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UNIFIED FACILITIES CRITERIA (UFC)

SEISMIC DESIGN FOR BUILDINGS

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UNIFIED FACILITIES CRITERIA (UFC)

SEISMIC DESIGN FOR BUILDINGS

U.S. ARMY CORPS OF ENGINEERS (Preparing Activity)

Record of Changes (changes are indicated by **\1** ... **/1/**)

Change No.	Date	Location

This UFC supersedes TI 809-04, dated 31 December 1998. The format of this UFC does not conform to UFC 1-300-01; however, the format will be adjusted to conform at the next revision. The body of this UFC is the previous TI 809-04, dated 31 December 1998.

FOREWORD

The Unified Facilities Criteria (UFC) system is prescribed by MIL-STD 3007 and provides planning, design, construction, sustainment, restoration, and modernization criteria, and applies to the Military Departments, the Defense Agencies, and the DoD Field Activities in accordance with USD(AT&L) Memorandum dated 29 May 2002. UFC will be used for all DoD projects and work for other customers where appropriate. All construction outside of the United States is also governed by Status of forces Agreements (SOFA), Host Nation Funded Construction Agreements (HNFA), and in some instances, Bilateral Infrastructure Agreements (BIA.) Therefore, the acquisition team must ensure compliance with the more stringent of the UFC, the SOFA, the HNFA, and the BIA, as applicable.

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- Whole Building Design Guide web site http://dod.wbdg.org/.

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UNIFIED FACILITIES CRITERIA (UFC)

SEISMIC DESIGN FOR BUILDINGS

The text of this UFC is the previous TI 809-04, dated 31 December 1998.

TI 809-04

US Army Corps
of Engineers

TI 809-04
31 December 1998

TECHNICAL INSTRUCTIONS

SEISMIC DESIGN FOR BUILDINGS

Headquarters
US Army Corps of Engineers
Engineering Division
Directorate of Military Programs
Washington, DC 20314-1000

CEMP-E

TI 809-04
31 December 1998

TECHNICAL INSTRUCTIONS

Seismic Design for Buildings

Approved for public release; distribution is unlimited.

Record of Changes (changes indicated by \1\..../1/)

No.	Date	Location

This document replaces the seismic design criteria contained in TM 5-809-10, Seismic Design for Buildings; TM 5-809-10-1, Seismic Design Guidelines for Essential Buildings; ETL 1110-3-479, Site-Specific Ground Motion Studies; and ETL 1110-3-480, Seismic Isolation and Energy Dissipation Systems.

FOREWORD

These technical instructions (TI) provide design and construction criteria and apply to all U.S. Army Corps of Engineers (USACE) commands having military construction responsibilities. TI will be used for all Army projects and for projects executed for other military services or work for other customers where appropriate.

TI are living documents and will be periodically reviewed, updated, and made available to users as part of the HQUSACE responsibility for technical criteria and policy for new military construction. CEMP-ET is responsible for administration of the TI system; technical content of TI is the responsibility of the HQUSACE element of the discipline involved. Recommended changes to TI, with rationale for the changes, should be sent to HQUSACE, ATTN: CEMP-ET, 20 Massachusetts Ave., NW, Washington, DC 20314-1000.

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FOR THE COMMANDER:

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Chief, Engineering and Construction Division
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SEISMIC DESIGN FOR BUILDINGS

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CHAPTER 1
GENERAL

1-1. Purpose and Scope.

This document is intended to provide qualified designers with the necessary criteria and guidance for the performance-based seismic analysis and design of new military buildings, and the nonstructural systems and components in the buildings.

1-2. Applicability.

a. General. The criteria in this document are applicable to all elements responsible for the design of military construction in the United States and its territories and possessions. The procedures in this document may be used to verify the performance objectives of any new construction.

b. Nonapplicability. Non-building structures and hazardous critical facilities (e.g., nuclear power plants, piers, wharves, dams, and liquefied gas facilities) are not within the scope of this document.

c. Design Team. When use of this document is required, the selected design team will include an engineer knowledgeable in seismic design. That engineer will be included in the facility planning process from the beginning to provide guidance in the selection of the appropriate seismic resisting system. Early input and a special peer review team are required when seismic isolation or energy dissipation devices are a potential alternative.

1-3. Basis for Design.

The primary basis for this document is the 1997 edition of the *NEHRP Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA 302), and the terminology and general design procedures are incorporated by reference in this document. This document provides guidance in the interpretation and implementation of the FEMA 302 provisions for the Life Safety performance objective for all buildings, and it provides criteria for the design and analysis of buildings with enhanced performance objectives.

a. Introduction to Seismic Design. Chapter 2 provides an introduction to principles of performance-based seismic design as prescribed in this document.

b. Classification of Buildings. All buildings are classified regarding use and/or function into one of four Seismic Use Groups indicated in Table 4-1. Based on these seismic use groups and the applicable design ground motion, the buildings are further assigned a Seismic Design Category, as shown in Tables 4-2a and 4-2b. The Seismic Use Group classification dictates the seismic performance objective for the building, while the Seismic Design Category influences the permissible structural system, allowable height, and other design parameters.

c. Ground Motion. Two levels of ground motion are prescribed in this document. Both levels are defined in terms of spectral ordinates with reference to the Maximum Considered Earthquake (MCE). Contours of spectral ordinates at periods of 0.2 second and 1.0 second are delineated on the

MCE maps that accompany FEMA 302. Ground Motion A, which is the reference ground motion in FEMA 302, is defined as two-thirds of the site-adjusted MCE spectral ordinates, and Ground Motion B is three-quarters of the same MCE ordinates. The derivation of the MCE ground motion and the representations of the design ground motions in seismic analysis of buildings are described in Chapter 3.

d. Performance Objectives. Three acceptable performance levels are established: 1) Life Safety, 2) Safe Egress, and 3) Immediate Occupancy, as described in Table 4-3. The three performance levels are combined with the two design ground motions to define a performance objective for each of the four seismic use groups, as indicated in Table 4-4. Performance Objective 1A (Life Safety) is the basic objective for Seismic Use Group I (Standard Occupancy) buildings in FEMA 302, and is the required minimum performance for all buildings governed by this document. The remaining three objectives define enhanced performance required for special occupancy, and hazardous or essential facilities. The expected seismic response of these performance objectives is indicated graphically in Figure 1-1 for ductile structures, and Figure 1-2 for non-ductile structures.

e. Seismic Design and Analysis Procedures.

(1) Seismic design and detail requirements. All structures are required to comply with the applicable requirements of the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA 302) as modified by this document. Additionally, structures requiring enhanced performance objectives shall comply with the applicable provisions of this document. For Seismic Use Group IIIE buildings that must be available for post-earthquake recovery and/or other mission-essential functions immediately following an earthquake, it is important to have structural engineering input early in the functional development and building layout phase of the project. When these buildings are assigned to Seismic Design Category D, E, or F, base isolation or energy dissipation, in accordance with Chapter 8, should be considered to reduce the ground shaking effects on the building and its contents.

(2) Additions will be designed as new buildings and will be kept structurally independent of the existing building, if at all possible. When an addition is *not* structurally independent, the addition will be designed as a new building and the combined building structure, new and old, will be evaluated using the provisions of this document. If found deficient, the existing structure will be upgraded to comply with the appropriate performance objective corresponding to the seismic use group assigned to the building.

(3) Compliance with agency manuals. Criteria and design standards in the agency manuals for ordinary or nonseismic design are applicable to seismic design except where criteria in this document are more stringent. Details of construction shown in this document represent those acceptable for conforming systems. Site adaption of standard drawings will include design revisions for the seismic

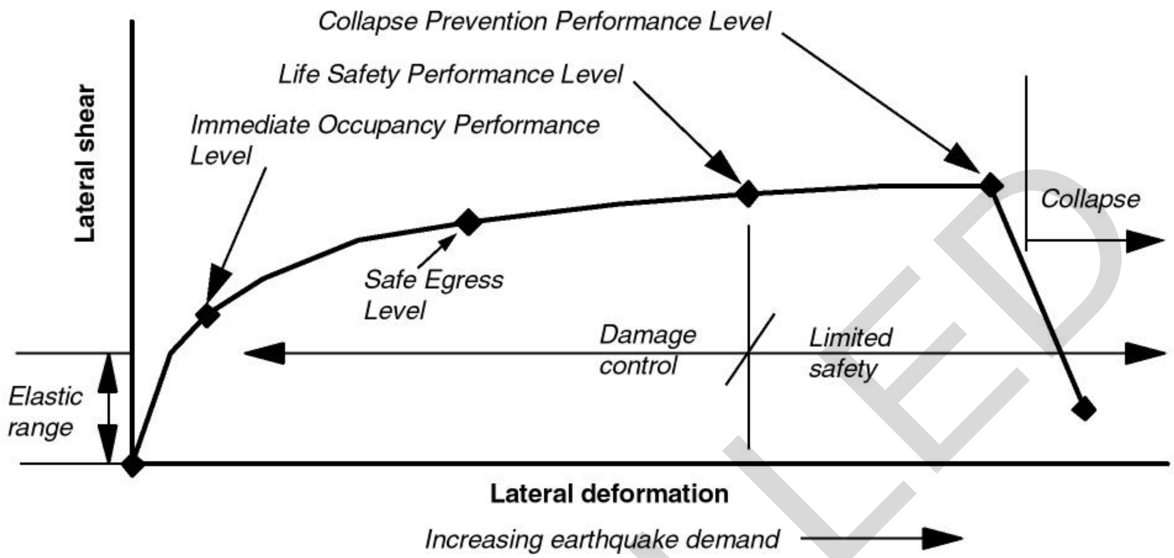


Figure 1-1 Performance and structural deformation demand for ductile structures.

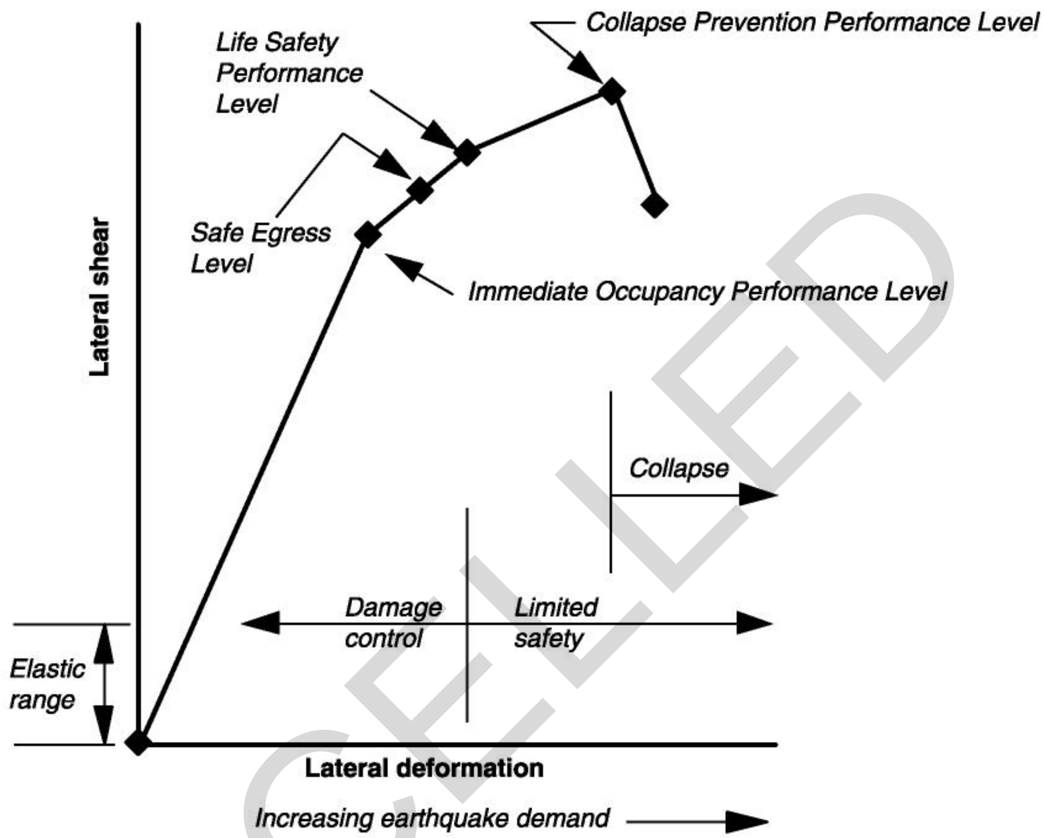


Figure 1-2 Performance and structural deformation demand for nonductile structures.

area as required. In overseas construction where local materials of grades other than those stated herein are used, the nominal capacities, grades, and other requirements of this document will be modified as acceptable.

(4) Minimum analytical procedures. The three basic analytical procedures prescribed by this document are described in Chapter 5, and the minimum analytical procedure permitted for each performance objective is indicated in Table 4-4. Chapter 5 also provides guidance as to the limitations of the minimum procedures and the need for more rigorous analyses.

(5) Acceptance criteria. The acceptance criteria for the various performance objectives are prescribed for each of the three analytical procedures in Chapter 6, and numerical values of the acceptance criteria for various structural and nonstructural systems are provided in Chapters 7 through 10.

(6) Seismic isolation and energy dissipation. Chapter 8 provides guidance for the design of seismic isolation and energy dissipation systems.

(7) Nonstructural systems and components. The seismic analysis and design of nonstructural systems and components for various performance objectives is prescribed in Chapter 10.

1-4. References.

Appendix A contains a list of references pertaining to this document.

1-5. Symbols and Notations.

Symbols and notations pertaining to the text are defined where they occur. Other symbols and notations pertaining to ground motion and design examples are defined in Appendix B.

1-6. Glossary.

Technical terms pertaining to seismic and geological hazards are defined in the Glossary in Appendix C.

1-7. Ground Motion Data.

Ground motion background data are provided in Appendix D; site-specific probabilistic seismic hazard analyses are described in Appendix E; geologic hazard evaluations are provided in Appendix F; and geologic screening examples are provided in Appendix G.

1-8. Design Examples.

Examples of structural design for buildings are provided in Appendix H; design for architectural components are provided in Appendix I; and design for mechanical and electrical components are provided in Appendix J.

1-9. Bibliography.

A bibliography of publications that may furnish additional information or background data is provided in Appendix K.

1-10. Quality Assurance.

a. Design Quality Assurance. In addition to the normal internal design and review procedures required of all design projects, peer review as discussed below will be required for the seismic design of buildings in Seismic Use Group III, Seismic Design Category D, E, or F; all buildings being designed with seismic isolation or energy dissipation; and other buildings that may be designated by the cognizant design authority.

(1) Peer review is the technical review of an engineering project by peers of the project design team in order to provide an increased level of confidence regarding the desired performance and safety of the project as envisioned by the design.

(2) The scope of the peer review should be defined in writing by the cognizant design authority, and in general should include review of the following:

(a) Compatibility of the design criteria with the following objectives:

1. Quality of the design and the design approach.

2. Quality of the documentation.

3. Constructibility.

4. Anticipated structural performance.

(3) After completing the review, the peer review team, consisting of one or more design professionals, shall discuss the results of the review with the project design team prior to submitting a report summarizing the scope and limitations of the review, conclusions, and recommendations.

b. Construction Quality Assurance. A quality assurance plan conforming to the requirements of Chapter 3 of FEMA 302 shall be developed and implemented for all projects governed by this document.

CHAPTER 2
INTRODUCTION

2-1. General.

This chapter provides an introduction to the basic concepts of designing buildings to resist inertia forces and related effects caused by earthquakes. An earthquake causes vibratory ground motions at the base of a structure, and the structure actively responds to these motions. For the structure responding to a moving base there is an equivalent system: the base is fixed and the structure is acted upon by forces (called inertia forces) that cause the same distortions that occur in the moving-base system. In design, it is customary to visualize the structure as a fixed-base system acted upon by inertia forces. Seismic design involves two distinct steps: determining (or estimating) the earthquake forces that will act on the structure, and designing the structure to provide adequate strength, stiffness, and energy dissipation capabilities to withstand these forces.

2-2. Ground Motion.

a. General. The response of a given structure depends on the characteristics of the ground motion; therefore, it would be highly desirable to have a quantitative description of the ground motion that might occur at the site of the building during a major earthquake. Unfortunately, there is no description that fits all the ground motions that might occur at any particular site. The characteristics of the ground motion are dependent on the magnitude of the earthquake (i.e., the energy released), the distance from the source of the earthquake (depth, as well as horizontal distance), the distance from the surface faulting (this may or may not be the same as the horizontal distance from the source), the nature of the geological formations between the source of the earthquake and the building, and the nature of the soil in the vicinity of the building site (e.g., hard rock or alluvium). Although fully accurate prediction of ground motion is not possible, the art of ground motion prediction has progressed in recent years to the point that nationally approved design criteria have been developed by consensus groups of geotechnical and building design professionals.

b. Representation of Ground Motion. The motion at the site can be described by a single number, such as peak ground acceleration (A_g). This single number, however, does not give the information on the characteristics (or signature) of the earthquake.

(1) Response spectra. For design purposes, it would be ideal to forecast the acceleration time history of a future earthquake having a given hazard of occurrence; however, the complex random nature of an accelerogram makes it necessary to employ a more general characterization of ground motion. Specifically, the most practical representation is the earthquake response spectrum. Although this spectrum is used to describe the intensity and vibration frequency content of accelerograms, its most important advantage is that spectra from several records can be normalized, averaged, and then scaled according to seismicity to predict future ground motion at a given site. The physical definition of an acceleration response spectrum is shown in Figure 2-1. A set of linear elastic single-degree-of-freedom (SDOF) systems having a common damping ratio, ***$***,

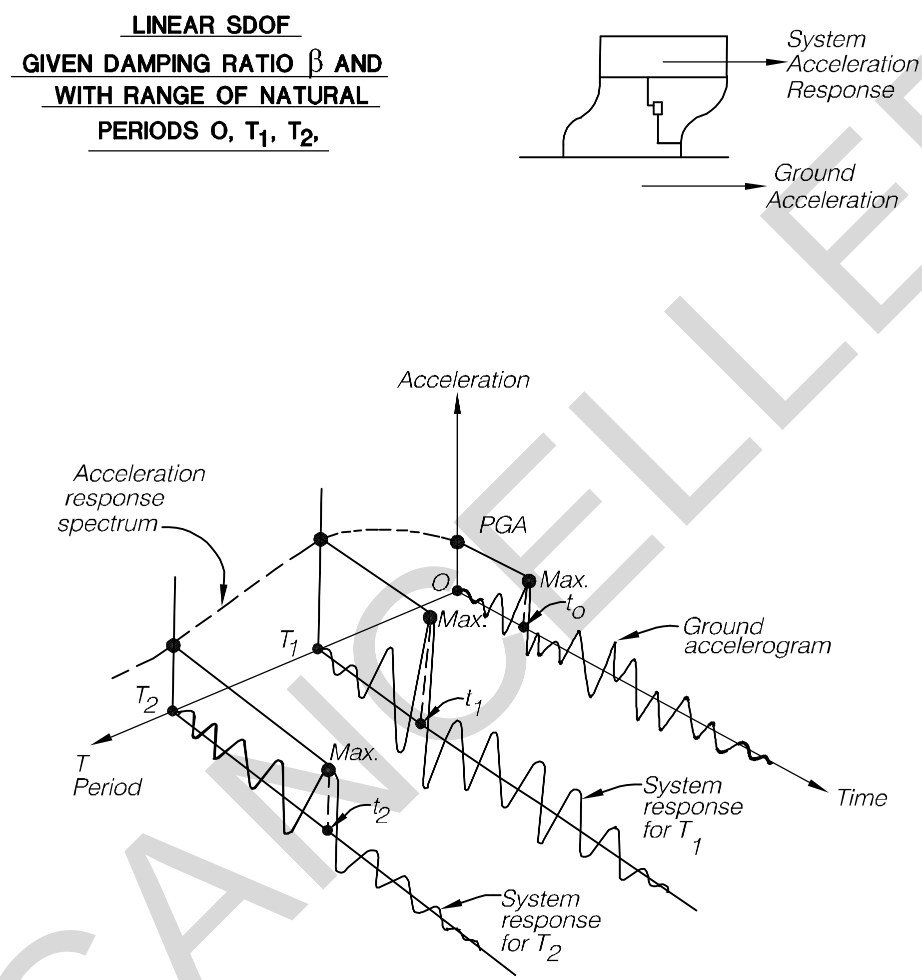


Figure 2-1 Description of acceleration response spectrum

but each having different harmonic periods over the range O, T_1, T_2, etc., is subjected to a given ground motion accelerogram. The entire time history of acceleration response is found for each system, and the corresponding maximum value, S_a, is plotted on the period axis for each system period. The curve connecting these S_a values is the acceleration response spectrum for the given accelerogram and damping ratio. The peak response of the oscillator (S_a) is a point on the response spectra for the period of the oscillator. The ground motion in this document is defined by two spectral ordinates, as described in Chapter 3. The two ordinates represent spectral response accelerations as a percentage of the acceleration due to gravity, g. Equations are also provided for the development of response spectra from the two ordinates, and for the modification of the spectral response for various soil conditions at the site. For firm sites, the design spectral ordinate at 0.2 second, S_{DS}, is roughly equivalent to two and one-half times the Z coefficient in the Uniform Building Code (UBC), or the A_a coefficient in the prior NEHRP provisions. The spectral ordinate at 1.0 second for firm sites is approximately equivalent to 1.2 times the A_v coefficient in the prior NEHRP provisions. The response spectra are prescribed for dynamic analyses, and the equations also define equivalent lateral forces for static analysis. These changes in the representation of ground motions were instigated by USGS as a result of an extensive national program to develop spectral parameters that better represent actual site response, and to incorporate the most current knowledge regarding regional seismicity. The new design values will result in higher seismic design forces, as compared to the 1997 UBC or FEMA 222A in sites near major faults and in areas of very low or negligible seismicity (e.g., UBC Zone 0). In some areas of previously low or moderate seismicity, the new seismic design forces may be lower than previously prescribed.

(2) Time histories of ground motion are required for nonlinear inelastic dynamic analyses, and can also be used for general dynamic analyses. Time histories may be actual or modified ground motion records, or may be synthetic time histories developed to match a target spectrum. Since a single time history cannot be completely representative of all possible ground motions at the site, a suite (e.g., usually at least three) of time histories is generally required. Time histories are not prescribed by this document, and their use requires authorization from the cognizant design authority.

2-3. Site Hazards Other than Ground Motion.

a. General. The analysis and design procedures of this document are primarily aimed at improving the performance of structures under the loads and resulting deformations imposed by seismic shaking. Other seismic hazards could, however, exist at the building site that could damage the structure, regardless of its ability to resist ground shaking. These hazards include fault rupture, liquefaction or other shaking-induced soil failures, landslides, and inundation from offsite effects such as dam failure or tsunami.

b. Evaluation and Mitigation. The risk and possible extent of damage from such site hazards should be considered in the site selection process. In some situations, it may be feasible to mitigate the site hazards. In many cases, the likelihood of the site hazard occurring will be sufficiently small that the design of the structure to resist ground shaking is appropriate. Where a site geological hazard exists, it may be feasible to mitigate it, either by itself or in connection with the design of the structure. It is also possible that the risk from a site hazard is so extreme and difficult to control that construction on the site will not be cost-effective. Chapter 3 and Appendices F and G provide guidance for the evaluation and mitigation of site geological hazards.

2-4. Behavior of Structures.

Buildings and other structures are composed of horizontal and vertical structural elements that resist lateral forces. The horizontal elements, diaphragms and horizontal bracing, are used to distribute the lateral forces to vertical elements. The vertical elements that are used to transfer lateral forces to the ground are shear walls, braced frames, and moment resisting frames. The structure must include complete lateral and vertical-force-resisting systems capable of providing adequate energy dissipation capacity to withstand the design ground motions within the prescribed limits of deformation and strength demand.

a. Demands of Earthquake Motion. The loads or forces that a structure sustains during an earthquake result directly from the distortions induced in the structure by the motion of the ground on which it rests. Ground motion is characterized by displacements, velocities, and accelerations that are erratic in direction, magnitude, duration, and sequence. Earthquake loads are inertia forces related to the mass, stiffness, and energy-absorbing (e.g., damping and ductility) characteristics of the structure. During the life of a structure located in a seismically active zone, it is generally expected that the structure will be subjected to many small earthquakes, some moderate earthquakes, one or more large earthquakes, and possibly a very severe earthquake. In general, it is uneconomical or impractical to design buildings to resist the forces resulting from the very severe or maximum credible earthquake within the elastic range of stress; instead, the building is designed to resist lower levels of force, using ductile systems. When the earthquake motion is large to severe, the structure is expected to yield in some of its elements. The energy-absorbing capacity (ductility) of the yielding structure will limit the degree of life-threatening damage; buildings that are properly designed and detailed can survive earthquake forces substantially greater than the forces associated with allowable stresses in the elastic range. Seismic design concepts must consider building proportions and details for their ductility and for their reserve energy-absorbing capacity for surviving the inelastic deformations that would result from the maximum expected earthquake. Special attention must be given to the connections that hold together the elements of the lateral-force-resisting system.

b. Analysis of Structural Response. As indicated above, the response of structures to severe ground motion is a complex combination of elastic and inelastic actions. Additionally, as yielding is initiated in individual structural elements, subsequent loads are redistributed among the remaining elastic elements. Linear analyses assume that the response can be adequately represented by an elastic mode of the structure with various response modification factors to represent ductility or the energy absorption capabilities of the structure. Linear elastic and dynamic analyses with a global response modification factor, *R*, are prescribed in FEMA 302, and are incorporated by reference in this document for compliance with Performance Objective 1A (Life Safety). These linear elastic analyses are also prescribed in this document for Performance Objectives 2A, 2B, and 3B with modification factors, *m*, for deformation-controlled structural components or elements. Nonlinear analyses can be either elastic or inelastic. Nonlinear elastic analyses, also known as "pushover" analyses, subject an elastic model of the structure to a predetermined pattern of static forces. A force/displacement curve is then constructed by iterative analyses with yield "hinges" placed at the yielding ends of the structural elements. Compliance is determined by matching a target displacement with acceptable inelastic deformation of the yielding elements. Nonlinear inelastic analyses are usually time-history dynamic analyses with predetermined elastic/inelastic characteristics for the structural elements. Guidance on the use and limitations of the above analytical procedures is provided in Chapters 4 and 5.

c. Response of Elements Attached to the Structure. Elements attached to the floors of the building or structure (e.g., mechanical equipment, ornamentation, piping, nonstructural partitions) respond to floor motion in much the same manner that the building responds to ground motion; however, the floor motion may vary substantially from the ground motion. The high-frequency components of the ground motion tend to be filtered out at the higher levels in the building, while the components of ground motion that correspond to the natural periods of vibration of the building tend to be magnified. If the elements are rigid and are rigidly attached to the structure, the forces on the elements will be in the same proportion to the mass as the forces on the structure, or $F = ma$ (i.e., the accelerations of the elements will be about the same as the acceleration of the floor on which they are supported). However, elements that are flexible and have periods of vibration close to any of the predominant modes of the building vibration will experience accelerations substantially greater than the accelerations on the structure (i.e., accelerations of elements will be greater than floor accelerations). The above actions are approximated by the design force equations in Chapter 6 of FEMA 302, and as prescribed in Chapter 10 of this document for the various performance objectives.

2-5. Fundamentals of Seismic Design.

The type of structural system used will determine the magnitude of the design lateral forces. The decision as to the type of structural system to be used will be based on the merits and relative costs for the individual building being designed. There are innovative systems available for particular structural configurations and conditions, such as eccentric braced frames, seismic isolation, friction devices, and other response control systems. These systems are described below.

a. Gravity-Load System. The basic elements of a gravity load system are: (a) horizontal elements (e.g., slabs, sheathing, beams, girders, or trusses) that collect the dead and live loads in various levels in the structure; (b) the vertical-resisting elements (e.g., columns and bearing walls) that receive the gravity loads from the horizontal elements: and (c) the foundations (e.g., footings, piers, piles) that receive the loads from the vertical elements and transfer them to the ground. The suitability of various foundation systems and allowable values for their design must be determined from available data or by a program of soil borings and laboratory tests.

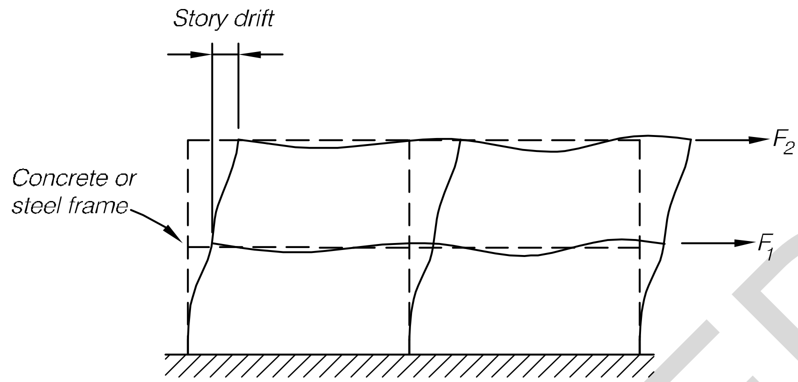
b. Lateral-Force-Resisting Systems.

(1) General. A building is not merely a summation of parts (walls, columns, trusses, and similar components), but is a completely integrated system or unit that has its own properties with respect to lateral-force response. The designer must trace the forces through the structure into the ground, and make sure that every connection along the path of stress is adequate to maintain the integrity of the system. It is necessary to visualize the response of the complete structure, and to keep in mind that the real forces involved are not static, but dynamic; are usually erratically cyclic and repetitive; may be significantly larger than the design forces; and can cause deformations well beyond those determined from the design forces.

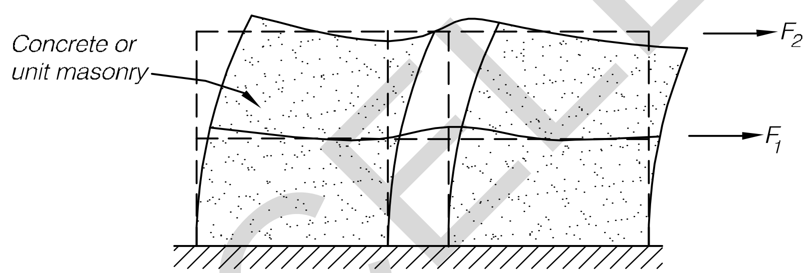
(2) Lateral force system types. Over a dozen approved lateral-force-resisting systems are described in Chapter 7. All of the vertical elements of these lateral-force systems consist of: (a) moment-resisting frames within a three-dimensional space frame system; (b) a coordinated system of shear walls; (c) a three-dimensional system of braced frames; or (d) a combination or "dual system" of moment-resisting frames with either shear walls or braced frames. These vertical elements may be used in various combinations within a building, as described herein. All of the horizontal elements of these lateral-force systems consist of either diaphragms or horizontal bracing systems. The vertical elements of the lateral-force-resisting systems are illustrated in Figure 2-2.

(a) In buildings where a moment-resisting frame resists the earthquake forces, the columns and beams act in bending (*a* of Figure 2-2). During a large earthquake, story-to-story deformation (story drift) may be a matter of inches, without causing failure of columns or beams. The drift, however, may be sufficient to damage elements that are rigidly tied to the structural system, such as brittle partitions, stairways, plumbing, exterior walls, and other elements that extend between floors. For this reason, buildings can have substantial interior and exterior nonstructural damage, possibly approaching 50 percent of the total building value, and still be considered structurally safe. Moment frames are desirable architecturally because they are relatively unobtrusive compared with shear walls or braced frames, but they may be a poor economic risk unless special damage control measures are taken.

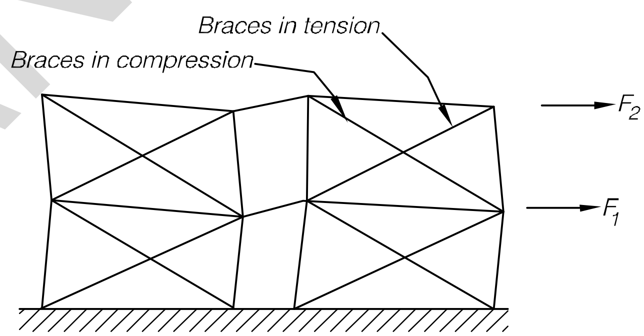
(b) Buildings with shear walls (*b* of Figure 2-2) are usually rigid compared with buildings with moment-resisting frames. With low design stress limits in shear walls, deformation due to shear forces (for low buildings) is negligible. Shear wall construction is an excellent method of bracing buildings to limit damage to nonstructural components, but architectural considerations may limit its applicability. Shear walls are usually of reinforced unit masonry or reinforced concrete, but may be of wood in wood-frame buildings up to and including three stories. Shear wall design is relatively simple, except when the height-to-width ratio of a



(a) FRAME ACTION BY MOMENT-RESISTING BENTS



(b) SHEAR WALLS AS VERTICAL CANTILEVERS



(c) BRACED FRAMES OF STEEL

Figure 2-2 Vertical elements of the lateral force resisting systems.

wall becomes large. Overturning may then be a problem, and if the foundation soil is relatively soft, the entire shear wall may rotate, causing localized damage around the wall. Another difficult case is the shear wall with openings such that it may respond more like a frame than a wall.

(c) Braced frames (Figure 2-2) generally have the stiffness associated with shear walls, but are somewhat less restrictive architecturally. It may be difficult to find room for doorways within a braced frame; however, braces are less obtrusive than solid walls. The concern for overturning mentioned above for shear walls also applies to braced frames. Braced frames may be concentric (*c* in Figure 2-2) or eccentric (Figure 7-23).

(d) Horizontal elements in the lateral-force-resisting system include floor and roof diaphragms and horizontal bracing systems. Diaphragms may consist of wood sheathing or plywood, steel decking with or without concrete fill, or cast-in-place or precast concrete slabs. Diaphragms and horizontal bracing systems are classified as flexible, stiff, or rigid, based on their deformation relative to the vertical-resisting elements. Design and acceptance criteria for these elements are provided in Paragraph 7-7 of Chapter 7.

(e) Structural systems may be used in various combinations. There may be different systems in the two directions, or systems may be combined in any one direction, or may be combined vertically. FEMA 302 permits the use of *R* factors applicable to the structural system in each orthogonal direction. Specific *R* values are provided for acceptable dual systems, and the lower *R* value is prescribed for a vertical combination of two structural systems.

c. Configuration and Simplicity. A great deal of a building's resistance to lateral forces is determined by its plan layout. The objective in this regard is symmetry about both axes, not only of the building itself, but of its lateral-force-resisting elements and of the arrangement of wall openings, columns, shear walls, and so on. It is most desirable to consider the effects of lateral forces on the structural system from the start of the layout, since this may save considerable time and money without detracting significantly from the usefulness or appearance of the building. Experience has shown that buildings that are asymmetrical in plan have greater susceptibility to earthquake damage than symmetrical structures with simple and direct load paths for lateral forces. The effect of asymmetry is to induce torsional oscillations of the structure and stress concentrations at re-entrant corners. Asymmetry in plan can be eliminated or improved by separating L-, T-, and U-shaped buildings into distinct units by use of seismic joints at the junctions of the individual wings. It should be noted, however, that this causes two new problems: providing floor joints that are capable of bridging gaps large enough to preclude adjacent structures from pounding each other, and providing wall and roof joints that are capable of keeping out the weather. Asymmetry caused by the eccentric location of lateral-force-resisting structural elements—such as in the case of a building that has a flexible front because of large openings and an essentially stiff (solid) rear wall—can usually be avoided by better conceptual planning. For example, modify the stiffness of the rear wall or add rigid structural partitions to make the center of rigidity of the lateral-force-resisting elements closer to the center of mass. When a building has irregular features, such as asymmetry in features, such as asymmetry in plan or vertical discontinuity, the assumptions used in developing seismic criteria for buildings with regular features may not apply. For example, planners often omit partitions and exterior walls in the first story of a building to permit an open ground floor; in this case, the columns at the ground level are the only elements available to resist lateral forces, and there is an abrupt change in the rigidity of the vertical elements of the lateral force resisting system at that level. This condition, generally referred to as soft story, is undesirable. It is advisable to carry all shear walls down to the foundation. It is best to avoid creating buildings with irregular features; however, when irregular features are unavoidable, special design considerations are required to account for the unusual dynamic characteristics and the load transfer and stress concentrations that occur at abrupt changes in structural resistance.

d. *Redundancy*. Redundancy is a highly desirable characteristic for earthquake-resistant design. Redundancy can be achieved with multiple load paths. For example, a multistory steel moment frame building, with all the joints designed to be moment-resisting, has greater redundancy than a similar building with only selective moment-resisting joints in that a flaw or unexpected failure of one joint can be offset by redistribution of loads to the other joints. Redundancy can also be achieved with parallel or "back-up" systems, such as the moment-resisting frames in a dual framing system in which the frames are designed for a nominal lateral force, but are expected to preclude collapse after the shear walls or braced frames have failed. Redundancy is defined by the reliability factor D described in paragraph 4-1, and lack of redundancy results in increased seismic load effects, as indicated in Equation 4-4 and 4-5.

e. *Ductile vs. Brittle Response*. Although ductile response is highly desirable from an earthquake energy dissipation standpoint, ductile structures will be more flexible, and the designer must give proper consideration to the resulting drift to preclude structural instability and undue damage to nonstructural elements. Similar consideration must be given to structural elements with anticipated brittle response (e.g., shear in concrete columns). These elements must be designed so as to preclude brittle response (e.g., adequate shear strength in concrete columns to permit flexural yielding of column or connecting beams) or designed with adequate capacity to resist the unreduced demand forces. When a building is subjected to earthquake ground motion, a pattern of lateral deformations that varies with time is induced into the structure. At any given point in time, a particular state of lateral deformation will exist in the structure, and at some time within the period in which the structure is responding to the ground motion, a maximum pattern of deformation will occur. At relatively low levels of ground motion, the deformations induced within the building will be limited, and the resulting stresses that develop within the structural components will be within the elastic range of behavior. Within this elastic range, the structure will experience no damage. All structural components will retain their original strength, stiffness, and appearance, and when the ground motion stops, the structure will return to its pre-earthquake condition. At more severe levels of ground motion, the lateral deformations induced into the structure will be larger. As these deformations increase, so will demands on the individual structural components. At different levels of deformation, corresponding to different levels of ground motion severity, individual components of the structure will be strained beyond their elastic range. As this occurs, the structure starts to experience damage in the form of cracking, spalling, buckling, and yielding of the various components. As components become damaged, they degrade in stiffness. In general, when a structure has responded to ground motion within this range of behavior, it will not return to its pre-earthquake condition when the ground motion stops. Some permanent deformation may remain within the structure, and damage will be evident throughout. Depending on how far the structure has been deformed, and in what pattern, the structure may have lost a significant amount of its original stiffness, and possibly, strength. Brittle elements are not able to sustain inelastic deformations and will fail suddenly; the consequences may range from local and repairable damage to collapse of the structural system. At higher levels of ground motion, the lateral deformations induced into the structure will strain a number of elements to a point at which elements behave in a brittle manner, or as a result of the decreased overall stiffness, the structure loses stability. Eventually, partial or total collapse of the structure can occur. The structural performance levels used in this document relate the extent of a building's response to earthquake hazards to these various possible damage states. Figure 1-1 illustrates the behavior of a ductile structure as it responds with increasing lateral deformation. The figure is a schematic plot of the lateral force induced in the structure as a function of lateral deformation. Four discrete points are indicated, representing the discrete performance levels: Immediate Occupancy, Safe Egress, Life Safety, and Collapse Prevention. At the Immediate Occupancy Level, damage is relatively limited. The structure retains a significant portion of its original stiffness, and most, if not all, of its strength. At the Collapse Prevention level, the building has experienced extreme damage. If laterally deformed beyond this point, the structure can experience instability and collapse. At the Life Safety Level, substantial damage has occurred to the structure, and it may have lost a significant amount of its original stiffness; however, a substantial margin remains for additional lateral deformation before collapse would occur. At the Safe Egress level, the damage is intermediate between the Immediate Occupancy and the Life Safety levels. It should be noted that for given buildings, the relative horizontal and vertical scales shown on this plot may vary significantly, and the margin of deformation between individual performance levels may not be as large as indicated in this figure. Figure 1-2 is a similar curve, representative of the behavior of a nonductile, or brittle, structure. Note that for such a structure, there may be relatively little margin in the responses that respectively define the three performance levels. For a given structure and design earthquake, it is possible to estimate the overall deformation and force demand on the structure, and therefore, the point on the corresponding curves shown in Figures 1-1 or 1-2 to which the earthquake will push the building. This either will or will not correspond to the desired level of performance for the structure. The building should also be checked for compliance with the allowable story drift levels prescribed in Table 6-1 to preclude unacceptable damage to nonstructural systems and components. When structural/seismic design is performed, modifications to the structural model are made to alter its strength, stiffness, or ability to dampen or resist induced deformations. These actions will alter the characteristics of both the shape of the curves in these figures, and the deformation demand produced by the design earthquake on the building, such that the expected performance at the estimated deformation level for the structure is acceptable.

f. Connectivity. It is essential to tie the various structural elements together so that they act as a unit. The connections between the elements are at least as important as the elements themselves. Prevention of collapse during a severe earthquake depends upon the inelastic energy-absorbing capacity of the structure, and this capacity should be governed by the elements rather than by their connections; in other words, connections should not be the weak link in the structure. As a general guide, if no other requirements are specified, connections should be adequate to develop the useful strength of the structural elements connected, regardless of the calculated stress due to the prescribed seismic forces.

g. Separation of Structures. In past earthquakes, the mutual hammering received by buildings in close proximity to one another has caused significant damage. The simplest way to prevent damage is to provide sufficient clearance so that free motion of the two structures will result. The motion to be provided for is produced partly by the deflections of the structures themselves, and partly by the rocking or settling of foundations. The gap must equal the sum of the total deflections from the base of the two buildings to the top of the lower building.

(1) In the case of a normal building less than 80 feet in height using concrete or masonry shear walls, the gap shall be not less than the arbitrary rule of 1 inch (25mm) for the first 20 feet (6.10m) of height above the ground, plus ½ inch (13mm) for each 10 feet (3.05m) of additional height.

(2) For higher or more flexible buildings, the gap or seismic joint between the structures should be based on the sum of the deflections determined from the required (prescribed) lateral forces. If the design of the foundation is such that rotation is expected to occur at the base due to rocking or due to settlement of foundations, this additional deflection (as determined by rational methods) will be included.

(3) In situations where it is impractical to provide adequate clearance, the consequences of potential damage due to hammering must be considered. If the floor levels of the two buildings are approximately the same and the floor systems are relatively robust (e.g., concrete beams and slabs), the resulting damage may be limited to local spalling that is readily repaired. If the floor levels are significantly offset and the bearing walls or columns of either building are vulnerable to hammering action from the rigid floor systems of the other building, the potential damage is unacceptable. In such instances, either adequate clearance must be provided, or the vulnerable structural components must be strengthened or provided with back-up elements to avoid the possibility of structural failure.

h. Seismic Joints. Junctures between distinct parts of buildings, such as the intersection of a wing of a building with the main portion, are often designed with flexible joints that allow relative movement. When this is done, each part of the building must be considered as a separate structure that has its own independent bracing system. The criteria for separation of buildings in Paragraph *a* above will apply to seismic joints for parts of buildings. Seismic joint coverage will be flexible and architecturally acceptable.

i. Elements that Connect Buildings. Certain types of structures commonly found in industrial installations are tied together at or near their tops by connecting parts such as piping, conveyors, and ducts. The support of these elements will allow for the relative movement between buildings.

j. Bridges Between Buildings. Clusters of buildings are often connected by bridges. In most cases it would not be economically feasible to make bridges sufficiently rigid to force both buildings to vibrate together. A sliding joint at one or both ends of the bridge can usually be installed.

k. Stairways. Concrete stairways often suffer seismic damage because they act like struts between the connected floors. This damage can be avoided by anchoring the stair structure at the upper end and providing a slip joint at the lower end of each stairway, or by tying stairways to stairway shear walls.

l. "Short Column" Effects. Whenever the lateral deflection of any column is restrained, when full height deflections were assumed in the analysis, it will carry a larger portion of the lateral forces than assumed. In past earthquakes, column failures have frequently been inadvertently caused by the stiffening (shortening) effect of deep spandrels, stairways, partial-height filler walls, or intermediate bracing members. Unless considered in the analysis, such stiffening effects will be eliminated by proper detailing for adequate isolation at the junction of the column and the resisting elements.

m. Design and Analysis Procedures. Step-by-step design and analysis procedures are provided for buildings conforming to Performance Objective 1A in Table 4-5, and illustrated in a flow chart in Figure 4-1. Similar procedures for buildings with enhanced performance objectives, using linear elastic analysis with the *m* modification factors, are provided in Table 4-6, and in a flow chart in Figures 4-2 and 4-3. The nonlinear elastic static procedures for Performance Objective 3B are described in Table 4-7, and in a flow chart in Figure 4-4.

n. Nonstructural Participation. For both analysis and detailing, the participation effects of nonstructural filler walls and stairs must be considered. The nonstructural elements that are rigidly tied to the structural system can have a substantial influence on the magnitude and distribution of earthquake forces. Such elements act somewhat like shear walls, stiffening the building and causing a reduction in the natural period, and an increase in the lateral forces and overturning moments. Any element that is not strong enough to resist the forces it attracts will be damaged, and should be isolated from the lateral-force-resisting system. Following are some design considerations to minimize damage to nonstructural components, and to preclude life safety hazards to the occupancy of the building.

(1) Details that allow structural movement without damage to nonstructural elements can be provided. Damage to items such as piping, glass, plaster, veneer, and partitions may constitute a major financial loss. To minimize this type of damage, special care in detailing, either to isolate these elements or to accommodate the movement, is required.

(2) Glass windows should be isolated with adequate clearance and flexible mountings at edges to allow for frame distortions.

(3) Rigid nonstructural partitions should have room to move at the top and sides.

(4) In piping installations, the expansion loops and flexible joints used to accommodate temperature movement are often adaptable to accommodating seismic deflections.

(5) Freestanding shelving can be fastened to walls to prevent toppling. Shelves can be provided with lips or edge restraints to prevent contents from falling off in an earthquake.

o. Alternatives to the Prescribed Provisions. Alternatives to the seismic provisions of this document are permitted if they can be properly substantiated. The most common alternatives are the use of more rigorous analytical procedures or the use of innovative systems.

(1) Rigorous analyses. Simple or approximate analyses are generally based on assumptions that require a significant degree of conservatism. A more rigorous analysis may require more precise knowledge of the physical characteristics of the structural elements and materials, but may incorporate less conservatism, thus permitting the acceptance of an otherwise nonconforming structure.

(2) Innovative systems. Systems and devices are available for controlling and/or limiting the response of structures to earthquake ground motion. The best known of these systems are seismic isolation systems (sometimes called base isolation systems). Seismic isolation is based on the premise that the structure can be substantially decoupled from potentially damaging earthquake motions. By decoupling the structure from the ground motion, seismic isolation reduces the level of response in the structure from the level that would otherwise occur in a conventional fixed-base building, or conversely, offers the advantage of designing with a reduced level of earthquake load to achieve the same degree of seismic protection and reliability as a conventional fixed-base building. Other innovative systems include passive and active energy dissipation devices. Limited guidance for the design of seismic isolation and energy dissipation systems is provided in Chapter 8. These systems are relatively new and sophisticated concepts that require more extensive design and detailed analysis than most conventional schemes. Peer review must be an essential part of any project that includes seismic isolation or energy dissipation devices.

CHAPTER 3
GROUND MOTION AND
GEOLOGICAL HAZARDS ASSESSMENT

3-1. Specification of Ground Motion.

a. General. This document prescribes two ground motions: Ground Motion A and Ground Motion B, as defined in the following paragraphs. The ground motions are expressed in terms of spectral ordinates at 0.20 sec (S_{DS}) and 1.0 sec (S_{D1}). These spectral values are derived from various seismic hazard maps prepared by the U.S. Geological Survey (USGS) and the Building Seismic Safety Council (BSSC) of the National Institute of Building Sciences (NIBS).

b. USGS Seismic Hazard Maps. At the request of the BSSC, USGS prepared probabilistic spectral acceleration maps for ground motions with 10 percent, 5 percent, and 2 percent probability of exceedance in 50 years. For each of these ground motions, probabilistic spectral ordinate maps were developed for peak ground accelerations and spectral response accelerations at 0.2, 0.3, and 1.0 seconds. Additionally, deterministic spectral ordinate maps were developed for areas adjacent to major active faults.

c. Maximum Considered Earthquake (MCE) Maps. In response to concerns regarding the use of the USGS maps by the building design professions, BSSC convened a nation-wide Design Values Group to review the maps and prepare design values for FEMA 302. The concerns of the design profession regarding the probabilistic maps included:

(1) The 10 percent probability of exceedance in 50 years ground motion generally used as a basis of seismic codes did not adequately capture the hazard due to large, but infrequent, events in some areas of the eastern and central U.S.

(2) Probabilistic values near major active faults tended to be very high because of the high rates of activity.

(3) Probabilistic values in some areas that appeared to be unreasonably low could be attributed to lack of sufficient data regarding source zones and frequency of events.

To address these concerns, the Design Values Group developed the MCE maps for spectral ordinates at 0.2 sec (denoted as S_S) and 1.0 sec (denoted as S_1). These maps are generally based on the USGS probabilistic maps for ground motion with 2 percent probability of exceedance in 50 years (approximately 2,500-year return period), but with deterministic values near major active faults and higher threshold values in selected areas of low seismicity. As indicated below, the design spectral ordinates were selected as two-thirds of the site-adjusted MCE values. The traditional seismic risk level considered by most model building codes is 10 percent probability of exceedance in 50 years (return period of about 500 years). Because the value of the ground motion for other risk levels is a function of the shape of the site-specific hazard curve, a valid comparison of the ground motion specified by prior codes with

2/3 of MCE can only be made on a site-specific or regional basis. However, the authors of the FEMA 302 provisions have indicated that, in many areas of the U.S., the new ground motions corresponding to 2/3 of MCE will be comparable to those specified by prior codes. It was also considered that, for most structural elements, the design criteria in FEMA 302 provided adequate reserve capacity to resist collapse at the MCE hazard level.

d. *Site Response Coefficients*. For all structures located within those regions of the maps having values of short-period spectral acceleration, S_S, greater than 0.15g, or values of the one-second period spectral acceleration, S_1, greater than 0.04g, the site shall be classified according to Table 3-1. Based on these Site Classes, FEMA 302 assigns Site Response Coefficients, F_a and F_v, as indicated in Tables 3-2a and 3-2b. The adjusted MCE spectral response acceleration for short periods, S_{MS}, and at 1 second, S_{M1}, are defined as:

$$S_{MS} = F_a S_S \qquad (3\text{-}1)$$

$$S_{M1} = F_V S_1 \qquad (3\text{-}2)$$

3-2. Design Parameters for Ground Motion A (FEMA 302).

a. *General*. Ground Motion A is the basic design ground motion for the FEMA 302 provisions. The design parameters for Ground Motion A are those used in this document for Performance Objectives 1A (Life Safety) and 2A (Safe Egress for Special Occupancy). The combination of performance levels and ground motions to form performance objectives is described in Paragraphs 4-7, 4-8, and 4-9, and is summarized in Tables 4-3 and 4-4.

b. *Design Spectral Response Accelerations*. The spectral response design values, S_{DS} and S_{D1}, adopted in FEMA 302 are defined as:

$$S_{DS} = 2/3\ S_{MS} \qquad (3\text{-}3)$$

$$S_{D1} = 2/3\ S_{M1} \qquad (3\text{-}4)$$

For regular structures, 5 stories or less in height, and having a period, T, of 0.5 seconds or less, the spectral accelerations, S_{MS} and S_{M1} need not exceed:

$$S_{MS}\ \#\ 1.5\ F_a \qquad (3\text{-}5)$$

$$S_{M1}\ \#\ 0.6\ F_v \qquad (3\text{-}6)$$

c. *Seismic Response Coefficients*.

(1) Equivalent Lateral Force (ELF) Procedure. For this procedure the seismic base shear is represented as V=$C_S W$ and the seismic response coefficient, C_S, is determined in accordance with the following equation:

$$C_S = \frac{S_{DS}}{R} \qquad (3\text{-}7)$$

where

R = Response modification factor defined in Section 5.2.2 of FEMA 302.

The value of C_S need not exceed the following:

$$C_S = \frac{S_{D1}}{TR} \tag{3-8}$$

Table 3-1
Site Classification

Class A	Hard rock with measured shear wave velocity, $\bar{v}_s$ > 5,000 ft/sec (1500 m/s)
Class B	Rock with 2,500 ft/sec < $\bar{v}_s$ ≤ 5,000 ft/sec (760 m/s < $\bar{v}_s$ ≤1500 m/s)
Class C	Very dense soil and soft rock with 1,200 ft/sec < $\bar{v}_s$ ≤ 2,500 ft/sec (360 m/s < $\bar{v}_s$ ≤ 760 m/s) or with either $\bar{N}$ > 50 or $\bar{s}_u$ >2,000 psf (100 kPa)
Class D	Stiff soil with 600 ft/sec ≤ $\bar{v}_s$ ≤ 1,200 ft/sec (180 m/s ≤ $\bar{v}_s$ ≤ 360 m/s) or with either 15≤ $\bar{N}$ ≤ 50 or 1,000 psf ≤ $\bar{s}_u$ ≤ 2,000 psf (50 kPa ≤ $\bar{s}_u$ ≤ 100 kPa)
Class E	A soil profile with $\bar{v}_s$ < 600 ft/sec (180 m/s) or with either $\bar{N}$ < 15, $\bar{s}_u$ < 1,000 psf, or any profile with more than 10 ft (3 m) of soft clay defined as soil with PI >20, $w \geq$ 40 percent, and s_u <500 psf (25 kPa).
Class F	Soils requiring site-specific evaluations: 1. Soil vulnerable to potential failure or collapse under seismic loading such as liquefiable soils; quick and highly sensitive clays; and collapsible, weakly cemented soils. 2. Peats and/or highly organic clays (H > 10 ft [3 m] of peat and/or highly organic clay where H = thickness of soil). 3. Very high plasticity clays (H > 25 ft [8 m] with PI > 75). 4. Very thick soft/medium stiff clays (H > 120 ft [36 m]).

Note: v_s is shear wave velocity; N is Standard Penetration Resistance (ASTM D1586-84), not to exceed 100 blows/ft as directly measured in the field without corrections; s_u is undrained shear strength, not to exceed 5,000 psf (250 kPa) (ASTM D2166-91 or D2850-87). $\bar{v}_s$, $\bar{N}$, and $\bar{s}_u$ are average values for the respective parameters for the top 100 feet of the site profile. Refer to FEMA 302 for the procedure to obtain average values for $\bar{v}_s$, $\bar{N}$, and $\bar{s}_u$.

Exception: When the soil properties are not known in sufficient detail to determine the *Site Class, Site Class D* shall be used. *Site Classes E* or *F* need not be assumed unless the authority having jurisdiction determines that *Site Classes E* or *F* could be present at the site or in the event that *Site Classes E* or *F* are established by geotechnical data.

Table 3-2a
Values of F_a as a Function of a Site Class and Mapped Short-Period Spectral Response Acceleration S_s

	Mapped Spectral Acceleration at Short Periods S_s				
Site Class	**S_s £ 0.25**	**$S_s = 0.50$**	**$S_s = 0.75$**	**$S_s = 1.00$**	**S_s ³ 1.25**
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	*
F	*	*	*	*	*

Note: Use straight-line interpolation for intermediate values of S_s.

*Site-specific geotechnical investigation and dynamic site response analyses should be performed.

Table 3-2b
Values of F_v as a Function of a Site Class and Mapped Spectral Response Acceleration at One-Second Period S_1

	Mapped Spectral Acceleration at One-Second Period S_1				
Site Class	**S_1 £ 0.10**	**$S_1 = 0.20$**	**$S_1 = 0.30$**	**$S_1 = 0.40$**	**S_1 ³ 0.50**
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	*
F	*	*	*	*	*

Note: Use straight-line interpolation for intermediate values of S_1.

*Site-specific geotechnical investigation and dynamic site response analyses should be performed
.

but shall not be less than:

$$C_S = 0.044\, S_{DS} \qquad (3\text{-}9)$$

where:

T = The fundamental period of the structure. The above equations are shown graphically in Figure 3-1.

(2) Modal Analysis Procedure. The required modal periods, mode shapes, and participation factors shall be calculated by established methods of structural, analysis assuming a fixed-base condition.

(a) General response spectrum. Where a design response spectrum is required in this document, and where site specific procedures are not used, the design response-spectrum curve shall be developed as indicated in Figure 3-2, and as follows:

1. For periods equal or less than T_o, the design spectral response acceleration, S_a, shall be as given by the following equation:

$$S_a = 0.4\, S_{DS} + 0.6\, S_{DS}\, (T/T_o) \qquad (3\text{-}10)$$

Where $T_O = 0.2T_S$ and T_S is defined by Equation 3-13.

2. For periods greater than T_o and less than or equal to T_s, the design spectral response acceleration, S_a, shall be as given by the following equation:

$$S_a = S_{DS} \qquad (3\text{-}11)$$

3. For periods greater than T_s, the design spectral response acceleration shall be as given by the following equation:

$$S_a = \frac{S_{D1}}{T} \qquad (3\text{-}12)$$

where the value of T_s shall be as given by the following equation:

$$T_s = \frac{S_{D1}}{S_{DS}} \qquad (3\text{-}13)$$

(b) Modal base shear. The portion of the base shear contributed by the m^{th} mode, V_m, shall be determined from the following equations:

$$V_m = C_{sm}\overline{W_m} \qquad (3\text{-}14)$$

$$\overline{W_m} = \frac{\left(\sum_{i=1}^{n} w_i f_{im}\right)^2}{\sum_{i=1}^{n} w_i f_{im}^2} \qquad (3\text{-}15)$$

where:

C_{sm} = the modal seismic response coefficient determined below,

$\overline{W_m}$ = the effective modal gravity load including portions of the live load as defined in Sec. 5.3.2 of FEMA 302,

w_i = the portion of the total gravity load of the structure at level i, and

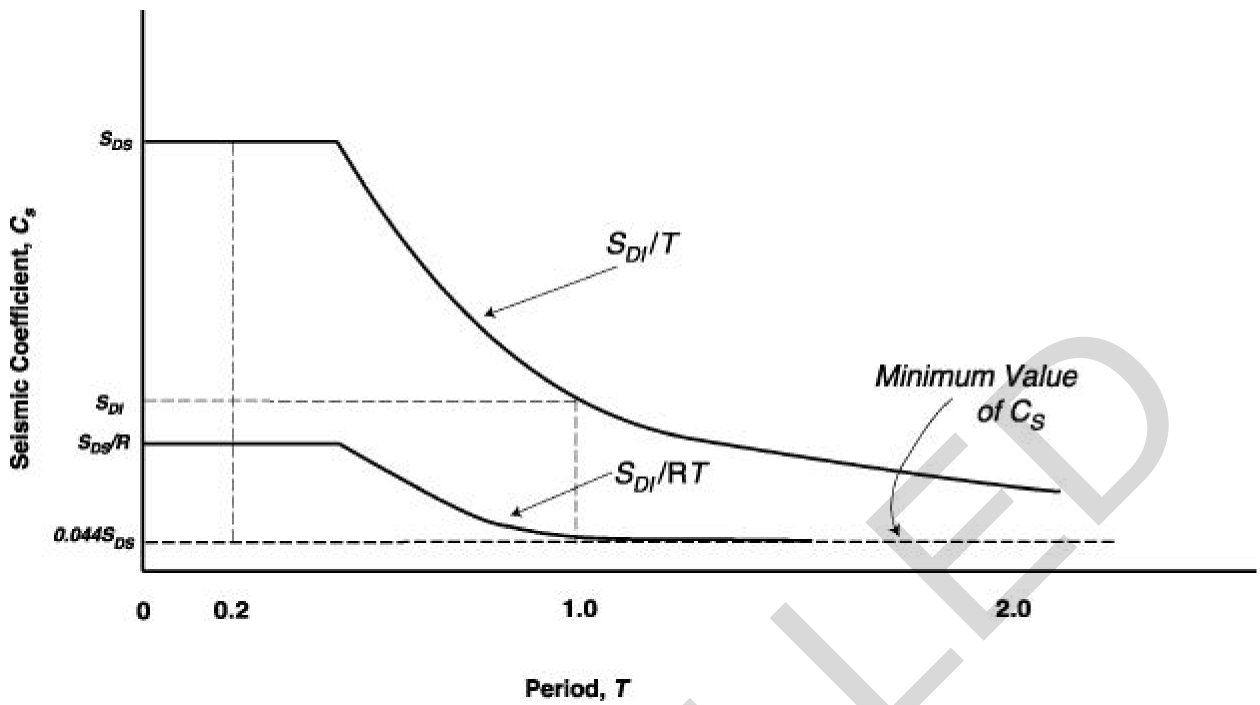


Figure 3-1 Seismic coefficient, C_{S}.

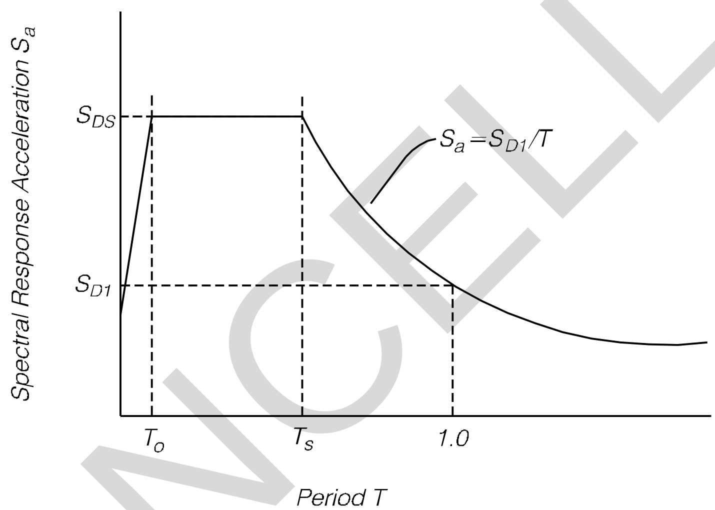


Figure 3-2 Design response spectrum.

f_{im} = the displacement amplitude of the i^{th} level of the structure when vibrating in its m^{th} mode.

The modal seismic response coefficient, C_{sm}, shall be determined in accordance with the following equation:

$$C_{sm} = \frac{S_{am}}{R} \qquad (3\text{-}16)$$

where:

S_{am} = The design response acceleration at period T_m determined from either the general design response spectrum of Paragraph 3-2c (2)(a), or a site-specific response spectrum per Paragraph 3-5,

R = the response modification factor determined from Table 7-1, and

T_m = the modal period of vibration (in seconds) of the m^{th} mode of the structure.

Exceptions:

1. When the general design response spectrum of Paragraph 3-2c (2)(a) is used for structures on Site Class D, E, or F soils, the modal seismic design coefficient, C_{sm}, for modes other than the fundamental mode that have periods less than 0.3 seconds is permitted to be determined by the following equation:

$$C_{sm} = \frac{0.4S_{DS}}{R}(1.0 + 5.0T_m) \qquad (3\text{-}17)$$

Where S_{DS} is as defined in Paragraph 3-2b, and R and T_m, are as defined above.

2. When the general design response spectrum of Paragraph 3-2c(2)(a) is used for structures where any modal period of vibration, T_m, exceeds 4.0 seconds, the modal seismic design coefficient, C_{sm}, for that mode is permitted to be determined by the following equation:

$$C_{sm} = \frac{4S_{D1}}{(R)T_m^2} \qquad (3\text{-}18)$$

Where R, and T_m are as defined above, and S_{D1} is the design spectral response acceleration at a period of 1 second as determined in Paragraph 3-2b.

(c) Modal forces, deflections, and drifts. The modal force, F_{xm}, at each level shall be determined by the following equations:

$$F_{xm} = C_{vxm} V_m \qquad (3\text{-}19)$$

and

$$C_{vxm} = \frac{w_x f_{xm}}{\sum_{i=1}^{n} w_i f_{im}} \qquad (3\text{-}20)$$

where:

C_{vxm} = the vertical distribution factor in the m^{th} mode,

V_m = the total design lateral force or shear at the base in the m^{th} mode,

w_p w_x = the portion of the total gravity load, *W*, located or assigned to Level *i* or *x*.

N_{xm} = the displacement amplitude at the x^{th} level of the structure when vibrating in its m^{th} mode,

and

N_{im} = the displacement amplitude at the i^{th} level of the structure when vibrating in its m^{th} mode.

The modal deflection at each level, $*_{xm}$, shall be determined by the following equations:

$$*_{xm} = C_d \; *_{xem} \qquad (3\text{-}21)$$

and

$$d_{xem} = \left(\frac{g}{4p^2}\right)\left(\frac{T_m^2 F_{xm}}{W_x}\right) \qquad (3\text{-}22)$$

where:

C_d = the deflection amplification factor determined from Table 7-1,

$*_{xem}$ = the deflection of Level x in the m^{th} mode at the center of the mass at Level x determined by an elastic analysis,

g = the acceleration due to gravity (ft/s^2 or m/s^2),

T_m = the modal period of vibration, in seconds, of the m^{th} mode of the structure,

F_{xm} = the portion of the seismic base shear in the m^{th} mode, induced at Level *x*, and

w_x = the portion of the total gravity load of the structure, *W*, located or assigned to Level *x*. The modal drift in a story, $)_m$, shall be computed as the difference of the deflections, $*_{xm}$, at the top and bottom of the story under consideration.

(d) Design values. The design values for the modal base shear, each of the story shear, moment, and drift quantities, and the deflection at each level shall be determined by combining their modal values as obtained above. The combination shall be carried out by taking the square root of the sum of the squares (SRSS) of each of the modal values or by the complete quadratic combinations (CQC) technique.

d. Design values for sites outside the U.S. Table 3-2 in TM 5-809-10 assigns seismic zones to selected locations outside the United States. The seismic zones in that table are consistent with the design values in the 1991 Uniform Building Code (UBC). Table 3-3 in this document provides spectral ordinates that have been derived to provide comparable base shear values.

(1) Algorithms to convert UBC zones to spectral ordinates. The UBC base shear equations are as follows:

$$V = \frac{ZIC}{R_w} W \qquad (3\text{-}23)$$

where

Location	S_s	S_I
AFRICA:		
Algeria:		
Alger	1.24	0.56
Oran	1.24	0.56
Angola:		
Luanda	0.06	0.06
Benin:		
Cotonou	0.06	0.06
Botswana:		
Gaborone	0.06	0.06
Burundi:		
Bujumbura	1.24	0.56
Cameroon:		
Douala	0.06	0.06
Yaounde	0.06	0.06
Cape Verde:		
Praia	0.06	0.06
Central African Republic:		
Bangui	0.06	0.06
Chad:		
Ndjamena	0.06	0.06
Congo:		
Brazzaville	0.06	0.06
Djibouti:		
Djibouti	1.24	0.56
Egypt:		
Alexandria	0.62	0.28
Cairo	0.62	0.28
Port Said	0.62	0.28
Equatorial Guinea:		
Malabo	0.06	0.06
Ethiopia:		
Addis Ababa	1.24	0.56
Asmara	1.24	0.56
Gabon:		
Libreville	0.06	0.06
Gambia:		
Banjul	0.06	0.06
Ghana:		
Accra	1.24	0.56
Guinea:		
Bissau	0.31	0.14
Conakry	0.06	0.06
Ivory Coast:		
Abidijan	0.06	0.06
Kenya:		
Nairobi	0.62	0.28
Lesotho:		
Maseru	0.62	0.28
Liberia:		
Monrovia	0.31	0.14
Libya:		
Tripoli	0.62	0.28
Wheelus AFB	0.62	0.28
Malagasy Republic:		
Tananarive	0.06	0.06
Malawi:		
Blantyre	1.24	0.56
Lilongwe	1.24	0.56
Zomba	1.24	0.56
Mali:		
Bamako	0.06	0.06
Mauritania:		
Nouakchott	0.06	0.06
Mauritius:		
Port Louis	0.06	0.06
Morocco:		
Casablanca	0.62	0.28
Port Lyautey	0.31	0.14
Rabat	0.62	0.28
Tangier	1.24	0.56
Mozambique:		
Maputo	0.62	0.28
Niger:		
Niamey	0.06	0.06
Nigera:		
Ibadan	0.06	0.06
Kaduna	0.06	0.06
Lagos	0.06	0.06
Republic of Rwanda:		
Kigali	1.24	0.56
Senegal:		
Dakar	0.06	0.06
Seychelles		
Victoria	0.06	0.06
Sierra Leone:		
Freetown	0.06	0.06
Somalia:		
Mogadishu	0.06	0.06
South Africa:		
Cape Town	1.24	0.56
Durban	0.62	0.28
Johannesburg	0.62	0.28
Natal	0.31	0.14
Pretoria	0.62	0.28
Swaziland:		
Mbabane	0.62	0.28
Tanzania:		
Dar es Salaam	0.62	0.28
Zanzibar	0.62	0.28
Togo:		
Lome	0.31	0.14
Tunisia:		
Tunis	1.24	0.56
Uganda:		
Kampala	0.62	0.28
Upper Volta:		
Ougadougou	0.06	0.06
Zaire:		
Bukavu	1.24	0.56
Kinshasa	0.06	0.06
Lubumbashi	0.62	0.28
Zambia:		
Lusaka	0.62	0.28
Zimbabwe:		
Harare (Salisbury)	1.24	0.56
ASIA		
Afghanistan:		
Kabul	1.65	0.75
Bahrain:		
Manama	0.06	0.06
Bangladesh:		
Dacca	1.24	0.56
Brunei:		
Bandar Seri Begawan	0.31	0.14
Burma:		
Mandalay	1.24	0.56
Rangoon	1.24	0.56
China:		
Canton	0.62	0.28
Chengdu	1.24	0.56
Nanking	0.62	0.28
Peking	1.65	0.75
Shanghai	0.62	0.28
Shengyang	1.65	0.75
Tibwa	1.65	0.75
Tsingtao	1.24	0.56
Wuhan	0.62	0.28
Cyprus:		
Nicosia	1.24	0.56
Hong Kong:		
Hong Kong	0.62	0.28
India:		
Bombay	1.24	0.56
Calcutta	0.62	0.28
Madras	0.31	0.14
New Delhi	1.24	0.56
Indonesia:		
Bandung	1.65	0.75
Jakarta	1.65	0.75
Medan	1.24	0.56
Surabaya	1.65	0.75
Iran:		
Isfahan	1.24	0.56
Shiraz	1.24	0.56
Tabriz	1.65	0.75
Tehran	1.65	0.75

Table 3-3

S_s	S_I	S_s	S_I	S_s	S_I

Table 3-3

	S_s	S_I
Iraq:		
Baghdad	1.24	0.56
Basra	0.31	0.14
Israel:		
Haifa	1.24	0.56
Jerusalem	1.24	0.56
Tel Aviv	1.24	0.56
Japan:		
Fukuoka	1.24	0.56
Itazuke AFB	1.24	0.56
Misawa AFB	1.24	0.56
Naha, Okinawa	1.65	0.75
Osaka/Kobe	1.65	0.75
Sapporo	1.24	0.56
Tokyo	1.65	0.75
Wakkanai	1.24	0.56
Yokohama	1.65	0.75
Yokota	1.65	0.75
Jordan:		
Amman	1.24	0.56
Korea:		
Kwangju	0.31	0.14
Kimhae	0.31	0.14
Pusan	0.31	0.14
Seoul	0.06	0.06
Kuwait:		
Kuwait:	0.31	0.14
Laos:		
Vientiane	0.31	0.14
Lebanon:		
Beirut	1.24	0.56
Malaysia:		
Kuala Lumpur	0.31	0.14
Nepal:		
Kathmandu	1.65	0.75
Oman:		
Muscat	0.62	0.28
Pakistan:		
Islamabad	1.68	0.75
Karachi	1.65	0.75
Lahore	0.62	0.28
Peshawar	1.65	0.75
Quatar:		
Doha	0.06	0.06
Saudi Arabia:		
Al Batin	0.31	0.14
Dhahran	0.31	0.14
Jiddah	0.62	0.28
Khamis Mushayf	0.31	0.14
Riyadh	0.06	0.06
Singapore:		
All	0.31	0.14
South Yemen:		
Aden City	1.24	0.56
Sir Lanka		
Colombo	0.06	0.06
Syria:		
Aleppo	1.24	0.56
Damascus	1.24	0.56
Taiwan:		
All	1.65	0.75
Thailand:		
Bangkok	0.31	0.14
Chinmg Mai	0.62	0.28
Songkhia	0.06	0.06
Udorn	0.31	0.14
Turkey:		
Adana	0.62	0.28
Ankara	0.62	0.28
Istanbul	1.65	0.75
Izmir	1.65	0.75
Karamursel	1.24	0.56
United Arab Emirates:		
Abu Dhabi	0.06	0.06
Dubai	0.06	0.06
Viet Nam:		
Ho Chi Minh City (Saigon)	0.06	0.06
Yemen Arab Republic		
Sanaa	1.24	0.56

ATLANTIC OCEAN AREA

	S_s	S_I
Azorea:		
All	0.62	0.28
Bermuda:		
All	0.31	0.14

CARIBBEAN SEA

	S_s	S_I
Bahama Islands:		
All	0.31	0.14
Cuba:		
All	0.62	0.28
Dominican Republic:		
Santo Domingo	1.24	0.56
French West Indies:		
Martinique	1.24	0.56
Grenada:		
Saint Georges	1.24	0.56
Haiti:		
Port au Prince	1.24	0.56
Jamaica:		
Kingston	1.24	0.56
Leeward Islands:		
All	1.24	0.56
Puerto Rico:		
All	0.83	0.38
Trinidad & Tobago:		
All	1.24	0.56

CENTRAL AMERICA:

	S_s	S_I
Belize:		
Beimopan	0.62	0.28
Canal Zone:		
All	0.62	0.28
Costa Rica:		
San Jose	1.24	0.56
El Salvador:		
San Salvador	1.65	0.75
Guatemala:		
Guatemala	1.65	0.75
Honduras:		
Tegucigalpa	1.24	0.56
Nicaragua:		
Managua	1.65	0.75
Panama:		
Colon	1.24	0.56
Galeta	0.83	0.38
Panama	1.24	0.56
Mexico:		
Ciudad Juarez	0.62	0.28
Guadalajara	1.24	0.56
Hermosillo	1.24	0.56
Matamoros	0.06	0.06
Mazatlan	0.60	0.28
Merida	0.06	0.06
Mexico City	1.24	0.56
Monterrey	0.06	0.06
Nuevo Laredo	0.06	0.06
Tijuana	1.24	0.56

EUROPE

	S_s	S_I
Albania:		
Tirana	1.24	0.56
Austria:		
Salzburg	0.62	0.28
Vienna	0.62	0.28
Belgium:		
Antwerp	0.31	0.14
Brussels	0.62	0.28
Bulgaria:		
Sofia	1.24	0.56
Czechoslovakia:		
Bratislava	0.62	0.28
Prague	0.31	0.14
Denmark:		
Copenhagen	0.31	0.14
Finland:		
Helsinki	0.31	0.14
France:		
Bordeaux	0.62	0.28
Lyon	0.31	0.14
Marseille	1.24	0.56
Nice	1.24	0.56
Strasbourg	0.62	0.28

Table 3-3

Location	S_s	S_I
Germany, Federal Republic:		
Berlin	0.06	0.06
Bonn	0.62	0.28
Bremen	0.06	0.06
Dusseldorf	0.31	0.14
Frankfurt	0.62	0.28
Hamburg	0.06	0.06
Munich	0.31	0.14
Stuttgart	0.62	0.28
Vaihigen	0.62	0.28
Greece:		
Athens	1.24	0.56
Kavalla	1.65	0.75
Makri	1.65	0.75
Rhodes	1.24	0.56
Sauda Bay	1.65	0.75
Thessaloniki	1.65	0.75
Hungary:		
Budapest	0.62	0.28
Iceland:		
Keflavick	1.24	0.56
Reykjavik	1.65	0.75
Ireland:		
Dublin	0.06	0.06
Italy:		
Aviano AFB	1.24	0.56
Brindisi	0.06	0.06
Florence	1.24	0.56
Genoa	1.24	0.56
Milan	0.62	0.28
Naples	1.24	0.56
Palermo	1.24	0.56
Rome	0.62	0.28
Sicily	1.24	0.56
Trieste	1.24	0.56
Turin	0.62	0.28
Luxembourg:		
Luxembourg	0.31	0.14
Malta:		
Valletta	0.62	0.28
Netherlands:		
All	0.06	0.06
Norway:		
Oslo	0.62	0.28
Poland:		
Krakow	0.62	0.28
Poznan	0.31	0.14
Waraszawa	0.31	0.14
Portugal:		
Lisbon	1.65	0.75
Oporto	1.24	0.56
Romania:		
Bucharest	1.24	0.56
Spain:		
Barcelona	0.62	0.28
Bilbao	0.62	0.28
Madrid	0.06	0.06
Rota	0.62	0.28
Seville	0.62	0.28
Sweden:		
Goteborg	0.62	0.28
Stockholm	0.31	0.14
Switzerland:		
Bern	0.62	0.28
Geneva	0.31	0.14
Zurich	0.62	0.28
United Kingdom:		
Belfast	0.06	0.06
Edinburgh	0.31	0.14
Edzell	0.31	0.14
Glasgow/Renfrew	0.31	0.14
Hamilton	0.31	0.14
Liverpool	0.31	0.14
London	0.62	0.28
Londonderry	0.31	0.14
Thurso	0.31	0.14
U.S.S.R.:		
Kiev	0.06	0.06
Leningrad	0.06	0.06
Moscow	0.06	0.06
Yugoslavia:		
Belgrade	0.62	0.28
Zagreb	1.24	0.56

NORTH AMERICA:

Location	S_s	S_I
Greenland:		
All	0.31	0.14
Canada:		
Argentia NAS	0.62	0.28
Calgary, Alb	0.31	0.14
Churchill, Man	0.06	0.06
Cold Lake, Alb	0.31	0.14
Edmonton, Alb	0.31	0.14
E. Harmon, AFB	0.62	0.28
Fort Williams, Ont	0.06	0.06
Frobisher N.W. Ter	0.06	0.06
Goose Airport	0.31	0.14
Halifax	0.31	0.14
Montreal, Quebec	1.24	0.56
Ottawa, Ont	0.62	0.28
St. John's Nfld	1.24	0.56
Toronto, Ont	0.31	0.14
Vancouver	1.24	0.56
Winnepeg, Man	0.31	0.14

SOUTH AMERICA:

Location	S_s	S_I
Argentina:		
Buenos Aires	0.25	0.10
Brazil:		
Belem	0.06	0.06
Belo Horizonte	0.06	0.06
Brasilia	0.06	0.06
Manaus	0.06	0.06
Porto Allegre	0.06	0.06
Recife	0.06	0.06
Rio de Janeiro	0.06	0.06
Salvador	0.06	0.06
Sao Paulo	0.31	0.14
Bolivia:		
La Paz	1.24	0.56
Santa Cruz	0.31	0.14
Chile:		
Santiago	1.65	0.75
Valparaiso	1.65	0.75
Colombia:		
Bogata	1.24	0.56
Ecuador:		
Quito	1.65	0.75
Guayaquil	1.24	0.56
Paraquay:		
Asuncion	0.06	0.06
Peru:		
Lima	1.65	0.75
Piura	1.65	0.75
Uruguay:		
Montevideo	0.06	0.06
Venezuela:		
Maracaibo	0.62	0.28
Caracas	1.65	0.75

PACIFIC OCEAN AREA:

Location	S_s	S_I
Australia:		
Brisbane	0.31	0.14
Canberra	0.31	0.14
Melbourne	0.31	0.14
Perth	0.31	0.14
Sydney	0.31	0.14
Caroline Islands:		
Koror, Paulau Is	0.62	0.28
Ponape	0.06	0.06
Fiji:		
Suva	1.24	0.56
Johnson Island:		
All	0.31	0.14
Mariana Islands:		
Guam	1.24	0.56
Saipan	1.24	0.56
Tinian	1.24	0.56
Marshall Islands:		
All	0.31	0.14
New Zealand:		
Auckland	1.24	0.56
Wellington	1.65	0.75
Papau New Guinea:		
Port Moresby	1.24	0.56
Phillipine Islands:		
Cebu	1.65	0.75
Manila	1.65	0.75
Baguio	1.24	0.56
Samoa:		
All	1.24	0.56
Wake Island:		
All	0.06	0.06

Table 3-3

S_s	S_I	S_s	S_I	S_s	S_I

Table 3-3

$$C = \frac{1.25S}{T^{2/3}} \quad (3\text{-}24)$$

but C need not exceed 2.75.

If the importance factor, I is eliminated in Equation 3-23, and if it is assumed that R_w with allowable stress design is comparable to the FEMA 302 reduction factor, R, with strength design, then by comparison with Equation 3-7,

$$S_{DS} = 2.75Z \quad (3\text{-}25)$$

Where Z is the seismic zone coefficient from Table 3-4. Similarly, Equation 3-24 can be compared with Equation 3-8 with T = 1.0 sec to yield:

$$S_{D1} = 1.25Z \quad (3\text{-}26)$$

(2) Spectral ordinates for Seismic Zone O. The Building Seismic Safety Council (BSSC) Design Values Group that developed the MCE maps recommended that, regardless of seismicity, all buildings should be designed to resist a lateral force of one percent of the building weight (i.e. C_z = 0.011x1). If an average value of 4.0 is assumed for the R factor in Equations 3-7 and 3-8, then

$$S_{DS} \text{ and } S_{D1} = 0.04 \quad (3\text{-}27)$$

(3) Conversion to S_S and S_1. The preceding subparagraph provides the basic relationship between the design parameters in the UBC and those in FEMA 302. It should be noted however, that the Site Adjustment Factor, S, is applied directly to the UBC design values in Equation 3-24, while the FEMA site factors, F_a and F_v, are applied to the MCE ordinates S_S and S_1 in Equations 3-1 and 3-2. The design parameters defined by Equations 3-25 and 3-26 have been multiplied by 1.50 to obtain the equivalent S_S and S_1 values listed in Table 3-3. The adjusted design values, S_{DS} and S_{D1}, for Earthquake A, can thus be obtained by multiplying the values in Table 3-3 by the appropriate local site adjustment factor, F_a or F_v, and multiplying the product by the 2/3 factor indicated in Equations 3-3 and 3-4. Similarly, for Ground Motion B, the product is multiplied by the ¾ factor indicated in Equations 3-28 and 3-29.

(4) Use of available data. As indicated in the above subparagraphs, the spectral ordinates listed in Table 3-3 are derived from the data contained in the current TM 5-809-10. These data are at least six years old, and the conversion is approximate. If better data are available in more recent publications, or from site-specific investigations, the data should be converted to the appropriate design parameters by the procedures outlined in this chapter.

3-3. Design Parameters for Ground Motion B.

a. General. The design parameters for Ground Motion B are those used in this document for Performance Objectives 2B (Safe Egress for Hazardous Occupancy) and 3B (Immediate Occupancy for Essential Facilities). Performance levels, ground motions, and performance objectives are summarized in Tables 4-3, and 4-4. Criteria for the seismic evaluation or design of essential military buildings have typically prescribed ground motion with 5 percent probability of exceedance in 50 years (i.e., a return period of about 1,000 years). This document prescribes three-quarters of the MCE as Ground Motion B for the design of essential buildings. As indicated in Paragraph 3-2, a direct comparison of the ground motion at ¾ of MCE, with that based on 5 percent probability of exceedance in 50 years, can only be made on a site-specific or regional basis. The pragmatic intent of ¾ of MCE was the specification of a ground motion for enhanced performance objectives that would be comparable to that specified in prior military documents.

b. Design Values. The design spectral response acceleration parameters, S_{DS} and S_{D1}, for Ground Motion B, shall be in accordance with the following:

$$S_{DS} = \tfrac{3}{4}\, S_{MS} \qquad (3\text{-}28)$$

$$S_{D1} = \tfrac{3}{4}\, S_{M1} \qquad (3\text{-}29)$$

Other design parameters, as defined in Paragraph 3-2 for the ELF or modal analysis procedures, shall be calculated using the above values of S_{DS} and S_{D1} and a response modification factor, R, of 1.0.

3-4. Site-Specific Determination of Ground Motion.

a. General. The site-specific determination of ground motion may be used for any structure, and should be considered where any of the following apply:

- The structure is assigned to Performance Objectives 2B or 3B.
- The site of the structure is within 10 kilometers of an active fault.
- The structure is located on Type F soils.
- A time history response analysis of the structure will be performed.
- The structure is to be designed with base isolation or energy dissipation.

Site-specific determination of the ground motion shall be performed only with prior authorization of the cognizant design authority. If a site-specific spectrum is determined for the design ground motion, the spectrum is permitted to be less than the general response spectrum given in Figure 3-2, but not less than 70 percent of that spectrum.

b. Required Expertise. Multi-disciplinary expertise is needed for the development of site-specific response spectra. Geological and seismological expertise are required in the characterization of seismic sources. The selection of appropriate attenuation relationships and the conduct of site response analyses requires expertise in geotechnical engineering and strong-motion seismology. Conduct of probabilistic seismic hazard analyses requires expertise in probabilistic modeling and methods. A team approach is therefore often appropriate for site-specific response spectrum development. It is important that the team or lead geotechnical specialist work closely with the design engineer to ensure a common understanding of design earthquakes, approaches to be followed in developing site-specific response spectra, and the nature and limitations of the ground motion outputs developed from the geotechnical studies. The peer review prescribed for Seismic Use Group III buildings in Paragraph 1-9a shall apply to the site-specific determination of ground motion for those buildings.

c. General Approaches. There are two general approaches to developing site-specific response spectra: deterministic approach, and probabilistic approach.

(1) In the deterministic approach, site ground motions are estimated for a specific, selected earthquake; that is, an earthquake of a certain size occurring on a specific seismic source at a certain distance from the site. Often, the earthquake is selected to be the largest earthquake judged to be capable of occurring on the seismic source, or the maximum earthquake, and is assumed to occur on the portion of the seismic source that is closest to the site. After the earthquake magnitude and distance are selected, site ground motions are then deterministically estimated using applicable ground-motion attenuation relationships (see Paragraph 3f below), statistical analyses of ground motion data recorded under similar conditions, or other techniques.

(2) In the probabilistic approach, site ground motions are estimated for selected values of annual frequency or return period for ground motion exceedance, or probability of ground motion exceedance in a certain exposure time (or design time period). The probability of exceeding a certain level of ground motion at a site is a function of the locations of seismic sources and the uncertainty of future earthquake locations on the sources, the frequency of occurrence of earthquakes of different magnitudes on the various sources, and the source-to-site ground motion attenuation, including its uncertainty.

(3) In this document, site specific Ground Motions A and B are determined using both probabilistic and deterministic parameters.

(a) In regions where active faults have not been identified, design ground motions shall be determined using a probabilistic approach as two-thirds (for Ground Motion A) and three-fourths (for ground Motion B) of ground motions having a 2 percent probability of exceedance in 50 years.

(b) In regions where active faults have been identified, ground motions shall be determined using both a probabilistic and a deterministic approach. Design ground motions may be the lesser of: (1) two-thirds (for Ground Motion A) or three-fourths (for Ground Motion B) of ground motions having a probability of exceedance of 2 percent in 50 years; and (2) two-thirds (for Ground Motion A) or three-fourths (for Ground Motion B) of ground motions determined deterministically as one- and one-half times the median (50th percentile) ground motions estimated assuming the occurrence of maximum magnitude earthquakes on portions of active faults closest to the site. Furthermore, in regions having active faults, design ground motions shall not be lower than ground motions that have a 10 percent probability of exceedance in 50 years for Ground Motion A, or 5 percent probability of exceedance in 50 years for Ground Motion B. The following paragraphs provide guidance for conducting a probabilistic ground motion analysis.

d. *Overview of Methodology*. The development of site-specific response spectra using a probabilistic approach involves the following basic steps: (1) characterizing earthquake sources in terms of their locations and geometrics, maximum earthquake magnitudes, and frequency of earthquake occurrence; (2) characterizing source-to-site ground motion attenuation; (3) carrying out a probabilistic ground motion analysis (often termed a probabilistic seismic hazard analysis, or PSHA) using inputs from (1) and (2); and (4) developing response spectra from the PSHA results. These basic steps are illustrated in Figures 3-3 and 3-4. Figure 3-3 is for the case where a PSHA is carried out for peak ground acceleration (PGA) only, and the response spectrum is then constructed by anchoring a selected response spectrum shape to the value of PGA obtained from the PSHA for the selected probability level. Figure 3-4 is for the case where a PSHA is carried out for response spectral values as well as for PGA, and an equal-probability-of-exceedance (equal-hazard) response spectrum is directly determined from the PSHA results for the selected probability level. The effects of local soil conditions on response spectra are incorporated either directly through the choice of appropriate attenuation relationships or spectral shapes, or by supplemental analyses of site effects in the case where the PSHA is carried out for rock motions at the site. The following paragraphs summarize the different steps involved in developing site-specific response spectra; details of the methodology, including the mathematical formulation of the probabilistic model, are described in Appendix E. Examples of the development of site-specific ground motions using PSHA methodology are also presented in Appendix E. Guidance and computer programs for PSHA are also described in Navy publications TR-2016-SHR and TR-2076-SHR (Ferritto, 1994, 1997).

e. Characterizing Earthquake Sources.

(1) Source identification. Seismic sources are identified on the basis of geological, seismological, and geophysical studies. In the western United States (WUS), i.e., west of the Rocky Mountains) major seismic sources include active faults that have been identified on the basis of surface and subsurface evidence. For example, major active faults in California are shown in map view in Figure 3-5. An example of faults mapped in a localized region of the western U.S. (San Francisco Bay area) is shown in Appendix E, Figure E-10. In some coastal regions of the WUS, specifically northwest California, Oregon, Washington, and southern Alaska, major earthquake sources also include subduction zones, which are regions where a tectonic plate of the earth's crust is thrusting beneath an adjacent tectonic plate. For example, a cross section through the subduction zone in the Puget Sound area of Washington is shown in Figure 3-6. In the eastern U.S. (EUS), earthquake faults typically do not have surface expression, and their subsurface location is usually not precisely known. Accordingly, earthquake sources in the EUS are usually characterized as zones with the zone boundaries selected on the basis of boundaries of geologic structures and/or patterns of seismicity. An example of seismic source zones developed for the EUS is described in Appendix E, Paragraph E-5c.

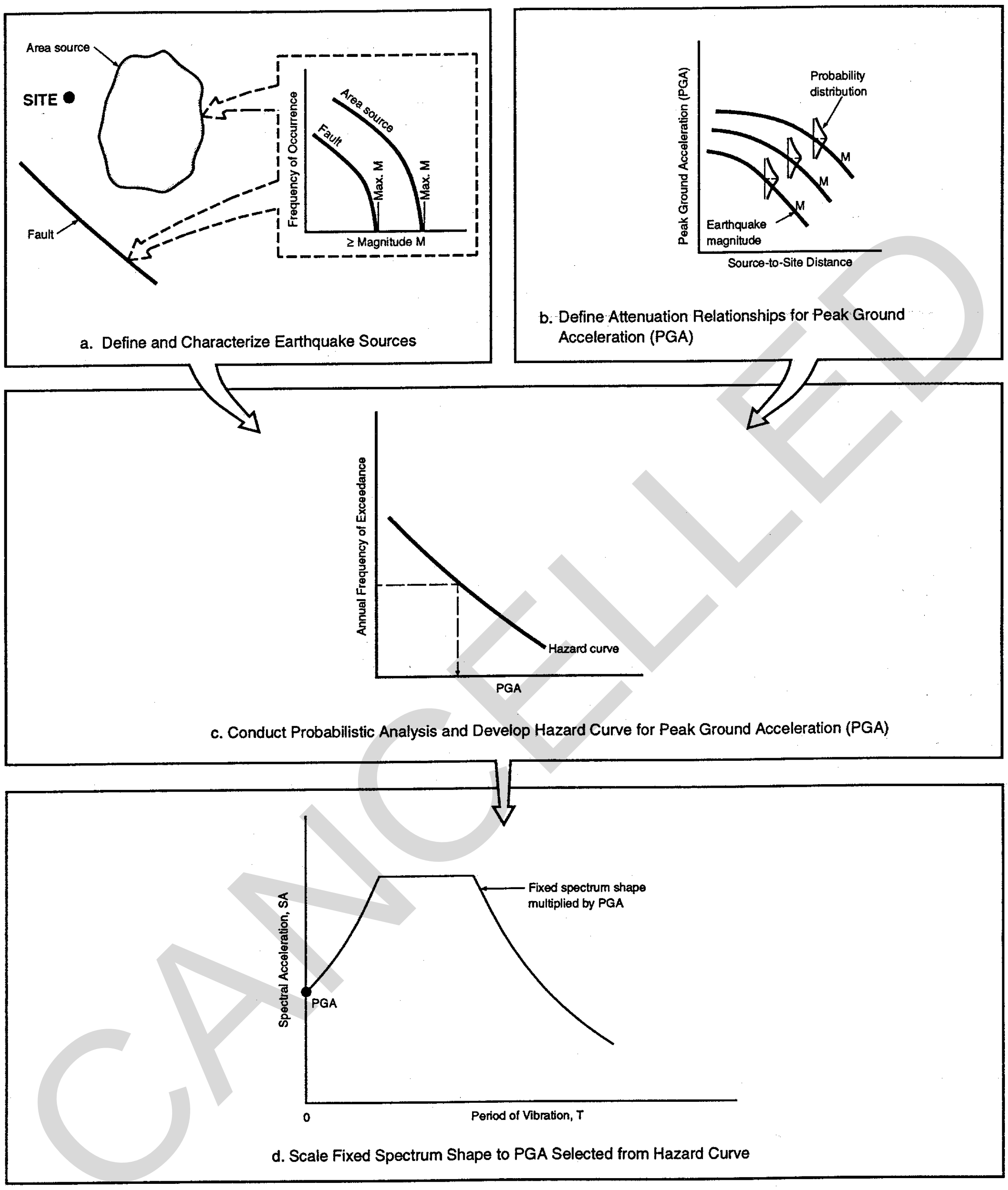


Figure 3-3 **Development of response spectrum based on a fixed spectrum shape and a probabilistic seismic hazard analysis for peak ground acceleration.**

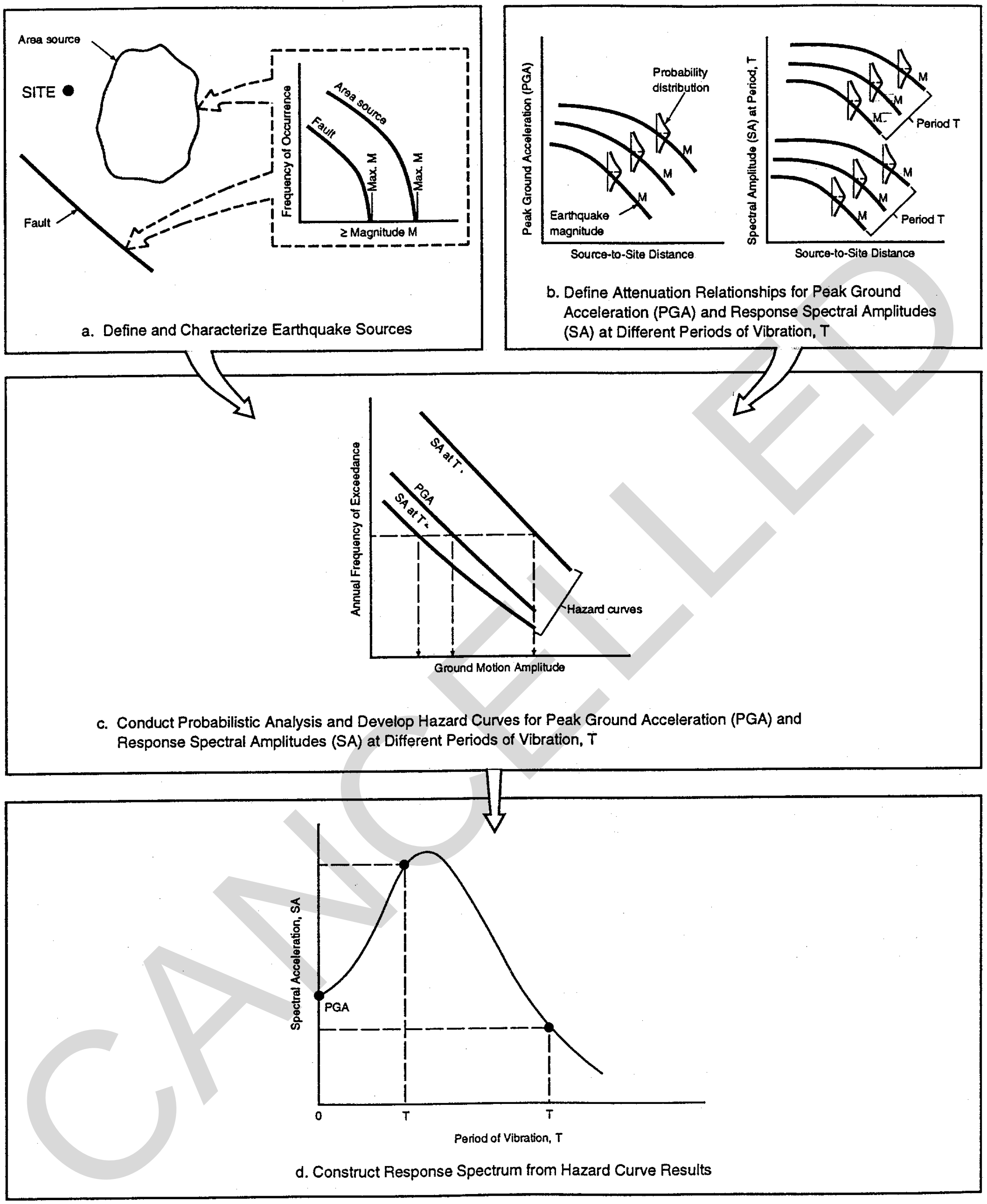


Figure 3-4 Development of equal-hazard response spectrum from probabilistic seismic hazard analysis for response spectral values.

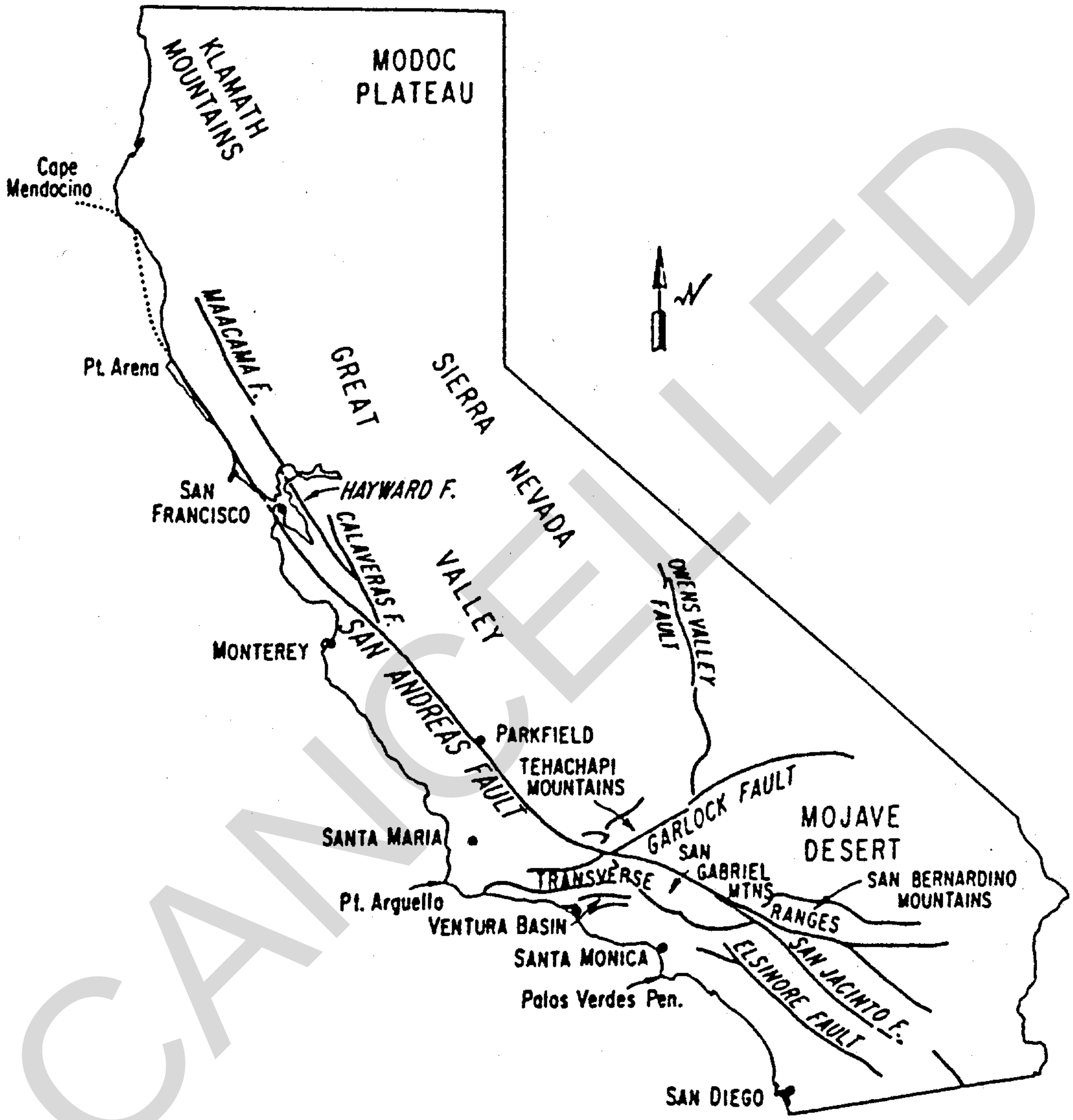
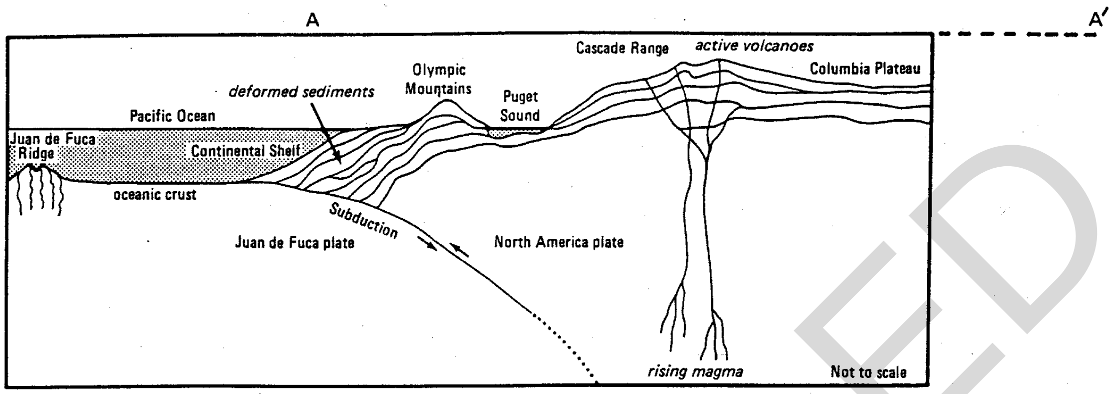
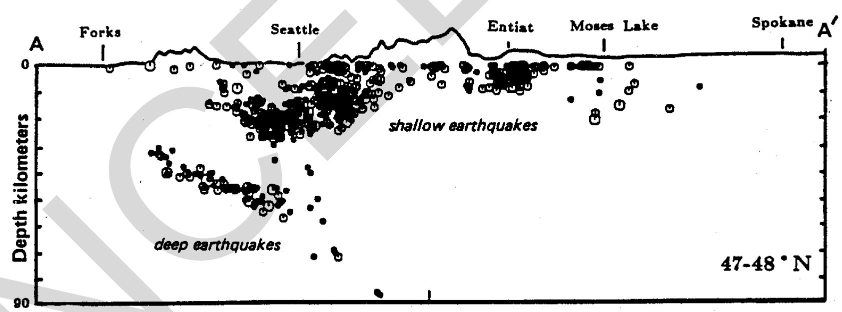


Figure 3-5 Major active faults in California (after Wesnousky, 1986).



(a) Geologic and geographic features



(b)) Seismicity (deep earthquakes indicate the slope of the zone of subduction; vertical exaggeration of 2 to 1 for seismicity plot indicates steeper than actual slope of subducting Juan de Fuca plate)

Figure 3-6 Cross section through Puget sound, Washington, showing subduction zone (from Nolson and others, 1988).

(2) Maximum earthquake magnitudes. Maximum magnitude is the physical limit of the size of an earthquake that can be generated by an earthquake source that is related to the dimensions of the source or source segments. For seismic sources in the WUS, maximum magnitudes are usually estimated by assessing the largest dimension (e.g., area) of the source expected to rupture in a single event, and then using empirical relationships that relate earthquake magnitude to rupture size. An example of a correlation between rupture area and earthquake moment magnitude is shown in Figure 3-7. In the EUS, because the source dimensions are typically unknown, there is a greater degree of uncertainty as to the maximum earthquake magnitude. Typically, maximum earthquake magnitudes in the EUS are estimated based on a conservative interpretation of (or extrapolation beyond) the historical seismicity on the source and by analogies to similar geologic structures and tectonic regimes throughout the world. Johnston et al. (1994) present a methodology for assessing maximum earthquake magnitude in the EUS based on an analysis of worldwide data for similar stable continental tectonic regions.

(3) Recurrence relationships. Recurrence relationships characterize the frequency of occurrence of earthquakes of various sizes, from the minimum magnitude of engineering significance to the maximum magnitude estimated for the source. Recurrence relationships are illustrated schematically in diagram A of Figure 3-3 and 3-4.

(a) Earthquake recurrence relationships must be developed for each identified seismic source that could significantly contribute to the seismic hazard at a site. Where earthquake sources are defined as area sources, recurrence relationships are usually developed on the basis of historical seismicity. For sources defined as faults, however, the available historical seismicity for the individual fault is usually insufficient to characterize recurrence rates, particularly for larger earthquakes, and use is typically made of geologic data to supplement the historical records. Geologic data include data on fault slip rates and data from paleo-seismic studies on the occurrence of large prehistoric earthquakes.

(b) Earthquake recurrence curves are usually described by either a truncated exponential recurrence model (Cornell and Vanmarke, 1969) based on Gutenberg and Richter's (1954) recurrence law, or a characteristic earthquake recurrence model (Schwartz and Coppersmith, 1984; Youngs and Coopersmith, 1985a, 1985b). The exponential relationship describes a rate of earthquake occurrence that increases exponentially as earthquake magnitude decreases. On the other hand, the characteristic relationship predicts that a relatively greater number of earthquakes (compared to the exponential relationship) will occur as "characteristic" magnitude events that are at or near the maximum magnitude for the source. A characteristic relationship is illustrated in Figure 3-8. Characteristic and exponential forms of recurrence relationships are compared in Figure 3-9. The exponential relationship is typically used for seismic sources defined as areas, whereas both exponential and characteristic earthquake models are used for individual fault sources. Detailed studies of earthquake recurrence in the Wasatch fault region, Utah, and in the San Francisco Bay region have shown excellent matches between regional seismicity rates and recurrence modeling when combining the

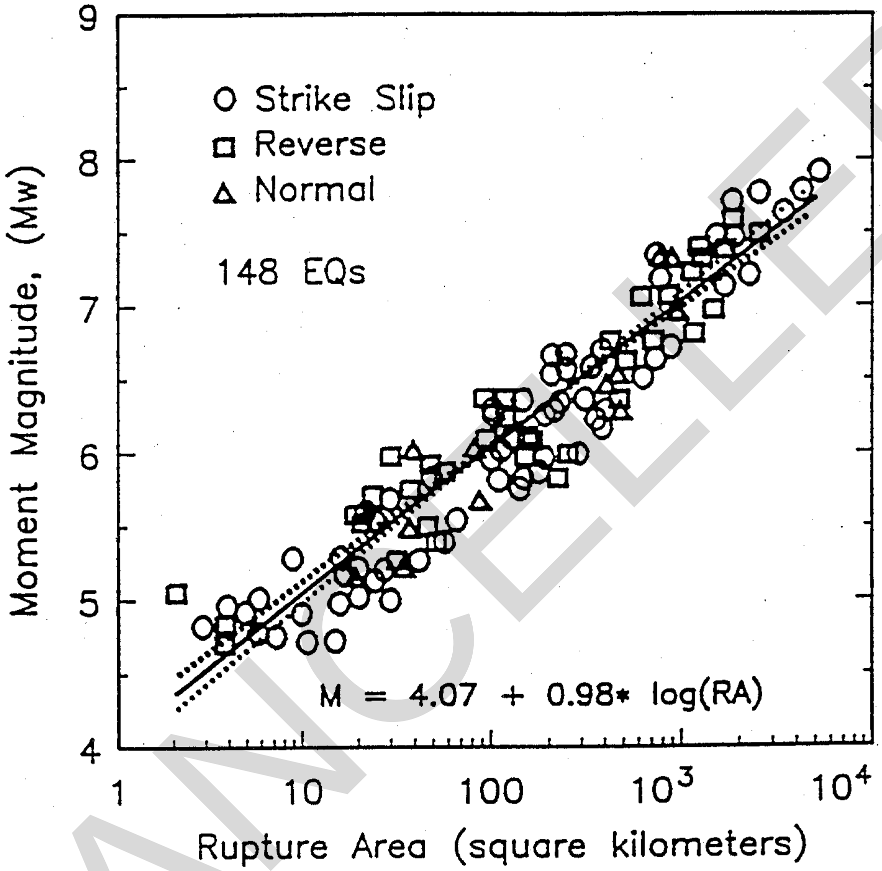


Figure 3-7 Relation between earthquake magnitude and rupture area (after Wells and Coppersmith, 1994).

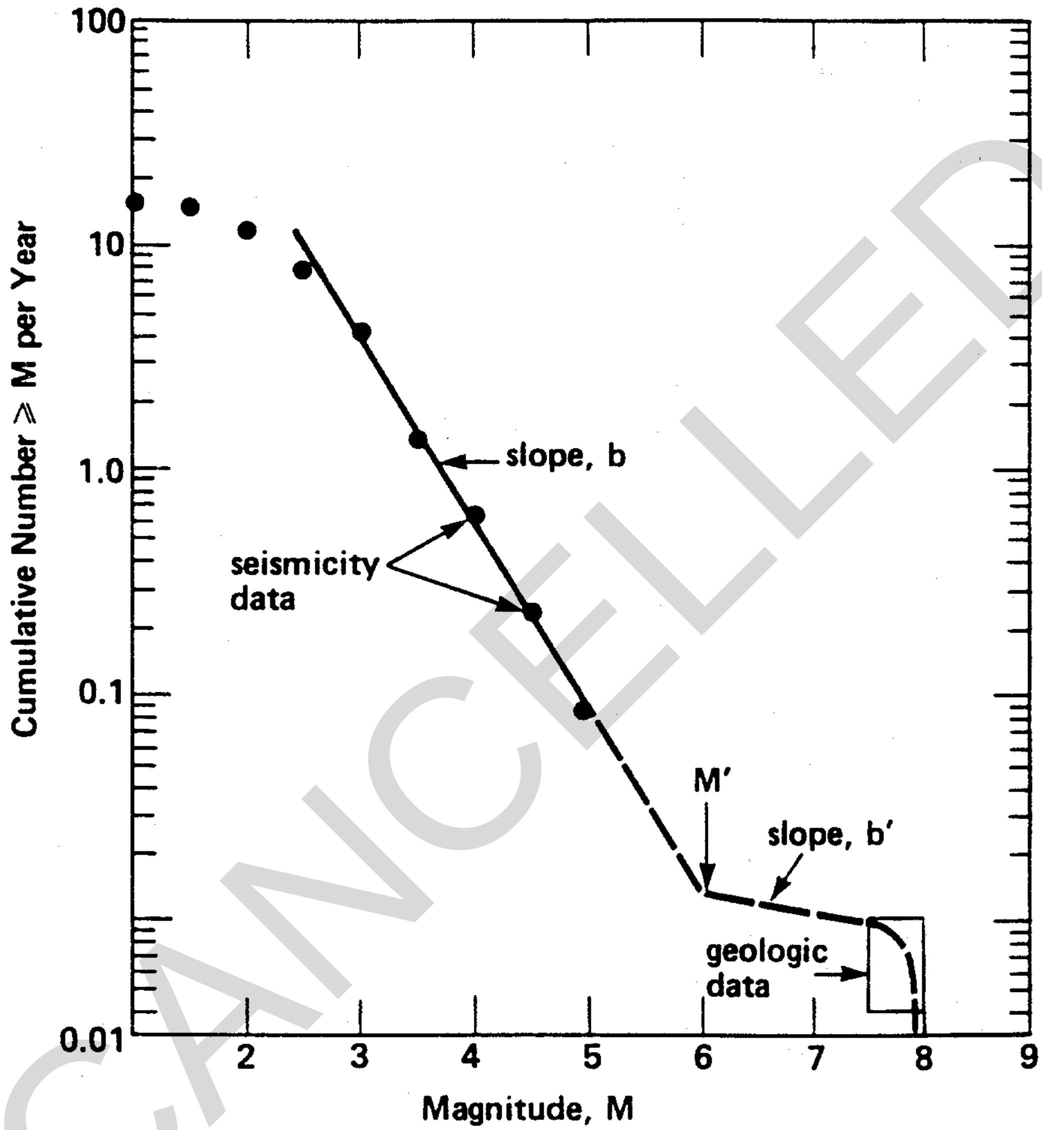


Figure 3-8 **Diagrammatic characteristic earthquake recurrence relationship for an individual fault or fault segment (from Schwartz and Coppersmith, 1984, and National Research council, 1988).**

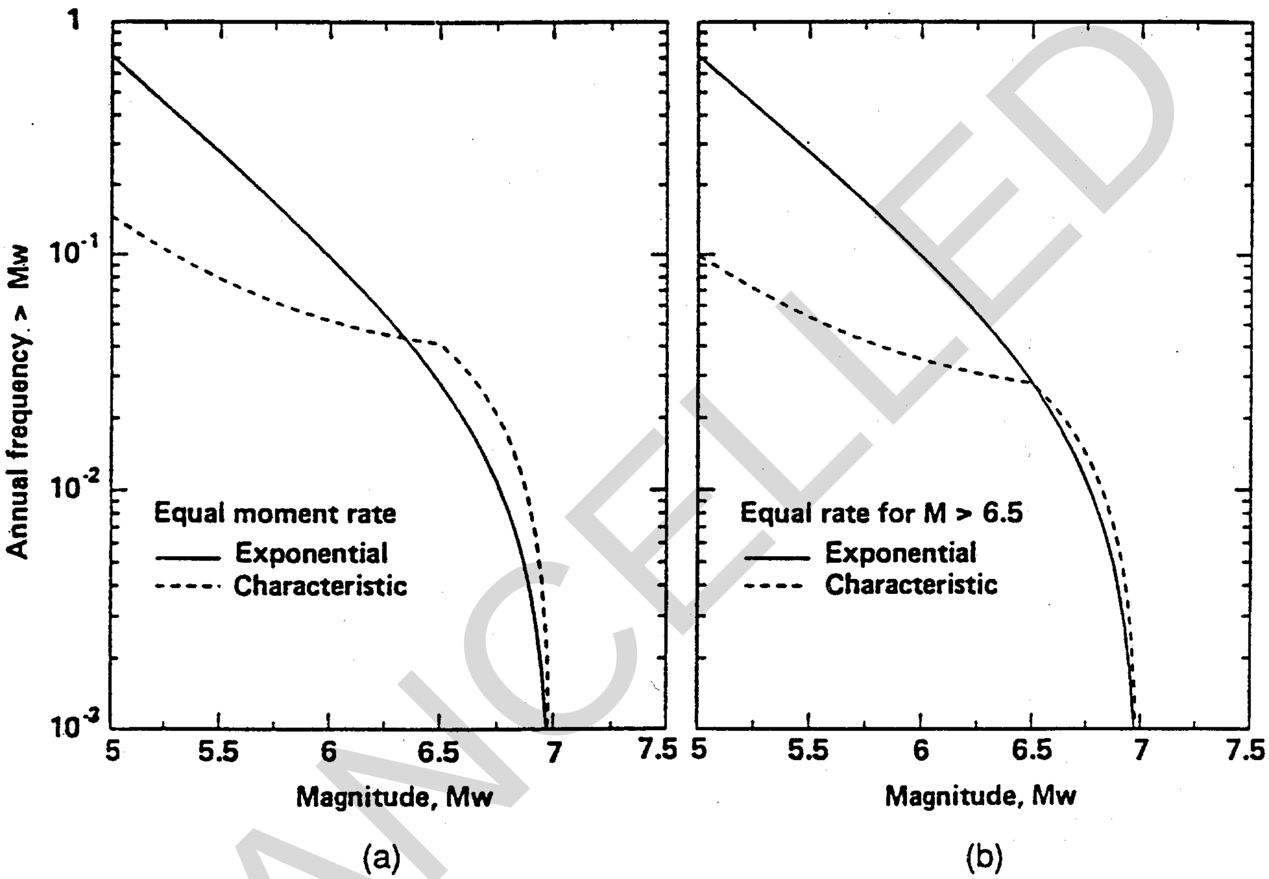


Figure 3-9 **Comparison of exponential and characteristic earthquake magnitude distributions.**

characteristic recurrence model for individual faults with the exponential model for distributed source areas (Youngs et al., 1987, 1988; Youngs et al., 1993); such a comparison is illustrated in the example in Appendix E for the San Francisco Bay region (Paragraph E-5b).

(c) A Poisson probability model is usually assumed for probabilistic ground motion analyses. In the Poisson model, earthquake occurrence in time is assumed to be random and memoryless. The probability of an earthquake in a given time period is thus determined by the average frequency of earthquakes, and is independent of when the last earthquake occurred. This model has been shown to be consistent with earthquake occurrence on a regional basis; however, it does not conform to the process believed to result in earthquakes on an individual fault — one of a gradual accumulation of strain followed by a sudden release. More realistic "real time" earthquake recurrence models have been developed that predict the probability of an earthquake in the next time period, rather than any time period, taking into account the past history (and paleo-history) of large earthquakes on a fault. Usually, there are insufficient geologic and seismic data on the timing of past earthquakes to justify the use of these models; however, real-time recurrence models have been used, for example, in the study of the probabilities of large earthquakes on the San Andreas fault system in Northern California by the Working Group on California Earthquake Probabilities (1990). These models can be considered for site-specific applications when there are sufficient data on the time-dependent occurrence of earthquakes on specific earthquake sources. Further discussion of earthquake recurrence models, including real-time models, is contained in Navy publication TR-2016-SHR (Ferritto, 1994).

f. Characterizing Ground Motion Attenuation.

(1) Attenuation relationships describe the variation of the amplitude of a ground motion parameter as a function of earthquake magnitude and source-to-site distance. A number of attenuation relationships have been developed for PGA and also for response spectral accelerations or velocities for different structural periods of vibration. Figure 3-10 illustrates typical attenuation relationships for PGA and response spectral accelerations for three periods of vibration. These relationships are in terms of earthquake moment magnitude, and the distance is the closest distance to the ruptured fault. The curves in Figure 3-10 are median (50th percentile) relationships. In a probabilistic ground-motion analysis, it is important to include the uncertainty in the ground motion estimates, which reflects the scatter in ground motion data. An example of ground motion data scatter for a single earthquake is illustrated in Figure 3-11. To model this source of uncertainty in ground motion estimation, a probabilistic distribution about the median-curves is assigned, as schematically illustrated in diagram b of Figures 3-3 and 3-4, and as illustrated by the plus-and-minus-one standard deviation curves in Figure 3-11. A log-normal distribution is typically used, and the standard deviation of the distribution is usually provided by the developer of the particular attenuation relationship.

(2) Attenuation relationships have been developed for different tectonic environments, including WUS shallow crustal, EUS, and subduction

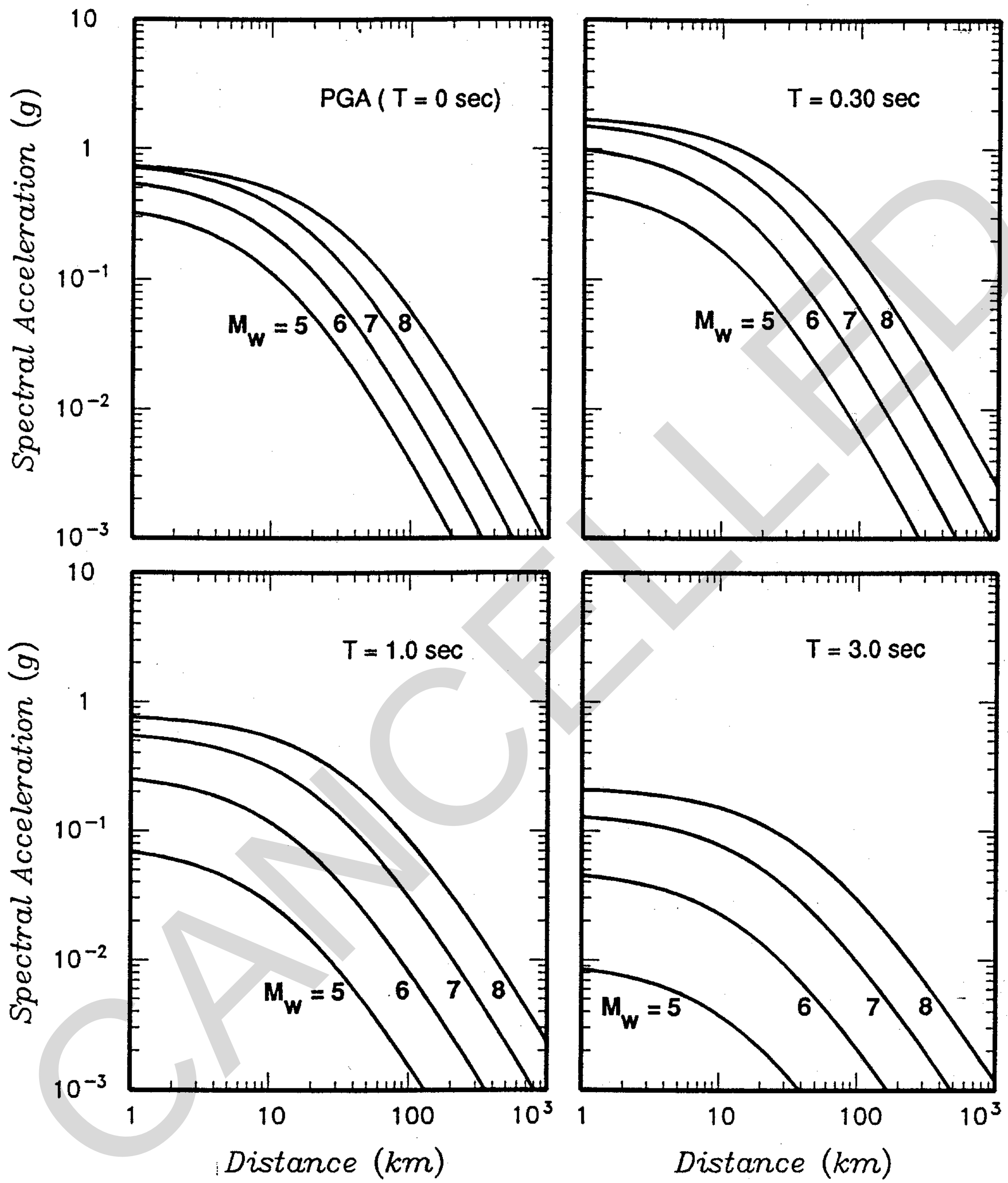


Figure 3-10 **Example of attenuation relationships for response spectral accelerations (5% damping).**

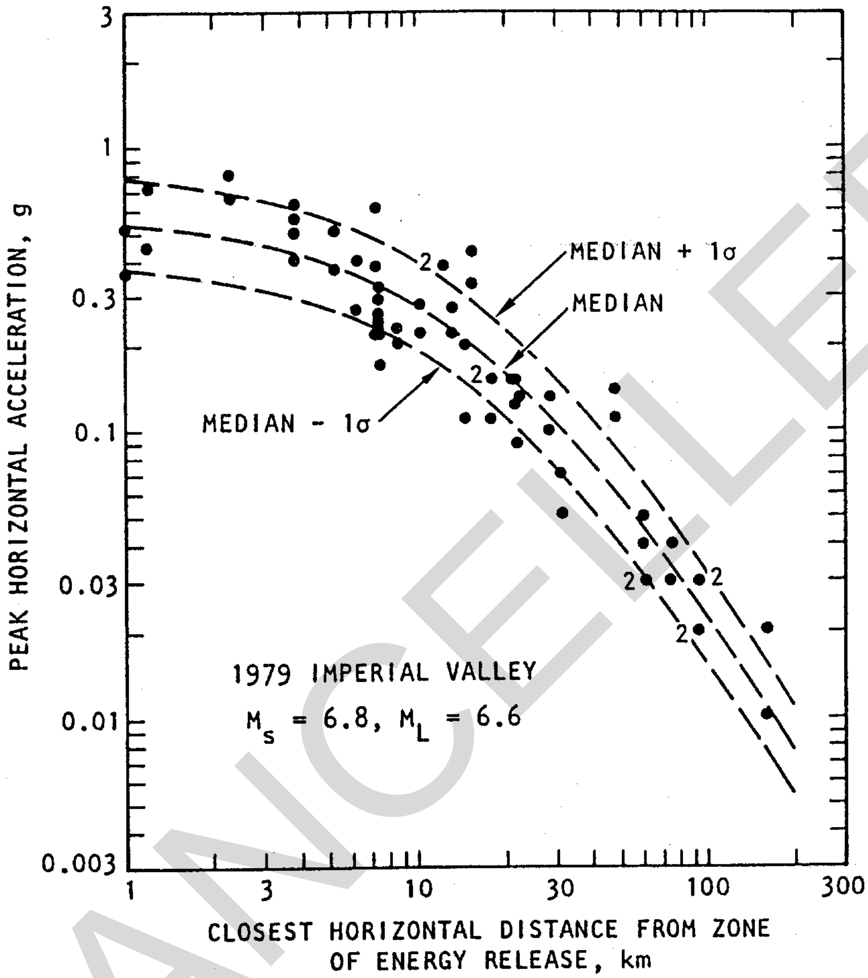


Figure 3-11 Example of ground motion data scatter for a single earthquake (from Seed and Idriss, 1982).

zone environments. Attenuation relationships have also been developed for different broad categories of subsurface conditions, particularly for the categories of rock and firm soils. In some cases, attenuation relationships have distinguished the effects of different types of faulting (e.g., strike-slip vs. reverse faulting). It is important to select a set of attenuation relationships that are most applicable to the site under consideration. Several recently developed relationships are summarized in Seismological Research Letters (1997).

g. *Conducting Probabilistic Seismic Hazard Analyses (PSHA*). The seismic source characterization and ground motion attenuation characterization are combined in a probabilistic model to develop relationships between the amplitude of a ground motion parameter and the probability or frequency of its exceedance (diagram c of Figures 3-3 and 3-4). These relationships are termed hazard curves. A hazard curve for PGA is illustrated in Figure 3-12. Appendix E describes the mathematical formulation for the seismic hazard model, and provides examples of its usage in obtaining hazard curves. The appendix also discusses the quantification of uncertainty in hazard curves as related to the uncertainty involved in the relationships and parameters of the model (i.e., uncertainty in seismic source parameters such as maximum earthquake magnitude, frequency of earthquake occurrence, etc., and uncertainty in the choice of appropriate attenuation relationships). It is important to incorporate these uncertainties in a PSHA in order to provide robust estimates of the mean hazard, and evaluate the uncertainties in the hazard.

h. Developing Response Spectra from the PSHA. Described below are two alternative approaches for obtaining response spectra based on PSHA: Approach 1 - anchoring a response spectrum shape to the PGAs determined from PSHA; Approach 2 - developing equal-hazard spectra directly from the PSHA. The two approaches are schematically illustrated in Figures 3-3 and 3-4, respectively.

(1) Approach 1 - Anchoring Response Spectrum Shape to PGA Determined from PSHA. In this alternative, the hazard analysis is carried out only for PGA, and the PGAs for the design ground motions are obtained from the hazard curve developed for the site. The response spectra are then constructed by anchoring appropriate response spectrum shapes to the PGA values. Typically, spectrum shapes for the appropriate category of subsurface condition, such as the shapes contained in the 1994 Uniform Building Code (UBC), are used. It should be noted, however, that widely used spectrum shapes, such as those in the 1994 UBC, were developed on the basis of predominantly WUS shallow crustal earthquake ground-motion data, and they may not be appropriate for EUS earthquakes or subduction zone earthquakes. Furthermore, such spectrum shapes are considered to be most applicable to moderate-magnitude earthquakes (magnitude . 6 1/2) and close to moderate distances (distance < 100 km). For larger magnitudes and distances, the shapes may be unconservative in the long-period range; conversely, for smaller magnitudes, the shapes may be overly conservative for long periods. To assess the appropriateness of the spectrum

shapes, the results of a PSHA may be analyzed to determine the dominant magnitude and distance contributions to the seismic hazard. The dominant magnitudes and

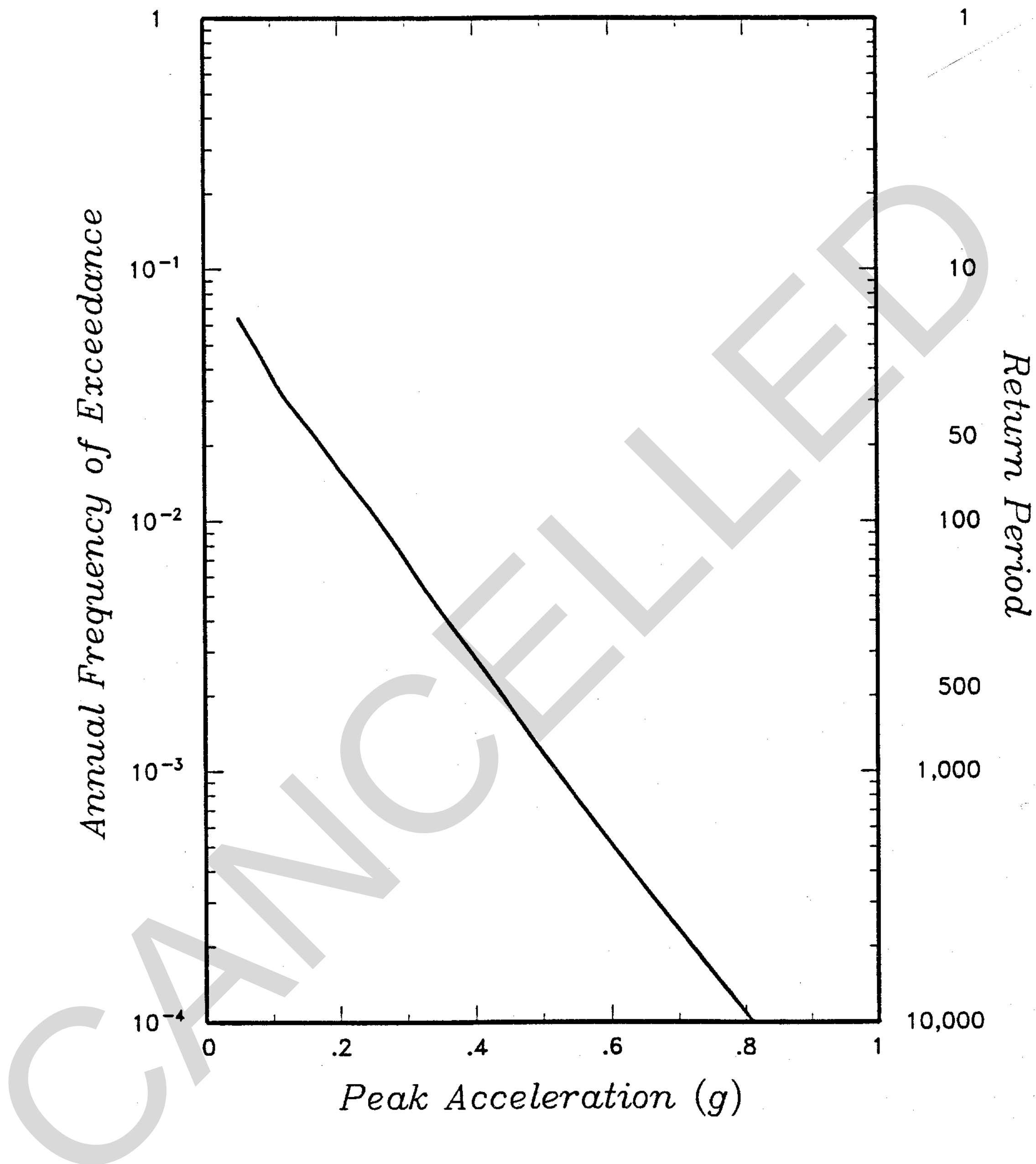


Figure 3-12 **Example seismic hazard curve showing relationship between peak ground acceleration and annual frequency of exceedance.**

distances will, in some cases, differ significantly for different probability levels. Usually the dominant magnitudes increase as the probability of exceedance decreases (e.g., larger dominant magnitudes for Ground Motion B than for Ground Motion A); this is illustrated in Appendix E.

(2) Approach 2 - Developing Equal Hazard Response Spectra Directly From PSHA. In Approach 2, the hazard analysis is carried out for response spectral values at a number of periods of vibration (using response spectral attenuation relationships), as well as for PGA. For the probability levels for the design ground motions, the response spectral values are obtained from the hazard curves, and are then plotted versus period of vibration. A smooth curve is then drawn through the response spectral values obtained for each earthquake, resulting in an equal-hazard response spectrum for each earthquake; that is, a spectrum having the same probability of exceedance at each period of vibration. The process of constructing equal-hazard response spectra from hazard curve results is illustrated in Figure 3-13 for the same site for which the PGA hazard curve was constructed in Figure 3-12. The example in Figure 3-13 is for a return period of 1,000 years, which is approximately equal to the return period for Ground Motion B. (Note in Figure 3-13 that PGA is identically equal to zero-period response spectral acceleration at periods equal to or less than 0.03 second). In general, response spectra should be developed using Approach 2 rather than Approach 1. This is partly because response spectral attenuation relationships needed for Approach 2 are available for both EUS and WUS, and are as reliable as attenuation relationships for PGA. Also, by using Approach 2, the resulting response spectrum will directly incorporate the effects of tectonic environment, magnitude, distance, and probability level on response spectral shape.

i. Accounting for Local Site Effects on Response Spectra.

(1) If the site is a rock site, local soil amplification effects are not applicable, and the response spectrum is directly obtained from the PSHA using attenuation relationships and response spectrum shapes for rock motions.

(2) If the site is a soil site, it is important to account for soil amplification effects on response spectra. Such effects can be very strong in many cases, such as the case illustrated in Figure 3-14, in which ground motions recorded on a soft soil site (Treasure Island) during the 1989 Loma Prieta earth- quake were amplified greatly in comparison to motions recorded on an adjacent rock site (Yerba Buena Island).

(3) Two approaches for incorporating soil amplification effects are: (1) by directly incorporating soil amplification effects in the PSHA through the use of attenuation relationships applicable to the soil conditions at the site; and (2) by developing rock response spectra at the site from a PSHA using rock attenuation relationships, and then carrying out site response analyses to assess the modifying influence of the soil column on the ground motions. The choice between Approaches 1 and 2 depends on whether attenuation relationships are available that are sufficiently applicable to the

soil conditions at the site (Approach 1), and whether site soil conditions are known in sufficient detail to be modeled for site

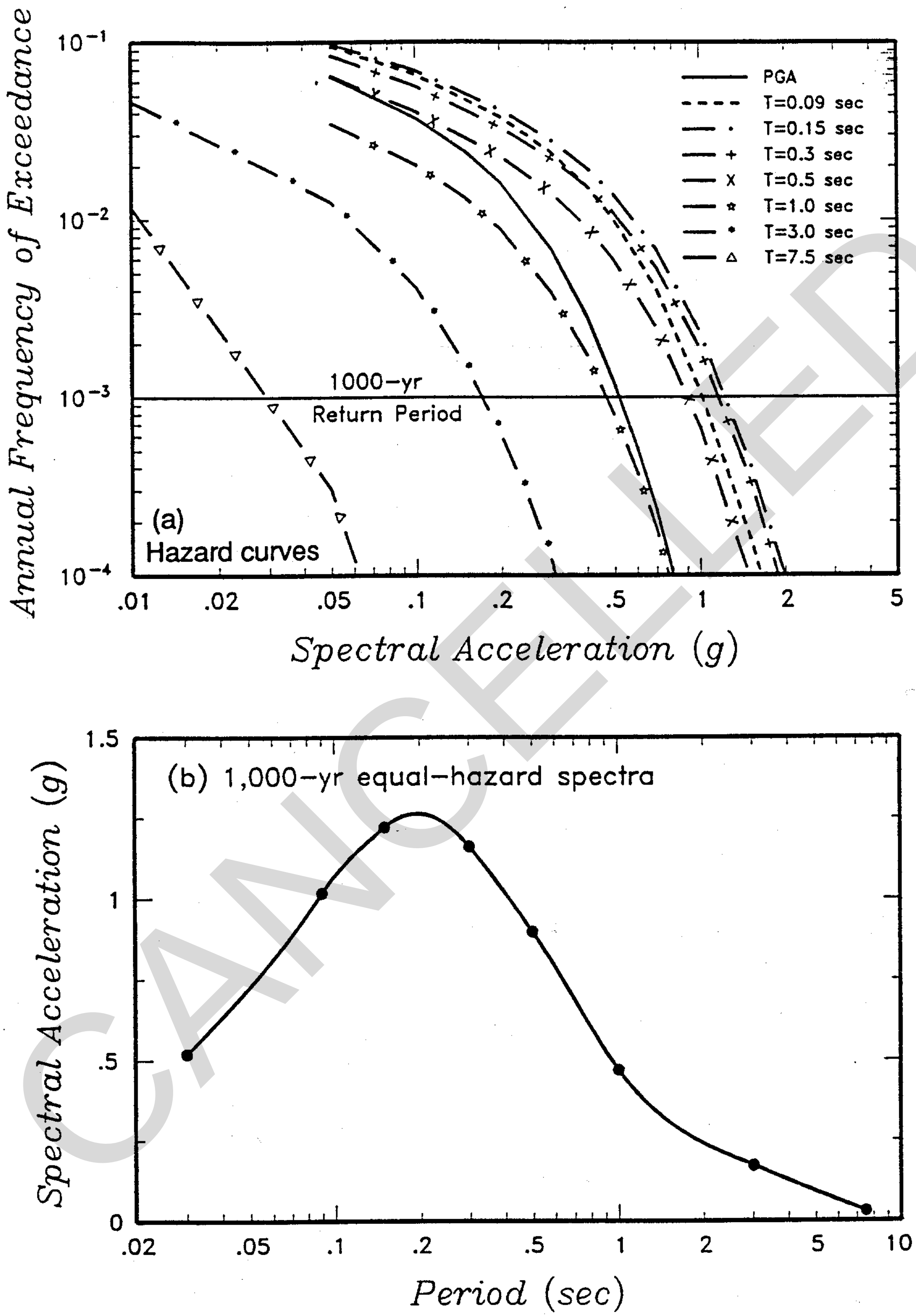
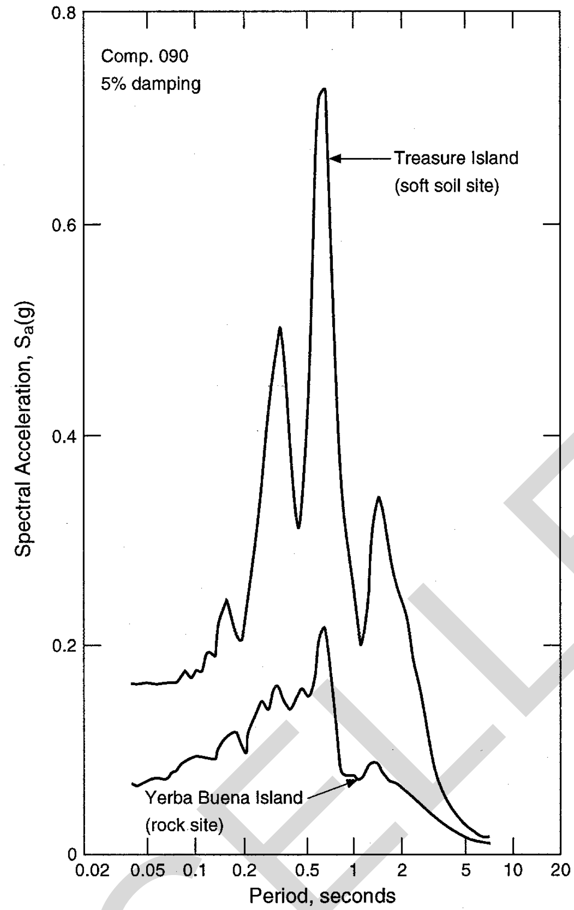
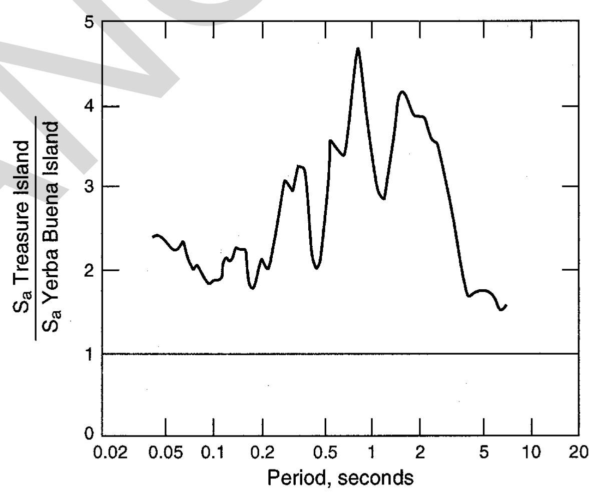


Figure 3-13 Construction of equal-hazard spectra.



(a) Response Spectra



(b) Ratio of Response Spectra

Figure 3-14 Response spectra and ratio of response spectra for ground motions recorded at a soft site and nearby rock site during the 1989 Loma Prieta earthquake.

response analysis (Approach 2). Approach 2 can always be considered as an alternative or supplement to approach 1.

(4) Soil amplification effects are stronger for soft clay soils than for stiff clays or dense sands, especially in the long-period range. Soil amplification is also increased by a large change in stiffness or shear wave velocity between the soils and underlying bedrock; therefore, it is particularly appropriate to conduct site response analyses when these conditions are present at a site.

(5) Site response analysis methodology is schematically illustrated in Figure 3-15. The soil profile between the ground surface and underlying rock is modeled in terms of its stratigraphy and dynamic soil properties. Acceleration time histories that are representative of the estimated rock motions are selected, and are propagated through the modeled soil profile using nonlinear or equivalent linear response analytical methods, and top-of-soil motions are obtained. As in other types of theoretical modeling and numerical analyses, site response analyses are sensitive to the details of the analytical procedures, soil dynamic properties, and input motions. The sensitivities should be carefully examined when these analyses are conducted.

(6) In certain cases, it may be appropriate to consider other types of site effects in developing site-specific ground motions. These include surface topographic effects when the surface topography is very irregular and could amplify ground motions, and subsurface basin or buried valley response effects when such two- and three-dimensional effects could significantly modify ground motions in comparison to the one-dimensional site response effects that are usually modeled.

j. Special Characteristics of Ground Motion for Near-Source Earthquakes. At close distances to the earthquake source, within approximately 10 to 15 km of the source, earthquake ground motions often contain a high energy pulse of medium- to long-period ground motion (at periods in the range of approximately 0.5 second to 5 seconds) that occurs when fault rupture propagates toward a site. It has also been found that these pulses exhibit a strong directionality, with the component of motion perpendicular (normal) to the strike of the fault being larger than the component parallel to the strike (see, for example, Somerville et al., 1997). These characteristics of near-source ground motions are illustrated in Figure 3-16, which shows the acceleration, velocity, and displacement time histories and response spectra of the Rinaldi recording obtained during the 1994 Northridge earthquake. These ground-motion characteristics should be incorporated in developing design response spectra, and when required, acceleration time histories for near-source earthquakes.

k. Vertical Ground Motions. For the design of some structures, it may be necessary to analyze the structure for vertical, as well as horizontal, ground motions. Generally, vertical design response spectra are obtained by applying vertical-to-horizontal ratios to horizontal design response spectra. Recent studies (e.g., Silva, 1997) indicate that vertical-to-horizontal response spectral ratios are a function of period of vibration, earthquake source-to-site distance,

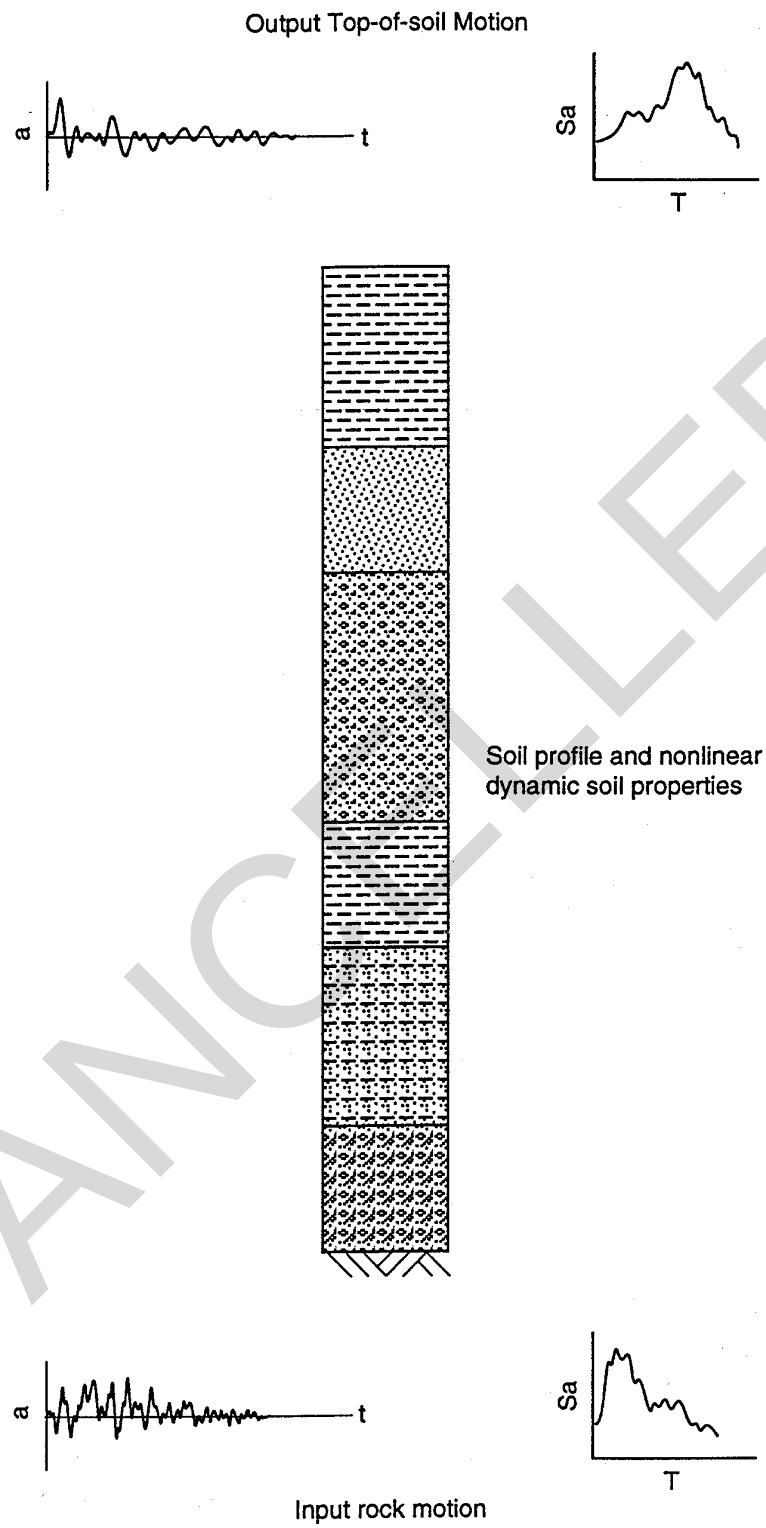


Figure 3-15 Schematic of site response analysis.

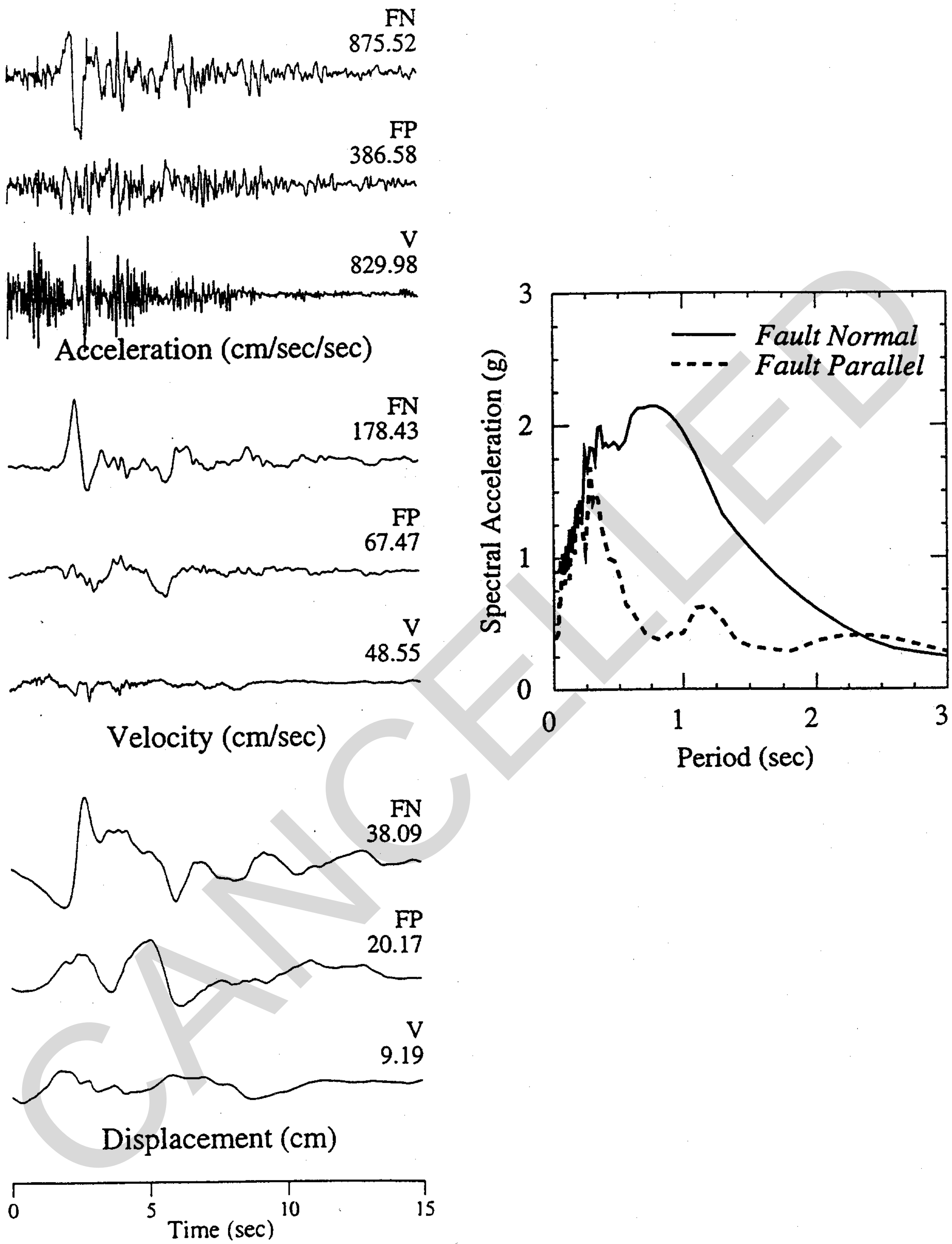


Figure 3-16 **Acceleration and velocity time histories for the strike-normal and strike-parallel horizontal components of ground motion, and their 5% damped response spectra, recorded at Rinaldi during the 1994 Northridge earthquake (Somerville, 1997).**

earthquake magnitude, tectonic environment (W.U.S. and E.U.S.) and subsurface conditions (soil and rock).

Figure 3-17 illustrates trends for these ratios as a function of period of vibration, source-to-site distance, and subsurface conditions for shallow crustal W.U.S. earthquakes of moment magnitude approximately equal to 6.5. The figure illustrates that the commonly used vertical-to-horizontal spectral ratio of two-thirds is generally conservative for longer-period ground motions, but is generally unconservative for short-period ground motions from near-source earthquakes. In fact, these ratios may significantly exceed 1.0 in some cases, as shown in Figure 3-17. Ratios such as those presented in Figure 3-17 may be used to construct design vertical response spectra of ground motions. However, the longer period (greater than 0.2 second) spectral values should be carefully examined, and it may be desirable to adopt for design vertical-to-horizontal spectral ratios for longer periods that are higher than the ratios shown in Figure 3-17.

3-5. Geologic Hazards.

Although, the hazard of strong ground shaking is generally the principal cause of damage to buildings and other structures during earthquakes, other seismic-geologic hazards have caused catastrophic damage to structures during earthquakes. These hazards include:

- *Surface fault rupture*, which is the direct, shearing displacement occurring along the surface trace of the fault that slips during an earthquake.

- *Soil liquefaction*, in which certain types of soil deposits below the groundwater table may lose a substantial amount of strength due to strong earthquake ground shaking, potentially resulting in reduced foundation-bearing capacity, lateral spreading, settlement, and other adverse effects.

- *Soil differential compaction*, which refers to the densification of soils and resulting settlements that may occur due to strong ground shaking.

- *Landsliding* of soil and rock masses on hillside slopes, due to earthquake-ground-shaking-induced inertia forces in the slope.

- *Flooding* induced by earthquakes, which includes the phenomena of tsunami, seiche, and dam, levee, and water tank failures.

The sites of all new buildings shall be evaluated to minimize the possibility that a structure which is adequately resistant to ground shaking could fail due to the presence of a severe site geologic hazard. Guidelines for conducting a geologic hazards study are described in Appendix F. As described in Appendix F, a screening procedure may be applied initially to ascertain whether the possibility of one or more geologic hazards can be screened out for a facility site. For those hazards that cannot be screened out, more detailed procedures should be used to evaluate whether a significant hazard exists, and if necessary, to develop hazard mitigation measures. Guidelines for more detailed evaluations of hazards and for hazard mitigation are also presented in Appendix F. Examples of geologic hazards studies are provided in Appendix G.

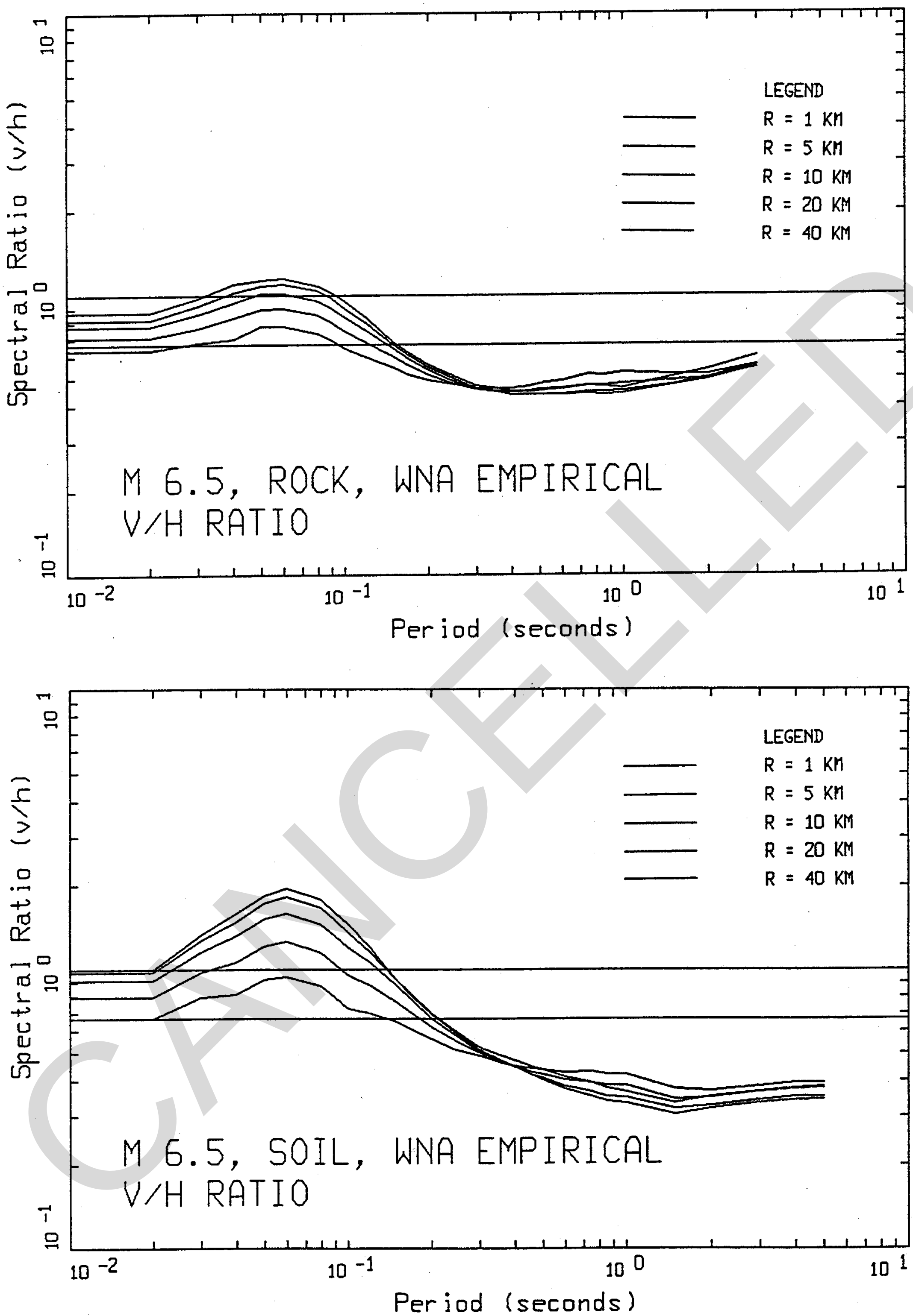


Figure 3-17 **Distance dependency of response spectral ratio (V/H) for M 6.5 at rock and soil sites in western North America. Line at 0.66 indicates the constant ratio of 2/3 (Silva, 1997).**

CHAPTER 4
APPLICATION OF CRITERIA

4-1. General.

a. Performance Objectives. Seismic performance objectives for a building are defined by a desired performance level for the building (e.g., damage state or ability to perform an essential function) when subjected to a specified seismic hazard (i.e., deterministic or probabilistic ground motion). A performance objective for each of the four Seismic Use Groups (Table 4-1) is prescribed in the following paragraphs. The performance objectives (Table 4-4) are derived from appropriate combinations of three performance levels (Table 4-3), and two design ground motions.

b. Basis of Provisions.

(1) Performance Objective 1A. All buildings governed by this document are required to comply with Performance Objective 1A, which is intended to protect life safety for Ground Motion A, defined as two-thirds of the Maximum Considered Earthquake (MCE). The Acceptance criteria for the performance objectives require compliance with the 1997 edition of the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA 302). The following are excerpted from Chapter 1 of those provisions:

"These provisions present criteria for the design and construction of structures to resist earthquake ground motions. The purposes of these provisions are as follows:

(a) To provide minimum design criteria for structures appropriate to their primary function and use considering the need to protect the health, safety, and welfare of the general public by minimizing the earthquake-related risk to life, and

(b) To improve the capability of essential facilities and structures containing substantial quantities of hazardous materials to function during and after design earthquakes. The design earthquake ground motion levels specified here could result in both structural and nonstructural damage. For most structures designed and constructed according to these Provisions, structural damage from the design earthquake would be repairable, although perhaps not economically so. For essential facilities, it is expected that the damage from a design earthquake would not be so severe as to preclude continued occupancy and function of the facility. The actual ability to accomplish these goals depends upon a number of factors, including the structural framing type, configuration, materials, and as-built details of construction. For ground motions larger than the design levels, the intent of these Provisions is that there be a low likelihood of structural collapse."

(2) Enhanced performance objectives. Performance levels and performance objectives prescribed in this document are generally based on guidance provided in SEAOC Vision 2000. The acceptance criteria for the enhanced performance objectives and the nonlinear analytical procedures are adapted from FEMA 273.

c. Analytical Procedures. Minimum requirements for the analytical procedures associated

Seismic Use Group	Occupancy or Function of Structure
I. Standard Occupancy Structures	All structures having occupancies or functions not listed below.
II. Special Occupancy Structures	Covered structures whose primary occupancy is public assembly with a capacity greater than 300 persons.
	Day care centers with a capacity greater than 150 persons.
	Educational buildings through the 12^{th} grade with a capacity greater than 250 persons.
	Buildings for colleges or adult education schools with a capacity greater than 500 students.
	Medical facilities with 50 or more resident incapacitated patients, but not otherwise designated as Seismic Use Group IIIE facility.
	Jails and detention facilities.
	All structures with occupancy capacity greater than 5,000 persons.
	Structures and equipment in power-generating stations and other public utility facilities not included in Seismic Use Group IIIE, and are required for continued operation.
	Water treatment facilities required for primary treatment and disinfecting of potable water.
	Waste water treatment facilities required for primary treatment.
	Facilities having high value equipment, when justification is provided by the using agency.

Table 4-1 Seismic Use Groups

III H. Hazardous Facilities	Structures housing, supporting or containing sufficient quantities of toxic or explosive substances to be dangerous to the safety of the general public if released.
III E. Essential Facilities[1]	Facilities involved in handling or processing sensitive munitions, nuclear weaponry or materials, gas and petroleum fuels, and chemical or biological contaminants.
	Facilities involved in operational missile control, launch, tracking or other critical defense capabilities.
	Mission-essential and primary communication or data handling facilities.
	Hospitals and other medical facilities having surgery and emergency treatment areas.
	Fire, rescue, and police stations.
	Designated emergency preparedness centers.
	Designated emergency operations centers.
	Designated emergency shelters.
	Power generating stations or other utilities required as emergency back-up facilities for Seismic Use Group IIIE facilities.
	Emergency vehicle garages and emergency aircraft hangars.
	Designated communications centers.
	Aviation control towers and air traffic control towers.
	Waste treatment facilities required to maintain water pressure for fire suppression.

[1] Essential facilities are those structures that are necessary for emergency operations subsequent to a natural disaster.

Table 4-1 Seismic Use Groups-Continued

with each performance objective are presented in Table 4-4, and each of the analytical procedures is described in Chapter 5. These procedures are considered to provide acceptable analytical results for most low-rise (i.e., six stories or less) regular buildings. Chapter 5 provides guidance regarding the limitations of these minimum analytical procedures, as well as when more rigorous analyses are required. It should be noted that most military buildings are classed as Seismic Use Group I (Standard Occupancy) that have Performance Objective 1A (Life Safety). For these buildings, the basic design approach used in this document prescribes a linear elastic (ELF or modal analysis) procedure that has not changed from previous criteria. The most basic change from previous design procedures is that the comparison of the demand of the design loads to the structural component capacity is performed at the strength level, rather than working or allowable stress.

d. Acceptance Criteria. The acceptance criteria for each of the performance objectives and the applicable analytical procedures are prescribed in Chapter 6. Numerical acceptance limits for specific structural systems are provided in Chapter 7.

4-2. Seismic Use Groups.

The following Seismic Use Groups are established based on the occupancy or function of a building:

a. Group IIIE. Seismic Use Group IIIE buildings are those containing essential facilities that are required for post-earthquake recovery, and/or those structures housing mission-essential functions. Mission-essential functions are those absolutely critical to mission continuation of the activity (there is no redundant back-up facility on- or off-site) as determined by the Commanding Officer at the activity and/or the Major Claimant.

b. Group IIIH. Seismic Use Groups IIIH buildings are those containing substantial quantities of hazardous substances that could be dangerous to the safety of the public, if released.

c. Group II. Seismic Use Group II buildings are those that constitute a substantial public hazard because of the occupancy or use of the building.

d. Group I. Seismic Use Group I buildings are those that are not assigned to Seismic Use Groups II or III.

e. Hazardous Critical Facilities. These facilities (e.g., nuclear power plants, dams, and LNG facilities) are not included within the scope of this document, but are covered by other publications or regulatory agencies. For any facilities housing hazardous items not covered by criteria in this document, guidance should be requested from DAEN-ECE-D (Army), NAVFAC Code 04BA (Navy), or HQ USAF/LEEE (Air Force).

Examples of buildings or structures in each of the above groups are provided in Table 4-1. Buildings with multiple occupancies will be categorized according to the most important occupancy, unless the portion of the building that houses the most important occupancy can be shown to satisfy all of the requirements for that occupancy.

4-3. Seismic Design Categories.

All buildings shall be assigned a Seismic Design Category based on their assigned Seismic Use Group and their applicable spectral acceleration coefficients, S_{DS} and S_{DI} for Ground Motion A. Each building or structure shall be assigned to the more severe Seismic Design Category in accordance with Table 4-2a or Table 4-2b. The category designations are used in FEMA 302 to determine permissible structural systems, limitations on height and irregularity, and coefficients related to overstrength and drift.

4-4. Redundancy.

FEMA 302 prescribes a reliability factor, *D*, to be assigned to all buildings based on the extent of structural redundancy inherent in the lateral-force-resisting system. The value of *D* may be taken as 1.0 for buildings in Seismic Design Categories A, B, C. For buildings in Seismic Design Categories D, E, and F, *D* shall be taken as the largest of the values of $?_x$ calculated at each story of the building, *x*, in accordance with the following equation:

$$?_x = 2 - \frac{20}{r_{\max_x}\sqrt{A_x}} \qquad (4\text{-}1)$$

where:

$r_{\max_x}$ = the ratio of the design story shear resisted by the single element carrying the most shear force in the story to the total story shear, for a given direction of loading. For braced frames, the value of $r_{\max_x}$ is equal to the lateral-force component in the most heavily loaded brace element divided by the story shear. For moment frames, $r_{\max_x}$ shall be taken as the maximum of the sum of the shears in any two adjacent columns, in the plane of a moment frame, divided by the story shear. For columns common to two bays with moment-resisting connections on opposite sides at the level under consideration, 70 percent of the shear in that column may be used in the column shear summation. For shear walls, $r_{\max_x}$ shall be taken as equal to the shear in the most heavily loaded wall or wall pier multiplied by $10/l_w$ (the metric equivalent is 3.3 (l_w), where l_w is the wall or wall pier length in feet (m), divided by the story shear. For dual systems, $r_{\max_x}$ shall be taken as the maximum value as defined above considering all lateral-load-resisting elements in the story. The lateral loads shall be distributed to elements based on relative rigidities considering the interaction of the dual system. For dual systems, the value of *D* need not exceed 80 percent of the value calculated above.

A_x = the floor area in square feet of the diaphragm level immediately above the story.

The metric equivalent of Equation 4-1 is:

$$r_x = 2 - \frac{6.1}{r_{\max_x} A_x}$$

where A_x is in square meters.

The value of *D* need not exceed 1.5, which is permitted to be used for any structure. The value of *D* shall not be taken as less than 1.0.

Exception: For structures with lateral-force-resisting systems in any direction comprised solely of special moment frames, the lateral-force-resisting system shall be configured such that the value of *D* calculated in accordance with this section does not exceed 1.25.

Table 4-2a Seismic Design Category Based on Short Period Response Accelerations

Value of S_{DS}	Seismic Use Group		
	I	II	III
$S_{DS} < 0.167g$	A	A	A
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D[a]	D[a]	D[a]

[a] See footnote on Table 4-2b.

Table 4-2b Seismic Design Category Based on 1 Second Period Response Accelerations

Value of S_{DI}	Seismic Use Group		
	I	II	III
$S_{DI} < 0.067g$	A	A	A
$0.067g \leq S_{DI} < 0.133g$	B	B	C
$0.133g \leq S_{DI} < 0.20g$	C	C	D
$0.20g \leq S_{DI}$	D[a]	D[a]	D[a]

[a] Seismic Use Group I and II structures located on sites with mapped maximum considered earthquake spectral response acceleration at 1 second period, S_1, equal to or greater than 0.75g shall be assigned to Seismic Design Category E and Seismic Use Group III structures located on such sites shall be assigned to Seismic Design Category F.

4.5. Overstrength.

A system overstrength factor, S_o, provided in Table 7-1 is intended to quantify the actual force that can be delivered to sensitive individual brittle elements, the failure of which could result in the loss of a complete lateral-force-resisting system or in instability or collapse. The factor is similar in application to the $3R_w/8$ factor prescribed in the UBC, and represents an estimate of the combined design, material, and system overstrengths that could effect a brittle or force-controlled element.

4-6. Combination of Load Effects.

a. Basic Load Combinations. The basic load combinations from ASCE 7 are:

$$1.2D + 1.0E + 0.5L + 0.2S \qquad (4\text{-}2)$$

or $$0.9D + 1.0E \qquad (4\text{-}3)$$

where: D,E,L, and S, are respectively, dead, earthquake, live, and snow loads.

The effect of the earthquake load, E, is defined by:

$$E = \mathsf{D}Q_E + 0.2S_{DS}D \qquad (4\text{-}4)$$

or $$E = \mathsf{D}Q_E - 0.2S_{DS}D \qquad (4\text{-}5)$$

Where:

E = the effect of horizontal and vertical earthquake-induced forces.

D = the reliability factor defined in Paragraph 4-4

Q_E = the effect of horizontal seismic forces

S_{DS} = the design spectral acceleration at 0.2 sec

D = the effect of dead load.

b. Special Combination of Loads. When specifically required by FEMA 302, or when in the judgement of the designer the effects of structural overstrength need to be considered, the design seismic force on brittle or force-controlled components shall be defined by the following equations:

$$E = \mathsf{S}_o Q_E + 0.2S_{DS}\,D \qquad (4\text{-}6)$$

$$E = \mathsf{S}_o Q_E - 0.2S_{DS}\,D \qquad (4\text{-}7)$$

Where E, Q_E, S_{DS} and D are as defined in Paragraph *a* above, and S_o is the system overstrength factor defined in Paragraph 4-5 and listed in Table 7-1. Example applications for Equations 4-6 and 4-7 include the design of columns under discontinuous shear walls or braced frames, and the design of frame members in braced frames effected by overstrength in the bracing.

4-7. Performance Levels.

Three structural performance levels, as described in Table 4-3, are considered to be acceptable by this document. Performance Level 1 (Life Safety) is the minimum performance level required of all Seismic

Structural Performance Levels

Performance Level	Building Response Range
CP	Collapse Prevention-The building barely remains standing with significant structural and nonstructural damage. This range of performance, where collapse is imminent, is an unacceptable performance range for all new military buildings.
LS (1)	Life Safety-The building remains stable with significant reserve capacity. Structural damage is moderate requiring significant post-earthquake repairs, however, collapse is precluded. This is the basic range of performance for all new military buildings, except as defined below.
SE (2)	Safe Egress-The building structural system remains fully safe for occupancy following the earthquake. Essential functions are sufficiently disrupted to prevent immediate post-earthquake occupancy of the building. Structural damage is light, allowing fairly rapid post-earthquake repairs.
IO (3)	Immediate Occupancy-The building structure remains safe to occupy and all essential functions remain operational. It may be used for post-earthquake recovery and to perform essential operational military missions within a few hours following an earthquake. The building has limited structural damage, which may be repairable while occupied.

Table 4-3. Structural Performance Levels

Use Groups. Performance Levels 2 (Safe Egress) and 3 (Immediate Occupancy) are enhanced performance levels for Seismic Use Groups II and III. The Collapse Prevention performance level indicated in Table 4-3 is an assumed ultimate response level for structural components and is not an applicable design performance level. The physical significance of these performance levels is indicated in Figures 1-1 and 1-2.

4-8. Design Ground Motions.

As indicated in Table 4-4, two ground motions, derived from the MCE, are considered in this document, and their derivation is discussed in Chapter 3. Ground Motion A is the basic ground motion in the FEMA 302 provisions; it is approximately equivalent to 10 percent probability of exceedence in 50 years, and is the ground motion associated with Performance Level 1 (Life Safety) for all seismic use groups. Ground Motion A is also used with Performance Level 2 to provide Performance Objective 2A (Safe Egress) for Seismic Use Group II, while Ground Motion B is used to provide Performance Objective 2B (Safe Egress) for Seismic Use Group IIIH and Performance Objective 3B (Immediate Occupancy) for Seismic Use Group IIIE. Note that while "Safe Egress" is used to describe the performance level in both Performance Objectives 2A and 2B, different ground motions and acceptance criteria are used to comply with the two performance objectives.

4-9. Performance Objectives.

The seismic performance objectives for the various seismic use groups in Table 4-1 consist of the combination of performance levels from Table 4-3 with an appropriate ground motion. As indicated above, Performance Objective 1A (Life Safety) is required of all seismic use groups, and the other objectives in Table 4-4 pertain to buildings in Seismic Use Group II or III.

4-10. Minimum Requirements for Analytical Procedures.

Minimum analytical procedures for the various performance objectives associated with each of the seismic use groups are indicated in Table 4-4. The analysis procedures are described in Chapter 5, the acceptance criteria for each of the procedures are prescribed in Chapter 6, and the numerical acceptance limits for the various structural systems are provided in Chapter 7. Note that for Performance Objective 1A, FEMA 302 provisions are prescribed with appropriate R factor and an Importance Factor, I, equal to 1.0. For enhanced performance objectives, more severe ground motion and/or more restrictive acceptance criteria are prescribed in lieu of the R or I factors.

4-11. General Design Procedures.

a. Performance Objectives 1A (All Buildings). Table 4-5 provides a step-by-step tabulation of procedures for the analysis and design of buildings, in accordance with FEMA 302, to comply with Performance Objective 1A (Protect Life Safety). Reference is made to applicable sections in FEMA 302 and to corresponding paragraphs in this document. Additional guidance pertaining to the total design process is provided in Chapter 2. The analyses and design procedures for this performance

Structural System Performance Objectives

Performance Parameters				Minimum Analysis Procedures	
Seismic Use Group	Performance Level	Ground Motion	Performance Objective	Linear Elastic with R Factors	Linear Elastic with m Factors
I	LS(1)	2/3 MCE (A)[1]	1A	√	
II	SE(2)	2/3 MCE (A)[1]	2A	√[2]	√[3]
IIIH	SE(2)	¾ MCE (B)[1]	2B	√[2]	√[3]
IIIE	IO(3)	¾ MCE (B)[1]	3B	√[2]	√[3]

[1] MCE refers to the spectral ordinate values from the Maximum Considered Earthquake spectral acceleration maps that accompany FEMA 302.

[2] All buildings will be initially designed to comply with Performance Objective 1A using a linear elastic analysis with R factors. This will establish the member sizes required to check compliance with the m factor criteria for the enhanced performance objectives.

[3] For certain buildings in Seismic Design Categories C, D, E, & F, the minimum analysis method to be used may be a nonlinear static (or pushover) procedure. See paragraph 5-4b to determine when nonlinear analysis procedures are required.

[4] For buildings in Seismic Design Categories A and B, the analysis for enhanced performance objectives may be performed with the applicable ground motion and a linear elastic analysis with the R and I factors from Table 7-1 and the I factors from FEMA 302.

Table 4-4. Structural System Performance Objectives

Step	Procedure	References FEMA 302	TI-809-04
A. Preliminary Determinations (See Flow Chart in Figure 4-1)			
1.	Determine appropriate Seismic Use Group, Performance Objectives & Analysis Procedures	Sec. 1.3	Table 4-1, Table 4-4 para. 4-2
2.	Determine site seismicity, (S_S&S_I)	MCE Maps	para. 3-1c
3.	Determine site characteristics	Sec. 4.1.2.1	Table 3-1 para. 3-4i
4.	Determine site coefficient: F_a F_v	Table 4.1.2.4.a Table 4.1.2.4.b	Table 3-2a Table 3-2b para. 3-1d
5.	Calculate $S_{MS} = F_aS_s$ $S_{MI} = F_VS_I$	Eq. 4.1.2.4-1 Eq. 4.1.2.4.-2	Eq. 3-1 Eq. 3-2
6.	Calculate $S_{DS} = 2/3\ S_{MS}$ $S_{DI} = 2/3\ S_{MI}$	Eq. 4.1.2.5-1 Eq. 4.1.2.4-2	Eq. 3-3 – Eq. 3-6 para. 3-2b
7.	Select Seismic Design Category	Table 4.2.1a Table 4.2.1b	Table 4-2a Table 4-2b para. 4-3
8.	Select structural system		para. 2-5b – para. 2-5e
9.	Select R, Ω_o, & C_d factors	Table 5.2.2	Table 7-1 para. 4-5 para. 6-2c
10.	Determine preliminary member sizes for gravity load effects	ASCE 7	ASCE 7 para. 2-5a
B. Equivalent Lateral Force Procedure (See Flow Chart in Figure 4-1)			
1.	Calculate fundamental period, T	Eq. 5.3.3.1-1 or Eq. 5.3.3.1-2	
2.	Determine dead load, W	Sec. 5.3.2	
3.	Calculate base shear, V	Eq. 5.3.2	para. 3-2c
4.	Calculate vertical distribution of seismic forces	Sec. 5.3.4.	
5.	Perform static analyses		para. 5-2
6.	Determine cr and cm		Figure 7-47
7.	Perform torsional analyses	Secs. 5.3.5.1 to 5..3.5.3	para. 7-7b(4)
8.	Determine need for redundancy factor, ρ.	Sec. 5.2.4.2	para. 2-5c, para. 4-4
9.	Determine need for everstrength factor Ω_o	Sec. 5.2.7.1	para. 4-5
10.	Calculate combined load effects	ASCE 7 and Sec. 5.27	para. 4-6

Table 4-5 Step-by-Step Procedures for Performance Objective 1A (Life Safety)

Sheet 1 of 2

Step	Procedure	FEMA 302	TI-809-04
11.	Determine structural member sizes	Chapters 8 through 12	Chapter 7 para. 2-5f
12.	Check allowable drift and PΔ effect	Sec. 5.2.8 & 5.3.7 and Table 5.2.8	para. 6-2d and Table 6-1
1.	C. Modal Analyses Procedure (See Flow Chart in Figure 4-1) Sec. 5.4 in FEMA 302 provides general guidance for modeling and performance of modal analyses. Available computer programs calculate the equivalent of steps B1 through B7 above with proper input of structural member properties, distributed dead loads, and appropriate response spectra. (The general response spectrum defined by Figure 3-1 is applicable to the fundamental mode while the spectrum shown in Figure 3-2 applies to higher modes). Most computer programs will combine the modal responses for individual structural members by SRSS, CQC, or absolute sum, at the designer's option. Steps B8 through B12 above also apply to this procedure.		

Table 4-5 Step-by-Step Procedures for Performance Objective 1A (Life Safety)

Sheet 2 of 2

objective are illustrated by the flow chart in Figure 4-1.

b. Enhanced Performance Objectives.

(1) Performance Objective 2A (Safe Egress for Special Occupancy). This performance objective, for Seismic Use Group II, uses the same ground motion as in Performance Objective 1A in Paragraph 4-11a above. Step-by-step procedures for analysis and design are provided in Table 4-6. Note that if proper consideration is given in the selection of the analytical procedure for Performance Objective 1A, the seismic effects may be scaled for compliance with the prescribed acceptance criteria in Chapter 6. The design and analysis procedures for this performance objective are illustrated by the flow chart in Figure 4-2.

(2) Performance Objective 2B (Safe Egress for Hazardous Occupancy and safe post-earthquake protection of hazardous materials stored in these buildings). This performance objective, for Seismic Use Group IIIH, uses the same performance level as Performance Objective 2A, but requires this level of performance for Ground Motion B (3/4 MCE). The seismic response effects for this ground motion can also be scaled from the initial analysis for Performance Objective 1A. The step-by-step procedures are similar to those in Table 4-6, and a flow chart illustrates these procedures in Figure 4-3.

(3) Performance Objective 3B (Immediate Occupancy for Essential Facilities). This performance objective for Seismic Use Group IIIE is the most demanding objective applicable to buildings governed by this document. The step-by-step design and analysis procedures described in Table 4-7 and the flow chart in Figure 4-4 illustrate the implementation of nonlinear elastic static analysis for this objective.

4-12. Performance Objectives for Nonstructural Systems and Components.

The minimum performance objective for nonstructural systems and components will be similar to structural Performance Objective 1A, as described in the preceding paragraph of this chapter. Compliance with the provisions of Chapter 6 of FEMA 302 will be considered as fulfilling this minimum performance objective. Provisions for enhanced performance objective and additional requirements for nonstructural systems and components are provided in Chapter 10 of this document.

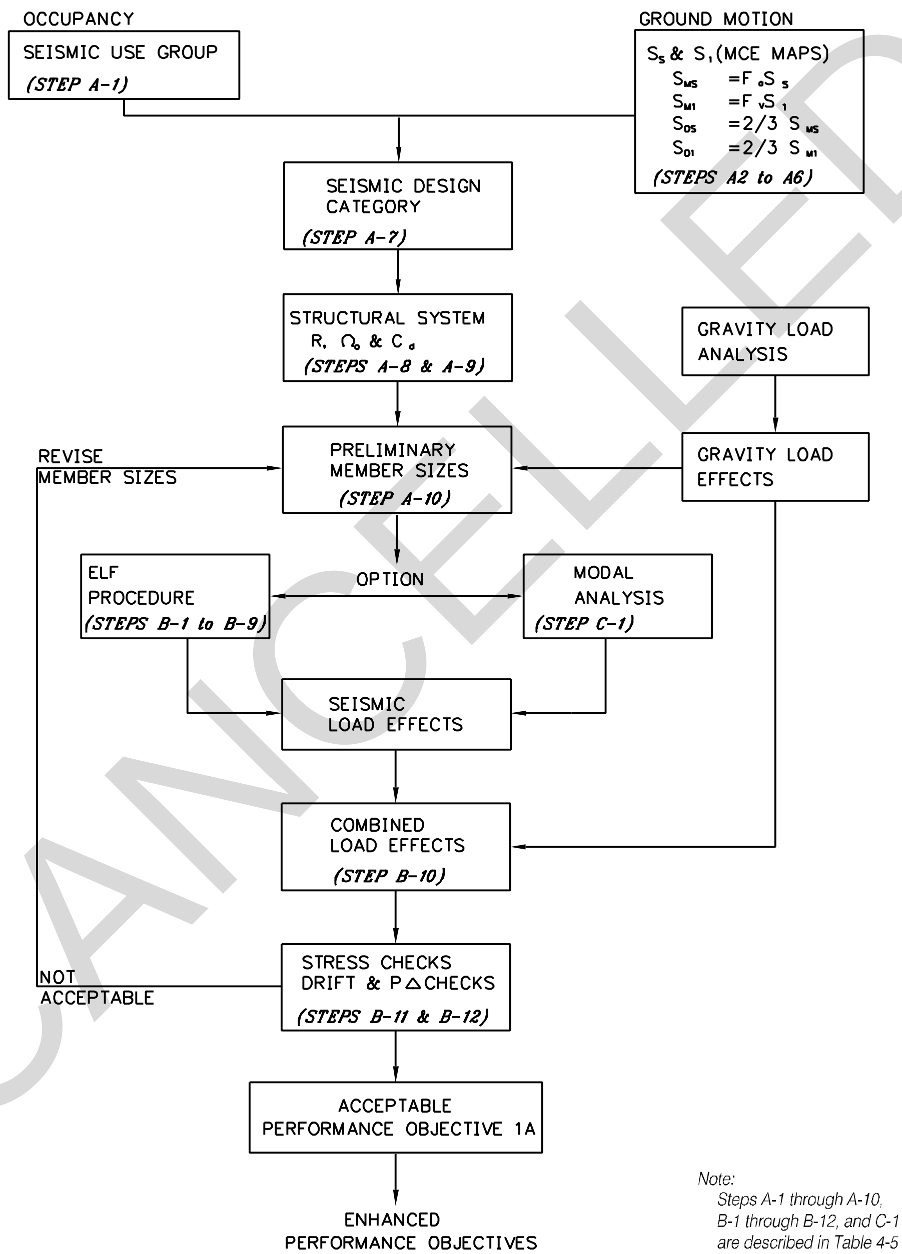
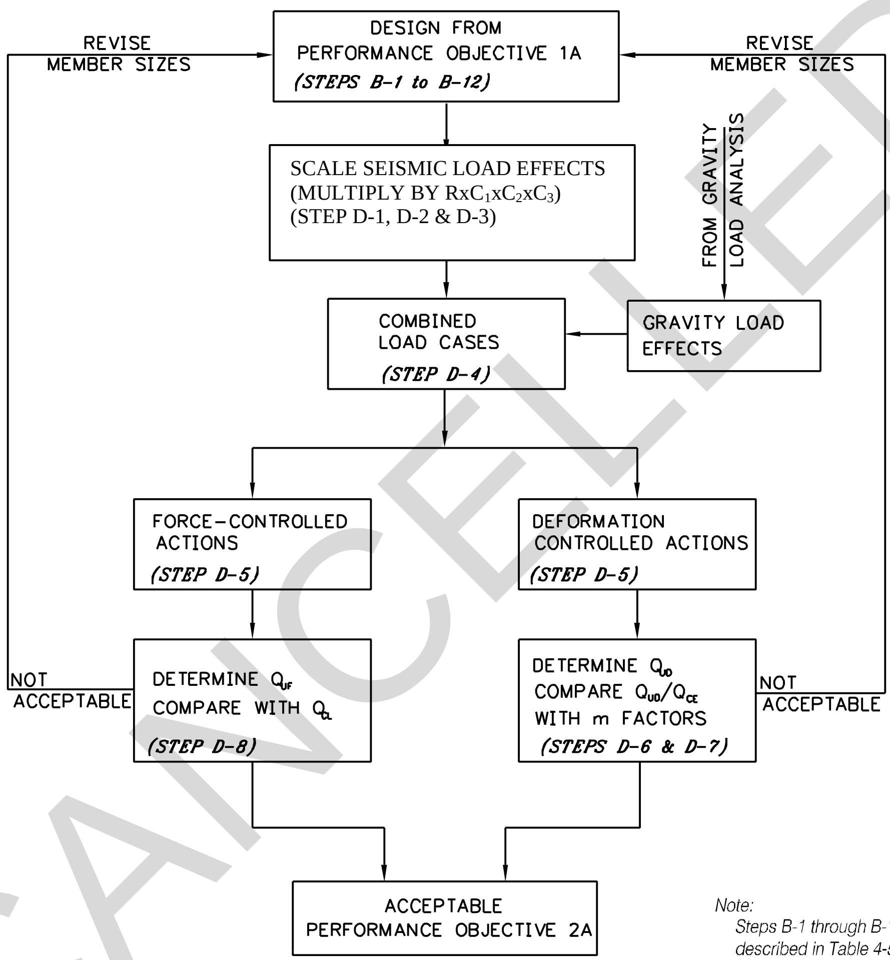


Figure 4-1. Flow Chart for Performance Objective 1A (All Buildings)

Step	Procedure	TI-809-04
1.	D. <u>Performance Objective 2A (Safe Egress for Special Occupancy)</u> (See Flow Chart in Figure 4-2) This performance objective utilizes the same basic ground motion as the Life Safety Performance Objective (1A). For one- or two-story buildings that have been analyzed by the ELF procedure or for buildings that have been subjected to a 3-D modal analysis procedure, the seismic effects, Q_E, in step B10 of Table 4-5 may be scaled to the appropriate values as indicated in Step 3 below.	
2.	Determine pseudo lateral load $V=C_1 C_2 C_3 S_a W$.	para. 6-3a(2) Eq. 6-1
3.	Determine seismic effects (Seismic effects in Steps B-4 through B-9 in Table 4-5 may be scaled by the factor $R \times C_1 \times C_2 \times C_3$.)	Steps B-4 through B-9 in Table 4-5
4.	Determine the combined load effects.	para. 4-6 and Eq. 6-2
5.	Identify force-controlled and deformation-controlled structural components.	para. 6-3a(3)(a)
6.	Determine Q_{UD} and Q_{CE} for deformation-controlled components.	Chapter 7 and para. 6-3a(3)(b)
7.	Determine DCR's for deformation-controlled components and compare with allowable m values for Safe Egress.	Chapter 7 and Eq. 6-3
8.	Determine Q_{UF} and Q_{CL} for force-controlled components and compare Q_{UF} with Q_{CL}.	para. 6-3a(3)(b) Eq. 6-4a and 6-4b and Chapter 7
9.	Revise member sizes, as necessary, and repeat analysis.	
1.	E. <u>Performance Objective 2B (Safe Egress for Hazardous Occupancy)</u> (See Flow Chart in Figure 4-3) Same as Performance Objective 2A above, except that Ground Motion B (3/4 MCE) is to be used . As indicated above, if a suitable analyses has been performed for Performance Objective 1A, the seismic effects may be scaled by $R \times C_1 \times C_2 \times C_3 \times 0.75/0.67$. The m values for Safe Egress are applicable.	
Step	Procedure	TI-809-04
	F. <u>Performance Objective 3B (Immediate Occupancy for an Essential Facility)</u> Same as Performance Objective 2B, except that the m values for Immediate Occupancy are applicable.	

Table 4-6 Step-by-Step Procedures for Enhanced Performance Objectives with Linear Elastic Analyses Using m Factors



Note:
Steps B-1 through B-12 are described in Table 4-5.
Steps D-1 through D-9 are described in Table 4-6

Figure 4-2. Flow Chart for Performance Objective 2A (Seismic Use Group II Buildings)

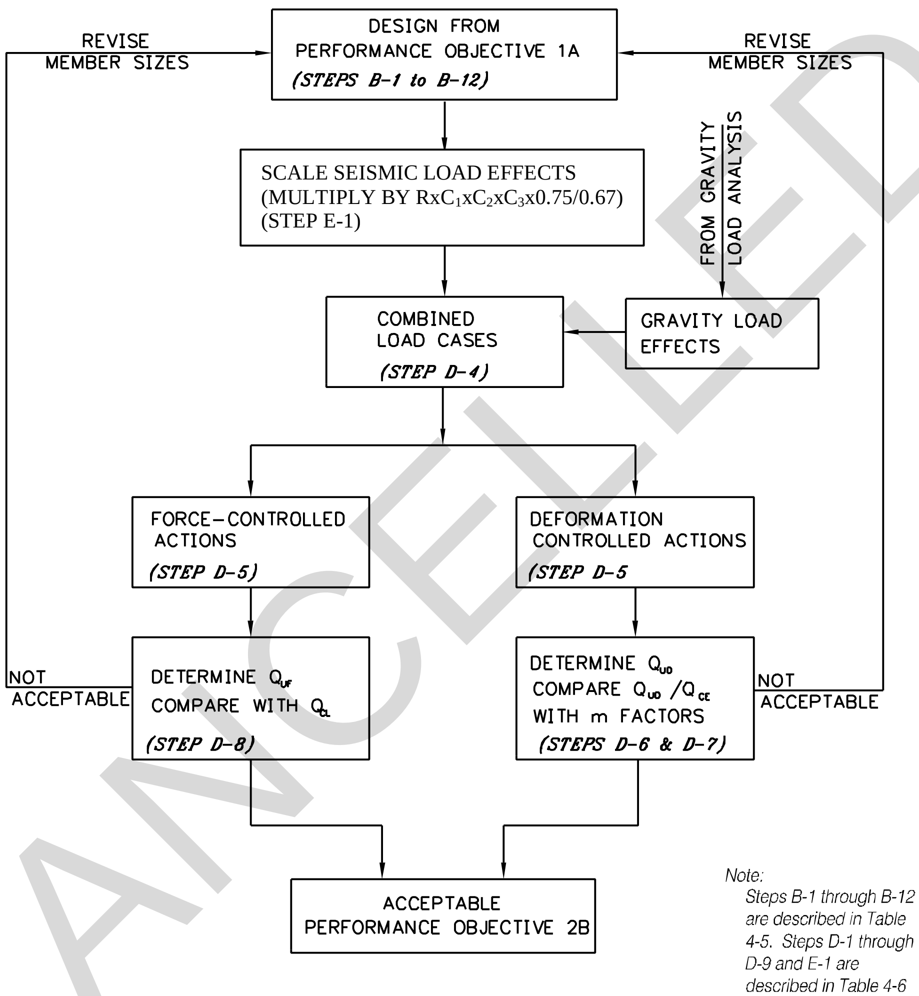


Figure 4-3. Flow Chart for Performance Objective 2B
(Seismic Use Group III H Buildings)

Step	Procedure	TI-809-04
	G. Performance Objective 3B (Immediate Occupancy for Essential Facility) (See Flow Chart in Figure 4-4)	
1.	Using the mathematical model of the building developed for Performance Objective 1A, perform a 3D spectral response analysis using Ground Motion B (3/4 x MCE).	para. 5-4e(3) para. 5-4e (5) and (6)
2.	Using only fundamental mode responses, calculate combined load effects, displacement of center of mass at roof level, and base shear.	para. 5-4f(1)
3.	Calculate DCR's for the structural elements and identify element with highest DCR and any other elements within 10% of that value.	para. 5-4f(1)
4.	Determine necessary reduction in seismic effects to reduce highest DCR to 1.0. Reduce the seismic effects ,base shear, and roof displacement by the seismic effects reduction. Plot base shear vs. roof displacement.	para. 5-4f(1) Fig. 5-3
5.	Modify elements identified in Step 4 by inserting plastic hinges at the yielding ends in the mathematical model.	para. 5-4f(1)(c)
6.	Repeat the analyses, add the new seismic effects to the reduced seismic effects from the prior analyses, and repeat Steps 2 and 3.	para. 5-4f(1)(d)
7.	Determine necessary reduction in the new seismic effects to reduce highest DCR to 1.0. Reduce the new seismic effects, base shear, and roof displacement by the reduction in the new seismic effects. Plot the new increment of base shear and roof displacement by superposition on prior plot.	Fig. 4-5
8.	Repeat Steps 5,6,and 7 until plot results in undesirable response or failure of critical elements.	para. 5-4f(1)(f)
9.	Idealize the plot to determine V_y, $0.6V_y$, and K_e.	Fig. 5-3
10.	Obtain the elastic fundamental period T_i, from step 2 and calculate the effective fundamental period, T_e, by $T_e = T_i\sqrt{K_i/K_e}$	para. 5-4e(4)
11.	Determine appropriate values for coefficients C_0, C_1, C_2, and C_3 and for spectral acceleration S_a.	para. 5-4f(2)
12.	Calculate target displacement, δ_t, by $\delta_t = C_0\, C_1\, C_2\, C_3\, S_a\, \dfrac{T_e^{\,2}}{4\Pi^2}$	Eq. 5-5

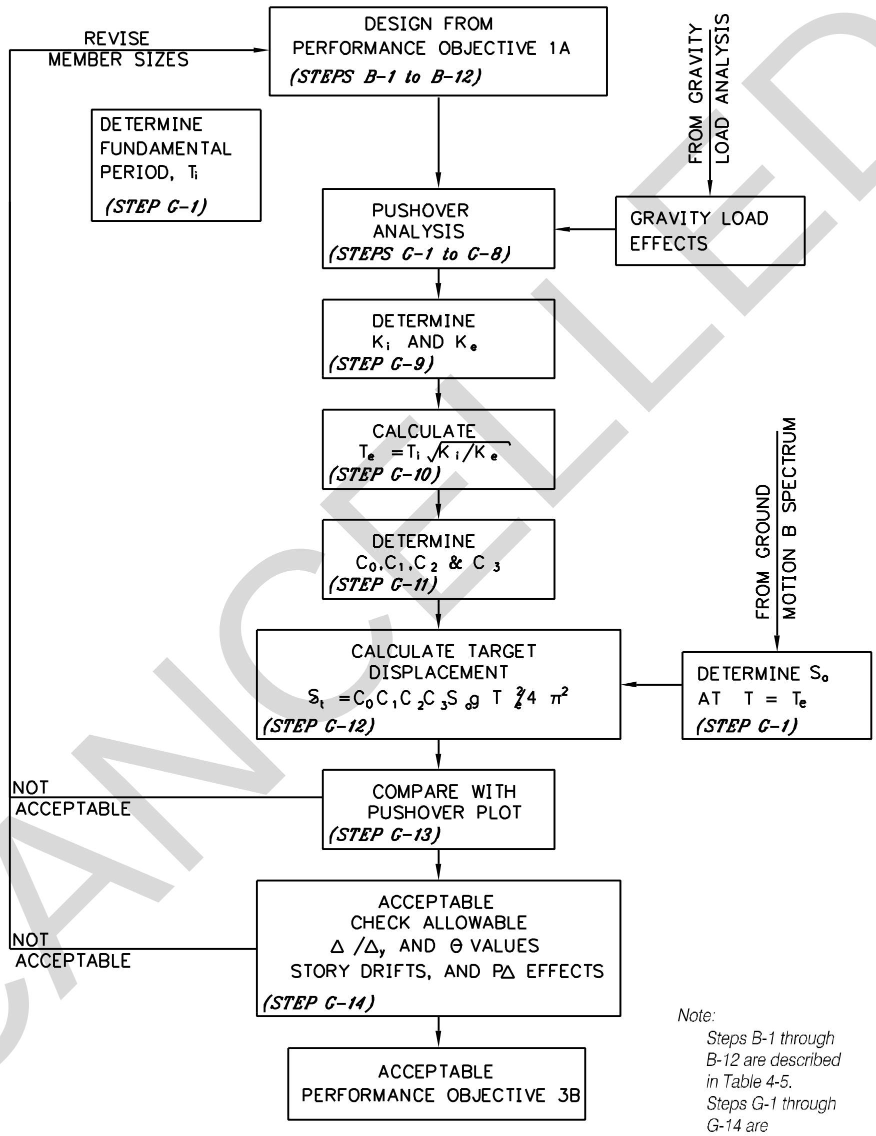
Table 4-7 Step-by-Step Procedure for Enhanced Performance Objective with Nonlinear Elastic Static Analysis

Sheet 1 of 2

Step	Procedure	TI-809-04
13.	Locate δ_t on Base Shear vs. Roof Displacement plot. a. If δ_t is beyond effective portion of the plot, significant strengthening of the yielding elements may be required. b. If δ_t is within the effective portion of the plot, check interstory drift and PΔ effects. If allowable limits have been exceeded, stiffening of yielding elements may be required. c. If interstory drift and PΔ effects are acceptable, check inelastic responses of yielding elements against acceptance criteria.	para. 5-4f(1)(i)
14.	If evaluations in Step 13 are negative, mathematical model must be strengthened and/or stiffened and the analyses repeated. Available computer programs can perform Steps 1 to 8 above, including PΔ effects. For a given building displacement, maximum inelastic responses (Δ/Δ_y or chord rotations) are developed which can be checked against the acceptance criteria. Chapter 7 provides acceptance criteria for the nonlinear response (Δ/Δ_y or chord rotations) for the structural components of various structural systems. Acceptance values are provided for Life Safety, Safe Egress, and Immediate Occupancy.	para. 6-3b Chapters 5 and 7

Table 4-7 Step-by-Step Procedure for Enhanced Performance Objective with Nonlinear Elastic Static Analysis

Sheet 2 of 2



Note:
Steps B-1 through B-12 are described in Table 4-5.
Steps G-1 through G-14 are described in Table 4-7

Figure 4-4. Flow Chart for Performance Objective 3B (Seismic Use Group III E Buidlings)

CHAPTER 5
ANALYSIS PROCEDURES

5-1. General.

a. Introduction. This chapter defines four basic analytical procedures; however, only the first three procedures are prescribed by this document. The first two procedures are linear elastic, and the latter two procedures are nonlinear. Limitations on the use of linear elastic static procedures are indicated in Paragraph 5-2b, and conditions when nonlinear procedures are required are provided in Paragraph 5-4b. The procedures are discussed in the following paragraphs in order of increasing rigor and complexity. Advantages, disadvantages, and limitations are indicated for each procedure. Paragraph 4-11 and Table 4-4 prescribe the minimum analytical procedure for each performance objective for the various seismic use groups. The prescribed minimum procedure is intended to apply to structures of average complexity for each performance objective. Unusual, or more complex, structures may require more complex or rigorous analytical procedures than the prescribed minimum.

b. Mathematical Modeling.

(1) Basic assumptions. In general, a building should be modeled, analyzed, and evaluated as a three-dimensional assembly of elements and components. Three-dimensional mathematical models shall be used for analysis and evaluation of buildings with plan irregularity. Two-dimensional modeling, analysis, and evaluation of buildings with stiff or rigid diaphragms is acceptable if torsional effects are either sufficiently small to be ignored or indirectly captured. Vertical lines of seismic framing in buildings with flexible diaphragms may be individually modeled, analyzed, and evaluated as two-dimensional assemblies of components and elements, or a three-dimensional model may be used with the diaphragms modeled as flexible elements. Connection modeling is not required for linear analysis. Explicit modeling of a connection is required for nonlinear analysis if the connection is weaker than the connected components, and/or the flexibility of the connection results in a significant increase in the relative deformation between the connected components.

(2) Horizontal torsion. The effects of horizontal torsion must be considered for buildings with diaphragms capable of resisting such torsion. The total torsional moment at a given floor level shall be set equal to the sum of the following two torsional moments:

- The actual torsion; that is, the moment, M_t, resulting from the eccentricity between the centers of mass at all floors above and including the given floor, and the center of rigidity of the vertical seismic elements in the story below the given floor, and

- The accidental torsion; that is, an accidental torsional moment, M_{ta}, produced by horizontal offset in the center of mass, at all floors above and including the given floor, equal to a minimum of 5 percent of the horizontal dimension at the given floor level measured perpendicular to the direction of the applied load.

For buildings in Seismic Design Categories C, D, E, and F, where torsional irregularity exists as defined in Table 5.2.3.2 of FEMA 302, the effects of the irregularity shall be accounted for by multiplying the sum of M_t and M_{ta} at each level by a torsional amplification factor, A_x, determined from:

$$A_x = \left\{ \frac{d_{\mathrm{max}}}{1.2\, d_{\mathrm{avg}}} \right\}^2 \qquad (5\text{-}1)$$

where:

d_{max} = maximum displacement at Level x

d_{avg} = average of the displacements at the extreme points of the building at Level x.

The torsional amplification factor, A_x is not required to exceed 3.0.

5-2. Linear Elastic Static Procedure.

a. General. This procedure, also known as the "Equivalent Lateral Force (ELF) Procedure," will be the procedure most widely employed for one-story buildings, and can be utilized for all regular buildings of two to six stories, and is the preferred procedure for structures of wood frame or light metal frame construction. The required calculations are relatively simple and can be performed by hand, although a number of computer programs are available to facilitate the analysis. The results of the linear static analysis procedure can be very inaccurate when applied to buildings with highly irregular structural systems, unless the building is capable of responding to the design earthquake(s) in a nearly elastic manner. Therefore, linear static analysis procedures should not be used for highly irregular buildings, except wood frame structures.

b. Limitations on Use of the Procedure. The linear elastic static procedure may be used unless one or more of the following conditions apply, in which case the linear elastic dynamic procedure, described in Paragraph 5-3, shall be used:

- The building height exceeds 100 feet.

- The ratio of the building's horizontal dimensions at any story to the corresponding dimensions at an adjacent story exceeds 1.4 (excluding penthouse).

- The building is found to have a severe torsional stiffness irregularity in any story. A severe torsional stiffness irregularity may be deemed to exist in a story if the diaphragm above the story is not flexible, and the results of the analysis indicate that the drift along any side of the structure is more than 150 percent of the average story drift.

- The building is found to have a severe vertical mass or stiffness irregularity. A severe vertical mass or stiffness irregularity may be deemed to exist when the average drift in any story (except penthouses) exceeds that of the story above or below by more than 150 percent.

- The building has a non-orthogonal lateral-force-resisting system.

c. Implementation of the Procedure shall be in accordance with the provisions of Section 5.3 of FEMA 302, with exceptions or modifications as noted in the following paragraphs.

(1) Performance Objective 1A. In accordance with Section 5.3 of FEMA 302, except that $I = 1.0$. This is the prescribed analysis in FEMA 302 for standard occupancy structures, and is prescribed in this document as a preliminary analysis for all seismic use groups to satisfy the Life Safety Performance Objective.

(2) Performance Objective 2A, 2B, and 3B. In accordance with Section 5.3 of FEMA 302, except that $R = 1.0$, $I = 1.0$, and the base shear is modified to represent the pseudo-lateral load described in paragraph 6-3a(2). The m modification factors used in these analyses are defined in Paragraph 6-3a.

Exception: Buildings with enhanced performance objectives in Seismic Design Categories A and B may be analyzed by the ELF procedure, or the modal analysis procedure described in the following paragraph, with the appropriate ground motions, the R factors from Table7-1, and the applicable I factor from FEMA 302.

5-3. Linear Elastic Dynamic Procedure.

This procedure, also known as the "Modal Analysis Procedure," shall be performed in accordance with the requirements of Section 5.4 of FEMA 302, with the exceptions noted in Paragraph 5-2c for use with the various performance objectives. For most moment frame systems, the contribution of panel zone deformations to overall story drift may be assumed to be adequately represented by the use of centerline-to-centerline dimensions in the mathematical model. This analytical procedure is considered acceptable for all structures and all performance objectives designed in accordance with this document, except for the structures incorporating the use of a supplemental energy dissipation system and some types of base isolation systems. For specific analysis procedures applicable to those structures, refer to Chapter 8. The ELF procedure described in Paragraph 5-2 may be more appropriate for some regular or rigid (one or two-story) structures. Unusual or complex structures in Seismic Use Groups II and III, with the characteristics described in Paragraph 5-4b, may require a nonlinear elastic static procedure for confirmation of the enhanced Performance Objectives 2A, 2B, and 3B.

5-4. Nonlinear Static Procedure.

a. General. Nonlinear procedures directly account for the redistribution of forces and deformations that occur in a structure as it undergoes inelastic response. Consequently, they are generally capable of providing a more accurate estimate of the demands produced in a structure than either of the linear procedures. However, the nonlinear static procedure is not able to predict accurately the higher mode response of flexible structures and a nonlinear dynamic procedure should be considered for tall buildings (i.e. in excess of six stories) or buildings with significant vertical irregularities.

b. When Nonlinear Procedures are Required. In order to determine whether a building may be analyzed with sufficient accuracy by linear procedures, it is necessary to perform a linear analysis and then examine the results to determine the magnitude and distribution of inelastic demands on the various components of the primary lateral-force-resisting elements. The magnitude and distribution of inelastic demands are indicated by demand-capacity ratios (DCRs). DCRs for existing and new building components shall be computed in accordance with the equation:

$$\text{DCR} = \frac{Q_{UD}}{Q_{CE}} \qquad (5\text{-}2)$$

where:

Q_{UD} = the combined effect of gravity loads and earthquake loads

Q_{CE} = the expected strength of the component or element at the deformation level under consideration for deformation-controlled actions.

DCRs should be calculated for each controlling action (such as axial force, moment, and shear) of each component. If all of the computed controlling DCRs for a component is less than or equal to 1.0, then the component is expected to respond elastically to the earthquake ground shaking being evaluated. If one or more of the computed DCRs for a component is greater than 1.0, then the component is expected to respond inelastically to the earthquake ground shaking. The largest DCR calculated for a given component defines the critical action for the component, i.e., the mode in which the deformation-controlled component will first yield, or fail in the case of a brittle force-controlled component. This DCR is termed the critical component DCR. If an element is composed of multiple components, then the components with the largest computed DCR is the critical component for the element, i.e., this will be the first component in the element to yield, or fail. The largest DCR for any component in an element at a particular story is termed the critical element DCR at that story. If the DCRs computed for all of the critical actions (axial force, moment, shear) of all of the components (such as beams, columns, wall piers, braces, and connections) of the primary elements are less than 2.0, then linear analysis procedures are applicable, regardless of considerations of regularity. If some computed DCRs exceed 2.0, then linear methods should not be used if any of the following apply:

- There is an in-plane discontinuity in any primary element of the lateral-force-resisting system. In-plane discontinuities occur whenever a lateral-force-resisting element is present in one story, but does not continue, or is offset, in the story immediately below. Figure 5-1 indicates such a condition.

- There is an out-of-plane discontinuity in any primary element of the lateral-force-resisting system. An out-of-plane discontinuity exists when an element in one story is offset relative to the continuation of that element in an adjacent story, as indicated in Figure 5-2.

- There is a severe weak story irregularity present at any story in any direction of the building. A severe weak story irregularity may be deemed to

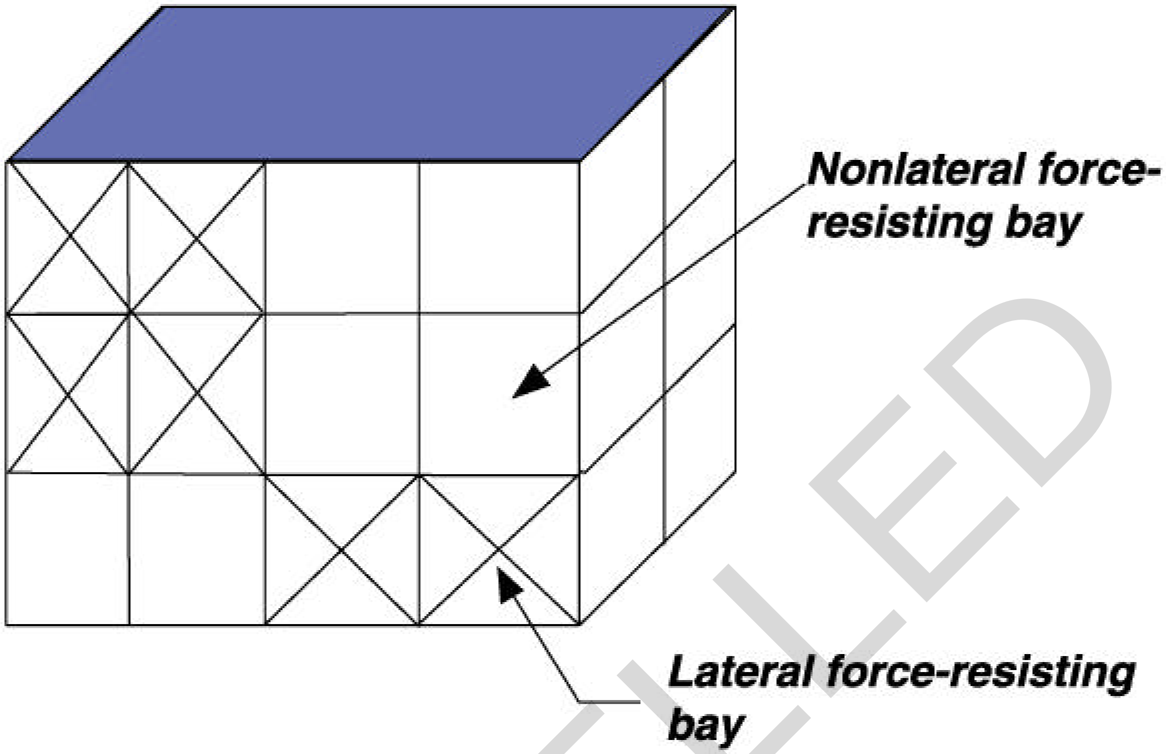


Figure 5-1: In-Plane Discontinuity in Lateral System

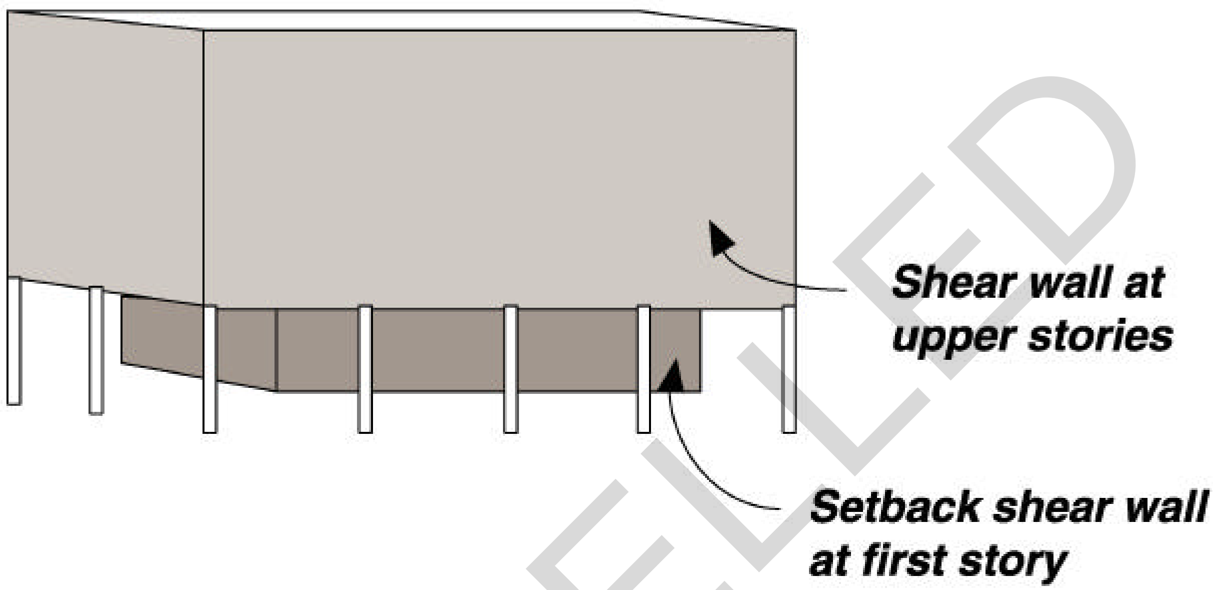


Figure 5-2: Typical Building with Out-of-Plane Offset Irregularity

exist if the ratio of the average shear DCR for any story to that for an adjacent story in the same direction exceeds 125 percent. The average DCR for a story may be calculated by the equation:

$$\overline{DCR} = \frac{\sum_{1}^{n} DCR_i V_i}{\sum_{1}^{n} V_i} \qquad (5\text{-}3)$$

where:

DCR = the average DCR for the story

DCR_I = the critical action DCR for element *i*

V_I = the total calculated lateral shear force in an element *i* due to earthquake response, assuming that the structure remains elastic

n = the total number of elements in the story.

- There is a severe torsional strength irregularity present in any story. A severe torsional strength irregularity may be deemed to exist in a story when the diaphragm above the story is not flexible, and the ratio of the critical element DCRs for primary elements on one side of the center of resistance in a given direction for a story, to those on the other side of the center of resistance for the story, exceeds 1.5.

c. Limitations on Use of the Procedure. The nonlinear static procedure may be used for any structure and for any performance objective, with the following exceptions and limitations:

- The use of the nonlinear static procedure in this document is required for those structures in Seismic Use Groups II and III with the structural characteristics described in Paragraph 5-4b, unless specific instructions to the contrary are received from the cognizant design authority.

- The procedure is not recommended for use with wood-frame structures.

- The procedure should not be used for structures in which higher-mode effects are significant unless a linear elastic dynamic procedure is also performed. To determine if higher-mode effects are significant, a modal response spectrum analysis shall be performed for the structure using sufficient modes to capture 90 percent mass participation, and a second modal response spectrum analysis shall be performed considering only the first mode participation. Higher-mode effects shall be considered significant if the shear in any story calculated from the analysis with 90 percent mass participation exceeds 130 percent of the corresponding story shear from the analysis considering only the first mode response. A linear elastic dynamic procedure may be performed to supplement the nonlinear static procedure for structures with significant higher-mode effects, in which case the acceptance criteria values for deformation-controlled actions (*m* values) in Chapter 7 may be increased by a factor of 1.3.

d. *Basis of the Procedure*. Under the Nonlinear Static Procedure, a model directly incorporating inelastic material response is displaced to a target displacement, and resulting internal deformations and forces are determined. The nonlinear-load-deformation characteristics of individual components and elements of the building are modeled directly. The mathematical model of the building is subjected to monotonically increasing lateral forces or displacements until either a target displacement is exceeded, or the building collapses. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. The target displacement may be calculated by any procedure that accounts for the effects of nonlinear response on displacement amplitude; one rational procedure is presented in Paragraph 5-4f, and further described in Paragraph 4-8b(3) and Table 4-7. Because the mathematical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake. The target displacement calculated using Equation 5-5 may be unconservative if the strength ratio of Equation 5-6 exceeds five, or if the building is located in the near field of the causative fault. Results of the Nonlinear Static Procedure are to be checked using the applicable acceptance criteria prescribed in Chapter 6, and provided in Chapter 7. Calculated displacements and internal forces are to be compared directly with the allowable values.

e. *Modeling and Analysis Criteria.*

(1) General. In this document, the Nonlinear Static Procedure involves the monotonic application of lateral forces or displacements to a nonlinear mathematical model of a building until the displacement of the control node in the mathematical model exceeds a target displacement. For buildings that are not symmetric about a plane perpendicular to the applied lateral loads, the lateral loads must be applied in both the positive and negative directions, and the maximum forces and deformations used for design. The relation between base shear force and lateral displacement of the target node shall be established for control node displacements ranging between zero and 150 percent of the target displacement, $*_t$, given by Equation 5-5. Acceptance criteria shall be based on those forces and deformations (in components and elements) corresponding to a minimum horizontal displacement of the control node equal to $*_t$. Gravity loads shall be applied to appropriate elements and components of the mathematical model during the nonlinear analysis. The loads and load combination presented in ASCE 7, as appropriate, shall be used to represent the gravity loads. The analysis model shall be discretized in sufficient detail to represent adequately the load-deformation response of each component along its length. Particular attention should be paid to identifying locations of inelastic action along the length of a component, as well as at its ends.

(2) Control node. The procedure requires definition of the control node in a building. This document considers the control node to be the center of mass at the roof of a building. The top of a penthouse should not be considered as the roof. The

displacement of the control node is compared with the target displacement, a displacement that characterizes the effects of earthquake shaking.

(3) Lateral load patterns. Lateral loads shall be applied to the building in profiles that approximately bound the likely distribution of inertia forces in an earthquake. For three-dimensional analysis, the horizontal distribution should simulate the distribution of inertia forces in the plane of each floor diaphragm. For both two- and three-dimensional analysis, the vertical distributions of lateral load shall be selected from one of the following two options:

- A lateral-load pattern represented by values of C_{VX} given in Equation 5.3.4-1 of FEMA 302, which may be used if more than 75 percent of the total mass participates in the fundamental mode in the direction under consideration.

- A lateral-load pattern proportional to the story inertia forces consistent with the story shear distribution calculated by combination of modal responses using (1) response spectrum analysis of the building including a sufficient number of modes to capture 90 percent of the total mass; and (2) the appropriate ground-motion spectrum.

(4) Period determination. The effective fundamental period T_e in the direction under consideration shall be calculated using the force-displacement relationship of the Nonlinear Procedure. The nonlinear relation between base shear and displacement of the target node shall be replaced with a bilinear relation to estimate the effective lateral stiffness, K_e, and the yield strength, V_y, of the building as indicated in Figure 5-3. The effective lateral stiffness shall be taken as the secant stiffness calculated at a base shear force equal to 60 percent of the yield strength. The effective fundamental period T_e shall be calculated as:

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \qquad (5\text{-}4)$$

where:

T_i = elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis

K_i = elastic lateral stiffness of the building in the direction under consideration

K_e = effective lateral stiffness of the building in the direction under consideration.

See Figure 5-3 for further information.

(5) Analysis of three-dimensional models. Static lateral forces shall be imposed on the three-dimensional mathematical model corresponding to the mass distribution at each floor level. The effects of accidental torsion shall be considered. Independent analysis along each principal axis of the three-dimensional mathematical model is permitted unless multi-directional evaluation is required, as prescribed in Section 5.2.6.3.1 of FEMA 302.

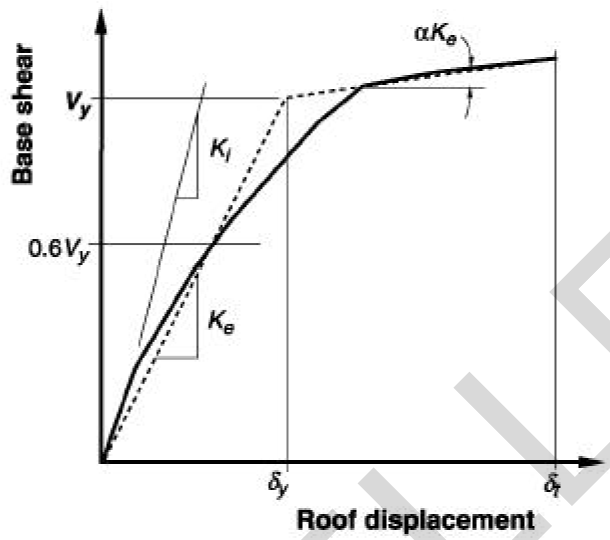


Figure 5-3: Calculation of Effective Stiffness, K_e

(6) Analysis of two-dimensional models. Mathematical models describing the framing along each axis (axis 1 and axis 2) of the structure shall be developed for two-dimensional analysis. The effects of horizontal torsion shall be considered. If multidirectional excitation effects are to be considered, component deformation demands and actions shall be computed for the following cases: 100 percent of the target displacement along axis 1, and 30 percent of the target displacement along axis 2; and 30 percent of the target displacement along axis 1, and 100 percent of the target displacement along axis 2.

f. Determination of Actions and Deformations.

(1) Pushover curve. The general procedure for the development of the load/displacement or pushover curve is as follows.

(a) An elastic structural model is developed that includes all components having significant contributions to the weight, strength, stiffness, and/or stability of the structure, and whose behavior is important in satisfying the desired level of seismic performance. The structure is loaded with gravity loads in the same load combination(s) as used in the linear procedures before proceeding with the application of lateral loads.

(b) The structure is subjected to a set of lateral loads, using one of the load patterns (distributions) described in Paragraph 5-4e(3).

(c) The intensity of the lateral load is increased until the weakest component reaches a deformation at which its stiffness changes significantly (usually the yield load or member strength). The stiffness properties of this "yielded" component in the structural model are modified to reflect post-yield behavior, and the modified structure is subjected to an increase in lateral-loads (load control) or displacements (displacement control), using the same shape of the lateral-load distribution, or an updated shape to reflect the revised fundamental mode shape. Modification of component behavior may be in one of the following forms:

- Placing a hinge where a flexural element has reached its bending strength; this may be at the end of a beam, column, or base of a shear wall.

- Eliminating the shear stiffness of a shear wall that has reached its shear strength in a particular story.

- Eliminating a bracing element that has buckled and whose post-buckling strength decreases at a rapid rate.

- Modifying stiffness properties if an element is capable of carrying more loads with a reduced stiffness.

(d) Step (c) is repeated as more and more components reach their strength. Note that although the intensity of loading is gradually increasing, the load pattern usually remains the same for all stages of the "yielded" structure, unless the user decides on the application of an adaptive load pattern. At each stage, internal forces and elastic and plastic deformations of all components are calculated.

(e) The loading process is continued until unacceptable performance is detected or a roof displacement is obtained that is larger than the maximum displacement specified in Paragraph 5-4e(1). Unacceptable performance may be defined as excessive drift of the building, or the undesirable response or failure of critical components or elements.

(f) The forces and deformations from all previous loading stages are accumulated to obtain the total forces and deformations (elastic and plastic) of all components at all loading stages.

Note: Steps (c) through (f) can be performed systematically with a nonlinear computer analysis program using an event-by-event strategy, or an incremental analysis with predetermined displacement increments in which iterations are performed to balance internal forces.

(g) The displacement of the control node versus first story (base) shear at various loading stages is plotted as a representative nonlinear response diagram of the structure. The changes in slope of this curve are indicative of the yielding of various components.

(h) The control node displacement versus base shear curve is used to estimate the target displacement, as described in the following paragraph. Note that this step may require iteration if the yield strength and stiffnesses of the simplified bilinear relation are sensitive to the target displacement.

(i) Once the target displacement is known, the accumulated forces and deformations at this displacement of the control node should be used to evaluate the performance of components and elements.

1. For deformation-controlled actions (e.g., flexure in beams), the deformation demands are compared with the maximum permissible values given in Chapter 7.

2. For force-controlled actions (e.g., shear in beams), the lower-bound strength capacity is compared with the force demand. Capacities are given in Chapters 7 through 10.

(j) If either (a) the force demand in force-controlled actions, components, or elements, or (b) the deformation demand in deformation-controlled actions, components, or elements, exceeds permissible values by more than 10 percent, then the action, component, or element is deemed to violate the performance criterion.

(k) When the demand exceeds the permissible capacity of the components or elements as described in Step (j) above, the mathematical model of the building shall be redesigned to provide additional strength and/or rigidity to the deficient components or elements, and the pushover procedure shall be repeated, as necessary. Similarly, if the evaluation indicates that a number of components or elements are overdesigned by a factor of 10 percent or more, the mathematical model shall be redesigned, and the pushover procedure repeated unless the overdesign can be justified as being cost-effective or otherwise beneficial.

(2) Target displacement. The target displacement δ_t for a building with rigid diaphragms at each floor level shall be estimated using an established procedure that accounts for the likely nonlinear response of the building. Actions and deformations corresponding to the control node displacement equaling or exceeding the target displacement shall be used for component checking in Chapter 7. The procedure for evaluating the target displacement is given by the following equation:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\Pi^2} \qquad (5\text{-}5)$$

where:

T_e = effective fundamental period of the building in the direction under consideration

C_0 = modification for C_0 can be calculated using one of the following:

- the first modal participation factor at the level of the control node

- the modal participation factor at the level of the control node calculated using a shape vector corresponding to the deflected shape of the building at the target displacement

- the appropriate value from Table 5-1.

C_1 = modification factor to relate expected maximum inelastic displacement to displacements calculated for linear elastic response.

= 1.0 for $T_e \geq T_s$

= $[1.0 + (R - 1)\, T_s/T_e]/R$ for $T_e < T_s$

but need not exceed:

C_1 = 1.5 for T_e < 0.10 sec.

C_1 may be interpreted between T_e = 0.10 sec and $T_e = T_s$.

T_s = a characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum.

R = ratio of elastic strength demand to calculated yield strength coefficient. See below for additional information.

C_2 = modification factor to represent the effect of hysteresis shape on the maximum displacement response. Values for C_2 are established in Table 5-2.

C_3 = Modification factor to represent increased displacement due to dynamic P-Δ effects. For buildings with positive post-yield stiffness, C_3 shall be set equal to 1.0. For buildings with negative post-yield stiffness, values of C_3 shall be calculated using Equation 5-7.

S_a = response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration.

The strength ratio R shall be calculated as:

Number of Stories	Modification Factor[1]
1	1
2	1.2
3	1.3
5	1.4
10+	1.5

1. Linear interpolation should be used to calculate intermediate values.

Table 5-1: Values for Modification Factor C_0

	T = 0.1 second		$T >= T_s$ second	
Performance Level	**Framing Type1[1]**	**Framing Type 2[2]**	**Framing Type 1[1]**	**Framing Type 2[2]**
Immediate Occupancy	1.0	1.0	1.0	1.0
Life Safety	1.3	1.0	1.1	1.0
Collapse Prevention	1.5	1.0	1.2	1.0

1. Structures in which more than 30% of the story shear at any level is resisted by components or elements whose strength and stiffness may deteriorate during the design earthquake. Such elements and components include: ordinary moment-resisting frames, concentrically-braced frames, frames with partially-restrained connections, tension-only braced frames, unreinforced masonry walls, shear-critical wall and piers, or any combination of the above.

2. All frames not assigned to Framing Type 1.

Table 5-2: Values for Modification Factor C_2

$$R = \frac{S_a}{V_y/W} \cdot \frac{1}{C_0} \qquad (5\text{-}6)$$

where S_a and C_0 are defined above, and:

V_y = yield strength calculated using results of Nonlinear Static Procedure, where the nonlinear force-displacement (i.e., base shear force versus control node displacement) curve of building is characterized by a bilinear relation (Figure 5-3).

W = total dead load and anticipated live load.

Coefficient C_3 shall be calculated as follows if the relation between base shear force and control node displacement exhibits negative post-yield stiffness.

$$C_3 = 1.0 + \frac{|a|(R-1)^{3/2}}{T_e} \qquad (5\text{-}7)$$

where R and T_e are defined above, and:

" = ratio of post-yield stiffness to effective elastic stiffness, where the nonlinear force-displacement relation is characterized by a bilinear relation (Figure 5-3).

For a building with flexible diaphragms at each floor level, a target displacement shall be estimated for each line of vertical seismic framing. The target displacements shall be estimated using an established procedure that accounts for the likely nonlinear response of the seismic framing. One procedure for evaluating the target displacement for an individual line of vertical seismic framing is given by Equation 5-5. The fundamental period of each vertical line of seismic framing, for calculation of the target displacement, shall follow the general procedures described for the Nonlinear Static Procedure; masses shall be assigned to each level of the mathematical model on the basis of tributary area. For a building with neither rigid nor flexible diaphragms at each floor level, the target displacement shall be calculated using rational procedures. One acceptable procedure for including the effects of diaphragm flexibility is to multiply the displacement calculated using Equation 5-5 by the ratio of the maximum displacement at any point on the roof, and the displacement of the center of mass of the roof, both calculated by modal analysis of a three-dimensional model of the building using the design response spectrum. The target displacement so calculated shall be no less than that displacement given by Equation 5-5, assuming rigid diaphragms at each floor level. No vertical line of seismic framing shall be evaluated for displacements smaller than the target displacement.

5-5. Nonlinear Dynamic Procedure.

Under the Nonlinear Dynamic Procedure, design seismic forces, and the corresponding internal forces and system displacements are determined using an inelastic response history dynamic analysis. The basis, modeling approaches, and acceptance criteria for the Nonlinear Dynamic Procedure are similar to those of the Nonlinear Static Procedure. The main exception is that the response calculations are carried out using time-history (also known as response-history) analysis. With the Nonlinear Dynamic Procedure, the design displacements are not established using a target displacement, but instead are determined directly through dynamic analysis using ground-motion histories. These analyses are highly sensitive to the modeling assumptions and to the representation of the ground motion. They are not prescribed by this document, and should only be employed by experienced analysts with express authorization of the cognizant design authority.

5-6. Alternative Rational Analyses.

Nothing in this document should be interpreted as preventing the use of any alternative analysis procedure that is rational and based on fundamental principles of engineering mechanics and dynamics. Such alternative analyses should not adopt the acceptance criteria contained in this document without careful review as to their applicability. All projects using alternative rational analysis procedures should be subject to review by an independent third-party professional engineer, approved by the cognizant design authority, with substantial experience in seismic design.

CHAPTER 6

ACCEPTANCE CRITERIA

6-1. General.

This chapter prescribes the acceptance criteria for the various performance objectives described in Chapter 4. The applicable acceptance criteria for each performance objective are provided for each of the analytical procedures described in Chapter 5. Numerical values of the criteria for specific structural systems are provided in Chapter 7.

6-2. Performance Objective 1A.

a. General. This is the basic Life Safety performance objective for all buildings, and is the only performance objective for Seismic Use Group I buildings, which constitute the vast majority of military construction. The design is based on the FEMA 302 seismic provisions with an applicable Response Modification Factor, *R*, and drift limits based on elastic analysis. The designer should not lose sight of the fact that an elastic analysis with an *R* factor greater than unity implies energy dissipation capacity in the structural system, and structural detailing that results in brittle or nonductile response could preclude the assumed energy dissipation, and lead to the development of a premature failure mechanism.

b. Analytical Procedure. As indicated in Table 4-4, the minimum analytical procedures for this performance objective are the linear elastic static or dynamic procedures with ELF or modal analysis in accordance with FEMA 302. More rigorous analytical procedures, as described in subsequent paragraphs for enhanced performance objectives, may be necessary or desirable for highly irregular or complex structural systems.

c. Design Coefficients and Factors for Basic Seismic-Force Resisting Systems. Table 7-1 (Table 5.2.2 of FEMA 302) lists the basic seismic-force-resisting systems, and for each system, provides detailing references; the applicable response modification factor, *R*; the systems overstrength factor, S_o; the deflection amplification factor, C_d; and system restrictions and building height limitations by Seismic Design Category.

d. Deflection and Drift Limits. Table 6-1 (Table 5.2.8 in FEMA 302) provides the allowable story drift applicable to each performance level for representative structural systems. The story drift,), is computed as the difference of the deflections $*_x$, of the center of mass at the top and bottom of the story under consideration. The story deflections are equal to the deflections $*_{xe}$, determined from the elastic analysis multiplied by the deflection amplification factor, C_d.

e. Acceptance Criteria. The acceptance criteria for Performance Objective 1A consists in confirming that the capacity of the structural components and elements satisfies the combined demand of the gravity and design loads in accordance with the LRFD procedures referenced for the various structural material in FEMA 302. Additionally, compliance with the drift and detailing requirements prescribed in FEMA 302, or incorporated by reference, must be met.

Table 6-1
Allowable Story Drift, ?$_a$ (in. or mm)

Structure	**Performance Level**		
	1	**2**	**3**
Structures, other than masonry shear wall or masonry wall frame structures, four stories or less in height with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	0.025 h_{sx} [b]	0.020 h_{sx}	0.015 h_{sx}
Masonry cantilever shear wall structures [c]	0.010 h_{sx}	0.010 h_{sx}	0.010 h_{sx}
Other masonry shear wall structures	0.007 h_{sx}	0.007 h_{sx}	0.007 h_{sx}
Masonry wall frame structures	0.013 h_{sx}	0.013 h_{sx}	0.010 h_{sx}
All other structures	0.020 h_{sx}	0.015 h_{sx}	0.010 h_{sx}

[a] h_{sx} is the story height below Level x.

[b] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts.

[c] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

6-3. Enhanced Performance Objectives.

The minimum analytical procedure for enhanced performance objectives is the linear elastic static or dynamic procedure using the modification factors, *m* (refer to Paragraph 5-2b(2) for the exception applicable to buildings with enhanced performance objectives in Seismic Design Categories A and B). The dynamic procedure shall be employed when the limitations described in Paragraph 5-2b preclude the use of the static procedure. The acceptance criteria for all performance objectives analyzed by the dynamic procedure are the same as for the Linear Elastic Static Procedure, except that the seismic design actions, Q_E in Equations 6-2, 6-4a, and 6-4b, for the individual structural components, are obtained by either square root of the sum of the squares (SRSS), or by the complete quadratic combination (CQC) of the modal values for each action. The nonlinear static procedure shall be used in lieu of the linear procedures when the conditions described in Paragraph 5-4b are present. Alternative analytical procedures and applicable acceptance criteria not prescribed by this document will require specific authorization by the cognizant design authority.

a. Linear Elastic Static Procedure. For those structures with a linear elastic static procedure permitted in accordance with Paragraph 5-2b, compliance with enhanced Performance Objectives 2A, 2B, and 3B shall be achieved by evaluation of the demand on individual structural components in accordance with the following procedures adopted from FEMA 273. Structural components or elements are classified as being either primary or secondary. Primary components and elements are those that provide the structure's overall ability to resist collapse under earthquake-induced ground motion. Although damage to these components, and some degradation of their strength and stiffness, may be permitted to occur, the overall function of these components in resisting structural collapse should not be compromised. Other components and elements are designated as secondary. For some structural performance levels, substantial degradation of the lateral-force-resisting strength and stiffness of secondary components and elements is permissible; however, the ability of these secondary components and elements to support gravity loads under the maximum deformations induced by the design ground motion, must be preserved.

(1) General. The analysis procedure indicates the structure's response to the design earthquake and the forces and deformations imposed on the various components, as well as the global drift demands on the structure. Acceptability of component behavior is evaluated for each of the component's various actions using Equation 6-2 for ductile (deformation-controlled) actions, and Equations 6-4a and 6-4b for nonductile (force-controlled) actions.

(a) Figure 6-1 indicates typical idealized force-deformation curves for various types of component actions. The Type 1 curve is representative of typical ductile behavior. It is characterized by an elastic range (point 0 to point 1 on the curve), a plastic range (points 1 to 2) that may include strain hardening or softening, and a strength-degraded range (points 2 to 3), in which the residual force that can be resisted is significantly less than the peak strength, but still substantial. Acceptance

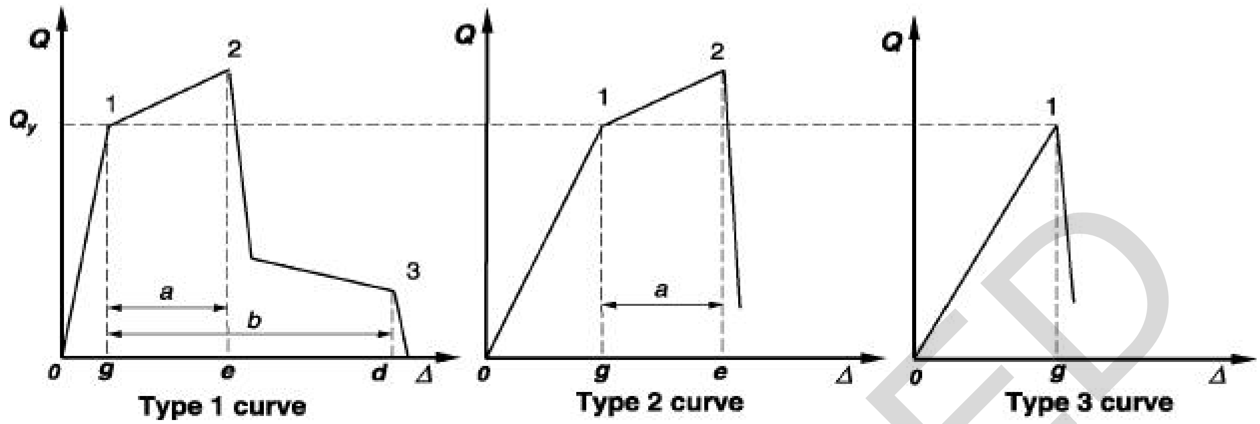


Figure 6-1 General Component Behavior Curves

criteria for primary elements that exhibit this behavior are typically within the elastic or plastic ranges, depending on the performance level. Acceptance criteria for secondary elements can be within any of the ranges. Primary component actions exhibiting this behavior are considered deformation-controlled if the plastic range is sufficiently large ($b \geq 2a$); otherwise, they are considered force-controlled. Structural steel and reinforced concrete members in flexural response are typical examples of deformation-controlled elements. Secondary component actions exhibiting this behavior are typically considered to be deformation-controlled.

(b) The Type 2 curve is representative of semi-ductile behavior. It is characterized by an elastic range and a plastic range, followed by a rapid and complete loss of strength if the behavior is categorized as deformation-controlled. Otherwise it is categorized as force-controlled. A reinforced concrete wall in shear response is a typical example of a deformation-controlled element with semi-ductile behavior. Acceptance criteria for primary and secondary components exhibiting this behavior will be within the elastic or plastic ranges, depending on the performance level.

(c) The Type 3 curve is representative of a brittle or non-ductile behavior. It is characterized by an elastic range, followed by a rapid and complete loss of strength. Component actions resulting in this behavior are always categorized as force-controlled. Shear critical (i.e., shear capacity is attained prior to flexural capacity) beams and columns in reinforced concrete frames are typical examples of force-controlled elements. Acceptance criteria for primary and secondary components exhibiting this behavior are always within the elastic range.

(d) Figure 6-2 shows an idealized force versus deformation curve that is used throughout this procedure to specify acceptance criteria for deformation-controlled components and element actions for any of the four basic types of materials. Linear response is depicted between point A (unloaded component) and an effective yield point B. The slope from B to C is typically a small percentage (0 to 10 percent) of the elastic slope, and is included to represent phenomena such as strain hardening. C has an ordinate that represents the strength of the component, and an abscissa value equal to the deformation at which significant strength degradation begins (line CD). Beyond point D, the component responds with substantially reduced strength to point E. At deformations greater than point E, the component strength is essentially zero. In Figure 6-1, Q_{CE} is the expected strength of a component or element at the deformation level under consideration for deformation-controlled actions. Expected strength is defined as the mean value of resistance at the deformation level anticipated, and includes phenomena such as strain hardening and plastic section development. Q_{CL} is the lower-bound strength of a component or element at the deformation level under consideration for force-controlled actions. Lower-bound strength is typically established by the lower five percentile of yield, buckling, or brittle failure strength. Q_{CE} and Q_{CL} are further defined in Paragraph 6-3a(3).

(e) For some components it is convenient to prescribe acceptance criteria in terms of deformation (e.g., 2 or)), while for others it is more convenient to give criteria in terms of deformation ratios. To accommodate this, two types of idealized force versus deformation curves are used in this procedure as illustrated in Figures 6-2a and 6-2b. Figure 6-2a shows normalized force (Q/Q_{CE}) versus deformation (or)) and the parameters a, b, and c. Figure 6-2b shows normalized force (Q/Q_{CE}) versus deformation ratio ($2/2_y$, $)/)_y$ or)/h) and the parameters d, e, and c. Elastic stiffness and values for the parameters a, b, c, d, and e that can be used for modeling components for various structural systems are given in Chapter 7. Figure 6-2c graphically shows the approximate deformation or deformation ratio, in relation to the idealized force versus deformation curve, that is deemed acceptable in this procedure for structural components for Immediate Occupancy (IO), Safe Egress (SE), and Life Safety (LS), Performance Levels. The Collapse Prevention (CP) performance level indicated in Figure 6-2c is not an acceptable performance level, and is indicated here and in the acceptance criteria tables in Chapter 7 as a limit state for ductile response. Numerical values of the acceptable deformations or deformation ratios are given in Chapter 7 for components and elements in various structural systems. Additional guidelines on the calculation of individual component force and deformation capacities may be found in the following chapters.

- Base isolation systems and energy dissipation systems - Chapter 8.

- Foundations - Chapter 9.

- Nonstructural Systems and Components – Chapter 10.

Acceptance criteria for elements and components for which criteria are not presented in this document shall be determined by an approved qualification-testing program.

(2) Pseudo-lateral load, V, in a given horizontal direction of a building, is given by Equation 6-1. This load shall be used for the design of the vertical seismic framing system when the linear elastic analysis procedures are used with the *m* values.

$$V = C_1\, C_2\, C_3\, S_a\, W \qquad (6\text{-}1)$$

where:

V = pseudo-lateral load. This force, when distributed over the height of the linearly elastic model of the building, is intended to produce calculated lateral displacements approximately equal to those that are expected in the real structure during the design event. If it is expected that the actual structure will yield during the design event, the force given by Equation 6-1 may be significantly larger than the actual strength of the structure to resist that force. The acceptance criteria in the following paragraph are developed to take this aspect into account.

C_1, C_2, C_3, and S_a are defined in Paragraph 5-4f(2).

W = Total dead and applicable live loads as defined in FEMA 302.

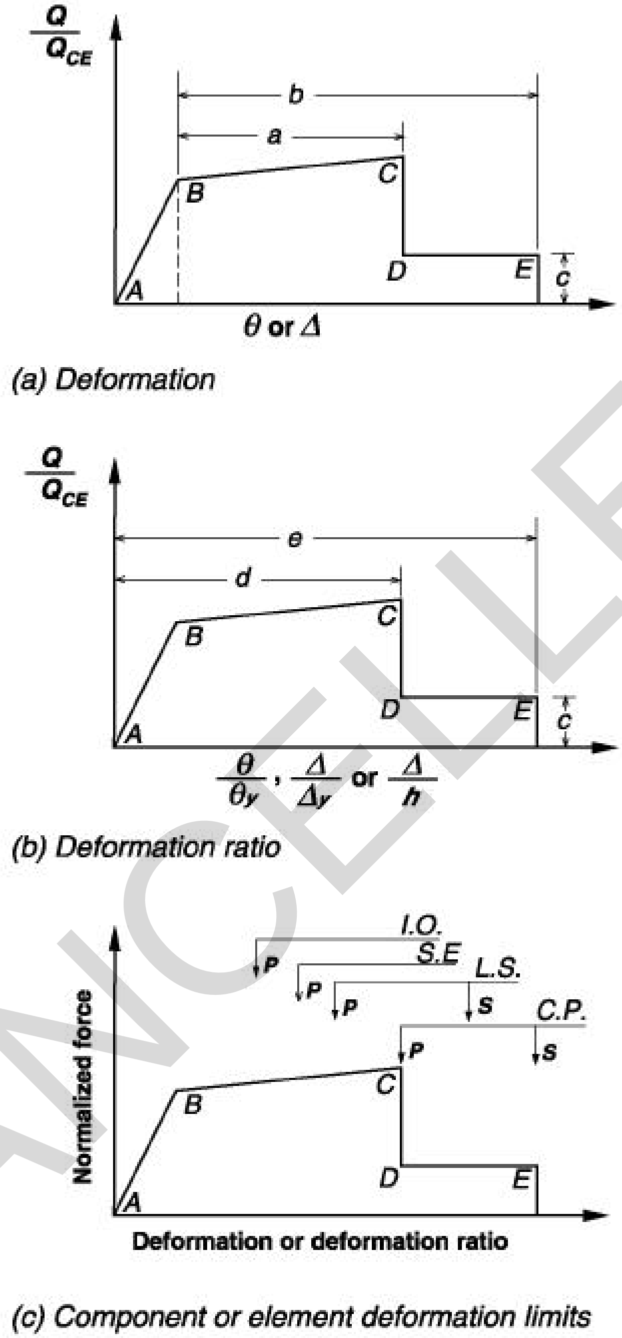


Figure 6-2 Idealized Component Load Versus Deformation Curves for Depicting Component Modeling and Acceptability

(3) Design actions.

(a) Deformation-controlled actions shall be calculated according to Equation 6-2:

$$Q_{UD} = Q_G \pm Q_E \qquad (6\text{-}2)$$

where:

Q_{UD} = design action due to gravity loads and earthquake loads.

Q_G = action due to design gravity loads as defined in ASCE 7.

Q_E = action due to design earthquake loads.

Deformation-controlled actions in structural components shall satisfy Equation 6-3:

$$mQ_{CE} \geq Q_{UD} \qquad (6\text{-}3)$$

where:

m = component or element demand modifier to account for expected ductility of the deformation associated with this action at selected Performance Level (see Chapters 7, 8, 9 and 10).

Q_{CE} = expected strength of the component or element at the deformation level under consideration for deformation-controlled actions.

For Q_{CE}, the expected strength shall be determined considering all co-existing actions acting on the component under the design loading condition. In this document, Q_{CE} is defined as the nominal strength, Q_N, multiplied by 1.25, unless otherwise noted in Chapters 7 through 10.

(b) Force-controlled actions. Force-controlled actions in structural or nonstructural components or elements are those responses generally characterized by the Type 3 curve and in some cases by the Type 2 curve in Figure 6-1. Acceptance criteria for the capacity of these components or elements are provided in Chapter 7, and the components or elements shall be evaluated in accordance with the provisions of this paragraph. The value of force-controlled design action, Q_{UF}, need not exceed the maximum action that can be developed in a component considering the nonlinear behavior of the structure. In lieu of more rational analysis, design actions may be calculated according to Equation 6-4a or 6-4b. Note that Q_E has been determined from the pseudo-lateral load, V, defined in Paragraph (2) above as the basic spectral response force, S_aW, modified by C_1, C_2, and C_3 to represent the expected deformation in the building. In Equation 6-4b, Q_E is divided by the modification factors to restore Q_E to a force-controlled action. The force delivery factor, J, in Equation 6-4a, represents an approximation of the additional reduction in the force delivered to a force-controlled component or element by a yielding component of the seismic framing system.

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3 J} \qquad (6\text{-}4a)$$

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3} \qquad (6\text{-}4b)$$

where:

Q_{UF} = design actions due to gravity loads and earthquake loads.

$C_1C_2C_3$ = coefficients as defined in Paragraph 5-4f.

J = a force-delivery reduction factor given by Equation 6-5.

The coefficient J shall be established using Equation 6-5:

$$J = 1.0 + S_{DS}, \text{ not to exceed } 2 \qquad (6\text{-}5)$$

where:

S_{DS} = spectral acceleration, described in Chapter 3.

Equation 6-4b can be used in all cases. Equation 6-4a can only be used if the forces contributing to Q_{UF} are delivered by yielding components of the seismic framing system. Force-controlled actions in primary and secondary components and elements shall satisfy Equation 6-6.

$$Q_{CL} \ \$ \ Q_U \qquad (6\text{-}6)$$

where:

Q_{CL} = lower-bound strength of a component or element at the deformation level under consideration for force-controlled actions.

For Q_{CL}, the lower-bound strength shall be determined considering all co-existing actions acting on the component under the design loading condition. In this document, Q_{CL} is defined as the nominal strength Q_N multiplied by the appropriate capacity reduction factor, N, unless otherwise noted in Chapters 7 through 10.

(3) Nonstructural components. As indicated in Paragraph 4-12, the minimum performance objective for all nonstructural components will be similar to structural Performance Objective 1A, and the acceptance criteria are satisfied by compliance with Chapter 6 of FEMA 301 with a component importance factor of 1.0. Selected nonstructural components shall be assigned component importance factors, in accordance with Paragraph 10-1d, regardless of the structural performance objectives of the building.

b. Nonlinear Static Procedure.

(1) General. This procedure shall be used for the evaluation of structures in Seismic Use Groups II and III, with the characteristics described in Paragraph 5-4b. Acceptance criteria are also provided for this procedure to satisfy Performance Objective 1A, but the use of this procedure for that performance objective requires specific authorization from the cognizant design authority.

(2) Actions and Deformations. With the procedures as described in Paragraph 5-4, compliance with the performance objective requires compliance with the global displacement criteria for the structure as a whole, and the local deformation criteria for individual structural elements.

(a) Global displacement. The displacement for the control node of the structure in the force/displacement plot (i.e., pushover analysis) must equal or exceed the target displacement, δ_t, described in Paragraph 5-4f. Story drifts shall not exceed the values indicated in Table 6-1.

(b) Deformation-controlled actions. Primary and secondary components shall have expected deformation capacities not less than the deformations derived from the pushover analysis when the target displacement, δ_t, is attained. Modeling parameters and numerical acceptance criteria are provided for each performance objective for the structural systems described in Chapters 7 through 10. The acceptance criteria are provided in terms of rotations, θ, in radians; rotation ratios, θ/θ_y; or deformation ratios Δ/Δ_y, as depicted in Figure 6-2.

1. Steel moment and braced frames. Acceptance criteria are provided in terms of either plastic rotations of ratios or plastic rotations to yield rotations. Figure 6-3 illustrates the definition of chord rotation for frame beams and columns. If it is assumed that the total chord rotation, θ, (elastic plus plastic rotation) is defined by the interstory drift, Δ/h, then the interstory drift ratio becomes a convenient parameter to monitor the inelastic deformations by subtraction of the yield deformations, θ_y.

i. For beams:

$$\theta_y = \frac{ZF_{ye}\ell_b}{6EI_b} \qquad (6\text{-}7)$$

ii. For columns:

$$\theta_y = \frac{ZF_{ye}\ell_c}{6EI_c}\left(1 - \frac{P}{P_{ye}}\right) \qquad (6\text{-}8)$$

where:

Z = Plastic section modulus, in^3 (mm^3).

F_{ye} = Expected yield strength, psi (kPa), as defined in the AISC Seismic provisions.

I_b = Moment of inertia of beams, in^4 (mm^4).

I_c = Moment of inertia of columns, in^4 (mm^4).

ℓ_b = Beam length, in (mm).

ℓ_c = Column length, in (mm).

P = Axial force in the columns, kips (kN).

P_{ye} = Expected axial yield strength, $A_g F_{ye}$ kips (kN).

iii. For beams in partially restrained moment frames, EI_b in Equation 6-7 is modified to:

$$EI_b \text{ (adjusted)} = \frac{1}{\dfrac{6h}{\ell_b^2 K_\Theta} + \dfrac{1}{EI_b}} \qquad (6\text{-}9)$$

where:

h = Average story height of the columns, in. (mm) K_θ = Rotational spring stiffness, estimated as $M_{CE}/0.005$, kip-in per rad ($M_{CE}/0.044$, kN-m per rad.).

M_{CE} = Expected moment capacity of the connection, kip-in. (kN-m)

iv. For link beams in eccentric braced frames:

$$\theta_y = Q_{CE}/eK_e \qquad (6\text{-}10)$$

where:

θ_y = Yield deformation of the link, rad.

Q_{CE} = Expected shear strength of link beam, kips (kN) = 0.6 F_{ye} A_w

F_{ye} = Expected yield strength, ksi (kPa)

A_w = Area of link beam $(d_b\text{-}2t_f)t_w$, in^2 (mm^2)

d_b = Depth of link beam, in (mm).

t_f = Thickness of link beam flanges, in. (mm).

t_w = Thickness of link beam web, in. (mm)

A_w = Area of link beam web, in^2 (mm^2)

e = Length of link beam, in. (mm)

K_e = Stiffness of link beam, kip/in (kN/mm) = $\dfrac{K_s K_b}{K_s + K_b}$

K_s = Shear stiffness of link beam, kip/in (kN/mm) = $\dfrac{GA_w}{e}$

G = Shear modulus, $kips/in^2$ (kPa)

K_b = Flexural stiffness of link beam, kips/in (kN/mm) = $12EI_b/e^3$.

2. Concrete moment frames. Acceptance criteria for reinforced concrete beams, columns, and beam/column joints in moment frames are tabulated in Chapter 7. The numerical values are given as the plastic rotation angles in radians as defined in Figure 6-2. As described in Paragraph 6-3b(2)(b)1 above, the total chord rotation may be assumed to be equal to the interstory drift ratio, Δ/h, and the yield chord rotation, θ_y, for beams and columns is assumed to be:

$$\theta_y = \frac{M_{CE}\, d}{E_c I_g} \qquad (6\text{-}11)$$

where:

M_{CE} = Expected moment capacity of the beam or column with the design axial load and F_y for the reinforcement equal to F_{ye}.

d = Depth of the beam or column, in. (mm).

E_c = Elastic modulus of the concrete, ksi (kPa).

I_g = Gross moment of inertia of the beam of column, in^4 (mm^4).

(Note that in Equation 6-11, the yield curvature, N_y, is calculated with $I_g/2$ and the plastic hinge length is assumed to be $d/2$.)

3. Reinforced concrete shear walls.

i. Controlled by flexure. For shear walls in which the vertical reinforcement is expected to yield in flexure prior to the wall exceeding its shear capacity, the acceptance criteria in Table 7-4 are provided in terms of the plastic rotation, 2, as indicated in Figure 6-4 and are similar to that for concrete moment-resisting frames in Paragraph 2 above with the depth, *d*, in Equation 6-11 to be replaced by the length of the wall.

ii. Controlled by shear. For shear walls when the shear capacity is attained prior to flexural yielding of the reinforcement, the tabulated acceptance criteria values in Table 7-5 represent allowable values of the interstory drift ratio, Δ/h, with reference to Figure 6-2b, and it is not necessary to determine Δy.

iii. Coupling beams. The acceptance criteria in Table 7-4, for coupling beams controlled by flexure, are evaluated as shown in Figure 6-5 for beams in moment frames. Coupling beams controlled by shear are evaluated as indicated above for walls controlled by shear, and the acceptance criteria are tabulated in Table 7-5.

4. Reinforced masonry shear walls. The acceptance criteria for these shear walls, tabulated in Table 7-9, are provided in terms of drift ratios, Δ/h, as indicated in Figure 6-2b. Acceptance criteria for coupling beams for reinforced masonry walls are similar to criteria for coupling beams in reinforced concrete shear walls described in Paragraph iii above.

(c) Force-controlled actions. Structural components shall have lower-bound strengths, Q_{CL}, not less than the required strength, Q_{UF}, from the appropriate combinations of seismic and gravity load effects. Lower-bound strengths, Q_{CL}, are defined in Paragraph 6-3a(3)b and in Chapters 7 through 10.

(d). Reanalysis. The results of the analysis must be carefully monitored to determine whether any of the structural components have exceeded the deformation limits indicated in Chapters 7 through 10 for the desired performance objectives. Minor exceedance (i.e., 10 to 15 percent) of the deformation limits in a limited number of components may be acceptable if it can be demonstrated that the additional deformation does not have an adverse effect on the performance of the structure. All other components with excessive deformations should be strengthened to meet the acceptance criteria. If the revised member sizes for the components are significant, a reanalysis may be required to confirm an acceptable response. Similarly, if the results of the analysis indicate that a number of the components or elements are overdesigned by a factor of 10 to 15 percent, the overdesigned components or elements shall be redesigned, and the analysis reported, unless it can be demonstrated that the overdesign is cost-effective, or otherwise beneficial. If the structural members are required to be substantially stronger or stiffer, as compared to the design for gravity loads, the designer should consider the use of a supplementary structural system; such as the use of shear walls or braced frames to stiffen a flexible moment frame system.

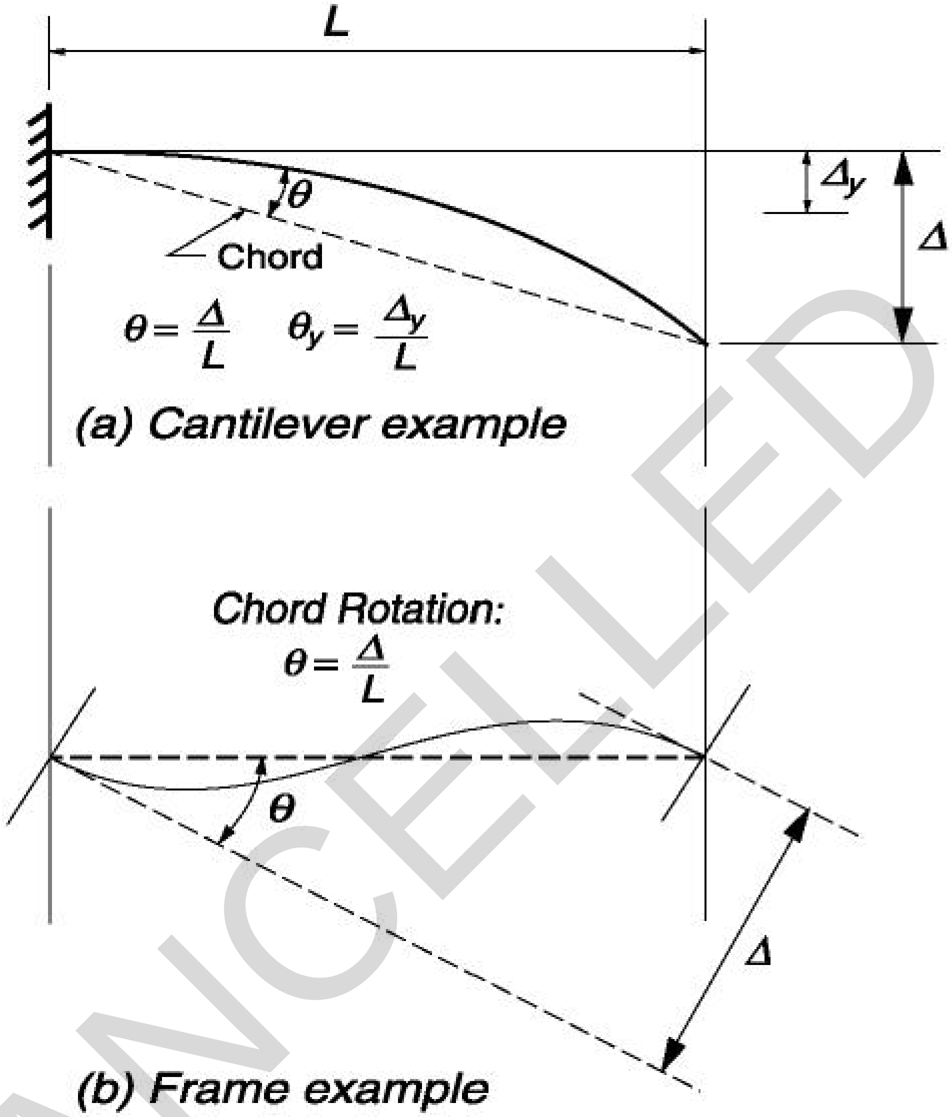


Figure 6-3 Definition of Chord Rotation

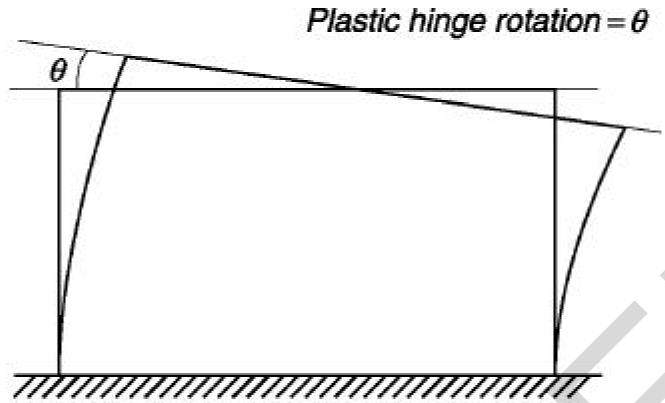


Figure 6-4 Plastic Hinge Rotation in Shear Wall where Flexure Dominates Inelastic Response

3. Reinforced concrete shear walls.

i. Controlled by flexure. For shear walls in which the vertical reinforcement is expected to yield in flexure prior to the wall exceeding its shear capacity, the acceptance criteria in Table 7-4 are provided in terms of the plastic rotation, θ, as indicated in Figure 6-4 and are similar to that for concrete moment-resisting frames in Paragraph 2 above with the depth, *d*, in Equation 6-11 to be replaced by the length of the wall.

ii. Controlled by shear. For shear walls when the shear capacity is attained prior to flexural yielding of the reinforcement, the tabulated acceptance criteria values in Table 7-5 represent allowable values of the interstory drift ratio, Δ/h, with reference to Figure 6-2b, and it is not necessary to determine Δy.

iii. Coupling beams. The acceptance criteria in Table 7-4, for coupling beams controlled by flexure, are evaluated as shown in Figure 6-5 for beams in moment frames. Coupling beams controlled by shear are evaluated as indicated above for walls controlled by shear, and the acceptance criteria are tabulated in Table 7-5.

4. Reinforced masonry shear walls. The acceptance criteria for these shear walls, tabulated in Table 7-9, are provided in terms of drift ratios, Δ/h, as indicated in Figure 6-2b. Acceptance criteria for coupling beams for reinforced masonry walls are similar to criteria for coupling beams in reinforced concrete shear walls described in Paragraph iii above.

(c) Force-controlled actions. Structural components shall have lower-bound strengths, Q_{CL}, not less than the required strength, Q_{UF}, from the appropriate combinations of seismic and gravity load effects. Lower-bound strengths, Q_{CL}, are defined in Paragraph 6-3a(3)b and in Chapters 7 through 10.

(d). Reanalysis. The results of the analysis must be carefully monitored to determine whether any of the structural components have exceeded the deformation limits indicated in Chapters 7 through 10 for the desired performance objectives. Minor exceedance (i.e., 10 to 15 percent) of the deformation limits in a limited number of components may be acceptable if it can be demonstrated that the additional deformation does not have an adverse effect on the performance of the structure. All other components with excessive deformations should be strengthened to meet the acceptance criteria. If the revised member sizes for the components are significant, a reanalysis may be required to confirm an acceptable response. Similarly, if the results of the analysis indicate that a number of the components or elements are overdesigned by a factor of 10 to 15 percent, the overdesigned components or elements shall be redesigned, and the analysis reported, unless it can be demonstrated that the overdesign is cost-effective, or otherwise beneficial. If the structural members are required to be substantially stronger or stiffer, as compared to the design for gravity loads, the designer should consider the use of a supplementary structural system; such as the use of shear walls or braced frames to stiffen a flexible moment frame system.

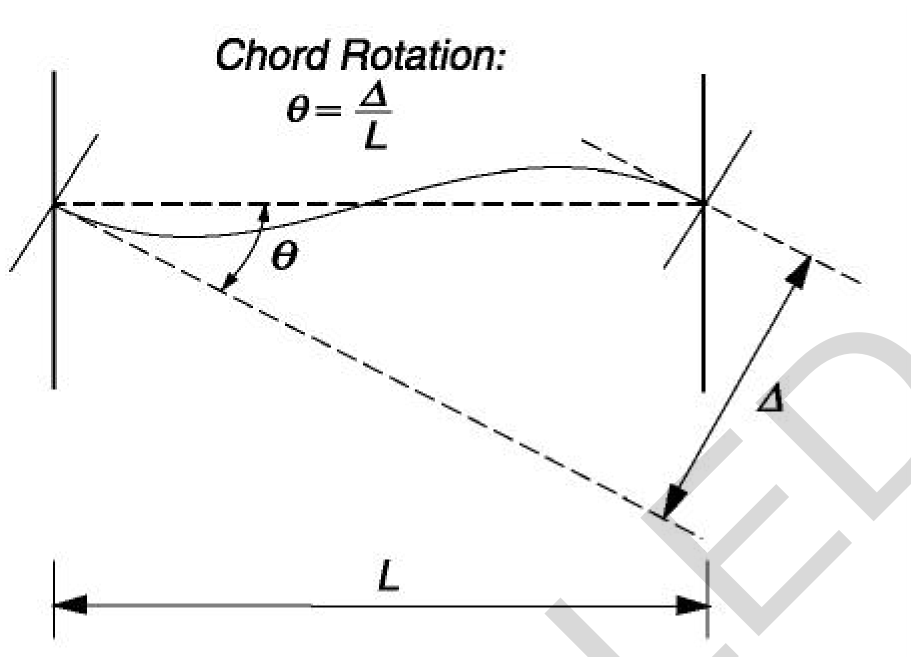


Figure 6-5 Chord Rotation for Shear Wall Coupling Beams

CHAPTER 7
STRUCTURE SYSTEMS
AND COMPONENTS

7-1. General.

This chapter provides acceptance criteria applicable to structural systems for the design of military buildings. Advantages and disadvantages of each system are discussed, and pertinent detailing provisions are provided and illustrated with typical details. Alternative structural systems, other than those described in this chapter, should not be used in the design of military buildings without specific approval from the proponent agency.

a. Design for Life Safety. As indicated in Chapter 4, all buildings will be designed to protect life safety (Performance Objective 1A). Following the selection of the appropriate Seismic Use Group, Seismic Design Category, and analytical procedures, the design of the building is performed in accordance with the provisions of FEMA 302. Table 5.2.2 of "Design Coefficients and Factors of Basic Seismic-Force-Resisting Systems" is reproduced in this document as Table 7-1 for ease of reference.

b. Enhanced Performance Objectives. Chapter 4 prescribes the minimum analytical procedures for the enhanced performance objectives (Performance Objectives 2A, 2B, and 3B); the analytical procedures are discussed in Chapter 5; and numerical values for the acceptance criteria prescribed in Chapter 6 are provided in this chapter for various components of each structural systems. These values, modified from tables in FEMA 273, represent the current state of the art as defined by panels of experts. Future modification of these values should be expected as they are tested by analytical case studies and actual earthquakes. Alternative values, derived from test data in the literature or performed on a project-specific basis, may be used in lieu of the tabulated values, subject to the approval of the proponent agency.

(1) *m* factor tables. Tables of numerical acceptance criteria for linear procedures (*m* factors) are provided in this chapter for various deformation-controlled components and elements of structural systems. The columns in the left-hand side of the tables refer to the applicable condition of the component or element, and the columns on the right list the appropriate *m* factor for the following performance levels:

IO = immediate occupancy

SE = safe egress

LS = life safety

CP = collapse prevention.

The performance levels are defined in Table 4-3, and their physical significance is indicated in Figures 6-1 and 6-2.

(2) Nonlinear acceptance criteria. Tables of modeling parameters and numerical acceptance criteria for nonlinear procedures are provided for the same components and elements and in the same format as in the *m* factor tables for linear procedures.

TABLE 7-1
Design Coefficients and Factors for Basic Seismic-Force-Resisting Systems

Basic Seismic-Force-Resisting System	Detailing Reference	Response Modification Co-Efficient, R[a]	System Overstrength Factor, Ω_o[g]	Deflection Amplification Factor, C_d[b]	System Limitations and Building Height Limitations (ft) by Seismic Design Category[e]				
					B	C	D[d]	E[e]	F[f]
Bearing Wall Systems									
Ordinary steel braced frames	14 [k]	4	2	3½	NL	NL	160	160	160
Special reinforced concrete shear walls	9.3.2.4 [l]	5½	2½	5	NL	NL	160	160	160
Ordinary reinforced concrete shear walls	9.3.2.3 [l]	4½	2½	4	NL	NL	NP	NP	NP
Detailed plain concrete shear walls	9.3.2.2 [l]	2½	2½	2	NL	NL	NP	NP	NP
Ordinary plain concrete shear walls	9.3.2.1 [l]	1½	2½	1½	NL	NP	NP	NP	NP
Special reinforced masonry shear walls	11.11.5 [l]	4	2½	3½	NL	NL	160	160	160
Intermediate reinforced masonry shear walls	11.11.4 [l]	3½	2½	3	NL	NL	NP	NP	NP
Ordinary reinforced masonry shear walls	11.11.3 [l]	2	2½	1¾	NL	NP	NP	NP	NP
Detailed plain masonry shear walls	11.11.2 [l]	2	2½	1¾	NL	160	NP	NP	NP
Ordinary plain masonry shear walls	11.11.1 [l]	1½	2½	1¼	NL	NP	NP	NP	NP
Light frame walls with shear panels	3[n] 12.3.4.[l] 12.4 [l]	6½	3	4	NL	NL	160	160	100
Building Frame Systems									
Steel eccentrically braced frames, moment resisting, connections at columns away from links	15 [k]	8	2	4	NL	NL	160	160	100
Steel eccentrically braced frames, nonmoment resisting, connections at columns away from links	15 [k]	7	2	4	NL	NL	160	160	100
Special steel concentrically braced frames	13 [k]	6	2	5	NL	NL	160	160	100
Ordinary steel concentrically braced frames	14 [k]	5	2	4½	NL	NL	160	100	100
Special reinforced concrete shear walls	9.3.2.4 [l]	6	2½	5	NL	NL	160	160	100
Ordinary reinforced concrete shear walls	9.3.2.3 [l]	5	2½	4½	NL	NL	NP	NP	NP
Detailed plain concrete shear walls	9.3.2.2 [l]	3	2½	2½	NL	NL	NP	NP	NP
Ordinary plain concrete shear walls	9.3.2.1 [l]	2	2½	2	NL	NP	NP	NP	NP

TABLE 7-1. (cont'd)
Design Coefficients and Factors for Basic Seismic-Force-Resisting-Systems

Basic Seismic-Force-Resisting System	Detailing Reference	Response Modification Co-Efficient, R[a]	System Overstrength Factor, Ω_o[g]	Deflection Amplification Factor, C_d[b]	System Limitations and Building Height Limitations (ft) by Seismic Design Category[e]				
					B	C	D[d]	E[e]	F[f]
Composite eccentrically braced frames	13 [m]	8	2	4	NL	NL	160	160	100
Composite concentrically braced frames	14 [m]	5	2	4½	NL	NL	160	160	100
Ordinary composite braced frames	12 [m]	3	2	3	NL	NL	NP	NP	NP
Composite steel plate shear walls	17 [m]	6½	2½	5½	NL	NL	160	160	100
Special composite reinforced concrete shear walls with steel elements	16 [m]	6	2½	5	NL	NL	160	160	100
Ordinary composite reinforced concrete shear walls with steel elements	15 [m]	5	2½	4½	NL	NL	NP	NP	NP
Special reinforced masonry shear walls	11.11.5 [l]	5	2½	4	NL	NL	160	160	100
Intermediate reinforced masonry shear walls	11.11.4 [l]	4½	2½	4	NL	NL	160	160	100
Ordinary reinforced masonry shear walls	11.11.3 [l]	2½	2½	2¼	NL	NL	NP	NP	NP
Detailed plain masonry shear walls	11.11.2 [l]	2½	2 ½	2¼	NL	160	NP	NP	NP
Ordinary plain masonry shear walls	11.11.1 [l]	1½	2½	1¼	NL	NL	NP	NP	NP
Light frame walls with shear panels	12.3.4 [l] 12.4 [l]	7	2½	4½	NL	NL	160	160	160
Moment Resisting Frame Systems									
Special steel moment frames	9[k]	8	3	5 ½	NL	NL	NL	NL	NL
Special steel truss moment frames	12[k]	7	3	5 ½	NL	NL	160	100	NP
Intermediate steel moment frames	10[k]	6	3	5	NL	NL	160	100	NP[l]
Ordinary steel moment frames	11[k]	4	3	3½	NL	NL	35[l]	NP[ij]	NP[ij]
Special reinforced concrete moment frames	9.3.1.3[l]	8	3	5½	NL	NL	NL	NL	NL
Intermediate reinforced concrete moment frames	9.3.1.2[l]	5	3	4½	NL	NL	NP	NP	NP
Ordinary reinforced concrete moment frames	9.3.1.1[l]	3	3	2½	NL [a]	NP	NP	NP	NP
Special composite moment frames	9 [m]	8	3	5½	NL	NL	NL	NL	NL
Intermediate composite moment frames	10 [m]	5	3	4½	NL	NL	NP	NP	NP
Composite partially restrained moment	8 [m]	6	3	5½	160	160	100	NP	NP

TABLE 7-1. (cont'd)
Design Coefficients and Factors for Basic Seismic-Force-Resisting-Systems

Basic Seismic-Force-Resisting System	Detailing Reference	Response Modification Co-Efficient, R^a	System Overstrength Factor, Ω_o^g	Deflection Amplification Factor, C_d^b	System Limitations and Building Height Limitations (ft) by Seismic Design Category[e]				
					B	C	D[d]	E[e]	F[f]
frames									
Ordinary composite moment frames	11 [m]	3	3	4	NL	NP	NP	NP	NP
Masonry wall frames		5½	3	5	NL	NL	160	160	100
Dual Systems with Special Moment Frames Capable of Resisting at Least 25% of Prescribed Seismic Forces									
Steel eccentrically braced frames, moment resisting connections at columns away from links	15 [k]	8	2½	4	NL	NL	NL	NL	NL
Steel eccentrically braced frames, non-moment resisting connections, at columns away from links	15 [k]	7	2½	4	NL	NL	NL	NL	NL
Special steel concentrically braced frames	13 [k]	8	2½	6½	NL	NL	NL	NL	NL
Ordinary steel concentrically braced frames	14 [k]	6	2½	5	NL	NL	NL	NL	NL
Special reinforced concrete shear walls	9.3.2.4 [l]	8	2½	6½	NL	NL	NL	NL	NL
Ordinary reinforced concrete shear walls	9.3.2.3 [l]	7	2½	6	NL	NL	NP	NP	NP
Composite eccentrically braced frames	13 [m]	8	2½	4	NL	NL	NL	NL	NL
Composite concentrically braced frames	14 [m]	6	2½	5	NL	NL	NL	NL	NL
Composite steel plate shear walls	17 [m]	8	3	6½	NL	NL	NL	NL	NL
Special composite reinforced concrete shear walls with steel elements	16 [m]	8	3	6½	NL	NL	NL	NL	NL
Ordinary composite reinforced concrete shear walls with steel elements	15 [m]	7	3	6½	NL	NL	NP	NP	NP
Special reinforced masonry shear walls	11.11.5 [l]	7	3	6½	NL	NL	NL	NL	NL
Intermediate reinforced masonry shear walls	11.11.4 [l]	6½	3	5½	NL	NL	NL	NP	NP
Dual Systems with Intermediate Moment Frames Capable of Resisting at Least 25% of Prescribed Seismic Forces									
Special steel concentrically braced frames [f]	13 [k]	6	2½	5	NL	NL	160	100	NP
Ordinary steel concentrically braced frames [f]	14 [k]	5	2½	4½	NL	NL	160	100	NP
Special reinforced concrete shear walls	9.3.2.4 [l]	6	2½	5	NL	NL	160	100	100
Ordinary reinforced concrete shear walls	9.3.2.3 [l]	5½	2½	4½	NL	NL	NP	NP	NP

TABLE 7-1. (cont'd)
Design Coefficients and Factors for Basic Seismic-Force-Resisting-Systems

Basic Seismic-Force-Resisting System	Detailing Reference	Response Modification Co-Efficient, R^a	System Overstrength Factor, Ω_o^g	Deflection Amplification Factor, C_d^b	System Limitations and Building Height Limitations (ft) by Seismic Design Category[e]				
					B	C	D[d]	E[e]	F[f]
Ordinary reinforced masonry shear walls	11.11.3 [l]	3	3	2½	NL	160	NP	NP	NP
Intermediate reinforced masonry shear walls	11.11.4 [l]	5	3	4½	NL	NL	160	NP	NP
Composite concentrically braced frames	14 [m]	5	2½	4½	NL	NL	160	100	NP
Ordinary composite braced frames	12 [m]	4	2½	3	NL	NL	NP	NP	NP
Ordinary composite reinforced concrete shear walls with steel elements	16 [m]	5	3	4½	NL	NL	NP	NP	NP
Inverted Pendulum Systems									
Special steel moment frames	9 [k]	2½	2	2½	NL	NL	NL	NL	NL
Ordinary steel moment frames	11 [k]	1¼	2	2½	NL	NL	NP	NP	NP
Special reinforced concrete moment frames	9.3.1.3 [l]	2½	2	1¼	NL	NL	NL	NL	NL
Structural Steel Systems Not Specifically Detailed for Seismic Resistance		3	3	3	NL	NL	NP	NP	NP

TABLE 7-1. (cont'd)
Design Coefficients and Factors for Basic Seismic-Force-Resisting-Systems

NOTES FOR TABLE 7-1

[a] Response modification coefficient, R, for use throughout the Provisions of FEMA 302.

[b] Deflection amplification factor, C_d for use in Section 5.3.7.1 and 5.3.7.2. of FEMA 302.

[c] NL – not limited; and NP = not permitted. If using metric units, 100 feet approximately equals 30 m and 160 feet approximately equals 50 m.

[d] See Section 5.2.2.4.1 of FEMA 302 for a description of building systems limited to buildings with a height of 240 feet (70 m) or less.

[e] See Section 5.2.2.5 of FEMA 302 for building systems limited to buildings with a height of 160 feet (50 m) or less.

[f] Ordinary moment frame is permitted to be used in lieu of intermediate moment frame in Seismic Design Categories B and C.

[g] The tabulated value of the overstrength factor, Ω_0, may be reduced by subtracting 1/2 for structures with flexible diaphragms, but shall not be taken as less than 2.0 for any structure.

[h] Ordinary moment frames of reinforced concrete are not permitted as a part of the seismic-force-resisting system in Seismic Design Category B structures founded on Site Class E or F soils (see Section 9.5.2 of FEMA 302).

[i] Steel ordinary moment frames and intermediate moment frames are permitted in single-story buildings up to a height of 60 feet (18m) when the moment joints of field connections are constructed of bolted end plates, and the dead load of the roof does not exceed 15 psf (103 kPa).

[j] Steel ordinary moment frames are permitted in buildings up to a height of 35 feet (11mm) where the dead load of the walls, floors, and roofs does not exceed 15 psf (103 kPa).

[k] Refers to Section in Part I of AISC Seismic Provisions.

[l] Refers to Section in FEMA 302.

[m] Refers to Section in Part II of AISC Seismic Provisions.

[n] Refers to Chapter 3, paragraph 3 in TI 809-07.

As in the *m* factor tables, the columns on the left refer to the applicable condition of the component or element. The next three columns provide the appropriate modeling parameters or limit states for the various performance levels, as indicated in Figure 6-2c. The four columns on the right-hand side provide the appropriate values for the performance levels, as defined in Paragraph (1) above.

(3) Symbols and notations contained in the above tables are defined in Appendix B.

(4) Expected and lower-bound strengths. Default values for the determination of expected strength, Q_{CE}, for deformation-controlled components, and the lower-bound strength, Q_{CL}, for force-controlled components, are provided in Paragraph 6-3a(3). Specific exception to the default values is provided for some of the systems described in this chapter.

7-2. Shear Walls.

a. General.

(1) Function. Shear walls are vertical elements in the lateral-force-resisting system that transmit lateral forces from the diaphragm above to the diaphragm below, or to the foundation. Shear walls may also be bearing walls in the gravity-load system, or they may be components in a dual system framed so as to resist only lateral loads.

(2) Shear wall types. General discussions of shear walls are presented in Paragraphs 7-2b through 7-2e. Details of reinforced concrete shear walls are covered in Paragraph 7-2f, precast concrete shear walls in 7-2g, masonry shear walls in 7-2h, wood-stud shear walls in 7-2i, and steel stud shear walls in 7-2j.

(3) Revisions to ACI 318. Various revisions to Chapter 21 of ACI 318 have been approved, but have not yet been published (September 1998). Many of these revisions have been incorporated in Chapter 6 of FEMA 302 as modifications to the referenced provisions of ACI 318. The following provisions pertaining to mechanical and welded splices of reinforcement are not included in FEMA 302, but have been approved as revisions to ACI 318, and are incorporated as provisions required by this document:

(a) Delete Sections 21.2.6, 21.2.6.1, and 21.2.6.2 of ACI 318.

(b) Add the following new sections to ACI 318:

"21.2.6 - Mechanical splices

21.2.6.1 – Mechanical splices shall be classified as either Type 1 or Type 2 mechanical splices, as follows:

(1) Type 1 mechanical splices shall conform to 12.14.3.4.

(2) Type 2 mechanical splices shall conform to 12.14.3.4 and shall develop the specified tensile strength of the spliced bar.

21.2.6.2 – Type 1 mechanical splices shall not be used within a distance equal to twice the member depth from the column or beam face or from sections where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacements. Type 2 mechanical splices shall be permitted to be used at any location.

21.2.7 – Welded splices

21.2.7.1 – Welded splices in reinforcement resisting earthquake-induced forces shall conform to 12.14.3.3 and shall not be used within a distance equal to twice the member depth from the column or beam face or from sections where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacements.

21.2.7.2 – Welding of stirrups, ties, inserts, or other similar elements to longitudinal reinforcement required by design shall not be permitted."

b. Design Forces. Walls may be subjected to both vertical (gravity) and horizontal (wind or earthquake) forces. The horizontal forces are both in-plane and out-of-plane. When considered under their in-plane loads, walls are called shear walls; when considered under their out-of-plane loads, they are called normal walls. Walls will be designed to withstand all vertical loads and horizontal forces, both parallel to and normal to the flat surface, with due allowance for the effect of any eccentric loading or overturning forces generated. Any wall, whether or not intended as part of the lateral-force-resisting system, is subjected to lateral forces unless it is isolated on three sides (both ends and top), in which case it is classified as nonstructural. Any wall that is not isolated will participate in shear resistance to horizontal forces parallel to the wall, since it tends to deform under stress when the surrounding framework deforms.

c. Wall Components. Reinforced concrete and reinforced masonry shear walls are seldom simple walls. Whenever a wall has doors, windows, or other openings, the wall must be considered as an assemblage of relatively flexible components (column segments and wall piers), and relatively stiff elements (wall segments).

(1) Column segments. A column segment is a vertical member whose height exceeds three times its thickness, and whose width is less than two and one-half times its thickness. Its load is usually predominantly axial. Although it may contribute little to the lateral-force resistance of the shear wall, its rigidity must be considered. When a column is built integral with a wall, the portion of the column that projects from the face of the wall is called a pilaster. Column segments shall be designed according to ACI 318 for concrete and ACI 530 for masonry.

(2) Wall piers. A wall pier is a segment of a wall whose horizontal length is between two and one-half and six times its thickness, and whose clear height is at least two times its horizontal length.

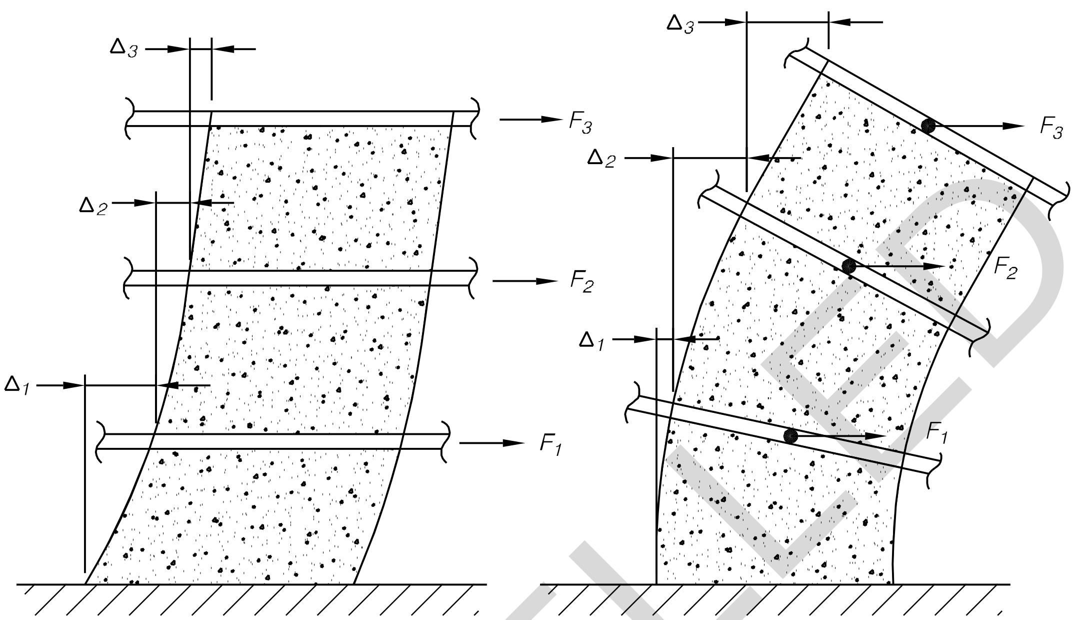
(3) Wall segments. Wall segments are components that are longer than wall piers. They are the primary lateral-load-resisting components in the shear wall.

d. In-Plane Effects. Horizontal forces at any floor or roof level are generally transferred to the ground (foundation) by using the strength and rigidity of shear walls (and partitions). A shear wall may be considered analogous to a cantilever plate girder standing on end in a vertical plane, where the wall performs the function of a plate girder web, the pilasters or floor diaphragms function as web stiffeners, and the integral reinforcement of the vertical boundaries functions as flanges. Axial, flexural, and shear forces must be considered in the design of shear walls. The tensile forces on shear wall elements resulting from the combination of seismic uplift forces and seismic overturning moments must be resisted by anchorage into the foundation medium unless the uplift can be counteracted by gravity loads (e.g., 0.90 of dead load) mobilized from neighboring elements. A shear wall may be constructed of materials such as concrete, wood, unit masonry, or metal in various forms. Design procedures for such materials as cast-in-place reinforced concrete and reinforced unit masonry are well known, and present no problem to the designer once the loading and reaction system is determined. Other materials frequently used to support vertical loads from floors and roofs have well- established vertical- load- carrying characteristics, but have required tests to demonstrate their ability to resist lateral forces. Various types of wood sheathing and metal siding fall into this category. Where a shear wall is made up of units such as plywood, gypsum, wallboard, tilt-up concrete units, or metal panel units, its characteristics are, to a large degree, dependent upon the attachments of one unit to another, and to the supporting members.

(1) Rigidity analysis. For a building with rigid diaphragms, there is a torsional moment, and a rigidity analysis is required. It is necessary to make a logical and consistent distribution of story shears to each wall. An exact determination of wall rigidities is very difficult, but is not necessary, because only *relative* rigidities are needed. Approximate methods in which the deflections of portions of walls are combined usually are adequate.

(a) Wall deflections. The rigidity of a wall is usually defined as the force required to cause a unit deflection. Rigidity is expressed in kips per inch. The deflection of a concrete shear wall is the sum of the shear and flexural deflections (see Figure 7-1). In the case of a solid wall with no openings, the computations of deflection are quite simple; however, where the shear wall has openings, as for doors and windows, the computations for deflection and rigidity are much more complex. An exact analysis, considering angular rotation of elements, rib shortening, etc., is very time-consuming. For this reason, several short-cut approximate methods have been developed. These do not always give consistent or satisfactory results. A conservative approach and judgment must be used.

(b) Deflection charts. The calculation of deflections is facilitated by the use of the deflection charts. See Figure 7-4 for fixed-ended corner and rectangular piers. Curves 5 and 6 are for cantilever corner and rectangular piers. The corner pier curves are for the special case where the moment of inertia,



(a) Shear Deformation **(b) Flexural Deformation**

Figure 7-1 Shear wall deformation.

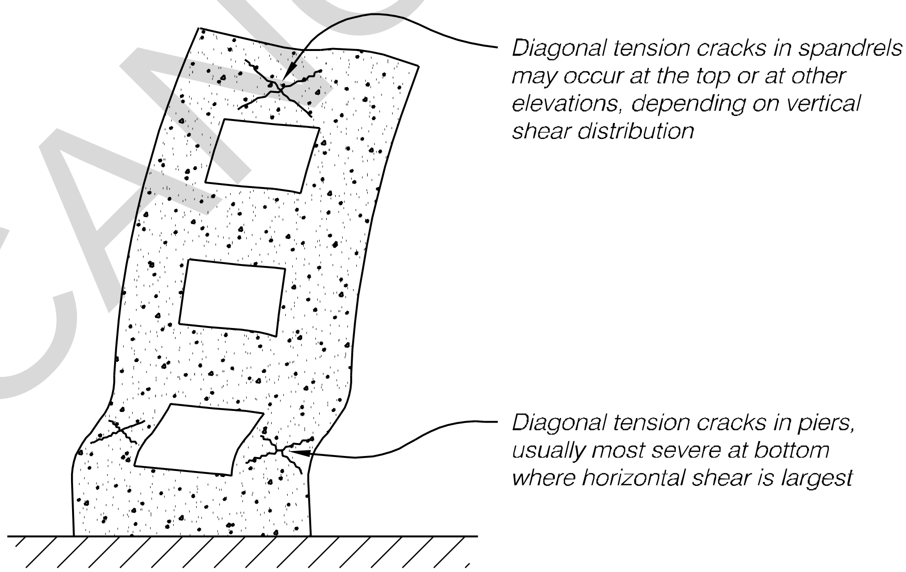


Figure 7-2 Deformation of shear wall with openings.

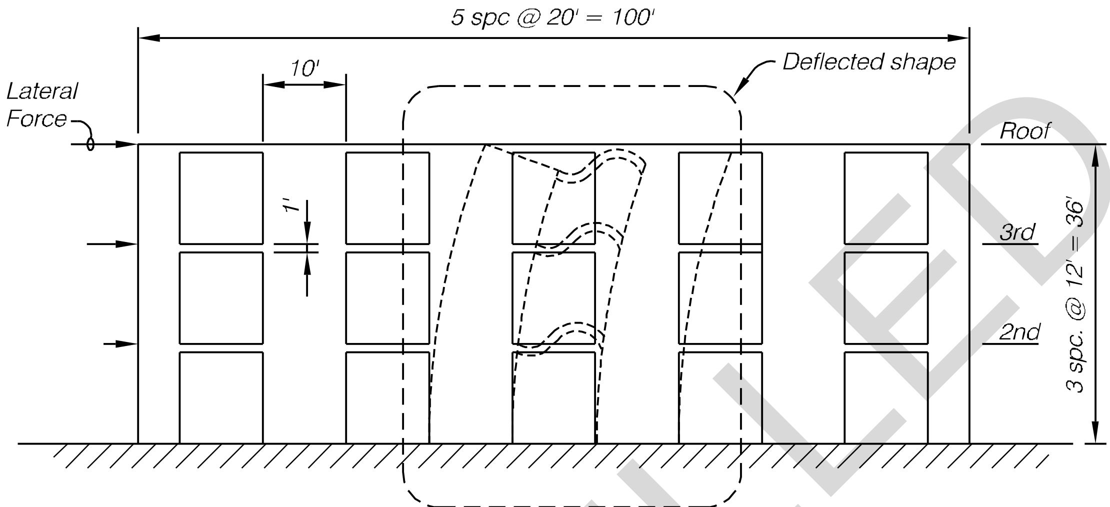
I, of the corner pier is 1.5 times that of a rectangular pier; for other *I*-values, the bending portion of the deflection would be proportional. The deflections shown on the charts are for a horizontal load, *P*, of 1,000,000 pounds. The deflections shown on the charts are reasonably accurate. The formulas written on the curves can be used to check the results; however, the charts will give no better results than the assumptions made in the shear wall analysis. For instance, the point of contraflexure of a vertical pier may not be in the center of the pier height. In some cases, the point of contraflexure may be selected by judgment and an interpolation made between the cantilever and fixed conditions.

(c) Foundation effects. The rotation at the foundation can greatly influence the overall rigidity of a shear wall because of the very rigid nature of the shear wall itself; however, the rotational influence on relative rigidities of walls for purposes of horizontal force distribution may not be as significant. Considering the complexities of soil behavior, a quantitative evaluation of the foundation rotation is generally not practical, but a qualitative evaluation will be provided.

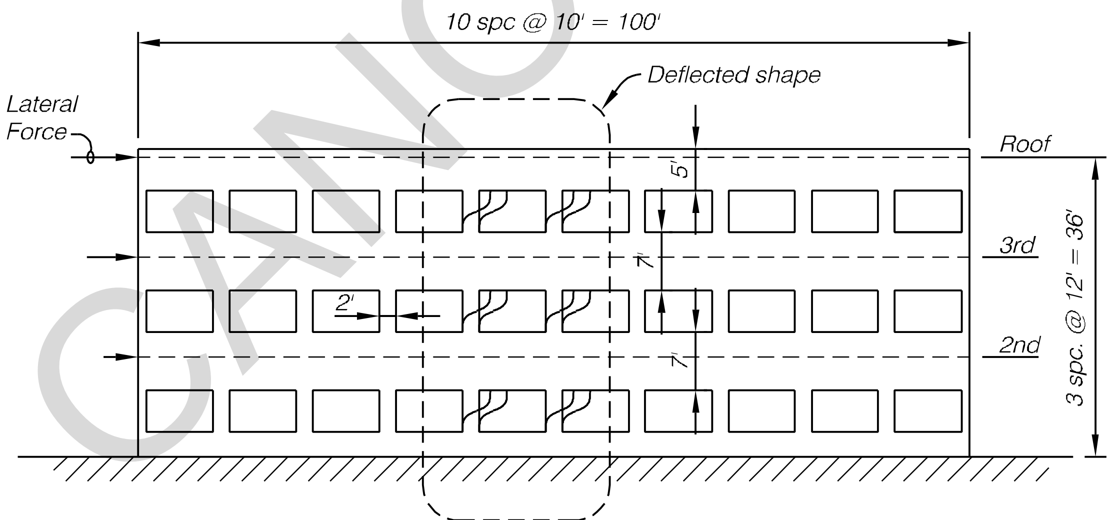
(d) Framework effects. The relative rigidity of concrete or unit masonry walls with nominal openings is usually much greater than that of the building framework; therefore, the walls tend to resist essentially all or a major part of the lateral force.

(2) Effect of openings. The effect of openings on the ability of shear walls to resist lateral forces must be considered. If openings are very small, their effect on the overall state of stress in a shear wall is minor. Large openings have a more pronounced effect, and if large enough, result in a system in which typical frame action predominates. Openings commonly occur in regularly spaced vertical rows throughout the height of the wall, and the connection between the wall sections is provided by either connecting beams (or spandrels) which form a part of the wall, or floor slabs, or a combination of both. If the openings do not line up vertically and/or horizontally, the complexity of the analysis is greatly increased. In most cases, a rigorous analysis of a wall with openings is not required. "Strut and Tie" procedures that depict shear walls as consisting of compression struts and tension ties are useful tools for the evaluation of shear walls with openings (see Paulay and Priestley, 1992). In the design of a wall with openings, the deformations must be visualized in order to establish some approximate method for analyzing the stress distribution to the wall. Figures 7-3 and 7-4 give some visual descriptions of such deformations. The major points that must be considered are the lengthening and shortening of the extreme sides (boundaries) due to deep beam action, the stress concentration at the corner junctions of the horizontal and vertical components between openings, and the shear and diagonal tension in both the horizontal and vertical components.

(a) Relative rigidities of piers and spandrels. The ease of methods of analysis for walls with openings is greatly dependent on the relative rigidities of the piers and the spandrels, as well as the general geometry of the building. Figure 7-3 shows two extreme examples of relative rigidities of exterior walls of a building. In Figure 7-3A, the piers are very rigid and the spandrels are very flexible. Assuming a rigid base, the shear walls act as vertical



A. RIGID PIERS AND FLEXIBLE SPANDRELS



B. FLEXIBLE PIERS AND RIGID SPANDRELS

Figure 7-3 Relative rigidities of piers and spandrels.

1 foot = 0.3 meter

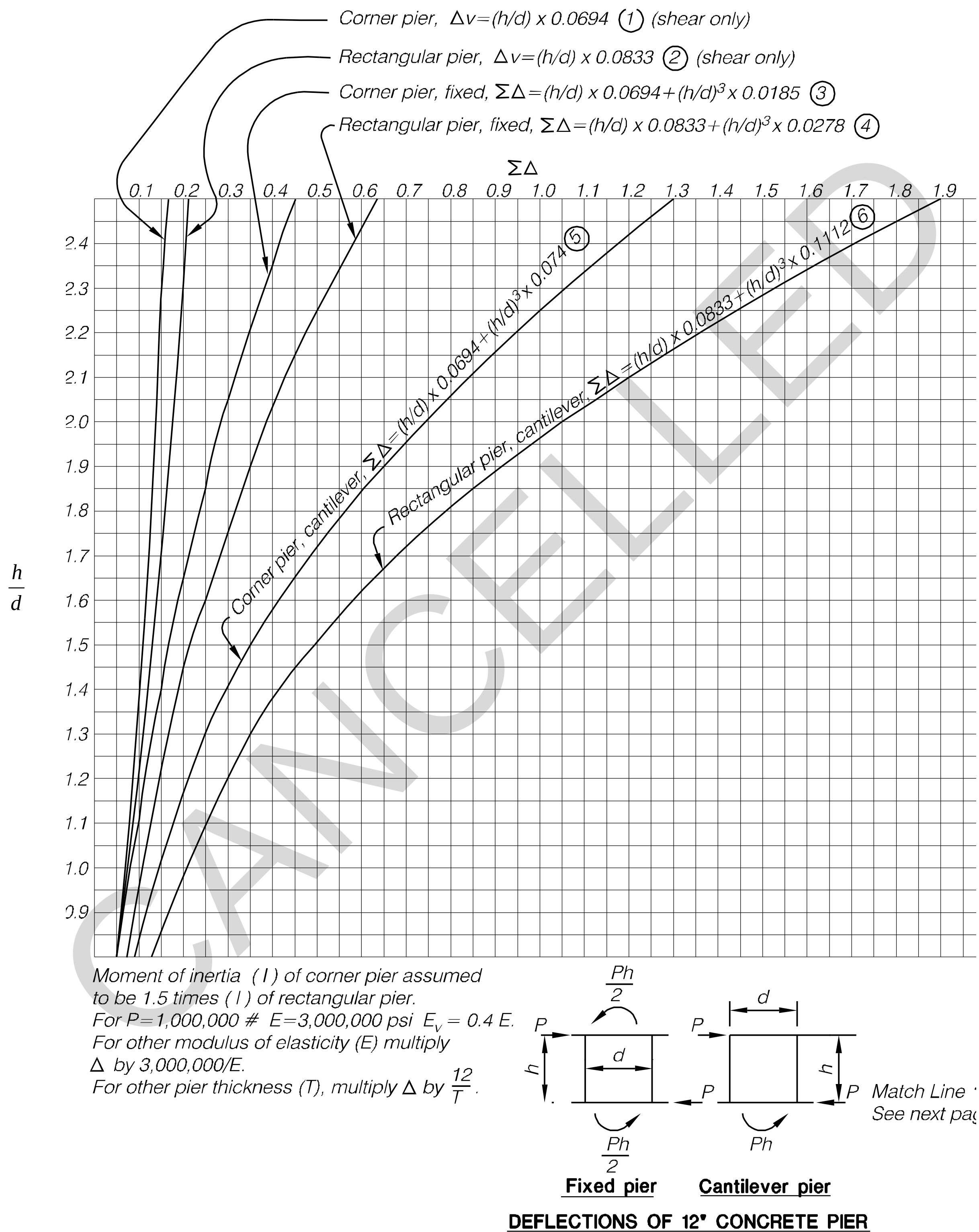
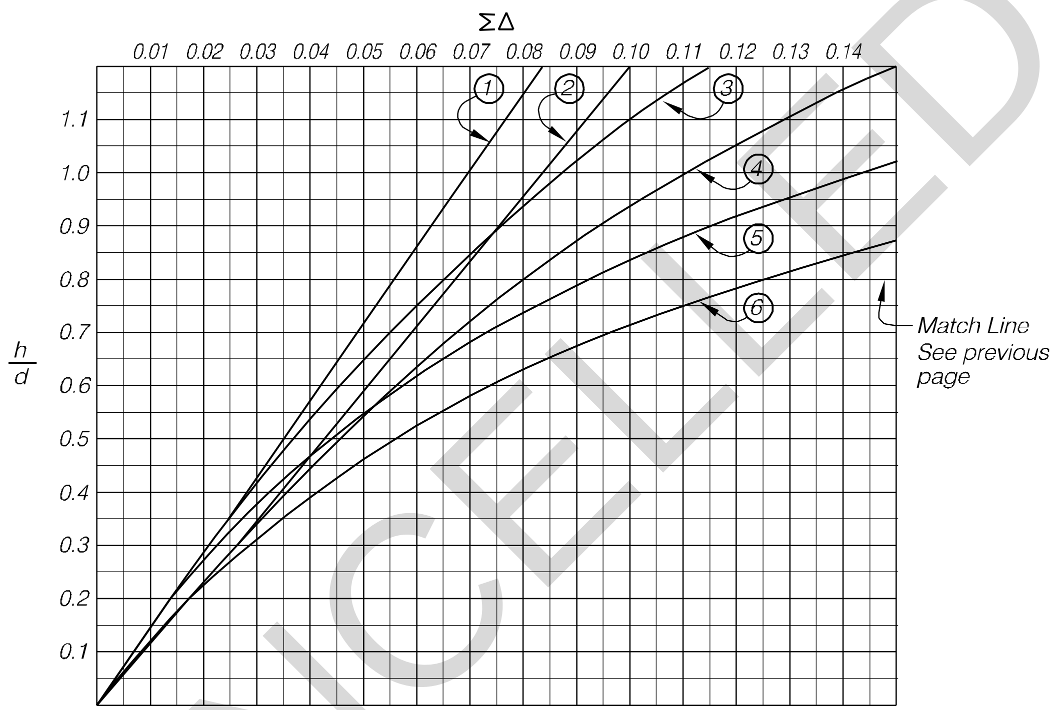


Figure 7-4 Design curves for masonry and concrete shear walls.



Metric equivalents:

Curves based on:

T = 305mm

P = 4448 MN

E = 20,685 Mpa

For other modulus of elasticity, (E), multiply Δ by 20,685/E.

For other pier thickness, (T), multiply Δ by 305/T.

Figure 7-4 Design curves - continued.

cantilevers. When a lateral force is applied, the spandrels act as struts that flexurally deform to be compatible with the deformation of the cantilever piers. It is relatively simple to determine the forces on the cantilever piers by ignoring the deformation characteristics of the spandrels. The spandrels are then designed to be compatible with the pier deformations. In Figure 7-3B, the piers are relatively flexible compared with the spandrels. The spandrels are assumed to be infinitely rigid, and the piers are analyzed as fixed-ended columns. The spandrels are then designed for the forces induced by the columns. The overall wall system is also analyzed for overturning forces that induce axial forces into the columns. The calculations of relative rigidities for both cases shown in Figure 7-3 can be aided by the charts in Figure 7-4. For cases of relative spandrel and pier rigidities other than those shown, the analysis and design become more complex.

(3) Methods of analysis. Approximate methods for analyzing walls with openings are generally acceptable. For the extreme cases shown in Figure 7-3, the procedure is straightforward. For other cases, a variety of assumptions may be used to determine the most critical loads on various elements, thus resulting in a conservative design. (Note: In some cases, a few additional reinforcing bars, at little additional cost, can greatly increase the strength of shear walls with openings.) When, however, the reinforcement requirements or the resulting stresses of this approach appear excessively large, the strut and tie procedure indicated in paragraph 7-2d (2) or a more rigorous analysis may be justified.

(4) Coupled shear walls. When two or more shear walls in one plane are linked together by coupling beams, interactive forces are transmitted to the shear walls by the beams. In addition to these axial forces, the beams develop moments and shears that contribute to the walls resisting overturning. The magnitude of the resisting beam bending moments and vertical shears is dependent on the relative stiffnesses of the walls and the coupling beams. It should be noted that the foundation itself functions as a coupling beam. Accurate determination of the resisting forces can be complex; therefore, approximate methods are generally used. One method may be used for calculating the axial forces, and another method may be used for calculating bending moments and shears to ensure that the structural elements are not underdesigned.

(5) Construction joints and dowels. The contact faces of shear wall construction joints have exhibited slippage and related drift damage in past earthquakes. Consideration must be given to the location and details of construction joints, which must be clean and roughened. Shear friction reinforcement may be utilized in accordance with ACI 318. For this procedure, a coefficient of friction of 0.6 is suggested for seismic effects.

e. Out-of-Plane Effects.

(1) Lateral forces. Walls and partitions must safely resist horizontal seismic forces normal to their flat surface (Figure 7-5, part *a*). At the same time, they must resist moments and shears induced by relative deflections of the diaphragms above and below (Figure 7-5, part *b*). The normal force on a wall is a function of its weight. Equations for the determination of the force are provided in Paragraph 10-1b(1); however, wind forces, other forces, or interstory drift will frequently govern the design.

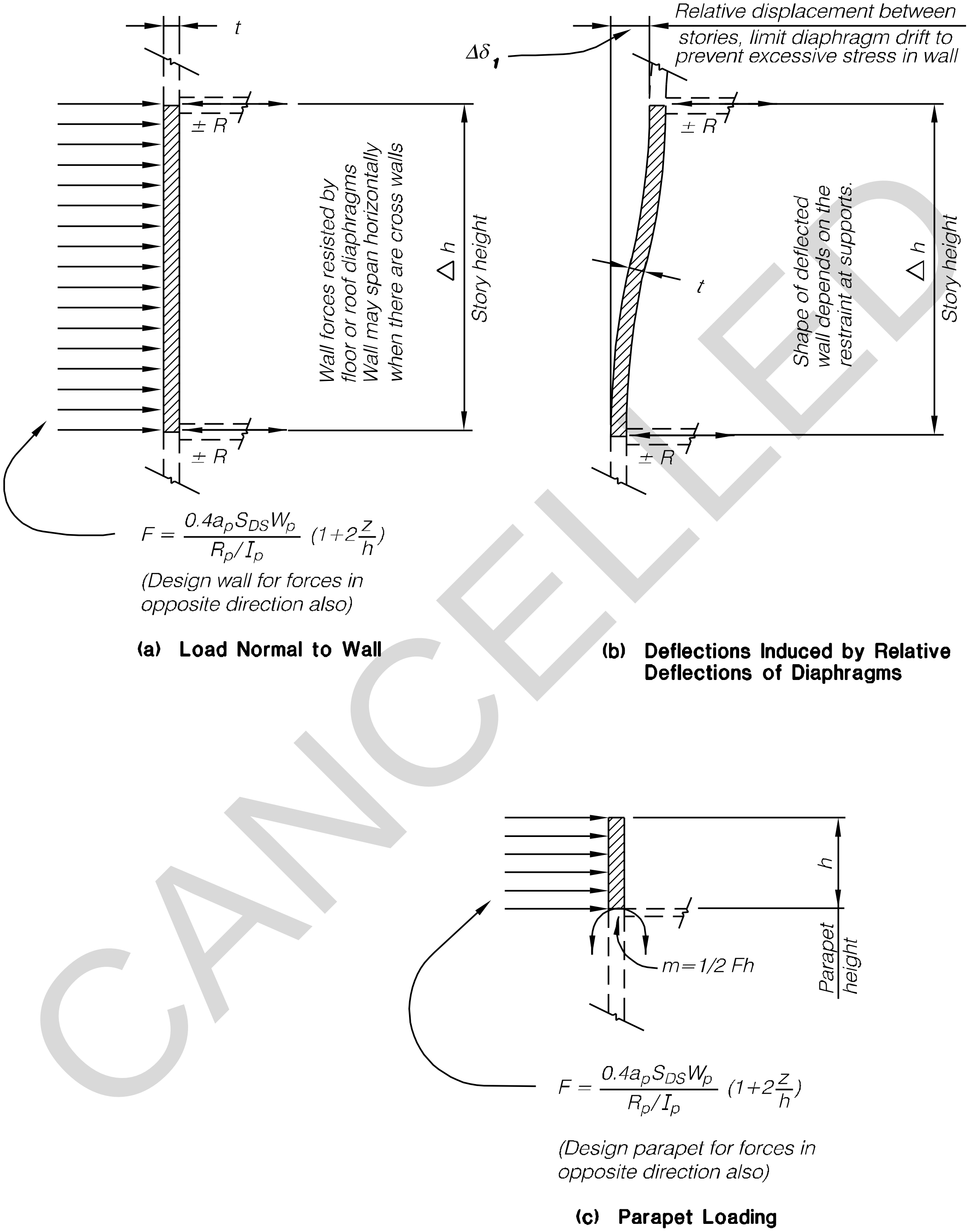


Figure 7-5 Out-of-plane effects.

(For cantilevered walls, see Paragraph 3 below.) The design force will be applied to the wall in both inward and outward directions.

(2) Wall behavior. Walls distribute normal forces vertically to the horizontal resisting elements above or below. They may also distribute normal forces to frames, or other walls or frames. A wall may be either continuous or discontinuous across its supports.

(3) Cantilevered walls. Where walls, such as parapets, are cantilevered, the anchorage for reaction and cantilever moment is required to be fully developed (Figure 7-5, part *c*). Where a parapet wall is anchored to a concrete roof slab and is not a continuation of a wall below, the roof slab will be designed for the cantilever moment. Where the parapet is a continuation of a wall below, the cantilever moment will be divided between the concrete slab and the wall below in proportion to their relative stiffnesses. Where the parapet is an extension of a wall below and is anchored to a roof or floor of wood, metal deck, or other similar materials, the moment at the base of the parapet will be developed into the wall below. In this case, the anchorage force to the roof will be determined by the usual methods of analysis, assuming a pinned condition for the connection of the roof to the wall.

(4) Connections. Walls will be anchored to the structural frame or diaphragm by dowels, anchor bolts, or other approved methods to withstand the design forces, but in no case less than 200 pounds per linear foot. Dovetail anchors are inadequate for this purpose. Nonstructural partitions will be isolated from exterior walls and shear partitions so as to prevent buttress action, which would restrict shear walls from deflecting with the diaphragms. Isolated partitions will be braced to overhead construction or anchored to other isolated cross walls to ensure lateral stability under out-of-plane loading.

f. Cast-in-Place Concrete Shear Walls.

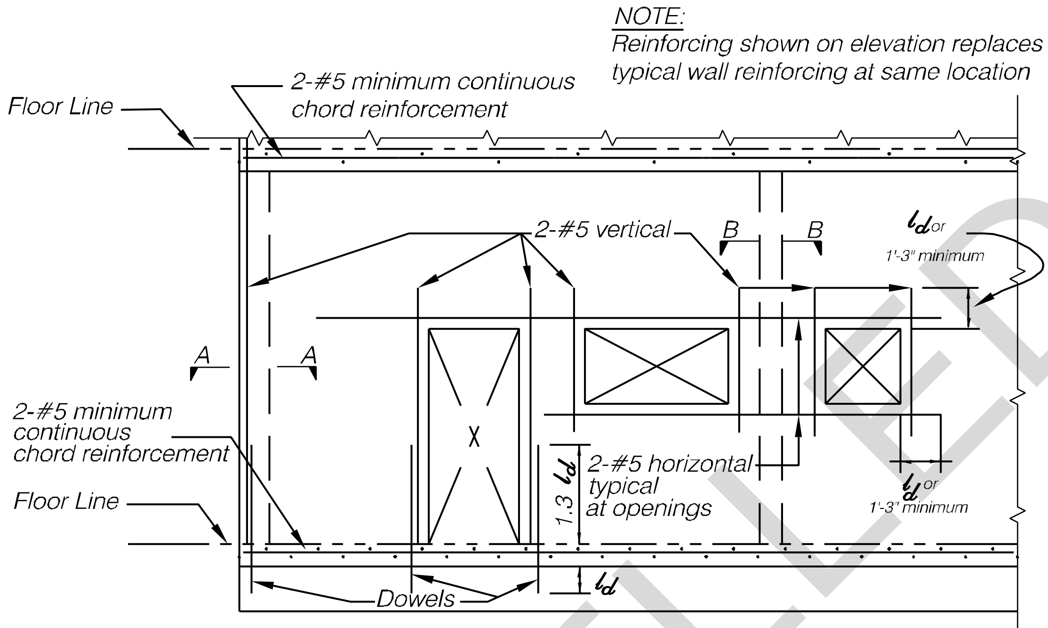
(1) General requirements. Reinforced concrete shear walls shall comply with the following provisions.

(a) Seismic Design Categories A and B. Shear walls may be of any type permitted by ACI 318 and FEMA 302.

(b) Seismic Design Category C. Shear walls may be detailed plain concrete shear walls, ordinary reinforced concrete shear walls, or special reinforced concrete shear walls as prescribed in Chapter 9 of FEMA 302.

(c) Seismic Design Categories D, E, and F. Shear walls shall be special reinforced concrete shear walls in accordance with Section 9.3.2.4 of FEMA 302.

(2) General design criteria. The criteria used to design reinforced concrete shear walls will be ACI 318 requirements for “structural walls,” as modified by the provisions given in Chapter 9 of FEMA 302. For tilt-up and other precast concrete shear walls, refer to Paragraph 7-2g. For details of reinforcement, see Figure 7-6 and 7-7.



WALL ELEVATION

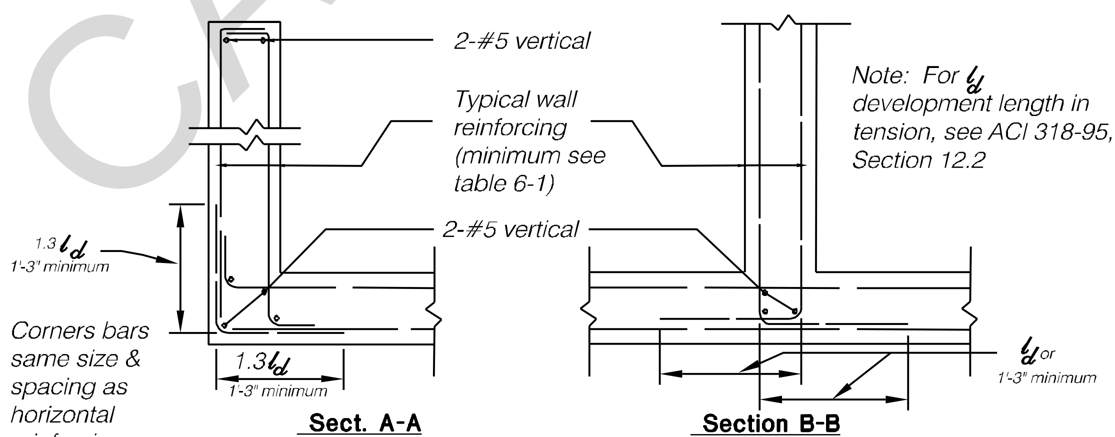
Without special boundary zone

NOTE:
All reinforcement shown is typical unless additional or larger bars are required by seismic design.

1 inch = 25mm
#4 bar ≈ 10M bar
#5 bar = 15M bar

Table	
MINIMUM CONCRETE WALL REINFORCING	
Wall Thickness	Vertical and horizontal Reinforcing *
8"	#4 at 18" o.c. both faces
9"	#4 at 18" o.c. both faces
10"	#4 at 16" o.c. both faces
12"	#4 at 13" o.c. both faces

* Max. spacing of bars = d/3 where d is dimension of the wall pier parallel to shear force



Sect. A-A
Typical wall corner

Section B-B
Typical wall intersection

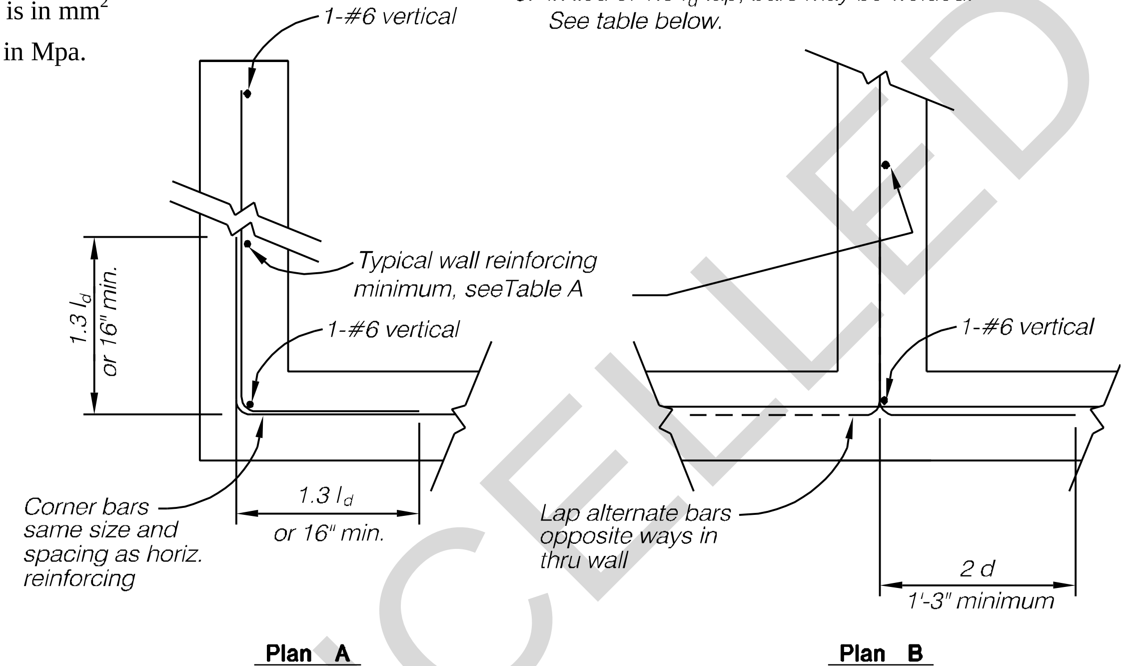
Figure 7-6 Minimum concrete shear wall reinforcement (two curtains)

1 inch = 25mm
#4 bar ≈ 10M bar
#5 bar = 15M bar
#6 bar ≈ 20M bar
#8 bar ≈ 25M bar

$$2A_{cv}\sqrt{f_c^{'}} = \frac{A_{cv}\sqrt{f_c^{'}}}{6}$$

where A_{cv} is in mm^2
and $f_c^{'}$ is in Mpa.

Note: 1. *Bars at jams, heads, and sills of openings will be 1 - #4 placed in same manner as indicated for two curtain reinf. or Figure 7-6.*
2. *All reinforcement shown is typical unless additional or larger bars are required by seismic design.*
3. *In lieu of 1.3 l_d lap, bars may be welded. See table below.*



TYPICAL WALL CORNER AND JAMS

TYPICAL WALL INTERSECTION

Table A

MINIMUM CONCRETE WALL REINFORCEMENT	
Wall Thickness	*Vertical and horizontal Reinforcing **
5"	*#4 at 16" o.c. in center*
6"	*#4 at 13" o.c. in center*
7"	*#4 at 11" o.c. in center*
8"	*#4 at 10" o.c. in center*

** Spacing of bars not to exceed d/3 where d is dimension of the wall pier parallel to shear force*

Table B

MINIMUM LENGTH OF STANDARD A.W.S. FLARE GROOVE WELDS TO DEVELOP LAPPED REINFORCING BARS	
Bar	*Welded length (each side)*
3	*2"*
4	*2 1/2"*
5	*3"*
6	*3 1/2"*
7	*4"*

Note: Special reinforced concrete shear walls shall have two curtains of reinforcement when the factored shear force exceeds $2\,A_{cv}\sqrt{f_c'}$

Figure 7-7 Minimum concrete shear wall reinforcement (one curtain).

(3) Boundary zone requirements for special reinforced concrete shear walls.

(a) Boundary zones are required for special reinforced concrete shear walls, except where the following conditions exist.

1. $P_u \leq 0.10\, A_g\, f_c^{'}$ for geometrically symmetrical wall sections $P_u \leq 0.05\, A_g\, f_c^{'}$ for geometrically unsymmetrical wall sections, and either

2. $M_u/V_u\, l_w \leq 1.0$

or

3. $V_u \leq 3A_{cv}\sqrt{f_c^{'}}$ and $M_u/V_u\, l_w \leq 3.0$

The metric equivalent is $v_u \leq A_{cv}\sqrt{f_e^{'}}\,/\,4$ where:

P_u = 1.2D+0.5L + E, kips (kN).

A_g = gross area of the wall, in^2 (mm^2).

M_u = required moment strength, kip-in (kN-m).

V_u = required shear strength, kips (kN).

l_w = horizontal length of wall, in (mm).

A_{cv} = net area of wall bounded by the web thickness and length of section in^2 (mm^2).

(b) Design and detailing of boundary zones shall be in accordance with the provisions of ACI 318, as modified by Section 9.1.1.13 of FEMA 302. Section 9.1.1.13 in FEMA 302 modifies the boundary zone provisions in Section 21.6.6 of ACI 318 with more explicit statements regarding when boundary zones are required; the design of boundary zones; and when the boundary zone is no longer required.

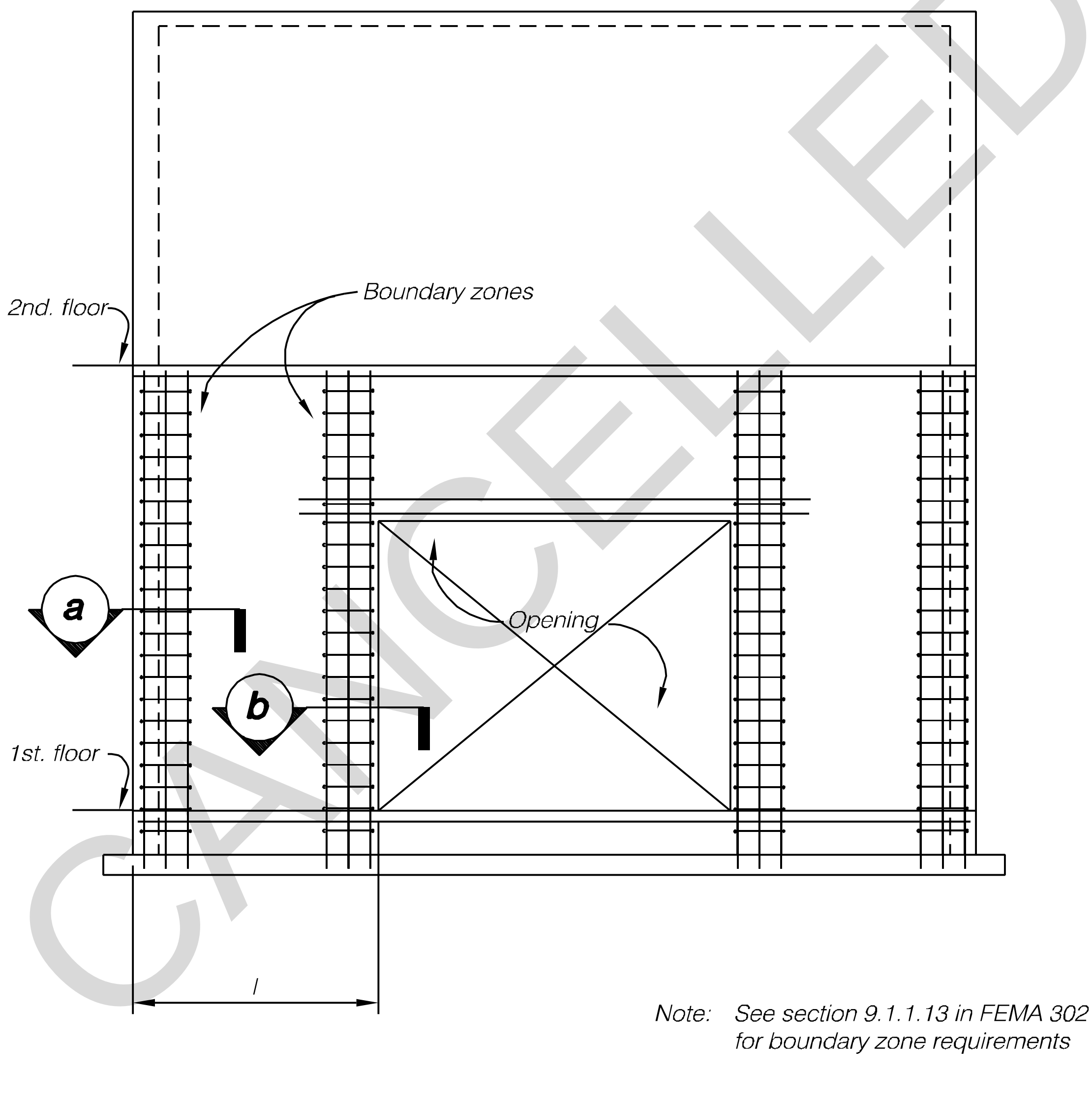
(c) Wall boundary elements may also occur in the building frame system and the dual system where the usual configuration is to place the shear walls within the bays between the frame columns. See Figure 7-8 for details of shear walls with boundary elements. Note that the vertical reinforcement in the boundary zones in Figure 7-8 is extended to be developed above and below the prescribed limit of the boundary zones.

(d) When boundaries are not required, special reinforced concrete shear walls shall comply with Section 21.6 of ACI 318, as modified by the applicable provisions of Section 9.1.1.13 of FEMA 302.

(4) Acceptance criteria.

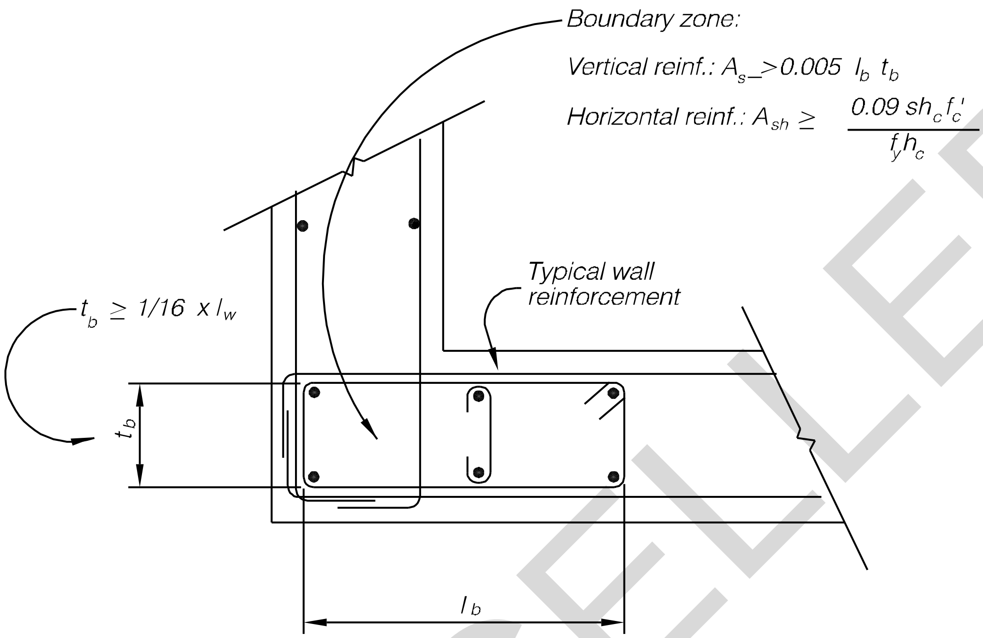
(a) Response modification factors, *R*, for Performance Objective 1A are provided in Table 7-1.

(b) Modification factors, *m*, for enhanced performance objectives are provided in Table 7-2 for components controlled by flexure, and in Table 7-3 for components controlled by shear.

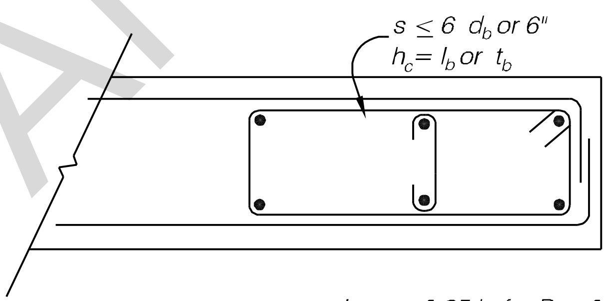


SHEAR WALL ELEVATION

Figure 7-8 Boundary zones in a special reinforced concrete shear wall



Section A



l_b $= 0.25\ l_w$ for $P_u = 0.35\ P_o$
$=0.15\ l_w$ for $P_u = 0.15\ P_o$
= 18 in (460 mm) min.

Section B

Figure 7-8 cont. Boundary zones in a special reinforced concrete shear wall

Conditions			IO	SE	LS	CP	LS	CP
			***m* factors**					
			Component Type					
			Primary				**Secondary**	
			Performance Level					
i. Shear walls and wall segments								
$\frac{(A_s - A'_s)f_y + P}{t_w l_w f'_c}$	$\frac{\text{Shear}}{t_w l_w \sqrt{f'_c}}$	Confined Boundary[1]						
≤ 0.1	≤ 3	Yes	2	3	4	6	6	8
≤ 0.1	≥ 6	Yes	2	2.5	3	4	4	6
≥ 0.25	≤ 3	Yes	1.5	2.3	3	4	4	6
≥ 0.25	≥ 6	Yes	1	1.5	2	2.5	2.5	4
≤ 0.1	≤ 3	No	2	2.3	2.5	4	4	6
≤ 0.1	≥ 6	No	1.5	1.8	2	2.5	2.5	4
≥ 0.25	≤ 3	No	1	1.3	1.5	2	2	3
≥ 0.25	≥ 6	No	1	1	1	1.5	1.5	2
ii. Columns supporting discontinuous shear walls								
Transverse reinforcement[2]								
Conforming			1	1.3	1.5	2	n.a.	n.a.
Nonconforming			1	1	1	1	n.a.	n.a.
iii. Shear wall coupling beams								
Longitudinal reinforcement and transverse reinforcement[3]		$\frac{\text{Shear}}{t_w l_w \sqrt{f'_c}}$						
Conventional longitudinal reinforcement with conforming transverse reinforcement		≤ 3	2	3	4	6	6	9
		≥ 6	1.5	2.3	3	4	4	7
Conventional longitudinal reinforcement with nonconforming transverse reinforcement		≤ 3	1.5	2.5	3.5	5	5	8
		≥ 6	1.2	1.5	1.8	2.5	2.5	4
Diagonal reinforcement		n.a.	2	3.5	5	7	7	10

1. Requirements for a confined boundary are the same as those given in *ACI 318-95*.
2. Requirements for conforming transverse reinforcement are: (a) closed stirrups over the entire length of the column at a spacing ≤ $d/2$, and (b) strength of closed stirrups V_s ≥ required shear strength of column.
3. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the beam at a spacing ≤ $d/3$, and (b) strength of closed stirrups V_s ≥ 3/4 of required shear strength of beam.

Table 7-2: Numeric Acceptance Criteria for Linear Procedures—Members Controlled by Flexure

Conditions		m factors					
		Component Type					
		Primary				Secondary	
		Performance Level					
		IO	SE	LS	CP	LS	CP
i. Shear walls and wall segments							
All shear walls and wall segments[1]		2	2	2	3	2	3
ii. Shear wall coupling beams							
Longitudinal reinforcement and transverse reinforcement[2]	$\frac{\text{Shear}}{t_w l_w \sqrt{f_c'}}$						
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	1.5	2.3	3	4	4	6
	≥ 6	1.2	1.6	2	2.5	2.5	3.5
Conventional longitudinal reinforcement with nonconforming transverse reinforcement	≤ 3	1.5	2	2.5	3	3	4
	≥ 6	1	1.1	1.2	1.5	1.5	2.5

1. For shear walls and wall segments where inelastic behavior is governed by shear, the axial load on the member must be $\leq 0.15\, A_g f_c'$, the longitudinal reinforcement must be symmetrical, and the maximum shear stress must be $\leq 6 \sqrt{f_c'}$, otherwise the shear shall be considered to be a force-controlled action.
2. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the beam at a spacing $\leq d/3$, and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of beam.

Table 7-3: Numeric Acceptance Criteria for Linear Procedures—Members Controlled by Shear

(c) Modeling parameters and numerical acceptance criteria for nonlinear procedures are provided in Table 7-4 for members controlled by flexure, and in Table 7-5 for members controlled by shear.

(d) Expected strength, $Q_{CE,}$ shall be determined using $1.25f_y$ for the contribution attributed to the reinforcement in flexure and 1.0 f_y in shear.

(e) Lower-bound strength for force-controlled actions (e.g., length of splices, dowels, or embedments) shall be equal to the values provided in ACI 318 without a strength reduction factor, *N*.

g. Tilt-up and Other Precast Concrete Shear Walls.

(1) Analysis. Precast concrete shear walls shall be designed in accordance with Section 9.1.1.5 of FEMA 302, and shall emulate the behavior of monolithic reinforced concrete construction. Where tilt-up or precast concrete walls are used as shear walls, the analysis is similar to that for walls of cast-in-place concrete; however, in this case, the boundary conditions become critical, and the shears between precast and cast-in-place elements must be analyzed. Shears between two precast elements or between a precast element and a cast-in-place element may be developed by shear keys, dowels, or welded inserts. The contact joint itself is a cold joint, and will be given no shear or tension value.

(2) Joints. Precast concrete elements tend to be structurally separate, one element from another. In the case of precast wall construction, for instance, these could be a series of concrete elements tied together at top and bottom, but structurally separated from each other by vertical joints. Since all elements in a line are tied together at the top, they must have equal horizontal deflections; therefore, a horizontal force parallel to the line of units will be resisted by the individual elements in proportion to relative rigidities. Such elements may not have equal rigidities, since some may contain large openings or may be of different height-width ratios. Some elements may deflect primarily in shear, and others primarily in flexure. Where significant dissimilar deflections are found, the building elements tying the individual units together must be analyzed to determine their ability to resist or accept such deformations, including angular rotation, without losing their ability to function as ties or diaphragm chords or footings. Mechanical keys or sleeved dowels may be used to assist in eliminating differential movement of adjacent precast panels separated by control joints where appearance and weather-tightness are otherwise satisfactorily provided.

(3) Connectors for shear walls. Past earthquakes have shown that the performance of weld plates or other nonductile connectors has often been poor, and in many cases they have resulted in failures. These connectors have been weak links in the shear wall connection. It is important that the load-bearing shear walls be more stringently or conservatively designed, since any connector failure during an earthquake may result in progressive

Conditions			Plastic Hinge Rotation (radians) a	b	Residual Strength Ratio c	Acceptable Plastic Hinge Rotation (radians) — Component Type — Primary — Performance Level IO	SE	LS	CP	Secondary LS	CP
i. Shear walls and wall segments											
$\frac{(A_s - A'_s)f_y + P}{t_w l_w f'_c}$	$\frac{\text{Shear}}{t_w l_w \sqrt{f'_c}}$	Confined Boundary[1]									
≤ 0.1	≤3	Yes	0.015	0.020	0.75	0.005	0.008	0.010	0.015	0.015	0.020
≤ 0.1	≥ 6	Yes	0.010	0.015	0.40	0.004	0.006	0.008	0.010	0.010	0.015
≥ 0.25	≤ 3	Yes	0.009	0.012	0.60	0.003	0.005	0.006	0.009	0.009	0.012
≥ 0.25	≥ 6	Yes	0.005	0.010	0.30	0.001	0.002	0.003	0.005	0.005	0.010
≤ 0.1	≤ 3	No	0.008	0.015	0.60	0.002	0.003	0.004	0.008	0.008	0.015
≤ 0.1	≥ 6	No	0.006	0.010	0.30	0.002	0.003	0.004	0.006	0.006	0.010
≥ 0.25	≤ 3	No	0.003	0.005	0.25	0.001	0.002	0.002	0.003	0.003	0.005
≥ 0.25	≥ 6	No	0.002	0.004	0.20	0.001	0.001	0.001	0.002	0.002	0.004
ii. Columns supporting discontinuous shear walls											
Transverse reinforcement[2]											
Conforming			0.010	0.015	0.20	0.003	0.005	0.007	0.010	n.a.	n.a.
Nonconforming			0.0	0.0	0.0	0.0	0.0	0.0	0.0	n.a.	n.a.
			Chord Rotation (radians) d	e							
iii. Shear wall coupling beams											
Longitudinal reinforcement and transverse reinforcement[3]		$\frac{\text{Shear}}{t_w l_w \sqrt{f'_c}}$									
Conventional longitudinal reinforcement with conforming transverse reinforcement		≤ 3	0.025	0.040	0.75	0.006	0.011	0.015	0.025	0.025	0.040
		≥ 6	0.015	0.030	0.50	0.005	0.008	0.010	0.015	0.015	0.030
Conventional longitudinal reinforcement with nonconforming transverse reinforcement		≤ 3	0.020	0.035	0.50	0.006	0.009	0.012	0.020	0.020	0.035
		≥ 6	0.010	0.025	0.25	0.005	0.007	0.008	0.010	0.010	0.025
Diagonal reinforcement		n.a.	0.030	0.050	0.80	0.006	0.012	0.018	0.030	0.030	0.050

1. Requirements for a confined boundary are the same as those given in *ACI 318-95*.
2. Requirements for conforming transverse reinforcement are: (a) closed stirrups over the entire length of the column at a spacing ≤ $d/2$, and (b) strength of closed stirrups V_s ≥ required shear strength of column.
3. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the beam at a spacing ≤ $d/3$, and (b) strength of closed stirrups V_s ≥ 3/4 of required shear strength of beam.

Table 7-4: Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Members Controlled by Flexure

Conditions		Drift Ratio (%), or Chord Rotation (radians)[1]		Residual Strength Ratio	Acceptable Drift (%) or Chord Rotation (radians)[1]					
					Component Type					
					Primary				Secondary	
					Performance Level					
		d	*e*	*c*	IO	SE	LS	CP	LS	CP
i. Shear walls and wall segments										
All shear walls and wall segments[2]		0.75	2.0	0.40	0.40	0.50	0.60	0.75	0.75	1.5
ii. Shear wall coupling beams										
Longitudinal reinforcement and transverse reinforcement[3]	$\frac{\text{Shear}}{t_w l_w \sqrt{f_c'}}$									
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	0.018	0.030	0.60	0.006	0.009	0.012	0.015	0.015	0.024
	≥ 6	0.012	0.020	0.30	0.004	0.006	0.008	0.010	0.010	0.016
Conventional longitudinal reinforcement with nonconforming transverse reinforcement	≤ 3	0.012	0.025	0.40	0.006	0.007	0.008	0.010	0.010	0.020
	≥ 6	0.008	0.014	0.20	0.004	0.005	0.006	0.007	0.007	0.012

1. For shear walls and wall segments, use drift; for coupling beams, use chord rotation; refer to Figures 6-4 and 6-5.
2. For shear walls and wall segments where inelastic behavior is governed by shear, the axial load on the member must be ≤ 0.15 $A_g f_c'$; otherwise, the member must be treated as a force-controlled component.
3. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the beam at a spacing ≤ $d/3$, and (b) strength of closed stirrups V_s ≥ 3/4 of required shear strength of beam.

Table 7-5: Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Members Controlled by Shear

failure to collapse. All connectors for load-bearing and non-load-bearing walls will therefore be designed in accordance with ACI 318, as modified by Section 9.1.1 of FEMA 302. The shear force will be uniformly distributed throughout the height or length of the shear wall with reasonably spaced connectors (maximum spacing 4 feet), rather than with a few that will have localized concentration of stresses. Detailed calculations will be made, including the localized effects in concrete walls attributed from these connectors. Sufficient details of connectors and embedded anchorage will be provided to preclude construction deficiency.

(4) Typical details. Refer to Figure 7-9 for details.

(5) Acceptance criteria. FEMA 302 requires that connections for precast concrete walls shall be designed to be stronger than the adjacent precast panels. The lateral-load response behavior is therefore comparable to that for monolithic shear walls, and the acceptance criteria of Paragraph 7-2f(3) will be applicable.

h. Masonry Shear Walls.

(1) General design criteria. This section prescribes the criteria for the structural design of shear walls of unit masonry construction. The basic reference documents are FEMA 302 and ACI 530.

(2) Unreinforced or plain masonry bearing walls or shear walls, where permitted, shall be used only for buildings in Seismic Design Category A or B. Design shall be in accordance with Section 11.3.3 of FEMA 302.

(3) Masonry construction prescribed by this document shall be in accordance with the following provisions:

(a) Seismic Design Categories A, B, and C. Masonry shear walls shall comply with the requirements of intermediate reinforced masonry shear walls (Section 11.11.4 of FEMA 302) or special reinforced masonry shear walls (Section 11.11.5 of FEMA 302).

(b) Seismic Design Categories D, E, and F. Masonry shear walls shall comply with the requirements of special reinforced masonry shear walls (Section 11.11.5 of FEMA 302).

(c) Basic requirements. Unit masonry will be reinforced with deformed bars for axial, flexural, shear, and diagonal tension stresses as determined by design calculations. Additional reinforcing bars are prescribed for use around openings, at corners, at anchored intersections, and at the ends of wall panels (for example, at control joints). The minimum reinforcement prescribed in FEMA 302 is intended to provide empirical requirements relative to damage control (ductility and boundary conditions). Layout and details of construction will be compatible with the application of the rules for modular measure.

(d) Types of reinforced masonry walls. Masonry will conform to one of the following basic types: reinforced grouted masonry, reinforced hollow masonry, or reinforced filled-cell masonry.

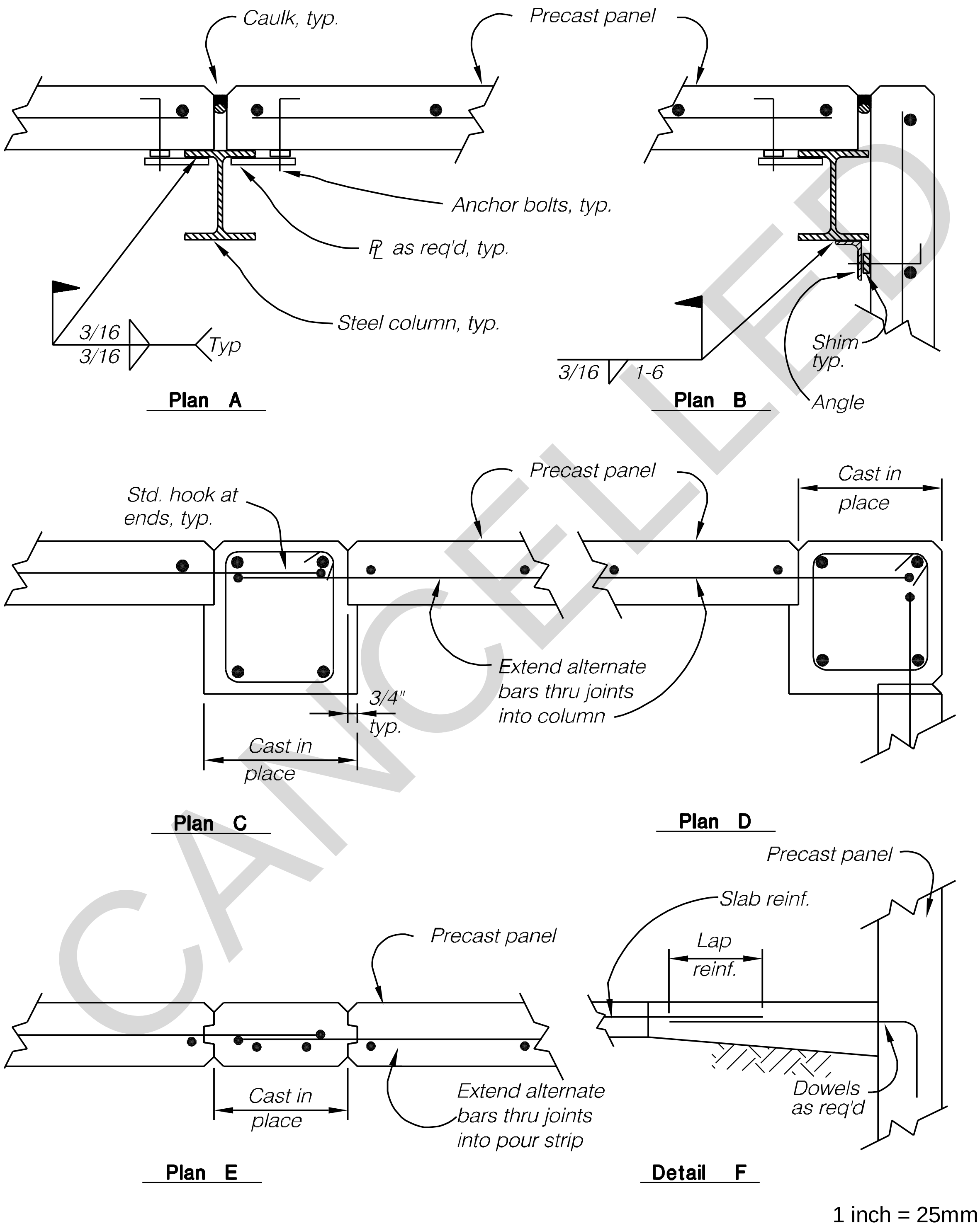


Figure 7-9 Tilt-up and other precast concrete walls - typical details of attachments

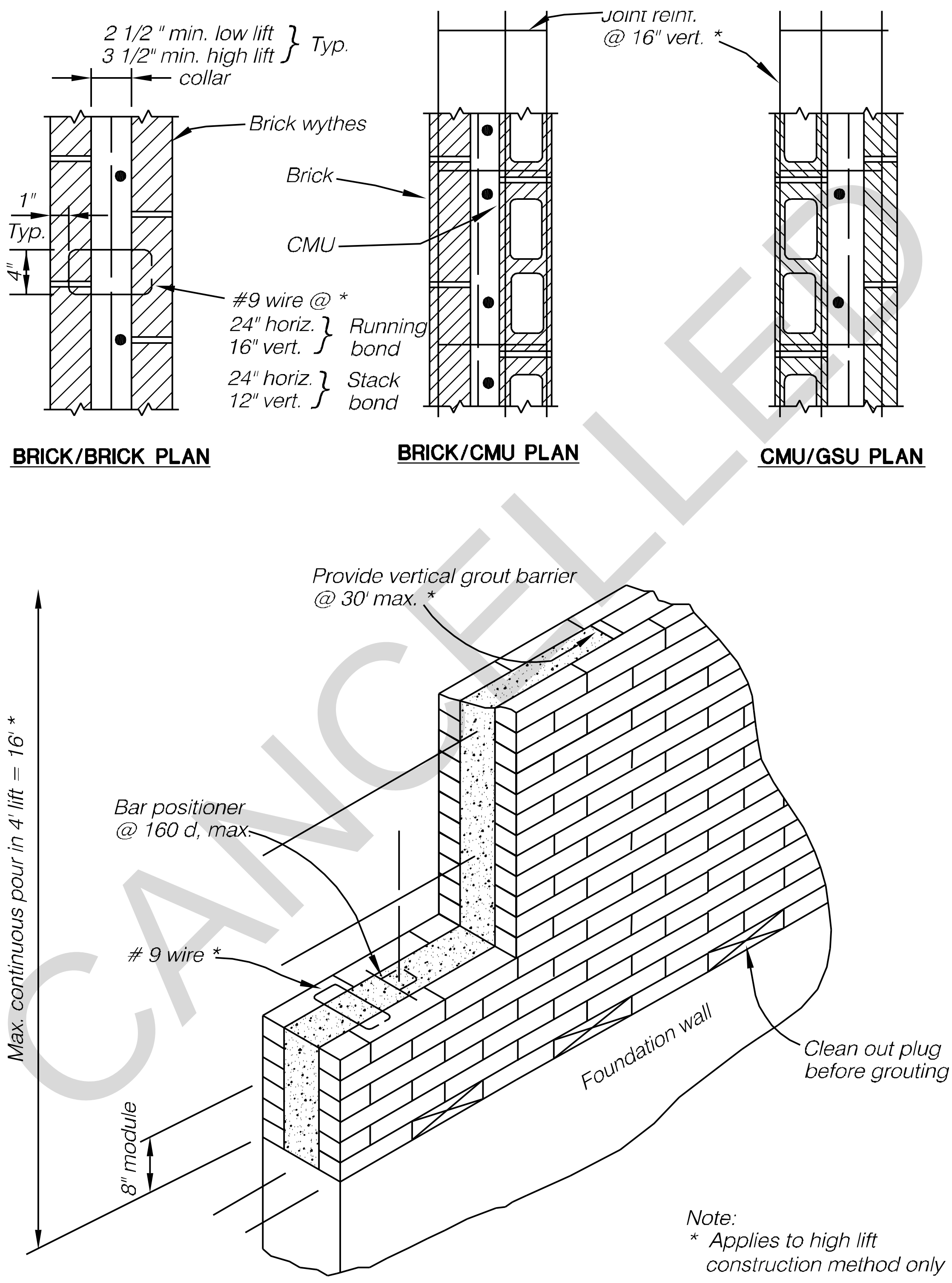
1. Reinforced grouted masonry is that type of construction made with two wythes of masonry units in which the collar joint between is reinforced and filled solidly with concrete grout. The grout may be placed as the work progresses or after the masonry units are laid. Collar joints will be reinforced with deformed bars, both vertical and horizontal. Reinforcement and embedded items such as structural connections and electrical conduit shall be positioned so as to allow proper placement of grout. All units will be laid in running bond with full shoved head and bed mortar joints. Masonry headers will not project into grout spaces. Clipped-brick headers will be used where the appearance of masonry headers is required (see Figure 7-10).

2. Reinforced hollow masonry is that type of construction made with a single wythe of hollow masonry units (concrete or clay blocks), reinforced vertically and horizontally with steel bars, and cores and voids containing reinforcing bars or embedded items are filled with grout as the work progresses (see Figure 7-11).

3. Reinforced filled-cell masonry is that type of construction made with a single wythe of hollow masonry units, reinforced vertically and horizontally with deformed steel bars, and all cores and voids are filled solidly with grout after the wall is laid (see Figure 7-12).

(e) Bond beams. Bond beams will be located as indicated in Figure 7-13. Reinforcement bars in bond beams will be lapped as prescribed in ACI 530 at splices, at intersections, and at corners. Bar splices will be staggered. Bond beams will be provided at top of masonry foundation wall stems, below and at top of openings or immediately above lintels, at floor and roof levels, and at top of parapet walls. Intermediate bond beams will be provided as required to conform to the maximum spacing of horizontal bars. When, however, the height is not a multiple of this normal spacing, the spacing may be increased up to a maximum of 24 inches (610mm) , provided the bond beams are supplemented with joint reinforcement. One line of joint reinforcement will be provided for each 8-inch (203mm) increase in the spacing. No additional bond beam will be required between window openings that do not exceed 6 feet (1.8m) in height, provided the prescribed supplemental joint reinforcement is installed. To facilitate the placement of steel or concrete core fill, the top bond beam for filler walls or partitions may be placed in the next-to-top course. The area of bond beam reinforcement will be included as part of the minimum horizontal steel.

(f) Design for crack control. Guidelines provided in TM 5-809-3/NAVFAC DM-2.9/AFM 88-3, Chapter 3, will be utilized to minimize cracking of masonry walls due to drying shrinkage and thermal expansion and contraction. The placement of control joints must be coordinated with the seismic design. Because the control joints provide a complete separation of the masonry, the location of control joints fixes the length of wall panels, and in turn, the rigidity of the walls, the distribution of seismic forces, and the resulting unit stresses. Therefore, adding, eliminating, or relocating control joints will not be permitted once the structural design is complete. Control joints will never be assumed to



1 inch = 25mm

Figure 7-10 Reinforced grouted masonry.

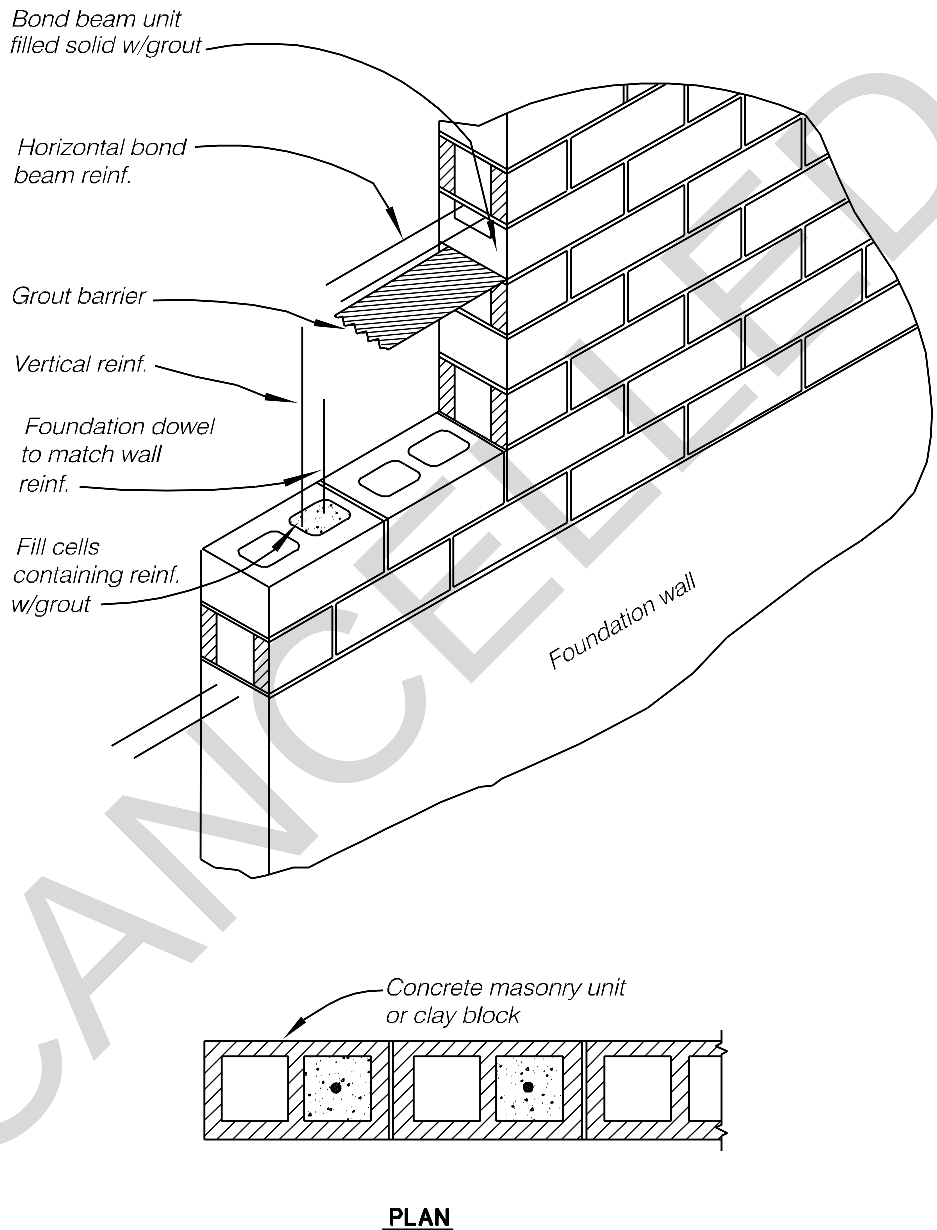
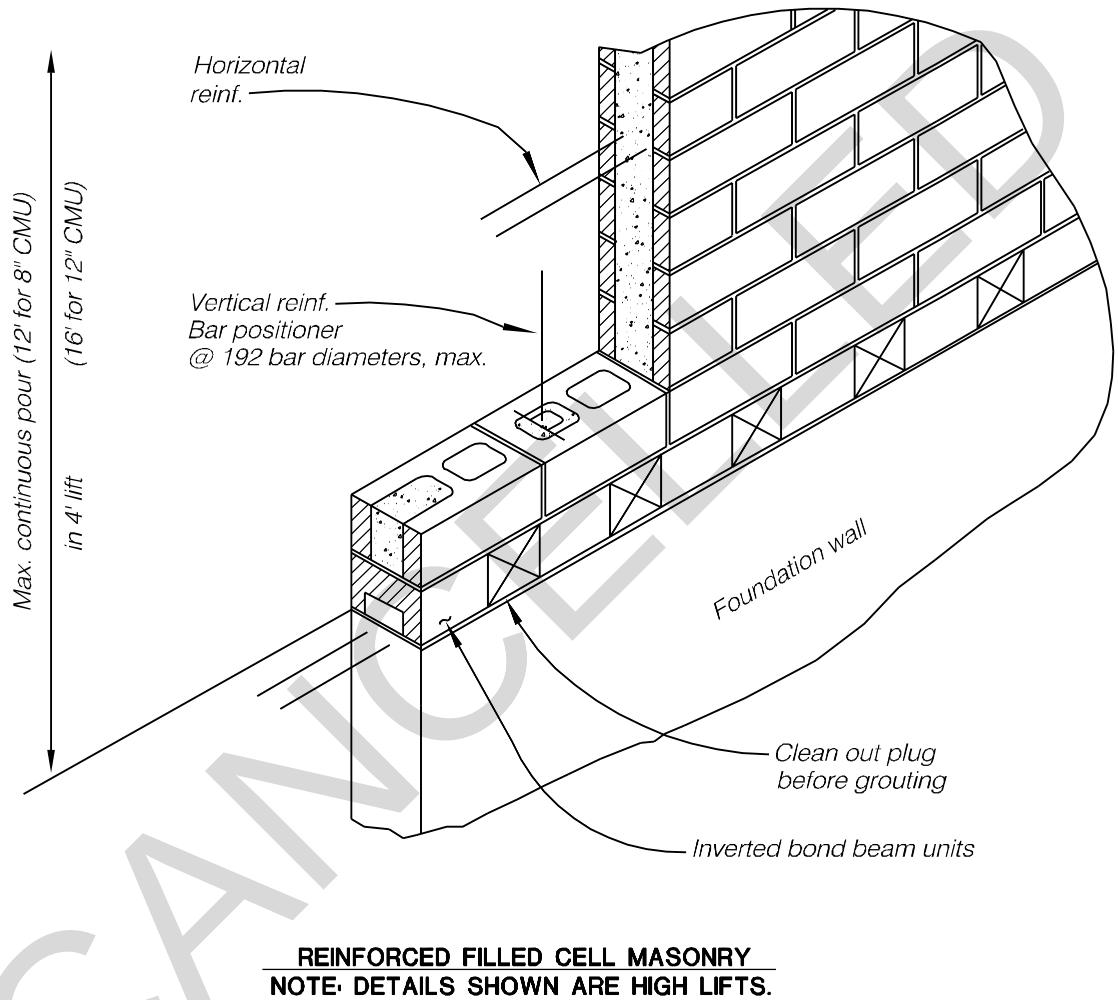
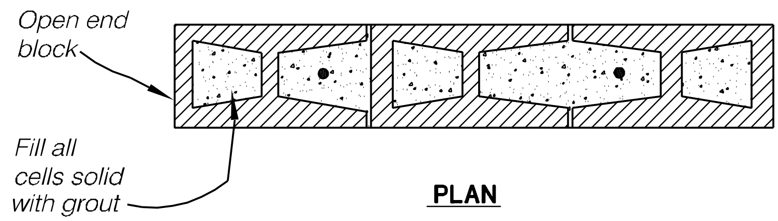


Figure 7-11 Reinforced hollow masonry.





1 inch = 25mm
1 foot = 0.30m

Figure 7-12 Reinforced filled-cell masonry.

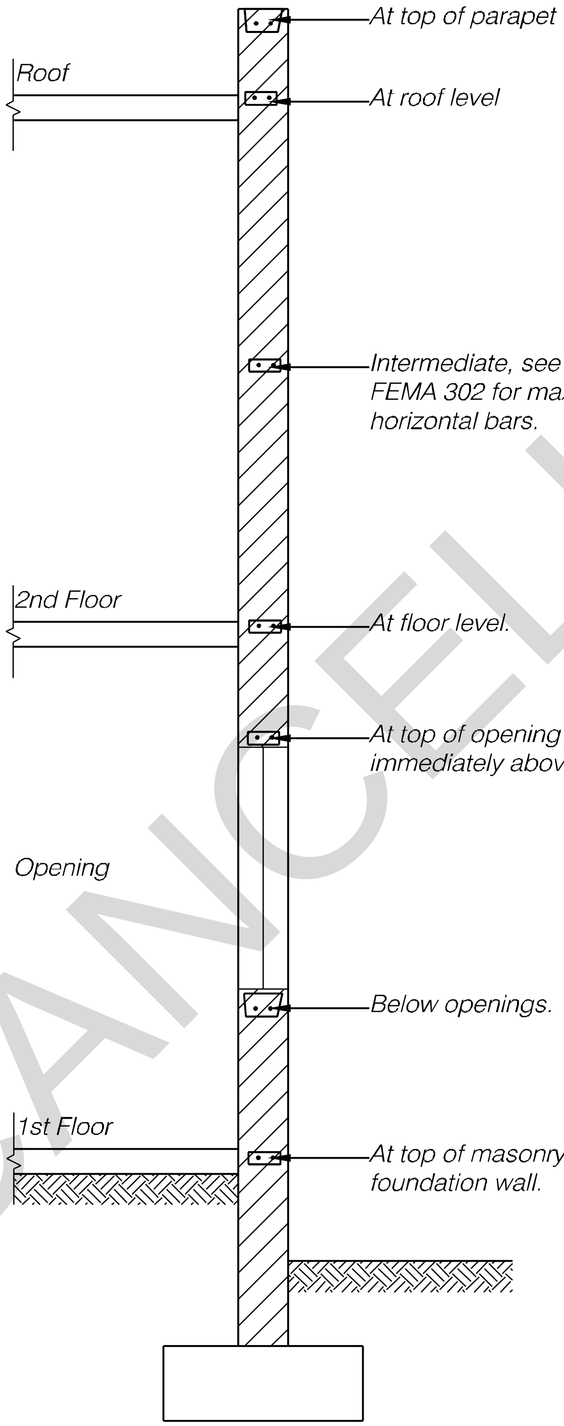


Figure7-13 Location of Bond Beams.

transfer bending moments or diagonal tension across the joint: joint reinforcement and bars in nonstructural bond beams will be terminated at control joints. Deformed bars in structural bond beams (those acting as chords and collectors) will be made continuous for the length of the diaphragm (refer to Figure 7-14).

(g) Design considerations.

1. Wall weights. Refer to ACI 530 for the average weight of concrete masonry units and the average weight of completed walls.

2. Shearing stresses in hollow masonry shall be based on area of the grouted cores plus the minimum net bedded cross-sectional area of the members under consideration.

3. Boundary Zones. When the compressive strains in special reinforced concrete shear walls exceed 0.0015 under combined loads, boundary zones shall be provided as prescribed for special reinforced concrete shear walls in Paragraph 7-2f(3).

(h) Reinforcing. Typical reinforcement is shown in Figure 7-16.

1. Minimum reinforcing. Unit masonry must be reinforced not only for structural strength, but to provide ductile properties and to hold it together in the event of severe seismic disturbance. All walls and partitions will be reinforced as required by structural calculations, but in no case with less than the minimum area of steel and the maximum spacing of bars prescribed in Chapter 11 of FEMA 302. Only reinforcement that is continuous in any wall panel will be considered in computing the minimum area of reinforcement. Joint reinforcement used for crack control or mechanical bonding may be considered as part of the total minimum horizontal reinforcement, but will not be used to resist computed stresses. Further additional bars will be provided around openings, at corners, at anchored intersections in wall piers, and at ends of wall panels, as prescribed elsewhere in this chapter. Vertical bars in walls will be spliced as prescribed in ACI 530.

2. Reinforcing in shear walls. In special reinforced masonry shear walls, reinforcement required to resist in-plane shear will be terminated with a standard hook or with an extension of proper embedment length beyond the reinforcing at the end of the wall section. The hook or extension may be turned up, down, or horizontally. Provisions will be made not to obstruct grout placement. Wall reinforcement terminating in columns or beams will be fully anchored into these elements.

3. Reinforcing in wall piers. Horizontal reinforcement will be in the form of ties as shown in Figure 7-16.

4. Column ties. For buildings in Seismic Design Categories D, E, and F, the spacing of column ties will not be more than 8 inches (203mm) for the full height for columns stressed by tensile or compressive axial overturning forces due to the seismic loads of Chapter 3; and 8 inches (203mm) for the tops and bottoms of all other columns for a distance of one-sixth of the clear column height, but not less than 18 inches (457mm), nor

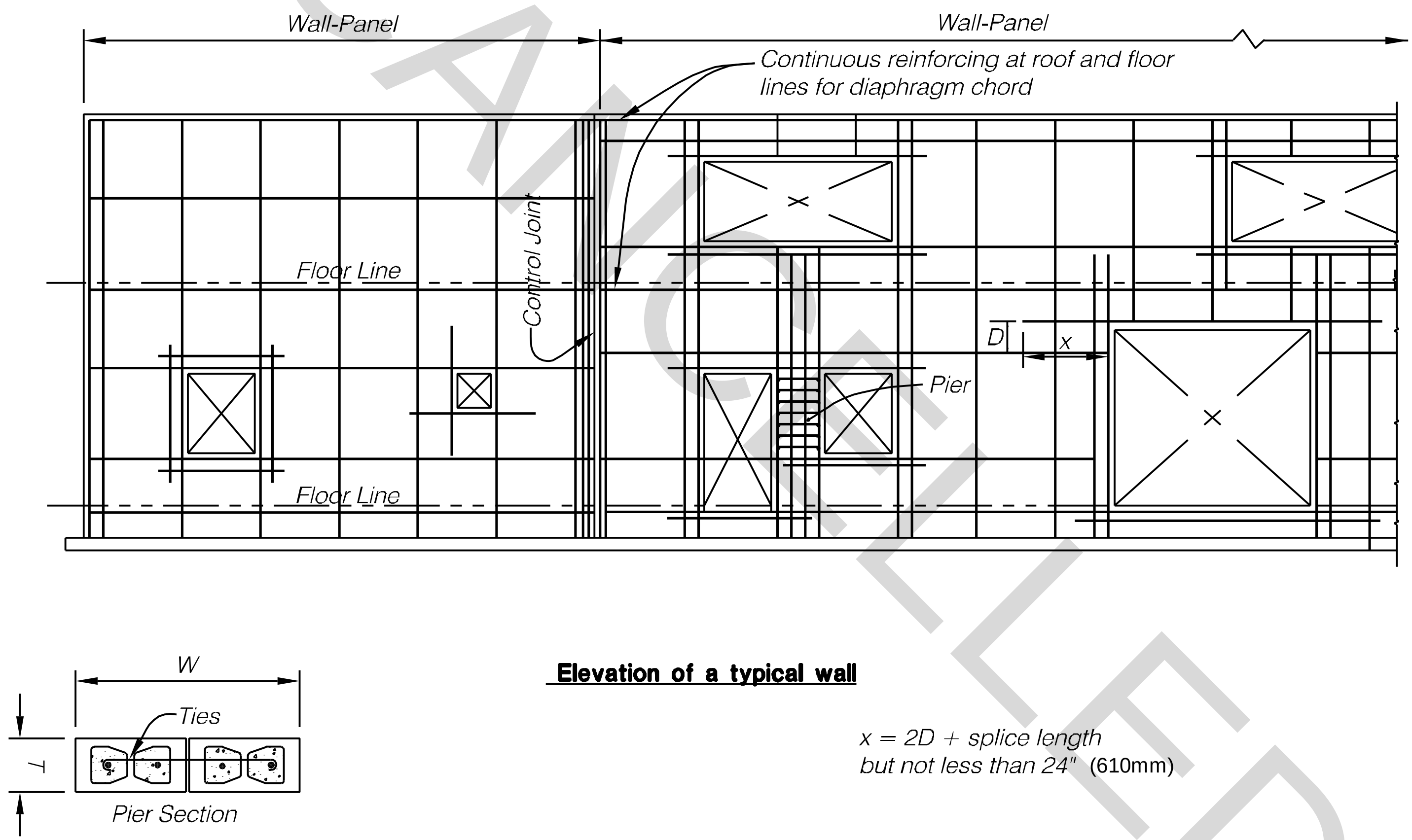


Figure 7-14. Typical wall reinforcement - reinforced masonry.

the maximum column dimension. Tie spacing for the remaining column height will be not more than 16 bar diameters, 48 tie diameters, or the least column dimension, but not more than 18 inches (457mm). Hooks in column ties will have a minimum turn of 135 degrees plus an extension of at least six bar diameters, but not less than 4 inches (102mm) at the free end of the bar, except that where the ties are placed in the horizontal bed joints, the hook will consist of a 90-degree bend having a radius of not less than four bar diameters, plus an extension of 32 bar diameters.

5. Reinforcing in stacked bond. For buildings in Seismic Design Categories A, B, and C, the minimum horizontal reinforcement ratio shall be .0007 bt. This ratio shall be satisfied by uniformly distributed joint reinforcement fully embedded in mortar or by horizontal reinforcement spaced not over 4 feet (1.2m), and fully embedded in grout. For buildings in Seismic Design Categories D, E, and F, the minimum horizontal reinforcement ratio shall be 0.015 bt. If open end units are used and grouted solid, then the minimum horizontal reinforcement ratio shall be .0007 bt. The above reinforcement ratios may be satisfied by combinations of joint and horizontal reinforcement.

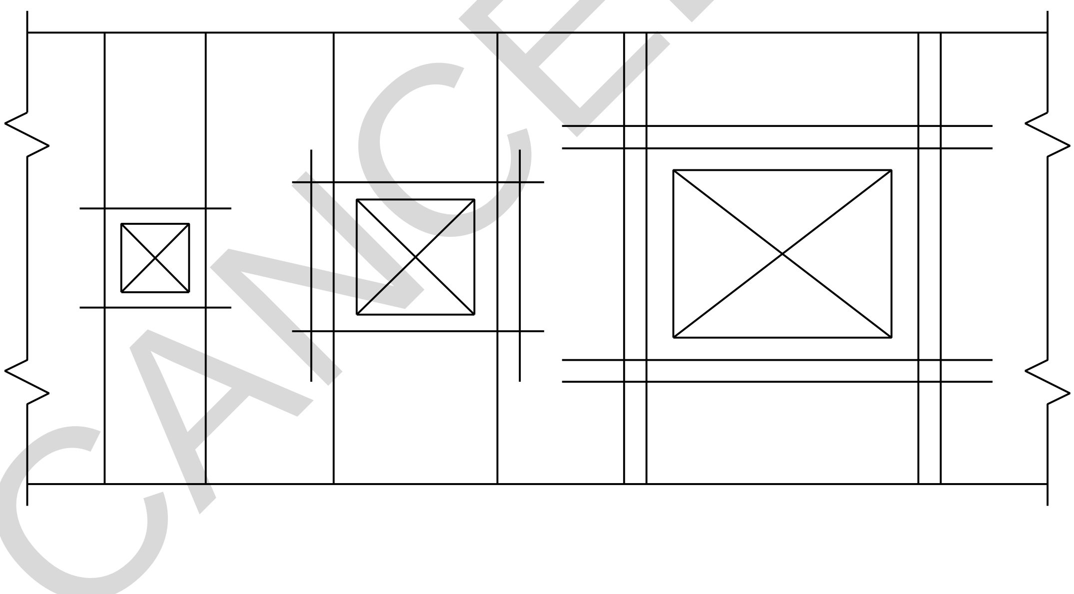
6. Reinforcing at wall openings. Since the area around wall openings is vulnerable to failure, supplemental reinforcement is prescribed herein. For purposes of this paragraph, the term "jamb bars" will mean bars of the same size, number, extent, and anchorages as the typical vertical stud reinforcement in that wall, and in no case less than one bar, #4 (10M) or larger (refer to Figure 7-15).

i. Case I. Case I applies to all openings in nonstructural partitions over 100 square inches (64.5 x 10^3 mm^2), and any opening in structural partitions or exterior walls that is 2 feet (0.6m) or less both ways, but over 100 square inches. Jamb bars will be provided on each side of the opening, and at least one bar, #4 (10M) or larger, will be provided at top and bottom of the opening. The lintel bars above the opening may serve as the top horizontal bar, and a bond beam bar at the bottom of the opening may serve as the bottom horizontal bar.

ii. Case II. Case II applies to exterior walls and structural partitions for any opening that exceeds 2 feet (0.6m), but is not over 4 feet (1.2m) in any direction. The perimeter reinforcement will be the same as in Case I, plus additional reinforcement as follows: #4 (10M) or larger will be provided on all four sides of the opening, in addition to the bars required in Case I, and shall extend not less than 40 bar diameters or 24 inches (0.6m), whichever is larger, beyond the corners of the opening.

iii. Case III. Case III applies to any opening that exceeds 4 feet (1.2m) in either direction in exterior walls or structural partitions. The perimeter reinforcement will be the same as in Case II, except that vertical jamb bars will be provided in lieu of the shorter vertical bars.

(i) Additional details (see Figure 7-16).



Refer to paragraph 7-2a(3)f for application of Cases I, II and III.

Figure 7-15 Reinforcement around wall openings.

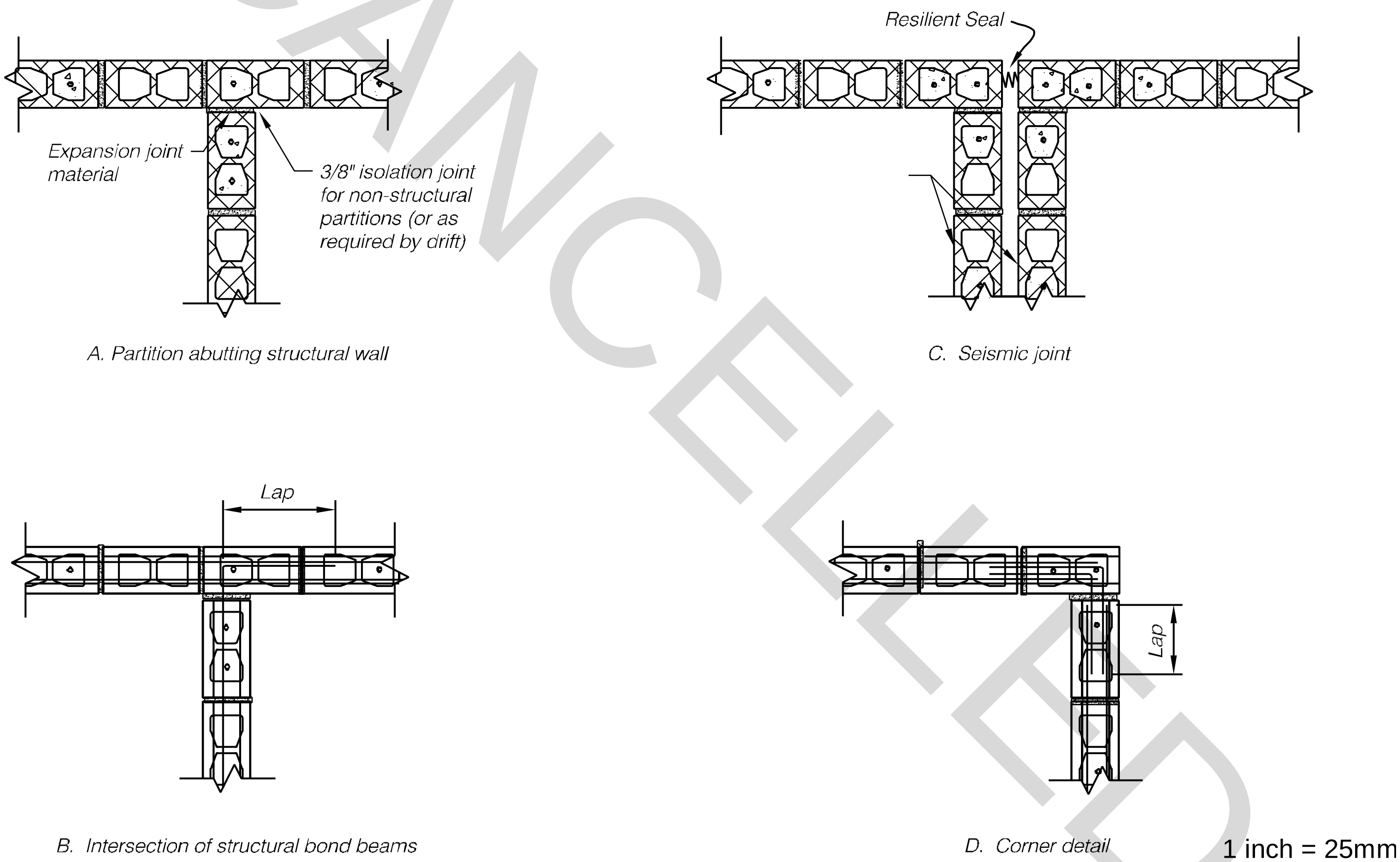


Figure 7-16 Masonry wall details.

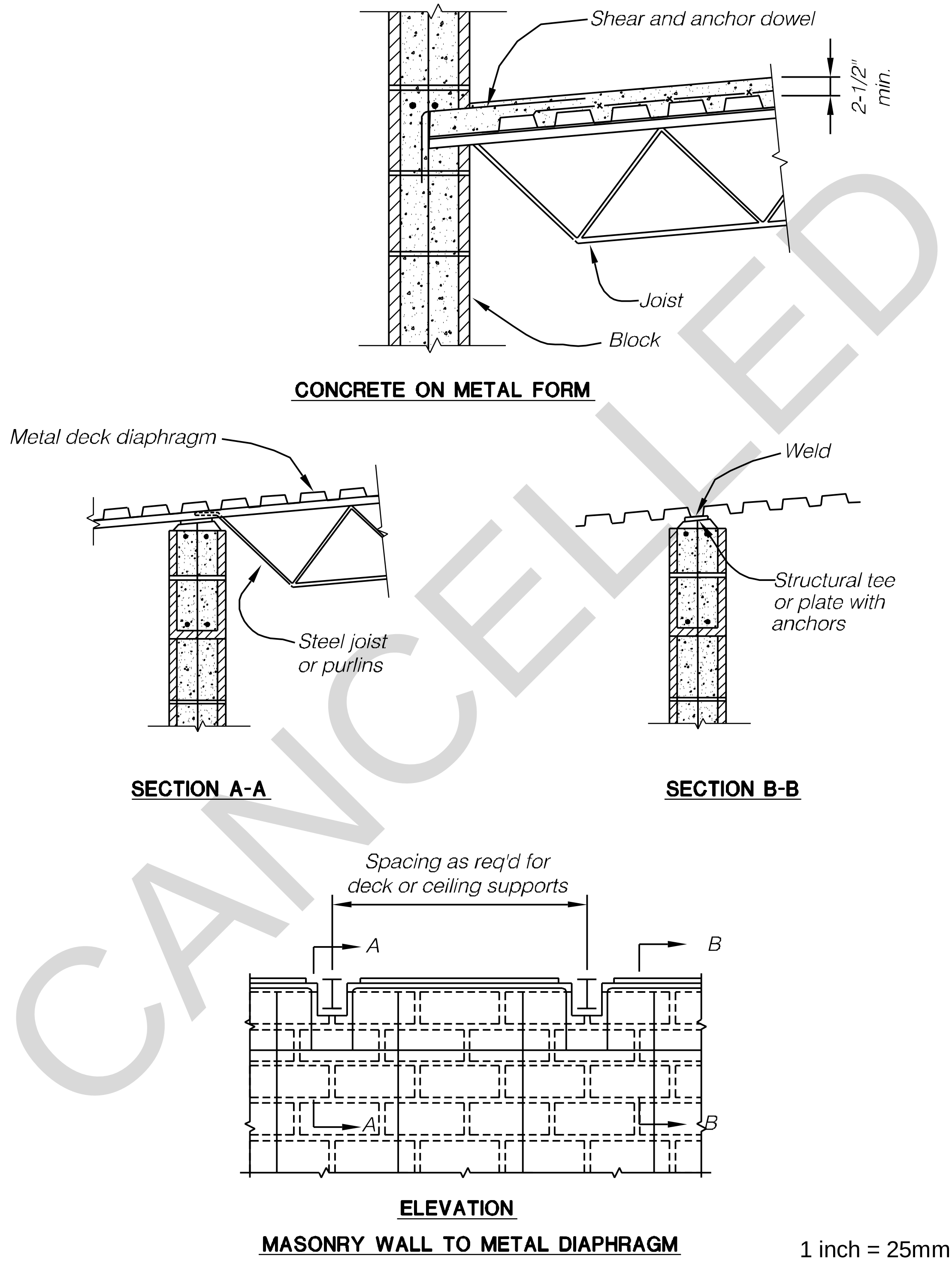


Figure 7-16 continued.

(4) Special requirements.

(a) Excluded materials. The following materials will not be used as part of the structural system:

1. In areas where $S_{DS} \geq 0.50$, glass block, non-load-bearing masonry units, plastic cement, masonry cement, and mortar with more than $1^{1}/_{4}$ parts by volume of hydrated lime or lime putty per one part of Portland cement.

2. In areas where $S_{DS} \geq 0.75$, glass block, non-load-bearing masonry units, plastic cement, masonry cement, and mortar with more than 1/2 part by volume of hydrated lime or lime putty per one part of Portland cement.

(b) Stacked bond. Since a running bond pattern is the strongest and most economical, the criteria in this document are based upon each wythe of masonry being constructed in a running bond pattern. The use of a stacked bond pattern will be restricted to reinforced walls essential to the architectural treatment. Filled-cell masonry or grouted masonry will be used. For filled-cell masonry, open-end blocks will be used and so arranged that closed ends are not abutting, and all head joints are made solid, and bond beam units shall be used to facilitate the flow of grout.

(c) Height limit. Unit masonry construction designed in accordance with the empirical procedure of ACI 530 will not be used for shear walls where the height of the building exceeds the limits given in Table 7-6.

(d) Joint reinforcement. Joint reinforcement will not be used in the calculation of shear strength.

(e) Mechanical splices. Mechanical splices will develop 125 percent of the specified yield strength of the bar in tension, except that for compression bars in columns that are not part of the seismic system and are not subject to flexure, the compressive strength only need be developed.

(f) Cavity walls. Cavity walls are not practical for use as shear walls because each wythe individually, and both wythes acting together in proportion to their relative rigidities, must be capable of carrying the required loads. It is usually much more economical to construct a two-wythe cavity-type wall by using an interior structural wythe and an exterior nonstructural anchored veneer wythe. See Chapter 10 for requirements for anchored veneer.

(g) Drawings. The locations of control joints and the identification of structural and nonstructural walls and partitions for all masonry construction will be shown on preliminary and contract drawings. On contract drawings, complete details for masonry, reinforcement, and connections to other elements will be shown. Detailing procedures outlined in ACI 318 are generally applicable to reinforced masonry.

(5) Acceptance criteria.

(a) Response modification factors, *R*, for Performance Objective 1A are provided in Table 7-1.

Construction	**Maximum l/t or h/t**
Bearing Walls	
Solid units or Fully grouted	20
All other	18
Nonbearing walls	
Exterior	18
Interior	36

Table 7-6. Lateral Support Requirements for Masonry Walls.

(b) Modification factors, m, for enhanced performance objectives and linear elastic procedures, are provided in Table 7-7 for deformation-controlled reinforced masonry in-plane walls and piers.

(c) Modeling parameters and numerical acceptance criteria for nonlinear procedures applied to deformation-controlled reinforced masonry walls and piers are provided in Table 7-8.

(d) Expected strength. The expected lateral strength of reinforced masonry, Q_{CE}, in either flexure or shear shall be determined using 1.25 f_y for the contribution attributed to the reinforcement.

(e) Lower-bound strength. The lower-bound strength for all other actions in URM or reinforced masonry shear walls shall be taken as the design strength defined by Section 11.5.3 of FEMA 302.

i. Wood Stud Shear Walls.

(1) General design criteria. The criteria used to design wood stud shear walls are presented in Chapter 12 of FEMA 302. Additional criteria and details are included in the following paragraphs.

(2) Allowable shears for plywood. Details of plywood sheathed walls are shown in Figure 7-17, and the allowable shears are shown in Table 7-9. When a combination of plywood and other materials is used, the shear strength of the walls will be determined by the values permitted for plywood alone.

(3) Conventional light frame construction, as defined in Section 12.5 of FEMA 302, may be used only for buildings required to comply with Performance Objective 1A.

(4) Deflections. Procedures for calculating the deflection of wood frame shear walls are not yet available. The maximum height-width limitations given herein are presumed to satisfactorily control deflections. Relative stiffness of wood stud shear walls will be measured by the effective lineal width of walls or piers between openings.

(5) Wall tie-down. The end studs of any plywood sheathed shear wall and/or shear wall pier will be tied down in such a manner as to resist the overturning forces produced by seismic forces parallel to the shear wall. This overturning force is sometimes of sufficient magnitude to require special steel attachment details. A commonly used detail is shown on Figure 7-18.

(6) Acceptance criteria.

(a) Compliance with the provisions of FEMA 302 constitutes the acceptance criteria for Performance Objective 1A for light frame construction.

(b) Response modification factors, R, for Performance 1A are provided in Table 7-1 for light frame walls in bearing-wall systems and building frame systems.

			m Factors					
			Primary				Secondary	
f_{ae}/f_{me}	L/h_{eff}	$\rho_g f_{ye}/f_{me}$	IO	SE	LS	CP	LS	CP
0.00	0.5	0.01	4.0	5.5	7.0	8.0	8.0	10.0
		0.05	2.5	3.8	5.0	6.5	8.0	10.0
		0.20	1.5	1.8	2.0	2.5	4.0	5.0
	1.0	0.01	4.0	5.5	7.0	8.0	8.0	10.0
		0.05	3.5	5.0	6.5	7.5	8.0	10.0
		0.20	1.5	2.3	3.0	4.0	6.0	8.0
	2.0	0.01	4.0	5.5	7.0	8.0	8.0	10.0
		0.05	3.5	5.0	6.5	7.5	8.0	10.0
		0.20	2.0	2.8	3.5	4.5	7.0	9.0
0.038	0.5	0.01	3.0	4.5	6.0	7.5	8.0	10.0
		0.05	2.0	3.3	3.5	4.5	7.0	9.0
		0.20	1.5	1.8	2.0	2.5	4.0	5.0
	1.0	0.01	4.0	5.5	7.0	8.0	8.0	10.0
		0.05	2.5	3.8	5.0	6.5	8.0	10.0
		0.20	1.5	2.0	2.5	3.5	5.0	7.0
	2.0	0.01	4.0	5.5	7.0	8.0	8.0	10.0
		0.05	3.5	5.0	6.5	7.5	8.0	10.0
		0.20	1.5	2.8	3.0	4.0	6.0	8.0
0.075	0.5	0.01	2.0	3.8	3.5	4.5	7.0	9.0
		0.05	1.5	2.3	3.0	4.0	6.0	8.0
		0.20	1.0	1.5	2.0	2.5	4.0	5.0
	1.0	0.01	2.5	3.8	5.0	6.5	8.0	10.0
		0.05	2.0	2.8	3.5	4.5	7.0	9.0
		0.20	1.5	2.0	2.5	3.5	5.0	7.0
	2.0	0.01	4.0	5.5	7.0	8.0	8.0	10.0
		0.05	2.5	3.8	5.0	6.5	8.0	10.0
		0.20	1.5	2.3	3.0	4.0	4.0	8.0

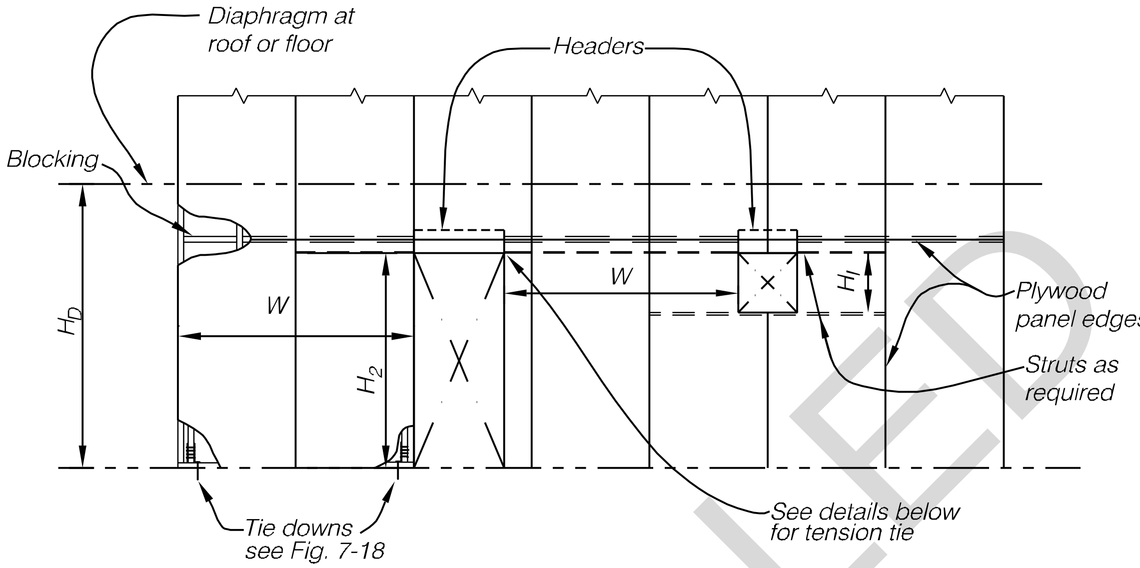
Note: Interpolation is permitted between table values.

Table 7-7: Linear Static Procedure—m Factors for Reinforced Masonry In-Plane Walls

						Acceptance Criteria					
						Primary				Secondary	
f_{ae}/f_{me}	L/h_{eff}	$\rho_g f_{ye}/f_{me}$	c	d %	e %	IO %	SE %	LS %	CP %	LS %	CP %
0.00	0.5	0.01	0.5	2.6	5.3	1.0	1.5	2.0	2.6	3.9	5.3
		0.05	0.6	1.1	2.2	0.4	0.6	0.8	1.1	1.6	2.2
		0.20	0.7	0.5	1.0	0.2	0.3	0.4	0.5	0.7	1.0
	1.0	0.01	0.5	2.1	4.1	0.8	1.2	1.6	2.1	3.1	4.1
		0.05	0.6	0.8	1.6	0.3	4.5	0.6	0.8	1.2	1.6
		0.20	0.7	0.3	0.6	0.1	0.2	0.2	0.3	0.5	0.6
	2.0	0.01	0.5	1.6	3.3	0.6	0.9	1.2	1.6	2.5	3.3
		0.05	0.6	0.6	1.3	0.2	0.4	0.5	0.6	0.9	1.3
		0.20	0.7	0.2	0.4	0.1	0.2	0.2	0.2	0.3	0.4
0.038	0.5	0.01	0.4	1.0	2.0	0.4	0.6	0.8	1.0	1.5	2.0
		0.05	0.5	0.7	1.4	0.3	0.4	0.5	0.7	1.0	1.4
		0.20	0.6	0.4	0.9	0.2	0.3	0.3	0.4	0.7	0.9
	1.0	0.01	0.4	0.8	1.5	0.3	0.5	0.6	0.8	1.1	1.5
		0.05	0.5	0.5	1.0	0.2	0.3	0.4	0.5	0.7	1.0
		0.20	0.6	0.3	0.6	0.1	0.2	0.2	0.3	0.4	0.6
	2.0	0.01	0.4	0.6	1.2	0.2	0.3	0.4	0.6	0.9	1.2
		0.05	0.5	0.4	0.7	0.1	0.2	0.3	0.4	0.5	0.7
		0.20	0.6	0.2	0.4	0.1	0.1	0.1	0.2	0.3	0.4
0.075	0.5	0.01	0.3	0.6	1.2	0.2	0.4	0.5	0.6	0.9	1.2
		0.05	0.4	0.5	1.0	0.2	0.3	0.4	0.5	0.8	1.0
		0.20	0.5	0.4	0.8	0.1	0.2	0.3	0.4	0.6	0.8
	1.0	0.01	0.3	0.4	0.9	0.2	0.3	0.3	0.4	0.7	0.9
		0.05	0.4	0.4	0.7	0.1	0.2	0.3	0.4	0.5	0.7
		0.20	0.5	0.2	0.5	0.1	0.2	0.2	0.2	0.4	0.5
	2.0	0.01	0.3	0.3	0.7	0.1	0.2	0.2	0.3	0.5	0.7
		0.05	0.4	0.3	0.5	0.1	0.2	0.2	0.3	0.4	0.5
		0.20	0.5	0.2	0.3	0.1	0.1	0.1	0.2	0.2	0.3

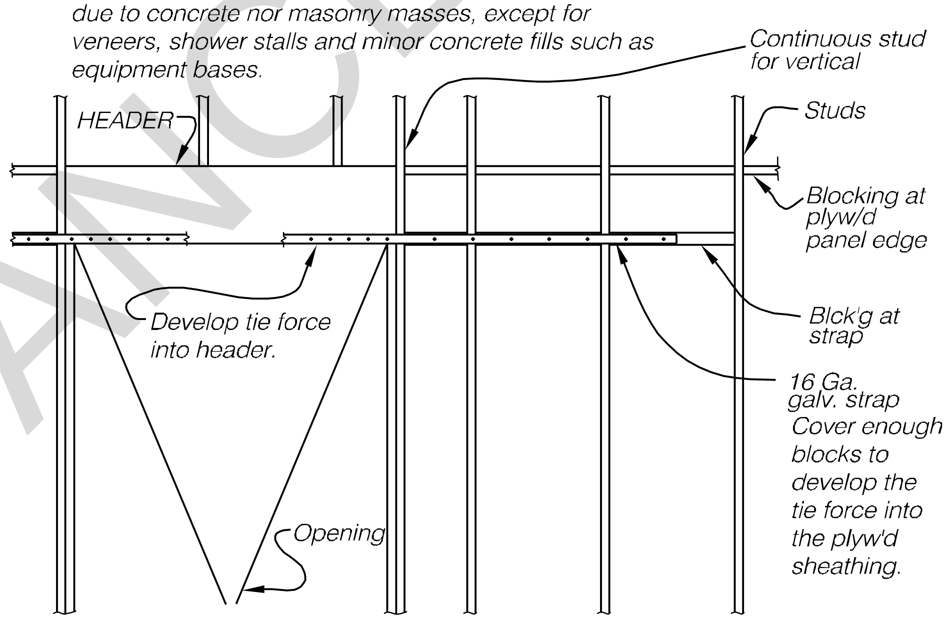
Note: Interpolation is permitted between table values.

Table 7-8: Nonlinear Static Procedure—Simplified Force-Deflection Relations for Reinforced Masonry Shear Walls



NOTES:

1. For values of plywood sheathed shear walls, see Table 7-10.
2. Height-width ratio of plywood sheated shear walls will be limited to 3-1/2 to 1. H_D / w will be used for the height-width ratio unless struts are developed at the top and bottom of the openings in which case H_1 / w or H_2 / w may be used.
3. These shear walls shall not be used to resist forces due to concrete nor masonry masses, except for veneers, shower stalls and minor concrete fills such as equipment bases.



Detail of tension tie-strut

Figure 7-17 Plywood sheathed wood stud shear walls.

TABLE 7-9 Factored Shear Resistance in Kips per Foot (KLF) for Seismic Forces on Structural Use Panel Shear Walls with Framing Members of Douglas Fir-Larch or Southern Pine

Panel Grade	Nail Size (Common or Hot-Dipped Galvanized Box)	Minimum Penetration in Framing (in.)	Panel Thickness (in.)	Panel Applied Direct to Framing Nail Spacing at Panel Edges (in.)				Nail Size (Common or Hot-Dipped Galvanized Box)	Panel Applied Over ½ in. or 5/8 in. Gypsum Sheathing Nail Spacing at Panel Edges (in.)			
				6	4	3	2 [d]		6	4	3	2 [d]
Structural 1	6d	1¼	3/8	0.23	0.35	0.46	0.60	8d	0.23	0.35	0.46	0.60
	8d	1½	3/8	0.27 [f]	0.42	0.54 [f]	0.71 [f]	10d [e]	0.27 [f]	0.42 [f]	0.54 [f]	0.71 [f]
	8d	1½	7/16	0.30 [f]	0.46 [f]	0.59 [f]	0.78 [f]	10d [e]	0.30 [f]	0.46 [f]	0.59 [f]	0.78 [f]
	8d	1½	15/32	0.33	0.50	0.64	0.85	10d [e]	0.33 [f]	0.50 [f]	0.64 [f]	0.85 [f]
	10d [e]	1-5/8	15/32	0.40	0.60	0.78	1.02		-	-	-	-
	14 ga staple	2	3/8	0.17	0.26	0.35	0.52		-	-	-	-
	14 ga staple	2	7/16	0.24	0.36	0.48	0.72		-	-	-	-
Sheathing, Panel Siding and Other Grades Covered in References 9.10 and 9.11	6d	1¼	3/8	0.23	0.35	0.46	0.60	8d	0.23	0.35	0.46	0.60
	8d	1½	3/8	0.26 [f]	0.37 [f]	0.48 [f]	0.62 [f]	10d [e]	0.26 [f]	0.37 [f]	0.48 [f]	0.62 [f]
	8d	1½	7/16	0.28 [f]	0.41 [f]	0.53 [f]	0.68 [f]	10d [e]	0.28 [f]	0.41 [f]	0.53 [f]	0.68 [f]
	8d	1½	15/32	0.30	0.44	0.57	0.75	10d [e]	0.30 [f]	0.44 [f]	0.57 [f]	0.75 [f]
	10d [e]	1-5/8	15/32	0.36	0.54	0.70	0.90		-	-	-	-
	10d [e]	1-5/8	19/32	0.40	0.60	0.78	1.02		-	-	-	-
	14 ga staple	2	3/8	0.15	0.23	0.30	0.45		-	-	-	-
	14 ga staple	2	7/16	0.21	0.32	0.42	0.63		-	-	-	-
	14 ga staple	2	15/32	0.24	0.36	0.48	0.72		-	-	-	-
	(Hot-Dipped Galvanized Casing Nail)							(Hot-Dipped Galvanized Casing Nail)				
Panel Siding as Covered in Reference 9.10	6d	1¼	3/8	0.16	0.25	0.32	0.42	8d	0.16	0.25	0.32	0.42
	8d	1½	3/8	0.19	0.28	0.36	0.48	10d [e]	0.19	0.28	0.36	0.48

TABLE 7-9 (cont.)

Factored Shear Resistance in Kips per Foot (KLF) for Seismic Forces on Structural Use Panel Shear Walls with Framing Members of Douglas Fir-Larch or Southern Pine

a $\lambda = 1.0$ $\varnothing = 0.65$

b All panel edges backed with 2-inch nominal or wider framing. Panels installed either horizontally or vertically. Space nails at 6 inches on center along intermediate framing members for 3/8-inch panels installed with strong axis parallel to studs spaced 24 inches on center and 12 inches on center for other conditions and panel thicknesses. Allowable shear values for fasteners in framing members of other species set forth in Table 12A of ASCE 16-95 shall be calculated for all grades by multiplying the values for fasteners in STRUCTURAL I by the following factors: 0.82 for species with a specific gravity greater than or equal to 0.42 but less than 0.49 ($0.42 \le G < 0.49$) and 0.65 for species with a specific gravity less than 0.42 ($G < 0.42$). For panel siding using hot-dipped galvanized casing nails, the shear values shall be the values in the table multiplied by the same factors.

c Where panels are applied on both faces of a wall and nail spacing is less than 6 inches on center on either side, panel joints shall be offset to fall on different framing members, or framing shall be 3-inch nominal or wider, and nails on each side of joint shall be staggered.

d Framing at adjoining panel edges shall be 3-inch nominal or wider, and nails shall be staggered where nails are spaced 2 inches on center.

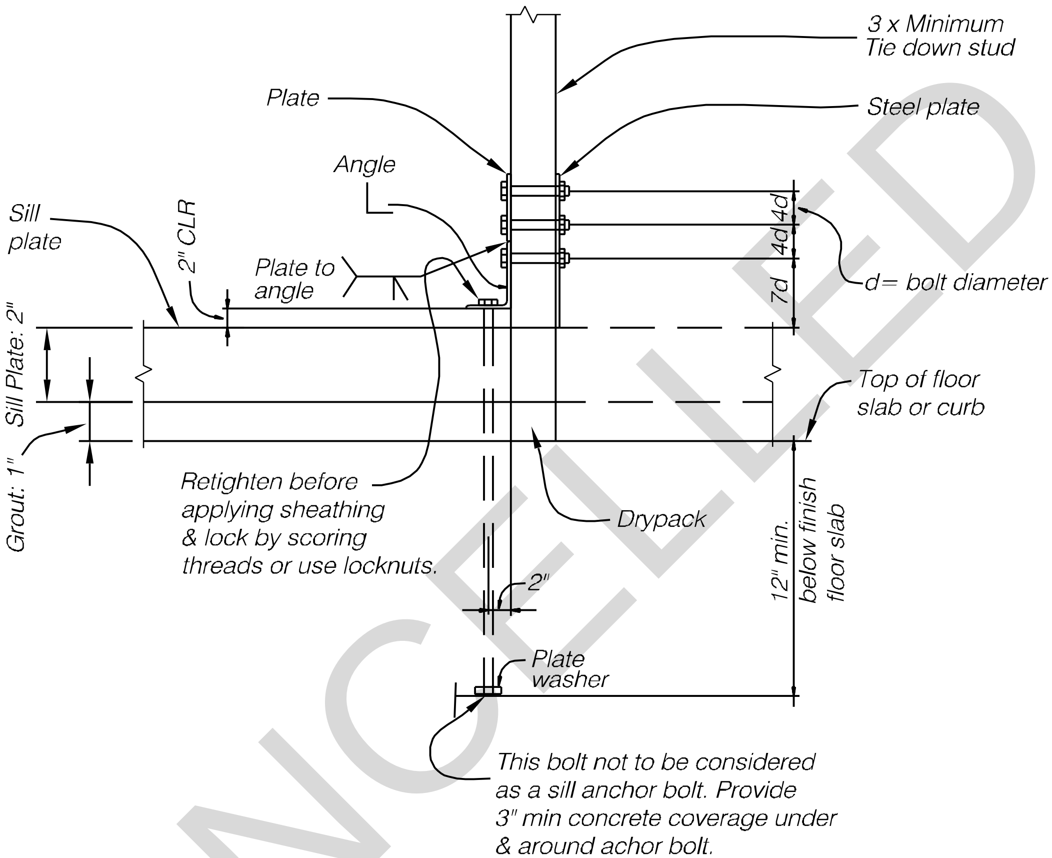
e Framing at adjoining panel edges shall be 3-inch nominal or wider, and nails shall be staggered where 10d nails having penetration into framing of more than 1-5/8 inches are spaced 3 inches or less on center.

f The values for 38-inch and 7/16-inch panels applied directly to framing are permitted to be increased to the values shown for 15/32-inch panels, provided studs are spaced a maximum of 16 inches on center or panel is applied with strong axis across studs.

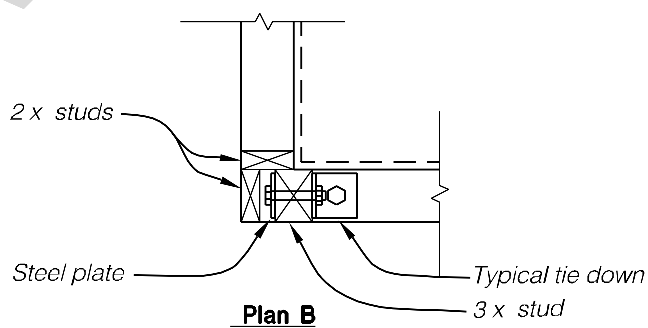
Metric equivalent:

1 inch = 25mm

1 kLF = 14.6 kN/m



Typical tie-down detail A



Plan B

Corner detail at tie-down

1 inch = 25mm

Figure 7-18 Wood stud walls - typical tie-down details

j. Steel Stud Shear Walls.

(1) Description of system. Steel studs may be used in lieu of wood studs in structural bearing walls. To function as shear walls, steel-stud walls need bracing. In principle, plywood sheathing could be used, but there are no available allowable shear values. Instead, it is customary to use diagonal braces made of steel straps welded to the face of the steel studs. Sheathing such as plywood or gypsum board may be used to serve architectural purposes such as containing insulation and backing up finishes.

(2) Design Criteria. The Department of Defense is currently reviewing tests performed by industry with the objective of providing approved design criteria for steel stud framing systems. A moratorium currently precludes the use of this system as a lateral-force-resisting system. It is anticipated that applicable criteria will be available prior to the final version of this document.

7-3. Steel Braced Frames.

a. General.

(1) Function. Vertical braced frames are used to transmit lateral forces from the diaphragm above to the diaphragm below or to the foundations. They are similar to shear walls in their general function and stiffness is compared with moment-resisting frames.

(2) Definition of braced frame. A braced frame is defined as an essentially vertical truss system of the concentric or eccentric type that is provided to resist lateral forces. Note that for braced frames, as for shear walls, the *R* value depends on whether the frame is in a building-frame system, a moment-resisting frame system, or a dual system.

(3) Redundancy. A sufficient number of braced frames should be provided so that a failure of a single member or connection will not result in instability of the entire lateral-force-resisting system.

(4) Braced frame types. The principal types of braced frame are the familiar concentric braced frame (CBF), the relatively new eccentric braced frame (EBF), and the knee-braced frame (KBF).

(5) Design criteria. The criteria governing the design of structural steel and wood vertical braced frames will be as prescribed in this chapter. Reinforced concrete braced frames are not permitted in buildings governed by this document.

(a) Structural steel braced frames. Structural steel braced frames will conform to the requirements of the AISC “Seismic Provisions for Structural Steel Buildings” and the further provisions of this document.

(b) Wood braced frames. Wood braced frames will be designed by using normal procedures illustrated in many easily obtainable texts and are not covered in this manual. Allowable loads and resistance factors for wood members shall be in accordance with ASCE 16-95.

b. Concentric Braced Frames.

(1) Eccentricities. Although the frame is called “concentric,” there may be minor eccentricities between member centerlines at the joints, and these

eccentricities must be provided for in the design. Such eccentricities do not mean that the frame is an EBF: the EBF has unique properties and design methods.

(2) Concentric braced frame types. Braced frames are usually of steel and may be of various forms. Figure 7-19 illustrates some of the common configurations for concentric braced frames. Trussed portal bracing and K-bracing sometimes used in the older industrial buildings are still occasionally used in bridge design, but have been replaced in buildings by one or more of the configurations shown in Figure 7-19. Braced frames with single diagonal members capable of resisting compression as well as tension are used to permit flexibility in the location of openings. Chevron bracing also permits openings in the middle of the braced bay, but the horizontal beam at bracing intersection must be capable of resisting an additional load equal to the vertical component of the tensile brace when the compressive brace buckles. For all of the bracing configurations in Figure 7-19, the deflection of the braced frame is readily computed using recognized methods.

(3) Direction of brace force. Braces that are designed for compression will, of course, act also in tension. Diagonal members designed to resist both compression and tension forces are preferred because they provide greater system redundancy. X-braced panels are the most effective bracing configurations as the tension diagonal provides direct in-plane lateral support to the compression diagonal and also provides out-of-plane resistance to compression buckling (as indicated in Figure 7-21 an unbraced length equal to two-thirds of the total length of the compression brace may be used for the effective out-of-plane length). Braces may be designed for tension only, but the use of such braces is discouraged because they tend to stretch under earthquake tension, go slack during the load reversal, then snap when tension is applied in a subsequent cycle. Diagonal cable bracing is permitted only for utilitarian one-story Seismic Use Group I buildings in areas with $S_{DS} \leq 0.50$g, and where the system is not required to provided lateral support for concrete or masonry walls.

(4) Effect of bracing on columns. The vertical component of brace force is transferred into the column, and adds to or subtracts from the gravity load on the column. When braces are few and heavily loaded, their vertical components may govern the design of the columns. The concern with braces of this type is that their true, as-built ultimate capacity may be greater than is assumed in design, and therefore, that such braces could overload the column to the point of collapse.

(5) Configurations. Diagonal X-bracing is the preferred configuration in that the tension brace can provide in-plane lateral support to the compression brace. The orientation of single braces should be alternated so that not all of the braces are in tension or compression at the same time. Chevron bracing may have an interaction with gravity-load-carrying beams; accordingly, special requirements are provided in the AISC Seismic Provisions. K-bracing has a potentially dangerous effect on columns;

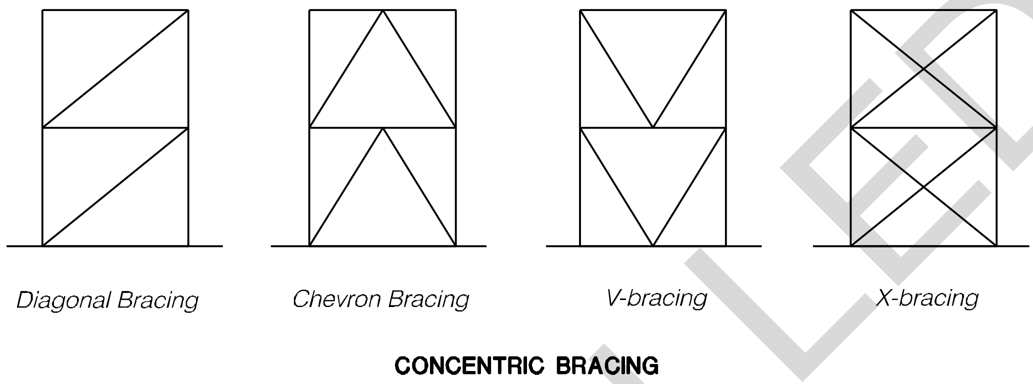


Figure 7-19 Concentric braced frames.

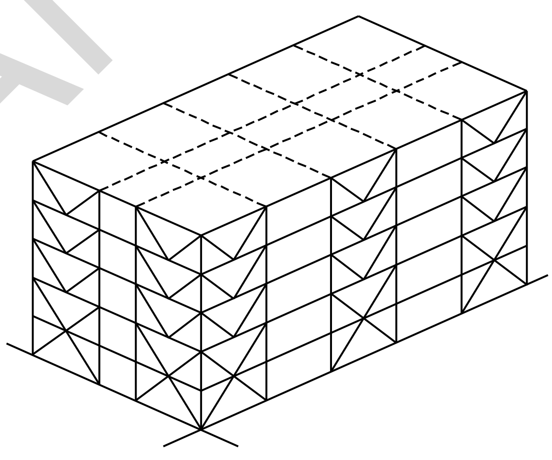


Figure 7-20 Bracing for a tier building.

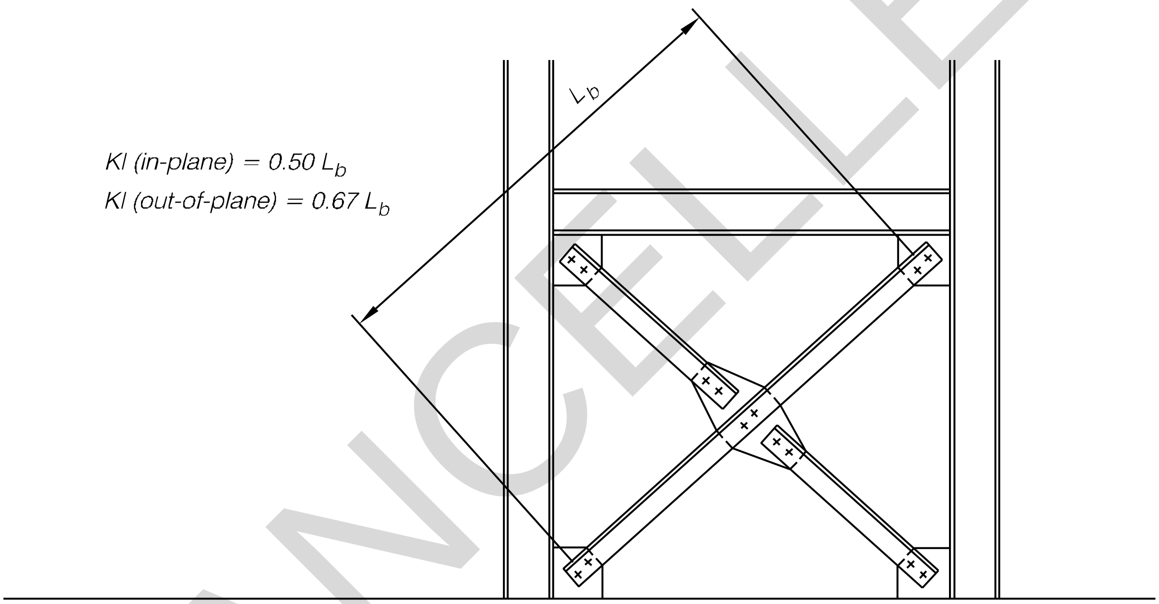


Figure 7-21 Effective length of cross-bracing.

accordingly, it is subject to the requirements of Section 14.4b, Part I, of the AISC Seismic Provision, and permitted only in buildings in Seismic Design Categories A and B.

(6) Low buildings. The AISC Seismic Provisions provide special provisions for concentric bracing in metal buildings not over two stories, and for light roof structures such as penthouses. Manufactured metal buildings are intended to be included in this category. In planning the use of manufactured metal buildings, the designer is cautioned that these buildings can perform well only when they are kept light and simple, as they are intended to be; they may have poor performance if extra weight, such as masonry veneer, is added, or if they are used as elements of a more complex system.

(7) Knee-braced frames (KBF).

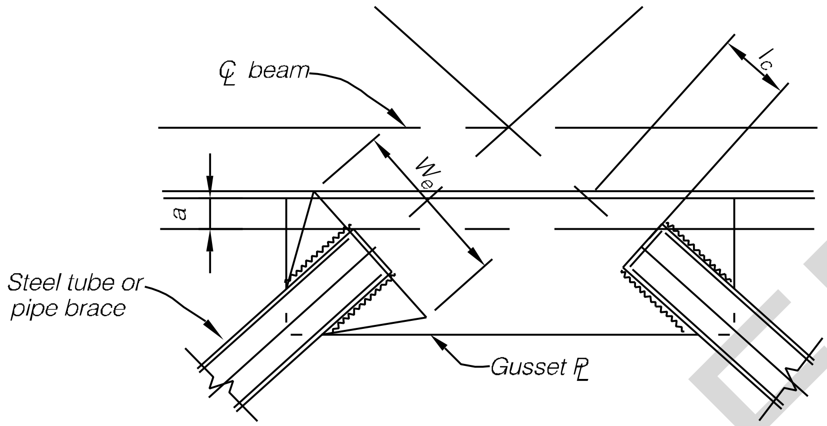
(a) Definition. A KBF is an assembly of a beam, a column, and a brace whose ends are significantly offset from the beam-column joints. The braces in CBFs are either truly concentric, or have small eccentricities with the beam-column joints; accordingly, they induce forces that are primarily axial, while the braces in KBFs have substantial eccentricities, and induce significant shearing, and flexural, as well as axial, stresses in the columns and beams.

(b) Function. Knee braces were often used in the past to stiffen beams and to provide a measure of lateral stability. Their popularity in recent years has decreased markedly, particularly in zones of high seismicity, because their seismic behavior has become recognized as potentially dangerous.

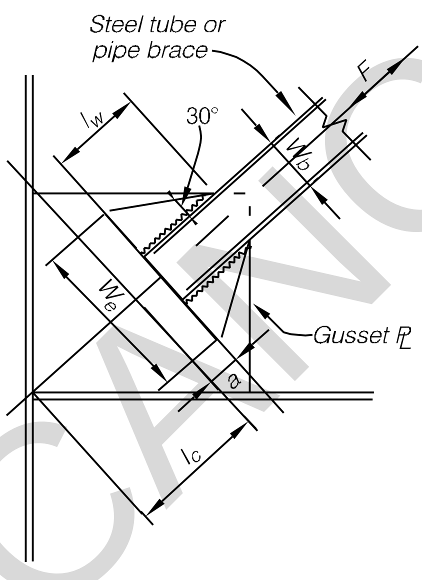
(c) Design considerations. There are two concerns with KBFs. The first concern involves gravity load: any change in the load on the beam after the brace is connected induces forces in all the components of the frame; moreover, the brace has a prying effect that can produce surprisingly large forces in the beam-column joint. The sequence of erection and the further application of superimposed loads must be carefully controlled. The second concern involves seismic loads: another set of loads is applied, and while the brace does stiffen the frame, its as-built ultimate capacity may cause bending in the column of sufficient magnitude to cause collapse.

(d) Design criteria. KBFs shall be designed in accordance with Section 9.4 of the AISC Seismic Provisions, and the use of KBFs shall be restricted to roof structures or to unoccupied storage or other utilitarian buildings with Performance Objective 1A, not over two stories in height.

(8) Connections. The AISC Seismic Provisions provide the requirements for design of connections. Figure 7-22 illustrates the design of gusset plates with welded connections. Note that most steel braces are designed as pin-ended members (K=1.0) for compressive forces. As the braces deflect out-of-plane in compression, the gusset must be able to accommodate the end-rotation. The AISC LRFD Specifications prescribe that the brace connection should provide a minimum length of gusset plate, a, equal to twice the plate thickness, t, to permit end-rotation of the brace as shown in Figure



DETAIL 1



DETAIL 2

Gusset Plate Design Criteria:

Max brace force F:
Tension: $= R_y F_y A_g$
Compr. $= A_g F_{cr}$

Shear/tension rupture:
$0.75t\,(0.6\,F_y \times 2\,l_w/\cos 30^\circ + F_u \times W_e)$ *or*
$0.75t\,(0.6\,F_u \times 2\,l_w + F_y \times W_b)$

Tension:
$0.75 \times W_e \times F_u \times t \geq R_y F_y A_g$

Compression:
$0.90 \times F_y \times W_e \times t \geq A_g F_{cr}$

Buckling:
$$0.90 \times \frac{4000\, t^3 \sqrt{F_y}}{l_c} \geq A_g F_{cr}$$

Rotation capability:
$a \geq 2\,t$

Figure 7-22 Gusset plate design criteria.

7-22. When the in-plane brace connection is welded as shown in Figure 7-21, the appropriate *K* value for restrained end conditions should be used, and the welds and gusset plate should be designed for the plastic moment capacity of the brace. For the gusset plate, a section, normal to the brace, or the midpoint of the connection, should have the necessary capacity to resist the above moment.

(9) Acceptance criteria.

(a) Response modification factors, *R*, for Performance Objective 1A are provided in Table 7-1. K-braced frames shall be classified as ordinary concentric braced frames and are subject to the limitations of Paragraph (6) above.

(b) Modification factors, *m*, for enhanced performance objective are provided in Table 7-10.

(c) Modeling parameters and numerical acceptance criteria for nonlinear procedures are provided on Table 7-11.

(d) The expected strength of deformation-controlled components or elements shall be determined using the expected yield strength, F_{ye}, as defined in the AISC Seismic provisions.

(e) The lower-bound strength of connections and other force-controlled components shall be taken as the nominal strength multiplied by the appropriate resistance factor, ϕ, determined from the provisions of the AISC LRFD Specifications.

c. *Eccentric Braced Steel Frames (EBF).*

(1) Definition. An EBF is a steel-braced frame designed in accordance with Section 15, Part I, of the AISC Seismic Provisions. At least one end of each brace intersects a beam at a point offset from the beam intersection with the column or with the opposing brace (see Figure 7-23). The short section of the beam between opposing braces, or between a brace and the beam-column intersection, is called the "link beam," and is the element of the frame intended to provide inelastic cyclic yielding.

(2) Purpose. The intent of the eccentric braced frame design is to provide a ductile link that will yield in lieu of buckling of its braces when the frame experiences dynamic loads in excess of its elastic strength. Although they are usually easier to detail, they are more complex to design than CBFs, and they are most useful in areas with $S_{DS} \geq 0.75$.

(3) Characteristics. To take advantage of the ductility of the link, it is important that all related framing elements be strong enough to force the link to yield, and that they maintain their integrity through the range of forces and displacements developed during the yielding of the link. The braces are the most vulnerable of the framing elements because seismic forces are by far the dominant forces in their design. Other elements, such as columns and collector beams, are less vulnerable, since their seismic loads constitute a smaller percentage of their total loads, and since there are frequently redundant load paths for portions of the forces they carry. The rotation demand on the link beam is a multiple of the lateral drift of the frame as a whole, a multiple that is

Component/Action	*m* Values for Linear Procedures					
	Primary				**Secondary**	
Component/Action	**IO**	**SE**	**LS**	**CP**	**LS**	**CP**
Concentric Braced Frames						
Columns:[1]						
a. Columns in compression[1]	Force-controlled member, use Equations 6-4a or 6-4b.					
b. Columns in tension[1]	1	2	3	5	6	7
Braces in Compression[2]						
a. Double angles buckling in plane	0.8	3.4	6	8	7	9
b. Double angles buckling out of plane	0.8	2.9	5	7	6	8
c. W or I shape	0.8	3.4	6	8	6	8
d. Double channel buckling in plane	0.8	3.4	6	8	7	9
e. Double channel buckling out of plane	0.8	2.9	5	7	6	8
f. Rectangular concrete-filled cold-formed tubes	0.8	2.9	5	7	5	7
g. Rectangular cold-formed tubes						
1. $\frac{d}{t} \leq \frac{90}{\sqrt{F_y}}$	0.8	2.9	5	7	5	7
2. $\frac{d}{t} \geq \frac{190}{\sqrt{F_y}}$	0.8	1.4	2	3	2	3
3. $\frac{90}{\sqrt{F_y}} \leq \frac{d}{t} \leq \frac{190}{\sqrt{F_y}}$	Use linear interpolation					
h. Circular hollow tubes						
1. $\frac{d}{t} \leq \frac{1500}{F_y}$	0.8	2.9	5	7	5	7
2. $\frac{d}{t} \geq \frac{6000}{F_y}$	0.8	1.4	2	3	2	3
3. $\frac{1500}{F_y} \leq \frac{d}{t} \leq \frac{6000}{F_y}$	Use linear interpolation					
Braces in Tension[3]	1	4	6	8	8	10
Eccentric Braced Frames						
a. Beams	Governed by link					
b. Braces	Force-controlled, use Equations 6-4a or 6-4b					
c. Columns in compression	Force-controlled, use Equations 6-4a or 6-4b					
d. Columns in tension	1	2	3	5	6	7

Table 7-10: Acceptance Criteria for Linear Procedures—Braced Frames and Steel Shear Walls

	$\frac{\Delta}{\Delta_y}$		Residual Force Ratio	Deformation					
				Primary				Secondary	
Component/Action	***d***	***e***	***c***	**IO**	**SE**	**LS**	**CP**	**LS**	**CP**
Concentric Braced Frames									
a. Columns in compression[1]	Force-controlled, use Equations 6-4a or 6-4b								
b. Columns in tension[1]	6	8	1.000	1	2.5	4	6	7	8
Braces in Compression[2,3]									
a. Two angles buckle in plane	1	10	0.2	0.8	3.4	6	8	8	9
b. Two angles buckle out of plane	1	9	0.2	0.8	2.9	5	7	7	8
c. W or I shape	1	9	0.2	0.8	3.4	6	8	8	9
d. Two channels buckle in plane	1	10	0.2	0.8	3.4	6	8	8	9
e. Two channels buckle out of plane	1	9	0.2	0.8	2.9	5	7	7	8
f. Concrete-filled tubes	1	8	0.2	0.8	2.9	5	7	7	8
g. Rectangular cold-formed tubes									
1. $\frac{d}{t} \le \frac{90}{\sqrt{F_y}}$	1	8	0.4	0.8	2.9	5	7	7	8
2. $\frac{d}{t} \ge \frac{190}{\sqrt{F_y}}$	1	4	0.2	0.8	1.4	2	3	3	4
3. $\frac{90}{\sqrt{F_y}} \le \frac{d}{t} \le \frac{190}{\sqrt{F_y}}$	Use linear interpolation								
h. Circular hollow tubes									
1. $\frac{d}{t} \le \frac{1500}{F_y}$	1	10	0.4	0.8	2.9	5	7	6	9
2. $\frac{d}{t} \ge \frac{6000}{F_y}$	1	4	0.2	0.8	1.4	2	3	3	4
3. $\frac{1500}{F_y} \le \frac{d}{t} \le \frac{6000}{F_y}$	Use linear interpolation								
Braces in Tension	12	15	0.800	1	4.5	8	10	12	14
Eccentric Braced Frames									
a. Beams	Governed by link								
b. Braces	Force-controlled, use Equations 6-4a or 6-4b								
c. Columns in compression	Force-controlled, use Equations 6-4a or 6-4b								
d. Columns in tension	6	8	1.000	1	2.5	4	6	7	8

Table 7-11: Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Braced Frames and Steel Shear Walls

	$\frac{\Delta}{\Delta_y}$		Residual Force Ratio	Deformation					
				Primary				Secondary	
Component/Action	***d***	***e***	***c***	**IO**	**SE**	**LS**	**CP**	**LS**	**CP**
Link Beam[3]									
a.[4] $\frac{2M_{CE}}{eV_{CE}} \le 1.6$	16	18	0.80	1.5	6.8	12	15	15	17
b. $\frac{2M_{CE}}{eV_{CE}} \ge 2.6$	Same as for beam in FR moment frame (see Table 7-28:)								
c. $1.6 \le \frac{2M_{CE}}{eV_{CE}} \le 2.6$	Use linear interpolation								
Steel Shear Walls[5]	15	17	.07	1.5	6.3	11	14	14	16

1. Columns in moment or braced frames need only be designed for the maximum force that can be delivered.
2. Δ_c is the axial deformation at expected buckling load.
3. Deformation is rotation angle between link and beam outside link or column. Assume Δ_y is 0.01 radians for short links.
4. Link beams with three or more web stiffeners. If no stiffeners, use half of these values. For one or two stiffeners, interpolate.
5. Applicable if stiffeners are provided to prevent shear buckling.

Table 7-11: Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Braced Frames and Steel Shear Walls (Continued)

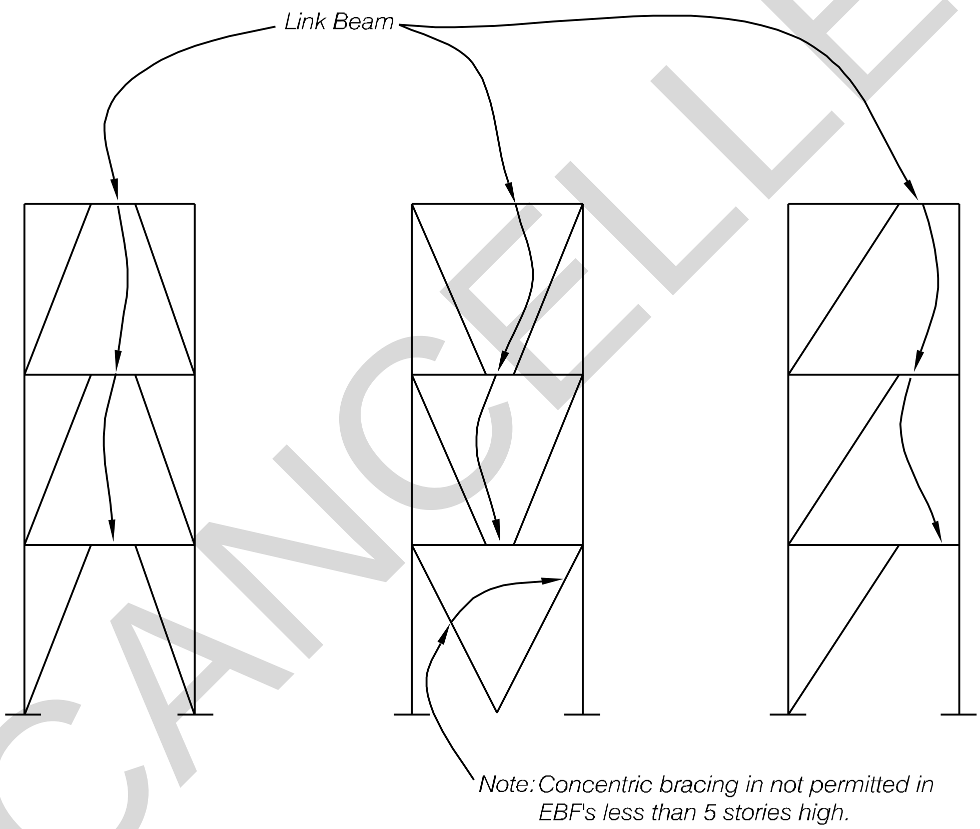


Figure 7-23 Eccentric braced frame configurations

a function of the geometry of the frame (see Figure 7-24). Link beams can yield in shear, in bending, or in both shear and bending at the same time. Which yield mechanism governs is a function of the relationship of link length to the ratio of its bending strength to shear strength. Where the length of the link beam is less than 1.6 M_s/V_s, the yielding is almost entirely in shear. Where the length is greater than 2.6 M_s/V_s, the yielding is primarily in bending. Where the length is between 1.6 M_s/V_s and 2.6 M_s/V_s, both shear and bending yield will occur. Since link beams that yield in shear are considered to have the most stable energy-dissipating characteristics, most of the EBF research has tested the cyclic inelastic capacity of link beams with shear yielding at large rotations. Consequently, most of the design provisions are concerned with limiting the link beam shear yield rotation to less than the maximum cyclic test rotations, and then requiring details indicated by the tests as necessary to ensure that this rotation can occur through a number of cycles without failure.

(4) Design criteria. The specific criteria governing the design of eccentrically braced frames are given in the AISC Seismic Provisions. Additional detail is provided in the following paragraphs.

(a) Link beam location and stability. Link beams are the fuses of the EBF structural system, and are to be placed at locations that will preclude buckling of the braces. A link beam must be located in the intersecting beam at least at one end of each brace. There are exceptions permitting concentric bracing at the roof level and/or at the bottom level of EBF over five stories in the AISC Seismic Provisions. Compact sections meeting the more restrictive flange-width-to-thickness ratio of $52/\sqrt{F_y}$ are required for the beam portions of eccentric braced frames in order to provide the beams with stable inelastic deformation characteristics. The same requirement is used for the beams of special moment-resisting space frames.

(b) Link beam strength. The basic requirement for link beam strength is given in the AISC Seismic Provisions, which states that the shear in the link beam web due to prescribed seismic forces be limited to 0.8 V_s. Paragraph 15.2f of the AISC Seismic Provisions addresses the concern for the effect that substantial axial loads in the link beam could have on its inelastic deflection performance. It presumes that in shear links, the web's capacity is fully utilized in shear, and that flanges provide the needed axial and flexural capacity. Shear links with a length less than 2.2 M_s/V_s are considered to be controlled by shear. Substantial axial loads occur in some EBF configurations when the link beam is required to transmit horizontal forces to or from the braces. It is recommended that, insofar as it is possible, link beams be located so that they are not required to transmit the horizontal force component of braces or drag struts. Where axial forces in the link cannot be avoided, the flexural strength shall be reduced by the axial stress f_a, giving $M_{RS} = Z\,(F_y - f_a)$. The f_a should correspond to the lesser value of the axial force corresponding to yield of the link beam in shear, or that which, when combined with link bending, causes the beam flanges to yield.

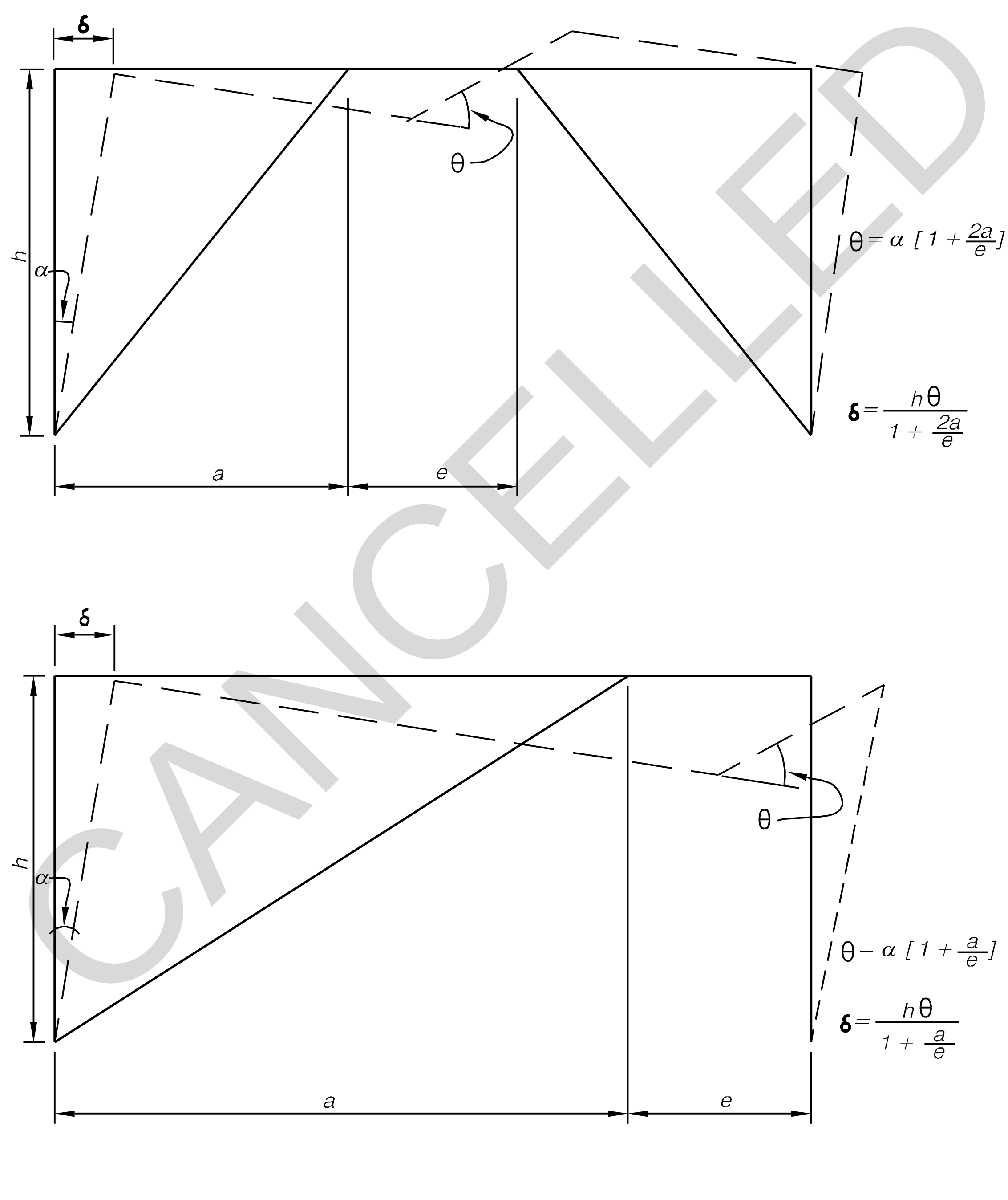


Figure7-24 Deformed frame geometry.

(5) Link beam rotation. The link beam rotation, at a frame drift 0.4*R* times the drift calculated from prescribed seismic forces, is limited to the values given in Paragraph 15.2g of the AISC Seismic Provisions. The procedure for calculating the rotations is as follows (refer to Figure 7-24):

(a) Perform an elastic analysis of the frame for the prescribed seismic forces, being certain that the analysis includes the contribution of the elastic shear deformation of the link beam.

(b) Calculate 0.4R times the drift angle obtained from the analysis in (1). This angle is denoted as ″ in Figure 7-24.

(c) Calculate the rotation angle *2*, as shown in Figure 7-24, for the appropriate configuration. This simplified procedure is slightly conservative, since the elastic curvature of the beam segments between hinges and of the brace deformations have been ignored, and would contribute a minor amount of the required deformation. It should be noted that calculation of the rotation by multiplying the elastic deflections of the link beam by 0.4*R* would be unconservative, since these deflections include elastic effects, such as the axial deformation of the braces, that would not increase proportionally after the link begins to yield.

(d) Link-beam web. Link-beam web doubler plates are prohibited in AISC Seismic Provisions because tests have shown that they are not fully effective. The performance of eccentric braced frames relies on the predictability of the strength and strain characteristics of the link beam. It is not considered advisable to complicate the behavior of the link beam by permitting doublers or allowing holes within it.

(e) Brace sizing. Once the link beam size has been selected, the brace size is determined by the requirement given in the AISC Seismic Provisions that its compressive strength be at least 1.5 times the axial force corresponding to the controlling strength of the link beam. The controlling strength is either the shear strength V_s or the reduced flexural strength M_{RS} described above, whichever results in the lesser force in the brace. Note that once the link beam is selected, the brace forces are determined from its strength, and the brace forces calculated in the elastic analysis will not govern, and will not be used in the brace design.

(f) Brace-to-beam connection. The AISC Seismic Provisions require that the brace-to-beam connection develop the compressive strength of the brace, and that no part of the brace-to-beam connection extend into the web area of the link. The required development may be at the strength level of the connection. The prohibition of the extension of the brace-to-beam connection into the link beam is intended to prevent physical attachments that might alter the strength and deflection characteristics of the link beam. It is not intended to prevent the centerline intersection of brace and link beam from intersecting within the link.

(g) Column sizing. FEMA 302 requires that the columns remain elastic at 1.25 times the forces causing yield of the link beam. "Remain elastic at" means the same as "have the strength to resist." The strength, including bending moments, can be calculated using Part 2 of AISC "Specifications for Structural Steel Buildings."

(h) Beam-to-column connections. For link beams that are adjacent to a column, special connection criteria are given in Section 15.4 of the AISC Seismic Provisions. Where the link beam is not adjacent to the column, a simpler criterion for connection is given in Section 15.7 of the AISC Seismic Provisions. Where the simpler connections are used, consideration must be given to transmission of collector forces into the EBF bay.

(i) Intermediate stiffeners. Section 15.3 of the AISC Seismic Provisions provides requirements for various types of stiffeners necessary for the intended performance of the link beams. Stiffener plates as described in those paragraphs are required at the following locations (see Figure 7-25):

1. At the brace end(s) of the link beam.

2. At b_f from each end where link beam length is between 1.6 M_s/V_s and 2.6 M_s/V_s.

3. At intermediate points along the link beam where shear stresses control or are high.

(6) Acceptance criteria.

(a) Response modification factors, *R*, for Performance Objective 1A, are provided in Table 7-1.

(b) Modification factors, *m*, for enhanced performance objectives, are provided in Table 7-12 for beams, columns, and fully restrained moment connections; in Table 7-13 for partially restrained moment connections; and in Table 7-10 for braces and link beams.

(c) Modeling parameters and numerical acceptance criteria for nonlinear procedures are provided in Table 7-11 for deformation-controlled components.

(d) Expected strength of deformation-controlled components and lower-bound strength of force-controlled components shall be determined as indicated in Paragraph 7-3b(9).

7-4. Concrete Moment-Resisting Frames.

a. General.

(1) Function. Moment frames, like shear walls, are vertical elements in a lateral-force-resisting system that transmit lateral forces to the ground; however, they differ from shear walls in that their deflections result primarily from flexural deformations of their elements.

(2) Frame behavior. The bending stiffness of the moment-resisting frame provides the lateral stability of the structure (Figure 7-26). It is important to remember that deformations resulting from the dynamic response to a major earthquake are much greater than those determined from the application of the prescribed design forces. This

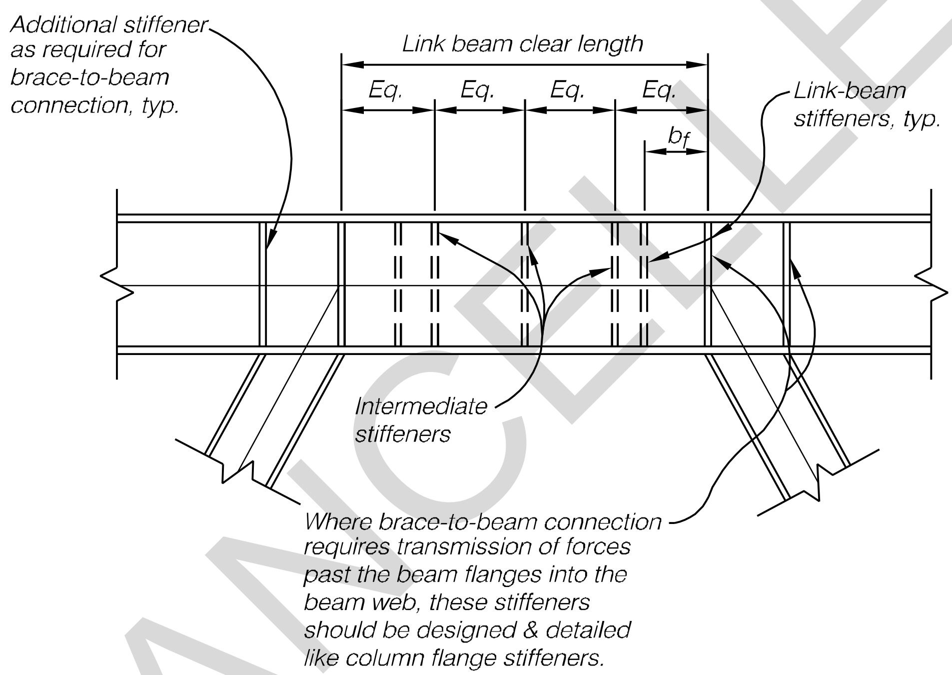


Figure 7-25 Link beam and intermediate stiffeners.

Component/Action	*m* Values for Linear Procedures[8]					
	Primary				Secondary	
	IO m	SE m	LS m	CP m	LS m	CP m
Moment Frames						
Beams:						
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	2	4	6	8	10	12
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	1	1.5	2	3	3	4
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation						
Columns:						
For $P/P_{ye} < 0.20$						
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	2	4	6	8	10	12
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	1	1	1	2	2	3
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation						
For $0.2 \le P/P_{ye} \le 0.50$[9]						
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	1	—[1]	—[2]	—[3]	—[4]	—[5]
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	1	1	1	1.5	2	2
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation						
Panel Zones	1.5	4.8	8	11	NA	NA
Fully Restrained Moment Connections[7]						

1. $m = 6\,(1 - 1.7\,P/P_{ye})$
2. $m = 9\,(1 - 1.7\,P/P_{ye})$
3. $m = 12\,(1 - 1.7\,P/P_{ye})$
4. $m = 15\,(1 - 1.7\,P/P_{ye})$
5. $m = 18\,(1 - 1.7\,P/P_{ye})$
6. $m = 5 - 0.125\,d_b$
7. $m = 6 - 0.125\,d_b$
8. $m = 7 - 0.125\,d_b$
9. If construction documents verify that notch-tough rated weldment was used, these values may be multiplied by two.
10. For built-up numbers where strength is governed by the facing plates, use one-half these *m* values.
11. If $P/P_{ye} > 0.5$, assume column to be force-controlled.

Table 7-12: Acceptance Criteria for Linear Procedures—Fully Restrained (FR) Moment Frames

Component/Action	*m* Values for Linear Methods					
	Primary				Secondary	
	IO	SE	LS	CP	LS	CP
Partially restrained moment connection						
For top and bottom clip angles[1]						
a. Rivet or bolt shear failure[2]	1.5	2.8	4	6	6	8
b. Angle flexure failure	2	3.5	5	7	7	14
c. Bolt tension failure[2]	1	1.3	1.5	2.5	4	4
For top and bottom T-stub[1]						
a. Bolt shear failure[2]	1.5	2.8	4	6	6	8
b. T-stub flexure failure	2	3.5	5	7	7	14
c. Bolt tension failure[2]	1	1.3	1.5	2.5	4	4
For composite top and clip angle bottom[1]						
a. Yield and fracture of deck reinforcement	1	1.5	2	3	4	6
b. Local yield and web crippling of column flange	1.5	2.8	4	6	5	7
c. Yield of bottom flange angle	1.5	2.8	4	6	6	7
d. Tensile yield of column connectors or OSL of angle	1	1.3	1.5	2.5	2.5	3.5
e. Shear yield of beam flange connections	1	1.8	2.5	3.5	3.5	4.5
For flange plates welded to column bolted or welded to beam[1]						
a. Failure in net section of flange plate or shear failure of bolts or rivets[2]	1.5	2.8	4	5	4	5
b. Weld failure or tension failure on gross section of plate	0.5	1.0	1.5	2	1.5	2
For end plate welded to beam bolted to column						
a. Yielding of end plate	2	3.8	5.5	7	7	7
b. Yield of bolts	1.5	1.8	2	3	4	4
c. Failure of weld	0.5	1.0	1.5	2	3	3

1. Assumed to have web plate or stiffened seat to carry shear. Without shear connection, this may not be downgraded to a secondary member. If $d_b > 18$ inches, multiply *m* values by $18/d_b$.
2. For high-strength bolts, divide these values by two.

Table 7-13: Acceptance Criteria for Linear Procedures—Partially Restrained (PR) Moment Frames

18 inches = 457mm

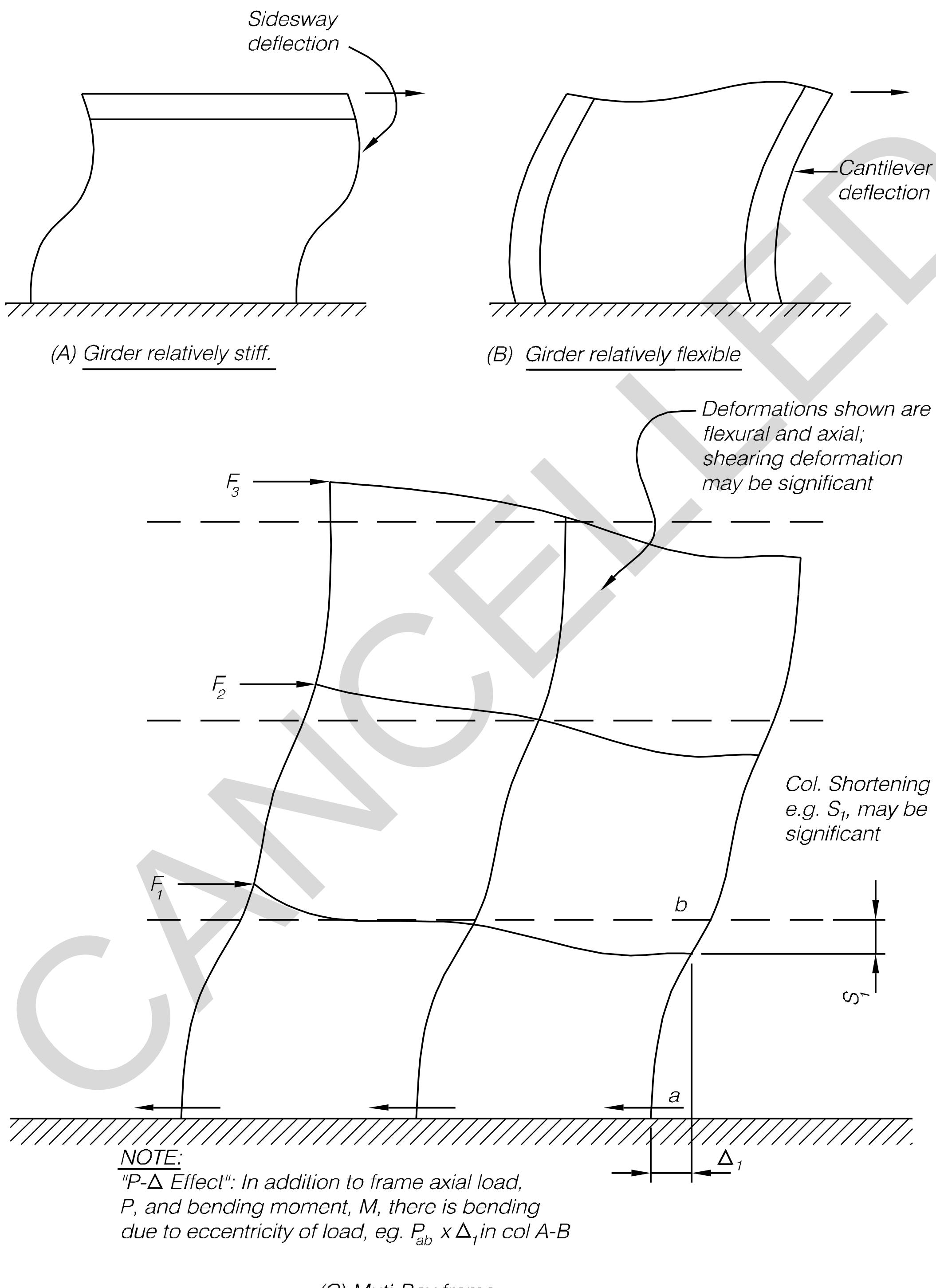


Figure 7-26 Frame deformations.

means that a frame meeting the minimum strength requirements of this manual will survive a major earthquake only if it can yield and sustain cyclic inelastic deformations without essential loss of lateral resistance and vertical load capacity. Since normal building materials have very limited energy-absorbing capacity in the elastic range of action, it follows that what is needed is a large energy capacity in the inelastic range. The term "ductility" is used to denote this property. Providing a ductile seismic frame will allow the structure to sustain tolerable, and in many cases, repairable damage, instead of suffering catastrophic failure. The energy dissipation, ductility, and structural response (deformation) of moment-resisting frames depend upon the types of members, connections (joints), and materials of construction used. The behavior of joints is a critical factor in the ability of building frames to resist high-intensity cyclic loading.

(3) Mechanical and welded splices. See Paragraph 7-2a(3) for revisions to ACI 318 provisions regarding mechanical and welded splices in reinforcement.

b. Classification of Concrete Moment-Resisting Frames. FEMA 302 classifies concrete moment-resisting frames as Ordinary Moment Frames (OMF), Intermediate Moment Frames (IMF), or Special Moment Frames (SMF). Restrictions regarding the use of the various frame classifications are summarized in Table 7-1, which also provides the appropriate *R* value for each classification.

c. Nonseismic Frames. Frame members assumed not to contribute to lateral resistance shall be detailed according to Section 21.7.2 or 21.7.3 of ACI 318, depending on the magnitude of the moment induced in those members when subjected to the calculated displacements in FEMA 302. When the effects of lateral displacement are not explicitly checked, the provisions of Section 21.7.3 shall apply.

d. Ordinary Moment Frames (OMF) are reinforced concrete moment frames conforming to the provisions of ACI 318, exclusive of Appendix A.

(1) Flexural members of OMF's forming part of a seismic-force-resisting system shall be designed in accordance with Section 7.13.2 of ACI 318, and at least two main flexural reinforcing bars shall be provided continuously top and bottom throughout the beams through, or developed within, exterior columns or boundary elements.

(2) Columns of OMFs having a clear height-to-maximum plan dimension ratio of 5 or less shall be designed for shear in accordance with Section 21.8.3 of ACI 318.

e. *Intermediate Moment Frames (IMFs)* are frames conforming to the requirements of Sections 21.1, 21.2.1.1, 21.2.1.2, 21.2.2.3, and 21.8 of ACI 318, in addition to the requirements of OMFs. Flat-plate or two-way slabs are permitted for the beam elements of IMFs. These slab systems have a potential for a brittle mode of punching shear failure at the column supports due to gravity load combined with the eccentric shear caused by moment transferred from the slab to the column. In order to prevent punching shear failure under the maximum expected earthquake deformation, the slab shall be designed in accordance with Section 21.8 of ACI

318. Details illustrating these requirements are presented in Figures 7-27 through 7-32.

f. *Special Moment Frames (SMFs)* are frames conforming to the requirements of Sections 21.1 through 21.5 of ACI 318, in addition to the requirements of OMFs.

(1) General design requirements. The basic concept of SMFs is to provide inelastic energy dissipation by flexural yielding in the girder elements. Columns must, therefore, be stronger than the flexural capacity of the girders, and all elements must have shear resistance and reinforcing bar anchorage capacity capable of developing the full flexural yield level in the girders. In order to provide the girder yield mechanism, the design provisions require:

(a) Compact proportions for the girder and column sections, along with closely spaced seismic ties or hoops for confinement of concrete in the regions of potential flexural yielding.

(b) Column interaction flexural capacity greater than 6/5 times the value required to develop girder yield.

(c) Girder, column, and joint shear capacity greater than shears induced by gravity loads and the strain-hardened flexural capacity of the girders.

(d) Reinforcing bar splices and straight and hooked bar anchorages capable of developing the strain-hardened yield of the girder steel.

(e) Details illustrating the above requirements are presented in Figure 7-33 through 7-40.

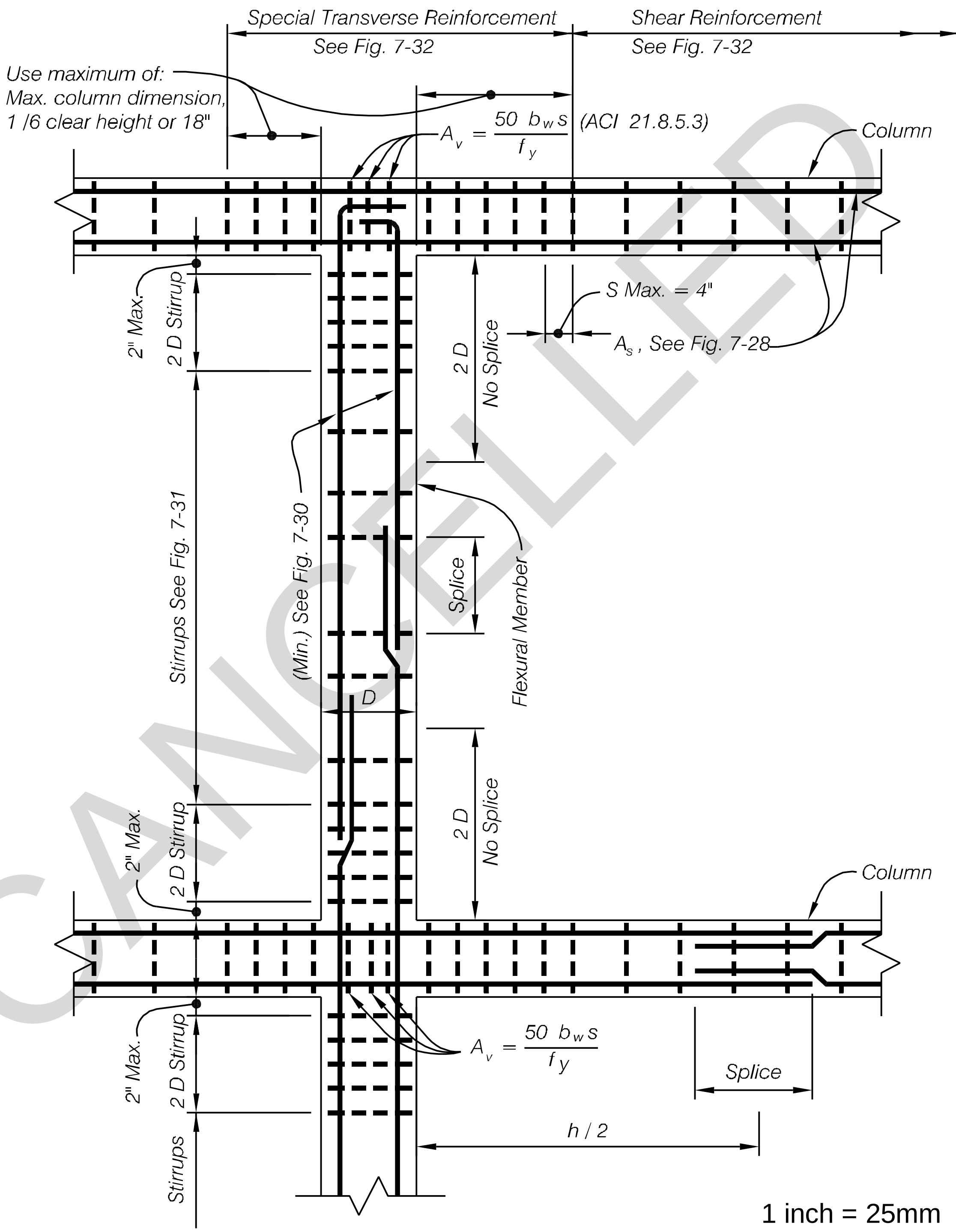
(2) The two phases of design. With the design concept that inelastic behavior and energy dissipation are to be restricted to flexural yielding in the confined concrete regions of the beam or girder elements, the design process consists of two phases. The first phase establishes the beam sizes and capacities needed to resist the specified factored gravity and seismic load combinations. Then, with the known girder strengths and some preliminary column sizes, the second phase proportions the shear resistance of the girders, columns, and joints, and establishes the column flexural strengths such that all of these elements are able to resist the effects of a strain-hardened flexural yielding in the beams along with unfactored gravity loads.

g. *Acceptance Criteria.*

(1) Response modification factors, *R*, for Performance Objective 1A, for concrete frames in various structural systems are provided in Table 7-1.

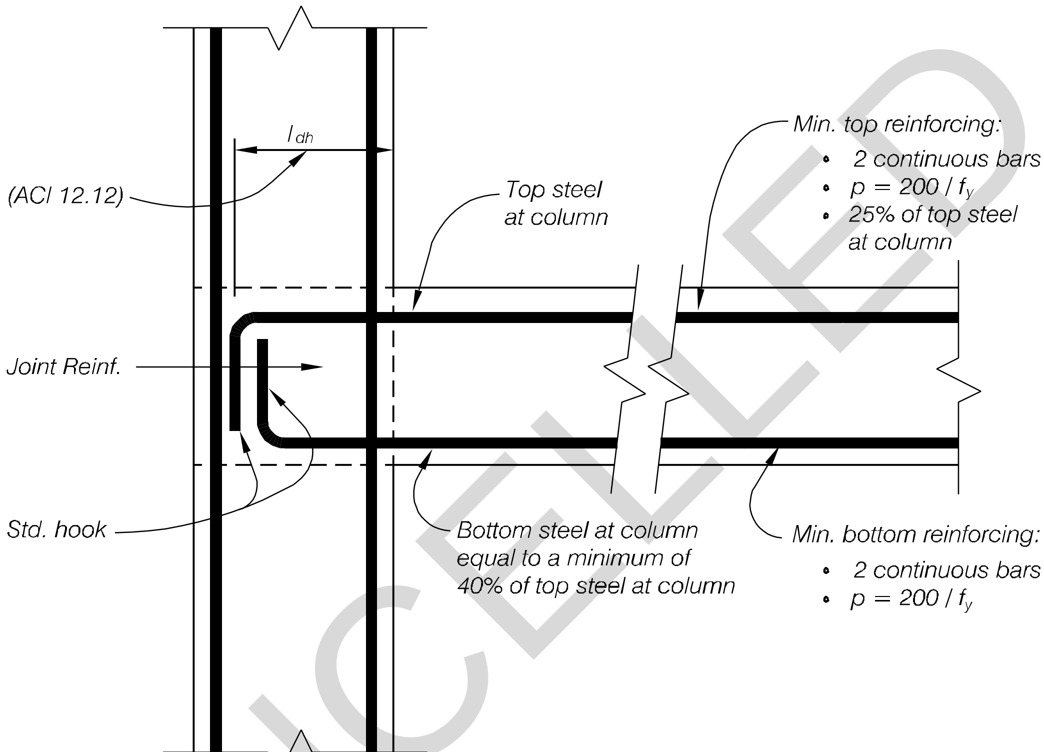
(2) Modification factors, *m*, for enhanced performance objectives are provided in Table 7-14 for beams; Table 7-15 for columns; Table 7-16 for beam/column joints; and Table 7-17 for slab/column frames.

(3) Modeling parameters and numerical acceptance criteria for nonlinear procedures are provided in Table 7-18 for beams; in Table 7-19 for



1 inch = 25mm

Figure 7-27 Intermediate Moment Frame frame requirements.



ELEVATION OF GIRDER AND COLUMN

FLEXURAL MEMBER:

f'_c = 3,000 p.s.i. min. at 28 days
f_y = 40 ksi or 60 ksi
Reinforcement ratio $p = A_S / bd$ or $p' = A'_S / bd$: $p = 0.025$ max.

1 ksi = 6.89 MPa

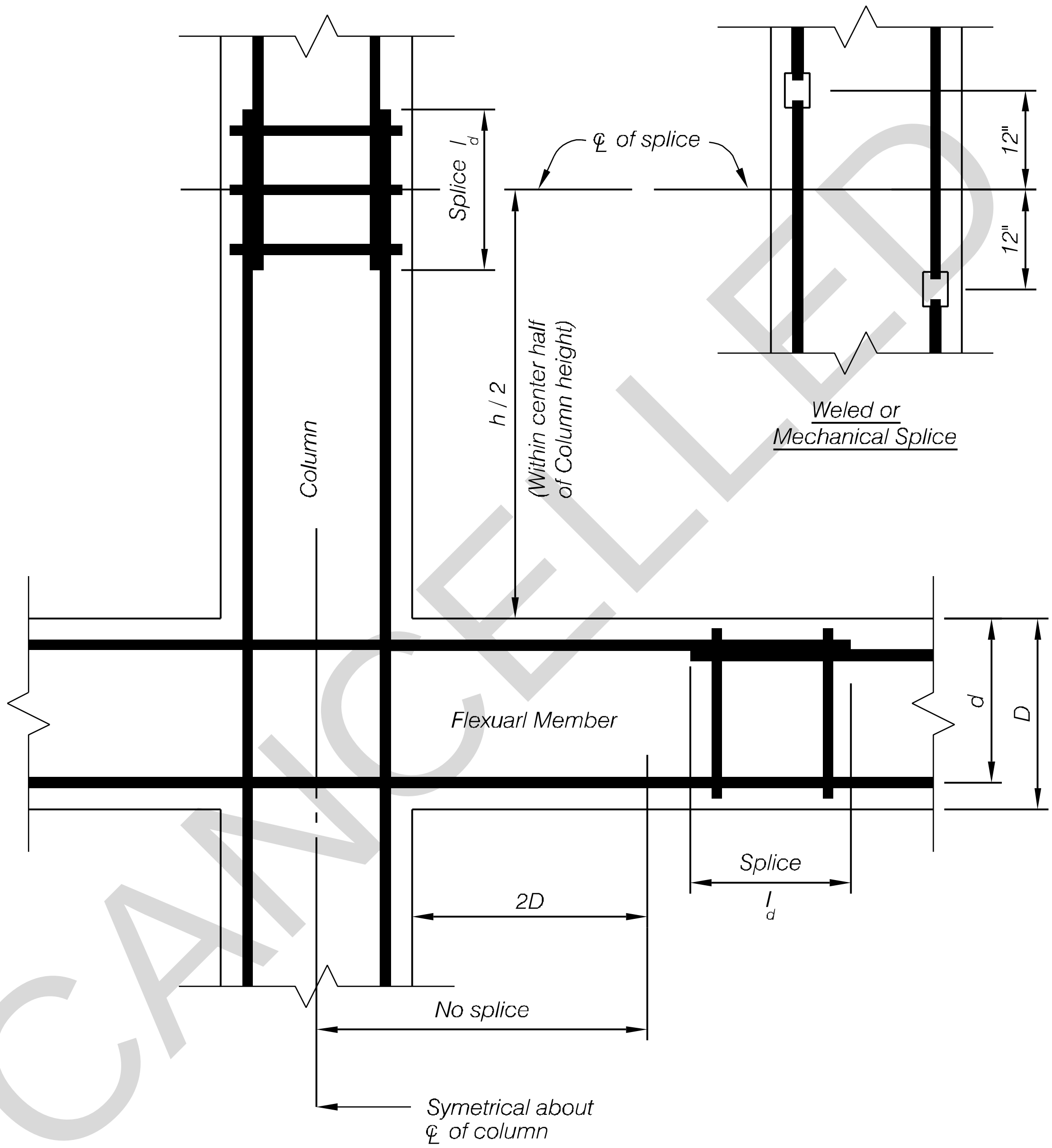
COLUMN:

f'_c = 3,000 p.s.i. min. at 28 days
f_y = 40 ksi or 60 ksi
Reinforcement ratio, p (for tied columns)

$0.01 \leq p \leq 0.06$

1 ksi = 6.89 Mpa

Figure 7-28 Intermediate Moment Frame longitudinal reinforcement



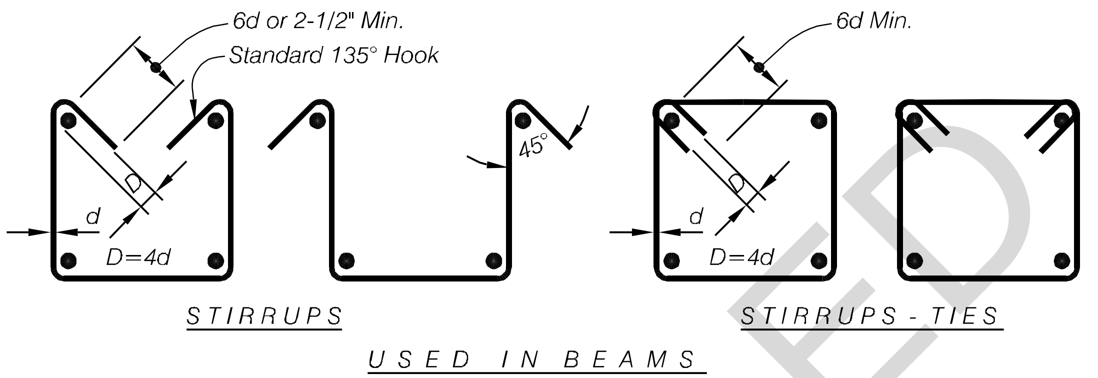
1 inch = 25mm

Column:

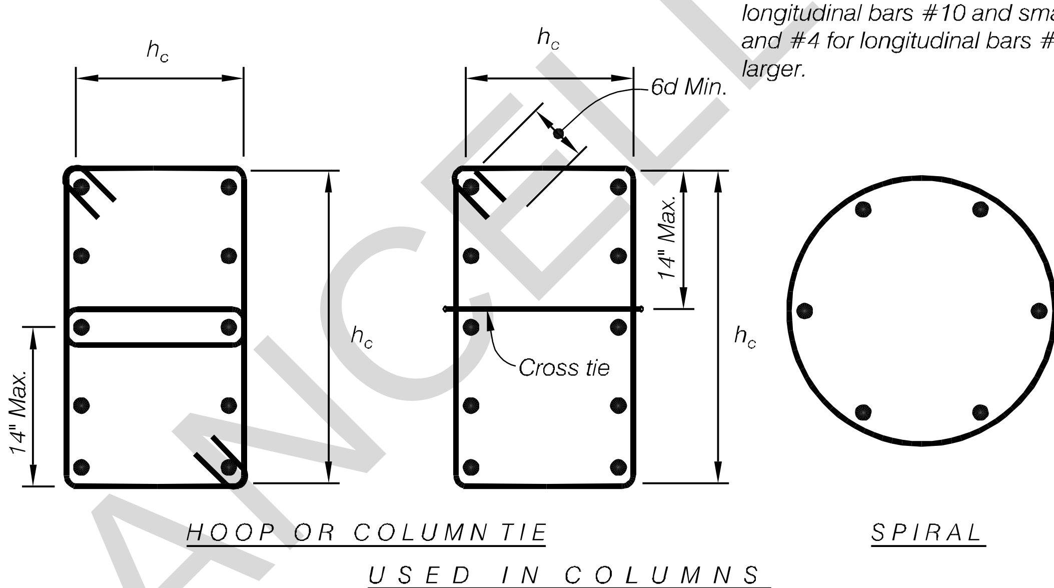
l_d Is the development length.

At any level, not more than alternate bars will be welded or mechanical spliced. Min. distance between two adjacent bar splices = 24".

Figure 7-29 Intermediate Moment Frame splices in reinforcement



Min. hoop and tie size is #3 for longitudinal bars #10 and smaller, and #4 for longitudinal bars #11 or larger.



Spiral Ratio:

$$\rho_s = 0.08 \frac{f'_c}{f_{yh}} \quad \text{or} \quad 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}$$

Whichever is greater.

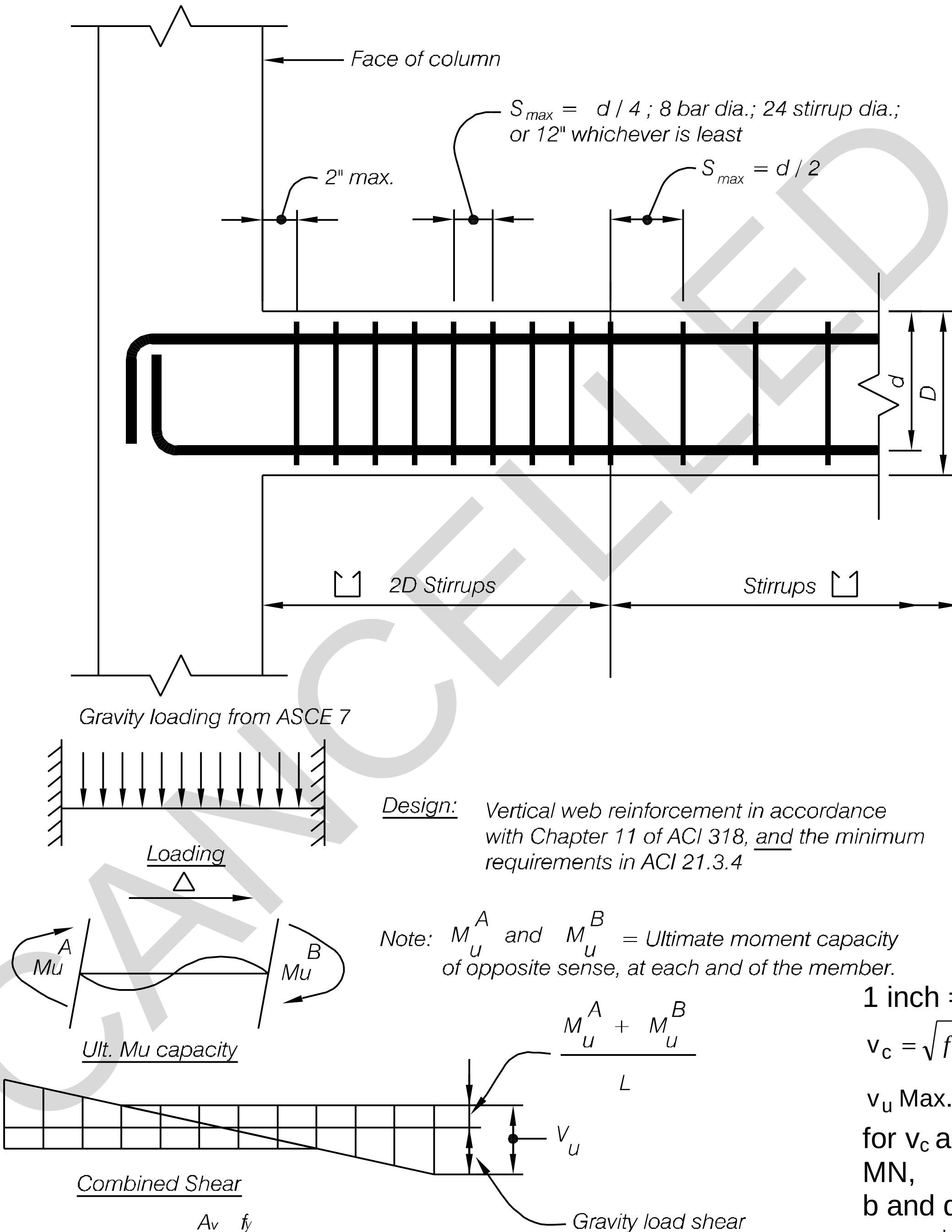
Hoop Requirements -Total Tie Area:

$$A_{sh} = 0.08\ sh_c \frac{f'_c}{f_{yh}}$$

Functions	Stirrups	Stirrup-Ties	Column Ties	Hoops	Spirals
Shear Reinforcement and "Caging"	•	•	•	•	•
Restrain Longitudinal Steel from Buckling		•	•	•	•
Confine Concrete				•	•

1 inch = 25mm
#4 bar ≈ 10M bar
#9 bar ≈ 30M bar
#11 bar ≈ 35M bar

Figure 7-30 Intermediate Moment Frame transverse reinforcement



$$v_u - v_c = v_s = \frac{A_v \; f_y}{Sb}$$

$$v_c = 2\sqrt{f'_c} \qquad v_u = \frac{V_u}{\text{Ø}\, bd}$$

$$\text{Ø} = 0.85$$

$$v_u \text{ Max.} = 10\sqrt{f'_c}$$

1 inch = 25mm

$v_c = \sqrt{f_c'}/6$ and

v_u Max. $= 0.83\sqrt{f_c'}$

for v_c and v_u in MN,

b and d in mm,

and f_c' in MPa

Figure 7-31 Intermediate Moment Frame Girder Web reinforcement

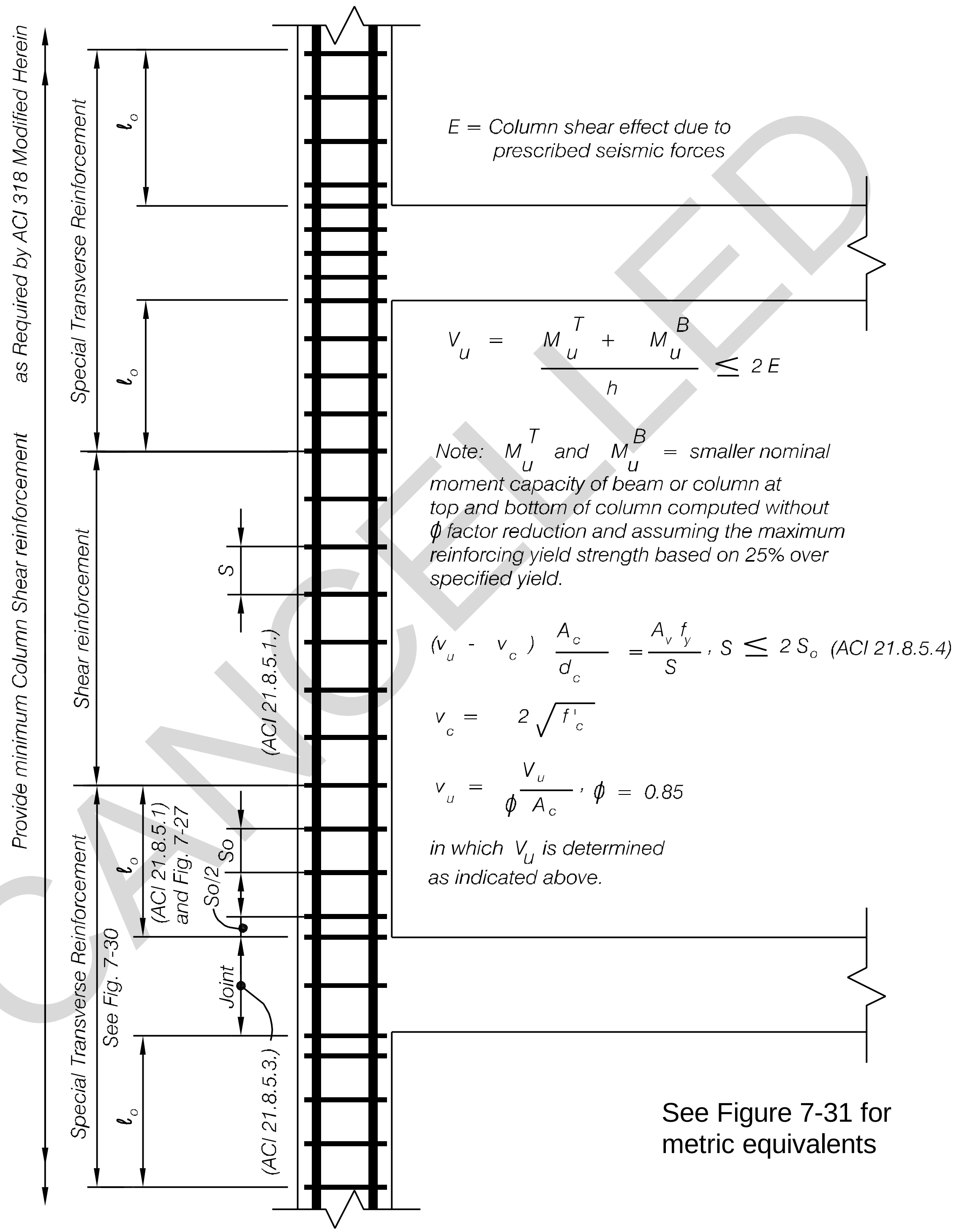


Figure 7-32 Intermediate Moment Frame transverse reinforcement

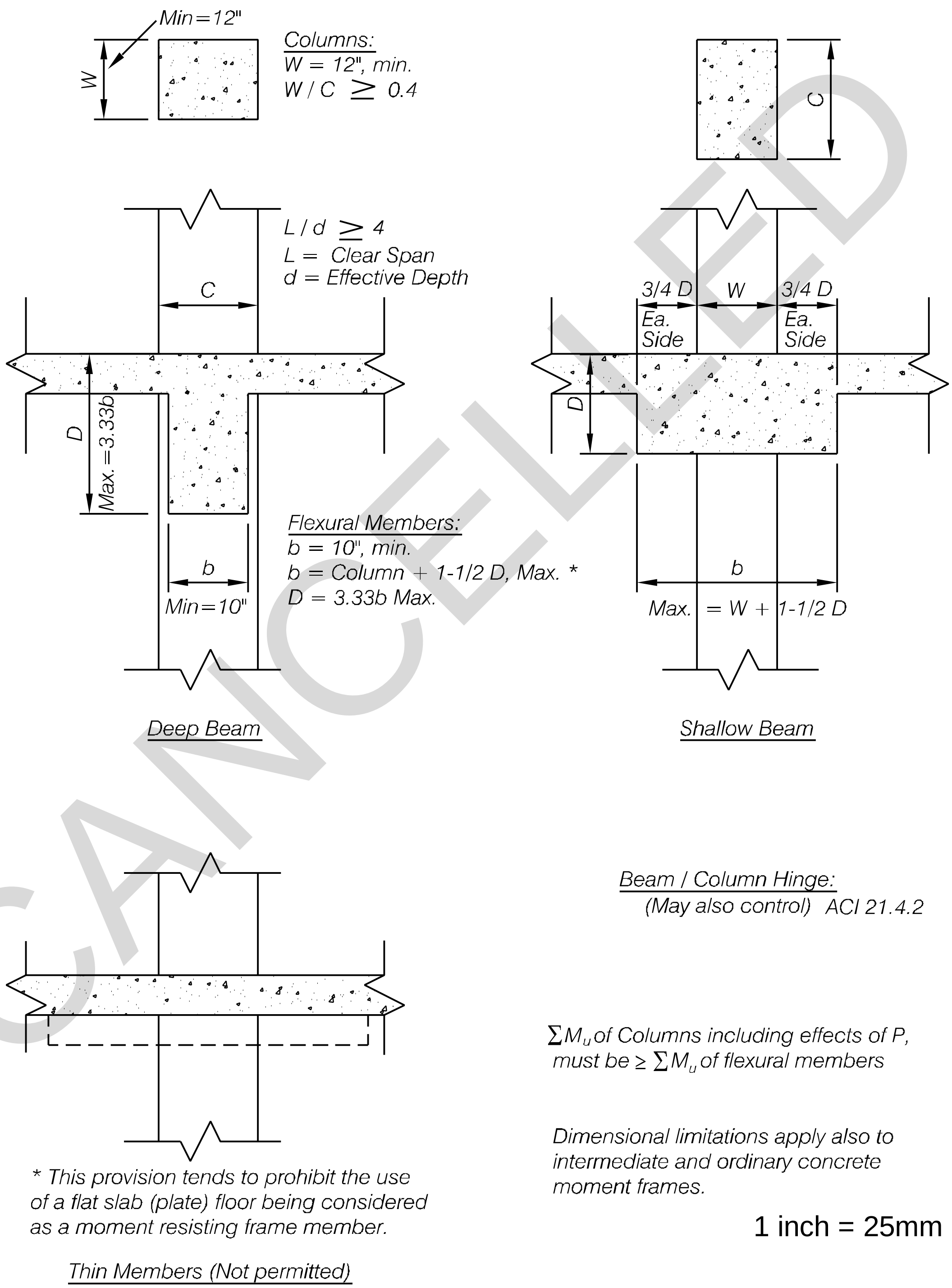
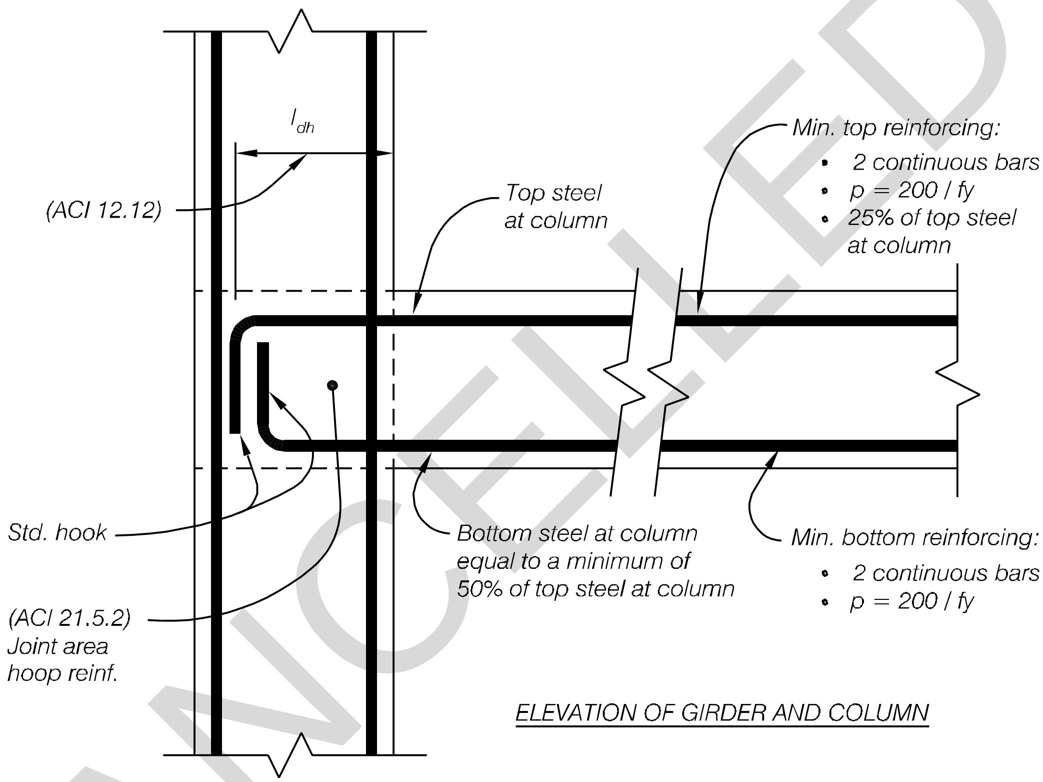


Figure 7-33 Special Concrete Moment Frame - limitations on dimensions.



ELEVATION OF GIRDER AND COLUMN

FLEXURAL MEMBER:

f'c = 3,000 p.s.i. min. at 28 days
fy = 40 ksi or 60 ksi
Reinforcement ratio $p = A_s / bd$ *or* $p' = A'_s / bd$: $p = 0.025$ *max.*

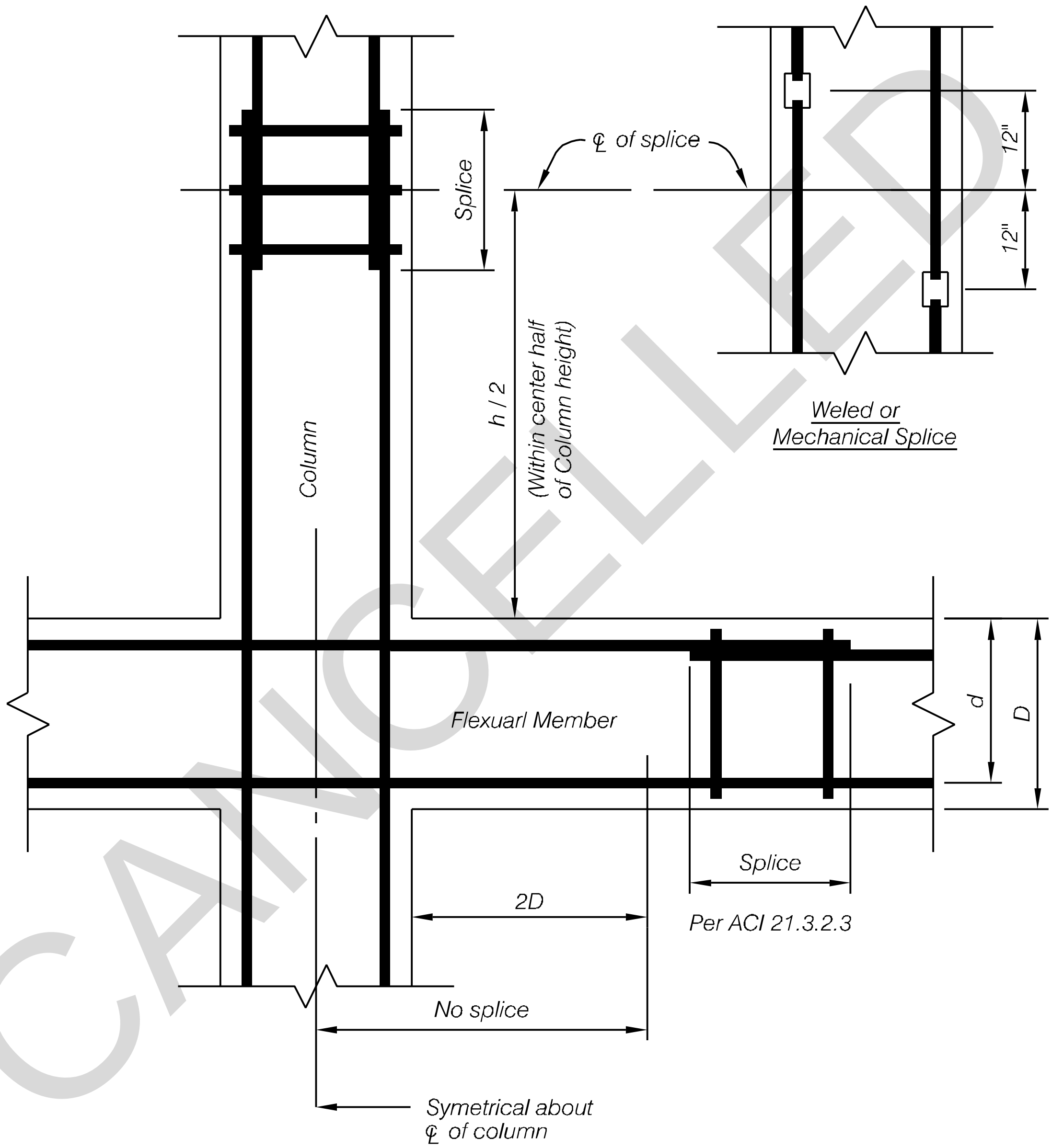
COLUMN:

f'_c *= 3,000 p.s.i. min. at 28 days*
$f_y = 40$ *ksi or 60 ksi*
Reinforcement ratio, p (for tied columns)

$0.01 \leq p \leq 0.06$

1 ksi = 6.89

Figure 7-34 Special Concrete Moment Frame longitudinal reinforcement



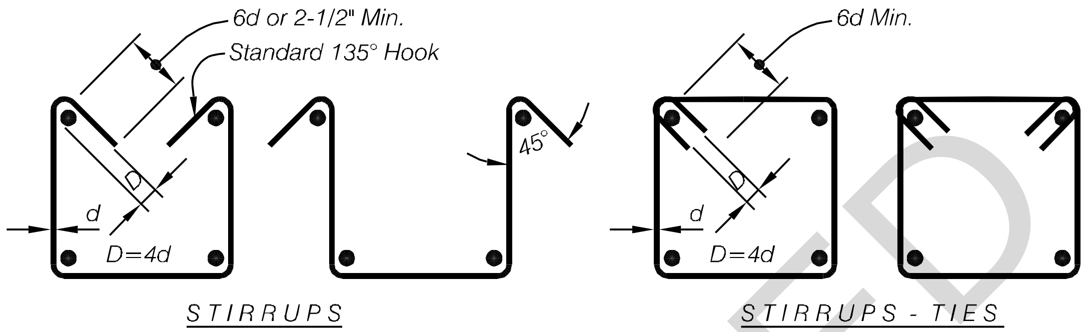
Column:

l_d *Is the development length. See ACI 318-95 Sect. 12.2*

1 inch = 25mm

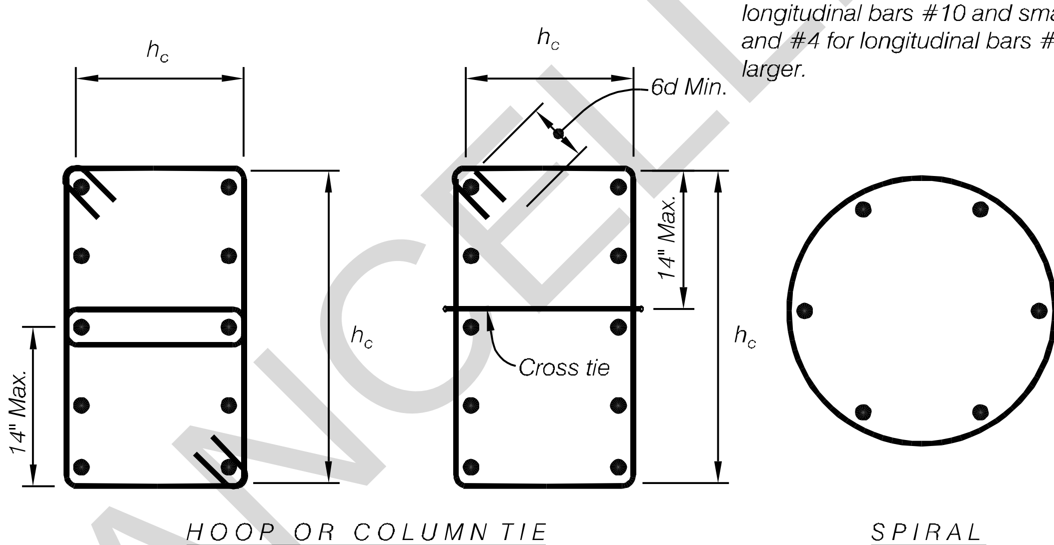
At any level, not more than alternate bars will be welded or mechanical spliced. Min. distance between two adjacent bar splices = 24".

Figure 7-35 Special Moment Frame splices in reinforcement



USED IN BEAMS

Min. hoop and tie size is #3 for longitudinal bars #10 and smaller, and #4 for longitudinal bars #11 or larger.



USED IN COLUMNS

Spiral Ratio:

$$\rho_s = 0.12 \frac{f'_c}{f_{yh}} \text{ or } 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}$$

Whichever is greater.

Functions	Stirrups	Stirrup-Ties	Column Ties	Hoops	Spirals
Shear Reinforcement and "Caging"	•	•	•	•	•
Restrain Longitudinal Steel from Buckling		•	•	•	•
Confine Concrete				•	•

Hoop Requirements -Total Tie Area:

$$A_{sh} = 0.3\left(\frac{sh_c}{f_{yh}}\right)\left(\frac{A_g}{A_{ch}} - 1\right)$$

Formula 21 - 3

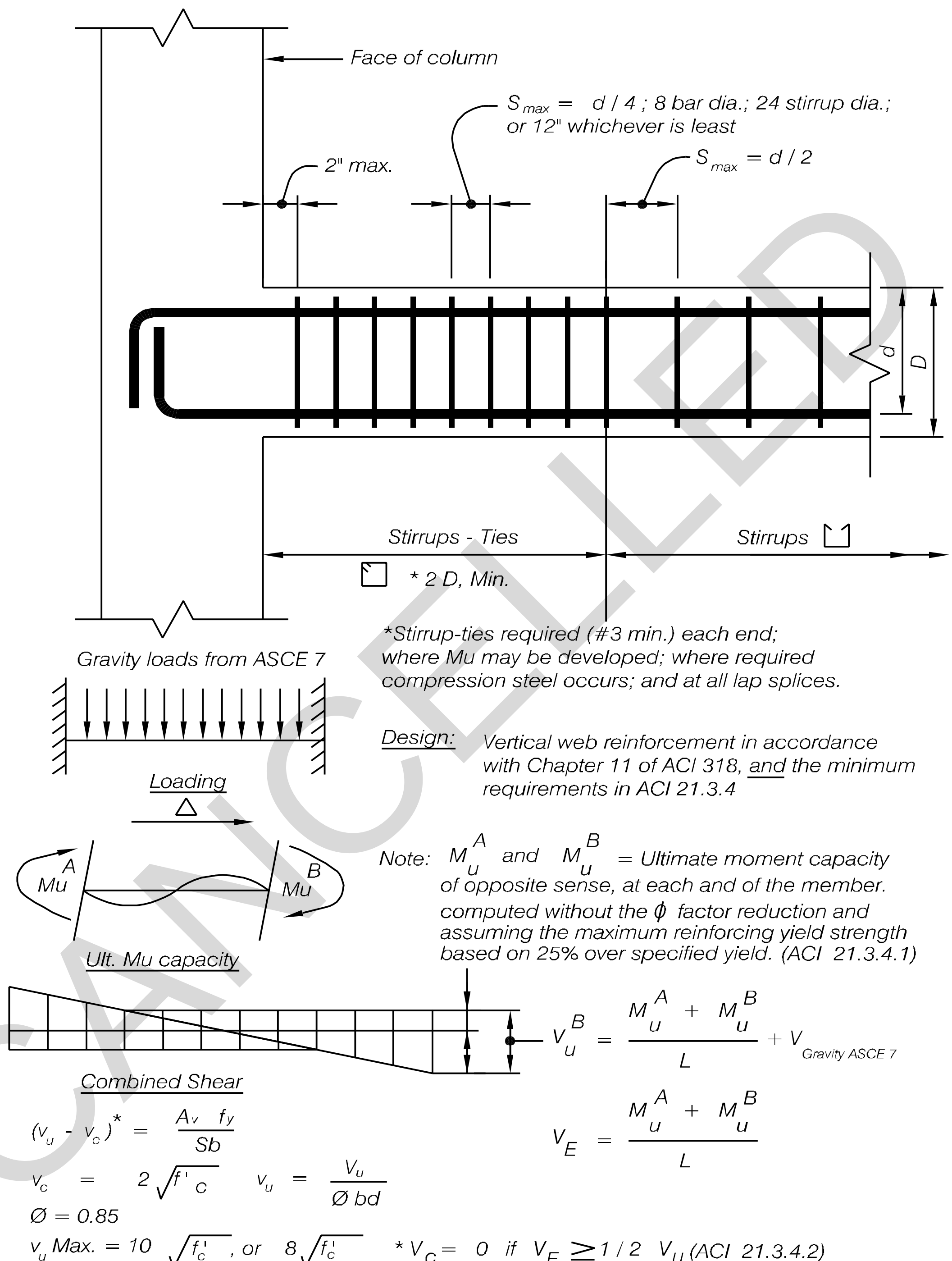
Formula 21 - 4

$$A_{sh} = 0.09 \; sh_c \frac{f'_c}{f_{yh}}$$, whichever is greater.

Provide hoops or spirals in columns where special transverse reinforcement is required. (ACI 21.4.4)

See Figure 7-30 for metric

Figure 7-36 Special Moment Frame - transverse reinforcement



See Figure 7-31 for

Figure 7-37 Special Moment Frame Girder Web reinforcement

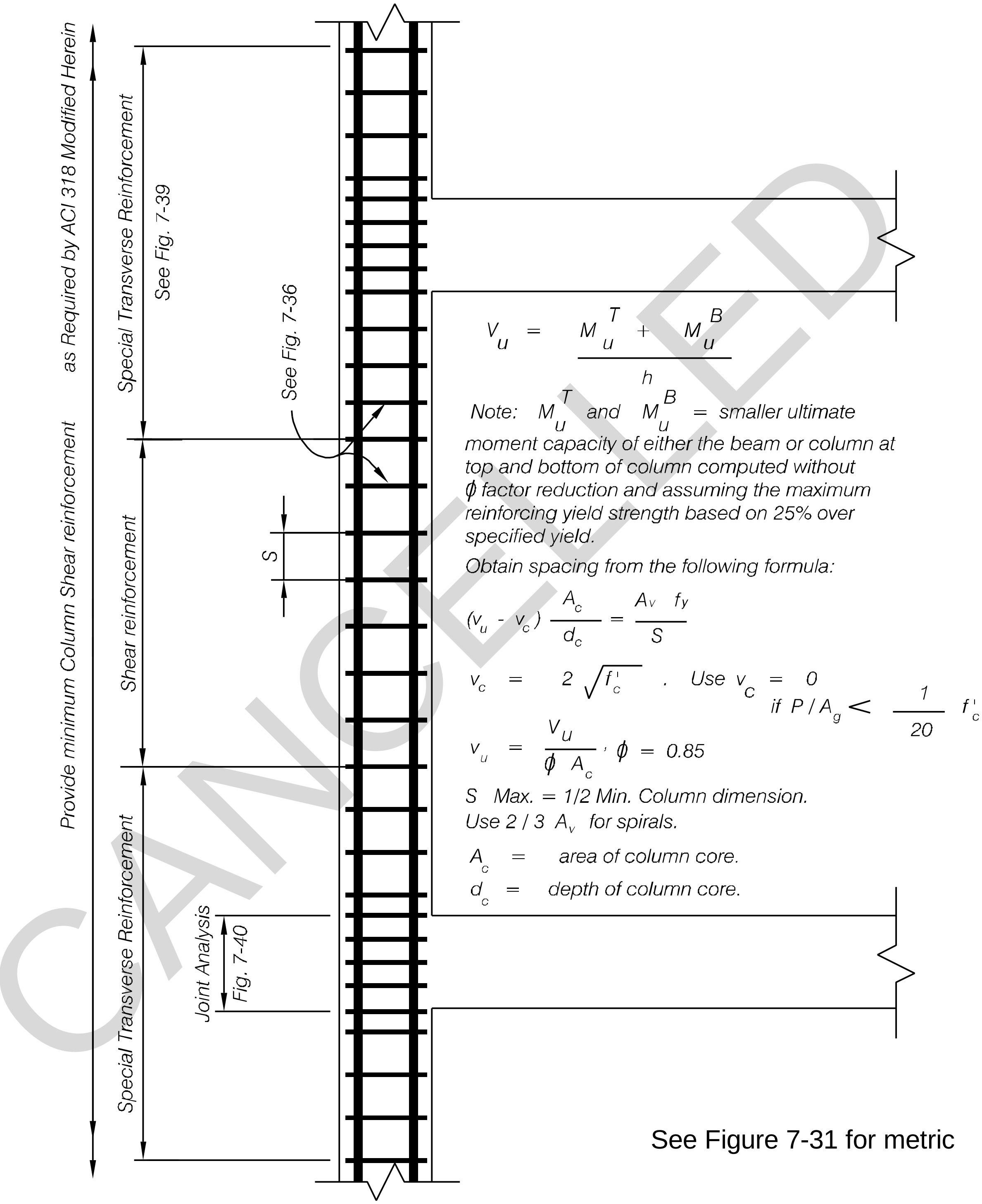
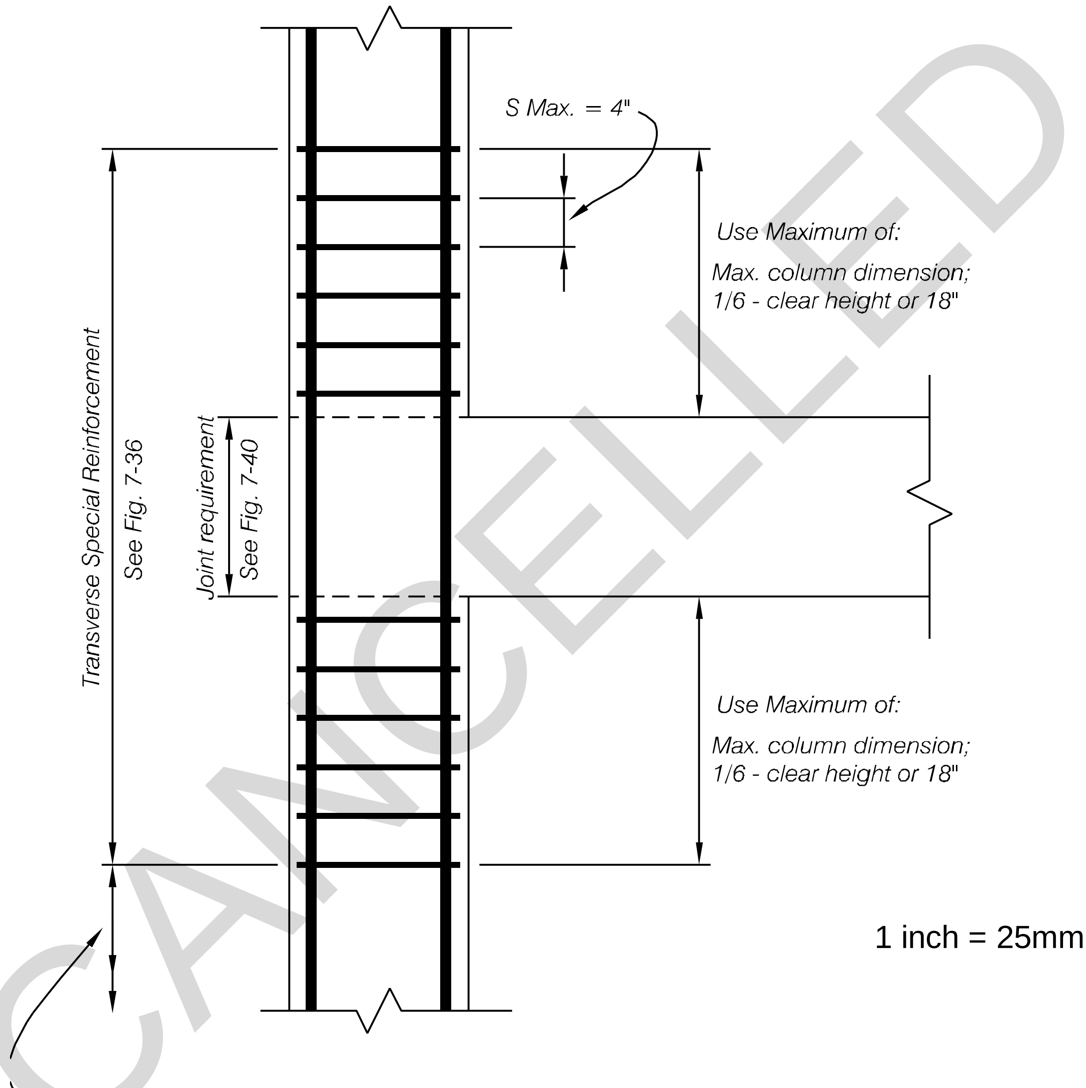
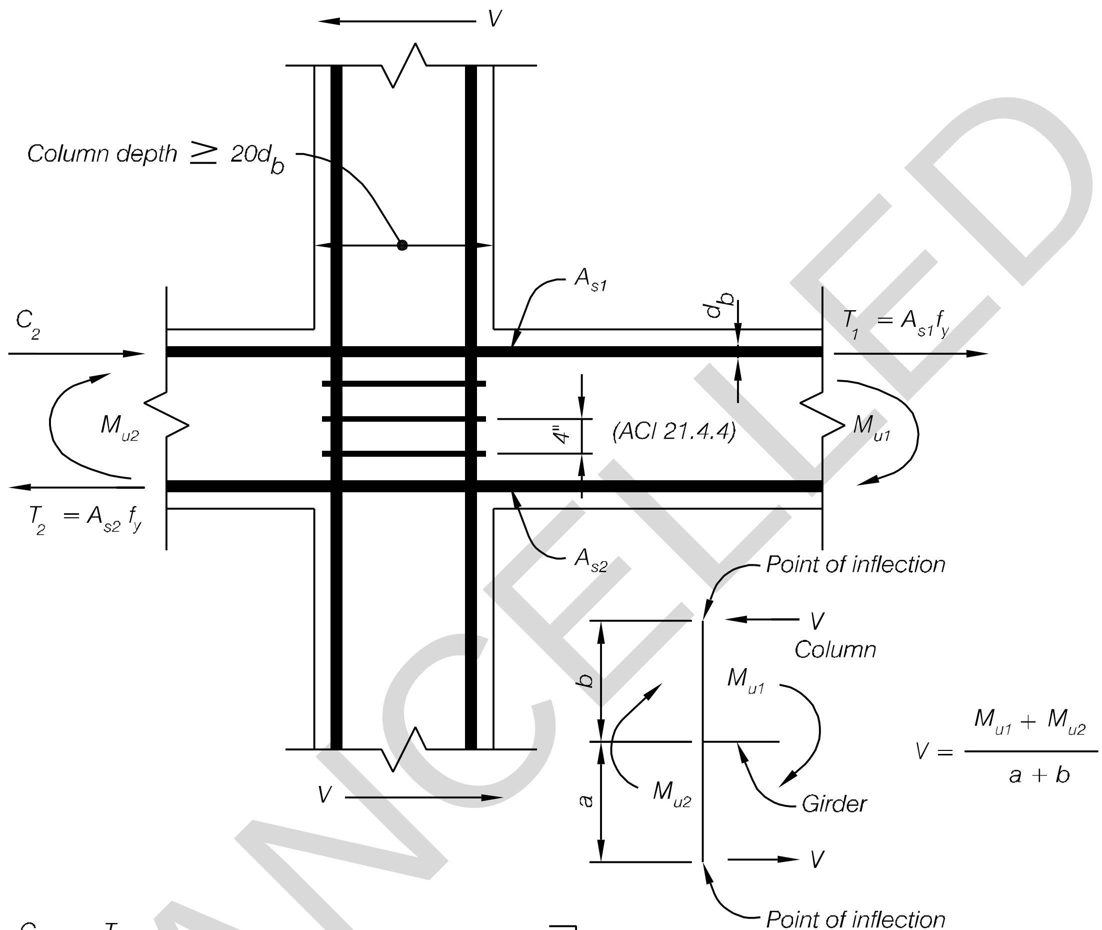


Figure 7-38 Special Moment Frame transverse reinforcement



At any section where the ultimate capacity of the column (P_u) is less than sum of the beam shears (ΣV_u) computed by $Vu = \dfrac{Mu^A + Mu^A}{L} + V_{Grav\ ASCE\ 7}$ *for all the beams. Above the level under consideration, confining reinforcement shall be provided (See Fig. 7-36). This confining reinforcement is also required when point of contra-flexure not in middle half of column (ACI 21.4.4.5) for columns supporting discontinued stiff members, such as walls.*

Figure 7-39 Special Moment Frame - special transverse reinforcement.



$C_2 = T_2$

$V_u = T_1 + C_2 - V$

$v_u = \dfrac{V_u}{A_J}$ Where A_J is defined in ACI 21.5.3.

Modified to concur with FEMA 302 and ACI 318-95

$S = 4"$ max. for non-confined joints (ACI 21.5.2.1)

only 1/2 the special transverse reinforcement is required for confined joints where girders frame into all four sides. (ACI 21.5.2.2)

Note: The intersection of the orthogonal beam steel and the column steel, along with the required joint confinement hoop steel frequently results in congestion of bars. A careful study of the bar layouts should be made during design and represented on the construction documents.

Figure 7-40 Special Frame - girder column joint analysis.

Conditions			m factors[3]					
			Component Type					
			Primary				Secondary	
			Performance Level					
			IO	SE	LS	CP	LS	CP
i. Beams controlled by flexure[1]								
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$						
≤ 0.0	C	≤ 3	2	4	6	7	6	10
≤ 0.0	C	≥ 6	2	2.5	3	4	3	5
≥ 0.5	C	≤ 3	2	2.5	3	4	3	5
≥ 0.5	C	≥ 6	2	2	2	3	2	4
≤ 0.0	NC	≤ 3	2	2.5	3	4	3	5
≤ 0.0	NC	≥ 6	1	1.5	2	3	2	4
≥ 0.5	NC	≤ 3	2	2.5	3	3	3	4
≥ 0.5	NC	≥ 6	1	1.5	2	2	2	3
ii. Beams controlled by shear[1]								
Stirrup spacing ≤ $d/2$			–	–	–	–	3	4
Stirrup spacing > $d/2$			–	–	–	–	2	3
iii. Beams controlled by inadequate development or splicing along the span[1]								
Stirrup spacing ≤ $d/2$			–	–	–	–	3	4
Stirrup spacing > $d/2$			–	–	–	–	2	3
iv. Beams controlled by inadequate embedment into beam-column joint[1]								
			2	2	2	3	3	4

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic region, closed stirrups are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. Linear interpolation between values listed in the table is permitted.

Table 7-14: Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Beams

Conditions			IO	SE	LS	CP	LS	CP
			m factors[4]					
			Component Type					
			Primary				Secondary	
			Performance Level					
i. Columns controlled by flexure[1]								
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$						
≤ 0.1	C	≤ 3	2	2.5	3	4	3	4
≤ 0.1	C	≥ 6	2	2.5	3	3	3	3
≥ 0.4	C	≤ 3	1	1.5	2	2	2	2
≥ 0.4	C	≥ 6	1	1	1	2	1	2
≤ 0.1	NC	≤ 3	2	2	2	3	2	3
≤ 0.1	NC	≥ 6	2	2	2	2	2	2
≥ 0.4	NC	≤ 3	1	1	1	2	1	2
≥ 0.4	NC	≥ 6	1	1	1	1	1	1
ii. Columns controlled by shear[1,3]								
Hoop spacing ≤ $d/2$, or $\frac{P}{A_g f'_c} \le 0.1$			–	–	–	–	2	3
Other cases			–	–	–	–	1	1
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]								
Hoop spacing ≤ $d/2$			–	–	–	–	3	4
Hoop spacing > $d/2$			–	–	–	–	2	3
iv. Columns with axial loads exceeding $0.70P_o$[1,3]								
Conforming reinforcement over the entire length			1	1	1	2	2	2
All other cases			–	–	–	–	1	1

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic hinge region, closed hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. To qualify, hoops must not be lap spliced in the cover concrete, and must have hooks embedded in the core or other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
4. Linear interpolation between values listed in the table is permitted.

Table 7-15: Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Columns

Conditions			m factors[4]					
			Component Type					
			Primary[5]				Secondary	
			Performance Level					
			IO	SE	LS	CP	LS	CP
i. Interior joints								
$\frac{P}{A_g f_c}$ [2]	Trans. Reinf.[1]	$\frac{V}{V_n}$ [3]						
≤ 0.1	C	≤ 1.2	–	–	–	–	3	4
≤ 0.1	C	≥ 1.5	–	–	–	–	2	3
≥ 0.4	C	≤ 1.2	–	–	–	–	3	4
≥ 0.4	C	≥ 1.5	–	–	–	–	2	3
≤ 0.1	NC	≤ 1.2	–	–	–	–	2	3
≤ 0.1	NC	≥ 1.5	–	–	–	–	2	3
≥ 0.4	NC	≤ 1.2	–	–	–	–	2	3
≥ 0.4	NC	≥ 1.5	–	–	–	–	2	3
ii. Other joints								
$\frac{P}{A_g f_c}$ [2]	Trans. Reinf.[1]	$\frac{V}{V_n}$ [3]						
≤ 0.1	C	≤ 1.2	–	–	–	–	3	4
≤ 0.1	C	≥ 1.5	–	–	–	–	2	3
≥ 0.4	C	≤ 1.2	–	–	–	–	3	4
≥ 0.4	C	≥ 1.5	–	–	–	–	2	3
≤ 0.1	NC	≤ 1.2	–	–	–	–	2	3
≤ 0.1	NC	≥ 1.5	–	–	–	–	2	3
≥ 0.4	NC	≤ 1.2	–	–	–	–	1	1
≥ 0.4	NC	≥ 1.5	–	–	–	–	1	1

1. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A joint is conforming if closed hoops are spaced at ≤ $h_c/3$ within the joint. Otherwise, the component is considered nonconforming. Also, to qualify as conforming details under ii, hoops must not be lap spliced in the cover concrete, and must have hooks embedded in the core or other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
2. This is the ratio of the design axial force on the column above the joint to the product of the gross cross-sectional area of the joint and the concrete compressive strength. The design axial force is to be calculated using limit analysis procedures as described in Chapter 5.
3. This is the ratio of the design shear force to the shear strength for the joint.
4. Linear interpolation between values listed in the table is permitted.
5. All interior joints are force-controlled, and no *m* factors apply.

Table 7-16: Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Beam-Column Joints

Conditions		*m* factors					
		Component Type					
		Primary				Secondary	
		Performance Level					
		IO	SE	LS	CP	LS	CP
i. Slabs controlled by flexure, and slab-column connections[1]							
$\frac{V_g}{V_o}$ [2]	Continuity Reinforcement[3]						
≤ 0.2	Yes	2	2	2	3	3	4
≥ 0.4	Yes	1	1	1	1	2	3
≤ 0.2	No	2	2	2	3	2	3
≥ 0.4	No	1	1	1	1	1	1
ii. Slabs controlled by inadequate development or splicing along the span[1]							
		–	–	–	–	3	4
iii. Slabs controlled by inadequate embedment into slab-column joint[1]							
		2	2	2	3	3	4

1. When more than one of the conditions i, ii, and iii occurs for a given component, use the minimum appropriate numerical value from the table.
2. V_g = the gravity shear acting on the slab critical section as defined by ACI 318; V_o = the direct punching shear strength as defined by ACI 318.
3. Under the heading "Continuity Reinforcement," assume "Yes" where at least one of the main bottom bars in each direction is effectively continuous through the column cage. Where the slab is post-tensioned, assume "Yes" where at least one of the post-tensioning tendons in each direction passes through the column cage. Otherwise, assume "No."

Table 7-17: Numerical Acceptance Criteria for Linear Procedures—Two-Way Slabs and Slab-Column Connections

Conditions			Modeling Parameters[3]			Acceptance Criteria[3]					
						Plastic Rotation Angle, radians					
						Component Type					
						Primary				Secondary	
			Plastic Rotation Angle, radians		Residual Strength Ratio	Performance Level					
			a	b	c	IO	SE	LS	CP	LS	CP
i. Beams controlled by flexure[1]											
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$									
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.005	0.013	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.008	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.008	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.008	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0	0.003	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.008	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0	0.003	0.005	0.005	0.005	0.01
ii. Beams controlled by shear[1]											
Stirrup spacing ≤ $d/2$			0.0	0.02	0.2	0.0	0.0	0.0	0.0	0.01	0.02
Stirrup spacing > $d/2$			0.0	0.01	0.2	0.0	0.0	0.0	0.0	0.005	0.01
iii. Beams controlled by inadequate development or splicing along the span[1]											
Stirrup spacing ≤ $d/2$			0.0	0.02	0.0	0.0	0.0	0.0	0.0	0.01	0.02
Stirrup spacing > $d/2$			0.0	0.01	0.0	0.0	0.0	0.0	0.0	0.005	0.01
iv. Beams controlled by inadequate embedment into beam-column joint[1]											
			0.015	0.03	0.2	0.01	0.01	0.01	0.015	0.02	0.03

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic region, closed stirrups are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. Linear interpolation between values listed in the table is permitted.

Table 7-18: Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams

Conditions			Modeling Parameters[4]			Acceptance Criteria[4]					
			Plastic Rotation Angle, radians		Residual Strength Ratio	Plastic Rotation Angle, radians					
						Component Type					
						Primary				Secondary	
						Performance Level					
			a	b	c	IO	SE	LS	CP	LS	CP
i. Columns controlled by flexure[1]											
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$									
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.008	0.01	0.02	0.015	0.03
≤ 0.1	C	≥ 6	0.015	0.025	0.2	0.005	0.008	0.01	0.015	0.01	0.025
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.0	0.003	0.005	0.015	0.010	0.025
≥ 0.4	C	≥ 6	0.01	0.015	0.2	0.0	0.003	0.005	0.01	0.01	0.015
≤ 0.1	NC	≤ 3	0.01	0.015	0.2	0.005	0.005	0.005	0.01	0.005	0.015
≤ 0.1	NC	≥ 6	0.005	0.005	–	0.005	0.005	0.005	0.005	0.005	0.005
≥ 0.4	NC	≤ 3	0.005	0.005	–	0.0	0.0	0.0	0.005	0.0	0.005
≥ 0.4	NC	≥ 6	0.0	0.0	–	0.0	0.0	0.0	0.0	0.0	0.0
ii. Columns controlled by shear[1,3]											
Hoop spacing ≤ $d/2$, or $\frac{P}{A_g f'_c}$ ≤ 0.1			0.0	0.015	0.2	0.0	0.0	0.0	0.0	0.01	0.015
Other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]											
Hoop spacing ≤ $d/2$			0.01	0.02	0.4	1	1	1	1	0.01	0.02
Hoop spacing > $d/2$			0.0	0.01	0.2	1	1	1	1	0.005	0.01
iv. Columns with axial loads exceeding 0.70P_o[1,3]											
Conforming reinforcement over the entire length			0.015	0.025	0.02	0.0	0.003	0.005	0.001	0.01	0.02
All other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic hinge region, closed hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. To qualify, hoops must not be lap spliced in the cover concrete, and hoops must have hooks embedded in the core or other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
4. Linear interpolation between values listed in the table is permitted.

Table 7-19: Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Columns

columns; Table 7-20 for beam/column joints, and Table 7-21 for slab/column frames.

(4) Expected strength of deformation-controlled components shall be the nominal flexural strengths determined in accordance with Chapter 9 of FEMA 302 with 1.25 f_y in lieu of f_y for the contribution of the reinforcement.

(5) The lower-bound strength for force-controlled component shall be taken as the applicable nominal strength, Q_m, times the appropriate capacity reduction factor, N, in accordance with ACI 318.

7-5. Steel Moment-Resisting Frames.

a. General.

(1) Function. Steel moment-resisting frames have functions and behavior similar to those of concrete moment frames, as discussed in Paragraph 7-4a(1).

(2) Frame types. FEMA 302 prescribes three types of steel moment frames: Ordinary Moment Frames (OMFs), Intermediate Moment Frames (IMFs), and Special Moment Frames (SMFs). Restrictions regarding the use of these frames are summarized in Table 7-1, which also provides the appropriate R value for each classification. Design of steel moment frames shall be in accordance with the provisions of AISC Seismic Provisions for Structural Steel Buildings.

b. Ordinary Moment Frames (OMFs).

(1) General. OMFs are expected to withstand limited inelastic deformations in their members when subjected to the forces resulting from the ground motions of the design earthquakes in combination with other loads. OMFs shall have a design strength, as provided in the AISC Seismic Provisions, to resist load combinations 4-1 through 4-2 of that document.

(2) Beam-to-column connections shall be made by welds or high-strength bolts. Connections shall be fully restrained or partially restrained (Type PR).

(a) Fully restrained connections. The required flexural strength, M_u, of each beam-to-column connection considered to be part of the lateral-force-resisting system shall be at least equal to $1.1R_yM_p$ of the beam or column, whichever is weaker. For pre-engineered steel structures, M_p is permitted to be taken as the critical buckling moment of the beam section. Welded joints in the connection shall be made with filler metal rated to have a Charpy V-notch toughness of 20 ft-lbs (27N-m) at a temperature of 0°F, as determined by ASTM A673. Except for connections of beams to end plates for use in pre-engineered metal structures, welded joints shall be complete penetration welds. At the bottom flange of welds, weld backing shall be removed, the root inspected and repaired, and a reinforcing fillet added. At the top flange welds, backing shall be removed and repaired or shall be attached by means of a continuous fillet weld on the edge away from the complete penetration weld. Alternately, only connections having a demonstrated inelastic rotation capability of at least 0.01 radian, based on tests as described in Paragraph 7-5c, shall be permitted to be

Conditions			Modeling Parameters[4]			Acceptance Criteria[4]					
						Plastic Rotation Angle, radians					
						Component Type					
			Shear Angle, radians		Residual Strength Ratio	Primary				Secondary	
						Performance Level					
			d	e	c	IO	SE	LS	CP	LS	CP
i. Interior joints											
$\frac{P}{A_g f_c}$ [2]	Trans. Reinf.[1]	$\frac{V}{V_n}$ [3]									
≤ 0.1	C	≤ 1.2	0.015	0.03	0.2	0.0	0.0	0.0	0.0	0.02	0.03
≤ 0.1	C	≥ 1.5	0.015	0.03	0.2	0.0	0.0	0.0	0.0	0.015	0.02
≥ 0.4	C	≤ 1.2	0.015	0.025	0.2	0.0	0.0	0.0	0.0	0.015	0.025
≥ 0.4	C	≥ 1.5	0.015	0.02	0.2	0.0	0.0	0.0	0.0	0.015	0.02
≤ 0.1	NC	≤ 1.2	0.005	0.02	0.2	0.0	0.0	0.0	0.0	0.015	0.02
≤ 0.1	NC	≥ 1.5	0.005	0.015	0.2	0.0	0.0	0.0	0.0	0.01	0.015
≥ 0.4	NC	≤ 1.2	0.005	0.015	0.2	0.0	0.0	0.0	0.0	0.01	0.015
≥ 0.4	NC	≥ 1.5	0.005	0.015	0.2	0.0	0.0	0.0	0.0	0.01	0.015
ii. Other joints											
$\frac{P}{A_g f_c}$ [2]	Trans. Reinf.[1]	$\frac{V}{V_n}$ [3]									
≤ 0.1	C	≤ 1.2	0.01	0.02	0.2	0.0	0.0	0.0	0.0	0.015	0.02
≤ 0.1	C	≥ 1.5	0.01	0.015	0.2	0.0	0.0	0.0	0.0	0.01	0.015
≥ 0.4	C	≤ 1.2	0.01	0.02	0.2	0.0	0.0	0.0	0.0	0.015	0.02
≥ 0.4	C	≥ 1.5	0.01	0.015	0.2	0.0	0.0	0.0	0.0	0.01	0.015
≤ 0.1	NC	≤ 1.2	0.005	0.01	0.2	0.0	0.0	0.0	0.0	0.005	0.01
≤ 0.1	NC	≥ 1.5	0.005	0.01	0.2	0.0	0.0	0.0	0.0	0.005	0.01
≥ 0.4	NC	≤ 1.2	0.0	0.0	–	0.0	0.0	0.0	0.0	0.0	0.0
≥ 0.4	NC	≥ 1.5	0.0	0.0	–	0.0	0.0	0.0	0.0	0.0	0.0

1. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A joint is conforming if closed hoops are spaced at ≤ $h_c/3$ within the joint. Otherwise, the component is considered nonconforming. Also, to qualify as conforming details under ii, hoops must not be lap spliced in the cover concrete, and must have hooks embedded in the core or other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
2. This is the ratio of the design axial force on the column above the joint to the product of the gross cross-sectional area of the joint and the concrete compressive strength. The design axial force is to be calculated using limit analysis procedures, as described in Chapter 5.
3. This is the ratio of the design shear force to the shear strength for the joint.
4. Linear interpolation between values listed in the table is permitted.

Table 7-20: Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beam-Column Joints

Conditions		Modeling Parameters[4]			Acceptance Criteria[4]					
					Plastic Rotation Angle, radians					
					Component Type					
					Primary				Secondary	
		Plastic Rotation Angle, radians		Residual Strength Ratio	Performance Level					
		a	b	c	IO	SE	LS	CP	LS	CP
i. Slabs controlled by flexure, and slab-column connections[1]										
$\frac{V_g}{V_o}$ [2]	Continuity Reinforcement[3]									
≤ 0.2	Yes	0.02	0.05	0.2	0.01	0.008	0.015	0.02	0.03	0.05
≥ 0.4	Yes	0.0	0.04	0.2	0.0	0.0	0.0	0.0	0.03	0.04
≤ 0.2	No	0.02	0.02	–	0.01	0.008	0.015	0.02	0.015	0.02
≥ 0.4	No	0.0	0.0	–	0.0	0.0	0.0	0.0	0.0	0.0
ii. Slabs controlled by inadequate development or splicing along the span[1]										
		0.0	0.02	0.0	0.0	0.0	0.0	0.0	0.01	0.02
iii. Slabs controlled by inadequate embedment into slab-column joint[1]										
		0.015	0.03	0.2	0.01	0.01	0.01	0.015	0.02	0.03

1. When more than one of the conditions i, ii, and iii occurs for a given component, use the minimum appropriate numerical value from the table.
2. V_g = the gravity shear acting on the slab critical section as defined by ACI 318; V_o = the direct punching shear strength as defined by ACI 318.
3. Under the heading "Continuity Reinforcement," assume "Yes" where at least one of the main bottom bars in each direction is effectively continuous through the column cage. Where the slab is post-tensioned, assume "Yes" where at least one of the post-tensioning tendons in each direction passes through the column cage. Otherwise, assume "No."
4. Interpolation between values shown in the table is permitted.

Table 7-21: Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures— Two-Way Slabs and Slab-Column Connections

used. Such connections shall be constructed using the same configurations, materials, processes, and quality control as was used in the tested connections. Member sizes used shall be similar to those tested. A typical pre-Northridge Earthquake fully restrained moment connection is shown in Figure 7-41. This connection is permitted by FEMA 302, provided it meets the requirements of this paragraph for ordinary moment frames, and the further requirements of Paragraph 7-5c for intermediate moment frames or 7-5d for special moment frames.

(b) Partially restrained connections shall be used, provided that the following requirements are met:

1. The strength requirements of Paragraph 7-5b(1) are met.

2. The nominal bending strength of the connection is at least equal to $0.5M_p$ of the connected beams.

3. The connections have been demonstrated by cyclic tests to have adequate rotation capacity at an interstory drift calculated from the design story drift, Δ, as determined in Section 5.3.8.1 of FEMA 302.

4. The additional drift and lower strength of the partially restrained connections is considered in the design, including the effects on overall frame stability.

Partially restrained connections are described in detail in Section A2 of AISC "Design Specifications for Structural Steel Buildings." A partially restrained connection using split wide-flange beam sections is shown in Figure 7-42.

(c) Required shear strength. The required shear strength V_u of a beam-to-column connection shall be determined as a minimum using the load combination $1.2D + 0.5L + 0.2S$, plus the shear resulting from M_u as defined in Paragraph 7-5b(2)(a) for fully restrained connections, on each end of the beam. For partially restrained connections, V_u shall be determined from the load combination above plus the shear resulting from the maximum end moments that the partially restrained connections are capable of resisting.

(d) Continuity of column-flange stiffener plates. Where fully restrained connections are made by means of welds of beam flanges or beam-flange connection plates directly to column flanges, continuity or column-flange stiffener plates shall be provided to transmit beam-flange forces to the column web or webs. Such plates shall have a minimum thickness of one-half that of the beam flange or beam-flange connection plate. The connections of the plates to the column flanges shall have a design strength equal to the design strength of the contact area of the plate with the column flange. The connection of the plate to the column web shall have a design shear strength equal to the lesser of the following:

1. The design strength of the connections of the plate to the column flanges, or

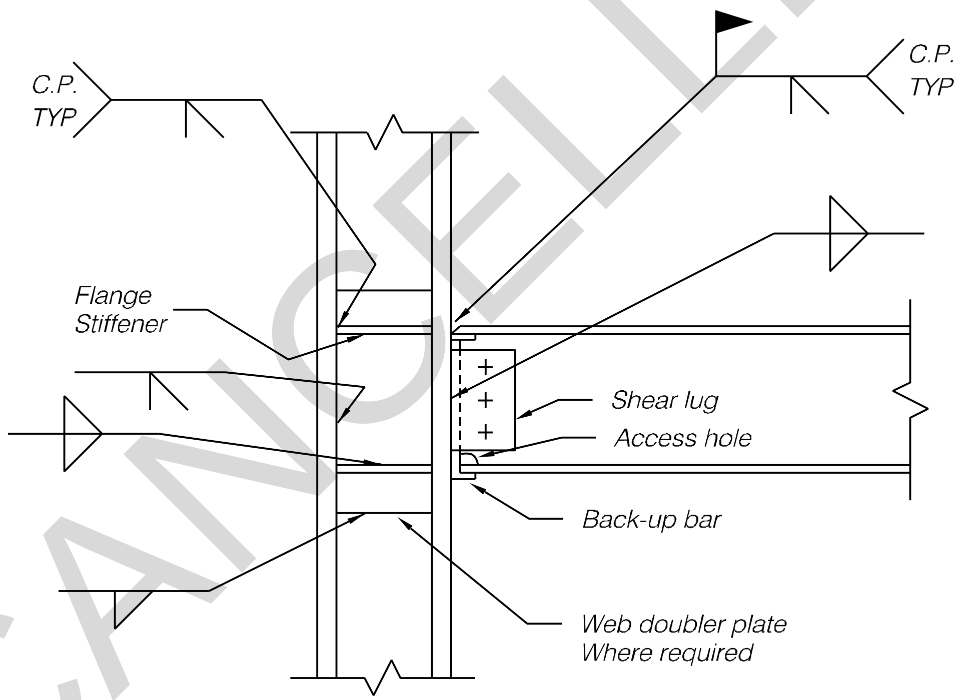


Figure 7-41 Typical pre-Northridge fully restrained moment connection

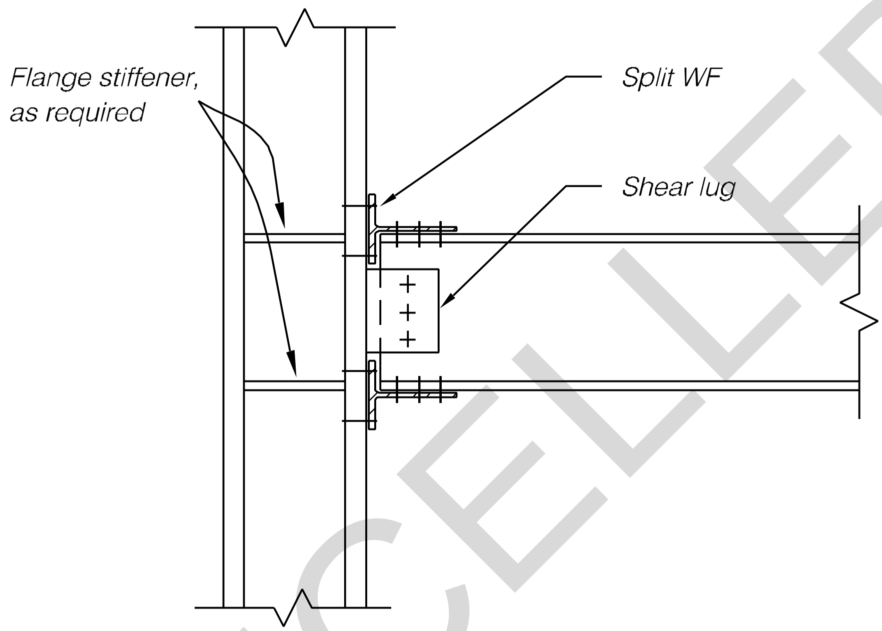


Figure 7-42 Typical partially restrained moment conection

2. The design shear strength of the contact area of the plate with the column web.

Continuity plates are not required if tested connections demonstrate that the intended inelastic rotation capacity can be achieved without their use. Partial penetration welds of the plates to the column flanges shall not be used.

c. Intermediate Moment Frames (IMFs). Intermediate moment frames are expected to withstand moderate inelastic deformations when subjected to the forces resulting from the motions of the design earthquake in combination with other loads. Intermediate moment frames shall conform to the AISC Seismic Provisions, Section 12, Requirements for Special Moment Frames, except as follows:

(1) Beam-to-column connections. Beam-to-column connection design shall be based on cyclic test results demonstrating inelastic rotation capacity of at least 0.020 radian. Inelastic rotation is defined as the total angle change between the column face at the connection and a line connecting the beam inflection point to the column face, less that part of the angle change occurring prior to yield of the beam. Qualifying test results shall consist of cyclic tests and shall be based on one of the following:

(a) Tests reported in research, or documented tests performed for other projects, which can be demonstrated to simulate project conditions.

(b) Tests conducted specifically for the project and representative of project member sizes, material strengths, connection configurations, and matching connection processes. At least two tests of each subassemblage type shall be performed successfully to qualify a connection for use. Interpolation or extrapolation of test results for different member sizes shall be justified by rational analysis that demonstrates stress distributions and magnitudes of internal stresses consistent with tested assemblies, and which considers potentially adverse effects of larger material and weld thickness and variations in material properties. Extrapolation of test results shall be limited to similar combinations of member sizes. Connections shall be constructed using materials, configurations, processes, and quality control and assurance methods that match as closely as is feasible those of the tested connections. Tests that utilize beams with tested F_y more than 10 percent lower than F_{ye} shall not be used.

(2) Connection flexural strength. The test results and analysis shall demonstrate a connection flexural strength, determined at the column face, at least equal to the nominal plastic moment, M_p, of the tested beam at the required inelastic rotation.

Exception: When beam flange buckling rather than connection strength limits the moment strength of the beam, and when connections using a reduced beam flange are used, then the limit shall be $0.8M_p$ of the tested beam. Figure 7-43 illustrates a fully restrained moment connection with haunches provided at the ends of the beam. This connection is designed such that the plastic hinge mechanism forms in the beam at the end of the haunch rather than at the column connection. If the beam size is based on strength considerations, haunches may

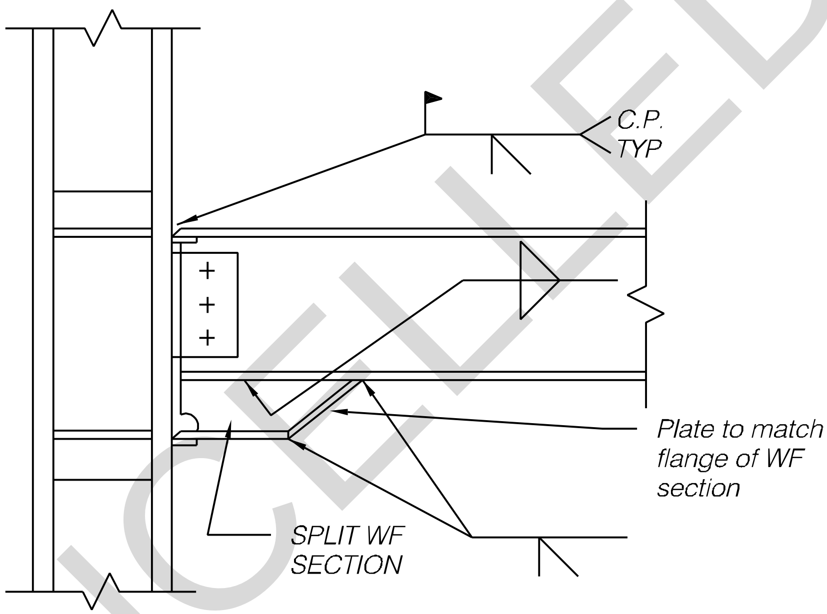


Figure 7-43 Typical post-Northridge fully restrained moment connection

permit selection of a shallower, and more economical, beam section. However, the beam depth and the length and depth of the haunch must be carefully selected to assure that the plastic hinge will occur at the end of the haunch, and not at the column face, for the combined seismic and factored gravity load moments. If the beam size is based on stiffness to control drift, the haunches may not contribute adequate stiffness to permit reduction in the size of the beam.

Exception: Connections that accommodate the required rotations within the connection itself and maintain the minimum required strength of Paragraph 7-5b(2)(a) are permitted to be used provided that the additional drift due to the connection deformation can be accommodated by the structure as demonstrated by rational analysis. Such rational analysis shall include consideration of overall frame stability, including the P-delta effect.

(3) Connection shear strength. The required shear strength, V_u, of a beam-to-column shall be determined using the load combination $1.2D + 0.5L + 0.2S$ plus the shear resulting from applying $1.1R_yF_yZ$ in the opposite sense on each end of the beam. Alternately, V_u shall be justified by rational analysis. The required shear strength is not required to exceed the shear resulting from the load combinations prescribed by Equations 4-6 and 4-7.

(4) Panel zone shear strength. The required shear strength, V_u, of the panel zone shall be the shear force determined by applying load combinations prescribed above to the moment-connected beam or beams in the plane of the frame at the column. V_u is not required to exceed the shear force determined from $0.8\Sigma M_p$ of the beams framing into the column flanges at the connection.

(5) Width-thickness ratios. Beams shall comply with λ_p in the AISC Design Specifications Table B5-1. When the ratio in Equation 7-3 is less than or equal to 1.25, columns shall comply with λ_p in Table I-9-1 of the AISC Seismic Provisions; otherwise, columns shall comply with Table B5-1 of the AISC Design Specifications.

(6) Continuity plates. Continuity plates shall be provided to match the tested connections. When tested connections do not include continuity plates, neither columns with thinner flanges nor beams with thicker or wider flanges shall be considered to be qualified by the test.

(7) Column/beam moment ratio. At any beam-to-column connection, the following strong column/weak beam relationship shall be satisfied:

$$\frac{\sum M^*_{pc}}{\sum M^*_{pb}} \geq 1.0 \qquad (7\text{-}3)$$

Where:

ΣM^*_{pc} = the moment at the intersection of the beam and column center-line determined by projecting the sum of the nominal column plastic moment strength, reduced by the axial stress P_{uc}/A_g, from the top and bottom of the beam moment connection (including haunches where used). It shall be permitted to take ΣM^*_{pc} as $\Sigma Z_c\,(F_{yc} - P_{uc}/A_g)$.

ΣM^*_{pb} = the moment at the intersection of the beam and column center-line determined by projecting the beam maximum developed moments from the column face thereto. Maximum developed moments shall be determined from test results as required by Paragraph 7-5b(2)(a) or by rational analysis based on the tests. Alternately, the maximum developed moment may be taken as 1.1 R_y M_p + M_v where M_v is the additional moment due to the shear amplification from the location of the plastic hinge to the column centerline. When connections with reduced beam sections are used, M_{pb} may be taken as 1.1 R_y F_y z + M_v, where z is the minimum plastic section modulus at the reduced section.

A_g = gross area of column, in^2 (mm^2).

F_{yc} = specified minimum yield strength of column ksi (Mpa).

P_{uc} = required axial strength in column, kips (kN).

Z_c = plastic section modulus of a column, in^3 (mm^3).

Ry = ratio of the expected yield strength, F_{ye}, to the minimum specified yield strength, F_y.

These requirements do not apply in any of the following cases, provided that the columns conform to the above minimum width-thickness ratios.

(a) Column with $P_{uc} < 0.3F_{yc}A_g$ for all load combinations:

1. Which are used in the top story of a multistory structure with a period greater than 0.7 seconds, or

2. Where the sum of their resistance is less than 20 percent of the shear in a story and is less than 33 percent of the shear on each of the column lines within that story. A column line is defined for the purpose of this exception as a single line of columns or parallel lines of columns located within 10 percent of the plan dimension perpendicular to the line of columns.

(b) Columns in any story that have a ratio of design shear strength to design force 50 percent greater than the story above.

(c) Any column not included in the design to resist the required seismic shears, but included in the design to resist axial overturning forces.

(8) Lateral support at beams. Both flanges of beams shall be laterally supported directly or indirectly. The unbraced length between lateral support shall not exceed $3{,}600 r_y/F_y$ (689.5 r_y/F_y for F_y in MPa). In addition, lateral supports shall be placed at concentrated loads where an analysis indicates a

hinge will be formed during inelastic deformation of the intermediate moment frame.

d. Special Moment Frames (SMFs). Special moment frames are expected to withstand significant inelastic deformation when subjected to the forces resulting from the motions of the design earthquake in combination with other loads. Special moment frames shall conform to all of the requirements for IMFs, except:

(1) Cyclic test results of the beam/column connection must demonstrate inelastic rotation capacity of at least 0.03 radian. The second exceptions in Paragraph 7-5c(2) shall not apply to SMFs.

(2) Circular sections shall have an outside-wall-diameter-to-thickness ratio not exceeding 1300/F_y (250/F_y for F_y in MPa). Rectangular tubes shall have an out-to-out width-to-wall thickness b/t not exceeding 110/F_y (21/F_y for F_y in MPa).

e. Special Truss Moment Frames (STMFs).

(1) General. Special truss moment frames, as shown in Figure 7-44, shall be designed so that when subjected to earthquake loading, yielding will occur in specially designed segments of the truss girders which are part of the lateral-force-resisting system. Such trusses shall be limited to span length between column not to exceed 60 feet (18m), and overall depth not to exceed 6 feet (1.8m). The columns and truss segments outside of the special segments shall be designed to remain elastic under the forces that can be generated by the fully yielded and strain-hardened special segment. Special truss moment frames shall have a design strength to resist the applicable load combinations of the AISC Seismic Provisions as modified by the following added requirements.

(2) Special segment. Each horizontal truss that is part of the moment frame shall have a special segment located within the middle one-half length of the truss. The length of the special segment shall range from 0.1 to 0.5 times the truss span length. The length-to-depth ratio of any panel in the special segment shall be limited to a maximum of 1.5 and a minimum of 0.67. All panels within a special segment shall be either Vierendeel or X braced, not a combination thereof. Where diagonal members are used in the special segment, they shall be arranged in an X pattern separated by vertical members. Such diagonal members shall be interconnected at points of crossing. The interconnection shall have a design strength adequate to resist a force at least equal to 0.25 times the diagonal member nominal tensile strength. Bolted connections shall not be used for web members within the special segment. Splicing of chord members shall not be permitted within the special segment, nor within ½ panel length from the ends of the special segment. Axial forces in diagonal web members due to factored dead plus live loads acting within the special segment shall not exceed 0.03 $F_y A_g$.

(3) Special segment nominal strength. In the fully yielded state, the special segment shall develop vertical nominal shear strength through the nominal flexural strength of the chord

members and through the nominal axial tensile and compressive strengths of the diagonal web members. The top and bottom chord members in the special segment shall be made of identical sections and in the fully yielded state shall provide at least 25 percent of the required vertical shear strength. The required axial strength in the chord members shall not exceed $0.45\ \phi\ F_y A_g$ where $f = 0.9$. Diagonal members in any panel of the special segment shall be made of identical sections. The end connections of diagonal members in the special segment shall have a design strength at

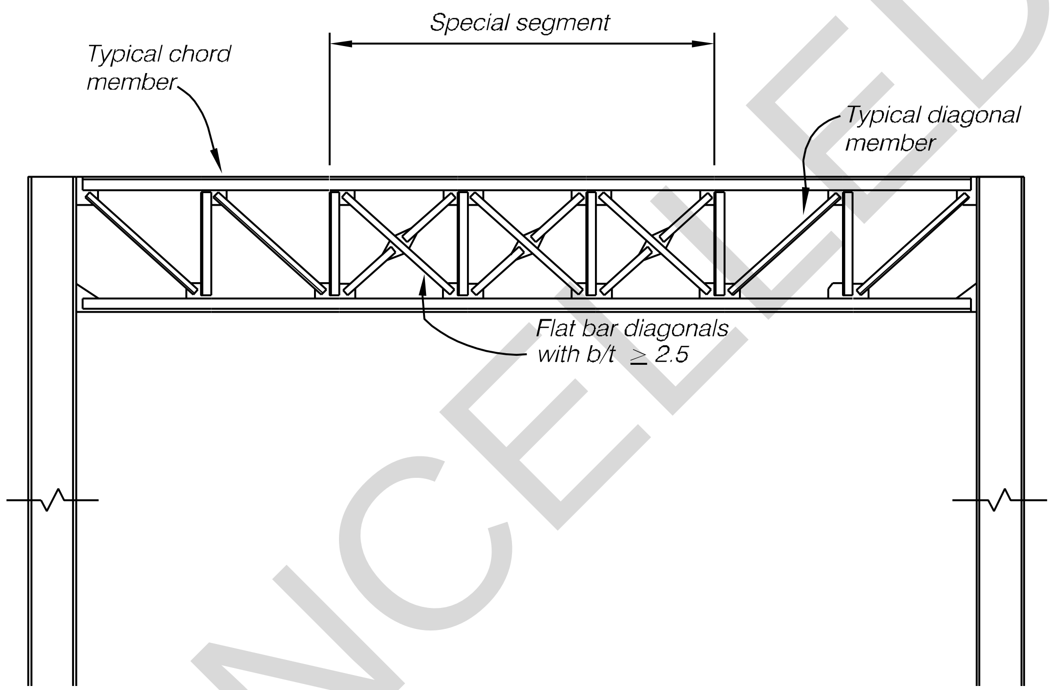


Figure 7-44 Special truss moment frame

least equal to the expected nominal axial tension strength of the web member, $R_y F_y A_g$.

(4) Non-special segment nominal strength. All members and connections of special truss moment frames, except those members identified as special segments, shall have a design strength to resist the factored gravity loads and the lateral loads necessary to develop the expected vertical nominal shear strength in all special segments, V_{ne}, given by the following formula:

$$V_{ne} = \frac{3.4 R_y M_{nc}}{L_s} + 0.07\ EI\left(\frac{L - L_s}{L_s^3}\right) + R_y\left(P_{nt} + 0.3 P_{nc}\right)\sin\ a \tag{7-4}$$

where:

R_y = defined in the AISC Design Specification;

M_{nc} = nominal flexural strength of the chord member of the special segment (kips-in.) (kN-m);

EI = flexural elastic stiffness of the chord members of the special segment (kip-in^2) (MPa);

L = span length of the truss (in.) (mm);

L_s = 0.9 times the length of the special segment (in.) (mm);

P_{nt} = nominal axial tension strength of diagonal members of the special segment (kips) (kN);

P_{nc} = nominal axial compression strength of the diagonal members of the special segment (kips) (kN);

″ = angle of diagonal members with the horizontal plane.

(5) Compactness. Diagonal web members of the special segment shall be made of flat bars. The width-thickness ratio of such flat bars shall not exceed 2.5. The width-thickness ratio of chord members shall not exceed the limiting l_p values from Table B5.1 of the AISC Design Specification. The width-thickness ratio of angles, and flanges and webs of tee sections used for chord members in the special segment, shall not exceed $52/\sqrt{F_y}$ $(137/\sqrt{F_y}$ for F_y in MPa.

(6) Lateral bracing. Top and bottom chords of the trusses shall be laterally braced at the ends of special segments, and at intervals not to exceed L_p, according to Section F1.1 of the AISC Design Specification, along the entire length of the truss. Each lateral brace at the ends of, and within, the special segment shall have a design strength to resist at least 5 percent of the required compressive axial strength, P_{nc}, of the largest adjoining chord member. Lateral braces outside of the special segment shall have at least 2.5 percent of the required P_{nc} of the largest adjoining chord members.

f. Acceptance Criteria.

(1) Response modification factors, *R*, for Performance Objective 1A for moment frames in

various structural systems are provided in Table 7-1.

(2) Modification factors, *m*, for enhanced performance objectives for beams, columns, and connections in fully restrained moment frames are provided in Table 7-12, and for partially restrained moment frames in Table 7-13.

(3) Modeling parameters and numerical acceptance criteria for nonlinear procedures are provided in Table 7-22 for fully restrained moment frames, and in Table 7-23 for partially restrained moment frames.

(4) Expected strength of columns, beams, and other deformation-controlled components shall be determined using the expected yield strength, F_{ye}, as defined in the AISC Seismic Provisions and the plastic section modulus, *Z*, where applicable.

(5) The lower bound strength of connections and other force-controlled components shall be determined in accordance with the nominal strength and N factors prescribed by AISC Seismic Provisions.

7-6. Dual Systems.

a. General.

(1) Combinations of structural systems. The connotation of dual systems is sometimes erroneously interpreted to mean different systems in each orthogonal direction of structural framing. To clarify this point, FEMA 302 describes that condition as "combinations of structural systems." In addition to the above interpretation, these combinations could also include different systems in the same vertical plane (e.g., a two-story building with steel moment frames in the second story and a concrete shear wall system in the first story). In the first case above, FEMA 302 permits the use of the appropriate *R* factor pertaining to the structural system in each orthogonal direction. For the second case, the FEMA provision states: "The response modification coefficient, *R*, in the direction under consideration at any story shall not exceed the lowest response modification factor, *R*, for the seismic-force-resisting system in the same direction considered above that story, excluding penthouses. For other than dual systems where a combination of different structural systems is utilized to resist lateral forces in the same direction, the value of *R* used in that direction shall not be greater than the least value of any of the systems utilized in the same direction. If a system other than a dual system with a response modification coefficient, *R*, with a value of less than 5 is used as part of the seismic-force-resisting system in any direction of the structure, the lowest such value shall be used for the entire structure. The system overstrength factor, Ω_o, in the direction under consideration at any story, shall not be less than the largest value of this factor for the seismic-force-resisting system in the same direction considered above that story."

Exceptions:

(a) Supported structural systems with a weight equal to or less than 10 percent of the weight of the structure.

Component/Action	$\frac{\Delta}{\Delta_y}$		Residual Strength Ratio	Plastic Rotation, Deformation Limits					
				Primary				Secondary	
	d	*e*	*c*	IO	SE	LS	CP	LS	CP
Beams[1]:									
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	10	12	0.6	2	5	7	9	10	12
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	5	7	0.2	1	2	3	4	4	5
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation									
Columns[2]: For $P/P_{ye} < 0.20$									
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	10	12	0.6	2	5	7	9	10	12
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$			0.2	1	2	3	4	4	5
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation									

1. Add θ_y from Equations 6-6 or 6-7 to plastic end rotation to estimate chord rotation.
2. Columns in moment or braced frames need only be designed for the maximum force that can be delivered.
3. Deformation = 0.072 (1 – 1.7 P/P_{ye})
4. Deformation = 0.100 (1 – 1.7 P/P_{ye})
5. Deformation = 0.042 (1 – 1.7 P/P_{ye})
6. Deformation = 0.060 (1 – 1.7 P/P_{ye})
7. 0.043 – 0.0009 d_b
8. 0.035 – 0.0008 d_b
9. If $P/P_{ye} > 0.5$, assume column to be force-controlled.

Table 7-22: Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Fully Restrained (FR) Moment Frames

Component/Action	$\frac{\Delta}{\Delta_y}$		Residual Strength Ratio	Plastic Rotation, Deformation Limits					
				Primary				Secondary	
	d	e	c	IO	SE	LS	CP	LS	C
For $0.2 \le P/P_{ye} \le 0.50$[9] a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	—[3]	—[4]	0.2	0.04	0.05	—[5]	—[6]	0.019	0.0
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	2	2.5	0.2	1	1.3	1.5	1.8	1.8	2
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation									
	Plastic Rotation								
	a	b							
Panel Zones	0.052	0.081	0.800	0.004	0.015	0.025	0.043	0.055	0.0
Connections									
For full penetration flange weld, bolted or welded web: beam deformation limits									
a. No panel zone yield	—[7]	—[7]	0.200	0.008	—[8]	—[8]	—[8]	0.017	0.0
b. Panel zone yield	0.009	0.017	0.400	0.003	0.004	0.005	0.007	0.010	0.0

1. Add θ_y from Equations 6-6 or 6-7 to plastic end rotation to estimate chord rotation.
2. Columns in moment or braced frames need only be designed for the maximum force that can be delivered.
3. Deformation = $0.072\,(1 - 1.7\,P/P_{ye})$
4. Deformation = $0.100\,(1 - 1.7\,P/P_{ye})$
5. Deformation = $0.042\,(1 - 1.7\,P/P_{ye})$
6. Deformation = $0.060\,(1 - 1.7\,P/P_{ye})$
7. $0.043 - 0.0009\,d_b$
8. $0.035 - 0.0008\,d_b$
9. If $P/P_{ye} > 0.5$, assume column to be force-controlled.

Table 7-22: Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Fully Restrained (FR) Moment Frames (Continued)

	Plastic Rotation[1]		Residual Force Ratio	Joint Rotation					
				Primary				Secondary	
	a	*b*	*c*	IO	SE	LS	CP	LS	CP
Top and Bottom Clip Angles[1]									
a. Rivet or bolt shear[2]	0.036	0.048	0.200	0.008	0.014	0.020	0.030	0.030	0.040
b. Angle flexure	0.042	0.084	0.200	0.010	0.018	0.025	0.035	0.035	0.070
c. Bolt tension	0.016	0.025	1.000	0.005	0.007	0.008	0.013	0.020	0.020
Top and Bottom T-Stub[1]									
a. Rivet or bolt shear[2]	0.036	0.048	0.200	0.008	0.014	0.020	0.030	0.030	0.040
b. T-stub flexure	0.042	0.084	0.200	0.010	0.018	0.025	0.035	0.035	0.070
c. Rivet or bolt tension	0.016	0.024	0.800	0.005	0.007	0.008	0.013	0.020	0.020
Composite Top Angle Bottom[1]									
a. Deck reinforcement	0.018	0.035	0.800	0.005	0.008	0.010	0.015	0.020	0.030
b. Local yield column flange	0.036	0.042	0.400	0.008	0.014	0.020	0.030	0.025	0.035
c. Bottom angle yield	0.036	0.042	0.200	0.008	0.014	0.020	0.030	0.025	0.035
d. Connectors in tension	0.015	0.022	0.800	0.005	0.007	0.008	0.013	0.013	0.018
e. Connections in shear[2]	0.022	0.027	0.200	0.005	0.009	0.013	0.018	0.018	0.023
Flange Plates Welded to Column Bolted or Welded to Beam[2]									
a. Flange plate net section or shear in connectors	0.030	0.030	0.800	0.008	0.014	0.020	0.025	0.020	0.025
b. Weld or connector tension	0.012	0.018	0.800	0.003	0.006	0.008	0.010	0.010	0.015
End Plate Bolted to Column Welded to Beam									
a. End plate yield	0.042	0.042	0.800	0.010	0.019	0.028	0.035	0.035	0.035
b. Yield of bolts	0.018	0.024	0.800	0.008	0.009	0.010	0.015	0.020	0.020
c. Fracture of weld	0.012	0.018	0.800	0.003	0.006	0.008	0.010	0.015	0.015

1. If $d_b > 18$, multiply deformations by $18/d_b$. Assumed to have web plate to carry shear. Without shear connection, this may not be downgraded to a secondary member.
2. For high-strength bolts, divide rotations by 2.

Table 7-23: Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Partially Restrained (PR) Moment Frames

(b) Detached one- and two-family dwelling of light-frame construction.

(2) Dual systems in the FEMA 302 provisions are defined as moment frames with either braced frames or shear walls that jointly resist lateral forces along the same line of force. The lateral forces are distributed to the various structural components in accordance with their relative rigidities, but the moment frames are designed to be capable of resisting at least 25 percent of the design forces. The moment frame shall be part of an essentially complete space frame system providing support for vertical loads. These dual systems are described further in the following paragraphs.

b. Moment Frame/Shear Wall Systems. As limited by this document, these dual systems shall consist of either structural steel or reinforced concrete moment frames resisting lateral forces jointly with either reinforced concrete or reinforced masonry shear walls. Appropriate *R* factors and other design coefficients for other systems are provided in Table 7-1.

c. Moment Frames/Bracing Systems. As defined by this document, these systems shall consist of steel moment frames with selected braced bays so that lateral forces are resisted partly by moment frame action and partly by braced frame action. The bracing system can consist of either concentrically or eccentrically braced frames. *R* factors and other design coefficients for these various systems are provided in Table 7-1. The use of concrete moment frames with either concrete or steel bracing is not prescribed by this document, as the detailing requirements are very demanding, and the performance of these systems has not been satisfactory.

d. Acceptance Criteria.

(1) Response modification factors, *R*, for Performance Objective 1A for various dual systems are provided in Table 7-1.

(2) Modification factors, *m*, for enhanced performance objectives, and modeling parameters and numerical criteria for nonlinear procedures are prescribed in the following paragraphs;

Reinforced concrete shear walls…..…para. 7-2f(3)

Precast concrete shear walls………...para. 7-2g(5)

Unreinforced masonry shear walls…para. 7-2h(5)

Concentric braced frames…………..para. 7-3b(9)

Eccentric braced frames…………….para. 7-3c(5)

Steel moment resisting frames………para. 7-5f.

7-7. Diaphragms.

a. General.

(1) Function. Floors and roofs, acting as diaphragms, are the horizontal resisting elements in a structure. Diaphragms are subject to lateral forces due to their own weight plus the tributary weight of walls connected to them. The diaphragms distribute the lateral forces to the vertical elements: the shear walls or frames, which resist the lateral forces and transfer them to lower levels of the building and finally to the ground. If floors or roofs cannot be made strong enough, their diaphragm function can be accomplished by horizontal bracing. In an industrial building, horizontal bracing can be the only resisting element. Where there is a horizontal offset between resisting vertical elements above and below, the diaphragm transfers lateral forces between the elements.

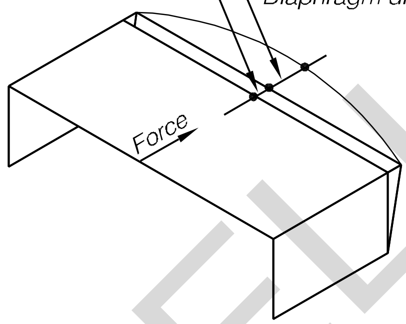
(2) Diaphragms. Usually the roof and floors of the structure perform the function of distributing lateral forces to the vertical-resisting elements (such as walls and frames). These elements, called diaphragms, make use of their inherent strength and rigidity, supplemented, when needed, by chords and collectors. A diaphragm is analogous to a plate girder laid in a horizontal plane (or inclined plane, in the case of a roof). The floor or roof deck functions as the girder web, resisting shear; the joists or beams function as web stiffeners; and the chords (peripheral beams or integral reinforcement) function as flanges, resisting flexural stresses (Figure 7-46). A diaphragm may be constructed of any material of which a structural floor or roof is made. Some materials, such as cast-in-place reinforced concrete and structural steel, have well-established properties and present no problems for diaphragm design once the loading and reaction system is known. Other materials, such as wood sheathing and metal deck, have properties that are well-established for vertical loads, but not so well established for lateral loads. For these materials, tests have been required to demonstrate their ability to resist lateral forces. Moreover, where a diaphragm is made up of units such as sheets of plywood or metal deck, or precast concrete units, the characteristics of the diaphragm are, to a large degree, dependent upon the connections that join one unit to another and to the supporting members.

(3) Horizontal bracing. A horizontal bracing system may also be used as a diaphragm to transfer the horizontal forces to the vertical-resisting elements. A horizontal bracing system may be of any approved material. A common system that is not recommended is the rod or angle tension-only bracing used in older

industrial buildings. The general layout of a bracing system and the sizing of members must be determined for each case in order to meet the requirements for load resistance and deformation control. The bracing system will be fully developed in both directions so that the bracing diagonals and chord members form complete horizontal trusses between vertical-resisting elements (Figure 7-47). Horizontal bracing systems will be designed using diaphragm design principles.

b. Diaphragm Flexibility.

(1) Diaphragm classification. Floor diaphragms shall be classified as either flexible, stiff, or rigid, as indicated in Figure 7-45. Diaphragms shall be considered flexible when the maximum lateral deformation of the diaphragm along its length is more than twice the average interstory drift of the story immediately below the diaphragm. For diaphragms supported by basement walls, the average interstory drift of the story above the diaphragm may be used in lieu of the basement story.



Deflection Ratio	Flexibility
a > 2b	Flexible
2b > a > 1/2b	Stiff
a < 1/2b	Rigid

Figure 7-45 Diaphragm flexibility

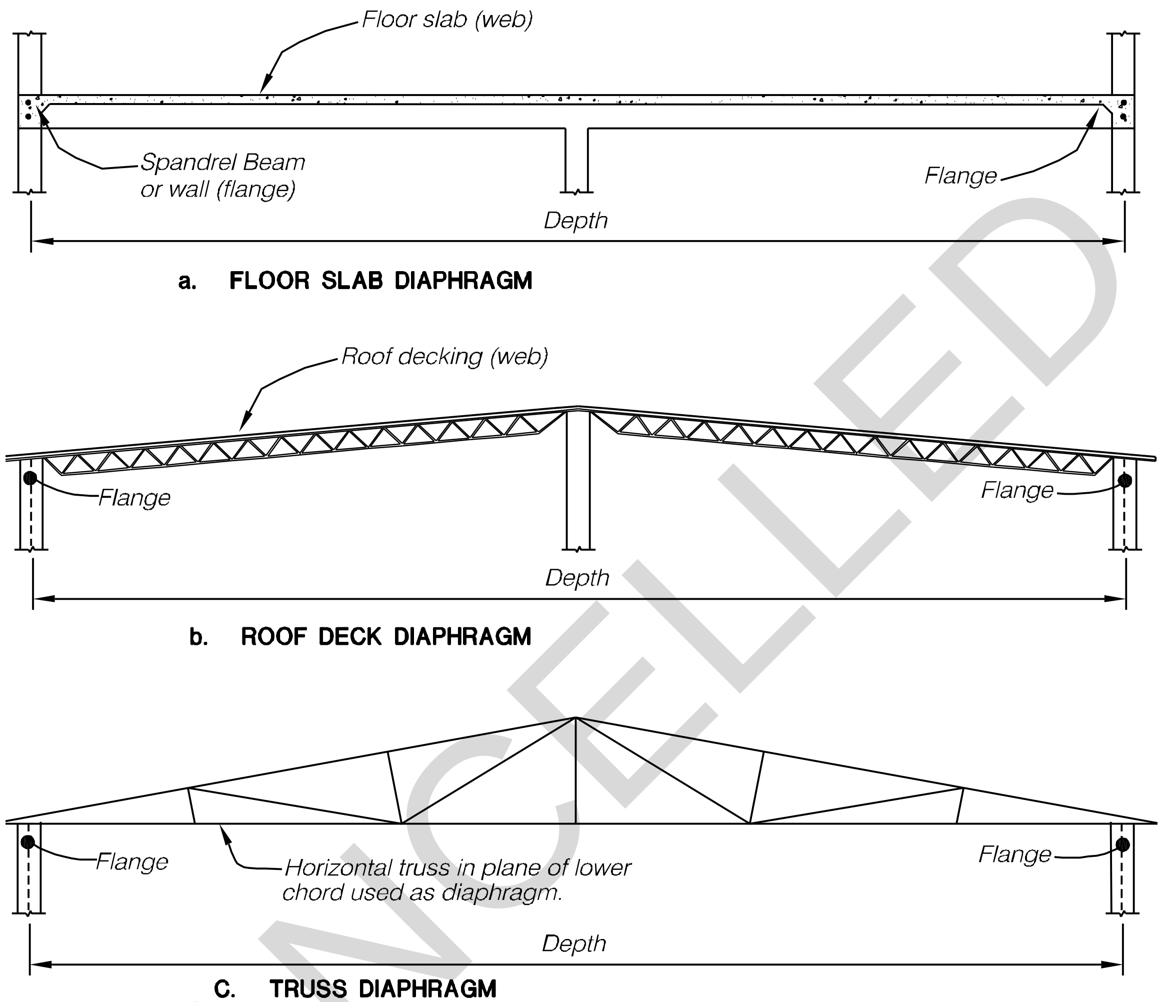


Figure 7-46 Diaphragms.

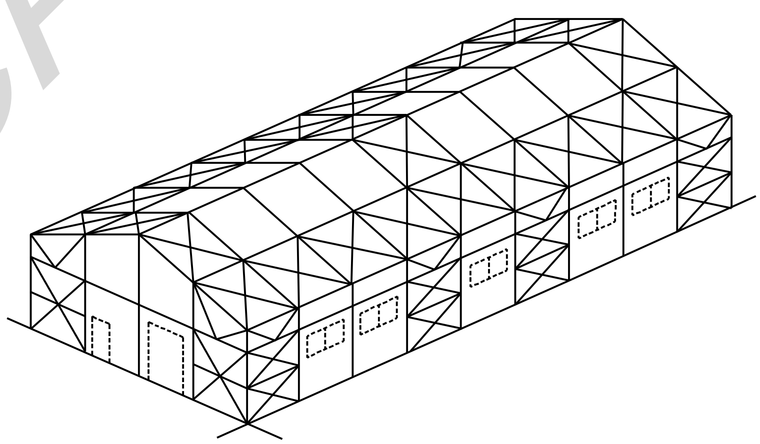


Figure 7-47 Bracing an industrial building.

Diaphragms shall be considered rigid when the maximum lateral deformation of the diaphragm is less than half the average interstory drift of the associated story. Diaphragms that are neither flexible nor rigid shall be classified as stiff. The interstory drift and diaphragm deformations shall be estimated using the seismic lateral forces from Section 5.3 or 5.4 of FEMA 302.

(2) Flexibility considerations. The in-plane deflection of the floor diaphragm shall be calculated for an in-plane distribution of lateral force consistent with the distribution of mass, as well as all in-plane lateral forces associated with offsets in the vertical seismic framing at that floor. The deformation of the diaphragm may be neglected in mathematical models of buildings with rigid diaphragms. Mathematical models of buildings with stiff diaphragms shall explicitly include diaphragm flexibility. Mathematical models of buildings with flexible diaphragms should explicitly account for the flexibility of the diaphragms. For buildings with flexible diaphragms at each floor level, the vertical lines of seismic framing may be designed independently, with seismic masses assigned on the basis of tributary area. Diaphragm flexibility results in: (1) an increase in the fundamental period of the building, (2) decoupling of the vibrational modes of the horizontal and vertical seismic framing, and (3) modification of the inertia force distribution in the plane of the diaphragm. There are numerous single-story buildings with flexible diaphragms. For example, precast concrete tilt-up buildings with timber-sheathed diaphragms are common throughout the United States. An equation for the fundamental period of a single-story building with a flexible diaphragm is presented in the following equation:

$$T = \left(0.1\Delta_w + 0.078\Delta_d\right)^{0.5} \qquad (7\text{-}5)$$

where Δ_w and Δ_d are in-plane wall and diaphragm displacements in inches, due to a lateral load, in the direction under consideration, equal to the weight of the building. For the displacements in mm, the calculated value of T shall be multiplied by 5. Wall displacements shall be estimated for each line of framing. For multiple-bay diaphragms, lateral load equal to the gravity weight tributary to the diaphragm bay under consideration shall be applied to each bay of the building to calculate a separate period for each diaphragm bay. The period so calculated that maximizes the equivalent base shear shall be used for design of all walls and diaphragms in the building.

(3) Rotation. In cases where there is a lack of symmetry either in the load or the reactions, the diaphragm will experience a rotation. Rotation is of concern because it can lead to vertical instability. This is illustrated in the following cases: the cantilever diaphragm, and the diaphragm supported on three sides.

(a) Building with a cantilever diaphragm (an example is shown in Figure 7-48). The layout of the resisting walls is shown in Figure 7-48, part a. If the backspan is flexible relative to the walls (Figure 7-48, part b), the forces exerted on the backspan by the cantilever are resisted by walls B, C, and D, provided there are adequate collectors. If the backspan is relatively rigid (Figure 7-48, part c), the load from the cantilever is resisted by all four walls

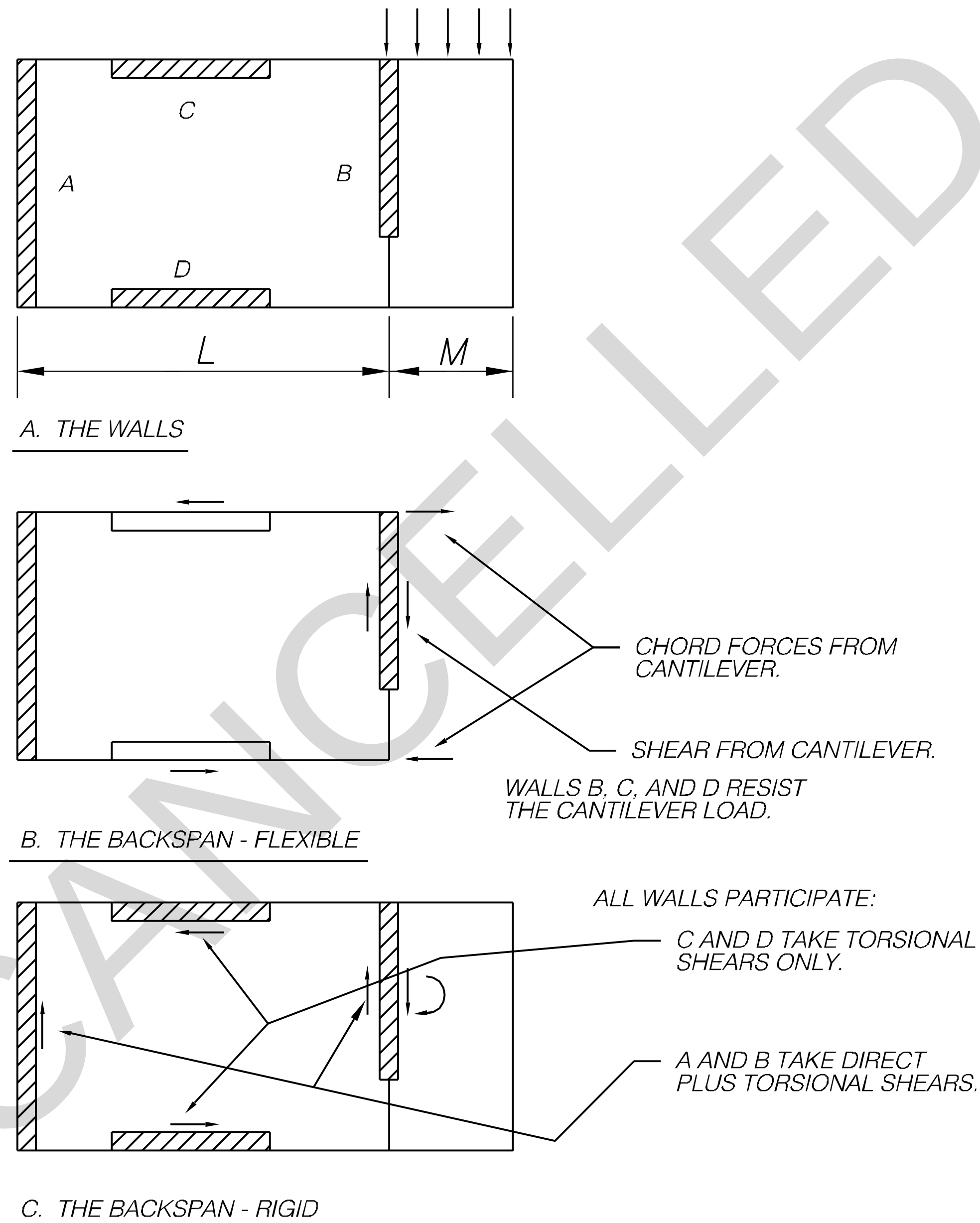


Figure 7-48 Cantilever diaphragm.

(A, B, C, and D); a rigidity analysis is needed in order to determine the forces in the walls.

(b) Building with walls on three sides (an example is shown in Figure 7-49). For transverse (north-south) forces (Figure 7-49, part a), this is a simple case: because of symmetry of load and reactions, the end walls share the load equally. For longitudinal (east-west) forces (Figure 7-49, part b), there is an eccentricity between the resultant of the load and the centerline of the one east-west resisting wall, wall C. The analysis is simplified by treating the load as a combination of the load, *W*, acting directly on the wall, and the couple $M = WD/2$ (Figure 7-49, part c). The direct force induces a direct shear, W, on the diaphragm and a reaction, *W*, in Wall C (Figure 7-49, part d); the moment, *M*, is resisted by walls A and B (Figure 7-49, part e), causing a counterclockwise rotation of the diaphragm. A particular concern with this type of building is the deflection on the corners at the open side. If this is excessive, it can lead to vertical instability in the southwest and southeast corners.

1. Flexible diaphragm. In an all-wood building, the concern about rotation is met by limitations on the size and proportions of the diaphragm. In buildings with walls of concrete or masonry, the greater weight causes greater concern for rotation, and there are special limitations on the span/width ratio of the diaphragms.

2. Rigid diaphragm. If the diaphragm is rigid, the design of the building will consider the effects of torsion. The concept of orthogonality does not apply.

(4) Torsion, in a general sense, occurs in a building whenever the location of the resultant of the lateral forces, i.e., the center of mass, *cm*, at and above a given level does not coincide with the center of rigidity, *cr*, of the vertical-resisting elements at that level. If the resisting elements have different deflections, the diaphragm will rotate. Torsion, in this general sense of rotation, occurs regardless of the stiffness properties of the diaphragms and the walls or frames. For purposes of design, however, the procedure for dealing with torsion does depend on these stiffness properties.

(a) Flexible diaphragms. Flexible diaphragms such as wooden diaphragms can rotate, but cannot develop torsional shears. For example, a single-span diaphragm with a relatively stiff shear wall at one end and a more flexible frame at the other end will rotate because the two resisting elements have different deflections. Flexible diaphragms, however, are considered incapable of inducing forces in the walls or frames that are perpendicular to the direction of the design forces; i.e., flexible diaphragms are said to be incapable of taking torsional moments. All of the lateral load is taken by the walls that are parallel to the lateral forces; none is taken by the other walls. (The building with walls on three sides is a special case and entails special limitations, as discussed above.) Lateral loads are usually distributed to the resisting walls by using the continuous beam analogy. There is no rigidity analysis, no calculation of the *cm* and the *cr*. If there are uncertainties about the locations of the loads and the rigidities of the structural elements, the design can be adjusted to bracket the range of possibilities.

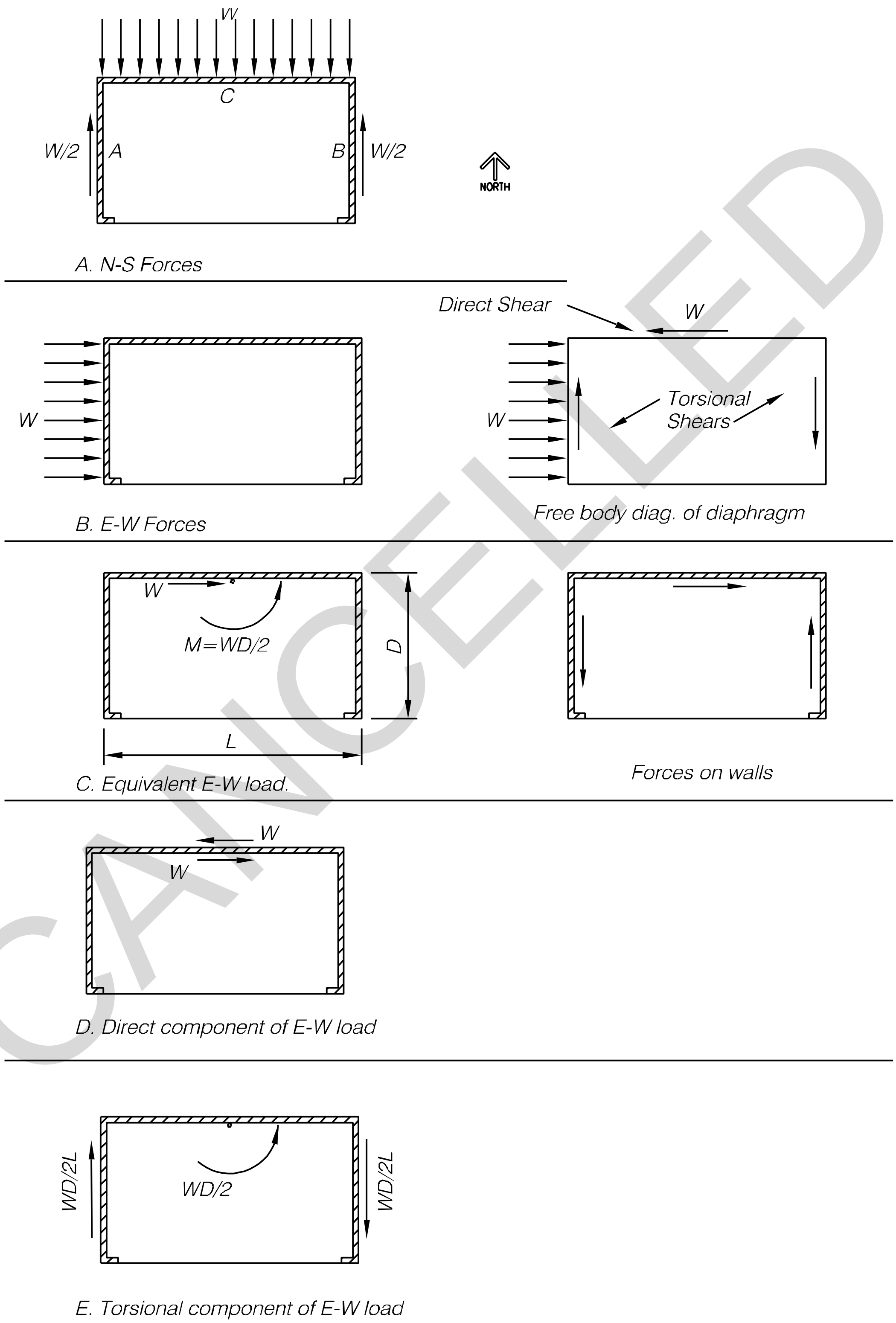


Figure7-49 Building with walls on three sides.

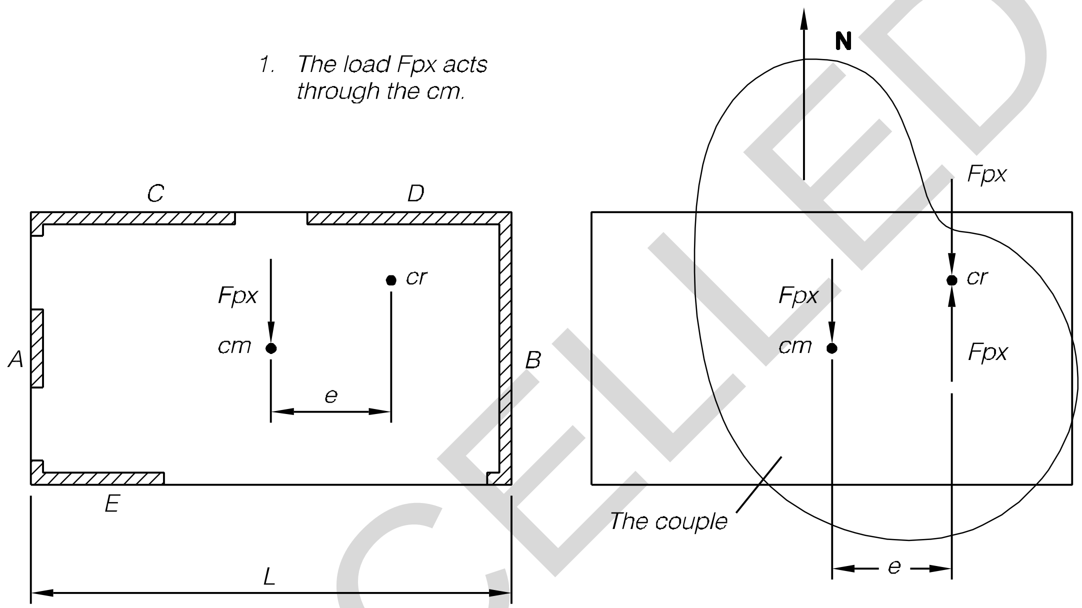
(b) Rigid diaphragms. When rigid diaphragms rotate, they develop shears in all of the vertical-resisting elements. In the example (Figure 7-50) there is an eccentricity in both directions, and all five walls develop resisting forces via the diaphragm.

(c) Deformational compatibility. When a diaphragm rotates, whether it is rigid or flexible, it causes displacements in all elements attached to it. For example, the top of a column will be displaced with respect to the bottom. Such displacements must be recognized and addressed.

(d) Analysis for torsion. The method of determining torsional forces is indicated in Figure 7-50. The diaphragm load, $F_{px,}$ which acts through the *cm*, is replaced by an equivalent set of new forces. By adding equal and opposite forces at the cr, the diaphragm load can now be described as a combination of a force component, F_{px} (which acts through the *cr*) and a moment component (which is formed by the couple of the two remaining forces F_{px} separated by the eccentricity *e*). The moment, called the torsional moment, M_T, is equal to F_{px} times *e*. The torsional moment is often called the "calculated" torsion, because it is based on a calculated eccentricity; also this name distinguishes it from the "accidental" torsion, which is described below. In the modified loading, the force F_{px} acts through the *cr* instead of *cm*; therefore, it causes no rotation and it is distributed to the walls, which are parallel to F_{px} in proportion to their relative rigidities. The torsional moment is resolved into a set of equivalent wall forces by a procedure similar to that used for finding forces on bolts in an eccentrically loaded group of bolts. The formula is analogous to the torsion formula $t = Tc/J$. The torsional shear forces can thus be expressed by the formula $F_t = M_T kd/\Sigma kd^2$, where *k* is the stiffness of a vertical-resisting element, *d* is the distance of the element from the center of rigidity, and Σkd^2 represents the polar moment of inertia. For the wall forces, the direct components due to F_{px} at the *cr* are combined with the torsional components due to M_T. In the example shown on Figure 7-50, the torsional moment is counterclockwise, and the diaphragm rotation will be counterclockwise around the *cr*. The direct component of the load is shared by walls A and B, while the torsional component of the load is resisted by walls A, B, D, C, and E. Where the direct and torsional components of wall force are the same direction, as in wall A, the torsional component adds to the direct component; where the torsional component is opposite to the direct component, as in wall B, the torsional component subtracts from the direct. Walls C, D, and E carry only torsional components; in fact, their design will most likely be governed by direct forces in the east-west direction.

(e) Accidental torsion. Accidental torsion is intended to account for uncertainties in the calculation of the locations of the *cm* and the *cr*. The accidental torsional moment, M_A, is obtained using an eccentricity, e_{acc}, equal to 5 percent of the building dimension perpendicular to the direction of the lateral forces; in other words, $M_A = F_{px} \times e_{acc}$. For the example of Figure 7-50, the accidental torsion for forces in the north-south direction is $M_T = F_{px} \times 0.05L$. In hand calculations, M_A is treated like $M_{T,}$ except that absolute values of the resulting forces are

1. The load Fpx acts through the cm.



2. Add an equal and opposite force, Fpx at the cr.

3. Use the equivalent load:

a Fpx acting through the cr
b. The moment formed by the two remainingFpx's. The torsional moment is

$$M_T = F_{px} \text{ x } e$$

Figure7-50 Calculated torsion.

used so that the accidental torsion increases the total design force for all walls. In computer calculations, the accidental torsion may be handled by running one analysis, using for eccentricity the calculated eccentricity plus the accidental eccentricity, then running a second analysis, using the calculated minus the accidental eccentricity, and finally, selecting the larger forces from the two cases.

(f) Dynamic amplifications of torsion. Section 5.3.5.3 of FEMA 302 specifies dynamic amplifications for Type 1 torsional irregularities in Seismic Design Category C, D, E, and F structures analyzed by the ELF procedure.

(5) Flexibility limitations. The deflecting diaphragm imposes out-of-plane distortions on the walls that are perpendicular to the direction of lateral force. These distortions are controlled by proper attention to the flexibility of the diaphragm. A diaphragm will be designed to provide such stiffness that walls and other vertical elements laterally supported by the diaphragm can safely sustain the stresses induced by the response of the diaphragm to seismic motion.

(a) Empirical rules. Direct design is not feasible because of the difficulty of making reliable calculations of the diaphragm deflections; instead, diaphragms are usually proportioned by empirical rules. The design requirement is considered to be met if the diaphragm conforms to the span and span/depth limitations of Table 7-24. These limitations are intended as a guide for ordinary buildings. Buildings with unusual features should be treated with caution. The limits of Table 7-24 may be exceeded, but only when justified by a reliable evaluation of the strength and stiffness characteristics of the diaphragm. If the diaphragm is providing out-of-plane lateral support to the top of a relatively short or stiff concrete or masonry wall, it should be noted that wall will experience the diaphragm deflections plus the in-plane deflection of the vertical lateral-load-resisting system. For use of Table 7-24, the flexibility category in the first column of the table can be determined with little or no calculation: concrete diaphragms are rigid; bare metal deck diaphragms can be stiff or flexible; plywood diaphragms can be considered to be rigid when used in light wood framing, but should be considered to be stiff or flexible with other framing systems; special diaphragms of diagonal wood sheathing are flexible; and conventional diaphragms of diagonal wood sheathing and diaphragms of straight wood sheathing are very flexible (very flexible diaphragms are seldom used in new construction because of their small capacities).

(b) Diaphragm deflections. When a deflection calculation is needed, the following procedure will be used.

1. Deflection criterion. The total deflection of the diaphragm under the prescribed static forces will be used as the criterion for the adequacy of the stiffness of a diaphragm. The limitation on the allowable amount relative to out-of-plane deflection (drift) of the walls, between the level of the diaphragm and the floor below, is equal to the deflection of the orthogonal walls at the ends of the diaphragm, plus the deflection of the diaphragm, as

		Diaphragm Span / Diaphragm Depth Limitations	
Flexibility Category	**Allowable Span of Diaphragm, ft.***	**Concrete or Masonry Walls**	**Other Walls**
Flexible	100	2:1	2½ :1
Stiff	200	2½ :1	3½ :1
Rigid	350	3½ :1	4:1

*1 foot = 0.3m

Table 7-24: Span and Depth Limitations on Diaphragms

computed in the following paragraphs.

2. Deflection calculations. The total computed deflection of diaphragms (Δ_d) under the prescribed static seismic forces consists of the sum of two components: the first component is the flexural deflection (Δ_f); the second component is the shearing deflection (Δ_w). When most beams are designed, the flexural component is usually all that is calculated, but for diaphragms, which are like deep beams, the shearing component must be added to the flexural component.

i. Flexural component. This is calculated in the same way as for any beam. For example, for a simple beam with uniform load, the flexural component is obtained from the familiar formula $\Delta_f = 5wL^4/384EI$. The only question is the value of the moment of inertia, I. For diaphragms whose webs have uniform properties in both directions (concrete or a flat steel plate), the moment of inertia is simply that of the diaphragm cross-section. For diaphragms of fluted steel deck, or diaphragms of wood, whose stiffness is influenced by nail slip and chord-joint slip, the flexural resistance of the diaphragm web is generally negligible, and the moment of inertia is based on the properties of the diaphragm chords. For a diaphragm of depth D with chord members each having area A, the moment of inertia, I, equals $2A(D/2)^2$, or $AD^2/2$.

ii. Shearing Component. The shearing component of deflection can be derived from the following equation:

$$\Delta_w = \frac{q_{ave} L_1 F}{10^6} \qquad (7\text{-}6)$$

where:

Δ_w = web component of diaphragm deflection, in. (mm).

q_{ave} = average shear in diaphragm, lbs. /ft. (N/m).

L_1 = distance from adjacent vertical resisting element (i.e., such as a shear wall) and the point to which the deflection is to be determined, ft. (m).

F = flexibility factor, micro inches per foot of span stressed with a shear of one pound per foot (micro millimeters per meter of span stressed with a shear of one Newton per meter of span).

Values of the flexibility factor, F, and the allowable shear per foot, q_D, for steel decking are given in manufacturers' catalogs, as well as the Diaphragm Design Manual of the Steel Deck Institute (SDI). Deflection calculations for concrete diaphragms are seldom required, but the deflection can be calculated by the conventional beam theory. For example, for a diaphragm with a single span of length, L, with a total load, W, that is uniformly distributed, the maximum shearing deflection is:

$$\Delta_w = \frac{a\,WL}{8A_w G} \qquad (7\text{-}7)$$

where:

″ = a form factor (L/D for prismatic webs)

A_w = area of the web

G = the shear modulus

noting that:

R, the end reaction, equals $W/2$ and q_{ave} = $R/2D = W/4D$, $L = 2L_1$, and $A_w = Dt$

Where t is the thickness of the web, and D is the depth of the diaphragm, the formula for shearing deflection can also be expressed as:

$$\Delta_w = \frac{q_{ave} L_1 a}{tG} \qquad (7\text{-}8)$$

As noted above, this is only applicable to webs of uniform properties. For a concrete slab with ″ = 1.5, $G = 0.4\ E$, and $E = 33w^{1.5}\sqrt{f'_c}$, the formula in English units becomes:

$$\Delta_w = \frac{q_{ave} L_1}{8.8tw^{1.5}\sqrt{f'_c}}. \qquad (7\text{-}9)$$

where:

t = thickness of the slab, in.

w = unit weight of the concrete, lbs. /cu. ft.

Recent editions of the SDI Diaphragm Design Manual provide the following alternative equation for the deflection of steel deck diaphragms:

$$)_w = \frac{wL^2}{8DG'} \qquad (7\text{-}10)$$

where:

w = uniform lateral shear load on the diaphragm, K/ft. (N/m).

L = diaphragm span, ft. (m).

D = depth of diaphragm, ft. (m).

G' = effective shear modulus calculated from tabulated values based on profile and thickness of deck and type and spacing of connectors.

The effective shear modulus , G', is related to the flexibility factor, F, as follows:

$$G' = 10^3 / F \qquad (7\text{-}11)$$

c. Design of Diaphragms. A deep-beam analogy is used in the design. Diaphragms are envisioned as deep beams with the web (decking or sheathing) resisting shear and the flanges (spandrel beams or other members) at the edges resisting the bending moment.

(1) Unit shears. Diaphragm unit shears are obtained by dividing the diaphragm shear by the length or area of the web, and are expressed in pounds per foot (N/m) (for wood and metal decks) or pounds per square inch (MPa) (for concrete). These unit shears are checked against allowable values for the material. Webs of precast concrete units or metal-deck units will require details for joining the units to each other and to their supports so as to distribute shear forces.

(2) Flexure. Diaphragm flexure is resisted by members called chords. The chords are often at the edges of the diaphragm, but may be located elsewhere. The design force is obtained by dividing the diaphragm moment by the distance between the chords. The chords must be designed to resist direct tensile or compressive stresses, both in the members and in the splices at points of discontinuity. Usually, chords are easily developed. In a concrete frame, continuous reinforcing in the edge beam can be used. In a steel frame building, the spandrel beams can be used as chords if they have adequate capacity and adequate end connections where they would otherwise be interrupted by the columns; or special reinforcing can be placed in the slab. Chords need not actually be in the plane of the diaphragm as long as the chord forces can be developed between the diaphragm and the chord. For example, continuous chord reinforcing can be placed in walls or spandrels above or below the diaphragm. In masonry walls, the chord requirements tend to conflict with the control joint requirements. At bond beams, control joints will have to be dummy joints so that reinforcement can be continuous, and the marginal connections must be capable of resisting the flexural and shear stresses developed.

(3) Openings. A diaphragm with openings such as cut-out areas for stairs or elevators will be treated as a plate girder with holes in the web. The diaphragm will be reinforced so that forces developing on the sides of the opening can be developed back into the body of the diaphragm.

(4) L-and T-shaped buildings. L- and T-shaped buildings will have the flange (chord) stresses developed through or into the heel of the L or T. This is analogous to a girder with a deep haunch. Figure 7-51 illustrates the calculation of the chord forces at a re-entrant diaphragm corner. These chord forces need to be developed by an appropriate connection to the floor or roof framing, or by the addition of a drag strut, to develop resistance to the chord forces within the adjacent diaphragm.

d. Concrete Diaphragms.

(1) General design criteria. The criteria used to design concrete diaphragms will be ACI 318, as modified by FEMA 302. Concrete diaphragm webs will be designed as concrete slabs; the slab may be designed to support vertical loads between the framing members, or the slab itself may be supported by other vertical-load-carrying elements, such as precast concrete elements or steel decks. If shear is transferred from the diaphragm web to the framing members through steel deck fastenings, the design will conform to the requirements in Paragraph 7-7e(1)(a).

(2) Span and anchorage requirements. The following provisions are intended to prevent diaphragm buckling.

(a) General. Where reinforced concrete slabs are used as diaphragms to transfer lateral forces, the clear distance (L_V) between framing members or mechanical anchors shall not exceed 38 times the total thickness of the slab (t).

Drag struts are required at re-entrant building corners to develop tensile chord forces F_1 and F_2 into adjacent diaphragms.

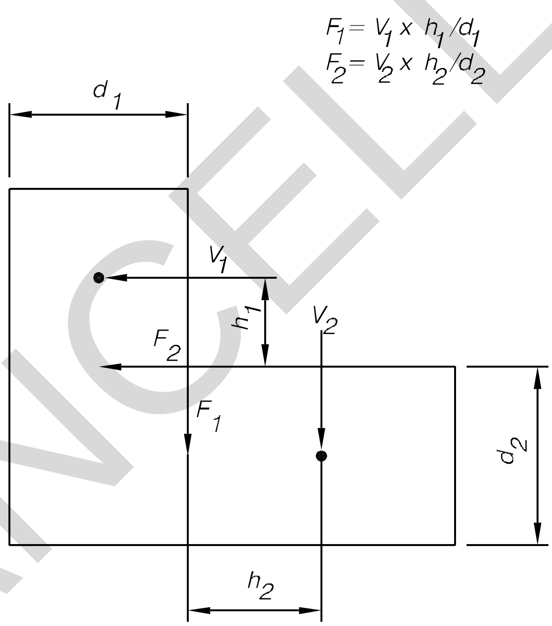


Figure 7-51 Drag struts at re-entrant building corners.

(b) Cast-in-place concrete slabs not monolithic with supporting framing. When concrete slabs used as diaphragms are not monolithic with the supporting framing members (e.g., slabs on steel beams), the slab will be anchored by mechanical means at intervals not exceeding 4 feet (1.2m) on center along the length of the supporting member. This anchorage is not a computed item, and should be similar to that shown in Figure 7-52, Detail A. For composite beams, anchorages provided in accordance with AISC provisions for composite construction will meet the requirements of this paragraph.

(c) Cast-in-place concrete diaphragms vertically supported by precast concrete slab units. If the slab is not supporting vertical loads but is supported by other vertical-load-carrying elements, mechanical anchorages will be provided at intervals not exceeding 38*t*; thus, the provisions above will be satisfied by defining L_V as the distance between the mechanical anchorages between the diaphragm slab and the vertical-load-carrying members. This mechanical anchorage can be provided by steel inserts or reinforcement, by bonded cast-in-place concrete lugs, or by bonded roughened surface, as shown in Figure 7-53. Positive anchorage between cast-in-place concrete and the precast deck must be provided to transmit the lateral forces generated from the weights of the precast units to the cast-in-place concrete diaphragm, and then to the main lateral-force-resisting system.

(d) Precast concrete slab units. If precast units are continuously bonded together as shown in Figure 7-54, they may be considered concrete diaphragms and designed accordingly, as previously described herein. Intermittently bonded precast units or precast units with grouted shear keys will not be used as a diaphragm. In areas with $S_{DS} \leq 0.25$g (Figure 7-55), there is an exception permitting the use of hollow-core planks with grouted shear keys and the use of connectors, in lieu of continuous bonding, for precast concrete members. The exception is permitted if the following considerations and requirements are satisfied:

1. Procedure conforms with PCI-MNL-120 seismic design requirements.

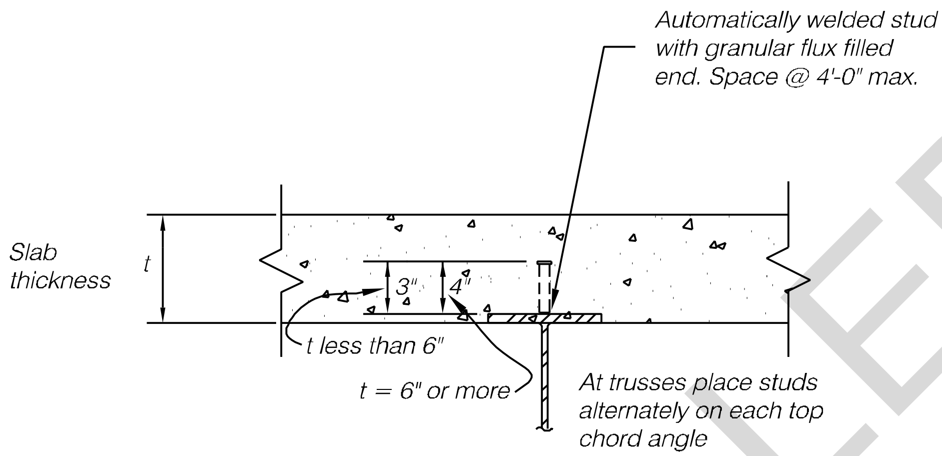
2. Shear forces for diaphragm action can be effectively transmitted through connectors. The shear is uniformly distributed throughout the depth or length of the diaphragm with reasonably spaced connectors, rather than with a few that will have localized concentration of shear stresses.

3. Connectors are designed for 0.6*R* times the prescribed shear force.

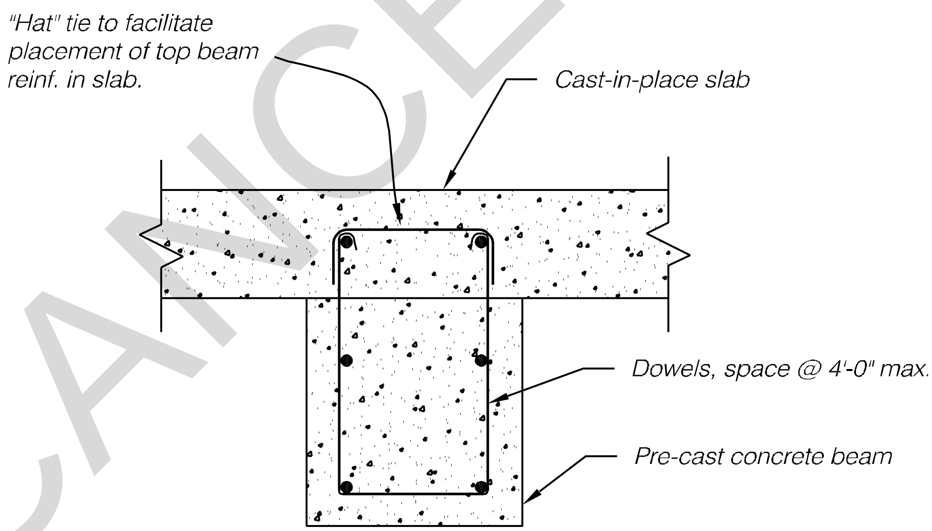
4. Detailed structural calculations are made including the localized effects in concrete slabs attributed from these connectors.

5. Sufficient details of connectors and embedded anchorage are provided to preclude construction deficiency.

(e) Metal-formed deck, Where metal deck is used as a form, the slab shall be governed by the requirements of Paragraph (b) above. Refer to Paragraph 7-7e, where the deck is used structurally.



DETAIL A

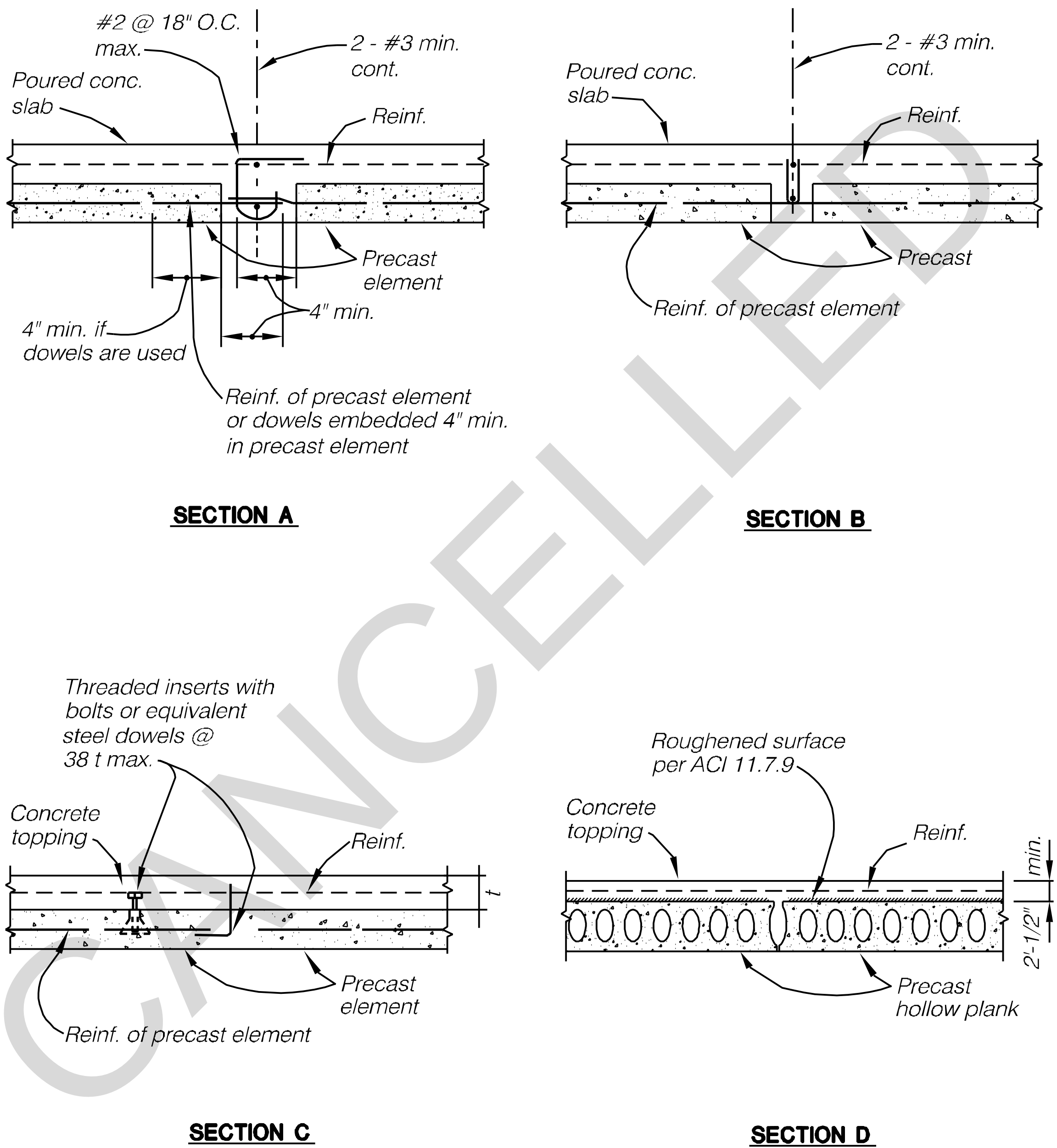


1 inch = 25mm
1 foot = 0.3m

DETAIL B

Slabs Not Monolithic with Supporting Framing

Figure7-52 Anchorage of cast-in-place concrete slab



1 inch = 25mm
#2 bar ≈ 5M bar
#3 bar ≈ 10M

Figure 7-53 Attachment of superimposed diaphragm slab to precast slab units.

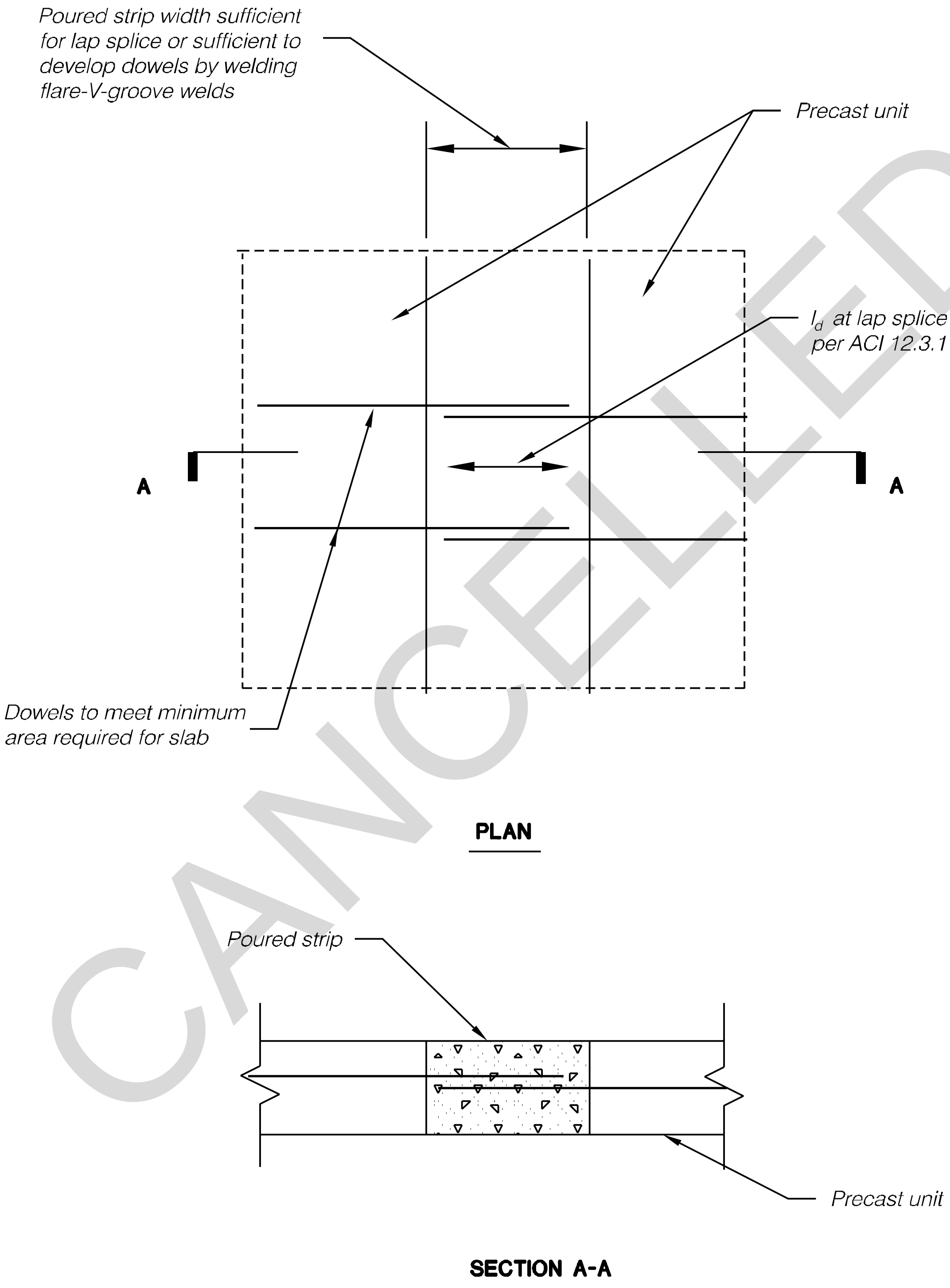


Figure7-54 Precast concrete diaphragms using precast units.

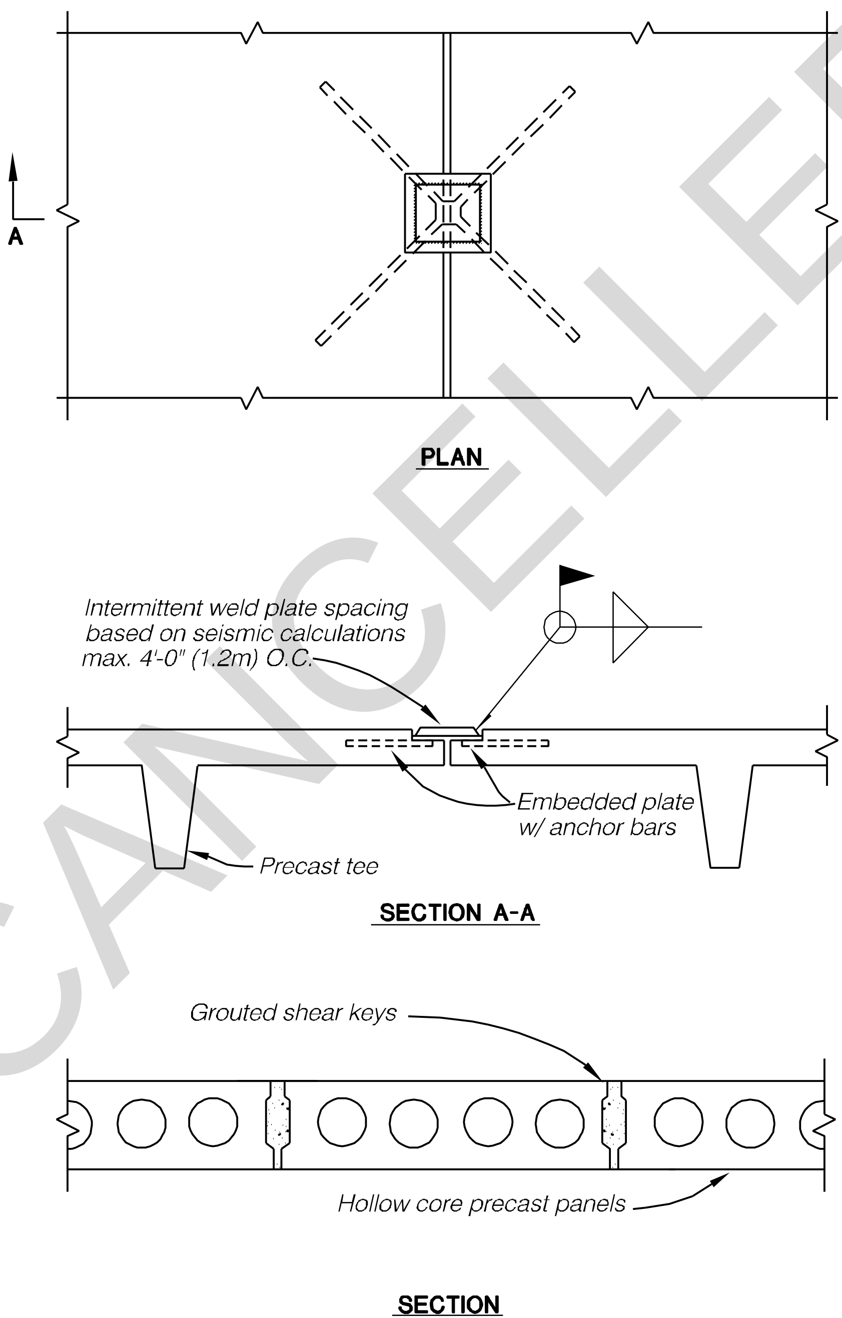


Figure 7-55 Concrete diaphragms using precast units-details permitted for Seismic Design Categories A and B

(3) Special reinforcements. Special diagonal reinforcement will be placed on corners of diaphragms, as indicated in Figure 7-56. Typical chord reinforcement and connection details are shown in Figure 7-57.

(4) Acceptance criteria.

(a) Reinforced concrete diaphragms for Performance Objective 1*A* shall be designed with the response reduction factor, *R,* in Table 7-1, based on the building's structural systems.

(b) For enhanced performance objectives, reinforced concrete diaphragms shall be considered as force-controlled rigid elements, and the demand / capacity ratio in shear shall not exceed 1.25.

e. Steel Deck Diaphragms.

(1) General design criteria. The following criteria will be used to design steel deck diaphragms. The three general categories of steel deck diaphragms are Type A, Type B, and decks with concrete fill. Design data from industry sources such as the Steel Deck Institute and the Research Reports of the International Conference of Building Officials may be used subject to the approval of the Agency Proponent.

(a) Typical deck units and fastenings. Deck units will be composed of a single fluted sheet or a combination of two or more sheets fastened together with welds. The special attachments used for field attachment of steel decks are shown in Figure 7-58. In addition to those shown, standard fillet (1/8-inch by 1-inch) (3mm by 25 mm) and butt welds are also used. The depth of deck units will not be less than 1½ inches (38mm).

(b) Connections at ends and at supporting beams. Refer to Type A and Type B details, Paragraphs 7-7e(2) and 7-7e(3).

(c) Connections at marginal supports. Marginal welds for all types of steel deck diaphragms will be spaced as follows:

for puddle welds

$$a_w = \frac{35{,}000\left(t_1 + t_2'\right)}{q_{ave}} \qquad (7\text{-}12)$$

metric equivalent:

$$a_w = \frac{200{,}000\left(t_1 + t_2'\right)}{q_{ave}} \qquad (7\text{-}12\text{m})$$

for fillet welds and seam welds

$$a_w = \frac{1200\ell_w'}{q_{ave}} \qquad (7\text{-}13)$$

metric equivalent:

$$a_w = \frac{6850\ell_w'}{q_{ave}} \qquad (7\text{-}13\text{m})$$

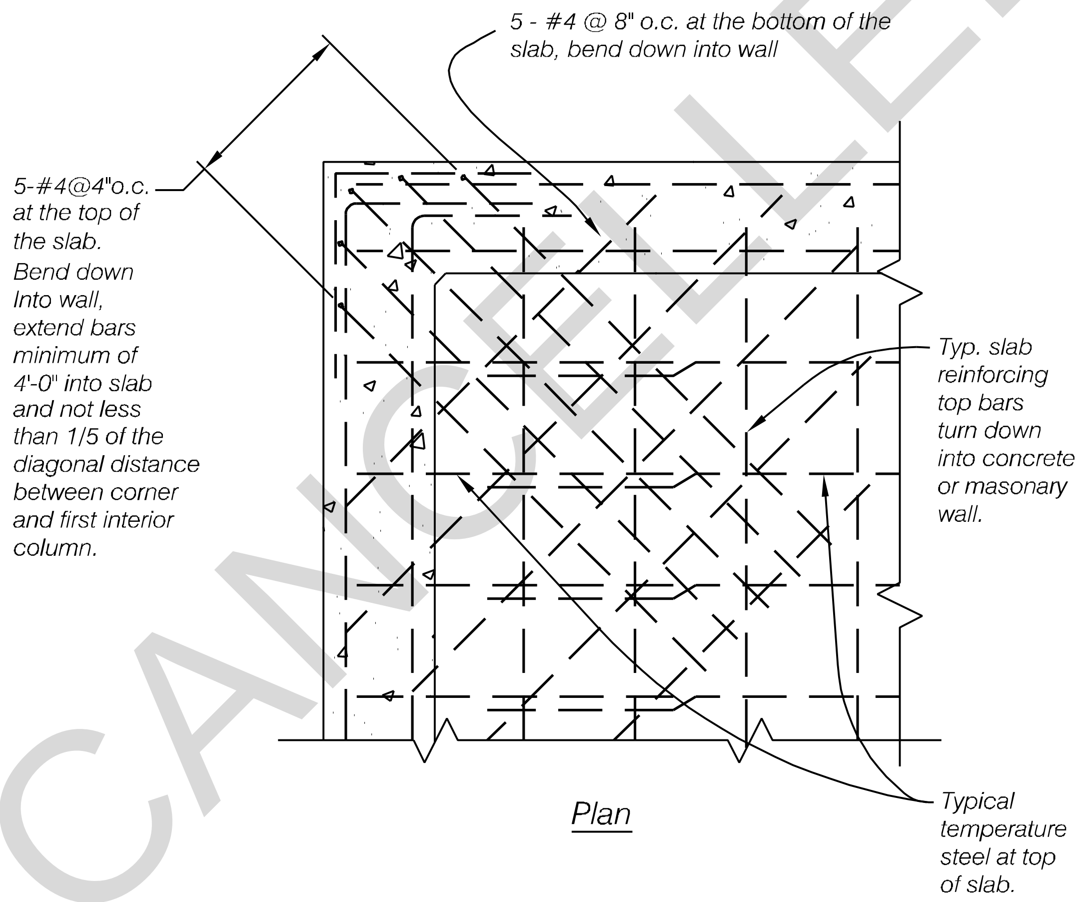
where:

a_w = spacing of marginal welds in feet (m).

t_1 = thickness of flat sheet elements in inches (mm) (22-gauge minimum).

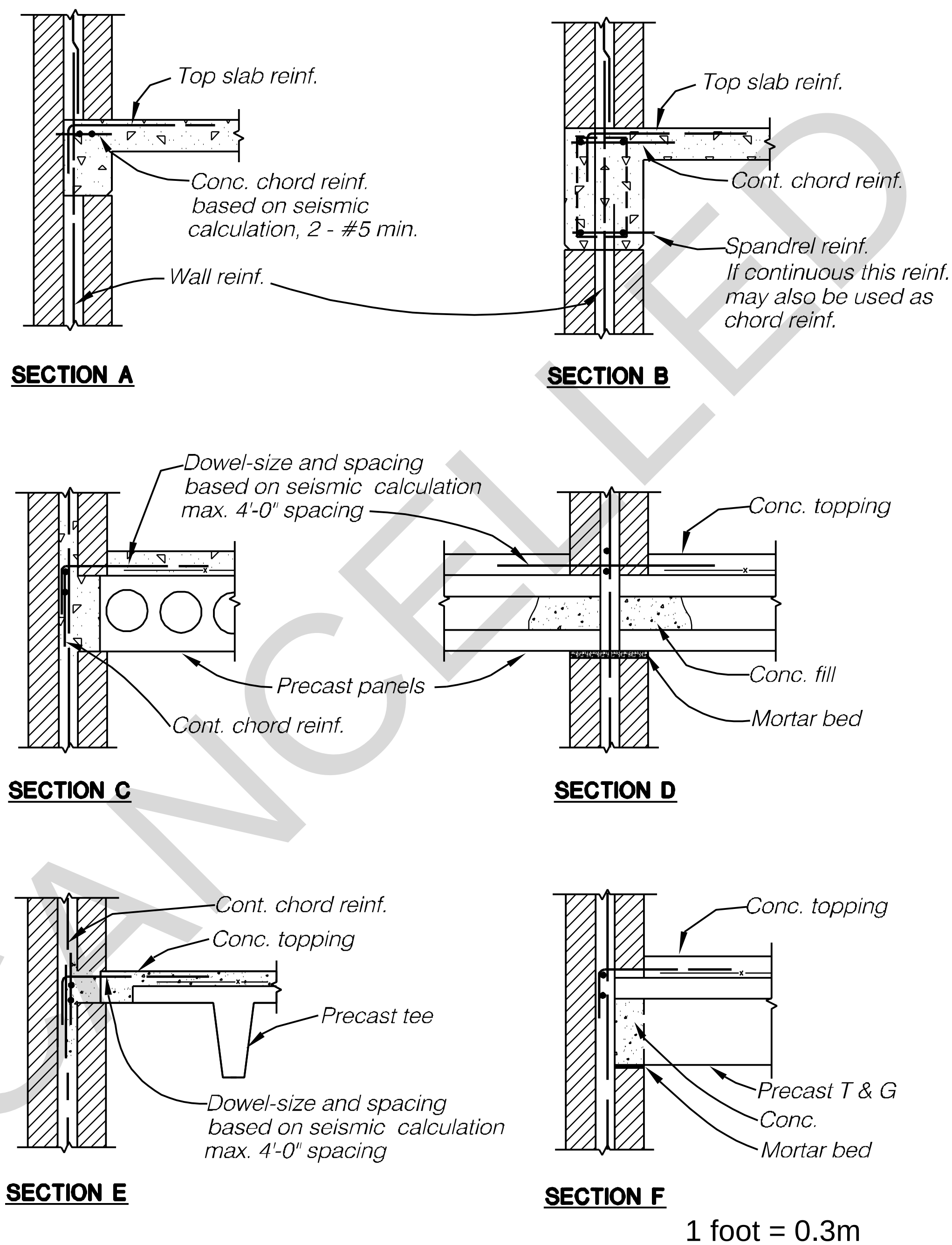
t'_2 = effective thickness of fluted elements in inches (mm).

q_{ave} = average shear in diaphragm over length L_1, in pounds per foot (N/m).



#4 bar ≈ 10M bar
1 inch = 25mm

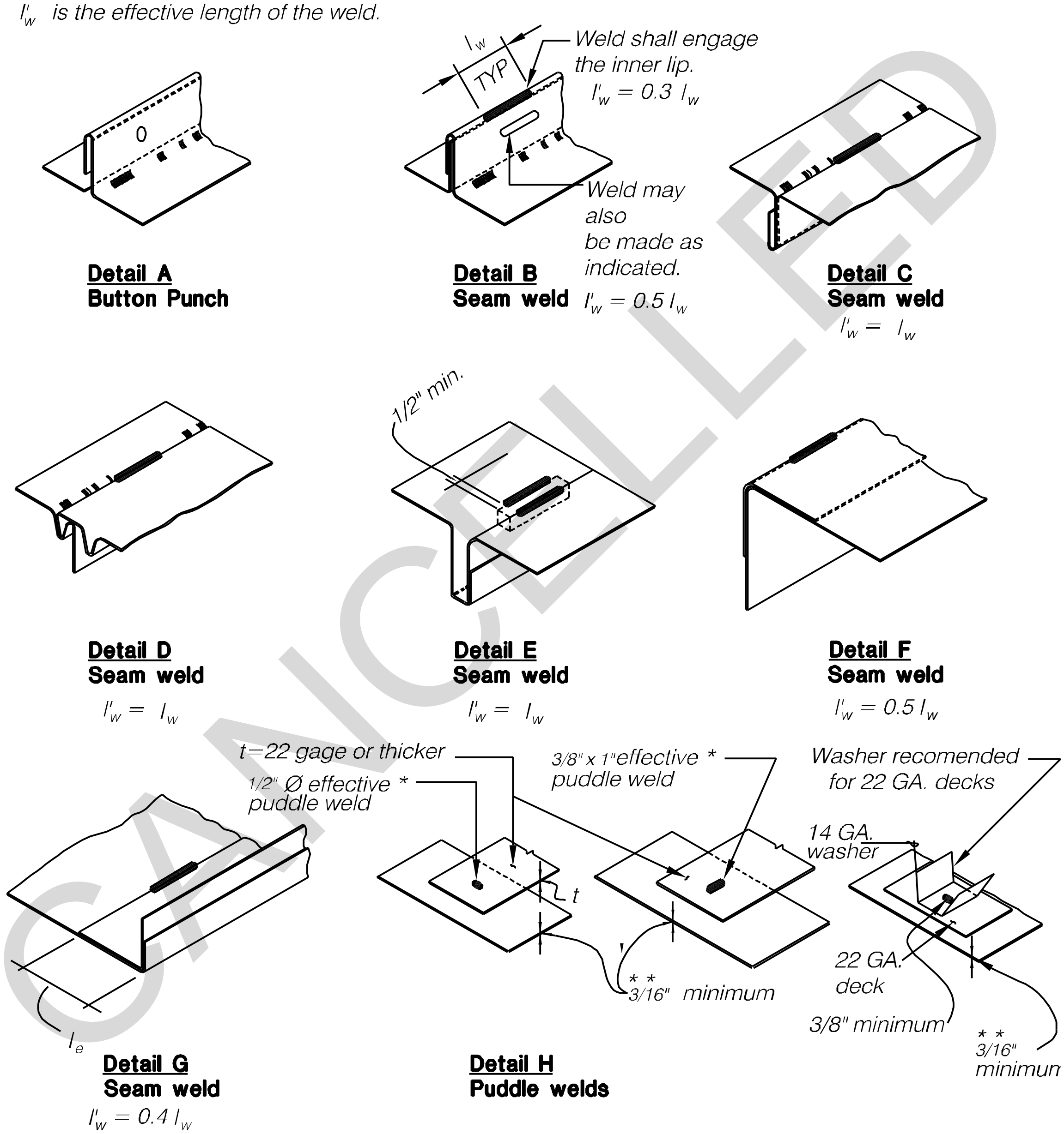
Figure 7-56 Corner of monolithic concrete diagragm.



1 foot = 0.3m
#5 bar = 15M bar

Figure 7-57 Concrete diaphragms - typical connection details.

NOTE: Maximum spacing of seam welds or button punches 3'-0". Minimum lenghth of seam welds - 1" for determining shears on diaphragms. Minimum spacing of seam welds or button punches - 1'-0". Maximum length of seam welds - 2". l'_w is the effective length of the weld.



**NOTE:*
Effective size of Puddle weld indicates size of fusion area of weld metal on framing members.

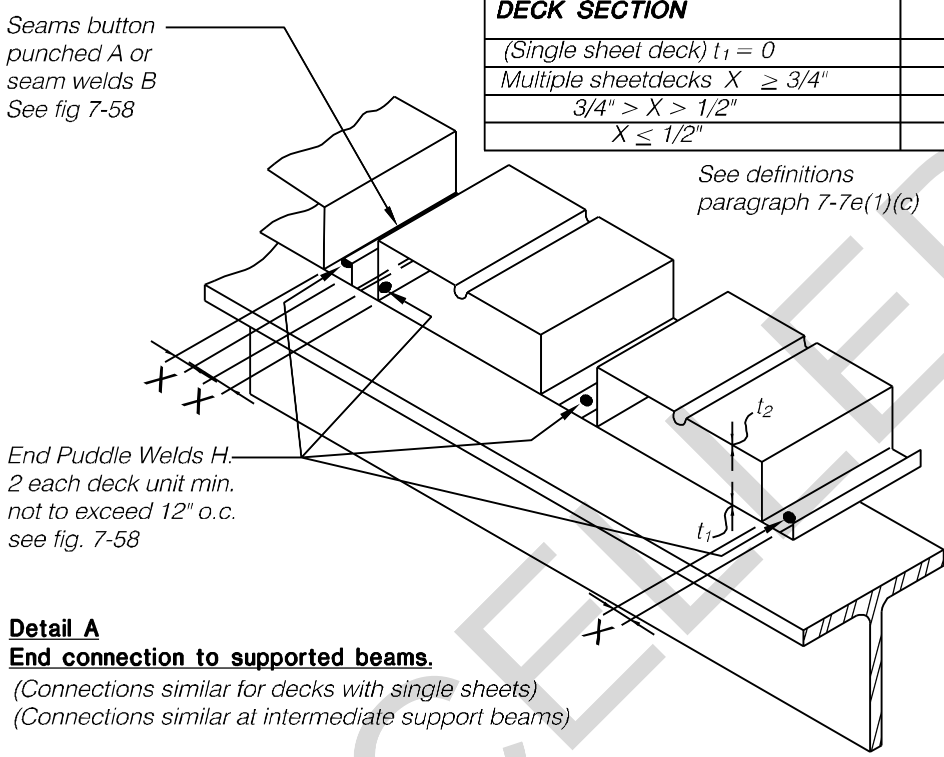
***NOTE:Minimum thickness may be waived by design agency based on manufacturers standard products.*

Figure 7-58 Steel deck diaphragms - Typical details of fastenings

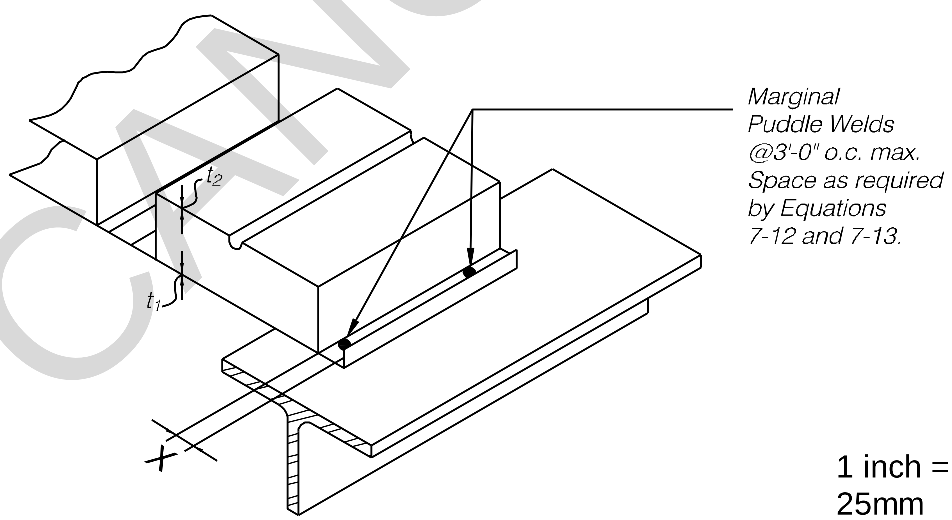
1 inch = 25mm
1 foot = 0.3m

DECK SECTION	t'_2 / t_2
(Single sheet deck) $t_1 = 0$	1
Multiple sheetdecks $X \geq 3/4''$	2/3
$3/4'' > X > 1/2''$	1/4
$X \leq 1/2''$	0

See definitions paragraph 7-7e(1)(c)



Detail A
End connection to supported beams.
(Connections similar for decks with single sheets)
(Connections similar at intermediate support beams)



1 inch = 25mm

Detail B
Connection to marginal beams.

Figure 7-59 Steel deck diaphragms Type A - Typical attachments.

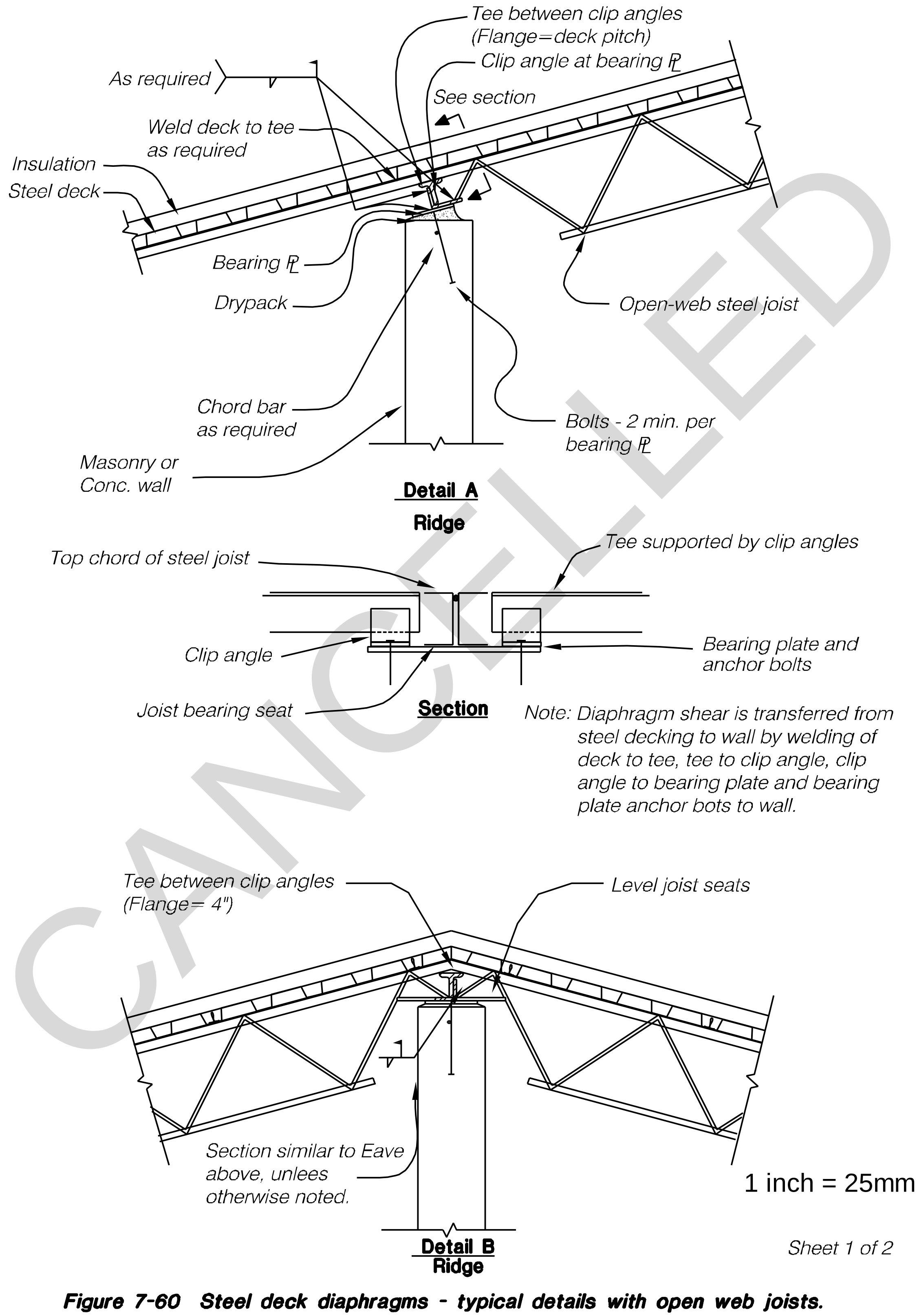
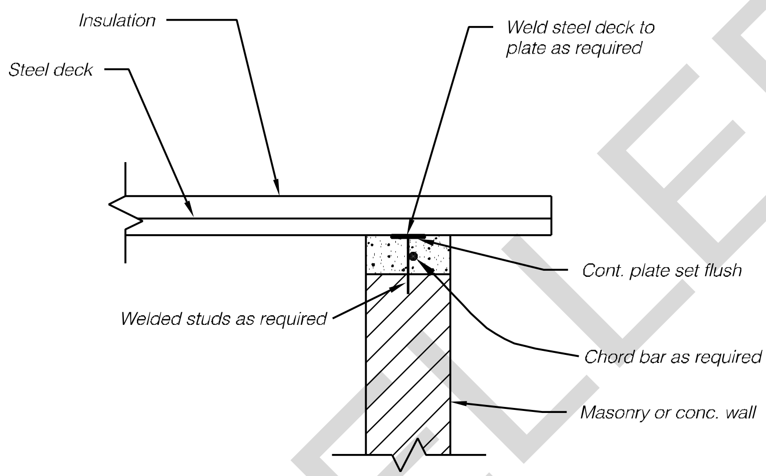
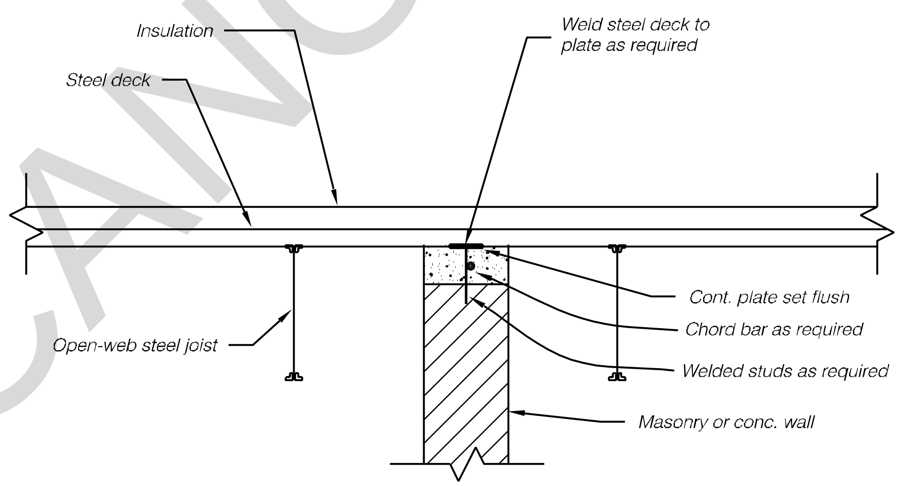


Figure 7-60 Steel deck diaphragms - typical details with open web joists.



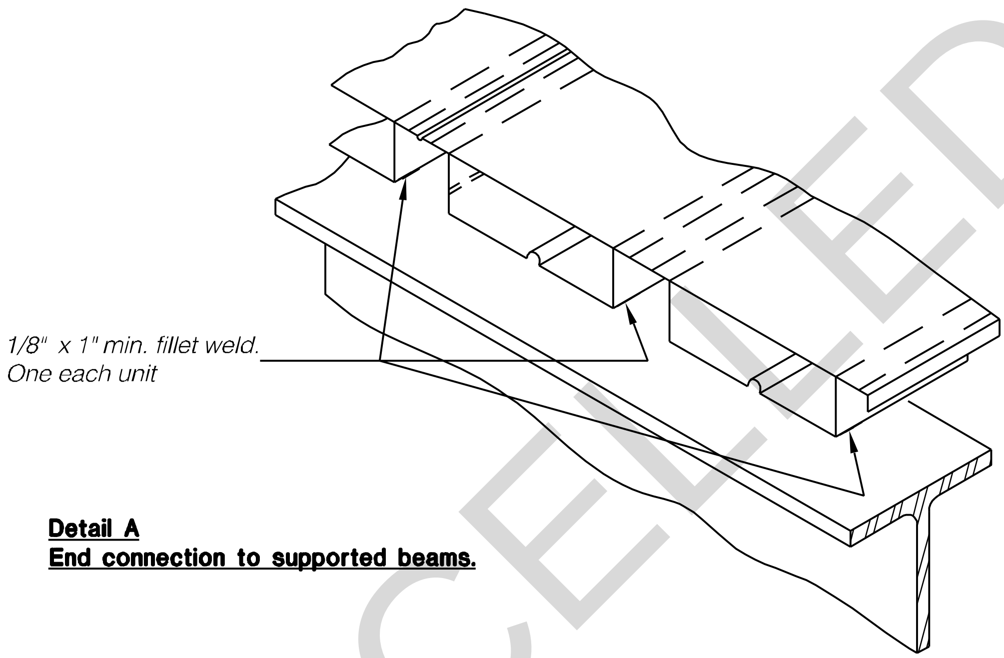
Detail A - Rake



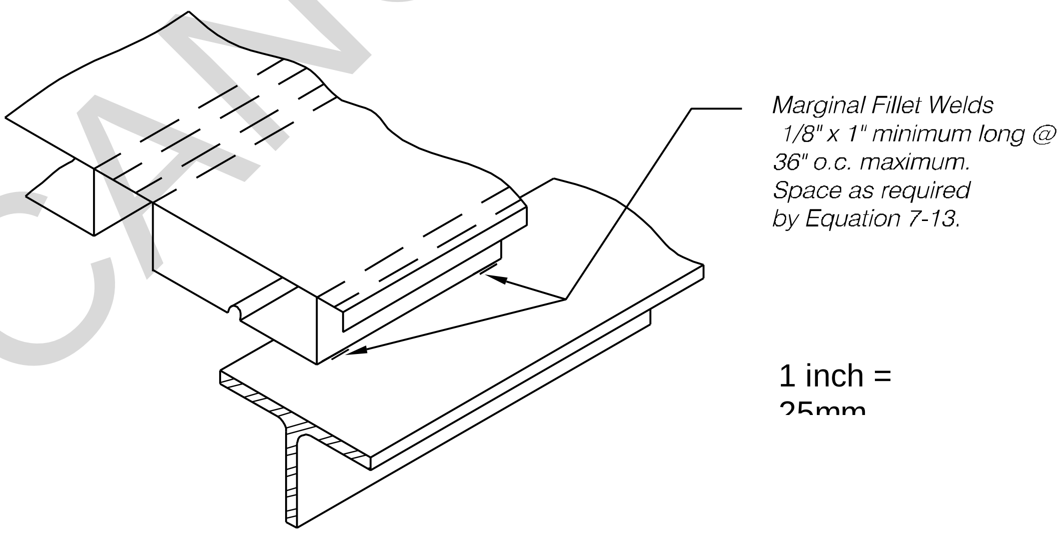
Detail B - Transverse shear wall

Sheet 2 of 2

Figure 7-60 continued



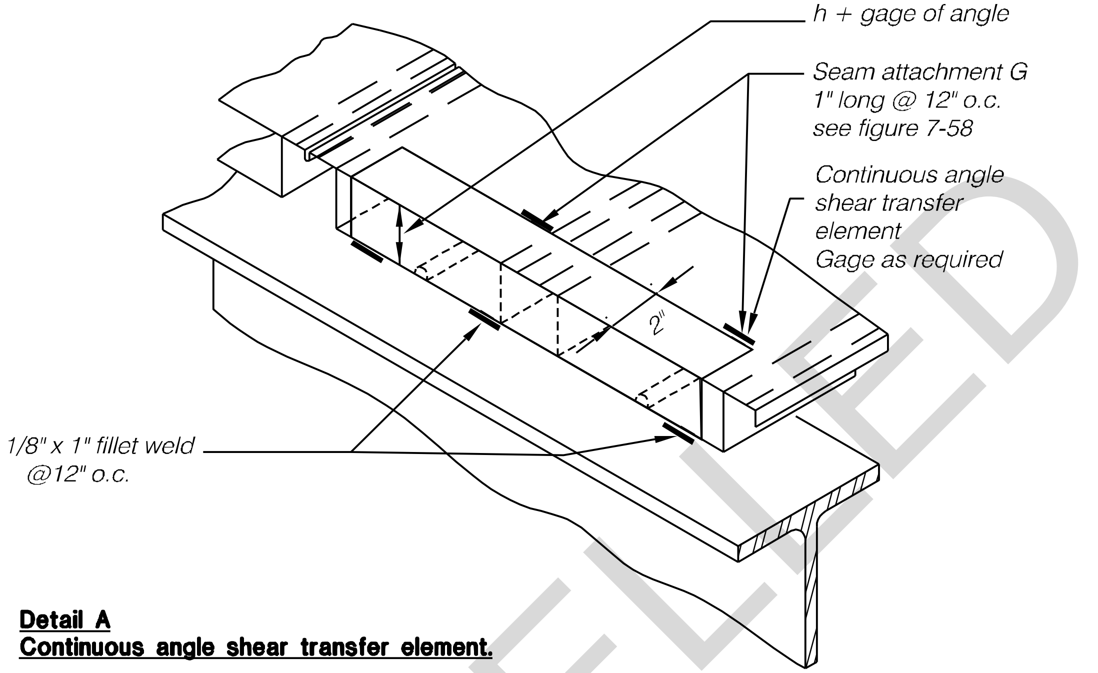
Detail A
End connection to supported beams.



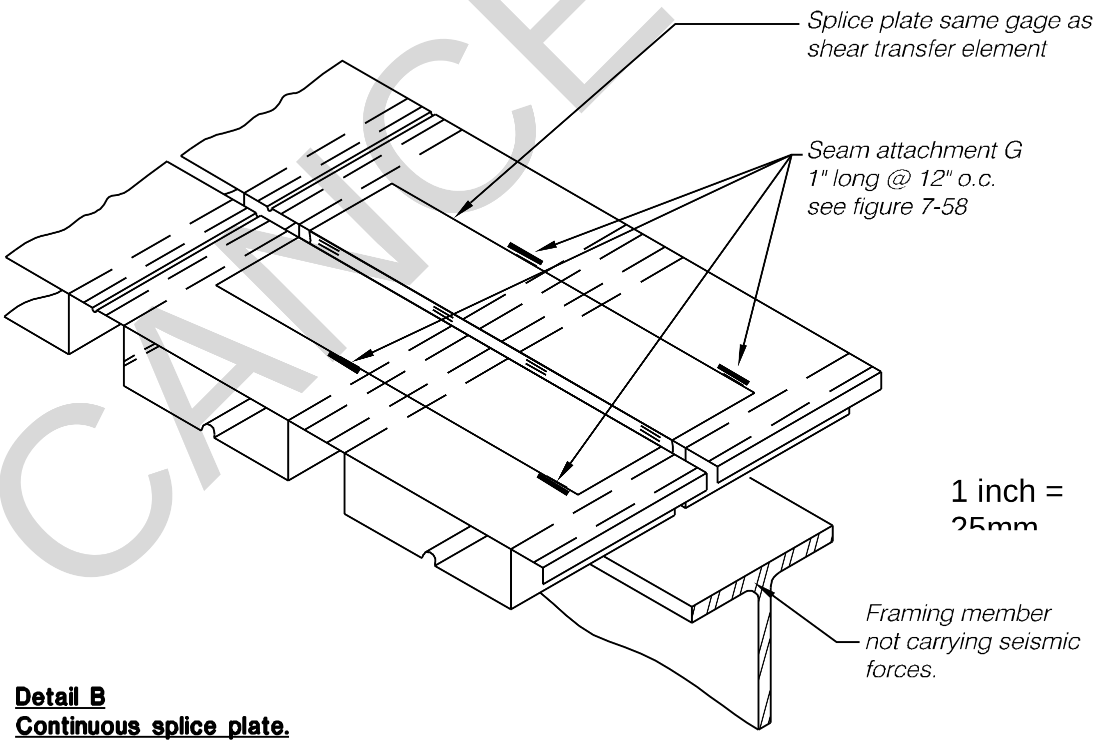
Detail B
Connection to marginal beams.

Sheet 1 of 3

Figure 7-61 Steel deck diaphragms Type B - Typical attachments to frame.



Detail A
Continuous angle shear transfer element.

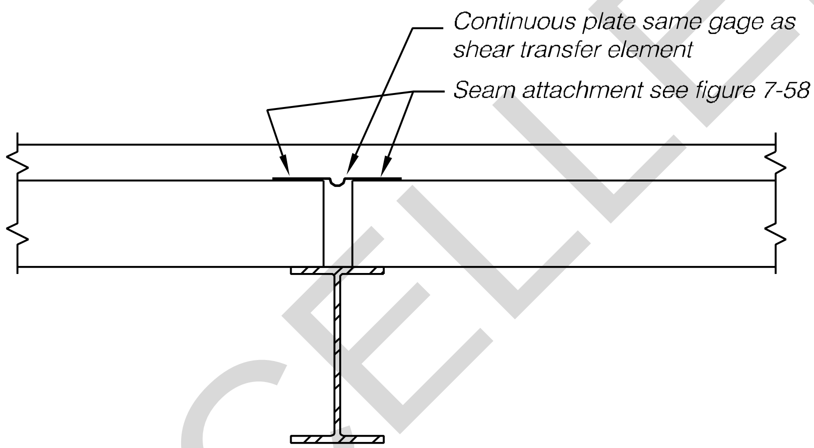


1 inch = 25mm

Detail B
Continuous splice plate.
See sheet 3 for cross section.

Sheet 2 of 3

Figure 7-61 Continued



Detail C

Splice at support

Sheet 3 of 3

Figure 7-61 continued

attached to the deck corrugations, holes placed in the corrugations, or deck profile in which the fluted elements are placed up so that the fill is keyed with the deck. If interlocking between the deck and the concrete is not achieved, then mechanical anchorages will be required to anchor the fill to the supporting member, as prescribed in Paragraph 7-7e(2).

(a) Concrete as a diaphragm. If the diaphragm is loaded and reacted without shear stresses passing through the steel deck or its attachments, the diaphragm is a concrete diaphragm as described in Paragraph 7-7d. Typical attachment details are shown in Figure 7-62, details A and B.

(b) Steel deck as a diaphragm.

1. Shear strength. Nominal shear strength of steel deck diaphragm shall be determined in accordance with approved analytical procedures or with test procedures prepared by a licensed design professional experienced in testing cold-formed steel assemblies, and approved by the authority having jurisdiction. The steel deck installation for the structures, including fasteners, should comply with the test assembly arrangement.

2. Acceptance criteria.

i. The response modification factors, *R*, for steel deck diaphragms conforming to Performance Objective 1A, shall be based on the factor for the applicable structural system in Table 7-1. The allowable shear strength shall be taken as 1.50 times the allowable stress values published by the Steel Deck Institute, or the International Conference of Building Officials.

ii. Modification factors, *m*, for enhanced performance objectives shall be taken as 1.0 for Performance Objective 3B; 1.5 for Performance Objectives 2A and 2B; and 2.0 for Performance Objective 1A.

iii. Steel decking and its attachments are considered to be force-controlled components, and the strength, Q_{CL}, shall be determined as indicated in Paragraph i above.

f. Wood Diaphragms.

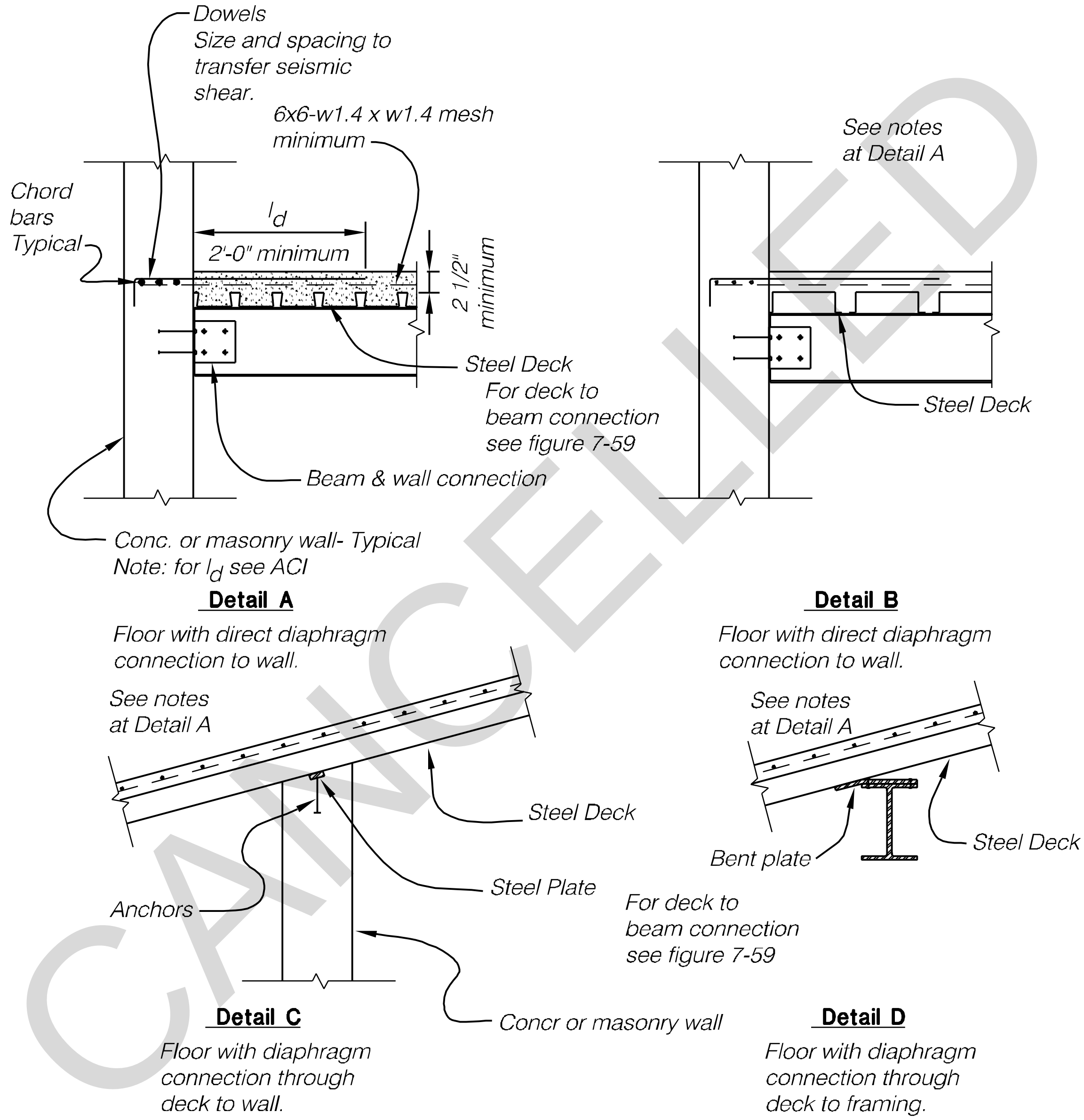
(1) General design criteria. Wood diaphragms will be designed with reference to Section 12.4 of FEMA 302, and the additional criteria of this section.

(2) Wood diaphragms in concrete and masonry buildings; refer to Section 12.3.4.1 of FEMA 302.

(3) Wood buildings with walls on three sides. Provide for rotation as discussed in Paragraph 7-7b(2). Straight sheathing will not be used to resist shears in rotation. The depth of the diaphragm normal to the open side will not exceed 25 feet (7.5m), or two-thirds of the diaphragm width, whichever is the smaller depth.

Exceptions:

(a) One-story wood-frame structures with the depth normal to the open side not greater than 25 feet



Note:

When decks are attached at all shear transfer points similar to details A and B, the diaphragms will be designed in accordance with paragraph 7-7, concrete diaphragms. When shear transfer is through the welds between the steel deck and framing, the diaphragm will be designed in accordance with paragraph 7-7e(4)(b).

1 inch = 25.4 mm
1 foot = 0.3 m

Figure 7-62 Steel deck diaphragms with concrete fill.

(7.5m) may have a depth equal to the width.

(b) Where calculations show that diaphragm deflections can be tolerated, the depth normal to the open end may be increased to a depth-to-width ratio not greater than 1½:1 for diagonal sheathing, or 2:1 for special diagonally sheathed or plywood diaphragms.

(4) Material requirements.

(a) Straight sheathing. Straight sheathing diaphragms will be constructed of 1- or 2-inch (25 or 50mm) nominal boards, 6 or 8 inches (150 or 200mm) nominal in width, with boards laid at right angles to the rafters or joists. Boards will be nailed to each rafter or joist and to peripheral blocking with two 8d common nails for 1-inch by 6-inch (25 x 150mm) and 1-inch by 8-inch (25 x 200mm) sheathing. For 2-inch (50mm) sheathing, nails will be three 16d. End joints of adjacent boards will be separated by at least two joist or rafter spaces, with at least two boards between joints on the same support. They will not be used for the lateral support of masonry, concrete, or other walls that would be seriously affected by high floor-to-floor deflection. Straight sheathing diaphragms are permitted only for buildings in Seismic Design Category A or B.

(b) Diagonal sheathing.

1. Conventional construction. These diaphragms will be made up of 1-inch (25mm) nominal sheathing boards laid at an angle of approximately 45 degrees to supports. Sheathing boards will be nailed directly to each intermediate bearing member with not less than two 8d nails for 1- by 6-inch (25 x 150mm) boards and three 8d nails for boards 8 inches (200mm) or wider, and in addition, three 8d nails and four 8d nails will be used for 6-inch (150mm) and 8-inch (200mm) boards, respectively, at the diaphragm boundaries. End joints in adjacent boards will be separated by at least two joist or stud spaces, and there will be at least two boards between joints on the same support. The boundary or chord members at the edges of diaphragms will be designed to resist direct tensile and compressive chord stresses. This category of diaphragms will also be considered very flexible; such diaphragms will not be used for the lateral support of masonry or concrete walls.

2. Special construction. Special diagonally sheathed diaphragms will include two adjoining layers of 1-inch (25mm) nominal sheathing boards laid diagonally and at 90 degrees to each other. Special diagonally sheathed diaphragms also include single-layered diaphragms, conforming to conventional construction, and which, in addition, will have all elements designed in conformance with the following provision: each chord or portion thereof may be considered as a beam loaded with a uniform load per foot equal to 50 percent of the unit shear due to diaphragm action. The load will be assumed as acting normal to the chord in the plane of the diaphragm, and either toward or away from the diaphragm. The span of the chord, or portion thereof, will be the distance between structural members of the diaphragm, such as joists or blocking, which serve to transfer the assumed load to the sheathing. Special diagonally sheathed diaphragms may be used to resist shears due to seismic forces, provided such shears do not stress the nails beyond their allowable safe lateral strength. For approximating deflections, a value of *F* of 75 will be used; thus, special diagonally sheathed diaphragms also fit into the category of flexible diaphragms.

(c) Plywood sheathing.

1. Boundary members. All boundary members will be proportioned and spliced where necessary to transmit direct stresses. The nominal width of the framing members will be at least 2 inches (50mm). In general, panel edges will bear on the framing members and butt along their centerlines. Nails will be placed not less than 3/8 inch (10mm) in from the panel edge, not more than 12 inches (300mm) apart along intermediate supports, and 6 inches (150mm) along panel edge bearings, and will be firmly driven into the framing members. No unblocked panels less than 12 inches (300mm) wide will be used.

2. Nailing. The use of pneumatically or mechanically driven steel wire staples with a minimum crown width of 7/16 inch (11mm) is an acceptable alternative method of attaching diaphragms. The crown of the staple must be installed parallel to the framing member.

Common Wire Nail	Staple	Minimum Staple Penetration In Framing Member
6d	14 gauge	1 inch
8d	13 gauge	1 inch
10d	1 1/8 inch	12 gauge

1 inch = 25mm

3. Typical details. Refer to Figure 7-63.

(5) Acceptance criteria.

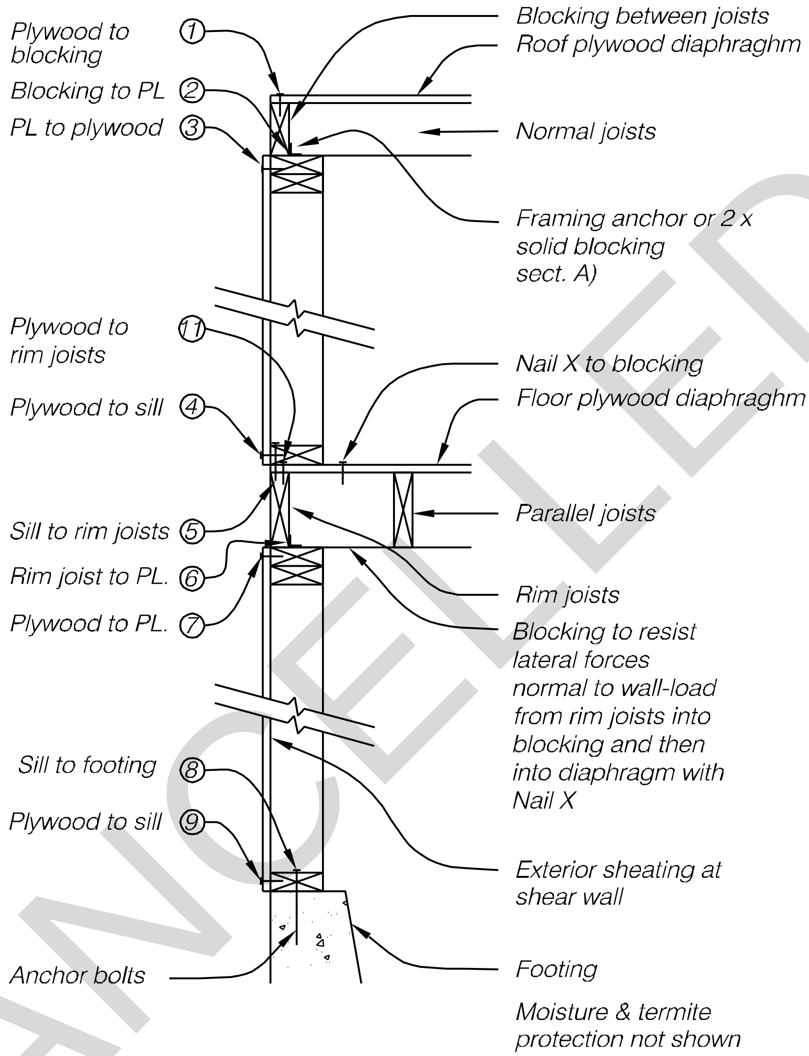
(a) The response modification factors, *R*, for wood diaphragms conforming to Performance Objectives 1A shall be based on the factor for the applicable structural system in Table 7-1. The nominal shear strength of wood diaphragms will be calculated by principles of mechanics using values of fastener strength and shear resistance in the sheathing material based on approved values from cyclical tests. The design strength shall be the nominal strength multiplied by a resistance factor, *N*, equal to 0.65. When approved allowable stress (including a one-third increase for wind or seismic loads), the design strength may be taken as 1.50 times the allowable values. Factored shear capacity values (i.e., including capacity reduction factor, *N*, equal to 0.65) for plywood diaphragm are provided in Table 7-25.

(b) Modification factors, *m*, for enhanced performance objectives are provided in Table 7-26.

(c) Modeling parameters and numerical acceptance criteria for nonlinear procedures are provided in Table 7-27.

(d) Expected strength, Q_{CE}, for deformation-controlled connections and other elements shall be taken as the nominal strength calculated in accordance with FEMA 302, without the *N* factor.

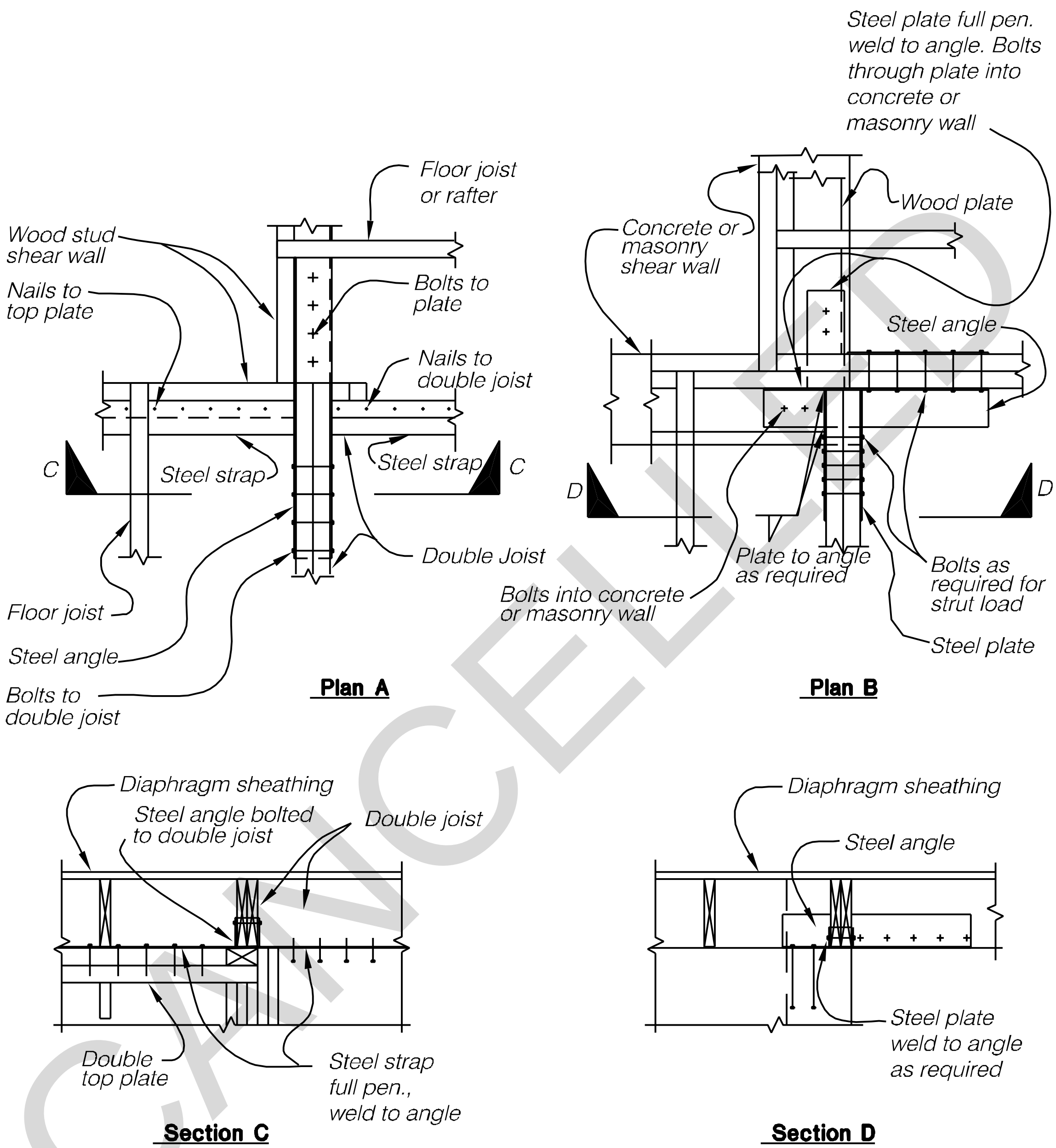
(e) Lower-bound strength for force-controlled connections and other elements shall be



① - ⑨ Path of forces from roof to foundation
⑪ , ⑥ - ⑨ Path for forces from floor diaphraghm
details above are schematic, the purpose is to show the path of forces in a particular arrangement of framing elements.

Sheet 1 of 4

Figure 7-63 Wood details.



Note:
Bolts and nailing to be designed for diaphragm strut or chord loads. Roof connections will be similar but modified because of roof slope.

Sheet 2 of 4

Figure 7-63 Wood details continued.

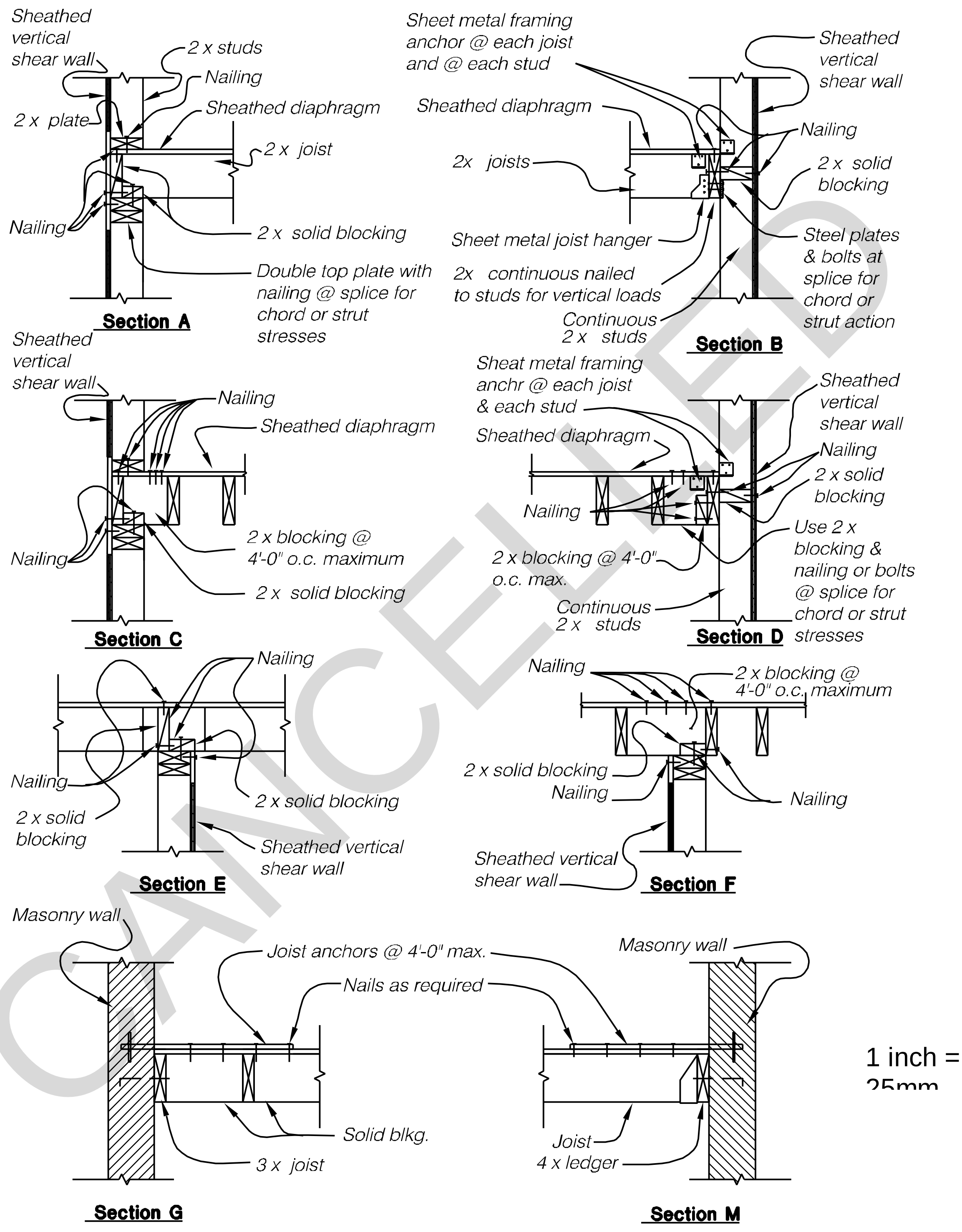
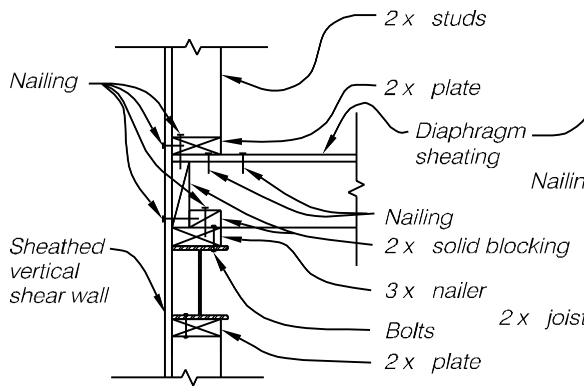
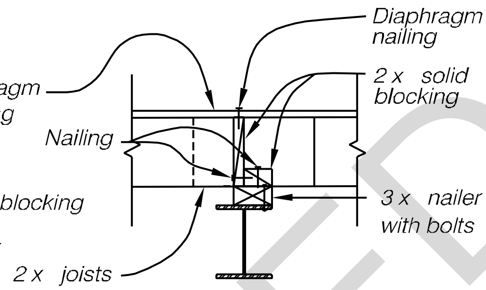


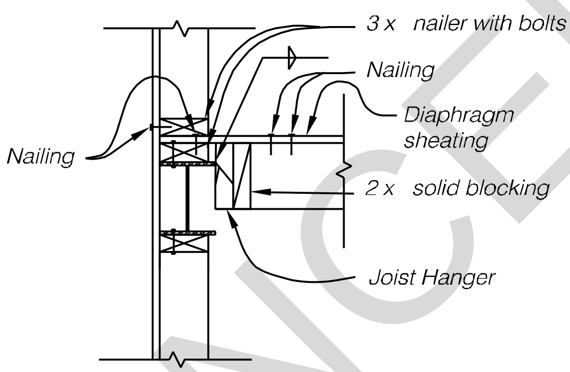
Figure 7-63 continued. Sheet 3 of 4



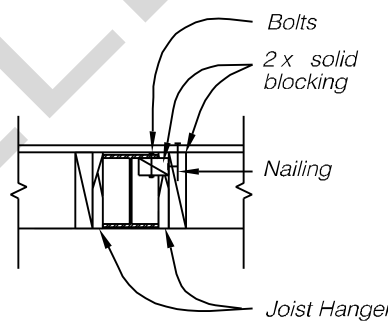
Section A



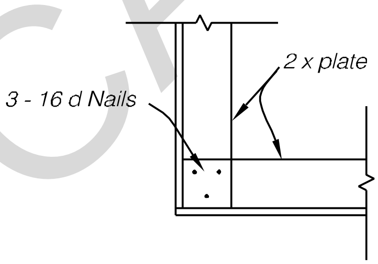
Section B



Section C



Section D



Plan E
Top Plate Lap at Corner

Note:
Nails and bolts shown on details will be designed to resist the prescribed seismic shears and wall anchorages. Roof connectoins will be similar but modified because of roof slope.

1 inch = 25mm

Sheet 4 of 4

Figure 7-63 Wood details continued.

TABLE 7-25

Factored Shear Resistance in Kips per Foot for Horizontal Wood Diaphragms With Framing Members of Douglas Fir-Larch or Southern Pine for Seismic Loading [a,b]

Panel Grade	Fastener [c] Type	Fastener [c] Minimum penetration in framing (in.)	Minimum nominal panel thickness (in.)	Minimal nominal width of framing (in.)	Lines of fasteners	Blocked Diaphragms: Fastener spacing at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 and 4) and at all panel edges (Cases 5 and 6) [c]: 6 / Spacing per line at other panel edges (in.): 6	4 / 6	4 / 4	2½ [f] / 4	2½ [f] / 3	2 [f] / 3	2 [f] / 2	Unblocked Diaphragms [d]: Fastener spacing at 6-inch centers at supported edges: Case 1	Cases 2, 3, 4, 5, and 6
Structural 1	6d common	1¼	3/8	2	1	0.24	0.33	---	0.49	---	0.55	---	0.21	0.16
				3	1	0.27	0.36	---	0.55	---	0.62	---	0.24	0.18
	8d common	1½	3/8	2	1	0.35	0.47	---	0.69	---	0.78	---	0.31	0.23
				3	1	0.39	0.52	---	0.78	---	0.88	---	0.34	0.26
	10d [g] common	1-5/8	15/32	2	1	0.42	0.55	---	0.83	---	0.95	---	0.37	0.28
				3	1	0.47	0.62	---	0.94	---	1.07	---	0.42	0.31
	10d [g] common	1-5/8	23/32	3	2	---	0.85	1.13	1.22	1.60	---	---	---	---
				4	2	---	0.98	1.27	1.40	1.83	---	---	---	---
				4	3	---	1.22	1.70	1.79	2.35	---	---	---	---
	14 gauge staples	2	23/32	3	2	---	0.78	0.78	1.09	1.17	1.35	1.56	---	---
				4	3	---	1.09	1.17	1.48	1.76	1.87	2.34	---	---
Sheathing, Single Floor and Other Grades Covered in References 9.10 and 9.11	6d common	1¼	3/8	2	1	0.24	0.33	---	0.49	---	0.55	---	0.21	0.16
				3	1	0.27	0.36	---	0.55	---	0.62	---	0.24	0.18
	8d common	1½	3/8	2	1	0.31	0.42	---	0.62	---	0.71	---	0.28	0.21
				3	1	0.35	0.47	---	0.70	---	0.79	---	0.31	0.23
			7/16	2	1	0.33	0.44	---	0.66	---	0.75	---	0.30	0.22
				3	1	0.37	0.49	---	0.74	---	0.84	---	0.33	0.25
			15/32	2	1	0.35	0.47	---	0.69	---	0.78	---	0.31	0.23
				3	1	0.39	0.52	---	0.78	---	0.88	---	0.34	0.26
	10d [g] common	1-5/8	15/32	2	1	0.38	0.50	---	0.75	---	0.85	---	0.33	0.25
				3	1	0.42	0.56	---	0.85	---	0.96	---	0.38	0.28
			19/32	2	1	.042	0.55	---	0.83	---	0.95	---	0.37	0.28
				3	1	0.47	0.62	---	0.94	---	1.07	---	0.42	0.31
			23/32	3	2	---	0.84	1.13	1.22	1.59	---	---	---	---
				4	2	---	0.98	1.27	1.40	1.81	---	---	---	---
				4	3	---	1.22	1.70	1.78	1.96	---	---	---	---
	14 gauge staples	2	23/32	3	2	---	0.78	0.78	1.07	1.17	1.33	1.56	---	---
				4	3	---	1.07	1.17	1.46	1.76	1.82	1.96	---	---

TABLE 7-25 (cont.)

Factored Shear Resistance in Kips per Foot for Horizontal Wood DiaphragmsWith Framing Members of Douglas Fir-Larch or Southern Pine for Seismic Loading [a,b]

a λ = 1.0 $\phi = 0.65$

b l/w shall not be more than 4/1 for blocked diaphragms or more than 3/1 for unblocked diaphragms. For framing members of other species set forth in ASCE 16-95, with the range of specific gravity (SG) noted, allowable shear values shall be calculated for all panel grades by multiplying the values for Structural I by the following factors: 0.82 for SG equal to or greater than 0.42, but less than 0.49 (0.42 = SG < 0.49) and 0.65 for SG less than 0.42 (SG < 0.42).

c Space nails along intermediate framing members at 12-inch centers except where spans are greater than 32 inches, space nails at 6-inch centers.

d Blocked values are permitted to be used for 1-1/8-inch panels with tongue-and-groove edges where 1 in. by 3/8-inch crown by No. 16 gauge staples are driven through the tongue-and-groove edges 3/8 inch from the panel edge so as to penetrate the tongue. Staples shall be spaced at one-half the boundary nail spacing for Cases 1 and 2, and at one-third the boundary nail spacing for Cases 3 through 6.

e Maximum shear for Cases 3 through 6 is limited to 1,500 pounds per foot.

f For values listed for 2-inch nominal framing member width, the framing members at adjoining panel edges shall be 3 inches nominal width. Nails at panel edges shall be placed in two lines at these locations.

g Framing at adjoining panel edges shall be 3-inch nominal or wider and nails shall be staggered where 10d nails having penetration into framing of more than 1-5/8 inches are spaced 3 inches or less on center.

Metric equivalents:
1 inch = 25mm
1 pound/foot = 14.58 N/m
1 kp/foot = 14.58 kN/m

		m Factors for Linear Procedures[2]					
		Primary				Secondary	
		IO	SE	LS	CP	LS	CP
Shear Walls	**Height/Length Ratio (h/L)**[1]						
Horizontal 1" x 6" Sheathing	$h/L < 1.0$	1.8	3.0	4.2	5.0	5.0	5.5
Horizontal 1" x 10" Sheathing	$h/L < 1.0$	1.6	2.5	3.4	4.0	4.0	5.0
Horizontal Wood Siding Over Horizontal 1" x 6" Sheathing	$h/L < 1.5$	1.4	2.0	2.6	3.0	3.1	4.0
Horizontal Wood Siding Over Horizontal 1" x 10" Sheathing	$h/L < 1.5$	1.3	1.8	2.3	2.6	2.8	3.0
Diagonal 1" x 6" Sheathing	$h/L < 1.5$	1.5	2.2	2.9	3.3	3.4	3.8
Diagonal 1" x 8" Sheathing	$h/L < 1.5$	1.4	2.1	2.7	3.1	3.1	3.6
Horizontal Wood Siding Over Diagonal 1" x 6" Sheathing	$h/L < 2.0$	1.3	1.8	2.2	2.5	2.5	3.0
Horizontal Wood Siding Over Diagonal 1" x 8" Sheathing	$h/L < 2.0$	1.3	1.7	2.0	2.3	2.5	2.8
Double Diagonal 1" x 6" Sheathing	$h/L < 2.0$	1.2	1.5	1.8	2.0	2.3	2.5
Double Diagonal 1" x 8" Sheathing	$h/L < 2.0$	1.2	1.5	1.7	1.9	2.0	2.5
Vertical 1" x 10" Sheathing	$h/L < 1.0$	1.5	2.3	3.1	3.6	3.6	4.1
Structural Panel or Plywood Panel Sheathing or Siding	$h/L < 1.0$*	1.7	2.3	3.8	4.5	4.5	5.5
	$h/L > 2.0$* $h/L < 3.5$	1.4	2.0	2.6	3.0	3.0	4.0
Stucco on Studs	$h/L<1.0$ *	1.5	2.3	3.1	3.6	3.6	4.0
	$h/L = 2.0$ *	1.3	1.8	2.2	2.5	2.5	3.0
Stucco over 1" x Horizontal Sheathing	$h/L < 2.0$	1.5	2.2	3.0	3.5	3.5	4.0
Gypsum Plaster on Wood Lath	$h/L < 2.0$	1.7	2.8	3.9	4.6	4.6	5.1
Gypsum Plaster on Gypsum Lath	$h/L < 2.0$	1.8	3.0	4.2	5.0	4.2	5.5
Gypsum Plaster on Metal Lath	$h/L < 2.0$	1.7	2.7	3.7	4.4	3.7	5.0
Gypsum Sheathing	$h/L < 2.0$	1.9	3.3	4.7	5.7	4.7	6.0
Gypsum Wallboard	$h/L < 1.0$ *	1.9	3.3	4.7	5.7	4.7	6.0
	$h/L = 2.0$ *	1.6	2.5	3.4	4.0	3.8	4.5
Horizontal 1" x 6" Sheathing With Cut-In Braces or Diagonal Blocking	$h/L < 1.0$	1.7	2.7	3.7	4.4	4.2	4.8
Fiberboard or Particleboard Sheathing	$h/L < 1.5$	1.6	2.4	3.2	3.8	3.8	5.0

1. For ratios greater than the maximum listed values, the component is considered not effective in resisting lateral loads.
2 Linear interpolation is permitted for intermediate value if h/L has asterisks.

Table 7-26: Numerical Acceptance Factors for Linear Procedures—Wood Components

		m Factors for Linear Procedures[2]					
		Primary				Secondary	
		IO	SE	LS	CP	LS	CP
Diaphragms	**Length/Width Ratio (*L/b*)[1]**						
Single Straight Sheathing, Chorded	*L/b* < 2.0	1	1.5	2.0	2.5	2.4	3.1
Single Straight Sheathing, Unchorded	*L/b* < 2.0	1	1.3	1.5	2.0	1.8	2.5
Double Straight Sheathing, Chorded	*L/b* < 2.5	1.25	1.6	2.0	2.5	2.3	2.8
Double Straight Sheathing, Unchorded	*L/b* < 2.5	1	1.3	1.5	2.0	1.8	2.3
Single Diagonal Sheathing, Chorded	*L/b* < 2.5	1.25	1.6	2.0	2.5	2.3	2.9
Single Diagonal Sheathing, Unchorded	*L/b* < 2.0	1	1.3	1.5	2.0	1.8	2.5
Straight Sheathing Over Diagonal Sheathing, Chorded	*L/b* < 3.0	1.5	2.0	2.5	3.0	2.8	3.5
Straight Sheathing Over Diagonal Sheathing, Unchorded	*L/b* < 2.5	1.25	1.6	2.0	2.5	2.3	3.0
Double Diagonal Sheathing, Chorded	*L/b* < 3.5	1.5	2.0	2.5	3.0	2.9	3.5
Double Diagonal Sheathing, Unchorded	*L/b* < 3.5	125	1.6	2.0	2.5	2.4	3.1
Wood Structural Panel, Blocked, Chorded	*L/b* < 3.0 * *L/b* = 4*	1.5 1.5	2.3 2.0	3.0 2.5	4.0 3.0	3.5 2.8	4.5 3.5
Wood Structural Panel, Unblocked, Chorded	*L/b* < 3* *L/b* = 4*	1.5 1.5	2.0 1.8	2.5 2.0	3.0 2.5	2.9 2.6	4.0 3.2
Wood Structural Panel, Blocked, Unchorded	*L/b* < 2.5 *L/b* = 3.5	1.25 1.25	1.9 1.6	2.5 2.0	3.0 2.5	2.9 2.6	4.0 3.2
Wood Structural Panel, Unblocked, Unchorded	*L/b* < 2.5 *L/b* = 3.5	1.25 1.0	1.6 1.3	2.0 1.5	2.5 2.0	2.4 2.0	3.0 2.6
Wood Structural Panel Overlay on Sheathing, Chorded	*L/b* < 3* *L/b* = 4*	1.5 1.5	2.0 1.8	2.5 2.0	3.0 2.5	2.9 2.6	4.0 3.2
Wood Structural Panel Overlay on Sheathing, Unchorded	*L/b* < 2.5 *L/b* = 3.5	1.25 1.0	1.6 1.3	2.0 1.5	2.5 2.0	2.4 1.9	3.0 2.6
Component/Element							
Frame elements subject to axial and bending stresses		1.0	1.8	2.5	3.0	2.5	4.0
Connections							
Nails - 8d and larger - Wood to Wood		2.0	4.0	6.0	8.0	8.0	9.0
Nails - 8d and larger - Metal to Wood		2.0	3.0	4.0	6.0	5.0	7.0
Screws - Wood to Wood		1.2	1.6	2.0	2.2	2.0	2.5
Screws - Metal to Wood		1.1	1.5	1.8	2.0	1.8	2.3
Lag Bolts - Wood to Wood		1.4	2.0	2.5	3.0	2.5	3.3
Lag Bolts - Metal to Wood		1.3	1.8	2.3	2.5	2.4	3.0
Machine Bolts - Wood to Wood		1.3	2.2	3.0	3.5	3.3	3.9
Machine Bolts - Metal to Wood		1.4	2.1	2.8	3.3	3.1	3.7
Split Rings and Shear Plates		1.3	1.8	2.2	2.5	2.3	2.7
Bolts - Wood to Concrete or Masonry		1.4	2.1	2.7	3.0	2.8	3.5

1. For ratios greater than the maximum listed values, the component is considered not effective in resisting lateral loads.
2 Linear interpolation is permitted for intermediate value if h/L has asterisks.

Table 7-26: Numerical Acceptance Factors for Linear Procedures—Wood Components (Continued)

		d	e	c
Shear Wall Type - Types of Existing Wood and Light Frame Shear Walls	**Height/Length Ratio *h/L*[1]**			
Horizontal 1" x 6" Sheathing	*h/L* < 1.0	5.0	6.0	0.3
Horizontal 1" x 10" Sheathing	*h/L* < 1.0	4.0	5.0	0.3
Horizontal Wood Siding Over Horizontal 1" x 6" Sheathing	*h/L* < 1.5	3.0	4.0	0.2
Horizontal Wood Siding Over Horizontal 1" x 10" Sheathing	*h/L* < 1.5	2.6	3.6	0.2
Diagonal 1" x 6" Sheathing	*h/L* < 1.5	3.3	4.0	0.2
Diagonal 1" x 8" Sheathing	*h/L* < 1.5	3.1	4.0	0.2
Horizontal Wood Siding Over Diagonal 1" x 6" Sheathing	*h/L* < 2.0	2.5	3.0	0.2
Horizontal Wood Siding Over Diagonal 1" x 8" Sheathing	*h/L* < 2.0	2.3	3.0	0.2
Double Diagonal 1" x 6" Sheathing	*h/L* < 2.0	2.0	2.5	0.2
Double Diagonal 1" x 8" Sheathing	*h/L* < 2.0	2.0	2.5	0.2
Vertical 1" x 10" Sheathing	*h/L* < 1.0	3.6	4.0	0.3
Structural Panel or Plywood Panel Sheathing or Siding	*h/L* < 1.0*	4.5	5.5	0.3
	h/L > 2.0* *h/L* < 3.5	3.0	4.0	0.2
Stucco on Studs	*h/L* < 1.0*	3.6	4.0	0.2
	h/L = 2.0*	2.5	3.0	0.2
Stucco over 1" x Horizontal Sheathing	*h/L* < 2.0	3.5	4.0	0.2
Gypsum Plaster on Wood Lath	*h/L* < 2.0	4.6	5.0	0.2
Gypsum Plaster on Gypsum Lath	*h/L* < 2.0	5.0	6.0	0.2
Gypsum Plaster on Metal Lath	*h/L* < 2.0	4.4	5.0	0.2
Gypsum Sheathing	*h/L* < 2.0	5.7	6.3	0.2
Gypsum Wallboard	*h/L* < 1.0*	5.7	6.3	0.2
	h/L = 2.0*	4.0	5.0	0.2
Horizontal 1" x 6" Sheathing With Cut-In Braces or Diagonal Blocking	*h/L* < 1.0	4.4	5.0	0.2
Fiberboard or Particleboard Sheathing	*h/L* < 1.5	3.8	4.0	0.2
Diaphragm Type - Horizontal Wood Diaphragms	**Length/Width Ratio (*L/b*)[1]**			
Single Straight Sheathing, Chorded	*L/b* < 2.0	2.5	3.5	0.2
Single Straight Sheathing, Unchorded	*L/b* < 2.0	2.0	3.0	0.3
Double Straight Sheathing, Chorded	*L/b* < 2.0	2.5	3.5	0.2
Double Straight Sheathing, Unchorded	*L/b* < 2.0	2.0	3.0	0.3

1. For ratios greater than the maximum listed values, the component is considered not effective in resisting lateral loads.

Notes: (a) Acceptance criteria for primary components

(Δ/Δ_y) IO = 1.0 + 0.2 (d - 1.0)
(Δ/Δ_y) SE = 1.0 + 0.5 (d - 1.0)
(Δ/Δ_y) LS = 1.0 + 0.8 (d - 1.0)
(Δ/Δ_y) CP = d

(b) Acceptance criteria for secondary components

(Δ/Δ_y) LS = d
(Δ/Δ_y) CP = e

(c) Linear interpolation is permitted for intermediate values if h/L or L/b has asterisks.

Table 7-27: Normalized Force-Deflection Curve Coordinates for Nonlinear Procedures—Wood Components

		d	e	c
Single Diagonal Sheathing, Chorded	*L/b* < 2.0	2.5	3.5	0.2
Single Diagonal Sheathing, Unchorded	*L/b* < 2.0	2.0	3.0	0.3
Straight Sheathing Over Diagonal Sheathing, Chorded	*L/b* < 2.0	3.0	4.0	0.2
Straight Sheathing Over Diagonal Sheathing, Unchorded	*L/b* < 2.0	2.5	3.5	0.3
Double Diagonal Sheathing, Chorded	*L/b* < 2.0	3.0	4.0	0.2
Double Diagonal Sheathing, Unchorded	*L/b* < 2.0	2.5	3.5	0.2
Wood Structural Panel, Blocked, Chorded	*L/b* < 3* *L/b* = 4*	4.0 3.0	5.0 4.0	0.3
Wood Structural Panel, Unblocked, Chorded	*L/b* < 3* *L/b* = 4*	3.0 2.5	4.0 3.5	0.3
Wood Structural Panel, Blocked, Unchorded	*L/b* < 2.5* *L/b* = 3.5*	3.0 2.5	4.0 3.5	0.3
Wood Structural Panel, Unblocked, Unchorded	*L/b* < 2.5* *L/b* = 3.5*	2.5 2.0	3.5 3.0	0.4
Wood Structural Panel Overlay On Sheathing, Chorded	*L/b* < 3* *L/b* = 4*	3.0 2.5	4.0 3.5	0.3
Wood Structural Panel Overlay On Sheathing, Unchorded	*L/b* < 2.5* *L/b* = 3.5*	2.5 2.0	3.5 3.0	0.4
Connection Type				
Nails - Wood to Wood		7.0	8.0	0.2
Nails - Metal to Wood		5.5	7.0	0.2
Screws - Wood to Wood		2.5	3.0	0.2
Screws - Wood to Metal		2.3	2.8	0.2
Lag Bolts - Wood to Wood		2.8	3.2	0.2
Lag Bolts - Metal to Wood		2.5	3.0	0.2
Bolts - Wood to Wood		3.0	3.5	0.2
Bolts - Metal to Wood		2.8	3.3	0.2

1. For ratios greater than the maximum listed values, the component is considered not effective in resisting lateral loads.

Notes: (a) Acceptance criteria for primary components

(Δ/Δ_y) IO = 1.0 + 0.2 (d - 1.0)
(Δ/Δ_y) SE = 1.0 + 0.5 (d - 1.0)
(Δ/Δ_y) LS = 1.0 + 0.8 (d - 1.0)
(Δ/Δ_y) CP = d

(b) Acceptance criteria for secondary components

(Δ/Δ_y) LS = d
(Δ/Δ_y) CP = e

(c) Linear interpolation is permitted for intermediate values if h/L or L/b has asterisks.

Table 7-27: Normalized Force-Deflection Curve Coordinates for Nonlinear Procedures—Wood Components (Continued)

determined in accordance with FEMA 302, includingthe *N* factor.

g. Horizontal Bracing.

(1) General. Diaphragms may be made of horizontal steel bracing. Usually, the bracing consists of members added to the top or bottom plane of a system of floor or roof trusses or beams. Transverse elements are added for components perpendicular to the trusses or beams, and diagonal members are added to form a triangulated plane of bracing.

(2) Acceptance criteria.

(a) Response modification factors, *R*, for horizontal bracing in buildings conforming to Performance Objective 1A will be based on the factors for the applicable structural systems in Table 7-1. Bracing members and connections will be designed as prescribed for vertical steel concentric braced frames in Paragraph 7-3b.

(b) Modification factors, *m*, for enhanced performance objectives and modeling parameters and numerical acceptance criteria shall be as prescribed for concentric steel braced frames in Paragraph 7-3b(9).

(c) Expected strength for deformation-controlled components and lower-bound strength for force-controlled components shall also be as prescribed for concentric braced frames in Paragraph 7-3b(9).

CHAPTER 8

SEISMIC ISOLATION AND ENERGY DISSIPATION SYSTEMS

8-1. Introduction.

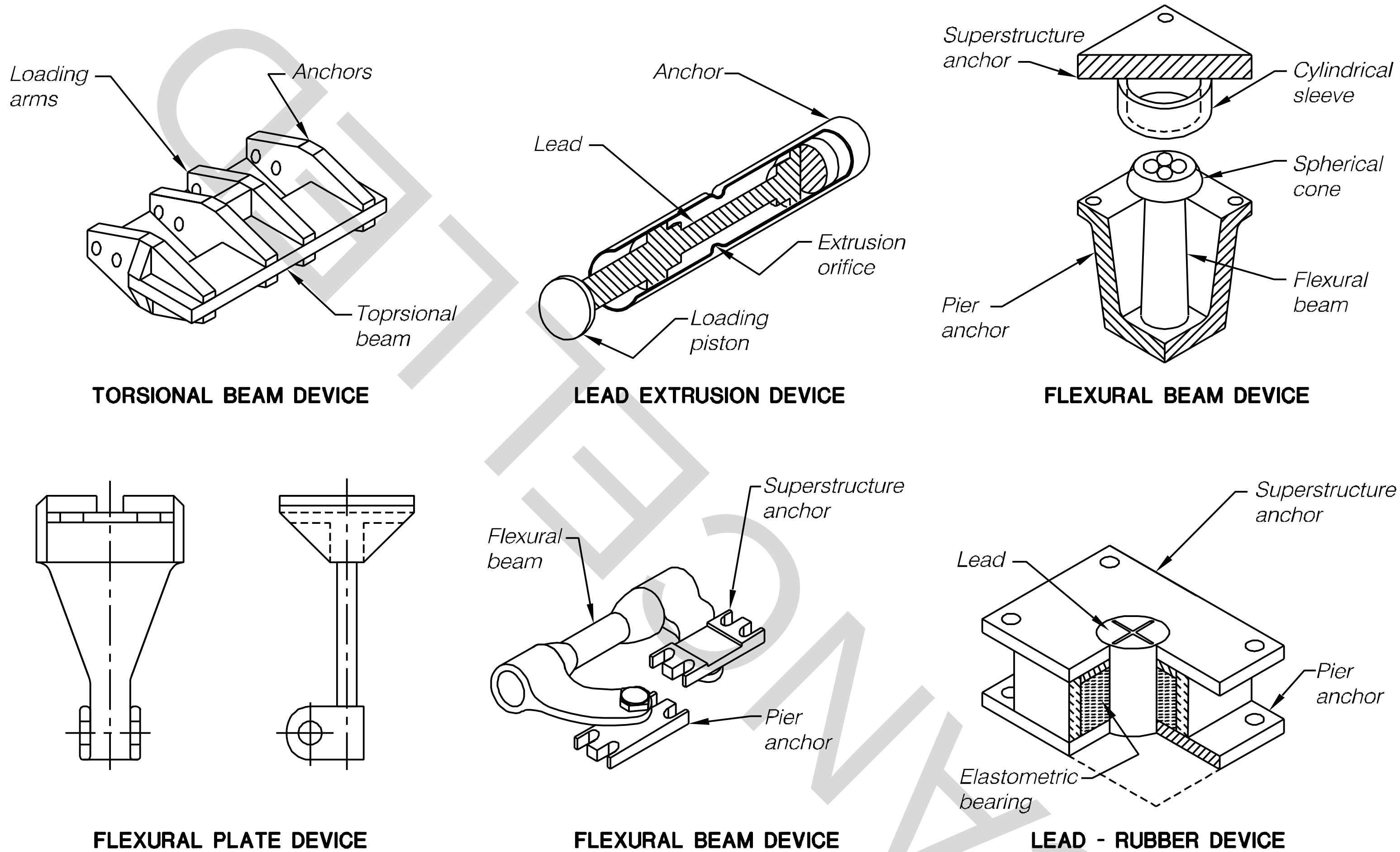
The purpose of this chapter is to provide a brief overview of many new technologies that are rapidly becoming more prevalent in the seismic design of building structures, and to provide guidance for the consideration and evaluation of the use of these systems in selected buildings. These technologies all involve the use of special details or specific devices to alter or control the dynamic behavior of buildings. The structural systems that utilize these technologies can be broadly categorized as passive, active, or hybrid control systems. Definitions of these terms are provided below, although the primary focus of this chapter is on passive control systems. Additional guidelines and design provisions for base isolation systems are provided in FEMA 302. Similar guidance for energy dissipation systems is provided in FEMA 273.

a. System Definitions.

(1) Passive control systems. These systems are designed to dissipate a large portion of the earthquake input energy in specialized devices or special connection details that deform and yield during an earthquake. Since the deformation and yielding are concentrated in the device, damage to other elements of the building may be reduced. These systems are passive in that they do not require any additional energy source to operate, and are activated by the earthquake input motion. Seismic isolation and passive energy dissipation are both examples of passive control systems. Some examples of these devices are presented in Figure 8-1. It is interesting to note that many of these devices can be used at the base of a structure as part of an isolation system, or in combination with braced frames or walls as energy dissipation devices.

(a) Seismic isolation systems. The objective of these systems is to decouple the building structure from the damaging components of the earthquake input motion, i.e., to prevent the superstructure of the building from absorbing the earthquake energy. The entire superstructure must be supported on discrete isolators whose dynamic characteristics are chosen to uncouple the ground motion. Some isolators are also designed to add substantial damping. Displacement and yielding are concentrated at the level of the isolation devices, and the superstructure behaves very much like a rigid body.

(b) Passive energy dissipation systems. The objective of these systems is to provide supplemental damping in order to significantly reduce structural response to earthquake motions. This may involve the addition of viscous damping through the use of viscoelastic dampers, hydraulic devices or lead extrusion systems; or the addition of hysteretic damping through the use of friction-slip devices, metallic yielding devices, or shape-memory alloy devices. Using these systems, a building will dissipate a large portion of the earthquake energy through inelastic deformations or friction concentrated in the energy dissipation devices,



(Note: Reprinted from "Seismic Isolation: History, Application, and Performance - A World View", Spectra, May 1990 with permission from The Earthquake Engineering Research Institute.)

Figure 8-1 Schematic drawings of representative isolation/energy dissipation devices

thereby protecting other structural elements from damage.

(2) Active control systems. These systems provide seismic protection by imposing forces on a structure that counter-balance the earthquake-induced forces. These systems are active in that they require an energy source and computer-controlled actuators to operate special braces or tuned-mass dampers located throughout the building. Active systems are more complex than passive systems, since they rely on computer control, motion sensors, feedback mechanisms, and moving parts that may require service or maintenance. In addition, these systems need an emergency power source to ensure that they will operate during a major earthquake and any immediate aftershocks.

(3) Hybrid control systems. These systems combine features of both passive and active control systems. In general, they have reduced power demands, improved reliability, and reduced cost when compared to fully active systems. In the future, these systems may include variable friction dampers, variable viscous dampers, and semi-active isolation bearings.

b. Mechanical Engineering Applications. It is important to note that the passive energy dissipation systems described here are "new" technologies when applied to civil engineering structures, but have been used in mechanical engineering for many years. There are numerous situations where dampers, springs, torsion bars, or elastomeric bearings have been used to control vibration or alter the dynamic behavior of mechanical systems. Several examples include vehicular shock absorbers, spring mounts that provide vertical vibration isolation for mechanical equipment, and hydraulic damping devices that utilize fluid flow through an orifice to provide shock isolation for military hardware. Many of these devices have been in use for decades and have performed well in situations where they are subjected to millions of cycles of loading; many more than would be required for seismic resistance. The immediate challenge is therefore not to develop new technologies, but to develop guidelines that will enable us to adapt existing technologies to civil/structural engineering applications.

c. Historical Overviews of Building Applications. Several types of isolation and supplemental damping systems have previously been used in building structures to solve problems related to vertical vibrations or wind loading. For example, a building in London is located on isolators in order to damp vibrations from the London Underground; the World Trade Center Towers in New York City were built with a system of viscoelastic dampers in order to alleviate human discomfort due to wind loading. The use of passive energy dissipation systems for seismic design is a relatively recent development, although there are now examples of these systems throughout the world (EERI 1990).

(1) Applications Outside the U.S. Beginning in the early 1970s, a number of bridge structures in New Zealand were constructed using seismic-isolation systems. The first building structure constructed using lead-rubber bearings was a government facility completed in Wellington, New Zealand in 1981. The most widespread use of both seismic isolation and energy dissipation systems is in Japan, where over a hundred structures have been

built using these technologies. Buildings in many countries, including Canada, Mexico, Italy, France, China, England, Russia, Iran, Chile, and South Africa, now use these systems. Facilities with isolation and supplemental damping systems include apartment houses, nuclear power plants, government office buildings, highrises, commercial structures, and monumental historic buildings.

(2) Applications within the U.S. In the United States, many projects are recently completed or under construction. The first new base-isolated building in the U.S. was completed in Rancho Cucamonga, California in 1985; the first seismic upgrade using steel yielding devices was completed in San Francisco, California in 1992. The most recent examples of seismic upgrading by base isolation includes the Oakland, California, City Hall completed in 1997, and the San Francisco City Hall, scheduled for completion in 1999. A number of essential facilities have been built using base-isolation systems, including the Fire Command and Control Facility and the Emergency Operations Center, both in East Los Angeles, California; the Titan Solid Rocket Motor Storage Facility at Vandenburg Air Force Base, California; and the V.A. Hospital in Long Beach, California.

8-2. Design Objectives.

a. General. Passive control systems can be used to achieve different design objectives or performance goals ranging from a life-safety standard to a higher standard that would provide damage control and post-earthquake functionality. The energy dissipation units used in passive control systems are generally simple devices that exhibit stable and predictable inelastic behavior when subjected to repeated cycles of seismic loading. Nevertheless, there is nothing inherent in these devices that guarantees better building performance. The addition of energy dissipation devices will only improve the seismic performance of a building if the devices have been carefully integrated into the seismic design of the structural system, taking into consideration the dynamic characteristics of the building, the dissipators, and the soil at the site.

b. Performance Objectives. Passive energy systems can be used to achieve building performance goals ranging from a life-safety standard to a higher standard that would provide damage control and post-earthquake functionality. The life-safety standard is currently reflected in the minimum design lateral-force requirements of conventional building codes. Damage control and post-earthquake functionality reflect higher performance goals that would provide additional protection from structural and nonstructural damage and loss of function. The discussion below compares how these various performance objectives can be met using either conventional design or passive control systems.

(1) Life Safety Standard. The philosophy embodied in building codes governing conventional fixed-base construction is that structures should resist minor earthquakes without damage; moderate earthquakes with nonstructural but without structural damage; and major earthquakes with structural damage but without collapse. This is often referred to as a life-safety standard, since the objective of these requirements is primarily to prevent loss of life due to catastrophic failures, not to prevent costly damage or loss of function.

(a) Structural Damage to Conventionally Designed Buildings. Based on observations from past earthquakes and laboratory tests, it is assumed that a properly detailed structure, designed to remain elastic for reduced seismic forces, will have sufficient strength and energy absorption capability to resist a major earthquake without collapse. The energy absorption capacity of conventional structural systems is a result of the yielding and degradation, i.e., damage to the structural and non-structural elements of the building. This includes degradation of beam-column joints, buckling of steel braces, cracking of shear panels and interior partition walls, etc. Following a major earthquake, buildings designed to meet the minimum life-safety standard are not expected to be functional, and may not be repairable.

(b) Passive Control Systems. To date, most projects where these technologies have been employed involve structures that were designed to a standard higher than life safety. In the future, these technologies may be useful in providing structures that meet the life safety objectives with lower life-cycle costs than for conventional design, or providing cost-effective seismic upgrades for older construction that does not comply with current life safety requirements.

(2) Damage control and post-earthquake functionality. In order to reduce or avoid damage to structures and building systems, a building's behavior must be investigated for a range of earthquake motions from smaller, more frequent events, to larger, infrequent events. Seismic demands on structural elements, stairs, ceiling systems, cladding, glazing, utilities, computer equipment, piping and mechanical systems, and other critical building components must be reviewed in order to assess the post-earthquake functionality of essential facilities.

(a) Conventional Design. In order to meet restrictive post-earthquake functionality requirements, most conventionally designed buildings must be designed to remain elastic for larger earthquake forces, with less reliance on ductility, increased damping, or significant inelastic behavior.

(b) Passive Control Systems. Seismic isolation and energy dissipation systems offer attractive alternatives to conventional design, since all these schemes can be used to reduce the earthquake input energy and concentrate the inelastic deformations in the isolators or damping devices, protecting critical elements of the structural frame from damage. Isolation and dissipation devices all have a yield threshold, and exhibit elastic behavior below this threshold and inelastic behavior after initial yielding. It is therefore especially important that response to both small and large earthquake motions be investigated, in order to capture the effective range of behavior of the particular device.

8-3. Seismic Isolation Systems.

a. Design Concept. The design of a seismic isolation system depends on many factors, including the period of the fixed-base structure, the period of the isolated structure, the dynamic characteristics of the soil at the site, the shape of the input response spectrum, and the force-deformation relationship for the particular isolation device. The primary objective of the design is to obtain a structure such that the isolated period of the building is sufficiently longer than both the fixed-base period of the building (i.e., the period of the superstructure), and the predominant period of the soil at the site. In this way, the superstructure can be decoupled from the maximum earthquake input energy. The spectral accelerations at the isolated period of the building are significantly reduced from those at the fixed-base period. The resultant forces on structural and nonstructural elements of the superstructure will be significantly reduced when compared with conventional fixed-base design. The benefits resulting from base isolation are attributed primarily to a reduction in spectral demand due to a longer period, as discussed in this Paragraph. Additional benefits may come from a further reduction in the spectral demand attained by supplemental damping provided by high-damped rubber components or lead cores in the isolation units. A preliminary evaluation of these benefits requires the following considerations:

(1) Select a target base shear, V_S, and an appropriate response modification factor, R_I, for the isolated building. Calculate $K_{D\max}$ D_D from Equation 8-8.

(2) From test data supplied by the isolation manufacturer, select units with effective stiffnesses $K_{D\min}$ and $K_{D\max}$ that approximately satisfy the calculated value of $K_{D\max}$ D_D.

(3) From the isolator damping characteristics provided by the manufacturers, assume an effective damping coefficient, $\$_D$, and obtain the appropriate value of B_D from Table 8-1.

(4) Calculate the design displacement, D_D, using Equation 8-1. Compare the calculated value with the assumed value, and if necessary, reiterate the process with revised values of $K_{D\max}$, T_D, and B_D until isolator properties provide the desired base shear, V_S, in the building.

(5) Calculate maximum displacement, D_M, using Equation 8-3 and total maximum displacement, D_{TM}, using Equation 8-6. The isolated building and all connecting utilities and appurtenances must be able to accommodate these displacements without interference.

b. Device Description. A number of seismic isolation devices are currently in use or proposed for use in the U.S. Although the specific properties vary, they are all designed to support vertical dead loads and to undergo large lateral deformations during a major earthquake. Some of these systems use elastomeric bearings; others use sliding systems that rely on frictional resistance.

Table 8-1

Damping Coefficient, B_D or B_M

Effective Damping, $\$_D$ or $\$_M$ (Percentage of Critical)[a,b]	B_D or B_M Factor
≤ 2%	0.8
5%	1.0
10%	1.2
20%	1.5
30%	1.7
40%	1.9
≥ 50%	2.0

a The damping coefficient shall be based on the effective damping of the isolation system determined in accordance with the requirements of Paragraph 8-3k.

b The damping coefficient shall be based on linear interpolation for effective damping values other than those given.

(1) Elastomeric Systems.

c. Applications. While base isolation is an ideal solution for some building structures, it may be entirely inappropriate for others. Since the objective of isolation design is to separate the response of the fixed-base structure from the predominant period of the underlying soil, it is most effective when these two periods coincide. In cases where they are already widely separated, base isolation may increase the response of the structure rather than reducing it. For instance, a very stiff structure on very soft soil would be a poor candidate, as would a very soft structure on very stiff soil. This is shown in Figures 8-2, 8-3, and 8-4 using three representative building types and three different soil types, represented by earthquake response spectra. The damping of the isolation devices may serve to further reduce the response of the building, but for the sake of simplicity, the effect of damping is not included in the following examples.

(1) Hard soil example. Three fixed-base structures are considered as potential candidates for isolation. The period of the isolated structure for all three cases is assumed to be 2.5 seconds. The three buildings, and fixed-base periods without isolators, are as follows:

- Concrete shear wall or steel braced frame building; T = 0.3 seconds;
- Concrete frame building; T = 0.7 seconds;
- Steel frame building; T = 1.2 seconds;

From Figure 8-2, it is evident that the seismic forces would be significantly reduced for the 0.3- and 0.7-second-period structures, and reduced by a smaller amount for the more flexible building with the 1.2-second period. It is important to remember that using conventional design principles, all three of these structures would soften during a major earthquake, and the forces would consequently be reduced, even without the addition of isolators. Nonetheless, these structures would be damaged, and if damage control and post-earthquake functionality are important issues, then isolation may still be useful even for the more flexible steel frame structure.

(2) Soft soil example. The same three fixed-base structures are considered as potential candidates for isolation. The period of the isolated structure for all three cases is assumed to be 2.5 seconds. From Figure 8-3, it may appear that none of the three buildings are good candidates for base isolation. The responses of the 0.7- and 1.2-second-period structures are reduced at a period of 2.5 seconds, but not dramatically. The response of the 0.3-second-period building would increase; nevertheless, the 0.3-second fixed-base structure would soften during a large earthquake, resulting in higher seismic forces and additional damage. Thus, if post-earthquake functionality is important, all of these structures might benefit from an appropriate isolation system.

(3) Very-soft-soil example. In this case, all three structures shown in Figure 8-4 would be subjected to higher seismic forces at the isolated period than at the fixed-base period, and no advantage would be gained from base isolation.

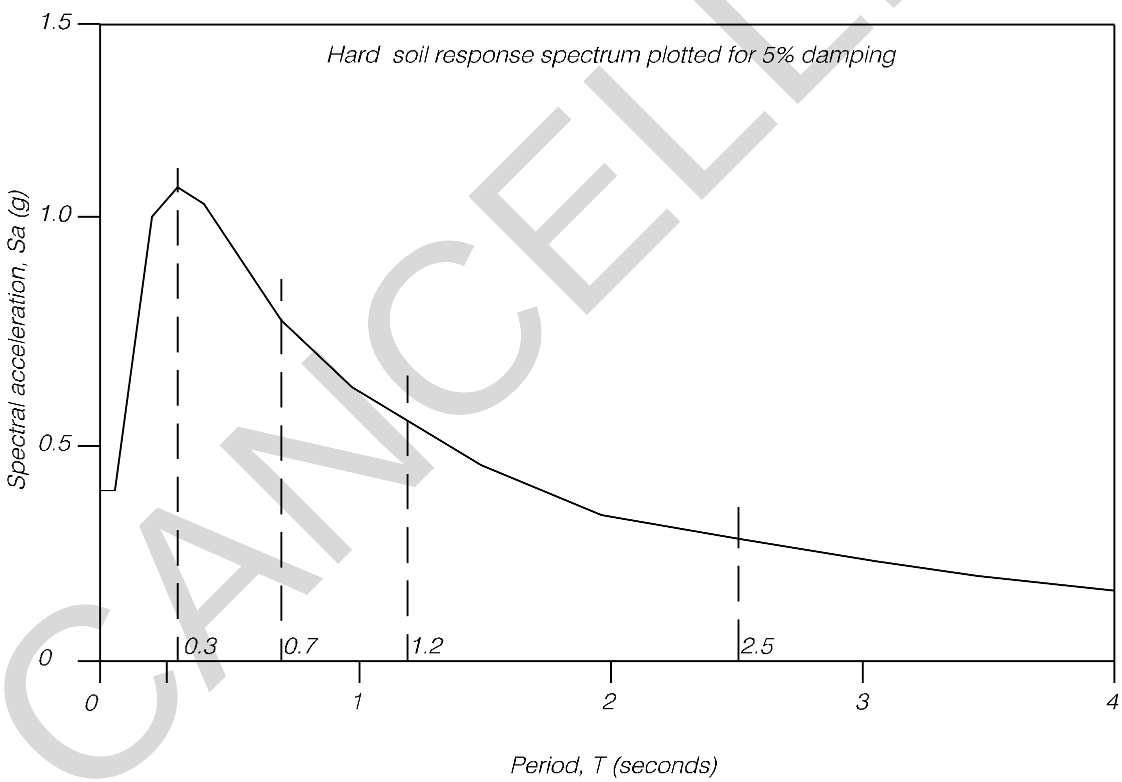


Figure 8-2 Seismic isolation hard soil example

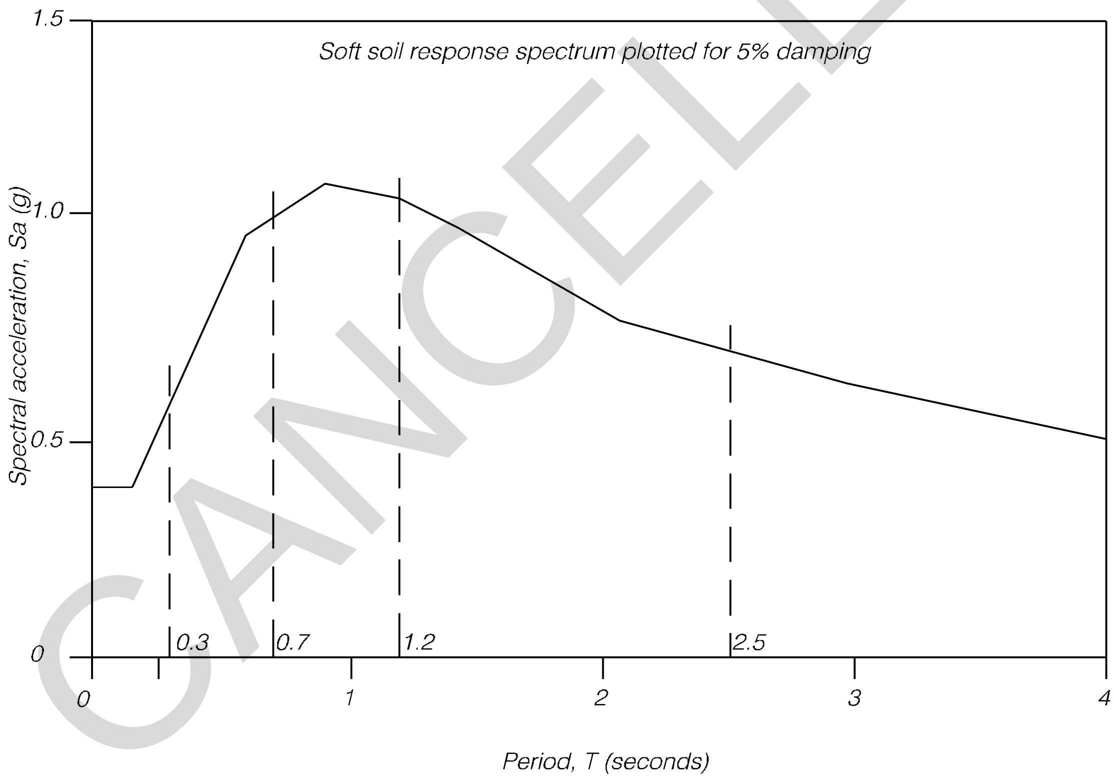


Figure 8-3 Seismic isolation soft soil example

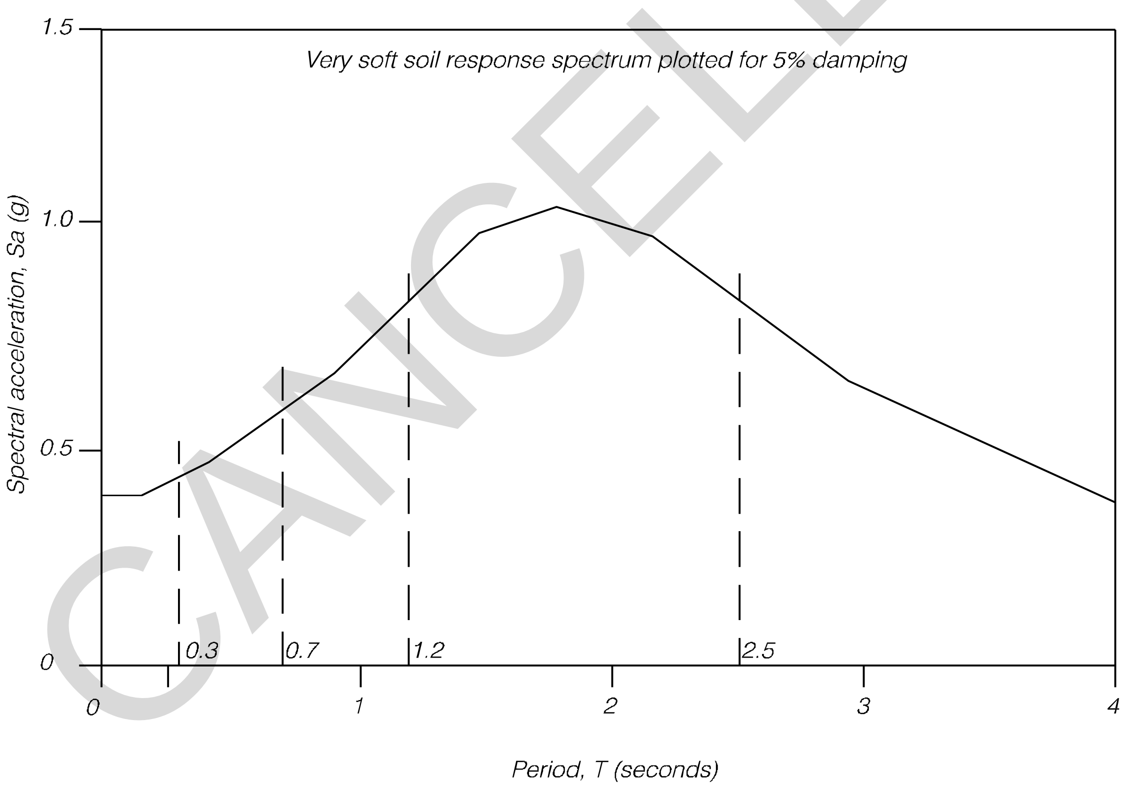


Figure 8-4 Seismic isolation very soft soil example

d. Design Criteria.

(1) Basis for design. The procedures and limitations for the design of seismically isolated structures shall be determined considering zoning, site characteristics, vertical acceleration, cracked section properties of concrete and masonry members, Seismic Use Group, configuration, structural system, and height in accordance with Section 5.2 of FEMA 302, except as noted below.

(2) Stability of the isolation system. The stability of the vertical-load-carrying elements of the isolation system shall be verified by analysis and test, as required, for lateral seismic displacement equal to the total maximum displacement.

(3) Selection of analytical procedure.

(a) General. Any seismically isolated structure is permitted to be designed using the dynamic lateral response procedure of Paragraph 8-3f, as are certain seismically designed structures defined below.

(b) Equivalent lateral-force procedures. The equivalent lateral-response procedure of Paragraph 8-3e is permitted to be used for design of a seismically isolated structure, provided that:

1. The structure is located at a site with S_1 less than or equal to 0.60g;

2. The structure is located on a Class A, B, C, or D site;

3. The structure above the isolation interface is not more than four stories or 65 feet (20 m) in height;

4. The effective period of the isolated structure, T_M, is less than or equal to 3.0 sec.;

5. The effective period of the isolated structure, $T_{D,}$ is greater than three times the elastic, fixed-base period of the structure above the isolation system, as determined by Equations 5.3.3.1-1 or 5.3.3.1-2 of FEMA 302;

6. The structure above the isolation system is of regular configuration; and

7. The isolation system meets all of the following criteria:

- The effective stiffness of the isolation system at the design displacement is greater than one-third of the effective stiffness at 20 percent of the design displacement;
- The isolation system is capable of producing a restoring force as specified in Paragraph 8-3i(2)(d);
- The isolation system has force-deflection properties that are independent of the rate of loading;
- The isolation system has force-deflection properties that are independent of vertical load and bilateral load; and

- The isolation system does not limit maximum capable earthquake displacement to less than S_{M1}/S_{D1} times the total design displacement.

(c) Dynamic analysis. A dynamic analysis is permitted to be used for the design of any structure, but shall be used for the design of all isolated structures not satisfying Paragraph 8-3d(3)(b). The dynamic lateral response procedure of Paragraph 8-3f shall be used for design of seismically isolated structures as specified below.

1. Response-spectrum analysis. Response-spectrum analysis is permitted to be used for design of a seismically isolated structure, provided that:

- The structure is located on a Class A, B, C, or D site; and

- The isolation system meets the criteria of Item 7 of Paragraph 8-3d(3)(b).

2. Time-history analysis. Time-history analysis is permitted to be used for design of any seismically isolated structure, and shall be used for design of all seismically isolated structures not meeting the criteria of Paragraph 1 above:

3. Site-specific design spectra. Site-specific ground-motion spectra of the design earthquake and the maximum considered earthquake developed in accordance with Paragraph 8-3f(4)(a) shall be used for design and analysis of all seismically isolated structures, if any one of the following conditions apply:

- The structure is located on a Class E or F site; or

- The structure is located at a site with S_1 greater than 0.60g.

e. Equivalent Lateral Force Procedure.

(1) General. Except as provided in Paragraph 8-3d, every seismically isolated structure or portion thereof may be designed and constructed to resist minimum earthquake displacements and forces, as specified by this Paragraph and the applicable requirements of FEMA 302.

(2) Minimum lateral displacements.

(a) Design displacement. The isolation system shall be designed and constructed to withstand minimum lateral earthquake displacements that act in the direction of each of the main horizontal axes of the structure in accordance with the following:

$$D_D = \left(\frac{g}{4\pi^2}\right)\frac{S_{D1}T_D}{B_D} \qquad \textbf{(}8\text{-}1)$$

where:

g = acceleration of gravity. The units of the acceleration of gravity, g, are in./sec$_2$ (mm/sec$_2$) if the units of the design displacement, D_D, are inches (mm).

S_{D1} = design 5 percent damped spectral acceleration in g units at 1 sec period for Ground

Motion A or Ground Motion B, as defined in Chapter 4.

T_D = effective period, in seconds (sec), of seismically isolated structure at the design displacement in the direction under consideration, as prescribed by Equation 8-2.

B_D = numerical coefficient related to the effective damping of the isolation system at the design displacement, $\$_D$, as set forth in Table 8-1.

(b) Effective period. The effective period of the isolated structure, T_D, shall be determined using the deformational characteristics of the isolation system in accordance with the following equation:

$$T_D = 2p\sqrt{\frac{W}{k_{D\min}g}} \qquad (8\text{-}2)$$

where:

W = total seismic dead load weight of the structure above the isolation interface as defined in Sections 5.3.2 and 5.5.3 of FEMA 302 (kip or kN).

$k_{D\min}$ = minimum effective stiffness, in kips/inch (kN/mm), of the isolation system at the design displacement in the horizontal direction under consideration.

g = acceleration of gravity. The units of the acceleration of gravity, g, are in./sec^2 (mm/sec^2) if the units of the design displacement, D_D, are inches (mm).

(c) Maximum displacement. The maximum displacement of the isolation system, D_M, in the most critical direction of horizontal response shall be calculated in accordance with the formula:

$$D_M = \frac{\left(\frac{g}{4p^2}\right)S_{M1}T_M}{B_M} \qquad (8\text{-}3)$$

where:

g = acceleration of gravity. The units of the acceleration of gravity, g, are in./sec^2 (mm/sec^2) if the units of the design displacement, D_D, are inches (mm).

S_{M1} = maximum considered 5 percent damped spectral acceleration at 1-second period as determined in Chapter 3.

T_M = effective period, in seconds, of seismic-isolated structure at the maximum displacement in the direction under consideration as prescribed by Equation 8-4.

B_M = numerical coefficient related to the effective damping of the isolation system at the maximum displacement, $\$_D$, as set forth in Table 8-1.

(d) Effective period at maximum displacement. The effective period of the isolated structure at maximum displacement, T_M, shall be determined using the deformational characteristics of the isolation system in accordance with the equation:

$$T_M = 2p\sqrt{\frac{W}{k_{M\min}g}} \qquad (8\text{-}4)$$

where:

W = total seismic dead load weight of the structure above the isolation interface as defined in Sections 5.3.2 and 5.5.3 of FEMA 302.

$k_{M\min}$ = minimum effective stiffness, in kips/inch (kN/mm), of the isolation system at the maximum displacement in the horizontal direction under consideration.

g = the acceleration due to gravity. The units of the acceleration of gravity, g, are in./sec^2 (mm/sec^2) of the units of the design displacement, D_D, are inches (mm).

(e) Total displacement. The total design displacement, D_{TD}, and the total maximum displacement, D_{TM}, of elements of the isolation system shall include additional displacement due to actual and accidental torsion calculated considering the spatial distribution of the lateral stiffness of the isolation system, and the most disadvantageous location of mass eccentricity.

1. The total design displacement, D_{TD}, and the total maximum displacement, D_{TM}, of elements of an isolation system with uniform spatial distribution of lateral stiffness shall not be taken as less than that prescribed by the following equations:

$$D_{TD} = D_D\left[1 + y\left(\frac{12e}{b^2 + d^2}\right)\right] \quad (8\text{-}5)$$

$$D_{TM} = D_M\left[1 + y\left(\frac{12e}{b^2 + d^2}\right)\right] \quad (8\text{-}6)$$

where:

D_D = design displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration as prescribed by Equation 8-1.

D_M = maximum displacement, in inches (mm) at the center of rigidity of the isolation system in the direction under consideration as prescribed in Equation 8-3.

y = the distance, in feet (mm), between the center of rigidity of the isolation system rigidity and the element of interest measured perpendicular to the direction of seismic loading under consideration.

e = the actual eccentricity, in feet (mm), measured in plan between the center of mass of the structure above the isolation interface and the center of rigidity of the isolation system, plus accidental eccentricity, in feet (mm), taken as 5 percent of the longest plan dimension of the structure perpendicular to the direction of force under consideration.

b = the shortest plan dimension of the structure, in feet (mm), measured perpendicular to d.

d = the longest plan dimension of the structure, in feet (mm).

(3) Minimum lateral force.

(a) Isolation system structural elements at or below the isolation system. The isolation system, the foundation, and all structural elements below the isolation system shall be designed and constructed to withstand a minimum lateral seismic force, V_s, using all of the appropriate provisions for a nonisolated structure, where:

$$V_s = k_{D\max} D_D \quad (8\text{-}7)$$

where:

$k_{D\max}$ = maximum effective stiffness, in kips/inch (kN/mm), of the isolation system at the design displacement in the horizontal direction under consideration.

D_D = design displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration as prescribed by Equation 8-1.

In all cases, V_b shall not be taken as less than the maximum force in the isolation system at any displacement, up to and including the design displacement.

(b) Structural elements above the isolation system. The structure above the isolation system shall be designed and constructed to withstand a minimum shear force, V_s, using all of the appropriate provisions for a nonisolated structure, where:

$$V_S = \frac{k_{D\max} D_D}{R_I} \quad (8\text{-}8)$$

where:

$k_{D\max}$ = maximum effective stiffness, in kips/inch (kN/mm), of the isolation system at the design displacement in the horizontal direction under consideration.

D_D = design displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration as prescribed by Equation 8-1.

R_I = numerical coefficient related to the type of lateral-force-resisting system above the isolation system.

The R_I factor shall be based on the type of lateral-force-resisting system used for the structure above the isolation system and shall be 3/8 of the R value given in Table 7-1, with an upper-bound value not to exceed 2.0, and a lower-bound value not to be less than 1.0.

(4) Vertical distribution of force. The total force shall be distributed over the height of the structure above the isolation interface in accordance with the following equation:

$$F_x = \frac{V_s w_x h_x}{\sum_{i=1}^{n} w_i h_i} \quad (8\text{-}9)$$

where:

V_s = total lateral seismic design force or shear on elements above the isolation system as prescribed by Equation 8-8.

w_x = portion of w that is located at or assigned to Level x.

h_x = height above the base Level x.

w_i = portion of w that is located at or assigned to Level I, respectively.

h_i = height above the base Level I.

At each level designated as x, the force, F_x, shall be applied over the area of the structure in accordance with the mass distribution at the level. Stresses in each structural element shall be calculated as the effect of force, F_x, applied at the appropriate levels above the base.

(5) Drift limits. The maximum interstory drift of the structure above the isolation system shall not exceed $0.015h_{sx}$. The drift shall be calculated by Equation 5.3.7-1 of FEMA 302, with the C_d factor of the isolated structure equal to the R_I factor defined in Paragraph 8-3e(3)(b).

f. Dynamic Lateral Response Procedure.

(1) General. Except as required by Paragraph 8-3d, every seismically isolated structure or portion thereof may be designed and constructed to resist earthquake displacements and forces as specified in this Paragraph and the applicable requirements of Section 5.4 of FEMA 302.

(2) Isolation system and structural elements below the isolation system.

(a) The total design displacement of the isolation system shall be taken as not less than 90 percent of D_{TD} as specified by Paragraph 8-3e(2)(e). The total maximum displacement of the isolation system shall be taken as not less than 80 percent of D_{TM}, as specified by Paragraph 8-3e(2)(e). The design lateral shear force on the isolation system and structural elements below the isolation system shall be taken as not less than 90 percent of V_b as prescribed by Equation 8-7. The limits of Paragraphs 8-3e(3)(a) and (b) shall be evaluated using values of D_{TD} and D_{TM} determined in accordance with Paragraphs 8-3e(2)(a) and (c), except that D'_D is permitted to be used on lieu of D_D and D'_M is permitted to be used in lieu of D_M where D'_D and D'_M are prescribed by the following equations:

$$D'_D = \frac{D_D}{\sqrt{1 + \left(\frac{T}{T_D}\right)^2}} \tag{8-10}$$

$$D'_M = \frac{D_M}{\sqrt{1 + \left(\frac{T}{T_M}\right)^2}} \tag{8-11}$$

where:

D_D = design displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration as prescribed by Equation 8-1.

D_M = maximum displacement in inches (mm), at the center of rigidity of the isolation system in the direction under consideration as prescribed by Equation 8-3.

T = elastic, fixed-base period of the structure above the isolation system as determined by Section 5.3.3 of FEMA 302.

T_D = effective period, in seconds, of the seismically isolated structure at the design displacement in the direction under consideration as prescribed by Equation 8-2.

T_M = effective period, in seconds, of the seismically isolated structure at the maximum displacement in the direction under consideration as prescribed by Equation 8-4.

(3) Structural elements above the isolation system. The design lateral shear force on the structure above the isolation system, if regular in configuration, shall be taken as not less than 80 percent of V_S, as prescribed by Equation 8-8 and the limits specified by Section 13.3.4.3 of FEMA 302.

Exception: The design lateral shear force on the structure above the isolation system, if regular in configuration, is permitted to be taken as less than 80 percent, but not less than 60 percent of V_S, provided time-history analysis is used for design of the structure.

The design lateral shear force on the structure above the isolation system, if irregular in configuration, shall be taken as not less than V_S, as prescribed by Equation 8-8 and the limits specified by section 13.3.4.3 of FEMA 302.

Exception: The design lateral shear force on the structure above the isolation system, if irregular in configuration, is permitted to be taken as less than 100 percent, but not less than 80 percent of V_S, provided time-history analysis is used for design of the structure.

(4) Ground motion.

(a) Design spectra. A design spectrum shall be constructed for the design earthquake. This design spectrum shall be taken as not less than the design earthquake response spectrum given in Figure 3-2. Properly substantiated site-specific spectra are required for the design of all structures located on a Class E or F site, or located at a site with S_1 greater than 0.60g. Structures that do not require site-specific spectra and for which site–specific spectra have not been calculated shall be designed using the response spectrum shape given in Figure 3-2.

Exception: If a site-specific spectrum is calculated for the design earthquake, the design spectrum is permitted to be taken as less than 100 percent, but not less than 80 percent, of the design earthquake response spectrum given in Figure 3-2.

A design spectrum shall be constructed for the maximum considered earthquake. This design spectrum shall be taken as not less than 1.5 times the design earthquake response spectrum given in Figure 3-2. This design spectrum shall be used to determine the total maximum displacement and overturning forces for design and testing of the isolation system.

Exception: If a site-specific spectrum is calculated for the maximum considered earthquake, the design spectrum is permitted to be taken as less than 100 percent, but not less than 80 percent of 1.5

times the design earthquake response spectrum given in Figure 3-2.

(b) Time histories. Pairs of appropriate horizontal ground-motion time-history components shall be selected and scaled from not less than three recorded events. Appropriate time histories shall be based on recorded events with magnitudes, fault distances, and source mechanisms that are consistent with those that control the design earthquake (or maximum considered earthquake). Where three appropriate recorded ground-motion time-history pairs are not available, appropriate simulated ground-motion time-history pairs are permitted to be used to make up the total number required. For each pair of horizontal ground-motion components, the square root sum of the squares of the 5 percent damped spectrum of the scaled, horizontal components shall be constructed. The motions shall be scaled such that the average value of the square-root-sum-of-the squares spectra does not fall below 1.3 times the 5 percent damped spectrum of the design earthquake (or maximum considered earthquake) by more than 10 percent for periods from $0.5T_D$ seconds to 1.25 T_M seconds.

(5) Analytical procedure.

(a) General. Response-spectrum and time-history analyses shall be performed in accordance with Section 5.4 of FEMA 302, and the requirements of the following Paragraphs.

(b) Input earthquake. The design earthquake shall be used to calculate the total design displacement of the isolation system and the lateral forces and displacements of the isolated structure. The maximum considered earthquake shall be used to calculate the total maximum displacement of the isolation system.

(c) Response-spectrum analysis. Response-spectrum analysis shall be performed using a modal damping value for the fundamental mode in the direction of interest not greater than the effective damping of the isolation system or 30 percent of critical, whichever is less. Modal damping values for higher modes shall be selected consistent with those appropriate for response spectrum analysis of the structure above the isolation system with a fixed base. Response-spectrum analysis used to determine the total design displacement and the total maximum displacement shall include simultaneous excitation of the model by 100 percent of the most critical direction of ground motion, and 30 percent of the ground motion on the orthogonal axis. The maximum displacement of the isolation system shall be calculated as the vectorial sum of the two orthogonal displacements. The design shear at any story shall not be less than the story shear obtained using Equation 8-9 and a value of V_S taken as that equal to the base shear obtained from the response-spectrum analysis in the direction of interest.

(d) Time-history analysis. Time-history analysis shall be performed with at least three appropriate pairs of horizontal time-history components as defined in Paragraph 8-3f(4)(b). Each pair of time histories shall be applied simultaneously to the model considering the most disadvantageous location of mass eccentricity. The maximum displacement of the isolation system shall be calculated from the vectorial sum of the two orthogonal components at each time step. The parameter of interest shall be calculated for each time-history analysis. If three time-history analyses are performed, the maximum response of the parameter of interest shall be used for design. If seven or more time-history analyses are performed, the average value of the response parameter of interest shall be used for design.

(6) Design lateral force.

(a) Isolation system and structural elements at or below the isolation system. The isolation system, foundation, and all structural elements below the isolation system shall be designed using all of the appropriate requirements for a non-isolated structure and the forces obtained from the dynamic analysis without reduction.

(b) Structural elements above the isolation system. Structural elements above the isolation system shall be designed using the appropriate provisions for a non-isolated structure and the forces obtained from the dynamic analysis divided by a factor of R_I. The R_I factor shall be based on the type of lateral-force-resisting system used for the structure above the isolation system.

(c) Scaling of results. When the factored lateral shear force on structural elements, determined using either response-spectrum or time-history analysis, is less than the minimum level prescribed by Paragraph 8-3f(2) and 8-3f(3), all response parameters, including member forces and moments, shall be adjusted proportionally upward.

(d) Drift limits. Maximum interstory drift corresponding to the design lateral force, including displacement due to vertical deformation of the isolation system, shall not exceed the following limits:

1. The maximum interstory drift of the structure above the isolation system calculated by response-spectrum analysis shall not exceed $0.015h_{sx}$, and

2. The maximum interstory drift of the structure above the isolation system calculated by time-history analysis considering the force-deflection characteristics of nonlinear elements of the lateral-force-resisting system shall not exceed $0.020h_{sx}$.

Drift shall be calculated using Equation 5.3.8.1 of FEMA 302 with the C_d factor of the isolated structure equal to the R_I factor defined in Paragraph 8-3e(3)(b). The secondary effects of the maximum considered earthquake lateral displacement) of the structure above the isolation system combined with gravity forces shall be investigated if the interstory drift ratio exceeds $0.010/R_I$.

g. Acceptance Criteria.

(1) Performance Objective 1A. Compliance with the provisions of Paragraphs 8-3e or 8-3f with Ground Motion as the design ground motion will be considered to satisfy this performance objective.

(2) Enhanced performance objectives. The design ground motion for enhanced performance objectives will be as indicated in Table 4-4. The analysis will be performed without the response modification factor, R_I, and the acceptance criteria

will be as prescribed in Chapter 6 with the appropriate *m* values from Chapter 7.

h. Lateral Load on Nonstructural Systems and Components Supported by Buildings.

(1) General. Parts or portions of an isolated structure, permanent nonstructural components and the attachments to them, and the attachments for permanent equipment supported by a structure shall be designed to resist seismic forces and displacements as prescribed by this section and the applicable requirements of Chapter 10. Buildings with isolation systems should use rigid horizontal diaphragms or bracing systems above and below the isolator level to provide deformation compatibility among the resisting structural elements. When the isolation system is located immediately above the building foundations, a reinforced concrete slab or a system of tie beams should be provided for displacement compatibility among the footings or pile caps.

(2) Forces and displacements.

(a) Components at or above the isolation interface. Elements of seismically isolated structures and nonstructural components, or portions thereof , that are at or above the isolation interface shall be designed to resist a total lateral seismic force equal to the maximum dynamic response of the element or component under consideration.

Exception: Elements of seismically isolated structures and nonstructural components or portions thereof are permitted to be designed to resist total lateral seismic force as prescribed by Equation 5.2.6-1 or 5.2.6-2 of FEMA 302, as appropriate.

(b) Components crossing the isolation interface. Elements of seismically isolated structures and nonstructural components, or portions thereof, that cross the isolation interface, shall be designed to withstand the total maximum displacement.

(c) Components below the isolation interface. Elements of seismically isolated structures and nonstructural components, or portions thereof, that are below the isolation interface shall be designed and constructed in accordance with the requirements of Section 5.2 of FEMA 302.

i. Detailed System Requirements. The isolation system and the structural system shall comply with the material requirements of FEMA 302. In addition, the isolation system shall comply with the detailed system requirements of this chapter, and the structural system shall comply with the requirements of this document and the applicable portions of Section 5.2 of FEMA 302.

j. Design and Construction Review.

(1) General. A design review of the isolation system and related test programs shall be performed by an independent peer review team of registered design professionals in the appropriate disciplines, and others experienced in seismic analysis methods and the theory and application of seismic isolation.

(2) Isolation system. Isolation system design review shall include, but not be limited to, the following:

(a) Review of site-specific seismic criteria, including the development of site-specific spectra and ground motion time histories and all other design criteria developed specifically for the project;

(b) Review of the preliminary design, including the determination of the total design displacement of the isolation system design displacement and the lateral force design level;

(c) Overview and observation of prototype testing, Paragraph 8-3k;

(d) Review of the final design of the entire structural system and all supporting analyses; and

(e) Review of the isolation system quality control testing program, Paragraph 8-3i(2)(i).

k. Required Tests of the Isolation System. Required testing to establish and validate the design perspectives of the isolation system shall be in accordance with the requirements of Section 13.9 of FEMA 302.

8-4. Energy Dissipation Systems.

a. Design Concept. These systems are designed to provide supplemental damping in order to reduce the seismic input forces. Most conventional buildings are designed assuming 5 percent equivalent viscous damping for structures responding in the elastic range. For structures that include viscous dampers or metallic yielding devices, the equivalent viscous damping may be increased to between 15 percent and 25 percent, depending on the specific characteristics of the device. In this way, seismic input energy to the structure is largely dissipated through the inelastic deformations concentrated in the devices, reducing damage to other critical elements of the building. The benefits resulting from the use of displacement-dependent energy dissipation devices are attributed primarily to the reduction in spectral demand due to supplemental damping provided by the devices. A preliminary evaluation of these benefits requires the following considerations:

(1) From a linear elastic static or modal analysis of the building, determine the story displacements without the energy dissipation devices.

(2) Select target design displacement, D_{Di}, at each story. From test data furnished by the manufacturer, determine the effective stiffness, K_{eff}, of the proposed devices at each story using Equation 8-13.

(3) Based on the effective stiffness of the devices and the assumed target displacements, calculate the effective damping, $\$$, in accordance with Equations 8-18 and 8-19.

(4) Modify the design response spectrum to represent the effective damping using Table 8-2 and Figure 8-8.

(5) Modify the mathematical model of the building to incorporate the effective stiffness of the devices in each story.

Effective Damping ß (percent of critical)[1]	B_S	B_1
<2	0.8	0.8
5	1.0	1.0
10	1.3	1.2
20	1.8	1.5
30	2.3	1.7
40	2.7	1.9
>50	3.0	2.0

[1] The damping coefficient should be based on linear interpolation for effective damping values other than those given.

Table 8-2 Damping Coefficients B_s and B_1 as a Function of Effective Damping b

(6) Perform the analysis of the revised model with the modified spectrum and compare the story displacements with the assumed target displacements. If necessary, revise the target displacements and reiterate the analysis.

(7) Optimize the design by using several assumed values of the effective stiffness of the devices and the target displacements.

Evaluation of the benefits of velocity-displacement energy-dissipation devices is much more complex and beyond the scope of this document. Guidance for such an evaluation can be obtained from the design examples in FEMA 274 (Commentary to FEMA 273).

b. *Device Description*. A number of energy-dissipation devices are currently in use or proposed for use in the U.S. The specific properties vary widely. Some of these systems use viscous fluids or viscoelastic materials; some rely on the hysteretic behavior of metallic elements; and others use sliding systems that rely on frictional resistance. The systems that use viscous and viscoelastic materials are rate-dependent (i.e., the hysteretic response of the device depends upon the rate of loading), and also may be temperature sensitive. The other systems are generally rate-independent.

c. *Applications*. Supplemental damping may significantly reduce the seismic input where the structural period is in resonance with the predominant period of the site. If the structural period and site period are widely separated, added damping may have only a marginal effect on the response. It should be noted that the reduction of the response is most dramatic when the frequency of the structural system (including the effects of the yielding device) coincides with the frequency at the peak of the input acceleration spectrum. This is shown in Figures 8-5 and 8-6 using four representative building types and two different soil types, represented by earthquake response spectra. These examples are constructed to demonstrate the effect of the supplemental damping. For the sake of simplicity, the effect of the added stiffness has been included with the building period cited below.

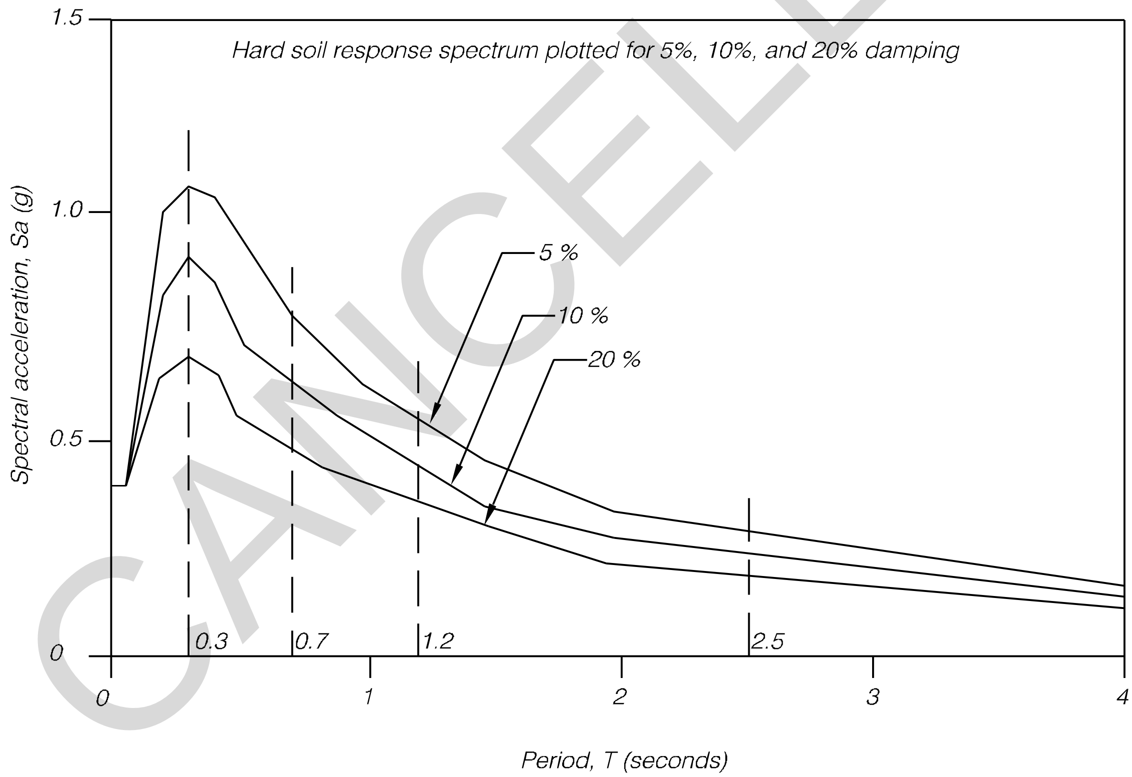


Figure 8-5 Supplemental damping hard soil example

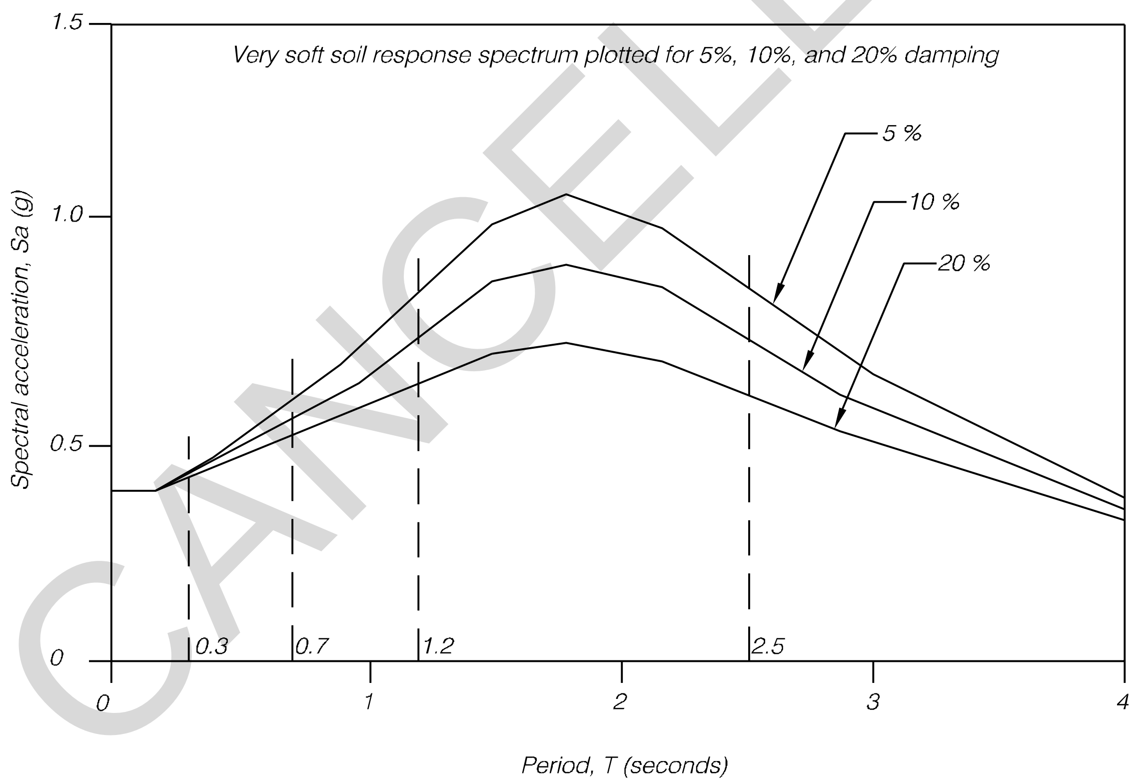


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Figure 8-6 Supplemental damping very soft soil example

- Concrete shear wall or steel braced frame building; T = 0.3 seconds;

- Concrete frame building; T = 0.7 seconds;

- Steel frame building; T = 1.2 seconds;

- Tall steel frame; T = 2.5 seconds.

d. Design Criteria.

(1) General.

(a) The energy-dissipation devices should be designed with consideration given to other environmental conditions, including wind, aging effects, creep, fatigue, ambient temperature, operating temperature, and exposure to moisture or damaging substances.

(b) The building height limitations should not exceed the limitations for the structural system into which the energy-dissipation devices are implemented.

(c) The mathematical model of a building should include the plan and vertical distribution of the energy-dissipation devices. Analysis of the mathematical model should account for the dependence of the devices on excitation frequency, ambient and operating temperature, velocity, sustained loads, and bilateral loads. Multiple analyses of the building may be necessary to capture the effects of varying mechanical characteristics of the devices.

(d) Energy-dissipation devices shall be capable of sustaining larger displacements (and velocities for velocity-dependant devices) than the maximum calculated in the MCE. The increase in displacement (and velocity) capacity is dependent on the level of redundancy in the supplemental damping system as follows:

1. If four or more energy dissipation devices are provided in a given story of a building, in one principal direction of the building, with a minimum of two devices located on each side of the center of stiffness of the story in the direction under consideration, all energy dissipation devices shall be capable of sustaining displacements equal to 130 percent of the maximum calculated displacement in the device in the MCE. A velocity-dependant device shall also be capable of sustaining the force associated with a velocity equal to 130 percent of the maximum calculated velocity for that device in the MCE.

2. If fewer than four energy dissipation devices are provided in a given story of a building, in one principal direction of the building, or fewer than two devices are located on each side of the center of stiffness of the story in the direction under consideration, all energy-dissipation devices shall be capable of sustaining displacements equal to 200 percent of the maximum calculated displacement in the device in the MCE. A velocity-dependant device shall also be capable of sustaining the force associated with a velocity equal to 200 percent of the maximum calculated velocity for that device in the MCE.

(e) The components and connections transferring forces between the energy dissipation devices shall be designed to remain linearly elastic for the forces described in items (d)1 or (d)2 above, depending upon the degree of redundancy in the supplemental damping system.

(2) Modeling of energy-dissipation devices.

(a) Energy-dissipation devices are classified as either displacement-dependent, velocity-dependent, or other. Displacement-dependent devices may exhibit either rigid-plastic (friction devices), bilinear (metallic yielding devices), or trilinear hysteresis. The response of displacement-dependent devices should be independent of velocity and/or frequency of excitation. Velocity-dependent devices include solid and fluid viscoelastic devices, and fluid viscous devices. The third classification (other) includes all devices that cannot be classified as either displacement- or velocity-dependent. Examples of "other" devices include shape memory alloys (superelastic effect), friction-spring assemblies with recentering capability, and fluid-restoring force-damping devices.

(b) Models of the energy dissipation system should include the stiffness of structural components that are part of the load path between the energy-dissipation devices and the ground, if the flexibility of these components is significant enough to affect the performance of the energy dissipation system. Structural components whose flexibility could affect the performance of the energy dissipation system include components of the foundation, braces that work in series with the energy dissipation devices, and connections between braces and the energy dissipation devices.

(c) Energy dissipation devices should be modeled as described in the following subsection, unless more advanced methods or phenomenological models are used.

(3) Displacement-dependent devices.

(a) The force-displacement response of a displacement-dependent device is primarily a function of the relative displacement between each end of the device. The response of such a device is substantially independent of the relative velocity between each end of the device, and/or frequency of excitation.

(b) Displacement-dependent devices should be modeled in sufficient detail so as to capture their force-displacement response adequately, and their dependence, if any, on axial-shear-flexure interaction, or bilateral deformation response.

(c) For the purposes of evaluating the response of a displacement-dependent device from testing data, the force in a displacement-dependent device may be expressed as:

$$F = k_{\mathrm{eff}} D \qquad (8\text{-}12)$$

where the effective stiffness k_{eff} of the device is calculated as:

$$k_{\mathrm{eff}} = \frac{|F^{+}| + |F^{-}|}{|D^{+}| + |D^{-}|} \qquad (8\text{-}13)$$

and where forces in the device, F^+ and F^-, are evaluated at displacements D^+ and D^-, respectively.

(4) Velocity-dependent devices.

The force-displacement response of a velocity-dependent device is primarily a function of the relative velocity between each end of the device.

(a) Solid viscoelastic devices. The cyclic response of viseoelastic solids is generally dependent on the frequency and amplitude of the motion, and the operation temperature (including temperature rise due to excitation).

1. Solid viseoelastic devices may be modeled using a spring and dashpot in parallel (Kelvin model). The spring and dashpot constants selected should adequately capture the frequency and temperature dependence on the device consistent with fundamental frequency of the building *(f_1)*, and the operating temperature range. If the cyclic response of a viseoelastic solid device cannot be adequately captured by single estimates of the spring and dashpot constants, the response of the building should be estimated by multiple analyses of the building frame, using limited values for the spring and dashpot constants.

2. The force in a viseoelastic device may be expressed as:

$$F = k_{\text{eff}} D + C \dot{D} \qquad (8\text{-}14)$$

where C is the damping coefficient for the viscoelastic device, D is the relative displacement between each end of the device, $\dot{D}$ is the relative velocity between each end of the device, and k_{eff} is the effective stiffness of the device calculated as:

$$k_{\text{eff}} = \frac{|F^+| + |F^-|}{|D^+| + |D^-|} = K^1 \qquad (8\text{-}15)$$

where K^1 is the so-called storage stiffness.

3. The damping coefficient for the device shall be calculated as:

$$C = \frac{W_D}{p w_1 D_{\text{ave}}^2} = \frac{K^{11}}{w_1} \qquad (8\text{-}16)$$

where K^{11} is the loss stiffness, the angular frequency ω_1 is equal to $2Bf_1$, D_{ave} is the average of the absolute values of displacements D^+ and D^-, and W_D is the area enclosed by one complete cycle of the force-displacement response of the device.

(b) Fluid viscoelastic devices. The cyclic response of viscoelastic fluid devices is generally dependent on the frequency and amplitude of the motion, and the operation temperature (including temperature rise due to excitation). Fluid viscoelastic devices may be modeled using a spring and dashpot in series (Maxwell model). The spring and dashpot constants selected should adequately capture the frequency and temperature dependence of the device consistent with fundamental frequency of the rehabilitated building *(f_1)*, and the operation temperature range. If the cyclic response of a viscoelastic fluid device cannot be adequately captured by single estimate of the spring and dashpot constants, the response of the building should be estimated by multiple analyses of the building frame, using limiting values for the spring and dashpot constants.

(c) Fluid viscous devices.

1. The cyclic response of a fluid viscous device is dependent on the velocity of motion; may be dependent on the frequency and amplitude of the motion; and is generally dependent on the operation temperature (including temperature rise due to excitation). Fluid viscous devices may exhibit some stiffness at high frequencies of cyclic loading. Linear fluid viscous dampers exhibiting stiffness in the frequency range 0.5 f_1 to 2.0 f_1 should be modeled as a fluid viscoelastic device.

2. In the absence of stiffness in the frequency range 0.5 f_1 to 2.0 f_1, the force in the fluid viscous device may be expressed as:

$$F = C_{\mathrm{O}} \left| \dot{\mathrm{D}} \right|^{a} sgn\left(\dot{\mathrm{D}} \right) \qquad (8\text{-}17)$$

where C_0 is the damping coefficient for the device, ″ is the velocity exponent for the device, $\dot{\mathrm{D}}$ is the relative velocity between each end of the device, and *sgn* is the signum function that, in this case, defines the sign of the relative velocity term.

(d) Other types of devices. Energy dissipation devices not classified as either displacement-dependent or velocity-dependent should be modeled using either established principles of mechanics or phenomenological models. Such models should accurately describe the force-velocity-displacement response of the device under all sources of loading (e.g., gravity, seismic, thermal).

e. Linear Analytical Procedures.

(1) General.

(a) Linear procedures are only permitted if it can be demonstrated that the framing system exclusive of the energy dissipation devices remains essentially linearly elastic for the level of earthquake demand of interest after the effects of added damping are considered. Further, the effective damping afforded by the energy dissipation shall not exceed 30 percent of critical in the fundamental mode. Other limits on the use of linear procedures are presented below.

(b) The secant stiffness, K_s, of each energy dissipation device, calculated at the maximum displacement in the device, in a manner similar to that indicated in Figure 8-7 for the target displacement of the building, shall be included in the mathematical model of the rehabilitated building. For the purpose of evaluating the regularity of a building, the energy dissipation devices shall be included in the mathematical mode.

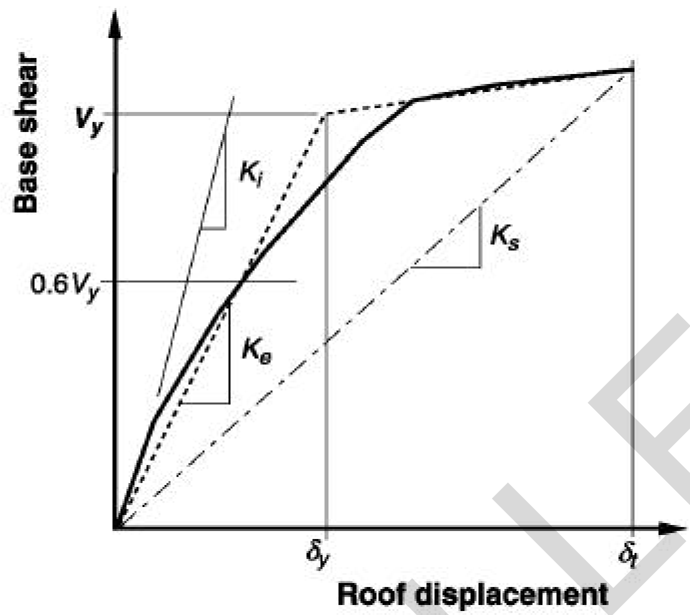


Figure 8-7 Calculation of Secant Stiffness, K_S

(2) Linear Static Procedures.

(a) Displacement-dependent device. The Linear Static Procedure (LSP) may be used to implement displacement-dependent energy dissipation devices, provided that the following requirements are satisfied:

1. The ratio of the maximum resistance in each story, in the direction under consideration, to the story shear demand calculated using Equations 5.3.4-1 and 5.3.4-2 in FEMA 302, shall range between 80 percent and 120 percent of the average value of the ratio for all stories. The maximum story resistance shall include the contributions from all components, elements, and energy-dissipation devices.

2. The maximum resistance of all energy-dissipation devices in a story, in the direction under consideration, shall not exceed 50 percent of the resistance of the remainder of the framing, where said resistance is calculated at the displacements anticipated in the MCE. Aging and environmental effects shall be considered in calculating the maximum resistance of the energy dissipation devices.

3. The base shear and story forces calculated by Equations 5.3.4-1 and 5.3.4-2 in FEMA should be reduced by the damping modification factors in Table 8-2 to account for the energy dissipation (damping) affected by the energy dissipation devices. Figure 8-8 indicates how the response spectrum is modified by the damping coefficient B_s and B_1 in Table 8-2. In Figure 8-8, the spectral ordinates S_{xs} and S_{x1} represent the 0.2 second and the 1.0 second ordinates for Ground Motion A or B, or for the MCE. The calculation of the effective damping is estimated as follows:

$$b_{\text{eff}} = b + \frac{\sum_{\text{i}} W_j}{4pW_k} \tag{8-18}$$

where $\$$ is the damping in the framing system, and is set equal to 0.05, unless modified. W_j is work done by device j in one complete cycle corresponding to floor displacements $*_i$, the summation extends over all devices j, and W_k is the maximum strain energy in the frame, determined using Equation 8-19.

$$W_k = \frac{1}{2}\sum_i F_i d_i \tag{8-19}$$

where F_{I} is the inertia force at floor level I, and the summation extends over all floor levels.

(b) Velocity-dependent devices.

1. The LSP may be used to implement velocity-dependent energy-dissipation devices, provided that the following requirement is satisfied:

- The maximum resistance of all energy-dissipation devices in a story, in the direction under consideration, shall not exceed 50 percent of the resistance of the remainder of the framing, where said resistance is calculated at the displacements anticipated in the MCE. Aging and environmental effects shall be considered in calculating the

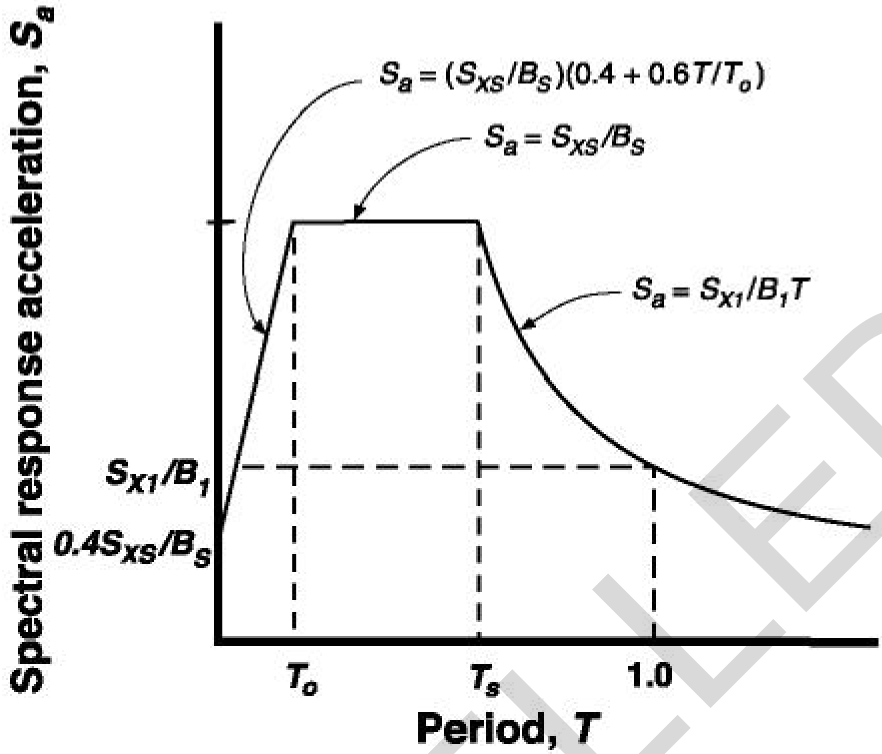


Figure 8-8 General Response Spectrum

maximum resistance of the energy-dissipation devices.

2. The base shear and story forces should be reduced, as described above, by the damping modification factors in Table 8-2 to account for the energy dissipation (damping) afforded by the energy-dissipation devices. The calculation for effective damping is estimated as:

$$\beta_{\text{eff}} = \beta + \frac{\sum_i W_j}{4\pi W_k} \qquad \text{(8-20)}$$

where β is the damping in the structural frame, and is set equal to 0.05 unless modified in Section 2.6.1.5, W_j is work done by device j in one complete cycle corresponding to floor displacements δ_i, the summation extends over all devices j, and W_k is the maximum strain energy in the frame, determined using Equation 8-19.

3. The work done by linear viscous device j in one complete cycle of loading may be calculated as:

$$W_j = \frac{2\pi^2}{T} C_j \delta_{rj}^2 \qquad \text{(8-21)}$$

where T is the fundamental period of the building, including the stiffness of the velocity-dependent devices, C_j is the damping constant for device j, and δ_{rj} is the relative displacement between the ends of device j along the axis of device j. An alternative equation for calculating the effective damping of Equation 8-20 is:

$$\beta_{eff} = \beta + \frac{T \sum_i C_j \cos^2\theta_j \phi_{rj}^2}{\pi \sum_i \left(\frac{w_i}{g}\right) \phi_i^2} \qquad \text{(8-22)}$$

where θ_j is the angle of inclination of device j to the horizontal, ϕ_{rj} is the first mode relative displacement between the ends of device j in the horizontal direction, w_i is the reactive weight of floor level i, ϕ_i is the first mode displacement at floor level i, and other terms are as defined above. Equation 8-22 applies to linear viscous devices only.

4. The design actions for components of the building should be calculated in three distinct stages of deformation, as follows. The maximum action should be used for design.

i. At the stage of maximum drift. The lateral forces at each level of the building should be calculated using Equations 5.3.4-1 and 5.3.4-2 in FEMA 302, where V is the modified equivalent base shear.

ii. At the stage of maximum velocity and zero drift. The viscous component of force in each energy dissipation device should be calculated by Equations 8-14 or 8-17, where the relative velocity $\dot{D}$ is given by $2\pi f_1 D$, where D is the relative displacement between the ends of the device calculated at the stage of maximum drift. The calculated viscous forces should be applied to the mathematical model of the building at the points of attachment of the device, and in directions consistent with the deformed shape of the building at maximum drift. The horizontal inertial forces at each floor level of the building should be applied concurrently with the viscous forces so that the horizontal displacement of each floor level is zero.

iii. At the stage of maximum floor acceleration**.** Design actions in components of the rehabilitated building should be determined as the sum of [actions determined at the stage of maximum drift] times [CF_1] and [actions determined at the stage of maximum velocity] times [CF_2], where

$$CF_1 = \cos\left[\tan^{-1}\left(2\beta_{\text{eff}}\right)\right] \qquad \text{(8-23)}$$

$$CF_2 = \sin\left[\tan^{-1}\left(2\beta_{\text{eff}}\right)\right] \qquad \text{(8-24)}$$

in which β_{eff} is defined by either Equation 8-20 or Equation 8-22.

(3) Linear Dynamic Procedure (LDP). The modal analyses procedure, described in Paragraph 3-2(c)(2), may be used when the effective damping in the fundamental mode of the building, in each principal direction, does not exceed 30 percent of critical.

(a) Displacement-dependent devices. Application of the LDP for the analysis of buildings incorporating displacement-dependent devices is subject to the restrictions set forth in Paragraph 8-4-e(2)(a).

1. For analysis by the Response Spectrum Method, the 5 percent damped response spectrum may be modified to account for the damping afforded the displacement-dependent energy-dissipation devices. The 5 percent damped acceleration spectrum should be reduced by the modal-dependent damping modification factor, *B*, and either B_s or B_l, for periods in the vicinity of the mode under consideration; note that the value of B will be different for each mode of vibration. The damping modification factor in each significant mode should be determined using Table 8-2 and the calculated effective damping in that mode. The effective damping should be determined using a procedure similar to the described in Paragraph 8-4e(2)(a).

2. If the maximum base shear force calculated by dynamic analysis is less than 80 percent of the modified equivalent base shear of Paragraph 8-4e(2)(a), component and element actions and deformations shall be proportionally increased to correspond to 80 percent of the modified equivalent base shear.

(b) Velocity-dependent devices.

1. For analysis by the Response Spectrum Method, the 5 percent damped response spectrum may be modified to account for the damping afforded by the velocity-dependent energy dissipation devices. The 5 percent damped acceleration spectrum should be reduced by the modal-dependent damping modification factor, *B*, either B_s or B_l, for periods in the vicinity of the mode under consideration; note that the value of *B* will be different for each mode of vibration. The damping modification factor in each significant mode should be determined using Table 8-2 and the calculated effective damping mode.

2. The effective damping in the *m*-th mode of vibration ($\beta_{eff\text{-}m}$) shall be calculated as:

$$\beta_{eff-m} = \beta_m + \frac{\sum_i W_{mj}}{4\pi W_{mk}} \qquad (8\text{-}25)$$

where β_m is the *m*-th mode damping in the building frame, W_{mj} is work done by device *j* in one complete cycle corresponding to modal floor displacements δ_{mi}, and W_{mk} is the maximum strain energy in the frame in the *m*-th mode, determined using Equation 8-26.

$$W_{mk} = \frac{1}{2}\sum_i F_{mi} d_{mi} \qquad (8\text{-}26)$$

where F_{mi} is the *m*-th mode horizontal inertia force at floor level *i* and δ_{mi} is the *m*-th mode horizontal displacement at floor level *i*. The work done by linear viscous device *j* in one complete cycle of loading in the *m*-th mode may be calculated as:

$$W_{mj} = \frac{2\pi^2}{T_m} C_j d_{mrj}^2 \qquad (8\text{-}27)$$

where T_m is the *m*-th mode period of the rehabilitated building, including the stiffness of the velocity-dependent devices, C_j is the damping constant for device *j*, and δ_{mrj} is the *m*-th mode relative displacement between the ends of device *j* along the axis of device *j*.

3. Direct application of the Response Spectrum Method will result in member actions at maximum drift. Member actions at maximum velocity and maximum acceleration in each significant mode should be determined using the procedure described in Paragraph 8-4e(2)(b). The combination factors CF_1 and CF_2 should be determined from Equations 8-23 and 8-24 using β_{eff-m} for the *m*-th mode.

4. If the maximum base shear force calculated by dynamic analysis is less than 80 percent of the modified equivalent base shear of Paragraph 8-4e(3), component and element actions and deformations shall be proportionally increased to correspond to 80 percent of the modified equivalent base shear.

f. Nonlinear Elastic Static Procedure.

The nonlinear static procedure, described in Paragraph 5-4, should be followed unless explicitly modified by the following paragraphs.

(1) The nonlinear mathematical model of the building should explicitly include the nonlinear force-velocity-displacement characteristics of the energy-dissipation devices, and the mechanical characteristics of the components supporting the devices. Stiffness characteristics should be consistent with the deformations corresponding to the target displacement and frequency equal to the inverse of period T_e, as defined in Paragraph 5-4(e)(4).

(2) The nonlinear mathematical model of the building shall include the nonlinear force-velocity-displacement characteristics of the energy-dissipation devices, and the mechanical characteristic components supporting the devices. Energy-dissipation devices with stiffness and damping characteristics that are dependent on excitation frequency and/or temperature shall be modeled with characteristics consistent with (1) the deformations

expected at the target displacement, and (2) a frequency equal to the inverse of the effective period.

(3) Equation 5-5 should be used to calculate the target displacement. For velocity-dependent energy-dissipation devices, the spectral acceleration in Equation 5-5 should be reduced to account for the damping afforded by the viscous dampers.

(a) Displacement-dependent devices. Equations 5-5 should be used to calculate the target displacement. The stiffness characteristics of the energy dissipation devices should be included in the mathematical model.

(b) Velocity-dependent devices. The target displacement of Equation 5-5 should be reduced to account for the damping added by the velocity-dependent energy-dissipation devices. The calculation of the damping effect is estimated as:

$$b_{\text{eff}} = b + \frac{\sum_i W_j}{4pW_k} \qquad \text{(8-28)}$$

where $\$$ is the damping in the structural frame and is set equal to 0.05, W_j, is work done by device in j in one complete cycle corresponding to floor displacements $*_i$, the summation extends over all devices j, and W_k is the maximum strain energy in the frame, determined using Equation 8-19. The work done by device j in one complete cycle of loading may be calculated as:

$$W_j = \frac{2p^2}{T_s} C_j d_j^2 \qquad \text{(8-29)}$$

where T_s is the secant fundamental period of the building, including the stiffness of the velocity-dependent devices (if any), calculated using Equation 5-3, but replacing the effective stiffness (K_e) with the secant stiffness (K_s) at the target displacement (see Figure 8-7); C_j is the damping constant for device j; and $*_{rj}$ is the relative displacement between the ends of device j along the axis of device j at a roof displacement corresponding to the target displacement.

g. Acceptance Criteria. The acceptance criteria for all performance objectives, prescribed in Chapter 6, and provided for building systems and components in Chapter 5, apply to buildings incorporating energy dissipation devices. The benefits of energy dissipation are realized by the reduced demand response spectrum using the damping coefficients in Table 8-2. Checking for force-controlled actions should use the component actions calculated for three limit states: maximum drift, maximum velocity, and maximum acceleration. In the nonlinear elastic static procedure, displacement-controlled actions must be checked for deformations corresponding to the target displacement. Maximum actions are to be used for design, temperature, and exposure to moisture and damaging substances.

h. Design and Construction Reviews. Design and construction review will be performed on all buildings incorporating energy-dissipation devices. The type and scope of the review will be in accordance with the requirements of Paragraph 1-9, unless modified by the requirements of this chapter. Design review of the energy-dissipation system and related test programs should be performed by an independent engineering peer review panel, including persons licensed in the appropriate disciplines, and experience in seismic analysis, including the theory and application of energy-dissipation methods. The design review should include, but should not necessarily be limited to, the following:

- Preliminary design including sizing of the devices;

- Prototype testing;

- Final design of the rehabilitated building and supporting analyses; and

- Manufacturing quality control program for the energy-dissipation devices.

i. Required Tests of Energy Dissipation Devices. Required testing, and peer review of the testing, to establish and validate the design properties of the energy-dissipation devices, shall be similar to that required by Section 13.9 and the appendix to Chapter 13 of FEMA 302.

8-5. Guidance for Selection and Use of Seismic Isolation and Energy Dissipation Systems.

a. Earthquake Damage Mitigation. Earthquake damage to nearly any structure could be reduced through the judicious use of some type of seismic isolation or energy-dissipation system. Although the initial design and construction costs for these systems may be higher than for conventional design, current data suggest that they will pay for themselves over the life of a structure in reduced earthquake damage. These systems might be appropriate for critical facilities where severe damage is unacceptable, and also for noncritical facilities where a long-term user is willing to accept the higher initial coats in exchange for reduced future damage costs.

(1) Conventional design using elastic design. Using conventional design, earthquake damage can generally be prevented only by designing for higher and higher seismic forces. Critical facilities built using conventional design may need to be designed to remain elastic even for major earthquakes. The resulting design forces must be resisted elastically by all of the critical structural and nonstructural building components. Such design procedures result in larger structural members and more costly construction than life-safety design procedures, and are rarely used except for facilities such as nuclear power plants.

(2) Seismic isolation and energy dissipation. Facilities that incorporate seismic isolation and energy dissipation systems can be designed to take advantage of the dynamic characteristics and the nonlinearities inherent in these systems to reduce the seismic accelerations and displacements. Thus, critical structural and nonstructural components may generally be designed using substantially lower element forces than would be required using elastic design procedures to achieve the same level of earthquake protection.

b. Type of Facility. Important, essential, and historic facilities may be good candidates for seismic isolation or energy-dissipation systems, since earthquake damage to such facilities may have costly and unacceptable consequences. Examples of such consequences might include a major hazardous materials release from a facility located in an urban area, major equipment malfunction at a regional emergency response center, or the destruction of an irreplaceable historic structure. Such events are unacceptable, particularly when techniques are available to prevent them. Seismic isolation or energy dissipation systems can be incorporated into the design of critical facilities to prevent these types of disasters from occurring.

c. Earthquake Effects - Acceleration vs. Displacement. Building components may be damaged by both seismic accelerations and seismic displacements. A particular type of component, either structural or nonstructural, may be more sensitive to one or other type of damage. In order to reduce earthquake damage, it is important to consider whether critical building components are vulnerable to acceleration damage, displacement damage, or both.

(1) Damage caused by seismic accelerations. Seismic accelerations cause intense shaking that may damage structural components, nonstructural components, and piping or sensitive equipment. A building component may be damaged when the seismic inertial forces generated within the component exceed the elastic capacity of the component to resist those forces. Some examples of damage due to excessive inertial forces caused by seismic accelerations include the following: shear cracking in a masonry shear wall; out-of-plane failure of a freestanding wall or heavy partition; shear failure of anchor bolts at the base of a piece of heavy equipment; and pipe rupture at an anchor point for a long, unbraced section of heavy pipe.

(2) Damage caused by seismic displacements. Seismic displacements may also damage building components. Nonstructural components attached to adjacent floors in multistory buildings are particularly vulnerable to displacement damage. Light items that are unlikely to generate large inertial forces may still be damaged by large imposed deformations. Nonstructural components such as glazing, precast cladding, rigid full-height partitions, sprinkler piping, hazardous material piping, and exterior veneer or ornamentation may be damaged by large interstory drifts caused by the seismic displacements of the building frame. Items that cross seismic joints between adjacent buildings are also vulnerable to displacement damage.

(3) Damage identification. It is important to identity what critical building components are vulnerable to damage, what type of damage they are vulnerable to, and what level of damage protection is desired for critical components of a given facility in order to identity effective damage reduction techniques. In some cases, acceleration control may be required in order to reduce potential acceleration damage. In other cases, displacement control may be most important. In still other cases, both acceleration and displacement control may he required to provide effective damage reduction.

d. System Selection - conventional design, seismic isolation, or energy dissipation. The selection of a structural system for a critical facility is a complex process that must take many factors into consideration. These factors include the dynamic characteristics of the building, the surrounding soil, and the critical nonstructural components. Both present construction costs and future damage costs should be considered. Proximity to an active fault may be another important consideration. Seismic isolation and energy-dissipation systems can both be effectively used to reduce earthquake damage when compared with conventional construction, but each type of system is most effective for a different range of dynamic characteristics. In addition, the selection of one or other system may depend on whether acceleration control, displacement control, or both, are required to reduce the earthquake damage at a particular facility.

(1) System comparison. Table 8-3 provides a comparison of building behavior for these three systems – conventional design, seismic isolation, and energy dissipation. Generally, seismic isolation systems are most effective in reducing damage to buildings that are already very flexible. Base isolation is most effective when the original building period is significantly shorter than the isolated building period, typically about 2.5 seconds. Energy dissipation systems are almost the reverse. They are most effective in reducing damage to flexible structures, and much less effective in reducing damage to rigid structures.

(2) Site selection - inappropriate sites. Particular care must be used in selecting a structural system for a building site located very close to an active fault or in an unmapped area that may be underlain by blind thrust faults. Recent seismic recording from near-fault sites include measurements of very large spectral displacements at some stations, and very large, one-cycle, energy pulses at other stations. Typical seismic isolation and energy dissipation systems are currently not designed to accommodate these extreme near-fault motions. In addition, seismic isolation systems are currently not designed for use at locations where the site period is in the range of 2 to 3 seconds, since this is also the range of most current isolators.

(a) Sites where seismic isolation systems are not recommended. During recent earthquakes, near-fault spectral displacements of approximately 40 inches have been measured for periods in the range of 2 to 3 seconds. Current isolators typically have periods of approximately 2.5 seconds. These isolators have not been designed to accommodate such large spectral displacements, and may fail and develop vertical instabilities. Deep soil sites with 2- to 3- second periods also would not be appropriate for seismic isolation. At such sites, the isolators could be in resonance with the ground motion, resulting in the undesired amplification of the structural response. In the future, isolation systems may he developed for these sites, but current seismic isolation techniques and hardware are not recommended for either the near-fault site, or the deep soil site with a 2- to 3- second period.

(b) Sites where energy dissipation systems are not recommended. During recent earthquakes, including both Northridge, California and Kobe, Japan, very large energy pulses have been recorded within the first few earthquake cycles at some near-fault sites. Very close to a fault, the majority of the total input energy at the site may be contained in an initial large pulse. Currently available energy dissipators are generally designed to dissipate a portion of the energy input during each of several cycles in order to obtain the maximum benefit. Current dissipators are not designed to dissipate the total input energy from a major earthquake in one or two cycles. In the future, special devices may be developed for this type of motion, but current energy-dissipation systems are not recommended for use at near-fault sites.

Table 8-3

Comparison of Building Behavior

Building Type	Conventional Design		Building with Seismic Isolation System		Building with Energy Dissipation System		Recommended System to Reduce Earthquake Damage
	Seismic Displ.	Seismic Accel.	Seismic Displ.	Seismic Accel.	Seismic Displ.	Seismic Accel.	
Rigid Buildings (shear wall, masonry construction	small	large	small	small	small	large	Seismic Isolation
Semi-rigid or semi-flexible Buildings (braced frames, stiff moment frames, tall shear walls)	moderate	moderate	small	small	small	small	Seismic Isolation, or Energy Dissipation
Very Flexible Buildings (steel or concrete frames)	large	normally large (note 1)	normally moderate (note 1)	normally small (note 1)	normally moderate (note 1, note 2)	normally small (note 1)	Energy Dissipation (note 2)

Note 1. Flexible buildings cover a broad range of building types and wide period range. Selection of an appropriate system to reduce earthquake damage depends on the dynamic characteristics of the building in question. Energy dissipation schemes are more likely to be effective for very flexible structures, but either system might be used for semi-rigid and semi-flexible structures.

Note 2. Special detailing may be required to protect elements vulnerable to displacement damage as a result of the moderate interstory displacements.

CHAPTER 9
FOUNDATIONS

9-1. Introduction.

Chapter 7 in FEMA 302 provides conventional foundation design provisions that are adequate for most military buildings. This chapter provides guidance in the implementation of those provisions, and also provides guidance in the use of load deformation characteristics, for soil/structure interaction, in the form of simplified soil springs. The determination of appropriate soil springs and the structural systems for which they provide a better representation of seismic response are discussed in Paragraph 9-2b.

9-2. Site Characterization.

Site characterization consists of the compilation of information on the site subsurface soil conditions, and the configuration and loading of the proposed foundations. The evaluation of the ground-shaking hazard and site geologic hazards is discussed in Chapter 3.

a. Site Foundation Conditions. Subsurface soil conditions must be defined in sufficient detail to assess the ultimate capacity of the foundation, and to determine if the site is susceptible to seismic-geologic hazards.

(1) Structural foundation type. Information regarding the structural foundation type, dimensions, and material are required irrespective of the subsurface soil conditions. This information includes:

- Foundation type: spread footings, mat foundation, piles, drilled shafts.

- Foundation dimensions: plan dimensions and locations. For piles, tip elevations, vertical variations (tapered sections of piles or belled caissons).

- Material composition/construction. For piles, type (concrete/steel/wood), and installation method (cast-in-place, open/closed-end driving).

(2) Subsurface soil conditions. The capacity of the foundation soil in bearing or the capacity of the soil interface between pile, pier, or caisson and the soil will be determined by a geotechnical investigation and shall be sufficient to support the structure with all prescribed loads, without seismic forces, taking due account of the settlement that the structure can withstand. For the load combination including earthquake, the soil capacities must be sufficient to resist loads at acceptable strains considering both the short duration of loading and the dynamic properties of the soil. If load-deformation characterization for the foundations are to be considered (i.e., Paragraph 9-2b), the soil unit weight, g soil shear strength, c, soil friction angle, f, soil compressibility characteristics, soil shear modulus, G, and Poisson's ratio, n, need to be determined for each soil type. Additionally, the shear wave velocity, $<_s$, the standard penetration resistance, N, or the undrained shear strength, S_u, need to be determined to define the site classification

in accordance with Table 3-1 in order to assign the appropriate site coefficients, F_a and F_v.

b. Load Deformation Characteristics for Foundations.

(1) General. Load-deformation characteristics are required where the effects of foundations are to be taken into account in linear elastic analyses or in nonlinear static (pushover) or nonlinear dynamic (time history) analyses. Foundation load-deformation parameters characterized by both stiffness and capacity can have a significant effect on both structural response and load distribution among structural elements. Load-deformation parameters, represented by appropriate soil springs, can provide significant reduction and/or redistribution of seismic force levels in some buildings. Vertical soil springs may effectively lengthen the fundamental period of slender, stiff buildings such as aircraft control towers, and could have a beneficial effect for buildings at relatively stiff soil sites. Rotational soil springs at interior column footings will tend to relieve the fixed end moment at the column base, and cause redistribution of seismic forces in the story. Foundation systems for buildings can in some cases be complex, but for the purpose of simplicity, three foundation types are considered in these guidelines:

- Shallow bearing foundations;
- Pile foundations; and
- Drilled shafts.

While it is recognized that the load-deformation behavior of foundations is nonlinear, because of the difficulties in determining soil properties and static foundation loads for existing buildings, together with the likely variability of soils supporting foundations, an equivalent elasto-plastic representation of load-deformation behavior is recommended. In addition, to allow for such variability or uncertainty, an upper- and lower-bound approach to defining stiffness and capacity is recommended (as shown in Figure 9-1a) to permit evaluation of structural response sensitivity. The selection of uncertainty represented by the upper and lower bounds should be determined jointly by the geotechnical and structural engineers.

(2) Shallow bearing foundations.

(a) Stiffness parameters. The shear modulus, G, for a soil is related to the modulus of elasticity, E, and Poisson's ratio, v, by the relationship:

$$G = \frac{E}{2(1+v)} \qquad (9\text{-}1)$$

1. Most soils are intrinsically nonlinear and the shear modulus and the shear wave velocity decrease with increasing shear strain. Experimental values obtained by laboratory testing at low strains need to be modified to reflect expected effective values at strains corresponding to the design ground motion.

2. To reflect the upper- and lower-bound concept illustrated in Figure 9-1 the upper-bound stiffness of rectangular footings should be based on

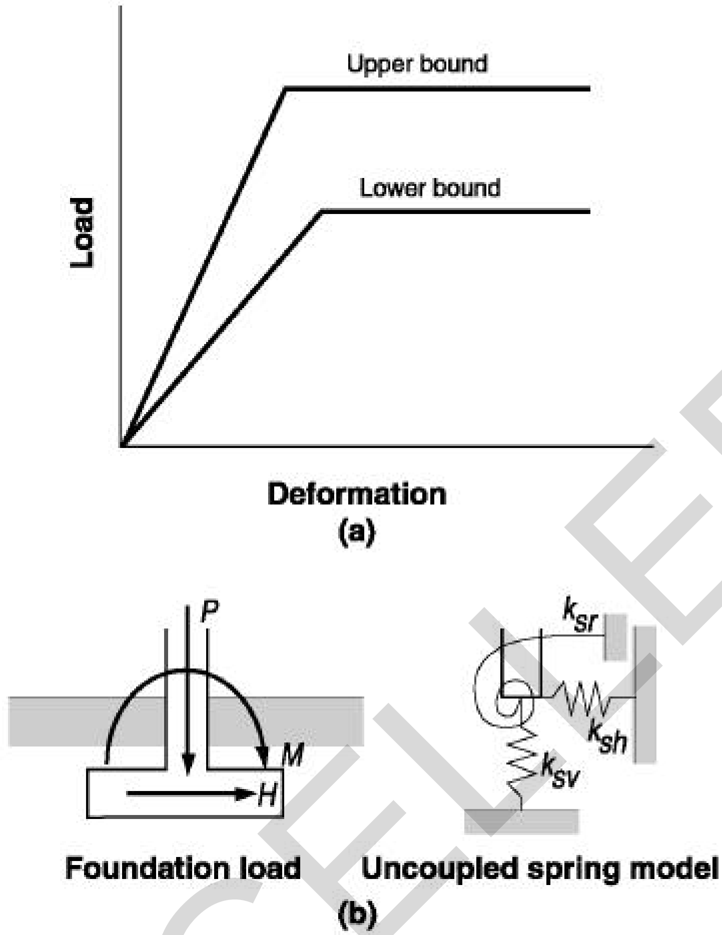


Figure 9-1 Idealized Elasto-Plastic Load-Deformation Behavior for Soils

twice the effective shear modulus, G, determined in the geotechnical investigation. The lower-bound stiffness should be based on one-half the effective shear modulus; thus, the range of stiffness should incorporate a factor of four from lower- to upper-bound. Most shallow bearing footings are stiff relative to the soil upon which they rest. For simplified analyses, an uncoupled spring model, as shown in Figure 9-1b, may be sufficient. The three equivalent spring constants may be determined using conventional theoretical solutions for rigid plates resting on a semi-infinite elastic medium. Although frequency-dependent solutions are available, results are reasonably insensitive to loading frequencies within the range of parameters of interest for buildings subjected to earthquakes. It is sufficient to use static stiffnesses as representative of repeated loading conditions. Figure 9-2 presents stiffness solutions for rectangular plates in terms of an equivalent circular radius. Stiffnesses are adjusted for shape and depth using factors similar to those in Figure 9-3. For the case of horizontal translation, the solution represents mobilization of base traction (friction) only. If the sides of the footing are in close contact with adjacent in situ foundation soil or well-compacted fill, significant additional stiffness may be assumed from passive pressure. A solution for passive pressure stiffness is presented in Figure 9-4. For more complex analyses, a finite element representation of linear or nonlinear foundation behavior may be accomplished using Winkler component models. Distributed vertical stiffness properties may be calculated by dividing the total vertical stiffness by the area. Similarly, the uniformly distributed rotational stiffness can be calculated by dividing the total rotational stiffness of the footing by the moment of inertia of the footing in the direction of loading. In general, however, the uniformly distributed vertical and rotational stiffnesses are not equal. The two may be effectively decoupled for a Winkler model using a procedure similar to that illustrated in Figure 9-5. The ends of the rectangular footing are represented by End Zones of relatively high stiffness, with overall length of approximately one-sixth of the footing width. The stiffness per unit length in these End Zones is based on the vertical stiffness of a B x $B/6$ isolated footing. The stiffness per unit length in the Middle Zone is equivalent to that of an infinitely long strip of footing. In some instances, the stiffness of the structural components of the footing may be relatively flexible compared to the soil material. A slender grade beam resting on stiff soil is an example. Classical solutions for beams on elastic supports can provide guidance regarding when such effects are important. For example, a grade beam supporting point loads spaced at a distance of L might be considered flexible if:

$$\frac{EI}{L^4} < 10 k_{sv} B \qquad (9\text{-}8)$$

where, for the grade beam,

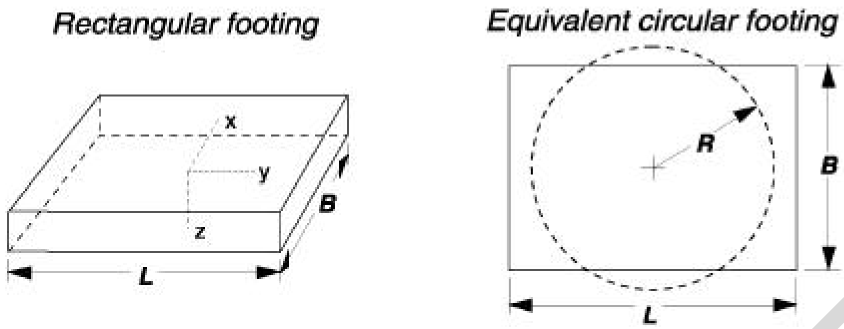
E = effective modulus of elasticity

I = moment of inertia

B = width.

For most flexible foundation systems, the unit subgrade spring coefficient, k_{sv}, may be taken as

Radii of circular footings equivalent to rectangular footings



	Degree of freedom			
	Translation	Rocking		Torsion
		About x-axis	About y-axis	About z-axis
Equivalent radius, R	$\left(\frac{BL}{\pi}\right)^{1/2}$	$\left(\frac{BL^3}{3\pi}\right)^{1/4}$	$\left(\frac{B^3L}{3\pi}\right)^{1/4}$	$\left[\frac{BL(B^2+L^2)}{6\pi}\right]^{1/4}$

Spring constants for embedded rectangular footings

Spring constants for shallow rectangular footings are obtained by modifying the solution for a circular footing, bonded to the surface of an elastic half-space, i.e., $k = \alpha\beta k_O$

where

k_O = Stiffness coefficient for the equivalent circular footing
α = Foundation shape correction factor (Figure 4-3a)
β = Embedment factor (Figure 4-3b)

To use the equation, the radius of an equivalent circular footing is first calculated according to the degree of freedom being considered. The figure above summarizes the appropriate radii. k_O is calculated using the table below:

Displacement degree of freedom	k_O
Vertical translation	$\frac{4GR}{1-\nu}$
Horizontal translation	$\frac{8GR}{2-\nu}$
Torsional rotation	$\frac{16GR^3}{3}$
Rocking rotation	$\frac{8GR^3}{3(1-\nu)}$

Note:
G and ν are the shear modulus and Poisson's ratio for the elastic half-space. G is related to Young's modulus, E, as follows:
$E = 2(1+\nu)G$
R = Equivalent radius

Figure 9-2 Elastic Solutions for Rigid Footing Spring Constants (based on Gazetas, 1991 and Lam et al., 1991)

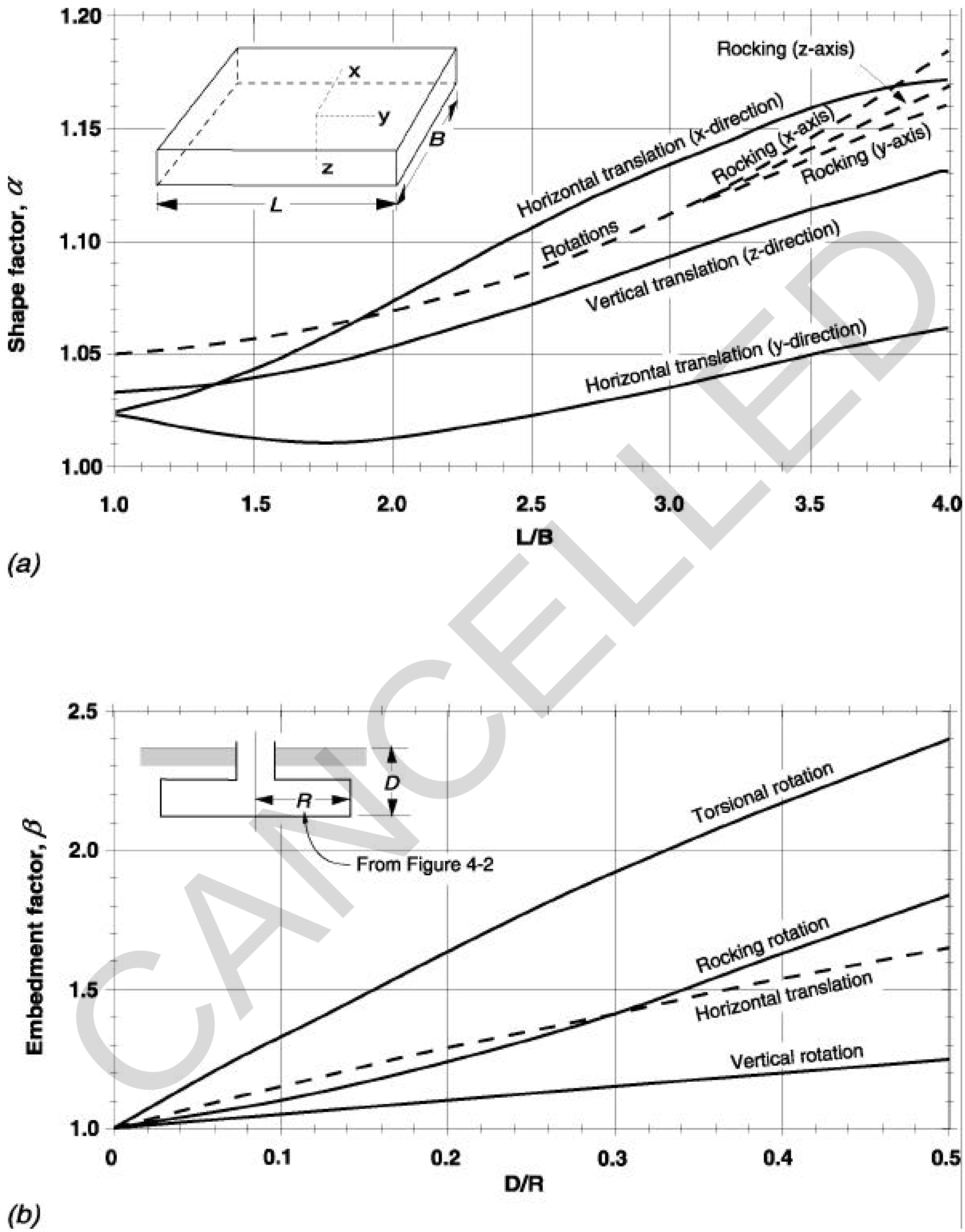


Figure 9-3 (a) Foundation Shape Correction Factors (b) Embedment Correction Factors

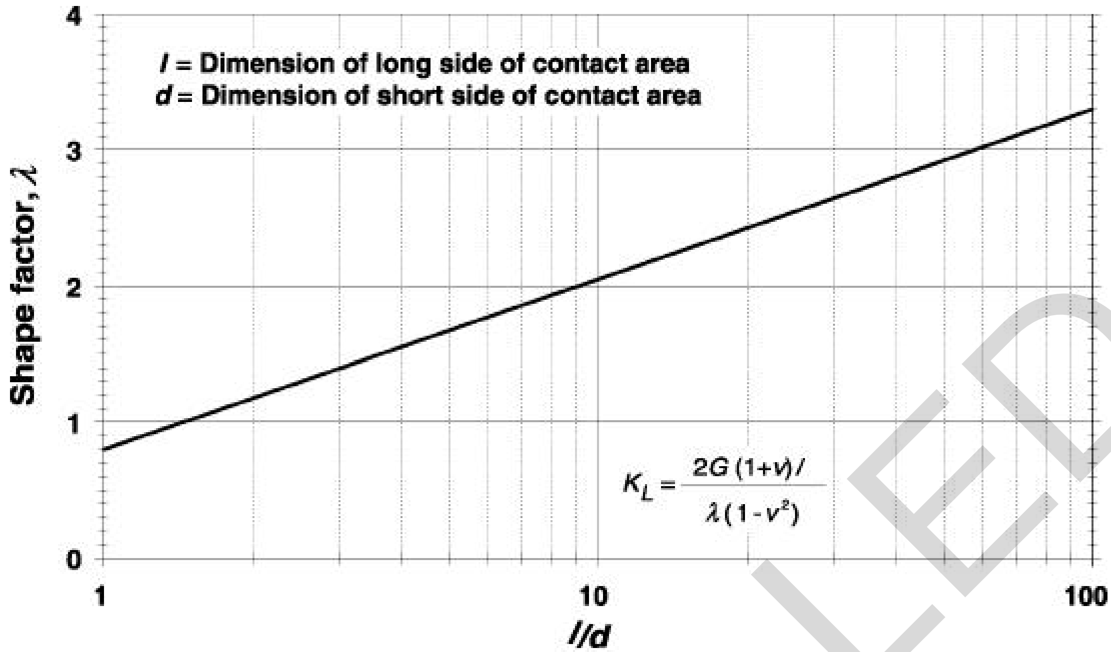


Figure 9-4   Lateral Foundation-Soil Stiffness for Passive Pressure (after Wilson, 1988)

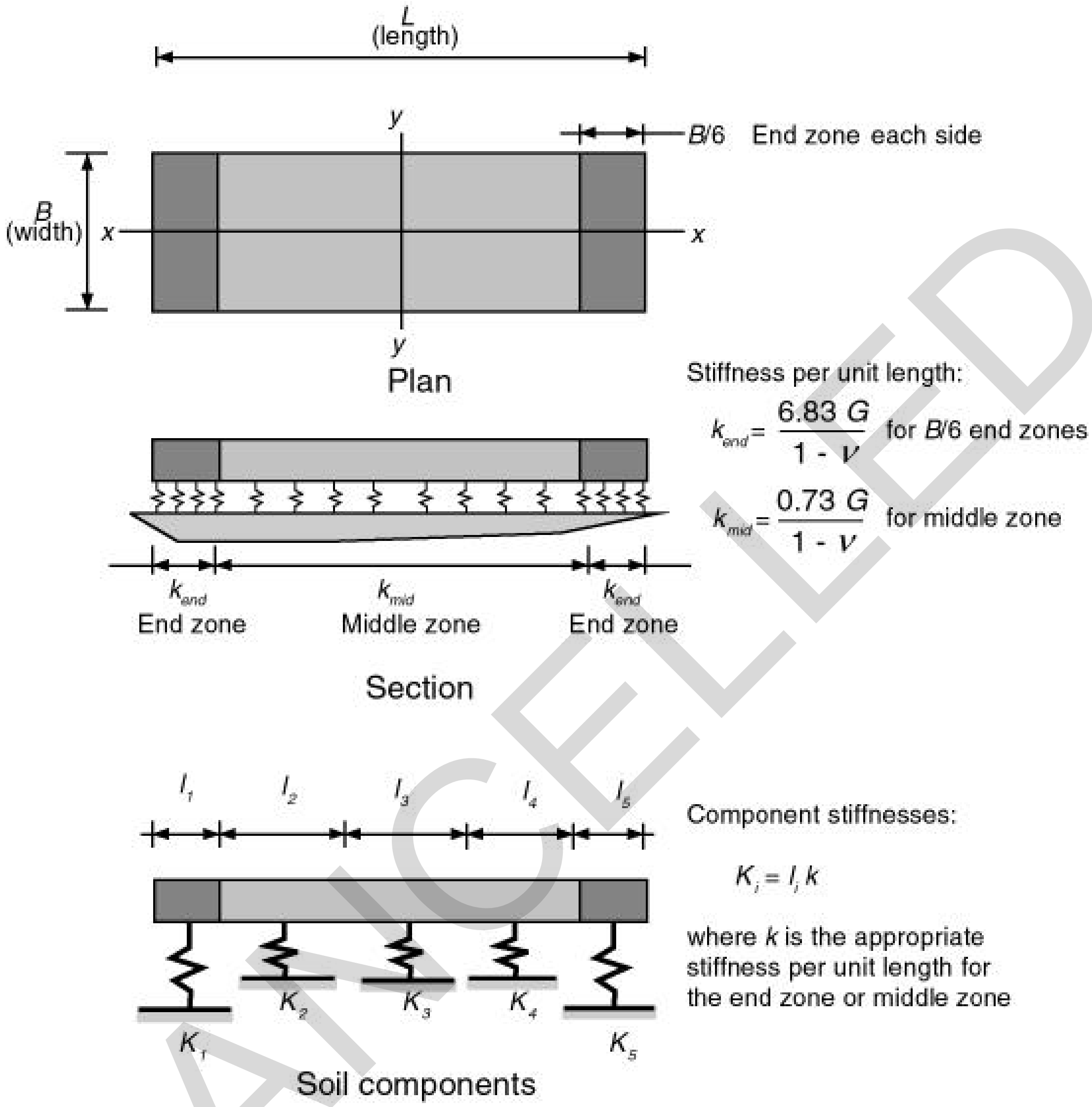


Figure 9-5 Vertical Stiffness Modeling for Shallow Bearing Footings

$$k_{sv} = \frac{1.3G}{B(1-v)} \qquad (9\text{-}9)$$

(b) Capacity parameters.

1. In the absence of moment loading, the vertical load capacity of a rectangular footing of width *B* and length *L* is

$$Q_c = q_c BL \qquad (9\text{-}10)$$

For rigid footings subject to moment and vertical load, contact stresses become concentrated at footing edges, particularly as uplift occurs. The ultimate moment capacity, M_c, is dependent upon the ratio of the vertical load stress, *q*, to the vertical stress capacity, q_c. Assuming that contact stresses are proportional to vertical displacement and remain elastic up to the vertical stress capacity, q_c, it can be shown that uplift will occur prior to plastic yielding of the soil when q/q_c is less than 0.5. If q/q_c is greater than 0.5, then the soil at the toe will yield prior to uplift. This is illustrated in Figure 9-6. In general, the moment capacity of a rectangular footing may be expressed as:

$$M_c = \frac{LP}{2}\left(1 - \frac{q}{q_c}\right) \qquad (9\text{-}11)$$

where:

P = vertical load

$q = \dfrac{P}{BL}$

B = footing width

L = footing length in direction of bending.

2. The lateral capacity of a footing should be assumed to be attained when the displacement, considering both base traction and passive pressure stiffnesses, reaches 2.0 percent of the thickness of the footing. Upper and lower bounds of twice and one-half of this value, respectively, also apply.

(3) Pile Foundations.

(a) General. Pile foundations, in the context of this paragraph, refer to those foundation systems that are composed of a pile cap and associated driven or cast-in-place piles, which together form a pile group. A single pile group may support a load-bearing column, or a linear sequence of pile groups may support a shear wall. Generally, individual piles in a group could be expected to be less than 2 feet (0.6m) in diameter. The stiffness characteristics of single large-diameter piles or drilled shafts are described in Paragraph 9-2c(4).

(b) Stiffness parameters. For the purpose of simplified analyses, the uncoupled spring model as shown in Figure 9-1b may be used where the footing in the figure represents the pile cap. In the case of the vertical and rocking springs, it can be assumed that the contribution of the pile cap is relatively small compared to the contribution of the piles. In general, mobilization of passive pressures by either the pile

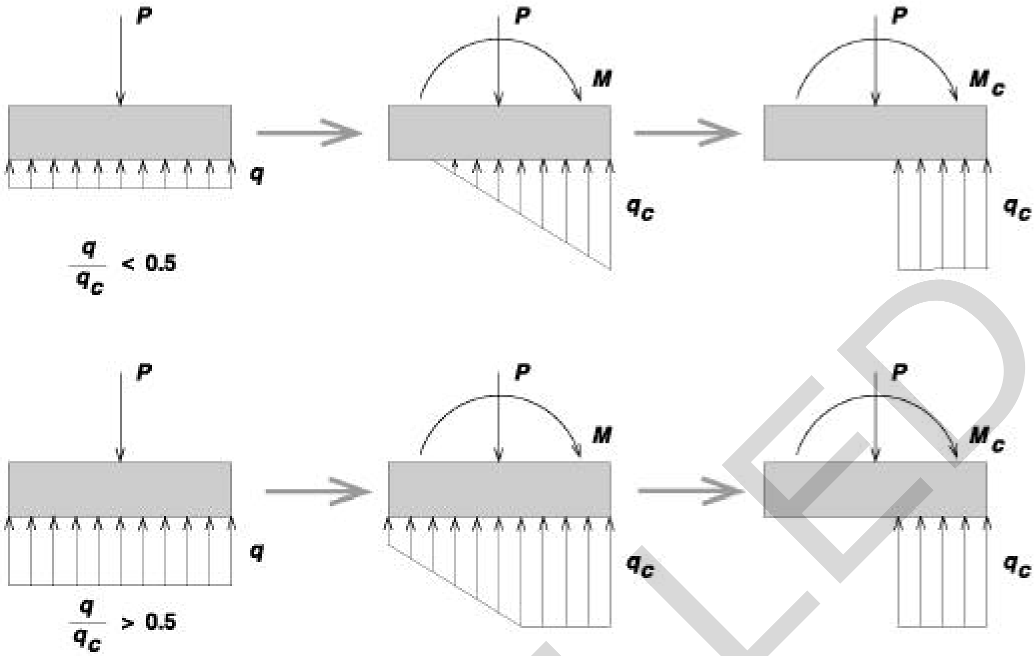


Figure 9-6   Idealized Concentration of Stress at Edge of Rigid Footings Subjected to Overturning Moment

caps or basement walls will control lateral spring stiffness; therefore, estimates of lateral spring stiffness can be computed using elastic solutions as described in Paragraph 9-2c(2)(a). In instances where piles may contribute significantly to lateral stiffness (i.e., very soft soils, battered piles) solutions using beam-column pile models are recommended. Axial pile group stiffness spring values, k_{sv}, may be assumed to be in an upper- and lower-bound range, respectively, given by:

$$k_{sv} = \sum_{n=1}^{N} \frac{0.5\,AE}{L} \text{ to } \sum_{n=1}^{N} \frac{2\,AE}{L} \qquad (9\text{-}12)$$

where:

A = cross-sectional area of a pile

E = modulus of elasticity of piles

L = length of piles

N = number of piles in group.

The rocking spring stiffness values about each horizontal pile cap axis may be computed by assuming each axial pile spring acts as a discrete Winkler spring. The rotational spring constant (moment per unit rotation) is then given by:

$$k_{sr} = \sum_{n=1}^{N} k_{vn} S_n^2 \qquad (9\text{-}13)$$

where:

k_{vn} = axial stiffness of the nth pile

S_n = distance between nth pile and axis of rotation.

Whereas the effects of group action and the influence of pile batter are not directly accounted for in the form of the above equations, it can be reasonably assumed that the latter effects are accounted for in the range of uncertainties expressed for axial pile stiffness.

(c) Capacity parameters. Vertical load capacity of piles (for both axial compression and axial tensile loading) should be determined and documented in the geotechnical investigation report. The investigation should be based on accepted foundation engineering practice using best estimate of soil properties. Consideration should be given to the capability of pile cap and splice connections to take tensile loads when evaluating axial tensile load capacity. The moment capacity of a pile group should be determined assuming a rigid pile cap, leading to an initial triangular distribution of axial pile loading from applied seismic moments. Full axial capacity of piles may, however, be mobilized when computing ultimate moment capacity, leading to a rectangular distribution of resisting moment in a manner analogous to that described for a footing in Figure 9-6. The lateral capacity of a pile group is largely dependent on that of the cap, as it is restrained by passive resistance of the adjacent soil material. The capacity may be assumed to be reached when the displacement reaches 1.0 percent of the depth of the cap in a manner similar to that for a shallow bearing foundation.

(4) Drilled shafts. In general, drilled shaft foundations or piers may be treated similarly to pile foundations. When the diameter of the shaft becomes large (>24 inches), the bending and the lateral stiffness and strength of the shaft itself may contribute to the overall capacity. This is obviously necessary for the case of individual shafts supporting isolated columns. In these instances, the interaction of the soil and shaft may be represented using Winkler-type models.

9-3. General Requirements.

a. Base. The base of the building is the level at which the earthquake motions are considered to be imparted to the structure. From the point of view of design, the base is the level at which the base shear is resisted. In a building without a basement, this is simply at grade, where footings develop lateral resistance. In a building with a basement, the base is at grade if grade-level framing or the upper portion of the basement wall is capable of developing the required lateral resistance, or at the basement level, if the lateral resistance cannot be developed at grade level. On sloping sites, the level at grade may be unrestrained at the downhill side, but restrained, like a basement, at the uphill side. The base of a building is determined by judgment, considering the mechanism for developing lateral resistance. The base should be taken at the highest level where the building can transmit lateral forces into the ground on all sides. Partial basements and sites with varying subsurface conditions are also potentially troublesome. The engineer should consider how the forces enter the substructure, and how they are transmitted into the ground. Simple three-dimensional free-body diagrams of whole substructures may be of great help in defining the design conditions.

b. Column Base. If a column is assumed to be fixed in the analysis of the superstructure, the foundation system must have the strength and stiffness required by this assumption.

c. Development of Forces into the Foundations. Foundations must be detailed to develop the horizontal and vertical components of seismic forces imparted by columns, shear walls, and braces. In instances where footing are subjected to lateral thrusts due to applied vertical loads, such horizontal thrust will be added to the lateral seismic force indicated above. An example of this case could be the outward thrusts on footings of a rigid gable bent due to applied vertical loads.

d. Interconnection of Foundation Elements. Foundation ties shall be provided as required by Chapter 7 of FEMA 302. The ties can be formed by an interconnecting grid network of reinforced concrete struts or structural steel shapes encased in concrete. As an alternative, a reinforced concrete floor slab, doweled to walls and footings to provide restraint in all horizontal directions, may be used in lieu of the grid network of ties. Slabs on grade will not be used as ties when a potential for liquefaction has been identified, or when significant differential settlement is expected between footings and slab. In such cases, slabs on grade will be cut loose from footings and made free-floating (note that the effective unsupported height of the wall is increased for this condition.) Strut ties placed below such slabs will be cushioned or separated from the slab such that slab settlement will not damage the slab or strut ties. Alternatively, it may be more economical to overexcavate the soil under the footings and recompact to control differential settlements under vertical loads, and to increase passive resistance of the sides of the footings under lateral loads so as to eliminate the need for footing ties. Slabs on ground when used as a foundation tie will have minimum reinforcing, according to ACI 318. As a minimum, a mat of #4 at 16 inches each way is recommended.

e. Overturning. The overturning moment at the base of the building is resisted by the soil through the foundation. The total load on the soil is not changed, but there is a change in the distribution of the soil pressure. For isolated spread footings, the design requirement is simply to provide for vertical components of the overturning moment in combination with the vertical forces due to dead and live loads. For wall footings, there may be enlarged footings under the boundary members, and these will have increased loads as indicated above for isolated footings, but there will also be loads on grade beams or other connecting elements.

f. Differential Settlement. Earthquake vibrations may cause consolidation or liquefaction of loose soils, and the resultant settlement of building foundations usually will not be uniform. For rigid structures supported on individual spread footings bearing on such material, excessive differential settlements can damage the superstructure. Stabilization of the soil prior to construction, or the use of piles, caissons, or deep piers bearing on a firm stratum, may be the solution to this problem.

9-4. Design of Elements.

a. General. The mechanism used for the transmission of horizontal forces may be friction between floor slab and ground; friction between bottom of footing and ground; and/or passive resistance of earth against vertical surfaces of pile caps, footings, grade beams, or basement walls. The overturning effects, which require a careful analysis of permissible overloads for the combined effect of vertical and lateral loads, must be considered in the foundation design. Although rocking of buildings about their foundations appears to have been beneficial in some instances, it is not permitted by this document because of the indeterminate nature of rocking as a means of energy dissipation. Net upward forces must be resisted by anchorage into the foundation. Stability against overturning must be provided for the short-time loading during an earthquake (or wind) without creating disparities in the foundation configuration that would result in significantly different foundation settlements due to gravity loads. These differential settlements could create more damage to the structure than the short-time deformations that might occur under the highly increased soil pressures due to earthquake effects.

b. Slabs on Ground. Slabs on ground are often thought of as nonstructural, but will in fact be nonstructural only if detailed to be unconstrained by adjacent elements. In seismic design, the slab on ground should be used as a connecting, tying, stiffening element by suitable details of joints and reinforcing in the slab and at the edges of the slab.

c. Grade Beams. Grade beams may be used to stiffen spread footings where columns are intended to have fixed bases; grade beams may also develop lateral resistance in passive pressure on their sides, especially if stiffened by an integral slab on ground. Passive-resistance values vary greatly with type of soil and depth. Adequacy of passive resistance should be determined by the geotechnical engineer. Passive resistance or lateral bearing values are permitted only where concrete is deposited directly against natural ground, or the backfill is well compacted. Passive resistance should not be used where the lateral bearing surface is close to an excavation, unless such excavation is carefully backfilled with well-compacted material. The shear capacity of the soil between such bearing surface and open or poorly compacted excavation or a similar depression may be inadequate to provide the needed resistance.

d. Basement Walls. Basement walls can develop passive pressure for normal forces. The comments on passive pressure for grade beams apply.

e. Spread Footings. Spread footings resist vertical loads through bearing pressure on the bottom, and resist horizontal loads through friction on the bottom and passive pressure on the sides.

f. Wall Footings. Wall footings resist lateral loads through friction on the bottom.

g. Piles. Piles driven into soft surficial soils must transfer the base shear into stiffer soils at lower levels. This involves bending of the piles. Criteria for design should be obtained from the geotechnical engineer. Where subsurface conditions vary over the site, the effective lengths of piles in bending may vary. The resulting variation in relative rigidity causes some piles to carry more lateral load than others, and must be considered in the foundation design. Passive pressure on the vertical surfaces of the pile cap may be a more effective method of shear transfer. For pile caps in weak soils, the use of select compacted backfill will enhance the lateral load capacity.

h. Batter Piles. The use of batter piles should be avoided. Their greater lateral stiffness relative to the vertical piles attracts most of the lateral forces to themselves, resulting in an unbalanced lateral-load-resisting system. Because the inclination of the batter piles is usually small, very large vertical components of force are developed between the vertical and adjacent batter piles. The pile cap must be detailed to accommodate these forces, and the caps may need to be stiffened by horizontal grade beams to prevent rotation under these forces.

i. Foundation Ties. Ties will be designed to carry an axial tension and compression horizontal force equal to 10 percent of the larger column load. The minimum tie will be 12 inches by 12 inches (305mm x 305mm) , with four #5 (15M) longitudinal bars and #3 (10M) ties at 12 inches (305mm) on center.

j. *Retaining Walls.* Building walls retaining soil should be evaluated for the effects of seismic earth pressures. The seismic earth pressure acting on a building wall retaining nonsaturated, level soil above the groundwater table may be approximated as:

$$Dp = 0.4 k_h \gamma_t H_{rw} \qquad (9\text{-}15)$$

where:

Dp = additional earth pressure due to seismic shaking, which is assumed to be a uniform pressure

k_h = horizontal seismic coefficient in the soil, which may be assumed equal to 0.5 S_{DS}/2.5

g = the total unit weight of soil

H_{rw} = the height of the retaining wall.

The seismic earth pressure given above should be added to the static earth pressure to obtain the total earth pressure on the wall. The expression in Equation 9-13 is a conservative approximation of the Mononabe-Okabe formulation. Seismic earth pressures much higher than summarized above may develop on walls that are required to develop passive pressures to resist lateral forces. In such cases, static passive earth pressure formulations, neglecting inertia forces in the soil, may be used to estimate the magnitude of total (static plus seismic) earth pressures on the wall. A triangular pressure distribution may be assumed.

k. *Mixed Systems.* When subsurface conditions vary significantly across a site, it is sometimes effective to use mixed systems, e.g., combinations of drilled piers and spread footings. Geotechnical consultation is especially important for mixed systems in order to control differential settlements. The difference in lateral stiffnesses between the spread footings and drilled piers must be considered in the foundation earthquake design. Nominal values of the soil springs, determined in accordance with the requirements of this chapter, may be used in the analysis.

9-5. Acceptance Criteria.

a. *Performance Objective 1A.* The response modification factors, *R*, for Performance Objective 1A, shall be in accordance with the structural system identified in Table 7-1. The design of the foundation shall be in accordance with Chapter 7 of NEHRP as modified by this chapter.

b. *Enhanced Performance Objectives.*

(1) Linear elastic analyses with *m* factors. Structural foundation components should be considered to be force-controlled, and their lower-bound capacity, Q_{CL}, will be the nominal capacity, in accordance with FEMA 302, multiplied by the appropriate capacity reduction factor, *N*. If soil springs are used in the analyses, the nominal stiffness coefficients prescribed in this chapter are to be multiplied by 0.5 for Life Safety, 1.0 for Safe Egress, and 2.0 for Immediate Occupancy performance levels.

CHANGES

Page 10-16, Para 10-3 b (1): Change "Paragraph 7-1 b (5) to "Paragraph 10-1 b (5)".

Page 10-20, Para 10-3 e (4) (b): Change "Equation 7-1" to "Equation 10-1".

CHAPTER 10
NONSTRUCTURAL SYSTEMS AND COMPONENTS

10-1. General.

a. Component Force Transfer. Components shall be supported or braced such that the component forces are transferred to the structure of the building. Component seismic attachment shall be bolted, welded, or otherwise positively fastened without consideration of frictional resistance produced by the effects of gravity. The design documents shall include sufficient information relating to the attachments to verify compliance with the requirements of this chapter. For buildings in Seismic Design Categories D, E, and F, if the supported weight of the nonstructural systems and components with flexible dynamic characteristics exceeds 25 percent of the weight of the building, the building shall be designed considering the interaction effects between the building and the supported items.

b. Seismic Forces. Seismic design of nonstructural components shall be in accordance with Chapter 6 of FEMA 302, and shall include the following considerations:

(1) Seismic forces (F_p) shall be determined in accordance with:

$$F_P = \frac{0.4 a_p S_{DS} W_p}{R_p / I_p}\left(1 + 2\frac{z}{h}\right) \quad (10\text{-}1)$$

F_p is not required to be taken as greater than:

$$F_P = 1.6 S_{DS} I_p W_p \quad (10\text{-}2)$$

nor less than

$$F_P = 0.3 S_{DS} I_p W_p \quad (10\text{-}3)$$

where:

F_P = Seismic design force centered at the component's center of gravity and distributed relative to component's mass distribution.

a_p = Component amplification factor that varies from 1.0 to 2.50 (Select appropriate value from Table 10-1 or Table 10-2.

S_{DS} = Spectral acceleration, short period, as determined from Chapter 3.

W_p = Component operating weight.

R_p = Component response modification factor that varies from 1.0 to 5.0 (select appropriate value from Table 10-1 or Table10-2.

I_p = Component importance factor that is either 1.0 or 1.5 (See Paragraph 10-1d).

z = Height in structure of highest point of attachment of component. For items at or below grade, the base, z, shall be taken as 0.

h = Average roof height of structure relative to grade elevation.

Table 10-1: Architectural Components Coefficients

Architectural Component or Element	A_p [a]	R_p [b]
Interior Nonstructural Walls and Partitions (See also Section 6.2.8 of FEMA 302)		
Plain (unreinforced) masonry walls	1.0	1.25
All other walls and partitions	1.0	2.5
Cantilever Elements (unbraced or braced to structural frame below its center of mass)		
Parapets and cantilever interior nonstructural walls	2.5	2.5
Chimneys and stacks where laterally supported by structures	2.5	2.5
Cantilever Elements (Braced to structural frame above its center of mass)		
Parapets	1.0	2.5
Chimneys and stacks	1.0	2.5
Exterior nonstructural walls	1.0 [c]	2.5
Exterior Nonstructural Wall Elements and Connections (see also Section 6.2.4 of FEMA 302)		
Wall element	1.0	2.5
Body of wall panel connections	1.0	2.5
Fasteners of the connecting system	1.25	1
Veneer		
High deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.25
Penthouses (except when framed by an extension of the building frame)	2.5	3.5
Ceilings (see also Section 6.2.6 of FEMA 302)		
All	1.0	2.5
Cabinets		
Storage cabinets and laboratory equipment	1.0	2.5
Access floors (see also Section 6.2.7 of FEMA 302)		
Special access floors (designed in accordance with Section 6.2.7.2 of FEMA 302)	1.0	2.5
All other	1.0	1.25
Appendages and Ornamentation	2.5	2.5
Signs and Billboards	2.5	2.5
Other Rigid Components		
High deformability elements and attachments	1.0	3.5
Limited deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.25
Other flexible components		
High deformability elements and attachments	2.5	3.5
Limited deformability elements and attachments	2.5	2.5
Low deformability elements and attachments	2.5	1.25

a A lower value for a_p may be justified by detailed dynamic analysis. The value for a_p shall not be less than 1.00. The value of $a_p = 1$ is for equipment generally regarded as rigid and rigidly attached. The value of $a_p = 2.5$ is for flexible components or flexibly attached components. See Chapter 2 of FEMA 302 for definitions of rigid components and flexible components, including attachments.

b $R_p = 1.25$ for anchorage design when component anchorage is provided by expansion anchor bolts, shallow chemical anchors, or shallow (nonductile) cast-in-place anchors, or when the component is constructed of nonductile materials. Powder-actuated fasteners (shot pins) shall not be used for component anchorage in tension applications in Seismic Design Categories D, E, or F. Shallow anchors are those with an embedment length-to-diameter ratio of less than 8.

c Where flexible diaphragms provide lateral support for walls and partitions, the design forces for anchorage to the diaphragm shall be as specified in Section 5.2.5.4.4 of FEMA 302.

Table 10-2: Mechanical and Electrical Components Coefficients

Mechanical and Electrical Component or Element[c]	a_p [a]	R_p [b]
General Mechanical		
Boilers and furnaces	1.0	2.5
Pressure vessels on skirts and free-standing	2.5	2.5
Stacks	2.5	2.5
Cantilevered chimneys	2.5	2.5
Other	1.0	2.5
Manufacturing and Process Machinery		
General	1.0	2.5
Conveyors (nonpersonnel)	2.5	2.5
Piping Systems		
High deformability elements and attachments	1.0	3.5
Limited deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.25
HVAC System Equipment		
Vibration isolated	2.5	2.5
Non-vibration isolated	1.0	2.5
Mounted in-line with ductwork	1.0	2.5
Other	1.0	2.5
Elevator Components	1.0	2.5
Escalator Components	1.0	2.5
Trussed Towers (free-standing or guyed)	2.5	2.5
General Electrical		
Distributed systems (bus ducts, conduit, cable tray)	1.0	3.5
Equipment	1.0	2.5
Lighting Fixtures	1.0	1.25

a A lower value for a_p is permitted provided a detailed dynamic analysis is performed which justifies a lower limit. The value for a_p shall not be less than 1.00. The value of $a_p = 1$ is for equipment generally regarded as rigid or rigidly attached. The value of $a_p = 2.5$ is for flexible components or flexibly attached components. See Chapter 2 of FEMA 302 for definitions of rigid components and flexible components, including attachments.

b $R_p = 1.25$ for anchorage design when component anchorage is provided by expansion anchor bolts, shallow chemical anchors, or shallow low deformability cast-in-place anchors or when the component is constructed of nonductile materials. Powder-actuated fasteners (shot pins) shall not be used for component anchorage in Seismic Design Categories D, E, or F. Shallow anchors are those with an embedment length-to-diameter ratio of less than 8.

c Components mounted on vibration isolation systems shall have a bumper restraint or snubber in each horizontal direction. The design force shall be taken as $2F_P$.

The force, F_p, shall be applied independently longitudinally and laterally in combination with service loads associated with the components. Horizontal and vertical load effects shall be combined as indicated in ASCE 7, substituting F_p for the term Q_E. The reliability/redundancy factor, D, in FEMA 302 shall be taken as equal to 1.0. When positive and negative wind loads exceed F_p for nonstructural exterior walls, these wind loads shall govern the design. Similarly, when building code horizontal loads exceed F_p for interior partitions, the specified building code loads shall govern the design.

c. Seismic Relative Displacement. Relative structural displacements that may affect the design of nonstructural systems and components shall be calculated in accordance with Section 6.1.4 of FEMA 302.

d.. Component Importance Factor. Compliance with the provisions in Chapter 6 of FEMA 302 with Component Importance Factor, I_p, equal to 1.0 satisfies the acceptance criteria for Performance Objective 1A (Life Safety). For buildings with enhanced performance objectives, a Component Importance Factor of 1.5 will be assigned to selected nonstructural components as follows:

I_p = 1.5 Life safety component is required to function after an earthquake.

I_p = 1.5 Component contains or can damage hazardous contents.

I_p = 1.5 Storage racks in occupancies with general access (e.g., warehouses or retail stores).

I_p = 1.5 Components needed for continued operation of an essential facility (Seismic Use Groups IIIE).

I_p = 1.0 All other components.

10-2. Architectural Components.

a. Introduction. This paragraph defines architectural components, discusses their participation and importance in relation to the seismic design of the structural system, and prescribes the criteria for their design to resist damage from seismic lateral forces. The fundamental principle and underlying criterion of this paragraph are that the design of architectural components will be such that they will not collapse and cause personal injury due to the accelerations and displacements induced by severe seismic disturbances, and that the architectural components will withstand more frequent but less severe seismic disturbance without excessive damage and economic loss.

b. Definition. Architectural components are elements such as partitions, stairways, windows, suspended ceilings, parapets, building ornamentation and appendages, and storage racks. They are called architectural because they are not part of the vertical-or lateral-load-carrying systems of the building, nor part of the mechanical or electrical systems. Although they are usually shown on the architectural drawings, they often have a structural aspect. The architect will consult with the structural, mechanical, and electrical engineers when dealing with these elements. Examples of architectural components that have a structural aspect follow.

(1) Nonstructural walls. A wall is considered “architectural” or “nonstructural” when it does not participate in the resistance to lateral forces. This is the case if the wall is isolated; that is, not connected to the structure at the top and the ends, or if it is very flexible relative to the structural wall frames. Note that an isolated wall must be capable of acting as a cantilever from the floor, or be braced laterally.

(2) Curtain walls and filler walls. A curtain wall is an exterior wall, usually of masonry, that lies outside of, and usually conceals, the structural frame. A filler wall is an infill, usually of masonry, within the members of a frame. These are often considered architectural if they are designed and detailed by the architect, but they can act as structural shear walls. If they are connected to the frame, they will be subjected to the deflections of the frame and will participate with the frame in resisting lateral forces. Curtain walls and infill walls in buildings governed by this document will be designed so as not to restrict the deformations of the structural framing under lateral loads. Lateral supports and bracing for these walls will be provided as prescribed in the following paragraphs.

(3) Partial infill wall. A partial infill wall is one that has a strip of windows between the top of the solid infill and the bottom of the floor above, or has a vertical strip of window between one or both ends of the infill and a column. Such walls require special treatment: if they are not properly isolated from the structural system they will act as shear walls. The wall with windows along the top is of particular concern because of its potential effect on the adjacent columns. The columns are fully braced where there is an adjacent infill, but are unbraced in the zone between the windows. The upper, unbraced part of the column is a “short column,” and its greater rigidity (compared with other unbraced columns in the system) must be accounted for in the design. As indicated above, all infills in buildings governed by this document will be considered to be nonstructural components, and will be designed so as not to restrict the deformation of the structural framing under lateral loads.

(4) Precast panels. Exterior walls that have precast panels attached to the frame are a special case. The general design of the walls is usually shown on the architectural drawings, while the structural details of the panels are usually shown on the structural drawings. Often, the structural design is assigned to the General Contractor so as to allow maximum use of the special expertise of the selected panel subcontractor. In such cases, the structural drawings will include design criteria and representative details in order to show what is expected. The design criteria will include the required design forces and the frame deflections that must be accommodated by the panels and their connections.

c. Design Criteria. Architectural elements must safely resist horizontal forces prescribed by Equation 10-1, and must be capable of conforming (accommodating) to the lateral deflections that they will be subjected to during the lateral deformations of the structure.

(1) Lateral force coefficients. Coefficients for Equation 10-1 applicable to architectural components are provided in Table 10-1.

(2) Displacements. Allowable story drift for structures is prescribed in Table 6-1. Determination of relative displacement applicable to architectural components is prescribed in Section 6.1.4 of FEMA 302.

d. Detailed Requirements

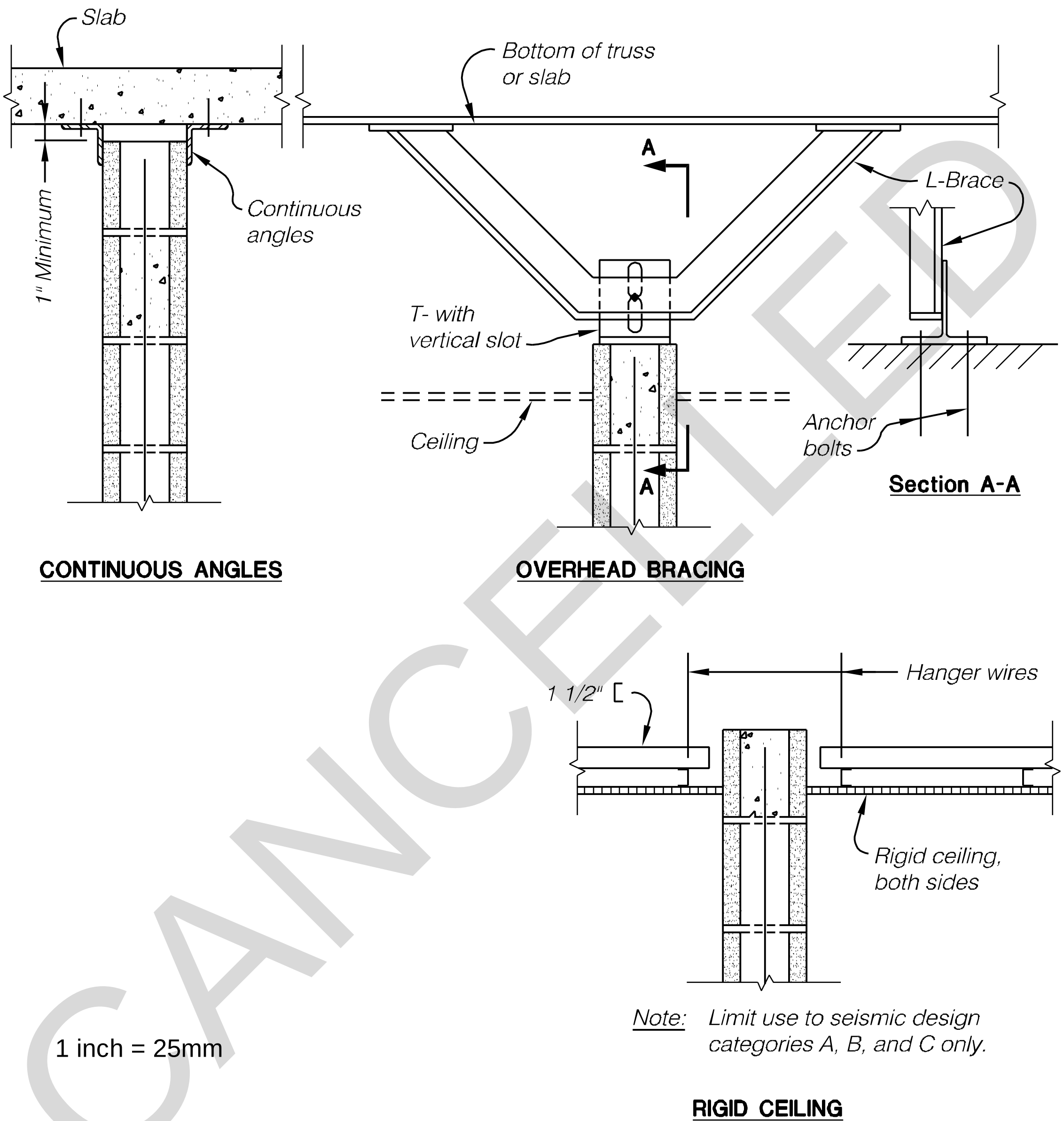
(1) Partitions. Partitions are classified into two general categories: rigid and nonrigid.

(a) Rigid partitions. This category generally refers to nonstructural masonry walls. Walls will be isolated where they are unable to resist in-plane lateral forces to which they are subjected, based on relative rigidities. Typical details for isolation of these walls are shown in Figure 10-1. These walls will be designed for the prescribed forces normal to their plane.

(b) Nonrigid partitions. This category generally refers to nonstructural partitions such as stud and drywall, stud and plaster, and movable partitions. When constructed according to standard recommended practice, it is assumed that the partitions can withstand the design in-plane drift of .005 times the story height (i.e., 1/16 inch per foot (5.2mm per meter) of height) without damage. Therefore, if the structure is designed to control drift within the prescribed limits, these partitions do not require special isolation details. They will be designed for the prescribed seismic force acting normal to flat surfaces; however, wind or the usual 5 pounds per square foot partition load will usually govern. If the structural design drift is not controlled within the prescribed limits, isolation of partitions will be required for reduction of nonstructural damage. Economic justification between potential damage and costs of isolation will be considered. A decision has to be made for each project as to the role, if any, such partitions will contribute to damping and response of the structure, and the effect of seismic forces parallel to the partition resulting from the structural system as a whole. Usually it may be assumed that this type of partition is subject to future alterations in layout location. The structural role of partitions may be controlled by height of partitions and method of support.

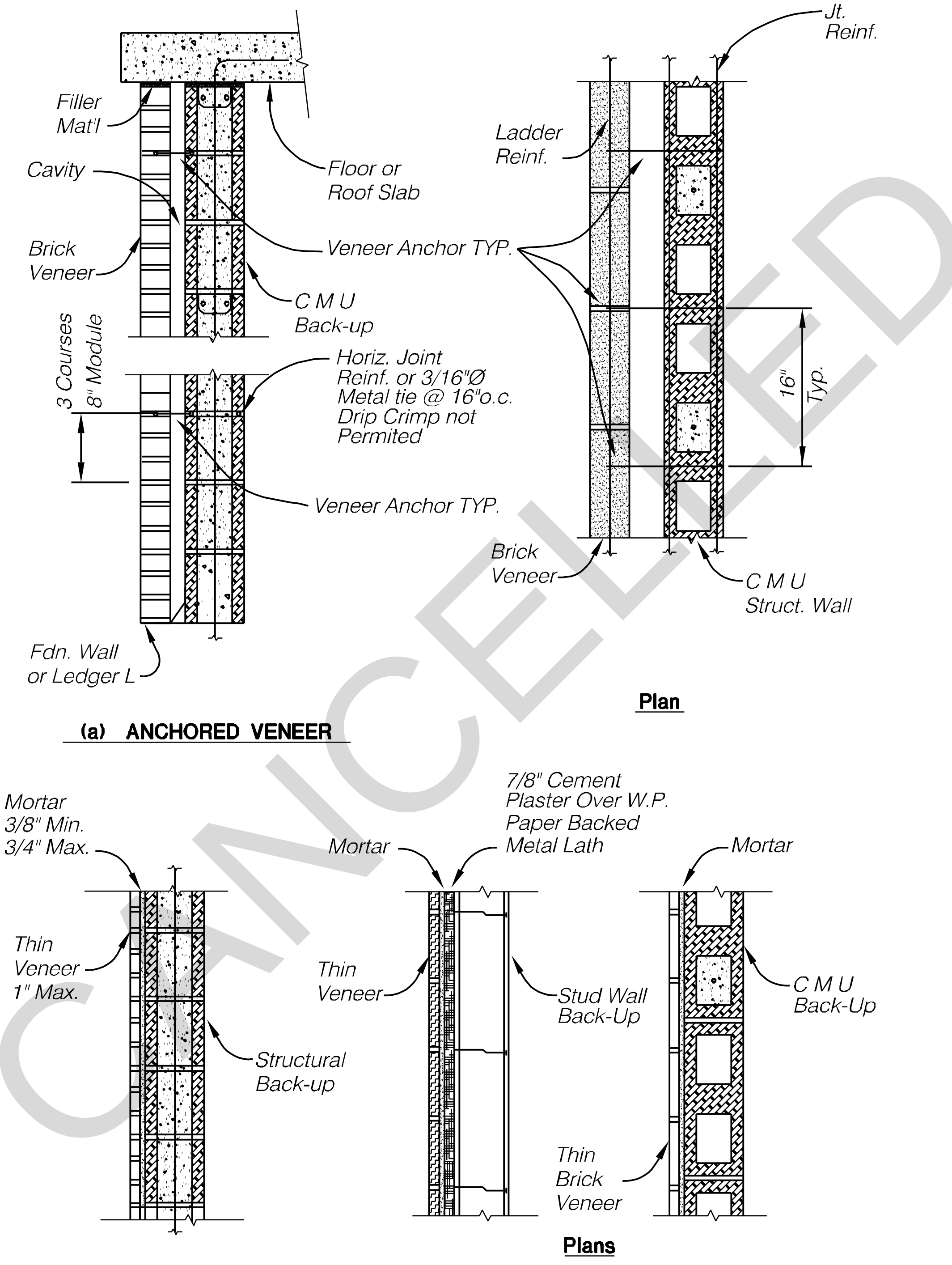
(2) Veneered walls. There are two methods for attaching veneer to a backup structural wall (see Figure 10-2).

(a) Anchored veneer is a masonry facing secured by joint reinforcement or equivalent mechanical tie attached to the backup. All required load-carrying capacity (both vertical and lateral) will be provided by the structural backup wall. The veneer will be nonbearing and isolated on three edges to preclude it from resisting any load other than its own weight, and in no case shall it be considered part of the wall in computing required thickness of a masonry wall. The veneer will be not less than 1½ inches (38mm), nor more than 5 inches (127mm) thick. The veneer will be tied to the structural wall with joint reinforcement



LATERAL SUPPORTS - NONSTRUCTURAL PARTITION

Figure 10-1 Typical details of isolation of walls.



Note: Limit wall deflection to h/720.

1 inch = 25mm

Figure 10-2 Veneered walls.

or 3/16-inch (5mm) round corrosion-resisting metal ties capable of resisting, in tension or compression, the wind load or two times the weight of veneer, whichever governs. Maximum spacing of ties is 16 inches (406mm), and a tie must be provided for each 2 square feet ($0.2m^2$) of wall area. Adjustable ties are not permitted in Seismic Design Categories D, E, and F. They may be used in Seismic Design Categories A, B, and C if the basic wind speed is less than 100 mph (160 kmph). If adjustable ties are used, they will be the double pintle-eye type, with a minimum wire size of 3/16 inch (5mm); play within the pintle will be limited to 1/16 inch (1.6mm), and the maximum vertical eccentricity will not exceed 1/2 inch (12.7mm). The maximum space between the veneer and the backing will not exceed 3 inches (75mm), unless spot mortar bedding is provided to stiffen the ties. A noncombustible, noncorrosive horizontal structural framing will be provided for vertical support of the veneer. The maximum vertical distance between horizontal supports will not exceed 25 feet (7.6m) above the adjacent ground, and 12 feet (3.7m) maximum spacing above the 25-foot (7.6m) height.

(b) Adhered veneer is masonry veneer attached to the backing with minimum 3/8-inch (9.5mm) to maximum 3/4 inch (19mm)mortar or with approved thin-set latex Portland cement mortar. The bond of the mortar to the supporting element will be capable of withstanding a shear stress of 50 psi (345 kPa). Maximum thickness of the veneer will be limited to 1 inch (25mm). Since adhered veneer is supported through adhesion to the mortar applied over a backup, consideration will be given for differential movement of supports, including that caused by temperature, shrinkage, creep, and deflection. A horizontal expansion joint in the veneer is recommend at each floor level to prevent spalling. Vertical control joints should be provided in the veneer at each control joint in the backup.

(3) Connections of Exterior Wall Panels. Precast, non-bearing, non-shear wall panels or other elements that are attached to or enclose the exterior will be designed and detailed to accommodate movements of the structure resulting from lateral forces or temperature changes. The concrete panels or other elements will be supported by means of cast-in-place concrete or by mechanical devices. Connections and panel joints will be designed to allow for the relative movement between stories, and will be designed for the forces specified in Section 6.1.6 of FEMA 302. Connections will have sufficient ductility and rotation capacity so as to preclude fracture of the concrete or brittle failures at or near welds. Inserts in concrete shall be attached to or hooked around reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel. Connections to permit movement in the plane of the panel for story drift may be properly designed sliding connections using slotted or oversized holes, or may be connections that permit movement by bending of steel components without failure. Typical design forces are shown in Figure 10-3.

(4) Suspended Ceiling Systems. Seismic design is required for structures conforming to Seismic Design Categories C, D, E, and F. Earthquake damage to suspended ceiling systems can be limited by proper support and detailing.

Suspended ceiling framing systems will be designed for forces prescribed in Section 6.2.6 of FEMA 302.

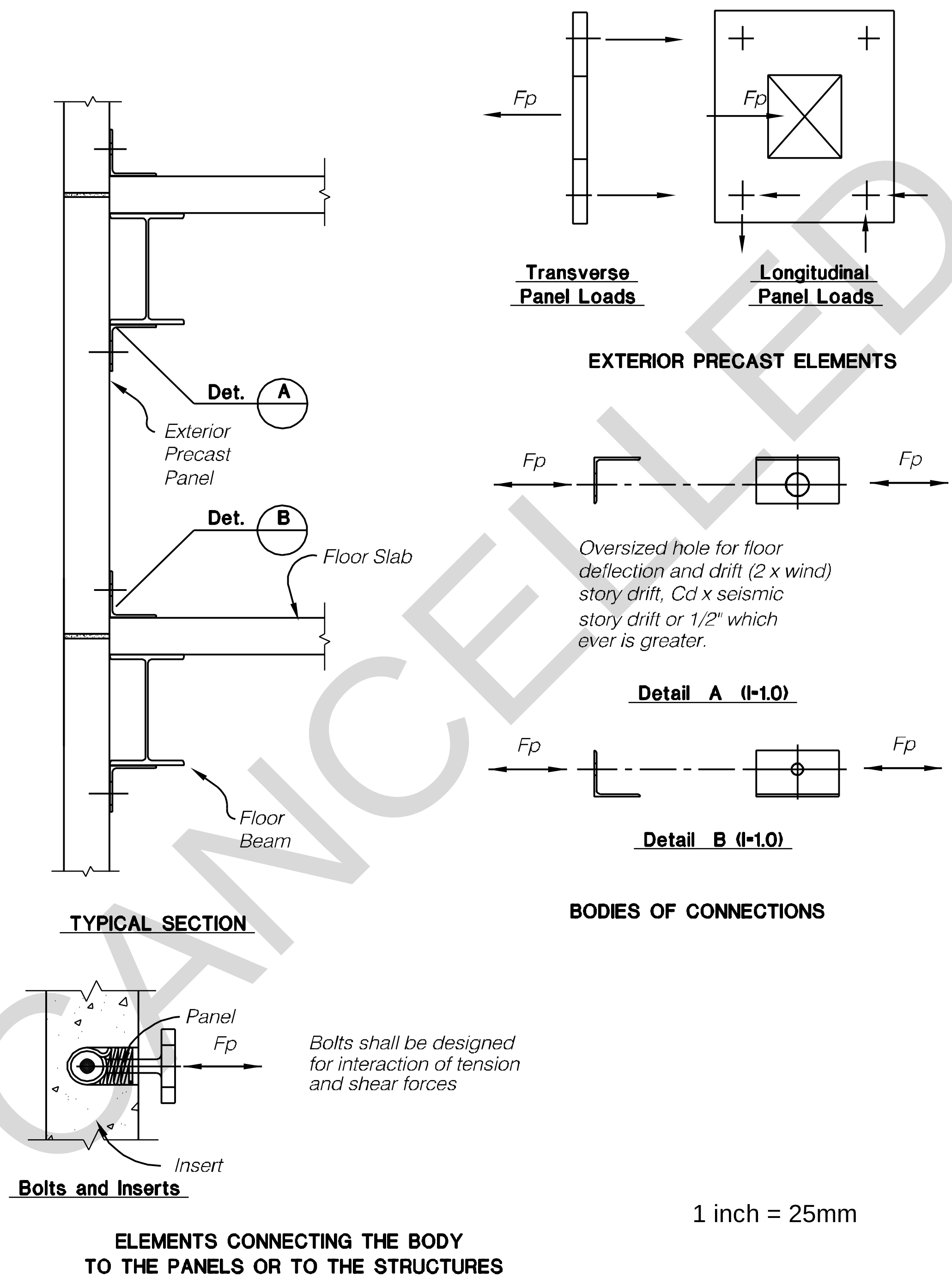


Figure 10-3 Design forces for exterior precast elements.

The ceiling weight, W_P, will include all light fixtures and other equipment laterally supported by the ceiling. For purposes of determining the lateral force, a ceiling weight of not less than 4 pounds per square foot (0.2kPa) will be used. The support of the ceiling systems will be by positive means, such as wire or an approved seismic clip system. Typical details of suspended acoustical tile ceilings are shown in Figure 10-4.

(5) Window Frames. Window frames will be designed to accommodate deflections of the structure without imposing a load on the glass. Because glass is a brittle material, a considerable hazard of falling glass may be present. It is particularly serious if the glass is above and adjacent to a public way. This hazard can be eliminated by proper isolation between glass and its enclosing frame. It is obvious that the magnitude of isolation required depends upon the drift and the size of the individual pane or enclosing frame; thus, a pane of glass in a full-story-height frame should have an isolation or movement capability as great as the maximum possible drift (e.g., C_d times the calculated elastic story displacement in Table 6-1). The actual isolation clearance will depend on the geometry and deformation characteristics of enclosing frame, frame support, and structural system. Special care will be exercised in the field to see that such isolation is actually obtained.

(6) Stairways. Stairways tend to act like struts; therefore, the rigidity of the stairway, relative to the structure, will be considered. In some cases, the stairway will be isolated in order to prevent damage to the stair by the building frame, or to prevent the stair from imposing an unwanted constraint on the frame.

(7) Cantilever parapets, ornamentation, and appendages shall be designed with a_p and R_p equal to 2.5 in accordance with Table 10-1.

(8) Storage racks.

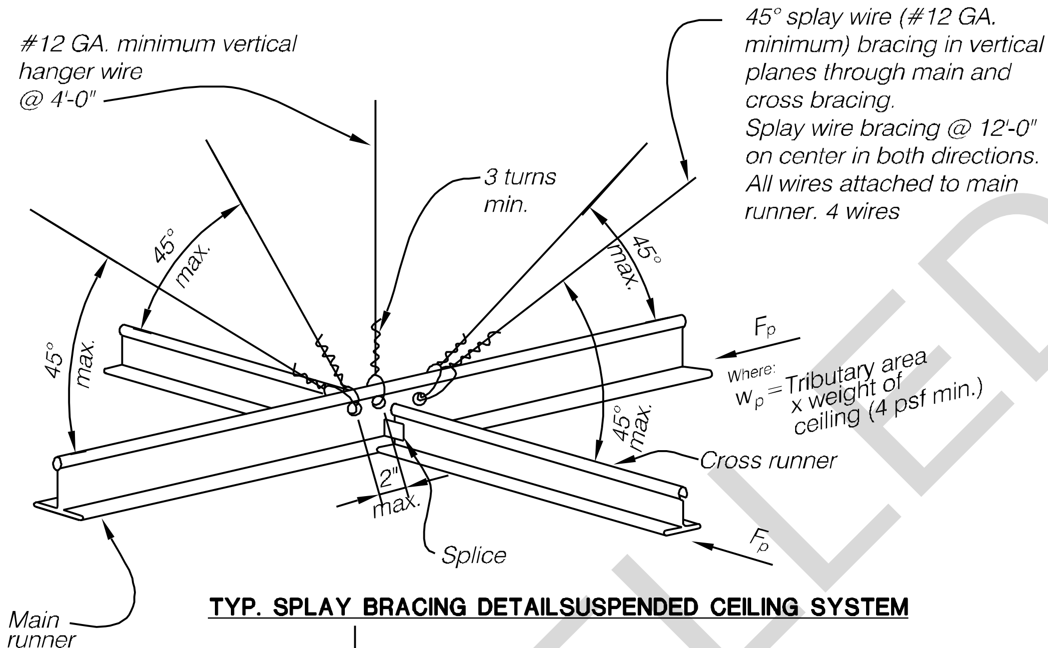
(a) Storage racks supported at grade will be treated as non-building structures in accordance with Chapter 14 of FEMA 302. The weight, *W*, will be equal to the weight of the rack plus its contents.

1. Rigid racks. Racks having periods of vibration less than 0.06 second will be governed by Section 14.2.2 of FEMA 302.

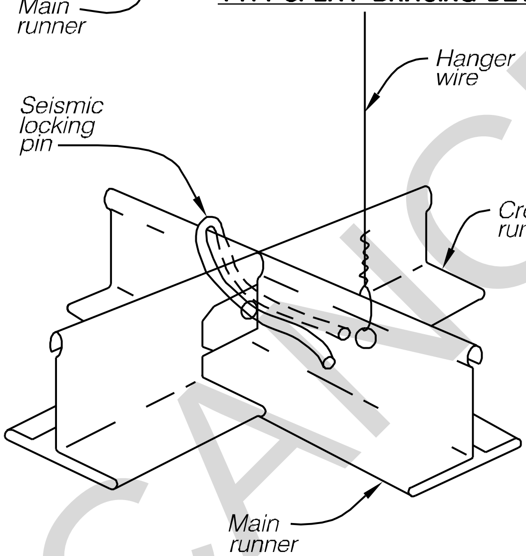
2. Flexible Rack. Racks having a period of vibration greater than 0.06 second will be governed by Section 14.3 of FEMA 302.

(b) Storage racks supported by other structures will be governed by Section 14.1.2 of FEMA 302.

e. Alternative Designs. Where an accepted national standard or approved test data provide the basis for earthquake resistance of a particular type of architectural element or rack, such standards of data may be accepted as a basis for design. Where approved standards or test data define acceptance criteria in terms of allowable stresses as opposed to strength, the design seismic forces shall be obtained from FEMA 302 and reduced by a factor of 1.4 for use with allowable stresses. Allowable stress

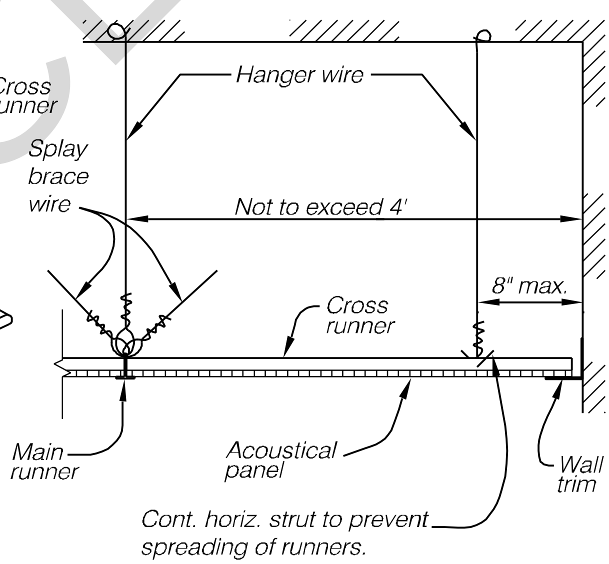


TYP. SPLAY BRACING DETAILSUSPENDED CEILING SYSTEM



Locking Clip to Fasten Cross Runner to Main Runner

Splices and intersections of runners shall be attached with mechanical interlocking connectors such as pop rivets, screws, pins, plates with bent tabs, or other approved connectors. Design connectors for 2x design load or ultimate axial tension or compression (minimum 120 pounds).



SUSPENDED CEILING DETAIL AT WALL

Note:
The details in this figure apply only in seismic design categories D, E, and F

1 inch = 25mm
1 foot = 0.3m
1 pound = 4.45 N

Figure 10-4 Suspended acoustical tile ceiling.

increases used in approved standards are permitted. Detailing shall be in accordance with the approved standards.

10-3. Mechanical and Electrical Equipment.

a. Introduction. This paragraph prescribes the criteria for structural design of anchorages and supports for mechanical and electrical equipment in seismic areas.

(1) Design Goals. The goal of design is that the anchorages and supports will withstand the accelerations induced by severe seismic disturbances without collapse or excessive deflection, and withstand the accelerations induced by less severe seismic disturbances without exceeding yield stresses. The design forces are related to the inertia forces on the equipment, and are calculated on the weight of the equipment; accordingly, design provisions often speak of equipment. The design is for the supports of the equipment, however, not the equipment itself. Ordinary equipment, which is fabricated at some distance from the site and is transported by truck and/or railroad, is assumed to have adequate strength. Critical equipment, which may have to be substantiated by design or test, is beyond the scope of this manual.

(2) Earthquake loadings. The earthquake loadings applied to equipment supports are generally higher than the earthquake loadings used in the design of the building structural system. One reason is the amplification of the ground motion acceleration transmitted to elements in the elevated stories of a building due to dynamic response. Another reason is that equipment supports often lack the extra margin of safety provided by reserve strength mechanisms, such as participation of architectural elements, inelastic behavior of structural elements, and redundancy in the structural system, which are characteristics of buildings.

b. General. All equipment anchorages and supports designed under the provisions of this chapter will conform to the following requirements:

(1) Equipment supports or bracing on buildings or other structures shall be designed in accordance with Paragraph 7-1 b (5).

(2) Equipment on the ground. Mechanical and electrical equipment that is supported at or below ground level will be considered to be non-building structures, and are governed by other agency documents.

(3) Weight limitations. Equipment in buildings will be considered to be within the scope of this chapter if the maximum weight of the individual item of equipment does not exceed 10 percent of the total building weight, or 20 percent of the total weight of the floor at the equipment level. The response of equipment is dependent upon the response of the building in which it is housed. If the weight of the equipment is appreciable, relative to the weight of the building, the interaction of the equipment with the building (i.e., the coupling effect) will change the building's response characteristics. It is assumed that equipment within the above weight limitations has a negligible effect

on the response of the building. Equipment that is not within the above limitations is outside the scope of this manual, and must be designed using a more rigorous method of analysis.

(4) Rigorous analysis. No portion of this chapter will be construed to prohibit a rigorous analysis of equipment and the supporting mechanism by established principles of structural dynamics. Such an analysis will demonstrate that the fundamental principle and underlying criterion of Paragraph 10-3a are satisfied. In no case will the design result in capacities less than 80 percent of those required by Paragraph 10-1b.

(5) Securing Equipment. Friction resulting from gravity loads as a method of resisting seismic forces is not acceptable and will not be allowed. Both vertical and horizontal accelerations are possible during an earthquake. Under vertical acceleration, the gravity force required to maintain friction can be greatly diminished. This could result in a reduction or elimination of the friction force available to resist horizontal seismic loads, as simultaneous vertical and horizontal accelerations are possible. Equipment will thus be secured by bolts, embedment, or other acceptable positive means of resisting horizontal forces. Refer to Figures 10-12 and 10-13 for typical details.

(6) Special requirements. Requirements for lighting fixtures and supports, piping, stacks, bridge cranes and monorails, and elevator systems are covered in Paragraphs 10-3d through 10-3h.

c. Seismic Design Forces. Equations 10-1, 10-2, and 10-3 prescribe seismic forces for equipment that is supported by buildings or other structures. The amplification of the floor response motion in the higher level of the structure is represented by the factor $(1+z/h)$, and the amplification of the design force due to the dynamic response of the equipment and/or its supports is represented by the component amplification factor, A_p. Equation 10-1 is a simplistic, but acceptable, determination of the seismic design force for equipment supports in structures. More rigorous determinations include the use of floor response spectra together with the determination of the fundamental period of the component and its attachment to the structure.

d. Lighting Fixtures in Buildings. In addition to the requirements of the preceding paragraphs, lighting fixtures and supports will conform to the following seismic requirements in structures conforming to Seismic Design Criteria C, D, E, and F.

(1) Materials and construction.

(a) Fixture supports will employ materials that are suitable for this purpose. Cast metal parts, other than those of malleable iron, and cast or rolled threads, will be subject to special investigation to ensure structural adequacy.

(b) Loop and hook or swivel hanger assemblies for pendant fixtures will be fitted with a restraining device to hold the stem in the support position during earthquake motions. Pendant-supported fluorescent fixtures will also be provided with a flexible hanger device at the attachment to the fixture channel to preclude breaking of the support. The motion of swivels or hinged joints will not cause sharp binds in conductors or damage to insulation.

(c) Each recessed individual or continuous row of fluorescent fixtures will be supported by a seismic-resisting suspended ceiling support system, and will be fastened thereto at each corner of the fixture; or will be provided with fixture support wires attached to the building structural members using two wires for individual fixtures, and one wire per unit of continuous row fixtures. These support wires (minimum 12-gauge wire) will be capable of supporting four times the support load.

(d) A supporting assembly that is intended to be mounted on an outlet box will be designed to accommodate mounting features on 4-inch (102mm) boxes, 3-inch (76mm) plaster rings, and fixture studs.

(e) Each surface-mounted individual or continuous row of fluorescent fixtures will be attached to a seismic-resisting ceiling support system. Support devices for attaching fixtures to suspended ceilings will be a locking-type scissor clamp or a full loop band that will securely attach to the ceiling support. Fixtures attached to the underside of a structural slab will be properly anchored to the slab at each corner of the fixture.

(f) Each wall-mounted emergency light unit will be secured in a manner that will hold the unit in place during a seismic disturbance.

(g) Tests. In lieu of the requirements for equipment supports given in Paragraph 10-3c, lighting fixtures and the complete fixture-supporting assembly may be accepted of they pass shaking-table tests approved by the using agency. Such tests will be conducted by an approved and independent testing laboratory, and the results of such tests will specifically state whether or not the lighting fixture supports satisfy the requirements of the approved tests. Suspension systems for light fixtures, as installed, that are free to swing a minimum of 45° from the vertical in all directions, and will withstand, without failure, a force of not less than four times the weight they are intended to support, will be acceptable.

e. Piping in Buildings. Pipes are categorized as pipes related to the fire protection system, critical piping in essential and hazardous facilities, and all other piping.

(1) Fire protection piping. All water pipes for fire protection systems will be designed under the provisions of the current issue of the “Standard for the Installation of Sprinkler Systems” of the National Fire Protection Association (NFPA No. 13). To avoid conflict with the NFPA recommendations, the criteria in the following paragraphs are not applicable to piping expressly designed for fire protection.

(2) Critical piping in essential and hazardous facilities. Critical piping is that which is required for life-safety systems, for continued operations after an earthquake, or for safety of the general public. All critical piping in essential and hazardous

facilities located in Seismic Design Criteria C, D, E, and F will be designed using the provisions in Paragraph 10-3e(4).

(3) All other piping.

(a) Piping in Seismic Design Category A structures is not required to have seismic restraint.

(b) Piping in Seismic Design Category B structures that is not categorized as essential or hazardous is not required to have seismic restraints.

(c) Piping in all other Seismic Design Category structures that is not categorized as essential or hazardous is required to have seismic restraints designed using the provisions in Paragraph 10-3e(4). Restraints may be omitted for the following installations:

1. Gas piping of less than 1-inch (25mm) inside diameter.

2. Piping in boiler and mechanical equipment rooms of less than 1¼ (32mm) inches inside diameter.

3. All other piping of less than 2½ inches (64mm) inside diameter.

4. All electrical conduit of less than 2½ inches (64mm) inside diameter.

5. All rectangular air-handling ducts of less than 6 square feet ($0.6m^2$) in cross-sectional area.

6. All round air-handling ducts less than 28 inches (711mm) in diameter.

7. All piping suspended by individual hangers 12 inches (0.3m) or less in length from the top of pipe to the bottom of the support for the hanger.

8. All ducts suspended by hangers 12 inches or less in length from the top of the duct to the bottom of the support for the hanger.

(4) Seismic restraint provisions. Seismic restraints that are required for piping by Paragraphs 10-3e(2) and 10-3e(3) will be designed in accordance with the following provisions.

(a) General. The provisions of this paragraph apply to the following:

1. Risers. All risers and riser connections.

2. Horizontal pipe. All horizontal pipes and attached valves. For the seismic analysis of horizontal pipes, the equivalent static force will be considered to act concurrently with the full dead load of the pipe, including contents.

3. Connections. All connections and brackets for pipe will be designed to resist concurrent dead and equivalent static forces. The seismic forces will be determined from the appropriate provisions below. Supports will be provided at all pipe joints unless continuity is

maintained. See Paragraph (4) below for acceptable sway bracing details.

4. Flexible couplings and expansion joints. Flexible couplings will be provided at the bottoms of risers for pipes larger than 3½ inches (89mm) in diameter. Flexible couplings and expansion joints will be braced laterally unless such lateral bracing will interfere with the action of the flexible coupling or expansion joint. When pipes enter buildings, flexible couplings will be provided to allow for relative movement between soil and building.

5. Spreaders. Spreaders will be provided at appropriate intervals to separate adjacent pipe lines unless the pipe spans and the clear distance between pipes are sufficient to prevent contact between the pipes during an earthquake.

(b) Rigid and rigidly attached pipes will be designed in accordance with Equation 7-1, where W_P is the weight of the pipes, the contents of the pipes, and the attachments. The forces will be distributed in proportion to the weight of the pipes, contents, and attachments. A piping system is assumed rigid if the maximum period of vibration is 0.05 second (for pipes that are not rigid, see Paragraph (3) below). Figures 10-5, 10-6, and 10-7, which are based on water-filled pipes with periods equal to 0.05 second, are to be used to determine the allowable span-diameter relationship for structures conforming to Seismic Design Categories C, D, E, and F for standard (40S) pipe; extra strong (80S) pipe; types K, L, and M copper tubing; and 85 red brass or SPS copper pipe.

(c) Flexible piping systems. Piping systems that are not in accordance with the rigidity requirements of Paragraph 10-3e(4)(b) (i.e., period less than 0.05 second) will be considered to be flexible (i.e., period greater than 0.05 second). Flexible piping systems will be designed for seismic forces with consideration given to both the dynamic properties of the piping system and the building or structure in which it is placed. In lieu of a more detailed analysis, the equivalent static lateral force is given by Equation 10-1, with $a_P = 2.5$. The forces will be distributed in proportion to the weight of the pipes, contents, and attachments. If the weight of the attachments is greater than 10 percent of the weight of the pipe, the attachments will be separately braced, or substantiating calculations will be required. If temperature stresses are appreciable, substantiating calculations will be required.

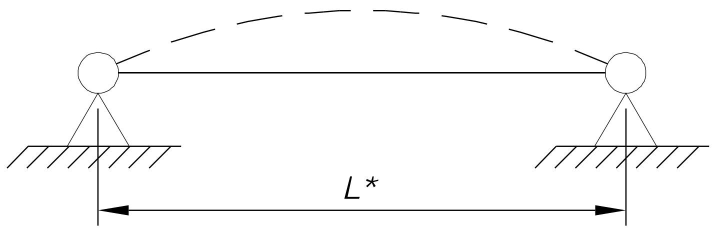
1. Separation between pipes. Separation will be a minimum of four times the calculated maximum displacement due to F_P, but not less than 4 inches (102mm) clear between parallel pipes, unless spreaders are provided.

2. Clearance. Clearance from walls or rigid elements will be a minimum of three times the calculated displacement due *to* F_P, but not less than 3 inches (76mm) clear from rigid elements.

3. Alternative method for flexible piping systems. If the provisions in the above paragraphs appear to be too severe for an economical design, alternative methods based on rational and substantial analysis may be applied to flexible piping systems.

4. Acceptable seismic details for sway bracing are shown in Figure 10-8.

f. Stacks. Stacks are actually beams with distributed mass, and as such, cannot be approximated accurately by single-mass systems.



Diameter Inches	Std. Wt. Steel Pipe 40S	Ex. Strong Steel Pipe 80S	Copper Tube Type K	Copper Tube Type L	Copper Tube Type M	85 Red Brass & SPS Copper Pipe
1	6'-6"	6'-6"	5'-0"	4'-9"	4'-6"	5'-6"
1½	7'-6"	7'-9"	5'-9"	5'-6"	5'-6"	6'-6"
2	8'-6"	8'-6"	6'-6"	6'-6"	6'-3"	7'-0"
2½	9'-3"	9'-6"	7'-3"	7'-0"	7'-0"	8'-0"
3	10'-3"	10'-6"	7'-9"	7'-6"	7'-6"	8'-9"
3½	11'-0"	11'-0"	8'-3"	8'-3"	8'-0"	9'-3"
4	11'-6"	11'-9"	9'-0"	8'-9"	8'-6"	9'-9"
5	12'-9"	13'-0"	10'-0"	9'-6"	9'-6"	10'-9"
6	13'-9"	14'-0"	10'-9"	10'-6"	10'-3"	11'-6"
8	15'-6"	16'-0"				
10	17'-0"	17'-6"				
12	18'-3"	19'-0"				

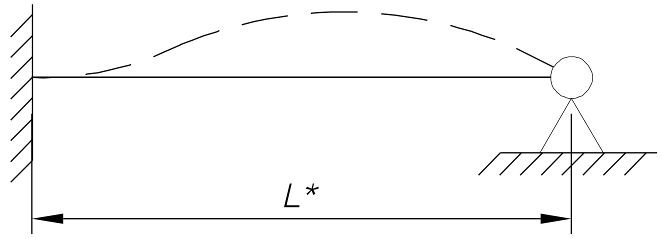
*Maximum unsupported or unbraced lengths (L) are based on water-filled pipes with period (T_a) equal to 0.05 Sec. Where

$$L^2 = 0.50 p T_a \sqrt{\frac{EIg}{w}}$$, in. or mm

1 inch = 25mm
1 foot = 0.3m

E = Modulus of Elasticity of Pipe, psi or MPa
I = Moment of Inertia of Pipe, in^4 or mm^4
w = Weight Per Unit Length of Pipe and Water, lbs/in. or N/mm

Figure 10-5. Maximum span for rigid pipe pinned-pinned.



Diameter Inches	Std. Wt. Steel Pipe 40S	Ex. Strong Steel Pipe 80S	Copper Tube Type K	Copper Tube Type L	Copper Tube Type M	85 Red Brass & SPS Copper Pipe
1	8'-0"	8'-0"	6'-0"	6'-0"	5'-9"	6'-9"
1½	9'-6"	9'-6"	7'-3"	7'-0"	7'-0"	8'-0"
2	10'-6"	10'-9"	8'-0"	8'-0"	8'-9"	9'-0"
2½	11'-9"	11'-9"	9'-0"	8'-9"	8'-6"	9'-9"
3	12'-9"	13'-0"	9'-9"	9'-6"	9'-3"	10'-9"
3½	13'-6"	14'-0"	10'-6"	10'-3"	10'-0"	11'-6"
4	14'-6"	14'-9"	11'-0"	11'-0"	10'-9"	12'-3"
5	16'-0"	16'-3"	12'-3"	12'-0"	11'-9"	13'-3"
6	17'-0"	17'-9"	13'-6"	13'-0"	12'-9"	14'-3"
8	19'-3"	20'-0"				
10	21'-3"	22'-0"				
12	23'-0"	23'-6"				

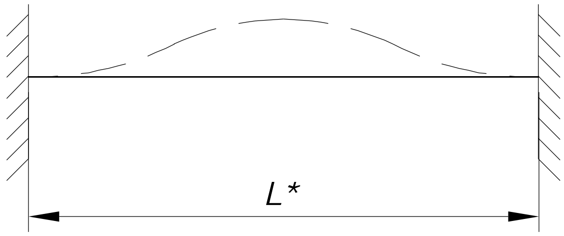
**Maximum unsupported or unbraced lengths (L) are based on water-filled pipes with period (T_a) equal to 0.05 Sec. Where*

$$L^2 = 0.78pT\sqrt{\frac{EIg}{w}}$$

1 inch = 25mm
1 foot = 0.3m

See Figure 10-5 for Notations

Figure 10-6. Maximum span for rigid pipe fixed-pinned.



Diameter Inches	Std. Wt. Steel Pipe 40S	Ex. Strong Steel Pipe 80S	Copper Tube Type K	Copper Tube Type L	Copper Tube Type M	85 Red Brass & SPS Copper Pipe
1	9'-6"	9'-6"	7'-3"	7'-3"	7'-0"	8'-0"
1½	11'-6"	11'-6"	8'-6"	8'-6"	8'-3"	9'-9"
2	12'-9"	13'-0"	9'-9"	9'-6"	9'-6"	10'-9"
2½	14'-0"	14'-3"	10'-9"	10'-6"	10'-6"	11'-9"
3	15'-6"	15'-9"	11'-9"	11'-6"	11'-3"	13'-0"
3½	16'-6"	16'-9"	12'-6"	12'-3"	12'-0"	14'-0"
4	17'-3"	17'-9"	13'-6"	13'-0"	13'-0"	14'-9"
5	19'-0"	19'-6"	15'-0"	14'-6"	14'-3"	16'-0"
6	20'-9"	21'-3"	16'-3"	15'-9"	15'-6"	17'-3"
8	23'-3"	24'-3"				
10	25'-9"	26'-6"				
12	27'-6"	28'-6"				

**Maximum unsupported or unbraced lengths (L) are based on water-filled pipes with period (T_a) equal to 0.05 Sec. Where*

$$L^2 = 1.125 p T_a \sqrt{\frac{EIg}{w}}$$

1 inch = 25mm
1 foot = 0.3m

See Figure 10-5 for Notations

Figure 10-7. Maximum span for rigid pipe fixed-fixed

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The design criteria presented herein apply to either cantilever or singly guyed stacks. All stacks designed under the provisions of this paragraph must have a constant moment of inertia. Stacks having a slightly varying moment of inertia will be treated as having a uniform moment of inertia with a value equal to the average moment of inertia.

(1). Stacks on buildings. Stacks that extend more than 15 feet (4.6m) above a rigid attachment to the building will be designed according to the criteria for cantilever stacks prescribed below. Stacks that extend less than 15 feet (4.6m) will be designed for the equivalent static lateral force prescribed in Section 6.3.11 of FEMA 302.

(a) Cantilever stacks.

1. The fundamental period of the stack will be determined from the period coefficient (i.e., C = 0.0909) provided in Figure 10-9, unless actually computed.

2. The dynamic response may be calculated from the appropriate base shear equations for the Equivalent Lateral Force procedure prescribed in Chapter 3.

(b) Guyed stacks. The analysis of guyed stacks depends on the relative rigidities of the cantilever resistance and the guy wire support system. If the wires are very flexible, the stack will respond in a manner similar to the higher modes of vibration of a cantilever, with periods and mode shapes similar to those shown on Figure 10-9. The fundamental period of vibration of the guyed system will be somewhere between the values for the fundamental and the appropriate higher mode of a similar cantilever stack. An illustration for a single-guyed stack is shown in Figure 10-10. The design of guyed stacks is beyond the scope of this manual.

1. Stacks on the ground. Where stack foundations are in contact with the ground and the stack is not supported by the building, the stack will be considered to be a non-building structure governed by other agency documents.

2. Anchor bolts. Anchor bolts for moment-resisting stack bases should be as long as possible. A great deal more strain energy can be absorbed with long anchor bolts than with short ones. The use of these long anchor bolts has been demonstrated to give stacks better earthquake performance. In some cases, a pipe sleeve is used in the upper portion of the anchor bolt to ensure a length of unbonded bolt for strain energy absorption. When this type of detail is used, provisions will be made for shear transfer (e.g., shear keys). The use of two nuts on anchor bolts is also recommended to provide an additional factor of safety.

g. Bridges, Cranes, and Monorails. In addition to the normal horizontal loads prescribed by the various other applicable government criteria, the design of bridge cranes and monorails will also include an investigation of lateral seismic forces and deformations as set forth in this paragraph.

(1) Equivalent static force. A lateral force equal to $0.5S_{DS}\ a_P$ times the weight of the bridge crane of monorail will be statically applied at the

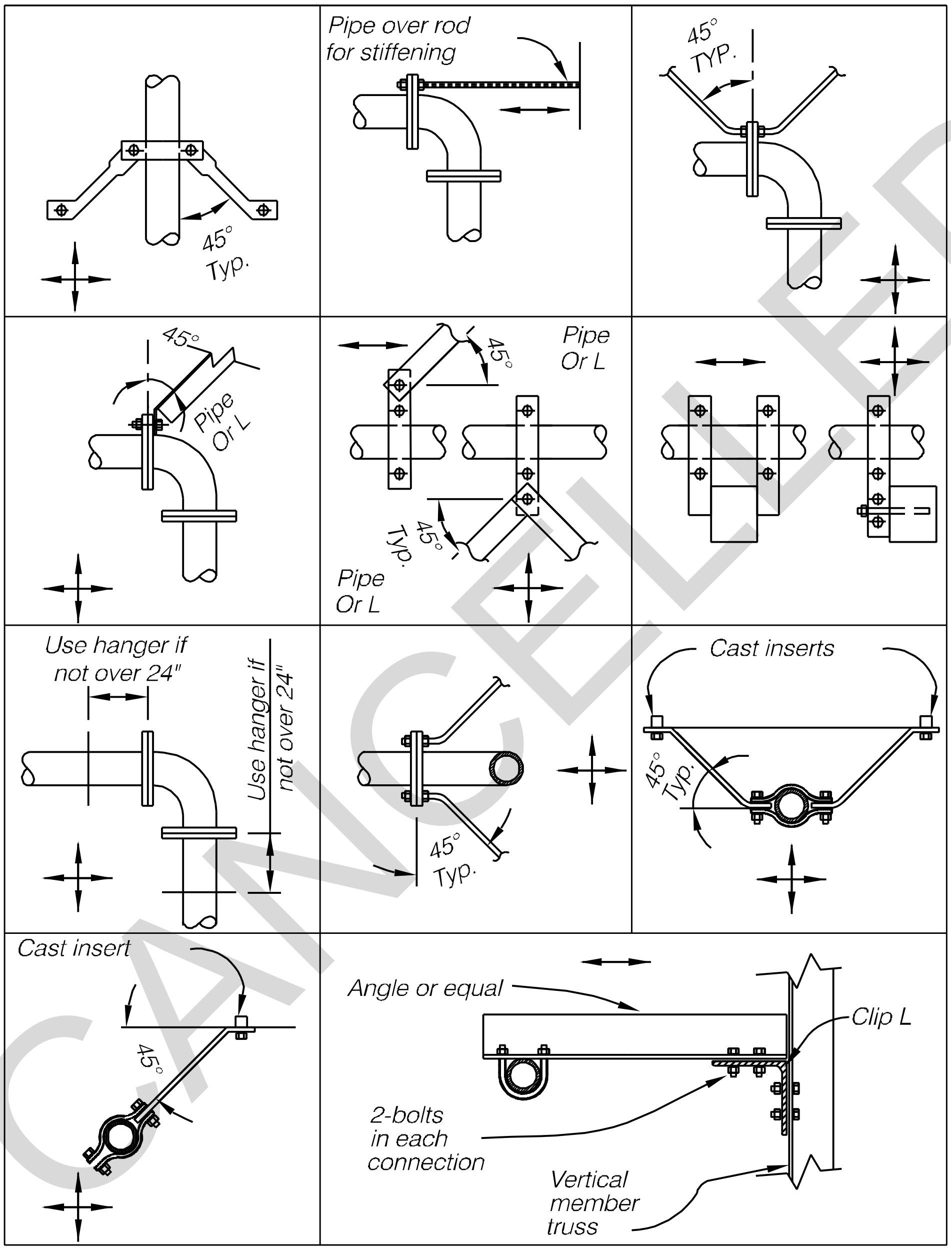
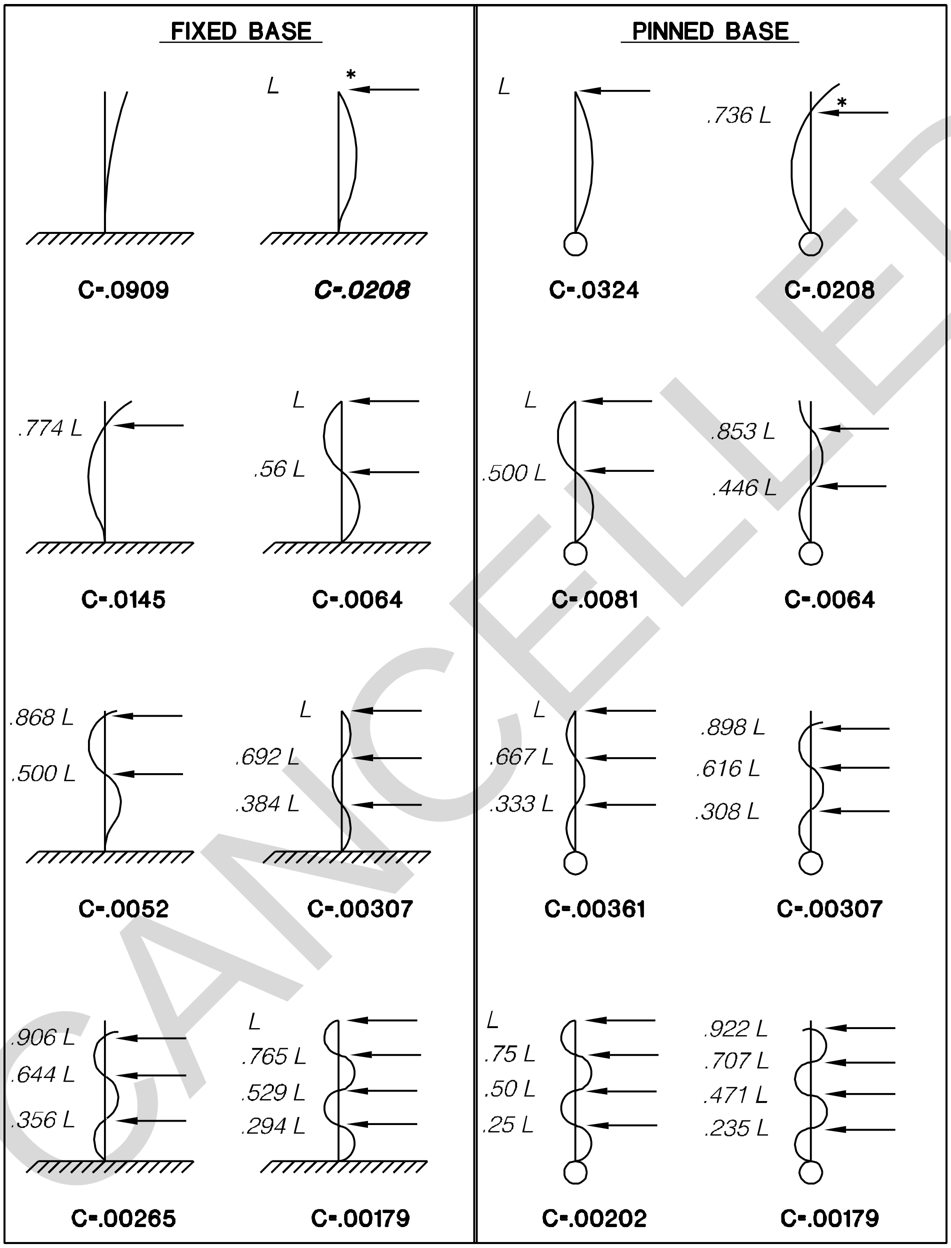


Figure 10-8 Acceptable seismic details for sway bracing.



$$T_a = C\sqrt{\frac{wL^4}{EI}}$$

T_a = Fundament period (sec)
w = Weight per unit length of beam (lb/in) (N/mm)
L = Total beam length (in) (mm)
I = Moment of inertia (in^4) (mm^4)
E = Modulus of elasticity (psi) (MPa)
C = Period constant

Figure 10-9 Period coefficients for uniform beams.

center of gravity of the equipment. This equivalent static force will be considered to be applied in any direction. a_P will be equal to 1.50.

(2) Weight of equipment. The weight of such equipment, W_P, need not include any live load, and the equivalent static force so computed will be assumed to act non-concurrently with other prescribed non-seismic horizontal forces when considering the design of the crane and monorails. When considering the design of the building, the weight of the equipment will be included with the weight of the building.

h. Elevators. Power-cable-driven elevators and hydraulic elevators with lifts over 5 feet (1.5m) will be designed for lateral force set forth in this chapter.

(1) Elements of the elevator support system. All elements that are part of the elevator support system, such as the car and counterweight frames, guide rails, supporting brackets and framing, driving machinery, operating devices, and control equipment, will be investigated for the prescribed lateral seismic forces (see Figure 10-11).

(2) Equivalent static forces. The lateral seismic forces will conform to the applicable provisions of Paragraphs 10-3b and 10-3c.

(a) The car and counterweight frames, roller guide assembly, retainer plates, guide rails, and supporting brackets and framing will be designed in accordance with Section 6.3.2 of FEMA 302. The lateral forces acting on the guide rails will be assumed to be distributed one-third to the top guide rollers and two-thirds to the bottom guide rollers of elevator cars and counterweights. The elevator car and/or counterweight will be assumed to be located at its most adverse position in relation to the guide fails and support brackets. Horizontal deflections of guide rails will not exceed ½ inch (12.7mm) between supports, and horizontal deflections of the brackets will not exceed ¼ inch (6.4mm).

1. In structures conforming to Seismic Design Categories D, E, and F, a retainer plate (auxiliary guide plate) will be provided at top and bottom of both car and counterweight. The clearances between the machined faces of the rail and the retainer plate will not be more than 3/16 inch (4.8mm), and the engagement of the rail will not be less than the dimension of the machined side face of the rail. When a car safety device attached to the lower members of the car frame complies with the lateral restraint requirements, a retainer plate is not required for the bottom of the car.

2. For Seismic Design Categories D, E, and F, the maximum spacing of the counterweight rail tie brackets tied to the building structure will not exceed 16 feet (4.9m). An intermediate spreader bracket, not required to be tied to the building structure, will be provided for tie brackets spaced greater than 10 feet (3.0m), and two intermediate spreader brackets are required for tie brackets spaced greater than 14 feet (4.3m).

(b) Machinery and equipment will be designed for a_P = 1.0 in Equation 7-1, when rigid and rigidly attached. Non-rigid or flexibly mounted equipment will be designed with $a_P = 2.5$.

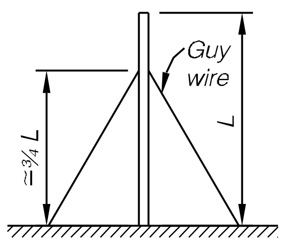
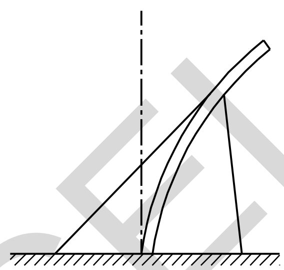
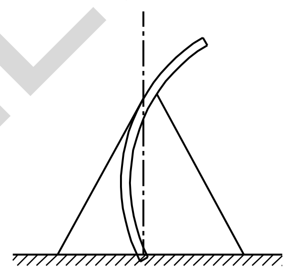
DESCRIPTION	DEFLECTED SHAPE	
	FLEXIBLE WIRE	RIGID WIRE
		

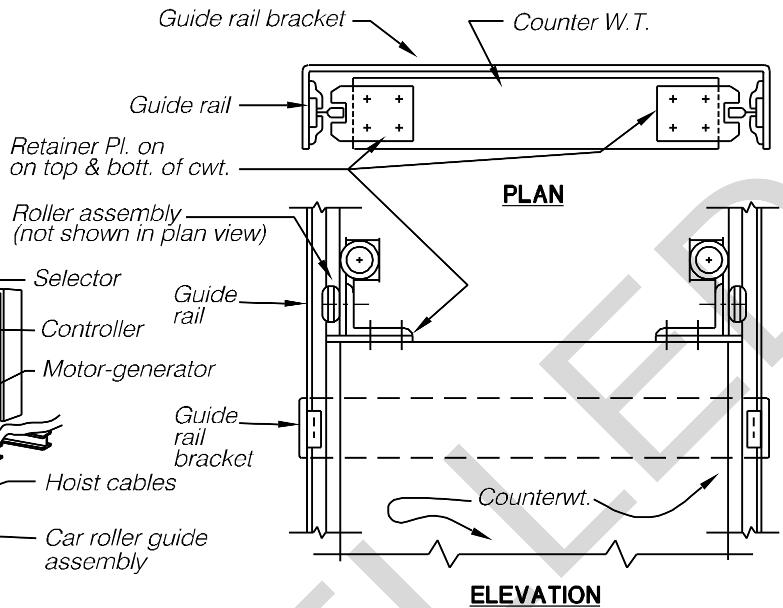
Figure 10-10 Single guyed-stack.

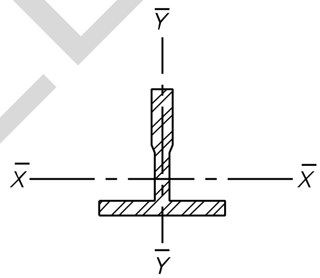
i. Typical Details for Securing Equipment. See Figures 10-12 and 10-13 for examples of seismic restraints for equipment.

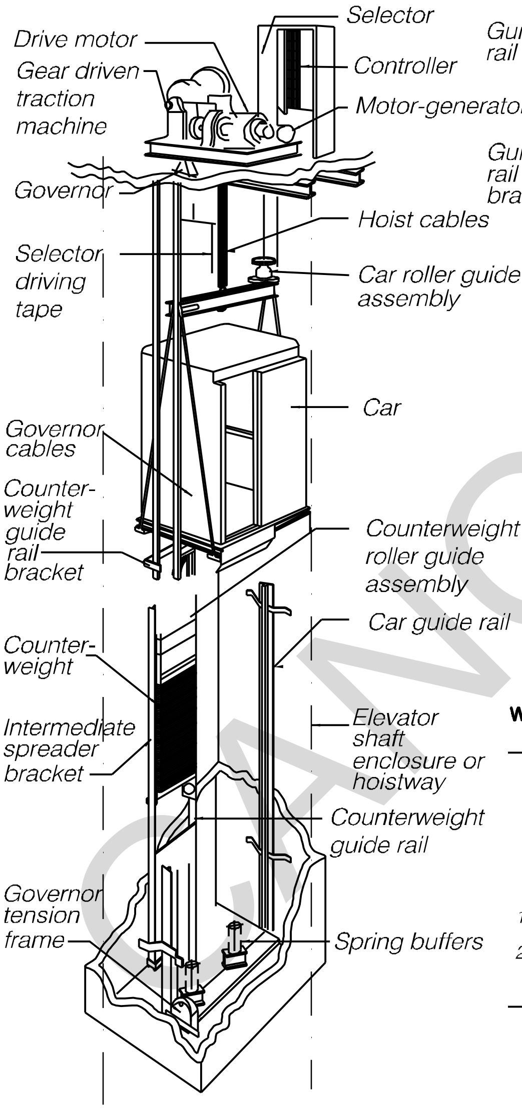
10-4. Acceptance Criteria.

a. Performance Objective 1A. The acceptance criteria for nonstructural components in Performance Objective 1A is conformance with the requirements of Chapter 6 of FEMA 302, with the importance factor, I_P, equal to 1.0, and as modified by this document. The required seismic forces are represented by Equation, 10-1, 10-2, and 10-3.

b. Enhanced Performance Objective. Performance Objective 1A is the minimum requirement for all nonstructural components. Buildings that are designed for enhanced performance objective, shall identify critical nonstructural components that require enhanced performance. The enhanced performance shall be achieved by compliance with the criteria prescribed in this chapter, with Ip selected from Paragraph 10-1d and with S_{DS} from Ground Motion A or B, as appropriate.







Section Properties of Elevator Guide Rails

Nom. Wt. per Foot	A	$\overline{y}$	I_x	S_x	r_x	I_y	S_y	r_y
8	2.40	0.81	1.40	0.86	0.78	1.28	0.73	0.74
11	3.24	1.12	4.29	1.80	1.15	2.67	1.19	0.90
12	3.51	1.08	4.49	1.85	1.13	3.61	1.45	1.01
15	4.41	0.97	4.78	1.89	1.04	5.51	2.21	1.12
18.5	5.44	1.26	9.67	3.23	1.33	7.45	2.71	1.17
22.5	6.73	1.38	11.17	4.20	1.28	8.64	3.10	1.13
30	8.93	1.74	24.00	7.37	1.64	11.08	4.03	1.11

A = Area (in.^{2})
$\overline{y}$ = Distance from back of rail to axis x-x (in.)
I_x = Moment of inertia about axis x-x (in.^{4})
S_x = Section modulus about axis x-x (in.^{3})
r_x = Radius of gyration about axis x-x (in.)
I_y = Moment of inertia about axis y-y (in.^{4})
S_y = Section modulus about axis y-y (in.^{3})
r_y = Radius of gyration about axis y-y (in.)

1 inch = 25mm

TRACTION TYPE ELEVATOR

Figure 10-11 Elevator Details

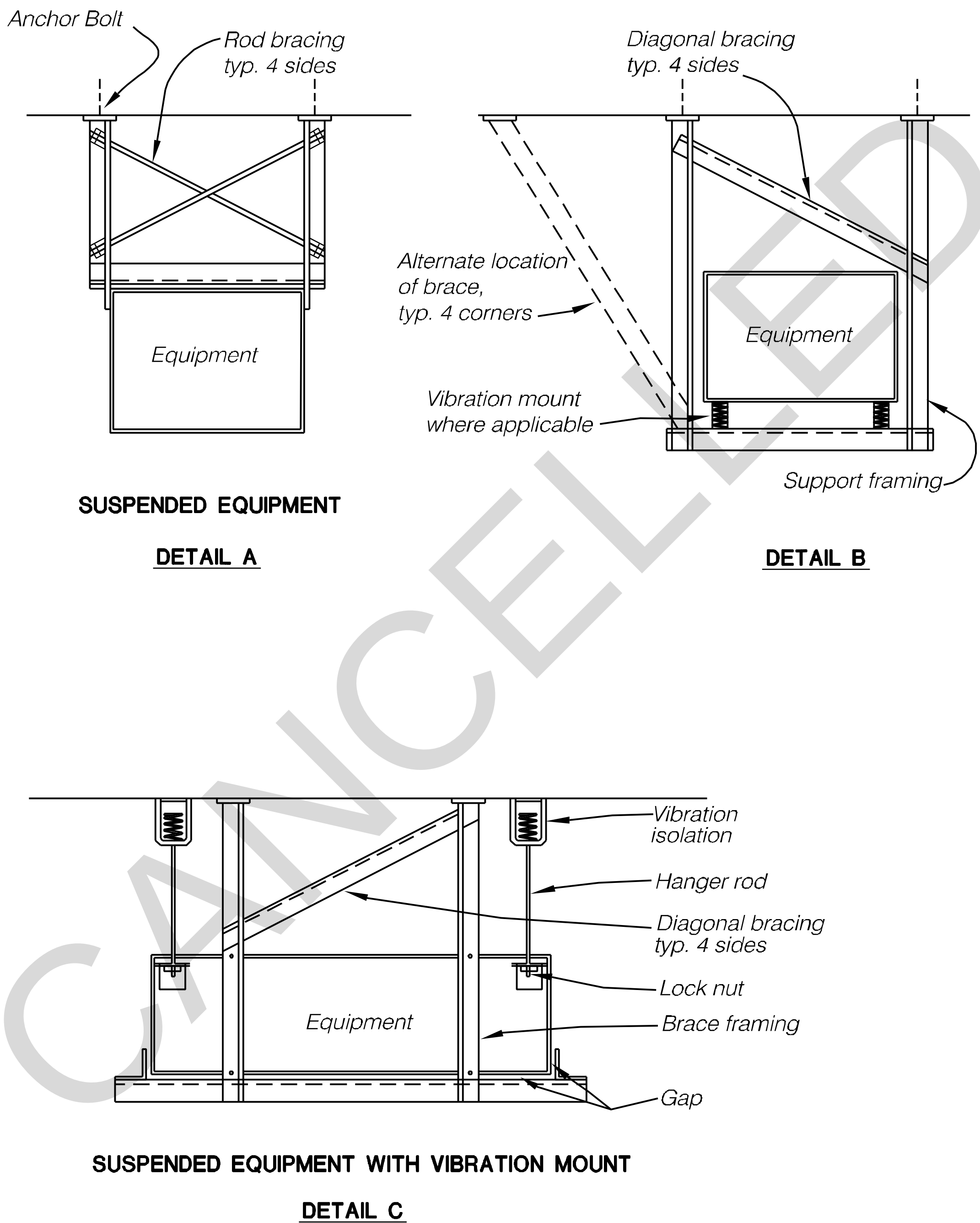


Figure 10-12 Typical seismic restraint of hanging equipment.

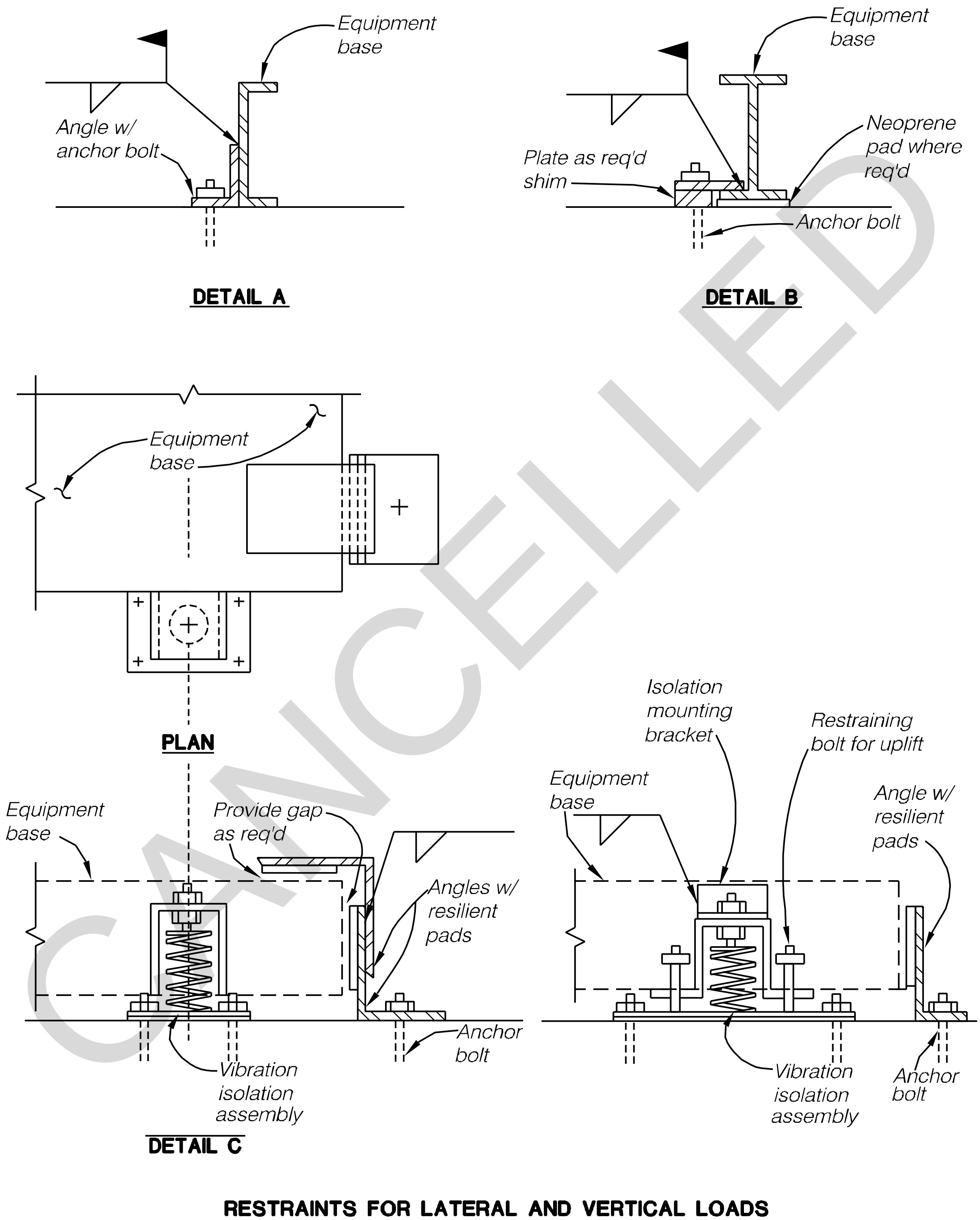


Figure 10-13 Typical seismic restraint of floor-mounted equipment.

APPENDIX A
REFERENCES

A-1. Government Publications.

a. Federal Energy Management Agency
FEMA 302, December 1997
NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures.

FEMA 303, December 1997
Commentary on the NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures.

A-2. Nongovernment Publications.

a. American Concrete Institute
Box 19150
Redford Station, Detroit, MI 48219

ACI 318-95, Building Code Requirements for Reinforced Concrete

ACI 530-95/ASCE 5-95/TMS 402-95, Building Code Requirements for Masonry Structures.

b. American Institutes of Steel Construction, Inc.
One Wacker Drive, Suite 3100,
Chicago, IL 60601-2001

Seismic Provisions for Structural Steel Buildings, April, 1997, Part1

Load and Resistance Factor Design for Structural Steel Buildings (LRFD), 1994

c. American Society of Civil Engineers
1801 Alexander Bell Drive
Reston, VA, 20191-4400

ANSI/ASCE 7-95, Minimum Design Loads for Buildings and Other Structures.

ASCE 16-95, Load and Resistance Factor Standard for Engineered Wood Construction.

d. American Society for Testing and Materials
1916 Race Street
Philadelphia, PA, 19103

A673-95, Standard Methods for Notched Base Import Testing of Metallic Materials.

e. International Conference of Building Officials
5360 Workman Mill Road
Whittier, CA, 90601-2298

Uniform Building Code, 1997
Volume 2, Structural Engineering Design Provisions

f. Steel Deck Institute
PO Box 9506
Canton, OH, 44711

DDMO2, Diaphragm Design Manual, 1995

APPENDIX B
SYMBOLS AND NOTATIONS

A_S Area of nonprestressed tension reinforcement (in.^{2} or mm^2).

A'_S Area of compression reinforcement (in.^{2} or mm^2).

Ag Gross cross-sectional area (in.^{2}or mm^2).

A_w Area of the diaphragm web in Equations 5-10 and 5-11 (in.^{2} or mm^2).

a Angle of the diagonal members with the horizontal plane in the special segment of a truss moment frame (degrees or radians).

a_w Spacing of marginal welds in steel deck diaphragm in Equation 5-14 (ft or m).

b_f Flange width of structural steel beam (in. or mm).

b/t Ratio of flange width to flange thickness for structural steel beams.

C_d The deflection amplification factor as given in Table 7-2.

C_s The seismic response coefficient (dimensionless) determined in paragraph 3-2c.

C_{sm} The modal seismic response coefficient (dimensionless) determined in paragraph 3-2c.

C_{vxm} The vertical distribution factor in the mth mode.

cm Center of mass.

cr Center of rigidity.

d Distance of a vertical resisting element from the center of rigidity in a torsional analyses (in. or mm).

EI Flexural elastic stiffness of the chord member of the special segment of a truss moment frame (kips - in.^{2} or (N-mm^2).

F Flexibility factor in paragraph 5-9b(4)(b) (micro in./ft, or mm/m).

F_a Acceleration-based site coefficient (at 0.3 sec period).

F_{px} The resultant of the diaphragm load acting through the center of mass (kips or K-N).

F_t The force on an individual vertical resisting element due to a torsional moment M_t (kips or K-N).

F_v Velocity-based site coefficient (at 1.0 sec period).

F_{xm} The portion of the seismic base shear, V_m, induced at level x as determined in paragraph 3-2c.

F_y Specified yield strength of structural steel (psi or MPa).

F_{ye} Expected yield strength of structural steel (psi or MPa).

f_a Axial stress in a structural member (psi or MPa).

f_{ae} Expected vertical compressive stress (psi or MPa).

f_{me} Expected compressive strength of masonry as determined in Section 7.3.2.1 (psi or MPa).

G Shear modulus (lb/in.^{2} or MPa).

H Thickness of soil (ft. or m).

h_{eff} Height to resultant of lateral force for wall or pier (in. or mm).

K Stiffness of a vertical resisting element in paragraph 5-9b(3)(d) (kips/in. or K-N/mm).

L_I Distance from adjacent vertical resisting element (i.e. such as a shear wall) and the point at which the deflection is to be determined (ft or m).

L Length of wall (in. or mm).

L single span length of a diaphragm in Equation 5-10 (ft or m).

L Span length of truss in a truss moment frame (in. or mm).

L_p Limiting laterally unbraced length for full plastic flexural strength, uniform moment case (in. or mm).

L_s 0.9 times the length of the special segment in a truss moment frame (in. or mm).

l'_w Effective length of seam weld in steel deck diaphragm (in. or mm).

M_a The accidental torsional moment in paragraph 5-9b(3)(e) (in.-lb. or N-mm).

M_{nc} Nonlinear flexural strength of the chord member of the special segment of a truss moment frame (in.-lb. or N-mm).

M_p Nominal plastic flexural strength of structural steel sections (in.-lb. or N-mm).

M_{RS} Reduced flexural strength of a link beam in an eccentric braced frame when subjected to axial stress combined with flexure (in.-lb or N-mm).

M_s Flexural strength of the link beam in an eccentric braced frame (in.-lb. or N-mm).

M_t The torsional moment resulting from the location of the building masses, paragraph 5-9b(3)(d) (in-lb or N-mm).

M_w Moment earthquake magnitude.

M_u Required flexural strength due to factored loads (in.-lb. or N-mm) paragraph 5-5b(2)(a).

m Modification factor used in the acceptance criteria of deformation-controlled components or elements, indicating the available ductility of a component action.

N Average field standard penetration test for the top 100 ft (30m); see Table 3-1.

P Axial force in a member (lbs. or N).

PI Plasticity index, ASTM D4318-93.

P_{nc} Nominal axial compression strength of the diagonal members of the special segment of a truss moment frame (kips or K-N).

P_{nt} Nominal axial torsion strength of the diagonal members of the special segment of a truss moment frame (kips or K-N).

P_{uc} Required axial strength for a column in compression (psi or MPa).

q_{avc} Average shear in diaphragm (lbs./ft or N/m).

R The response modification coefficient as given in Table 7-2.

Ry Ratio of the expected yield strength, Fye,, to the specified minimum yield strength, Fy. Ry to be taken as 1.5 for ASTM A36 steel rolled shapes and bars and 1.3 for ASTM A572, Grade42.

S Section modulus based on net cross sectional area of a wall (in.^{3} or mm^3).

S_1 The mapped maximum considered earthquake, 5% damped, spectral response acceleration at a period of 1 second as defined in paragraph 3-1d (g).

Sa The design spectral response acceleration as determined by Equation 3-10, 3-11, or 3-12 (g).

S_{am} The design response acceleration at period Tm, (sec) (g).

S_{D1} The design, 5% damped, spectral response acceleration at a period of one second as defined in paragraph 3-2b (g).

S_{DS} The design, 5% damped, spectral response acceleration at short periods as defined in paragraph 3-26 (g).

S_{M1} The maximum considered earthquake, 5% damped, spectral response acceleration at a period of 1 second adjusted for *site class* effects as defined in paragraph 3-1d (g).

S_{MS} The maximum considered earthquake, 5% damped, spectral response acceleration at short periods adjusted for *site class* effects as defined in paragraph 3-1d (g).

S_S The mapped maximum considered earthquake, 5% damped, spectral response acceleration at short periods as defined in paragraph 3-1d (g).

$\bar{s}_u$ Average undrained shear strength in top 100 ft (30.5); see Table 3-1, ASTM D2166-9 or ASTM D2850-87 (psf or kPa).

T The fundamental period (sec) of the building as determined in paragraph 3-2c.

T_0 Period at which the response spectrum shifts from constant response acceleration to constant response velocity as given by Eq. 3-13.

T_m The modal period of vibration (sec) of the m^{th} mode of the building as determined in paragraph 3-2c.

t Thickness of the web of a diaphragm in Equation 5-12 and 5-13 (in. or mm).

t_1 Thickness of flat sheet element in a steel deck diaphragm (in or mm).

t_2^1 Effective thickness of fluted element in a steel deck diaphragm (in. or mm).

t_w Thickness of wall web (in. or mm).

V_m The portion of the base shear contributed by the m^{th} mode (kips or K-N).

V_{me} The expected vertical nominal shear strength in a non-special segment of a truss moment frame (kips or K-N).

V_s Shear strength of the link beam in an eccentric braced frame (kips or K-N).

V_s Shear strength provided by shear reinforcement (lb or N).

V_u Required shear strength (lb or N) due to factored loads in paragraph 5-5b(2)(c).

$\bar{v}_s$ Average shear wave velocity in top 100 ft (30 m); see Table 3-1 (fps or mps).

W Uniformly distributed load on a diaphragm in Equation 5-10 (lbs. or N).

W_m The effective modal gravity load determined in accordance with Equation 3-15 (kip or kN).

w Moisture content (in percent), ASTM D2216-92.

w Unit weight of concrete in Equation 5-13 (lbs/cu ft or N/m^3).

w_i, w_x The portion of the total gravity load, W, located or assigned to Level i or x (kip or kN).

Z Plastic section modules of a member (in^3 or mm^3).

α A form factor in Equation 5-10 and 5-12.

Δ The design story drift as determined in Section 5.3.8.1 of NEHRP 97 (in. or mm).

Δ_d Diaphragm displacement, in a one-story building with a flexible diaphragm, due to a lateral load equal to the weight of the building (in. or mm).

Δ_w In-plane wall displacement in a one-story building with a flexible diaphragm, due to a lateral load equal to the weight of the building (in or mm).

Δ_w Web component of diaphragm deflection (in. or mm).

δ_{xem} The modal of Level x at the center of the mass at and above Level x determined by an elastic analysis, paragraph 3-2c (in or mm).

δ_{xm}, δ_{xm} The modal deflection of Level x at the center of the mass at and above Level x as determined by Equation 3-21 (in or mm).

ϕ Resistance factor.

ϕ_{im} The displacement amplitude at the i^{th} level of the building for the fixed base condition when vibrating in its m^{th} mode, Paragraph 3-2c.

ϕ_{xm} The displacement amplitude at the x^{th} level of the structure when vibrating in its m^{th} model

ρ_g Ratio of area of total wall or pier reinforcement to area of gross section.

ΣM^*_{pb} Moment at the intersection of the beam and column centerlines determined by projecting the beam maximum developed moment from the column face in paragraph 5-5 (c)(7) (in-lb or N-mm).

ΣM^*_{pc} Moment at intersection of the beam and column centerline determined by projecting the sum of the nominal column plastic moment strengths, induced by the axial stress, Puc/Ag, from the top and bottom of the beam moment connection in paragraph 5-5(c)(7) (in-lb or N-mm).

APPENDIX C
GLOSSARY

Acceptance Criteria: Permissible values of such properties as drift, component strength demand, and inelastic deformation used to determine the acceptability of a component's projected behavior at a given Performance Level.

Action: Sometimes called a generalized force, most commonly a single force or moment. However, an action may also be a combination of forces and moments, a distributed loading, or any combination of forces and moments. Actions always produce or cause displacements or deformations; for example, a bending moment action causes flexural deformation in a beam; an axial force action in a column causes axial deformation in the column; and a torsional moment action on a building causes torsional deformations (displacements) in the building.

Addition: An increase in building area, aggregate floor area, height, or number of stories of a structure.

Alteration: Any construction or renovation to an existing structure other than an addition.

Appendage: An architectural component such as a canopy, marquee, ornamental balcony, or statuary.

Approval: The written acceptance by the regulatory agency of documentation that establishes the qualification of a material, system, component, procedure, or person to fulfill the requirements of these provisions for the intended use.

Architectural Component Support: Those structural members or assemblies of members, including braces, frames, struts and attachments, that transmit all loads and forces between architectural systems, components, or elements and the structure.

Attachments: Means by which components and their supports are secured or connected to the seismic-force-resisting system of the structure. Such attachments include anchor bolts, welded connections, and mechanical fasteners.

Base: The level at which the horizontal seismic ground motions are considered to be imparted to the structure.

Base Shear: Total design lateral force or shear at the base.

Basement: A basement is any level below the first story.

Boundary Elements: Diaphragm and shear wall boundary members to which sheathing transfers forces. Boundary members include chords and drag struts at diaphragm and shear wall perimeters, interior openings, discontinuities, and reentrant corners.

Boundary Members: Portions along wall and diaphragm edges strengthened by longitudinal and transverse reinforcement and/or structural steel members.

Braced Frames: An essentially vertical truss, or its equivalent, of the concentric or eccentric type that is provided in a building frame system or dual frame system to resist in-plane lateral loads.

> **Concentrically Braced Frame (CBF):** A braced frame in which the members are subjected primarily to axial forces.
>
> **Eccentrically Braced Frame (EBF):** A diagonally braced frame in which at least one end of each brace frames into a beam a short distance from a beam-column joint or from another diagonal brace.
>
> **Ordinary Concentrically Braced Frame (OCBF):** A steel concentrically braced frame in which members and connections are designed in accordance with the AISC Seismic Provisions.
>
> **Special Concentrically Braced Frame (SCBF):** A steel or composite steel and concrete concentrically braced frame in which members and connections are designed for ductile behavior. Special concentrically braced frames shall conform to Sec. 8.2.1 or NEHRP'97.
>
> **V-Braced Frame:** A concentric braced frame (CBF) in which a pair of diagonal braces located either above or below a beam is connected to a single point within the clear beam span. Where the diagonal braces are below the beam, the system also is referred to as an "inverted V-brace frame," or "chevron bracing."
>
> **X-Braced Frame:** A concentric braced frame (CBF) in which a pair of diagonal braces crosses near the mid-length of the braces.
>
> **Y-Braced Frame:** An eccentric braced frame (EBF) in which the stem of the Y is the link of the EBF system.

Brittle: Systems, members, materials, and connections that do not exhibit significant energy dissipation capacity in the inelastic range.

Building: Any structure whose use could include shelter of human occupants.

Building Performance Level: A limiting damage state, considering structural and nonstructural building components, used in the definition of Performance Objectives.

Capacity: The permissible strength or deformation for a component action.

Component: A part or element of an architectural, electrical, mechanical, or structural system.

> **Component, Equipment:** A mechanical or electrical component or element that is part of a mechanical and/or electrical system within or without a building system.
>
> **Component, Flexible:** Component, including its attachments, having a fundamental period greater than 0.06 sec.
>
> **Component, Rigid:** Component, including its attachments, having a fundamental period less than or equal to 0.06 sec.

Concrete:

> **Plain Concrete:** Concrete that is either unreinforced or contains less reinforcement than the minimum amount specified in ACI-318 for reinforced concrete.
>
> **Reinforced Concrete:** Concrete reinforced with no less than the minimum amount required by ACI-318, prestressed or nonprestressed, and designed on the assumption that the two materials act together in resisting forces.

Confined Region: That portion of a reinforced concrete component in which the concrete is confined by closely spaced special transverse reinforcement restraining the concrete in directions perpendicular to the applied stress.

Construction Documents: The written, graphic, electronic, and pictorial documents describing the design, locations, and physical characteristics of the project.

Control Node: The node in the mathematical model of a building used to characterize mass and earthquake displacement.

Coupling Beam: A beam that is used to connect adjacent concrete wall piers to make them act together as a unit to resist lateral loads.

Critical Action: That component action that reaches its elastic limit at the lowest level of lateral deflection, or loading, for the structure.

Damping: The exponential decay of the free vibration of an elastic single-degree-of-freedom system due to internal energy dissipation. Usually expressed as a percentage of critical damping.

> **Critical Damping:** The amount of energy dissipation required to restrain a displaced elastic single-degree-of-freedom system from vibration beyond the initial "at rest" position.

Demand: The amount of force or deformation imposed on an element or component.

Design Earthquake Ground Motion: The earthquake effects that buildings and structures are specifically proportioned to resist as defined in Sec. 4.1 of NEHRP'97.

Design Earthquake: The earthquake for use with Chapter 10 that is two-thirds the maximum considered earthquake.

Diaphragm: A horizontal or nearly horizontal system acting to transfer lateral forces to the vertical resisting elements. Diaphragms are classified as either flexible or rigid according to the requirement of Sec. 12.3.4.2 of NEHRP'97.

Diaphragm, Blocked: A diaphragm in which all sheathing edges not occurring on a framing member are supported on a fastened to blocking.

Diaphragm Boundary: A location where shear is transferred into or out of the diaphragm sheathing. Transfer is either to a boundary element or to another force-resisting element.

Diaphragm Chord: A diaphragm boundary element perpendicular to the applied load that is assumed to take axial stresses due to the diaphragm moment in a manner analogous to the flanges of a beam. Also applies to shear walls.

Diaphragm Collector: A diaphragm component provided to transfer lateral force from the diaphragm to vertical elements of the later-force-resisting system or to other portions of the diaphragm.

Displacement:

> **Design Displacement:** The design earthquake lateral displacement, excluding additional displacement due

to actual and accidental torsion, required for design of the isolation system.

Total Design Displacement: The design earthquake lateral displacement, including additional displacement

Displacement continued:

due to actual and accidental torsion, required for design of the isolation system or an element thereof.

Total Maximum Displacement: The maximum capable earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for verification of the stability of the isolation system or elements thereof, design of building separations, and vertical load testing of isolator unit prototypes.

Displacement Restraint System: A collection of structural elements that limits lateral displacement of seismically isolated structures due to the maximum considered earthquake.

Drag Strut (Collector, Tie, Diaphragm Strut): A diaphragm or shear wall boundary element parallel to the applied load that collects and transfers diaphragm shear forces to the vertical-force-resisting elements or distributes forces within the diaphragm or shear wall. A drag strut often is an extension of a boundary element that transfers forces into the diaphragm or shear wall.

Effective Damping: The value of equivalent viscous damping corresponding to energy dissipated during cyclic response of the isolation system.

Effective Stiffness: The value of the lateral forces in the isolation system, or an element thereof, divided by the corresponding lateral displacement.

Element:

Ductile Element: An element capable of sustaining large cyclic deformations beyond the attainment of its nominal strength without any significant loss of strength.

Limited Ductile Element: An element that is capable of sustaining moderate cyclic deformations beyond the attainment of nominal strength without significant loss of strength.

Nonductile Element: An element having a mode of failure that results in an abrupt loss of resistance when the element is deformed beyond the deformation corresponding to the development of its nominal strength. Nonductile elements cannot reliably sustain significant deformation beyond that attained at their nominal strength.

Equipment Support: Those structural members or assemblies of members or manufactured elements, including braces, frames, legs, lugs, snuggers, hangers or saddles, that transmit gravity load and operating load between the equipment and the structure.

Essential Facility: A facility or structure required for post-earthquake recovery.

Factored Resistance ($\lambda\phi D$): Reference resistance multiplied by the time effect and resistance factors. This value must be adjusted for other factors such as size effects, moisture conditions, and other end-use factors.

Flexible Diaphragm: A diaphragm with stiffness characteristics indicated in paragraph 5-9b(1).

Flexible Equipment Connections: Those connections between equipment components that permit rotational and/or translational movement without degradation of performance. Examples include universal joints, bellows expansion joints, and flexible metal hose.

Foundations:

Allowable Bearing Capacity: Foundation load or stress commonly used in working-stress design (often controlled by long-term settlement rather than soil strength).

Deep Foundation: Piles or piers.

Differential Compaction: An earthquake-induced process in which loose or soft soils become more compact and settle in a nonuniform manner across a site.

Footing: A structural component transferring the weight of a building to the foundation soils and resisting lateral loads.

Foundation Soils: Soils supporting the foundation system and resisting vertical and lateral loads.

Foundation Springs: Method of modeling to incorporate load-deformation characteristics of foundation soils.

Foundation System: Structural components (footings, piles).

Landslide: A down-slope mass movement of earth resulting from any cause.

Liquefaction: An earthquake-induced process in which saturated, loose, granular soils lose a substantial amount of shear strength as a result of increase in porewater pressure during earthquake shaking.

Foundations continued:

Pier: Similar to pile; usually constructed of concrete and cast in place.

Pile: A deep structural component transferring the weight of a building to the foundation soils and resisting vertical and lateral loads; constructed of concrete, steel, or wood; usually driven into soft or loose soils.

Retaining Wall: A free-standing wall that has soil on one side.

Shallow Foundation: Isolated or continuous spread footings or mats.

SPT N-Values: Using a standard penetration test (ASTM Test D1586), the number of blows of a 140-pound (623N) hammer falling 30 inches (0.76m) required to drive a standard 2-inch-(50mm) diameter sampler a distance of 12 inches (0.30m).

Ultimate Bearing Capacity: Maximum possible foundation load or stress (strength); increase in deformation or strain results in no increase in load or stress.

Frame Systems:

Building Frame System: A structural system with an essentially complete space frame system providing support for vertical loads. Seismic-force resistance is provided by shear walls or braced frames.

Dual Frame System: A structural system with an essentially complete space frame system providing support for vertical loads. Seismic force resistance is provided by moment resisting frames and shear walls or braced frames as prescribed in Sec. 5.2.2.1 of NEHRP'97.

Moment Frame System: A structural system with an essentially complete space frame system providing support for vertical loads, with restrained connections between the beams and columns to permit the frames to resist lateral forces through the flexural rigidity and strength of its members.

Fundamental Period: The first mode period of the building in the direction under consideration.

Grade Plane: A reference place representing the average of finished ground level adjoining the building at all exterior walls. Where the finished ground level slopes away from the exterior walls, the reference plane shall be established by the lowest point within the area between the buildings and the lot line or, where the lot line is more than 6 ft. (1829mm) from the building, between the building and a point 6 ft. (1829mm) from the building.

Hazardous Contents: A material that is highly toxic or potentially explosive and in sufficient quantity to pose a significant life-safety threat to the general public if an uncontrolled release were to occur.

High Temperature Energy Source: A fluid, gas, or vapor whose temperature exceeds 220 degrees F (378 K).

Inspection, Special: The observation of the work by the special inspector to determine compliance with the approved construction documents.

Continuous Special Inspection: The full-time observation of the work by an approved special inspector who is present in the area where work is being performed.

Periodic Special Inspection: The part-time or intermittent observation of the work by an approved special inspector who is present in the area where work has been or is being performed.

Inspector, Special (who shall be identified as the Owner's Inspector): A person approved by the regulatory agency as being qualified to perform special inspection required by the approved quality assurance plan. The quality assurance personnel of a fabricator may be approved by the regulatory agency as a special inspector.

Inter-Story Drift: The relative horizontal displacement of two adjacent floors in a building. Inter-story drift can also be expressed as a percentage of the story height separating the two adjacent floors.

Inverted Pendulum Type Structure: Structures that have a large portion of their mass concentrated near the top and, thus, have essentially one degree of freedom in horizontal translation. The structures are usually T-shaped with a single column supporting the beams or framing at the top.

Joint: That portion of a column bounded by the highest and lowest surfaces of the other members framing into it.

Lateral-Force-Resisting System: Those elements of the structure that provide its basic lateral strength and stiffness, and without which the structure would be laterally unstable.

Load:

> **Dead Load:** The gravity load due to the weight of all permanent structural and nonstructural components of

Load continued:

> a building such as walls, floors, roofs, and the operating weight of fixed service equipment.
>
> **Gravity Load (W):** The total dead load and applicable portions of other loads as defined in Sec. 5.3.2 of NEHRP'97.
>
> **Live Load:** The load superimposed by the use and occupancy of the building not including the wind load, earthquake load, or dead load; see Sec. 5.3.2 of NEHRP'97.
>
> **LRFD (Load and Resistance Factor Design):** A method of proportioning structural components (members, connectors, connecting elements, and assemblages using load and resistance factors that no applicable limit state is exceeded when the structure is subjected to all design load combinations.

Masonry: The assemblage of masonry units, mortar, and possibly grout and/or reinforcement. Types of masonry are classified herein with respect to the type of the masonry units, such as clay-unit masonry, concrete masonry, or hollow-clay tile masonry.

> **Bed Joint:** The horizontal layer of mortar on which a masonry unit is laid.
>
> **Cavity Wall:** A masonry wall with an air space between wythes. Wythes are usually joined by wire reinforcement, or steel ties. Also known as a noncomposite wall.
>
> **Clay-Unit Masonry:** Masonry constructed with solid, cored, or hollow units made of clay. Hollow clay units may be ungrouted, or grouted.
>
> **Clay Tile Masonry:** Masonry constructed with hollow units made of clay tile. Typically, units are laid with cells running horizontally, and are thus ungrouted. In some cases, units are placed with cells running vertically, and may or may not be grouted.

Collar Joint: Vertical longitudinal joint between wythes of masonry or between masonry wythe and back-up construction that may be filled with mortar or grout.

Composite Masonry Wall: Multiwythe masonry wall acting with composite action.

Concrete Masonry: Masonry constructed with solid or hollow units made of concrete. Hollow concrete units may be ungrouted, or grouted.

Head Joint: Vertical mortar joint placed between masonry units in the same wythe.

Hollow Masonry Unit: A masonry unit whose net cross-sectional area in every plane parallel to the bearing surface is less than 75% of the gross cross-sectional area in the same plane.

Infill: A panel of masonry placed within a steel or concrete frame. Panels separated from the surrounding frame by a gap are termed "isolated infills." Panels that are in tight contact with a frame around its full perimeter are termed "shear infills."

In-plane Wall: See shear wall.

Nonbearing Wall: A wall that is designed and detailed so as not to participate in providing support for gravity loads.

Noncomposite Masonry Wall: Multiwythe masonry wall acting without composite action.

Out-of-plane Wall: A wall that resists lateral forces applied normal to its plane.

Parapet: Portions of a wall extending above the roof diaphragm. Parapets can be considered as flanges to roof diaphragms if adequate connections exist or are provided.

Partially Grouted Masonry Wall: A masonry wall containing grout in some of the cells.

Perforated Wall or Infill Panel: A wall or panel not meeting the requirements for a solid wall or infill panel.

Pier: A vertical portion of masonry wall between two horizontally adjacent openings. Piers resist axial stresses from gravity forces, and bending moments from combined gravity and lateral forces.

Reinforced Masonry (RM) Walls: A masonry wall that is reinforced in both the vertical and horizontal directions. Reinforced walls are assumed to resist loads through resistance of the masonry in compression and the reinforcing steel in tension or compression. Reinforced masonry is partially grouted or fully grouted.

Intermediate Reinforced Masonry Walls Shear Walls: To be designed in accordance with Section 11.11.4 of FEMA 302.

Masonry continued:

Special Reinforces Masonry Shear Walls: To be designed in accordance with Section 11.11.5 of FEMA 302.

Running Bond: A pattern of masonry where the head joints are staggered between adjacent courses by more than a third of the length of a masonry unit. Also refers to the placement of masonry units such that head joints in successive courses are horizontally offset at least one-quarter the unit length.

Solid Masonry Unit: A masonry unit whose net cross-sectional area in every plane parallel to the bearing surface is 75% or more of the gross cross-sectional area in the same plane.

Solid Wall or Solid Infill Panel: A wall or infill panel with openings not exceeding 5% of the wall surface area. The maximum length or height of an opening in a solid wall must not exceed 10% of the wall width or story height. Openings in a solid wall or infill panel must be located within the middle 50% of a wall length and story height, and must not be contiguous with adjacent openings.

Stack Bond: In contrast to running bond, usually a placement of units such that the head joints in successive courses are aligned vertically.

Transverse Wall: A wall that is oriented transverse to the in-plane shear walls, and resists lateral forces applied normal to its plane. Also known as an out-of-plane wall.

Unreinforced Masonry (URM) Wall: A masonry wall containing less than the minimum amounts of reinforcement as defined for reinforced masonry (RM) walls. An unreinforced wall is assumed to resist gravity and lateral loads solely through resistance of the masonry materials.

Wythe: A continuous vertical section of a wall, one masonry unit in thickness.

Maximum Considered Earthquake Ground Motion: The most severe earthquake effects considered by this document as defined in Chapter 3.

Moment Frames:

Intermediate Moment Frames (IMF): Moment frames of reinforced concrete or structural steel conforming to detailing requirements that provide capability for moderate inelastic rotation of the beam/column joint.

Ordinary Moment Frames (OMF): Moment frames of reinforced concrete or structural steel conforming to limited detailing requirements that provide capability for nominal inelastic rotation of the beam column joint.

Special Moment Frames (SMF): Moment Frames of reinforced concrete or structural steel conforming to detailing requirements that provide capability for significant inelastic rotation of the beam/column joint.

Eccentric Braced Frame (EBF): A diagonal braced frame in which at least one end of each diagonal bracing member connects to a beam a short distance from either a beam-to-column connection or another brace end.

Nonstructural Performance Level: A limiting damage state for nonstructural building components used to define Performance Objectives.

Partition: A nonstructural interior wall that spans from floor to ceiling, to the floor or roof structure immediately above, or to subsidiary structural members attached to the structure above. A partition may receive lateral support from the floor above, but shall be designed and detailed so as not to provide lateral or vertical support for that floor.

P-Delta Effect: The secondary effect on shears and moments of frame members due to the action of the vertical loads induced by displacement of the building frame resulting from the design loads.

Primary Component: Those components that are required as part of the building's lateral-force-resisting system (as contrasted to secondary components).

Primary Element: An element that is essential to the ability of the structure to resist earthquake-induced deformations.

Quality Assurance Plan: A detailed written procedure that establishes the systems and components subject to special inspection and testing.

Reference Resistance (D): The resistance (force or moment as appropriate) of a member or connection computed at the reference end use conditions.

Registered Design Professional: An architect or engineer, registered or licensed to practice professional architecture or engineering, as defined by the statutory requirements of the professional registrations laws of the state in which the project is to be constructed.

Required Strength: The load effect (force, moment, stress, as appropriate) acting on a component or connection, determined by structural analysis from the factored loads (using the most appropriate critical load combinations).

Resistance Factor: A reduction factor applied to member resistance that accounts for unavoidable deviations of the actual strength from the nominal value, and the manner and consequences of failure.

Rigid Diaphragm: A diaphragm that meets requirements of paragraph 5-9b (1).

Secondary Component: Those components that are not required for lateral force resistance (contrasted to primary components). They may or may not actually resist some lateral forces.

Secondary Element: An element that does not affect the ability of the structure to resist earthquake-induced deformations.

Seismic Demand: Seismic hazard level commonly expressed in the form of a ground shaking response spectrum. It may also include an estimate of permanent ground deformation.

Seismic Design Category: A classification assigned to a structure based on its Seismic Use Group and the severity of the design earthquake ground motion at the site.

Seismic-Force-Resisting System: That part of the structural system that has been considered in the design to provide the required resistance to the shear wall prescribed herein.

Seismic Forces: The assumed forces prescribed herein, related to the response of the structure to earthquake motions, to be used in the design of the structure and its components.

Seismic Isolation and Energy Dissipation:

Design Displacement: The design earthquake displacement of an isolation or energy dissipation system, or elements thereof, excluding additional displacement due to actual and accidental torsion.

Design Earthquake: A user-specified earthquake for the design of an isolated building, having ground shaking criteria described in Chapter 3.

Displacement-Dependent Energy Dissipation Devices: Devices having mechanical properties such that the force in the device is related to the relative displacement in the device.

Displacement Restraint System: Collection of structural components and elements that limit lateral displacement of seismically-isolated buildings during the maximum considered earthquake.

Effective Damping: The value of equivalent viscous damping corresponding to the energy dissipated by the building, or element thereof, during a cycle of response.

Energy Dissipation Device (EDD): Non-gravity-load-supporting element designed to dissipate energy in a stable manner during repeated cycles of earthquake demand.

Energy Dissipation System (EDS): Complete collection of all energy dissipation devices, their supporting framing, and connections.

Isolation Interface: The boundary between the upper portion of the structure (superstructure), which is isolated, and the lower portion of the structure, which moves rigidly with the ground.

Isolation System: The collection of structural elements that includes all individual isolator units, all structural elements that transfer force between elements of the isolation system, and all connections to other structural elements. The isolation system also includes the wind-restraint system, if such a system is used to meet the design requirements of this section.

Isolator Unit: A horizontally flexible and vertically stiff structural element of the isolation system that permits large lateral deformations under seismic load. An isolator unit may be used either as part of or in addition to the weight-supporting system of the building.

Maximum Displacement: The maximum earthquake displacement of an isolation or energy dissipation system, or elements thereof, excluding additional displacement due to actual or accidental torsion.

Tie-Down System: The collection of structural connections, components, and elements that provide restraint against uplift of the structure above the isolation system.

Total Design Displacement: The design displacement of an isolation or energy dissipation

Seismic Isolation and Energy Dissipation continued:

system, or elements thereof, including additional displacement due to actual and accidental torsion.

Velocity-Dependent Energy Dissipation Devices: Devices having mechanical characteristics such that the force in the device is dependent on the relative velocity in the device.

Wind-Restraint System: The collection of structural elements that provides restraint of the seismic-isolated structure for wind loads. The wind-restraint system may be either an integral part of isolator units or a separate device.

Seismic Response Coefficient: Coefficient C_s as determined from Sec. 5.3.2.1 of NEHRP'97.

Seismic Use Group: A classification assigned to a building based on its use as defined in Sec. 1.3 of NEHRP'97.

Shear Panel: A floor, roof, or wall component sheathed to act as a shear wall or diaphragm.

Site Class: A classification assigned to a site based on the types of soils present and their engineering properties as defined in Sec. 4.1.2 of NEHRP'97.

Site Coefficients: The values of F_a and F_v, indicated in Tables 1.4.2.3a and 1.4.2.3b, respectively.

Special Transverse Reinforcement: Reinforcement composed of spirals, closed stirrups, or hoops and supplementary cross-ties provided to restrain the concrete and qualify the portion of the component, where used, as a confined region.

Steel Frame Elements:

Connection: A link between components or elements that transmits actions from one component or element to another component or element. Categorized by type of action (moment, shear, or axial), connection links are frequently nonductile.

Continuity Plates: Column stiffeners at the top and bottom of the panel zone.

Diagonal Bracing: Inclined structural members carrying primarily axial load, employed to enable a structural frame to act as a truss to resist lateral loads.

Dual System: A structural system included in building with the following features:

- An essentially complete space frame provides support for gravity loads.
- Resistance to lateral load is provided by concrete of steel shear walls, steel eccentrically braced frames (EBF), or concentrically braced frames (CBF) along with moment-resisting frames (Special Moment Frames, or Ordinary Moment Frames) that are capable of resisting at least 25% of the lateral loads.

Joint: An area where two or more ends, surfaces, or edges are attached. Categorized by the type of fastener or weld used and the method of force transfer.

Lateral Support Member: A member designed to inhibit lateral buckling or lateral-torsional buckling of a component.

Link: In an EBF, the segment of a beam that extends from column to brace, located between the end of a diagonal brace and a column, or between the ends of two diagonal braces of the EBF> The length of the link is defined as the clear distance between the diagonal brace and the column face, or between the ends of two diagonal braces.

Link Intermediate Web Stiffeners: Vertical web stiffeners placed within the link.

Link rotation angle: The angle of plastic rotation between the link and the beam outside of the link derived using the specified base shear, V.

Panel Zone: The area of a column at the beam-to-column connection delineated by beam and column flanges.

Storage Racks: Include industrial pallet racks, movable shelf racks, and stacker racks made of cold-formed or hot-rolled structural members. Does not include other types of racks such as drive-in and drive-through racks, cantilever racks, portable racks, or racks made of materials other than steel.

Story: The vertical distance from the top to top of two successive tiers of beams or finished floor surfaces; and, for the topmost story, from the top of the floor finish to the top of the ceiling joists or, where there is not a ceiling, to the top of the roof rafters.

Story Above Grade: Any story having its finished floor surface entirely above grade, except that a basement shall be considered as a story above grade where the finished floor surface of the floor above the basement is:

1. More than 6 feet (1829mm) above the grade plane,

2. More than 6 feet (1829mm) above the finished ground level for more than 40 percent of the total building perimeter, or

3. More than 12 feet (3658mm) above the finished ground level at any point.

Story Drift Ratio: The story drift, as determined in Sec. 5.3.7 of NEHRP'97, divided by the story height.

Story Shear: The summation of design lateral forces at levels above the story under consideration.

Strength:

Design Strength: Nominal strength multiplied by a strength reduction factor, ϕ.

Effective Strength: Nominal strength multiplied by a strength increase factor to represent the expected mean strength at the expected deformation value. Includes variability in material strength and such phenomena as strain hardening and plastic section development.

Nominal Strength: Strength of a member or cross section calculated in accordance with the requirements and assumptions of the strength design methods of NEHRP'97 (or the referenced standards) before application of any strength reduction factors.

Required Strength: Strength of a member, cross section, or connection required to resist factored loads or related internal moments and forces in such combinations as stipulated by NEHRP'97.

Structure: That which is built or constructed and limited to buildings or no building structures as defined herein.

Structural Observations: The visual observations performed by the registered design professional in responsible charge (or another registered design professional) to determine that the seismic-force-resisting system is constructed in general conformance with the construction documents.

Structural Performance Level: A limiting structural damage state, used in the definition of Performance Objectives.

Structural Use Panel: A wood-based panel product that meets the requirements of NEHRP'97 and is bonded with a waterproof adhesive. Included under this designation are plywood, oriented strand board, and composite panels.

Subdiaphragm: A portion of a diaphragm used to transfer wall anchorage forces to diaphragm cross ties.

Target Displacement: An estimate of the likely building roof displacement in the design earthquake.

Testing Agency: A company or corporation that provides testing and/or inspection services. The person in responsible charge of the special inspector(s) and the testing services shall be an engineer licensed by the state to practice as such in the applicable discipline.

Tie-Down (Hold-down): A device used to resist uplift of the chords of shear walls. These devices are intended to resist load without significant slip between the device and the shear wall chord or be shown with cyclic testing to not reduce the wall capacity or ductility.

Time Effect Factor (λ): A factor applied to the adjusted resistance to account for effects of duration of load.

Torsional Force Distribution: The distribution of horizontal shear wall through a rigid diaphragm when the center of mass of the structure at the level under consideration does not coincide with the center of rigidity (sometimes referred to as diaphragm rotation).

Toughness: The ability of a material to absorb energy without losing significant strength.

Utility or Service Interface: The connection of the building's mechanical and electrical distribution systems to the utility or service company's distribution system.

Veneers: Facings or ornamentation of brick, concrete, stone, tile, or similar materials attached to a backing.

Wall: A component that has a slope of 60 degrees or greater with the horizontal plane used to enclose or divide space

> **Bearing Wall:** An exterior or interior wall providing support for vertical loads.
>
> **Cripple Wall:** A framed stud wall, less than 8 feet (2400mm) in height, extending from the top of the foundation to the underside of the lowest floor framing. Cripple walls occur in both engineered structures and conventional construction.
>
> **Light-Framed Wall:** A wall with wood or steel studs.
>
> **Light-Framed Wood Shear Wall:** A wall constructed with wood studs and sheathed with material rated for shear resistance.

Wall continued:

> **Nonbearing Wall:** An exterior or interior wall that does not provide support for vertical loads, other than its own weight or as permitted by the building code administered by the regulatory agency.
>
> **Nonstructural Wall:** All walls other then bearing walls or shear walls.
>
> **Shear Wall (Vertical Diaphragm):** A wall designed to resist lateral forces parallel to the plane of the wall (sometimes referred to as a vertical diaphragm).

Wall System, Bearing: A structural system with bearing walls providing support for all or major portions of the vertical loads. Shear walls or braced frames provide seismic-force resistance.

Wind-Restraint System: The collection of structural elements that provides restraint of the seismic-isolated structure for wind loads. The wind-restraint system may be either an integral part of isolator units or a separate device.

Wood and Light Metal Framing:

Aspect Ratio: Ratio of height to width for vertical diaphragms, and width of depth for horizontal diaphragms.

Balloon Framing: Continuous stud framing from sill to roof, with intervening floor joists nailed to studs and supported by a let-in ribbon. (See platform framing.)

Cripple Wall: Short wall between foundation and first floor framing.

Cripple Studs: Short studs between header and top plate at opening in wall framing or studs between base sill and sill of opening.

Decking: Solid sawn lumber or glued laminated decking, nominally two to four inches thick and four inches and wider. Decking may be tongue-and-groove or connected at longitudinal joints with nails or metal clips.

Dimensional Lumber: Lumber from nominal two through four inches thick and nominal two or more inches wide.

Dressed Size: The dimensions of lumber after surfacing with a planing machine. Usually 1/2 to 3/4 inch (13 to 19mm) less than nominal size.

Edge Distance: The distance from the edge of the member to the center of the nearest fastener. When a member is loaded perpendicular to the grain, the loaded edge shall be defined as the edge in the direction toward which the fastener is acting.

Gypsum Wallboard or Drywall: An interior wall surface sheathing material sometimes considered for resisting lateral forces.

Hold-Down: Hardware used to anchor the vertical chord forces to the foundation or framing of the structure in order to resist overturning of the wall.

Moisture Content: The weight of the water in wood expressed as a percentage of the weight of the over-dried wood.

Nominal Size: The approximate rough-sawn commercial size by which lumber products are known and sold in the market. Actual rough-sawn sizes vary from the nominal. Reference to standards or grade rules is required to determine nominal to actual finished size relationships, which have changed over time.

Oriented Strandboard: A structural panel comprising thin elongated wood strands with surface layers arranged in the long panel direction and core layers arranged in the cross panel direction.

Panel: A sheet-type wood product.

Panel Rigidity or Stiffness: The in-plane shear rigidity of a panel, the product of panel thickness and modulus of rigidity.

Panel Shear: Shear stress acting through the panel thickness.

Platform Framing: Construction method in which stud walls are constructed one floor at a time, with a floor or roof joist bearing on top of the wall framing at each level.

Plywood: A structural panel comprising plies of wood veneer arranged in cross-aligned layers. The plies are bonded with an adhesive that cures upon application of hear and pressure.

Pressure-Preservative Treated Wood: Wood products pressure-treated by an approved process and preservative.

Row of Fasteners: Two or more fasteners aligned with the direction of load.

Wood and Light Metal Framing continued:

Sheathing: Lumber or panel products that are attached to parallel framing members, typically forming wall, floor, ceiling, or roof surfaces.

Structural-Use Panel: A wood-based panel product bonded with an exterior adhesive, generally 4’ x 8’ (1.2 x 2.4m) or larger in size. Included under this designation are plywood, oriented strand board, waferboard, and composite panels. These panel products meet the requirements of PS 1-95 (NIST, 1995) or PS 2-92 (NIST, 1992) and are intended for structural use in residential, commercial, and industrial applications.

Stud: Wood member used as vertical framing member in interior or exterior walls of a building, usually 2” x 4” or 2” x 6” (50mm x 100mm or 50mm x 150mm) sizes, and precision end-trimmed.

Tie: See drag strut.

Tie-Down: Hardware used to anchor the vertical chord forces to the foundation or framing of the structure in order to resist overturning of the wall.

Time Effect Factor: A factor applied to adjusted resistance to account for effects of duration of load.

APPENDIX D
GROUND MOTION BACKGROUND DATA

D-1. Earthquake Source and Earthquake Size Definition

a. A Simple Earthquake Source Model. The actual release of earthquake energy along a fault plane in the crust of the earth is a very complex phenomenon. All the physical processes that occur just before, during and after a seismic event are still not completely understood, and considerable research is going on to better describe this phenomenon. However, for engineering purposes, the above complex phenomenon is idealized, and Figure D-1(a) gives the resulting simplified model representation of the earthquake source. In this model, an earthquake is caused by the sudden release of energy accumulated during tectonic processes. The energy is released via faulting (rupture) of rock along a plane (the fault plane). Part of the energy is converted into elastic energy carried by seismic waves and thus the shaking that is felt during earthquakes.

b. Earthquake Location. Even though a substantial volume of the earth's crust is involved in the energy release, it is generally assumed that the faulting initiates at a discrete point (the hypocenter or focus) and then spreads over a larger area (Figure D-1(a)). The term epicenter is used to denote the point on the earth's surface directly above the hypocenter. In recent times (since the installation of seismographs), the locations of the hypocenter are determined by means of instruments. Before the advent of instrumentation, the epicenter was located by means of finding the region of most intense shaking. Quite often, the field epicenter (region of intense shaking) and the instrumentally located epicenter do not coincide.

c. Types of Faulting. Figure D-2 shows the three basic types of faulting. They are defined by the sense of relative displacement between the two adjoining blocks along the fault plane. In a normal fault, the upper block slides downward relative to the lower block. In a reverse fault, the upper block rides up. In a strike-slip fault, one block moves horizontally past the other. Any faulting may be described as a combination of these three basic types of faulting.

d. Types of Seismic Waves. Seismic waves generated by an earthquake source are of three main types: P, S, and surface waves (Figure D-1(b)). The P wave has the fastest travel speed and its particle motion involves compression and expansion of the rock. The S waves travel more slowly than P waves, arrive after the P waves, and exhibit particle motion transverse to the direction in which they travel. Both P and S waves move through the body of the earth and are thus called body waves. Body waves are followed by surface waves that travel along the earth's surface and have motion that is restricted to near the earth's surface.

e. Earthquake Size.

(1) Magnitude. Among the various quantitative measures of earthquake size, magnitude is undoubtedly the most successful and widely used. The basic concept of magnitude is to compare sizes of earthquakes in a relative manner. In his definition of magnitude, Richter (Richter, 1958) rates an earthquake relative to a standard size earthquake by comparing their maximum amplitudes recorded by the same type of seismometer at the same distance to the epicenter,

$$M = \log \left[\frac{A(\Delta)}{A_o(\Delta)} \right] \qquad \text{(D-1)}$$

where Δ is the distance from observation location to the epicenter (epicentral distance), A and $A_{@}$ denote the recorded maximum amplitudes of an earthquake and the standard size earthquake, respectively. The standard size earthquake is defined as to have $A_{@} = 1\mu$m (10^{-6} meter or $3.3\mathrm{x}10^{-6}$ feet) recorded by a Wood-Anderson seismometer at Δ = 100 km (62 miles). Tables were constructed empirically to reduce from 100 km (62 miles) to any distance. A graphical representation of the table is given in Figure D-3. Since the scale is logarithmic, an increase of one step on the magnitude scale increases the amplitude scale by a factor of 10 (see Figure D-3). Richter magnitude scale was originally defined for local earthquakes in southern California; the definition has been adopted and expanded to become applicable to other regions using different type of instruments. Richter magnitude is only used for shallow local (Δ < 600 km or 375 miles) earthquakes, hence it is also called the local magnitude (M_L). Body-wave magnitude (m_b) and surface-wave magnitude (M_s) have been introduced to measure the size of distant earthquakes (Δ > 600 km or 375 miles). Surface-wave magnitude M_s is usually based on the amplitude of 20 seconds period surface waves recorded at distances of thousands of kilometers, where seismograms are dominated by surface waves. Body wave magnitude is based on the maximum amplitude of 1 second period P-waves.

(2) Seismic moment. As more is known about the earthquake source mechanism and about the size of earthquake events, it is becoming increasingly clear that the

existing magnitude scales are inadequate to describe the overall size or the energy content of earthquake events. To

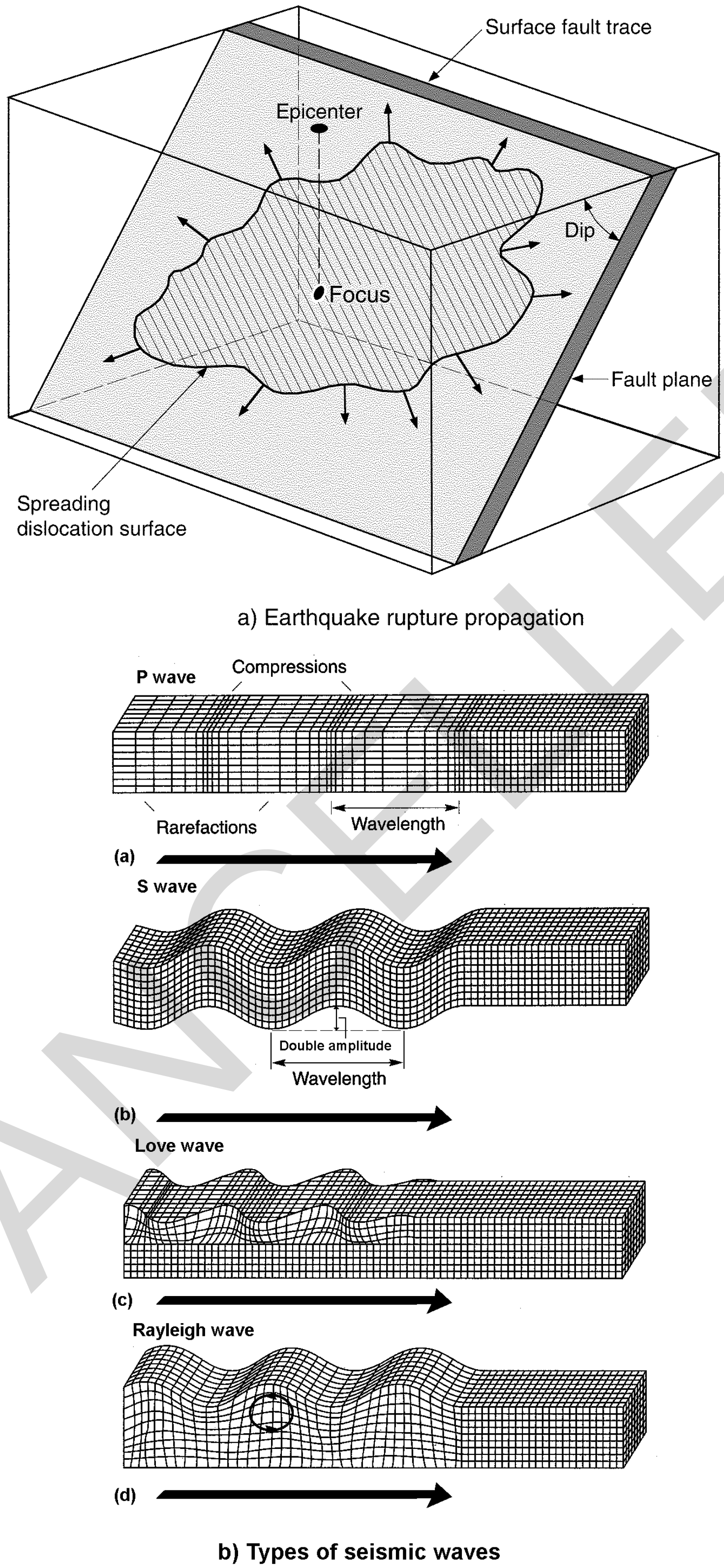


Figure D-1 Earthquake source model and types of seismic waves (from Bolt, 1993).

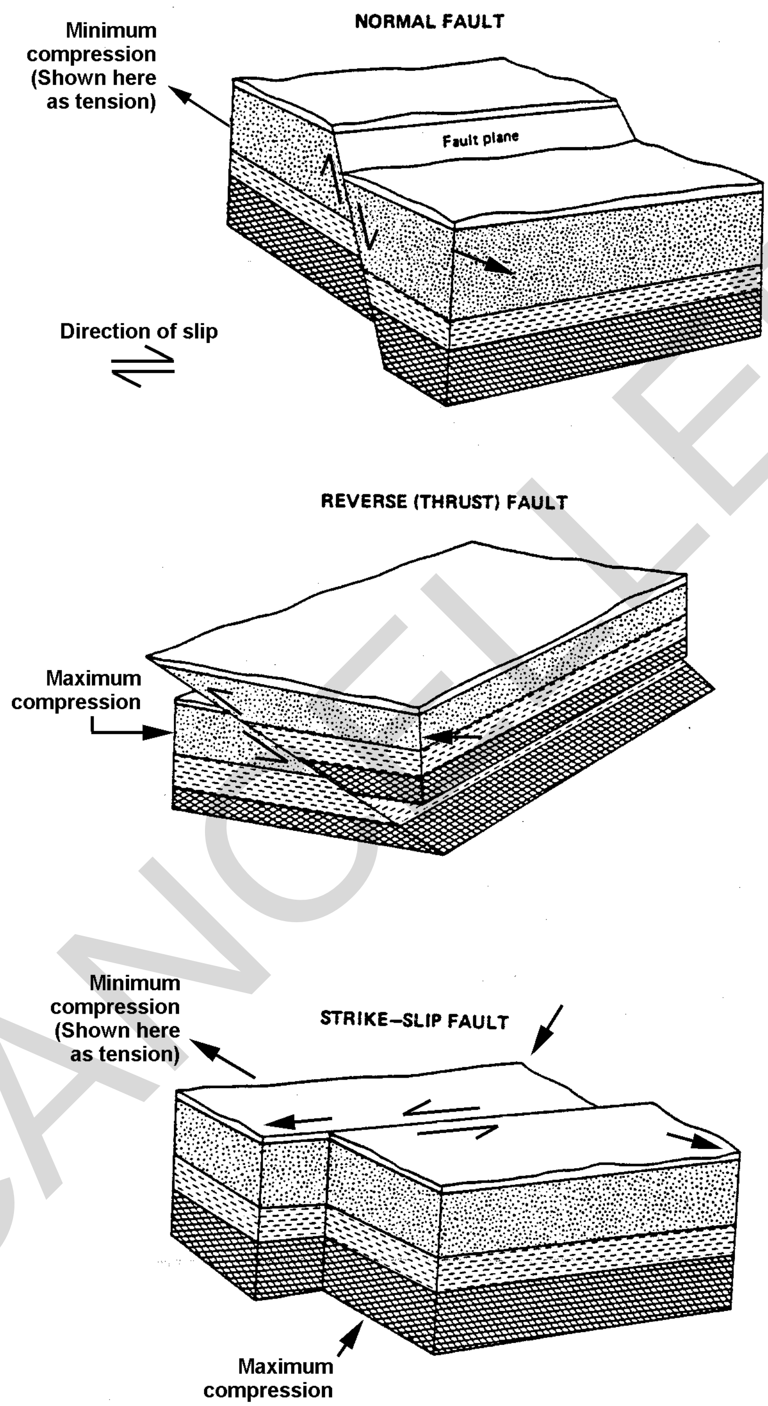
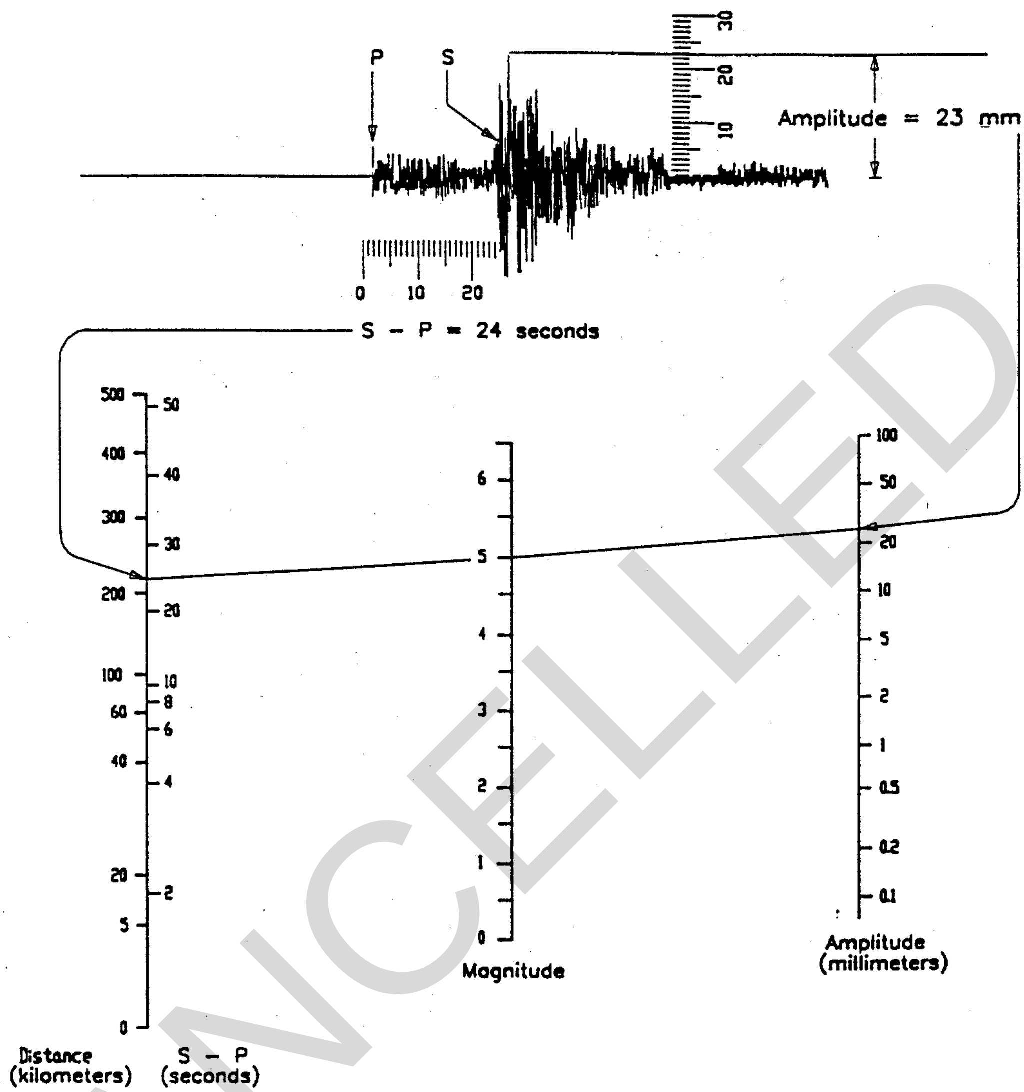


Figure D-2 Types of fault slip.



Procedure for calculating the local magnitude, M_L

1. Measure the distance to the focus using the time interval between the S and the P waves (S − P = 24 seconds).
2. Measure the height of the maximum wave motion on the seismogram (23 millimeters).
3. Place a straight edge between appropriate points on the distance (left) and amplitude (right) scales to obtain magnitude M_L = 5.0.

1 km = 0.62 miles; 1 mm = 0.04 inches

Figure D-3 The Richter Scale (after Bolt, 1988).

overcome this deficiency, seismologists have introduced a "physical" parameter called seismic moment, M_o, to describe the size of an earthquake. This parameter is directly related to the size of the fault rupture area, the average slip on the fault, and the property in shear of the ruptured zone (recall that the magnitude scale is a relative scale). M_o is defined as:

$$M_o = G\ A\ S \qquad \text{(D-2)}$$

where

G = average shear modulus over the rupture zone
A = fault rupture area
S = average slip on the fault during the earthquake.

(3) Moment magnitude. In order to relate seismic moment to the existing magnitude scales, a moment magnitude (M_w) has been introduced. In the M_L range of 3 to 6, M_w corresponds to M_L. M_w is related to seismic moment M_o by the following relationship (Hanks and Kanamori, 1979).

$$M_w = \frac{2}{3} Log M_o - 10.7 \qquad \text{(D-3)}$$

where M_o is in units of dyne-cm.

Comparative values of the moment magnitudes and seismic moments of some well-known earthquakes are given in Table D-1.

(4) Intensity measures. Another means of describing the size of an earthquake at a given location is the intensity scale. The two intensity scales used in the United States are the Rossi-Forel Scale (RF Scale) and the Modified Mercalli Scale (MM Scale).

(a) The Modified Mercalli scale is the most common. A simplified version of this scale is given in Table D-2. The RF scale, which was developed in the late 19th century, was used in this century until 1930. Since then, use of the MM scale has become more common. It is important to note that the above scales are subjectively assigned by investigators after observing and reviewing the earthquake effects in a given region. The assignment of proper intensity value therefore requires a careful analysis of the affected region. Unless the guidelines for assigning intensities are properly and correctly followed, there could be an error in the assigned value.

(b) Empirical relationships are available in the literature to relate the magnitude of an earthquake and the intensity in the epicentral area. The following illustrate such relationships.

(Gutenberg and Richter, 1956)

$$M_L = 1 + \frac{2}{3} I_o \qquad \text{(D-4)}$$

(Krinitzky and Chang, 1975)

$$M_L = 2.1 + \frac{1}{2} I_o \qquad \text{(D-5)}$$

(Chinnery and Rogers, 1973) for Northeastern United States,

$$M_L = 2.1 + 0.6 I_o \qquad \text{(D-6)}$$

where

M_L = Richter magnitude or local magnitude
I_o = Modified Mercalli intensity in the epicentral area

(c) All such relationships, including those derived for specific sites where specific data are available, are extremely approximate and the scatter of data about the predicted lines is large. Note that much of the scatter is due to the necessity of empirically converting site intensity data to the equivalent I_o value in the epicentral area, so as to normalize the site distance attenuation effects. Figure D-4 (Krinitzky and Chang, 1975) shows the relationships given by Equations D-4 and D-5 along with earthquake data.

D-2. Ground Motion Recordings and Ground Motion Characteristics

a. Characteristics in the Time Domain. With the introduction of modern strong motion instruments, the actual ground motion at a given location is often derived from instrumentally recorded motions. The most commonly used instruments for engineering purposes are strong motion accelerographs. These instruments record the acceleration time history of ground motion at a site, called an accelerogram. Figure D-5(a) shows a typical accelerogram. By proper analysis of a recorded accelerogram to account for instrument distortion and base line correction, the resulting corrected acceleration record can be used by engineers. This corrected acceleration record can yield ground velocity and ground displacement by appropriate integration (Figure D-5(a)).

(1) A number of parameters may be used to characterize strong ground motion in the time domain.

Table D-1 Moment Magnitude, M_w, and Seismic Moment, M_o of some well-known earthquakes.

Earthquake	M_w	M_o (dyne-cm)
1960 Chile Earthquake	9.6	2.5×10^{30}
1964 Alaska Earthquake	9.2	7.5×10^{29}
1906 San Francisco, CA Earthquake	7.9	9.3×10^{27}
1971 San Fernando, CA Earthquake	6.6	1.0×10^{26}
1976 Tangshan, China Earthquake	7.5	1.8×10^{27}
1989 Loma Prieta, CA Earthquake	6.9	2.7×10^{26}
1992 Cape Medocino, CA Earthquake	7.0	4.2×10^{26}
1994 Northridge, CA Earthquake	6.7	1.3×10^{26}
1995 Kobe, Japan Earthquake	6.9	2.5×10^{26}

1 dyne-cm = 7.4×10^{-8} foot-lbs

Table D-2 Modified Mercalli Intensity Scale.

Mercalli's improved intensity scale (1902) served as the basis for the scale advanced by Wood and Neumann (1931), known as the modified Mercalli scale and commonly abbreviated MM. The modified version is described below with some improvements by Richter (1958).

To eliminate many verbal repetitions in the original scale, the following convention has been adopted. Each effect is named at that level of intensity at which it first appears frequently and characteristically. Each effect may be found less strongly or more often at the next higher grade. A few effects are named at two successive levels to indicate a more gradual increase.

Masonry A, B, C, D. To avoid ambiguity of language, the quality of masonry, brick or otherwise, is specified by the following lettering.

Masonry A. Good workmanship, mortar, and design; reinforced, especially laterally, and bound together by using steel, concrete, etc.; designed to resist lateral forces.

Masonry B. Good workmanship and mortar; reinforced, but not designed in detail to resist lateral forces.

Masonry C. Ordinary workmanship and mortar; no extreme weaknesses like failing to tie in at corners, but neither reinforced nor designed against horizontal forces.

Masonry D. Weak materials, such as adobe; poor mortar; low standards of workmanship; weak horizontally.

Modified Mercalli Intensity Scale of 1931 (Abridged and Rewritten by C.F. Richter)

I. Not felt. Marginal and long-period effects of large earthquakes.

II. Felt by person at rest, on upper floors, or favorably placed.

III. Felt indoors. Hanging objects swing. Vibration like passing of light trucks. Duration estimated. May not be recognized as an earthquake.

IV. Hanging objects swing. Vibration like passing of heavy trucks; or sensation of a jolt like a heavy ball striking the walls. Standing motor cars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. In the upper range of IV, wooden walls and frame creak.

V. Felt outdoors; direction estimated. Sleepers wakened. Liquids disturbed, some spilled. Small unstable objects displaced or upset. Doors swing, close, open. Shutters, pictures move. Pendulum clocks stop, start, change rate.

VI. Felt by all. Many frightened and run outdoors. Persons walk unsteadily. Windows, dishes, glassware broken. Knickknacks, books, etc., off shelves. Pictures off walls. Furniture moved or overturned. Weak plaster and masonry D cracked. Small bells ring (church, school). Trees, bushes shaken visibly, or heard to rustle.

VII. Difficult to stand. Noticed by drivers of motor cars. Hanging objects quiver. Furniture broken. Damage to masonry D, including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices, also unbraced parapets and architectural ornaments. Some cracks in masonry C. Waves on ponds; water turbid with mud. Small slides and caving in along sand or gravel banks. Large bell rings. Concrete irrigation ditches damaged.

VIII. Steering of motor cars affected. Damage to masonry C; partial collapse. Some damage to masonry B; none to masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down; loose panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on steep slopes.

IX. General panic. Masonry D destroyed; masonry C heavily damaged, sometimes with complete collapse; masonry B seriously damaged. General damage to foundations. Frames structure, if not bolted, shifted off foundation. Frame racked. Serious damage to reservoirs. Underground pipes broken. Conspicuous cracks in ground. In alluviated areas, sand and mud ejected, earthquake fountains, sand craters.

Table D-2 Modified Mercalli Intensity Scale.

X. Most masonry and frame structures destroyed with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankments. Large Landslides. Water thrown on banks of canals, rivers, lakes, etc. Sand and mud shifted horizontally on beaches and flat land. Rails bent slightly.

XI. Rails bent greatly. Undergournd pipelines completely out of service.

XII Damage nearly total. Large rock masses displaced. Lines of sight and level distorted. Objects thrown into the air.

(Reprinted from "*Elementary Seismology*", C.F. Richter, 1958, with permission from W.H. Freeman and Company.)

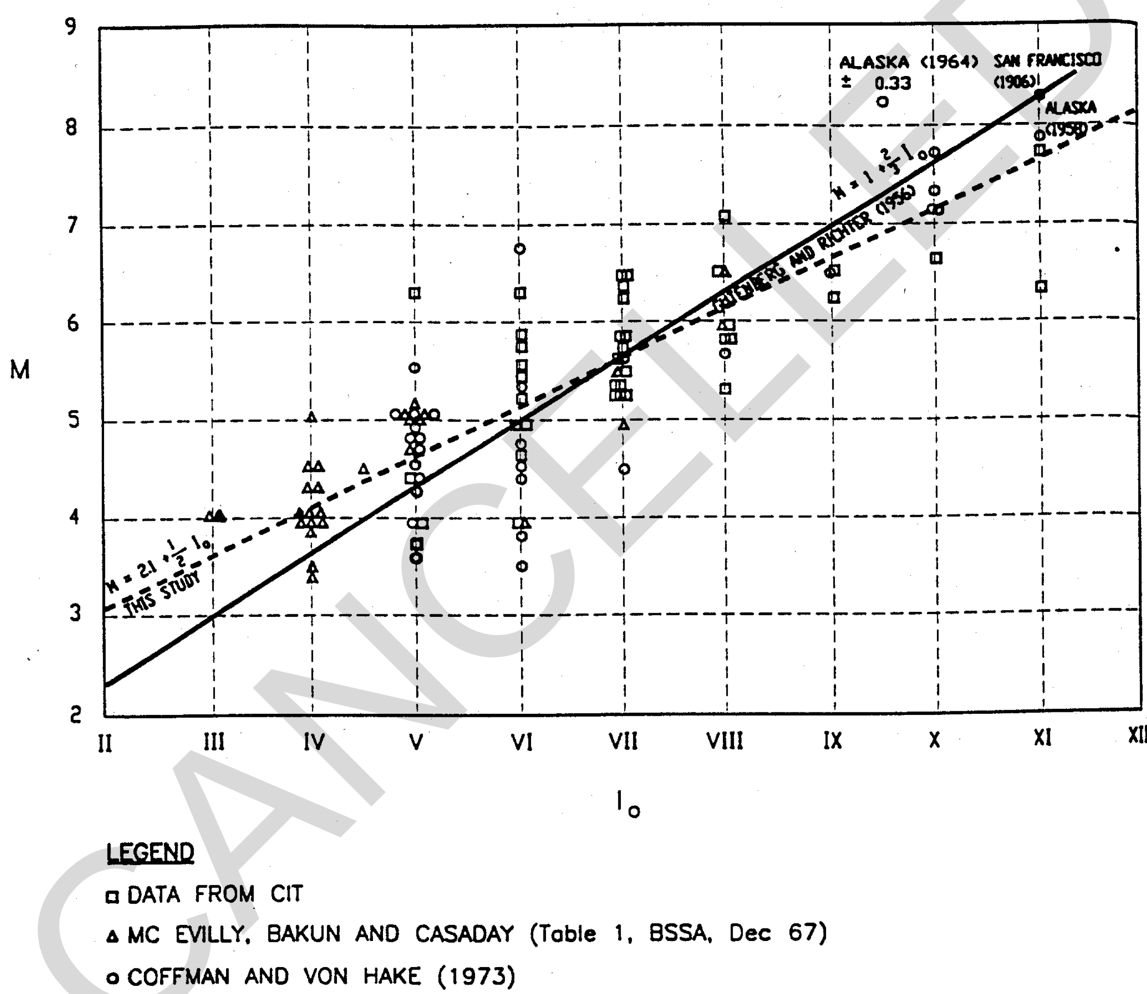
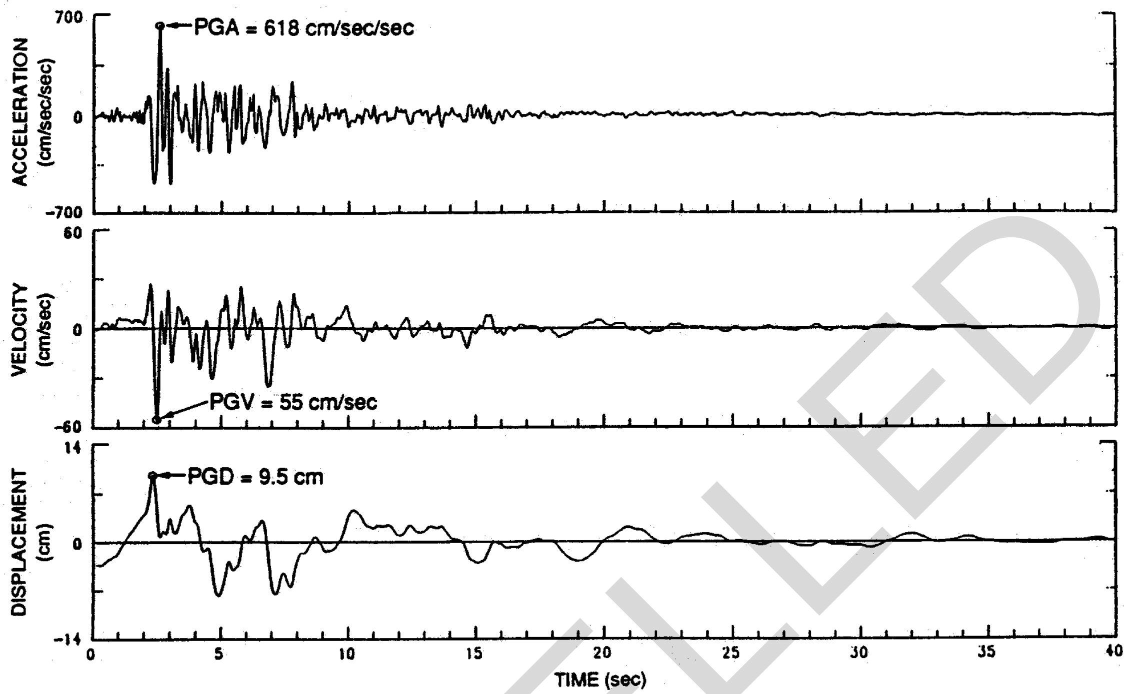
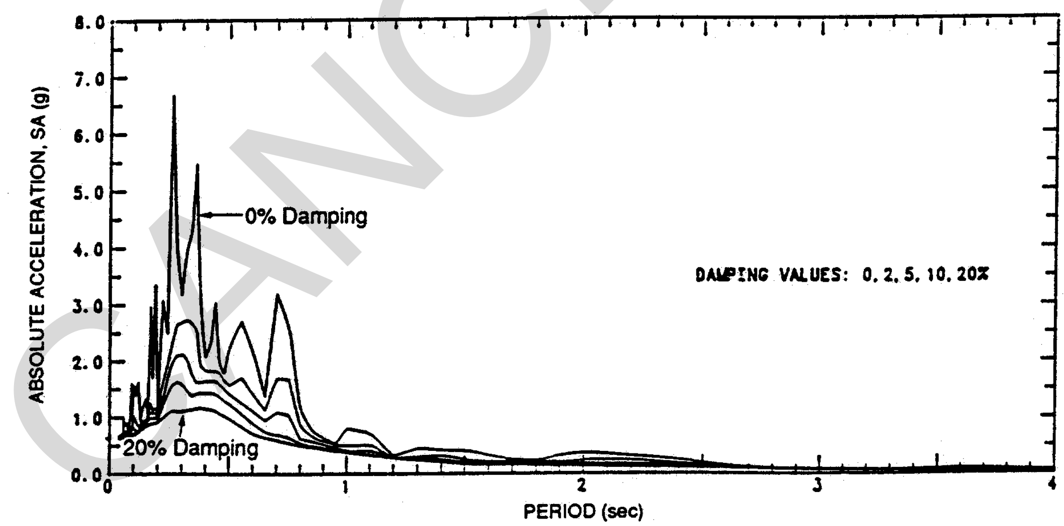


Figure D-4 Relation between earthquake magnitude and epicentral intensity in the western United States (after Krinitzky and Chang, 1975).



a) Acceleration, velocity, and displacement time histories.



b) Acceleration response spectra.

1 cm = 0.4 inches

Figure D-5 Corralitos ground motion recording, component 0E, October 17, 1989, Loma Prieta, California earthquake (after California Division of Mines and Geology, 1989).

These include peak ground acceleration (PGA), peak ground velocity (PGV), peak ground displacement (PGD), and strong motion duration. It should be noted that these ground motion parameters provide only gross descriptions of the recorded ground motions. The PGA value, normally expressed as a fraction of the earth's gravity (note that one gravity unit, or 1g, is equal to 980.7 cm/sec^2 or 32.2 ft/sec^2), has been the key parameter in the past characterizing the level of ground shaking for engineering purposes; while duration has been used to characterize the time duration of significant shaking during earthquakes. Different definitions of strong motion duration have been used. Bolt (1973) defined a bracketed duration as the lapsed time between the first and last acceleration greater than a given level (0.05 g and 0.10 g as used by Bolt (1973)). Trifunac and Brady (1975) and Dobry et al. (1978) defined significant duration as the time needed for the integral of $(x^2(t))^2$, where $x^2(t)$ is the ground acceleration at time t, to build up between 5 and 95 percent of its total value for the accelerogram. The integral of $(x^2(t))^2$ is a measure of the energy of an accelerogram (Arias, 1969). There are empirical relationships between duration and earthquake magnitude (e.g., Bolt, 1973; Dobry et al., 1978).

(2) In general, the recorded ground motion consists of the three main types of seismic waves described in paragraph D-1d.. Experience indicates that each accelerogram has a variable degree of detail. For example, at distances close to the earthquake fault, the onset of the main S waves is often associated with a longer-period pulse related to the fault slip (see Figure D-6). It is important to take this into consideration when designing structures near an active fault.

b. Response Spectrum. Seismic ground motion may be characterized as the superposition of a set of harmonic motions having a fairly broad range of frequencies. This characterization of the ground motion (called the Fourier spectrum) is often used by seismologists and is different from the response spectrum discussed here. Structures subjected to the input ground motion tend to amplify the harmonics near their own natural frequencies and filter or attenuate the others. The resulting structural response therefore depends upon the frequency content of the harmonics in the ground motion and their relation to the dynamic frequency characteristics of the structure. This paragraph provides the definitions and discussions of the response spectrum representation of this inter-relationship between ground motion input and structural response.

(1) Single degree-of-freedom system response. Figure D-7 shows the system and the definition for seismic input and response.

(a) Response to arbitrary ground motion input *x(t).* For any given ground acceleration $x^2(t)$, the relative displacement response *u(t)* is

$$u(t) = -\frac{1}{w_D} \int_o^t x''(t)\; e^{-wb(t-t)}\; \sin\left[w_D(t-t)\right]\; dt \quad \text{(D-7)}$$

where $w_D = w\,(1-b^2)^{1/2}$ is the damped natural frequency of the single-degree-of-freedom system and b is the damping ratio. For the case of zero damping, this equation simplifies to

$$u(t) = -\frac{1}{w} \int_o^t x''(t)\; \sin\left[w\,(t-t)\right]\; dt \qquad \text{(D-8)}$$

where w is the undamped natural frequency of the system. Relative velocity and acceleration responses are given by the time derivatives $u'(t)$ and $u''(t)$, respectively.

(b) Response to sinusoidal input. If the ground acceleration $x''(t)$ were to be a single unit amplitude sinusoid at frequency Ω, $x''(t) = sin\Omega t$, then the corresponding response is given by $u(t) = H(T)\, sin[\Omega t + \phi]$, where ϕ is a phase angle and

$$H(w) = \frac{1}{\left[\left(1-(\Omega/w)^2\right)^2 + \left(2b\;\;\Omega/w\right)^2\right]^{1/2}} \qquad \text{(D-9)}$$

is the system's frequency-response function which either amplifies or attenuates the response according to the frequency ratio Ω/w, and the damping ratio b, see Figure D-8. This function is most useful in the explanation of how predominant harmonics in ground motion can amplify the ordinates of the response spectrum.

(2) Response spectra. For a given ground acceleration $x^2(t)$ such as shown in Figure D-5(a), and given damping ratio, the absolute maximum values found from the complete time history solution of equation D-7 provide the response spectrum values at the system frequency w, or period, $T=2\pi/w$. A response spectrum is traditionally presented as a curve connecting the maximum response values for a set of prescribed frequency or period values, such as shown in Figure D-5(b). The different response spectra quantities are defined as:

SD = $[\,u(t)\,]_{max}$ = Relative Displacement Response

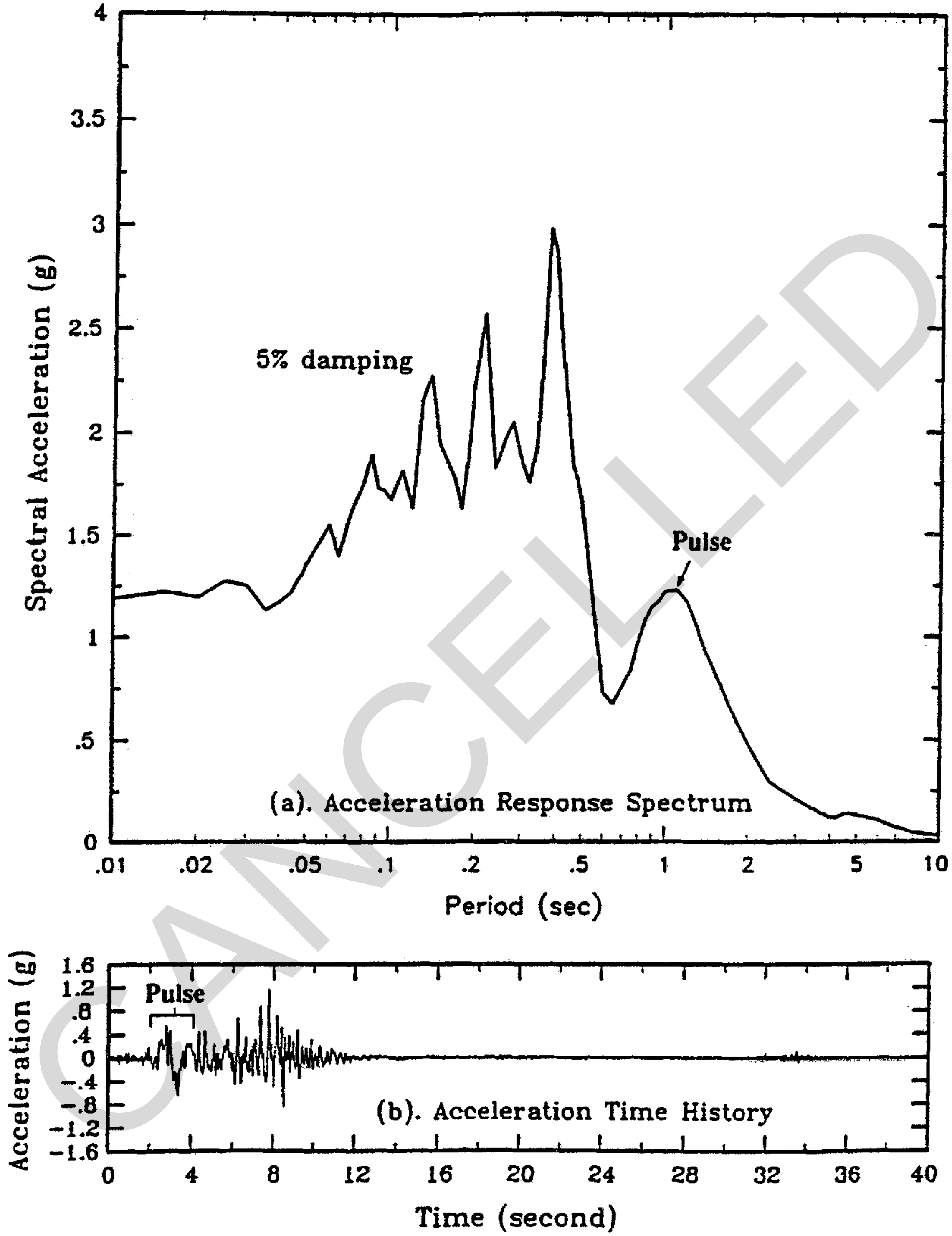
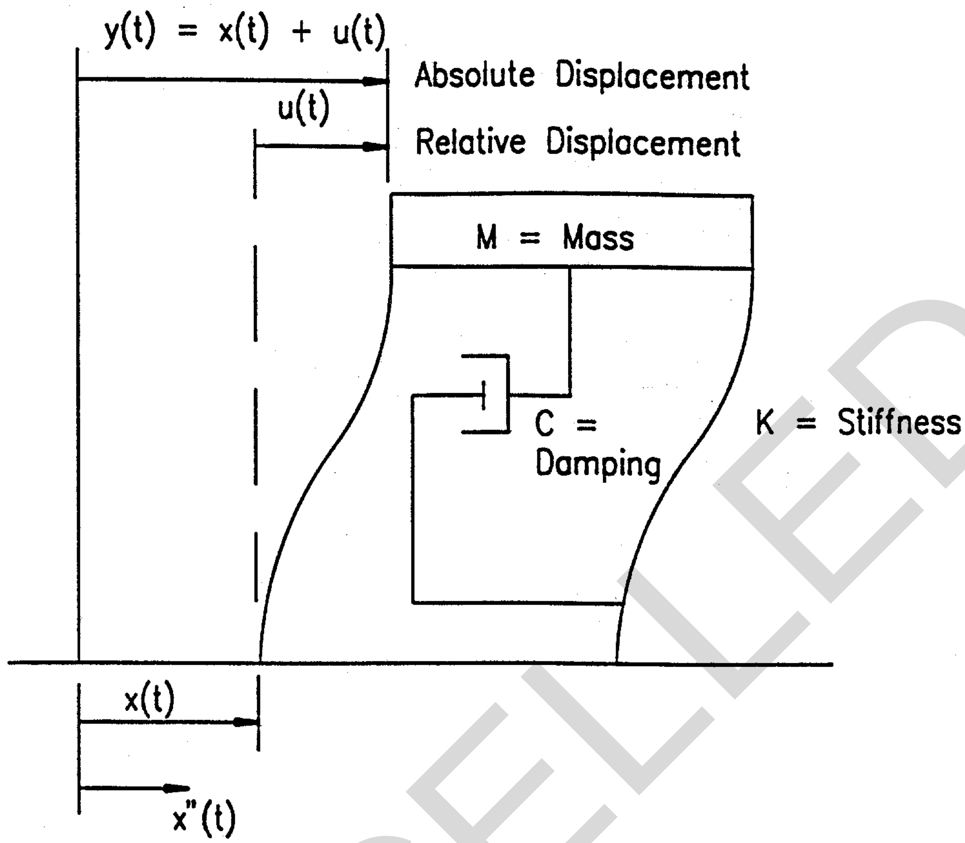


Figure D-6 Pacoima dam recording (S14W component) obtained 3 km (1.9 miles) from the causative fault during the 1971 San Fernando, California earthquake.



<u>System Properties</u>

$\omega = \sqrt{K/M}$ = undamped natural frequency

$\beta = \dfrac{C}{2M\omega}$ = fraction of critical damping

$\omega_D = \omega\sqrt{1-\beta^2}$ = damped natural frequency

<u>Ground Motion</u>

$x(t)$ = displacement

$x'(t) = \dfrac{dx}{dt}$ = velocity

$x''(t) = \dfrac{d^2x}{dt^2}$ = acceleration

Figure D-7 Single degree of freedom system.

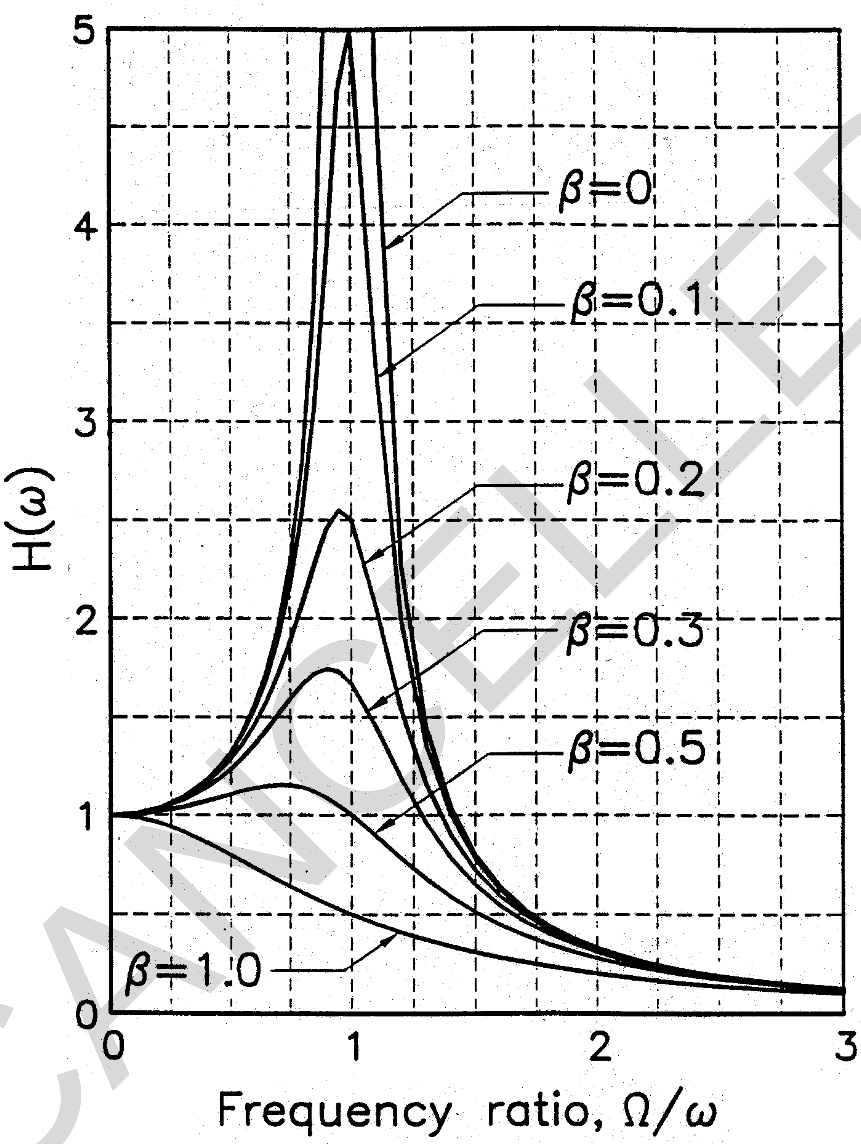


Figure D-8 Maximum dynamic load factor for sinusoidal load.

$SV = [\ u'(t)\ ]_{max}$ = Relative Velocity Response

$SA = [\ y''(t)\ ]_{max} = [\ u''(t) + x''(t)]_{max}$ = Absolute Acceleration Response

Then using the close approximation of $w = w_D$ for b £ 0.1, the more commonly employed versions for engineering purposes are:

$S_v = w \cdot SD$ = Pseudo-Relative-Velocity Response

(D-10)

$S_a = w^2 \cdot SD$ = Pseudo-Acceleration Response

(D-11)

For the common structural damping ratios, and the earthquake type of input motion, there is essential equality for the real and pseudo values,

$$S_v \cong SV \qquad \text{(D-12)}$$

$$S_a \cong SA \qquad \text{(D-13)}$$

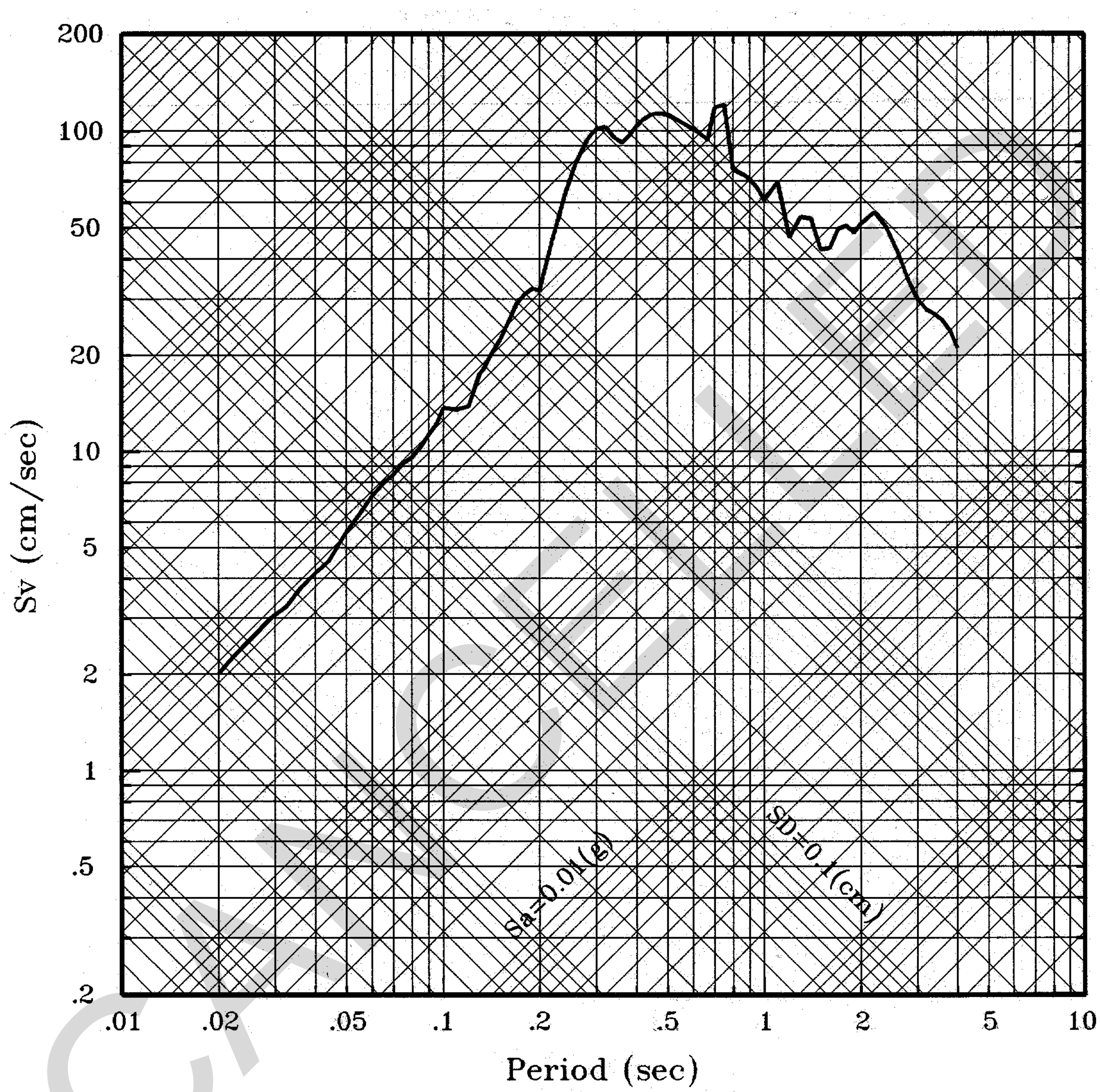
Of course, for long period structures, the velocity equality breaks down since S_v approaches zero, while SV approaches peak ground velocity (PGV). The relationships between SD and S_a can be justified by the following physical behavior of the vibrating system. At maximum relative displacement SD, the velocity is zero, and maximum spring force equals maximum inertia force,

$K \cdot SD = m \cdot S_a$,

where K is stiffness and *m* is mass, giving

$$S_a = K/m \cdot SD = w^2 \cdot SD \qquad \text{(D-14)}$$

Detailed discussions on response spectra and their computation from accelerograms are given in Ebeling 1992, Chopra 1981, Clough and Penzien 1993, and Newmark and Rosenblueth 1971. An example of a typical acceleration response spectrum is shown in Figure D-5(b). Also, because of the relation $S_a = w\ S_v = w^2\ SD$, it is possible to represent spectra on tripartite log paper (Figure D-9).



1 cm = 0.4 inches

Figure D-9 Tripartite plot of the response spectrum from the Corralitos recording, component 0E, of the 1989 Loma Prieta, California Earthquake.

APPENDIX E
SITE-SPECIFIC PROBABILISTIC SEISMIC HAZARD ANALYSIS

E-1. Introduction

a. Purpose. The purpose of this appendix is to describe details of the methodology used in probabilistic seismic hazard analysis (PSHA) to develop site-specific response spectra. More general aspects of the site-specific approach are presented in Chapter 3. In paragraph E-2, the formulation of the basic probabilistic model is described. Paragraph E-3 discusses the incorporation of uncertainty in PSHA. Paragraph E-4 describes the results of a PSHA and how they can be analyzed to determine the dominant contributors to the seismic hazard and sources of uncertainty. In paragraph E-5, two examples of applications of PSHA to develop site-specific response spectra are presented.

E-2. Mathematical Formulation of the Basic Seismic Hazard Model.

a. General Formulation.

(1) Formulation for probability of exceedance. The methodology used to conduct PSHA was initially developed by Cornell (1968). The formulation of the basic seismic hazard model is summarized herein. Additional discussion and guidance for conducting a PSHA is described in several publications, including National Research Council (1988), Earthquake Engineering Research Institute (1989), and Ferritto (1994, 1997). Using a Poisson probability model, the probability of exceedance, p_z, (z), of a ground motion level, z, in an exposure time or design time period, t, at a site is related to the annual frequency (or rate) of ground motion exceedance at the site, *v(z)*, by:

$$p_z(z)=1-e^{-(v(z)\cdot t)} \qquad \text{(E-1)}$$

A PSHA is carried out to obtain *v(z)* and $p_z(z)$ can then be obtained using Equation E-1. The return period (RP) for ground motion exceedance at a site is equal to the reciprocal of *v(z)*. The results of a PSHA are, in practice, expressed in terms of one or more of the parameters, $p_z(z)$, *v(z)*, and RP. Note that when $(v(z)\times t)$ is small (approximately ≤ 0.1) $p_z(z)$ is approximately equal to $(v(z)\times t)$. For larger values of $(v(z)\times t)$, $p_z(z)$ is less than $(v(z)\times t)$.

(2) Formulation for frequency of exceedance. The annual frequency of ground motion exceedance, *v(z),* is evaluated using the following expression:

$$n(z)=\sum_{n=1}^{N}\sum_{m_i=m^o}^{m_i=m^U} l_n(m_i)\sum_{r_j=0}^{r_j=r_{\max}} P_n\left(R=r_j \mid m_i\right)\times P\left(Z>z \mid m_i, r_j\right)$$

(E-2)

in which

$l_n(m_i)$ = the annual frequency of occurrence of earthquakes on seismic source *n* in a magnitude interval centered at m_i. m_i is above a minimum size of engineering significance, m^o, and below the maximum event size, m^U.

$P_n(R=r_j \mid m_i)$ = the probability of an earthquake of magnitude m_i on source *n* occurring at a certain distance r_j from the site

$P(Z>z \mid m_i,r_j)$ = the probability that ground motion level z will be exceeded, given an earthquake of magnitude m_i at distance r_j from the site

Thus, for a given source, the annual frequency or rate of exceeding a certain ground motion level at the site is obtained by summing over all magnitudes (the second summation of Equation E-2) and source-to-site distances (the last summation of Equation E-2) for that source. Then, the total rate of ground motion exceedance at the site, *v(z),* is obtained by adding the rates for all the sources (the first summation of Equation E-2). The components of equation E-2 are discussed in paragraphs b, c, and d below.

b. Frequency of Occurrence of Earthquakes. The incremental rate of earthquakes occurrence $l_n(m_i)$ is obtained from earthquake recurrence relationships. Two recurrence models are typically used in PSHA, the truncated exponential model and the characteristic earthquake recurrence model. These two recurrence models are also discussed in paragraph 3-4e(3)(b) of Chapter 3. For convenience, the subscript *n* for the source region is eliminated in the following paragraphs.

(1) The truncated exponential model of Cornell and Vanmarcke (1969) represents the truncation of the Gutenberg-Richter (1954) earthquake frequency law at a finite upper bound magnitude m^U. The cumulative form, which expresses the rate of occurrence of earthquakes equal to or greater than a certain magnitude *m*, is specified by

$$N(m) = N(m^o)\frac{e^{-b(m-m^o)} - e^{-b(m^U-m^o)}}{1.0 - e^{-b(m^U-m^o)}}, \quad for \quad m^o \le m \le m^U$$

(E-3)

where b = b *ln(10)* and b is the b-value of the Gutenberg-Richter frequency law. Parameters b and $N(m^o)$ are estimated by fitting the recurrence relationship E-3 to the observed recurrence rates obtained from a catalog of historic seismicity. These parameters can be further constrained by the geological slip rate, if it is available. An example of such a truncated exponential recurrence relationship is given on the upper left of Figure E-1. The incremental recurrence rate $l(m_i)$ is obtained by discretizing the cumulative recurrence curves into narrow magnitude intervals as illustrated in the lower left of Figure E-1.

(2) The characteristic earthquake recurrence model is based on the hypothesized fault behavior that individual fault and fault segments tend to generate same-size or characteristic earthquakes (Schwartz and Coppersmith, 1984; Youngs and Coppersmith, 1985a). "Same-size" usually means within about one-half magnitude unit. There are two implementations of the characteristic earthquake model that are commonly used in PSHA. In the characteristic earthquake recurrence model implemented by Youngs and Coppersmith (1985a), the maximum magnitude m^U is taken to be the expected magnitude for the characteristic event, with individual events uniformly distributed in the range of m^U ±**3** magnitude units, representing random variability in individual "maximum" ruptures. The cumulative form of the earthquake recurrence relationship thus becomes

$$N(m) = N^e \frac{10^{-b(m-m^o)} - 10^{-b(m^U - 1/4 - m^o)}}{1 - 10^{-b(m^U - 1/4 - m^o)}} + N^c, \qquad for \quad m^o \le m < m^U - 1/4$$

$$= N^c \frac{m^U + 1/4 - m}{1/2}, \qquad for \quad m^U - 1/4 \le m \le m^U + 1/4$$

(E-4)

where the terms N^e and N^c represent the rate of exponential and characteristic events, respectively. N^e and N^c are specified by the slip rate of the individual fault using the formulation of Youngs and Coppersmith (1985a).

$$N^e = \frac{\dot{M}_o^T (1 - 10^{-b(m^U - \frac{1}{4} - m^o)})}{M_o^U 10^{-b(m^U - \frac{1}{4} - m^o)} \left[\frac{b10^{-\frac{c}{2}}}{c-b} + \frac{b10^b(1-10^{-\frac{c}{2}})}{c} \right]} \qquad \text{(E-5)}$$

$$N^c = \frac{\frac{1}{2} b \ln(10) N^e 10^{-b(m^U - \frac{1}{4} - m^o - 1)}}{1 - 10^{-b(m^U - \frac{1}{4} - m^o)}}$$

where $\dot{M}_o^T$ is the rate of seismic moment release along a fault and M_o^U is the seismic moment for the upper limit event m^U +**3** . $\dot{M}_o^T$ is estimated by μ A_f S, where μ is the shear modulus of fault zone rock (assumed to be $3 \cdot 10^{11}$ dyne/cm^2), A_f is the total fault surface area, S is the slip rate, An example of such a characteristic recurrence relationship is given on the upper right of Figure E-1 and the incremental rate $l(m_i)$ is given on the lower right.

(3) In another implementation of the characteristic earthquake model (Wesnousky, 1986), no allowance is made for the occurrence of events of sizes other than the characteristic size. The characteristic size (m^c) is proportional to fault length and can be determined using relations such as those in Wells and Coppersmith (1994). The recurrence rate for this characteristic size earthquake is thus

$$l(m^c) = \frac{\dot{M}_o^T}{M_o^c} \qquad \text{(E-6)}$$

where M_o^c is the seismic moment of the characteristic size earthquake m^c. This version of the characteristic earthquake recurrence model (called the maximum magnitude model by Wesnousky, 1986) has been used by USGS (1996) and others in PSHAs (e.g. Ferritto, 1994).

c. Distance Probability Distribution. The distance probability distribution, $P(R=r_j\,|m_i)$, depends on the geometry of earthquake sources and their distance from the site; an assumption is usually made that earthquakes occur with equal likelihood on different parts of a source. The function $P(R=r_j\,|m_i)$ also should incorporate the magnitude-dependence of earthquake rupture size; larger-magnitude earthquakes have larger rupture areas, and thus have higher probability of releasing energy closer to a site than smaller-magnitude earthquakes on the same source. An example of probability distributions for the closest distance to an earthquake source is shown in Figure E-2. In this particular example, the source (fault) is characterized as a line source and the probability distributions are based on the formulations presented by Der Kiureghian and Ang (1977). Figure E-2 (diagram a) illustrates the probability distributions for a fault rupture length of 5 km (3.1 miles); Figure E-2 (diagram b) illustrates the probability distributions for a fault rupture length of 25 km (15.5 miles). The longer rupture length corresponds to a larger magnitude. The figure shows the distributions for both the probability of the closest distance to the fault rupture, R, being less than a certain value, $P(R<r_j|m_i)$ and the probability of earthquakes occurring at a certain distance

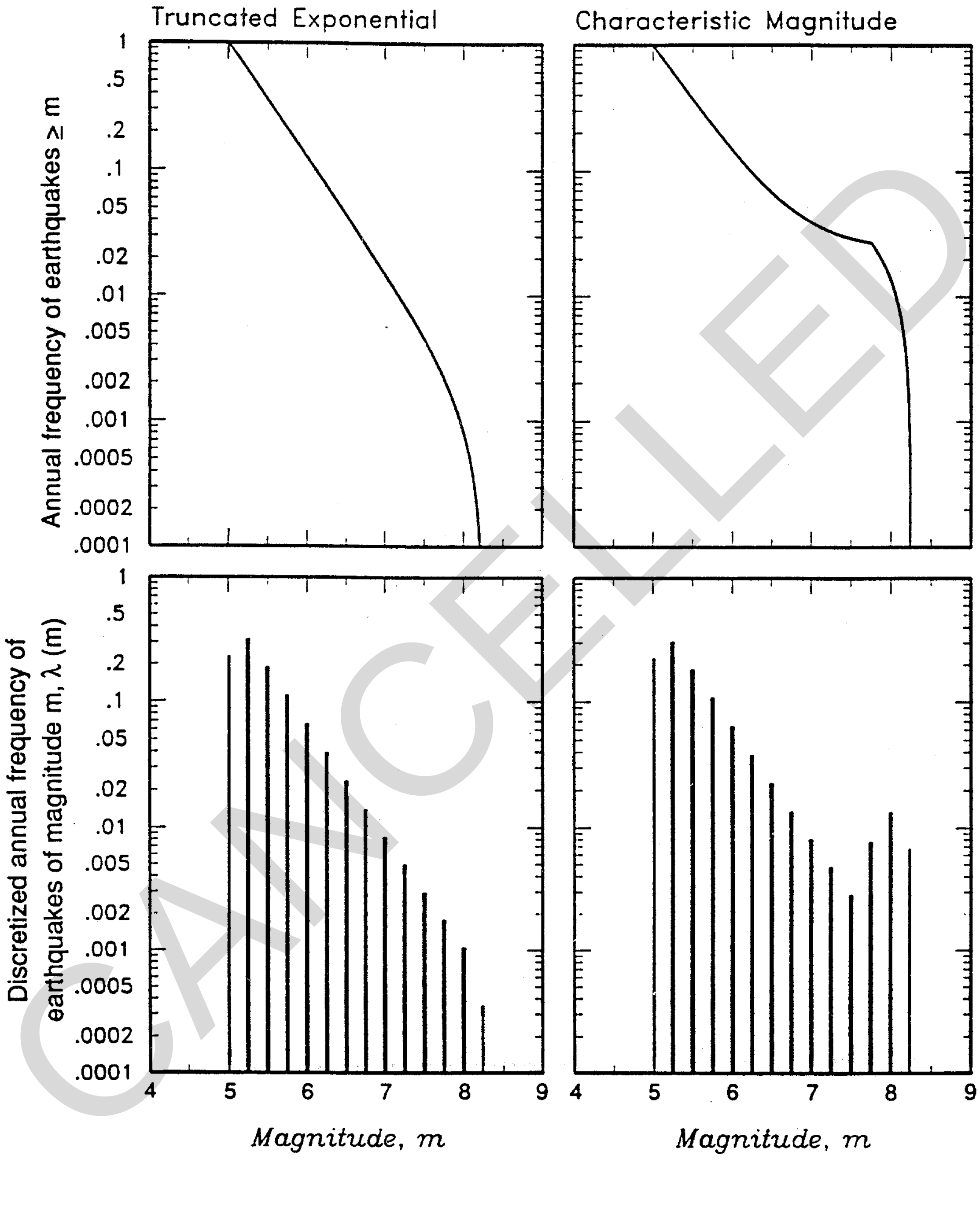
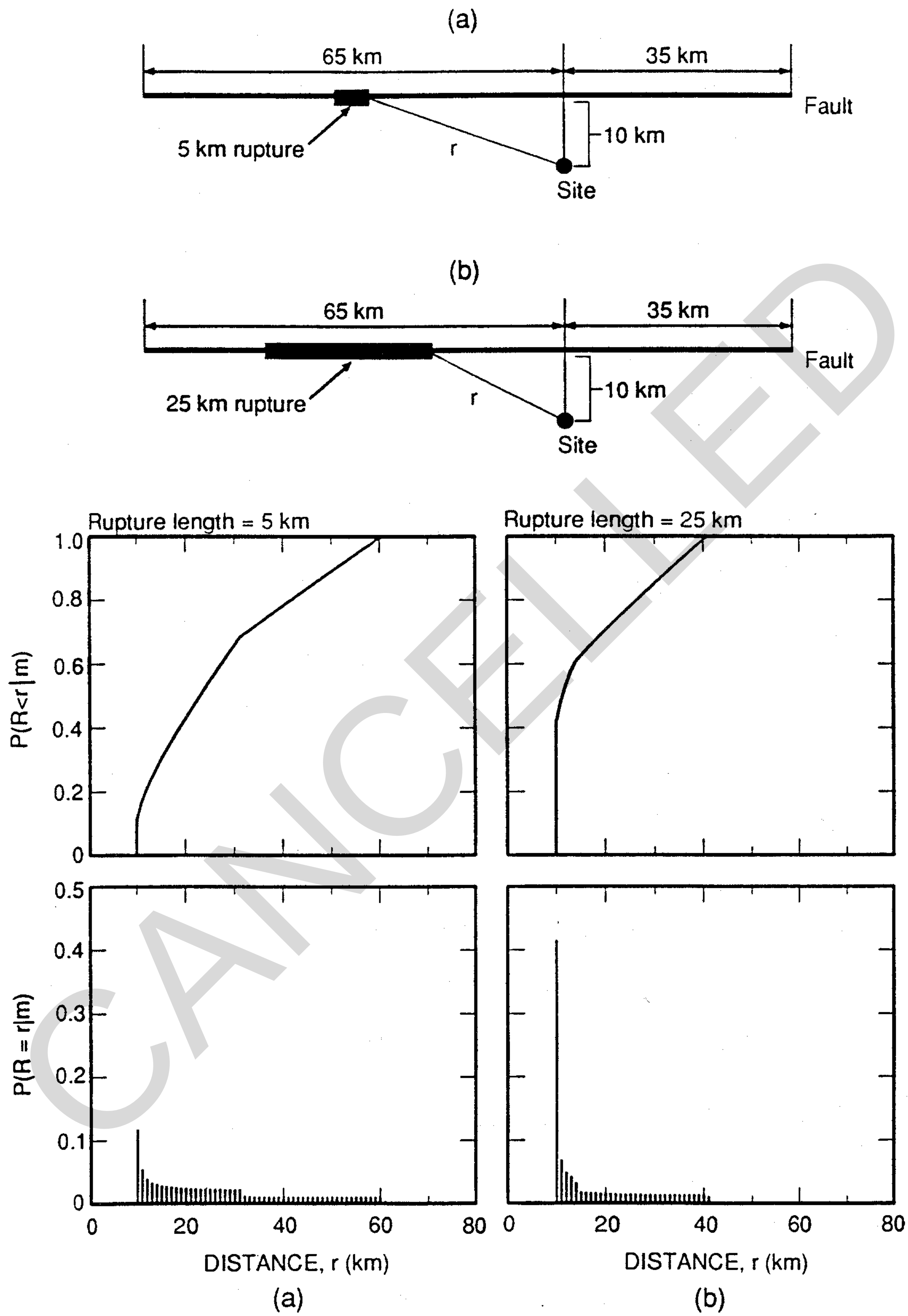


Figure E-1 Typical earthquake recurrence curves and discretized occurrence rates.



1 km = 0.62 miles

Figure E-2 Illustration of distance probability distribution.

($P(R=r_j\,|m_i)$), which is obtained by discretizing the curves for $P(R<r_j|m_i)$. The higher probability for earthquakes to occur at closer distances for longer rupture lengths (larger magnitudes) can be noted by comparing Figure E-2 (diagram b) with E-2 (diagram a). Note that the distance to the earthquake rupture must be expressed in terms of the same definition of distance as used in the ground motion attenuation relationships. Typically, some form of closest distance to rupture definition is used for attenuation relationships (variations in this definition include: closest distance to rupture, closest distance to rupture of the seismogenic zone (at some depth below ground surface), closest horizontal distance to surface projection of rupture, etc.).

d. Ground Motion Exceedance Probability Distribution. The conditional probability of exceeding a ground motion level for a certain earthquake magnitude and distance, $P(Z>z|m_i,r_j)$, is determined from the ground motion attenuation relationships selected for the site. As noted in paragraph 3-4f of Chapter 3 and illustrated in Figure 3-11, attenuation relationships are available for response spectral values as well as for peak ground acceleration. Uncertainty in the median attenuation curves is incorporated, as illustrated in Figures 3-3, 3-4 and 3-11. The function $P(Z>z|m_i,r_j)$ is usually evaluated assuming that ground motion values are log-normally distributed about the median value; the calculation of this function is illustrated in Figure E-3.

E-3. Treatment of Modeling and Parameter Uncertainties in PSHA.

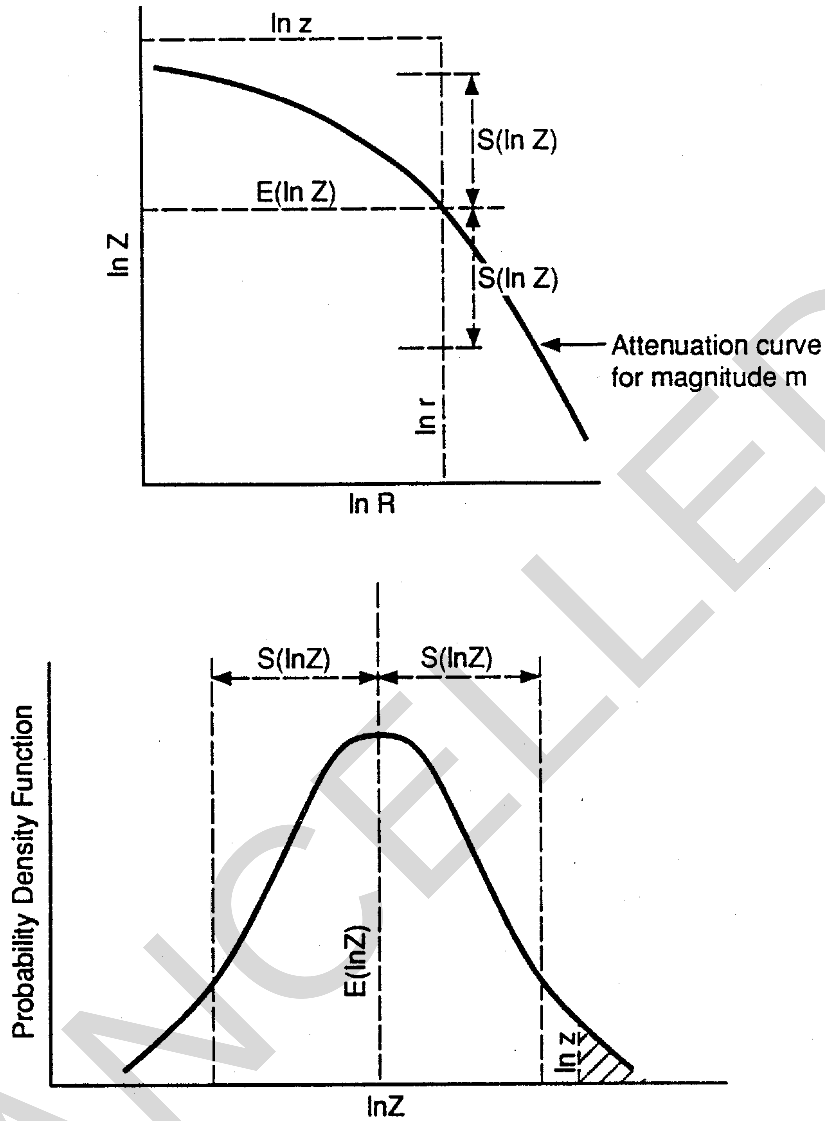
The basic probability formulations in Equations E-1 and E-2 incorporate the randomness of the physical process of earthquake generation and seismic wave propagation. Although these formulations incorporate the inherent uncertainty due to randomness, they do not incorporate additional sources of uncertainty that may be associated with the choice of particular models or model parameters. For example, there could be uncertainty as to which ground motion attenuation relationship is most applicable to a site, uncertainty as to whether an exponential or characteristic earthquake recurrence model is most applicable, uncertainty in the geometry of earthquake sources, uncertainty in the values of maximum earthquake magnitude, uncertainty in earthquake recurrence parameters, etc. In a deterministic analysis, these uncertainties, which are termed epistemic uncertainties, are usually treated by applying conservatism in selecting design earthquakes and estimating ground motions. In PSHA, these uncertainties can be directly modeled within the analysis framework to provide an assessment of the uncertainty in the result. The technique of “logic trees” has been widely used to incorporate scientific uncertainty in a PSHA (Kulkarni et al., 1984; Youngs et al., 1985; Coppersmith and Youngs, 1986; National Research Council, 1988; SSHAC, 1997). Figure E-4 shows an example of a logic tree used in a PSHA. Although only a few branches of the logic tree are shown, there may be many thousands of branches in the tree. Each path through the tree to an end branch (on the right-hand side of the Figure E-4) defines a set of parameters that are used to conduct a basic seismic hazard analysis for that path and end branch using Equation E-2. Basic hazard analyses are carried out for each path. Each path also has an associated probability or weight that is determined by the product of the relative probabilities or weights assigned to the various models and parameters along the path. (The relative probabilities or weights of the alternative models and parameters are illustrated by the numbers in parentheses in Figure E-4.) The basic hazard analysis results for all the paths are combined using the associated weights to arrive at best estimates (mean or median values) for the frequencies of exceedance of ground motions as well as uncertainty bands for the estimates. Through the approach of incorporating scientific uncertainty, PSHA incorporates the alternative hypotheses and data interpretations that may significantly affect the computed results. The display and analysis of uncertainty in the seismic hazard is discussed in the following section.

E-4. Analysis Results.

a. Basic Results. The basic results of a PSHA are seismic hazard curves (curves of the amplitude of a ground motion parameter at a site vs. frequency of exceedance). An example of the typical form of results is illustrated in Figure E-5 for the parameter of peak ground acceleration. A distribution of seismic hazard curves ranging from the 5th to the 95th percentile is shown. This distribution results from the incorporation of scientific uncertainty in the PSHA through the use of logic trees as discussed above. Typically, the mean curve or median (50th percentile) curve is used to obtain design parameters, while the various percentiles of the distribution are a measure of the uncertainty in the result. Note in Figure E-5 that the mean curve lies above the median curve. This result is typical of seismic hazard analysis. In general, the mean curve rather than the median curve is the preferred measure of the hazard results. The use of hazard curve results to develop response spectra is described in paragraph 3-4h of Chapter 3.

b. Analysis of Contribution to the Seismic Hazard. A hazard curve incorporates contributions from different earthquake sources, magnitudes, and source-to-site distances. The results can be analyzed to determine the

major contributions to the hazard. For example,
contributions of different earthquake sources to the mean



$$P(Z>z \mid m, r) = 1 - F_u \left(\frac{\ln z - E(\ln Z \mid m, r)}{S(\ln Z \mid m, r)} \right)$$

EXPLANATION

E(lnZ) is the expected value of lnZ

S(lnZ) is the standard deviation of the estimate of lnZ

F_u is cumulative distribution function of a unit normal variable

Figure E-3 Ground motion estimation conditional probability function.

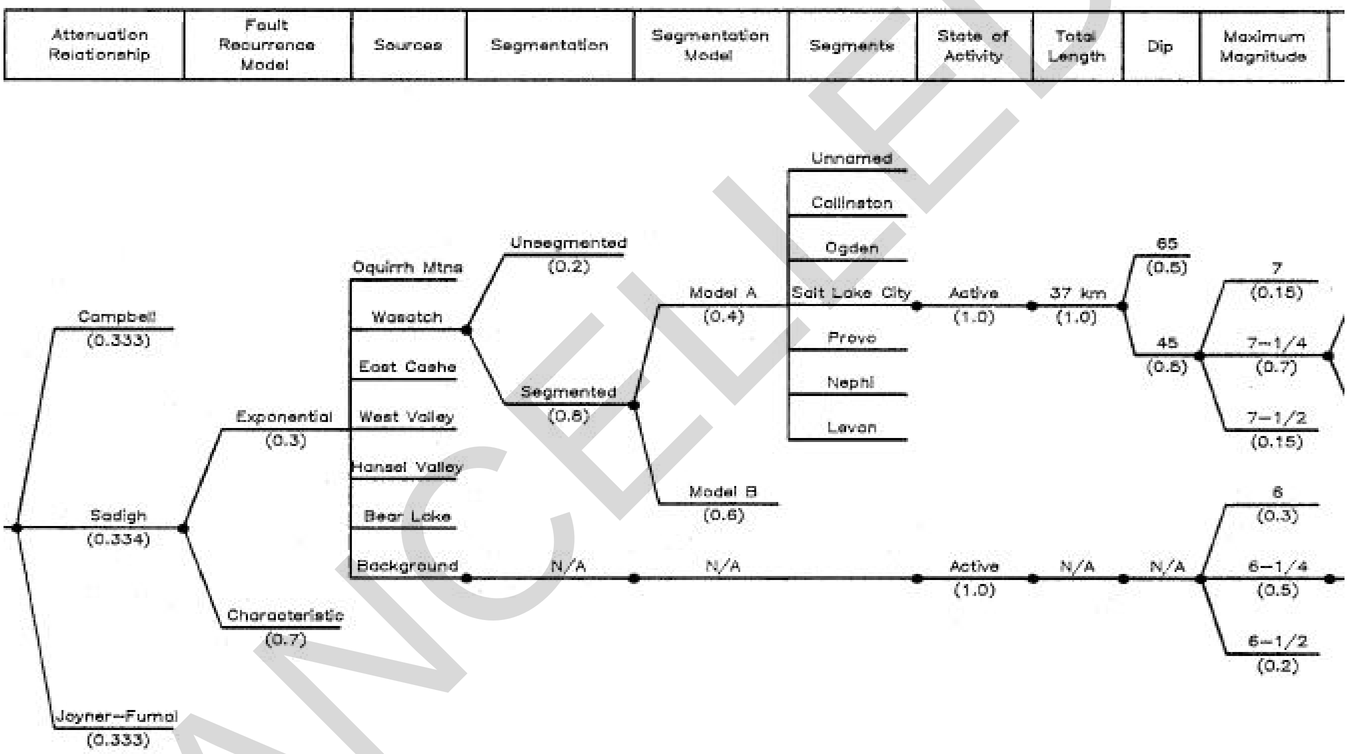


Figure E-4 Example logic tree for characterizing uncertainty in seismic hazard input (Youngs et al., 1988)

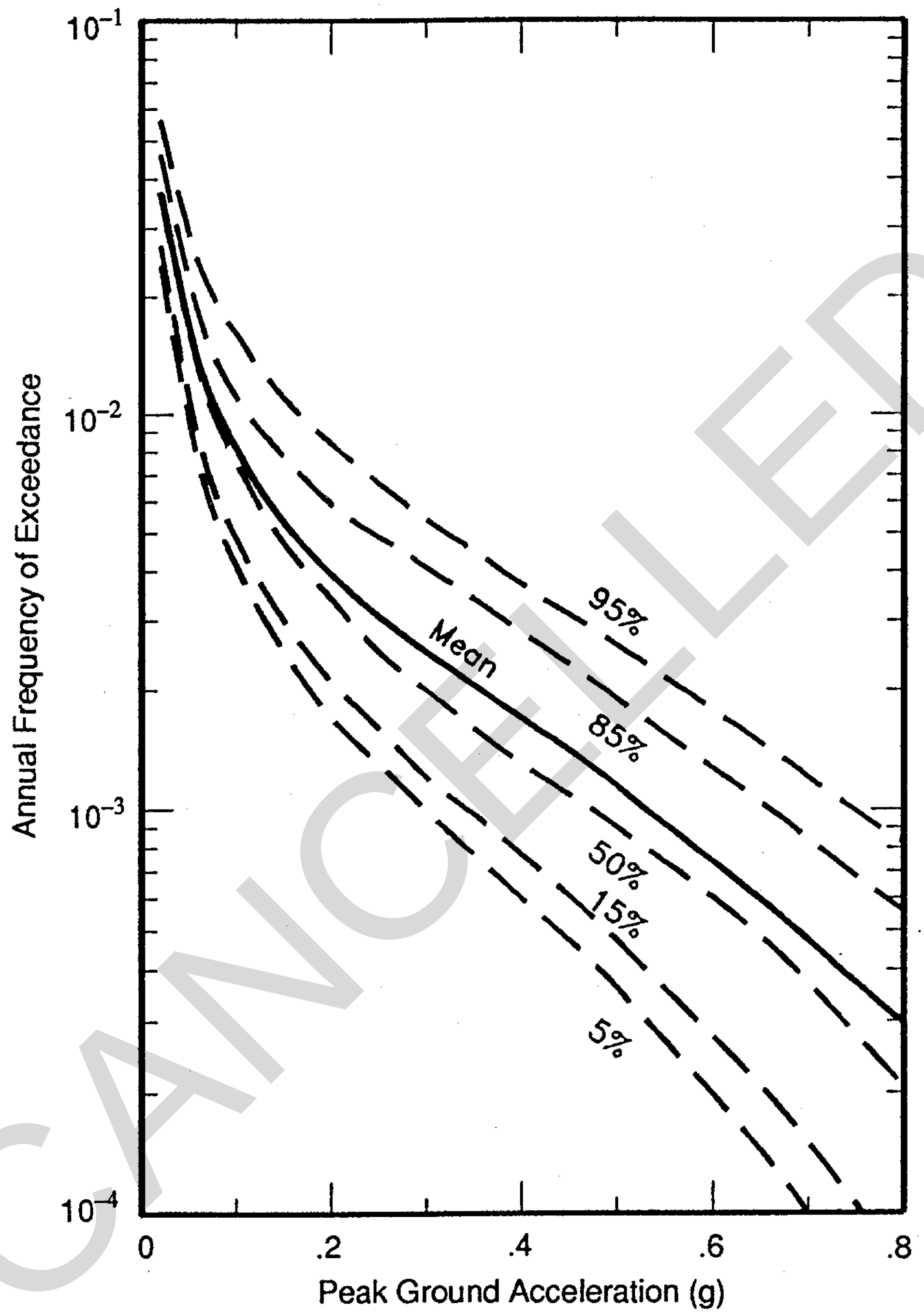


Figure E-5 Example of distribution of seismic hazad results.

hazard curves for three ground motion parameters (peak ground acceleration and response spectral values at periods of vibration of 0.3 and 3.0 seconds) at a site are illustrated in Figure E-6. The contributions of different earthquake magnitudes to the seismic hazard at the same site are illustrated in Figure E-7. As shown in the example in Figure E-7, there is increasing contribution to the hazard from large magnitude earthquakes as the response spectral period of vibration increases. This result is typical and reflects the larger influence of magnitude on ground motions at longer periods, as illustrated in the attenuation curves in Figure 3-10 of Chapter 3. Also as shown, the contribution of larger magnitude earthquakes increases as the return period increases (probability level decreases). This result is also typical and reflects the lesser ability of smaller magnitude earthquakes to produce high levels of ground motion. An analysis similar to that shown in Figure E-7can also be made to identify the dominant distance ranges contributing to the seismic hazard (although the distance contributions may be adequately described by the source contributions in many cases). In cases where site-specific acceleration time histories are required, such analyses of the dominant contributors to the site ground motion hazard are essential to the process of selecting or developing time histories that have appropriate characteristics, including an appropriate duration of strong shaking (duration is strongly correlated with earthquake magnitude).

c. Analysis of Contributions to Uncertainty in the Seismic Hazard. The results of a PSHA can also be analyzed to identify those components of the seismic hazard model that primarily contribute to uncertainty in the hazard results, as reflected in the hazard curve distributions such as illustrated in Figure E-5. This uncertainty is due to the alternative models and parameter values incorporated in the logic tree. The analysis of two potential contributors to uncertainty in seismic hazard results is illustrated in Figures E-8 and E-9. In Figure E-8, it can be seen that uncertainty in the choice of ground motion attenuation relationships contributed substantially to the overall uncertainty in seismic hazard (as measured by the 5^{th} to 95^{th} percentile hazard curve results) for this particular site. In Figure E-9, it can be similarly seen that uncertainty in maximum earthquake magnitude contributed only moderately to the overall uncertainty in seismic hazard for the same site.

E-5. Examples of PSHA Usage in Developing Site-Specific Response Spectra.

a. Introduction. In the following two subsections, examples of the application of PSHA in developing site-specific response spectra are presented. These examples illustrate the characterization of analysis inputs, analysis of the results, and development of equal hazard response spectra. The first example is a relatively high-hazard site in the San Francisco Bay Area in California; the second example is a moderate hazard site in southern Illinois.

b. Site in San Francisco Bay Area.

(1) Seismic source characterization.

(a) The site is a rock site located approximately 21 km (13 miles) east of the San Andreas fault and 7 km (4.3 miles) west of the Hayward fault, as shown in Figure E-10. The seismic sources, including discrete faults and area sources, are shown in Figure E-11. The corridors shown around the faults are for the purposes of analyzing the seismicity that is likely associated with the faults.

(b) For each fault, cumulative earthquake recurrence based on seismicity was plotted and compared with earthquake recurrence based on geologic slip rate data for the fault. For the slip-rate-based recurrence assessments, two magnitude distribution models were initially used exponential model; and characteristic model. Comparisons of recurrence estimated for each model with seismicity were made. Examples of these comparisons for the San Andreas fault and Hayward fault are shown in Figures E-12 and E-13. These comparisons and comparisons for other faults indicate that the characteristic magnitude distribution used in conjunction with fault slip rate data provided recurrence characterizations in good agreement with seismicity data. On the other hand, the exponential magnitude distribution used with the fault slip rate data resulted in recurrence rates that exceeded the rates from seismicity data. From these comparisons and comparisons for the other faults, it was concluded that the fault-specific recurrence was appropriately modeled using the characteristic magnitude distribution model and this model was used for all the fault-specific sources. Recurrence on the area sources was modeled using both: (1) the exponential magnitude distribution and seismicity data; and (2) both the exponential and characteristic magnitude distributions and tectonic data on plate convergence rates in the San Francisco Bay Area. For the entire central Bay Area, a comparison was made between the recurrence predicted by the adopted recurrence models and the observed seismicity. This comparison is shown in Figure E-14 and illustrates good agreement. The faults contribute much more to the regional recurrence than the area sources. Because the fault recurrence is modeled using geologic slip-rate data, the comparison in Figure E-14 is indicative of good agreement between seismicity and

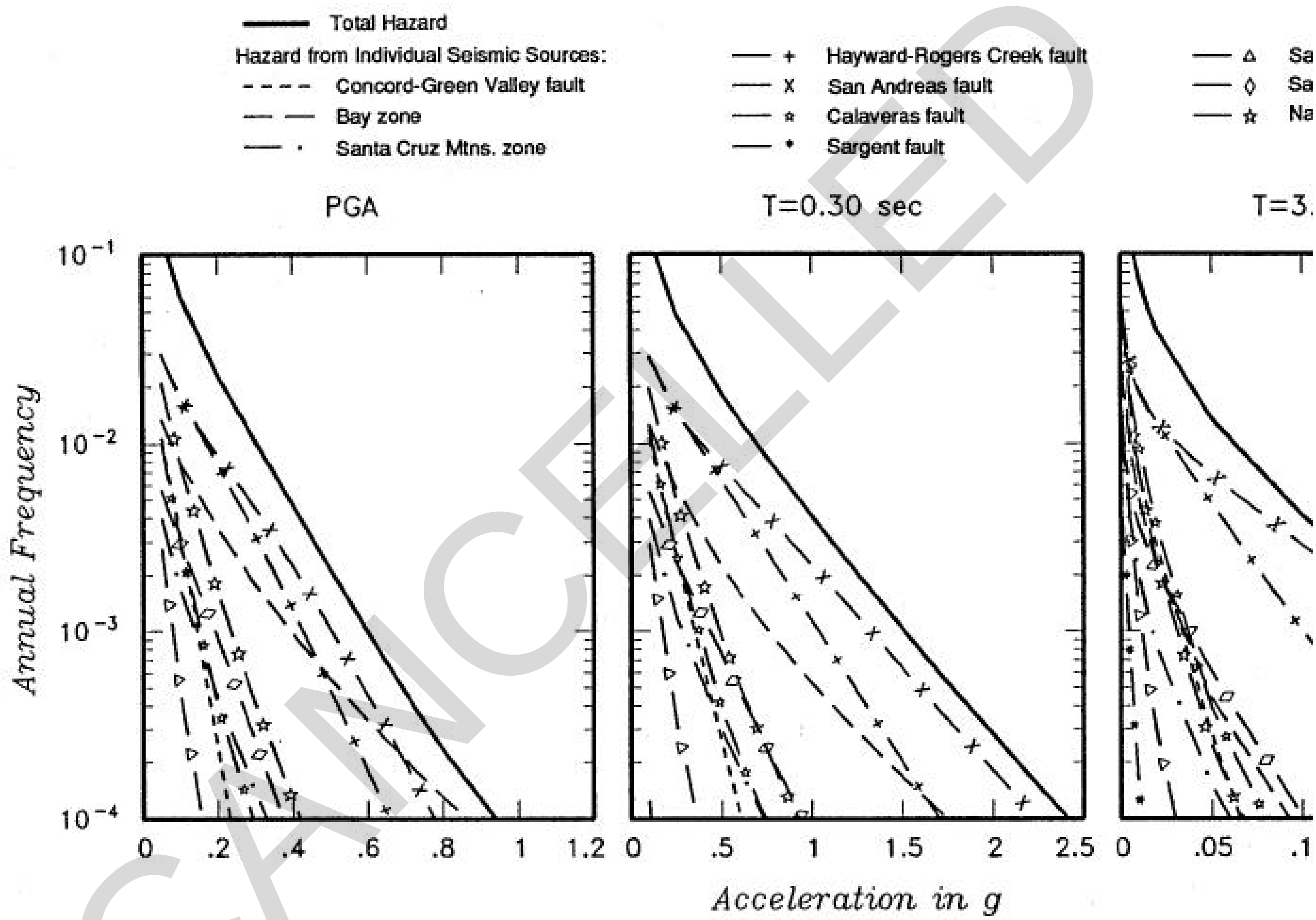


Figure E-6 Example of contributions of various seismic sources to the mean hazard at a site.

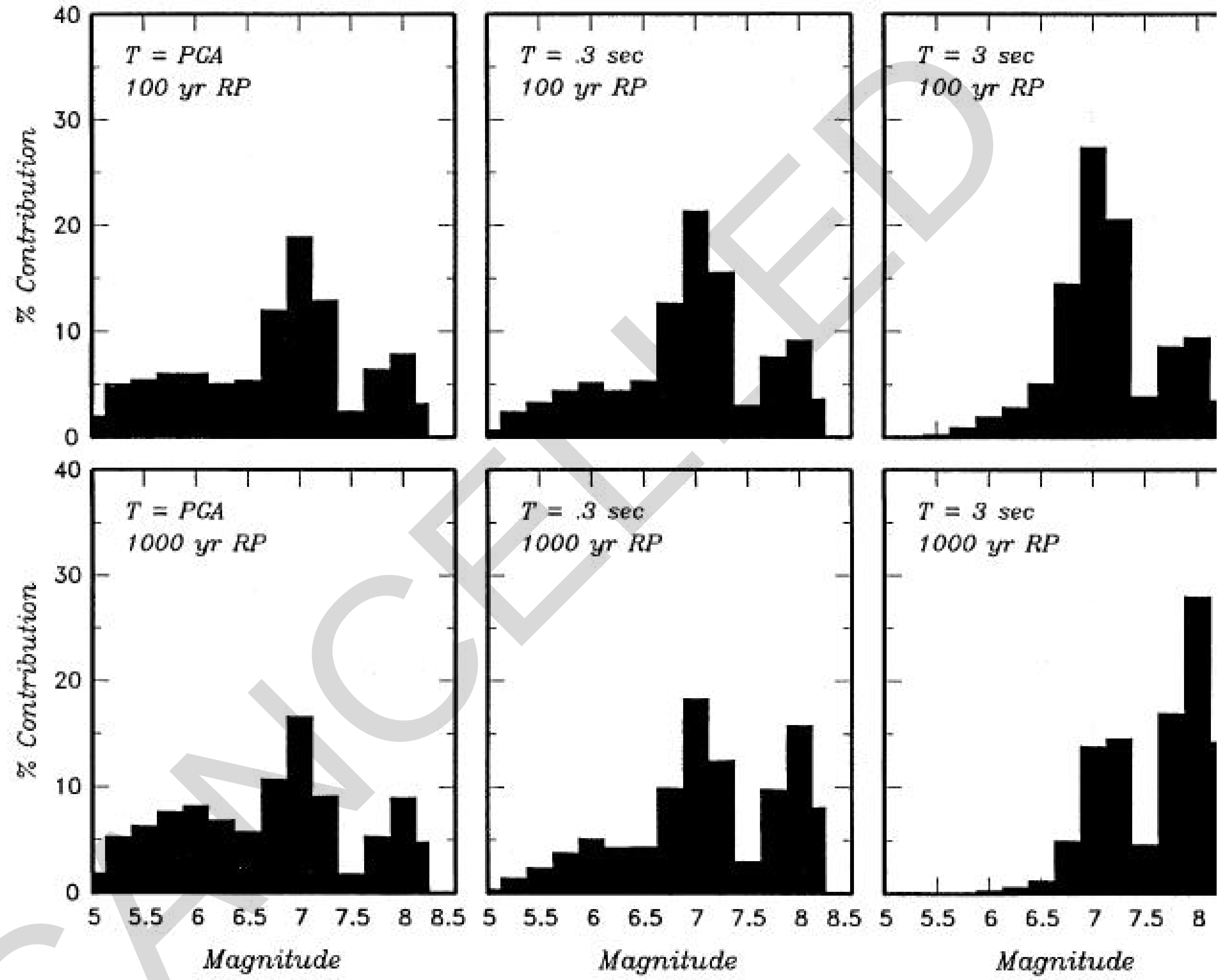


Figure E-7 Example of contributions of events in various magnitude intervals to the hazard for peak accele spectral accelerations at periods of 0.3 and 3.0 seconds.

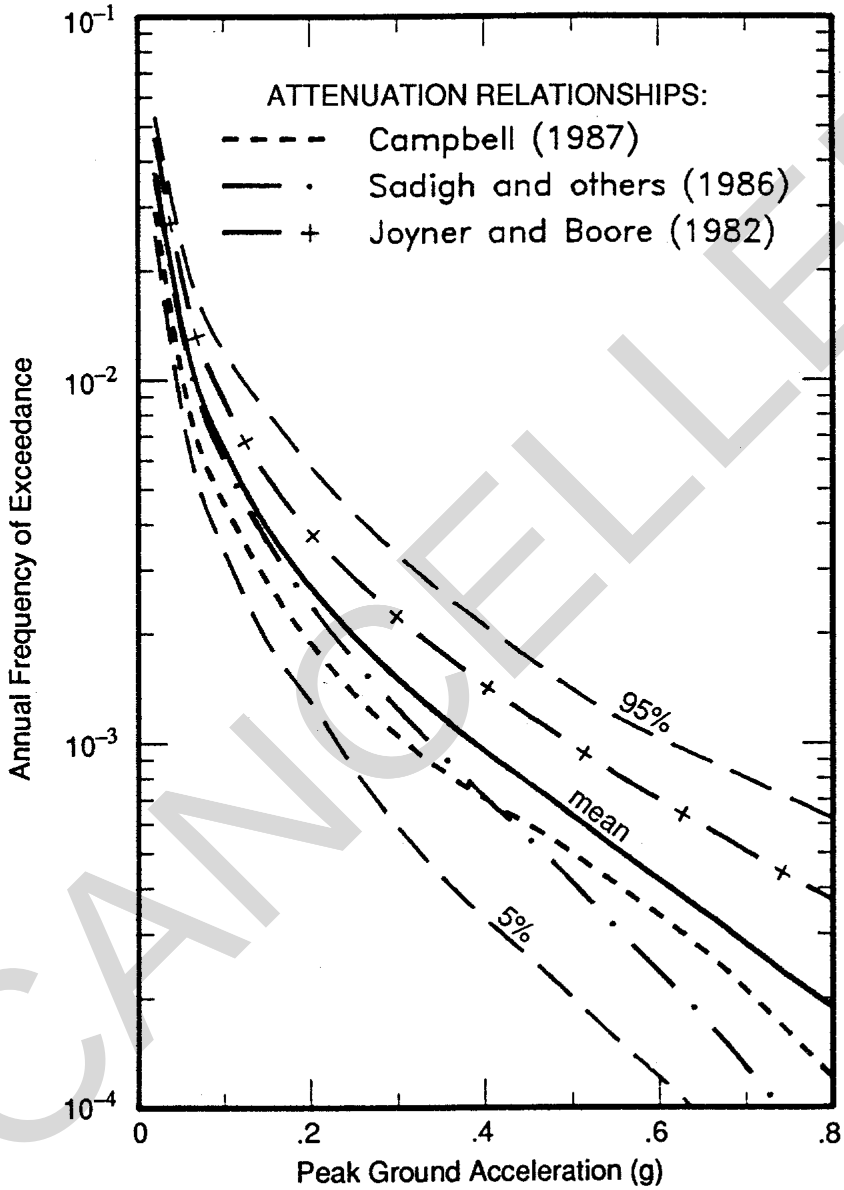


Figure E-8 Example of uncertainty in attenuation contribution to seismic hazard uncertainty.

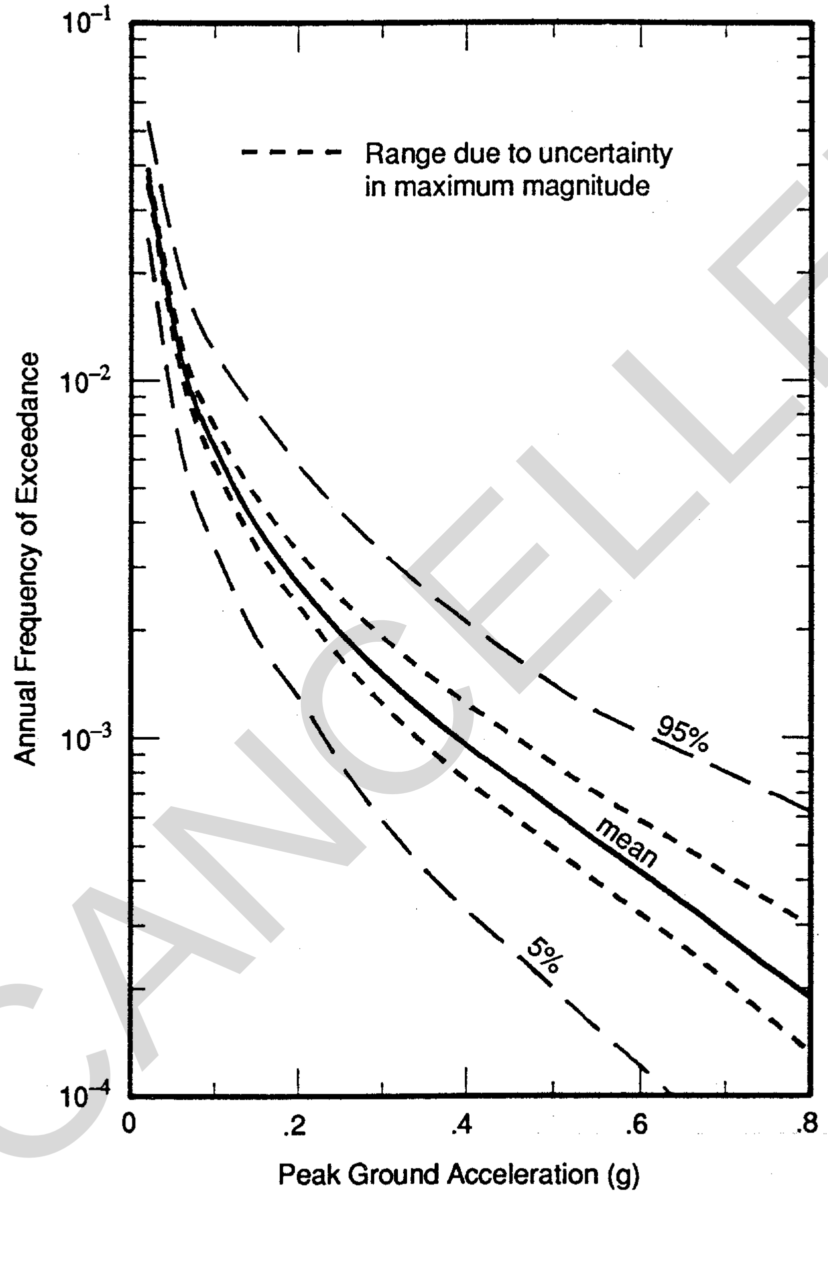


Figure E-9 Example of uncertainty in maximum magnitude contribution to seismic hazard uncertainty.

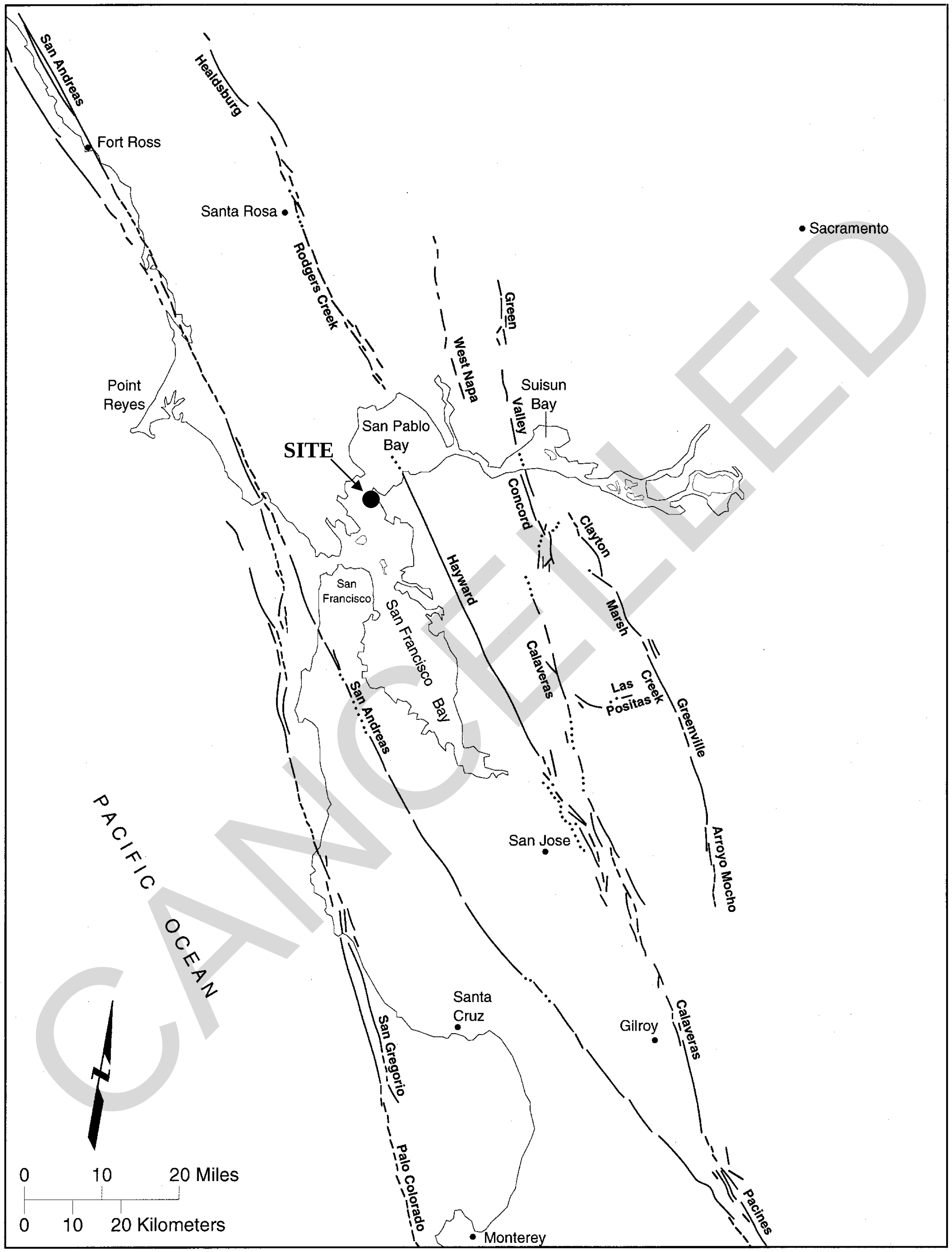


Figure E-10 Regional active fault map, San Francisco Bay area.

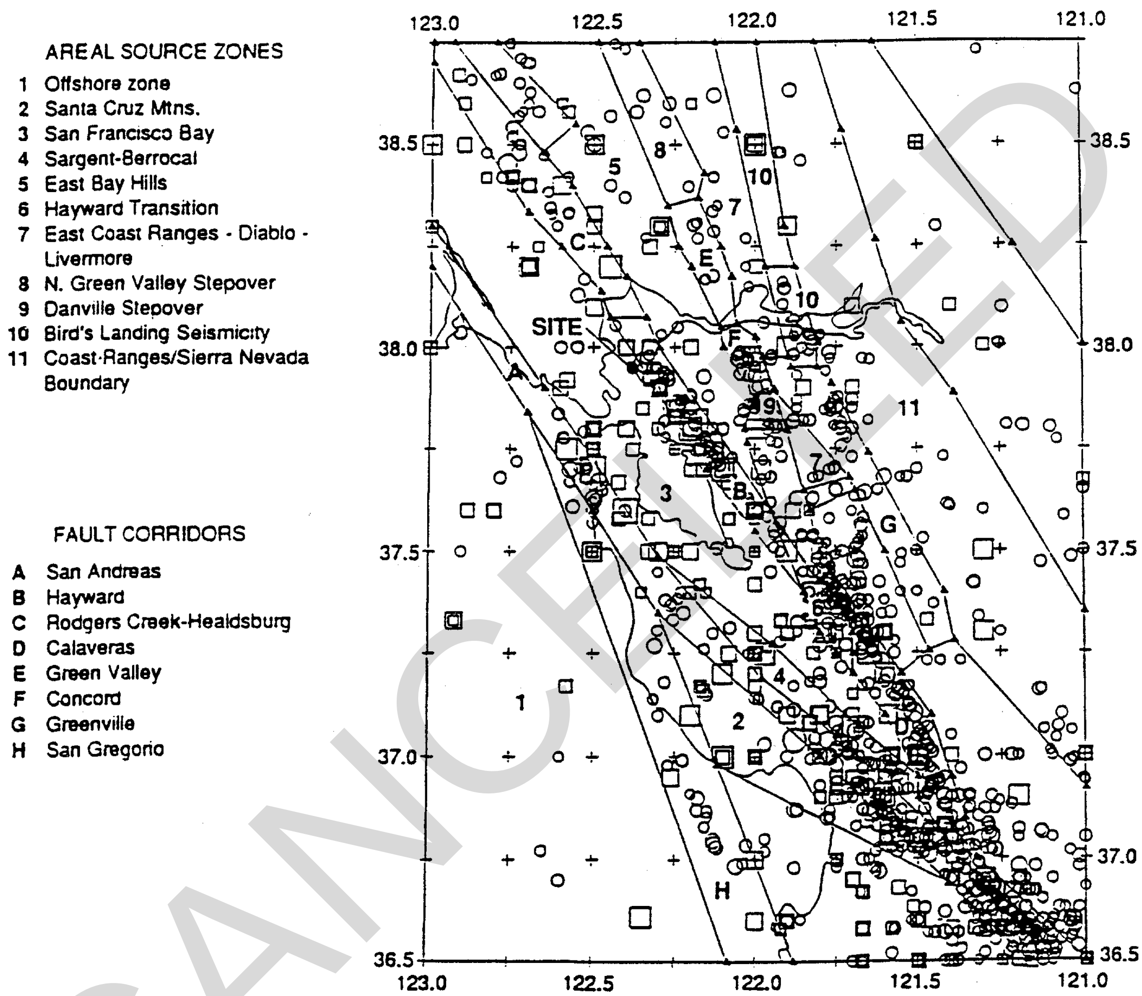


Figure E-11 Map of the San Francisco Bay Area showing independent earthquakes, fault corridors, and areal source zones.

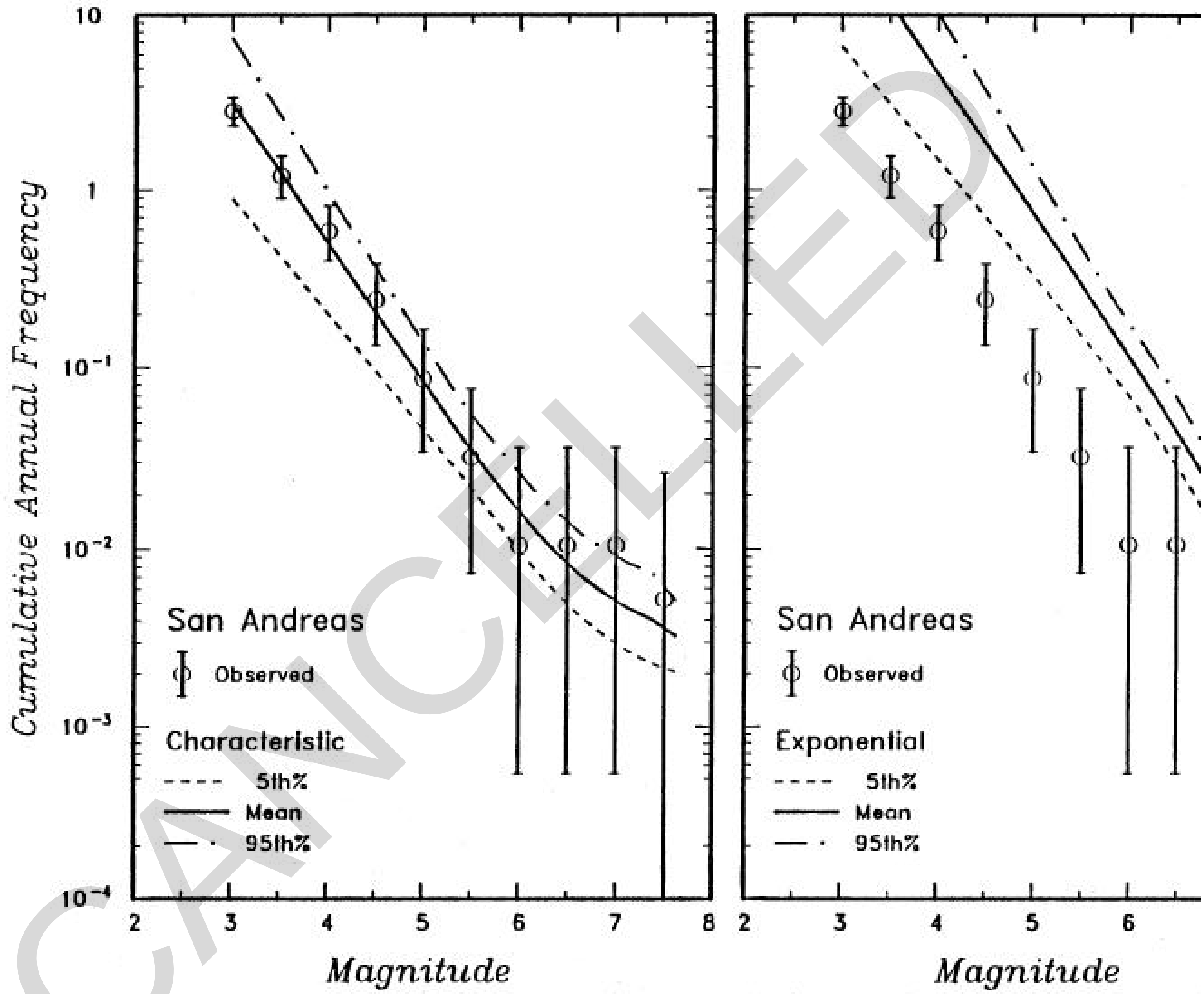


Figure E-12 Comparison of recurrence rates developed from independent seismicity and from fault slip rat fault.

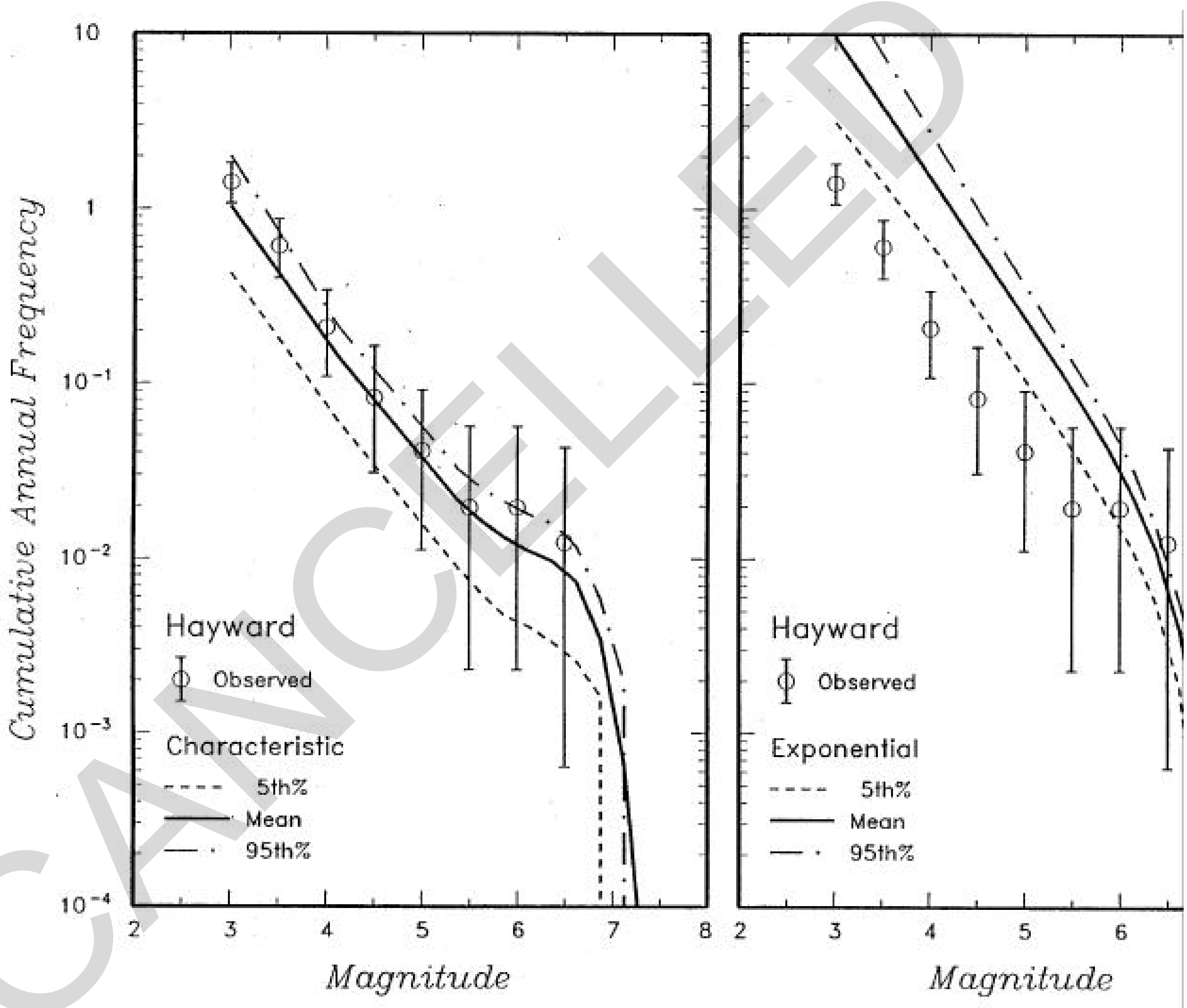


Figure E-13 Comparison of recurrence rates developed from independentseismicity and from fault slip rat

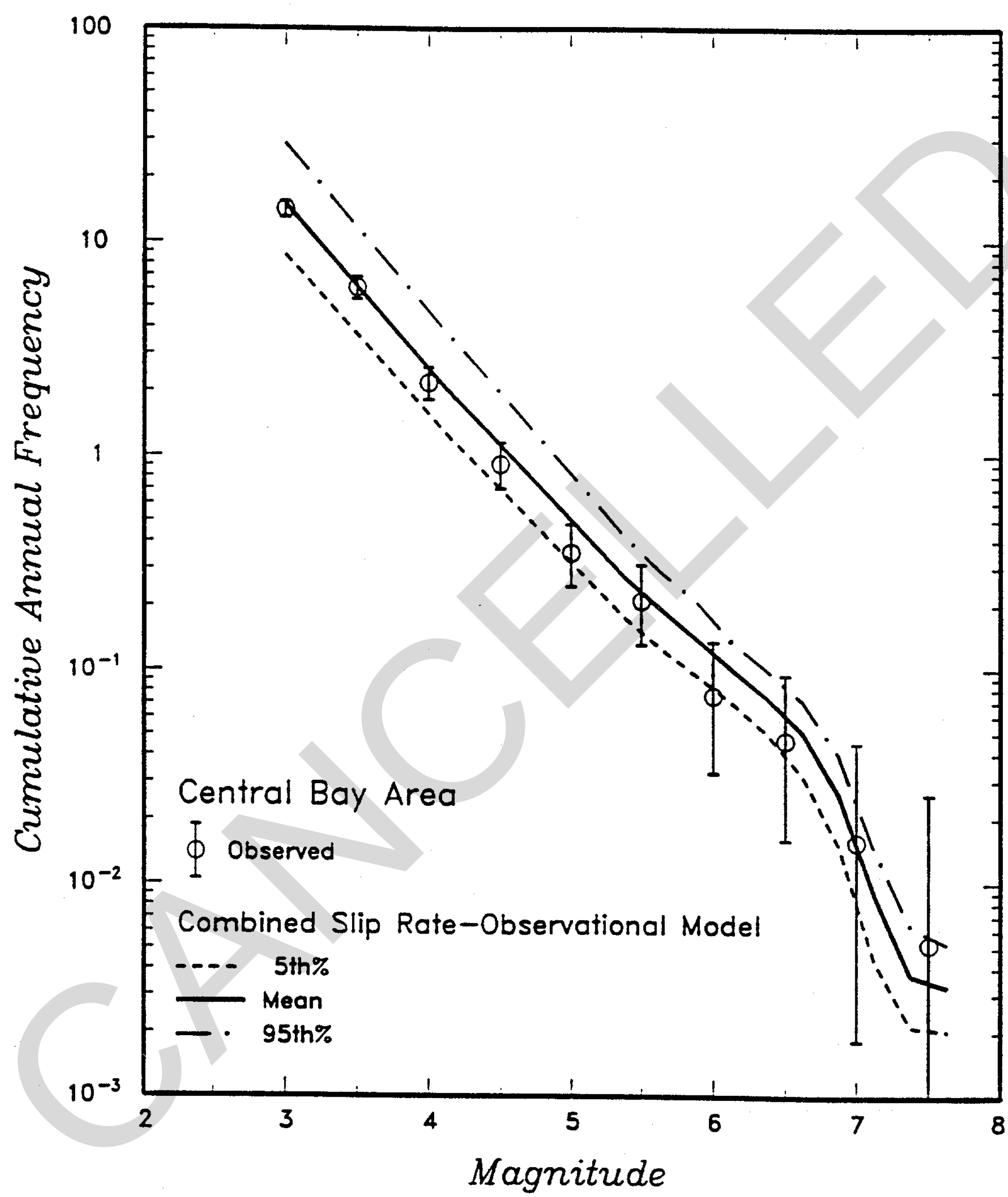


Figure E-14 Comprehensive recurrence model for the Central Bay Area.

geologic data in defining the regional rate of earthquake activity.

(c) Figure E-15 illustrates the generic logic tree for seismic source characterization used for the PSHA. As shown, the study incorporated uncertainty and alternative hypotheses and parameter values for segmentation, maximum rupture length (influencing maximum earthquake magnitude), maximum magnitude estimate correlations, recurrence approach (alternatives of using seismicity data and tectonic convergence rate data for source zones), recurrence rates and b-values, and magnitude distribution model for recurrence assessments (characteristic for faults and characteristic and exponential for area sources).

(2) Ground motion attenuation characterization. Three different sets of rock ground motion attenuation relationships for response spectral acceleration at different periods of vibration (5 % damping) as well as for peak acceleration were utilized. Median values for these relationships (for magnitudes 5, 6, and 7) are illustrated in Figure E-16 for peak acceleration and spectral acceleration at two periods of vibration. Each set of these relationships also has its associated model of uncertainty (dispersion) around the median curves. The dispersion relationships for the preferred model (designated Caltrans, 1991, in Figure E-16 are summarized in Table E-1. (The attenuation model designated Caltrans, 1991, is the relationship of Sadigh et al., 1993). Note that this model predicts increasing dispersion for decreasing magnitude and increasing period of vibration, based on analysis of ground motion data. The three sets of attenuation relationships comprise three additional branches that are added to the logic tree in Figure E-15.

(3) PSHA Results

(a) Typical results of the PSHA are illustrated in Figure E-17 in terms of the hazard curves obtained for peak acceleration and response spectral acceleration at two periods of vibration. The distribution about the mean hazard curve represents the uncertainty in seismic source characterization and ground motion attenuation characterization modeled in the logic tree.

(b) Figure E-18 shows the contributions of different seismic sources to the hazard (sources are shown in Figures E-10 and E-11). As shown, the Hayward fault, which is closest to the site, dominates the hazard for PGA and spectral values at low periods of vibration, but the San Andreas fault contribution increases with increasing vibrational period (reflecting the potential for larger magnitude earthquakes on the San Andreas fault than on the Hayward fault and the relatively greater influence of magnitude on long-period motions than short-period motions).

(c) Magnitude contributions to the mean hazard curves are illustrated in Figure E-19. The contributions of higher magnitudes increase both with increasing period of vibration and with increasing return period (RP).

(d) Analyses of two of the components of the seismic hazard model that contribute to the uncertainty in the hazard curves are illustrated in Figures E-20 and E-21. From Figure E-20 it can be seen that much of the uncertainty in the hazard curves is associated with uncertainties as to the appropriate attenuation relationship. The uncertainty in the hazard associated with different models of earthquake recurrence for the San Andreas fault (different segmentation models) (Figure E-21) is small, particularly at lower frequencies of exceedance.

(e) Equal hazard response spectra (expressed in the form of tripartite plots) constructed from the mean hazard results are shown in Figure E-22 for return periods varying from 100 to 2000 years.

c. Site in Illinois

(1) Seismic Source Characterization. The site location is shown in Figure E-23 and is in southern Illinois on the Ohio River. The dominant source zone for this site is the Iapetan Continental Rifts source zone (ICR), which represents an interconnected system of partially developed and failed continental rifts that lie within the mid-continent region of the United States and includes the New Madrid source zone (NSZ) where the large 1811 and 1812 earthquakes occurred. The extent of ICR is shown by the heavy line in Figure E-23 along with source zones outside ICR and the historical seismicity. Modeling of earthquake recurrence within the dominant ICR can be summarized as follows:

(a) The recurrence rate for large (1811-1812 type) earthquakes in NSZ is modeled based on paleoseismic evidence. As shown in Figure E-24, the paleoseismic-determined rate of these earthquakes exceed the rate of large earthquakes predicted from the historical seismicity.

(b) The recurrence rate for smaller earthquakes in ICR is determined by the historical seismicity. Two basic models are used within a logic tree framework for defining subzones for characterizing

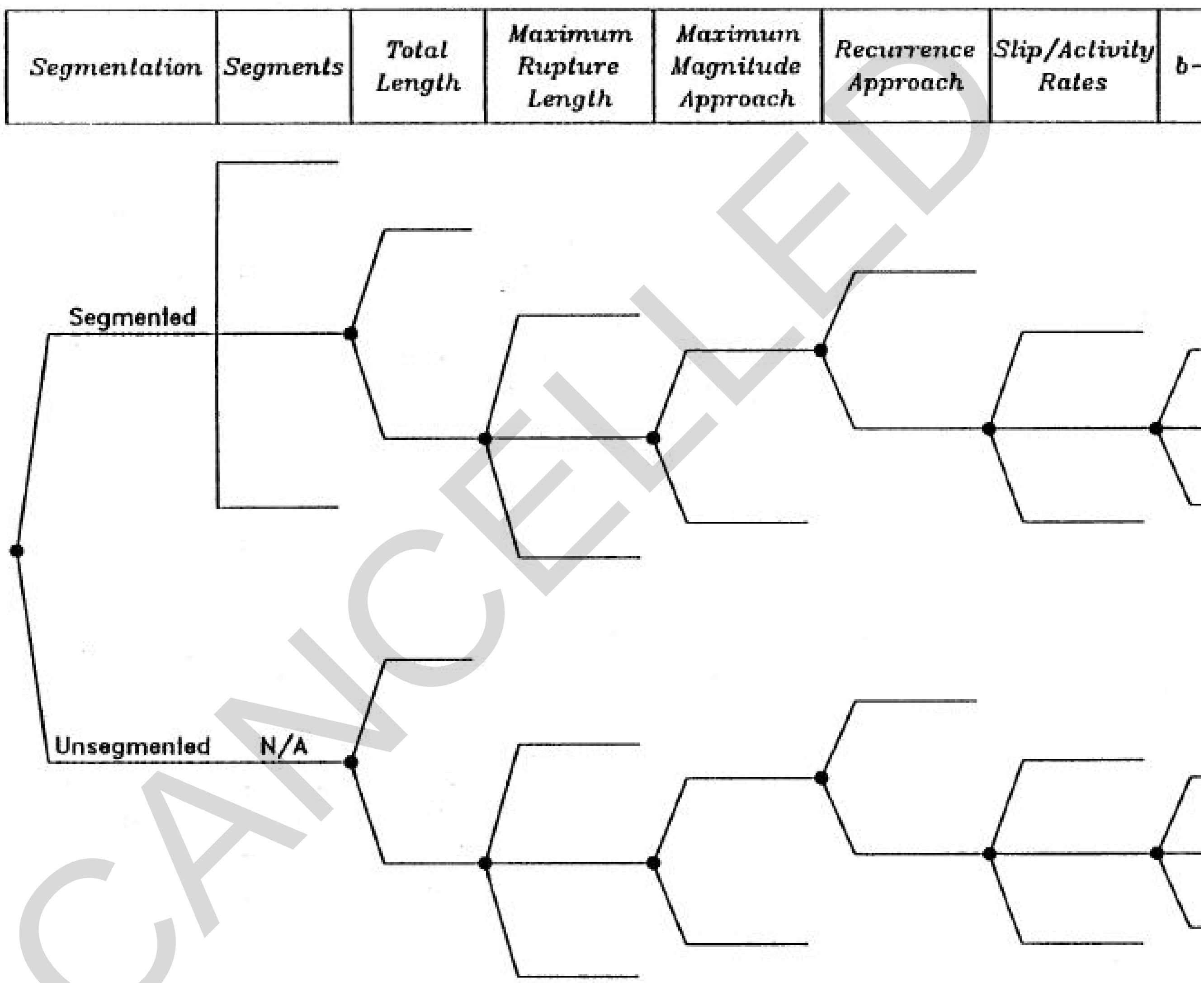


Figure E-15 Generic logic tree used to characterize seismic sources for probabilistic seismic hazard analysis.

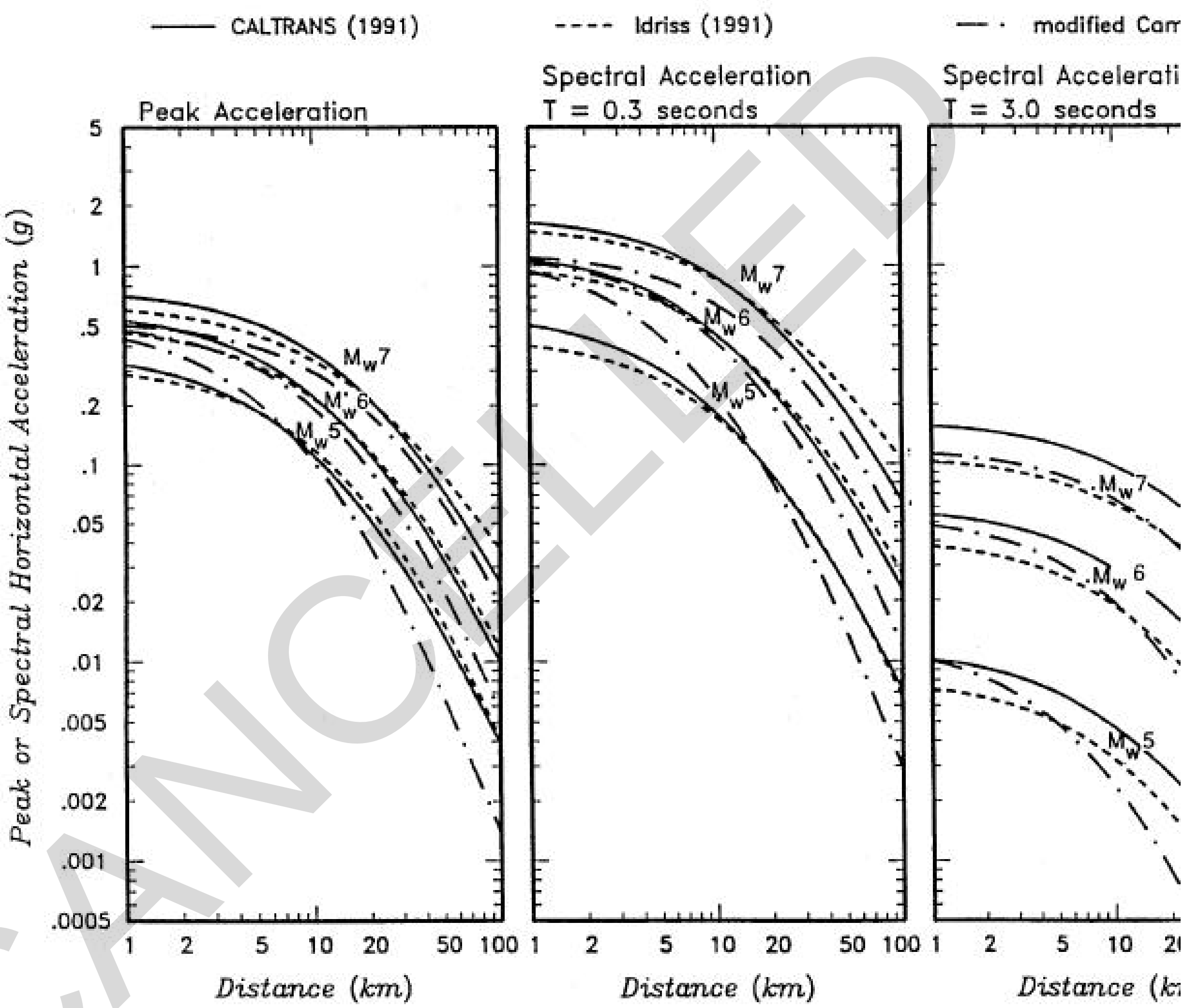


Figure E-16 Ground motion attenuation relationships.

Table E-1 Dispersion relationships for horizontal rock motion from the attenuation relationships of Sadigh et al. (1993).

Ground Motion Parameter	**Period**	**Sigma [ln(y)]**
Peak Ground Acceleration	--	1.39 - 0.14*M; 0.38 for M ≥ 7.21
Response Spectra Acceleration	0.05	1.39 - 0.14*M; 0.38 for M ≥ 7.21
Response Spectra Acceleration	0.07	1.40 - 0.14*M; 0.39 for M ≥ 7.21
Response Spectra Acceleration	0.09	1.40 - 0.14*M; 0.39 for M ≥ 7.21
Response Spectra Acceleration	0.10	1.41 - 0.14*M; 0.40 for M ≥ 7.21
Response Spectra Acceleration	0.12	1.41 - 0.14*M; 0.40 for M ≥ 7.21
Response Spectra Acceleration	0.14	1.42 - 0.14*M; 0.41 for M ≥ 7.21
Response Spectra Acceleration	0.15	1.42 - 0.14*M; 0.41 for M ≥ 7.21
Response Spectra Acceleration	0.17	1.42 - 0.14*M; 0.41 for M ≥ 7.21
Response Spectra Acceleration	0.20	1.43 - 0.14*M; 0.42 for M ≥ 7.21
Response Spectra Acceleration	0.24	1.44 - 0.14*M; 0.43 for M ≥ 7.21
Response Spectra Acceleration	0.30	1.45 - 0.14*M; 0.44 for M ≥ 7.21
Response Spectra Acceleration	0.40	1.48 - 0.14*M; 0.47 for M ≥ 7.21
Response Spectra Acceleration	0.50	1.50 - 0.14*M; 0.49 for M ≥ 7.21
Response Spectra Acceleration	0.75	1.52 - 0.14*M; 0.51 for M ≥ 7.21
Response Spectra Acceleration	1.00	1.53 - 0.14*M; 0.52 for M ≥ 7.21
Response Spectra Acceleration	>1.00	1.53 - 0.14*M; 0.52 for M ≥ 7.21

Note: Sigma [ln(y)] is the standard deviation of the natural logarithm of the respective ground motion parameter, y. M is the earthquake moment magnitude.

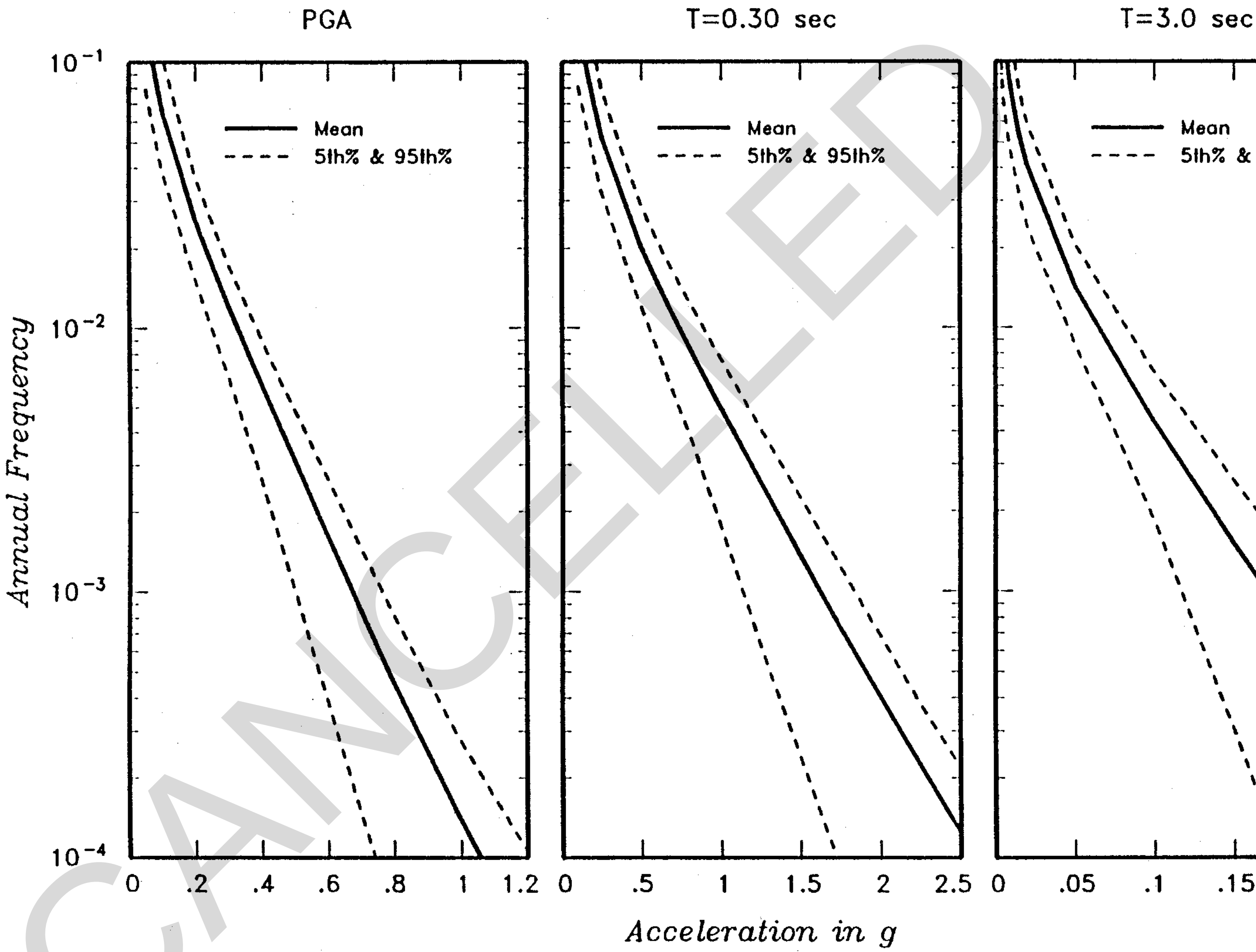


Figure E-17 Mean, 5th, and 95th percentile hazard curves for the site for peak acceleration and 5 percen accelerations at periods of 0.3 and 3.0 seconds.

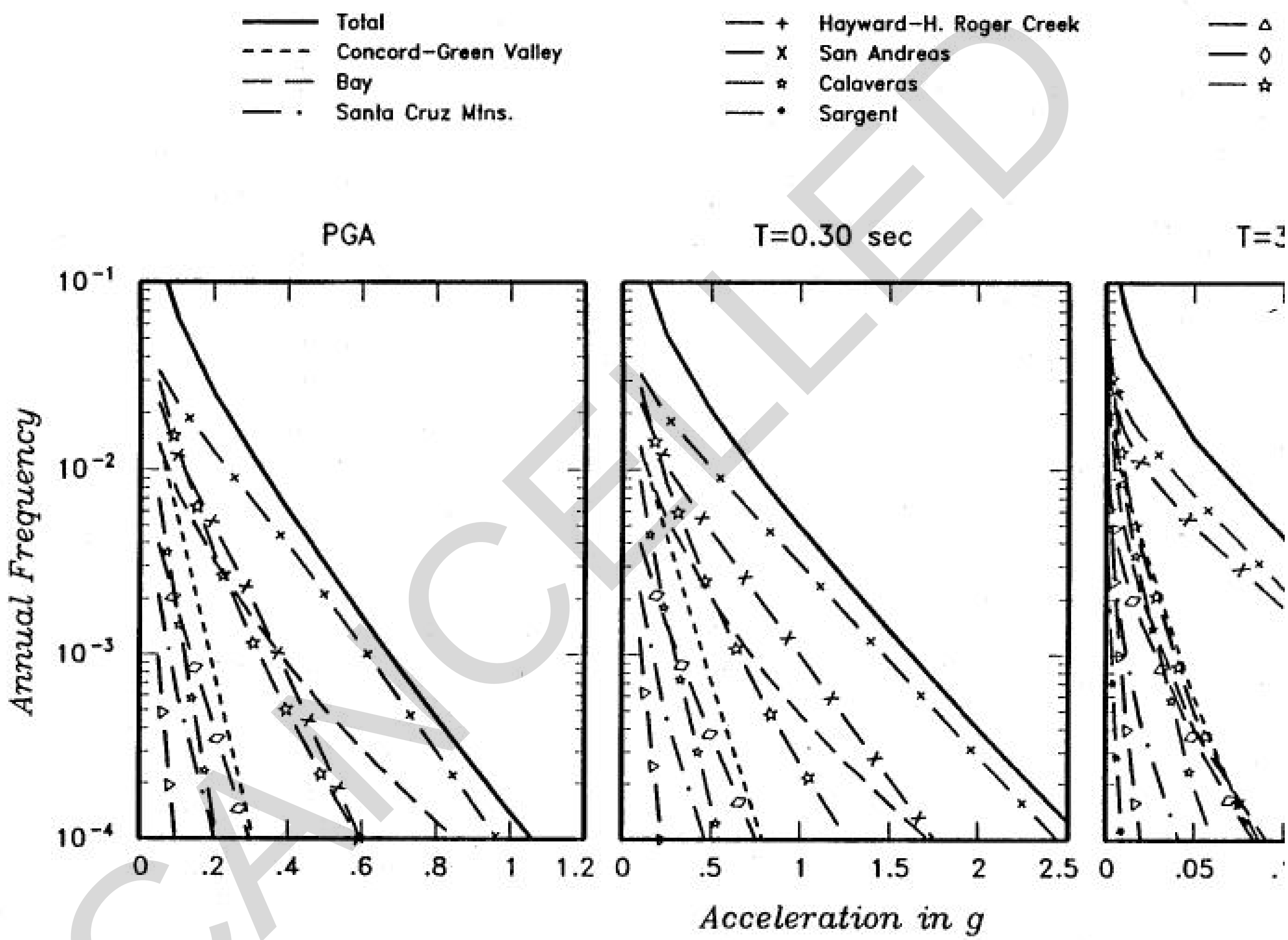


Figure E-18 Contributions of various sources to mean hazard at the site.

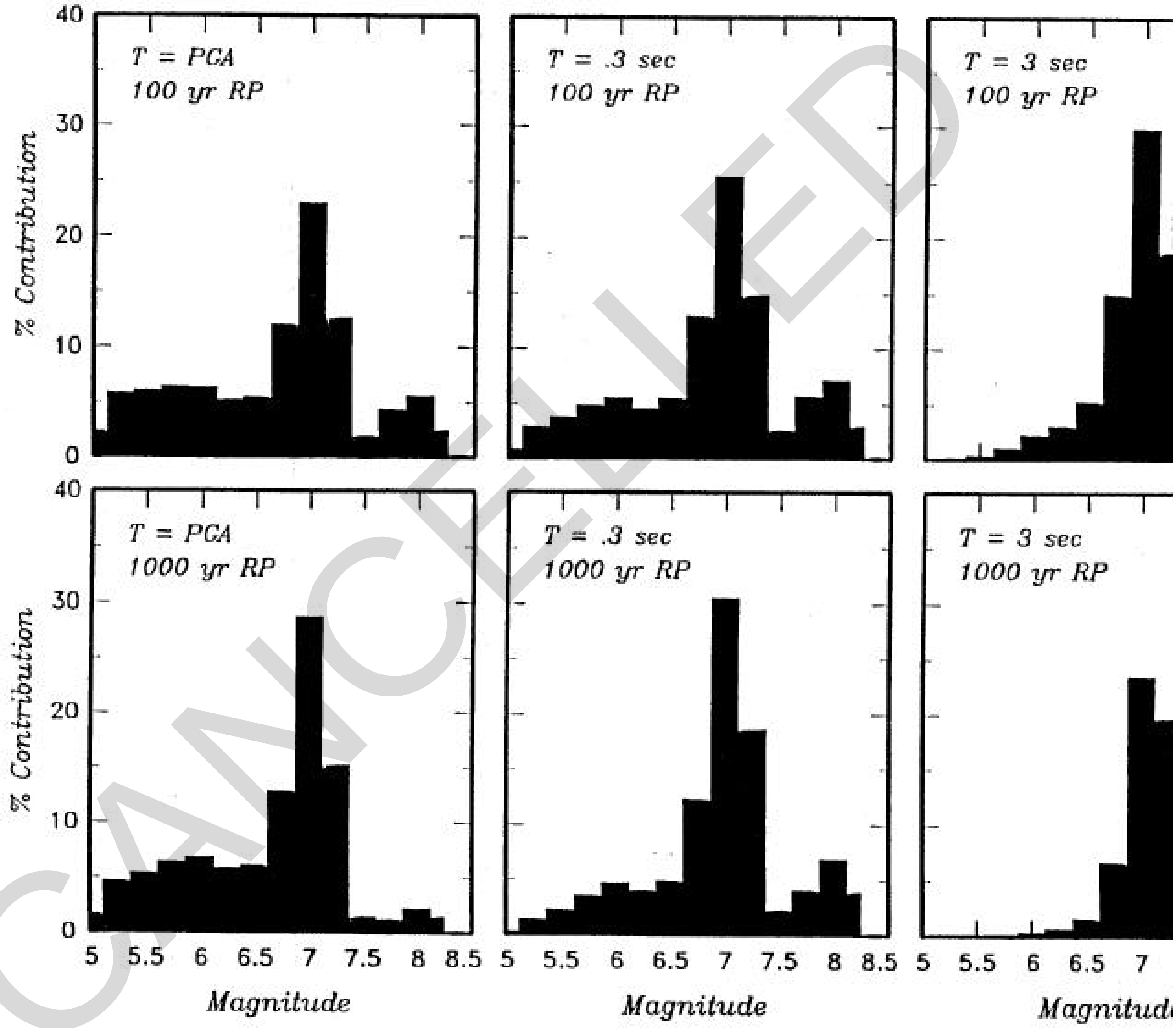


Figure E-19 Contributions of events in various magnitude intervals to the mean hazard at the site.

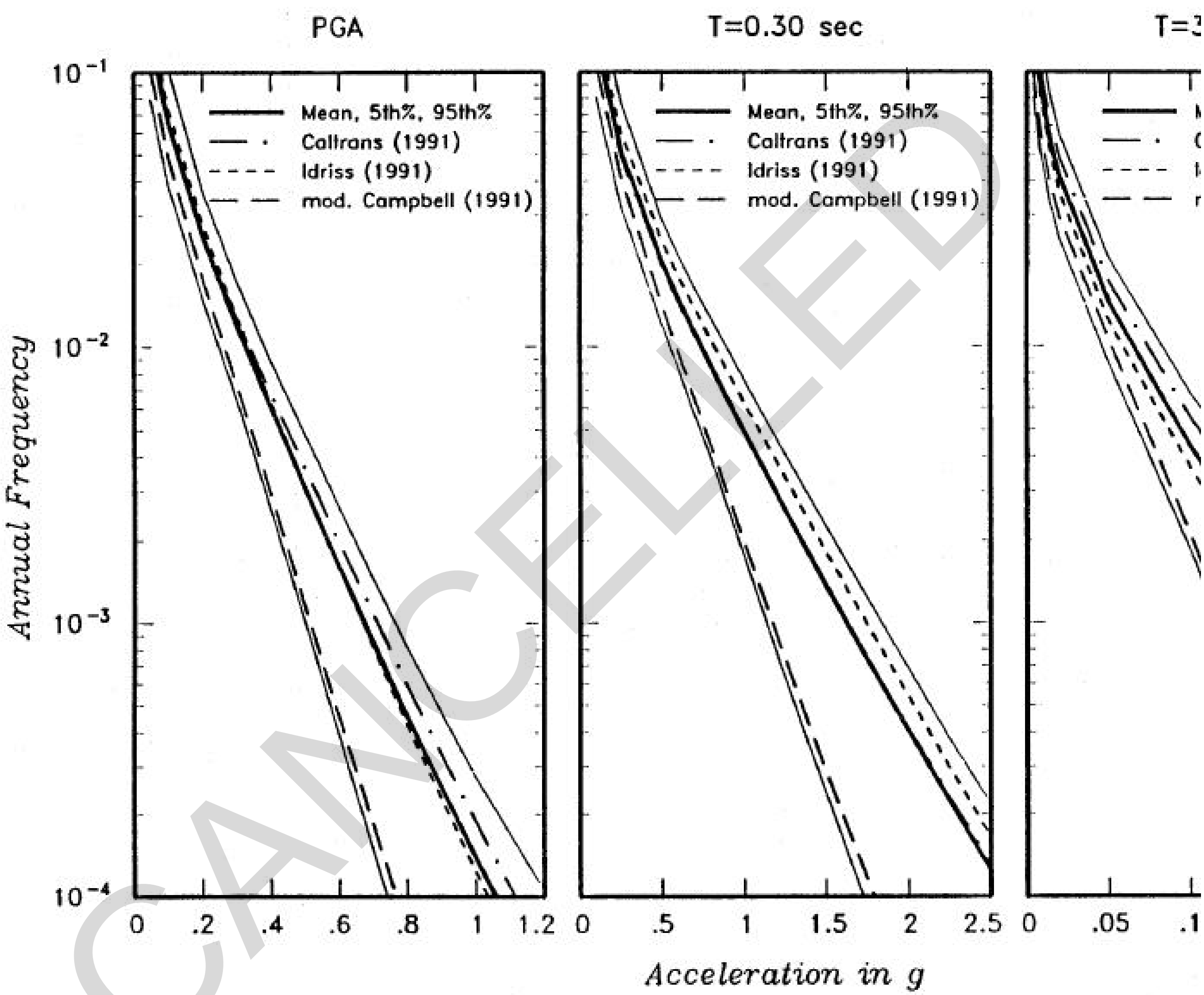


Figure E-20 Sensitivity of mean hazard at the site from the choice of attenuation model.

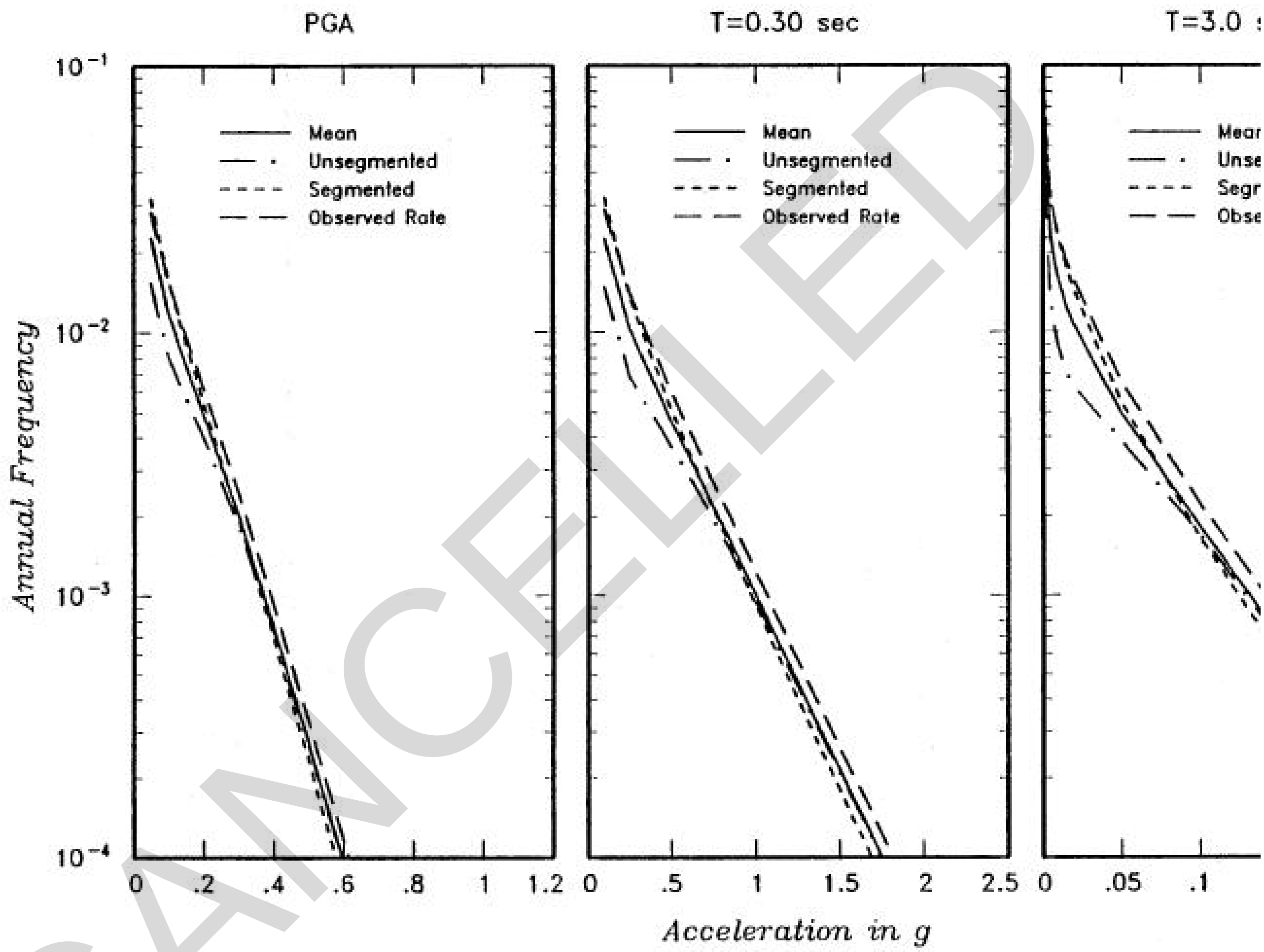
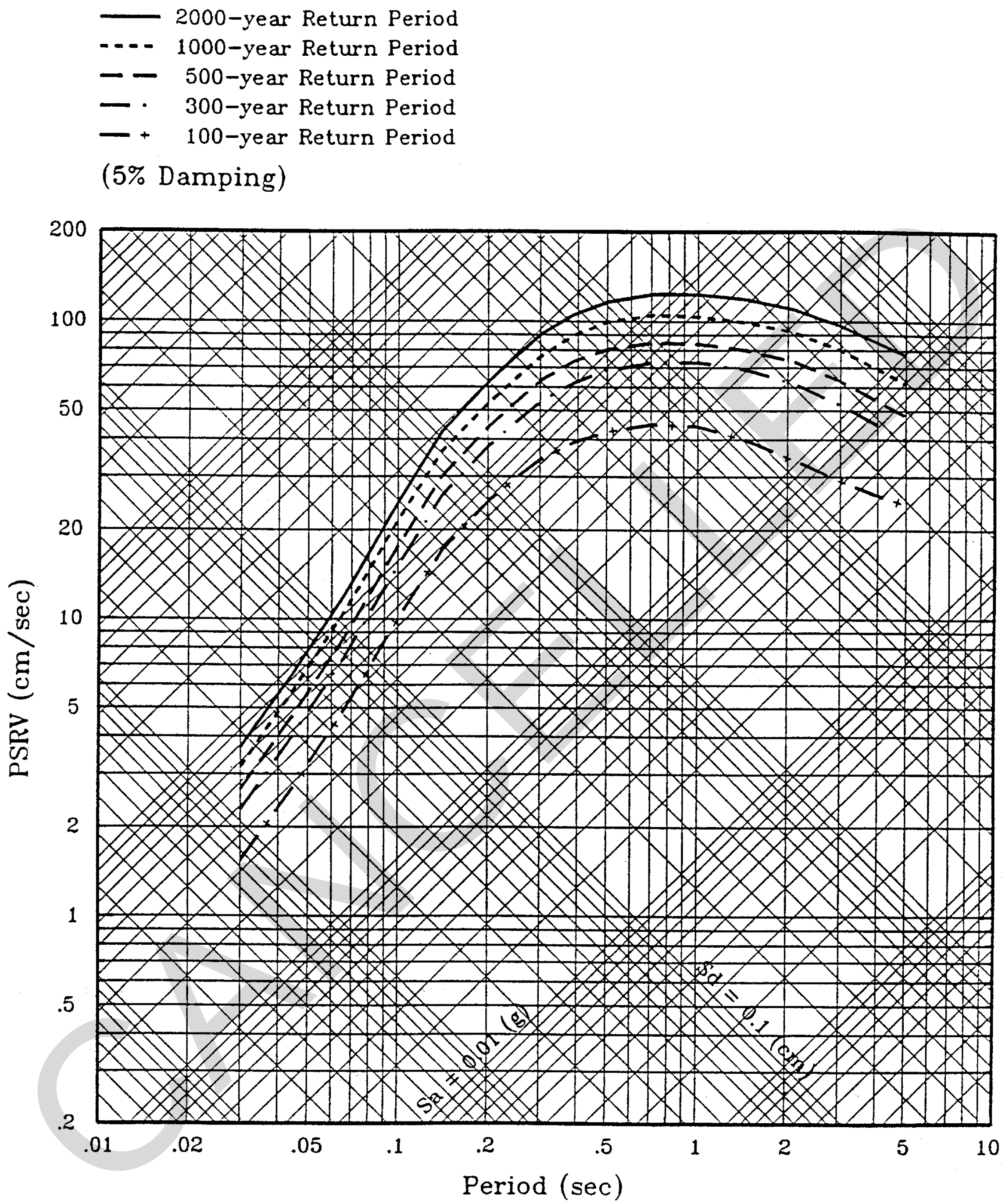


Figure E-21 Sensitivity of mean hazard at the site from the San Andreas fault only due to choice of earthqu
the San Andreas fault.



1 cm = 0.4 inches

Figure E-22 Equal-hazard pseudo-velocity response spectra for the site (5 percent damping).

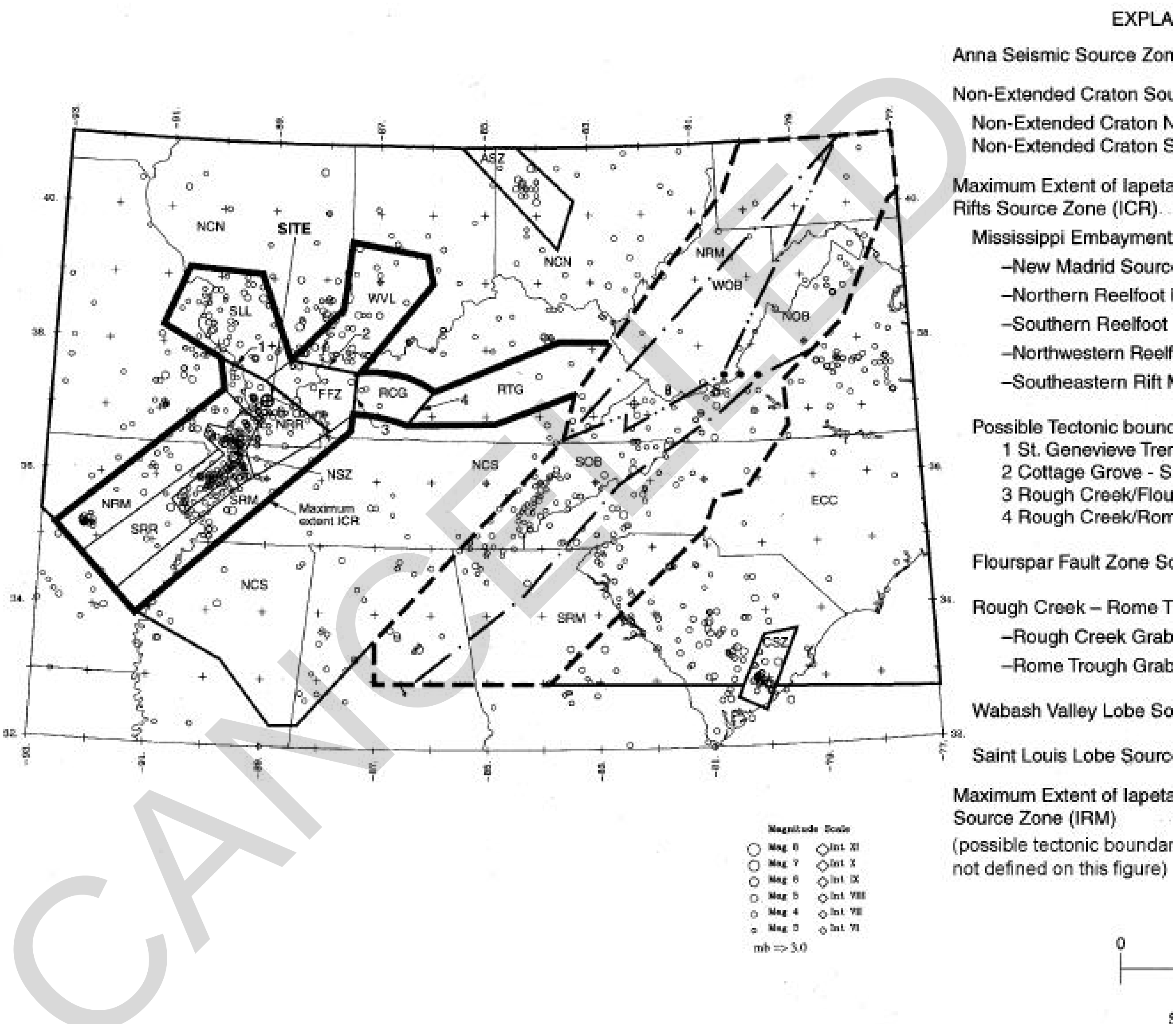


Figure E-23 Seismic source zonation model for the central and southeastern United States.

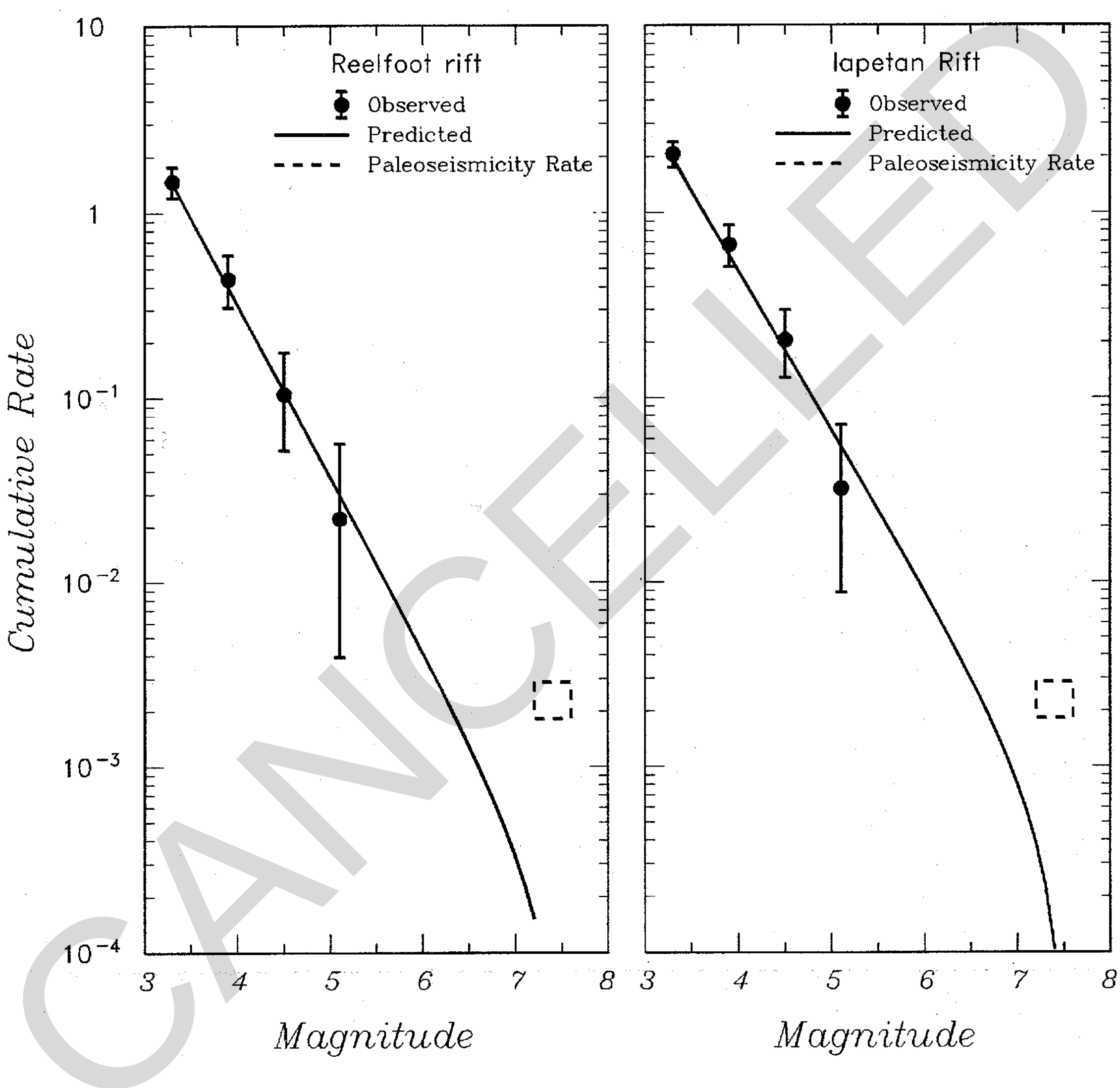


Figure E-24 Comparison of historical andpaleoseismic recurrence estimates for the Reelfoot Rift and Iapetan Rift Seismic Zone.

recurrence within ICR: a seismicity-based model (given a weight of 0.25); and a geology-based model (given a weight of 0.75). The seismicity-based model divides ICR into cells of one-half degree latitude and longitude and calculates recurrence rates based on the historical seismicity in the cell. Different degrees of smoothing of seismicity rates and b-values among adjacent cells is accomplished using the methodology developed by EPRI (1988). In the geology-based model, Zone ICR is divided into subzones as indicated in Figure E-23. Different combinations of subzones are defined in a logic tree approach. The possible combinations are controlled in part by the presence or absence of four possible tectonic boundaries within the ICR (Figure E-23) and the assessed likelihood that these features represent fundamental boundaries that control the distribution, rate, and maximum magnitudes of seismicity. The logic tree for weights assigned to these boundaries is shown on Figure E-25. Thirty alternativesubzonations (not shown herein) of ICR result from the logic tree of Figure E-25. Within each subzone of each alternative, seismicity rates are determined based on the seismicity within the subzone and assuming the rate is uniform within the subzone.

(c) Probabilistic distributions of maximum earthquake magnitudes are also part of the source model logic tree. These probabilistic distributions were determined using the methodology developed by EPRI (Johnston et al., 1994) that utilized worldwide data bases to assess maximum earthquake magnitudes in stable continental regions (like the eastern United States (EUS)) where active faults have not been identified and therefore maximum magnitude cannot be estimated on the basis of fault dimensions (as is done in the western United States (WUS)). However, for the New Madrid zone, maximum earthquake magnitudes were estimated on the basis of both (1) estimated rupture models by Johnston (1996) and Gomberg and Ellis (1994) and correlations of magnitude with rupture dimensions, and (2)estimates of magnitudes of the 1811-1812 earthquakes by Johnston (1996).

(2) Ground Motion Attenuation Characterization.

(a) It was desired to estimate ground motions on rock at the site. Two attenuation relationships applicable to hard rock in the EUS for horizontal peak ground acceleration and response spectral accelerations of ground motions at different periods of vibration were used. The relationships are those of EPRI (1993), (later published as Toro et al., 1997) and Atkinson and Boore (1995) (later published as Atkinson andBoore, 1997).

(b) The relationship for response spectral acceleration of EPRI (1993) extends to periods as long as 1 second, and that of Atkinson and Boore extends to a period of 2 seconds. The EPRI (1993) relationship was extrapolated to a period of 2 seconds. This was accomplished by extrapolating the coefficients of the attenuation relationship and examining the reasonableness of the resulting spectral prediction. The smooth quadratic form of the relationship of Atkinson andBoore (1995) underestimates their simulations of longer period ground motions at distances beyond 100 km (62 miles). Therefore, their relationships were modified at periods greater than 0.5 second to result in ground motion estimates closer to the simulation results. Plots of the attenuation relationships of EPRI (1993) and Atkinson andBoore (1995) for peak ground acceleration and response spectral accelerations at 1.0 second are presented in Figure E-26. The modifications to the 1-second motion at distances greater than 100 km (62 miles) can be seen in the figure. The plots in Figure E-26 clearly indicate the distinctive differences between the two eastern United States attenuation relationships: the Atkinson andBoore (1995) relationships result in higher spectral values than those of EPRI (1993) for peak ground acceleration and for short-period response spectral accelerations (less than about 0.2 second period), but lower values than those of EPRI (1993) at longer periods.

(c) In the hazard analysis, the relationship of EPRI (1993) was given a higher weight (0.67) than that of Atkinson and Boore (1995) (0.33). The reason for this judgment was that the EPRI (1993) relationship resulted from an EPRI study that involved input from a number of ground motion experts and thus could be viewed as having achieved a certain degree of consensus regarding the model. The practical effect of higher weighting on the EPRI (1993) model is to increase longer period ground motions and reduce short-period ground motions.

(3) PSHA Results. Hazard curves obtained from the analysis for peak ground acceleration and response spectral acceleration at two periods of vibration are shown in Figure E-27. The uncertainty bands around the mean curves, reflecting the alternative seismic source models and attenuation relationships incorporated into the logic tree, are shown in the figure. The contributions to the hazard are almost entirely from Zone ICR. Figure E-28 shows contributions within ICR from large New Madrid Zone earthquakes with rates defined by paleoseismic data (dashed-dotted line) and smaller earthquakes defined by seismicity (dashed line). It can be seen that the smaller earthquakes dominate hazard at higher frequencies (probabilities) of exceedance and the larger, 1811-1812-type earthquakes dominate at lower frequencies (probabilities) of exceedance. Figure E-29 compares the hazard obtained from geology-based and seismicity-based

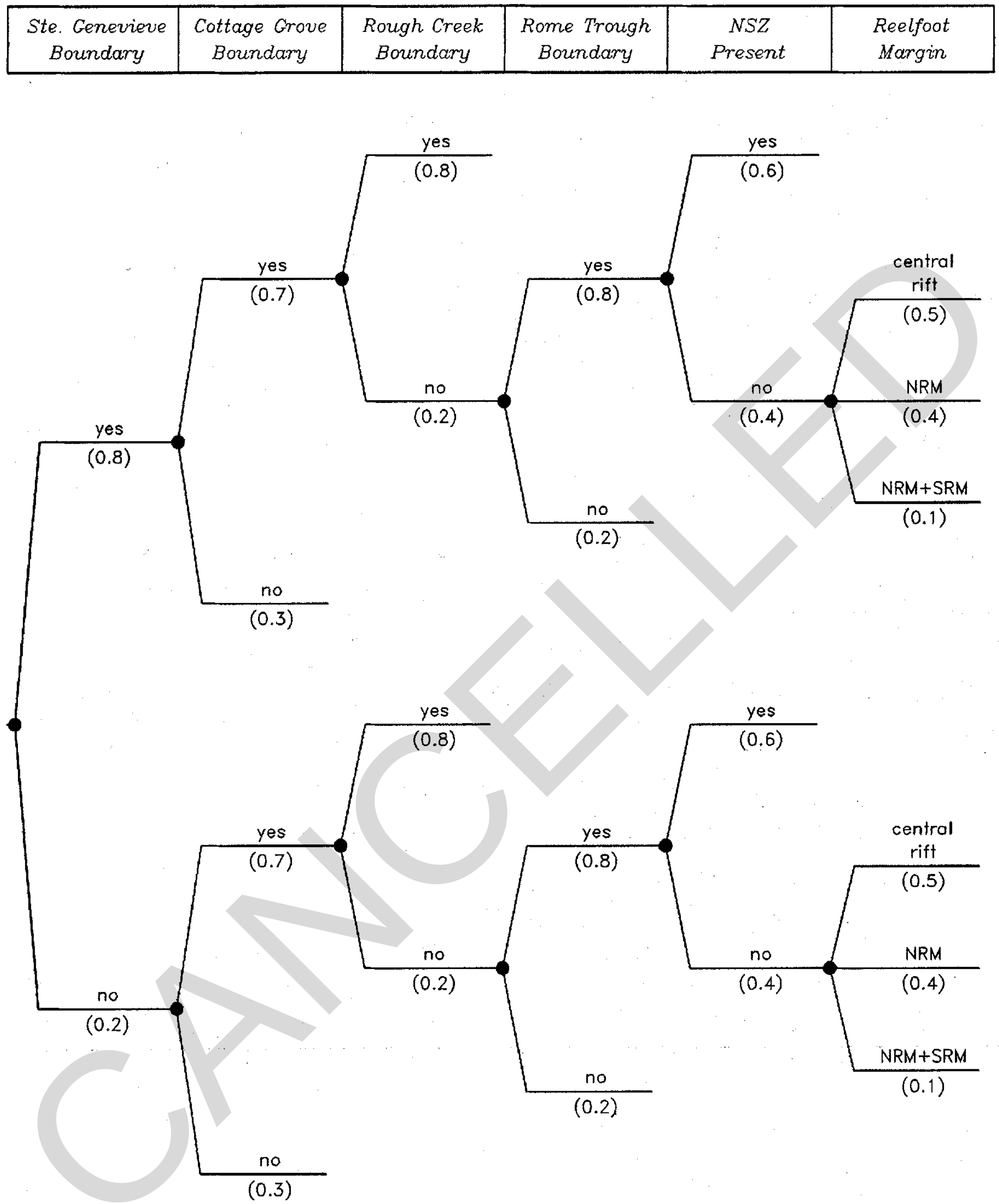


Figure E-25 Logic tree showing relative weights assigned to boundaries separating potential subzones of the Iapetan Rift Seismic Zone.

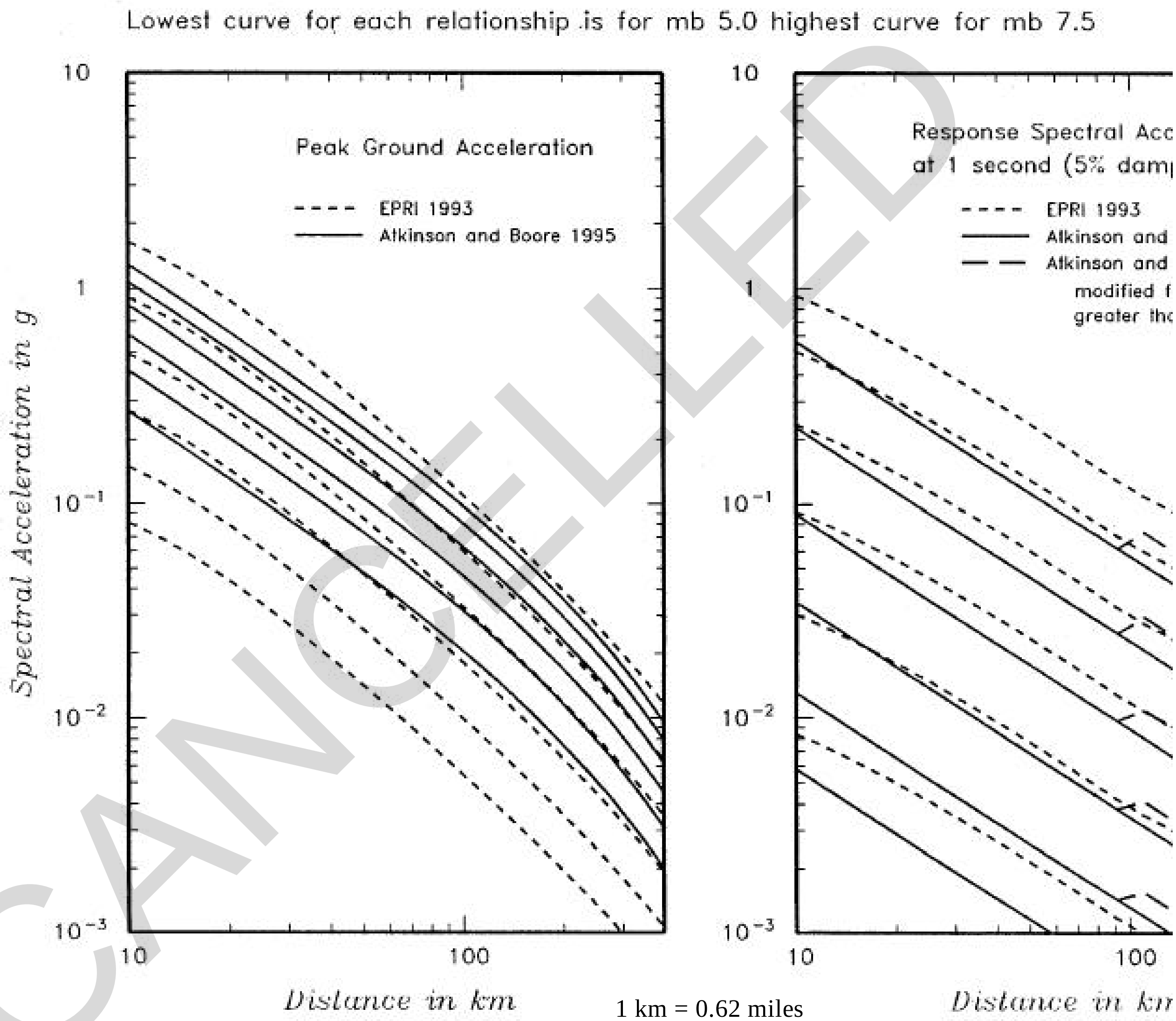


Figure E-26 Attenuation curves of Atkinson and Boore (1995) and EPRI (1993) for peak ground accelerat acceleration at 1.0 second period.

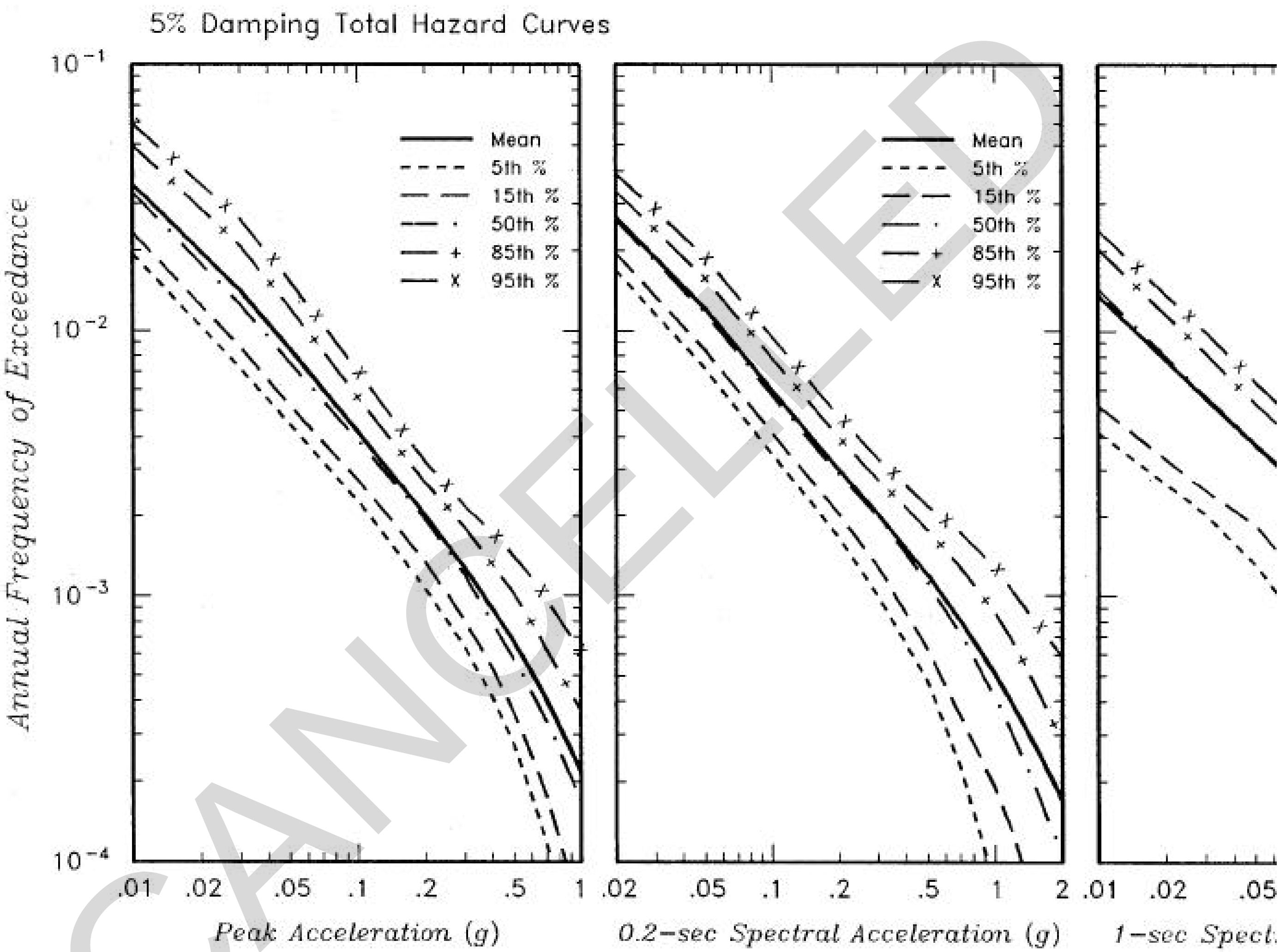


Figure E-27 Computed hazard for peak ground acceleration and response spectral accelerations at 0.2 and

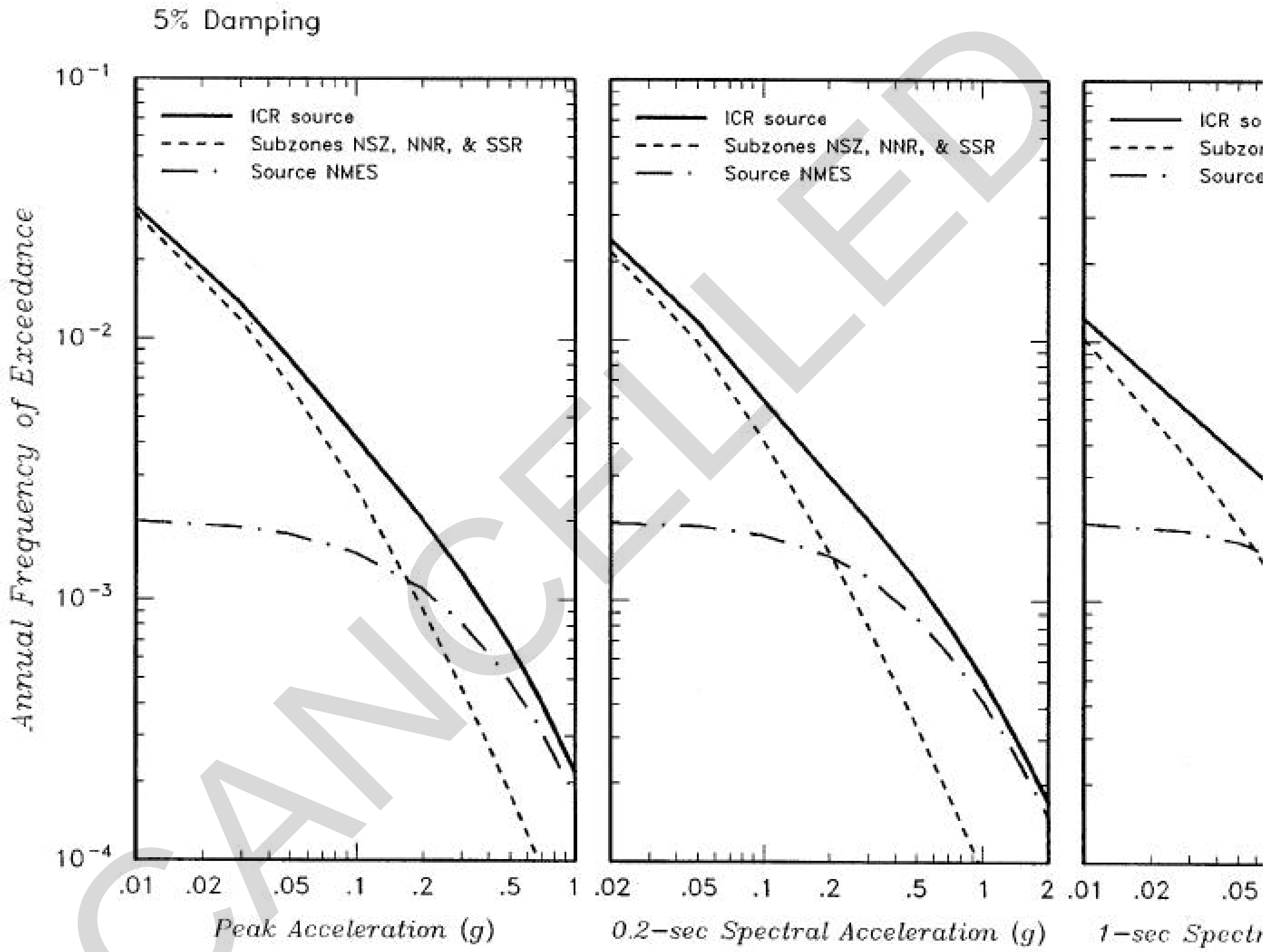


Figure E-28 Contributions of components of the ICR source to the hazard.

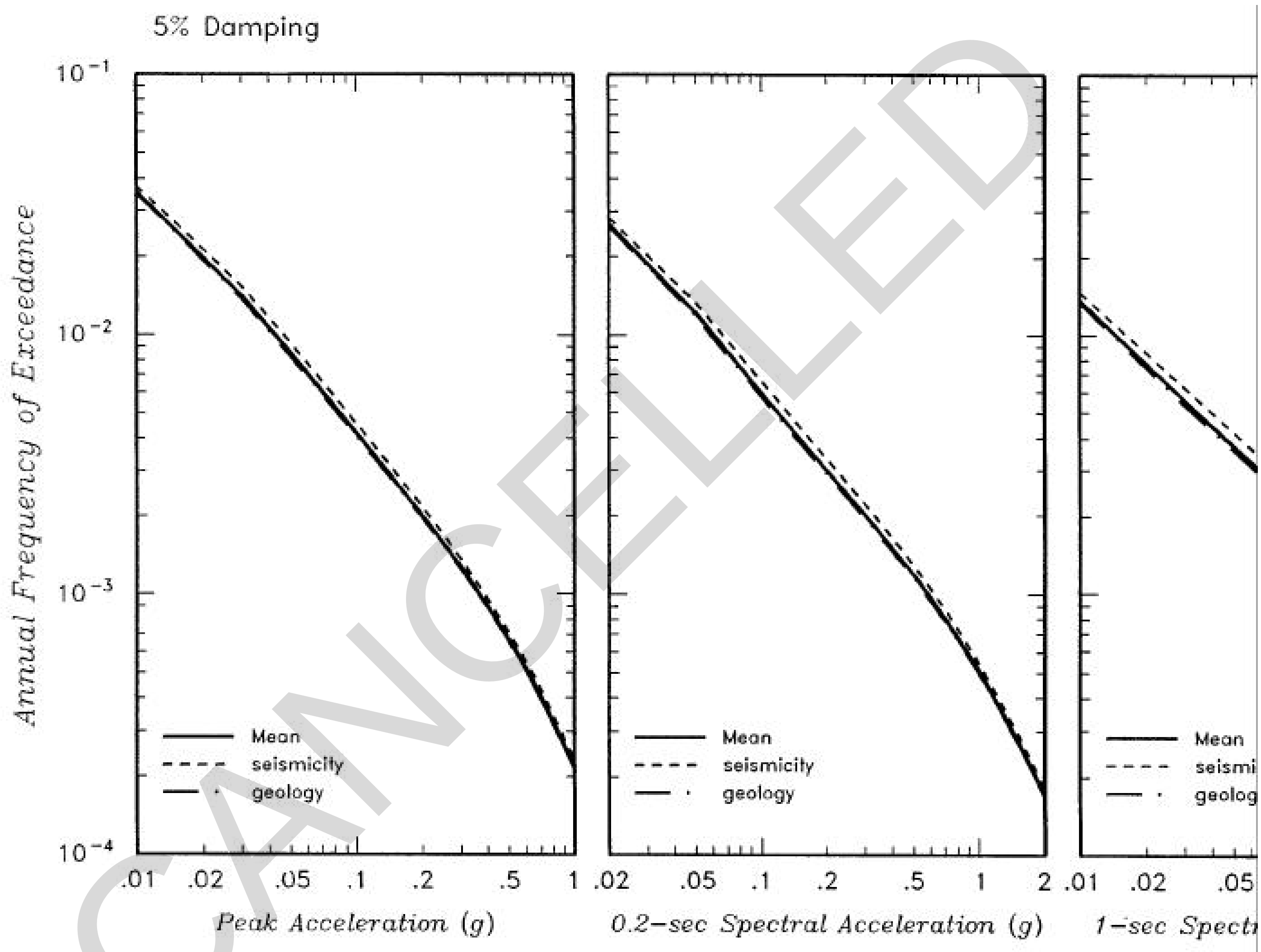


Figure E-29 Comparisons of hazard from the geology andseismicity-based models.

models. It can be seen that, for this site, the two modeling approaches lead to almost identical results. Equal-hazard response spectra obtained from the mean hazard results for all the periods of vibration analyzed for are shown in Figure E-30 for return periods varying from 144 to 10,000 years.

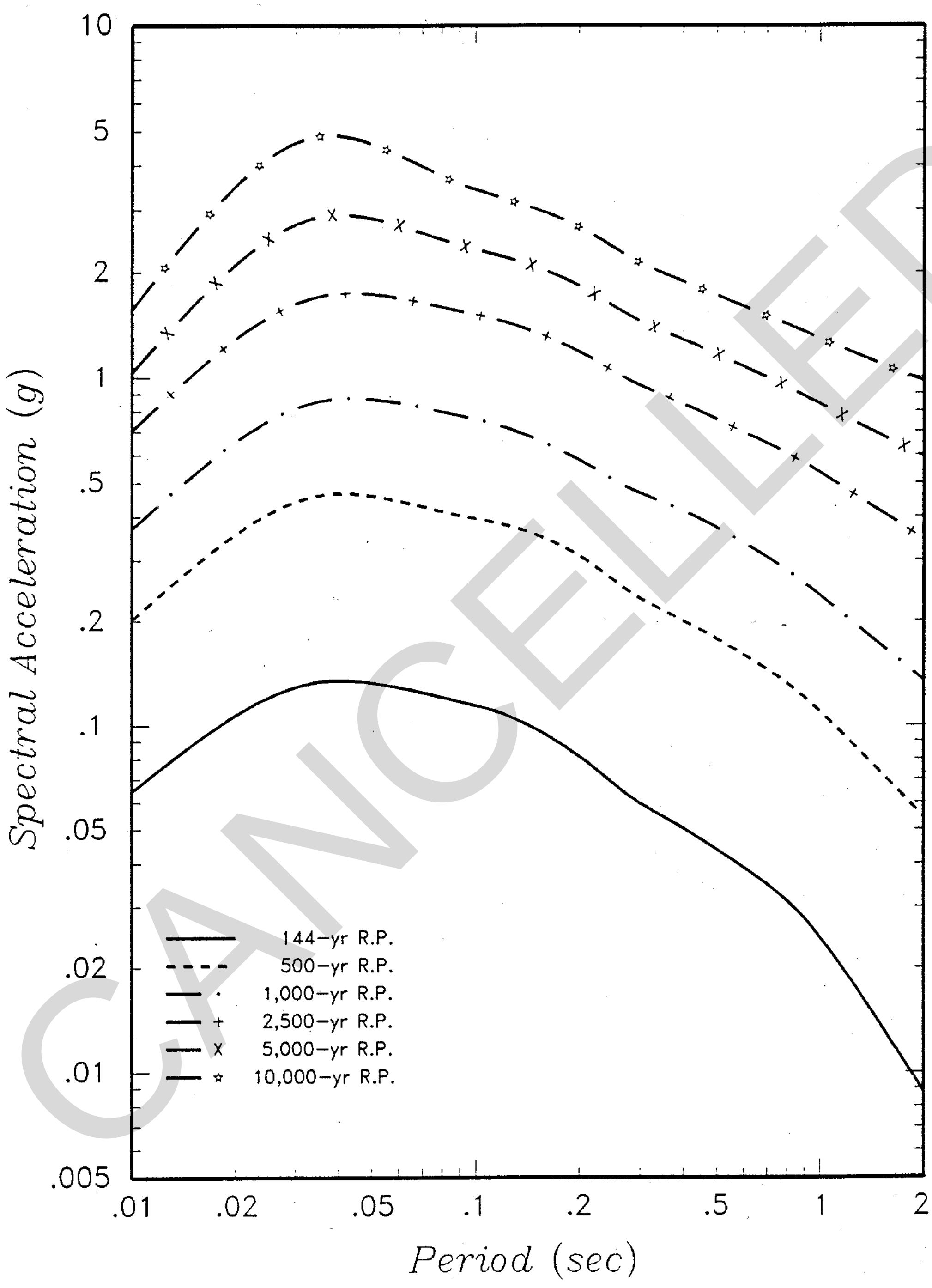


Figure E-30 Equal hazard response spectra (5% damping).

APPENDIX F
GEOLOGIC HAZARDS EVALUATIONS

F-1. Introduction

This appendix describes guideline procedures for the evaluation of seismic-geologic site hazards, other than the ground shaking hazard. These hazards include: (a) surface fault rupture; (b) soil liquefaction; (c) soil differential compaction; (d) landsliding; and (e) flooding. The evaluations of the hazards described in this appendix should be carried out by qualified geotechnical professionals. Depending on the hazard, disciplinary expertise in geotechnical engineering, geology, and seismology may be needed.

a. Overview of process for conducting geologic hazards evaluations. The process described herein for seismic-geologic hazards evaluation is a two-step process—screening and evaluation. If a significant hazard is disclosed by this process, then hazard remediation should be developed.

b. Organization of remainder of this appendix. Paragraph F-2 describes and illustrates the geologic hazards. Screening procedures for these hazards are presented in paragraph F-3. The intent in the screening process is to utilize readily available data and criteria to ascertain whether a significant potential for any of the hazards exists at the site. Paragraph F-4 presents hazard evaluation procedures in the event that the screening process results in a conclusion that more detailed evaluation is required to assess the hazard and its significance. Paragraph F-5 provides preliminary information regarding hazard mitigation. Requirements for documentation of the evaluations of geologic hazards are described in paragraph F-6. Examples of geologic hazard evaluations are presented in Appendix G.

F-2. Description of Geologic Hazards

The following paragraphs provide brief descriptions of the seismic-geologic hazards of surface fault rupture, soil liquefaction, soil differential compaction, landsliding, and flooding. Hazard significance in terms of potential ground movements and effects on structures are also summarized.

a. Surface fault rupture. Earthquakes are caused by the sudden slip or displacement along a zone of weakness in the earth's crust, termed a fault. Surface fault rupture is the manifestation of the fault displacement at the ground surface for those cases where the fault slip extends to the ground surface. Generally, fault rupture extends to the ground surface only during moderate- to large-magnitude earthquakes (magnitudes equal to or greater than 6). However, not all moderate-to large-magnitude earthquakes produce fault slip at the ground surface. In some cases, the fault displacement may occur entirely at depth, with little or no apparent permanent surface deformation (e.g., 1989 Loma Prieta, California earthquake of moment magnitude 7.0), or with more subdued or diffuse surface warping and fracturing (as may have accompanied the 1994 Northridge, California earthquake of moment magnitude 6.7).

(1) Mode of fault movement. The mode of surface fault deformation is influenced by the type of faulting. Different types of faults are illustrated in Figure F-1. These types are distinguished by the primary sense of relative displacement between the two sides of the fault. Strike-slip faults are characterized by horizontal movement; reverse or thrust faults involve relative upward movement of the crustal block above the fault plane; normal faults involve relative downward movement of the block above the fault plane; and oblique faults are characterized by both strike-slip and reverse or normal types of movement.

(2) Magnitude of displacements. Surface fault displacements may range from a fraction of an inch to several feet or more depending on the earthquake magnitude, steepness of the fault plane, type of movement, and other factors. These same factors, as well as the nature of the surface geologic materials, also influence how wide the zone of surface rupture is likely to be. Because fault displacements tend to occur abruptly, often across a narrow zone, surface fault rupture can be catastrophic to structures situated directly astride the rupture zone. Figure F-2 illustrates surface fault rupture that occurred in the 1992 Landers, California earthquake. During this moment magnitude 7.3 earthquake, the displacement was mainly of the strike-slip type (see Figure F-1) and the maximum observed horizontal displacement along the fault was 5.5 m (18 feet). Figure F-3 illustrates damage to a structure astride the surface

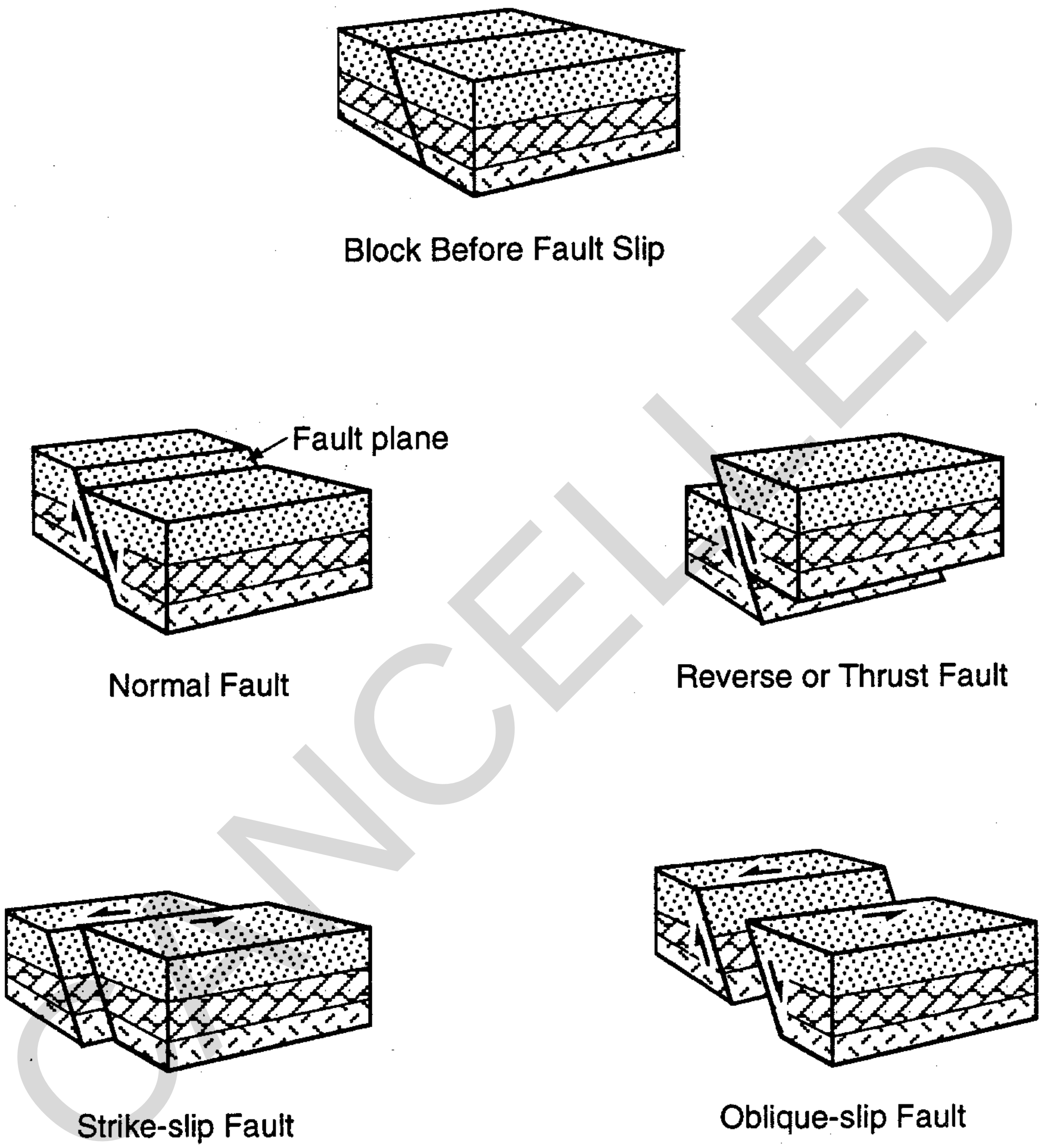


Figure F-1 Types of faults.

Figure F-2 Surface faulting accompanying Landers, California earthquake of June 28, 1992.

Figure F-3 House damaged by ground displacement caused by surface faulting accompanying the San Fernando, California earthquake of February 9, 1971.

fault rupture of the 1971 San Fernando, California, earthquake (moment magnitude 6.6), which was of the reverse- or thrust-fault type (see Figure F-1). More than 1.8 m (6 feet) of combined vertical and horizontal displacement occurred along the surface trace of the fault during the San Fernando earthquake.

b. Soil liquefaction. Soil liquefaction is a phenomenon in which a soil deposit below the groundwater table loses a substantial amount of strength due to strong earthquake ground shaking. The reason for the strength loss is that some types of soil tend to compact during earthquake shaking and this tendency for compaction will induce excess pore water pressures which, in turn, causes strength reduction in the soil. Recently deposited (i.e. geologically young) and relatively loose natural soils and uncompacted or poorly compacted fills are potentially susceptible to liquefaction. Loose sands and silty sands are particularly susceptible. Loose silts and gravels also have potential for liquefaction. Dense natural soils and well-compacted fills have low susceptibility to liquefaction. Clay soils are generally not susceptible, except for highly sensitive clays found in some geographic locales.

(1) Potential consequences of liquefaction include: (1) reduction or loss of foundation bearing strength, which can lead to large structure settlements due to shear failure in the weakened soils; (2) flotation of lightweight structures embedded in liquefied soil; (3) differential compaction, due to soil densification as excess pore water pressures dissipate, that can lead to structure differential settlement; (4) horizontal movements due to lateral spreading or flow sliding of liquefied soils, which can lead to total and differential lateral movements of structures; and (5) increased lateral pressures on retaining walls for liquefied soils. Other manifestations of liquefaction can also occur and may or may not pose a risk to structures. Sand boils are common surface manifestations of liquefaction, in which the liquefied soil under pressure is ejected to the ground surface through a vent and forms a conical-shaped "sand boil" deposit around the vent. Although sand boils are usually not a cause of damage to structures, the ejection of subsurface materials in a sand boil may pose a settlement hazard to an immediately adjacent structure. Another phenomenon accompanying liquefaction is ground oscillation, in which the ground overlying liquefied soil experiences large-displacement transient oscillations that can result in extensional and compressional ground failures such as opening and closing of fissures, buckling of sidewalks, thrusting of sidewalks and curbs over streets, breakage of utility lines, and the like.

(2) Figure F-4 illustrates the consequence of loss of foundation bearing capacity that occurred during the 1964 Niigata earthquake in Japan. As shown, apartment buildings experienced large settlements and tilts due to liquefaction of the underlying soil.

(3) Liquefaction-induced lateral movements can occur on extremely flat slopes, less than 1 percent in some cases. The potential for lateral movements is increased if there is a "free face," such as a river channel or the sloping shoreline of a lake or bay, toward which movements can occur. The hazard of lateral spreading is illustrated diagrammatically in Figure F-5. Figure F-6 illustrates the effect of lateral spreading on a building during the 1989 Loma Prieta earthquake; the movements pulled the structure apart.

c. Soil differential compaction. Differential compaction refers to the densification of soils that may occur due to strong earthquake ground shaking. As noted above, densification can occur with time following liquefaction as soil excess pore water pressures dissipate. In soils that are above the groundwater table and thus not susceptible to liquefaction, densification can occur as the strong ground shaking occurs. Loose natural soils and uncompacted and poorly compacted fills are susceptible to densification. If densification does not occur uniformly over an area, the resulting differential settlements can be damaging to structures. In general, the amounts of movement associated with the hazard of differential compaction are less than those due to liquefaction-induced bearing capacity failure or lateral spreading.

d. Landsliding. Landsliding can occur due to the loss of soil strength accompanying liquefaction, as mentioned above. However, landsliding can also occur in soils and rocks on hillside slopes in the absence of liquefaction, due to the inertia forces induced by the ground shaking. Consequences of landsliding include differential lateral and vertical movements of a structure located within the landslide zone, or landslide debris impacting a structure located below a landslide. An example of a structure within a zone of earthquake-induced

Figure F-4 Bearing capacity failure due to liquefaction, Niigata, Japan earthquake of June 16, 1964.

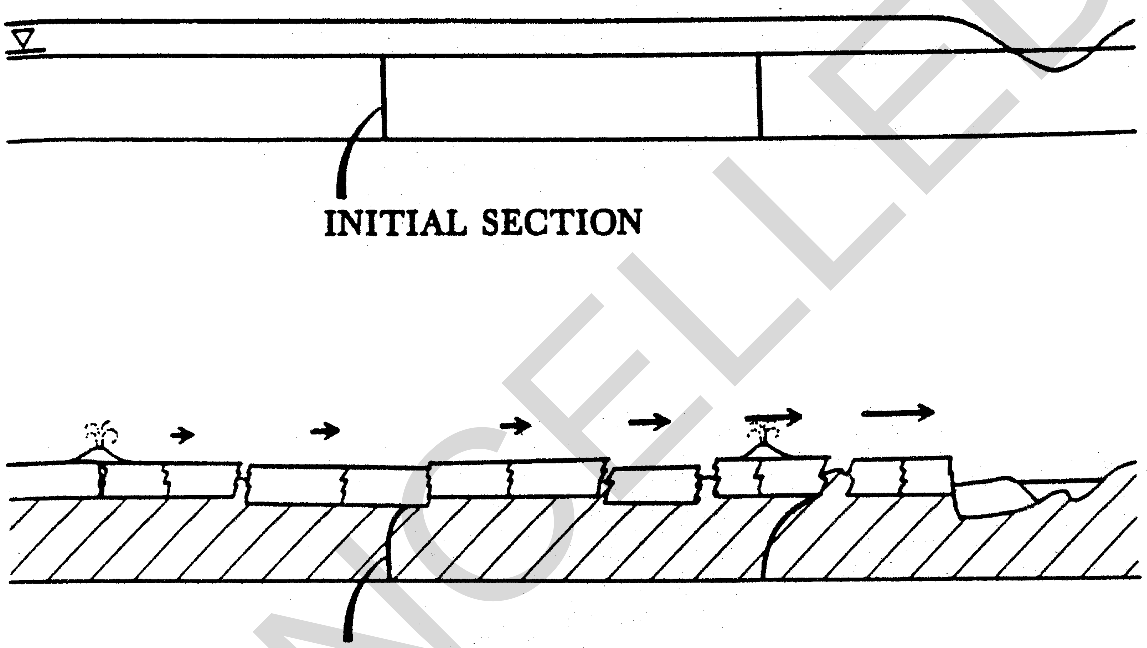


Figure F-5 Diagram of lateral spread before and after failure. Liquefaction occurs in the cross-hatched zone. The surface layer moves laterally down the mild slope, breaking up into blocks bounded by fissures. The blocks also may tilt and settle differentially with respect to one another (from Youd, 1984; National Research Council, 1985).

Figure F-6 Lateral spreading failure due to liquefaction, University of California Marine Laboratory Building at Moss Landing, Loma Prieta, California earthquake of October 17, 1989.

landsliding is shown in Figure F-7. Figure F-8 illustrates the hazard of landslide material (rockfall debris in this case) impinging on a structure below a slope. Even a single large boulder dislodged from a slope can cause considerable damage to a structure below.

e. Flooding. Earthquake-induced flooding at a site can be caused by a variety of phenomena including seiche, tsunami, landsliding, and dam, levee, and water storage tank failures. Seiches are waves induced in an enclosed body of water such as a bay, lake, or reservoir by interaction of the water body with the arriving seismic waves. Seiches can be caused by earthquakes that occur either in the region of a site or thousands of miles away. Seiche waves may reach several feet in height and can be damaging to facilities located at or very near the shoreline.

(1) Tsunamis are ocean waves generated by vertical seafloor displacements associated with large offshore earthquakes. Tsunami waves at a site may be produced by local or distant earthquakes; and wave heights may reach tens of feet at some coastal locations. Onshore tectonic movements accompanying earthquakes can also cause flooding, such as crustal tilting causing water to overflow a dam or uplift along a thrust fault causing damming of a river.

(2) Another source of tsunami waves is rapid landsliding into bodies of water, either from hillside slopes above the water body or from submarine slopes within the water body. Another type of flooding hazard is that caused by earthquake-induced failure of a dam, levee, or water storage tank.

F-3. Screening Procedures

The following sections describe screening procedures for the geologic hazards described above. The possible conclusions from screening for each hazard are: (1) a significant hazard potential does not exist; or (2) further evaluation (described in paragraph F-4) is required to assess the hazard and its significance. There are two screening procedures that should be followed for all the hazards. First, a check should be made as to whether a hazard has previously occurred at the site (or in the near vicinity of the site in similar geotechnical conditions) during historical earthquakes. This check may involve review of the earthquake history of an area, review of published post-earthquake reconnaissance reports, and discussions with engineers and geologists knowledgeable of the prior earthquake performance of an area. Although such information does not exist for all locations, it is available for numerous locations throughout the country; for example, in Northern California (Youd and Hoose, 1978; Tinsley et al., 1994); in the New Madrid, Missouri region (Obermeier, 1989; Wesnousky et al., 1989); in the Charleston, South Carolina region (Obermeier et al., 1986; Gohn et al., 1984); in the northeastern United States (Tuttle and Seeber, 1989); among others. If a hazard has previously occurred at the site, then the evaluations described in paragraph F-4 should be conducted; its absence, however, does not preclude the occurrence of the hazard during future seismic events. Second, a check should be made as to whether the site is included in an area for which a regional earthquake hazard map has been prepared by a federal or state agency. For example, under the U.S. Geological Survey's National Earthquake Hazard Reduction Program (NEHRP), liquefaction potential maps have been prepared for several urban areas of the United States. If the area containing the site has been mapped as having a high risk with respect to any geotechnical hazard (e.g., in an area of "high liquefaction potential"), then evaluations described in paragraph F-4 should be conducted.

a. Surface fault rupture. The potential for experiencing fault rupture (or not) at a site is controlled primarily by the regional and local tectonic environment. For the hazard of surface fault rupture to be present, an active fault or faults must pass beneath the site. A fault is considered to be active and capable of producing surface rupture if the fault exhibits any of the following characteristics indicative of recent tectonic activity:

- It is a documented source of historical earthquakes or is associated spatially with a well-defined pattern of microseismicity.

- Its trace (the zone where the fault intersects the ground surface) is marked by well-defined geomorphic features like scarps, deflected drainages, closed depressions, etc. that are suggestive of geologically recent faulting. Because such features are easily modified or destroyed by erosion and deposition, their

Figure F-7 House and street damaged by several inches of landslide displacement caused by the San Fernando, California earthquake of February 9, 1971.

Figure F-8 Damage to store front caused by rock fall during the San Fernando, California earthquake of February 9, 1971.

presence in the landscape indicates geologically recent tectonic activity.

- It has experienced at least one episode of surface rupture (including fault creep) during approximately the past 11,000 years (Holocene time) or multiple episodes of rupture during the last 100,000 years (the late Quaternary period).

(1) Regional potential for surface fault rupture. The potential for surface fault rupture varies greatly in different parts of the United States. The potential exists mainly along and near the active deformation boundary between the North American and Pacific tectonic plates, which extends along coastal California, Oregon, Washington, and southeastern and southern Alaska. The tectonic effects of this plate boundary, including surface faulting, extend to the eastern margin of the Rocky Mountains. Beyond the plate boundary, intraplate earthquakes occur within the North American plate but generally have not been accompanied by surface fault rupture. In the eastern United States, the only active faults that have been mapped at the ground surface to date are the Meers and Criner faults in southern Oklahoma. These faults, which comprise two segments of the Frontal Wichita Fault System, have well developed geomorphic expression and geologically documented episodes of slip during Holocene time. Intraplate earthquakes within the Pacific plate occur beneath the state of Hawaii and are triggered by the underground movement of basaltic magma from which the island volcanoes have been built. Ground fissuring can occur due to the swelling of volcanoes prior to eruption.

(2) Steps involved in screening. Screening for surface fault rupture should include:

- A review of geologic maps available from the U.S. Geological Survey, state geological agencies, and local government agencies. The geologic maps typically show the location of faults and identify the ages of the geologic units displaced by the fault. Large-scale geologic maps (e.g., 1:24,000 or larger scale) prepared within the last 30 years generally provide the most reliable information for this type of assessment. In California, "Alquist-Priolo" maps, published by the California Division of Mines and Geology, define those zones within the state in which surface fault rupture is a significant risk. The U.S. Geological Survey in Denver is currently preparing maps that show the major active faults in the Western Hemisphere. In the process of obtaining and reviewing these maps, government geologists who may be actively working on the geology of the area including the site should be contacted as needed.

- A review of topographic maps available from the U.S. Geological Survey. These maps depict the topography in the general site vicinity and can be used to identify geomorphic features that might indicate the presence of faults.

- A reconnaissance of the site and review of aerial photographs. With respect to the surface fault rupture hazard, a site reconnaissance and review of available aerial photographs, aimed at detecting geologic or geomorphic evidence of faulting, should be conducted if adequate geologic and topographic maps are not available.

(3) Screening criteria. It can be assumed that a severe hazard due to surface fault rupture does not exist at the site if, based upon a review of the available information, both of the following screening criteria are met:

(a) Geologic and topographic maps show no faults passing beneath the site or in the vicinity of the site; or if the maps show faults and folds in the vicinity of the site, the geologic maps and related cross sections clearly show that earth materials that are as least as old as Quaternary (1.8 million years old), like soils, alluvium, terrace surfaces and/or deposits, lie across the folds and faults and are not deformed by them.

(b) Site reconnaissance and air photo review does not detect evidence of faulting at the site.

(4) Example. An example of screening for surface fault rupture potential is given in Appendix G.

b. Soil liquefaction. The potential for experiencing liquefaction (or not) at a site during an earthquake is primarily influenced by the characteristics of the subsurface soils (e.g., geologic age and depositional environment, soil type, density), the depth to the groundwater table, and the amplitude and duration of ground shaking. As such, these factors can provide a basis for evaluating a site for liquefaction hazard. For screening level evaluations, criteria are given for assessing subsurface soils and groundwater information available for a site. Screening criteria are not made a function of ground shaking level because current understanding of liquefaction behavior does not preclude its occurrence at any ground shaking level, although there are no reported/known cases of historical liquefaction for peak ground accelerations less than about 0.07g.

(1) Sources of information. Sources of available information to be reviewed in conducting a screening evaluation for liquefaction hazard include:

- Geologic maps - Large-scale (e.g., 1:24,000) or smaller-scale (e.g., 1:250,000) geologic maps are generally available for many areas from geologists of regional U.S. Geological Survey offices, state geological agencies, or local government agencies. The geologic maps typically identify the age, depositional environment, and material type for a particular mapped geologic unit.

- Topographic maps - Similar availability as geologic maps. These maps depict the general slope gradient and direction for the general site vicinity and the presence of any significant nearby free-face. Site grading plans may also be available for review.

- Boring logs - Foundation engineering reports prepared for a facility typically contain logs of geotechnical borings drilled at the site. The logs typically contain information regarding the stratigraphy (soil type), penetration resistance (density) and the depth at which groundwater was encountered. The foundation engineering reports may also contain laboratory test data such as grain size distributions, Atterberg limits, unit weights, shear strength, etc.; these data are commonly reported on the boring logs and reflected in the soil descriptions given on the logs. In the absence of site-specific boring logs, logs for borings drilled on an adjacent site may provide useful screening information, as may logs of water wells drilled on site or nearby. If off-site information is utilized, it is important to examine the appropriateness of the off-site data by checking the mapped geologic similarity of the sites (see above).

- Groundwater depth - The depth of the groundwater table below the existing ground surface is commonly reported on boring logs or water well logs; regional groundwater depth (elevation) contour maps may also be available and utilized if site-specific or nearby measurements are not. Possible seasonal and historic fluctuations of groundwater levels should also be reviewed/considered.

- Building foundation - Available drawings and other information on the proposed building foundation system should be reviewed to ascertain the type and depth of foundation (e.g., spread footings, piles).

- Site ground reconnaissance - Walkdown of the site and buildings should be conducted to observe and note the existing characteristics of the site (e.g., topography, especially slopes or free faces). During the site reconnaissance, observations of ground distress and/or building distress at the site and nearby sites that may be related to geotechnical processes should also be recorded.

(2) Screening criteria. It can be assumed that a significant hazard due to liquefaction does not exist at a site if, based on the review of available information, one of the following screening criteria is met:

(a) The geologic materials underlying the site are either bedrock or have a very low liquefaction susceptibility according to the relative susceptibility ratings that Youd and Perkins (1978) assigned based upon general depositional environment and geologic age of the deposit. These susceptibility ratings are shown in Table F-1.

(b) The soils below the groundwater table at the site are: stiff clays or clayey silts and have a clay content (grain size< 0.005 mm or 0.0002 inches) greater than 15 percent, liquid limit greater than 35 percent, or natural moisture content less than 90 percent of the liquid limit (Seed and Idriss, 1982); or cohesionless soils (i.e. clean or silty sands, silts, or gravels) with a minimum normalized Standard Penetration Test (SPT) resistance, $(N_1)_{60}$, value of 30 blows/0.3 m (30 blows/foot); or cohesionless

Table F-1 Estimated susceptibility of sedimentary deposits to liquefaction during strong ground motion (after Youd and Perkins, 1978).

Type of Deposit	General Distribution of Cohesionless Sediments in Deposits	Likelihood that Cohesionless Sediments, When Saturated, Would be Susceptible to Liquefaction (by Age of Deposit)			
		<500 yr Modern	Holocene >11 ka	Pleistocene 11 ka - 2 Ma	Pre-Pleistocene >2 Ma
(a) Continental Deposits					
River channel	Locally variable	Very high	High	Low	Very low
Floodplain	Locally variable	High	Moderate	Low	Very low
Alluvial fan and plain	Widespread	Moderate	Low	Low	Very low
Marine terraces and plains	Widespread	---	Low	Very low	Very low
Delta and fan-delta	Widespread	High	Moderate	Low	Very low
Lacustrine and playa	Variable	High	Moderate	Low	Very low
Colluvium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dunes	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low
Tephra	Widespread	High	High	?	?
Residual soils	Rare	Low	Low	Very low	Very low
Sebka	Locally variable	High	Moderate	Low	Very low
(b) Coastal Zone					
Delta	Widespread	Very high	High	Low	Very low
Estuarine	Locally variable	High	Moderate	Low	Very low
Beach					
High wave-energy	Widespread	Moderate	Low	Very low	Very low
Low wave-energy	Widespread	High	Moderate	Low	Very low
Lagoonal	Locally variable	High	Moderate	Low	Very low
Fore shore	Locally variable	High	Moderate	Low	Very low
(c) Artificial					
Uncompacted fill	Variable	Very high	---	---	---
Compacted fill	Variable	Low	---	---	---

soils that classify as clayey sand (SC) or clayey gravel (GC) with $(N_1)_{60}$ greater than 20. (The parameter $(N_1)_{60}$ is defined in paragraph F-4.) However, cohesive soils that are highly sensitive based on measured soil properties or local experience are not screened out. To be classified as highly sensitive, a soil must possess each of the following property values: sensitivity greater than 4; liquid limit less than 40%; moisture content greater than 0.9 times the liquid limit; liquidity index greater than 0.6; and $(N_1)_{60}$ less than 5 or normalized cone penetration resistance, q_{c1}, less than 1 MPa (20 ksf). Areas of the U.S. with known highly sensitive soils include some coastal areas of Alaska, along the St. Lawrence River, some eastern and western coastal areas with estuarine soil deposits, and near saline lakes in the Great Basin and other arid areas. (Refer to Youd, 1998).

(c) The groundwater table is at least 15 m (49 feet) below the ground surface, including considerations for seasonal and historic groundwater level rises, and any slopes or free-face conditions in the site vicinity do not extend below the groundwater elevation at the site.

(3) Example. An example of screening for the hazard of liquefaction is given in Appendix G.

c. Soil differential compaction. Information sources to be reviewed in conducting a screening evaluation for differential compaction are the same as those identified above for the liquefaction potential hazard. The site reconnaissance observations for the liquefaction potential hazard can be used for the screening of the hazard of differential compaction.

(1) Screening criteria. It can be assumed that a significant hazard due to differential compaction does not exist if the soil conditions meet both of the following criteria:

(a) The geologic materials underlying foundations and below the groundwater table do not pose a significant hazard due to liquefaction.

(b) The geologic materials underlying foundations and above the groundwater table are either: Pleistocene in geologic age (older than 11,000 years); stiff clays or clayey silts; or cohesionless sands, silts, and gravels with a minimum $(N_1)_{60}$ of 20 blows/0.3 m (20 blows/foot).

d. Landsliding. The potential for landsliding or downslope movement is dependent on slope geometry, subsurface soil, rock and groundwater conditions, past slope performance, and level of ground shaking. The screening procedures involve a review of geologic and topographic maps, review of available data on the subsurface conditions, and performing reconnaissance of the site and adjacent areas. Review of available aerial photographs is desirable, especially if adequate geologic and topographic maps are not available. In some areas, governmental agencies have prepared slope stability maps showing existing landslides and/or relative slope stability. These should be reviewed if available. If appropriate, geologists and engineers in government agencies knowledgeable of the performances of slopes in the area should be contacted.

(1) Screening criteria. It can be assumed that a significant hazard due to earthquake-induced landsliding does not exist if all of the following criteria are satisfied:

(a) The building site is not located within a pre-existing active or ancient landslide, and there are no landslides on slopes of similar geometry and geology in the site vicinity. The site is not located on, above, or below a slope that displays cracking or other signs of actual or incipient slope movement. There is not an obvious hazard to the building from falling rocks or shallow soil flows on slopes located above the building.

(b) The site is not located adjacent to a shoreline.

(c) The site is not located in a zone that has been mapped as having a high landslide potential (static or seismic).

(d) The building is located above a slope, is a horizontal distance of at least three times the slope height from the toe of the slope, and is set back a distance at least equal to the slope height from the top of the slope. The geologic materials in the slope are stiff cohesive (and nonsensitive) clays or clayey silts, dense sands that do not have a significant liquefaction potential, or bedrock. There are no obvious planes of weakness in the slope, such as bedding planes dipping out of the slope. If fill is present in the slope, there is evidence that it has been engineered, well compacted, and placed with engineering inspection and testing.

(e) The building is located below a slope, is a horizontal distance of at least twice the slope height from the toe of the slope, and the slope is underlain by geologic materials as stated in (d) above.

(2) Example. An example of screening for the hazard of landsliding is given in Appendix G.

e. Flooding. The hazard of flooding due to many causes, including tsunami, seiche, tectonic movements, and failure of water retention structures can be assumed to be not significant if the facility is not located near a body of water nor in an area that could be inundated by the hazard.

(1) Tsunami and seiche. For facilities located near coastal waters, the hazard of tsunami due to earthquake-induced seafloor displacements can be assumed to be not significant if the ground surface elevation of the facility above sea level is greater than the estimated potential maximum tsunami wave height as given in Figure F-9. Although records of seiche occurrence are relatively incomplete, it would appear to be rare for a seiche wave to exceed about 2 m (7 feet) in height. Therefore, the seiche hazard can be screened out for sites located more than 2 m (7 feet) above the adjacent water body.

(2) Landsliding-induced tsunami. The potential for rapid hillside landsliding into bodies of water can be assumed to be not significant if slopes in similar geologic materials in the vicinity have performed well historically and the slopes are not oversteepened. If similar slopes and geologic formations extend underwater, they are also unlikely to be susceptible to significant submarine landsliding. Loose or soft submarine deposits such as deltaic deposits could be susceptible to rapid landsliding.

(3) Flooding due to tectonic movements. The potential for flooding due to tectonic movements can be assumed to be not significant if the regional faults would not be expected to produce tectonic movements to a degree that could interact with water bodies and cause flooding. Such judgements should be made by experienced geologists or seismologists who are knowledgeable of the regional tectonic setting.

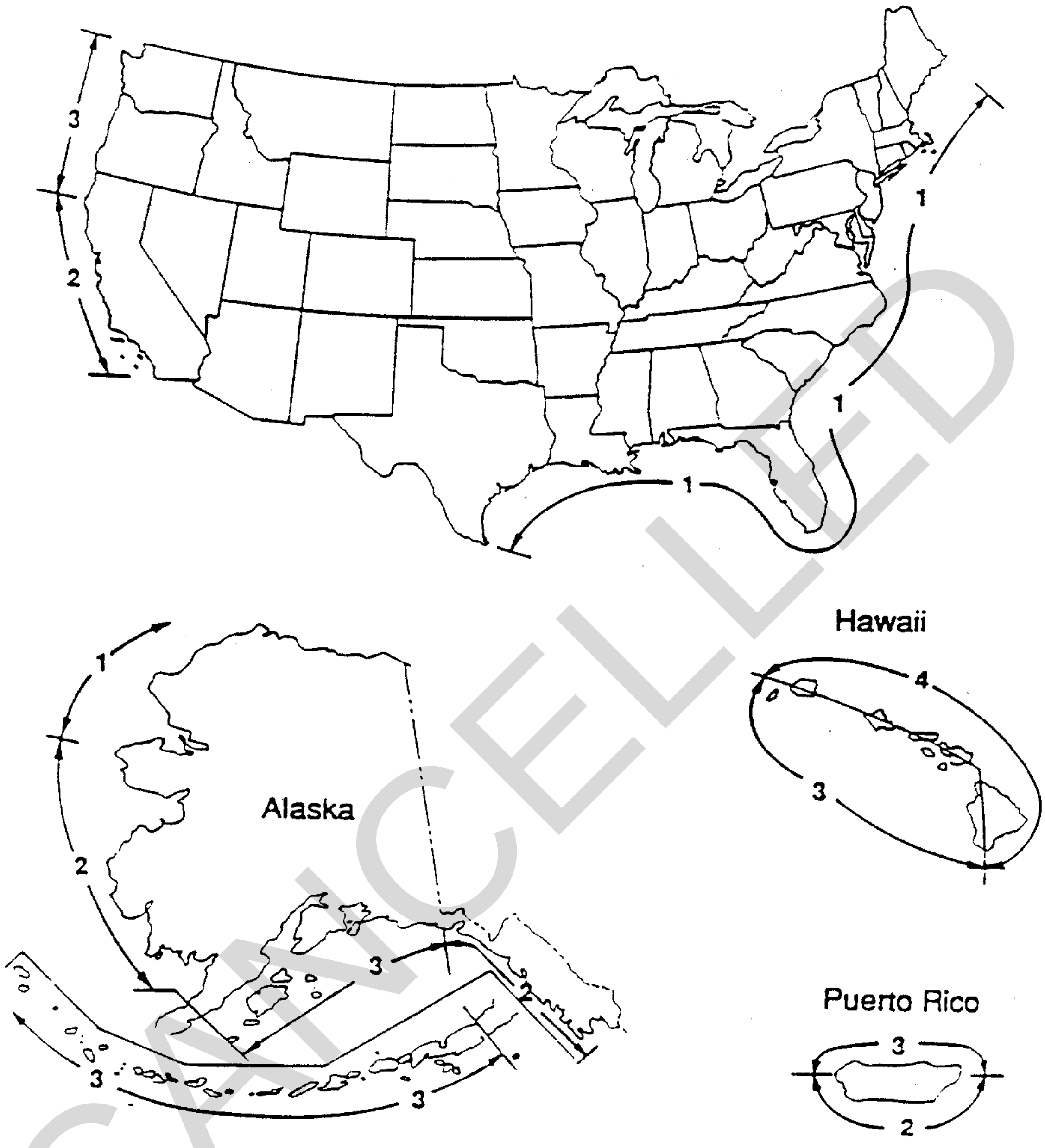
(4) Flooding due to failure of water retention structures. The potential for flooding due to the failure of water retention structures can be assumed to be not significant if the facility is located outside of areas that could be subject to inundation. City, county, state, and federal agencies (e.g., U.S. Army Corps of Engineers, U.S. Bureau of Reclamation) should be contacted as needed to ascertain the location of such water retention structures and inundation areas.

F-4. Evaluation Procedures

The following sections describe evaluation procedures for hazards that are not screened out using the procedures in paragraph F-3. An important element in the evaluations is to assess the consequences of the hazard in terms of the significance of the hazard to the structure. Thus, for example, the occurrence of liquefaction may or may not pose a significant risk to a structure depending on whether or not significant ground and structural deformations could occur as a result of liquefaction. The possible conclusions from these evaluations are: (1) a hazard posing a significant risk to structures does not exist; (2) the hazard exists, but further structural evaluation is required to ascertain whether the risk to structures is significant; or (3) the hazard exists, poses a significant risk of damage to a structure and mitigation measures should be considered.

a. Estimated ground motion. When estimates of earthquake ground shaking parameters are required for these evaluations, they should be consistent with MCE ground motions as defined in Chapter 3. The corresponding performance objectives should be collapse prevention for Seismic Use Groups I and II; for Seismic Use Groups IIIH and IIIE, performance objectives should be 2B and 3B, respectively, as defined in Chapter 4. Estimates of the duration of strong shaking should be based on the assumption of the occurrence of maximum earthquakes in the site region.

b. Surface fault rupture. After a site has been evaluated by the screening criteria developed above and (1) either there is insufficient information to rule out a surface fault rupture hazard, or (2) there is seismic, geomorphic, and/or geologic data that suggests active fault(s) might be present at or near



Zone	Wave Height (feet)
1	5
2	15
3	30
4	50

1 foot = 0.3 meters

Figure F-9 Tsunami zone map and wave heights.

the site, the following information is required to refine definition of the hazard:

- the location of fault traces (if any) with respect to the site
- the timing of most recent slip activity on the fault
- the ground rupture characteristics for a design earthquake on the fault (e.g., type of faulting (Figure F-1), amount of slip and distribution into strike-slip and dip-slip components, and width of the zone of ground deformation)

(1) Fault location. There are several steps that can be taken to confirm and define the location of faults. Further assessments will not be required if it can be shown on the basis of the evaluation procedures outlined below that there are no faults passing beneath the site.

(a) Interpretation of aerial photographs. Aerial photographs can be an excellent supplementary resource to geologic and topographic maps of the site and vicinity for identifying faults. Older photographs are particularly useful if they depict the site and/or its environs prior to development activities that would have altered or destroyed landforms that indicate the presence of faults. For many parts of the country, stereo photographic coverage is available as far back as the 1920s or 1930s. Aerial photographs are usually available from several sources including private companies and from various governmental agencies including the U.S. Geological Survey, U.S. Department of Agriculture (Soil Conservation Service), Bureau of Land Management, Forest Service, etc. The USGS maintains the repository for federal photographic resources at its EROS Data Center, Sioux Falls, South Dakota 57198.

(b) Contacting knowledgeable geologists. There probably are geologists/earth scientists familiar with geologic and tectonic conditions in the site vicinity who will be willing to share their knowledge. These geologists might work for governmental agencies (federal, state, and local), teach and conduct research at nearby colleges and universities, or practice as consultants.

(c) Ground reconnaissance of site and vicinity. Walkdown of the site and its vicinity should be conducted to observe unusual topographic conditions and to evaluate any geologic relationships visible in cuts, channels or other exposures. Features requiring a field assessment might have been identified previously during the geologic and topographic map review, aerial photographic interpretation, and/or during conversations with geologists.

(d) Subsurface exploration. Faults obscured by overburden soils, site grading, and/or structures can be potentially located by one or more techniques. Geophysical techniques such as seismic refraction surveying provide a remote means of identifying the location of steps in a buried bedrock surface and the juxtaposition of earth materials with different elastic properties. Geophysical surveys require specialized equipment and expertise, and their results may sometimes be difficult to interpret. Trenching investigations are commonly used to expose subsurface conditions to a depth of 4.6 to 6.1 m (15 to 20 feet). While expensive, trenches have the potential to locate faults precisely and provide exposures for assessing their slip geometry and slip history. Borings can also be used to assess the nature of subsurface materials and to identify discontinuities in material type or elevation that might indicate the presence of faults.

(2) Fault activity. If it is determined that faults pass beneath the site, it is essential to assess their activity by determining the timing of the most recent slip(s). If it is determined, based on the procedures outlined below, that the faults are not active faults (see paragraph F-3a), then further assessments are not required.

(a) Assess fault relationship to young deposits/surfaces. The most definitive assessment of the recency of fault slip can be made in natural or artificial exposures of the fault where it is in contact with earth materials and/or surfaces of Quaternary age (last 1.8 million years). Deposits might include native soils, glacial sediments like till and loess, alluvium, colluvium, beach and dune sands, and other poorly consolidated surficial materials. Surfaces might include marine, lake, and stream terraces, and other erosional and depositional surfaces. A variety of age-dating techniques, including radiocarbon analysis and soil profile development, can be used to estimate the timing of most recent fault slip.

(b) Evaluate local seismicity. If stratigraphic data are not available for assessment of fault activity, historical seismicity patterns might provide useful information. Maps and up-to-date plots depicting historical seismicity surrounding the site and vicinity can be obtained from the USGS at its National Earthquake Information Center in Golden, Colorado. Additional seismicity information may be obtained from state geologic agencies and from colleges and universities that maintain a network of seismographs (e.g., California Institute of Technology; University of California, Berkeley; University of Nevada, Reno; University of Washington; National Center for Earthquake Engineering Research, Buffalo, New York; etc.). If the fault(s) that pass beneath the site are spatially associated with historical seismicity, and particularly if the seismicity and fault trends are coincident, the faults should probably be considered active.

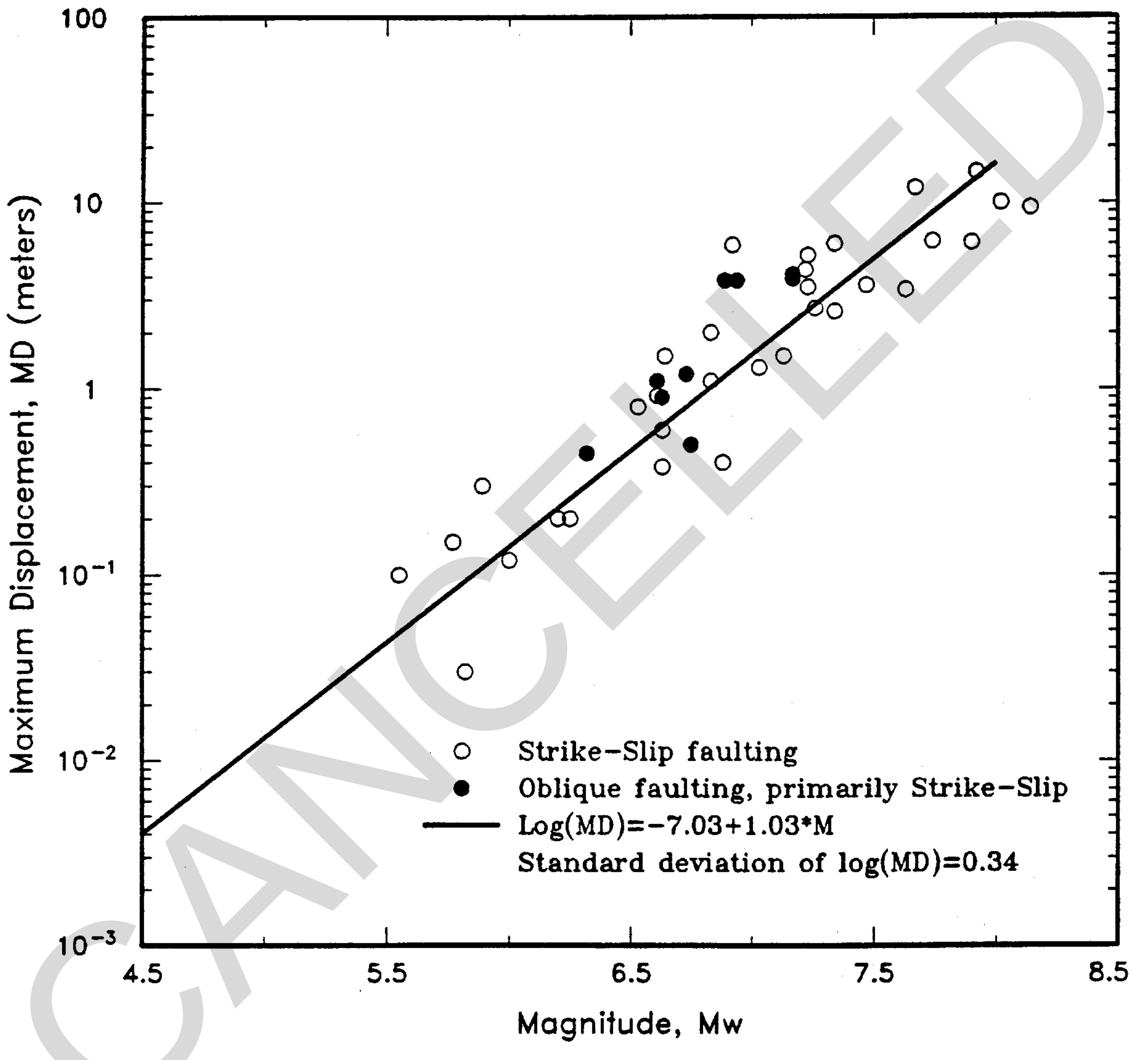
(c) Evaluate structural relationships. In the absence of both stratigraphic and seismological data, an assessment of the geometric/structural relationships between fault(s) at the site and faults of known activity in the region could be useful. Although less definitive than the two prior criteria, the probability that the site fault is active increases if it is structurally associated with another active fault, and if it is favorably oriented relative to stresses in the current tectonic environment.

(3) Fault rupture characteristics. If the evaluation indicates one or more active faults are present beneath the site, the characteristics of future slip on the fault(s) can be estimated. Based on analysis of moderate and large magnitude earthquakes worldwide, Wells and Coppersmith (1994) have developed empirical relationships among earthquake moment magnitude and a variety of fault characteristics including maximum displacement (Figure F-10) that are based on fault type (e.g., strike-slip, reverse, and normal). These curves provide a convenient means for assessing the amount of slip or displacement fault. Amounts of fault displacement should be estimated assuming the occurrence of a maximum earthquake on the fault. Predicting the width of the zone of surface deformation associated with a surface faulting event is more difficult because empirical relationships having general applicability have not yet been developed. The best means for assessing the width of faulting at the site is site-specific trenching that crosses the entire zone. In the absence of such information, the historical record indicates that steeply dipping faults, such as vertical strike-slip faults, tend to have narrower zones of surface deformation than shallow dipping faults like thrust and normal faults. An example of an evaluation of the potential for surface fault rupture following a screening process is given in Appendix G.

c. Soil liquefaction. If a site has been filtered through the screening criteria and liquefaction-susceptible materials are identified, the potential for liquefaction to occur due to earthquake ground shaking may be assessed by a variety of available approaches (National Research Council, 1985). The most commonly utilized approach is the Seed-Idriss simplified empirical procedure presented by Seed and Idriss (1971, 1982), as updated by Seed et al. (1985) and Youd and Idriss (1997) that utilizes Standard Penetration Test (SPT) blowcount data. The latter citation refers to the Proceedings of the Workshop on Evaluation of Liquefaction Resistance of Soils conducted by the National Center for Earthquake Engineering Research (NCEER). The purpose of the workshop was to update and augment the simplified liquefaction evaluation procedures. Where consensus has been achieved by the workshop participants on changes and additions to the evaluation procedures, these changes and additions are incorporated herein. However, as of October 1998, workshop participants are continuing to evaluate several aspects of the evaluation procedures.

The following paragraphs briefly summarize simplified state-of-the-art approaches for evaluating liquefaction potential and its consequences. Guidance for liquefaction potential evaluations is also presented in Navy Technical Report TR-2077-SHR (Ferritto, 1997b) and Department of Defense Handbook MIL-HDBK-1007/3 (Department of Defense, 1997). Ferritto (1997b) also presents guidance for safety factors against liquefaction and allowable displacements for different facility types.

(1) Seed-Idriss evaluation procedure. Peak ground-surface acceleration, earthquake magnitude, total and effective overburden stresses at the point of interest, and the standardized SPT blowcount are needed to perform the evaluation using the Seed-Idriss simplified empirical procedure. The standardized blowcount index used in the method is $(N_1)_{60}$, which represents the SPT blowcount to advance a 51-mm (2-inch) O.D. split-spoon sampler



1 meter = 3.3 feet

Figure F-10 Relationship between maximum surface fault displacement (MD) and earthquake moment magnitude, M_w, for strike-slip faulting (based on Wells and Coppersmith, 1994).

0.3 m (1 foot) at a 60 percent hammer energy efficiency, with correction to an effective overburden pressure of 96 kPa (2 ksf). The procedure is based on the empirical correlation between cyclic stress ratio (computed from the peak ground surface acceleration) and $(N_1)_{60}$ blow count that differentiates the observed occurrence or non-occurrence of liquefaction in sand deposits during earthquakes. The basic correlation presented by Seed et al. (1985) for magnitude 7.5 earthquakes for materials with different fines contents (FC), and adjusted in Youd and Idriss (1997) for very low blowcounts, is illustrated in Figure F-11; the correlation may be adjusted to other earthquake magnitudes using adjustment factors developed by Seed and Idriss (1982) given in Table F-2. Youd and Idriss (1997) present several alternative magnitude scaling factors; however, at present, consensus has not been attained on revisions to these factors.

(a) For a given value of peak ground surface acceleration (PGA) (in g units) and the total and effective overburden pressures at the depth of interest (s_o and s'_o, respectively), a value of the average induced cyclic stress ratio (CSR) can be computed using the expression (Seed and Idriss, 1971):

$$CSR = \frac{t_a}{s'_o} = 0.65 \frac{PGA}{g} \frac{s_o}{s'_o} r_d$$

in which t_a is the induced average cyclic shear stress at the depth of interest, and r_d is a stress reduction factor that decreases from a value of 1 at the ground surface to a value of 0.9 at a depth of about 10.7 m (35 feet). It is noted that the participants in the NCEER workshop (Youd and Idriss, 1997) have not achieved consensus regarding possible changes to the values for r_d. The relationship for r_d developed by Seed and Idriss (1971) and still in engineering usage is shown in the liquefaction potential evaluation example in Appendix G (Figure G-7). Using values of cyclic stress ratio from the preceding equation and a plot such as Figure F-11 for the appropriate earthquake magnitude, a critical value of the $(N_1)_{60}$ blowcount can be determined, such that those $(N_1)_{60}$ blowcounts exceeding the critical $(N_1)_{60}$ would likely not liquefy and those having a value less than the critical $(N_1)_{60}$ would likely liquefy. By comparing the critical blowcount $(N_1)_{60}$ with the measured $(N_1)_{60}$ of the material, it is possible to assess whether liquefaction would be expected to occur or not at the site. The critical blowcount $(N_1)_{60}$ condition corresponds to a factor of safety against liquefaction equal to unity (i.e., 1.0). Factor of safety is defined as the ratio of the ground-shaking induced cyclic stress ratio (from the preceding equation) to the cyclic resistance ratio (CRR) (see Figure F-11) that defines the boundary between liquefaction and non-liquefaction behavior.

To facilitate the use of electronic computational aids, Youd and Idriss (1997) present equations that may be used to approximate the CRR curves given in Figure F-11. The clean sand curve (fines content < 5 %) is approximated by the following equation:

$$CRR_{7.5} = \frac{a + cx + ex^2 + gx^3}{1 + bx + dx^2 + fx^3 + hx^4} \quad \text{for x < 30}$$

where:

a	=	0.048
b	=	-0.1248
c	=	-0.004721
d	=	0.009578
e	=	0.0006136
f	=	-0.0003285
g	=	-0.00001673
h	=	0.000003714
x	=	$(N_1)_{60\ cs}$

The curves for silty sands in Figure F-11 may be approximated by correcting the penetration resistance of a silty sand to an equivalent clean sand penetration resistance, $(N_1)_{60cs}$. The equivalent clean sand blowcount may then be used in the preceding equation to estimate liquefaction resistance. The equivalent clean sand blowcount is approximated by the following equation:

$$(N_1)_{60cs} = a + b(N_1)_{60}$$

where:

a	=	0	for FC#5%
a	=	exp[1.76-(190/FC^2)]	for 5%<FC<35%
a	=	5.0	for FC$35%
b	=	1.0	for FC#5%
b	=	[0.99+(FC$^{1.5}$/1000)]	for 5%<FC<35%
b	=	1.2	for FC$35%

where FC is the fines content (expressed as a percentage) measured from laboratory gradation tests from retrieved soil samples.

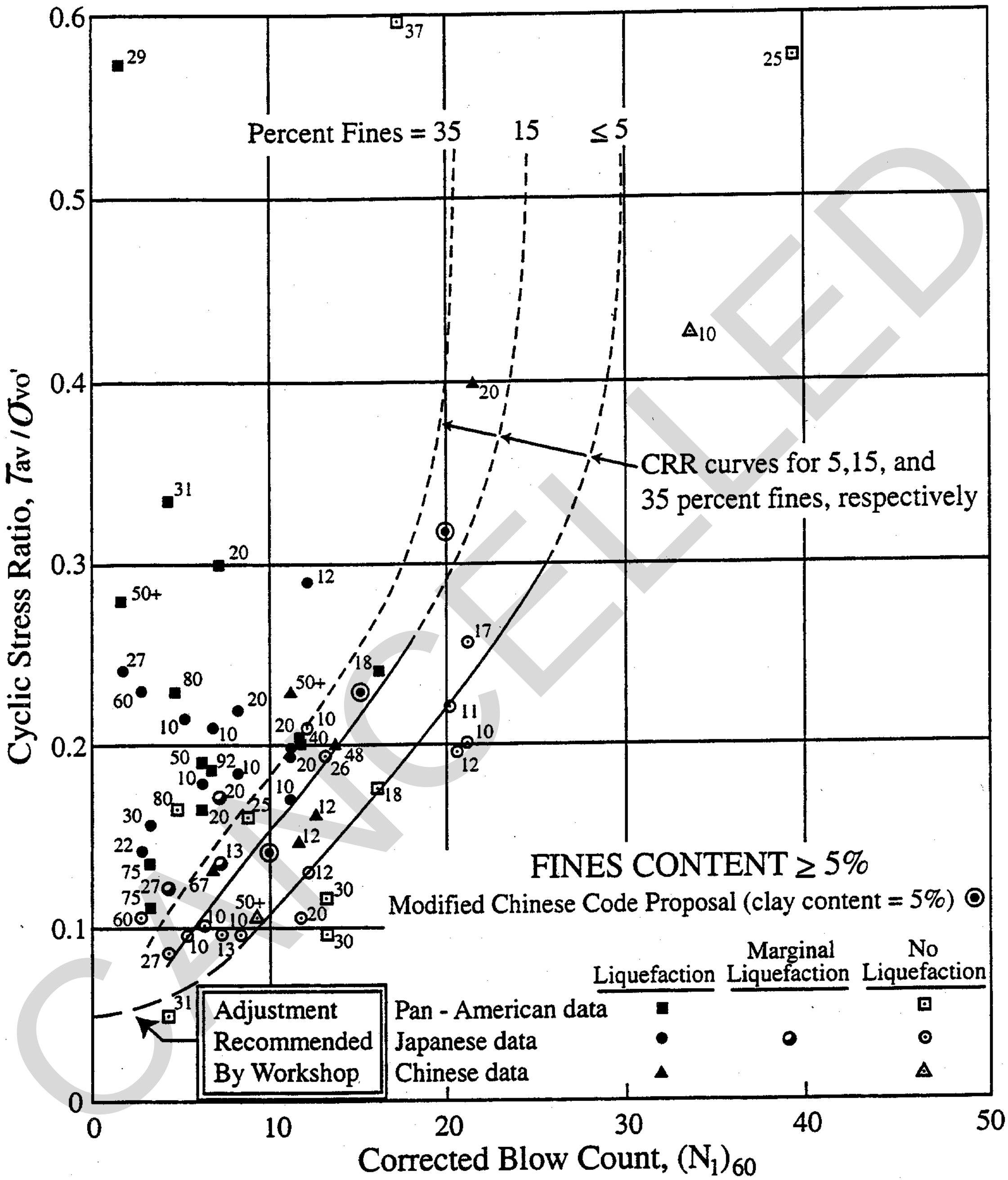


Figure F-11 Relationship between cyclic stress ratio (CSR) causing liquefaction and $(N_1)_{60}$ values for M_w = 7.5 earthquakes (Seed et al., 1985; Youd and Idriss, 1997).

Table F-2 Scaling factors for influence of earthquake magnitude on liquefaction resistance (from Seed et al., 1985).

Earthquake Magnitude	Magnitude Scaling Factor
M_w	K_m
8½	0.89
7½	1.00
6:	1.13
6	1.32
53	1.50

Note: scaling factors are applied to the ordinates of the curves in Figure F-11.

An example of liquefaction potential evaluation using the simplified empirical procedure is presented in Appendix G. The Navy has developed a computer program, LIQUFAC, for analyzing liquefaction potential using the Seed-Idriss simplified procedure (Ferritto, 1997b). Figure F-12 is a graphic plot illustrating results of LIQUFAC analysis for a soil profile.

(2) Cone Penetration Test (CPT) data are also utilized with the Seed-Idriss evaluation procedure by conversion of the CPT data to equivalent SPT blowcounts, using correlations developed among cone tip resistance (Q_c), friction ratio, soil type, and Q_c/N in which N is the SPT blowcount (e.g., Seed and DeAlba, 1986; Robertson and Campanella, 1985). Direct correlations of CPT data with liquefaction potential have also been developed. The most recent of these are those by Robertson and Wride (1997) and Olsen (1997) in the proceedings of the 1997 NCEER workshop (Youd and Idriss, 1997). To date these are not as widely used as the Seed-Idriss correlation with $(N_1)_{60}$ blowcount in Figure F-11.

(3) Shear wave velocity data have also been correlated with liquefaction potential in a manner similar to the correlations with SPT and CPT data. A recent correlation is presented by Andrus and Stokoe (1997) in the proceedings of the 1997 NCEER workshop (Youd and Idriss, 1997).

(4) Other approaches. The Becker hammer is a larger-diameter penetrometer that has been used to obtain penetration test data in gravelly soils. These data are then correlated to SPT measurements so that liquefaction potential of gravelly soils can be evaluated using Figure F-11. The approach is described by Harder (1997). The threshold strain approach of Dobry et al. (1981) utilizes shear wave velocity as a parameter to estimate a level of cyclic shear strain below which excess pore water pressure will not be generated and accumulated. If the cyclic shear strains induced by an earthquake's ground shaking do not exceed the threshold level, liquefaction cannot occur during that earthquake. National Research Council (1985) notes that this is a conservative evaluation because liquefaction may not occur even if the strains do exceed the threshold.

(5) Consequences of liquefaction -- general. The predicted occurrence of liquefaction does not necessarily imply unacceptable adverse consequences to a structure. If liquefaction is estimated to occur under design ground motion levels, the consequences should be assessed. Deformations accompanying liquefaction may or may not be tolerable depending on the specific structure design and performance objectives. Guidance for allowable displacements due to liquefaction for different types of Navy facilities is presented by Ferritto (1997b). Guidelines for assessing consequences of liquefaction are presented in the following paragraphs.

(6) Consequences of liquefaction -- lateral spreads. Lateral spreads are ground-failure phenomena that can occur on gently sloping ground underlain by liquefied soil. Earthquake ground-shaking affects the stability of sloping ground containing liquefiable materials by seismic inertia forces within the slope and by shaking-induced strength reductions in the liquefiable materials. Temporary instability due to seismic inertia forces are manifested by lateral "downslope" movement that can potentially involve large land areas. For the duration of ground shaking associated with moderate-to large-magnitude earthquakes, there could be many such occurrences of temporary instability, producing an accumulation of "downslope" movement.

(a) Various relationships for estimating lateral spreading displacement have been proposed, including the Liquefaction Severity Index (LSI) by Youd and Perkins (1978), a relationship incorporating slope and liquefied soil thickness by Hamada et al. (1986), a modified LSI approach presented by Baziar et al. (1992), and a relationship by Bartlett and Youd (1992, 1995), in which they characterize displacement potential as a function of earthquake and local site characteristics (e.g., ground slope, liquefiable layer thickness, and soil grain size distribution). Equations given by Bartlett and Youd (1992, 1995) for lateral spreading of sloping ground and free-face conditions are as follows:

for free-face conditions:

$$\mathrm{LOG}(D_H+0.01) = -16.366 + 1.178\,M - 0.927\,\mathrm{LOG}\,R - 0.013\,R + 0.657\,\mathrm{LOG}\,W + 0.348\,\mathrm{LOG}\,T_{15} + 4.527\,\mathrm{LOG}\,(100-F_{15}) - 0.922\,D50_{15}$$

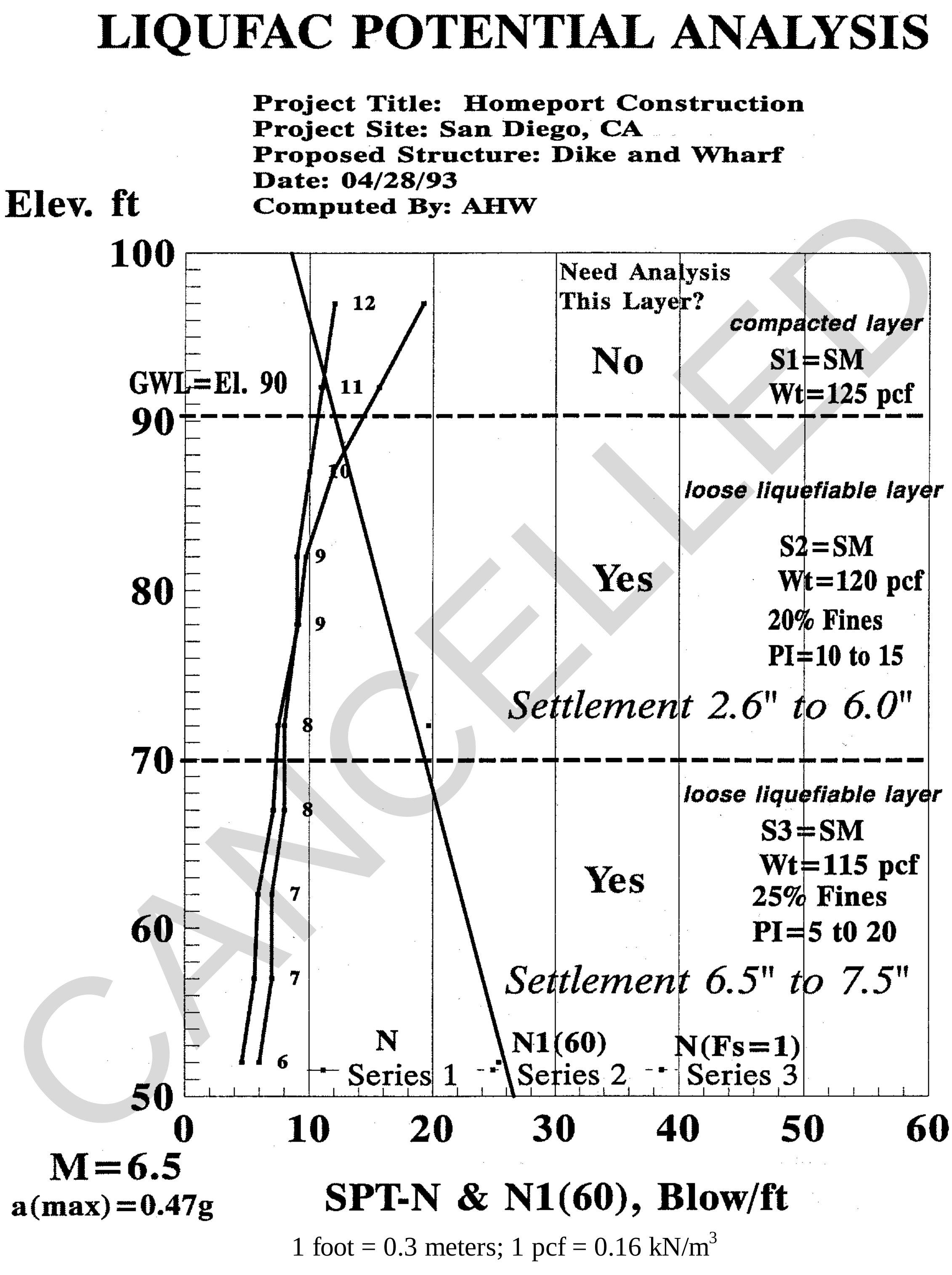


Figure F-12 Example of LIQUFAC analysis graphic plot (Department of Defense, 1997).

and for sloping ground conditions:

$$\begin{aligned}\text{LOG}(D_H+0.01) = {} & -15.787 + 1.178\,M \\ & -0.927\,\text{LOG}\,R - 0.013\,R + 0.429\,\text{LOG}\,S \\ & + 0.348\,\text{LOG}\,T_{15} + 4.527\,\text{LOG}\,(100-F_{15}) \\ & - 0.922\,D50_{15}\end{aligned}$$

in which:

D_H	=	Displacement (m)
M	=	Earthquake moment magnitude
R	=	Horizontal distance from the seismic energy source, (km).
W	=	100 x (height (H) of the free face / distance (L) from the free face).
S	=	Ground slope (%).
T_{15}	=	Cumulative thickness of saturated granular layers with $(N_1)_{60} \leq 15$, (m).
F_{15}	=	Average fines content of saturated granular layers included in T_{15}, (%).
$D50_{15}$	=	Average mean grain size in layers included in T_{15}, (mm).

(b) This set of relationships is considered to be adequate for most applications to obtain an order of magnitude (i.e., generally within a factor of 2) of the lateral spreading hazard for a site. More site-specific relationships may be developed based on slope stability and deformation analysis for lateral spreading conditions using undrained residual strengths for liquefied sand (Seed and Harder, 1990; Stark and Mesri, 1992) along with simplified Newmark-type (1965) and Makdisi and Seed (1978) displacement approaches, or using more detailed displacement analysis approaches.

(7) Consequences of liquefaction -- flow slides. Flow slides generally occur in liquefied materials located on steeper slopes and may involve ground movements of hundreds of meters. As a result, flow slides can be the most catastrophic of the liquefaction-related ground-failure phenomena. Fortunately, flow slides are much less common occurrences than lateral spreads. Whereas lateral spreading requires earthquake inertia forces to create instability for movement to occur, flow movements occur when the gravitational forces acting on a ground slope exceed the strength of the liquefied materials within the slope.

(8) Consequences of liquefaction -- settlement. With time following the occurrence of liquefaction, the excess pore water pressures built up in the soil will dissipate, drainage will occur, and consolidation or compaction of the soil will occur that will be manifested at the ground surface as settlement. An approach to estimate the magnitude of such ground settlement that is analogous to the simplified empirical procedure for liquefaction potential evaluation (i.e., using SPT blowcount data and cyclic stress ratio) has been presented by Tokimatsu and Seed (1987) and is suggested herein to the user. The relationships presented by Tokimatsu and Seed (1987) are shown on Figure F-13. An example illustrating the estimation of liquefaction-related ground settlement using the Tokimatsu and Seed (1987) procedure is provided in Appendix G. Relationships presented by Ishihara and Yoshimine (1992) are also available for assessing settlement.

(9) Consequences of liquefaction -- bearing capacity reduction. Shaking-induced strength reductions in liquefiable materials that are associated with the generation and accumulation of excess pore water pressure can have effects on the support capacity of foundation elements. For spread-type footings, these effects may be substantial where the groundwater and liquefiable materials are situated at shallow depths relative to the size of the footing and when liquefaction or high levels of excess pore water pressure occur (i.e., when the factor of safety against liquefaction is less than about 1.5; see, for example Figure 27 of Marcuson et al., 1990). Figure F-14 illustrates the relative effects that high excess pore water pressure or liquefaction may have on the calculated ultimate bearing capacity of a spread footing. The effects illustrated in Figure F-14 were developed considering representative density and strength properties for non-liquefied soil (i.e., friction angle) and liquefied soil (i.e., undrained residual strength [e.g., Seed and Harder, 1990; Stark and Mesri, 1992]), the Marcuson et al. (1990) relationship between excess pore water pressure and factor of safety against liquefaction, and static ultimate bearing capacity formulations for layered systems (e.g., Meyerhof, 1974; Hanna and Meyerhof, 1980; Hanna, 1981). Meyerhof (1974) and Hanna and Meyerhof (1980) address footings in sand overlying clay, which can be used for evaluation of a liquefaction condition, treating the liquefied material as a clay with strength characterized by undrained residual strength, whereas Hanna (1981) addresses footings in strong sand overlying weak sand, which can be used for either liquefaction or high excess pore pressure.

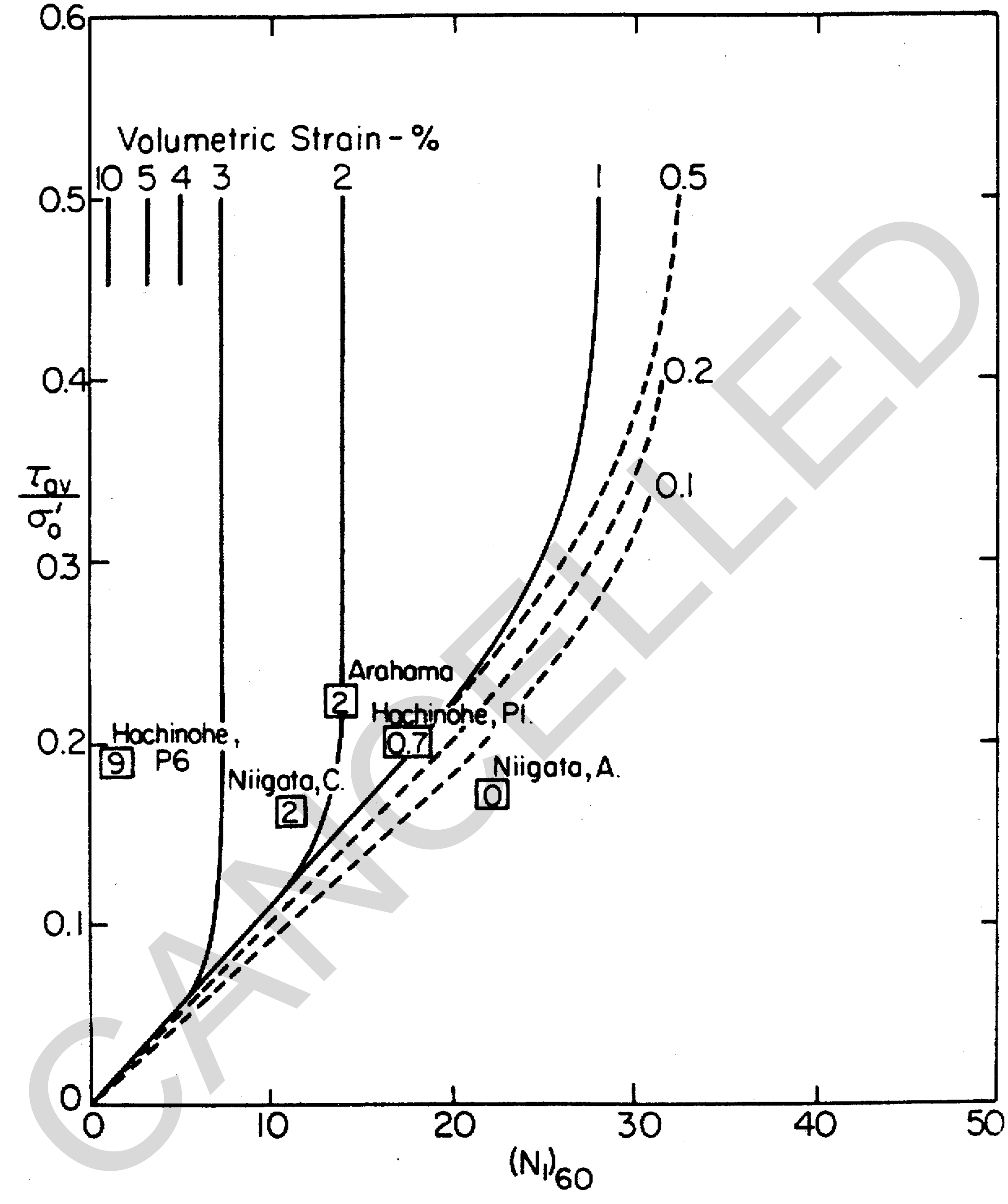


Figure F-13 Relationship between cyclic stress ratio (CSR), $(N_1)_{60}$, and volumetric strain for saturated clean sands (from Tokimatsu and Seed, 1987).

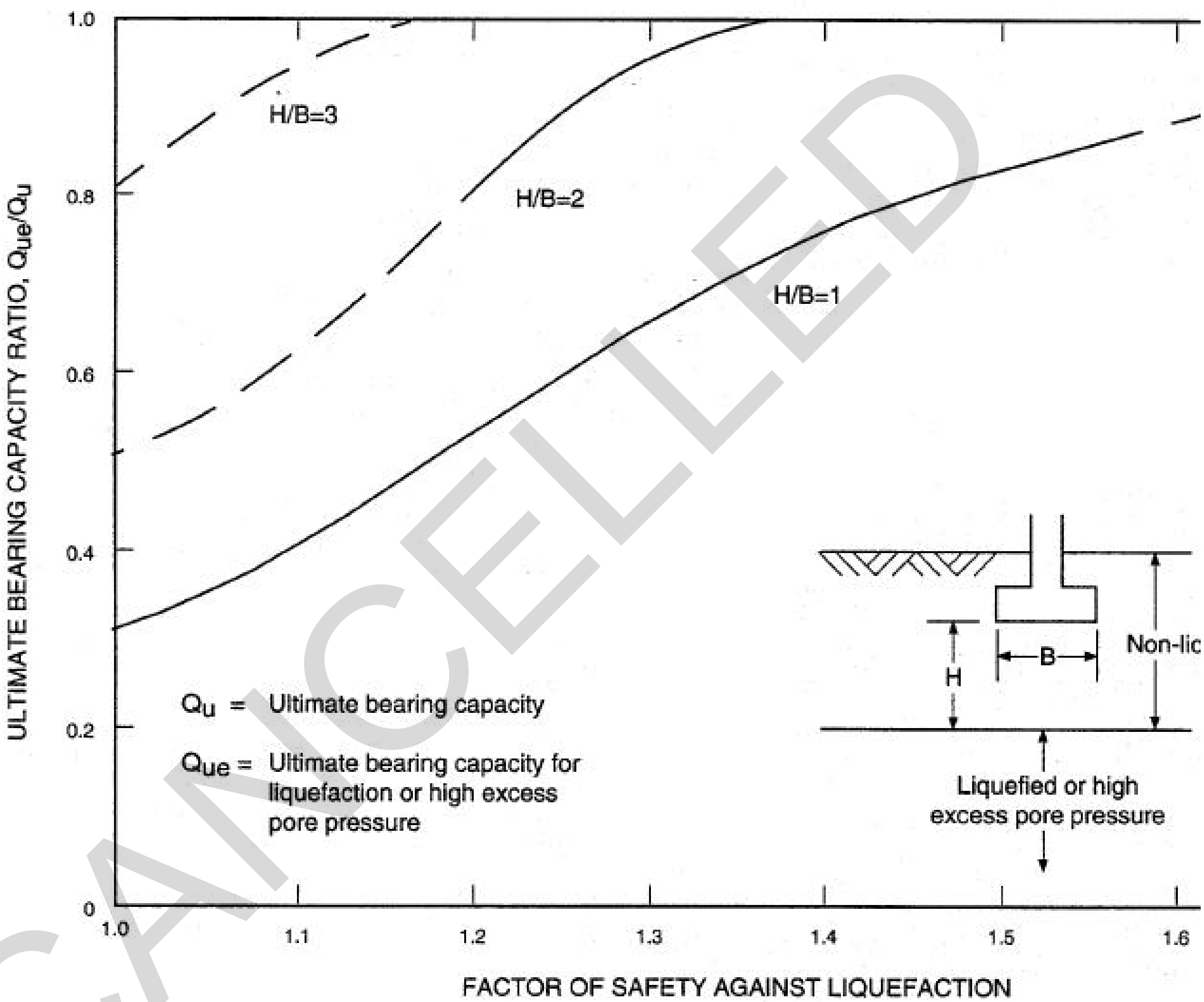


Figure F-14 Illustration of effects of liquefaction or increased pore water pressures on ultimate bearing capa
foundations.

(a) Richards et al. (1993) suggest that, in addition to strength reductions accompanying high excess pore water pressures and liquefaction, lateral inertial forces in the soil may reduce the bearing capacity of a shallow foundation system, thereby affecting the settlement performance of the foundation. However, the importance of this phenomenon in comparison to the geologic hazards addressed in this appendix is not yet clearly demonstrated by case histories. The phenomenon should be considered when evaluating foundation bearing capacities as part of a seismic rehabilitation design process.

(10) Consequences of liquefaction -- increased lateral pressures on walls. Behind a wall, the buildup of pore water pressures during the liquefaction process increases the pressure on the wall. This pressure is a static pressure which reduces with time after the earthquake as pore pressures dissipate. Ebeling and Morrison (1992) provide procedures for assessing effects of variable amounts of pore pressure buildup on the lateral pressures behind walls. In addition, the Ebeling and Morrison (1992) procedures cover the transient, dynamic pressures on walls induced by earthquake ground shaking. Both types of increases in lateral pressures due to earthquakes may influence the behavior of retaining walls, although most cases of retaining wall failures during earthquakes have been associated with liquefaction of loose sand backfills behind waterfront retaining walls. Department of Defense (1997) presents design procedures for steel sheet pile walls based on the procedures developed by Ebeling and Morrison (1992).

(11) Consequences of liquefaction -- flotation of buried structures. The potential for flotation of a buried or embedded structure can be evaluated by comparing the total weight of the buried or embedded structure with the increased uplift forces occurring due to the buildup of liquefaction-induced pore water pressures.

d. Differential compaction. The procedures described by Tokimatsu and Seed (1987) are suggested for estimating earthquake-induced settlements due to densification of saturated and unsaturated cohesionless soils. Other procedures can be used if justified. The principal soil parameter required for evaluations using the Tokimatsu and Seed (1987) method is the normalized Standard Penetration Test (SPT) resistance, $(N_1)_{60}$, in blows/foot. Appendix G provides an example of the application of this methodology. It is noted that the procedure provides an estimate of the total earthquake-induced settlement at a site for a given soil profile. The differential settlement must then be assessed based on considerations of soil variability and other factors.

e. Landsliding. Prior to performing engineering analyses to assess landslide potential, the data gathered in the screening stage should be supplemented if necessary. More detailed geologic reconnaissance and mapping may be needed. If preexisting landslides were identified at the site in the screening stage, subsurface investigations may be required to assess the slide geometry. Geotechnical data should be reviewed to assess the engineering properties of the subsurface materials in the slope(s). If sufficient data are lacking, supplemental field and laboratory testing may be required. For slopes located in stiff, nonsensitive clays, dry sands, and saturated sands that do not liquefy or lose their strength during earthquake shaking, the stability of the slopes can be evaluated using either pseudo-static analysis or deformation analysis procedures. The deformation behavior of slopes that liquefy is addressed in paragraph F-4c.

(1) Pseudo-static analysis procedure. The pseudo-static analysis can be used in the initial evaluation. In the pseudo-static analysis, inertial forces generated by the earthquake are represented by an equivalent static horizontal force (seismic-coefficient) acting on the potential sliding mass. In this analysis, the seismic coefficient should be equal to the peak ground acceleration in the vicinity of the slope. The factor of safety for a given seismic coefficient can be estimated using limit equilibrium slope stability methods. A computed factor of safety greater than one indicates that the slope is stable and further evaluations are not required. A computed factor of safety of less than one indicates that the slope will yield and deformations can be expected. In this case, an estimate of the expected slope deformations should be made using the procedures described below.

(2) Deformation analysis procedures. Simplified procedures for estimating deformations of slopes during earthquake shaking are based on the concept of yield acceleration originally proposed by Newmark (1965). Newmark's method has been modified and augmented by several investigators (Goodman and Seed, 1966; Ambraseys, 1973;

Sarma, 1975; Franklin and Chang, 1977; Makdisi and Seed, 1978; Hynes-Griffin and Franklin, 1984; Wilson and Keefer, 1985; Lin and Whitman, 1986, Yegian et al., 1991). The procedure assumes that movement occurs on a well-defined slip surface and that the material behaves elastically at acceleration levels below the yield acceleration but develops a perfectly plastic behavior above yield. The procedure involves the following steps:

- A yield acceleration, k_y, i.e., the acceleration at which a potential sliding surface would develop a factor of safety of unity, is determined using limit equilibrium pseudo static slope stability methods. Values of the yield acceleration are dependent on the slope geometry, groundwater conditions, the undrained shear strength of the slope material (or the reduced strength due to earthquake shaking), and the location of the potential sliding surface.

- The peak or maximum acceleration, k_{max}, induced within a potential sliding mass (average of the peak accelerations over the mass) must be estimated. Often this value is assumed equal to the free field ground surface acceleration, a_{max}. This neglects possible amplification of accelerations on a slope due to topographic effects, but also neglects reduction of acceleration due to reduction of ground motion with depth and averaging over the sliding mass. A specific evaluation of k_{max} considering amplifying and reducing effects can always be made using dynamic response analysis or simplified methods.

- If the maximum induced acceleration, k_{max}, exceeds the yield acceleration, k_y, downslope movement of the sliding mass occurs. Conceptually, if there is a time history of induced accelerations, some of which exceed the yield acceleration, downslope movement occurs when the induced accelerations exceed the yield acceleration. Movement stops after the time when the induced acceleration level drops below the yield acceleration. The magnitude of the potential displacements can be calculated by a simple double integration procedure of an accelerogram (see Figure F-15 for an illustration).

(a) The above procedure was used by Makdisi and Seed (1978) to develop a simplified procedure for estimating displacements in dams and embankments. Charts relating the displacements as a function of the ratio of the yield acceleration to the maximum induced acceleration (k_y/k_{max}) are shown on Figures F-16 and F-17. The displacements shown on Figures F-16 and F-17 are normalized with respect to the amplitude of the peak induced acceleration, k_{max} (expressed as a decimal fraction of gravity), and the predominant period of the induced acceleration time-history, T_o.

(b) A convenient relationship (Egan, 1994) derived from the results of Makdisi and Seed (1978) is shown on Figure F-18. The displacement per cycle of significant shaking normalized with respect to the induced peak acceleration (expressed as a decimal fraction of gravity) is plotted against the ratio of the yield acceleration to the induced peak acceleration. The curves are most representative for ground motions having a predominant period of about one second. Shown on the same figure is a relationship between earthquake magnitude and number of cycles of significant shaking (Seed and Idriss, 1982).

(c) The Newmark sliding block analysis concept was also employed by Franklin and Chang (1977) who computed permanent displacements based on a large number of recorded acceleration time-histories from previous earthquakes and a number of synthetic records. Their results are shown on Figure F-19 in terms of upper bound envelop curves for standardized maximum displacements versus the ratio of the yield acceleration to the maximum earthquake acceleration. The time-histories used by Franklin and Chang (1977) were all scaled to a peak ground acceleration of 0.5g and peak ground velocity of 30 inches per second. The displacement (inches) for particular values of peak ground acceleration, A, and velocity, V, may be obtained by multiplying the standardized maximum displacement by the quantity $V^2/1800A$, where V is in units of inches per second and A is a decimal fraction of gravity.

(d) Yegian et al. (1991) performed similar analyses using 86 ground motion records. Their computed normalized displacements are shown on Figure F-20. Their computed displacements were normalized with respect to the peak-induced

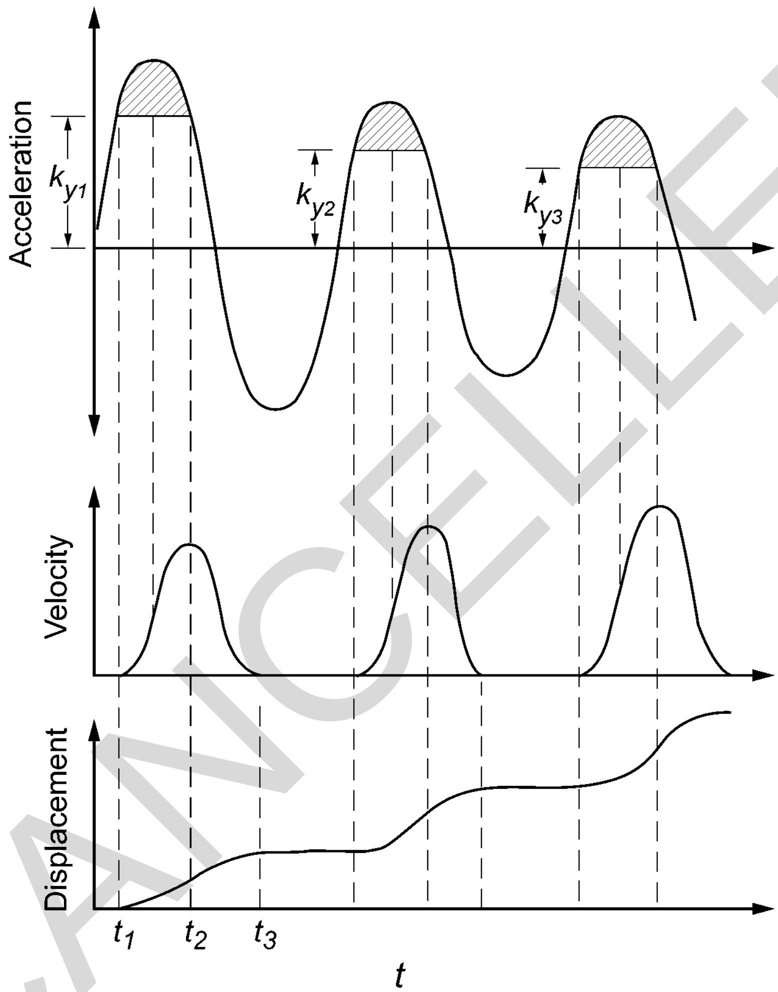


Figure F-15 Integration of acceleration time-history to determine velocities and displacements (from Goodman and Seed, 1966).

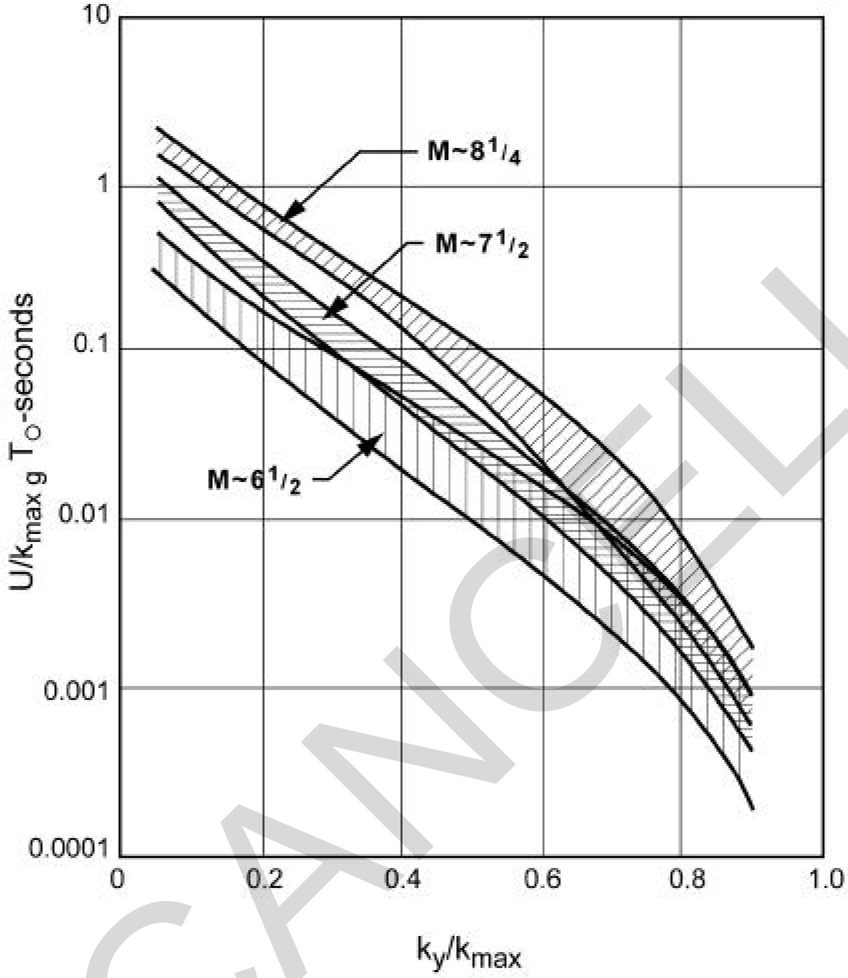


Figure F-16 Variation of normalized permanent displacement with yield acceleration-summary of all data (from Makdisi and Seed, 1978)

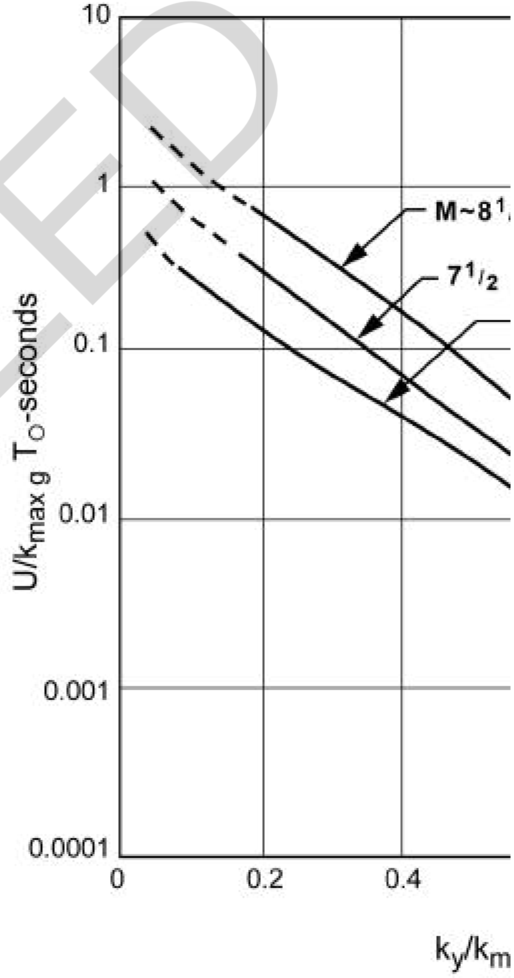


Figure F-17 Variation of averag
displacement with
Makdisi and Seed,

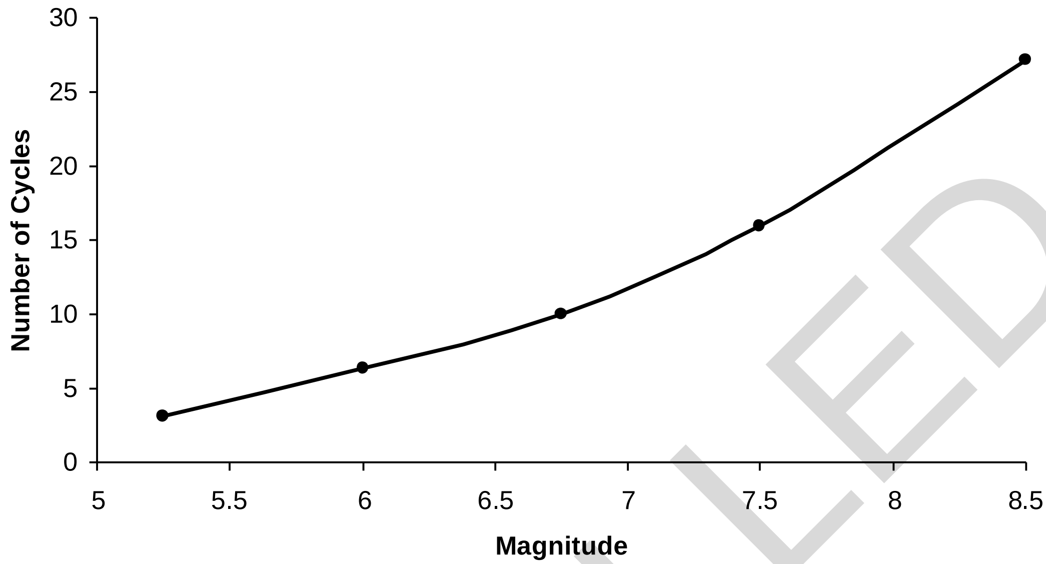


Figure F-18a Relationship between earthquake moment magnitude and number of cycles (after Seed and Idriss, 1982).

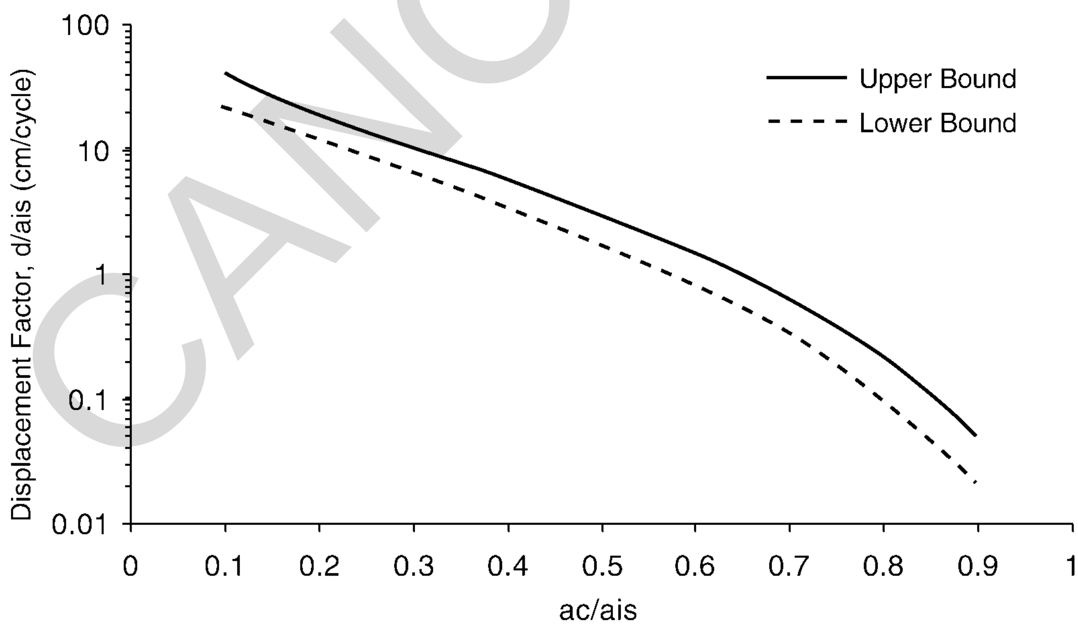
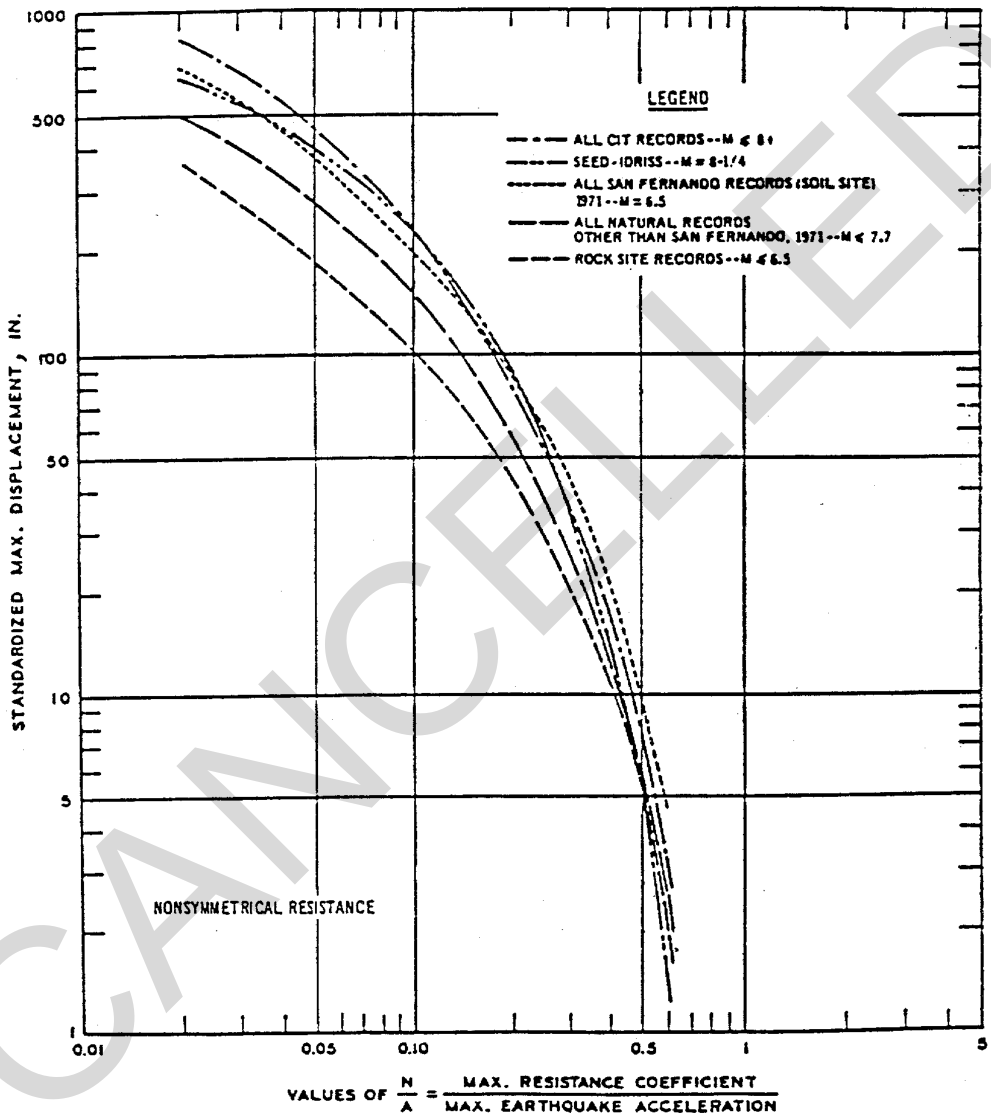


Figure F-18b Relationship between displacement factor and ratio of critical acceleration and induced acceleration (after Egan, 1994).



1 inch = 2.5 cm

Figure F-19 Upper bound envelope curves of permanent displacements for all natural and synthetic records analyzed (from Franklin and Chang, 1977).

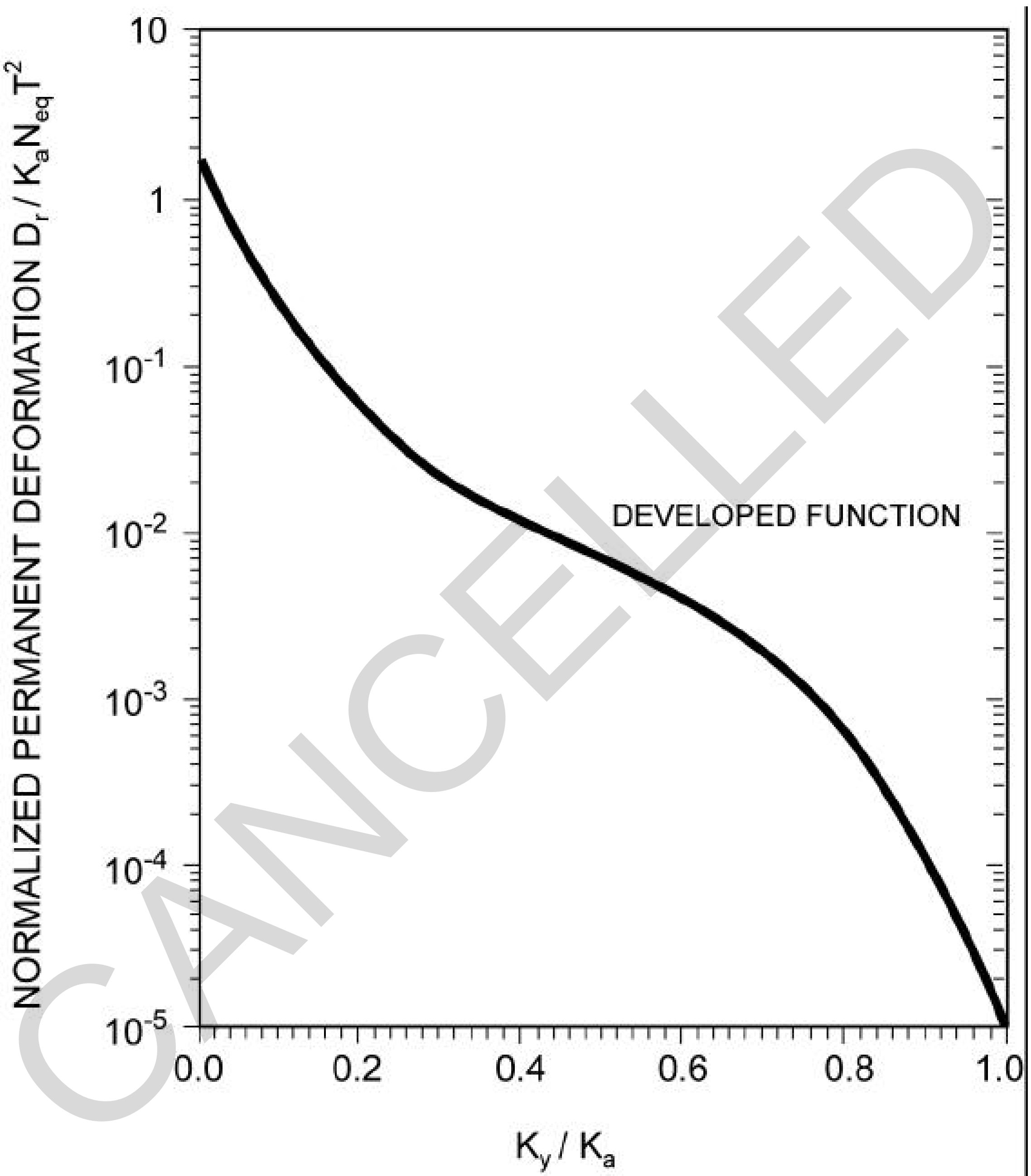


Figure F-20 Variation of normalized permanent deformation with yield acceleration (from Yegian et al., 1991).

acceleration, k_a (units of g), the number of equivalent cycles, N_{eq}, and the square of the natural period of the time-history, T.

(3) Example. An example of a detailed evaluation of landslide potential is given in Appendix G.

f. Flooding. If a facility has possible exposure to earthquake-induced flooding after applying the screening criteria in Section F-3, then further evaluations should be directed at assessing the potential, severity, consequences, and likelihood of the hazard. The evaluation of the potential for landsliding into or within a body of water utilizes methodologies described previously in the section for assessing liquefaction and landsliding. Evaluation of the height of waves that could be produced by a tsunami, seiche, or landslide requires special expertise in fields such as fluid dynamics and coastal engineering as well as seismological, geophysical, and earthquake engineering expertise in characterizing the earthquakes and ground shaking that cause these phenomena. Similarly, geological, seismological, and geophysical expertise are required to assess tectonic movements such as uplift or tilting that could cause flooding. Such studies of hazard potential and severity should be undertaken unless it can be concluded that the effects of flooding on the facility site are tolerable considering the performance objective for the facility, or the probability of occurrence of the hazard is sufficiently low that the risk can be accepted.

(1) If a facility has possible exposure to flooding from failure of a water retention structure, the agencies having jurisdiction over these facilities should be contacted to ascertain whether the structure has been evaluated or designed for appropriate ground shaking using modern seismic analysis and design methods. The potential effects of the flooding at the site should also be evaluated.

F-5. Mitigation Techniques and Considerations

In the event that a significant geologic hazard is found to exist at a facility site, alternatives for mitigating the hazard should be identified and evaluated.

a. Overall approaches to hazard mitigation. The overall approaches to hazard mitigation are (1) eliminating or reducing the hazard; (2) eliminating or reducing the consequences of the hazard; and (3) resiting the proposed facility to a less hazardous location. The following paragraphs summarize hazard mitigation strategies that have been used or considered for the different geologic hazards.

b. Surface fault rupture. There is no mitigation technique that can prevent fault rupture from occurring. Therefore, if the risk posed by the hazard of surface fault rupture is unacceptable, then the mitigation options are either avoiding the hazard by resiting or designing for the displacements.

(1) Generally, it is not feasible to design for the large and concentrated displacements associated with surface fault rupture. However, during the 1978 Managua, Nicaragua earthquake, the foundation and basement of the Banco Central building were apparently rigid and strong enough to divert a fault slippage of several inches around the building and the building sustained only minor damage due to the faulting (Wyllie et al., 1977; Youd, 1989). Thus, the possibility of mitigation by designing for fault displacement should be considered unless the displacements are of a magnitude that obviously would not be tolerable.

c. Soil liquefaction. Ground modification techniques can be considered to eliminate or reduce the liquefaction potential hazard. Soil modification techniques that can be considered include soil removal and replacement, vibratory soil densification, soil grouting, installation of drains, and installation of permanent dewatering systems. A number of ground modification techniques are summarized in Table F-3 (National Research Council, 1985; Ferritto, 1997b).

(1) Soil removal and replacement. Removing liquefiable soil and replacing it with soil that is not liquefiable (including recompaction of the excavated soil in lifts to a dense, nonliquefiable state) is a positive method for mitigating a liquefaction hazard. However, it may not be economically feasible in many cases because of the need to dewater a site to remove the soil as well as the need to retain the area surrounding the site if existing facilities are nearby. The effect of dewatering and excavation on adjacent facilities should also be evaluated.

(2) In-place soil densification. Various techniques can be considered to increase the density of the in-place soil, thereby reducing its tendency to compact and buildup pore pressures during an earthquake. A number of methods are summarized in Table F-3. In-place soil densification is often the

Table F-3 Liquefaction remediation measures (National Research Council, 1985; Ferritto, 1997b).

Method	Principle	Most Suitable Soil Conditions/Types	Maximum Effective Treatment Depth	Relative Cost
1. Blasting	Shock waves and vibrations cause limited liquefaction, displacement, remolding, and settlement to higher density.	Saturated, clean, sands; partly saturated sands and silts after flooding.	>40 m	Low
2. Vibratory probe a. Terraprobe b. Vibrorods c. Vibrowing	Densification by vibration; liquefaction-induced settlement and settlement in dry soil under overburden to produce a higher density.	Saturated or dry clean sand; sand.	20 m routinely (ineffective above 3-4 m depth); >30 m sometimes; vibrowing, 40 m	Moderate
3. Vibrocompaction a. Vibroflot b. Vibro-Composer System	Densification by vibration and compaction of backfill material of sand or gravel.	Cohesionless soils with less than 20% fines.	>30 m	Low to moderate
4. Compaction piles	Densification by displacement of pile volume and by vibration during driving; increase in lateral effective earth pressure.	Loose sandy soil; partly saturated clayey soil; loess.	>20 m	Moderate to high
5. Heavy tamping (dynamic compaction)	Repeated application of high-intensity impacts at surface.	Cohesionless soils best; other types can also be improved.	30 m (possibly deeper)	Low
6. Displacement/ compaction grout	Highly viscous grout acts as radial hydraulic jack when pumped in under high pressure.	All soils.	Unlimited	Low to moderate
7. Surcharge/ buttress	The weight of a surcharge/ buttress increases the liquefaction resistance by increasing the effective confining pressures in the foundation.	Can be placed on any soil surface.	--	Moderate if vertical drains used
8. Drains a. Gravel b. Sand c. Wick d. Wells (for permanent dewatering)	Relief of excess pore water pressure to prevent liquefaction. (Wick drains have comparable permeability to sand drains.) Primarily gravel drains; sand/wick may supplement gravel drain or relieve existing excess pore water pressure. Permanent dewatering with pumps.	Sand, silt, clay.	Gravel and sand >30 m; depth limited by vibratory equipment; wick >45 m	Moderate to high
9. Particulate grouting	Penetration grouting—fill soil pores with soil, cement, and/or clay	Medium to coarse sand and gravel	Unlimited	Lowest of grout methods
10. Chemical grouting	Solutions of two or more chemicals react in soil pores to form a gel or a solid precipitate.	Medium silts and coarser	Unlimited	High
11. Pressure-injected lime	Penetration grouting—fill soil pores with lime.	Medium to coarse sand and gravel.	Unlimited	Low

Table F-3 Liquefaction remediation measures (National Research Council, 1985; Ferritto, 1997b).

Method	Principle	Most Suitable Soil Conditions/Types	Maximum Effective Treatment Depth	Relative Cost
12. Electrokinetic injection	Stabilizing chemicals move into and fill soil pores by electro-osmosis or colloids into pores by electro-phoresis.	Saturated sands, silts, silty clays.	Unknown	High
13. Jet grouting	High-speed jets at depth excavate, inject, and mix a stabilizer with soil to form columns or panels.	Sands, silts, clays.	Unknown	High
14. Mix-in-place piles and walls	Lime, cement, or asphalt introduced through rotating auger or special in-place mixer.	Sand, silts, clays, all soft or loose inorganic soils.	>20 m (60 m obtained in Japan)	High
15. In-situ vitrification	Melts soil in place to create an obsidian-like vitreous material.	All soils and rock.	>30 m	Moderate
16. Vibro-replacement stone and sand columns a. Grouted b. Not grouted	Hole jetted into fine-grained soil and backfilled with densely compacted gravel or sand hole formed in cohesionless soils by vibro techniques and compaction of backfilled gravel or sand. For grouted columns, voids filled with a grout.	Sands, silts, clays.	>30 m (limited by vibratory equipment)	Moderate
17. Root piles, soil nailing	Small-diameter inclusions used to carry tension, shear, compression.	All soils.	Unknown	Moderate to high

most cost-effective way to mitigate a liquefaction hazard if the densification process can be undertaken without adverse effects on adjacent structures (e.g., potential effects of settle ment or vibration). Figure F-21 illustrates the technique of vibro-replacement in which a vibrating probe is inserted into the soil at close spacings and gravel or crushed rock is also placed at the vibration locations to create a dense gravel column surrounded by densified in-placed soil.

(3) Different types of grouting that can be considered include permeation grouting, compaction grouting, and formation of grouted soil columns. Permeation grouting involves injecting chemical grout into liquefiable sands to essentially replace the pore water and create a non-liquefiable solid material in the grouted zone. The more fine-grained and silty the sands, the less effective is permeation grouting. Compaction grouting involves pumping a mixture of soil, cement, and water into the ground to form bulbs of grouted material. The formation of these bulbs compresses and densifies the surrounding soil, thus reducing its liquefaction potential. However, the amounts of densification that can be achieved may be limited because static compression is less effective than vibration in densifying sands. Compaction grouting must be done carefully to avoid creating unacceptable heaving or lateral displacements of adjacent structures during the grouting process. The mixing or injection of grout locally beneath foundation locations can also be accomplished to form stabilized columns of soil to transfer vertical foundation loads to deeper nonliquefiable strata.

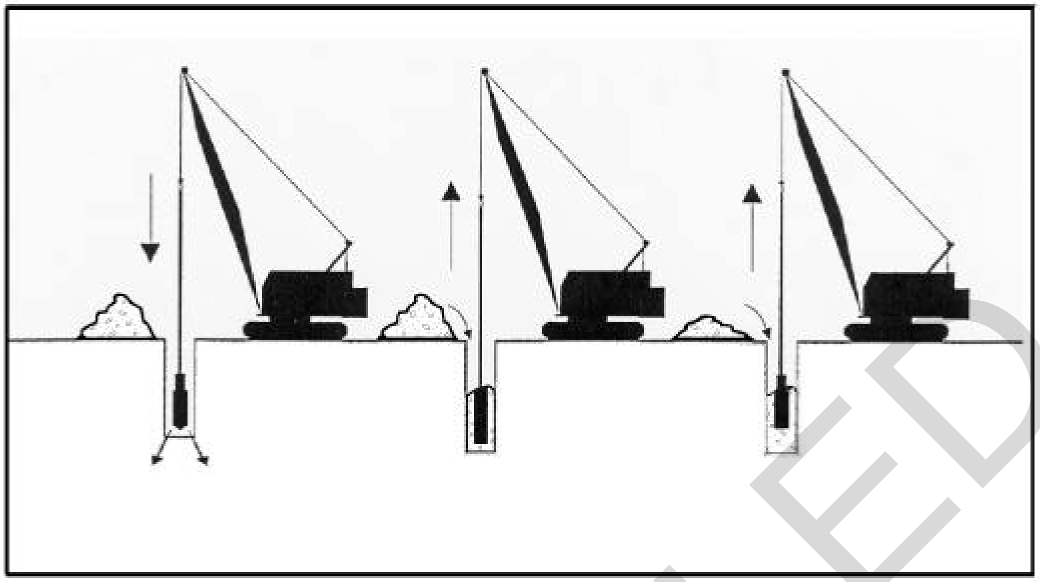
(4) Drain installation (e.g., stone or gravel columns) involves creating closely spaced, vertical columns of permeable material in the liquefiable soil strata. Their purpose is to dissipate soil pore water pressures as they build up during the earthquake shaking, thus preventing liquefaction from occurring. To achieve the objective of high permeability in the gravel column, it must be constructed by a method that avoids contamination by a mixing of the gravel with the surrounding soil. Permanent dewatering systems lower groundwater levels below liquefiable soil strata, thus preventing liquefaction.

(5) All of the above techniques can potentially be applied beneath the building area to prevent the occurrence or reduce the extent and effects of liquefaction. If the assessed consequences of liquefaction are reduction of bearing capacity and/or building settlements, these measures should be sufficient. However, if a potential for significant liquefaction-induced lateral spreading exists at a site, then ground modification beyond the immediate building area may need to be considered. This is because the potential for lateral spreading movements beneath a building may depend on the behavior of the soil mass at distances well beyond the building as well as immediately beneath it. Thus, measures to prevent lateral spreading may, in some cases, require stabilizing large soil volumes and/or constructing buttressing structures that can reduce the potential for or the amount of lateral movements.

(6) Modifications to the structure or its foundation may also be considered to mitigate the consequences. If the predicted movements are small, the structure can be strengthened to resist the deformations. The foundation system can be designed to reduce or eliminate the potential for large foundation displacements, for example by using deep foundations to achieve bearing on a deeper, non-liquefiable strata. Alternatively, a shallow foundation system can be made more rigid (for example by a system of well-reinforced grade beams or mats between isolated footings) in order to reduce the differential ground movements transmitted to the structure.

(7) Conceptual schemes to mitigate liquefaction-induced settlement or bearing capacity reductions are illustrated in Figure F-22. Conceptual schemes to mitigate liquefaction induced lateral spreading are illustrated in Figure F-23. Remediation methodologies are discussed in more detail in a number of publications, including Mitchell (1981), Ledbetter (1985), National Research Council (1985), ASCE (1997), Department of Defense (1997), and Ferritto (1997b), Mitchell et al. (1998).

d. Soil differential compaction. For cases of predicted significant differential settlements of a building, mitigation options are similar to those for mitigating liquefaction potential beneath a building. These options include modifying the soil or groundwater conditions beneath the building, designing the structure to withstand the ground movements, or modifying the foundation system by deepening or stiffening.



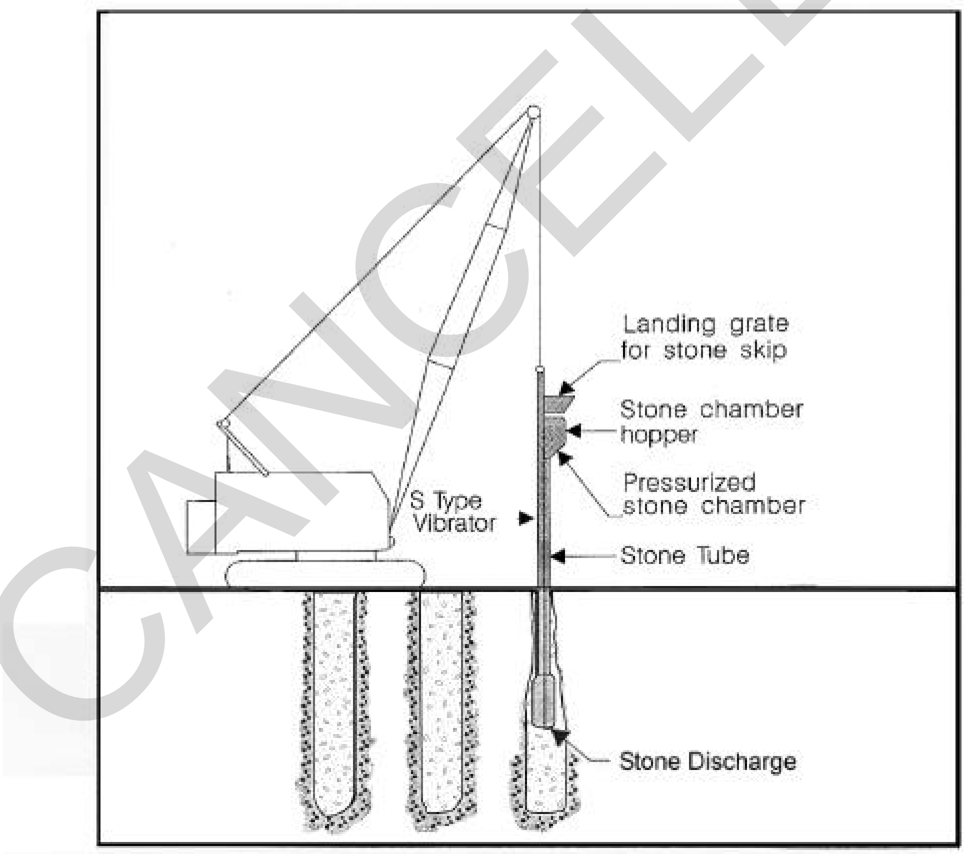
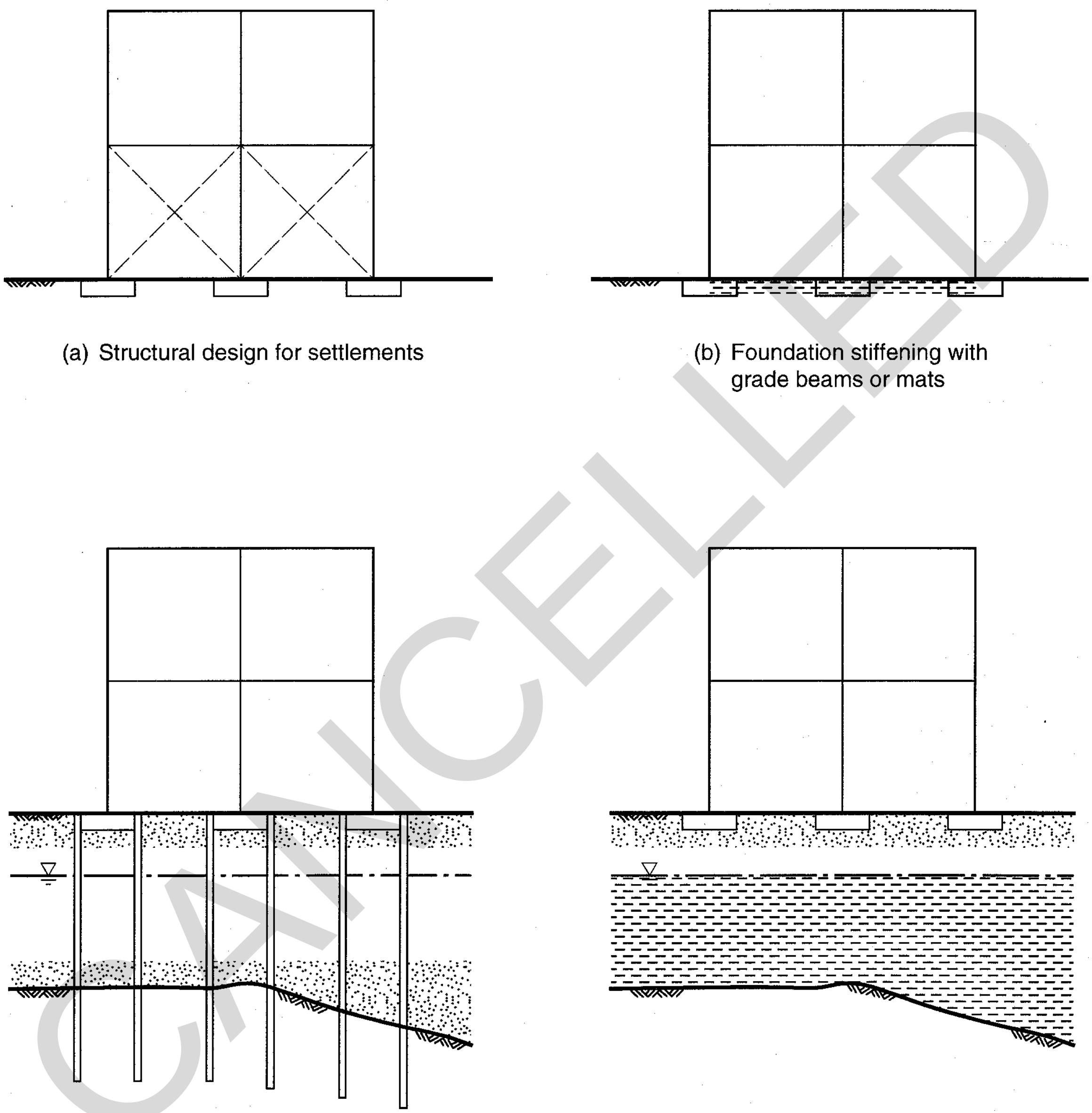


Figure F-21 Vibroreplacement and installation of stone columns (after Baez and Martin, 1992; Department of Defense, 1997).



(a) Structural design for settlements

(b) Foundation stiffening with grade beams or mats

(c) Deep foundation (structural piles, grout columns beneath foundations, etc.)

(d) Areal ground improvement (densification, grouting, stone columns, dewatering, etc.)

Figure F-22 Conceptual schemes to resist liquefaction-induced settlement or bearing capacity reductions.

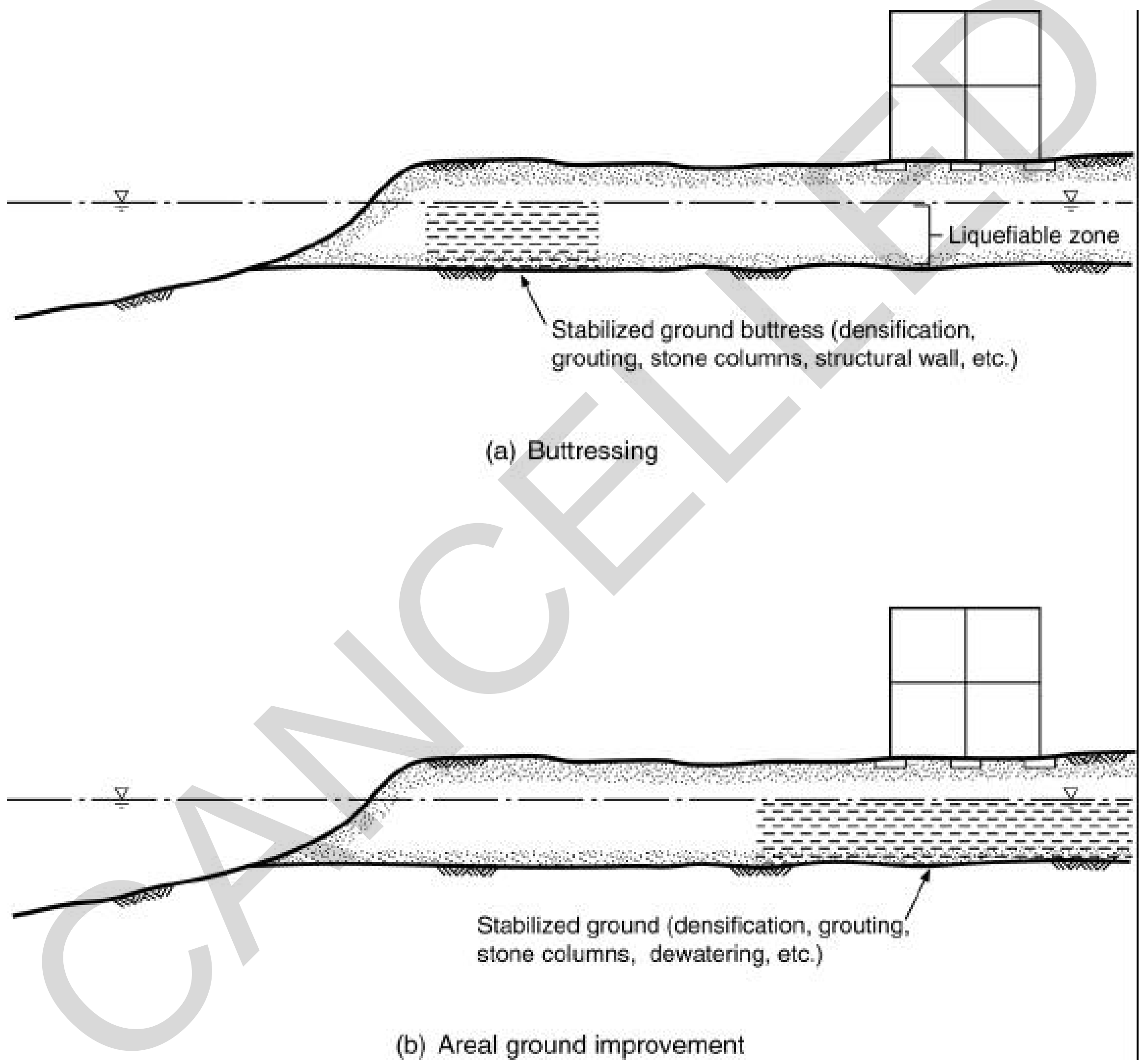


Figure F-23 Conceptual schemes to resist liquefaction-induced lateral spreading.

e. *Landsliding*. If a significant landslide risk to a facility exists, it is generally difficult to design the structure or its foundation to withstand the landslide movement. Mitigating measures typically involve some form of slope stabilization, such as regrading, buttressing, subsurface drainage, or ground modification. If a hazard exists to a structure from rockfalls or shallow soil flows on a slope above the structure, mitigating measures include removal of the material susceptible to failure, buttressing or other stabilization to prevent failure, or creating walls or earth berms to catch or deflect falling rocks or soil flows.

f. *Flooding*. If the depth and velocity of water associated with flooding is not too great, the hazard can be mitigated by creating walls or breakwaters to prevent the water from reaching the structure or dissipating its energy. For floodwaters substantially above the facility elevation or moving with great velocity, resiting may be the only feasible alternative to mitigate the hazard.

F-6. Documentation of Geologic Hazards Evaluations

The methods employed for evaluating geologic hazards, the results of the evaluations, and the conclusions should be documented in a report prepared by the geotechnical professional.

APPENDIX G
GEOLOGIC HAZARD SCREENING AND EVALUATION EXAMPLES

G-1. Example 1 - Surface Fault Rupture Hazard Screening and Evaluation

This example illustrates the steps involved in screening a site for a surface fault rupture hazard and a subsequent site evaluation using the criteria described in paragraphs F-3 and F-4. The example given below is based on a case history study for an existing building.

a. Review of available information

The building site is located within a developing metropolitan area in a tectonically active region. Twelve moderate- to large-magnitude earthquakes have occurred in the region surrounding the site within the last 160 years. The building is a 137-m (450-foot) long by 61-m (200-foot) wide, one- to two-story structure.

(1) Geotechnical investigations indicated that the site is underlain by volcanic (basalt) and sandstone materials located within a meter of the ground surface. Based on these studies, the original building design called for the building to be supported on shallow foundations extending to bedrock, which was reported to occur close to the surface beneath the building footprint. However, during construction of the foundations in the western part of the building, no rock or rock-like materials were encountered, requiring design and utilization of deep auger cast-in-place piles in this part of the building.

(2) Examination of as-built construction documents indicated the possible presence of an abrupt interface between the rock subsurface conditions to the east and deep soil conditions to the west on the property. When plotted on a map (Figure G-1), these data supported the presence of a steeply dipping rock/soil contact that extended across the site on a north-northwesterly alignment. This trend is consistent with the orientation of known active faults within the site's tectonic environment and also with the direction of the channel of a nearby river.

b. Fault rupture hazard screening

Based on the data available in the geotechnical reports and the construction documents, it was not possible to judge whether the apparent soil/rock contact was a buried erosional channel margin, reflecting a former position of the river, the manifestation of geologically young faulting with a down-on-the-west dip-slip component, or the result of some other process. To evaluate the possibility of active faulting beneath the site, a surface fault rupture hazard screening was performed.

(1) The screening consisted of the three steps outlined in paragraph F-3. First, geological maps from the U.S. Geological Survey, the state geologic survey, and the county were reviewed. These maps showed that the site is located on a gentle, west-plunging anticlinal fold that was not interpreted to be cut by faults. The maps did reveal, however, the presence of an active, northwest-trending fault within 0.6 miles (1 km) west of the site and another potentially active fault within 0.6 miles (1 km) north of the site. The potentially active fault is not well expressed topographically, and it appeared not to cut deposits interpreted to be of Holocene age (last 11,000 years) but does displace rocks of Quaternary age (last 1.8 million years). Secondly, a review of topographic maps of the site and vicinity revealed no features suggestive of a fault-related origin beneath the site. Third, black and white aerial photographs of the building and vicinity, flown prior to site modification by grading, were examined stereographically. The photos confirmed the presence of the anticlinal fold, but soil cover at the site obscured any fault-related dislocations that might be present in the volcanic layers beneath the site.

(2) The screening process yielded no evidence that faults were present beneath the site. However, the close proximity of a known active fault west of the site having a nearly identical trend to the inferred soil/rock boundary beneath the western part of the building, as well as the close proximity of a fault north of the site that displaced Quaternary-aged sediments, meant that the possibility of a surface fault rupture hazard could not be ruled out. Therefore, further evaluation of the potential for surface fault rupture at the site was performed.

c. Fault rupture hazard evaluation

An exploratory trench was excavated to a depth of 6 to 9 feet (1.8 to 2.7 m) across a portion of an open field

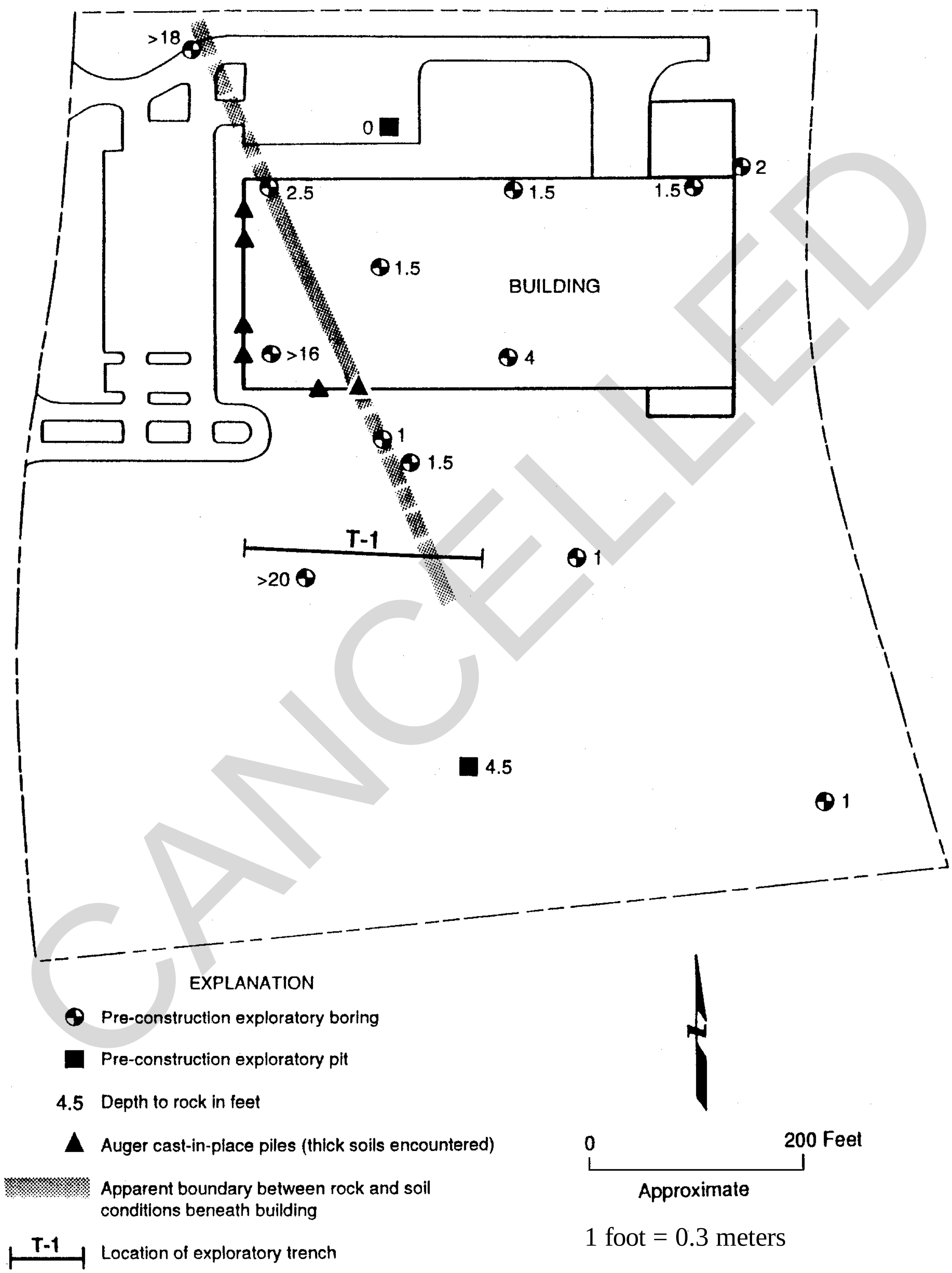


Figure G-1 Map of building site.

south of the building. The trench was sited to intersect the projected trend of the apparent linear soil/rock contact interpreted to lie beneath the western end of the building (Figure G-1). The walls of the trench were cleaned of the smeared soil coating, examined for evidence of faulting, and logged.

(1) The trench exposed no soil-bedrock contact. On the basis of the trench, it was concluded that the entire building is underlain by alluvial fan channel deposits consisting of cobbles and boulders of unweathered basalt in a fine-grained matrix. These hard channel deposits, which had a source east of the site, may have caused "refusal" during the pre-development geotechnical borings, leading to an interpretation that bedrock had been encountered in the easterly portion of the site. It was observed in the trench that the alluvial deposits became finer-grained toward the west. Toward the east, there was an increasing concentration of cobbles and boulders, reflecting the deposition of coarser material toward the upstream margin of the alluvial fan. The coarser materials in the eastern portion of the site were apparently interpreted as bedrock in the original geotechnical investigation. Examination of the trench did not reveal any evidence of faulting, and it was concluded that the potential for ground rupture due to faulting beneath the site was very low.

G-2. Example 2 - Liquefaction Hazard Screening

This example illustrates the steps involved in screening a site for liquefaction hazard using the criteria described in paragraph F-3. This example is based on a case history study for an existing building.

a. Review of available information

(1) Site development. The building site is located in a metropolitan area of a moderately-to-highly active tectonic region. The site development consists of two high-rise office buildings connected by an elevated bridge; both buildings were designed and constructed in the early-1970s. Each building is rectangular-shaped with a below-ground basement extending beneath the footprint of the building. Available drawings and plans for the buildings indicate that the structures are supported on systems of shallow foundations and footings situated beneath their respective basement-floor slabs. The foundation plans show the finished basement-floor slabs to be approximately 30 feet (9 m) below the existing grade adjacent to the building. The drawings and plans indicate that the footings of the perimeter walls are approximately 2.6 to 3.6 feet (0.3 to 1.1 m) below the finished basement-floor slab. The column footings for both buildings are typically square, with dimensions ranging between about 7 and 15 feet (2.1 and 4.6 m); the bottoms of these footings are generally about 5.5 to 7 feet (1.7 to 2.1 m) below the finished basement-floor slab, although some extend as deep as 11.5 to 13 feet (3.5 to 4 m).

(2) Soil conditions. The site is situated in an area mapped geologically as a Pleistocene-age formation, generally described as poorly consolidated, fine- to medium-grained sand and/or sandstone that was deposited in nearshore marine, lagoonal, and non-marine environments. Eleven soil borings drilled at the site during the buildings' original design phase encountered predominantly fine sand, silty fine sand, and fine sandy silt from the ground surface to the maximum exploration depth of about 63 feet (19.2 m). The logs of the soil borings indicate that beneath an approximately 3-foot (1-m) thick surficial veneer of sandy and clayey fill materials, the sands, silty sands, and sandy silts encountered within depths of about 15 feet (4.6 m) below the ground surface are loose to medium dense [$7 < (N_1)_{60} < 25$]. Underlying these near-surface sands and silts is a 12 to 15 foot (3.6 to 4.5 m) thick stratum of dense to very dense poorly graded fine- to medium-grained sand [$35 < (N_1)_{60} < 55$]. This sand stratum is in turn underlain by various thinner strata of generally dense to very dense sands, silty sands and occasional sandy silts [$30 < (N_1)_{60} < 60$], interbedded with very stiff to hard silty clays and clayey silts extending at least to the penetration depth of the borings. The nature of the soil materials encountered in the borings is consistent with characteristics of the mapped geology.

(3) Groundwater conditions. Groundwater was encountered at the time of drilling the borings at depths varying between about 30 and 45 feet (9.1 and 13.7 m); it is not known to what extent, if any, tidal fluctuation affected this range of variation. Based on the available data, however, it is evident that groundwater elevations along the eastern boundary of the site are shallower than those in the western portions of the site by as much as about 6 to 8 feet (1.8 to 2.4 m).

(4) Topographic conditions. According to U.S. Geological Survey topographic maps and logs of soil borings drilled at the site during the design phase for the buildings, the ground surface across the site varies between elevations of about 25 and 34 feet (7.6 and 10.5 m) above mean sea level (MSL) and slopes very gently downward to the south at a gradient of less than a degree. The only significant topographic change in the site vicinity is at the waterfront along the bay situated approximately 2500 feet (750 m) to the south and west.

(5) Historic earthquake effects. The buildings, having been constructed in the early-1970s, have experienced only relatively distant, moderate- to large-magnitude earthquakes during their existence. The ground motions from such earthquakes have been merely felt in the site area, producing Modified Mercalli Intensity (MMI) V effects, and therefore have not been of consequence to the buildings or the site. During the historical time period prior to construction of the buildings (i.e. since about 1800), the site experienced ground shaking from several moderate to large earthquakes that reportedly produced MMI V-VII effects in the area. Two events in the nineteenth century, an estimated magnitude 6.5 in 1800 and an estimated magnitude 5.9 earthquake in 1862, reportedly produced MMI VII effects in the site area; both of these events are thought to have occurred on faulting in the offshore region west of the site area. There are no reports of ground failure distress for the site vicinity associated with these or other historic earthquakes.

b. Liquefaction hazard screening

(1) Susceptibility. Based upon review of the available geologic information, the site is underlain by Pleistocene-age deposits with soil-like characteristics, rather than rock-like characteristics. Although the liquefaction susceptibility of the deposits is probably not greater than low according to Table F-1, the deposits cannot be categorically rated as having a very low susceptibility; therefore, liquefaction hazard cannot be screened out on the basis of the susceptibility criterion.

(2) Groundwater. The groundwater table at the site was encountered at depths between about 30 and 45 feet (9.1 and 13.7 m). These depths are less than 50 feet (15 m) below the ground surface; therefore, liquefaction hazard cannot be screened out on the basis of the groundwater depth.

(3) Soil conditions. The available logs of borings drilled at the site indicate that predominantly fine sand, silty fine sand, and fine sandy silt (cohesionless) deposits underlie the site and the reported penetration resistance (blowcount) data suggest that these deposits vary in compactness from loose to very dense. The loose to medium dense [i.e., $(N_1)_{60} < 30$] deposits were encountered at shallow depths within the profile, well above the groundwater table; whereas, the cohesionless deposits situated below the groundwater table are dense to very dense [i.e., $(N_1)_{60} > 30$]. Additionally, the silty clay and clayey silt strata interbedded with the deeper cohesionless deposits are described as very stiff to hard. On the basis of these soil conditions, the screening criteria indicate that the liquefaction hazard at the site is not significant and that the site may be eliminated from further liquefaction evaluation.

G-3. Example 3 - Liquefaction Potential Evaluation

a. Introduction

This example illustrates the steps involved in evaluating liquefaction potential using the Seed-Idriss empirically based methodology described in paragraph F-4. This procedure is a widely used procedure that would typically be employed for a site for which there remains the potential for a liquefaction hazard after applying the screening criteria in paragraph F-3. Also included in this example is an assessment of the consequences of liquefaction in terms of settlements.

(1) The site conditions are illustrated in Figure G-2. Approximately 50 feet (15 m) of predominantly loose to medium dense sand with lenses of clay of Holocene geologic age overlies dense (non liquefiable) sands and stiff clays. The water table is at a depth of 20 feet (6.1 m). The site cannot be screened out as having an insignificant potential for liquefaction using any of the three criteria given in paragraph F-3; therefore, the soils below 20 feet (6.1 m) depth are evaluated for their liquefaction potential and consequences. The soils above the water table cannot be screened out for differential compaction using the criteria in paragraph F-3; therefore, settlements in the upper 20 feet (6.1 m) are evaluated also.

(2) The proposed structure is a light, two-story structure to be supported on isolated spread footings bearing at a depth of 2 feet (0.6 m) below the ground surface. Because the foundation loads are light and the footings are well above the water table, there is not a potential for liquefaction to result in a foundation bearing capacity failure. Rather, the primary concern is settlement due to consolidation of the liquefied sand as pore pressures dissipate following liquefaction. The sands above the water table may also densify due to the ground shaking and contribute to the overall settlement. Settlements are estimated using the Tokimatsu and Seed (1987) methods. The site and vicinity is flat, with a slope gradient of less than 0.1 percent, and there are no free faces within thousands of meters of the site. The potential for lateral spreading movements is therefore judged to be negligible.

b. Liquefaction potential

A plot of the Standard Penetration Test (SPT) blowcounts (N-values) in sands versus depth is shown in Figure G-3. These blowcounts were obtained in borings using recommended methods described in Seed et al. (1985) and Youd and Idriss (1997) and no energy correction to the values is required. For assessment of liquefaction potential, the N-values are converted or normalized to $(N_1)_{60}$ values. This involves adjusting the values to a common effective overburden pressure of 1 tsf (96 kPa) using the relationship in Figure G-4. The calculations for each of the five borings drilled at the site are shown in Table G-1. The sands at the site contain varying amounts of silty fines (i.e., the percentage of minus No. 200 sieve material). The percentage of fines influences the liquefaction susceptibility, as shown in Figure G-5 (Youd and Idriss, 1997; Seed et al.,1985), which will be used to assess the liquefaction potential. For this evaluation, it is desired to use the correlation curve for clean sands (# 5 percent fines) in Figure G-5. Therefore, it is necessary to further adjust the $(N_1)_{60}$ values of the silty sands (> 5 percent fines) to a clean sand condition. The following equations are utilized to make this adjustment in the $(N_1)_{60}$ values:

$$(N_1)_{60cs} = a + b(N_1)_{60}$$

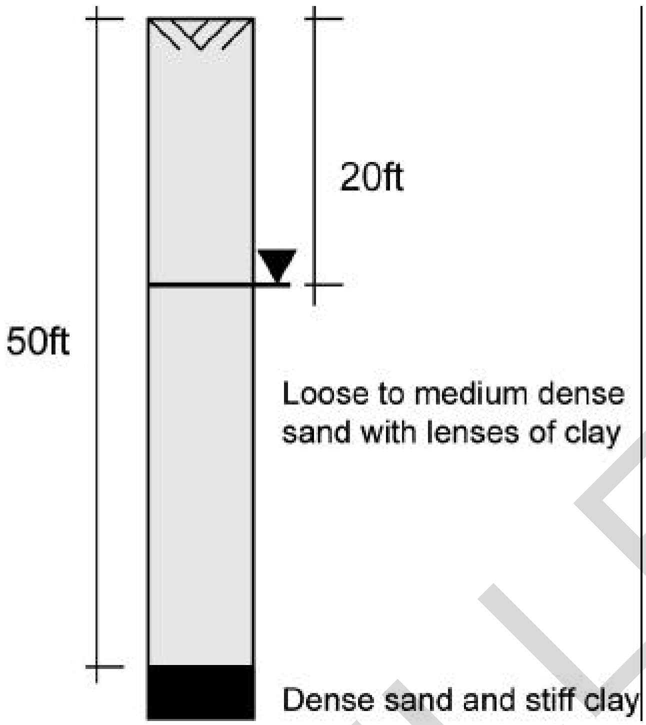
where:

α	=	0	for FC#5%
α	=	$\exp[1.76-(190/FC^2)]$	for 5%<FC<35%
α	=	5.0	for FC$35%
β	=	1.0	for FC#5%
β	=	$[0.99+(FC^{1.5}/1000)]$	for 5%<FC<35%
β	=	1.2	for FC$35%

where FC is the fines content (expressed as a percentage) measured from laboratory gradation tests from retrieved soil samples.

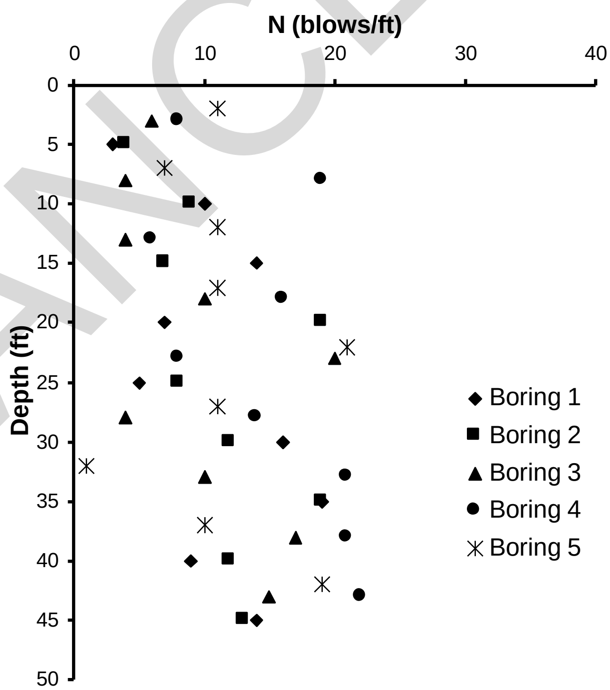
The adjusted $(N_1)_{60}$ clean-sand values are shown in the right-hand column of Table G-1 and plotted versus depth in Figure G-6.

(1) The next step is to assess the "critical" $(N_1)_{60}$ values for the site, i.e. the $(N_1)_{60}$ values dividing expected liquefaction and non-liquefaction behavior. To accomplish this, the cyclic stress ratio, t_a / s'_o, induced in the soil by the earthquake ground shaking is calculated as a function of depth for depths below the ground water table. The simplified procedure developed by Seed and Idriss (1971) is used to



1 foot = 0.3 meters

Figure G-2 Site soil profile.



1 foot = 0.3 meters

Figure G-3 Plot of SPT blowcounts vs. depth.

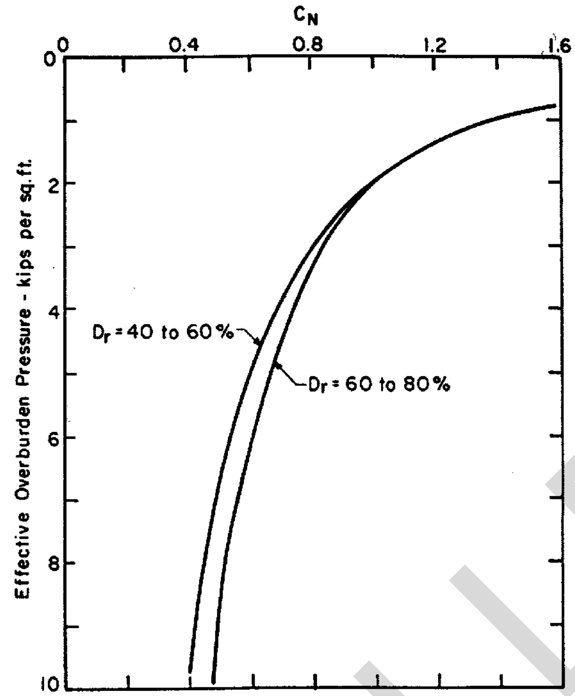


FIG. 4.—Chart for Values of C_N

1 ksf = 47.9 kPa

Figure G-4 Relationship between C_N and σ_o' (from Seed et al., 1985).

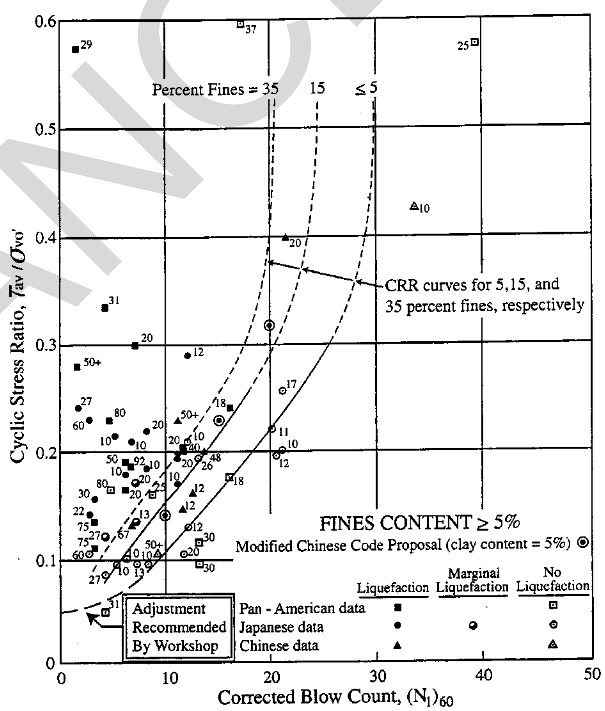


Figure G-5 Relationship between cyclic stress ratio (CSR) causing liquefaction and $(N_1)_{60}$ (from Seed et al., 1985; Youd and Idriss, 1997).

Table G-1 Calculation of the $(N_1)_{60\ cs}$ values.

Boring 1							
Depth	σ_o	σ_o'	N	C_N	$(N_1)_{60}$	Fines Content	$(N_1)_{60\ cs}$
ft	psf	psf	blows/ft		blows/ft	%	blows/ft
5	575	575	3	1.60	5	10	6
10	1150	1150	10	1.26	13	5	13
15	1725	1725	14	1.05	15	2	15
20	2300	2300	7	0.90	6	15	9
25	2875	2563	5	0.86	4	3	4
30	3450	2826	16	0.82	13	8	13
35	4025	3089	19	0.78	15	5	15
40	4600	3352	9	0.75	7	7	7
45	5175	3615	14	0.71	10	9	11

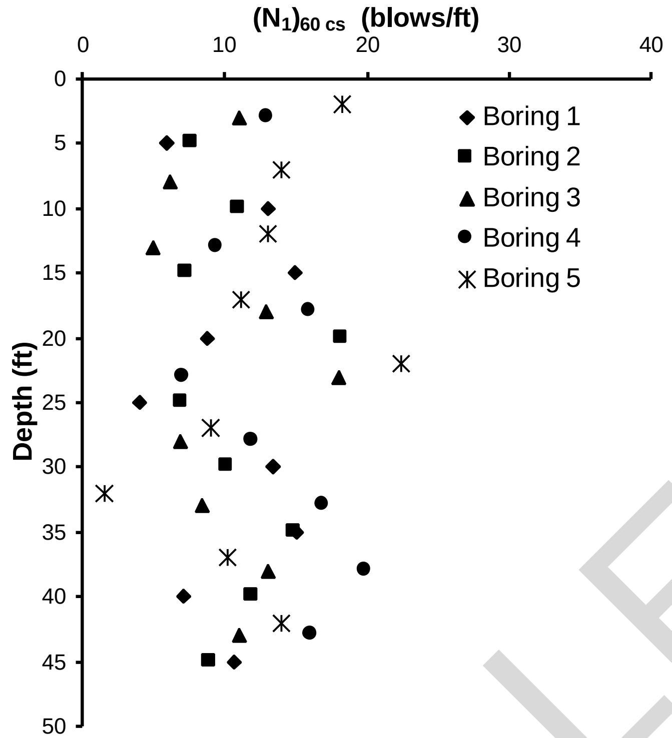
Boring 2							
Depth	σ_o	σ_o'	N	C_N	$(N_1)_{60}$	Fines Content	$(N_1)_{60\ cs}$
ft	psf	psf	blows/ft		blows/ft	%	blows/ft
5	575	575	4	1.60	6	12	8
10	1150	1150	9	1.26	11	4	11
15	1725	1725	7	1.05	7	8	7
20	2300	2300	19	0.90	17	10	18
25	2875	2563	8	0.86	7	2	7
30	3450	2826	12	0.82	10	7	10
35	4025	3089	19	0.78	15	4	15
40	4600	3352	12	0.75	9	15	12
45	5175	3615	13	0.71	9	3	9

Boring 3							
Depth	σ_o	σ_o'	N	C_N	$(N_1)_{60}$	Fines Content	$(N_1)_{60\ cs}$
ft	psf	psf	blows/ft		blows/ft	%	blows/ft
3	345	345	6	1.60	10	10	11
8	920	920	4	1.45	6	7	6
13	1495	1495	4	1.13	5	3	5
18	2070	2070	10	0.97	10	15	13
23	2645	2458	20	0.88	18	4	18
28	3220	2721	4	0.84	3	20	7
33	3795	2984	10	0.80	8	8	8
38	4370	3247	17	0.75	13	2	13
43	4945	3510	15	0.73	11	6	11

Table G-1 Calculation of the $(N_1)_{60\,cs}$ values. (continued)

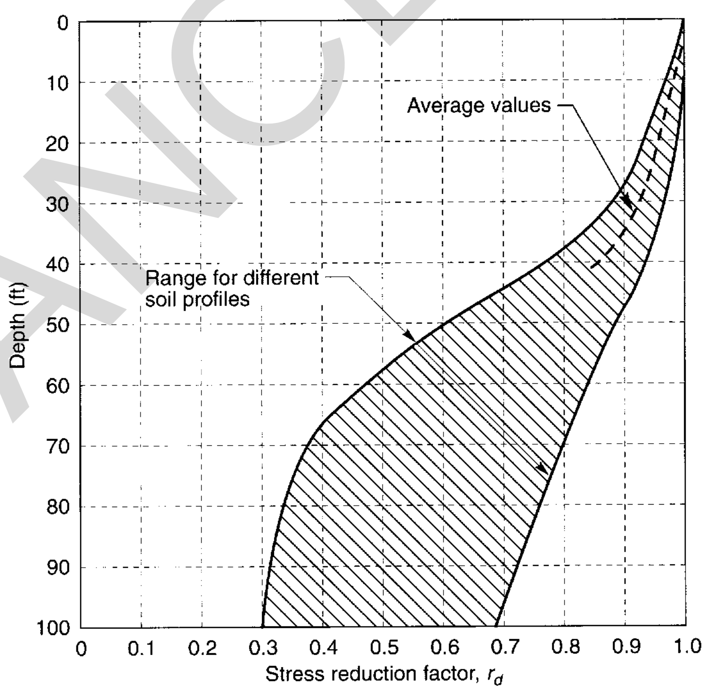
Boring 4							
Depth	**σ_o**	**σ_o'**	**N**	**C_N**	**$(N_1)_{60}$**	**Fines Content**	**$(N_1)_{60\,cs}$**
ft	**psf**	**psf**	**blows/ft**		**blows/ft**	**%**	**blows/ft**
3	345	345	8	1.60	13	5	13
8	920	920	19	1.45	28	9	29
13	1495	1495	6	1.13	7	14	10
18	2070	2070	16	0.97	16	2	16
23	2645	2458	8	0.88	7	7	7
28	3220	2721	14	0.84	12	3	12
33	3795	2984	21	0.80	17	5	17
38	4370	3247	21	0.75	16	17	20
43	4945	3510	22	0.73	16	6	16

Boring 5							
Depth	**σ_o**	**σ_o'**	**N**	**C_N**	**$(N_1)_{60}$**	**Fines Content**	**$(N_1)_{60\,cs}$**
ft	**psf**	**psf**	**blows/ft**		**blows/ft**	**%**	**blows/ft**
2	230	230	11	1.60	18	7	18
7	805	805	7	1.50	11	15	14
12	1380	1380	11	1.20	13	3	13
17	1955	1955	11	1.00	11	7	11
22	2530	2405	21	0.91	19	15	22
27	3105	2668	11	0.84	9	4	9
32	3680	2931	1	0.81	1	9	2
37	4255	3194	10	0.77	8	13	10
42	4830	3457	19	0.74	14	2	14



1 foot = 0.3 meters

Figure G-6 Plot of $(N_1)_{60\ cs}$ vs. depth.



1 foot = 0.3 meters

Figure G-7 Relationship between r_d and depth (from Seed and Idriss, 1971).

calculate the cyclic stress ratio, as follows (Seed and Idriss, 1971; Seed et al., 1985):

$$CSR = \frac{t_a}{s'_o} = 0.65\frac{a_{\max}}{g}\cdot\frac{s_o}{s'_o}\cdot r_d$$

where a_{max} is the free field surface peak ground acceleration, which is equal to 0.25g for this example problem, s_o is to total vertical stress at depth z, s'_o is the effective vertical stress at depth z, and r_d is a stress reduction factor with values given by Figure G-7. The first five columns of Table G-2 show the calculation of induced cyclic stress ratio, t_a / s'_o . Having this stress ratio, Figure G-5 is used to obtain the corresponding values of critical $(N_1)_{60}$ from the CRR curve for clean sands (# 5 percent fines). This curve is approximated by the following equation:

$$CRR_{7.5} = \frac{a + cx + ex^2 + gx^3}{1 + bx + dx^2 + fx^3 + hx^4} \quad \text{for x < 30}$$

where:

a	=	0.048
b	=	-0.1248
c	=	-0.004721
d	=	0.009578
e	=	0.0006136
f	=	-0.0003285
g	=	-0.00001673
h	=	0.000003714
x	=	$(N_1)_{60\ cs}$

However, for this site, the peak ground acceleration of 0.25g is caused by a magnitude 6.75 earthquake, whereas the curve in Figure G-5 is for a magnitude 7.5 earthquake. Therefore the curve needs to be adjusted to a magnitude 6.75 condition using the factors in Table F-2 (Seed and Idriss, 1982; Seed et al., 1983, 1985). The adjustment factor to the ordinate of the curve is 1.13. This factor, denoted K_m, is shown in Column VI of Table G-2. A further adjustment of the curves has been recommended by Seed and Harder (1990) to account for the possible reduction in values of t_a / s'_o causing liquefaction if values of the effective overburden pressure, s'_o, exceed 1 tsf (96 kPa). Their recommended adjustment factors, K_σ, are shown in Figure G-8 and are a function of s'_o. Column VII in Table G-2 shows the K_s factors. Column VIII shows the critical $(N_1)_{60}$ values for a magnitude 7.5 earthquake and s'_o equal to 1 tsf (96 kPa). Column IX shows the final critical $(N_1)_{60}$ values for the design earthquake of magnitude 6.75 and the site values of s'_o. For the linear portions of the curve in G-5, the final critical $(N_1)_{60}$ values are obtained as:

$$(N_1)_{60critical}(M\,6.75, s'_o) = \frac{(N_1)_{60critical}(M\,7.5, s'_o = 1tsf)}{K_m \cdot K_s}$$

(2) The critical $(N_1)_{60}$ curve is superimposed on the $(N_1)_{60}$ data in Figure G-9. Most of the data lie to the left of the curve, indicating liquefaction is likely to occur.

c. Settlement

The next step is to estimate the settlement of the soils below 20 feet (6.1 m) depth and also associated with the compaction of the soils above 20 feet (6.1 m) depth. The procedures presented in Tokimatsu and Seed (1987) are used. The Tokimatsu and Seed correlation for volumetric strain (percent settlement) of saturated clean sand for a magnitude 7.5 earthquake is shown in Figure G-10. The correlation is similar to that for liquefaction shown in Figure G-5. For a magnitude 6.75 earthquake, the curves in Figure G-10 are adjusted upward by the factor K_m equal to 1.13. The $(N_1)_{60}$ data below the water table average about 10 blows/0.3 m (10 blows/foot) (Figure G-9). The induced cyclic stress ratio below the water table is in the range of about 0.16 to 0.19 (Table G-2). Comparing this stress ratio and a value of $(N_1)_{60}$ equal to 10 blows/foot with curves in Figure G-10 (after adjusting them upward by a factor of 1.13) indicates a volumetric strain of about 2.5 percent. Thus, for a 30-foot (9.1 m) thickness of liquefied sand, the estimated settlement is 0.025 x 30 feet (9.1 m) = 9 inches (23 cm).

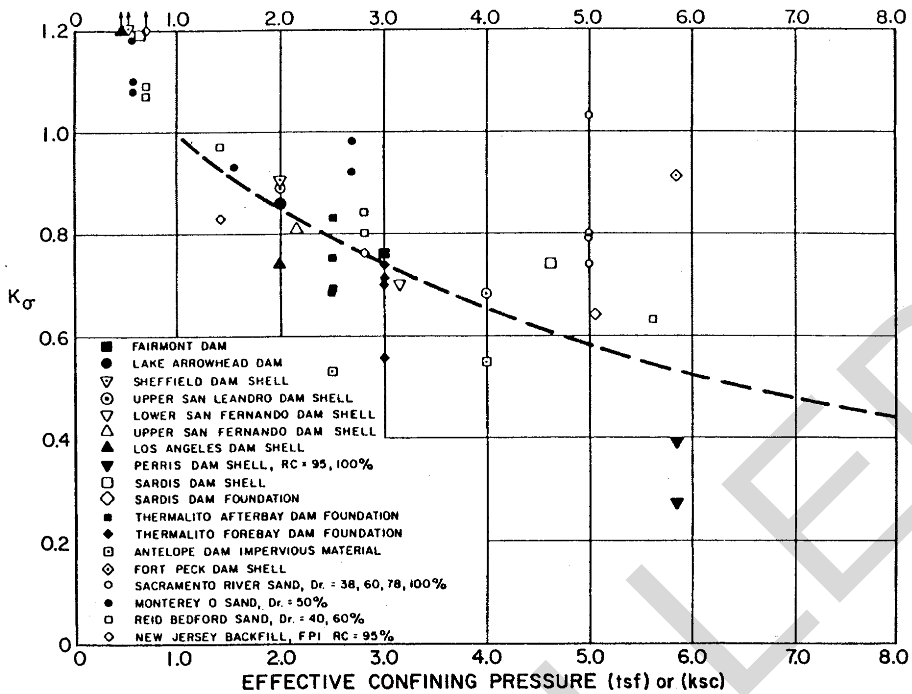
(1) Estimates of settlements in the upper 20 feet (6.1 m) of sands above the water table are made using the procedures described in Tokimatsu and Seed (1987). The first step is to calculate the shear strain developed in the soils using the relationship:

$$g_{eff}\left(\frac{G_{eff}}{G_{\max}}\right) = \frac{0.65 \cdot a_{\max} \cdot s_o \cdot r_d}{g \cdot G_{\max}}$$

Table G-2 Calculation of CSR and $(N_1)_{60\ critical}$.

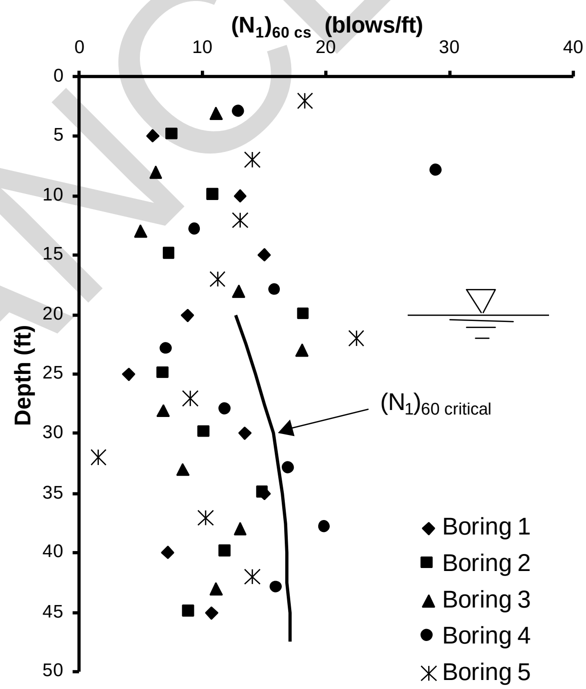
Water Table at 20 ft	Design Earthquake:
γ_t = 115 pcf	PGA = 0.25 g
	M_w = 6.75

Depth	S_o	S_o'	r_d	CSR	K_m	K_S	$(N_1)_{60\ cr}$ M = 7.5	$(N_1)_{60\ cr}$ M = 6.75
ft	psf	psf					blows/ft	blows/ft
I	II	III	IV	V	VI	VII	VIII	IX
5	575	575	0.99	0.16	1.13	N/A	N/A	N/A
10	1150	1150	0.98	0.16	1.13	N/A	N/A	N/A
15	1725	1725	0.97	0.16	1.13	N/A	N/A	N/A
20	2300	2300	0.95	0.15	1.13	0.99	14.2	12.7
22.5	2588	2432	0.95	0.16	1.13	0.99	15.0	13.5
25	2875	2563	0.94	0.17	1.13	0.98	15.8	14.3
27.5	3163	2695	0.93	0.18	1.13	0.97	16.4	15.0
30	3450	2826	0.92	0.18	1.13	0.96	17.0	15.7
32.5	3738	2958	0.91	0.19	1.13	0.94	17.2	16.2
35	4025	3089	0.90	0.19	1.13	0.93	17.3	16.4
37.5	4313	3221	0.87	0.19	1.13	0.92	17.4	16.7
40	4600	3352	0.85	0.19	1.13	0.92	17.4	16.8
42.5	4888	3484	0.82	0.19	1.13	0.91	17.4	16.9
45	5175	3615	0.80	0.19	1.13	0.90	17.3	17.1
47.5	5463	3747	0.77	0.18	1.13	0.89	17.2	17.1



1 tsf = 96 kPa

Figure G-8 Relationship between K_s and s_o' (from Seed and Harder, 1990).



1 foot = 0.3 meters

Figure G-9 $(N_1)_{60\,critical}$ superimposed on $(N_1)_{60\,cs}$ data with depth.

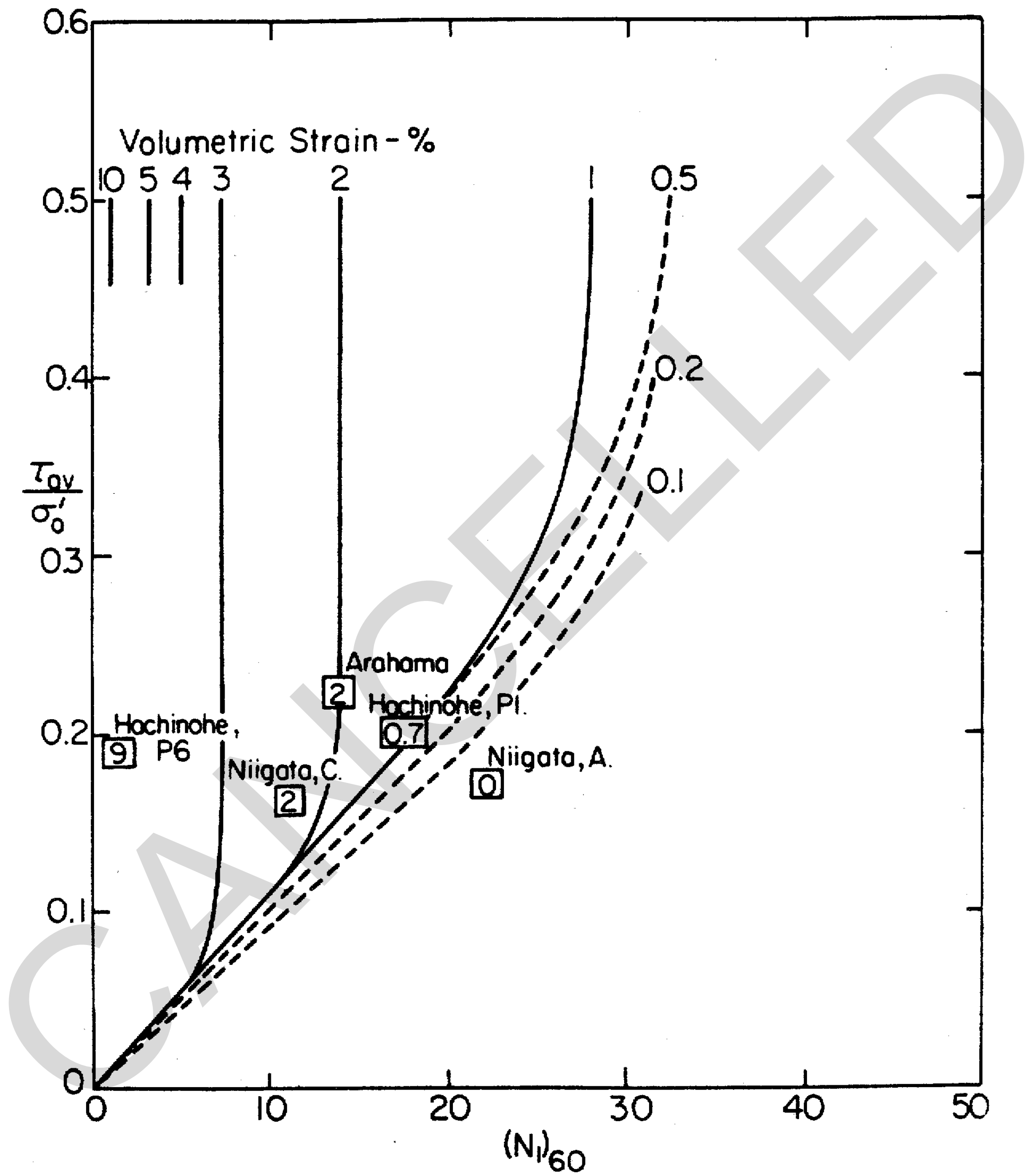


Figure G-10 Correlation for volumetric strain, cyclic stress ratio (CSR), and $(N_1)_{60}$ for sands (from Tokimatsu and Seed, 1987).

in which γ_{eff} is the effective average shear strain induced in the soil at a certain depth by the design earthquake ground shaking, G_{eff} is the shear modulus at this strain level, and G_{max} is the maximum shear modulus at a very low strain. This calculation is made for three soil layers in the upper 20 feet (6.1 m) in Table G-3. Then, using Figure G-11, γ_{eff} is obtained for the respective values of effective overburden pressure. From Figure G-12, the volumetric strains or percent settlements are obtained for the effective shear strain in each layer using an average $(N_1)_{60}$ value equal to 10 blows/0.3 m (10 blows/foot) in the upper 20 feet (6.1 m). These volumetric strains are for magnitude 7.5 and should be reduced for the shorter duration of shaking for magnitude 6.75 using Table G-4. Finally, the correlations in Figure G-12 are based on unidirectional shaking, and research by Pyke, et al. (1975) indicates that the volumetric strains due to multidirectional shaking are about twice those for unidirectional shaking. Therefore, the volumetric strains are doubled. The sum of the estimated settlements in the upper 20 feet (6.1 m) is only 0.3 inches (0.7 cm), which is additive to the 9 inches (23 cm) of settlement due to liquefaction of the underlying sands, leading to a total estimated settlement of about 9**2** inches (24 cm) beneath the building. (Note that the settlement estimates for the sand above the water table are sensitive to the level of acceleration. For example, the calculated settlements in the upper 20 feet (6.1 m) would increase from only 0.3 inches (0.7 cm) to approximately 1.6 inches (4 cm) if the peak ground acceleration increased from 0.25g to 0.50g, yet the settlements associated with liquefaction 20 feet (6.1 m) in depth would not change as long as liquefaction occurs.)

(2) Consideration of the sand variability from boring to boring as well as varying thicknesses of the sand due to presence of clay lenses across the site would lead to estimates of differential settlements between footings. If these would lead to unacceptable structural distress, then alternative mitigation measures (described in paragraph F-5) would include: (1) densifying the soils; (2) grouting the soils; (3) installing permeable drainage columns; (4) installing a permanent dewatering system to lower the ground water table to the base of the liquefiable layer (note that the effects of this method in causing consolidation of shallow clay lenses and deeper clay strata would have to be evaluated); (5) using pile or pier foundations to extend below the liquefiable layer; and (6) stiffening the foundation system by tying isolated footings together with well-reinforced grade beams or mats.

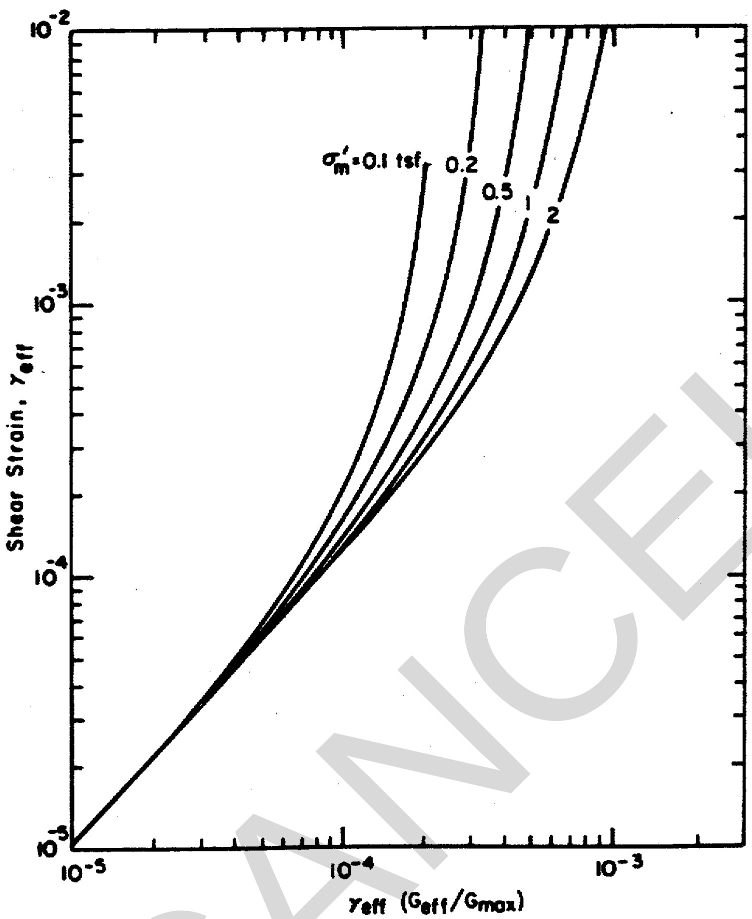


Figure G-11 Plot of induced shear strain for sands (from Tokimatsu and Seed, 1987).

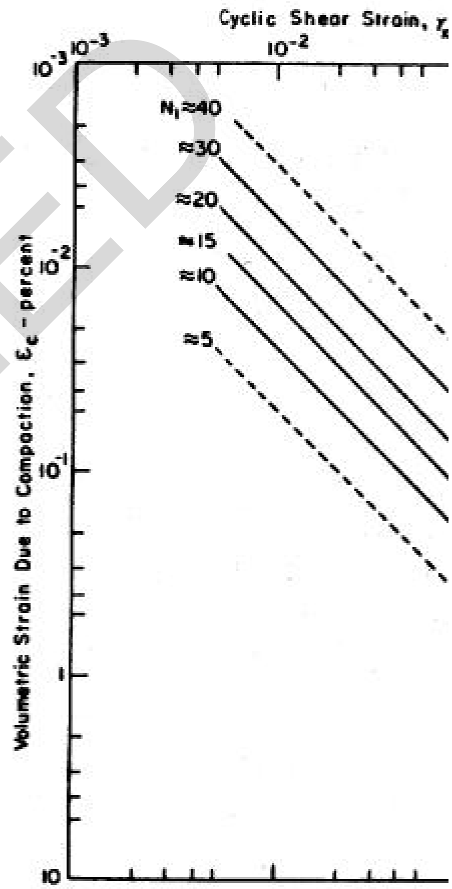


Figure G-12 Correlation for volumetr and $(N_1)_{60}$ (from Tokima

Table G-3 Calculation of settlement of sand above ground water table..

Water Table at 20 ft	Design Earthquake:
γ_t = 115 pcf	PGA = 0.25 g
	M_w = 6.75

Depth	Thickness	s_o	r_d	N_1	G_{max} [1]	g_{eff}	g_{eff}	$e_{v,}$ M_w=7.5	$e_{v,}$ M_w=6.75	2(
ft	ft	psf			psf	(G_{eff}/G_{max})		%	%	
5	7.5	575	0.99	10	8.44E+05	1.1E-04	2.2E-04	0.05	0.04	
10	5	1150	0.98	10	1.19E+06	1.5E-04	2.8E-04	0.07	0.06	
15	7.5	1725	0.97	10	1.46E+06	1.9E-04	3.4E-04	0.09	0.08	

[1]: $G_{max} = 20 * (N_1)^{1/3} * (s_m')^{1/2} * 1000$ **Total :**

Table G-4 Scaling factors for influence of earthquake magnitude on volumetric strain for dry sand: Seed, 1987).

Earthquake Magnitude	$e_{M=m}$ / $e_{M=7.5}$
M_w	
8½	1.25
7½	1.00
6:	0.85
6	0.60
53	0.40

G-4. Example 4 – Landslide Hazard Screening

This example presents the steps involved in screening a site for potential earthquake-induced landsliding hazards. The example problem is illustrated in Figure G-13.

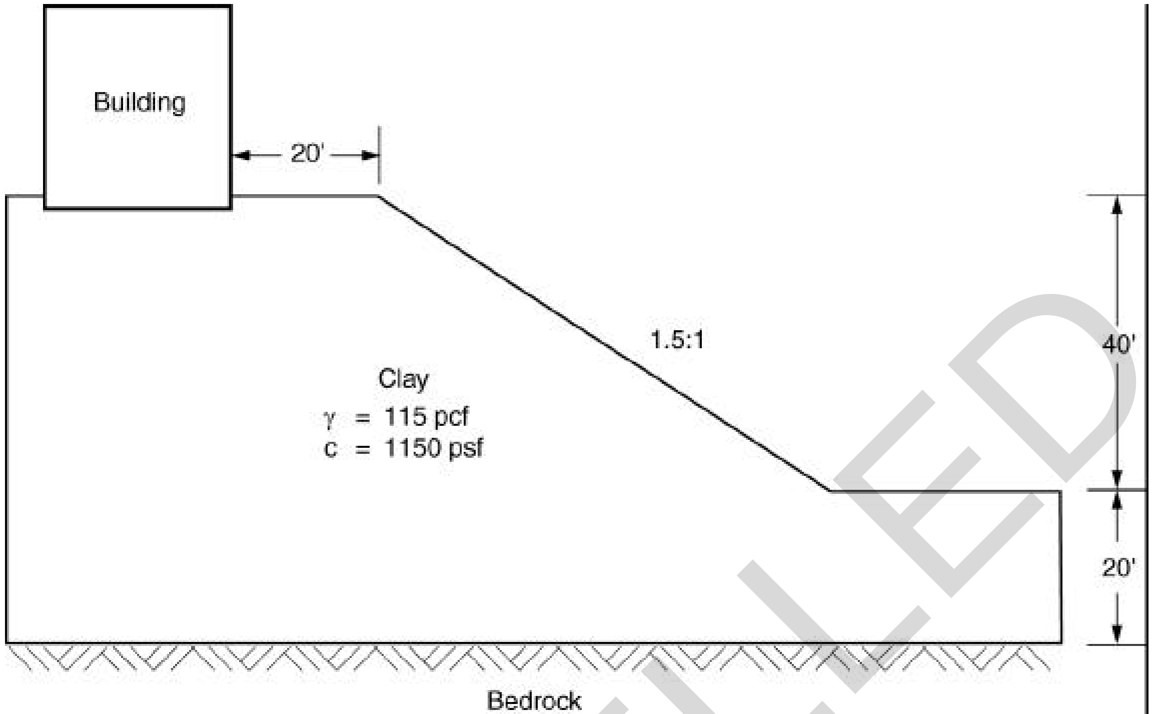
a. Review of available information

(1) Site development and soil conditions. A barracks building is located 20 feet (6.1 m) back from the crest of a 40 feet (12 m) high slope (Figure G-13). The ratio of the slope width to the slope height is approximately 1.5:1. The building is to be supported on a shallow foundation system. Soil conditions at the site consist of clay with a uniform undrained shear strength (cohesion) of approximately 1150 psf (55.2 kPa) and a unit weight of 115 pcf (18.0 kN/m^3). Bedrock is located approximately 60 feet (18 m) below the building and groundwater is not present at the site.

(2) Historic earthquake effects and postulated earthquake parameters. This site has been shaken by several moderate earthquakes. However, no known historic information indicates that earthquake-induced landsliding occurred. Inspection of the building site shows that the slope is stable under static conditions. No cracking above the slope crest or other evidence of present instability were observed. Seismic landslide hazard maps have not been developed for this area. Site specific analyses determined the MCE to have a moment magnitude of approximately 6.5 and a peak horizontal acceleration of 0.40 g. The predominant period of the induced acceleration time-history, T_o, was estimated to be 0.3 seconds.

b. Earthquake-induced landslide screening

(1) Susceptibility. To conclude that a landsliding hazard does not, each of the landslide screening criteria presented in paragraph F-3 must be satisfied. The stability of the slope during past earthquakes and present site conditions indicate no significant susceptibility to landsliding. The site is not adjacent to a shoreline. The building is located approximately 20 feet (6.1 m) from the top of the slope and a horizontal distance of 80 feet (24 m) from the toe of the slope (Figure G-13). According to the screening criteria, the building cannot be located closer than the distance of the slope height (40 feet or 12 m) from the top of the slope or closer than three times the slope height (120 feet or 37 m) from the toe of the slope. This criterion is not satisfied indicating further evaluation is required.



1 foot = 0.3 meters; 1 pcf = 0.16 kN/m^3; 1 psf = 48 Pa

Figure G-13 Profile of earthquake-induced landsliding example problem.

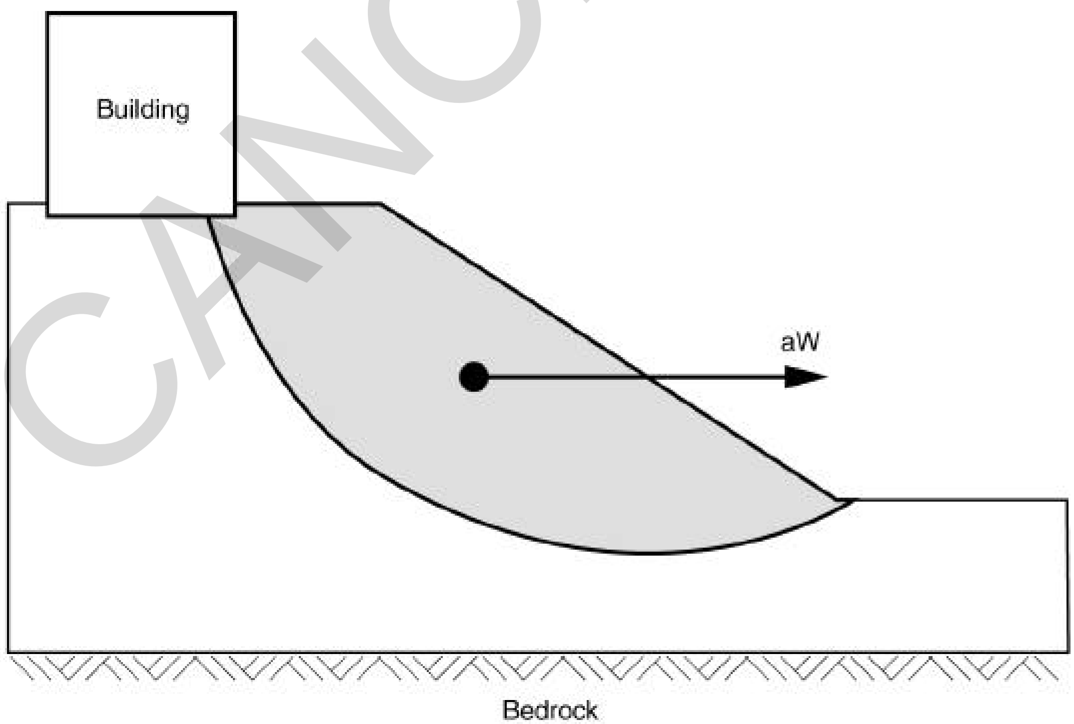


Figure G-14 Failure surface and pseudo-static load for earthquake-induced landsliding example problem.

G-5. Example 5 - Landslide hazard evaluation

The general method for evaluating the seismic stability of slopes involves both pseudo-static and deformation analysis procedures, as illustrated below.

a. Pseudo-static slope stability analysis

Pseudo-static slope stability analyses conservatively evaluate the occurrence of a slope failure due to earthquake loading. If the results of the pseudo-static analysis indicate potential deformation of the slope (factor of safety < 1), a deformation analysis is performed to estimate the displacement. A static limit-equilibrium slope stability analysis performed for the site determined that the critical failure surface would intersect the foundation of the building (Figure G-14). This failure surface was then used for the pseudo-static slope stability analysis. The seismic coefficient was assumed to be equal to the peak horizontal acceleration of 0.40 g. The results of the pseudo-static analysis indicate a marginal susceptibility to earthquake-induced landsliding with a factor of safety of 0.92. A deformation analysis was then performed to estimate the displacement.

b. Deformation analysis

The deformation analysis procedure is based on Newmark's (1965) concept of yield acceleration. For a specified potential sliding mass, the acceleration induced by the earthquake is compared with the yield acceleration. When the induced acceleration exceeds the yield acceleration, downslope movements will occur along the direction of the assumed failure plane. The movement will stop when the induced acceleration drops below the yield acceleration.

(1) Yield acceleration, k_y. The yield acceleration is the acceleration at which the potential sliding surface would develop a factor of safety of unity. For this site, k_y was determined to be 0.30 g by iteratively adjusting the seismic coefficient in the pseudo-static analysis until the factor of safety reached a value of unity.

(2) Peak or maximum acceleration, k_{max}. This parameter represents the peak or maximum acceleration induced within the sliding mass. k_{max} was assumed to be equal to the peak horizontal acceleration of 0.40 g.

(3) Acceleration ratio. The acceleration ratio is calculated by dividing the yield acceleration, k_y, by the maximum acceleration, k_{max}. For this example, the acceleration ratio is equal to 0.75.

(4) Deformation. Several simplified methods based on the concept of yield acceleration originally proposed by Newmark (1965) are utilized to estimate deformation.

(a) Makdisi and Seed (1978). The Makdisi and Seed (1978) method normalizes displacement by k_{max}, T_o, and gravity (Figure F-17). Based on the ratio of k_y to k_{max} of 0.75 and a moment magnitude of 6.5, the normalized displacement is equal to approximately 0.003 seconds (note that the units of seconds will be replaced by inches when the normalizing values are factored out). An estimated deformation of 0.14 inches (0.4 cm) was calculated by multiplying the normalized displacement by the values of k_{max}, T_o, and gravity.

(b) Egan (1994). The Egan (1994) relationship between deformation and the ratio of critical acceleration is normalized by k_{max} and the number of earthquake cycles. A magnitude 6.5 earthquake contains approximately eight cycles (Figure F-18a). Based on the ratio of k_y to k_{max} of 0.75, the displacement factor was estimated to be 0.3 (Figure F-18b). An estimated deformation of 0.4 inches (1 cm) was determined by multiplying the displacement factor by the values of k_{max} and the number of cycles.

(c) Franklin and Chang (1977). The range of the Franklin and Chang (1977) simplified method has a lower bound of one inch of deformation (Figure F-19). The critical acceleration ratio of 0.75 is outside this range. However, judging from the trend of the curves, a deformation of less than one inch (2.5 cm) can be assessed.

(d) Yegian et al. (1991). The Yegian et al. (1991) simplified method for estimating permanent deformation normalizes displacement by k_{max}, T_o^2, number of cycles, and gravity (Figure F-20). A magnitude 6.5 earthquake contains approximately eight cycles (Figure F-18a). Based on the ratio of k_y to k_{max} of 0.75, the normalized permanent deformation was estimated to be 0.001. An estimated deformation of 0.1 inches (0.03 cm) was determined by multiplying the normalized displacement value of 0.001 by the values of k_{max}, T_o^2, number of cycles, and gravity.

c. Hazard mitigation

The amount of acceptable deformation is dependent on several factors including: foundation rigidity, age of the building, building function, regulatory requirements, and economic alternatives. For this example problem, each deformation analysis method predicted less than one inch (2.5 cm) of displacement. This magnitude of displacement was determined to be acceptable considering the use and structural characteristics of the building. Thus, stabilization methods were not needed at this site.

APPENDIX H

H.1 Vehicle maintenance facility

a. Introduction: This example problem demonstrates the design of a vehicle maintenance facility. The structure is considered to be a Standard Occupancy Structure. This type of building is to be designed to meet Performance Objective 1A (protect Life Safety). Most of the structures designed for military use fall into this category. For this example it is assumed that the site has spectral accelerations of 0.75% g at 0.2 seconds and 0.40% g at 1.0 seconds per the MCE maps. The soil classification is type D.

(1) Purpose. The purpose of the is example is to demonstrate the design of a structure to meet Performance Objective 1A following the steps outlined in Table 4.5.

(2) Scope. The scope of this example problem includes; the design of all major structural elements such as the steel gravity framing, CMU shear walls and the steel braced frames, as well as the connections between the various structural elements. The design of the foundations, nonstructural elements and their connections and detailed design of some structural elements such as the concrete floor slab and pilasters are not included.

b. Building Description

(1) Function. This building is to be used as a vehicle maintenance facility. The building is not considered to be mission critical and is therefore is designed to meet the Life Safety Performance Level.

(2) Seismic Use Group. The occupancy or function of the structure does not match any of the conditions required for Special, Hazardous, or Essential Facilities set forth in Table 4-1. Therefore, the building is categorized as Seismic Use Group I.

(3) Configuration. The building is a rectangular, six bay, one-story structure. At each end of the building is an office and bathroom space. Above the office space at both ends is a mezzanine accessed by a staircase and is used for storage. The building measures 160'-0" (48.80m) long by 40'-0" (12.20m) wide in plan. The top of the roof is 20'-0" (6.10m) above the grade on average with the roof sloping in the transverse direction (N-S) to allow for drainage.

(4) Structural Systems.

Gravity System

Steel framing is selected to support gravity loads. The frames provide for the large open floor areas needed for the motor pool. The steel beams around the perimeter of the building are used to span the large roll-up door openings, carrying the gravity loads from the roof as well as the weight of the metal roll-up doors.

The roof consists of untopped 1-1/2 inch (38.1mm) metal deck that spans 6'-8" (2.03m) to open web steel joists. The joists are selected due to their ability to span 40'-0" (12.2m) transversely to steel beams which are supported by steel columns spaced at 20'-0" (6.10m) on center. The columns are supported by spread footings and the walls are supported by strip footings (the design of the footings is omitted for this example).

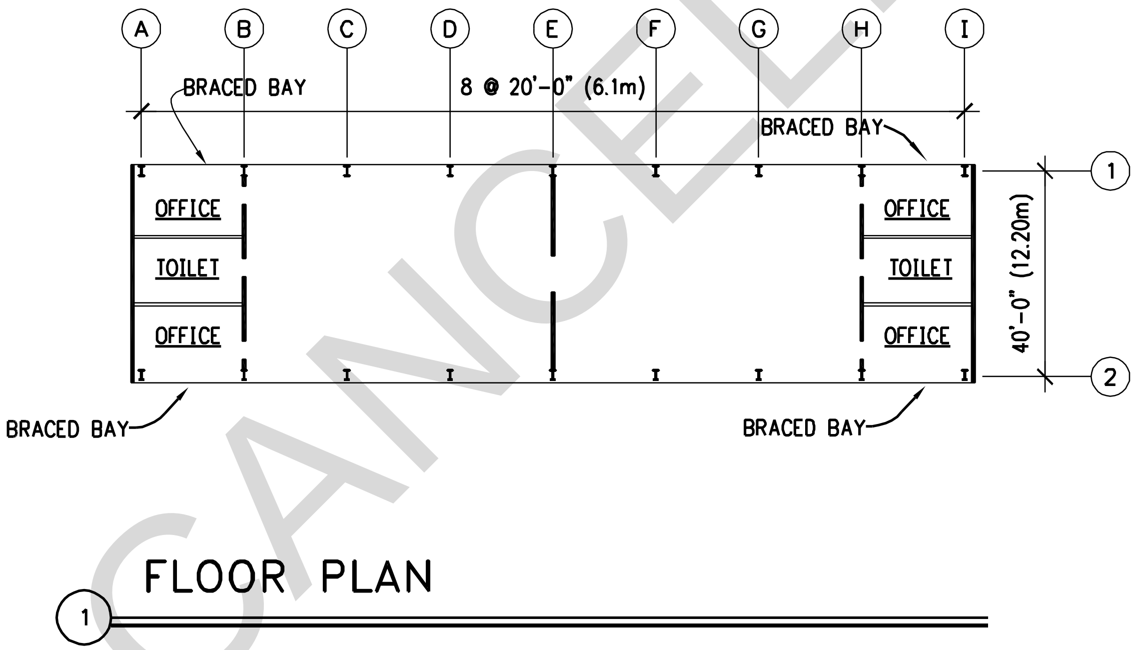
The mezzanines at the end bays of the building must support the large storage live loads (assumed 125 psf or 5.99KN/m^2). This calls for the use of some type of concrete slab to support the high loads. A concrete filled metal deck is selected from a manufacture’s catalog consisting of normal weight concrete fill on 1-1/2" (38mm) metal deck. The deck spans in the transverse direction over steel beams at 8’-0” (2.44m) on center. The beams bear on pilasters projecting from the CMU walls (design of pilasters not in scope of problem).

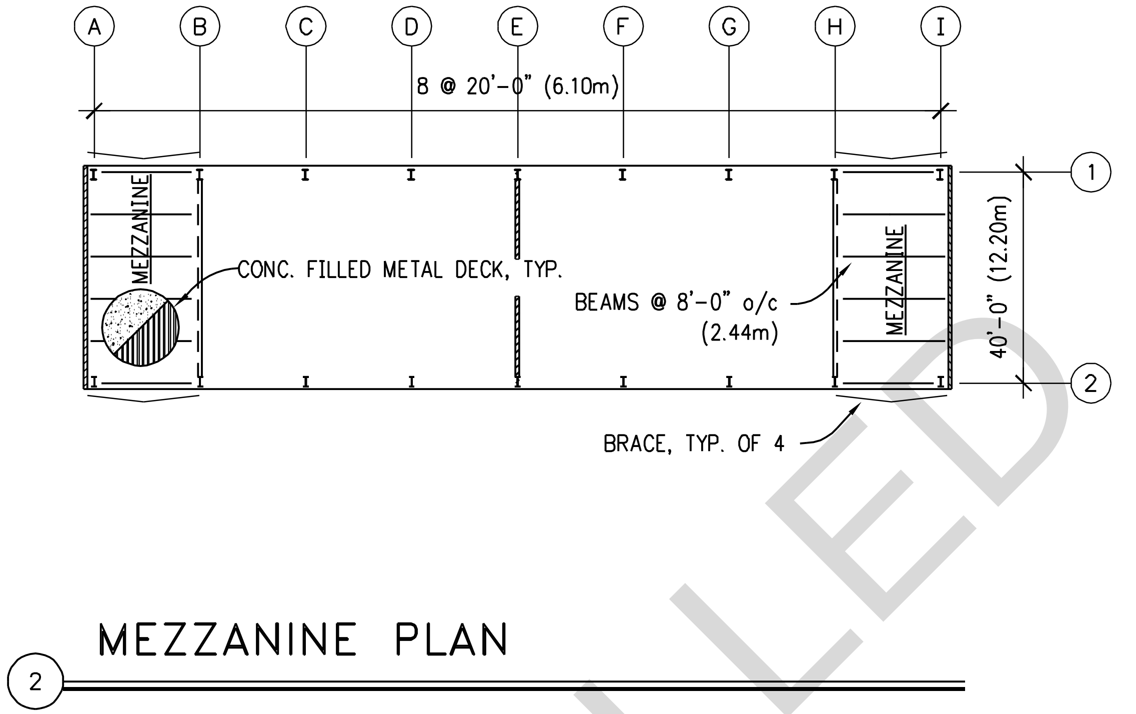
Lateral Systems

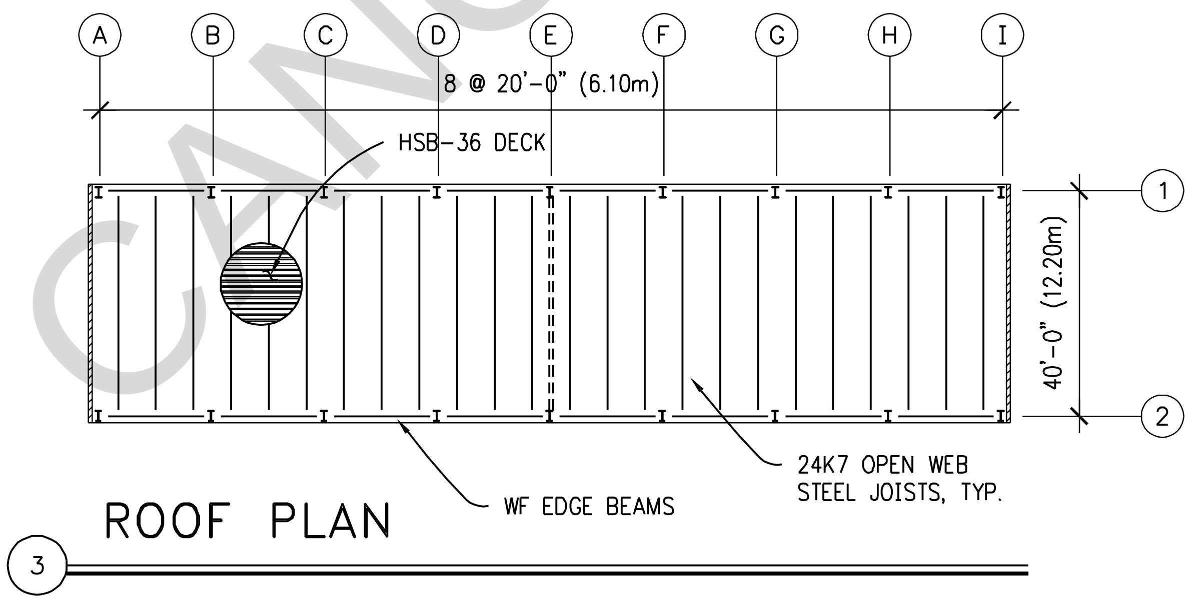
The primary lateral system in the transverse direction consists of reinforced CMU walls. The building has a complete frame system so the walls are considered nonbearing. There is no need for large openings in the transverse walls, which allows for the use of shear walls. The metal decking at the roof level acts as a flexible diaphragm that transfers shear to the exterior CMU shear walls and the interior CMU shear wall based on tributary areas. The metal deck diaphragm spans 80' (24.40m) with a depth of 40' (12.20m). This is within the limits for diaphragm span and depth set forth in Table 7-24. The shear walls are detailed as Special Reinforced Masonry Shear Walls. This calls for more stringent reinforcing details to allow the structure to respond in a ductile manor in the event of inelastic deformations. The building is of lightweight construction, which translates into low seismic demand. The shear walls are very strong and stiff and it is likely that the minimum reinforcing details will control the design due to the inherent strength of the wall.

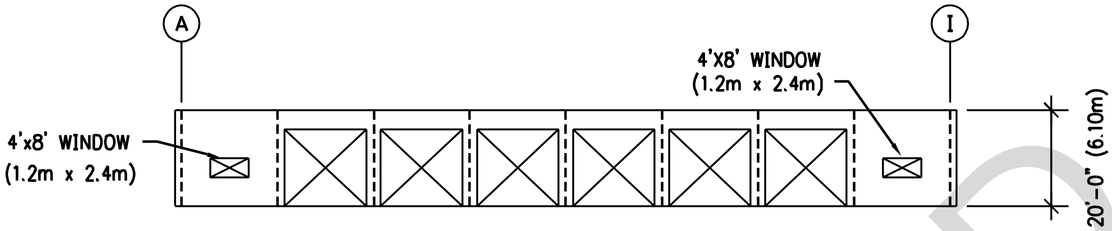
The lateral system in the longitudinal direction consists of four steel braced frames (one at each end-bay of the building.). The braced frames allow for the large door openings while providing high strength and stiffness. The building is detailed as an Ordinary Concentrically Braced Frame with V-Type bracing from the roof level to the mezzanine edge beam and Chevron bracing from the mezzanine edge beams to the base of the columns. The building is not likely to see large inelastic deformation demands due to the lightweight construction. Therefore, the bracing members and connections are detailed as ordinary braced frames rather than special braced frames.

The mezzanines are analyzed as rigid diaphragms due to the high stiffness of the concrete filled metal deck. The shear forces from the mezzanines are distributed to the vertical resisting elements based on their relative rigidities. In the transverse direction, the mezzanines are braced by the exterior shear walls along wall lines A & I and by interior shear walls along lines B & H. Longitudinally, the mezzanines transfer shear forces through metal studs into the supporting beams of the braced frames. The frames relative rigidities are all the same due to symmetry, and thus, each resists the same amount of shear. The effects of torsion must be checked due to the mezzanine diaphragm rigidity.

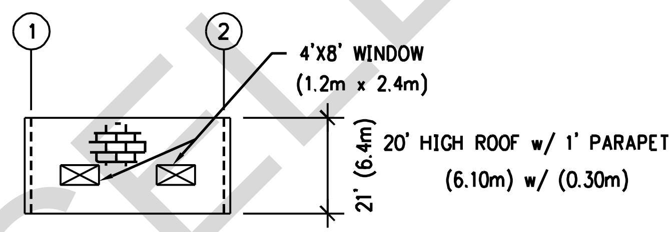



A
B
C
D
E
F
G
H
I
8 @ 20'-0" (6.10m)
MEZZANINE
CONC. FILLED METAL DECK, TYP.
BEAMS @ 8'-0" o/c
(2.44m)
MEZZANINE
40'-0" (12.20m)
1
2
BRACE, TYP. OF 4
MEZZANINE PLAN
2


A
B
C
D
E
F
G
H
I
8 @ 20'-0" (6.10m)
HSB-36 DECK
40'-0" (12.20m)
1
2
24K7 OPEN WEB
STEEL JOISTS, TYP.
WF EDGE BEAMS
ROOF PLAN
3

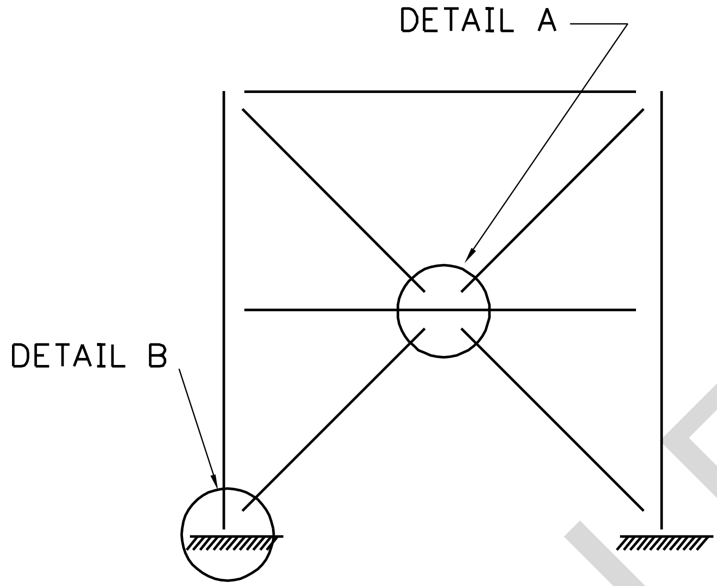


NORTH & SOUTH ELEVATIONS

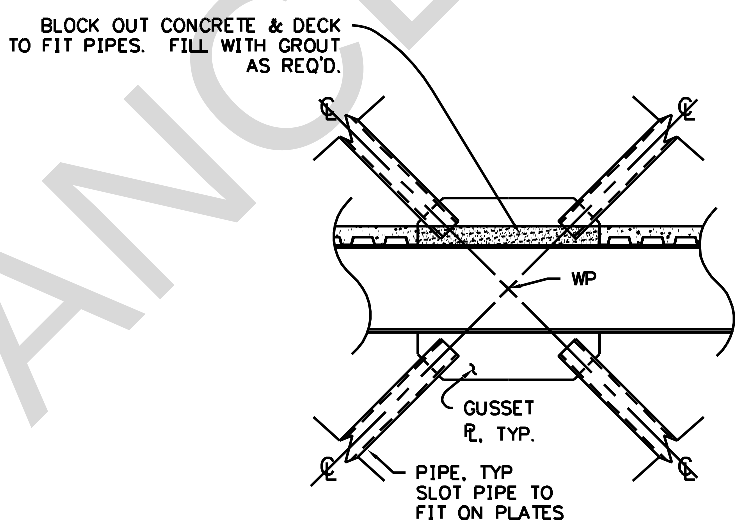


EAST & WEST ELEVATIONS

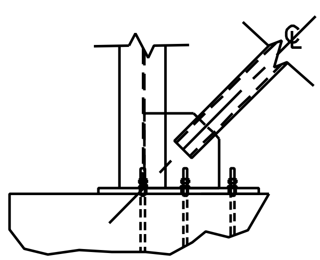
4 BUILDING ELEVATIONS



BRACE ELEVATION



DETAIL A



DETAIL B

c. Design of building

Note: Many of the calculations in this example were carried out in a spreadsheet format. The calculations carry more significant digits than are shown in the steps below. Some of the results may be slightly different in the last digit due to accuracy carried by spreadsheet as compared to the accuracy shown in steps.

The building design follows the steps for Performance Objective 1A set forth in Table 4-5.

A.1 *Determine appropriate Seismic Use Group and analysis procedure.*

The garage structure is a Standard Occupancy Structure. This classifies it as Seismic Use Group I. The structural system performance objectives are determined from Table 4-4.

Seismic Use Group:	I	Table 4-1
Performance Level:	LS(1)	Table 4-4
Ground Motion:	2/3 MCE (A)	Table 4-4
Performance Objective:	1A	Table 4-4
Minimum Analysis Procedure:	Linear Elastic w/ R Factors	Table 4-4

A.2 *Determine site seismicity.*

It is assumed for this problem that we have the values:

$S_S = 0.75$ g — MCE Maps

$S_1 = 0.40$ g

A.3 *Determine site characteristics.*

It is assumed for this problem that we have soil type D

Soil Type: D — Table 3-1

A.4 *Determine site coefficients.*

From Tables 3-2a and 3-2b for the given site seismicity and soil characteristics the site coefficients are:

$F_a = 1.2$ — Table 3.2a

$F_v = 1.6$ — Table 3.2b

A.5 *Determine adjusted MCE spectral response accelerations.*

$$S_{MS} = F_a S_S = (1.2)(0.75g) = 0.90g \qquad \text{Eq. 3-1}$$

$$S_{M1} = F_v S_1 = (1.6)(0.40g) = 0.64g \qquad \text{Eq. 3-2}$$

A.6 *Determine design spectral response accelerations.*

For Performance Objective IA (Protect Life Safety) FEMA 302 requires that the design spectral accelerations be calculated as 2/3 of the adjusted MCE spectral response accelerations.

$$S_{DS} = 2/3 S_{MS} = (2/3)(0.90g) = 0.60g \qquad \text{Eq. 3-3}$$

$$S_{D1} = 2/3 S_{M1} = (2/3)(0.64) = 0.43g \qquad \text{Eq. 3-4}$$

For regular structures, 5-stories or less in height, and having a period, T, of 0.5 seconds or less, the design spectral accelerations need not exceed:

$$S_{DS} \le 1.5F_a = (1.5)(1.2) = 1.80g > 0.60g \qquad \text{Eq. 3-5}$$

$$S_{D1} \le 0.6F_v = (0.6)(1.6) = 0.96g > 0.43g \qquad \text{Eq. 3-6}$$

A.7 *Determine* Seismic Design Category

From Tables 4-2a and 4-2b for the Seismic Use Group and design spectral response accelerations:

Seismic Design Category = D	Table 4-2a
Seismic Design Category = D	Table 4-2b

A.8 *Select structural system*

(See discussion of structural systems in the building description section).

A.9 *Select R,* W_o *&* C_d *factors.*

<u>Transverse (North-South):</u> Building Frame System with Special reinforced masonry shear walls. The building is assumed to act as a frame system rather than a bearing wall system. The beams that support the mezzanine deck bear on pilasters that project from the exterior and interior CMU shear walls. The pilasters are considered to act as a part of the gravity system and are neglected for lateral force resistance.

R =	5	
Ω_o =	2.5	Table 7-1
C_d =	4	

<u>Longitudinal (East-West):</u> Building frame system with ordinary steel concentrically braced frames.

R =	5	
Ω_o =	2.0	Table 7-1
C_d =	4.5	

A.10 *Determine preliminary member sizes for gravity load effects.*

ASCE 7 is used for the load combinations to be checked. Wind and snow load effects are neglected in this example. The governing load combinations for the gravity load system are:

$1.2\ D + 1.6L + 0.5L_r$ — ASCE 7-95 Sec. 2.3.2.2
$1.2\ D + 1.6L_r + 0.5L$ — ASCE 7-95 Sec. 2.3.2.3

ROOF (psf)

Item	Deck	Joist	Beam	Column	Seismic
Built-up Roofing	5.0	5.0	5.0	5.0	5.0
2" (51mm) Rigid Insulation	3.0	3.0	3.0	3.0	3.0
20 Gage (1mm) Metal Decking	2.0	2.0	2.0	2.0	2.0
24K7 Open Web Steel Joists @ 6'-8" (2.03m) O.C.	-	2.0	2.0	2.0	2.0
Perimeter Beams @ 20' (6.10m)	-	-	2.0	2.0	2.0
(10' (3.05m) of col. @ 20'x20' (6.10m x 6.10m))	-	-	-	1.0	1.0
Misc. (Mech., Elec., etc.)	3.0	3.0	3.0	3.0	3.0
Totals	13.0	15.0	17.0	18.0	18.0

MEZZANINE (psf)

Item	Deck	Joist	Beam	Column	Seismic
Finish	1.0	1.0	1.0	1.0	1.0
20 gage metal decking w/ NWT conc. fill (3-1/2")	30.5	30.5	30.5	30.5	30.5
Ceiling	1.0	1.0	1.0	1.0	1.0
Beams @ 6.7' O.C.	-	-	4.5	4.5	4.5
Partitions*	-	-	-	-	10.0
Misc. (M&E)	2.0	2.0	2.0	2.0	2.0
Totals	34.5	34.5	39.0	39.0	49.0

Masonry Walls (psf, vertical)

8" NWT CMU, Grouted @ 40" O.C.	57

Metal Walls (psf, vertical)

Metal Siding	1
Girts	1
Insulation	2
Totals	4

1 lb. = 4.448N

1 psf = 47.88 N/m^2

Roll-Up Doors + Mechanical (lb.)

Weight of each door	1200

Live Loads (psf)

Roof	20
Mezzanine	125

Snow Load (psf)

Temperate Climate - no snow	0

*Note: ASCE 7-95 Section 4.2.2 requires that provisions for partition weights should be made if the live load is less than 80 psf (3.83 N / m^2) for gravity design. The live load for the mezzanines is 125 psf (5.99 KN / m^2) for this example, and therefore, no provision for partition weight is made for the gravity load design.

*Note: No live load reductions are taken in this example to be conservative

Metal Roof Decking

Note: Since the Government cannot procure proprietary materials and systems, the following reference is provided. To select the metal decking, refer to and use the Steel Joist Institute (SJI) LOAD TABLES in the current SDI Design Manual. Select the deck rib type (narrow, intermediate, or wide) and the gage thickness that will be given on the contract documents.

For this example the metal roof decking is chosen based on the given live and dead loads from any metal deck manufacturer's catalog. The manufacturer's catalog used for this example lists allowable loads based on the expected service loads.

Dead Loads to Deck:	13 psf
Live Loads to Deck:	20 psf
Total:	33 psf (1.58 KN / m^2)

Entering the catalog with the 33 psf load and assuming that the deck will span 6'-8" between each open-web joist a 20 gage deck is selected.

Roof Joists

Note: Since the Government cannot procure proprietary materials and systems, the following reference is provided. To select the open web joists, refer to and use the Steel Joist Institute (SJI) K-Series STANDARD LOAD TABLE in the current edition of the SJI publication, STANDARD SPECIFICATIONS, LOAD TABLES AND WEIGHT TABLES. (Note: The assumptions for joist selection from the K-Series STANDARD LOAD TABLE are parallel chord simple span joists that are uniformly loaded and are placed on a slope not greater than ½ inch / foot. If any of these conditions do not exist, the joist loading diagram, the span, the slope, and the desired joist depth must be given on the contract documents. With this information, the required K-Series joist chord size will be determined and certified by the manufacturer.)

For this example the open-web steel joists are chosen from a manufacture's catalog. The allowable loads for the joists are listed for unfactored service loads

Dead Loads to Joists:	15 psf
Live Loads to Deck:	20 psf
Total:	35 psf (1.68 KN / m^2)

Running Load = Load (psf) x tributary width (6'-8" or 2.03m)
Running Load = (35 psf)(6.67') = 233 plf (3.40 KN / m)

Entering the catalog with a 40' span and running load of 233 plf a 24K7 joist is selected. The design and details of the bridging and weld connections to the supporting beams shall be as per manufacture's specs in the design catalogs.

Edge Beams of Gravity System

The beams of the gravity system are designed in accordance with the AISC LRFD specifications.

Tributary width = 20' (6.10m)

Dead Loads:

Dead Load:	17psf
Roll-up Doors:	1200 lb. / door (5.34KN)

Total = (17psf)(20') + (1200 lb. / 20') = 400 plf (5.84 KN / m)

Live Loads:

Live Loads:	(20psf)(20') = 400 plf (5.84 KN / m)

w_u = 1.2 DL + 1.6 LL = 1.2(400) + 1.6(400) = 1120 plf   (16.34 KN / m)
$M_u = wL^2/8$ = (1120plf)(20ft.)^2/8 = 56 kipft. (75.9 KN-m)

Try W12 x 26, BF=2.99 L_p = 6.3' $\phi_b M_p$ = 100 kipft. A36 steel. Assume the beam is laterally supported at 6'-8" by the joists. Therefore, $L_b > L_p$, use the following check for the beam design;

$$\phi_b M_n = C_b\left[\phi_b M_p - BF(L_b - Lp)\right]$$ From AISC LRFD 93 Part 4 (Beam Design)

$\phi_b M_n = 1.0\left[100 - 2.99(6.67 - 6.3)\right] = 98.9$ kipft (134KN-m)> 56 kipft. (75.9 KN-m) OK.

The beam is slightly overdesigned in anticipation of the combined action due to collector and chord action.

<u>Steel Columns</u>

Tributary Area = 20' x 20' = 400 ft.^{2} (37.21 m^2)

Dead Loads:

Dead Load:	18psf
Roll-Up Doors:	1.2 kips (Each column must support ½ the door wt. on either side)

Live Loads: 20 psf

P_u = 1.2[(18psf)(400 ft.^{2}) + 1.2 k] + 1.6(20psf)(400 ft.^{2}) = 23 kips (102.3 KN)

This is a very low axial load. The columns at the end bays where the mezzanines occur must support the axial forces generated by the braced frames in addition the extra weight from the mezzanine beams. The columns are slightly overdesigned in anticipation of these forces.

Try W10x33, r_y = 1.94, A = 9.71 in.^{2}

KL/r = (1.0)(20')(12"/')/1.94" = 124   look up design strength from AISC design manual:
$\phi_c F_{cr}$ = 13.62 ksi — AISC LRFD '93 Table 3-36
$P_n = A_g F_{cr}$ — AISC LRFD '93 Eq. E2-1
ϕP_n = (13.62 ksi)(9.71in.^{2}) = 132 kips (589.1 KN) > 23 kips (102.3 KN),   OK

<u>Mezzanine Roof / Deck Slab</u>

The mezzanine deck will be selected from a manufacturer's catalog. The mezzanine must support high live loads (125 psf or 5.99 KN/m^2) which suggest the use of metal decking with a topping slab. The catalog lists allowable superimposed service loads.

Dead Loads to Deck:	4 psf	(Only superimposed loads are considered)
Live Loads to Deck:	<u>125 psf</u>	
Total:	129 psf	(6.18 KN/m^2)

Enter the catalog with 8' (2.44m) span between supporting beams and select 20 gage (1mm) decking with 3-1/2" (89mm) total depth including concrete topping.

<u>Mezzanine Beams</u>

The beams span 20' (6.10m) and are spaced at 8' (2.44m) on centers. The interior beams are simply supported on base plates anchor bolted to CMU pilasters at the exterior and interior walls, while the exterior beams frame into the steel columns.   It is assumed that the beams are supported laterally along the full length by the decking and concrete fill.

Tributary Width = 8' (2.44m)

Dead Loads = 39psf

Live Loads = 125psf

w_u = 1.2 DL + 1.6 LL = 1.2(39psf)(8') + 1.6(125psf)(8') = 1974 plf (28.80 KN/m)

$M_u = wL^2/8$ = (1974plf)(20ft.)^2/8 = 99 kipft. (134.24 KN-m)

Try W12 x 26, A36 steel. This is the same member as used for the upper roof edge beams.

$\phi_b M_p$ = 100 kipft. (135.60 KN-m) > 99 kipft. (134.24 KN-m) OK.

B.1 *Calculate fundamental period, T*

$T_a = C_T h_n^{3/4}$ FEMA 302 Eq. 5.3.3.1-1

C_T = 0.020 for both the transverse and longitudinal directions

h_n = 20 ft. (6.10m)

$T_a = (0.020)(20)^{3/4}$ = 0.19 sec.

B.2 *Determine dead load, W*

ROOF LEVEL TRIBUTARY SEISMIC WEIGHTS (ROOF & TRIBUTARY WALLS)

Item	Number	Tributary Height / Width (ft.)	Length / Width (ft.)	Area (ft.2)	Unit Weight (psf / lb.)	Seismic Weight (kips)
Roof	1	41	161	6601	18.0	118.8
CMU Wall A1-A2	1	6	41	246	57.0	14.0
CMU Wall I1-I2	1	6	41	246	57.0	14.0
CMU Firewall E1-E2	1	10	40	400	57.0	22.8
Metal Panel Wall 1A-1B	1	5	20	100	4.0	0.4
Metal Panel Wall 1H-1I	1	5	20	100	4.0	0.4
Metal Panel Wall 2A-2B	1	5	20	100	4.0	0.4
Metal Panel Wall 2H-2I	1	5	20	100	4.0	0.4
Metal Panel Walls Between Doors	10	10	3	300	4.0	1.2
Metal Roll-Up Doors	12	---	---	---	1200	14.4
					TOTAL	***186.9***
						(831.3 KN)

MEZZAINIE LEVEL TRIBUTARY SEISMIC WEIGHTS (ROOF & TRIBUTARY WALLS)

Item	Number	Tributary Height / Width (ft.)	Length / Width (ft.)	Area (ft.2)	Unit Weight (psf / plf)	Seismic Weight (kips)
Decks	2	20	40	1600	49.0	78.4
25% of Live storage loads*	2	20	40	1600	31.3	50.0
CMU Wall A1-A2	1	10	41	410	57.0	23.4
CMU Wall I1-I2	1	10	41	410	57.0	23.4
CMU Wall B1-B2	1	5	40	200	57.0	11.4
CMU Wall H1-H2	1	5	40	200	57.0	11.4
Metal Panel Wall 1H-1I	1	10	20	200	4.0	0.8
Metal Panel Wall 1A-1B	1	10	20	200	4.0	0.8
Metal Panel Wall 2A-2B	1	10	20	200	4.0	0.8
Metal Panel Wall 2H-2I	1	10	20	200	4.0	0.8
					TOTAL	***201.1***

TOTAL SEISMIC WEIGHT = 186.9 + 201.1 = 388 kips (894.5 KN)

*Note: ASCE 7-95 Section 9.2.3.2-1.1 requires that 25% of the floor live load be included in the determination of the seismic weight in areas used for storage.

B.3 *Calculate base shear, V*

$V = C_s W$	FEMA 302 Eq. 5.3.2
$C_s = S_{DS}/R$,	Eq. 3-7
but need not exceed $C_s = S_{D1}/TR$,	Eq. 3-8
but shall not be less than $C_s = 0.044S_{DS}$	Eq. 3-9

Transverse Direction:

$C_s = (0.6)/5 = 0.12$
$C_s = (0.43)/(0.19)(5) = 0.45 > 0.12$
$C_s = 0.044(0.43) = 0.02 < 0.12$

$V = C_s W = (0.12)(388 \text{ kips}) = 47 \text{ kips } (209 \text{ KN})$

Longitudinal Direction:

$C_s = (0.6)/5 = 0.12$
$C_s = (0.43)/(0.19)(5) = 0.45 > 0.12$
$C_s = 0.044(0.43) = 0.02 < 0.12$

$V = C_s W = (0.12)(388 \text{ kips}) = 47 \text{ kips } (209 \text{ KN})$

B.4 *Calculate the vertical distribution of seismic forces*

This building is a combination of one & two story area. The main building is one story while the mezzanines act as two-story areas. It is assumed that the mezzanine diaphragms will not act to drive the overall building response. Therefore, the building is analyzed as a one-story structure.

The upper roof metal decking acts as a flexible diaphragm distributing the shears to the vertical resisting elements according to tributary areas. In the transverse direction, the end CMU walls resist ¼ of the lateral force while ½ of the force is resisted by the firewall. In the longitudinal direction, it is assumed that each of the braced bays will resist ¼ of the shear force. The shear forces developed at the mezzanine level will be distributed to the vertical resisting elements in relation to their rigidities due to the rigid diaphragm action of the concrete topping. In addition, torsional forces developed must be considered.

Shear Forces to Roof & Mezzanine Diaphragms:

The seismic coefficient, C_s, is the same in both directions for this structure.

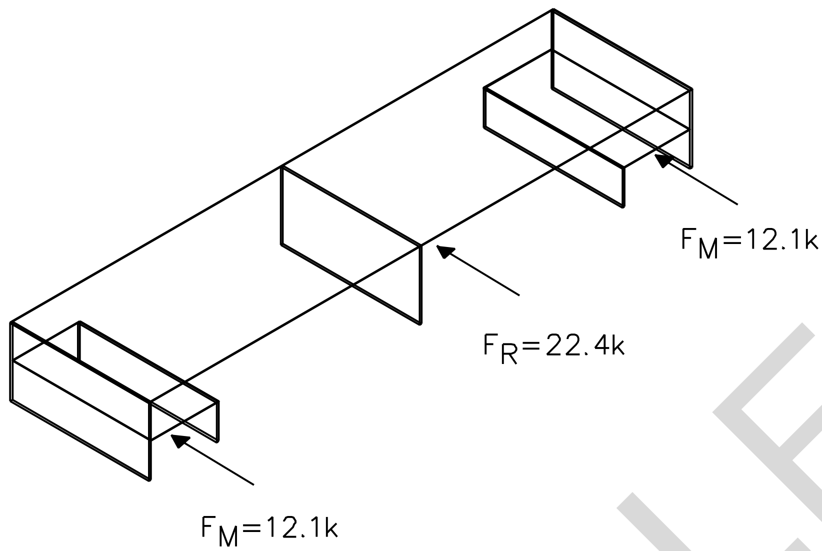
Roof level: $F_r = C_s$ x weight tributary to roof level
$F_r = (0.12)(187 \text{ kips}) = 22.4 \text{ kips } (99.6 \text{ KN})$

Mezzanine level: $F_m = C_s$ x weight tributary to the mezzanines
$F_m = (0.12)(201 \text{ kips}) = 24.1 \text{ kips } (107.2 \text{ KN})$

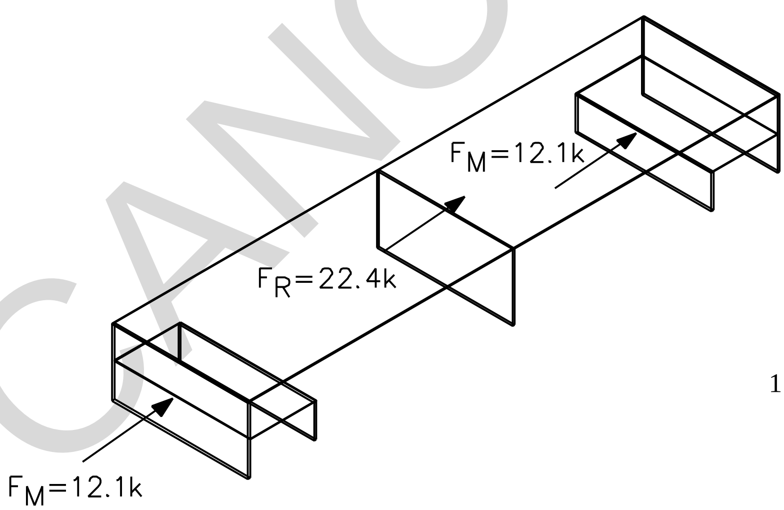
Due to symmetry, the mezzanine level forces are assumed to act evenly between each mezzanine.

$F_m = ½(24.1 \text{ kips}) = 12.1 \text{ kips / mezzanine } (53.8 \text{ KN})$

Transverse Seismic Forces



Longitudinal Seismic Forces



1 kip = 4.448 KN

B.5 *Perform Static Analysis*

<u>Seismic Analysis</u>

The seismic design will follow the load path from the diaphragms to the vertical resisting elements. The upper roof metal decking is assumed to act as a flexible diaphragm and will distribute shear to the vertical resisting elements based on tributary area. The concrete filled deck of the mezzanine acts as a rigid diaphragm, distributing the shear to the vertical resisting elements based on their relative rigidities.

Diaphragm Shear forces

The first step in the load path is the shear force to the diaphragms. The diaphragm shear forces are due to their own weight as well as the tributary normal walls.

ROOF DIAPHRAGM WEIGHTS & NORMAL WALLS

Item	Number	Tributary Height / Width (ft.)	Length / Width (ft.)	Area (ft.2)	Unit Weight (psf / lb.)	Trans. Seismic Weight (kips)	Long. Seismic Weight (kips)
Roof	1	41	161	6601	18.0	118.8	118.8
CMU Wall A1-A2	1	6	41	246	57.0	0.0	14
CMU Wall I1-I2	1	6	41	246	57.0	0.0	14
CMU Firewall E1-E2	1	10	40	400	57.0	0.0	22.8
Metal Panel Wall 1A-1B	1	5	20	100	4.0	0.4	0.0
Metal Panel Wall 1H-1I	1	5	20	100	4.0	0.4	0.0
Metal Panel Wall 2A-2B	1	5	20	100	4.0	0.4	0.0
Metal Panel Wall 2H-2I	1	5	20	100	4.0	0.4	0.0
Metal Panel Walls Between Doors	10	10	3	300	4.0	1.2	0.0
Metal Roll-Up Doors	12	---	---	---	1200	14.4	0.0
			TOTAL			***136.0 (605 KN)***	***169.7 (755 KN)***

MEZZAINIE LEVEL WEIGHTS & NORMAL WALLS

Item	Number	Tributary Height / Width (ft.)	Length / Width (ft.)	Area (ft.2)	Unit Weight (psf / plf)	Trans. Seismic Weight (kips)	Long. Seismic Weight (kips)
Decks	2	20	40	1600	49.0	78.4	78.4
25% of Live storage loads	2	20	40	1600	31.3	50.0	50.0
CMU Wall A1-A2	1	10	41	410	57.0	0.0	23.4
CMU Wall I1-I2	1	10	41	410	57.0	0.0	23.4
CMU Wall B1-B2	1	5	40	200	57.0	0.0	11.4
CMU Wall H1-H2	1	5	40	200	57.0	0.0	11.4
Metal Panel Wall 1H-1I	1	10	20	200	4.0	0.8	0.0
Metal Panel Wall 1A-1B	1	10	20	200	4.0	0.8	0.0
Metal Panel Wall 2A-2B	1	10	20	200	4.0	0.8	0.0
Metal Panel Wall 2H-2I	1	10	20	200	4.0	0.8	0.0
TOTAL WEIGHT TRIBUTARY TO MEZZANINE DIAPHRAGMS						***131.6***	***197.9***
WEIGHT TRIBUTARY TO EACH MEZZANINE DIAPHRAGM = 1/2 WEIGHT =						***65.8 (293 KN)***	***99 (440 KN)***

Determine Shear to Diaphragms: Diaphragm shear, $V = C_s$ x Tributary weight

Transverse: $C_s = 0.12$

Roof: $V_r = (0.12)(136\text{kips}) = 16.32$ kips (72.6 KN)

w = unit loading to diaphragm $= V_r$ / diaph. span = 16.32 kips / 160’ = 102 plf (1.49 KN/m)

Mezz: $V_m = (0.12)(65.8\text{kips}) = 7.89$ kips / mezzanine (35.1 KN)

w = 7.89 kips / 20’ = 395 plf (5.76 KN/m)

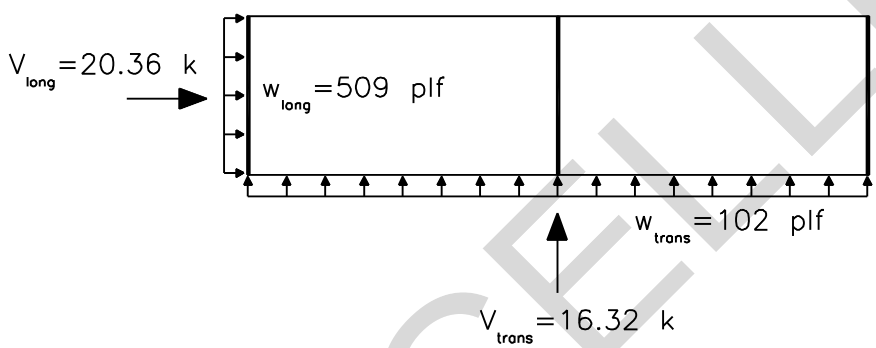
Longitudinal: $C_s = 0.12$

Roof: $V_r = (0.12)(169.7\text{kips}) = 20.36$ kips (90.56 KN)

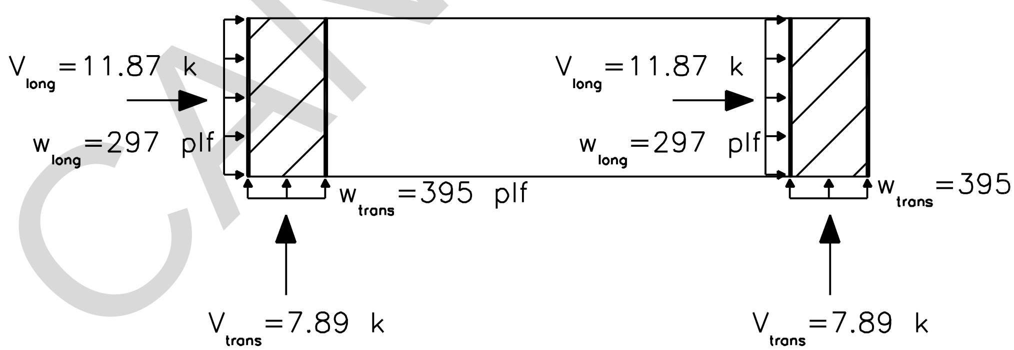
w = 20.36 kips / 40’ = 509 plf (7.43 KN/m)

Mezz: $V_m = (0.12)(99\text{ kips}) = 11.87$ kips / mezzanine (52.8 KN)

w = 11.87 kips / 40’ = 297 plf (4.33 KN/m)



UPPER ROOF DIAPHRAGM FORCES



MEZZANINE DIAPHRAGM FORCES

1 kip = 4.448 KN
1 plf = 14.59 N/m

Distribute upper roof diaphragm shear forces to vertical resisting elements

The shear force from the upper roof diaphragm is distributed to the vertical elements based on tributary areas. In the transverse direction, each of the end exterior CMU shear walls resists ¼ of the roof shear while the interior CMU wall resist ½ of the roof shear. The upper roof shear force in the longitudinal direction is assumed to be resisted by each of the braced frame bays evenly. Therefore, each bay will resist ¼ of the roof shear.

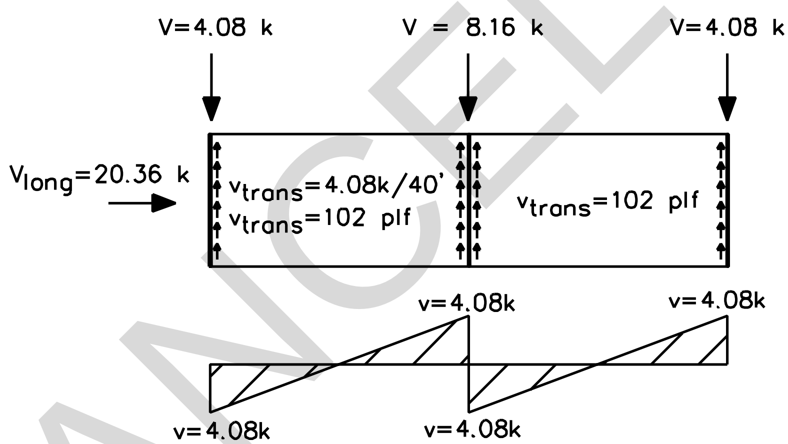
Transverse:

Shear to each exterior CMU wall = (1/4)(16.32kips) = 4.08 kips (18.15 KN)
Shear to interior CMU firewall = (1/2)(16.32kips) = 8.16 kips (36.30 KN)
The unit shear force in the diaphragm, v, will be the maximum at the shear walls.
v = shear at walls / diaphragm depth
v = 4.08 kips / 40’ = 102 plf (1.49 KN/m)

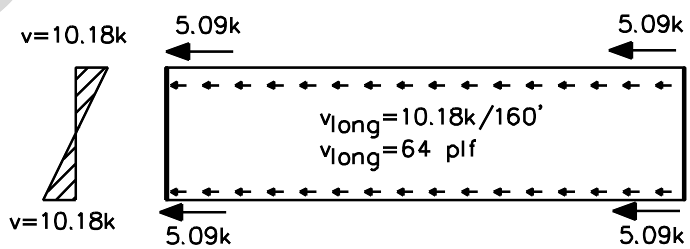
Longitudinal:

Shear to each braced frame bay = (1/4)(20.36kips) = 5.09 kips / braced bay (22.64 KN)
The diaphragm is assumed to act as a simply supported beam spanning between the braced frame wall lines. The maximum shear in the diaphragm is located at the edges. The unit diaphragm shear , v = 2(5.09kips)/160’ = 64 plf (933 N/m)

UPPER LEVEL DIAPHRAGM SHEAR FORCES



TRANSVERSE DIAPHRAGM SHEAR FORCES



1 kip = 4.448 KN
1 plf = 14.59 N/m

LONGITUDINAL DIAPHRAGM SHEAR FORCES

<u>Determine shear in vertical elements due to self-weight inertial effects</u>

The vertical resisting elements must resist the shear force transferred to them by the diaphragm in addition to the shear forces generated by parallel walls and self-weight.

Transverse:

Typical exterior wall (A1-A2 & I1-I2)
Shear = C_s x wall weight trib to upper roof = (0.12)(14 kips) = 1.68 kips
This shear is now added to the diaphragm shear transferred to the wall;
Total shear in exterior wall above mezzanine = (1.68k + 4.08k) = 5.8 kips (25.8 KN)

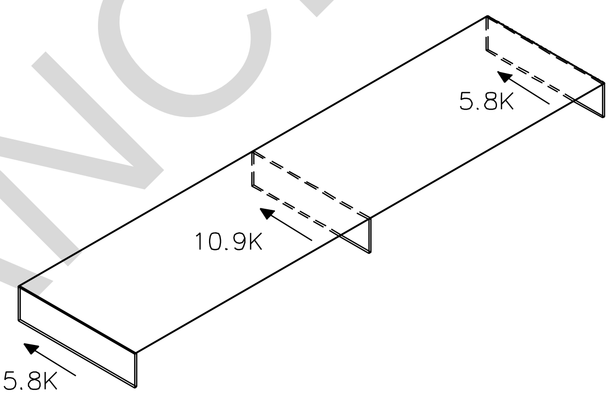
Interior CMU wall E1-E2
Shear = (0.12)(22.8 kips) = 2.74 kips
Total Shear = (2.74 k + 8.16 k) = 10.9 kips (48.48 KN)

Longitudinal:

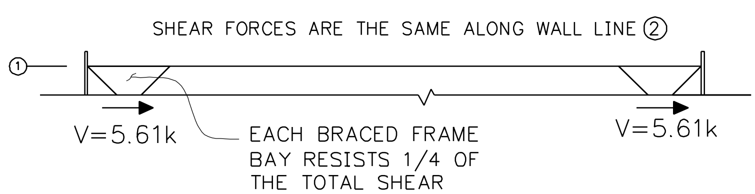
The inertial forces of the metal panel walls and roll-up doors are collected by the edge beam of the steel frame and delivered to the braced frames.

Tributary weight of metal panel walls at braced frame bays = 1.6k
Tributary weight of panel walls between doors = 1.2k
Weight of all 12 metal roll-up doors = 14.4k
Total = 1.6k + 1.2k + 14.4k = 17.2 kips
Seismic Force = (0.12)(17.2) = 2.06 kips
¼ of the longitudinal shear force goes to each braced frame bay = ¼(2.06) = 0.515 kips
Total shear force = (5.09k + 0.515k) = 5.61 kips (24.95 KN)

SHEAR FORCES IN VERTICAL RESISTING ELEMENTS ABOVE
MEZZANINES (FORCES TRIBUTARY TO ROOF)



TRANSVERSE



LONGITUDINAL

1 kip = 4.448 KN

Determine distribution or mezzanine shear force to vertical resisting elements

Mezzanine shear forces in the transverse direction are distributed to the interior CMU shear walls (B1-B2 & H1-H2) and the exterior walls (A1-A2 & I1-I2) based on their relative rigidities. For longitudinal forces, it is assumed that each braced frame bay receives ½ of the force due to symmetry.

Wall Rigidity Calculations

Mechanical Properties of Masonry

$f'_m := 1500\,psi$ (10.34 MPa) — Assume masonry strength of 1500 psi with Type S mortar.

$E_m := 1.6\cdot 10^6\cdot psi$ (11032 MPa) — Elastic Modulus of CMU (ACI 530-95 Table 5.5.2.3)

$E_v := 0.4\cdot E_m$ $\quad E_v = 6.4\cdot 10^5\,psi$ (4413 Mpa) — Shearing Modulus of CMU (ACI 530-95 Sec. 5.5.2.3 b)

$est := 4.7\cdot in$ (117 mm) — Equivalent solid thickness 8" CMU grouted at every 40" o/c

$P := 1\cdot kip$ (4.45 N) — Shear load to determine pier stiffness

Masonry Stiffness Functions:

$A(d) := est\cdot d$ — Area of wall segment

$I(d) := \frac{1}{12}\cdot est\cdot d^3$ — Use uncracked section (Assuming that wall piers will not crack, per FEMA 273 Sec. 7.4.4.1)

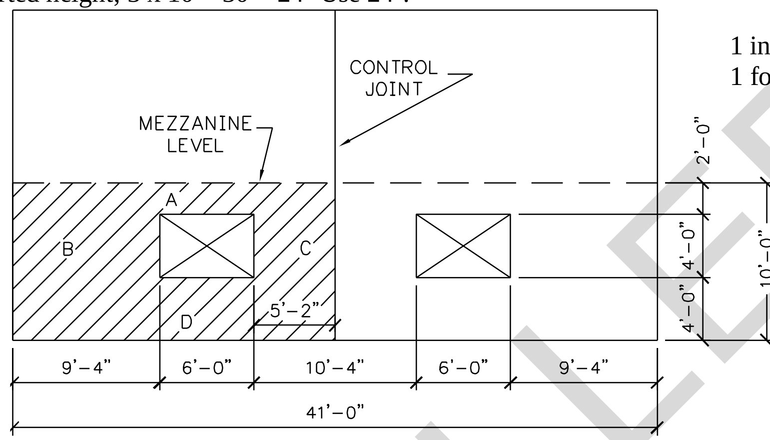
$\Delta_c(h,d) := \frac{(1.2\cdot P\cdot h)}{A(d)\cdot E_v} + \frac{P\cdot h^3}{3\cdot E_m\cdot I(d)}$ — Deflection of a cantilevered pier

$\Delta_f(h,d) := \frac{(1.2\cdot P\cdot h)}{A(d)\cdot E_v} + \frac{P\cdot h^3}{12\cdot E_m\cdot I(d)}$ — Deflection of a fixed-fixed pier

$R_f(h,d) := \frac{1}{\Delta_f(h,d)}$ — Rigidity of fixed pier

Typical Exterior Wall

The rigidity of the exterior wall is based on the height of the wall below the rigid mezzanine diaphragm. The walls are assumed to act as separate units between control joints. The wall is symmetric about the control joint, therefore, the rigidity of the cross-hatched portion of the wall is used for both wall areas. Recommended control joint spacing is taken from Table 4-1 of TM 5-809-3. All of the CMU shear walls have horizontal joint reinforcement of 2-#9 wires at every other course (16"). The recommended maximum ratio of panel length to wall height is 3 with a maximum spacing of 24'. The exterior walls have a 10' unsupported height; 3 x 10 = 30 > 24' Use 24'.



1 inch = 25.4 mm
1 foot = 0.305m

Deflection of solid wall ABCD:

$\Delta_{abcd} := \Delta_c(10 \cdot ft, 20.5 \cdot ft)$ $\quad\quad$ $\Delta_{abcd} = 0.00026 \cdot in$

Subtract strip BC:

$\Delta_{bc} := \Delta_f(4 \cdot ft, 20.5 \cdot ft)$ $\quad\quad$ $\Delta_{bc} = 7.88291 \cdot 10^{-5} \cdot in$

$\Delta := \Delta_{abcd} - \Delta_{bc}$ $\quad\quad$ $\Delta = 0.00018 \cdot in$

Add back in piers B & C

$R_b := R_f(4 \cdot ft, 9.33 \cdot ft)$ $\quad\quad$ $R_b = 5509.26578 \cdot \frac{1}{in}$

$R_c := R_f(4 \cdot ft, 5.17 \cdot ft)$ $\quad\quad$ $R_c = 2700.94107 \cdot \frac{1}{in}$

$\Delta_{bc} := \frac{1}{R_b + R_c}$ $\quad\quad$ $\Delta_{bc} = 0.00012 \cdot in$

$\Delta_{abcd} := \Delta + \Delta_{bc}$ $\quad\quad$ $\Delta_{abcd} = 0.0003 \cdot in$

$$R_{wall} := \frac{1 \cdot kip}{\Delta_{abcd}}$$

$$R_{wall} = 3340.95897 \cdot \frac{kip}{in}$$

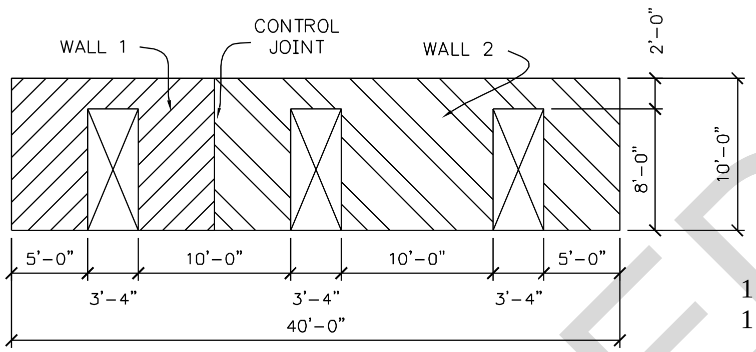
$R_{total} := 2 \cdot R_{wall}$ $\quad\quad$ Add the two wall segments

$R_{total} = 6681.91794 \cdot \frac{1\ kip}{in}$ $\quad\quad$ This value is kips per inch, (6682 kips / in)

$$R_{total} = 1170.15584 \cdot \frac{KN}{mm}$$

Typical Interior Mezzanine Wall

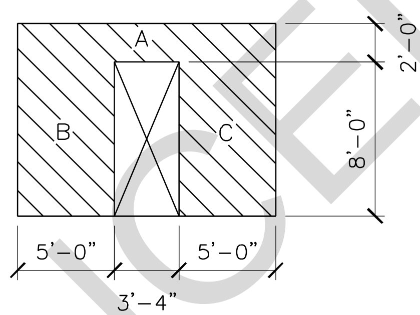
The recommended maximum ratio of panel length to wall height is 3. The interior walls have a 10' unsupported height, therefore, the maximum spacing of control joints is 30'. (This value is greater than 24', assume OK.)



TYPICAL INTERIOR CMU MEZZANINE WALL

1 inch = 25.4 mm
1 foot = 0.305m

Wall 1



Deflection of solid wall ABC:

$\Delta_{abc} := \Delta_c(10\cdot ft, 13.33\cdot ft)$ $\qquad \Delta_{abc} = 0.00052 \circ in$

Subtract strip BC:

$\Delta_{bc} := \Delta_f(8\cdot ft, 13.33\cdot ft)$ $\qquad \Delta_{bc} = 0.00027 \circ in$

$\Delta_a := \Delta_{abc} - \Delta_{bc}$ $\qquad \Delta_a = 0.00026 \circ in$

Add back in piers B & C

$R_b := R_f(8\cdot ft, 5\cdot ft)$ $\qquad R_b = 845.32502 \circ \frac{1}{in}$

$R_c := R_f(8\cdot ft, 5\cdot ft)$ $\qquad R_c = 845.32502 \circ \frac{1}{in}$

$\Delta_{bc} := \frac{1}{R_b + R_c}$ $\qquad \Delta_{bc} = 0.00059 \circ in$

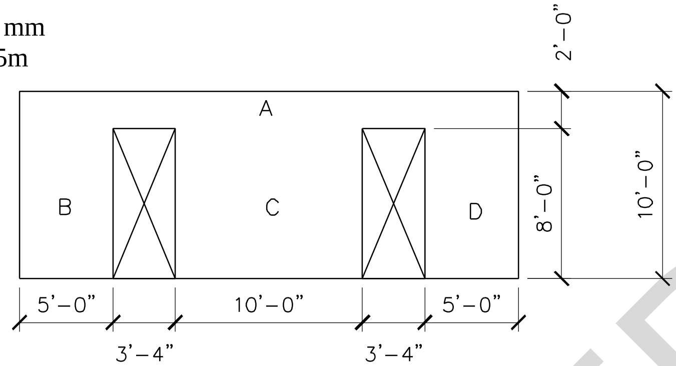
$\Delta_{abc} := \Delta_a + \Delta_{bc}$ $\qquad \Delta_{abc} = 0.00085 \circ in$

$R_{wall.1} := \frac{1}{\Delta_{abc}}$

$R_{wall.1} = 1180.40272 \circ \frac{1}{in}$ This values is kips per inch, (1180 kips / in) $\left(207 \frac{KN}{mm}\right)$

Wall 2

1 inch = 25.4 mm
1 foot = 0.305m



Deflection of solid wall ABCD:

$\Delta_{abcd} := \Delta_c(10 \cdot ft, 26.67 \cdot ft)$ $\quad\quad$ $\Delta_{abcd} = 0.000178 \cdot in$

Subtract strip BCD:

$\Delta_{bcd} := \Delta_f(8 \cdot ft, 26.67 \cdot ft)$ $\quad\quad$ $\Delta_{bcd} = 0.00012 \cdot in$

$\Delta_a := \Delta_{abcd} - \Delta_{bcd}$ $\quad\quad$ $\Delta_a = 5.4367 \cdot 10^{-5} \cdot in$

Add back in piers B, C, & D

$R_b := R_f(8 \cdot ft, 5 \cdot ft)$ $\quad\quad$ $R_b = 845.32502 \cdot \frac{1}{in}$

$R_c := R_f(8 \cdot ft, 10 \cdot ft)$ $\quad\quad$ $R_c = 2582.42148 \cdot \frac{1}{in}$

$R_d := R_b$ $\quad\quad$ $R_d = 845.32502 \cdot \frac{1}{in}$

$\Delta_{bcd} := \frac{1}{R_b + R_c + R_d}$ $\quad\quad$ $\Delta_{bcd} = 0.00023 \cdot in$

$\Delta_{abcd} := \Delta_a + \Delta_{bcd}$ $\quad\quad$ $\Delta_{abcd} = 0.00029 \cdot in$

$R_{wall.2} := \frac{1}{\Delta_{abcd}}$

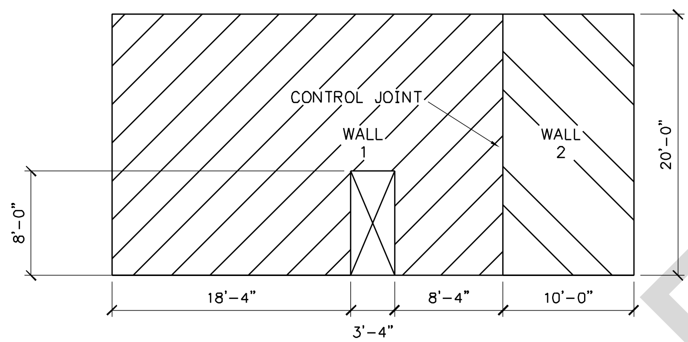
$R_{wall.2} = 3467.51827 \cdot \frac{1}{in}$ This value is kips per inch, (3468 kips / in) $\left(607 \frac{KN}{mm}\right)$

$R_{total} := R_{wall.1} + R_{wall.2}$

$R_{total} = 4647.921 \cdot \frac{1}{in}$ $\left(813 \frac{KN}{mm}\right)$ Total Rigidity of interior CMU wall

Interior Shear Wall E1-E2

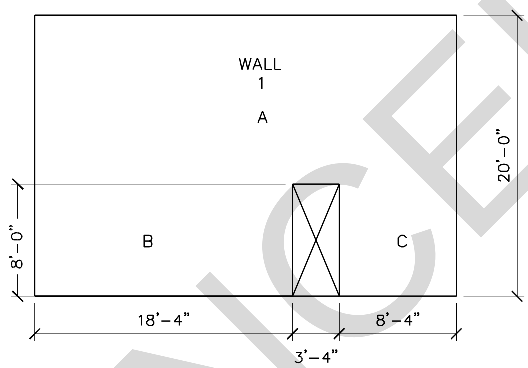
The stiffness of the interior CMU shear wall is not needed for the mezzanine forces. It is calculated here so that the shear force tributary to the wall line may be assigned to the individual wall piers based on their relative rigidities.



1 inch = 25.4 mm
1 foot = 0.305m

INTERIOR SHEAR WALL E1–E2

Wall 1



Deflection of solid wall ABC:

$\Delta_{abc} := \Delta_c(20 \cdot ft, 30 \cdot ft)$ $\qquad \Delta_{abc} = 0.00042 \circ in$

Subtract strip BC:

$\Delta_{bc} := \Delta_f(8 \cdot ft, 30 \cdot ft)$ $\qquad \Delta_{bc} = 0.00011 \circ in$

$\Delta_a := \Delta_{abc} - \Delta_{bc}$ $\qquad \Delta_a = 0.00031 \circ in$

Add back in piers B & C

$R_b := R_f(8 \cdot ft, 18.33 \cdot ft)$ $\qquad R_b = 5400.50795 \circ \frac{1}{in}$

$R_c := R_f(8 \cdot ft, 8.33 \cdot ft)$ $\qquad R_c = 1996.3125 \circ \frac{1}{in}$

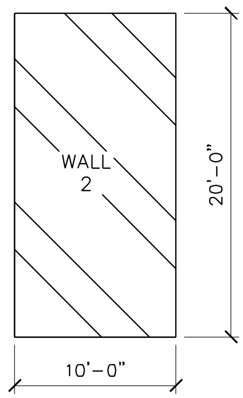
$\Delta_{bc} := \frac{1}{R_b + R_c}$ $\qquad \Delta_{bc} = 0.00014 \circ in$

$\Delta_{abc} := \Delta_a + \Delta_{bc}$ $\qquad \Delta_{abc} = 0.00045 \circ in$

$R_{wall.1} := \frac{1}{\Delta_{abc}}$

$R_{wall.1} = 2222.96341 \circ \frac{1}{in}$ This values is kips per inch, (2223 kips / in) $\left(389 \frac{KN}{mm}\right)$

Wall 2



1 inch = 25.4 mm
1 foot = 0.305m

$\Delta_{wall.2} := \Delta_c(20 \cdot ft, 10 \cdot ft)$ $\qquad \Delta_{wall.2} = 0.00505 \cdot in$

$R_{wall.2} := \dfrac{1}{\Delta_{wall.2}}$

$R_{wall.2} = 197.89504 \dfrac{1}{in}$ This value is kips per inch, (198 kips / in) $\left(35 \dfrac{KN}{mm}\right)$

$R_{total} := R_{wall.1} + R_{wall.2}$

$R_{total} = 2420.85845 \dfrac{1}{in}$ $\left(424 \dfrac{KN}{mm}\right)$ Total Rigidity of interior CMU wall

Transverse forces:

Shear to wall = mezzanine diaphragm shear x relative rigidity

Shear to A1-A2 & I1-I2 = $(7.89\text{kips})\dfrac{6682}{6682+4648} = 4.66\text{kips}$ (20.7 KN)

Shear to B1-B2 & H1-H2 = $(7.89\text{kips})\dfrac{4648}{6682+4648} = 3.24\text{kips}$ (14.4KN)

The unit shear force in the diaphragm, v, will be the maximum at the exterior walls.

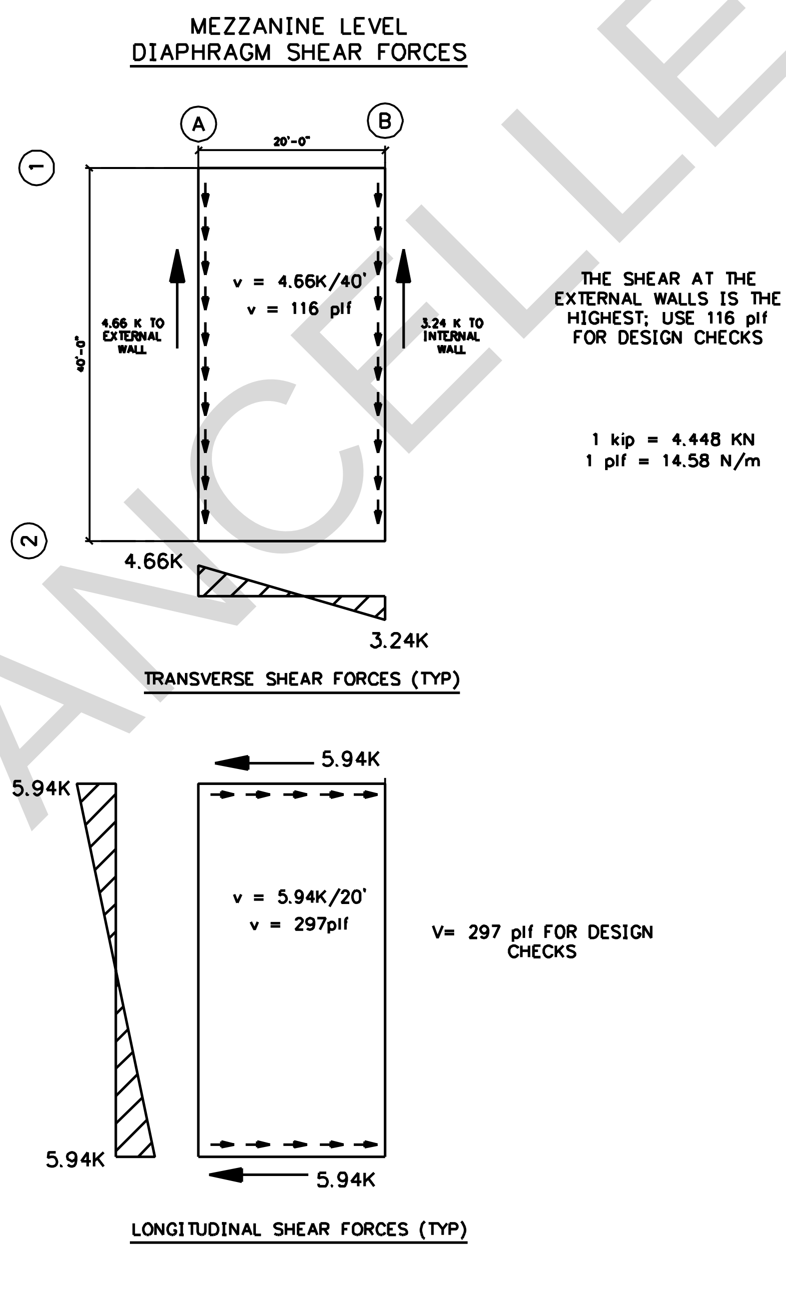
v = 4.66 kips / 40’ = 116 plf (1.69 KN/m)

Longitudinal forces:

Shear to each braced frame = ½ mezzanine diaphragm shear

Shear to each braced frame = (1/2)(11.87kips) = 5.94 kips (26.4 KN)

The unit diaphragm shear , v =(5.94kips)/20’ = 297 plf (4.33 KN/m)



<u>Determine shear in vertical elements due to self-weight inertial effects</u>

Add in the shear forces due to self-weight of vertical elements tributary to the mezzanines

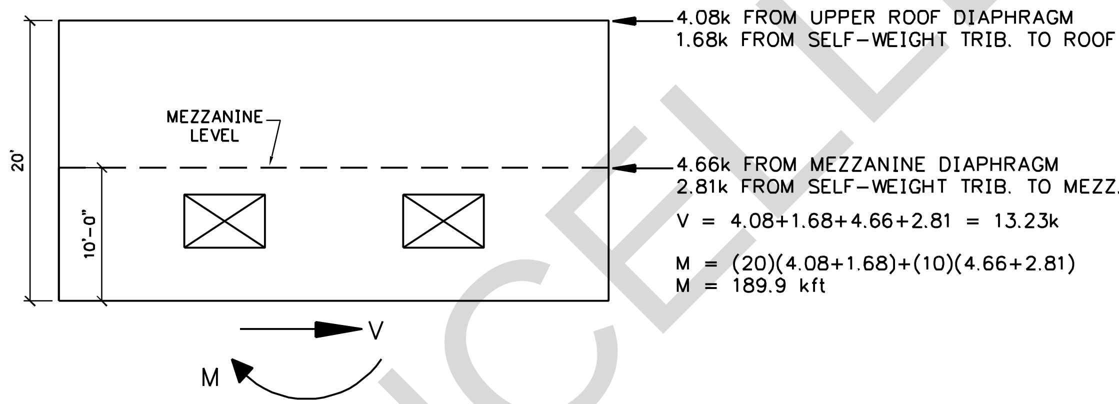
Transverse:

Typical exterior wall (A1-A2 & I1-I2)
Shear = C_s x wall weight trib to mezz. = (0.12)(23.4kips) = 2.81 kips
This shear is now added to the diaphragm shear transferred to the wall by the mezzanine;
Shear in exterior wall tributary to mezzanine = (2.81k + 4.66k) = 7.47 kips (33.2 KN)

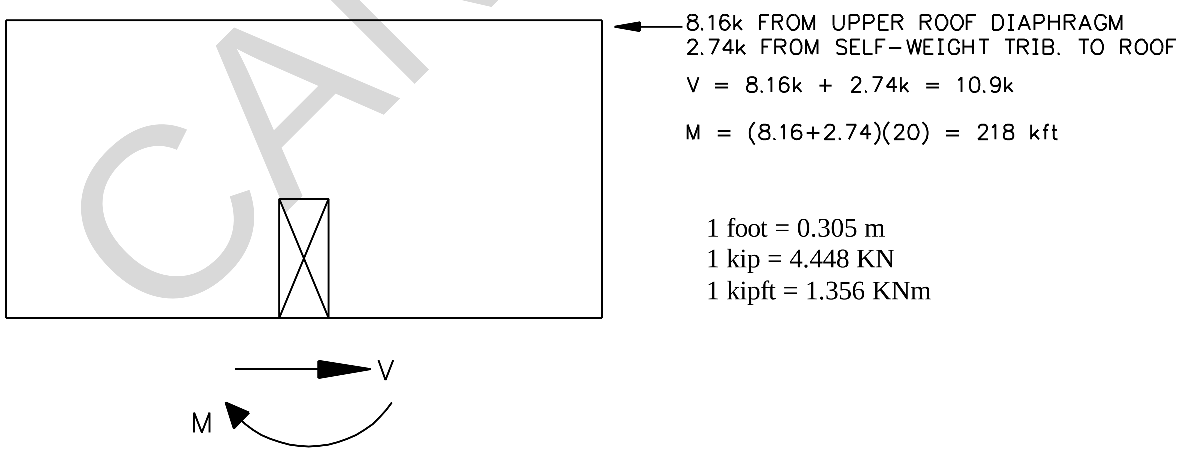
Interior mezzanine wall (B1-B2 & H1-H2)
Shear = (0.12)(11.4kips) = 1.37 kips
Shear in interior walls tributary to mezzanine = (1.37k + 3.24k) = 4.61 kips (20.5 KN)

Longitudinal:

Weight of metal panel walls tributary to each braced bay = 0.8 k
Shear = (0.12)(0.8 kips) = 0.096 kips
Shear force to each braced bay from mezz trib loads = (5.94 k + 0.096k) = 6.04 kips (26.9KN)

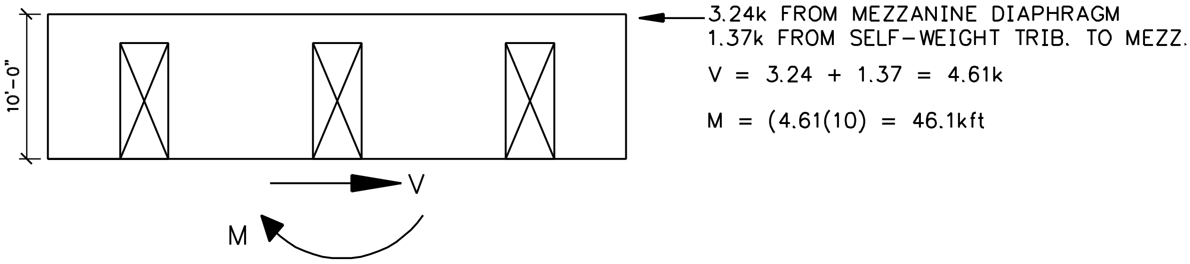


TYPICAL EXTERIOR CMU WALL

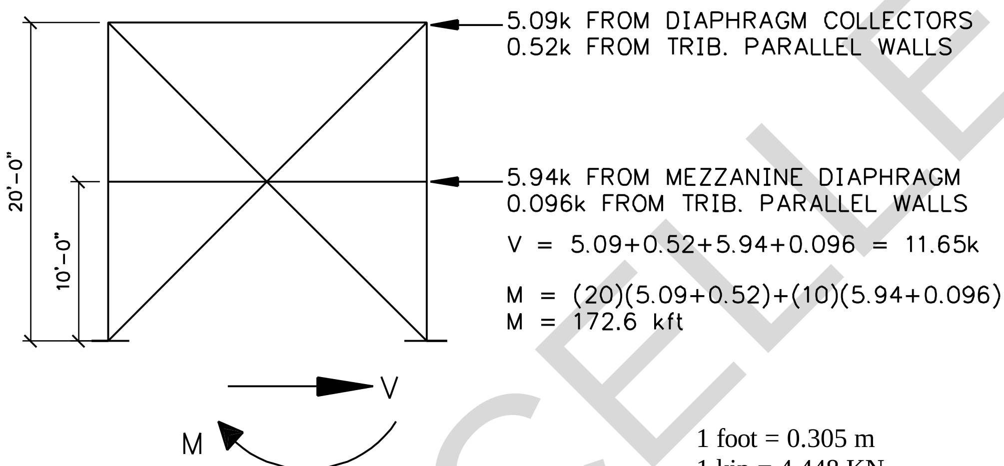


1 foot = 0.305 m
1 kip = 4.448 KN
1 kipft = 1.356 KNm

INTERIOR CMU SHEAR WALL (E1–E2)



TYPICAL INTERIOR CMU MEZZANINE WALL
(B1-B2 & H1-H2)



1 foot = 0.305 m
1 kip = 4.448 KN
1 kipft = 1.356 KNm

TYPICAL BRACED FRAME BAY

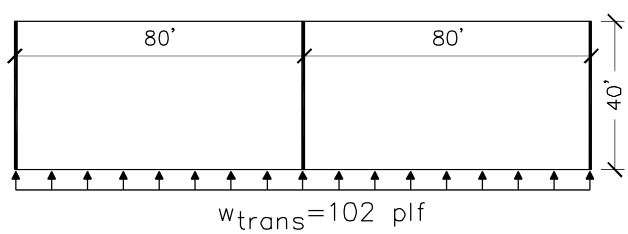
<u>Determine diaphragm chord and collector forces</u>

<u>Chord Forces: Upper roof</u>

Transverse direction:

The chord members for the upper roof diaphragm are the edge beams (W 12 x 26).
The diaphragm is assumed to be simply supported between the exterior shear walls and the interior CMU wall.

w = 102 plf, span = 80 ft., depth = 40 ft.
M = $wL^2/8$ = $(102)(80')^2/8$ = 81.6 kipft. (110.6 KNm)
T = M/d = (81.6 kipft) / 40 ft. = 2.04kips (9.07KN)

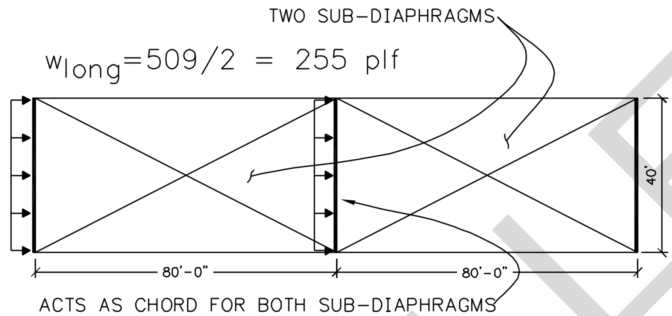


TRANSVERSE FORCE TO UPPER DIAPHRAGM

Longitudinal direction:

The steel in the bond beams (2- #5) resists seismic chord forces in the longitudinal direction. The diaphragm is assumed to act as two subdiaphragms spanning between the longitudinal collectors on the edges and the CMU walls (see diagram on next sheet). The steel in the firewall bond beam must resist the chord forces from both subdiaphragms. The chord forces at the interior wall would tend to cancel each other out, but assume they are additive to be conservative.

w = 509 plf (use 255 plf for each subdiaphragm), span = 40 ft., depth = 80 ft.
$M = wL^2/8 = (255)(40')^2/8 = 51.0$ kipft. (69.2 KNm)
T = M/d = (51.0 kipft) / 80 ft. = 0.64 kips (2.85KN)
Chord force to interior CMU wall bond beam = 2(0.64) = 1.28 kips (5.69 KN)



LONGITUDINAL FORCES TO UPPER DIAPHRAGM

Chord Forces: Mezzanines
Transverse direction

The diaphragm is assumed to act as a simply supported beam between the exterior and interior CMU shear walls. The mezzanine chord members for transverse seismic forces are the edge beams at the edge of the mezzanine.

w = 395 plf, span = 20 ft., depth = 40 ft.
$M = wL^2/8 = (395)(20')^2/8 = 19.7$ kipft. (26.7 KNm)
T = M/d = (19.7 kipft) / 40 ft. = 0.49 kips (2.18 KN)

Longitudinal direction

The steel in the bond beams of the exterior and interior mezzanine shear walls (2- #5) resists chord forces for seismic loading in the longitudinal direction.

w = 297 plf, span = 40 ft., depth = 20 ft.
$M = wL^2/8 = (297)(40')^2/8 = 59.4$ kipft.
T = M/d = (59.4 kipft) / 20 ft. = 2.97 kips

1 foot = 0.305m
1 plf = 14.59 N/m

MEZZANINE DIAPHRAGM FORCES

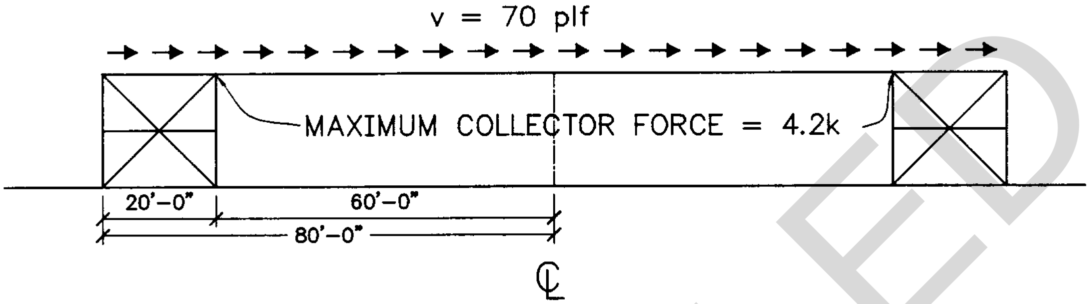
Collector Forces

The only collector members in the structure are the edge beams for seismic forces in the longitudinal direction. Each braced frame resists 5.61k from the upper roof diaphragm and tributary parallel walls/roll-up doors. This 5.61k is distributed into the collectors over a distance of 80'.

Shear to collectors = 5.61k / 80' = 70 plf

The maximum collector force occurs at the first brace (collector length of 60')

Maximum collector force = (70plf)(60') = 4.2k (18.7KN)



Out-of-plane wall forces

The shear walls and parapets must be checked for out-of-plane forces.

$$F_p = \frac{0.4a_pS_{DS}w_p}{R_p / I_p}\left(1+2\frac{z}{h}\right) \qquad \text{Eq. 10-1}$$

Shear walls:

$a_p = 1.0$, $R_p = 2.5$ Table 10-1
$I_p = 1.0$, $S_{DS} = 0.6$
$z = 20$ft., $h = 20$ ft. (for interior wall E1-E2)
$w_p = 57$ psf

$$F_p = \frac{0.4(1.0)(0.6)w_p}{2.5/1.0}\left(1+2\frac{20}{20}\right) = 0.288w_p\,,$$

but F_p in not required to be greater than

$$F_p = 1.6S_{DS}I_pw_p = 1.6(0.6)(1.0)w_p = 0.96w_p > 0.288w_p \qquad \text{Eq. 10-2}$$

nor is F_p to be less than

$$F_p = 0.3S_{DS}I_pw_p = 0.3(0.6)(1.0)w_p = 0.18w_p < 0.288w_p \qquad \text{Eq. 10-3}$$

Therefore, the governing value of F_p is:

$$F_p = 0.288w_p = 0.288(57\text{psf})(1'\text{strip}) = 16.4\text{plf (239N/m)}$$

Force at top & bottom of interior wall = (16.4 plf)(10') = 164 plf (2.39 KN/m)

Force at top of exterior wall due to wall below roof level = (16.4 plf)(5') = 82 plf

Parapets:

$a_p = 2.5$, $R_p = 2.5$ Table 10-1
$I_p = 1.0$, $S_{DS} = 0.6$
$z = 20$ft., $h = 20$ ft. (for interior wall E1-E2)
$w_p = 57$ psf

$$F_p = \frac{0.4(2.5)(0.6)w_p}{2.5/1.0}\left(1+2\frac{20}{20}\right) = 0.72w_p,$$

but F_p in not required to be greater than

$$F_p = 1.6S_{DS}I_pw_p = 1.6(0.6)(1.0)w_p = 0.96w_p > 0.72w_p \qquad \text{Eq. 10-2}$$

nor is F_p to be less than

$$F_p = 0.3S_{DS}I_pw_p = 0.3(0.6)(1.0)w_p = 0.18w_p < 0.72w_p \qquad \text{Eq. 10-3}$$

Therefore, the governing value of F_p is:

$$F_p = 0.72w_p = 0.72(57\text{psf})(1'\text{strip}) = 41.0\text{plf} \text{ (598 N/m)}$$

Force at bottom of parapet = (41.0 plf)(1') = 41.0 plf of wall length.

Total anchorage force of exterior wall + parapet = 164 plf + 41 plf = 205 plf (2.99 KN/m)

B.6 *Determine cm and cr*

The braces must be designed before the center of rigidity is determined.

Each braced bay must resist a total shear of 11.65 kips. It is assumed that the compressive and tension braces resist ½ of the load to each braced frame and that the braces do not resist gravity loads. Therefore, the unfactored load to each brace is :

(1/2)(11.65 kips) = 5.83 kips / brace. (25.9 KN)

Note: Section 14.5 of the AISC Seismic Provisions exempt low rise buildings designed to Load Combinations 4-1 and 4-2 from the special requirements of Sec. 14.2. – 14.4. However, some of the provisions will be followed as they are considered good design practice.

$1.2D + 0.5L + 0.2S + \Omega_0 Q_E$ (AISC Seismic Provisions Eq. 4-1)
$0.9D - \Omega_0 Q_E$ (AISC Seismic Provisions Eq. 4-2)

Note: The live load factor of 1.0 is required for use in Eq. 4-1 for garages, areas occupied as places of public assembly and all areas where the live load is greater than 100 psf. This structure is considered to be a garage.
$1.2D + 1.0L + 0.2S + \Omega_0 Q_E$ (AISC Seismic Provisions Eq. 4-1)

Required bracing strength based on Eq. 4-1:
$\Omega_0 Q_E = 2.0\ (5.83 \text{ kips}) = 11.7 \text{ kips}$ (52.0 KN)
The braces are at a 45 degree angle, axial force = 1.414 x hor. shear = (1.414)(11.7 k) = 16.5 k (73.4 KN)

AISC Seismic Provisions require that braces in V-Type or Chevron-Type configurations have a design strength of at least 1.5 times the required strength using LRFD Specification Load Combinations A4-5 and A4-6.

$1.2D \pm 1.0E + 0.5L + 0.2S$ (AISC LRFD Eq. A4-5)
$0.9D \pm (1.3W \text{ or } 1.0E)$ (AISC LRFD Eq. A4-6)

1.5 x factored force = (1.5)(5.83 kips) = 8.7 kips (38.7 KN) (AISC 14.4.a). This provision is exempt as noted above but is included here for illustrative purposes.

The braces are at a 45 degree angle, axial force = 1.414 x hor. shear = (1.414)(8.7 k) = 12.4 K (55.2 KN)

The axial force from equation 4-1 governs. Use 16.5 kips (73.4KN) for design.

The required compressive strength of a bracing member in axial compression shall not exceed 0.8 times $\phi_c P_n$. (AISC Seismic Provisions 14.2.b)

Try 3" standard pipe brace, A = 2.23 in^2, L = 14.1 ft., r = 1.16in., A36, K =1 per AISC LRFD Sec. C2.1

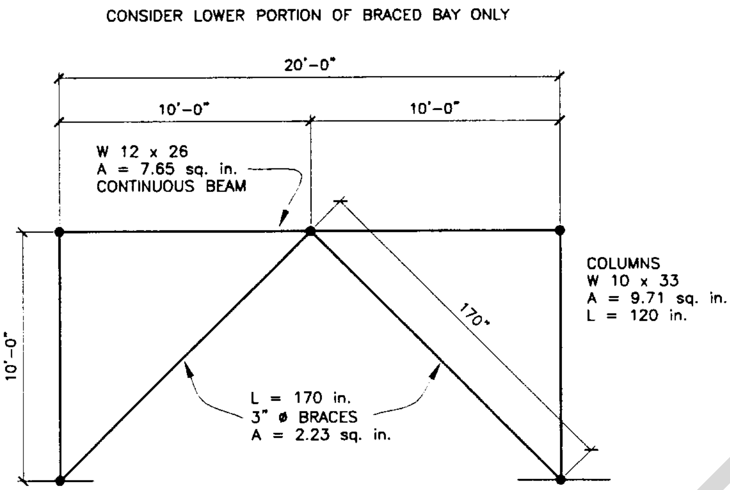
$$\lambda = \frac{KL}{\pi r}\sqrt{\frac{F_y}{E}} = \frac{(1)(14.1)(12)}{\pi(1.16)}\sqrt{\frac{36}{29000}} = 1.64$$ (AISC LRFD '93 Eq. E2-4)

$$F_{cr} = \frac{0.866}{\lambda^2} F_y = 11.65\text{ksi}$$ (AISC LRFD '93 Eq. E2-2)

$$\phi P_n = (0.85)(11.65\text{ksi})(2.23\text{in}^2) = 22.1\text{kips}$$

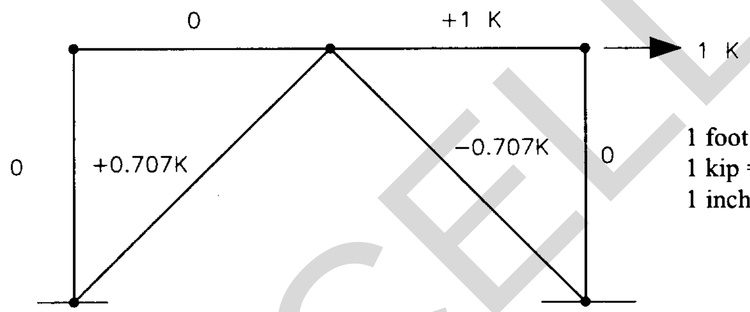
$0.8\phi P_n$ = (0.8)(22.1 kips) = 17.68 (78.6 KN) kips > 16.5 kips (73.4 KN), OK

Use 3" Standard pipe braces



Determine rigidity of braced frames

Use virtual work to determine frame stiffness;



1 foot = 0.305 m
1 kip = 4.448 KN
1 inch = 25.4 mm

$$1*\Delta = \int P\frac{pL}{AE}dx = \sum P\frac{pL}{AE}$$

$$\Delta = \frac{\left(\frac{(1)(1)(120)}{7.65}\right) + \left(\frac{(.707)(.707)(170)}{2.23}\right) + \left(\frac{(-.707)(-.707)(170)}{2.23}\right)}{29000}$$

$\Delta = 0.00317 \text{in} / \text{kip}$

$R = 1/\Delta$

$R = 316 \text{kip} / \text{in}\ (55.3\ \text{KN} / \text{mm}) =$ Rigidity of each braced frame bay.

CANCELLED

Mezzanine center of mass (typ)

Element	Weight (kips)	x (ft.)	y (ft.)	Wx (kip*ft)	Wy (kip*ft)
Deck	39.1821	10	20	391.8209	783.6418
CMU Wall A1-A2	23.37	0	20	0	467
CMU Wall B1-B2	11.4	20	20	228	228
Panel Wall 1A-1B	0.8	5	40	4	32
Panel Wall 2A-2B	0.8	5	0	4	0
S =	75.55			627.8	1511

$$cm_x = \frac{\sum Wx}{\sum W} \qquad cm_x = 8.31 \text{ ft.}$$

$$cm_y = \frac{\sum Wy}{\sum W} \qquad cm_y = 20.00 \text{ ft.}$$

Mezzanine center of rigidity

Element	R_{cx} (kip / in)	R_{cy} (kip / in)	x (ft.)	y (ft.)	yR_{cx}	xR_{cy}
CMU Wall A1-A2	0	6682	0	0	0	0
CMU Wall B1-B2	0	4648	20	0	0	92960
Braced Frame 1A-1B	316	0	0	40	12640	0
Braced Frame 2A-2B	316	0	0	0	0	0
S =	632	11330			12640	92960

$$cr_x = \frac{\sum xR_{cy}}{\sum R_{cy}} \qquad cr_y = \frac{\sum yR_{cx}}{\sum R_{cx}}$$

1 inch = 25.4 mm
1 foot = 0.305 m
1 kip = 4.448 KN

$cr_x =$ 8.20 ft. $\qquad cr_y =$ 20.00 ft.

B.7 *Perform torsional analysis*

Torsion due to eccentricity between the centers of mass and rigidity must be checked for each mezzanine. The design eccentricity is taken as the calculated eccentricity plus 5% of the perpendicular length of the structure under consideration.

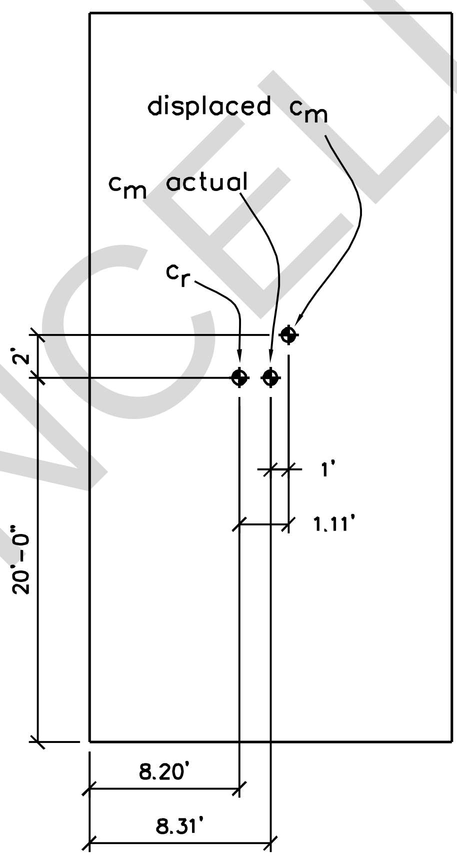
Transverse Seismic Forces

Calculated eccentricity = (8.31’)-(8.20’) = 0.11’
Accidental eccentricity = (0.05)(20’) = 1’
Design eccentricity = (1’)+(0.11’) = 1.11’
Shear forces tributary to each mezzanine = 12.1 kips
Torsion due to shear = (12.1k)(1.11’) =13.4 kipft

Longitudinal Seismic Forces

Calculated eccentricity = (20’)-(20’) = 0’
Accidental eccentricity = (0.05)(40’) = 2’
Design eccentricity = (0’)+(2’) = 2.0’
Shear forces tributary to each mezzanine = 12.1 kips
Torsion due to shear = (12.1k)(2.0’) =24.2 kipft

$$F_T = T\frac{Rd}{\sum Rd^2}$$



1 inch = 25.4 mm
1 foot = 0.305 m
1 kip = 4.448 KN

Distribution of Forces for Transverse Seismic Forces

Element	R	d	Rd	Rd^2	Torsional Force (kip)
CMU Wall A1-A2*	6682	-8.27	-55260	457001	-0.546
CMU Wall B1-B2	4648	11.73	54521	639532	0.539
Braced Frame 1A-1B	316	20	6320	126400	0.062
Braced Frame 2A-2B	316	20	6320	126400	0.062
S				1349333	

*Note: The torsional force to wall A1-A2 and I1-I2 acts in the opposite sense of the direct shear force. Only forces that are additional will be considered. Therefore, the torsional forces to walls A1-A2 and I1-I2 will be taken as zero.

Distribution of Forces for Longitudinal Seismic Forces

Element	R	d	Rd	Rd^2	Torsional Force (kip)
CMU Wall A1-A2	6682	8.27	55260	457001	0.988
CMU Wall B1-B2	4648	11.73	54521	639532	0.975
Braced Frame 1A-1B*	316	-20	-6320	126400	-0.113
Braced Frame 2A-2B*	316	20	6320	126400	0.113
S				1349333	

*Note: Since the braced frames 1A-1B & 2A-2B are symmetrical, use F = 113 # for both frames (Earthquake force can act in either direction, and the only eccentricity is due to accidental eccentricity which means the center of mass can be on either side of the center of rigidity).

Determine total shear forces to vertical resisting elements (Direct shear + Torsional force)

Note: The vertical elements (shear walls and braced frames) will be designed for the shear force that acts below the mezzanine level; i.e. the upper braced frames and portions of shear walls above the mezzanine level will be designed for the shear force levels at the base of the element.

Transverse Seismic Forces:

Element	Direct Shear Force (kips)	Torsional Shear Force (kips)	Total Shear Force (kips)
CMU Wall A1-A2	13.22	0.00	13.22
CMU Wall B1-B2	4.61	0.54	5.15
CMU Firewall E1-E2	10.90	0.00	10.90
CMU Wall H1-H2	4.61	0.54	5.15
CMU Wall I1-I2	13.22	0.00	13.22
Typical Braced Frame Bay	0.00	0.06	0.06

1 inch = 25.4 mm
1 foot = 0.305 m
1 kip = 4.448 KN

Longitudinal Seismic Forces:

Element	Direct Shear Force (kips)	Torsional Shear Force (kips)	Total Shear Force (kips)
CMU Wall A1-A2	0.00	0.99	0.99
CMU Wall B1-B2	0.00	0.98	0.98
CMU Firewall E1-E2	0.00	0.00	0.00
CMU Wall H1-H2	0.00	0.98	0.98
CMU Wall I1-I2	0.00	0.99	0.99
Typical Braced Frame Bay	11.64	0.11	11.75

1 kip = 4.448 KN

B.8 *Determine need for redundancy factor, r*

$$\rho_x = 2 - \frac{20}{r_{max}\sqrt{A_x}}$$ Eq. 4-1

Transverse Direction (CMU shear walls):

$$r_{max} = \frac{V_{max}(10/l_w)}{V_{story}}$$

r_{max} = (13.22)(10/40)/(47) = 0.07

$$\rho_x = 2 - \frac{20}{0.070\sqrt{(40)(160)}} = -1.5 \text{, use 1.0}$$

Longitudinal Direction (Braced frames):

$r_{max} = V_{max} / V_{story}$ = ½ (11.75) / 47 = 0.125

$$\rho_x = 2 - \frac{20}{0.125\sqrt{(40)(160)}} = 0.0 \text{, use 1.0}$$

Both the longitudinal and transverse directions have sufficient redundancy.

B.9 *Determine need for overstrength factor, W_o*

FEMA 302 requires the use of the overstrength factor when designing collectors (Sec. 5.2.6.4.2) and the design of diagonal bracing connections (Sec. 8.6.2). Therefore, the overstrength factor will be used for the collector force demand in the edge beams and their connections, and the braced frame connection demands.

B-10 *Calculate combined load effects*

The load combinations from ASCE 7-95 are:

(1) 1.4D
(2) 1.2D + 1.6L + 0.5L_r
(3) 1.2D + 0.5L + 1.6L_r
(4) 1.2D + E + 0.5L
(5) 0.9D + E

Where $E = \rho Q_E \pm 0.2\ S_{DS}D$ Eq. 4-4 & 4-5

When specifically required by FEMA 302 (Collectors, their connections, and bracing connections for this example) the design seismic force is defined by:

$E = \Omega_0 Q_E \pm 0.2 S_{DS}D$ Eq. 4-6 & 4-7

The term $0.2S_{DS}D$ is added to account for the vertical earthquake accelerations.

$0.2S_{DS}D = 0.2(0.6)D = 0.12\ D$
Therefore, 0.12 will be added to the dead load factor for load combinations 4 and 5.

B-11 *Determine structural member sizes*

<u>Upper roof diaphragm</u>

Unit shear check:

The applicable load combination for diaphragm shear reduces to 1.0E.

The allowable unit shear is determined by multiplying the value from the deck manufacturer's catalog by 1.5 (Sec. 7.7e4(b)2.i.)

q_{all} = 1.5 x 520 plf = 780plf (11.4 KN/m) (20-gage deck (1mm), top-seam welded at 24" (0.61m), 5 welds per end, 6'-8"span (2.03m))

Transverse shear = 102 plf (1.49 K/m) < 780 plf (11.4 KN/m), O.K.
Longitudinal shear = 64 plf (934 N/m) < 780 plf (11.4 KN/m), O.K.

<u>Mezzanine diaphragm forces</u>

Unit shear check:

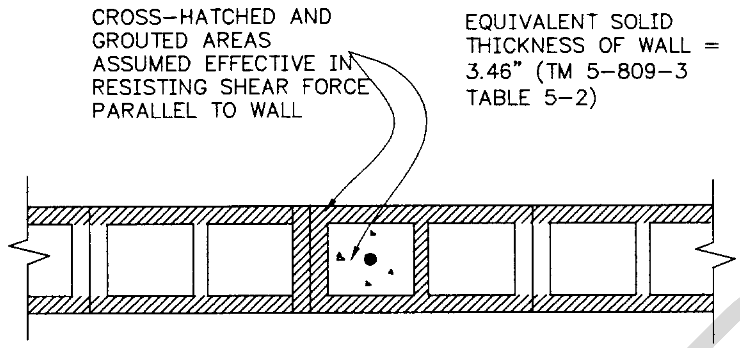
The applicable load combination for diaphragm shear reduces to 1.0E.

The allowable unit shear is determined by multiplying the value from the deck manufacturer's catalog by 1.5 (Sec. 7.7e4(b)2.i.)

q_{all} = 1.5 x 1745 plf = 2618 plf (38.2 KN/m) (20-gage (1mm) deck with 3-1/2" (89 mm) n.w. concrete fill, span = 8' (2.44m))

Transverse shear = 116 plf (1.69 KN/m) < 2618 plf (38.2 KN/m), O.K.
Longitudinal shear = 297 plf (4.33 KN/m) < 2618 plf (38.2 KN/m), O.K.

Shear walls



– Shear strength of wall (per FEMA 302 Sec. 11.7.2)

$V_u \leq \phi V_n$ (FEMA 302 Eq. 11.7.2.1)

$\phi = 0.80$ for shear (FEMA 302 Tab. 11.5.3)

$V_n = V_m + V_s$ (FEMA 302 Eq. 11.7.3.1-1)

The shear forces on the CMU walls are seen to be low. Therefore, the strength of the walls will be calculated based on the strength provided by the masonry only to see if they need only minimum steel reinforcing details.

$$V_m = \left[4.0 - 1.75\left(\frac{M}{Vd}\right)\right] A_n \sqrt{f'_m} + 0.25P \quad \text{(FEMA 302 Eq. 11.7.3.2)}$$

The M/Vd will be taken as one and the axial contribution of 0.25P is neglected to be conservative.

$V_m = [4.0 - 1.75(1)]A_n\sqrt{1500} = 87.1\text{psi} * A_n$, $(601\ KN/m^2)*A_n$

Based on the masonry only:
$\phi V_n = 0.8\ (87.1\text{psi}) * A_n = (69.7\text{psi})\ A_n$, $(481\ KN/m^2)*A_n$
$\phi v_n = 69.7$ psi $(481\ KN/m^2)$

The calculated shear stress in the masonry is determined by the relation:

$$f_v = \frac{V}{bd}$$

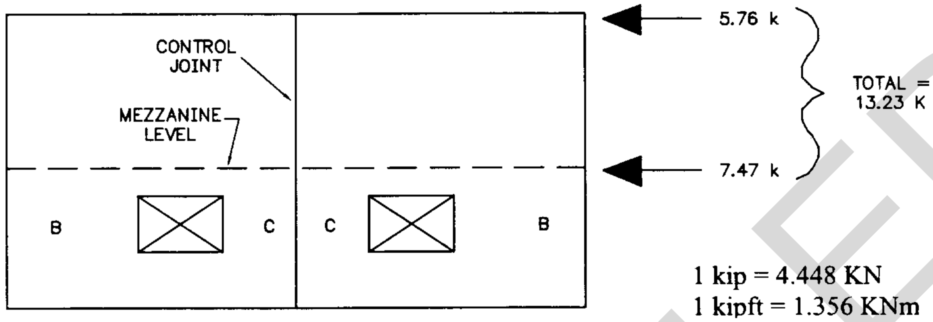
In this equation, b = equivalent solid thickness = 3.46' (88mm). (Note: This equivalent solid thickness is taken from TM 5-809-3 Table 5-2 and is a conservative value. The value used for the rigidity calculations (4.7" or 119 mm) is taken from the 'Reinforced Masonry Engineering Handbook' by Amrhein. The 3.46" thickness is used for strength calculations.)

The shear force resisted by each wall will be distributed to the individual piers based on their relative rigidities. The individual pier rigidities have been calculated previously.

Shear forces to individual piers

FEMA 302 Section 11.7.2.2 requires that the design shear strength of masonry members shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength of the member, except that the nominal shear strength need not exceed 2.5 V_u. The shear walls are all very strong in flexure due to their low h/l ratio and therefore it is nearly impossible to detail them to develop the shear strength corresponding to the flexural strength. Therefore, the shear demands on each wall pier will be scaled up by 2.5 and compared to the wall pier capacity.

Exterior CMU shear walls A1-A2 & I1-I2



1 kip = 4.448 KN
1 kipft = 1.356 KNm

The shear force will be split equally between the two wall areas on either side of the control joint.

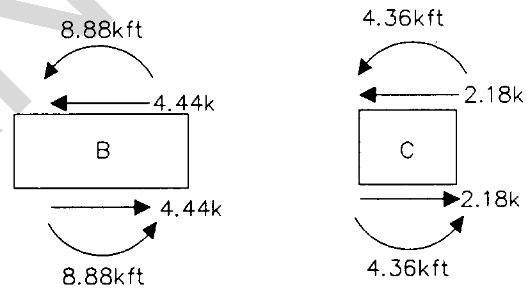
Vwall = ½ (13.23 k) = 6.62 k (29.4 KN)

This shear force is resisted by the individual piers based on their relative rigidities.

$R_B = 5509$ $\qquad R_C = 2701$

$$V_B = 6.62\left(\frac{5509}{5509+2701}\right) = 4.44\text{k}\ \ (19.7\ \text{KN}) \qquad V_C = 6.62\left(\frac{2701}{5509+2701}\right) = 2.18\text{k}\ (9.7\ \text{KN})$$

M_B = Vh / 2 = (4.44k)(4') / 2 = 8.88 kft = 107 kipin (12.0 KNm)
M_C = Vh / 2 = (2.18k)(4') / 2 = 4.36 kft = 52.3 kipin (5.9 KNm)



Shear stress to piers:

f_v = V / bd $\qquad$ where b is the equivalent solid wall thickness and d = pier length

f_{vB} = 2.5(4.44 k) / (9.33')(12"/')(3.46") = 28.7 psi (198 KN/m^2) < 69.7 psi (481 KN/m^2) (minimum shear reinforcement governs)
f_{vC} = 2.5(2.18 k) / (62")(3.46") = 25.5 psi (176 KN/m^2) < 69.7 psi (481 KN/m^2) (minimum shear reinforcement governs).

Determine need for trim steel:

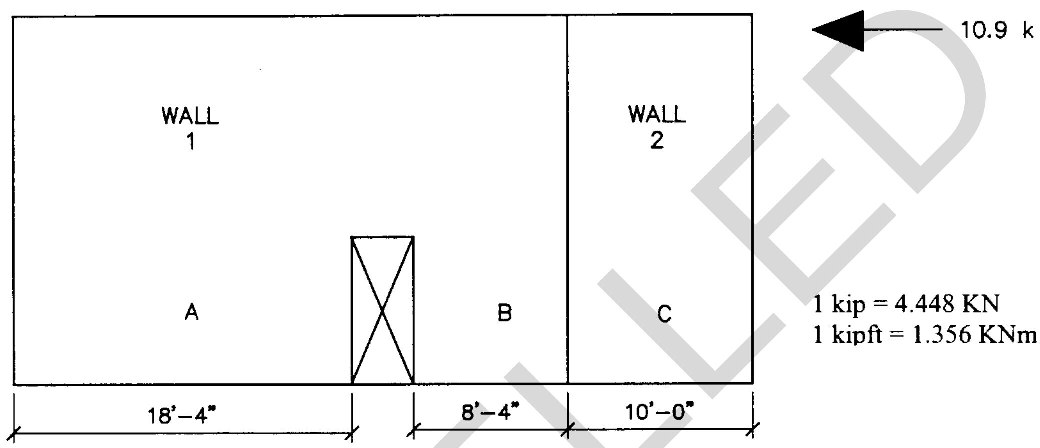
$M_{rs} = F_sA_sjd \quad A_s = M_{rs} / F_sjd$ (TM 5-809-3 Eq. 5-14)

Assume that $j \approx 0.9$ and that $F_s = 1.33(24\text{ksi}) = 32$ ksi (221 N/mm^2)

$As_B = (8.88 \text{ kft})(12''/') / (32 \text{ ksi})(0.9)(9.33')(12''/') = 0.03 \text{ in}^2$ (19.35 mm^2)

$As_C = (4.36 \text{ kft})(12''/') / (32 \text{ ksi})(0.9)(62'') = 0.03 \text{ in}^2$ (19.35 mm^2)

<u>Interior shear wall E1-E2</u>



1 kip = 4.448 KN
1 kipft = 1.356 KNm

The shear force will be resisted by Walls 1 and 2 in relation to the wall rigidities.

$R_{wall\,1} = 2223 \qquad R_{wall\,2} = 198$

$V_{wall\,1} = (10.9 \text{ k})(2223 / 2223+198) = 10 \text{ k}$ (44.5KN)
$V_{wall\,2} = (10.9 \text{ k})(198 / 2223+198) = 0.9 \text{ k}$ (4.0 KN)

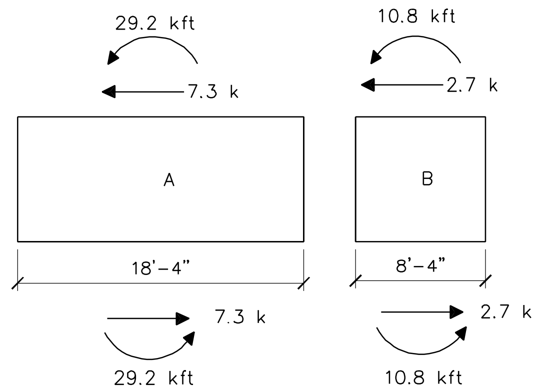
The shear force in wall 1 is resisted by the individual piers based on their relative rigidities.

$R_A = 5401 \qquad R_B = 1996$

$V_A = 10\left(\frac{5401}{5401+1996}\right) = 7.3\text{k}$ (32.5 KN) $\qquad V_B = 10\left(\frac{1996}{5401+1996}\right) = 2.70\text{k}$ (12.0 KN)

$V_C = V_{wall\,2} = 0.9\text{k}$ (4.0 KN)

$M_A = Vh / 2 = (7.3\text{k})(8') / 2 = 29.2 \text{ kft} = 350 \text{ kipin}$ (39.6 KNm)
$M_B = Vh / 2 = (2.7\text{k})(8') / 2 = 10.8 \text{ kft} = 130 \text{ kipin}$ (14.6 KNm)
$M_C = Vh = (0.9\text{k})(20') = 18 \text{ kft} = 216 \text{ kipin}$ (24.4 KNm)



1 kip = 4.448 KN
1 kipft = 1.356 KNm

Shear stress to piers:

$f_v = V / bd$ where b is the equivalent solid wall thickness and d = pier length

f_{vA} = 2.5(7.3 k) / (18.33')(12"/')(3.46") = 24 psi (165 KN/m^2) < 69.7 psi (481 KN/m^2) (minimum shear reinforcement governs)

f_{vB} = 2.5(2.70 k) / (8.33')(12"/')(3.46") = 19.5 psi (134 KN/m^2) < 69.7 psi (481 KN/m^2) (minimum shear reinforcement governs)

f_{vC} = 2.5(0.9 k) / (120")(3.46") = 5.42 psi (37 KN/m^2) < 69.7 psi (481 KN/m^2) (minimum shear reinforcement governs)

Determine need for trim steel:

$M_{rs} = F_sA_sjd$ $\quad A_s = M_{rs} / F_sjd$ (TM 5-809-3 Eq. 5-14)

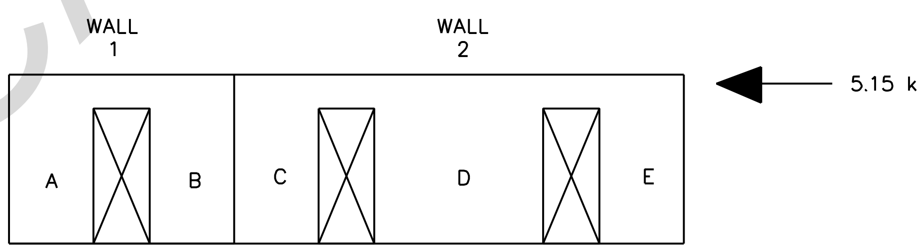
Assume that j ≈ 0.9 and that F_s = 1.33(24ksi) = 32 ksi (221 N/mm^2)

As_A = (29.2 kft)(12"/') / (32 ksi)(0.9)(18.33')(12"/') = 0.06 in^2 (39 mm^2)

As_B = (10.8 kft)(12"/') / (32 ksi)(0.9)(8.33')(12"/') = 0.05in^2 (32 mm^2)

As_C = (18 kft)(12"/') / (32 ksi)(0.9)(120") = 0.06 in^2 (39 mm)

<u>Interior mezzanine CMU shear walls B1-B2 & H1-H2</u>



The 5.15k (22.9 KN) force is distributed to the two wall segments separated by the control joint in relation to their relative rigidities.

$R_{wall\ 1}$ = 1180 $\quad R_{wall\ 2}$ = 3468

$V_{wall\,1} = 5.15(1180/1180+3468) = 1.31$ k (5.83 KN)
$V_{wall\,2} = 5.15(3468/1180+3468) = 3.84$ k (17.01 KN)

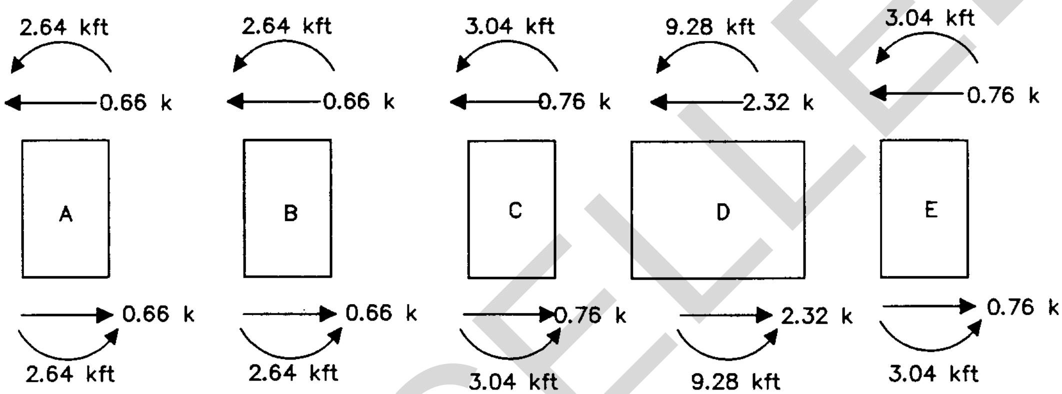
The shear force in wall 1 is resisted by the individual piers based on their relative rigidities.

$R_A = R_B = R_C = R_E = 845 \quad R_D = 2582$

$$V_A = V_B = 1.31\left(\frac{845}{845+845}\right) = 0.66\text{k} \ \ (2.94\text{ KN})$$

$$V_C = V_E = 3.84\left(\frac{845}{845+845+2582}\right) = 0.76\text{k}\ (3.38\text{ KN}) \qquad V_D = 3.84\left(\frac{2582}{2582+845+845}\right) = 2.32\text{k}\ (10.32\text{ KN})$$

$M_A = M_B = Vh / 2 = (0.66k)(8') / 2 = 2.64$ kft = 31.7 kipin (3.6 KNm)
$M_C = M_E = Vh / 2 = (0.76k)(8') / 2 = 3.04$ kft = 36.5 kipin (4.1 KNm)
$M_D = Vh / 2 = (2.32k)(8') / 2 = 9.28$ kft = 111.4 kipin (12.6 KNm)



1 kip = 4.448 KN
1 kipft = 1.356 KNm

Shear stress to piers:

$f_v = V / bd$ where b is the equivalent solid wall thickness and d = pier length

$f_{vA} = f_{vB} = 2.5(0.66 \text{ k}) / (60")(3.46") = 7.95$ psi (54.8 KN/m^2) < 69.7 psi (481 KN/m^2) (minimum shear reinforcement governs)

$f_{vC} = f_{vE} = 2.5(0.76 \text{ k}) / (60")(3.46") = 9.15$ psi (63 KN/m^2) < 69.7 psi (481 KN/m^2) (minimum shear reinforcement governs)

$f_{vD} = 2.5(2.32 \text{ k}) / (120")(3.46") = 13.97$ psi (96 KN/m^2) < 69.7 psi (481 KN/m^2) (minimum shear reinforcement governs)

Determine need for trim steel:

$M_{rs} = F_sA_sjd \quad A_s = M_{rs} / F_sjd$ (TM 5-809-3 Eq. 5-14)

Assume that $j \approx 0.9$ and that $F_s = 1.33(24\text{ksi}) = 32$ ksi (221 N/mm^2)

$As_A = As_B = (2.64 \text{ kft})(12"/') / (32 \text{ ksi})(0.9)(60") = 0.02 \text{ in}^2$ (13mm^2)

$As_C = As_E = (3.04 \text{ kft})(12"/') / (32 \text{ ksi})(0.9)(60") = 0.02\text{in}^2$ (13mm^2)

As_D = (9.28 kft)(12"/') / (32 ksi)(0.9)(120") = 0.03 in^2 (19mm^2)

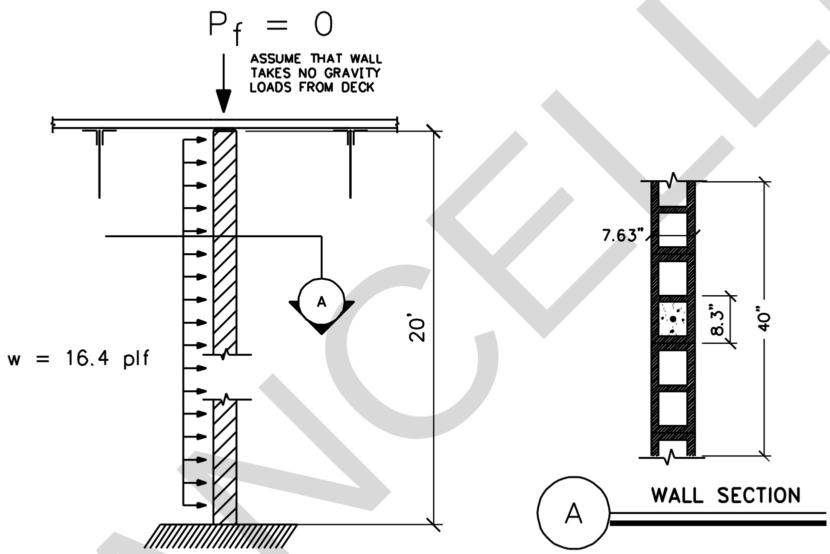
It is seen from the low shear stress values that the masonry alone can resist the shear forces without the reinforcement contribution. Therefore, the horizontal reinforcement will be based on the minimum reinforcement ratio details for all wall piers.

The trim steel requirement for each pier will be satisfied by having 2 - # 6 bars at the edges of openings, at wall ends, and at control joints.

Out-of-plane forces on CMU walls

The CMU walls must be checked for the interaction of axial loads (due to self-weight) and flexural moments. Wall E1-E2 is the most critical of the walls due to its slenderness and long unbraced height (20'). For walls with $h/t_w > 24$ it is suggested that the moment magnification due to P-Δ effects be considered (TM 5-809-3 Section 6-5). Wall E1-E2 has h/t_w = 240"/8" = 30 > 24, include P-Δ effects.

Determine out-of-wall strength:



1 plf = 14.59 N/m
1 inch = 25.4 mm

- Assume #6 bar at 40" o/c
- f'_m = 1500 psi (10.3 N/mm^2), F_m = 1/3 f'_m * (1.33) = 1/3(1500)(1.33) = 665 psi (4.6 N/mm^2)

- E_m = 1125 ksi * (7.76 KN/mm^2)
- f_y = 60 ksi (414 N/mm^2), F_s = 24 ksi (165 N/mm^2), E_s =29000 ksi (200 KN/mm^2)
- n = E_s / E_m = 29000 / 1125 = 25.7 (where n is the modular ratio)

*Note: The elastic modulus used for the out-of-plane deflections (1125 ksi) is determined from the equation $E_m = 750f'_m$ (from FEMA 302). This is lower than the value used for the wall rigidity calculation. The use of a lower modulus of elasticity for out-of-plane wall forces is conservative.

P_w = Weight of the wall at mid-height = (57psf)(10 ft.)(40/12) = 1900 lb. / 40"

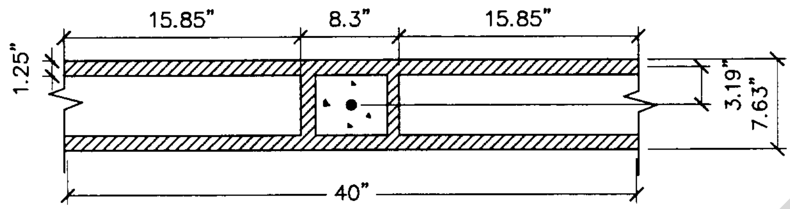
Axial Load Check: $f_a = \dfrac{P + w(10')}{A_e} = \dfrac{1.9k + (0.0164)(10)}{31.62} = 65.3\text{psi}$ (450 KN/m^2) (TM 5-809-3 Eq. 6-10)

Allowable Stress = $F_a = 0.20f'_m\left[1-\left[\frac{12h}{40t_n}\right]^3\right] = 0.20(1500)\left[1-\left[\frac{12(20)}{40(8)}\right]^3\right] = 173\text{psi}$ (1193 KN/m^2)

(TM 5-809-3 Eq.'s 5-24 & 5-25)

$f_a < F_a$, 65.3 psi < 173 psi, OK (450 < 1193)

- Determine gross moment of inertia, Ig:



1 inch = 25.4 mm

$$I_g = \frac{(8.3)(7.63)^3}{12} + 2\left[\left(\frac{31.7 * 1.25^3}{12}\right) + (31.7)(1.25)(3.19)^2\right]$$

$$I_g = 1124\text{in.}^4/40\text{" wide}$$

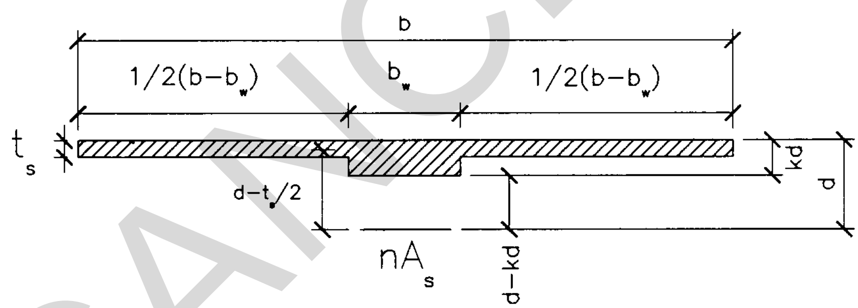
- Determine cracking moment

$$M_{cr} = \frac{2I_g f_r}{t}$$ TM 5-809-3 Eq. 5-7

$$f_r = 2.5\sqrt{f'_m} = 2.5\sqrt{1500} = 96.8\text{psi (668 KN/m}^2)$$ TM 5-809-3 Section 5.3.b.(5)

$$M_{cr} = \frac{2(1124)(96.8)}{7.63} = 28520\text{in.lbs} \quad (3.22\text{ KNm})$$

- Determine cracked moment of inertia, I_{cr}:



First determine if compression block lies with the shell or extends into to cells;

ρ = As / bd = (0.44) / (40)(3.81) = 0.0029 TM 5-809-3 Eq. 5-8
$n\rho$ = 25.7(0.0029) = 0.074

$k = \sqrt{(n\rho)^2 + 2n\rho} - n\rho = \sqrt{0.074^2 + 2(0.074)} - 0.074 = 0.32$ TM 5-809-3 Eq. 5-9

j = 1-k/3 = 1 - .32/3 = 0.89 TM 5-809-3 Eq. 5-10
kd = depth of compression block = 0.32(3.81") = 1.22" < 1.25"
Therefore, the compression block lies within the shell.

$$I_{cr} = nA_s(d-kd)^2 + \frac{1}{12}(t_s)^3(b) + (t_s)(b)\left(kd - \frac{t_s}{2}\right)^2$$

$$I_{cr} = 25.7(0.44)(3.81-1.22)^2 + \frac{1}{12}(1.25)^3(40) + (1.25)(40)\left(1.22 - \frac{1.25}{2}\right)^2 = 100\text{in.}^4$$

Calculate the mid-height moment, M_s, and the lateral deflection, Δ_s, due to service loads by iteration method.

$$M_s = \frac{wh^2}{8} + P_f(\text{eccentricity}/2) + (P_f + P_w)\Delta_s \text{, but } P_f = 0$$

$$M_s = \frac{(16.4)(40/12)(20)^2}{8}\frac{12"}{\text{ft.}} + 1900\Delta_s = 32800 + 1900\Delta_s(\text{in.lbs}) > M_{cr}$$

$M_{mid} = M_s$

- First iteration, $\Delta_s = 0$

$M_s = 32800 + 1900(0) = 32800$

$\Delta_s = 0.135 + 0.0000533(32800\text{-}28520) = 0.363"$

- Second iteration, $\Delta_s = 0.363$

$M_s = 32800 + 1900(0.363) = 33490$

$\Delta_s = 0.135 + 0.0000533(33490\text{-}28520) = 0.4"$

-Third iteration, $\Delta_s = 0.4$

$M_s = 32800 + 1900(0.4) = 33560$

$\Delta_s = 0.135 + 0.0000533(33560\text{-}28520) = 0.404"$, close enough

$M_s = 33560$ in.lbs. (3.79 KNm)= flexural demand on wall at mid-height.

Determine out-of-plane bending strength of wall: (Per TM 5-809-3)

$$M_{rs} = \frac{F_s A_s jd}{12}(\text{ft}-\text{lb}) \quad \text{for steel controlled} \qquad \text{(TM 5-809-3 Eq. 5-14)}$$

$$M_{rm} = \frac{F_m kjbd^2}{2(12)}(\text{ft}-\text{lb}) \quad \text{for masonry controlled} \qquad \text{(TM 5-809-3 Eq. 5-15)}$$

where;

$F_m = 1.33(1/3)f'_m = (1.33)(1/3)(1500 \text{ psi}) = 665 \text{ psi}$ (4.59 N/mm^2) (TM 5-809-3 Table 5-7)

$F_s = 1.33(24 \text{ ksi}) = 32 \text{ ksi}$ (221 N/mm^2)

$k = \sqrt{(n\rho)^2 + 2n\rho} - n\rho$ (TM 5-809-3 Eq. 5-9)

$\rho = A_s / bd = (0.44 \text{ in.}^2) / (40")(3.81") = 0.0029$ (TM 5-809-3 Eq. 5-8)

$$k = \sqrt{[(25.7)(0.0029)]^2 + (2)(25.7)(0.0029)} - (25.7)(0.0029) = 0.32$$

$j = 1 - k/3 = 1 - 0.32/3 = 0.89$ (TM 5-809-3 Eq. 5-10)

$$M_{rs} = \frac{(32\text{ksi})(0.44\text{in.}^2)(0.89)(3.81)}{12} = 3.98\text{kipft} = 47.7\text{kipin (5.4 KNm)} > 33.6\text{kipin (3.8 KNm)}\text{, OK}$$

$$M_{rm} = \frac{(665psi)(0.32)(0.89)(40)(3.81)^2}{2(12)} = 4.58kipft = 54.98kipin\ (6.21\ KNm) > 33.6kipin\ (3.8\ KNm)\ ,\ OK$$

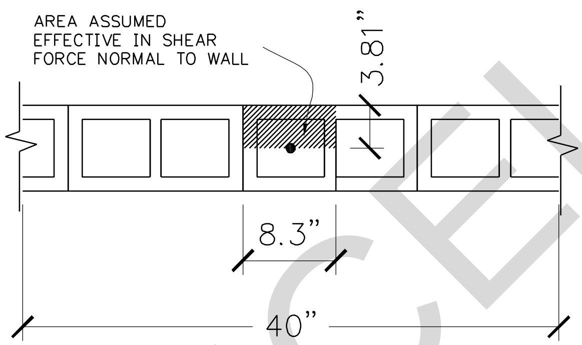
FEMA 302 Section 11.10.1 requires that the nominal flexural strength of the wall for out-of-plane flexure be at least equal to 1.3 times the cracking moment of the wall. The cracking moment was calculated previously to be 28.52 kipin. The flexural strength of the wall determined by allowable stress design was calculated as 47.7 kipin > 37.1 kipin (= 1.3 x 28.52). The flexural strength of the wall calculated using ultimate strength design is much greater than the strength calculated from allowable stresses. Therefore, assume OK.

Out-of-plane shear strength check

– Shear force demand;

The out-of-plane shear force demand is determined from the horizontal force on the wall face of 16.4 psf. Wall E1-E2 is the most critical with an unbraced span of 20’. Therefore, the shear demand for a 40” wide section is:

f = wL/2 = (16.4 psf)(40”)(1 ft. / 12”)(20 ft. / 2) = 547 lb / 40” (2.39 KN/m)



1 inch = 25.4 mm

Shear capacity;

Effective shear area, A_e = (8.3”)(3.81”) = 31.62 in.^{2} / 40” (TM 5-809-3 Fig. 5-2)

$f_v = R_a / b_w d$, where $b_w d = A_e$ and R_a = 547 lb. (TM 5-809-3 Eq. 6-17)

F_a = 547 lb / 31.62 in.2 = 17.3 psi (119 KN/m^2) < 69.7 psi (481 KN/m^2), OK

Out-of-plane bracing forces

Anchorage of walls to flexible diaphragms shall have the strength to develop the out-of-plane force give by:

$F_p = 1.2 S_{DS} I W_p$ FEMA 302 Eq. 5.2.6.3.3

– Interior wall E1-E2

W_p = (57psf)(20’/2) = 570 plf (8.32 KN/m)

F_p = 1.2(0.6)(1.0)(570) = 410 plf Equivalent to 0.41 (40 / 12) = 1.37 kips / 40” (5.98 KN/m)
Minimum anchorage demand = 200 plf (2.92 KN/m) < 410 plf (5.98KN/m) (Per Sec. 7-2.e(2))

This value is higher than the previously determined out-of-plane shear forces on the wall (547 lb / 40" = 164 plf). Therefore, use 410 plf (5.98 KN/m) for the anchorage of the interior CMU walls for out-of-plane forces.

– Exterior walls A1-A2 & I1-I2

At top of wall: W_n = (57 psf)[(10'/2) + (1' parapet)] = 342 plf
At mezz level: W_n = (57 psf)(5' top + 5' below) = 570 plf (8.32 KN/m) > 342 plf (4.99 KN/m)

Design all walls for out-of-plane anchorage force of 570 plf (8.32 KN/m)

Standard reinforcement details for CMU shear walls

Typical CMU wall reinforcing details are taken from Figures 7-14 – 7-16 and Section 7-2.h.(3)(h).6.iii . Additional development requirements are taken from FEMA 302 Section 11.4.5

– Embedment length, l_d: The embedment length of reinforcing bars is determined as:
  $l_d = 0.0015d_bF_s$ ACI 530 Eq. 8-1

  For #4 bar: l_d = 0.0015(4/8)(24000) = 18" (46 cm)
  For #5 bar: l_d = 0.0015(5/8)(24000) = 22.5", use 24" (61 cm)
  For #6 bar: l_d = 0.0015(6/8)(24000) = 27", use 28" (71 cm)

– Lap Splices: The minimum length of lap for bars in tension or compression is taken as:
  $l_d = 0.002d_bF_s$ ACI 530 Eq. 8-2

  For #4 bar: l_d = 0.002(4/8)(24000) = 24" (61 cm)
  For #5 bar: l_d = 0.002(5/8)(24000) = 30" (76 cm)
  For #6 bar: l_d = 0.002(6/8)(24000) = 36" (91 cm)

– Standard hooks: The typical standard hook for this structure (per FEMA 302 Sec. 11.4.5.3) shall be a 180 degree turn plus extension of at least $4d_b$ but not less than 2.5" (64mm), a 135-degree turn plus extension of at least 6 bar diameters at free end of bar, or a 90-degree turn plus extension of at least 12 bar diameters at free end of bar.

– Shear reinforcement: The shear shall extend to a distance *d* from the extreme compression face and shall be carried as close to the compression and tension surfaces of the member as cover requirements and the proximity of other reinforcement permit. Shear reinforcement shall be anchored at both ends for its calculated stress. The ends of a single leg shall be anchored by a standard hook plus an effective embedment of 0.5 l_d ACI 530 Sec. 8.5.6

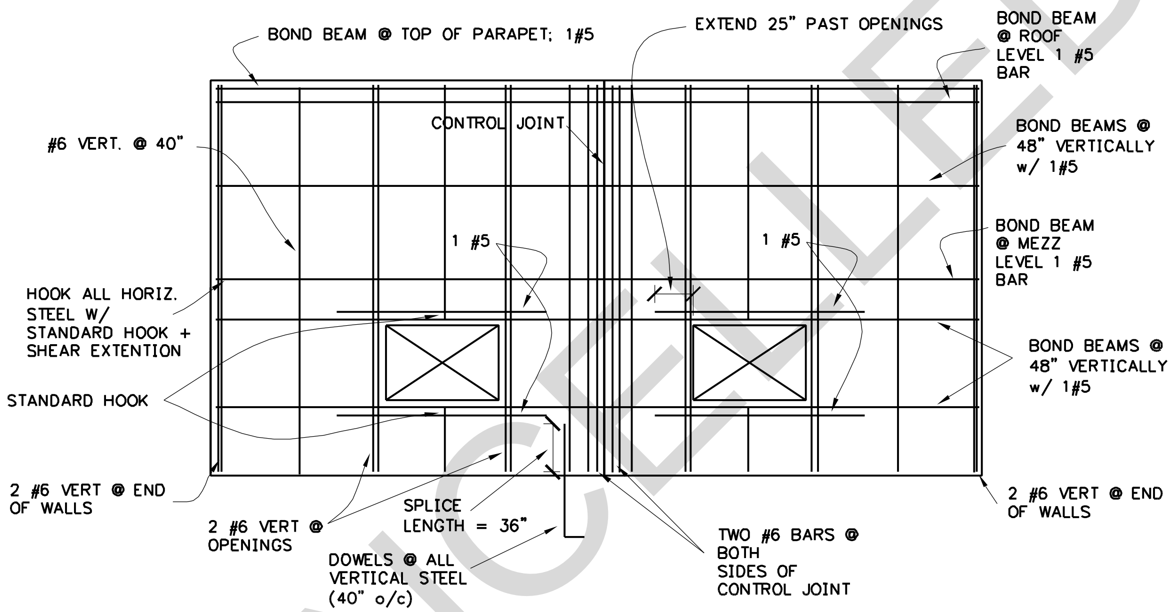
– Horizontal reinforcement at openings: Horizontal reinforcement of at least one #4 bar shall be placed on both sides of openings and extend a minimum of 24" (61cm) or $40d_b$ (Sec. 7.2.h.3.(h).6.)
  For #5 bars, $40d_b$ = 40(5/8) = 25" >24", use 25" (64 cm).

– Minimum wall reinforcement: All walls shall be reinforced with both vertical and horizontal reinforcement. The sum of the areas of horizontal and vertical reinforcement shall be at least 0.002 times the gross cross-sectional area of the wall and the minimum area of reinforcement in each direction shall not be less than 0.0007 times the gross cross-sectional area of the wall (per FEMA 302 Sec. 11.3.8.3).

For this structure #6 bars @ 40” (102 cm) were used for the vertical steel because the wall spans vertically between the supports. Therefore, the higher reinforcing ratio is used in the direction.

Vertical steel: 1 #6 bar @ 40” (102 cm); $\rho_v = 0.44/(8)(40) = 0.001375 > 0.0007$, OK
Horizontal steel: 1 #5 bar @ 48” (120 cm); $\rho_h = 0.31/(8)(48) = 0.0008 > 0.0007$, OK
$\rho_v + \rho_h = 0.0008 + 0.00138 = 0.0022 > 0.002$, OK

(Note: The horizontal joint reinforcing may be used to satisfy the reinforcing steel ratio but the strength contribution is neglected. For this example the horizontal joint reinforcing contribution to the steel ratio is neglected.)



Steel Members

Perimeter roof beams

The roof beams must support the gravity loads from the joist in addition to acting as collectors for longitudinal forces and chords for transverse forces. The beams were sized previously for the governing gravity load combination 1.2D + 1.6L. They must now be checked for the seismic load case: 1.2D + 0.5L + 1.0E

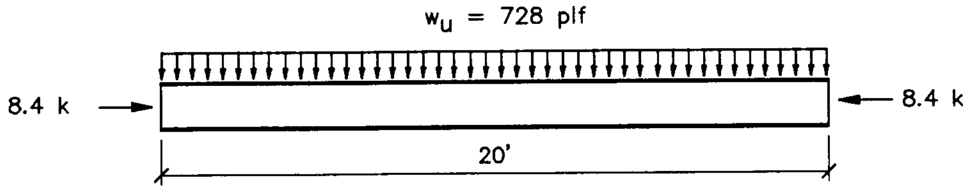
– FEMA 302 Section 5.2.6.4.2 requires that collector elements for structures in Seismic Design Category D be designed for the special load combination:
  $E = \Omega_o Q_E + 0.2S_{DS}D$ (FEMA 302 Eq. 5.2.7.1-1)

Loads:
Dead: 400 plf (5.84 KN/m) Live: 400 plf (5.84 KN/m)
Q_E: 2.04k (9.07 KN) (chord) Q_E: 4.2k (18.68 KN) (collector)

The collector force governs the design.

$w_u = 1.2D + 0.5L + 0.2S_{DS}D = 1.2(400) + 0.5(400) + (0.6)(0.2)(400) = 728$ plf (10.62 KN/m)

$E = \Omega o Q_E = 2.0(4.2) = 8.4k$ (37.4 KN) (collector axial force)



$M_u = w_u L^2/8 = (0.728klf)(20')^2/8 = 36.4$ kft (49.4 KN/m)
$P_u = 8.4$ k (37.4 KN)

1 kip = 4.448 KN
1 plf = 14.59 N/m

Check W 12 x 26;
- $A = 7.65$ in.^{2} (49.3 cm^2), $r_x = 5.17$in.(13.13 cm.), $r_y = 1.51$ in. (3.84 cm.)
- $L_x = 20'$(6.10m), $L_y = 6'$-8" (2.03 m) (braced by joists)

$(KL/r)_x = (1.0)(20)(12)/5.17 = 46.4$ $(KL/r)_y = (1.0)(6.67)(12)/1.51 = 53.0$ (controls)

$$\lambda = \frac{KL}{r\pi}\sqrt{\frac{F_y}{E}} = \frac{53}{\pi}\sqrt{\frac{36}{29000}} = 0.59$$ AISC LRFD '93 Eq. E2-4

$$F_{cr} = (0.658)^{\lambda^2} F_y = (0.658)^{0.59^2}(36) = 31.1\text{ksi } (214.4 \text{ N/mm}^2)$$ AISC LRFD '93 Eq. E2-2

$$P_n = A_g F_{cr} = (7.65)(31.1) = 238\text{k} \quad (1059 \text{ KN})$$ AISC LRFD '93 Eq. E2-1

$$\phi_c P_n = 0.85(238) = 202\text{k } (898 \text{ KN})$$

$$\phi_b M_n = 98.9\text{kft} \quad (138.0 \text{ KNm})$$ (determined previously)

$$\frac{P_u}{\phi_c P_n} = \frac{8.4}{202} = 0.04 < 0.2$$, Use interaction equation H1-1b

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$ AISC LRFD '93 Eq. H1-1b

$$\frac{8.4}{2(202)} + \left(\frac{36.4}{92.8} + 0\right) = 0.41 < 1.0$$ Use W12 x 26 for perimeter roof beams

<u>Braced frames (typical bay)</u>

The beams and columns have already been checked for the gravity load combinations, therefore, only seismic load combinations are checked here.

Members of the braced frame are checked for $1.2D + 1.0L + 0.2S + \Omega_0 Q_E$ (AISC Seismic Prov. Eq. 4-1) and $1.2D + 0.5L + 1.0E$ (where $E = \rho Q_E + 0.2S_{DS}D = 1.0Q_E + 0.12D$). It is assumed that all gravity load effects are resisted by the beams and columns with no loads being resisted by the braces (This is accomplished by analyzing the frame with the area of the braces set to zero). The braced frames are analyzed with the factored gravity loads applied and these member forces are superimposed with the forces due to the factored lateral loads for the load combinations involving seismic action. (The lateral load analysis uses the true brace area = 2.23 in.^{2} or 14.38cm^2).

Gravity loads to roof beams:

D = (roof load) + (tributary weight of walls) = (17 psf)(20') + (4psf)(5') = 360 plf (5.25 KN/m)

L = (roof load) = (20 psf)(20’) = 400 plf (5.84 KN/m)

Gravity loads to mezzanine beams:

D = (mezz. load) + (tributary weight of walls) = (39 psf)(4’) + (4 psf)(10’) = 196 plf (2.86 KN/m)
L = (mezz. load) = (125 psf)(4’) = 500 plf (7.30 KN/m)

Gravity loads to columns:

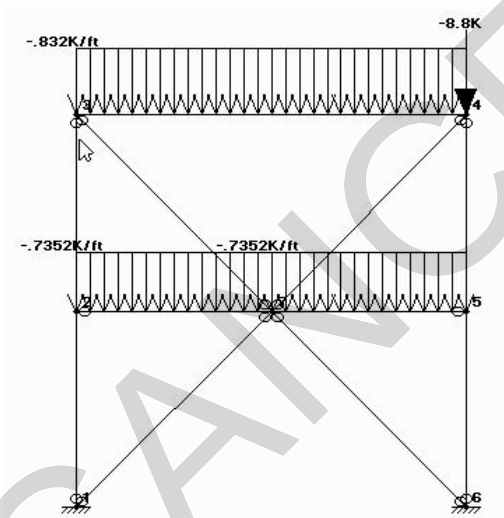
The interior columns of the braced bays (along wall lines B & H) must support the loads from the upper roof beams in both the braced bay and the adjacent door bay. Therefore, point loads are applied to these interior columns equal to the beam reaction force from the adjacent bay.

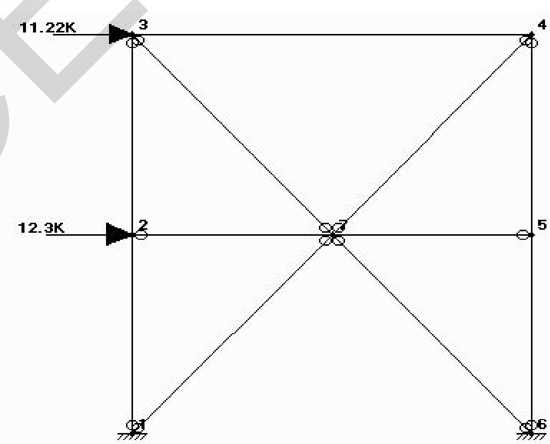
$D_{adj.\ beam}$ = 400 plf (see step A.10 for calculation)(20’/2) = 4k (17.8KN)
$L_{adj.\ beam}$ = 400 plf (see step A.10 for calculation)(20’/2) = 4k (17.8KN)

Lateral forces to braced frame:

$Q_{E\ top\ level}$ = 5.61 k (25.0 KN)
$Q_{E\ bot\ level}$ = 6.15 k (27.4 KN) (includes torsional force)

Load Case: 1.2D + 1.0L + 0.2S + $\Omega_0 Q_E$ (Assume L applies to all live loads, Ω_0 = 2.0).

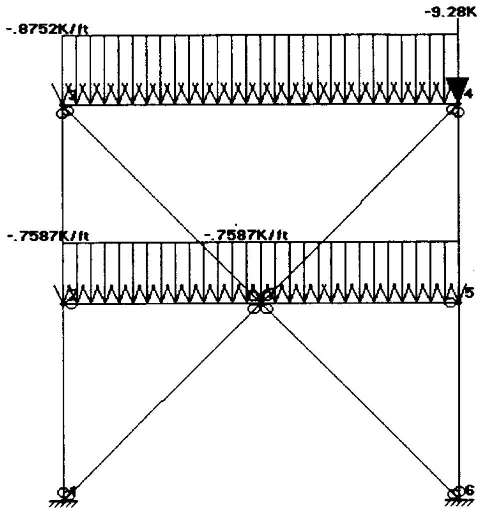




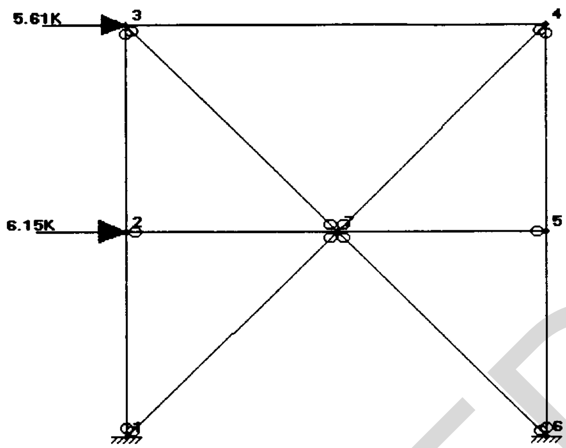
Gravity Loads

Lateral Loads

Load Case: 1.2D + 0.5L + 1.0E (where $E = \rho Q_E + 0.2S_{DS}D = 1.0Q_E + 0.12D$).



Gravity Loads



Lateral Loads

The Load Case: $1.2D + 1.0L + 0.2S + \Omega_0 Q_E$ governs for all member forces.

Column Check:

Axial load, P_u = 29.86k (132.8 KN) Maximum moment, M_{uy} = 0.3kft = 3.6 kipin. (0.41 KNm)

$\phi_c P_n = 132\text{k}$ (587 KN) (determined previously)

$\phi_b M_{ny} = \phi_b Z_y F_y = 0.9(38.8\text{in.}^3)(36\text{ksi}) = 105\text{kft}$ (142KNm)

$\dfrac{P_u}{\phi_c P_n} = \dfrac{29.86}{132} = 0.23 > 0.2$, Use interaction equation H1-1a

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$ AISC LRFD '93 Eq. H1-1a

$\dfrac{29.86}{132} + \dfrac{8}{9}\left(0 + \dfrac{.3}{105}\right) = 0.23 < 1.0$ Use W10x33 for columns

Roof Beam Check:

Axial load, P_u = 5.35 k (23.8 KN) Maximum moment, M_u = 41.6 kft = 499 kipin (56.4 KNm)

$\phi_c P_n = 202\text{k}$ (898KN) (determined previously)

$\phi_b M_n = 98.9\text{kft}$ (134.1 KNm) (determined previously)

$\dfrac{P_u}{\phi_c P_n} = \dfrac{5.35}{202} = 0.03 < 0.2$, Use interaction equation H1-1b

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$ AISC LRFD '93 Eq. H1-1b

$\dfrac{5.35}{2(202)} + \left(\dfrac{41.6}{98.9} + 0\right) = 0.43 < 1.0$ Use W12 x 26 for perimeter roof beams

Mezzanine Beam Check:

Axial load, P_u = 12.23 k (54.4 KN) Maximum moment, M_u = 36.84 kft = 442 kipin (50KNm)

$\phi_c P_n = 202\text{k}$ (898 KN) (determined previously)

$\phi_b M_n = 100\text{kft}$ (136 KNm) (determined previously)

$\frac{P_u}{\phi_c P_n} = \frac{12.23}{202} = 0.06 < 0.2$, Use interaction equation H1-1b

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0$$ AISC LRFD '93 Eq. H1-1b

$\frac{12.23}{2(202)} + \left(\frac{36.84}{100} + 0 \right) = 0.40 < 1.0$ Use W12 x 26 for perimeter roof beams

Brace Check:

Brace Axial Force from Load Case: 1.2D + 1.0L + 0.2S + $\Omega_0 Q_E$ = 16.94 k (75.3 KN) (Governs)

Brace Axial Force from Load Case: 1.2D + 0.5L + 1.0E (where E = $\rho Q_E + 0.2 S_{DS} D = 1.0 Q_E + 0.12D$) = 8.47 k (37.7KN)
The brace force is required to be scaled by 1.5 for this load combination (AISC Seismic Provisions 14.4.a.1) = 1.5(8.47 k) = 12.71 k (56.5 KN).

Brace compression capacity = 17.68 kips (78.6 KN) > 16.94 kips (75.3 KN), OK (see section B.6 for capacity calculation).

Diaphragm Connections

- Axial stregnth: (needed for out-of-plane bracing forces)
  Assume anchors are ¾" headed bolts (A = 0.44 in.^{2} (2.88cm^2), f_y = 50 ksi (345 N/mm^2)) with a 4" (10.2cm) embedment length spaced at 4' o/c (10.2 cm). There are no anchor bolts in tension for which edge distances are a concern.

  $B_a = 4\phi A_p \sqrt{f'_m}$ (FEMA 302 Eq. 11.3.12.1-1)

  $B_a = \phi A_b f_y$ (FEMA 302 Eq. 11.3.12.1-2)

  ϕ = 0.5 for Eq. 11.3.12.1-1 and ϕ = 0.9 for Eq. 11.3.12.1-2 (FEMA 302 Sec. 11.3.12.1)

  $A_p = \pi l_b^2$ (edge distance > embedment length) (FEMA 302 Eq. 11.3.12.1.1-1)

  A_p = π(4")2 = 50.23 in.^{2} (324 cm^2)

  $B_a = 4(0.5)(50.23)\sqrt{1500} = 3.89\text{kips}$ (17.3 KN) (governs)

  $B_a = (0.9)(0.44)(50) = 19.8\text{k}$ (88.1 KN)

  Anchor bolt tensile strength = 3.89 kips (17.3 KN)

- Shear strength:

$B_v = 1750\phi\sqrt{f'_m A_b}$ (FEMA 302 Eq. 11.3.12.3-1)

$B_v = 0.6\phi A_b f_y$ (FEMA 302 Eq. 11.3.12.3-2)

$\phi = 0.5$ for Eq. 11.3.12.3-1 and $\phi = 0.9$ for Eq. 11.3.12.3-2 (FEMA 302 Sec. 11.3.12.1)

$$B_v = 1750(0.5)\sqrt{(1500)(0.44)} = 22.5\text{k } (100\text{KN})$$

$$B_v = (0.6)(0.9)(0.44)(50) = 11.9\text{k } (52.9 \text{ KN}) \text{ (governs)}$$

Interior wall E1-E2 is braced for out-of-plane forces by anchor bolts that are subject to shear. The edge distance for these bolts = 3.81" (9.68 cm) is less than 12db = 12(3/4") = 9" (22.86 cm). The shear capacity of these bolts must be reduced per FEMA 302 Sec. 11.3.12.3.

$$\text{Reduced Strength} = \left(\frac{3.81-1}{(12)(3/4)-1}\right)(11.9\text{k}) = 4.18\text{k } (18.6\text{KN})$$

All other shear connections have adequate edge distance with a bolt shear strength = 11.9k (52.9 KN)

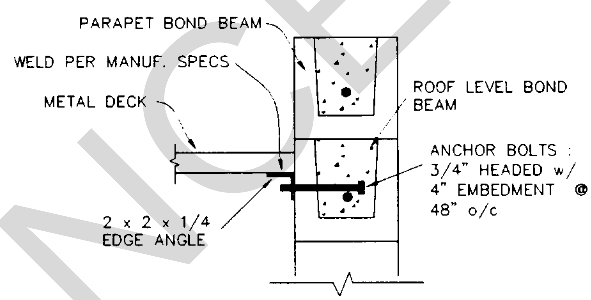
Shear transfer mechanism for upper-roof diaphragm

Transverse direction:

Deck-to-exterior wall connection: The deck is welded to an edge angle per the manufacturer's specs; the angle is anchor-bolted to the CMU bond beam; the bolts carry only shear from forces parallel to the walls (no gravity loads).

Shear demand = 102 plf (1.49 KN/m)
Shear capacity of bolts @ 4' (10.2 cm) o/c = 11.9k / 4' = 2975 (43.4KN/m) plf > 102 plf (1.49 KN/m), OK

1 inch = 25.4 mm

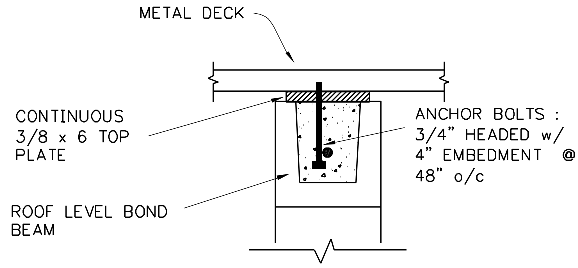


TYPICAL CONNECTION OF UPPER ROOF DIAPHRAGM TO EXTERIOR CMU WALL

Deck-to-interior firewall connection: The deck is welded to a continuous 3/8" (9.5mm) top plate per the manufacturer's specs; the top plate is anchor-bolted to the CMU bond beam. The firewall receives 102 plf from both the adjacent sub-diaphragm spans; therefore, design the anchor bolts for:

V = 2(102 plf) = 204 plf (2.98 KN/m)
Shear capacity of bolts @ 4' o/c = 11.9k / 4' = 2975 plf (43.4 KN/m) > 204 plf (2.98 KN/m), OK

1 inch = 25.4 mm



TYPICAL CONNECTION OF UPPER ROOF DIAPHRAGM TO INTERIOR CMU WALL

Longitudinal direction:

Deck-to-braced frame collectors: The metal deck is welded to an edge angle per the manufacturer's specs. The edge angles are welded continuously to the edge beam collectors (detail not shown).

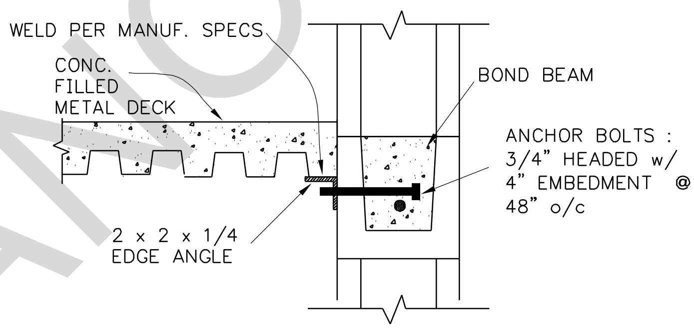
Shear transfer mechanism for mezzanine-to-vertical element connection

Transverse direction:

Exterior walls: The deck is welded to an edge angle per the manufacturer's specs; the angle is anchor-bolted to the CMU bond beam; the bolts carry only shear from forces parallel to the walls (no gravity loads).

Shear demand = 116 plf (1.69 KN/m)
Shear capacity of bolts @ 4' o/c = 11.9k / 4' = 2975 plf (43.4 KN/m) > 116 plf (1.69 KN/m), OK

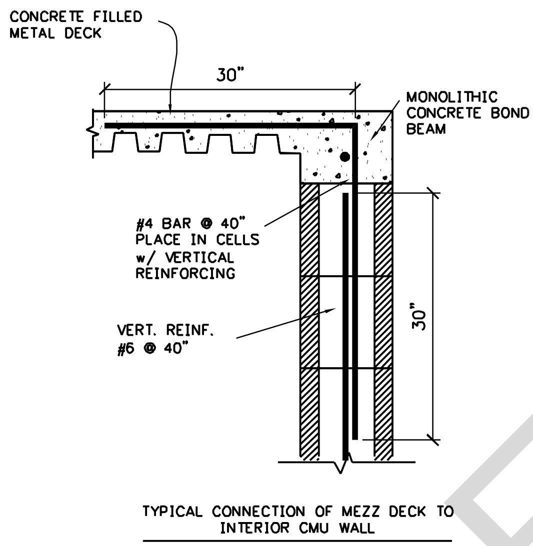
1 inch = 25.4 mm



TYPICAL CONNECTION OF MEZZ DECK TO EXTERIOR CMU WALL

Interior walls: Shear forces are transferred from the mezzanines to the CMU shear walls through dowels. The dowels are embedded into the concrete deck topping and bent around the bond beam steel in the wall. Shear demand on dowels = transverse mezzanine diaphragm shear = 116 plf (1.69 KN/m). By inspection it is seen that this connection has adequate capacity.

1 inch = 25.4 mm



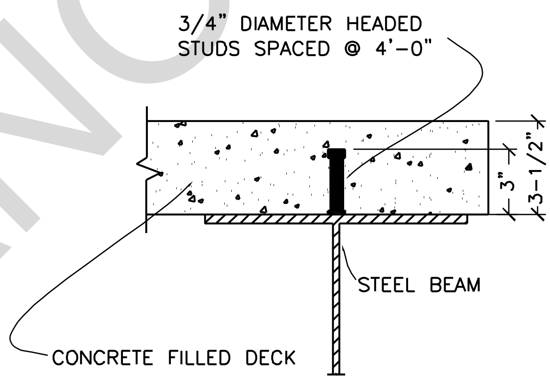
Longitudinal direction:

Shear forces are transferred from the mezzanines to edge beams of the braced frame bays through shear studs.

Shear demand on shear studs = longitudinal diaphragm shear = 297 plf (4.33 KN/m)

Details for cast-in-place concrete slabs not monolithic with supporting framing are given in Figure 7-52. The concrete filled deck has a total thickness of less than 6" (15.2 cm) which calls for the use of 3" (7.6cm) automatically welded studs with granular flux filled ends. The studs shall be spaced at every four feet (1.22m).

1 inch = 25.4mm
1 foot = 0.305m

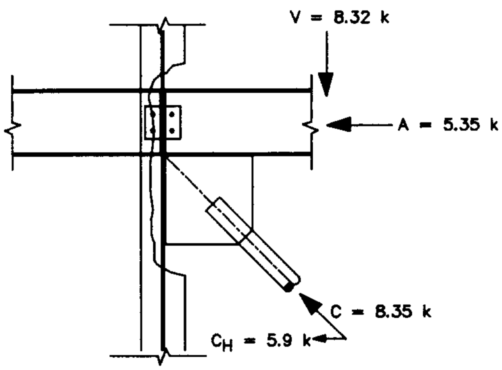


Steel Connections

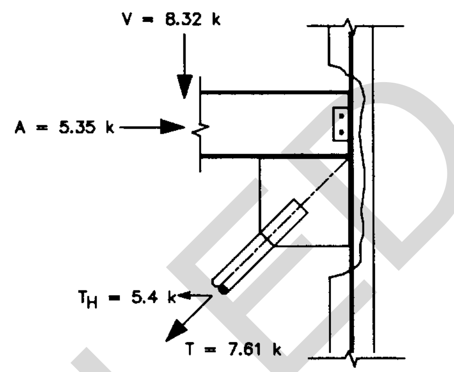
Roof edge beam-to-column connection at braced bay

The connection must be checked for the seismic load combination 1.2D + 1.0L + $\Omega_0 Q_E$. This seismic load combination includes the horizontal component of the brace force on the connection.

Forces from elastic analysis of braced frame (analysis not shown).



BRACE IN COMPRESSION



BRACE IN TENSION

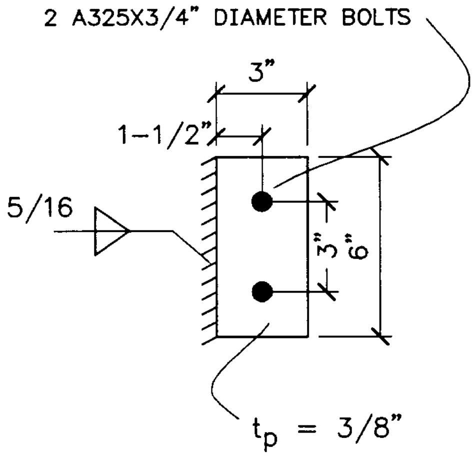
1 kip = 4.448 KN

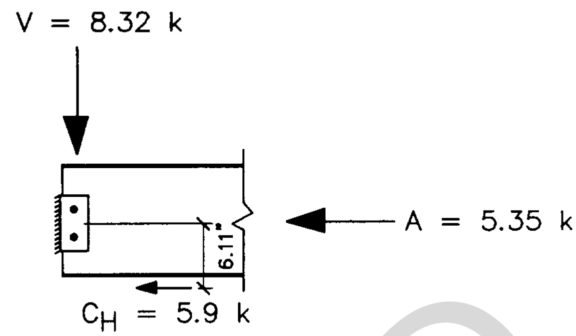
For brace in compression;

Total shear , V = 8.32 k (37.0 KN)
Total axial, A = 5.35 + 5.9 = 11.25 k (50.0 KN)
Bolt design eccentricity, $e_b = |(n-1)-a| \geq a = |(2-1)-1.5| = 0.5$, use 1.5" (3.81cm) AISC LRFD Part 9
Total moment about bolt cg = 5.9(6.11) + 8.32(1.5) = 48.53 kip*in (5.48KNm)

For brace in tension;

Total shear , V = 8.32 k (37.0 KN)
Total axial, A = 5.35 - 5.4 = 0.05 k (0.224 KN) (tension)
Bolt design eccentricity, $e_b = |(n-1)-a| \geq a = |(2-1)-1.5| = 0.5$, use 1.5" (3.81 cm) AISC LRFD Part 9
Total moment about bolt cg = 5.4(6.11) - 8.32(1.5) = -20.5 kip*in (2.32 KNm)





1 kip = 4.448 KN
1 inch = 25.4 mm

– Design weld (determine minimum size required)

$$f_b = Mc/I = 48.53(3)/(1/12)(1)(6)^3(2) = 4.04\text{ksi / inch of weld (10.97 N/mm}^2\text{ per cm. of weld)}$$
$$f_t = \text{Axial}/A = 11.25\text{k}/2(6) = 0.938\text{ksi / inch of weld (2.55 N/mm}^2\text{ per cm. of weld)}$$
$$f_v = V/A = 8.32\text{k}/2(6) = 0.69\text{ksi / inch of weld (1.87 N/mm}^2\text{ per cm. of weld)}$$
$$f_r = \sqrt{\left((4.04+0.938)^2+0.69^2\right)} = 5.03\text{ kip / inch of weld (13.65 N/mm}^2\text{ per cm. of weld)}$$

Weld strength = $\phi(0.6)(70\text{ksi})(0.707)(\text{size})$, $\phi = 0.75$ AISC LRFD Sec. J4

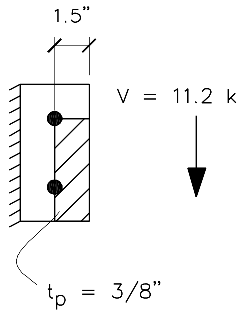
Weld size required = $\dfrac{5.03}{(.707)(1)(0.75)(0.6)(70)} = 0.23"$ (5.84 mm)

The AISC LRFD Manual Part 9 discussion for design checks of single-plate connections requires that the minimum weld size be equal to ¾ of the plate thickness to ensure that the weld is not the critical element (plate yielding before weld). = ¾(3/8)= 0.28" (7.11 mm), use 5/16" weld = 0.31" (7.87 mm) > 0.28" (7.11 mm)

– Check plate net section:
The plate connection never has a large net tension load as is seen from the connection force diagrams above. However, this connection will be used for all of the gravity frame connections. The gravity beams act as collectors; check the net section capacity against the collector demand.
Collector demand = 8.4 k (37.4 KN) (previously determined)

$A_n = (6-2(3/4+1/16))3/8 = 1.64\text{ in.}^2\ (10.58\text{ cm}^2)$
$\phi P_n = \phi_t F_u A_n = 0.75(58)(1.64) = 71.3$ kips (317 KN) > 8.4 kips (37.4 KN), OK AISC LRFD Eq. D1-2

– Check block shear:
Check block shear for vertical beam reaction. The vertical reaction from the seismic load combination (8.32 kips or 37.01 KN) is less than that from the gravity loads to the edge beams from the load combination 1.2D + 1.6L. The design vertical shear = $w_u L/2$ = (1120 plf)(20')/2 = 11.2 k (49.8 KN)



1 kip = 4.448 KN
1 inch = 25.4 mm

d_b = 3/4 + 1/16 = 0.813 in. (2.07 cm)
A_{nt} = (3/8)(1.5-.813/2) = 0.41 in.^{2} (2.64 cm^2)
A_{gv} = (3/8)(4.5) = 1.69 in.^{2} (10.90 cm^2)
A_{nv} = (3/8)(4.5 – 1.5(0.813)) = 1.23 in.^{2} (7.93 cm^2)
A_{gt} = (3/8)(1.5) = 0.56 in.^{2} (3.61 cm^2)
$\phi R_n = \phi(0.6F_yA_{gv} + F_uA_{nt})$ AISC LRFD Eq. J4-3a
ϕR_n = 0.75(0.6(36)(1.69) + (58)(0.41)) = 45.21 k (201 KN) >11.2 k (50 KN), OK
$\phi R_n = \phi(0.6F_uA_{nv} + FyA_{gt})$ AISC LRFD Eq. J4-3b
ϕR_n = 0.75(0.6(58)(1.23) + (36)(0.56)) = 47.2 k (210 KN)> 11.2 k (50 KN), OK

– Check shear yielding of plate
  $\phi_vV_n = \phi_v(0.6)(F_y)(A)$ AISC LRFD Eq. J5-3
  ϕ_vV_n = 0.9(0.6)(36)(6 x 3/8) = 43.7k (194 KN) > 11.2 k (50KN), OK

– Check shear fracture of plate
  $\phi_vV_n = \phi_v(0.6)F_uA_n$ AISC LRFD Eq. J4-1
  ϕ_vV_n = 0.75(0.6)(58)(6-2(0.813))3/8 = 42.8 k (190KN) > 11.2 k (50 KN), OK

– Check bolt shear and plate bearing
  Bearing strength of one bolt: $\phi r_n = \phi 2.4dtF_u$ AISC LRFD Eq. J3-1a
  ϕr_n = (0.75)(2.4)(3/4)(3/8)(58) = 29.4 k (131KN)

  Shear strength of bolt: $\phi r_n = \phi F_nA_b$ AISC LRFD Sec. J-3
  ϕr_n = 0.75(60)(0.44) = 19.8 k (18 KN) (governs)

Determine bolt shear demand based on elastic analysis of bolt group:
eccentricity of horizontal component of brace force = 6.11" (15.52 cm)
M = 5.9(6.11) = 36 kip*in (4.07 KNm)

$$\sum d^2 = 2(0)^2 + 2(1.5)^2 = 4.5\,\text{in.}^2\ (29.0\ \text{cm}^2)$$

$$\text{Force to each bolt} = \frac{Mv}{\sum d^2} = \frac{36(1.5")}{4.5} = 12.0\text{k (53.4 KN)}$$

Each bolt must also resist ½ of the shear and axial forces;
1/2V = 8.32/2 = 4.16k (18.5 KN), ½ Axial = 11.25/2 = 5.63k (25.0 KN)

$$\text{Resultant} = \sqrt{4.16^2 + (12 + 5.63)^2} = 18.11\text{k (80.5KN)} < 19.8\ \text{kip / bolt(88.1 KN), OK}$$

The connection passes all limit states. By inspection it is seen that this connection is adequate for all of the beam-to-column connections, including the mezzanine edge beam-to-column connection.

Gusset plates

Single Gusset

– *Weld of brace-to-plate*: The welds are to be designed to have a capacity greater than the tension capacity of the brace. This is not required by the AISC Seismic Provisions for ordinary concentrically braced frames but is used to be conservative since the brace tensile strength is higher than the demand from $\Omega_0 Q_E$.

  Tensile strength of the brace = $R_y F_y A_g$ AISC Seismic Provisions Sec. 14.3.a
  Tensile strength = 1.5(36)(2.23) = 120.4 kips (536 KN)

  Assume E70 welds and ½" (13 mm) thick gusset plate
  Minimum weld size = 3/16" = 0.188" (4.78mm) AISC LRFD Table J2.4
  Maximum weld size = thickness of welded material for materials less than ¼" in thickness; the brace has a wall thickness of 0.216" (5.49mm). Use a weld size of 3/16" (4.78mm)

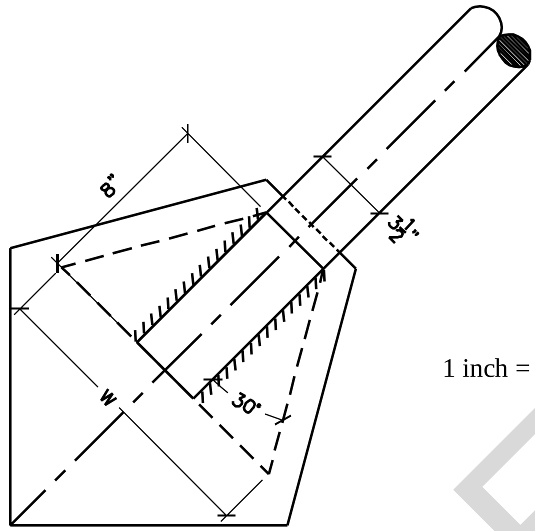
  Design strength of weld:
  $\phi 0.6(F_{EXX}) = 0.75(0.6)(70) = 31.5$ ksi (217 N/mm^2) AISC LRFD Table J2.5
  ϕR_n = (31.5 ksi)(0.707)(3/16")(length) = 4.18 kips / inch (11.34 N/mm^2 per cm. of weld) (controls)

  Design strength of base material (based on pipe)
  $\phi F_{UBM} A_{BM}$ = 0.75(0.6)(58)(0.216")(length) = 5.64 kips / inch (15.31 N/mm^2 per cm. of weld)

  Length = 120.4 kips / (4.18 kips / inch) = 28.8 inch (73.2 cm)
  Use 3/16" (4.78 mm) fillet welds, 8" (20.3 cm) long along each edge of pipe

– *Tension rupture of plate*: The tension rupture strength of the plate is based on Whitmore's area. This area is calculated as the product of the plate thickness times the length W, shown in the sketch as a 30 degree angle offset from the connection line. The tension rupture strength of the plate is designed to exceed the tensile strength of the brace, 120.4 kips.

  W = 2(8"*tan 30) + 3.5" = 12.7" (32.26 cm)
  $\phi_t P_n = \phi t F_u A_e$ = 0.75(58)(12.7")(0.5") = 276 k (1228 KN)> 120.4 k (536 KN) AISC LRFD Eq. D1-2



1 inch = 25.4 mm

– *Block shear rupture strength of plate*:

$\phi R_n = \phi(0.6F_yA_{gv} + F_uA_{nt})$ AISC LRFD Eq. J4-3a

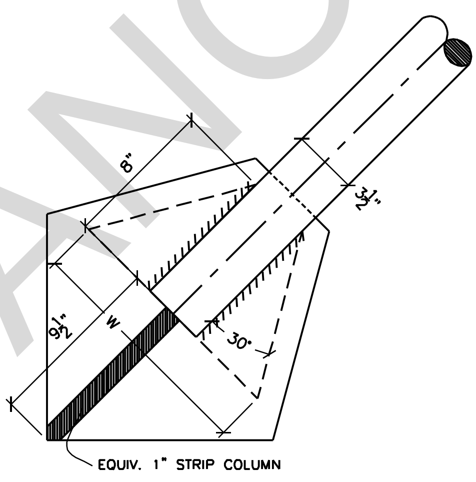
$\phi R_n = 0.75(0.50)[(0.6)(36)(2 \text{ x } 8\text{"}/\cos 30) + (58)(12.7)] = 388$ k (1726 KN) > 120.4 k (536 KN)

$\phi R_n = \phi[0.6F_uA_nv + F_yA_{gt}]$ AISC LRFD Eq. J4-3b

$\phi R_n = 0.75(0.5)\ [(0.6)(58)(2 \text{ x } 8) + (36)(12.7)] = 380$ k (1690 KN) >120.4 k (536 KN)

– *Buckling of plate*:

Buckling capacity of the brace = A_gF_{cr} = (2.23)(11.65) = 25.98 k (116 KN) (buckling strength determined previously)

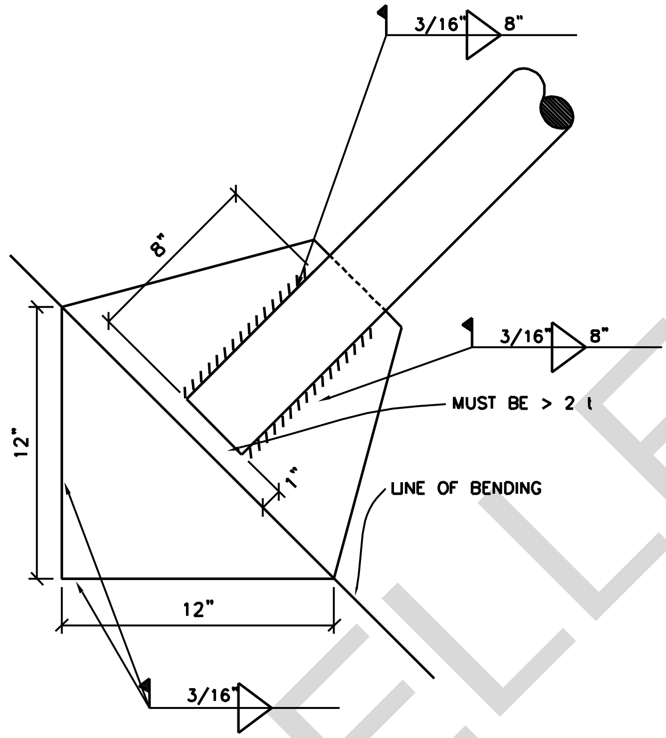


1 inch = 25.4 mm

$$0.90\frac{4000t^3\sqrt{f_y}}{l_c} = 0.90\frac{4000(1/2)^3\sqrt{36}}{9.5} = 284\,\text{k}\ \ (1263\,\text{KN}) > 25.98\ \ (116\ \ \text{KN})$$

– *Out-of-plane strength of plate*: The bracing member can buckle both in and out of plane due to the round section used. For out-of-plane buckling the gusset plate must be able to accommodate the rotation by bending. The brace shall terminate on the gusset a minimum of two times the gusset thickness from the theoretical line of bending which is unrestrained by the column or beam joints.

This ensures that the mode of deformation in the plate will be through plastic hinging rather than torsional fracture.

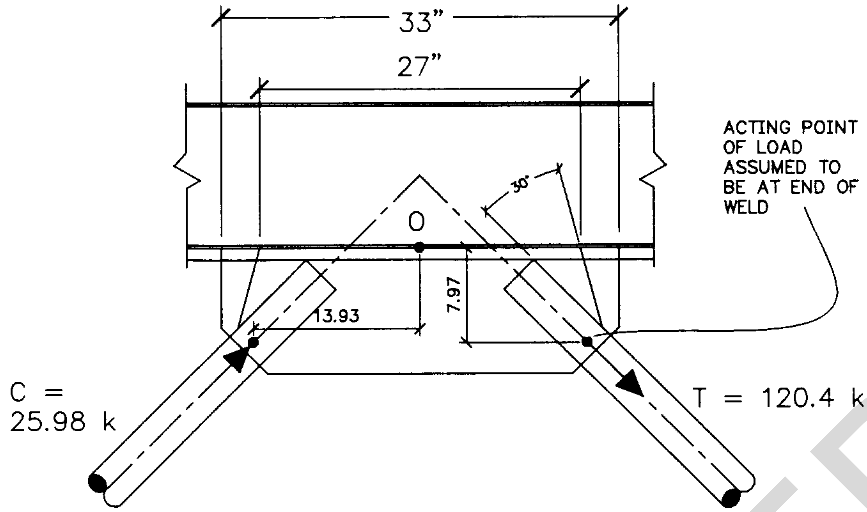


1 inch = 25.4 mm

Double Gusset

Assume that the gusset plate is ½ inch thick. The welds for the brace-to-plate are the same as for the single gusset. The limit states of tension rupture and block shear rupture of the plate have already been checked for the single gusset plate. The buckling capacity of the double gusset is greater than that of the single plate due to the shorter equivalent column length for the double gusset, and therefore, the limit state of plate buckling is satisfied.

– Interaction of shear and moment at plate edge: The interaction of the shear and moment forces is now checked at the plate / beam connection on the Whitmore stress area. The compression and tension forces are assumed to be equal to the brace capacities determined previously (this gives verty conservative results since it is not an equilibrium condition). The forces are assumed to act at the end of the welds of the brace-to-plate connection.



1 kip = 4.448 KN
1 inch = 25.4 mm

Moment at plate-beam edge;
$M_O = (T \cos 45)(7.97) + (C \cos 45)(7.97) - (T \sin 45)(13.93) - (C \sin 45)(13.93) = -617$ kipin (69.7 KNm)
Shear at plate-beam edge;
$V_O = (25.98)(\cos 45) + (120.4)(\cos 45) = 104$ kips (463 KN)
Shear strength of plate = $\phi(0.6)(A_w)(F_y) = (0.9)(0.6)(27)(0.5)(36) = 262$ kips (1165 KN)
Moment strength of plate = $f(Z)(F_y)$
Z of plate = $tW^2/4 = (0.5)(27)^2/4 = 91$ in.^{3} or 1491 cm^3)
Moment strength of plate = (0.9)(91)(36) = 2948 kipin (333 KNm)

Interaction;

$$\left(\frac{M}{M_p}\right)+\left(\frac{V}{V_p}\right) \le 1 = \left(\frac{617}{2952}\right)+\left(\frac{104}{262}\right) = 0.61 < 1, \text{ OK}$$

B.12 *Check allowable drift and PΔ effect*

<u>Drift:</u>

Transverse Direction: The drift is checked for the interior CMU wall E1-E2

Total shear to wall = 10.9 k (48.5 KN)
Rigidity of wall = 2421 kips / inch (424 KN/mm) (determined previously)
δ = 10.9 / 2421 = 0.005" (0.13mm)
Allowable Story Drift, $\Delta_a = 0.010\ h_{sx} = 0.010$ (20') = 0.2' = 1.2" (3.05 cm) (Table 6-1)
Design Drift = $C_d\delta$ = 4(0.005") = 0.02" (0.51mm) < 1.2"(30.5 mm), OK

Longitudinal Direction: The drift of the structure is checked at the interior 20' (6.1m) high section and at the mezzanines (10' sections or 3.05m).

The drift of the 20' building section is check first.

The deflection in the longitudinal direction at the top level of a typical braced frame was determined from computer analysis (not shown) to be = 0.05" (1.27mm).
Allowable Story Drift, $\Delta_a = 0.020\ h_{sx} = 0.020$ (20') = 0.4' = 2.4" (10.2mm) (Table 6-1)
Design Drift = $C_d\delta$ = 4.4(0.05") = 0.23" (5.84mm) < 2.4" (61mm), OK
By inspection, it is seen that the mezzanine drifts will be OK as well.

H-2 Bachelor Enlisted Quarters

a. Introduction

This design example illustrates the seismic design of a three-story Navy bachelor enlisted quarters (BEQ) building. The layout of the building is based on the Navy 1+1 module which allocates approximately 462 sq. ft. ($42.9m^2$) to a two-person living suite as indicated in Figure 1.

(1) Purpose. The purpose of this example is to illustrate the design of a representative military building in an area of high seismicity, using the provisions of FEMA 302 as modified by this document.

(2) Scope. The scope of this example problem includes; the design of all major structural members such as beams, columns, and shear walls. The design of the foundations, nonstructural elements and their connections, and detail design of some structural elements such as reinforced concrete slabs on grade were not considered part of the scope of this problem and are therefore not included. Additionally, this problem considers only seismic and gravity loads.

b. Building description

(1) Function. The function of a BEQ is to provide living quarters for enlisted personnel. The Department of the Navy has various standard modules for living areas. The modules can be arranged together with designated administrative and communal spaces to form the BEQ. In this example, a two-person living suite was chosen and the designated administrative and communal areas were provided on the 1st Floor. The building as indicated in Figure 1 would house 70 enlisted personnel.

(2) Seismic Use Group. Since the building is not described by any of the occupancies in Table 4-1 for special, hazardous, or essential facilities, it will be designated as a standard occupancy structure within Seismic Use Group I.

(3) Configuration. The standard Navy modules may be arranged in any desired configuration. The selected module, as shown in Figure 1, was designated for access from an interior hallway. This typical "motel" type configuration using a double-loaded interior corridor was selected as being the most efficient and economical configuration. A small reception and lounge area by the main stairway was provided at the main entrance on the 1st Floor and an additional stairway was provided at each end of the building.

(4) Structural Systems. The continuous vertical alignment of the transverse walls between the suites makes these walls ideal candidates for bearing and shear walls. Similarly, the need for fenestration at the exterior walls makes the use of longitudinal moment frames a logical choice. Precast cored concrete slabs were chosen for the framed floor system. These commercially available units are capable of spanning between the separation walls without intermediate supports. The soffit of the precast slabs forms the exposed ceiling in the service and sleeping areas. Furred ceilings in the corridors, bath, and storage areas can accommodate the heating and ventilation ducts for each of the modules. Lateral loads are transferred by the reinforced topping through dowels to the shear walls or frames and, in turn, to the foundations.

(5) Choice of materials. The bearing/shear walls could be designed as reinforced masonry rather than cast-in-place concrete. The masonry would be equally functional and in some areas of the U.S. may be more economical. Many alternatives, using cast-in-place or precast configurations, are available for the floor framing. A desirable prerequisite is that the floor be relatively stiff and have low acoustic transmission. The precast slabs with reinforced topping were chosen for the reasons discussed in subparagraph (4) above. All walls not shown on the floor framing plans are intended to be nonstructural and shall be constructed so as to not impair the response of the concrete frames or shear walls.

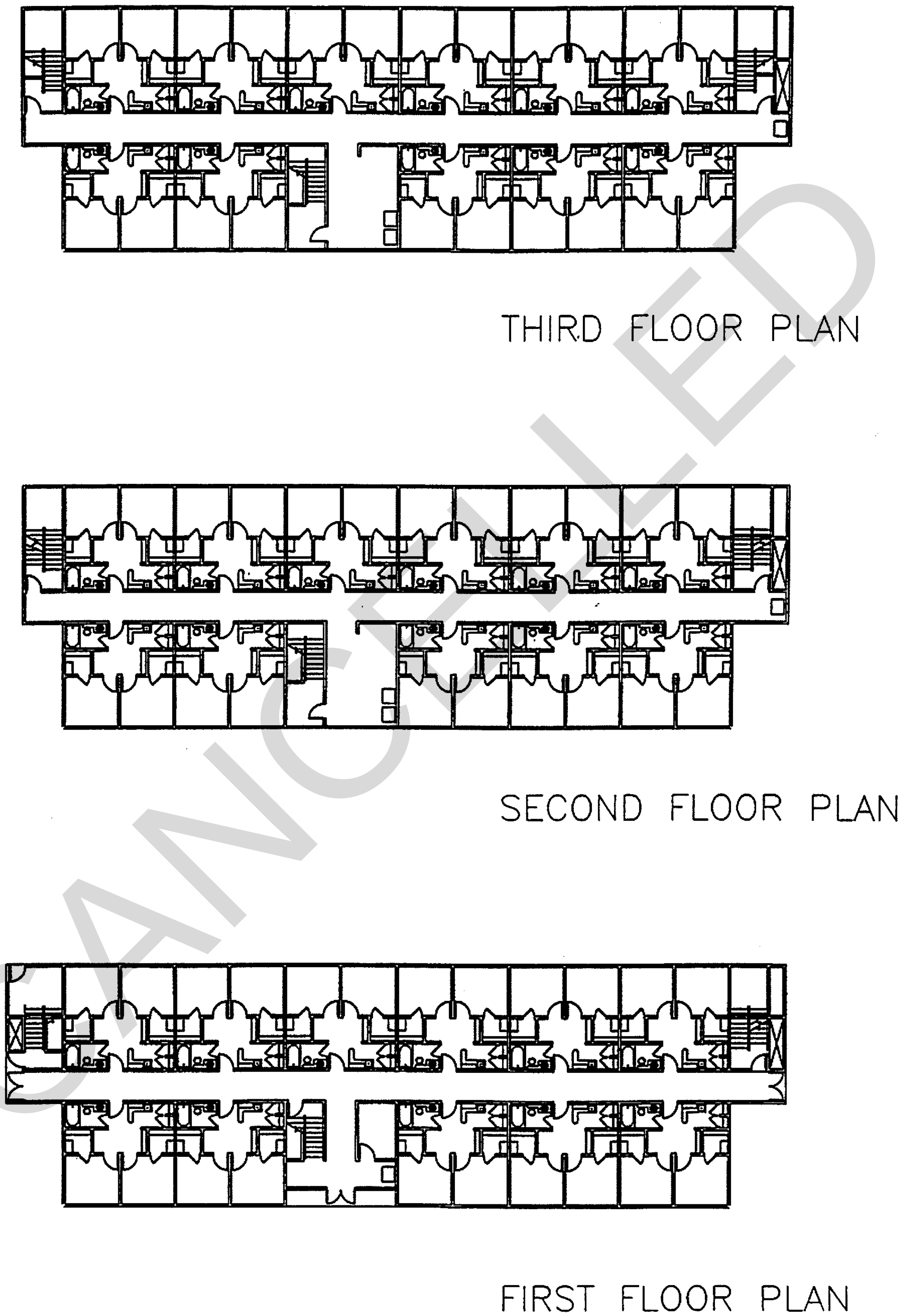
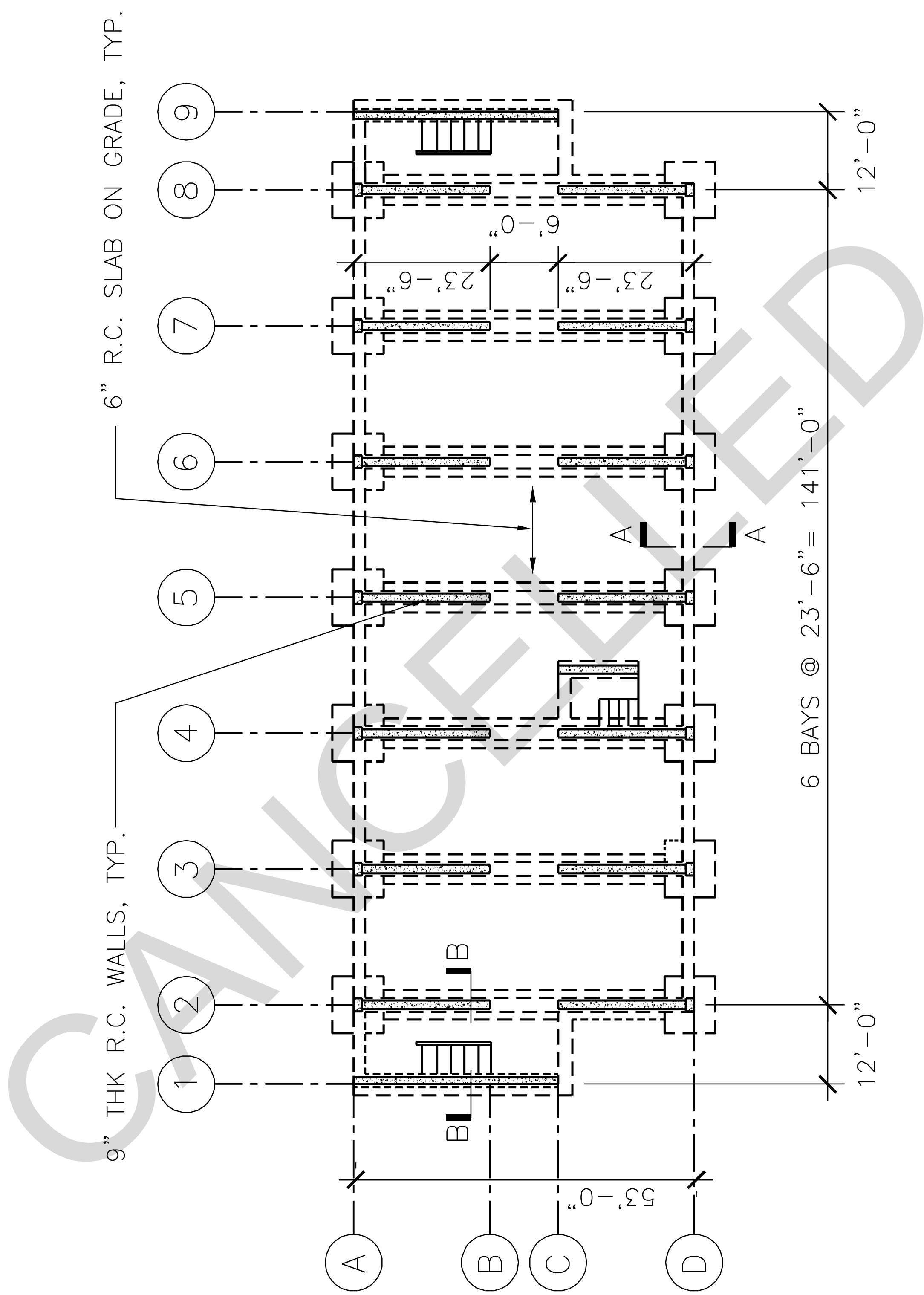
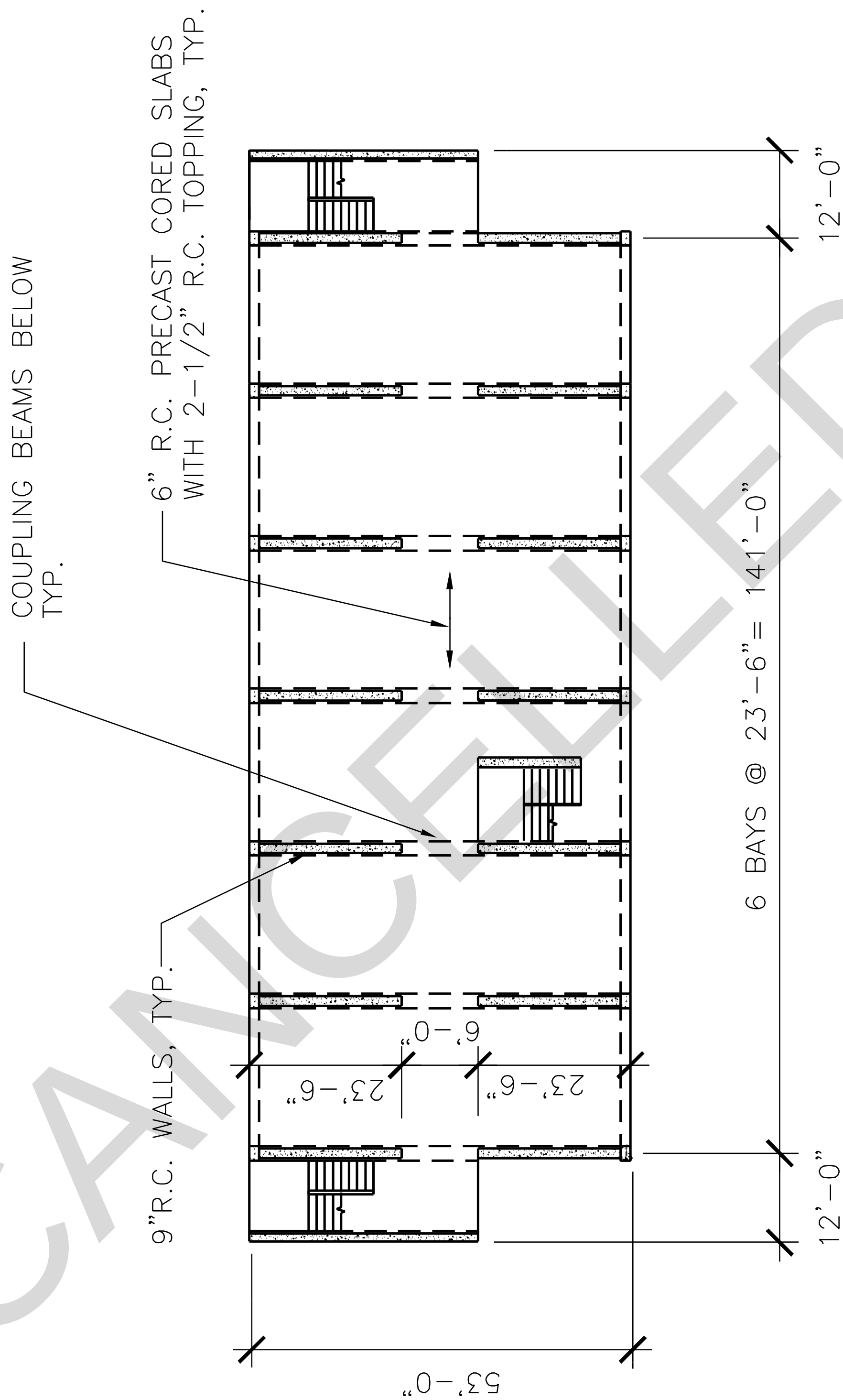


Figure 1. Architectural floor plan



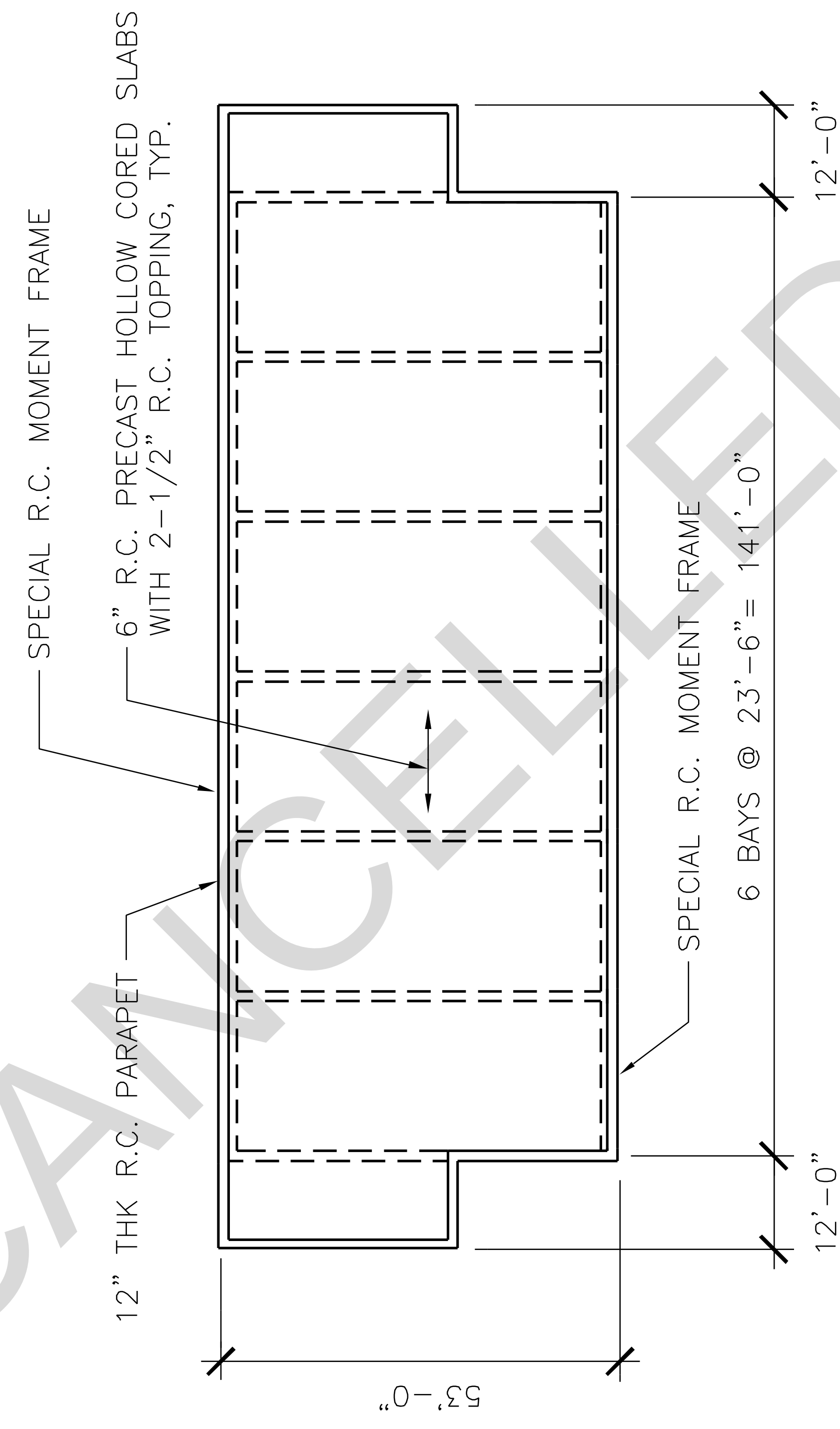
Note: For metric equivalents; 1-in = 25.4mm, 1-ft = 0.30m

Figure 2. Foundation and first floor plan



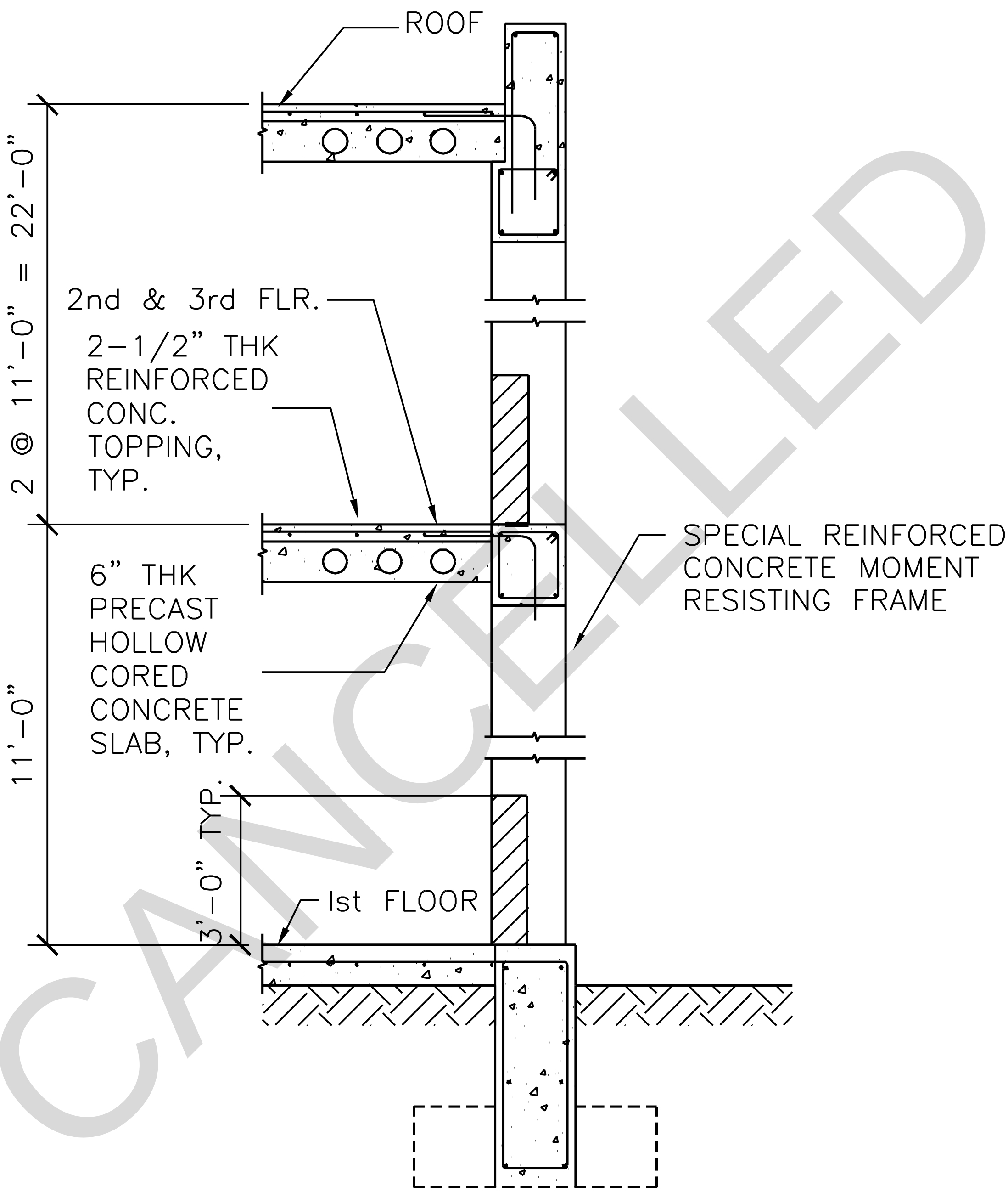
Note: For metric equivalents; 1-in = 25.4mm, 1-ft = 0.30m

Figure 3. Typical floor framing plan



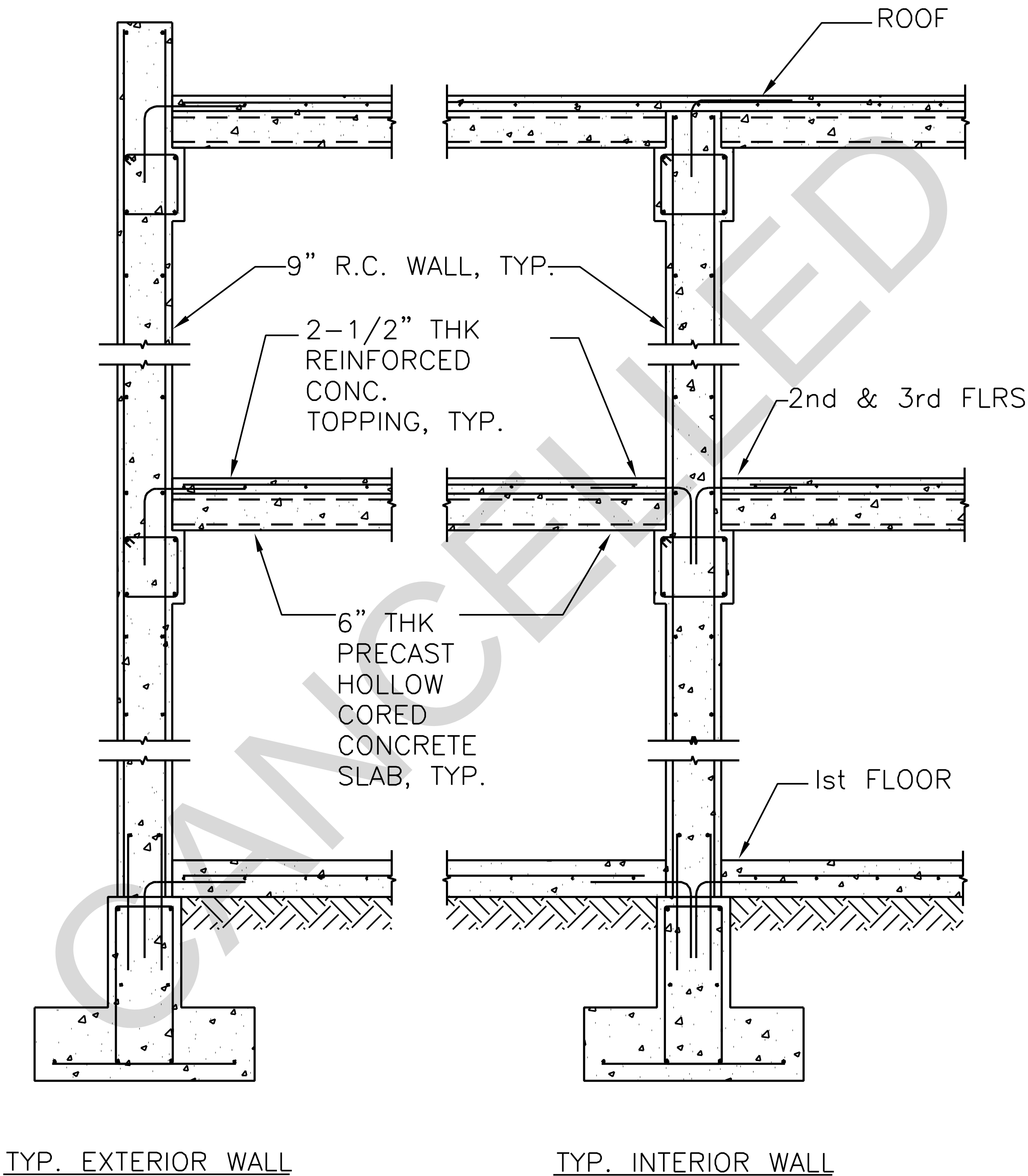
Note: For metric equivalents; 1-in = 25.4mm, 1-ft = 0.30m

Figure 4. Roof framing plan



Note: For metric equivalents; 1-in = 25.4mm, 1-ft = 0.30m

Figure 5. Section A-A



Note: For metric equivalents; 1-in = 25.4mm, 1-ft = 0.30m

Figure 6. Section B-B

c. *Building design (following steps in Table 4-5 for Life Safety).* The design of the building follows steps outlined in Table 4-5 for Performance Objective 1A.

A-1 *Determine appropriate Seismic Use Group.* Per Table 4-3 of the manual, the building is allowed to incur moderate structural damage. Therefore, it is a standard occupancy structure with a Seismic Use Group of I.

A-2 *Determine Site Seismicity*. The site seismicity for this example from the MCE maps is assumed as:
S_s = 1.40g, and S_1 = 0.50g.

A-3 *Determine Site Characteristics.* For the purpose of this problem, a stiff soil condition was assumed corresponding to a site classification of 'Class D' per Table 3-1 of the manual.

A-4 *Determine Site Coefficients, F_a and F_v.* From Tables 3-2a and 3-2b for the given site seismicity and soil characteristics, the site response coefficients were interpolated as follows:

$F_a = 1.00$ (Table 3-2a)
$F_v = 1.50$ (Table 3-2b)

A-5 *Determine adjusted MCE Spectral Response Accelerations:*

$S_{MS} = F_a(S_S) = 1.00(1.40) = 1.40$ (EQ. 3-1)
$S_{M1} = F_v(S_1) = 1.50(0.50) = 0.75$ (EQ. 3-2)

A-6 *Determine Design Spectral Response Accelerations:*

$S_{DS} = (2/3)S_{MS} = (2/3)1.40 = 0.93$ (EQ. 3-3)
$S_{D1} = (2/3)S_{M1} = (2/3)0.75 = 0.50$ (EQ. 3-4)

The approximate period of the structure (based on T = 0.1N, where N = number of stories) is:

$T_{approx} = 0.1N = 0.1(3) = 0.3 < 0.5$ (EQ. 5.3.3.1-2 FEMA 302)

Since $T_{approx} < 0.5$, and because the structure is less than 5 stories in height, Equations 3-5 and 3-6 must be checked in the short period range:

S_{MS} [$1.5F_a$ (EQ. 3-5)
S_{M1} [$0.6F_v$ (EQ. 3-6)

Therefore, $S_{MS} = 1.4 < 1.5(1.00) = 1.5$
$S_{M1} = 0.75 < 0.6(1.50) = 0.9$

Use S_{DS} = 0.93, & S_{D1} = 0.50

A-7 *Determine Seismic Design Category*. With S_{DS} = 0.93, S_{D1} = 0.50, and a Seismic Use Category of I, enter Tables 4-2a and 4-2b to obtain a Seismic Design Category of 'D'.

A-8 *Select Structural System.*

Gravity: Reinforced concrete hollow core slabs to span between transverse bearing walls.

Lateral: As permitted by table 7-1 of the manual;
Transverse direction: Special reinforced concrete shear walls.
Longitudinal direction: Special reinforced concrete moment frames.

A-9 *Select R, V_o, & C_d factors.*

From Table 7-1 of the manual:

Transverse direction: Special reinforced concrete shear walls **$R = 5.5$, $V_o = 2.5$, $C_d = 5.0$**

Longitudinal direction: Special reinforced concrete moment frames **$R = 8$, $V_o = 3.0$, $C_d = 5.5$**

A-10 *Determine preliminary member sizes for gravity load effects.*

(1) Roof and Floor framing

Roof and floor framing to consist of hollow core concrete slabs. Slabs span in the longitudinal direction between bearing walls.

Roof:

Determine Loading:

Live load; w_L = 20 psf (0.96KN/m^2) (per ANSI/ASCE 7-95 Table 4-1)

Dead load; w_D;

Roofing;	5 psf
Insulation;	3 psf
Suspended ceiling;	1 psf
Mechanical & Electrical;	3 psf
Total = w_D =	12 psf (0.57KN/m2)

Therefore, the total superimposed service load = $w_D + w_L$ = 12 psf + 20 psf = 32 psf (1.53KN/m^2). Per the hollow-core concrete slab manufacturers catalog, using a span of 23'-6" (7.17m) and the superimposed service load of 32 psf, choose a 6" (152.4mm) thick x 4'-0" (1.22m) wide hollow-core slab of normal weight concrete (wt. of slab with a 2-1/2" (63.5mm) topping = 80.3 psf or 3.84KN/m^2).

Floor:

Determine design loads:

Live load; w_L = 40 psf (1.92KN/m^2) (per ANSI/ASCE 7-95 Table 4-1)

Dead load; w_D;

Floor finish;	1 psf
Partitions;	10 psf
Suspended ceiling;	1 psf
Mechanical & Electrical;	3 psf
Total = w_D =	15 psf (0.72KN/m^2)

Therefore, the total superimposed service load = $w_D + w_L$ = 15 psf + 40 psf = 55 psf (2.63KN/m^2). Per the hollow-core concrete slab manufacturers catalog using a span of 23'-6", and the superimposed service load of 55 psf, choose a 6" thick x 4'-0" wide hollow-core slab of normal weight concrete (wt. of slab with a 2-1/2" topping = 80.3 psf or 3.84KN/m^2).

(2) Transverse bearing wall design

The transverse bearing walls run almost continuously through the building being interrupted only by the 6-ft. wide interior corridor. The thickness of the walls is determined based on their dual function as both shear and bearing walls; the building is located in a high seismic zone and two curtains of reinforcement are anticipated. Therefore, a wall thickness of 9-in (228.6mm). is chosen in order to accommodate the placement of two curtains of reinforcement. For gravity loads, the empirical design method of ACI 318-95 Section 14.5 will be used to check this 9-in. thickness for bearing. The empirical design method is determined to be applicable because the resultant of all factored loads is located within the middle third of the wall. Additionally, it is anticipated that reinforcement requirements will be governed by seismic considerations, and therefore, the design of the wall reinforcement will be addressed during the lateral load design.

Determine design loads;

Note: Loading will be determined per unit of wall length using a tributary width per floor or roof of 23'-6" (7.17m).

Live load;

Floor live load reduction per ANSI/ASCE 7-95;
$A_T = 23.5' \times 53' = 1{,}246\text{-ft}^2 > 400\text{-ft}^2$ $(115.8\text{-m}^2 > 37.2\text{m}^2)$

$$L = L_o\left(0.25+\frac{15}{\sqrt{A_T}}\right) > 0.4L_o \qquad \text{(EQ. 4-1 ANSI/ASCE 7-95)}$$

$$L = 40\text{psf}\left(0.25+\frac{15}{\sqrt{1{,}246-\text{ft}^2}}\right) = 27\text{psf} > 16\text{psf} = 0.4(40\text{psf}) \quad (1.29\text{KN/m}^2 > 0.77\text{KN/m}^2)$$

Therefore, $w_{FL} = 27\text{psf}\,(1.29\text{KN/m}^2)$, $w_{RL} = 20\text{psf}\,(0.96\text{KN/m}^2)$
The total roof live load per foot of wall length is;
$P_{RL} = 20\text{psf}(23.5') = 470\text{plf}$ (6.85KN/m)
The total floor live load per foot of wall length is;
$P_{FL} = 2\text{-floors} \times 27\text{psf}(23.5') = 1{,}269\text{plf}$ (18.51KN/m)

Dead load;
From the design of the roof and floor framing, the roof and floor dead loads are;
$w_{FD} = 80.3\text{psf} + 15\text{psf} = 95.3\text{psf}$ (4.56KN/m^2), $w_{RD} = 80.3\text{psf} + 12\text{psf} = 92.3\text{psf}$ (4.42KN/m^2)
Dead load due to the wall self weight at the first floor is;
$w_{WD} = 9''(1'/12'')150\text{pcf} \times 33'$(bldg. Height) $= 3{,}713\text{plf}$ (54.15KN/m)
The total dead load per foot of wall length (at the first story) is;
$P_{DL} = 95.3\text{psf}(2)23.5' + 92.3\text{psf}(1)23.5' + 3{,}713\text{plf} = 10{,}361\text{plf}$ (157.10KN/m)

Factored load combinations (per ANSI/ASCE 7-95);

Load case 1: $U = 1.4D$
$P_u = 1.4(10{,}361\text{plf}) = 14{,}505\text{plf}$ (211.53KN/m)

Load case 2: $U = 1.2D+1.6L+0.5L_r$
$P_u = 1.2(10{,}361\text{plf})+1.6(1{,}269\text{plf})+0.5(470\text{plf}) = 14{,}700\text{plf}$ (governs)
(214.38KN/m)

Load case 3: $U = 1.2D+0.5L+1.6L_r$
$Pu = 1.2(10{,}361\text{plf})+0.5(1{,}269\text{plf})+1.6(470\text{plf}) = 13{,}820\text{plf}$ (201.55KN/m)

Check minimum wall thickness requirements per ACI 318-95 Section 14.5.3;

$h = 9'' > 4''$ **O.K.**

$$\frac{l_c}{25} = \frac{11'(12''/1')}{25} = 5.28'' < 9''(134.1\text{mm} < 228.6\text{mm}) = h \qquad \textbf{O.K.}$$

Check strength per ACI 318-95 Section 14.5.2;

$$\phi P_{nw} = 0.55\phi f_c' A_g\left[1-\left(\frac{kl_c}{32h}\right)^2\right] \qquad \text{(EQ. 14-1 ACI 318-95)}$$

where; $\phi = 0.7$

$k = 1.0$ For ends are unrestrained against rotation at each floor level, but are braced against lateral translation by the orthogonal moment frames.

$f_c' = 4{,}000\text{psi}$ (27.58MPa)

$A_g = 9''(12''/1') = 108\text{-in}^2/\text{ft}$ $(228.4\text{X}10^3\ \text{mm}^2/\text{m})$

$$\phi P_{nw} = 0.55(0.70)4{,}000\text{psi}(108-\text{in}^2/\text{ft})\left[1-\left(\frac{1.0(11'(12''/1'))}{32(9'')}\right)^2\right] = 131.4\text{klf} \quad (1.92\text{MN/m})$$

$P_u = 14.7\text{klf} < 131.4\text{klf} = \phi P_{nw}$ $(0.21\text{MN/m} < 1.92\text{MN/m})$ **O.K.**

Choose a wall thickness h = 9" (228.6mm)

(3) Corbel design

The precast hollow-core concrete slabs span between transverse walls and are supported at these walls by rectangular corbels. Corbels are 4-in (101.6mm) and 12-in. (304.8mm) deep. One design for the worst case loading will be used throughout the building.

Determine if provisions of ACI 318-95 Section 11.9.1 are satisfied;

Shear span is chosen as a = 2-in. Therefore; $\frac{a}{d} = \frac{2}{12} = 0.17 < 1.0$

Also, by inspection, $N_{uc} < V_u$ **O.K.**

Determine if provisions of ACI 318-95 Section 11.9.2 are satisfied;

Depth outside edge of bearing area = d > 0.5d **O.K.**

Determine design loads;

Note: Worst case occurs at an interior span at either the second or third floors.

Live load; $w_L = 40\text{psf}$ (1.92KN/m^2)

Note: Corbels are conservatively designed without a live load reduction.

Dead load; $w_D = 95.3\text{psf}$ (4.56KN/m^2) (calculated previously for the transverse bearing wall design)

Factored load combinations (per ANSI/ASCE 7-95);

By inspection, load case 2 governs;

$$V_u = \frac{1}{2} w_u L = \frac{1}{2}(1.2(95.3\text{psf}) + 1.6(40\text{psf}))23.5'\left(\frac{\text{kip}}{1000-\text{lb}}\right) = 2.1\text{klf} \quad (30.63\text{KN/m})$$

By inspection, the tensile loads on the support are negligible. Therefore, the requirements of ACI 318-95 Section 11.9.3.4 apply;

$N_{uc} = 0.2V_u = 0.2(2.1\text{klf}) = 0.42\text{klf}$ (6.13KN/m)

The moment at the face of support is calculated from the requirements of ACI 318-95 Section 11.9.3;

$$M_u = [V_u a + N_{uc}(h-d)]$$

where; $d = 12" - 0.75" - \frac{0.50"}{2} = 11"$ (279.4mm) (assuming #4 (10M) bars for reinforcement)

a = half of the bearing width = 2" (50.8mm)

$$\therefore M_u = 2.1\text{klf}(2")(1'/12") + 0.42\text{klf}(12" - 11")(1'/12") = 0.39\frac{\text{ft}-\text{kips}}{\text{ft}} \quad (1.73\text{KN-m/m})$$

Design for shear;

$$\phi V_n \geq V_u \qquad \text{(EQ. 11-1 ACI 318-95)}$$

where; $\phi = 0.85$ per ACI 318-95 Section 11.9.3.1

$$V_n = A_{vf} f_y \mu \qquad \text{(EQ. 11-25 ACI 318-95)}$$

where; $\mu = 1.4(\lambda) = 1.4(1.0) = 1.4$

$$\therefore A_{vf} = \frac{V_u}{\phi f_y \mu} = \frac{2.1\text{klf}}{0.85(60\text{ksi})1.4} = 0.029\frac{\text{in}^2}{\text{ft}} \quad (61.3\text{mm}^2/\text{m})$$

Try 3 #3 (~10M) crossties spaced at 3-in. (76.2mm) o.c. ($A_{vf} = 0.33\text{-in}^2$ or $0.21\text{X}10^3\ \text{mm}^2$);

$\phi V_n = 0.85(27.7\text{klf}) = 23.6\text{klf} > 2.1\text{klf}(344.17\text{KN/m} > 30.63\text{KN/m}) = V_u$ **O.K.**

Check that $V_n < 0.2f'_c b_w d$ or $800 b_w d$ in accordance with ACI 318-95 Section 11.9.3.2.1;

$0.2f'_c b_w d = 0.2(4\text{ksi})(12"/\text{ft})11" = 106\text{klf} > 27.7\text{klf}(1.55\text{MN/m} > 0.40\text{MN/m}) = V_n$ **O.K.**

$800 b_w d = 800(12"/\text{ft})11" = 106\text{klf} > 27.7\text{klf} = V_n$ **O.K.**

Therefore use 3 #3 (~10M) bar crossties at 3-in. (76.2mm) o.c. vertical and spaced at 12-in. (0.30mm) o.c. horizontal

Determine reinforcement requirements for flexure 'A_f';

Per ACI 318-95 Section 11.9.3.3;

$$M_u \le \phi M_n = \phi A_f f_y\left(d - \frac{a}{2}\right) \qquad \text{where;} \quad a = \frac{A_f f_y}{0.85 f_c^{'} b}$$

Solve for A_f per foot of wall length;

$$\therefore M_u \le \phi A_f f_y\left(d - \frac{1}{2}\left(\frac{A_f f_y}{0.85 f_c^{'} b}\right)\right)$$

$$\Rightarrow \left[\frac{\phi(0.59)(f_y)^2}{f_c^{'} b}\right]A_f^2 - \left[\phi f_y d\right]A_f + M_u = 0$$

The terms of the quadratic equation are evaluated as follows;

$$\left[\frac{\phi(0.59)(f_y)^2}{f_c^{'} b}\right] = \frac{0.85(0.59)(60\text{ksi})^2}{4\text{ksi}(12")} = 37.6 - \text{kips}/\text{in}^2 \quad (0.26\text{KN/mm}^2)$$

$$\left[\phi f_y d\right] = 0.85(60\text{ksi})11" = 561 - \text{kips}/\text{in} \quad (98.2\text{KN/mm})$$

$$M_u = 0.39^k(12"/1') = 4.68^{\text{in-kips}} \quad (0.53\text{KN-m})$$

The roots of the quadratic equation are;

$$A_f \ge \frac{561 - \text{kips}/\text{in} \pm \sqrt{(561 - \text{kips}/\text{in})^2 - 4(37.6 - \text{kips}/\text{in}^2)4.68^{\text{in-kips}}}}{2(37.6 - \text{kips}/\text{in}^2)} = 0.0083 - \text{in}^2 \quad (5.35\text{mm}^2)$$

Note: The larger root is extraneous and therefore rejected

Therefore, $A_f \geqq 0.0083\text{-in}^2/\text{ft}$ (17.5mm²/m)

Determine reinforcement requirements for tension A_n;
Per ACI 318-95 section 11.9.3.4;

$$N_{uc} \le \phi A_n f_y$$

$$A_n \ge \frac{N_{uc}}{\phi f_y} = \frac{0.42\text{klf}}{0.85(60\text{ksi})} = 0.0082 - \text{in}^2 \quad (5.29\text{mm}^2)$$

Therefore, $A_n \geqq 0.0082\text{-in}^2/\text{ft}$ (17.3mm²/m)

Determine area of primary tension reinforcement A_s;
Per ACI 318-95 Section 11.9.3.5;

$$A_f + A_n = 0.0083 - \text{in}^2/\text{ft} + 0.0082 - \text{in}^2/\text{ft} = 0.017 - \text{in}^2/\text{ft} \quad (35.9\text{mm}^2/\text{m})$$

$$\frac{2}{3}A_{vf} + A_n = \frac{2}{3}(3(0.11 - \text{in}^2/\text{ft})) + 0.0082 - \text{in}^2/\text{ft} = 0.228 - \text{in}^2/\text{ft} \quad (482.2\text{mm}^2/\text{m}) \text{ (governs)}$$

Try 1 #4 (~10M) bar at 6" o.c.;

$$A_s = 2\left(\frac{0.20 - \text{in}^2}{\text{ft}}\right) = 0.40 - \text{in}^2/\text{ft} > 0.228 - \text{in}^2/\text{ft} \quad (845.9\text{mm}^2/\text{m} > 482.2\text{mm}^2/2)$$ **O.K.**

Check minimum reinforcement requirements per ACI 318-95 Section 11.9.5;

$$\rho = \frac{A_s}{bd} \ge 0.04\left(\frac{f_c^{'}}{f_y}\right)$$

Per foot of wall length;

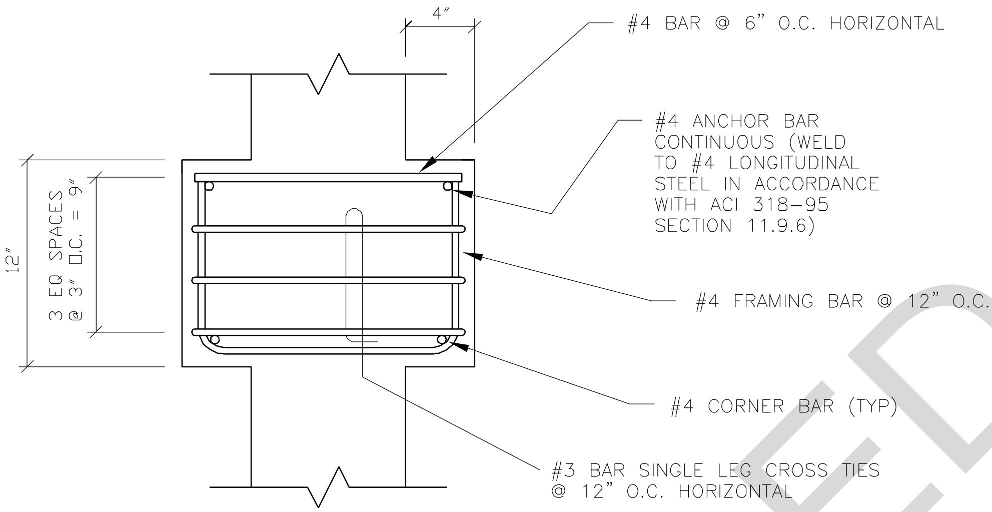
$$\rho = \frac{0.40 - \text{in}^2}{12"(11")} = 0.003 > 0.0027 = 0.04\left(\frac{4\text{ksi}}{60\text{ksi}}\right) = 0.04\left(\frac{f_c^{'}}{f_y}\right)$$ **O.K.**

Determine if additional ties are required per ACI 318-95 Section 11.9.4;

A_n provided within (2/3)d = 2(0.11-in²) = 0.22-in² (141.9mm²)
$(A_n)_{req'd} \geqq 0.5(A_s\text{-}A_n) = 0.5(0.40\text{-in}^2\text{-}0.0082\text{-in}^2) = 0.196\text{-in}^2$ (126.4mm²)
$(A_n)_{provided} = 0.22\text{-in}^2 > 0.196\text{-in}^2 = (A_n)_{req'd}$ (141.9mm² > 126.4mm²) **O.K.**

Therefore, use 1 #4 (~10M) bar at 6-in. (0.15m) on center for primary reinforcement

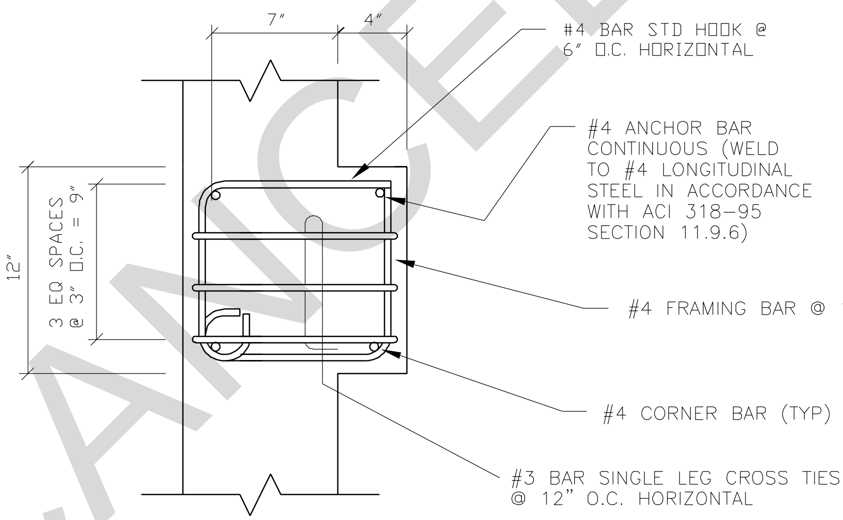
Therefore, the corbel design is as follows;



1-in = 25.4mm
#3 bar ~ 10M
#4 bar ~ 10M

At the end walls A_s must be anchored within the wall. A hoop arrangement as shown below will be used for this purpose.

The corbel design at the end walls is as follows;



See above for metric equivalents

(4) Transverse beam design

Transverse beams span over the interior 6-ft. (1.83m) wide corridor and support the floor and roof framing at these locations. The transverse beams are formed as a continuation of the corbel between the shear walls. One design for the worst case loading will be used throughout the building.

Determine design loads;
Note: Loading will be determined per unit length of the beam. Also, the worst case occurs at an interior span at either the second or third floor levels.

Tributary width per floor = 23'-6" (7.17m) (worst case conditon)

Live load;

Note: A_T = 23.5' x 6' =141-ft^2 < 400-ft^2 (13.1m^2 < 37.16m^2). Therefore, no live load reduction is allowed.

$\therefore w_{FL} = 40\text{psf} \quad (1.92\text{KN/m}^2)$

Dead load;

$w_{FD} = 95.3\text{psf} \quad (4.56\text{KPa})$ (as calculated previously)

By inspection, load case 2 (U=1.2D+1.6L+0.5L_r) governs;

$$w_u = [1.2(95.3\text{psf}) + 1.6(40\text{psf})]23.5' = 4.2\text{klf} \quad (61.25\text{KN/m})$$

Note: Beam has a fixed-fixed condition with the maximum positive moment occuring at mid span and the maximum negative moment occuring at its ends. The maximum shear also occurs at the member ends.

Design for flexure;

Positive moment at mid span;

$$(M_u^+)_{@\text{mid-span}} = \frac{w_u L^2}{24} = \frac{4.2\text{klf}(6')^2}{24}(12''/1') = 75.6^{\text{in-k}} \quad (8.54\text{KN-m})$$

Assume j = 0.9, and d = 12"-0.75"-0.375"-0.625"/2 = 10.6" (269.2mm) (assuming #5 (15M) bars for longitudinal steel, and #3 (~10M) bars for ties)

$$(A_s)_{\text{trial}} = \frac{M_u}{\phi f_y jd} = \frac{75.6^{\text{in-k}}}{0.9(60\text{ksi})0.9(10.6'')} = 0.15-\text{in}^2 \quad (96.8\text{mm}^2)$$

Try 2-#5 bars, A_s = 2(0.31-in^2) = 0.62-in^2 (399.9mm^2)

Check minimum reinforcement per ACI 318-95 Sections 10.5, and 21.3.2.1;

$$A_{s.min} = \frac{3\sqrt{f_c'}}{f_y} b_w d \geq \frac{200 b_w d}{f_y} \qquad \text{(EQ. 10-3 ACI 318-95)}$$

$$\therefore \rho_{min} = \frac{A_{s,min}}{b_w d} = \frac{3\sqrt{f_c'}}{f_y} = \frac{3\sqrt{4{,}000\text{psi}}}{60{,}000\text{psi}} = 0.00316$$

$$\text{or } \rho_{min} = \frac{A_{s,min}}{b_w d} = \frac{200}{f_y} = \frac{200}{60{,}000\text{psi}} = 0.0033 \qquad \text{(governs)}$$

$$\rho = \frac{0.62-\text{in}^2}{17''(10.6'')} = 0.0034 > \rho_{min} = 0.0033$$ **O.K.**

Check crack control of flexural reinforcement per ACI 318-95 Section 10.6;

$$z = f_s\sqrt[3]{d_c A} \leq 175-\text{kips/in} \quad \text{(for interior exposure)} \qquad \text{(EQ. 10-5 ACI 318-95)}$$

where; f_s = 0.60(60ksi) = 36ksi (248.2MPa)

d_c = 12"-10.6" = 1.4" (35.6mm)

A = 17"(2)(1.4")/2 = 23.8-in^2 (15.35X10^3 mm^2)

$$\therefore z = 36\text{ksi}\sqrt[3]{1.4''(23.8-\text{in}^2)} = 116-\text{kips/in} < 175-\text{kips/in} \quad (30.65\text{KN/mm})$$ **O.K.**

Note: There is no need to check the upper limit on the amount of reinforcement (i.e., 0.75ρ_b) because the design is governed by the minimum amount of reinforcement.

Check capacity;

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right), \qquad \text{where;} \quad a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{0.62-\text{in}^2(60\text{ksi})}{0.85(4\text{ksi})17''} = 0.64'' \quad (16.3\text{mm})$$

$$\phi M_n = 0.9(0.62-\text{in}^2)60\text{ksi}\left(10.6'' - \frac{0.64''}{2}\right) = 344^{\text{in-kip}} \quad (38.87\text{KN-m})$$

$$\phi M_n = 344^{\text{in-kips}} > 75.6^{\text{in-kips}}(38.8\text{KN-m} > 8.54\text{KN-m}) = M_u^+$$ **O.K.**

Therefore, use 2 #5 (15M) bottom bars

Negative moment at member ends;

$$(M_u^-)_{@ends} = \frac{w_u L^2}{12} = 144^{in-kips} \quad (16.27KN\text{-}m)$$

By inspection, 2 #5 (15M) bars are required;

Use 2 #5 (15M) top bars

Design for shear;

Determine the strength reduction factor 'ϕ' in accordance with ACI 318-95 Section 9.3.4;

Nominal shear strength = $\phi V_n = V_u$

$$V_u = \frac{w_u L}{2} = \frac{4.2klf(6')}{2} = 12.6^k \quad (56.0KN)$$

The shear corresponding to the development of the nominal flexural strength of the beam is;

$$V_e = \frac{M_{pr1} + M_{pr2}}{L} \pm \frac{W}{2}$$

where; $L = 6'$ (1.83m) (clear span between shear walls)

$M_{pr1} = M_{pr2}$, and is calculated using $f_s = 1.25f_y$, and $\phi = 1.0$ as follows;

$$M_{pr} = 1.25A_s f_y\left(d - \frac{a}{2}\right) \quad \text{where;} \quad a = \frac{1.25A_s f_y}{0.85f_c'b}$$

$$a = \frac{1.25(0.62 - in^2)60ksi}{0.85(4ksi)17"} = 0.80" \quad (20.3mm)$$

$$\therefore M_{pr} = 1.25(0.62 - in^2)60ksi\left(10.6" - \frac{0.80"}{2}\right) = 474^{in-kips} \quad (53.56KN\text{-}m)$$

W = factored gravity load along span = w_u = 4.2klf (61.3KN/m)

$\therefore$ $W = Lw_u = 6'(4.2klf) = 25.2^k$ (112.1KN)

$$\therefore V_e = \frac{2(474^{in-kips})(1'/12")}{6'} + \frac{25.2^k}{2} = 25.8^k \quad (114.8KN)$$

$$V_u = 12^k < 25.8^k (53.4KN < 114.8KN) = V_e$$

Therefore, ϕ = 0.6

Note: This design considers gravity loads only and will be reevaluated in the lateral load design (step B-11 of Table 4-5).

Determine if only minimum shear reinforcement is required per ACI 318-95 Section 11.5.6.1;

$$V_c = 2\sqrt{f_c'}b_w d = 2\sqrt{4000psi}\,17"(11")(1 - k/1000 - lb) = 23.7^k \quad (105.4KN)(\text{EQ. 11-1 ACI 318-95})$$

$$V_u = 12.6^k < 14.2^k (53.4KN < 63.2KN) = 0.6(23.7^k) = \phi V_c$$

Therefore, provide only minimum shear reinforcement per ACI 318-95 Section 11.5.5.3;

$$A_v = 50\frac{b_w s}{f_y} \qquad \text{(EQ. 11-13 ACI 318-95)}$$

Determine spacing for #3 (~10M) hoops;

$$s = \frac{A_v f_y}{50b_w} = \frac{2(0.11 - in^2)60{,}000psi}{50(17")} = 15.5" \quad (393.7mm)$$

or $s = \frac{d}{2} = \frac{11"}{2} = 5.5"$ (139.7mm) (governs) (Per ACI 318-95 Section 11.5.4.1)

or $s = 24"$ (609.7mm)

Therefore, use #3 (~10M) hoops at 5" (127.0mm) o.c. over full length of beam

d. Equivalent Lateral Force Procedure.

B-1 *Calculate Fundamental Period, T:*

$$T_a = C_t h_n^{3/4} \qquad \text{(EQ. 5.3.3.1-1 FEMA 302)}$$

Transverse direction;	$C_t = 0.020$	Shear wall system
Longitudinal direction;	$C_t = 0.030$	Reinforced concrete moment frame Resisting 100% of seismic forces
	h_n = 33-ft (10.06m)	Height to highest level

Therefore; $T_a = 0.020(33')^{3/4} = 0.28\,\text{sec}$ transverse

$T_a = 0.030(33')^{3/4} = 0.41\,\text{sec}$ longitudinal

B-2 *Determine Dead Load 'W':*

Note: For determining the base shear, the proportions of the moment frame elements are initially guessed. This is judged to provide adequate results because most of the weight of the building is due to the shear walls and the pre-cast concrete floor and roof framing thereby making the moment frames a relatively small percentage of the total building weight. The reproportioning of the moment frame elements is judged to have negligible effect on the base shear. The beams are proportioned first using a rule of thumb of one inch (25.4mm) of depth for every foot (0.305m) of span with the width being conservatively taken as 3/4 of the depth. The columns are then proportioned to have the same dimensions as the beams. The columns are proportioned as such because, by inspection, they support very little axial load and function primarily in flexure with a loading similar to that of the beams (pre-cast concrete planks spanning between bearing walls provide the primary gravity support, and the concrete frames are left to support only their own self weight).

Try a 24" (609.6mm) deep x 18" (457.2mm) wide floor beam and column, and 16" (406.4mm) deep x 18" (457.2mm) wide roof beam;

Check proportion requirements of ACI 318-95 Section 21.3, and 21.4;

Floor & Roof Beams;

Clear span = 21.5'>4(24")(1'/12") = 8' (2.44m)floor or >4(16")(1'/12") = 5.33' (1.63m)roof **O.K.**

$\frac{b}{h} = \frac{18''}{24''} = 0.75 > 0.3$ (floors) $\frac{b}{h} = \frac{18''}{16''} = 1.13 > 0.3$ (roof) **O.K.**

$b = 18'' > 10''$ (floors) $b = 18'' > 10''$ (roof) **O.K.**

(457.2mm > 254.0mm) (457.2mm > 254.0mm)

$b = 18'' \leq W + \frac{3}{2}h = 54''$ (floors) $b = 18'' \leq W + \frac{3}{2}h = 54''$ (roof) **O.K.**

(1371.6mm) (1371.6mm)

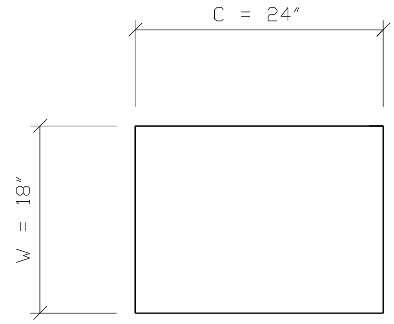
Column;

$\frac{W}{C} = \frac{18''}{24''} = 0.75 > 0.4$ **O.K.**

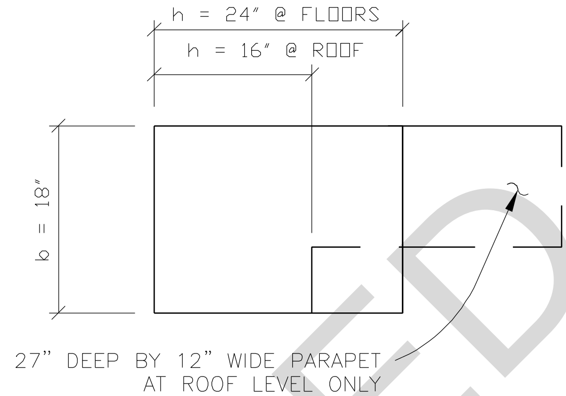
$W = 18'' > 12''$ (457.2mm > 304.8mm) **O.K.**

Therefore, the moment frame cross sections are as follows;

Columns:



Beams:



1-in = 25.4mm

Building weights are calculated on spread sheet and are shown in figures 3, and 4.
Total seismic weight is;

$$W = 4{,}428^k \ \ (19.70\text{MN})$$

B-3 *Calculate Base Shear, V:*

$$V = C_s W \qquad \text{(EQ. 5.3.2 FEMA 302)}$$

where;

$$C_s = \frac{S_{DS}}{R} \qquad \text{(EQ. 3-7)}$$

and,

$$C_s < \frac{S_{D1}}{TR} \qquad \text{(EQ. 3-8)}$$

$$C_s > 0.044 S_{DS} \qquad \text{(EQ. 3-9)}$$

Transverse direction;

$$C_{s,trans} = \frac{S_{DS}}{R} = \frac{0.93}{5.5} = 0.17$$

$$C_{s,trans} = 0.17 < 0.33 = \frac{0.50}{0.28(5.5)} = \frac{S_{D1}}{TR}$$

$$C_{s,trans} = 0.17 > 0.041 = 0.044(0.93) = 0.044 S_{DS}$$

Therefore, $V_{trans} = C_s W = 0.17(4{,}428^k) = 753^k$ (3.35MN)

Longitudinal direction;

$$C_{s,long} = \frac{S_{DS}}{R} = \frac{0.93}{8} = 0.12$$

$$C_{s,long} = 0.12 < 0.15 = \frac{0.50}{0.41(8)} = \frac{S_{D1}}{TR}$$

$$C_{s,long} = 0.12 > 0.041 = 0.044(0.93) = 0.044 S_{DS}$$

Therefore, $V_{long} = C_s W = 0.12(4{,}428^k) = 531^k$ (2.36MN)

$V_{trans} = 753^k$ (3.35MN), $V_{long} = 531^k$ (2.36MN)

ASSEMBLY WEIGHTS (PSF):

DIAPHRAGMS:

<u>Level - Roof</u>

Built-Up roofing;		5.00		
2" rigid insulation @ 1.5psf/in., 1.5x2 =		3.00		
6" Thick hollow core concrete slab;		49.00		
2-1/2" Concrete topping; 2.5"(1'/12")150pcf =		31.30		
Suspended ceiling;		1.00		
Mech., Elec., & Miscellaneous		3.00		
	Total =	92.3	psf	(4.42KN/m^2)

<u>Level - Floors</u>

Floor finish;		1.00		
Partitions (10psf per FEMA 310 section 3.5.2.1);		10.00		
6" Thick hollow core concrete slab;		49.00		
2-1/2" Concrete topping; 2.5"(1'/12")150pcf =		31.30		
Suspended ceiling;		1.00		
Mech., Elec., & Miscellaneous		3.00		
	Total =	95.3	psf	(4.56KN/m^2)

TRANSVERSE WALLS:

9" Thick reinforced concrete shear walls; 9"(1'/12")150pcf = 112.50 psf (5.39KN/m^2)

LONGITUDINAL WALLS:

Infill;				
6" Thick medium weight concrete block wall grouted at 48" o.c.;		40.00	psf	
Windows;				
Glass, frame and sash;		8.00	psf	
Columns;				
18" wide by 24" deep columns (use 23.5' spacing); 18"(24")(1-ft^2/144-in^2)(150pcf)(1/23.5') =		19.15	psf	
Spandral Beams at floors;				
24" deep by 18" wide spandrel beams (use 11' tributary height); 24"(18")(1-ft^2/144-in^2)(150pcf)(1/11') =		40.91	psf	
Spandral Beams and parapet at roof;				
16" deep by 18" wide spandrel beams (use 5.5' tributary height); 16"(18")(1-ft^2/144-in^2)(150pcf)(1/5.5') =		54.55		
27" deep by 12" wide parapet; 27"(12")(1-ft^2/144-in^2)(150pcf)(1/5.5') =		61.36		
	Total =	115.9	psf	(5.55KN/m^2)

Figure 7. Calculation of component weights

BUILDING WEIGHTS (KIPS):

Item Desc.	Grid Line	Width or Trib. ht. (ft)	Length (ft)	Number	Trib Area $(ft)^2$	Unit Wt. (psf)	Weight Trans. (kips)	Weight Long. (kips)	Weight Total (kips)
Level - Roof									
Diaph.	A1-C2	29.5	12	1	354	92.3	32.7	32.7	32.7
Diaph.	A2-D8	53	141	1	7473	92.3	689.8	689.8	689.8
Diaph.	A8-D9	29.5	12	1	354	92.3	32.7	32.7	32.7
Longitudinal Walls									
Beam	A, C, D	5.5	165	2	1815	115.9	210.4		210.4
Glass[1]	A, C, D	3.46	165	2	1142	8.0	9.1		9.1
Columns[1]	A, C, D	3.46	165	2	1142	19.2	21.9		21.9
Transverse Walls									
Wall	2 to 8	5.5	53	7	2041	112.5		229.6	229.6
Wall	1 & 9	5.5	29.5	2	325	112.5		36.5	36.5
Parapet	2 & 8	1.71	23.5	2	80	112.5		9.0	9.0
Parapet	1 & 9	1.71	29.5	2	101	112.5		11.4	11.4
Total Roof Tributary Weight =							996	1042	1283
Level - Floor									
Diaph.	A1-C2	29.5	12	1	354	95.3	33.7	33.7	33.7
Diaph.	A2-D8	53	141	1	7473	95.3	712.2	712.2	712.2
Diaph.	A8-D9	29.5	12	1	354	95.3	33.7	33.7	33.7
Longitudinal Walls									
Beam	A, C, D	11	165	2	3630	40.9	148.5		148.5
Infill	A, C, D	3	165	2	990	40.0	39.6		39.6
Glass[2]	A, C, D	6	165	2	1980	8.0	15.8		15.8
Columns[3]	A, C, D	9	165	2	2970	19.2	56.9		56.9
Transverse Walls									
Wall	2 to 8	11	53	7	4081	112.5		459.1	459.1
Wall	1 & 9	11	29.5	2	649	112.5		73.0	73.0
Typical Floor Tributary Weight =							1040	1312	1573
Total Building Weight =									4428

[1] Trib height at roof = 1/2(story height) - depth to bottom of beam = 5.5' - 16.0" - 8.5" = 3.46'

[2] Trib height at floor = story height - beam depth -infill depth = 11' - 2' - 3' = 6'

[3] Trib height at floor = story height - beam depth = 11' - 2' = 9'

1-in = 25.4mm
1-ft = 0.30m
1-kip = 4.448KN

Figure 8. Calculation of building weights

B-4 *Calculate Vertical Distribution of Forces.*

$$F_x = C_{vx} V \qquad \text{(EQ. 5.3.4-1 FEMA 302)}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \text{(EQ. 5.3.4-2 FEMA 302)}$$

where; k = 1 in both directions for the building period is less than 0.5 seconds

The calculations are tabularized below*;

Story Level	w_i (kips)	h_i (ft)	$w_i x h_i$ (ft-kips)	C_{vx}	$C_{vx} x V_{trans}$ $=F_{x,trans}$ (kips)	$C_{vx} x V_{long}$ $=F_{x,long}$ (kips)
Roof	1283	33	42337	0.45	338	239
3^{rd}	1573	22	34597	0.37	276	195
2^{nd}	1573	11	17299	0.18	138	98
SUM =	4428		94232	1.00	753	531

*Note: For metric equivalents; 1-ft = 0.30m, 1-kip = 4.448KN, 1-ft-kip = 1.36KN-m

Therefore;

Transverse direction;
$F_{roof} = 338^k$
$F_{3rd} = 276^k$
$F_{2nd} = 138^k$

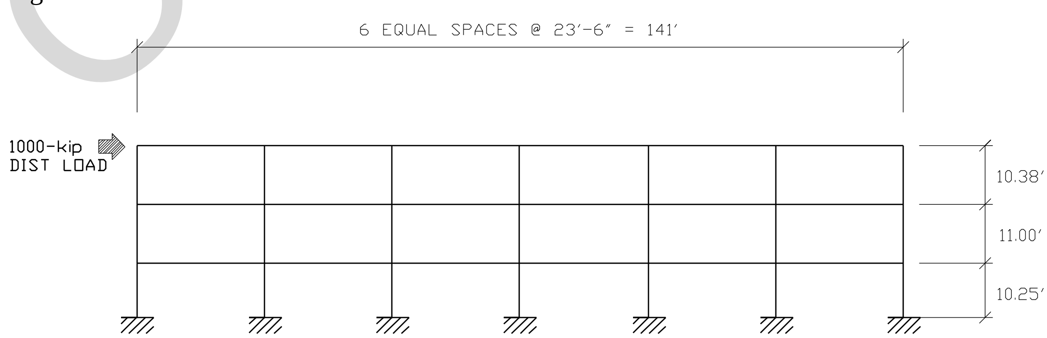
Longitudinal direction;
$F_{roof} = 239^k$
$F_{3rd} = 195^k$
$F_{2nd} = 98^k$

B-5 *Perform Static Analysis.*

General;

Because the diaphragms are rigid, relative rigidities of the lateral load resisting elements must be determined in order to establish the distribution of seismic loads. In the transverse direction, the shear walls are analyzed based on their flexural and shear deformations of a cantilever wall using closed form equations. In the longitudinal direction, moment frames are analyzed using a two-dimensional computer analysis program (RISA-2D, version 4.0) to determine their rigidity. Increased flexibility due to cracking for both the shear walls and the moment frames was accounted for by using cracked section properties in accordance with Section 10.11.1 of ACI 318-95.

The following diagram shows the computer model geometry used to model the longitudinal moment frames. Relative rigidities are determined for each floor level. For example, the stiffness at the roof is determined by applying a 1000^k load at the roof level (distributed uniformly along the length of the frame at that level) with no other loads acting on the model. The deflection of the frame is then taken as the average of all nodes at that level.



1-ft = 0.30m
1-kip = 4.448KN

Cracked section properties are determined as follows;

Floor Beams;

$$I_{cracked} = 0.35 I_g = 0.35\left[\frac{1}{12}(18'')(24'')^3\right] = 7{,}258 - in^4 \ \ (3.02X10^9\ mm^4),$$

Area = 432-in^2 (278.6X10^3 mm^2)

Roof Beams;

$$I_{cracked} = 0.35 I_g = 0.35\left[\frac{1}{12}(18'')(16'')^3\right] = 2{,}150 - in^4 \ \ (0.89X10^9\ mm^4),$$

Area = 288-in^2 (185.8X10^3 mm^2)

Note: It is conservative not to include the parapet in the roof beam cross section

Columns;

$$I_{cracked} = 0.70 I_g = 0.70\left[\frac{1}{12}(18'')(24'')^3\right] = 14{,}515 - in^4 \ \ (6.04X10^9\ mm^4),$$

Area = 432-in^2 (278.6X10^3 mm^2)

Therefore, the relative rigidities of the longitudinal moment frames are as follows;

$$R_{roof} = \frac{F_{applied}}{\Delta_{3^{rd}\ floor}} = \frac{1{,}000^k}{5.26''} = 190^{kips/in} = 2{,}280^{kips/ft} \ \ (33.25MN/m)$$

$$R_{roof} = \frac{F_{applied}}{\Delta_{3^{rd}\ floor}} = \frac{1{,}000^k}{2.35''} = 426^{kips/in} = 5{,}106^{kips/ft} \ \ (74.46MN/m)$$

$$R_{roof} = \frac{F_{applied}}{\Delta_{3^{rd}\ floor}} = \frac{1{,}000^k}{0.61''} = 1{,}639^{kips/in} = 19{,}672^{kips/ft} \ \ (286.89MN/m)$$

Shear walls are analyzed using the following equation;

$$\Delta = \frac{Vh^3}{3E_c I_{cr}} + \frac{1.2Vh}{A_{cr}G}$$

(formula from the "Reinforced Masonry Handbook" by J. Amrhein, 5th ed. Section 4-3)

$$R = \frac{1{,}000^k}{\Delta}$$

where; V = lateral force on pier in (kips)
h = story to story height (in)
E_c = Modulus of elasticity of cracked concrete = 3,605ksi (for f_c' = 4,000psi) (24.86MPa for f_c' = 27.58MPa)
$I_{cr} = 0.70I_g$ = Moment of inertia of cracked wall
$A_{cr} = A_g$ = Gross wall area; thickness x length
G = Shear modulus = $E_c/(2(1+v))$, with v = 0.18.
G = 1,528ksi (10.53MPa)

Note: The shear walls consist of two walls in series. These walls are connected by a flexible floor beam, but are not a coupled wall system. Calculations for the stiffness of the walls at each floor level are tabularized below;

Note: Calculations use V = 1000^k (4.448MN). Also, I_{cr} uses a value of 0.7 > 0.35 as specified in the code because it is anticipated that the walls are relatively understressed and that a value of 0.35 would be overly conservative.

Floor Level	Grid Lines	h (in)	$I_{cr} = 0.7(tL^3/12)$ t (in)	L (in)	I_{cr} ($x10^3$-in^4)	A_{cr} (in^2)	Δ (in)	$\Sigma\Delta_{below}$ (in)	K = $1^k/\Sigma\Delta_{below}$ (kips/ft)
Second	2 to 8	132	9	282	11,774	2,538	5.89E-05	5.89E-05	1,415
Third	2 to 8	132	9	282	11,774	2,538	5.89E-05	1.18E-04	707
Roof	2 to 8	132	9	282	11,774	2,538	5.89E-05	1.77E-04	472
Second	1 & 9	132	9	354	23,290	3,186	4.17E-05	4.17E-05	2,000
Third	1 & 9	132	9	354	23,290	3,186	4.17E-05	8.33E-05	1,000
Roof	1 & 9	132	9	354	23,290	3,186	4.17E-05	1.25E-04	667

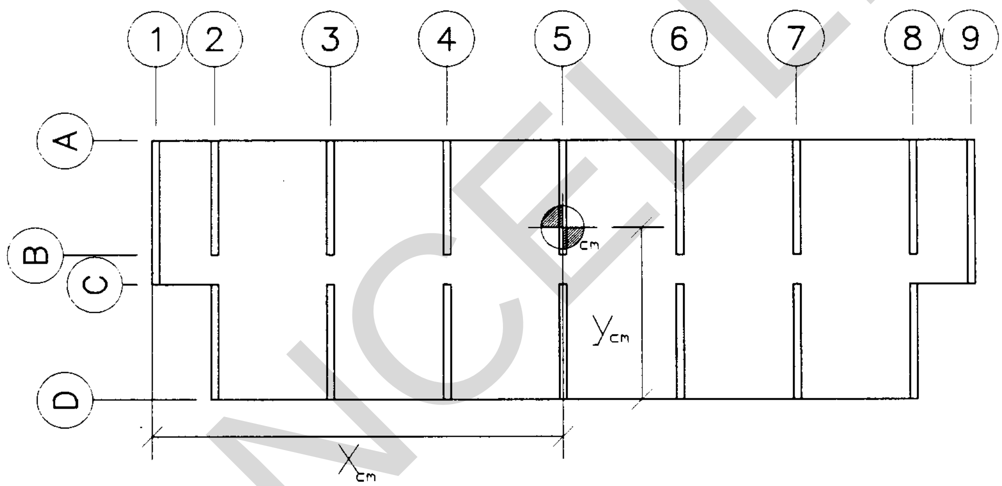
1-in = 25.4m
$1\text{-in}^2 = 0.645X10^3\ mm^2$
$1\text{-in}^4 = 0.416X10^6\ mm^4$

B-6 *Determine cr and cm.*

Center of mass, cm;

By inspection, the center of mass in the longitudinal direction lies at the plan centroid. Therefore, it is only necessary to calculate the center of mass in the transverse direction.

The center of mass is defined as; $y_{cm} = \dfrac{\sum w_i y_i}{\sum w_i}$



For the calculations which follow, it is convenient to use an equivalent weight for exterior walls in pounds per square foot (N/m^2). Unit weights and tributary areas were previously calculated in the seismic weights section of this problem.

Roof;
Walls on grid lines A, C, and D;

$$\text{Equivalent wall wt.} = 115.9\text{psf} + 19.15\text{psf}\left(\frac{3.46'}{5.5'}\right) + 8\text{psf}\left(\frac{3.46'}{5.5'}\right) = 133\text{psf} \quad (6.37\text{KN/m}^2)$$

Walls on grid lines 1 through 9;
Equivalent wall wt. = Actual wall wt. = 112.5psf ($5.39KN/m^2$)

Floors;
Walls on grid lines A, C, and D;

$$\text{Equivalent wall wt.} = 40.91\text{psf} + 19.15\text{psf}\left(\frac{9'}{11'}\right) + 8\text{psf}\left(\frac{6'}{11'}\right) + 40\text{psf}\left(\frac{3'}{11'}\right) = 71.9\text{psf} \quad (3.44\text{KN/m}^2)$$

Walls on grid lines 1 through 9;
Equivalent wall wt. = Actual wall wt. = 112.5psf ($5.39KN/m^2$)

Calculations for the location of the mass centroid are tabularized below;

LOCATION OF MASS CENTROID OF ROOF IN THE TRANSVERSE DIRECTION

Element	Grid Line	Width or Height (ft)	Length (ft)	y_i (ft)	Area (ft^2)	Unit wt. (psf)	Number	w_i (kips)	$w_i y_i$ (ft-kips)
Diaph	A1-C2	29.5	12	38.25	354	92.3	1	33	1,250
	A2-D8	53	141	26.5	7473	92.3	1	690	18,279
	A8-C9	29.5	12	38.25	354	92.3	1	33	1,250
Trans Walls	A1-C1	7.21	29.5	38.25	213	112.5	1	24	915
	A9-C9	7.21	29.5	38.25	213	112.5	1	24	915
	A2-D2 to A8-D8	5.5	53	26.5	291.5	112.5	7	230	6,083
Trans Parapet	C2-D2 & C8-D8	2	24	12	40	113	2	9	106
Long Walls	A1-A9	5.5	165	53	907.5	133	1	121	6,397
	C1-C2	5.5	12	23.5	66	133	1	9	206
	C8-C9	5.5	12	23.5	66	133	1	9	206
	D2-D8	5.5	141	0	775.5	133	1	103	0
Therefore, for the roof level;							SUM =	1,283	35,608

y_{cm} = **27.75 ft** **(8.46m)** x_{cm} = **82.50 ft** **(25.2m)**

LOCATION OF MASS CENTROID OF FLOOR IN THE TRANSVERSE DIRECTION

Element	Grid Line	Width or Height (ft)	Length (ft)	y_i (ft)	Area (ft^2)	Unit wt. (psf)	Number	w_i (kips)	$w_i y_i$ (ft-kips)
Diaph	A1-C2	29.5	12	38.25	354	95.3	1	34	1,290
	A2-D8	53	141	26.5	7473	95.3	1	712	18,873
	A8-C9	29.5	12	38.25	354	95.3	1	34	1,290
Trans Walls	A1-C1	11	29.5	38.25	324.5	112.5	1	37	1,396
	A9-C9	11	29.5	38.25	324.5	112.5	1	37	1,396
	A2-D2 to A8-D8	11	53	26.5	583	112.5	7	459	12,166
Long Walls	A1-A9	11	165	53	1815	71.9	1	130	6,916
	C1-C2	11	12	23.5	132	71.9	1	9	223
	C8-C9	11	12	23.5	132	71.9	1	9	223
	D2-D8	11	141	0	1551	71.9	1	112	0
Therefore, at the floor level;							SUM =	1,573	43,775

y_{cm} = **27.83 ft** **(8.48m)** x_{cm} = **82.50 ft** **(25.2m)**

1-in = 25.4mm 1psf = 47.9KN/m^2
1-ft = 3.0mm 1-kip = 4.448KN
1-ft^2 = 0.093m^2 1-ft-kip=1.36KN-m

Center of rigidity, cr;

Due to the symmetrical layout of the lateral load resisting elements, the center of rigidity can be located by inspection.

For both the floors and the roof; **y_{cr} = 26.5 ft (8.08m)** **x_{cr} = 82.5 ft (25.2m)**

B-7 *Perform torsional analysis.*

The load distributed to each element of the lateral load resisting system in a particular direction can be determined by its stiffness relative to all the other elements in that direction.

The direct shear component is F_v;

$$F_v = \frac{R_i}{\sum R_i} V$$

Additionally, there are two torsional components; F_{tors}, and F_{acc};

$$F_{tors} = \frac{Rd_i}{\sum Rd_i^2} Ve$$

where; e = eccentricity due to the displacement of the center of mass to the center of rigidity

$$F_{acc} = \frac{Rd_i}{\sum Rd_i^2} Ve_{acc}$$

where; e_{acc} = eccentricity to account for possible errors in calculating the cm or cr.

The shear component and the two torsional components add together to determine the total force transferred to an element of the lateral load resisting system. Calculations for the distribution of forces to the lateral load resisting elements at each floor level are tabularized below;

Roof;

V_{trans} = 338 kips e_{trans} = 0 ft $e_{trans,acc}$ = 8.25 ft

V_{long} = 239 kips e_{long} = 1.25 ft $e_{long,acc}$ = 2.65 ft

EQ Direction	Element (grid line)	R_{yi} (k/ft)	R_{xi} (k/ft)	$d_{xi}^{(1)}$ (ft)	$d_{yi}^{(1)}$ (ft)	$R_x d_y$ or $R_y d_x$ (k-ft/ft)	$R_x d_y^2$ or $R_y d_x^2$ (k-ft/ft)	F_v (kips)	$F_t^{(2)}$ (kips)	F_{acc} (kips)	$F_{total} = F_v+F_t+F_{acc}$ (kips)
Trans	A1-C1	667	-	82.5	-	55027.5	4539769	28.40	0.00	5.71	34.1
	A2-B2 & C2-D2[3]	944	-	70.5	-	66552.0	4691916	40.20	0.00	6.91	47.1
	A3-B3 & C3-D3[3]	944	-	47	-	44368.0	2085296	40.20	0.00	4.61	44.8
	A4-B4 & C4-D4[3]	944	-	23.5	-	22184.0	521324	40.20	0.00	2.30	42.5
	A5-B5 & C5-D5[3]	944	-	0	-	0.0	0	40.20	0.00	0.00	40.2
	A6-B6 & C6-D6[3]	944	-	23.5	-	22184.0	521324	40.20	0.00	2.30	42.5
	A7-B7 & C7-D7[3]	944	-	47	-	44368.0	2085296	40.20	0.00	4.61	44.8
	A8-B8 & C8-D8[3]	944	-	70.5	-	66552.0	4691916	40.20	0.00	6.91	47.1
	A9-C9	667	-	82.5	-	55027.5	4539769	28.40	0.00	5.71	34.1
Long	A2-A8	-	2280	-	26.5	60420	1601130	119.37	0.67	1.42	121.5
	D2-D8	-	2280	-	26.5	60420	1601130	119.37	-0.67	1.42	120.8
	ΣR_i =	7942	4560			J_p =	26878870				

[1]Note: distance 'd' is measured from the cr

[2] Note: only positive components are added

[3] Note: there are two walls in parallel with a total rigidity of; 2 x 472-kips/ft = 944-kips/ft

Note: For metric equivalents; 1-kip/ft = 14.58KN/m, 1-ft = 0.30m, 1-kip = 4.48KN

Second Floor;

V_{trans} =	138	kips	e_{trans} =	0	ft	$e_{trans,acc}$ =	8.25 ft
V_{long} =	98	kips	e_{long} =	1.33	ft	$e_{long,acc}$ =	2.65 ft

EQ Direction	Element (grid line)	R_{yi} (k/ft)	R_{xi} (k/ft)	d_{xi}[1] (ft)	d_{yi}[1] (ft)	R_xd_y or R_yd_x (k-ft/ft)	$R_xd_y^2$ or $R_yd_x^2$ (k-ft/ft)	F_v (kips)	F_t[2] (kips)	F_{acc} (kips)	$F_{total} = F_v+F_t+F_{acc}$ (kips)
Trans	A1-C1	2,000	-	82.5	-	165000.0	13612500	11.61	0.00	1.91	13.5
	A2-B2 & C2-D2[3]	2,830	-	70.5	-	199515.0	14065808	16.42	0.00	2.31	18.7
	A3-B3 & C3-D3[3]	2,830	-	47	-	133010.0	6251470	16.42	0.00	1.54	18.0
	A4-B4 & C4-D4[3]	2,830	-	23.5	-	66505.0	1562868	16.42	0.00	0.77	17.2
	A5-B5 & C5-D5[3]	2,830	-	0	-	0.0	0	16.42	0.00	0.00	16.4
	A6-B6 & C6-D6[3]	2,830	-	23.5	-	66505.0	1562868	16.42	0.00	0.77	17.2
	A7-B7 & C7-D7[3]	2,830	-	47	-	133010.0	6251470	16.42	0.00	1.54	18.0
	A8-B8 & C8-D8[3]	2,830	-	70.5	-	199515.0	14065808	16.42	0.00	2.31	18.7
	A9-C9	2,000	-	82.5	-	165000.0	13612500	11.61	0.00	1.91	13.5
Long	A2-A8	-	19672	-	26.5	521308	13814662	48.77	0.69	1.37	50.8
	D2-D8	-	19672	-	26.5	521308	13814662	48.77	-0.69	1.37	50.1
	ΣR_i =	23810	39344			J_p =	98614614				

Third Floor;

V_{trans} =	276	kips	e_{trans} =	0	ft	$e_{trans,acc}$ =	8.25 ft
V_{long} =	195	kips	e_{long} =	1.33	ft	$e_{long,acc}$ =	2.65 ft

EQ Direction	Element (grid line)	R_{yi} (k/ft)	R_{xi} (k/ft)	d_{xi}[1] (ft)	d_{yi}[1] (ft)	R_xd_y or R_yd_x (k-ft/ft)	$R_xd_y^2$ or $R_yd_x^2$ (k-ft/ft)	F_v (kips)	F_t[2] (kips)	F_{acc} (kips)	$F_{total} = F_v+F_t+F_{acc}$ (kips)
Trans	A1-C1	1000	-	82.5	-	82500.0	6806250	23.23	0.00	4.41	27.6
	A2-B2 & C2-D2[3]	1414	-	70.5	-	99687.0	7027934	32.85	0.00	5.33	38.2
	A3-B3 & C3-D3[3]	1414	-	47	-	66458.0	3123526	32.85	0.00	3.55	36.4
	A4-B4 & C4-D4[3]	1414	-	23.5	-	33229.0	780882	32.85	0.00	1.78	34.6
	A5-B5 & C5-D5[3]	1414	-	0	-	0.0	0	32.85	0.00	0.00	32.8
	A6-B6 & C6-D6[3]	1414	-	23.5	-	33229.0	780882	32.85	0.00	1.78	34.6
	A7-B7 & C7-D7[3]	1414	-	47	-	66458.0	3123526	32.85	0.00	3.55	36.4
	A8-B8 & C8-D8[3]	1414	-	70.5	-	99687.0	7027934	32.85	0.00	5.33	38.2
	A9-C9	1000	-	82.5	-	82500.0	6806250	23.23	0.00	4.41	27.6
Long	A2-A8	-	5106	-	26.5	135309	3585689	97.55	0.82	1.64	100.0
	D2-D8	-	5106	-	26.5	135309	3585689	97.55	-0.82	1.64	99.2
	ΣR_i =	11898	10212			J_p =	42648559				

See page H2-24 for metric equivalents

B-8 *Determine need for redundancy factor, r.*

The building has a seismic design category of D, and therefore, per Paragraph 4-4, the redundancy factor is calculated as follows;

$$r_x = 2 - \frac{20}{r_{max}\sqrt{A_x}} \qquad \text{(EQ. 4-1)}$$

Evaluation of r_{max};

For the longitudinal moment frames, r_{max} is taken as the maximum of the sum of the shears in any two adjacent columns in the plane of the frame divided by the story shear. The portal method is used as an approximation for the distribution of shear in the columns.

Therefore, since each story has 7 columns per frame, 2 frames per story, and the frames are each equally loaded;

$$r_{max} = \frac{2}{12} = 0.17 \text{ at each floor level}$$

For the transverse shear walls, r_{max} is taken as the shear in the most heavily loaded wall divided by $10/l_w$. Where l_w is the wall length in feet divided by the story shear. At every story level, the most heavily loaded wall occurs at either grid lines 1 or 9.

Therefore, at each story level;

$$\frac{10}{l_w} = \frac{10}{29.5'} = 0.339$$

At the third story level;

$$r_{max} = \frac{34.1^k}{338^k}(0.339) = 0.034$$

At the second story level;

$$r_{max} = \frac{27.6^k}{276^k}(0.339) = 0.034$$

At the first story level;

$$r_{max} = \frac{13.7^k}{138^k}(0.339) = 0.034$$

Calculations to determine the redundancy factor are tabularized below;

Story Level	Earthquake Direction	r_{max}	A_x (ft^2)	r_x
Third	Transverse	0.034	8,181	-4.5 < 1
	Longitudinal	0.170	8,181	0.70 < 1.0
Second	Transverse	0.034	8,181	-4.50 < 1.0
	Longitudinal	0.170	8,181	0.70 < 1.0
First	Transverse	0.034	8,181	-4.50 < 1.0
	Longitudinal	0.170	8,181	0.70 < 1.0

Therefore, for all story levels; $\mathbf{r_{x,trans} = 1.0,\ r_{x,long} = 1.0}$

B-9 *Determine need for overstrength factor* Ω_o.

Per Paragraph 5.2.6.3.2 of FEMA 302, collector elements, splices, and their connections to resisting elements shall be designed for the load combinations of Section 5.2.7.1 of FEMA 302. Therefore, for these force controlled elements the following seismic load will be used;

For both the transverse or the longitudinal direction;

$$E = \Omega_{\circ} Q_E \pm 0.2 S_{DS} D \qquad \text{(EQ. 4-6)}$$
$$E = 3Q_E \pm 0.2(0.93)D$$
$$E = 3Q_E \pm 0.186D$$

B-10 *Calculate combined load effects.*

Load combinations per ANSI/ASCE 7-95 are as follows;

(1) 1.4D
(2) 1.2D+1.6L+0.5Lr
(3) 1.2D+0.5L+1.6Lr
(4) 1.2D+E+0.5L
(5) 0.9D+E

However, per Paragraph 4-6;

$E = \rho Q_E$ 6 $0.2S_{DS}D$
$= 1.0Q_E$ 6 $0.2(0.93)D$
$= Q_E$ 6 $0.186D$

Therefore, Equations 4 and 5 become;

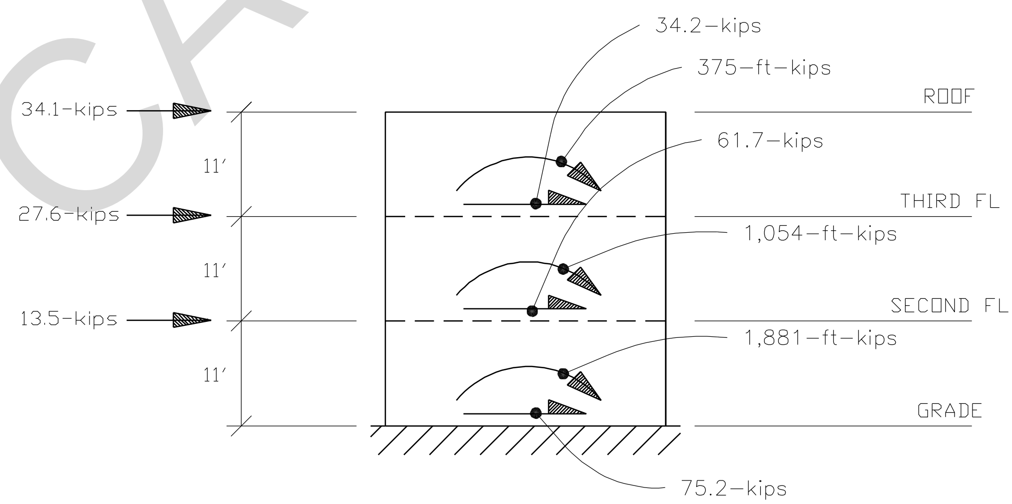
(4a) $1.386D+Q_E+0.5L$
(5a) $0.714D+Q_E$

B-11 *Determine structural member sizes.*

(1) Transverse shear wall design; use f_c' = 4,000psi (27.58MPa), and f_y = 60ksi (413.7MPa)

Per Paragraph 7-2.f. (1), shear walls will be designed in accordance with ACI 318-95 as modified by the provisions given in Chapter 9 of FEMA 303.

Walls on grid lines 1 and 9;
Determine design loads;
The following diagram shows the maximum shear and overturning moment at each level;



1-ft = 0.30m, 1-kip = 4.448KN

Therefore, at the base of the wall V_{max} = 75.2-kips (0.33MN), and $M_{max} = 1{,}881^{\text{ft-kips}}$ (2.55MN-m).

Design for shear;
Determine if reinforcement is required to resist shear;

$$V_u \le A_{cv}\sqrt{f_c'} \qquad \text{(per ACI 318-95 Section 21.6.2.1)}$$

$$V_u = 75.2^k < 202^k (334.5KN < 898.5KN) = (29.5')9"\sqrt{4{,}000psi}(12"/1') = A_{cv}\sqrt{f_c'}$$

Therefore, only minimum reinforcement is required.

By inspection, only one curtain of reinforcement is required. However, because a 9-in. thick wall is being used, it is decided to use two curtains in order to distribute the reinforcement more evenly throughout the section. Also, per ACI 318-95 Section 21.6.2.1, the minimum reinforcement required need only comply with Section 14.3 of ACI 318-95. Therefore, the minimum ratio of vertical reinforcement area to gross concrete area is 0.0012 (for bars smaller than number 5), and the minimum ratio of horizontal reinforcement area to gross concrete area is 0.0020 (also for bars smaller than number 5).

Try 2 curtains of #4 (~10M) bar at 12-in. (308.8mm) on center each way;
Check minimum reinforcement per ACI 318-95 Section 14.3;

$$\rho_h = \frac{2A_{sh}}{bt} = \frac{2(0.20-in^2)}{12"(9")} = 0.0037 > 0.0020 = \rho_{req'd} \qquad \textbf{O.K.}$$

$$\rho_v = \frac{2A_{sv}}{bt} = \frac{2(0.20-in^2)}{12"(9")} = 0.0037 > 0.0012 = \rho_{req'd} \qquad \textbf{O.K.}$$

Check spacing per ACI 318-95 Section 14.3.5;

18" < 3 x (wall thickness) = 3(9") = 27" Therefore, s_{max} = 18" (457.2mm)

$s_{vert} = s_{horiz}$ = 12" < s_{max} = 18" (308.8mm < 457.2mm) **O.K.**

Check capacity;

$$V_u \le \phi V_n \quad \text{with } \phi = 0.6 \qquad \text{(EQ. 11-1 ACI 318-95)}$$

Since h_w/l_w = 33'/29.5' = 1.12 < 2.0 in accordance with Section 21.6.5.3 of ACI 318-95;

$$V_n = A_{cv}(\alpha_c\sqrt{f_c'} + \rho_n f_y) \qquad \text{(EQ. 21-7 ACI 318-95)}$$

where; A_{cv} = 29.5'(9")(12"/1') = 3,186-in^2 (2.05X10^6 mm^2)
f_c' = 4,000psi (46.90MPa)
α_c = 3.0 for h_w/l_w = 1.4 < 1.5
ρ_n = 0.0037
f_y = 60ksi (413.7MPa)

Note: The more conservative Equation 21-6 could have been used due to the light loading.

$$\therefore \phi V_n = \frac{0.6(3{,}186-in^2)}{1000-kips/lb}\left(3\sqrt{4{,}000psi} + 0.0037(60{,}000psi)\right) = 787^k \ (3.50MN)$$

$$\phi V_n = 787^k > 75.2^k = V_u \ (3.50MN > 0.33MN) \qquad \textbf{O.K.}$$

Design for flexural and axial loads;
Determine axial loads acting at each story;
Note: The tributary width of diaphragm is 6-ft., and the story to story height is 11-ft (3.36m).
Dead loads;

At the roof; P_{RD} = 92.3psf(6') + 112.5psf(5.5') = 1,173plf (17.11KN/m)
At the floors; P_{FD} = 95.3psf(6') + 112.5psf(11') = 1,810plf (26.40KN/m)

Live loads; (Live loads are unreducable due to the small tributary area)

At the roof; P_{RL} = 20psf(6') = 120plf (1.75KN/m)
At the floors; P_{FL} = 40psf(6') = 240plf (3.50KN/m)

Confirm that walls are structural;
Per FEMA 302 Section 9.1.1.13, walls with $P_u > 0.35P_o$ shall not be considered to contribute to the calculated strength of the structure for resisting earthquake induced forces. By inspection, load case 4a governs; U = 1.386D + Q_E + 0.5L;

P_u = [1.386(1.17+2(1.81))klf + 0.5(2(0.24))klf]29.5' = 203^k (902.9KN)

The nominal strength of the wall is given by;

$$P_\circ = 0.85f_c^{'}(A_g - A_{st}) + f_y A_{st} \qquad \text{(EQ. 10-2 ACI 318-95)}$$

where; A_g = 29.5'(12"/1')9" = 3,186-in^2 (2.05X10^6 mm^2)
$A_{st} = l_w t \rho_v$ = 29.5'(12"/1')9"(0.0037) = 11.8-in^2 (7.61X10^3 mm^2)
f_c' = 4,000psi (46.90MPa)

$$P_\circ = 0.85(4{,}000\text{psi})(3{,}186 - \text{in}^2 - 11.8 - \text{in}^2) + 60{,}000\text{psi}(11.8 - \text{in}^2) = 11{,}500^k \ (51.15\text{MN})$$

$$\therefore P_u = 203^k << 4{,}025^k = 0.35P_\circ \ (902.9\text{KN}<<18.70\text{MN}) \qquad \text{Therefore, walls are structural}$$

Determine if boundary elements are required;
This requirement will be checked at the first story level only for this is the worst case condition, and if boundary elements are not required at that level they will not be required at the levels above.
Per FEMA 302 Section 9.1.1.13, boundary elements are not required if;

(1) $P_u \le 0.10 A_g f_c^{'}$

and either

(2) $\dfrac{M_u}{V_u l_w} \le 1.0$

or (3) $V_u \le 3A_{cv}\sqrt{f_c^{'}}$, and $\dfrac{M_u}{V_u l_w} \le 3.0$

Check condition (1);
By inspection, load case 4a governs; U = 1.386D + Q_E + 0.5L;
P_u = 203^k << 1,273^k = 0.1(29.5')(12"/1')9"(4,000psi)(1^k/1000-lb) = 0.1$A_g f_c$' **O.K.**
(902.9KN<<5.66MN)

Check condition (2);
By inspection, M_u and V_u are the same for all load cases. Therefore;

$$\frac{M_u}{V_u l_w} = \frac{1{,}881^{\text{ft-kips}}}{75.2^k(29.5')} = 0.85 < 1.0 \qquad \textbf{O.K.}$$

Therefore, no boundary elements are required

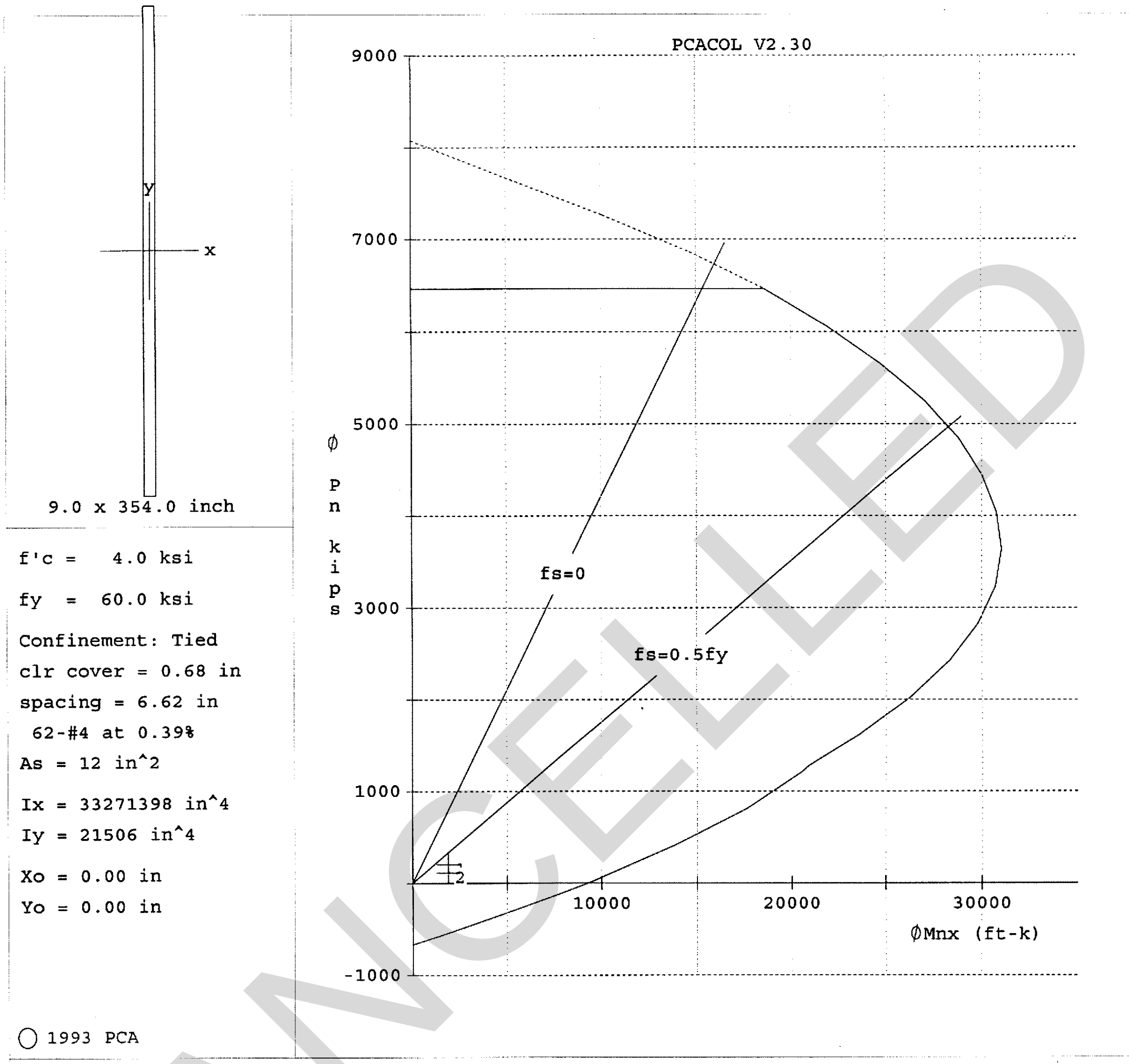
Determine if wall has adequate capacity for flexural and axial loads combined;
Per FEMA 302 Section 9.1.1.13, walls subject to combined flexural and axial loads shall be designed in accordance with Sections 10.2 and 10.3 of ACI 318-95 except that Section 10.3.6 of ACI 318-95 and the nonlinear strain requirements of 10.2.2 do not apply. To satisfy these requirements, an analysis program entitled 'PCACOL' produced by the Portland Cement Association was utilized. This program produces an interaction diagram for the wall cross section and plots the loads acting on the section.

Note: At the time of writing of this problem, the current version of PCACOL program was developed for ACI 318-89. However, there are no changes between the 95 code and the 89 code which will affect the results of this shear wall analysis.

Determine design loads;
Load case 4a; P_u = [1.386(1.17+2(1.81))klf + 0.5(2(0.24))klf]29.5' = 203^k (902.9KN)
M_u = 1.0(1,881-ft-kips) = 1,881-ft-kips (2.55MN-m)
Load case 5a; P_u = [0.714(1.79+2(1.81))klf]29.5' = 114^k (507.1KN)
M_u = 1.0(1,881-ft-kips) = 1,881-ft-kips (2.55MN-m)

Figure 9 shows the design interaction diagram (obtained from PCACOL) for the shear wall section. The section has a 9-in. (220.6mm) thick web reinforced with two curtains of reinforcement each having #4 (~10M) vertical and horizontal bars spaced at 12-in. (308.8mm) on center with f_c' = 4,000psi (46.90MPa) and f_y = 60,000psi (413.7MPa). Two design load combinations are listed. The point marked "1" represents the P_u-M_u combination corresponding to load case 4a, and the point marked "2" represents the P_u-M_u combination corresponding to load case 5a. This figure shows that the walls have sufficient capacity for axial and overturning forces.

Therefore, use 2 curtains of #4 (~10M) bars at 12" (308.8mm) o.c. each way



Licensed To: Licensee name not yet specified.

File name: H:\USERS\SBURNS\H2\WALL1-9.COL

Project: TI 809-04

Column Id:

Engineer:

Date: Time:

Code: ACI 318-89

Units: in-lb

X-axis slenderness is not considered.

Material Properties:

Ec = 3834 ksi eu = 0.003 in/in

fc = 3.40 ksi Es = 29000 ksi

Beta1 = 0.85

Stress Profile: Block

phi(c) = 0.70, phi(b) = 0.90

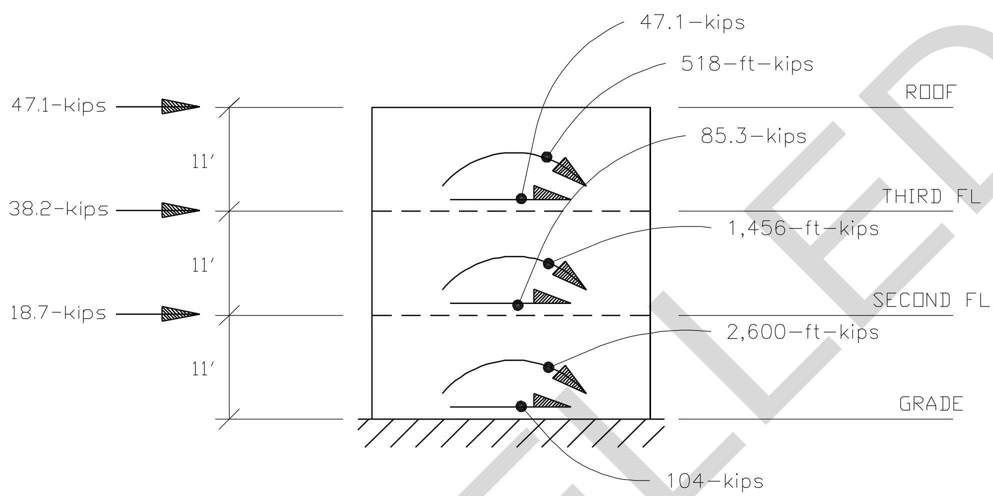
Note: For metric equivalents; 1-in = 25.4mm, 1-ft-kip = 1.356KN-m, 1-ksi = 6.895MPa

Figure 9. Design strength interaction diagram for shear wall section on grid lines 1 and 9

Walls on grid lines 2 through 8;

One design will be produced for the worst case and used for all walls on grid lines 2 through 8. By inspection, the worst case situation occurs on either grid line 2 or 8 between grid lines C and D because these walls carry the largest shear, largest overturning moment, and smallest axial load (the walls moment capacity increases with an increase in axial load).

Determine design loads;
The following diagram shows the maximum shear and overturning moment at each level;



1-ft = 0.30m, 1-kip = 4.448KN

Therefore, at the base of the wall V_{max} = 104-kips (462.6KN), and M_{max} = 2,600-ft-kips (3.53MN-m).

Design for shear;
By inspection, only minimum reinforcement will be required.

Try 2 curtains of #4 (~10M) bar at 12-in. (308.8mm) on center each way as used for walls on grid lines 1 and 9;

Check capacity;

$V_u \le \phi V_n$ with $\phi = 0.6$ (EQ. 11-1 ACI 318-95)

Since h_w/l_w = 33'/26.5' = 1.25 < 2.0 in accordance with Section 21.6.5.3 of ACI 318-95;

$V_n = A_{cv}(\alpha_c\sqrt{f_c'} + \rho_n f_y)$ (EQ. 21-7 ACI 318-95)

where; A_{cv} = 23.5'(9")(12"/1') = 2,538-in^2 (1.64X10^6 mm^2)
f_c' = 4,000psi (46.90MPa)
α_c = 3.0 for h_w/l_w = 1.3 < 1.5
ρ_n = 0.0037
f_y = 60ksi (413.7MPa)

Note: The more conservative Equation 21-6 could have been used due to the light loading.

$$\therefore \phi V_n = \frac{0.6(2{,}538-\text{in}^2)}{1000-\text{kips/lb}}\left[\sqrt{4{,}000\text{psi}} + 0.0037(60{,}000\text{psi})\right] = 627^k \quad (2.79\text{MN})$$

$$\phi V_n = 627^k > 104^k = V_u \quad (2.79\text{MN} > 462.6\text{KN})$$ **O.K.**

Design for flexural and axial loads;
Determine axial loads acting at each story;
Note: The tributary width of diaphragm is 23.5-ft(1/2) = 11.8-ft. (3.60m), and the story to story height is 11-ft (3.36m).

Dead loads;

At the roof; P_{RD} = 92.3psf(11.8') + 112.5psf(5.5') = 1,708plf (24.91KN/m)

At the floors; P_{FD} = 95.3psf(11.8') + 112.5psf(11') = 2,362plf (34.45KN/m)

Live loads; (Live loads are unreducable due to the small tributary area)

At the roof; P_{RL} = 20psf(11.8') = 236plf (3.44KN/m)

At the floors; P_{FL} = 40psf(11.8') = 472plf (6.88KN/m)

Confirm that walls are structural;

Per FEMA 302 Section 9.1.1.13, walls with $P_u > 0.35P_o$ shall not be considered to contribute to the calculated strength of the structure for resisting earthquake induced forces. By inspection, load case 4a governs; U = 1.386D + Q_E + 0.5L;

P_u = [1.386(1.71+2(2.36))klf + 0.5(0.236+2(0.472))klf]29.5' = 280^k (1.25MN)

The nominal strength of the wall is given by;

$$P_\circ = 0.85f_c^{'}(A_g - A_{st}) + f_y A_{st} \quad \text{(EQ. 10-2 ACI 318-95)}$$

where; A_g = 23.5'(12"/1')9" = 2,538-in^2 (1.64X10^6 mm^2)

$A_{st} = l_w t\rho_v$ = 23.5'(12"/1')9"(0.0037) = 9.39-in^2 (6.06X10^3 mm^2)

f_c' = 4,000psi (46.9MPa)

$$P_\circ = 0.85(4{,}000\text{psi})(2{,}538-\text{in}^2 - 9.39-\text{in}^2) + 60{,}000\text{psi}(9.39-\text{in}^2) = 9{,}161^k \ \ (40.75\text{MN})$$

$$\therefore P_u = 280^k << 3{,}206^k = 0.35P_\circ \ \ (1.25\text{MN} << 14.26\text{MN}) \qquad \text{Therefore, walls are structural}$$

Determine if boundary elements are required;

This requirement will be checked at the first story level only for this is the worst case condition, and if boundary elements are not required at that level they will not be required at the levels above.

Per FEMA 302 Section 9.1.1.13, boundary elements are not required if;

(1) $P_u \le 0.10A_g f_c^{'}$

and either

(2) $\dfrac{M_u}{V_u l_w} \le 1.0$

or (3) $V_u \le 3A_{cv}\sqrt{f_c^{'}}$, and $\dfrac{M_u}{V_u l_w} \le 3.0$

Check condition (1);

By inspection, load case 4a governs; U = 1.386D + Q_E + 0.5L;

P_u = 280^k < $1{,}015^k$ = 0.1(23.5')(12"/1')9"(4,000psi)(1^k/1000-lb) = $0.1A_g f_c$' (1.25MN < 4.51MN)

O.K.

Check condition (2);

By inspection, M_u and V_u are the same for all load cases. Therefore;

$$\frac{M_u}{V_u l_w} = \frac{2{,}600^{ft-kips}}{104^k(23.5')} = 1.06 > 1.0 \qquad \textbf{N.G.}$$

Therefore, check condition 3;

$$V_u = 104^k < 3(9")282"\sqrt{4{,}000\text{psi}} = 482^k \ \ (0.46\text{MN} < 2.14\text{MN}) \qquad \textbf{O.K.}$$

$$\frac{M_u}{V_u l_w} = 1.06 < 3.0 \qquad \textbf{O.K.}$$

Therefore, no boundary elements are required

Determine if wall has adequate capacity for flexural and axial loads combined;

Determine design loads;

Load case 4a; P_u = [1.386(1.71+2(2.36))klf + 0.5(2(0.472))klf]23.5' = 221^k (0.98MN)

M_u = 1.0(2,600-ft-kips) = 2,600-ft-kips (3.53MN-m)

Load case 5b; P_u = [0.714(1.71+2(2.36))klf]23.5' = 108^k (480.4KN-m)

M_u = 1.0(2,600-ft-kips) = 2,600-ft-kips (3.53MN-m)

Figure 10 shows the design interaction diagram (obtained from PCACOL) for the shear wall section. As in Figure 9 only two design load combinations are listed. The point marked "1" represents the P_u-M_u combination corresponding to load case 4a, and the point marked "2" represents the P_u-M_u combination corresponding to load case 5a. This figure shows that the walls have sufficient capacity for axial and overturning forces.

Therefore, use 2 curtains of #4 (~10M) bars at 12" (308.8mm) o.c. each way

(2) Longitudinal moment frame design: use f_c' = 4,000psi (46.90MPa), f_y = 60ksi (413.7MPa)

Per paragraph 7-4.f, special moment frames (SMF's) are frames conforming to the requirements of Sections 21.1 through 21.5 of ACI 318-95 in addition to the ACI 318-95 requirements for ordinary moment frames (OMF's).

Determine design loads;

The RISA-2D model, used previously to determine the relative rigidity of the frames, was reanalyzed for lateral loading. The resultant forces acting on the frame at each floor level, determined in section 3d of this problem solution, were distributed over the length of the structure at each floor level. Note that there are no live loads acting on the frame, and the dead loads are due only to the frames self weight, weight of windows, and non-structural infill walls. Cracked section properties in accordance with ACI 318-95 Section 10.11.1 were used.

Note: The RISA-2D program does not distribute loads over rigid end offsets. Therefore, distributed loads acting within the rigid end offsets were added as point loads on the accompanying node.

Dead loads;

At the roof ; w_{RD} = distributed load due to weight of beam and parapet

w_{RD} = {16"(18")+27"(12")}150pcf(1^k/1000-lb)/144-in^2/ft^2 = 0.638klf (9.30KN/m)

P_{RD} = point load due to tributary weight of column and parapet over rigid end offsets

P_{RD} = {18"(24")}150pcf(3.46')(1^k/1000-lb)/144-in^2/ft^2+0.638klf(2') =2.83^k (12.6KN)

Note: The tributary 3.46-ft (1.06m) column height was calculated in the seismic weights section of this problem.

At the floor; w_{FD} = distributed load due to weight of beam + infill + windows

w_{FD} = {18"(24")}150pcf(1^k/1000-lb)/144in^2/ft^2+…

…+{3'(40psf)+6'(8psf)}(1^k/1000-lb) = 0.618klf (9.01KN/m)

Note: The tributary 6-ft glass height was calculated in the seismic weights section of this problem.

P_{FD} = point load due to tributary weight of column and over rigid end offsets

P_{FD} = {18"(24")}150pcf(11')(1^k/1000-lb)/144in^2/ft^2 = 4.95^k (22.02KN)

The earthquake story forces were uniformly distributed over a length equal to the length of the frame minus the length of the rigid end offsets.

At the roof; $(Q_E)_R$ = 121.5^k/(141'-12(1')) = 0.941klf (13.72KN/m)

At the 3rd floor; $(Q_E)_3$ = 100^k/(141'-12(1')) = 0.775klf (11.30KN/m)

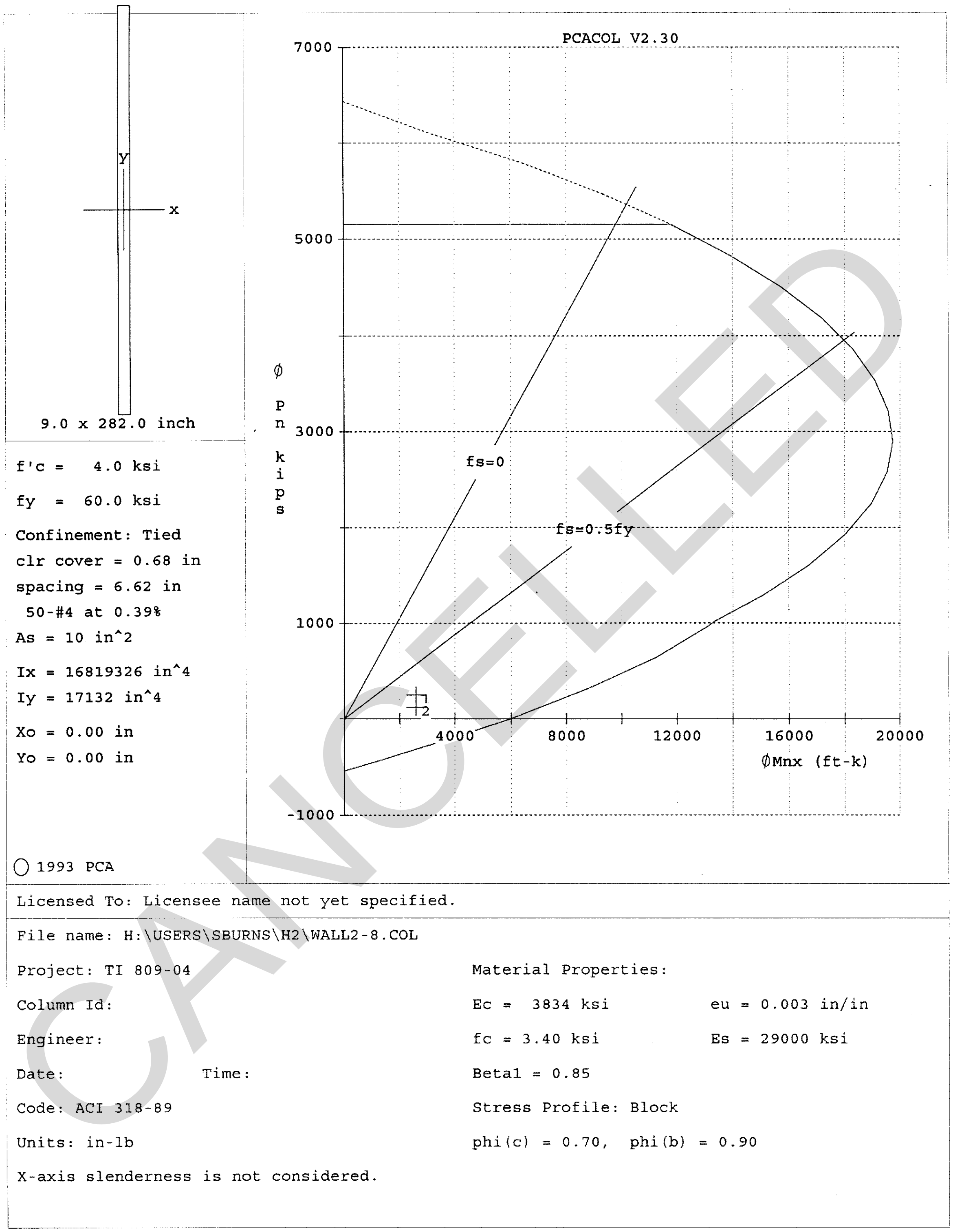
At the 2nd floor; $(Q_E)_2$ = 50.8^k/(141'-12(1')) = 0.394klf (5.75KN/m)

Design beams;

Check that axial loading may be ignored for beam design per Section 21.3.1.1 of ACI 318-95;

From the analysis output, the largest compressive axial load in a floor beam is 11.6^k (51.6KN) (due to load combination 5a), and the largest compressive axial load in a roof beam is 16.8^k (74.73KN) (due to load combination 4a).

At the roof; $\frac{A_g f_c^{'}}{10} = \frac{16''(18'')4ksi}{10} = 115^k > 16.8^k$ (511.5KN > 74.73KN) **O.K.**



Note: For metric equivalents; 1-in = 25.4mm, 1-ft-kip = 1.356KN-m, 1-ksi = 6.895MPa

Figure 10. Design strength interaction diagram for shear wall section on grid lines 2 through 8

At the floors, $\frac{A_g f'_c}{10} = \frac{18"(24")4ksi}{10} = 173^k > 11.6^k$ (0.77MN > 51.6KN) **O.K.**

Therefore, axial loading may be ignored in beam design

The moment and shear diagrams resulting from the RISA-2D analysis for the beams are shown in Figures 11 and 12 for lateral loads and gravity loads respectively. Inspection of these diagrams shows that there is relatively little variation in forces amongst beams at the second and third floor levels, and relatively little variation amongst roof beams. Therefore, it is decided to produce two beam designs, one for the second and third floor levels and one for the roof beams.

Design second and third floor moment frame beams;

Design for flexure;
Negative moment at face of column;
By inspection, the governing load combination is 4a ($U = 1.386D + Q_E + 0.5L$);

$$M_u^- = 1.386(-24.43^{ft-k}) + (-157.40^{ft-k}) = -191^{ft-k} \quad (259.0\text{KN-m})$$

Assume j = 0.9, and d = h – 2.5" = 24" – 2.5" = 21.5" (546.1mm)

$$(A_s)_{trial} = \frac{M_u}{\phi f_y jd} = \frac{191^{ft-k}(12"/1')}{0.9(60ksi)0.9(21.5")} = 2.19-in^2 \quad (1.41X10^3\ mm^2)$$

Try 5 #6 (~20M) bars; $A_s = 5(0.44\text{-}in^2) = 2.20\text{-}in^2$ ($1.42X10^3\ mm^2$)
Determine d (assuming #3 (~10M) bars for transverse reinforcement);

d = 24" – 2" – 0.375" – (0.75"/2) = 21.25" (539.8mm)

Check spacing of bars in one layer per ACI 318-95 Section 7.6.1;

2(clear cover + $d_{stirrup}$) + Σd_{bar} + (No. spaces) x 1"
= 2(2" + 0.375") + 5(0.75") + 4(1") = 12.5" < 18" = beam width (317.5mm < 457.2mm) **O.K.**

Check capacity;

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{A_s f_y}{0.85 f'_c b}$$

$$a = \frac{2.20-in^2(60ksi)}{0.85(4ksi)18"} = 2.16" \quad (54.9\text{mm})$$

$$\phi M_n = 0.9(2.20-in^2)60ksi\left(21.25" - \frac{2.16"}{2}\right)(1'/12") = 200^{ft-k} \quad (271.2\text{KN-m})$$

$$\phi M_n = 200^{ft-k} > 191^{ft-k} = M_u \quad (271.2\text{KN-m} > 259.0\text{KN-m})$$ **O.K.**

Check crack control of flexural reinforcement per ACI 318-95 Section 10.6;

$$z = f_s\sqrt[3]{d_c A} \le 145^{k/in} \quad (25.39\text{KN/mm})$$ (EQ. 10-5 ACI 318-95)

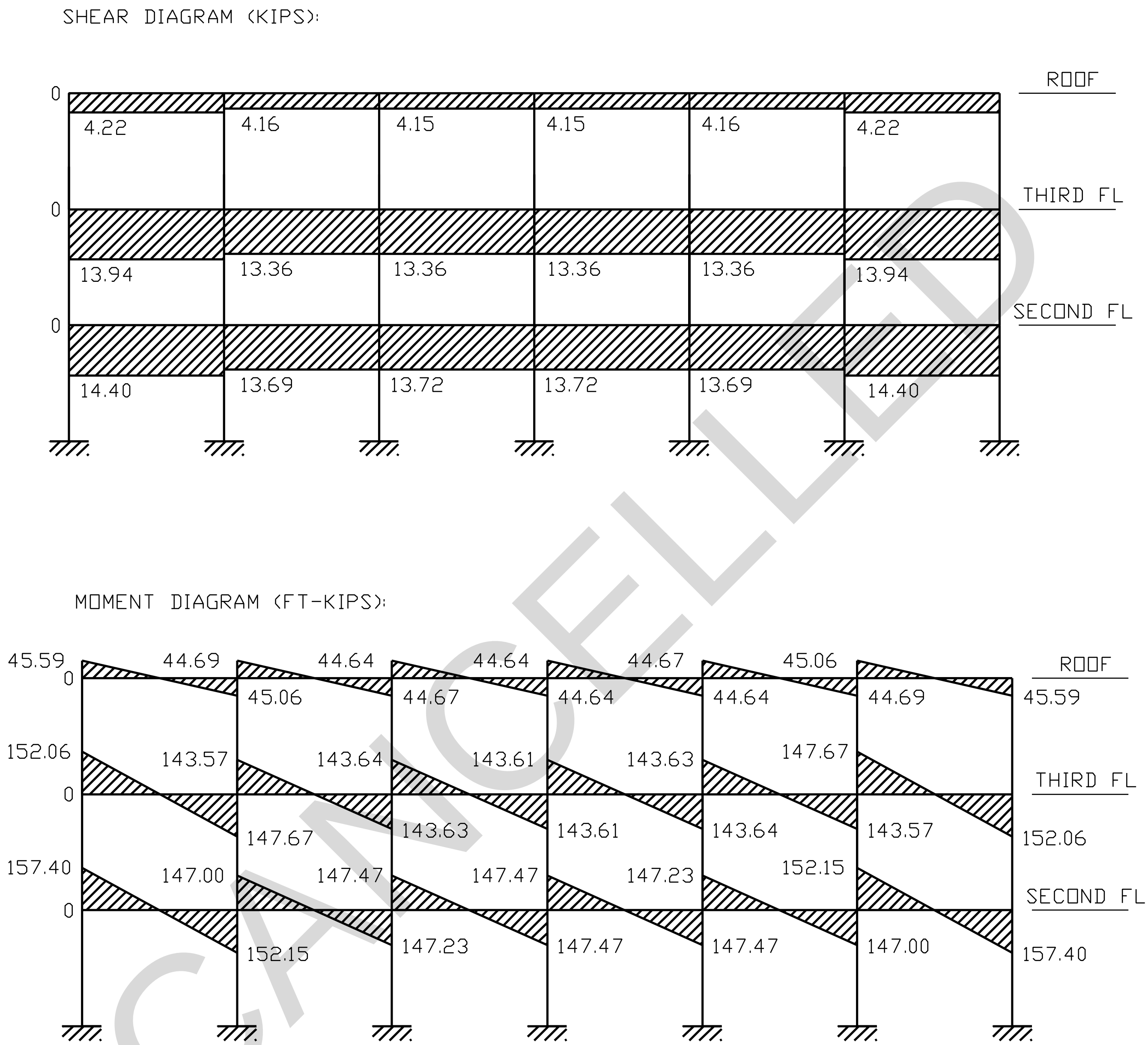
where; $f_s = 0.60(60ksi) = 36ksi$ (248.2MPa)
d_c = 24" – 21.25" = 2.75" (69.9mm)
A = 18"(2)(24"-21.25")/5 = 19.8-in² ($12.8X10^3\ mm^2$)

$$\therefore z = 36ksi\sqrt[3]{2.75"(19.8-in^2)} = 136^{k/in} < 145^{k/in} \quad (23.82\text{KN/mm} < 25.39\text{KN/mm})$$ **O.K.**

Check minimum reinforcement per ACI 318-95 Sections 10.5, and 21.3.2.1;

$$A_{s,min} = \frac{3\sqrt{f'_c}}{f_y} b_w d \ge \frac{200 b_w d}{f_y}$$ (EQ. 10-3 ACI 318-95)

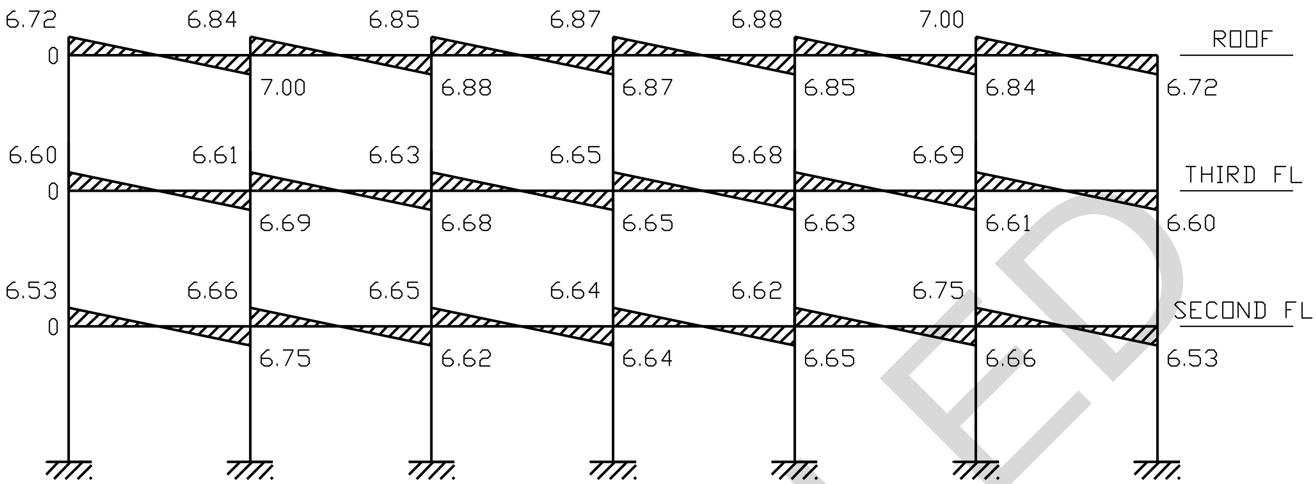
$$\text{or } \rho_{min} = \frac{200}{f_y} = \frac{200}{60{,}000psi} = 0.00333 \quad \text{(governs)}$$



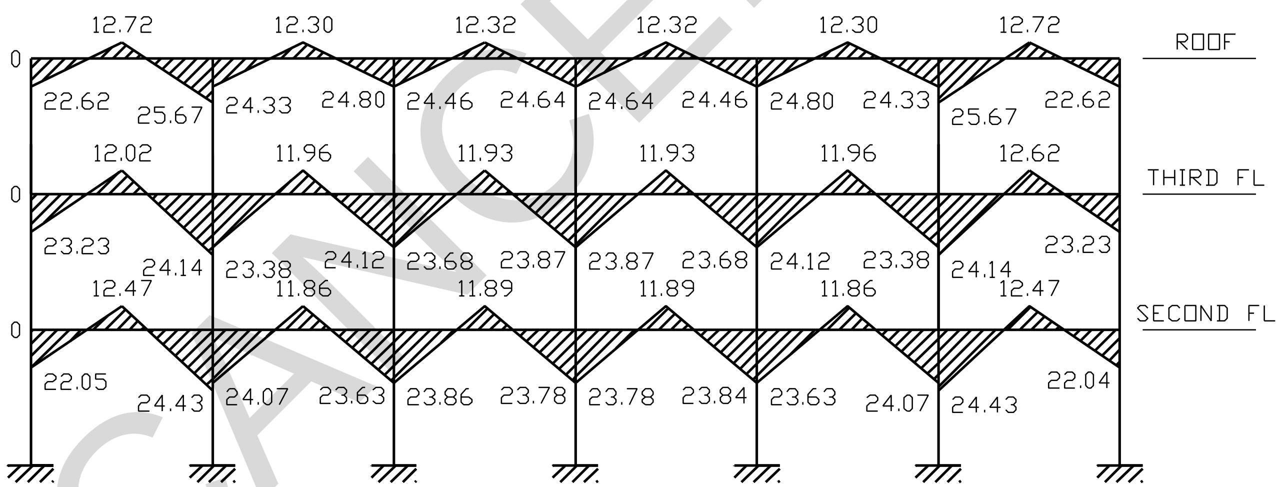
Note: For metric equivalent; 1-kip =4.48KN, 1-ft-kip = 1.356KN-m

Figure 11. Shear and moment diagrams for frame beams due to lateral loading

SHEAR DIAGRAM (KIPS):



MOMENT DIAGRAM (FT-KIPS):



Note: For metric equivalent; 1-kip =4.48KN, 1-ft-kip = 1.356KN-m

Figure 12. Shear and moment diagrams for frame beams due to gravity loads

$$\rho_{min} = \frac{A_{s,min}}{b_w d} = \frac{3\sqrt{f_c'}}{f_y} = \frac{3\sqrt{4{,}000\text{psi}}}{60{,}000\text{psi}} = 0.00316$$

$$\rho = \frac{2.20-\text{in}^2}{18"(21.25")} = 0.00575 > 0.00316 = \rho_{min}$$ **O.K.**

Check upper limit of reinforcement per ACI 318-95 Sections 10.3.3, and 21.3.2.1;
By inspection, $P_u < \phi P_n < 0.10 f_c' A_g$. Therefore, ρ must be less then $0.75\rho_b$ or 0.025.

$$0.75\rho_b = 0.75\left[\frac{0.85\beta_1 f_c'}{f_y}\left(\frac{87{,}000}{87{,}000+f_y}\right)\right] = 0.75\left[\frac{0.85(0.85)4\text{ksi}}{60\text{ksi}}\left(\frac{87{,}000}{87{,}000+60{,}000\text{psi}}\right)\right] = 0.0214$$

and $0.75\rho_b = 0.0214 < 0.025$

$\therefore \rho = 0.00575 < 0.0214 = \rho_{max}$ **O.K.**

Therefore, choose 5 #6 top bars at column face

Positive moment at face of column;
By inspection, the governing load combination is 5a ($U = 0.714D + Q_E = 0.5L$);

$$M_u^+ = 0.714(-24.43^{ft-k}) + 157.40^{ft-k} = +140^{ft-k} \quad (189.8\text{KN-m})$$

Assume j = 0.9, and d = h – 2.5" = 24" – 2.5" = 21.5" (546.1mm)

$$(A_s)_{trial} = \frac{M_u}{\phi f_y jd} = \frac{140^{ft-k}(12"/1')}{0.9(60\text{ksi})0.9(21.5")} = 1.61-\text{in}^2 \quad (1.04\text{x}10^3\ \text{mm}^2)$$

Try 5 #5 (15M) bars; $A_s = 5(0.31\text{-in}^2) = 1.55\text{-in}^2$ ($1.00\text{x}10^3$ mm^2)
Determine d (assuming #3 (~10M) bars for transverse reinforcement);

d = 24" – 1.5" – 0.375" – (0.625"/2) = 21.81" (554.0mm)

Check capacity;

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{1.55-\text{in}^2(60\text{ksi})}{0.85(4\text{ksi})18"} = 1.52" \quad (38.6\text{mm})$$

$$\phi M_n = 0.9(1.55-\text{in}^2)60\text{ksi}\left(21.81"-\frac{1.52"}{2}\right)(1'/12") = 147^{ft-k} \quad (199.3\text{KN-m})$$

$$\phi M_n = 147^{ft-k} > 140^{ft-k} = M_u \quad (199.3\text{KN-m} > 189.8\text{KN-m})$$ **O.K.**

Check crack control of flexural reinforcement per ACI 318-95 Section 10.6;

$$z = f_s\sqrt[3]{d_c A} \le 145^{k/in} \quad (25.39\text{KN/mm})$$ (EQ. 10-5 ACI 318-95)

where; $f_s = 0.60(60\text{ksi}) = 36\text{ksi}$ (248.2MPa)
d_c = 24" – 21.81" = 2.19" (55.6mm)
A = 18"(2)(24"-21.81")/5 = 15.8 -in^2 ($10.19\text{x}10^3$ mm^2)

$$\therefore z = 36\text{ksi}\sqrt[3]{2.19"(15.8-\text{in}^2)} = 117^{k/in} < 145^{k/in} \quad (20.49\text{KN/mm} < 25.39\text{KN/mm})$$ **O.K.**

Check minimum reinforcement per ACI 318-95 Sections 10.5, and 21.3.2.1;

$$A_{s,min} = \frac{3\sqrt{f_c'}}{f_y} b_w d \ge \frac{200 b_w d}{f_y}$$ (EQ. 10-3 ACI 318-95)

$$\text{or } \rho_{min} = \frac{200}{f_y} = \frac{200}{60{,}000\text{psi}} = 0.00333$$ (governs)

$$\rho_{min} = \frac{A_{s,min}}{b_w d} = \frac{3\sqrt{f_c'}}{f_y} = \frac{3\sqrt{4{,}000\text{psi}}}{60{,}000\text{psi}} = 0.00316$$

$$r = \frac{1.55 - \text{in}^2}{18''(21.81'')} = 0.00395 > 0.00333 = r_{\min}$$ **O.K.**

Check moment strength at face of joint per ACI 318-95 Section 21.3.2.2;

$$\frac{1}{2}(fM_n^-)_{@\text{joint}} = \frac{1}{2}(200^{\text{ft-k}}) = 100^{\text{ft-k}} < 147^{\text{ft-k}} = (fM_n^+) \quad (135.6\text{KN-m} < 199.3\text{KN-m})$$ **O.K.**

Therefore, choose 5 #5 (15M) bottom bars at column face

Positive moment at midspan;
By inspection, the governing load combination is 1(U = 1.4D);

$$M_u^+ = 1.4(12.5^{\text{ft-k}}) = 17.5^{\text{ft-k}} \quad (23.7\text{KN-m})$$

Due to the load demand at this section, steel will be governed by detailing requirements. By inspection, the minimum reinforcement requirements of ACI 318-95 Section 10.5, and 21.3.2.1 will govern. Since the steel at the column face is very close to the minimum, it is not possible to terminate any bars.

Check capacity per ACI 318-95 Section 21.3.2.2;

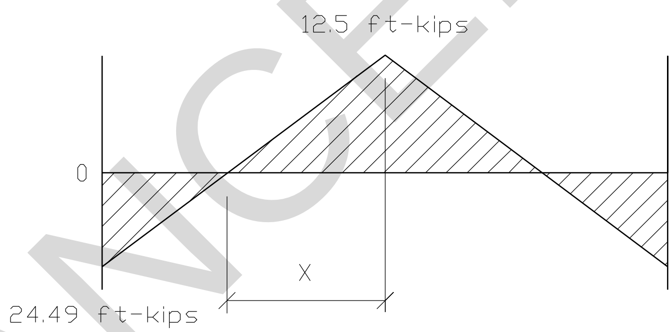
$$\frac{1}{4}(fM_n^-)_{@\text{joint}} = \frac{1}{4}(200^{\text{ft-k}}) = 50^{\text{ft-k}} < 147^{\text{ft-k}} = (fM_n^+) \quad (67.8\text{KN-m} < 199.3\text{KN-m})$$ **O.K.**

Check development of positive moment reinforcement at inflection points per ACI 318-95 Section 12.11;

$$l_d \le \frac{M_n}{V_u} + l_a$$ (EQ. 12-2 ACI 318-95)

where; $M_n = M_u = 147^{\text{ft-k}}/0.9 = 163^{\text{ft-k}}$ (221.0KN-m)
V_u; V_u is evaluated at the inflection point as follows;

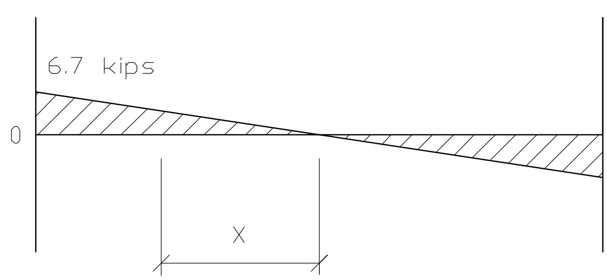
Inflection point is located from the beam centerline using similar triangles;



1-ft-kip = 1.356KN-m

$$x = \frac{21.5'/2}{(24.5^{\text{ft-k}} + 12.5^{\text{ft-k}})}(12.5^{\text{ft-k}}) = 3.63' \quad (1.11\text{m})$$

Similarly, V_u is determined at the inflection point;



1-kip = 4.448KN

$$V_u = \frac{6.7^k}{(21.5'/2)}(3.63') = 2.26^k \quad (10.05\text{KN})$$

Also,

$$l_a = \text{minimum}(d, 12d_b)$$
$$d = 21.31\text{"} \quad (541.3\text{mm})$$
$$12d_b = 12(0.625\text{"}) = 7.5\text{"} \quad (190.5\text{mm}) \qquad \text{(governs)}$$

$$\therefore l_d \leq \frac{163^{ft-k}}{2.26^{k}}(12\text{"}/1') + 7.5\text{"} = 873\text{"} \quad (22.17\text{X}10^3 \text{ mm})$$

$$l_d = \frac{f_y \alpha \beta \gamma}{25\sqrt{f_c'}} d_b$$ where; $\alpha = 1.3$ (> than 12" (304.8mm) conc. per ACI 318-95 Section 12.2.2)

$\beta = 1.0$ (uncoated reinforcement)

$\gamma = 0.8$ (#5 (15M) bars)

$$\therefore l_d = \frac{60{,}000\text{psi}(1.3)1.0(0.8)}{25\sqrt{4{,}000\text{psi}}}(0.625\text{"}) = 24.7\text{"} \quad (637.4\text{mm})$$

$$l_d = 24.7\text{"} < 873\text{"} = \frac{M_u}{V_u} + l_a$$ **O.K.**

Negative moment at mid span;
There is no negative moment occuring at mid span. Therefore, sreel will be governed by detailing requirements. Per ACI 318-95 Section 21.3.2.1, at least two bars are required.

Try 3 #6 (~20M) bars (assuming two of the top steel bars at the column face are to be cut-off); A_s = $3(0.44\text{-in}^2) = 1.32\text{-in}^2$ $(0.85\text{X}10^3 \text{ mm}^2)$, and d = 21.25" (539.8mm)

Check capacity;

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{1.32 - \text{in}^2(60\text{ksi})}{0.85(4\text{ksi})18\text{"}} = 1.29\text{"} \quad (32.8\text{mm})$$

$$\phi M_n = 0.9(1.32 - \text{in}^2)60\text{ksi}\left(21.25\text{"} - \frac{1.29\text{"}}{2}\right)(1'/12\text{"}) = 122^{ft-k} \quad (165.4\text{KN-m})$$

$$\frac{1}{4}(\phi M_n^-)_{@joint} = \frac{1}{4}200^{ft-k} = 100^{ft-k} < 122^{ft-k} = \phi M_n \quad (135.6\text{KN-m} < 165.4\text{KN-m})$$ **O.K.**

Therefore, choose 3 #6 (~20M) bars top steel at mid span

Determine cut-off point for two #6 top steel bars in accordance with ACI 318-95 section 12.10;

Determine the critical section;
By inspection, the governing load combination is 4a ($U = 1.386D + Q_E + 0.5L$). This combination governs because it requires the longest span from column face for the steel. At column face the moment capacity of the beam was calculated as $\phi M_n = 191^{ft-kips}$ (259.0KN-m), and at mid span the required strength is M_u = $1.386(12.5^{ft-kips}) = 17.3^{ft-kips}$ (23.5KN-m).

The slope of the moment diagram is;

$$\frac{dM_u}{dx} = \frac{-191^{ft-k} - (1.386(12.5^{ft-kips}))}{21.5'/2} = -19.4^{k} \quad (86.29\text{KN})$$

The capacity of the 2 remaining #6 bars was previously calculated at $\phi M_n = 122^{ft-kips}$ (165.4KN-m). Therefore, the distance from the face of column to where $M_u = \phi M_n = 122^{ft-kips}$ is;

$$x = \frac{122^{ft-k} - 191^{ft-k}}{-19.4^{ft-k}} = 3.6', \text{ Say } 4' \quad (1.22\text{m})$$

Determine the extension past the critical section;

$$\frac{l_d}{d_b} = \frac{f_y \alpha \beta \gamma}{25\sqrt{f_c'}}$$ where; α = 1.3 (> than 12" (304.8mm) conc. per ACI 318-95 Section 12.2.2)

β = 1.0 (uncoated reinforcement)
γ = 1.0 (#6 (~20M) bars)

$$\therefore l_d = \frac{60{,}000\text{psi}(1.3)1.0(0.8)}{25\sqrt{4{,}000\text{psi}}}(0.75") = 29.6" = 2.47' < 4' \quad (0.75\text{m} < 1.22\text{m})$$

48" + d = 48" + 21.25" = 69.3", Say 5'-9" (1.75m) (governs)
48" + 12d_b = 48" + 12(0.75") = 57" (1.45m)

Therefore, cut off 2 #6 (~20M) top steel bars at 5'-9" (1.75m) from face of column

Design for shear;
Determine the strength reduction factor in accordance with ACI 318-95 Section 9.3.4;

Nominal shear strength $= \phi V_n \le V_u$

By inspection, load combination 4a (U = 1.386D + Q_E + 0.5L) provides the largest factored shear force;

$$(V_u)_{max} = 1.386(6.75^k) + 14.4^k = 23.8^k \quad (105.9\text{KN})$$

The shear corresponding to the development of the nominal flexural strength of the beam is;

$$V_e = \frac{M_{pr1} + M_{pr2}}{L} \pm \frac{W}{2}$$

where; L =21.5' (6.56m) (beam clear span)
M_{pr1} and M_{pr2} are calculated as follows using ϕ = 1.0, and f_s = 1.25 f_y;

$$M_{pr} = 1.25A_s f_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{1.25A_s f_y}{0.85 f_c' b}$$

W/2 = gravity load reaction = 21.5'(w_{FD}/2)

At the beam left side using top bars;

$$a = \frac{1.25(2.20 - \text{in}^2)60\text{ksi}}{0.85(4\text{ksi})18"} = 2.70" \quad (68.6\text{mm})$$

$$\therefore M_{pr1} = 1.25(2.20 - \text{in}^2)60\text{ksi}\left(21.25" - \frac{2.70"}{2}\right)(1'/12") = 274^{\text{ft-k}} \quad (371.5\text{KN-m})$$

At the beam right side using bottom bars;

$$a = \frac{1.25(1.55 - \text{in}^2)60\text{ksi}}{0.85(4\text{ksi})18"} = 1.90" \quad (48.3\text{mm})$$

$$\therefore M_{pr2} = 1.25(1.55 - \text{in}^2)60\text{ksi}\left(21.81" - \frac{1.90"}{2}\right)(1'/12") = 202^{\text{ft-k}} \quad (273.9\text{KN-m})$$

$$\text{Also, } \frac{W}{2} = \frac{L(1.386w_{FD})}{2} = \frac{21.5'(1.386(0.618\text{klf}))}{2} = 9.20^k \quad (40.9\text{KN})$$

$$(V_e)_{max} = \frac{274^{\text{ft-k}} + 202^{\text{ft-k}}}{21.5'} + \frac{9.20^k}{2} = 26.7^k \quad (118.8\text{KN})$$

$$(V_u)_{max} = 23.8^k < 26.7^k = (V_e)_{max} \quad (105.9\text{KN} < 118.8\text{KN})$$ **Therefore, ϕ = 0.6**

Determine if V_c = 0 per ACI 318-95 Section 21.3.4.2;

Condition (1); $(V_e)_{max} = 26.7^k > 11.9^k = \frac{1}{2}(V_u)_{max}$ (118.8KN > 52.9KN)

Condition (2); From the computer RISA-2D analysis, the largest axial load in a floor beam is 11.45^k (50.9KN).

$$\frac{A_g f_c'}{20} = \frac{24"(18")4\text{ksi}}{20} = 86.4^k > 11.45^k \quad (384.3\text{KN} > 50.9\text{KN})$$ **Therefore, V_c = 0**

Design stirrups;

Note: There are 5 top and bottom longitudinal bars, and per ACI 318-95 Section 7.10.5.3, three stirrup legs are required in order to insure that every alternate bar is provided lateral support. Therefore, a single hoop with an extra interior cross tie will be used.

Spacing based on strength requirements;

$$V_s = \frac{A_v f_y d}{s} \qquad \text{(EQ. 11-15 ACI 318-95)}$$

$$V_u = (V_e)_{max} = \phi V_n = \phi V_s$$

within 2d of column face; 2d = 2(21.81") = 43.6" Say 4' (1.22m)

$$s = \frac{\phi A_v f_y d}{(V_e)_{max}} = \frac{0.6(3(0.11 - in^2))60ksi(21.81")}{26.7^k} = 9.70" \quad (246.4mm)$$

Spacing based on detail requirements;

s = d/4 = 21.25"/4 = 5.31" (134.9mm)

s = 8(0.625") = 5" (127.0mm) (governs)

s = 24(0.375") = 9" (228.6mm)

s = 12" (304.8mm)

Therefore, within 4-ft. (1.22m) of column face, provide stirrups consisting of 3 legs of #3 (~10M) bars at 5-in. (127.0mm) o. c.

For the remainder of the beam;

Spacing based on strength requirements;

At 4-ft (1.22m) from column face;

$$(V_e)_{@2h} = (V_e)_{max} - \frac{(V_e)_{max} - (V_e)_{min}}{L} 2h$$

where; $(V_e)_{max} = 26.7^k$ (calculated previously)

$$(V_e)_{min} = \frac{274^{ft-k} + 197^{ft-k}}{21.5'} - \frac{9.20^k}{2} = 17.3^k \quad (77.0KN)$$

$$(V_e)_{@2h} = 26.7^k - \left(\frac{26.7^k - 17.3^k}{21.5'}\right)4' = 25^k \quad (111.2KN)$$

$$\therefore s = \frac{0.6(3(0.11 - in^2))60ksi(21.25")}{25^k} = 10.1" \quad (256.5mm) \qquad \text{(governs)}$$

Spacing based on detail requirements;

s = d/2 = 21.25"/2 = 10.6" (269.2mm)

$$A_v = 50\frac{b_w s}{f_y} \qquad \text{(EQ. 11-13 ACI 318-95)}$$

$$s = \frac{A_v f_y}{50 b_w} = \frac{3(0.11 - in^2)60,000psi}{50(18")} = 22" \quad (558.8mm)$$

For the remainder of the beam, provide stirrups consisting of 3 legs of #3 (~10M) bars at 10-in. (254.0mm) o.c. max

Check transverse reinforcement requirements at longitudinal bar cut-offs;

Determine shear at the cut-off point;

$(V_u)_{max}$ at column face = 23.8^k (105.9KN) (previously calculated)

$(V_u)_{max}$ at beam mid span = 14.4^k (64.1KN) ($1.0Q_E = 1.0(14.4^k)$)

Therefore, at 5'-9" from column face;

$$(V_u)_{@5'-9"} = 23.8^k - \left(\frac{23.8^k - 14.4^k}{21.5'}\right)5.75' = 21.3^k \quad (94.7KN)$$

Note: The cut-off point occurs outside of the length identified in ACI 318-95 Section 21.3.3.1 (i.e., 2h from column face). Therefore, ϕV_c can be included in the section capacity ϕV_n.

$$\phi V_s = \frac{\phi A_v f_y d}{s} = \frac{0.6(3(0.11 - in^2))60ksi(21.25")}{10"} = 25.2^k \quad (112.1KN)$$

$$V_c = 2\sqrt{f'_c}\,b_w d$$ (EQ. 11-3 ACI 318-95)

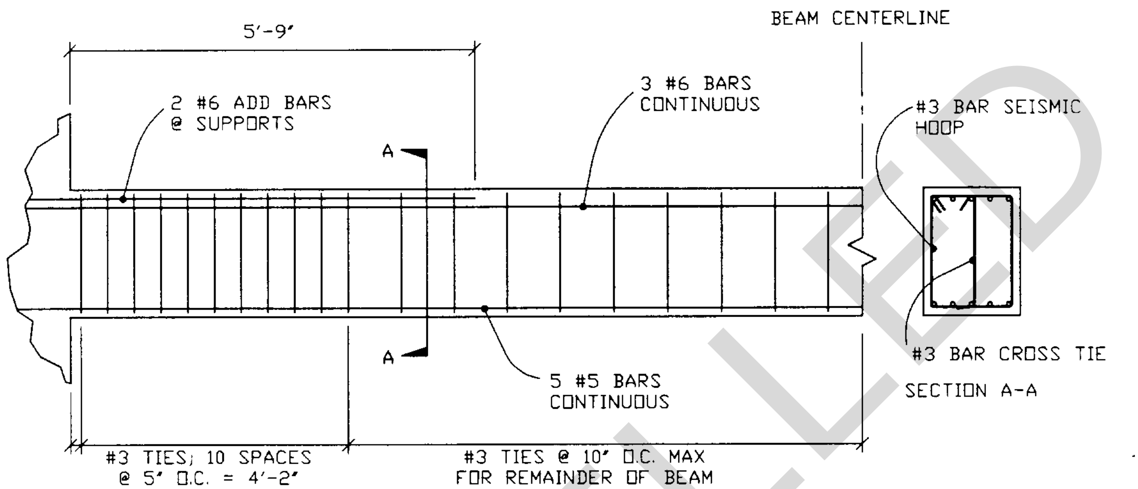
$$\phi V_c = 0.6\left(2\sqrt{4{,}000\text{psi}}\right)18''(21.25'')(1^k/1{,}000^{lb}) = 29^k \quad (129.0\text{KN})$$

$$\phi V_n = \phi(V_c + V_s) = 0.6\left(29^k + 25.2^k\right) = 32.5^k \quad (144.6\text{KN})$$

$$V_u = 18.8^k < 21.7^k = \frac{2}{3}(32.5^k) = \frac{2}{3}\phi V_n$$ **O.K.**

Therefore, no additional transverse reinforcement is required at bar cut-offs

Therefore, the second and third floor beam design is as follows;



#3 bar ~ 10M bar
#5 bar = 15M bar
#6 bar ~ 20M bar
1-ft = 0.30m
1-in = 25.4mm

Design roof level moment frame beams;

Design for flexure;
Negative moment at face of column;
By inspection, the governing load combination is 4a ($U = 1.386D + Q_E + 0.5L$);

$$M_u^- = 1.386(-25.67^{ft-kips}) + (-45.59^{ft-kips}) = -81.2^{ft-kips} \quad (110.1\text{KN-m})$$

Assume j = 0.9, and d = h – 2.5" = 16" – 2.5" = 13.5" (342.9mm)

$$(A_s)_{trial} = \frac{M_u}{\phi f_y jd} = \frac{81.2^{ft-kips}(12''/1')}{0.9(60\text{ksi})0.9(13.5'')} = 1.49 - \text{in}^2 \quad (0.96\text{X}10^3\ \text{mm}^2)$$

Try 5 #5 (15M) bars; $A_s = 5(0.31\text{-in}^2) = 1.55\text{-in}^2 > 1.49\text{-in}^2$ ($1.00\text{X}10^3\ \text{mm}^2 > 0.96\text{X}10^3\ \text{mm}^2$)
Determine d (assuming #3 (~10M) bars for transverse reinforcement);

d = 16" – 1.5" – 0.375" – (0.625"/2) = 13.81" (350.8mm)

Check capacity;

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{A_s f_y}{0.85 f'_c b}$$

$$a = \frac{1.55 - \text{in}^2(60\text{ksi})}{0.85(4\text{ksi})18''} = 1.52'' \quad (38.6\text{mm})$$

$$\phi M_n = 0.9(1.55 - \text{in}^2)60\text{ksi}\left(13.81'' - \frac{1.52''}{2}\right)(1'/12'') = 91^{ft-kips} \quad (123.4\text{KN-m})$$

$$\phi M_n = 91^{ft-kips} > 72.9^{ft-kips} = M_u \quad (12.34\text{KN-m} > 98.8\text{KN-m})$$ **O.K.**

Check crack control of flexural reinforcement per ACI 318-95 Section 10.6;

$$z = f_s\sqrt[3]{d_cA} \le 145^{k/in} \quad (25.4KN/m) \qquad \text{(EQ. 10-5 ACI 318-95)}$$

where; $f_s = 0.60(60ksi) = 36ksi$ (248.2MPa)
$d_c = 16" - 13.81" = 2.19"$ (55.6mm)
$A = 18"(2)(16"-13.81")/5 = 15.8\text{-}in^2$ ($10.2X10^3$ mm^2)

$$\therefore z = 36ksi\sqrt[3]{2.19"(15.8-in^2)} = 117^{k/in} < 145^{k/in} \quad (20.49KN/mm < 25.4KN/mm) \qquad \textbf{O.K.}$$

Check minimum reinforcement per ACI 318-95 Sections 10.5, and 21.3.2.1;

$\rho_{min} = 0.00333$ (previously calculated)

$$\rho = \frac{1.55-in^2}{16"(13.81")} = 0.00702 > 0.00333 = \rho_{min} \qquad \textbf{O.K.}$$

Check upper limit of reinforcement per ACI 318-95 Sections 10.3.3, and 21.3.2.1;
By inspection, $P_u < \phi P_n < 0.10f_c'A_g$. Therefore, ρ must be less then $0.75\rho_b$ or 0.025.

and $0.75\rho_b = 0.0214 < 0.025$ (previously calculated)

$$\therefore \rho = \frac{1.55-in^2}{16"(13.81")} = 0.00701 < 0.0214 = \rho_{max} \qquad \textbf{O.K.}$$

Therefore, choose 5 #5 (15M) top bars at column face

Positive moment at face of column;
By inspection, the governing load combination is 5a ($U = 0.714D + Q_E$);

$$M_u^+ = 0.714(22.62^{ft-kips}) + 45.59^{ft-kips} = +61.7^{ft-kips} \quad (83.7KN\text{-}m)$$

Assume j = 0.9, and d = h – 2.5" = 16" – 2.5" = 13.5" (342.9mm)

$$(A_s)_{trial} = \frac{M_u}{\phi f_y jd} = \frac{61.7^{ft-kips}(12"/1')}{0.9(60ksi)0.9(13.5")} = 1.13-in^2 \quad (0.73X10^3\ mm^2)$$

Try 5 #5 (15M) bars; $A_s = 5(0.31\text{-}in^2) = 1.55\text{-}in^2$ ($1.00X10^3$ mm^2)
Determine d (assuming #3 (~10M) bars for transverse reinforcement);

d = 16" – 1.5" – 0.375" – (0.625"/2) = 13.8" (350.5mm)

Check capacity;

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{A_s f_y}{0.85f_c'b}$$

$$a = \frac{1.55-in^2(60ksi)}{0.85(4ksi)18"} = 1.52" \quad (38.6mm)$$

$$\phi M_n = 0.9(1.55-in^2)60ksi\left(13.8" - \frac{1.52"}{2}\right)(1'/12") = 91^{ft-kips} \quad (123.4KN\text{-}m)$$

$$\phi M_n = 91^{ft-kips} > 61.7^{ft-kips} = M_u \quad (123.4KN\text{-}m > 83.7KN\text{-}m) \qquad \textbf{O.K.}$$

Check crack control of flexural reinforcement per ACI 318-95 Section 10.6;

$$z = f_s\sqrt[3]{d_cA} \le 145^{k/in} \quad (25.4KN/mm) \qquad \text{(EQ. 10-5 ACI 318-95)}$$

where; $f_s = 0.60(60ksi) = 36ksi$ (248.2MPa)
$d_c = 16" - 13.8" = 2.20"$ (55.9mm)
$A = 18"(2)(16"-13.8")/5 = 15.8\ \text{–}in^2$ ($10.19X10^3$ mm^2)

$$\therefore z = 36ksi\sqrt[3]{2.20"(15.8-in^2)} = 118^{k/in} < 145^{k/in} \quad (20.7KN/mm < 25.4KN/mm) \qquad \textbf{O.K.}$$

Check minimum reinforcement per ACI 318-95 Sections 10.5, and 21.3.2.1;

$\rho_{min} = 0.00333$ (previously calculated)

$$\rho = \frac{1.55-in^2}{16"(13.8")} = 0.0070 > 0.00333 = \rho_{min} \qquad \textbf{O.K.}$$

Check upper limit of reinforcement per ACI 318-95 Sections 10.3.3, and 21.3.2.1;
By inspection, $P_u < \phi P_n < 0.10f_c'A_g$. Therefore, ρ must be less then $0.75\rho_b$ or 0.025.

$0.75\rho_b = 0.0214$ (previously calculated) and $0.75\rho_b = 0.0214 < 0.025$

$$\therefore \rho = \frac{1.55-\text{in}^2}{16''(13.8'')} = 0.0070 < 0.0214 = \rho_{max}$$ **O.K.**

Check moment strength at face of joint per ACI 318-95 Section 21.3.2.2;

$$\frac{1}{2}(\phi M_n^-)_{@joint} = \frac{1}{2}(91^{ft-kips}) = 45.5^{ft-kips} < 91^{ft-kips} = (\phi M_n^+) \quad (61.7\text{KN-m} < 123.4\text{KN-m})$$

O.K.

Therefore, choose 5 #5 (15M) bottom bars at column face

Positive moment at mid span;
Because the longitudinal reinforcement is close to the minimum requirement, it is not possible to terminate any bars along the beams length. By inspection, all other detailing and strength requirements are satisfied.

Therefore, choose 5 #5 (15M) bottom bars at mid span

Negative moment at mid span;
As was demonstrated for the second and third floor beams, it is possible to cut off two of the 5 #5 (15M) top bars. However, in order to reduce the number of repetitive calculations it is decided to continue the 5 #5 (15M) bars along the member length. By inspection, all detailing and strength requirements are satisfied.

Therefore, choose 5 #5 (15M) top bars at mid span

Design for shear;
Determine the strength reduction factor in accordance with ACI 318-95 Section 9.3.4;

Nominal shear strength $= \phi V_n \le V_u$

By inspection, load combination 4a ($U = 1.386D + Q_E + 0.5L$) provides the largest factored shear force;

$$(V_u)_{max} = 1.386(7.0^k) + 4.22^k = 13.9^k \quad (61.8\text{KN})$$

The shear corresponding to the development of the nominal flexural strength of the beam is;

$$V_e = \frac{M_{pr1} + M_{pr2}}{L} \pm \frac{W}{2}$$

where; L =21.5' (beam clear span)

M_{pr1} and M_{pr2} are calculated as follows using $\phi = 1.0$, and $f_s = 1.25\ f_y$;

$$M_{pr} = 1.25A_s f_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{1.25A_s f_y}{0.85f_c' b}$$

W/2 = gravity load reaction = 21.5'(w_{RD}/2) (6.56m)

Since $M_{pr1} = M_{pr2}$;

$$a = \frac{1.25(1.55-\text{in}^2)60\text{ksi}}{0.85(4\text{ksi})18''} = 1.90'' \quad (48.3\text{mm})$$

$$\therefore M_{pr1} = M_{pr2} = 1.25(1.55-\text{in}^2)60\text{ksi}\left(13.8'' - \frac{1.90''}{2}\right)(1'/12'') = 125^{ft-kips} \quad (169.5\text{KN-m})$$

$$\text{Also, } \frac{W}{2} = \frac{L(1.386w_{RD})}{2} = \frac{21.5'(1.386(0.638\text{klf}))}{2} = 9.51^k \quad (42.3\text{KN})$$

$$(V_e)_{max} = \frac{125^{ft-kips} + 125^{ft-kips}}{21.5'} + \frac{9.51^k}{2} = 16.4^k \quad (92.9\text{KN})$$

$$(V_u)_{max} = 13.9^k < 16.4^k = (V_e)_{max} \quad (61.8\text{KN} < 72.9\text{KN})$$ **Therefore, ϕ = 0.6**

Determine if $V_c = 0$ per ACI 318-95 section 21.3.4.2;

Condition (1); $(V_e)_{max} = 16.4^k > 7.0^k = \frac{1}{2}(V_u)_{max}$ (72.9KN > 31.1KN)

Condition (2); From the computer RISA-2D analysis, the largest axial load in a roof beam is 16.8^k (74.7KN).

$$\frac{A_g f_c'}{20} = \frac{18"(16")4\text{ksi}}{20} = 57.6^k > 16.8^k \quad (256.2\text{KN} > 74.7\text{KN}) \qquad \textbf{Therefore, } \mathbf{V_c = 0}$$

Design stirrups;

Note: As in the case of the floor beams, there are 5 top and bottom longitudinal bars and per ACI 318-95 Section 7.10.5.3, three stirrup legs are required in order to insure that every alternate bar is provided lateral support. Therefore, a single hoop with an extra interior cross tie will be used.
Spacing based on strength requirements;

$$V_s = \frac{A_v f_y d}{s} \qquad \text{(EQ. 11-15 ACI 318-95)}$$

$$V_u = (V_e)_{max} = \phi V_n = \phi V_s$$

within 2d of column face; 2d = 2(13.88") = 27.8" Say 2.5' (0.76m)

$$s = \frac{\phi A_v f_y d}{(V_e)_{max}} = \frac{0.6(3(0.11-\text{in}^2))60\text{ksi}(13.8")}{16.4^k} = 10.0" \quad (3.05\text{m})$$

Spacing based on detail requirements;
s = d/4 = 13.8"/4 = 3.45" (87.6mm) (governs)
s = 8(0.50") = 4" (101.6mm)
s = 24(0.375") = 9" (228.6mm)
s = 12" (304.8mm)

Therefore, within 2.5-ft. (0.76m) of column face, provide stirrups consisting of 3 legs of #3 (~10M) bars at 3.5-in. (88.9mm) o. c.

For the remainder of the beam;
Spacing based on strength requirements;
At 2.5-ft from column face;

$$(V_e)_{@2h} = (V_e)_{max} - \frac{(V_e)_{max} - (V_e)_{min}}{L} 2h$$

where; $(V_e)_{max} = 16.4^k$ (72.9KN) (calculated previously)

$$(V_e)_{min} = \frac{125^{\text{ft-kips}} + 125^{\text{ft-kips}}}{21.5'} - \frac{9.51^k}{2} = 6.9^k \quad (30.7\text{KN})$$

$$(V_e)_{@2h} = 16.4^k - \left(\frac{16.4^k - 6.9^k}{21.5'}\right) 2.5' = 15.3^k \quad (68.1\text{KN})$$

$$\therefore s = \frac{0.6(3(0.11-\text{in}^2))60\text{ksi}(13.8")}{15.3^k} = 10.7" \quad (271.8\text{mm})$$

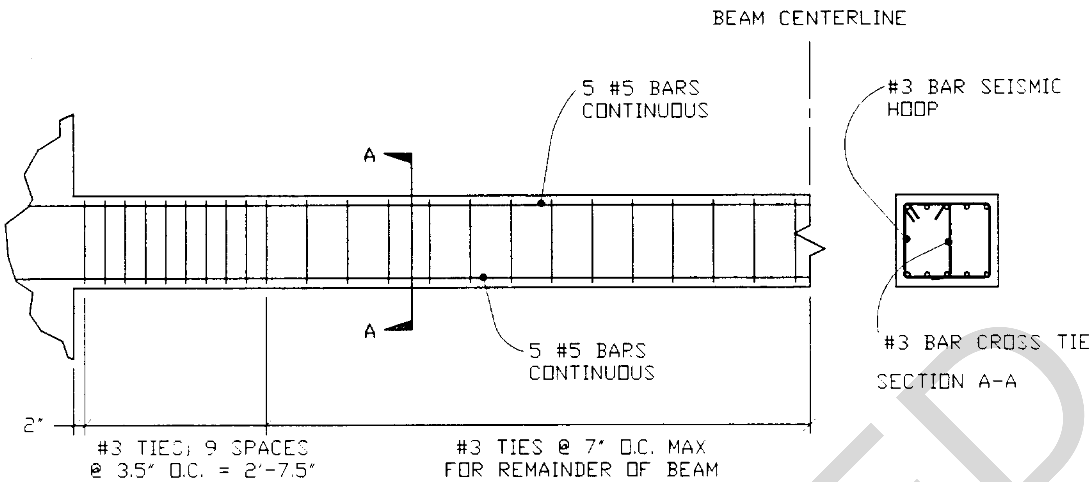
Spacing based on detail requirements;
s = d/2 = 13.8"/2 = 6.9" (175.3mm) (governs)

$$A_v = 50\frac{b_w s}{f_y} \qquad \text{(EQ. 11-13 ACI 318-95)}$$

$$s = \frac{A_v f_y}{50 b_w} = \frac{3(0.11-\text{in}^2)60{,}000\text{psi}}{50(18")} = 22" \quad (558.8\text{mm})$$

For the remainder of the beam, provide stirrups consisting of 3 legs of #3 (~10M) bars at 7-in. (177.8mm) o.c. max

Therefore, the roof beam design is as follows;



#3 bar ~ 10M bar
#5 bar = 15M bar
1-ft = 0.30m
1-in = 25.4mm

Design splices for beams;

Longitudinal reinforcement for beams shall be spliced using welded splices or mechanical connectors in conformance with ACI 318-95 Section 21.2.6.1. Only alternate bars in each layer of longitudinal reinforcement shall be spliced at a section. The center to center distance between splices of adjacent bars shall be 24-in. or more.

Design columns;

General;
The columns support only their own self weight, the weight of windows, and some minor infill. Therefore, the columns will first be checked to see if they are true columns or instead may be proportioned only for flexure.

Determine if ACI 318-95 Section 21.4 applies;
This section applies only if the column is proportioned to resist an axial compressive force exceeding $A_g f_c'/10$. All columns have the same cross sectional area therefore it is only necessary to check the most heavily loaded column. From the RISA-2D analysis the most heavily loaded column is and end column located at the first story ($E = 32.55^k$, and $D = 32.58^k$). By inspection, the controling load combination is 4a ($U = 1.386D + Q_E + 0.5L$);

$$P_u = 1.386(32.58^k) + 32.55^k = 77.7^k \quad (345.6KN)$$

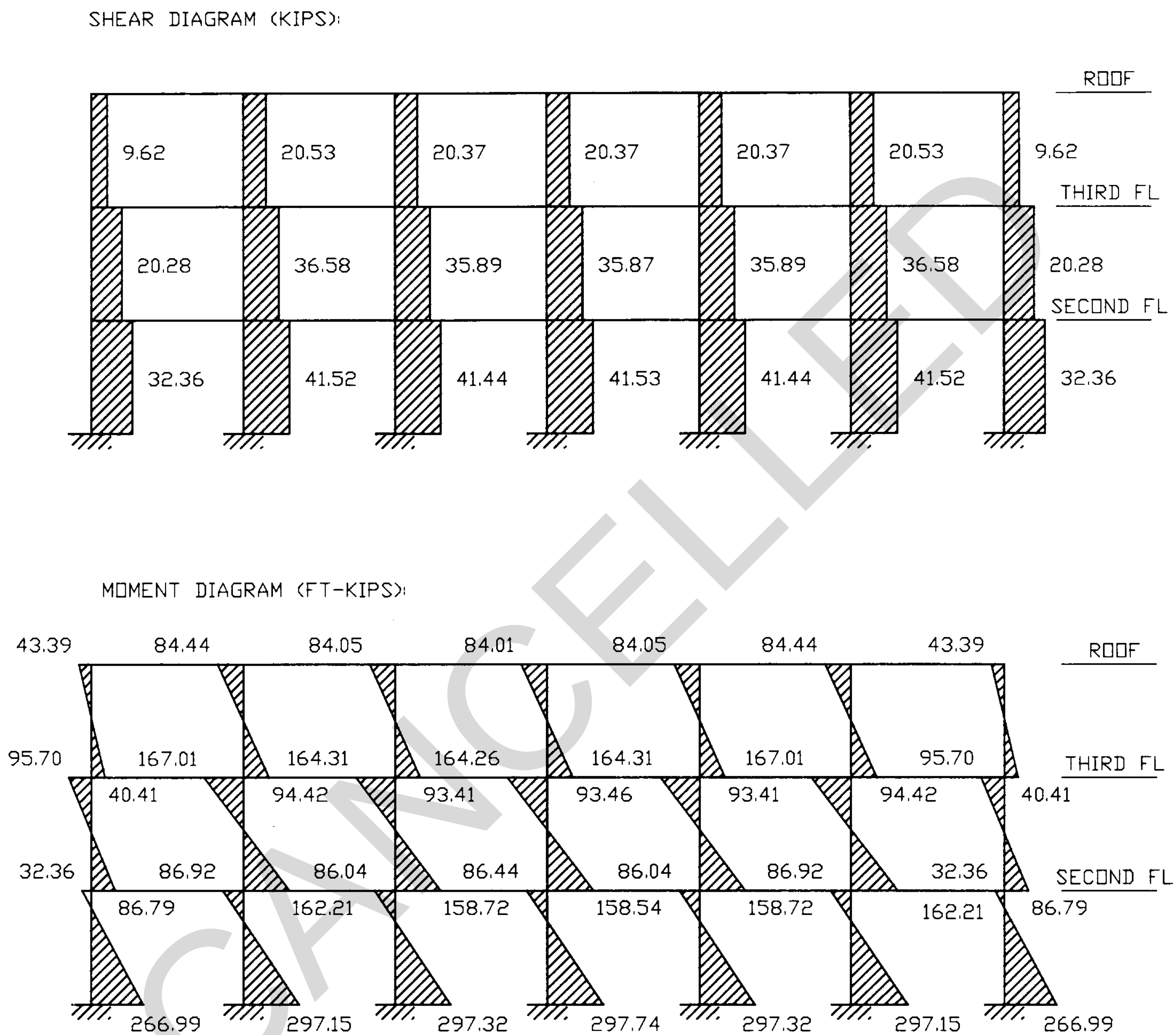
$$\frac{A_g f_c'}{10} = \frac{18"(24")4ksi}{10} = 173^k \quad (769.5KN)$$

$$P_u = 77.7^k < 173^k = \frac{A_g f_c'}{10} \quad (345.6KN < 769.5KN)$$

Therefore, columns shall be proportioned primarily for flexure

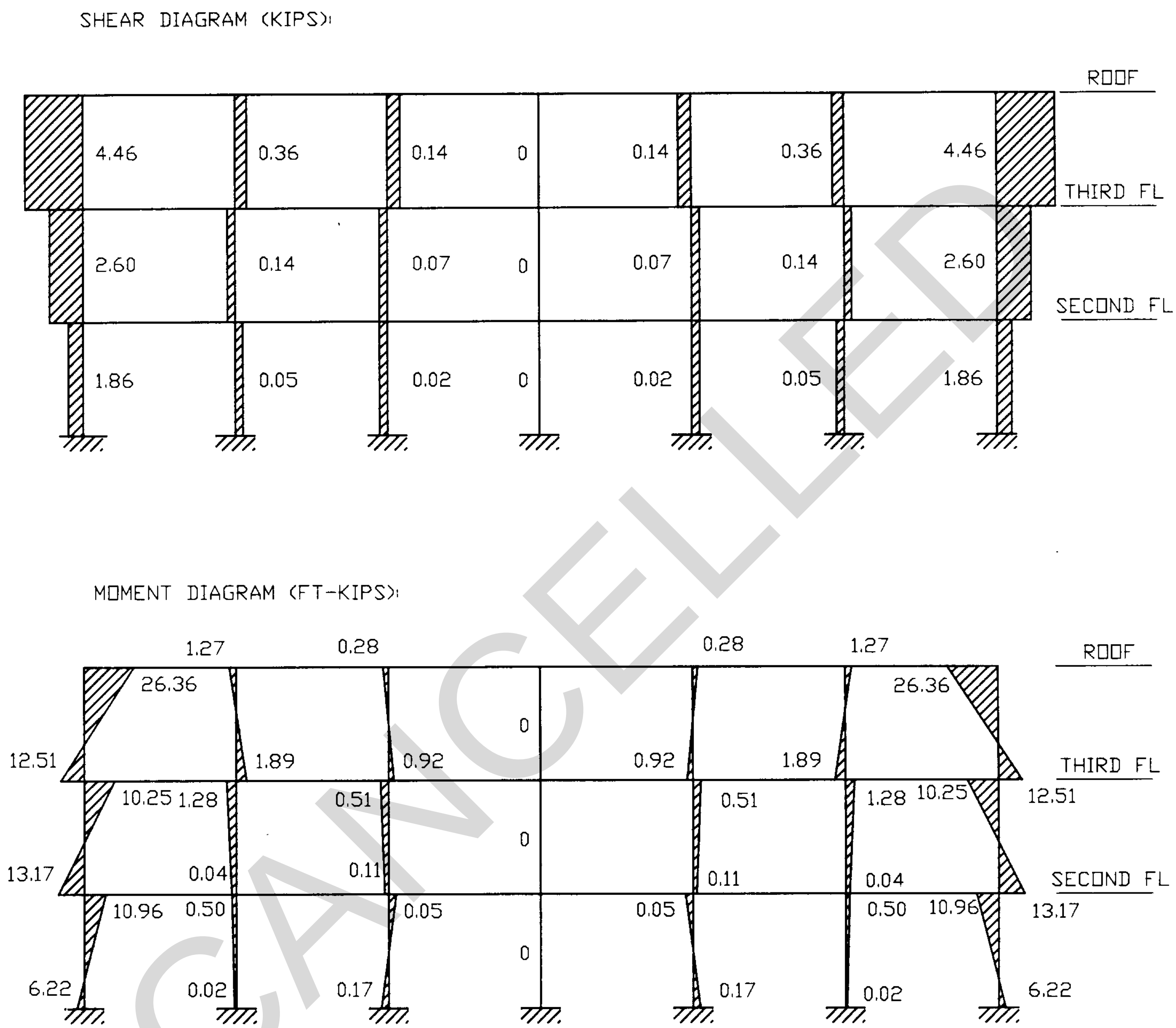
Note: It is good practice to apply the strong column/weak beam criteria of ACI 318-95 Section 21.4.2.2 even though it is not required. Therefore, in this solution, ACI 318-95 Section 21.4.2.2 shall be condsidered.

The moment and shear diagrams for the columns are shown in Figures 13 and 14 for lateral loads and gravity loads respectively. Inspection of these diagrams shows that there is relatively little variation in forces amongst columns in a story level with the exception of the end columns at the second and third story



Note: For metric equivalent; 1-kip =4.48KN, 1-ft-kip = 1.356KN-m

Figure 13. Shear and moment diagrams for frame columns due to lateral loading



Note: For metric equivalent; 1-kip =4.48KN, 1-ft-kip = 1.356KN-m

Figure 14. Shear and moment diagrams for frame columns due to gravity loading

level. Therefore, one design will be produced for the first story columns, one for the interior second story columns, one for the interior third story columns, one design for the second and third story end columns.

Design columns for first story level;
Determine design loads;
By inspection, the governing load combination is 4a for all loads ($U = 1.386D + Q_E + 0.5L$);

$$M_u = 1.386(6.22^{\text{ft-kips}}) + 298^{\text{ft-kips}} = 307^{\text{ft-kips}} \quad (416.3\text{KN-m})$$

$$V_u = 1.386(1.86^k) + 41.53^k = 44.1^k \quad (196.2\text{KN})$$

Determine if ACI 318-95 section 21.4.2.2 governs;

$$\sum M_e \geq \left(\frac{6}{5}\right)\sum M_g \qquad \text{(EQ. 21-1 ACI 318-95)}$$

For first story columns at the second floor beams;

$$\sum M_e = 2(\phi M_n)_{col} \geq \frac{6}{5}(147^{\text{ft-kips}} + 200^{\text{ft-kips}}) = 416.4^{\text{ft-kips}} \quad (564.6\text{KN-m})$$

$$\Rightarrow (\phi M_n)_{col} \geq 208.2^{\text{ft-kips}} \quad (282.3\text{KN-m})$$

However, $M_u = 307^{\text{ft-kips}} > 208^{\text{ft-kips}}$ (416.3KN-m > 282.3KN-m). Therefore, strong column/weak beam criteria does not govern design.
Assume j = 0.9, and d = h – 2.5" = 24" – 2.5" = 21.5" (546.1mm)

$$(A_s)_{trial} = \frac{M_u}{\phi f_y jd} = \frac{307^{\text{ft-kips}}(12''/1')}{0.9(60\text{ksi})0.9(21.5'')} = 3.53\text{-in}^2 \quad (2.28\text{X}10^3\ \text{mm}^2)$$

Try 6 #7 (20M) bars (on both faces of the column); $A_s = 6(0.60\text{-n}^2) = 3.60\text{-in}^2$ ($2.32\text{X}10^3$ mm^2)
Determine d (assuming #3 (~10M) bars for transverse reinforcement);

d = 24" – 2" – 0.375" – (0.875"/2) = 21.19" (538.2mm)

By inspection, the 6 #7 (~20M) bars can fit evenly around and through the 5 #6 (~20M) bars of the beam longitudinal reinforcement.
Check capacity;

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{3.60\text{-in}^2(60\text{ksi})}{0.85(4\text{ksi})18''} = 3.53'' \quad (89.7\text{mm})$$

$$\phi M_n = 0.9(3.60\text{-in}^2)60\text{ksi}\left(21.19'' - \frac{3.53''}{2}\right)(1'/12'') = 315^{\text{ft-k}} \quad (427.1\text{KN-m})$$

$$\phi M_n = 315^{\text{ft-kips}} > 307^{\text{ft-kips}} = M_u \quad (427.1\text{KN-m} > 416.3\text{KN-m}) \qquad \textbf{O.K.}$$

Check minimum reinforcement per ACI 318-95 Sections 10.5, and 21.3.2.1;

$\rho_{min} = 0.00333$ (calculated previously in the beam design)

$$\rho = \frac{3.60\text{-in}^2}{18''(21.19'')} = 0.00944 > 0.00333 = \rho_{min} \qquad \textbf{O.K.}$$

Check upper limit of reinforcement per ACI 318-95 Sections 10.3.3, and 21.3.2.1;

$\rho_{max} = 0.0214$ (calculated previously in the beam design)

$\rho = 0.00944 < 0.0214 = \rho_{max}$ **O.K.**

Check crack control of flexural reinforcement per ACI 318-95 Section 10.6;

$$z = f_s\sqrt[3]{d_c A} \leq 145^{k/in} \quad (25.4\text{KN/m}) \qquad \text{(EQ. 10-5 ACI 318-95)}$$

where; fs = 0.6(60ksi) = 36ksi (248.2MPa)
dc = 24" – 21.19" = 2.81" (71.4mm)
A = 18"(2)(24" – 21.19")/6 = 16.86-in^2 ($10.87\text{X}10^3$ mm^2)

$$\therefore z = 36\text{ksi}\sqrt[3]{2.81''(16.86\text{-in}^2)} = 130^{k/in} < 145^{k/in} \quad (22.8\text{KN/mm} < 25.4\text{KN/mm}) \qquad \textbf{O.K.}$$

Therefore, provide 6 #7 (~20M) longitudinal bars on opposite faces of the column

Design for shear;

Determine the strength reduction factor in accordance with ACI 318-95 Section 9.3.4;

Nominal shear strength $= \phi V_n \le V_u = 44.1^k$ (196.2KN)

The shear corresponding to the development of the nominal flexural strength of the column is;

$$V_e = \frac{M_{pr1} + M_{pr2}}{L}$$

where; L =9.25' (2.82m) (column clear span)

$M_{pr1} = M_{pr2}$ and is calculated as follows using $\phi = 1.0$, and $f_s = 1.25\ f_y$;

$$M_{pr} = 1.25 A_s f_y \left(d - \frac{a}{2}\right), \text{ where } a = \frac{1.25 A_s f_y}{0.85 f_c^{'} b}$$

Therefore;

$$a = \frac{1.25(3.60 - \text{in}^2)60\text{ksi}}{0.85(4\text{ksi})18"} = 4.41" \quad (112.0\text{mm})$$

$$\therefore M_{pr1} = 1.25(3.60 - \text{in}^2)60\text{ksi}\left(21.19" - \frac{4.41"}{2}\right)(1'/12") = 427^{\text{ft-k}} \quad (579.0\text{KN-m})$$

$$\therefore (V_e)_{max} = \frac{2(427^{\text{ft-k}})}{9.25'} = 92.3^k \quad (410.6\text{KN})$$

$$(V_u)_{max} = 44.1^k < 92.3^k = (V_e)_{max} \quad (196.2\text{KN} < 410.6\text{KN})$$

Therefore, $\phi = 0.6$

Determine if $V_c = 0$ per ACI 318-95 Section 21.3.4.2;

Condition (1); $(V_e)_{max} = 92.3^k > 22.05^k = \frac{1}{2}(V_u)_{max}$ (410.6KN > 98.1KN)

Condition (2); As previously mentioned, the largest axial load in a first story column is 77.7^k.

$$P_u = 1.386(32.58^k) + 32.55^k = 77.7^k \quad (345.6\text{KN})$$

$$\frac{A_g f_c^{'}}{20} = \frac{24"(18")4\text{ksi}}{20} = 86.4^k > 77.7^k \quad (384.3\text{KN} > 345.6\text{KN}$$

Therefore, $V_c = 0$

Design stirrups;

Note: There are 6 longitudinal bars, and per ACI 318-95 Section 7.10.5.3, four stirrup legs are required in order to insure that every alternate bar is provided lateral support. Therefore, a single hoop with two extra interior cross ties will be used.

Spacing based on strength requirements;

$$V_s = \frac{A_v f_y d}{s} \qquad \text{(EQ. 11-15 ACI 318-95)}$$

$$V_u = (V_e)_{max} = \phi V_n = \phi V_s$$

within 2d of the end of the beam; 2d = 2(21.19") = 42.4" Say 4' (1.22m)

$$s = \frac{\phi A_v f_y d}{(V_e)_{max}} = \frac{0.6(4(0.11 - \text{in}^2))60\text{ksi}(21.19")}{92.3^k} = 3.64" \text{ Say } 3.5" \quad (88.9\text{mm}) \qquad \text{(governs)}$$

Spacing based on detail requirements;

s = d/4 = 21.19"/4 = 5.30" (134.6mm)

s = 8(0.875") = 7.0" (177.8mm)

s = 24(0.375") = 9" (228.6mm)

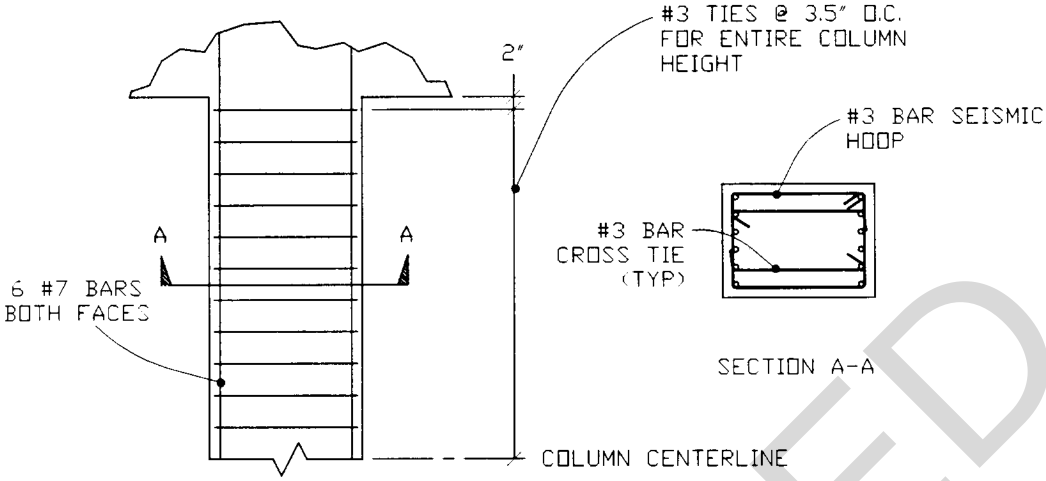
s = 12" (304.8mm)

For the remainder of the column;

Since Ve is constant over the length of the column s = 3.5-in. (88.9mm) as determined by strength requirements governs.

Therefore, provide stirrups consisting of 4 legs of #3 (~10M) bars at 3.5-in. (88.9mm) o.c. over the entire column length

Therefore, the first floor column design is as follows;
Note: Due to their low axial load ($P_u < A_g f_c'/10$), columns are proportioned primarily to resist flexure, and are not compression members. Therefore, the detailing requirements of ACI 318-95 Section 7.10 do not apply.



#3 bar ~ 10M bar
#7 bar ~ 20M bar
1-ft = 0.30m
1-in = 25.4mm

Design interior columns for the second story level;

Determine design loads;
By inspection, the governing load combination is 4a for all loads ($U = 1.386D + Q_E + 0.5L$);

$$M_u = 1.386(1.28^{\text{ft-kips}}) + 167.01^{\text{ft-kips}} = 169^{\text{ft-kips}} \quad (229.2\text{KN-m})$$

$$V_u = 1.386(0.14^k) + 36.58^k = 36.8^k \quad (163.7\text{KN})$$

Determine if ACI 318-95 Section 21.4.2.2 governs;

$$\sum M_e \geq \left(\frac{6}{5}\right)\sum M_g \qquad \text{(EQ. 21-1 ACI 318-95)}$$

For second story columns at the second or third floor beams;

$$\sum M_e = 2(\phi M_n)_{col} \geq \frac{6}{5}(147^{\text{ft-k}} + 200^{\text{ft-k}}) = 416.4^{\text{ft-k}} \quad (564.6\text{KN-m})$$

$$\Rightarrow (\phi M_n)_{col} \geq 208.2^{\text{ft-k}} \quad (282.3\text{KN-m})$$

Since $M_u = 169^{\text{ft-kips}} < 208^{\text{ft-kips}}$ (229.2KN-m < 282.3KN-m) strong column/weak beam criteria governs design.
Assume j = 0.9, and d = h – 2.5" = 24" – 2.5" = 21.5" (546.1mm)

$$(A_s)_{trial} = \frac{M_u}{\phi f_y jd} = \frac{208^{\text{ft-kips}}(12''/1')}{0.9(60\text{ksi})0.9(21.5'')} = 2.39\text{-in}^2 \quad (1.54\text{X}10^3\ \text{mm}^2)$$

Try 6 #6 (~20M) bars (on both faces of the column); $A_s = 6(0.44\text{-n}^2) = 2.64\text{-in}^2$ ($1.70\text{X}10^3$ mm^2)
Determine d (assuming #3 (~10M) bars for transverse reinforcement);
d = 24" – 2" – 0.375" – (0.750"/2) = 21.25" (539.8mm)
By inspection, the 6 #6 (~20M) bars can fit evenly around and through the 5 #6 (~20M) bars of the beam longitudinal reinforcement.
Check capacity;

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{2.64 - \text{in}^2(60\text{ksi})}{0.85(4\text{ksi})18''} = 2.59'' \quad (65.8\text{mm})$$

$$\phi M_n = 0.9(2.64-in^2)60ksi\left(21.25"-\frac{2.59"}{2}\right)(1'/12") = 237^{ft-kips} \quad (321.4KN\text{-}m)$$

$$\phi M_n = 237^{ft-kips} > 208^{ft-kips} = M_u \quad (321.4KN\text{-}m > 282.3KN\text{-}m)$$ **O.K.**

Check minimum reinforcement per ACI 318-95 Sections 10.5, and 21.3.2.1;

$\rho_{min} = 0.00333$ (calculated previously in the beam design)

$$\rho = \frac{2.64-in^2}{18"(21.25")} = 0.00690 > 0.00333 = \rho_{min}$$ **O.K.**

Check upper limit of reinforcement per ACI 318-95 Sections 10.3.3, and 21.3.2.1;

$\rho_{max} = 0.0214$ (calculated previously in the beam design)

$\rho = 0.00690 < 0.0214 = \rho_{max}$ **O.K.**

Check crack control of flexural reinforcement per ACI 318-95 Section 10.6;

$$z = f_s\sqrt[3]{d_c A} \le 145^{k/in} \quad (25.4KN/mm)$$ (EQ. 10-5 ACI 318-95)

where; $fs = 0.6(60ksi) = 36ksi$ (248.2MPa)

$dc = 24" - 21.25" = 2.75"$ (69.9mm)

$A = 18"(2)(24" - 21.25")/6 = 16.50\text{-}in^2$ ($10.64X10^3$ mm^2)

$$\therefore z = 36ksi\sqrt[3]{2.75"(16.50-in^2)} = 128^{k/in} < 145^{k/in} \quad (22.4KN/mm < 25.4KN/mm)$$ **O.K.**

Therefore, provide 6 #6 (~20M) longitudinal bars on opposite faces of the column

Design for shear;

Determine the strength reduction factor in accordance with ACI 318-95 Section 9.3.4;

Nominal shear strength $= \phi V_n \le V_u = 36.8^k$ (163.7KN)

The shear corresponding to the development of the nominal flexural strength of the column is;

$$V_e = \frac{M_{pr1} + M_{pr2}}{L}$$

where; L =9.00' (2.75m) (column clear span)

$M_{pr1} = M_{pr2}$ and is calculated as follows using $\phi = 1.0$, and $f_s = 1.25\ f_y$;

$$M_{pr} = 1.25A_s f_y\left(d-\frac{a}{2}\right), \text{ where } a = \frac{1.25A_s f_y}{0.85f_c' b}$$

Therefore;

$$a = \frac{1.25(2.64-in^2)60ksi}{0.85(4ksi)18"} = 3.24" \quad (82.3mm)$$

$$\therefore M_{pr1} = 1.25(2.64-in^2)60ksi\left(21.25"-\frac{3.24"}{2}\right)(1'/12") = 324^{ft-kips} \quad (439.3KN\text{-}m)$$

$$\therefore (V_e)_{max} = \frac{2(324^{ft-kips})}{9.0'} = 72^k \quad (320.3KN)$$

$$(V_u)_{max} = 36.8^k < 72^k = (V_e)_{max} \quad (163.7 < 320.3KN)$$ **Therefore, ϕ = 0.6**

Determine if $V_c = 0$ per ACI 318-95 Section 21.3.4.2;

Condition (1); $(V_e)_{max} = 72^k > 18.4^k = \frac{1}{2}(V_u)_{max}$ (320.3KN > 81.8KN)

Condition (2); From the RISA-2D analysis, the largest axial load in an interior second story column is;

$$P_u = 1.386(31.91^k) + 0.64^k = 49^k \quad (218.0KN)$$

$$\frac{A_g f_c'}{20} = \frac{24"(18")4ksi}{20} = 86.4^k > 49^k \quad (384.3KN > 218.0KN)$$ **Therefore, $V_c = 0$**

Design stirrups;

Note: There are 6 longitudinal bars, and per ACI 318-95 Section 7.10.5.3, four stirrup legs are required in order to insure that every alternate bar is provided lateral support. Therefore, a single hoop with an two extra interior cross ties will be used.
Spacing based on strength requirements;

$$V_s = \frac{A_v f_y d}{s} \qquad \text{(EQ. 11-15 ACI 318-95)}$$

$$V_u = (V_e)_{max} = \phi V_n = \phi V_s$$

within 2d of the end of the beam; 2d = 2(21.25") = 42.5" Say 4' (1.22m)

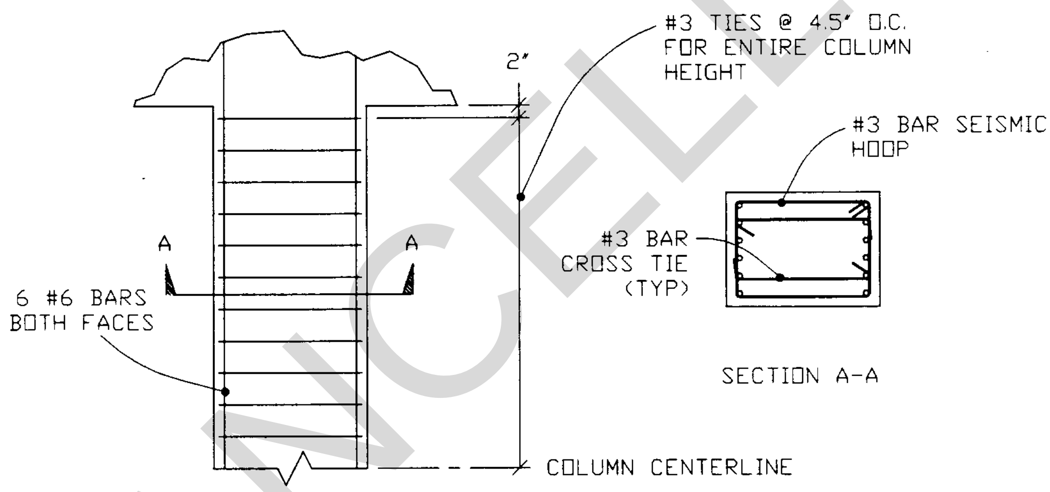
$$s = \frac{\phi A_v f_y d}{(V_e)_{max}} = \frac{0.6(4(0.11 - in^2))60ksi(21.25")}{72^k} = 4.68" \text{ Say } 4.5" \text{ (114.3mm)} \qquad \text{(governs)}$$

Spacing based on detail requirements;
s = d/4 = 21.25"/4 = 5.31" (138.9mm)
s = 8(0.750") = 6.0" (152.4mm)
s = 24(0.375") = 9" (228.6mm)
s = 12" (304.8mm)

For the remainder of the column;
Since Ve is constant over the length of the column s = 4.5-in. (114.3mm) as determined by strength requirements governs.

Therefore, provide stirrups consisting of 4 legs of #3 (~10M) bars at 4.5-in. o.c. over the entire column length

Therefore, the second story interior column design is as follows;
Note: Due to their low axial load ($P_u < A_g f_c'/10$), columns are proportioned primarily to resist flexure, and are not compression members. Therefore, the detailing requirements of ACI 318-95 Section 7.10 do not apply.



#3 bar ~ 10M bar
#6 bar ~ 20M bar
1-ft = 0.30m
1-in = 25.4mm

Design interior columns for the third story level;
Determine design loads;
By inspection, the governing load combination is 4a for all loads ($U = 1.386D + Q_E + 0.5L$);

$$M_u = 1.386(1.89^{ft-kips}) + 94.42^{ft-kips} = 97^{ft-kips} \text{ (131.5KN-m)}$$

$$V_u = 1.386(0.36^k) + 20.53^k = 21^k \text{ (93.4KN)}$$

Determine if ACI 318-95 Section 21.4.2.2 governs;

$$\sum M_e \geq \left(\frac{6}{5}\right)\sum M_g \qquad \text{(EQ. 21-1 ACI 318-95)}$$

For second story columns at the second or third floor beams;

$$\sum M_e = 2(\phi M_n)_{col} \geq \frac{6}{5}(147^{ft-kips} + 200^{ft-kips}) = 416.4^{ft-kips} \quad (564.6KN\text{-}m)$$

$$\Rightarrow (\phi M_n)_{col} \geq 208.2^{ft-k} \quad (282.3KN\text{-}m)$$

Since $M_u = 97^{ft\text{-}kips} < 208^{ft\text{-}kips}$ (131.5KN-m < 282.3KN-m) strong column/weak beam criteria governs design. Also, because the design moment is the same as used at the second story level, the same design will result.

Therefore, use the same design for the third story interior columns as used at the second story interior columns

Design end columns for the second and third story levels;
Determine design loads;
By inspection, the governing load combination is 4a for all loads ($U = 1.386D + Q_E + 0.5L$), and the worst case column occurs at the second floor level;

$$M_u = 1.386(10.25^{ft-kips}) + 95.7^{ft-kips} = 110^{ft-kips} \quad (149.2KN\text{-}m)$$

$$V_u = 1.386(2.60^k) + 20.28^k = 24^k \quad (106.8KN)$$

Determine if ACI 318-95 section 21.4.2.2 governs;

$$\sum M_e \geq \left(\frac{6}{5}\right)\sum M_g \qquad \text{(EQ. 21-1 ACI 318-95)}$$

For second story columns at the second or third floor beams;

$$\sum M_e = 2(\phi M_n)_{col} \geq \frac{6}{5}(200^{ft-kips}) = 240^{ft-kips} \quad (325.4KN\text{-}m)$$

$$\Rightarrow (\phi M_n)_{col} \geq 120^{ft-kips} \quad (162.7KN\text{-}m)$$

Since $M_u = 110^{ft\text{-}kips} < 120^{ft\text{-}kips}$ strong column/weak beam criteria governs design.
Assume j = 0.9, and d = h – 2.5" = 24" – 2.5" = 21.5" (546.1mm)

$$(A_s)_{trial} = \frac{M_u}{\phi f_y jd} = \frac{120^{ft-kips}(12"/1')}{0.9(60ksi)0.9(21.5")} = 1.38 - in^2 \quad (0.89X10^3\ mm^2)$$

Try 6 #5 (15M) bars (on both faces of the column); $A_s = 6(0.31\text{-}n^2) = 1.86\text{-}in^2$ ($1.20X10^3\ mm^2$)
Determine d (assuming #3 (~10M) bars for transverse reinforcement);

d = 24" – 1.5" – 0.375" – (0.625"/2) = 21.81" (554.0mm)

By inspection, the 6 #5 (15M) bars can fit evenly around and through the 5 #6 (~20M) bars of the beam longitudinal reinforcement.
Check minimum reinforcement per ACI 318-95 Sections 10.5, and 21.3.2.1;

$\rho_{min} = 0.00333$ (calculated previously in the beam design)

$$\rho = \frac{1.86 - in^2}{18"(21.81")} = 0.00473 > 0.00333 = \rho_{min} \qquad \textbf{O.K.}$$

Check capacity;

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{A_s f_y}{0.85 f'_c b}$$

$$a = \frac{1.86 - in^2(60ksi)}{0.85(4ksi)18"} = 1.82" \quad (46.2mm)$$

$$\phi M_n = 0.9(1.86 - in^2)60ksi\left(21.81" - \frac{1.82"}{2}\right)(1'/12") = 175^{ft-k} \quad (237.3KN\text{-}m)$$

$$\phi M_n = 175^{ft-k} > 120^{ft-k} = M_u \quad (237.3KN\text{-}m > 162.7KN\text{-}m) \qquad \textbf{O.K.}$$

Check upper limit of reinforcement per ACI 318-95 Sections 10.3.3, and 21.3.2.1;

$\rho_{max} = 0.0214$ (calculated previously in the beam design)

$\rho = 0.00473 < 0.0214 = \rho_{max}$ **O.K.**

Check crack control of flexural reinforcement per ACI 318-95 Section 10.6;

$$z = f_s \sqrt[3]{d_c A} \leq 145^{k/in} \quad (25.4KN/mm) \qquad \text{(EQ. 10-5 ACI 318-95)}$$

where; fs = 0.6(60ksi) = 36ksi (288.2MPa)

dc = 24" – 21.81" = 2.19" (55.6mm)

A = 18"(2)(24" – 21.81")/6 = 13.14-in^2 (8.48X10^3 mm^2)

$$\therefore z = 36ksi\sqrt[3]{2.19"(13.14 - in^2)} = 110^{k/in} < 145^{k/in} \quad (19.26KN/mm < 25.4KN/mm)$$ **O.K.**

Therefore, provide 6 #5 (15M) longitudinal bars on opposite faces of the column

Design for shear;

Determine the strength reduction factor in accordance with ACI 318-95 Section 9.3.4;

Nominal shear strength $= \phi V_n \leq V_u = 24^k$ (106.8KN)

The shear corresponding to the development of the nominal flexural strength of the column is;

$$V_e = \frac{M_{pr1} + M_{pr2}}{L}$$

where; L =8.71' (2.66m) (column clear span)

$M_{pr1} = M_{pr2}$ and is calculated as follows using $\phi = 1.0$, and $f_s = 1.25\ f_y$;

$$M_{pr} = 1.25A_s f_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{1.25A_s f_y}{0.85f'_c b}$$

Therefore;

$$a = \frac{1.25(1.86 - in^2)60ksi}{0.85(4ksi)18"} = 2.28" \quad (57.9mm)$$

$$\therefore M_{pr1} = 1.25(1.86 - in^2)60ksi\left(21.81" - \frac{2.28"}{2}\right)(1'/12") = 240^{ft-k} \quad (325.4KN\text{-}m)$$

$$\therefore (V_e)_{max} = \frac{2(240^{ft-k})}{8.71'} = 55^k \quad (244.6KN)$$

$$(V_u)_{max} = 24^k < 55^k = (V_e)_{max} \quad (106.8KN < 244.6KN)$$ **Therefore, ϕ = 0.6**

Determine if $V_c = 0$ per ACI 318-95 Section 21.3.4.2;

Condition (1); $(V_e)_{max} = 55^k > 12^k = \frac{1}{2}(V_u)_{max}$ (244.6KN > 53.4KN)

Condition (2); From the RISA-2D analysis, the largest axial load in an interior second floor column is;

$$P_u = 1.386(19.25^k) + 18.09^k = 44.8^k \quad (199.3KN)$$

$$\frac{A_g f'_c}{20} = \frac{24"(18")4ksi}{20} = 86.4^k > 44.8^k \quad (384.3KN > 199.3KN)$$ **Therefore, $V_c = 0$**

Design stirrups;

Note: There are 6 longitudinal bars, and per ACI 318-95 Section 7.10.5.3, four stirrup legs are required in order to insure that every alternate bar is provided lateral support. Therefore, a single hoop with an two extra interior cross ties will be used.

Spacing based on strength requirements;

$$V_s = \frac{A_v f_y d}{s} \qquad \text{(EQ. 11-15 ACI 318-95)}$$

$$V_u = (V_e)_{max} = \phi V_n = \phi V_s$$

within 2d of the end of the beam; 2d = 2(21.81") = 43.62" Say 4' (1.22m)

$$s = \frac{\phi A_v f_y d}{(V_e)_{max}} = \frac{0.6(4(0.11 - in^2))60ksi(21.81")}{55^k} = 6.28" \text{ Say } 6.0" \quad (152.4mm)$$

Spacing based on detail requirements;

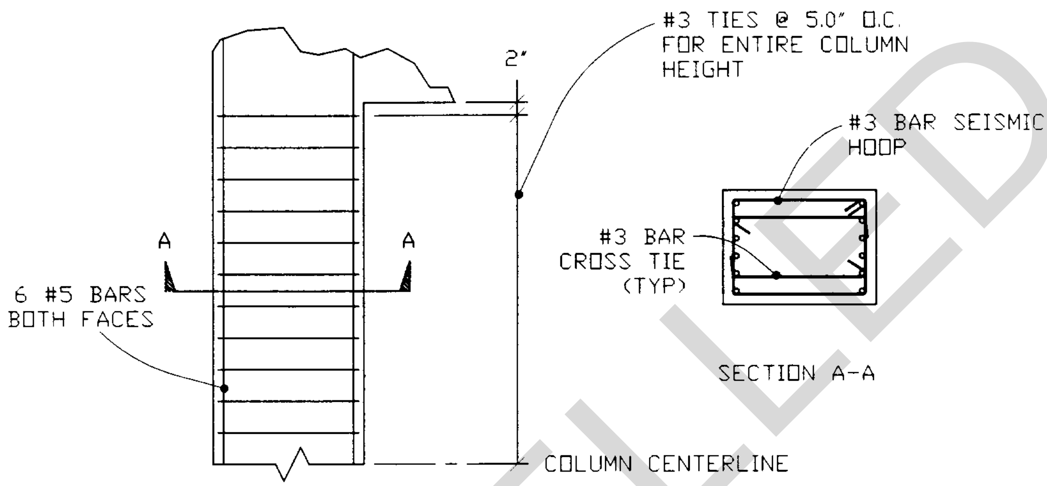
s = d/4 = 21.81"/4 = 5.5" (139.9mm)

s = 8(0.625") = 5.0" (127.0mm) (governs)
s = 24(0.375") = 9" (228.6mm)
s = 12" (304.8mm)

For the remainder of the column;
Since V_e is constant over the length of the column s = 5.0 -in. (127.0mm) as determined by strength requirements governs.

Therefore, provide stirrups consisting of 4 legs of #3 (~10M) bars at 5.0-in. (127.0mm) o.c. over the entire column length

Therefore, the second and third story end column design is as follows;
Note: Due to their low axial load ($P_u < A_g f_c'/10$), columns are proportioned primarily to resist flexure, and are not compression members. Therefore, the detailing requirements of ACI 318-95 Section 7.10 do not apply.



#3 bar ~ 10M bar
#6 bar ~ 20M bar
1-ft = 0.30m
1-in = 25.4mm

Design splices for columns;

Longitudinal reinforcement for columns shall be spliced using welded splices or mechanical connectors in conformance with ACI 318-95 Section 21.4.3.2. Splices are allowed only within the center half of the column, and only alternate bars will be spliced at a section. The center to center distance between splices of adjacent bars shall be 24-in. (0.61m) or more.

Design joints;

Note: All joints have the same cross-sectional dimensions. Therefore, the following design for transverse reinforcement applies to all joints at each floor level in the frame. Also, the shear strength check will be performed only once for the worst case joint in the frame.

Design transverse reinforcement for confinement;
Per ACI 318-95 Section 21.5.2.1, the total cross-sectional area of rectangular hoop reinforcement shall not be less than that given by the following equations;

$$A_{sh} = 0.3\left(\frac{sh_c f_c'}{f_{yh}}\right)\left[\left(\frac{A_g}{A_{ch}}\right) - 1\right] \qquad \text{(EQ. 21-3 ACI 318-95)}$$

$$A_{sh} = 0.09\frac{sh_c f_c'}{f_{yh}} \qquad \text{(EQ. 21-4 ACI 318-95)}$$

where; h_c = 24" - 2(2") – 0.375" = 19.63" (498.6mm)
f_{yh} = 60ksi (413.7MPa)

$f_c' = 4\text{ksi}$ (27.6MPa)
$A_g = 18"(24") = 432\text{-in}^2$ ($278.6\text{X}10^3$ mm^2)
$A_{ch} = (18" - 2(2"))(24"-2(2")) = 280\text{-in}^2$ ($180.6\text{X}10^6$ mm^2)
$A_{sh} = 4(0.11\text{-in}^2) = 0.44\text{-in}^2$ ($0.28\text{X}10^3$ mm^2)(using #3 (~10M) seismic hoops w/ 4 legs)

$$\therefore s = \frac{A_{sh}}{\left\{0.3\left(\frac{h_c f_c'}{f_{yh}}\right)\left[\left(\frac{A_g}{A_{ch}}\right)-1\right]\right\}} \frac{0.44-\text{in}^2}{\left\{0.3\left(\frac{19.63"(4\text{ksi})}{60\text{ksi}}\right)\left[\left(\frac{432-\text{in}^2}{280-\text{in}^2}\right)-1\right]\right\}} = 2.07" \quad \text{Say } 2" \ (50.8\text{mm})$$

$$\text{or} \quad s = \frac{A_{sh}}{\left\{0.09h_c\frac{f_c'}{f_{yh}}\right\}} = \frac{0.44-\text{in}}{\left\{0.09(19.63")\left(\frac{4\text{ksi}}{60\text{ksi}}\right)\right\}} = 3.74" \ (95.0\text{mm})$$

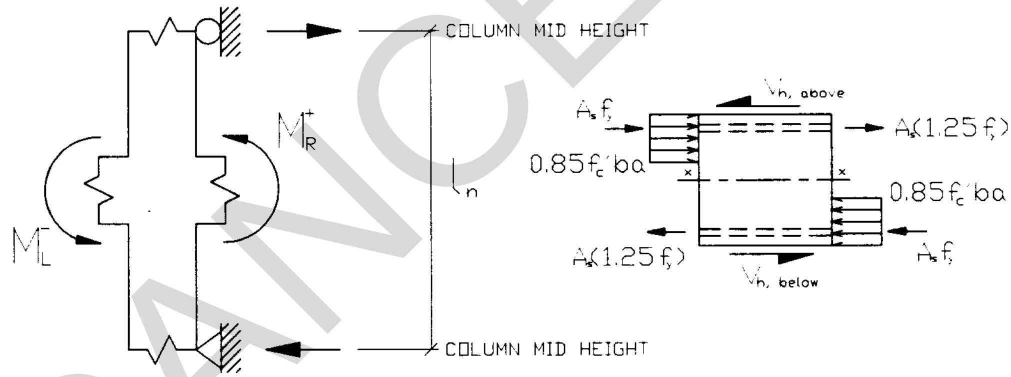
Provide stirrups consisting of 4 legs of #3 (~10M) bars at 2.0-in. (50.8mm) o.c. throughout joint

Determine (worst case) loading;

By inspection, the worst case loading occurs at an interior joint at the second floor level. It is assumed that beams in adjoining floors have formed plastic hinges at their junctions with the column, and that the end moments in the beams are resisted equally by the columns above and below the joint.

$$V_h \approx \frac{\sum (M_{pr})_{beams@joint}}{l_n}$$

where; $\Sigma(M_{pr})_{beams@joint} = 274^{ft\text{-}k} + 202^{ft\text{-}k} = 476^{ft\text{-}k}$ (645.5KN-m)
l_n = average of column clear spans between first and second floors
$l_n = (9.25' + 9')/2 = 9.13'$ (2.78m)



$$V_h = \frac{476^{ft-k}}{9.13'} = 52.1^k \ (231.7\text{KN})$$

$$T_1 = 1.25A_s^- f_y = 1.25(5(0.44-\text{in}^2))60\text{ksi} = 165^k \ (733.9\text{KN})$$

$$C_2 = T_2 = 1.25A_s^+ f_y = 1.25(2(0.31-\text{in}^2))60\text{ksi} = 46.5^k \ (206.8\text{KN})$$

Joint shear is evaluated at section x-x as follows;

$$\therefore V_{joint} = V_h - T_1 - C_2 = 52.1^k - 165^k - 46.5^k = -160^k \ (711.7\text{KN})$$

Determine shear capacity;
Per ACI 318-95 Section 21.5.3, the nominal shear strength of the joint shall be taken as;

$$\phi V_n = \phi V_c = \phi 12\sqrt{f_c'}A_j$$

where; $A_j = 18"(24") = 432\text{-in}^2$ ($278.6\text{X}10^3$ mm^2)

$$\phi = 0.85$$

$$\therefore \phi V_n = 0.85(12)\sqrt{4{,}000\text{psi}}\,432 - \text{in}^2 = 279^k \text{ (1.24MN)}$$

Check capacity of joints;

$$\phi V_n = 279^k > 160^k = V_{joint} \text{ (1.24MN > 711.7KN)}$$ **O.K.**

Determine embedment length of beam reinforcement within joint

Beam longitudinal steel will be anchored in the core of the end columns using a standard 90-deg hook. These bars consist of three sizes. The embedment length for each bar size is determined based on ACI 318-95 section 21.5.4 as follows;

$$l_{dh} = \frac{f_y d_b}{\left(65\sqrt{f_c'}\right)}$$ (EQ. 21-5 ACI 318-95)

where; l_{dh} shall not be less than $8d_b$ or 6-in. (152.4mm)
f_y = 60,000psi (413.7MPa)
d_b = 0.5" (12.7mm) for #4 (~10M) bars, 0.625" (15.9mm) for #5 (15M) bars, and 0.75" (19.1mm) for #6 (~20M) bars
fc' = 4,000psi

The embedment length for each bar size is calculated in the table below;

Bar Size (#)	Bar Diameter (in)	$8d_b$ (in)	$l_{dh} = f_y d_b/(65\text{sqrt}(f_c'))$ (in)	l_{dh} (in)
4	0.500	4.00	7.3	7.5
5	0.625	5.00	9.1	9.5
6	0.750	6.00	10.9	11

#4 bar ~ 10M bar
#5 bar = 15M bar
#6 bar ~ 20M bar
1-in = 25.4mm

Design diaphragm;

By inspection, the maximum diaphragm shear occurs at the landing along grid lines 1 or 2. At this location a 15-ft. (4.58m) length of diaphragm must transmit the shear to the shear wall on grid line 1. This shear has been previously determined to be $v_u = 34.1^k$ (151.7KN) and includes torsional effects.

Determine shear capacity of diaphragm; use f_c' = 4,000psi (27.6MPa), and f_y = 60ksi (413.7MPa)

Capacity of diaphragm is determined from ACI 318-95 Section 21.6.5.2;

$$V_n = A_{cv}(2\sqrt{f_c'} + \rho_n f_y)$$ (EQ. 21-6 ACI 318-95)

Try minimum reinforcement in conformance with ACI 318-95 Section 21.6.2;

$$\rho_{min} = 0.0018$$

$$\therefore \phi V_n = 0.85(2.5")(15')(12"/1')\left[2\sqrt{4{,}000\text{psi}} + 0.0018(60{,}000\text{psi})\right](1^k/1000^{lb}) = 89.7^k \text{ (399.0KN)}$$

$$\phi V_n = 89.7^k > 34.1^k = V_u \text{ (399.0KN > 151.7KN)}$$ **O.K.**

Therefore, design for minimum reinforcement;

Determine spacing of #4 (~10M) bars;

$$\rho_{min} = \frac{0.20 - \text{in}^2}{s(2.5")} = 0.0018 \Rightarrow s = 44.4" \text{ (1.13m)}$$

However, minimum spacing per ACI 318-95 Section 7.12.2.2 is;

s = 5 x (slab thickness) = 12.5" (317.5mm) (governs)

or s = 18" (457.2mm)

Also, per ACI 318-95 section 21.6.5.5, reinforcement must be equal in both directions.

Therefore, use #4 (~10M) bars at 12-in. (304.8mm) o.c. each way for slab reinforcement

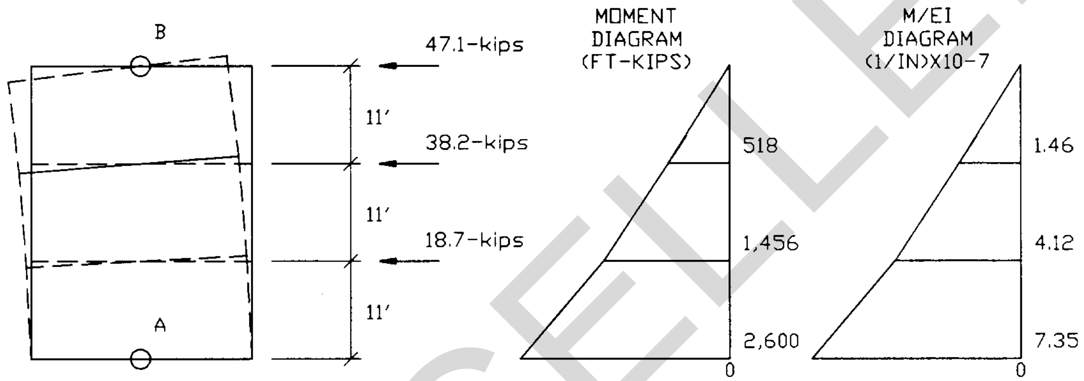
Design transverse coupling beams;

Transverse coupling beams connect transverse shear walls between grid lines B and C and were designed previously for gravity loads. These beams are considered flexible and are intended not to restrict the independent cantilever action of the walls. Instead, these beams serve only to transfer gravity loads between the walls acting in bearing. However, they must be checked to ensure that they can accommodate the in-plane rotations of the walls under lateral loads. The worst case condition occurs at the roof on grid lines 2 and 8 where the rotations are largest. The rotation of a shear wall at the roof level id determined by the moment area method.

$$\phi_{roof} - \phi_{base} = \int_A^B \frac{M_u}{E_c I_{cr}} dx$$

where; E_c = modulus of elasticity of cracked concrete = 3,605ksi (24.9×10^3 MPa)
$I_{cr} = 0.7 I_g$ = moment of inertia of cracked wall = $11{,}774 \times 10^3$-in^4 (4.90×10^{12} mm^4)

Note: $\phi_{base} = 0$



1-kip = 4.448KN   1-ft-kip = 1.356KN-m   1-in = 25.4mm

$$\phi_{roof} = \left[\frac{1}{2}(1.46) + (1.46) + \frac{1}{2}(4.12 - 1.46) + 4.12 + \frac{1}{2}(7.35 - 4.12)\right] 132" \text{x} 10^{-7} = 0.000122\text{rad} = 0.007 \deg$$

The moment acting in the beam due to this rotation will now be calculated. Due to symmetry, this moment is the same at both ends of the beam and puts the beam into reverse curnature. This moment will be added to the moment caused by gravity loads, and will then be checked against the beams flexural capacity. Additionally, to insure that a shear failure will not occur, the shear reinforcement will be revised to that required for the beams nominal flexural capacity per ACI 318-95 Section 21.3.4.1.

Using the moment area method, the relationship between the beams end rotation and the moment causing that rotation can be obtained;

$$2\phi = \frac{1}{2}\left(\frac{M}{E_c I_{cr}}\right)\frac{L}{2} \quad \text{or} \quad M = \frac{8\phi E_c I_{cr}}{L}$$

where; L = the beam length = 6' (1.83m)
E_c = 3,605ksi (24.9×10^3 MPa)
$I_{cr} = 0.35 I_g = 0.35(1/12)(17")(12")^3$ = 857-in^4 (356.7×10^6 mm^4)

$$\therefore M = \frac{8(0.000122)\text{rad}(3{,}605\text{ksi})857 - \text{in}^4}{6'(12"/1')} = 41.8^{\text{in-k}} \quad (56.7\text{KN-m})$$

The moment due to gravity loads was previously calculated as $M_u = 144^{\text{in-k}}$ (195.3KN-m), and the capacity was previously calculated as $\phi M_n = 344^{\text{in-k}}$ (466.5KN-m).

Therefore;

$$M_u = 41.8^{in-k} + 144^{in-k} = 186^{in-k} < 344^{in-k} = \phi M_n \quad (252.2\text{KN-m} < 466.8\text{KN-m}) \qquad \textbf{O.K.}$$

Design for shear;

In the gravity load design section of this problem, V_e was calculated as 25.8^k (114.8KN), and V_u was calculated as 12^k (53.4KN). It was also established that $\phi = 0.6$.
Determine if $V_c = 0$ per ACI 318-95 Section 21.3.4.2;

Condition (1); $V_e = 25.8^k > 6^k = \frac{1}{2}V_u$

Condition (2); By inspection, the axial load is negligible. **Therefore, $V_c = 0$**

Design stirrups;

$$V_s = \frac{A_v f_y d}{s} \qquad \text{(EQ. 11-15 ACI 318-95)}$$

$$V_u = (V_e)_{max} = \phi V_n = \phi V_s$$

within 2d of the end of the beam; 2d = 2(10.6") = 21.2" Say 2' (0.61m)

$$s = \frac{\phi A_v f_y d}{V_e} = \frac{0.6(2(0.11 - in^2))60ksi(10.6")}{25.8^k} = 3.25" \quad (82.6mm)$$

Spacing based on detail requirements;

s = d/4 = 10.6"/4 = =2.65" Say 2.5" (63.5mm) (governs)
s = 8(0.625") = 5" (127.0mm)
s = 24(0.375") = 9" (228.8mm)
s = 12" (304.8mm)

Therefore, within 2-ft. (0.61m) of the end of the beams, provide stirrups consisting of 2 legs of #3 (~10M) bars at 2.5-in. (63.5mm) o. c.

For the remainder of the beam;
Spacing based on strength requirements;
Since Ve is constant over the member length; s = 3.25" Say 3" (76.2mm) (governs)
Spacing based on detail requirements;

s = d/2 = 13.88"/2 = 6.94" (176.3mm)

$$A_v = 50\frac{b_w s}{f_y} \qquad \text{(EQ. 11-13 ACI 318-95)}$$

$$s = \frac{A_v f_y}{50 b_w} = \frac{3(0.11 - in^2)60{,}000psi}{50(18")} = 22" \quad (558.8mm)$$

For the remainder of the beam, provide stirrups consisting of 2 legs of #3 (~10M) bars at 3-in. (76.2mm) o.c. max

B-12 *Determine allowable drift and PΔ effect.*

Per TI 809-04 Table 6-1, the allowable interstory drift is $0.020h_{sx}$. The calculated story drifts are to be multiplied by the C_d factors listed in Table 7-1 before comparison to the allowable drift.

For both the transverse and the longitudinal directions, the allowable story drift is;

$$\delta_{allow} = 0.020(11')(12"/1') = 2.64" \quad (67.1mm)$$

Check drift in the transverse direction;

$C_d = 5.0$ (for specially reinforced concrete shear walls per Table 7-1)

The worst case condition occurs at walls along grid lines 2 or 8. The displacements at a level relative to ground are calculated using the moment area method as follows;

$$\Delta_{roof} = \left\{\frac{1}{2}(1.46)\frac{2}{3} + \frac{1}{2}(4.12-1.46)\frac{5}{3} + (1.46)\frac{3}{2} + \frac{1}{2}(7.35-4.12)\frac{8}{3} + (4.12)\frac{5}{2}\right\}(132)^2 x10^{-7} = 0.034 - \text{in}$$

(0.86mm)

$$\Delta_{2^{nd}\ floor} = \left\{\frac{1}{2}(4.12-1.46)\frac{2}{3} + (1.46)\frac{1}{2} + \frac{1}{2}(7.35-4.12)\frac{5}{3} + (4.12)\frac{3}{2}\right\}(132)^2 x10^{-7} = 0.018 - \text{in}$$

(0.46mm)

$$\Delta_{3^{rd}\ floor} = \left\{\frac{1}{2}(7.35-4.12)\frac{2}{3} + (4.12)\frac{1}{2}\right\}(132)^2 x10^{-7} = 0.006 - \text{in} \quad (0.15\text{mm})$$

Third story drift; $\delta_3 = \Delta_{roof} - \Delta_{3^{rd}\ floor} = 0.034\text{"} - 0.018\text{"} = 0.016 - \text{in}$ (0.41mm) (governs)

Second story drift; $\delta_2 = \Delta_{3^{rd}\ floor} - \Delta_{2^{nd}\ floor} = 0.018\text{"} - 0.006\text{"} = 0.012 - \text{in}$ (0.30mm)

First story drift; $\delta_1 = \Delta_{2^{nd}\ floor} - \Delta_{1^{st}\ floor} = 0.006\text{"} - 0.0\text{"} = 0.006 - \text{in}$ (0.15mm)

$$C_d x \delta_3 = 5.0(0.016\text{"}) = 0.08\text{"} < 2.64\text{"} = \delta_{allow} \quad (2.03\text{mm} < 67.1\text{mm})$$

Therefore, story drift is satisfied in the transverse direction

Check drift in the longitudinal direction;

$C_d = 5.5$ (for specially reinforced concrete moment frames per Table 7-1)

The displacements at a level relative to ground were calculated in the RISA-2D analysis and are as follows;

$\Delta_{roof} = 1.085 - \text{in}$ (27.6mm)
$\Delta_{2^{nd}\ floor} = 0.710 - \text{in}$ (18.0mm)
$\Delta_{3^{rd}\ floor} = 0.279 - \text{in}$ (7.1mm)

Third story drift; $\delta_3 = \Delta_{roof} - \Delta_{2^{nd}\ floor} = 1.085\text{"} - 0.710\text{"} = 0.375 - \text{in}$ (9.5mm)
Second story drift; $\delta_2 = \Delta_{2^{nd}\ floor} - \Delta_{3^{rd}\ floor} = 0.710\text{"} - 0.279\text{"} = 0.431 - \text{in}$ (10.9mm) (governs)
First story drift; $\delta_1 = \Delta_{2^{nd}\ floor} - \Delta_{1^{st}\ floor} = 0.279\text{"} - 0.0\text{"} = 0.279 - \text{in}$ (7.1mm)

$$C_d x \delta_3 = 5.5(0.431\text{"}) = 2.37\text{"} < 2.64\text{"} = \delta_{allow} \quad (60.2\text{mm} < 67.1\text{mm})$$

Therefore, story drift is satisfied in the longitudinal direction

Per FEMA 302 P-delta effects need not be considered when the following equation is equal to or less than 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \qquad \text{(EQ. 5.3.7.2-1 FEMA 302)}$$

where; P_x = total vertical design load at and above level x without load factors
Δ = story drift at level x
V_x = seismic story shear force at level x
h_{sx} = story height below level x

Check the requirements for P-delta effects in the transverse direction;

$$\theta_{\text{roof}} = \frac{(1{,}283^{k} + 0.20\text{ksf}(8{,}181 - \text{ft}^{2}))0.016''}{338^{k}(10.38')(12''/1')5.0} = 0.00022 << 0.10$$

$$\theta_{3^{\text{rd}}\text{ floor}} = \frac{(1{,}283^{k} + 1{,}573^{k} + (0.20 + 0.16)\text{ksf}(8{,}181 - \text{ft}^{2}))0.012''}{276^{k}(11')(12''/1')5.0} = 0.00038 << 0.10$$

$$\theta_{2^{\text{nd}}\text{ floor}} = \frac{(4{,}428^{k} + (0.20 + 2(0.16))\text{ksf}(8{,}181 - \text{ft}^{2}))0.006''}{138^{k}(10.25')(12''/1')5.0} = 0.00061 << 0.10$$

Therefore, P-delta effects need not be considered in the longitudinal direction

Check the requirements for P-delta effects in the longitudinal direction;

$$\theta_{\text{roof}} = \frac{(1{,}283^{k} + 0.20\text{ksf}(8{,}181 - \text{ft}^{2}))0.375''}{239^{k}(10.38')(12''/1')5.5} = 0.007 < 0.10$$

$$\theta_{3^{\text{rd}}\text{ floor}} = \frac{(1{,}283^{k} + 1{,}573^{k} + (0.20 + 0.16)\text{ksf}(8{,}181 - \text{ft}^{2}))0.431''}{195^{k}(11')(12''/1')5.5} = 0.018 << 0.10$$

$$\theta_{2^{\text{nd}}\text{ floor}} = \frac{(4{,}428^{k} + (0.20 + 2(0.16))\text{ksf}(8{,}181 - \text{ft}^{2}))0.279''}{98^{k}(10.25')(12''/1')5.5} = 0.037 << 0.10$$

Therefore, P-delta effects need not be considered in the transverse direction

H-3 CHAPEL

a. Introduction.

This design example illustrates the seismic design of a church building. The layout of the building is based on a typical military church structure.

(1) Purpose. The purpose of this example is to illustrate the design of a representative military building in an area of high seismicity, using the provisions of FEMA 302 as modified by this document.

(2) Scope. The scope of this example problem includes; the design of all major structural members such as steel gravity and moment framing, reinforced concrete shear walls and horizontal steel pipe bracing. The design of the foundations, nonstructural elements and their connections, and detail design of some structural elements such as reinforced concrete slabs (roof and slab on grade) were not considered part of the scope of this problem and are therefore not included (See Problem H-2 for the design of concrete floor and roof slab and Problem H-4 for the detailed design of steel moment connections)

b. Building Description.

(1) Function. This building functions as a Chapel with a capacity of more than 300 people.

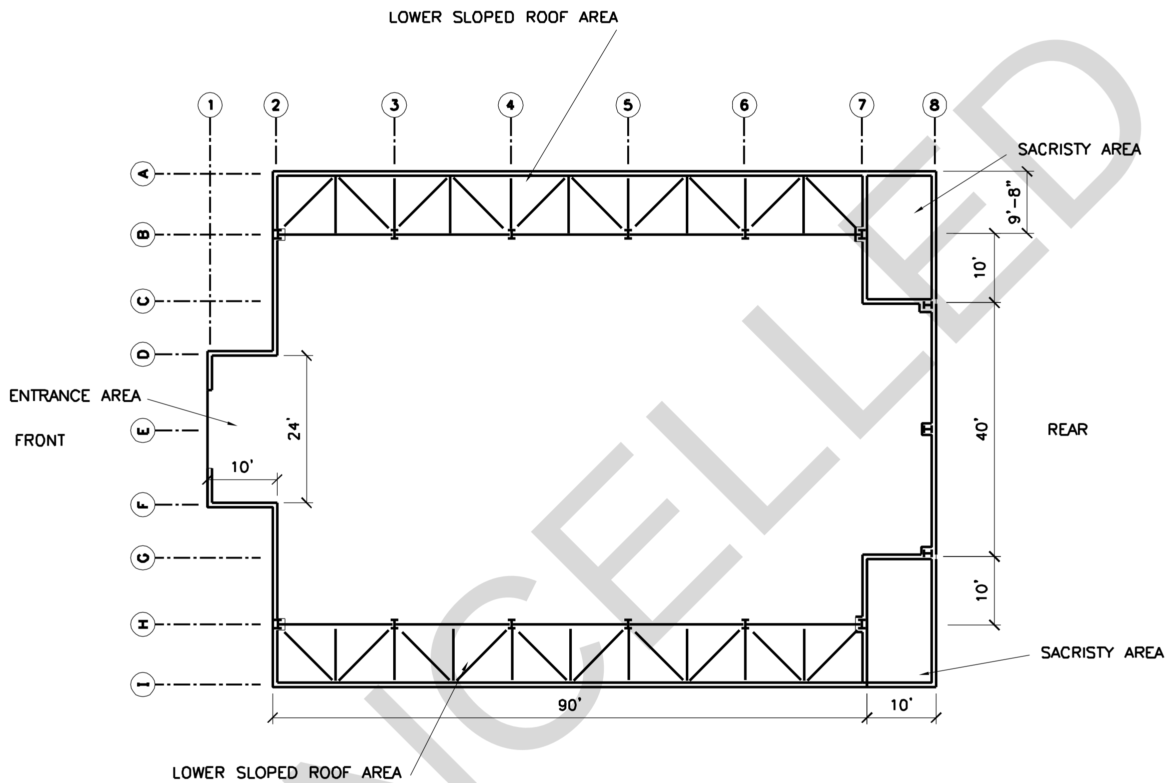
(2) Seismic Use Group. The Seismic Use Group is determined from Table 4-1. The primary occupancy of this structure is public assembly with a capacity greater than 300 persons. This type of occupancy places the building in Seismic Use Group II, Special Occupancy Structures. With the Seismic Use Group known, the Structural System Performance Objectives are obtained from Table 4-4. Structures in Seismic Use Group II are to be designed for Performance Level 2, Safe Egress. Ground Motion A (2/3 MCE) is to be used for Performance Objective 2A. The Minimum Analysis Procedure to be used is the Linear Elastic with R Factors and Linear Elastic with m Factors. The structure is designed first for Performance Objective 1A following the steps laid out in Table 4-5. After completion of the preliminary design, the enhanced performance objectives outlined in Table 4-6 for Performance Objective 2A are checked and the building design updated accordingly to meet those objectives.

(3) Configuration. The main chapel area has a high roof area (roof at ridge is 33'-3" high or 10.14m). There are low roof areas (10' in height or 3.05m) that run 90' (27.45m) on each side of the main open area. There are two sacristy areas at the rear end of the building that measure 10' x 19'-8" by 10' high (3.05m x 6.00m by 3.05m high)

(4) Structural systems. Steel transverse moment frames support the gravity loads from the high roof area. Metal decking spans over purlins spaced at 10' (3.05m), and the purlins span to the steel moment frames (spaced at 18' or 5.49m o.c.). In the low sloped roof areas that run parallel to the main high roof area, gravity loads are supported by metal decking, the decking spans between the shear walls along lines A & I and the window walls along lines B & H. The upper level concrete window/shear walls along lines B & H are supported by beams that span in the longitudinal direction between the columns of the transverse moment frames. Gravity loads at the sacristy areas are supported by reinforced concrete slabs (slab design not included in scope of problem).

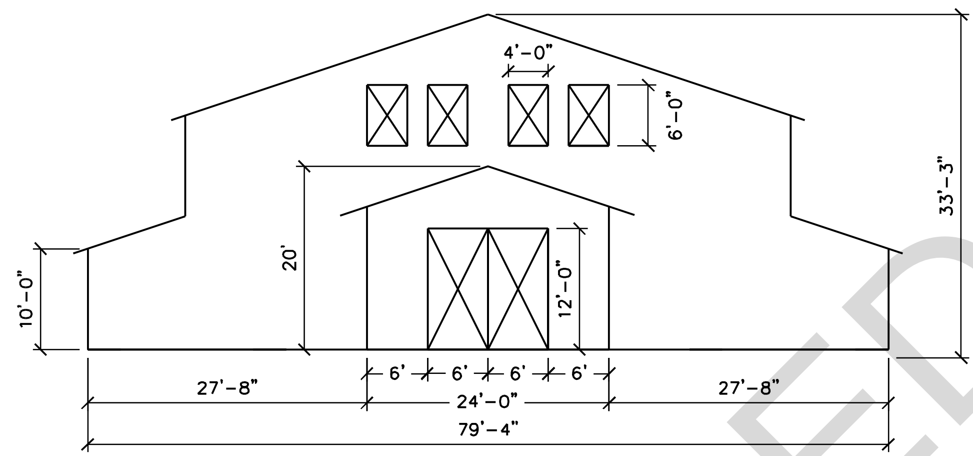
The primary lateral force resisting elements for this structure consists of specially reinforced concrete shear walls. In the longitudinal direction the concrete shear walls resist the entire shear force. Seismic forces from the upper roof area are transferred from the diaphragm to the shear walls along lines B & H. These walls are supported by steel beams at the level of the lower sloped roof diaphragms and are not continuous to the ground. Horizontal pipe bracing transfers the shear from these walls to the exterior shear walls. In the transverse direction lateral forces are also resisted by a combination of concrete shear walls and steel moment frames. The upper roof diaphragm is assumed to act as a flexible diaphragm. Therefore, inertial forces are resisted by the concrete shear walls and steel moment frames based on tributary areas. The moment frames are assumed to be braced by the horizontal bracing at the level of the lower sloped roof areas (along wall lines B & H). The horizontally braced diaphragms of the low sloped area are assumed to act as rigid diaphragms (the diaphragm action falls between flexible and rigid. Analyzing the diaphragms as rigid produce was found to produce the most conservative design for the shear walls and horizontal bracing). The sacristy roof diaphragms are composed of reinforced concrete slabs which are assumed to act as rigid diaphragms.

(5) Choice of materials. The concrete shear walls are chosen due to their high stiffness and strength. The transverse steel moment frames that support the high roof area allows for a large open space with a high ceiling. Large shear forces are required to be transferred in the longitudinal direction from the shear walls along lines B & H to the exterior walls along lines A & I. Horizontal pipe bracing consisting of extra strong sections were chosen to transfer these large forces to the exterior shear walls.



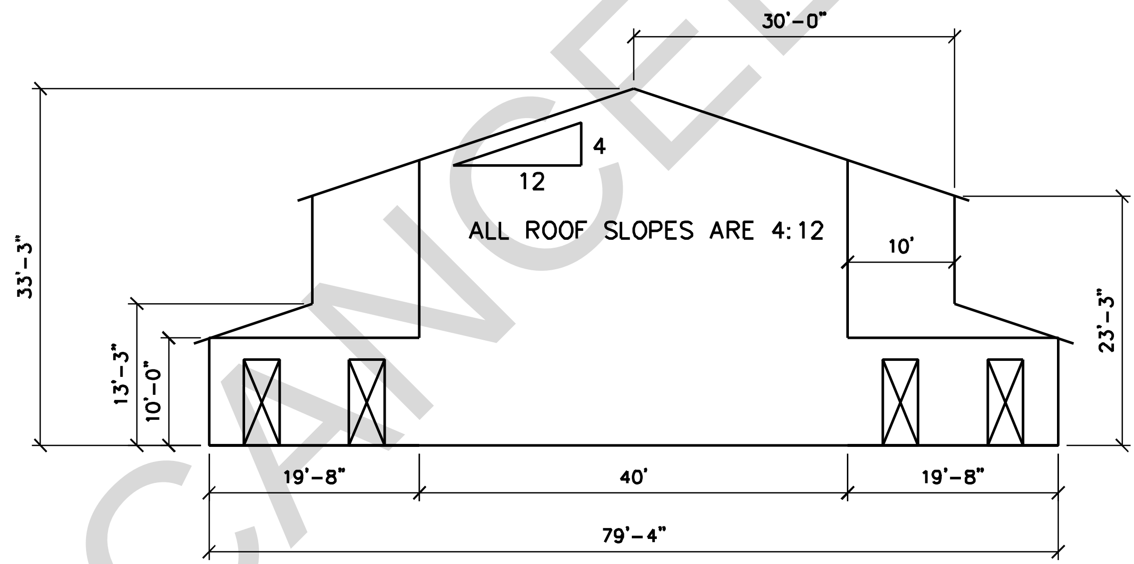
1 foot = 0.305 m
1 inch = 25.4 mm

FLOOR PLAN

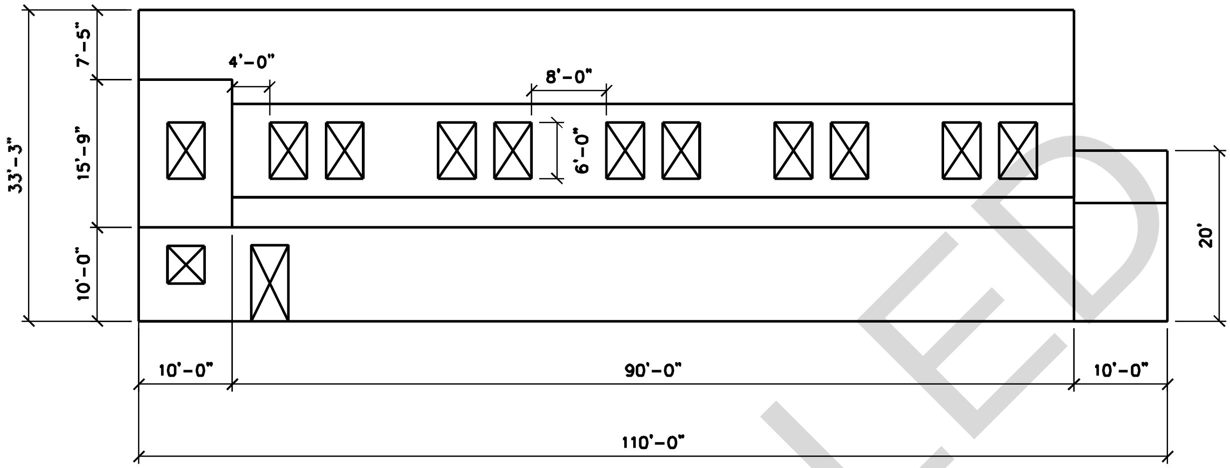


FRONT ELEVATION

1 foot = 0.305 m
1 inch = 25.4 mm

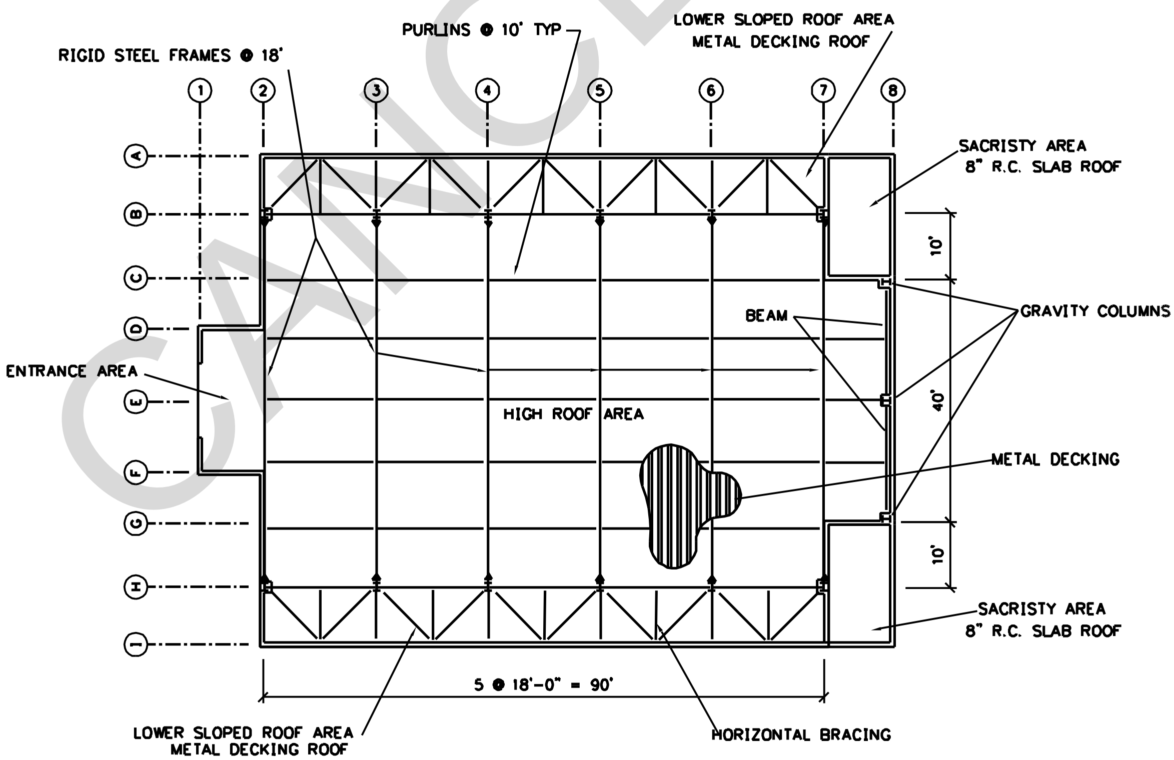


REAR ELEVATION

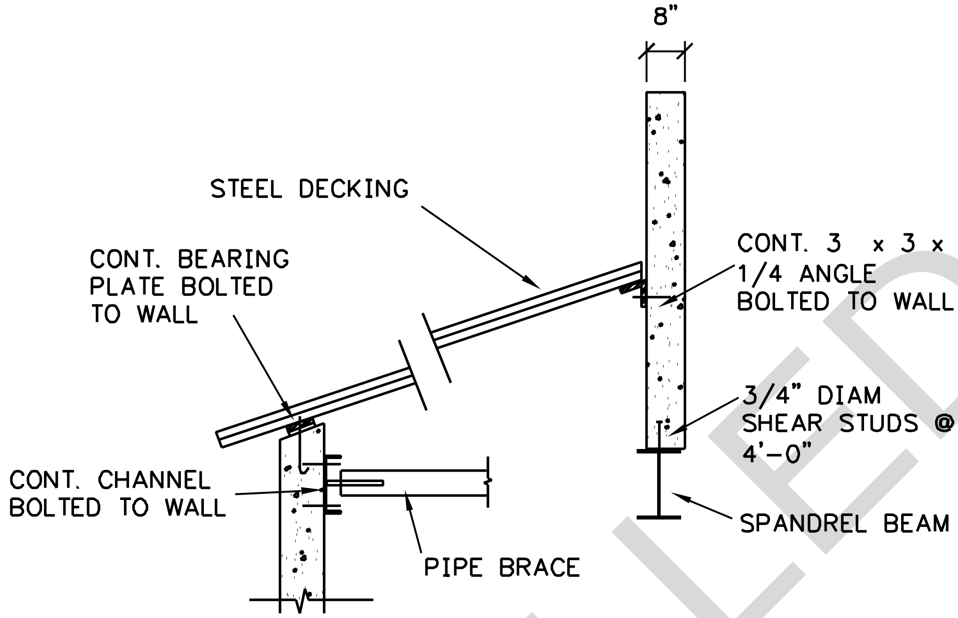


SIDE ELEVATION

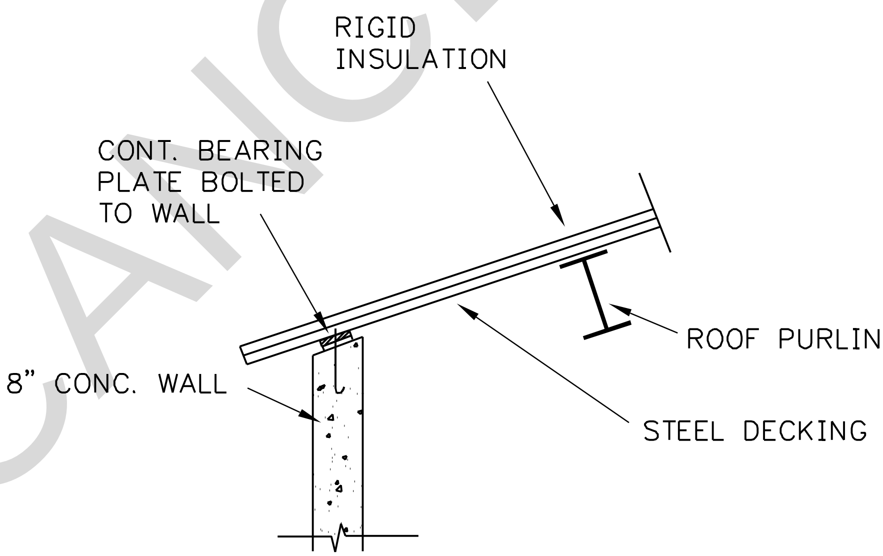
1 foot = 0.305 m
1 inch = 25.4 mm



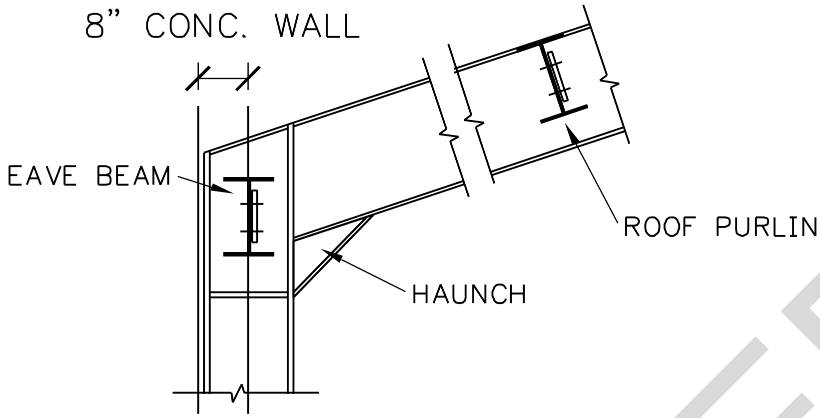
ROOF FRAMING



SECTION AT LOW ROOF



TYPICAL EAVE DETAIL



KNEE DETAIL FOR RIGID FRAME

c. Preliminary building design (Following steps in Table 4-5 for Life Safety). The preliminary design of the building follows the steps outlined in Table 4-5 for Performance Objective 1A. The design is then updated to meet the enhanced performance objectives laid out in Table 4-6 for Performance Objective 2A.

A-1 *Determine appropriate Seismic Use Group.* The structure falls into Seismic Use Group II.

A-2 *Determine Site Seismicity.* The site seismicity for this example from the MCE maps is:
$S_S = 1.50g$ $S_1 = 0.75g$.

A-3 *Determine Site Characteristics.* The soil for this example site is assumed to correspond to site class D.

A-4 *Determine Site Coefficients, F_a and F_v.* From Tables 3-2a and 3-2b for the given site seismicity and soil characteristics the site coefficients are:
$F_a = 1.0$ $F_v = 1.5$

A-5 *Determine adjusted MCE spectral response accelerations:*
$S_{MS} = F_a S_S = (1.0)(1.5g) = 1.50g$ Eq. 3-1
$S_{M1} = F_v S_1 = (1.5)(0.75g) = 1.13g$ Eq. 3-2

A-6 *Determine design spectral response accelerations:*
$S_{DS} = 2/3 S_{MS} = 2/3(1.5g) = 1.0g$ Eq. 3.3
$S_{D1} = 2/3 S_{M1} = 2/3(1.13g) = 0.75g$ Eq. 3.4
For regular structures, 5-stories or less in height, and having a period, T, of 0.5 seconds or less, the design spectral acceleration need not exceed:
$S_{DS} \le 1.5F_a = (1.5)(1.0) = 1.50g > 1.0g$ Eq. 3-5
$S_{D1} \le 0.6F_v = (0.6)(1.5) = 0.9g > 0.75g$ Eq. 3-6

A-7 *Select structural design category.* From Tables 4-2a and 4-2b for Seismic Use Group II and the design spectral response accelerations:
Seismic Design Category = D[a] Table 4-2a
Seismic Design Category = D[a] Table 4-2b
Footnote 'a' requires that structures with $S_1 \ge 0.75g$ be assigned to Seismic Design Category E.

A-8 *Select structural system.* The lateral system consists of a combination of special concrete shear walls and intermediate steel moment frames. In the transverse direction, the structure has concrete shear walls at the ends and moment frames in the interior. Longitudinally, concrete shear walls resist the lateral forces.

The upper roof diaphragm consists of metal decking that acts as a flexible diaphragm. The metal deck diaphragm spans are less than 2:1 in accordance with the limits for diaphragm span and depth set forth in Table 7-24. The lower roof diaphragm in the longitudinal direction consists of horizontal pipe bracing that transfers the shear from the suspended upper concrete shear walls to the exterior shear walls.

A-9 *Select R, W_o & C_d factors.*

Transverse (North-South): Building frame system with special reinforced concrete shear walls:
$R =$ 6
$\Omega_o =$ 2.5 Table 7-1
$C_d =$ 5

Longitudinal (East-West): Building frame system with special reinforced concrete shear walls:
$R =$ 6
$\Omega_o =$ 2.5 Table 7-1
$C_d =$ 5

A-10 *Determine preliminary member sizes for gravity load effects.*

Roof Purlins

- Assume simply supported, compression flange supported by roof deck along entire length
- Tributary width = spacing = 10’ (3.05m)
- Strength reduction factor, $\phi = 0.9$ (AISC LRFD Section F1.2)

Loads:

Live loads: $L_r = 20R_1R_2$ ASCE 7-95 Eq. 4-2
A_t = (10’)(18’) = 180 ft.^{2} (16.7 m^2)
$R_1 = 1$ (A_t < 200 ft.^{2} or 18.6 m^2)
$R_2 = 1$ (Rise of roof = 4” per foot or 3.33 cm per 10 cm)
L_r = (20 psf)(1)(1) = 20 psf (958 N/m^2)

Dead loads:		
	Finish	1.0 psf
	Metal Decking	2.0 psf
	Purlins (self wt.)	1.5 psf
	Ceiling / Covering	1.0 psf
	Misc.	3.0 psf
	Total:	8.5 psf (407 N/m^2)
	Adjust for slope:	(8.5)(12.65/12) = 8.96 plf (429 N/m^2)

Load Combination: 1.2 D + 1.6 Lr
w_u = (10’)[(1.2)(8.96psf) + (1.6)(20psf)] = 428 plf (6.25 KN/m)

$Z_{req} = M_u / \phi F_y$,
$M_u = w_uL^2/8$ = (428 plf)(18’)^2/8 = 17.3 kipft (23.46 KNm)
Z_{req} = (17.3)(12)/(0.9)(36) = 6.41 in.^{3} (105 cm^3)
Try C8x11.5, Z_x = 9.55 in.^{3} (156 cm^3) > 6.41 in.^{3}
Check shear;
V_u = (428 plf)(18’)/2 = 3.9 kips (17.35 KN)
h/t_w = 6.125/.22 = 27.8 < 418/(36)$^{1/2}$ = 70, use AISC LRFD Eq. F2-1
$\phi_vV_n = 0.6F_{yw}A_w$ = 0.6(36ksi)(0.22”)(8.0”) =38.0 kip (169 KN) > 3.9 kips (17.35KN), O.K.

Beams along grid line 8

- Assume simply supported, compression flange supported by roof deck
- Tributary width = 5’ (1.53m)

Loads:

Live Loads: $L_r = 20R_1R_2$ ASCE 7-95 Eq. 4-2
A_t = (5’)(20’) = 100 ft.^{2} (9.3m^2)
$R_1 = 1$ (A_t < 200 ft.^{2})
$R_2 = 1$ (Rise of roof = 4” per foot)
L_r = (20 psf)(1)(1) = 20 psf (958 N/m^2)

Dead Loads: (10.5)(12.65/12); Same as for purlins but add 2 psf (96 N/m^2) for self-weight
= 11.07 psf (530 N/m^2)

Load Combination: 1.2D + 1.6Lr
w_u = (5)[1.2(11.07)+1.6(20)] = 226 plf (3.30 KN/m)

M_u = (226)(20)^2/8 = 11.3 kipft (15.32 KNm)
Z_{req} = (11.3)(12)/(0.9)(36) = 4.2 in.^{3} (68.8 cm^3)
Try W12x14, Z_x = 17.4 in.^{3} (285 cm^3) > 4.2 in.^{3} (68.8 cm^3)
Check shear;
V_u = (428 plf)(18’)/2 = 3.9 kips (17.3 KN)
h/t_w = 54.3 < 418/(36)$^{1/2}$ = 70, use AISC LRFD Eq. F2-1

$\phi_v v_n = 0.6F_{yw}A_w = 0.6(36ksi)(0.20")(11.91") = 51.5$ kip (229 KN) > 3.9 kips (17.3 KN), O.K

Columns C8, E8 and G8

- Columns are designed for gravity loads only
- Assume columns are pin-pin connected, K=1.0
- Use L = 33.33' (10.17m) for all columns (conservative)
- Tributary area = 100 ft.^{2} (9.3 m^2)

  Loads:

  Live Loads: 20 psf (958 N/m^2)

  Dead Loads: 11.07 psf (530 N/m^2); Same as for beams along grid line 8

  Load Combination: 1.2D + 1.6Lr

  $P_u = (100)[(1.2)(11.07)+(1.6)(20)] = 4.53$ kips (20.15 KN)

  Try W10x33, A = 9.71 in.^{2} (62.6 cm^2), r_y = 1.94 in. (4.93 cm)

$$\lambda_c = \frac{KL}{r\pi}\sqrt{\frac{F_y}{E}} = \frac{(1.0)(33.3)(12)}{1.94\pi}\sqrt{\frac{36}{29000}} = 2.31 > 1.5 \quad \text{AISC LRFD Eq. E2-4}$$

$$F_{cr} = \left(\frac{0.866}{\lambda_c^2}\right)F_y = \left(\frac{0.866}{2.31^2}\right)36 = 5.84\text{ksi} \quad \text{AISC LRFD Eq. E2-3}$$

$$\phi_c P_n = \phi_c A_g F_{cr} = (0.85)(9.71)(5.84) = 48.2\text{kips } (214\text{KN}) > 4.53\text{k } (20.2\text{ KN}) \quad \text{AISC LRFD Eq. E2-1}$$

Beams along grid lines B & H

These beams support the discontinuous 8" (20.32 cm) thick concrete window walls. It is assumed that the beams carry no live load. It is further assumed that the beams are completely supported by the concrete walls for torsion. Therefore, torsion is not considered.

Loads:

Dead Loads: (from concrete wall above)

Weight of wall supported by beams = (unit weight)(wall height) = (8/12)(150 pcf)(13.25') = 1325 plf

w_u = 1.4 D = 1.4(1325 plf) = 1855 plf (27.1 KN/m) (ASCE 7-95 Sec. 2.3.2)
$M_u = (1855)(18)^2/8 = 75.13$ kipft (102 KNm)
$Z_{req} = (75.13)(12)/(0.9)(36) = 28$ in.^{3} (459 cm^3)

Try W14 x 38, Z_x = 61.5 in.^{3} > 28 in.^{3}

Check shear;
V_u = (1855 plf)(18')/2 = 16.7 kips (74.3 KN)
$h/t_w = 39.6 < 418/(36)^{1/2} = 70$, use AISC LRFD Eq. F2-1
$\phi_v v_n = 0.6F_{yw}A_w = 0.6(36ksi)(0.31")(14.10") = 96.4$ kip (428.8 KN) > 16.7 kips (74.3 KN) O.K

Moment Frames

For the preliminary gravity design it was assumed that $I_c / I_b = 1.75$ & $A_c / A_b = 1.75$ to ensure strong col / weak beam. Trial sections were obtained and the analysis repeated with the assumed sections.

- Frames are spaced at 18' (5.49m)
- Dead load from roof = 12.1 psf (579 N/m) on horizontal projection (See step B-2)

Columns: Loads:
The columns support the loads from the upper roof as well as the beams along grid lines B & H (one on each side of column)

Dead Loads:
12.1 psf (roof dead load from step B-2)
Beam Reaction = 2*(wL/2) = (2)(1325 plf)(18')/2 = 23.9 k (106 KN)

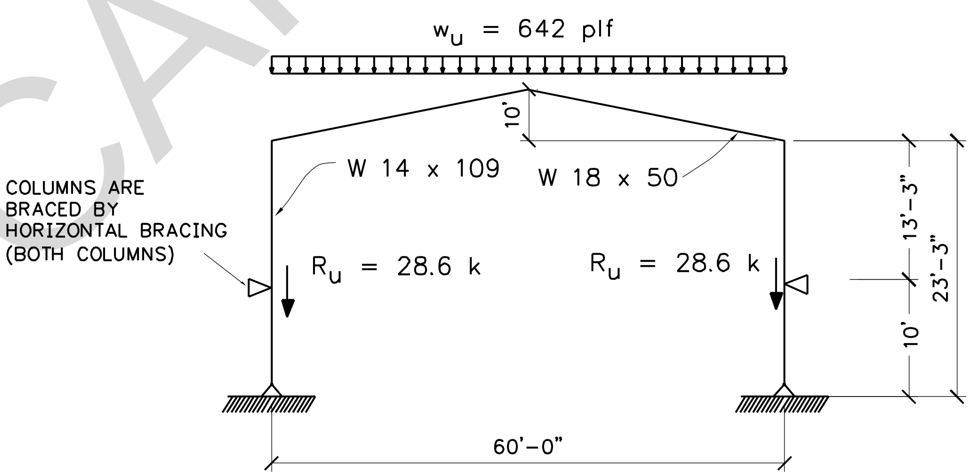
Live Loads:
Live loads: $L_r = 20R_1R_2$ ASCE 7-95 Eq. 4-2
A_t = (18')(30') = 540 ft.^{2} (50.22 m^2)
$R_1 = 1.2 – 0.001A_t = 1.2 – 0.001(540) = 0.66$
R_2 = 1 (Rise of roof = 4" per foot)
L_r = (20 psf)(0.66)(1) = 13.2 psf (193 N/m^2)

Beams: (Assume beam is one member for design, Length = 60' or 18.3m)
Loads;
Dead Loads: (12.1 psf)(18') = 218 plf (3.18 KN/m)

Live Loads:
The tributary area of the beams (18')(60') = 1080 ft.^{2} (100.4m^2)
The trib. area of the beams is greater than that of the columns (540 ft.^{2} or 50.2 m^2)
A higher live load reduction could be used, however use the same live loads that the columns to be conservative
L_r = (20 psf)(0.66)(1) = 13.2 psf = (13.2psf)(18') = 238 plf (3.47 KN/m)
w_u = 1.2(218) + 1.6(238) = 642 plf (9.37 KN/m)

1 plf = 14.59 N/m
1 kip = 4.448 KN
1 foot = 0.305 m
1 inch = 25.4 mm

Input for elastic analysis



Beam Design

The beams are laterally braced by the purlins, L_b = 10' (3.1m)

- Results from elastic analysis:

  Maximum moment at end of beam = 125.3 kft (170 KNm)
  Maximum axial force at end of beam = 21.25 k (94.5 KN)

Design member as beam-column
- Try a W 18 x for trial size
- KL_x = 1.2(31.6) = 38 ft., Kl_y = 1.0(10') = 10
- L_b = 10' (spacing of purlins)
- M_y = 0 (no lateral forces on beam)

Try W 18 x 50 (A = 14.7 in.^{2}, r_x = 7.38 in., r_y = 1.65 in., L_p = 6.9', BF = 7.31, $\phi_b M_p$ = 273 kft)
$(KL/r)x$ = (38')(12")/(7.38") = 61.8
$(KL/r)y$ = (10')(12")/(1.65") = 72.7
$\phi_c F_{cr}$ = 23.17 ksi (160 N/mm^2) (AISC Table 3-36)
$\phi P_n = A\phi_c F_{cr}$ = (14.7)(23.17) = 340 k (1512 KN)
$\phi_b M_{nx} = C_b\left[\phi_b M_{nx} - BF(L_b - L_p)\right] = 1.0(273\text{kft} - 7.31(10 - 6.9)) = 250\text{kft}$ (339 KNm) (AISC LRFD Chap 4)
$P_u / \phi P_n$ = 21.25 / 340 = 0.06 < 0.2, use interaction equation H1-1b

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}}\right) \le 1 = \frac{21.25}{2(340)} + \left(\frac{125.28}{250}\right) = 0.53 < 1, \text{OK}$$

By inspection assume that beam is OK in shear.
The beam is slightly over designed in anticipation of the higher stresses due to the addition of the lateral loads to the frames.
Use W 18 x 50 as a trial beam size.

Column Design

The interaction of axial and moment forces are check at the top of the column (highest moment) and at the horizontal support point (highest axial load)

- Results from elastic analysis:

  Moment at top of column = 125.3 kft = 1504 kipin (170 KNm)
  Axial force at top of column = 20.31 k (90.3KN)

  Moment at horizontal pin support (at horizontal bracing) = 81.9 kft = 983 kipin (111 KNm)
  Axial force at pin support = 48.99 k (218 KN)

Design member as beam-column

Try a W 14 x 109 for trial size to ensure strong column / weak beam
W 14 x 109 (A = 32.0 in.^{2}, r_x = 6.22 in., r_y = 3.73 in., L_p = 15.5', Z_x = 192 in.^{3})

- Top of column:
  The upper portion of the column (above the horizontal bracing level) is braced in both directions from the concrete window walls (B1-B2 & H1-H2). However, an unbraced column length of 13.3' will be used to determine the axial capacity (based on $K_x L_x / r_x$) to be conservative. It is assumed that the compression flange of the columns is continuously supported by the concrete walls.

Determine K from alignment chart for effective length of columns in continuous frames (AISC LRFD Fig C-C2.2)
Assume that the column is pinned at the lateral support (G=10).

At top of column, $G = \dfrac{\sum(I_c / L_c)}{\sum(I_g / L_g)} = \dfrac{(1240/(13.3)}{(800/31.6)} = 3.68$, $K \approx 2.4$ (sidesway uninhibited)

$(KL/r)x = (2.4)(13.3')(12)/(6.22") = 61.58$
$\phi_c F_{cr} = 25.06$ ksi (173 N/mm^2) (AISC LRFD Table 3-36)
$\phi_c P_n = 25.06$ ksi (32 in.^{2}) = 802 kips (3567 KN)

$M_{ux} = B_{1x}M_{ntx}+B_2xM_{ltx}$; for this preliminary design the moment magnification factors are assumed to be unity. Therefore, M_{ux} = moment at top of column = 125.3 kft (170 KNm)

$\phi_b M_{nx} = \phi_b Z_x F_y = (0.9)(192)(36)/(12) = 518$ kft (702 KNm); (see AISC LRFD Chapter F)

$P_u / \phi P_n = 20.31 / 802 = 0.025 < 0.2$, use interaction equation H1-1b

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}}\right) \le 1 = \frac{20.31}{2(802)} + \left(\frac{125.3}{518}\right) = 0.25 < 1, \text{ OK}$$

- Bottom of column:
  The lower portion of the column (below the horizontal bracing level) is assumed to be pinned at both the footing support and the horizontal bracing for determining the second order effects.

K = 1.0 in both x and y directions (pin support)

$(KL/r)_x = (1.0)(10')(12)/(6.22") = 19.3$
$(KL/r)_y = (1.0)(10')(12)/(3.73") = 32.2$ (governs)
$\phi_c F_{cr} = 29$ ksi (200 N/mm^2) (AISC LRFD Table 3-36)
$\phi_c P_n = 29$ ksi (32 in.^{2}) = 927 kips (4123 KN)

$M_{ux} = B_{1x}M_{ntx}+B_2xM_{ltx}$; for this preliminary design the moment magnification factors are assumed to be unity. Therefore, M_{ux} = moment at bracing support = 125.3 kft (170 KNm)

$\phi_b M_{nx} = \phi_b Z_x F_y = (0.9)(192)(36)/(12) = 518$ kft (702 KNm), since $L_b < L_p$ $(10 < 15.5)$ (see AISC LRFD Chapter F)

$P_u / \phi P_n = 48.99/ 927 = 0.05 < 0.2$, use interaction equation H1-1b

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}}\right) \le 1 = \frac{48.99}{2(927)} + \left(\frac{81.9}{518}\right) = 0.18 < 1, \text{ OK}$$

The column is over designed in anticipation of the higher stresses due to the addition of the lateral loads to the frames and to ensure strong beam / weak column.
Use W 14 x 109 as a trial column size.

B-1 *Calculate fundamental period, T*

$T_a = C_T h_n^{3/4}$ where $C_T = 0.020$ and $h_n = 28.25$ ft. (ave. roof ht.) (FEMA 302 Eq. 5.3.3.1-1)

$T_a = (0.020)(28.25)^{3/4} = 0.25$ seconds for both the transverse and longitudinal directions.

B-2 *Determine the dead load, W*

Sloped Upper Roof Level*

Finish	1.0 Psf
20 Gage Metal Deck	2.0 Psf
Roof Purlins	1.5 Psf
Rigid Frame Beams	2.0 Psf
Rigid Frame Columns	1.0 Psf
Ceiling / Covering	1.0 Psf
Misc. (Mech., Elec. & Framing)	3.0 Psf
Total:	11.5 Psf

Sloped Lower Roof Level*

Built-Up Roofing	5.0 psf
2" Rigid Insulation	3.0 psf
20 Gage Metal Deck	2.0 psf
Horizontal Bracing	2.0 psf
Ceiling / Covering	1.0 psf
Misc. (Mech., Elec.& Framing)	3.0 psf
Total:	16.0 psf

Roof at Sacristy (Horizontal)

Finish	1.0 Psf
Concrete Slab (Assume 4" Thick)	50.0 Psf
Ceiling / Covering	1.0 Psf
Misc. (Mech, Elec.)	1.0 Psf
Total:	53.0 Psf

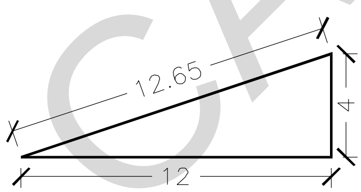
Sloped Roof at Entrance*

Built-Up Roofing	5.0 psf
2" Rigid Insulation	3.0 psf
20 Gage Metal Deck	2.0 psf
Steel Beams	1.0 psf
Ceiling / Covering	1.0 psf
Misc. (Mech, Elec. & Framing)	1.0 psf
Total:	13.0 psf

Reinforced Concrete Shear Walls

Normal Weight Concrete (8" Thick) 100 Psf

*Note: The weights of the sloped roofs are calculated based on the sloped area. In order to calculate the weight based on the horizontally projected area the loading on the sloped roof must be adjusted.



Upper roof level: (11.5)(12.65/12) = 12.12 psf

Sloped lower roof level: (16.0)(12.65/12) = 16.87 psf

Roof at entrance: (13.0)(12.65/12) = 13.70 psf

1 psf = 47.88 N/m^2

BUILDING SEISMIC WEIGHTS

UPPER ROOF LEVEL TRIBUTARY SEISMIC WEIGHTS (ROOF & TRIBUTARY WALLS)

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)
UPPER ROOF						
Roof (grid 2 - 7)	62.0	90.0	100.0	5580.0	12.1	67.6
Roof (grid 7-8)	10.0	42.0	100.0	420.0	12.1	5.1
TRANSVERSE WALLS						
Wall 8C-8G	---	---	---	650.0	100.0	65.0
Wall 7A-7C	---	---	---	66.0	100.0	6.6
Wall 7G-7I	---	---	---	66.0	100.0	6.6
Wall 2B-2H	---	---	---	800.0	100.0	80.0
LONGITUDINAL WALLS						
Wall B2-B7	6.5	90.0	65.0	380.3	100.0	38.0
Wall H2-H7	6.5	90.0	65.0	380.3	100.0	38.0
Wall C7-C8	8.0	10.0	75.0	60.0	100.0	6.0
Wall G7-G8	8.0	10.0	75.0	60.0	100.0	6.0
					TOTAL	319.0

1419 KN

LOWER SLOPED ROOF LEVEL TRIBUTARY SEISMIC WEIGHTS (ROOF & TRIBUTARY WALLS)

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)
LOWER ROOF						
Roof A2-B7	10.0	90.0	100.0	900.0	16.9	15.2
Roof H2-I7	10.0	90.0	100.0	900.0	16.9	15.2
TRANSVERSE WALLS						
Wall 7A-7B	---	---	---	62.0	100.0	6.2
Wall 7H-7I	---	---	---	62.0	100.0	6.2
Wall 2A-2B	---	---	---	62.0	100.0	6.2
Wall 2H-2I	---	---	---	62.0	100.0	6.2
LONGITUDINAL WALLS						
Wall B2-B7	6.5	90.0	65.0	380.3	100.0	38.0
Wall H2-H7	6.5	90.0	65.0	380.3	100.0	38.0
Wall A2-A7	5.0	90.0	95.0	427.5	100.0	42.8
Wall I2-I7	5.0	90.0	95.0	427.5	100.0	42.8
					TOTAL	216.7

964 KN

LOWER SACRISTRY TRIBUTARY SEISMIC WEIGHTS

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)
LOWER ROOF						
Roof A7-C8	10.0	19.7	100.0	196.7	53.0	10.4
Roof G7-I8	10.0	19.7	100.0	196.7	53.0	10.4
TRANSVERSE WALLS						
Wall 8A-8C	5.0	20.0	80.0	80.0	100.0	8.0
Wall 8G-8I	5.0	20.0	80.0	80.0	100.0	8.0
LONGITUDINAL WALLS						
Wall A7-A8	5.0	10.0	75.0	37.5	100.0	3.8
Wall I7-I8	5.0	10.0	75.0	37.5	100.0	3.8
Wall C7C8	13.0	10.0	75.0	97.5	100.0	9.8
Wall G7-G8	13.0	10.0	75.0	97.5	100.0	9.8
					TOTAL	63.9

284 KN

LOWER SLOPED ROOF @ ENTRANCE TRIBUTARY SEISMIC WEIGHTS

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)
LOWER ROOF						
Roof D1-F2	10.0	24.0	100.0	240.0	13.7	3.3
TRANSVERSE WALLS						
Wall D1-F1	---	---	100.0	160.0	100.0	16.0
LONGITUDINAL WALLS						
Wall D1-D2	8.0	10.0	100.0	80.0	100.0	8.0
Wall F1-F2	8.0	10.0	100.0	80.0	100.0	8.0
					TOTAL	35.3 157 KN

Total Seismic Weight = 634.8 Kips (2824 KN)

B-3 *Calculate Base Shear*

$V = C_sW$ FEMA 302 Eq. 5.3.2
$C_s = S_{DS}/R$, but need not exceed: Eq. 3-7
$C_s = S_{D1}/TR$, but shall not be less than: Eq. 3-8
$C_s = 0.044S_{DS}$ Eq. 3-9

Transverse Direction:

$C_s = (1.0)/6 = 0.167$
$C_s = (0.75)/(0.25)(6) = 0.5 > 0.167$
$C_s = 0.044(1.0) = 0.044 < 0.167$

$V = C_sW = (0.167)(634.8 \text{ kips}) = 106 \text{ kips } (471 \text{ KN})$

Longitudinal Direction:

$C_s = (1.0)/6 = 0.167$
$C_s = (0.75)/(0.25)(6) = 0.5 > 0.167$
$C_s = 0.044(1.0) = 0.044 < 0.167$

$V = C_sW = (0.167)(634.8 \text{ kips}) = 106 \text{ kips } (471 \text{ KN})$

B-4 *Calculate vertical distribution of seismic forces*

The building is analyzed as a one-story structure. The seismic forces to the separate roof areas are determined by multiplying the tributary weights by the seismic coefficient C_s.

<u>Transverse Direction:</u> $C_s =$ 0.167

$W_{roof} =$ 318.99 Kips $V_{roof} =$ 53.2 kips (237 KN)
$W_{lowroof} =$ 108.36 Kips each $V_{lowroof} =$ 18.1 kips each (81 KN)
$W_{sacr} =$ 31.93 Kips each $V_{sacr} =$ 5.3 kips each (23.6 KN)
$W_{entrance} =$ 35.29 Kips $V_{entrance} =$ 5.9 kips (26.2 KN)

<u>Longitudinal Direction:</u> $C_s =$ 0.167

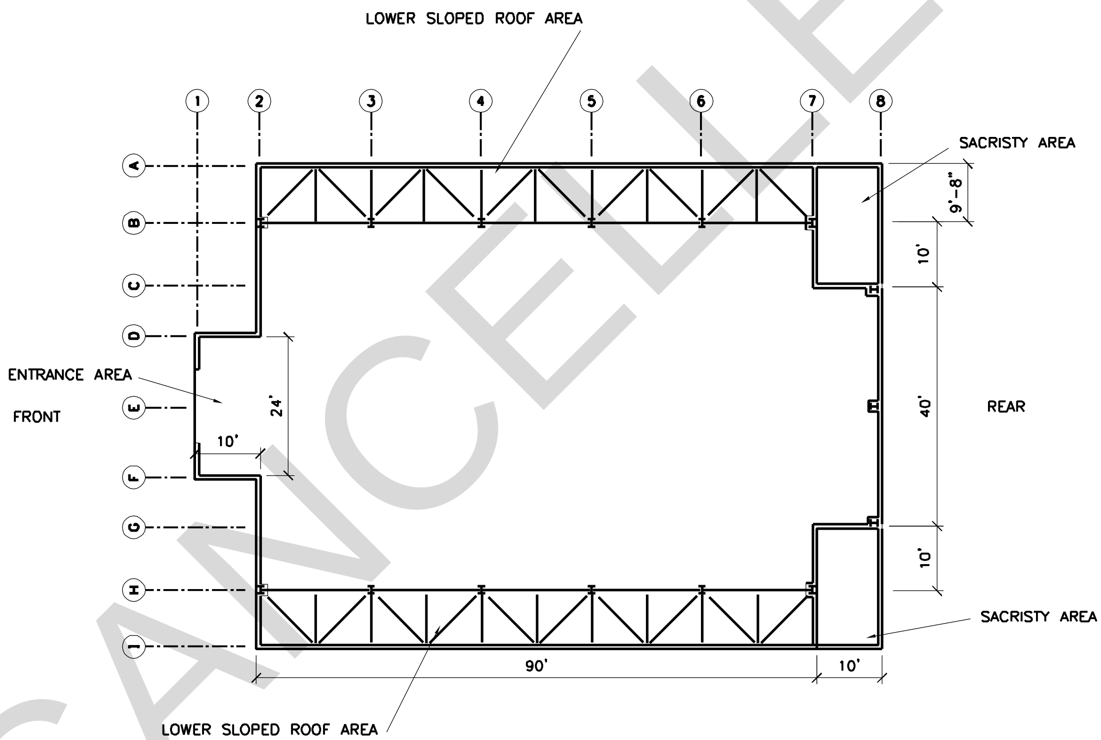
$W_{roof} =$ 318.99 Kips $V_{roof} =$ 53.2 kips (237 KN)
$W_{lowroof} =$ 108.36 Kips each $V_{lowroof} =$ 18.1 kips each (81 KN)
$W_{sacr} =$ 31.93 Kips each $V_{sacr} =$ 5.3 kips each (23.6 KN)
$W_{entrance} =$ 35.29 Kips $V_{entrance} =$ 5.9 kips (26.2 KN)

B-5 *Perform Static Analysis*

The seismic forces from the upper sloped roof and entrance are resisted by the vertical elements based on tributary areas due to flexible diaphragm action. The seismic forces tributary to the sacristies and lower sloped roof areas are resisted by the vertical elements based on relative rigidity due to rigid diaphragm action.

<u>Seismic forces to vertical resisting elements from upper sloped roof diaphragm</u>

<u>Transverse direction:</u> Seismic forces tributary to the sloped upper roof in the transverse direction are resisted by the steel moment frames and the concrete shear walls based on tributary areas. It is assumed that the frames along grid lines 2 and 7 resist no loads due to their low stiffness as compared to the shear walls along the same grid lines. The diaphragm forces are determined from the weight of the roof and tributary normal walls. It is assumed that the diaphragm acts as a simply supported beam element between the frames and shear walls.



FLOOR PLAN

1 inch = 25.4mm
1 foot = 0.305m

Weight of roof and normal walls between grid lines 2 and 7

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)
UPPER ROOF						
Roof (grid 2 - 7)	62.0	90.0	100.0	5580.0	12.1	67.6
LONGITUDINAL WALLS						
Wall B2-B7	6.5	90.0	65.0	380.3	100.0	38.0
Wall H2-H7	6.5	90.0	65.0	380.3	100.0	38.0
					TOTAL	143.7

(639 K)N

Total weight = 143.7 kips Seismic coefficient, $C_s = 0.167$
Seismic force = $C_sW = (0.167)(143.7) = 24$ kips (107 KN)
Equivalent Running Load $w_{2\text{-}7} = 24$ k / 90 ft. = 267 plf (3.90 KN/m)

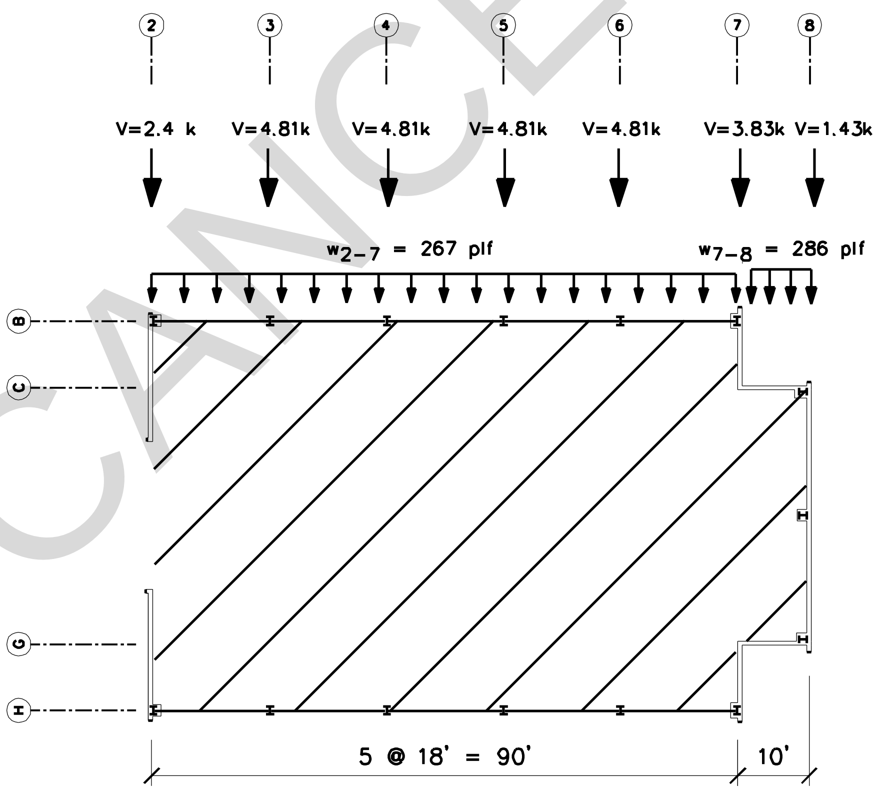
Weight of roof and normal walls between grid lines 7 and 8

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)
UPPER ROOF						
Roof (grid 7-8)	10.0	42.0	100.0	420.0	12.1	5.1
LONGITUDINAL WALLS						
Wall C7-C8	8.0	10.0	75.0	60.0	100.0	6.0
Wall G7-G8	8.0	10.0	75.0	60.0	100.0	6.0
					TOTAL	17.1

76.1 KN

Total weight = 17.1 kips Seismic coefficient, $C_s = 0.167$
Seismic force = $C_sW = (0.167)(17.1) = 2.86$ kips (12.7 KN)
Equivalent Running Load $w_{7\text{-}8} = 2.86$ k / 10 ft. = 286 plf (4.17 KN/m)

Shear force to wall line 2 from upper roof diaphragm:
$V = w_{2\text{-}7}(9') = (267 \text{ plf})(9') = 2.40$ k (10.7 KN)
Shear force to each rigid frame (grid lines 3, 4, 5 and 6)
$V = w_{2\text{-}7}(18') = (267 \text{ plf})(18') = 4.81$ k (21.4 KN)
Shear force to wall line 7
$V = w_{2\text{-}7}(9') + w_{7\text{-}8}(5') = (267)(9) + (286)(5) = 3.83$ k (17.0 KN)
Shear force to wall line 8 $V = w_{7\text{-}8}(5') = (286)(5) = 1.43$ k (6.36 KN)



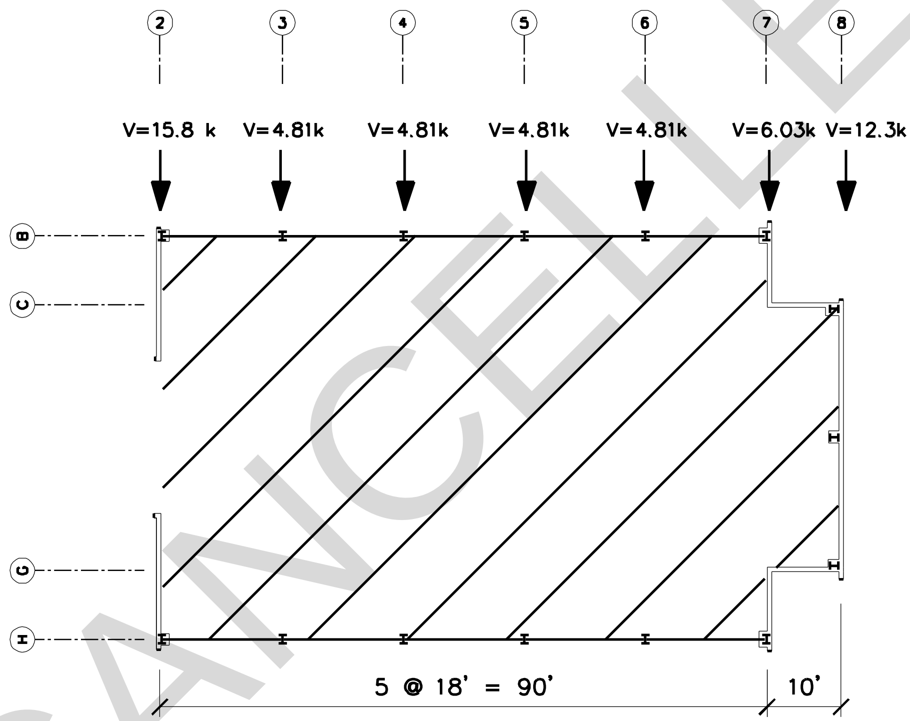
1 kip = 4.448 KN
1 foot = 0.305m
1 plf = 14.59 N/m

The self-weight inertial effects of the shear walls due to the weight tributary to the upper sloped roof must now be added to the shears determined for shear wall lines 2, 7 and 8.

– Weight of wall line 2 tributary to the upper roof = 80 kips
  Self-weight inertial force = C_sW = (0.167)(80) = 13.4 kips
  Total shear to wall line 2 tributary to upper roof diaphragm = (2.4 k) + (13.4 k) = 15.8 k (70.3 KN)

– Weight of wall line 7 tributary to the upper roof = (6.6) + (6.6) = 13.2 k (58.7 KN)
  Self-weight inertial force = C_sW = (0.167)(13.2) = 2.2 k (9.8 KN)
  Total shear to wall line 7 tributary to upper roof diaphragm = (3.83 k) + (2.2 k) = 6.03 k (26.8 KN)

– Weight of wall line 8 tributary to the upper roof = 65 k
  Self-weight inertial force = CW = (0.167)(65 k) = 10.9 k
  Total shear to wall line 8 tributary to upper roof diaphragm = (1.43 k) + (10.9 k) = 12.3 k (54.7 KN)



1 kip = 4.448 KN
1 foot = 0.305m

Longitudinal Direction: Seismic forces tributary to the upper roof in the longitudinal direction are resisted by the concrete shear walls based on tributary areas. Shear wall lines B2-B7 & H2-H7 each resist ½ of the shear from the upper roof between lines 2 and 7. Shear wall lines C7-C8 and G7-G8 each resist ½ of the shear from the upper roof between lines 7 and 8. The shear force associated with normal wall line 8 is assumed to be resisted by shear wall lines C and G, while the shear force associated with normal wall line 7 is assumed to be resisted by shear wall lines B & H. It is assumed that the diaphragm acts as a simply supported beam element between the shear walls.

Weight of roof and normal walls between grid lines 2 and 7

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)
UPPER ROOF						
Roof (grid 2 - 7)	62.0	90.0	100.0	5580.0	12.1	67.6
TRANSVERSE WALLS						
Wall 7A-7C	---	---	---	66.0	100.0	6.6
Wall 7G-7I	---	---	---	66.0	100.0	6.6
Wall 2B-2H	---	---	---	800.0	100.0	80.0
					TOTAL	160.8

715 KN

Total weight = 160.8 kips Seismic coefficient, C = 0.167
Seismic force = C_sW = (0.167)(160.8) = 26.85 kips
Equivalent Running Load w_{b-h} = 26.85 k / 60 ft. = 448 plf

Weight of roof and normal walls between grid lines 7 and 8

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)
UPPER ROOF						
Roof (grid 7-8)	10.0	42.0	100.0	420.0	12.1	5.1
TRANSVERSE WALLS						
Wall 8C-8G	---	---	---	650.0	100.0	65.0
					TOTAL	70.1

312 KN

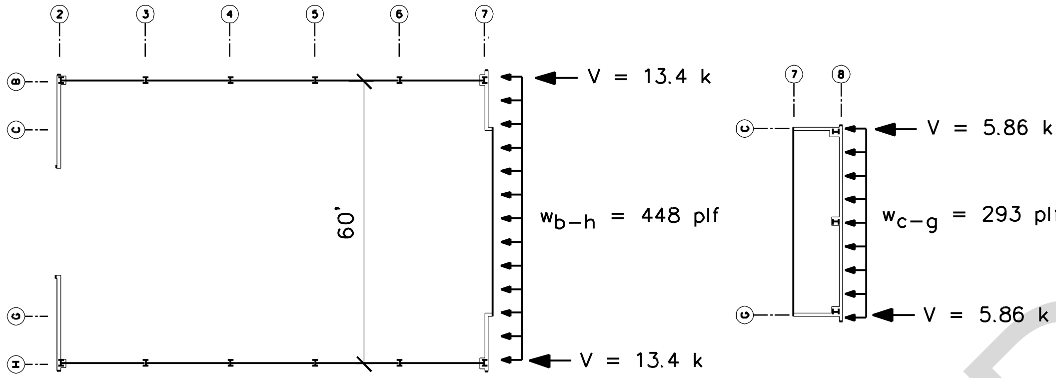
Total weight = 70.1 kips Seismic coefficient, C = 0.167
Seismic force = C_sW = (0.167)(70.1) = 11.71 kips (52.1 KN)
Equivalent Running Load w_{c-g} = 11.71 k / 40 ft. = 293 plf (4.27 KN/m)

– Shear force to wall line B from upper roof diaphragm
  V = w_{b-h}(30’) = (448)(30) = 13.4 kips (59.6 KN)

– Shear force to wall line H (same as wall line B due to symmetry)
  V = 13.4 kips (59.6 KN)

– Shear force to wall line C
  V = w_{c-g}(20’) = (293)(20) = 5.86 kips (26.1 KN)

– Shear force to wall line G (same as wall line C due to symmetry)
  V = 5.86 kips (21.6 KN)



1 kip = 4.448 KN
1 plf = 14.59 N/m

The self-weight inertial effects of the shear walls due to the weight tributary to the upper roof must now be added to the shears determined for shear wall lines B, C, G and H.

- Weight of wall line B tributary to the upper roof = 38 kips
  Self-weight inertial force = C_sW = (0.167)(38) = 6.35 kips
  Total shear to wall line B tributary to upper roof diaphragm = (13.4 k) + (6.35 k) = 19.8 k (88.1 KN)

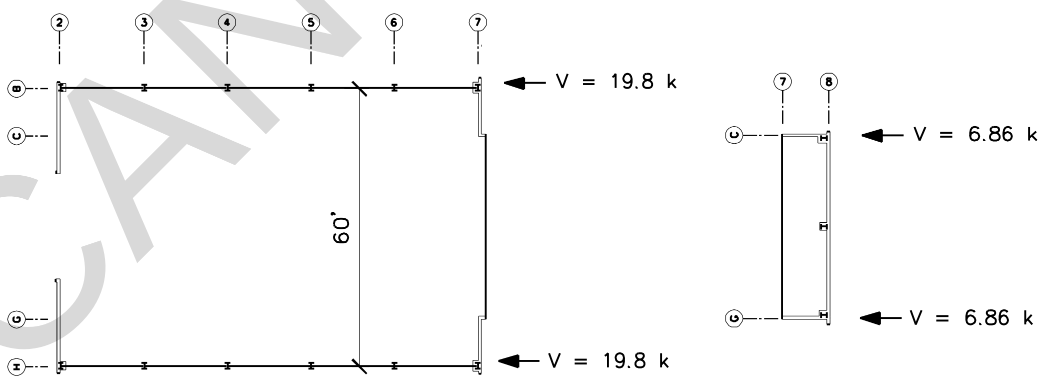
- Wall line H same as line B, Total shear = 19.8 k (88.1 KN)

  The shear from wall lines B & H is transferred through the horizontal bracing to shear wall lines A & I, respectively.

- Weight of wall line C tributary to the upper roof = 6 k
  Self-weight inertial force = C_sW = (0.167)(6) = 1.0 k
  Total shear to wall line C tributary to upper roof diaphragm = (5.86 k) + (1.0 k) = 6.86 k (30.5 KN)

- Wall line G same as line C, Total shear = 6.86 (30.5 KN)

TOTAL SHEAR TO VERTICAL ELEMENTS TRIBUTARY TO UPPER ROOF



Chord forces:
The upper roof diaphragm is analyzed as two subdiaphragms (one for grid lines 2-7 and one for grid lines 7-8).

Transverse direction:
The spans act as simply supported elements between the moment frames and concrete shear walls.

w_{2-7} = 267 plf (3.90 KN/m)
M = $wL^2/8$ = $(0.267)(18\ ft)^2$ / 8 = 10.8 kipft (14.64 KNm)
T = M / d = (10.8 / 60 ft) = 0.18 kips (801N)

w_{7-8} = 286 plf (4.17 KN/m)
M = $wL^2/8$ = $(0.286)(10\ ft)^2$ / 8 = 3.58 kipft (4.85 KNm)
T = M / d = (3.58 / 40 ft) = 0.09 kips (0.40 KN)

Longitudinal direction:
The spans act as simply supported elements between the concrete shear walls.

w_{2-7} = 448 plf (6.54 KN/m)
M = $wL^2/8$ = $(0.448)(60\ ft)^2$ / 8 = 202 kipft (274 KNm)
T = M / d = (202 / 90 ft) = 2.24 kips (9.96 KN)

w_{7-8} = 293 plf (4.27 KN/m)
M = $wL^2/8$ = $(0.293)(40\ ft)^2$ / 8 = 58.6 kipft (79.5 KNm)
T = M / d = (58.6 / 10 ft) = 5.86 kips (26.1 KN)

Seismic forces to vertical resisting elements from lower sloped roof diaphragm

Transverse direction: The horizontally braced diaphragms are assumed to act as rigid diaphragms spanning between the steel rigid frames and the end shear walls along lines 2 and 7. Due to the low stiffness of the moment frames as compared to that of the end concrete shear walls, it is assumed that all of the shear is distributed to the end walls in relation to their relative rigidities.

Determine relative rigidities of concrete shear walls at ends of low sloped roof area:

Wall Rigidity Equations

$f'_c := 3000\,psi$ — Concrete strength = 3000 psi

$E_c := 57000\sqrt{\dfrac{f'_c}{psi}}\cdot psi$ — ACI 318 Section 8.5.1

$E_c = 3122\,ksi$ $\quad E_c = 21.5251\,\dfrac{KN}{mm^2}$ — Elastic Modulus of concrete

$E_v := 0.4\cdot E_c$ $\quad E_v = 1249\,ksi$ — Shearing Modulus of concrete

$t := 8\cdot in$ $\quad E_v = 8.61\,\dfrac{KN}{mm^2}$ — Wall thickness

$P := 1\cdot kip$ — Shear load to determine pier stiffness

Concrete Functions:

$A(d) := t\cdot d$ — Area of wall pier

$I(d) := \dfrac{1}{12}\cdot t\cdot d^3\cdot 0.8$ — Assume stiffness = 80% of I per FEMA 273 Sec. 6.8.2.2

$$\Delta_c(h,d) := \frac{(1.2\cdot P\cdot h)}{A(d)\cdot E_v} + \frac{P\cdot h^3}{3\cdot E_c\cdot I(d)}$$

Deflection of a cantilevered pier

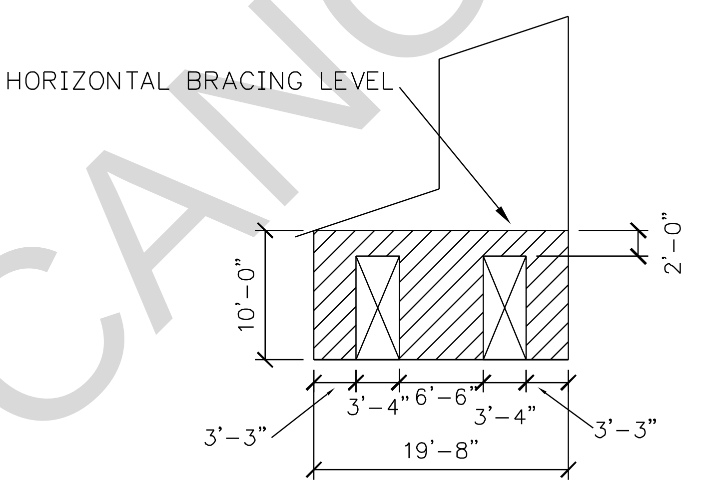
$$\Delta_f(h,d) := \frac{(1.2\cdot P\cdot h)}{A(d)\cdot E_v} + \frac{P\cdot h^3}{12\cdot E_c\cdot I(d)}$$

Deflection of a fixed-fixed pier

$$R_f(h,d) := \frac{1}{\Delta_f(h,d)}$$

Rigidity of fixed wall pier

Shear Walls 7A-7C and 7G-7I:



LOWER PORTION OF 7A–7C AND 7G–7I

1 foot = 0.305m

Solid wall ABCD $\Delta_{solid} := \Delta_c(10 \cdot ft, 19.67 \cdot ft)$ $\Delta_{solid} = 8.737 \cdot 10^{-5} \cdot in$

Subtract strip BCD $\Delta_{bcd} := \Delta_f(8 \cdot ft, 19.67 \cdot ft)$ $\Delta_{bcd} = 5.2219 \cdot 10^{-5} \cdot in$

$\Delta_a := \Delta_{solid} - \Delta_{bcd}$ $\Delta_a = 3.5151 \cdot 10^{-5} \cdot in$

Add back in piers B, C and D $R_b := R_f(8 \cdot ft, 3.25 \cdot ft)$ $R_b = 959.581 \cdot \frac{1}{in}$

$R_d := R_b$ $R_d = 959.581 \cdot \frac{1}{in}$

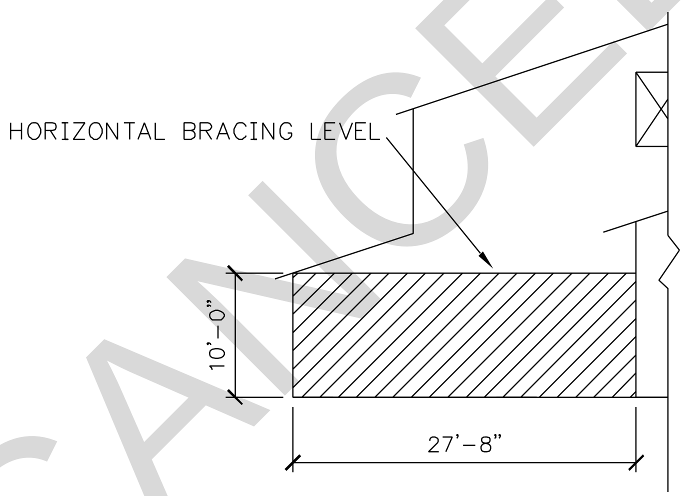
$R_c := R_f(8 \cdot ft, 6.5 \cdot ft)$ $R_c = 4146.9681 \cdot \frac{1}{in}$

$\Delta_{bcd} := \frac{1}{R_b + R_c + R_d}$ $\Delta_{bcd} = 0.0002 \cdot in$

$\Delta_{total} := \Delta_a + \Delta_{bcd}$ $\Delta_{total} = 0.0002 \cdot in$

Rigidity of wall $R_{wall} := \frac{1 \cdot kip}{\Delta_{total}}$ $R_{wall} = 4999.9934 \cdot \frac{kip}{in}$ $R_{wall} = 875.6126 \cdot \frac{KN}{mm}$

Shear walls 2A-2D & 2F-2I



SHEAR WALLS 2A–2D AND 2F–2I

Solid wall $\Delta_{solid} := \Delta_c(10 \cdot ft, 27.67 \cdot ft)$ $\Delta_{solid} = 5.28593 \cdot 10^{-5} \cdot in$

Rigidity of wall $R_{wall} := \frac{1 \cdot kip}{\Delta_{solid}}$ $R_{wall} = 18918.15 \cdot \frac{kip}{in}$ $R_{wall} = 3313 \cdot \frac{KN}{mm}$

Distribute the shear force tributary to the lower sloped roof areas to the shear walls.

Total weight tributary to the lower sloped roof areas = 216.7 kips (964KN)
Weight of each lower sloped roof area = 216.7 / 2 = 108.4 kips (482 KN)

Shear force tributary to each lower sloped roof area = C_sW = 0.167(108.4 kips) = 18.1 kips (80.5 KN)

$R_{2A\text{-}2D}$ = 18918 kips / in $R_{7A\text{-}7C}$ = 5000 kips / in $V_{element} = V \dfrac{R_{element}}{\sum R}$

$V_{2A\text{-}2D}$ = 18.1(18918) / (18918+5000) = 14.3 kips (63.6KN)
$V_{7A\text{-}7C}$ = 18.1(5000)/(18918+5000) = 3.78 kips (16.8 KN)

Due to symmetry, $V_{2F\text{-}2I} = V_{2A\text{-}2D}$ = 14.3 kips (63.6KN) $V_{7G\text{-}7I} = V_{7A\text{-}7C}$ = 3.78 kips (16.8 KN)

The horizontal bracing is assumed to act as a rigid support to the transverse moment frames. It is further assumed that the end shear walls (along grid lines 2 and 7) are infinitely rigid (relative to the frames). Therefore, the horizontally braced diaphragms of the low sloped roof will transmit the shear forces from the moment frames (shear forces tributary to the high roof) into the end shear walls.

Shear in moment frames from upper roof diaphragm = 4 (4.81 k) = 19.24 k (41.4 KN)

There are two low sloped roof areas (2A-B7 & H2-I7); assume each resists ½ of this shear force.

Shear to each low sloped roof area = ½(19.24k) = 9.62 k (42.8KN)

Distribute shear to walls based on rigidity;

$V_{2A\text{-}2D}$ = 9.62(18918) / (18918+5000) = 7.61 kips (33.8 KN)
$V_{7A\text{-}7C}$ = 9.62(5000)/(18918+5000) = 2.01 kips (8.94 KN)

Due to symmetry, $V_{2F\text{-}2I} = V_{2A\text{-}2D}$ = 7.61 kips (33.8 KN) $V_{7G\text{-}7I} = V_{7A\text{-}7C}$ = 2.01 kips (8.94 KN)

Total Shear to walls from lower roof diaphragms;
$V_{2A\text{-}2D}$ = 14.3 k + 7.61 k = 21.91 k (97.5 KN) $V_{7A\text{-}7C}$ = 3.78 k + 2.01 k = 5.79 k (25.8 KN)
Due to symmetry, $V_{2F\text{-}2I} = V_{2A\text{-}2D}$ = 21.91 kips (95.7 KN) $V_{7G\text{-}7I} = V_{7A\text{-}7C}$ = 5.79 kips (25.8 KN)

<u>Longitudinal direction:</u> The shear walls along grid lines A and I resist all of the shear tributary to the low sloped roofs.

Diaphragm force:

Weight tributary to each low sloped roofs (minus weight of exterior shear wall A or I):

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.^{2})	Unit Weight (psf / lb)	Seismic Weight (kips)
LOWER ROOF						
Roof A2-B7	10.0	90.0	100.0	900.0	16.9	15.2
TRANSVERSE WALLS	0.0	0.0	0.0	0.0	0.0	0.0
Wall 7A-7B	---	---	---	62.0	100.0	6.2
Wall 2A-2B	---	---	---	62.0	100.0	6.2
LONGITUDINAL WALLS	0.0	0.0	0.0	0.0	0.0	0.0
Wall B2-B7	6.5	90.0	65.0	380.3	100.0	38.0
					TOTAL	65.61
						292 KN

V = C_sW = 0.167(65.61 kips) = 10.96 k (48.8 KN)

This shear force must be transferred to the shear walls (A2-A7 & I2-I7) through the horizontal bracing. The bracing also transfers the shear force from wall lines B & H (tributary to the upper roof diaphragm).

Total shear force transferred through horizontal bracing = 19.8 k + 10.96 kips = 30.8 kips (137 KN)

Total shear to walls A2-A7 and I2-I7 = Horizontal bracing shear + shear due to self-weight:

Weight of wall A2-A7 = 42.75 kips
Shear due to self-weight = C_sW = (0.167)(42.75k) = 7.14 k (318 KN)

Total shear to walls A2-A7 & I2-I7 trib. to high and low sloped roofs = 30.8 k + 7.14 k = 37.9 k (169 KN)

Chord forces:
The low sloped roof diaphragm areas are conservatively assumed to span 90 feet (27.45 m) between shear wall lines 2 and 7 for transverse seismic forces. An equivalent running load, w, is found by dividing the total shear to the diaphragm by the span. The shear is made up of forces tributary to the low sloped roof areas and the reactions at the moment frames.

Shear force tributary to the low sloped roof areas = 18.1 kips (80.5KN)
Shear force to each diaphragm from moment frame reactions = 4(4.81 k) / 2 = 9.62 kips (42.8KN)
Total shear force = 18.1 k + 9.62 k = 27.7 kips (123 KN)
Equivalent running load, w = V / L = 27.7 / 90’ = 308 plf (4.49 KN/m)
Moment = $wL^2 / 8 = (.308)(90)^2 / 8$ = 312 kft (423 KNm)
T = M / d = 312 / 9.67’ = 32.3 kips (143 KN)

Chord forces for longitudinal seismic forces are negligible by inspection.

Collector forces:
The beams along grid lines B & H must collect the shear forces from the upper concrete shear / window walls and distribute them to the horizontal bracing of the low sloped roof area. The beams collect the shear force from the upper roof area and the shear force associated with wall lines B & H tributary to the low sloped roof areas.

Shear force from wall line B tributary to the upper roof diaphragm = 19.8 kips (88.1 KN)
Weight of wall line B tributary to the low sloped roof area = 15.2 k
Shear force from wall line B tributary to the low sloped roof area = C_sW = (0.167)(15.2k) = 2.54k (11.3KN)
Total shear force collected by beams along grid lines B & H = 19.8 + 2.54 = 22.34 kips (99.4 KN)
Unit collector shear force, v = V / L = 22.34 kips / 90’ = 248 plf (3.62 KN/m)
Collector force = $v*L_{collector}$ = (248 plf)(18’) = 4.46 kips (19.8 KN)

<u>Seismic forces to vertical resisting elements from entrance area diaphragm</u>

The concrete shear walls resist seismic shear forces tributary to the entrance area according to their tributary areas.

Transverse direction:

LOWER SLOPED ROOF @ ENTRANCE TRIBUTARY SEISMIC WEIGHTS (ROOF & NORMAL WALLS)

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)
LOWER ROOF						
Roof D1-F2	10.0	24.0	100.0	240.0	13.7	3.3
LONGITUDINAL WALLS						
Wall D1-D2	8.0	10.0	100.0	80.0	100.0	8.0
Wall F1-F2	8.0	10.0	100.0	80.0	100.0	8.0
					TOTAL	19.3

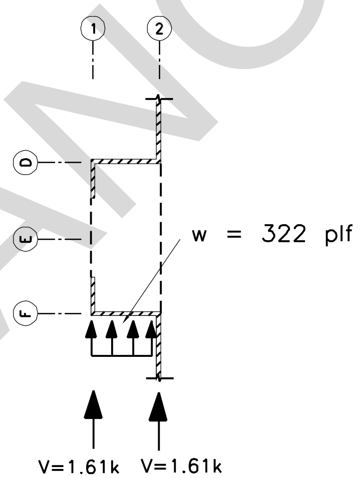
Total weight = 19.3 kips Seismic coefficient, C = 0.167 85.8 KN
Seismic force = C_sW = (0.167)(19.3) = 3.22 kips (14.3 KN)
Equivalent Running Load w = 3.22 k / 10 ft. = 322 plf (4.70 KN/m)

- Shear force to shear wall line 1 from entrance roof
  V = wL/2 = (322)(5') = 1.61 kips (7.16 KN)

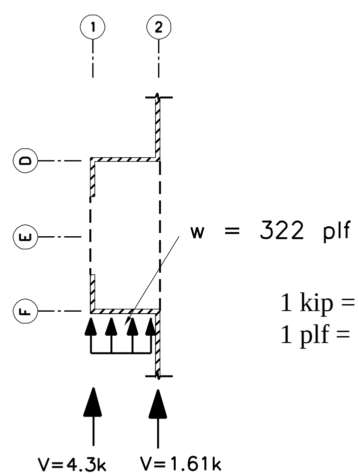
- Shear force to wall line 2 (same as wall line 1 due to symmetry)
  V = 1.61 kips (assume ½ goes to 2A-2D and ½ to 2F-2I = 0.805 (3.58 KN))

The shear walls along grid line 1 must also resist the forces associated with their self-weight. Wall line 2 has no seismic weight tributary to the entrance roof.

Weight of wall line 1 tributary to the entrance roof = 16.0 kips
Self-weight inertial force = C_sW = (0.167)(16.0) = 2.67 kips (11.88 KN)
Total shear to wall line 1 tributary to entrance roof diaphragm = (1.61 k) + (2.67 k) = 4.3 k (19.1 KN)



SHEAR TO VERTICAL ELEMENTS FROM ENTRANCE ROOF



SHEAR TO VERTICAL ELEMENTS FROM ENTRANCE ROOF + SELF INERTIA FORCES

1 kip = 4.448 KN
1 plf = 14.59 N/m

<u>Longitudinal direction:</u> Seismic forces in the longitudinal direction are resisted by shear wall lines D & F evenly due to symmetry. The diaphragm forces are due to the weight of the roof area and normal wall line 1. The weight of wall line 2 is included in the weight tributary to the upper roof.

LOWER SLOPED ROOF @ ENTRANCE TRIBUTARY SEISMIC WEIGHTS (ROOF AND NORMAL WALLS)

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)
LOWER ROOF						
Roof D1-F2	10.0	24.0	100.0	240.0	13.7	3.3
TRANSVERSE WALLS						
Wall D1-F1	---	---	100.0	160.0	100.0	16.0
					TOTAL	19.3

Total weight = 19.3 kips Seismic coefficient, C = 0.167 85.6 KN
Seismic force = C_sW = (0.167)(19.3) = 3.22 kips (14.3 KN)
Equivalent Running Load w = 3.22 k / 24 ft. = 134 plf (1.96 KN/m)

– Shear force to wall line D from entrance roof
  V = wL/2 = (134)(12') = 1.61 k (7.16 KN)

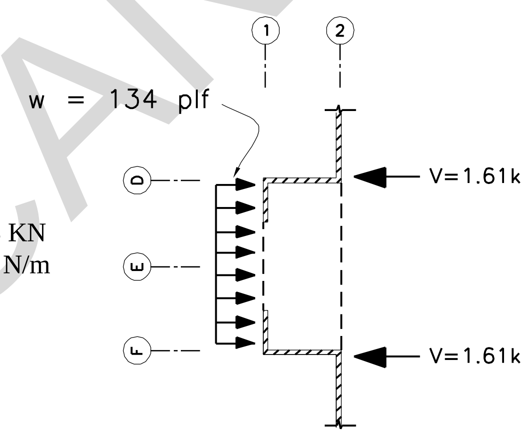
– Shear force to wall line F (same as wall line D)
  V = 1.61 k (7.16 KN)

The self-weight inertial effects of the shear walls due to the weight tributary to the entrance roof must now be added to the shears determined for shear wall lines D and F.

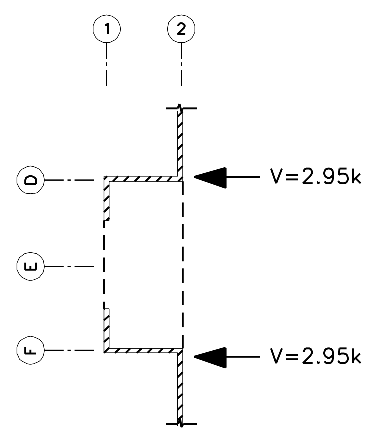
– Weight of wall line D tributary to the entrance roof = 8.0 kips
  Self-weight inertial force = C_sW = (0.167)(8.0) = 1.34 kips
  Total shear to wall line D tributary to entrance roof diaphragm = (1.61k) + (1.34k) = 2.95 k (13.1 KN)

– Wall line F same as line D
  Total shear = 2.95 k (13.1 KN)

1 kip = 4.448 KN
1 plf = 14.59 N/m



SHEAR TO VERTICAL ELEMENTS FROM ENTRANCE ROOF



SHEAR TO VERTICAL ELEMENTS FROM ENTRANCE ROOF + SELF INERTIA FORCES

Seismic forces to vertical resisting elements from sacristy area diaphragm

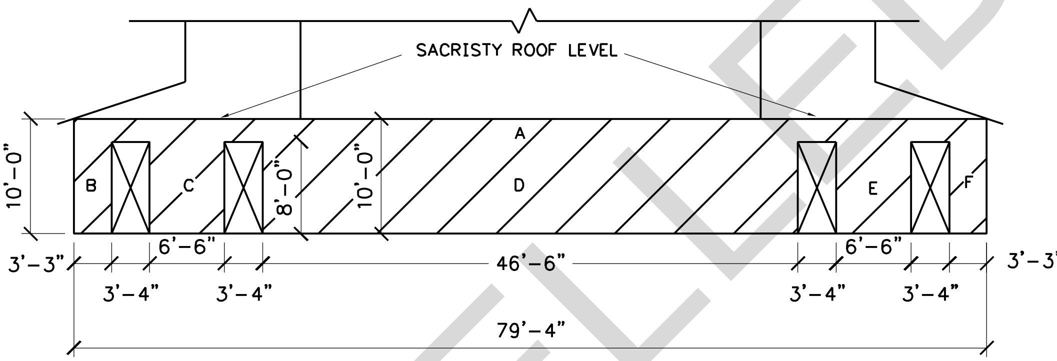
The concrete slab roofs of the sacristy areas act as rigid diaphragms. The shear force from these diaphragms are resisted by the vertical elements based on relative rigidities.

Transverse direction:

Shear wall line 7:

The rigidity of wall segments 7A-7C and 7G-7I was determined to be 5000 kips / in. (876 KN/mm) (see calculation in low sloped roof shear force section).

Shear wall line 8:



SHEAR WALL LINE 8 RIGIDITY (AT SACRISTY LEVEL)

Solid wall ABCDEF $\Delta_{solid} := \Delta_c(10\cdot ft, 76.33\,ft)$ $\Delta_{solid} = 1.6186 \cdot 10^{-5} \cdot in$

Subtract strip BCDEF $\Delta_{bcdef} := \Delta_f(8\cdot ft, 76.33\,ft)$ $\Delta_{bcdef} = 1.2647 \cdot 10^{-5} \cdot in$

$\Delta_a := \Delta_{solid} - \Delta_{bcdef}$ $\Delta_a = 3.5398 \cdot 10^{-6} \cdot in$

Add back in piers B, C, D, E and F $R_b := R_f(8\cdot ft, 3.25\,ft)$ $R_b = 959.581 \cdot \frac{1}{in}$

$R_F := R_b$ $R_F = 959.581 \cdot \frac{1}{in}$

$R_c := R_f(8\cdot ft, 3.33\,ft)$ $R_c = 1017.8097 \cdot \frac{1}{in}$

$R_e := R_c$ $R_e = 1017.8097 \cdot \frac{1}{in}$

$R_d := R_f(8\cdot ft, 46.5\,ft)$ $R_d = 47801.829 \cdot \frac{1}{in}$

$\Delta_{bcdef} := \dfrac{1}{R_b + R_c + R_d + R_e + R_F}$ $\Delta_{bcdef} = 1.9321 \cdot 10^{-5} \cdot in$

$\Delta_{wall} := \Delta_a + \Delta_{bcdef}$ $\Delta_{wall} = 2.2861 \cdot 10^{-5} \cdot in$

Rigidity of wall $R_{wall} := \dfrac{1\cdot kip}{\Delta_{wall}}$ $R_{wall} = 43742.6663 \frac{kip}{in}$ $R_{wall} = 7660.336 \cdot \frac{KN}{mm}$

Shear wall line 8 receives shear from both sacristy areas. Therefore, it is assumed that ½ of the rigidity of the entire wall (=1/2 * 43743 = 21872 k/in) will be used in determining the distribution of forces to vertical resisting elements tributary to each sacristy area.

For each sacristy; $R_{wall\ 7}$ = 5000 kips / in (876 KN/mm) $R_{wall\ 8}$ = 21872 kips /in (7660 KN/mm)

Shear force tributary to each sacristy = 5.3 kips (23.6 KN)

R_7 = 5000 kips / in R_8 = 21872 kips / in $V_{element} = V\dfrac{R_{element}}{\sum R}$

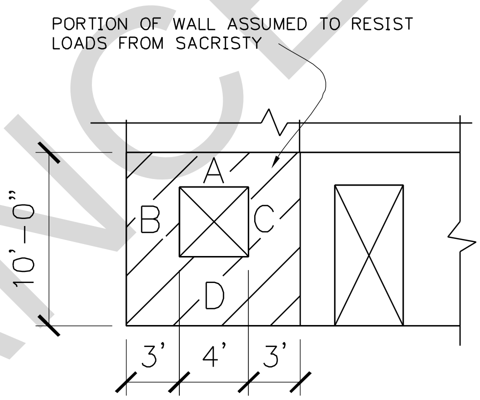
$V_{7A\text{-}7C}$ = 5.3(5000) / (21872+5000) = 0.99 kips (4.40 KN)
V_8= 5.3(21872)/(21872+5000) = 4.31 kips (19.2 KN)

Due to symmetry, $V_{7G\text{-}7I} = V_{7A\text{-}7C}$ = 0.99 kips (4.40 KN)

Total shear to wall line 8 (for forces trib. to sacristies) = 2(4.31 k) = 8.62 kips (38.3 KN)

<u>Longitudinal Direction</u>

Rigidity of shear wall lines A & I: It is assumed that only the end portion ot the walls (between grid lines 7 and 8) resist shear forces tributary to the sacristy areas (this is a very conservative assumption since the walls run the full length of the building).



SHEAR WALL LINES A & I

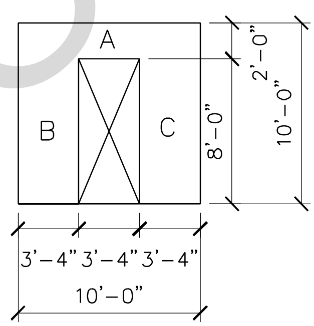
1 foot = 0.305m

Wall lines A & I at sacristy

Solid wall ABCD	$\Delta_{solid} := \Delta_c(10 \cdot ft, 10 \cdot ft)$	$\Delta_{solid} = 0.0003 \circ in$
Subtract strip BC	$\Delta_{bc} := \Delta_f(4 \cdot ft, 10 \cdot ft)$	$\Delta_{bc} = 5.1249 \cdot 10^{-5} \circ in$
	$\Delta := \Delta_{solid} - \Delta_{bc}$	$\Delta = 0.0003 \circ in$
Add back in piers B & C	$R_b := R_f(4 \cdot ft, 3 \cdot ft)$	$R_b = 3587.0055 \circ \frac{1}{in}$
	$R_c := R_f(4 \cdot ft, 3 \cdot ft)$	$R_c = 3587.0055 \circ \frac{1}{in}$
	$\Delta_{bc} := \frac{1}{R_b + R_c}$	$\Delta_{bc} = 0.0001 \circ in$
	$\Delta_{wall} := \Delta + \Delta_{bc}$	$\Delta_{wall} = 0.0004 \circ in$

$R_{wall} := \frac{1 \cdot kip}{\Delta_{wall}}$ $\quad R_{wall} = 2448 \circ \frac{kip}{in}$ $\quad R_{wall} = 429 \circ \frac{KN}{mm}$

Rigidity of lower portions of walls C7-C8 and G7-G8



LOWER PORTIONS OF WALLS
C7-C8 AND G7-G8

1 foot = 0.305m

Solid wall ABCD	$\Delta_{solid} := \Delta_c(10\cdot ft, 10\cdot ft)$	$\Delta_{solid} = 0.0003 \cdot in$
Subtract strip BCD	$\Delta_{bcd} := \Delta_f(8\cdot ft, 10\cdot ft)$	$\Delta_{bcd} = 0.0001 \cdot in$
	$\Delta_a := \Delta_{solid} - \Delta_{bcd}$	$\Delta_a = 0.0002 \cdot in$
Add back in piers B and C	$R_b := R_f(8\cdot ft, 3.33\cdot ft)$	$R_b = 1017.8097 \cdot \frac{1}{in}$
	$R_c := R_b$	$R_c = 1017.8097 \cdot \frac{1}{in}$
	$\Delta_{bc} := \frac{1}{R_b + R_c}$	$\Delta_{bc} = 0.0005 \cdot in$
	$\Delta_{total} := \Delta_a + \Delta_{bc}$	$\Delta_{total} = 0.0007 \cdot in$
Rigidity of wall	$R_{wall} := \frac{1\cdot kip}{\Delta_{total}}$ $R_{wall} = 1449.6 \cdot \frac{kip}{in}$	$R_{wall} = 253.9 \cdot \frac{KN}{mm}$

For each sacristy;

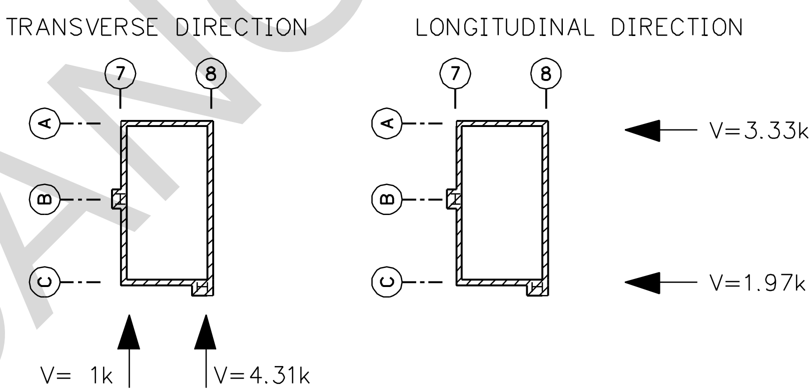
Shear force tributary to each sacristy = 5.3 kips (23.6 KN)

$R_{A7\text{-}A8}$ = 2448 kips / in $R_{C7\text{-}C8}$ = 1450 kips /in $V_{element} = V \dfrac{R_{element}}{\sum R}$

$V_{A7\text{-}A8}$ = 5.3(2448) / (2448 + 1450) = 3.33 kips (14.8 KN)
$V_{C7\text{-}C8}$= 5.3(1450)/(2448+1450) = 1.97 kips (8.76 KN)

Due to symmetry, $V_{I7\text{-}I8} = V_{A7\text{-}A8}$ = 3.33 kips (14.8 KN) $V_{G7\text{-}G8} = V_{C7\text{-}C8}$ = 1.97 kips (8.97 KN)

1 kip = 4.448 KN



B-6 *Determine cr and cm*

The sacristy and lower sloped roof areas have rigid diaphragms. The torsional forces to the vertical resisting elements are calculated by finding the tributary mass and stiffness eccentricities.

<u>Sacristy</u>

Center of Mass

Element	Weight (kips)	x (ft.)	y (ft.)	Wx (kip*ft)	Wy (kip*ft)
Roof Deck	10.4	5.0	9.8	52.1	102.5
Wall 8A-8C	8.0	10.0	9.8	80.0	78.4
Wall A7-A8	3.8	5.0	19.7	18.8	73.8
Wall C7-C8	9.8	5.0	0.0	48.8	0.0
S =	31.9			199.6	254.6

$cm = \sum Wx / \sum w$

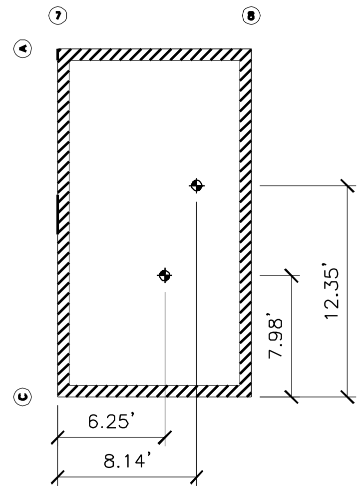
cm_x = (199.6) / (31.9) = 6.25' (1.91m)
cm_y = (254.6) / (31.9) = 7.98' (2.43m)

Center of Rigidity

Element	R_{cx} (kip / in)	R_{cy} (kip / in)	x (ft.)	y (ft.)	yR_{cx}	xR_{cy}
Wall A7-A8	2448	0	0	19.67	48152.2	0
Wall C7-C8	1450	0	0	0	0.0	0
Wall 7A-7C	0	5000	0	0	0.0	0
Wall 8A-8C	0	21872	10	0	0.0	218720
S =	3898	26872			48152	218720

$cr = \sum xR / \sum R$

cr_x = (48152) / (3898) = 12.35' (3.77m)
cr_y = (218720) / (26872) = 8.14' (2.48m)



1 foot = 0.305m

CENTER OR MASS & RIGIDITY FOR SACRISTY AREAS

Lower sloped roof areas

Center of mass:

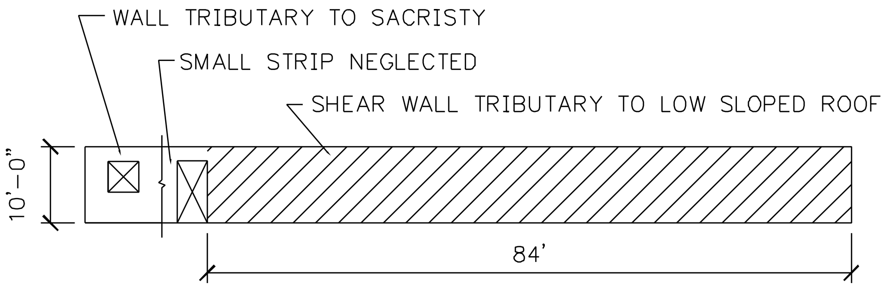
Element	Weight (kips)	x (ft.)	y (ft.)	Wx (kip*ft)	Wy (kip*ft)
Roof Deck	15.2	45.0	4.8	683.1	73.4
Wall A2-A7	42.8	45.0	9.7	1923.8	413.4
Wall B2-B7	38.0	45.0	0.0	1711.1	0.0
Wall 2A-2B	6.2	0.0	4.8	0.0	30.0
Wall 7A-7B	6.2	90.0	4.8	558.0	30.0
S =	108.4			4876.0	546.7

$cm = \sum Wx / \sum w$

cm_x = (4876) / (108.4) = 45' (13.73m)
cm_y = (546.7) / (108.4) = 5.04' (1.54m)

Center of rigidity

The rigidity of shear wall lines A and I must be calculated;



SHEAR WALL LINES A & I

Solid wall $\Delta_{solid} := \Delta_c(10 \cdot ft, 84 \cdot ft)$ $\Delta_{solid} = 1.4637 \cdot 10^{-5} \cdot in$

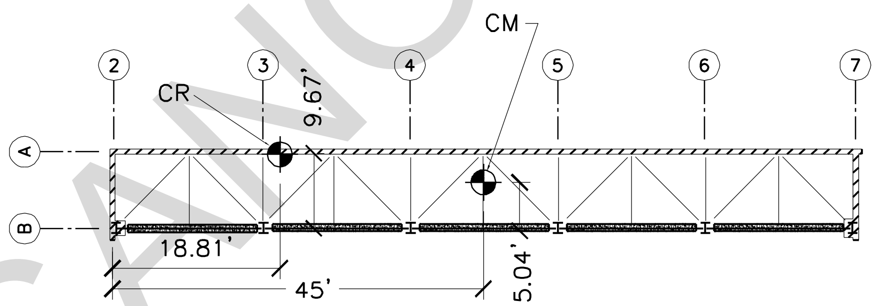
Rigidity of wall $R_{wall} := \dfrac{1 \cdot kip}{\Delta_{solid}}$ $R_{wall} = 68320 \cdot \dfrac{kip}{in}$ $R_{wall} = 11964 \cdot \dfrac{KN}{mm}$

Element	R_{cx} (kip / in)	R_{cy} (kip / in)	x (ft.)	y (ft.)	yR_{cx}	xR_{cy}
Wall A2-A7	68320	0	0	9.67	660654.4	0
Wall 2A-2D	0	18918	0	0	0	0
Wall 7A-7C	0	5000	90	0	0	450000
S =	68320	23918			660654	450000

$cr = \sum xR / \sum R$

cr_x = (660654) / (68320) = 9.67' (2.95 m)
cr_y = (450000) / (23918) = 18.81' (5.74 m)

1 foot = 0.305 m



CENTER OF MASS AND RIGIDITY FOR LOW SLOPED ROOF AREAS

B-7 *Perform torsional analysis*

<u>Sacristy Areas</u>

Due to symmetry both of the sacristies will behave the same in torsion. Only one of the sacristies is analyzed and the results are used for both. The torsional eccentricity is taken as the offset between the centers of mass and rigidity and an additional 5% accidental eccentricity.

Torsional force resisted by each vertical element = $F_T = T \dfrac{Rd}{\sum Rd^2}$

Transverse Forces:

Center of Mass in X direction =	6.25 ft.
Center of Rigidity in X direction =	8.14 ft.
Actual Eccentricity in X direction =	1.89 ft.
Perpendicular Span in X direction =	10 ft.
Accidental Eccentricity (5% of Span) =	0.5 ft.
Design Eccentricity in X direction =	2.39 ft. (0.73m)
Seismic Force Tributary to each Sacristy =	5.3 kips (23.6 KN)
Torsional Force (V * e) =	12.70 kip*ft (17.2 KNm)

Element	R	d	Rd	Rd^2	Torsional Force (kip)
Wall 7A-7C	5000	8.14	40697	331243	0.68
Wall 8	21872	-1.86	-40697	75723	-0.68*
Wall A7-A8	2448	7.32	17912	131061	0.30
Wall C7-C8	1450	12.35	17912	221267	0.30
			Σ =	759294	

*Note: Negative torsional force contributions are neglected.

Longitudinal Forces:

Center of Mass in Y direction =	7.98 ft.
Center of Rigidity in Y direction =	12.35 ft.
Actual Eccentricity in Y direction =	4.38 ft.
Perpendicular Span in Y direction =	19.67 ft.
Accidental Eccentricity (5% of Span) =	0.9835 ft.
Design Eccentricity in Y direction =	5.36 ft. (1.63m)
Seismic Force Tributary to each Sacristy =	5.3 kips (23.6 KN)
Torsional Force (V * e) =	28.52 kip*ft (38.7 KNm)

Element	R	d	Rd	Rd^2	Torsional Force (kip)
Wall 7A-7C	5000	8.14	40697	331243	1.529
Wall 8	21872	1.86	40697	75723	1.529
Wall A7-A8	2448	-7.32	-17912	131061	-0.673*
Wall C7-C8	1450	12.35	17912	221267	0.673
			Σ =	759294	

*Note: Negative torsional force contributions are neglected.

Lower Sloped Roof Areas

Due to symmetry both of the lower sloped roof areas will behave the same in torsion. Only one of the areas is analyzed and the results are used for both. In addition to the torsion created between the offset between the centers of mass and rigidity, the transfer of shear into the diaphragm from vertical resisting elements tributary to the upper roof diaphragm creates torsion. In the transverse direction the braced moment frames transfer their shear into the horizontal diaphragm. This creates a torsional force equal to the shear transferred times the distance between the center of application of the moment frame forces and the center of rigidity (45’-18.81’= 26.2’). In the longitudinal direction the offset between the upper shear walls (along lines B & H) and the lower resisting elements (walls along lines A & I) creates torsional forces.

Transverse Forces:

Center of Mass in X direction = 45.00 ft.
Center of Rigidity in X direction = 18.81 ft.
Actual Eccentricity in X direction = 26.19 ft.
Perpendicular Span in X direction = 90 ft.
Accidental Eccentricity (5% of Span) = 4.5 ft.
Design Eccentricity in X direction = 30.69 ft.
Seismic Force Tributary to each Diaphragm = 18.1 kips
Eccentricity Torsional Force (V * e) = 554.2 kipft
Shear Force from Upper Roof (moment frames) = 19.24 kips =4 x 4.81
Distance to Center of Rigidity = 26.19 ft
Torsion Force from Upper Roof = 503.8 kipft
Total Torsion = 1058.0 kipft (1434 KNm)

Element	R	d	Rd	Rd^2	Torsional Force (kip)
Wall A2-A7	68320	0.00	0	0	0.0
Wall 2A-2D	18918	-18.81	-355848	6693493	-11.8*
Wall 7A-7C	5000	71.19	355950	25340081	11.8
			Σ =	32033573	

*Note: Negative torsional force contributions are neglected.

Longitudinal Forces:

Center of Mass in Y direction = 5.05 ft.
Center of Rigidity in Y direction = 9.67 ft.
Actual Eccentricity in Y direction = 4.62 ft.
Perpendicular Span in Y direction = 9.67 ft.
Accidental Eccentricity (5% of Span) = 0.4835 ft.
Design Eccentricity in Y direction = 5.11 ft.
Seismic Force Tributary to each Diaphragm = 18.1 kips
Eccentricity Torsional Force (V * e) = 92.25 kipft
Shear Force from Upper Roof (wall line B) = 19.8 kips
Distance to Center of Rigidity = 9.67 ft
Torsion Force from Upper Roof = 191.5 kipft
Total Torsion = 283.7 kipft (385 KNm)

Element	R	d	Rd	Rd^2	Torsional Force (kip)
Wall A2-A7	68320	0.00	0	0	0.0
Wall 2A-2D	18918	18.81	355848	6693493	3.2
Wall 7A-7C	5000	71.19	355950	25340081	3.2
			$\Sigma =$	32033573	

Total Shear Force to Shear Walls:

1 kip = 4.448 KN

Transverse Forces

Element	Shear from Upper Roof (kips)	Shear from Lower Roof (kips)	Shear from Entrance Roof (kips)	Shear from Sacristy (kips)	Torsional Shear from Lower Roof (kips)	Torsional Shear from Sacristy (kips)	Total Shear force (kips)
Wall 1D-1F	0	0	4.3	0	0	0	4.3
Wall 2A-2D	7.9	21.91	0.805	0	0	0	30.6
Wall 2F-2I	7.9	21.91	0.805	0	0	0	30.6
Wall 7A-7C	3.015	5.79	0	0.99	11.8	0.68	22.3
Wall 7G-7I	3.015	5.79	0	0.99	11.8	0.68	22.3
Wall 8A-8I	12.3	0	0	8.62	0	0	20.9
Wall D1-D2	0	0	0	0	0	0	0.0
Wall F1-F2	0	0	0	0	0	0	0.0
Wall A2-A7	0	0	0	0	0	0	0.0
Wall I2-I7	0	0	0	0	0	0	0.0
Wall B2-B7	0	0	0	0	0	0	0.0
Wall H2-H7	0	0	0	0	0	0	0.0
Wall A7-A8	0	0	0	0	0	0.3	0.3
Wall I7-I8	0	0	0	0	0	0.3	0.3
Wall C7-C8	0	0	0	0	0	0.3	0.3
Wall G7-G8	0	0	0	0	0	0.3	0.3

Longitudinal Forces

Element	Shear from Upper Roof (kips)	Shear from Lower Roof (kips)	Shear from Entrance Roof (kips)	Shear from Sacristy (kips)	Torsional Shear from Lower Roof (kips)	Torsional Shear from Sacristy (kips)	Total Shear force (kips)
Wall 1D-1F	0	0	0	0	0	0	0
Wall 2A-2D	0	0	0	0	3.2	0	3.2
Wall 2F-2I	0	0	0	0	3.2	0	3.2
Wall 7A-7C	0	0	0	0	3.2	1.53	4.73
Wall 7G-7I	0	0	0	0	3.2	1.53	4.73
Wall 8A-8I	0	0	0	0	0	3.06	3.06
Wall D1-D2	0	0	2.95	0	0	0	2.95

Wall F1-F2	0	0	2.95	0	0	0	2.95
Wall A2-A7	19.8	18.1	0	0	0	0	37.9
Wall I2-I7	19.8	18.1	0	0	0	0	37.9
Wall B2-B7	19.8	0	0	0	0	0	19.8
Wall H2-H7	19.8	0	0	0	0	0	19.8
Wall A7-A8	0	0	0	3.33	0	0	3.33
Wall I7-I8	0	0	0	3.33	0	0	3.33
Wall C7-C8	6.86	0	0	1.97	0	0.67	9.5
Wall G7-G8	6.86	0	0	1.97	0	0.67	9.5

B-8 *Determine need for redundancy factor, r.*

Transverse Direction: Seismic forces in the transverse direction are resisted by a combination of concrete shear walls and steel moment frames. The majority of the shear force is resisted by shear wall line 8. For shear walls, r_{max} is equal to the shear in the most heavily loaded wall multiplied by $10/l_w$, divided by the story shear:

$$\rho = 2 - \frac{20}{r_{max}\sqrt{A}} \quad , \; r_{max} = \frac{V}{V_T}\frac{10}{l_w}$$

$$r_{max} = \frac{20.9}{106}\frac{10}{40'} = 0.05\,, \quad \rho = 2 - \frac{20}{0.05\sqrt{8174}} = -2.4\,, \text{ use } 1.0$$

Longitudinal Direction Seismic forces in the longitudinal direction are resisted entirely by concrete shear walls.

$$r_{max} = \frac{57.7}{106}\frac{10}{84} = 0.065\,, \quad \rho = 2 - \frac{20}{0.065\sqrt{8174}} = -1.40\,, \text{ use } 1.0$$

B-9 *Determine need for overstrength factor, W_o*

The overstrength factor is used for the design of the collectors (beams along lines B and H that support the upper window/shear walls.

B-10 *Calculate combined load effects*

The load combinations from ASCE 7-95 are:

(1) 1.4D
(2) 1.2D + 1.6L + 0.5L_r
(3) 1.2D + 0.5L + 1.6L_r
(4) 1.2D + E + 0.5L
(5) 0.9D + E

Where $E = \rho Q_E \pm 0.2\, S_{DS}D$ Eq. 4-4 & 4-5

When specifically required by FEMA 302 (Collectors, their connections, and bracing connections for this example) the design seismic force is defined by:

$E = \Omega_0 Q_E \pm 0.2 S_{DS}D$ Eq. 4-6 & 4-7

The term $0.2S_{DS}D$ is added to account for the vertical earthquake accelerations.

$0.2S_{DS}D = 0.2(1.0)D = 0.2\ D$
Therefore, 0.2 will be added to the dead load factor for load combinations 4 and 5.

B-11 *Determine structural member sizes*

(a) *Upper roof diaphragm forces* - The upper roof diaphragm consists of metal decking. The decking is selected from a manufacture's catalog with the required shear and gravity capacities.

Transverse direction:
Diaphragm shear force demand = 4.21k (18.7 KN)
Diaphragm shear depth = 60' (18.3m)
Unit shear demand = (4.21k) / (60') = 70 plf (1021 N/m)

Longitudinal direction:
Diaphragm shear force demand (grid lines 2-7) = 14.5 k
Diaphragm shear depth (grid lines 2-7) = 90'
Unit shear demand (grid lines 2-7) = (14.5k) / (90') = 161 plf (2.35 KN/m)
Diaphragm shear force demand (grid lines 7-8) = 6.42 k
Diaphragm shear depth (grid lines 7-8) = 10'
Unit shear demand (grid lines 7-8) = (6.42k) / (10') = 642 plf (9.37 KN/m)

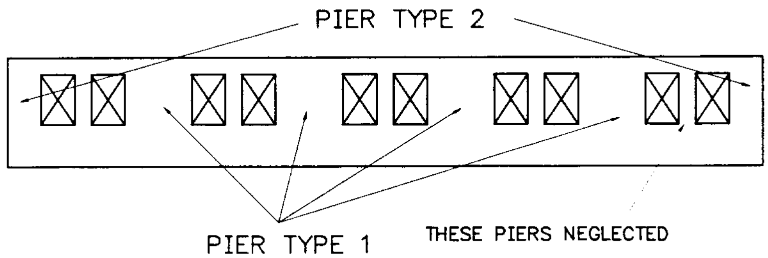
(b) *Reinforced Concrete Shear Walls*
Some of the shear walls resist forces in both the transverse and longitudinal direction due to torsion. Therefore, per FEMA 302 Section 5.2.7 orthogonal effects must be considered. The walls are checked for 100% of the force in one direction plus 30% in the orthogonal direction.

Element	Total Transverse Shear (kips)	Total Longitudinal Shear (kips)	100% Transverse + 30% Longitudinal (kips)	100% Longitudinal + 30% Transverse (kips)
Wall 1D-1F	4.3	0.0	4.3	1.3
Wall 2A-2D	30.6	3.2	31.6	12.4
Wall 2F-2I	30.6	3.2	31.6	12.4
Wall 7A-7C	22.3	4.7	23.7	11.4
Wall 7G-7I	22.3	4.7	23.7	11.4
Wall 8A-8I	20.9	3.1	21.8	9.3
Wall D1-D2	0.0	3.0	0.9	3.0
Wall F1-F2	0.0	3.0	0.9	3.0
Wall A2-A7	0.0	37.9	11.4	37.9
Wall I2-I7	0.0	37.9	11.4	37.9
Wall B2-B7	0.0	19.8	5.9	19.8
Wall H2-H7	0.0	19.8	5.9	19.8
Wall A7-A8	0.3	3.3	1.3	3.4
Wall I7-I8	0.3	3.3	1.3	3.4
Wall C7-C8	0.3	9.5	3.2	9.6
Wall G7-G8	0.3	9.5	3.2	9.6

1 kip 4.448 KN

Supported concrete shear / window walls - The supported concrete walls along grid lines B & H transfer the shear forces from the upper roof diaphragm to the beam collectors along the same grid lines. The shear from the collectors is then transferred to exterior shear wall lines A & I through horizontal bracing.

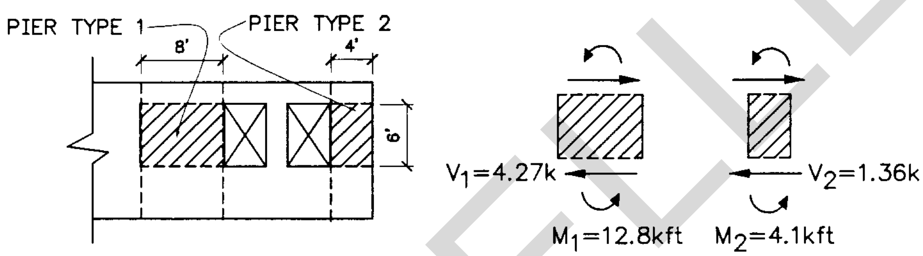
Shear from upper roof diaphragm = 19.8k (88.1 KN)



The pier elements resist shear in relation to their relative rigidities. It is assumed that the slender piers between the windows resist no shear.

R_1 = 8993 kips / in. (157 KN/mm) R_2 = 2865 kips / in. (502 KN/mm) (calcs. not shown)
ΣR = 4(8993) + 2(2865) = 41702 kips / in. (7303 KN/mm)

V_1 = 19.8(8993/41702) = 4.27 k (19.0 KN) V_2 = 19.8(2865/41702) = 1.36 k (6.05 KN)
M_1 = VL/2 = (4.27)(6) / 2 = 12.8 kft (17.4 KNm) M_2 = (1.36)(6) / 2 = 4.1 kft (5.56 KNm)



Shear strength of concrete: $V_n = A_{cv}\left(2\sqrt{f_c'} + \rho_n f_y\right)$ (ACI 318 Eq. 21-6)

ACI 318 Sec. 21.6.2.1 requires that $\rho > 0.0025$ for shear stress > $A_{cv}\sqrt{f_c'} = A_{cv}\sqrt{3000} = A_{cv}(54.8\text{psi})$
Assume all walls have stress exceeding this value (conservative). (378 KN/m^2)
Use 2-#5 bars at 12" in each direction for all walls (ρ=(0.31in.^{2}/(12 x 8) = 0.003 > 0.0025))

$V_n = A_{cv}\left(2\sqrt{3000} + 0.003(60000)\right) = A_{cv}(290\text{psi})$ or (2000 KN/m^2)

$\phi V_n = 0.6(290)A_{cv} = A_{cv}(174\text{ psi})$ (1200 KN/m^2) (ϕ = 0.6 per ACI 318 Sec. 9.3.4.1)
Shear strength of pier type 1 = (96")(8")(174) = 134 k (596 KN) > 4.27 k (19.0 KN), OK
Shear strength of pier type 2 = (48")(8")(174) = 67 k (298 KN) > 1.36 k (6.05 KN), OK

Check need for boundary zones; (per FEMA 302 Sec. 9.1.1.13)
Section 9.1.1.13 of FEMA 302 exempts walls & wall segments that meet the following conditions from needing boundary zones;

1. $P_u \le 0.10A_g f'_c$ (all wall segments in the structure meet this requirement since they are non bearing with low axial loads).

and either:

2. $M_u/V_u l_w \le 1.0$ or

$V_u \le 3A_{cv}\sqrt{f_c'} = A_{cv}(164\text{psi})$ and $M_u/V_u l_w \le 3.0$

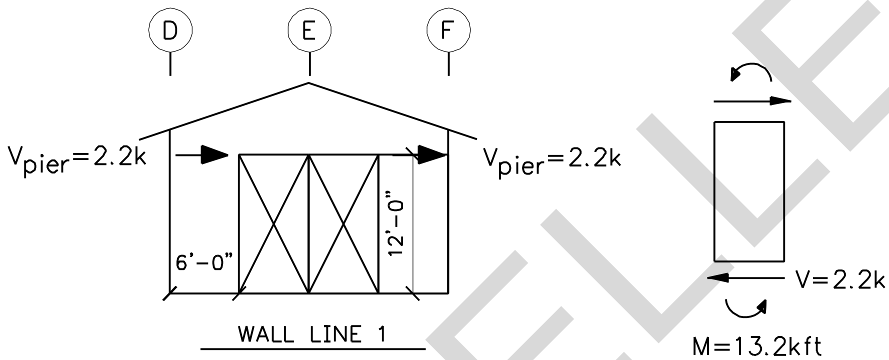
All of the shear walls have low shear stresses due to large amount of wall length; thus, by inspection,

the $V_u \le 3A_{cv}\sqrt{f_c'} = A_{cv}(164\text{psi})$ term is satisfied. Therefore, no boundary zones will be required in any wall segment if $M_u/V_u l_w \le 3.0$ is satisfied.
Pier 1 M/Vl = 12.8/4.27(8) = 0.37 <3.0 (No boundary zones required)
Pier 2 M/Vl = 4.1/1.36(4) = 0.75 <3.0 (No boundary zones required)

Shear wall line 1

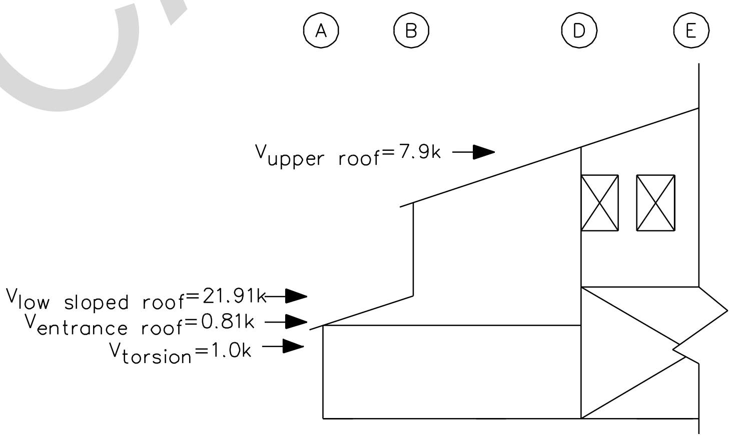
Shear wall line 1 resists forces tributary to the entrance only. The wall line consists of two pier elements which each resist ½ of the total loads.

$V_{wall\ 1}$ = 4.3 kips $V_{each\ pier}$ = ½(4.3) = 2.2 k (9.79 KN)
M = Vl / 2 = (2.2)(12') / 2 = 13.2kft (17.90 KNm)

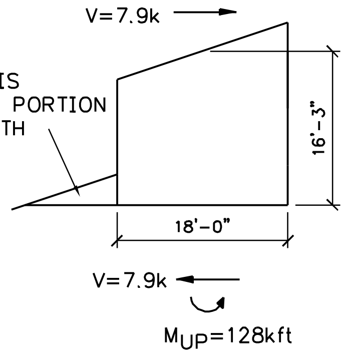


Shear strength of pier = (72")(8")(174) = 100 k (445 KN) > 2.2 k (9.79 KN), OK
Check need for boundary zones;
M/Vl = 13.2/2.2(6) =1.0 < 3.0 (No boundary zones required)

Shear wall line 2 (Between grid lines B & H)
Shear wall line 2 resists shear forces from the upper roof, lower sloped areas and entrance diaphragms.
V_{upper} = 7.9k (35.1 KN)
$V_{low\ sloped\ roof}$ = 21.91k (97.5 KN)
$V_{entrance}$ = .81k (3.6 KN)
$V_{torsion}$ = 1 k (4.448 KN) (= 30% of longitudinal torsion force for 100% trans + 30% longit. load combination).

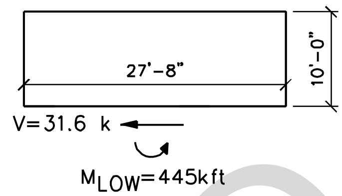


1 kip = 4.448 KN
1 kipft = 1.356 KNm
1 foot = 0.305m



UPPER PORTION

1 kip = 4.448 KN
1 kipft = 1.356 KNm
1 foot = 0.305m



LOWER PORTION

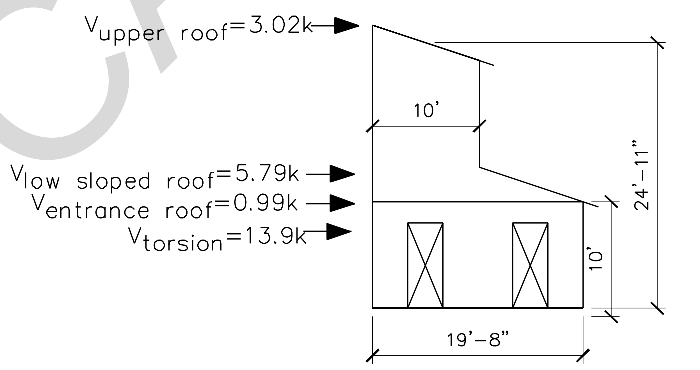
Shear strength of upper portion of wall (18')(12"/')(8")(174psi) = 301k (1348KN)> 7.9k (35.1 KN), OK
Shear strength of lower portion of wall (27.7')(12"/')(8")(174psi) = 463k (2059KN) >31.6k (141KN), OK

M_{upper} = (7.9k)(16.25') = 128 kft (174 KNm)
M/Vl = 128 / (7.9)(18) = 0.9 < 3.0 (no boundary zones required in upper wall portion)
M_{base} = (7.9)(26.25) + (21.91+0.81+1)(10') = 445 kft (603 KNm)
M/Vl = 445 / (24)(27.7) = 0.67 <3.0 (no boundary zones required in lower portion of wall)

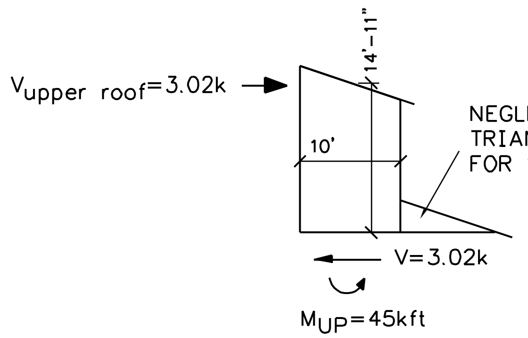
Shear wall line 7 (Walls 7A-7C & 7G-7I same by symmetry)

Shear wall line 7 resists shear forces from the upper roof, lower sloped areas and the sacristies.
V_{upper} = 3.015k (13.41 KN)
$V_{low\ sloped\ roof}$ = 5.79k (25.75 KN)
$V_{sacristy}$ = .99k (4.40 KN)
Torsion:
From lower sloped roofs in transverse direction = 11.8 k
From sacristy diaphragm in transverse direction = 0.68 k
From lower sloped roof in longitudinal direction x 30% 3.2(0.30) = 0.96 k
From sacristy in longitudinal direction x 30% = 1.53(0.30) = 0.46k
Total torsion (for orthogonal effects = 13.9k (61.8 KN)

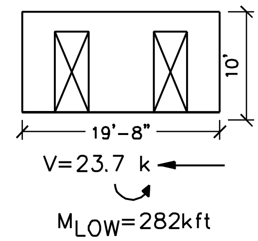
Total shear = 3.02 k + 5.79k + 0.99k + 13.9k = 23.7 k (105 KN)



SHEAR WALLS 7A-7C AND 7G-7I



UPPER PORTION



LOWER PORTION

Shear strength of upper portion of wall (10')(12"/')(8")(174psi) = 167k (743KN)> 3.02k (13.4KN), OK
M_{upper} = (3.02k)(14.92') = 45kft (61.0 KNm)
M/Vl = 45 / (3.02)(10) = 1.5 < 3.0 (no boundary zones required in upper wall portion)

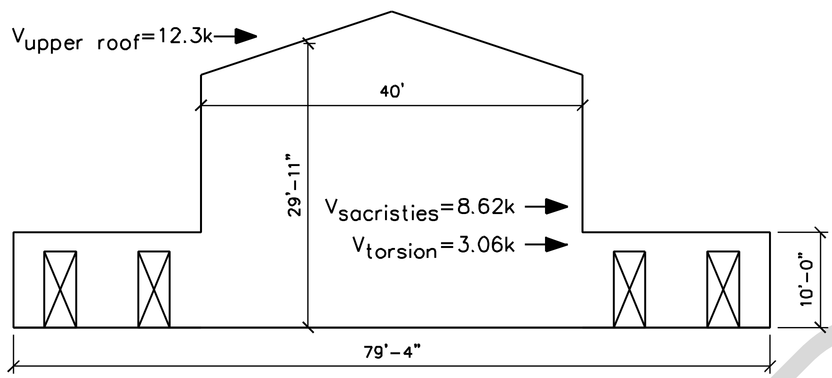
Distribute shear to wall piers in lower portion of wall based on relative rigidities;
R_{outer} = 960 kip / in R_{middle} = 4147 kip / in (previously calculated)
V_{outer} = 23.7k (960)/(2 x 960 + 4147) = 3.75 kips (16.7 KN)
V_{middle} = 23.7k (4147)/(2 x 960 + 4147) = 16.2 kips (72.1 KN)
Shear strength of outer pier = (3.25')(12"/')(8")(174) = 54.2 k (241 KN) > 3.75 k (16.68 KN), OK
Shear strength of middle pier = (6.5')(12"/')(8")(174) = 109 k (485 KN) > 16.2 k (72.1 KN), OK
M_{outer} = (8')(3.75k)/2 = 15 kft (20.34 KNm)
M/Vl = 15 / (3.75)(3.25') = 1.23 < 3.0 (no boundary zones required in outer piers)
M_{middle} = (8')(16.2k)/2 = 65 kft (88.14 KNm)
M/Vl = 65 / (16.2)(6.5) = 0.62 < 3.0 (no boundary zones required in middle pier)

Check overall action of lower portion of wall;

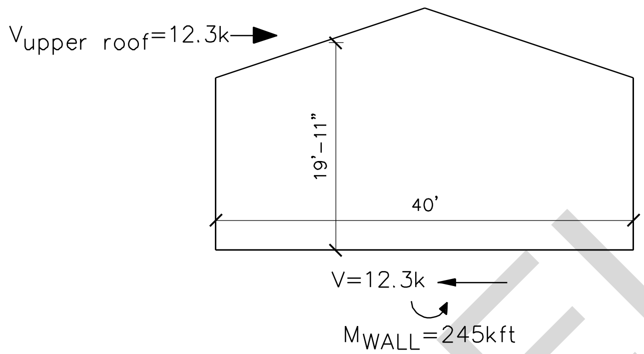
M_{base} = (3.02)(24.92) + (20.7)(10') = 282 kft (382 KNm)
M/Vl = 282 / (23.7)(19.67) = 0.61 <3.0 (no boundary zones required in lower portion of wall)

Shear wall line 8

Shear wall line 8 resists shear forces from the upper roof and the sacristies.
V_{upper} = 12.3k (54.7 KN)
$V_{sacristy}$ = 8.62k (38.3 KN)
Torsion: From sacristies in longitudinal direction x 30% =2 x 1.53(0.30) = 0.92k (4.09 KN)
Total shear = 12.3 k + 8.6k + 0.92 = 21.82 k (97.1 KN)

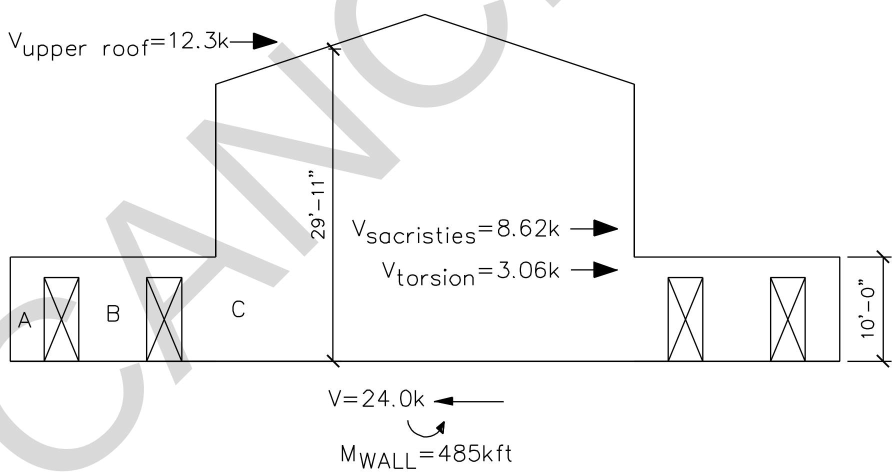


SHEAR WALL LINE 8



1 kip 4.448 KN
1 kipft = 1.356 KNm
1 foot = 0.305m

UPPER PORTION OF WALL



LOWER PORTION OF WALL

Shear strength of upper portion of wall (40')(12"/')(8")(174psi) = 668k (2971KN) > 12.3k (54.7KN), OK
M_{upper} = (12.3k)(19.92') = 245kft (332 KNm)
M/Vl = 245 / (12.3)(40) = 0.5 < 3.0 (no boundary zones required in upper wall portion)

Distribute shear to wall piers in lower portion of wall based on relative rigidities;
R_A = 960 kip / in $\quad$ R_B = 1018 kip / in $\quad$ R_C = 47802 kip / in (previously calculated)
ΣR = 2(960) + 2(1018) + 47802 = 51758 kips / in

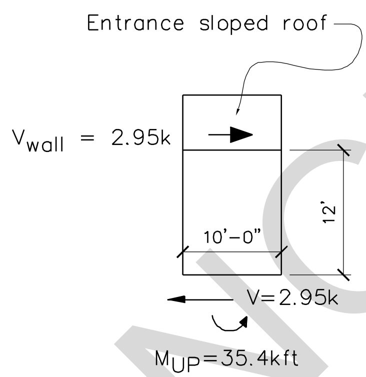
V_A = 24.0k (960)/(51758) = 0.45 kips (2.00 KN)
V_B = 24.0k (1018)/(51758) = 0.47 kips (2.09 KN)
V_c = 24.0k (47802)/(51758) = 22.2 kips (98.7 KN)
Shear strength of pier A= (3.25')(12"/')(8")(174) = 54.2k (241KN) > 0.45k (2.00 KN), OK
Shear strength of pier B= (3.33')(12"/')(8")(174) = 55.6k (247KN) > 0.47k (2.09 KN), OK
Shear strength of pier C= (46.5')(12"/')(8")(174) = 777 k > 22.2 k, OK
M_A = (8')(.45)/2 = 1.8 kft (2.44 KNm)
M/Vl = (1.8) / (0.45)(3.25) = 1.23 < 3.0 (no boundary zone required)
M_B = (8')(.47)/2 = 1.9 kft (2.58 KNm)
M/Vl = (1.9) / (0.47)(3.33) = 1.23 < 3.0 (no boundary zone required)
M_C = (8')(22.2)/2 = 89 kft (121 KNm)
M/Vl = 89 / (22.2)(46.5') = 0.09 < 3.0 (no boundary zones required in outer piers)

Check overall action of lower portion of wall;

M_{base} = (12.3)(29.92) + (11.68)(10') = 478 kft (648 KNm)
M/Vl = 478 / (24)(79.3) = 0.3 <3.0 (no boundary zones required in lower portion of wall)

Shear walls D1-D2 and F1-F2

Shear wall lines D & F resist force tributary to the entrance only.

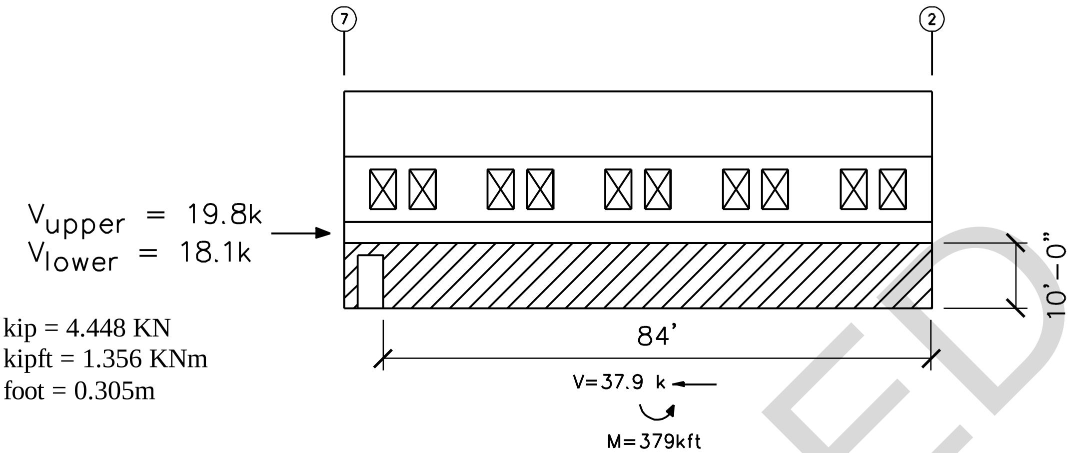


1 kip = 4.448 KN
1 kipft = 1.356 KNm
1 foot = 0.305 m

Shear Wall Line D1–D2 (Wall Line F1–F2 similar)

V_{wall} = 2.95 kips (13.12 KN)
M = Vl = (2.95)(12') = 35.4kft (48.0 KNm)
Shear strength of wall = (120")(8")(174) = 167 k (743 KN) > 2.95 k (13.1 KN), OK
Check need for boundary zones;
M/Vl = 35.4/2.95(10) =1.2 < 3.0 (No boundary zones required)

Shear walls A2-A7 and I2-I7



1 kip = 4.448 KN
1 kipft = 1.356 KNm
1 foot = 0.305m

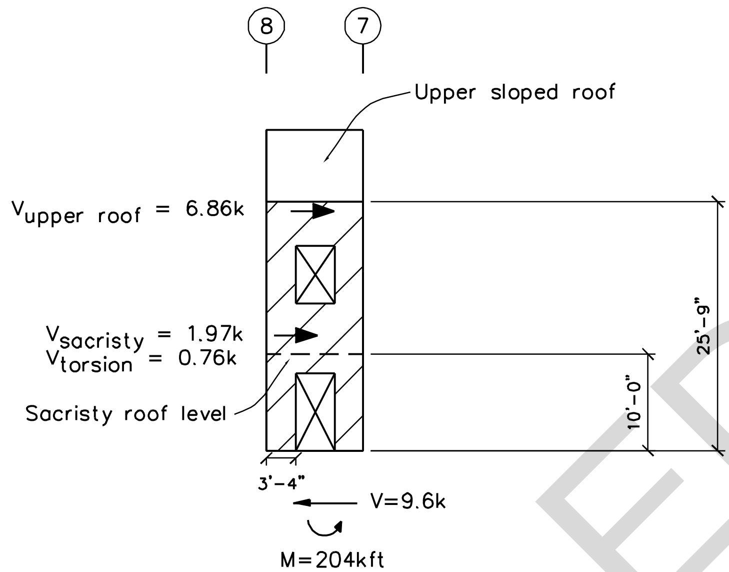
WALL LINE A2–A7 (WALL I2–I7 SIMILAR)

V_{wall} = 37.9 kips (169 KN)
M = Vl = (37.9)(10') = 379kft (514 KNm)
Shear strength of wall = (84 x 12")(8")(174) = 1403 k (6241 KN) > 37.9 k (169 KN), OK
Check need for boundary zones;
M/Vl = 379/37.9(84) =.12 < 3.0 (No boundary zones required)

Shear wall lines (C7-C8 and G7-G8)

Shear wall lines C & G resist shear forces from the upper roof and the sacristies.
V_{upper} = 6.86k (30.5 KN)
$V_{sacristy}$ = 1.97k (8.76 KN)
Torsion:
From sacristy in longitudinal direction = 0.673 k (2.99 KN)
From sacristy in transverse direction x 30% = 0.3(0.3kip) = 0.09kip (400N)
Total torsion (for orthogonal effects = 0.673 + 0.09 = 0.763k (3.39 KN)
Total shear = 6.86 k + 1.97k + 0.76k = 9.6 k (42.7 KN)

1 kip = 4.448 KN
1 kipft =- 1.356 KNm
1 foot = 0.305 m



Wall Line C7–C8 (Wall Line G7–G8 similar)

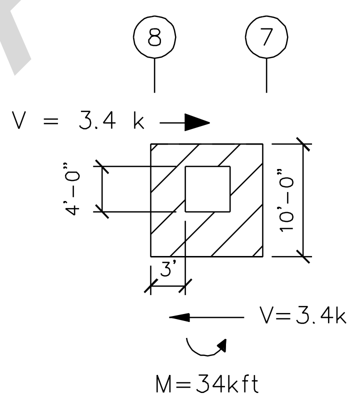
Distribute shear to wall piers in lower portion of wall (1/2 each)
V = ½(9.6) = 4.8 kips (21.3 KN)
Shear strength of pier = (3.33')(12"/')(8")(174) = 55.6 k (247 KN)> 4.8 k (21.3 KN), OK
M = (8')(4.8k)/2 = 19.2 kft (26.0 KNm)
M/Vl = 19.2 / (4.8)(3.33') = 1.2 < 3.0 (no boundary zones required in outer piers)

Check overall action of wall;

M_{base} = (6.86)(25.67) + (2.73)(10') = 204 kft (277 KNm)
M/Vl = 204 / (9.6)(10) = 2.12 <3.0 (no boundary zones required in lower portion of wall)

Shear wall lines A7-A8 & I7-I8

These walls resist shear forces from the sacristies.
V = 3.33k (14.8 KN)
Torsion:
From sacristy in transverse direction x 30% = 0.3(0.30) = 0.09k
Total shear = 3.33 k + 0.09k = 3.4 k (15.1 KN)



Wall Line A7–A8 (Wall Line I7–I8 similar)

Distribute shear to wall piers in lower portion of wall (1/2 each);
V = 3.4k/2 = 1.7 k (7.56 KN)
Shear strength of pier = (3')(12"/')(8")(174) = 49 k (218 KN) > 1.7 k (7.56 KN), OK
M = (4')(1.7)/2 = 3.4 kft (4.61 KNm)
M/Vl = 3.4 / (1.7)(3') = 0.67 < 3.0 (no boundary zones required in outer piers)

Check overall action of lower portion of wall;

M_{base} = (3.4)(10') = 34 kft (46.1 KNm)
M/Vl = 34 / (3.4)(10) = 1.0 <3.0 (no boundary zones required in lower portion of wall)

Typical Reinforcing; (See Figure 7-6 for typical reinforcing in concrete shear walls)

Use #5 bars @ 12" on center in horizontal and vertical direction for all concrete shear walls.

Development length per ACI 318 Sec. 12.2 $\frac{l_d}{d_b} = \frac{f_y \alpha\beta\lambda}{25\sqrt{f_c^{'}}}$

α = 1.3, β = 1.0 $\alpha\beta$ need not be greater than 0.8, use 0.8
λ = 1.0

$$\frac{l_d}{d_b} = \frac{60000(0.8)(1.0)}{25\sqrt{3000}} = 35$$

For # 5 bar l_d = 35*(5/8) = 22", use 24" (61 cm)

Use splice length = 1.3 l_d = 1.3(24") = 31.2", use 32" (81.3cm) for all splices.

Use 2 #5 vertical bars at ends of all wall segments and at openings.
Use 2 #5 horizontal bars typical above and below openings (extend bars 24" (61 cm) past edge of openings to develop bars) and continuously at top of walls for chord reinforcement.

(d) *Horizontal bracing:*
The bracing resist seismic forces only; load factor = 1.0
Design bracing for highest shear force (at shear wall line 2)
Shear to wall line 2A-2D passing through horizontal bracing = direct + torsion
V = 21.91 kips + 30%(3.2 kips) = 22.9 kips (102 KNm)
Axial (braces are at 45 degress); Axial force = 22.9 kips (1.41) = 32.3 kips (144 KN)

Try 4" Extra Strong Round Tubing (f_y = 36 ksi, r = 1.48 in., A = 4.41 in.^{2})

The perpendicular length of the brace is approximately 8', length of brace = 8'(1.41)=11.3'

$$\frac{KL}{\pi r}\sqrt{\frac{F_y}{E}} = \frac{(1.0)(11.3')(12''/')}{\pi(1.48)}\sqrt{\frac{36}{29000}} = 1.03$$

$$\phi_c F_{cr} = (0.85)(36ksi)(0.658)^{\lambda^2} = 19.63ksi \text{ (135 N/mm}^2\text{)}$$

$\phi_c P_n$ = (19.63 ksi)(4.41 in.2) = 86.6 k (385 KN)

Check AISC Seismic Provisions; Design bracing as ordinary concerntrically braced frame (OCBF)

Slenderness: Bracing members shall have Kl/r $\leq 720/\sqrt{F_y} = 720/6 = 120$ (Section 14.2.a)

Kl/r = (1.0)(11.3)(12)/(1.48) = 92 < 120, OK

Required Compressive Strength of brace ≤ 0.8 times $\phi_c P_n$ (Section 14.2.b)
$0.8f_c P_n$ = (0.8)(86.6 k) = 69.3 k (308 KN) > 32.3 k (144 KN), OK

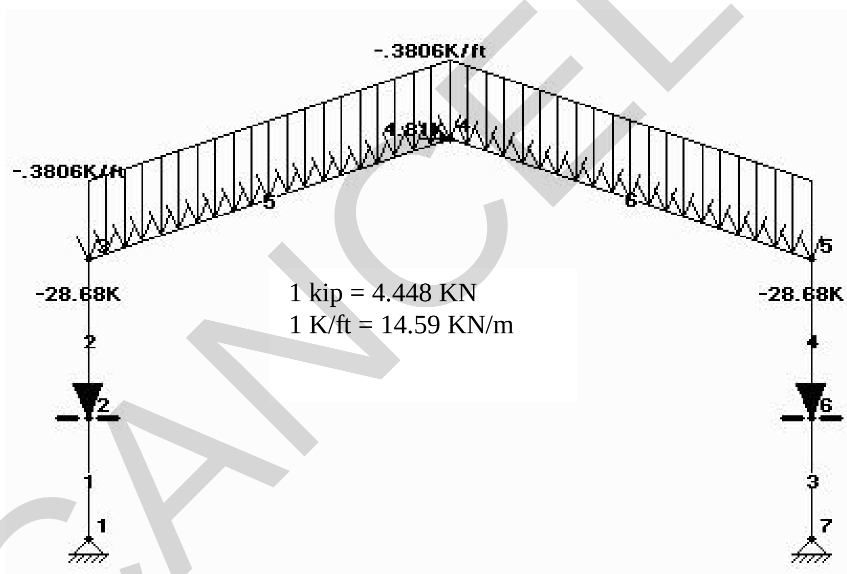
Width-to-Thickness Ratio: (Section 14.2.d)
$D/t \leq 1300/F^y = 1300/36 = 36.11$ (AISC Seismic Provisions Table I-9-1)
$D/t = 4.5 / 0.337 = 13.4 < 36.11$, OK

(e) *Moment frames*:
The moment frames resist seismic forces from the upper roof diaphragm and are braced by the lower sloped diaphragms (by horizontal bracing). The frames also support gravity loads from the upper roof and from the beam reactions along grid lines B & H.

Gravity:	Dead = 218 plf (3.18 KN/m)
	Live = 238 plf (3.47 KN/m)
	Beam reactions = 23.9 k (106.3 KN)
Seismic:	$V_{upper\ roof}$ =4.81 k (21.4 KN)

The members of the moment frames have already been checked for the gravity load combination and now are checked for the seismic load combination: 1.2D + 0.5L + 1.0E where $E = \rho Q_E + 0$. The vertical seismic effects are captured by the term $0.2\ S_{DS}D = 0.2(1.0) = 0.2D$. This term is added to the 1.2D load term to a total of 1.4D for this load combination.

Loads for elastic analysis: It is assumed that the lateral load to the upper portion of the frames is applied at the top middle node.



Beam Design:
The elastic analysis results show that the maximum moment (84.0 kft) and axial force in the beam (15.1 k) are lower than those for the load combination of 1.2D + 1.6L. By inspection it is seen that the beams are adequate.

Column Design:
Top portion: $\phi_c P_n$ = 802 k (3567 KN) $\phi_b M_n$ = 518 kipft (702 KNm)
Bottom portion: $\phi_c P_n$ = 927 k (4123 KN) $\phi_b M_n$ = 518 kipft (702 KNm)
For the upper portion of the column (above the horizontal bracing level) the maximum axial force = 13.2k (58.7 KN) and the moment is 84.0 kipft (114 KNm). For the lower portion of the columns the maximum axial force is 41.84 kips (186 KN) and the moment is 70.6 kipft (95.7 KNm). The columns are seen to be adequate by inspection.

Check AISC Seismic Provision Section 8.2 for columns;
Section 8.2 requires columns with high axial loads to be checked for increased demand.
$P_u / \phi P_n = 41.84 / 927 = 0.05 < 0.4$, OK

Check the colmn-beam moment ratio (AISC Seismic Provisions Section 9.6):

The relationship for the beam-to-column connection ratio, $\dfrac{\sum M^*_{pc}}{\sum M^*_{pb}} > 1.0$, is now checked:

$$\sum M^*_{pc} = Z_c(F_{yc} - P_{uc} / A_g) = 192(36 - 41.8/32.0) = 6661\text{kipin (752 KNm)}$$

$$\sum M^*_{pb} = 1.1(R_y M_p + M_v)$$

M_p = 303 kft (411 KNm)

The shears at the end of the beam when plastic hinges occur at the beam ends, V_p; (Note: The plastic hinge location must be verified to make sure they will occur at the toe of the haunch connections. See Problem H-4 for detailed calculation of plastic hinge locations. For this problem assume the platic hinges are located at the toe of the haunch.)
$V_p = 2M_p / L = 2(303) / 58$ ft = 10.4 kip (46.5 KN) assuming the hinges occur at the ends of the haunches.
The distance from the plastic hinge location to the connection centerline is approximatly 18" (45.7 cm).
$M_v = V_p(18") = 10.4k(18")$ = 187 kipin. (16.3 KNm)
1.1(1.5(303)(12) + 187) = 6205kipin (701 KNm)
$M_{pc} / M_{pb} = 6661 / 6205 = 1.07$, OK

Check the limits of compactness for the column set forth in AISC Seismic Provisions Sec. 9.4b.
Limiting value from AISC Seismic Provisions Table I-9-1 = $52/\sqrt{F_y} = 52/6 = 8.67$
For W14x109 b/t = 8.5 < 8.67, OK.

Limiting value $\dfrac{520}{\sqrt{F_y}}\left[1 - 1.54\dfrac{P_u}{\phi_b P_y}\right] = \dfrac{520}{6}\left[1 - 1.54\dfrac{41.8}{(32\text{in}^2)(36\text{ksi})(0.9)}\right] = 81.3$
For W14x109 h/t_w = 21.7 < 81.3, OK.

Check of the panel-zone of beam-to column connection:
Shear strength: (per AISC Seismic Provisions Sec. 9.3)
The required shear strength Ru of the panel-zone shall be determined by applying AISC Seismic Provsions Load Combinations 4-1 and 4-2 to the connected beam.

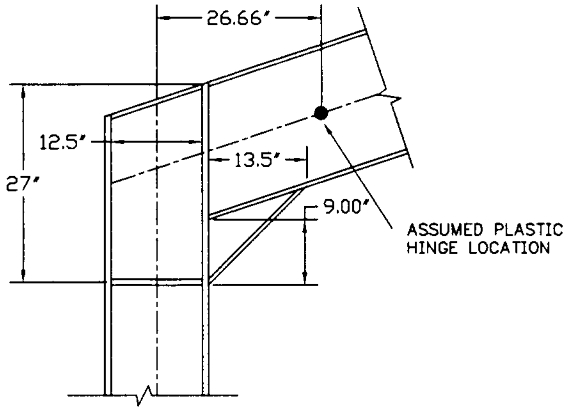
Governing Load Combination: $1.2D + 0.5L + \Omega_0 Q_E$ where $\Omega_0 = 3.0$ (AISC Seismic Provisions Eq. 4-1).
Note: The overstrength factor used for this check (3) is taken from the AISC Seismic Provisions. It is higher than the value required by FEMA 302 (2.5). The higher value is used to be conservative.

The maximum moment on the end of beam at the column interface due to seismic actions is 105 kipft (142 KNm). The shear to the joint is equal to this moment divided by the depth of the beam (including haunch) Ru = 105 kft / 27" = 47 kips (209 KN). The shear strength is determined from AISC Seimsic Provisions Equation 9-1:

$$\phi_v R_v = 0.75(0.6)F_y d_c t_p\left[1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_p}\right] = 0.75(0.6)(36)(14.32)(0.525)\left[1 + \frac{3(14.6)(0.86)^2}{18(14.32)(0.525)}\right] = 151\text{k (672 KN)}$$

151 kips (672 KN) > 47 kips (249 KN), Shear strength is OK.

Panel-Zone thickness:



1 inch = 25.4 mm

$t \geq (d_z + w_z) / 90$; (27" + 12.5) / 90 = 0.44 < 0.525, OK (AISC Seismic Eq. 9-2)

(f) Check beams along grid lines B & H that support concrete shear walls.

These beams act as collectors and chords for the low sloped roof areas in addition to supporting the concrete walls. They must be checked for the 1.2D + 0.5L + 1.0E load combination where $E = \Omega_0 Q_E + 0.2S_{DS}D$. The $0.2S_{DS}$ term adds 0.2 to the dead load factor. The collectors must be designed for $\Omega_0 = 2.5$. The beams act as collectors for longitudinal forces and chords for transverse forces.

Chord force = 32.3 kips (144 KN) Collector force = 4.46 kips (19.8 KN)
100% longit. + 30% transv. = 2.5(4.46) + (0.30)(32.3) = 20.84 kips (92.7 KN)
30% longit. + 100% transv. = (0.30)(2.5)(4.46) + 32.3 = 36 kips (160 KN) (governs)

w_u = 1.2D + 0.2D = 1.4D = 1.4(1325 plf) = 1855 plf (27.1 KN/m)
$M_u = w_uL^2/8 = (1855)(18')^2/8 = 75$ kft (102 KNm)

The compression flanges of the beams are completely supported by the concrete shear wall. It is assumed that the wall is connected to the beam by automatically welded studs @ 4' spacing (1.22m). Assume that the unsupported length for determing axial capacity is equal to KL = 4' (1.22m).

Determine axial capacity; W 14 x 38 (r_y = 1.55", A = 11.2 in^2, ϕM_p = 166 kft)
Kl/r = (48")/1.55" = 30.97
From AISC Table 3-36 ϕF_{cr} = 29.1 ksi (201 N/mm^2)
ϕP_n = (29.1 ksi)(11.2 in^2) = 326 kips (1450 KN)
$P_u / \phi P_n = 36 / 326 = 0.11 < 0.2$, use interaction equation H1-1b

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}}\right) \leq 1 = \frac{36}{2(326)} + \left(\frac{102}{166}\right) = 0.67 < 1, \text{ OK}$$

By inspection assume that beam is OK in shear.

B-12 *Check allowable drift and PΔ effect*

The drift of the steel moment frames is checked due to their low rigidity as compared to that of the concrete shear walls. It is assumed that the concrete shear wall drift is within the limits specified in Table 6-1. The interstory drift is found by adding the displacement of the moment frame at the top center node and adding that to the deflection of the shear walls at the low sloped roof areas.
Deflection of wall 2A-2D: Shear = 31.6 k (141 KN) = deflection = V/R R = 31.6 / 18920 k / in. ≈ 0.
Deflection of moment frames at center of gravity of story (from elastic analysis) = 0.11 inches (2.8 mm)
The story deflection is determined by multiplying this deflection by C_d.

Story deflection = C_d(0.11") = 5(0.11) = 0.55 inches (14mm)
Allowable Deflection = 0.02(33.3') = 8" (203 mm) > 0.55" (14mm), OK (From Table 6-1)

D-1 *Check for Performance Objective 2A (Safe Egress).*
This performance objective uses the same ground motion as the Life Safety Performance Objective (1A). The structure is a one-story building analyzed by the ELF procedure, and therefore, the seismic effects, Q_E, in step B10 may be scaled up in a linear manner.

D-2 *Determine the pseudo lateral load, $V=C_1C_2C_3S_aW$*
C_1: Modification factor to relate expected maximum inelastic displacement to displacements calculated for linear elastic response. (per FEMA 273 Section 3.3.1.3)
$T_0 = S_{D1}/S_{DS} = 0.75$
T = Fundamental period of building = 0.25 seconds
C_1 = 1.5 – (0.25-0.10)/(0.75-0.10)*(0.5) = 1.38

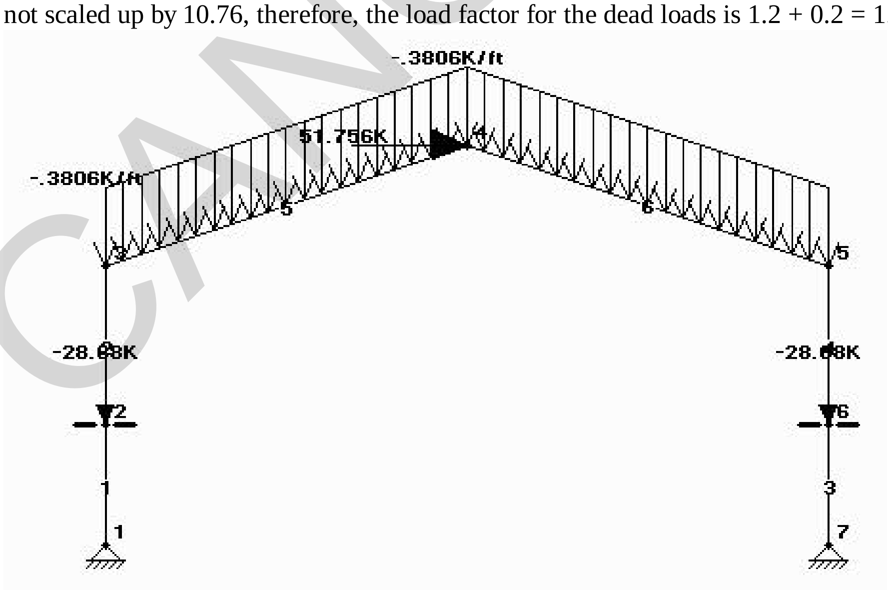
C_2: Hysterisis modification factor, from Table 5-2;
C_2 = 1.3 (Framing Type 1, Life Safety and T = 0.1 sec)

C_3: Modification factor to account for P-delta effects.
Assume that the building exhibits positive post-yield stiffness.
C_3 = 1.0 for positive post-yield stiffness.

V = (1.38)(1.3)(1.0)(1.0g)(635 k) = 1139 k (5066 KN)

D-3 *Determine seismic effects.*
The seismic effects in Steps B-4 through B-9 are scaled up by the factor R x C1 x C2 x C3.
Scale factor = (6)(1.38)(1.3)(1.0) = 10.76

D-4 *Determine the combined load effects.*
The shear force to the wall segments are scaled up by the factor 10.76 and the resulting shear demand is checked against the shear strength multiplied by the appropriate m-factor from Table 7-4.
The moment frames will be analyzed for the load combination 1.2D + 0.5L + E, where the E term represents the seismic actions determined from step B.10 scaled up by 10.76 (Note: the term $0.2S_{DS}$ x D is not scaled up by 10.76, therefore, the load factor for the dead loads is 1.2 + 0.2 = 1.4)



Horizontal Bracing:

The axial force in the horizontal pipe bracing is scaled up by 10.76 to 10.76(32.3) = 344 k (1530 KN). The bracing enhanced performance objectives will be checked as if the bracing were a concentrically braced frame.

Collectors / Chords along grid lines B & H
These beams act as collectors and chords for the low sloped roof areas in addition to supporting the concrete walls. Scale up forces by 10.96
Chord force = 32.3 kips (10.96) = 354 k (1575 KN)   Collector force = 4.46 k (10.96) = 48.7 k (217 KN)

w_u = 1.2D + 0.2D = 1.4D = 1.4(1325 plf) = 1855 plf (27.1 KN/m)
$M_u = w_u L^2/8$ = (1855)(18')^2/8 = 75 kft (102 KNm)

D-5 *Identify force-controlled and deformation controlled structural components.*
The concrete shear walls have very low axial load demands. Footnote 1 of Table 7-3 requires that the axial load be less than 0.15 $A_g f'_c$ to be governed by shear. The maximum shear stress in all of the wall is less than $6\sqrt{f_c'}$ and therefore, the walls are checked as deformation controlled structural components.
The steel moment frames are checked as deformation controlled components.
The horizontal bracing is checked as a deformation controlled component.

The connection of the horizontal bracing to the shear walls along lines 2 & 7 and the collectors along lines B & H are checked as force controlled actions.

D-6 *Determine Q_{UD} and Q_{CE} for deformation controlled components*

<u>Shear Wall Segments</u>

The highest demand / capacity ratio to any wall pier element from step B.11 is 16.2k / 109k = 0.15 (for the lower portions of wall line 7). Only this element is checked;

Q_{UD} = (10.76)(16.2 k) = 174.3 kips (775 KN)
Q_{CE} = 109 kips (485 KN) (determined previously)

<u>Moment Frames</u>

The expected strength of the steel members is based on $F_{ye} = R_y F_y$ = 1.5(36) = 54 ksi (372 N/mm^2) and Z for the section.

Beams;
Moment Strength = ZF_{ye} = 101(54) = 455 kft (617 KNm)
Shear Strength = $0.6F_{ye}(d \times t_w)$ = 0.6(54)(17.99" x 0.355") = 207 k (921 KN)
Axial Strength = 340 kips (this is for 36 ksi; scale up by 54/36 to obtain F_{YE} strength)
Axial Strength = (340 k)(54/36) = 510 k (2268 KN)

Moment Demand = 180 kft (244 KNm)
Shear Demand = 11.8 k (52.5 KN)   (Elastic analysis not shown)
Axial Demand = 41 k (182 KN)

Columns;

Moment Strength = ZF_{ye} = 192(54) = 864 kft (1179 KNm)
Shear Strength = $0.6F_{ye}(d \times t_w)$ = 0.6(54)(14.32" x 0.525") = 244 k (1085 KN)
Axial Strength for 36 ksi column: Top segment = 802k; Bottom segment = 927k
Scale up to F_{YE} strength: Top segment = (802k)(54/36) = 1203 k (5351 KN);
Bottom segment = (927k)(54/36) = 1391k (6187 KN)

Moment Demand = 286 kft (388 KNm) (at horizontal bracing level)
Shear Demand = 35.14 k (156 KN) (Elastic analysis not shown)
Axial Demand @ top portion = 24.18 k (108 KN)
Axial Demand @ bottom portion = 52.86 k (235 KN)

Panel Zones;

The maximum moment on the end of beam at the column interface due to seismic actions is 179.8 kipft (244 KNm). The shear to the joint is equal to this moment divided by the depth of the beam (including haunch) = 179.8(12)/27 = 80 kips (356 KN). The shear strength is determined from AISC Seismic Provisions Equation 9-1:

$$\phi_v R_v = 0.75(0.6)F_{ye}d_c t_p\left[1+\frac{3b_{cf}t_{cf}^2}{d_b d_c t_p}\right] = 0.75(0.6)(54)(14.32)(0.525)\left[1+\frac{3(14.6)(0.86)^2}{18(14.32)(0.525)}\right] = 227\text{k (1010 KN)}$$

D-7 *Determine DCR's for deformation-controlled components and compare with allowable m-values for Safe Egress*

Shear Walls

The highest demand / capacity ratio to any wall pier element from step B.11 is 16.2k / 109k = 0.15 (for the lower portions of wall line 7). Only this element is checked;

Q_{UD} / Q_{CE} = 174.3 / 109 = 1.60

From Table 7-3 for concrete shear wall segments controlled by shear: m = 2.0.
1.60 < 2.0, OK

Steel Moment Frames

The m-factors for fully restrained moment resisting frames is taken from Table 7-12. The m-factor for all actions is 4.

Beams;

Moment DCR = 180 / 455 = 0.4 <4, OK
Shear DCR = 11.8 / 207 = 0.06 <4, OK
Axial DCR = 41 / 510 = 0.08 <4, OK

Columns;

Moment DCR = 1022 / 864 = 1.18 <4, OK
Shear DCR = 75.6 / 244 = 0.31 <4, OK
Top portion Axial DCR = 24.2 / 1203 = 0.02 <4, OK
Bottom portion Axial DCR = 52.9 / 1391 = 0.04 < 4, OK

Panel Zones;

The m factor for panel zones from Table 7-12 is 4.8 for Safe Egress
Demand / Capacity = 80 / 227 = 0.35 < 4.8 OK

Horizontal Bracing

The m-factor for the horizontal bracing from Table 7-10 is 2.9 for braces in compression and $\frac{d}{t} \le \frac{1500}{F_y}$.

Axial Demand = 344 kips (1530 KN)
Axial Capacity = 86.6 kips (this is based on 36 ksi; scale it up by 54/36 for enhanced performance objectives)
Axial Capacity = (86.6 k)(54/36) = 130 kips (578 KN)
Axial DCR = 344 / 130 = 2.65 < 2.9, OK

D-8 *Determine Q_{UF} and Q_{CL} for force-controlled components and compare Q_{UF} with Q_{CL}*

Note: Q_{CL} contains the appropriate strength reduction factor per paragraph 6-3a(3)b

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1C_2C_3J} \qquad \text{(Eq. 6-4a)}$$

where $J = 1.0 + S_{DS} = 1.0 + 0.2 = 1.2$ and C_1, C_2 and C_3 have been determined previously.

Therefore, the scale factor for QE is;

$$SF = \frac{1}{C_1C_2C_3J} = \frac{1}{(1.38)(1.3)(1.0)(1.2)} = 0.46$$

Chord / Collector elements along grid lines B & H

Worst case is for the chord force:
Q_E: Chord force =354 kips (1574KN)
Q_G = 1.4D = 1855 plf (27.06 KN/m) , M_u = 75 kft (102 KNm) (determined previously)

Q_{UF} = SF(Q_E) = (0.46)(354) = 163 kips (725 KN) axial and M_u = 75 kft (102 KNm)

ϕP_n = 326 kips (1450 KN) ϕM_p = 166 kft (225 KNm) (determined previously)

$P_u / \phi P_n = 163 / 326 = 0.5 > 0.2$ Use AISC LRFD equation H1-1a

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_u}{\phi M_p}\right) = \frac{163}{326} + \frac{8}{9}\left(\frac{75}{225}\right) = 0.80 < 1.0\text{, OK}$$

Shear wall to horizontal bracing connections.

These connections are designed in Section D-9 based on the capacity of the brace. The design force is greater than the force that results from Q_{UF}. Therefore, assume connections are OK for force-controlled action.

D-9 *Revise member sizes, as necessary, and repeat analysis.*
No member sizes need to be revised. Design of steel connections is done now;

Moment Connections:

Determine if continuity plates are required:

Determine demand;
$P_{bf} = A_f F_{ye} = A_f R_y F_y = (0.57)(7.495)(1.5)(36) = 231$ kips

Determine capacity;

$$\phi R_n = \phi\left[(2.5k + N)F_{yw}t_w + A_{st}F_{yst}\right]$$

$$A_{st} = \frac{P_{bf} - (2.5k + N)F_{yw}t_w}{F_{yst}} = \frac{231 - (2.5(1.56") + (0.57"))(36ksi)(0.525")}{36ksi} = 4.07\ in^2\ (26.3\ cm^2)$$

Design stiffeners in accordance with AISC LRFD Sec. K.9;

$$b_{st} + \frac{t_{cw}}{2} \geq \frac{b_b}{3} \rightarrow b_{st} \geq \frac{b_b}{3} - \frac{t_{cw}}{2}$$

where b_{st} = width of single stiffener, b_b = width of beam flange, t_{cw} = thickness of column web

$$b_{st} = \frac{7.495}{2} - \frac{0.525}{2} = 3.48"\ (88mm),\ \text{use } 4.5"\ (114mm)$$

$t_{st} \geq \frac{t_{bf}}{2}$ where t_{st} = thickness of a single stiffener, t_{bf}= thickness of beam flange

$$t_{st} = \frac{.57}{2} = 0.285"\ \ (7.24mm),\ \text{use } 1/5"\ (12.7mm)$$

Use 4.5" x ½" (114 mm x 88mm) stiffeners on both sides of column. (Area = 2 x 4.5 x ½ = 4.5 in^2 > 4.07 in^2)

Design welds for the stiffeners;
Stiffener to column web;
F = P_{bf} – (2.5k + N)$F_{yw}t_w$ = 231 – (2.5(1.56) + 0.57)36(0.525) = 147 k
Assume minimum weld size = 3/16" (4.8mm)
Strength of weld = 0.75(0.6)(70)(0.707)(3/16) = 4.18 kip / in (0.73 KN/mm)
Length of weld required = 147 / 4.18 = 35.2" (89.4 cm).
The length of weld available = 11.25" x 2 sides x 2 stiffeners = 45" (114 cm) > 35.2"
Check shear strength of the base material;

$\phi R_n = \phi R_{nw} = \phi(0.6F_u)t = (0.75)(0.6)(58)(0.525)2 = 27.4$ kips / in (4.8KN/mm) > 4.18 kips / in

Stiffener to column flange; Use full penetration groove welds

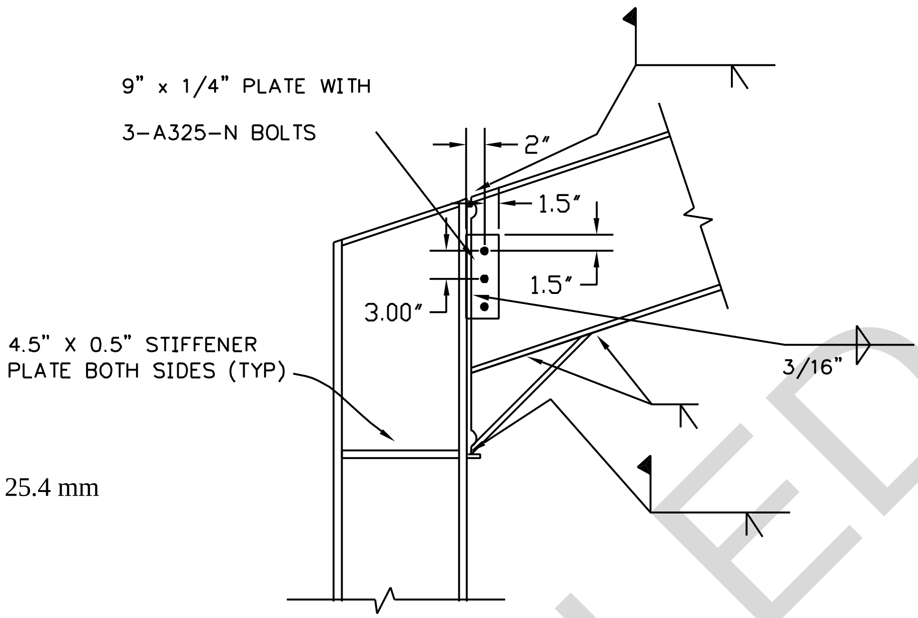
Design the single-plate web connection;

The governing load combination = 1.2D + 1.6L_r + 0.5L
w_u = 1.2(218 plf) + 1.6(238 plf) = 642 plf (9.37 KN/m)
$V_u = w_uL/2$ = (642 plf)(60')/2 = 19.3 k (85.8 KN)

From AISC LRFD Table 9-10; assuming the column provides a rigid support, for ¾" diameter A325-N bolts and single-plate material with F_y = 36 ksi and F_u = 58 ksi, select three rows of bolts, ¼" single-plate thickness, and 3/16" fillet weld size: ϕRn = 27.8 kips (124KN)

Check supported beam web: From Table 9-2, for three rows of bolts, beam material with Fy = 36 ksi and Fu = 58 ksi, and L_{ev} = 1-1/2" and L_{eh} = 1-1/2" (Assumed to be 1-1/4" for calculation purposes to account for possible underrun in beam length),
ϕRn = 235 kips / inch (0.355) = 83.4 kips (371 KN) > 27.8 kips (124KN)

Use ¼" thick plate (6.35mm) f_y = 36 ksi (248 N/mm^2) with three ¾" diameter A325-N bolts. Weld plate to column with 3/16" (4.76 mm) welds along entire plate length on both sides.



1 inch = 25.4 mm

<u>Design of horizontal bracing connections:</u>

The design of these connections follows Figure 7-22.
Assume plate thickness = ½" (12.7 mm); thickness of brace = 0.337" (8.56mm); E70XX welds

Design of brace-to-gusset weld;

Design weld capacity to be greater than axial capacity of brace = $R_yF_yA_g$ = 1.5(36ksi)(4.41 in.^{2}) = 238 k (1059 KN)
Minimum weld size = 3/16" (4.76mm) (AISC LRFD Table J2.4)
Maximum weld size = brace thickness – 1/16" = 0.337"-1/16" = 0.28" (7.11mm) (AISC LRFD Sec. J2.b)
Use ¼" welds (0.25") (6.35mm) 3/16<1/4<0.28, OK

Strength of weld; (per AISC LRFD Sec. J.4 and Table J2.5)

Weld material: $\phi R_{nw} = \phi(t_e)$ (0.6 F_{EXX}) = 0.75(0.6)(70ksi)(0.707)(0.25) = 5.6 kips / inch (0.98 KN/mm) (governs)
Base material: $\phi R_{nw} = \phi(0.6F_u)t$ = (0.75)(0.337")(0.6)(58) = 8.8 kips / in (1.54 KN/mm)

Length of weld required = 238 k / (5.6 k/") = 43" (109cm) (4 welds at connection, use 11" (27.9 cm) welds, 4 x 11 = 44 > 43)

Use 11" (27.9 cm) long ¼" (6.35 mm) fillet welds on all four sides

Check gusset plate capacity

Tension rupture of plate: The tension rupture strength of the plate is based on Whitmore's area. This area is calculated as the product of the plate thickness times the length W, shown in the sketch as a 30 degree angle offset from the connection line. The tension rupture strength of the plate is designed to exceed the tensile strength of the brace, 238 kips.

W = 2(11"*tan 30) + 4.5" = 17.2" (43.7 cm)
$\phi_tP_n = \phi tF_uA_e$ = 0.75(58)(17.2")(0.5") = 374 k (1664 KN) > 238k (1058 KN) AISC LRFD Eq. D1-2

Block shear rupture strength of plate:
$\phi R_n = \phi(0.6F_yA_{gv} + F_uA_{nt})$ AISC LRFD Eq. J4-3a
ϕR_n = 0.75(0.50)[(0.6)(36)(2 x 11"/cos 30) + (58)(17.2)] = 528 k (2349 KN) > 238 k (1058 KN)

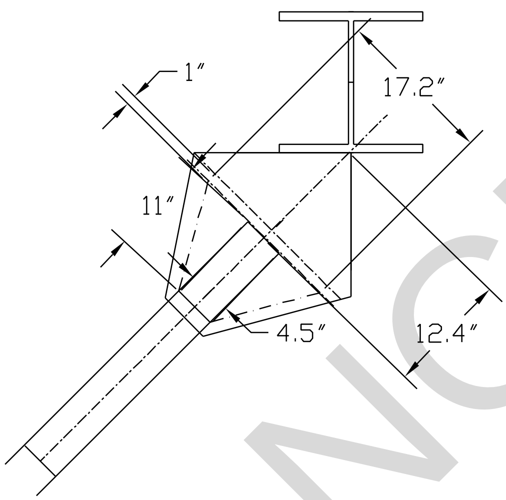
$\phi R_n = \phi[0.6F_uA_nv + F_yA_{gt}]$ AISC LRFD Eq. J4-3b
$\phi R_n = 0.75(0.5)\ [(0.6)(58)(2 \times 11) + (36)(17.2)] = 519$ k (2309 KN) > 238 k (1058 KN)

Buckling of plate:

Buckling capacity of the brace = $A_gF_{cr} = \phi_cP_n / \phi_c = 87$ kips / 0.85 = 102 kips (454 KN) (buckling strength determined previously)

$$0.90\frac{4000t^3\sqrt{f_y}}{l_c} = 0.90\frac{4000(1/2)^3\sqrt{36}}{12.4} = 218 \text{ kips (970KN)} > 102\text{kips (457 KN)}\text{ , OK}$$

Out-of-plane strength of plate: The bracing member can buckle both in and out of plane due to the round section used. For out-of-plane buckling the gusset plate must be able to accommodate the rotation by bending. The brace shall terminate on the gusset a minimum of two times the gusset thickness from the theoretical line of bending which is unrestrained by the column or beam joints. This ensures that the mode of deformation in the plate will be through plastic hinging rather than torsional fracture.



1 inch = 25.4 mm

Design of gusset-to-column flange and beam web weld;

This connection requires a weld length greater than the column flange. The plate must be welded to both the column flange and the beam web to develop the brace force.
Design weld capacity to be greater than horizontal component of brace capacity = (0.707)(238) = 168 kips (747 KN)
Column flange thickness = 0.86 in. (21.8mm)
Beam web thickness = 0.31"
Minimum weld size = ¼" (6.35mm) (AISC LRFD Table J2.4)
Maximum weld size = beam web thickness ≈ 0.25" (6.35 mm)
Use 1/4" (6.35 mm) welds

Strength of weld; (per AISC LRFD Sec. J.4 and Table J2.5)

Weld material: $\phi R_{nw} = \phi(t_e)(0.6\ F_{EXX})$ = 0.75(0.6)(70ksi)(0.707)(0.25) = 5.57 kips / inch (0.98 KN/mm) (governs)

Base material: $\phi R_{nw} = \phi(0.6F_u)$ = (0.75)(0.31")(0.6)(58) = 8.1 kips / in (1.42 KN/mm)

Length of weld required = 168 k / (5.57 k/") = 30.16" (76.6 cm) (2 welds at connection weld 16" (40.64 cm) long = 32" > 30.16")

Use 1/4" (6.35mm) fillet welds on top and bottom of plate.

Design member to develop force into shear wall

Vertical component = (0.707)(238k) = 168 kips (747 KN)

Use 6 x 6 x ½ angle to develop forces; Assume 7/8" anchor bolts to concrete shear wall

Check gross section yielding; $P_u = \phi_t F_y A_g$ = (0.9)(36)(5.75) = 186 kips (827 KN) > 168 kips (747 KN)
Check net section fracture; $P_u = \phi_t F_u A_e$ = (0.75)(58)(0.85)(5.75 – (7/8+1/16)(0.5)) = 195 kips (867 KN) > 168 kips

Design weld of plate to angle
Minimum weld size = ¼" (6.35mm) (AISC LRFD Table J2.4)
Maximum weld size = plate thickness – 1/16" = 0.5-1/16 = 0.44" (11.18mm)
Use 7/16" (11.11 mm) welds (0.438")

Strength of weld; (per AISC LRFD Sec. J.4 and Table J2.5)

Weld material: $\phi R_{nw} = \phi(t_e)(0.6\ F_{EXX})$ = 0.75(0.6)(70ksi)(0.707)(0.438) = 9.75 kips / inch (1.71 KN/mm) (governs)

Base material: $\phi R_{nw} = \phi(0.6F_u)t$ = (0.75)(0.5")(0.6)(58) = 13.05 kips / in (2.29 KN/mm)

Length of weld required = 168 k / (9.75 k/") = 17.23" (43.8 cm) (2 welds at connection, weld for 16" on both sides of plate. Total length = 2 x 16 = 32" (81.3 cm) > 17.23" (43.8 cm), OK

Use 16" long (81.3 cm) 7/16" (11.11mm) fillet welds along top and bottom of plate.

Design bolts for angle to wall connection.

Design per FEMA 302

Tensile strength of bolts:

-Assume 7/8" diameter bolts @ 12"o/c with a 6" embedment length

-Assume that the edge distance for anchors > than 6" = embedment length

Strength based on steel: $P_s = 0.9A_bF_un$ (FEMA 302 Eq. 9.2.4.1-1)

$P_s = (0.9)(0.6\text{in.}^2)(58\text{ ksi})(1) = 31.3$ kips (139 KN)

Strength based on concrete: $\phi P_c = \phi\lambda\sqrt{f_c^{'}}(2.8A_s)n$ (FEMA 302 Eq. 9.2.4.1-2)

$\phi P_c = (0.65)(1.0)\sqrt{3000}(2.8)(\pi)(6)^2(1) = 11.3$ kips (50.3 KN) (governs)

Shear strength of bolts:

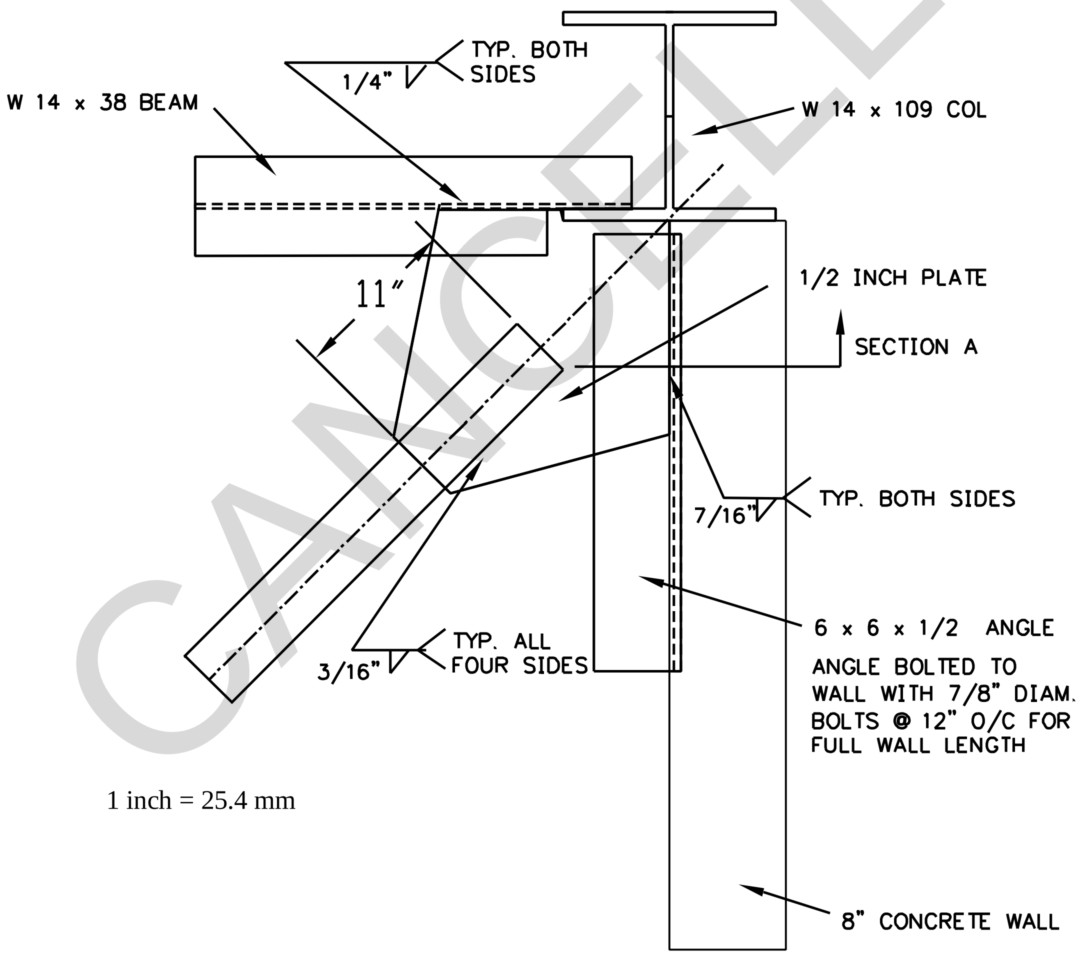
Strength based on steel: $V_s = 0.75A_bF_un$ (FEMA 302 Eq. 9.2.4.2-1)

$V_s = (0.75)(0.60)(58)(1) = 26.1$ kips (116 KN)

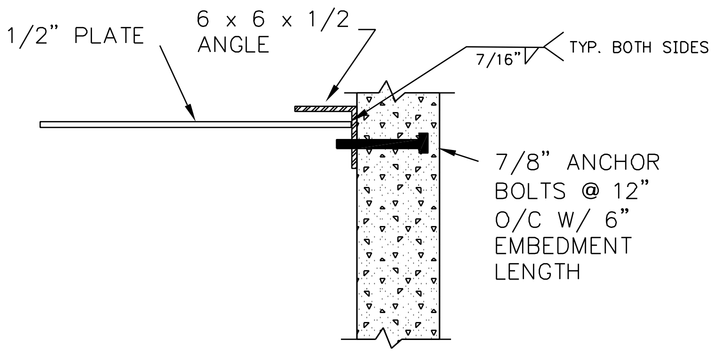
Strength based on concrete: $\phi V_c = \phi 800A_b\lambda\sqrt{f_c^{'}}n$ (FEMA 302 Eq. 9.2.4.2-2)

$\phi V_c = (0.65)(800)(0.6)(1.0)\sqrt{3000}(1) = 17.1$ kips (76.1 KN) (governs)

Shear force demand = vertical component of brace capacity = 168 kips (753 KN)
Number of bolts required = 168 / 17.1 = 9.8 bolts, use 10 bolts @ 12" (0.31m) on center.



1 inch = 25.4 mm



SECTION A

Design of gusset-to-column weld;

Design weld capacity to be greater than brace capacity = 238 k (1059 KN)
Column flange thickness = 0.86 in. (21.8mm)
Minimum weld size = ¼" (6.35mm) (AISC LRFD Table J2.4)
Maximum weld size = plate thickness – 1/16" = 0.5-1/16 = 0.44" (11.18mm)
Use 7/16" (11.11 mm) welds (0.438")

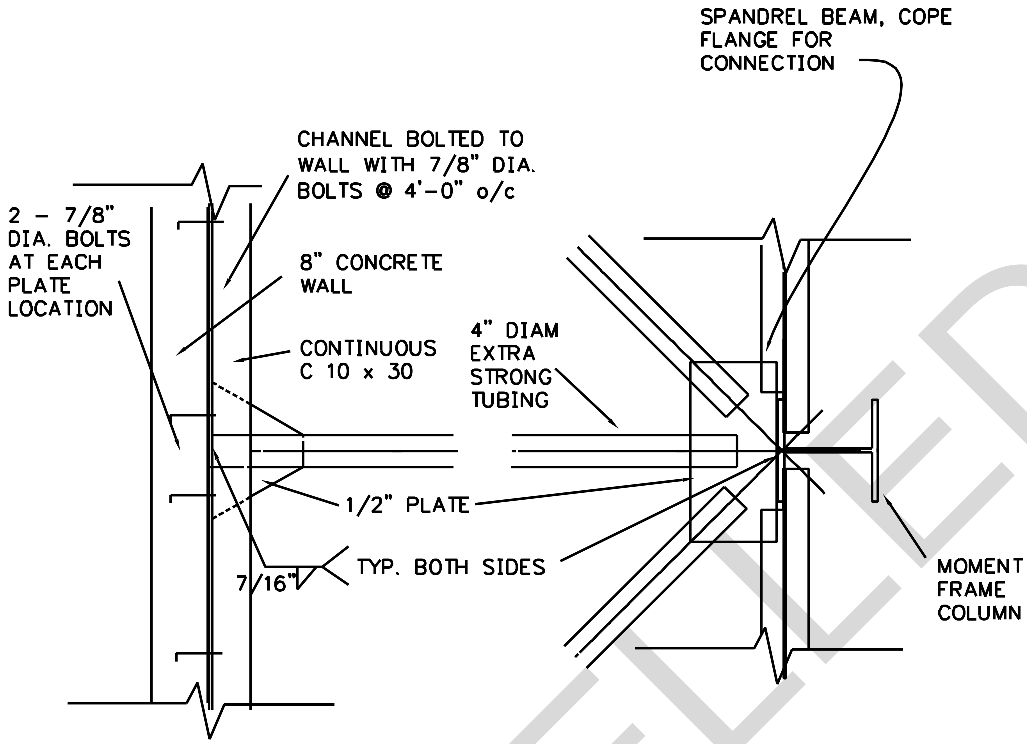
Strength of weld; (per AISC LRFD Sec. J.4 and Table J2.5)

Weld material: $\phi R_{nw} = \phi(t_e)\ (0.6\ F_{EXX}) = 0.75(0.6)(70ksi)(0.707)(0.438) = 9.75$ kips / inch (1.71 KN/mm) (governs)

Base material: $\phi R_{nw} = \phi(0.6F_u)t = (0.75)(0.5")(0.6)(58) = 13.05$ kips / in (2.29 KN/mm)

Length of weld required = 238 k / (9.75 k/") = 24.4" (62 cm) (2 welds at connection, weld for full length of column flange = 14.61" (37.11 cm) along top and bottom of plate. Total length = 2 x 14.61 = 29.22" (74.2 cm) > 24" (43.8 cm), OK

Use 7/16" (11.11mm) fillet welds along entire length of column flange on top and bottom.



PLAN OF HORIZONTAL BRACING AT LOW ROOF

1 inch = 25.4 mm

H-4 FIRE STATION

a. Introduction

This design example illustrates the seismic design of a two-story fire station. The step by step procedure as shown in Tables 4-5 and 4-6 was followed almost verbatim for the design of the buildings basic structural elements. This rigid adherence to the outlined procedures was done in order to provide a clear demonstration of the use of this manual.

(1) Purpose. The objective of this problem is to demonstrate the procedure to be used for designing a building with an enhanced performance objective.

(2) Scope. The scope of this example problem includes; the design of all major structural steel members such as beams, columns, and braces, as well as the design of several example structural steel connections. The design of the foundations, nonstructural elements and their connections, and detail design of some structural elements such as concrete floor slabs were not considered part of the scope of this problem and are therefore not included. Additionally, this problem considers only seismic and gravity loads.

b. Building Description

(1) Function. This building functions as a fire station, and provides living quarters to station personnel as well as garage space for equipment such as fire engines.

(2) Seismic Use Group. As a fire station, this building will be required for post-earthquake recovery and as such performs a mission essential function. Therefore, this building is categorized with a seismic use group of IIIE, Essential Facilities. With the Seismic Use Group known, the structural system performance objectives are obtained from Table 4-4. Structures in seismic use group IIIE are to be designed for performance level 3; Immediate Occupancy. Ground motion B (3/4 MCE) is to be used for performance objective 3B. The minimum analysis procedure to be used is the liner elastic with R factors and linear elastic with m factors. The structure is designed first for performance objective 1A following the steps laid out in Table 4-5. After completion of the preliminary design, the enhanced performance objectives outlined in Table 4-6 for performance objective 3B are checked and the building design is updated accordingly to meet those objectives.

(3) Configuration. As shown in Figure 1, the building is rectangular in plan measuring 70 feet (21.35m) by 30 feet (9.15m). It contains a one-story low roof garage area that is connected to an adjacent two-story high roof office area and dormitory. Story height of the low roof area is 15 feet (4.58m), and of the high roof area is 11 feet (3.36m).

(4) Structural Systems. The building consists primarily of steel frame construction composed of wide flange shapes, hollow structural sections, and metal decking. However, the second floor incorporates a reinforced concrete slab. Structural systems are shown in Figure 1.

The gravity load resisting system consists of untopped metal decking that spans to open web steel joists, which span to wide flange steel beams and columns. The second floor consists of a reinforced concrete slab that spans between wide flange beams, which are supported by the same columns that support the roof decking.

The lateral load resisting system consists of both flexible and rigid diaphragms that span between steel moment frames in the transverse direction, and steel braced frames in the longitudinal direction. Roof diaphragms consisting of flexible untopped metal decking that place tributary load on the lateral load resisting system. The second floor diaphragm, however, consists of a rigid concrete slab for which torsion must be considered.

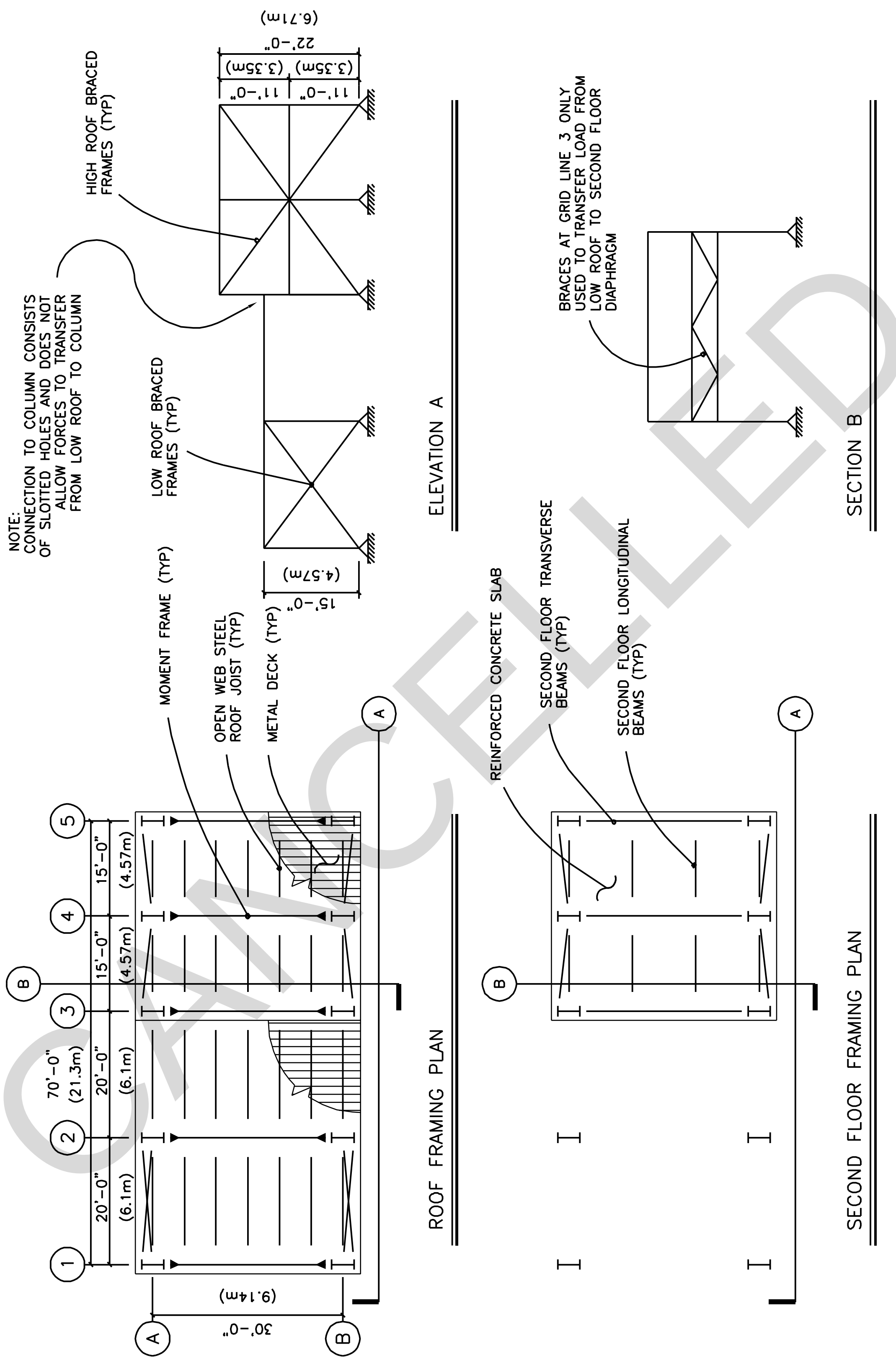


Figure. 1 Building Plan Layout

The high roof and low roof structures share a common transverse moment frame at their interface. However, in the longitudinal direction, the low roof structure is essentially isolated from the high roof structure at this location. This is accomplished by the use of a simple gravity connection consisting of elongated slotted holes that attach the low roof support beam to the moment frame column.

The building is considered regular both in plan and vertically.

(5) Choice of materials. Columns shall be designed using ASTM A572 Grade 50 and braces using ASTM A24 Grade 46. All other steel components will use ASTM A36 Grade 36. All walls not shown on the floor framing plans are intended to be nonstructural and shall be constructed so as to not impair the response of the steel moment frames and steel braced frames.

c. Preliminary building design (following steps in Table 4-5 for Life Safety). The preliminary design of the building follows the steps outlined in Table 4-5 for Performance Objective 1A. The design is then updated to meet the enhanced performance objectives laid out in Table 4-6 for Performance Objective 2A.

A-1 *Determine appropriate Seismic Use Group.* Per Table 4-3 of the manual, the building must be safe to occupy immediately after an earthquake and is required for post earthquake recovery. Therefore, it is an essential structure with a Seismic Use Group of IIIE.

A-2 *Determine Site Seismicity.* The site seismicity for this example from the MCE maps is:
$S_S = 0.80g$, and $S_1 = 0.40g$.

A-3 *Determine Site Characteristics.* For the purpose of this problem, a very dense soil and soft rock condition was assumed corresponding with a site classification of 'Class C' per Table 3-1 of the TI manual.

A-4 *Determine Site Coefficients, F_a and F_v.* From Tables 3-2a and 3-2b for the given site seismicity and soil characteristics, the site response coefficients were interpolated as follows:

$$F_a = 1.08 \qquad \text{(Table 3-2a)}$$
$$F_v = 1.40 \qquad \text{(Table 3-2b)}$$

A-5 *Determine adjusted MCE Spectral Response Accelerations:*

$$S_{MS} = F_a(S_S) = 1.08(0.80) = 0.86 \qquad \text{(EQ. 3-1)}$$
$$S_{M1} = F_v(S_1) = 1.40(0.40) = 0.56 \qquad \text{(EQ. 3-2)}$$

A-6 *Determine Design Spectral Response Accelerations:*

$$S_{DS} = (2/3)S_{MS} = (2/3)0.86 = 0.57 \qquad \text{(EQ. 3-3)}$$
$$S_{D1} = (2/3)S_{M1} = (2/3)0.56 = 0.37 \qquad \text{(EQ. 3-4)}$$

The approximate period of the structure (based on T = 0.1N, where N = number of stories) is:

$$T_{approx} = 0.1N = 0.1(2) = .2 < .5 \qquad \text{(EQ. 5.3.3.1-2 FEMA 302)}$$

Since $T_{approx} = .2 < .5$, and because the structure is less then 5 stories is height; equations 3-5 and 3-6 of the TI 809-04 manual must be checked in the short period range:

$$S_{DS} \leq 1.5F_a \qquad \text{(EQ. 3-5)}$$
$$S_{D1} \leq 0.6F_v \qquad \text{(EQ. 3-6)}$$

Therefore,

$$S_{DS} = 0.57 < 1.5(1.08) = 1.62 = 1.5F_a$$
$$S_{D1} = 0.37 < 0.6(1.40) = 0.84 = 0.6F_v$$

Use S_{DS} = 0.57, & S_{D1} = 0.37

A-7 *Determine Seismic Design Category*. With S_{DS} = 0.57, S_{D1} = 0.37, and a Seismic Use Category of IIIE, enter tables 4-2a and 4-2b to obtain a Seismic Design Category of 'D'.

A-8 *Select Structural System.*

Gravity: Steel frame with metal deck roof and reinforced concrete second floor slab. Roof deck spans between interior open web steel joists and perimeter edge beams. Open web steel joists span to transverse beams. Perimeter edge beams and interior transverse beams span between columns. Second floor slab spans between interior beams and perimeter edge beams. Second floor beams span to girders that span between columns.

Lateral: As permitted by Table 7-1:

Transverse direction: Special steel concentrically braced frames.
Longitudinal direction: Special steel moment frames.

A-9 *Select R, Ω_0, & C_d factors.*

From Table 7-1:

Transverse direction - Special Steel Moment Frames	**R = 8, Ω_0 = 3, C_d = 5.5**
Longitudinal direction – Special Steel Concentrically Braced Frames	**R = 6, Ω_0 = 2, C_d = 5.0**

A-10 *Determine preliminary member sizes for gravity load effects.*

(1) Roof Decking.

Live Load (per Table 4-1 of ANSI/ASCE 7-95):		20	psf
Dead Load (estimated):	Roofing;	5	psf
	Rigid Insulation;	3	psf
	Decking;	2	psf
	Misc. (Mechanical & Electrical);	3	psf
	Total =	13	psf (0.62KN/m^2)

Total Load: Dead + Live = 20 + 13 = 33 psf (1.58KN/m^2)

Per steel deck manufacturers catalog with 3 or more 5-ft (1.53m) spans;

Choose 1-1/2" (38.1mm)-22 gage HSB36

(2) Roof Joists.

Loading: 33 psf (1.58KN/m^2) + self weight
Assume self weight $\approx$ 5 plf (0.073KN/m), and a spacing of 4-ft (1.22m) on center
Loading = 33psf(4')+5plf
= 137plf (2.00KN/m)

Per steel joist manufacturers catalog with span = 20' (6.10m), a total load = 137plf (2.00KN/m), and a live load = 80plf (1.17KN/m);

Choose 10K1 Joists @ 4-ft (1.22m) o.c.

(3) Longitudinal Roof Edge Beams. Use F_y = 36ksi. One design will be produced for the worst case and used throughout. The beam must have a minimum depth of 12" (304.8mm) to ensure installation of 3 bolts per connection in anticipation of chord/collector forces. Worst cases situation occurs at the low roof that has the longest span and largest tributary area. Also, all beams are simply supported.

Determine Design Loads;
Tributary area "A_T" = 2.5'(20') = 50-ft^2 (4.65m^2)
Dead Load = 13psf(2.5')+(estimated self wt. of 25plf)
= 57.5 plf (0.84KN/m)
Note: Due to the small tributary area, live load reduction (as per ANSI/ASCE 7-95) is not permitted.
Live Load = 2.5'(20psf)
= 50plf (0.73KN/m)
Strength requirements;
Note: Beam has continuous lateral support from attached roof deck
W_u = 1.2(57.5plf)+1.6(50plf) = 149plf (2.17KN/m)
$M_u = w_u L^2/8 = 149\text{plf}(20')^2/8 = 7.45^{\text{ft-kips}}$ (10.10KN-m)

Deflection requirements;

$$\Delta_{allow} = \frac{L}{240} = \frac{20'(12''/1')}{240} = 1.0'' \ (25.4\text{mm})$$ (Per ANSI/ASCE 7-95 section B.1.1)

$$I_{req'd} \geq \frac{5w_L L^4}{384E\Delta_{allow}} = \frac{5(0.05\text{klf})(20')^4(12''/1')^3}{384(29{,}000\text{ksi})1.0''} = 6.2-\text{in}^4 \ (2.5\text{X}10^6 \text{ mm}^4)$$

By inspection, a W12X14 (W304.8mm X 0.20KN/m) will work due to the light loading condition

Choose W12X14 (W304.8mm X 0.20KN/m)

(4) Transverse Beams. Transverse beams are part of the moment frames. As a first approximation, it will conservatively be assumed that the beams are simply supported. Over designing for gravity loads is not expected to produce overly conservative results considering that drift due to seismic loads will probably govern the final design. One design will be produced for the worst case gravity load and used throughout. The worst case situation occurs at the low roof interior beam, which has the largest tributary area.

Determine Design Loads;
Tributary area "A_T" = 20'(30') = 600-ft^2 (55.7m^2)
Dead Load = 20psf (0.96KN/m^2) (assumed)
Live Load Reduction per ANSI/ASCE 7-95;
A_T = 600-ft^2 $\Rightarrow R_1 = 0.6$
Roof Slope < 4:12 $\Rightarrow R_2 = 1.0$
$\therefore$ Reduced Live Load = 20psf(0.6)(1.0) = 12psf (0.57KN/m^2)
$w_U = 1.2w_D+1.6w_L$ = [1.2(20psf)+1.6(12psf)]20' = 864plf (12.6KN/m)

Strength requirements; use F_y = 36ksi (248.2MPa)

$$Z_{req'd} \geq \frac{M_u}{f_b F_y} \quad \text{with} \quad M_u = \frac{w_u L^2}{8}$$

$$\geq \frac{864\text{plf}(30')^2(12''/1')}{8(0.9)36{,}000\text{psi}} = 36-\text{in}^3 \ (589.9\text{X}10^3 \text{ mm}^3)$$

$$f_v V_n \geq \frac{w_u L}{2} = \frac{864\text{plf}(30')}{2} = 13^k \ (57.8\text{KN})$$

Deflection requirements;

$$I_{req'd} \geq \frac{5w_L L^4}{384E\Delta_{allow}} \quad \text{with} \quad \Delta_{allow} = \frac{L}{240} = \frac{30'(12''/1')}{240} = 1.5'' \ (38.1\text{mm})$$

$$\geq \frac{5(0.24\text{klf})(30')^4(12''/1')^3}{384(29{,}000\text{ksi})(1.5'')} = 101-\text{in}^4 \ (42.0\text{X}10^6 \text{ mm}^4)$$

Compact section criteria (per AISC seismic provisions);
Try W14X26 (W355.6mm X 0.38KN/m);

$$\frac{b_f}{2t_f} = 6.0 < 8.7 = \frac{52}{\sqrt{36ksi}} = \frac{52}{\sqrt{F_y}}$$ **O.K.**

Choose W14X26 (W355.6mm X 0.38KN/m), Z = 40.2-in^3 (658.8X10^3 mm^3), f_vV_n = 69-kip (306.9KN), I = 245-in^4 (102X10^6 mm^4)

(5) Columns. Columns are sized based on the strong column /weak beam requirements of the AISC Seismic Provisions for Structural Steel Buildings (dated April 15, 1997) herein referred to as the AISC seismic provisions.

Using ASTM A572 Grade 50;

$$\frac{\sum M^*_{pc}}{\sum M^*_{pb}} \geq 1.0$$ (EQ. 9-3 AISC Seismic Provisions)

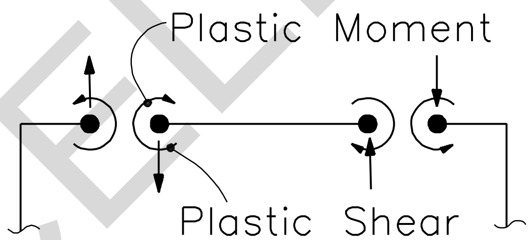
$$\sum M^*_{pb} = \sum (1.1 R_y M_p + M_v)$$

where; R_y = 1.5

M_p = Plastic Moment = ZF_y = 40.2-in^3(36ksi)

$= 1{,}447^{in-kips}$ (163.5KN-m)

M_v = Additional moment due to offset of the plastic hinge from the column centerline (this is equal to the plastic shear 'V_p' times the offset distance). M_p is determined using the following free body diagram;



Note; The offset from the column centerline is determined by choosing the column depth approximately equal to the beam depth, the haunch length to be 3/4 of the beam depth, and as recommended in FEMA 267, by noting that the plastic hinge will occur another 1/3 of the beam depth beyond the toe of the haunch.

Therefore; Offset distance 'x' $= \frac{d_c}{2} + \frac{3d_b}{4} + \frac{d_b}{3} \approx 1.6d_b$

= 1.6(13.91")

= 22.25" or 1.85' (0.56m)

$$V_p = \frac{2M_p}{(L-2x)} = \frac{2(1{,}447^{in-kips})}{(30'-2(1.85'))(12''/1')} = 9.17^k \text{ (40.8KN)}$$

$$\therefore M_v = V_p(1.85') = 9.17^k(22.25'') = 204^{in-kips} \text{ (23.1KN-m)}$$

Therefore; $$\sum M^*_{pb} = 1.1(1.5)1{,}447^{in-kips} + 204^{in-kips} = 2{,}591^{in-kips} \text{ (292.8KN-m)}$$

and; $$\sum M^*_{pc} = \sum Z_c (F_{yc} - P_{uc} / A_g)$$

where; $P_{uc} \approx$ 1.2(20psf)20'(15')(1^k/1000^{lb}) = 7.2^k (32.0KN)

$A_g \approx$ 10-in^2 (6.45X10^3 mm^2) (assumed)

$\therefore$ $P_{uc}/A_g \approx$ 0.7ksi (4.83MPa)

Therefore; $$\sum M^*_{pc} = Z_{req'd}(50ksi - 0.7ksi) = 49.3Z_{req'd}$$

and; $$\frac{\sum M_{pc}^*}{\sum M_{pb}^*} \Rightarrow Z_{req'd} \geq \frac{2{,}591^{"k}}{49.3\text{ksi}} = 53-\text{in}^3 \;\; (868.5\text{X}10^3 \text{ mm}^3)$$

Note: By inspection, shear does not govern.

Choose; W14X34 (W355.6 X 0.50KN/m), Z=54.6-in^3 ((894.7X10^3 mm^3), I = 340-in^4 (141.5X10^6 mm^4)

(6) Second Floor Slab. Per Table 9.5(a) of ACI 318-95 the minimum thickness of a one way slab when deflections are not computed is;

l/24	(one end continuous)	(governs)
l/28	(both ends continuous)	

Therefore, for a 10-ft (3.05m) span; 10'(12"/1')/24 = 5" (127.0mm)

Choose a 5" (127.0mm) thick reinforced concrete slab

(7) Second Floor Longitudinal Beams. Use F_y = 36ksi (248.2MPa)

Strength requirements;

Note: Beam has continuous lateral support from attached roof deck.

$w_u = 1.2w_{DL}+1.6w_{LL}$

where; w_{DL} = (5/12)'(150pcf)10'+10psf(10')+(1psf+3psf+1psf)10'

Partition wt.
Beam Spacing
Floor finish wt.
Suspended ceiling wt.
Miscellaneous wt.

w_{DL} = 775plf (11.30KN/m)

w_{LL} = 40psf(10') = 400plf (5.83KN/m) (Per ANSI/ASCE 7-95 Table 4-1 Residential)

w_u = 1.2(0.775klf)+1.6(0.400klf) = 1.57klf (22.90KN/m)

$$M_u = \frac{1.57\text{klf}(15')^2}{8} = 44.2^{\text{ft-kips}} = 530^{\text{in-kips}} \;\; (59.0\text{KN/m})$$

$$Z_{req'd} \geq \frac{M_u}{f_b F_y} = \frac{530^{\text{in-kips}}(1000^{\text{lb/k}})}{0.9(36{,}000\text{psi})} = 16.4-\text{in}^3 \;\; (268.7\text{X}10^3 \text{ mm}^3)$$

$$f_v V_n \geq \frac{w_u L}{2} = \frac{1{,}570\text{plf}(15')}{2} = 12^k \;\; (53.4\text{KN})$$

Deflection requirements;

$$I_{req'd} \geq \frac{5w_L L^4}{384E\Delta_{allow}} = \frac{5(0.40\text{klf})(15')^4(12"/1')^3}{384(29{,}000\text{ksi})0.75"} = 20.95-\text{in}^4 \;\; (8.72\text{X}10^6 \text{ mm}^4)$$

Choose W10X17 (W254mm X 0.25KN/m), Z = 18.7-in^3 (306.4X10^3 mm^3), I = 81.9-in^4 (34.1X10^6 mm^4), $F_v V_n$ =47.2-kip (209.9KN)

(8) Transverse Second Floor Beams. Use F_y = 36ksi (248.2MPa). Produce one design for the worst case and use throughout. Worst case situation occurs at the interior bay.

Strength requirements;

Note: Beam has continuous lateral support from attached slab.

$w_u = 1.2w_{DL}+1.6w_{LL}$

where; w_{DL} = [(5/12)'(150pcf)+17plf(1/10')+12psf]15'+30plf

Self wt.
Tributary width
Concrete floor slab wt.
Partitions & Misc. wts.
Long beam wts.

w_{DL} = 1,173plf (17.11KN/m)
w_{LL} = 40psf(15') = 600plf (8.75KN/m)

Note: Floor live load reduction, per ANSI/ASCE 7-95, is not permitted because this is a one way slab and the floor live load = 40psf < 100psf (1.92KN/m^2 < 4.79KN/m^2).

w_u = 1.2(1.218klf)+1.6(0.60klf) = 2.42klf (35.29KN/m)

$$M_u = \frac{2.42\text{klf}(30')^2}{8} = 272^{\text{ft-kips}} = 3264^{\text{in-kips}} \ (368.8\text{KN-m})$$

$$Z_{req'd} \geq \frac{M_u}{f_b F_y} = \frac{3264^{\text{in-kips}}(1000^{\text{lb/k}})}{0.9(36{,}000\text{psi})} = 101-\text{in}^3 \ (1.66\text{X}10^6 \text{ mm}^3)$$

$$f_v V_n \geq \frac{w_U L}{2} = \frac{2{,}420\text{plf}(30')}{2} = 36.3^k \ (161.5\text{KN})$$

Deflection requirements;

$$I_{req'd} \geq \frac{5 w_L L^4}{384 E \Delta_{allow}} = \frac{5(0.60\text{klf})(30')^4(12''/1')^3}{384(29{,}000\text{ksi})1.5''} = 251.4-\text{in}^4 \ (104.6\text{X}10^6 \text{ mm}^4)$$

Choose W21X50 (W533.4mm X 0.73KN/m), Z = 110-in^3 (1.80X10^6 mm^3), I = 984-in^4 (410X10^6 mm^4), $f_v V_n$ = 154-kip

d. Equivalent Lateral Force Procedure

B-1 *Calculate Fundamental Period, T:*

$$T_a = C_t h_n^{3/4} \qquad \text{(EQ. 5.3.3.1-1 FEMA 302)}$$

Transverse direction; C_t = 0.035 Moment frame resisting 100% of the seismic force.

Longitudinal direction; C_t = 0.020 Braced frame system.

h_n = 22-ft (6.71m) Height to highest level.

Therefore; $T_a = 0.035(22')^{3/4} = 0.36\sec$ transverse

$T_a = 0.020(22')^{3/4} = 0.20\sec$ longitudinal

B-2 *Determine Dead Load,'W':*

Building weights were calculated on spread sheet and are shown in Figures 2 and 3. Total seismic weight is;

W = 129-kips (574KN)

B-3 *Calculate Base Shear, V:*

$$V = C_s W \qquad \text{(EQ. 5.3.2 FEMA 302)}$$

where; $$C_s = \frac{S_{DS}}{R} \qquad \text{(EQ. 3-7)}$$

$$C_s < \frac{S_{D1}}{TR} \qquad \text{(EQ. 3-8)}$$

$$C_s > 0.044 S_{DS} \qquad \text{(EQ. 3-9)}$$

Longitudinal direction;

R = 6 (Table 7-1)

C_s = 0.57/6 = 0.095 < 0.37/{0.2(0.6)} = 0.31

> 0.044(0.57) = 0.025

Transverse direction;

R = 8 (Table 7-1)

$C_s = 0.57/8 = 0.071 \qquad < 0.37/\{0.36(8)\} = 0.13$
$\qquad\qquad\qquad\qquad\quad > 0.044(0.57) = 0.025$

Therefore; $\mathbf{C_{s,long} W = 0.095(129^k) = 12.2^k}$ **(54.3KN),** $\mathbf{C_{s,trans} W = 0.071(129^k) = 9.1^k}$ **(40.5KN)**

B-4 *Calculate Vertical Distribution of Forces.*

Note: The building will behave as two separate structures. In the transverse direction, the single story structure will distribute some tributary loads to the common lateral load resisting moment frame at the interface with the two-story structure, but will otherwise behave independently in this direction. In the longitudinal direction, the single story and two story structures are completely independent due to the elongated slotted holes in their adjoining connection. Therefore the two structures will be analyzed independently of each other.

Divide the base shear between the single story and the two story structures

The base shear will be divided between the structures based on the ratio of their masses;

Single story weight = 26.1^k (116.1KN)
Two story weight = $19.8^k + 82.2^k = 102^k$ (453.7KN)

Therefore;
Base shear for the single story building;

$$V_{\text{single story, trans}} = \frac{26.1^k}{128^k}(9.1^k) = 1.9^k \quad (8.45\text{KN})$$

$$V_{\text{single story, long}} = \frac{26.1^k}{128^k}(12.2^k) = 2.5^k \quad (11.12\text{KN})$$

Base shear for the two-story building;

$$V_{\text{two story,trans}} = \frac{102^k}{128^k}(9.1^k) = 7.2^k \quad (32.03\text{KN})$$

$$V_{\text{two story, long}} = \frac{102^k}{128^k}(12.2^k) = 9.7^k \quad (43.15\text{KN})$$

Calculate vertical distribution on two story structure;

$$F_x = C_{vx} V \qquad \text{(EQ. 5.3.4-1 FEMA 302)}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \text{(EQ. 5.3.4-2 FEMA 302)}$$

where; k = 1 in both directions for the building period is less than 0.5 sec.

The calculations are tabularized below*;

Story	w_i (kips)	h_i (ft)	$w_i x h_i$ (ft-kips)	C_{vx}	$C_{vx} x V_t$ (kips)	$C_{vx} x V_L$ (kips)
Roof	19.8	22	435.6	0.33	2.34	3.15
2^{nd}	82.2	11	904.2	0.67	4.86	6.55
SUM =	102		1339.8	1.00	7.20	9.70

Note: For metric equivalent; 1-kip = 4.448KN, 1-ft = 0.30m, 1-ft-kip = 1.36KN-m

Therefore;

Transverse direction;
$\mathbf{F_{roof} = 2.34^k}$ **(10.41KN)**
$\mathbf{F_{2nd} = 4.86^k}$ **(21.62KN)**

Longitudinal direction;
$\mathbf{F_{roof} = 3.15^k}$ **(14.01KN)**
$\mathbf{F_{2nd} = 6.55^k}$ **(29.13KN)**

ASSEMBLY WEIGHTS (PSF):

DIAPHRAGMS:

<u>Level - High Roof</u>

Built-Up roofing;		5.00	
2" rigid insulation @ 1.5psf/in., 1.5x2 =		3.00	
1-1/2"-22 GA Decking (galvanized);		1.90	
10K1 Open Web Steel Joists @ 4' o.c.; 5plf(1/4') =		1.25	
W12x14 Edge Beam @ equivalent 15' o.c.; 14plf(1/15') =		0.93	
W14X26 Beam @ avg spacing of 10' o.c.; 26plf(1/10') =		2.60	
Suspended ceiling;		1.00	
Mech., Elec., & Miscellaneous		3.00	
	Total =	18.7 psf	(0.9KN/m^2)

<u>Level - Low Roof</u>

Built-Up roofing;		5.00	
2" rigid insulation @ 1.5psf/in., 1.5x2 =		3.00	
1-1/2"-22 GA Decking (galvanized);		1.90	
10K1 Open Web Steel Joists @ 4' o.c.; 5plf(1/4') =		1.30	
W12x14 Edge Beam @ equivalent 15' o.c.; 14plf(1/15') =		0.93	
W14X26 Beam @ avg spacing of 13' o.c.; 26plf(1/13') =		1.00	
Suspended ceiling;		1.00	
Mech., Elec., & Miscellaneous		3.00	
	Total =	17.1 psf	(0.82KN/m^2)

<u>Level - Second Floor</u>

Floor Finish;		1.00	
5" Thick Concrete Floor Slab; (5/12)'(150pcf) =		62.50	
W10x17 Beam @ avg spacing of 7.5' o.c.; 17plf(1/7.5') =		2.27	
W21x50 Girders @ avg spacing of 10' o.c.; 50plf(1/10') =		5.00	
Partitions (10 psf per FEMA 310 section 3.5.2.1);		10.00	
Suspended ceiling;		1.00	
Mech., Elec., & Miscellaneous		3.00	
	Total =	84.8 psf	(4.06KN/m^2)

METAL SIDE WALLS:

Metal Siding;		1.00	
Girts;		1.00	
2" insulation @ 1.0psf/in., 1.0x2 =		2.00	
W14X34 Col @ avg spacing of 17.5'; 34plf(1/17.5') =		1.94	
	Total =	5.9 psf	(0.3KN/m^2)

Figure 2. Calculation of component weights.

BUILDING WEIGHTS (KIPS):

Item Desc.	Width or Height (ft)	Length (ft)	Number	Trib Area $(ft)^2$	Unit Wt. (psf)	Weight Trans. (kips)	Weight Long. (kips)	Weight Total (kips)
Level - High Roof								
Diaph.	30	30	1	900	18.7	16.8	16.8	16.8
Longitudinal Walls (above and below the diaphragm)								
Below	5.5	30	2	330	5.9	2.0		2.0
Transverse Walls (above and below the diaphragm)								
Below	5.5	30	1	165	5.9		1.0	1.0
Below	3.5	30	1	105	5.9		0.6	0.6
Total High Roof Tributary Weight =						18.8	18.4	20.4
Level - Low Roof								
Diaph.	30	40	1	1200	17.1	20.6	20.6	20.6
Longitudinal Walls (above and below the diaphragm)								
Below	7.5	40.0	2	600	5.9	3.6		3.6
Transverse Walls (above and below the diaphragm)								
Above	3.5	30.0	1	105	5.9		0.6	0.6
Below	7.5	30.0	1	225	5.9		1.3	1.3
Total Low Roof Tribrtary Weight =						24.1	22.5	26.1
Level - Second Floor								
Diaph.	30	30	1	900	84.8	76.3	76.3	76.3
Longitudinal Walls (above and below the diaphragm)								
Above	5.5	30.0	2	330	5.9	2.0		2.0
Below	5.5	30.0	2	330	5.9	2.0		2.0
Transverse Walls (above and below the diaphragm)								
Above	5.5	30.0	1	165	5.9		1.0	1.0
Below	5.5	30.0	1	165	5.9		1.0	1.0
Total Second Floor Tributary Weight =						80.2	78.3	82.2
Total Building Weight =								129

Note: For metric conversions; 1-ft = 0.30m, 1-ft^2 = 0.093m^2, 1psf = 47.88N/m^2, 1-kip = 4.448KN

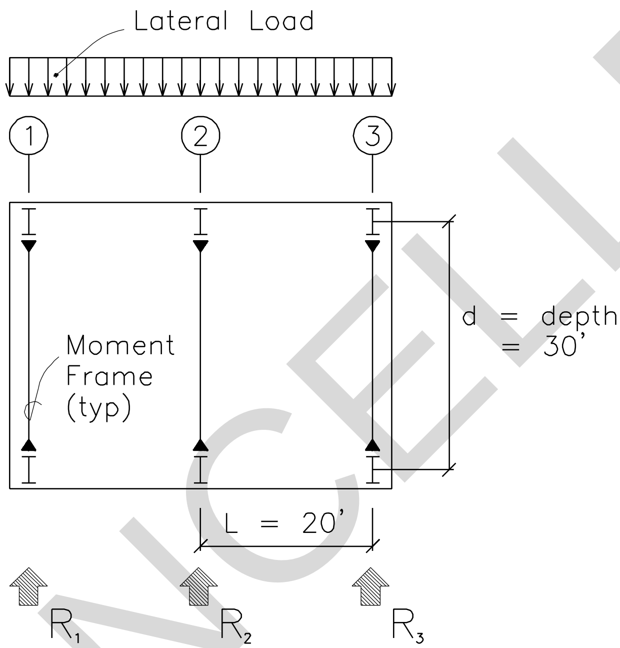
Figure 3. Calculation of building weights.

B-5 *Perform Static Analysis.*

General;

The roof diaphragms are composed of metal decking without fill and are therefore flexible diaphragms. Therefore, loads at the roof will be distributed to the lateral load resisting elements by the tributary area method. The second floor diaphragm is composed of a reinforced concrete slab and is considered a rigid diaphragm. Loads at the second floor will be distributed to the lateral load resisting elements in accordance with their rigidities.

Perform static analysis on single story structure;
Transverse direction:



1-ft = 0.30m

Lateral load: $w_u = F_{trans}/2L = 1.9^k/40' = 47.5\text{plf}$ (0.69KN/m)

Diaphragm moment: $M_u = \dfrac{w_u L^2}{8} = \dfrac{47.5\text{plf}(20')^2}{8} = 2{,}375^{ft-lbs}$ (3.22KN-m)

Chord forces: $T = C = \dfrac{M_u}{d} = \dfrac{2{,}375^{ft-lbs}}{30'} = 79.2 - \text{lb}$ (352.3N)

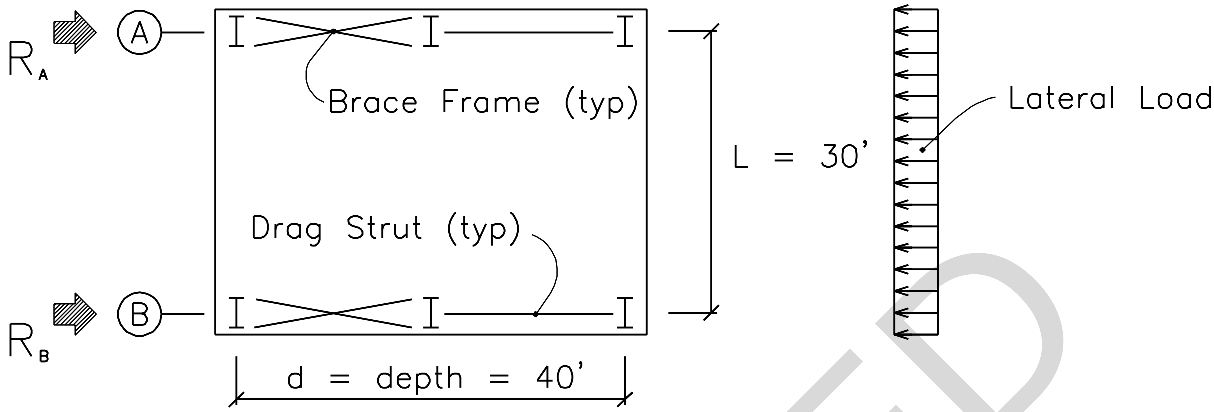
Reactions on moment frames:

End frames; $R_1 = R_3 = w_u$x(tributary length) = 47.5plf(10') = 475-lb (2.11KN)

Interior frame; R_2 = 47.5plf(20') = 950-lb (4.23KN)

Unit shear: v = R/d = 475-lb/30' = 16plf (0.23KN/m)

Longitudinal direction:



1-foot = 0.30m

Lateral load: $w_u = F_{trans}/L = 2.5^k/30' = 83.3\text{plf}$ (1.22KN/m)

Diaphragm moment: $M_u = \dfrac{w_u L^2}{8} = \dfrac{83.3\text{plf}(30')^2}{8} = 9{,}371^{\text{ft-lbs}}$ (12.71KN-m)

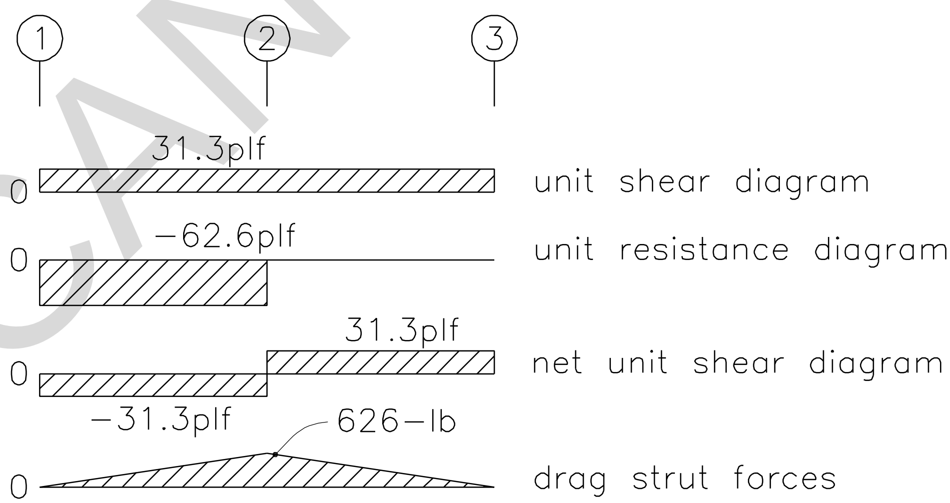
Chord forces: $T = C = \dfrac{M_u}{d} = \dfrac{9{,}371^{\text{ft-lbs}}}{40'} = 234 - \text{lb}$ (1.04KN)

Reactions on braced frames:

Both side frames; $R_A = R_B = w_u$x(tributary length) = 83.3plf(15') = 1,250-lb (5.56KN)

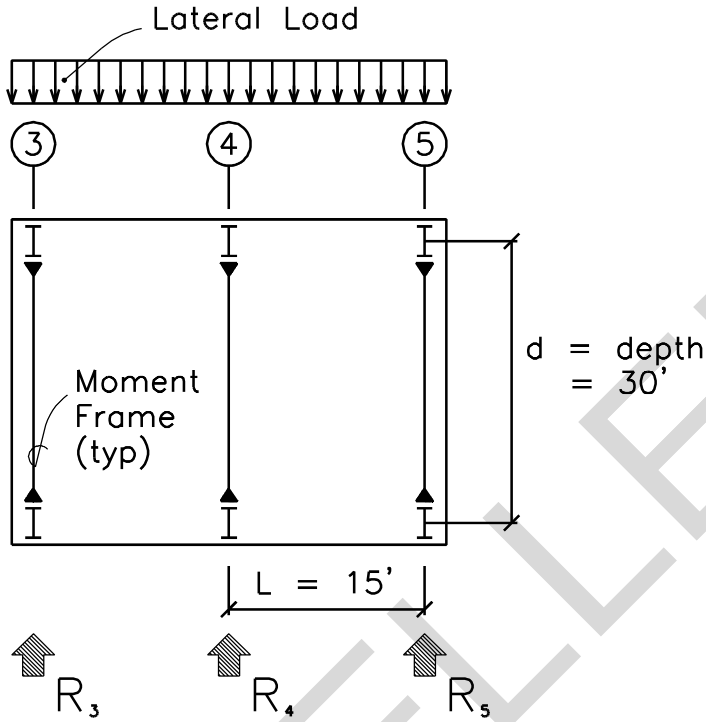
Unit shear: v = R/d = 1,250-lb/40' = 31.3plf (0.46KN/m)

Drag strut forces:



1-lb = 4.448N
1psf = 14.58KN/m

Perform static analysis on two-story structure;
Roof Level
Transverse direction:



1-ft = 0.30m

Lateral load: $w_u = F_{trans}/2L = 2.34^k/30' = 78$ plf (1.14KN/m)

Diagrammatically, Diaphragm moment: $M_u = \frac{w_u L^2}{8} = \frac{78\text{plf}(15')^2}{8} = 2{,}194^{\text{ft-lbs}}$ (2.98KN-m)

Chord forces: $T = C = \frac{M_u}{d} = \frac{2{,}194^{\text{ft-lbs}}}{30'} = 73 - \text{lb}$ (0.32KN)

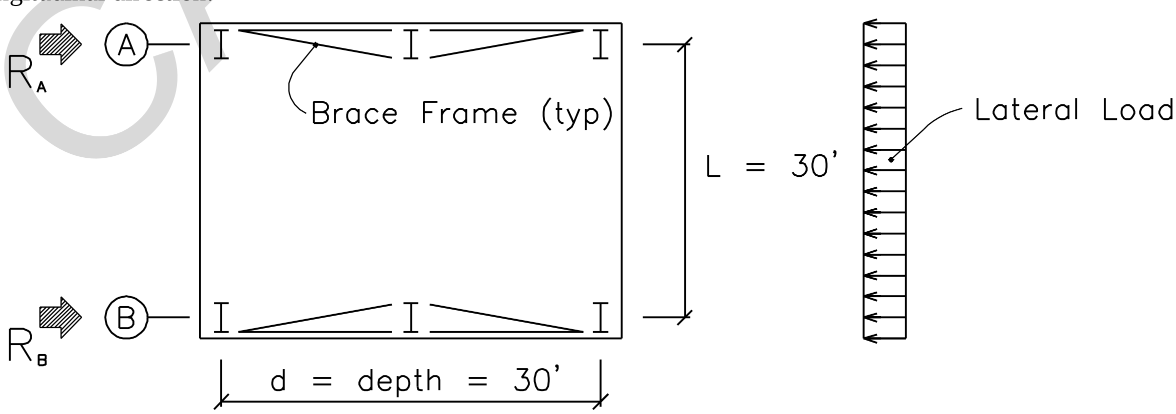
Reactions on moment frames:

End frames; $R_3 = R_5 = w_u$x(tributary length) = 78plf(7.5') = 585-lb (2.60KN)

Interior frame; R_4 = 78plf(15') = 1,170-lb (5.20KN)

Unit shear: v = R/d = 585-lb/30' = 19.5 plf (0.28KN/m)

Longitudinal direction:



1-ft = 0.30m

Lateral load: $w_u = F_{long}/L = 3.15^k/30' = 105\text{plf}$ (1.53KN/m)

Diaphragm moment: $M_u = \dfrac{w_u L^2}{8} = \dfrac{105\text{plf}(30')^2}{8} = 11{,}813^{\text{ft-lbs}}$ (16.02KN-m)

Chord forces: $T = C = \dfrac{M_u}{d} = \dfrac{11{,}813^{\text{ft-lbs}}}{30'} = 394\text{-lb}$ (1.75KN)

Reactions on braced frames:

Both side frames; $R_A = R_B = w_u x(\text{tributary length}) = 105\text{plf}(15') = 1{,}575\text{-lb}$ (7.01KN)

Unit shear: v = R/d = 1,575-lb/30’ = 52.5plf (0.67KN/m)

Drag strut forces: $T_{drag} = C_{drag} = R/2 = 1{,}575\text{-lb}/2 = 788\text{-lb}$ (3.51KN)

Second Floor Level

Note: The relative rigidities of the vertical lateral load resisting elements must be determined in order to establish the distribution of seismic loads. The transverse moment frames were analyzed using a two-dimensional computer analysis program (RISA-2D, version 4.0) to determine their stiffness. The size of the braces, in the moment frame at the interface between the high and low roof structures, was assumed, as they have not yet been designed. The following diagram shows the computer model input and results. Haunch properties are calculated as shown below. The deflection ‘Δ’ was taken at the point of applied loading.

Frame with truss;

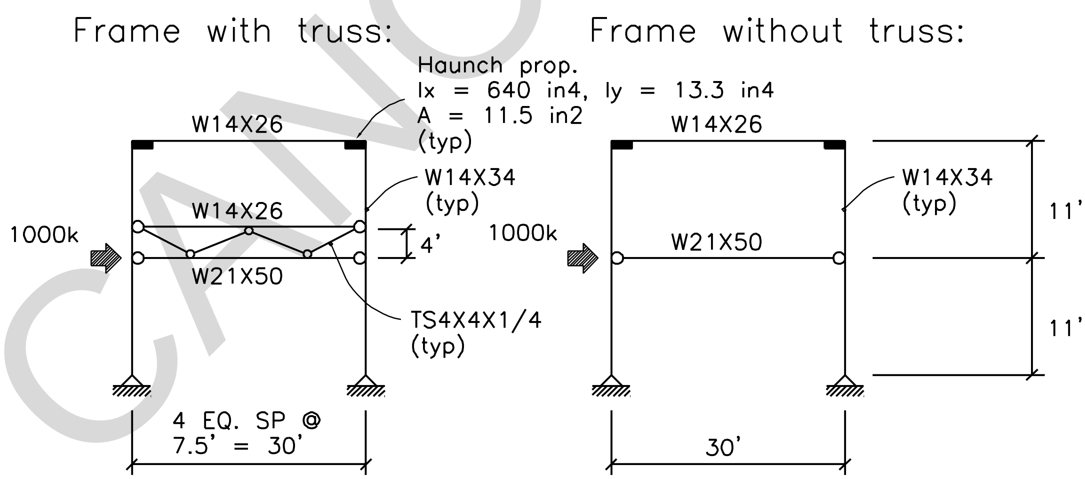
$\Delta_3 = 63.9"$ ($1.62\text{X}10^3$ mm)

$K_3 = \dfrac{1000^k}{63.9"} = 15.7^{k/in}$ ($2.75^{\text{KN/mm}}$)

Frame without truss;

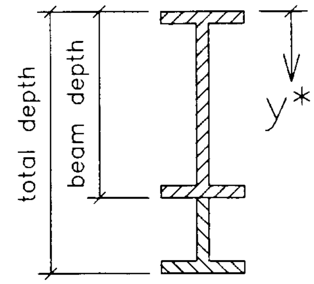
$\Delta_{4,5} = 216"$ ($5.49\text{X}10^3$ mm)

$K_{4,5} = \dfrac{1000^k}{216"} = 4.63^{k/in}$ ($0.81^{\text{KN/mm}}$)



1-in = 25.4mm
1-ft = 0.30m
$1\text{-in}^2 = 645.2\ \text{mm}^2$
$1\text{-in}^4 = 416.2\text{X}10^3\ \text{mm}^4$
1-kip = 4.448KN

Haunch properties are calculated on spreadsheet as follows;



Beam = W14X26

Beam I (in) =	245.00
Beam A (in^2) =	7.69
Beam depth (in) =	13.91
Flange Thickness (in) =	0.420
Web Thickness (in) =	0.255
Flange Width (in) =	5.025
Total Depth (in) =	20.91
Haunch Depth (in) =	7.00

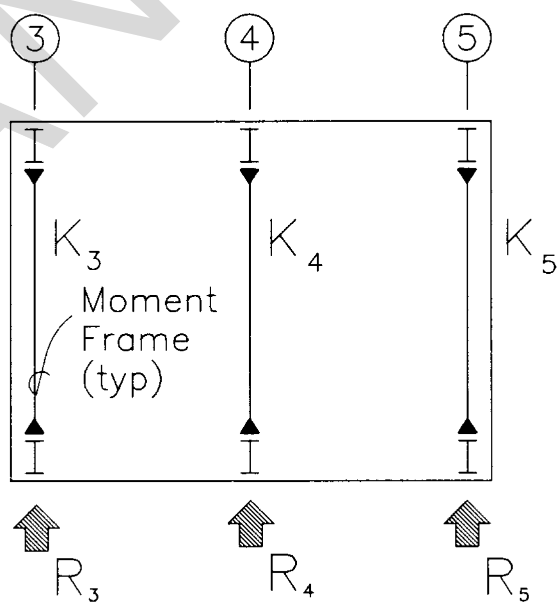
y* (in) =	10.98
Haunch I_x (in^4) =	640
Total A (in^2) =	11.48
Haunch I_y (in^4) =	13.3
Haunch S_x (in^3) =	58
Haunch Z_x (in^3) =	43.24

1-in = 25.4mm
1-in^2 = 645.2 mm^2
1-in^3 = 16.37X10^3 mm^3
1-in^4 = 416.2X10^3 mm^4

Therefore, in the transverse direction:

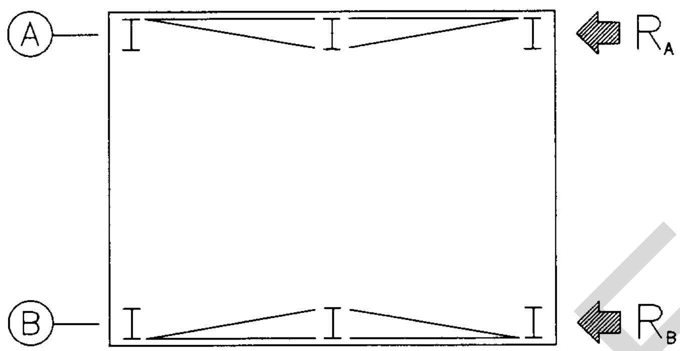
$$R_{trans} = F_{2^{nd}}\left(\frac{K_i}{\sum K_i}\right) \qquad \text{where; } \sum K_i = 15.7^{k/in} + 2(4.63^{k/in}) = 25^{k/in} \ (4.38\text{kn/MM})$$

$$\text{Therefore; } R_3 = 4.86^k\left(\frac{15.7^{k/in}}{25^{k/in}}\right) = 3.05^k \ (13.6\text{KN}) \quad R_4 = R_5 = 4.86^k\left(\frac{4.63^{k/in}}{25^{k/in}}\right) = 0.90^k \ (4.45\text{KN})$$



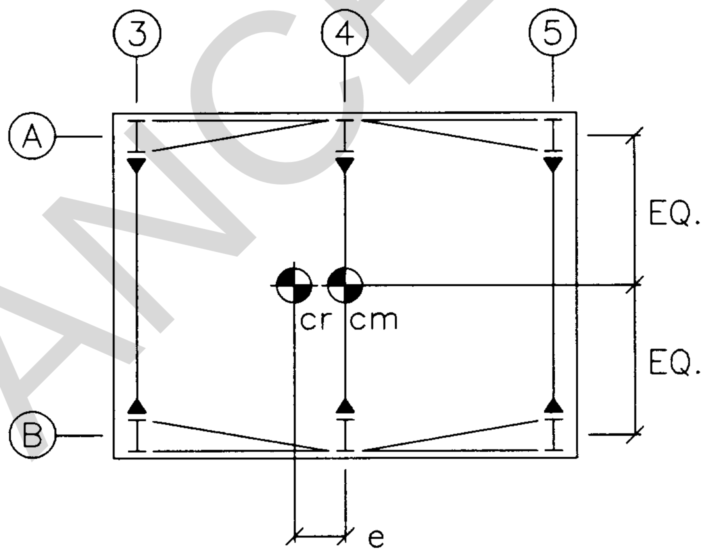
In the longitudinal direction:

$$R_{long} = F_{2^{nd}}\left(\frac{K_i}{\sum K_i}\right) = F_{2^{nd}}\left(\frac{1}{2}\right) \qquad \text{Therefore;} \qquad R_A = R_B = 6.55^k\left(\frac{1}{2}\right) = 3.28^k \ (13.13KN)$$



B-6 *Determine cr and cm.*

Note: Because the roof diaphragms are composed of flexible elements, torsion is only an issue for the second floor diaphragm. Also, by inspection, the cm (center of mass) is located at the geometric plan center of the second floor diaphragm. Additionally, the center of rigidity in the longitudinal direction is also located at the geometric plan center. However, the center of rigidity in the transverse direction must be calculated.



Locate cr_{trans};

$$cr_{trans} = \frac{\sum K_i x_i}{\sum K_i} = \frac{15.7^{k/in}(0) + 4.63^{k/in}(15' + 30')}{15.7^{k/in} + 2(4.63^{k/in})} = 8.35' \ (2.56m)$$

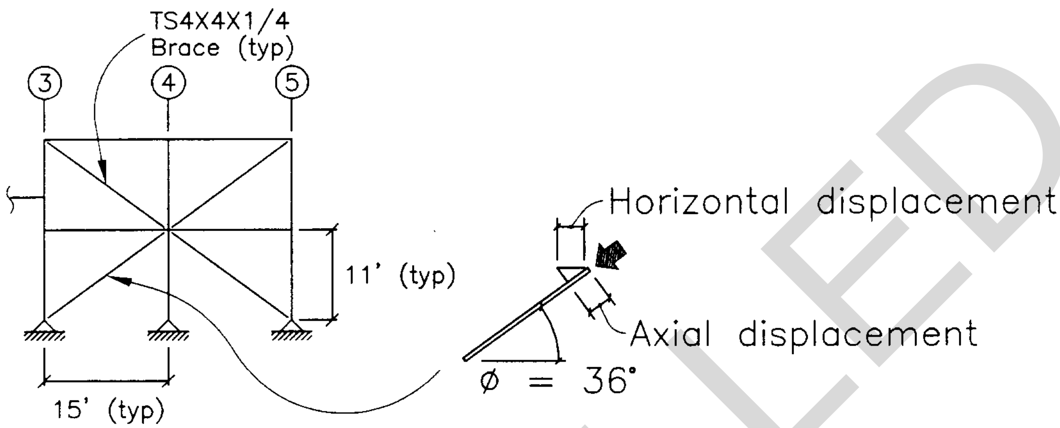
Therefore the eccentricity is;

$$e_{trans} = 15' - 8.35' = 6.65' \ (2.03m)$$

B-7 *Perform torsional analysis.*

Note: In order to determine the polar torsional rigidity 'J_p', the longitudinal brace elements had to be assumed as they have not yet been designed. The analysis could have proceeded using a polar torsional rigidity based on the transverse moment frames alone, but the results would have been overly conservative for the moment frames (this is because the relatively stiffer braced frames and their effect to increase J_p would have been ignored). Therefore, the braced frame elements were chosen as hollow structural sections consisting of TS4X4X1/4.

Calculate Stiffness of braced frames;



1-in = 25.4mm
1-ft = 0.30m
TS4X4X1/4 = TS 101.6mmX101.6mmX6.35mm

Consider a typical brace;

$P_{horiz} = P_{axial} \cos\phi$

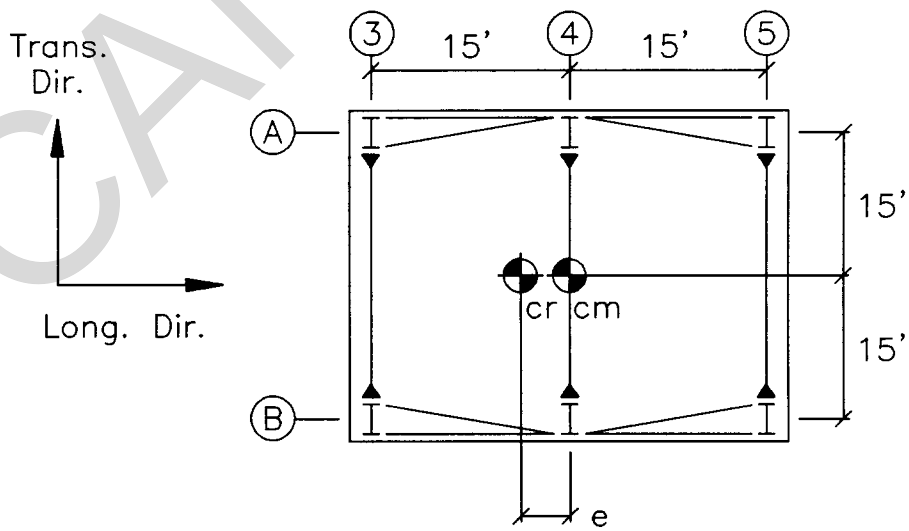
using small angle approximation;

$\Delta_{horiz} = \dfrac{\Delta_{axial}}{\cos\phi}$ Therefore; $K_{horiz} = \dfrac{P_{horiz}}{\Delta_{horiz}} = \dfrac{AE}{L}\cos^2\phi$

For two braces;

$$K_{horiz} = \frac{2AE}{L}\cos^2\phi = \frac{2(3.59in^2)29{,}000ksi}{18.6ft(12''/1')}\left(\frac{15}{18.6}\right)^2 = 607^{k/in}\ (106.3KN/mm)$$

Torsional loads are calculated as follows;



1-ft = 0.30m

$$F_{tors} = \frac{Rd_i}{\sum Rd_i^2} Ve$$

$$F_{acc} = \frac{Rd_i}{\sum Rd_i^2} Ve_{acc}$$

where;

e_{acc} = 0.05% of the building dimension perpendicular to the direction considered.

$e_{acc.\,trans}$ = 0.05(30') = 1.5' (0.46m)

$e_{acc.\,long}$ = 0.05(30') = 1.5' (0.46m)

The calculations were done on spreadsheet and are tabulized below;

V_{trans} =	4.86	kips	e_{trans} =	6.65 ft	$e_{trans.acc}$ =	1.5 ft
V_{long} =	6.55	kips	e_{long} =	0 ft	$e_{long,acc}$ =	1.5 ft

EQ Direction	Element (grid line)	R_{yi} (k/in)	R_{xi} (k/in)	d_{xi} [(1)] (ft)	d_{yi} [(1)] (ft)	R_xd_y or R_yd_x (k-ft/in)	$R_xd_y^2$ or $R_yd_x^2$ (k-ft/in)	F_t [(2)] (kips)	F_{acc} (kips)	$F_{total} = F_t + F_{acc}$ (kips)
Trans	3	15.7	-	8.35	-	131.1	1095	-0.0153	0.0035	0.003
	4	4.63	-	6.65	-	30.8	205	0.0036	0.0008	0.004
	5	4.63	-	21.65	-	100.2	2170	0.0117	0.0026	0.014
Long	A	-	607	-	15	9105	136575	0.0000	0.3234	0.323
	B	-	607	-	15	9105	136575	0.0000	0.3234	0.323

[1]Note: distance 'd' is measured from the cr J_p = 276620

[2] Note: only positive components are added

1-ft = 0.30m
1-kip = 4.448KN
1klf = 0.175KN/mm
1kip-ft/in = 0.053KN-m/mm

B-8 *Determine need for redundancy factor, ρ.*

The building has a seismic design category of D, and therefore, per paragraph 4-4, the redundancy factor is calculated as follows;

$$\rho = 2 - \frac{20}{r_{max}\sqrt{A_x}} \qquad \text{(EQ. 4-1)}$$

Evaluation of r_{max};

For braced frames (except at the first story of the high roof structure);

$$r_{max} = \left(\frac{1}{4}\right) \approx 0.25 \qquad \text{(four braces having equal loads at an angle } \theta \approx 36^\circ\text{)}$$

For braced frames at the first story of the high roof structure;

$$r_{max} = \frac{(3.15^k + 6.55^k + 0.323^k)\left(\frac{1}{4}\right)}{3.15^k + 6.55^k} = 0.26 \qquad \text{(includes torsion effects)}$$

For moment frames (except at the first story of the high roof structure);

$$r_{max} = 1/2 \qquad \text{(two columns per frame with equal shears)}$$

For moment frames at the first story of the high roof structure;

$$r_{max} = \frac{3.05^k + 0.003^k + 0.585^k}{2.34^k + 4.86^k} = 0.51 \qquad \text{(includes torsion effects)}$$

B-9 *Determine need for overstrength factor,* $\Omega_{\circ}$.

Per paragraph 8.6.2 of FEMA 302, connections for diagonal bracing members and collectors shall have a design strength equal to or greater than the nominal tensile strength of the members being connected or $\Omega_{\circ}$ times the design seismic force.

Therefore, for these *force controlled elements* the following seismic load will be used;

Longitudinal direction;

$$E = \Omega_{\circ} Q_E \pm 0.2 S_{DS} D \quad \text{(EQ. 4-6)}$$
$$E = 2Q_E \pm 0.2(0.57)D$$
$$E = 2Q_E \pm 0.114D$$

Transverse direction;

$$E = \Omega_{\circ} Q_E \pm 0.2 S_{DS} D \quad \text{(EQ. 4-7)}$$
$$E = 3Q_E \pm 0.2(0.57)D$$
$$E = 3Q_E \pm 0.114D$$

B-10 *Calculate combined load effects.*

Load combinations per ANSI/ASCE 7-95 and TI 809-04 are as follows;

(1) 1.4D
(2) $1.2D+1.6L+0.5L_r$
(3) $1.2D+0.5L+1.6L_r$
(4) 1.2D+E+0.5L
(5) 0.9D+E

However, per paragraph 4-6 of TI 809-04;

$$E = rQ_E \pm 0.2S_{DS}D$$
$$= 1.0Q_E \pm 0.2(0.57)D$$
$$= Q_E \pm 0.114D$$

Therefore, equations 4 and 5 become;

(4a) $1.314D+Q_E+0.5L$
(4b) $1.086+Q_E+0.5L$
(5a) $1.014D+Q_E$
(5b) $0.786D+Q_E$

B-11 *Determine structural member sizes.*

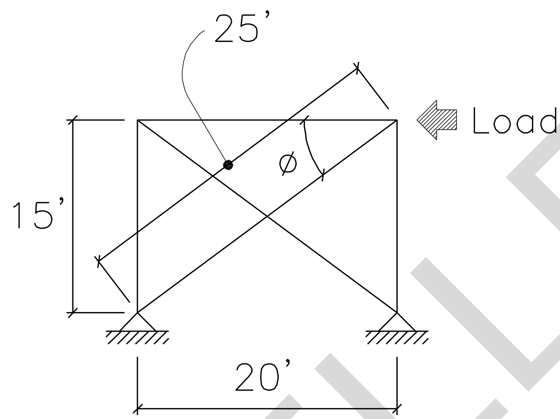
(1) Braced Frames (Low Roof).

Determine Design Loads:

Per paragraph 7-3.a (5) (a), structural steel braced frames will conform to the requirements of the AISC seismic provisions for structural steel buildings.

Axial load per braced frame;

Note: All load combinations reduce to $P_{horiz} = Q_E = F_{roof}/2$



1-ft = 0.30m

Therefore; $$P_u = P_{axial} = \frac{P_{horiz}}{2\cos\phi} = \frac{2.5^k/2}{2(0.8)} = 0.78^k \ \ (3.47KN)$$

Two brace elements per brace frame

Design Members (use HSS w/ F_y = 46ksi or 317.2MPa):

Per paragraph 7-3.b (3), the effective out-of-plane unbraced length is equal to two thirds of the total length.

Therefore; KL = (2/3)25' = 16.7' or 200" (5.09m)

Per AISC seismic provisions; $$\frac{KL}{r} \le \frac{1000}{\sqrt{F_y}} \qquad \frac{h}{t} \le \lambda_p = \frac{110}{\sqrt{F_y}}$$

Therefore; $$r \ge \frac{KL\sqrt{F_y}}{1000} = \frac{200''\sqrt{46^{ksi}}}{1000} = 1.36-\text{in} \ \ (34.5mm)$$

$$\lambda_p = \frac{110}{\sqrt{46^{ksi}}} = 16.2$$

Try TS 4X4X1/4;

From AISC column load tables for an effective length KL = 17' $\phi_c P_n = 42^k > 0.78^k = P_u$ **O.K.**

$\phi_t P_n = 0.9(46^{ksi})3.59-\text{in}^2 = 149^k > 0.86^k$ (662.8KN > 3.83KN) **O.K.**

r = 1.51" > 1.36" = $r_{req'd}$ **O.K.**

h/t = 16 < 16.2 = λ_p **O.K.**

Choose TS 4X4X1/4 (TS101.6mmX101.6mmX6.35mm)

(2) Chord/Collector Elements (Low Roof).

Determine Design Loads:

Seismic;

Note: Only Chord/Collector elements in the plane of the braced frames will be designed. Chord/Collector elements perpendicular to the braced frame are a part of the transverse moment frames. From the static analysis the axial loads are determined as follows;

Maximum Chord Force = 79.2-lb (0.35KN)
Maximum Collector Force = 626-lb (2.78KN) (governs)

Gravity;

w_D = 18.7psf(2.5') = 46.8plf (0.68KN/m)
w_L:
There is no live load reduction per ANSI/ASCE 7-95 for A_T = 2.5'(20') = 50-ft^2 < 200-ft^2 (4.65m^2 < 18.58m^2).
Therefore w_L = 20psf(2.5') = 50plf (0.73KN/m)
Therefore; $w_U = 1.314w_D+0.5w_L$ = 1.314(46.8plf)+0.5(50plf) = 86.5plf (1.26KN/m)

$$M_u = \frac{w_u L^2}{8} = \frac{86.5\text{plf}(20')^2}{8} = 4.33^{\text{ft-kips}} \text{ (5.87KN-m)}$$

$$P_u = \Omega_\circ(Q_E) = 2(626-\text{lb}) = 1.25^k \text{ (5.56KN)} \qquad \text{(Per FEMA 302 Section 8.6.2)}$$

Design Members:
Check the W12X14 choosen previously.
Capacity;

$\phi_b M_n = 47^{\text{ft-kips}}$ Per AISC LRFD 2nd edition load factor design selection table.

$\phi_c P_n = 110^k$ Per AISC LRFD 2nd edition table 3-36 with bending about the x-axis only:

$$\frac{KL}{r} = \frac{1.0(20')(12''/1')}{4.62''} = 52 \Rightarrow \phi_c F_{cr} = 26.54\text{ksi (174.0MPa)},$$

and $\phi_c P_n = A_g(\phi_c F_{cr}) = 4.16 - \text{in}^2(26.54\text{ksi}) = 110^k$ (489.3KN)

Check which interaction equation to use;

$$\frac{P_u}{\phi_c P_n} = \frac{1.25^k}{110^k} = 0.011 < 0.2 \Rightarrow \text{AISC LRFD EQ. H1-1b}$$

$$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) = \frac{1.25}{2(110^k)} + \left(\frac{4.33^{\text{ft-kips}}}{47^{\text{ft-kips}}} + 0\right) = 0.097 < 1.0 \qquad \textbf{O.K.}$$

Choose W12X14 (W304.8mmX0.204KN/m)

(3) Braced Frames (High Roof).

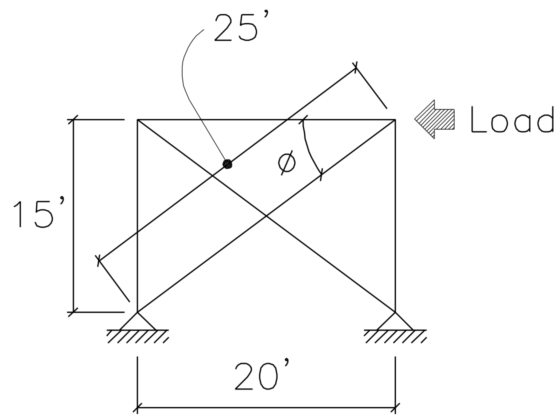
Note: Braces will be designed for the worst case loading at the first story level.

Determine Design Loads:

$$P_{horiz} = Q_E = (F_{roof} + F_{2nd})/2 + F_{torsion} = (3.15^k + 6.55^k)/2 + 0.323^k = 5.17^k \text{ (23.0KN)}$$

Axial load per brace;

$$P_u = P_{axial} = \frac{P_{horiz}}{2\cos\phi} = \frac{5.17^k}{2(0.806)} = 3.21^k \text{ (14.3KN)}$$



1-ft = 0.30m

Design Members (use HSS w/ F_y = 46ksi, or 317.2MPa):

Per paragraph 7-3.b (3), the effective out-of-plane unbraced length is equal to two thirds of the total length.

Therefore; KL = (2/3)18.6' = 12.4' or 149" (3.78m)

Per AISC seismic provisions; $\frac{KL}{r} \leq \frac{1000}{\sqrt{F_y}}$ $\qquad \frac{h}{t} \leq l_p = \frac{110}{\sqrt{F_y}}$

Therefore; $r \geq \frac{KL\sqrt{F_y}}{1000} = \frac{149"\sqrt{46^{ksi}}}{1000} = 1.01 - \text{in} \ \ (2.57\text{mm})$

$$l_p = \frac{110}{\sqrt{46^{ksi}}} = 16.2$$

Try TS 4X4X1/4;

From AISC column load tables for an effective length KL = 13'

$f_c P_n = 68^k > 3.21^k (302.5KN > 14.3KN) = P_u$ **O.K.**

$f_t P_n = 0.9(46^{ksi})3.59 - in^2 = 149^k > 3.21^k (662.8KN > 14.3KN)$ **O.K.**

r = 1.51" > 1.01" = $r_{req'd}$ **O.K.**

h/t = 16 < 16.2 = l_p **O.K.**

Choose TS 4X4X1/4 (TS101.6mmX101.6mmX6.35mm)

(4) Chord/Collector Elements (High Roof).

Determine Design Loads:

Seismic;

Note: Only Chord/Collector elements in the plane of the braced frames will be designed for Chord/Collector elements perpendicular to the braced frame are a part of the transverse moment frames. From the static analysis, the axial loads were determined as follows;

Maximum Chord Force = 73-lb (0.32KN)
Maximum Collector Force = 788-lb (3.51KN) (governs)

By inspection with the design for the low roof, a W12X14 is adequate.

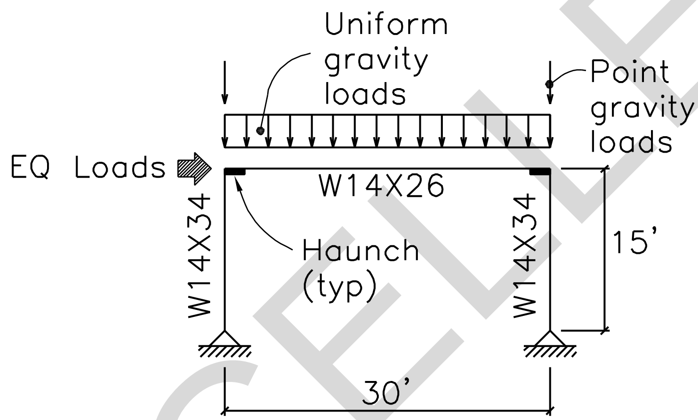
Choose W12X14 (W304.8mmX0.204KN/m)

(5) Moment Frames (Low Roof).

There will be one design for the worst case situation and this design will be used throughout the low roof. The worst case situation is the interior moment frame because it supports the largest tributary area.

Determine Design Loads:

w_L:
Live load reduction per ANSI/ASCE 7-95;
$A_T = 20'(30') = 600\text{-ft}^2\ (55.7\text{m}^2) \Rightarrow R_1 = 0.6$
Roof slope is flat $\therefore R_2 = 1.0$
$\therefore w_L = 20'(20\text{psf})0.6 = 240\text{plf}\ \ (3.50\text{KN/m})$
$w_D = 20'(17.1\text{psf})$
$= 342\text{plf}\ \ (4.99\text{KN/m})$
$P_D = 5.9\text{psf}(7.5')(20') = 885\text{-lb}\ (3.94\text{KN})$ (point load due to wt. of side walls)
$E = 1.0Q_E = 0.95^k\ (4.23\text{KN})$ (applied as a uniform load of $0.95^k/30' = 31.7\text{plf}\ (0.46\text{KN/m})$ along the beam length)



1-in = 25.4mm
1-ft = 0.30m
1-lb = 4.448N

Design Members:

Note; Haunch properties I_x, I_y, S_x, and A as well as the length of the haunch were previously calculated as; $I_x = 640\text{-in}^4\ (266.4\text{X}10^6\ \text{mm}^4)$, $I_y = 13.3\text{-in}^4\ (5.54\text{X}10^6\ \text{mm}^4)$, $S_x = 58\text{-in}^3\ (950.4\text{X}10^3\ \text{mm}^3)$, $A = 11.48\text{-in}^2\ (7.41\text{X}10^3\ \text{mm}^2)$, and 'L' length from centerline of column to toe of haunch is 1.85' (22.25-in or 0.56m).

General;

The moment frame was analyzed using a two-dimensional computer analysis program (RISA-2D, version 4.0). All load combinations were investigated to determine the worst case loading for each element and the worst case deflection for the frame. In all cases, the controlling load combination was equation 4a; $1.314D+Q_E$. After comparing the frame deflection to the allowable story drift, an investigation was undertaken to ensure that plastic hinges would form in their predetermined locations (within the beam at the toe of the haunch as opposed to the face of the column). Last, a check on the strength requirements of the frame was completed.

Drift requirements;

Calculated drift; $d_{calc} = 0.2''\ \ (5.1\text{mm})$

Allowable story drift; $\Delta_{allow} = 0.025h_{sx}$ (Table 6.1)

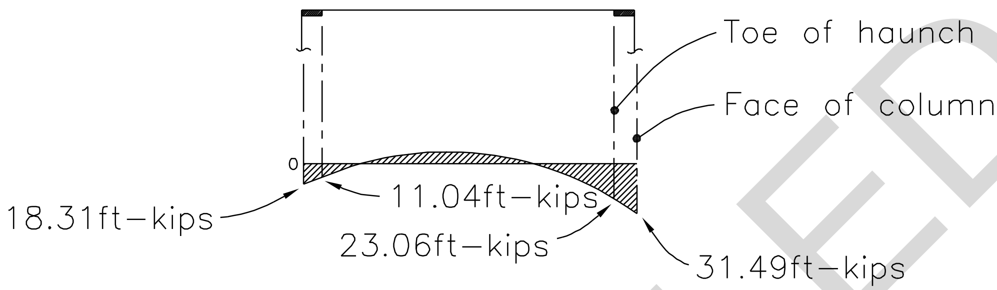
$\Delta_{allow} = 0.025(15'(12''/1')) = 4.5''\ (114.3\text{mm})$

Therefore; $C_d \text{ x } d_{calc} = 5.5(0.2'') = 1.10''(27.9\text{mm}) < \Delta_{allow} = 4.5''\ \ (114.3\text{mm})$ **O.K.**

Check plastic hinge location;

Note: At the time of writing of this problem, it is industry practice to ensure formation of a plastic hinge at the toe of the haunch (rather then at the face of the column) by keeping the ratio of stresses at the toe of the haunch relative to the face of the column greater then 1.2. Also, after the formation of a hinge on one side of the beam, the other side must be checked to ensure that the hinge does not form some where else along the length of the beam. The method is demonstrated as follows;

The resulting moment diagram, showing moments at the face of column and at the toe of the haunch, is as follows;



1-ft-kip = 1.36KN-m

By inspection of this diagram, it is clear that a plastic hinge will form on the right side of the beam first because the moments are greatest at this location (with increased loading, the moments will increase proportionately until yielding occurs).

The stress ratio on the right side is;

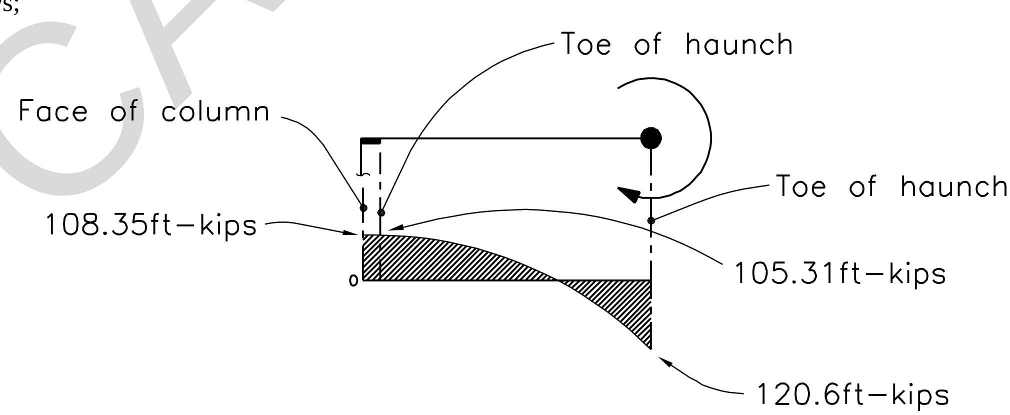
$$\frac{s_{\text{toe-of-haunch}}}{s_{\text{face-of-column}}} = \frac{M_{toe} / S_{x,toe}}{M_{face} / S_{x,haunch}} = \frac{23.06^{\text{ft-kips}}(12''/1') / 35.3 - \text{in}^3}{31.49^{\text{ft-kips}}(12''/1') / 58 - \text{in}^3} = 1.2 \qquad \textbf{O.K.}$$

The left side was investigated by placing a plastic hinge at the assumed hinge location on the right side and analyzing the resulting configuration. The lateral load was increased until yielding occurred at the toe of the haunch on the left side.

$$M_p = Z_x F_y = 40.2 - \text{in}^3(36\text{ksi}) = 120.6^{\text{ft-kips}} \text{ (164KN-m)}$$

$$M_y = S_x F_y = 35.3 - \text{in}^3(36\text{ksi}) = 106^{\text{ft-kips}} \text{ (144KN-m)}$$

The resulting moment diagram, showing moments at the face of column and at the toe of the haunch, is as follows;



1-ft-kip = 1.36KN-m

The resulting stress ratio on the left side is;

$$\frac{s_{\text{toe-of-haunch}}}{s_{\text{face-of-column}}} = \frac{M_{toe} / S_{x,toe}}{M_{face} / S_{x,haunch}} = \frac{105.31^{ft-kips}(12''/1')/35.3-in^3}{108.35^{ft-kips}(12''/1')/58-in^3} = 1.6 > 1.2 \qquad \textbf{O.K.}$$

Strength requirements;

Roof Beam:
The following maximum loads were obtained from the analysis output at the toe of the haunch;

$M_{u,beam} = 23.06^{ft-kips}(31.3KN-m), V_{u,beam} = 6.37^{k}(28.33KN)$

$f_b M_n = 109^{ft-kips}$ (147.8KN-m) per AISC LRFD 2nd ed. load factor design selection table (using an unbraced length 'L_b' of the compression flange of 5')

$f_v V_n = 69^{k}$ (306.9KN) per AISC LRFD 2nd ed. maximum uniform load tables

$\therefore M_{u,beam} = 23.06^{ft-kips} < 109^{ft-kips} = f_b M_n$ (31.3KN-m < 147.8KN-m) **O.K.**

$V_{u,beam} = 6.37^{k} < 69^{k} = f_v V_n$ (28.33KN < 306.9KN) **O.K.**

Check unbraced length of beam flanges (per AISC seismic provisions);

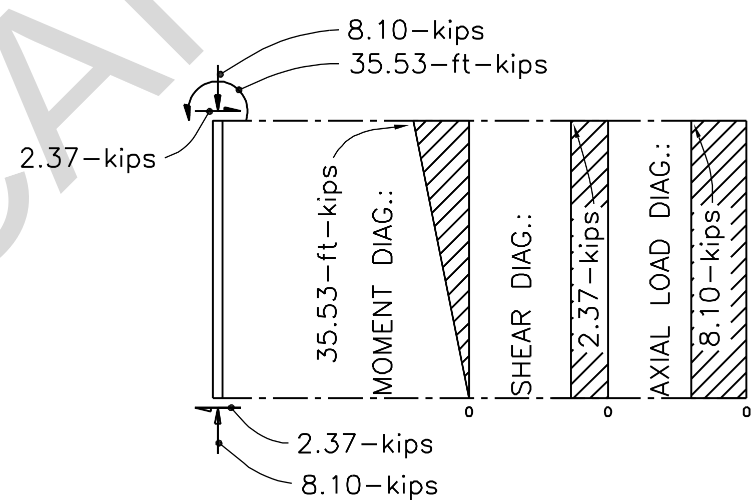
Try 5' on center (same spacing as the perpendicular floor joists);

$$L_b = 5' < 6.25' = \frac{2500(1.08'')(1'/12'')}{36ksi} = \frac{2500 r_y}{F_y} \quad (1.53m < 1.91m) \qquad \textbf{O.K.}$$

Therefore provide lateral support to beam flanges at 5-ft (1.53m) o.c.

Column:
Columns are loaded both in flexure and axially and are therefore beam columns. Additionally, the frames the columns are in are not braced against sidesway and therefore magnified moments due to sidesway must be determined.

A free body diagram showing loading as well as moment, shear, and axial load diagrams for the most highly loaded column are shown below.



1-kip = 4.448KN
1-ft-kip = 1.36KN

Per FEMA 302 paragraph 5.2.6.4.1, 30% of the seismic load effects from the orthogonal direction will be included. Since in the orthogonal direction brace frames are acting, this results in only an additional axial load. Therefore, the total axial load is as follows;

$$P_{u.total} = 8.10^k + 0.3(0.78^k \sin\phi) = 8.10^k + 0.3\left(0.78^k\left(\frac{15'}{25'}\right)\right) = 8.24^k \text{ (36.7KN)}$$

Determine $M_u = B_1 M_{nt} + B_2 M_{lt}$;

M_{nt} was determined by placing a restraint against sidesway at the top of the column (a translation fixed restraint) and re-running the RISA-2D analysis with the governing load combination.

$M_{nt} = +28.74^{ft-kips}$ (38.97KN-m) (calculated reaction at the restraint = 0.91^k) (4.05KN)

M_{lt} was determined by removing all loads from the RISA-2D analysis, including the restraint placed at the top of the column in determining M_{nt}, and then placing only the opposite of the reaction determined previously (-0.91^k) at the top of the column and re-running the RISA-2D analysis.

$M_{lt} = +6.83^{ft-kips}$ (9.26KN-m)

B_1 is determined as follows;

$$B_1 = \frac{C_m}{\left(1 - \frac{P_u}{P_{e1}}\right)} \le 1.0 \quad \text{(EQ. C1-2 AISC LRFD)}$$

where; $C_m = 0.85$ (since member end are restrained at the roof)

$P_u = 8.24^k$ (36.7KN) (as calculated)

$P_{e1} = \frac{\pi^2 E A_g}{(KL/r)^2}$ with; $A_g = 10.0\text{-in}^2$ (6.45×10^3 mm^2)

$E = 29{,}000$ksi (200.0×10^3 MPa)

$K_x = 1.0$ (sidesway inhibited)

$$\frac{K_x L}{r_x} = \frac{2.3(15')(12''/1')}{5.65''} = 73.4$$

$$P_{e1} = \frac{\pi^2(29{,}000\text{ksi})10.0 - \text{in}^2}{(73.4)^2} = 531^k \text{ (2.36MN)}$$

$$B_1 = \frac{0.85}{\left(1 - \frac{8.24^k}{531^k}\right)} = 0.86 < 1.0$$

Therefore, $B_1 = 1.0$

B_2 is determined as follows;

$$B_2 = \frac{1}{1 - \sum P_u\left(\frac{\Delta_{oh}}{\sum HL}\right)} \quad \text{(EQ. C1-4 AISC LRFD)}$$

where; $\Delta_{oh} = 0.2''$ (from RISA-2D analysis)

$\sum H = 0.95^k$ (4.23KN) (lateral loading, "Q_E")

$L = 15'(12''/1') = 180''$ (4.58m) (column height)

$\sum P_u = 8.10^k + 7.18^k + 2(0.14^k) = 15.6^k$ (69.4KN)

orthogonal effects; $0.3(0.78^k(15'/25')) = 0.14^k$ (0.62KN)

load in other column (from RISA-2D analysis)

$$\therefore B_2 = \frac{1}{1 - 15.6^k\left(\frac{0.2"}{0.95^k(180")}\right)} = 1.02 \quad \text{Therefore, } B_2 = 1.02$$

Therefore; $\quad M_u = B_1 M_{nt} + B_2 M_{lt} = 1.0(28.74^{ft-kips}) + 1.02(6.83^{ft-kips}) = 35.7^{ft-kips}$ (48.4KN-m)

Note: Since both M_{nt} and M_{lt} carry the same sign, only the magnitude is shown.

Check compact section criteria (per AISC seismic provisions);

Web local buckling;

$$P_y = A_g F_y = 10.0 - in^2(50ksi) = 500^k \ \ (2.22MN)$$

$$\frac{P_u}{\phi_b P_y} = \frac{8.24^k}{0.9(500^k)} = 0.018 < 0.125$$

$$\therefore \lambda_p = \frac{520}{\sqrt{F_y}}\left[1 - 1.54\frac{P_u}{\phi_b P_y}\right] = \frac{520}{\sqrt{50ksi}}\left[1 - 1.54\frac{8.24^k}{0.9(500^k)}\right] = 71.5$$

$$\lambda_{W10X30} = \frac{h}{t_w} = 43.1 < 71.5 = \lambda_p$$ **O.K.**

Flange local buckling;

$$\lambda_{W10X30} = \frac{b_f}{2t_f} = 7.4 \approx 7.35 = \frac{52}{\sqrt{50ksi}} = \frac{52}{\sqrt{F_y}} = \lambda_p$$ **O.K.**

Determine which interaction equation to use;

Maximum $\frac{KL}{r}$, $\quad K_y = 1.0$ due to the braced frames

K_x is determined using AISC alignment charts as follows;

$$\sum I_c / L_c = 340 - in^4 / 15' = 22.67$$

$$\sum I_g / L_g = 245 - in^4 / 30' = 8.17$$

$$\frac{\sum I_c / L_c}{\sum I_g / L_g} = \frac{22.67}{8.17} = 2.77 = G_A$$

At column base; $G_B = 10$ due to the pinned condition

Therefore, from the charts; $K_x = 2.3$

$$\frac{K_x L_x}{r_x} = \frac{2.3(15'(12"/1'))}{5.83"} = 71$$

$$\frac{K_y L_y}{r_y} = \frac{1.0(15'(12"/1'))}{1.53"} = 117.6 \qquad \text{(governs)}$$

From the AISC table 3-50, $\phi_c F_{cr} = 15.43ksi$ (106.4MPa) (interpolated)

$$\phi_c P_n = A_g(\phi_c F_{cr}) = 10.0 - in^2(15.43ksi) = 154^k \ \ (685.0KN)$$

$$\frac{P_u}{\phi_c P_n} = \frac{8.24^k}{154^k} = 0.054 < 0.2 \ \therefore \text{ use AISC LRFD equation H1-1b}$$

$$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0 \qquad \text{(EQ. H1-1b AISC LRFD)}$$

$\phi_b M_{nx} = 112^{ft-kips}$ per AISC LRFD 2nd ed. beam design charts (using an unbraced length of 15', $C_b = 1.0$, and $\phi_b = 0.9$)

Therefore; $$\frac{8.24^k}{2(154^k)} + \left(\frac{35.7^{ft-kips}}{112^{ft-kips}} + 0\right) = 0.35 < 1.0$$ **O.K.**

$$\therefore B_2 = \frac{1}{1 - 15.6^k\left(\frac{0.2''}{0.95^k(180'')}\right)} = 1.02 \quad \text{Therefore, } B_2 = 1.02$$

Therefore; $M_u = B_1M_{nt} + B_2M_{lt} = 1.0(28.74^{ft-kips}) + 1.02(6.83^{ft-kips}) = 35.7^{ft-kips}$ (48.4KN-m)

Note: Since both M_{nt} and M_{lt} carry the same sign, only the magnitude is shown.

Check compact section criteria (per AISC seismic provisions);

Web local buckling;

$$P_y = A_gF_y = 10.0 - in^2(50ksi) = 500^k \ \ (2.22MN)$$

$$\frac{P_u}{\phi_b P_y} = \frac{8.24^k}{0.9(500^k)} = 0.018 < 0.125$$

$$\therefore \lambda_p = \frac{520}{\sqrt{F_y}}\left[1 - 1.54\frac{P_u}{\phi_b P_y}\right] = \frac{520}{\sqrt{50ksi}}\left[1 - 1.54\frac{8.24^k}{0.9(500^k)}\right] = 71.5$$

$$\lambda_{W10X30} = \frac{h}{t_w} = 43.1 < 71.5 = \lambda_p$$ **O.K.**

Flange local buckling;

$$\lambda_{W10X30} = \frac{b_f}{2t_f} = 7.4 \approx 7.35 = \frac{52}{\sqrt{50ksi}} = \frac{52}{\sqrt{F_y}} = \lambda_p$$ **O.K.**

Determine which interaction equation to use;

Maximum $\frac{KL}{r}$, $K_y = 1.0$ due to the braced frames

K_x is determined using AISC alignment charts as follows;

$$\sum I_c / L_c = 340 - in^4 / 15' = 22.67$$

$$\sum I_g / L_g = 245 - in^4 / 30' = 8.17$$

$$\frac{\sum I_c / L_c}{\sum I_g / L_g} = \frac{22.67}{8.17} = 2.77 = G_A$$

At column base; $G_B = 10$ due to the pinned condition

Therefore, from the charts; $K_x = 2.3$

$$\frac{K_xL_x}{r_x} = \frac{2.3(15'(12''/1'))}{5.83''} = 71$$

$$\frac{K_yL_y}{r_y} = \frac{1.0(15'(12''/1'))}{1.53''} = 117.6 \qquad \text{(governs)}$$

From the AISC table 3-50, $\phi_c F_{cr} = 15.43ksi$ (106.4MPa) (interpolated)

$$\phi_c P_n = A_g(\phi_c F_{cr}) = 10.0 - in^2(15.43ksi) = 154^k \ \ (685.0KN)$$

$$\frac{P_u}{\phi_c P_n} = \frac{8.24^k}{154^k} = 0.054 < 0.2 \ \therefore \ \text{use AISC LRFD equation H1-1b}$$

$$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0 \qquad \text{(EQ. H1-1b AISC LRFD)}$$

$\phi_b M_{nx} = 112^{ft-kips}$ per AISC LRFD 2nd ed. beam design charts (using an unbraced length of 15', $C_b = 1.0$, and $\phi_b = 0.9$)

Therefore; $$\frac{8.24^k}{2(154^k)} + \left(\frac{35.7^{ft-kips}}{112^{ft-kips}} + 0\right) = 0.35 < 1.0$$ **O.K.**

Check Shear; From the analysis output; $V_{u,column} = 2.37^k$ (10.5KN)

$f_v V_n = 77.5^k$ (344.7KN) per AISC LRFD 2nd ed. maximum uniform load tables

$\therefore V_{u,column} = 2.37^k < 77.5^k = f_v V_n$ (10.5KN < 344.7KN) **O.K.**

(1) Moment Frames without truss (High Roof).

There will be one design used for the worst case situation and this design will be used throughout the high roof. The worst case situation is the interior moment frame because it supports the largest tributary area.

Determine Design Loads:
Roof:

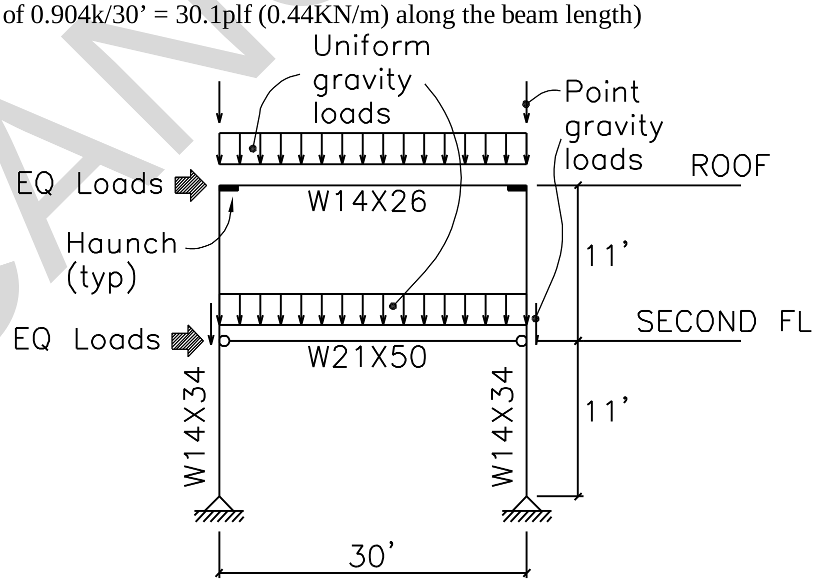
w_{LR}:
Live load reduction per ANSI/ASCE 7-95;
A_T = 15'(30') = 450-ft^2 (41.8m^2)
$R_1 = 1.2\text{-}0.001(A_T)$
= 1.2-0.001(450-ft^2)
= 0.75
Roof slope is flat $\therefore R_2 = 1.0$
$\therefore$ w_{LR} = 15'(20psf)0.75 = 225plf (3.28KN/m)
w_{DR} = 15'(18.7psf)
= 280plf (4.08KN/m)
P_{DR} = 5.9psf(11')(15') = 974-lb (4.33KN) (point load due to wt. of side walls)

Note: Weight of story panel is conservatively placed at the top of the column.

$E_R = 1.0Q_E = 1.17^k$ (5.20KN) (applied as a uniform load of 1.17^k/30' = 39plf (0.57KN/m) along the beam length)

Second Floor:

Note: there is no live load reduction at this level because the floor consists of a one way slab.
w_{LF} = 15'(40psf) = 600plf (8.75KN/m)
w_{DF} = 15'(84.8psf)
= 1,272plf (18.55KN/m)
P_{DF} = 5.9psf(5.5')(15') = 487-lb (2.17KN) (point load due to wt. of side walls)
$E_F = 1.0Q_E = 1.0(0.90^k+0.004^k) = 0.904^k$ (4.02KN)(includes torsion effects, and is applied as a uniform load of 0.904k/30' = 30.1plf (0.44KN/m) along the beam length)



1-in = 25.4mm
1-ft = 0.30m
1plf = 14.58N/m

Design Members:

Note; As for the low roof area haunch properties I_x, I_y, S_x, and A as well as the length of the haunch were previously calculated as; I_x = 640-in^4 (266.4X10^6 mm^4), I_y = 13.3-in^4 (5.54X10^6 mm^4), S_x = 58-in^3 (950.4X10^3 mm^3), A = 11.48-in^2 (7.41X10^3 mm^2), and 'L' length from centerline of column to toe of haunch is 1.85' (22.25")(0.56m).

General;

As in the case of the low roof, the moment frame was analyzed using a two-dimensional computer analysis program (RISA-2D, version 4.0). All load combinations were investigated to determine the worst case loading for each element and the worst case deflection for the frame. In all cases, the controlling load combination was again equation 4a; 1.314D+Q_E+0.5L. After comparing the frame deflection to the allowable story drift, a check on the strength requirements of the frame were completed. Since the same members being used are the same as for the frame elements as at the low roof, a check to ensure the location of the plastic hinge was determined to be unnecessary.

Drift requirements;

Calculated drift; $d_{calc} = 0.565''$ (14.4mm) (worst case at first story level)

Allowable story drift; $\Delta_{allow} = 0.025h_{sx}$ (Table 6.1)

$\Delta_{allow} = 0.025(11'(12''/1')) = 3.3''$ (83.8mm)

Therefore; $C_d \, x \, d_{calc} = 5.5(0.565'') = 3.11'' < \Delta_{allow} = 3.3''$ (79.0mm < 83.8mm) **O.K.**

Strength requirements;

Roof Beam:
The following maximum loads were obtained from the analysis output at the toe of the haunch;

$M_{u,beam} = 30.79^{ft-kips}, V_{u,beam} = 5.99^{k}$ (26.6KN)

$f_b M_n = 109^{ft-kips}$ (147.8KN-m) per AISC LRFD 2nd ed. load factor design selection table (using an unbraced length 'Lb' of the compression flange of 5'(1.53m))

$f_v V_n = 69^{k}$ (306.9KN) per AISC LRFD 2nd ed. maximum uniform load tables

$\therefore M_{u,beam} = 30.79^{ft-kips} < 109^{ft-kips} = f_b M_n$ (41.8KN-m < 147.8KN-m) **O.K.**

$V_{u,beam} = 5.99^{k} < 69^{k} = f_v V_n$ (26.6KN < 306.9KN) **O.K.**

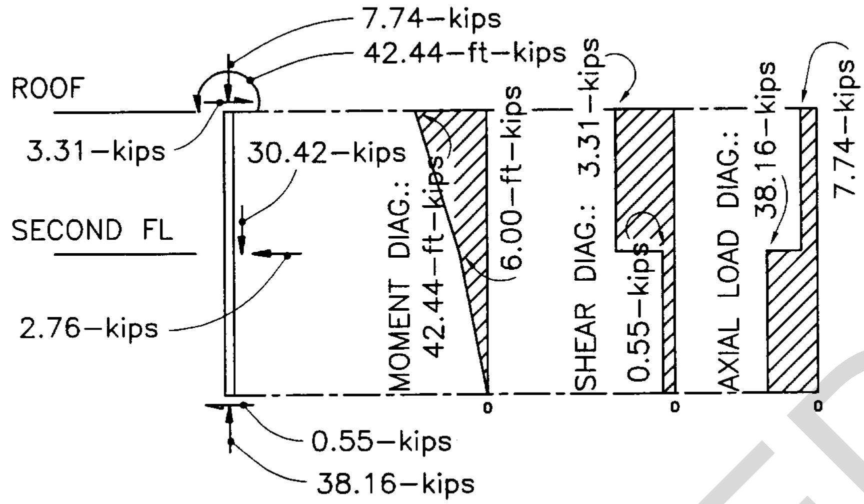
Column:

Note: The columns will be analyzed as if it were a single story column spanning from the first floor level to the roof and subjected to transverse loading (caused by the second floor diaphragm). Therefore, the interstory drift in the moment magnification calculation will be taken as the displacement of the roof level relative to the first floor level.

Per FEMA 302 paragraph 5.2.6.4.1, 30% of the seismic load effects from the orthogonal direction will be included. Since in the orthogonal direction brace frames are acting, this results in only an additional axial load. However, in this case the load resulting from the braced frames is zero.

$P_{u,total} = 38.16^{k}$ (169.7KN)

A free body diagram showing loading as well as moment, shear, and axial load diagrams for the most highly loaded column are shown below.



1-kip = 4.48KN
1-ft-kip = 1.356KN-m

Determine $M_u = B_1 M_{nt} + B_2 M_{lt}$;

$M_{nt} = -25.39^{\text{ft-kips}}$ (34.4KN-m) (calculated reaction at the restraints; @ roof = -1.01^k (4.49KN), @ floor = -1.09^k (4.85KN))

$M_{lt} = -17.11^{\text{ft-kips}}$ (23.2KN-m)

B_1 is determined as follows;

$$B_1 = \frac{C_m}{\left(1 - \frac{P_u}{P_{e1}}\right)} \le 1.0 \qquad \text{(EQ. C1-2 AISC LRFD)}$$

where; $C_m = 0.85$ (since member end is restrained at the roof)

$P_u = 38.16^k$ (169.7KN) (as calculated)

$P_{e1} = \dfrac{\pi^2 E A_g}{(KL/r)^2}$ with; $A_g = 10.0\text{-in}^2$ (6.45×10^3 mm^2)

$E = 29{,}000$ksi (200×10^3 MPa)

$K_x = 1.0$ (sidesway inhibited)

$$\frac{K_x L}{r_x} = \frac{2.3(11')(12''/1')}{5.83''} = 22.7$$

$$P_{e1} = \frac{\pi^2 (29{,}000\text{ksi})10.0-\text{in}^2}{(22.7)^2} = 5{,}555^k \quad (27.71\text{MN})$$

$$\therefore B_1 = \frac{0.85}{\left(1 - \frac{38.16^k}{5{,}555^k}\right)} = 0.856 < 1.0$$

Therefore, $B_1 = 1.0$

B_2 is determined as follows;

$$B_2 = \frac{1}{1 - \sum P_u \left(\frac{\Delta_{oh}}{\sum HL}\right)} \qquad \text{(EQ. C1-4 AISC LRFD)}$$

where; $\Delta_{oh} = 0.910''$ (23.1mm) (from RISA-2D analysis)

$$\sum H = 1.17^k + 0.904^k = 2.07^k \text{ (9.20KN)(lateral loading, "}Q_E\text{")}$$
$$L = 22'(12"/1') = 264" \text{ (6.71m)} \quad \text{(column height)}$$
$$\sum P_u = 38.16^k + 35.85^k = 74.01^k \text{ (329.2KN)}$$

Note: The 35.85^k load in other column (from RISA-2D analysis)

$$\therefore B_2 = \frac{1}{1 - 74.01^k\left(\dfrac{0.910"}{2.07^k(264")}\right)} = 1.14 \qquad \text{Therefore, } B_2 = 1.14$$

Therefore; $\quad M_u = B_1 M_{nt} + B_2 M_{lt} = 1.0(25.39^{ft-kips}) + 1.14(17.11^{ft-kips}) = 45^{ft-kips}$ (61.0KN-m)

Check compact section criteria (per AISC seismic provisions);
Web local buckling;

$$P_y = A_g F_y = 10.0 - in^2(50ksi) = 500^k \text{ (2.22MN)}$$

$$\frac{P_u}{\phi_b P_y} = \frac{38.3^k}{0.9(500^k)} = 0.085 < 0.125$$

$$\therefore \lambda_p = \frac{520}{\sqrt{F_y}}\left[1 - 1.54\frac{P_u}{\phi_b P_y}\right] = \frac{520}{\sqrt{50ksi}}\left[1 - 1.54\frac{38.16^k}{0.9(500^k)}\right] = 63.9$$

$$\lambda_{W10X30} = \frac{h}{t_w} = 43.1 < 63.9 = \lambda_p$$ **O.K.**

Note: Flange local buckling has been checked previously.
Determine which interaction equation to use;

Maximum $\dfrac{KL}{r}$, $\quad K_y = 1.0$ due to the braced frames

K_x is determined using AISC alignment charts as follows;

$$\sum I_c / L_c = 340 - in^4 / 22' = 15.5$$

$$\sum I_g / L_g = 245 - in^4 / 30' = 8.17$$

$$\frac{\sum I_c / L_c}{\sum I_g / L_g} = \frac{15.5}{8.17} = 1.90 = G_A$$

At column base; $G_B = 10$ due to the pinned condition
Therefore, from the charts; $K_x = 2.1$

$$\frac{K_x L_x}{r_x} = \frac{2.1(22'(12"/1'))}{5.83"} = 95.1$$ (governs)

$$\frac{K_y L_y}{r_y} = \frac{1.0(11'(12"/1'))}{1.53"} = 86.3$$

From the AISC table 3-50, $\phi_c F_{cr} = 21.94ksi$ (151.3MPa) (interpolated)

$$\phi_c P_n = A_g(\phi_c F_{cr}) = 10.0 - in^2(21.94ksi) = 219^k \text{ (974.1KN)}$$

$$\frac{P_u}{\phi_c P_n} = \frac{38.16^k}{219^k} = 0.17 < 0.2 \therefore \text{ use AISC equation H1-1b}$$

$$\frac{P_u}{2\phi_c P_n}+\left(\frac{M_{ux}}{\phi_b M_{nx}}+\frac{M_{uy}}{\phi_b M_{ny}}\right)\le 1.0 \qquad \text{(EQ. H1-1b AISC LRFD)}$$

$\phi_b M_{nx} = 128^{\text{ft-kips}}$ (173.6KN-m) per AISC LRFD 2nd ed. beam design charts (using an unbraced length of 11' (3.36m), $C_b = 1.0$, and $\phi_b = 0.9$)

Therefore; $\dfrac{38.16^k}{2(219^k)}+\left(\dfrac{45^{\text{ft-kips}}}{128^{\text{ft-kips}}}+0\right)=0.44<1.0$ **O.K.**

Check Shear; From the analysis output; $V_{u.column} = 3.31^k$ (14.7KN)

$\phi_v V_n = 77.5^k$ (344.7KN) per AISC LRFD 2nd ed. maximum uniform load tables

$\therefore V_{u.column} = 3.31^k < 77.5^k = \phi_v V_n$ (14.7KN < 344.7KN) **O.K.**

(7) Moment Frame with truss (High Roof).

Determine Design Loads:
Roof:

w_{LR}:
Live load reduction per ANSI/ASCE 7-95;
$A_T = 7.5'(30') = 225\text{-ft}^2$ (20.96m^2)
$R_1 = 1.2-0.001(A_T)$
$= 1.2-0.001(225\text{-ft}^2)$
$= 0.975$
Roof slope is flat $\therefore R_2 = 1.0$
$\therefore\ w_{LR} = 7.5'(20\text{psf})0.975 = 146\text{plf}$ (2.13KN/m)
$w_{DR} = 7.5'(18.7\text{psf})$
$= 140\text{plf}$ (2.04KN/m)
$P_{DR} = 5.9\text{psf}(11')(7.5') = 490\text{-lb}$ (2.18KN) (point load due to wt. of side walls)

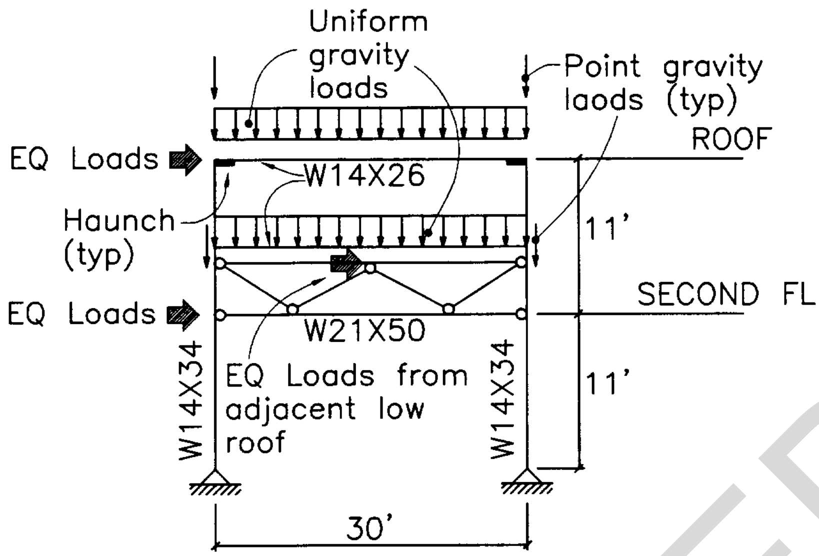
Note: Weight of story panel is conservatively placed at the top of the column.
$E_R = 1.0Q_E = 0.585^k$ (2.60KN)(applied as a uniform load of $0.585^k/30' = 19.5\text{plf}$ (0.28KN/m) along the beam length)

Second Floor:

Note: there is no live load reduction at this level because the floor consists of a one way slab.
$w_{LF} = 7.5'(40\text{psf}) = 300\text{plf}$ (4.38KN/m)
$w_{DR} = 7.5'(84.8\text{psf})$
$= 636\text{plf}$ (9.28KN/m)
$P_{D2} = 5.9\text{psf}[(7.5')(5.5')+(5.5')(10')] = 568\text{-lb}$ (2.53KN) (point load due to wt. of side walls)
$E_F = 1.0Q_E = 3.05^k+0.003^k=3.053^k$ (13.58KN) (applied as a uniform load of $3.053^k/30' = 102\text{plf}$ (1.49KN/m) along the beam length)

Loads from adjacent low roof:

w_{LR}:
Live load reduction per ANSI/ASCE 7-95;
$A_T = 10'(30') = 300\text{-ft}^2$ (27.9m^2)
$R_1 = 1.2-0.001(A_T)$
$= 1.2-0.001(300\text{-ft}^2)$
$= 0.90$
Roof slope is flat $\therefore R_2 = 1.0$
$\therefore\ w_{LR} = 10'(20\text{psf})0.90 = 180\text{plf}$ (2.63KN/m)
$w_{DR} = 10'(17.1\text{psf})$
$= 171\text{plf}$ (2.49KN/m)
$E_R = 1.0Q_E = 0.475^k$ (2.11KN) (applied as a uniform load of $0.475^k/30' = 15.8\text{plf}$ (0.23KN/m) along the beam length)



1-in = 25.4mm
1-ft = 0.30mm
1plf = 14.58N/m

Design Members:

Note; As for the low roof area haunch properties I_x, I_y, S_x, and A as well as the length of the haunch were previously calculated as; I_x = 640-in^4 (266.4X10^6 mm^4), I_y = 13.3-in^4 (5.54X10^6 mm^4), S_x = 58-in^3 (950.4X10^3 mm^3), A = 11.48-in^2 (7.41X10^3 mm^2), and 'L' length from centerline of column to toe of haunch is 1.85' (22.25") (0.56m).

General;

As in the case of the low roof, the moment frame was analyzed using a two-dimensional computer analysis program (RISA-2D, version 4.0). All load combinations were investigated to determine the worst case loading for each element and the worst case deflection for the frame. In all cases, the controlling load combination was again equation 4a; 1.314D+Q_E. After comparing the frame deflection to the allowable story drift, a check on the strength requirements of the frame were completed. Since the same members are being used for the moment frame elements as at the low roof, a check to ensure the location of the plastic hinge formation was determined to be unnecessary.

Drift requirements;

Calculated drift; δ_{calc} = 0.267" (6.8mm)

Allowable story drift; $\Delta_{allow} = 0.025h_{sx}$ (Table 6.1)

Δ_{allow} = 0.025(11'(12"/1')) = 3.3" (83.8mm)

Therefore; $C_d x \delta_{calc}$ = 5.5(0.267") = 1.47" < Δ_{allow} = 3.3" (37.3mm < 83.8mm) **O.K.**

Strength requirements;

Roof Beam:
The following maximum loads were obtained from the analysis output at the toe of the haunch;

$M_{u,beam} = 10.14^{ft-kips}(13.74KN-m), V_{u,beam} = 2.49^{k}(11.08KN)$

$\phi_b M_n = 109^{ft-kips}$ (147.8KN-m) per AISC LRFD 2nd ed. load factor design selection table (using an unbraced length 'Lb' of the compression flange of 5' (1.53m))

$\phi_v V_n = 69^k$ (306.9KN) per AISC LRFD 2nd ed. maximum uniform load tables

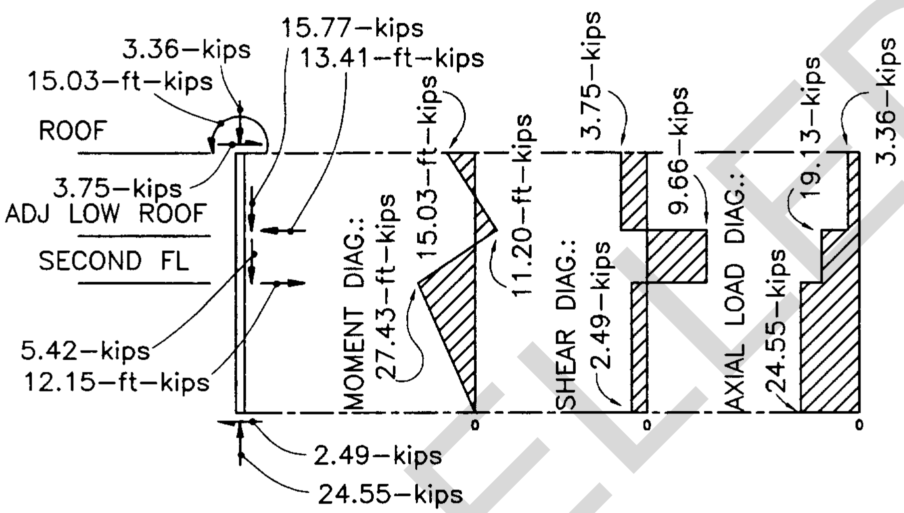
$\therefore M_{u,beam} = 10.14^{ft-kips} < 109^{ft-kips} = \phi_b M_n$ (13.74KN-m < 147.8KN-m) **O.K.**

$V_{u,beam} = 2.49^k < 69^k = \phi_v V_n$ (11.08KN < 306.9KN) **O.K.**

Column:

Note: The columns will be analyzed as if it were a single story column spanning from the first floor level to the roof and subjected to transverse loading (caused by the second floor diaphragm). Therefore, the interstory drift in the moment magnification calculation will be taken as the displacement of the roof level relative to the first floor level.

A free body diagram with loads as well as moment, shear, and axial load diagrams for the most highly loaded column are shown below.



1-kip = 4.448KN
1-ft-kip = 1.356KN-m

Per FEMA 302 paragraph 5.2.6.4.1, 30% of the seismic load effects from the orthogonal direction will be included. Since in the orthogonal direction brace frames are acting this results in only an additional axial load. The axial load due to the brace frame is calculated as follows;

$$P_{axial} = \frac{P_{horiz}}{2\cos\phi} = \frac{(3.15^k/2)}{2(0.806)} = 0.98^k \text{ (4.36KN)}$$

$$P_{u,total} = 24.55^k + 0.3(0.98^k \sin\phi) = 24.55^k + 0.3\left(0.98^k\left(\frac{11'}{18.6'}\right)\right) = 24.7^k \text{ (109.9KN)}$$

Determine $M_u = B_1 M_{nt} + B_2 M_{lt}$;

$M_{nt} = -12.60^{ft-kips}$ (17.09KN-m) (calculated reaction at the restraints; @ roof = -1.92^k (8.54KN), @ floor = -20.97^k (93.3KN), and @ low roof = $+18.39^k$ (81.8KN))

$M_{lt} = -27.37^{ft-kips}$ (37.11KN-m)

B_1 is determined as follows;

$$B_1 = \frac{C_m}{\left(1 - \frac{P_u}{P_{e1}}\right)} \le 1.0 \qquad \text{(EQ. C1-2 AISC LRFD)}$$

where; $C_m = 0.85$ (since member end is restrained at the roof)

$P_u = 24.7^k$ (109.9KN) (as calculated)

$P_{e1} = 5{,}555^k$ (24.71MN) (as previously calculated)

$$\therefore B_1 = \frac{0.85}{\left(1 - \dfrac{24.7^k}{5{,}555^k}\right)} = 0.854 < 1.0$$ Therefore, $B_1 = 1.0$

B_2 is determined as follows;

$$B_2 = \frac{1}{1 - \sum P_u\left(\dfrac{\Delta_{oh}}{\sum HL}\right)}$$ (EQ. C1-4 AISC LRFD)

where; $\Delta_{oh} = 0.279"$ (7.1mm) (from RISA-2D analysis)

$\sum H = 5.85^k + 0.475^k + 3.053^k = 9.38^k$ (41.7KN) (lateral loading, "Q_E")

L = 22'(12"/1') = 264" (6.71m) (column height)

$$\sum P_u = 24.7^k + 21.00^k + 0.3\left(0.98\left(\frac{11'}{18.6'}\right)\right) = 45.9^k \text{ (204.2KN)}$$

load in other column (from RISA-2D analysis)

$$\therefore B_2 = \frac{1}{1 - 45.9^k\left(\dfrac{0.279"}{9.38^k(264")}\right)} = 1.01$$ Therefore, $B_2 = 1.01$

Therefore; $M_u = B_1 M_{nt} + B_2 M_{lt} = 1.0(12.60^{ft-kips}) + 1.01(27.37^{ft-kips}) = 40.2^{ft-kips}$ (54.51KN-m)

Note: Since both M_{nt} and M_{lt} carry the same sign, only the magnitude is shown.

Note: Compact section criteria have been checked previously.

Determine which interaction equation to use;
Note: By inspection, K_yL/r_y governs;

$$\frac{K_y L_y}{r_y} = \frac{1.0(11'(12"/1'))}{1.53"} = 86.3$$

From the AISC table 3-50, $\phi_c F_{cr} = 24.7\text{ksi} \Rightarrow \phi_c P_n = A_g(\phi_c F_{cr}) = 10-\text{in}^2(24.7\text{ksi}) = 247^k$ (1.10MN)

$$\frac{P_u}{\phi_c P_n} = \frac{24.7^k}{247^k} = 0.05 < 0.2 \therefore \text{ use AISC equation H1-1b}$$

$$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$$ (EQ. H1-1b AISC LRFD)

$\phi_b M_{nx} = 128^{ft-kips}$ (173.6KN-m) (calculated previously)

Therefore; $$\frac{24.7^k}{2(247^k)} + \left(\frac{40.2^{ft-kips}}{128^{ft-kips}} + 0\right) = 0.36 < 1.0$$ **O.K.**

Check Shear; From the analysis output; $V_{u,column} = 9.66^k$ (42.97KN)

$f_v V_n = 77.5^k$ (344.7KN) (calculated previously)

$\therefore V_{u,column} = 9.66^k < 77.5^k = f_v V_n$ (42.97KN < 344.7KN) **O.K.**

B-12 *Check allowable drift and P Δ effect.*

Transverse direction;

Drift;

In the transverse direction, allowable drift was checked during the moment frame design.

P Δ effect;

$$q = \frac{P_x \Delta}{V_x h_{sx} C_d} \qquad \text{(EQ. 5.3.7.2-1 FEMA 302)}$$

By inspection, the worst case condition occurs at the first story of the interior high roof moment frame because it has the largest total vertical load "P_x" and the largest design story drift "Δ".

P_x = Total vertical design load without load factors
= 15'(0.225+0.280+0.600+1.272)klf+2(0.974^k+0.649^k)
= 38.9^k (73.0KN)
Δ = 0.565" (14.4mm)
$V_x = 1.17^k + 0.90^k + 0.004 = 2.074^k$ (9.23KN)
h_{sx} = 11'(12"/1') = 132" (3.36m)
C_d = 5.5

$$\therefore q = \frac{38.9^k (0.565")}{2.074^k (132")5.5} = 0.015 < 0.10$$

Therefore, P Δ effects need not be considered in the transverse direction.

Longitudinal direction;

Check Drift;

Low Roof;
The stiffness for a single 'x' brace (one per building side) consisting of two braces in the horizontal direction is;

$$K_{horiz} = \frac{2AE}{L}\cos^2 f = \frac{2(3.59 - in^2)29{,}000ksi}{25'(12"/1')}\cos^2(37^\circ) = 444 - k/in \ \ (77.8KN/mm)$$

Note: All load combinations reduce to $P_u = E = Q_E = 1.25^k$ (5.56KN).

$$\therefore d_{calc} = \frac{P_u}{K_{horiz}} = \frac{1.25^k}{444 - k/in} = 0.003" \ \ (0.08mm)$$

$$\Delta_{allow} = 0.025h_{sx} = 0.025(15'(12"/1')) = 4.5" \ \ (114.3mm)$$

$C_d \, x \, d_{calc} = 5.0(0.003") = 0.015" < 4.5" = \Delta_{allow}$ (0.38mm < 114.3mm) **O.K.**

High Roof;
The stiffness of two diagonal braces (two per building side per story) in the horizontal direction is;

$$K_{horiz} = \frac{2AE}{L}\cos^2\phi = \frac{2(3.59 - in^2)29{,}000ksi}{18.6'(12"/1')}\cos^2(36^\circ) = 611 - k/in\ \ (107.0KN/mm)$$

Note: The worst case condition occurs at the first story because the stiffness is the same while the loads are larger. The worst case condition is checked s follows:

$$\therefore \delta_{calc} = \frac{P_u}{K_{horiz}} = \frac{1.575^k + 3.28^k + 0.323^k}{611 - k/in} = 0.008"\ \ (0.20mm)$$

$$\Delta_{allow} = 0.025h_{sx} = 0.025(11'(12"/1')) = 3.3"\ \ (83.8mm)$$

$$C_d x \delta_{calc} = 5.0(0.008") = 0.04" < 3.3" = \Delta_{allow}\ \ (1.02mm < 83.8mm)$$ **O.K.**

$P\Delta$ effect;

By inspection, $P\Delta$ effect will not be an issue because it is not as issue for the more flexible transverse moment frames.

e. Enhanced Performance Objective (following steps in table 4-6 for Immediate Occupancy for an Essential Facility)

Note: Ground Motion B (3/4 MCE) is to be used. Seismic effects are to be scaled by the factor R x 0.75/0.67, and 'm' values for Immediate Occupancy are applicable.

F-1 through F-3 *Determine seismic effects.*

Note: Step F of Table 4-6 for performance objective 3B (Immediate Occupancy for an Essential Facility) specifies that seismic effects for the enhanced performance objective may be determined by scaling the values used for performance objective 1A as follows;

$$\text{Base Shear} = V = Rx\left(\frac{0.75}{0.67}\right)xC_1xC_2xC_3x(C_sxW)$$

where; C_s = Previously calculated seismic response coefficient determined in performance objective 1A (step B-3)

W = Previously calculated dead load and applicable live load

R = Response modification coefficient used in performance objective 1A

Coefficients C_1, C_2, and C_3 are defined in paragraph 5-4f(2).

$T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.37}{0.57} = 0.65$ (Note: T_s is calculated by equating equations 3-11 and 3-12)

$T_{transverse} = 0.36\sec$, $T_{longitudinal} = 0.2\sec$

$T_{transverse} = 0.36\sec < 0.65\sec = T_s$, $T_{longitudinal} = 0.2\sec < 0.65 = T_s$

C_1 is interpolated between 1.5 for $T \le 0.10\sec$, and 1.0 for $T \ge T_s$ in accordance with the alternate procedure listed in section 3.3.1.3 of FEMA 273 (dated October 1997).

$C_{1,transverse} = 1.26$, $C_{1,longitudinal} = 1.41$

$C_2 = 1.0$ per table 5-2

$C_3 = 1.0$ for the building has a positive post yield stiffness

Therefore, the scale factors in each of the buildings principle directions are calculated as follows;

$$SF_{trans} = 8\left(\frac{0.75}{0.67}\right)1.26(1.0)1.0 = 11.3$$

$$SF_{long} = 6\left(\frac{0.75}{0.67}\right)1.41(1.0)1.0 = 9.47$$

Therefore use $SF_{trans} = 11.3$, and $SF_{long} = 9.47$

F-4 *Determine combined load effects.*

Load combinations were previously determined in step B-10.

F-5 *Identify force controlled and deformation controlled structural components.*

Note: Table 7-10 for concentric braced frames, and Table 7-12 for fully restrained moment frames identify deformation controlled components for these lateral load resisting systems and supply m values for them. Components of concentric braced frame or fully restrained moment frame lateral load resisting systems not identified in these tables are force-controlled components. Therefore, connections for concentric braced frames, and Chord/Collector elements and their connections are force-controlled components. Additionally, braces in the moment frame with a truss are considered force-controlled because yielding of the deformation-controlled components controls the forces that can be delivered to them.

Deformation controlled components and associated m factors;

Moment frames;

Beams at plastic hinge location in flexure;

$F_{ye} = R_y F_y = 1.5(36\text{ksi}) = 54\text{ksi}$ (372.3MPa), with $R_y = 1.5$ per AISC seismic provisions sec. 6.2 for ASTM A36

$$\therefore \left(\frac{b}{2t_f}\right)_{W14X26} = 6.0 < 7.08 = \frac{52}{\sqrt{F_{ye}}}\text{, therefore;} \qquad m = 2.0$$

Columns in flexure with $P/P_{ye} < 0.20$;

Consider the most highly loaded column which is the interior first story column at the high roof area;

$P = [1.2(280 + 1{,}272)\text{plf} + 1.6(600\text{plf}) + 0.5(225\text{plf})]15' + 1.2(980 + 653)\text{lbs} = 46^k$ (204.6KN)
$P_{ye} = R_y F_y A_g = 1.1(50\text{ksi})10\text{-in}^2 = 550^k$ (2.47MN), with $R_y = 1.1$ per AISC seismic provisions sec. 6.2 for Gr. 50
$\therefore P/P_{ye} = 46^k/550^k = 0.084 < 0.2$

$F_{ye} = R_y F_y = 1.1(50\text{ksi}) = 55\text{ksi}$ (379.2MPa)

$$\frac{52}{\sqrt{F_{ye}}} = 7.01 < \left(\frac{b}{2t_f}\right)_{W14X34} = 7.4 < 12.8 = \frac{95}{\sqrt{F_{ye}}}\text{, therefore, by interpolation;} \qquad m = 1.93$$

Panel zones in shear; $m = 1.5$

Fully restrained moment connections;
For full penetration flange welds and bolted or welded web connections with no panel zone yielding;

$m = 1.0$

Braced Frames;

Columns in tension; $m = 1.0$

Braces in compression; $m = 0.8$

Braces in tension; $m = 1.0$

F-6 *Determine Q_{CE} for deformation-controlled components.*

Note: Q_{UD} for each component will be determined in step F-7.

Note: Per paragraph 6-3a(3)(a), Q_{CE} is defined as the nominal strength multiplied by 1.25.

W14X34 (W355.6mmX0.50KN/m) Column in flexure;

$Q_{CE} = 1.25M_n = 1.25(187^{\text{ft-kips}}) = 234^{\text{ft-kips}}$ (317.3KN-m)

where; M_n is calculated using AISC load factor design selection table as follows;
For an unbraced length of 11-ft (3.36m);

$$\phi_b M_n = C_b[\phi_b M_n - BF(L_b - L_p)]$$

$$\phi_b M_n = 1.0[205^{\text{ft-kips}} - 6.58(11'-5.4')] = 168^{\text{ft-kips}}(227.8\text{KN}-\text{m})$$

$$\therefore M_n = 168^{\text{ft-kips}} / 0.9 = 187^{\text{ft-kips}}(253.6\text{KN}-\text{m})$$

W14X34 (W355.6mmX0.50KN/m) Column in tension;

$Q_{CE} = 1.25P_n = 1.25A_gF_y = 1.25(10.0\text{-in}^2)50\text{ksi} = 625^k$ (2.78MN)

W14X34 (W355.6mmX0.50KN/m) Panel zone in shear;

$Q_{CE} = 1.25\ R_v$

Per AISC seismic provisions; $R_v = 0.6F_yd_ct_p\left[1 + \dfrac{3b_{cf}t_{cf}^2}{d_bd_ct_p}\right]$

where; $b_{cf} = 6.745"$, $d_b = 20.91"$, $d_c = 13.98"$, $t_p = 0.285"$, $t_{cf} = 0.455"$ (11.6mm)

$$\therefore R_v = 0.6(50\text{ksi})13.97"(0.285")\left[1 + \frac{3(6.745")(0.455")^2}{20.91"(13.98")0.285"}\right] = 126^k \ (560.4\text{KN})$$

$Q_{CE} = 1.25(126^k) = 158^k$ (702.8KN)

W14X26 (W355.6mm X 0.38KN/m) Beam in flexure;

$Q_{CE} = 1.25M_n = 1.25M_p = 1.25(119^{\text{ft-kips}}) = 149^{\text{ft-kips}}$ (202.0KN-m)

where; M_n is calculated using AISC LRFD 2nd ed. load factor design selection table as follows;
For an unbraced length of 5-ft (1.53m);

$$\phi_b M_n = C_b[\phi_b M_n - BF(L_b - L_p)]$$

$$\phi_b M_n = 1.0[109^{\text{ft-kips}} - 4.44(5'-4.5')] = 107^{\text{ft-kips}}(145.1\text{KN}-\text{m})$$

$$\therefore M_n = 107^{ft-kips} / 0.9 = 119^{ft-kips}(161.4KN - m)$$

TS 4X4X1/4 (TS101.6mmX101.6mmX6.35mm) Brace in compression;

$Q_{CE} = 1.25P_n = 1.25(22.4^k) = 28^k$ (124.5KN)

where; P_n is calculated using AISC LRFD 2nd ed. column load tables as follows;
From the tables with KL = 1.0(25') = 25' (7.63m)
$P_n = 19^k/0.85 = 22.4^k$ (99.6KN)

TS 4X4X1/4 (TS 101.6mmX101.6mmX6.35mm) Brace in tension;

$Q_{CE} = 1.25P_n = 1.25F_yA_g = 1.25(46ksi)3.59\text{-in}^2 = 206^k$ (916.3KN)

F-7 *Determine DCR's for deformation-controlled components and compare with allowable m values for Immediate Occupancy.*

Low Roof Structure;

Transverse direction;

Note: Risa-2D analysis was rerun using the scale factors applied to the seismic loading. In all cases the governing load combination was 4a; $1.314D + Q_E$..

$(Q_E)_{roof} = SF_{trans} x F_{roof} = 11.3 \times 0.95^k = 10.7^k$ (47.6KN)

W14X26 (W355.6mm X 0.38KN/m) Beam at plastic hinge location in flexure;

$(Q_{UD})_{worst\ case} = 84.96^{ft\text{-}kips}$ (115.2KN-m)

$$DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{84.96^{ft-kips}}{149^{ft-kips}} = 0.57 < 2.0 = m$$ **O.K.**

W14X34 (W355.6mm X 0.50KN/m) Column in flexure;

$(Q_{UD})_{worst\ case} = 106.14^{ft\text{-}kips}$ (143.9KN-m)

$$DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{106.71^{ft-kips}}{234^{ft-kips}} = 0.46 < 1.93 = m$$ **O.K.**

W14X34 (W355.6mmX0.50KN/m) Panel zone in shear;

$$Q_{UD} = \frac{M_u}{d_{beam} + d_{haunch} - t_f}$$

$$(Q_{UD})_{worst\ case} = \frac{99.63^{ft-kips}(12''/1')}{13.91''+7''-0.420''} = 58.2^{ft-kips} \text{ (78.9KN-m)}$$

$$DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{58.2^{ft-kips}}{158^{ft-kips}} = 0.37 < 1.5 = m$$ **O.K.**

Check Deflection;

$$\Delta_{allow} = 0.015h_{sx} = 0.015(15'(12''/1')) = 2.7'' \text{ (68.6mm)}$$

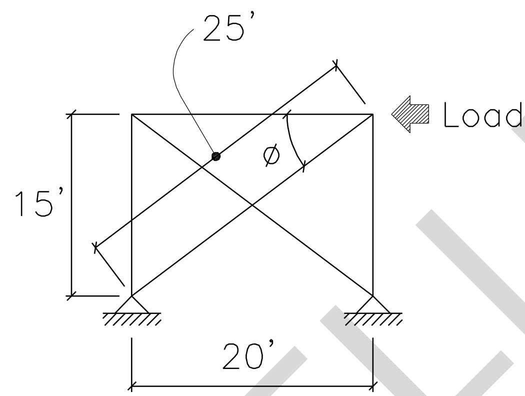
$$d_{calc} = 2.14'' < 2.7'' = \Delta_{allow} \quad (54.5mm < 68.6mm)$$ **O.K.**

Longitudinal direction;

$$(Q_E)_{roof} = SF_{long} x F_{roof} = 9.47(2.5^k) = 23.7^k \quad (105.4KN)$$

W14X34 (W355.6mmX0.50KN/m) Column in tension;

Note: Axial load in column is conservatively calcuated
Ignoring the gravity loads as follows;



1-ft = 0.30m

$$(Q_{UD})_{col\ axial} = \frac{23.7^k}{2(2)}(\sin f) = 5.93^k\left(\frac{15'}{25'}\right) = 3.56^k \quad (15.8KN)$$

$$DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{3.56^k}{625^k} = 0.006 < 1.0 = m$$ **O.K.**

TS 4X4X1/4 (TS 101.6mmX101.6mmX6.35mm) Braces in tension or compression;

$$(Q_{UD})_{brace\ axial} = \frac{23.7^k}{2(2)}\left(\frac{1}{\cos f}\right) = 5.93^k\left(\frac{25'}{20'}\right) = 7.41^k \quad (33.0KN)$$

Compression; $$\frac{Q_{UD}}{Q_{CE}} = \frac{7.41^k}{28^k} = 0.27 < 0.8 = m$$ **O.K.**

Tension; $$\frac{Q_{UD}}{Q_{CE}} = \frac{7.41^k}{206^k} = 0.04 < 1.0 = m$$ **O.K.**

High roof structure;

Transverse direction;

Note: Risa-2D analysis was rerun using the scale factors applied to the seismic loading. In all cases the governing load combination was 4a; $1.314D+Q_E+0.5L$.

For the moment frame without a truss;

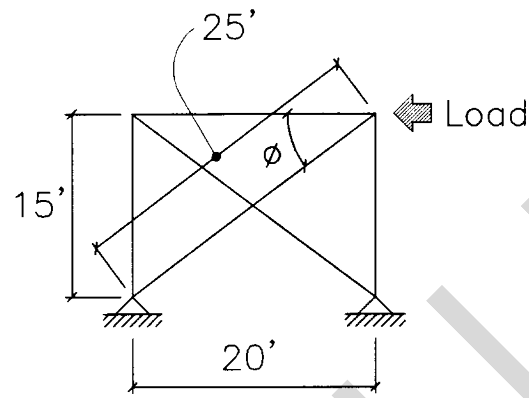
$\delta_{calc} = 2.14" < 2.7" = \Delta_{allow}$ (54.5mm < 68.6mm) **O.K.**

Longitudinal direction;

$(Q_E)_{roof} = SF_{long} x F_{roof} = 9.47(2.5^k) = 23.7^k$ (105.4KN)

W14X34 (W355.6mmX0.50KN/m) Column in tension;

Note: Axial load in column is conservatively calcuated
Ignoring the gravity loads as follows;



1-ft = 0.30m

$$(Q_{UD})_{col\ axial} = \frac{23.7^k}{2(2)}(\sin\phi) = 5.93^k\left(\frac{15'}{25'}\right) = 3.56^k \text{ (15.8KN)}$$

$$DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{3.56^k}{625^k} = 0.006 < 1.0 = m$$ **O.K.**

TS 4X4X1/4 (TS 101.6mmX101.6mmX6.35mm) Braces in tension or compression;

$$(Q_{UD})_{brace\ axial} = \frac{23.7^k}{2(2)}\left(\frac{1}{\cos\phi}\right) = 5.93^k\left(\frac{25'}{20'}\right) = 7.41^k \text{ (33.0KN)}$$

Compression; $\frac{Q_{UD}}{Q_{CE}} = \frac{7.41^k}{28^k} = 0.27 < 0.8 = m$ **O.K.**

Tension; $\frac{Q_{UD}}{Q_{CE}} = \frac{7.41^k}{206^k} = 0.04 < 1.0 = m$ **O.K.**

High roof structure;

Transverse direction;

Note: Risa-2D analysis was rerun using the scale factors applied to the seismic loading. In all cases the governing load combination was 4a; $1.314D+Q_E+0.5L$.

For the moment frame without a truss;

$(Q_E)_{roof} = SF_{trans} x F_{roof} = 11.3 \text{ x } 1.17^k = 13.2^k$ (58.7KN)

$(Q_E)_{floor} = SF_{trans} x F_{floor} = 11.3 \text{ x } 0.904^k = 10.2^k$ (45.4KN)

For the moment frames with a truss;

$(Q_E)_{roof} = SF_{trans} x F_{roof} = 11.3 \text{ x } 0.585^k = 6.42^k$ (28.6KN)

$(Q_E)_{floor} = SF_{trans} x F_{floor} = 11.3 \text{ x } 3.053^k = 34.3^k$ (152.6KN)

$(Q_E)_{floor} = SF_{trans} x F_{roof\ adj} = 11.3 \text{ x } 0.475^k = 5.37^k$ (23.9KN)

Note: In all of the following checks, the moment frame without a truss governed.

W14X26 (W355.6mmX0.38KN/m) Beam at plastic hinge location in flexure;

$(Q_{UD})_{worst\ case} = 187.19^{ft\text{-}kips}$ (253.8KN-m)

$$DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{187.19^{ft-kips}}{149^{ft-kips}} = 1.26 < 2.0 = m$$ **O.K.**

W14X34 (W355.6mmX0.50KN/m) Column in flexure;

$(Q_{UD})_{worst\ case} = 220.84^{ft\text{-}kips}$ (299.5KN-m)

$$DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{220.84^{ft-kips}}{234^{ft-kips}} = 0.94 < 1.93 = m$$ **O.K.**

W14X34 (W355.6mmX0.50KN/m) Panel zone in shear;

$$Q_{UD} = \frac{M_u}{d_{beam} + d_{haunch} - t_f}$$

$$(Q_{UD})_{worst\ case} = \frac{210.15^{ft-kips}}{13.91'' + 7'' - 0.420''} = 123^{ft-kips}$$ (166.8KN-m)

$$DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{123^{ft-kips}}{158^{ft-kips}} = 0.80 < 1.5 = m$$ **O.K.**

Check Deflection;

$$\Delta_{allow} = 0.015 h_{sx} = 0.015(11'(12''/1')) = 1.98''$$ (50.3mm)

$$d_{calc} = 6.375'' > 1.98'' = \Delta_{allow}$$ (161.9mm > 50.33mm) **N.G.**

Note: High roof moment frames will have to be redesigned.

Longitudinal direction;

$Q_E = SF_{long} x \{F_{roof} + F_{floor} + F_{torsion}\} = 9.47(1.575^k + 3.28^k + 0.323^k) = 9.47(5.18^k) = 49.0^k$ (218.0KN)

W14X34 (W355.6mmX0.50KN/m) Column in tension;

$$(Q_{UD})_{col\ axial} = \frac{49.0^k}{2}(\sin\phi) = \frac{49.0^k}{2}\left(\frac{11'}{18.6'}\right) = 14.5^k \ (64.5KN)$$

$$DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{14.5^k}{625^k} = 0.023 < 1.0 = m$$ **O.K.**

TS 4X4X1/4 (TS101.6mmX101.6mmX6.35mm) Braces in tension or compression;

$$(Q_{UD})_{brace\ axial} = \frac{49.0^k}{2}\left(\frac{1}{\cos\phi}\right) = \frac{49.0^k}{2}\left(\frac{18.6'}{15'}\right) = 30.4^k \ (135.2KN)$$

Compression; $$\frac{Q_{UD}}{Q_{CE}} = \frac{30.4^k}{28^k} = 1.09 < 0.8 = m$$ **N.G.**

Note: Braces will have to be redesigned.

Tension; $$\frac{Q_{UD}}{Q_{CE}} = \frac{30.4^k}{206^k} = 0.15 < 1.0 = m$$ **O.K.**

F-8 *Determine QUF and QCL for force-controlled components and compare QUF with QCL.*

Note: Q_{CL} contains the appropriate strength reduction factor per paragraph 6-3a(3)(b).

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1C_2C_3J} \qquad \text{(EQ. 6-4a)}$$

where; $J = 1.0 + S_{DS} \leq 2.0$ (EQ. 6-5)

$= 1.0+0.57 = 1.57 < 2.0$

$C_{1,trans} = 1.26$, $C_{1,long} = 1.41$ (as previously calculated)

$C_2 = C_3 = 1.0$ (as previously calculated)

Therefore, the scale factor for Q_E is;

Transverse; $$SF_{trans} = \frac{1}{C_1C_2C_3J} = \frac{1}{1.26(1.0)1.0(1.57)} = 0.51$$

Longitudinal; $$SF_{long} = \frac{1}{C_1C_2C_3J} = \frac{1}{1.41(1.0)1.0(1.57)} = 0.45$$

Chord/Collector elements;

Worst case condition is used.

Q_E;

Maximum Chord Force = 234-lb (1.04KN)

Maximum Collector Force = 788-lb (3.51KN) (governs)

Q_G;

$w_u = 1.314D = 1.314(2.5')18.7psf + 0.5(50plf) = 86.4plf$ (1.26KN/m)

$M_u = w_uL^2/8 = 86.4plf(15')^2/8 = 2.43^{ft\text{-}kips}$ (3.30KN/m)

$\therefore Q_{UF} = SF_{long}(Q_E) = 0.45(788\text{-lb}) = 355\text{-lb}$ (1.58KN) compressive seismic axial load with $2.43^{ft\text{-}kips}$ (3.30KN-m) in flexure due to gravity loads.

Determine Q_{CL};

$(Q_{CL})_{flexure} = \phi_b M_n = 47^{ft-kips}$ (63.7KN-m) per AISC LRFD 2nd ed. load factor design selection table for a beam with continuous lateral support of the compression flange.

$(Q_{CL})_{compression} = \phi_c P_n = 118^k$ (0.52KN) using AISC LRFD 2nd ed. table 3-36 as follows;

$$\frac{KL}{r} = \frac{1.0(15')(12''/1')}{4.62''} = 39 \Rightarrow \phi_c F_{cr} = 28.25\text{ksi}\ (194.4\text{MPa})$$

$$\phi_c P_n = \phi_c F_{cr} A_g = 28.25\text{ksi}(4.16 - \text{in}^2) = 118^k\ (524.9\text{KN})$$

Check interaction equation;

$$\frac{Q_{UF}}{Q_{CL}} \geq 1.0$$

$$\frac{P_u}{\phi_c P_n} = \frac{0.355^k}{118^k} = 0.003 < 0.2$$

Therefore, use AISC LRFD equation H1-1b

$$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$$

(EQ. H1-1b AISC LRFD)

$$\frac{0.355^k}{2(118^k)} + \left(\frac{2.43^{ft-kips}}{47'^k} + 0\right) = 0.053 < 1.0$$

O.K.

Note: By inspection, the collector element at the low roof is acceptable.

Braces in moment frame with a truss;

The Risa-2D analysis for this frame was rerun with the scale factor SF_{trans} = 0.51 applied to the seismic loading. The following load combination governed;

$$Q_{UF} = Q_G + \frac{Q_E}{C_1 C_2 C_3 J} = 1.314D + 0.5L + 0.51E$$

The governing loads are as follows;

$(P_u)_{max} = 29.50^k$ (131.2KN) Tension

$(P_u)_{max} = 7.10^k$ (31.6KN) Compression

Determine Q_{CL};

$(Q_{CL})_{tension} = \phi_t A_g F_y = 0.9(3.59 - \text{in}^2)46\text{ksi} = 149^k$ (662.8KN)

$(Q_{CL})_{compression} = 103.5^k$ (460.4KN) per AISC LRFD 2nd ed. column tables using KL = 8.5' (2.60m) and $\phi_c = 0.85$.

Tension; $\frac{Q_{UF}}{Q_{CL}} = \frac{29.5^k}{149^k} = 0.20 < 1.0$ **O.K.**

Compresion; $\frac{Q_{UF}}{Q_{CL}} = \frac{7.10^k}{103.5^k} = 0.07 < 1.0$ **O.K.**

F-9 *Revise member sizes as necessary and repeat analysis.*

Moment frames of the two-story structure (high roof) do not meet deflection requirements and must be resized. The beam size will be chosen first and then the column size will be chosen to comply with the strong column/weak beam requirements of the AISC seismic provisions. The beam size (required moment of inertia 'I_x') is scaled up in inverse proportion to the drift as follows;

$$\frac{I_{req'd}}{I_{W14X26}} = \frac{\delta_{calc}}{\Delta_{allow}} = \frac{6.375"}{1.98"} = 3.22\text{, Say } 3.5$$

$$\therefore (I_{req'd})_{beam} \geq 3.5(I_{W14X26}) = 3.5(245-\text{in}^4) = 858-\text{in}^4\ (359.1X10^6\ \text{mm}^4)$$

Check compact section criteria (per AISC seismic provisions):
Try W14X82 (W355.6mm X 1.20KN/m);

$$\frac{b_f}{2t_f} = 5.9 < 8.7 = \frac{52}{\sqrt{36\text{ksi}}} = \frac{52}{\sqrt{F_y}} \qquad \textbf{O.K.}$$

Choose W14X82 (W355.6mm X 1.20KN/m) I = 882-in^4 (367.1X10^6 mm^4) > 858-in^4 (357.1X10^6 mm^4), Z = 139-in^3 (2.28X10^6 mm^3)

Determine column size;

Assume a W14 section for the column providing a column depth 'd_c' approximately equal to the beam depth 'd_b'. Therefore, the location of the plastic hinge from column centerline 'x' is calculated as follows;

$$x = \frac{d_c}{2} + \frac{3d_b}{4} + \frac{d_b}{3} \approx 1.6d_b = 1.6(14.31") = 22.9"\ (581.7\text{mm}), \qquad \text{Say 24" or 2' (0.6m)}$$

$$\frac{\sum M^*_{pc}}{\sum M^*_{pb}} \geq 1.0 \qquad \text{(EQ. 9-3 AISC Seismic Provisions)}$$

$$\sum M^*_{pb} = \sum(1.1R_yM_p + M_v)$$

where; $R_y = 1.5$

M_p = Plastic Moment = ZF_y = 139-in^3(36ksi) = 5,004$^{\text{in-kips}}$ (565.5KN-m)

M_v;

$$V_p = \frac{2M_p}{(L-2x)} = \frac{2(5{,}004^{\text{in-kips}})}{(30'-2(2'))(12"/1')} = 32.1^k\ (142.8\text{KN})$$

$$\therefore M_v = V_p(2') = 32.1^k(24") = 770^{\text{in-kips}}\ (87.0\text{KN-m})$$

Therefore;

$$\sum M^*_{pb} = 1.1(1.5)5{,}004^{\text{in-kips}} + 770^{\text{in-kips}} = 9{,}027^{\text{in-kips}}\ (1.02\text{MN-m})$$

and;

$$\sum M^*_{pc} = \sum Z_c(F_{yc} - P_{uc}/A_g)$$

where; $P_{uc} \approx 1.2(20\text{psf})15'(15')(1^k/1000^{lb}) = 5.4^k$ (24.0KN)
$A_g \approx$ 10-in^2 (6.45X10^3 mm^2) (assumed)
$\therefore P_{uc}/A_g \approx 0.5$ksi (3.44MPa)

Therefore;

$$\sum M^*_{pc} = Z_{req'd}(50\text{ksi} - 0.5\text{ksi}) = 49.5Z_{req'd}$$

and; $$\frac{\sum M_{pc}^{*}}{\sum M_{pb}^{*}} \Rightarrow Z_{req'd} \geq \frac{9{,}027^{"k}}{49.5\text{ksi}} = 182 - \text{in}^3 \ (2.98\text{X}10^6 \text{ mm}^3)$$

Check compact section criteria (per AISC seismic provisions):
Try W14X132 (W355.6mmX1.93KN/m);

Web local buckling;

$$P_y = A_g F_y = 32.0 - \text{in}^2(50\text{ksi}) = 1{,}600^k \ (7.12\text{MN})$$

$$\frac{P_u}{\phi_b P_y} = \frac{38.3^k}{0.9(1{,}600^k)} = 0.027 < 0.125$$

$$\therefore \lambda_p = \frac{520}{\sqrt{F_y}}\left[1 - 1.54\frac{P_u}{\phi_b P_y}\right] = \frac{520}{\sqrt{50\text{ksi}}}\left[1 - 1.54\frac{38.3^k}{0.9(1{,}600^k)}\right] = 70.5$$

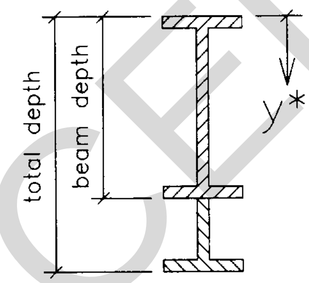
$$\lambda_{W14X132} = \frac{h}{t_w} = 17.7 < 70.5 = \lambda_p$$ **O.K.**

Flange local buckling;

$$\lambda_{W14X132} = \frac{b_f}{2t_f} = 7.1 < 7.35 = \frac{52}{\sqrt{50\text{ksi}}} = \frac{52}{\sqrt{F_y}} = \lambda_p$$ **O.K.**

Choose; W14X132 (W355.6mm X 1.93KN/m), Z=234-in^3 (3.83X10^6 mm^3), I = 1530-in^4 (636.8X10^{6}mm^4)

Haunch properties are calculated on spread sheet as follows;



Beam = W14X82

Beam I (in) =	882.00
Beam A (in^2) =	24.10
Beam depth (in) =	14.31
Flange Thickness (in) =	0.855
Web Thickness (in) =	0.510
Flange Width (in) =	10.130
Total Depth (in) =	21.31
Haunch Depth (in) =	7.00

y* (in) =	11.36
Haunch I_x (in^4) =	2218
Total A (in^2) =	35.90
Haunch I_y (in^4) =	222.2
Haunch S_x (in^3) =	195
Haunch Z_x (in^3) =	177.16

1-in = 25.4mm
1-in^2 = 645.2 mm^2
1-in^3 = 16.37X10^3 mm^3
1-in^4 = 416.2X10^3 mm^4

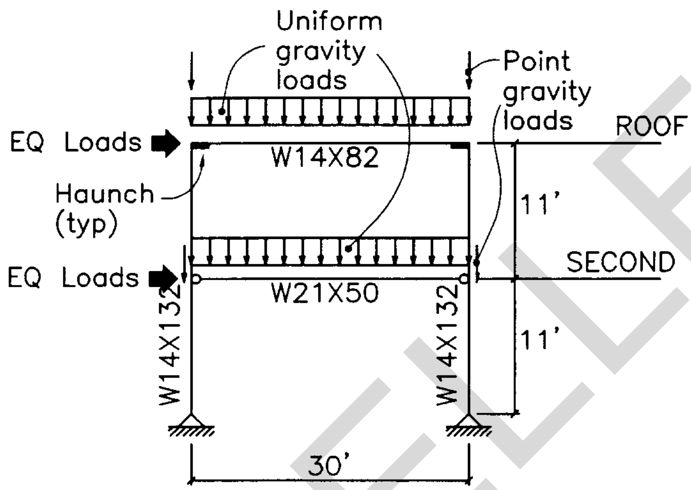
This frame was analyzed using RISA-2D. All load combinations were investigated to determine the worst case deflection. The controlling load combination was equation 4a; 1.314D+Q_E+0.5L. Because the members of the frame have changed, a check to ensure the location of the plastic hinge was made. The following frame and loading was analyzed;

Design Loads:
Roof:

w_{LR} = 225plf (3.28KN/m)
w_{DR} = 280plf (4.08KN/m)
P_{RD} = 974-lb (4.33KN)
E_R = 13.2^k (58.7KN) (applied as a uniform load of 13.2^k/30' = 440plf (6.42KN/m) along the beam length)

Floor:

w_{LF} = 600plf (8.75KN/m)
w_{DF} = 1,272plf (18.55KN/m)
P_{FD} = 649-lb (2.89KN)
E_R = 10.2^k (45.4KN) (applied as a uniform load of 10.2k/30' = 340plf (4.96KN/m) along the beam length)



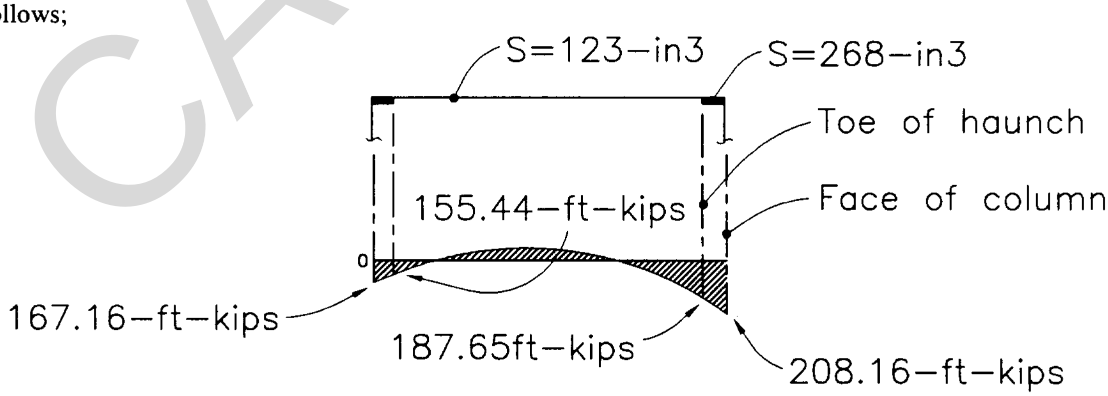
1-in = 25.4mm
1-ft = 0.3m
1plf = 14.58N/m

Drift requirements;

Calculated drift; δ_{calc} = 1.539" (39.1mm)

Therefore; δ_{calc} = 1.539" < Δ_{allow} = 1.98" (39.1mm < 50.3mm) **O.K.**

Check plastic hinge location;
The resulting moment diagram, showing moments at the face of column and at the toe of the haunch, is as follows;



1-in^3 = 16.4X10^3 mm^3
1-ft-kip = 1.356KN-m

By inspection of this diagram, it is clear that a plastic hinge will form on the right side of the beam where the moments are greatest (with increased loading, the moments will increase proportionately until yielding occurs).

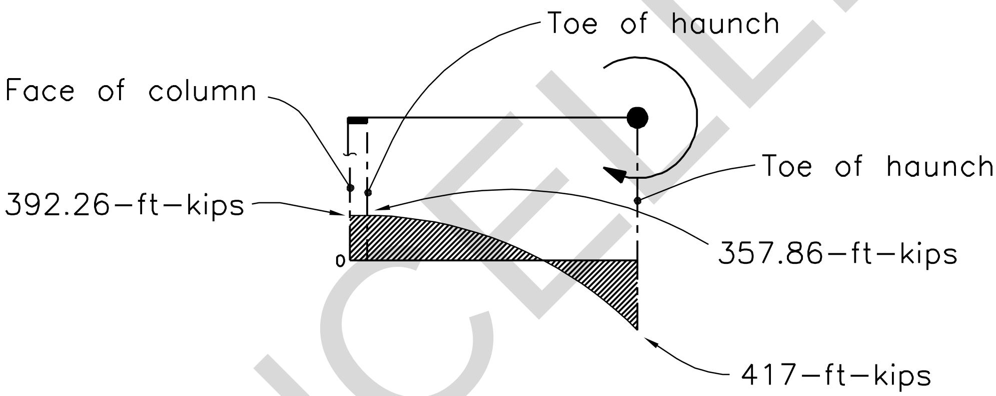
The stress ratio on the right side is;

$$\frac{s_{\text{toe-of-haunch}}}{s_{\text{face-of-column}}} = \frac{M_{\text{toe}} / S_{x,\text{toe}}}{M_{\text{face}} / S_{x,\text{haunch}}} = \frac{187.65^{\text{ft-kips}}(12''/1')/123-\text{in}^3}{208.16^{\text{ft-kips}}(12''/1')/268-\text{in}^3} = 1.96 > 1.2 \qquad \textbf{O.K.}$$

The left side was investigated by placing a plastic hinge at the assumed hinge location on the right side and analyzing the resulting configuration. The lateral load was increased until yielding occurred at the toe of the haunch on the left side.

$$M_p = Z_x F_y = 139-\text{in}^3(36\text{ksi}) = 417^{\text{ft-kips}} \text{ (565.5KN-m)}$$
$$M_y = S_x F_y = 123-\text{in}^3(36\text{ksi}) = 369^{\text{ft-kips}} \text{ (500.4KN-m)}$$

The resulting moment diagram, showing moments at the face of column and at the toe of the haunch, is as follows;



1-ft-kip = 1.356KN-m

The resulting stress ratio on the left side is;

$$\frac{s_{\text{toe-of-haunch}}}{s_{\text{face-of-column}}} = \frac{M_{\text{toe}} / S_{x,\text{toe}}}{M_{\text{face}} / S_{x,\text{haunch}}} = \frac{357.86^{\text{ft-kips}}(12''/1')/123-\text{in}^3}{392.26^{\text{ft-kips}}(12''/1')/268-\text{in}^3} = 1.99 > 1.2 \qquad \textbf{O.K.}$$

Check unbraced length of the compression flanges (per AISC seismic requirements);

Try 5' on center (same spacing as the perpendicular floor joists);

$$L_b = 5' < 14.4' = \frac{2500(2.48'')(1'/12'')}{36\text{ksi}} = \frac{2500 r_y}{F_y} \quad (1.53\text{m} < 4.39\text{m}) \qquad \textbf{O.K.}$$

Note: By inspection, the high roof moment frame with a truss will be acceptable.

Braces:
Try TS 4.5x4.5x1/4 (TS 114.3mmX114.3mmX6.4mm)
Check DCR in compression;
$Q_{CE} = 1.25P_n = 1.25(34.1^k) = 42.6^k$ (189.5KN)

where; P_n is calculated using AISC LRFD 2nd ed. column load tables as follows;
From the tables with KL = 1.0(25’) = 25’ (7.62m)
$P_n = 29^k/0.85 = 34.1^k$ (151.7KN)

Check DCR ratio in compression;

$$\frac{Q_{UD}}{Q_{CE}} = \frac{30.4^k}{42.6^k} = 0.71 < 0.8 = m$$ **O.K.**

Use TS 4.5x4.5x1/4 (TS 114.3mmX114.3mmX6.4mm)

e. Design connections.

General;

Governing design loads can be from either of ground motions A or B. Ground motion A consists of two thirds of the Maximum Considered Earthquake (MCE), includes R values, and was used in performance objective 1A. Ground motion B consists of three-quarters of the MCE, does not include R-values, and was used in performance objective 3B. Further, for force controlled components, the seismic load from ground motion B is divided by the modification factors C_1, C_2, C_3, and J to restore it to a force-controlled action. However, some elements of the connections may derive their design loads from the expected strength of other connection elements. For example, panel zone shear used to determine the need for doubler plates is limited by expected strength of the beams framing to the column flanges, and the design load for continuity plates is taken from the expected strength of the beam flanges. Additionally, brace frame connections may be designed for the expected strength of the brace.

Two example connection designs will be shown; a moment connection and a braced frame connection. Both connections are from the low roof portion of the building.

Low Roof;

Transverse direction;

Moment frames;

In designing the main elements of the moment frame (i.e., the beams and columns) most of the requirements of the AISC seismic provisions for moment connections had been met. What remains is to determine the need for doubler plates, the need for continuity plates, and the design of the bolted gravity load connection at the beam web.

Determine if doubler plates are required;

Determine Demand;

Note: Loading from performance objective 1A including the structural overstrength factor ($\Omega_\circ = 3$) could have been used to determine a smaller demand ‘R_u’. However, it is more convenient to apply the expected strength requirement of 0.8 times $\sum R_y M_p$ as follows;

$$R_u = \frac{0.8\sum R_y M_p}{d_b + d_{haunch} - t_f}$$ (per AISC seismic provisions dated April 15, 1997)

where; $R_y = 1.5$ (for ASTM A36)
$M_p = Z_x F_y = 43.24\text{-in}^3(36\text{ksi}) = 1{,}557^{\text{in-kips}}$ (175.9KN-m)
$d_b = 13.91”$ (353.3mm)

d_{haunch} = 7.00” (177.8mm)
t_f = 0.420” (10.7mm)

$$\therefore R_u = \frac{0.8(1.5(1,557^{"k}))}{13.91"+7.00"-0.42"} = 91.1^k \quad (405.2KN)$$

Determine Capacity;

$$(R_v)_{W14X34} = 158^k \ (702.8KN)$$ (calculated previously)

Check;

$$f_v R_v = 0.75(158^k) = 119^k > 91.1^k = R_u \ (529.3KN > 405.2KN)$$ **O.K.**

Check panel-zone thickness;

$$t \geq (d_z + w_z)/90$$ (EQ. 9-2 AISC Seismic provisions)

where; d_z = panel zone depth between continuity plates (which includes haunch depth)
d_z = 13.91”+7.00”-2(0.42”) = 20.1” (510.5mm)
w_z = d_c-2t_f = 13.98”-2(0.455”) = 13.1” (332.7mm)

$$\therefore t = 0.285" < 0.369" = \frac{20.1"+13.1"}{90} = \frac{d_z + w_z}{90} \quad (7.24mm < 9.37mm)$$ **N.G.**

Try a 3/8” (9.53mm) thick doubler plate (using ASTM A36);

Provide weld to match shear strength of the required thickness of doubler plate (use E70XX electrode);

Shear capacity of doubler plate = $f F_{BM} A_g = f F_{BM} t_d b_d$

where; F_{BM} = nominal shear capacity of base metal = 0.6F_y
A_g = gross area of the doubler plate
t_d = thickness of doubler plate
b_d = width of doubler plate

Shear capacity of welds = $f F_w (0.707s) b_d$

Where; F_W = nominal shear capacity of welds = 0.6F_{EXX}
s = weld leg length

Therefore; $f F_{BM} t_d b_d = f F_W (0.707s) b_d$

or $$s = \frac{f F_{BM} t_d}{0.707(f F_W)} = \frac{0.9(0.6(36ksi))0.369"}{0.707(31.5ksi)} = 0.322" \quad (8.18mm)$$

From AISC J2.2b, the maximum weld size is;

$$s = t_d - \frac{1"}{16} = \frac{3"}{8} - \frac{1"}{16} = 0.313" \ (7.95mm)$$ **N.G.**

Try a 1/2” (12.7mm) thick doubler plate with a 3/8” (9.53mm) weld;

For a 1/2” (12.7mm) thick doubler plate; s = 0.438” (11.13mm) **O.K.**

From AISC table J2.4, the minimum weld size is; s = 3/16” < 3/8” (4.76mm < 9.53mm) **O.K.**

Choose 1/2" (12.7mm) doubler plate secured with a 3/8" (9.53mm) fillet weld min

Determine if continuity plates are required;

Determine demand;

$$P_{bf} = A_f F_{ye} = A_f R_y F_y = 0.420''(5.025'')1.5(36ksi) = 114^k \quad (507.1KN)$$

Determine capacity;

$$\phi R_n = \phi[(2.5k + N)F_{yw} t_w + A_{st} F_{yst}]$$

where; $k = 1.0"$ (25.4mm) — distance from outer surface to toe of fillet for the column

$N = t_{bf} = 0.420"$ (10.67mm) — thickness of the beam flange

$F_{yw} = 50ksi$ (344.8MPa) — yield strength of the column web

$t_w = 0.285"$ (7.24mm) — thickness of column web

$A_{st} =$ — area of the stiffner

$F_{yst} = 36ksi$ (248.2MPa) — yield strength of the stiffener

$\phi = 1.0$

$$A_{st} = \frac{P_{bf} - (2.5k + N)F_{yw} t_w}{F_{yst}} = \frac{114^k - (2.5(1.0'') + 0.420'')50ksi(0.285'')}{36ksi} = 2.01 - in^2 > 0$$

($A_{st} = 1.30X10^3\ mm^2$)

Therefore, stiffeners with a total area of at least 2.01-in^2 (1.30X10^3 mm^2) total are required.

Design stiffeners in accordance with AISC LRFD 2nd ed. section K.9;

$$b_{st} + \frac{t_{cw}}{2} \geq \frac{b_b}{3} \Rightarrow b_{st} \geq \frac{b_b}{3} - \frac{t_{cw}}{2}$$

where; $b_{st} =$ width of a single stiffener

$b_b =$ width of the beam flange

$t_{cw} =$ thickness of the column web

$$b_{st} = \frac{b_b}{2} - \frac{t_{cw}}{2} = \frac{5.025''}{2} - \frac{0.285''}{2} = 2.37''$$ (60.2mm), Say two stiffeners with $b_{st} = 3.0"$ (76.2mm)

$$t_{st} \geq \frac{t_{bf}}{2}$$

where; $t_{st} =$ thickness of a single stiffener

$t_{bf} =$ thickness of the beam flange

use; $t_{st} = 0.375" > 0.210" = \frac{0.420''}{2} = \frac{t_b}{2}$, $(A_{st})_{total} = 0.375"(3.0")2 = 2.25\text{-}in^2 > 2.01\text{-}in^2$ **O.K.**

($t_{st} = 9.53mm > 5.33mm$) (A_{st})total = 1.45X10^3 mm^2 > 1.30X10^3 mm^2)

Check local buckling of the stiffeners;

$$\frac{b_{st}}{t_{st}} = \frac{3.0''}{0.375''} = 8.0 < 15.83 = \frac{95}{\sqrt{36ksi}} = \frac{95}{\sqrt{F_{y,st}}}$$ **O.K.**

where; $F_{y,st}$ = yield strength of the stiffener

Design welds for the stiffeners;

Stiffener to column web;

Minimum weld size = 3/16" (4.76mm) (per AISC LRFD 2nd ed. table J2.4, based on stiffener thickness)

Size required for strength;

Force to be resisted by a stiffener is;

$$F = P_{bf} - (2.5k + N)F_{yw}t_w = 114^k - (2.5(1.0") + 0.420")50ksi(0.285") = 72.4^k \quad \text{with } N = t_{bf}$$

(F = 322.0KN)

Length available for welding stiffeners to the column web is;

L = 12” (assuming a 1/2” chamfer) x 2 sides x 2 stiffeners = 48” (1.22m)

The required weld size is;

Note; For E70XX electrodes; $\phi F_w = 0.75[0.60(70ksi)] = 31.5ksi$ (265.0MPa)

$$s = \frac{R_u}{(0.707)L(\phi F_w)} = \frac{72.4^k}{0.707(48")31.5ksi} = 0.068" < \frac{3"}{16} \quad (1.72mm < 4.76mm)$$

Therefore, the minimum weld size governs

Check shear strength of the base metal;

$$\phi R_n = \phi(0.6t_{st}F_{st})x2 = 0.9(0.6)0.375"(36ksi)2 = 14.6 - kips/in \quad (2.56KN/mm)$$

$$\phi R_n = 14.6 - kips/in > 3.02 - kips/in = \frac{72.4^k}{48"/2} = \frac{F}{L/2} \quad (2.56KN/m > 0.53KN/m)$$ **O.K.**

Stiffener to column flange; Use full penetration groove welds

Choose two 3/8” (9.53mm) thick by 3.0” (76.2mm) wide stiffeners with a 3/16” (4.76mm) weld at the column web and a full pen-etration groove weld at the column flange

Design the single-plate web connection;

Note: The governing load combination ($1.2D+0.5L+1.6L_r$) is based solely on gravity loads.

$w_u = 1.2D+0.5L+1.6L_r = 1.2(343plf)+0.5(0)+1.6(240plf) = 796plf$ (11.6KN/m)

$$V_u = \frac{w_u(L - d_c)}{2} = \frac{796plf(30' - 13.98"(1'/12"))}{2} = 11.5^k \quad (51.2KN)$$

Try a 3/8” (9.53mm) plate;

Determine the number of 3/4” (19.05mm) diameter A325-N bolts required for shear;

From AISC LRFD 2nd ed. table 8-11;

$$n_{min} = \frac{R_u}{\phi r_n} = \frac{11.5^k}{15.9 - kips/bolt} = 0.72 \text{ bolts, Say 2 bolts}$$

Determine the number of 3/4” (19.05mm) diameter A325-N bolts required for bearing, assuming L_e = 1-1/2” (38.1mm), and s = 3” (76.2mm). The .255” (6.48mm) beam web is more critical then the 3/8” (9.53mm).

From AISC LRFD 2nd ed. Table 8-13;

$$n_{min} = \frac{R_u}{\phi r_n} = \left(\frac{11.5^k}{78.3 - \text{kips/bolt} - \text{in}}\right)\frac{1}{0.255"} = 0.58 \text{ bolts} < 2 \text{ bolts} \qquad \textbf{O.K.}$$

Check shear yielding of the plate;

$$\phi R_n = 0.9(0.6F_y A_g) = 0.9[0.6(36\text{ksi})6"(0.375")] = 43.7^k > 11.5^k \qquad \textbf{O.K.}$$

(193.4KN > 51.2KN)

Check shear rupture of the plate;

$$\phi R_n = 0.75(0.6F_u A_n) = 0.75\left[0.6(58\text{ksi})\left(6"-2\left(\frac{3"}{4}+\frac{1"}{8}\right)\right)\frac{3"}{8}\right] = 41.6^k > 11.5^k \qquad \textbf{O.K.}$$

(185.0KN > 51.2KN)

Check block shear rupture of the plate;

From AISC LRFD 2nd ed. table 8-47a, using L_{eh} = 1.5" (38.1mm);

$$\frac{\phi[F_u A_{nt}]}{t} = 46.2 - \text{kips/in} \quad (8.09\text{KN/m})$$

From AISC LRFD 2nd ed. table 8-48a, using L_{ev} = 1.5" (38.1mm);

$$\frac{\phi[0.6F_u A_{nv}]}{t} = 83 - \text{kips/in} \quad (14.53\text{KN/mm})$$

Therefore, $0.6F_uA_{nv} > F_uA_{nt}$. Thus, from AISC LRFD tables 8-48a and 8-48b;

$$\phi R_n = \phi[0.6F_u A_{nv} + F_y A_{gt}] = (83 - \text{kips/in} + 40.5 - \text{kips/in})0.375" = 46.3^k > 11.5^k \qquad \textbf{O.K.}$$

(205.9KN > 51.2KN)

Determine required weld size for fillet welds to supporting column flange;

From AISC LRFD 2nd ed. table J2.4, since the column flange thickness t_f = 0.455" > 1/4" (11.56mm > 6.35mm). The minimum fillet weld size is 3/16" (4.76mm). The length available for welding the plate to the column flange is;

L = 6" x 2 sides = 12" (304.8mm)

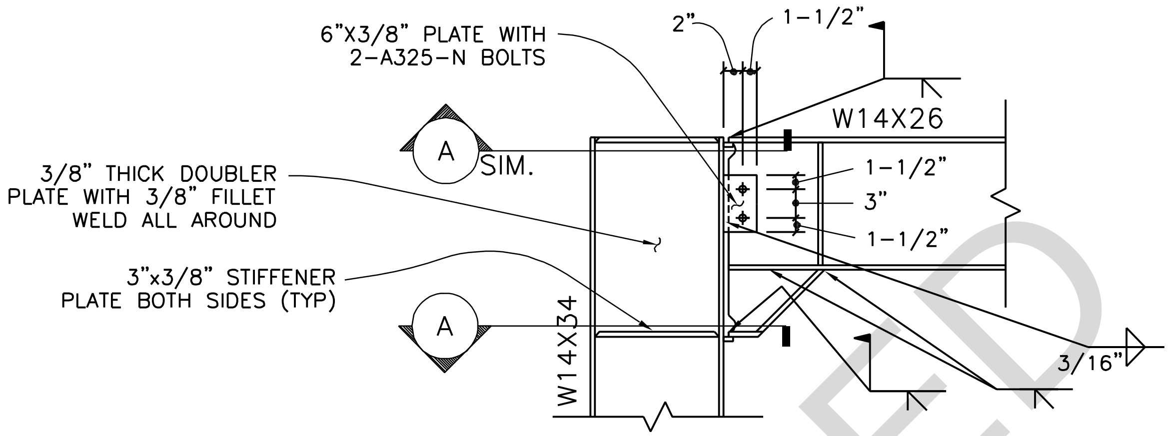
The required weld size 's' is;

$$s = \frac{F}{(0.707)L(\phi F_W)} = \frac{11.5^k}{0.707(12")31.5\text{ksi}} = 0.043" < \frac{3"}{16} \quad (1.09\text{mm} < 4.76\text{mm})$$

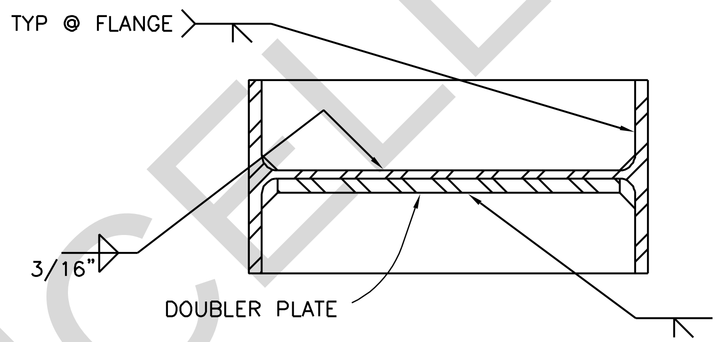
Therefore, the minimum weld size governs.

Choose 6"x3/8" (152.4mmx9.53mm) plate with two 3/16" (4.76mm) welds to the column flange and two 3/4" (19.05mm) A325-N bolts at the beam web

The final design of the low roof moment connection is as follows;



MOMENT FRAME DETAIL FOR LOW ROOF



A CONTINUITY PLATE DETAIL

1-in = 25.4mm
1plf = 14.58n/m

Longitudinal direction;

Braced frames;

A typical brace connection located at the foot of the column will be designed;

Determine demand;

It is decided to design the brace connection using the maximum load that can be delivered to it by the brace;

Tension; $R_u = R_yF_yA_g = 1.5(46ksi)4.0\ 9\text{-}in^2 = 282^k$ (1.25MN)

Compression; $R_u = F_{cr}A_g = \dfrac{\phi_c P_n}{\phi_c} = \dfrac{29^k}{0.85} = 34.1^k$ (151.7MN)

Try a 3/8" (9.53mm) thick gusset plate;

Design welds to gusset plate; Use E70XX electrodes and ASTM A36 for the gusset plate

Minimum weld size = 3/16" (4.76mm)	(per AISC LRFD 2nd ed. table J2.4, based on gusset plate thickness)
Maximum weld size = 3/16" (4.76mm)	(per AISC LRFD 2nd ed. J2.2b.(b), based on brace thickness)

Required length of four 3/16" (4.76mm) welds;

$$l_w = \frac{R_u}{4(0.707)s(\phi F_w)} = \frac{282^k}{4(0.707)0.188''(31.5\text{ksi})} = 16.8 - \text{in} \qquad \text{Say 17-in (431.8mm)}$$

Check base metal;

$$\phi R_n = \phi(0.6F_y)A_g = 0.9(0.6(46\text{ksi})4(17'')0.25'' = 373^k > 282^k \qquad \textbf{O.K.}$$

(1.66MN > 1.25MN)

Check shear/tension rupture;

$$\phi R_n = \phi t\left(0.6F_y\left(\frac{2l_w}{\cos 30^\circ}\right) + F_u w_1\right)$$

where; $w_1 = 4.5'' + 2l_w(\tan 30^\circ) = 4.5'' + 2(17'')0.577 = 24.1''$ (612.1mm)

$$\therefore \phi R_n = 0.75(0.375'')\left(0.6(36\text{ksi})\left(\frac{2(17'')}{0.866}\right) + 58\text{ksi}(24.1'')\right) = 632^k < 282^k \qquad \textbf{O.K.}$$

(2.81MN < 1.25MN)

or, $\phi R_n = \phi t\left(0.6F_u(2l_w) + F_y w_b\right) =$

$$\phi R_n = 0.75(0.375'')(0.6(58\text{ksi})2(17'') + 36\text{ksi}(4.5'')) = 378^k > 282^k \qquad \textbf{O.K.}$$

(1.68MN > 1.25MN)

Check tensile capacity;

$$\phi R_n = \phi w_1 F_u t = 0.75(24.1'')58\text{ksi}(0.375'') = 348^k > 282^k \qquad \textbf{O.K.}$$

(1.55MN > 1.25MN)

Check compressive capacity;

$$\phi R_n = \phi F_y w_1 t = 0.9(36\text{ksi})24.1''(0.375'') = 293^k > 34.1^k \qquad \textbf{O.K.}$$

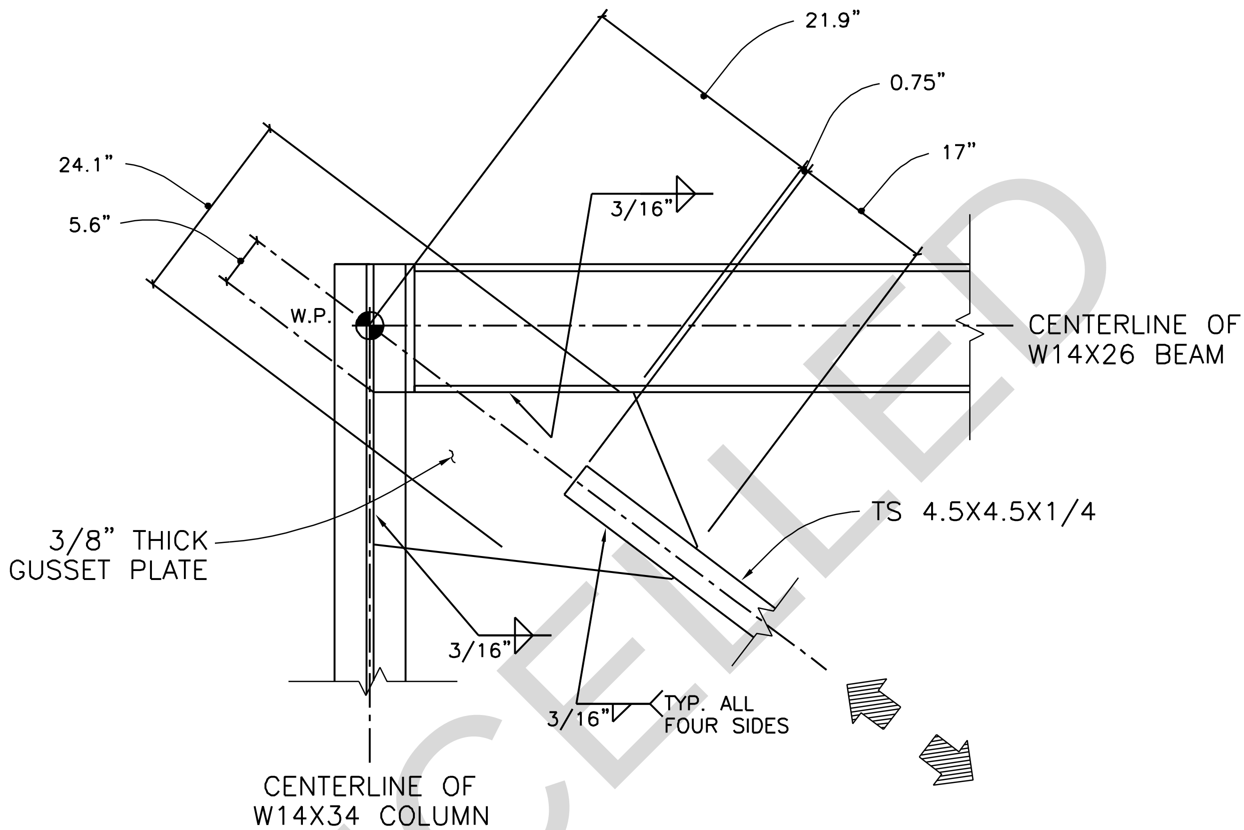
(1.30MN > 0.15MN)

Check buckling of the gusset plate;

$$\phi R_n = \phi\left(\frac{4000t^3\sqrt{F_y}}{l_1}\right) = 51.9^k > 34.1^k \qquad (230.9\text{KN} > 157.7\text{KN}) \qquad \textbf{O.K.}$$

Choose four 3/16" x 17" (4.76mmx431.8mm) long fillet welds with a 3/8" (9.53mm) thick gusset plate

The final design of the low roof brace frame connection at the roof is as follows;



1-in = 25.4mm
1plf = 14.58N/m

APPENDIX I

I-1 SUSPENDED CEILING BRACING

a. Introduction.

(1) Purpose. The purpose of this example problem is to illustrate the design of suspended ceiling bracing using Chapter 10 of this manual, and Chapter 6 of FEMA 302 (Components). Suspended ceiling systems without adequate lateral bracing have collapsed in a number of earthquakes causing injury to building occupants and disruption of safe egress and building function.

(2) Scope. The problem follows the steps in Tables 4.5 and 4.6 to analyze the ceiling bracing and anchors. Typical ceiling bracing details found in Figure 10-4 are inadequate for immediate occupancy in high seismic zones.

b. Component description.

The ceiling used in this example problem is suspended from a roof framing system consisting of steel joists and metal decking. The ceiling and equipment laterally supported by the ceiling, such as light fixtures and HVAC registers, are required to function after an earthquake.

c. Component design.

A.1 *Determine appropriate Seismic Use Group*

Due to the requirement that the ceiling and associated equipment must be functional after an earthquake, the ceiling and attachments are given a performance level of immediate occupancy (IO). The Seismic Use Group and other performance parameters are determined from Table 4-4, as follows;

Performance Level:	IO	(per problem statement)
Seismic Use Group:	IIIE	(Table 4-4)
Ground Motion:	3/4 MCE (B)	(Table 4-4)
Performance Objective:	3B	(Table 4-4)

A.2 *Determine site seismicity.*

The following value is assumed for this example:

$S_S = 1.50g$ (MCE Maps)

A.3 *Determine site characteristics.*

Soil type D is assumed for this problem

Soil type: D (Table 3-1)

A.4 *Determine site coefficients.*

$F_a = 1.0$ (Table 3-2a)

A.5 *Determine adjusted MCE spectral response accelerations.*

$S_{MS} = F_aS_S = 1.0(1.50)g = 1.50g$ (EQ. 3-1)

A.6 *Determine design spectral response accelerations.*

$S_{DS} = 3/4\ S_{MS} = 3/4(1.50) = 1.125g$ (EQ. 3-3)

A.7 *Bracing system.*

Assume that the suspension system consists of 5 wires; one vertical and four inclined at 45 degrees. Inclined wires are oriented parallel and perpendicular to the direction of the steel deck flutes as shown in Figure I1-2.

A.8 *Select R_p, a_p, and I_p factors.*

$a_p = 1.0$ (Table 10-1)
$R_p = 2.5$ (Table 10-1)
$I_p = 1.5$ (per Paragraph 10-1d)

A.10 *Determine member sizes for gravity load effects.*

Vertical hanger wire design;
Assume suspension system is placed at 12-ft. (3.66m) on center each way. Assume actual load is 3.5-lbs. per square foot (0.17KN/m), but must use the minimum value of 4-lbs. per square foot (0.19KN/m) per Paragraph 10-2d(4).

$W_p = 4\text{psf}(12')12' = 576\text{-lb}$ (2.56KN)

Use #8 galvanized soft steel wire (ASTM A651)

$f_u = 70\text{ksi}$ (482.7MPa)
$f_y = 50\text{ksi}$ (344.8MPa) (assumed) use $f_{allow} = f_y = 50\text{ksi}$ (344.8MPa)
$A_s = 0.0206\text{-in}^2$ (13.3mm^2)

Determine factored loads for gravity load effects;

$P_u = 1.4D$ (ANSI/ASCE 7-95)
$P_u = 1.4(576\text{-lb}) = 806\text{-lb-tension}$ (3.59KN)

Therefore,
$f_s = P_u/A_s = 806\text{-lb}/0.0206\text{-in}^2 = 39.1\text{ksi} < 50\text{ksi} = f_{allow}$ (269.6MPa < 344.8MPa) **O.K.**

Note: Table 4-6 was created as an aid for building design and is not entirely applicable in the design of nonstructural systems and components. The following steps are based on the intent of this document, and do not have a one to one correspondence to steps as listed in Table 4-6.

F.1 *Determine seismic force effects.*

Seismic forces (F_p) shall be determined in accordance with Chapter 10 as follows:

$$F_p = \frac{0.4a_p S_{DS} W_p}{R_p / I_p}\left(1 + 2\frac{z}{h}\right) \qquad \text{(EQ. 10-1)}$$

where; $z/h = 1.0$ (assumed at the top story)

F_p is not required to be greater than:

$$F_p = 1.6 S_{DS} I_p W_p \qquad \text{(EQ. 10-2)}$$

nor less than:

$$F_p = 0.3 S_{DS} I_p W_p \qquad \text{(EQ. 10-3)}$$

$$F_p = \frac{0.4(1.0)1.125(576 - \text{lb})}{2.5/1.5}(1 + 2(1)) = 0.81(576 - \text{lb}) = 467 - \text{lb} \quad (2.07\text{KN})$$

$$(F_p)_{max} = 1.6(1.125)1.5(576 - \text{lb}) = 1{,}555 - \text{lb} > 467 - \text{lb} = F_p \quad (6.92\text{KN} > 2.07\text{KN}) \qquad \textbf{O.K.}$$

$$(F_p)_{min} = 0.3(1.125)1.5(576-\text{lb}) = 292-\text{lb} < 467-\text{lb} = F_p \quad (1.30\text{KN} < 2.04\text{KN})$$ **O.K.**

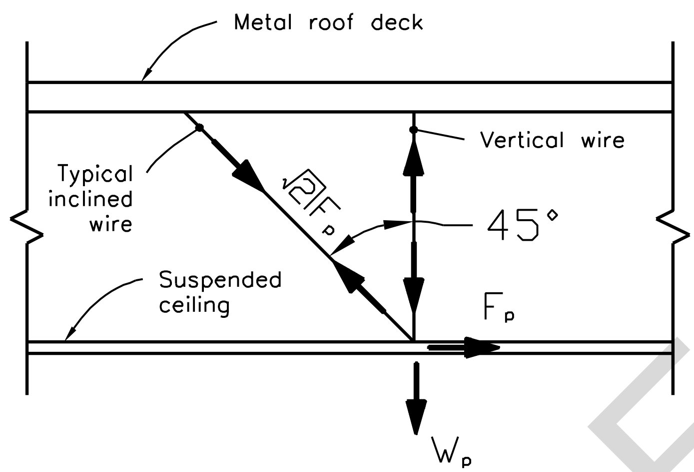


Figure I1-1. Force diagram for bracing wires

F.2 *Design members.*

Inclined wires;
Tension in inclined wire $= \sqrt{2}F_p = 660-\text{lb}$ (2.94KN) (see Figure I1-1)
Since only lateral loads are acting in the inclined wires all load combinations reduce to;

$Q_u = 1.0E = 1.0F_p = 660\text{-lb}$ (2.94KN)
(ANSI/ASCE 7-95)

Use #10 galvanized soft steel wire (ASTM A651)

$f_u = 70\text{ksi}$ (482.7MPa)
$f_y = 50\text{ksi}$ (344.8MPa) (assumed) use $f_{allow} = f_y = 50\text{ksi}$ (344.8MPa)
$A_s = 0.0143\text{-in}^2$ (9.23mm^2)

$f_s = Q_u/A_s = 660\text{-lb}/0.0143\text{-in}^2 = 46.2\text{ksi} < 50\text{ksi} = f_{allow}$ (318.5MPa < 344.8MPa) **O.K.**

Vertical wires;
Dead load on the wire was previously calculated as $W_p = 576\text{-lb}$ (2.56KN). The ability of the dead load to keep the wire taught is to be checked.

$Q_u = 0.9D + 1.0Q_E = 0.9(-576\text{-lb}) + 467\text{-lb} = -51\text{-lb-tension}$ (0.23KN) **O.K.**

Connections;
A L2X2X3/16X1'-3" (L50.8mmX50.8mmX4.8mmX0.38m) angle is used to transfer load to the steel deck. Angle is to be welded to at least two flutes of the deck as shown in Figure I1-2. The worst case loading is due to gravity load effects acting alone and was calculated as $P_u = 806\text{-lb}$ (3.59MN). It is decided to weld to each flute using 2-in. by 1/8-in. (50.8mmX3.2mm) fillet welds as shown (total weld length is 8-in. (203.2mm)).

Check capacity of welds;

$$A_{eff} = \frac{0.125-in}{\sqrt{2}} = 0.088-\frac{in^2}{in}$$

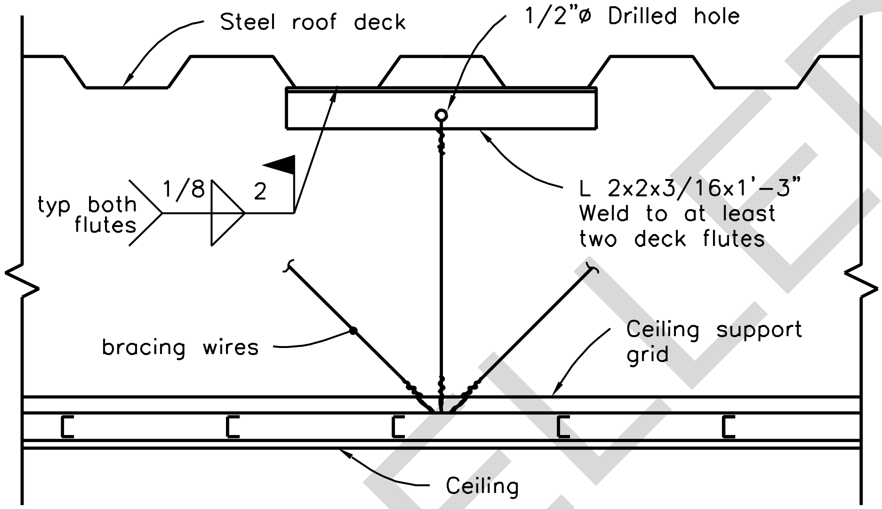
Use E70 electrodes: f_{exx} = 70ksi (482.7MPa)

Therefore,

$Q_E = \phi(0.6)f_{exx}(A_{eff})$ x (the total length of welds)

$Q_E = 0.75(0.6)70ksi(0.088\text{-}in^2)8" = 22.3^k > 0.806^k$ (99.2KN > 3.59KN) **O.K.**

Note: Connections are overdesigned, but required to distribute forces to the steel deck.



Note: For metric equivalents; 1-in = 25.4mm, 1-ft = 0.30m

Figure I1-2. Wire support and bracing system

I-2 MASONRY PARTITION BRACING

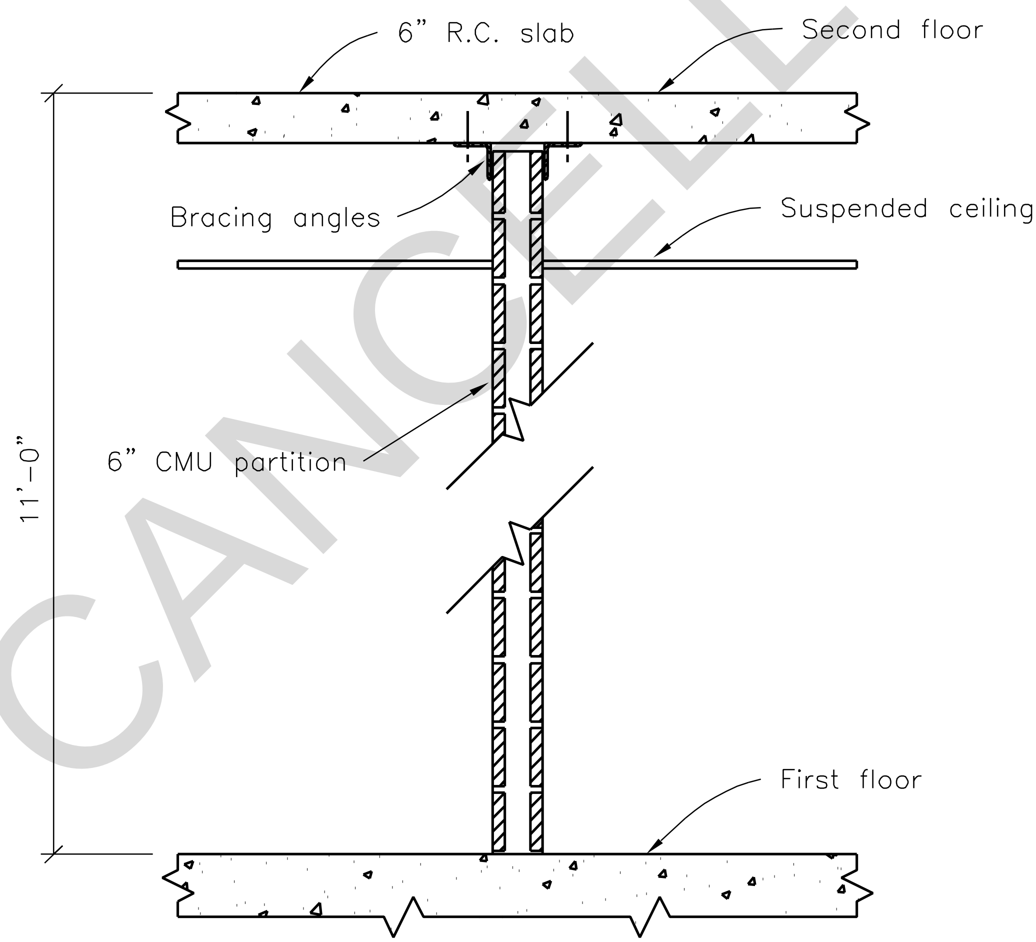
a. Introduction.

(1) Purpose. The purpose of this example problem is to illustrate the design of masonry partition bracing using Chapter 10 of TI 809-04, and Chapter 6 of FEMA 302 (Components). Unbraced masonry partitions are vulnerable to out-of-plane failure when subjected to lateral loads. Failure of these heavy partitions can cause injury to the occupants, preclude safe egress, and can obstruct essential functions in the building.

(2) Scope. The problem follows the steps in Table 4-6 to analyze the bracing and anchors. The solution is a modification of the bracing detail found in Figure 10-1. The building housing the partition is required to be functional after an earthquake.

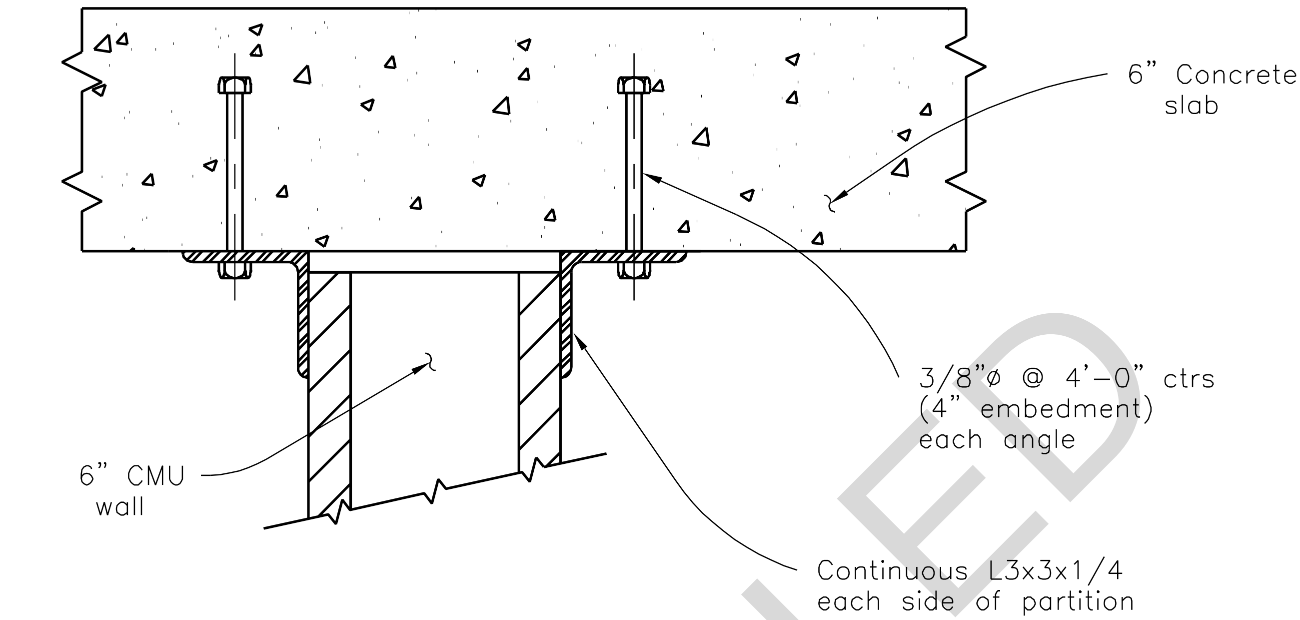
b. Component description.

The reinforced masonry partition used in this example problem forms part of an exit corridor below a concrete slab in a fire station. The partition is 10-1/2 (3.20m) feet high as shown in Figure I2-1. The bracing scheme is shown in Figure I2-2. The bracing is checked using a linear elastic analysis as described in paragraph 6-3a for force-controlled components.



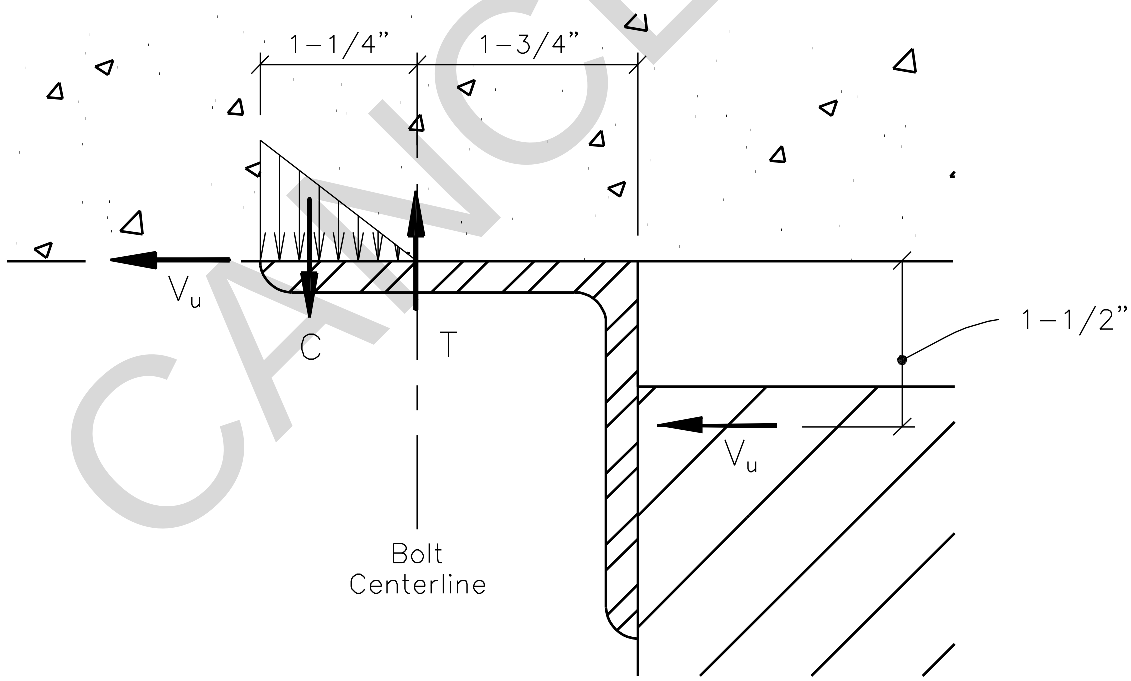
Note: For metric equivalents; 1-in = 25.4mm, 1-ft = 0.30m

Figure I2-1. Masonry partition in fire station



Note: For metric equivalents; 1-in = 25.4mm, 1-ft = 0.30m

Figure I2-2. Detail of lateral bracing for masonry partition



Note: For metric equivalents; 1-in = 25.4mm

Figure I2-3. Detail of forces in brace connection *c.* *Component design.*

A.1 *Determine appropriate Seismic Use Group*

Due to the requirement that the building be functional after and earthquake, the partition is given a performance level of immediate occupancy (IO). The Seismic Use Group and other performance parameters are determined from Table 4-4, as follows;

Performance Level:	IO	(per problem statement)
Seismic Use Group:	IIIE	(Table 4-4)
Ground Motion:	3/4 MCE (B)	(Table 4-4)
Performance Objective:	3B	(Table 4-4)

A.2 *Determine site seismicity.*

The following values are assumed for this example:

$S_S = 0.80g$ (MCE Maps)

A.3 *Determine site characteristics.*

Soil type C is assumed for this problem

Soil type: C (Table 3-1)

A.4 *Determine site coefficients.*

$F_a = 1.08$ (Table 3-2a)

A.5 *Determine adjusted MCE spectral response accelerations.*

$S_{MS} = F_a S_S = 1.08(0.80)g = 0.86g$ (EQ. 3-1)

A.6 *Determine design spectral response accelerations.*

$S_{DS} = 3/4\ S_{MS} = 3/4(0.86) = 0.65g$ (EQ. 3-3)

A.7 *Bracing system.*

Brace partitions with steel angles on each side. Angles to be bolted to 2nd floor concrete slab as indicated in Figure I2-2.

A.8 *Select R_p, a_p, and I_p factors.*

$a_p = 2.5$ (Table 10-1)

$R_p = 2.5$ (Table 10-1)

$I_p = 1.5$ (per Paragraph 10-1d)

A.10 *Determine member sizes for gravity load effects.*

No gravity load design is required.

Note: Table 4-6 was created as an aid for building design and is not entirely applicable in the design of nonstructural systems and components. The following steps are based on the intent of this document, and do not have a one to one correspondence to steps as listed in Table 4-6.

F.1 *Determine seismic force effects.*

Seismic forces (F_p) shall be determined in accordance with Chapter 10 as follows:

$$F_p = \frac{0.4a_pS_{DS}W_p}{R_p/I_p}\left(1+2\frac{z}{h}\right) \qquad \text{(EQ. 10-1 TI 809-04)}$$

where; z/h = 1/2 (1st story of a 2-story building)

W_p;

Dead load = 40psf (1.92KN/m^2) (6-in. (152.4mm) CMU)

$$\therefore W_p = \frac{10.5'(40psf)4'}{2} = 840-lb \quad (3.74KN)$$

F_p is not required to be greater than:

$$F_p = 1.6S_{DS}I_pW_p \qquad \text{(EQ. 10-2)}$$

nor less than:

$$F_p = 0.3S_{DS}I_pW_p \qquad \text{(EQ. 10-3)}$$

$$F_p = \frac{0.4(2.5)0.65(840-lb)}{2.5/1.5}\left(1+2(\frac{1}{2})\right) = 0.78(840-lb) = 655-lb \quad (2.91KN)$$

$$(F_p)_{max} = 1.6(0.65)1.5(840-lb) = 1{,}310-lb > 655-lb = F_p \quad (5.83KN > 2.91KN) \qquad \textbf{O.K.}$$

$$(F_p)_{max} = 0.3(0.65)1.5(840-lb) = 246-lb < 655-lb = F_p \quad (1.09KN < 2.91KN) \qquad \textbf{O.K.}$$

F.2 *Design members.*

Design of angle brace;
Try L3x3x1/4 (L76.2mmX76.2MMX6.4mm) with 3/8-in. (9.53mm) ϕ bolt (A-307) @ 4'-0" (1.22m) on center with 4" (101.6mm) embedment (see Figure I2-2).

Check flexure in angle
Assume simple beam moment for angle spanning between bolts;

$$\phi M_n = \phi ZF_y > M_u$$

$w_p = F_p/4' = 655\text{-lb}/4' = 164plf$ (2.39KN/m)

$$M_u = \frac{w_pL^2}{8} = \frac{164plf(4')^2(12''/1')}{8} = 3{,}936^{in-lb} \text{ or } 3.94^{in-k} \quad (0.44kn\text{-}m)$$

For L3x3x1/4, Z = 1.04-in^3 (17.04X10^3 mm^3)

$$\therefore \phi M_n = 0.9ZF_y = 0.9(1.04-in^3)36ksi = 33.7^{in-k} > 3.94^{in-k} = M_u \quad (3.81KN\text{-}m > 0.44KN\text{-}m) \qquad \textbf{O.K.}$$

Check bolt capacity
Determine loading;

$$V_u = 655-lb/bolt \quad (2.91KN)$$

The force in the bolt due to prying action is determined by summing moments about the bolt centerline (see Figure I2-3);

$665^{lb}(1.5") = C(1.25")2/3$
$P_u = T = C = 1{,}197\text{-lbs}$ or 1.2^k (5.34KN)

Check capacity in shear;
Steel;

$$V_s = (0.75A_bF_u)n \qquad \text{(EQ. 9.2.4.2-1 FEMA 302)}$$

For 3/8" (9.53mm) ϕ bolt (A 307)

where; $A_b = 0.11\text{-in}^2$ (70.97mm^2)
$F_u = 60\text{ksi}$ (413.7MPa)
$n = 1\text{-bolt}$

$$V_s = 0.75(0.11 - \text{in}^2)60\text{ksi}(1.0 - \text{bolt}) = 4.95^k > 0.665^k = V_u \quad (22.02\text{KN} > 2.96\text{KN}) \qquad \textbf{O.K.}$$

Concrete;

$$\phi V_c = (\phi 800 A_b \lambda \sqrt{f_c'})n \qquad \text{(EQ. 9.2.4.2-2 FEMA 302)}$$

where; $\phi = 0.65$
$\lambda = 1.0$ (normal weight concrete)
$f_c' = 4{,}000\text{psi}$ (27.6MPa)
$n = 1\text{-bolt}$

$$\therefore \phi V_c = 0.65(800)0.11 - \text{in}^2(1.0)\sqrt{4{,}000\text{psi}} = 0.65(5.57^k) = 3.62^k \quad (16.1\text{KN})$$

$$\phi V_c = 3.62^k > 0.665^k = V_u \quad (16.1\text{KN} > 2.96\text{KN}) \qquad \textbf{O.K.}$$

Check capacity in tension;
Steel;

$$P_s = (0.9 A_b F_u)n \qquad \text{(EQ. 9.2.4.1-1 FEMA 302)}$$

where; $A_b = 0.11\text{-in}^2$ (70.97mm^2)
$F_u = 60\text{ksi}$ (413.7MPa)
$n = 1\text{-bolt}$

$$\therefore P_s = 0.9(0.11 - \text{in}^2)60\text{ksi}(1 - \text{bolt}) = 5.94^k > 1.2^k = P_u \quad (26.4\text{KN} > 5.3\text{KN}) \qquad \textbf{O.K.}$$

Concrete;

$$\phi P_c = \phi \lambda \sqrt{f_c'}(2.8 A_s)n \qquad \text{(EQ. 9.2.4.1-2 FEMA 302)}$$

where; $\phi = 0.65$
$\lambda = 1.0$ (normal weight concrete)
$f_c' = 4{,}000\text{psi}$ (27.6MPa)
$A_s = \pi(4")4" = 50.3\text{-in}^2$ $(32.45\text{X}10^3\ \text{mm}^2)$ (for a 4-in. (101.6mm) embedment)

$$\therefore \phi P_c = 0.65(1.0)\sqrt{4{,}000\text{psi}}(2.8(50.3 - \text{in}^2))(1 - \text{bolt}) = 0.65(8.91^k) = 5.79^k > 1.2^k = P_u \qquad \textbf{O.K.}$$

(25.8KN > 5.3KN)

Check combined tension and shear;
Per section 9.2.4.3 of FEMA 302 , all of the following conditions shall be met;

condition (a) $$\frac{1}{\phi}\left(\frac{V_u}{V_c}\right) \le 1.0 \qquad \text{(EQ. 9.2.4.3-1a FEMA 302)}$$

$$\frac{1}{0.65}\left(\frac{0.665^k}{5.57^k}\right) = 0.18 \le 1.0 \qquad \textbf{O.K.}$$

condition (b) $$\frac{1}{\phi}\left(\frac{P_u}{P_c}\right) \le 1.0 \qquad \text{(EQ. 9.2.4.3-1b FEMA 302)}$$

$$\frac{1}{0.65}\left(\frac{1.2^k}{8.91^k}\right) = 0.21 \le 1.0 \qquad \textbf{O.K.}$$

condition (c) $$\frac{1}{\phi}\left[\left(\frac{P_u}{P_c}\right)^2 + \left(\frac{V_u}{V_c}\right)^2\right] \le 1.0 \qquad \text{(EQ. 9.2.4.3-1c FEMA 302)}$$

$$\frac{1}{0.65}\left[\left(\frac{1.2^k}{8.91^k}\right)^2 + \left(\frac{0.665^k}{5.57^k}\right)^2\right] = \frac{1}{0.65}(0.018 + 0.014) = 0.05 \le 1.0 \qquad \textbf{O.K.}$$

condition (d) $\left(\frac{P_u}{P_s}\right)^2 + \left(\frac{V_u}{V_s}\right)^2 \leq 1.0$ (EQ. 9.2.4.3-1d FEMA 302)

$$\left(\frac{1.2^k}{5.94^k}\right)^2 + \left(\frac{0.665^k}{4.95^k}\right)^2 = 0.041 + 0.018 = 0.06 \leq 1.0$$ **O.K.**

Check flexure in angle leg;

$$\phi M_n = \phi Z F_y > M_u$$

Moment @ bolt (assume 8" (203.2mm) length of angle is effective)

$M_u = 1.2^k(2/3)1.25" = 1.0^{in\text{-}k}$ (0.11KN-m)

Calculate Z;

For a rectangular section: $Z = \frac{bh^2}{4} = \frac{8"(0.25")^2}{4} = 0.125 - in^3$ ($2.05X10^3\ mm^3$)

$$\therefore \phi M_n = 0.9ZF_y = 0.9(0.125 - in^3)36ksi = 4.05^{in-k} > 1.0^{in-k} = M_u$$ **O.K.**

(0.46KN-m > 0.11KN-m)

Conclusion

In view of the stresses in the bolt and the angle, bolt spacing will be revised to 6'-0" (1.83m) on center. This should be confirmed by re-iteration of the shear calculations.

APPENDIX J

J-1 ELEVATOR GUARD RAIL BRACING

a. Introduction.

(1) Purpose. The purpose of this example problem is to illustrate the design of elevator counterweight guide rail bracing using Chapter 10 and Chapter 6 of FEMA 302 (Components). A common occurrence in high-rise buildings is loss of elevator service due to the counterweight leaving the guide rails and impacting the cab. The problem is attributed to flexibility or failure of the counterweight guide rails and their supports.

(2) Scope. The problem follows the steps in Tables 4-5 and 4-6 to analyze the counterweight guide rails and their supports. The building housing the elevator is required to be functional after an earthquake.

b. Component description.

The elevator used in this example problem is located in a five-story steel moment frame building. The elevator counterweight is assumed to be 5,000 pounds (22.24KN). Typical elevator details are shown in Figure 10-11.

c. Component design.

A.1 *Determine appropriate Seismic Use Group*

Due to the requirement that the building be functional after an earthquake, the elevator is given a performance level of immediate occupancy (IO). The Seismic Use Group and other performance parameters are determined from Table 4-4, as follows;

Performance Level:	IO	(per problem statement)
Seismic Use Group:	IIIE	(Table 4-4)
Ground Motion:	3/4 MCE (B)	(Table 4-4)
Performance Objective:	3B	(Table 4-4)

A.2 *Determine site seismicity.*

The following values are assumed for this example:

$S_S = 1.20g$ (MCE Maps)

A.3 *Determine site characteristics.*

Soil type D is assumed for this problem

Soil type: D (Table 3-1)

A.4 *Determine site coefficients.*

$F_a = 1.02$ (interpolated) (Table 3-2a)

A.5 *Determine adjusted MCE spectral response accelerations.*

$S_{MS} = F_aS_S = 1.02(1.20)g = 1.22g$ (EQ. 3-1)

A.6 *Determine design spectral response accelerations.*

$S_{DS} = 3/4\ S_{MS} = 3/4(1.22) = 0.92g$ (EQ. 3-23)

A.7 *Bracing system.*

The guide rails are supported at each story by the floor framing (see Figure J1-1). Since it is not feasible to brace the rails between floor levels, the rails will be stiffened to span between supports to resist the seismic forces from the counterweight.

A.8 *Select R_p, a_p, and I_p factors.*

$a_p = 1.0$ (Table 10-2)
$R_p = 2.5$ (Table 10-2)
$I_p = 1.5$ (per Paragraph 10-1d)

A.10 *Determine member sizes for gravity load effects.*

No gravity load design is required.

Note: Table 4-6 was created as an aid for building design and is not entirely applicable in the design of nonstructural systems and components. The following steps are based on the intent of TI 809-04, and do not have a one to one correspondence to steps as listed in table 4-6.

F.1 *Determine seismic force effects.*

Seismic forces (F_p) shall be determined in accordance with chapter 10 as follows (see Figure J1-1):

$$F_p = \frac{0.4 a_p S_{DS} W_p}{R_p / I_p}\left(1 + 2\frac{z}{h}\right) \qquad \text{(EQ. 10-1)}$$

where; $z/h = 1.0$ (top story of building)
$W_p = 5{,}000\text{-lb}$ (22.24KN) (counterweight)

F_p is not required to be greater than:

$$F_p = 1.6 S_{DS} I_p W_p \qquad \text{(EQ. 10-2)}$$

nor less than:

$$F_p = 0.3 S_{DS} I_p W_p \qquad \text{(EQ. 10-2)}$$

$$F_p = \frac{0.4(1.0)0.92(5{,}000-\text{lb})}{2.5/1.5}\left(1+2(1)\right) = 0.66(5{,}000-\text{lb}) = 3{,}300-\text{lb or } 3.30\text{-kips} \ \ (14.68\text{KN})$$

$$(F_p)_{max} = 1.6(0.92)1.5(5{,}000-\text{lb}) = 11{,}040-\text{lb} > 3{,}300-\text{lb} = F_p \ \ (49.11\text{KN} > 14.68\text{KN}) \qquad \textbf{O.K.}$$

$$(F_p)_{min} = 0.3(0.92)1.5(5{,}000-\text{lb}) = 2{,}070-\text{lb} < 3{,}300-\text{lb} = F_p \ \ (9.21\text{KN} < 14.68\text{KN}) \qquad \textbf{O.K.}$$

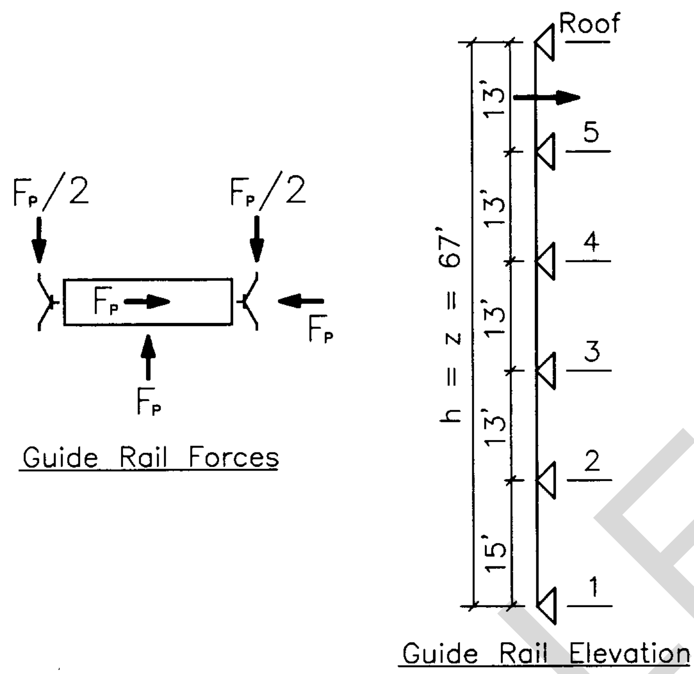
Because of the importance of the guide rails, assume that equation 4-1 of AISC Seismic Provisions (dated April 15, 1997) will apply with $\Omega_0 = 2.0$.

$$\therefore F_p = 3.30^k(2) = 6.60^k \ \ (29.36\text{KN})$$

F.2 *Design members.*

Check guide rail;
Assume elevator uses an 18.5plf (0.27KN/m) guide rail. The properties for an 18.5plf (0.27KN/m) guide rail, obtained from the guide rail manufacturer, are as follows;

$A = 5.44\text{-in}^2$ $(3.51\text{X}10^3\ \text{mm}^2)$	$I_x = 9.67\text{-in}^4$ $(4.02\text{X}10^6\ \text{mm}^4)$	$I_y = 7.45\text{-in}^4$ $(4.02\text{X}10^6\ \text{mm}^4)$
$\bar{y} = 1.26\text{-in}$ (32.0mm)	$S_x = 3.23\text{-in}^3$ $(52.93\text{X}10^3\ \text{mm}^3)$	$S_y = 2.71\text{-in}^3$ $(44.41\text{X}10^3\ \text{mm}^3)$
$J = 0.78\text{-in}^4$ $(324.7\text{X}10^3\ \text{mm}^4)$	$r_x = 1.33\text{-in}$ (33.8MM)	$r_y = 1.17\text{-in}$ (29.7mm)



Note: For metric equivalents; 1-ft = 0.30m

Figure J1-1. Guide rail elevation

Check flexure in rail;

Assume F_y = 36ksi (288.2MPa) (obtained from the guide rail manufacturer), and E = 29 x 10^6psi (200X10^3 MPa).

Note: The guide rail is essentially a tee section. Therefore, per AISC LRFD 2nd edition section F1.2c, its flexural capacity may be calculated as follows;

$$\phi_b M_n > M_u$$

where; $\phi_b = 0.9$

$$M_n = M_{cr} = \frac{\pi\sqrt{EI_yGJ}}{L_b}\left[B + \sqrt{1+B^2}\right] \qquad \text{(EQ. F1-15 AISC LRFD)}$$

$$M_n < 1.0M_y = 1.0S_xF_y$$

$$E = 29{,}000\text{ksi} \ \ (200\text{X}10^3\text{ MPa})$$

$$I_y = 7.45\text{-in}^4 \ \ (3.10\text{X}10^6\text{ mm}^4)$$

$$G = \frac{E}{2(1+\mu)} = \frac{29{,}000\text{ksi}}{2(1+0.3)} \approx 11{,}200\text{ksi} \ \ (77.2\text{X}10^3\text{ MPa}),$$

μ is poisson's ratio ≈ 0.3

$$J = 0.78\text{-in}^4 \ \ (324.7\text{X}10^3\text{ mm}^4)$$

$$L_b = 13\text{-ft.} = 156\text{-in.} \ \ (3.97\text{m})$$

$$B = \pm 2.3(d / L_b)\sqrt{I_y / J} \qquad \text{(EQ. F1-16 AISC LRFD)}$$

(the minus sign will be used for the stem in compression, and d, the depth of the rail section, = 4.25" (108.0mm) per manufacturers specifications)

$$\therefore B = -2.3(4.25''/156'')\sqrt{7.45-\text{in}^4 / 0.78-\text{in}^4} = -0.194$$

$$\therefore M_n = \frac{\pi\sqrt{29{,}000\text{ksi}(7.45-\text{in}^4)11{,}200\text{ksi}(0.78-\text{in}^3)}}{156-\text{in}}\left[-0.194 + \sqrt{1+(-0.194)^2}\right] = 722^{\text{in-k}} \ \ (81.59\text{KN-m})$$

$$M_y = 1.0S_xF_y = 1.0(3.23-\text{in}^3)36\text{ksi} = 116.3^{\text{in-k}} < 722^{\text{in-k}} = M_{cr} \ \ (13.14\text{KN-m} < 81.59\text{KN-m})$$

$$\therefore \phi_b M_n = \phi_b S_x F_y = 0.9(116.3^{in-k}) = 105^{in-k} \quad (11.89\text{KN-m})$$

Determine factored load;

Assume simple beam moment for rail spanning between floors;

$$(M_u)_x = \frac{PL}{4} = \frac{6.60^k(13')(12''/1')}{4} = 257^{in-k} \quad (29.04\text{KN-m}),$$

and $(M_u)_y = 0.5(M_u)_x = 129^{in-k}$ (14.58KN-m)

Therefore,

$$(\phi_b M_n)_x = 105^{in-k} < 257^{in-k} = (M_u)_x \quad (11.87\text{KN-m} < 29.04\text{KN-m})$$

N.G.

Determine required plastic section modulus;

$$(Z_x)_{req'd} = \frac{M_u}{\phi F_y} = \frac{257^{in-k}}{0.9(36ksi)} = 7.93 - in^3 \quad (129.9\text{X}10^3\ mm^3)$$

$$(Z_y)_{req'd} = \frac{M_u}{\phi F_y} = \frac{129^{in-k}}{0.9(36ksi)} = 3.98 - in^3 \quad (65.2\text{X}10^3\ mm^3)$$

Check deflection;

Assume maximum deflection to be limited to 1/2-in. (12.7mm);

$$\Delta_x = \frac{PL^3}{48EI} = \frac{6.60^k(13')^3(1{,}728 - in^3/ft^3)}{48(30{,}000ksi)9.67 - in^4} = 1.80 - in > 0.5 - in \quad (45.7mm > 12.7mm) \qquad \textbf{N.G.}$$

$$\Delta_y = \frac{3.30^k(13')^3(1{,}728 - in^3/ft^3)}{48(30{,}000ksi)7.45 - in^4} = 1.17 - in > 0.5 - in \quad (29.7mm > 12.7mm) \qquad \textbf{N.G.}$$

Determine required moment of inertia's;

$$I_x \geq \frac{PL^3}{48E\Delta_x} = \frac{6.60^k(13')^3(1{,}728 - in^3/ft^3)}{48(30{,}000ksi)0.5''} = 34.8 - in^4 \quad (14.48\text{X}10^6\ mm^4)$$

$$I_y \geq \frac{PL^3}{48E\Delta_x} = \frac{3.30^k(13')^3(1{,}728 - in^3/ft^3)}{48(30{,}000ksi)0.5''} = 17.4 - in^4 \quad (7.24\text{X}10^6\ mm^4)$$

Note: To simplify calculations, the contribution of the rail will be ignored in determining flexural strength.

Try TS 5x5x1/4 welded to back of guide rails (see Figure J1-2);

Properties of TS 5x5x1/4:

$Z_x = Z_y = 8.07\text{-}in^3$ ($132.2\text{X}10^3\ mm^3$) $I_x = I_y = 16.9\text{-}in^4$ ($7.03\text{X}10^6\ mm^4$)

$S_x = S_y = 6.78\text{-}in^3$ ($111.1\text{X}10^3\ mm^3$) $A = 4.59\text{-}in^2$ ($2.96\text{X}10^3\ mm^2$)

$\bar{y} = 2.50$-in (63.5mm) r = 1.92-in (48.8mm)

$J = 27.4\text{-}in^4$ ($11.40\text{X}10^6\ mm^4$)

Check deflection;

Determine properties of composite section for deflection calculations;

$$\bar{y} = \frac{\sum A_i y_i}{\sum A_i} = \frac{(5.44 - in^2)(-1.26'') + (4.59 - in^2)(2.5'')}{5.44 - in^2 + 4.59 - in^2} = +0.46 - in \quad (1.7mm)$$

Calculate moment of inertia about the neutral axis

$$I = \sum(I_o + Ad^2)$$

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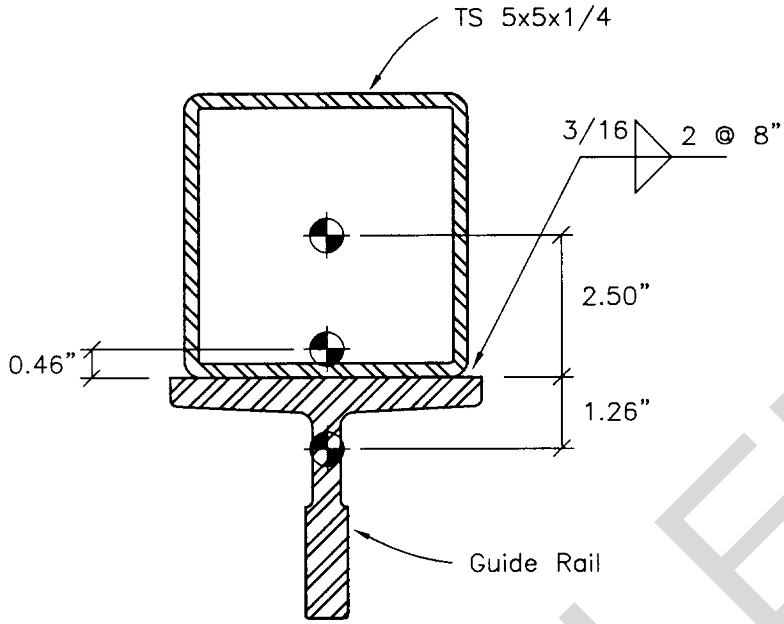


Note: For metric equivalents; 1-in = 25.4mm

Figure J1-2. Composite section of stiffened guide rail

Calculations about the x-axis are performed in tabular form as follows;

Element	A (in^2)	d (in)	Ad^2 (in^3)	I_0 (in^4)
TS 5x5x1/4	4.59	2.04	19.10	16.90
Guide rail	5.44	1.72	16.09	9.67
			35.19	26.57

1-in = 25.4mm
$1\text{-in}^2 = 645.2\text{mm}^2$
$1\text{-in}^3 = 16.4\text{X}103\ \text{mm}^3$
$1\text{-in}^4 = 416.2\text{X}103\ \text{mm}^4$

$I_x = 26.57\text{-in}^4 + 35.19\text{-in}^4 = 61.76\text{-in}^4$ $(25.7\text{X}10^6\ \text{mm}^4)$
$I_y = 7.45\text{-in}^4 + 16.90\ \text{-in}^4 = 24.35\ \text{-in}^4$ $(10.1\text{X}10^6\ \text{mm}^4)$

Therefore;

$$\Delta_2 = \Delta_1\left(\frac{I_1}{I_2}\right)$$

$$\Delta_x = 1.80 - \text{in}\left(\frac{9.67 - \text{in}^4}{61.76 - \text{in}^4}\right) = 0.28" < 0.50" \quad (7.1\text{mm} < 12.7\text{mm})$$ **O.K.**

$$\Delta_y = 1.17 - \text{in}\left(\frac{7.45 - \text{in}^4}{24.35 - \text{in}^4}\right) = 0.36" < 0.50" \quad (9.1\text{mm} < 12.7\text{mm})$$ **O.K.**

Check flexural capacity of rail assuming TS 5X5X1/4 (TS 127.0mmX127.0mmX6.4mm) supports all loads;

L_b = 13-ft or 156-in (3.97m)

$$L_p = \frac{300r_y}{\sqrt{F_{yf}}} = \frac{300(1.92")}{\sqrt{36\text{ksi}}} = 96.0 - \text{in} \quad (2.44\text{m})$$ (EQ. F1-4 AISC LRFD)

Since $L_b > L_p$ must calculate L_r, and M_r;

$$L_r = \frac{r_y X_1}{F_L}\sqrt{1+\sqrt{1+X_2 F_L^2}}$$ (EQ. F1-6 AISC LRFD)

$$M_r = F_L S_x$$ (EQ. F1-7 AISC LRFD)

where; F_L = smaller of (F_{yf} - F_r) or F_{yw}
Since $F_{yf} = F_{yw}$, and F_r = 10ksi (69.0MPa)
F_L = 36ksi – 10ksi = 26ksi (179.3MPa)

$$\therefore M_r = 26\text{ksi}(6.78 - \text{in}^3) = 176^{\text{in-k}} \quad (19.89\text{KN-m})$$

$$X_1 = \frac{\pi}{S_x}\sqrt{\frac{EGJA}{2}}$$ (EQ. F1-8 AISC LRFD)

where; E = 29,000ksi ($200X10^3$ MPa)
G = 11,200ksi ($77.2X10^3$ MPa)
J = 27.4-in^4 ($11.4X10^6$ mm^4)

$$\therefore X_1 = \frac{\pi}{S_x}\sqrt{\frac{EGJA}{2}} = \frac{\pi}{6.78-\text{in}^3}\sqrt{\frac{29{,}000\text{ksi}(11{,}200\text{ksi})27.4-\text{in}^4(4.59-\text{in}^2)}{2}} = 66{,}220\text{ksi}$$

($456.6X10^3$ MPa)

$$X_2 = 4\frac{C_w}{I_y}\left(\frac{S_x}{GJ}\right)^2 = 0$$ ($C_w = 0$ for a doubly symmetric closed section) (EQ. F1-9 AISC LRFD)

$$\therefore L_r = \frac{1.92''(66{,}221\text{ksi})}{26\text{ksi}}\sqrt{2} = 6{,}916'' \quad (175.78\text{m})$$

$$M_n = C_b\left[M_p - (M_p - M_r)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p$$ (EQ. F1-2 AISC LRFD)

where; $M_n \le 1.5M_y = 1.5S_xF_y$
$M_p = Z_xF_y = 8.07 - \text{in}^3(36\text{ksi}) = 291^{\text{in-k}}$ (32.88KN-m)
$C_b = 1.0$ (conservative for all load cases)

$$M_n = 1.0\left[291^{\text{in-k}} - (291^{\text{in-k}} - 176^{\text{in-k}})\left(\frac{156''-75.5''}{6{,}916''-75.5''}\right)\right] = 290^{\text{in-k}} \quad (32.77\text{KN-m})$$

Check that $M_n \le 1.5M_y$;

$$\frac{Z_x}{S_x} = \frac{8.07-\text{in}^3}{6.78-\text{in}^3} = 1.19 < 1.5$$

$\therefore (\phi M_n)_x = 0.9(290^{\text{in-k}}) = 261^{\text{in-k}} > 257^{\text{in-k}} = (M_u)_x$ (29.48KN-m > 29.04KN-m) **O.K.**

$\therefore (\phi M_n)_y = 261^{\text{in-k}} > 129^{\text{in-k}} = (M_u)_y$ (29.49KN-m > 14.58KN-m) **O.K.**

Check shear assuming guide rail supports all loads;
Note: The worst case shear occurs when the counterweight is positioned close to the support. The TS 5x5x1/4 section will be stopped short of the support thus placing the entire shear load on to the guide rail (see Figure J1-4). Assume data obtained from the manufacturer of the guide rail shows the guide rail stem and flange to be 0.5-in. (12.7mm) thick at their thinnest points, the overall depth of the rail section to be 4.25-in. (108.0mm), and the width of the flange to be 5.50-in. (139.7mm) By inspection, loading on the stem is the worst case to be investigated because it has the smallest shear area supporting the largest load.

Check the stem parallel to the 6.60^k (29.4KN) load;
$\phi_v V_n \le V_u$ where; $\phi_v = 0.90$

$$\frac{h}{t_w} = \frac{4.25''}{0.50''} = 8.5 < 69.7 = \frac{418}{\sqrt{36\text{ksi}}} = \frac{418}{\sqrt{F_y}}$$

$$\therefore V_n = 0.6F_{yw}A_w \qquad \text{(EQ. F2-1 AISC LRFD)}$$

$$\phi_v V_n = 0.9(0.6)36\text{ksi}(4.25''(0.5'')) = 41.3^k \ \ (183.7\text{KN})$$

$$\phi_v V_n = 41.3^k > 6.60^k = V_u \quad (183.7\text{KN} > 29.4\text{KN})$$ **O.K.**

Use TS 5x5x1/4 (TS 127mmX127mmX6.4mm) attached to guide rail

Design weld of TS to guide rail

Vertical shear at the TS to guide rail interface is determined from;

$$v = \frac{VQ}{I} \text{ (kips/in)}$$

where; $V_{max} = 6.60^k$ (29.4KN)

Q = static moment of the TS about the interface

$= 2.50''(4.59\text{-in}^2) = 11.48\text{-in}^3$ $(188.1\text{X}10^3\ \text{mm}^3)$

$$R_u = v = \frac{6.60^k(11.48 - \text{in}^3)}{61.76 - \text{in}^2} = 1.23^{k/in} \quad (0.22\text{KN/mm})$$

Since the thicker part being joined is 0.5-in. thick, the minimum thickness of weld per Table J2.4 of AISC LRFD is 3/16-in. Try a 3/16" fillet weld using E70 electrodes 2-in. in length spaced at 8-in. on center on both sides;

$$\phi R_n = 0.75t_e(0.60F_{EXX}) \qquad \text{(per AISC LRFD J2.2)}$$

where; $t_e = \frac{3''}{16}\left(\frac{1}{\sqrt{2}}\right) = 0.133 - \text{in}$ (3.38mm)

$$\phi R_n = 0.75(0.133'')(0.60(70\text{ksi})) = 4.19^{k/in} \text{ for a 3/16-in. (4.76mm) weld}$$

For two welds 2-in. long and spaced at 8-in. (203.2mm) on center;

$$\therefore \phi R_n = \frac{4.19^{k/in}(2'')}{8''}\text{x}2 = 2.10^{k/in} > 1.23^{k/in} = R_u \qquad (0.37\text{KN/mm} > 0.22\text{KN/mm})$$ **O.K.**

Use two 2-in. (50.8mm) by 3/16" (4.76mm) fillet welds spaced at 8-in. (20.32mm) on center to connect TS section to guide rail

Design bolted connection of guide rail to horizontal spreader bracket (Figures J1-3, and J1-4);

Note that the 6.60^k (29.4KN) load can only push against the guide rail as shown and can not pull it. Because the bracket angle is as yet not designed, the connecting members are conservatively assumed to be 3/8-in. (9.53mm) thick for these calculations.

Try 2-5/8" (15.9mm) ϕ bolts in a slip critical connection;

Check Shear;

By inspection, the worst case occurs for loads perpendicular to the long leg of the bracket angle.

$R_u = 6.60^k$ (29.4KN)

Per Table 8-16 of Volume 2 of AISC LRFD 2nd edition;

$$\phi R_n = 5.22^{k/bolt} \ \ (23.2\text{KN})$$

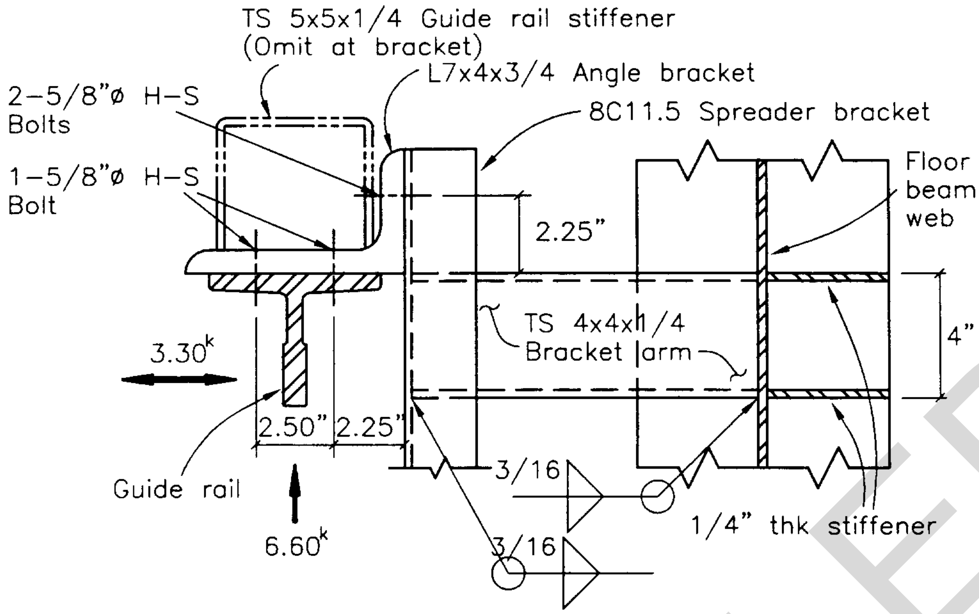
$$\therefore \phi R_n = 2(5.22^k) = 10.4^k > 6.60^k = R_u \ \ (46.3 > 29.4\text{KN})$$ **O.K.**

Note: Although slip critical connections theoretically are not subject to bearing, they must have sufficient strength in the event of an overload that may cause slip to occur.

Check bearing;

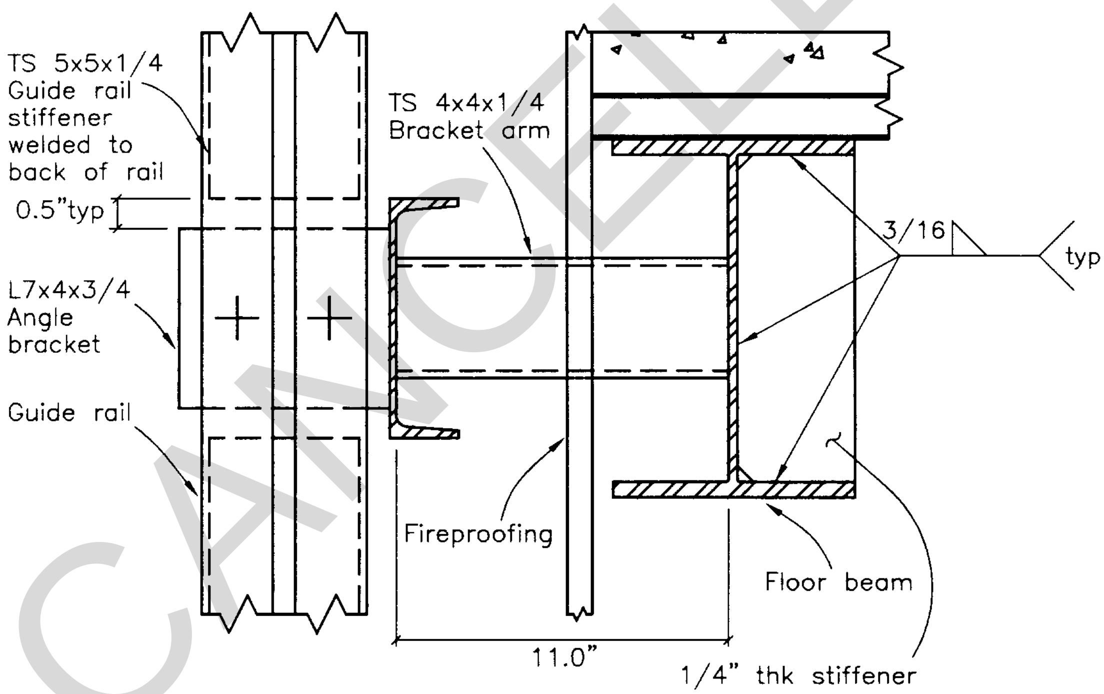
From Figure J1-3, the maximum spacing of bolts aligned in the direction of load is 2.5-in. (63.5mm) > 3d = 3(5/8-in.) = 1.88-in. (47.8mm), and the minimum edge distance is 1.5-in. (38.1mm) >1.5d = 1.5(5/8-in.) = 0.938-in. (23.8mm). Therefore, spacing and edge distance requirements of AISC LRFD J3.3 are satisfied.

Per Table 8-13 of Volume 2 of AISC LRFD 2nd edition;



Note: For metric equivalents; 1-in = 25.4mm, 1-kip = 4.48KN

Figure J1-3. Plan view of guide rail bracket



Note: For metric equivalents; 1-in = 25.4mm, 1-kip = 4.48KN

Figure J1-4. Section through guide rail bracket

$\phi R_n = 65.3^{k/in}(0.375") = 24.5^{k/bolt}$ (109.0KN/bolt)

$\therefore \phi R_n = 2(24.5^k) = 49^k > 6.60^k = R_u$ (218.0KN > 29.4KN) **O.K.**

Check prying action;

The worst case tensile load in the bolts connecting the angle bracket to the C channel spreader bracket must be determined. This load can result from either one of the load cases shown in Figure J1-3.

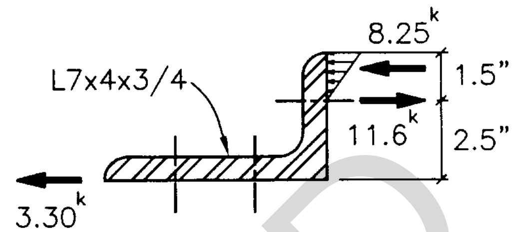
Consider bending about the bolt centerline at angles attachment to the spreader bracket;

Load condition (1); a 3.30^k (14.7KN) load parallel to the long leg on bracket angle (Figure J1-5):

$3.30^k(2.5") = C(2/3)1.5"$

$$C = \frac{(3.30^k)2.5"}{1.5}\left(\frac{3}{2}\right) = 8.25^k \quad (36.7KN)$$

$P_u/bolt = (8.25^k + 3.30^k)/2 = 5.78^{k/bolt}$ (25.7KN/bolt)



(1-in = 25.4mm, 1-kip = 4.448KN)

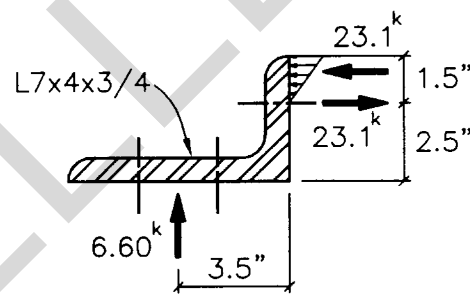
Figure J1-5. Detail of forces in angle bracket

Load condition (2); a 6.60^k (29.4KN) load perpendicular to the long leg on bracket angle (Figure J1-6):

$6.60^k(3.5") = C(2/3)1.5"$

$$C = \frac{(6.60^k)3.5"}{1.5}\left(\frac{3}{2}\right) = 23.1^k \quad (102.7KN)$$

$P_u/bolt = (23.1^k)/2 = 11.6^{k/bolt}$ (51.6KN/bolt)
(governs)



(1-in = 25.4mm, 1-kip = 4.448KN)

Figure J1-6. Detail of forces in angle bracket

Per Table 8-15 of volume 2 of AISC LRFD 2nd edition;

$\phi R_n = 20.7^{k/bolt} > 11.6^{k/bolt} = R_u$ (92.1KN/bolt > 51.6KN/bolt) **O.K.**

Use 2-5/8" (15.9mm) ϕ bolts at each angle bracket leg

Design bracket angle that connects guide rail to the horizontal spreader bracket (Figure J1-3, and J1-4);
Try L 7x4x3/8x0'-6" (L177.8mmX101.6mmX9.5mmX0.15m)
Design for flexure;
Critical section for bending occurs about the bolt centerline at angles attachment to the spreader bracket.
By inspection, the shape is compact and the unbraced length is negligible so that equation F1-1 of AISC LRFD may be used.

$\phi M_n = \phi Z_x F_y > M_u$ with; $\quad M_n \le 1.5 M_y = 1.5 S_x F_y$

As shown in the design of the bolts, the worst case moment is caused by the 6.60^k (29.4KN) load perpendicular to the Long leg of the angle bracket. Also, the reduction in cross sectional area due to the boltholes is considered in calculating the plastic section modulus.

$M_u = 6.60^k(3.5") = 23.1^{in\text{-}k}$ (2.61KN-m)

$$Z_x = \frac{bh^2}{4} = \frac{(6"-2(0.75"))(0.375")^2}{4} = 0.158-in^3 \quad (2.59X10^3\ mm^3)$$

$$S_x = \frac{bh^2}{6} = \frac{(6"-2(0.75"))(0.375")^2}{6} = 0.106-in^3 \quad (1.74X10^3\ mm^3)$$

$$\frac{Z_x}{S_x} = \frac{0.158}{0.106} = 1.49 < 1.5$$

$$\therefore \phi M_n = 0.9 Z_x F_y = 0.9(0.158 - \text{in}^3)36\text{ksi} = 5.12^{\text{in-k}} < 23.1^{\text{in-k}} = M_u \quad (2.02\text{KN-m} < 9.14\text{KN-m}) \ \textbf{N.G.}$$

Try L 7x4x3/4x0'-8"

$$Z_x = \frac{bh^2}{4} = \frac{(8"-2(0.75"))(0.75")^2}{4} = 0.914 - \text{in}^3 \quad (14.98\text{X}10^3 \text{ mm}^3)$$

$$S_x = \frac{bh^2}{6} = \frac{(8"-2(0.75"))(0.75")^2}{6} = 0.609 - \text{in}^3 \quad (9.98\text{X}10^3 \text{ mm}^3)$$

$$\frac{Z_x}{S_x} = \frac{0.914}{0.609} = 1.50$$

$$\therefore \phi M_n = 0.9 Z_x F_y = 0.9(0.914 - \text{in}^3)36\text{ksi} = 29.6^{\text{in-k}} > 23.1^{\text{in-k}} = M_u \quad (11.71\text{KN-m} > 9.14\text{KN-m})$$

O.K.

Use L7x4x3/4x0'-8" (L177.8mmX101.6mmX19.1mmX0.20m) for angle bracket

Design horizontal spreader bracket (Figures J1-3, and J1-4);
By inspection, the worst case moment is caused by the 6.60^k (29.4KN) load perpendicular to the Long leg of the angle bracket.

$M_u = 6.60^k(3.5") = 23.1^{\text{in-k}}$ (9.14KN/m)

By inspection, the shape is compact and the unbraced length is negligible so that equation F1-1 of AISC LRFD may be used.
Try C8x11.5;

$Z_x = 9.55\text{-in}^3$ $(156.5\text{X}10^3 \text{ mm}^3)$ $\quad S_x = 8.14\text{-in}^3$ $(144.4\text{X}10^3 \text{ mm}^3)$ $\quad \frac{Z_x}{S_x} = \frac{9.55}{8.14} = 1.17 < 1.5$

$$\therefore \phi M_n = 0.9 Z_x F_y = 0.9(9.55 - \text{in}^3)36\text{ksi} = 309^{\text{in-k}} > 23.1^{\text{in-k}} = M_u \quad (122.20\text{KN-m} > 9.14\text{KN-m})$$

O.K.

Use C8x11.5 (C203.2mmX0.17KN/m) for horizontal spreader

Design bracket arm to connect horizontal spreader to web of floor beam (Figure J1-3, and J1-4);
Assume that the spreader bracket distributes the load evenly between two bracket arms supporting the two guide rails on either side of the counter weight. Therefore, a simple bracket arm takes half of the loading. Also, note that the welded connection is eccentrically loaded.

Try TS 4x4x1/4 (TS 101.6mmX101.6mmX6.4mm) with a 3/16-in. (4.8mm) fillet weld all around;

Check TS 4x4x1/4 in flexure;
By inspection, the worst case moment is caused by the 6.60^k (29.4KN) load perpendicular to the Long leg of the angle bracket. The moment arm is L = 11" + 3" + 0.22" = 14.22" (361.2mm) using the C8x11.5 web thickness of 0.22-in (5.6mm).

$\therefore M_u = (1/2)6.60^k(14.22") = 46.9^{\text{in-k}}$ (5.30KN-m)

For a TS 4x4x1/4;

$Z_x = 4.97\text{-in}^3$ $\quad S_x = 4.11\text{-in}^3$ $\quad \frac{Z_x}{S_x} = \frac{4.97}{4.11} = 1.21 < 1.5$

$$\therefore \phi M_n = 0.9 Z_x F_y = 0.9(4.97 - \text{in}^3)36\text{ksi} = 161^{\text{in-k}} > 46.9^{\text{in-k}} = M_u \quad (18.16\text{KN-m} > 5.30\text{KN-m})$$

O.K.

Use TS 4x4x1/4 (TS 101.6mmX101.6mmX6.4mm) ***for bracket to floor beam web***

Check weld of TS4x4x1/4 to beam web and to spreader bracket;

$\phi R_n = 0.75 t_e (0.60 F_{EXX})$ (per AISC LRFD J2.2)

$$\text{where;}\quad t_e = \frac{3''}{16}\left(\frac{1}{\sqrt{2}}\right) = 0.133-\text{in}\quad (3.38\text{mm})$$

$$\phi R_n = 0.75(0.133'')(0.60(70\text{ksi})) = 4.19^{k/in} \text{ for a 3/16-in. (4.8mm) weld}$$

Combined shear and bending of welds:

Note: Even through the tube steel section is welded all around, only the two vertical welds opposite the stiffeners on the other side of the beam web will be considered effective for bending. Formula for the section modulus of the weld group is taken from Table 5.18.1 of 'Steel Structures' by C. Salmon and J. Johnson, 3rd edition.

$$f_s = \frac{P}{A} = \frac{6.60^k}{(4)4''} = 0.41^{k/in}\quad (71.8\text{N/mm})$$

For a weld pattern formed from two parallel lines of length 'b' and spaced a length 'd' apart;

f_b: $\quad I_p = bd = 4''(4'') = 16.0 - \text{in}^3/\text{in}\quad (262.2\text{X}10^3\ \text{mm}^3)$

where; b = 4" (101.6mm) = the length of the horizontal welds
d = 4" (101.6mm) = the length of the vertical welds

$$f_b = \frac{M_u}{S} = \frac{46.9^{in-k}}{16.0-\text{in}^3/\text{in}} = 2.93^{k/in}\quad (0.51\text{KN/mm})$$

Therefore,

$$R_u = f_r = \sqrt{f_s^2 + f_b^2} = \sqrt{(0.41^{k/in})^2 + (2.93^{k/in})^2} = 2.96^{k/in}\quad (0.52\text{KN/mm})$$

$$\phi R_n = 4.19^{k/in} > 2.96^{k/in} = R_u\quad (0.73\text{KN-m} > 0.52\text{KN-m})$$ **O.K.**

Use a 3/16-in. (4.8mm) weld to attach TS bracket to beam web and spreader bracket

Design stiffeners for beam web;

To prevent out of plane buckling of the beam web, stiffeners will be added as shown in Figures J1-3, and J1-4. By inspection, two 1/4-in. (6.4mm) thick stiffeners spaced at 4-in. (101.6mm) apart and connected with a 3/16-in. (4.8mm) fillet weld as shown will be sufficient.

Note: By inspection, the deflection of the guide rail due to the deflection of the stiffeners combined with the deflection of the bracket arm is less then 1/2-in. (12.7mm).

Use two 1/4-in. (6.4mm) thick stiffeners attached with a 3/16-in. (4.8mm) fillet weld

J-2 EQUIPMENT PLATFORM BRACING

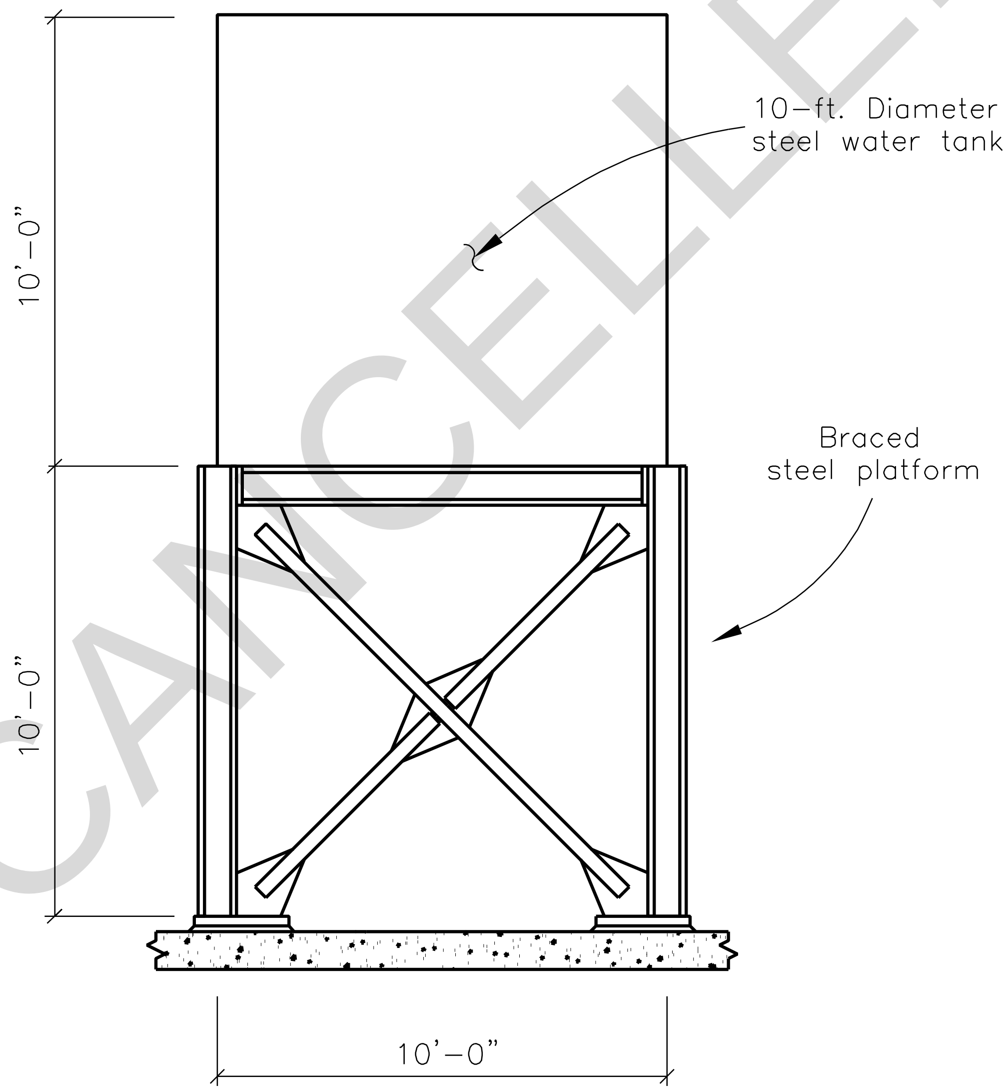
a. Introduction.

(1) Purpose. The purpose of this example problem is to illustrate the design of a braced steel platform supporting heavy equipment.

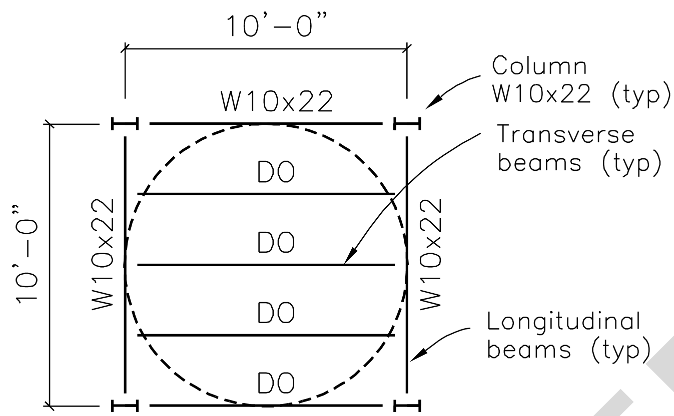
(2) Scope. The problem generally follows the steps in Table 4-5 and the procedures in Chapter 10 of this document and Chapter 6 of FEMA 302.

b. Component description.

The equipment in this example problem is an elevated steel water tank on a braced steel platform located on the roof of a two story building. An elevation of the tank and platform is shown in Figure J2-1 and a framing plan of the platform is shown in Figure J2-2.



Note: For metric equivalents; 1-ft = 0.30m
Figure J2-1. Elevation of tank and platform support structure



1-in = 25.4mm
1-ft = 0.30m
1plf = 14.58KN/m

Figure J2-2. Platform framing plan

c. Component design.

A.1 *Determine appropriate Seismic Use Group*

It is decided that the building supporting the tank be functional after an earthquake, therefore the tank is given a performance level of immediate occupancy (IO). The Seismic Use Group and other performance parameters are determined from Table 4-4, as follows;

Performance Level:	IO	(per problem statement)
Seismic Use Group:	IIIE	(Table 4-4)
Ground Motion:	3/4 MCE (B)	(Table 4-4)
Performance Objective:	3B	(Table 4-4)

A.2 *Determine site seismicity.*

The following values are assumed for this example:

$S_S = 1.20g$ (MCE Maps)

A.3 *Determine site characteristics.*

Soil type D is assumed for this problem

Soil type: D (Table 3-1)

A.4 *Determine site coefficients.*

$F_a = 1.02$ (interpolated) (Table 3.2a)

A.5 *Determine adjusted MCE spectral response accelerations.*

$S_{MS} = F_aS_S = 1.02(1.20)g = 1.22g$ (EQ. 3-1)

A.6 *Determine design spectral response accelerations.*

$S_{DS} = 3/4\ S_{MS} = 3/4(1.22) = 0.92g$ (EQ. 3-3)

A.7 *Lateral load reisiting system.*

Steel tank to be welded to platform and platform to be laterally supported by cross bracing in both directions.

A.8 *Select R_p, a_p, and I_p factors.*

$a_p = 2.5$ (Table 10-1)
$R_p = 2.5$ (Table 10-1)
$I_p = 1.5$ (per Paragraph 10-1d)

A.10 *Determine member sizes for gravity load effects.*

Determine structural weights;
Weight above platform;
Weight of water;
Note: Assume tank is normally full

$$V = pR^2 h = p(5')^2 10' = 785-\mathrm{ft}^3 \quad (22.23\mathrm{m}^3) \qquad \text{density of water} = 62.4\text{-lb/ft}^3 \quad (9800\mathrm{KN/m}^3)$$

$$W_{water} = 785-\mathrm{ft}^3(62.4^{\mathrm{lbs/ft}^3})(1^k/1000-\mathrm{lb}) = 49.0^k \quad (218.0\mathrm{KN})$$

Weight of tank shell (1/4-in. (6.4mm) plate);

$$\mathrm{Area} = pDh = p(10')10' = 314.2-\mathrm{ft}^2 \quad (29.19\mathrm{m}^2)$$

$$w_{plate} = 0.25''(1'/12'')490-\mathrm{lb/in}^3 = 10.2\mathrm{psf} \quad (0.49\mathrm{KN/m}^2)$$

$$W_{shell} = 314.2-\mathrm{ft}^2(10.2\mathrm{psf})(1^k/1000-\mathrm{lb}) = 3.2^k \quad (14.2\mathrm{KN})$$

Weight of tank top and bottom (3/8-in. (9.53mm) plate);

$$\mathrm{Area} = 2pr^2 = 2p(5')^2 = 157-\mathrm{ft}^2 \quad (14.59\mathrm{m}^2)$$

$$w_{plate} = 0.375''(1'/12'')490-\mathrm{lb/in}^3 = 15.3\mathrm{psf} \quad (0.73\mathrm{KN/m}^2)$$

$$W_{top\&bot} = 157-\mathrm{ft}^2(15.3\mathrm{psf})(1^k/1000-\mathrm{lb}) = 2.4^k \quad (10.7\mathrm{KN})$$

Therefore, the weight above the platform is;

$$W_{p1} = W_{water} + W_{shell} + W_{top\&bot} = 49.0^k + 3.2^k + 2.4^k = 54.6^k \quad (242.9\mathrm{KN})$$

Weight of platform;
Weight of platform beams (conservatively assume beam weight at 30psf over platform plan area);

$$W_{beams} = 30\mathrm{psf}(10')10'(1^k/1000-\mathrm{lb}) = 3.0^k \quad (13.3\mathrm{KN})$$

Weight of legs and braces (assume legs and braces at 10psf projected horizontally);
Note: Tributary height of 5-ft. (1.53m) is taken in lumping load at platform level

$$W_{legs\&braces} = 4\mathrm{sides}(10\mathrm{psf})5'(10')(1^k/1000-\mathrm{lb}) = 2^k \quad (8.9\mathrm{KN})$$

Therefore, the weight to lump at the platform level is;

$$W_{p2} = W_{beams} + W_{legs\&braces} = 3^k + 2^k = 5^k \quad (22.2\mathrm{KN})$$

Design transverse platform beams (see Figure J2-2, and J2-3);
Note: It is conservatively assumed that the middle transverse beam supports half of W_{p1} with the distribution as shown in Figure J2-3. One design will be made for this beam and used throughout for all transverse beams.

Using load combination U = 1.4D;

$$W_u = 1.4(0.5(54.6^k)) = 38.2^k \quad (169.9\mathrm{KN})$$

$$M_u = \frac{W_u L}{6} = \frac{38.2^k(10')(12''/1')}{6} = 764^{in-k} \quad (86.3\mathrm{KN})$$

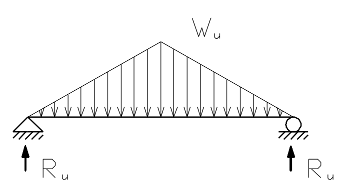


Figure J2-3. Loading on middle transverse beam

Note: Beam is laterally supported throughout its span

$$Z_{req'd} = \frac{M_u}{\phi F_y} = \frac{764^{in-k}}{0.9(36ksi)} = 23.6 - in^3 < 26.0\text{-}in^3 = Z_{W10x22} \;\; (386.7X10^3\ mm^3 < 426.1X10^3\ mm^3)$$

Use W10x22 (254mmX1.05KN/m) for transverse beams

Design longitudinal platform beams (see Figures J2-2, and J2-4);

$P_{1u} = (1/2)38.2^k = 19.1^k$ (89.96KN)

$P_{2u} = (1/2)P_{1u} = 9.6^k$ (42.70KN)

Therefore, reactions are; $R_u = 19.1^k$ (89.96KN)

The maximum moment occurs at mid span as;

$M_u = [19.1^k(5') - 9.6^k(2.5')](12''/1') = 858^{in\text{-}k}$ (96.95KN-m)

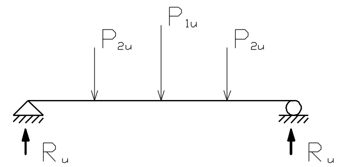


Figure J2-4. Loading on longitudinal beams

Note: Beam is laterally supported throughout its span

$$Z_{req'd} = \frac{M_u}{\phi F_y} = \frac{858^{in-k}}{0.9(36ksi)} = 26.5 - in^3 > 26.0 = Z_{W10x22} \;\; (434.3X10^3\ mm^3 > 426.1X10^3\ mm^3)$$

(okay because of conservative assumptions)

Use W10x22 (254mmX1.05KN/m) for longitudinal beams

Design columns;

Using load combination U = 1.4D;

$$P_u \text{ / column} = 1.4(W_{p1} + W_{p2})/\#\text{columns} = 1.4(54.6^k + 5^k)/4 = 20.9^{k/column} \quad (93.0KN/column)$$

Try W10x22;

Relevant properties of a W10x22 are as follows;

$A = 6.49\text{-}in^2$ ($4.19X10^3\ mm^2$) $r_y = 1.33$-in (33.8mm) $r_x = 4.27$-in (108.5mm)

Check capacity of W10x22;

Note: Elements are in a braced frame with K = 1.0

Since $KL_y = KL_x$;

$$\frac{KL_y}{r_y} = \frac{1.0(10')(12''/1')}{1.33''} = 90.2$$

From AISC LRFD Table 3-36; $\phi_c F_{cr} = 19.94ksi$ (137.5MPa) (interpolated)

$$\phi_c P_n = \phi_c F_{cr} A_g = 19.94ksi(6.49 - in^2) = 129.4^k \quad (575.6KN)$$

$$\phi_c P_n = 129.4^k > 20.9^k = P_u \quad (575.6KN > 93.0KN)$$

Use W10x22 (W254mmX1.05KN/m) for columns

Design transverse beam to longitudinal beam connection (see Figures J2-2, and J2-5);

Use; F_y = 36ksi (248.2MPa), and F_u = 58ksi (399.9MPa)

Note: It was previously determined that the middle transverse beam supports half of W_{p1} with the distribution as shown in Figure J2-3. Therefore, the reaction can be taken as $R_u = (1/2)38.2^k = 19.1^k$ (89.96KN). One design will be made for this beam and used throughout for all transverse beam connections. Formulas are taken from Part 9 of AISC LRFD volume II, 2nd edition.

Relevant properties of a W10x22 are as follows;

$t_w = 0.240$-in (6.1mm) $b_f = 5.750$-in (146.1mm)

$t_f = 0.360$-in (9.1mm) d = 10.17-in (258.3mm)

Determine coping dimensions c and d_c;

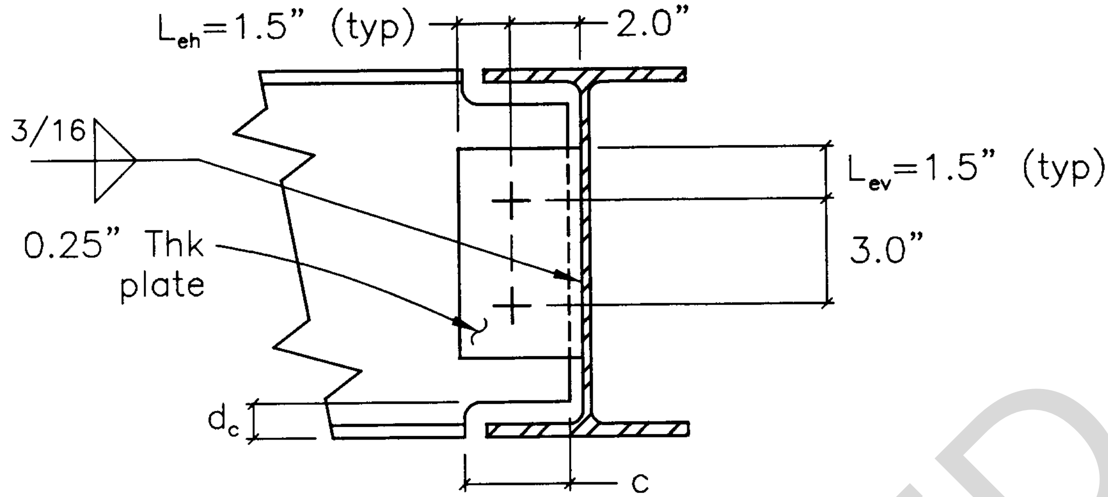
$$c = \frac{5.750''-0.240''}{2} = 2.76 - in \quad (70.1mm) \qquad \text{Say } c = 3.0\text{-in} \;\; (76.2mm)$$

$d_c = 0.360'' + 0.500'' = 0.86''$ (21.8mm) Say $d_c = 1.0$-in (25.4mm)

Check flexural yielding of coped section assuming two 7/8-in. (22.2mm) ϕ bolts;

$$R_u \le \frac{\phi_b M_n}{e} \qquad \text{where;} \quad \phi = 0.9$$

$$M_n = F_y S_{net}$$



Note: For metric equivalents; 1-in = 25.4mm
Figure J2-5. Transverse beam connection, elevation view

$$S_{net} = \frac{1}{12}(0.240")(10.17"-2(1.0"))^3\left(\frac{2}{(10.17"-2(1.0"))}\right) = 2.67-in^3 \quad (43.8X10^3\ mm^3)$$

$$\therefore M_n = 36ksi(2.67-in^3) = 96.1^{in-k} \quad (10.86KN\text{-}m)$$
$$e = c + 0.5" = 3.5" \quad (88.9mm)$$

$$\frac{\phi_b M_n}{e} = \frac{0.9(96.1^{in-k})}{3.5"} = 24.7^k > 19.1^k = R_u \quad (109.9KN > 85.0KN)$$ **O.K.**

Check local web buckling of coped section;

$$R_u \leq \frac{\phi_b M_n}{e}$$ where; $\phi = 0.9$

$$M_n = F_{bc}S_{net}$$

$$F_{bc} = 0.62\pi E\frac{t_w^2}{ch_0}f_d$$

$$f_d = 3.5 - 7.5\left(\frac{d_c}{d}\right) = 3.5 - 7.5\left(\frac{1.0"}{10.17"}\right) = 2.76$$

$$h_0 = 10.17" - 2(1") = 8.17" \quad (207.5mm)$$

$$\therefore F_{bc} = \frac{0.62\pi(29{,}000ksi)(0.240")^2}{3.0"(8.17")}(2.76") = 366ksi \quad (2.52X10^3\ MPa)$$

$$\therefore M_n = 366ksi(2.67-in^3) = 977^{in-k} \quad (110.4KN\text{-}m)$$

$$\frac{\phi_b M_n}{e} = \frac{0.9(977^{in-k})}{3.5"} = 251^k > 19.1^k = R_u \quad (1.12MN > 0.085MN)$$ **O.K.**

Design bolts, single plate, and welds;

Note: The tank being welded along the top of the beam prevents the beam from rotating thereby making the support for the connection relatively rigid.

From Table 9-10 of volume 2 of AISC LRFD 2nd edition, assuming the girder provides a rigid support, for 7/8-in. (22.2mm) diameter A325-N bolts and single plate material with F_y = 36ksi (248.2MPa) and F_u = 58ksi (400MPa), select two rows of bolts, 1/4-in. (6.4mm) single plate thickness, and 3/16-in. (4.8mm) fillet weld size.

$$\phi R_n = 25.5^k > 19.1^k = R_u \quad (113.4KN > 85KN)$$ **O.K.**

Check supported beam web;
From table 9-2 of volume 2 of AISC LRFD 2nd edition, for two rows of 7/8-in. (22.2mm) diameter bolts, beam material with F_y = 36ksi (248.2MPa) and F_u = 58ksi (400MPa), and L_{ev} = 1-1/2-in. (38.1mm) minimum and L_{eh} = 1-1/2-in. (38.1mm) minimum;

$$\phi R_n = 104^{k/in}(0.240") = 25^k > 19.1^k = R_u \quad (11.2KN > 85.0KN)$$

O.K.

Use two 7/8-in. (22.2mm) ϕ bolts with 1/4-in. (6.4mm) single plate and 3/16-in. (4.8mm) welds as shown in Figure J2-5

Design longitudinal beam to column connection (see Figure J2-2);
By inspection, since the end reactions are the same, and the connection to a column is also a rigid connection, the same single plate connection design connecting the transverse beams to the longitudinal beams can be used for this connection.

Use two 7/8-in. (22.2mm) ϕ bolts with 1/4-in. (6.4mm)single plate and 3/16-in. (4.8mm) welds as shown in Figure J2-5

Note: The following steps do not have a one to one correspondence to steps listed in table 4-6.

F.1 *Determine seismic force effects.*

Seismic forces (F_p) shall be determined in accordance with chapter 10 as follows:

$$F_p = \frac{0.4a_p S_{DS} W_p}{R_p / I_p}\left(1 + 2\frac{z}{h}\right) \quad \text{(EQ. 10-1)}$$

where; z/h = 1.0 (Equipment attached at roof of building)

F_p is not required to be taken greater than:

$$F_p = 1.6 S_{DS} I_p W_p \quad \text{(EQ. 10-2)}$$

nor less than:

$$F_p = 0.3 S_{DS} I_p W_p \quad \text{(EQ. 10-3)}$$

Seismic load at the tank;

$$\therefore F_{p1} = \frac{0.4(2.5)0.92(54.6^k)}{2.5/1.5}(1+2(1)) = 1.66(54.6^k) = 90.4^k \quad (402.1KN)$$

$$(F_p)_{max} = 1.6(0.92)1.5(54.6^k) = 121^k > 90.4^k = F_p \quad (538.2KN > 402.1KN)$$ **O.K.**

$$(F_p)_{min} = 0.3(0.92)1.5(54.6^k) = 22.6^k < 90.4^k = F_p \quad (100.5KN < 402.1KN)$$ **O.K.**

Seismic load at the platform level;

$$\therefore F_{p2} = \frac{0.4(2.5)0.92(5.0^k)}{2.5/1.5}(1+2(1)) = 1.66(5.0^k) = 8.3^k \quad (36.9KN)$$

$$(F_p)_{max} = 1.6(0.92)1.5(5.0^k) = 11^k > 8.3^k = F_p \quad (48.9KN > 36.9KN)$$ **O.K.**

$$(F_p)_{min} = 0.3(0.92)1.5(5.0^k) = 2.1^k < 8.3^k = F_p \quad (9.3KN < 36.9KN)$$ **O.K.**

F.2 *Design members.*

Design tank connections to support frame (see Figure J2-6);
Connections, to resist overturning and shear, will consist of welds at four opposite faces of the tank and located at the mid center of the perimeter beams. Loading is eccentric to the plane of the welds.

Try 3/16-in. (4.8mm) welds;
Capacity for a 3/16-in. (4.8mm) weld;

$$\phi R_n = 0.75 t_e (0.6 F_{exx})$$

where; $t_e = \frac{3"}{16}\left(\frac{1}{\sqrt{2}}\right) = 0.133-\text{in} \quad (3.38\text{mm})$

$F_{exx} = 70\text{ksi} \quad (482.7\text{MPa}) \quad (\text{E70XX electrodes})$

$$\therefore \phi R_n = 0.75(0.133")(0.6(70\text{ksi})) = 4.19^{k/in} \quad (0.73\text{KN/mm})$$

By inspection, load combination 'U = 0.9D + E' governs, and the overturning moment is calculated as;

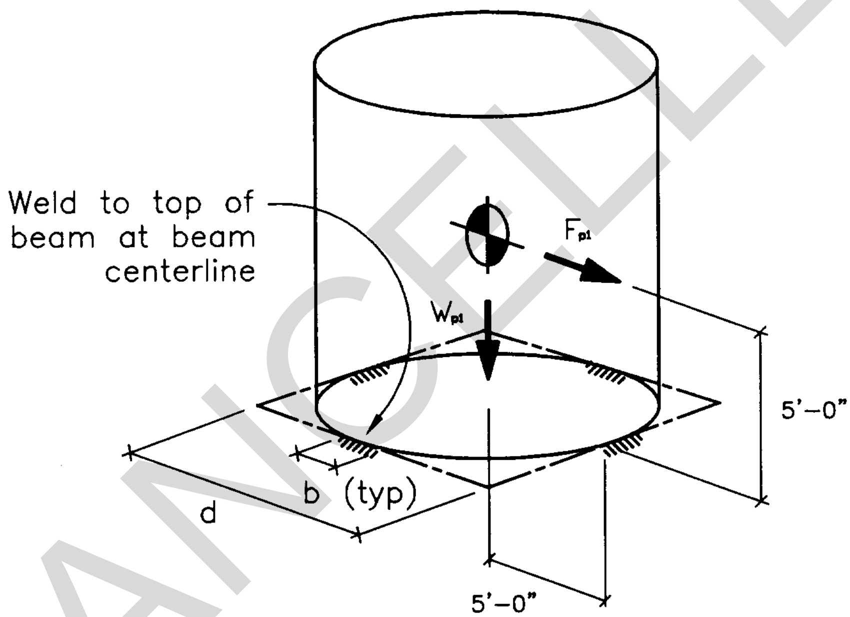
$$\therefore M_u = \left[90.4^k(5') - 0.9(54.6^k)5'\right](12"/1') = 2{,}476^{in-k} \quad (433.6\text{KN/mm})$$

The section modulus for the weld group is calculated as follows;

$$S_{welds} = (I_o + Ad^2)/(d/2)$$

$$S_{welds} \approx 2x\left[(1")b\left(\frac{d}{2}\right)^2 + \frac{1}{12}(1")b^3\right] \div \left(\frac{d}{2}\right) = bd + \frac{b^3}{3d}$$

Try b = 8-in. (203.2mm) (note that d = 120-in. (3.05m))



Note: For metric equivalents; 1-ft = 0.30m
Figure J2-6. Force diagram for tank to platform welds

$$\therefore S_{welds} = 8"(120") + \frac{(8")^3}{3(120")} = 961-\text{in}^2 \quad (620.0\text{X}10^3\ \text{mm}^2)$$

$$f_s = \frac{P_u}{A} = \frac{90.4^k}{4(8")} = 2.83^{k/in} \quad (0.50\text{KN/mm})$$

$$f_b = \frac{M_u}{S_{welds}} = \frac{2{,}476^{in/k}}{961-\text{in}^2} = 2.58^{k/in} \quad (0.45\text{KN/mm})$$

$$R_u = f_r = \sqrt{f_s^2 + f_b^2} = \sqrt{(2.83^{k/in})^2 + (2.58^{k/in})^2} = 3.83^{k/in} < 4.19^{k/in}$$

(0.67KN/mm < 0.73KN/mm) **O.K.**

Use four 3/16-in. (4.8mm) welds to secure tank to perimeter beams

Check transverse perimeter beams;
The transverse perimeter beams support relatively little of the tanks dead weight, but resist the overturning reaction. This reaction was previously calculated to be $P_u = 20.7^k$ (92.1KN). These beams were sized in the gravity load design as W10x22's and will be checked here. It is conservatively assumed that the beams support only the overturning seismic reaction.
The maximum moment occurs at mid span as;

$$M_u = \frac{P_u L}{4} = \frac{20.7^k(10')(12''/1)}{4} = 621^{in-k} \quad (70.17\text{KN-m})$$

Note: Beam is laterally supported throughout its span

$$Z_{req'd} = \frac{M_u}{f F_y} = \frac{621^{in-k}}{0.9(36ksi)} = 19.2 - in^3 < 26.6 = Z_{W10x22}$$

$(314.6X10^3\ mm^3 < 435.9X10^3\ mm^3)$ **O.K.**

Keep W10x22 (W254mmX1.05KN/m) for transverse beams

Check longitudinal beams;
The longitudinal beams support all of the tanks dead weight (transferred to it from the interior perimeter beams), and also resist the overturning reaction of $P_u = 20.7^k$ (92.1KN). These beams were sized in the gravity load design as W10x22's and will be checked here.
Per load combination 'U = 0.9D + E' the center load P_{1u} in figure J2-4 is reduced to an uplift load of;

$20.7^k - 19.1^k = 1.6^k$ (7.12KN) (uplift)

By inspection, this reduces the end reactions and the maximum moment acting within the beam. Therefore, the W10x22 is still adequate.

Keep W10x22 for longitudinal beams

Design transverse beam to column connection;
The worst case beam reaction is $R_u/2 = 20.7^k/2 = 10.4^k$ (46.3KN). By inspection, the same single plate connection used for the other beam to beam or beam to column connections is adequate.

Use two 7/8-in. (22.2mm) f **A325-N bolts with 1/4-in. (6.4mm) single plate and 3/16-in. (4.8mm) welds similar to Figure J2-5**

Check column for combined loading (see Figure J2-7);
Determine design loads;
Calculate reactions;
From symmetry;

$R_{1H} = R_{2H} = (1/2)(45.2^k + 4.15^k) = 24.7^k$ (109.9KN)

$$\sum M_2 = 0;$$

$$(R_{1V})10' - 45.2^k(15') - 4.15^k(10') = 0$$

$R_{1V} = 72^k$ (320.3KN) (tension)

$$\sum F_y = 0;$$

$R_{2V} = 72^k$ (compression)

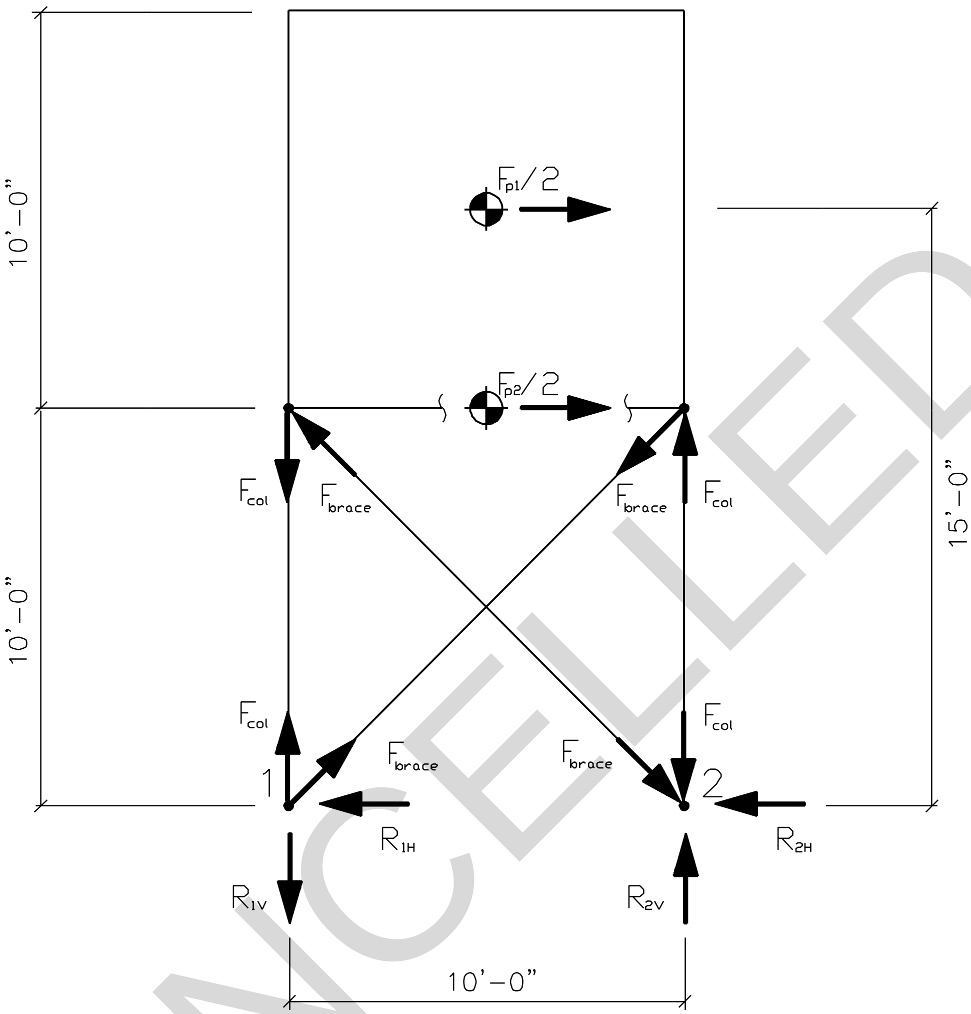
Calculate compressive force in column;
Summation of loads at point 2;
Due to the 45 degree inclination of the brace;

$$(F_{brace})_{horz} = (F_{brace})_{vert} = 24.7^k \quad (109.9\text{KN})$$

$$\sum F_y = 0;$$

$$(F_{col})_{vert} = R_{2V} - (F_{brace})_{vert} = 72^k - 24.7^k = 47.3^k \quad (210.4\text{KN})$$

Superimposing the dead load;
From load combination 'U = 1.2D + E';



Note: For metric equivalents; 1-ft = 0.30

Figure J2-7. Seismic force diagram for supporting legs and braces

Dead load = $(W_{p1} + W_{p2})/4 = (54.6^k + 5^k)/4 = 14.9^k$ (66.3KN)

$P_u = 1.2(14.9^k) + 47.3^k = 65.2^k$ (290.0KN)

Check column capacity;

$$f_c P_n = f_c F_{cr} A_g = 129.4^k \quad \text{(575.6KN) (calculated previously)}$$

$$f_c P_n = 129.4^k > 65.2^k = P_u \quad \text{(575.6KN > 290.0KN)}$$ **O.K.**

Design brace;

Determine deign loads (see Figure J2-7);

Note: Brace does not support gravity loads. Therefore, all load combinations reduce to $P_u = E$.

$$F_{brace} = \pm\sqrt{(F_{brace})^2_{horz} + (F_{brace})^2_{vert}} = \pm\sqrt{2(24.7^k)^2} = \pm 34.9^k \quad \text{(155.2KN)}$$

Compression;

A doubly symmetric section will be choosen. Therefore, per Figure 7-21, the out-of-plane buckling will govern capacity. Assume K = 0.67;

$$KL_b = 0.67(10')\sqrt{2} = 9.5' \quad (2.90\text{m})$$

Try a 3-in. (76.2mm) ϕ standard weight pipe;
From the columns tables in Part 3 of the AISC LRFD manual;

$$\phi_c P_n = 41^k > 34.9^k = P_u \quad (182.4\text{KN} > 155.2\text{KN})$$ **O.K.**

Tension;
Okay by inspection.

Use 3-in. (76.2mm) std wt pipe for braces

Design brace connection to column and beam (see Figure J2-8);
One design for the worst case situation will be used throughout the structure. Worst case occurs where the gusset plate attaches to the column web (this condition has the smallest buckling capacity due to the increased length of plate). Per the AISC Seismic provisions (dated April 15, 1997) section 13.2b, the required compressive strength is $\phi_c P_n$, and per section 13.3.a, the required tensile strength is $R_y F y A_g$.

Therefore;
Compressive design load = $\phi_c P_n = 41^k$ (182.4KN)
Tensile design load = $R_y F_y A_g = 1.5(46\text{ksi})2.23\text{-in}^2 = 154^k$ (685.0KN)

Try a 3/8-in. (9.5mm) thick gusset plate (F_y = 36ksi (248.2MPa));
Design welds to gusset plate using E70XX electrodes
Minimum weld size = 3/16-in. (4.8mm) (per AISC LRFD 2nd edition Table J2.4, based on gusset plate thickness)
Maximum weld size = 0.216-in. (5.49mm) (per AISC LRFD 2nd edition section J2.2b.(b), based on brace thickness)
Required length of four 3/16-in. fillet welds;

$$L_w = \frac{R_u}{\phi R_n} = \frac{154^k / 4-\text{sides}}{4.19^{k/in}} = 9.19" \quad \text{Say } 9.25" \ (235.0\text{mm})$$

Check base metal;

$$\phi R_n = \phi(0.6F_y)A_g = 0.9(0.6(36^{ksi}))0.216"(9.25")4 = 155^k \quad (689.4\text{KN})$$

$$\phi R_n = 155^k > 154^k = P_u \quad (689.4\text{KN} > 685.0\text{KN})$$ **O.K.**

Check shear tension rupture (refer to Figure J2-9 for nomenclature);

$$\phi R_n = \phi t_{brace}\left[0.6F_y\left(\frac{2l_w}{\cos 30°}\right) + F_u w_l\right]$$

where; $w_l = \phi_{brace} + 2l_w(\tan 30°) = 3.5"+2(9.25")0.577 = 14.2"$ (360.7mm)

$$\therefore \phi R_n = 0.75(0.375")\left[0.6(36\text{ksi})\left(\frac{2(9.25")}{0.866}\right) + 58\text{ksi}(14.2")\right] = 361^k > 154^k = P_u$$

(1.61MN > 0.68MN) **O.K.**

or, $\phi R_n = \phi t(0.6F_u(2l_w) + F_y w_b)$

$$\phi R_n = 0.75(0.375")(0.6(58\text{ksi})(2(9.25") + 36\text{ksi}(3.5")) = 217^k > 154^k = P_u$$

(0.97MN > 0.68MN) **O.K.**

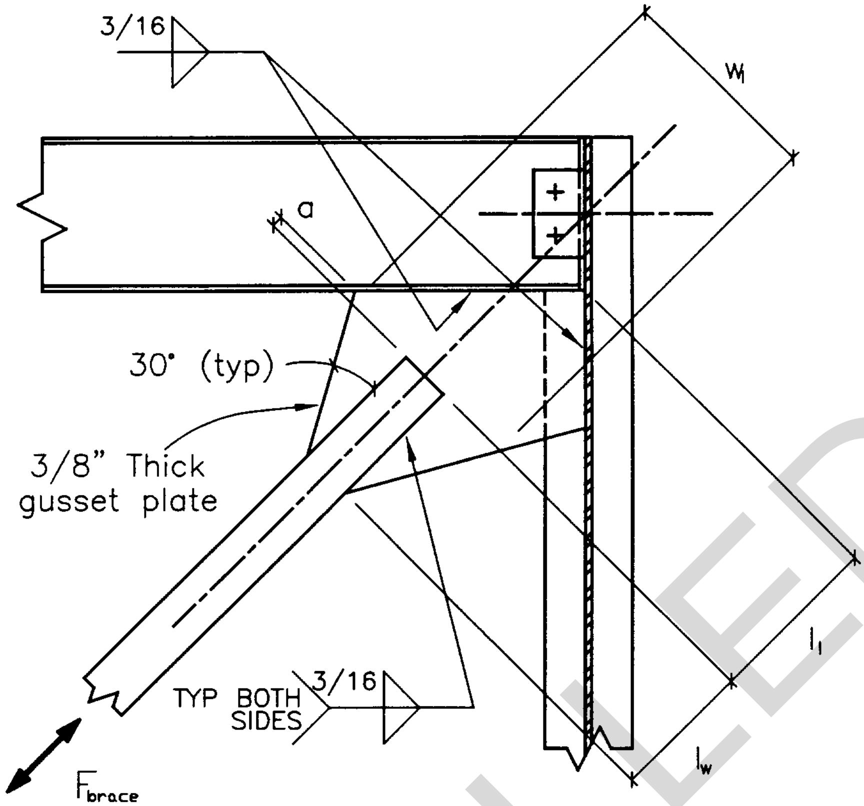
Check tensile capacity;

$$\phi R_n = \phi w_l F_u t = 0.75(14.2")58\text{ksi}(0.375") = 232^k > 154^k = P_u$$

(1.03KN > 0.68KN) **O.K.**

Check compressive capacity;

Note: The critical buckling strength for the brace is $A_g F_{cr} = \frac{\phi_c P_n}{\phi_c} = \frac{41^k}{0.85} = 48.2^k$ (214.4KN)



Note: For metric equivalents; 1-in = 25.4mm

Figure J2-8. Brace connection details and nomenclature

$$\phi R_n = \phi F_y w_1 t = 0.9(36\text{ksi})14.2''(0.375'') = 173^k > 48.2^k = A_g F_{cr}$$

(769.5KN > 214.4KN) **O.K.**

Check buckling of gusset plate;

$$\phi R_n = \phi\left(\frac{4{,}000t^3\sqrt{F_y}}{l_1}\right) \ge A_g F_{cr}$$

where; $l_1 = \dfrac{w_1}{2} + a(\tan 30^\circ) + \dfrac{d_b}{2}(\sin 45^\circ)$ (for this bracing configuration only)

$a = 2t = 2(0.375") = 0.75"$ (19.1mm)

$l_1 = \dfrac{14.2''}{2} + 0.75''(0.577) + \dfrac{10.17''}{2}(0.707) = 11.1''$ (281.9mm)

$$\phi R_n = 0.90\left(\frac{4{,}000(0.375'')^3\sqrt{36\text{ksi}}}{11.1''}\right) = 103^k > 48.2^k = A_g F_{cr}$$

(458.1KN > 214.4KN) **O.K.**

Design weld of gusset plate to beam and column;

Note: Length of weld to be along the entire length of contact between gusset plate and beam or column;

$(F_{brace})_{horz} = (F_{brace})_{vert} = 24.7^k$ (109.9KN)

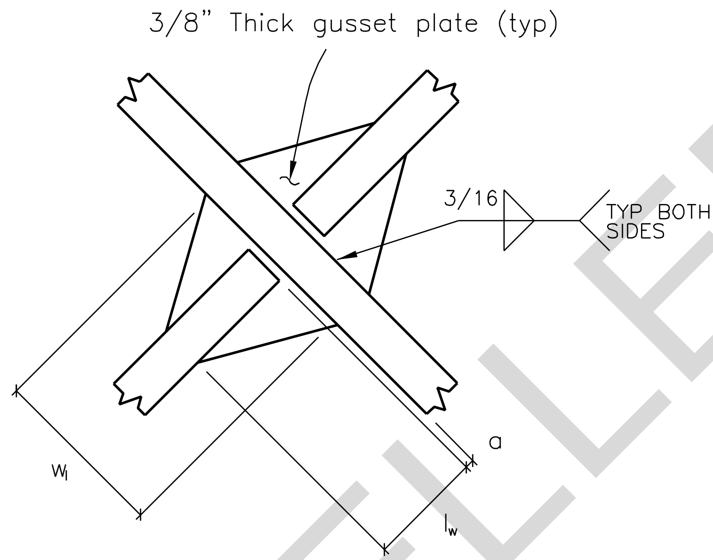
$L_{provided} \ge (w_1 + 2a(\tan 30^\circ))\cos 45^\circ$ (for this bracing configuration only)

$L_{provided} \ge (14.2''+2(0.75'')0.577)0.707 = 10.7''$ (271.8mm)

Required length of two 3/16-in. (4.8mm) fillet welds;

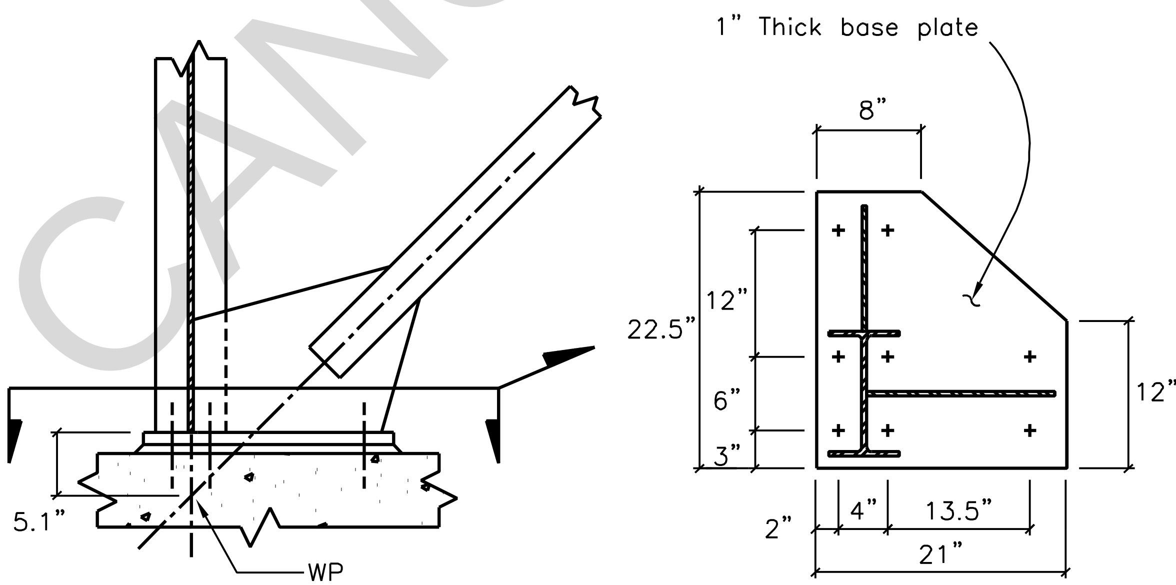
$$(L_w)_{req'd} = \frac{R_u}{\phi R_n} = \frac{24.7^k / 2 - \text{sides}}{4.19^{k/in}} = 2.95'' << 10.7'' \le L_{provided} \quad (74.9\text{mm} << 271.8\text{mm}) \qquad \textbf{O.K.}$$

Design brace to brace connection (see Figure J2-9);
Use similar design as used at the brace connection to the column and beam. By inspection, the connection meets the requirements of shear/tension rupture, tensile capacity, and compressive capacity. Also, buckling of the gusset plate does not govern because l_l is so much less.



Note: For metric equivalents; 1-in = 25.4mm
Figure J2-9. Brace to brace connection

Design gusset plate to column and column base plate (see Figure J2-10);
Use similar design as used at the column to beam connection. All parameters are the same except the gusset plate attaches to the column base plate instead of the beam.



Note: For metric equivalents; 1-in = 25.4mm
Figure J2-10. Brace to column and column base plate connection

Design column base plate;

For compressive loading, the base plate is approximated as a rectangular plate enclosing only the column cross section and having dimensions b_f + 1" (25.4mm) by d + 1" (25.4mm). Part 11 of AISC LRFD 2nd edition outlines procedures for designing base plates that will be used here.

Determine design loads;

$R_u = 72^k$ (compression or tension)

Area of base plate = BN = (5.75" + 1")(10.17" + 1") = 75.4-in^2 (46.6X10^3 mm^2)

$$(A_1)_{req'd} = \frac{R_u}{\phi_c(0.85f_c')}$$

where; $\phi_c = 0.6$

f_c' = 3,500psi (24.13MPa) (assumed)

$$\therefore (A_1)_{req'd} = \frac{72^k}{0.6(0.85)3.5\text{ksi}} = 40.3 - \text{in}^2 < 75.4 - \text{in}^2 = BN$$ (26.2X10^3 mm^2 < 46.6X10^3 mm^2) **O.K.**

Determine required plate thickness for compressive loading;

$$t_{req'd} = l\sqrt{\frac{2R_u}{0.9F_yBN}}$$

where; $l = \lambda n'$ (small base plate)

$$n' = \frac{\sqrt{db_f}}{4}$$

$$\lambda = \frac{2\sqrt{X}}{1+\sqrt{1-X}} \le 1$$

$$X = \left(\frac{4db_f}{(d+b_f)^2}\right)\frac{P_u}{\phi_c P_p}$$

$\phi_c = 0.6$ (LRFD Specification section J9)

$P_p = 0.85f_c'A_1$ (load acts on full area of concrete support)

$P_p = 0.85(3.5\text{ksi})94.3\text{-in}^2 = 281^k$ (1.28MN)

$$\therefore X = \left(\frac{4(10.17")5.75"}{(10.17"+5.75")^2}\right)\frac{72^k}{0.6(281^k)} = 0.394$$

$$\lambda = \frac{2\sqrt{0.394}}{1+\sqrt{1-0.394}} = 0.706$$

$$n' = \frac{\sqrt{10.17"(5.75")}}{4} = 1.91"$$ (48.5mm)

$l = 0.706(1.91") = 1.35"$ (34.3mm)

$$\therefore t_{req'd} = \sqrt{\frac{2(72^k)}{0.9(36\text{ksi})94.3 - \text{in}^2}} = 0.22"$$ (5.6mm)

Determine required base plate thickness for tensile load;

Closest spacing of bolts is 4-in. (see Figure J2-10) and plate is assumed to span between these bolts;

$$M_u = \frac{P_uL}{4} = \frac{72^k(4")}{4} = 72^{in-k}$$ (8.14KN-m)

$$\phi_b M_n \ge M_u$$

where; $\phi_b = 0.9$

$M_n = Z_xF_y$

$$Z_x = \frac{Nt^2}{4}$$

$$\therefore \phi_b M_n = 0.9\left(\frac{Nt^2}{4}\right)F_y = M_u \Rightarrow t = \sqrt{\frac{4.44M_u}{NF_y}} = \sqrt{\frac{4.44(72^{in-k})}{(10.17"+2")36ksi}} = 0.85 \qquad \text{(governs)}$$

Use 1.0-in. (25.4mm) thick base plate

Design anchor bolts;
Note: The four bolts surrounding the column will be conservatively designed for the entire tension load, but all eight bolts will be assumed active in resisting shear.
Determine design loads;

$$V_u = \frac{24.7^k}{8-\text{bolts}} = 3.09^{k/bolt} \quad (13.7\text{KN/bolt})$$

From load combination 'U = 0.9D + E';

Dead load = $(W_{p1} + W_{p2})/4 = (54.6^k + 5^k)/4 = 14.9^k$ (66.3KN)

$P_u = 0.9(-14.9^k) + 72^k = 58.6^k$ (260.7KN) (tension)

Try 1.0-in. (25.4mm) ϕ A307 bolts;
Check capacity in shear;
Steel;

$$V_s = (0.75A_bF_u)n \qquad \text{(EQ. 9.2.4.2-1 FEMA 302)}$$

For 1.0" ϕ bolt (A 307)

where; $A_b = 0.785\text{-in}^2$ ($506.5mm^2$)
$F_u = 60ksi$ (413.7MPa)
n = 8-bolt

$$V_s = 0.75(0.785-\text{in}^2)60ksi(8-\text{bolts}) = 283^k > 24.7^k = V_u \quad (1.26MN > 0.11MN) \qquad \textbf{O.K.}$$

Concrete;

$$\phi V_c = (\phi 800A_b\lambda\sqrt{f_c'})n \qquad \text{(EQ. 9.2.4.2-2 FEMA 302)}$$

where; ϕ = 0.65
λ = 1.0 (normal weight concrete)
f_c' = 3,500psi
n = 8-bolts

$$\therefore \phi V_c = 0.65(800)0.785-\text{in}^2(8)\sqrt{3{,}500psi} = 0.65(297^k) = 193^k \quad (0.86MN)$$

$$\phi V_c = 193^k > 24.7^k = V_u \quad (0.86MN > 0.11MN) \qquad \textbf{O.K.}$$

Check capacity in tension;
Steel;

$$P_s = (0.9A_bF_u)n \qquad \text{(EQ. 9.2.4.1-1 FEMA 302)}$$

where; $A_b = 0.785\text{-in}^2$ ($506.5mm^2$)
$F_u = 60ksi$ (413.7MPa)
n = 4-bolts

$$\therefore P_s = 0.9(0.785-\text{in}^2)60ksi(4-\text{bolts}) = 170^k > 58.6^k = P_u \quad (0.76MN > 0.26MN) \qquad \textbf{O.K.}$$

Concrete;

$$\phi P_c = \phi\lambda\sqrt{f_c'}(2.8A_p + 4A_T) \qquad \text{(EQ. 9.2.4.1-3 FEMA 302)}$$

where; ϕ =0.65
λ = 1.0 (normal weight concrete)
f_c' = 3,500psi (24.13MPa)
$A_T = (4")6" = 24.0\text{-in}^2$ ($15.5X10^3\ mm^2$)

$$A_p = 2(8")\sqrt{2"}[4"+6"] + 4\left[8"\sqrt{2}\right]^2 = 738-\text{in}^2 \quad (476X10^3\ mm^4)$$

(for a 8-in. (203.2mm) embedment)

$$\therefore \phi P_c = 0.65(1.0)\sqrt{3{,}500\text{psi}}\left[2.8(738-\text{in}^2)+4(24-\text{in}^2)\right] = 0.65(128^k) = 83^k > 58.6^k = P_u$$

(369.2KN > 260.7KN) **O.K.**

Check combined tension and shear;
Per section 9.2.4.3 of FEMA 302 , all of the following conditions shall be met;

condition (a) $\frac{1}{\phi}\left(\frac{V_u}{V_c}\right) \le 1.0$ (EQ. 9.2.4.3-1a FEMA 302)

$$\frac{1}{0.65}\left(\frac{24.7^k}{297^k}\right) = 0.13 \le 1.0$$ **O.K.**

condition (b) $\frac{1}{\phi}\left(\frac{P_u}{P_c}\right) \le 1.0$ (EQ. 9.2.4.3-1b FEMA 302)

$$\frac{1}{0.65}\left(\frac{58.6^k}{128^k}\right) = 0.20 \le 1.0$$ **O.K.**

condition (c) $\frac{1}{\phi}\left[\left(\frac{P_u}{P_c}\right)^2 + \left(\frac{V_u}{V_c}\right)^2\right] \le 1.0$ (EQ. 9.2.4.3-1c FEMA 302)

$$\frac{1}{0.65}\left[\left(\frac{57.1^k}{128^k}\right)^2 + \left(\frac{24.7^k}{297^k}\right)^2\right] = \frac{1}{0.65}(0.199+0.007) = 0.32 \le 1.0$$ **O.K.**

condition (d) $\left(\frac{P_u}{P_s}\right)^2 + \left(\frac{V_u}{V_s}\right)^2 \le 1.0$ (EQ. 9.2.4.3-1d FEMA 302)

$$\left(\frac{58.6^k}{339^k}\right)^2 + \left(\frac{24.7^k}{283^k}\right)^2 = (0.030+0.008) = 0.038 \le 1.0$$ **O.K.**

Use 1.0-in. (25.4mm) ϕ A307 anchor bolts

J-3 PIPE BRACING

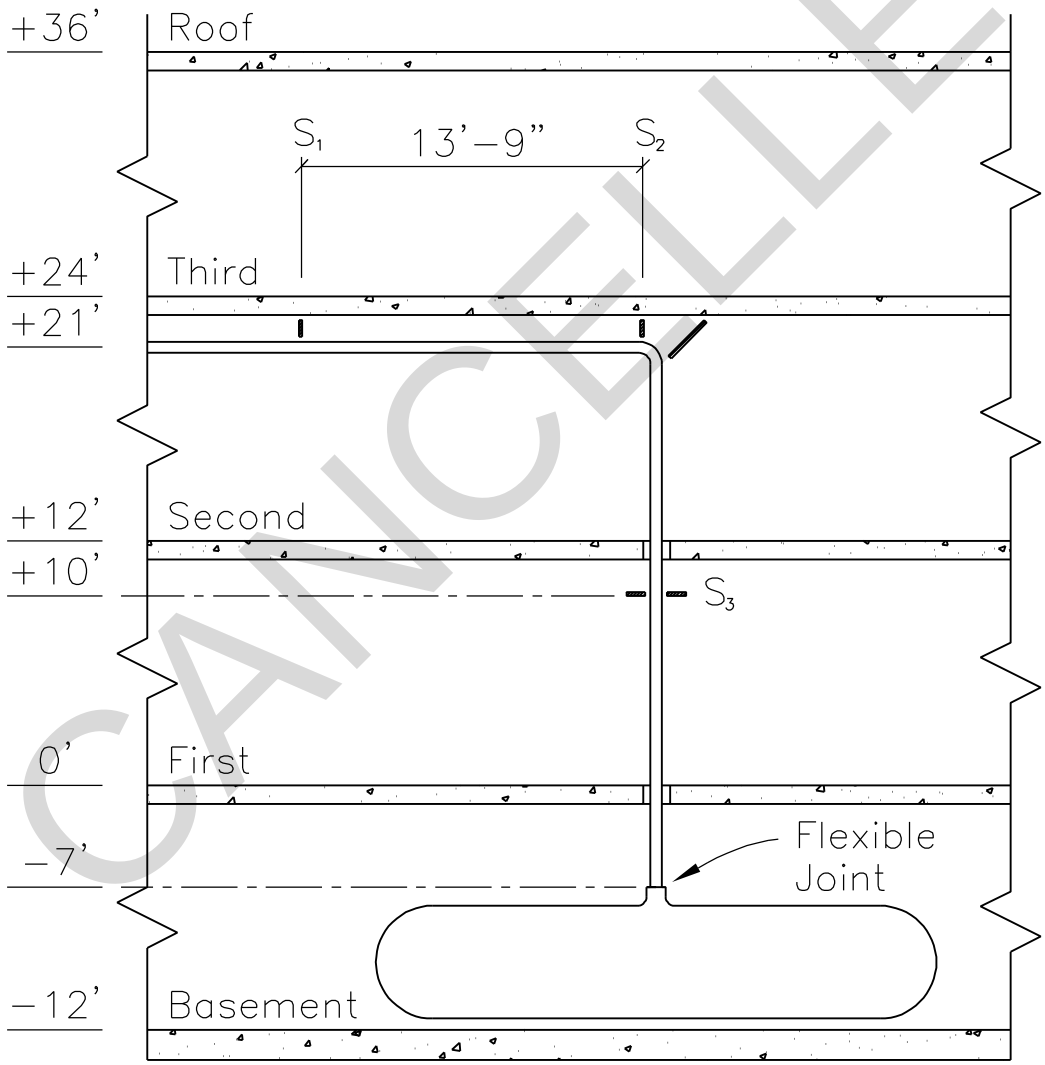
a. Introduction.

(1) Purpose. The purpose of this example problem is to illustrate the design of pipe bracing using Chapter 10 and Chapter 6 of FEMA 302 (Components).

(2) Scope. The problem generally follows the steps in Table 4-5 to analyze the pipe bracing and anchorage. Typical bracing details may be found in Figure 10-8

Component description.

The steel water pipe used in this example problem is a 6-inch (152.4mm) diameter standard wall pipe, extending from the basement to the second floor of a three story concrete building (see Figure J3-1). The piping is used to distribute chilled water for HVAC. The building and equipment performance objective is life safety (LS). Additionally, the equipment is not required to function after an earthquake.



Note: For metric equivalent; 1-ft = 0.30m
Figure J3-1. Water pipe and bracing elevation

b. Component design.

A.1 *Determine appropriate Seismic Use Group*

Because the equipment is not required to be functional after an earthquake, the water pipe bracing is given a performance level of life safety (LS). The Seismic Use Group and other performance parameters are determined from Table 4-4, as follows;

Performance Level:	LS	(per problem statement)
Seismic Use Group:	I	(Table 4-4)
Ground Motion:	2/3 MCE (A)	(Table 4-4)
Performance Objective:	1A	(Table 4-4)

A.2 *Determine site seismicity.*

The following values are assumed for this example:

$S_S = 1.20g$ (MCE Maps)

A.3 *Determine site characteristics.*

Soil type D is assumed for this problem

Soil type: D (Table 3-1)

A.4 *Determine site coefficients.*

$F_a = 1.02$ (interpolated) (Table 3.2a)

A.5 *Determine adjusted MCE spectral response accelerations.*

$S_{MS} = F_aS_S = 1.02(1.20)g = 1.22g$ (EQ. 3-1)

A.6 *Determine design spectral response accelerations.*

$S_{DS} = 2/3\ S_{MS} = 2/3(1.22) = 0.81g$ (EQ. 3-23)

A.7 *Bracing system.*

The bracing system will consist of structural steel bracing assumed to be installed using low deformability anchor bolts in concrete. To ensure rigidity, the pipe spans are limited to values shown if Figures 10-6 of this document.

A.8 *Select R_p, a_p, and I_p factors.*

$a_p = 1.0$ (Table 10-1)

$R_p = 1.25$ (Table 10-1)

$I_p = 1.0$ (per paragraph 10-1d)

A.10 *Determine member sizes for gravity load effects.*

Note: Only supports S_1 and S_2 support gravity loads. Additionally, connection design will be postponed until the seismic analysis is complete.

Dead load acting at supports is determined as follows;

a. Determine unit weight of pipe;

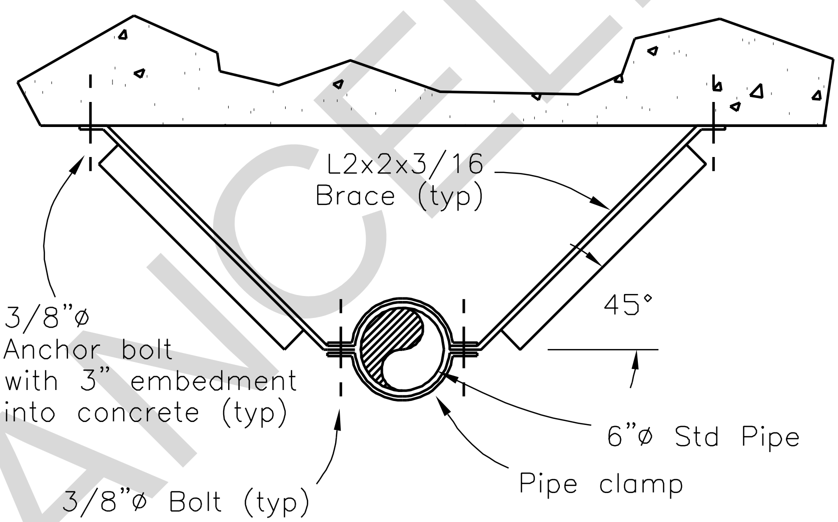
6" ϕ pipe;	19plf
Flanges;	4plf
Supports;	2plf
Water;	<u>20plf</u>
Total =	45plf (0.66KN/m)

b. Determine required support spacing;
Use 13'-9" (4.19m) for a pinned-pinned rigid pipe per Figure 10-5.

c. Determine dead loads;
$W_{P@S1}$ = 13.75'(45plf) = 619-lbs (2.75KN) (typical for horizontal spans)
$W_{P@S2}$ = (13.75' + 28')(1/2)45plf = 939-lbs (4.18KN) (governs for gravity load design)

Therefore, one design, based on the worst case loading at location S_2, will be used throughout for gravity loads.

Member design (see Figure J3-2);



Note: For metric equivalent; 1-in = 25.4mm
Figure J3-2. Transverse pipe restraint

Factored dead load acting at support; $P_u = 1.4W_{S2}$ =1.4(939-lbs) = 1,315-lbs, or 1.32^k (5.85KN)

For two braces inclined at 45 degrees (one on either side of pipe), the ultimate load per brace is determined as;

$$P_u = \frac{(1.32^k)\sqrt{2}}{2} = 0.93^{k/brace} \quad (4.14KN/brace)$$

Try L2x2x3/16 (L50.8mmX50.8mmX4.8mm) (Area = 0.715-in^2 (461.2mm^2)) attached using one 3/8" (9.53mm) ϕ bolt per connection;

Check yielding on the gross area;

$$f_t P_n = f_t F_y A_g \qquad \text{where;} \quad f_t = 0.90 \qquad \text{(EQ. D1-1 AISC LRFD)}$$

$$\phi_t P_n = 0.90F_yA_g = 0.9(36ksi)0.715-in^2 = 23.2^k > 0.93^k = P_u \quad (103.2KN > 4.14KN) \qquad \textbf{O.K.}$$

Check fracture on the effective net area;

$\phi_t P_n = \phi_t F_u A_e$ where; $\phi_t = 0.75$ (EQ. D1-2 AISC LRFD)

$A_e = 2"(3/16") - (1/8" + 3/8")(3/16") = 0.281\text{-in}^2$ $(181.2mm^2)$

$$\phi_t P_n = 0.75F_uA_e = 0.75(58ksi)0.281-in^2 = 12.2^k > 0.93^k = P_u \quad (54.3KN > 4.14KN) \qquad \textbf{O.K.}$$

Use 2-L2x2x3/16 (L50.8mmX50.8mmX4.8mm) braces

Note: The following steps do not have a one to one correspondence to steps as listed in table 4-6.

F.1 *Determine seismic force effects.*

Seismic forces (F_p) shall be determined at each pipe location (i.e., S_1, S_2, and S_3) in accordance with chapter 10 as follows:

$$F_p = \frac{0.4a_pS_{DS}W_p}{R_p/I_p}\left(1+2\frac{z}{h}\right) \qquad \text{(EQ. 10-1)}$$

where; $z/h = 21/36 = 0.58$ (Figure J3-1)

W_P: $W_{P@S1}$ = 619-lbs or 0.619^k (2.75KN) (as previously calculated)

$W_{P@S2}$ = 939-lbs or 0.939^k (4.18KN) (as previously calculated)

$W_{P@S3}$ = (21' + 7')(1/2)45plf = 630lb or 0.630k (2.80KN)

F_p is not required to be taken greater than:

$$F_p = 1.6S_{DS}I_pW_p \qquad \text{(EQ. 10-2)}$$

nor less than:

$$F_p = 0.3S_{DS}I_pW_p \qquad \text{(EQ. 10-3)}$$

Seismic load at location S_1;

$$F_{p1} = \frac{0.4(1.0)0.81(0.619^k)}{1.25/1.0}(1+2(0.58)) = 0.56(0.619^k) = 0.347^k \quad (1.54KN)$$

Note: The seismic coefficient is 0.56 and is applicable at all locations along the pipe.

$$(F_{p1})_{max} = 1.6(0.81)1.0(0.619^k) = 0.802^k > 0.347^k = F_{p1} \quad (3.57KN > 1.54KN) \qquad \textbf{O.K.}$$

$$(F_{p1})_{min} = 0.3(0.81)1.0(0.619^k) = 0.150^k < 0.347^k = F_{p1} \quad (0.67KN < 1.54KN) \qquad \textbf{O.K.}$$

Seismic load at location S_2;

$$F_{p2} = 0.56(0.939^k) = 0.526^k \quad (2.34KN)$$

$$(F_{p2})_{max} = 1.6(0.81)1.0(0.939^k) = 1.22^k > 0.526^k = F_{p2} \quad (5.43 > 2.34KN) \qquad \textbf{O.K.}$$

$$(F_{p2})_{min} = 0.3(0.81)1.0(0.939^k) = 0.228^k < 0.526^k = F_{p2} \quad (1.01KN < 2.34KN) \qquad \textbf{O.K.}$$

Seismic load at location S_3;

$$F_{p3} = 0.56(0.636^k) = 0.356^k \quad (1.58KN)$$

$$(F_{p3})_{max} = 1.6(0.81)1.0(0.630^k) = 0.817^k > 0.356^k = F_{p3} \quad (3.63KN > 1.58KN) \qquad \textbf{O.K.}$$

$$(F_{p3})_{min} = 0.3(0.81)1.0(0.630^k) = 0.153^k < 0.356^k = F_{p3} \quad (0.68KN < 1.58KN) \qquad \textbf{O.K.}$$

F.2 *Design members.*

Design for loads transverse to pipe axis (see Figure J3-2);
Note: Supports for loads transverse to pipe axis are located at locations S_1, S_2, and S_3.
At location S_1;
Seismic load per brace;

$$P_u = \frac{(0.347^k)\sqrt{2}}{2} = 0.245^{k/brace} \quad (1.09KN/brace)$$

Dead load per brace;

$$P_u = \frac{(0.619^k)\sqrt{2}}{2} = 0.438^{k/brace} \quad (1.99KN/brace)$$

Load combinations;

U = 1.4D;	$P_u = 1.4(0.438^{k/brace}) = 0.61^{k/brace}$ (2.71KN/brace)
U = 1.2D + E;	$P_u = 1.2(0.438^{k/brace}) + 0.245^{k/brace} = 0.77^{k/brace}$ (3.42KN/brace)
U = 0.9D – E;	$P_u = 0.9(0.438^{k/brace}) - 0.245^{k/brace} = 0.15^{k/brace}$ (0.67KN/brace)

At location S_2;
Seismic load per brace;

$$P_u = \frac{(0.526^k)\sqrt{2}}{2} = 0.372^{k/brace} \quad (1.65KN/brace)$$

Dead load per brace;

$$P_u = \frac{(0.939^k)\sqrt{2}}{2} = 0.664^{k/brace} \quad (2.95KN/brace)$$

Load combinations;

U = 1.4D;	$P_u = 1.4(0.664^{k/brace)} = 0.93^{k/brace}$ (4.14KN/brace)
U = 1.2D + E;	$P_u = 1.2(0.664^{k/brace}) + 0.372^{k/brace} = 1.17^{k/brace}$ (5.20KN/brace) (governs)
U = 0.9D – E;	$P_u = 0.9(0.664^{k/brace}) - 0.372^{k/brace} = 0.23^{k/brace}$ (1.02KN/brace)

At location S_3;
Note: No dead load acts at location S_3.
Seismic load per brace;

$$P_u = \frac{(0.356^k)\sqrt{2}}{2} = 0.25^{k/brace} \quad (1.11KN/brace)$$

Load combinations;

U = E; $P_u = 0.25^{k/brace}$ (1.11KN/brace)

Note: Braces always act in tension.
Check capacity of angle braces;
Check yielding on the gross area;

$$f_t P_n = 0.90 F_y A_g = 0.9(36ksi)0.715-in^2 = 23.2^k > 1.17^k = P_u \quad (103.2KN > 5.20KN) \qquad \textbf{O.K.}$$

Check fracture on the effective net area;

$$f_t P_n = 0.75 F_u A_e = 0.75(58ksi)0.281-in^2 = 12.2^k > 1.17^k = P_u \quad (54.3KN > 5.20KN) \qquad \textbf{O.K.}$$

Use 2-L2x2x3/16 (2-L50.8mmX50.8mmX4.8mm) braces

Design connections to pipe;
Determine design loads;

$$T_u = V_u = \frac{1.17^{k/brace}}{\sqrt{2}} = 0.83^{k/connection} \quad (3.69KN/connection)$$

Try a single 3/8" ϕ A307 bolt in a bearing type connection;
Check shear;

$$fR_n = fF_n A_b$$

where; $f_t = 0.75$ (per AISC LRFD section J3.6)
F_n = 24ksi (per AISC LRFD Table J3.2)

$$A_b = \frac{p(3/8")^2}{4} = 0.11-in^2 \quad (71.0mm^2)$$

$$fR_n = fF_n A_b = 0.75(24ksi)0.11-in^2 = 1.98^k > 0.83^k = V_u \quad (8.81KN > 3.69KN) \qquad \textbf{O.K.}$$

Check bearing;
By inspection, $L_e > 1.5d$;

$R_n = 2.4dtF_u$ with; $f = 0.75$ (EQ. J3-1a AISC LRFD)

$fR_n = 0.75(2.4(3/8")3/16"(58ksi)) = 7.35^k > 0.83^k = V_u$ (32.7KN > 3.69KN) **O.K.**

Check combined tension and shear;
The tension limit stress is;

$59 - 1.9f_v \leq 45\,ksi$ (310.3MPa) (per AISC LRFD Table J3.5)

Therefore,

$$f_v = \frac{P_u}{A_b} = \frac{0.83^k}{0.11 - in^2} = 7.55ksi \quad (52.1MPa)$$

$59ksi - 1.9(7.55ksi) = 44.7ksi < 45ksi$ Therefore, $F_t = 44.7ksi$ (308.2MPa)

$fR_n = fF_tA_b = 0.75(44.7ksi)0.11 - in^2 = 3.69^k > 0.83^k = T_u$ (16.4KN > 3.69KN) **O.K.**

Use a single 3/8" f bolt for connections to pipe

Design connections to concrete;
Note: For anchors in concrete without special inspection, Section 9.2.1 in FEMA 302 requires an additional load factor of 2.0. Therefore;

$P_u = V_u = 2(0.83^k) = 1.66^k$ (7.38KN)

Try a single 3/8" (9.53mm) ϕ A307 bolt in a bearing type connection;
Check capacity in shear;
Steel;

$V_s = (0.75A_bF_u)n$ (EQ. 9.2.4.2-1 FEMA 302)

For 3/8" (9.53mm) ϕ bolt (A 307)

where; $A_b = 0.11\text{-}in^2$ ($71.0mm^2$)
$F_u = 60ksi$ (413.7MPa)
n = 1-bolt

$V_s = 0.75(0.11 - in^2)60ksi(1.0 - bolt) = 4.95^k > 1.66^k = V_u$ (22.0KN > 7.38KN) **O.K.**

Concrete;

$fV_c = (f800A_b\, l\sqrt{f_c'})n$ (EQ. 9.2.4.2-2 FEMA 302)

where; ϕ = 0.65
λ = 1.0 (normal weight concrete)
f_c' = 4,000psi (27.6MPa)
n = 1-bolt

$\therefore fV_c = 0.65(800)0.11 - in^2(1.0)\sqrt{4{,}000psi} = 0.65(5.57^k) = 3.62^k$ (16.1KN)

$fV_c = 3.62^k > 1.66^k = V_u$ (16.1KN > 7.38KN) **O.K.**

Check capacity in tension;
Steel;

$P_s = (0.9A_bF_u)n$ (EQ. 9.2.4.1-1 FEMA 302)

where; $A_b = 0.11\text{-}in^2$ ($71.0mm^2$)
$F_u = 60ksi$ (413.7MPa)
n = 1-bolt

$\therefore P_s = 0.9(0.11 - in^2)60ksi(1 - bolt) = 5.94^k > 1.66^k = P_u$ (26.4KN > 7.38KN) **O.K.**

Concrete;

$fP_c = f\, l\sqrt{f_c'}(2.8A_s)n$ (EQ. 9.2.4.1-2 FEMA 302)

where; ϕ =0.65
λ = 1.0 (normal weight concrete)
f_c' = 4,000psi (27.6MPa)
$A_s = \pi(3")3" = 28.3\text{-}in^2$ ($18.25X10^3\ mm^2$) (for a 3-in. (76.2mm) embedment)

$\therefore \phi P_c = 0.65(1.0)\sqrt{4{,}000\text{psi}}(2.8(28.3-\text{in}^2))(1-\text{bolt}) = 0.65(5.01^k) = 3.26^k > 1.66^k = P_u$

(14.5KN > 7.38KN) **O.K.**

Check combined tension and shear;
Per section 9.2.4.3 of FEMA 302 , all of the following conditions shall be met;

condition (a) $\frac{1}{\phi}\left(\frac{V_u}{V_c}\right) \leq 1.0$ (EQ. 9.2.4.3-1a FEMA 302)

$$\frac{1}{0.65}\left(\frac{1.66^k}{5.57^k}\right) = 0.46 \leq 1.0$$ **O.K.**

condition (b) $\frac{1}{\phi}\left(\frac{P_u}{P_c}\right) \leq 1.0$ (EQ. 9.2.4.3-1b FEMA 302)

$$\frac{1}{0.65}\left(\frac{1.66^k}{5.01^k}\right) = 0.51 \leq 1.0$$ **O.K.**

condition (c) $\frac{1}{\phi}\left[\left(\frac{P_u}{P_c}\right)^2 + \left(\frac{V_u}{V_c}\right)^2\right] \leq 1.0$ (EQ. 9.2.4.3-1c FEMA 302)

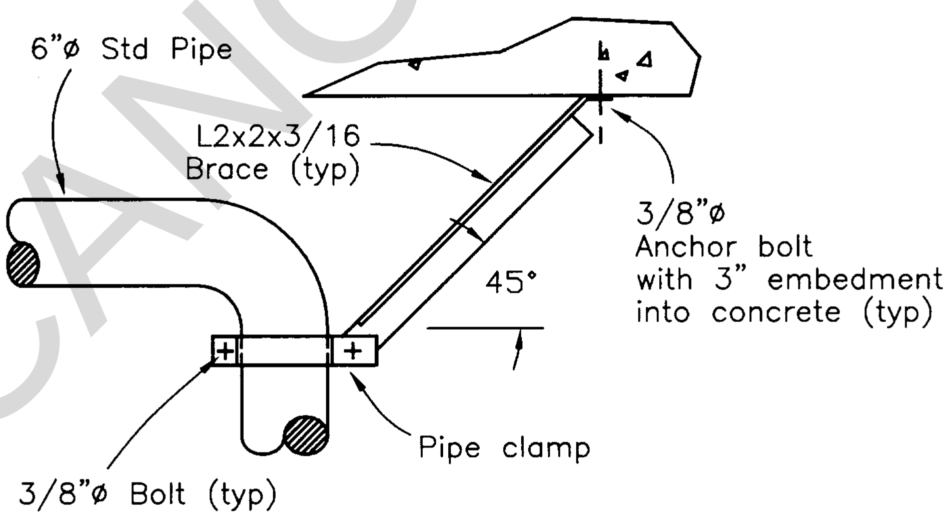
$$\frac{1}{0.65}\left[\left(\frac{1.66^k}{5.57^k}\right)^2 + \left(\frac{1.66^k}{5.01^k}\right)^2\right] = \frac{1}{0.65}(0.089 + 0.11) = 0.31 \leq 1.0$$ **O.K.**

condition (d) $\left(\frac{P_u}{P_s}\right)^2 + \left(\frac{V_u}{V_s}\right)^2 \leq 1.0$ (EQ. 9.2.4.3-1d FEMA 302)

$$\left(\frac{1.66^k}{5.94^k}\right)^2 + \left(\frac{1.66^k}{4.95^k}\right)^2 = (0.078 + 0.112) = 0.19 \leq 1.0$$ **O.K.**

Use a single 3/8" (9.53mm) ϕ A307 bolt to concrete

Design for loads longitudinal to pipe axis (see Figure J3-3);



Note: For metric equivalents; 1-in = 25.4mm
Figure J3-3. Longitudinal pipe restraint and force diagram

Note: Supports for loads longitudinal to pipe axis are located at location S_2 only. Assume that the horizontal section of pipe at the 3rd floor has longitudinal braces spaced at 60-ft (18.3m). on center.

Therefore; $(W_{P@S2})_{long} = (60' + 11')(1/2)45plf = 1{,}598\text{-lbs or } 1.60^k$ (7.11KN)

$F_p = 0.56(1.60^k) = 0.896^k$ (3.99KN)

Seismic load per brace;

$$P_u = \pm\frac{(0.896^k)\sqrt{2}}{2} = \pm 0.633^{k/brace} \text{ (2.82KN/brace)}$$

Note: Brace acts in both tension and compression.

Try a single L2x2x3/16 (L50.8mmX50.8mmX4.8mm) brace.

Check capacity of brace;

Note: By inspection, the brace is adequate in tension. Also, the brace is a singly symmetric shape that is susceptible to flexural-torsional buckling. Therefore, equations from Appendix E3 of AISC LRFD are invoked.

Compare slenderness ratios;

The length of brace is established from Figure J3-1. Assume floor slab is 6-in. (152.4mm) thick, and that attachment to the pipe is made one pipe diameter below the horizontal pipe centerline.

$$L = \sqrt{2}(24'-21'-0.5'+0.5') = 4.24' \quad (1.29\text{m})$$

$$\frac{KL_y}{r_y} = \frac{KL_x}{r_x} = \frac{1.0(4.24')(12''/1')}{0.617''} = 82.5$$

$$\frac{KL_z}{r_z} = \frac{1.0(4.24')(12''/1')}{0.394''} = 129.1 \qquad \text{(governs)}$$

Check flexural buckling;

$$\phi_c P_n = \phi_c A_g F_{cr} \qquad \text{with;} \quad \phi_c = 0.85 \qquad \text{(EQ. E2-1 AISC LRFD)}$$

From Table 3-36 of AISC LRFD;

$$\phi_c F_{cr} = 12.72\text{ksi} \quad (87.7\text{MPa}) \qquad \text{(interpolated)}$$

and

$$\phi_c P_n = \phi_c A_g F_{cr} = (12.72\text{ksi})0.715-\text{in}^2 = 9.1^k \quad (40.5\text{KN})$$

Check flexural-torsional buckling;

$$\phi_c P_n = \phi_c A_g F_{cr} \qquad \text{with;} \quad \phi_c = 0.85 \qquad \text{(EQ. A-E3-1 AISC LRFD)}$$

$$F_e = \frac{F_{ey}+F_{ez}}{2H}\left(1+\sqrt{1-\frac{4F_{ey}F_{ez}H}{(F_{ey}+F_{ez})^2}}\right) \qquad \text{(EQ. A-E3-6 AISC LRFD)}$$

$$F_{ey} = \frac{\pi^2 E}{(K_y l/r_y)^2} \qquad \text{(EQ. A-E3-11 AISC LRFD)}$$

$$F_{ez} = \left(\frac{\pi^2 EC_w}{(K_z l)^2}+GJ\right)\frac{1}{A\bar{r}_o^2} \qquad \text{(EQ. A-E3-12 AISC LRFD)}$$

where; $A = 0.715\text{-in}^2$ (461.2mm^2)

$l = 4.24'(12''/1') = 50.9\text{-in}^2$ ($32.8\text{X}10^3\ \text{mm}^2$)

$K_y = 1.0$ (no sway)

$K_z = 1.0$ (ends are rotationally restrained)

$r_y = 0.617\text{-in}$ (15.67mm) (from AISC LRFD properties tables)

$\bar{r}_o = 1.10\text{-in}$ (27.94mm) (from AISC LRFD torsional properties tables)

$C_w = 0.00254\text{-in}^6$ ($682.1\text{X}10^3\ \text{mm}^6$)(from AISC LRFD torsional properties tables)

$E = 29{,}000\text{ksi}$ ($200\text{X}10^3$ MPa)

$G = 11{,}200\text{ksi}$ ($77.2\text{X}10^3$ MPa)

$J = 0.00880\text{-in}^4$ ($3.67\text{X}10^3\ \text{mm}^4$) (from AISC LRFD torsional properties tables)

$H = 0.628$ (from AISC LRFD torsional properties tables)

$$F_{ey} = \frac{\pi^2(29{,}000\text{ksi})}{(82.5)^2} = 42.1\text{ksi} \quad (290.3\text{MPa})$$

$$F_{ez} = \left(\frac{\pi^2(29{,}000\text{ksi})0.00254-\text{in}^6}{(1.0(4.24'(12''/1')))^2} + 11{,}200\text{ksi}(0.00880-\text{in}^4)\right)\frac{1}{0.715-\text{in}^2(1.10-\text{in})^2} = 125.7\text{ksi}$$

$$(866.7\text{MPa})$$

$$F_{ey} + F_{ez} = 42.1\text{ksi} + 125.7\text{ksi} = 168\text{ksi} \quad (1.16\text{X}10^3\ \text{MPa})$$

$$F_e = \frac{168\text{ksi}}{2(0.628)}\left(1+\sqrt{1-\frac{4(42.1\text{ksi})114\text{ksi}(0.628)}{(173)^2}}\right) = 237\text{ksi} \quad (1.63\text{X}10^3\ \text{MPa})$$

Check local buckling;

$$\frac{b}{t} = \frac{2.0}{3/16} = 10.67 < 12.67 = \frac{76}{\sqrt{F_y}} \Rightarrow Q = 1.0$$ (per paragraph E3 of Appendix E of AISC LRFD)

Determine which equation to use;

$$\lambda_e = \sqrt{F_y / F_e}$$ (EQ. A-E3-4 AISC LRFD)

$$\lambda_e = \sqrt{36\text{ksi}/237\text{ksi}} = 0.39 < 1.5$$

$$F_{cr} = Q\left(0.658^{Q\lambda_e^2}\right)F_y$$ (EQ. A-E3-2 AISC LRFD)

$$F_{cr} = 1.0\left(0.658^{1.0(0.39)^2}\right)36\text{ksi} = 33.8\text{ksi} \quad (233.1\text{MPa})$$

$$\therefore \phi_c P_n = \phi_c A_g F_{cr} = 0.85(0.715-\text{in}^2)33.8\text{ksi} = 20.5^k \quad (91.2\text{KN})$$

Therefore, flexural buckling controls.

$$\phi_c P_n = 7.73^k > 0.633^k = P_u \quad (33.4\text{KN} > 282\text{KN})$$ **O.K.**

Use L2x2x3/16 (L50.8mmX50.8mmX4.8mm) brace

APPENDIX K
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