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1 March 2005

UNIFIED FACILITIES CRITERIA (UFC)

STRUCTURAL DESIGN CRITERIA FOR BUILDINGS

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UNIFIED FACILITIES CRITERIA (UFC)

STRUCTURAL DESIGN CRITERIA FOR BUILDINGS

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

NAVAL FACILITIES ENGINEERING COMMAND

AIR FORCE CIVIL ENGINEER SUPPORT AGENCY

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

Change No.	Date	Location

This UFC supersedes TI 809-02, dated 1 September 1999. 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 809-02, dated 1 September 1999.

FOREWORD

\1\
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.

UFC are living documents and will be periodically reviewed, updated, and made available to users as part of the Services' responsibility for providing technical criteria for military construction. Headquarters, U.S. Army Corps of Engineers (HQUSACE), Naval Facilities Engineering Command (NAVFAC), and Air Force Civil Engineer Support Agency (AFCESA) are responsible for administration of the UFC system. Defense agencies should contact the preparing service for document interpretation and improvements. Technical content of UFC is the responsibility of the cognizant DoD working group. Recommended changes with supporting rationale should be sent to the respective service proponent office by the following electronic form: Criteria Change Request (CCR). The form is also accessible from the Internet sites listed below.

UFC are effective upon issuance and are distributed only in electronic media from the following source:

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

Hard copies of UFC printed from electronic media should be checked against the current electronic version prior to use to ensure that they are current.

AUTHORIZED BY:

DONALD L. BASHAM, P.E. Chief, Engineering and Construction U.S. Army Corps of Engineers	DR. JAMES W WRIGHT, P.E. Chief Engineer Naval Facilities Engineering Command
Kathleen I. Ferguson KATHLEEN I. FERGUSON, P.E. The Deputy Civil Engineer DCS/Installations & Logistics Department of the Air Force	Dr. GET W. MOY, P.E. Director, Installations Requirements and Management Office of the Deputy Under Secretary of Defense (Installations and Environment)

TI 809-02
1 September 1999

US Army Corps
of Engineers®

Technical Instructions

Structural Design Criteria for Buildings

Headquarters
U.S. Army Corps of Engineers
Engineering and Construction Division
Directorate of Military Programs
Washington, DC 20314-1000

CEMP-E

TI 809-02
1 September 1999

TECHNICAL INSTRUCTIONS

Structural Design Criteria for Buildings

Approved for public release; distribution is unlimited.

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

No.	Date	Location

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-ED 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-ED, 20 Massachusetts Ave., NW, Washington, DC 20314-1000.

TI are effective upon issuance. TI are distributed only in electronic media through the TECHINFO Internet site *http://www.hnd.usace.army.mil* /techinfo/index.htm and the Construction Criteria Base (CCB) systems maintained by the National Institute of Building Sciences at Internet Site *http://www.nibs.org/ccb/*. Hard copies of these instructions produced by the user from the electronic media should be checked against the current electronic version prior to use to assure that the latest instructions are used.

FOR THE COMMANDER:

DWIGHT A. BERANEK, P.E.
Chief, Engineering and Construction Division
Directorate of Military Programs

DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
CEMP-E Washington, DC 20314-100

Technical Instructions
No. 809-02
1 September 1999

STRUCTURAL DESIGN CRITERIA FOR BUILDINGS

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CHAPTER 1

GENERAL

1-1. PURPOSE AND SCOPE. This document provides general structural design guidance for buildings and for building systems constructed of concrete, masonry, steel and wood. The design requirements provided herein, or cited by reference, are based on national building codes, industry standards, and technical manuals developed by the Army, Navy, and Air Force. Instructions necessary to provide serviceable buildings and to assure load path integrity and continuity is included. Requirements unique to Army, Navy, and Air Force facilities are indicated. Supplemental information to help engineers interpret and apply code provisions, and meet serviceability and strength performance objectives is also included in this document. Design guidance presented herein is applicable to buildings constructed in the continental USA (CONUS).

1-2. APPLICABILITY. These Technical Instructions (TI) are applicable to all USACE, Air Force and Navy elements involved with the design of buildings.

1-3. REFERENCES. Appendix A contains a list of references used in these instructions.

1-4. BACKGROUND. TI 809-02, "Structural Design Criteria for Buildings" is one in a series of Technical Instructions used to update the Army, Air Force, and Navy guidance requirements related to the design of buildings. The TI 809 Series documents are listed in Tables 1-1, 1-2, and 1-3. Where applicable the Army, Navy, and Air Force guidance documents replaced by, or to be replaced by, the TI are indicated. In some instances the TI document listed in the table has not as yet been developed, or is currently under development. In such cases, the appropriate Army, Navy, and Air Force guidance document will be used until such time as the TI is published.

Table 1-1. Design Criteria Documents Summary.

TI 809 Series No.	Title	Replaces Army Document	Replaces Navy Document	Replaces Air Force Document
TI 809-01	Load Assumptions for Buildings	TM 5-809-1		AFM 88-3, Chap. 1
TI 809-02	Structural Design Criteria for Buildings	TM 5-809-02	MIL-HDBK-1002/1 MIL-HDBK-1002/3 MIL-HDBK-1002/5 MIL-HDBK-1002/6 DM 2.04	AFM 88-3, Chap. 2
TI 809-03	Structural Design Criteria for Structures Other than Buildings			
TI 809-04	Seismic Design for Buildings	TM 5-809-10 TM 5-809-10-1	NAVFAC, P-355 NAVFAC, P-355.1	AFM 88-3, Chap. 13 AFM 88-3, Chap.13, Sec. A
TI 809-05	Seismic Design for the Rehabilitation of Buildings	TM 5-809-10-2	NAVFAC, P-355-2	AFM 88-3, Chap. 3, Sec. B
TI 809-06	Masonry Design for Buildings	TM 5-809-3	NAVFAC DM 2.9	AFM 8-3, Chapter 3
TI 809-07	Design of Load Bearing Cold-Formed Steel Systems and Masonry Veneer/Steel Stud Walls			

Table 1-2. Guidance Documents Summary.

TI 809 Series No.	Title	Replaces Army Document	Replaces Navy Document	Replaces Air Force Document
TI 809-26	Welding Guidance for Buildings	TM 5-809-7		
TI 809-27	Concrete Floor Slabs on Grade Subjected to Heavy Loads	TM 5-809-12		AFM 88-3, Chap.13
TI 809-28	Design and Construction of Reinforced Ribbed Mat Slabs			
TI 809-29	Structural Considerations for metal Roofing			
TI 809-30	Metal Building Systems			

Table 1-3. Commentary Documents Summary.

TI 809 Series No.	Title	Replaces Army Document	Replaces Navy Document	Replaces Air Force Document
TI 809-51	Seismic Review Procedures for Existing Military Buildings	TI 809-51 (Previous Edition)		
TI 809-52	Commentary on Snow Loads			
TI 809-53	Commentary on Roofing Systems			

1-5. ALTERNATIVE AND SPECIAL DESIGNS. Deviation from these criteria, where a valid need exists or an alternative solution is more desirable, may be accepted subject to approval. Requirements for alternative and special designs are described in Chapter 13.

1-6. DESIGN LOADS. Dead load (D), loads due to fluids (F), flood loads (F_a), loads due to soil and water in soil (H), live loads (L) , roof live load (L_r), and rain load (R) are to be based on the requirements of ASCE 7. Snow and wind loads are also based on the requirements of ASCE 7, except as indicated in TI 809-01, "Load Assumptions for Buildings," for Army and Air Force buildings, and in ITG (Interim Technical Guidance) “Minimum Design Loads for Buildings and Other Structures" for NAVFAC buildings. Earthquake loads are based on the requirements of TI 809-04, "Seismic Design for Buildings," and FEMA 302, "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures."

1-7. COMBINATION OF LOAD EFFECTS. The effect on the structure and its components due to gravity loads and seismic forces will be combined in accordance with the following factored load combinations, and in accordance with TI 809-04 guidelines:

$$1.2\,D + 1.0\,E + 0.5\,L + 0.2\,S \qquad \text{(ASCE 7, Paragraph 2.3.2)}$$

$$0.9\,D + 1.0\,E \qquad \text{(ASCE 7, Paragraph 2.3.2)}$$

Where:
D = dead load
E = earthquake load as defined by FEMA 302
L = live load as defined by TI 809-01
S = snow load as defined by TI 809-01

For the above load combinations the strength reduction factors (ϕ) for concrete will be in accordance with Appendix C of ACI 318-95. All other load combinations (non-seismic load combinations) will be in accordance with the applicable material design standard (ACI 318 for concrete, AISC for steel, etc.).

1-8. STRENGTH AND SERVICEABILITY PERFORMANCE OBJECTIVES. General requirements with respect to strength and serviceability are indicated in the following paragraphs. Specific requirements on strength and serviceability with respect to the various types of building systems and construction materials are provided in the following chapters of the TI.

a. Strength. Buildings and other structures, and all parts thereof, will be designed to support safely the loads and load combinations indicated above.

b. Serviceability. Structural systems and members thereof will be designed to have adequate stiffness to limit deflections, lateral drift, vibration, or any other deformations that adversely affect the intended use and performance of the building. Building designs must also consider deformation loads such as temperature, differential settlement, creep, and shrinkage. Measures necessary to keep buildings free from deformation load cracking and deterioration, such as crack control joints, will be considered an essential part of the building design. In addition buildings, when necessary, will be capable of withstanding severe environmental effects without incurring damage or deterioration that would reduce the building's service life.

c. Deflection and Drift Limits. Deflections of structural members will not be greater than allowed by the applicable material standard (ACI, AISC, etc.). Deflection limits are needed to restrict damage to ceilings, partitions, and other fragile nonstructural elements. Therefore, the deflection over span length (l) will not exceed that permitted by Table 1-4. Drift limits applicable to earthquake loading are provided in Table 6-1 TI 809-04. In certain cases, drift limits lower than those specified in TI 809-04 will be required to restrict damage to partitions, stair and shaft enclosures, glass, and other fragile nonstructural elements.

Table 1-4. Deflection Limits.
(Adapted from the International Building Code (IBC)-Final Draft)

Construction	Live Loads	Snow or Wind Loads	Dead + Live Load
Roof members			
Supporting plaster ceiling	l / 360	l / 360	l / 240
Supporting Non-plaster ceiling	l / 240	l / 240	l / 180
Not supporting Ceiling	l / 180	l / 180	l / 120
Floor Members	l / 360	----------	l / 240
Exterior walls and interior partitions			
With brittle finishes	l / 240	l / 240	----------
With flexible finishes	l / 120	l / 240	----------

1-9. LOAD PATH CONTINUITY AND INTEGRITY.

a. General. Building designers must understand how the building will respond to vertical and lateral loads. Follow all loads through the structure to assure all the structural elements and connections along the load path have sufficient strength and stiffness to maintain structural integrity. Direct and continuous load paths from the roof to foundation must be provided. Building configuration, continuous and redundant load paths, connection detailing, system ductility, quality of materials, and construction are important to building performance. These ancillary aspects of building design are covered in Chapter 2.

b. Structural Integrity. In accordance with ASCE 7, buildings and other structures will be designed to sustain local damage under extreme loading conditions with the structural system as a whole remaining stable. This objective can be achieved by an arrangement of structural elements which assures loads can be transferred from any locally damaged region to adjacent regions capable of resisting those loads. This can be accomplished by providing sufficient continuity, redundancy, and energy dissipating capability (ductility).

1-10. OVERSEAS CONSTRUCTION. Where local materials of grades other than those referenced herein are to be used, the working or yield stresses and details of construction will be modified as required by the structural property characteristics of the local material. The material, as far as practicable, will be of equivalent or better grade than the comparable grades referenced herein.

1-11. SERVICE LIFE. Service life of various buildings and facilities is defined as follows:

a. Permanent construction. Permanent construction will be designed and constructed to serve a life expectancy of 25 years or more, will be energy efficient, and will include finishes, materials, and systems selected for low maintenance and low life cycle costs.

b. Limited life structures. Limited life structures include both semi-permanent and temporary construction as defined below.

(1) Semi-permanent construction will be designed and constructed to serve a life expectancy of more than 5 years but less than 25-years (generally a 15-year service life), will be energy efficient, and will include finishes, materials, and systems selected for a moderate degree of maintenance using the life-cycle approach.

(2) Temporary construction will be designed and constructed to serve life expectancy of 5 years or less, will use low cost construction, and systems selected with maintenance factors being a secondary consideration.

1-12. STABILITY. The building foundation must be capable of safely transferring all vertical and horizontal forces, due to specified design load combinations, to the supporting soil or rock. The mechanism used for the transmission of horizontal forces may be friction between the bottom of the footing and ground, friction between the floor slab and ground, and /or lateral resistance of soil against vertical surfaces of grade beams, basement walls, footings, piles, or pile caps. Net upward forces on footings and piles, which must be resisted to prevent overturning and/or flotation, will be considered in the foundation design. Dead load should include the benefits of weight of the overlying fill in resisting sliding, overturning, and flotation. Structures will be designed to resist overturning effects caused by seismic forces in accordance with TI 809-04. For load combinations other than earthquake when load combinations applicable to allowable stress design (ASCE 7, paragraph 2.4.1) are used for the foundation design, the building will have a minimum safety factor of 1.5 against sliding, overturning, and flotation.

1-13. SPECIAL INSPECTIONS. Earthquake resistant structures designed in accordance with TI 809-04 and FEMA 302 are required to undergo special inspections. The term "special inspection" means that experts qualified in the installation, fabrication, erection or placement of structural components and connections will be available on a periodic or continuous basis to ensure the adequacy of the work. The type of special inspection depends on the type of construction used (concrete, masonry, steel, etc.) and on the seismic region in which the building is located. Special inspection requirements from FEMA 302 are addressed in the various chapters of this report. The requirements in some cases have been generalized to cover all types of loading conditions and combinations. Special inspections should not be limited to those required by FEMA 302 and this document. Rather they should be specifically required in the contract documents where ever they are needed to assure quality in construction. It is the designers' responsibility to assure all special inspection requirements are included in the contract specifications.

CHAPTER 2

BASIC STRUCTURAL SYSTEMS FOR BUILDINGS

2-1. SELECTION OF STRUCTURAL SYSTEM. The goals in the selection of a load resisting system are simplicity in the structural framing layout and symmetry in the structural system reaction to design loadings. The selections must consider the need for economy, function, and reliability. Structural systems selected must have deformation characteristics that are compatible with the architectural and other nonstructural building elements and features. Regular structure configuration, continuous and redundant load paths, and system ductility are attributes encouraged. These attributes are required of buildings constructed in high seismic areas.

2-2. COMMONLY USED STRUCTURAL SYSTEMS (Adapted from FEMA 178). The following is a list of building systems commonly found in existing building construction. These systems are provided for informational purposes with the understanding that not all the systems listed in this paragraph are recommended for use in new building construction. Figures illustrating the types of construction described herein can be found in FEMA 154, "Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook".

a. Building Type 1 - Wood, Light Frame. Light frame wood buildings are typically single, or multiple family dwellings of one or two stories. The essential structural character of this type is repetitive framing by wood joists on wood studs. Loads are light and spans are small. Some of these buildings are engineered: however, most are not, but are constructed in accordance with the International One and Two Family Dwelling Code. Shear walls are exterior walls sheathed with plank siding, stucco, plywood, gypsum board, particleboard, or fiberboard. Interior partitions are typically sheathed with gypsum board.

b. Building Type 2 - Wood, Commercial and Industrial. Commercial and industrial wood buildings usually have floor areas in excess of 5000 square feet with few, if any, interior walls. The essential structural character is framing by beams on columns. The beams may be glulam beams, steel beams, or trusses. Lateral forces are resisted by wood, or metal diaphragms, and exterior walls sheathed with plywood, or particle board. The walls may have rod, or metal strap bracing. Large openings for stores or garages often require post-and-beam framing. Lateral force resistance on walls with large openings can be achieved with steel frames or diagonal bracing. Type 2 buildings have been used with some frequency in military construction, in particular with respect to child development centers.

c. Building Type 3 - Steel Moment Frame. These buildings have frames of steel columns and beams. The beam-column connections are usually fully developed as a moment connection to resist lateral forces. Lateral loads are transferred by diaphragms to the moment resisting frames. The diaphragms can be steel decking, reinforced concrete, or a composite of steel decking with a concrete topping slab. The frames develop their stiffness by full or partial moment connections. The frames can be located almost anywhere in the building. Usually the columns have their strong directions orientated so that some columns act primarily in one orthogonal direction while others act in the other orthogonal direction, and the frames consist of lines of strong columns and their intervening beams. Steel moment frame buildings are typically more flexible than shear wall buildings, and their design is often controlled by drift limitations.

d. Building Type 4 - Steel Braced Frame. Steel braced frame buildings are similar in construction to steel moment frame buildings except lateral resistance is provided by bracing rather than beam to column moment connections.

e. Building Type 5 - Steel Light Frame. Steel light frame buildings are typically pre-engineered and prefabricated with rigid frames in the transverse orthogonal direction. The roofs and walls usually consist of lightweight metal panels. The frames are often designed for maximum efficiency, often with tapered beam and column sections built up of light plates. The frames are built in segments and assembled in the field with bolted joints. Lateral loads in the transverse direction are resisted by the rigid frames with loads distributed to them by the roof and wall panels. Lateral loads in the longitudinal direction are resisted by steel strap or rod bracing, or by shear panels located in the roof and walls.

f. Building Type 6 - Steel Frame with Concrete Shear Walls. These buildings are of typical steel frame construction with lateral loads resisted by cast-in-place reinforced concrete shear walls. The steel frame is designed for vertical loads only. The shear walls may also serve as bearing walls carrying vertical loads that would otherwise be carried by steel columns. The steel frames may provide a secondary lateral force resisting system if the steel columns and beams are rigidly connected as in Building Type 3. This combined system is termed a "dual" system in which the steel moment frames are designed to work together with the concrete shear walls with load sharing dependent on the stiffness of the two systems. In this case, the walls would be evaluated as concrete shear walls, and the frames would be evaluated as steel moment frames.

g. Building Type 7 - Steel Frame with Infill Shear Walls. In this older type of construction, some of which still remains, solidly infilled masonry panels act as a diagonal strut between moment frames. This type of construction is <u>not recommended</u>, because if the infill walls do not fully engage the frame members (i.e., lie in the same plane), diagonal compression strut action will not develop.

h. Building Type 8 - Concrete Moment Frame. These buildings are similar to steel moment frame buildings except the frames are of reinforced concrete construction. In high seismic areas the concrete frames have large quantities of longitudinal and transverse reinforcing steel, with closely spaced transverse steel (spiral reinforcement and stirrups) required to confine the concrete and produce a ductile response to earthquake ground motion.

i. Building Type 9 - Concrete Shear Walls. The vertical components of the lateral-force resisting system in these buildings are concrete shear walls that are usually bearing walls. Buildings in which the shear wall area is relatively large with respect to the floor area, there often is no need to provide boundary elements to handle the large compressive strains that can occur in the wall extremities. Buildings with limited shear wall area will require shear wall boundary elements in accordance with ACI 318.

j. Building Type 10 - Concrete Frame with Infill Shear Walls. These buildings are similar to Type 7 buildings except that the frame is of reinforced concrete. The capacity of this system during major earthquakes is often limited by the shear strength of the columns (non-ductile response) since once the infill cracks much of the shear is transferred to the columns. For this reason the use of Type 10 buildings is <u>not recommended for high seismic regions.</u>

k. Building Type 11 - Precast/Tilt-Up Concrete Walls with Lightweight Flexible Diaphragm. These buildings often have a metal deck or wood roof diaphragms that distributes lateral forces to precast concrete shear walls. The connections between the diaphragms and walls and between precast concrete wall elements are extremely important in high seismic regions. Tilt-up buildings often are more than one story.

l. Building Type 12 - Precast Concrete Frames with Concrete Shear Walls. These buildings typically have floor and roof diaphragms composed of precast concrete elements with or without cast-in-place concrete topping slabs. In high seismic regions, cast-in-place concrete topping slabs are generally used unless diaphragm spans are very small. Precast concrete girders and columns support the diaphragms. The girders often bear on column corbels. Closure strips between precast floor elements and beam column joints usually are cast-in-place concrete. Welded steel inserts often are used to interconnect precast elements. The capacity of these type buildings to resist lateral loads is dependent on connection strength and ductility. Buildings with precast frames and concrete shear walls should perform as intended during major earthquakes if the connections have sufficient strength and displacement capacity.

m. Building Type 13 - Reinforced Masonry Bearing Walls with Metal Deck Roof Diaphragms. These buildings have perimeter bearing walls of reinforced concrete masonry units (CMU) or reinforced brick construction. The bearing walls are also the vertical elements of the lateral force resisting system. The roof diaphragm is of metal deck construction with or without a cast-in-place topping. Floor diaphragms are generally a reinforced concrete slab or precast concrete slab supported by steel beams or CMU walls. The roof and floor diaphragms should have sufficient strength and stiffness to transfer lateral loads to the transverse shear walls without imposing excessive out-of-plane displacements on the longitudinal walls.

n. Building Type 14 - Reinforced Masonry Walls with Precast Concrete Roof Diaphragm. These buildings are similar to Type 13 buildings except the roof diaphragm is composed of interconnected precast concrete elements such as planks, or tee beams with or without a cast-in-place topping. The roof diaphragm is stiffer than the metal deck diaphragm of the Type 13 building and therefore roof diaphragm deflections will generally not impose excessive displacements on the longitudinal walls. Because of efflorescence problems double wythe construction with grout fill between wythes is not permitted for military buildings.

o. Building Type 15 - Unreinforced Masonry Bearing Wall Building. These buildings are similar in construction to Type 13 and Type 14 buildings except the masonry is unreinforced or has the minimum reinforcement required by code. Unreinforced masonry construction is typical in the eastern and mid-western United States. The performance of unreinforced masonry buildings when subjected to earthquake ground motions has often been unsatisfactory. For this reason Type 15 building construction <u>is not permitted by the Army and Air Force</u>.

2-3. BUILDING SYSTEM USE IN MILITARY CONSTRUCTION. Various bearing wall systems, building frame systems, and moment resisting frame systems were described in the preceding paragraph. It should be noted that a particular building type (such as Type 9) might be either a bearing wall system or a building frame system. Descriptions of systems commonly used in military construction, and discussions of particular systems that should be avoided, are presented in the following paragraphs.

a. Bearing Wall Systems. A bearing wall system is a structural system without a complete vertical load carrying space frame. Bearing walls or bracing systems support gravity loads. Shear walls or braced frames resist lateral loads. Bearing wall systems of reinforced concrete and reinforced masonry (Types 9, 11, 12, 13, and 14) are common to all types of military construction. Bearing wall systems of steel construction (Types 4 and 6) are also common.

b. Building Frame Systems. A building frame system is a structural system with an essentially complete space frame that supports gravity loads. Shear walls or braced frames resist lateral loads. Building frame systems in which non-load bearing shear walls or braced frames resist lateral loads (Types 4, 6, 9, 11, and 12 buildings) are excellent systems for high seismic regions and are commonly used in military construction.

c. Moment Frame Systems. A moment resisting frame system is a structural system with an essentially complete space frame that supports gravity loads and resists lateral wind and earthquake forces. Moment resisting frame systems of steel and concrete (Types 3, and 8 buildings) are sometimes used. Moment frames are recommended in cases where it is undesirable to restrict the interior of the building with interior bracing or shear walls. Interstory drift is a problem with moment frame construction and the interaction of the moment frames with architectural and other nonstructural building elements must be considered.

d. Other Systems. Because of fire protection concerns, systems of wood construction (Type 1 and Type 2 buildings) are generally limited to residential construction. Although, in the western United States it is not uncommon to find wood construction used for the construction of one and two story office buildings, school buildings, and commercial buildings. Steel light frame (Type 5) buildings are often used for industrial type buildings, and for one and two story dormitory construction. Infill systems (Type 7, and 8 buildings) although commonly used in older construction, <u>are not recommended</u> for new military buildings because of undesirable interaction that occurs between the infill walls and the frames when the building is subjected to lateral loads. Infill wall systems are particularly undesirable in high seismic regions.

2-4. CHARACTERISTICS IMPORTANT TO BUILDING PERFORMANCE. (A*dapted from DOE "Design and Evaluation Guidelines for Department of Energy Facilities Subjected to Natural Phenomena Hazards")* Proper design and detailing of structural systems and their connections is critical. Structural engineers should work closely with the building architect in the early phases of building design to assure characteristics important to building performance such as structural system configuration, load path continuity, redundancy, ductility, and the quality of materials and construction, become a integral part of the concept design.

a. System Configuration. The configuration of the structural system is important. Irregularly configured structures under extreme loading conditions, especially earthquake loadings, experience greater damage than regularly configured systems. Irregular structures are structures having one or more plan irregularities or one or more vertical irregularities. Plan irregularities are defined in FEMA 302, Table 5.2.3.2. Vertical irregularities are described in FEMA 302, Table 5.2.3.3. Plan irregularities, such as large reentrant corners or differences in stiffness between portions of diaphragms, produce stress concentrations and high localized forces. Other plan irregularities can cause an undesirable torsional response to lateral loads. Buildings that are T, L, U-shaped, or cruciform configurations are examples of irregular plans. Often seismic joints, which separate the various wings of buildings, are provided to allow each wing to perform as an individual structure. This is often more practical than using a rigorous three-dimensional analysis to determine how the wings of the building will interact if connected together. Vertical irregularities, such as an abrupt change in stiffness from one story to the next, or offsets in the lateral force resisting systems from one story to the next, can also produce stress concentrations and high-localized forces. Regular structural system configuration should be encouraged for new designs, especially for buildings located in high seismic areas. It should be realized however that an regular configuration is not always possible. In such cases the designer must recognize the effect each particular irregularity will have on structural response and make a conscious effort to ensure his design will meet all strength and serviceability requirements.

b. Load Path Continuity. Direct and continuous load paths should be provided to assure that all loads to which the structure is subjected can be delivered from the load point of application to the foundation. All elements and connections along the load path must have sufficient strength, stiffness, and deformation capability to deliver the loads to the foundation without impairing the buildings ability to perform as a unit. When connection is required to develop the strength of the connected members (such as in the design of earthquake-resistant systems), the effect member over-strength will have on system performance must be considered. Different parts of the building should be adequately interconnected to resist extreme loads, to prevent progressive type failures, and resist foundation settlement. Beams and girders should be adequately tied to columns, and columns should be adequately tied to footings. Concrete and masonry walls should be adequately anchored to floors and roofs for lateral support. Diaphragms that distribute lateral loads to vertical resisting elements must be adequately tied to those elements. Collector or drag struts should be provided to collect shear forces and deliver them to shear-resisting elements, such as shear walls or other bracing elements, that may be spaced at various intervals around the diaphragm. Shear walls must be adequately tied to floor and roof slabs and to footings. Non-structural elements such as exterior cladding and interior stairs should be isolated from structural load paths to assure loads are delivered as intended from the point of application to the foundation. Rigid non-structural features if not properly isolated can attract loads that will most likely damage non-structural elements and in the process create unintended load paths that can damage structural elements.

c. Redundancy. Redundancy of load paths is a desirable structural system characteristic, especially with respect to earthquake-resistant design. Redundancy means that when one structural element or system fails a new load path develops allowing the loads once carried by the failed structural element or system to be safely transferred to another primary or secondary system thereby preventing progressive collapse of the structure. Redundant systems if properly designed will also prevent the formation of unwanted load paths. For instance if two or more bays of lateral bracing are provided on each side of a building the failure of one bay of bracing will not cause a serious torsional response which could jeopardize the bracing on the other sides of the building. A single row of bracing could be considered to have redundant characteristics if each bracing element is designed to carry both tension and compressive loads. This is not, however, as reliable as a system containing two or more braced bays. The practice of limiting the bracing on a side of a building to a single bay should be avoided if possible. A single braced bay consisting of tension-only bracing is unacceptable. In a building without redundant components, every component must function as intended to preserve the overall structural integrity of the building.

d. Ductility. It is desirable, and required by code, that structures be ductile to avoid brittle failure mechanisms which could lead to an unexpected failure. Ductility is imperative for the design of earthquake resistant structures.

(1) Structural steel is an inherently ductile material. The ductility of steel structures is achieved by designing connections to avoid tearing or fracture and by ensuring an adequate path for loads to travel across the connection. Detailing for adequate stiffness and restraint of compressive braces, outstanding legs of members, compression flanges, etc. must be provided to avoid local and global instability by buckling of relatively slender steel members acting in compression. Deflections must be limited to prevent overall frame instability due to P-Δ effects. Steel bracing systems must be configured such that bracing forces can not distort columns in a manner that would amplify P-Δ effects Refer to TI 809-04 for additional information on acceptable and unacceptable bracing systems).

(2) Less ductile materials, such as concrete and unit-masonry, require steel reinforcement to provide ductility. Concrete structures should be designed to prevent brittle failure mechanisms such as compressive failure, shear failure, anchorage failure, and bond failures from occurring. Compressive failures in flexural members can be controlled by limiting the amount of tensile reinforcement or by providing compressive reinforcement and confining the reinforcement with closely spaced transverse reinforcement such as spirals, or stirrup ties. Confinement increases the strain capacity and the compressive, shear and bond strengths of concrete and masonry. Shear failures in concrete and masonry can be prevented through the use of adequate shear reinforcing. Anchorage and splice failures can be controlled by providing adequate splice development length, or by providing suitable mechanical or welded connections. Masonry walls must be adequately reinforced and anchored to floors and roofs. Additional information on the detailing of structures to provide a ductile response to earthquake ground motions can be found in TI 809-04.

e. Quality of Materials and Construction. It has been observed that the quality of materials used in construction and the quality of construction are important factors, which can determine whether or not a building will survive extreme loadings due to earthquakes or wind. Testing and inspection programs are necessary to ensure the finished structure meets code requirements. It is the designer's responsibility to assure all testing and special inspections required by code are part of the contact documents.

CHAPTER 3

FOUNDATIONS

3-1. GENERAL. This chapter prescribes criteria for the design of building foundations including spread footings, pile supported foundations, pier supported foundations, and machinery foundations.

3-2. DESIGN. Allowable bearing pressures and allowable stresses where required in the design of pile and pier foundations will be used with the allowable stress design load combinations of ASCE 7. Where foundations are designed for seismic overturning using the strength design load combinations of ASCE 7, the seismic overturning moment need not exceed 75% of the value determined by either the equivalent lateral force method, or the modal analysis method of TI 809-04, "Seismic Design for Buildings".

3-3. FOUNDATION AND SOILS INVESTIGATIONS. Foundation and soils investigations when required will be in accordance with EM 1110-1-1804, " Geotechnical Investigations". Additional information on foundation investigations can be found in the Navy Design Manual (DM) 7.1, "Soil Mechanics", and Military Handbook (MIL-HDBK) 1007/3, "Soil Dynamics and Special Design Aspects". The latter document describes the requirements for site specific studies for seismically active areas. Pile and pier foundations will be designed and installed on the basis of a foundations investigation and pile load test when required.

3-4. ALLOWABLE LOAD BEARING VALUES AND LATERAL SLIDING RESISTANCE OF SOILS.

a. Allowable Bearing Pressures. Maximum allowable bearing pressures will be determined for soil foundations in accordance with EM 1110-2-1905, "Bearing Capacity of Soils", and for rock foundations in accordance with EM 1110-1-1908, "Rock Foundations". The presumptive allowable bearing pressures for spread footing as provided in Table 4-8 of EM 1110-1-1905 can be used with caution for spread footings supporting small or temporary structures. Design guidance for foundations can be found in TM 5-818-1 / AFM 88-3, Chapter 7, "Soils and Geology Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures)", and in Navy Design Manual (DM) 7.02, " Foundations and Earth Structures".

b. Lateral Sliding Resistance. The resistance of footings to lateral sliding will be calculated by combining shear frictional resistance ($V \tan \phi$) and lateral soil resistance,

where:

V = vertical load
ϕ = Angle of internal friction

The lateral soil resistance will not exceed one-half the computed passive resistance.

c. Increases in Allowable Bearing Values. Allowable bearing pressures may be increased by 1/3 for load combinations involving wind or earthquake.

3-5. FOOTINGS AND FOUNDATIONS. Footings and foundations will be built on undisturbed soil or compacted fill material.

a. Frost Protection. The minimum design depth of building foundations to protect against frost penetration will be in accordance with Tables 1 and 2 of TI 809-01 "Load Assumptions for Buildings". The minimum depth when not governed by frost protection requirements will be 300 mm (12 inches). The DOD Weather Manual (a Tri-Service document) provides additional information that can be used to determine frost depth penetration.

b. Design. Footings will be so designed that the allowable bearing capacity of the soil is not exceeded and that differential settlement is minimized. The minimum width of footings will be 300mm (12 inches). Footing design will be in accordance with TM 5-818-1 / AFM 88-3, Chapter 7, or Navy Design Manual (DM) 7.02, “Foundations and Earth Structures.” Footing design will also meet the requirements of ACI 318, except loads and load combinations will be per the requirements of Chapter 1.

c. Seismic Footing Ties. Where a building is assigned to Seismic Design Category D, E, or F, as defined in FEMA 302, "NEHRP Recommended Provisions for the Seismic Design of New Buildings and Other Structures," individual spread footings founded on soil will be interconnected by ties in accordance with the provisions of FEMA 302 and TI 809-04, "Seismic Design for Buildings."

d. Gable Bent Footing Ties. The gable bent type of moment frame requires a horizontal reaction force at the foundation to resist horizontal spreading. Because often it is unreliable to count on the soil surrounding the footings to provide this reaction, (i.e., excavation next to the building may reduce lateral bearing resistance), footing ties are advisable. These ties, may consist of reinforced concrete tension tie beams that are located below the slab-on-grade, or for short span frames may be reinforcing steel which anchors the gable bent footing directly to the slab-on-grade. When gable bent frames are anchored to the slab-on-grade it is imperative that the location of the ties be coordinated with the slab-on-grade jointing to assure tie capacity is not reduced or impaired by the joints. Reinforced concrete tension tie beams are required for gable bent frames with spans of 15 meters (50 feet) or more.

e. Footings on Expansive Soils. Expansive soils change volume from changes in water content leading to total and differential foundation movements. Seasonal wetting and drying cycles have caused soil movements that lead to long-term deterioration of structures. Soils can have large strengths and bearing capacity when relatively dry. Expansive soils consist of plastic clays and clay shales that often contain colloidal clay materials. They include marls, clayey siltstone, sandstone, and saprolites. Some of these soils, especially dry residual clayey soil, may heave under low applied pressure but collapse under higher pressure. In certain cases, clay soils may not exhibit swelling characteristics if undisturbed. This same soil, when dried by manipulation and re-compacted at less than the initial moisture content, may exhibit some swell. Other soils may collapse initially but heave later on. Estimates of the potential heave of these soils are necessary for consideration of the foundation design. Information regarding the design of foundations on expansive soils can be found in TM 5-818-1. Additional information on expansive soils can be found in EM 1110-1-1905 and Navy Design Manual (DM) 7.02.

3-6. DEEP FOUNDATIONS. Deep foundations such as piles or pier foundations are needed to transmit the load of a building through a material of poor bearing capacity, or to eliminate differential settlements. The choice of pile or pier type for a given foundation should be made on the basis of a comparative study of cost, permanency, stability under vertical and horizontal loading, and the required method of installation. Additional information on pile and pier foundations can be found in the publications listed in Appendix A.

a. Pile Foundations. Pile foundations can consist of concrete, wood, or steel elements either driven or drilled into the ground. Piles are relatively slender in comparison to their length. Piles derive their load carrying capacity through skin friction, end bearing, or a combination of both. Two of the pile types common to building construction, along with their advantages and disadvantages, are described below. Design of pile foundations will comply with the provisions of EM 1110-2-2906, "Design of Pile Foundations," or Navy Design Manual (DM) 7.02, “Foundations and Earth Structures,” and Military Handbook (MIL-HDBK) 1007/3, “Soil Dynamics and Special Design Aspects.”

(1) Steel H-Piles. Steel H-piles are rolled steel sections with wide flanges so the depth of the section and widths of the flanges are about equal dimension. The cross-sectional area and volume of the H-pile are relatively small; consequently, they can be driven through compacted granular materials and into soft rock. Steel H-piles because of their small volume displacement have little or no effect in causing ground swelling or rising of adjacent piles. Steel piles protruding above the ground lines are subject to corrosion at or somewhat below the ground line. Steel piles are ductile and therefore are suitable for use in high seismic areas.

(2) Prestressed Concrete Piles. Prestressed concrete piles are used and have replaced for the most part the reinforced concrete precast pile. Some of its advantages are prestressing eliminates open cracks in a concrete pile, permits ease in handling, and reduces the tendency to spall during driving. The compression induced in the pile permits piles to sustain considerable bending stresses. However when used in high seismic areas, prestressed concrete piles must contain large quantities of confinement steel in the form of spiral reinforcement to resist the curvature demands place on the pile by differential subsurface ground distortions. The transverse confinement reinforcement is similar to that required for reinforced concrete columns in high seismic areas (Site Class D, E, or F per TI 809-04). Information on the amounts of transverse reinforcement required for various regions of the pile can be found in two PCI Journal Papers: "Seismic Design of Prestressed Concrete Piling" PCI JOURNAL, V.28, No. 2, March-April 1983, and "Simulated Seismic Load Tests on Prestressed Concrete Piles and Pile-Pile Cap Connections," PCI JOURNAL, V.35, No. 6, November-December 1990. The later PCI Journal Paper also provides valuable information on pile to pile cap connections.

b. Pier Foundations. Pier foundations are constructed by digging, drilling, or otherwise excavating a hole in the soil, which is subsequently filled with plain or reinforced concrete. Steel casings may or may not be used to facilitate pier construction. Two pier foundation types common to building construction, along with their advantages and disadvantages, are described below. The design of pier foundations will follow the recommendations contained in ACI Committee Report 336, "Suggested Design and Construction Procedures for Pier Foundations."

(1) Augured Uncased Piers. Augured uncased pier foundations are constructed by depositing concrete into an uncased augured hole. The drill hole diameter can range from 250 mm (10-inches) to 1825 mm (72-inches), and up to 60 meters (200-feet) deep. In advancing through granular materials, drilling mud is often required to keep the hole open. The drilled shaft is filled with concrete in the dry, or by means of a tremie pipe through the drilling mud. When the concrete is to be reinforced, care and planning is required to assure the reinforcement can be placed in the desired location and to the depth required. For drilled piers installed with a hollow stem auger, where longitudinal steel reinforcement is placed without lateral ties, the reinforcement will be placed through ducts in the auger prior to the placement of concrete. When transverse reinforcement is required, the reinforcement is fabricated in cages which are securely tied so they will not rack or otherwise distort when handled and placed in the augured hole. Transverse confinement reinforcing similar to that indicated for prestressed piling is required for uncased concrete piers constructed in high seismic areas.

(2) Drilled Shaft Piers. Drilled shaft piers can be generalized as large diameter cast-in-place concrete filled pipes. Pier diameter range from 300mm (12-inches) to 900 mm (36-inches), and the casing may, or may not remain part of the load-carrying element. Casings where used are usually thick-walled. Drilled shaft piers can be designed to carry extremely heavy loads to extreme depths. Once installed to the desired depth, the pipe is cleaned, reinforcement placed, and filled with concrete if dry, or filled by the tremie method if water is present. The pipe can then either be pulled for reuse, or left in place to increase load carrying capacity. Transverse confinement reinforcing similar to that indicated for prestressed piling is required for caisson piers constructed in high seismic areas in those cases where the pipe is to be pulled. When the pipe is left in place, the pipe can be used to provide the necessary concrete confinement.

3-7. FOUNDATIONS FOR MACHINERY. Commonly used machines such as centrifugal pumps, fans, centrifuges, blowers, generator engines and compressors have vibrational characteristics that can be damaging to foundations. The design of foundations supporting these types of equipment requires special consideration to assure the equipment and foundations supporting the equipment are not damaged due to resonant vibration. Information and references pertaining to the design of foundations for vibratory loads can be found in the ACI Committee 350 Report, "Environmental Engineering Concrete Structures". Additional information can be found in ACI Committee 351 Report, " Foundations for Static Equipment," and in Military Handbook (MIL-HDBK) 1007/3, “Soil Dynamics and Special Design Aspects.”

a. Minimum Requirements for Spread Footings Supporting Machinery. Machinery and generator foundations will be reinforced as required by design loads but in no case with less than 0.15 percent reinforcing each way distributed at top and bottom. Minimum bar size will be No. 4, and maximum spacing of bars will be 300 mm (12 inches). These foundations will be completely isolated from floor slab on grade with isolation joints. The allowable bearing pressure will be one-half that assumed for static load conditions. When the depth of foundation (D) is 900mm (36 inches) or more and its length-to-width (L /W) ratio is 3 or more, the following reinforcing steel requirements will be met.

(1) Longitudinal reinforcing will be distributed at top, bottom, and faces of foundation within 150 mm (6 inches) of the surface.

(2) Horizontal bars, bent and lapped to be continuous with side-wall, top and bottom bars will be provided on end and side-wall faces. See Figure 3-1 for distribution of reinforcement in heavy machinery foundations.

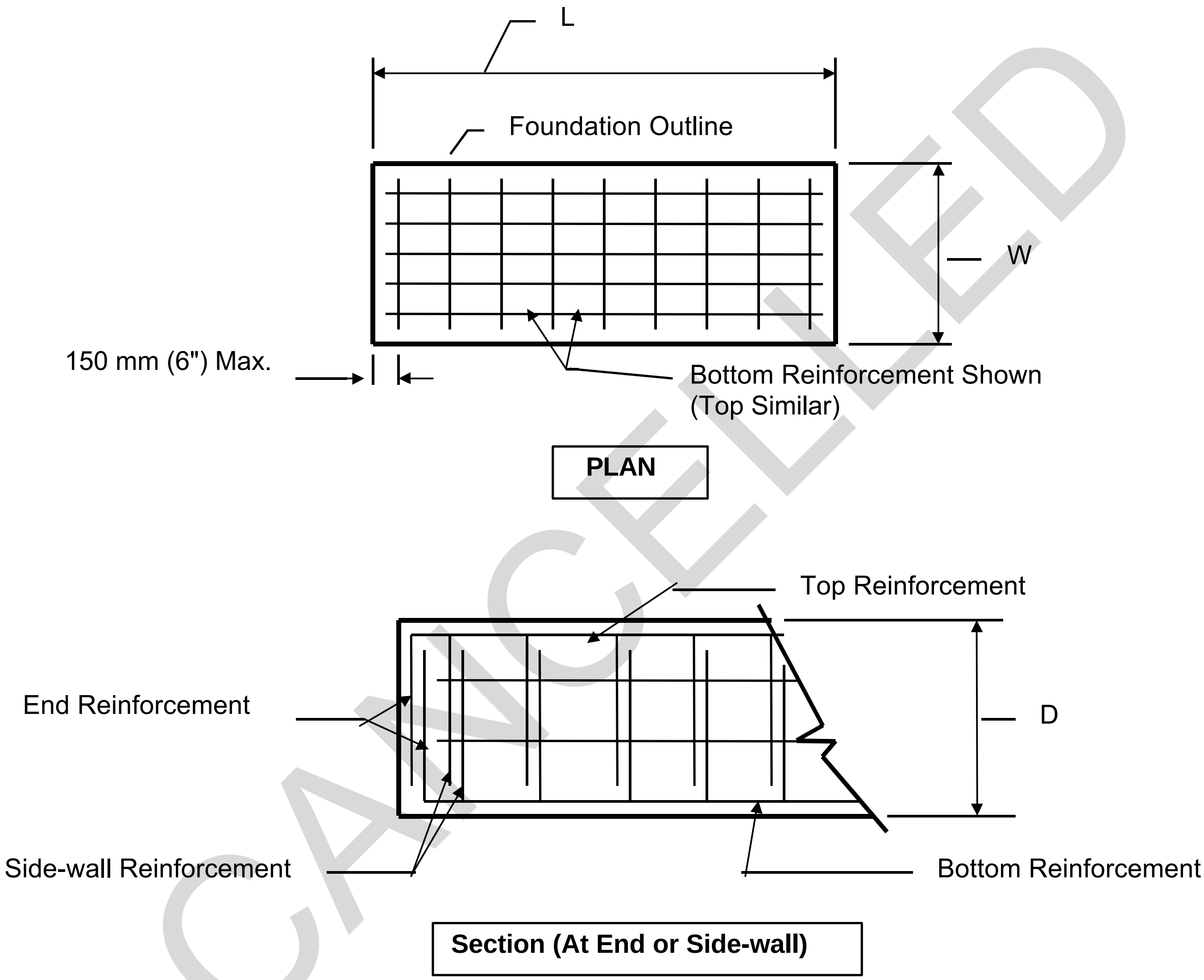

Figure 3-1. Machinery Foundation Reinforcing.

3-8. SPECIAL INSPECTIONS.

- Pile and Pier Foundations. Inspectors familiar with pile and pier construction will be present when pile or pier foundations are being installed, and during load testing. Records are to be kept for each pile or pier installed, including the results of all load tests, and the final tip elevation.

CHAPTER 4

CONCRETE STRUCTURE DESIGN REQUIREMENTS

4-1. INTRODUCTION. This chapter prescribes criteria for the design of buildings using cast-in-place or precast construction with plain, reinforced, or prestressed concrete.

4-2. BASIS FOR DESIGN. The basis for design for buildings and building components constructed of reinforced concrete, prestressed concrete, or plain concrete will be ACI 318, "Building Code Requirements for Structural Concrete and Commentary". Additional provisions for buildings constructed in severe environments, and buildings designed to resist the effects of accidental explosions (blast-resistant construction) are provided in Chapter 12. In executing designs in accordance with ACI 318, cognizance will be given to ACI 318R; Portland Cement Association (PCA) Notes on ACI 318 Building Code Requirements for Reinforced Concrete with Design Applications, and to ACI standards and committee reports referenced in this document.

4-3. EARTHQUAKE RESISTANT DESIGN. Concrete structures are to be designed to resist the effects of earthquake ground motions. The additional requirements of TI 809-04, "Seismic Design for Buildings" and FEMA 302, "NEHRP Recommended Provisions for the Seismic Design of New Buildings and Other Structures" will apply.

4-4. DESIGN STRENGTHS. Concrete strengths for various applications and various exposures are listed in Table 4-1. Use of high strength concrete will be in accordance with ACI Committee 211 Report, "Guide for Selecting Proportions for High-Strength Concrete with Portland Cement and Fly Ash," and ACI Committee 363 Report, "State-of-the-Art Report on High Strength Concrete."

Table 4-1. Minimum Concrete Strength Requirements.

Usage	Minimum Strength
Concrete fills.	14 Mpa (2000 psi)
Encasements for utility lines and ducts.	17 Mpa (2500 psi)
Foundation walls, footings and cast-in-place concrete piles.	20 Mpa (3000 psi)
Slabs on grade.	24 Mpa (3500 psi)
Reinforced concrete buildings	20 Mpa (3000 psi)
Precast members (including architectural and structural members and piles).	28 Mpa (4000 psi)
Walls or floors subjected to severe exposure (Severe exposure includes extreme heat or cold and exposure to deicing or other aggressive chemicals.)	20 Mpa (3000 psi)

Table 4-1. Continued

Concrete deposited under water (tremie concrete).	20 Mpa (3000 psi)
Columns in multistory buildings carrying heavy loads.	28 Mpa (4000 psi)
Reinforced concrete in contact with sea-water, alkaline soils or waters, or other destructive agents.	28 Mpa (4000 psi)
Prestressed concrete construction.	35 Mpa (5000 psi)

4-5. DESIGN CHOICES. The selection of the structural concrete framing system, strength of concrete and reinforcement, conventional versus lightweight concrete, conventional versus prestressed design, and cast-in-place versus precast construction will be based on economic and functional considerations. Designers should take into account the specific type and size of structure, architectural features or special performance requirements, seismic exposure, construction cost factors for the building site, and the availability of materials and labor. For further discussion of considerations in selecting appropriate composition and properties for concrete, see ACI Committee 201 Report, "Guide to Durable Concrete."

4-6. SERVICEABILITY. Buildings must remain serviceable throughout their service life. This means for concrete buildings and concrete structural elements, the concrete must be durable, free from objectionable cracking, and with adequate protection of the reinforcing steel to prevent corrosion. In additions, structural deflections that can damage interior partition walls, ceilings and various architectural features must be kept within acceptable limits.

a. Durability. Durability of Portland cement concrete is defined as its ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration. Durable concrete will retain its original form, quality, and serviceability when exposed to its environment. Causes of concrete deterioration, such as freezing and thawing, aggressive chemical exposure, abrasion, corrosion of steel and other materials embedded in concrete, and chemical reactions of aggregates are described in the ACI Committee 201 Report, "Guide to Durable Concrete". This report also covers various preventive measures to assure durability problems do not occur. The most significant causes of concrete deterioration are freezing and thawing, and corrosion of reinforcing steel.

(1) Freeze-thaw Protection. Concrete made with good aggregates, low water-cement ratio, and air entrainment will have good resistance to cyclic freezing. Air entrained concrete which contains an appropriate distribution of air voids provides good freeze-thaw protection, because when the concrete freezes there is room for any water which has saturated the concrete to expand without causing damage to the concrete. Table 4-2 provided recommended air contents to prevent freeze-thaw damage.

Table 4-2. Recommended Air Contents for Frost-resistant Concrete
(From ACI Committee 201 Report)

Nominal Maximum Aggregate Size	Average air content % Severe Exposure	Moderate Exposure
10 mm (3/8 inch)	7 1/2	6
12 mm (1/2 inch)	7	5 1/2
19 mm (3/4 inch)	6	5
38 mm (1 1/2 inch)	5	4 1/2
75 mm (3 inch)	4 1/2	3 1/2
150 mm (6 inch)	4	3

(2) Corrosion Protection. Corrosion protection is accomplished primarily by providing a sufficient thickness of concrete cover over reinforcing steel and other embedded items. A complete discussion of corrosion causes and preventive measures can be found in the ACI Committee 201 Report, "Guide to Durable Concrete," and in ACI Committee 222 Report, “Corrosion of Metals in Concrete.” For normal exposure conditions, or those conditions where the concrete is not exposed to chlorides, the minimum concrete cover protection specified in ACI 318 will be provided. Concrete cover requirements for severe exposure conditions is covered in Chapter 12.

b. Crack Control. Cracking in concrete occurs mainly when volume changes due to drying shrinkage and temperature effects are restrained. Cracking can also occur due to externally applied loads. Cracks indicate a major structural problem, or a serviceability problem. Reinforcing steel exposed to moisture and air can corrode. The corroded steel has a volume several times that of the parent material. Cracking and spalling occurs due to the expansion of the steel as it corrodes. A discussion of the factors that cause cracking in concrete and measures that can be used to control cracking are provided in the ACI Committee 224 Report, "Control of Cracking in Concrete Structures." Cracking can be controlled by providing adequate temperature and shrinkage reinforcement, by reducing steel stresses at service load conditions, and by reducing restraint through the use of joints. Tolerable crack widths for reinforced concrete under various exposure conditions are provided in Table 4-3.

Table 4-3. Tolerable Crack Widths for Reinforced Concrete
(From ACI Committee 224 Report)

Exposure Condition	Tolerable Crack Width
Dry air or protective membrane	0.40 mm (0.016 inch)
Humidity, moist air, soil	0.30 mm (0.012 inch)
Deicing chemicals	0.20 mm (0.007 inch)
Sea water and saltwater spray	0.15 mm (0.006 inch)
Water retaining structures	0.10 mm (.004 inch)

(1) Shrinkage and Temperature Reinforcement. To keep cracks widths within acceptable limits for buildings under normal exposure conditions the minimum shrinkage and temperature reinforcement as required by ACI 318 will be provided. Shrinkage and temperature steel requirements for buildings under severe exposure conditions are provided in Chapter 12.

(2) Reducing Steel Stresses under Service Load Conditions. Cracking due to service loads can be controlled by limiting the maximum stress in the reinforcing steel, and by providing small diameter bars at close spacing, rather than large size bars at wide spacing. Rules for distributing flexural reinforcement in beams and slabs to control flexural cracking are provided in ACI 318. Suitable distribution of flexural reinforcement in beams and slabs is measured by a z-factor. Z-factors for normal interior and exterior exposure conditions will comply with ACI 318 requirements.

(3) Joints and Joint Sealants. The effects of deflection, creep, shrinkage, temperature contraction and expansion, and the need for vibration isolation will all be addressed when determining the location of expansion and contraction joints in concrete buildings. Appropriate allowances for the aforementioned effects will be included in the design; location, details or provisions for required contraction joints, control (weakened-plane) joints, expansions joints, isolation joints, and seismic joints. The location of expansion, contraction, and seismic joints will be shown on the drawings since joints are critical with respect to other design considerations, e.g., configuration of the structural concrete, effects of joints on structural strength and shrinkage cracking, and the appearance of joint lines on exposed concrete surfaces. Where reinforced concrete foundation walls support masonry, crack control measures will be designed to be compatible with crack control measures in the masonry. All crack control joints in the foundation wall will be carried upward into masonry crack control joints. The following are basic

requirements for the more common types. Additional information on joints for concrete buildings can be found in ACI Committee 224.3 Report, "Joints in Concrete Construction," and the Portland Cement Association Report (PCA), "Building Movements and Joints".

(a) Expansion Joints. Expansion joints are seldom needed in buildings less than 200 feet in length, the exception being for brick masonry construction where expansion joints are provided at close intervals. The maximum permitted spacing of expansion joints in brick walls are provided in TI 809-06, “Masonry Structural Design for Buildings”. The maximum length a building can be without expansion joints depends on the temperature change that can occur in the region in which the building is located. In general, expansion joints should be provided in accordance with the following rules:

- Where the temperature differential (TD), defined as the greater of the differences between the annual mean air temperature and the highest and lowest air temperature to be expected, is not greater than 20 degrees C (70 degrees F) and no excessive change in atmospheric moisture is anticipated, expansion joints should be spaced so straight lengths of building measure no more than 90 meters (300 feet) between joints.

- Where the TD is greater than 20 degrees C (70 degrees F), or where excessive change in atmospheric moisture is likely, expansion joints should be spaced so straight lengths of building measure no more than 60 meters (200 feet) between joints.

- An expansion (or seismic) joint is usually required between adjoining building areas which are different in shape, or between areas where different rates of building settlement are anticipated.

- Joints for structural or seismic reasons are often located at junctions in L-, T-, or U-shaped buildings.

Expansion joints should extend entirely through the building, completely separating it into independent units. Column footings located at expansion joints need not be cut through unless differential settlements or other foundation movements are anticipated. Expansion joints should be carried down through foundation walls: otherwise the restraining influence of the wall below grade, without a joint, may cause the wall above to crack in spite of it's joint. Reinforcement must never pass through an expansion joint. An empirical approach for determining the need for expansion joints is provided in the PCA Report, "Building Movements and Joints".

(b) Control Joints. Control joints are needed to eliminate unsightly cracks in exposed building walls by controlling the location in which cracking due to volume change effects takes place. As a general rule:

- In walls with openings, space control joints at 6-meter (20-foot) intervals; in walls with infrequent openings, space at 8-meter (25-foot) intervals.

- Provide a control joint within 3 to 5 meters (10 or 15) feet of a corner.

- Where steel columns are embedded in the walls, provide joints in the plane of the columns.

- If the columns are more than 8 meters (25 feet) apart, provide intermediate joints.

Numerous ways have been developed for forming control joints in walls. Whatever method is used, the thickness of the wall section at the joint should be reduced at least 20% by the depth of the joint; and the sum of the depths of the inside and outside grooves should not be less than 50 mm (2 inches).

(c) Construction Joints. Construction joints are used to allow concrete placement of separate construction elements at different times, e.g., between columns and beams, footings and pedestals, etc. Construction joints will be made with tie bars, dowels, or keys to provide shear transfer. The location and details of critical construction joints will be shown on the drawings and, to the extent practicable, will coincide with the location of expansion or control joints. The location of other construction joints need not be shown. Cautionary and advisory notes regarding acceptable joint locations will be included on the drawings.

(d) Seismic Joints. Buildings that are irregular in plan such as T, L, U, or cruciform shaped buildings can generate high torsional or twisting effects when subjected to earthquake ground motions. These structures would require a three-dimensional analysis for a rigorous determination of stress distribution. Since such analyses are generally not practical, seismic joints are provided to separate various blocks of the structure into regular shaped units that will not exhibit a torsional response. The joints should be of sufficient width to prevent hammering on adjacent blocks during earthquakes, and should be adequately sealed to protect the structure from the environment.

(e) Sealing joints. Exterior expansion, control, and construction joints should be sealed against moisture penetration using methods such as waterstops or sealants as appropriate for the prevailing conditions.

4-7. LOAD PATH INTEGRITY. Loads must be transferred from their point of application to the foundation. All structural elements and connections along the load path must have sufficient strength, and in the case of seismic resistant structures, sufficient ductility to transfer the loads in a manner that will not impair structural performance. Most load path deficiencies are a result of inadequate connections between precast elements, or between cast-in-place concrete elements and precast elements. Connections are often required to:

- Transfer shear from floor and roof diaphragms to the walls

- Transfer shear from the walls to the foundations

- Transfer shear between individual wall panels

- Transfer tension caused by overturning forces

- Transfer shear, bending, and axial loads between beams and columns and between beams and walls.

Connections between precast elements, or between cast-in-place concrete elements and precast elements can include the following types of connections:

- Column to foundation
- Column to column
- beam to column
- Slab to beam
- Beam to girder
- Beam to beam
- Slab to slab
- Wall to foundation
- Slab to wall
- Beam to wall
- Wall to wall

Details for these various types of connection can be found in the Prestressed Concrete Institute (PCI) Technical Report No. 2, "Connections for Precast Prestressed Concrete Buildings".

a. Shear Connections. Shear connections are classified as either "wet" or "dry". Wet connections use reinforced or unreinforced cast-in-place concrete to form the junction between members. Dry connections utilize a mechanical anchor, such as bolts or welded metal, to transfer load. Wet and dry connections use shear-friction resistance to transfer forces. In wet connections the reinforcing steel placed across the potential failure plane provides the clamping force needed to provide the shear-friction resistance. The most common type of dry connection involves embedded plates or other structural steel shapes that are anchored to the concrete by welded studs, anchor bolts, or expansion anchors. The embedded plates or structural steel shapes embedded in each of the concrete elements to be connected, are then connected themselves by weldments or by bolting. Contractors prefer the “dry” type connections because they are the easiest types to construct. The "wet" type connectors however are usually the best performers, especially under cyclic loading conditions such as occur during earthquakes.

b. Embedded Bolt and Headed Stud Anchors. Embedded anchor bolts and headed studs are commonly used to transfer shear and tension loads between cast-in-place concrete and precast concrete members and between cast-in-place concrete and structural steel shapes. Anchor connections should be designed and detailed to assure connection failure will be initiated by failure of the anchor steel rather than by failure of the surrounding concrete. The design of anchor bolt connectors will be based on the requirements of FEMA 302, Paragraph 9.2, "Bolts and Headed Anchors in Concrete." Additional information regarding the use of headed anchor bolts for anchorage can be found in the American Institute of Steel Construction (AISC) Engineering Journal, Second Quarter / 1983 Report, "Design of Headed Anchor Bolts," and in the Prestressed Concrete Institute (PCI) Design Handbook. Where strength design is used, the required strength, and the design of the anchors will be in accordance with FEMA 302. When allowable stress design (ASD) is used, the allowable service load for headed anchors in shear or tension, assuming the anchor bolts conform to ASTM 307 or an approved equivalent can be assumed equal to that indicated in Table 4-4. For ASD design, when anchors are subject to combined shear and tension, the following relationship will be used:

$$\left(\frac{P_s}{P_t}\right)^{5/3} + \left(\frac{V_s}{V_t}\right)^{5/3} \leq 1$$

Where:

P_s = applied tension service load

P_t = allowable tension service load from Table 4-4

V_s = applied shear service load

V_t = allowable shear service load from Table 4-4

The allowable service loads in tension and shear specified in Table 4-4 are for the edge distances and spacing specified. The edge distance and spacing can be reduced to 50 percent of the values specified with an equal reduction in allowable service load. Where edge distance and spacing are reduced less than 50 percent, the allowable service load will be determined by linear interpolation. Increase of the values in Table 4-4 by one-third is permitted for load cases involving wind or earthquake. Where special inspection is provided for the installation of anchors, a 100 percent increase in the allowable tension values of Table 4-4 is permitted. No increase in shear value is permitted.

c. Expansion Anchors. Expansion anchors will be designed in accordance with the provisions of ACI Committee 449 Report, "Concrete Nuclear Structures," Appendix B, "Steel Embedments." The engineer will review expansion anchor design features, failure modes, test results and installation procedures prior to selecting a specific expansion anchor for an application. Expansion anchors will not be used to resist vibratory loads in tension zones of concrete members unless tests are conducted to verify the adequacy of the specific anchor and application. Expansion anchors will not be installed in concrete where there is obvious signs or cracking, or deterioration.

Table 4-4. Allowable Service Load on Embedded Bolts
(Adapted from the International Building Code (IBC) - Final Draft, Table 1912.2)

Minimum Concrete Strength (f_c')

Bolt Dia.	Minimum Embed.	Edge Distance	Spacing	20 MPa (2500psi)	20 Mpa (2500psi)	25 MPa (3000psi)	25 MPa (3000psi)	30 MPa (4000psi)	30 MPa (4000psi)
				Tension	Shear	Tension	Shear	Tension	Shear
6.4 mm (1/4")	63.5 mm (2-1/2")	38.1 m (1-1/2")	76.2 mm (3")	890 N (200#)	2225 N (500#)	890 N (200#)	2125 N (500#)	390 N (200#)	2225 N (500#)
9.5 mm (3/8")	76.2 mm (3")	57.2 m (2-1/4")	114.3mm (4-1/2")	2225 N (500#)	4890 N (1,100#)	2225 N (500#)	4890 N (1,100#)	2225 N (500#)	4890 N (1,100#)
12.7 mm (1/2")	101.6 mm (4")	76.2 mm (3")	152.4mm (6")	4225 N (950#)	5560 N (1,250#)	4225 N (950#)	5560 N (1,250#)	4225 N (950#)	5560 N (1,250#)
12.7 mm (1/2")	101.6 mm (4")	127 mm (5")	152.4mm (6")	6450 N (1,450#)	7120 N (1,600#)	6670 N (1,500#)	7340 N (1,650#)	6890 N (1,550#)	7785 N (1,750#)
15.9 mm (5/8")	114.3 mm (4-1/2")	95.3 mm (3-3/4")	190.5mm (7-1/2")	6670 N (1,500#)	12,230 N (2,750#)	6670 N (1,500#)	12,270 N (2,750#)	6670 N (1,500#)	12,270 N (2,750#)
15.9 mm (5/8")	114.3 mm (4-1/2")	158.8mm (6-1/4")	190.5mm (7-1/2")	9460 N (2,125#)	13,120 N (2,950#)	9785 N (2,200#)	13,340 N (3,000#)	10,670 N (2,400#)	13,570 N (3,050#)
19.1 mm (3/4")	127.0 mm (5")	114.3mm (4-1/2")	228.6mm (9")	10,000 N (2,250#)	14,460 N (3,250#)	10,000 N (2,250#)	15,835 N (3,560#)	10,000 N (2,250#)	15,835 N (3,560#)
19.1 mm (3/4")	127.0 mm (5")	190.5mm (7-1/2")	228.6mm (9")	12,565 N (2,825#)	19,015 N (4,275#)	13,120 N (2,950#)	19,130 N (4,300#)	14,230 N (3,200#)	19,570 N (4,400#)
22.2 mm (7/8")	152.4 mm (6")	133.4mm (5-1/4")	266.7mm (10-1/2")	11,300 N (2.550#)	16,460 N (3,700#)	11,300 N (2,550#)	18,015 N (4,050#)	11,300 N (2,550#)	18,015 N (4,050#)
25.4 mm (1")	177.8 mm (7")	152.4mm (6")	304.8mm (12")	13,570 N (3.050#)	18,350 N (4,125#)	14,460 N (3,250#)	20.020 N (4,500#)	16,235 N (3,650#)	23,575 N (5,300#)
28.6 mm (1 1/8")	203.2 mm (8")	171.5mm (6-3/4")	342.9mm (13-1/2")	15,120 N (3,400#)	21,130 N (4,750#)	15,120 N (3,400#)	21,130 N (4,750#)	15,120 N (3,400#)	21,130 N (4,750#)
31.8 mm (1 1/4")	228.6 mm (9")	190.5mm (7-1/2")	381.0mm (15")	17,790 N (4,000#)	25,800 N (5,800#)	17,790 N (4,000#)	25,800 N (5,800#)	17,790 N (4,000#)	25,800 N (5,800#)

d. Adhesive (Chemical) Anchors. Adhesive anchors consist of a threaded rod installed in a hole drilled in hardened concrete and filled with a two-component epoxy, polyester, or vinylester resin adhesive. The hole is about 3 mm (1/8") larger than the bolt diameter. Adhesive anchors should not be used in structural elements that are required to be fire resistant. They should not be installed in wet or damp conditions or in concrete where there is obvious signs or cracking, or deterioration. No specific design codes are available for adhesive anchors. Therefore, design should be based on the manufacture recommendations, and testing should be required to assure the installed anchors meet strength requirements. Additional guidance on adhesive anchors can be found in the ACI Paper entitled "Bond Stress Model for Design of Adhesive Anchors", by Cook, R.A., Doerr, G.T., and Klingner, R.E., ACI Journal / Sept-Oct 1993.

e. Other Steel Embedments. Other steel embedments will be designed in accordance with the provisions of ACI Committee 449 Report, "Concrete Nuclear Structures," Appendix B, "Steel Embedments."

f. Special Considerations.

(1) Shear Transfer. The analysis of shear transfer will be in accordance with provisions of ACI 318. Special attention will be given to transfer of shear at locations such as shear heads, bases of walls, brackets and corbels.

(2) Compatibility. The combined action of flexible and rigid shear connectors will not be considered as providing simultaneous shear transfer. Rigid shear connectors include roughened surfaces and structural shapes. Flexible connectors include bolts, stirrups, dowel bars, and ties.

(3) Mechanical Anchorage of Reinforcing Steel. Mechanical connections are permitted for reinforcing steel in accordance with the provisions of ACI 318. There are many applications that make the use of mechanical connections feasible or more practical. Information on the types of mechanical connectors, their use, and design requirements can be found in the ACI Committee 439 Report, "Mechanical Connections of Reinforcing Bars".

4-8. DETAILING REQUIREMENTS. Details and detailing of concrete reinforcement will conform to ACI 315, "Details and Detailing of Concrete Reinforcement". Engineering and placing drawings for reinforced concrete structures will conform to ACI 315R, "Manual of Engineering and Placing Drawings for Reinforced Structures". For seismic areas, the design and details will conform to TI 809-04, "Seismic Design for Buildings".

4-9. SPECIAL INSPECTIONS.

- Periodic special inspection during and upon completion of the placement of reinforcing steel in intermediate moment frames, in special moment frames, and in shear walls is required.

- Continuous special inspection during the welding of reinforcing steel for structural members is required.

- Periodic special inspection during and upon completion of the placement concrete in intermediate moment frames, in special moment frames, and in shear walls is required.

- Periodic special inspection is required during the placement and after completion of placement of prestressing steel, and continuous special inspection is required during all stressing and grouting operations and during the placement of concrete.

- Periodic special inspection is required during the placement of anchor bolts, expansion anchors, and chemically grouted anchors to verify the anchor system is in conformance with approved plans and specifications.

CHAPTER 5

SLABS ON GRADE

5-1. INTRODUCTION. This chapter provides design and construction requirements for industrial type (non-residential) lightly loaded slabs on grade. Industrial type slabs refer to those slabs that are reinforced to minimize the number of crack control joints (maximize joint spacing), Guidance for unreinforced residential type concrete slabs, which utilize crack control joints spaced at frequent intervals to minimize cracking, can be found in the PCA Publication, "Concrete Floors on Ground." Lightly loaded slabs on grade are those supporting stationary live loads of not more than 20 kPa (400 pounds per square foot), stationary concentrated line (wall) loads of not more than 15 kPa (300 pounds per foot), or vehicle axle loads of not more than 2275 kg (5000 pounds). The guidance is applicable to usual exposure conditions meaning interior locations other than airplane hangars where slabs are not subject to extreme climatic changes, and to typical subgrade conditions characterized by sufficient underdrainage to prevent frost penetration, the absence of a wet environment, i.e., volume change due to change in moisture content is limited, and the absence of expansive soils. In addition, typical subgrade conditions are deemed to include only soils classified according to ASTM D 2487, "Classification of Soils for Engineering Purposes," as either Class ML, any of the S or G groups, or Class CH, CM, or CL having a modulus of subgrade reaction (k) of 2.75 kg per cubic centimeter (100 pounds per cubic inch) or greater. Although slabs on grade may be designed to perform satisfactorily on subgrades of lower strength, design for such conditions is beyond the scope of this manual. Refer to TI 809-27, "Concrete Floor Slabs on Grade Subjected to Heavy Loads," for the design of slabs on grade subjected to heavy loads, and to TI 809-28, "Design and Construction of Reinforced Ribbed Mat Slab" for the design of a commonly used slab-on-grade system for resisting potential foundation movements at sites containing expansive soils. Additional information on the design of slabs on grade, primarily industrial floors, can be found in ACI Committee 360 Report, “Design of Slabs on Grade.”

5-2. BASIS FOR DESIGN.

a. General. Slabs-on-grade will be designed for bending stresses due to uniform loads and concentrated loads and for in-plane stresses due to drying shrinkage and subgrade drag resistance. When appropriate for the type facility being designed, slabs will be designed for the effects of warehouse loadings involving aisles, posts and racks, etc. In such instances, particular attention will be given to the design for negative moment in aisles.

b. Guidance. Proper construction methods, workmanship, and concrete mix proportioning will follow the guidelines of ACI Committee 302 Report, "Guide to Concrete Floor and Slab Construction". Slabs are required to have a minimum thickness of 100 mm (4 inches). The following thickness for maximum uniform design live loads will be used provided the modulus of subgrade reaction (k) is at least 2.75 kg per cubic centimeter (100 pounds per cubic inch).

Thickness of Slab	Maximum Uniform Design Live Load
100 mm (4”)	7 Kpa (150 psf)
150 mm (5”)	12 Kpa (250 psf)
200 mm (6”)	20 Kpa (400 psf)

Unless otherwise specified above, the correct slab thickness will be determined in accordance with the Portland Cement Association (PCA) Publication entitled, "Slab Thickness Design for Industrial Concrete Floors on Grade." In the PCA design process compressive strength is converted to modulus of rupture, which is then reduced by a factor of safety to obtain the maximum allowable flexural tensile stress. The maximum allowable flexural tensile stress is then used to find the required slab thickness. For typical values of modulus of subgrade reaction refer to TI 809-27, "Concrete Floor Slabs on Grade Subjected to Heavy Loads." When wall loads exceed 15 kPa (300 lb./foot) slabs-on grade will be thickened in accordance with the provisions of TI 809-27.

5-3. SERVICEABILITY.

a. General. Cracking, warping, and curling can impair slab-on-grade serviceability. These problems are directly attributable to drying shrinkage. Cracking can be controlled by minimizing drying shrinkage, by providing adequate crack control and isolation joints, and through the use of reinforcing steel. Water penetrating the slab is a common serviceability problem that can be cured by proper drainage and by the use of vapor barriers.

b. Minimizing Drying Shrinkage. Cracking in slabs generally results from drying shrinkage and restraint caused by friction between the slab and subgrade. Curling and warping occur due to differential shrinkage when the top of the slab dries to lower moisture content than the bottom of the slab. Drying shrinkage, curling, and warping can be reduced by using less water in the concrete. Ways to reduce water content include using the largest maximum sized aggregate (MSA), using an MSA equal to 1/3 the slab thickness, and by using coarse sand. Water content can also be reduced by using coarser ground cement and cement with a low C_3A content. On large and critical slab-on-grade projects the designer should request by specification that shrinkage tests be made of several concrete mixes to obtain a mix with the lowest drying shrinkage potential.

c. Controlling Cracking through Control and Isolation Joints. Control and isolation joints can be used to minimize cracking and to force cracking to occur at joint locations. Designers should attempt to minimize the number of joints occurring in the slab. However, in most instances, the maximum slab area bound by crack control joints should not exceed 60 square meters (625 square feet), and distance between crack control joints should not exceed 7.5 meters (25 feet). The length/width ratio of panels bounded by joints should be as near 1.0 as possible and should not exceed 1.25. In localities where extreme conditions of heat or dryness tend to produce excessive shrinkage, the maximum area and joint spacing may need to be decreased. Crack control joints may also be construction joints. Joints in the vicinity of column pedestals will be placed at column centerline, with diamond shaped or circular isolation joints provided at columns or square shape isolation joints provided at column pedestals. When thickened slabs are used under column bases or partitions, joints should be offset from the thickened areas. Corners of isolation joints will meet at a common point with other joints so far as practicable. Where discontinuous joints, (i.e., joints which are not continuous across their perpendicular joints (see Figure 5-1)), cannot be avoided, two No. 13 bars, 1.25 meters long (two No. 4 bars, 4 feet long), will be placed parallel to the edge opposite the end of the discontinuous joint. Bars will be at mid-depth and 100 mm (4 inches) apart starting 50 mm (2 inches) from edge of slab. Except for openings of less than 300 mm by 300 mm (12 inches by 12 inches), corners of openings and reentrant corners in slabs will be reinforced with two No. 13 bars, 1.25 meters long (two No. 4 bars, 4 feet long), placed diagonally to the corner.

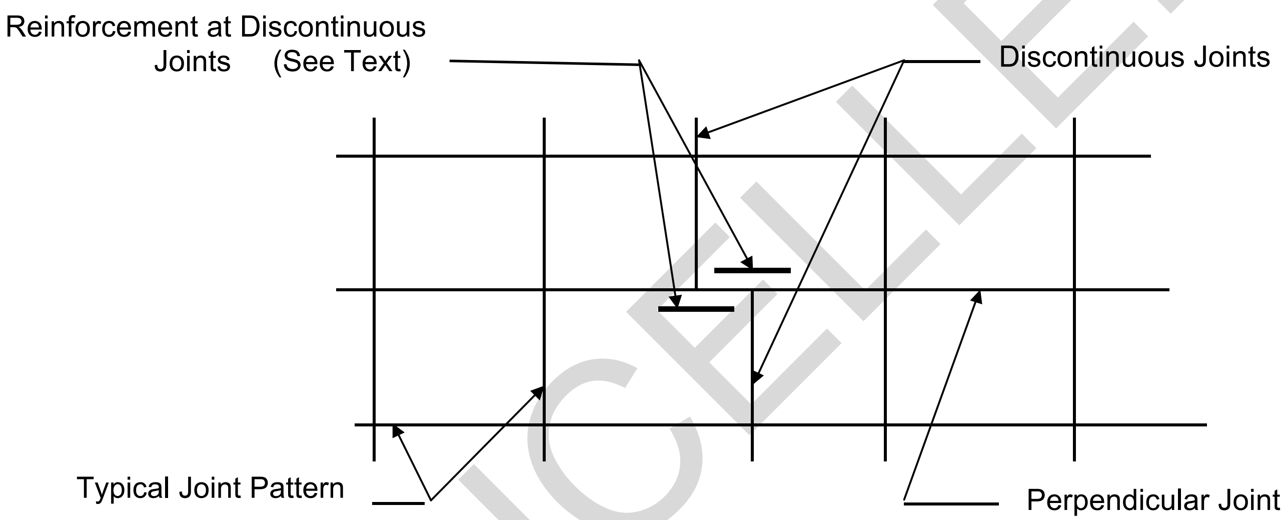

Figure 5-1. Discontinuous Joints.

(1) Control Joints. Control joints form a weakened plane to direct cracking to preselected locations. Sawed control joints will be cut to one-fourth depth of slab thickness (H). Details of control joints are shown in figure 5-2. Control joints may be made in floors scheduled to receive a floor covering by inserting fiberboard strips in the unset concrete. Depth of fiberstrip should be one-fourth of the slab thickness. Location and details of control joints will be shown on drawings.

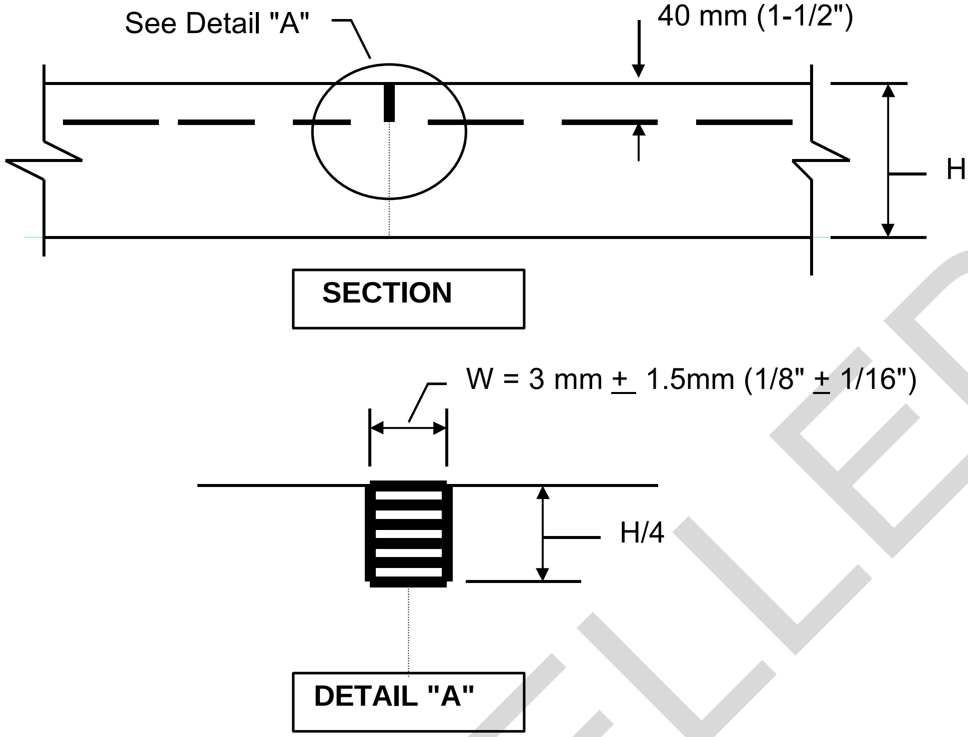

Figure 5-2. Control Joints.

Figure 5-2 Notes:

1. Concrete cover of 40 mm (1-1/2 inches) will be provided over reinforcement.

2. One-half of the welded wire mesh reinforcement (alternate wires) will be interrupted within 50 mm (2 inches) of each side of slab control joints

(2) Isolation joints. Isolation joints form a separation of elements from the slab on grade and permit both horizontal and vertical relative movement. Isolation joints should be provided between the abutting faces of floor slab and fixed parts of the structure such as columns, walls, and machinery bases. At locations where slabs abut vertical surfaces, such as interior and exterior foundation walls and column pedestals, isolation joints will ordinarily be a strip of 15-kg (30-pound) felt serving as a bond breaker. At exterior walls, perimeter insulation extended to the top of slab will serve the purpose. Where slabs will expand due to radiant heating systems, or extreme temperature changes, and where isolation from vibrations of machinery and equipment foundations is required, joint filler 10 mm (3/8 inch) or more thick will be required. Location and details of isolation joints will be shown on drawings. A typical isolation joint is shown in Figure 5-3.

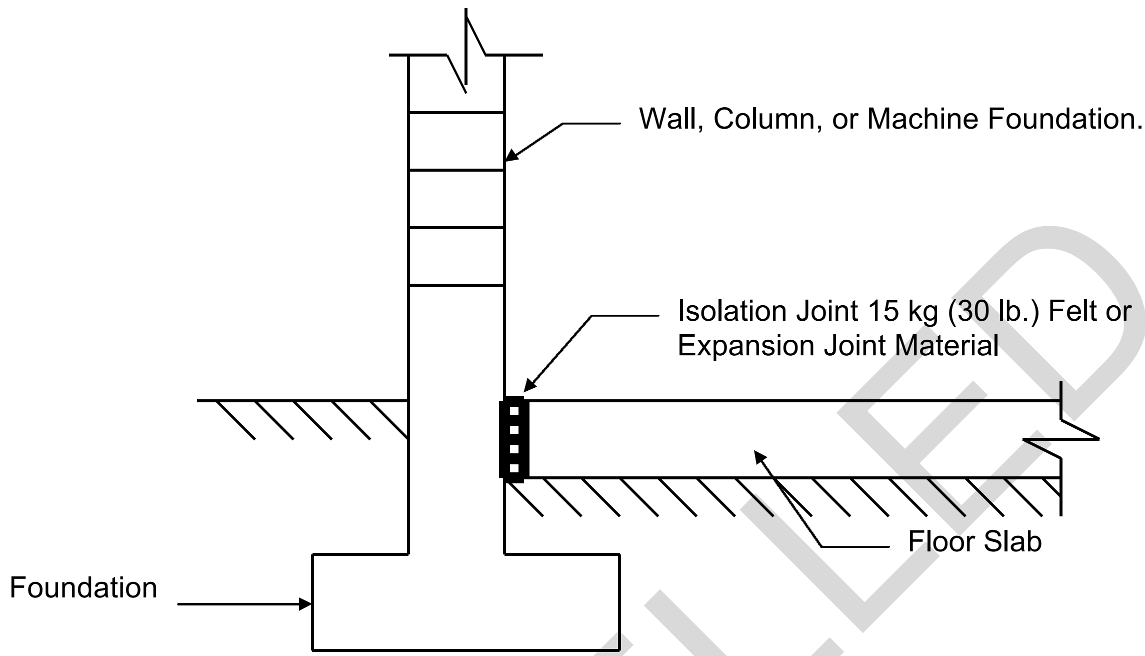

Figure 5-3. Isolation Joints

(3) Construction joints. Construction joints are used to allow separate concrete placement. Construction joints should be kept to a minimum, and should generally be in conformity with a predetermined joint layout. Construction joints will have dowels, or keys to provide shear transfer. Dowel size and spacing should be in accordance with ACI Committee 302 Report, "Guide to Concrete Floor and Slab Construction." Formed-keyed joints will only be used in slabs having a thickness of 150mm (6 inches) or more. In order to accommodate keyed joints in 100 mm (4-inch) thick slabs it is acceptable to taper the slab so the slab at joint locations is 150 mm (6-inches) thick. The taper should begin 1 meter (3 feet) each side of the joint. Preformed keys left in place may be used for 100 mm (4 inch) and thicker slabs. The key will be centered on the depth of the slab with the base of the male portion about one-third the depth of the slab. Location and details of construction joints will be shown on drawings. Details of construction joints are shown in Figure 5-4.

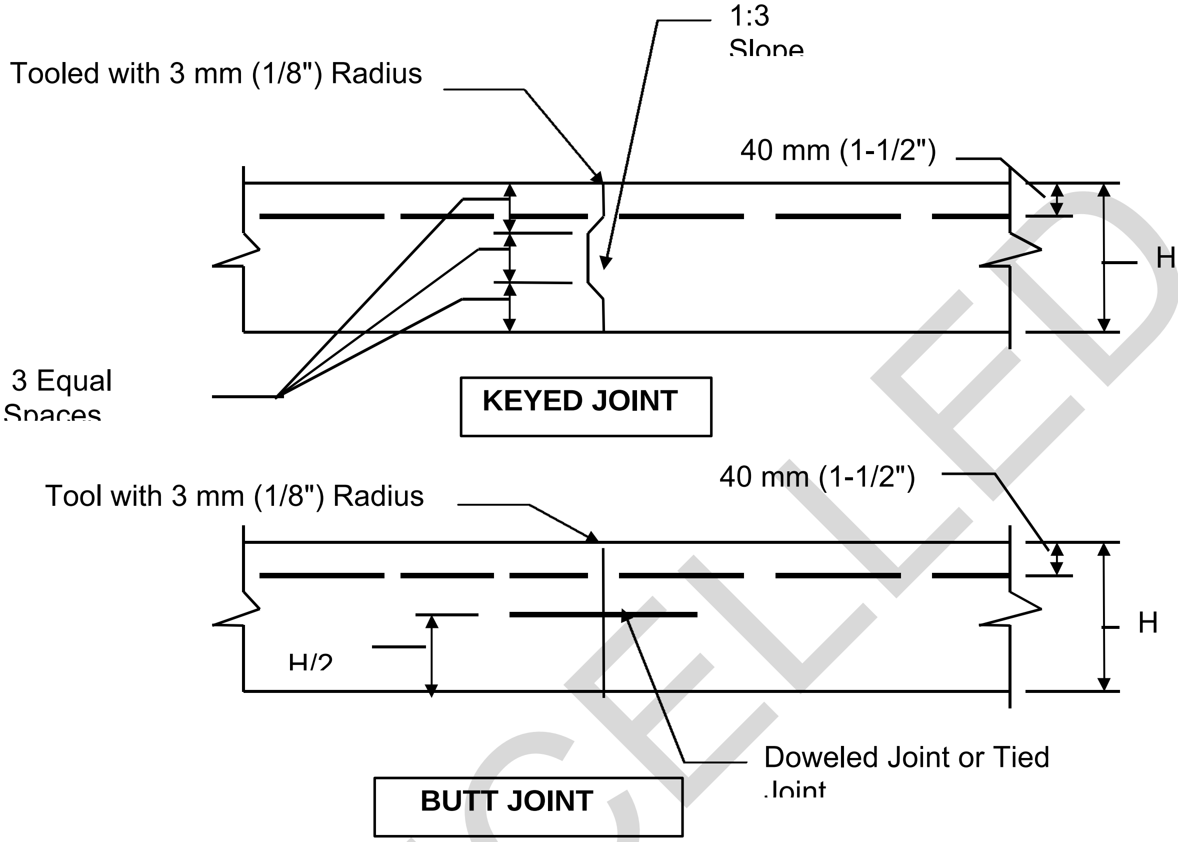

Figure 5-4. Construction Joints.

Figure 5-4 Notes:

1. Keyed joints or doweled joints should align with and function as a control joint or expansion joint. For doweled joints use a 20 mm (3/4") diameter x 400 mm (16 ") long bar and lubricate one half the bar. Dowels should be spaced at 400 mm (16" o.c.) for slab thickness 150 mm (6 inches) or less, and 300 mm (12 inch) spacing for slab thickness greater than 150 mm (6 inches).

2. Tied butt joints constructed with deformed rather than smooth bars will be used when the construction joint is not at a planned control or expansion joint location. This type of joint restrains movement. Use 13 mm (1/2") diameter x 750 mm (30 ") long deformed tie bars spaced at 750 mm (30 ") on centers.

3. Concrete cover of 40 mm (1-1/2 inches) will be provided over reinforcement.

4. Welded wire reinforcement (sheets not rolls) will be stopped 50 mm (2 inches) of each side of planned control or expansion joint locations. Welded wire reinforcement will be continuous through tied butt joints.

d. Slab-On-Grade Reinforcement.

Slab-on-grade shrinkage reinforcement will be located 40 mm (1-1/2 inches) from the top of the slab in order to restrain shrinkage and reduce curling. Maximum spacing of reinforcing bars should not exceed three times the slab thickness. For plain wire reinforcement the spacing should not be more than 350 mm (14 inches) longitudinally and 350 mm (14 inches) transversely. The percentage of steel determined should not be less than 0.15 percent. Wire mesh reinforcements meeting the 0.15 percent steel requirement, for various slab thickness, is provided in Table 5-1. Deformed welded wire fabric in flat sheets, or deformed reinforcing bars will be used. The positioning of the steel in the slab is critical for proper crack control. Reinforcing steel will be supported on chairs and every precaution taken to assure the reinforcing bars are positioned, as intended after construction is complete.

Table 5-1. Minimum Slab-On-Grade Reinforcement Requirements.

Slab Thickness	Wire Mesh Reinforcement
100 mm (4-inch)	305 x 305 - MW 48.4 x MW 48.4 (12 x 12 - W 7.4 x W 7.5)
125 mm (5-inch)	305 x 305 - MW58.1 x MW 58.1 (12 x 12 - W9 x W 9)
150 mm (6-inch)	305 x 305 - MW 71.0 x MW 71.0 (12 x 12 - W11 x W 11)

Suppliers because of the large bar sizes and wide spacing may not normally stock the wire mesh reinforcement sizes indicated in Table 5-1. The large bar sizes are desirable to prevent bending of the steel and provide adequate stiffness to keep the steel in the upper half of the slab during concrete placement. For smaller jobs [less than 7000 square meters (75,000 square feet)] in those cases when the wire mesh sizes listed in Table 5-1 are unavailable, wire mesh spacing with reduced bar diameters and closer spacing may be used provided minimum steel requirements are met.

e. Vapor Barriers. High levels of moisture in the subgrade increase slab curling. If the subgrade can become moist because of ground water an impermeable vapor barrier should be provided. The minimum thickness of the vapor barrier should be 1.3 mm (50 mil) and be covered with a 150 mm (6 inches) of crushed stone topped with a 13 mm (1/2 inch) thick layer of sand. The advantage of the 165mm (6-1/2 inch) stone / sand cover over the vapor barrier is that the vapor barrier will not be punctures nor will the fill material be easily displaced as construction equipment is driven over the stone and sand cover (Ytterberg, 1987). If the only purpose of the vapor barrier is to reduce friction between the slab and subgrade in order to reduce drag, then a polyethylene slip sheet can be placed directly under the slab provided holes are punched in the polyethylene to allow water to leave the bottom of the slab before final set occurs in the concrete.

5-4. SHINKAGE COMPENSATING CONCRETE. Edge curling and shrinkage cracking can be reduced through the use of shrinking compensating concrete that is properly reinforced. The ACI Committee 223 Report, "Standard Practice for the Use of Shrinkage-Compensating Concrete" recommends that the area of steel reinforcement be greater than or equal to 0.15 percent of the cross-sectional area of the concrete and less than or equal to 0.20 percent, and that it be located in the upper half of the slab.

5-5. POST-TENSIONED SLABS. Post-tensioning of slabs-on-grade can be useful where it is desirable to eliminate joints, design the slab-on-grade to resist foundation movements at sites containing expansive soils, or to provide the slab-on-grade system with the capacity to span over weak subgrade areas. The Post-Tensioning Institute (PTI) Reports, "Design and Construction of Slabs-on-Ground," and "Post-Tensioned Commercial and Industrial Floors" can be used as a basis for design of post-tensioned slabs.

5-6. SPECIAL TESTS AND SPECIAL INSPECTIONS. In addition to the usual concrete testing, shrinkage testing of concrete mixtures to minimize shrinkage potential, modulus of rupture testing of concrete mixtures to maximize the ability of the concrete to resist tensile stress form shrinkage and load induced flexure, should be considered.

CHAPTER 6

MASONRY

6-1. INTRODUCTION. Design guidance for reinforced masonry structures is provided. Plain (unreinforced) masonry design and design using "empirical" methods are generally not permitted.

6-2. BASIS FOR DESIGN. Reinforced masonry will be designed by allowable stress design (ASD) methods in accordance with TI 809-06, "Masonry Structural Design for Buildings. Loads will be in accordance with Chapter 1, Paragraph 7. Load combinations will be in accordance with Chapter 1, Paragraph 8. Special design and detailing requirements for masonry structures subjected to earthquake ground motions are provided in FEMA 302, "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and in TI 809-04, "Seismic Design for Buildings". For buildings in Seismic Design Categories D, E, and F, shear keys at the base of masonry walls may be necessary. Designers must be aware of, and comply with all seismic design and detailing requirements.

6-3. SERVICEABILITY. Detailing of masonry structures to prevent efflorescence, and to prevent cracking due to shrinkage and temperature movements, should be in accordance with recommendations in TI 809-06. Large expanses of roof deck with supporting systems rigidly attached to masonry walls, pilasters, or columns can result in cracking due to thermal changes that take place during construction. Designers should therefore include provisions in the design to accommodate thermal movement both during and after construction. Control joints and expansion joints are used to control cracking due to shrinkage and thermal movement. The joints should permit movement but have sufficient strength to resist required loads. Joints should be weather tight when located in exterior walls. Joint location and detailing requirements are provided in TI 809-06.

6-4. LOAD PATH INTEGRITY. Information on the design and detailing of connections between shear walls (vertical resisting elements) and floor and roof diaphragms (horizontal resisting elements) are provided in TI 809-04, TI 809-06, and FEMA 302. The design of headed anchor bolts and bent-bar anchors embedded in masonry, and used to connect diaphragms and structural members to masonry, will be in accordance with FEMA 302, Paragraph 11.3.12, "Headed and Bent-Bar Anchor Bolts".

6-5. SPECIAL INSPECTIONS.

- Periodic special inspection during the preparation of mortar, the laying of masonry units.

- Periodic inspection of vertical, horizontal, and bond beam corner reinforcement.

- Continuous special inspection during the welding of reinforcement, grouting, consolidation, reconsolidation and placement of anchors (bent bar or anchor bolts).

CHAPTER 7

STEEL STRUCTURE DESIGN REQUIREMENTS

7-1. INTRODUCTION. This chapter prescribes criteria for the design of structural steel, open-web joists, and cold-formed steel structural members in buildings.

7-2. GENERAL. Structural framing systems and elements of buildings will be designed in accordance with the accepted industry standards described below. The type of steel and unit dimension, (bay size, story, height, etc.), the system for structural framing, and the design method used will be based on a comparative economic study and will be those that result in the least cost for the required structure.

7-3. BASIS FOR DESIGN.

a. Structural Steel Construction. The design, fabrication and erection of structural steel for buildings and structures will be in accordance with either the AISC Load and Resistance Factor Design (AISC-LRFD) or AISC Specification for Structural Steel Buildings-Allowable Stress Design (AISC-ASD). The seismic design of steel structures will be in accordance with the additional provisions of TI 809-04, "Seismic Design for Buildings". Guidance on cold-formed steel can be found in TI 809-07, "Design of Load Bearing Cold-Formed Steel Systems and Masonry Veneer / Steel Stud Walls".

b. Cold-Formed Steel. The design of cold-formed carbon and low alloy steel structures will be in accordance with the TI 809-07. The design of cold-formed stainless steel members will be in accordance with ASCE 8, "Specifications for the Design of Cold-Formed Stainless Steel Structural members. Composite slabs of concrete on steel deck will be designed and constructed in accordance with ANSI/ASCE 9, "Standard for the Structural Design of Composite Slabs". The seismic design of cold-formed steel structures will be in accordance with the additional provisions of TI 809-04.

c. Steel Joists. The design, manufacture, and use of open web steel joists and joist girders will be in accordance with one of the following Steel Joist Institute (SJI) specifications:

- Standard Specifications for Open Web Steel Joists, K Series

- Standard Specifications for Longspan Steel Joists, LH Series, and Deep Longspan Steel Joists, DLH Series, or

- Standard Specifications for Joist Girders.

d. Steel Cables Structures. The design, fabrication, and erection, including related connections, and protective coatings of steel cables for buildings will be in accordance with ASCE 19, "Structural Applications of Steel Cables for Buildings" except as modified as follows for seismic design:

- Section 5d of ASCE 19 will be modified by substituting (1.5 T_4) where T_4 is the net tension in the cable due to dead load, prestress, live load, and seismic load. A load factor of 1.1 will be applied to the prestress force to be added to the load combination of Section 3.1.2 of ASCE 19.

e. Crane Runways and Supports.

(1) Stops and Bumpers. Stops refer to rigid assemblies installed at the ends of crane runways to prevent traveling cranes from running beyond the ends of the runway. Bumpers refer to those devices (usually fitted onto the crane) which are resilient or other energy absorbing construction designed to limit the deceleration force resulting from the crane's hitting the runway stops. Stops engaging the thread of the wheel will not be less than the radius of the wheel. Stops engaging other parts of the crane are recommended. Requirements for the design of crane stops are controlled largely by the design of the crane bumpers. Procurement documents for cranes will mandate that crane bumpers be designed in accordance with requirements of the Occupational Safety and Health Act (OSHA) including.

- Bumpers will be capable of stopping the crane (not including lifted load) at an average deceleration of no more than 3 feet per second per second with the crane traveling at 20 percent of rated speed.

- Bumpers will, at a minimum, have sufficient energy absorbing capacity to stop the crane when its traveling at 40 percent of rated speed. The forces to be resisted by the stops will either be indicated by the crane manufacturer or determined as set forth in Whiting Corporation Overhead Crane Handbook.

(2) Deflections. Vertical deflection of crane runway girders will be limited as set forth in Crane Manufacturer's Association of America (CMAA) 70 and 74. Horizontal deflection will be checked to assure compatibility with clearance between flanges of double-flanged wheels and bearing area of single-flanged wheels.

(3) Additional Information. Additional information useful in the design of crane runways and supports can be found in the Navy MIL-HDBK-1038, "Weight Handling Equipment", and Navy guidance document No. P-307, "Management of Weight Handling Equipment". These documents provide information on crane loadings that can occur during operation and testing.

7-4. SERVICEABILITY.

a. General. AISC Specification for Structural Steel Buildings-Allowable Stress Design (AISC-ASD) provides design guidance with respect to the following serviceability related issues:

- Camber
- Expansion and contraction
- Deflection, vibration and drift
- Connection slip

- Corrosion

b. Camber. Camber is especially important in truss type structures such as those typically used over rolling doors in aircraft hangars. In general, however, the use of camber to offset dead load deflections in long span beams, girders, and joists has limited benefits with respect to building serviceability. Camber however can improve the appearance of structures where the support systems are exposed to view.

c. Expansion and Contraction. Expansion joints should be provided when necessary to minimize detrimental effects resulting from the lateral movement of long structures due to thermal expansion and contraction. In general, for non-rectangular steel-framed buildings, the maximum allowable building length without an expansion joint (or maximum length between expansion joints) is 90 meters (300 feet) for moderate climates (temperature change less than 0^{o} C (30^{o} F)). This decreases to 60 meters (200 feet) for extreme climates (temperature change greater than 20^{o} C (70^{O} F)). These distances may be increased for rectangular buildings, however, they should be decreased if the building is unheated, if the building is air conditioned, or if the building has fixed-base columns. Additional information on expansion joints in buildings can be found in Technical Report No. 65, “Expansion Joints in Buildings,” National Academy of Sciences, 1974. A double column arrangement is the preferred method of establishing an expansion joint. Additional joints may be required at the junctures of T-, L-, U-shaped and other irregularly shaped buildings, or when foundation conditions create a potential for differential settlement.

d. Deflection, Vibration and Drift. Displacements of structural framing system under service load conditions must be controlled to prevent damage to various architectural features such as interior walls, partitions, ceilings, and exterior cladding. Deflection limits, expressed as a function of span length are provided in Table 1-4 and in various codes (AISC, AISI, MBMA, etc.). Drift limits for earthquake loadings are provided in FEMA 302. Designers should verify the deflection and drift limits imposed by code are suitable. Drift limits more stringent than those imposed by FEMA 302 may be warranted for those conditions where non-ductile cladding is used.

e. Connection Slip. When connection slip can cause a significant displacement increase in the framing system and thereby raise concerns about building serviceability, the use of slip-critical connections should be considered. Other reasons for using slip-critical connections are provided in Paragraph 5d.

f. Corrosion. Corrosion can reduce structural capacity as well as cause serviceability problems. Painting of structural steel will comply with the requirements contained in either AISC-LRFD or AISC-ASD. Except where fabricated of approved corrosion resistant steel or of steel having a corrosion resistant or other approved coating, individual structural members and assembled panels of cold-formed steel construction must be protected against corrosion with an approved coat of paint. Requirements for the protection of steel in corrosive environments are provided in Chapter 12.

g. Requirements for Wear Protection. The total thickness of design sections subject to wear will be increased beyond that required to meet stress requirements. The amount of such increase will be based on the material involved, the frequency of use, and the designed service life. Estimates of the wear requirement will be based on previous experience or accepted practice for the application. Use of replaceable wear plates should be considered where extremely severe conditions exist.

7-5. LOAD PATH INTEGRITY.

a. General. Connections between framing elements are critical. They must perform at limit state and service load levels as intended to assure load path integrity is maintained during extreme loading events. Under service loading conditions, connection displacements and rotations should not lead to serviceability problems. Connections can be welded or bolted, or a combination of both welds and bolts. Design of welded and bolted connections will be in accordance with the either AISC Load and Resistance Factor Design (AISC-LRFD) or AISC Specification for Structural Steel Buildings-Allowable Stress Design (AISC-ASD). Bolted connections can be shear-bearing type connections, slip critical connections, or direct tension connections. Field welding of connections should be minimized to the maximum extent possible.

b. Design of Connections for Wind and Seismic Loads. Connection design philosophy is different for earthquake than it is for wind. Connections used to resist wind load are designed to perform in the elastic range. Connections used to resist earthquake loads, although designed in a fashion similar to that for wind load, are expected to experience forces greater than the code level design forces. This requires that earthquake resistant connections perform in a ductile manner. Under certain circumstances, such as moment frame connections, bearing connections may be desirable for wind load, but slip-critical connections may be recommended to resist earthquake loads. Slip critical connections are recommended for earthquake resistant beam-column connections where web connections are bolted and flange connections are welded. Slippage of the bolts in these type connections can increase loads on the flange welds resulting in connection failure (Reference UCB/EERC-83/02 Report, "Seismic Moment Resisting Connections for Moment-Resisting Steel Frames": Keep in mind that the ultimate load capacity of a slip-critical connection is the capacity of the bolts loaded in shear/bearing. If slip-critical bolts in the web connection of a moment resistant joint are too near the yielding flanges, the bolts may experience sufficient force to slip into a shear/bearing type of response, which may have a lower capacity than the slip-critical type of response.). There are significant differences between the strengths allowed for shear-bearing, and slip critical bolts. Therefore, the designer must check both wind and seismic loading conditions to make sure the connection satisfies both shear-bearing and slip-critical requirements.

c. Shear-bearing Type Connections. In a shear-bearing type connection, shear forces are transferred through the connection by bolts that act in shear. The connected material is in bearing adjacent to the bolts and must be evaluated for its load bearing capacity. Bolt shear strength is about 62 percent of the bolt-ultimate tensile strength. It should be recognized that all shear and moment connections at ultimate load conditions act in shear-bearing whether they are designed as shear-bearing connections or as slip-critical connections. The bolts in shear-bearing type connections are installed "snug tight" since shear strength in this type of connection is not related to the pretension load in the bolt. The bolts are usually ASTM A307 bolts with an ultimate tensile strength of 450 MPa (60ksi), although "snug tight" high-strength ASTM A325 and ASTM A490 bolts are also permitted for shear-bearing type connections with some restrictions. ASTM A325 bolts have an ultimate tensile strength of 830MPa (120 ksi), and ASTM A490 bolts have an ultimate tensile strength of 1040 MPa (150 ksi).

d. Slip Critical Connections. Slip critical connections transfer shear load by shear-friction with the normal force provided by pretensioning the bolts. The shear strength at service load levels is dependent on the pretension load (normal force on joint) and the friction between joined materials. Slip-critical connections are generally required when loads are repetitive and fatigue is a concern, where connection slip would effect structure performance and where bolts and welds must act in unison to resist applied loads. Bolts are pretensioned to at least 70 percent of the bolt-minimum tensile strength. The tension load is applied by tightening the nut, or otherwise elongating the bolt sufficiently to develop the prescribed pretensioning force. Turn-of-the-nut, calibrated wrench, and direct tension indicators are all methods used to assure the bolt has been properly tensioned.

e. Bolts in Direct Tension. When bolts act in tension to resist applied loads, the possibility of, and the effect of prying action on bolt tension must be considered.

f. Steel Properties. Strength and ductility are properties important to building performance. Ductility is dependent on the yield strength and chemical composition of the steel. Buildings located in high seismic areas must be ductile if they are to survive major earthquakes. Designers must select steels that meet needed strength and ductility requirements. They must recognize that ASTM A572 Grade 50 steel will meet ASTM A36 requirements and most likely will be used even though ASTM A36 steel is specified. Designers specifying ASTM A36 steel must consider the impact a higher yield strength will have on the seismic design and make sure the substitution does not adversely impact seismic performance. Connections designed to develop the strength of the connected members may not perform as intended if the connected members were specified to be ASTM A36 steel and the contractor elected to substitute members of ASTM A572, Grade 50 steel.

7-6. SPECIAL INSPECTIONS.

- Periodic special inspection of the installation and tightening of fully tensioned high-strength bolts in slip critical connections and in connections subject to direct tension is required.

- Continuous special inspection of all structural welding, except where periodic special inspection is allowed in FEMA 302, Chapter 3.

CHAPTER 8

STRUCTURAL WELDING

8-1. INTRODUCTION. A weldment may contain metals of different compositions, and the components of the framing system may be rolled shapes, pipes, tubes, or plates. In general, all metals are weldable, but some are much more difficult to weld than others. Certain types of reinforcing steels used in concrete and masonry construction can be very difficult to weld. Various types of joints are used to connect framing components, including butt joints, corner joints, edge joints, lap joints, and T-joints. Various types of welds are used in joining steel components together, and several different welding processes can be used to make the weld. Structural engineers involved in the design of weldments must be familiar not only with the various types of joints and types of welding procedures, but also with the effect welding position has on the selection of welding procedures, and with the effect factors such as base metal composition, electrode selection, preheat and interpass temperature, wire feed speed, travel speed, post weld treatment, ambient temperature, etc., have on weld performance. Volume change effects due to temperature gradients that occur due to welding, joint restraint conditions, and weld toughness are factors which must also be considered to assure weldments will be free from cracks that could reduce joint capacity and ductility. Corner joints and poor weld termination can cause stress concentrations that can also lead to weldment cracking.

8-2. BASIS FOR DESIGN. Weldments will comply with the American Welding Society (AWS) AWS D1.1, "Structural Welding Code - Steel". AWS D1.1 contains many prequalified joint details that are known to produce quality weldments when fabricated in accordance with AWS Welding Procedure Specifications (WPS). WPS's are required for all welding, including prequalified procedures. Additional guidance on welding can be found in TI 809-26, "Welding Guidance for Buildings."

8-3. ARC WELDING PROCESSES. The most popular arc welding procedures are Shielded Metal Arc Welding (SMAW) and Flux Core Arc Welding (FCAW). The SMAW process is defined as "an arc welding process that produces coalescence of metals by heating them with an arc between a covered metal electrode and the work. Shielding is obtained from decomposition of the electrode covering. Pressure is not used and filler metal is obtained from the electrode." The FCAW process is defined as " an arc welding process that produces coalescence of metals by heating them with an arc between a continuous filler metal (consumable) electrode and the work. Shielding is produced by a flux contained within the tubular electrode. Additional shielding may or may not be obtained from an externally supplied gas or gas mixture". Shielding refers to methods used to prevent the molten weld metal from coming in contact with gasses contained in the surrounding air. These gases, especially oxygen, nitrogen, and hydrogen, are the most detrimental to weld quality. Other types of arc welding processes are carbon arc welding (CAW), gas tungsten arc welding (GTAW), plasma arc welding (PAW), submerged arc welding (SAW), and gas metal arc welding (GMAW). The type of base metal usually determines which welding processes can be used.

8-4. WELDING POSITIONS. Welding positions include the flat position, the horizontal position, the vertical position, and the overhead position. Certain welding processes have limitations on which welding positions can be used. The structural engineer designing the weldment must be familiar with these restrictions, and make sure the type of weldment specified can actually be applied as intended considering access conditions, weld position, and any environmental conditions that could adversely influence weldment fabrication.

8-5. WELDMENTS SUBJECTED TO EARTHQUAKE AND CYCLIC LOADING CONDITIONS. Certain types of welds are not permitted to join members that may experience cyclic loadings due to earthquake ground motions, or due to vibratory motion caused by equipment. Partial penetration butt joints in tension, intermittent groove welds, and intermittent fillet weld are examples. Special Moment Resisting Frames (SMRF's) and Ordinary Moment Resisting Frames (OMRF's) used for earthquake resistance must be capable of performing in the nonlinear range (beyond yield) during major earthquakes. For SMRF's this means the weldments designed for beam column joints must have greater strength than the connected members so that yielding occurs in the beam span away from the beam column joint. Welded steel frame design for seismic resisting moment frame systems is covered by the American Institute of Steel Construction (AISC), Seismic Provisions for Structural Steel Buildings".

8-6. WELDMENT STRENGTH AND DUCTILITY. Base metals selected for components of the structural framing system must possess certain strength and ductility characteristics. It is important that the electrodes for joint weldments, as well as welding procedures, be selected to produce a joint with strength and ductility properties equal to, or superior to, those of the base metal.

8-7. ENVIRONMENTAL FACTORS IMPORTANT TO WELDMENT PERFORMANCE. Cracking can occur due to environmental factors such as the presence of moisture, and low ambient temperatures. Field welding should be avoided as much as possible. However, when field welding is required, especially under adverse environmental conditions, it is important that AWS welding specification procedures be followed to the letter. Preheating and inter-pass temperatures are critical if cracking is to be prevented. Preheating also helps to drive of excess surface moisture, and retards the cooling rate thereby minimizing temperature gradients. Controlling cool-down rates, especially during cold weather, is critical if cracking is to be eliminated.

8-8. WELDING REINFORCING STEEL. The welding of reinforcing steels is covered by AWS D 1.4, "Structural Welding Code - Reinforcing Steel." Most reinforcing steel bars can be welded. However, the preheat and other quality control measures that are required for bars with high carbon equivalents are extremely difficult to achieve. It is recommended that carbon equivalents be limited to 0.45 percent for No. 23 bars (No. 7 bars) and higher, and to 0.55 percent for smaller bars. ASTM A615, Grade 60 reinforcing steel will most likely not meet the aforementioned carbon equivalent requirements. However, reinforcing bars meeting ASTM Specification A706 have a low carbon equivalent, are easy to weld, and should be considered when welding is required. Mechanical connectors are another way of connecting reinforcing bars. Mechanical connectors are covered in Chapter 4.

8-9. SPECIAL INSPECTIONS.

- Continuous special inspection for all structural welding, except periodic special inspection is permitted for single pass fillet or resistance welds and welds loaded to less than 50 percent of their design strength provided the qualifications of the welder and the welding electrodes are inspected at the beginning of the work and all welds are inspected for compliance with the approved construction documents at the completion of welding.

- Continuous special inspection during the welding of reinforcing steel used in concrete or masonry construction.

CHAPTER 9

METAL ROOFING AND METAL (STEEL) DECK DIAPHRAGMS

9-1. INTRODUCTION. This Chapter prescribes the criteria and procedures for the design of metal roofing and steel deck diaphragms for buildings.

9-2. METAL ROOFING. Metal roofing consists of cold-formed, corrugated, fluted or ribbed metal sheets attached to the exterior of building structures and exposed to weather to serve as the exterior covering of the structure. Metal roofing may be either structural, or non-structural. The structural metal roof is designed so the roofing panels support all out of plane loads. Forces in the plane of the roof due to lateral wind and earthquake forces are typically resisted by steel deck diaphragms, or in plane X-bracing. The non-structural roof depends on substrate to carry the applied loads. Metal roofing may have lapped side seams and exposed fasteners, standing side seams and hidden metal clip fasteners, or hybrid types such as those with snap seams or battens that fall somewhere in between. There are many types of metal roofing produced by the metal manufacturing industry and care must be exercised to ensure that the type specified is compatible with the main structural system or substrate.

9-3. METAL DECK DIAPHRAGMS.

a. Metal Deck Diaphragms without Structural Concrete Topping. Metal deck diaphragms without structural concrete topping are usually used for roofs of buildings where the gravity loads are light (live loads are 1000 kg / meter^2 (20 psf or less). The metal deck units are often composed of steel sheets ranging in thickness from 0.75 mm (0.03 inches) to 2 mm (0.08 inches). The sheets are 600 mm (2 feet) to 900 mm (3 feet) wide, and formed in a repeating pattern with ridges and valleys. Rib depths vary from 40 mm to 100 mm (1-1/2 to 4 inches) in most cases. Decking units are attached to each other and to the structural steel supports by welds or mechanical fasteners. Chords and collector elements in these diaphragms are composed of steel frame elements attached to the diaphragm. Load transfer to frame elements that act as chords or collectors is thorough shear connectors, puddle welds, screws, or shot pins.

b. Metal Deck Diaphragms with Structural Concrete Topping. Metal deck diaphragms with structural concrete topping are frequently used on floor and roofs of buildings where the loads are moderate to heavy. The metal deck may be either a composite deck, which has indentations, or a non-composite deck. In both cases the slab and deck act together to resist diaphragm loads. The concrete fill may be normal weight or lightweight concrete, with reinforcing composed of wire mesh or small diameter reinforcing steel. Additional reinforcing may be added in areas of high stress. The metal deck units are composed of gage thickness steel sheets, 600 mm (2 feet) to 900 mm (3 feet) wide, and are formed in a repeating pattern with ridges and valleys. Decking units are attached to structural steel supports by welds or mechanical fasteners. The concrete topping has structural properties that significantly add to diaphragm stiffness and strength. The topping should be a minimum of 65 mm (2-1/2 inches) thick. Concrete reinforcing ranges from light mesh reinforcement to a regular grid of small reinforcing bars. Metal decking is typically composed of corrugated sheet steel from 22 gage down to 14 gage. Rib depths vary from 40 mm to 75 mm (1-1/2 inches to 3 inches) in most cases. Chord and collector elements in these diaphragms are considered to be composed of the steel frame elements attached to the diaphragm. Load transfer to frame elements that act as chords or collector elements is usually through welds or headed studs.

c. Metal Deck Diaphragms with Nonstructural Concrete Topping. Metal deck diaphragms with nonstructural concrete fill are typically used on roofs where gravity loads are light. The concrete fill, such as very lightweight insulating concrete (e.g. vermiculite), does not have useable structural properties. If the concrete is reinforced, reinforcing steel consists of wire mesh or small diameter reinforcing steel. To act as a diaphragm load transfer must be the same as that provided for a metal deck diaphragm without concrete topping.

d. Horizontal Steel Bracing (Steel Truss Diaphragms). Horizontal steel bracing (steel truss diaphragms) are often used in conjunction with structural metal roofing systems where the strength and stiffness of the metal roofing is incapable of carrying in-plane loads. Steel truss diaphragm elements are typically found in conjunction with vertical framing systems that are of structural steel framing. Steel trusses are more common in long span situations, such as special roof structures for arenas, exposition halls, auditoriums, industrial buildings, and aircraft hangars. For steel truss elements with large in-plane loads, diagonal elements may consist of angles, tubes, or wide flange shapes that can act in both tension and compression. Diagonals which can act in both tension and compression are preferred, however with lightweight metal buildings the diagonals are often steel rods which can act only in tension. Sufficient load path reliability should be provided for diagonals in accordance with the redundancy recommendations of Chapter 2. Truss element connections are generally concentric, to provide the maximum lateral stiffness and ensure that the truss members act under pure axial load.

9-4. BASIS FOR DESIGN.

a. Metal Roofing. The basis for the design of metal roofing is provided in TI 809-29, "Structural Considerations for Metal Roofing."

b. Metal (Steel) Deck Diaphragms. Steel deck diaphragms will be made from materials conforming to the requirements of the American Iron and Steel Institute (AISI), Specifications for the Design of Cold-formed Structural Members," or ANSI/ASCE 8, "Specifications for the Design of Cold-formed Stainless Structural Steel Members."

(1) In-plane Loads. Nominal in-plane shear strengths will be determined in accordance with approved analytical procedures. Design strengths will be determined by multiplying the nominal strength by a resistance factor, ϕ, equal to 0.60 for mechanically connected diaphragms, and equal to 0.50 for welded diaphragms. Analytical procedures contained in the Steel Deck Institute, Inc., "Diaphragm Design Manual #DDM01" are accepted means for calculating in-plane shear strengths. Limits are placed on diaphragm span and depth to span ratios to keep diaphragm in-plane displacements small enough to prevent cracking of walls. The maximum span and span to depth ratio depends on diaphragm stiffness and wall ductility. These and other additional requirements for diaphragms and their connections are provided in TI 809-04, "Seismic Design for Buildings."

(2) Out-of-Plane Loads. Design loads and design requirements for out-of-plane loads for bare metal deck roofing will be in accordance with TI 809-29, "Structural Considerations for

Metal Roofing." The design for out-of-plane loads for metal deck with structural concrete topping (composite steel floor deck) will be in accordance with the Steel Deck Institute, Inc., "Design Manual for Composite Decks, Form Decks, and Roof Decks."

9-5. METAL DECK DIAPHRAGMS - STIFFNESS FOR ANALYSIS (FEMA 273). Diaphragms can be considered to be flexible, or rigid. For flexible diaphragms, the lateral forces are distributed from the metal deck diaphragm to the vertical lateral force resisting elements by assuming the diaphragm acts as a simple beam spanning between vertical lateral-force resisting elements. For rigid diaphragms, the lateral forces are distributed to the vertical lateral-force resisting elements based on the relative stiffnesses of the vertical lateral-force resisting elements. Flexibility factors, provided in manufacturers' catalogs as well as in the Diaphragm Design Manual of the Steel Deck Institute, can be used to determine whether the diaphragm should be considered as flexible or rigid.   For bare metal deck diaphragms the stiffness is a function of metal thickness, rib geometry, fastener type, and fastener spacing. Procedures for calculating diaphragm flexibility are also provided in TI 809-04, "Seismic Design for Buildings."

9-6. SPECIAL INSPECTIONS.

- Periodic special inspection is required for all welding of all steel deck and steel truss elements of the seismic force resisting system.

- Periodic special inspections are required for screw attachment, bolting, anchoring, and other fastening components within the seismic force resisting system including diaphragms, drag struts, collector elements, and truss elements.

CHAPTER 10

METAL BUILDING SYSTEMS

10-1. INTRODUCTION. A metal building system is an engineered product furnished by metal building manufacturers. The metal building can be selected from a catalogue of standard designs or can be a custom design. Metal building systems, in general, consist of:

- rigid frames which act as the primary vertical load carrying system and as a lateral force system in the transverse direction,
- rod or angle x-bracing for the roof truss diaphragm and for vertical lateral bracing in the longitudinal direction,
- girts to support wall system cladding and resist wind loads,
- roof purlins to support a standing seam metal roof system and roof live , loads, and
- an exterior cladding system.

Many variations in framing systems are possible, and many different wall-cladding systems can be used including metal panels, brick veneer, masonry, and precast concrete wall panels. Additional information on metal building systems can be found in TI 809-30, "Metal Building Systems".

10-2. METAL BUILDING OPTIMIZATION. The metal building system will be optimized, based on ASCE 7 load and load combination requirements, to provide the lightest weight structural system possible. The designer must determine if additional requirements related to displacement, drift, durability, and redundancy should be written into the specifications to assure the final design will satisfy short and long term performance goals including function, durability, serviceability, and future expansion needs.. The major optimization occurs with respect to the rigid frames. These frames are constructed of relatively thin plates that are welded together. The frames use tapered webs with increased depth in the areas of high moments. Web thickness and flange size are varied as needed. The rigid frame members are designed with bolted end connections for easy field assembly. Most often the flanges are weld connected to the web on only one side to reduce fabrication costs. The secondary members including girts and purlins are cold-formed with a high strength to weight ratio.

10-3. BASIS FOR DESIGN.

a. Criteria. Specifications for metal building systems usually require that design be in accordance with the Metal Building Manufacturers Association (MBMA), "Low Rise Building Systems Manual," except that design loads and load combinations will be in accordance with TI 809-01, "Load Assumptions for Buildings." Secondary members (purlins, girts, etc.) are normally light gage cold-formed sections for which design is governed by TI 809-07, "Design of Load Bearing Cold-Formed Steel Systems and Masonry Veneer / Steel Stud walls".

b. Roofing. Roofing will comply with the requirements of TI 809-29, "Structural Considerations for Metal Roofing."

c. Certification. American Institute of Steel Construction, Inc. (AISC) certification is required of all metal building system manufacturers. However, this requirement may be waived for small storage type buildings with areas less than 140 square meters (1500 square feet).

10-4. STRENGTH AND SERVICEABILITY ISSUES.

a. General. Typically metal building systems have the minimum strength required, and use high strength materials to keep strength to weight ratios at a maximum. This approach to design under certain conditions can lead to serviceability problems.

b. Lateral Drift. Special attention should be paid to building drift under wind and seismic loading conditions. The maximum allowable drift will depend on the type of exterior cladding. For rigid claddings, such as precast concrete, brick, or block masonry, the maximum drift should be limited to h / 600. This is much lower than the h / 60 limit commonly accepted for buildings with flexible metal cladding.

c. Earthquake loadings. Metal building systems with heavyweight cladding must have suitable lateral force resisting systems to resist the inertial forces generated by the cladding during earthquake ground motions. The usual tension-only type bracing conventionally used in metal building construction will not be adequate when heavyweight cladding is used. If rigid cladding is used, the roof diaphragm must also have sufficient stiffness to limit out-of-plane wall displacements so that the cladding will not fail when the building is subjected to earthquake ground motions.

d. Mechanical Equipment Loads. The roof purlin system commonly provided with metal building systems would not have the capacity to support hanging mechanical HVAC units or rooftop units. Where the roof is required to support such units, special framing must be provided.

e. Roof In-Plane Load Resistance. Metal building systems are commonly constructed with a standing seam metal roof. Standing seam metal roof systems are incapable of resisting in-plane loads due to wind and earthquake forces. Therefore, a separate horizontal bracing system is required.

f. Serviceability Guidance. Metal building systems must meet the same serviceability requirements specified for steel framed buildings in Chapter 7, "Steel Structure Design Requirements". Additional guidance on serviceability can be found in the American Institute of Steel Construction, Inc. (AISC) Steel Design Guide Series 3, "Serviceability Design Considerations for Low Rise Buildings".

10-5. DESIGN RESPONSIBILITY. Performance specifications are generally used to obtain metal building systems. A professional engineer representing the owner will specify the metal building system including all framing elements, roofing system, exterior cladding, interior partition walls, and architectural finishes. The same engineer will specify which design codes, loads, and load combinations are to be considered in the design. Unusual loads such as unbalanced snow loads, and concentrated roof loads must be clearly defined. If future expansion is required, this must also be conveyed so end bays can be designed without intermediate columns. Architectural requirements such as "R" factors for insulation should also be included in the metal building specification. Metal building manufacturer's typically design the building and furnish plans and specifications for building construction. The building manufacturer's design must include all framing elements and their connections, and all exterior wall and cladding systems including openings, framing around openings, and connections. All bays where roof and wall bracing is to be installed should be identified. All interior walls including connections, or separations, from the building frame system should be detailed on the contract drawings. The foundation design will be provided with the contract documents in order to provide a basis for bid. The foundation design will be reviewed after the building manufacturer submits the final design. The final design submitted by the manufacturer should describe all loads, load combinations, and foundation reactions. With the use of gable bent type rigid frames, the foundation must be designed to resist lateral spreading forces. The lateral force resisting anchors are usually hairpin type reinforcing bars embedded in the slab-on-grade. When spreading forces are large, direct tension ties between exterior footings may be necessary. The foundation designer must carefully design and detail all foundation anchor systems to make sure they can transfer loads to the foundation, and make sure they do not intercept slab-on-grade control joints or otherwise interfere with other building features. The professional engineer representing the owner should track the building though the metal building system design review, shop drawing review, and construction process to assure the metal building system actually supplied and erected on the site meets all design requirements. It may be required that certain features of the project, such as foundations, cladding, and connections be redesigned during the metal building system design review phase.

10-6. SPECIAL INSPECTIONS.

- Periodic special inspections will be provided during all welding of elements of the lateral force resisting system.

- Periodic special inspections will be provided for bolting, anchoring, and other fastening components of the lateral force resisting systems including struts, braces, and hold-downs.

- Periodic special inspections will be provided during the erection and fastening of exterior cladding to the metal building framing system.

CHAPTER 11

WOOD STRUCTURE DESIGN REQUIREMENTS

11-1. INTRODUCTION. This chapter provides a list of guidance documents to be used for the design of wood buildings. Properties of wood and other considerations influencing design including design of plywood elements and built-up members, wood preservation, termite control, fire retardant treatment, and climatic influences are included either in this chapter or in the referenced guidance documents in Appendix A. Guidance documents referenced include design standards and specifications. The use of timber construction will consider the type of occupancy and meet all fire protection criteria and requirements. Detailed design information on wood buildings is not provided because wood construction is generally limited to residential construction since strict fire protection standards preclude the use of wood construction for most other types of military buildings.

11-2. BASIS FOR DESIGN. The design of structural elements or systems constructed partially or wholly of wood or wood-based products will be by allowable stress design or load and resistance factor design. The structural analysis and construction of wood elements and structures using allowable stress design methods will be in accordance with the applicable standards indicated by reference in Appendix A. The structural analysis and construction of wood elements and structures using load and resistance factor design methods will be in accordance with AF&PA/ASCE 16, " Standard for Load and Resistance Factor Design (LRFD) for Engineered Wood Construction." The design and construction of wood structures to resist seismic forces and the material used therein will comply with the requirements of TI 809-04, "Seismic Design for Buildings," and FEMA 302, "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures."

11-3. SERVICEABILITY CONSIDERATIONS.

a. Climatic Considerations. Engineering properties usually are not appreciably affected when wood is subjected to extremely low temperatures. For cold region limitations on wood construction, see TM 5-852-9/AFR 88-19, Volume IX. The engineering properties of wood are not appreciably affected in tropical climates. Rot and insect attacks, however, are aggravated in tropical humid areas, and all timber for permanent construction in tropical areas should be preservative treated. Bonding of wood to wood can be made by a variety of adhesives. In tropical climates, structural bonding of wood to other materials should be by means of epoxy resin adhesive.

b. Fire Retardant Treatment. Recommendations regarding the use of fire retardant treatments are provided in the USDA Wood Handbook and the National Fire Protection Handbook. Pressure impregnation is the preferred treatment method.

c. Termite Control. Termite control measures will be used in areas prone to termite infestation. Soil will be treated with commonly accepted termite control products prior to construction.

d. Oriented Strand Board. The use of oriented strand board (OSB) for non-vertical applications <u>is not permitted</u>. For floor and roof sheathing, APA structural rated plywood sheathing only will be used. Specifically, for floors, use as a minimum, 18mm (23/32 inch) thickness APA rated STURD-I-FLOOR, 600 mm (24inches) on center span rating, Exposure 1, Tongue and Grove, glued and nailed. In addition, all of the requirements of the APA "Code Plus Floor" will be met. Ring or screw-shank nails will be used.

11-4. SPECIAL INSPECTIONS.

- Continuous special inspection during all field gluing of elements of the lateral force resisting system is required.

- Periodic special inspections for nailing, bolting, anchoring, and other fastening of components within the lateral forces resisting system including drag struts, braces, and tie-downs is required.

CHAPTER 12

BUILDINGS SUBJECT TO SEVERE ENVIRIONMENTS AND EXPLOSIVE EFFECTS

12-1. INTRODUCTION. In some cases, buildings may be constructed in severe corrosive environments, constructed in environments with extreme temperatures, or constructed such that protection of the environment due to toxic chemicals, wastewater, or explosives must be provided. This chapter provides information so buildings constructed in severe environments can meet their intended performance objectives without deterioration, and without degradation that could increase maintenance costs or shorten service life.

12-2. STEEL STRUCTURES IN CORROSIVE ENVIRONMEMTS.

a. General. When steel members will be exposed to heavy industrial pollution, salt spray, salt air, chemicals, or are to be embedded in corrosive soils, a corrosion engineer will be consulted to recommend materials, protection, or both and to review design drawings to assure the structure will be serviceable and durable. When appropriate, an increased thickness, i.e., corrosion allowance, will be used to attain the required service life. For additional discussion and guidance in designing for corrosive conditions, see the National Association of Corrosion Engineers (NACE) Corrosion Handbook and other relevant NACE publications. Additional information can also be found in the NAVFAC Maintenance and Operations Manual MO-307, “Corrosion Control” and in NAVFAC Interim Technical Guidance (ITG), “Cathodic Protection Systems”, dated May 1994.

b. Corrosion Protection of Steel Members. Corrosion protection is less of a problem in structures which have been designed to provide good drainage, designed to provide air circulation, and designed to provide access for maintenance. Both oxygen and moisture must be present if corrosion is to occur. The most severe corrosion occurs in marine environments where chloride ions are present. Paint systems and zinc coatings applied by the hot dip process or by spray on process are all good corrosion protection systems when metal surfaces are prepared properly and the coating systems are applied properly. In extremely corrosive conditions, or where maintenance is difficult, the use of cathodic protection or stainless steel may be warranted.

c. Design Considerations. Steel structures designed for corrosive environments should consider the following corrosion protection measures.

(1) Box-shaped members should be designed so that all inside surfaces may be readily inspected, cleaned, and painted, or should be closed entirely to prevent exposure to moisture.

(2) The flanges of two (back to back)angle members, when not in contact, should have a minimum separation of 3/8 inch to permit air circulation.

(3) Pockets or depressions in horizontal members should have drain holes to prevent water from ponding in low areas. Positive drainage should be provided away from exposed

steel. Column bases should be terminated on concrete curbs or piers above grade, and tops of curbs or piers will be pitched to drain.

(4) Where extremely corrosive conditions exist, consideration should be given to providing cathodic protection in addition to protective coatings for steel members exposed to salt water moisture environments.

(5) Structural members embedded in concrete and exterior railing, handrails, fences, guardrails, and anchor bolts will be galvanized or constructed of stainless steel.

(6) Dissimilar metals, (e.g., aluminum and steel, stainless steel and carbon steel, zinc coated steel and uncoated steel) should be isolated by appropriate means to avoid the creation of galvanic cells which can occur when dissimilar metals come in contact.

12-3. STEEL STRUCTURES EXPOSED TO EXTREME CLIMATIC CONDITIONS.

a. Arctic and Antarctic Zones. For carbon steel, the transition from ductile to brittle behavior occurs within temperatures to be expected in Arctic and Antarctic zones. Ductility is important for structures in high seismic areas. Toughness, a characteristic also affected by cold temperatures, is important for structures which could be subjected to cyclic or impact loadings. Information in ASTM A 709, "Structural Steel for Bridges", although related to bridge structures, does contain information on fatigue and toughness useful in the design of buildings. Structures in cold climates, which could be subjected to cyclic or impact loads, should considered the following measures to mitigate potential fatigue and fracture problems.

(1) Provide ample fillets to avoid stress risers.

(2) Use bolted joints whenever possible. If welded joints are used, take precautions to eliminate gas and impurities in welds. Proper preheating and post cooling are essential.

(3) Use low-carbon steels and nickel-alloy steel that have good toughness characteristics at low temperatures.

b. Tropical Zones. The effect high ambient temperatures have on steel properties, with respect to buildings constructed in tropic zones, is minimal.

12-4. STEEL STRUCTURES SUBJECTED TO ELEVATED TEMPERATURES.

a. Hot-rolled Carbon Steel. Up to 65 degrees C (150 degrees F), strength of steel will be assumed to be same as the strength at normal temperature. Above 65 degrees C (150 degrees F), the yield strength decreases with increasing temperature.

b. High-strength and Heat-treated Steels. The effect of elevated temperatures on high strength and heat-treated steels should be thoroughly investigated. For example, quenched and tempered materials will undergo radical changes in their mechanical properties as well as toughness when subjected to temperatures above 260 degrees C (500 degrees F).

12-5. CONCRETE STRUCTURES IN CORROSIVE ENVIRONMENTS.

a. General. Corrosion of reinforcing steel and other steel in concrete causes not only damage to the reinforcing steel, but also damage to the concrete. Rust that forms during the corrosion process has a volume several times that of the parent material. This generates expansive forces that causes spalling and delamination of the concrete. Chloride ions, most often attributed to a salt water, or salt spray, are the major cause of reinforcing steel corrosion. As with structural steel the contact between dissimilar metals, and also between exposed steel and embedded reinforcing steel, can lead to corrosion that can damage both the steel and concrete. The high alkaline environment of concrete protects reinforcing steel, prestressing steel, and other embedded steel from corrosion provided the concrete is durable, has low permeability, and provided there is adequate concrete cover. Additional information of the corrosion of concrete reinforcing steel can be found in ACI Committee 222 Report, "Corrosion of Metals in Concrete".

b. Concrete Cover. Concrete in a marine environment and subject to salt-water wave action and spray should have concrete cover protection greater than that specified in ACI 318. For concrete walls that at 500 mm (20 inches) or greater in thickness the minimum concrete cover will be 65 mm (2.5 inches). If possible the cover should also be 65 mm (2.5 inches) in concrete walls that are less than 500 mm (20 inches). For concrete that is not placed within forms, or for concrete in contact with earth, the cover requirements will be increased 15 mm (1/2 inches). However in thin members where clearances are restricted the cover can be limited to the maximum of either:

(1) 1.5 times the maximum size of aggregate, or

(2) 1.5 times the maximum diameter of reinforcement, or

(3) 20 mm (3/4 inch) cover to all steel including stirrups.

Additional information on reinforcing steel and prestressing steel detailing for structures in marine environments, and for the design of reinforced concrete structures in marine environments, can be found in ACI Committee 357 Report, "Guide for the Design and Construction of Fixed Offshore Concrete Structures".

12-6. CONCRETE STRUCTURES PROTECTING THE ENVIRONMENT.

a. General. Concrete building features that must be designed to protect the environment from chemical spills, or to protect water supplies from contamination, will be designed in accordance with the provisions of the ACI Committee 350 Report, "Environmental Engineering Concrete Structures". These features include tanks, reservoirs, sewers, wet wells, pump stations, and other similar structures and appurtenances. The main purpose of ACI Committee 350 Report is to minimize cracking to avoid leakage of chemicals and wastewater.

b. Design Approach. In accordance with the ACI Committee 350 Report the design strength required by the ACI 318 load factor equations is to be multiplied by a durability coefficient equal to 1.3 to obtain the required design strength for environmentally engineered concrete structures. The purpose of the durability coefficient is to reduce reinforcing steel stresses (and cracking potential) at service load conditions. Small diameter bars at close spacing are encouraged in order to limit crack widths. Cover requirements greater than those of ACI 318 are required to provide increased protection against reinforcing steel corrosion. Information on joints, joint details, and waterstops are also covered in the ACI Committee 350 Report.

12-7. CONCRETE BLAST RESISTANT STRUCTURES. Design of structures to resist the effects of accidental explosions will be in accordance with TM 5-1300/AFM 88-22/P-397, "Structures to Resist the Effects of Accidental Explosions" and Navy Design Manual (DM) 2.08, "Blast Resistant Structures." The design of blast-resistant structures must consider the transient loadings and dynamic response of the structure that result from the specified design event. Blast-resistant design is often required in conjunction with the construction of weapons system facilities, both developmental and operational, as well as for hardened structures designed to resist the effects of intentional attack.

CHAPTER 13

SPECIAL CONSTRUCTION

13-1. INTRODUCTION. This chapter covers the design requirements for aluminum structures, membrane structures, and other types of structures constructed using unique materials and unique construction methods not covered by the previous chapters. All special construction must comply to the requirements of the National Fire Protection Association.

13-2. NEW MATERIALS AND METHODS. The use of special construction is permitted whenever it appears necessary, advantageous, and economical. However, specifying new or untried materials or methods of construction should be avoided until the merits of the methods or materials have been established. New, unusual, or innovative materials, systems or methods previously untried may be incorporated into designs when evidence shows that such use is in the best interest of the Government from the standpoint of economy, lower-life-cycle costs, and quality of construction. Manufacturers should prove the merits of their products by certified laboratory results, by evidence of satisfactory installation under conditions similar to those anticipated for the proposed construction, and by demonstrating compliance with appropriate industry standards.

13-3. SPECIAL CONSTRUCTION STANDARDS. Special construction often involves design requirements and construction methods different from those covered by current design manuals, construction standards, and guide specifications. In cases where special constructions has application beyond the scope of a particular project, the agency responsible for design will submit a recommended change report to the agency/or agencies responsible for maintaining and updating design guidance, construction standards and guide specifications. The recommended change report will allow current design guidance, construction standards, and/or guide specifications, to be updated to cover the new special construction method. The recommended change report should contain justification and documentation supporting the new special construction method, including cost benefits, proposed special criteria and controls, performance history, and tests.

13-4. ALUMINUM STRUCTURES. Aluminum when used for structural purposes in building construction will be designed in accordance with the Aluminum Association AA-94, Part 1-A, "Specifications for Aluminum Structures - Allowable Stress Design", and Part 1-B, "Specifications for Aluminum Structures - Load and Resistance Factor Design of Buildings and Similar Type Structures". Approval under the provisions of Paragraphs 2 and 3 is not required unless the aluminum structure is part of a unique structural system.

13-5. MEMBRANE STRUCTURES. Membrane structures include membrane-covered frame structures, cable supported membrane structures, and air supported structures. Membrane structures are those which utilize an enclosure membrane acting in tension as a structural element. Membrane structures must either be noncombustible, or flame resistant. Structures will be designed to sustain dead loads, loads due to tension or inflation, and live loads including wind, snow and seismic loads. The use of membrane structures is particularly applicable to temporary construction, to situations requiring minimum structure weight, or to conditions requiring large, column-free spaces. Membrane structures are also used to cover water storage facilities, water clarifiers, water treatment plants, greenhouses, and other facilities not used for human occupancy. For additional guidance on the design of fabric structures, see Tension Structures Behavior and Analysis by Leonard and refer to literature available from the major manufacturers of structural fabrics and air-supported structures. Use of fabric structures is subject to prior approval as set forth above except when they are used as temporary enclosures.

13-6. GLASS FIBER REINFORCED CONCRETE, FIBER COMPOSITES AND REINFORCED PLASTICS.

a. General. Although the use of glass fiber reinforced concrete (GRFC) is used in many instances for building cladding, the use of other fiber composites and reinforced plastics has been very limited with respect to building systems and components. These materials do offer unique advantages because of their high strength-to-weight ratio, because of the ease with which they can be molded into various shapes, and because of the excellent resistance they provide against corrosion. Other properties such as creep, modulus of elasticity, coefficient of thermal expansion, and long term resistance to weathering and other environmental effects, however, may result in overall performance that does not measure up to the standards associated with the commonly used materials described in the previous chapters of this report.

b. Glass Fiber Reinforced Concrete (GRFC). Glass Fiber Reinforced Concrete (GRFC) consists of cement aggregate slurry reinforced throughout with alkali resistant glass fibers. In building construction, GRFC is primarily used for architectural precast cladding. The cladding has an appearance similar to precast concrete, except it is much lighter which can help to reduce the cost of the building structural framing. GRFC is not used as a vertical load-bearing component, although it can resist wind loads, and seismic inertial forces due to its own weight. In addition to being lightweight, GRFC panels have high impact resistance and are non-combustible. Information on use, design and construction of GFRC can be found in the Prestressed Concrete Institute (PCI) Committee Report, "Recommended Practice for Glass Fiber Reinforced Concrete Panels." When used, GFRC should be evaluated for its history of performance under the types of climatic conditions to which the building will be exposed. Climates with temperature extremes and with high rainfall and moisture conditions have in some cases have caused deterioration of GRFC thereby effecting cladding serviceability.

c. Reinforced Plastics and Fiber Composites. Construction using fiber reinforced plastic and fiber reinforced plastic composites is relatively new with respect to structural applications. Glass fiber reinforced plastic (GFRP) bars can be used in concrete beams and columns in order to prevent damage from the corrosive effects of salts, acids, and other aggressive elements. The ultimate strength of GFRP bars is about twice that of reinforcing steel, however the modulus of elasticity is only about 1/4 that of steel that could possibly cause serviceability problems related to increased deflection. Glass fiber reinforced plastic (GFRP) plates and fabric can be epoxy bonded to concrete or masonry to increase strength and ductility. This shows great promise as a means for increasing the structural performance of existing structural systems and components, especially where structural components lack the strength and ductility to survive major earthquake ground motion demands. Information on reinforced plastics and fiber composites can be found in the ACI Journal Technical, Paper, Title No. 91-S34, "Fiber Composites for New and Existing Structures," May-June 1994.

APPENDIX A

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TI 809-02	Structural Design Criteria for Buildings
TI 809-03	Structural Design Criteria for Structures Other than Buildings
TI 809-04	Seismic Design for Buildings
TI 809-05	Seismic Design for the Rehabilitation of Buildings
TI 809-06	Masonry Structural Design for Buildings
TI 809-07	Design of Load Bearing Cold-Formed Steel Systems and Masonry Veneer / Steel Stud Walls
TI 809-26	Welding Guidance for Buildings
TI 809-27	Concrete Floor Slabs on Grade Subjected to Heavy Loads
TI 809-28	Design and Construction of Reinforced Ribbed Mat Slabs
TI 809-29	Structural Considerations for Metal Roofing
TI 809-30	Metal Building Systems
TI 809-51	Seismic Screening Procedures for Military Buildings
TI 809-52	Commentary on Snow Loads
TI 809-53	Commentary on Roofing systems

Department of the Army - Engineering Manuals

Reference Designation	Title
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EM 1110-1-1905	Bearing Capacity of Soils
EM 1110-1 1908	Rock Foundations
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Department of the Army - Technical Manuals

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TM 809-3/AFM 88-3, Ch. 3	Masonry Structural Design for Buildings
TM 809-10/ P-355/AFM 88-3, Ch. 13	Seismic Design for Buildings
TM 809-10-1/ P-355.1/AFM 88-3, Ch. 13, Sec A	Seismic Design Guidelines for Essential Buildings
TM 809-10-2/ P-355.2/AFM 88-3, Ch. 13, Sec B	Seismic Design Guidelines for Upgrading Existing Buildings
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TM 5-809-11	Design Criteria for Facilities in Areas Subject to Typhoons and Hurricanes.
TM 5-852-9/ AFR 88-19, Vol. IX	Arctic and Subarctic Construction: Buildings.
TM 5-1300/AFM 88-2	Structures to Resist the Effects of Accidental Explosions.

Department of the Navy

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Navy Manual P-89 (To become a Tri-Service Manual)	DOD Weather Manual
Navy Manual P-307	Management of Weight Handling Equipment
Navy Manual P-397	Structures to Resist the Effects of Accidental Explosions
Navy Design Manual DM 2.04	Structural Engineering, Concrete Structures
Navy Design Manual DM 2.08	Blast Resistant Structures
Navy Design Manual DM 7.1	Field Exploration, Testing, and Instrumentation
Navy Design Manual DM 7.02	Foundations and Earth Structures
Navy Military Handbook MIL-HDBK 1002/1	Structural Engineering – General Requirements
Navy Military Handbook MIL-HDBK 1002/3	Steel Structures
Navy Military Handbook MIL-HDBK 1002/5	Timber Structures
Navy Military Handbook MIL-HDBK 1002/6	Aluminum Structures, Composite Steel, Other Structural Materials
Navy Military Handbook MIL-HDBK 1007/3	Soil Dynamics and Special Design Aspects
Navy Handbook MIL-HDBK 1038	Weight Handling Equipment
NAVFAC Maintenance and Operations Manual M-307	Corrosion Control
NAVFAC Interim Technical Guidance (ITG)	Cathodic Protection Systems, May 1994
NAVFAC Interim Technical Guidance (ITG)	Minimum Design Loads for Buildings and Other Structures, 31 August 1998

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FEMA 178	NEHRP Handbook for the Seismic Evaluation of Existing Buildings.
FEMA 267	Interim Guidelines, Welding of Steel Moment Frame Structures
FEMA 273	NEHRP Guidelines for the Seismic Rehabilitation of Buildings
FEMA 302	NEHRP Recommended Provisions for Seismic Regulations for New Buildings and other Structures

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Aluminum Association (AA)
900-19th Street NW, Suite 300
Washington DC 20006

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AA-94, Part 1-B	Specifications for Aluminum Structures - Load and Resistance Factor Design of Buildings and Similar Type Structures".

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PO Box 9094
Farmington Hills, MI 48333-9094

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ACI Committee 224.3 Report	Joints in Concrete Construction
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ACI Committee 315R Report	Manual of Engineering and Placing Drawings for Reinforced Structures.
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ACI Committee 357 Report	Guide for the Design and Construction of Fixed Offshore Concrete Structures.
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1111 19th Street NW, Suite 700
Washington DC 20036

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One East Wacker Drive, Suite 3100
Chicago, IL 60601-2001

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7012 S. Revere Parkway, Suite 140
Englewood, CO 80112

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AITC 112	Standard for Tongue and Groove Heavy Timber Roof Decking.
AITC 113	Standard for Dimensions of Structural Glued - Laminated Timber.
AITC 117	Structural Glued-Laminated Timber.
AITC 119	Standard Specifications for Hardwood Glued Laminated Timber.
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AITC 200	Inspection Manual
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1101 - 17th Street NW, Suite 1300
Washington DC 20036-4700

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AISI	Design of Cold Formed Steel Structural Members

APA-Engineered Wood Association (APA)
PO Box 11700
Tacoma, WA 98411-0700

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APA	Supplement 3- Design & Fabrication of Plywood - Stressed-Skin Panels
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1801 Alexander Bell Drive
Reston, VA 20191-4400

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550 NW LeJeune Road
Miami, FL 33126

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8720 Red Oak Boulevard, Suite 201
Charlotte, NC 28217

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1300 Sumner Avenue
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5420 Old Orchard Road
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175 W. Jackson Boulevard, Suite 1859
Chicago, IL 60604-9773

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1717 W. Northern Avenue
Phoenix, AZ 85021

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PO Box 9506
Canton, OH 44711

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1205 -48th Avenue North, Suite A
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583 D'Onofrio Drive, Suite 200
Madison, WI 53719

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