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UNIFIED FACILITIES CRITERIA (UFC)

O&M MANUAL: STANDARD PRACTICE FOR AIRFIELD PAVEMENT EVALUATION

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O&M MANUAL: STANDARD PRACTICE FOR AIRFIELD PAVEMENT EVALUATION

Indicate the Military Department Preparing Activity responsible for the document.

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

NAVAL FACILITIES ENGINEERING SYSTEMS COMMAND

AIR FORCE CIVIL ENGINEER CENTER

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

Change No.	**Date**	**Location**

This UFC supersedes UFC 3-260-03, *Airfield Pavement Evaluation*, dated 15 April 2001.

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FOREWORD

The Unified Facilities Criteria (UFC) system is prescribed by MIL-STD 3007 and provides planning, design, construction, sustainment, restoration, and modernization criteria, and applies to the Military Departments, the Defense Agencies, and the DoD Field Activities in accordance with USD (AT&L) Memorandum dated 29 May 2002. UFC will be used for all DoD projects and work for other customers where appropriate. All construction outside of the United States, its territories, and possessions 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 most stringent of the UFC, the SOFA, the HNFA, and the BIA, as applicable.

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

- Whole Building Design Guide website https://www.wbdg.org/ffc/dod.

Refer to UFC 1-200-01, *DoD Building Code*, for implementation of new issuances on projects.

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CHAPTER 1 INTRODUCTION

1-1 REISSUES AND CANCELS.

This UFC reissues and cancels UFC 3-260-03, *Airfield Pavement Evaluation*, dated 15 April 2001.

1-2 PURPOSE AND SCOPE.

This document provides guidance in the structural evaluation of existing pavements. It incorporates recent and applied research that has resulted in improved reliability in evaluation results obtained with DoD engineering tools that address the purpose and scope of this UFC for all DoD Services.

UFC 3-260-03 presents criteria for evaluating the load-carrying capability of airfield pavements in terms of allowable traffic that a pavement can sustain for given loading conditions or the allowable load for a specified traffic mix, without producing unexpected or uncontrolled distress. It is not for use in contractor quality assurance or quality control (QA/QC). This document outlines procedures for nondestructive testing (NDT) and direct testing to gather data for use in conventional and layered elastic pavement analysis. The Pavement-Transportation Computer Assisted Structural Engineering (PCASE) application implements the pavement evaluation criteria in this document.

1-3 APPLICABILITY.

This document applies to evaluations of DoD airfields and heliports or those used by DoD aircraft or missions.

1-4 NATO AIRFIELDS AND OPERATIONS.

Comply with NATO STANAG 7131, *Aircraft Classification Number (ACN)/Pavement Classification Number (PCN)*, and NATO STANDARD AEP-46, *ACN/PCN*, when evaluating airfields used by NATO forces or NATO campaigns. Comply with TSPWG M 3-260-00.NS7210*, Standards for NATO Deployed Air Operations*.

1-5 PAVEMENT TYPES.

The pavement types considered in this UFC are the following.

1-5.1 Flexible Pavement.

A pavement with an asphalt concrete (AC) surface course and one or more supporting base or subbase courses, placed over a prepared subgrade.

1-5.2 Plain Concrete Pavement.

A single thickness of non-reinforced portland cement concrete (PCC) resting directly on a prepared subgrade, granular base course, or stabilized layer.

1-5.3 Rigid Overlay on Rigid Pavement.

A rigid overlay pavement placed on an existing rigid pavement. Placing a rigid overlay can include or exclude a bond-breaking course between the existing rigid pavement and the overlay. If the thickness of the bond-breaking course between the two rigid pavements is 4 inches (102 millimeters) or more, evaluate the entire pavement as a composite pavement (see paragraph 1-5.6 and paragraph 7-8).

1-5.4 Non-rigid Overlay on Rigid Pavement.

An AC surface layer or combination of AC layer and granular base course placed on an existing rigid pavement.

1-5.5 Rigid Overlay on Non-rigid Pavement.

A rigid overlay pavement placed on an existing non-rigid pavement.

1-5.6 Composite Pavement.

A composite pavement consists of a rigid overlay placed on an existing pavement that already has an existing flexible overlay on a rigid base slab. The existing flexible overlay may be asphalt for its full depth or a combination of asphalt and granular base course over the rigid base slab. When the thickness of the flexible overlay is less than 4 inches (102 millimeters), consider the entire pavement as an unbonded rigid overlay on rigid pavement. The asphalt overlay material is considered a bond-breaking course.

1-5.7 Reinforced Concrete Pavement.

A concrete pavement reinforced with deformed steel bar or welded-wire fabric. Measure the diameter and spacing of the steel in both the longitudinal and transverse directions to determine the percent steel.

1-5.8 Fiber Reinforced Concrete.

A concrete pavement reinforced with fibers. Previous evaluation manuals contained curves for evaluating concrete pavements with steel fibers. These curves are no longer used because there are no airfield pavements in DoD with steel fibers due to the fibers causing surface problems. Do not use steel fibers unless approved by the Pavements Discipline Working Group (DWG) or its designated representative. If using other types of fibers in pavements, do not reduce the pavement thickness requirement.

1-6 GLOSSARY.

Appendix F contains acronyms, abbreviations, and terms.

1-7 REFERENCES.

Appendix G contains a list of references used in this document. The publication date of the code or standard is not included in this document. Unless otherwise specified, the most recent edition of the referenced publication applies.

CHAPTER 2 EVALUATION CONCEPTS AND PROCESS

2-1 RELATIONSHIP OF DESIGN TO EVALUATION.

Pavement design requires selecting materials with the necessary strength and placing them at the proper thickness, density, and depth to construct a pavement capable of carrying the anticipated number of passes of a given load. Due to variations in material and placement conditions, the strengths and thicknesses of the as-constructed pavement may differ from the design. Over time, the strength of layers in the pavement structure will change. An evaluation determines the physical properties of a pavement as constructed and in its current condition to verify its aircraft load-supporting capability.

2-2 CONCEPTS.

The primary function of a pavement is to distribute the wheel loads over a larger area than the wheel contact area. Each airfield has its own natural soil and environmental conditions, and the in situ soils must ultimately sustain the stresses resulting from loads applied to the pavement. Since the strengths of native soils can vary widely from site to site, the ability to support loads also varies widely. In most cases, aircraft tire loads cannot be sustained directly on the native soils.

2-2.1 Pavement Structure.

A pavement design limits the tensile strain in an AC surface layer and tensile stress in a PCC surface layer to prevent excessive shear deformation (e.g., vertical strain) in the underlying unbound layers, including the subgrade. Flexible and rigid pavement structures are designed to limit tensile strains and stress for a defined mix of aircraft at specified loads and passes. Based on the magnitude of the applied surface load, contact pressure, and gear configuration, a pavement structure must distribute surface loads to that which the subgrade soil can accept for the aircraft mix. The evaluation process looks at load capability of an existing pavement in two ways. First, given a specified aircraft mix at a specified load, determine the allowable passes. Second, given a specified aircraft mix at a required number of passes, determine the allowable load.

Flexible pavements distribute load by broadening the effective area supporting the load, from the tire contact area on the surface to a wider area on the base, to a still wider area on the subbase, and so on. Each layer must be of sufficient quality to sustain the load intensity or stress and each must be thick enough to broaden or distribute the load and reduce intensity to that which its supporting layer can sustain without excessive permanent deformation. Rigid pavements are stiffer and have a “beam action” or flexural capability that spreads or distributes load more widely but must still have sufficient support to distribute the load and reduce flexural and tensile stresses in the slab.

2-2.2 Performance Models.

Performance models act as a “transfer function” between pavement response models and actual pavement performance. DoD uses several different pavement evaluation models, all of which are mechanistic-empirical models that associate an empirically derived pavement failure indicator (e.g., vertical stress) that defines the response with the required performance (e.g., coverages to failure). These performance models include the following:

- CBR-Alpha-Beta Hybrid model for flexible pavements that uses the California Bearing Ratio (CBR) as a strength index for base, subbase, and subgrade layers
- Westergaard Medium-Thick Plate Solution for rigid pavements that uses the modulus of subgrade reaction (k) as a strength index for layers supporting the slab
- CBR-Alpha model for unsurfaced and mat pavements that uses the CBR as strength index for all supporting layers
- Layered Elastic model for both rigid and flexible pavements that uses a material's Modulus of Elasticity (E) and Poisson's Ratio (*v*) values to characterize each layer

2-3 PAVEMENT EVALUATION PROCESS.

Pavement evaluation requires a structured approach to gather and organize information, perform testing and analysis, and generate report products for a variety of stakeholders to use in decision making. In addition to the structural evaluation process and procedures defined in this UFC, UFC 3-260-16, *O&M Manual: Standard Practice for Airfield Pavement Condition Surveys*, outlines the guidance for pavement condition index (PCI) inspections and UFC 3-270-08, *Pavement Maintenance Management*, provides guidance on using PCI and structural evaluation results in the overall pavement management process. The processes and procedures in all three of these documents are interrelated, follow the same general steps, and use the same inventory organization.

2-3.1 Evaluation Planning.

Gather and review information regarding the site and the pavement at the site from the sources outlined below. These data are used to determine the scope and validity of available data and develop a test plan. While this step in the process begins prior to any field work, it typically continues through the other phases of the evaluation as you contact people at the installation and get access to additional information.

2-3.1.1 Previous Evaluation Reports, Design, and Construction Documents.

Begin the planning process by gathering any previous evaluation reports. They typically have much of the background data needed for planning and conducting the evaluation. In addition to physical property and surface condition data, they contain site, construction history (also known as work history), and previous traffic information. Design and construction documents are another good source of information, including, but not limited to, the following:

- Pavement, base, and subbase layer thicknesses

- Asphalt physical properties such as mix design aggregate gradation and testing, binder properties, and asphalt mix properties such as density and voids
- PCC physical properties such as mix design, aggregate gradation, and slump
- Base and subbase strength and material properties
- Rigid pavement flexural strength
- Rigid pavement joint layout and load transfer devices or thickened edges

These data are particularly useful for forensic analysis when testing uncovers issues with existing pavements. This type of information is also available in the sources described below when no previous evaluations exist and is also used to validate and supplement information in previous reports.

2-3.1.2 Geographic Location and Mapping.

Determine the geographic location of the airfield and obtain mapping data. Geospatially correct mapping is normally furnished by the installation when performing an evaluation at a DoD installation or forward operating location with a current DoD mission. Obtain imagery and mapping from other sources such as the National Geospatial-Intelligence Agency (NGA) when not available from the installation or operating location.

2-3.1.3 Geological Data.

Identifying the general geology in the vicinity of the airfield is critical to determine the general type of soil deposition (e.g., alluvial, residual), the parent rock from which the soil derives, and other pertinent information. Soil type data is available in U.S. Geological Survey publications or Department of Agriculture soil maps as well as from state geological departments, state highway departments, subsurface exploration companies, and similar organizations, including NAVFAC and USACE construction offices. Soil boring or well logs from the installation and aerial photographs showing pertinent geologic features are also valuable data sources.

2-3.1.4 Drainage and Groundwater Conditions.

Identify the natural drainage pattern and general surface-drainage system for the area from contour maps published by the U.S. Geological Survey, the National Oceanic and Atmospheric Administration, or the NGA. Collect detailed information concerning drainage at the airfield, including descriptions of any drainage structures and shoulder slopes, and whether excessive vegetation or soil along the pavement edges ponds water on the pavements. Determine the depth to groundwater table near the airfield and at the airfield perimeter and note the presence of any perched water tables in the airfield subgrade. Obtain groundwater data and the location of springs and seeps from well logs, cuts, or borings in the vicinity. Also, identify and evaluate subsurface drainage systems.

2-3.1.5 Climatic Data.

The Pavement-Transportation Computer Assisted Structural Engineering (PCASE) application has a world index database with the average daily maximum and minimum temperatures for each month, average annual rainfall, and the freezing index. This, as well as other information such as the average humidity and description of the prevailing winds for the period of record, can be found in routine National Weather Service publications, from records of the airfield weather station, or from the U.S. Air Force 14th Weather Squadron (formerly Combat Climatology Center [AFCCC]) Asheville, NC.

2-3.1.6 Construction/Work History.

Having an accurate construction history is essential to analyze the pavement surface condition deterioration and is used for structural analysis if field testing cannot be conducted. Information on other work performed, such as dates for overlays, surface treatments, joint seals, patches, and other repairs, enhances analysis capability. Obtain detailed information on the construction and maintenance performed on each facility from the installation engineer organization responsible for base maintenance. The construction office responsible for construction on the installation (e.g., NAVFAC or USACE) may also be able to provide this information.

2-3.1.7 Traffic Data.

Collect data from airfield management on the type, gross weight, and typical operating weight of each type of aircraft regularly using the airfield on a day-to-day basis. Specific traffic data (type, weight, passes) for all fixed or rotary wing aircraft using each runway, taxiway, and apron system will enhance the evaluation accuracy if available. These data will be used to define future expected traffic loading and pass levels. Specific traffic analysis procedures are discussed in detail in Chapter 4.

2-3.2 Mapping and Inventory.

Having a geospatially correct map linked to areas of pavement with similar characteristics provides organization for pavement testing, analysis, and reporting. Mapping and inventory standards and procedures are described in detail in UFC 3-270-08. The following is a process summary.

2-3.2.1 Pavement Inventory.

Pavement inventory is the term used to describe all the airfield pavement on an installation. The pavement is divided into a hierarchy consisting of a network, branches, and sections. A site typically has one airfield network but can have more than one in some situations. Branches are divided based on pavement use (e.g., runways, taxiways, and aprons) and sections are areas of pavement with similar physical characteristics. Each of these entities has an ID and the combination of the network, branch, and section IDs is the pavement ID (PID). The PID is associated with the pavement evaluation data in the database and its respective polygon on the map. See UFC 3-270-08 for more detail.

2-3.2.2 Creating and Updating Maps.

Use a Geographic Information System (GIS) application such as ArcMap or AutoCAD 3D Map to create or update mapping. There is also the option to use the GIS application to update inventory data and work history associated with section polygons. When implementing this option, the inventory and work history data structure must follow the PAVER standard. Export the map to a shape (.shp) file or table and import that file to PCASE or PAVER for use in either application.

2-3.2.3 Importing Maps to PCASE or PAVER Applications.

The PCASE and PAVER applications both use the same database. When you import the map in PCASE, the updated map is available in PAVER and vice versa. Details on importing GIS/tabular data are available in the PCASE and PAVER user guides. The process is the same for both.

2-3.2.4 Creating and Updating Inventory in PCASE or PAVER.

When the map imported from the GIS application does not include inventory or work history data, it is updated in PCASE or PAVER using the Define Inventory tool. The updated section data is then assigned to the section polygons using the GIS Assignment tool. When starting with an empty inventory, add a network to the inventory, then add branches to the network and add sections to the branches.

2-3.2.5 Linear Segmentation.

Linear segmentation is the process of linking the pavement inventory to the corresponding facilities in the Real Property data structure. This linkage is created between the branch and the facility using the Real Property Unique ID (RPUID) such that a pavement network can have one or many facilities and a facility can have one or many branches. The objective of creating this linkage is to be able to report pavement management data in Real Property terms (facility) for use at the Service and Office of Secretary of Defense (OSD) level. See UFC 3-270-08 for more detail.

2-3.3 Test Plan.

Using the mapping, inventory structure, and data gathered from previous reports and other sources, develop a test plan that defines the types and estimated number of tests required as outlined in paragraph 3-1 to accurately characterize the pavement structure of each section in the inventory. This historical data provides an indication of the uniformity of the pavement structure for each section. It is used to identify gaps in the data or the need to validate the historical data with testing. Note that even data captured twenty, thirty, or more years ago remains valuable. Once a soil reaches an equilibrium moisture content, strength and thickness may not change significantly over time. Where moisture varies seasonally or frost issues exist, address these seasonal variations with appropriate testing described in Chapter 8 and Appendix A. When performing testing such as coring or using a dynamic cone penetrometer (DCP), develop a map with the approximate locations of these tests. The map is helpful to communicate the test plan to the installation engineers and airfield management. Depending on the Service, installation, and type of testing, a work clearance request may be required before the start of field work.

2-3.4 Perform Field Testing.

Conduct field testing based on the test plan to determine the pavement characteristics and structure of each section in the inventory, using one or a combination of the procedures below. Note that additional testing is often required to supplement the test plan when test results deviate from previous evaluation, design, or construction data. When no previous evaluation or construction data are available and there is significant variability in the test plan testing results, conduct additional tests as required. Finally, access time on the airfield may limit the number of tests that can be performed. In these cases, prioritize the tests. Details on the procedures below are included in Chapter 3.

2-3.4.1 PCI Inspection.

Conduct a PCI survey to determine the PCI for each section or validate the results from a previous inspection.

2-3.4.2 Coring or Test Pits.

Coring is used to determine the pavement thickness, get asphalt or concrete samples, and provide access for DCP testing and collecting soil samples. Test pits are rarely used due to operational restrictions but provide the capability to collect more samples and do more robust material testing.

2-3.4.3 Dynamic Cone Penetrometer (DCP).

Use the DCP or automated DCP (ADCP) to determine soil strengths and layer thickness.

2-3.4.4 Falling Weight Deflectometer (FWD).

Provides deflection data used to backcalculate the moduli for the layered elastic analysis procedure.

2-3.4.5 Ground-Penetrating Radar (GPR).

Use GPR to determine pavement layer thicknesses for each material type and the presence of anomalies in a structure.

2-3.4.6 Portable Seismic Pavement Analyzer (PSPA).

Uses wave propagation and elastic theory to determine structural properties for the layered elastic analysis procedure.

2-3.4.7 MIRA Ultrasonic Tomography.

Used to estimate the average thickness of the section.

2-3.5 Perform Laboratory Testing.

Perform laboratory testing on asphalt, concrete, and soil samples taken during field work. This step in the process is used less frequently than in the past, but when performed, is typically done after the initial field work data compilation, modeling, and analysis. It is used to validate the initial results and improve the level of detail and overall quality of the report.

2-3.6 Compile Evaluation Data.

Select representative layer thickness, strength, and material types for the pavement surface, base course, subbase course, and subgrade of each section from available data and summarize the data in the physical property data (PPD) or construction history table of the report. These layer structures are used in the modeling and analysis process, so it is important to document any assumptions or limitations made in compiling the data. For example, if limited time did not permit additional testing or when data was taken from a previous report.

2-3.7 Modeling and Analysis.

There are two approaches to pavement modeling and analysis based on the performance models described in paragraph 2-2.2. The first is commonly known as airfield pavement evaluation (APE) analysis and the second is layered elastic analysis. Either or both models may be used, depending on the situation. While all of these procedures use different models to compute stresses, they all compute allowable passes, allowable load, the pavement classification number (PCN), and overlay requirements when required for each analyzed section.

2-3.7.1 APE Analysis.

APE analysis uses the CBR-Alpha-Beta Hybrid model for flexible pavement, the Westergaard model for rigid pavements, and the CBR-Alpha model for unpaved and mat airfields. These models are implemented in the PCASE APE module, which is typically used for contingency evaluations at forward operating locations or when layered elastic analysis does not yield reasonable results (e.g., on low strength pavements). APE inputs include the layer structure, thickness, and strength (CBR or k). Layers may be combined in some complex structures to facilitate analysis (e.g., a multi-layer composite pavement).

2-3.7.2 Layered Elastic Analysis.

Layered elastic analysis uses the YULEA model for both flexible and rigid pavement analysis. Layered Elastic analysis is implemented in the PCASE Layered Elastic Evaluation Program (LEEP). Layered elastic analysis is more commonly used to evaluate main operating installations but is also used at forward operating locations with an enduring mission. LEEP inputs include the layer structure, thickness, and properties (E and v). Similar layers are typically combined to simplify the structure being analyzed (e.g., combine the subbase and subgrade when they have similar material properties). Once the layer structure is defined, select FWD deflection (basin) data that define the pavement's response to loading and use it to determine the pavement layer moduli by matching the deflection basin with an elastic layer model. This process is known as backcalculation. Finally, select a representative model from the backcalculation procedure for layered elastic analysis.

2-3.8 Report Generation.

Report format and content varies by Service and mission (e.g., the report format and content for a contingency evaluation is different than that for a main operating installation). In general, a pavement evaluation report for a main operating installation has the report elements listed below. Contingency evaluations focus on tabular summaries of data collected in the field, PCI and structural analysis results, and any limitations to the proposed mission. More detailed information on report content is outlined in Chapter 10 of this UFC, UFC 3-270-08, and TSPWG M 3-260-03.02-19.

- Discuss construction changes that have occurred since the last evaluation
- Discuss changes in the installation mission regarding aircraft traffic mixes and define the critical aircraft and required overlays for deficient sections
- Discuss field data collection efforts and provide a tabular summary of the data structure for each section
- Tabular summary of PCI ratings
- Tabular summary of analysis results, including allowable aircraft loads, allowable aircraft passes, and PCN ratings
- Color maps for the inventory, PCI, and structural condition (e.g., PCN or Structural Index [ACN/PCN] ratios)
- Discuss structural capacity and functional condition deficiencies
- Recommend localized and global preventive maintenance and repair (M&R) requirements
- Recommend major M&R requirements and alternatives to address deficiencies

CHAPTER 3 DATA COLLECTION

3-1 GENERAL.

Selecting representative physical characteristics for a pavement section requires a thorough study of all existing information as well as field and, in some cases, laboratory testing. Previous evaluations, and when available, design and construction control data, provide a starting point for the evaluation test plan described in Chapter 2 that identifies test requirements for the evaluation. These tests fall into two general categories: nondestructive testing (NDT) and direct sampling.

3-1.1 Nondestructive Testing (NDT).

As the name implies, NDT does not require taking physical samples. This category includes methods such as the falling weight deflectometer (FWD), ground penetrating radar (GPR), and the MIRA ultrasonic tomography device.

3-1.2 Direct Sampling.

Direct sampling includes methods such as coring, DCP, split tensile, and soil classification as well as methods conducted in test pits such as in-place CBR, plate bearing, and soil density testing. More robust laboratory testing can be performed on asphalt, concrete, and soil samples collected from test pits. This chapter outlines data collection requirements and procedures. More detailed information on sampling and testing methods are discussed in Appendix A.

3-1.3 Determining Testing Methods.

There are a several factors that dictate the testing methods for an evaluation, including the purpose of the evaluation, logistics limitations, and site-specific testing limitations.

3-1.3.1 Purpose of the Evaluation.

Pavement evaluations fall into two general categories based on the nature of the mission: permanent and contingency. Contingency evaluations are further categorized as expedient or sustainment. A third general category is special purpose, in which the evaluation is focused on a specific issue. All require the same basic procedures as outlined in this chapter but differ in amount of data used in the evaluation and, in turn, the reliability of the results and the level of detail in the report. The evaluation classification is driven primarily by the purpose and time allotted for field work and analysis.

- Permanent: Managing pavement maintenance and repair (M&R) and long-term aircraft operations
- Contingency: Managing pavement and aircraft operations at forward locations
    - Expedient (100 passes or initial surge of mission aircraft)
    - Sustainment (5,000 passes or throughout anticipated operation)

- Special purpose: Address specific issues (e.g., void detection)

3-1.3.2 Logistics Limitations.

The ability to get test equipment to a site, time available for the evaluation, and access to the pavements are all limiting factors that determine the approach to pavement evaluation testing and analysis.

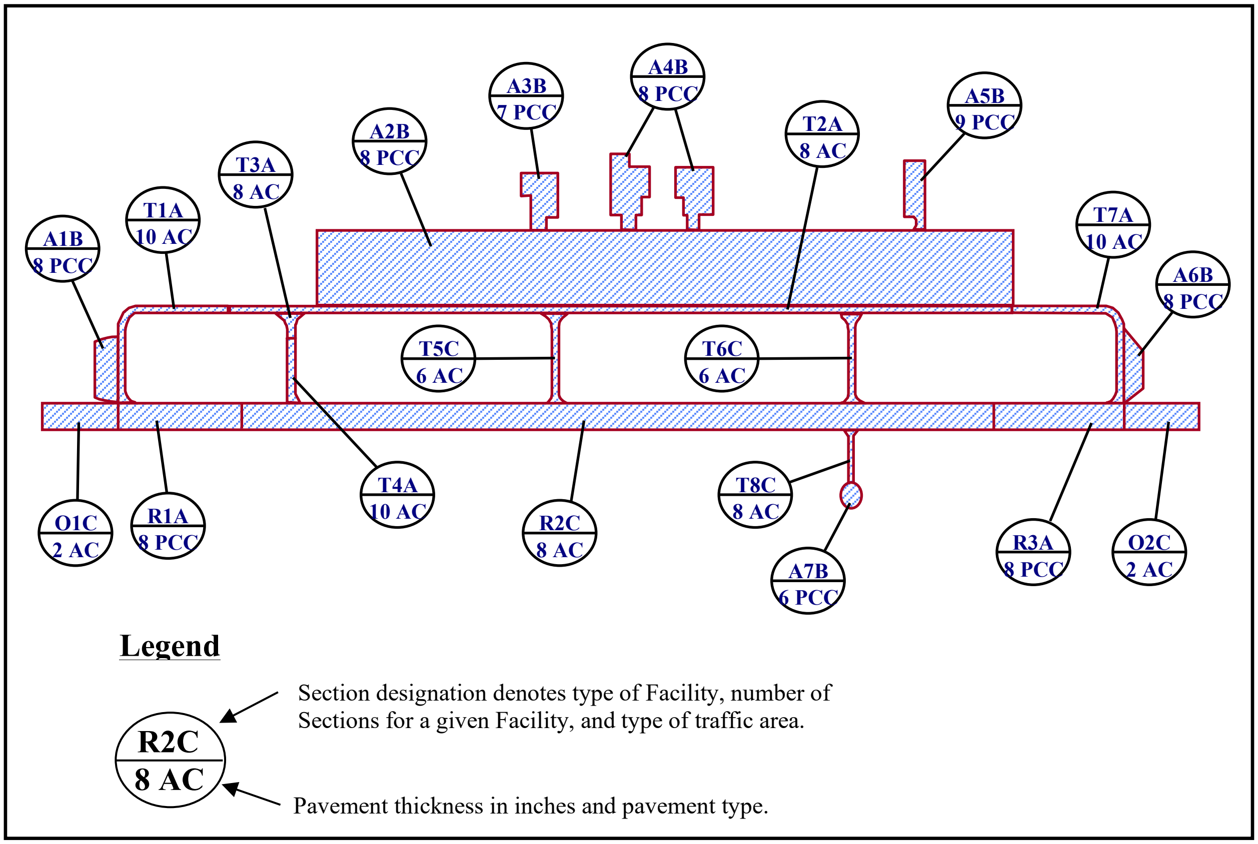
3-1.3.3 Site-Specific Testing Limitations.

The nature of the pavement or soils at the site can limit the reliability of the data. For example, FWD testing can provide unreliable results in certain soil types or when there is a high water table. DCP results can be unreliable in rocky soils. Research and understand the limitations of each test method to determine its suitability for the pavements or soils at each site. TSPWG 3-260-03.02-19, *Airfield Pavement Evaluation Standards and Procedures*, provides alternative testing procedures when site conditions limit the use of testing equipment.

3-2 MAPPING AND INVENTORY.

All testing described in this chapter is intended to determine representative physical characteristics at the pavement inventory section level as shown in Figure 3-1. It assumes that the mapping and inventory is established as outlined in paragraph 2-3.2, with specific details outlined in UFC 3-270-08. Note that Figure 3-1 is typical for a contingency evaluation. In an evaluation for a main installation, section IDs have a leading zero before the number (e.g., A01B and pavement thicknesses are typically rounded to the quarter inch, e.g., 8.25 AC).

Figure 3-1 Typical Airfield Section Map



3-3 PAVEMENT SURFACE CONDITION INSPECTION.

The Pavement Condition Index (PCI) is the standard measure of pavement surface condition used by DoD. PCI data are collected using the procedures outlined in UFC 3-260-16, which is the DoD equivalent of ASTM D5340, *Standard Test Method for Airport Pavement Condition Index Surveys*, with additional DoD-specific requirements. The PCI uses a scale from 0 to 100 to define the condition of the pavement as shown in Figure 3-2 and described in Table 3-1. Ideally a structural pavement evaluation includes a project-level PCI inspection (aka a PCI survey) but the situation may dictate a network-level inspection or even a cursory inspection, which is often the case in a contingency environment. The determining factors on the level of inspection are the intended use of the data and the time and manpower available.

Figure 3-2 PCI Rating Scale

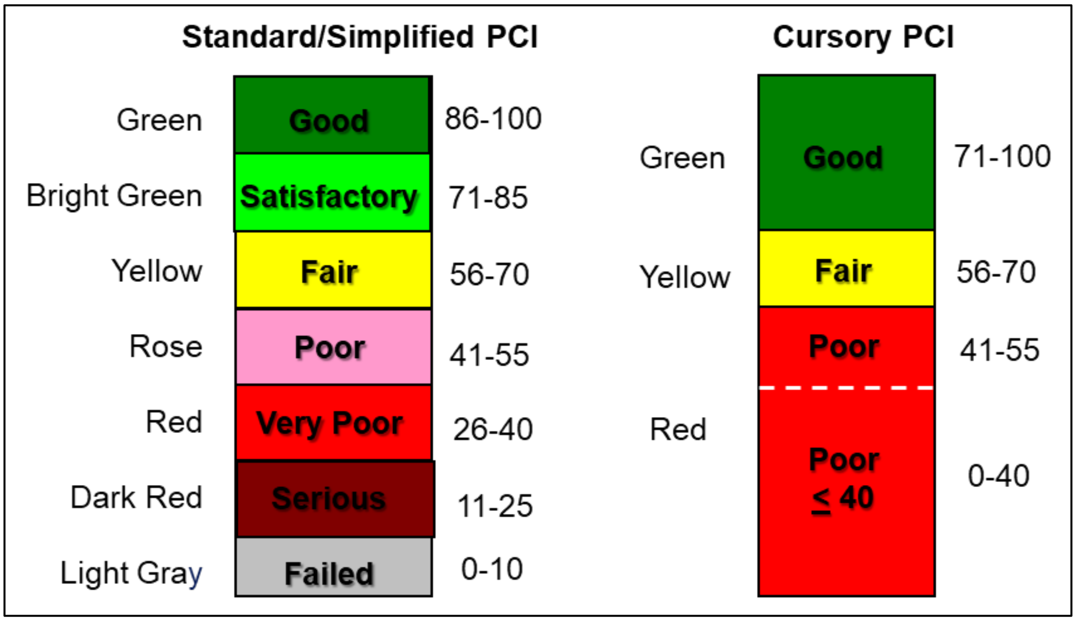


Table 3-1 PCI Rating Definitions

Rating	Definition
86–100	**GOOD:** Pavement has minor or no distresses and will require only routine maintenance.
71–85	**SATISFACTORY:** Pavement has scattered low-severity distresses, which should require routine maintenance.
56–70	**FAIR:** Pavement has a combination of generally low- and medium-severity distresses. Near-term maintenance and repair needs should be routine to major.
41–55	**POOR:** Pavement has low-, medium-, and high-severity distresses, which probably cause some operational problems. Near-term maintenance and repair needs should range from routine to reconstruction.
26–40	**VERY POOR:** Pavement has predominantly medium- and high-severity distresses causing considerable maintenance and operational problems. Near-term maintenance and repair needs will be intensive in nature.
11–25	**SERIOUS:** Pavement has mainly high-severity distresses, which cause operational restrictions; immediate repairs are needed.
0–10	**FAILED:** Pavement deterioration has progressed to the point that safe aircraft operations are no longer possible; complete reconstruction is required.

3-3.1 Project-Level PCI Inspection.

The project-level PCI is referred to as a standard PCI inspection in some contingency pavement evaluation material and in Figure 3-2 above. The PAVER pavement management application implements the inspection process outlined in UFC 3-260-16. It requires inspecting sufficient samples to achieve a 95 percent confidence level. Determine the samples to be inspected based on a systematic random sampling process. The formula for determining the number of samples is in UFC 3-260-16 but Table 3-2 provides a general idea of sampling requirements. Use the seven-tier PCI scale shown in Figure 3-2 and Table 3-1 when reporting results. Use project-level inspections when the data is used to develop project management plans for main installations or to meet a specific requirement for a higher confidence level. PAVER uses the same database structure as the Pavement-Transportation Computer Assisted Structural Engineering (PCASE) application, so PCI inspection data are also accessible in PCASE for use in structural analysis.

Table 3-2 Project-Level Sampling Requirements

PCC Sampling			
Total # of SU	Survey #	Total # of SU	Survey #
1-10	ALL	50-55	22
11-13	10	56-61	23
14-15	11	62-70	24
16-17	12	71-79	25
18-19	13	80-91	26
20-22	14	92-105	27
23-24	15	106-122	28
25-27	16	123-145	29
28-31	17	146-175	30
32-35	18	176-217	31
36-39	19	218-280	32
40-43	20	281-330	33
44-49	21	>= 331	10%

ACC Sampling	
Total # of SU	Survey #
1-7	ALL
8-11	7
12-15	8
16-19	9
20-24	10
25-32	11
33-44	12
45-64	13
65-104	14
105-150	15
≥151	10%

3-3.2 Network-Level PCI Inspection.

The network-level PCI is referred to as a simplified or contingency pavement condition survey in some contingency pavement evaluation references. It is also conducted in accordance with UFC 3-260-16 but requires a lower sample rate than a project-level PCI, as shown in Table 3-3. Another difference between the project and network-level PCI is that the network-level PCI requires representative rather than random samples. The inspector must determine the typical distress types in the section and inspect samples that are typical of the entire section. Place emphasis on structural or foreign object damage (FOD) -related distresses. Use the seven-tier PCI scale shown in Figure

3-2 and Table 3-1 when reporting results. The network-level inspection is typically used for contingency evaluations at forward or en-route operating locations but may also be used at sites such as auxiliary fields when there is not a specific requirement for a higher confidence level.

Table 3-3 Network-Level PCI Sampling Requirements

Section Size (Total Samples)	Sample Units to Survey
1 to 5	1
6 to 10	2
11 to 15	3
16 to 40	4
Greater than 40	10%

3-3.3 Cursory Pavement Condition Inspection.

In a cursory pavement condition inspection, the number of inspected sample units may be less than the minimum requirements for a network-level inspection. Use the same process outlined in UFC 3-260-16 when time permits or, when time is limited, conduct a visual assessment noting the primary distresses with a focus on distresses that cause limitations or mission impacts to aircraft. Mission-critical PCI values typically occur when the value is less than 40 or 25. In either case, report the results of a cursory survey as a qualitative assessment of the pavement surface condition using the Cursory three-color scale in Figure 3-2. When a cursory condition survey is conducted using the simplified evaluation procedures, the evaluation is considered "expedient" and valid for limited or immediate use only. Cursory inspections are typically used in contingency pavement evaluations.

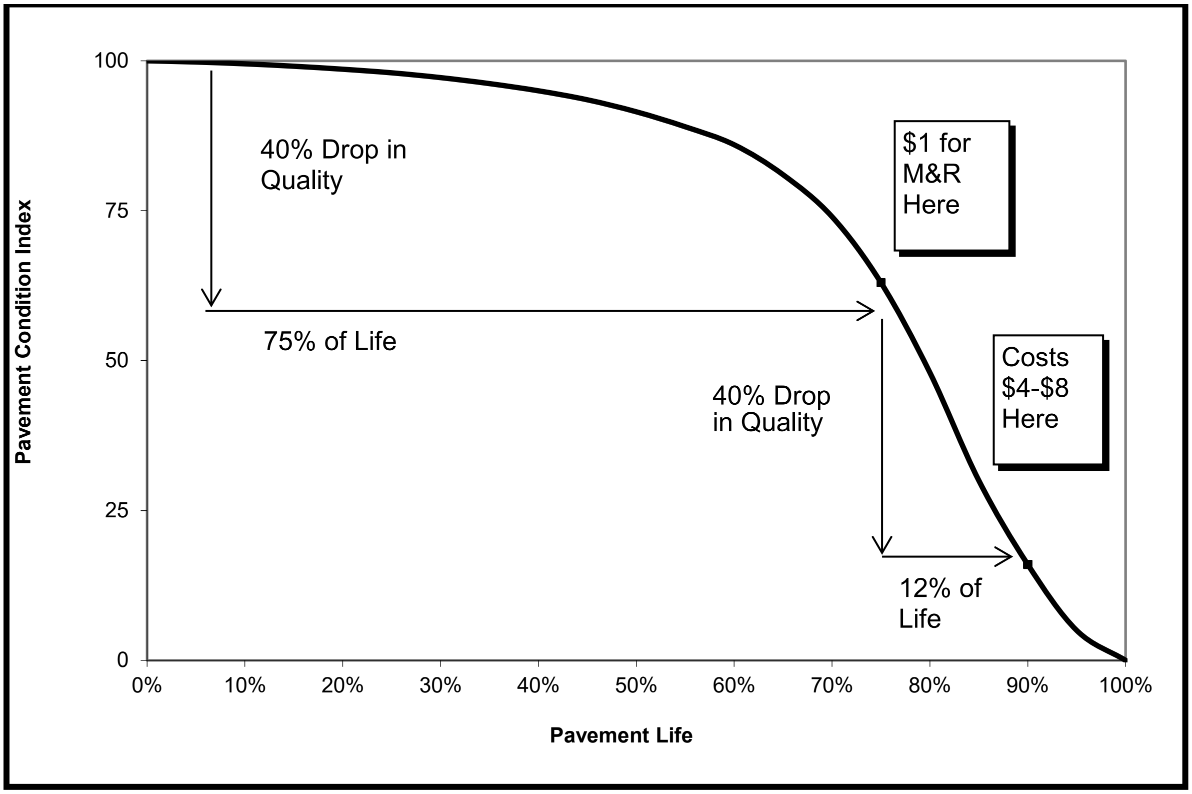
3-3.4 Using PCI Inspection Results.

The PCI of a pavement plays a critical role in determining localized and global preventive as well as major M&R requirements in pavement management plans. It is also used in all the structural evaluation pavement analysis procedures.

3-3.4.1 PCI Use in Pavement Management.

The PCI is used to determine the rate of deterioration of the surface condition and predict the future condition of the pavement. The predicted PCI is used to plan appropriate cost-effective M&R actions. When a pavement deteriorates to a condition where it is no longer cost-effective to do localized or global M&R (known as the critical PCI), major M&R is triggered to address issues before the pavement deteriorates to the point that reconstruction is required (see Figure 3-3). While each Service establishes PCI levels that trigger major M&R and reconstruction based on their respective missions, the general principles remain the same: invest in localized and global M&R to extend the service life of GOOD pavement and invest in major M&R at the appropriate time to delay the need for reconstruction.

Figure 3-3 Typical Pavement Life Cycle (APWA,1983)



3-3.4.2 PCI Use in Pavement Structural Analysis.

The pavement condition can adversely affect aircraft operations because some distresses generate foreign object debris (FOD) that poses a risk to aircraft operations. It can also help identify potential structural problems (e.g., structural distresses that indicate the pavement is overloaded or at the end of its service life). When the PCI is 40 or lower, reported allowable gross loads (AGL) are reduced by 25 percent. In addition, the PCI is used to compute the structural condition index (SCI). The SCI is like the PCI but only considers the load-related distresses. The SCI is used as the failure criteria for rigid pavement when doing layered elastic analysis and used to determine the condition factors Cb and Cr that determine the equivalent thickness of existing overlays on rigid pavements or new overlay requirements when an existing pavement is not capable of supporting the evaluation traffic.

3-3.5 Additional Contingency Evaluation Considerations.

The amount of time available to conduct PCI surveys impacts the number of sample units inspected. The evaluator’s most important task is to accurately identify the correct distress type and severity level as described in UFC 3-260-16. Acceptable errors in distress quantity will have less of an impact on the PCI value and FOD risk to mission aircraft. Typically, medium- and high-severity distresses create the highest FOD potential that may cause operational limitations or impacts to the mission aircraft. While the PCI value may provide an indirect measure of subsurface deficiencies, it is important to consider both the surface condition (e.g., PCI) from a function perspective and structural evaluation results. A pavement surface may rate GOOD (PCI 71 to 100) but have underlying pavement or soil conditions that could result in pavement failure under repeated aircraft operations. On the other hand, a pavement may be structurally sound, but the surface condition may be hazardous for aircraft traffic (e.g., FOD).

3-4 NONDESTRUCTIVE PAVEMENT TESTING.

Paragraph 3-1.1 describes three nondestructive pavement testing techniques currently in use, the FWD, GPR, and the MIRA device. Each is used to determine different pavement characteristics are used in conjunction with each other or in conjunction with direct sampling testing.

3-4.1 Falling Weight Deflectometer (FWD).

The falling or heavy-weight deflectometer (FWD/HWD) is an impulse loading device that measures the response of a pavement system to a falling dynamic load that simulates a moving vehicle or aircraft wheel. FWD is the generic term for the device, with the HWD capable of applying a heavier load than other FWDs, but the terms are used synonymously. The heavier load of the HWD is preferred on thick AC and PCC airfield pavement structures. The objective is to apply the maximum load possible to simulate aircraft loading without overloading the FWD sensors. ASTM D4694, *Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load Device*, provides detailed guidance on the FWD and HWD test procedures.

3-4.1.1 FWD Description.

The load on the pavement (impulse force) from an FWD is created by dropping weights from different heights onto a rubber or spring buffer system. The standard loading plates used to transmit the applied force to the pavement are either 12 inches (300 millimeters) or 18 inches (450 millimeters) in diameter, with the 12-inch plate being most commonly used for AC or PCC pavement surfaces while the 18-inch load plate used for unbound aggregate layers and stabilized subgrades. The drop height is varied to produce an impact force up to 56,000 pounds (25,401 kilograms), depending on the HWD. Other FWD models are limited to lower loads more typical of road traffic.

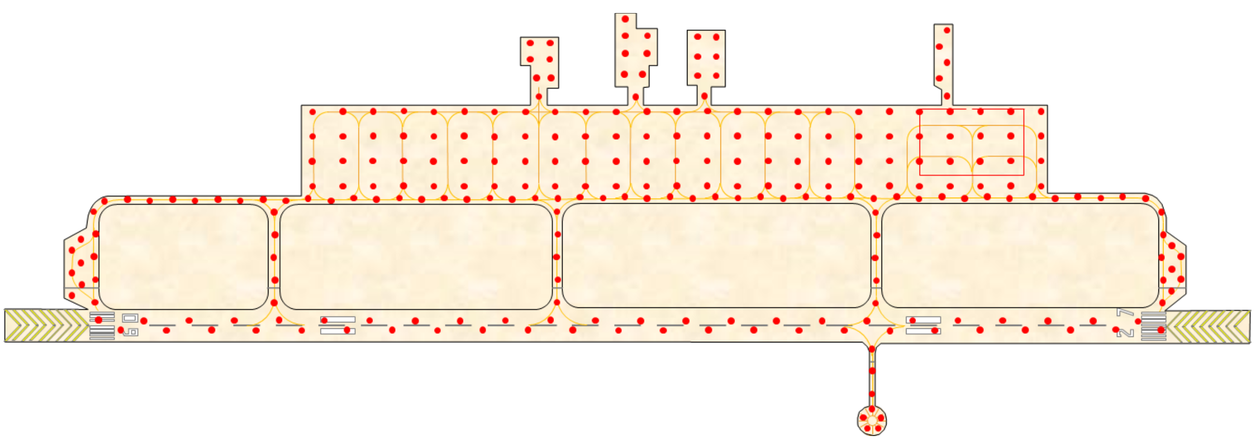
3-4.1.2 Measuring Pavement Response.

The FWDs currently in use by DoD use geophones, which are velocity transducers that convert ground movement (velocity) into voltage to measure the pavement response to the applied load. The load is measured by a load cell integral to the FWD load system and the pavement response is captured by the pavement deflection which is obtained by integrating the surface velocity measured by the velocity transducers. Other systems use seismometers but both are typically referred to as sensors. Seven sensors are preferred, with a sensor located at the center of the load plate and the remaining sensors at 12-inch (300-millimeter) intervals, with the outermost sensor (farthest from the falling weight load application) at 72 inches (1829 millimeters). There may be instances when fewer sensors are used but in no case should the outermost sensor be less than 48 inches (1219 millimeters) from the load.

3-4.1.3 Test Location and Density.

The time required to measure the deflection basin at each testing location is short (one to two minutes), allowing for many tests in a short period of time. Conduct FWD testing at 100-foot (30-meter) intervals on runways and taxiways. Alternate tests on either side of the centerline at an offset that is within the main gear wheel paths of aircraft that frequently use the airfield or are based at the site. The centerline offset is usually 10 to 12 feet (3 to 4 meters) for flexible pavements. Adjust this offset distance for rigid pavements as required to accommodate joint layouts and PCC slab size. Conduct FWD tests on apron areas in a grid pattern at 100- to 200-feet (30- to 61-meter) spacing. As seen in Figure 3-4, the procedure outlined above establishes longitudinal profiles along the runways, taxiways, and aprons to produce a test density that gives a comprehensive assessment of subgrade, base, and pavement structural condition. The uniformity of results will dictate whether test spacing can be increased or whether additional tests should be conducted when there are large variations in pavement response. When failed areas or areas of excessive pavement distress are encountered, locate enough FWD and other tests in the failed or distressed areas to determine the cause of the failure or distress. Conduct a minimum of five FWD tests on all pavement sections.

Figure 3-4 FWD Test Locations



3-4.1.4 Performing the FWD Test.

Position the FWD equipment at each test location and initiate the test sequence through the FWD application provided with the system. Test sequences require a minimum of three drops (loads) and use of the same drop heights (e.g., 2-4-4) throughout a given section. The first loading is at a lower drop height and is considered a seating load and results are not typically used in backcalculation. The second and third loadings are set to maximize the magnitude of the loading without exceeding the geophone limitations. They should produce similar results and are used for the analysis. If inconsistencies are observed in either of these test sequences (e.g., high errors or inconsistent basin shape), select the better of the two drop sequences for analysis. The load is applied for each drop in the sequence, the resulting surface deflections are determined at each geophone location, and the results are stored in a data file. There are several data file formats, but a formatted text file with a .fwd or .hwd file extension is typically used by DoD. Import the .fwd file (or other chosen format) into PCASE for analysis.

3-4.1.5 FWD Testing for Asphalt.

The modulus of bituminous concrete is temperature dependent. There are relationships between the temperature and the modulus used in backcalculation and analysis as described in Chapter 5. The relationship for backcalculation requires the mean pavement temperature at the time of testing. This datum can be captured by measuring the temperatures with thermometers installed 1 inch (25 millimeters) below the pavement surface, 1 inch (25 millimeters) above the bottom of the AC layer, and at mid-depth of the bituminous layer, but this procedure is seldom used. The standard approach is to collect data on the average (mean) air temperature for the five-day period prior to the day of testing and adding it to the measured pavement surface temperature, which is captured by the FWD at the time of the test to determine the mean pavement temperature using the relationship described in Chapter 5.

3-4.1.6 FWD Testing for Concrete.

Perform tests on rigid pavements near the center of the PCC slabs but at a minimum of 3 to 6 feet (1 to 2 meters) away from the joints and linear cracks that may exist within the slab. When a slab width or length is less than 20 feet (6 meters), center the entire sensor array on the slab, keeping the outer sensor at least 3 feet (1 meter) from the joint.

3-4.1.7 FWD Testing for Joint Load Transfer.

Rigid airfield pavements are commonly designed to transfer at least 25 percent of the load on a slab to adjacent slabs. FWD testing is used to verify that the load is being transferred across the joint. Figure 3-5 shows the test configuration with the plate (and sensor 1) on the loaded slab and the second sensor on the unloaded slab. The deflection ratio of the unloaded slab to the loaded slab is the deflection ratio used to determine the joint load reduction factor using the relationship shown in Figure 3-6 to define joint transfer efficiency. If the joint load transfer is poor, the load-carrying capacity of the PCC slabs is reduced, with a corresponding decrease in the pavement service life.

Joint testing policy varies. In some cases, joint testing is always done and in other cases it is only done when there are indications that there is poor load transfer such as longitudinal cracking in the wheel path on multiple slabs in a section. In either case, determine the number of center slab tests, take 20 percent of that number and perform joint tests on that number of slabs. Joint load transfer is temperature dependent; testing in the morning can yield different results in the afternoon as slabs heat up and expand. While not always feasible, it is best to perform NDT work in the spring or fall to avoid high temperatures in the summer and cold temperatures in the winter that may not represent typical load-transfer for a pavement system. If NDT work must be performed in the summer, consider early-morning testing when temperatures are typically cooler than in the afternoon. This is especially relevant if joint load transfer exists primarily from aggregate interlock because no dowel bars exist in the jointed PCC pavement. Reference point tests can establish a relationship between air temperature and the deflection ratio from NDT such that adjustments are made to test results collected over a wide range of temperatures. Select a reference slab within each section to be tested on a given day. Conduct joint tests on each reference slab at one- to two-hour intervals throughout the testing period, or at closer intervals if the testing period is less than four hours on a given section.

Figure 3-5 NDT Configuration for Determining PCC Joint Load Transfer

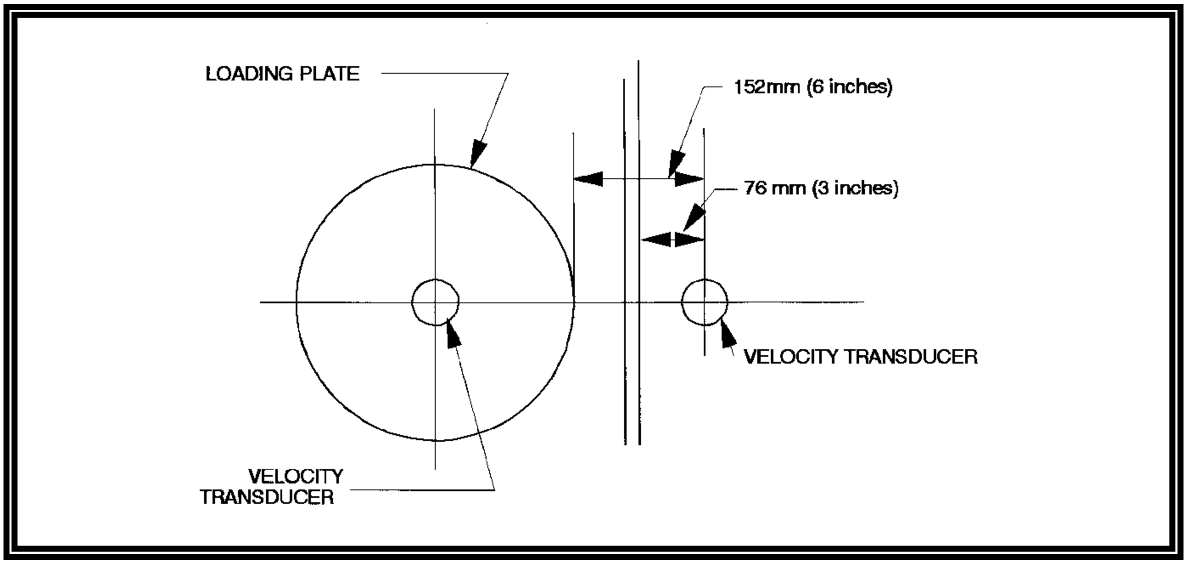
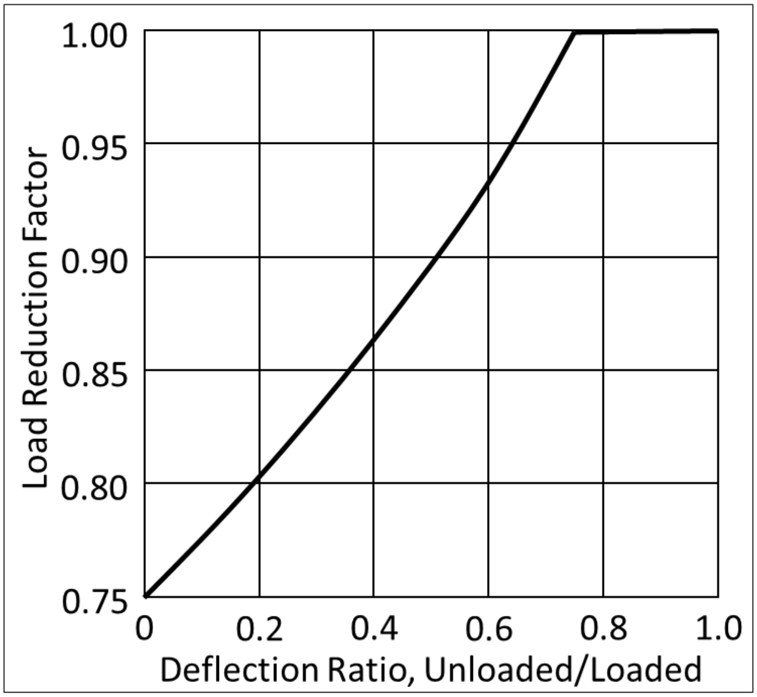


Figure 3-6 Joint Load Reduction Factor



3-4.2 Ground Penetrating Radar (GPR).

The primary benefit of GPR is that it can collect large amounts of detailed data in a short time. Use GPR to determine pavement layer thicknesses for each material type and the presence of anomalies in a structure. There are air-coupled and ground-coupled GPR variants that use electromagnetic radiation, usually in the range of 10 MHz to 2.6 GHz. Higher frequencies do not penetrate as far as lower frequency antennae but may provide better resolution. A GPR transmitter emits electromagnetic energy into the structure. When the energy encounters a buried object or a boundary between materials having different permittivity, it is reflected, refracted, or scattered back to the surface. The receiving antenna records the variations in the return signal.

GPR is sensitive to specific site conditions. The material types encountered will dictate the ability of the GPR to evaluate the layered structure. Dry, sandy soils or materials such as granite or limestone tend to be resistive rather than conductive and can penetrate up to 49 feet (15 meters). Moist or clay-laden soils and materials with high electrical conductivity can limit penetration to as little as a few inches. Materials with similar dielectric constants will limit the ability to discern layer changes. Before testing, calibrate the GPR system (see Figure 3-7) at each site using cores and a steel plate. Take measurements along FWD testing paths on each side of the centerline for taxiways and runways and along the FWD testing path on aprons. The data collection system records the voltage and time history of the signal and GPS location and camera images for use in post-processing. Post-process the data to determine layer thickness by comparing voltage peaks (amplitude) and the time between peaks to estimate the layer thickness and use these data to determine the average layer thicknesses for each section. Use the procedure outlined in Appendix B for void detection.

Figure 3-7 Air Coupled GPR

3-4.3 MIRA Ultrasonic Tomography.

The MIRA ultrasonic tomography device is used to determine concrete pavement thickness. The standard procedure is to take measurements near each FWD test location with the objective of achieving a 95 percent confidence level that the average value reported for each section is within 0.5 inch (13 millimeters) of the true value. Round the computed average value to the nearest 0.25 inch. While coring has the benefit of providing a sample that can be measured and tested for flexural strength, the MIRA can test more locations to determine an average thickness without the need to repair core holes. Figure 3-8 shows a MIRA device being used for testing.

Figure 3-8 MIRA Ultrasonic Tomography

3-5 DIRECT SAMPLING.

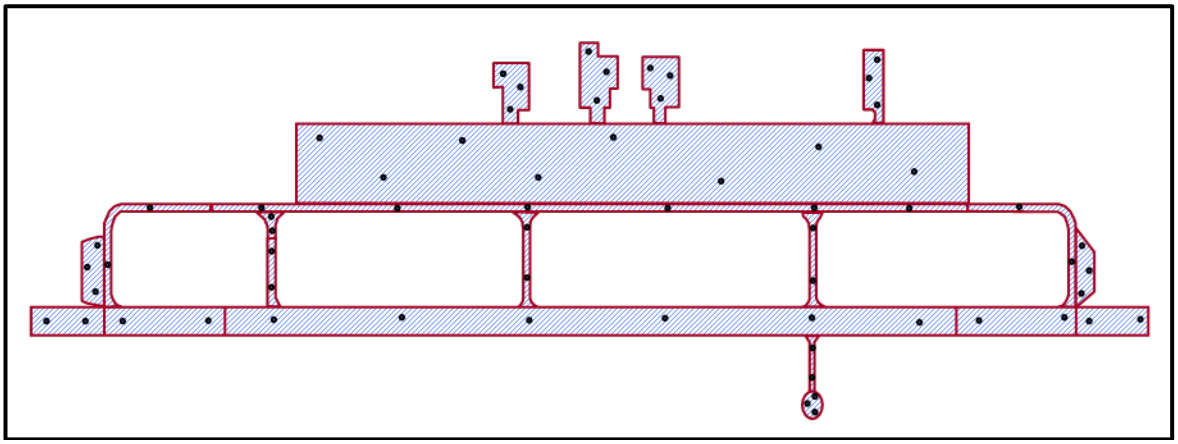
3-5.1 Pavement Coring and Drilling.

Pavement coring or drilling is used to collect pavement and soil samples, verify the pavement thickness, and provide access to the subsurface layers for DCP testing.

3-5.1.1 Coring or Drilling Locations.

When there were previous evaluations at a site, select locations that were not previously tested and use the data from both the previous and new evaluation to define the representative pavement structure. For rigid pavements, core in the center of the slabs to avoid thickened edges. Recording new location GPS coordinates can assist in preparing GIS maps. For contracted coring or drilling work, obtain GPS data for each location if the airfield owner allows GPS data collection. When coring or drilling is done in conjunction with FWD testing, use the FWD data to identify locations for additional testing where there are anomalies or changes in strength. The size and uniformity of the section dictates the number of tests required but test at least three locations for any new pavement not previously tested. Perform tests in the aircraft wheel paths on alternating sides of the centerline and in any weak areas. Conduct additional tests to verify the boundaries of these areas. When test time is limited, prioritize runway and taxiway tests. Figure 3-9 shows a typical test plan for coring or drilling.

Figure 3-9 Typical Coring/Drilling Test Plan



3-5.1.2 Pavement Coring.

Figure 3-10 shows a typical coring operation. The core drill uses 4- to 8-inch (102- to 203-millimeter) -diameter, diamond-tipped coring barrels (6-inch [152-millimeter] is the norm) to cut through asphalt or concrete pavements. This type of pavement coring system can cut through pavements to depths greater than 36 inches (914 millimeters) using a technique known as double dipping to remove the core in sections. Measure the cores to the nearest 0.25 inch and inspect them in the field for evidence of defects such as alkali-silica reaction (ASR).

Figure 3-10 Typical Coring Operation

3-5.1.3 Pavement Drilling.

Impact or rock drills are commonly used for contingency pavement evaluations because they have a smaller logistics footprint. Drill 1- to 1.25-inch (25- to 32-millimeter) -diameter holes through bound materials or any layers impenetrable by a DCP. Pavement thickness is measured to the nearest 0.25 inch in the drill hole.

3-5.2 Dynamic Cone Penetrometer (DCP).

The DCP is a device used to determine the thickness and strength of the soil layers in a pavement structure. There are manual, semi-automated, and automated DCP variants but all apply the same principles to measure the depth of penetration for a known applied load to determine a DCP index (in. or mm / blow). The DCP index is empirically correlated to CBR, k, or modulus.

3-5.2.1 DCP Description.

The four main components of the DCP are the 0.79-inch (20-millimeter) -diameter 60-degree cone, the rod, the anvil, and the 17.6-pound (7.98-kilogram) hammer, as described in ASTM D6951, *Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications*. The cone is driven into the ground by raising and dropping a hammer 22.6 inches (575 millimeters) against the anvil. The manual hand-held version shown in Figure 3-11 is portable, requires the hammer be lifted manually, and the depth of penetration measurements be taken manually using an incremented measuring stick. The correct number and length of extensions in the field must account for the thickness of all bound layers or materials that cannot be penetrated by the DCP. The semi-automated version requires lifting the hammer manually but the depth of penetration is measured automatically using a magnetic rule, string potentiometer, or similar device, and the blow count and depth of penetration are saved in a data file.

The automated DCP (ADCP) has a mechanism for lifting the hammer to the prescribed height and releasing it, then recording the blow count and depth of penetration, which are saved in a data file. Figure 3-12 shows an example of a system that has both a core drill and an ADCP.

Figure 3-11 Schematic of DCP

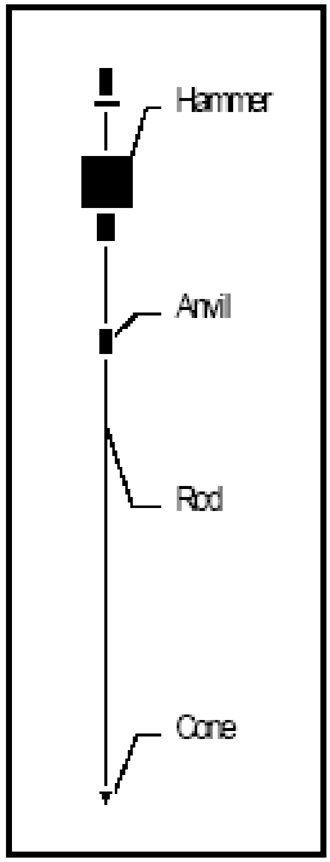


Figure 3-12 ADCP

3-5.2.2 DCP Test Procedure.

A 1-inch (25-millimeter) or 6-inch (152-millimeter) -diameter hole is drilled or cored through the pavement until the top of the base or subgrade is encountered. The rod with the cone attached is placed in the hole until it is in contact with the soil, then the hammer is raised and dropped and the depth of penetration recorded. The standard test is designed to penetrate soils to a depth of 48 inches (1219 millimeters) from the top of the pavement, although extension kits can go deeper, with a maximum recommended depth of 6.5 feet (2 meters). Testing is normally done to 36 inches (914 millimeters) in contingency evaluations. Once the DCP test is completed, the DCP is removed from the hole and soil samples for lab testing can be taken using a hand auger. Detailed test procedures and correlations for using the DCP and ADCP are provided in TM 3-34.48-2, *Theater of Operations: Roads, Airfields, and Heliports - Airfield and Heliport Design*.

3-5.3 Test Pits.

Test pits are seldom used for evaluations due to the number of tests required to characterize an entire airfield and the time it takes to open the pit and conduct testing, both of which typically have a significant impact on the mission. They are used more frequently for geotechnical work associated with a project or when doing forensic analysis to determine the cause of a pavement failure. Test pits provide greater opportunity to collect more pavement samples and larger soil samples for testing. An alternative to test pits is a minimum of three core holes up to 8 inches (203 millimeters) in diameter to permit in-place small aperture CBR tests and obtain samples for laboratory tests. The size of the test pits and some test procedures vary between flexible and rigid pavements. Following are descriptions for both pavement types.

3-5.3.1 Test Pits for Flexible Pavements.

Test pits for flexible pavements are approximately 4 feet (1 meter) wide by 5 feet (1.5 meters) long. Whether doing a full test pit or core holes for small aperture CBR testing, record the general condition of the pavement and a visual classification of materials from each test pit or core hole. Take several measurements around the perimeter of the test pit or core hole to determine the representative pavement thickness to the nearest 0.25 inch. For each test pit, perform CBR and field density tests on the base and collect disturbed and undisturbed soil samples of the base material for laboratory testing. Remove the remaining base material and measure the thickness of the base at several locations around the perimeter to determine the representative base thickness to the nearest 1 inch. Repeat this process for each subbase and the subgrade. Describe each soil course, noting color, in situ conditions, texture, and a visual classification. Sampling procedures, test descriptions, and testing references are included in Appendix A.

3-5.3.2 Test Pits for Rigid Pavements.

Test pits for rigid pavements are a minimum of 4 feet by 5 feet (1 meter by 1.5 meters), although the size of the test pits for rigid pavements depends, in part, on the thickness of the pavement because the length of the beams for flexural strength tests cut from the slab must be at least three times the pavement thickness, except when 6-inch by 6-inch (152-millimeter by 152-millimeter) beams are cut from the top and bottom of the slab for a three-point beam test. Record the general condition of the pavement and a visual classification of materials from each test pit. Take several measurements around the perimeter of the test pit to determine the representative pavement thickness to the nearest 0.25 inch. For each test pit, perform a plate bearing test, field density tests and, in some cases, CBR testing on the base. Collect disturbed and undisturbed soil samples of the base material for laboratory testing. Remove the remaining base material and measure the thickness of the base at several locations around the perimeter to determine the representative base thickness to the nearest 1 inch. Repeat this process (without the plate bearing test) for each subbase and the subgrade. Describe each soil course, noting color, in situ conditions, texture, and a visual classification. Sampling procedures, test descriptions, and testing references are included in Appendix A.

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CHAPTER 4 TRAFFIC

4-1 TRAFFIC DEFINITION.

A fundamental component of pavement evaluation is the traffic concept. Traffic is the mix of different aircraft types, loads, and number of passes used for the evaluation analysis. This group of aircraft defines the anticipated applied stress and number of stress repetitions the pavement will experience. Traffic is applied in the analysis procedures in different ways, depending on the Service and mission. Following is a summary of traffic terms and concepts and the various ways traffic is defined in an evaluation.

4-1.1 Traffic Pattern.

Traffic pattern is a term used to describe one or more aircraft or ground vehicles, with the weight and number of passes defined for each. The term traffic pattern is often used interchangeably with traffic mix and aircraft group when the loads and passes are defined for the aircraft in the group.

4-1.2 Traffic Mix.

Traffic mix is a term used to describe one or more aircraft or ground vehicles with the weight and number of passes defined for each. The term traffic mix is often used interchangeably with traffic pattern and for aircraft group when the loads and passes are defined for the aircraft in the group.

4-1.3 Aircraft Group.

An aircraft group is a collection of one or more aircraft organized by a specific criterion (e.g., pavement effect, gear type, or mission). When the load and passes are defined for each aircraft in the group, the term is synonymous with the term traffic pattern or traffic mix.

4-1.4 Representative Aircraft.

An aircraft in an aircraft group that is representative of the group based on a specified criterion such as gear configuration, weight, or a combination of these that defines the effect on the pavement for that group.

4-1.5 Controlling Aircraft.

The controlling aircraft is used in a mixed traffic analysis. In design, it is the aircraft in the traffic mix that requires the greatest pavement thickness. In evaluation, it is the aircraft with the fewest allowable passes.

4-2 AIRCRAFT PASSES.

Passes are defined as the number of aircraft movements across an imaginary transverse line placed within 500 feet (152 meters) of the end of the runway. Since touch-and-go aircraft operations will not pass this line, they are not counted. For taxiways and aprons, passes are determined by the number of aircraft movements across a line on the primary taxiway that connects the runway and the parking apron. At single-runway airfields with a parallel taxiway, the pass levels for the runway, taxiway, and apron could be the same, but passes can vary based on the airfield configuration as shown in Figures 4-1 through 4-4.

Figure 4-1 Takeoff and Land in Same Direction with No Back Taxiing

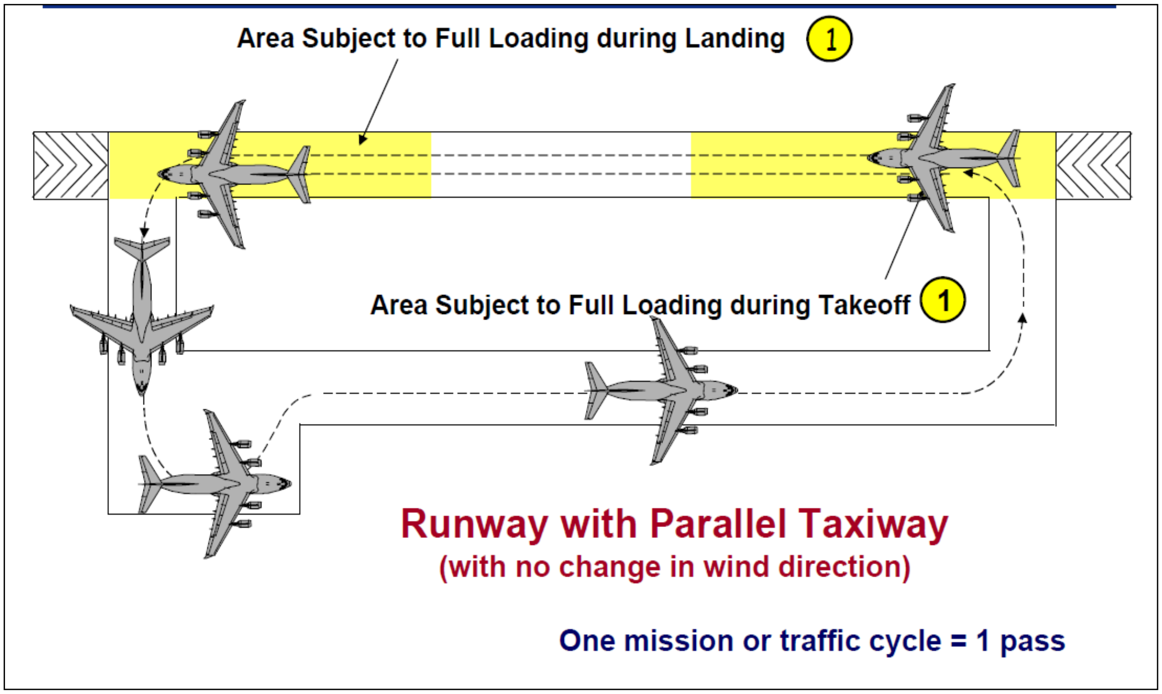


Figure 4-2 Takeoff and Land in Opposite Directions with No Back Taxiing

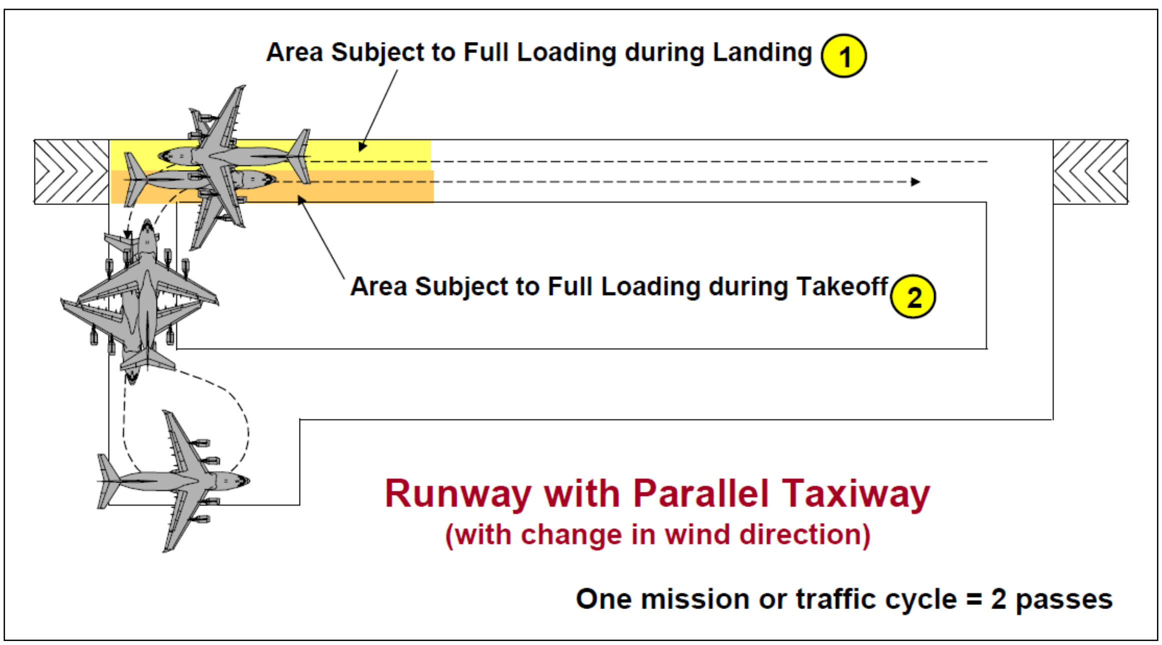


Figure 4-3 Takeoff and Land in Same Direction with Back-Taxiing

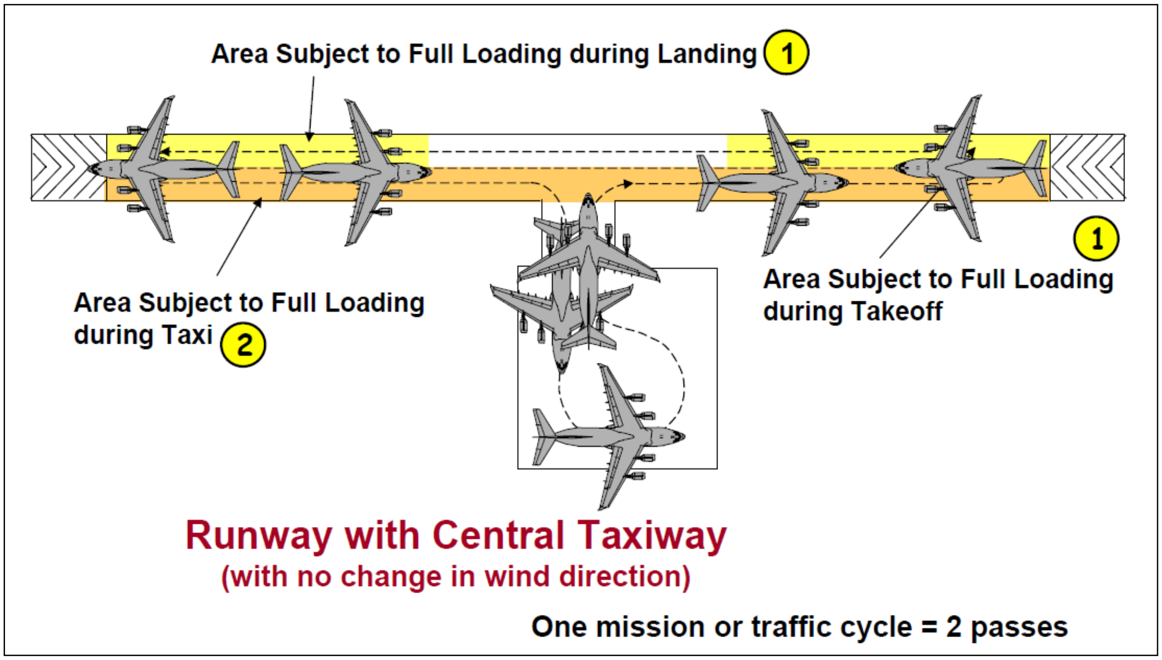
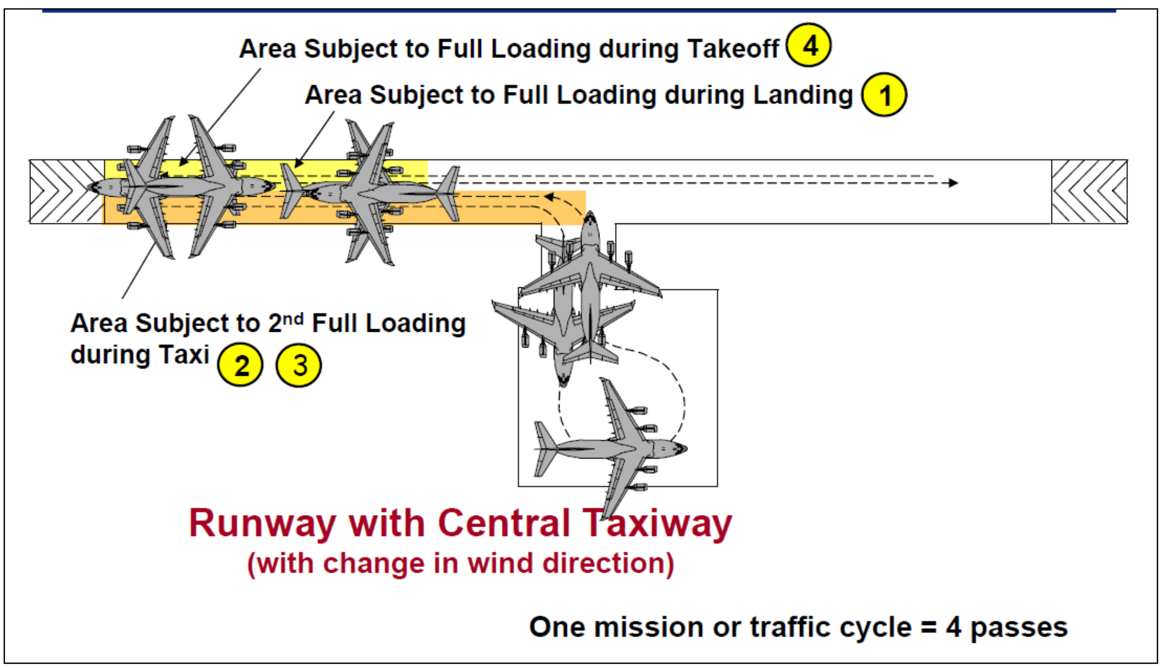


Figure 4-4 Takeoff and Land in Opposite Directions with Back-Taxiing



4-3 AIRCRAFT COVERAGES.

Passes are converted to coverages for analysis. Coverage is a term used to define the number of maximum stress repetitions that occur in a pavement due to aircraft operations. For flexible pavement, a coverage occurs when every point on the pavement surface within the traffic lane has been subjected to one application of maximum stress by operating aircraft. For rigid pavement, a coverage occurs when each point in the pavement within the limits of the traffic lane has been subjected to a maximum stress by operating aircraft. Maximum stress is the stress induced in the pavement by the aircraft wheels when the aircraft is operating at its maximum gross weight. An important point is that the surface criteria (AC and PCC) are based on coverages to failure, while the subgrade criteria are based on repetitions to failure. The lateral distribution of traffic has a greater effect on the number of maximum stress applications that occur at a point near the surface than for a point deep within the pavement structure (ERDC Miscellaneous Paper S-73-56, *Lateral Distribution of Aircraft Traffic*). A coverage is a function of gear configuration and tire width as well as the traffic area, so the pass/coverage (P/C) ratio varies for each aircraft and for each traffic area. The Pavement-Transportation Computer Assisted Structural Engineering (PCASE) application implements the P/C ratio concept for rigid and flexible pavement design and evaluation. These ratios are shown in TSPWG M 3-260-03.02-19.

4-4 TRAFFIC AREA.

The traffic area defines the wander width and load condition on specific portions of the airfield.

4-4.1 Wander Width.

Wander width is defined by whether aircraft traffic is close to the centerline of the runway or taxiway or whether they tend to deviate from the centerline. The first scenario is known as channelized traffic and is used when 75 percent of traffic occurs within ±35 inches (889 millimeters) from the center line for a runway or taxiway (a 70-inch [1778-millimeter] wander width). The second scenario is known as unchannelized traffic, which is used when 75 percent of traffic occurs within ±70 inches (1778 millimeters) from the centerline of a runway, taxiway, or apron (a 140-inch [3556-millimeter] wander width). The pass-to-coverage ratio for a given aircraft is lower for channelized traffic than for unchannelized traffic.

4-4.2 Load Condition.

An aircraft is typically fully loaded as it moves from the apron onto the taxiway and to the runway end. As the aircraft takes off, the wings provide lift and the interior portion of the runway is not typically experiencing the full aircraft weight. As an aircraft lands, the wings are still providing lift until the aircraft comes to taxi speed at the end of the runway, onto the taxiway and then to the apron. When the airfield configuration requires back-taxiing, as seen in Figures 4-3 and 4-4, the pavement will experience the full weight of the aircraft.

4-4.3 Traffic Area Designations.

Table 4-1 summarizes the wander width and load condition of the different traffic areas. Details are available in UFC 3-260-02, *Pavement Design for Airfields*.

Table 4-1 Traffic Area Summary

Traffic Area	Load Condition	Distribution	Usage
A	Full weight	Channelized	Runway ends and primary taxiways
B	Full weight	Unchannelized	Aprons
C	75% weight	Unchannelized	Runway interiors and secondary taxiways
D	75% weight	Unchannelized and 1% of passes	Overruns

4-5 STANDARD VERSUS MISSION AIRCRAFT TRAFFIC.

There are three approaches to defining the traffic mix used in an evaluation: standard aircraft traffic groups, mission aircraft traffic groups, and representative/mission aircraft groups. One or more of these approaches may be used for any given evaluation, depending on the Service and mission.

4-5.1 Standard Aircraft Groups.

In this approach, the Service defines a standard mix of aircraft types, weights, and passes for a standard aircraft traffic group based on its mission and operations. They are used for both design and evaluation, although these groups are different for each. Standard groups are used in design when the Service wants to address future uncertainty. For example, it is often difficult to predict future mission changes, aircraft loads and passes, and potential maintenance and repair (M&R) funding constraints over the design life of the pavement. Using a standard aircraft group reduces this risk. The same concept applies to evaluation and has the benefit of better evaluation results comparison between installations. Standard groups may be supplemented with specific aircraft for use in an area of operations or specific mission and often include the same aircraft at different loads.

4-5.1.1 Standard Aircraft Group by Aircraft Effect and Pass Level.

The standard 14-aircraft group in Table 4-2 has aircraft with a similar load effect on the pavement grouped together. This load effect is termed an index and each group has a designated controlling aircraft based on its gear configuration and load. Note that a given group can have more than one gear configuration as shown in Table 4-3. Each group has a minimum weight based on the unloaded weight of the lightest aircraft in the group and a maximum weight based on the fully loaded weight of the heaviest aircraft in the group. This standard aircraft group is used in an individual analysis procedure described in paragraph 4-6. Each of the 14 groups is analyzed at each pass intensity level. The primary benefit of this approach is that it can be used to consider the impact of a wide array of aircraft at different pass intensity levels and can be used to compare the capability of different installations for specific aircraft groups.

Table 4-2 14-Aircraft Group Index Table

Aircraft Group Index

Group	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Included Aircraft	C-12 C-21 C-23 C-38A C-41A HH-60 RC-26 RQ-4-Bk 10 T-1* T-6 T-7A T-37 UH-1H	A-10 AT-38 F-15* F-16 F-22 F-35 F-117 RQ-4-Bk 20+ T-38	CV-580* MH-53 MV-22 CV-22	C-130* C-27J C-295 CN-235	C-20* C-37	B-717* C-9 DC-9 T-43	A-319 A-320 A-321 B-727 B-737 C-22 C-40 MD-81 MD-82 MD-83 MD-87 MD-90 P-3* P-8A	A-300 A-310 B-2A B-707 B-720 B-757 C-32A* DC-8 E-3 E-8C KC-135 RC-135 VC-137	A-330 B-1 B-767 B-787 DC-10-10 KC-46A L-1011 MD-10 B-767 -400ER*	C-17* IL-76	C-5*	A-340 A-350 B-777 DC-10-30 DC-10-40 KC-10 MD-11*	A-380 AN-124 B-747 B-747-8 E-4 VC-25 B-747 -400*	B-52*

Note: * Denotes Controlling Landing Gear Configuration in Group

Pass Intensity Levels (in Passes)

Level	1	2	3	4	5	6	7	8	9	10	11	12	13	14
I		300,000						50,000						15,000
II		50,000						15,000						3,000
III		15,000						3,000						500
IV		3,000						500						100

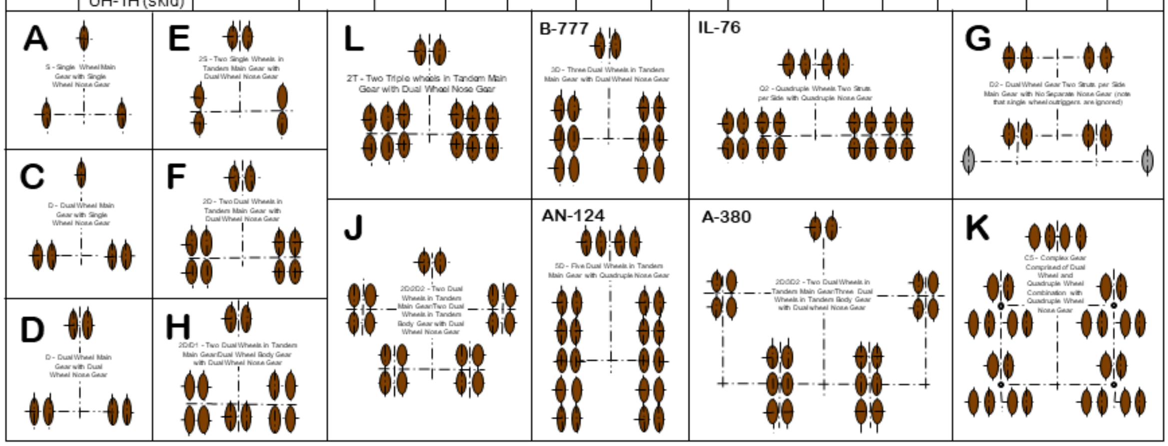
Gross Weight Ranges for Aircraft Groups (in KIPs)

Group	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Lowest Gross Weight	4	8	23	22	39	49	55	110	177	178	374	240	342	230
Highest Gross Weight	27	84	61	175	91	121	210	376	507	585	840	775	1,301	488

Table 4-3 14-Aircraft Group Gear Types

Aircraft Group Index: Gear Types

Group	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Included Aircraft	A C-23 C-41A HH-60 T-1* T-6 T-7A T-37 C C-12 RQ-4-Bk 10 D C-21 C-38A RC-26 UH-1H (skid)	A A-10 AT-38 F-15* F-16 F-22 F-35 F-117 RQ-4-Bk 20+ T-38	D CV-580* MH-53 MV-22 CV-22	E C-130* C-27J C-295 CN-235	D C-20* C-37	D B-717* C-9 DC-9 T-43	D A-319 A-320 A-321 B-727 B-737 C-22 C-40 MD-81 MD-82 MD-83 MD-87 MD-90 P-3* P-8A	F A-300 A-310 B-2A B-707 B-720 B-757 C-32A* DC-8 E-3 E-8C KC-135 RC-135 VC-137	F A-330 B-1 B-767 B-787 DC-10-10 KC-46A L-1011 MD-10 B-767-400ER*	L C-17* IL-76	K C-5*	H A-340 A-350 DC-10-30 DC-10-40 KC-10 MD-11* B-777	J B-747 B-747-8 E-4 VC-25 B-747-400* A-380 AN-124	G B-52*



4-5.1.2 Standard Aircraft Group for Contingency Evaluation.

Table 4-4 shows an example of a standard aircraft group with aircraft that might operate at a forward operating location or en-route airfield. Each Service will have their own pattern, dependent on the mission requirements. The primary objective of this approach is to determine the allowable passes for each aircraft at the defined load. Note that the C-17 at 585,000 pounds is evaluated for 50,000 passes. In this example standard aircraft group, the C-17 is evaluated for 50,000 passes and the resulting allowable gross load (AGL) is used to determine the PCN. This concept is further discussed in Chapter 9.

Table 4-4 Contingency Evaluation Traffic Group

Aircraft	Load (lb)	Passes
C-5	840,000	1,000
E-3A	325,000	1,000
F-15D	68,000	1,000
KC-10	590,000	1,000
KC-135R/T	322,500	1,000
MV-22	60,500	1,000
C-130J	135,000	1,000
C-130J	155,000	1,000
C-130J	175,000	1,000
C-17	450,000	1,000
C-17	500,000	1,000
C-17	585,000	50,000

4-5.1.3 Standard Aircraft Group by Gear Type.

Table 4-5 shows a standard aircraft group categorized by gear types, with a defined representative aircraft for each group. This standard traffic group assumes the maximum load for the aircraft in each group but differs from the previous groups in that there are no predefined pass levels. This group is typically used in conjunction with the mission aircraft group for a specific location using the procedure described in paragraph 4-5.3.

Table 4-5 Aircraft Gear Type Groups

Single Tricycle	Dual Tricycle	Single-Tandem Tricycle	Dual-Tandem Tricycle	Triple Tandem Tricycle
AV-8	C-9	C-130[1]	E-6B	C-17[1]
C-2	C-12		KC-135[1]	
E-2	C-20			
EA-6	C-26			
EA-18	C-37			
F-5	C-38			
F-16	C-40			
F-35B	EP-3			
F-35C[1]	H-3			
FA-18	H-53			

Single Tricycle	Dual Tricycle	Single-Tandem Tricycle	Dual-Tandem Tricycle	Triple Tandem Tricycle
H-60	H-92			
MQ-4C	KC-10			
MQ-25	P-3			
NU-1	P-8[1]			
T-6	V-22			
T-34				
T-38				
T-44				
U-6				
UC-35				
Note 1: Designated representative aircraft for each group.				

4-5.2 Mission Aircraft Groups.

Mission aircraft groups are used for both design and evaluation and are based on the anticipated aircraft traffic at the specific airfield over the design life of the pavement, which is currently 20 years as defined in UFC 3-260-02. Note that different sections on an airfield can have different mission aircraft groups depending on the aircraft that use that specific section. It is not unusual to have two or more mission aircraft groups for any given airfield as shown in the example from a specific airfield in Table 4-6. The primary benefit of a mission aircraft group is that it gives a higher level of fidelity for managing pavements at that specific location but does not serve as well as the standard group when trying to compare evaluation results between installations. Mission aircraft groups are typically used with a mixed traffic analysis.

Table 4-6 Mission Aircraft Group Example

Aircraft	Gross Weight (lb)	20-year Projected Aircraft Passes	20-Year Equivalent PCC Passes	20-Year Equivalent AC Passes
Fixed-Wing Pavements				
C-9A	108,000	600	49	1
C-12J	16,600	1,100	1	1
C-130H	155,000	10,000	42	52
C-17A	585,000	6,300	6,300	6,300
C-23	24,600	800	1	1
C-26	16,500	340	1	1
Equivalent C-17 Passes at 585,000 lb			9,436	6,356
Rotary-Wing Pavements				
UH-60	16,300	6,700	283	3,954
AH-64	18,000	4,720	160	1,929
CH-47	50,000	4,820	4,820	4,820
MH-60	16,300	1,340	57	791
Equivalent CH-47 Passes at 50,000 lb			5,321	11,494
Fixed-Wing and Rotary-Wing Pavements				
CH-47	50,000	9,600	1	1
C-17	585,000	2,000	2,000	2,000
AH-64	18,000	19,000	1	1
C-130H	155,000	800	5	9
Equivalent C-17 Passes at 585,000 lb			2,475	2,011

4-5.3 Mission/Representative Aircraft Group.

The goal of this traffic approach is to determine the equivalent passes of the mission aircraft group in terms of each of the five representative aircraft gear-type groups listed in Table 4-5. The first step in achieving this goal is defining the aircraft and pass levels in the mission aircraft group for the specific location using the procedure in paragraph 4-5.2, then append this traffic mix with each representative aircraft from Table 4-5 that is not already included in the mission aircraft group, with each aircraft at full load and one pass. The next step is to use a mixed traffic analysis to determine the controlling aircraft in the group. Next, manually set the first representative aircraft gear type as the controlling aircraft and determine the equivalent passes. Repeat the process for each representative aircraft shown in Table-4-5.

This approach results in equivalent passes for each of the five representative aircraft. The equivalent passes will vary by section depending on the pavement type, subgrade category, and traffic area as described in paragraph 4-6.2. The load and equivalent passes for each of these patterns are used in the structural analysis procedure for each section. The intent of this approach is to get the fidelity of the mission traffic approach while facilitating comparison between installations. There may be specific instances where aircraft not represented in Table 4-5 would also be presented this way and there may also be sections or sites whose missions support aircraft significantly lighter than the representative aircraft. In these instances, only add the appropriate representative aircraft (up to three) that do not overload the pavement.

4-6 INDIVIDUAL VERSUS MIXED TRAFFIC ANALYSIS.

Both standard and mission traffic patterns can be used in an individual or mixed traffic analysis although a mixed traffic analysis is typically used for a mission traffic pattern. Individual or mixed traffic analysis can be used for either a conventional (APE) or layered elastic (LEEP) analysis.

4-6.1 Individual Traffic Analysis.

In an individual traffic analysis, each aircraft in the group is analyzed individually. The allowable passes for the specified aircraft load and the allowable load for the specified evaluation passes are computed irrespective of the other aircraft.

4-6.2 Mixed Traffic Analysis

In a mixed traffic analysis, the controlling aircraft is determined based on the pavement type (rigid or flexible), traffic area (defined above), subgrade category (see Table 4-7), and number of passes. The equivalent passes of each aircraft in the mix are determined in terms of the controlling aircraft and the equivalent passes for all aircraft are added together. The result is a controlling aircraft at a specified weight and number of equivalent passes that is used in analysis.

Table 4-7 Subgrade Category

Subgrade Category	Rating	Flexible (CBR %)	Representative CBR	Rigid (k pci)	Representative k
A	High	CBR ≥ 13	15	k ≥ 442	452.6
B	Medium	8 < CBR < 13	10	221 < k < 442	294.7
C	Low	4 < CBR ≤ 8	6	92 < k ≤ 221	147.4
D	Ultra Low	CBR ≤ 4	3	k ≤ 92	73.7

4-7 GEAR CONFIGURATIONS.

Early aircraft were primarily supported on two main landing gear wheels, referred to as “single” wheels. With the large increases in aircraft gross weights, landing gear have changed to twin (two per strut) wheel loadings, to twin-tandem (four wheel) loadings, and to more complex (16 and 24 main-gear wheels, extra “belly” gear) wheel support systems. The two main wheels of single-wheel aircraft are generally spaced far enough apart that there is no significant overlap of the distributed loads for even very thick pavement structures protecting weak subgrades. For twin wheels, however, and closely spaced tandem wheels or complex wheel groups, the patterns of distributed surface loadings at and near the bottom of pavement structures overlap so the intensities (pressures or stresses) combine between adjacent wheels. This combining effect of load intensities is greater as the adjacent wheels become closer. The aircraft gear configurations and nomenclature used by the Services are shown in Appendix C.

4-8 TIRE PRESSURE.

The intensity of stress at a given point in a flexible pavement is affected by the tire contact pressure, which, for large aircraft tires, is roughly equivalent to the inflation pressure. The major difference in stress intensities caused by variation in tire pressure occurs near the surface; consequently, the pavement surface and upper base-course layers are most seriously affected by high tire pressures. Current evaluation criteria outlined in this UFC and implemented in PCASE are based on constant tire pressure. Previous versions of UFC 3-260-03 had criteria based on constant contact area. This difference does result in changes to evaluation results.

4-9 MANAGING AIRCAFT TRAFFIC.

The goal in defining the anticipated load and passes for the aircraft in a traffic mix is to determine whether each pavement section can structurally support the traffic to accomplish the mission, typically for a defined period. When the evaluation determines the pavement is not structurally capable, there are several options for managing the traffic:

- Reducing the departure weights of one or more aircraft in the traffic mix
- Reducing the number of daily operations of some aircraft in the mix
- Decreasing the pavement service life and programming repairs

The first two options typically focus on large, heavy aircraft that can generate unacceptable amounts of structural damage. Structural damage is often sensitive to changes (5 percent or less) in the aircraft gross loads for heavier aircraft, so it is often more advantageous to restrict the operations of one to three heavy aircraft that typically cause 90 percent of the pavement fatigue damage rather than limiting day-to-day operations. Whether the focus is on using up the service life and performing timely repairs for each inadequate section or managing the traffic as in the first two options, color-coded structural and condition maps convey this information. Chapter 10 describes these report products in more detail.

CHAPTER 5 LAYERED ELASTIC PAVEMENT EVALUATION

5-1 PERFORMANCE CRITERIA.

5-1.1 Flexible Pavement Performance Criteria.

The flexible pavement structural evaluation procedure considers two performance criteria: cracking in the asphalt surface course by limiting values of the tensile strain at the bottom of the AC layer and rutting due to deformation in the subgrade by limiting values of the vertical strain at the top of the subgrade. The limiting performance criterion is typically the vertical strain at the top of the subgrade. There are cases where the tensile strain controls the allowable number of passes or allowable gross load (AGL) for thin asphalt surfaces (e.g., less than 3 inches [76 millimeters]), but this scenario could also exist for thicker AC layers when there are no bases or subbases, or these layers are weak.

5-1.2 Rigid Pavements Performance Criteria.

Performance criteria for rigid pavements are based on limiting the tensile stress in the PCC slabs such that failure occurs only after the pavement with has sustained many load repetitions. Failure is based on a SCI of 50 or 0, as discussed in paragraphs 3-3.4.2 and 5-3.9.1.3.

5-2 PAVEMENT RESPONSE MODEL.

The YULEA linear elastic modeling subroutine computes the pavement responses that implement the performance criteria in the Pavement-Transportation Computer Assisted Structural Engineering (PCASE) Layered Elastic Evaluation Program (LEEP) module. The following assumptions apply in YULEA.

5-2.1 Pavement Structure.

Pavement is a multilayered structure, with each layer characterized by its thickness, modulus of elasticity, and Poisson's ratio. Layers are assumed to be homogeneous, isotropic, and extend infinitely in the horizontal direction.

5-2.2 Layer Interface.

The interface between layers is continuous, meaning the friction resistance between layers is greater than the developed shear force.

5-2.3 Bedrock Layer.

The bedrock layer is located 20 feet (6 meters) from the surface and is of infinite thickness. When geotechnical information indicates the depth to the bedrock layer is less than 20 feet (6 meters), adjust the depth to bedrock to the known depth.

5-2.4 Loads.

All loads are static, circular, and uniform over the contact area.

5-3 LAYERED ELASTIC EVALUATION PROCEDURE.

The layered elastic evaluation procedure is based on a layered linear elastic model that characterizes multilayer pavement systems as outlined above. It applies to flexible, plain concrete, plain concrete overlays, and non-rigid overlays on plain concrete pavements. Layered elastic criteria are not currently available for reinforced or fibrous pavements. Refer to Chapter 7 for methods to evaluate reinforced pavements. It uses layer properties determined from in situ measurements (at the time fieldwork is conducted) to compute allowable loads for a selected number of aircraft passes, allowable passes at a specified load, and the Pavement Classification Number (PCN). When the pavement structure cannot support the defined pass level and aircraft load, PCASE can determine overlay requirements to strengthen the pavement. More detailed information on the following procedure is available in the PCASE User Manual.

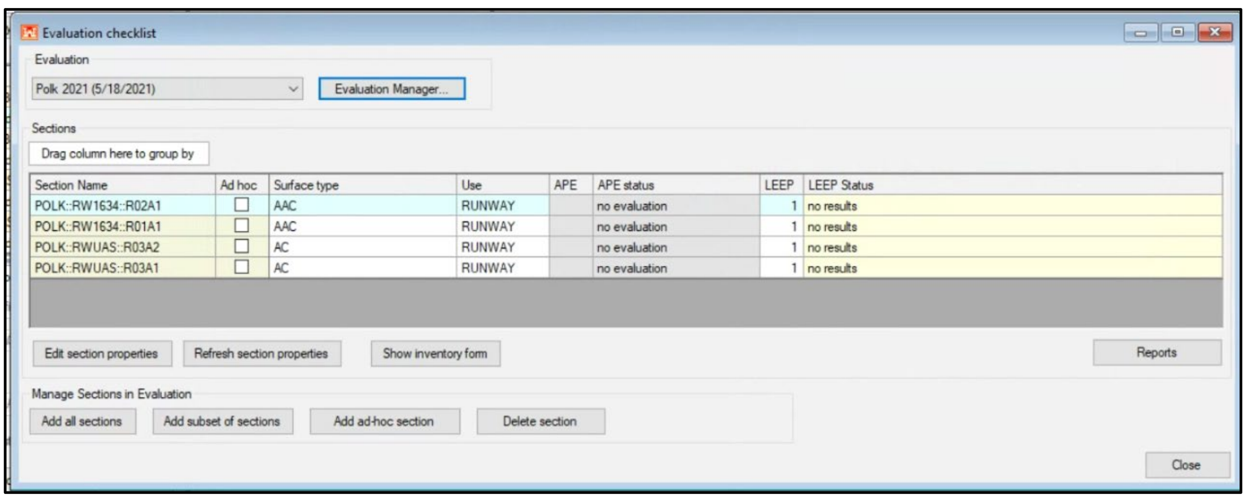
5-3.1 Layered Elastic Evaluation Using PCASE.

The Services use the PCASE application for design and evaluation of pavements (see Appendix E). Use the PCASE LEEP module to compute allowable loads, allowable passes, and PCNs using layered linear elastic evaluation criteria.

5-3.2 Step 1 – Create a New Evaluation.

Open the PCASE Evaluation Checklist to create a new evaluation using the Evaluation Manager. Define the Service, climate data, evaluation traffic, and rigid failure criteria for the evaluation, then assign the inventory sections to be included in the evaluation.

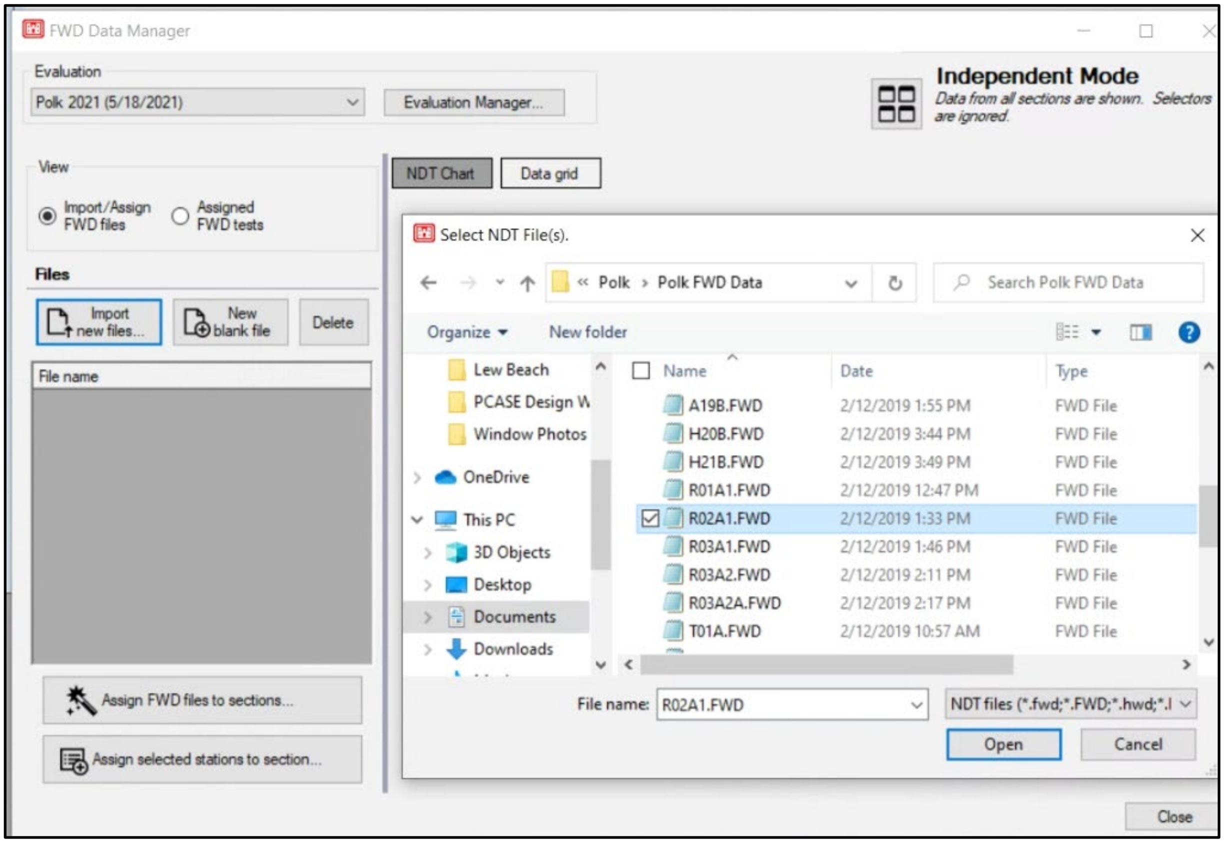
Figure 5-1 Evaluation Checklist



5-3.3 Step 2 – Import HWD Test Data.

Use the FWD Module to import the NDT files created during FWD testing (as described in Chapter 3) into PCASE.

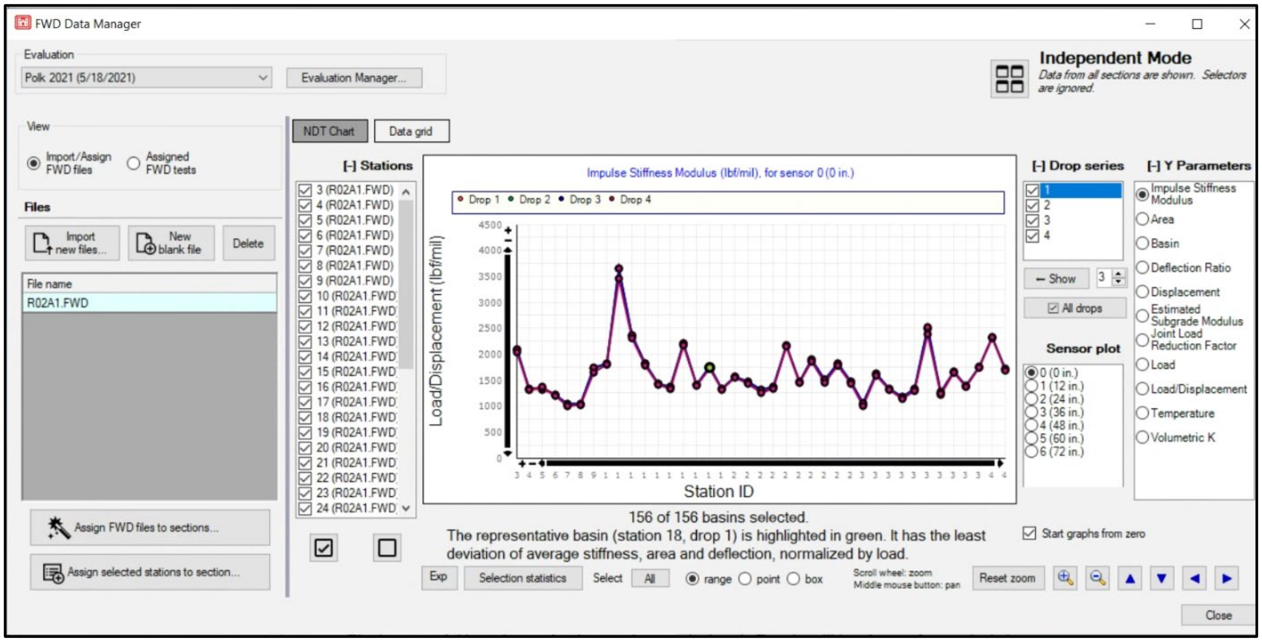
Figure 5-2 FWD Data Import



5-3.4 Step 3 - Assign Basins to Sections.

Each FWD test (each drop at each station) defines a deflection basin, which is viewed in PCASE as a two-dimensional plot, as shown in Figure 5-3. When the HWD file has data for an entire branch (e.g., an entire runway), use the FWD tool to “Assign selected stations to sections.” When all the stations in an HWD file were collected for a specific section, use the “Assign FWD files to sections” option.

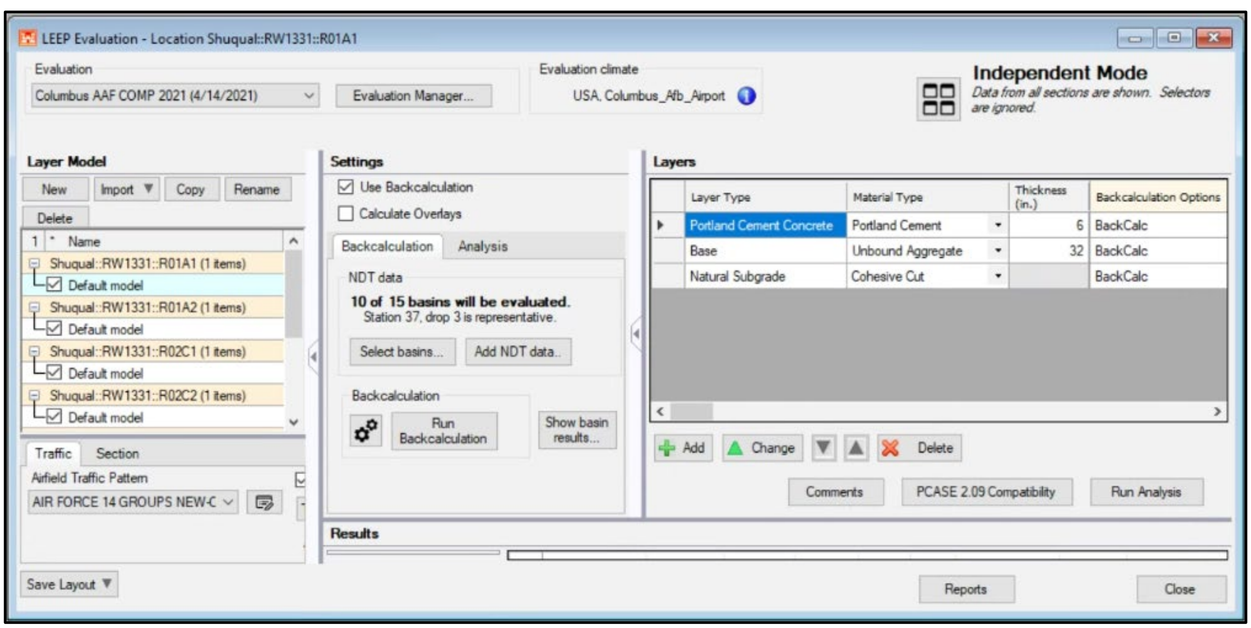
Figure 5-3 Assign FWD Files to Sections



5-3.5 Step 4 – Select Basins for Backcalculation.

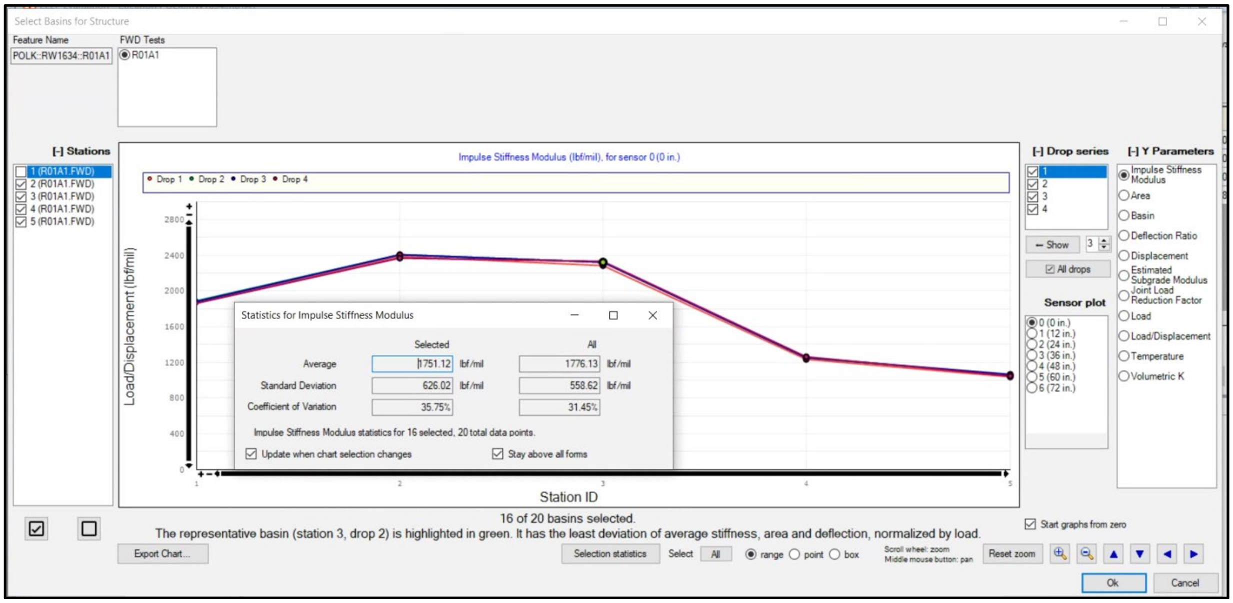
Open the LEEP module and use the Select Basins tool on the Settings tab to define the basins used in backcalculation.

Figure 5-4 Select Basins for Backcalculation



The Select Basins tool is like the NDT tool but only displays the basins for the section that is the current focus. Use the Selection Statistics tool to determine which basins to include or exclude from backcalculation. This tool displays statistics on the impulse stiffness modulus by default. Other views of the basin data are available including Area, Basin, Displacement, Estimated Subgrade Modulus, Load, Load Displacement, Temperature, and Volumetric K, Deflection Ratio, and Joint Load Reduction Factor (testing rigid pavement joints) as shown in Figure 5-5 and described in the following paragraphs.

Figure 5-5 FWD Analysis Parameters



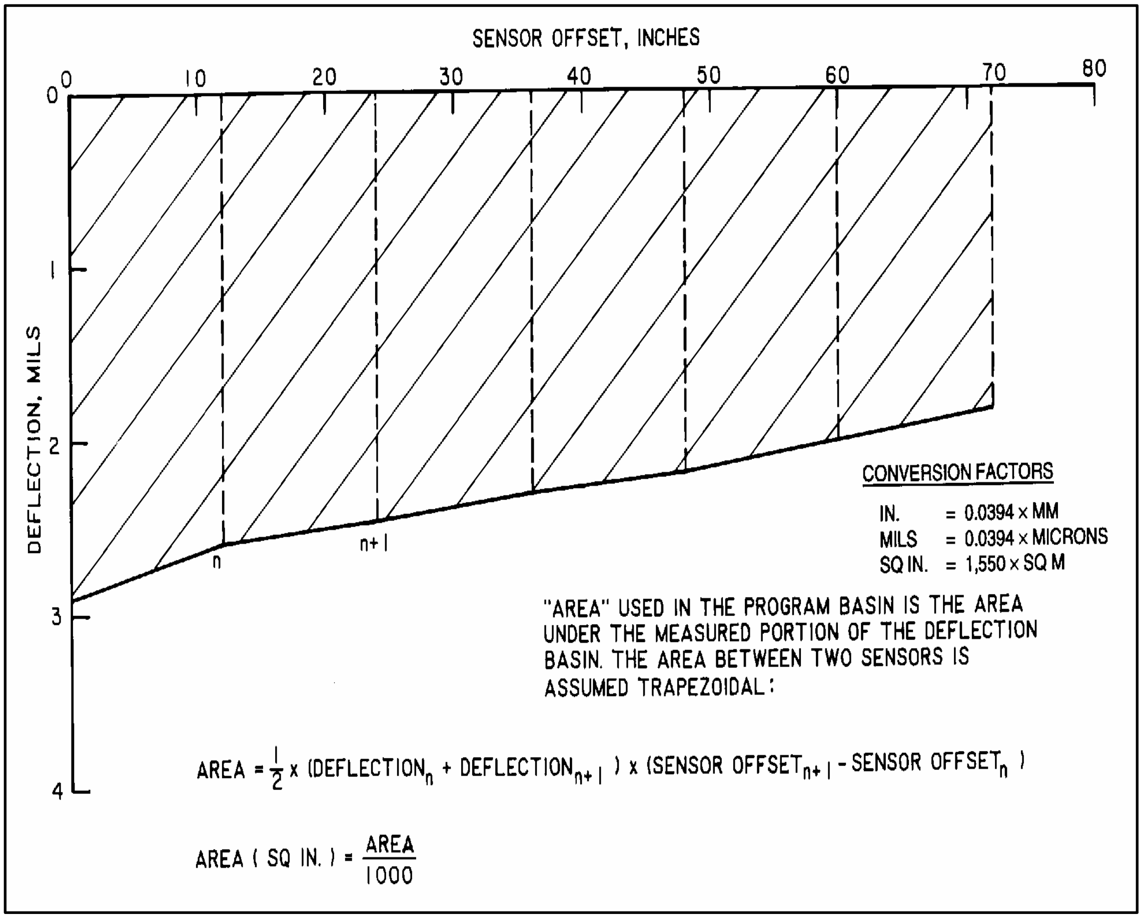
5-3.5.1 Impulse Stiffness Modulus.

The impulse stiffness modulus (ISM) is defined as the FWD force or load in kips divided by the deflection measured at the center of the load in inches. ISM values computed for the load-plate sensor (geophone) represent the overall strength of the pavement structure. These ISMs provide a quantitative stiffness comparison between test points and between pavement sections. The ISM values are plotted on the Y axis for each station (test point) in the section. This data is used to visually determine if a change in strength exists and define where sections change when the FWD file has basin data from multiple sections. Even when a pavement section has the same pavement type and construction, the ISMs measured in one area of the section can be statistically different from those in another area of the section. In this case, consider splitting the section. Ideally the Coefficient of Variation of the selected basins within a section should be less than 20 percent. PCASE also displays ISM values for the other sensors that can be used to compare the relative strength of the base, subbase, or subgrade at each NDT location.

5-3.5.2 Basin Area.

The AREA parameter displays the area of each deflection basin determined using the procedure illustrated in Figure 5-6. Only the hatched area (under the measured portion of the basin) is considered in this computation, and the area between two sensors is assumed trapezoidal. Selection Statistics for AREA displays the average deflection basin area for the section.

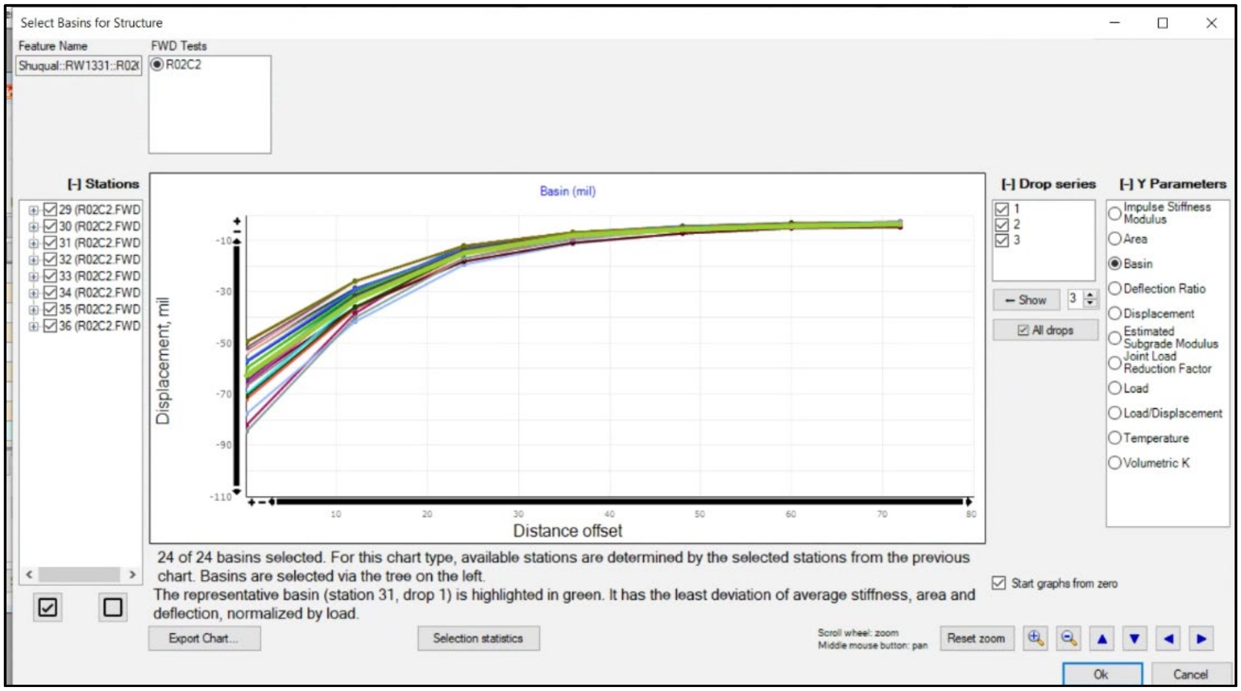
Figure 5-6 Determining AREA Beneath Deflection Basin



5-3.5.3 Basin.

The basin plot displays the deflection (in mils) on the Y axis for each sensor at its respective offset distance on the X axis. The plot provides a visual indication of the quality of the basin data. When the lines in the plot are well organized, as in the Figure 5-7 example, the data is likely good. When there are discontinuities in the data, such as varying basin shapes and crossing lines, the quality of the data is questionable.

Figure 5-7 Basins



5-3.5.4 Displacement.

Displacement plots show the sensor deflection (in mils) for each drop in the test series on the Y axis for each station on the X axis. Like the basin plot, this plot provides a visual indication of the quality of the data. Discontinuities such as lines crossing can indicate anomalies such as voids or delamination or indicate data quality issues.

5-3.5.5 Estimated Subgrade Modulus.

The estimated subgrade modulus is displayed on the Y axis for each drop in the series for each station in the section. The estimate is computed in Equation 5-1 using the deflection measured at the 72-inch (1829-millimeter) offset. These values are also used as the seed moduli for the subgrade layer in the backcalculation procedure.

Equation 5-1. Estimated Subgrade Modulus

$$E = 59{,}304.82\,(D72)^{-0.98737}$$

Where:
E = *subgrade modulus, psi*
D72 = *deflection measured at 72 inches (1829 millimeters) from the NDT load normalized to 25,000 pounds (11,340 kilograms)*

5-3.5.6 Load.

Load plots show the load (in lbf or kN/µm) for each drop in the test series on the Y axis for each station on the X axis. This plot provides a visual indication of the quality of the data. Discontinuities such as lines crossing can indicate data quality issues.

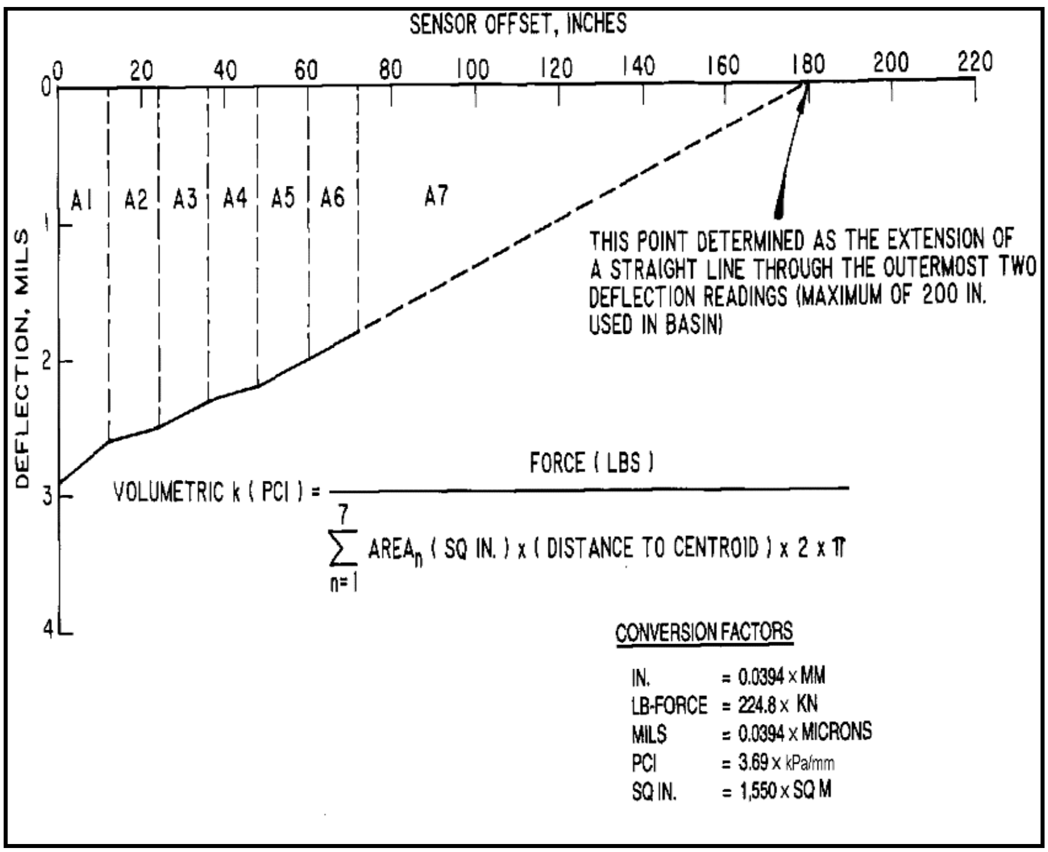
5-3.5.7 Temperature.

Temperature plots show the temperature (in °F or °C) on the Y axis for each drop in the test series for each station on the X axis. The air and the surface temperature are captured at the time of testing and the pavement temperature (at depth) is calculated.

5-3.5.8 Volumetric Estimation of k Value.

This procedure estimates the modulus of subgrade reaction, k, beneath rigid pavement, or rigid pavement with a flexible overlay. It computes the volume of the deflection bowl as illustrated in Figure 5-8. The k value obtained in this manner is only an approximate value that can be used for comparison with results from other test procedures such as plate bearing or dynamic cone penetrometer (DCP) tests used to determine k values. When no other test data to determine k is available, use the volumetric k in an airfield pavement evaluation (APE) analysis for comparison with the layered elastic analysis results. Note that volumetric k values are not typically sufficiently accurate to compute allowable aircraft loads and PCN values.

Figure 5-8 Determining Volumetric k (Estimate of Modulus of Subgrade Reaction)



5-3.5.9 Deflection Ratio.

The layered elastic rigid pavement analysis procedure assumes a 25 percent load transfer by default. Validate this assumption by testing joints with the FWD as described in more detail later in this chapter. The result of this test is used to compute the joint deflection ratio and, when results indicate, reduce the percent of load transfer. PCASE can also compute the deflection ratio between other sensors, but this capability is not typically used at this time.

5-3.5.10 Joint Load Reduction Factor.

The deflection ratio is used to determine and, when appropriate, adjust the load transfer percentage between slabs for the analysis. The Joint Load Reduction Factor is equal to one whenever the joint deflection ratio is greater than or equal to 0.76.

5-3.6 Step 5 – Layer Model and Backcalculation Options.

LEEP populates a default layer model for each section based on the pavement type, but the user must update the layer structure as shown in Figure 5-9 based on the data collected during fieldwork, including the type and thickness of each layer and the flexural strength (for PCC). In addition, the user can select different backcalculation options and edit the seed moduli used to initiate the backcalculation procedure as well as the lower (min) and upper (max) limits used in the procedure. Following are the backcalculation options for the various layer types.

Figure 5-9 Layer Model for Backcalculation

Layers

Layer Type	Material Type	Thickness (in.)	Backcalculation Options	Seed Modulus (psi)	Min Modulus (psi)	Max Modulus (psi)	Apply Limit
Portland Cement Concrete	Portland Cement	6	Flexural Strength	5,000,000	2,500,000	10,000,000	☑
Base	Unbound Aggregate	32	BackCalc	60,000	5,000	150,000	☑
Natural Subgrade	Cohesive Cut		BackCalc	9,132	4,132	14,132	☑

5-3.6.1 Backcalculation Option.

The Backcalculation option can be selected for any layer type. It uses estimated initial modulus values, a minimum, and a maximum modulus that are set for each layer but the number of backcalculated layers cannot exceed the number of measured deflections. Table 5-1 provides an example of typical default values used in PCASE that can be edited when test data is available. When the Apply Limit box is checked, the backcalculation routine keeps the solution within the limits and when it is unchecked, the backcalculation routine is not restricted by the limits for that layer.

Table 5-1 WESDEF Default Modulus Values (psi)

Material	Range		Initial Estimate	Poisson's Ratio
	Minimum	Maximum		
Asphalt concrete	100,000	2,500,000	350,000	0.35
Portland cement concrete	2,500,000	10,000,000	4,000,000	0.15
High-quality stabilized base	500,000	2,500,000	1,000,000	0.20
Base-subbase, stabilized	100,000	1,000,000	650,000	0.25
Base-subbase, unstabilized	5,000	150,000	61,000	0.35
Subgrade	1,000	75,000	15,000	0.40

5-3.6.2 Subgrade Seed Modulus.

The seed modulus for the subgrade is determined differently than other layer types. It is estimated using the deflection measured at the 72-inch (1829-millimeter) offset from the load using Equation 5-1. The maximum and minimum moduli are set to ±5,000 psi (34 MPa) respectively. This relationship is not valid when bedrock is present near the pavement surface (< 20 feet [6 meters]). In this case use the depth to bedrock estimation tool (for asphalt pavements) or other geotechnical information to adjust the depth to bedrock and determine a reasonable subgrade seed modulus.

5-3.6.3 Flexural Strength Option.

The Flexural Strength option uses Equations 5-2 and 5-3 to estimate the modulus value based on the flexural strength of the pavement.

Equation 5-2. Compressive Strength

$$C = 0.4036 * {M_R}^{1.4281}$$

Equation 5-3. PCC Layer Modulus

$$E = 57{,}000 * C^{0.5}$$

Where:

C = *Compressive strength, psi*

M_R = *Flexural strength, psi*

E = *Modulus of elasticity, psi*

5-3.6.4 Backcalculation Temperature Option.

This option adds the surface temperature at the time of testing to the previous five-day mean air temperature to determine the pavement temperature at depth as shown in Figure 5-10. Use this mean calculated mean pavement temperature to estimate the AC modulus using the relationship in Figure 5-11. The FWD or HWD device normally produces a load frequency at or near 20 Hz. The curves in Figure 5-11 are extrapolated from laboratory relationships for new AC mixes; therefore, predicted values may not always agree with actual field values.

Figure 5-10 Determining Mean Pavement Temperature

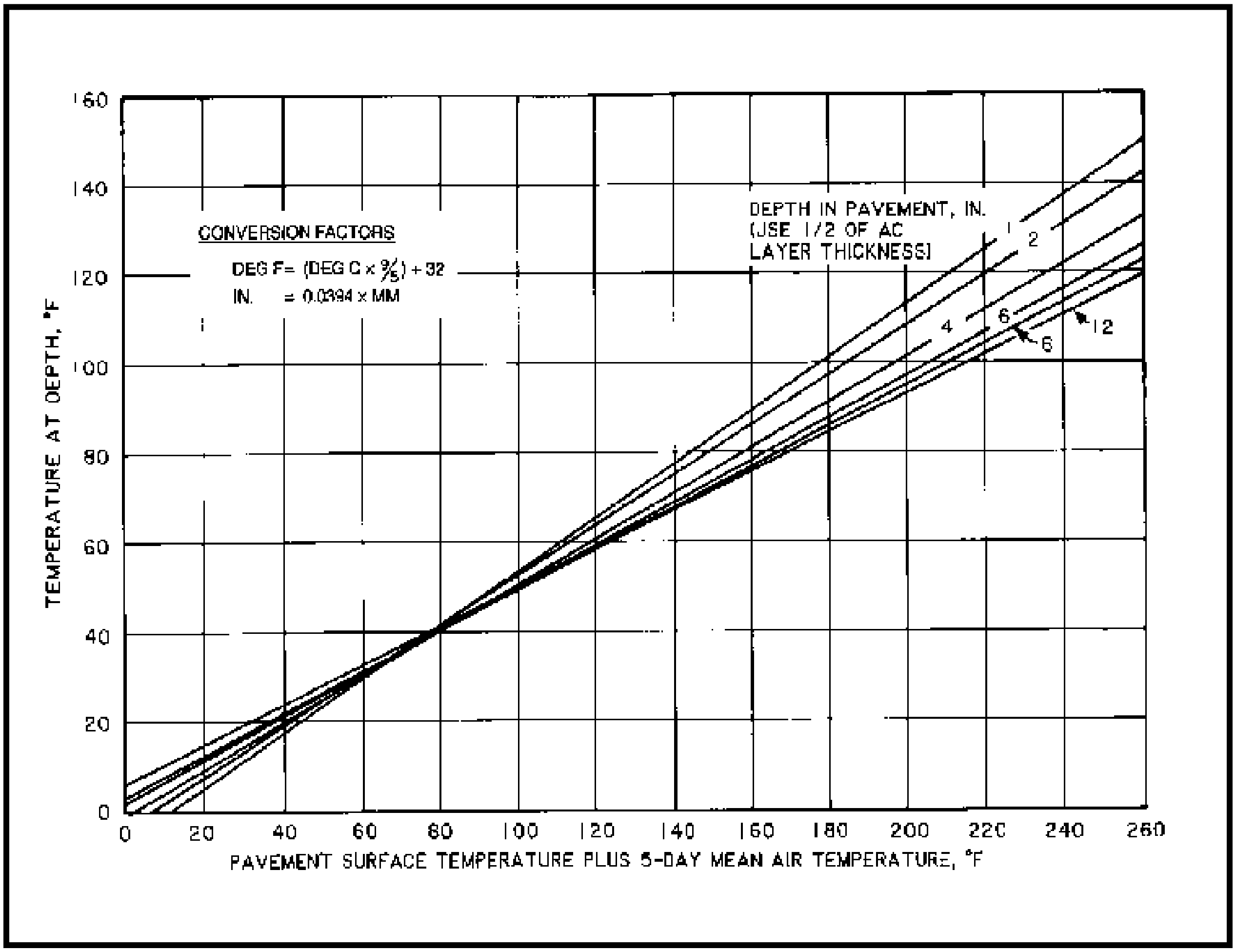
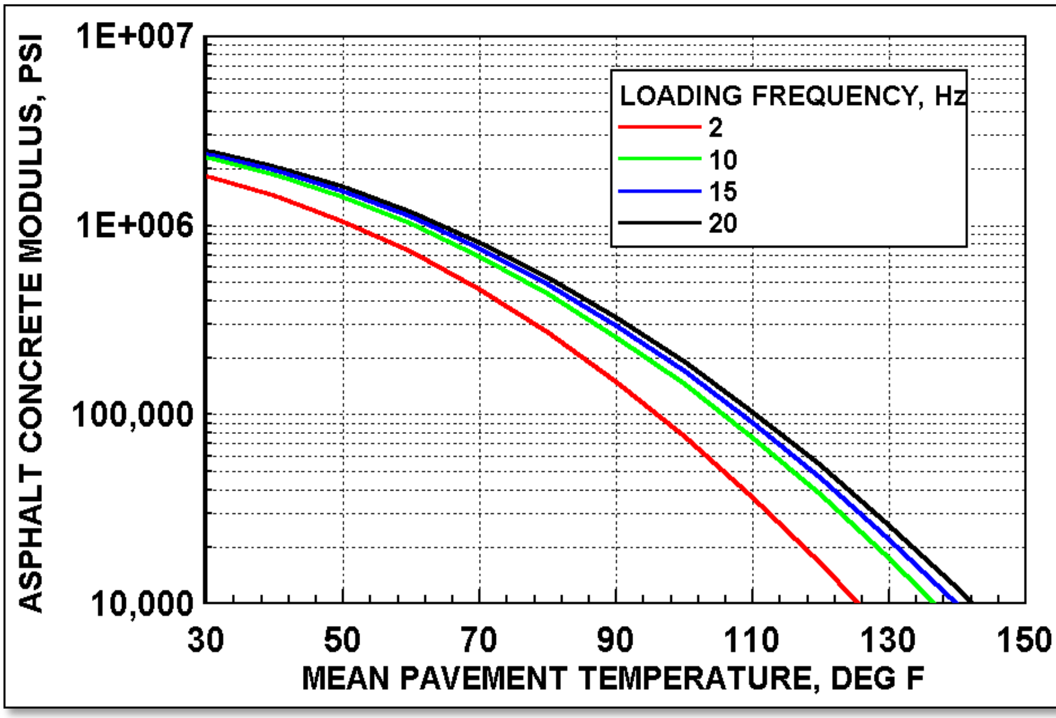


Figure 5-11 Predicting AC Modulus for Asphalt Layers



5-3.6.5 En+1 Option.

The E_{n+1} option uses the modulus of the layer below to estimate the base or subbase modulus using Equation 5-4 for base course and Equation 5-5 for subbase layers:

Equation 5-4. Base Course

$$E_{n+1} = E_n * (1.0 + 10.52 \log t - 2.1 \log E_n * \log t)$$

Where:

E_{n+1} = Modulus of base layer with a maximum value of 100,000 psi

E_n = Modulus of subbase or subgrade layer

t = Thickness of base layer

Equation 5-5. Subbase Layer

$$E_{n+1} = E_n * (1.0 + 7.18 \log t - 1.56 \log E_n * \log t)$$

Where:

E_{n+1} = Modulus of subbase layer with a maximum value of 40,000 psi

E_n = Modulus of subgrade layer

t = Thickness of subbase layer

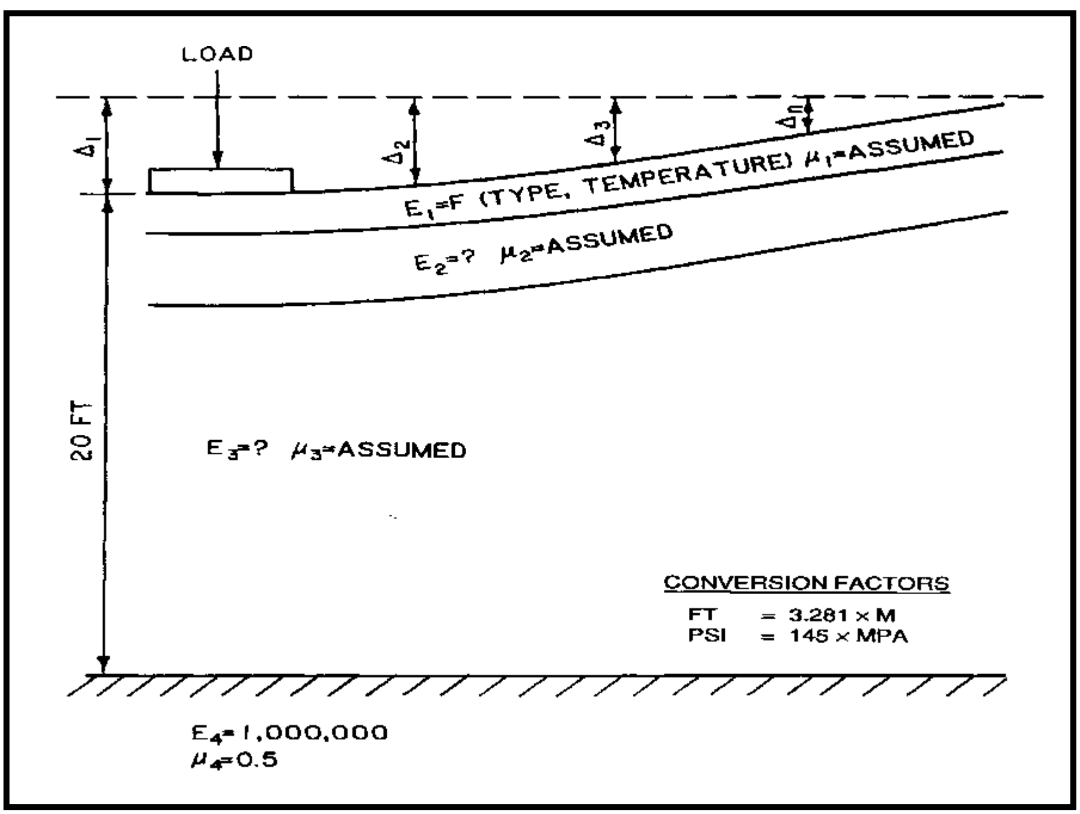
5-3.6.6 Manual Option.

Typically, the modulus of any layer can be backcalculated; however, when backcalculated results are erratic, assigning a modulus value to a base or subbase layer based on its material type or other tests (e.g., DCP) can resolve backcalculation issues. In general, use the backcalculation, flexural strength, or temperature options for surface layers. If the results are reasonable for all unbound layers but not surface layers, adjust the surface layer modulus in analysis rather than in backcalculation. Modulus values developed from the portable seismic pavement analyzer (PSPA) are also used for the surface layer modulus in analysis when this testing is performed.

5-3.7 Step 6 – Backcalculate Layer Modulus Values.

The deflection basin produced by applying a load to the pavement with an NDT device gives input parameters to the system analysis that are used to derive the relative strength parameters of the pavement layers. To determine modulus values, model the pavement structure as a layered system like that illustrated in Figure 5-12. PCASE uses the YULEA module to determine a set of modulus values that provides the best fit between a measured and a computed deflection basin when given an initial estimate of the elastic modulus values, a range of modulus values, and a set of measured deflections. The following paragraphs summarize the layered elastic modulus backcalculation routine.

Figure 5-12 Layered Pavement Structure



5-3.7.1 Backcalculation Objective.

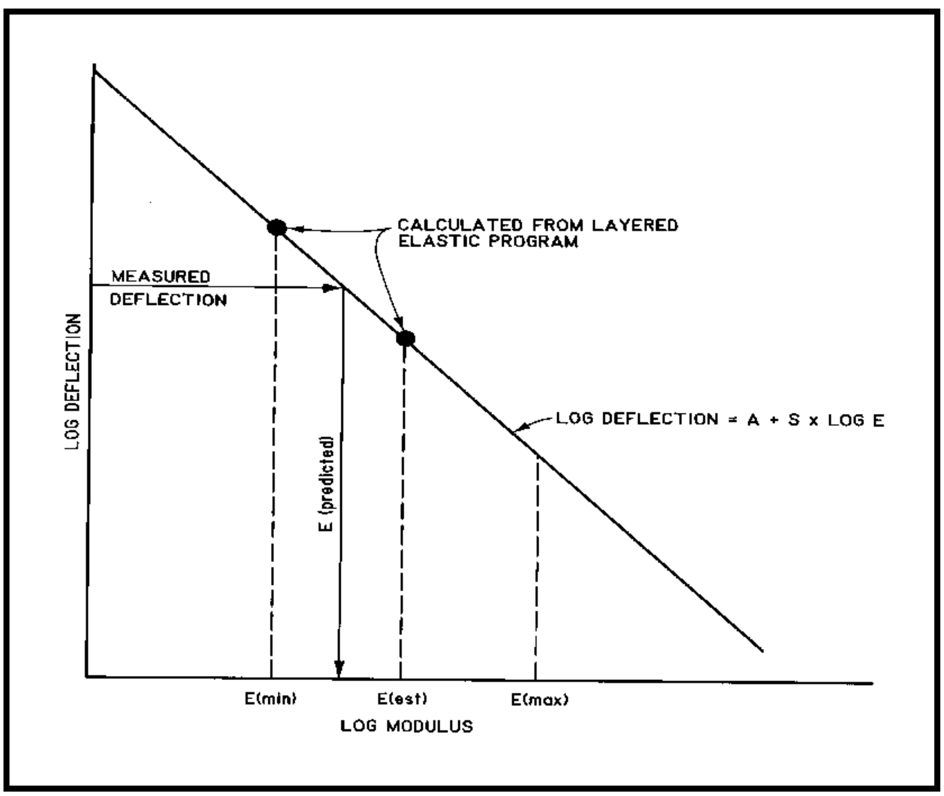
Consider the pavement system where:

- The modulus is unknown for a number of layers (NL).
- The deflection due to an NDT load is measured at a number of deflection sensors (ND).
- The number of deflection sensors (ND) is greater than the number of layers (NL).
- The objective is to determine the set of elastic moduli values that minimizes the error between the computed deflection (CD) and the measured deflection (MD).

5-3.7.2 Elastic Modulus Backcalculation from NDT Data.

Assume a set of E values and compute the deflection at the sensor location corresponding to the measured deflection. Vary each unknown E individually and compute a new set of deflections for each variation. Figure 5-13 presents a simplified description of how the deflection basins are matched. This illustration is for one deflection and one layer. For multiple deflections and layers, obtain the solution by developing a set of equations that defines the slope and intercept for each deflection and each unknown layer modulus using Equation 5-6.

Figure 5-13 Simplified Description of Matching Deflection Basins in YULEA (One Deflection and One Layer)



Equation 5-6. Backcalculated Layer Modulus

$$Deflection_j = A_{ji} + S_{ji}\left(\log E_i\right)$$

Where:
A = *intercept*
S = *slope*
j = *1 to the number of deflections*
i = *1 to the number of layers with unknown modulus values*

5-3.7.3 Depth to Bedrock Estimation.

PCASE assumes a stiff layer having a modulus of elasticity of 1,000,000 psi (6,895 MPa) and Poisson's ratio of 0.5 below the subgrade layer. This stiff layer defaults to a 20-foot (6-meter) depth and is infinitely thick. When modulus values for the subgrade seem excessively high for the material type, adjust the depth to bedrock using geotechnical information such as boring logs or the PCASE Depth to Bedrock tool for asphalt pavements that uses Equations 5-7 through 5-10 to estimate the depth of bedrock for each station and then uses equation 5-11 to determine the average depth to bedrock (see Report No. FHWA/TX-91/1123-3, *Modulus 4.0: Expansion and Validation of the Modulus Backcalculation System*)

Equation 5-7. Depth to Bedrock, Asphalt Thickness < 2 in.

$$\frac{1}{B} = 0.0362 - 0.3242r_0 + 10.2717r_0^2 - 23.6609r_0^3 - 0.0037BCI$$

Equation 5-8. Depth to Bedrock, Asphalt Thickness > 2, ≤ 4 in.

$$\frac{1}{B} = 0.065 + 0.1652r_0 + 5.42898r_0^2 - 11.0026r_0^3 - 0.0004BDI$$

Equation 5-9. Depth to Bedrock, Asphalt Thickness > 4, ≤ 6 in.

$$\frac{1}{B} = 0.0413 + 0.9929r_0 - 0.0012SCI + 20.0063BDI - 0.0778\log(BCI)$$

Equation 5-10. Depth to Bedrock, Asphalt Thickness > 6,

$$\frac{1}{B} = 0.0409 + 0.5669r_0 + 3.0137r_0^2 + 0.0033BDI - 0.0665\log(BCI)$$

Where:
$r_0 = {}^1/_r$ $intercept\ by\ extrapolating\ the\ steepest\ section\ of\ the$
${}^1/_r\ vs.deflection\ curve\ \left({}^1/_{ft.}\ units\right)$
$SCI = D_0 - D_1\ (Surface\ Curvature\ Index)$
$BDI = D_1 - D_2\ (Base\ Damage\ Index)$
$BCI = D_2 - D_3\ (Base\ Curvature\ Index)$
$D_i = Surface\ deflection\ (inches\ 10^{-3})\ normalized\ to\ 9{,}000\ lb.load$
$at\ an\ offset\ i\ in\ feet.$

Equation 5-11. Average Depth to Bedrock

$$D = \left[\frac{n}{\sum_{i=1}^{n} \frac{1}{B_i}}\right]$$

Where:
D = *Average depth to an apparent rigid layer in feet*
B_i = *Depth to the apparent rigid layer for the ith deflection bowl*
n = *Number of deflection bowls within one standard deviation of the mean* $1/B_i$

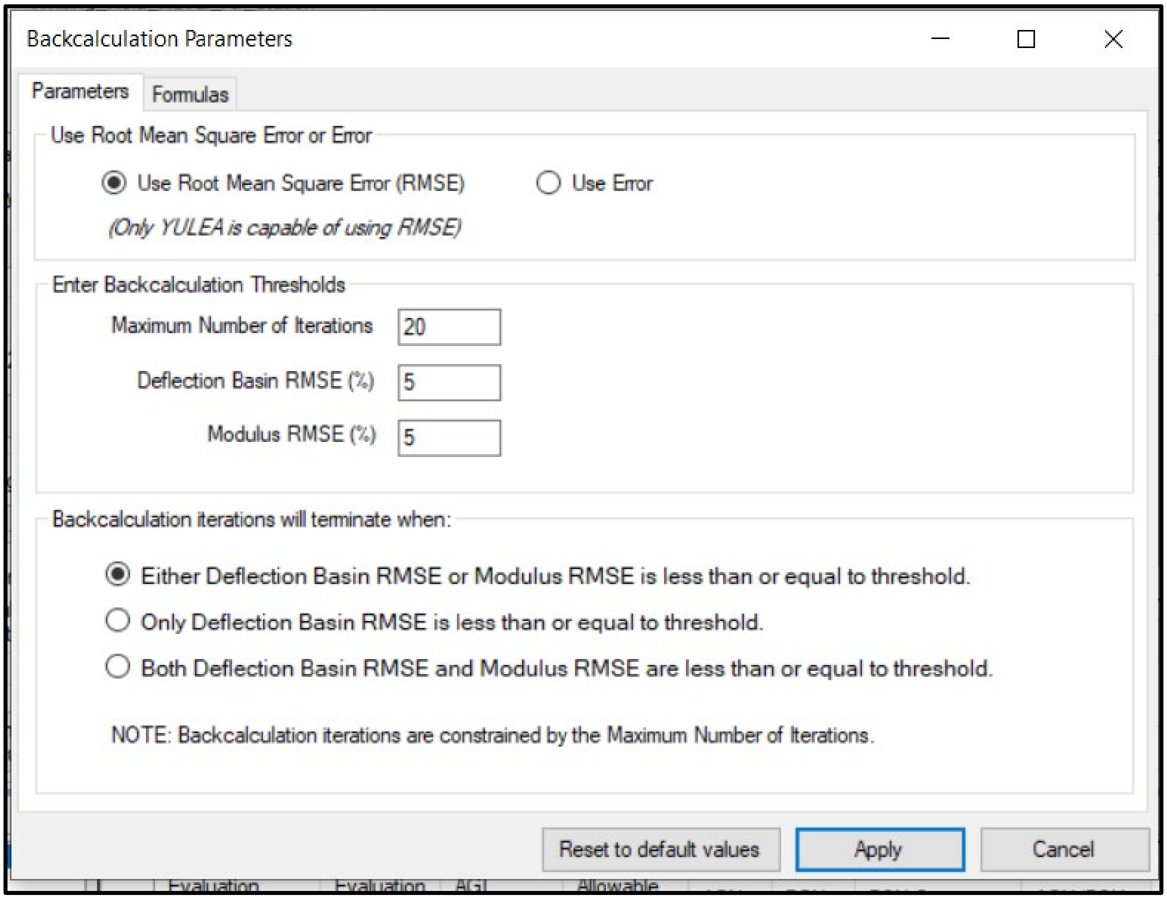
5-3.7.4 Layered Elastic Interface Conditions.

YULEA can accommodate multiple loads and variable interface conditions. For a given layer (n) and underlying layer (n + 1), set the interface value to “Fully Bonded” for complete adhesion between the layers or “Partially Bonded” for almost frictionless bond between the layers. The procedure assumes a partially bonded condition at the bottom of a PCC layer and a fully bonded interface condition for all other layers.

5-3.7.5 Backcalculation Procedure Closure.

PCASE allows the user to define the backcalculation procedure closure parameters. The user can choose whether the to use the Error (historically used by DoD) or Root Mean Square Error (RMSE) which is more commonly used in industry. The user can define the maximum number of iterations and the closure parameters, including the percent error (or RMSE) for the deflection basin (Equation 5-12) and the percent error (or RMSE) for the modulus (Equation 5-13). There are also options for defining the backcalculation termination parameters, including when both the basin and modulus error (or RMSE) are less than or equal to the thresholds, only the basin error (or RMSE) is less than or equal to the threshold, or either the basin or modulus error (or RMSE) is less than or equal to the threshold. The latter is the default setting. The maximum iterations defaults to 20, and both the basin and modulus error defaults to five percent as shown in Figure 5-14. When the backcalculation results meet the parameters for each basin, the procedure closes and presents the results. The targeted error for deflection basin and modulus is less than 3 percent after one or two iterations. Compare the results from the basin and modulus backcalculation methods to obtain optimum results with low standard deviations and low coefficients of variation. A coefficient of variation that is less than 15 percent is good but this statistic depends heavily on the variability of the pavement layer thicknesses, material types, and strength.

Figure 5-14 PCASE Backcalculation Closure Options



Equation 5-12. Basin RMSE

$$RMSE_{Deflection\ Basin} = 100 * \sqrt{\frac{\sum_i^n \left(\frac{D_{measured} - D_{computed}}{D_{measured}}\right)^2}{n}}$$

Where:

$RMSE_{Deflection\ Basin}$ = *Deflection basin root mean square error*
i = *ith Sensor*
n = *Total number of sensors*
$D_{measured}$ = *Measured deflection at sensor i*
$D_{computed}$ = *Computed deflection at sensor i*

Equation 5-13. Modulus RMSE

$$RMSE_{Modulus} = 100 * \sqrt{\frac{\sum_{i}^{n}\left(\frac{E_{i-1,j} - E_{i,j}}{D_{i-1,j}}\right)^2}{n}}$$

Where:

$RMSE_{Modulus}$ = *Modulus root mean square error*
i = *ith Iteration*
j = *jth Layer*
n = *Total number of layers*
$E_{i-1,j}$ = *Modulus from previous iteration for layer j*
$E_{i,j}$ = *Modulus for current iteration for layer j*
$D_{i-1,j}$ = *Deflection from previous iteration for layer j*

5-3.8 Step 7 – Select Layer Model for Analysis.

In addition to the basin and modulus error closure procedure described above and shown in Figure 5-15, PCASE provides several other statistics to aid in selecting a basin for layered elastic analysis. These include the representative basin (mean modulus error) (Equation 5-15) and the mean measurement error (Equation 5-16).

Figure 5-15 Detailed Basin Results

Detailed Basin Results

Backcalculated moduli

Representative Basin	Mean modulus and error	Mean measurements error	Station	Drop	Basin Error	Modulus Error	Iterations	Hit Limit?	E1 (psi)	E2 (psi)	E3 (psi)
☐	☑ 47.80%	34.84%	1	2	18.2%	✓ 2.9%	✓ 5	☐	240,208	10,775	17,724
☐	☑ 45.06%	132.41%	1	2	12.9%	✓ 2.3%	✓ 5	☐	240,208	13,759	17,076
☐	☑ 38.77%	70.03%	2	2	21.2%	✓ 3.4%	✓ 5	☐	240,208	17,274	19,966
☐	☑ 53.08%	111.44%	2	2	19.0%	✓ 0.1%	✓ 4	☑	240,208	5,000	19,775
☑	☑ 7.18%	85.06%	3	2	16.3%	✓ 1.0%	✓ 4	☐	240,208	52,969	22,888
☐	☑ 26.14%	✓ 6.25%	4	2	13.6%	✓ 4.8%	✓ 4	☐	240,208	26,607	23,426
☐	☑ 40.05%	12.93%	4	2	10.7%	✓ 3.7%	✓ 4	☐	240,208	15,158	27,364
☐	☑ 23.38%	11.77%	5	2	15.1%	✓ 1.4%	✓ 4	☐	240,208	32,125	30,487
☐	☑ 47.20%	46.94%	6	2	28.2%	✓ 2.4%	✓ 4	☐	240,208	18,581	38,198
☐	☑ 56.82%	62.67%	7	2	6.7%	✓ 0.2%	✓ 4	☐	240,208	93,023	33,119
☐	☑ 126.07%	149.00%	8	2	13.8%	✓ 0.1%	✓ 3	☑	240,208	150,000	39,205
☐	☑ 29.63%	11.27%	9	2	8.5%	✓ 0.0%	✓ 4	☐	240,208	72,666	27,189
☐	☑ 77.44%	246.87%	10	2	5.8%	✓ 3.0%	✓ 4	☐	240,208	110,285	15,140
☐	☑ 23.39%	173.53%	11	2	5.9%	✓ 3.0%	✓ 4	☐	240,208	58,004	16,094

5-3.8.1 Representative Basin.

PCASE determines the representative basin using Equation 5-14. It highlights the row for the basin with the lowest mean modulus error, which is based solely on the backcalculation results. The basin with the lowest error is sent to LEEP for analysis unless the user selects another basin (e.g., the basin with the lowest mean measurement error).

Equation 5-14. Mean Modulus Error

$$Error_k = \sum_{i=1}^{NL} \left(\frac{\bar{E}_i - E_i}{\bar{E}_i} \right)^2$$

Where:

$\bar{E}_i$	*= Average of the modulus of the i-th layer among all the basins 1 to k*
k	*= basin number*
NL	*= number of layers*

5-3.8.2 Mean Measurement Error.

The mean measurement error is computed using Equation 5-15 and is based solely on the FWD data, not the backcalculation results. The basin with the lowest error is indicated by a green circle with a white checkmark in the mean measurements error column as shown in Figure 5-15.

Equation 5-15. Representative Basin

$$Error_k = \left(\frac{\overline{ISM} - ISM_k}{\overline{ISM}} \right)^2 + \sum_{1}^{ND} \left(\frac{\overline{DF} - DF_k}{\overline{DF}} \right)^2 + \left(\frac{\overline{AREA} - AREA_k}{\overline{AREA}} \right)^2$$

Where:

ISM	*= computed ISM*
DF	*= measured deflection*
AREA	*= computed area*
k	*= basin number*
ND	*= number of deflection sensors*
$\overline{\text{ISM}}$	*= average ISM*
$\overline{\text{DF}}$	*= average deflection*
$\overline{\text{AREA}}$	*= average basin area*

5-3.8.3 Basin Selection for Analysis.

Ideally, we want the basin and modulus error of closure (paragraph 5-3.7.5) to be below five percent, but there can be situations when one or both values exceed this threshold. In addition, having low errors for any of the statistics outlined above does not guarantee modulus values for the layers are reasonable. When results are not reasonable, adjust the model or backcalculation parameters as outlined in the following paragraphs and run backcalculation again. Select a basin with reasonable results for the material type even if the error is higher. Simply taking the average of each deflection reading from each FWD sensor and computing engineering properties from an “average deflection basin” is not a best-practices procedure. Each FWD test within a section represents a unique pavement response (e.g., deflection basin) for a unique pavement cross-section.

5-3.8.4 Backcalculation Analysis Guidelines.

Contributing factors that affect the reasonableness of results include errors between measured and calculated values, compensating adjacent layer E-values, or assigning inappropriate E-values. To overcome these issues, first identify the cause of the issue and do not make random changes to the structure. The following backcalculation guidelines are helpful in determining layer moduli.

- If modulus values are against the limits, turn off the limits and backcalculate again or modify the limits to include the computed elastic modulus. Results can come back within the original boundary conditions.
- Fix the modulus of an AC or PCC surface layer using the Temp or Flex option or based on tests conducted with the PSPA or on material type and condition at the time of testing rather than computing the modulus.
- Combine base and subbase into one layer and compute a composite modulus or divide the base course into two layers.
- Fix the subgrade modulus based on results of a preliminary run or on the deflection of sensor #7. In some cases, subdividing the subgrade into two layers is warranted.
- When a rigid pavement has a base and/or subbase, best practice is to include them in the model. However, if results are not reasonable, use a two-layer model with a composite modulus for the combined base and subgrade. Note that this can impact the PCN subgrade category in analysis.
- Do not attempt to compute the modulus of layers less than 3 inches (76 millimeters) thick. Assign the modulus of a thin layer based on material type, temperature, etc., or combine a thin layer with an adjacent layer with similar material properties to determine a composite modulus.
- Exercise caution when using modulus values outside the default ranges. Because the ranges are quite broad, values outside these limits can be unrealistic.

5-3.9 Step 8 – Layered Elastic Analysis.

The PCASE LEEP module uses YULEA to compute load-carrying capabilities and required overlay thicknesses for the defined traffic pattern (e.g., aircraft gear configuration, load, pass intensity level) on an existing pavement structure using layer moduli obtained through backcalculation or assigned based on one of the other previously described options. YULEA computes stresses (rigid and non-rigid overlay on rigid pavement) and strains (flexible pavement) that occur in the pavement system. Next, it calculates the limiting stress or strain values from empirically developed layered elastic values. LEEP compares the predicted stress or strain to the limiting value and outputs the allowable load for the defined pass level and allowable passes for the defined traffic (aircraft) load. The specific criteria and methodology are outlined below.

5-3.9.1 Analysis Criteria.

Maximum stresses and strains within a pavement system are computed using the controlling wheels of the design aircraft. The location of the maximum stress and strain value is influenced by factors such as pavement structure, wheel load, and wheel spacing. For a single wheel aircraft, the maximum stress and strain always occurs directly underneath the wheel. For other more complicated gear configurations, compute stresses and strains at several positions to determine the critical values. The PCASE LEEP module uses YULEA to determine the limiting values of stress/strain for a particular pavement type using the following.

5-3.9.1.1 AC Pavement Analysis Criteria.

The horizontal tensile strain at the bottom of the AC layer and vertical subgrade strain at the top of the subgrade are considered when evaluating flexible pavements. The limiting AC strain criterion (shown graphically in Figure 5-16) is as follows:

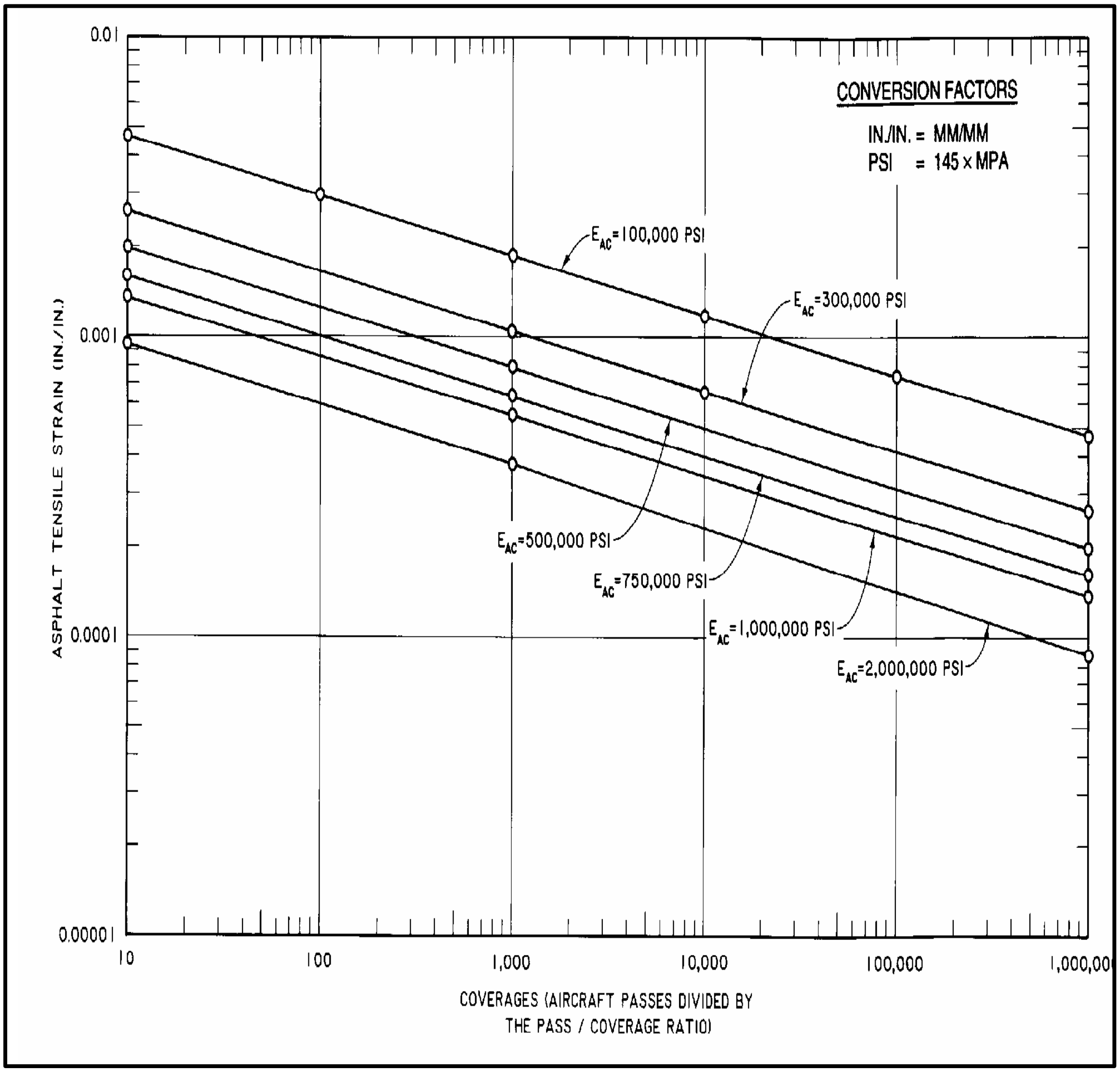
Equation 5-16. Allowable AC Strain

$$ALLOWABLE\ STRAIN_{AC} = 10^{-A}$$

Where:

ALLOWABLE STRAIN$_{AC}$ = allowable tensile strain at the bottom of the asphalt layer, inches/inches

Figure 5-16 Limiting Horizontal Tensile Strain Criteria for an AC Layer



The allowable subgrade strain criterion (shown graphically in Figure 5-17) is calculated using Equation 5-17.

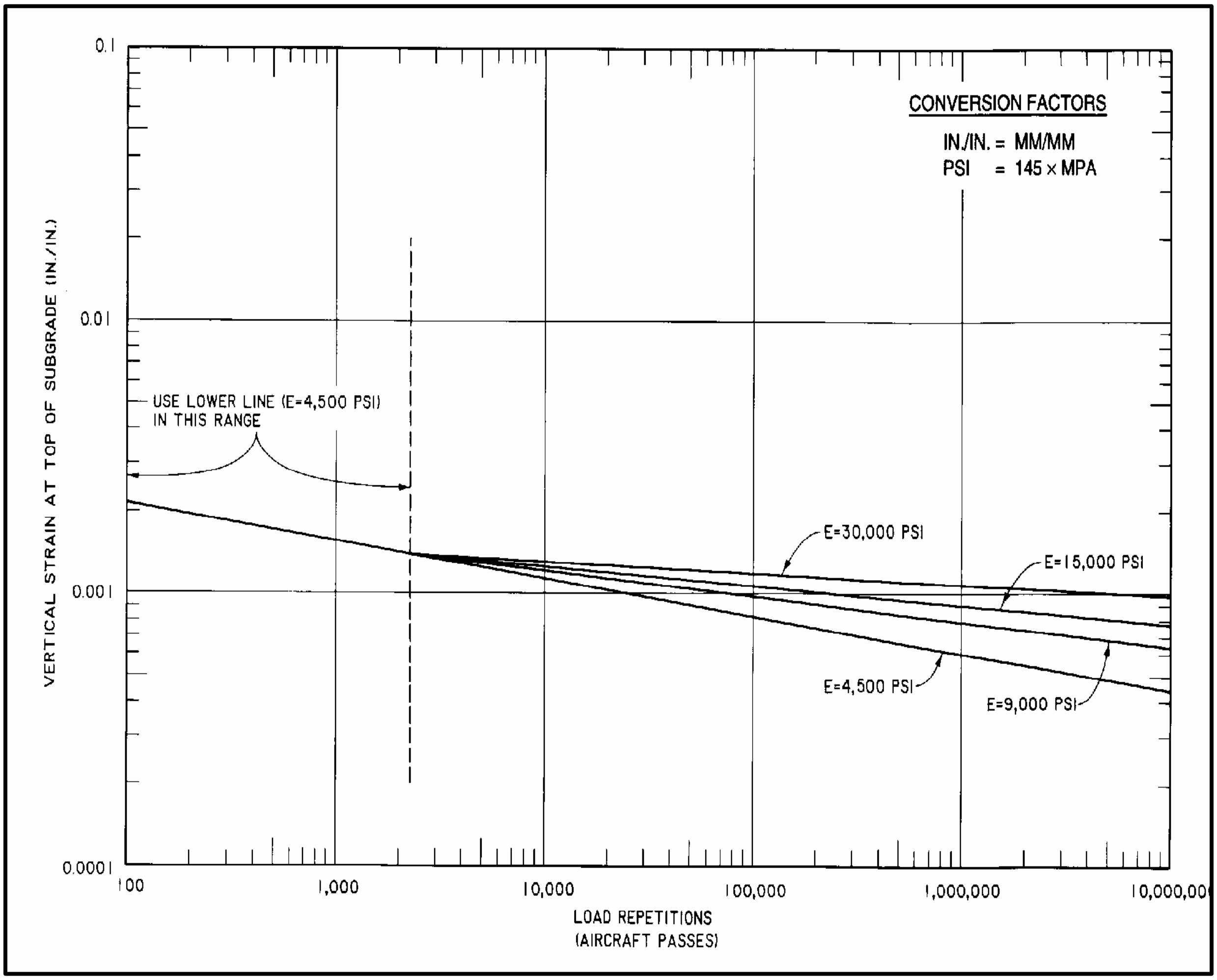
Equation 5-17. Allowable Subgrade Strain

$$ALLOWABLE\ STRAIN_{SG} = \left(\frac{10{,}000}{N}\right)^{1/B} A$$

Where:

ALLOWABLE STRAIN$_{SG}$	*= allowable vertical strain at the top of the subgrade, inches/inches*
N	*= aircraft repetitions (passes)*
A	*= 0.000247 + 0.000245 LOG(E_{SG})*
B	*= 0.0658 (E_{SG})$^{0.559}$*
E_{SG}	*= subgrade modulus, psi*

Figure 5-17 Limiting Vertical Subgrade Strain Criteria for Flexible Pavement



5-3.9.1.2 Asphalt Design Modulus.

While the backcalculation procedure uses the surface and five-day mean to determine a modulus, the analysis procedure uses the design air temperature that is the average of the hottest month's mean and maximum temperatures. The LEEP module pulls this data from the world index (climate) database to determine the design pavement temperature using the relationship in Figure 5-18. The design pavement temperature is then used in the relationship shown in Figure 5-19 to determine the asphalt modulus for the specific load frequency. Use the 10 Hz load frequency for runways and the 2 Hz load frequency for taxiways and aprons.

Figure 5-18 Design Pavement Temperature

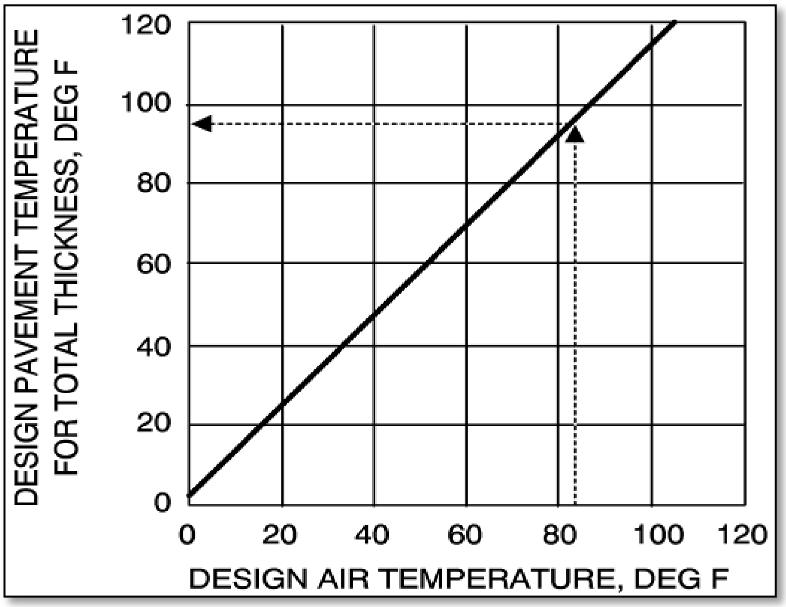
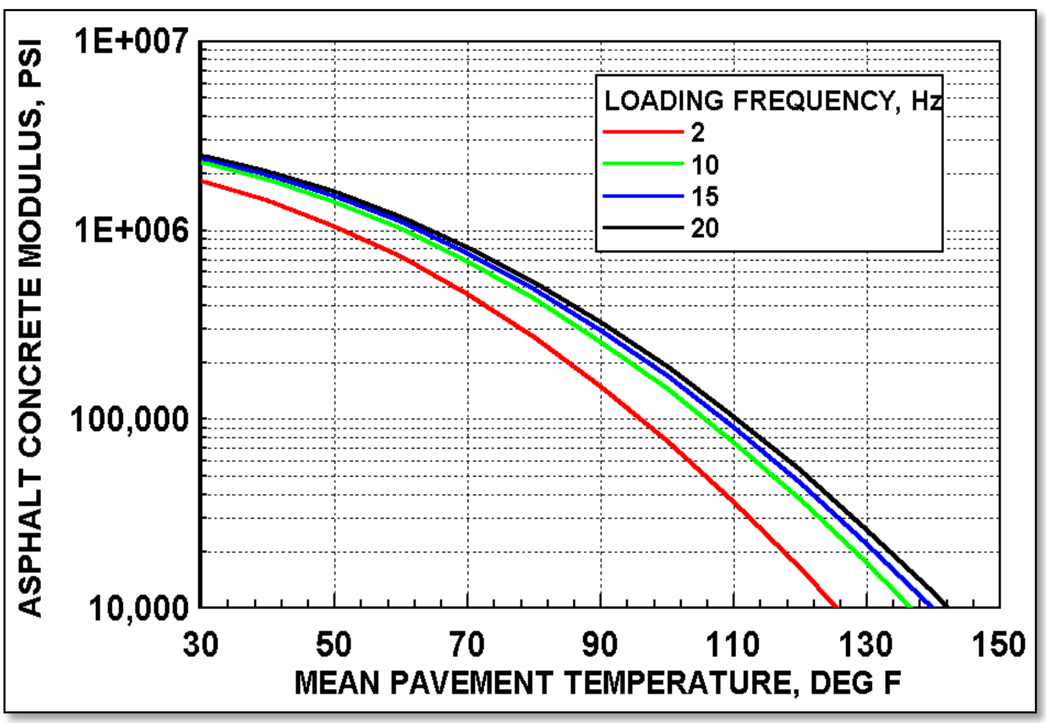


Figure 5-19 Asphalt Concrete Modulus



5-3.9.1.3 PCC Pavement Analysis Criteria.

LEEP assumes that an AC over PCC structure is a rigid pavement unless the backcalculated modulus of the PCC layer is less than 1,000,000 psi (6,895 MPa), then evaluate it as a flexible pavement. Rigid and non-rigid overlays of rigid pavements are evaluated based on the tensile stress at the bottom of the PCC slab and the predicted pavement deterioration in terms of the Structural Condition Index (SCI) as defined in Equation 5-18.

Equation 5-18. Structural Condition Index

$$SCI = 100 - A * (\,sum\ of\ structural\ deducts)$$

A is an adjustment factor based on the number of distress types with load-related PCI deduct values greater than five points as determined from the PCI survey procedure. The load-related PCI distresses are established and computed in the PAVER software program. These structural deducts are a function of distress types, severities, and densities associated with repeated aircraft and vehicle loads. The SCI prediction is based on a relationship between design factor and stress repetitions as related to crack formation in the PCC slabs due to load. An SCI of 50 corresponds well to the formation of one or more cracks per slab in 50 percent of the trafficked slabs (first crack failure criteria) and an SCI = 0 correlates approximately to a shattered-slab condition. The design factor, DF, is the concrete flexural strength divided by the flexural stress in a PCC slab.

Equation 5-19 shows the SCI-based equation for determining the DF. Using the PCC flexural strength, determine the allowable PCC slab flexural stress using Equation 5-20.

Equation 5-19. Design Factor

$$DF = A + B\ LOG\ \ C$$

Where:
DF = *design factor*
A = *0.2967 + 0.002267 (SCI)*
B = *0.3881 + 0.000039 (SCI)*
C = *coverage level at selected SCI*
SCI = *structural condition index*

Equation 5-20. Allowable PCC Slab Flexural Stress

$$ALLOWABLE\ STRESS_{PCC} = \frac{R}{DF}$$

Where:
$ALLOWABLE\ STRESS_{PCC}$ = *allowable tensile stress at the bottom of the slab, psi*
R = *PCC flexural strength, psi*

5-3.9.2 PCC Joint Load Transfer Efficiency Using NDT Tests.

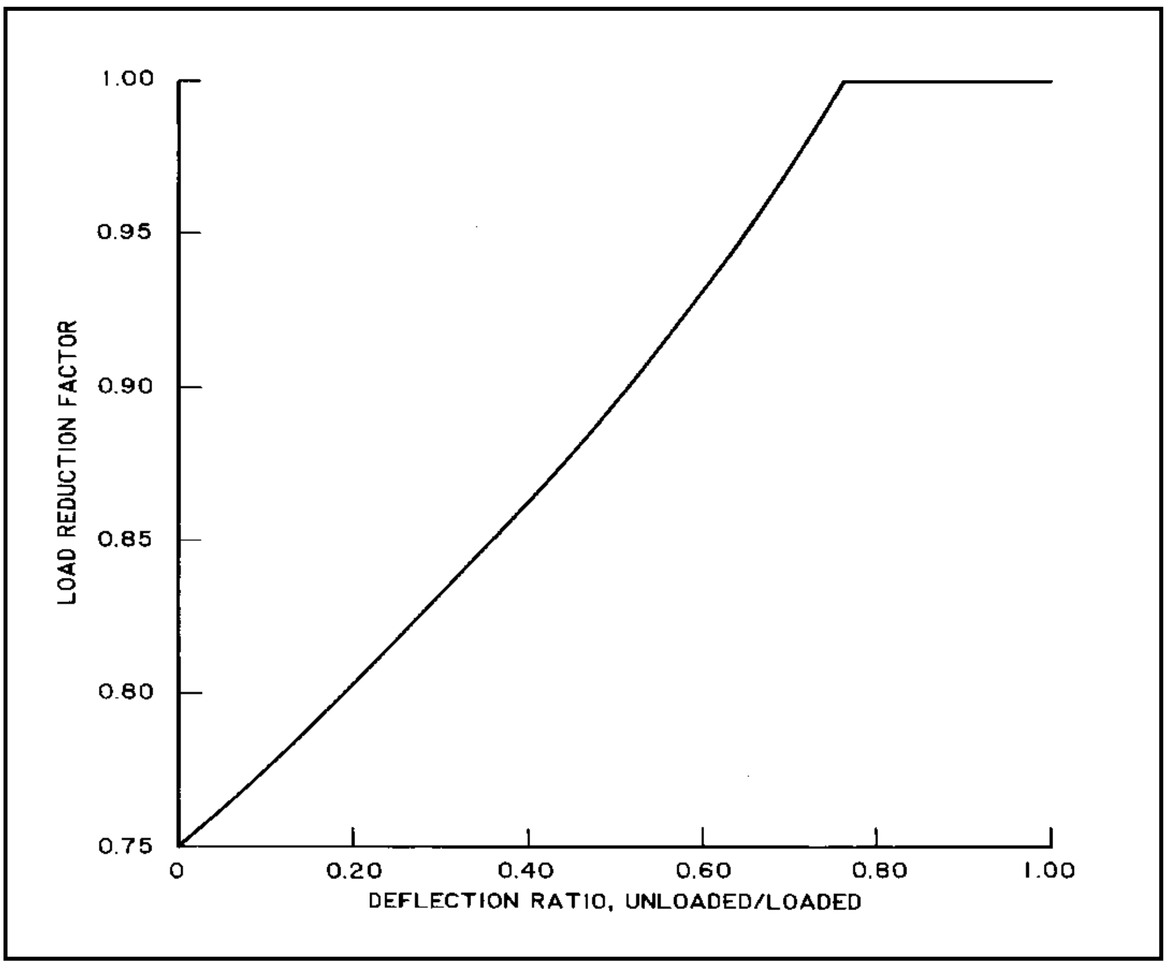
Rigid pavement analysis assumes 25 percent load transfer between slabs. The allowable loads determined at the slab centers can be reduced for poor joint transfer using load reduction factors shown in Figure 5-20. So, when there is evidence that there is a lack of load transfer (e.g., longitudinal cracking along the length of a section), test the joint load transfer as outlined in paragraph 3-4.1.7 and use the PCASE FWD module to compute the deflection ratio and load transfer efficiency as follows:

Equation 5-21. Deflection Ratio and Load Transfer Efficiency

$$DEFLECTION\ RATIO\ = \frac{DEFLECTION\ OF\ UNLOADED\ SLAB}{DEFLECTION\ OF\ LOADED\ SLAB}$$

The relationship in Figure 5-20 was developed using finite element programs to compute edge stresses for a range of pavement thicknesses and subgrade moduli and k values to relate the deflection ratio to the percent maximum edge stress. The maximum edge stress condition is a free edge with no load transfer. The edge stress is reduced as more load is transferred across a PCC joint from the loaded to the unloaded slab. For a load reduction factor of 1.0 (e.g., 100 percent of the aircraft design load), the deflection ratio is at least 76 percent as shown in Figure 5-20. As the deflection ratio falls below 76 percent, the load factor and corresponding design load decrease. The load reduction factor varies from 0.75 to 1.00, with a minimum load reduction factor of 75 percent when the deflection ratio is zero. This procedure is also used for both rigid and non-rigid overlays of rigid pavements.

Figure 5-20 Load Reduction Factors for Load-Transfer Analyses

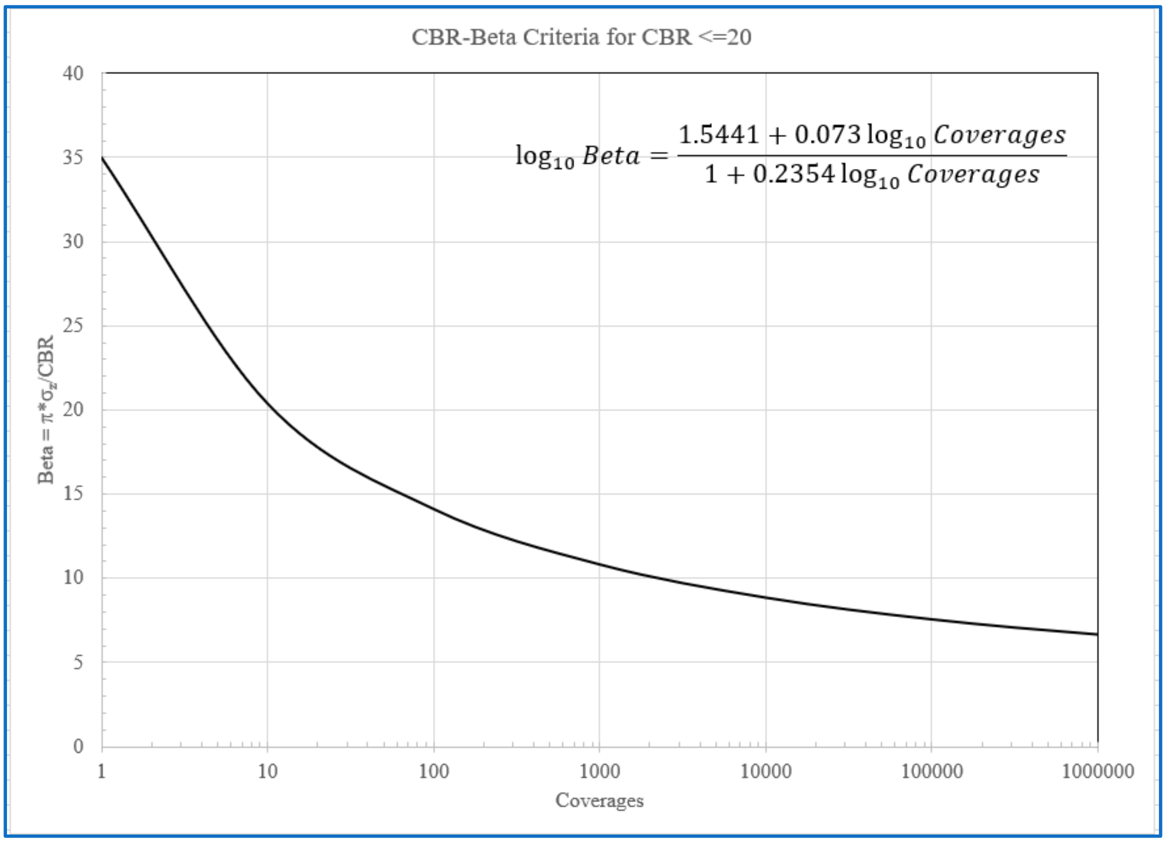


CHAPTER 6 FLEXIBLE PAVEMENT EVALUATION – CBR PROCEDURE

6-1 PERFORMANCE CRITERIA.

This flexible pavement structural evaluation procedure is a mechanistic-empirical approach known as the Alpha-Beta hybrid procedure which uses the California Bearing Ratio (CBR) as a measure of strength to analyze the vertical stress at the top of each layer and determine the allowable load and passes for an existing structure. Figure 6-1 shows the CBR Beta Performance model, which is based on the test points gathered in multiple full-scale test sections. The CBR Beta model is used when the CBR of a layer is less than or equal to 20. When the CBR of a layer is greater than or equal to 30, the CBR Alpha model is used for analysis, and when the CBR is greater than 20 and less than 30, the Alpha-Beta Hybrid model is used for analysis. The term CBR procedure is commonly used to describe the alpha-beta hybrid procedure. The details of this procedure are outlined in Appendix D and ERDC/GL TR-12-16, *Reformulation of the CBR Procedure*.

Figure 6-1 CBR-Beta Performance Criteria



6-2 FACTORS LIMITING LOAD-CARRYING CAPABILITY.

Structural failure criterion for a flexible pavement is based on a 1-inch (25-millimeter) rut. The load-carrying capability of a flexible pavement is limited by its critical or controlling layer, either the pavement surface, base, subbase, or subgrade.

6-2.1 Controlling Layer.

The ability of a given subsurface layer to withstand the loads imposed on it depends on the thickness and strength of material above it and its strength in its weakest condition. The critical or controlling layer is the layer that will support the least allowable load. To be realistic, an evaluation must consider possible future changes in moisture content and density as well as the effects of freezing and thawing.

6-2.2 Surface Condition.

A flexible pavement is assumed to have lost some structural capability when the PCI is less than or equal to 40 (VERY POOR, SERIOUS, or FAILED). When this occurs, a 25 percent load reduction is imposed on the section.

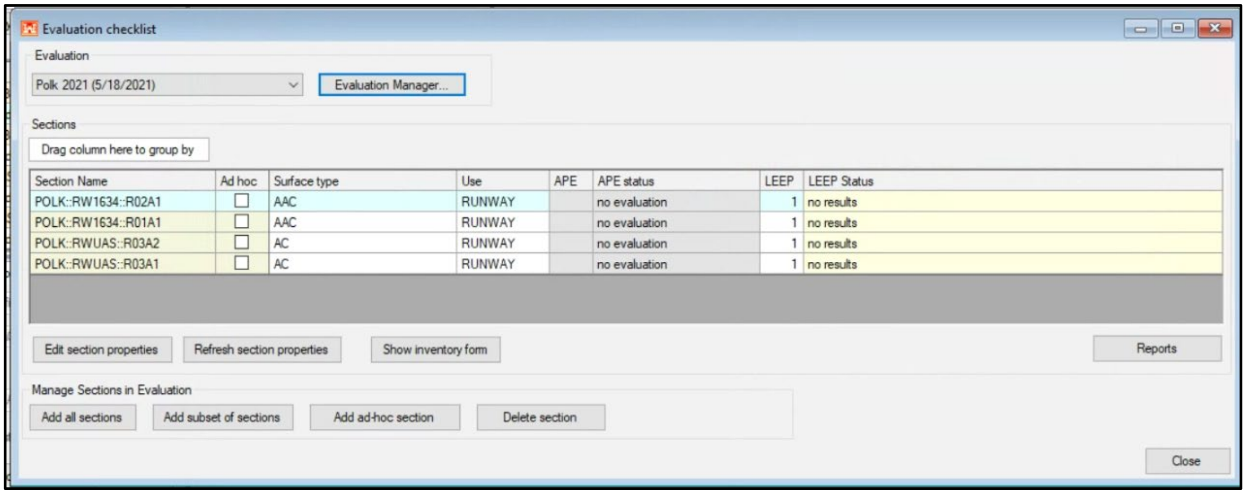
6-3 FLEXIBLE PAVEMENT (CBR) EVALUATION PROCEDURE.

The CBR evaluation procedure applies to flexible pavements. It analyzes the shear stress at the top of each layer using the CBR as a measure of the shear strength. It uses layer properties determined from in situ measurements to compute allowable loads for a selected number of aircraft passes, allowable passes at a specified load, and the Pavement Classification Number (PCN). When the pavement structure cannot support the defined pass level and aircraft load, determine overlay requirements to strengthen the pavement when desired. Following is a step-by-step procedure for evaluating a pavement section. Repeat Steps 2 through 4 of this process for each section being evaluated. The Pavement-Transportation Computer Assisted Structural Engineering (PCASE) Airfield Pavement Evaluation (APE) module implements the CBR criteria. More detailed information on using PCASE is available in the PCASE *User Manual*.

6-3.1 Step 1 – Create a New Evaluation.

Open the PCASE Evaluation Checklist to create a new evaluation using the Evaluation Manager. Define the Service, climate data, evaluation traffic, and rigid failure criteria for the evaluation, then assign the inventory sections to be included in the evaluation.

Figure 6-2 Evaluation Checklist



6-3.2 Step 2 - Input Pavement Layers and Thickness.

Open the APE module, edit the default layer structure, and enter the pavement thickness for each section. Determine the in-place thicknesses of asphaltic concrete to the nearest 0.25 inch and underlying unbound layers to the nearest inch by testing or from construction data when testing is not possible. Layer thickness testing can include measurements from coring, DCP, soil boring, GPR, or a combination of these tests. The number of tests required will vary based on the area and use of the pavement as well as the uniformity of the structure. When the layer thicknesses vary for a given section, evaluate the section using different models that replicate what was seen in the field, but only report the controlling evaluation for the facility.

6-3.2.1 Equivalency Factors.

When the measured thickness of a layer exceeds the required minimum thickness as defined in UFC 3-260-02, the excess measured thickness is converted to an equivalent thickness of base course and added to the existing base thickness. Then, any excess base-course thickness is converted to an equivalent thickness of subbase and added to the subbase thickness. This adjusted section is then used for evaluation. The equivalency factors for converting asphalt to base and subbase are 1.15 and 2.3 respectively, and for converting base course to subbase is 2.0, as shown in Table 6-1. This means that 1 inch (25.4 millimeter) of asphalt is equal to 1.15 inches (29 millimeters) of base and 2.3 inches (58 millimeters) of subbase, and 1 inch (25.4 millimeter) of base course is equal to 2 inches (51 millimeters) of subbase. The following example illustrates the use of equivalency factors.

Table 6-1 Equivalency Factors

Material	Base Equivalency Factor	Subbase Equivalency Factor
Unbound crushed stone	1.00	2.00
Unbound subbase*	-	1.00
Asphalt-stabilized and all-bituminous concrete GW, GP, GM, GC (SW, SP, SM, SC)*	1.15 1.00 -	2.30 2.00 1.50
Cement-stabilized GW, GP, SW, SP GC, GM (ML, MH, CL, CH)* (SC, SM)*	 1.15 1.00 - -	 2.30 2.00 1.70 1.50
Lime-stabilized (ML, MH, CL, CH)* (SC, SM, GC, GM)*	 - -	 1.00 1.10
Lime-, cement-, fly ash-stabilized (ML, MH, CL, CH)* (SC, SM, GC, GM)*	 - -	 1.30 1.40

* **Note:** Material is not to be used as a base layer.

6-3.2.2 Equivalent Thickness Example.

Evaluate a runway touchdown section for C-130 operations. The measured thickness of the pavement section and the equivalent thickness used to evaluate the pavement are shown in Table 6-2. The C-130 requires a minimum surface thickness of 4 inches (102 millimeters) and a minimum base thickness of 6 inches (152 millimeters). The base is unbound crushed stone.

Table 6-2 Equivalent Thicknesses

Layer	Measured Thickness (in.)	Equivalent Thickness of Base (in.)	Equivalent Thickness of Subbase (in.)	Evaluation Thickness (in)
Asphalt surface	5	5" - 4" min = 1" excess	-	4
Base	7	8.15 = 7.0 +1 x 1.15)	8.15" – 6" min = 2.15" excess	6
Subbase	10	-	14.30 = 10 + 2.15 x 2	14.3
Subgrade	-	-	-	-

6-3.2.3 Stabilized Layer Equivalent Thickness.

Stabilized layers are incorporated in the design of pavement sections to make use of locally available materials that cannot otherwise meet the criteria for base or subbase courses. Materials must meet the requirements in UFC 3-250-11, *Soil Stabilization and Modification for Pavements*. In design, the equivalency factors shown in Table 6-1 are assigned to the stabilized material and result in a thickness reduction as compared with an unbound base course or subbase course. These same equivalency factors result in an increase in thickness of the layer in evaluation. If no information is available on the condition and strength of the stabilized layer, it should be treated as a high-quality granular layer. If DCP results indicate the layer is well stabilized (refusal for DCP), then consider the layer for the equivalency factors.

6-3.2.4 Stabilized Layer Equivalent Thickness Example.

Assume that an Air Force pavement structure consists of a 4-inch (102-millimeter) asphaltic concrete, an 8-inch (203-millimeter) bituminous concrete base, and an 8-inch (203-millimeter) cement-stabilized gravelly clay subbase with an unconfined compressive strength of 700 psi (4.83 MPa). From Table 6-1, the 8-inch (203-millimeter) bituminous concrete base equivalency factor is 1.15, which increases the thickness of the stabilized base for evaluation to 9.2 inches (234 millimeters). Table 6-1 shows that the 8-inch (203-millimeter) cement-stabilized subbase has an equivalency factor of 2.0, which increases the thickness of the stabilized subbase for evaluation to 16 inches (406 millimeters).

6-3.3 Step 3 - Soil Layer Strength Values.

Enter the CBR for the subgrade and overlying subbase and base courses. Both in-field and laboratory CBR tests are described in CRD-C654, *Standard Test Method for Determining the California Bearing Ratio of Soils*. Field DCP tests are described in Appendix A and TM 3-34.48-2, Appendix G. Use construction data in conjunction with testing or when testing is not possible. The CBR test results from an individual test pit or from multiple DCP tests are seldom uniform. Therefore, analyze the data carefully as described in Chapter 3 to determine reasonable CBR values to use for an evaluation.

6-3.3.1 Base Course CBR.

Base course CBR or DCP testing can produce inaccurate CBR values when performing in-place tests or for laboratory tests due to inherent difficulties in processing samples. For example, DCP test results may show a 100 CBR for a Poorly Graded Gravel however, it is likely the DCP encountered large aggregates that skewed the test results. In this case, assign CBR values based on the material's typical behavior, as shown in Table 6-3.

Table 6-3 Assigned CBR values for Base Course Materials

Aggregate Base Course	Assigned CBR
Graded crushed aggregate	100
Aggregate	80
Limerock	80
Coral	80
Shell Rock	80

6-3.4 Step 4 – Flexible Pavement Analysis.

6-3.4.1 Alpha-Beta Hybrid (CBR) Evaluation Procedure.

Once the thickness and CBR values are selected for each of the layers, use these values to determine the shear stress at the top of each layer based on the stress-based CBR Alpha-Beta hybrid procedure assuming constant tire pressure. The objective of the analysis is to determine the allowable load and allowable passes for the structure. Note that results using the current criteria will differ from the CBR Alpha criteria and constant contact area assumption used in past versions of this UFC. PCASE automates the analysis procedure outlined in this chapter and in Appendix D. The procedure for generating aircraft curves using the current criteria is included in TSPWG M 3-260-03.02-19.

6-3.4.2 Procedure for Determining Allowable Gross Load (AGL).

The inputs for this analysis are the traffic mix with the load and number of passes for each vehicle in the mix defined, the pavement structure, and the traffic area. Determine the controlling/representative vehicle and equivalent passes based on one of the traffic analysis procedures outlined in Chapter 4. Perform the allowable coverages calculation using the Alpha-Beta Hybrid procedure in which limiting (vertical) stress is calculated for each layer in the pavement structure based on load of the controlling/representative vehicle load. Compute the cumulative damage factor (CDF). If the CDF is less than 1, increase the gross load and repeat the analysis procedure. If the CDF is greater than 1, decrease the load and repeat the analysis procedure. When CDF equals 1, use that value for the AGL. See Appendix D for details on this procedure.

6-3.4.3 Procedure for Determining Allowable Passes.

The inputs for this analysis are the traffic mix with the load and number of passes for each vehicle in the mix defined, the pavement structure, and the traffic area. Determine the controlling/representative vehicle and equivalent passes based on one of the traffic analysis procedures outlined in Chapter 4. Perform the allowable coverages calculation using the Alpha-Beta Hybrid procedure in which limiting (vertical) stress is calculated for each layer in the pavement structure based on load of the controlling/representative vehicle load. See Appendix D for details on this procedure.

6-3.4.4 Load, Tire Pressure, and Contact Area Relationship.

Typically, the relationship between weight on a tire, tire pressure, and contact area is:

Tire Contact Area = Load on Tire/Constant Tire Pressure

This relationship is good for AGLs up to approximately the maximum aircraft load. At that point, contact area begins increasing to unrealistic values to the extent that the limiting stress is not reached. Therefore, a solution for allowable load is not achievable. To resolve this issue, the following relationship is used for the allowable loads above the maximum aircraft loads.

Equation 6-1. Tire Pressure Relationship

$$T_p = T_{pml} + \frac{D}{T_{ca}\left[1 + \left(\frac{AGL}{D}\right)^3\right]}$$

where

$T_p = Tire\ pressure\ used\ for\ calculations$

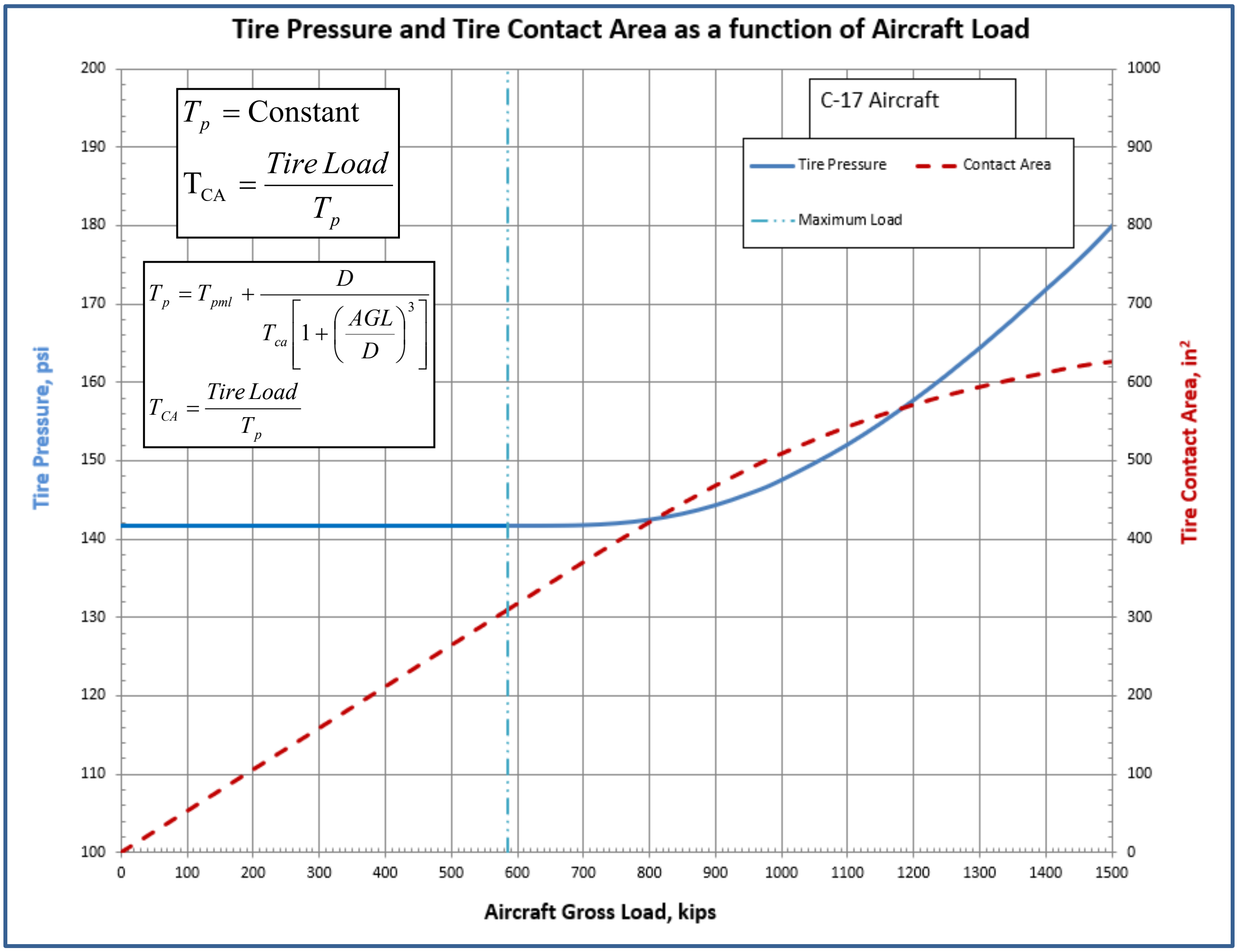
$T_{ca} = Tire\ contact\ area\ at\ MaxLoad$

$T_{pml} = Tire\ pressure\ at\ MaxLoad$

$D = AGL - MaxLoad$

An example of this relationship for the C-17 is shown in Figure 6-3.

Figure 6-3 Tire Pressure/Contact Area vs. AGL



6-3.5 Pavement Classification Number (PCN).

The process described above is used to calculate the allowable load which is then used to compute the PCN. Comparing the aircraft classification number (ACN) to the PCN of a pavement section is an expedient way to determine if it can support a particular aircraft. Chapter 9 presents the PCN procedure.

6-4 OTHER FLEXIBLE PAVEMENT EVALUATION CONSIDERATIONS.

The structural analysis procedure above assumes the quality of the materials and construction procedures used to construct a flexible pavement meet the criteria outlined in UFC 3-260-02 and the various Unified Facility Guide Specifications (UFGS). When field and laboratory testing indicate that this assumption is not valid, adjust evaluation inputs or at least fully document any anomalies in the report. The following paragraphs discuss evaluation issues that should be considered.

6-4.1 Ability to Support Traffic.

The type and gradation of the aggregate, the amount of bitumen in the mix, and the compaction of the mix all affect the ability of a mix to support traffic of a given load. Mixes with rounded aggregates are less stable than those with crushed-face aggregates. Mixes with aggregates of irregular grading are less stable than those with well-graded aggregates. A bitumen deficiency produces a pavement that may ravel, but too much bitumen produces a pavement that may rut and shove. Compare the test data from the laboratory recompacted core sample specimens taken during the evaluation with the design criteria in UFC 3-260-02. The condition of surface or binder course pavement at the time of sampling can be an indication of future behavior under additional traffic. Table 6-4 shows the prediction of behavior from tests on cores and on laboratory recompacted surface course specimens. Assume the thickness and aggregate gradation are satisfactory.

Table 6-4 Example Test Data

Tests	Field Cores	Recompacted Sample - 50 Blows*	Recompacted Sample - 75 Blows
Unit weight (density), pcf	144.2	149.7	150.9
Unit weight, percent of 50-blow laboratory compaction	96	-	-
Unit weight, percent of 75-blow laboratory compaction	95	-	-
Stability (pounds)	1,883	2,929	3,276
Flow (1/100 inch)	15	16	16
Voids total mix, percent	8.5	4.5	3.7
Voids filled, percent	57.2	72.1	75.8

***Note:** For shoulders and overruns

The test data from recompacted specimens shown above indicates the current density (field cores) is relatively low, the flow is approaching the upper limit, and the void relations are outside the acceptable ranges, but the stability is satisfactory. This means that additional compaction from traffic will likely increase the stability but also cause some rutting of the pavement. Therefore, the pavement should be able to withstand heavier loads than it sustained in the past and is satisfactory under traffic having up to 200 psi (1.4 MPa) tire pressure. At 75 blow laboratory compaction, the voids total mix value is below the midpoint of the acceptable range and the flow is at the upper limit, indicating a mix slightly rich of optimum. However, no danger from flushing is expected.

6-4.2 Ability to Withstand Fuel Spillage.

Fuel dripping on a given area at frequent intervals or a pervious pavement mix that allows considerable penetration of the fuel will cause pavement distresses because asphaltic cements are readily soluble in fuels. The voids in the total mix control the rate at which penetration occurs. Fuel will penetrate very little into pavements with 3 percent voids but will rapidly penetrate pavements with high (over 7 percent) voids. Therefore, an AC layer with higher density will typically increase the pavement's resistance to jet fuel penetration and weathering. Pavements about one year or older usually perform better in this respect than new pavements. Evaluate the surface course characteristics for resistance to jet fuel. Table 6-5 serves as a guide for evaluating asphalt pavements regarding fuel spillage for use in different areas of the airfield.

Table 6-5 Surface Course Fuel Resistance

Pavement Type	Texture	Satisfactory for
Asphaltic concrete	Dense	Runway interiors and areas of taxiways where aircraft do not warm up or stop frequently
Asphaltic concrete	Open	Runway interiors or any high-speed areas

6-4.3 Ability to Withstand Jet Blast.

Tests have shown that about 300 °F (149 °C) is the critical temperature for asphaltic concrete. Field tests simulating pre-takeoff checks at the ends of runways indicate that the maximum temperatures induced in the pavements when afterburners are not used are less than 300 °F (149 °C). Maximum temperatures induced in pavement tests simulating maintenance checkups are 315 °F (157 °C). When afterburners are turned on after the aircraft has begun the takeoff run, little or no damage occurs.

Thin-surface courses, not well-bonded to the underlying layers, are subject to erosion (e.g., weathering, raveling, jet blast) by a high-velocity blast, even though the binder is not melted. All jet aircraft currently in use are believed to produce blasts of sufficiently high velocity to flay such courses. Setback distances for running-up engines are established and included in UFC 3-260-01, *Airfield and Heliport Planning and Design*. Surface layers less than 1 inch (25.4 millimeters) thick and poorly bonded are considered unsatisfactory for parking areas and the 1,000-foot (304.8-meter) ends of runways and are so reported in the narrative portion of the evaluation report for all aircraft. DoD aircraft inventories now include aircraft with thrust vectors that potentially negatively impact airfield pavements, depending on operational usage. When these aircraft are present, the evaluation should consider the expected decrease in performance due to thrust vector forces.

6-4.4 Effects of Traffic Compaction on Paving Mixes.

Traffic tends to densify flexible pavements, depending on the gear loads applied and the characteristics of the mix. Densification is limited where traffic is widely distributed and is greatest where traffic is channelized. High tire pressures produce greater densification than low tire pressures. The probability of densification under a given loading decreases somewhat with pavement age because of hardening of the asphalt. A comparison of the in-place density and void relations of the pavement with the results of comparable tests on specimens recompacted in the laboratory gives an indication of future behavior. If the pavement is constructed so the voids fall near the lower limit of the specified allowable range, it is probable that aircraft with relatively high-pressure tires will produce sufficient densification to appreciably reduce the voids in the total mix. The pavement is considered unstable and may rut when the voids fall below the specified minimum (see UFC 3-260-02). These conditions cannot be translated into numerical evaluations, but they should be discussed in the evaluation report and summarized so engineers will have the information available.

6-4.5 Effects of Traffic Compaction on Base Course and Subgrade.

6-4.5.1 Degree of Compaction for CBR Values.

Definite degrees of compaction are specified for the subgrade and base course in airfield pavement construction to prevent excessive densification under traffic, the consequent development of surface roughness “birdbaths,” and loss of grade. The design CBR values are based on assumed degrees of compaction outlined in the specifications.

6-4.5.2 Density Requirements.

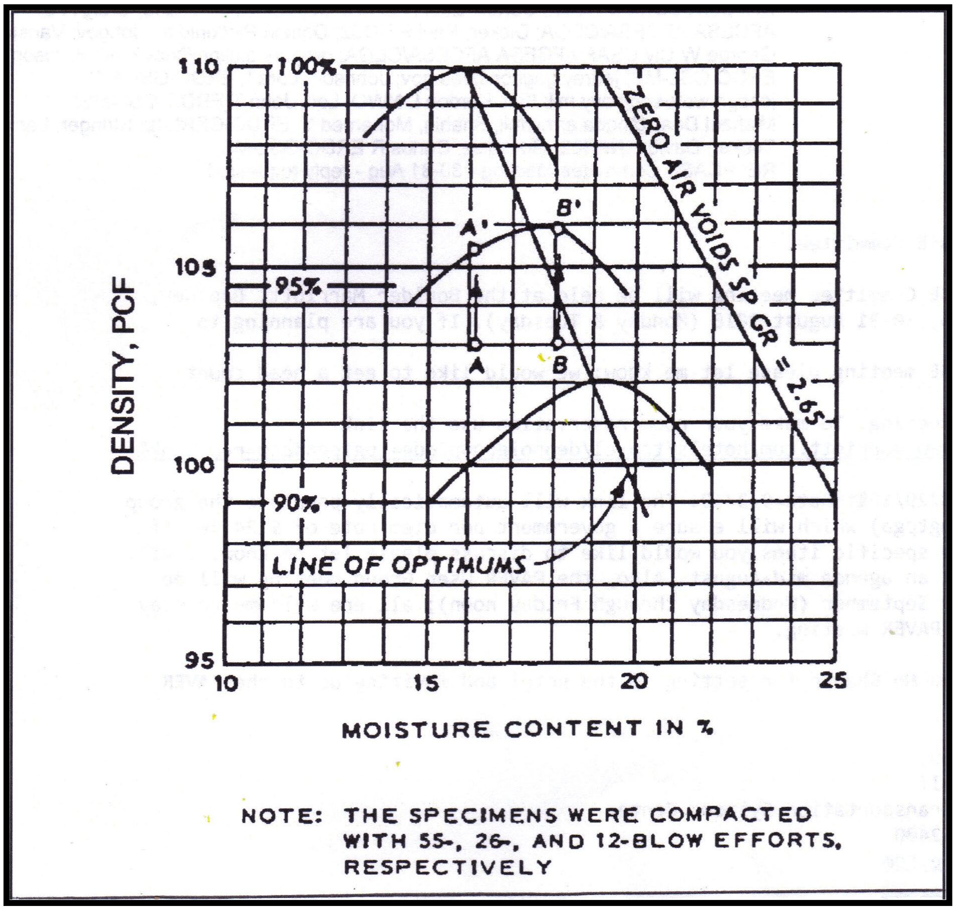
Compare the in-place densities, as a percentage of ASTM D1557, *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3 [2,700 kN-m/m^3])*, maximum density, with the design requirements for the various loads and gear configurations that the pavement is expected to support to evaluate the base, subbase, and subgrade from the standpoint of future compaction. When the in-place density of a layer is appreciably lower than that required, assume that traffic will densify the layer in time. Density requirements at various depths are discussed in UFC 3-260-02.

6-4.5.3 Selection of Evaluation CBR Value.

Consider the effect of further compaction on strength of base and subgrade. Some cohesive soils, when highly saturated, potentially develop pore pressures under traffic of heavy wheel loads and show serious loss of strength. Compare the in-place density and moisture contents with those of the laboratory compaction tests made at three compaction efforts to determine if there is potential for strength loss. These data are used to determine the line of optimums illustrated in Figure 6-4 by a line drawn through the three optimum moisture contents. Pore pressure seldom develops unless the moisture and density results fall to the right of the line of optimums. When this occurs, it is likely that future compaction will produce pore pressures. For example, consider point

A plotted in Figure 6-4 at a moisture content of 16 percent and a density of 103 pounds per cubic foot (1,651 kilograms per cubic meter). Assume this represents a subgrade that has 95 percent of ASTM D1557 maximum density. If further compaction occurs, the density will increase to approximately 105 pounds per cubic foot (1,682 kilograms per cubic meter) (point A' on the curve for 26 blow effort). Since this is to the left of the line of optimums, no pore pressures will develop. If the subgrade had a moisture content of 18 percent (point B), the increased compaction would cause the density to be plotted to the right of the line of optimums (B') and pore pressures would result. The CBR that would develop under this condition could be estimated from laboratory CBR tests in which the material was compacted to the same density and moisture content.

Figure 6-4 Line of Optimums



It is not necessary to lower the load-carrying capacity of the facility below that derived based on thickness and CBR because compaction does not meet specifications. However, if the measured densities are considerably less than those specified, the deterioration of the pavement may be high, resulting in a decrease in service life. Note that materials of low density combined with low moisture content may not densify under traffic, but subsequent increases in moisture content will permit densification. There may be possible settlement due to densification in the evaluation of pavements being subjected to channelized and heavy wheel-load traffic. In the case of cohesive materials that may develop pore pressures and a loss in strength, consider a lower CBR when evaluating allowable aircraft loads.

6-5 EVALUATIONS IN ARID REGIONS.

The danger of saturation beneath flexible pavements is reduced when the annual rainfall is less than 15 inches (381 millimeters), the water table (including perched water table) is at least 15 feet (5 meters) below the surface, and the water content of the subgrade will not increase above the optimum as determined by the ASTM D1557 compaction test. Under such conditions, the total design thickness of the pavement, when based on a soaked CBR, can be reduced 20 percent. This reduction is subtracted from the thickness of the select material or the subbase course having the lowest design CBR value. Therefore, when flexible pavements are evaluated using a soaked CBR value, the total thickness above the subgrade is increased 25 percent before entering the evaluation curves. This increase in thickness is added to the select material, or the subbase course having the lowest CBR, or to the same layer in which the reduction was made in the design analysis. This increase in thickness does not apply for evaluations using in-place data.

6-6 EVALUATION FOR FROST CONDITIONS.

If the existing soil, water, and temperature conditions are conducive to detrimental frost effects in the base-course, subbase, or subgrade materials, then the pavement evaluation is based on criteria for frost areas as given in Chapter 8.

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CHAPTER 7 RIGID PAVEMENT EVALUATION USING THE K PROCEDURE

7-1 PERFORMANCE CRITERIA.

This chapter presents criteria for evaluating rigid pavements using the Westergaard solution that uses medium-thick plate theory and treats the combined support of the base, subbase, and subgrade as a bed of independent springs (Winkler's Foundation) represented by the Modulus of Subgrade Reaction (k). Chapter 3 outlines how to use plate bearing tests or dynamic cone penetrometer (DCP) test data to determine the k value.

7-2 FACTORS LIMITING LOAD-CARRYING CAPACITY.

Jointed, plain PCC pavements are evaluated using stresses due to edge loading of a slab. Either first crack or shattered slab failure criteria are used for rigid pavements as described below. The Service dictates which criterion it uses in its evaluations.

7-2.1 Standard Evaluation Failure Criterion.

First crack failure (sometimes referred to as initial failure or standard failure) means that 50 percent of the slabs in a sample or section are cracked into two or three pieces.

7-2.2 Shattered Slab Failure Criterion.

Shattered slab failure (sometimes referred to as extended life failure) means that 50 percent of the slabs in a sample or section are cracked into approximately six pieces or when 50 percent of slabs are cracked into four pieces and cracks are medium or high severity.

7-2.3 Basis of Load-carrying Capability.

The load-carrying capability of rigid pavements depends on the thickness and flexural strength of the PCC surface layer and the support in terms of the modulus of subgrade reaction (k) value provided by the base, subbase, and subgrade. Long-term rigid pavement system performance depends on many elements, including PCC and stabilized base material durability, unbound material and subgrade gradations, moisture content and density, and regional climatic effects such as freezing and thawing.

7-2.4 Surface Condition

Assume a rigid pavement has lost some structural capability when the PCI is less than or equal to 40 (VERY POOR, SERIOUS, or FAILED). When this occurs, reduce the load on the section by 25 percent.

7-2.5 Load Transfer.

The rigid pavement analysis procedure assumes a 25 percent load transfer across joints from aggregate interlock in sawn joints or by dowels. When testing (by FWD) indicates inadequate joint load transfer, change the percent load transfer. This change recalculates the joint deflection ratio and increases the maximum edge stress to a maximum of 100 percent. The effect of this change is an allowable load reduction to 25 percent.

7-2.6 Combined Load Reduction.

Any allowable aircraft load reduction is based on engineering judgment, but there is typically no combined load reduction for both PCI and load transfer. The engineer must investigate all possible sources of pavement engineering data to ensure that site conditions, including field and laboratory test results, are consistent with proposed reductions in allowable loads.

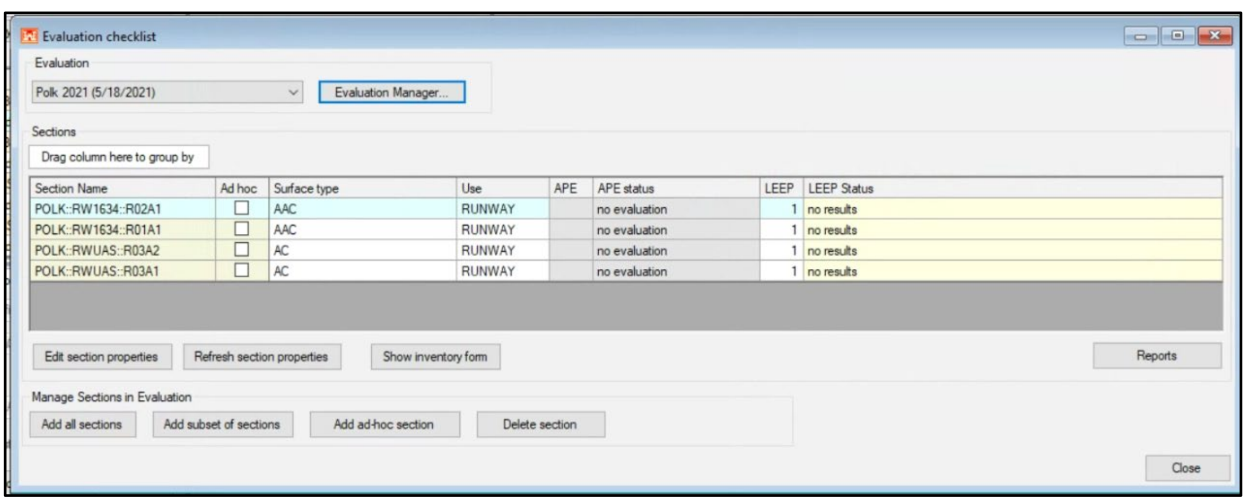
7-3 RIGID PAVEMENT (K) EVALUATION PROCEDURE.

The Westergaard (k) evaluation procedure applies to rigid pavements. It analyzes the critical tensile stresses produced within the slab by the vehicle loading. It uses layer properties determined from in situ measurements to compute allowable loads for a selected number of aircraft passes, allowable passes at a specified load, and the Pavement Classification Number (PCN). When the pavement structure cannot support the defined pass level and aircraft load, determine overlay requirements to strengthen the pavement when desired. The following paragraphs present a step-by-step procedure for evaluating a pavement section using the Pavement-Transportation Computer Assisted Structural Engineering (PCASE) Airfield Pavement Evaluation (APE) module that implements the CBR criteria. Repeat Steps 2 through 5 of this process for each section evaluated. More detailed information on PCASE is available in the PCASE *User Manual*.

7-3.1 Step 1 – Create a New Evaluation.

Open the PCASE Evaluation Checklist to create a new evaluation using the Evaluation Manager. Define the Service, climate data, evaluation traffic, and rigid failure criteria for the evaluation, then assign the inventory sections included in the evaluation.

Figure 7-1 Evaluation Checklist



7-3.2 Step 2 - Input Pavement Layers and Thickness.

Open the APE module, edit the default layer structure, and enter the pavement thickness. Determine the in-place thicknesses of PCC pavement to the nearest 0.25 inch and underlying unbound layers to the nearest inch by testing or from construction data when testing is not possible. Layer thickness testing can include measurements from coring, DCP, soil boring, GPR, or a combination of these tests. The number of tests required varies based on the area and use of the pavement as well as the uniformity of the structure. When the layer thicknesses vary for a given section, evaluate the section using different models that replicate field observations, but only report the controlling evaluation for the facility. Repeat this process for each section.

7-3.2.1 Stabilized Base Equivalent Thickness

When a pavement structure contains a stabilized base layer, determine the modulus of elasticity and thickness of the stabilized layer. The modulus of elasticity of the stabilized layer is more difficult to determine than the PCC layer. If the stabilized layer is a high-quality lean concrete or cement-stabilized layer, assign it a modulus value of 1,200,000 psi (8274 MPa). If the stabilized layer is lower quality, such as a lime or asphalt stabilized layer, assign it a modulus value of 500,000 psi (3447 MPa). Use the following equation to determine the equivalent thickness of the combined PCC and stabilized layers. Use this equivalent thickness value (h_e) with the PCC flexural strength and the modulus of subgrade reaction, k, of the material below the stabilized base layer in the analysis. PCASE automates this procedure.

Equation 7-1. Equivalent Thickness

$$h_e = 1.4\sqrt{(h_c)^{1.4} + \left(3\sqrt{\left(\frac{E_s}{E_c}\right)}\, h_s\right)^{1.4}}$$

Where:

h_e = *thickness of plain PCC equivalent to the combined PCC and stabilized base layer thicknesses, inches*

h_c = *thickness of PCC pavement, inches*

h_s = *thickness of stabilized base layer, inches*

E_c = *modulus of elasticity of PCC. The modulus values that are used in PCASE can be modified, based on engineering judgment. However, the UFC and PCASE should be consistent unless there is evidence to suggest otherwise.*

E_s = *modulus of elasticity of the stabilized base layer, psi. Estimate from Table 7-1 or calculate using deflections resulting from ASTM D1635.*

Table 7-1 E Values for Pavement Materials (Guide When E is not Available)

Material	Range (psi)	Typical Modulus (psi)
Portland cement concrete	3,000,000 – 6,000,000	4,000,000
Cement-treated bases	1,000,000 – 3,000,000	2,000,000
Soil cement materials	50,000 – 2,000,000	1,000,000
Lime-fly ash materials	500,000 – 2,500,000	1,000,000
Granular bases	40,000 – 100,000	60,000
Stiff clay	7,600 – 17,000	12,000
Medium clay	4,700 – 12,300	8,000
Soft clay	1,800 – 7,700	5,000
Very soft clay	1,000 – 5,700	3,000

7-3.2.2 Poisson's Ratios of Pavement Materials

Table 7-2 shows typical values for Poisson's ratios for different pavement materials.

Table 7-2 Poisson's Ratios for Pavement Materials

Material	Range	Typical Value
Hot mix asphalt	0.30 – 0.40	0.35
Portland cement concrete	0.15 – 0.20	0.15
Untreated granular base	0.30 – 0.40	0.35
Cement-treated granular base	0.10 – 0.20	0.15
Cement-treated fine soils	0.15 – 0.35	0.25
Lime-stabilized materials	0.10 – 0.25	0.20
Lime-fly ash mixtures	0.10 – 0.15	0.15
Loose sand or silty sand	0.20 – 0.40	0.30
Dense sand	0.30 – 0.45	0.35
Fine-grained soils	0.30 – 0.50	0.40
Saturated soft clays	0.40 – 0.50	0.45

7-3.3 Step 3 – Input the PCC Flexural Strength.

Enter the flexural strength for each PCC layer using the guidance below. Repeat the process for each section

7-3.3.1 PCC Flexural Strength, M_R Based on Testing.

Determine the representative M_R value using the results of split tensile tests or by conducting flexural strength beam tests. The M_R value used for each section in the evaluation is the arithmetical mean of all M_R values as described in Appendix A. Do not discard high or low results unless it is established that results were erroneous because the sample was defective or due to incorrect test procedures. In special instances the evaluating engineer may use a slightly lower or higher value that is more representative of existing conditions. Round the flexural strength to the nearest 5 psi (0.03 MPa), limit the maximum flexural strength for individual tests to 850 psi (5.9 MPa), and the average flexural strength to 800 psi (5.5 MPa) when reporting physical property data (PPD) and modeling.

7-3.3.2 PCC Flexural Strength, M_R Based on Construction Data.

For evaluations based on design or construction data, the representative M_R value is the arithmetical mean of the M_R values obtained in the construction-control beam tests. Disregard small changes in mix design necessary during construction to obtain the design strength when selecting representative M_R values. However, if there is a design strength change that necessitated a change in mix design, consider this change and a representative M_R value obtained for each facility for which the design strength was changed.

7-3.3.3 PCC Flexural Strength, M_R When No Data is Available.

When there is no test, design, or construction data available for an evaluation, assume a 650-psi (4.5-MPa) flexural strength when probability of construction quality control is high and 600 psi (4.1 MPa) when it is not.

7-3.4 Step 4 – Input Soil Layer Strength (k) Values.

Determine the subgrade and overlying subbase and base courses strengths by means of plate bearing tests described in CRD-C655, *Standard Test Method for Determining the Modulus of Soil Reaction*, and ASTM D1196, *Standard Test Method for Nonrepetitive Static Plate Tests of Soils and Flexible Pavement Components for Use in Evaluation and Design of Airport and Highway Pavements*, or DCP tests described in TM 3-34.48-2, Appendix G. Use construction data in conjunction with testing or when testing is not possible. The test results from an individual test pit or from multiple DCP test are seldom uniform; therefore, analyze the data carefully as described in Chapter 3 to determine reasonable k values to use for the evaluation.

7-3.4.1 Determining Representative k Values.

Compute an average k value for each pavement section, limiting the maximum k value to 500 psi/inch (13,840 grams per cubic centimeter) for evaluations. When the average k value exceeds 200 psi/inch (5,536 grams per cubic centimeter), round down to the nearest 25 psi/inch (692 grams per cubic centimeter). When it is less than 200 psi/inch (5,536 grams per cubic centimeter), round down to the nearest 10 psi/inch (277 grams per cubic centimeter). When test results are considerably higher or lower than the average of most values, conduct a thorough study of foundation conditions to determine whether the test was erroneous or whether the foundation is non-uniform. If the test is erroneous, discard the unusually high or low value. If the foundation is non-uniform, conduct more testing to select a representative k value. Do not make a saturation correction for k values since the material has likely reached an equilibrium moisture content.

7-3.4.2 Determining k Values with Plate Bearing Tests.

The plate bearing test procedures as described in CRD-C 655 and ASTM D1196 are the preferred methods to determine k values. However, existing pavement must be removed to create a test pit to conduct a plate bearing test. Operational considerations typically limit the ability to do a plate bearing test during an evaluation but will be used when the geotechnical work is for a specific project design.

7-3.4.3 Estimating k Values with DCP Tests.

When operational limitations prevent performing a plate bearing test for an evaluation, use the DCP test discussed in Chapter 3. The CBR is correlated to the k value for each layer using Figure 7-2 (based on Equations 7-2 through 7-4) and these values are used to determine the effective k at the bottom of the slab as described in the effective k procedure below. When performing DCP testing in conjunction with HWD testing, the volumetric k from HWD testing can be compared with the effective k derived from the DCP test as a checkpoint to determine the reasonableness of the k value. Note that k values derived from either of these procedures should be used with caution since CBR, volumetric k, and k values derived from plate bearing testing are fundamentally different soil engineering properties with poor correlations for many real-world cases.

Equation 7-2. CBR to k Coarse Grained Non-Plastic Subgrade Material

$$k = 129.58076 * CBR^{0.5} - 5.49306 * CBR - 242.93236$$

Equation 7-3. CBR to k Fine Grained Subgrade Material with LL < 50

$$k = 60.2282 * CBR^{0.5} + 2.1854046 * CBR - 11.245482$$

Equation 7-4. CBR to k Fine Grained Subgrade Material with LL > 50

$$k = 20 * CBR$$

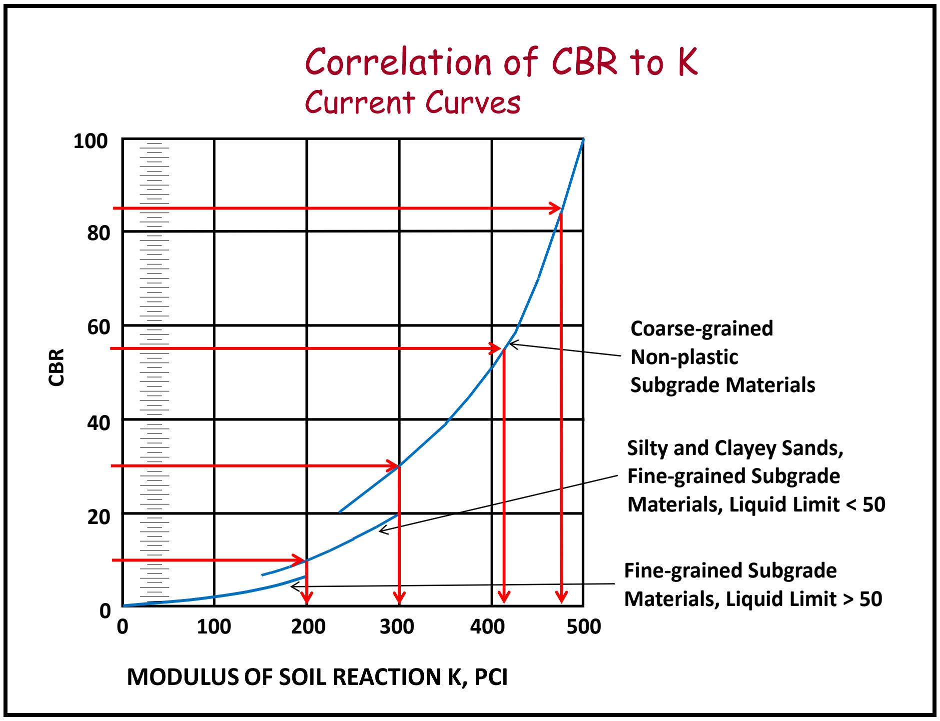
Where:
k = *Modulus of Subgrade Reaction*
CBR = *California Bearing Ratio*

Source: ERDC/GSL TR-12-20

7-3.4.4 Procedure to Determine Effective k Values.

Determine the k value for each layer in the pavement structure by inputting the CBR results from the DCP test into Equation 7-2, 7-3, or 7-4, depending on the material type. Figure 7-2 is derived from these equations. Determine the effective k for each layer using Figures 7-2 through 7-8. Compare the effective k for each layer to the k for each layer determined by Figure 7-2 and use the lower value for computing the effective k of the next layer. This process is automated in the PCASE APE module.

Figure 7-2 Estimation of K values from CBR



7-3.4.5 Determine Layer Structure and CBR Values from DCP Test.

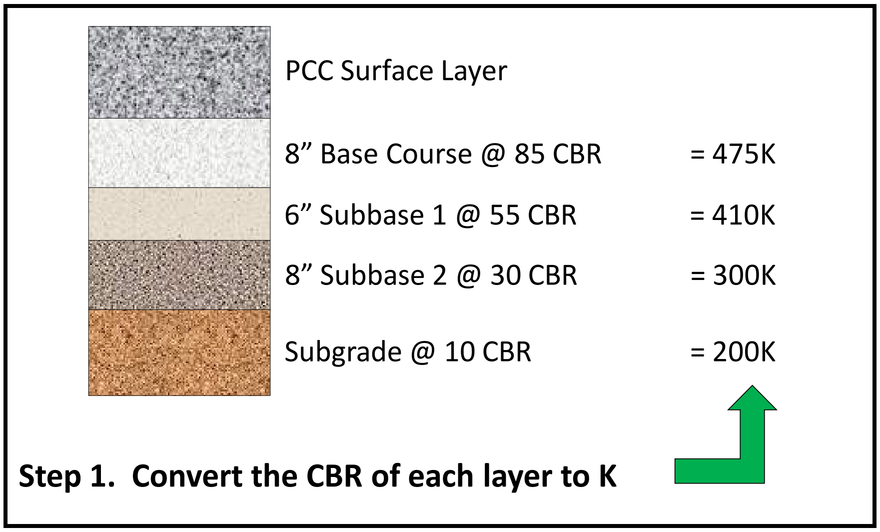
Using the following layer structure, determine k and effective k values.

- Base course is 8 inches (203 millimeters) thick with a CBR = 85
- Subbase 1 is 6 inches (152 millimeters) thick with a CBR = 55
- Subbase 2 is 8 inches (203 millimeters) thick with a CBR = 30
- Subgrade CBR = 10

7-3.4.5.1 Step 1 - Determine k Value for Each Layer Using Figure 7-2.

Use Figure 7-2 to determine the k values for the subgrade, subbase 1, subbase 2, and base course to compare with tentative k values at the top of each of these layers as shown in Figure 7-3.

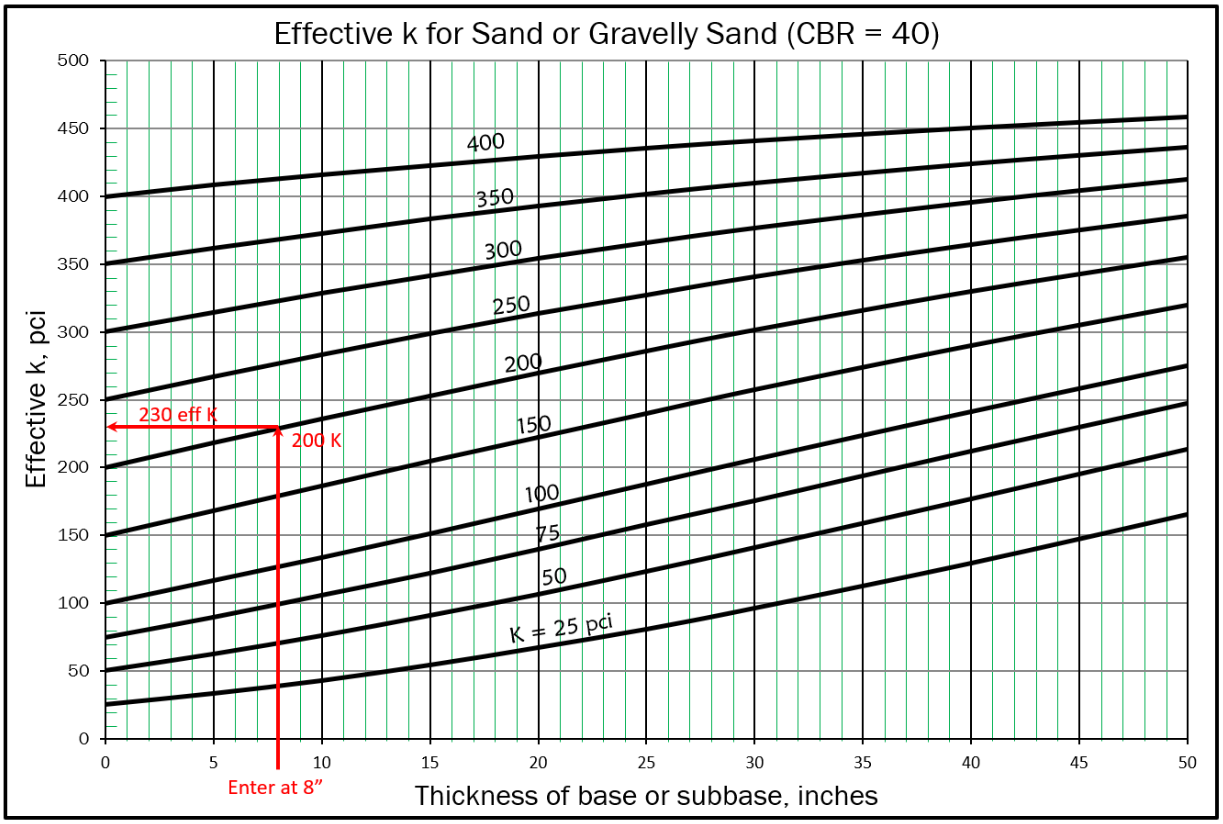
Figure 7-3 CBR to k Conversion Profile



7-3.4.5.2 Step 2 - Determine Effective k for Subbase 2.

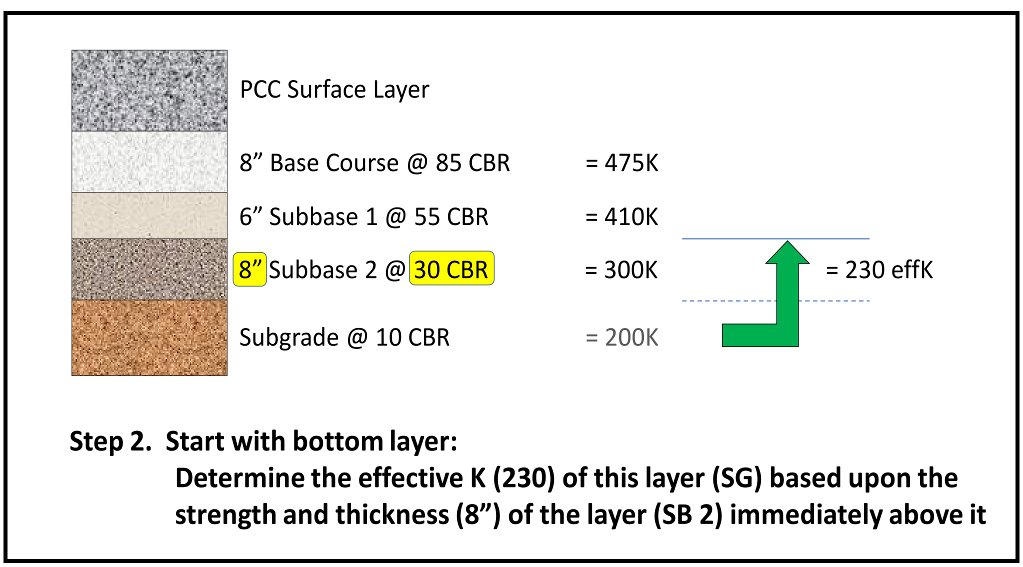
Use Figure 7-4 to determine effective k for layers with CBR values < 50. Results for this step are show in Figure 7-5.

Figure 7-4 Determine Effective k for Subbase 2



For base or subbase layers with CBR < 50 / (K < 399)

Figure 7-5 Effective k Subbase 2 Profile



7-3.4.5.3 Steps 3 & 4 - Determine Effective k for Subbase 1.

Use Figure 7-6 to determine effective k for layers with CBR values 50 ≤ CBR < 70. Results for this step are show in Figure 7-7.

Figure 7-6 Determine Effective k for Subbase 1

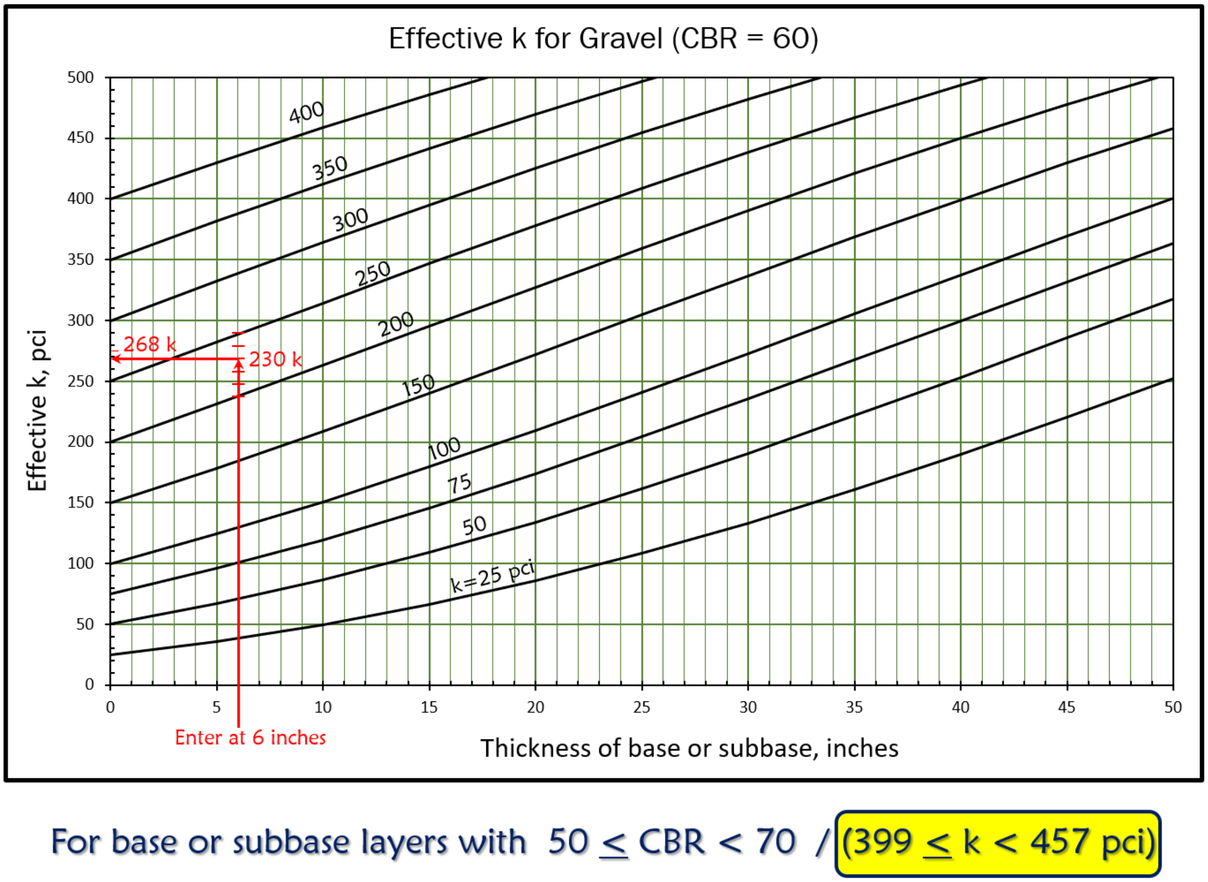
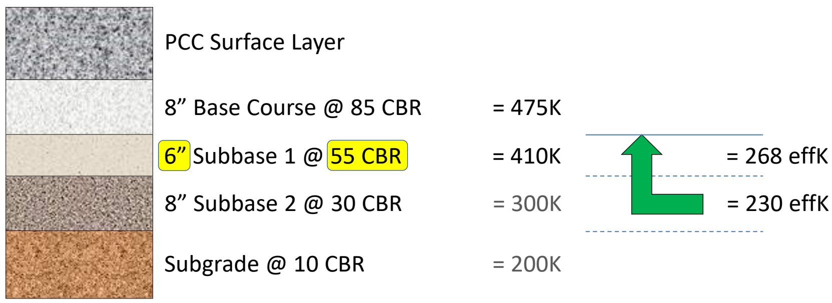


Figure 7-7 Effective k Subbase 1 Profile



Step 3. Compare the k values (300 and 230) at the top of SB 2 and continue upward with the lowest one (230):
Step 4. Determine the effective K (268) of this layer based upon the strength and thickness (6") of the layer (SB 1) above it

7-3.4.5.4 Steps 5 & 7 - Determine Effective k for Base Course.

Use Figure 7-8 to determine effective k for layers with CBR values 70 ≤ CBR <90. Results for this step are shown in Figure 7-9. Use the lower of the measured k or computed effective k. In this case, use 435 pci to evaluate the structure.

Figure 7-8 Effective k Base Course Determination

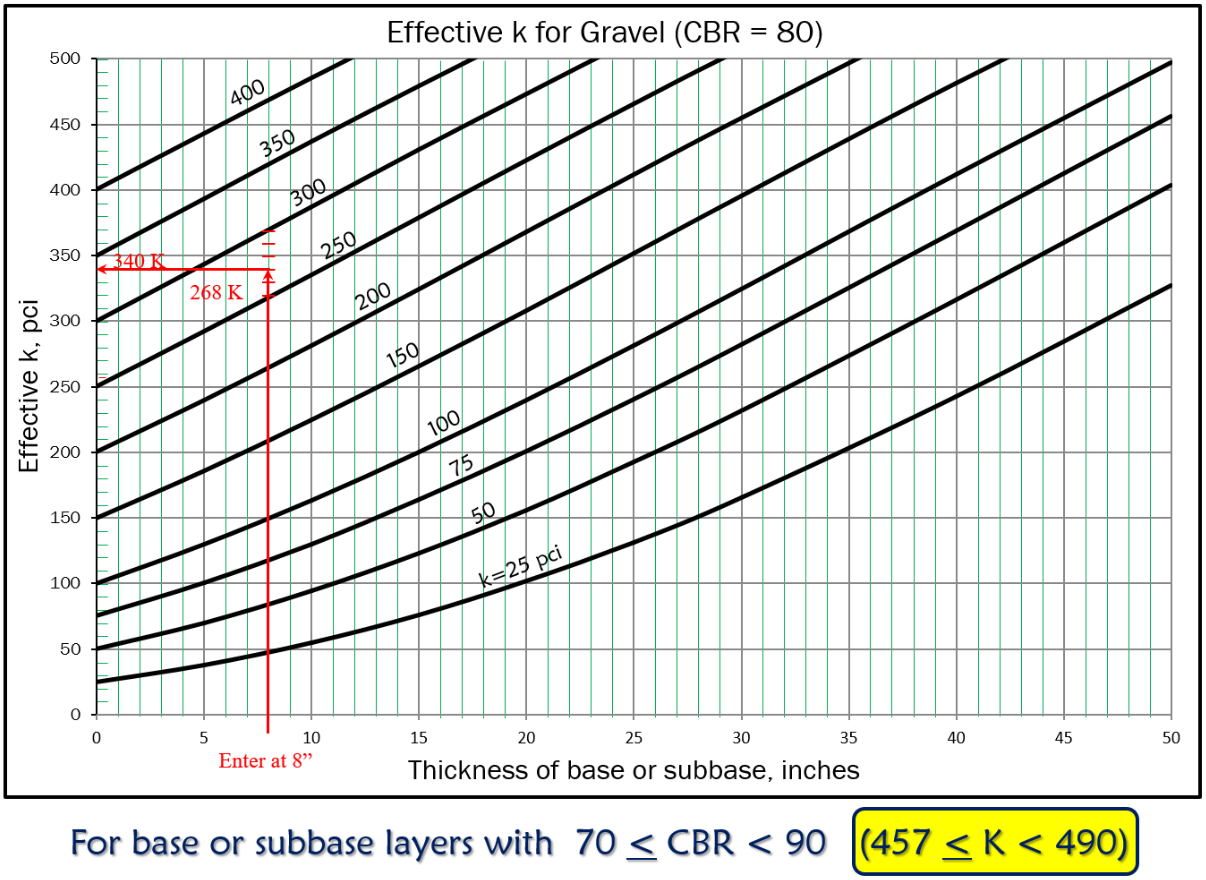
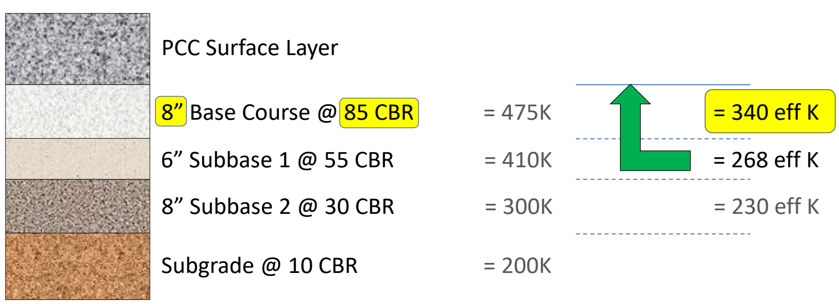


Figure 7-9 Effective k Base Course Profile



Step 5. Compare the K values (410 and 268) at the top of SB 1 and continue upward with the lowest one (268):

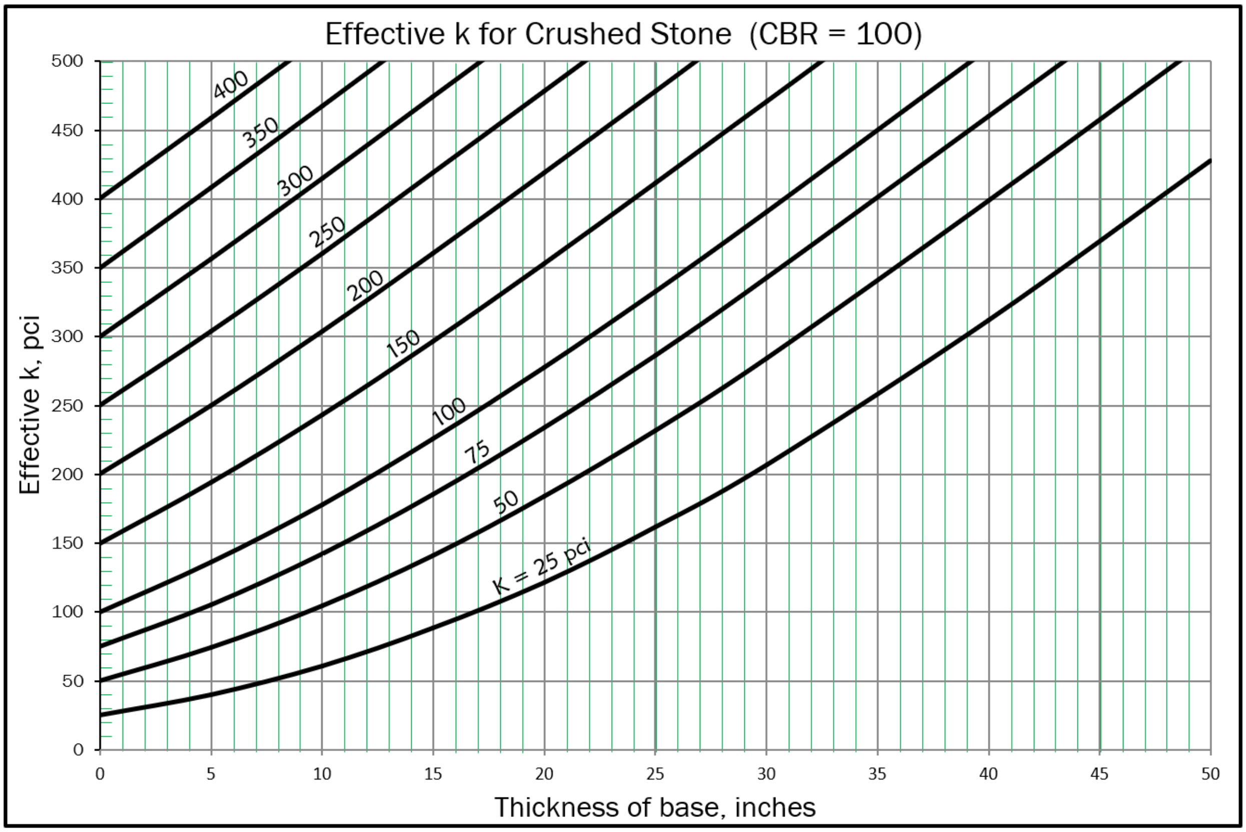
Step 6. Determine the effective K (340) of this layer based upon the strength and thickness (8") of the layer (BC) above it

Step 7. Compare the eff K values of the layers immediately beneath the PCC (in this case 475 and 340) and use the lowest eff K (340) to evaluate the PCC

7-3.4.5.5 Effective k Crushed Stone Base Course.

While not used in the example problem, the set of curves in Figure 7-10 are used to determine the effective k when the material is crushed stone (CBR 100).

Figure 7-10 Effective k Crushed Stone Base Course Determination



For base layers with CBR $\geq$ 90 / (K $\geq$ 490)

7-3.5 Rigid Pavement Analysis.

7-3.5.1 Westergaard (k) Evaluation Procedure.

Once the thickness, flexural strength, and k values are selected for the respective layers, use these values to determine the critical tensile stresses produced within the slab by the vehicle loading based on the Westergaard procedure and assuming constant tire pressure. Note that results using the current criteria will differ from earlier criteria given that the procedure now assumes constant pressure rather than constant contact area used in past versions of this UFC. The objective of the analysis is to determine the allowable load and allowable passes for the structure. PCASE automates the analysis procedure outlined in this chapter and in Appendix D. The procedure for generating aircraft curves using the current criteria is in TSPWG M 3-260-03.02-19.

7-3.5.2 Procedure for Determining Allowable Gross Load (AGL).

The inputs for this analysis are the traffic mix with the load and number of passes for each vehicle in the mix defined, the pavement structure, and the traffic area. Determine the controlling/representative vehicle and equivalent passes based on one of the traffic analysis procedures outlined in Chapter 4. Compute the design factor based on the flexural strength and load transfer. Perform the allowable coverages calculation using the Westergaard (k) procedure to determine the free edge bending stress for the pavement structure. The bending stress is based on the controlling/representative vehicle load and the design factor. Then use the allowable coverages to compute the cumulative damage factor (CDF). If the CDF is less than 1, increase the Gross load and repeat the analysis procedure. If the CDF is greater than 1, decrease the load and repeat the analysis procedure. When CDF equals 1, use that value for the AGL. See Appendix D for details on this procedure.

7-3.5.3 Procedure for Determining Allowable Passes.

The inputs for this analysis are the traffic mix with the load and number of passes for each vehicle in the mix defined, the pavement structure, and the traffic area. Determine the controlling/representative vehicle and equivalent passes based on one of the traffic analysis procedures outlined in Chapter 4. Perform the allowable coverages calculation using the Westergaard (k) procedure with the free edge bending stress calculated for the pavement structure based on the controlling/representative vehicle load and the design factor that is computed based on the flexural strength and load transfer. See Appendix D for details on this procedure.

7-4 REINFORCED CONCRETE PAVEMENTS.

The process and data required to evaluate reinforced concrete pavements is essentially the same as those for plain concrete pavements, except the percent steel is also required.

7-4.1 Reinforcing Steel.

The reinforcing steel in a reinforced concrete pavement is normally located at or above the neutral axis of the pavement section. If the steel is below the neutral axis, it affects the determination of the flexural strength and the static modulus of elasticity in flexure. Therefore, when the reinforcing steel falls below the neutral axis in a test beam, turn the beam over and test it with the reinforcing steel above the neutral axis. The split tensile test cannot be performed on a core with reinforcing steel although it may be possible to obtain a core to test with no reinforcing steel. If the pavement is thick enough, saw the core just below the reinforcing steel and perform the split tensile test on the lower, non-reinforced portion.

7-4.2 Reinforced PCC Evaluation Procedure.

Determine the percentage of steel reinforcement S per foot of pavement cross-sectional area using Equation 7-5 then use Figure 7-11 to convert the existing reinforced pavement thickness (h_r) to an equivalent thickness (h_E) of plain concrete pavement and calculate the load-bearing capability using plain concrete equivalent thickness.

Equation 7-5. Steel Reinforcement Required

$$S = \frac{A_s}{A_p} * 100$$

Where:

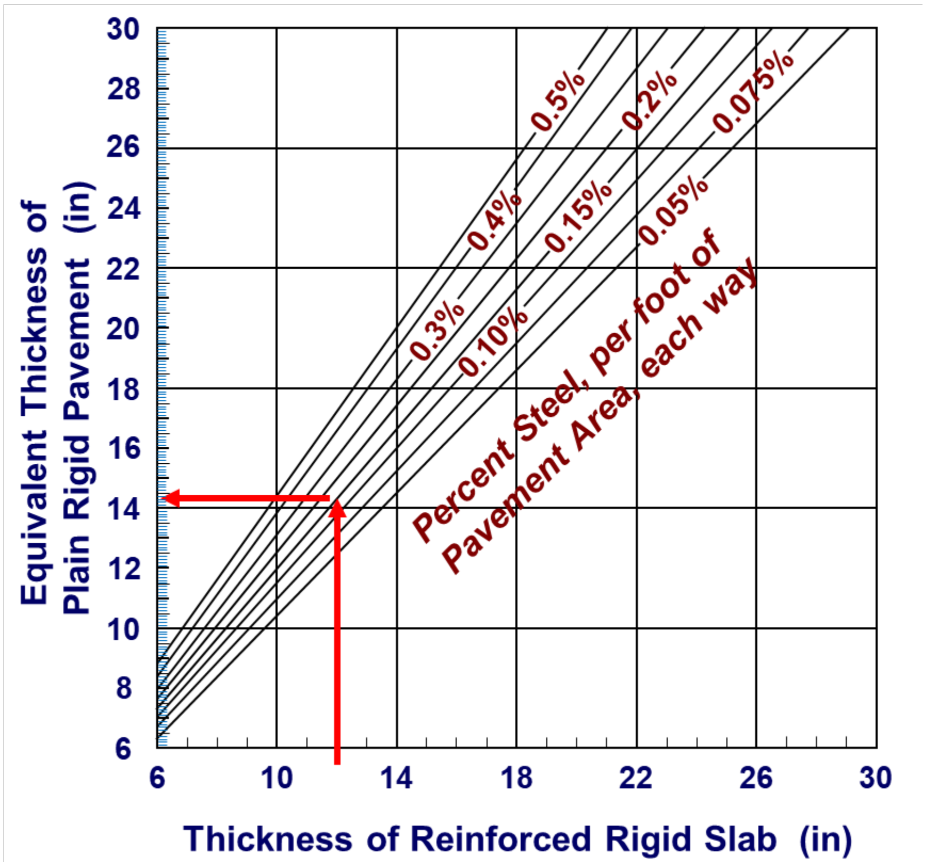
A_s = *cross-sectional area of the reinforcing steel per foot of pavement width or length, square inches*

A_p = *cross-sectional area of pavement per foot of pavement width or length, square inches*

7-4.2.1 Determine Equivalent Plain PCC Thickness of Reinforced PCC.

Compute the percent steel in both the longitudinal and transverse directions. Typically, it will be the same in both directions, but if there is a difference, use the smaller value. Next, enter Figure 7-11 with the known value of h_r, thickness of reinforced PCC pavement. Make a vertical projection and extend it until it intersects the diagonal line representing the computed value of S. Then make a horizontal projection to the left until it intersects the scale line representing the value for h_E, thickness of plain PCC equivalent that would have the same load-carrying capacity as the reinforced concrete pavement. When S is less than 0.05, h_E will equal h_r. When S is greater than 0.5, use the diagonal line representing S = 0.5 percent to determine h_E.

Figure 7-11 Reinforced to Plain PCC, Equivalent Thickness



7-4.2.2 Evaluating Reinforced PCC Overlay or with Stabilized Base.

Determining the equivalent thickness of a reinforced PCC overlay is a two-step process. First, determine the equivalent plain PCC thickness, then determine the equivalent thickness using the fully, partially, or unbonded overlay equivalent thickness calculation described in paragraph 7-4. When a reinforced concrete pavement is placed over a stabilized layer, determine the equivalent thickness of plain concrete pavement as described above using Figure 7-11, then determine the equivalent thickness h_E of the PCC and stabilized layer using Equation 7-1. See reinforced PCC equivalent thickness calculation examples in the PCASE Getting Started module and *User Guide*.

7-5 RIGID OVERLAY ON RIGID PAVEMENT.

The first step in rigid pavement structure with rigid overlay(s) is determining the equivalent thickness of the combined pavement structure. The equivalent thickness is defined as a single thickness of plain concrete pavement with the same load-carrying capacity as the combined thickness of the rigid overlay(s) and the rigid base pavement. Overlay equivalent thickness calculation examples are provided in the PCASE Getting Started module and *User Guide*.

7-5.1 Determining Equivalent PCC Thickness of a Rigid Overlay.

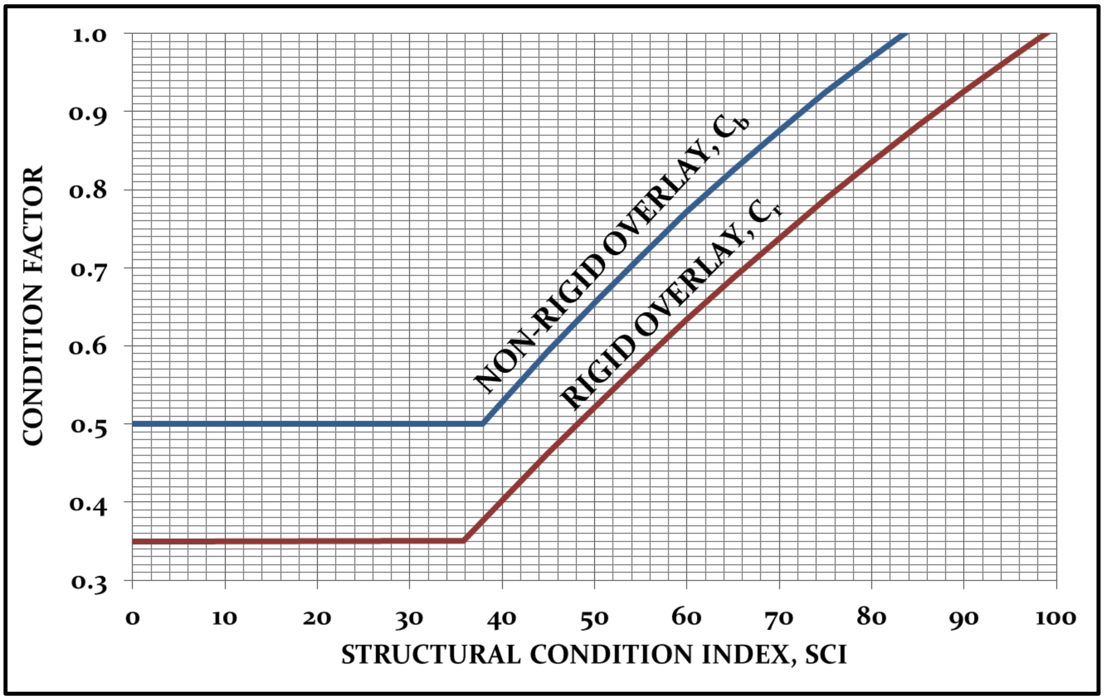
The condition of the base pavement is a key input in determining the equivalent thickness. Structural distresses in the base slab will migrate through the overlay so if the overlay pavement only has minimal structural defects (e.g., reflected longitudinal and transverse cracking as opposed to joint reflective cracking), it is an indication the base pavement is still in good condition. FWD testing or evaluation data from prior to the overlay can also help discern the condition of the base slab and strength of underlying surface layers. Start at the bottom of the structure and determine the equivalent thickness of the base pavement and overlay using the appropriate equation for the overlay type. If there is more than one overlay, use that equivalent thickness and the next overlay to determine the combined equivalent thickness. Continue this procedure with any remaining overlays. When there is variability in the base slab and overlay thicknesses across a section, use the average thickness of each layer in the section to determine the equivalent thickness.

7-5.1.1 PCC Overlay Condition Factors.

Estimate the rigid overlay condition factor (Cr) based on the current surface condition (PCI) and percent of load-related distresses that are used to compute the structural condition index (SCI). PCASE computes the Cr value, but if PCI data is not available, use the recommended Cr values below. In addition, use the values below when it is not possible to visually determine the condition of the existing base PCC slab. The relationship between the SCI and Cr is shown in Figure 7-12. An SCI of 100 indicates good condition and an SCI of 0 indicates poor condition.

- Cr = 1.00 for base PCC in very good condition. There are no structural or reflective cracks in the rigid overlay. If the condition of the base pavement cannot be determined or is unknown, do not use this value.
- Cr = 0.75 for base PCC in good condition. There are a few initial cracks in the surface PCC due to loading or reflective cracks from the base PC slabs, but no progressive cracks.
- Cr = 0.35 for badly cracked base PCC layer. Approximately 60 percent of the slabs in the overlay contain medium- or high-severity cracking or 50 percent of the slabs contain high-severity cracks.

Figure 7-12 SCI – Condition Factor Relationship



7-5.1.2 Partially Bonded PCC Overlays.

If the overlay slab was cast directly on the base slab and no effort was made to break the bond between the overlay and the base pavement by means of a tack coat, sand, paper, bituminous concrete, or other materials placed between the overlay and the base pavement, treat it as a partially bonded overlay. Compute the equivalent thickness h_E of the combined partially bonded overlay and base pavement using Equation 7-6.

Equation 7-6. Equivalent Thickness for Partially Bonded PCC Overlays

$$h_E = \sqrt[1.4]{(h_o)^{1.4} + Cr\,(h_b)^{1.4}}$$

Where:

h_o = thickness of rigid overlay pavement, inches
C_r = coefficient representing condition of rigid base PCC layer
h_b = thickness of rigid base pavement, inches

7-5.1.3 Unbonded PCC Overlays.

If a bond-breaker layer was used between the rigid overlay and the rigid base pavement, treat it as an unbonded overlay. Compute the equivalent thickness h_E of the combined unbonded overlay and base pavement using Equation 7-7. Do not give any thickness credit to the bond breaker layer if it is less than 4 inches (102 millimeters). If the thickness of the bond breaker is greater than 4 inches (102 millimeters), evaluate it as a composite pavement.

Equation 7-7. Equivalent Thickness for Unbonded PCC Overlays

$$h_E = \sqrt{(h_o)^2 + Cr\,(h_b)^2}$$

7-5.2 Structural Analysis Using the h_E Value.

After determining the h_E value using Equation 7-6 or 7-7, determine the weighted average flexural strength (R) of the overlay and base pavement using Equation 7-8 and use these values to determine the load capability the same as a plain PCC pavement.

Equation 7-8. Weighted Average Flexural Strength

$$R = \frac{h_o(R_o) + h_b(R_b)}{h_o + h_b}$$

Where:
h_o = thickness of overlay
R_o = flexural strength of overlay
h_b = thickness of base slab
R_b = flexural strength of base slab

7-6 FLEXIBLE OVERLAY ON RIGID PAVEMENT.

First determine if the flexible (e.g., asphalt) overlay meets the structural design (minimum thickness) requirements in UFC 3-260-02. Thin overlays used to correct surface defects are not given structural credit. When the overlay meets minimum thickness requirements, the procedures outlined below recommend evaluating the pavement as both a rigid and flexible structure and using the method that yields the higher AGL. Use the procedures in Chapter 6 and treat the base slab as a base course for the flexible analysis. For the rigid pavement analysis, determine the equivalent thickness h_E of the combined pavement structure. The equivalent thickness is defined as a single thickness of plain concrete pavement with the same load-carrying capacity as the combined thickness of the flexible overlay(s) and the rigid base pavement. Overlay equivalent thickness calculation examples are provided in the PCASE Getting Started module and *User Guide*.

7-6.1 Determining Equivalent PCC Thickness of a Flexible Overlay.

Just as with a rigid overlay, the condition of the base pavement is a key input in determining the equivalent thickness. In addition, the degree of cracking allowed in the base slab is also required for the equivalent thickness computation. Start at the bottom of the structure and determine the equivalent thickness of the base pavement and overlay using the appropriate equation for the overlay type. If there is more than one overlay, use that equivalent thickness and the next overlay to determine the combined equivalent thickness. Continue this procedure with any remaining overlays. When there is variability in the base slab and overlay thicknesses across a section, use the average thickness of each layer in the section to determine the equivalent thickness.

7-6.1.1 Flexible Overlay Condition Factor.

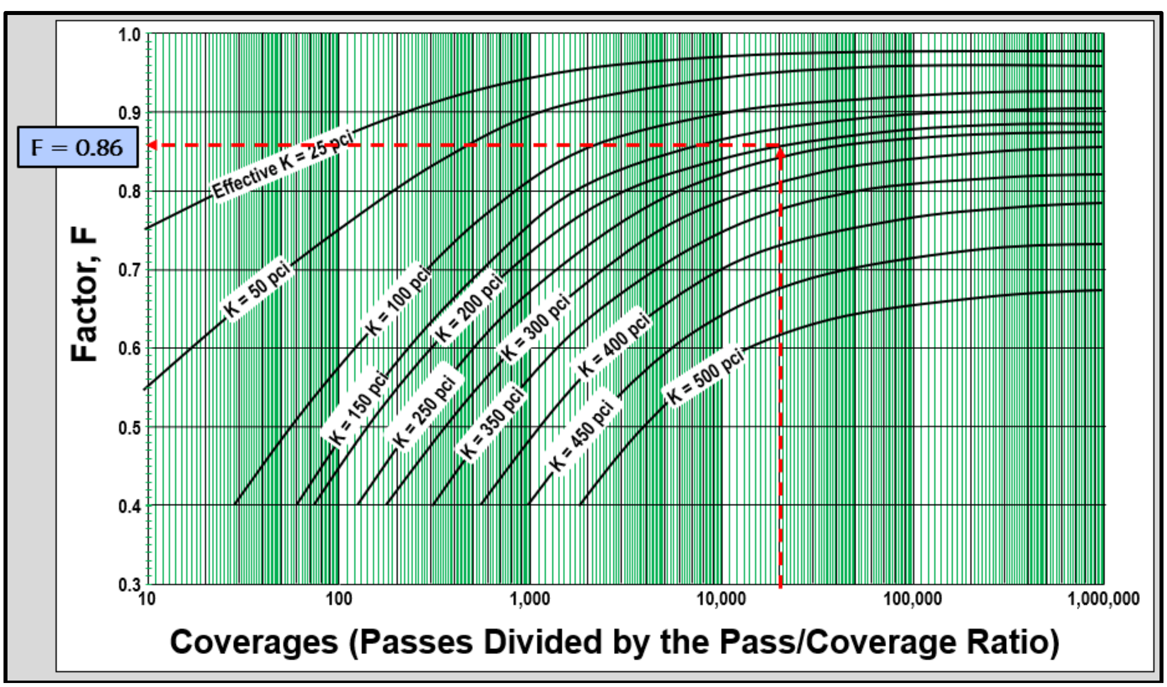
Estimate the flexible overlay condition factor (C_b) based on the current surface condition (PCI) and percent of load-related distresses, which are used to compute the SCI. PCASE will compute the Cb value, but if PCI data is not available or if it is not possible to visually determine the condition of the existing base PCC slab, use the recommended Cb values below. The relationship between the SCI and Cb is shown in Figure 7-11. An SCI of 100 indicates good condition and an SCI of 0 indicates poor condition:

- C_b = 1.0 Use if there are no reflective distresses on the asphalt surface and it is positive that the base pavement is in good condition.
- C_b = 0.8 Use if there are only joint reflective distresses on the asphalt surface.
- C_b = 0.5 Use if there are reflective cracks in addition to reflective joints.

7-6.1.2 Controlled Cracking with F Factor.

The F factor in the equivalent thickness Equation 7-9 defines the degree of cracking allowed in the rigid base pavement during the life of the pavement. It is dependent on the modulus of subgrade reaction k (measured or computed directly under the pavement) and traffic intensity in terms of coverages (passes/pass to coverage ratio of the critical aircraft). PCASE computes the F factor based on the relationship shown in Figure 7-13. The maximum k value used to compute the h_E value is 500 pci. The equivalent thickness equation can yield h_E values greater than the combined thickness of h_b + t for some F factors. If this occurs, use the h_b + t value for h_E.

Figure 7-13 SCI – Condition Factor Relationship



7-6.1.3 Flexible Overlay on Rigid Pavement – Rigid Analysis.

Use the measured pavement thicknesses for the base slab and overlay and the Cb and F values described above to calculate the equivalent thickness h_E in Equation 7-9. The equivalent thickness, h_E, is defined as the thickness of a plain concrete pavement having the same load-carrying capacity as the combined overlay section. Use h_E and the flexural strength of the base slab to determine the load capability using the same procedure as a plain PCC pavement.

Equation 7-9. Equivalent Thickness for Flexible Overlay

$$h_E = \frac{1}{F}(0.33\,t + C_b\,h_b)$$

Where:

t = *thickness of non-rigid overlay pavement, inches*
h_b = *thickness of rigid base pavement, inches*
C_b = *coefficient representing the condition of the rigid base*
F = *a factor which controls the degree of cracking in the rigid base pavement (Several F Factor curves are included in TSPWG M 3-260-03.02-19.)*

7-6.2 Flexible Overlay on Rigid Pavement – Flexible Analysis.

The flexible pavement evaluation method uses the procedures in Chapter 6 and considers the flexible overlay on rigid pavement as a flexible pavement, with the rigid base pavement assumed to be a high-quality base course with a CBR of 100 and the subbase and subgrade characterized by their respective CBR values.

7-6.3 Other Considerations.

7-6.3.1 PCC Material Property Limitations.

When conditions indicate PCC or soil properties are not typical, modify the evaluation accordingly. Consider the possible factors that are influencing the material properties such as those outlined below. Discuss the effect that any of the following factors may have on the evaluation of the pavement in the narrative portion of an evaluation report.

- High moisture absorption and shrinkage of the PCC
- High variations in daily ambient air temperature
- Wide variation in the flexural strength within a given pavement section
- Heterogeneous subgrade, base, or moisture conditions resulting in wide variations in modulus of subgrade reaction values
- Non-rigid overlays (bituminous concrete and flexible overlay) that do not meet design requirements for flexible pavements
- Poor PCC joint load transfer (e.g., NDT deflection ratios)

7-6.3.2 Joint Load Transfer.

As stated previously (see paragraph 5-3.9.2), rigid pavement criteria assume 25 percent joint load transfer. If test data indicate this assumption is not valid, adjust the percent load transfer to reflect what was measured.

7-6.3.3 Asphalt Overlay Quality.

The evaluation procedure outlined above assumes the asphalt concrete meets UFC 3-260-02 design requirements. Determine whether surface cracking is the result of inadequate strength in the overlay or reflective cracking from joints and structural defects in the rigid base pavement. When the surface condition indicates the quality assumption is not valid, it may be necessary to conduct additional tests on the asphalt overlay as outlined in Chapter 3 to determine whether it meets design requirements. Construction records may also be used to determine the quality of the overlay materials. When the asphalt concrete does not meet design requirements, discuss the consequences, such as rutting and raveling, in the narrative portion of the evaluation report. Raveling can be a sign of a poor mix design or construction issues. Rutting or surface cracking can be signs of inadequate strength or asphalt compaction.

7-6.3.4 Comparing the Rigid and Flexible Analysis Procedure.

Typically, the rigid overlay evaluation method yields higher allowable gross weights than the flexible procedure and will be used for reporting purposes. However, when the flexural strength of the rigid base pavement is less than 400 psi (2.8 MPa) or the k value of the foundation is greater than 200 pci, the flexible pavement evaluation method can yield the higher allowable gross weight at a selected pass level. Therefore, especially when the test results indicate that the flexural strength of the rigid base pavement is less than 400 psi (2.8 MPa) or the k value is greater than 200 pci, evaluate the flexible overlay on rigid pavement by both methods to determine which yields the higher allowable gross weight for a selected pass level.

7-6.4 Asphalt Additional Overlay Thickness.

Use Equation 7-10 to determine the additional asphalt overlay thickness required to support aircraft operations.

Equation 7-10. Additional Overlay Thickness Calculation

$$t_{ao} = 3 \cdot [F \cdot h_d - h_E]$$

Where:

t_{ao} = *additional overlay required, inches*

h_d = *new pavement PCC layer design thickness, inches*

h_E = *equivalent thickness of existing PCC base and overlay, inches*

F = *a factor which controls the degree of cracking in the rigid base pavement, see paragraphs 7-6.1.2 and 7-6.1.3.*

7-7 RIGID OVERLAY ON FLEXIBLE PAVEMENT.

The flexible pavement layer (e.g., asphalt concrete) in a rigid overlay on flexible pavement structure is treated as a base course for the rigid overlay. Determine the k value on the surface of the flexible pavement with the plate-bearing test subject to the limitations described in paragraph A-5.9 or using the effective k procedure described in paragraph 7-3, but in no case use a k value greater than 500 pci.

7-7.1 Rigid Overlay on Flexible Pavement Analysis.

Select representative thickness values for the rigid overlay and other layers in the structure, determine the flexural strength of the rigid overlay, and modulus of subgrade reaction (k) on the surface of the existing flexible pavement as described in paragraph A-5.9.2. Evaluate the rigid overlay on flexible pavement using the same procedures used for plain PCC pavement on a base course.

7-8 COMPOSITE PAVEMENT.

A composite pavement consists of three or more pavement layers. This section specifically addresses the situation in which there is a rigid layer over asphalt over a rigid base slab. The analysis procedure depends on whether the asphalt layer is less than or greater than 4 inches (102 millimeters).

7-8.1 Asphalt Layer Less than 4 Inches (102 Millimeters).

When the thickness of the asphalt layer is less than 4 inches (102 millimeters), treat the rigid surface as an unbonded overlay, with the thickness of the asphalt layer assumed to be a bond-breaking layer. Determine the layer thicknesses of each layer in the structure, the equivalent thickness of the asphalt and base pavement, the flexural strength of the rigid overlay and base pavement, and the k value of the foundation materials beneath the rigid base pavement. Estimate the condition of the base slab as described in paragraph 7-5.1.1 and use Equation 7-7 for an unbonded overlay to determine h_E.

7-8.2 Asphalt Layer Greater than or Equal to 4 Inches (102 Millimeters).

When the thickness of the asphalt between the rigid pavements is greater than or equal to 4 inches (102 millimeters), treat the surface layer as rigid pavement on a base. Determine thickness of each layer in the structure, the flexural strength of the rigid overlay, and the k value on the surface of the asphalt layer beneath the rigid surface layer. Use the procedure described in paragraph 7-7.1 for a rigid overlay on a flexible pavement to determine the effective k.

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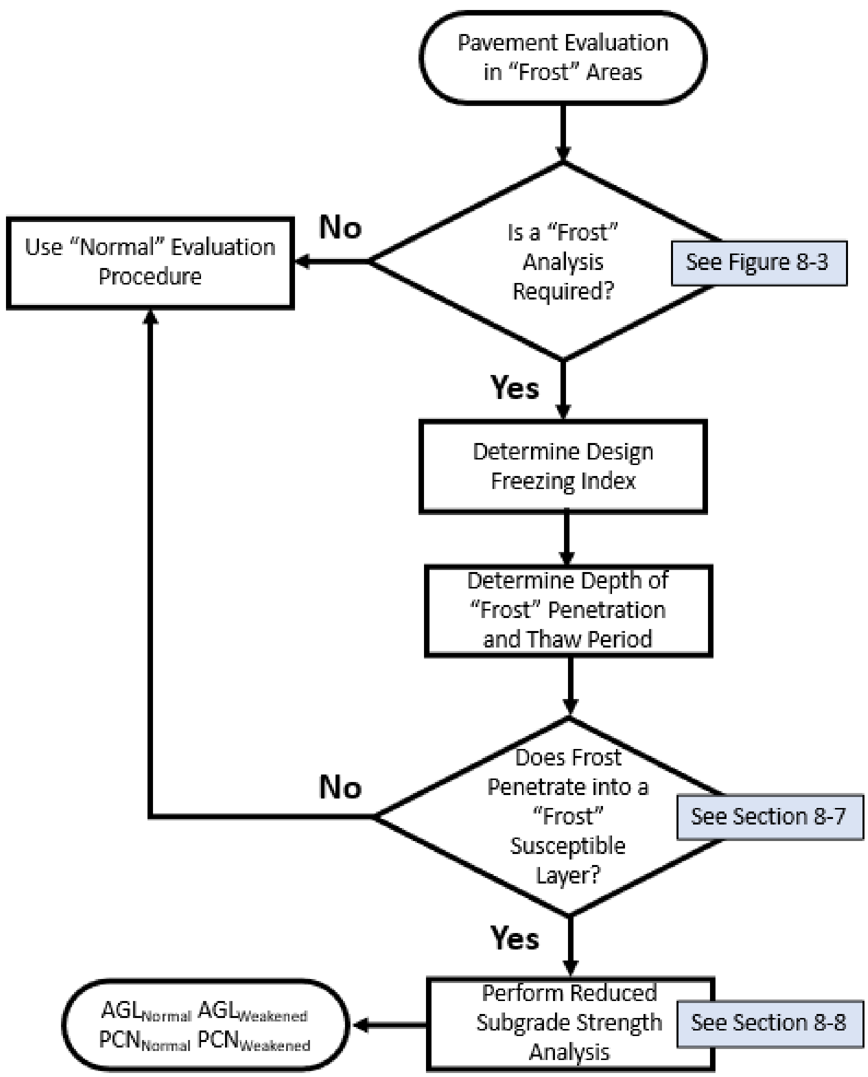
CHAPTER 8 PAVEMENT FROST EVALUATION

8-1 PAVEMENT FROST EVALUATION PROCESS.

The term frost evaluation is used to describe the process of determining if a pavement is susceptible to the detrimental effects of frost action and, if so, analyzing the pavement structure to determine the impact of these effects on the load-carrying capacity of the pavement during the thaw period. Figure 8-1 describes the overall process. The first step is to determine if a frost analysis is warranted. If it is, compute the design freezing index (DFI) using climate data from the WorldIndex database, then determine the depth of frost penetration and start and duration of the freezing season. If the depth of frost penetration does not penetrate into a layer of frost-susceptible material, use the normal evaluation procedure. If it does, then use the reduced subgrade strength procedure. Terms used in this chapter that are not explained in the text are defined in the glossary in Appendix F.

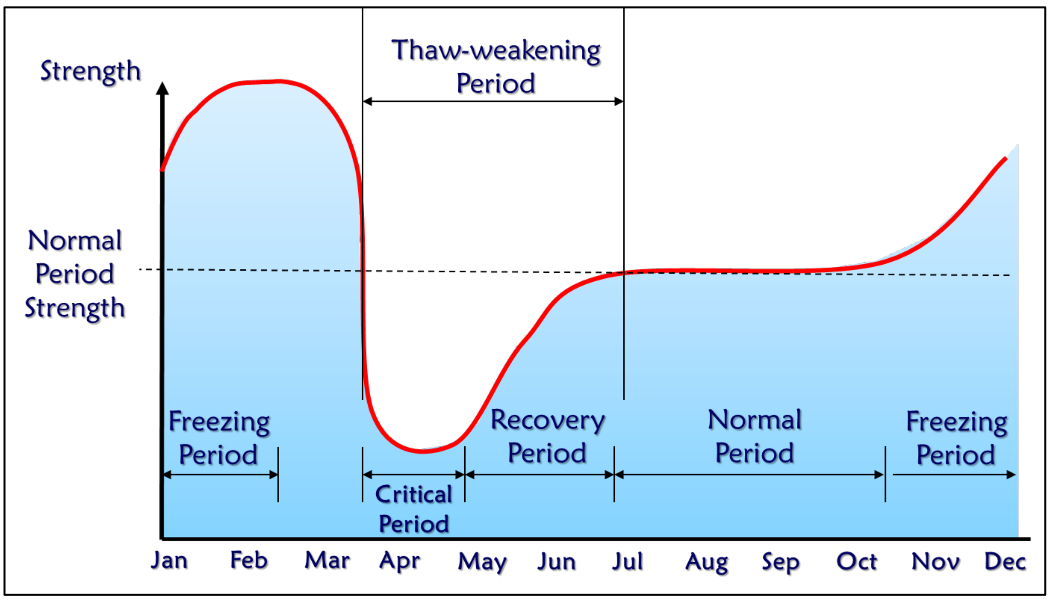
Figure 8-1 Pavement Frost Evaluation Process



8-2 FROST ACTION.

Frost action is a general term for freezing and thawing of moisture in materials and the resultant effects on these materials, the overall structure, and adjacent structures. Detrimental effects can occur when a pavement structure is exposed to freezing temperatures, has frost-susceptible soils, and has a source of water near the freezing front. When these conditions exist, water is drawn upward to the freezing front, creating ice lenses, which can result in pavement frost heave that increases the roughness of the pavement surface. As the ice melts, water does not readily drain or redistribute itself, saturating and thus weakening the soil and reducing the pavement structure's load-bearing capacity during the thaw period. The weakened pavement transitions back to a normal state as the soil drains, pore water pressure dissipates, and soil reconsolidates. Figure 8-2 provides a conceptual graphic of the freeze-thaw cycle.

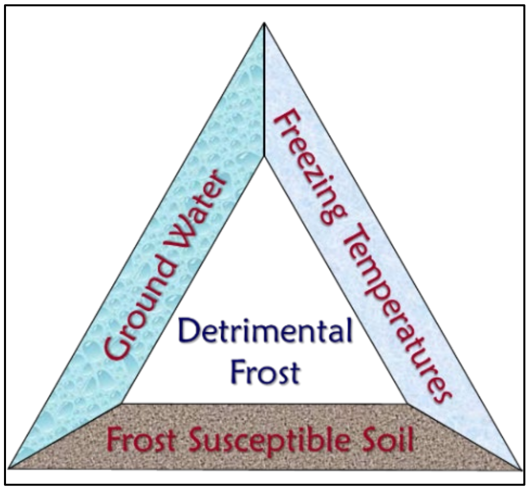
Figure 8-2 Freeze-thaw Cycle



8-3 WHEN TO PERFORM A FROST EVALUATION.

A frost evaluation is not warranted unless all the elements that increase the risk of detrimental frost action exist as shown in Figure 8-3. Additional soil sampling and laboratory testing is required to determine the moisture content, dry density, and frost susceptibility of the soil. This information is used to determine the depth of frost penetration and potential loss of load-carrying capability.

Figure 8-3 Detrimental Frost Action



8-3.2 Temperature.

The average daily air temperature at the location must remain below freezing for sufficient time for the freezing front to extend into frost-susceptible soil layers. If it does not, frost analysis is not warranted.

8-3.3 Frost-Susceptible Soil.

The frost susceptibility of soil is defined by the percent of soil finer than a #200 (.075 mm) sieve by weight and the percent finer than 0.02 mm by weight. This determines the frost group of the soil, which is described in more detail in paragraph 8-6.1.2. Non-frost-susceptible soils have a minimal risk of detrimental frost action.

8-3.4 Ground Water.

Generally, if the water table is below 10 feet (3 meters), detrimental frost action is not a problem.

8-3.5 Evidence of Detrimental Frost Effects.

Even when it appears all the elements shown in Figure 8-3 and described above exist, check for evidence of damage due to detrimental frost action. If the conditions described below do not exist, frost analysis may not be required.

- Pavement heave and cracking
  - Differential heave caused by swelling of materials in subgrade and base due to frost action
  - Differential settlement caused by soil reconsolidation after heave
- Excessive cracking of rigid pavement
  - Pumping along cracks and joints
  - Durability (D) cracking

    - Excessive joint and crack spalling
    - Longitudinal cracking or other load-related distresses in non-traffic areas
- Accelerated cracking of flexible pavement
    - Alligator cracking or other load-related distresses
    - Distresses located in non-traffic areas
    - Accelerated deterioration along cracks

8-3.5.1 Detrimental Frost Effects.

Detrimental frost effects include frost heave and thaw weakening. Frost heave occurs when the pavement surface is raised. It is directly associated with ice segregation and is visible evidence on the surface that ice lenses have formed in the subgrade, subbase, or base layer materials. When ice segregation occurs in a frost-susceptible soil, the soil is subsequently weakened during prolonged thaw periods that can occur during winter partial thaws and early in the spring. When the segregated ice melts, it leads to excess water in the base, subbase, or subgrade that cannot drain through the still-frozen underlying soil. Drainage could also be restricted laterally at this time of the year; thus, the period of severe weakening may last several weeks. When the pavement structure has a drainage layer, this period of severe thaw-weakening can decrease.

8-3.5.2 Frost Heave.

Pavements constructed over F4 subgrade soils, and in some instances over F3 soils, as described in Table 8-1, may experience heave. Heave can be uniform or non-uniform, depending on variations in exposure to solar radiation, the character of the soil, and groundwater conditions underlying the pavement. Non-uniform heave results in unevenness or abrupt changes in grade at the pavement surface. This surface roughness may be objectionable for aircraft with high landing and takeoff speeds. If experience indicates this is the case, the report should include the locations and descriptions of the objectionable roughness. Obtain surface elevations at least once a month during the following winter to determine the magnitude of the detrimental heave.

8-3.5.3 Thaw Weakening.

The load-bearing capacity of both flexible and rigid pavements can be severely reduced during critical weakening periods; however, the reduction is less critical for rigid than for flexible pavements. Rigid pavements experience a smaller reduction because the subgrade has less influence on the supporting capacity of rigid pavements than on that of flexible pavements. Subgrade soils under rigid pavements are subjected to less shearing deformation and remolding during critical weakening periods.

Soils, such as clays, which often show no frost heave, may significantly lose supporting capacity during thawing periods. Frost-susceptible granular unbound base materials may also weaken significantly during frost-melting periods because of increased saturation and associated decrease of moisture tension, combined with reduced density that is derived from expansion in the previously frozen state. As the percent of fines in granular material increases, so does its potential for thaw weakening during frost-melting periods due to reduction of its permeability.

Traffic loads may cause excess hydrostatic pressures within the pores of the frost-affected soil during thaw-weakening periods, resulting in further reduction in strength or even failure. The degree to which a soil loses strength during a frost-melting period and the duration of the period of thaw weakening depend on the soil type, temperature conditions during freezing and thawing, the amount and type of traffic during frost melting, the availability of water during freezing and thawing, and drainage conditions.

8-3.5.4 Visible Surface Effects.

Visible surface effects associated with frost action include random cracking and roughness due to differential frost heave as described above. Noticeable cracking and weakening or deflection can also occur in flexible pavements during the thaw period but may not become visible in rigid pavements during thaw until subsurface damage accumulates and leads to visible surface cracking. As a result, thaw weakening may not always be recognized as the dominant factor causing accelerated deterioration. In either case damage due to thaw weakening may be more severe than cracks caused by frost heave or low-temperature contraction because it leads to destruction of the pavement, requiring reconstruction.

Cracks in flexible pavements may be the result of contraction of the pavement during periods of extremely low temperatures. Flexible pavements that experience accelerated deterioration because of thaw weakening can show alligator cracking or other load-associated cracking at an early age. Rigid pavements can exhibit slab cracking or pumping at cracks and joints. Studies of rigid pavements have shown that cracks may develop more rapidly during and immediately following the spring frost-melting period due to differential thaw than during the period of active heave. D cracking is also a common indication of freeze-thaw damage to PCC pavements but is primarily associated with aggregates of poor quality in the concrete mixture. These are closely spaced crescent-shaped cracks that occur adjacent to longitudinal and transverse joints or free edges.

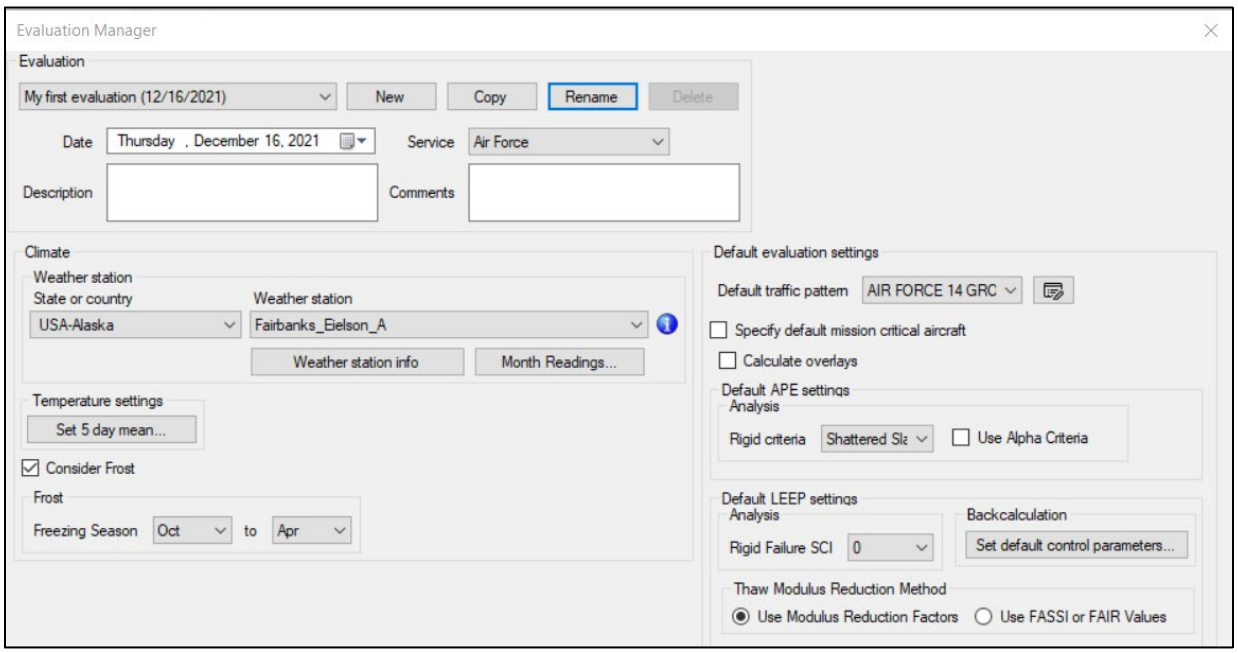
8-3.5.5 Field Inspection and Previous Records.

During pavement inspections, note any cracking, faulting, or pumping. Give particular attention to locations of transitions between cuts and fills and at any boundaries of subgrade soils of varying frost susceptibility. HWD testing can help determine where these transitions take place. Note all spalling at the edges of open cracks which can be an indication of “working cracks” caused by frost action. Construction maintenance and previous evaluation records may help in confirming whether frost-susceptible conditions exist. Records of highway performance in the vicinity of the airfield that have similar subgrade conditions may provide a clue as to whether weakening occurs because of frost melting. In the analysis of highway performance records, the evaluator should carefully note and assess the many local influences that may affect frost action, such as variations in ground-water level, soil conditions, type of pavement surface, degree of shading, north versus south slope, frequency of snow plowing, position of underlying bedrock, etc.

8-4 SETTING UP PCASE FOR FROST ANALYSIS.

When a frost analysis is required, in the Pavement-Transportation Computer Assisted Structural Engineering (PCASE) application the user must check the “Consider Frost” box on the Evaluation Manager form as shown in Figure 8-4, then select the state/country and station for the evaluation location. If there is no station for the specific location, select the station closest to the location at a similar elevation and with similar climatic conditions. This process identifies the climate data from the WorldIndex database to be used for the evaluation. Selecting the “Consider Frost” checkbox also activates additional fields on the airfield pavement evaluation (APE) and Layered Elastic Evaluation Program (LEEP) forms for use in frost analysis.

Figure 8-4 “Consider Frost” Checkbox and Select Station



8-5 DESIGN FREEZING INDEX (DFI).

The DFI is based on climate data from the WorldIndex database for the selected station. The DFI is a description of the length and severity of the winter for a given location. It is used to determine the surface freezing index which, in turn, is used to determine the depth of frost penetration. While the term “design” is used, the DFI is equally applicable to evaluation.

Historically, the DFI was defined as the average air freezing index (AFI) of the three coldest winters in the latest 30 years of record. If 30 years of record were not available, the AFI for the coldest winter in the latest ten-year period was used. This climate data was presented in maps showing the distribution of design AFI values or mean air freezing index values as described in paragraphs 8-5.5 and 8-5.6. The WorldIndex database provides a numerical solution for determining the DFI. The USACE Cold Regions Research and Engineering Laboratory (CRREL) created the WorldIndex database to aggregate climate data from the National Oceanic and Atmospheric Administration (NOAA).

8-5.1 WorldIndex Database.

The WorldIndex database is available through the PCASE program. The current version of the database was published in 2018 and uses historical surface air-temperature observations from 1980 to 2017 (37 years) at over 16,000 locations around the globe. Each station has a minimum of five years of continuous data available. The WorldIndex database is updated every five years. It aggregates data from the Global Surface Summary of Day (GSOD) database (version 7) and the Global Historical Climatology Network (GHCN) Cooperative Observer Program database, which are published by NOAA's National Centers for Environmental Information (NCEI) and the National Weather Service, respectively.

The database contains 80 air-temperature-based parameters determined for each station, including the parameters PCASE uses to determine the depth of frost penetration, the start and duration of the thaw season, and temperature data used to determine asphalt design/evaluation modulus values. All data is stored in Celsius units. Note that the WorldIndex database uses the term freezing degree-days (FDD) rather than the terms AFI or DFI. More details on the structure and content of the WorldIndex database are available in the ERDC/CRREL Technical Report TR-19-13, *WorldIndex Database Update 2018*.

8-5.2 Degree Days.

The number of degree-days for any given day is the difference between the average daily air temperature and 32 °F. The degree-days are negative when the average daily temperature is below 32 °F (freezing degree-days) and positive when above (thawing degree-days) although in both cases, the sign is typically omitted when presenting the data. Air temperatures are measured approximately 4.5 feet (1 meter) above the ground. Degree days may be computed in either Fahrenheit or Celsius units.

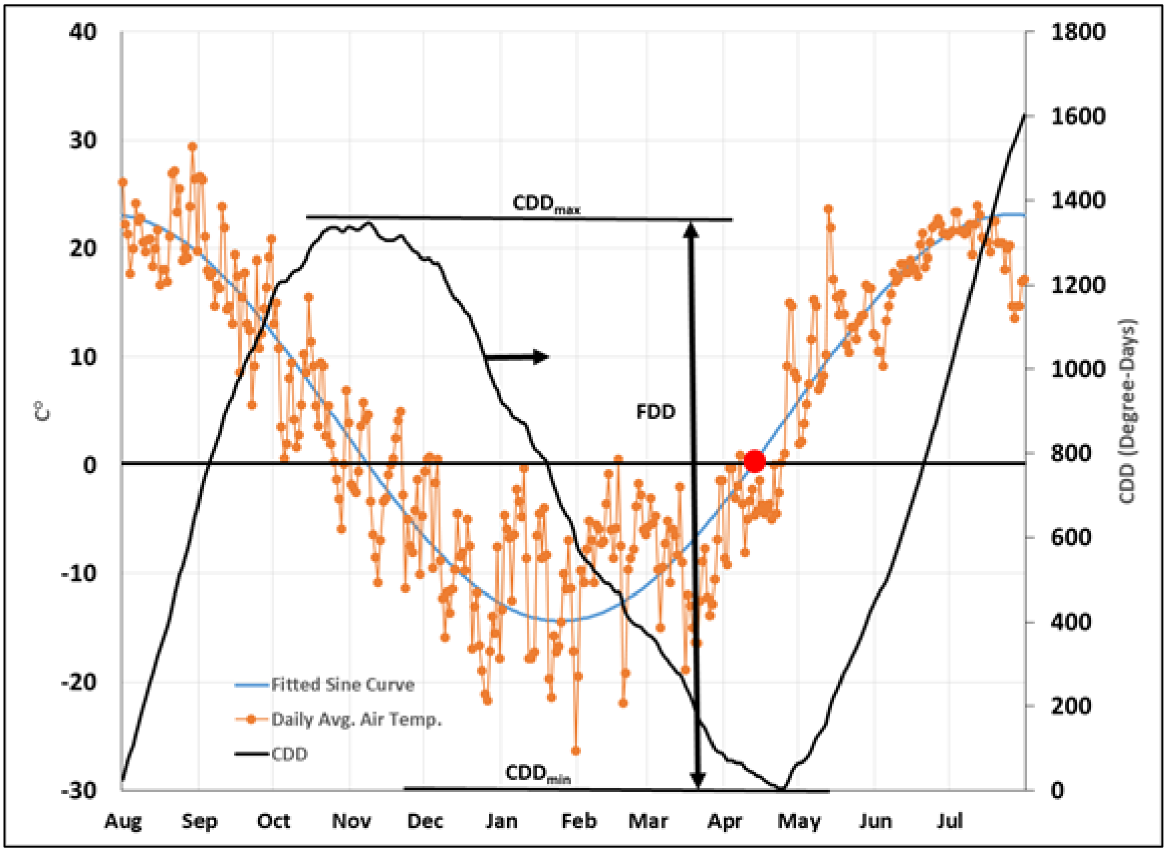
8-5.3 Cumulative Degree Days (CDD).

CDD are the arithmetic sum of FDD over time (typically a year). When CDD are plotted versus time, it generates a curve used to determine the AFI, as shown in Figure 8-5.

8-5.4 Air Freezing Index (AFI).

The AFI is the number of FDD between the highest (CDD_{max}) and lowest points (CDD_{min}) on a CDD curve versus time for a single freezing season. It is also called the annual AFI and is used as a measure of the combined duration and magnitude of below-freezing temperatures occurring during any given freezing season. Note that the AFI is shown as FDD in Figure 8-5 to reflect the terminology used in the WorldIndex database.

Figure 8-5 Air Freezing Index



8-5.5 Average Annual Air Freezing Index (AFI).

The average AFI is the average of all the annual AFIs over the period of record. As noted above, the period of record for the data in the WorldIndex database is 37 years rather than 30 years as described in past criteria documents. The WorldIndex database uses the term "average annual maximum cumulative freezing degree days (YRLY_AVG_FDD)." This value and the standard deviation of the average annual maximum cumulative FDD (YRLY_STDEV_FDD) are passed to PCASE to compute the DFI.

8-5.6 Computing the Design Freezing Index (DFI).

Past criteria defined the DFI as the average AFI of the three coldest winters in the latest 30 years of record. The current procedure uses the average annual AFI and its standard deviation to define the DFI as shown in Equation 8-1. This approach represents the 91st percentile of the freezing indices for the period of record for a given location assuming a normal distribution (Reference: Cortez, E.R., M.A. Kestler, and R.L. Berg. 2000, *Computer-Assisted Calculations of the Depth of Frost Penetration in Pavement-Soil Structures*).

Equation 8-1. DFI Computation

$$\mathrm{DFI}\ (°\mathrm{F\ days}) = 1.8 * \left(\overline{AFI_{Ann}} + (1.5 * \sigma_{AFI})\right)$$

Where:

$1.8 = constant\ to\ convert\ ℃\ days\ to\ ℉\ days$

$\overline{AFI_{Ann}} = Average\ AFI = YRLY\_AVG\_FDD\ (℃\ days)$

$\sigma_{AFI} = standard\ deviation\ of\ the\ average\ AFI = YRLY\_STDEV\_FDD\ (℃\ days)$

For example, if the average annual AFI is 990 ℃ days and the standard deviation is 310 ℃ days, the DFI is 2619 ℉ days.

$$1.8 * \left(990\ ℃\ Days + (1.5 * 310\ ℃\ days)\right) = 2619\ ℉\ days$$

Figure 8-6 is taken from the previous version of this UFC based on data prior to 1987. It graphically depicts DFI distribution in North America and is presented here as a general guide. Note that the current release of the WorldIndex database identified overall decreases in global annual AFI values from previous data.

8-5.7 Alternate DFI Procedure.

In cases where climate data for a location is not available in the WorldIndex database but the mean freezing index is available from other data sources, the DFI can be roughly estimated using the equations below and used to manually compute the depth of frost penetration for use in PCASE. It is important to note the period of record for the available data. At a minimum, it should cover the latest 10 years and preferably at least 30 years. Special considerations will be necessary to compensate for local topographic conditions that will cause deviations from general freezing index values. Note that the mean freezing value must be multiplied by 13.33 to convert from degrees F days to degrees C hours in the second equation.

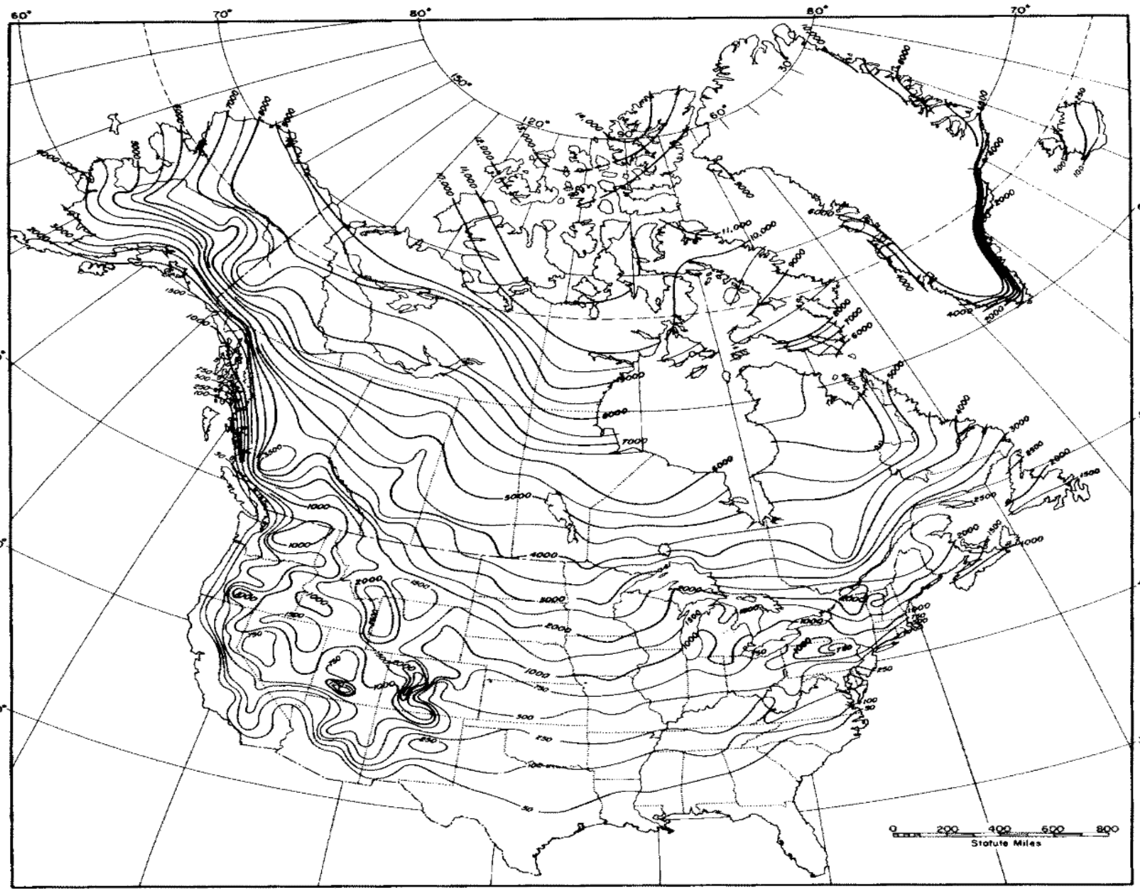
English Units

$$(DFI) = 429 + 1.143 \times mean\ freezing\ (^{\circ}\ F\ days)$$

SI Units

$$(DFI) = 5{,}718 + 1.143 \times mean\ freezing\ index\ (^{\circ}\ C\ hours)$$

Figure 8-6 Distribution of Design Air Freezing Indices in North America



CONVERSION FACTOR

DEG F DAYS = 0.075 × DEG C HOURS

8-6 DETERMNE DEPTH OF FROST PENETRATION AND THAW PERIOD.

The general process to determine the depth of frost penetration and the start and duration of the thaw period for a given pavement structure is outlined below. It focuses on the numerical procedure used in PCASE, but both the depth of frost penetration and length of the thaw period can be entered in PCASE based on manual calculations or experience at the location.

- Define the pavement layer structure
  - Pavement type and thickness
  - Soil layer type classifications and thicknesses
    - Soil frost group
    - Moisture content
    - Dry unit weight
- Compute the surface freezing index

- Determine the start and length of the thaw season

8-6.1 Define the Pavement Layer Structure.

Chapter 3 outlines field procedures for testing to determine the pavement layer structure. Chapters 5, 6, and 7 outline the procedure for entering the layer structure data for layered elastic and conventional (CBR and k) structural analysis. Frost analysis requires a frost code (soil frost group), gravimetric moisture content, and dry unit weight for each layer in addition to entering the pavement and soil layer types and thicknesses.

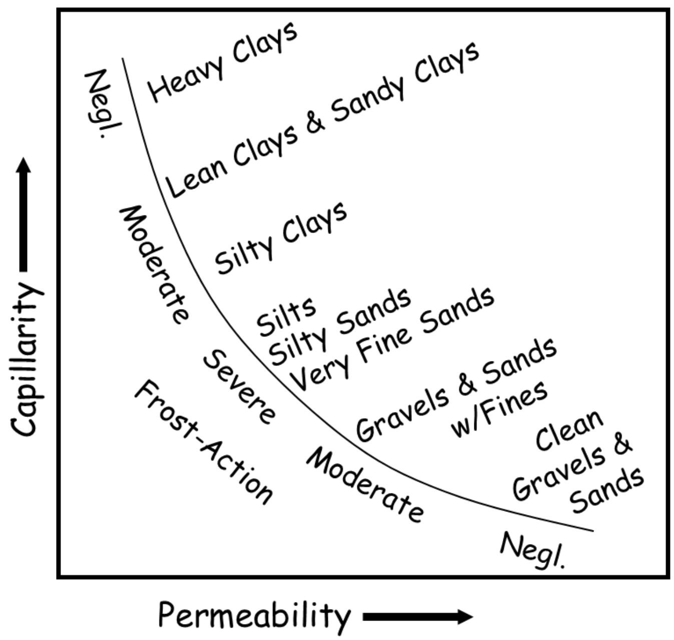
8-6.1.1 Supplementary Soil Testing for Frost Analysis.

When a frost analysis is warranted based on the criteria in paragraph 8-3, conduct testing on the base, subbase, and subgrade material to determine its frost susceptibility. Even if the materials were not frost-susceptible at construction, base and subbase materials can degrade due to freeze-thaw cycles and traffic loads over time. This degradation may introduce additional fines, increasing its thaw-weakening potential.

8-6.1.2 Frost Susceptibility of Base, Subbase, and Subgrade.

Use sieve and Atterberg limits testing to classify base, subbase, and subgrade soils according to the Unified Soil Classification System (USCS) (ASTM D2487, *Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)*). The frost susceptibility of soil is defined by the percent of soil finer than a #200 (.075 mm) sieve by weight and the percent finer than 0.02 mm by weight. Additional testing is required to characterize the percent finer than 0.02 mm by weight using methods described in ASTM D1140, *Standard Test Methods for Determining the Amount of Material Finer than 75-µm (No. 200) Sieve in Soils by Washing*, and ASTM D7928, *Standard Test Method for Particle-Size Distribution (Gradation) of Fine-Grained Soils Using the Sedimentation (Hydrometer) Analysis*. Table 8-1 identifies the frost susceptibility by soil type and percent fines. They are listed in approximate order of increasing frost susceptibility and decreasing bearing capacity during periods of thaw. The percent of fines defines the potential for capillary action and the permeability of soil, which both effect the potential for detrimental frost action as shown in Figure 8-7. Note that while clay materials may not show frost heave, they can still have significant loss of bearing capacity during thawing periods.

Figure 8-7 Frost Action Severity Based on Soil Type



Most cohesive soils have high capillarity but low permeability, water moves too slowly for growth of thick ice lenses.

Low-plasticity soils with high percentage of silt-sized particles, have pore sizes small enough to develop capillary potential, but large enough to permit travel of water to the frozen zone.

Coarser soils have higher rates of flow, but do not have the capillary potential to lift enough moisture for heaving.

Table 8-1 Frost Susceptibility Soil Classification

Frost Group	Soil Type	% Finer than 0.02 mm by Weight	% Finer than #200 Sieve by Weight[1]	Typical Soil Types (Unified Soil Classification System)
NFS[2]	(a) Gravel, crushed stone, crushed rock	0 - 1.5	0 - 3	GW, GP
	(b) Sands	0 – 3	0 - 7	SW, SP
PFS[3]	(a) Gravel, crushed stone, crushed rock	1.5 - 3	3 - 7	GW, GP
	(b) Sands	3 – 10		SW, SP
S1	Gravelly soils	3 - 6	7 - 15	GW, GP, GW-GM. GP-GM
S2	Sandy soils	3 - 6	7 - 15	SW, SP. SW-SM, SP-SM
F1	Gravelly soils	6-10		GM, GW-GM, GP-GM
F2	(a) Gravelly soils	10-20		GM, GW-GM, GP-GM
	(b) Sands	6-15		SM, SW-SM, SP-SM
F3	(a) Gravelly soils	Over 20		GM, GC
	(b) Sands, except very fine silty sands	Over 15		SM, SC
	(c) Clays, PI > 12	--		CL, CH
F4	(a) Silts	--		ML, MH
	(b) Very fine silty sands	Over 15		SM
	(c) Clays, PI < 12	--		CL, CL-ML
	(d) Varved clays and other fine-grained, banded sediments	--		CL, ML, and SM, CL, CH, and ML, CL, CH, ML, and SM

Notes: 1. These are rough estimates. If there are surface indications of frost action, then frost-susceptibility tests should be conducted.
2. Nonfrost susceptible.
3. Possibly frost susceptible; requires lab test to determine frost soil classification.

8-6.1.3 Moisture Content.

The moisture content and dry unit weight of the soil in each layer are required inputs that PCASE uses to compute the frozen and unfrozen soil thermal properties of each layer. Optimally, these values are determined based on field and laboratory testing such as ASTM D6938, *Standard Test Methods for In-Place Density and Water Content of*

Soil and Soil-Aggregate by Nuclear Methods (Shallow Depth), or ASTM D2216, *Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass*. These values are used to estimate average moisture contents in the base subbase and subgrade at the start of the freezing period. When testing is not possible, average estimated values can be used based on pit data from previous pavement evaluations or data from construction projects.

PCASE assigns default moisture contents as shown in Table 8-2 based on the layer types. Use these conservative defaults when no other data is available. Note that the depth of frost penetration is sensitive to the moisture content. In general, for a given dry density, frost penetration increases with decreasing moisture content. For example, as shown in Figure 8-8, for a soil with a dry density of 115 pcf and an AFI of 2,000 °F Days, frost is able to penetrate 70 inches at 15 percent moisture content but penetrates to 80 inches at 5 percent moisture content.

Figure 8-8 Effect of Moisture on Frost Penetration

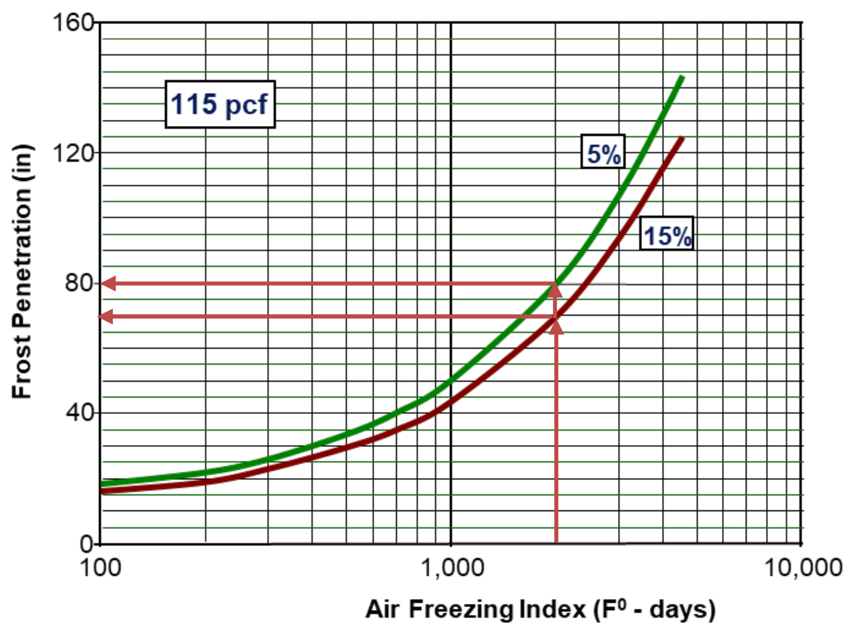


Table 8-2 Default Moisture Contents and Dry Densities

Layer Type	Moisture Content (%)	Dry Unit Weight (pcf)
Asphalt	0	140
Concrete	0	145
Stabilized base AC / PCC	5 / 0	140
Base	5	135
Drainage layer	5	135
Separation layer	5	135
Subbase	5	135
Stabilized subbase AC / PCC	5 / 0	130 / 140
Select fill	5	135
Stabilized subgrade PCC / Lime	0 / 10	130 / 110
Modified subgrade cohesive / cohesionless	18 / 10	100 / 120
Compacted subgrade	18 / 10	100 / 120
Natural subgrade cohesive / cohesionless	18 / 10	100 / 120

8-6.1.4 Dry Unit Weight.

The dry unit weight of a soil is determined based on field and laboratory testing using a nuclear gauge, sand cone, or compaction testing. Table 8-2 has the default dry unit weight values assigned in PCASE and Figure 8-9 provides estimated ranges of dry unit weight for various soil types. Note that in general, the depth of frost penetration will be greater for a soil with a higher dry unit weight at a given moisture content. For example, given a 12-inch rigid pavement overlying a homogeneous material at an infinite depth, at an AFI of 2,000 ˚F Days with 5 percent moisture content, the depth of frost penetration would be 100 inches for a soil with a density of 150 pcf, 80 inches at 115 pcf, and 70 inches at 100 pcf.

Figure 8-9 Soil Density Chart

Soil Types		Symbol		Drainage Characteristics	Unit Dry Weight Lb. per Cu Ft	Field CBR	Subgrade Modulus K Lb per Cu In
Coarse-grained Soils	Gravels and Gravelly Sands	GW		Excellent	125 - 140	60 - 80	300 or more
		GP		Excellent	110 - 130	25 - 60	300 or more
		GM	d	Fair to poor	130 - 145	40 - 80	300 or more
			u	Poor to impervious	120 - 140	20 - 40	200 to 300
		GC		Poor to impervious	120 - 140	20 - 40	200 to 300
	Sands and Sandy Gravels	SW		Excellent	110 - 130	20 - 40	200 to 300
		SP		Excellent	100 - 120	10 - 25	200 to 300
		SM	d	Fair to poor	120 - 135	20 - 40	200 to 300
			u	Poor to impervious	105 - 130	10 - 20	200 to 300
		SC		Poor to impervious	105 - 130	10 - 20	200 to 300
Fine-grained Soils	Silts and Clays LL <50	ML		Fair to poor	100 - 125	5 - 15	100 to 200
		CL		Impervious	100 - 125	5 - 15	100 to 200
		OL		Poor	90 - 105	4 - 8	100 to 200
	Silts and Clays LL >50	MH		Fair to poor	80 - 100	4 - 8	100 to 200
		CH		Impervious	90 - 110	3 - 5	50 to 100
		OH		Impervious	80 - 105	3 - 5	50 to 100
Highly Organic Soils		Pt		Fair to poor	-----	-----	-----

GM and SM groups are divided into subdivisions d and u for roads and airfields

Suffix d is used when LL $\leq$ 28 and PI $\leq$ 6 Suffix u is used when LL > 28

8-6.2 Surface Freezing Index.

As noted earlier, the DFI is based on the average annual AFI and standard deviation but this index is determined for air temperatures at 4.5 feet (1 meter) above the ground. We use the AFI because the data is more readily available than surface temperatures. The DFI and the n-Factor (Figure 8-10) are used in Equation 8-2 to estimate the surface freezing index, which is the temperature immediately below the pavement surface. PCASE determines the n-Factor based on the surface type of the layer model being analyzed (see Figure 8-10).

Equation 8-2. Surface Freezing Index

$$\text{Surface Freezing Index} = \text{n Factor} * \text{DFI}$$

Figure 8-10 n-Factors

Surface Type[1]	n-Factor for Freezing Conditions	n-Factor for Thawing Conditions
Snow Surface	1.00	-
Portland Cement Concrete	0.75	1.50
Bituminous Pavement	0.70	1.60 - 2.00[2]
Bare Soil	0.70	1.40 - 2.00[2]
Shaded Surface	0.90	1.00
Turf	0.50	0.80
Tree-Covered	0.30[3]	0.40

1. Surface exposed directly to sun or air without any overlying dust, soil, snow or ice, except as noted otherwise and with no building heat involved.
2. Use lowest value except in extremely high latitudes or at high elevations where a major portion of summer heating is from solar radiation.
3. Data from Fairbanks, Alaska, for single season with snow cover permitted to accumulate naturally.

8-6.3 Determine Start and Length of Thaw Season.

8-6.3.1 Thaw-weakened Period.

As shown in Figure 8-11, thaw-weakened periods are intervals of the year when the base, subbase, or subgrade strength are below normal summer values. These intervals correspond to frost melting periods. The period ends when the material is either refrozen or when the subgrade strength has returned to the normal summer value at the end of the spring thaw-weakening period.

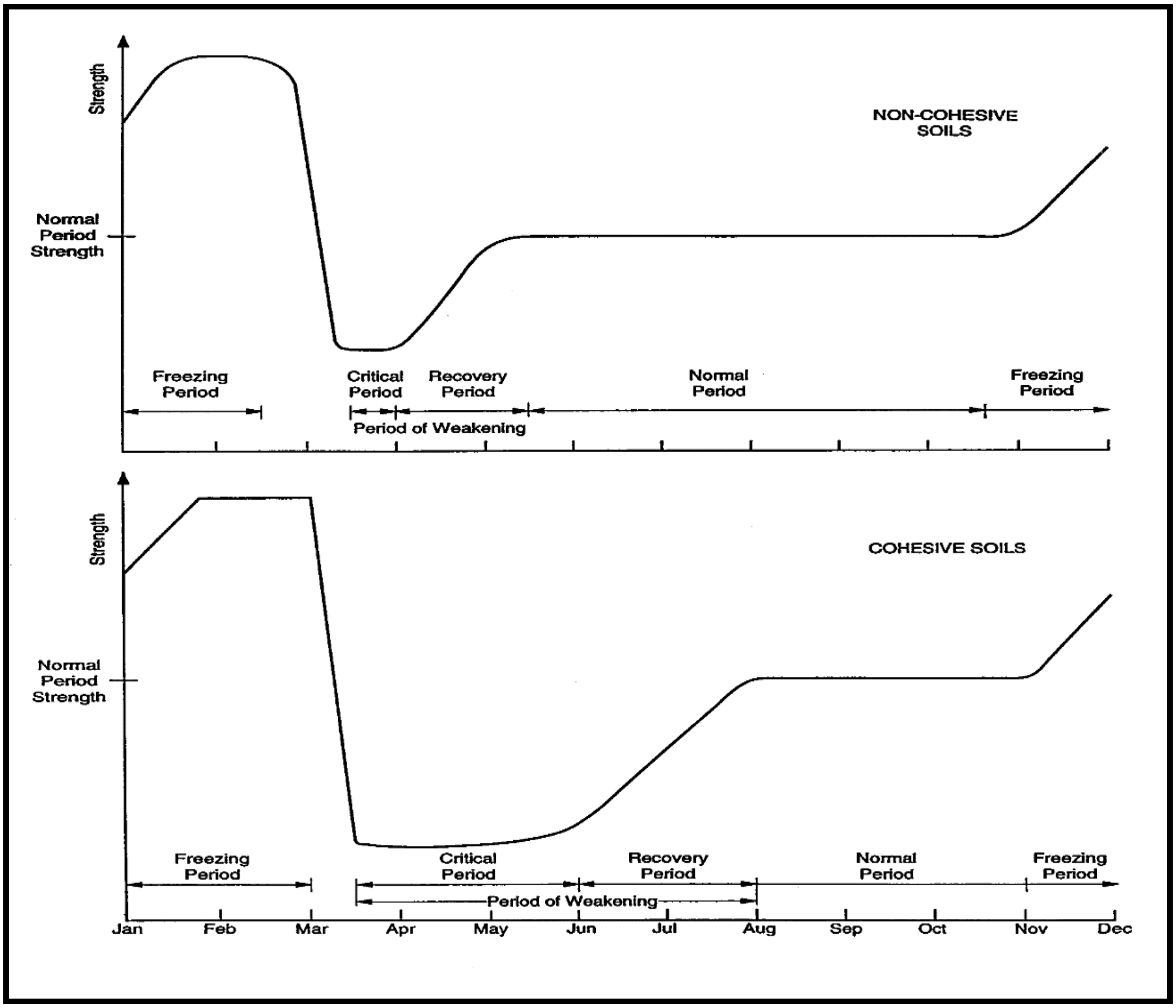
8-6.3.2 Critical Weakening Period.

The critical weakening period is the time interval during the thaw-weakened period when the base, subbase, or subgrade are at their lowest strength. As shown in Figure 8-11, the critical weakening period comes during the early stages of frost-melting and may occur intermittently during the winter when the segregated ice in the base, subbase, or subgrade is melting. This critical period can last from a week to several months, depending on the soil type and environmental conditions. The likely duration of the critical period can be estimated based on the performance of highways with a comparable subgrade in the vicinity of the airfield. However, since airfield pavements are wider and drainage paths longer, the thaw-weakened period is likely to be longer.

8-6.3.3 Recovery Period.

As the soil drains and reconsolidates, the pavement gradually regains full normal-period bearing capacity. The length of the recovery period varies from a few weeks to several months, depending on the intensity of ice segregation, depth of frost penetration, rate of thawing, permeability of the soil, drainage conditions, precipitation, and atmospheric humidity.

Figure 8-11 Illustration of Thaw-Weakening Period



8-6.3.4 Estimating Thaw Weakened Period Start and Duration.

Several frost-melting periods may occur during a typical winter period. The procedure outlined below is used to estimate the start and total period of weakening, including frost-melting periods during the winter. The length of the thaw-weakened period can be changed based on local experience. Principal factors affecting the recovery time are depth of frost penetration, type of frost-susceptible material, and subsurface drainage. Normally, the time for recovery will be from several weeks to several months.

The end of the freezing period defines the start of the thaw period. The thaw-weakened periods for different frost-susceptible soils are presented in Table 8-3. These values are adjusted based on whether the DFI ≤ 1,000-degree F-days or DFI > 1000 degree F-days as shown in Table 8-4. Note that the general soil type (cohesive or cohesionless) is used to determine the period adjustment for soils in the F3 and F4 frost groups.

Table 8-3 Length of End-of-Winter Thaw-Weakened Period

Frost Group	Thaw-Weakened Period (Months)
F1	1
F2	1
F3 and F4 (Cohesionless)	2
F3 and F4 (Cohesive)	3

Table 8-4 Thaw-Weakened Period Adjustment

Frost Group	Soil Type	DFI	Adjusted Thaw-weakened Period (Months)
F1, F2	N/A	< = 1,000 deg F days	1
F1, F2	N/A	> 1,000 deg F days	2
F3, F4	Cohesionless	< = 1,000 deg F days	2
F3, F4	Cohesionless	> 1,000 deg F days	3
F3, F4	Cohesive	< = 1,000 deg F days	3
F3, F4	Cohesive	> 1,000 deg F days	4

8-7 DETERMINE DEPTH OF FROST PENETRATION.

The objective of this step is to determine whether the freezing front penetrates a frost-susceptible layer. If not, use the normal evaluation procedure. Once the layer type and thickness, frost code, dry unit weight, and moisture content are entered for each layer and the analysis is run, PCASE will display the allowable passes, AGL, and Pavement Classification Number (PCN) for the thaw weakened period if the freezing front penetrates a frost-susceptible layer or will report only the normal values if it does not. Instructions for computing the depth of frost using PCASE are included in the PCASE *User Manual*.

This step can be performed manually by determining the surface thickness of the pavement (p), the total pavement (surface, base, and subbase) thickness (x) and estimate the depth of frost penetration (d) and following the criteria below:

- If ($x \geq d$) use the normal evaluation procedure.

- If ($x < d$), the pavement structure is inadequate for complete frost protection. If there are indications of frost action, evaluate the pavement structure with the reduced subgrade strength approach.
- If ($x - p \geq 60$ inches) or the base, subbase, and/or subgrade is classified as NFS, S1, or S2 and there are no surface indications of frost action, use the normal evaluation procedure.
- If ($x - p \geq 60$ inches) or the base, subbase, and/or subgrade is classified as NFS, S1, or S2 and there are indications of frost action, evaluate pavement structure with the reduced subgrade strength approach.

8-8 EVALUATE PAVEMENT FOR REDUCED SUBGRADE STRENGTH.

Both conventional and layered elastic reduced subgrade strength (RSS) evaluation procedures are based on the application of the fatigue damage concept (Miner's Hypothesis). The conventional evaluation procedure substitutes frost area soil support indices (FASSI) values for California Bearing Ratio (CBR) values in flexible pavement analysis and Frost Area Index of Reaction (FAIR) values for modulus of subgrade reaction values (k) in rigid analysis. In layered elastic evaluation, either the FASSI/FAIR procedure or the reduced modulus procedure can be used.

8-8.1 Frost Area Soil Support Indices (FASSI).

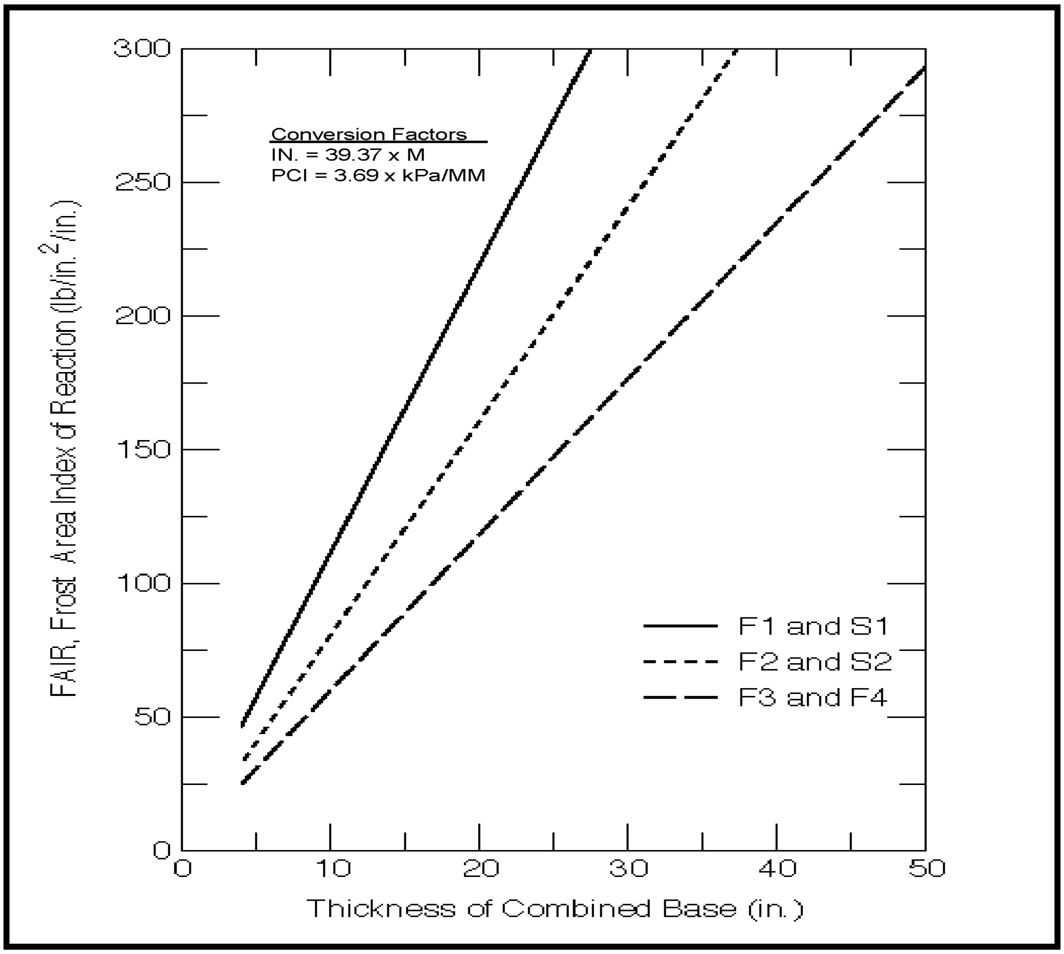
The FASSI values are used as if they were CBR values. The term CBR is not applied to them, however, because they are weighted average values for the annual cycle and their values cannot be determined by CBR tests. FASSI values are assigned based on the soil layer's frost group. Ideally, base and subbase layers are not frost susceptible but this is not always the case, so, for any layer with a frost code assigned (other than NFS), PCASE will replace the CBR with a FASSI value when the depth of frost penetrates that layer. If the depth of frost does not penetrate the layer, PCASE uses the assigned CBR. FASSI values for the respective frost codes are listed below.

- F1 and S1 soils = 9.0 FASSI
- F2 and S2 soils = 6.5 FASSI
- F3 and F4 soils = 3.5 FASSI

8-8.2 Frost Area Index of Reaction (FAIR).

FAIR values are used as if they were modulus of subgrade reaction values, k, and have the same units. However, the term modulus of subgrade reaction is not applied to them because the FAIR values are weighted average values for an annual cycle and cannot be determined by a plate-bearing test. Figure 8-12 shows the equivalent weighted average FAIR values for an annual cycle that includes a thaw-weakening period in relation to the combined thickness of the base and subbase. This figure is based on the FAIR value Equations 8-3, 8-4, and 8-5. If the depth of frost does not penetrate the layer, PCASE uses the assigned k value, or when the modulus of subgrade reaction k, determined from tests on the equivalent base course and subgrade for the normal period, is less than the FAIR value obtained from the equations, use the test value for the analysis.

Figure 8-12 Determination of FAIR Value



Equation 8-3. FAIR Value for F1 or S1 Material

$$FAIR\ ((psi/in.) = 4.2 + 10.8 \times \text{Combined } Base\ Course\ Thickness\ (inches)$$

Equation 8-4. FAIR Value for F2 or S2 Material

$$FAIR\ ((psi/in.) = 1.3 + 8.0 \times \text{Combined } Base\ Course\ Thickness\ (inches)$$

Equation 8-5. FAIR Value for F3 or F4 Material

$$FAIR\ ((psi/in.) = 1.6 + 5.9 \times \text{Combined } Base\ Course\ Thickness\ (inches)$$

8-9 MODULUS REDUCTION FACTORS.

There are two alternatives for reduced subgrade strength analysis using layered elastic procedures. In the first, the FASSI or FAIR values determined using the procedures above are correlated to modulus values and used for the RSS analysis. In the second approach, the modulus values of the respective layers during the normal period are reduced using the reduction factors shown in Table 8-5. The modulus for the normal period is multiplied by the reduction factor for the given frost group and this reduced modulus is used for the analysis. If the depth of frost does not penetrate the layer, PCASE uses the normal period modulus value. If modulus values were determined during the thaw period, then use those values in the thaw period analysis. In this case, the modulus for the normal period can be estimated by dividing the thaw period modulus value by the modulus reduction factor.

Table 8-5 Modulus Reduction Factors for Seasonal Frost Areas

Frost Group	Modulus Reduction Factors
NFS	1.00
PFS	0.90
S1	0.75
S2	0.70
F1	0.60
F2	0.50
F3/F4	0.30

8-10 EVALUATION USING DAMAGE CONCEPTS.

The fatigue damage concept (Miner's Hypothesis) is used to evaluate pavement structures when frost is a consideration. Damage is defined as the ratio of applied load repetitions to the allowable load repetitions to a failure and is expressed as:

Equation 8-6. Damage Definition

$$d = \frac{n}{N}$$

Where:
n = applied load repetitions
N = allowable load repetitions

When the applied load repetitions equal the allowable number of load repetitions, the damage ratio is equal to 1.0. This means that 100 percent of the pavement life has been consumed. The goal is to determine the allowable load or the allowable passes such that the damage ratio is equal to one (*d* = 1.0) for an analysis period.

8-10.1 Cumulative Damage Factor (CDF).

The damage concept can be extended to multiple aircraft and multiple seasons. A CDF can be expressed as the sum of individual damage contributions from different aircraft and different seasons or evaluation periods. The general equation for CDF, where the aircraft load and passes result in the maximum utilization of pavement life, is defined as:

Equation 8-7. General Cumulative Damage Factor

$$CDF = \sum_{i=1}^{nac} \sum_{j=1}^{ns} d_{ij} = 1.0$$

Where:

$d_{i,j}$ = damage for aircraft i and season *j*
nac = number of aircraft in mix
ns = number of seasons or analysis period

In pavement evaluation, the analysis is performed for only a single aircraft (*nac* = 1) and two seasons (*ns* = 2). The two evaluation seasons considered are the normal and thaw-weakened periods. The cumulative damage factor represented by Equation 8-7 can now be reduced to Equations 8-8 and 8-9.

Equation 8-8. Reduced Cumulative Damage Equation

$$CDF = d_{normal} + d_{thaw} = 1.0$$

Equation 8-9. Expanded Cumulative Damage Equation

$$CDF = \frac{n_{normal}}{N_{normal}} + \frac{n_{thaw}}{N_{thaw}} = 1.0$$

Where:
n_{normal}, N_{normal} are the applied and allowable number of coverages (or passes) during the normal period
n_{thaw}, N_{thaw} are the applied and allowable number of coverages (or passes) during the thaw-weakened period

8-10.2 Prorating Passes for Normal and Thaw Periods.

When evaluating pavement structures, the aircraft AGL is analyzed for a life expectancy expressed as the evaluation passes, *n*. Since *n* is the total number of passes applied throughout the whole year, a fraction of *n* is applied during the normal period and a fraction of *n* is applied during the thaw-weakened period. Each of these fractions will contribute to the total accumulated damage during a year. These fractions of *n* are determined as the prorated number of passes during the normal months and thaw-weakened months. For example, if evaluating the subgrade of a pavement structure for *n* = 50,000 passes and one month of the year is in a thaw-weakened condition, then Equations 8-10 and 8-11 represent the contribution of the total number of passes *n* in each season.

Equation 8-10. Prorated Evaluation Passes during the Normal Period

$$n_{normal} = n\left(\frac{11}{12}\right) = 50{,}000\left(\frac{11}{12}\right) = 45{,}833\ Passes$$

Equation 8-11. Prorated Evaluation Passes during the Thaw-weakened Period

$$n_{thaw} = n\left(\frac{1}{12}\right) = 50{,}000\left(\frac{1}{12}\right) = 4{,}167\ Passes$$

8-10.3 Damage in Conventional Flexible and Rigid Pavement Evaluation (APE)

The airfield pavement evaluation (APE) CBR / k models were not developed with the cumulative damage concept in mind. Therefore, the damage concept cannot be applied directly because the analysis is based on a single evaluation period and the APE analysis will only return the allowable number of coverages (or passes) for the entire pavement life, *N*. Equation 8-9 is solved in APE by allowing a predetermined amount of damage to occur during the thaw-weakened period as shown in Figure 8-11. The predetermined damage is assumed to be 25 percent during the thaw period (d_{thaw} = 0.25) and 75 percent during the normal period (d_{normal} = 0.75), as shown in Equations 8-12 and 8-13:

Equation 8-12. Damage Allowed during the Thaw Period

$$d_{thaw} = 0.25 = \frac{n_{thaw}}{N_{thaw}}$$

Equation 8-13. Damage Allowed during the Normal Period

$$d_{normal} = 0.75 = \frac{n_{normal}}{N_{normal}}$$

Both N_{thaw} and N_{normal} are the allowable number of coverages for the respective analysis periods or seasons. If we define N'_{thaw} as the allowable number of coverages (or passes) at failure when d'_{thaw} = 1.0, then 100 percent of the life will be consumed. Therefore, we must adjust N'_{thaw} by a factor of 0.25 as shown in Equation 8-14, because only one-quarter of its life is allowed be consumed during this analysis period.

Equation 8-14. Pavement Life Adjustment during the Thaw-weakened Period

$$N_{thaw} = 0.25\ N'_{thaw}$$

This leads to the equation of damage during the thaw-weakened period (Equation 8-15), that is interpreted as an adjustment to the allowable coverages (or passes), N'_{thaw} or as an adjustment to the evaluation coverages (or passes) n_{thaw}.

Equation 8-15. Adjustment to Evaluation Passes during the Thaw-weakened Period

$$d_{thaw} = \frac{n_{thaw}}{N_{thaw}} = \frac{n_{thaw}}{0.25\ N'_{thaw}} = \frac{4\ n_{thaw}}{N'_{thaw}}$$

Similarly, make an adjustment during the normal period using Equation 8-16, because only three-quarters of its life is consumed during the normal period.

Equation 8-16. Pavement Life Adjustment during Normal Period

$$N_{normal} = 0.75\ N'_{normal}$$

The damage during the normal period can then be estimated using Equation 8-17.

Equation 8-17. Damage Allowed during the Normal Period

$$d_{normal} = \frac{n_{normal}}{N_{normal}} = \frac{n_{normal}}{0.75\ N'_{normal}} = \frac{\frac{4}{3}\ n_{normal}}{N'_{normal}} = \frac{1.33\ n_{normal}}{N'_{normal}}$$

Aircraft AGL for the normal and thaw-weakened periods satisfying Equations 8-15 and 8-17 are individually calculated by an iteration process. In APE, the AGL is determined by evaluating for the following adjusted number of passes:

$$Thaw\ period = \ 4\ times\ prorated\ passes = 4\ n_{thaw}$$
$$Normal\ period = \frac{4}{3}\ times\ prorated\ passes = \frac{4}{3} n_{normal}$$

For our example in Equations 8-10 and 8-11, the cumulative damage factor becomes:

Equation 8-18. Example Cumulative Damage Factor for Two Analysis Periods

$$CDF = \frac{\frac{4}{3}(45{,}833)}{N'_{normal}} + \frac{4(4{,}167)}{N'_{thaw}} = 1.0$$

Equation 8-18 is solved iteratively by assuming an AGL for each season (AGL_{normal}, AGL_{frost}) and calculating the allowable number of passes (N'_{normal}, N'_{thaw}) until the *CDF* = 1.0 within an acceptable tolerance value. Here, N'_{normal} is a function of the subgrade strength during the normal period and N'_{thaw} is a function of the subgrade strength during the thaw-weakened period. In summary, in APE the AGL is determined by evaluating for the following equivalent number of passes:

$$Thaw\ period = \ 4\ times\ prorated\ passes = 4\ n_{thaw}$$
$$Normal\ period = \frac{4}{3}\ times\ prorated\ passes = \frac{4}{3} n_{normal}$$

8-11 DEFINING DAMAGE IN LAYERED ELASTIC EVALUATION PROGRAM (LEEP).

When designing flexible pavements using the layered elastic method, load repetitions are equated to coverages and both the limiting asphalt horizontal strain and the subgrade vertical strain criteria are checked in terms of coverages. However, when evaluating flexible pavements, the subgrade strain criterion is only checked in terms of passes. This simplification was adopted during the early stages of development of the evaluation criteria because passes at the subgrade level are approximately equal to coverages. Equation 8-19 defines damage in terms of the subgrade criterion for layered elastic evaluation of flexible pavements and Equation 8-20 defines the damage in terms of the asphalt criterion.

Equation 8-19. Subgrade Damage in Terms of Passes

$$d = \frac{applied\ passes}{allowable\ passes}$$

Equation 8-20. Asphalt Damage in Terms of Coverages

$$d = \frac{applied\ coverages}{allowable\ coverages}$$

When evaluating rigid pavements, the bending stress criterion at the bottom of the concrete layer is checked against the bending stress criterion in terms of coverages as shown by Equation 8-21.

Equation 8-21. Concrete Damage in Terms of Coverages

$$d = \frac{applied\ coverages}{allowable\ coverages}$$

Even though the layered elastic method permits the calculations of the damage for normal and thaw-weakened periods directly without having to assume $d_{thaw} = 0.25$ and $d_{normal} = 0.75$, the same procedure outlined here for APE is also applied with layered elastic analysis. In theory, the damage distribution between analysis periods could be changed to any another value if the field conditions warrant a different damage distribution between the two analysis periods.

8-11.2 LEEP (WESPAVE) Frost Analysis.

Earlier PCASE versions used the WESPAVE layered elastic model for LEEP analysis. The current PCASE version allows the user to set the WESPAVE option to replicate previous pavement evaluation results. When the WESPAVE option is on, the AGLs for the normal and thaw-weakened periods are determined by evaluating for the following adjusted number of passes:

$$Thaw\ period = \ 4\ times\ prorated\ passes = 4\ n_{thaw}$$
$$Normal\ period = \frac{4}{3}\ times\ prorated\ passes = \frac{4}{3} n_{normal}$$

8-11.3 LEEP (YULEA) Frost Analysis.

The current PCASE version (7.x and later) uses the YULEA layered elastic model in LEEP. When using YULEA, the AGLs for the normal and thaw-weakened periods are determined by setting the target damages during the two analysis periods directly as follows:

$$Thaw\ period - Set\ d_{thaw} = 0.25$$
$$Normal\ period - Set\ d_{normal} = 0.75$$

8-12 EVALUATING PAVEMENTS ON PERMAFROST.

Typically, pavements on permafrost are in their weakest condition during the summer. The permafrost thaws from the top down and provides excess water that cannot drain because of the underlying frozen permafrost. Pavement evaluations are performed during the weakened state, which may only last a few months. This is essentially the opposite of the evaluation procedures previously discussed in this chapter.

In this case, the pavement evaluation is conducted during the summer with a heavy weight deflectometer (HWD) when the pavement is in a weakened condition. Modulus values are established for each layer, including the saturated thawed layers and the frozen permafrost. This establishes the basis for the AGLs and PCNs published for the summer period. For the winter period, assume that the previously thawed layers (base, subbase, and subgrade) would have the same modulus values when frozen as those established for the frozen permafrost during the summer evaluation. Alternatively, apply the factors in Table 8-5 using the procedure in paragraph 8-9 to determine a conservative estimate of the modulus during the winter period. Assume that modulus values for the asphalt surface would be the same as established during the summer. These modulus values are used to determine the AGLs and PCNs during the winter period.

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CHAPTER 9 STANDARDIZED METHOD FOR REPORTING AIRFIELD PAVEMENT STRENGTH

9-1 BACKGROUND.

The Aircraft Classification Number (ACN) - Pavement Classification Number (PCN) method was adopted by the International Civil Aviation Organization (ICAO) as an effective, simple, and readily comprehensible means for reporting the aircraft weight-bearing capability of runways. As a cooperating member of the ICAO, the United States report airfield weight-bearing limits by this method as outlined in Doc 9157-AN/901, *Aerodrome Design Manual*, and Amendment 35 to *International Standards and Recommended Practices, Aerodromes (*Annex 14, Volume I to the Convention on International Civil Aviation). These weight-bearing limits are included in evaluation reports and reported in flight information pamphlets (FLIP).

9-2 ACN DEFINITION.

The ACN is a number that expresses the relative structural effect of an aircraft at a given weight on different pavement types (flexible or rigid) for four specified standard subgrade strengths in terms of a standard single-wheel load. ACN values are determined by the aircraft manufacturers. These same numbers can be calculated using computer programs such as Pavement-Transportation Computer Assisted Structural Engineering (PCASE) as outlined below.

9-2.1 Computing the ACN.

Computing the ACN requires detailed information on the operational characteristics of the airplane, such as maximum aft center of gravity, maximum weight, wheel spacing, tire pressure, and other factors. ACN values are available from aircraft manufacturers, Technical Report TSC 13-2, *Aircraft Characteristics for Airfield Pavement Design and Evaluation - Air Force and Army Aircraft*, and Technical Report TSC 13-3, *Aircraft Characteristics for Airfield Pavement Design and Evaluation - Commercial Aircraft*, or are computed by PCASE.

9-2.2 Subgrade Category.

The ACN-PCN method adopts four standard levels of subgrade strength for rigid pavements and four standard levels of subgrade strength for flexible pavements. These standard support conditions are used to represent a range of subgrade conditions as shown in Tables 9-1 and 9-2. Modulus values (E) for use in layered elastic analysis are shown in Tables 9-3 and 9-4. E values in Table 9-3 were obtained using $k=.07906(E^{0.7788})$. E values in Table 9-4 were obtained using $E = 1500*CBR$.

Table 9-1 Rigid Pavement k Subgrade Categories

Subgrade Strength Category	k-Value (pci)	Represents (pci)	Code Designation
High	552.6	K ≥ 442	A
Medium	294.7	221 < k <442	B
Low	147.4	92 < k ≤221	C
Ultra-low	73.7	k ≤ 92	D

Table 9-2 Flexible Pavement CBR Subgrade Categories

Subgrade Strength Category	CBR Value	Represents	Code Designation
High	15	CBR ≥ 13	A
Medium	10	8 < CBR <13	B
Low	6	4 < CBR ≤ 8	C
Ultra-low	3	CBR ≤4	D

Table 9-3 Rigid Pavement Modulus Subgrade Categories

Subgrade Strength Category	E Value (psi)	Represents (psi)	Code Designation
High	86,374	E≥ 64,840	A
Medium	38,530	22,627 <E< 64,840	B
Low	15,829	8,642 <E≤ 22,627	C
Ultra-low	6,500	E ≤ 8,642	D

Table 9-4 Flexible Pavement Modulus Subgrade Categories

Subgrade Strength Category	E Value	Represents	Code Designation
High	22,500	E≥ 19,500	A
Medium	15000	12,000 <E< 19,500	B
Low	9000	6,000 <E≤ 12,000	C
Ultra-low	4,500	E ≤ 6,000	D

9-2.3 Rigid Pavement ACN.

The aircraft landing gear flotation requirements for rigid pavements are determined by the Westergaard solution for a loaded elastic plate on a Winkler foundation (interior load case) for each subgrade category, assuming a concrete working stress of 399 psi (2.8 MPa).

9-2.4 Flexible Pavement ACN.

The airplane landing gear flotation requirements for flexible pavements are determined by the California Bearing Ratio (CBR) method for each subgrade support category. The CBR method uses a Boussinesq solution for stresses and displacements in a homogeneous, isotropic elastic half-space. To standardize the ACN calculation and to remove operational frequency from the relative rating scale, ACN values are determined for 10,000 coverages.

9-2.5 ACN Calculation.

A mathematically derived single-wheel load is calculated to define the landing gear/pavement interaction using the parameters defined for each type of pavement. The derived single-wheel load implies equal stress to the pavement structure and eliminates the need to specify pavement thickness for comparative purposes. This is achieved by equating the thickness derived for a given airplane landing gear to the thickness derived for a single wheel load at a standard tire pressure of 181 psi (1.2 MPa). The ACN is defined as two times the derived single wheel load (expressed in thousands of kilograms). The procedure for determining ACN is outlined in ICAO Doc 9157-AN/901, *Aerodrome Design Manual, Part 3 - Pavements*.

9-2.6 Variables Involved in Determining ACN Values.

Because airplanes can be operated at various weight and center of gravity combinations, ICAO adopted standard operating conditions for determining ACN values. The ACN is determined at the weight and center of gravity combination that creates the maximum ACN value. Tire pressures are assumed to be those recommended by the manufacturer for the noted conditions. Airplane manufacturers publish maximum weight and center of gravity information in their Airplane Characteristics for Airport Planning (ACAP) manuals.

9-2.7 PCASE ACN Example.

Figure 9-1 is from the PCASE ACN calculator. It shows the ACN information for the P-8 Poseidon aircraft operating at a weight of 188.2 kips on a flexible pavement. The ACN values for each subgrade category are displays as well as an ACN curve plot. Table 9-5 shows a summary of the ACNs for this aircraft on each subgrade category, assuming unlimited tire pressure as indicated by the “W” tire-pressure category.

Figure 9-1 Rigid and Flexible ACN Values for P-8 Poseidon Aircraft

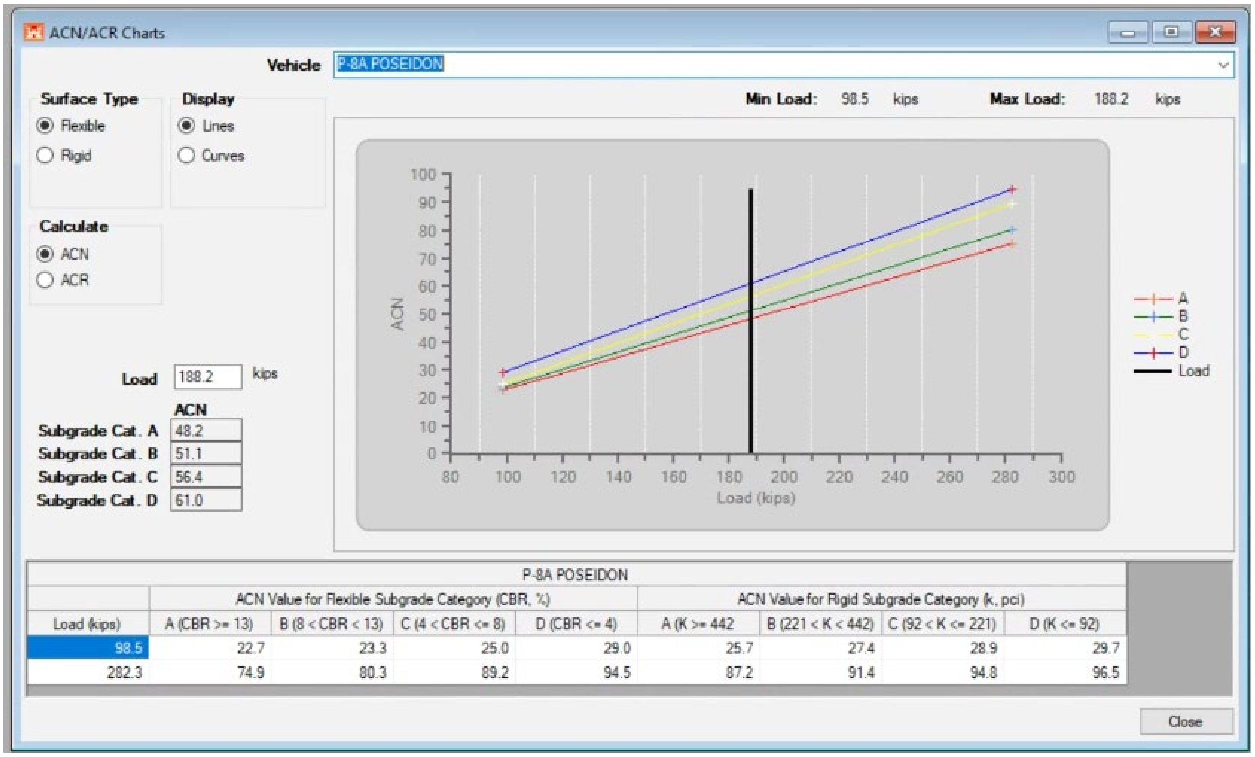


Table 9-5 ACN Ratings for P-8 Operating at 188.2 Kips

Rigid	Flexible
56/R/A	48/F/A
59/R/B	51/F/B
61/R/C	56/F/C
62/R/D	61/F/D

9-2.8 Adjusted ACN Due to Increase/Decrease in Tire Pressure.

Tire pressure is a secondary factor in determining an ACN; however, ICAO procedures can determine the increase or decrease in ACN if the aircraft is operating at a tire pressure different from the one used to determine the ACN. The adjusted ACN can be used if conditions, such as a thin asphalt surface or a weak upper pavement layer, exist. Figures 9-2 and 9-3 are used to adjust the ACN for flexible pavements and Figure 9-4 is used for rigid pavements. On flexible pavements, it is assumed that adjustments will only be one category. Figures 9-2 and 9-3 were developed by ERDC based on the equation in FAA Advisory Circular 150/5335-5D, *Standardized Method of Reporting Airport Pavement Strength – PCR*.

Figure 9-2 Adjusting Flexible Pavement ACN Due to an Increase or Decrease in Tire Pressure (Z to Y or Y to Z)

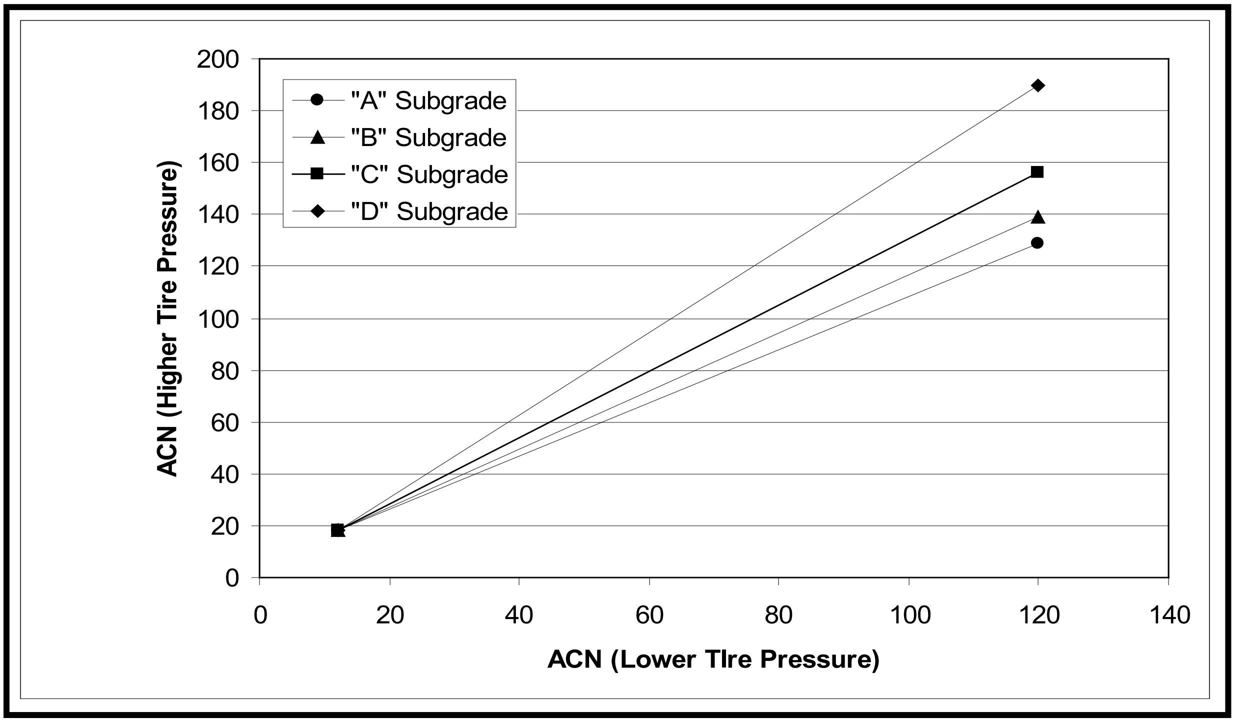


Figure 9-3 Adjusting Flexible Pavement ACN Due to an Increase or Decrease in Tire Pressure (Y to X, X to Y, X to W, W to X)

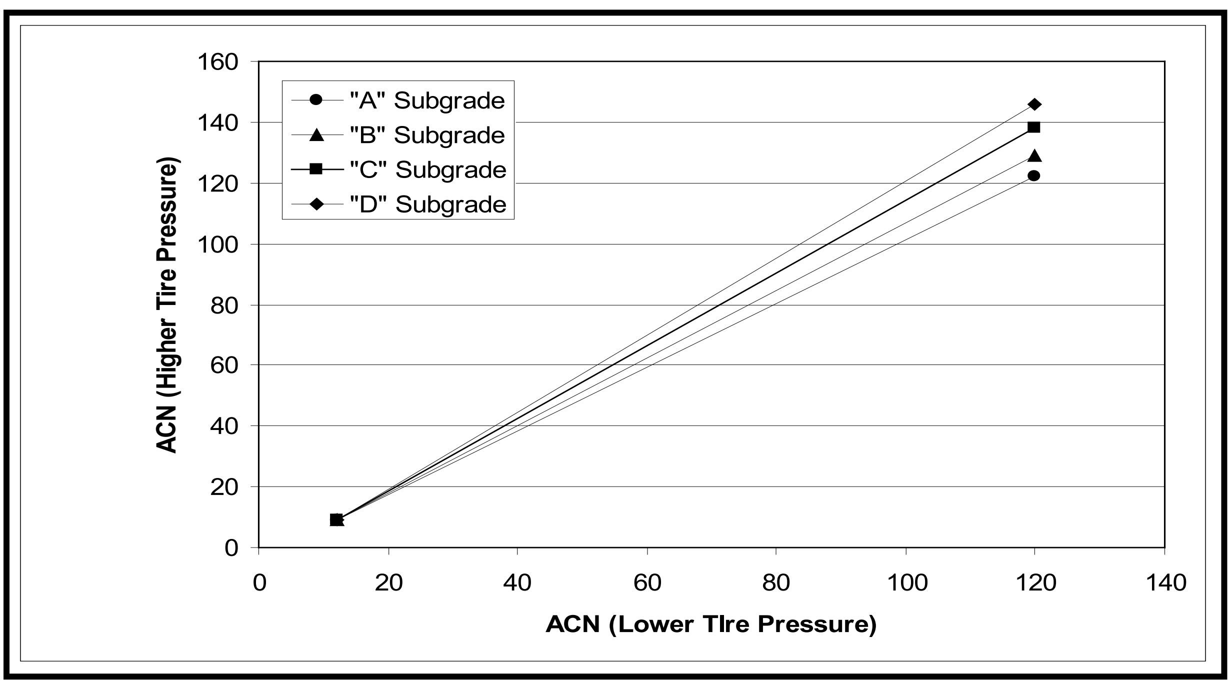
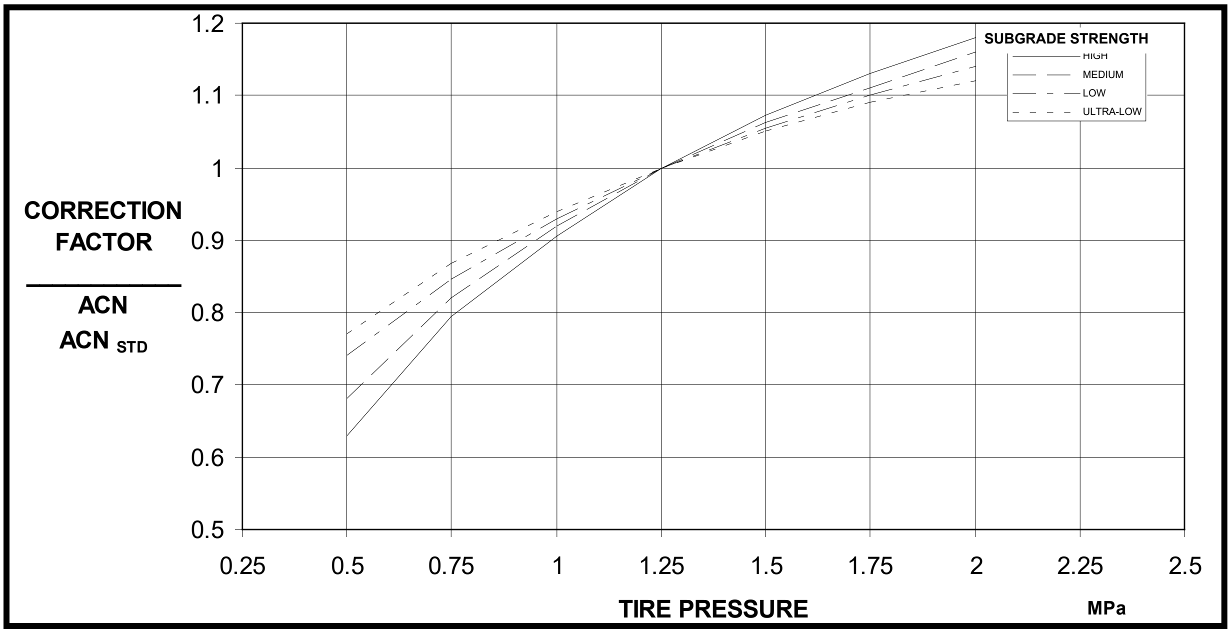


Figure 9-4 Adjusting Rigid Pavement ACN for Changes in Tire Pressure



9-2.8.2 Flexible Pavement ACN Adjustment Due to Tire Pressure.

The ACN for an aircraft was determined to be 60/F/D using a tire pressure of 160 psi (1.1 MPa). Using Figure 9-3, the ACN will be 55/F/D if the tire pressure is reduced to 140 psi (1.0 MPa).

9-2.8.3 Rigid Pavement ACN Adjustment Due to Tire Pressure.

An aircraft operating at a tire pressure of 180 psi (1.2 MPa) on a medium-strength subgrade has an ACN of 50/R/B. The ACN will be 53/R/B if the tire pressure is increased to 215 psi (1.5 MPa). (Enter Figure 9-4 with a tire pressure of 1.5 MPa and proceed vertically until the medium subgrade is intercepted. Proceed horizontally and read the correction factor of 1.06. (50 x 1.06 = 53).

9-3 PCN DEFINITION.

The PCN is a number that expresses the bearing strength (load-carrying capability) of a pavement based on a specified aircraft in terms of a standard single-wheel load at a specified number of passes. The aircraft and number of passes are determined based on the traffic approach as defined in Chapter 4 that is used in the evaluation. This can be a set number of passes of a specific aircraft (e.g., 50,000 passes of the C-17), the equivalent passes of the controlling aircraft over the next 20 years as defined in the mission aircraft group approach, or equivalent passes for representative groups as defined in the mission/representative aircraft group approach. The PCN value is for reporting pavement strength only. The PCN value expresses the results of pavement evaluation in relative terms and cannot be used for pavement design or as a substitute for a structural evaluation.

9-3.1 Computing the PCN.

The numerical PCN value is simply the ACN value of the specified aircraft at the specified number of passes as defined in paragraph 9-3. The entire PCN is a five-part code made up of the numerical portion of the PCN as well as a four-letter code that defines the pavement type, the subgrade category, and the tire pressure described below, as well as the procedure used to determine the numerical PCN value, the using aircraft method, or the technical evaluation method. ICAO procedures permit member states to determine how PCN values will be determined based upon internally developed pavement evaluation procedures. DoD PCN values are based on the technical evaluation method whenever possible.

9-3.1.1 Using Aircraft Method.

The using aircraft method is a simple procedure where ACN values for all aircraft currently permitted to use the pavement facility are determined and the largest ACN value is reported as the PCN. An underlying assumption is that the pavement structure has the structural capacity to accommodate all aircraft in the traffic mix and that each aircraft can operate on the pavement structure without restriction. Using an excessively damaging aircraft that uses the pavement on a very infrequent basis to determine the PCN can result in significant over-estimation of the pavement capacity. This procedure can also result in significant under-estimation of the pavement capacity, preventing acceptable traffic from operating. Use of the using aircraft method is discouraged due to these concerns.

9-3.1.2 Technical Evaluation Method.

The accuracy of a technical evaluation is better than the using aircraft method but requires more time and resources. Pavement evaluation may require a combination of on-site testing and engineering judgment. Numerical PCN values for DoD are determined from an allowable load determined by a technical pavement evaluation conducted in accordance with this UFC.

9-3.1.3 Numerical PCN Value.

Report the numerical PCN value in whole numbers, rounding off any fractional parts to the nearest whole number. For pavements of diverse strengths, the weakest section PCN value normally controls the reported numerical PCN value. Engineering judgment is required if the weakest section is not in the most heavily used part of the runway. In this case, consider another representative section as the basis for the reported PCN.

9-3.1.4 Pavement Type.

Pavement types are either flexible or rigid structures. Table 9-6 lists the pavement codes for purposes of reporting PCN.

Table 9-6 Pavement Codes for Reporting PCN

Pavement Type	Pavement Code
Flexible	F
Rigid	R

9-3.1.4.1 Flexible Pavement.

Flexible pavements support loads through bearing rather than flexural action. They are normally composed of several layers of selected materials designed to gradually distribute loads from the surface to the layers beneath. For a CBR evaluation, each layer in the pavement structure is evaluated to determine structural capacity. The layer that produces the lowest allowable gross load (AGL) is the controlling layer. For a Layered Elastic Evaluation Program (LEEP) evaluation, the tensile strain in the AC surface layer and the vertical strain in the subgrade are used to determine the structural capacity. For the two location points in a LEEP evaluation, the location that produces the lowest allowable load controls the AGL. For LEEP evaluations, the default locations are the bottom of the AC surface layer and the top of the subgrade.

9-3.1.4.2 Rigid Pavement.

Rigid pavements employ a single structural layer, which is very stiff or rigid, to support the pavement loads. The rigidity of the structural layer and resulting beam action enable a rigid pavement to distribute loads over a large area of the subgrade. The load-carrying capacity of a rigid structure is highly dependent upon the strength of the structural layer, which relies on uniform support from the layers beneath.

9-3.1.4.3 Composite Pavement.

Various combinations of pavement types and stabilized layers can result in complex pavements that could be classified as either rigid or flexible. A pavement section may comprise multiple structural elements representative of both rigid and flexible pavements. Composite pavements are most often the result of pavement surface overlays applied at various stages in the life of the pavement structure. If a pavement is of composite construction, the pavement type should be reported as the type which provides the highest allowable load.

9-3.1.5 Subgrade Strength Category.

As discussed in paragraph 9-2.2, there are four standard subgrade strengths identified for calculating and reporting ACN or PCN values. The standard values for rigid and flexible pavements are shown in the section on ACNs in Tables 9-1 through 9-4.

9-3.1.6 Allowable Tire Pressure.

Table 9-7 lists the allowable tire pressure categories used in the ACN-PCN system. The tire pressure codes apply equally to rigid or flexible pavement sections; however, the application of the allowable tire pressure differs substantially for rigid and flexible pavements.

Table 9-7 Tire Pressure Codes for Reporting PCN

Category	Code	Tire Pressure Range
Unlimited	W	No pressure limit
High	X	Pressure ≤ 254 psi
Medium	Y	Pressure ≤ 181 psi
Low	Z	Pressure ≤ 73 psi

9-3.1.6.2 Tire Pressures on Rigid Pavements.

Tire pressure has little effect on pavements with PCC surfaces. Rigid pavements are inherently strong enough to resist high tire pressures and can usually be rated as code W. However, when the rigid layer is very thin (less than 4 inches [102 millimeters]) or is thoroughly shattered (PCI ≤ 25, with pieces less than about 3 feet [1 meter] wide), the tire pressure code is reduced to X.

9-3.1.6.3 Tire Pressures on Flexible Pavements.

Tire pressures may be restricted on asphaltic concrete, depending upon the quality of the asphalt mixture, climatic conditions, or thickness and condition of the surface. Tire pressure effects on an asphalt layer relate to the stability of the mix to resist shearing or densification. A poorly constructed asphalt pavement can be subject to rutting due to consolidation under load. A properly prepared and placed mixture that conforms to DoD specifications can withstand tire pressures more than 254 psi (1.8 MPa) and be rated as tire pressure code W. A flexible pavement that has a PCI > 25 and is ≥ 4 inches (102 millimeters) thick but less than the minimum required thickness per UFC 3-260-02 is assigned code X. Pavement that has a PCI > 25 but is < 4 inches (102 millimeters) thick is assigned code Y. Pavement with a PCI ≤ 25 (aged or severely cracked pavements) is assigned code Y.

9-3.1.6.4 Method Used to Determine PCN.

As discussed in paragraph 9-3.1, two pavement evaluation methods are recognized in the PCN system. If the evaluation represents the results of a technical study, the evaluation method should be coded T. If the evaluation is based on “using airplane” experience, the evaluation method should be coded U. Technical evaluation implies that some form of technical study and computation were involved in the determination of the PCN. Using airplane evaluation means the PCN was determined by selecting the highest ACN among the airplanes currently using the facility.

9-3.2 Critical PCN for a Runway.

When selecting the critical PCN rating for a runway with multiple sections, it is important to examine the entire rating, not just the numerical value. A PCN rating that includes the lowest numerical value may not be the critical PCN rating. It depends on the subgrade category. Examine the AGL when PCN values with different subgrade categories are similar and then use the PCN rating with the lower AGL. Typically, this critical PCN selection for multiple-section runways will occur either within the 75-foot keel or the full-width ends in the first 1,000 feet of the runway.

9-3.3 Example PCN Reporting.

An example of a PCN code is 80/R/B/W/T, with 80 expressing the PCN numerical value, R for rigid pavement, B for medium-strength subgrade, W for high allowable tire pressure, and T for a PCN value obtained by a technical evaluation.

9-3.4 Reporting the PCN Value.

The Service determines the traffic analysis approach used to determine the allowable load for critical/representative aircraft and passes used in the PCN numerical computation for each section. The standard aircraft approach (e.g., the C-17 at a pavement life of 50,000 passes) facilitates PCN comparison between installations. The mission group approach looks at the critical mission aircraft and equivalent passes for a specific installation for a 20-year period. This provides more value to the installation but does not allow for a comparison of load-carrying capacity between multiple installations. The mission/representative approach bases the PCN on the mission aircraft group in terms of a representative aircraft for each gear type group. This last approach provides fidelity for managing pavements at the installation level while facilitating comparison between installations. Note that the PCN and the ACN/PCN procedure described in the following paragraphs provides the first look for managing aircraft traffic at an installation. Ultimately, the allowable loads and allowable passes should be used to manage operations when questions arise. Once a PCN value and the coded entries are determined, the PCN code should be reported to:

National Geospatial-Intelligence Agency (NGA)
Attn: Air Information Library, L27
3838 Vogel Rd.
Arnold MO, 63010

9-4 AIRCRAFT/PAVEMENT (ACN/PCN) CLASSIFICATION NUMBERS.

The ACN/PCN method is a weight-bearing capability reporting tool and is not an evaluation procedure. The NGA publishes PCNs from the Services in their FLIPs for civil and international use. The FLIPs are used to determine weight-bearing limits in terms of the ACN/PCN ratio. The intent is to avoid either overloading pavement facilities or refused landing permission by providing planning information for individual flights or multi-flight missions.

9-4.1 ACN/PCN Concept.

The pavement PCN for a pavement structure is simply the ACN for the selected or most critical aircraft. Under these conditions, any aircraft with an ACN equal to or less than the reported PCN value can safely operate on the pavement, subject to limitations on tire pressure.

9-4.2 Limitations of the ACN/PCN System.

The ACN/PCN system is only intended as a method of reporting relative pavement strength so airport operators can evaluate acceptable operations of airplanes. It is not intended as a pavement design or pavement evaluation procedure, nor does it restrict the methodology used to design or evaluate a pavement structure. Operators should use the allowable loads or allowable passes contained in each Service's pavement evaluation reports to manage day-to-day operations. The use of the standardized method of reporting pavement strength applies only to pavements with bearing strengths of 12,500 pounds (5,700 kg) or greater.

9-5 PAVEMENT OVERLOAD.

Pavement overloading can result from aircraft loads that are too high, a substantial increase in operations rate, or both. Loads larger than the defined design or evaluation load shorten the design life, while smaller loads extend it. Except for massive overloading, pavements are not subject to a particular limiting load above which they suddenly or catastrophically fail. The structural behavior of pavements is such that a pavement can sustain a definable load for an expected number of repetitions during its design life. As a result, occasional overloading is acceptable, when expedient, with only a limited loss in pavement life expectancy and a relatively small acceleration of the pavement deterioration rate. Examples of situations where operators may decide that it is acceptable to overload a pavement are emergency landings, short-term contingencies, exercises, and air shows.

9-5.1 Structural Index (ACN/PCN Ratio) Standards.

9-5.1.1 Structural Index (ACN/PCN) ≤ 1.1.

Structural index (SI) values less than or equal to 1.1 have minimal impact on pavement life.

9-5.1.2 1.1 ≤ Structural Index (ACN/PCN) ≤ 1.4.

When the SI value is greater than 1.1 and less than or equal to 1.4, limit aircraft operations to ten passes and inspect the pavement after each operation, or consult the AGL and Pass-Level tables, or request a pavement engineer evaluate the structural capacity of the pavement to support the required mission. Ensure the airfield surface meets aircraft and mission requirements such as FOD and smoothness.

9-5.1.3 Structural Index (ACN/PCN) ≥ 1.4.

When the SI value is greater than 1.4, do not allow aircraft operations except for emergencies, or request a pavement engineer evaluate the structural capacity of the pavement to support the required mission.

9-5.2 Aircraft Movements.

The annual number of movements by aircraft exceeding an ACN/PCN ratio of 1.0 should not exceed 5 percent of the total annual aircraft movements.

9-5.2.1 Aircraft Movements During a Thaw Period.

Movements by aircraft exceeding an ACN/PCN ratio of 1.0 are normally not permitted on pavements exhibiting substantial signs of distress or failure. If the pavement must be used for operations, perform an analysis using the PCN criteria in Chapter 8 during any periods of thaw-weakening following frost penetration or when the strength of the pavement or its subgrade could be weakened by the presence of water.

9-5.2.2 AGL/Pass Level Methodology.

The AGL/pass level methodology must be used to determine airfield structural capability when the ACN/PCN ratios exceeds a value of 1.1.

CHAPTER 10 REPORTING EVALUATION RESULTS

10-1 OVERVIEW.

Evaluation results are reported using maps, tables, and figures compiled in an evaluation report. The content of the report varies based on the scope and intended use of the evaluation by each Service, such as a contingency evaluation versus an evaluation at a main operating installation where the report is used to generate pavement management plans. In either case, the report always describes and discusses the type and number of tests performed, the analytical process used to determine thickness and strength values, and any limitations or assumptions.

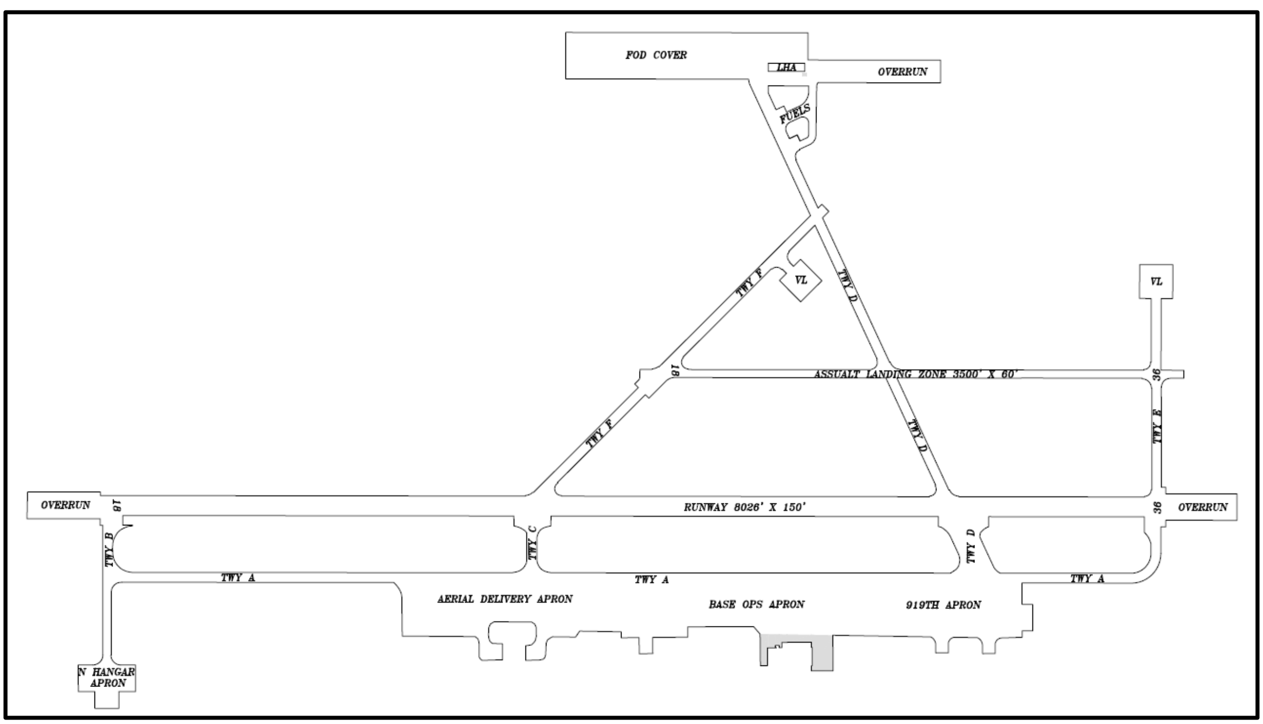
10-2 INVENTORY.

The pavement network, branch, and section inventory structure defines each pavement area for testing, analysis, and reporting. Tabular reports are sorted by branch and section or just by section, depending on the information presented and Service preferences. Maps also use branch and section IDs to organize and present inventory and analysis results. The following paragraphs present examples of inventory maps and tables.

10-2.1 Branch Maps.

Branch maps show the location of each branch on the airfield as shown in Figure 10-1. Variations of this basic map show both branches and sections or are color-coded to show condition data as shown in other examples in this chapter.

Figure 10-1 Branch Map Example



10-2.2 Section Maps.

Section maps show the location of each section on the airfield. They can include the pavement type and thickness as shown in Figure 10-2 or show only the section ID or branch and section ID as shown in Figure 10-3. Variations of these basic maps are color-coded to indicate condition data as shown in other examples in this chapter.

Figure 10-2 Section Map Example 1

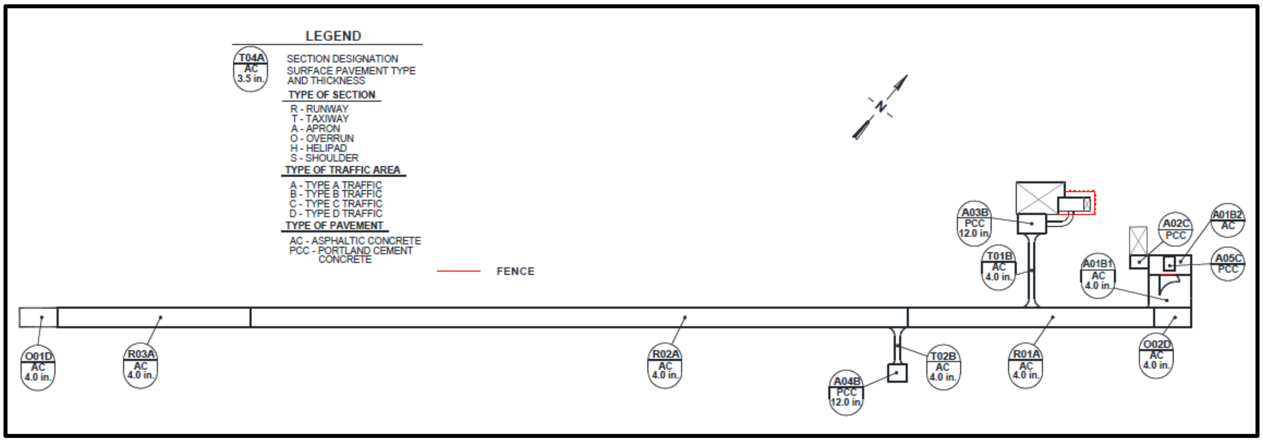
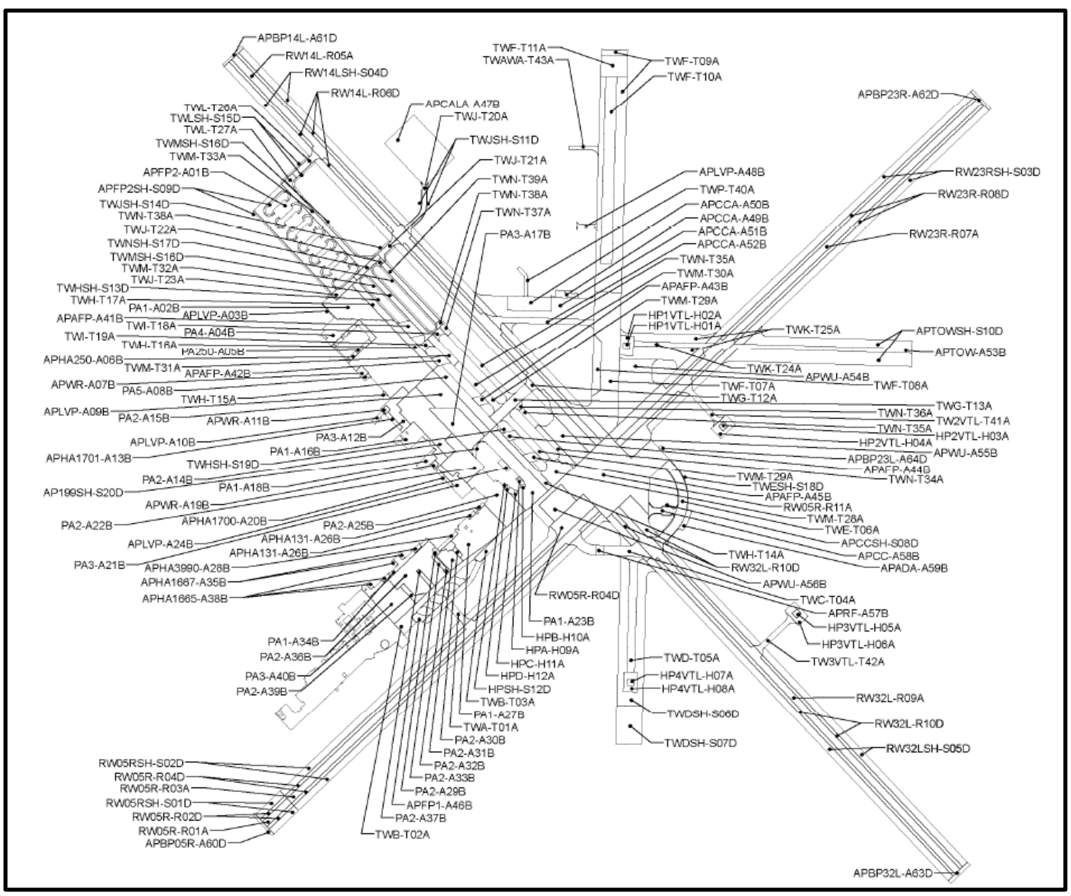


Figure 10-3 Section Map Example 2



10-2.3 Construction History.

Construction history, also known as work history, is integral to developing deterioration models and maintenance and repair (M&R) strategies. Typically, construction history is presented in tables and, while there are variations, construction history tables always have the section ID, the pavement type and thickness, and the construction date. Figure 10-4 is an example of a report generated by the PAVER application and Figure 10-5 is an example of a report generated in a spreadsheet. Variations of the report include information on subsurface layers or surface treatments.

Figure 10-4 Construction History Report Example 1

6/24/2021 **Work History Report** Page 1 of 3

Pavement Database: Mettie AS 2020_data_withGIS

Network: Mettie Airstrip, Yaki **Branch:** APNWTURN NW TURNAROU **Section:** A02B **Surface:**AC
L.C.D. 1/1/2019 **Use:** APRON **Rank:** P **Length:** 239.00 (Ft) **Width:** 114.00 (Ft) **True Area:** 27246.00000 (SqFt

Work Date	Work Code	Work Description	Cost	Thickness (in)	Major M&R	Comments
1/1/2019	NC-AC	New Construction - AC	0.00	0.00	☑	
1/1/2019	BA-AG	Base Course - Aggregate	0.00	0.00	☐	
1/1/2019	SG-CO	Subgrade - Compacted	0.00	0.00	☐	SILTY SAND (SP-SM)

Network: Mettie Airstrip, Yaki **Branch:** APSETURN SE TURNAROUN **Section:** A01B **Surface:**AC
L.C.D. 1/1/2019 **Use:** APRON **Rank:** P **Length:** 240.00 (Ft) **Width:** 112.00 (Ft) **True Area:** 26880.00000 (SqFt

Work Date	Work Code	Work Description	Cost	Thickness (in)	Major M&R	Comments
1/1/2019	NC-AC	New Construction - AC	0.00	0.00	☑	
1/1/2019	BA-AG	Base Course - Aggregate	0.00	0.00	☐	
1/1/2019	SG-CO	Subgrade - Compacted	0.00	0.00	☐	SILTY SAND (SP-SM)

Figure 10-5 Construction History Report Example 2

Pavement Section	Surface Pavement		Construction Date	Agency[a]
	Thicknesses in.	Type		
Runway 04-22				
R01A	24.0 20.0	PCC PCC	1957-1958 2021	IE CE
R02C	19.0[d]	PCC	1958-1959	CE
R03C	12.0[b] 2.0[c] 3.0[c] 17.0	AC AC AC PCC	1955-1956 1959 Unknown 2021	CE CE CE
R04A	16.0 21.5[d] 20.0	PCC PCC PCC	1956-1957 1996-2000 2019	CE CE CE

10-3 TRAFFIC.

An evaluation report typically includes a table that outlines the traffic used in the analysis. This may be one of the standard traffic patterns shown in Chapter 4 or the mission traffic as shown in Figure 10-6. In all cases, traffic tables include the aircraft used in the analysis, the load, the number of passes and, for a mixed traffic analysis, the controlling vehicle(s) and equivalent passes.

Figure 10-6 Mission Traffic Example

Aircraft from Table A-5	Aircraft Used for Evaluation in PCASE	Gross Weight lb	20-year Projected Aircraft Passes	20-year Equivalent C-17 Passes
Main Pattern PCC and AC Pavements[a]				
C-17	C-17A	585,000	50,000	50,000
20-year Total Equivalent C-17 passes @ 585,000 lb = 50,000				
Fixed-Wing Aircraft	Aircraft Used for Evaluation in PCASE	Gross Weight lb	20-year Projected Aircraft Passes	20-year Equivalent C-130 Passes
Limited Pattern PCC Pavements[a]				
A-10	A-10	50,000	360	31
A6E	A-6 Intruder	60,421	20	115
H64D	AH-64	18,000	52,140	1
BE20/C12	C-12J Huron	16,600	20,780	1
C-130J-30	C-130J-30 Hercules	164,000	1,560	1,560
D0328/C146	C-146 Wolfhound	30,843	60	1

10-4 ACN CHARTS AND TABLES.

The report uses charts, tables, or both to present ACN data. In all cases, the information includes the aircraft, pavement type, load, subgrade category, and the ACN.

10-4.1 ACN Table Example.

Figure 10-7 contains ACN values for the representative aircraft in Table 4-5 at various evaluation loads and Figure 10-8 shows an example at a single load. Figure 10-9 shows the ACN values for a single aircraft at varying operational loads.

Figure 10-7 ACN Values for Representative Aircraft

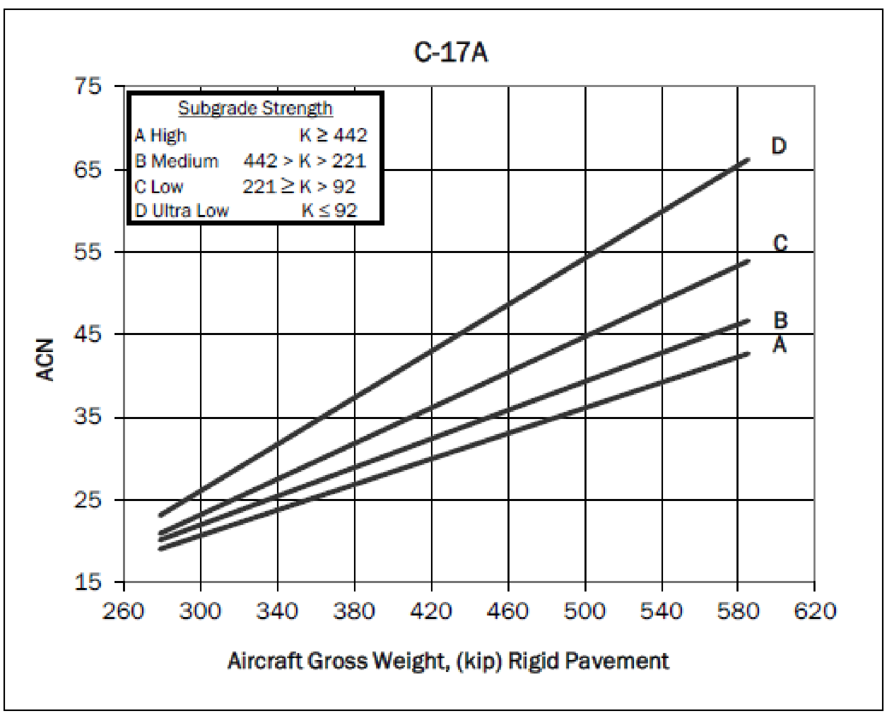
AIRCRAFT CLASSIFICATION NUMBERS										
AIRCRAFT	LOAD	MAX TAKE OFF WEIGHT (LBS)	RIGID PAVEMENT				FLEXIBLE PAVEMENTS			
			A	B	C	D	A	B	C	D
			K > 442	K < 442 K > 221	K < 221 K > 92	K < 92	CBR>13	CBR<13 CBR>8	CBR<8 CBR>4	CBR<4
			E>64,822	E<64,822 E>26,618	E<26,618 E>8,640	E<8,640	E>19,500	E<19,500 E>12,000	E<12,000 E>6,000	E<6,000
F-35C LIGHTNING II	FULL	70,400	30.1	30.1	30.1	30.1	28.2	28.2	28.2	28.2
	HALF	52,787	22.5	22.5	22.5	22.5	21.2	21.2	21.2	21.2
	EMPTY	35,174	14.9	14.9	14.9	14.9	14.1	14.1	14.1	14.1

AIRCRAFT	LOAD	MAX TAKE OFF WEIGHT (LBS)	RIGID PAVEMENT				FLEXIBLE PAVEMENTS			
			A	B	C	D	A	B	C	D
			K > 442	K < 442 K > 221	K < 221 K > 92	K < 92	CBR>13	CBR<13 CBR>8	CBR<8 CBR>4	CBR<4
			E>64,822	E<64,822 E>26,618	E<26,618 E>8,640	E<8,640	E>19,500	E<19,500 E>12,000	E<12,000 E>6,000	E<6,000
P-8A POSEIDON	FULL	188,200	55.7	58.7	61.1	62.3	48.2	51.1	56.4	61.0
	HALF	143,348	40.7	43.1	45.0	46.0	35.5	37.2	40.7	45.0
	EMPTY	98,495	25.7	27.4	28.9	29.7	22.7	23.3	25.0	29.0
C-130H HERCULES	FULL	175,000	30.9	34.1	37.3	39.7	27.4	32.0	34.8	40.6
	HALF	122,000	21.2	23.1	25.1	26.7	18.7	21.8	23.6	27.2
	EMPTY	69,000	11.4	12.1	12.9	13.7	9.9	11.5	12.3	13.7
C-17A GLOBE-MASTER III	FULL	585,000	42.7	46.7	53.9	66.3	50.5	57.0	68.5	90.2
	HALF	432,000	30.9	33.4	37.4	44.7	35.0	38.9	46.3	60.6
	EMPTY	279,000	19.0	20.1	20.9	23.1	19.5	20.7	24.0	31.0
KC-135 STRATO-TANKER	FULL	323,000	35.0	42.8	51.2	58.0	36.7	40.8	49.4	63.8
	HALF	213,650	22.1	26.3	31.2	35.4	23.0	25.2	30.0	38.3
	EMPTY	104,300	9.1	9.8	11.1	12.7	9.3	9.6	10.5	12.7
MV-22 OSPREY	FULL	60,500	10.6	11.7	13.6	15.4	12.2	13.2	14.1	14.6
	HALF	47,016	8.0	8.8	10.0	11.6	9.1	9.9	10.6	11.0
	EMPTY	33,531	5.3	5.9	6.4	7.7	6.0	6.5	7.0	7.4

Figure 10-8 ACN Values for Controlling Aircraft

Design Aircraft	Weight, lb	Subgrade Category[a]	ACN or Required PCN
PCC Pavements			
C-17A	585,000	A	43
		B	47
		C	54
		D	66
C-130-J	164,000	A	34
		B	37
		C	39
		D	42
AC Pavements			
C-17A	585,000	A	40
		B	45
		C	53
		D	70
C-130-J	164,000	A	31
		B	34
		C	36
		D	42

Figure 10-9 ACN Chart Example



10-5 REPORTING TEST RESULTS.

As discussed in Chapter 3, testing typically includes a mix of one or more of the following: coring or drilling, GPR, or MIRA testing to determine pavement thickness; GPR testing to determine subsurface layer structure; DCP testing to determine subsurface layer structure and strength; and FWD/HWD testing to determine the modulus of the layers in the pavement structure. Although not used as frequently as in the past, test pits are an option. The scope and intended use of the evaluation typically defines testing requirements. A comprehensive report includes test results to ensure readers have the necessary information to make decisions and provides a foundation for future evaluations.

10-5.1 Surface Condition.

The PCI provides an objective measure of the surface condition that is used in most instances. Use a cursory inspection with a direct rating of GOOD, FAIR, or POOR for contingency evaluations when a full PCI inspection is not performed. In either case, report the surface condition in either a table or a map.

10-5.1.1 Surface Condition Maps.

Surface condition maps vary depending on the scope and intended use of the evaluation. Maps show either the standard seven-tier PCI rating scale for a full PCI as shown in Figure 10-10 or a three-color condition map as shown in Figure 10-11. If a full PCI is not performed or when Service standards dictate, use three-color maps. Figure 10-10 includes the section information with the pavement types. At a minimum, the map includes either a branch or section ID.

Figure 10-10 Seven-Color PCI Map Example

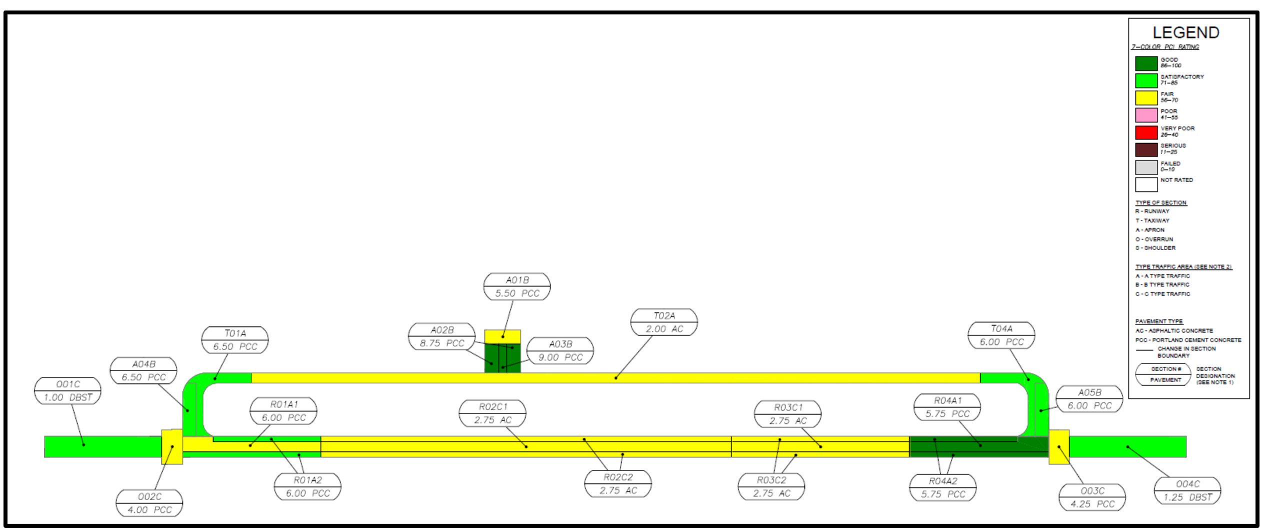
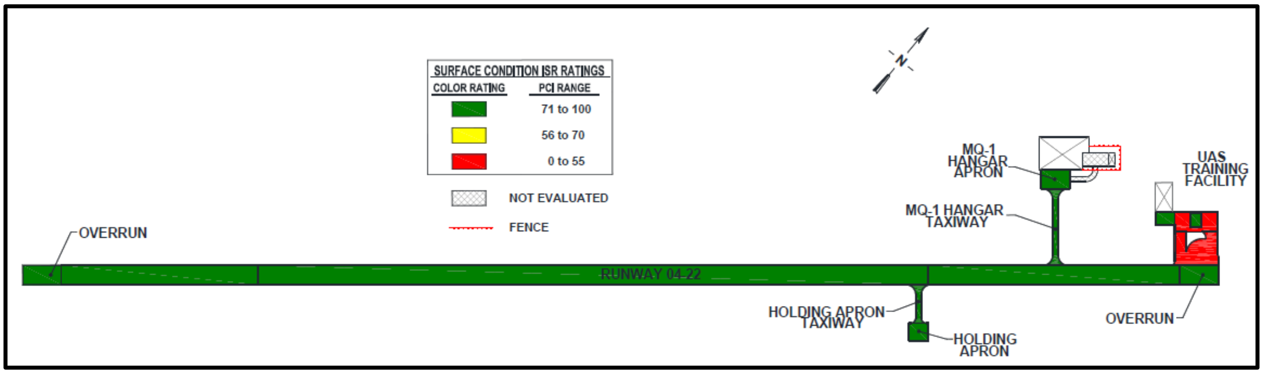


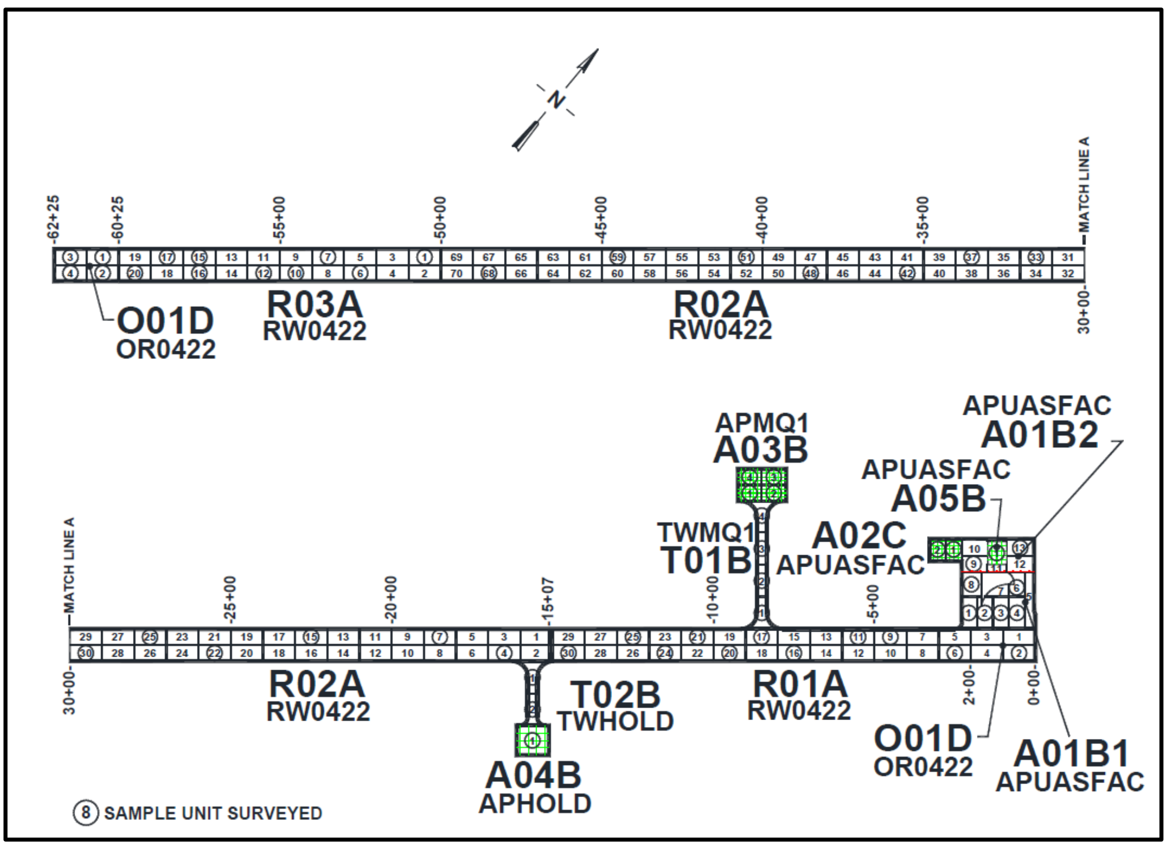
Figure 10-11 Three-Color PCI Map Example



10-5.1.2 Sample Unit Maps.

Sample unit maps define the location of each sample unit in an inspection. Sample unit maps show the section delineation and identification and include each sample unit with its number. In addition, sample unit maps for rigid pavements include the slab layout. All sample units are numbered and inspected samples have a circle around the number.

Figure 10-12 Sample Unit Map Example



10-5.1.3 Surface Condition Table.

The PCI is often included in other tables such as the physical property data (PPD) or M&R summary tables to provide context. When the PCI is the primary focus, summarize the data at the branch level as shown in Figure 10-13 or at the section level as shown in Figure 10-14, depending on the intended use. In either case, at a minimum, a surface condition table shows the branch and/or section ID, the condition, and last inspection date. Tables typically include other inventory information as well as the deterioration rate and predicted PCI.

Figure 10-13 Branch Condition Report Table

6/24/2021 **Branch Condition Report** Page 1 of 2

Pavement Database: Mettie AS 2020_data_withGIS

Branch ID	Number of Sections	Sum Section Length (Ft)	Avg Section Width (Ft)	True Area (SqFt)	Use	Average PCI	Standard Deviation PCI	Weighted Average PCI
APNWTUR	1	239.00	114.00	27,246.00	APRON	100.00	0.00	100.00
APSETURN	1	240.00	112.00	26,880.00	APRON	100.00	0.00	100.00
APUAS	1	40.00	40.00	1,600.00	APRON	55.00	0.00	55.00
RW1230	3	5,200.00	100.00	520,000.00	RUNWAY	100.00	0.00	100.00
TW1	1	1,082.00	80.00	87,594.00	TAXIWAY	100.00	0.00	100.00
TW2	1	2,664.00	60.00	185,435.00	TAXIWAY	100.00	0.00	100.00
TWUAS	3	182.00	15.00	2,735.00	TAXIWAY	77.00	24.54	83.93

Figure 10-14 Section Condition Report Table

Branch ID	Section ID	Surface	Use	Rank	Length (ft)	Width (ft)	Slabs	Slab Dimensions (ft)		Joint Length (ft)	True Area (sqft)	True Area (sy)	Last Inspection Date	Age at Time of Inspection (yrs)	Last Major Work Date	SCI	PCI Pct Load	PCI	Predicted PCIs	
								Width	Length							2018		2018	2021	2024
AP199SH	S20D	AC	SHOULDER-AF	T	331	40					11,601	1,289	11/8/18	8	01/06/11	93	0	32	6	0
APADA	A59B	PCC	APRON	P	326	74	131	12	15	3,138	24,508	2,723	11/8/18	43	06/15/76	100	0	85	84	83
APAFP	A41B	PCC	APRON	P	200	255	271	12	15	7,018	50,848	5,650	11/8/18	43	06/01/76	100	7	85	84	83
APAFP	A42B	PCC	APRON	P	200	130	140	12	15	3,473	26,290	2,921	11/8/18	43	06/01/76	91	41	87	86	85
APAFP	A43B	PCC	APRON	P	154	130	110	12	15	2,659	20,532	2,281	11/8/18	43	06/01/76	96	23	85	84	83
APAFP	A44B	PCC	APRON	P	200	130	140	12	15	3,480	26,189	2,910	11/8/18	43	06/01/76	94	30	84	83	82
APAFP	A45B	PCC	APRON	P	200	130	137	12	15	3,480	25,745	2,861	11/8/18	43	06/01/76	100	0	98	98	98
APBP05R	A60D	AAC	APRON	S	37	400					14,533	1,615	11/8/18	38	01/01/81	88	0	24	18	12
APBP14L	A61D	AAC	APRON	S	35	400					14,000	1,556	11/8/18	38	01/01/81	63	0	32	27	21
APBP23L	A64D	PCC	APRON	S	70	50	12	17	17	291	3,500	389	11/8/18	8	01/01/11	100	0	93	90	88
APBP23R	A62D	AAC	APRON	S	40	397					15,197	1,689	11/8/18	38	01/01/81	73	0	46	42	37
APBP32L	A63D	AAC	APRON	S	55	394					21,865	2,429	11/8/18	38	01/01/81	100	0	11	4	0
APCALA	A47B	PCC	APRON	P	881	611	2,867	12	15	77,457	537,491	59,721	11/8/18	34	06/15/85	100	0	94	93	93
APCC	A58B	PCC	APRON	S	420	119	235	13	15	6,791	44,005	4,889	11/8/18	43	06/15/76	100	0	95	95	94

10-5.2 Dynamic Cone Penetrometer (DCP) Report.

DCP reports include the location of the evaluation as well as the date and location or number of the test. Include the blow and penetration data as well as a data plot. This is typically a CBR plot, but a plot of k or other values may be presented as well. Identify the hammer correlation and soil correlation used to determine the CBR values. The layer structure used to populate the PPD table may also be included on the graph, in tabular form, or both. See Figure 10-15.

Figure 10-15 DCP Test Report

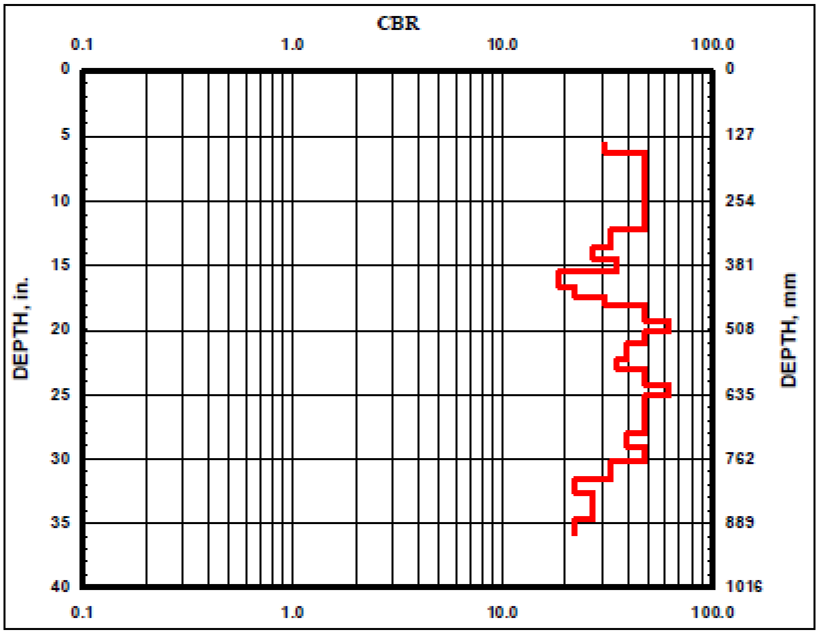
DCP TEST DATA

Project: *Stallion AAF* Date: 3-Oct-09
Location: R01A-Sta 0+65, 20' W of CL Soil Type(s):

Hammer
○ 10.1 lbs.
◉ 17.6 lbs.
○ Both hammers used

Soil Type
○ CH
○ CL
◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	145	1
2	160	1
5	185	1
5	210	1
5	235	1
5	260	1
5	285	1
5	310	1
5	345	1
3	370	1
3	390	1
3	425	1
2	445	1
2	460	1
2	470	1
4	490	1
5	510	1
5	535	1
5	565	1
3	585	1



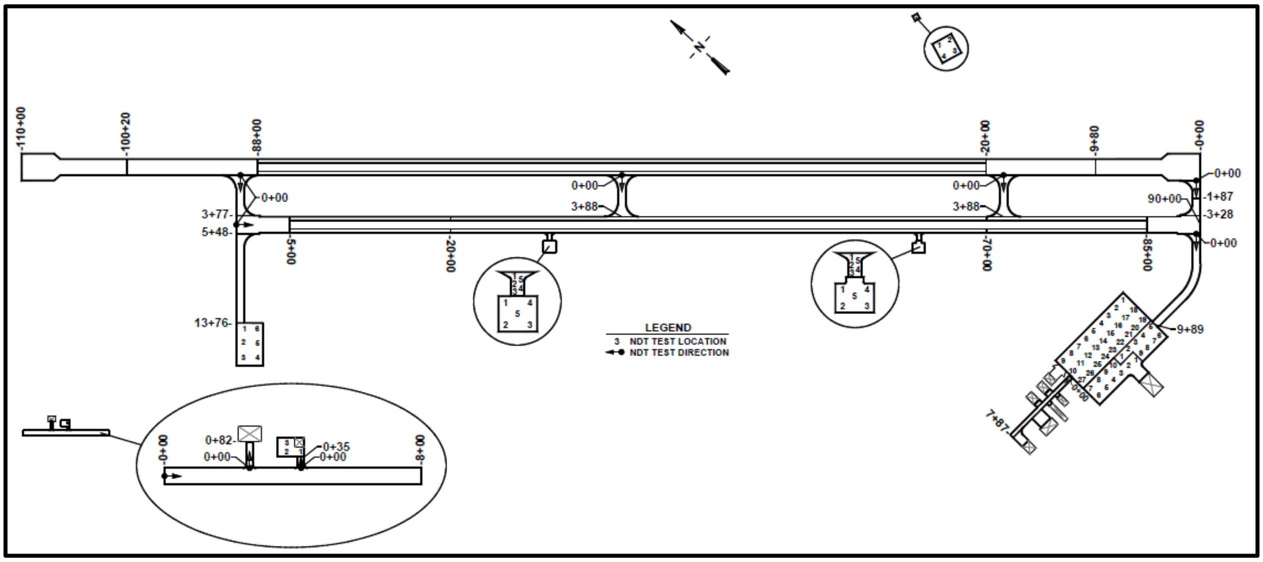
10-5.3 Falling/Heavy Weight Deflectometer (FWD/HWD) Data.

As described in paragraph 3-4.1, FWD is the generic term for the device, with the HWD capable of applying a heavier load than other FWDs, but the terms are used synonymously. FWD data is reported using one or more report types, including maps, charts, and tables.

10-5.3.1 FWD Data Maps.

FWD data maps report the test locations. They can vary from a map that shows the general location of tests to one based on the GPS coordinates of each test location. The latter are typically included at a larger scale. Figure 10-16 shows a map of the general location and direction of testing. Both the general map and the more detailed GPS-based map show distance measurements on long, linear structures such as a runway. The distance measurement can be used to identify the test location or, more typically, the test is just given a number. The term station is used for either the test number or the test location.

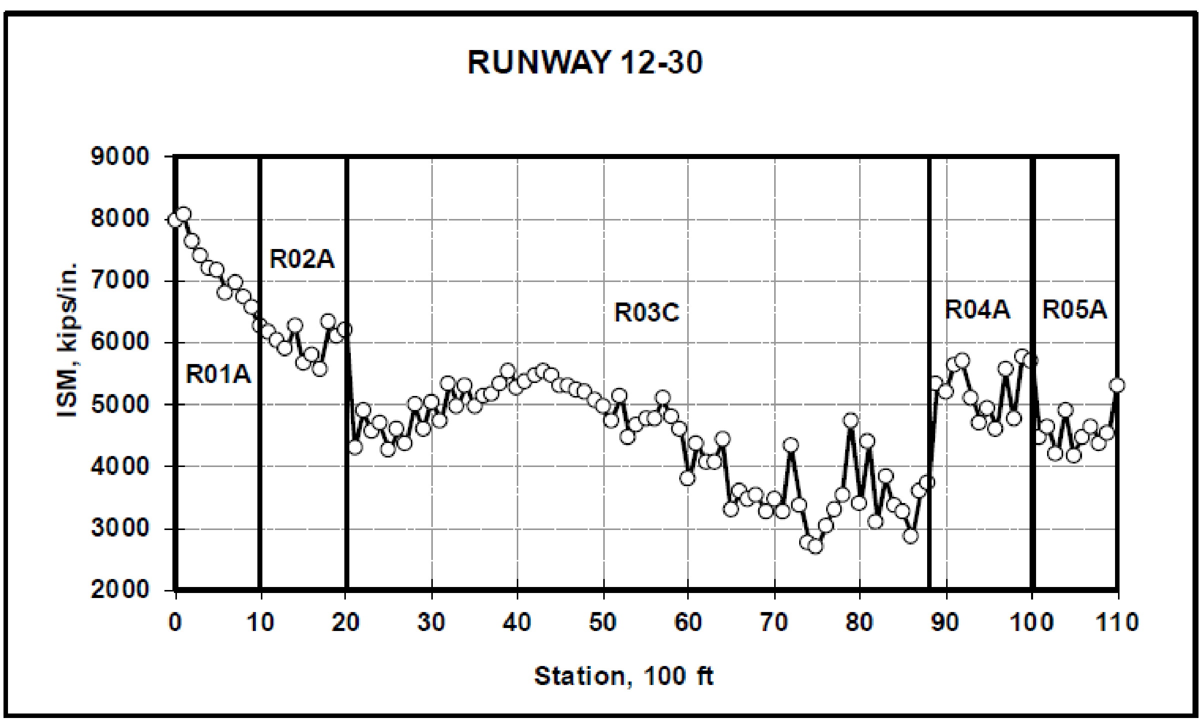
Figure 10-16 FWD Test Location Map



10-5.3.2 FWD Data Charts.

FWD data charts typically display the impulse stiffness modulus (ISM) as described in paragraph 5-3.5.1. These ISMs provide a quantitative stiffness comparison between test stations and between pavement sections. The ISM values are plotted on the Y axis for each station (test point) in the section. These data are used to visually determine if a change in strength exists and define where sections change when the FWD file has basin data from multiple sections, as shown in Figure 10-17.

Figure 10-17 ISM Plot



10-5.3.3 FWD Data Tables.

When included in a report, FWD data are typically limited to the representative basin that is used for analysis for each section (see paragraph 5-3.8) rather than reporting all basin data. The representative basin data is presented in a table that includes the section identification, the ISM, and load, as well as the deflection data (in mils) for each FWD sensor. Figure 10-18 shows a typical representative basin table.

Figure 10-18 Representative Basin Table

Section	ISM, kips/in.	Load, lb	Deflection, mils						
			D1	D2	D3	D4	D5	D6	D7
Runway 12-30									
R01A	7,182	49,985	6.96	6.41	5.85	5.42	5.00	4.52	4.03
R02A	6,075	50,303	8.28	7.95	7.38	6.93	6.38	5.70	5.15
R03C	4,411	50,062	11.35	10.74	9.78	8.82	7.83	6.68	5.62
R04A	5,323	50,040	9.40	8.88	8.16	7.71	6.93	6.05	5.30
R05A	4,602	49,930	10.85	10.26	9.48	8.73	7.84	6.96	5.94

10-5.4 Ground Penetrating Radar (GPR).

As described in paragraph 3-4.2, GPR is used to determine pavement and soil layer thicknesses and determine the presence of voids. When used to determine thickness, the average thickness values for a section are populated in the physical property data table. When used to identify the location of anomalies such as voids, images from the GPR application may be included in the report.

10-6 PHYSICAL PROPERTY DATA (PPD).

PPD is the term used to describe the pavement layer structure. When included in a report, these data are presented in tabular format. At a minimum, this table will include the section ID as well as the thickness, description, and strength index or property for each layer. Flexural strength is reported for any PCC layer. The description can range from a general layer description (e.g., drainage layer or stabilized base) to a USCS when soil laboratory testing was performed. The table may include the backcalculated modulus value as shown in Figure 10-19 or a strength index like CBR or k for soil layers as shown in Figure 10-20. When testing includes coring and DCP tests, it is typical to report the CBR and k values. Reporting modulus values is typically done when only HWD data is collected although there may be instances when a PPD table may include both, e.g., some sections had coring, DCP, and HWD data collected because they were new or when the impulse stiffness modulus was below 400, making an airfield pavement analysis (APE) analysis preferable for that section.

Figure 10-19 PPD Table with Modulus

Branch	Section	Length (ft)	Width (ft)	General Condition PCI	Overlay Pavement			Pavement			Base			Subbase			Subgrade	
					Thick-ness[a] (in.)	Type	Flex. Str.[a] (psi)	Thick-ness[a] (in.)	Type	Flex. Str. (psi)	Thick-ness[a] (in.)	Material[a]	Modulus psi	Thick-ness[a] (in.)	Material[a]	Modulus (psi)	Material[a]	Modulus (psi)
TWB	T07A	500	150	Good				15.0	PCC	650	4.0 6.0	Drainage GW	225,000[c] 225,000[c]				Lean Clay (CL)	18,031
	T08A	3,000	75	Fair				5.0	AC		10.0 4.0	GW Drainage	–	6.0	Separation	–	Lean Clay (CL)	–
	T09A	5,000	75	Fair				4.0	AC		11.0 4.0	GW Drainage	–	6.0	Separation	–	Lean Clay (CL)	–

Figure 10-20 PPD Table with CBR/k

SUMMARY OF PHYSICAL PROPERTY DATA Installation Name																
SECT	IDENT	OVERLAY PAVEMENT			PAVEMENT			BASE			SUBBASE			SUBGRADE	
		THICK (in)	DESCRP	FLEX (psi)	THICK (in)	DESCRP	FLEX (psi)	THICK (in)	DESCRP	K/CBR	THICK (in)	DESCRP	K/CBR	DESCRP	K/CBR
A01B	South Arm/Disarm Pad	-	-	-	12.25	PCC	710	6.00	$GP\text{-}GM_{88}$	400 -	5.00	SC_{88}	275 -	$SP\text{-}SM_{88}$	200 -
A02B	Apron 01	-	-	-	14.00	PCC	575	6.00	STABILZED BASE	400 -	8.00	$SP\text{-}SM_{88}$	225 -	$SM\text{-}SC_{88}$	125 -
A03B	Apron 01	-	-	-	12.75	PCC	650	6.00	$GW\text{-}GM_{88}$	400 -	5.00	$SP\text{-}SM_{88}$	325 -	SC_{88}	300 -

10-7 BACKCALCULATION RESULTS.

Backcalculation results are presented in a layered elastic modulus table. This table may include either the backcalculated modulus values for each section or the modulus values used for Layered Elastic Evaluation Program (LEEP) analysis for each section, the distinction being that the latter shows values that were manually set for analysis as in the case of capping PCC modulus values or using the temp option for asphalt modulus values in analysis. At a minimum, a modulus table shows the section ID and the modulus for each layer. It may also include information on the layer type (e.g., PCC or AC), the layer thickness (including depth to bedrock), or the percent error of closure for reported backcalculated modulus values. Figure 10-21 shows a layered elastic modulus value generated by the Pavement-Transportation Computer Assisted Structural Engineering (PCASE) application.

Figure 10-21 Layered Elastic Modulus Table

Layered Elastic Model Data Installation Name															
Section	Layer 1				Layer 2				Layer 3			Layer 4		
	Thickness (in)	Type	Modulus (psi)	Flex. Str. (psi)	Thickness (in)	Type	Modulus (psi)	Flex. Str. (psi)	Thickness (in)	Type	Modulus (psi)	Thickness (in)	Type	Modulus (psi)
A01B	10.00	PCC	3,930,965	616	-	-	-	-	-	-	-	230.00	SUBG	18,025
A02B	5.50	AC	240,208	650	6.00	BASE	52,969	-	12.00	SUBAS	31,000	216.50	SUBG	22,888
A03B	8.25	PCC	4,586,409	727	-	-	-	-	-	-	-	231.75	SUBG	27,573
A04B	5.50	AC	360,647	650	6.00	BASE	70,334	-	-	-	-	228.50	SUBG	31,907
A05B	10.00	PCC	2,679,401	427	-	-	-	-	-	-	-	230.00	SUBG	27,622
A06B	6.00	PCC	2,982,971	474	12.00	BASE	73,316	-	24.00	SUBAS	28,220	198.00	SUBG	19,936
A07B	5.50	AC	249,994	650	6.00	BASE	100,998	-	-	-	-	204.50	SUBG	22,328

10-8 ANALYSIS RESULTS.

Analysis results are typically in tables but the results from these tables may also be displayed on maps. At a minimum, analysis results tables will include the section ID, allowable passes, allowable gross load (AGL), Pavement Classification Number (PCN), and the basis of the PCN. These results are reported in one or more tables, e.g., PCNs may be reported in a separate table than allowable passes and AGLs. These tables may also include other data, such as overlay requirements, critical aircraft, Aircraft Classification Number (ACN, and the structural index (ACN/PCN ratio). The presentation of the results is largely defined by whether the analysis is done based on standard, mission, or mission/representative aircraft groups as well as whether an individual or mixed traffic analysis was performed.

10-8.1 Aircraft/Gear Type Load Table.

An analysis results table may be organized based on specified loads for specific aircraft or gear types. In this case, the report in Figure 10-22 shows the PCN and design passes (basis of the PCN) as well as the allowable passes.

Figure 10-22 Aircraft/Gear Type PCN – Allowable Pass Table

GEAR TYPE	ST			DT			STT			DTT			TRT		
AIRCRAFT	F-35C			P-8A			C-130H			KC-135			C-17A		
SECTION	PCN	DESIGN PASSES	ALLOWA BLE PASSES	PCN	DESIGN PASSES	ALLOWA BLE PASSES	PCN	DESIGN PASSES	ALLOWA BLE PASSES	PCN	DESIGN PASSES	ALLOWA BLE PASSES	PCN	DESIGN PASSES	ALLOWA BLE PASSES
APCR-A06D	35/R/C/W/T	87,393	564,964	75/R/C/W/T	289	1,033	32/R/C/W/T	298,545	237,157	52/R/C/W/T	10,866	12,360	47/R/C/W/T	2,239	789
APLA-A05D	38/R/C/W/T	87,393	1,585,438	79/R/C/W/T	289	1,545	34/R/C/W/T	298,545	404,170	54/R/C/W/T	10,866	17,982	49/R/C/W/T	2,239	1,112
APLA-A49D	24/F/B/W/T	7,406	1,202	46/F/B/W/T	331	181	29/F/B/W/T	5,872	8,458	43/F/B/W/T	725	1,013	52/F/B/W/T	216	540
APRP-A08D	9/F/A/W/T	8,543,298	4,407	31/F/A/W/T	20,041	278	17/F/A/W/T	8,928,664	24,662	27/F/A/W/T	159,846	1,554	29/F/A/W/T	18,873	709
APRP-A09D	39/R/B/W/T	80,153,228	UNLIMITED	86/R/B/W/T	20,054	1,624,848	37/R/B/W/T	> 99 M	UNLIMITED	57/R/B/W/T	2,552,572	UNLIMITED	54/R/B/W/T	1,092,516	4,685,595

10-8.2 Aircraft Group Allowable Gross Load (AGL) Table.

An aircraft group AGL table is organized by groups of aircraft with similar characteristics. AGLs are reported for specified pass levels of each group. These tables may be color-coded to indicate when the AGL is above the maximum aircraft load for the group (green), when it is between the maximum and minimum load for the group (yellow), and when it is below the minimum load for the group. The AGL table is accompanied by a separate table that defines the aircraft in each group and the pass levels. Figure 10-23 shows an AGL table from PCASE.

Figure 10-23 Aircraft Group AGL Table

PAVEMENT CAPACITY IN KIPS FOR AIRCRAFT GROUP INDEX NUMBERS																
SECTION	PCN	PASS INTENSITY LEVEL	1	2	3	4	5	6	7	8	9	10	11	12	13	14
R01A2	17/R/B/W/T	I	27	29	45	82	42	53	58	146	162	202	382	222	338	121
		II	31	34	52	91	48	59	65	164	181	224	423	248	377	142
		III	36	39	58	109	57	70	77	195	215	262	496	294	448	176
		IV	43	47	69	140	72	89	97	246	273	323	612	370	565	224
R02C1	19/F/B/W/T	I	34	36	67	100	61	77	85	181	190	301	574	261	405	183
		II	39	41	76	109	66	84	93	198	208	329	628	286	443	206
		III	42	45	84	124	75	95	105	224	234	371	709	323	500	258
		IV	48	51	94	155	94	119	132	281	294	466	889	404	626	322
R02C2	19/F/B/W/T	I	34	36	67	100	61	77	85	182	190	302	576	262	406	183
		II	39	41	77	110	66	84	93	199	208	330	630	286	444	206
		III	42	45	84	124	75	95	105	224	235	372	711	323	501	259
		IV	48	51	95	155	94	119	132	281	294	467	891	405	628	323

10-8.3 Controlling Aircraft AGL Table.

A controlling aircraft AGL table is organized by section like the other tables but defines the controlling aircraft load and passes and reports the AGL and PCN. The example in Figure 10-24 also shows overlays.

Figure 10-24 Controlling Aircraft AGL Table

Pavement Facility	Section	Test Number or Station, ft	Type Traffic Area	Subgrade Strength[a] CBR or K, % or psi/in.	Design Aircraft[b]				Allowable Gross Load, kips	PCN	Theoretical Overlay Requirements, in.		
					Aircraft	Weight, lb	Passes	ACN			AC	PCC No Bond	PCC Partial Bond
Taxiway Maintenance	T27B	0+00-5+00	B	174	C-17	585,000	9,436	54/R/C/W/T	528	48/R/C/W/T	6.1	6.5	8.6
Hanger 2,3,4 TW	T28B	5+00-6+00	B	136	CH-47	50,000	5,321	11/R/C/W/T	50	18/R/C/W/T	-	-	-
	T29B	6+00-5+00	B	116	CH-47	50,000	5,321	11/R/C/W/T	50	18/R/C/W/T	-	-	-
	T30B	5+00-5+00	B	171	CH-47	50,000	5,321	11/R/C/W/T	50	20/R/C/W/T	-	-	-
	T31B	5+00-5+00	B	300	CH-47	50,000	5,321	10/R/B/W/T	50	22/R/B/W/T	-	-	-
Assault Ramp TW	T20B	0+00-8+00	B	5	CH-47	50,000	11,495	10/F/C/W/T	50	12/F/C/W/T	-	-	-
Compass Rose Ramp	A01B	1-5	B	207	C-17	585,000	9,436	54/R/C/W/T	567	52/R/C/W/T	-	-	-
Maintenance Ramp	A02B1	1-7	B	199	C-17	585,000	9,436	54/R/C/W/T	284	22/R/C/W/T	17.8	11	12.8

10-8.4 Pavement Classification Number (PCN) Table.

As with other reports, PCN tables are organized by section and show the PCN for each respective section. These tables may also show the structural index (ACN/PCN ratio) for the controlling aircraft or may show the controlling PCN for a branch with multiple sections. Figure 10-25 shows an example of a basic PCN table from PCASE.

Figure 10-25 PCN Table

PAVEMENT CLASSIFICATION NUMBER Normal Period							
SECTION	PCN	SECTION	PCN	SECTION	PCN	SECTION	PCN
A01B	21/R/C/W/T	A08B	2/R/C/W/T	H08A	3/R/C/W/T	R05A	116/F/A/W/T
A02B	16/F/A/W/T	A09C	29/R/C/W/T	H09A	4/R/D/W/T	R06A	135/F/A/W/T
A03B	19/R/B/W/T	A10B	12/R/B/W/T	H11A	18/R/C/W/T	T09A	15/F/A/Y/T
A04B	32/F/A/W/T	A11B	15/R/C/W/T	R01C	201/F/A/W/T	T14A	175/F/A/Y/T
A05B	16/R/B/W/T	H02A	10/R/C/W/T	R02A	102/F/A/W/T	T15A	34/F/A/Y/T
A06B	5/R/B/W/T	H04A	11/R/C/W/T	R03A	97/F/A/W/T	T16A	2/F/A/Y/T
A07B	19/F/A/W/T	H06A	8/R/C/W/T	R04C	156/F/A/W/T	T17A	120/F/B/Y/T

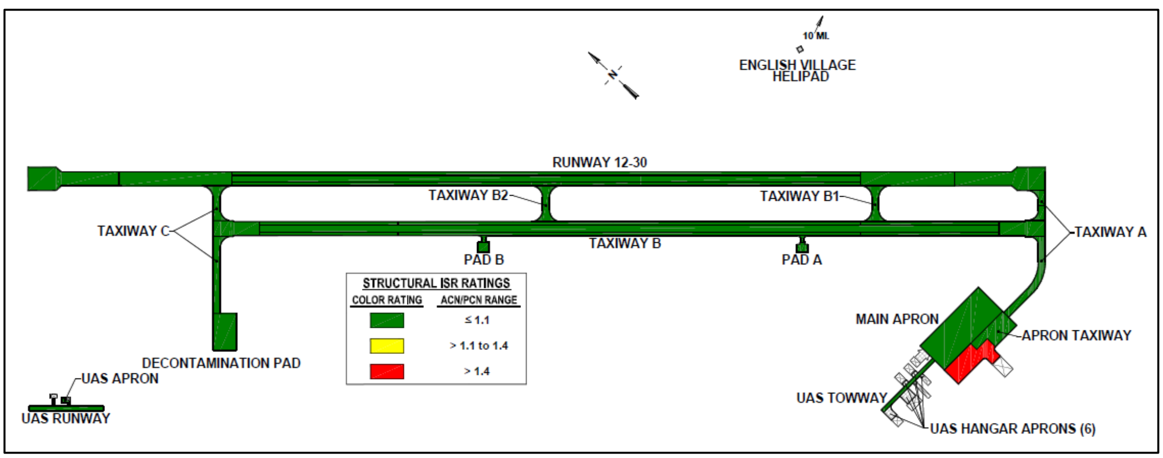
10-8.5 Structural Index (ACN/PCN) Ratio.

As described in paragraph 9-5.1, the structural index (SI) is used to define whether a pavement is structurally adequate to sustain mission traffic. As mentioned in paragraph 10-8.4, the SI can be incorporated in other tabular reports as shown in the example in Figure 10-26 in which it is included in the PCN table or it can be displayed in a map as shown in Figure 10-27. This objective is to indicate whether each pavement section is structurally capable of supporting the anticipated mission traffic for the expected life of the pavement. The concept can be extended to address the issue of service life, which is discussed in paragraph 10-8.6.

Figure 10-26 SI Example

SECTION	PRIMARY AIRCRAFT	ACN	PCN	STRUCTURAL INDEX
A01B	T-6A	3/R/B	21/R/B/W/T	0.14
A02B	T-6A	3/R/C	23/R/C/W/T	0.13
A03B	T-6A	3/R/B	31/R/B/W/T	0.10
A04B	T-6A	3/R/B	33/R/B/W/T	0.09
A05B	T-6A	3/R/B	18/R/B/W/T	0.17

Figure 10-27 Pavement Life Expectancy Based on SI



10-8.6 Pavement Life Expectancy.

The ACN/PCN ratio concept can be extended to similar Service-specific criteria as described below and shown in the color-coded example map in Figure 10-28. In this example, the ACN of the critical aircraft at various load levels (loaded, half-loaded, and unloaded aircraft) is compared to the PCN for the section. These ACN/PCN relationships correspond to the color codes shown below:

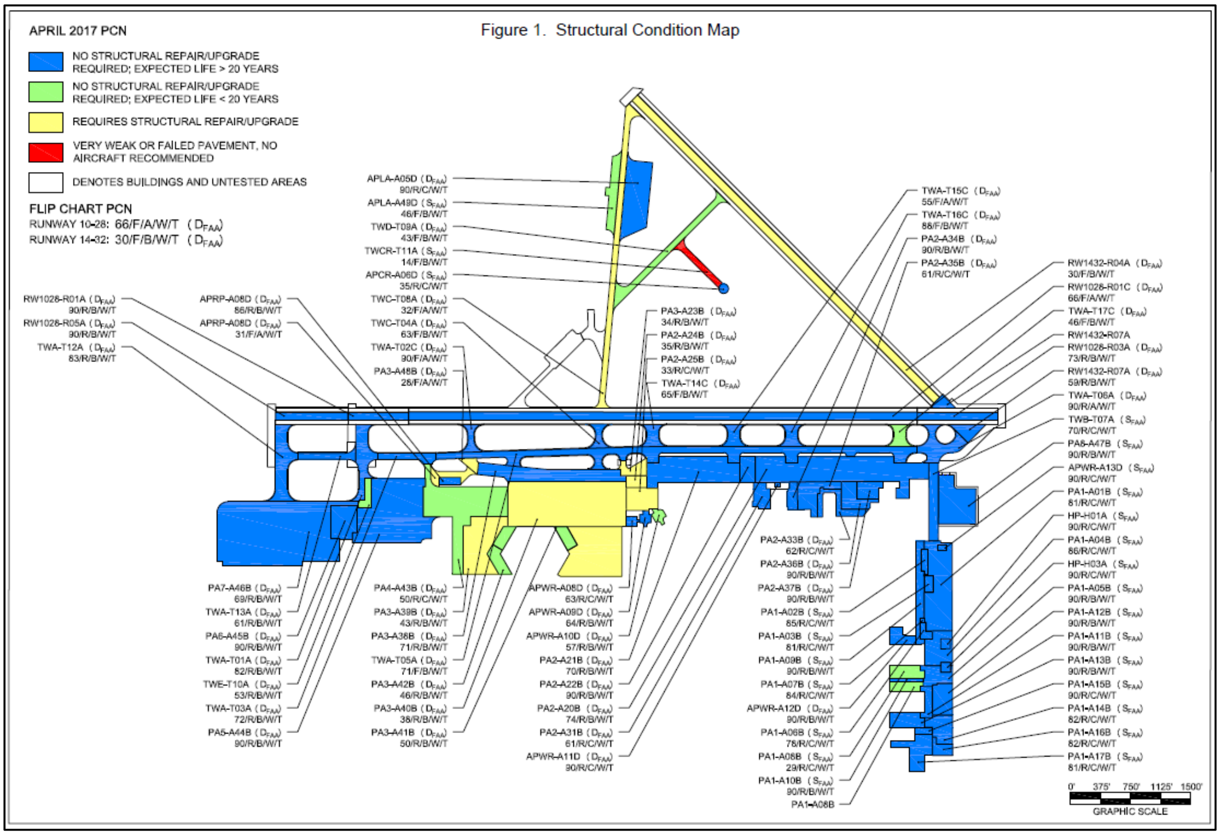
- B (BLUE): $ACN_{\text{fully loaded}} \leq PCN$
- G (GREEN): $ACN_{\text{half-loaded}} \leq PCN \leq ACN_{\text{fully loaded}}$
- Y (YELLOW): $ACN_{\text{empty}} \leq PCN \leq ACN_{\text{half-loaded}}$

- R (RED): PCN ≤ ACN_{empty}.

Figure 10-28 depicts this 4-color structural condition map in terms of life expectancy:

- B (BLUE): Expected pavement life is greater than 20 years
- G (GREEN): Expected pavement life is less than 20 years
- Y (YELLOW): Pavement in need of structural repair/upgrade
- R (RED): Very weak or failed pavement, limit or cease aircraft operations

Figure 10-28 Pavement Life Expectancy Based on Service-Specific Criteria



10-8.6.1 Life Expectancy Weight Restrictions.

The example map in Figure 10-28 also identifies weight restrictions based on the color code. The color codes below define the limitations necessary (at the original level of passes) to ensure the section will provide a service life of twenty years.

- B (BLUE): No weight restriction
- G (GREEN): Reduce departure weights by 50 percent
- Y (YELLOW): Reduce use to unloaded aircraft
- R (RED): Aircraft traffic operations suspended until pavement is repaired.

10-8.6.2 Life Expectancy Pass Level Restrictions.

The colors in the map in Figure 10-28 also imply pass level restrictions (at the original weight) necessary to ensure that the Section provides a 20-year service life. Areas with a blue color code can accommodate pass level increases up to 50 percent without significantly affecting the pavement service life.

APPENDIX A SAMPLING AND TESTING METHODS

A-1 INTRODUCTION.

The Table A-1 provides a summary of the sampling and testing procedures used in pavement evaluation and published standards for the respective tests. The remainder of this appendix provides an overview of some of the commonly used test methods and details on procedures that do not have established standards.

Table A-1 Sampling and Testing Procedures

Testing or Sampling Procedure	Publication
General Testing	
Pavement condition index (PCI) inspection	UFC 3-260-16 ASTM D5340
Falling weight deflectometer (FWD)	ASTM D4694 and D4695
Ground penetrating radar (GPR)	ASTM D6432
Pavement coring	Paragraph A-2.4
Dynamic cone penetrometer	TM 3-34.48-2
Asphalt Testing	
Sampling bituminous paving mixtures	ASTM D979
Unit weight, Marshall stability, and flow of bituminous mixtures	CRD-C 649
Density and percent voids of compacted bituminous paving mixtures	CRD-C 650
Recovery of asphalt from solution by Abson method	ASTM D1856
Extraction of bitumen from bituminous paving mixtures	ASTM D2172
Asphaltic concrete recompaction	Paragraph A-3.1
Penetration of bituminous materials	ASTM D5
Ductility of bituminous materials	ASTM D113
Softening point of asphalt and tar materials	ASTM D36
Test for bitumen	ASTM D4

Testing or Sampling Procedure	Publication
Concrete Testing	
MIRA ultrasonic tomography	ASTM C597
Flexural strength of concrete	ASTM C78 as modified in paragraph A-4.1
Compressive strength tests	ASTM C39
Splitting tensile strength tests	ASTM C496
Specific gravity of concrete	ASTM C642
Absorption by concrete	ASTM C642
Voids in concrete	ASTM C642
Soil Testing	
In-place density, sand cone method	ASTM D1556
In-place (field) CBR	CRD-C 654
Laboratory CBR relations of soils	CRD-C 654
Moisture-density relations of soils	CRD-C 653
Sieve analysis	ASTM C136
Particle size analysis	ASTM D6913
Specific gravity of soils	ASTM D854
Specific gravity and absorption of coarse aggregate	ASTM C127
Specific gravity and absorption of fine aggregate	ASTM C128
Moisture content of soil or aggregate (total sample)	ASTM D2216
In-place density, drive cylinder method	ASTM D2937
Liquid limit, plastic limit, and plasticity of soils	ASTM D4318
Soils sampling	ASTM D1586M
Plate-bearing tests	CRD-C 655
Classification tests	ASTM D2487

Testing or Sampling Procedure	Publication
Sampling and preparing test specimens	ASTM C42
Flexural strength of soil-cement	ASTM D1635
Deep, quasi-static, cone, and friction-cone penetration tests of soils	ASTM D3441

Note: ASTM is the designation of standards and test methods issued by the American Society for Testing and Materials.

A-2 GENERAL PAVEMENT EVALUATION TESTING.

A-2.1 Pavement Condition Index (PCI) Inspection.

UFC 3-260-16 implements the PCI inspection procedures in ASTM D5340 for DoD. Additional details are found in the PAVER User manual and Distress Identification Manuals posted on the Tri-Service Pavement-Transportation site, https://transportation.erdc.dren.mil/paver/Manuals.htm

A-2.2 Falling Weight Deflectometer (FWD).

Chapter 3 provides DoD-specific procedures on use of the FWD. This guidance supplements that found in the Dynatest user manual, ASTM D4694, and ASTM D4695.

A-2.3 Ground Penetrating Radar (GPR).

Chapter 3 provides a general description on DoD GPR use. Appendix B addresses GPR use for void detection. This guidance supplements the manufacturer's user manual and ASTM D6432.

A-2.4 Pavement Coring and Drilling.

Chapter 3 provides general DoD guidance on the number and locations for taking pavement core samples.

A-2.5 Dynamic Cone Penetrometer (DCP).

Chapter 3 provides a general overview of the types of DCP devices used by DoD as well as guidance on the number and locations for drilling pavement to perform DCP testing.

The DCP (or automated DCP) consists of a rod that is driven into the soil using a 17.6-pound hammer dropped from a constant height of 22.6 inches (574 millimeters). The manual system is portable and has options that automate data collection so testing is performed by a single operator. DCPs are designed to penetrate to a depth of 36 to 48 inches (900 to 1,220 millimeters), which is typically sufficient to test weak areas for voids.

A-2.6 Standard Penetration Test.

The standard penetration test (SPT) is also called the split-spoon test because of the split-barrel used for soil sampling. The test (ASTM D1586) provides a representative soil sample and a measure of the soil resistance to penetration by driving a split-barrel sampler using a 140-pound mass from a 30-inch height. The number of blows is recorded for each 6-inch increment of penetration and is assumed to be representative of the soil strength. Typically, the DCP has been easier to conduct than the SPT.

A-3 ASPHALT TESTING.

A-3.1 Recompaction of Asphaltic Concrete.

Samples of existing pavements may be recompacted in the laboratory for comparison with the in-place conditions. Pavement samples should be approximately 10-inch (254-millimeter) maximum dimension so the various layers or course can be identified. If the pavement consists of more than one course, the courses should be separated and treated individually. The courses may be separated by heating the pieces of pavement and driving a hot knife between the layers or by other similar methods. After a course has been separated, break it into small pieces and heat it to a temperature of 240 °F to 260 °F (115 °C to 127 °C) as rapidly as possible in an oven or on a hotplate, with constant stirring to ensure uniform heating. Thoroughly mix the material during heating and compact the hot mixture in accordance with the standard Marshall method procedures. Compact samples with 50 or 75 blows on each side of the specimen for comparison with criteria for tire pressures of 100 psi and 200 psi (0.7 MPa and 1.4 MPa), respectively. Compact six to eight specimens with each effort and test in accordance with standard procedures for the Marshall method. Note that reheating produces a hardening of the asphalt cement. This hardening causes somewhat higher stability values but has little effect on the other test values when analyzing the test data.

A-3.2 All-Bituminous Concrete and Flexible Overlays.

Use the same procedures described in paragraph A-3.1 when testing all-bituminous concrete base-course material and flexible overlays except when the all-bituminous concrete or flexible overlay exists between two thicknesses of rigid pavement (composite pavement). In this case, the only test necessary on the bituminous concrete portion of the overlay is an extraction test to determine the gradation of the aggregate and the bitumen content, so only one or two samples of the bituminous concrete are needed from each test pit. Take a large enough sample of the base course portion of the flexible overlay for a gradation test.

A-3.3 AC Layer Separation by Construction.

Split the cores at the interface of each AC layer (or lift) when a flexible pavement consists of more than one AC layer so that each layer can be tested separately. Test each layer for Marshall stability, flow, percentage of asphalt by weight, penetration of bitumen, aggregate type, shape and gradation, specific gravity of bitumen and aggregate, and density (CRD-C 649). Evaluate each course of the cores for percentage of asphalt by weight, aggregate gradation, and specific gravity according to ASTM D2172, ASTM D2726, and ASTM D5444 when the pavement was designed according to Superpave criteria. Compute the voids in the total mix and the percentage of voids filled with asphalt from the test results (CRD-C 650, ASTM D2041 and ASTM D2726).

A-3.4 AC Core Extraction Analysis.

Determine aggregate gradation, specific gravity of bitumen and aggregate, and penetration, ductility, and softening point of the bitumen from a portion of the samples. Other samples are recompacted as described in paragraph A-3.1 to determine Marshall stability, flow, density, and their voids relations. The stability of the cores cut from the pavement will often be lower than the recompacted sample. A part of this difference is due to differences in density since the field cores seldom have density as high as the laboratory-compacted samples. A major part of this variation in stability is due to differences in the structure of the field and laboratory samples and also the fact that the asphalt hardens some during reheating. Since the stability value is not the sole criterion for evaluating the mix, the lack of correlation between the stability of the field and laboratory samples is not particularly significant.

A-3.5 Resistance to Fuel Spillage.

There are currently no standard tests to determine resistance to spillage. However, spilling a small amount of jet fuel on one of the samples from each test pit to see if the fuel penetrates the samples quickly or if it “puddles” on the surface gives an indication of resistance to fuel spillage.

A-3.6 AC Separation Tests between PCC Layers.

The gradation and bitumen content of the bituminous concrete and the gradation of the base-course material, if any, are the only tests required when the non-rigid overlay is between two thicknesses of rigid pavement.

A-4 CONCRETE TESTING.

Retain all concrete cores collected and all test specimens cut from test pits for laboratory tests to determine the flexural strength. Visually examine concrete samples to determine the type of aggregate and estimate the maximum size of aggregate. Determine flexural strength from cores by conducting tensile splitting tests on 6-inch (152-millimeter) -diameter cores. Ensure the test specimens from pits are three times as long and three times as wide as the pavement thickness except when cutting 6- by 6-

inch (152- by 152-millimeter) beams from the top and bottom of the specimens for three-point load beam tests.

A-4.1 Flexural Strength Test.

Determine the flexural strength of rigid pavement using the third-point loading procedure set forth in ASTM C78, with the following modifications.

A-4.1.1 Test Specimens.

The test specimens should have a square section with the width and thickness equal to the pavement thickness for pavement thicknesses less than or equal to 12 inches (305 millimeters). For pavement greater than 12 inches (305 millimeters), either cut a square section with width and thickness equal to the pavement thickness or cut 6- by 6-inch (152- by 152-millimeter) beams from the top and bottom of the slab then average test to obtain a strength representative of the full section. When cutting 6- by 6-inch (152- by 152-millimeter) beams from the top and bottom of the slab, ensure the length of the specimen is three times the thickness of the specimen plus approximately 6 inches (152 millimeters).

A-4.1.2 Procedure.

Place the specimen in the third point loading apparatus and test it in the as-cast position. That is, apply the load at the third points on the surface of the beam that represents the pavement surface. Locate the load reaction on the bottom of the beam, which represents the bottom of the pavement.

A-4.2 Splitting Tensile Strength Tests.

The splitting tensile test can be conducted in the laboratory or in the field in accordance with ASTM C496 standard practices and uses the correlation in Equation A-3 to measure concrete flexural strength. Portable field splitting tensile test equipment is a modified version of the laboratory test equipment shown in Figure A-1.

Figure A-1 Portable Split Tensile Tester

The test involves laying a concrete core with its longitudinal axis horizontal and then applying a vertical compressive load at a constant rate along the longitudinal until the core fails in tension across the diameter from stresses induced by the compression load. Figure A-1 shows a failed specimen following a splitting tensile test. Record the diameter and the length of the core and maximum load at failure. Use Equation A-3 to calculate the tensile splitting strength and use the empirically developed relationship (WES, 1974) in Equation A-1 to compute flexural strength (Equation A-2 is a variation of Equation A-1).

Equation A-1 Flexural Strength

$$f = \left[\frac{2p}{\pi \cdot ld} \right] 1.02 + 210$$

Where:

f = *flexural strength (pounds per square inch)*

p = *applied load (pound-force)*

l = *length of the sample (inches)*

d = *diameter of the sample (inches)*

Equation A-2 Flexural Strength

$$F = 1.02T + 210$$

Where:

F = flexural strength in psi

T = tensile splitting strength in psi

The splitting tensile strength T is then computed from the equation:

Equation A-3 Tensile Splitting Strength

$$T = \frac{2P}{\pi l d}$$

Where:

P = maximum load at rupture, pounds-force (Newtons)

l = length of core, inches (millimeters)

d = diameter of core, inches (millimeters)

A-5 SOIL TESTING.

Conduct laboratory testing on samples of the base course, subbase course, and subgrade materials to classify them using the USCS in accordance with ASTM D2487. The size of the samples depends on the type of sampling and laboratory tests performed.

A-5.1 Disturbed Sampling.

Auger borings and bag samples are the two types of disturbed sampling used for airfield pavement evaluation.

A-5.1.1 Auger Borings.

The most suitable method of obtaining samples of the foundation materials for developing soil profiles is by auger borings. These borings are taken in test pits or through small 4- or 6-inch (102-millimeter or 152-millimeter) -diameter holes cored through the pavement. Take samples of the foundation materials at each 6-inch (152-millimeter) vertical increment to a depth of 2 feet (610 millimeter) and for each 12-inch (305-millimeter) increment thereafter to the desired depth. Take additional samples whenever there is a change in materials or moisture conditions. Seal the samples in clearly marked jars before transporting to the laboratory for moisture content testing and soil classification.

A-5.1.2 Bag Samples.

Bag samples of the foundation materials from test pits are used for compaction tests. Take samples of each type of material encountered. The size of the bag samples required depends on the type of material and the type of test to be performed. Collect a 100-pound (45 kilogram) sample of fine-grained soil for determining moisture-density. Collect a 450-pound (204-kilogram) sample of fine-grained soil when developing the moisture-density-CBR relationship. Increase the sample size to 200 pounds (90 kilograms) for the moisture-density tests and 600 pounds (272 kilograms) for moisture-density-CBR tests of granular soils.

A-5.2 Undisturbed Sampling.

Undisturbed samples may be required for laboratory CBR tests if the subgrade is composed of a fine-grained cohesive material. There is no prescribed method for obtaining undisturbed samples of subgrade material. Any method that provides enough material and maintains it in its existing condition is satisfactory. The method most widely used for undisturbed sampling is to trim a sample by hand to fit into a split cylinder of galvanized metal approximately 8 inches (203 millimeters) in diameter and at least 12 inches (305 millimeters) high. Seal the sample at the sides and ends with paraffin to prevent moisture loss.

A-5.3 Soil Testing for Rigid Pavements.

Collect bag samples of base and subbase courses underlying rigid pavements for classification and compaction tests. In general, a 200-pound (91 kilograms) sample is sufficient. However, when laboratory CBR tests are necessary, which may be the case when evaluating a non-rigid overlay on rigid pavements, a minimum 600-pound (272 kilogram) base-course sample is required. Determine gradation, Atterberg limits, specific gravity, and moisture-density relations. The moisture-density and CBR values may be required when evaluating a non-rigid overlay on rigid pavements. Perform an adaptation of the consolidation test on undisturbed samples of the subgrade to determine the correction for saturation of the plate-bearing test results. The undisturbed samples may also be used for density determinations. Soaked laboratory CBR tests on undisturbed subgrade material may be required when evaluating a non-rigid overlay on rigid pavement.

A-5.4 Soil Testing for Flexible Pavement.

Conduct tests on samples of base course, subbase, and subgrade materials, including Atterberg limits, gradation, dry soil color, and specific gravity, to classify the soil. Table A-2 summarizes testing requirements for project design that UFC 3-260-02 describes in more detail. Determine moisture-density and CBR relations from available data or from samples of base course, subbase, and subgrade materials remolded at three compaction efforts as described in CRD-C 653 and CRD-C 654. Take the base and subgrade samples in a manner that assures representative materials.

Table A-2 Flexible Pavement Sampling Requirements

Material	Samples Per Pit	Remarks
Pavement	8 cores, 200 pounds (91 kilograms) per sample	Samples should be 8 to 10 inches (203 to 254 millimeters) in minimum dimension to permit separation of courses
Base and subbase courses	600 pounds (272 kilograms)	Disturbed sample
	3 samples	Undisturbed cylinders to be taken of material with plastic fines where applicable
Subgrade	450 pounds (204 kilograms)	Disturbed sample; increase to 600 pounds (272 kilograms) if much coarse material is present
	3 samples	Undisturbed cylinders

A-5.5 Subgrade Soil Testing.

Collect a 100-pound (45-kilogram) bag sample of fine-grained material when samples of the subgrade are required. Obtain a 200-pound (91-kilogram) bag sample when the subgrade is composed of a granular material. If laboratory CBR tests are required, which may be the case in the evaluation of a non-rigid overlay on rigid pavements, increase the bag samples of subgrade material to 450 pounds (204 kilograms) and 600 pounds (272 kilograms) for fine-grained and granular materials, respectively.

A-5.6 Field Density Tests.

A-5.6.1 Field Density Frequency.

When taking samples of 0.5 cubic foot (0.014 cubic meter) volume or less, make three density determinations at each elevation tested. When taking larger samples, decrease the number of density determinations to two. When there is not a reasonable agreement between the tests results, perform two additional tests. A reasonable agreement is a tolerance of 5 pounds per cubic foot (80 kilograms per cubic meter) wet density. For example, test results of 108, 111, and 113 pounds per cubic foot (1,730, 1,778, and 1,810 kilograms per cubic meter) wet density are in reasonable agreement, and their average is 111 pounds per cubic foot (1,778 kilograms per cubic meter).

A-5.6.2 Field Density Test Procedure.

Field density tests are performed on the base course and subgrade materials. The most satisfactory methods of obtaining the density are by the sand-displacement or balloon methods when the base course or subgrade is composed of granular materials. These tests are described in ASTM D1556 and ASTM D2167, respectively. If the subgrade is composed of a fine-grained cohesive material, the density is best obtained either by drive-sampling (ASTM D2937) or balloon methods (ASTM D2167) or by the undisturbed sampling that may be required in connection with the plate-bearing test. Conduct all field density tests adjacent to the area that was loaded during the plate-bearing test. When the overlay portion of a non-rigid overlay on rigid pavement is composed of a bituminous concrete and base course, conduct density tests on the base-course portion of the overlay.

A-5.7 Moisture Content Tests.

The strength of base courses composed of substantial portions of fine materials is governed by the moisture content of the fine fraction. Therefore, moisture-content determinations are made on the fine-grained portion of the soil. The fine fraction is that portion passing any of several sieve sizes ranging from No. 200 to No. 4. For the purposes of this UFC, material passing the No. 40 sieve is the critical sieve size. This is the same sieve used for separations for liquid and plastic limit determinations. Determine the moisture content of both the material passing the No. 40 sieve and the total sample and recorded in the test data tables. If it is impractical to separate the material at the No. 40 sieve without affecting the moisture present, perform an absorption test following ASTM C127. The percentage of absorption thus determined is considered the moisture content of the coarse fraction. It is used to determine the moisture content of the remainder (assuming all other moisture to be in this finer fraction) mathematically. Comparing the moisture content of the material passing the No. 40 sieve with the liquid limit of the material is an indication of the stability of the base-course material. If the moisture content is near the liquid limit, the material is considered unstable. When the moisture content exceeds the liquid limit, the base material becomes more unstable as the percentage of fines increase.

A-5.8 California Bearing Ratio (CBR) Test.

A-5.8.1 CBR Test Locations within Test Pit.

When selecting CBR test locations in the test pit, place the CBR piston at a location that represents an average condition of the surface being tested, ensuring it is not set on unusually large pieces of aggregate or other unusual materials. The general practice is to space the CBR tests in the pit where the areas covered by the surcharge weights of the individual tests do not overlap. Perform these tests on the surface and at each full 6-inch (152-millimeter) depth (especially if a strength problem is suspected) in the base and subbase courses, on the surface of the subgrade, and on underlying layers in the subgrade as needed. Make density and moisture-content determinations in the subgrade at 1-foot (0.3 meter) intervals to a total depth of 4 feet (1 meter) below the surface of the subgrade. Use the results of the density and moisture tests at these depths to ascertain whether there is a need for additional CBR tests. Make density determinations between adjacent CBR tests. Perform three in-place CBR tests in test pits at each elevation tested. However, if the results of these three tests do not show reasonable agreement, make three additional tests. Reasonable agreement means a tolerance of 3 between three tests when the CBR is less than 10; a tolerance of 5 when the CBR is from 10 to 30; and a tolerance of 10 when the CBR is from 30 to 60. Variations in the individual readings are not as important for CBR values greater than 60. For example, actual test results of 6, 8, and 9 are reasonable and their average is 8; results of 23, 18, and 20 are reasonable and their average is 20. If the first three tests do not fall within this tolerance, then perform three additional tests at the same location and use the numerical average of the six tests as the CBR for that location. Round off CBR values below 20 to the nearest point. For example, round off 18.7 to 19. Round off to the nearest five points for CBR values above 20. For example, round off 23.4 to 25. Obtain a moisture content at the point of each penetration.

A-5.8.2 Using CBR Tests for Rigid Pavements.

In-place CBR tests may be required on the subgrade materials in addition to plate-bearing tests to evaluate a non-rigid overlay on rigid pavement. When the k value of the subgrade material is greater than 200 pci (5,536 g/cm3) or the concrete flexural strength is less than 400 psi (2.8 MPa), the load-carrying capability for the non-rigid overlay or rigid pavement should be evaluated using both rigid and flexible pavement evaluation procedures. In the latter case, assume the rigid pavement is a high-quality base course material and conduct in-place CBR tests on the base and subgrade materials in addition to the plate-bearing tests. Conduct the in-place CBR tests the same as if it was for a flexible pavement evaluation.

A-5.8.3 Moisture-Density-CBR Relations.

Develop the moisture-density-CBR relationships of the foundation materials as outlined in UFC 3-260-02 when required to evaluate a non-rigid overlay on rigid pavement.

A-5.9 Plate-Bearing Tests.

A-5.9.1 Estimating the Subgrade k Value.

Determine the modulus of subgrade reaction of the subgrade or base course using the plate-bearing test for rigid pavements as discussed in CRD-C 655. Conduct the plate-bearing test on the surface of the unbound material immediately beneath the pavement, that is, on the granular base course or on the subgrade when there is no base course. When the plate-bearing test cannot be conducted, determine an approximate k value by determining CBR values of each layer in the pavement structure using the DCP and use the procedure outlined in paragraph 7-3.4 to determine the effective k. When the pavement structure includes a high-quality stabilized base course as defined in UFC 3-260-02, conduct the plate-bearing test on the layer beneath the stabilized layer and test the stabilized layer to determine its modulus.

A-5.9.2 Plate-Bearing Tests on Rigid Overlays of Flexible Pavement.

When evaluating a rigid overlay of a flexible pavement, conduct the test in a pit with the concrete overlay removed. When the temperature of the existing asphalt pavement surface is above 75 °F (24 °C), remove the asphalt concrete pavement and run the plate bearing test on the base, then use the effective k procedure in paragraph 7-3.4 to determine the effective k at the top of the asphalt and use that value for the analysis. When the temperature of the existing asphalt pavement surface is below 75 °F (24 °C), run the tests on the asphalt concrete pavement and use that value for the analysis. Place load reaction far enough away from the plates so the stresses created by the load reaction will not influence the results of the plate-bearing tests. In general, place the load reactions on slabs adjacent to the slab being tested and not less than 12.5 feet (3.8 meters) from the bearing plate.

A-5.9.3 Plate Bearing Tests on Composite Pavements.

A composite pavement is composed of rigid overlay over a rigid base pavement with 4 inches (102 millimeters) or more of flexible or all-bituminous overlay. If the flexible overlay is less than 4 inches (102 millimeters), evaluate the rigid overlay using the unbonded overlay equation. When evaluating a composite pavement, perform the plate-bearing test on the surface of the granular base course beneath the rigid pavement layer or on the subgrade when there is no base course. Place load reaction far enough away from the plates so the stresses created by the load reaction will not influence the results of the plate-bearing tests. In general, place the load reactions on slabs adjacent to the slab being tested and not less than 12.5 feet (3.8 meters) from the bearing plate.

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APPENDIX B VOID DETECTION UNDER AIRFIELD PAVEMENTS

B-1 GENERAL.

This appendix outlines a reliable, cost-effective method to detect subsurface voids under airfield pavements to minimize the risk of premature airfield pavement failure. The Services determine whether to perform airfield void detection surveys as part of regularly scheduled evaluation or as required based on the airfield location and history of issues with voids. The term void encompasses actual voids, voids filled with water, or pockets of very loose subgrade with low bearing capacity.

B-2 BACKGROUND.

Past aircraft accidents and airfield pavement failures due to subsurface voids increase the risk of future accidents and threats to life safety, especially given aging DoD facilities and scarce M&R resources. A pavement failure under the front gear of a trainer aircraft at NAS Pensacola in 1999 prompted a review of available pavement evaluation technology and procedures to detect subsurface weakness. The resulting approach outlined below uses a combination of visual, nondestructive, and destructive testing and targets pavements above drainpipe crossings, but the same method is applied for any airfield pavement where there is the potential for a void.

B-3 VOID DETECTION.

B-3.1 Visual Inspection.

Perform visual inspection of the airfield pavements with sufficient frequency to locate potential problem areas and satisfy the airfield manager of their operational safety. Base testing frequency on local physical conditions and operational tempo. Monitor pavements for conditions that may affect aircraft movement, with a focus on depressions and cracking that are indicative of subsurface deterioration. Depressions are evident in flexible pavements after a rainfall or by the concentric marks left by the evaporated water. Concrete slabs cracked into two or more pieces or slabs that exhibit faulting at joints may indicate underlying voids or loss of support. Carefully inspect areas above drainpipe crossings since most problems appear above or near drainage structures. Inspect unpaved areas adjacent to the pavement above drainage structures. Problems observed in these areas are early warning signs of problems in nearby paved areas. Depressed pavement or shattered slabs surrounding drainage structures (catch basins) indicate infiltration of soil materials into the structure or pipe. Use UFC 3-260-16 (ASTM D5340 and ASTM D6433) for visual inspections.

B-3.2 Heavy-Weight Deflectometer (HWD) Testing.

After performing a visual inspection, evaluate areas of concern using an HWD. Test all drainage structure crossings and any other areas that have visual indications of voids or loss of subgrade support.

B-3.2.1 Data Collection.

- The following outlines the data collection procedure for drainage structures under asphalt pavements. The procedure for concrete pavements is the same, with adjustments for performing HWD tests at the center of each slab.
  - Identify the location of each pipe and mark it on the pavement.
  - Test at 10-foot (3-meter) intervals, offset 10 feet (3 meters) to the left of the drainage structure. This is “Line A” for reporting purposes.
  - Test at 10-foot (3-meter) intervals above the drainage structure in the same direction as Line A. This is “Line B” for reporting purposes.
  - Test at 10-foot (3-meter) intervals, offset 10 feet (3 meters) to the right of the drainage structure in the same direction as Line A. This is “Line C” for reporting purposes.
- This procedure typically produces three sets of readings at 10-foot (3-meter) intervals along the drainage structure except for the case where the pipe falls just in between two rows of concrete slabs, then only two sets of readings are needed. The procedure uses 10-foot (3-meter) intervals based on the assumption the HWD cannot “sense” loss of pavement support beyond a 5-foot (1.5-meter) radius.
- A single drop at each location is typically sufficient to compare successive drops at adjacent locations. Configure the HWD with seven geophones numbered D1 though D7, where D1 is the deflection under the load point and D2 through D7 are typically at 12 inches (305 millimeters) (15 inches [381 millimeters] for some configurations), 24, 36, 48, 60, and 72 inches (381, 610, 914, 1219, 1524, and 1829 millimeters) from D1, respectively. This results in seven deflection measurements at each test location.
- Use the deflection data to determine the impulse stiffness modulus (ISM), which is a measure of the relative pavement strength at each test location. Calculate ISM1 by dividing the load by the deflection at D1. Determine ISM2 through ISM7 using the same procedure: ISM(X) = load/deflection at D(X).
- D1 primarily indicates the state of the pavement itself, whereas D7 primarily indicates the state of the subgrade. Therefore, using D1 alone is not sufficient to successfully detect voids under the pavement.

B-3.2.2 Data Analysis.

Analyze data during data acquisition and mark weak areas immediately for penetrometer testing. Plot ISM1 through ISM7 results for each test along the drainage structure. Normalize the data by dividing each plot by the highest value in the plot to determine relative effects of pavement weaknesses on each sensor. Once the ISM plots are completed, use the following rules to determine potentially weak areas.

- An absolute ISM1 value below about 300 kips/inch is of concern for asphalt pavements.
- An absolute ISM1 value below 1000 kips/inch is of concern for concrete pavements.
- A relative ISM decay indicates an unexpected weakness.
  - A relative weakness in ISM1 indicates it is shallow.
  - A relative weakness in ISM7 indicates it is deep (3 to 20 feet [1 to 6 meters]).
  - A relative weakness in both ISM1 and ISM7 indicates a general lack of support.

B-3.2.3 Load-Carrying Capacity.

Use the HWD data to determine the effect of any subgrade weakness (or void) on the load-carrying capacity of the pavement using the layered elastic evaluation procedure in paragraph 5-3.

B-3.2.4 Frequency.

When an airfield has a history of problem with voids, perform void detection procedures at all drainage structures as outlined in paragraphs B-3.1 and B-3.2, in conjunction with regularly scheduled structural evaluations.

B-3.2.5 Large Area Testing.

When large areas may be subject to voids, such as where karst formations are prevalent, adjust the procedure on asphalt pavement by testing at 10- to 20-foot (3- to 6-meter) intervals along a linear structure (e.g., a runway) at 10-foot (3 meters) offsets on both sides of the centerline. Perform testing at each slab center (e.g., 15-foot (4.6 meters) spacing for Navy airfields) on PCC pavements. Test composite pavements as asphalt pavements when overlaid joints are not visible. Perform testing with the concrete procedure when the overlaid concrete joints are visible. Test outer portions of the linear structures with GPR if deemed necessary. GPR testing is faster but less reliable. Perform HWD testing on any anomalies found with GPR.

B-3.3 Penetrometer Testing.

Test weak areas revealed by the HWD (or the GPR and the HWD) with DCP or SPT as outlined in Chapter 3 and Appendix A to determine the depth of the weakness and identify the type of repair needed. Ensure there are no buried utilities present prior to testing.

Use PCASE or a spreadsheet to plot CBR or k (modulus of subgrade reaction) versus depth. A low CBR value (less than 3) or a low k (less than 75) indicates a weak layer or an actual void. When coring concrete pavement, the core may drop, indicating a void between the concrete pavement and the underlying base. When drilling the pavement, this separation is more difficult to observe, so a bore scope may be used to assess the existing void.

B-3.4 Capture Drainage Structure Video.

When testing and/or visible failure is evident near or around drainage structures, capture video of the interior of these drainage structures to help pinpoint the location of potential problem areas and define the need for M&R. Give special attention to assessing pipe joints because accumulations of fines near joints or other penetrations are a good indicator of a loss of subgrade material and concurrent subgrade strength loss.

B-3.5 Alternative Nondestructive Testing (NDT).

As described in paragraph 3-4.2, GPR is an alternate nondestructive void detection technique. Acoustic reflection sounding is another technique. Based on testing, neither is as effective for detecting voids in all circumstances as the procedures outlined in paragraph B-3.2. However, they can provide useful complementary information.

B-3.5.1 Ground Penetrating Radar (GPR).

GPR provides a cost-effective and nondestructive means of examining subsurface conditions. On the airfield, GPR can be used to detect utilities, pavement reinforcement, and anomalies in base and subgrade material, such as voids. There are distinct advantages and disadvantages to using GPR in lieu of the HWD. GPR techniques will never provide an estimate of the pavement strength. However, GPR methodologies are very fast and accurate if used in areas suitable for GPR technology. It is essential to make sure GPR is suitable for the site. The United States Department of Agriculture's Natural Resources Conservation Service Ground-Penetrating Radar Soil Suitability Maps are an essential resource in early planning to determine if GPR is the correct approach to take in a pavement void detection analysis. The GPR works by sending a tiny pulse of energy into a material and recording the strength and the time required for the return of any reflected signal. A series of pulses make up what is called a scan. Reflections are produced whenever the energy pulse enters a material with different electrical conduction properties or dielectric permittivity. The strength, or amplitude, of the reflection is determined by the contrast in dielectric constants and conductivities of the two materials. The GPR displays the changes in dielectric constant on the screen, indicating changes in material type. The display allows interpretation of data in the field. Subsurface features can be identified with the GPR by how fast the energy can travel though the material type. Air has a dielectric constant of 1 and energy travels very quickly through materials with a dielectric constant of 1. Water has a dielectric constant of 81 and energy travels slowly through water. Metal has a dialectic constant of infinity and acts as a reflector. The dielectric constant of subsurface soils typically ranges from

4 to 32, depending on the soil type and saturation. With the knowledge of pavement, base, and material types, unanticipated anomalies and changes in scan response can be identified as weak base or subgrade or air- and water-filled voids.

The following procedures are used for effective void detection using GPR.

- The GPR will first be calibrated to accurately measure stations along the storm drainage lines. This is done at each pavement type to account for the roughness in the pavement surface.
- At each differing pavement and geological condition, the dielectric constant of the cross-section of pavement, base, and subgrade must be determined for accurate depth calculations and appropriate contrast while performing the scan onsite. This can be done by either identifying the soil type or ground truthing to known underground structure depths. The dielectric constants and the GPR data are recorded at each run.
- Once the storm drainage structures have been identified and the approximate location of the storm drainage line to be tested is known, the GPR unit will make several perpendicular scans to determine the exact location of the storm pipe. The pipe will then be marked on the pavement surface for future data collection. Sometimes, as with concrete or PVC pipes, it is difficult to locate the pipe due to the dielectric constant being similar to the in situ soil. In these instances, the field team uses maps and locations of drainage structures to determine the most likely place for the pipe to be located and the tests are run along those alignment locations.
- Once the location of the pipe is properly identified, collection of data is taken parallel to the storm pipe approximately 12 inches (305 millimeters) left and right of the pipe edge. Another pass is made directly over the pipe. Data is collected in the same direction each time so the stations of each run are comparable.
- Based on the observed pavement conditions and the observations made with the GPR scans, suspicious void locations may be marked on the pavement for further testing to verify the presence of a void or soft subgrade.
- The identified areas could be voids or could be utilities. Before proceeding with soil penetration techniques, it is best to locate the utilities and ensure the suspect areas are not utilities. If the suspect areas are not utilities, then they can be investigated using one of the soil penetration techniques such as the DCP.

B-3.5.2 Acoustic Reflection Sounding

Acoustic reflection sounding (ASTM D4580, *Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding*) has been used to detect concrete bridge deck delamination. In some instances, a person walking alongside the HWD can hear a difference in the sound of the pavement when it is spanning a shallow void, especially with thin concrete pavements. This provides another tool in detecting potential voids just under the slab. Once coring or drilling is complete, a borescope is useful to assess the existence of voids.

B-4 VOID REPAIR AND PREVENTION.

- Prior to proceeding with repair, determine if an actual void (or very loose area) is present or if a deep layer of weak material is responsible for the readings, using FWD/HWD testing.
- Void repair methods include pressure grouting, polymer injection, and removal and replacement. If an actual void is present, lightweight polymer or grout injection is generally preferred to removal and replacement because of the minimal impact on aircraft operations.
  - Pressure grouting and polymer injection may successfully fill a void (or compact a locally loose area) but may only have very limited success for a deep layer of weak material.
  - When a void or weak layer is deep, injection may simply create polymer (or grout) lenses (i.e., thin layers) that will lift the pavement but not provide additional support.
  - When pavement surface integrity is sound and load-carrying capacity is adequate, pressure grouting or polymer injection can be used to lift pavement and re-establish ride quality.
  - Once set, grout provides a stiff material typically usable for any type of subgrade and can also be used to fill gaps just under the slab.
  - If no void is present and a weak subgrade is undermining the load carrying capacity, remove and replace the weak layer. If the weak layer is under the water table, removal and replacement can become very difficult.
- Lightweight polymer injection has some advantages over grout injection.
  - Polymer injection adds less weight when dealing with soft subgrades and large voids.
  - A properly mixed polymer typically reaches most of its strength in a few minutes.
  - Quick-setting polymer can seal large cracks in drainpipes or fill deep sinkholes, while grout could flow down the sinkhole and proceed into the pipes.
- If polymer injection is used, the modulus of elasticity of the polymer needs to exceed the stiffness of the layer where it is injected; therefore, it should typically only be injected into the subgrade. Even then, tests on some limited data indicate that this requires a minimum density of:

  - 6 pcf (96 kg/m^3) for subgrades with elastic modulus of 6,000 psi (41 MPa)
  - 10 pcf (160 kg/m^3) for subgrades with elastic modulus of 15,000 psi (103 MPa)
  - 15 pcf (240 kg/m^3) for subgrades with elastic modulus of 25,000 psi (172 MPa)
- If lightweight polymer or grout injection is not available, then use pavement and base removal and replacement down to the prescribed depth. When pipe deterioration is extensive, consider internal pipe repair, jacketing, or pipe replacement.

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APPENDIX C AIRCRAFT GEAR CONFIGURATION NOMENCLATURE

C-1 PURPOSE.

This appendix outlines the standard convention for naming and characterizing aircraft landing gear configurations. It is primarily intended for fixed-wing aircraft but is applicable to any aircraft using wheels for landing. This appendix is used in conjunction with FAA Order 5300.7, *Standard Naming Convention for Aircraft Landing Gear Configurations*.

C-2 BACKGROUND.

Landing gear configuration and aircraft gross weight are an integral part of airfield pavement design and evaluating pavement strength. Historically, most aircraft used relatively simple gear geometries such as a single wheel per strut or two wheels side by side on a landing strut. As aircraft became larger and heavier, they required additional wheels in groups or placed side-by-side and in tandem configurations to prevent excessively high individual wheel loads that impart high stresses on the pavement structure.

C-2.1 Typical Gear Configurations.

Originally, most civilian and military aircraft used three basic gear configurations: the “single wheel” (one wheel per strut), the “dual wheel” (two wheels side by side on a strut), and the “dual tandem” (two wheels side by side followed by two additional side-by-side wheels). As aircraft gross weight increased, manufacturers added additional landing struts to the aircraft. For example, Boeing used four landing struts with dual tandem configurations on the B-747 to reduce its impact on airfield pavement.

C-2.2 Complex Gear Configurations.

Other aircraft used gear configurations with multiple wheels in arrangements that could not be described by the three simple gear configurations. There was no coordinated effort between the FAA and the Services to provide a uniform naming convention resulting in naming systems that were not easily cross-referenced.

C-3 DEFINITIONS.

C-3.1 Main Gear.

“Main gear” means the primary landing gear that is symmetrical on either side of an aircraft. When multiple landing gears are present and are not in line with each other, the outermost gear pair is considered the main gear. Multiples of the main gear exist when a gear is in line with other gears along the longitudinal axis of the aircraft.

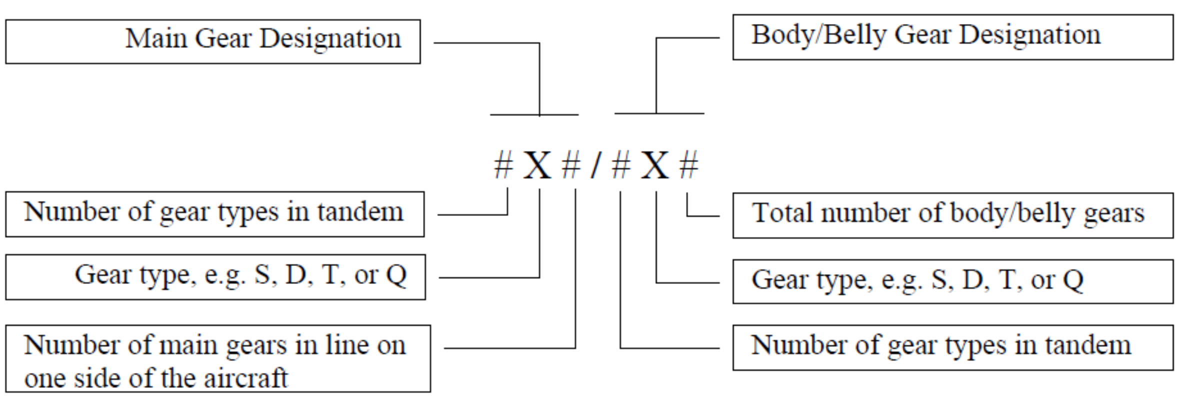
C-3.2 Body/Belly Gear.

“Body/belly gear” refers to an additional landing gear or gears in the center portion of the aircraft between the main gears. Body/belly gears may be different than the main gear and may be asymmetric.

C-4 INTENDED USE.

DoD is adopting the naming convention used by the FAA, as shown in Figure C-1.

Figure C-1 Aircraft Gear Naming Convention



C-5 AIRCRAFT GEAR GEOMETRY NAMING CONVENTION.

C-5.1 Basic Name for Aircraft Gear Geometry.

As shown in Figure C-1, abbreviated aircraft gear designations may include up to three variables. The two primary variables are the main gear configuration and the body/belly gear configuration if body/belly gears are present. An optional tire pressure code can also be used.

C-5.2 Basic Gear Type.

Gear type for an individual landing strut is determined by the number of wheels across a given axle (or axle line) and whether wheels are repeated in tandem. There are instances when multiple struts are in close proximity and are best treated as a single gear, e.g., the Antonov AN-124 (see Figure C-14). If body/belly gears are not present, the second portion of the name is omitted. For aircraft with multiple gears, such as the B-747 and the A380, the outer gear pair is treated as the main gear.

C-5.3 Basic Gear Codes.

The naming convention in Figure C-1 uses the gear designation codes in Table C-1.

Table C-1 Gear Designations

S	Single
D	Dual
T	Triple
Q	Quadruple

C-5.4 Use of Historical Tandem Designation.

Note that while the verbal description continues to use the term “tandem” to describe tandem gear configurations, the tandem designation “T” no longer appears in the gear name. “T” now indicates triple wheels.

C-5.5 Main Gear Portion of Gear Designation.

As shown in Figure C-1, the first portion of the aircraft gear name is the main gear designation that may consist of up to three characters. The first character indicates the number of tandem sets or wheels in tandem and the second character indicates the gear code (S, D, T, or Q). If a tandem configuration is not present, the leading value of “1” is omitted. Typical names are S = single, 2D = two dual wheels in tandem, 3D = three dual gears in tandem, 5D = five dual wheels in tandem, and 2T = two triple wheels in tandem.

The main gear designation indicates the number of gears on one side of the aircraft but assumes the gear is present on both sides (symmetrical) of the aircraft. The third character of the gear designation is a numeric value that indicates multiples of gears. An aircraft with one gear on each side of the aircraft has a value of 1 but, for simplicity, it is omitted from the main gear designation. Aircraft with more than one main gear on each side of the aircraft and where the gears are in line will use a value indicating the number of gears in line. For example, as shown in Figure C-20, the Ilyushin IL-76 has two gears containing quadruple wheels on each side of the aircraft and has the designation Q2.

C-5.6 Body/Belly Gear Portion of Gear Designation.

The second portion of the aircraft gear name is used when body/belly gears are present. If body/belly gears are present, the main gear designation is followed by a forward slash (/), then the body/belly gear designation. For example, the B-747 aircraft has two dual wheels in tandem main gear and two dual wheels in tandem body/belly gears. The full gear designation for this aircraft is 2D/2D2. The body/belly gear designation is similar to the main gear designation except that the trailing numeric value denotes the total number of body/belly gears present, e.g., 2D1 = one dual tandem body/belly gear; 2D2 = two dual tandem body/belly gears. Because body/belly gear arrangement may not be symmetrical, the gear code must identify the total number of gears present; a value of 1 is not omitted if only one gear exists.

C-5.7 Extension of Naming Convention.

Future aircraft might require additional body/belly gears that are nonsymmetrical and/or non-uniform. In these instances, the body/belly gear designation will contain a hyphen to indicate the non-uniform gear geometry. For demonstration purposes, consider adding one dual wheel body/belly gear to the existing 2D/2D2 gear configuration. The resulting gear name would be 2D/2D2-D.

C-5.8 Unique Gear Configurations.

The Lockheed C-5 Galaxy has a unique gear type and is difficult to name using this method. This aircraft will continue to be referred to directly as the C5. Gear configurations such as those on the Boeing C-17, Antonov AN-124, and Ilyushin IL-76 might also cause some confusion. In these cases, it is important to observe the number of landing struts and the proximity of the struts. In the case of the AN-124, it is more advantageous to address the multiple landing struts as one gear, i.e., 5D or five duals in tandem, rather than use D5 or dual wheel gears with five sets per side of the aircraft. Due to wheel proximity, the C-17 gear is more appropriately called a 2T as it appears to have triple wheels in tandem. In contrast, the IL-76 has considerable spacing between the struts and has a Q2 designation.

C-5.9 Gear Geometry Naming Convention Examples.

Table C-1 and paragraphs C-5.3 to C-5.8 provide examples of generic gear types in individual and multiple tandem configurations. Figures C-2 through C-20 provide examples of known gear configurations.

C-5.10 Tire Pressure Information.

The gear naming convention includes a third variable to report tire pressure using ICAO codes. While tire pressure effects on airfield pavements are secondary to aircraft load and wheel spacing, they can have a significant impact on the ability of the pavement to accommodate a specific aircraft.

C-5.10.1 ICAO codes associated with the ACN and the PCN system categorize aircraft tire pressures into four groups for reporting purposes. Table C-2 lists tire pressure codes by category.

Table C-2 Standard Tire Pressure Categories

Category	Range		Code Designation
	psi	MPa	
Unlimited	No limit	No limit	W
High	182–254	1.26–1.75	X
Medium	74–181	0.51–1.25	Y
Low	0–73	0.0–0.5	Z

C-5.10.2 Include the ICAO tire pressure in parentheses after the standard gear name. Table C-3 shows sample gear names with and without the additional tire pressure code.

Table C-3 Sample Gear Names with and without Tire Pressure Codes

Gear Name without Tire Pressure	Gear Name with Tire Pressure
S	S(W)
2S	2S(X)
2D/2D1	2D/2D1(Z)
Q2	Q2(Y)
2D/3D2	2D/3D2(Z)

C-5.11 Historical Naming Convention Comparison.

Table C-4 provides a comparison of the naming convention outlined in this UFC and past FAA, Air Force, and Navy methods. Note that while the old Air Force methodology addresses nose gear configuration, the new method does not due to the minimal impact of the nose gear on the pavement load.

Table C-4 Naming Convention with Historical FAA, U.S. Air Force, and U.S. Navy Nomenclatures

		Historic FAA Designations					U.S. Air Force Designations				U.S. Navy Designations			
Proposed Nomenclature	Reference Figure	FAA Name	Main Gear	Belly Gear	# Belly Gear	Total # Wheels, Excluding Nose	Air Force Designation	Air Force Types	Air Force Name	Nose Gear	Navy Name	Navy Designation	DOD Flight Information	Typical Aircraft
S	3	Single Wheel	SW			2	S	A	Single, Tricycle	Single Wheel	Single Tricycle	ST	S	F-14, F-15
S	4	Single Wheel	SW			2	S	B	Single, Tricycle	Dual wheel				
D	5	Dual wheel	DW			4	T	C	Twin, Tricycle	Single Wheel				Beech 1900
D	6	Dual wheel	DW			4	T	D	Twin, Tricycle	Dual wheel	Dual Tricycle	DT	T	B-737, P3 (C-9)
2S	7	Single Tandem				4	S-TA	E	Single, Tandem Tricycle	Dual wheel	Single Tandem Tricycle	STT	ST	C-130
2T	8					12	TR-TA	L	Twin-Tandem, Tricycle	Dual wheel	Triple Tandem	TRT	TRT	C-17
2D	9	Dual Tandem	DT			8	T-TA	F	Twin-Tandem, Tricycle	Dual wheel	Dual Tandem Tricycle	DTT	TT	B-757, KC-135, C-141
2D/D1	10	Dual tandem	DT	DW	1	10	T-TA	H	Twin-Tandem, Tricycle	Dual wheel	Single Belly Twin Tandem	SBTT	SBTT	L1011, DC-10
2D/2D1	11	Dual Tandem	DT	DT	1	12				Dual wheel				A340-600
2D/2D2	12	Double Dual Tandem	DT	DT	2	16	T-TA	J	Twin-Tandem, Tricycle	Dual wheel	Double Dual Tandem	DDT	DDT	B-747, (E-4)
3D	13	Triple dual Tandem	TDT			12				Dual wheel				B-777
5D	14					20				4 across				An-124
7D	15					28				4 across				An-225
2D/3D2	16		DT	TDT	2	20				Dual wheel				A380
C5	17					24	T-D-TA	K	Twin-Delta-Tandem, Tricycle	4 across	Twin Delta Tandem	TDT	TDT	C-5
D2	18					8	T-T	G	Twin-Twin, Bicycle	No Nose Gear - single outrigger	Twin Twin Tricycle	TT	TT	B-52
Q	19					8								HS-121 Trident
Q2	20					16								IL-76

Figure C-2 Generic Gear Configurations (Increase Numeric Value for Additional Tandem Axles)

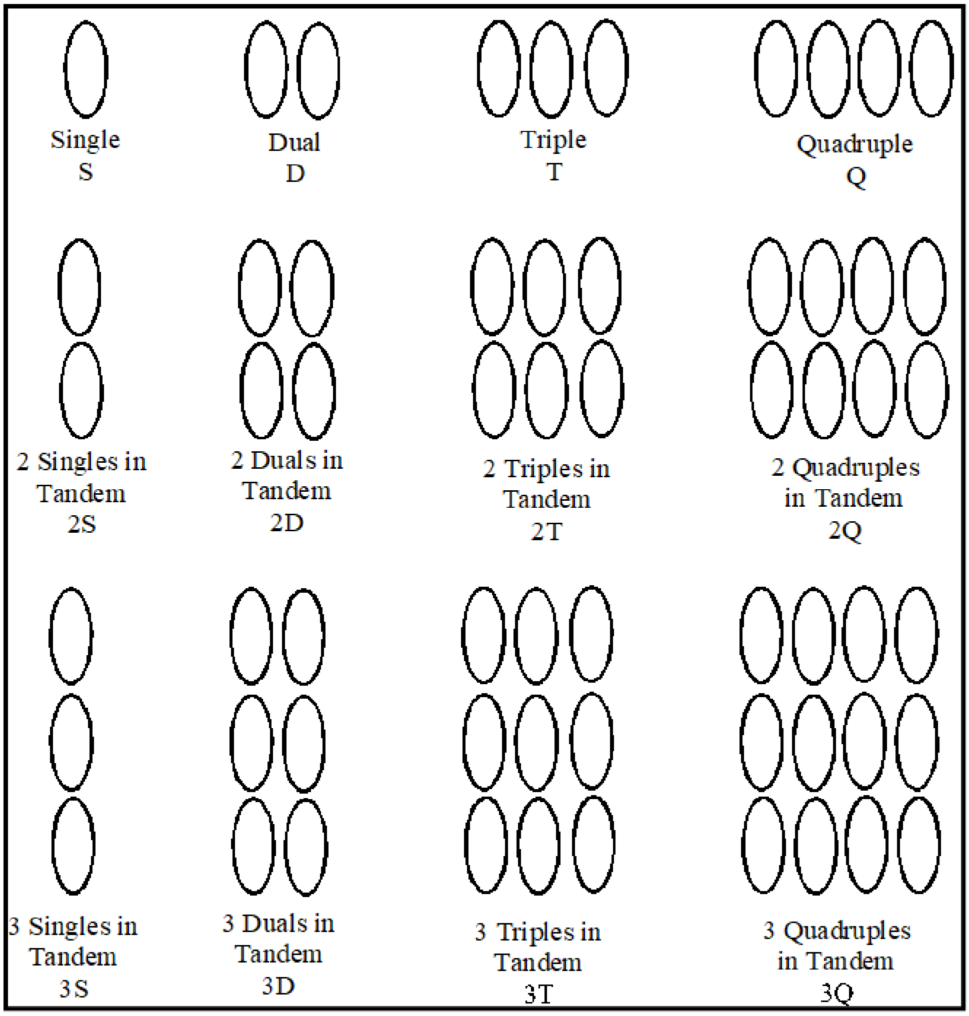


Figure C-3 S - Single Wheel Main Gear with Single Wheel Nose Gear

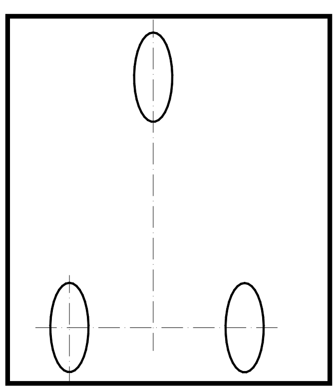


Figure C-4 S - Single Wheel Main Gear with Dual Wheel Nose Gear

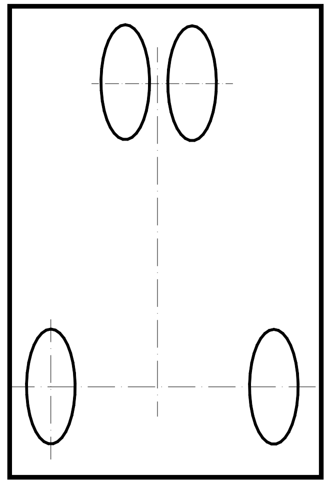


Figure C-5 D - Dual Wheel Main Gear with Single Wheel Nose Gear

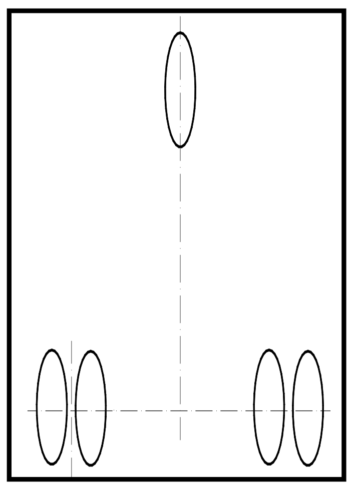


Figure C-6 D - Dual Wheel Main Gear with Dual Wheel Nose Gear

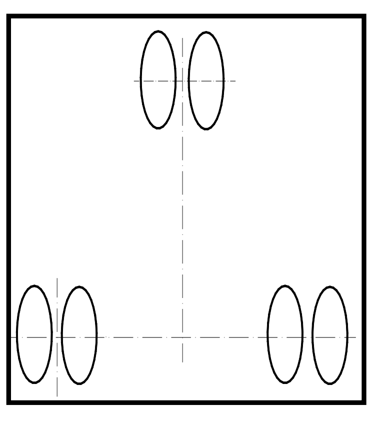


Figure C-7 2S - Two Single Wheels in Tandem Main Gear with Dual Wheel Nose Gear, Lockheed C-130

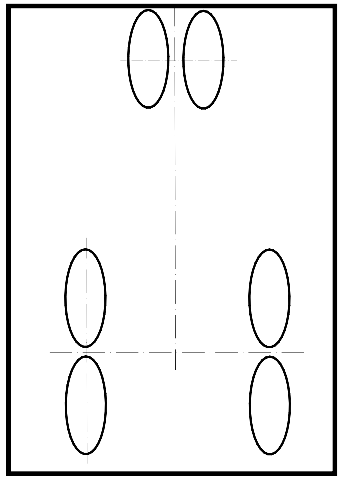


Figure C-8 2T - Two Triple wheels in Tandem Main Gear with Dual Wheel Nose Gear, Boeing C-17

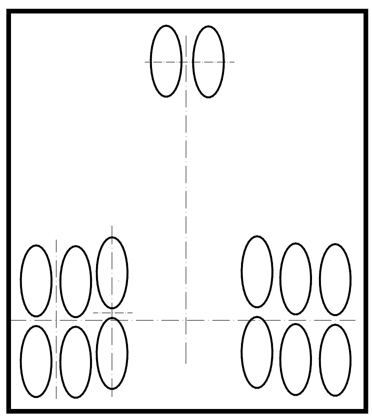


Figure C-9 2D - Two Dual Wheels in Tandem Main Gear with Dual Wheel Nose Gear

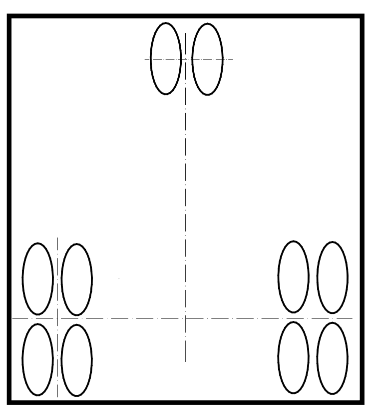


Figure C-10 2D/D1 - Two Dual Wheels in Tandem Main Gear/Dual Wheel Body Gear with Dual Wheel Nose Gear, McDonnell Douglas DC-10, Lockheed L-1011

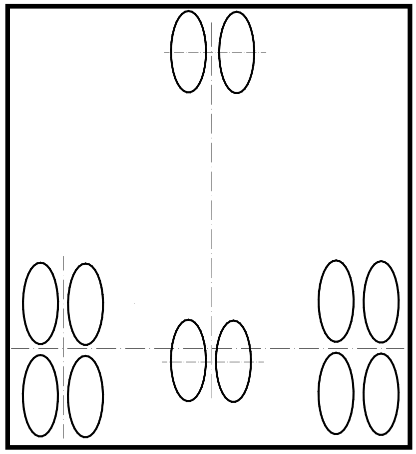


Figure C-11 2D/2D1 Two Dual Wheels in Tandem Main Gear/Two Dual Wheels in Tandem Body Gear with Dual Wheel Nose Gear, Airbus A340-600

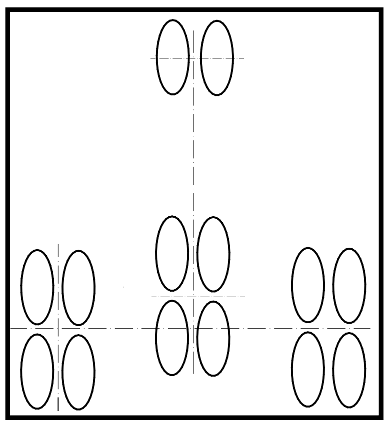


Figure C-12 2D/2D2 - Two Dual Wheels in Tandem Main Gear/Two Dual Wheels in Tandem Body Gear with Dual Wheel Nose Gear, Boeing B-747

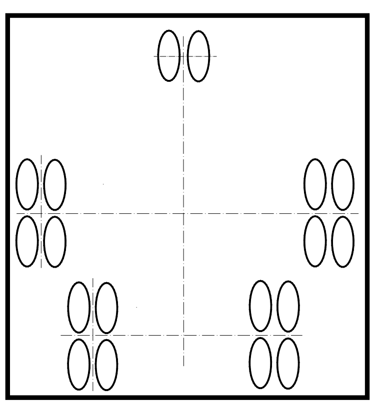


Figure C-13 3D - Three Dual Wheels in Tandem Main Gear with Dual Wheel Nose Gear, Boeing B-777

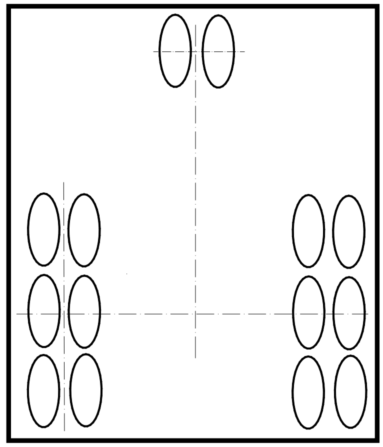


Figure C-14 5D - Five Dual Wheels in Tandem Main Gear with Quadruple Wheel Nose Gear, Antonov AN-124

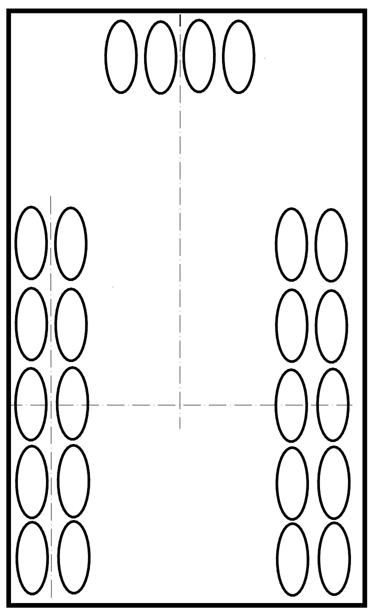


Figure C-15 7D - Seven Dual Wheels in Tandem Main Gear with Quadruple Nose Gear, AN-225

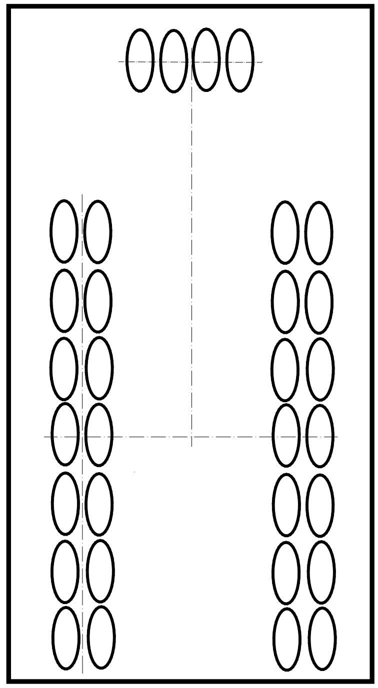


Figure C-16 2D/3D2 - Two Dual Wheels in Tandem Main Gear/Three Dual Wheels in Tandem Body Gear with Dual Wheel Nose Gear, Airbus A380

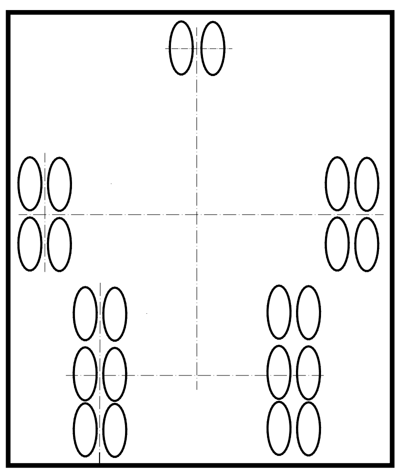


Figure C-17 C5 - Complex Gear Comprised of Dual Wheel and Quadruple Wheel Combination with Quadruple Wheel Nose Gear, Lockheed C5 Galaxy

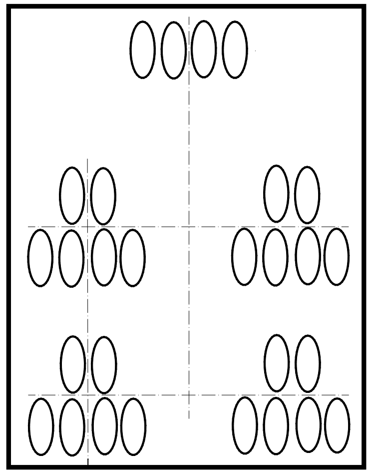


Figure C-18 D2 - Dual Wheel Gear Two Struts per Side Main Gear with No Separate Nose Gear (note that single wheel outriggers are ignored), Boeing B-52 Bomber

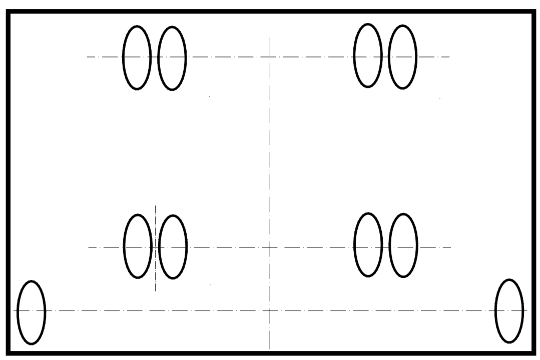


Figure C-19 Q - Quadruple Wheel Main Gear with Dual Wheel Nose Gear, Hawker Siddeley HS-121 Trident

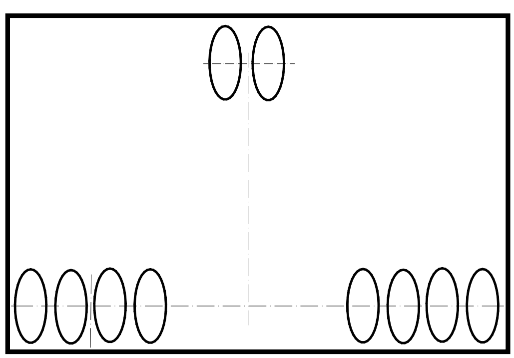
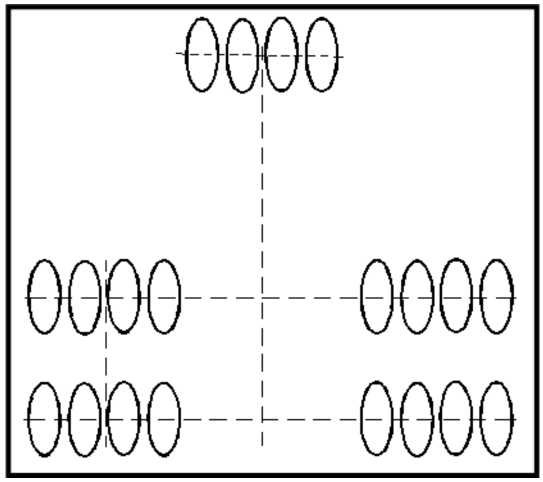


Figure C-20 Q2 - Quadruple Wheels Two Struts per Side with Quadruple Nose Gear, Ilyushin IL-76



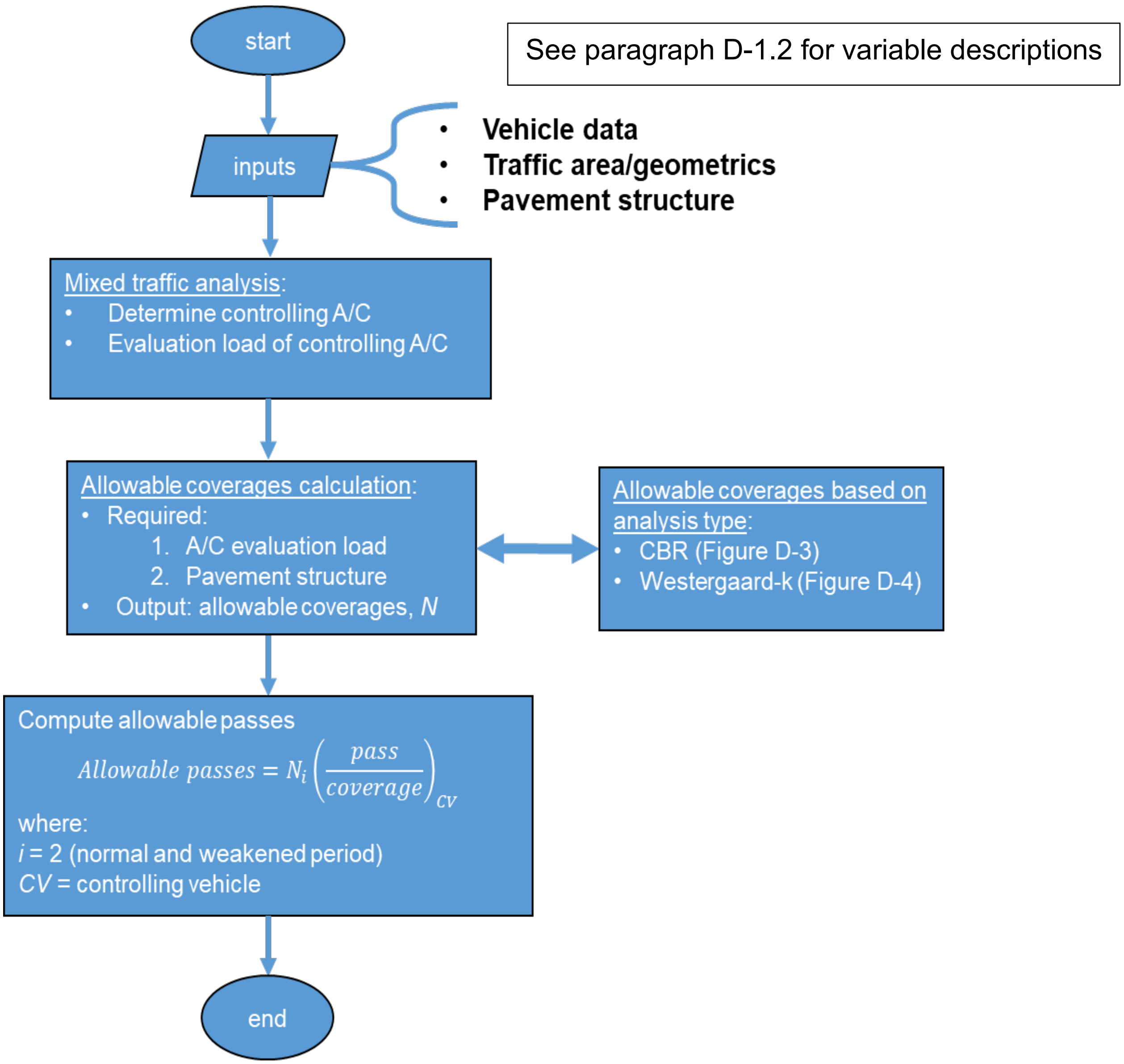
APPENDIX D STRUCTURAL ANALYSIS PROCESS

D-1 CBR-K PAVEMENT ANALYSIS PROCESS FLOW.

D-1.1 Overall CBR-k Analysis Process Flow.

The general process flows for each of the analysis models, Alpha-Beta Hybrid (CBR), and Westergaard (k) are the same. The objective of the first process flow shown in Figure D-1 is to determine the allowable passes. The second, shown in Figure D-2, is to determine the allowable load. Each of these process flows call the CBR procedure for flexible pavement analysis in Figure D-3 and the Westergaard procedure for rigid pavement analysis shown in Figure D-4.

Figure D-1 CBR-k Procedure for Allowable Passes Computation



D-1.2 Analysis Variables for Figure D-1.

- A/C = aircraft
- AGL = allowable gross load of controlling aircraft
- AGL_{trial} = the trial AGL assumed by PCASE during a given iteration
- CBR = California Bearing Ratio (evaluation methodology for flexible pavements)
- CDF = cumulative damage factor, a ratio of applied coverages to allowable coverages of the controlling aircraft
- CV = controlling vehicle (aircraft) determined from mixed traffic analysis
- k = Modulus of subgrade reaction (evaluation methodology for rigid pavements)
- N = Allowable coverages of controlling aircraft at trial AGL
- n = applied coverages of controlling aircraft based on equivalent evaluation passes
- N_i = the number of allowable coverages determined for the controlling aircraft, where the subscript i = 2 for a normal period and a weakened (typically due to thawing) period
- pass/coverage = the ratio of passes to coverage for the controlling aircraft for a specified traffic area

Figure D-2 CBR-k Procedure for Allowable Gross Load Computation

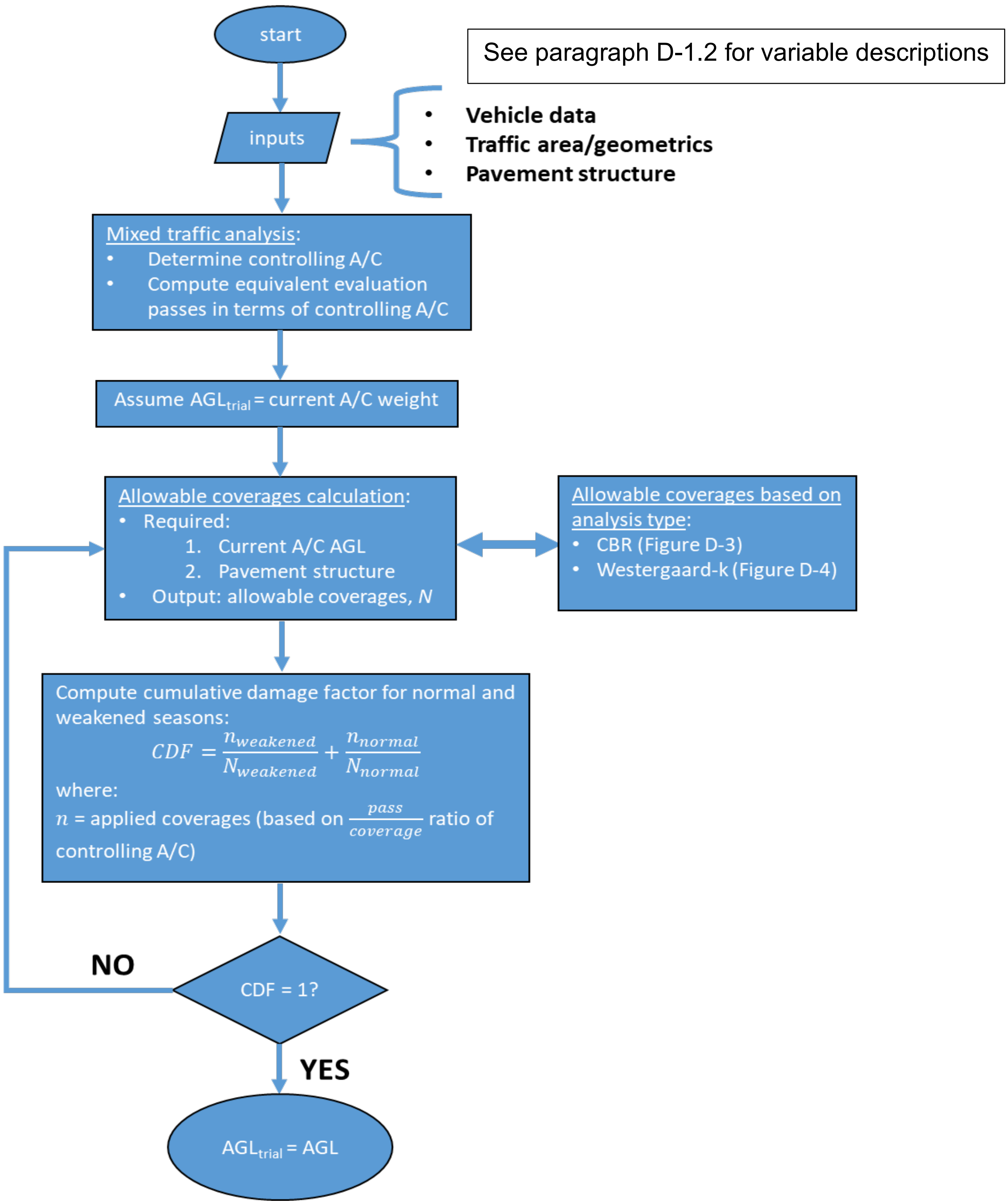
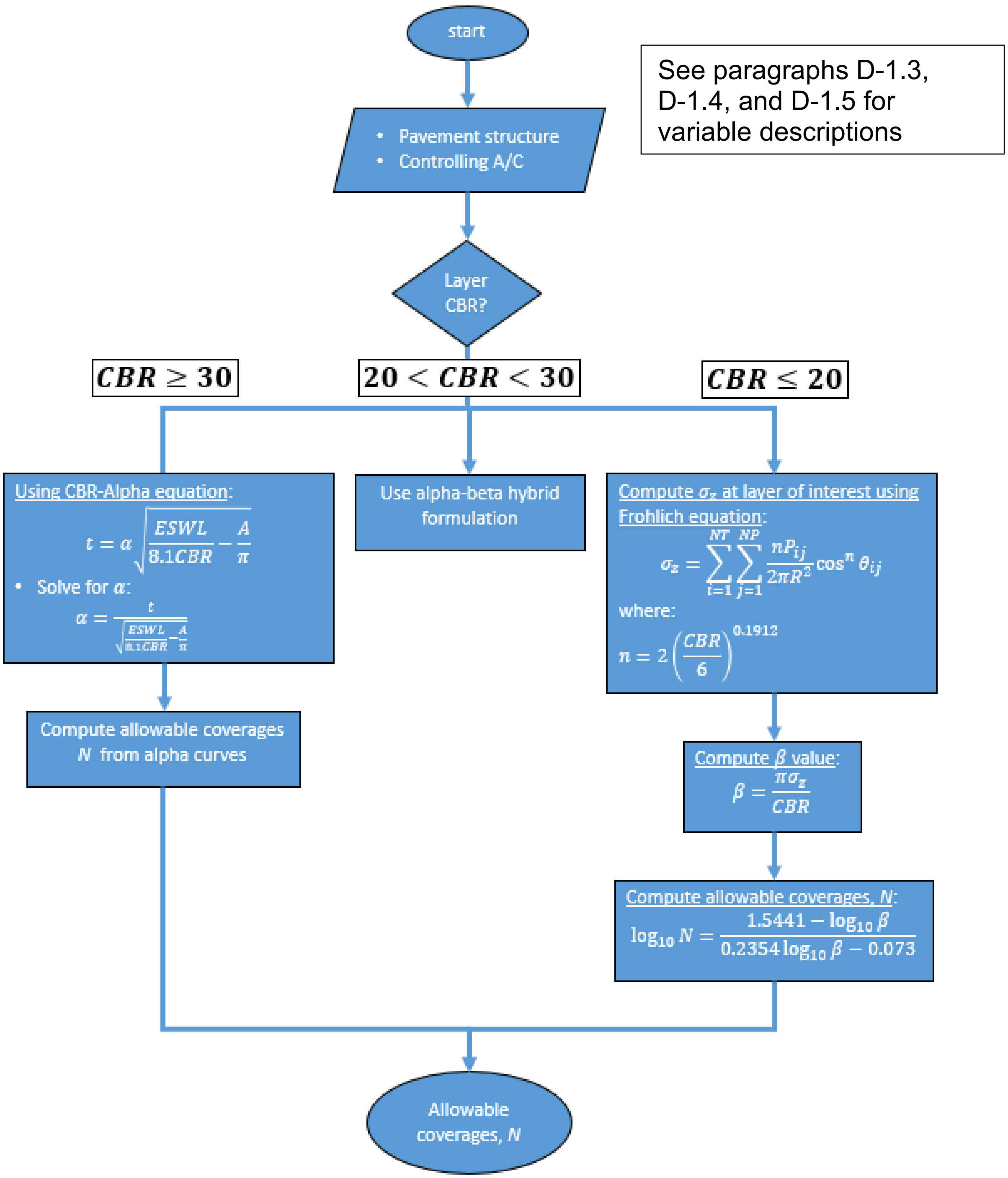


Figure D-3 CBR Procedure for Allowable Coverage Computation



D-1.3 CBR-Alpha Equation Variables in Figure D-3.

- t = total thickness of pavement structure above layer of interest
- α = thickness reduction factor, from alpha curves (function of number of tires in controlling vehicle)
- ESWL = equivalent single wheel load of controlling vehicle
- CBR = California Bearing Ratio of layer of interest
- A = contact area of controlling vehicle

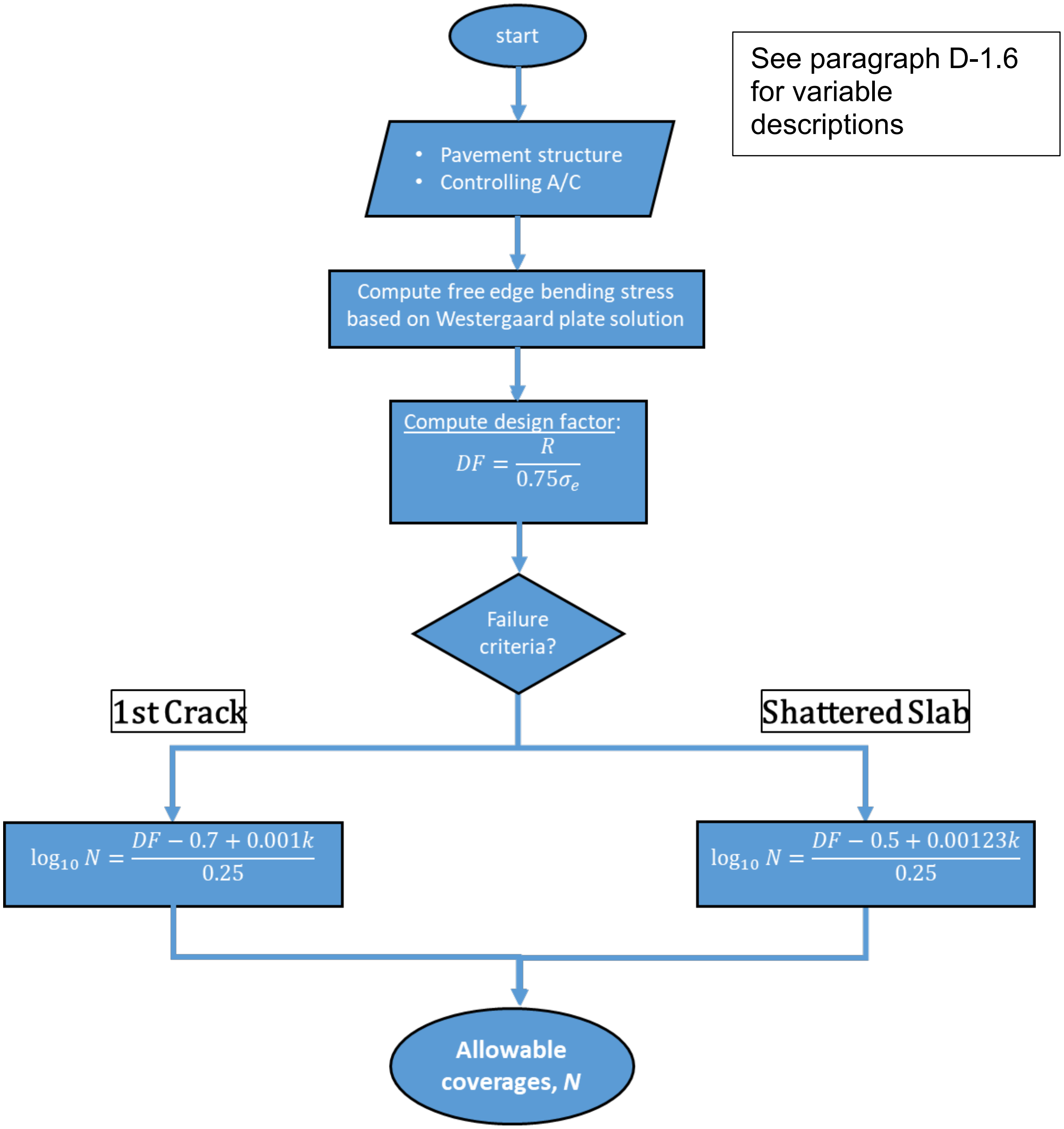
D-1.4 CBR-Beta Frohlich Equation Variables in Figure D-3.

- σ_z = vertical stress computed at the top of the layer of interest
- NT = number of tires in the controlling vehicle main landing gear
- NP = as the Frohlich equation is a *point load solution* (as opposed to a uniform pressure over a circular area solution), the inner summation is conducted over a given number of point loads NT used to estimate the tire inflation pressure in the shape of an elliptical contact area
- i,j subscripts = "i" is the current tire number in the outer summation and "j" is the current point load number in the inner summation
- n = Frohlich stress concentration factor, an empirical value which modifies the vertical stress distribution with depth
- P_{ij} = j^{th} point load in the i^{th} main landing gear tire of the controlling vehicle
- θ_{ij} = horizontal angle between the (x,y) coordinate of point load P_{ij} and the (x,y) coordinate of the calculation point at the top of the layer of interest
- R_{ij} = straight line distance from the z coordinate of the surface point load P_{ij} and the z coordinate of the calculation point at the top of the layer of interest
- β = Beta parameter, a function of the computed Frohlich σ_z and layer CBR, used to compute the allowable coverages of the controlling vehicle

D-1.5 Alpha-Beta Hybrid Formulation.

To compute the allowable gross load for a given flexible pavement structure with CBR > 20 and < 30, the Alpha-Beta Hybrid formulation must be used. This requires the CBR-Beta procedure to be used to compute the AGL at CBR = 20 and the CBR-Alpha procedure to compute the AGLs at CBR = 25 and CBR = 30 to establish a bilinear approximation of the nonlinear relationship between the two formulations. Interpolation is then used to compute the AGL at a specified CBR. This is only an interim solution to address the problems that arise when evaluating contingency structures with thin asphalt layers and marginal base materials.

Figure D-4 Westergaard-k Procedure for Allowable Coverage Computation



D-1.6 Variables for Figure D-4.

- σ_e = "free edge bending stress" = the maximum tensile stress at the bottom edge of the slab due to loading on the slab's free edge (i.e., no joint) computed from Westergaard plate solution
- *DF* = design factor, a function of concrete flexural strength and the free edge stress reduced by 25 percent considering joint load transfer, used to compute the allowable coverages of the controlling vehicle
- *R* = concrete flexural strength
- *k* = modulus of subgrade reaction used to characterize entire supporting structure beneath slab

Figure D-5 Layered Elastic Procedure for Allowable Passes Computation

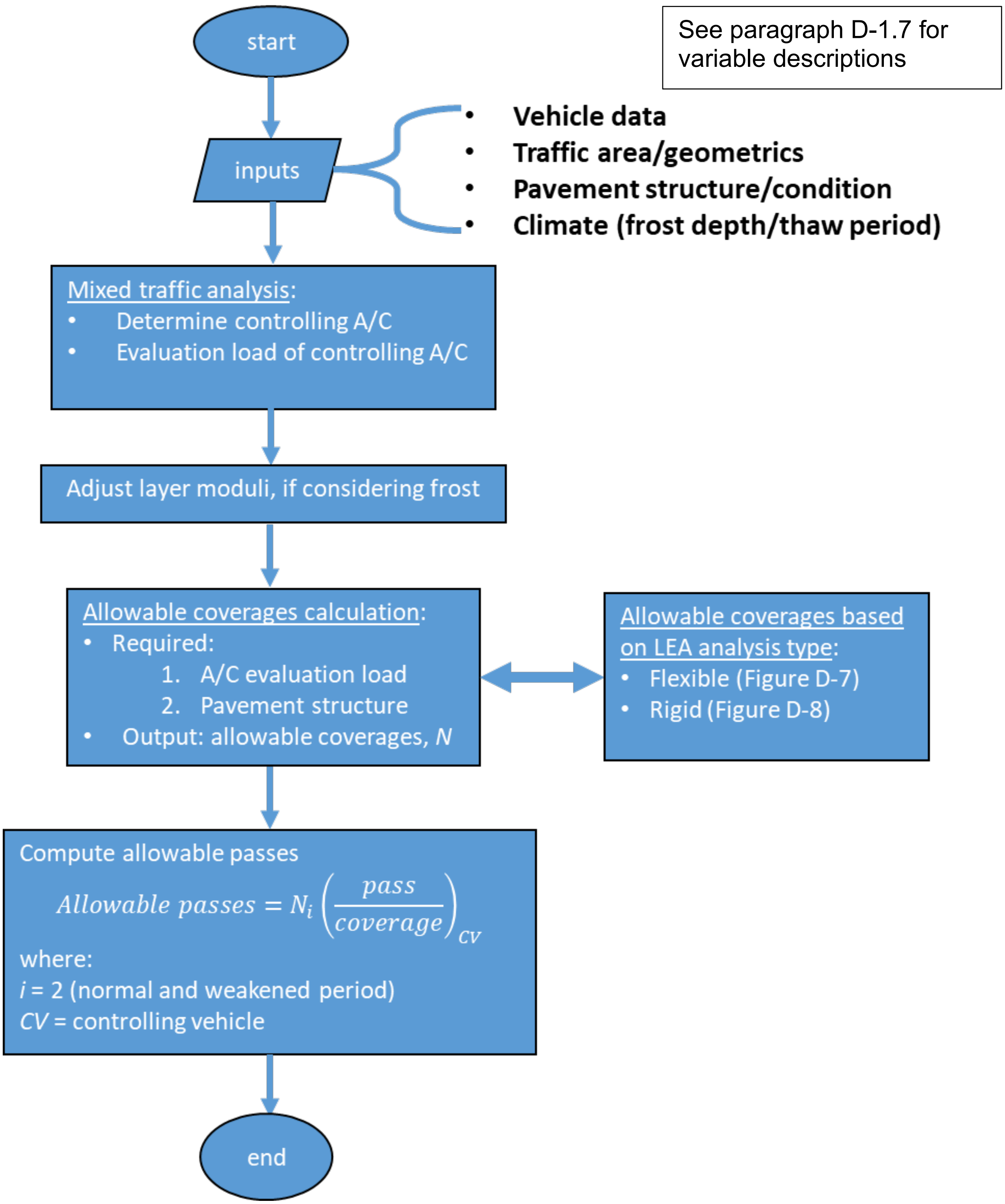
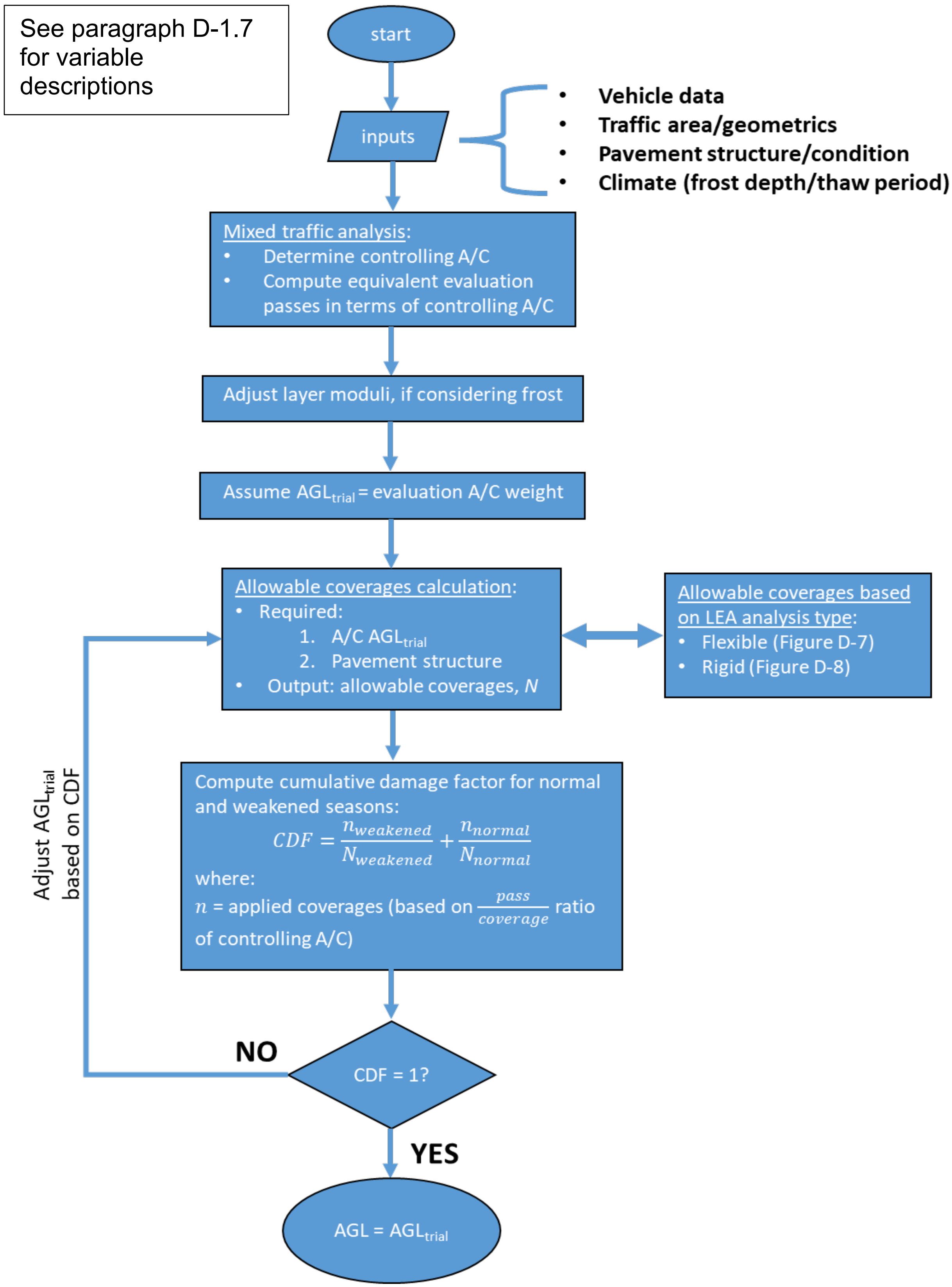


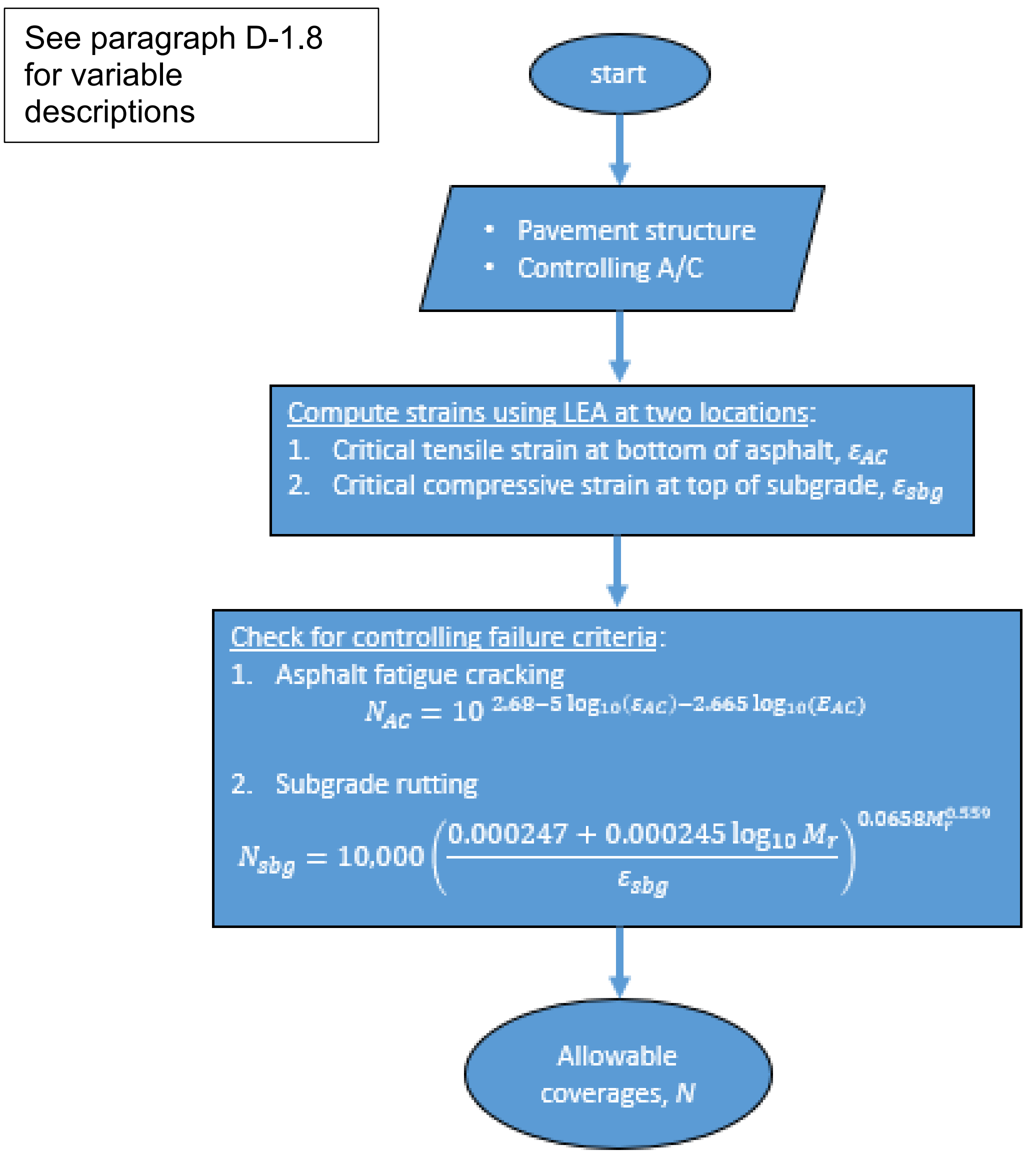
Figure D-6 Layered Elastic Procedure for Allowable Gross Load Computation



D-1.7 Variables for Figures D-5 and D-6.

- n_{thaw} = the number of applied coverages during the thaw period
- n_{normal} = the number of applied coverages during the normal period
- N_{thaw} = computed allowable coverages from performance model during thaw period
- N_{normal} = computed allowable coverages from performance model during normal period

Figure D-7 Layered Elastic Analysis Flexible Pavement Allowable Coverage Computation

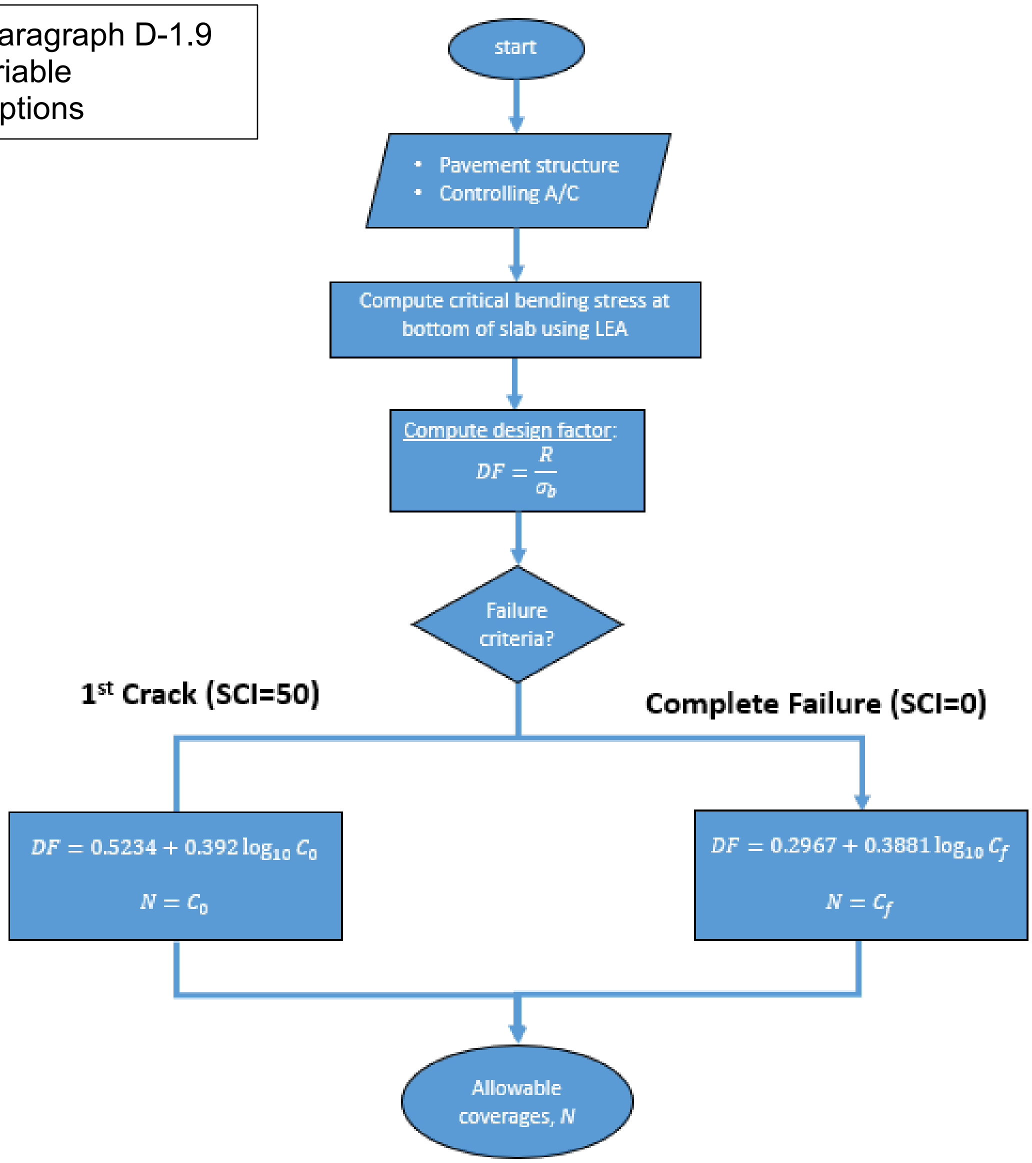


D-1.8 Variables for Figure D-7.

- ε_{AC} = tensile strain at the bottom of the asphalt layer (in/in)
- ε_{sbg} = compressive strain at the top of the subgrade layer (in/in)
- N_{AC} = computed allowable coverages using asphalt fatigue cracking performance model
- N_{sbg} = computed allowable coverages using subgrade rutting performance model
- E_{AC} = asphalt layer Young's modulus of elasticity (psi)
- Mr = subgrade layer resilient modulus (psi)

Figure D-8 Layered Elastic Analysis Rigid Procedure for Allowable Coverage Computation

See paragraph D-1.9 for variable descriptions



D-1.9 Variable for Figure D-8.

- σ_b = interior bending stress in slab
- SCI = structural condition index
- C_0 = computed allowable coverages before first crack failure criteria (SCI = 50)
- C_f = computed allowable coverages before complete failure criteria (SCI = 0)

APPENDIX E PCASE PAVEMENT EVALUATION APPLICATION

E-1 BACKGROUND.

The Services use the Pavements-Transportation Computer Assisted Structural Engineering (PCASE) application to design and evaluate airfield and road and parking pavements. The program is managed Jointly by the US Army Corps of Engineers Transportation Systems Center (TSC) and Engineer Research and Development Center (ERDC) Geotechnical and Structures Lab, with support from a Tri-Service governance working group. TSC and ERDC continuously update, expand, and improve the application and provide technical assistance, consulting services, and training. PCASE training is highly encouraged to ensure the latest criteria and technology is used to design and evaluate pavements. TSC contact information is below:

U.S. Army Corps of Engineers
Transportation Systems Center
1616 Capitol Ave.
Omaha, NE 68102-4901
Telephone: 402-995-2399

E-2 USING PCASE.

Details on PCASE installation and use for design and evaluation are available in the *PCASE User Manual.* The guide and latest version of the application are available at:

https://transportation.erdc.dren.mil/pcase/

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APPENDIX F GLOSSARY

F-1 ACRONYMS.

A/C	Aircraft
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Concrete
ACN	Aircraft Classification Number
ADCP	Automated Dynamic Cone Penetrometer
AFI	Air Freezing Index
AGL	Allowable Gross Load
APE	Airfield Pavement Evaluation
ASTM	American Society for Testing and Materials
BIA	Bilateral Infrastructure Agreement
CBR	California Bearing Ratio
CDD	Cumulative Degree Days
CDF	Cumulative Damage Factor
CP	Coordinating Panel
CRREL	Cold Regions Research and Engineering Laboratory
DCP	Dynamic Cone Penetrometer
DFI	Design Freezing Index
DoD	Department of Defense
DWG	Discipline Working Group
ERDC	Engineer Research and Development Center
FAIR	Frost Area Index of Reaction
FASSI	Frost Area Soil Support Indices
FDD	Freezing Degree Days

FLIP	Flight Information Pamphlet
FOD	Foreign Object Debris
FWD	Falling Weight Deflectometer
GHz	Gigahertz
GIS	Geographic Information System
GPR	Ground Penetrating Radar
GPS	Global Positioning System
Hz	Hertz
ICAO	International Civil Aviation Organization
ISM	Impulse Stiffness Modulus
LEEP	Layered Elastic Evaluation Program
LL	Liquid Limit
M&R	Maintenance and Repair
MHz	Megahertz
MPa	Megapascal
NDT	Nondestructive Testing
NFS	Nonfrost Susceptible
NGA	National Geospatial-Intelligence Agency
PCC	Portland Cement Concrete
pcf	Pound per Cubic Foot
PCI	Pavement Condition Index
pci	Pound per Cubic Inch
PCN	Pavement Classification Number
PPD	Physical Property Data
psi	Pound per Square Inch

PSPA	Portable Seismic Pavement Analyzer
RMSE	Root Mean Square Error
RSS	Reduced Subgrade Strength
SCI	Structural Condition Index
SHRP	Strategic Highway Research Program
SI	Structural Index
TM	Technical Manual
TSPWG M	Tri-Service Pavements Working Group Manual
UFC	Unified Facilities Criteria
UFGS	Unified Facilities Guide Specifications
USCS	Unified Soil Classification System

F-2 DEFINITION OF TERMS.

Average Daily Temperature: The average of the maximum and minimum temperatures for one day, or the average of several temperature readings taken at equal time intervals, generally hourly, during a day.

Combined Base Thickness: Term used in frost analysis that means the combined thickness of base, subbase, drainage layer, and separation layer.

Critical Weakening Period: Interval during the period of thaw weakening when the base, subbase, or subgrade are at their lowest strength.

Degree-Days: The Fahrenheit degree days for any given day equal the difference between the average daily air temperatures and 32 degrees F (0 degrees C). The Centigrade degree hours for any given day equal the average daily temperatures (degrees C) multiplied by 24 hours. The degree-days or degree-hours are negative when the average daily temperature is below 32 degrees F (0 degrees C) (freezing degree-days or hours) and positive when above (thawing degree-days or hours). Usually, the degree-days or hours are reported in terms of their absolute values and the distinction is made between freezing and thawing.

Design Freezing Index: The average air freezing index of the three coldest winters in the latest 30 years of record. If 30 years of record are not available, the air freezing index for the coldest winter in the latest 10-year period may be used. The design freezing index at a site need not be changed more than once in 5 years unless the more recent temperature records indicate a significant change in thickness requirements for frost protection.

Discipline Working Group: Representatives from the DoD components responsible for the unification and maintenance of criteria documents. (MIL-STD-3007)

Freezing Index: The number of degree-days between the highest and lowest points on a curve of cumulative degree-days versus time for one freezing season. It is used as a measure of the combined duration and magnitude of below-freezing temperatures occurring during any given freezing season. The index is determined from air temperatures measured approximately 4.5 feet (1 meter) above the ground and is commonly designated as the air freezing index.

Frost Action: A general term for freezing and thawing of moisture in materials and the resultant effects on these materials and on structures of which they are a part, or with which they are in contact.

Frost Area Soil Support Indices (FASSI): The weighted average of CBR values for the annual cycle. These values are used in flexible pavement evaluation for the frost-melt period, as if they are true CBR values.

Frost Area Index of Reaction (FAIR): The weighted average of k values for the annual cycle. These values are used for rigid pavement evaluation for the frost-melt period, as if they are true k values.

Frost Susceptible Soil: Soil in which significant detrimental ice segregation will occur when the requisite moisture and freezing conditions are present. These soils will lose a substantial portion of their strength upon thawing.

Frost Heave: The raising of the pavement surface due to formation of ice lenses in the underlying soil.

Frost-melting (Thaw) Periods: Intervals of the year when the ice in the base, subbase, or subgrade returns to a liquid state. A period ends when all the ice in the ground has melted or when the previously frozen material is refrozen. In general, there may be several significant frost-melting periods during the winter months prior to the spring thaw.

Mean Daily Temperature: The mean of the average daily temperatures for a given day, usually calculated over a period of several years.

Mean Freezing Index: The freezing index determined based on mean daily temperatures. The period of record over which average daily temperatures are averaged is usually a minimum of the latest 10 years, preferably 30.

Non-frost Susceptible Materials: Cohesionless materials such as crushed rock, gravel, sand, slag, and cinders that do not experience significant detrimental ice segregation under normal freezing conditions. Cemented or stabilized materials that do not experience significant detrimental ice segregation, loss of strength upon thawing, and freeze thaw degradation are also considered to be non-frost susceptible materials.

Normal Period: Interval during the year when the base, subbase, and subgrade strengths are at their normal strength.

Recovery Period: Interval from the end of the critical weakening period to the beginning of the normal period. During this time the base, subbase, and subgrade strengths are recovering to normal strength from their lowest strength.

Surface Freezing Index: The n factor * Design Freezing Index = the Surface Freezing Index.

Thaw-Weakened Periods: Intervals of the year when the base, subbase, or subgrade strength are below normal summer values. These intervals correspond to thaw periods. The period ends when either the material is refrozen or when the subgrade strength has returned to the normal summer value at the end of the spring thaw-weakening period.

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