U.S. Department
of Transportation
Federal Aviation
Administration

Advisory Circular

Subject: Airport Pavement Design and Evaluation

Date: 11/10/2016
Initiated by: AAS-100

AC No: 150/5320-6F
Change:

1. **Purpose.**

This advisory circular (AC) provides guidance to the public on the design and evaluation of pavements used by aircraft at civil airports. For reporting of pavement strength, see AC 150/5335-5C, *Standardized Method of Reporting Airport Pavement Strength – PCN*.

2. **Cancellation.**

This AC cancels AC 150/5320-6E, *Airport Pavement Design and Evaluation,* dated September 30, 2009.

3. **Application.**

The FAA recommends the guidance and standards in this AC for airport pavement design and evaluation. In general, use of this AC is not mandatory. However, use of the standards in this AC is mandatory for all projects funded under the Airport Improvement Program (AIP) or with revenue from the Passenger Facility Charge (PFC) Program.

This AC does not apply to the design of pavements that are not used by aircraft, i.e., roadways, parking lots, and access roads.

4. **Principal Changes.**

This AC contains the following changes:

1. Reformatted to comply with FAA Order 1320.46, *FAA Advisory Circular System*.
2. Revised text and design examples to incorporate changes in FAARFIELD v1.41 pavement design software. Also added general guidance on how to use FAARFIELD.
3. Simplified and moved guidance on economic analysis to Chapter 1.

4. Included all pavement design in Chapter 3, including previous guidance on pavement design for airplanes weighing less than 30,000 pounds (13 610 kg).
5. Defined "Regular use" for pavement design as at least 250 annual departures, which is equivalent to 500 annual operations.
6. Removed information on embedded steel and continuously reinforced concrete pavement.
7. Added table on allowable modulus values and Poisson's Ratios used in FAARFIELD.
8. Added tables for minimum layer thickness for flexible and rigid pavement structures.
9. Added detail on reinforcement at a reinforced isolation joint.
10. Added detail for transition between PCC and HMA pavement sections.
11. Added appendix, Nondestructive Testing (NDT) Using Falling-Weight Type Impulse Load Devices in the Evaluation of Airport Pavements.

5. Related Reading Material.

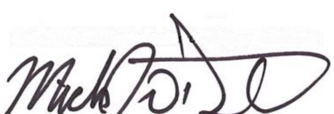
The publications listed in Appendix F provide further guidance and detailed information on the design and evaluation of airport pavements.

6. Units.

Through this AC, customary English units will be used followed by "soft" (rounded) conversion to metric units for tables and figures and hard conversion for the text. The English units govern.

7. Feedback on this AC.

If you have suggestions for improving this AC, you may use the Advisory Circular Feedback form at the end of this AC.



Michael O'Donnell
Director of Airport Safety and Standards

Contents

Paragraph **Page**

Contents (CONTINUED)

Contents (CONTINUED)

Contents (CONTINUED)

Paragraph **Page**

Contents (CONTINUED)

Paragraph **Page**

FIGURES

Number **Page**

TABLES

CHAPTER 1. AIRPORT PAVEMENTS—THEIR FUNCTION AND PURPOSES

1.1 General.

1.1.1 An airport pavement is a complex engineering structure. Pavement analysis and design involves the interaction of four equally important components: the subgrade (naturally occurring soil), the paving materials (surface layer, base, and subbase), the characteristics of applied loads, and climate.

1.1.2 Airport pavements are designed and constructed to provide adequate support for the loads imposed by airplanes and to produce a firm, stable, smooth, skid resistant, year-round, all-weather surface free of debris or other particles that can be blown or picked up by propeller wash or jet blast. To fulfill these requirements, the quality and thickness of the pavement must not fail under the imposed loads. The pavement must also possess sufficient inherent stability to withstand, without damage, the abrasive action of traffic, adverse weather conditions, and other deteriorating influences. This requires coordination of many design factors, construction, and inspection to assure the best combination of available materials and workmanship.

1.1.3 The pavement design guidance presented in this AC is based on layered elastic theory for flexible pavement design and three-dimensional finite element theory for rigid pavement design.. These methodologies address the impact of landing gear configurations and increased pavement load conditions on airport pavements without modifying the underlying design procedures. The failure curves have been calibrated with full scale pavement tests at the FAA National Airport Pavement Test Facility (NAPTF). The FAA has developed the computer program FAA Rigid and Flexible Iterative Elastic Layer Design (FAARFIELD) to assist with pavement design, see chapter 3 for detailed information on FAARFIELD.

1.1.4 Guidance on airport pavement structural evaluation necessary to assess the ability of an existing pavement to support different types, weights, or volume of airplane traffic is presented this AC. Since the pavement failure models have been updated, current pavement design methodology may produce different pavement thicknesses than the methods used to design the original pavement. Engineering judgment must be used when evaluating results.

1.2 Construction Specifications and Geometric Standards.

1.2.1 <u>Specifications</u>.

Construction material specifications referenced by Item Number (e.g. P-401, Hot Mix Asphalt (HMA) Pavements; P-501, Portland Cement Concrete (PCC) Pavement, etc.) are contained in AC 150/5370-10, *Standards for Specifying Construction of Airports.*

1.2.2 Geometric Standards.

Airport design standards and recommendations including runway and taxiway geometric design, widths, grades, and slopes are contained in AC 150/5300-13, *Airport Design;* and lengths of runways is discussed in AC 150/5325-4, *Runway Length Requirements for Airport Design.*

1.3 Airfield Pavements.

1.3.1 Types of Pavement.

Pavements discussed in this AC include flexible, rigid, and flexible and rigid overlays. Various combinations of pavement types and stabilized layers result in complex pavements classified between flexible and rigid.

- **Flexible pavements** are those in which each structural layer is supported by the layer below and ultimately supported by the subgrade. Hot mix asphalt (HMA) and P-401/403 refer to flexible pavements.
- **Rigid pavements** are those in which the principal load resistance is provided by the slab action of the surface concrete layer. Portland cement concrete (PCC) and P-501 refer to rigid pavements.

1.3.2 Selection of Pavement Type.

1.3.2.1 With proper design, materials, construction, and maintenance, any pavement type can provide the desired pavement service life. Historically, airport pavements have performed well for 20 years as shown in *Operational Life of Airport Pavements*, (DOT/FAA/AR-04/46). However, no pavement structure will perform for the desired service life without using quality materials installed and maintained with timely routine and preventative maintenance.

1.3.2.2 The selection of a pavement section requires the evaluation of multiple factors including cost and funding limitations, operational constraints, construction time-frame, cost and frequency of anticipated maintenance, environmental constraints, material availability, future airport expansion plans, and anticipated changes in traffic. The engineer must document the rationale for the selected pavement section and service life in the engineer's report.

1.3.3 Cost Effectiveness Analysis.

1.3.3.1 When considering alternative pavement sections it is assumed that all alternatives will achieve the desired result. The question is which design alternative results in the lowest total cost over the life of the project and what are the user-cost impacts of alternative strategies. Present worth or present value economic analyses are considered the best methods for evaluating airport pavement design or rehabilitation alternatives. Refer to

OMB Circular A-94, Appendix C, *Discount Rates for Cost-Effectiveness, Lease Purchase, and Related Analysis,* for real discount rates for the design analysis period. For federally funded projects, use the most recent discount rate published by the Office of Management and Budget (OMB) appropriate for a cost effectiveness analysis. When applicable calculate residual salvage values on the straight-line depreciated value of the alternative at the end of the analysis period. The initial cost and life expectancy of the various alternatives should be based on the engineer's experience with consideration given to local materials, environmental factors, and contractor capability. When considering the effectiveness of various routine and preventative maintenance alternatives, refer to Airfield Asphalt Pavement Technology Program (AAPTP) Project 05-07, *Techniques for Prevention and Remediation of Non-Load Related Distresses on HMA Airport Pavements (Phase I).*

The basic equation for determining present worth is shown below:

$$\mathrm{PW} = C + \sum_{i=1}^{m} M_i \left(\frac{1}{1+r}\right)^{n_i} - S\left(\frac{1}{1+r}\right)^{z}$$

Where:

PW	=	Present Worth
C	=	Present Cost of initial design or rehabilitation activity
m	=	Number of maintenance or rehabilitation activities
M_i	=	Cost of the ith maintenance or rehabilitation alternative in terms of present costs, i.e., constant dollars
r	=	Discount rate
n_i	=	Number of years from the present of the ith maintenance or rehabilitation activity
S	=	Salvage value at the end of the analysis period
Z	=	Length of analysis period in years. The FAA design period is 20 years. For federally funded projects, the FAA must approve other analysis periods.
$\left(\frac{1}{1+r}\right)^{n}$		is commonly called the single payment present worth factor in most engineering economic textbooks

1.3.3.2 From a practical standpoint, if the difference in the present worth of costs between two design or rehabilitation alternatives is 10 percent or less, it is normally assumed to be insignificant and the present worth of the two alternatives can be assumed to be the same.

1.3.3.3 A cost effectiveness determination includes a life-cycle cost analysis (LCCA). LCCA methodology includes the following steps:

1. Establish alternative design strategies;
2. Determine activity timing (analysis period should be sufficient to reflect long term cost differences including at least one rehab of each alternative); and
3. Estimate direct costs (future costs should be estimated in constant dollars and discounted to the present using real discount rate).
4. Note: Analysis period is period of time over which alternative pavement sections are compared and is not the design life used for the pavement design.

1.3.3.4 Routine maintenance costs, such as incidental crack sealing, have a marginal effect on net present value (NPV). Focus should be on initial construction, preventative maintenance, and rehabilitation costs. Salvage value should be based on the remaining life of an alternative at the end of the analysis period.

Note: LCCA, at a minimum, should include a sensitivity analysis to address the variability within major analyses input assumptions and estimates. Traditionally, sensitivity analysis has evaluated different discount rates or assigned value of time. The ultimate sensitivity analysis is to perform a probabilistic analysis, which allows multiple inputs to vary simultaneously.

1.3.3.5 Just because a life cycle cost analysis supports a pavement section does not ensure that funds will be available to support the initial construction. On federally funded projects coordination with and approval by the local FAA Region/ADO is required when considering design periods greater or less than 20 years.

1.3.3.6 For additional information on performing LCCA, refer to Airfield Asphalt Pavement Technology Program (AAPTP) Report 06-06, *Life Cycle Cost Analysis for Airport Pavements*, and the Federal Highway Administration *Life-Cycle Cost Analysis Primer*. AIRCOST, an Excel based spreadsheet for performing LCCA analysis, can be downloaded from http://www.aaptp.us.

1.3.4 Pavement Structure.

Pavement structure consists of surface course, base course, subbase course, and subgrade as illustrated Figure 1-1 and described in Table 1-1.

1. **Surface.** Surface courses typically include Portland cement concrete (PCC) and Hot-Mix Asphalt (HMA).
2. **Base.** Base courses generally fall into two classes: unstabilized and stabilized.

a. **Unstabilized bases** consist of crushed and uncrushed aggregates.

b. **Stabilized bases** consist of crushed and uncrushed aggregates stabilized with cement or asphalt.

3. **Subbase.** Subbase courses consist of granular material, which may be unstabilized or stabilized.

4. **Subgrade.** Subgrade consists of natural or modified soils.

Figure 1-1. Typical Pavement Structure

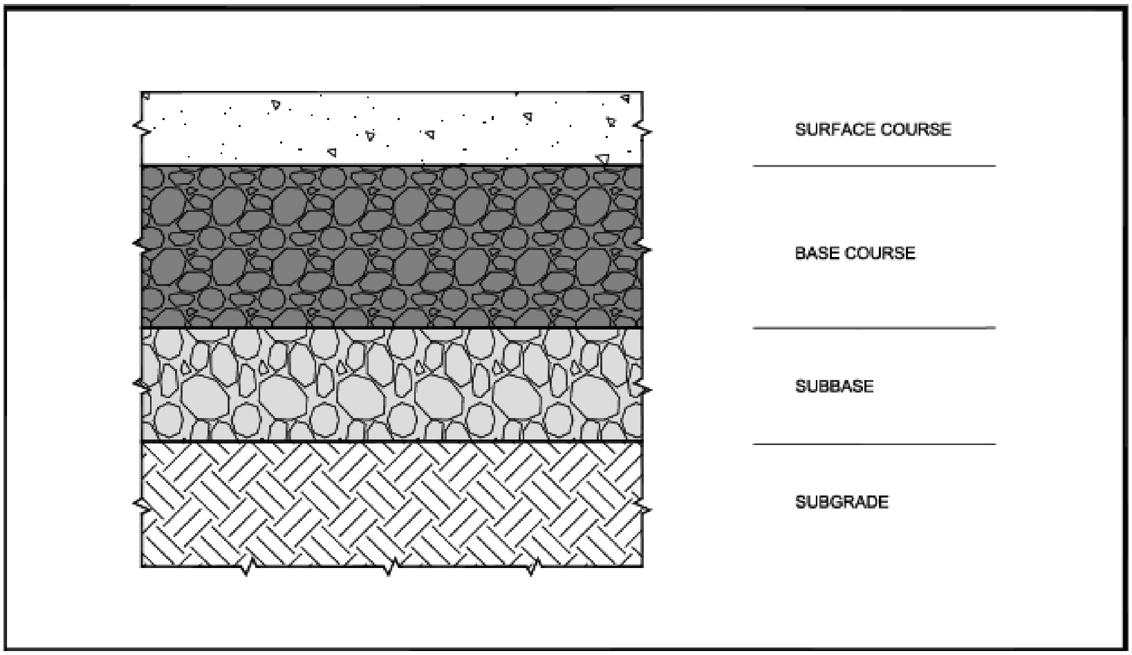


Table 1-1. Typical Pavement Specifications for Pavement Layers[1]

Pavement Layer	Flexible Pavement	Rigid Pavement
Surface Course	P-401/P-403[2]	P-501
Stabilized Base Course	P-401/403 P-304[3] P-306[3]	P-401/403 P-304[3] P-306[3]
Base Course	P-209[4] P-208[5] P-211	P-209[4] P-208[5] P-211
Subbase Course	P-154 P-213[6] P-219[7]	P-154 P-301[6] P-219[7]
Subgrade	P-152 P-155 P-157 P-158	P-152 P-155 P-157 P-158

Notes:

1. Refer to AC 150/5370-10, *Standards for Specifying Construction of Airports,* for the individual specifications.
2. P-601 may be used for locations that need a fuel resistant surface.
3. P-304 and P-306 should be used with caution because it is susceptible to reflective cracking.
4. P-209, Crushed Aggregate Base Course, used as a base course is limited to pavements designed for gross loads of 100,000 pounds (45 360 kg) or less.
5. P-208, Aggregate Base Course, used as base course is limited to pavements designed for gross loads of 60,000 pounds (27 200 kg) or less.
6. Use of P-213 and P-301 as subbase course is not recommended where frost penetration into the subbase is anticipated.
7. P-219, Recycled Concrete Aggregate Base Course, may be used as base depending on quality of materials and gradation.

1.4 Skid Resistance.

Airport pavements should provide a skid resistant surface that will provide good traction during all weather conditions. Refer to AC 150/5320-12, *Measurement, Construction, and Maintenance of Skid Resistant Airport Pavement Surfaces,* for information on skid resistant surfaces.

1.5 Staged Construction.

It may be necessary to construct the airport pavement in stages to accommodate changes in traffic, increases in aircraft weights, frequency of operation or to address funding limitations. The stages may be vertical (i.e. successive layer strengthening) or lateral (i.e. widening, lengthening, etc). When designing airport pavements, give consideration for planned runway/taxiway extensions, widening, parallel taxiways, and other changes to ensure that each stage provides an operational surface that can safely accommodate the current traffic. Consider alignments of future development when selecting the longitudinal grades, cross-slope grade, stub-taxiway grades, etc. Design each stage to safely accommodate the traffic using the pavement until the next stage is constructed. Initial construction must consider the future structural needs for the full service life of the pavement. Design and construction of the underlying layers and drainage facilities must be to the standards required for the final pavement cross-sections. Refer to AC 150/5320-5, *Airport Drainage*, for additional guidance on design and construction of airport surface and subsurface drainage systems for airports.

1.6 Design of Structures.

Refer to Appendix B for recommended design parameters for airport structures such as culverts and bridges.

CHAPTER 2. SOIL INVESTIGATIONS AND EVALUATION

2.1 General.

Accurate identification and evaluation of pavement foundations is necessary. The following sections highlight some of the more important aspects of soil mechanics that are important to the geotechnical and pavement engineers.

2.1.1 Soil.

For engineering purposes, soil includes all natural deposits that can be moved and manipulated with earth moving equipment, without requiring blasting or ripping. The soil profile is the vertical arrangement of individual soil layers exhibiting physical properties different than the adjacent layer. Subgrade soil is the soil layer that forms the foundation for the pavement structure; it is the soil directly beneath the pavement structure. Subsurface soil conditions include the elevation of the water table, the presence of water bearing strata, and the field properties of the soil. Field properties include the density, moisture content, frost susceptibility, and typical depth of frost penetration.

2.1.2 Classification System.

Use ASTM D 2487, *Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)*, to classify soils for civil airport pavements for engineering purposes. Appendix A provides a summary of general soil characteristics pertinent to pavements.

2.1.3 Subgrade Support.

2.1.3.1 The subgrade soil provides the ultimate support for the pavement and the imposed loads. The pavement structure serves to distribute the imposed load to the subgrade over an area greater than the tire contact area. The available soils with the best engineering characteristics should be incorporated in the upper layers of the subgrade.

2.1.3.2 The design value for subgrade support should be conservatively selected to ensure a stable subgrade and should reflect the long term subgrade support that will be provided to the pavement. The FAA recommends selecting a value that is one standard deviation below the mean. Where the mean subgrade strength is lower than a California Bearing Ratio **(**CBR) of 5, it may be necessary to improve the subgrade through stabilization or other means in order to facilitate compaction of the subbase. When the design CBR is lower than 3, it is required to improve the subgrade through stabilization or other means. See paragraph 2.6.

2.1.4 Drainage.

Soil conditions impact the size, extent, and nature of surface and subsurface drainage structures and facilities. General guidance on basic drainage layers is discussed in

Chapter 3. For detailed guidance on design of subsurface drainage layers, refer to AC 150/5320-5, *Airport Drainage Design,* Appendix G.

2.2 Soil Conditions.

2.2.1 Site Investigation.

Soil type and properties for soils to be used on the project must be assessed. If sufficient soils are not available within the boundaries of the airport, identify and investigate additional borrow areas. Investigations should determine the distribution and physical properties of the various types of soil present. This, combined with site topography and climate data, provides the information necessary for planning the development of the airport pavement structure. An investigation of in-situ soil conditions at an airport site will typically include the collection of representative samples of the soils to determine the soil profile and properties identifying the arrangement of the different soils. The site investigation should also include an evaluation of local materials and their availability for possible use in construction of the pavement structure.

2.2.2 Procedures.

ASTM D 420, *Standard Guide to Site Characterization for Engineering Design and Construction Purposes,* can be used for sampling and surveying procedures and techniques. This method is based on the soil profile. In the field, ASTM D 2488, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedures),* is commonly used to identify soils by such characteristics as color, texture, structure, consistency, compactness, cementation, and, to varying degrees, chemical composition.

2.2.3 Soil Maps.

Department of Agriculture, Natural Resources Conservation Service soils maps, United States Geological Survey (USGS) geologic maps, and engineering geology maps are valuable aids in the study of soils at and in the vicinity of the airport. The pedagogical classification determined from these maps does not treat soil as an engineering or construction material; however, the data obtained is useful for the engineer conducting preliminary investigations of site selection, development costs, and alignment, as well as for the agronomist in connection with the development of turf areas on airports. Much of this information is available on the respective agency websites.

2.2.4 Aerial Photography.

Relief, drainage, and soil patterns may be determined from aerial photography. A review of historical aerial site photographs may reveal prior drainage patterns and deposits of different soil types. Many websites now provide access to aerial photographs and maps useful for preliminary site investigations.

2.3 Surveying and Sampling.

2.3.1 Subsurface Borings and Pavement Cores of Existing Pavement.

2.3.1.1 The initial step in an investigation of subsurface conditions is a soil survey to determine the quantity and extent of the different types of soil, the arrangement of soil layers, and the depth of any subsurface water. Profile borings are usually obtained to determine the soil or rock profile and its lateral extent. The spacing of borings cannot always be definitely specified by rule or preconceived plan because of the variations at a site. Sufficient borings should be taken to identify the extent of soils encountered.

2.3.1.2 Additional steps that may be taken to characterize the subsurface include: Nondestructive testing (NDT) and Dynamic Cone Penetrometer (DCP) tests. Nondestructive testing (NDT), as described in Appendix C, can be used to evaluate subgrade strength and to assist with establishing locations for soil borings as well as sampling locations for evaluation of existing pavements. Dynamic Cone Penetrometer (DCP) tests, per ASTM D 6951 *Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications*, provide useful information. DCP tests can easily be run as each soil layer is encountered as a boring progresses or DCP tests can be run after taking pavement cores of existing pavements. DCP results can provide a quick estimate of subgrade strength with correlations between DCP and CBR. In addition, plots of DCP results provide a graphical representation of the relative strength of subgrade layers. Boring logs from original construction and prior evaluations can also provide useful information.

2.3.1.3 Cores of existing pavement provide information about the existing pavement structure. It is recommended to take color photographs of pavement cores and include with the geotechnical report.

2.3.2 Number of Borings, Locations, and Depths.

The locations, depths, and numbers of borings should be sufficient to determine and map soil variations. If past experience indicates that settlement or stability in deep fill areas at the location may be a problem, or if in the opinion of the geotechnical engineer more investigations are warranted, additional and/or deeper borings may be required to determine the proper design, location, and construction procedures. Where uniform soil conditions are encountered, fewer borings may be acceptable. Suggested criteria for the location, depth, and number of borings for new construction are given in Table 2-1. Wide variations in these criteria can be expected due to local conditions.

Table 2-1. Typical Subsurface Boring Spacing and Depth[1]

Area	Spacing	Depth
Runways, Taxiways and Taxilanes	Random Across Pavement at 200-foot (60 m) Intervals	Cut Areas - 10' (3 m) Below Finished Grade Fill Areas - 10' (3 m) Below Existing Ground
Other Areas of Pavement	1 Boring per 10,000 Square Feet (930 sq m) of Area	Cut Areas - 10' (3 m) Below Finished Grade Fill Areas - 10' (3 m) Below Existing Ground
Borrow Areas	Sufficient Tests to Clearly Define the Borrow Material	To Depth of Borrow Excavation

Note:
1. Boring depths should be sufficient to determine if consolidation and/or location of slippage planes will impact the pavement structure.

2.3.3 Boring Log.

2.3.3.1 The results of the soil explorations should be summarized in boring logs. A typical boring log includes location of the boring, date performed, type of exploration, surface elevation, depth of materials, sample identification numbers, classification of the material, water table, and standard penetration resistance. Refer to ASTM D 1586 Standard Test Method for Standard Penetration Test (SPT) and Split Barrel Sampling of Soils. Representative samples of the different soil layers encountered should be obtained and tested in the laboratory to determine their physical and engineering properties. If samples not obtained with split barrel, e.g. grab sample from flight auger extreme care should be used to assure that sample is representative and not a mixture of layers. In-situ properties, such as in-place moisture, density, shear strength, consolidation characteristics etc., may require obtaining "undisturbed" core samples per ASTM D 1587 *Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes*. Because test results only represent the sample being tested, it is important that each sample be representative of a particular soil type and not be a mixture of several materials.

2.3.3.2 Identification of soil properties from composite bag samples can lead to misleading representation of soil properties.

2.3.4 In-place Testing.

Pits, open cuts, or both may be required for making in-place bearing tests, taking undisturbed samples, charting variable soil strata, etc. This type of soil investigation may be necessary for projects involving in-situ conditions that warrant a high degree of accuracy.

2.3.5 Number of Cores

Sufficient cores should be taken to evaluate condition of existing pavement to help characterize extent and possible causes of distress. Cores of existing pavement structure aid in the determination of the extent of rehabilitation and/or reconstruction required to correct the distress.

2.4 **Soil Tests.**

2.4.1 Soil Testing Requirements.

2.4.1.1 The geotechnical engineer should identify the tests necessary to characterize the soil properties for the project. Subsurface evaluations may include the following standards:

1. ASTM D 421 *Standard Practice for Dry Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants*.

   This procedure is used to prepare samples for particle-size and plasticity tests to determine test values on air-dried samples.

2. ASTM D 422 *Standard Test Method for Particle-Size Analysis of Soils*.

   This analysis covers the quantitative determination of the particle sizes in soils.

3. ASTM D 4318 *Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils***.**

2.4.1.2 The plastic and liquid limits of a soil define the lowest moisture content at which a soil will change from a semisolid to a plastic state and a solid passes from a plastic to a liquid state, respectively. The plasticity index is the numerical difference between the plastic limit and the liquid limit and indicates the range in moisture content over which a soil remains in a plastic state prior to changing into a liquid. The plastic limit, liquid limit, and plasticity index of soils are used with the Unified Soil Classification System (ASTM D 2487) to classify soils. They are also used, either individually or together, with other soil properties to correlate with engineering behavior such as compressibility, permeability, compactibility, shrink-swell, and shear strength.

2.4.2 Moisture-Density Relations of Soils.

For compaction control during construction, the following ASTM test methods can be used to determine the moisture-density relations of the different soil types:

1. **Pavements Loads of 60,000 Pounds (27 216 kg) or More.** For pavements designed to serve airplanes weighing 60,000 pounds (27 200 kg) or more, use ASTM D 1557, *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort* (56,000 ft-lbf/ft^3 (2,700 kN-m/m^3)).

2. **Pavement Loads Less than 60,000 Pounds (27 216 kg).** For pavements designed to serve airplanes weighing less than 60,000 pounds (27 200 kg), use ASTM D 698, *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort* (12 400 ft-lbf/ft^3 (600 kN-m/m^3)).

2.5 Soil Strength Tests.

2.5.1 Soil classification for engineering purposes provides an indication of the suitability of the soil as a pavement subgrade. However, the soil classification does not provide sufficient information to predict pavement behavior. Performance variations can occur due to a variety of reasons including degree of compaction, degree of saturation (moisture content), height of overburden, etc.

2.5.2 For pavement design and evaluation, subgrade materials are characterized by a suitable strength or modulus parameter. For pavements to be designed with FAARFIELD, subgrade quality is best characterized by the Elastic Modulus (E), which is the material parameter used in internal FAARFIELD calculations. The value of E to be used in design or evaluation may be obtained by a variety of means.

2.5.3 For flexible pavements, the strength of the subgrade is typically measured by CBR tests. The elastic modulus E can be estimated from CBR using the following correlation: E (psi) = 1500 × CBR or E (MPa) = 10 × CBR. This is only an approximate relationship that is generally adequate for pavement design and analysis.

2.5.4 For rigid pavements, the strength of the subgrade is ideally measured by a plate load test, which gives the modulus of subgrade reaction (k-value). The elastic modulus E can be estimated from k-value using the following correlation: E (psi) = $20.15 \times k^{1.284}$ (k in pci). This is only an approximate relationship that is generally adequate for pavement design and analysis. If plate-load test data are unavailable, then the elastic modulus E should be estimated from CBR using the formula in paragraph 2.5.3.

2.5.5 In some cases, for example when designing overlays on existing pavements, it is not possible to obtain estimates of E from CBR or plate load data. In these cases, an estimate of E may be obtained by back-calculation from falling-weight deflectometer (FWD) data or other nondestructive testing (NDT) using the methods described in Chapter 5 and Appendix C.

2.5.6 California Bearing Ratio (CBR).

The CBR test is basically a penetration test conducted at a uniform rate of strain. The force required to produce a given penetration in the material under test is compared to the force required to produce the same penetration in a standard crushed limestone. The result is expressed as a ratio of the two forces (e.g., a material with a CBR of 15 means the material offers 15 percent of the resistance to penetration that the standard crushed limestone offers). Laboratory CBR tests should be performed in accordance with ASTM D 1883, *Standard Test Method for California Bearing Ratio (CBR) of Laboratory-Compacted Soils.* Field CBR tests should be conducted in accordance with

ASTM D 4429, *Standard Test Method for CBR (California Bearing Ratio) of Soils in Place.*

1. **Laboratory CBR.** Laboratory CBR tests are conducted on materials obtained from the site and remolded to the density that will be obtained during construction. Pavement foundations tend to reach nearly complete saturation after about 3 years. The CBR test should be run at a moisture content that simulates the condition of a pavement that has been in service for time, typically this is what is referred to as a 'soaked' or 'saturated' CBR. Seasonal moisture changes also dictate the use of a soaked CBR design value since traffic must be supported during periods of high moisture such as spring thaw.
2. **Field CBR.** Field CBR tests provide information on foundation materials that have been in place for several years. The materials should be in place for a sufficient time to allow for the moisture to reach an equilibrium condition, i.e. a fill that has been constructed and surcharged for a long period of time prior to pavement construction.
3. **CBR Gravelly Materials.** CBR tests are difficult to interpret on gravelly materials. Laboratory CBR tests on gravel often yield CBR results that are too high due to the confining effects of the mold. The assignment of CBR values to gravelly subgrade materials may be based on judgment and experience. The FAA pavement design procedure recommends a maximum subgrade *E* value of 50,000 psi (345 MPa) (CBR=33) for use in design.
4. **Lime Rock Bearing Ratio.** If the lime rock bearing ratio (LBR) is used to express soil strength, it may be converted to CBR by multiplying the LBR by 0.8.
5. **Number of CBR Tests.** The number of CBR tests required to establish a design value cannot be simply stated. Variability of the soil conditions encountered at the site combined with the low reliability of CBR tests has a significant influence on the number of tests needed. From three to seven CBR tests on each different major soil type should be sufficient.

2.5.7 Plate Bearing Test.

2.5.7.1 The plate bearing test measures the bearing capacity of the pavement foundation. The result, modulus of subgrade reaction (*k* value) is a measure of the pressure required to produce a unit deflection of the pavement foundation. The *k* value has the units pounds per cubic inch (Mega-newton per cubic meter). Plate bearing tests should be performed in accordance with the procedures contained in AASHTO T 222 *Standard Method of Test for Non-repetitive Static Plate Load Test of Soils and Flexible Pavement Components for Use in Evaluation and Design of Airport and Highway*. This method covers the making of non-repetitive static plate load tests on subgrade soils and flexible pavement components, in either the compacted condition or the natural state, and is intended to provide data for use in the evaluation and design of rigid and flexible-type airport and highway pavements.

2.5.7.2 In lieu of the plate bearing test, the *k* value may be estimated from the CBR per paragraph 3.14.4.

1. **Plate Bearing Test Conditions.** Plate bearing tests are conducted in the field on test sections constructed to the design compaction and moisture conditions. A correction to the k value is required to simulate the moisture conditions likely to be encountered by the in-service pavement.
2. **Plate Size.** The rigid pavement design presented in this AC is based on the elastic modulus (E) or resilient modulus (k value). The k value can be determined by a static plate load test using a 30-inch (762 mm) diameter plate. Using a smaller plate diameter may result in a higher k value.
3. **Number of Plate Bearing Tests.** Plate bearing tests are expensive to perform and the number of tests that can be conducted to establish a design value is limited. Generally only two or three tests can be performed for each pavement feature. The design *k* value should be conservatively selected.

2.5.8 Additional Soil Strength Tests.

Other tests that may be used to assist in evaluating subgrade soils include ASTM D 3080, *Standard Test Method for Direct Shear Tests of Soils Under Consolidated Drained Conditions,* ASTM D 2573, *Standard Test Method for Field Vane Shear Tests in Cohesive Soil,* or ASTM D 2166 Standard Test Method for Unconfined Compressive Strength of Cohesive Soil.

2.6 Subgrade Stabilization.

2.6.1 Where the mean subgrade strength is lower than a modulus of 7,500 psi (CBR 5), it may be necessary to improve the subgrade chemically, mechanically, or by replacement with suitable subgrade material. When the design modulus is lower than 4,500 psi (CBR is lower than 3), it is necessary to improve the subgrade through stabilization or replacement with suitable subgrade material. Subgrade stabilization should also be considered if any of the following conditions exist: poor drainage, adverse surface drainage, frost, or need for a stable working platform. Subgrade stabilization can be accomplished through the use of chemical agents or by mechanical methods. It is often beneficial to stabilize the subgrade to create a stable construction working platform. When it is not possible to create a stable subgrade with either chemical or mechanical stabilization it may be necessary to remove and replace the unsuitable material.

2.6.2 A geotechnical engineer should be consulted to determine what long-term strength can be achieved with stabilized layers. It is recommended to use a very conservative estimate of the benefit unless you have tests results to substantiate the long-term benefit. Note: Generally the stabilized layer should be 12 in (300 mm) or otherwise recommend by the geotechnical engineer. When designing pavements that include a layer of stabilized material it may be necessary to model this layer as a user defined layer when performing pavement structural design in FAARFIELD, see Chapter 3

2.6.3 Chemical Stabilization.

Different soil types require different stabilizing agents for best results. The following publications are recommended to determine the appropriate type and amount of chemical stabilization for subgrade soils: Unified Facilities Criteria (UFC) Manual Pavement Design for Airfields, UFC 3-260-02; Soil Cement Construction Handbook, Portland Cement Association; The Asphalt Institute Manual Series MS-19, Basic Asphalt Emulsion Manual; and AC 150/5370-10, Items P-155, P-157, and P-158. See paragraph 3.13.5.5 for information regarding how to model chemical stabilized layers in FAARFIELD.

2.6.4 Mechanical Stabilization.

In some instances, subgrades cannot be adequately stabilized through the use of chemical additives. The underlying soils may be so soft that stabilized materials cannot be mixed and compacted over the underlying soils without failing the soft soils. To facilitate construction of the pavement section, extremely soft soils may require bridging of the weak soils. Bridging can be accomplished with the use of thick layers, 2-3 feet (600-900mm), of shot rock or cobbles. If open-graded aggregate layers are used for subgrade replacement, ensure that the layer is fully wrapped in geotextile fabric to prevent migration of fine soil particles into the layer. Thick layers of lean, porous concrete or geosynthetics may also be used as the first layer of mechanical stabilization over soft, fine-grained soils.

2.6.5 Geosynthetics.

2.6.5.1 The term geosynthetics describes a range of manufactured synthetic products used to address geotechnical problems. The term is generally understood to encompass four main products: geotextiles, geogrids, geomembranes, and geocomposites. The synthetic nature of the materials in these products makes them suitable for use in the ground where high levels of durability are required. These products have a wide range of applications, including use as a separation between subbase aggregate layers and the underlying subgrade.

2.6.5.2 The need for geosynthetics within a pavement section depends on subgrade soil conditions, groundwater conditions, and the type of overlying pavement aggregate. The geotechnical engineer should clearly identify what the geosynthetic is intended to provide to the pavement structure. The most common use on airports is as a separation layer to prevent migration of fines.

2.6.5.3 Currently, the FAA does not consider any reductions in pavement structure for the use of any geosynthetics. The FAA is currently researching the use of geosynthetics with aircraft loadings.

2.7 Seasonal Frost.

The design of pavements in areas subject to seasonal frost action requires special consideration. The detrimental effects of frost action may include non-uniform heave and a loss of soil strength during warm periods and spring thaw. Other detrimental effects include possible loss of compaction, development of pavement roughness, restriction of drainage, and cracking and deterioration of the pavement surface. Three conditions must exist simultaneously for detrimental frost action:

1. The soil must be frost susceptible,
2. Freezing temperatures must penetrate into the frost susceptible soil, and
3. Free moisture must be available in sufficient quantities to form ice lenses.

2.7.1 Frost Susceptibility.

The frost susceptibility of soils is dependent to a large extent on the size and distribution of voids in the soil mass. Voids must be of a certain critical size for the development of ice lenses. Empirical relationships have been developed correlating the degree of frost susceptibility with the soil classification and the amount of material finer than 0.02 mm by weight. Soils are categorized into four frost groups for frost design purposes as defined in Table 2-2: Frost Group 1 (FG-l), FG-2, FG-3, and FG-4. The higher the frost group number, the more susceptible the soil, i.e., soils in FG-4 are more frost susceptible than soils in frost groups 1, 2, or 3.

Table 2-2. Soil Frost Groups

Frost Group	Kind of Soil	Percentage Finer than 0.02 mm by Weight	Soil Classification
FG-1	Gravelly Soils	3 to 10	GW, GP, GW-GM, GP-GM
FG-2	Gravelly Soils Sands	10 to 20 3 to 5	GM, GW-GM, GP-GM SW, SP, SM, SW-SM, SP-SM
FG-3	Gravelly Soils Sands, except very fine silty sands Clays, PI above 12	Over 20 Over 15 -	GM, GC SM, SC CL, CH
FG-4	Very fine silty sands All Silts Clays, PI = 12 or less Varved Clays and other fine grained banded sediments	Over 15 - - -	SM ML, MH CL, CL-ML CL, CH, ML, SM

2.7.2 Depth of Frost Penetration.

The depth of frost penetration is a function of the thermal properties of the pavement and soil mass, the surface temperature, and the temperature of the pavement and soil mass at the start of the freezing season. In determining the frost penetration depth, give primary consideration to local engineering experience. Local construction practice, including the experience of local building departments, is generally a good guide to frost penetration depth, e.g. depth of water mains and depth of local foundation designs. The pavement design program PCASE includes a module to help evaluate the depth of frost penetration. PCASE is available at https://transportation.erdc.dren.mil/pcase/software.aspx.

2.7.3 Free Water.

For frost action to occur, there must be free water in the soil mass that can freeze and form ice lenses. Water can enter the soil from many different sources, e.g. by infiltration from the surface or sides of the pavement structure, by condensation of atmospheric water vapor, or drawn from considerable depths by capillary action. Generally speaking, if the degree of saturation of the soil is 70 percent or greater, frost heave will probably occur. The designer should assume that sufficient water will be present to cause detrimental frost action for any soil that may be susceptible to frost action. Edge drain systems may help reduce the amount of available water however the effectiveness of the edge drain system will be impacted by the type of subgrade soil present. Generally edge drain systems are most effective in removing free water when combined with a subsurface drainage layer. See AC 150/5320-5, *Airport Drainage Design.*

2.7.4 Frost Design.

The design of pavements to offset seasonal frost effects is discussed in Chapter 3. A more rigorous evaluation for frost effects is necessary when designing for pavement service life greater than 20 years. A discussion of frost action and its effects can be found in Research Report No. FAA-RD-74-030, *Design of Civil Airfield Pavement for Seasonal Frost and Permafrost Conditions.*

2.8 **Permafrost.**

In arctic regions, soils are often frozen to considerable depths year round. Seasonal thawing and refreezing of the upper layer of permafrost can lead to severe loss of bearing capacity and/or differential heave. In areas with continuous permafrost at shallow depths, utilize non-frost susceptible base course materials to prevent degradation (thawing) of the permafrost layer. The frost susceptibility of soils in permafrost areas is classified the same as in Table 2-2.

Note: In areas of permafrost, an experienced pavement/geotechnical engineer familiar with permafrost protection must design the pavement structure.

2.8.1 Depth of Thaw Penetration.

Pavement design for permafrost areas must consider the depth of seasonal thaw penetration. The thawing index used for design (design thawing index) should be based on the three warmest summers in the last 30 years of record. If 30-year records are not available, data from the warmest summer in the latest 10-year period may be used.

2.8.2 Muskeg.

Muskeg is a highly organic soil deposit that is essentially a swamp that is sometimes encountered in arctic areas. If construction in areas of muskeg is unavoidable, and the soil survey shows the thickness of muskeg is less than 5 feet (1.5 m), the muskeg should be removed and replaced with granular fill. If the thickness of muskeg is too great to warrant removal and replacement, a 5-foot (1.5 m) granular fill should be placed over the muskeg. These thicknesses are based on experience. Differential settlement will occur and considerable maintenance will be required to maintain a smooth surface. Use of a geosynthetic between the muskeg surface and the bottom of granular fill may be necessary to prevent migration of the muskeg up into the granular fill.

CHAPTER 3. PAVEMENT DESIGN

3.1 Design Considerations.

This chapter provides pavement design guidance for airfield pavements. Since the FAA computer program FAARFIELD is used for all pavement designs, there is no longer a differentiation between pavement design for light and aircraft greater than 30,000 pounds. Procedures for overlay design are covered in Chapter 4, and procedures for evaluating pavements are covered in Chapter 5.

3.2 FAA Pavement Design.

The design of airport pavements is a complex engineering problem that involves the interaction of multiple variables. This chapter presents mechanistic-empirical pavement design procedures that are implemented in the FAARFIELD computer program. FAARFIELD uses layered elastic and three-dimensional finite element-based design procedures for new and overlay designs of flexible and rigid pavements respectively. The structural design of pavements on federally funded projects must be completed using FAARFIELD, and a copy of the pavement design report must be included with the engineer's report.

3.3 Flexible Pavements.

For flexible pavement design, FAARFIELD uses the maximum vertical strain at the top of the subgrade and the maximum horizontal strain at the bottom of all asphalt layers as the predictors of pavement structural life. FAARFIELD provides the required thickness for all individual layers of flexible pavement (surface, base, and subbase) required to support a given airplane traffic mix for the structural design life over a given subgrade.

3.4 Full-Depth Asphalt Pavements.

Full-depth asphalt pavements that contain asphaltic cement in all components above the prepared subgrade may be used for pavements less than 60,000 pounds (27215 kg). FAARFIELD has the ability to analyze full depth asphalt pavements by only including HMA surface layer and a subgrade layer; however the program will identify it as a nonstandard layer. The preferred method of analyzing a full-depth asphalt pavement is to use a 3-layer structure consisting of a HMA surface layer on top of a HMA flexible stabilized base. The Asphalt Institute (AI) has published guidance on the design of full depth asphalt pavements for light airplanes in Information Series No. 154 (IS 154) Thickness Design - Asphalt Pavements for General Aviation. Use of the AI design method requires approval by the FAA. On federally funded projects full-depth asphalt pavements may be used in other applications when approved by the FAA.

3.5 Rigid Pavements.

For rigid pavement design, FAARFIELD uses the maximum horizontal stress at the bottom of the PCC slab as the predictor of the pavement structural life. The maximum horizontal stress for design is determined considering both PCC slab edge and interior loading conditions. FAARFIELD provides the required thickness of the rigid pavement slab required to support a given airplane traffic mix for the structural design life over a given base/subbase/subgrade.

3.6 Stabilized Base Course.

3.6.1 If aircraft in the design traffic mix have gross loads of 100,000 pounds (45,359 kg) or more, then use of a stabilized base is required. Crushed aggregates that can be proven to exhibit a remolded soaked CBR of 100 or greater may be substituted for a stabilized base course. In areas subject to frost penetration, the materials should meet permeability and non-frost susceptibility tests in addition to the CBR requirements. Other exceptions to the policy include proven performance under similar airplane loadings and climatic conditions comparable to those anticipated. Subbases used under stabilized bases should exhibit a remolded soaked CBR (per ASTM D1883) of at least 35. Suitable subbases for use under a stabilized base include P-209, P-208, or P-211. Other materials, such as P-219, may be acceptable with FAA approval.

3.6.2 Full scale performance tests have proven that pavements which include stabilized bases have superior performance. Long term performance gains should be considered before making substitutions to eliminate stabilized base. Exceptions to use of stabilized base may be considered when less than 5% of the traffic is aircraft with gross loads of 100,000 pounds (45,359 kg) or more but all aircraft gross loads are less than 110,000 pounds (49,895 kg).

3.7 Base or Subbase Contamination.

Contamination of subbase or base aggregates may occur during construction and/or once pavement is in service. A loss of structural capacity can result from contamination of base and/or subbase elements with fines from underlying subgrade soils. The contamination reduces the quality of the aggregate material, thereby reducing its ability to protect the subgrade. Geosynthetic separation fabrics can be effectively used to reduce aggregate contamination (refer to paragraph 2.6).

3.8 Drainage Layer

3.8.1 General guidance on basic drainage layers is discussed below. For detailed guidance on subsurface drainage layers, refer to AC 150/5320-5 *Airport Drainage Design, Appendix G, Design of Subsurface Drainage Systems.*

3.8.2 Pavements constructed in non-frost areas constructed on subgrade soils with a coefficient of permeability less than 20 ft/day (6 m/day) should include a subsurface drainage layer. Pavements in frost areas constructed on FG2 or higher subgrade soils should include a subsurface drainage layer. For rigid pavements the drainage layer is usually placed immediately beneath the concrete slab. For flexible pavements the drainage layer is usually placed immediately above the subgrade. An effective drainage layer will attain 85 percent drainage in 24 hours for runways and taxiways, and 85 percent drainage in 10 days for aprons and other areas with low speed traffic. In the structural design of the concrete slab the drainage layer along with the granular separation layer is considered a base layer. In flexible pavement structures when the required thickness of the granular subbase is equal to or greater than the thickness of the drainage layer plus the thickness of the separation layer, the drainage layer is placed beneath the aggregate base. Where the total thickness of the pavement structure is less than 12 inches (300 mm), the drainage layer may be placed directly beneath the surface layer and the drainage layer used as a base. When the drainage layer is placed beneath an unbound aggregate base, limit the material passing the No. 200 (0.075 mm) sieve in the aggregate base to less than 8 percent or less in accordance with AC 150/5370-10.

3.9 **Subgrade Compaction.**

3.9.1 FAARFIELD computes compaction requirements for the specific pavement design and traffic mixture and generates tables of required minimum density requirements for the subgrade. The values in these tables denote the range of depths for which densities should equal or exceed the indicated percentage of the maximum dry density as specified in Item P-152. Since compaction requirements are computed in FAARFIELD after the thickness design is completed, the computed compaction tables indicate recommended depth of compaction as measured from both the pavement surface and the top of finished subgrade. FAARFIELD determines whether densities are in accordance with ASTM D 698 or ASTM D 1557 based on weight of aircraft. ASTM D 698 applies for aircraft less than 60,000 pounds (27 200 kg) and ASTM D 1557 applies for aircraft 60,000 pounds (27 200 kg) and greater.

3.9.2 The compaction requirements implemented in the FAARFIELD computer program are based on the Compaction Index (CI) concept. More information may be found in U.S. Army Engineer Waterways Experiment Station, Technical Report No. 3-529, *Compaction Requirements for Soil Components of Flexible Airfield Pavements* (1959).

3.9.3 FAARFIELD generates two tables applicable to non-cohesive and cohesive soil types respectively. The appropriate compaction controls should be used for the actual soil type. Note: Non-cohesive soils, for the purpose of determining the compaction requirement, are those with a plasticity index of less than 3.

3.9.4 The subgrade for new flexible and rigid pavements in cut areas should have natural in-place densities equal to or greater than those computed by FAARFIELD for the given soil type. If the natural in-place densities of the subgrade are less than required, the subgrade should be (a) compacted to achieve the required densities (b) removed and replaced with suitable material at the required densities, or (c) covered with sufficient select or subbase material so the in-place densities of the natural subgrade meet the design requirements. It is a good practice to rework and recompact at least the top 12 inches (300 mm) in cut areas; however, depending upon the in-place densities, it may be necessary to rework and recompact additional subgrade material. The maximum practical depth of compaction of soils in cut areas is generally limited to 72 inches (1,829 mm) below the top of finished pavement.

3.9.5 For cohesive soils used in fill sections, for new flexible and rigid pavements, the entire fill must be compacted to 90 percent maximum density. For non-cohesive soils used in fill sections, the top 6 inches (150 mm) of fill must be compacted to 100 percent maximum density, and the remainder of the fill must be compacted to 95 percent maximum density, regardless of any lesser requirement indicated by FAARFIELD. When supported by a geotechnical engineers report lower compaction requirements may be justified to address unique local soil conditions.

3.10 **Swelling Soils.**

3.10.1 Swelling soils are clayey soils that exhibit a significant volume change caused by moisture variations. Airport pavements constructed on swelling soils are subject to differential movements causing surface roughness and cracking. When swelling soils are present, the pavement design should incorporate methods to prevent or reduce the effects of soil volume changes. Local experience and judgment should be applied in dealing with swelling soils to achieve the best results.

3.10.2 The clay minerals that cause swelling, in descending order of swelling activity, are smectite, illite, and kaolinite. These soils usually have liquid limits above 40 and plasticity indexes above 25.

3.10.3 Soils that exhibit a swell of greater than 3 percent when tested for the CBR, per ASTM D 1883 *Standard Test Method for California Bearing Ration (CBR) of Laboratory-Compacted Soils,* require treatment. Treatment of swelling soils consists of removal and replacement, stabilization, and compaction efforts in accordance with Table 3-1. Adequate drainage is important when dealing with swelling soils.

3.10.4 Additional information on identifying and handling swelling soils is presented in FAA Reports No. FAA-RD-76-066 *Design and Construction of Airport Pavements on Expansive Soils,* and DOT/FAA/PM-85115 *Validation of Procedures for Pavement Design on Expansive Soils*.

Table 3-1. Recommended Treatment of Swelling Soils

Swell Potential (Based on Experience)	Percent Swell Measured (ASTM D 1883)	Potential for Moisture Fluctuation[1]	Treatment
Low	3-5	Low	Compact soil on wet side of optimum (+2% to +3%) to not greater than 90% of appropriate maximum density.[2]
		High	Stabilize soil to a depth of at least 6 in. (150 mm)
Medium	6-10	Low	Stabilize soil to a depth of at least 12 in. (300 mm)
		High	Stabilize soil to a depth of at least 12 in. (300 mm)
High	Over 10	Low	Stabilize soil to a depth of at least 12 in. (300 mm)
		High	For uniform soils, i.e., redeposited clays, stabilize soil to a depth of at least 36 in. (900 mm) or raise grade to bury swelling soil at least 36 in. (900 mm) below pavement section or remove and replace with non-swelling soil. For variable soil deposits depth of treatment should be increased to 60 in. (1 500 mm).

Notes:

1. Potential for moisture fluctuation is a judgment determination and should consider proximity of water table, likelihood of variations in water table, as well as other sources of moisture, and thickness of the swelling soil layer.
2. When control of swelling is attempted by compacting on the wet side of optimum at a reduced density, the design subgrade strength should be based on the higher moisture content and reduced density.

3.11 Pavement Life.

3.11.1 Design Life in FAARFIELD refers to structural life. Structural life for design is related to the total number of load cycles a pavement structure will carry before it fails. Structural life is distinguished from functional life, which is the period of time that the pavement is able to provide an acceptable level of service as measured by performance indicators such as: foreign object debris (FOD), skid resistance, or roughness.

3.11.2 The structural design of airport pavements consists of determining both the overall pavement thickness and the thickness of the component parts of the pavement structure. A number of factors influence the required thickness of pavement including: the impact of the environment, the magnitude and character of the airplane loads it must support, the volume and distribution of traffic, the strength of the subgrade soils, and the quality of materials that make up the pavement structure. Pavements are designed to provide a finite structural life at design fatigue limits. It is theoretically possible to perform a pavement structural design for any service period. However, to achieve the intended life requires consideration of many interacting factors including: airplane mix, quality of materials and construction, and routine and preventative pavement maintenance.

3.11.3 Pavements on federally funded FAA projects are designed for a 20-year structural life. Designs for longer periods may be appropriate at airfields where the configuration of the airfield is not expected to change and where future traffic can be forecast with relative confidence beyond 20 years. A longer design life may be appropriate for a runway at a large hub airport where the future aircraft traffic can be forecast and where both the location and size of the runway and taxiways is not anticipated to change. However, when designing a taxiway at a smaller airport, it may be more prudent to design for no more than 20 years than to forecast the composition and frequency of future activity. Similarly, a phased project may only require a temporary pavement for 1-2 years. Many airports have significant changes planned, but whether these plans ultimately become reality depends on local economic conditions (e.g., business upturns or downturns at the fixed base operator (FBO), or the number and composition of based aircraft). Typically a life cycle cost effectiveness analysis is utilized to support design periods other than 20 years. However, fiscal constraints (i.e., funds available) may dictate which pavement section(s) and design life are considered. On federally funded projects FAA approval is required to use a design period other than 20 years.

3.11.4 To achieve intended design life all pavements require quality materials and construction combined with routine and/or preventative maintenance. To maximize a pavement's life, routine crack sealing and applications of pavement seal coats will be required for flexible pavements, and crack sealing and joint sealant repair/replacement will be required for rigid pavement. In addition, isolated slab replacement may be needed for some rigid pavements as well as small patches for some flexible pavements. Due to deterioration from normal use and the environment, rehabilitation of surface grades and renewal of skid-resistant properties may also be needed for both flexible and rigid pavements. Functional life may be longer or shorter than structural life, but is generally much longer when pavements are maintained properly.

3.12 Pavement Design Using FAARFIELD.

The FAA developed FAARFIELD using failure models based on full-scale tests conducted from the 1940s through the present. FAARFIELD is based on layered elastic and three-dimensional finite element-based structural analysis developed to calculate design thicknesses for airfield flexible and rigid pavements respectively.

3.12.1 Application.

The procedures and design software identified in this chapter provide standard pavement thickness designs meeting structural requirements for all airfield pavements. FAARFIELD currently does not take into account provisions for frost protection and permafrost discussed in paragraph 3.12.14. It is the responsibility of the user to check these provisions separately from FAARFIELD and to modify the thickness design if necessary to provide additional frost and or permafrost resistant materials. Functional failures in pavements (e.g., excessive roughness, FOD, or surface deformations) can often be traced to material or construction issues that are not addressed directly by FAARFIELD. FAARFIELD design assumes that all standard pavement layers meet the applicable requirements of AC 150/5370-10 for materials, construction, and quality control. Mix design requirements for HMA and PCC materials are covered in Items P-401/403 and P-501 respectively.

3.12.2 Cumulative Damage Factor (CDF).

FAARFIELD is based on the cumulative damage factor (CDF) concept in which the contribution of each aircraft type in a given traffic mix is summed to obtain the total cumulative damage from all aircraft operations in the traffic mix. FAARFIELD does not designate a design aircraft; however, using the CDF method, it identifies those aircraft in the design mix that contribute the greatest amount of damage to the pavement. Thickness designs using FAARFIELD use the entire traffic mix. Using departures of a single "design" aircraft to represent all traffic is not equivalent to designing with the full traffic mix in the CDF method and will generally result in excessive thickness.

3.12.3 Current Version FAARFIELD.

3.12.3.1 The current version of FAARFIELD is designated Version 1.4. It has been calibrated using the most recent full scale pavement tests at the FAA's National Airport Pavement Test Facility (NAPTF). Due to updates to the failure models for both rigid and flexible pavements, computed pavement thicknesses using FAARFIELD v1.4 may be different than those computed using earlier versions of FAARFIELD.

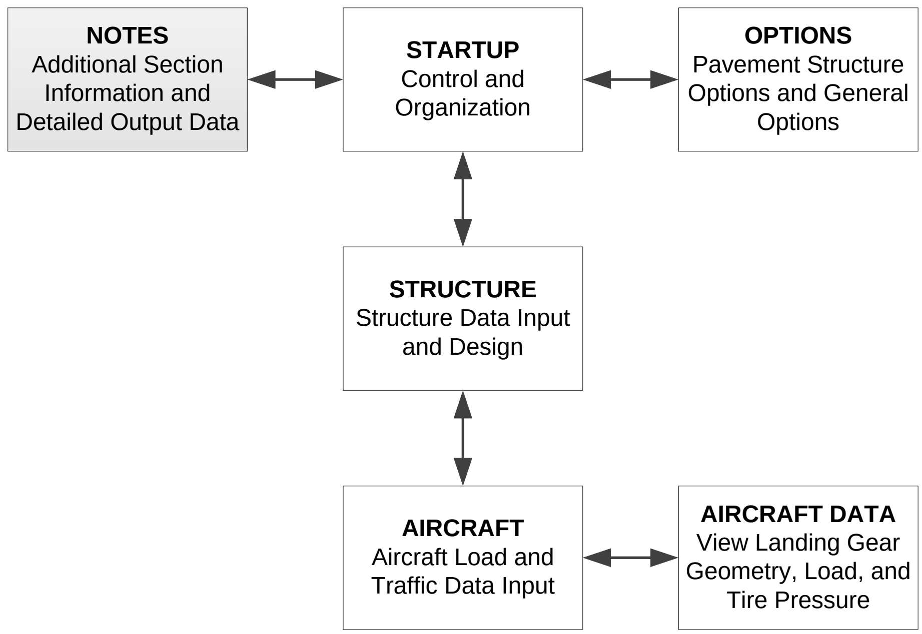
3.12.3.2 The internal help file for FAARFIELD contains a user's manual, which provides detailed information on proper execution of the program. The manual also contains additional technical references for specific details of the FAARFIELD design procedure.

3.12.3.3 FAARFIELD can be downloaded from the FAA website (http://www.faa.gov/airports/engineering/design_software/).

3.12.4 Overview of FAARFIELD Program.

FAARFIELD consists of five main forms linked as schematically shown in Figure 3-1. The primary forms are Startup, Structure and Aircraft. Startup establishes which job and section will be evaluated. Structure establishes the pavement structure to be analyzed. Aircraft establishes the aircraft operating weight and frequency of operation that will be used to apply loads to the pavement. Notes contains output data and other section information. Options contains analysis and output options. Note: The program may be operated with U.S. customary or metric dimensions, which can be selected on the Options form (see Figure 3-11).

Figure 3-1. Overview of FAARFIELD Program



3.12.5 FAARFIELD Pavement Design Process.

Pavement Design with FAARFIELD is an iterative process for both flexible and rigid design, see paragraphs 3.13 and 3.14 for specific information regarding flexible and rigid design including examples. The basic FAARFIELD design steps include:

Step 1: From Startup, create a new job and add the basic sections to analyze.

Step 2: From Structure, modify the pavement structure to be analyzed.

Step 3: From Airplane, add Airplane Load and Traffic Data.

Step 4: Return to Structure and Design Pavement Structure.

Step 5: Adjust Layer Thicknesses, Change Layer Types. Repeat Step 4.

Step 6: Select Life/Compaction, print out design report.

Step 7: Return to Startup and view pavement design report.

Step 8: Print pavement design report to be included in engineer's report.

Optional: Evaluate Life of Final Section to be Constructed. Disable automatic base design in options screen, return to structure screen and adjust layer thickness to match construction

3.12.6 Aircraft Traffic Considerations.

3.12.6.1 **Load.**

Pavements should be designed for the maximum anticipated takeoff weights of the airplanes in the fleet regularly operating on the section of pavement being designed. The design procedure generally assumes 95 percent of the gross weight is carried by the main landing gears and 5 percent is carried by the nose gear. FAARFIELD provides manufacturer-recommended gross operating weights and load distribution, for many civil and military airplanes. Using the maximum anticipated takeoff weight provides a conservative design allowing for changes in operational use and traffic, at airports where traffic regularly operates at less than maximum load. Where arrivals constitute 85% or greater of that runway's operations, and for high-speed exit taxiways, the use of aircraft landing weights for design is permitted.

3.12.6.2 **Landing Gear Type and Geometry.**

Gear type and configuration dictate how airplane weight is distributed to a pavement and how the pavement responds to airplane loadings. Refer to Order 5300.7, *Standard Naming Convention for Aircraft Landing Gear Configurations*, for standard gear designations.

3.12.6.3 **Tire Pressure.**

Tire pressure varies depending on gear configuration, gross weight, and tire size. In FAARFIELD, the tire pressure is linked to the gross weight. An increase in gross weight causes a proportional increase in tire pressure, such that the tire contact area is maintained constant. Tire pressure has a more significant influence on strains in the asphalt surface layer than at the subgrade. For flexible pavements constructed with a high stability asphalt, tire pressures up to 254 psi (1.75 MPa) may be accommodated. Tire pressure has a negligible impact on rigid pavement design.

3.12.6.4 **Aircraft Traffic Volume.**

Forecasts of annual departures by airplane type are needed for pavement design. In general, pavements should be designed to accommodate regularly using aircraft, where regular use is defined as at least 250 annual departures (500 operations). However, in some cases seasonal or other non-regular use aircraft may have significant impact on the pavement structure required. A sensitivity analysis is recommended to compare the structure needed to accommodate all planes in the fleet to the structure needed for all planes that have at least 250 annual departures. On federally funded projects when occasional or seasonal use aircraft are included in the traffic, documentation verifying actual activity (as opposed to planned) must be submitted with the sensitivity analysis to the local FAA region/ADO office as part of the engineer's report.

3.12.6.5 **Departure Traffic.**

Airfield pavements are generally designed considering only aircraft departures. This is because typically aircraft depart at a heavier weight than they arrive. If the aircraft arrive and depart at essentially the same weight, then the number of departures used for pavement design should be adjusted to reflect the number of times the pavement is loaded with each aircraft operation in the FAARFIELD pavement analysis.

3.12.6.6 **Total Departures Over Design Life.**

FAARFIELD evaluates the total number of departures over the design life period. For example, FAARFIELD considers 250 annual departures for a 20-year design life to be 5,000 total departures. Similarly, FAARFIELD considers 225 annual departures at a 1% annual growth rate to be 4,950 total departures.

3.12.6.7 **Airplane Traffic Mix.**

Nearly any traffic mix can be developed from the airplanes in the program library. The actual anticipated traffic mix must be used for the design analysis. Attempts to substitute equivalent aircraft for actual aircraft can lead to erroneous results.

3.12.6.8 **Total Cumulative Damage.**

FAARFIELD analyzes the damage to the pavement for each airplane and determines a final thickness for the total cumulative damage of all aircraft in the evaluation. FAARFIELD calculates the damaging effects of each airplane in the traffic mix based upon its gear spacing, load, and location of gear relative to the pavement centerline. Then the effects of all airplanes are summed under Miner's law. Since FAARFIELD considers where each airplane loads the pavement, the pavement damage associated with a particular airplane may be isolated from one or more of the other airplanes in the traffic mix. When the cumulative damage factor (CDF) sums to a value of 1.0, the structural design conditions have been satisfied.

3.12.7 Non-Aircraft Vehicles.

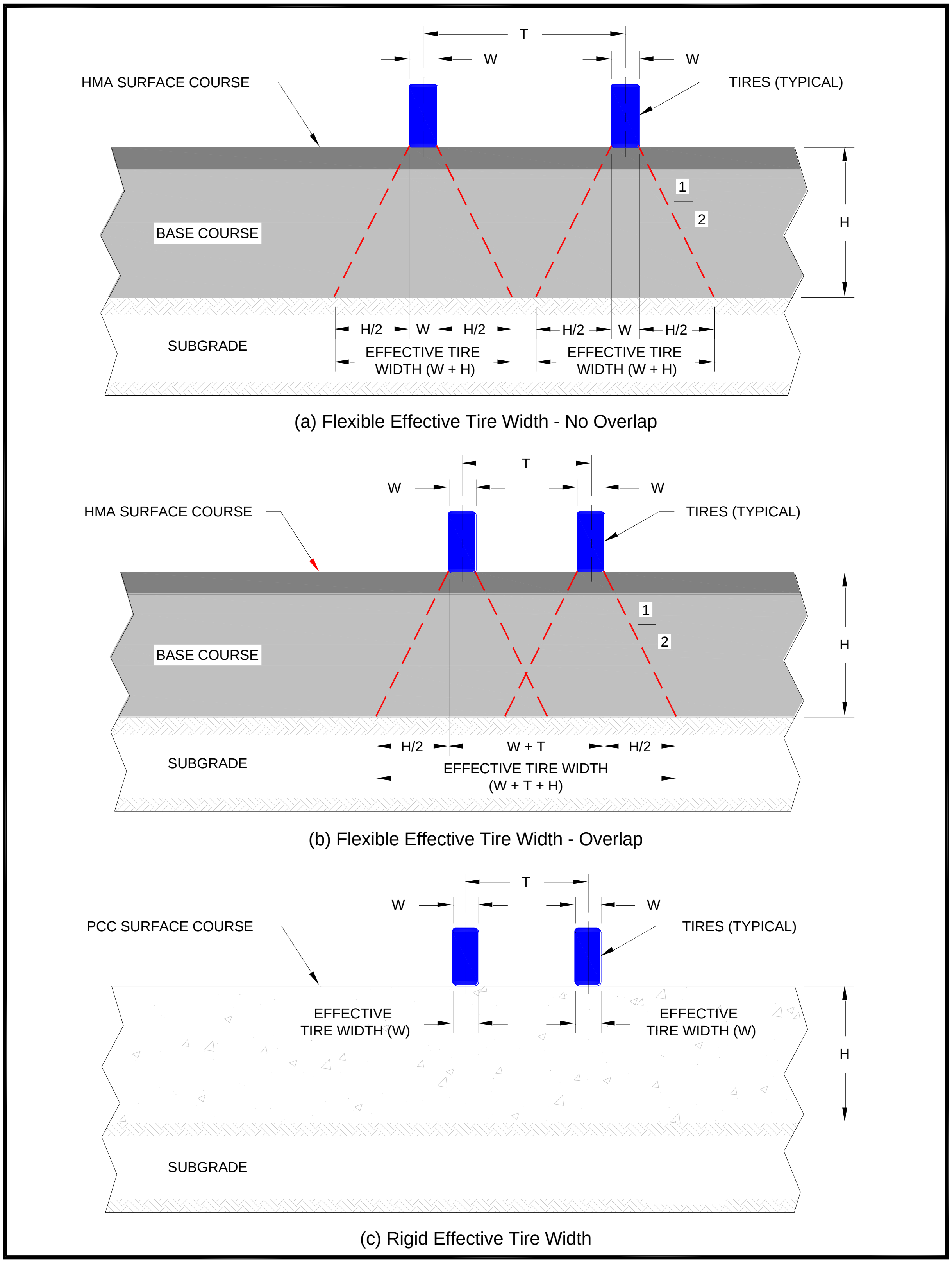
3.12.7.1 In some situations, non-aircraft vehicles such as aircraft rescue and firefighting, snow removal, or fueling equipment may place heavier wheel loads on the pavement than aircraft. FAARFIELD allows these types of vehicles to be included in the traffic mix. The "Non-Airplane Vehicles" airplane group includes several types of truck axles (single, dual, tandem, and dual-tandem) that may be used to represent common truck types. The included truck axles should be adequate for most light-duty pavement designs.

3.12.7.2 For small GA airports, it may be necessary to consider one or more of the following options: (1) limit the size of fuel trucks used for supply and refueling; (2) locate the fuel storage tanks in a location such that the trucks supplying fuel to the airport can access the storage tanks without entering the airfield; (3) strengthen the fuel truck access route; or (4) limit the size of maintenance vehicles (e.g., snow removal equipment).

3.12.8 Pass-to-Coverage Ratio.

An airplane seldom travels along a pavement section in a perfectly straight path or along the same path each time. This lateral movement is known as airplane wander and is modeled by a statistically normal distribution. As an airplane moves along a taxiway or runway, it may take several trips or passes along the pavement for a specific point on the pavement to receive a full-load application. The ratio of the number of passes required to apply one full load application to a unit area of the pavement is expressed by the pass-to-coverage (P/C) ratio. It is easy to observe the number of passes an airplane may make on a given pavement, but the number of coverages is mathematically derived internally in FAARFIELD. By definition, one coverage occurs when a unit area of the pavement experiences the maximum response (stress for rigid pavement, strain for flexible pavement) induced by a given airplane. For flexible pavements, coverages are a measure of the number of repetitions of the maximum strain occurring at the top of subgrade. For rigid pavements, coverages are a measure of repetitions of the maximum stress occurring at the bottom of the PCC layer (see Report No. FAA-RD-77-81, *Development of a Structural Design Procedure for Rigid Airport Pavements*). Coverages resulting from operations of a particular airplane type are a function of the number of airplane passes, the number and spacing of wheels on the airplane main landing gear, the width of the tire-contact area, and the lateral distribution of the wheel-paths relative to the pavement centerline or guideline markings (see Report No. FAA-RD-74-036, *Field Survey and Analysis of Aircraft Distribution on Airport Pavements*). In calculating the P/C ratio, FAARFIELD uses the concept of effective tire width. For flexible pavements, the effective tire width is defined at the top of the subgrade. “Response lines” are drawn at a 1:2 slope from the edges of the tire contact surface to the top of the subgrade, as illustrated in Figure 3-2. Tires are considered to be either separate or combined, depending on whether the response lines overlap. For rigid pavements, the effective tire width is defined at the surface of the pavement and is equal to a nominal tire contact surface width. All effective tire width and P/C ratio calculations are performed internally within the FAARFIELD program.

Figure 3-2. Effective Tire Width



3.12.9 Annual Departures.

3.12.9.1 Airport pavement design using FAARFIELD only considers departures and ignores the arrival traffic when determining the number of airplane passes. This is because in most cases airplanes arrive at an airport at a significantly lower weight than at takeoff due to fuel consumption. During touchdown, remaining lift on the wings and the landing gear shock absorber alleviates most of the dynamic vertical force that is transmitted to the pavement through the landing gears.

3.12.9.2 When arrival and departure weights are not significantly different or when the airplane must travel along the pavement more than once, it may be appropriate to adjust the number of annual departures used for thickness design to recognize that each departure results in multiple pavement loadings. For example, when an airplane is required to traffic a large part of the runway during the taxi movement (i.e., a runway with a central taxiway configuration), the airplane must travel along the same portion of the runway pavement twice during the take-off operation. In this case, it would be appropriate to double the number of departures in FAARFIELD. The pavement engineer must document all adjustments to traffic in the engineer's report.

3.12.10 Cumulative Damage Factor.

3.12.10.1 In FAARFIELD, fatigue failure is expressed in terms of a cumulative damage factor (CDF) using Miner's rule. CDF represents the amount of the structural fatigue life of a pavement that has been used up. It is expressed as the ratio of applied load repetitions to allowable load repetitions to failure. For a new pavement design, the pavement structure is adjusted until the cumulative CDF=1 for the traffic mix applied over the structural design life period being evaluated. For a single airplane and constant annual departures, CDF can be expressed by the following:

$$CDF = \frac{\text{number of applied load repetitions}}{\text{number of allowable repetitions to failure}}$$

or

$$CDF = \frac{(\text{annual departures}) \times (\text{life in years})}{\left(\text{pass}/\text{coverage ratio}\right) \times (\text{coverages to failure})}$$

or

$$CDF = \frac{\text{applied coverages}}{\text{coverages to failure}}$$

3.12.10.2 In the program implementation, CDF is calculated for each 10-inch (254-mm) wide strip along the pavement over a total width of 820 inches (20.8

m). Pass-to-coverage ratio is computed for each strip assuming that traffic is normally distributed laterally, and that 75 percent of passes fall within a “wander width” of 70 inches (1,778 mm). Statistically, this results in a normally distributed wander pattern with a standard deviation of 30.435 inches (773 mm). The CDF for design is taken to be the maximum CDF computed over all 82 strips. Even with the same gear geometry, airplanes with different main gear track widths will have different pass-to-coverage ratios in each of the 10-inch (254 mm) strips and may show little cumulative effect on the maximum CDF. Removing the airplanes with the lowest stress or strain may have little effect on the design thickness, depending on how close the gear tracks are to each other and the number of departures.

3.12.10.3 In FAARFIELD, the ‘CDF Graph’ function displays plots of CDF versus lateral effect for each gear in the design mix, as well as plot of cumulative CDF for all airplanes in the mix. For a completed design the peak value of cumulative CDF = 1.0. The following example illustrates the concept.

Given the following pavement structure:

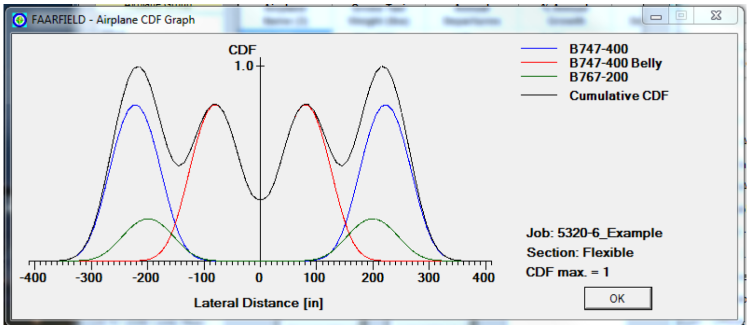
Thickness	**Pavement Structure**
4 inches	P-401 HMA Surface Course
8 inches	P-403 Stabilized Base Course
12 inches	P-209 Crushed Aggregate Base Course
10 inches	P-154 Aggregate Base Course
	Subgrade CBR 5 (7,500 psi Modulus)

Designed for the following airplane traffic:

Airplane	**Gross Weight (lbs)**	**Annual Departures**
B747-8	990,000	50
B747-8 Belly	990,000	50
B767-200	361,000	3000

3.12.10.4 To view the graph after the design is complete, return to the Aircraft screen and select CDF Graph. This provides a graph depicting the impact of each aircraft as well as the combined total contribution, showing that the critical location is between the main gear locations for the aircraft being evaluated. In this example even though the belly gear has a large contribution, it does not contribute the controlling damage to the pavement as shown in Figure 3-3.

Figure 3-3. CDF Contribution for Airplane Mix



3.12.11 FAARFIELD Material Properties.

3.12.11.1 In FAARFIELD pavement layers are assigned a thickness, elastic modulus, and Poisson's ratio. The same layer properties are used in flexible and rigid analysis. Layer thicknesses can be varied, subject to minimum thickness requirements. Poisson's ratio is fixed for all materials and the elastic moduli are either fixed or variable (within a permissible range) depending upon the material. Materials in FAARFIELD are identified by their corresponding specification designations as used in AC 150/5370-10; for example, crushed aggregate base course is identified as Item P-209. The list of materials also contains a user defined layer with variable properties that can be defined by the user. Table 3-2 lists the modulus values and Poisson's Ratios used in FAARFIELD.

3.12.11.2 In a rigid analysis, FAARFIELD requires a minimum of 3 layers (PCC Surface, base and subgrade) but allows up to a total of five (5) layers. A flexible design may have as few as 2 layers (HMA surface and subgrade), however an unlimited number of layers can be added.

3.12.11.3 On federally funded projects FAA standard materials as specified in AC 150/5370-10 must be used unless use of other materials has been approved by the FAA as a modification to standards (see FAA Order 5100.1). When analyzing existing sections, user defined layers may be the most accurate way to model performance of existing material. The designer should utilize a modulus that reflects the weakest in service strength of the existing material.

Table 3-2. Allowable Modulus Values and Poisson's Ratios Used in FAARFIELD

Layer Type	FAA Specified Layer	Rigid Pavement psi (MPa)	Flexible Pavement psi (MPa)	Poisson's Ratio
Surface	P-501 PCC	4,000,000 (30,000)	NA	0.15
	P-401/P-403/P-601 HMA	NA	200,000 (1,380)[1]	0.35
Stabilized Base and Subbase	P-401/P-403HMA	400,000 (3,000)		0.35
	P-306 Lean Concrete	700,000 (5,000)		0.20
	P-304 cement treated base	500,000 (3,500)		0.20
	P-301 soil cement	250,000 (1,700)		0.20
	Variable stabilized rigid	250,000 to 700,000 (1,700 to 5,000)	NA	0.20
	Variable stabilized flexible	NA	150,000 to 400,000 (1,000 to 3,000)	0.35
Granular Base and Subbase	P-209 crushed aggregate	Program Defined		0.35
	P-208, aggregate	Program Defined		0.35
	P-219, Recycled concrete aggregate	Program Defined		0.35
	P-211, Lime rock	Program Defined		0.35
	P-154 uncrushed aggregate	Program Defined		0.35
Subgrade	Subgrade	1,000 to 50,000 (7 to 350)		0.35
User-defined	User-defined layer	1,000 to 4,000,000 (7 to 30,000)		0.35

Notes:

1. A fixed modulus value for hot mix surfacing is set in the program at 200,000 psi (1380 MPa). This modulus value was conservatively chosen and corresponds to a pavement temperature of approximately 90°F (32°C).

3.12.12 Minimum Layer Thickness.

Table 3-3 and Table 3-4 establish minimum layer thicknesses for flexible and rigid pavements respectively, applicable to different airplane weight classes. Minimum thickness requirements are determined by the gross weight of the heaviest aircraft in the design traffic mix, regardless of the traffic level. FAARFIELD automatically establishes the minimum layer thickness requirements based on the traffic mix entered. However, the user should consult the applicable paragraphs of this AC and Table 3-3 and Table 3-4 to ensure that all minimum thickness requirements are met.

Table 3-3. Minimum Layer Thickness for Flexible Pavement Structures

Layer Type	FAA Specification Item	Maximum Airplane Gross Weight Operating on Pavement, lbs (kg)		
		<12,500 (5 670)	< 100,000 (45 360)	≥100,000 (45 360)
HMA Surface[1, 2,3]	P-401, Hot Mix Asphalt (HMA) Pavements	3 in. (75 mm)	4 in. (100 mm)	4 in. (100 mm)
Stabilized Base	P-401 or P-403; P-304; P-306[4]	Not Required	Not Required	5 in. (125 mm)
Crushed Aggregate Base[5,6]	P-209, Crushed Aggregate Base Course	3 in. (75 mm)	6 in. (150 mm)	6 in. (150 mm)
Aggregate Base[5,7,8]	P-208,Aggregate Base Course	3 in. (75 mm)	Not Used[7]	Not Used
Subbase[5,8]	P-154, Subbase Course	4 in. (100 mm)	4 in. (100 mm) (If required)	4 in. (100 mm) (if required)

Notes:

1. P-601-Fuel Resistant Hot Mix Asphalt may be used to replace the top 2 in (75 mm) of P-401 where a fuel resistant surface is needed; structurally, P-601 considered same as P-401.
2. Additional HMA surface above minimum typically in 0.5-inch (10-mm) increments.
3. P-403 may be used as surface course < 12,500 pounds (5,760 kg) or for HMA base or leveling course.
4. Use of P-306 requires FAA approval on federally funded projects to assure adequate measures taken to control potential for reflective cracking.
5. Use the larger of the thicknesses in this table or the thickness calculated by FAARFIELD rounded to the nearest 0.5 inch (10 mm). Additional thickness may be required for frost protection above minimums.
6. P-209, Crushed Aggregate Base Course, when used as a stabilized base course, is limited to pavements designed for gross loads of 100,000 pounds (45,360 kg) or less, except as noted in paragraph 3.6, Stabilized Base Course.
7. P-208, Aggregate Base Course, when used as a base course, is limited to pavements designed for gross loads of 60,000 pounds (27,220 kg) or less.
8. P-219 Recycled Concrete Aggregate Base Course may be used as an aggregate base or subbase. How P-219 will perform is related to the quality of the material it is made from combined with the method used to process it into an aggregate base.

Table 3-4. Minimum Layer Thickness for Rigid Pavement Structures

Layer Type	FAA Specification Item	Maximum Airplane Gross Weight Operating on Pavement, lbs (kg)		
		<12,500 (5,670)	< 100,000 (45,360)	≥ 100,000 (45,360)
PCC Surface	P-501, Portland Cement Concrete (PCC) Pavements	5 in. (125 mm)	6 in. (150 mm)[1]	6 in. (150 mm)[1]
Stabilized Base	P-401 or P-403; P-304; P-306	Not Required	Not Required	5 in. (125 mm)
Base	P-208, P-209, P-211, P-301	Not Required	6 in. (150 mm)[2]	6 in. (150 mm)
Subbase[3,4]	P-154, Subbase Course	4 in. (100 mm)	As needed for frost or to create working platform	As needed for frost or to create working platform

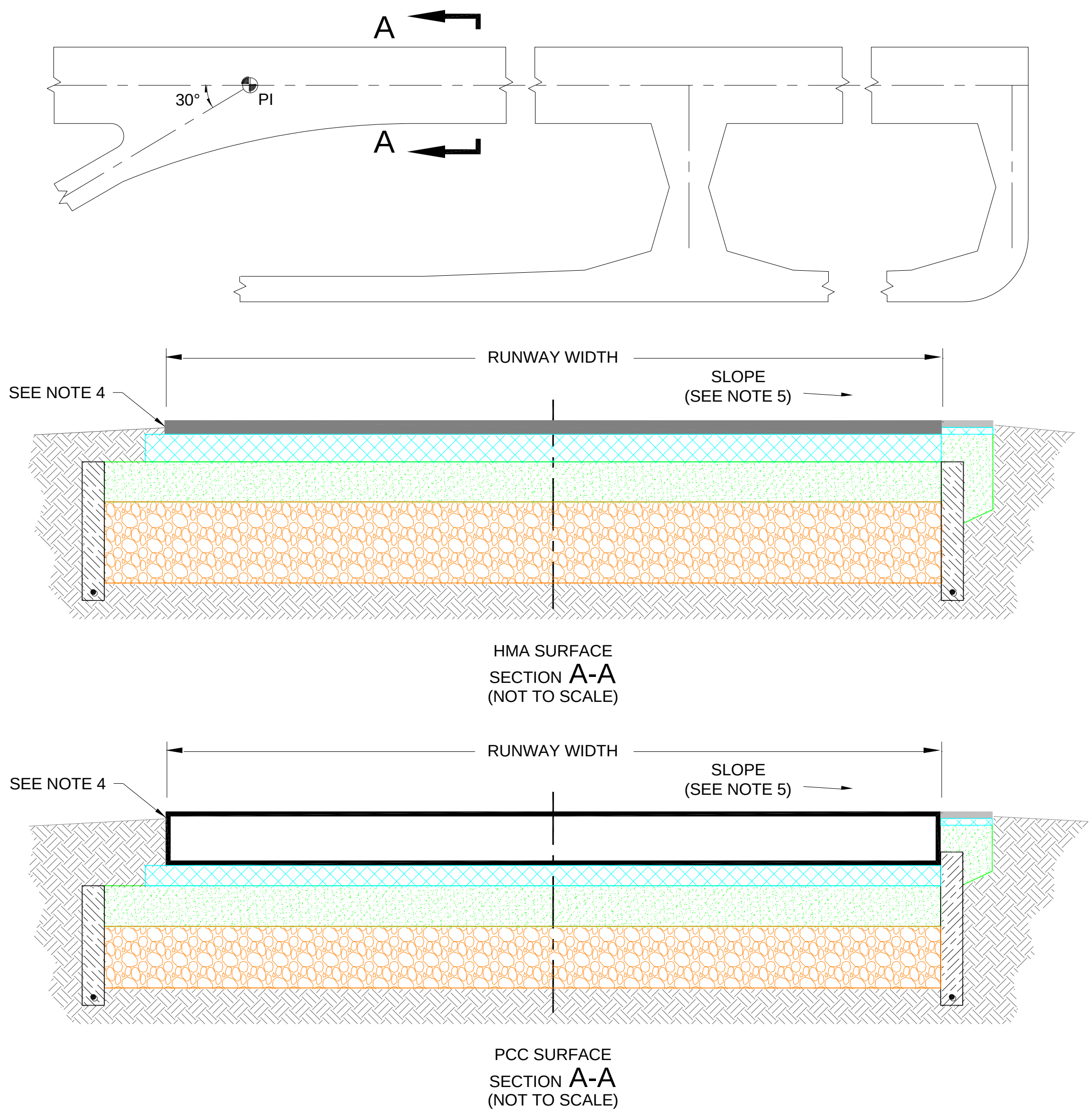
Notes:
1. FAARFIELD thickness to be rounded to the nearest 0.5 inch (10 mm).
2. For pavements for aircraft greater than 30,000 lbs (13,610 kg), base may be replaced with subbase.
3. Subbase layer is required for pavements designed for gross loads of 12,500 pounds (5,670 kg) or less only when the following soil types are present: OL, MH, CH, or OH.
4. The following specification items may also be used as subbase: P-208, Aggregate Base Course; P-209, Crushed Aggregate Base Course; P-211, Lime Rock Base Course; P-219 Recycled Concrete Aggregate Base Course; P-301, Soil-Cement Base Course. If more than one layer of subbase is used, each layer should meet the minimum thickness requirement in this table.

3.12.13 Typical Pavement Sections.

3.12.13.1 The FAA recommends uniform full width pavement sections, with each pavement layer constructed a uniform thickness for the full width of the pavement. See Figure 1-1. Typical Pavement Structure and Figure 3-4. Typical Plan and Sections for Pavements.

3.12.13.2 Since traffic on runways is distributed with majority of traffic in the center (keel) portion of the runway, the runways may be constructed with a transversely variable section. Variable sections permit a reduction in the quantity of materials required for the upper pavement layers of the runway. However, construction of variable sections may be more costly due to the complex construction associated with variable sections and this may negate any savings realized from reduced material quantities (see Appendix E).

Figure 3-4. Typical Plan and Sections for Pavements



NOTES:

1. RUNWAY, TAXIWAY AND SHOULDER WIDTHS; TRANSVERSE SLOPES, ETC. PER AC 150/ 5300-13, AIRPORT DESIGN
2. SURFACE, BASE, PCC, ETC. THICKNESS PER AC 150/5320-6.
3. BASE AND SUBBASE MINIMUM 12 INCHES [30 CM] UP TO 36 INCHES [90 CM] BEYOND FULL STRENGTH PAVEMENT.
4. CONSTRUCT A 1.5 INCH [4 CM] DROP BETWEEN PAVED AND UNPAVED SURFACES.
5. RUNWAY "CROWN" DEPICTION OMITTED FOR CLARITY.

LEGEND:

- HMA SURFACE
- PCC SURFACE
- SHOULDER
- STABILIZED BASE
- BASE
- SUB BASE
- SUB GRADE
- EDGE DRAIN

3.12.14 Frost and Permafrost Design.

The design of an airport pavement must consider the environmental conditions affecting the pavement during its construction and service life. In areas where frost and permafrost impact pavements, the pavement design should address the adverse effects of seasonal frost and permafrost. The maximum practical depth of frost protection provided is normally 72 inches (180 cm) below the top of the finished pavement. Frost considerations may result in thicker base or subbase courses than needed for structural support.

3.12.15 Seasonal Frost.

The adverse effects of seasonal frost are discussed in Chapter 2. Soil frost groups are described in Table 2-2. The design of pavements in seasonal frost areas can be based on either of two approaches: frost protection or reduced subgrade strength. The first approach is based on the control of pavement deformations resulting from frost action. Using this approach, the combined thickness of the pavement and non-frost-susceptible material must be sufficient to eliminate, or limit, the adverse effects of frost penetration into the subgrade. The second approach is based on providing adequate pavement load carrying capacity during the critical frost melting period and provide for the loss of load carrying capacity due to frost melting, ignoring the effects of frost heave. The procedures that address these design approaches are discussed below.

3.12.16 Complete Frost Protection.

3.12.16.1 Complete frost protection is accomplished by providing a sufficient thickness of pavement and non-frost-susceptible material to totally contain frost penetration within the pavement structure. The depth of frost penetration is determined by engineering analysis or by local codes and experience. The thickness of pavement required for structural support is compared with the computed depth of frost penetration. The difference between the pavement thickness required for structural support and the computed depth of frost penetration is made up with additional non-frost susceptible material.

3.12.16.2 Complete protection may involve removal and replacement of a considerable amount of subgrade material. Complete frost protection is the most effective method of providing frost protection. The complete frost protection method applies only to soils in FG-3 and FG-4, which are extremely variable in horizontal extent, characterized by very large, frequent, and abrupt changes in frost heave potential.

3.12.17 Limited Subgrade Frost Penetration.

The limited subgrade frost penetration method is based on engineering judgment and experience to control frost heave to an acceptable level of maintenance (less than 1 inch (250 mm) of frost heave). Frost is allowed to penetrate a limited amount into the underlying frost susceptible subgrade. Additional frost protection is required if the thickness of the non-frost susceptible structural section is less than 65 percent of the frost penetration. This method applies to soils in all frost groups when the functional requirements of the pavement permit a minor amount of frost heave.

3.12.18 Reduced Subgrade Strength.

3.12.18.1 The reduced subgrade strength method is based on providing a pavement with adequate load-carrying capacity during the frost melting period and does not address the effects of frost heave. To use the reduced subgrade strength method, the design assigns a subgrade strength rating to the pavement for the frost melting period.

3.12.18.2 This method applies to soils in FG-1, FG-2, and FG-3, which are uniform in horizontal extent or where the functional requirements of the pavement permit some degree of frost heave. Frost heave should be such that it does not impact safe operation of aircraft. The method may also be used for variable FG-1 through FG-3 soils for pavements subject to slow speed traffic where heave can be tolerated.

3.12.18.3 The required pavement thicknesses are determined using FAARFIELD, inputting the lower of the reduced subgrade strength value from Table 3-5 in lieu of the nominal subgrade CBR or k-value determined by testing. The pavement thicknesses established reflect the requirements for the weakened condition of the subgrade due to frost melting. The various soil frost groups, as defined in Chapter 2 should be assigned the lower of the strength ratings in Table 3-5 or that determined from geotechnical investigations.

Table 3-5. Reduced Subgrade Strength Ratings

Frost Group	Flexible Pavement CBR Value	Rigid Pavement *k*-value (pci)
FG-1	9	50
FG-2	7	40
FG-3	4	25
FG-4	Reduced Subgrade Strength Method Does Not Apply	

3.12.19 Permafrost.

The design of pavements in permafrost regions must consider the effects of seasonal thawing and refreezing, as well as the thermal effects of construction on the permafrost. Pavements can lead to thermal changes that may cause degradation of the permafrost resulting in severe differential settlements and drastic reduction of pavement load carrying capacity. Gravel-surfaced pavements are common in permafrost areas and generally provide satisfactory service. These pavements often exhibit considerable distortion but are easily regraded. Typical protection methods for permafrost may include complete protection, reduced subgrade strength, and insulated panels. In areas of permafrost, an experienced pavement/geotechnical engineer familiar with permafrost protection should design the pavement structure.

3.13 Flexible Pavement Design.

3.13.1 General

Flexible pavements consist of a HMA wearing surface placed on a base course and a subbase (if required) to protect the subgrade. In a flexible pavement structure, each pavement layer must protect its supporting layer. A typical pavement structure is shown in Figure 1-1 and Figure 3-4. Non-drained pervious granular layers must not be located between two impervious layers, which is referred to as sandwich construction. This is to prevent trapping water in the granular layer, which could result in a loss of pavement strength and performance.

3.13.2 Hot Mix Asphalt (HMA) Surfacing.

3.13.2.1 The HMA surface or wearing course limits the penetration of surface water into the base course, provides a smooth, skid resistant surface free from loose particles that could become foreign object debris (FOD), and resists the shearing stresses induced by airplane wheel loads. To meet these requirements the surface must be composed of a mixture of aggregates and asphalt binders which will produce a uniform surface of suitable texture possessing maximum stability and durability. A dense-graded HMA, such as Item P-401, meets these requirements.

3.13.2.2 Item P-401 is to be used as the surface course for pavements serving aircraft weighing more than 12,500 pounds (5,670 kg). Item P-403 may be used as a surface course for pavements serving aircraft weighing 12,500 pounds (5,670 kg) or less. See AC 150/5370-10, Items P-401 and P-403, for additional discussion on HMA pavement material specifications. See Table 3-3 for minimum requirements for HMA surface thickness.

3.13.2.3 In FAARFIELD, the HMA surface or overlay types have the same properties, with modulus fixed at 200,000 psi (1,380 MPa) and Poisson’s ratio fixed at 0.35. The HMA Overlay type can be placed over HMA or PCC surface types or User-Defined. Refer to Table 3-2 for material properties used in FAARFIELD.

3.13.2.4 A solvent-resistant surface (such as P-601) should be provided at areas subject to spillage of fuel, hydraulic fluid, or other solvents, such as airplane fueling positions and maintenance areas.

3.13.3 Base Course.

3.13.3.1 The base course distributes the imposed wheel loadings to the pavement subbase and/or subgrade. The best base course materials are composed of select, hard, and durable aggregates. The base course quality depends on material type and gradation, physical properties, and compaction. The quality and thickness of the base course must prevent failure in the support layers, withstand the stresses produced in the base, resist vertical pressures that may produce consolidation and distortion of the surface course, and resist volume changes caused by fluctuations in moisture content.

3.13.3.2 Base courses are classified as either stabilized or unstabilized. If aircraft in the design traffic mix have gross loads of 100,000 pounds (45,359 kg) or more then use of a stabilized base is required (see paragraph 3.6). AC 150/5370-10, *Standards for Specifying Construction of Airports*, includes the material specifications that can be used as base courses: stabilized (P-401, P-403, P-306, P-304) and unstabilized (P-209, P-208, P-219, P-211). The use of Item P-208 *Aggregate Base Course*, as base course is limited to pavements designed for gross loads of 60,000 pounds (27,200 kg) or less.

3.13.3.3 **Stabilized Base Course.**

FAARFIELD includes two types of stabilized layers, classified as stabilized (flexible) and stabilized (rigid). The two stabilized flexible base options are designated P-401/P-403 and Variable. The word flexible is used to indicate that these bases have a higher Poisson's ratio (0.35), act as flexible layers as opposed to rigid layers, and are less likely to crack. The standard FAA stabilized base is P-401/P-403, which has a fixed modulus of 400,000 psi (2,760 MPa). The variable stabilized flexible base can be used to characterize a stabilized base which does not conform to the properties of P-401/P-403. It has a variable modulus ranging from 150,000 to 400,000 psi (1,035 to 2,760 MPa). Stabilized (rigid) bases, P-304, and P-306 may also be used as base courses for flexible pavements. However, depending on the strength of the stabilized base material, the potential for reflective cracking must be considered and appropriate measures taken to control. Note: In AC 150/5370-10, Item P-304 and Item P-306 both contain limits on strength of concrete. On federally funded projects, FAA approval must be obtained before using P-306 as a base under flexible pavements. The properties of the various stabilized base layer types used in FAARFIELD are summarized in Table 3-2. Stabilized bases are offset 12 inches (300 mm) from the edge of the full strength pavement (see Figure 3-4).

3.13.3.4 **Aggregate Base Course.**

3.13.3.4.1 The standard aggregate base course for flexible pavement design is Item P-209, *Crushed Aggregate Base C*ourse. Item P-208, *Aggregate Base*

Course, may be used as a base for pavements accommodating aircraft fleets with all aircraft less than 60,000 pounds (27,200 kg) gross weight.

3.13.3.4.2 The modulus of non-stabilized layers is computed internally by FAARFIELD and the calculated modulus is dependent on the modulus of the underlying layer. Details on the sublayering procedure used by FAARFIELD may be found in the FAARFIELD help file.

3.13.3.4.3 Aggregate layers can be placed anywhere in the flexible pavement structure except at the surface or subgrade. The maximum number of aggregate layers that may be present in a structure is two, one of each type, and the crushed layer must be above the uncrushed layer.

3.13.3.4.4 Once the FAARFIELD design is complete, the modulus value displayed in the structure table for an aggregate layer is the average value of the sublayer modulus values. (Note: When a new P-209 crushed aggregate layer is created, the initial modulus value displayed is 75,000 psi (517 MPa). When a new P-154, uncrushed aggregate layer is created, the initial modulus value displayed is 40,000 psi (276 MPa). However, these initial default modulus values are not used in calculations.)

3.13.3.4.5 Compaction control for unstabilized base course material should be in accordance with ASTM D698 for areas designated for airplanes with gross weights of 60,000 pounds (27,200 kg) or less and ASTM D 1557 for areas designated for airplanes with gross weights greater than 60,000 pounds (27,200 kg).

3.13.3.5 **Minimum Base Course Thickness.**

FAARFIELD first computes the structural thickness of base required to protect a layer with a CBR of 20. FAARFIELD then compares it to the applicable minimum base thickness requirement from Table 3-3, and reports the thicker of the two values as the design base course thickness.

3.13.3.6 **Base Course Width.**

The base course may be offset 12 inches (300 mm) from the edge of the HMA surface.

3.13.4 Subbase.

3.13.4.1 A subbase is required as part of the flexible pavement structure on subgrades with a CBR value less than 20. The standard subbase layer (P-154) provides the equivalent bearing capacity of a subgrade with a CBR of 20. Subbases may be aggregate or treated aggregate. The minimum thickness of subbase is 4 inches (100 mm), see Table 3-3. Additional thickness may be required for practical construction limitations or if subbase is being utilized as non-frost susceptible material. The material requirements for subbase are not as strict as for the base course since the subbase is subjected to lower load intensities. Allowable subbase materials include P-154, P-210, P-212, P-213, and P-301. Use of items P-213 or P-301 as subbase course is not recommended in areas where frost penetration into the subbase is anticipated. Any material suitable for use as base course can also be used as subbase. AC 150/5370-10, *Standards for Specifying Construction of Airports*, covers the quality of material, methods of construction, and acceptance of material.

3.13.4.2 Compaction control for subbase material should be in accordance with ASTM D 698 for areas designated for airplanes with gross weights of 60,000 pounds (27,200 kg) or less and ASTM D1557 for areas designated for airplanes with gross weights greater than 60,000 pounds (27,200 kg).

3.13.5 Subgrade.

3.13.5.1 The ability of a particular soil to resist shear and deformation varies with its properties, density, and moisture content. Subgrade stresses decrease with depth, and the controlling subgrade stress is usually at the top of the subgrade.

3.13.5.2 Specification Item P-152, *Excavation, Subgrade, and Embankment*, covers the construction and density control of subgrade soils. Subgrade soils must be compacted sufficiently to ensure that the anticipated traffic loads will not cause additional consolidation of the subgrade.

3.13.5.3 In FAARFIELD, the subgrade thickness is assumed to be infinite and is characterized by either a modulus *(E)* or CBR value. Subgrade modulus values for flexible pavement design can be determined in a number of ways. The applicable procedure in most cases is to use available CBR values as calculated at in-service moisture content and allow FAARFIELD to compute the design elastic modulus using the following relationship:

$$E = 1500 \times CBR \text{ , } (E \text{ in psi})$$

3.13.5.4 It is also acceptable to enter the elastic modulus (*E*) directly into FAARFIELD. Flexible thickness design in FAARFIELD is sensitive to the strength of subgrade. For this reason, it is recommended to use a subgrade strength that reflects the in service strength. For guidance on determining the CBR value to use for design, refer to paragraph 2.5.6.

3.13.5.5 In cases where the top layer of subgrade is stabilized using a chemical stabilizing agent (lime, cement, fly ash, etc.) per paragraph 2.6.3, the properties of the top layer of subgrade will be different from those of the untreated subgrade below. To model this situation in FAARFIELD, the following procedure is recommended: Enter a user-defined layer immediately above the subgrade. Prior to designing the structure, change the design layer (indicated by the small arrow to the left of the cross section) to the layer immediately above this user-defined layer.

FAARFIELD will display a message that this is a nonstandard structure, since it includes a user-defined layer. The user must select the modulus of the user-defined layer. It is recommended to choose a modulus equal to 1500 × CBR (in psi) or 10 × CBR (in MPa), where the design CBR is one standard deviation below the laboratory CBR average for the stabilized material. The thickness of the user-defined material should be equal to the depth of field stabilization. The CBR entered for the subgrade (lowest layer) should be equal to the estimated CBR of the natural (unstabilized) subgrade.

3.13.6 FAARFIELD Flexible Pavement Design Failure Mode.

The design process for flexible pavement considers two failure modes: vertical strain in the subgrade and horizontal strain in the asphalt layer. Limiting vertical strain in the subgrade guards against failure by subgrade rutting, and limiting horizontal strain at the bottom of the asphalt layer guards against pavement failure initiated by cracking of the asphalt layer. For the horizontal strain mode, FAARFIELD considers horizontal strain in all asphalt layers in the structure, including asphalt stabilized base layers and asphalt overlays. By default, FAARFIELD computes only the vertical subgrade strain for flexible pavement thickness design. However, the user has the option of enabling the asphalt strain computation by selecting the “HMA CDF” checkbox in the FAARFIELD options screen. In most cases the thickness design is governed by the subgrade strain criterion however it is good engineering practice to perform the asphalt strain check for the final design.

3.13.7 Flexible Design Example.

The design of a pavement structure is an iterative process in FAARFIELD. Enter the pavement structure and airplane traffic to be applied to the section. FAARFIELD then evaluates the minimum pavement layer requirements and adjusts the pavement layer thicknesses to give a predicted structural life equal to the design structural life. This example follows the steps as outlined in paragraph 3.12.5.

Step (1): From ‘Startup Screen’ create new job, and add basic section(s) from sample sections to be analyzed.

Step (2a): For this example, assume the following starting pavement structure and airplane traffic:

Starting pavement structure:

Thickness	Pavement Structure
4 inches	P-401 HMA Surface Course
5 inches	P-401/P-403 Stabilized Base Course
6 inches	P-209 Crushed Aggregate Base Course

Thickness	Pavement Structure
12 inches	P-154 Aggregate Base Course
	Subgrade, CBR=5 (E = 7500 psi)

Airplane traffic:

Airplane	Gross Weight (lbs)	Annual Departures
B737-800	174,700	3000
A321-200 opt	207,014	2500
EMB-195 STD	107,916	4500
Regional Jet – 700	72,500	3500

Step (2b): The pavement structure to be used for the design is entered by going to the STRUCTURE screen by clicking on the 'STRUCTURE' button (see Figure 3-5) and modifying the existing structure to match proposed pavement section by selecting the 'Modify Structure' button (see Figure 3-6). Layers can then be added by selecting the 'Add/Delete Layer' button. Layer types can be changed by 'clicking' on the layer material and thickness of the layer can be adjusted by clicking on the layer thickness. To change the layer modulus for a layer, click on the layer modulus. A dialog box will display the allowable range of values, and have a field where the new value can be entered. If the modulus is not user-changeable, the dialog box displays a message that the value is fixed by FAARFIELD. When done making adjustments select 'End Modify' button.

Figure 3-5. Flexible Design Example Step 1

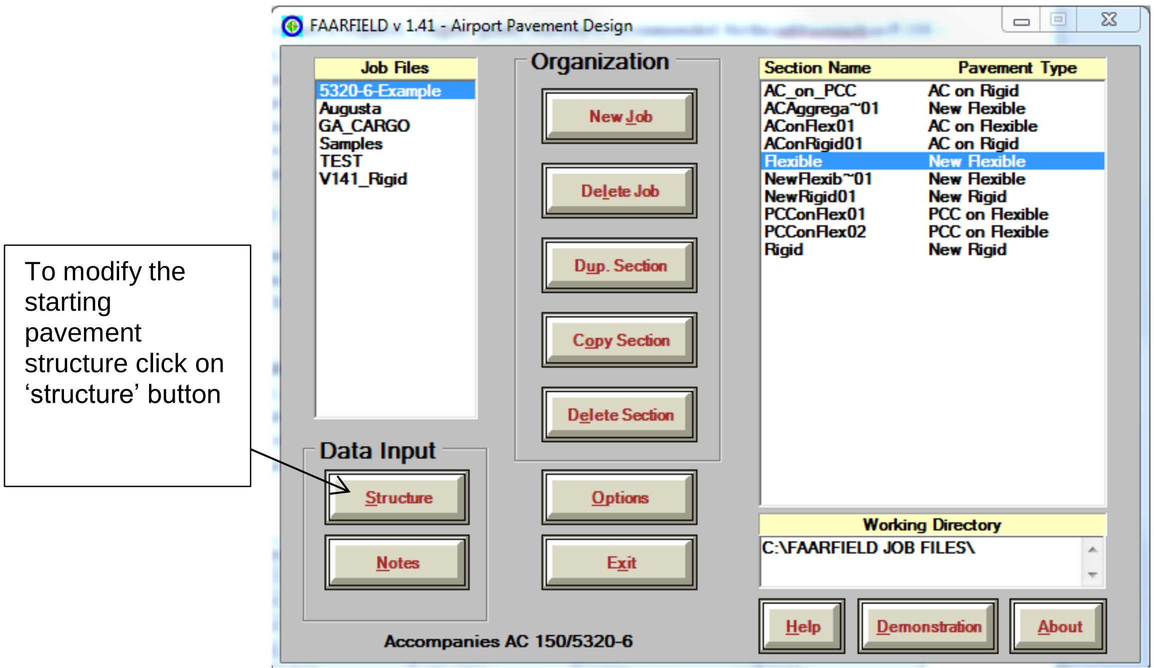
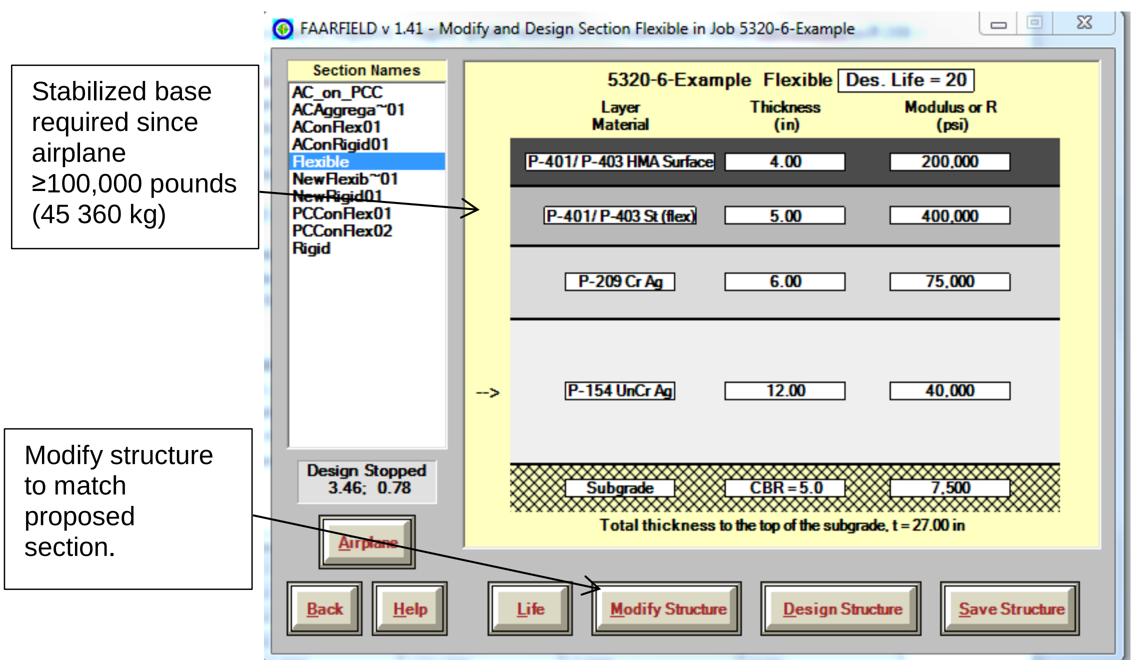


Figure 3-6. Flexible Design Example Step 2



Step (3): The design traffic to be applied to the pavement is entered by going to the Airplane screen by selecting the 'Airplane' button (see Figure 3-7). Airplanes are selected from the airplane library at the left of the screen (see Figure 3-8). For each airplane selected, the following data may be adjusted: gross taxi weight, annual departures, and percent annual growth. Airplanes are organized by group based upon airplane manufacturer. In addition there is a group of generic airplanes based upon type and size of airplane gear. In many cases specific airplane models not in the airplane library can be adequately represented by a generic airplane. After entering all of the airplanes, you return to the structure screen by selecting the 'back' button (see Figure 3-8).

Figure 3-7. Flexible Design Example Step 4

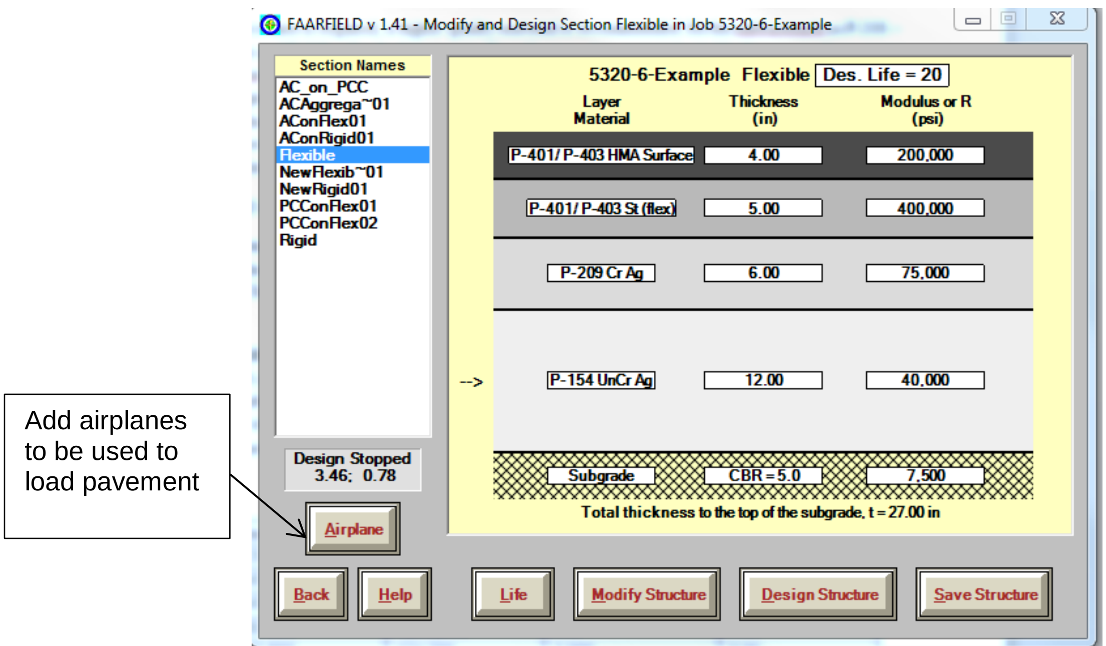
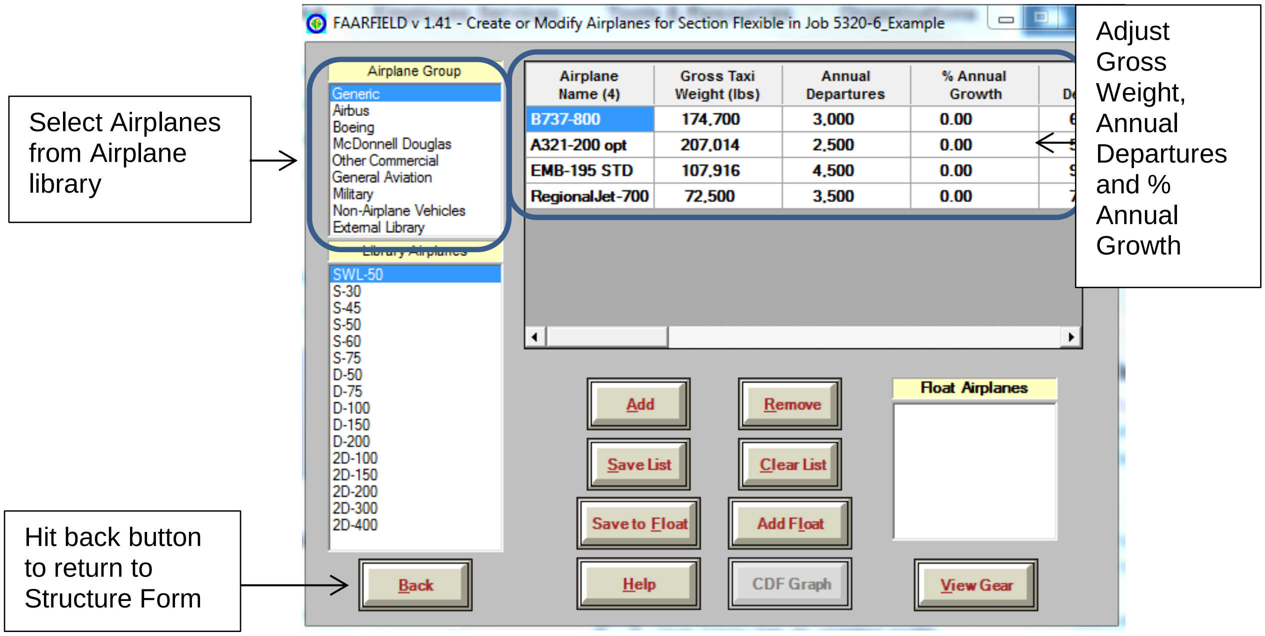


Figure 3-8. Flexible Design Example Step 4b



Step (4): Select 'Design Structure' button to start design (see Figure 3-9). During the design process, FAARFIELD checks the P-209 subbase thickness, assuming that the underlying layer has a CBR of 20. In this example, the thickness of P-209 required to protect the layer with a CBR of 20 is 6.1 inches, which is greater than the 6 inch minimum allowable thickness for a P-209 layer from Table 3-3. Next, FAARFIELD designs the thickness of the P-154 aggregate subbase layer. The layer being iterated on by FAARFIELD (the design layer) is indicated by the small black arrow on the left. The results of the completed design are shown in Figure 3-10.

Figure 3-9. Flexible Design Example Step 6 Perform Design Analysis

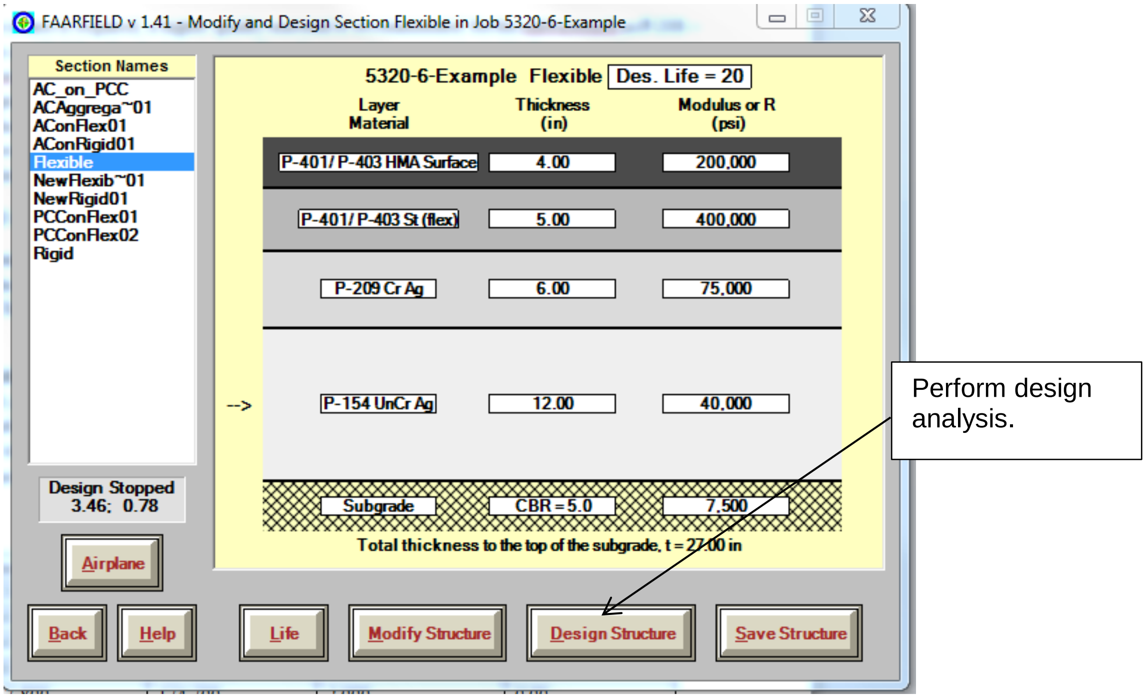
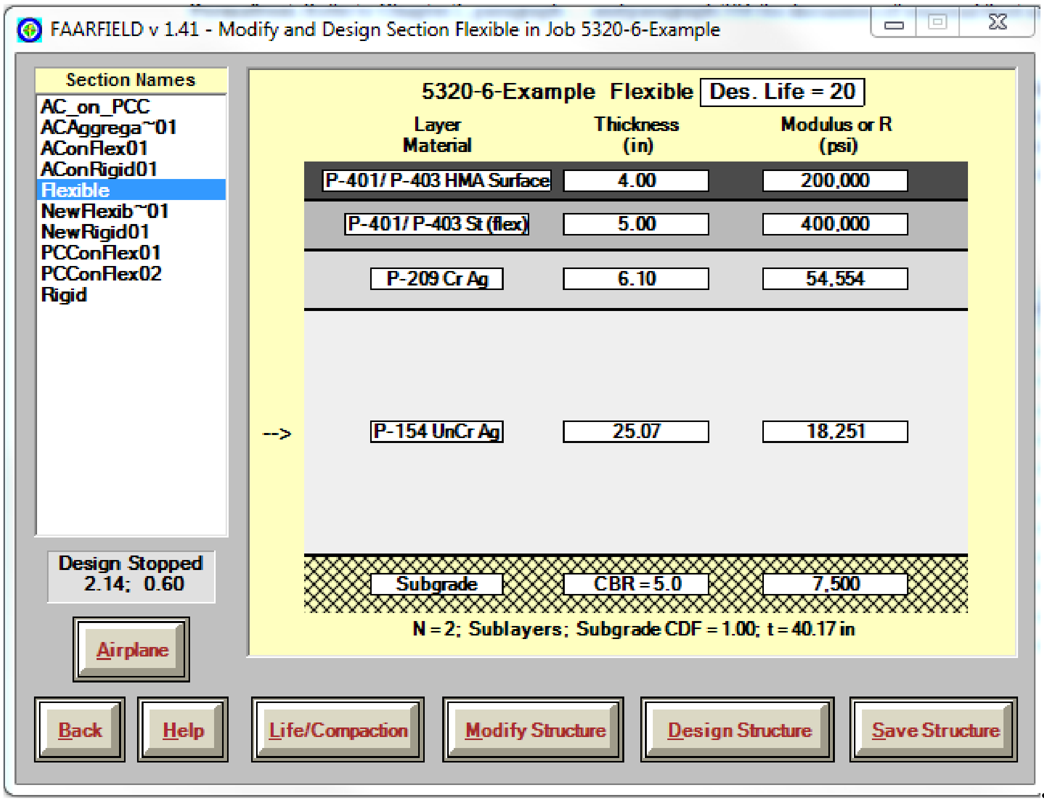


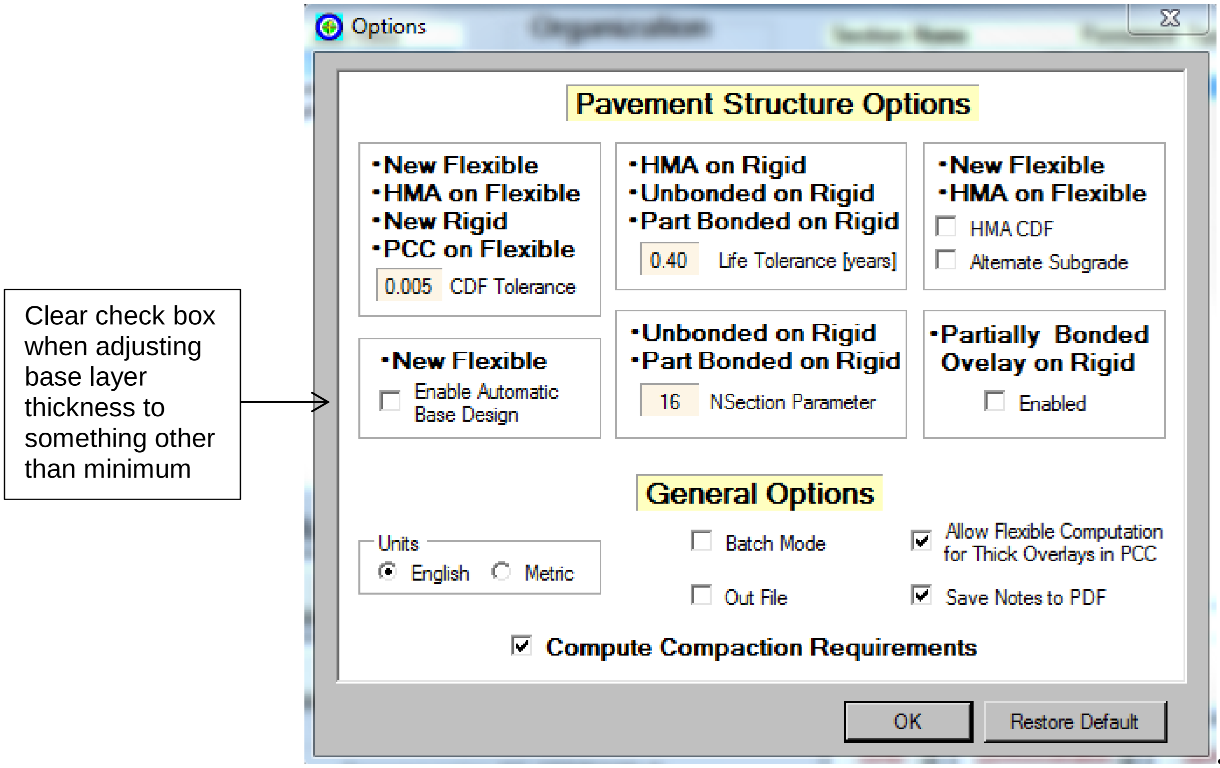
Figure 3-10. Flexible Design Example Step 6 Results



Step (5): To design the final (adjusted) structure:

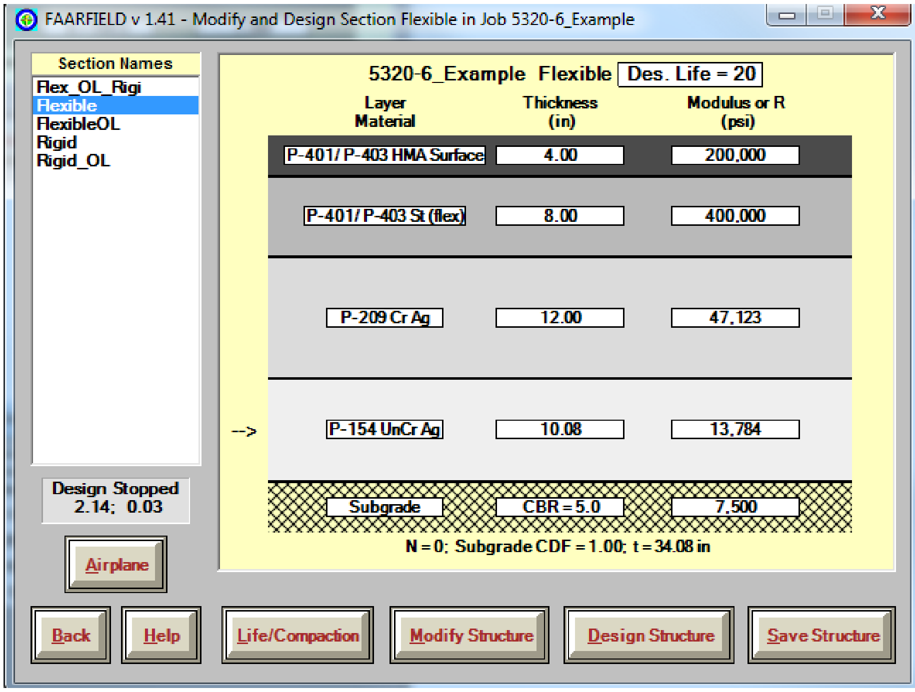
- Turn off automatic base design by clearing check box 'Enable Automatic Base Design' in the options screen. To navigate to the options screen (Figure 3-11) select the 'back button' followed by selecting the 'Options' button.
- Close the options screen by selecting 'ok' button
- Return to the structure screen by selecting the 'Structure' button.
- Select the 'modify structure' button
- Adjust the layers (surface, stabilized base and base) to reflect the final thickness to be constructed; when layer adjustments are complete select 'End Modify' button. For example, consider the following pavement structure, which meets minimum layer thickness requirements, and is proposed to be constructed: 4 inches P-401, 8 inches P-403, 12 inches P-209 and 10 inches P-154.
- Select the 'Design Structure' button to complete the final analysis.

Figure 3-11. Flexible Design Example Options Screen



Note: The final thickness design check for this example is shown in Figure 3-12. Note it indicates that 10.08 inches of subbase is needed, which would be rounded to nearest 0.5 inch or 10 inches.

Figure 3-12. Flexible Design Example Final Pavement Structure



Step (6): FAARFIELD v1.4 includes the ability to evaluate the depth of subgrade compaction required. After completing your design, select the ‘Life/Compaction’ button. The design report then includes a subgrade compaction table for Non-Cohesive and Cohesive subgrade. Check that the “compute compaction requirements” is selected on the Options Screen (Figure 3-8). A detailed example of how to generate the FAARFIELD compaction tables is given in paragraph 3.13.8. (Note: The compaction function will not be available if the design has not been completed, or if the 'Compute Compaction Requirements' option has not been selected in the Options screen. In this case, the button will just read 'Life.')

Step (7/ 8): The Airport Pavement Design report (Figure 3-13) is automatically saved to a file named as follows: [*job* name]_[*section* name].pdf.into the same working directory that you designated for your FAARFIELD job files. The report can also be viewed from the startup screen by selecting ‘Notes’ button. The design report summarizes the Pavement Structure, Airplane Traffic and the CDF contribution of each aircraft evaluated.

Figure 3-13. Airport Pavement Design Report

FAARFIELD

FAARFIELD v 1.41 - Airport Pavement Design

Section Flexible in Job 5320-6_Example.

Working directory is C:\FAARFIELD JOB FILES\

The structure is New Flexible. Asphalt CDF = 0.2329.

Design Life = 20 years.

A design for this section was completed on 11/16/15 at 08:27:44.

Pavement Structure Information by Layer, Top First

No.	Type	Thickness in	Modulus psi	Poisson's Ratio	Strength R,psi
1	P-401/ P-403 HMA Surface	4.00	200,000	0.35	0
2	P-401/ P-403 St (flex)	8.00	400,000	0.35	0
3	P-209 Cr Ag	12.00	47,123	0.35	0
4	P-154 UnCr Ag	10.08	13,784	0.35	0
5	Subgrade	0.00	7,500	0.35	0

Total thickness to the top of the subgrade = 34.08 in

Airplane Information

No.	Name	Gross Wt. lbs	Annual Departures	% Annual Growth
1	B737-800	174,700	3,000	0.00
2	A321-200 opt	207,014	2,500	0.00
3	EMB-195 STD	107,916	4,500	0.00
4	RegionalJet-700	72,500	3,500	0.00

Additional Airplane Information

Subgrade CDF

No.	Name	CDF Contribution	CDF Max for Airplane	P/C Ratio
1	B737-800	0.03	0.03	1.23
2	A321-200 opt	0.97	0.97	1.20
3	EMB-195 STD	0.00	0.00	1.24
4	RegionalJet-700	0.00	0.00	1.41

HMA CDF

No.	Name	CDF Contribution	CDF Max for Airplane	P/C Ratio
1	B737-800	0.00	0.00	2.69
2	A321-200 opt	0.00	0.00	2.65
3	EMB-195 STD	0.00	0.00	2.91

Figure 3-13. Airport Pavement Design Report (continued)

FAARFIELD

FAARFIELD v 1.41 - Airport Pavement Design

Section Flexible in Job 5320-6_Example.
Working directory is C:\FAARFIELD JOB FILES\

The structure is New Flexible. Asphalt CDF = 0.2667.
Design Life = 20 years.
A design for this section was completed on 09/04/15 at 13:43:50.

Pavement Structure Information by Layer, Top First

No.	Type	Thickness in	Modulus psi	Poisson's Ratio	Strength R,psi
1	P-401/ P-403 HMA Surface	4.00	200,000	0.35	0
2	P-401/ P-403 St (flex)	8.00	400,000	0.35	0
3	P-209 Cr Ag	12.00	47,121	0.35	0
4	P-154 UnCr Ag	10.08	13,783	0.35	0
5	Subgrade	0.00	7,500	0.35	0

Total thickness to the top of the subgrade = 34.08 in

Airplane Information

No.	Name	Gross Wt. lbs	Annual Departures	% Annual Growth
1	B737-800	174,700	3,000	0.00
2	A321-200 opt	207,014	2,500	0.00
3	EMB-195 STD	107,916	4,500	0.00
4	RegionalJet-700	72,500	3,500	0.00

Additional Airplane Information

Subgrade CDF

No.	Name	CDF Contribution	CDF Max for Airplane	P/C Ratio
1	B737-800	0.03	0.03	1.23
2	A321-200 opt	0.97	0.97	1.20
3	EMB-195 STD	0.00	0.00	1.24
4	RegionalJet-700	0.00	0.00	1.41

HMA CDF

No.	Name	CDF Contribution	CDF Max for Airplane	P/C Ratio
1	B737-800	0.00	0.00	2.69
2	A321-200 opt	0.00	0.00	2.65
3	EMB-195 STD	0.00	0.00	2.91
4	RegionalJet-700	0.00	0.00	3.23

P-401/P-403 St (flex) CDF

No.	Name	CDF Contribution	CDF Max for Airplane	P/C Ratio
1	B737-800	0.09	0.11	1.84
2	A321-200 opt	0.16	0.16	1.83
3	EMB-195 STD	0.01	0.02	1.94
4	RegionalJet-700	0.00	0.01	2.00

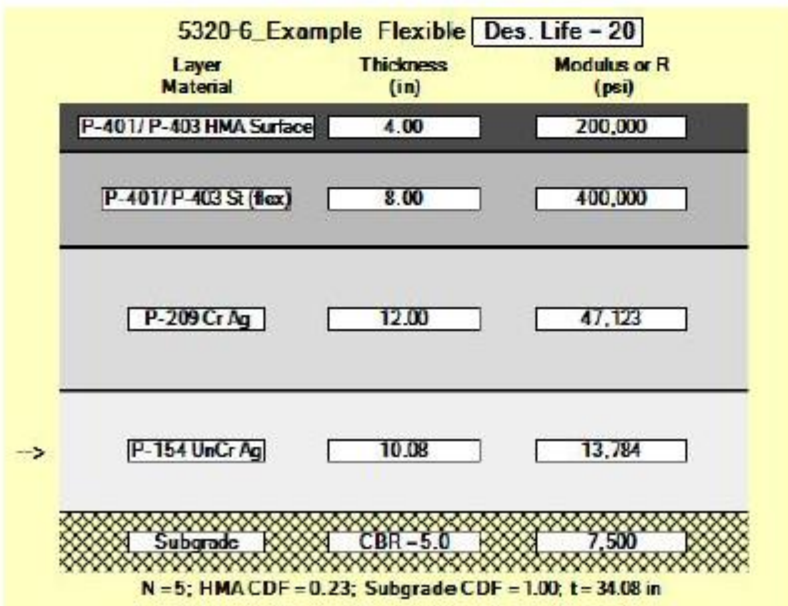
Figure 3-13. Airport Pavement Design Report (continued)

4	RegionalJet-700	0.00	0.00	3.23

P-401/P-403 St (flex) CDF

No.	Name	CDF Contribution	CDF Max for Airplane	P/C Ratio
1	B737-800	0.08	0.09	1.84
2	A321-200 opt	0.13	0.14	1.83
3	EMB-195 STD	0.02	0.02	1.94
4	RegionalJet-700	0.00	0.01	2.00

User is responsible for checking frost protection requirements.



Note: Perform a final check for failure by fatigue cracking in the asphalt layers by selecting the "HMA CDF" checkbox in the Options screen (see Figure 3-8). In example shown in Figure 3-13, the subgrade strain controls and the CDF in the HMA is only 0.23.

3.13.8 Detailed Example FAARFIELD Compaction Table.

1. An apron extension is to be built to accommodate the following airplane mix:

Airplane mix:

Airplane	Gross Weight (lbs)	Annual Departures
B737-800	174,700	3000
A321-200 opt	207,014	2500
EMB-195 STD	107,916	4500
Regional Jet – 700	72,500	3500

2. A soils investigation has shown the subgrade will be cohesive, with a design CBR of 5. In-place densities of the soils have been determined at even foot increments below the ground surface in accordance with Chapter 2.

3. Depths and densities are tabulated as follows:

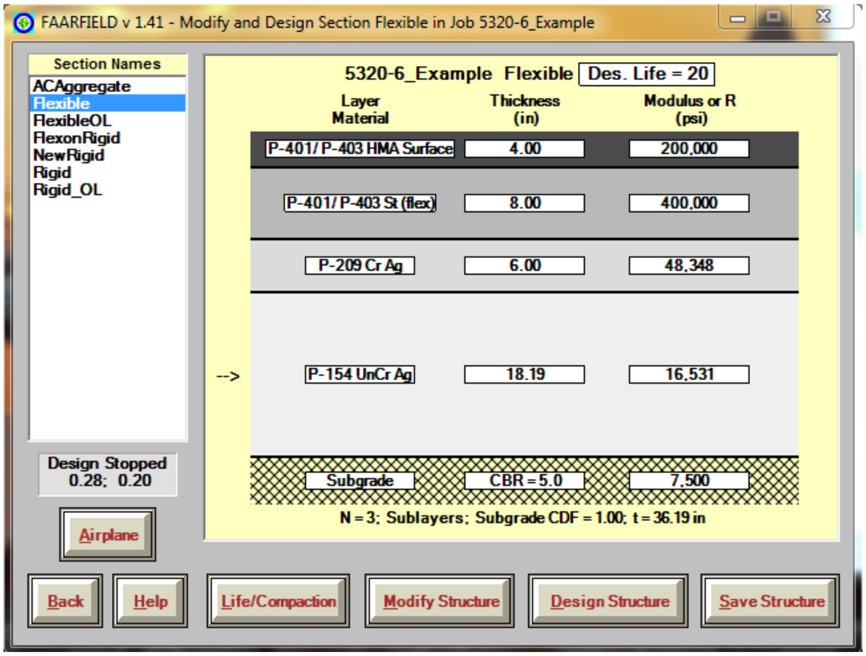
Depths and densities:

Depth Below Existing Grade	In-Place Density[1]
1ft (0.3 m)	75%
2 ft (0.6 m)	89%
3 ft (0.9 m)	91%
4 ft (1.2 m)	95%
5 ft (1.5 m)	96%

Note: In-place densities determined in accordance with ASTM D 1557 since aircraft mix includes aircraft greater than 60,000 pounds (27,200 kg) per paragraph 2.4.2.

4. The FAARFIELD flexible pavement thickness design results in the following pavement structure (Figure 3-14): 4 inches P-401 / 8 inches P-403 / 6 inches P-209 / 18 inches P-154 for a total thickness of 36 inches above the subgrade.

Figure 3-14. FAARFIELD Pavement Structure for Compaction Example



5. After completing the thickness design, ensure that the "Compute Compaction Requirements" check box is checked on the FAARFIELD Options screen.

6. From the FAARFIELD Structure screen "Life/compaction." Compaction requirements for the section will then be displayed on the FAARFIELD "Design Information" screen under Notes. For this example, the computed compaction requirements for cohesive soils are shown in Table 3-6. For this example assume that the top of the subgrade will be 30 inches below the top of the existing subgrade. Since the existing density at this level is greater than the compaction requirements calculated by FAARFIELD, for this example the existing subgrade density is adequate.

Table 3-6. Computed Compaction Requirements for the Sample Section

Cohesive Soil

Percent Maximum Dry Density(%)	Depth of compaction from pavement surface (in)	Depth of compaction from top of subgrade (in)	Critical Airplane for Compaction
95	0 - 22	--	A321-200 opt
90	22 - 38	0 - 4	A321-200 opt
85	38 - 60	4 - 26	A321-200 opt
80	60 - 83	26 - 49	A321-200 opt

Note: Due to different aircraft gear configurations it is possible that there may be a different critical airplane at each density level.

7. Note: The specific compaction requirements above apply only to the particular set of design and traffic data used for this example. Compaction requirements will differ depending on the design CBR or E-value, soil type, and design pavement thickness, as well as the traffic mix.

3.14 Rigid Pavement Design.

3.14.1 General.

3.14.1.1 Rigid pavements for airports are composed of PCC placed on a granular or stabilized base course supported on a compacted subgrade. See Figure 1-1 for typical pavement structure.

3.14.1.2 The FAARFIELD design process currently considers only one mode of failure for rigid pavement, bottom up cracking of the concrete slab. Cracking is controlled by limiting the horizontal stress at the bottom of the PCC slab and does not consider failure of subbase and subgrade layers. FAARFIELD iterates on the concrete layer thickness until the CDF reaches a value of 1.0 which satisfies the design conditions. When the minimum PCC layer thickness controls, FAARFIELD will not reduce the PCC thickness below the minimum thickness of 6 inches (150 mm), or 5 inches (125 mm) if all aircraft are less than 12,500 pounds (5,760 kg) gross weight. If minimum thickness is reached the design process will abort with a CDF < 1.0 and the design report will indicate "Minimum layer thickness was reached."

3.14.1.3 A three-dimensional finite element model is used to compute the edge stresses in concrete slabs. The model has the advantage of considering where the critical stresses for slab design occur. Critical stresses normally occur at slab edges, but may be located at the center of the slab with certain aircraft gear configurations. FAARFIELD uses LEAF to compute interior stress and takes the larger of 95% of the interior or 3D-FEM computed edge stress (reduced by 25 percent) as the design stress.

Note: FAARFIELD does not consider functional pavement design issues such as the need for additional material for frost protection and permafrost. Seasonal frost and permafrost effects are discussed in Chapter 2.

3.14.2 Concrete Surface Layer.

The concrete surface must provide a nonskid texture, prevent the infiltration of surface water into the subgrade, and provide structural support for airplane gears. The quality of the concrete, acceptance and control tests, methods of construction and handling, and quality of workmanship are covered in Item P-501 Portland Cement Concrete Pavement. See AC 150/5370-10, Item P-501 for additional discussion regarding PCC specifications. See Table 3-4 for minimum PCC surface thicknesses. The modulus value for concrete is fixed in FAARFIELD at 4,000,000 psi (27,580 MPa) and Poisson's ratio is set at 0.15, see Table 3-2.

3.14.3 Base / Subbase Layers.

3.14.3.1 The base layer provides a uniform, stable support for the rigid pavement slabs. Refer to Table 3-4 for minimum base thicknesses required under rigid pavements. Stabilized base is required for base under pavements designed to serve airplanes over 100,000 pounds, see paragraph 3.6. Two layers of base may be used, e.g. P-306 over a layer of P-209. Layering must be done in such a way as to avoid producing a sandwich (granular layer between two stabilized layers) section or a weaker layer over a stronger layer. Subbase may be substituted for base under rigid pavements designed to serve airplanes weighing 30,000 pounds (13,610 kg) or less. Subbase may be used as: a base under rigid pavement; for frost protection; or as a substitution for unsuitable subgrade material. The following materials are acceptable for use under rigid pavements: stabilized base (P-401, P-403, P-306, P-304) and unstabilized base/subbase (P-209, P-208, P-219, P-211, P-154).

3.14.3.2 The first base layer directly under the PCC surface must be offset 12 to 36 inches from the edge of the PCC layer. Subsequent base or subbase layers beneath the base layer should be offset 12 inches from the edge of the layer immediately above.

3.14.3.3 Up to three base/subbase layers can be added to the pavement structure in FAARFIELD for new rigid pavement design. The layer thickness must be entered for each base/subbase layer. For standard base/subbase materials, the modulus and Poisson’s ratio are internally set and cannot be changed by the user. When using the variable stabilized or user-defined layers, the modulus value can be input directly. If a variable layer or user-defined layer is used, a warning will appear in the Structure Screen stating that a ‘non-standard’ material has been selected and its use in the pavement structure will require FAA approval. Refer to Table 3-4 for minimum subbase layer thicknesses.

3.14.4 Subgrade: Determination of Modulus (E Value) for Rigid Pavement Subgrade.

3.14.4.1 In addition to the soils survey and analysis and classification of subgrade conditions, the determination of the foundation modulus is required for rigid pavement design. The foundation modulus is assigned to the subgrade layer; i.e., the layer below all structural layers. The foundation modulus can be expressed as the modulus of subgrade reaction, *k*, or as the elastic (Young’s) modulus *E*. The subgrade modulus can be input into FAARFIELD directly in either form; however, all structural computations are performed using the elastic modulus *E*. If the foundation modulus is input as a *k*-value it is automatically converted to the equivalent *E* value using the following equation:

$$E_{SG} = 20.15 \times k^{1.284}$$

where:

E_{SG}	=	Elastic modulus (E-modulus) of the subgrade, psi
k	=	Modulus of Subgrade Reaction of the subgrade, pci

The following formula can be used to convert CBR to an approximate *k*-value for the subgrade:

$$k = 28.6926 \times \text{CBR}^{0.7788}, \text{(k, pci)}$$

The pavement engineer must refer to the project geotechnical report for the subgrade strength to be used for the pavement design. See paragraph 2.5, Soil Strength Tests.

3.14.4.2 For existing pavements the *E* modulus can be determined in the field from non-destructive testing (NDT). Generally, a heavy-falling weight deflectometer (HWD) is used on airfields. See Appendix C, Nondestructive Testing (NDT) Using Falling Weight Type Impulse Load Devices, or AC 150/5370-11, *Use of Nondestructive Testing in the Evaluation of Airport Pavements*.

3.14.5 Frost Effects.

Frost protection should be provided for rigid pavements in areas where conditions conducive to detrimental frost action exist. PCC slabs <9 in (230 mm) are more susceptible to cracking from frost heave. This is generally most pronounced at the boundary between marked and unmarked areas on a runway, e.g. adjacent to the fixed distance marking. For PCC slabs < 9 in (230 mm), if complete frost protection is not provided, reinforcement with embedded steel no less than 0.050 percent steel in both directions should be provided for slabs on the runway which include large areas of markings, (e.g., threshold bars, runway designation and fixed distance markings) and for those slabs immediately adjacent to the markings. Refer to paragraph 2.7 for guidance on the determination of the depth of frost protection required. Local experience may be used to refine the calculations of depth of protection required.

3.14.6 FAARFIELD Calculation of Concrete Slab Thickness.

3.14.6.1 FAARFIELD utilizes a three-dimensional finite element model to compute the edge stresses in concrete slabs. The model has the advantage of considering where the critical stresses for slab design occur. Critical stresses normally occur at slab edges, but may be located at the center of the slab with certain aircraft gear configurations. FAARFIELD uses LEAF to compute interior stress and takes the larger of 95% of the interior or 3D-FEM computed edge stress (reduced by 25 percent accounting for load transfer) as the design stress.

3.14.6.2 FAARFIELD calculates the slab thickness based on the assumption that the airplane gear induces a maximum stress on the bottom surface of the slab. Loads that induce top-down cracks (such as corner loads) are not considered for design. The maximum design stress may be caused by airplane gear loading on the interior or the edge of the slab. The airplane gear may be positioned either parallel or perpendicular to the slab edge to determine the maximum edge stress.

3.14.6.3 FAARFIELD does not calculate the thickness of layers other than the PCC slab in rigid pavement structures, but will enforce the minimum thickness requirements for all layers as shown in Table 3-4 to assure the minimum thickness requirements are met.

3.14.6.4 FAARFIELD requires design input data from the following five areas: design life (years), concrete flexural strength (psi), structural layer data (type and thickness), subgrade modulus (*k* or *E*), and airplane traffic mix (type, weight, frequency). For thicknesses greater than the minimum, the pavement thickness should be rounded to nearest 0.5 inch (1 cm).

3.14.7 Concrete Flexural Strength.

3.14.7.1 The required concrete pavement thickness is related to the strength of the concrete. For pavement design, the strength of the concrete is characterized by the flexural strength since the primary action and failure mode of a concrete pavement is in flexure. Concrete flexural strength is measured in accordance with the ASTM C 78, *Standard Test Method for Flexural Strength of Concrete.*

3.14.7.2 When establishing the flexural strength for the thickness design the designer should consider the capability of the industry in a particular area to produce concrete at a particular strength and the need to avoid high cement contents, which may have a negative effect on concrete durability. In addition, high cement contents may lead to increased alkali content which may exacerbate alkali-silica reactivity issues in the concrete mixture.

3.14.7.3 A design flexural strength between 600 and 750 psi (4.14 to 5.17 MPa) is recommended for most airfield applications. Design strengths outside of this range must be approved by the FAA. In general, design flexural strengths higher than 750 psi (5.17 MPa) should be avoided, unless it can be shown that higher strength mixes are produced by normal methods using local materials, i.e., without relying on excessive cement contents or additives likely to negatively impact durability. The strength used in thickness design is different than the strength used for material acceptance in P-501. The acceptance strength in P-501 should reflect the strength needed to ensure the actual (in-service) strength meets or exceeds the strength used in the FAARFIELD thickness design. Item P-501 typically uses a 28-day strength as a practical construction acceptance measure. However, the long-term strength achieved by the concrete is normally expected to be at least 5 percent more than the strength measured at 28 days.

3.14.8 Jointing of Concrete Pavements.

Variations in temperature and moisture content can cause volume changes and slab warping which may cause significant stresses. Use joints to divide the pavement into a series of slabs of predetermined dimension to reduce the detrimental effects of these stresses and to minimize random cracking. Slabs should be as nearly square as possible when no embedded steel is used. Refer to table 3-9 for recommended Maximum Joint Spacing. Note, joint spacing is controlled by slab thickness; it is not intended to imply that you can establish slab thickness based upon joint spacing.

3.14.9 Joint Type Categories and Details

3.14.9.1 **Joint Type Categories.**

Pavement joints are categorized according to the function that the joint is intended to perform. The categories of joints are isolation, contraction, and construction joints, as described in Table 3-7 and below. All joints should be finished in a manner that permits the joint to be sealed. Pavement joint details are shown in figures 3-15a, 3-15b, and 3-16. Longitudinal joints should be designed to minimize pavement width changes. All longitudinal construction joints should be doweled joints, unless the joint also serves as an isolation joint. For aprons, undoweled construction joints are acceptable for intermediate longitudinal joints unless the joint is within 20 feet (6 m) of a free edge. For narrow (75 ft (20 m) or less) taxiway pavements less than 9 inches (225 mm) thick on unstabilized granular bases, it is acceptable to create a ‘tension ring’. This is done by using tied longitudinal contraction joints and tied transverse contraction joints for the last three transverse joints from the end.,

Taxiway pavements greater than 9 inches (225 mm) require doweled intermediate longitudinal contraction joints adjacent to a free edge, as well as doweled joints for the last three transverse joints from a free edge.

3.14.9.2 **Isolation Joints (Types A, A-1).**

Isolation joints are used when conditions preclude load transfer across the joint by dowels or aggregate interlock. Isolation joints are only needed where the pavement abuts a structure or to isolate intersecting pavements where differences in direction of movement of the pavements may occur, e.g., between a connecting taxiway and a runway. Type A joints are created by increasing the thickness of the pavement along the edge of the slab (see Figure 3-15a). This thickened edge will accommodate the load that would be transferred with dowels or aggregate interlock in contraction and construction joints. Type A-1 joints are reinforced to provide equivalent load carrying capacity as a thickened edge, and may only be used for PCC pavements greater than 9 inches (228 mm). The joint between the runway and connecting, crossover, and exit taxiways are locations where the Type A-1 joint may be considered. See Appendix D, *Reinforced Isolation Joint,* for detail and example Type A-1 Isolation Joint.

3.14.9.3 **Contraction Joints (Types B, C, D).**

Contraction joints provide controlled cracking of the pavement when the pavement contracts due to a decrease in moisture content or a temperature drop. Contraction joints also decrease stresses caused by slab warping. Details for contraction joints are shown as Types B, C, and D in Figure 3-15b. Details for joint sealant are shown in Figure 3-15c.

3.14.9.4 **Construction Joints (Types E and F).**

Construction joints are required when two abutting slabs are placed at different times, such as at the end of a day's placement or between paving lanes. For pavements serving airplanes 30,000 pounds (13,610 kg) or greater, use Type E construction joints. Type F butt joints may be used for pavements serving airplanes less than 30,000 pounds gross weight, constructed on a stabilized base. Details for construction joints are shown in Figure 3-15b.

Table 3-7. Pavement Joint Types

Type	Description	Longitudinal	Transverse
A	Thickened Edge Isolation Joint	Use at: -Pavement Intersections -Free edge that is location of future expansion -edge of structures	Use at: -pavement feature intersections when the pavement intersects at an angle. -free edge that is location of future expansion , -where pavement abuts a structure.
A-1	Reinforced Isolation Joint	For PCC slabs > 9 in (230 mm). Use at: -Pavement Intersections -Free edge that is location of future expansion - edge of structures	For PCC slabs > 9 in (230 mm). Use at: -Pavement Intersections -Free edge that is location of future expansion - edge of structures

Type	Description	Longitudinal	Transverse
B	Hinged Contraction Joint	Longitudinal contraction joint in slabs < 9 in. (230 mm) thick; longitudinal contraction joints located 20ft (6m) or less from the pavement free edge in slabs < 9 in. (230 mm) thick	Not used except for slabs < 9” when using ‘tension ring’
C	Doweled Contraction Joint	For use in longitudinal contraction joints 20 ft (6 m) or less from free edge in slabs > 9 in. (230 mm) thick	Use on the last three joints from a free edge, and for three joints on either side of isolation joints. Use at other locations with FAA approval.
D	Dummy Contraction Joint	For all other contraction joints in pavement.	For all other contraction joints in pavement.
E	Doweled Construction Joint	All construction joints excluding isolation joints.	Use for construction joints at all locations separating successive paving operations (“headers”).
F	Butt Construction Joint	All construction joints for pavements serving airplanes less than 30,000 lbs (13,610 kg) on a stabilized base.	All construction joints for pavements serving airplanes less than 30,000 lbs (13,610 kg) on a stabilized base.

Figure 3-15a. Rigid Pavement Isolation Joint

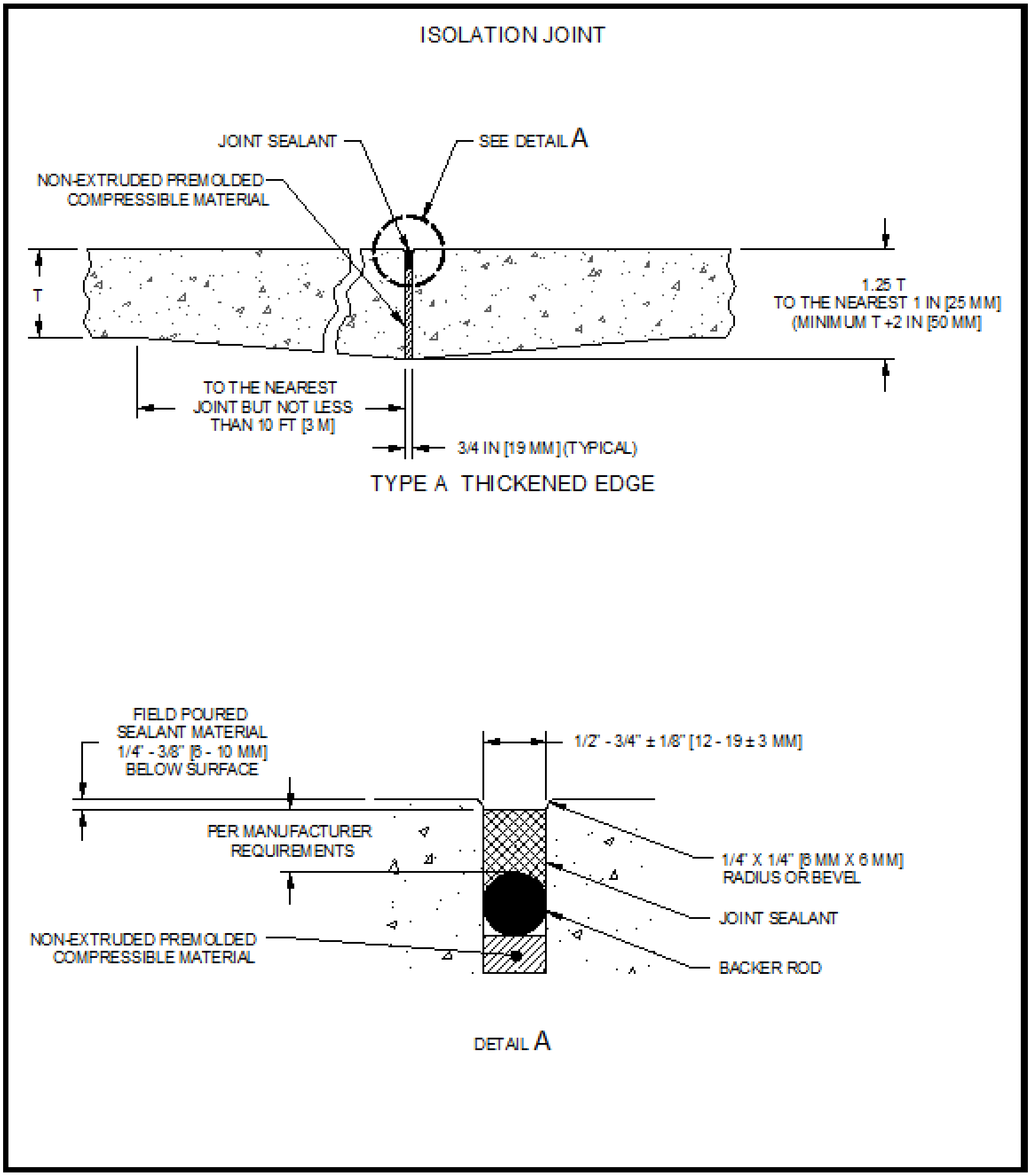


Figure 3-15b. Rigid Pavement Contraction and Construction Joints

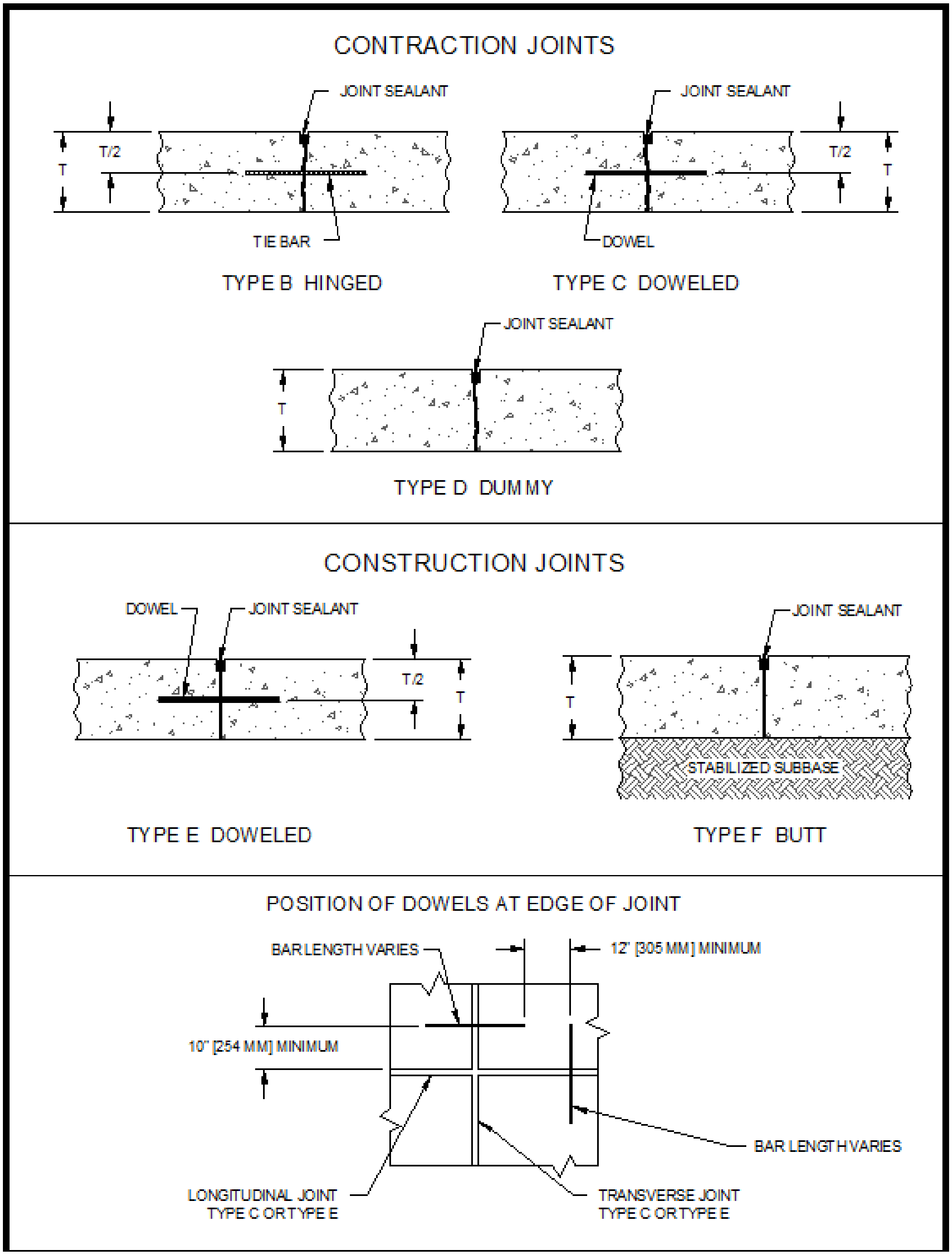
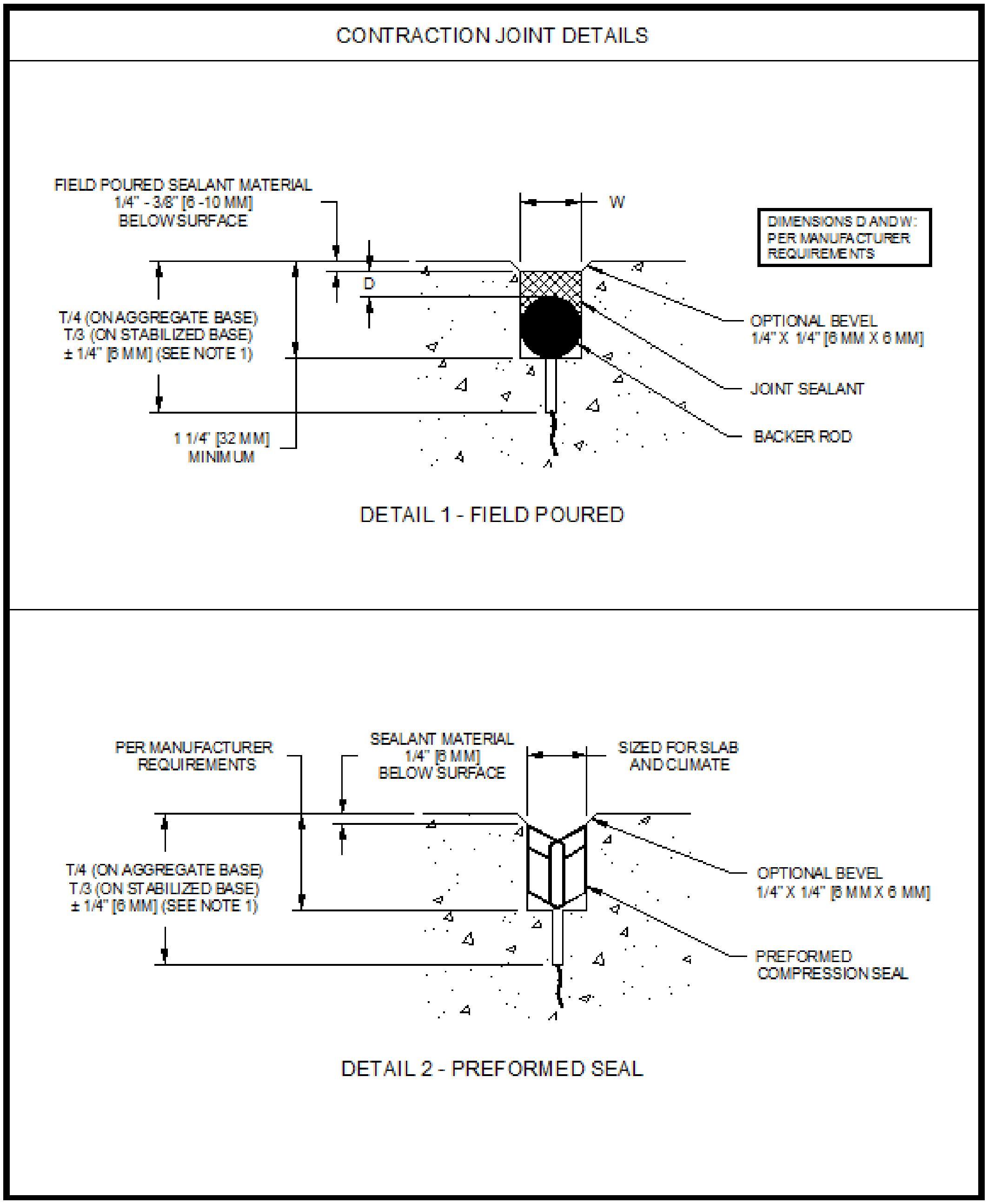


Figure 3-15c. Rigid Pavement Joint Sealant Details



Notes:

1. Initial saw cut T/6 to T/5 (minimum 1 in. (25 mm) when using early entry saw.
2. Field Poured Sealant reservoir sized to provide proper shape factor, depth (D): width (W) base upon sealant manufacturer requirements. Typically hot pour sealants require a 1:1 shape factor and silicon sealants a 1:2 shape factor, for individual projects refer to sealant manufacturer recommendations.
3. Hold all sealants down 3/8” on grooved RW.

3.14.10 Dowels and Tie Bars for Joints.

3.14.10.1 **Tie Bars.**

For slabs less than or equal to 9 inches (225 mm), longitudinal contraction joints within 20 feet (6 m) of a free edge must be tied to hold the slab faces in close contact. In this case the tie bars do not act as load transfer devices, but prevent opening of the joint, facilitating load transfer by aggregate interlock. Tie bars should be deformed bars conforming to the specifications given in Item P-501. For slabs less than or equal to 6 inches (150 mm), tie bars must be 20 inch long (510 mm), No.4 bars spaced at 36 inches (900 mm) on center For slabs greater than 6 inches or greater (150 mm), tie bars must be 30 inch long (762 mm), No. 5 bars spaced at 30 inches on center. Do not use tie bars to create continuous tied joints greater than 75 feet (23 m).

3.14.10.2 **Dowels.**

Dowels provide load transfer across the joint and prevent relative vertical displacement of adjacent slab ends. If installed properly dowels permit relative movement of adjacent slabs. For all slab thicknesses, dowels must be provided for three transverse joints from a free edge. FAA approval is required to use dowels at other transverse joints.

3.14.10.2.1 Size Length and Spacing of Dowels.

Dowels are sized to resist the shearing and bending stresses produced by the loads on the pavement. Dowel length and spacing should be sufficient to prevent failure of the concrete slab due to the bearing stresses exerted on the concrete. Table 3-8 gives dowel dimensions and spacing for various pavement thicknesses.

3.14.10.2.2 Dowel Positioning.

The alignment and elevation of dowels is important to ensure the performance of a joint. Transverse dowels will require the use of a fixture, usually a wire cage or basket firmly anchored to the base, to hold the dowels in position. Alternatively, a paving machine equipped with an automated dowel bar inserter may be used for placing dowels in the transverse joint.

Table 3-8. Dimensions and Spacing of Steel Dowels

Thickness of Slab	Diameter	Length	Spacing
6-7 in (152-178 mm)	¾ in (20 mm)	18 in (460 mm)	12 in (305 mm)
7.5-12 in (191-305 mm)	1 in (25 mm)	18 in (460 mm)	12 in (305 mm)
12.5-16 in (318-406 mm)	1 ¼ in (30 mm)	20 in (510 mm)	15 in (380 mm)
16.5-20 in (419-508 mm)	1 ½ in (40 mm)	20 in (510 mm)	18 in (460 mm)
20.5-24 in (521-610 mm)	2 in (50 mm)	24 in (610 mm)	18 in (460 mm)

3.14.11 Joint Sealants and Fillers.

Sealants are used in all joints to prevent the ingress of water and foreign material into the joint. In isolation joints premolded compressible filler is used to accommodate movement of the slabs, and sealant is applied above the filler to prevent infiltration of water and foreign material. The depth (D) and width (W) of the joint sealant reservoir is a function of the type of sealant material used. For joint sealants to perform as intended, the joint sealant material and reservoir be must constructed in accordance with the joint sealant manufacturer's recommendations for that type of sealant. For example typically hot pour sealants perform best with a 1:1 D/W ratio, where silicone sealants perform best with a 1:2 D/W ratio. See Figure 3-15c for typical joint reservoir details. Backer rod material must be compatible with the type of sealant used and sized to provide the desired shape factor. Specifications for joint sealants are given in Item P-605, *Joint Sealants for Concrete Pavements*, and Item P-604, *Compression Joint Seals for Concrete Pavements.*

3.14.12 Joint Layout and Spacing.

3.14.12.1 Pavement joint layout requires the selection of the proper joint type(s), spacing, and dimensions to ensure the joints perform their intended function. Construction considerations are also important in determining the joint layout pattern. Generally, it is more economical to keep the number and width of paving lanes to a minimum. Keep the slab width (w) to length (l) ratio no greater than 1:1.25. Paving lane widths and location of in-pavement light fixtures will affect joint spacing and layout. Joints should be placed with respect to light fixtures in accordance with AC 150/5340-30, *Design and Installation Details for Airport Visual Aids*. Innovative Pavement Research Foundation (IPRF) Report 01-G-002-03-01 *Constructing In-pavement Lighting, Portland Cement Pavement* includes sample details for the installation of in pavement lights. In addition, Figure 3-16 shows a typical jointing plan for a runway end, parallel taxiway, and connector. Figure 3-17 shows a typical jointing plan for pavement for a 75-foot (23-m) wide runway. For sample PCC Joint plans, see http://www.faa.gov/airports/engineering/pavement_design/.

3.14.12.2 When designing joint layouts for intersections, consider the following for isolation joints, odd-shaped slabs, and slabs with structures or other embedments.

1. **Isolation Joints.** Intersecting pavements, such as a taxiway and runway, should be isolated to allow the pavements to move independently. In addition at locations where it is necessary to change the joint pattern, isolation joints are required. Isolation can be accomplished by using a Type A isolation joint between the two pavements where the two pavements meet. The isolation joint should be positioned to allow the two pavements to move independently of each other.

2. **Odd-Shaped Slabs and Slabs with Structures or Other Embedments.** Because cracks tend to form in odd-shaped slabs and in slabs with structures and other embedments, it is good practice to maintain sections that are nearly square or rectangular in shape. Where odd-shaped slabs or slabs with structures cannot be avoided, embedded steel is required. In slabs where the length-to-width ratio exceeds 1.25, or in slabs that are not rectangular in shape, the slabs must be reinforced. The embedded steel should consist of no less than 0.050 percent of the gross cross-sectional area of the slab in both directions. In addition embedded steel must be placed around the perimeter of embedded structures. Although steel does not prevent cracking, it helps keep the cracks that do form tightly closed. The interlock of the irregular faces of the cracked slab provides structural integrity of the slab maintaining pavement performance. In addition, by holding the cracks tightly closed, the steel minimizes the infiltration of debris into the cracks. The embedded steel should be placed in accordance with the recommendations given below.

3. The thickness of pavements with crack control steel is the same as for plain concrete pavement.

4. Steel may be either bar mats or welded wire fabric installed with end and side laps to provide steel throughout the slab. Longitudinal members should be not less than 4 inches (100 mm) or more than 12 inches (305 mm) apart; transverse members should be not less than 4 inches (100 mm) or more than 24 inches (610 mm) apart. End laps should be a minimum of 12 inches (305 mm) but not less than 30 times the diameter of the longitudinal bar or wire. Side laps should be a minimum of 6 inches (150 mm) but not less than 20 times the diameter of the transverse bar or wire. End and side clearances should be a maximum of 6 inches (150 mm) and a minimum of 2 inches (50 mm). Steel should be placed approximately in the middle of the slab.

3.14.13 Joint Spacing.

3.14.13.1 Joint spacing is impacted by many factors including: total width and thickness of pavement to be constructed, location and size of in-pavement objects, type of aggregates used in the concrete, range of temperatures that pavement is exposed to, base restraint as well as warping stresses. Shorter joint spacing generally provides better long-term in-service performance. See Table 3-9 for recommended maximum joint spacing.

3.14.13.2 **Without Stabilized Base.**

Shorter spacing may be required to provide minimum clearance between pavement joints and in-pavement objects such as light bases. On federally funded projects exceeding the spacing as shown in Table 3-9 requires FAA approval.

3.14.13.3 **With Stabilized Base.**

Rigid pavements supported on stabilized base are subject to higher warping and curling stresses than those supported on unstabilized base. A maximum spacing of 20 feet (6.1 m) is recommended for slabs equal to or thicker than 16 inches (406 mm). On federally funded projects exceeding the spacing as shown in Table 3-9 requires FAA approval.

Figure 3-16. Typical Joint Layout Pattern for Runway, Parallel Taxiway and Connector

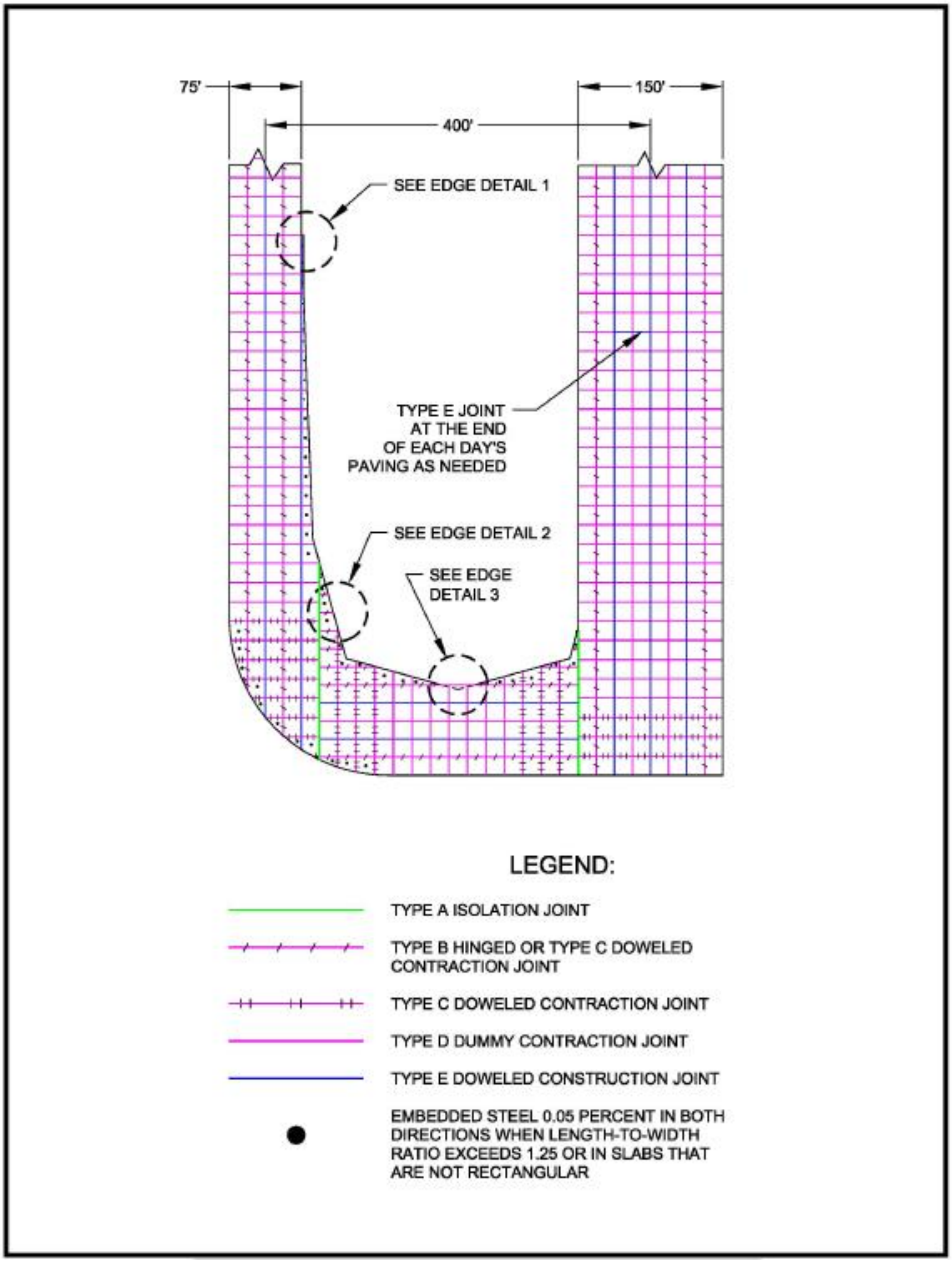
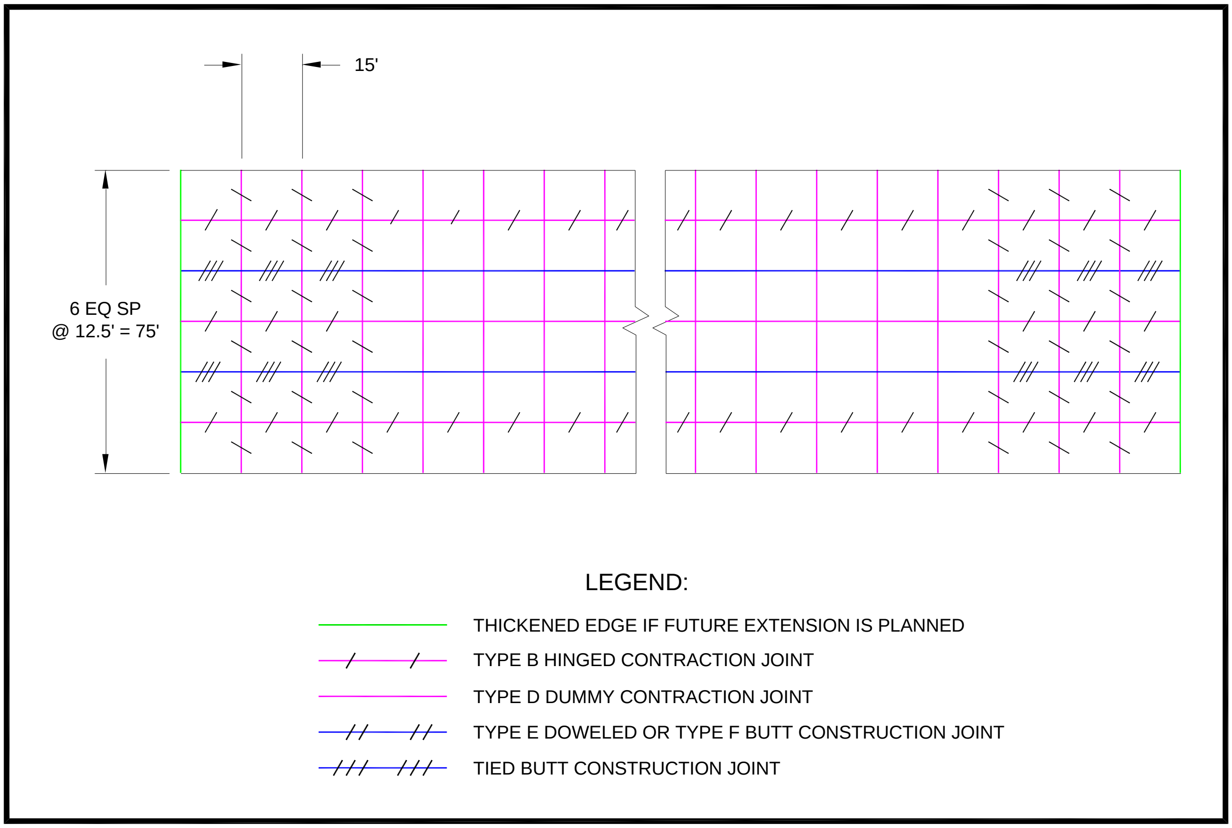


Figure 3-17. (Optional) Joint Layout PCC Pavement – 75 Foot Runway Width (Pavements ≤ 9 inches)[1]



Notes:

1. The concept behind the jointing pattern shown is the creation of a "tension ring" around the perimeter of the pavement to hold the joints in the interior of the paved area tightly closed. The last three transverse contraction joints and the longitudinal joints nearest the free edge of the pavement are tied with #4 deformed bars, 20 inches (508 mm) long, spaced at 36 inches (914 mm) center to center.

Table 3-9. Recommended Maximum Joint Spacing - Rigid Pavement[1]

a. Without Stabilized Subbase

Slab Thickness	Joint Spacing
6 inches or less (152 mm)	12.5 feet (3.8 m)
6.5-9 inches (165-229 mm)	15 feet (4.6 m)
>9 inches (>229 mm)	20 feet (6.1 m)[2]

b. With Stabilized Subbase

Slab Thickness	Joint Spacing
8–10 inches (203-254 mm)	12.5 feet (3.8 m)
10.5-13 inches (267-330 mm)	15 feet (4.6 m)
13.5-16 inches (343-406 mm)	17.5 feet (5.3 m)
>16 inches (>406 mm)	20 feet (6.1 m)[2]

Notes:

1. Longitudinal joint spacing shown in the tables. Transverse spacing should not exceed 1.25 the longitudinal spacing.
2. On Group IV Taxiways, 20.5 feet (6.2 m).

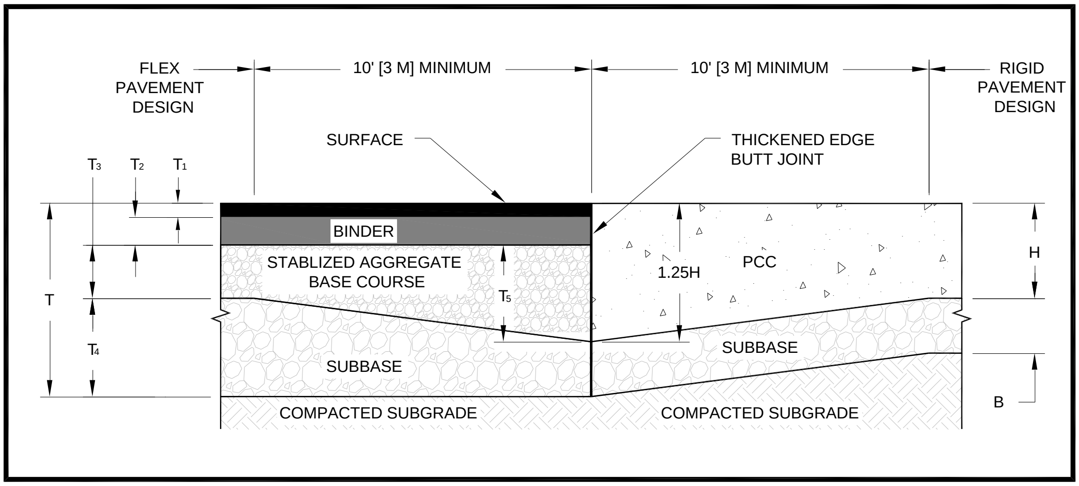
3.14.14 Jointing Considerations for Future Pavement Expansion.

When a runway or taxiway is likely to be extended, the construction of a thickened edge joint (for Type A, see Figure 3-15a) should be provided at that end of the runway or pavement. At locations where there may be a need to accommodate a future connecting taxiway or apron entrance, a thickened edge should be provided as appropriate. To avoid trapping water under a pavement, it is critical to maintain a constant transverse cross slope for the subgrade under the pavement that supports the base (or subbase).

3.14.15 Transition Between PCC and HMA.

When rigid pavement abuts a flexible pavement section at a location that will be subjected to regular aircraft loading, a transition should be provided using a detail similar to Figure 3-18. Note: This is one example of how a transition could be constructed. At the point of transition it is necessary to match subgrade elevation on both sides of the transition, as well as to provide a stabilized base under the flexible pavement.

Figure 3-18. Transition between PCC and HMA Pavement Sections



Dimension	Description
H	Design thickness of PCC pavement
B	Thickness of base
T	Design thickness of flexible (HMA) pavement
T_1	Design thickness of surface course
T_2	Design thickness of binder course
T_3	Design thickness of base course
T_4	Design thickness of subbase course
T_5	$(H + B) – (T_1 + T_2)$ or $2(T_3)$, whichever is greater

3.14.16 Rigid Design Example.

The design of a pavement structure is an iterative process in FAARFIELD. The user enters the pavement structure and airplane traffic to be applied to the section. FAARFIELD evaluates the minimum pavement layer requirements and adjusts the PCC thickness to give a predicted life equal to the design period (generally 20 years). This example follows the steps as outlined in paragraph 3.12.5.

Step (1): From 'Startup Screen' create new job, and add basic section(s) from sample sections to be analyzed.

Step (2a): For this example, assume the following starting pavement structure:

Pavement structure:

Thickness	Pavement Structure
Thickness to be determined by FAARFIELD	P-501 PCC Surface Course (R = 600 psi)
5 inches	P-401/P-403 Stabilized Base Course
12 inches	P-209 Crushed Aggregate Base Course
	Subgrade, CBR=5 (E = 7500 psi)

Step (2b): With the following airplane traffic:

Airplane traffic:

Airplane	Gross Weight (lbs)	Annual Departures
B737-800	174,700	3000
A321-200 opt	207,014	2500
EMB-195 STD	107,916	4500
Regional Jet – 700	72,500	3500

Step (2c): The pavement structure to be analyzed is entered by clicking on the 'STRUCTURE' button (Figure 3-19) and modifying the existing structure to match proposed pavement section by selecting the 'Modify Structure' button (see Figure 3-20). Layers can then be added by selecting the 'Add/Delete Layer' button. Layer types can be changed by 'clicking' on the layer material and thickness of the layer can be adjusted by clicking on the layer thickness To change the modulus for a layer, click on the modulus. A dialog box will display. If the modulus for the layer type is user-changeable, then the dialog box will display the allowable range of values and a box to enter a new value. If the modulus is not user-changeable, the dialog box displays the message the value is fixed in FAARFIELD. When done making adjustments select 'End Modify' button.

Figure 3-19. Rigid Design Example Step 2

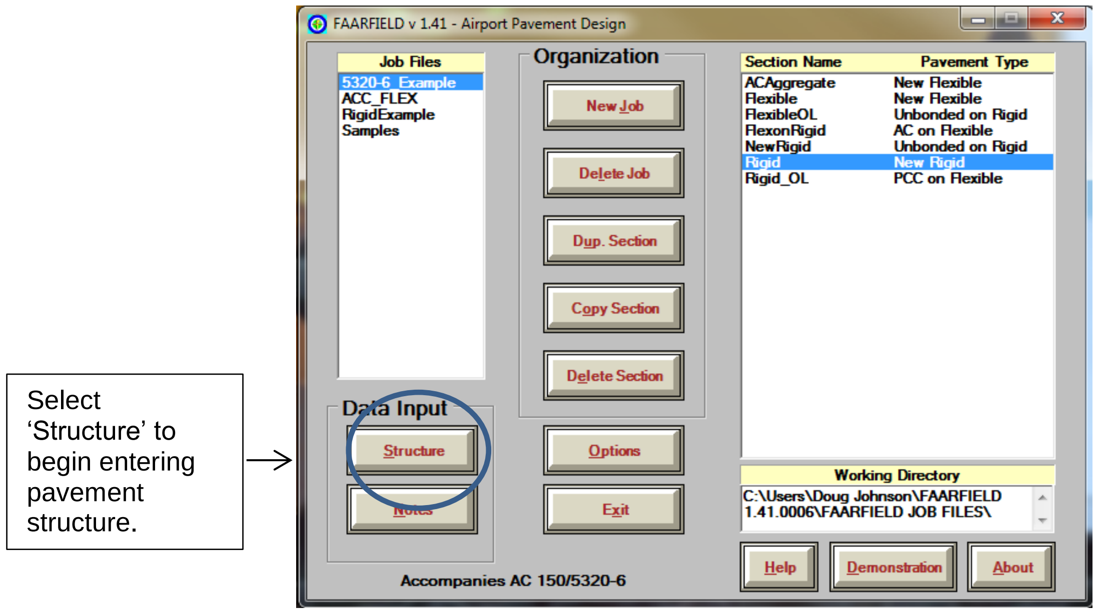
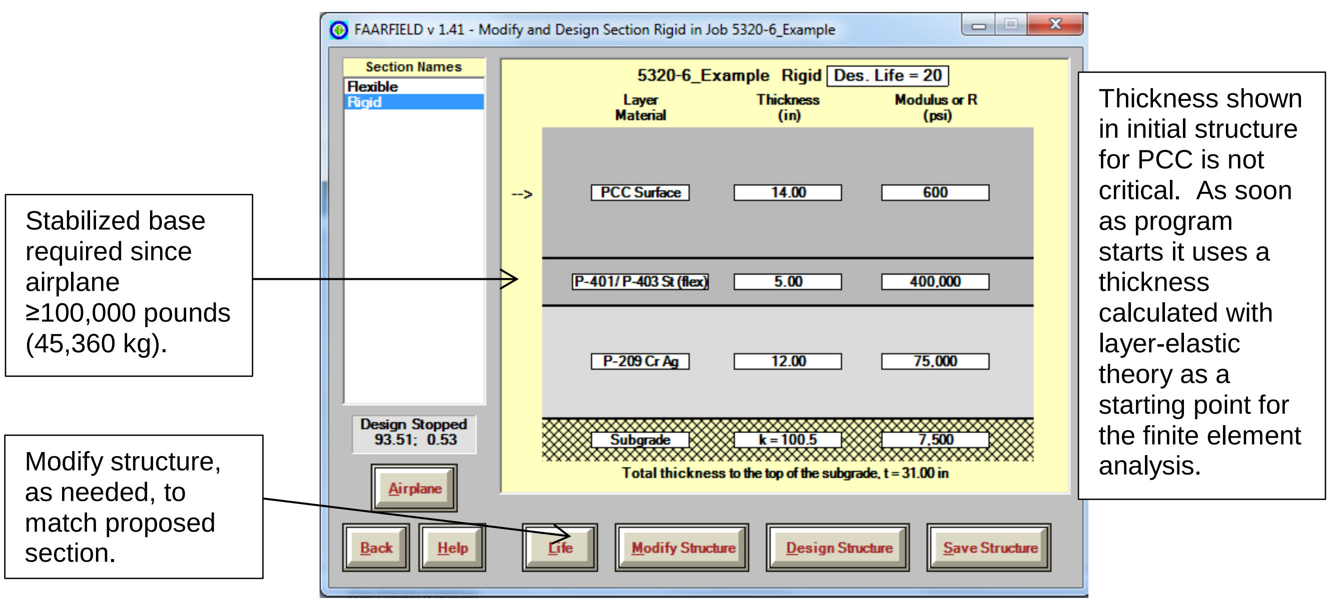


Figure 3-20. Rigid Design Example Step 2C Modify Structure Information



Step (3) Enter the Airplane screen by selecting the 'Airplane' button at the lower left of the Structure screen (Figure 3-21). Airplanes are added to the traffic mix by selecting them from the airplane library located on the left side of the Airplane screen. For each airplane selected, the following data may be adjusted: Gross Taxi Weight, Annual Departures, and percent annual growth (Figure 3-22). Airplanes are organized by group based on the airplane manufacturer. In addition there is a group of generic airplanes. After entering all of the airplanes, return to the Structure Screen by selecting the 'Back" button.

Figure 3-21. Rigid Design Example Step 3

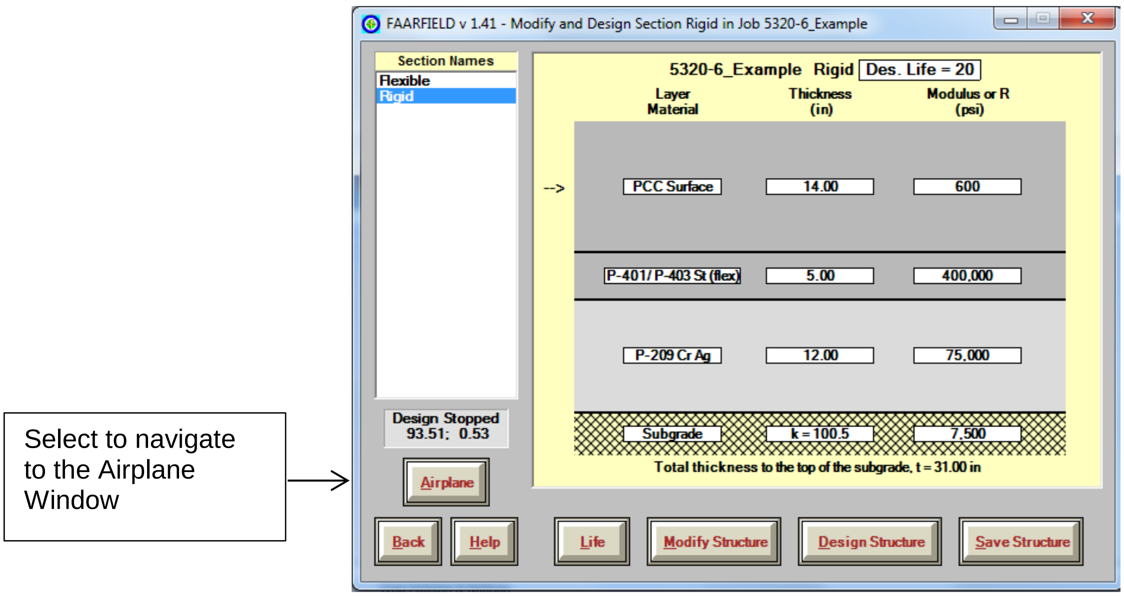


Figure 3-22. Rigid Design Example Step 3 Airplane Data

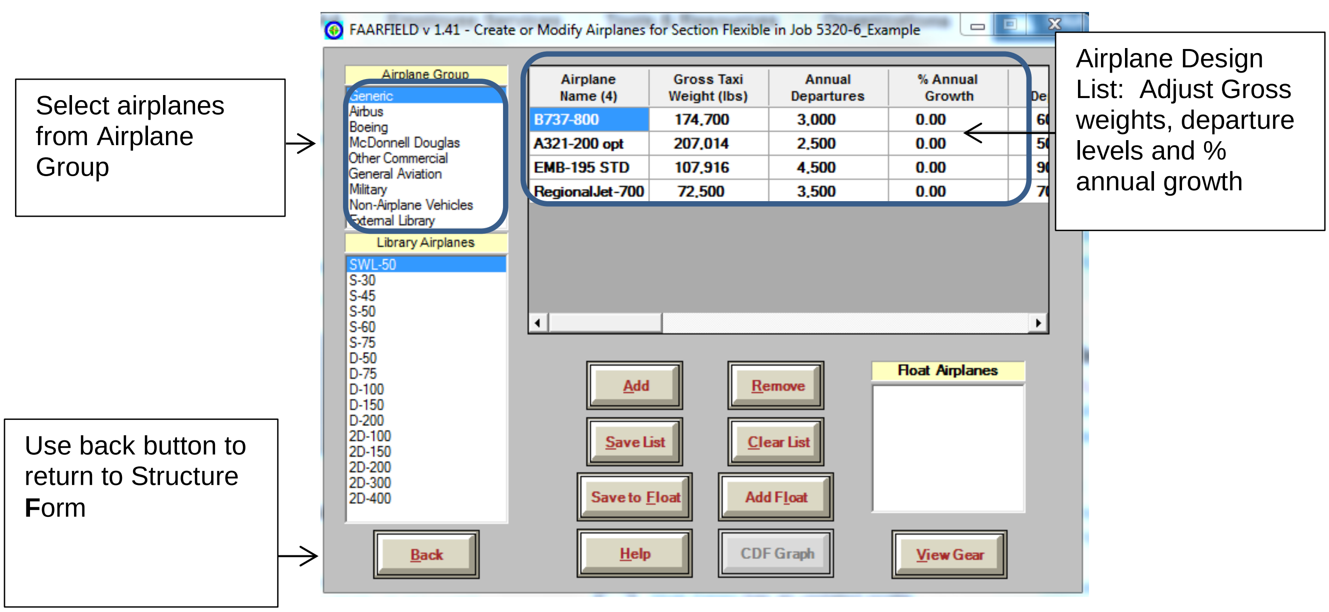


Figure 3-23. Rigid Design Example Step 4 Design Structure

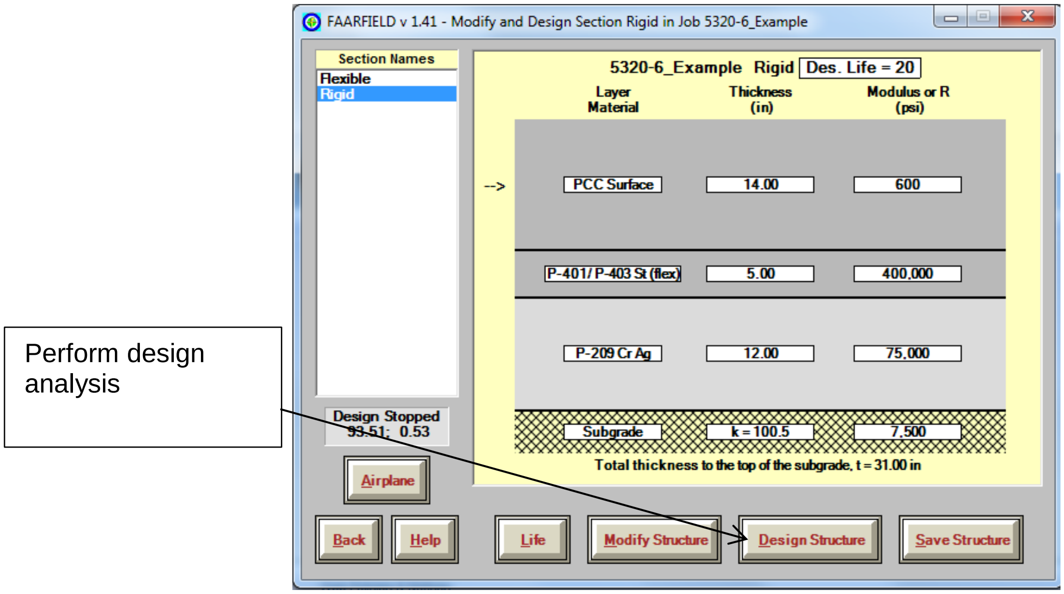
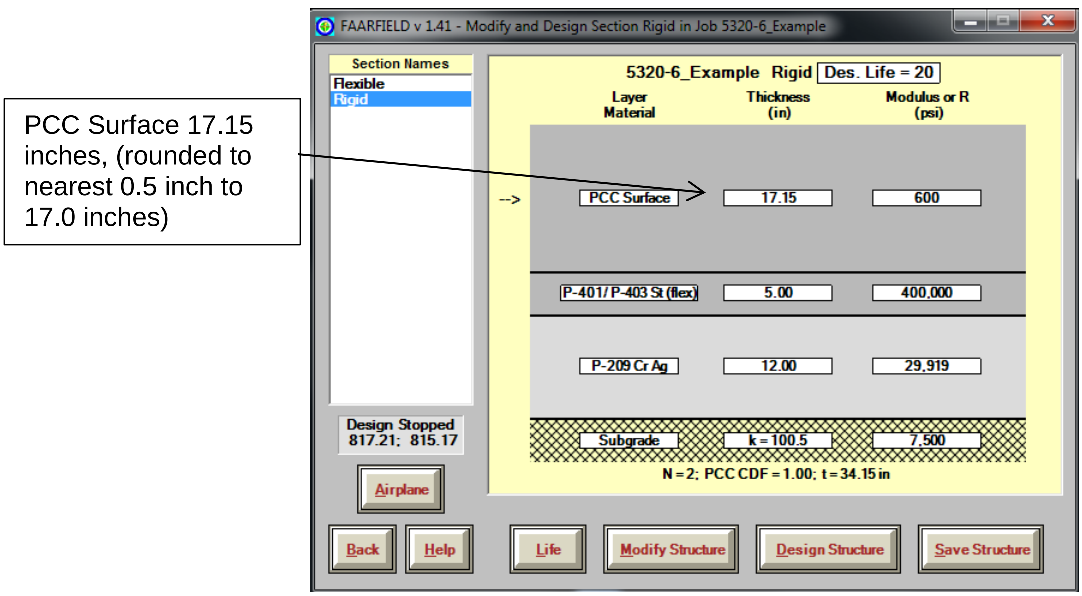


Figure 3-24. Rigid Design Example Step 4 Pavement Structure

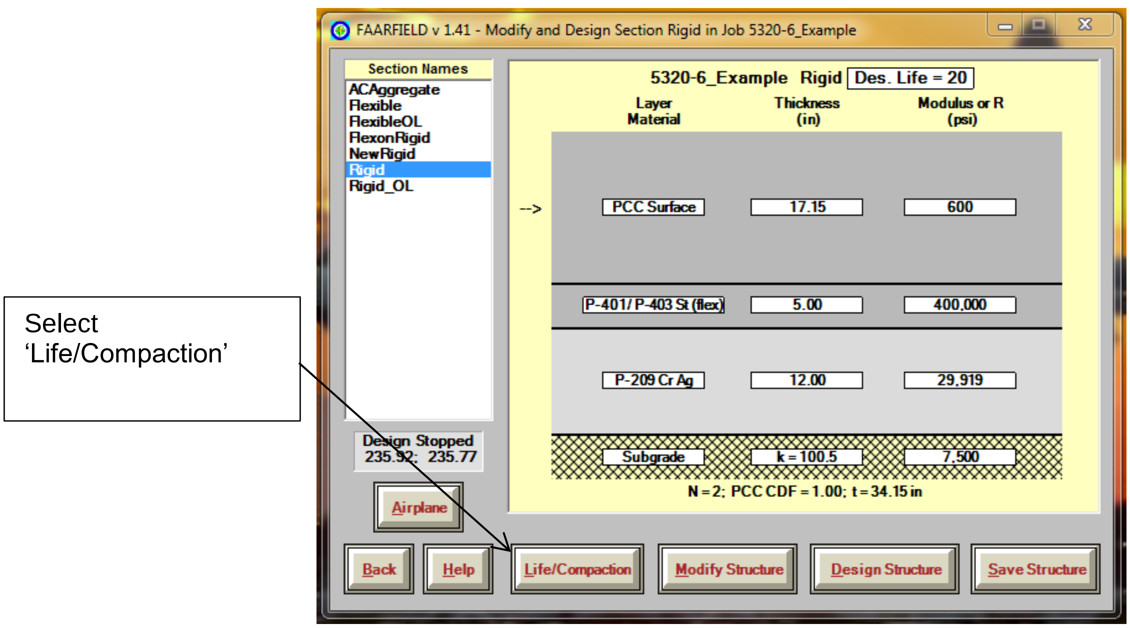


Step (4): Select 'Design Structure' button to start design analysis, see Figure 3-23. FAARFIELD iterates on the thickness of PCC Surface until a CDF of 1.0 is reached. The procedure gives a thickness of 17.15 inches (43 cm), see Figure 3-24. FAARFIELD does not design the thickness of pavement layers other than the PCC slab in rigid pavement structures, but will enforce the minimum thickness requirements for all layers as shown in Table 3-4. The solution time depends upon many factors, including the structure and the number of aircraft. In general, rigid designs take longer than flexible designs due to the finite element process. While FAARFIELD is running, the time clock at the lower left will update periodically.

Step (5): If the structure to be built includes layer thicknesses different than those used in the initial analysis, adjust layer thickness and repeat design analysis.

Step (6): FAARFIELD v1.4 includes the ability to evaluate the depth of subgrade compaction required after you have completed the design of the pavement structure. After completing your design, select the 'Life/Compaction' button, see Figure 3-25. (Note: If you have not completed a design, this button will just be labeled 'Life', or if you have not selected the 'Compute Compaction Requirements' in the option screen.) The design report then includes a subgrade compaction table for Non-Cohesive and Cohesive subgrade. Check that the "compute compaction requirements" is selected on the Options Screen.

Figure 3-25. Rigid Design Example Step 6 Life/Compaction



Steps (7/8): The Airport Pavement Design report (Figure 3-26) is automatically saved into the same working directory that you designated for your FAARFIELD job files or the report can be viewed from the startup screen by selecting 'Notes' button. The design report summarizes the Pavement Structure, Airplane Traffic and the CDF contribution of each aircraft evaluated.

Note: In this example for construction, the PCC thickness would be rounded to the nearest 0.5 inch (12.5 mm), or to 17.0 inches (425 mm)

Figure 3-26. Airport Pavement Step 7 and 8 Design Report

FAARFIELD

FAARFIELD v 1.41 - Airport Pavement Design

Section Rigid in Job 5320-6_Example.

Working directory is C:\Users\Doug Johnson\FAARFIELD 1.41.0006\FAARFIELD JOB FILES\

The structure is New Rigid.

Design Life = 20 years.

A design for this section was completed on 05/05/16 at 13:25:41.

Compaction requirements for this section were computed on 05/05/16 at 13:29:57.

Pavement Structure Information by Layer, Top First

No.	Type	Thickness in	Modulus psi	Poisson's Ratio	Strength R,psi
1	PCC Surface	17.15	4,000,000	0.15	600
2	P-401/ P-403 St (flex)	5.00	400,000	0.35	0
3	P-209 Cr Ag	12.00	29,919	0.35	0
4	Subgrade	0.00	7,500	0.40	0

Total thickness to the top of the subgrade = 34.15 in

Airplane Information

No.	Name	Gross Wt. lbs	Annual Departures	% Annual Growth
1	B737-800	174,700	3,000	0.00
2	A321-200 opt	207,014	2,500	0.00
3	EMB-195 STD	107,916	4,500	0.00
4	RegionalJet-700	72,500	3,500	0.00

Additional Airplane Information

No.	Name	CDF Contribution	CDF Max for Airplane	P/C Ratio
1	B737-800	0.03	0.05	3.52
2	A321-200 opt	0.97	0.97	3.42
3	EMB-195 STD	0.00	0.00	3.90
4	RegionalJet-700	0.00	0.00	4.71

Subgrade Compaction Requirements

NonCohesive Soil

Percent Maximum Dry Density(%)	Depth of compaction from pavement surface (in)	Depth of compaction from top of subgrade (in)	Critical Airplane for Compaction
100	0 - 13	--	A321-200 opt
95	13 - 17	--	A321-200 opt

Figure 3-26. Airport Pavement Step 7 and 8 Design Report (continued)

90	17 - 25	--	A321-200 opt
85	25 - 68	0 - 34	A321-200 opt

Cohesive Soil

Percent Maximum Dry Density(%)	Depth of compaction from pavement surface (in)	Depth of compaction from top of subgrade (in)	Critical Airplane for Compaction
95	0 - 13	--	A321-200 opt
90	13 - 16	--	A321-200 opt
85	16 - 18	--	A321-200 opt
80	18 - 24	--	A321-200 opt

Subgrade Compaction Notes:

1.Noncohesive soils, for the purpose of determining compaction control, are those with a plasticity index (PI) less than 3.

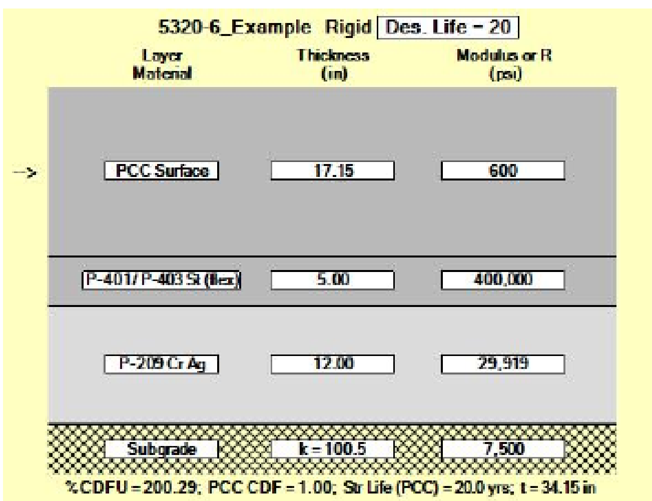
2.Tabulated values indicate depth ranges within which densities should equal or exceed the indicated percentage of the maximum dry density as specified in item P-152.

3.Maximum dry density is determined using ASTM Method D 1557.

4.The subgrade in cut areas should have natural densities shown or should (a) be compacted from the surface to achieve the required densities, (b) be removed and replaced at the densities shown, or (c) when economics and grades permit, be covered with sufficient select or subbase material so that the uncompacted subgrade is at a depth where the in-place densities are satisfactory.

5.For swelling soils refer to AC 150/5320-6E paragraph 313.

User is responsible for checking frost protection requirements.



3.15 Pre-stressed, Precast, Reinforced and Continuously Reinforced Concrete Pavement.

Pre-stressed, precast, reinforced, and continuously reinforced concrete pavements have been used in airport applications to a limited extent. Use of pre-stressed, precast, reinforced, and continuously reinforced concrete airport pavements on federally funded projects require FAA approval.

3.16 Aggregate Turf Pavements.

Aggregate-turf should be considered only for areas designed to serve non-jet airplanes having gross weights of 12,500 pounds (5,670 kg) or less. Some areas of airports serving light airplanes may not require hard surfacing. In these areas, the development of an aggregate-turf or turf surface may be adequate for limited operations of these light airplanes. Aggregate-turf surfaces are constructed by improving the stability of a soil with the addition of aggregate prior to development of the turf. The stability of the underlying soil is increased by the addition of granular materials prior to establishment of the turf. This provides a landing area that will not soften appreciably during wet weather and has sufficient soil to promote the growth of grass.

3.16.1 Materials.

Material and construction requirements are covered in Item P-217, Aggregate-Turf Pavement. Aggregate-turf construction consists of a soil seedbed layer (soil or soil/aggregate combination) over a soil aggregate base course. The soil aggregate base course meeting the requirements of P-217 consists of crushed stone, gravel, or sand stabilized with soil.

3.16.2 Thickness.

The thickness varies with the soil type, drainage, and climatic conditions. The minimum thickness of the soil aggregate can be computed by FAARFIELD using the CBR of the subgrade. The minimum thickness of the soil seedbed is determined by the thickness required to support the growth of grass.

3.16.3 Aggregate Turf Pavement Example.

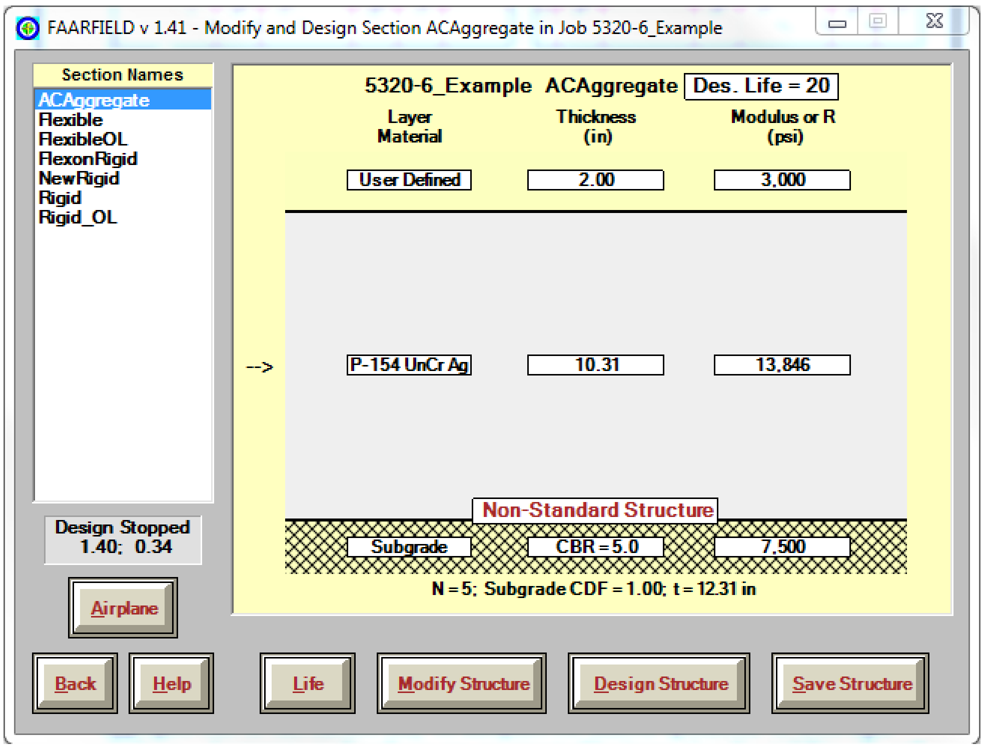
Assume that the airplane mix consists of the following:

Airplane	Gross Weight (lbs)	Annual Departures
King Air B-100	11,500	1,200
Conquest 441	9,925	500

3.16.3.1 The aggregate turf pavement will be constructed on a subgrade CBR = 5 and FAARFIELD will be used to determine the thickness of the aggregate stabilized base course layer.

3.16.3.2 A minimum thickness of 2 inches (50 mm) is assigned to the turf seedbed, although the actual thickness of soil will be determined by growing requirements. The turf seedbed is represented as a user-defined layer, with a nominal E-modulus of 3,000 psi (21 MPa). The design layer (aggregate stabilized base) is represented as P-154 uncrushed aggregate. In this example, the thickness required for the aggregate stabilized base course is 10.3 inches (287 mm), which will be rounded to 10.5 inches (265 mm) (Figure 3-27).

Figure 3-27. Aggregate Turf Pavement Structure



3.17 Heliport Design.

3.17.1 The guidance contained in this chapter is appropriate for pavements designed to serve rotary-wing airplanes. Refer to AC 150/5390-2, *Heliport Design*, for additional guidance on heliport gradients and heliport pavement design.

3.17.2 Generally, heliports are constructed with a PCC surface. The pavement is designed considering a dynamic load equal to 150 percent of the gross helicopter weight, equally distributed between the main landing gears. See Appendix B of AC 150/5390-2 for Helicopter Data. For the majority of helicopters, which have a maximum gross weight less than 30,000 pounds (13,610 kg), a 6-inch (150-mm) PCC slab will generally be sufficient. However, the loads of fuel or maintenance vehicles may be more demanding than the helicopter loads and may require additional pavement thickness.

3.18 Passenger Loading Bridge.

Design of the passenger loading bridge operating area is separate from the design of the adjacent aircraft apron. Loads of passenger loading bridges range from 40,000 – 100,000 pounds supported on two solid tires resulting in loads ranging from 600-700 psi per tire. Due to the large range of potential loads, verify the actual loads and contact tire pressure with the manufacturer of the passenger loading bridge. The FAA recommends rigid pavement be used where the passenger loading bridge will operate. Drainage structures and fuel hydrants should not be located in the jet bridge operation area. The design of the adjacent aircraft parking apron should only consider the aircraft and any equipment that will use the apron and not the load of the passenger loading bridge.

CHAPTER 4. PAVEMENT REHABILITATION

4.1 General.

4.1.1 Reason for Rehabilitation.

Pavements may require rehabilitation for a variety of reasons, for example to correct surface conditions that affect airplane performance (roughness, surface friction, and/or drainage) or material-related distresses or to repair localized structural damage due to overloading. A pavement in good condition may require strengthening to serve heavier airplanes and/or more frequent operations than the original pavement design supported. Some types of pavement rehabilitation methods incorporate the recycling of existing pavement materials. Rehabilitation or reconstruction can make use of recycled materials since techniques and equipment are readily available to recycle old pavement materials into base and subbase materials. However on federally funded projects, due to the variability of recycling materials and methods, the use of recycled materials other than those meeting P219, requires a Modification of Standards (MOS) in accordance with FAA Order 5100.1, *Modification of Agency Airport Design, Construction and Equipment Standards*.

4.1.2 Full Width Section.

Generally pavements are rehabilitated in full width sections. Pavements that are severely distressed in the center (keel) sections can sometimes be economically rehabilitated by reconstructing just the keel section. Structurally, reconstruction is no different than designing a new pavement structure. Refer to Chapter 3 when reconstruction of pavements is required.

4.1.3 Transitions.

Consider the transition to existing pavement structures and drainage when selecting the rehabilitation method. It may be necessary to remove sections of the existing pavement structure beyond the area of distressed pavement to comply with airport design gradients.

4.2 Condition of Existing Pavement Structure.

4.2.1 A complete condition assessment of the pavement materials and the structural integrity of the existing pavement structure is the first step in design of a rehabilitation project. The assessment of the existing pavement properties includes the thickness, condition, and strength of each layer; the subgrade soil classification; and an estimate of foundation strength (CBR or subgrade modulus (k or E)).

4.2.2 Sufficient investigations must be done during the design phase to ensure that the rehabilitation method chosen can be performed on the pavement. The construction plans and specifications may have to include limitations on the size and/or weight of construction equipment to avoid damaging the remaining pavement structure.

4.2.3 The rehabilitation design procedures in this AC assume the overlay will be placed on a base pavement with significant remaining structural integrity. Distressed areas in the existing pavement should be studied to determine the cause of the distresses and to determine potential mitigation. For example, pavement and material distresses such as alkali-silica reactivity in existing rigid pavements or highly weathered and cracked existing flexible pavements should be mitigated or removed, as necessary, before adding an overlay. When removing areas of distressed HMA by milling you must either remove the entire layer or leave at least 2 inches of HMA. The remaining pavement structure must be able to support the milling and all other construction equipment required.

4.2.4 Overlaying an existing pavement without correcting poor subsurface drainage usually results in poor overlay performance. Before overlaying, assess subsurface drainage conditions and correct any deficiencies. Corrections of subsurface drainage deficiencies may require reconstruction of the entire pavement structure.

4.2.5 A valuable technique for assessing the structural condition of the existing pavement is nondestructive pavement testing (NDT) (see Appendix C). NDT can help estimate foundation strength, measure joint load transfer, and possibly detect voids beneath existing pavements. NDT can also be used to determine structural capacity, assist with calculating PCN, and assess areas of localized weakness.

4.3 Material Selection Considerations.

This AC includes criteria for overlay of both flexible and rigid pavements. The type of overlay material selected should take into account existing pavement type, locally available materials, and cost of materials and construction. Consideration of the total life cycle cost of the reconstructed pavement or overlay will assist in determination of the most cost-effective solution. Generally the sooner preservation techniques are implemented, the bigger the long-term benefit. It is always more effective to extend the life of a pavement in good condition as opposed to rehabilitate a pavement in fair or poor condition. The condition of the pavement at the time of rehabilitation greatly impacts the cost of the rehabilitation. Life cycle costs should include initial construction and maintenance costs over the life of the pavement. User costs associated with the amount of time required for the pavement rehabilitation may also have a significant impact on the rehabilitation method selected.

4.4 Overlay Design.

An overlay consists of a new flexible or rigid surface course on top of an existing pavement. FAARFIELD overlay design is based on layered elastic and three-dimensional finite element methods of analysis. FAARFIELD designs the overlay thickness required to provide a 20-year (or other chosen) design life by meeting the limiting stress or strain criterion, subject to minimum thickness requirements (Table 3-3 and Table 3-4). There are four types of overlay pavements: HMA overlay of existing flexible or rigid pavement, and PCC overlay of existing flexible or rigid pavement.

4.5 Overlays of Existing Flexible Pavements.

The design of an overlay for an existing flexible pavement is similar to designing a new pavement. A pavement engineer should characterize the existing flexible pavement, assigning the appropriate thicknesses and moduli of the existing layers. An HMA overlay requires consideration of many factors including the condition, thickness, and properties of each layer of the existing HMA pavement structure. Milling of the HMA surface may be required to correct surface and grade deficiencies and/or remove deteriorated existing HMA material.

4.5.1 FAARFIELD Design HMA Overlay of an Existing Flexible Pavement.

FAARFIELD design of an HMA overlay on existing HMA is similar to the design of a new HMA pavement, except the HMA overlay is the design layer. In FAARFIELD, a trial overlay thickness is selected, and the program iterates on the overlay thickness until a CDF of 1.0 is reached at the top of the subgrade. The minimum HMA overlay of an existing flexible pavement is 2 inches (50 mm), however a thicker overlay typically performs better. The minimum structural overlay is 3 inches (75 mm). The design thickness of the overlay is the larger of the minimum thickness or the thickness required to achieve a subgrade or HMA CDF of 1, whichever controls.

4.5.1.1 Flexible Overlay on Existing Flexible Pavement Example.

FAARFIELD indicates a 4.57 inch overlay is needed. Depending on other factors, this may be rounded either to 4.5 inches or 5 inch. For an additional example of flexible pavement evaluation, refer to Chapter 5 and Appendix C.

Given the following airplane mix:

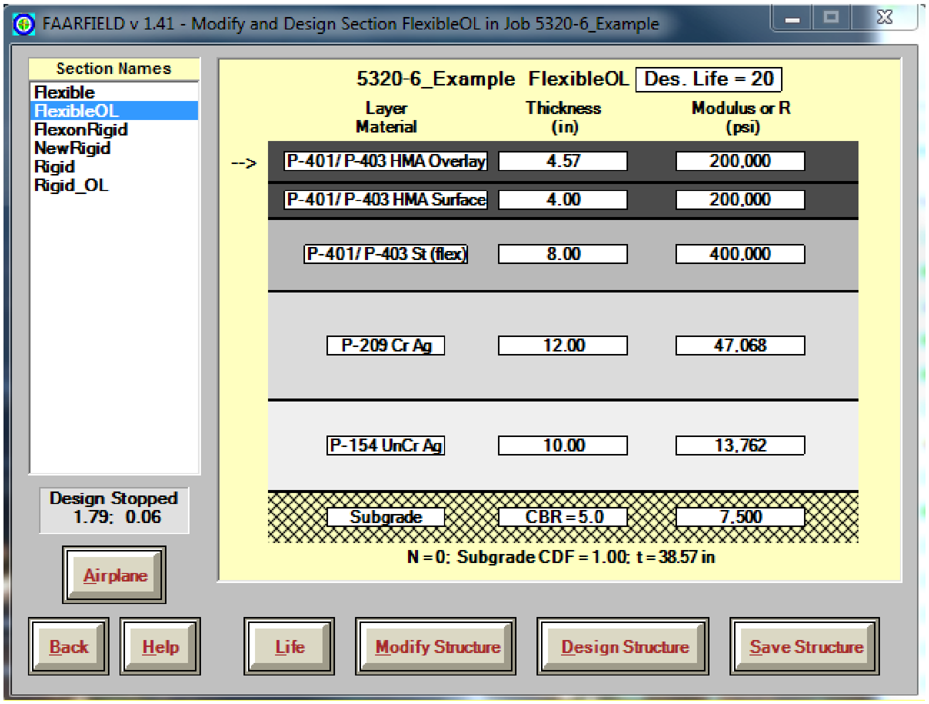
Airplane	Gross Weight (lbs)	Annual Departures
B737-800	174,700	3000
A321-200 opt	207,014	2500
EMB-195 STD	107,916	4500
Regional Jet – 700	72,500	3500
A380n	1,238,998	50
A380n Belly	1,238,998	50
B777-300 ER	777,000	50

And starting with the following structure:

Thickness	Pavement Structure
4 inches	P-401 HMA Surface Course
8 inches	P-403 Stabilized Base Course

Thickness	Pavement Structure
12 inches	P-209 Crushed Aggregated Base Course
10 inches	P-154 Aggregate Base Course
	Subgrade CBR 5.0

Figure 4-1. Flexible Overlay Structure



4.5.2 Nonstructural HMA Overlays.

An overlay may be required to correct nonstructural problems such as restoring the crown, correcting longitudinal profile, and/or improving skid-resistance. Thickness calculations are not required in these situations since thickness is controlled by minimum overlay thickness or other design considerations. Overlay thickness should be in 0.5-inch increments starting at least 2 inches. If a portion of the existing surface is to be removed prior to the removal, it is imperative to take sufficient pavement cores to determine the thickness and condition of the existing surface. When removing existing HMA surface course, either remove the entire course or leave sufficient surface material to maintain the integrity of the layer. Leaving less than 2 inches of surface course may result in the creation of a thin layer that is susceptible to delamination under construction traffic. On federally funded projects, overlay thicknesses less than 2 inches need FAA approval.

4.5.3 Concrete Overlay of an Existing Flexible Pavement.

The design of a concrete overlay on an existing flexible pavement is essentially the same as designing a new rigid pavement. The existing flexible pavement is characterized by assigning the appropriate thicknesses and moduli of the existing layers. A trial overlay thickness is selected and FAARFIELD iterates until a CDF of 1 is reached. The design thickness is the larger of the minimum PCC thickness or the overlay thickness required to achieve a CDF of 1. A pavement engineer should characterize the existing pavement layers. FAARFIELD assumes a frictionless (unbonded) interface between the concrete overlay and the existing flexible surface. The use of non-stabilized material below the rigid pavement overlay is not allowed because it results in the creation of a sandwich pavement. The use of a ¼ inch (5 mm) or less bond breaker (choke stone), however, is not considered a sandwich pavement. The minimum allowable thickness for a concrete overlay of an existing flexible pavement is 6 inches (150 mm). Concrete overlays constructed on existing flexible pavements should meet the joint spacing requirements of paragraph 3.14.3.

4.5.3.1 **Rigid Overlay on Existing Flexible Pavement Example.**

FAARFIELD indicates a 17.47 inch overlay is needed. Round to a 17.5 inch overlay (Figure 4-2).

Given the following airplane mix:

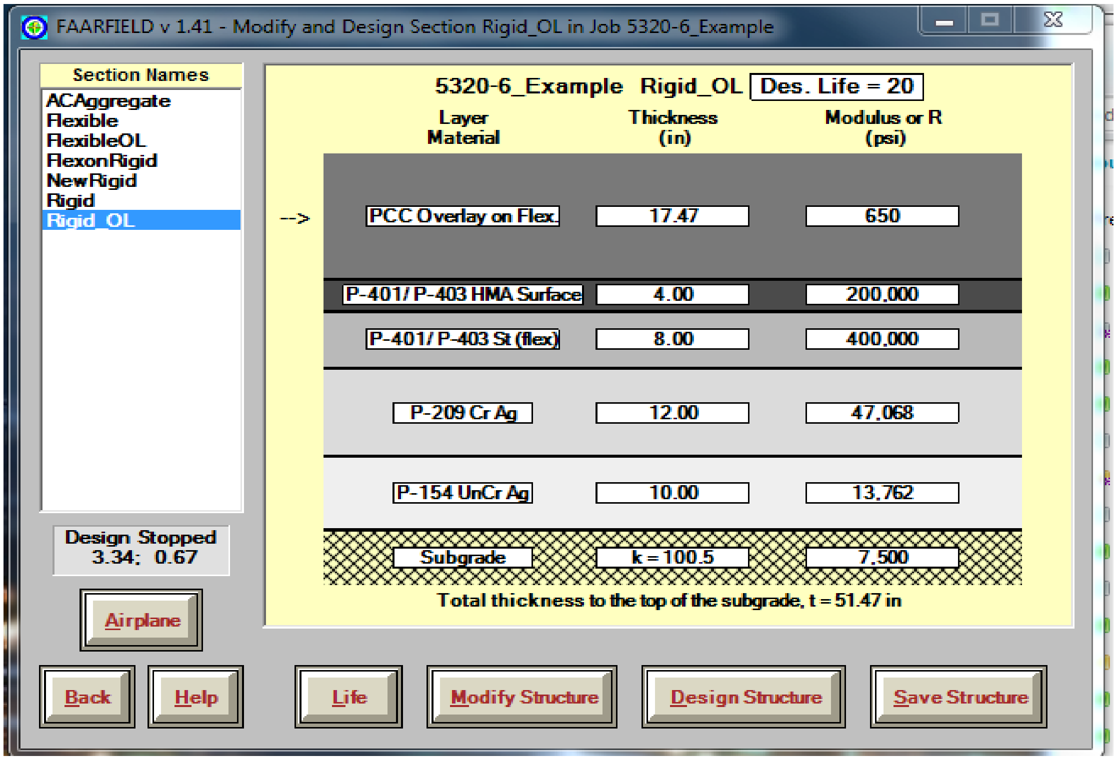
Airplane	Gross Weight (lbs)	Annual Departures
B737-800	174,700	3,000
A321-200 opt	207,014	2,500
EMB-195 STD	107,916	4,500
Regional Jet – 700	72,500	3,500
A380n	1,238,998	50
A380n Belly	1,238,998	50
B777-300 ER	777,000	50

And starting with the following pavement structure:

Thickness	Pavement Structure
	P-501 PCC Overlay on Flex (Mr=650)
4 inches	P-401 HMA Surface Course
8 inches	P-403 Stabilized Base Course
12 inches	P-209 Crushed Aggregated Base Course
10 inches	P-154 Aggregate Base Course

Thickness	Pavement Structure
	Subgrade Modulus 7,500 psi

Figure 4-2. Rigid Overlay Pavement Structure



4.6 Overlays of Existing Rigid Pavements.

The design of overlays for an existing rigid pavement must consider the structural condition of the existing pavement. Non-destructive testing (NDT), borings, or engineering judgment can help determine the flexural strength of the existing rigid pavement. The condition of the existing rigid pavement prior to overlay is expressed in terms of the structural condition index (SCI). When SCI is equal to 100 (no visible structural cracks), then you calculate the Cumulative Damage Factor Used (CDFU). In general, thicker HMA overlays perform better than thin HMA overlays, and overlay thickness on rigid pavements may need to exceed minimum HMA thickness. Issues with reflective cracking, slippage, and rutting may be exacerbated with thin HMA overlays of rigid pavements.

4.6.1 Structural Condition Index (SCI).

The SCI does not include conventional shrinkage cracks due to curing or other non-load-related problems. The SCI is the summation of structural components from the pavement condition index (PCI). Guidance on PCI can be found in Chapter 5 and ASTM D 5340, *Standard Test Method for Airport Pavement Condition Index Survey*.

An SCI of 80 is the FAA definition of structural failure of a rigid pavement and is consistent with 50 percent of slabs in the traffic area exhibiting a structural crack. The SCI can be computed automatically with computer programs such as MicroPAVER or FAA PAVEAIR. Additional guidance on deriving an SCI is also provided in the FAARFIELD Help feature. For fully unbonded concrete overlay, the modulus of the base pavement varies as a function of the SCI of the base pavement when the SCI is less than 100. This computation is done automatically within FAARFIELD. The equations for the modulus reduction as a function of the SCI are given in Report No. DOT-FAA-PM-87/19, *Design of Overlays for Rigid Airport Pavements*.

4.6.2 Cumulative Damage Factor Used (CDFU).

4.6.2.1 When the SCI of the existing pavement is 100 (i.e., no visible distresses contributing to a reduction in SCI) and the pavement is not new (has received some traffic), estimate the amount of fatigue life that has been consumed up to the time of the overlay. In this case, the condition of the existing pavement is described by the cumulative damage factor used (CDFU). For aggregate base layers, assuming that traffic on the pavement has been constant over time, a good estimate of CDFU can be obtained from:

$$\begin{aligned} CDFU &= \frac{L_U}{0.75\,L_D} \quad \text{when } L_U < 0.75\,L_D \\ &= 1 \quad \text{when } L_U \geq 0.75\,L_D \end{aligned}$$

where:

L_U = number of years of operation of the existing pavement until overlay

L_D = structural design life of the existing pavement in years

4.6.2.2 This equation was derived from the empirical relationship between traffic coverages and SCI and only applies to pavements on conventional (aggregate) base. For rigid pavements on stabilized bases, this relationship is not valid, and FAARFIELD must be used to compute the CDFU. The percent CDFU is computed and displayed when the Life button is clicked in the STRUCTURE screen.

4.6.2.3 When computing percent CDFU for a rigid pavement on stabilized base, FAARFIELD defaults to setting CDFU = 100, which will give the most conservative design. To calculate a CDFU other than 100:

1. Set up the structure based upon what was constructed.
2. Estimate the traffic that has actually been applied to the pavement and enter it into the airplane design list.

3. Set “Structural Design Life” to the number of years the pavement will have been in operation up to the time of overlay.
4. Run Life.

4.6.2.4 The percent CDFU will be displayed when the Life computation is completed. A computed value of percent CDFU greater than 100 indicates that, based on the estimated structural properties and traffic inputs, the FAARFIELD procedure predicts the SCI of the pavement should be less than 100. In this case, a value of 100 should be entered for percent CDFU as input data for the overlay design. However, since the computation of percent CDFU is based on estimated structure properties and traffic, the value is likely to be unreliable. An alternative procedure is to run Design Structure for the original structure with structural design life set to the actual design life, where actual design life is typically the 20-year design period. Then repeat the steps above and use the new value of percent CDFU.

4.6.2.5 If the pavement has been subjected to more or heavier traffic than assumed in the Life computation, increase the percent CDFU from the computed value. Setting percent CDFU to 100 will always give the most conservative design.

4.6.2.6 **CDFU Example.**

1. The following steps illustrate the procedure for calculating CDFU:
2. The existing PCC surface does not currently exhibit structural damage, i.e., SCI = 100, to determine what CDFU is:

Set up the structure based on the actual section constructed.

Thickness	**Pavement Structure**
17.5 inches	P-501 PCC Surface Course (Mr=625 psi)
5 inches	P-401/403 Stabilized Base Course
12 inches	P-209 Crushed Aggregate Base Course
	Subgrade, k = 100.5 pci equivalent to E-modulus = 7500 psi

Note: The existing pavement was designed to accommodate the following airplane mix: 3000 departures of B737-800, 2,500 departures of A321-200 opt, 4,500 departures of EMB-195 STD and 3,500 departures of RJ-700. FAARFIELD determined 17.15 inches was needed; however 17.5 inches was constructed.

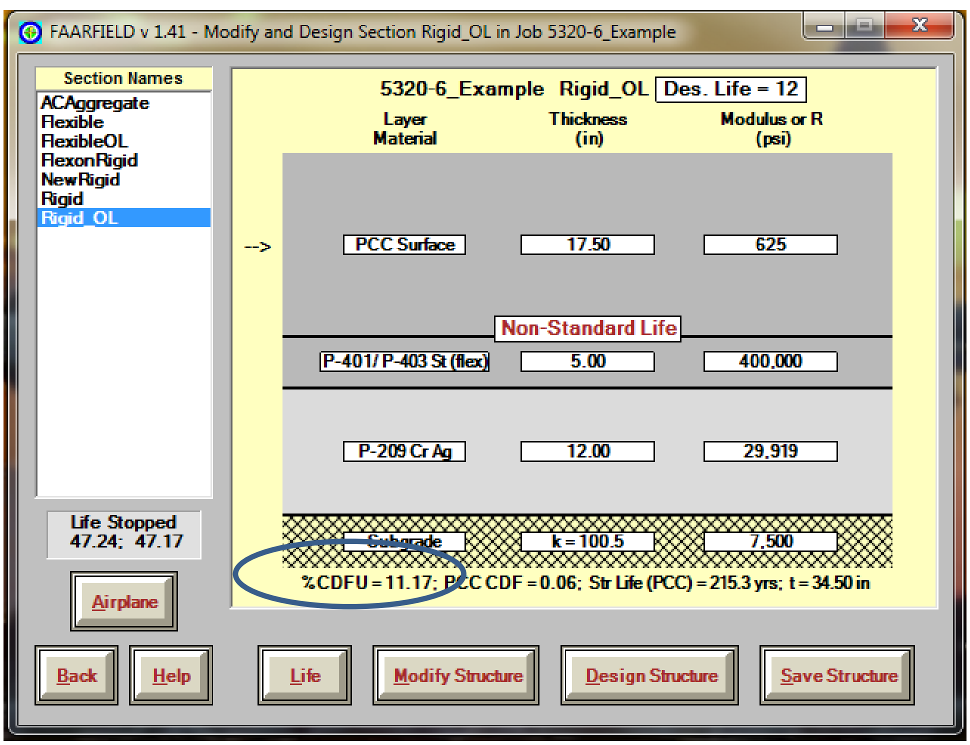
3. Using the traffic applied to the pavement, enter it into the airplane design list.

Assume the annual traffic levels actually applied to the pavement are as follows:

Airplane	Gross Weight (lbs)	Annual Departures
B737-800	174,700	1500
A321-200 opt	207,014	1250
EMB-195 STD	107,916	2250
Regional Jet – 700	72,500	1750

4. Set the "Structural Design Life" to the number of years the pavement has been in operation. For this example, assume the taxiway has been in service for 12 years. In the Structure screen, click on "Structural Design Life" and change to 12 years. A message "Non-Standard Life" will display, indicating that a life equal to other than 20 years has been selected.
5. Run Life. The calculated percent CDFU will appear on the Structure screen, at the lower left of the pavement section.
6. For the above case, FAARFIELD calculates percent CDFU equal to 11.17 (Figure 4-1). For overlay design, the value CDFU = 12 percent would be used.

Figure 4-3. Rigid Overlay Percent CDFU



4.6.2.7 One potential source of confusion is that the value percent CDFU = 12 does not mean that 12 percent of the original structural design life has been used up. Rather, this value should be interpreted as indicating that, at the time of the overlay, the pavement will have received 12 percent of the number of traffic passes predicted to result in a first full structural crack (i.e., 12 percent of the number of passes theoretically needed to bring the pavement to the point at which its SCI is less than 100 or perfect structural condition). At this point, the pavement still has significant structural life.

4.7 Hot Mix Asphalt Overlays of Existing Rigid Pavements.

The design process for hot mix overlays of rigid pavements considers two conditions for the existing rigid pavement to be overlaid: (1) SCI of the existing pavement less than 100 and (2) SCI equal to 100. The minimum thickness of hot mix asphalt overlay on existing rigid pavements is 3 inches (75 mm).

4.8 SCI Less Than 100.

The most likely situation is one in which the existing pavement is exhibiting some structural distress, i.e., the SCI is less than 100. If the SCI is less than 100, the base pavement will continue to deteriorate at the rate predicted by the rigid pavement failure model. The pavement structure is assumed to have failed when the SCI of the concrete base pavement reaches a terminal value. FAARFIELD assumes an initial overlay thickness and iterates on the overlay thickness until a 20-year life is predicted. A 20-year predicted life satisfies the design requirements.

Note: The design thickness generated by FAARFIELD does not currently address reflection cracking of the hot mix asphalt overlay as a potential failure mode.

4.8.1 HMA Overlay Over PCC Example.

The existing concrete strength is estimated as 625 psi (4.5 MPa). Based on a visual survey, the existing pavement is assigned an SCI of 80. Frost action is negligible. Based on these conditions, the flexible pavement overlay required will be a 6.42 inch HMA overlay, which is rounded to 6.5 inches (Figure 4-4).

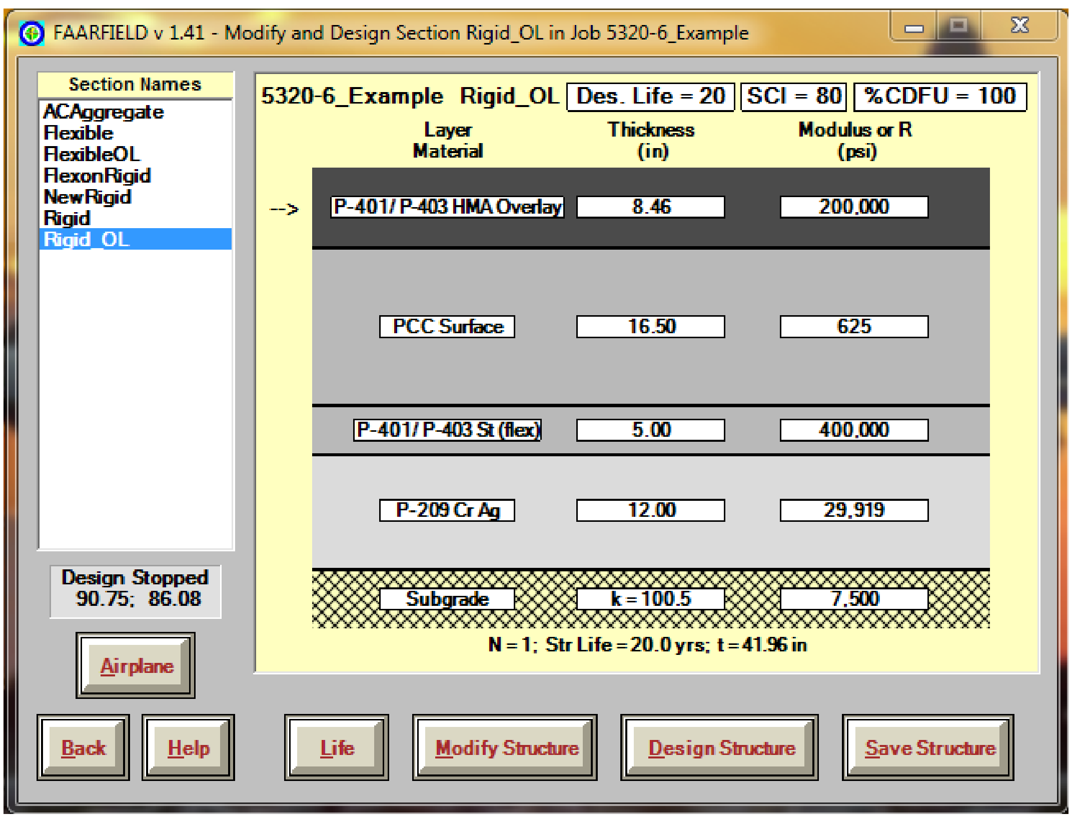
Assume an existing taxiway pavement composed of the following section:

Thickness	Pavement Structure
16.5 inches	P-501 PCC Surface Course (Mr=625)
5 inches	P-401/P-403 Stabilized Base Course
12 inches	P-209 Crushed Aggregate Base Course
	Subgrade, k = 100.5 (E = 7500 psi)

Assume the existing pavement will be strengthened to accommodate the following airplane mix:

Airplane	Gross Weight (lbs)	Annual Departures
B737-800	174,700	3,000
A321-200 opt	207,014	2,500
EMB-195 STD	107,916	4,500
RegionalJet-700	72,500	3,500
A380n	1,238,998	50
A380n Bely	1,238,998	50
B777-300 ER	777,000	50

Figure 4-4. Flexible Overlay on Rigid Pavement



4.9 SCI Equal to 100.

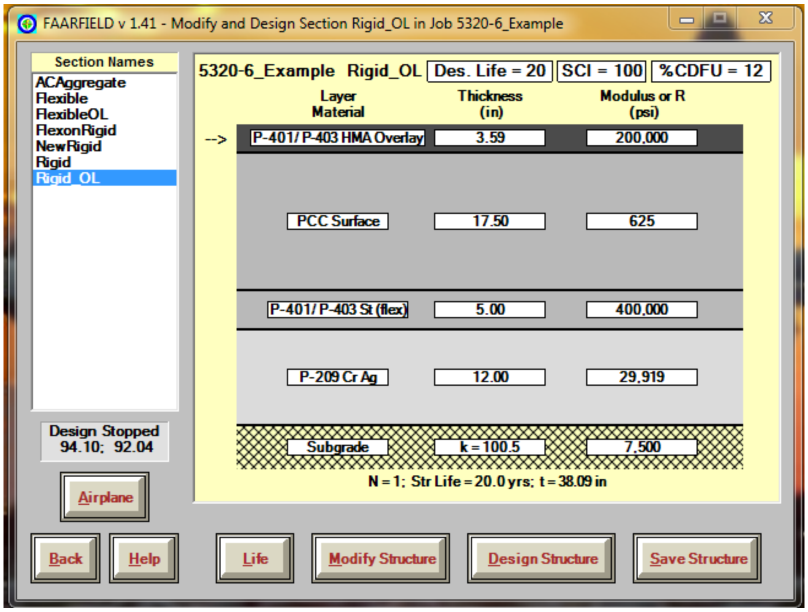
4.9.1 An existing pavement with an SCI of 100 may require an overlay to strengthen the pavement to accept heavier airplanes. If the SCI of the base pavement is equal to 100, an additional input is required: the percent CDFU. FAARFIELD assumes the base pavement will deteriorate at different rates when the SCI is equal to 100 and after the SCI drops below 100. As with case (1), a trial overlay thickness is input, and the program iterates on that thickness until a 20-year life is predicted. The design thickness is the thickness that provides a 20-year predicted life.

4.9.2 After adjusting structure to add in an HMA overlay layer, design life = 20 years, SCI =100 and CDFU = 12, the FAARFIELD analysis indicates than a 3.59 inch overlay is needed. Typically this would be rounded to 3.5 inches. Traffic is what is expected over the next 20 years.

Figure 4-5. HMA overlay of Rigid Traffic

Airplane Name (7)	Gross Taxi Weight (lbs)	Annual Departures
B737-800	174,700	3,000
A321-200 opt	207,014	2,500
EMB-195 STD	107,916	4,500
RegionalJet-700	72,500	3,500
B777-300 ER	777,000	50
A380	1,238,998	50
A380 Belly	1,238,998	50

Figure 4-6. Flexible Overlay of Rigid



4.10 Concrete Overlays of Existing Concrete Pavements.

The design of a concrete overlay of an existing rigid pavement is the most complex type of overlay design. The overlay design must consider the condition of the existing pavement, degree of bond between the overlay and existing pavement, and the deterioration of both the concrete overlay and the existing rigid pavement. FAARFIELD considers two possible degrees of bond: fully unbonded and fully bonded.

Note: Rigid pavements that have significant structural distress are not candidates for an overlay. Generally, pavements with an SCI less than 80 are not acceptable candidates because they would require extensive repairs prior to the overlay.

4.10.1 Fully Unbonded Concrete Overlay.

You must intentionally eliminate bonding between the overlay and existing pavement for an unbonded concrete overlay of an existing rigid pavement. Typically, this is achieved by applying a thin hot mix layer or fabric bondbreaker to the existing rigid pavement. An SCI is required to describe the condition of the existing pavement. A trial overlay thickness is input and FAARFIELD iterates until a 20-year structural design life overlay thickness is achieved. The minimum thickness for a fully unbonded concrete overlay is 6 inches (150 mm).

4.10.1.1 Example - Rigid Overlay on Existing Fully Unbonded Rigid Pavement.

Assume the existing pavement is to be strengthened to accommodate the following airplane mix:

Airplane	Gross Weight (lbs)	Annual Departures
B747-200B Combi Mixed	836,000	850
B747-200B Combi Mixed Belly	836,000	850
B777-200 ER	634,500	1,000
B767-200	361,000	2,500

Assume an existing taxiway pavement composed of the following section:

Thickness	Pavement Structure
14 inches	P-501 PCC Surface Course (Mr =700)
12 inches	P-209 Base Course
12 inches	P-154 Aggregate Base Course
	Subgrade, k = 172 pci (E = 15,000 psi)

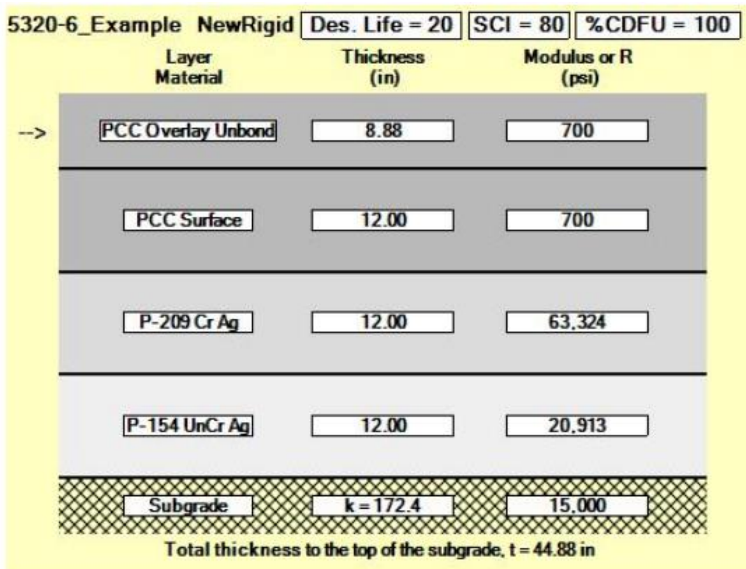
1. SCI is 80 for the existing PCC surface. Frost action is negligible. Assume that the PCC strength is 700 psi (4.83 MPa) for both the overlay and the existing concrete.

The overlay structure computed by FAARFIELD for these conditions is:

Thickness	Pavement Structure
8.88 inches	PCC unbonded overlay
1.0 inches	Debonding layer
14 inches	PCC Surface Course
12 inches	P-209 Base Course
12 inches	P-154 Aggregate Subbase

2. This gives a total pavement thickness of 44.88 inches. Note that FAARFIELD does not include the debonding layer in thickness calculations.

Figure 4-7. Rigid Overlay on Existing Fully Unbonded Rigid Pavement



3. The required overlay thickness is 8.88 inches (225 mm), which will be rounded to the nearest 0.5 inches, or 9.0 inches (228 mm)).

4.10.2 <u>Bonded Concrete Overlays</u>.

4.10.2.1 On federally funded projects, FAA approval is required for the use of a bonded overlay. Bonded overlays should only be considered when the existing rigid pavement is in good to excellent condition. Defects in the existing pavement are more likely to reflect through a bonded overlay than other types of concrete overlays. Good surface preparation and construction techniques are required to ensure a good bond.

4.10.2.2 The new section behaves as a monolithic slab by bonding the concrete overlay to the existing rigid pavement. In FAARFIELD, a bonded overlay can be designed as a new rigid pavement, treating the existing concrete surface and the concrete overlay as a single layer. The flexural strength used in the FAARFIELD computation should be the strength of the existing concrete. The thickness of the bonded overlay required is computed by subtracting the thickness of the existing pavement from the total thickness of the required slab as computed by FAARFIELD.

4.10.3 Jointing of Concrete Overlays.

4.10.3.1 Some modification to jointing criteria in paragraph 3.14.8 may be necessary because of the design and joint arrangement of the existing pavement. Unbonded concrete overlays constructed on existing rigid pavements should meet the joint spacing requirements of paragraph 3.14.12, based on the overlay slab thickness. Joints in bonded overlays should be located within 0.5 inch (13 mm) of joints in the existing base pavement.

4.10.3.2 The following may be used as a guide in the design and layout of joints in concrete overlays. Joints do not need to be the same type as in the old pavement except for some bonded overlay applications. It is not necessary to provide an isolation joint for each isolation joint in the old pavement; however, a saw cut or plane of weakness should be provided within 1 foot (0.3 m) of the existing isolation joint.

- The timing for sawing joints is extremely critical on concrete overlays to minimize the curling and warping stresses and prevent random cracking.
- Contraction joints in unbonded overlays must be placed approximately over but within 1 foot (0.3 m) of existing isolation, construction, or contraction joints. Should spacing result in slabs too long to control cracking, additional intermediate contraction joints are necessary.
- Overlay slabs longer or wider than 20 feet (6.1 m) should contain reinforcing steel regardless of overlay thickness. Reinforcement may be required any time that overlay joint spacing is different than the underlying existing slab joint spacing.

4.10.4 Previously Overlaid Rigid Pavement.

The design of an HMA overlay for a rigid pavement that already has an existing HMA overlay must consider many factors, including the condition and thickness of each layer of the existing HMA overlay. Depending on the existing HMA material condition and pavement grades, the HMA may require some or complete milling. The designer should treat the problem as if the existing HMA overlay were not present, calculate the overlay thickness required, and then adjust the calculated thickness to compensate for the existing overlay. Inconsistent results will often be produced if this procedure is not used. Use engineering judgment to determine the condition of the rigid pavement.

4.10.5 Treatment of Thick HMA Overlays on Existing Rigid Pavements.

For HMA overlay thickness, FAARFIELD assumes the existing rigid pavement supports load through flexural action. As the overlay thickness increases, the existing rigid pavement will tend to act more like a high quality base material. As the overlay thickness approaches the thickness of the rigid pavement, it may be more economical to treat the design as a new flexible pavement design on a high quality base material. In FAARFIELD options screen, one of the general options is to allow flexible computation for Thick Overlays. If this option is selected, FAARFIELD will do both computations and report out the thinner HMA overlay.

4.11 Alternatives for Rehabilitation of Existing Pavement.

4.11.1 An evaluation of the condition of the existing pavement will assist in the determination of what rehabilitation alternatives should be considered. For example, if the condition of the existing rigid pavement is very poor (i.e., extensive structural cracking, joint faulting, "D" cracking, etc.), rubblization may not be appropriate.

4.11.2 In addition to the previously discussed flexible and rigid overlays, the following may also be considered:

4.11.2.1 Full-Depth Reclamation (FDR) of In-Place HMA.

4.11.2.1.1 This technique consists of pulverizing the full pavement section and may include mixing in a stabilization agent (fly ash, cement, emulsified or foamed asphalt), leveling, and compacting the reclaimed material layers into a uniform, base layer prior to placement of additional structural layer(s). The quality and quantity of the material being recycled combined with the composition of traffic to be accommodated will determine the number and type of additional structural layers.

4.11.2.1.2 At smaller general aviation airports it is conceivable that a surface layer of HMA or PCC could be placed directly upon the recycled base. However at larger airports a crushed aggregate base as well as a stabilized base may be required with the new surface layer.

4.11.2.1.3 In FAARFIELD the FDR layer may be modeled as a 'User Defined' layer with recommended modulus values ranging from 25,000-psi to 50,000-psi, higher values must be supported with in-place field testing. Engineering judgment is required for the selection of an appropriate modulus value for the FDR layer.

4.11.2.1.4 The FAA is investigating the strength of FDR materials at the National Airfield Pavement Material Research Center (NAPMRC) and plan to release a specification regarding FDR when this research is complete. Until the FAA develops a standard specification for FDR, on federally funded projects the use of FDR requires a MOS in accordance with FAA Order 5100.1.

4.11.2.2 Rubblization of Existing PCC Pavement.

4.11.2.2.1 Rubblization of existing PCC may be effective in mitigating reflective cracking. Using this process, the section is designed as a flexible pavement, treating the broken rigid pavement as base course. Reflective cracking is reduced or eliminated.

4.11.2.2.2 Rubblization of deteriorated concrete pavements is a method of pavement rehabilitation. The rubblization process eliminates the slab action by breaking the concrete slab into 1- to 3-inch (25- to 75-mm) pieces at the top and 3- to 15-inch (75- to 381-mm) pieces at the bottom. The rubblized concrete layer behaves as a tightly interlocked, high-density, non-stabilized base, which prevents the formation of reflective cracks in the overlay.

4.11.2.2.3 The thickness design procedure for an overlay over a rubblized concrete base is similar to the design of a new flexible or rigid pavement design. In FAARFIELD rubblized PCC layer may be modeled as a 'User Defined' layer with recommended modulus values ranging from 100,000-psi to 400,000-psi. Engineering judgment is required for the selection of an appropriate modulus value for the rubblized PCC layer.

4.11.2.2.4 The following ranges are suggested for selecting a design modulus value of rubblized PCC on airfields:

- For slabs 6 to 8 inches thick: Moduli from 100,000 to 135,000 psi
- For slabs 8 to 14 inches thick: Moduli from 135,000 to 235,000 psi
- For slabs greater than 14 inches thick: Moduli from 235,000 to 400,000 psi

4.11.2.2.5 The selected value is influenced by many factors, including the slab thickness and strength of the layer being rubblized, the condition and type of subbase and subgrade materials, and rubblized particle sizes. Reference AAPTP 04-01, *Development of Guidelines for Rubblization*, and Engineering Brief 66, *Rubblized Portland Cement Concrete Base Course*, for further information.

4.11.2.2.6 Note: Subsurface drainage for rubblized layers must be provided. In the AAPTP 04-01 report, it is recommended that edge drains be installed prior to rubblization.

Note: See EB 66 Rubblized PCC base course for additional guidance on rubblization.

4.11.2.3 **Crack and Seat.**

The crack and seat process involves cracking a PCC layer into pieces typically measuring 1.5 to 2 feet (0.46 m to 0.6 m) and firmly seating the pieces into the subgrade prior to overlaying with asphalt concrete. Crack and seat has generally been replaced by rubblization methods. Coordinate with FAA regarding the use of rubblization or crack and seat techniques during the design phase on federally funded projects.

4.11.2.4 **Pavement Interlayers.**

4.11.2.4.1 In general the FAA does not recommend the use of pavement interlayers. The use of interlayers does not eliminate the need to fill cracks in existing pavement. Pavement Interlayers may retard reflective cracking in limited applications, but should be compared to providing additional thickness of HMA. Pavement interlayers are located immediately on top of the surface being overlayed. Interlayers may be: an aggregate binder course; stress absorbing membrane interlayer (SAMI); paving fabric; a grid; or a combination.

4.11.2.4.2 Pavement interlayers must not be considered when existing pavements, flexible or rigid, show evidence of excessive deflections, substantial thermal stresses, and/or poor drainage. In addition, interlayers may impede future maintenance or rehabilitation.

4.11.2.4.3 Paving fabrics may provide limited waterproofing capability when overlaying full depth asphalt pavement structures. The paving fabrics provide some degree of water protection of the existing pavement subgrade. However the fabric may trap water in the upper layers of the pavement structure leading to premature surface deterioration and/or stripping.

4.11.2.4.4 FAARFIELD does not attribute any structural benefits to pavement for any type of interlayers in HMA thickness design. On federally funded projects, the pavement engineer must evaluate the cost and benefits of an interlayer versus additional HMA thickness.

4.12 Preparation of the Existing Pavement Surface for an Overlay.

Before proceeding with construction of an overlay, correct defective areas in the existing surface, base, subbase, and subgrade. If not corrected, deficiencies in the base pavement will often be reflected in an overlay. Refer to AC 150/5370-10, Item P-101, Surface Preparation, and AC 150/5380-6, *Guidelines and Procedures for Maintenance of Airport Pavements*, for additional information on pavement repair methods and procedures.

4.12.1 <u>Flexible Pavements</u>.

Distresses in flexible pavements typically consist of: pavement cracking, disintegration, and distortion. Refer to AC 150/5380-6 for additional guidance on pavement distresses.

4.12.1.1 **Patching.**

Remove localized areas of distressed and failed pavement and replace with new HMA. Failures usually occur when the pavement is deficient in thickness, the subgrade consists of unstable material, or poor drainage has reduced subgrade support. To correct this condition, the subgrade material should be replaced with a select subgrade soil or proper drainage facilities installed. Following the correction of the subgrade condition, the subbase, base, and surface courses must be placed and compacted.

4.12.1.2 **Milling.**

Surface irregularities and depressions, such as shoving, rutting, scattered areas of settlement, "birdbaths," and bleeding should be corrected by milling and by leveling with suitable HMA mixtures. The leveling course should consist of high-quality HMA. See AC 150/5370-10 P-401 or P-403.

4.12.1.3 **Cracks and Joints.**

Repair cracks and joints in accordance with P-101, Surface Preparation. Refer to AC 150/5380-6 for additional guidance on crack and joint repair.

4.12.1.4 **Grooves.**

It is generally not necessary to remove existing pavement grooves prior to an asphalt or concrete overlay, unless the grooves are exhibiting other irregularities such as shoving, rutting or other types of pavement distress.

4.12.1.5 **Porous Friction Courses (PFC).**

Existing PFCs must be removed prior to any overlay.

4.12.1.6 **Paint and Surface Contaminants.**

Paint must be removed or scarified prior to an asphalt overlay to ensure bonding of the overlay to the existing pavement. Surface contaminants that will prevent bonding of the surface overlay (e.g., rubber, oil spills, etc.) must be removed prior to an asphalt overlay.

4.12.2 Rigid Pavements.

Narrow transverse, longitudinal, and corner cracks need no special attention unless there is a significant amount of displacement and faulting between the separate slabs. If the subgrade is stable and no pumping has occurred, the low areas can be addressed as part of the overlay and no other corrective measures are needed. If pumping has occurred at the slab ends or the slabs are subject to rocking under the movement of airplanes, subgrade support may be improved by chemical or cement grout injection to fill the voids that have developed. Grouting is a specialized technique that must be done under the direction of an experienced pavement or geotechnical engineer.

4.12.2.1 **Broken and Unstable Slabs.**

Localized replacement of broken slabs may be required before starting construction of an overlay. However, badly broken and unstable pavement slabs due to uneven bearing on the subgrade can also be broken into smaller pieces to obtain a firmer seating. Crack and seat procedures may be used when broken and unstable slabs are extensive throughout the pavement area. Refer to AAPTP 05-04, *Techniques for Mitigation of Reflective Cracks*, for additional information.

4.12.2.2 **Leveling Course.**

When the existing pavement is uneven due to slab distortion, faulting, settlement, or after a crack and seat procedure, an HMA leveling course may be required.

4.12.2.3 **Cracks and Joints.**

Repair cracks and joints in accordance with P-101, Surface Preparation. Refer to AC 150/5380-6 for additional guidance on crack and joint repair.

4.12.2.4 **Surface Cleaning.**

The pavement surface should be swept clean of all dirt, dust, and foreign material after all repairs have been completed and prior to the placing of the overlay. Any excess joint-sealing material should be trimmed from rigid pavements. Paint does not require removal prior to an unbonded concrete overlay.

4.12.3 Bonded Concrete Overlays.

The bond between existing concrete and a concrete overlay is extremely difficult to achieve and special attention is required to ensure the bond with the existing pavement. Surface cleaning and preparation by shot peening or mechanical texturing by cold milling have been used successfully to achieve an adequate bonding surface. A bonding agent may be required on the prepared surface immediately ahead of the overlay placement to achieve a bond. For federally funded projects, FAA approval is required prior to the design of a bonded PCC overlay.

4.12.4 Materials and Methods.

AC 150/5370-10, *Standards for Specifying Construction of Airports*, specifies quality of materials and mixes, control tests, methods of construction, and workmanship for pavement materials. For federally funded projects, use of materials other than PCC pavement (Item P-501) or HMA pavement (Item P-401) requires FAA approval.

CHAPTER 5. PAVEMENT STRUCTURAL EVALUATION

5.1 Purposes of Structural Evaluation.

This chapter covers the structural evaluation of pavements for all weights of airplanes. Airport pavement and structure (e.g., bridge, culvert, storm drain) evaluations are necessary to assess the ability of an existing pavement to support different types, weights, and volumes of airplane traffic and for use in the planning and design of improvements to the airport. When visual inspection indicates structural distresses, the pavement strength may not be adequate and physical testing may be required. See AC 150/5380-7, *Airport Pavement Management Program (PMP)*, for information on visual inspection and evaluation of pavement condition and pavement management.

5.2 Evaluation Process.

The structural evaluation of airport pavements is a methodical step-by-step process. The steps described below should be used for all pavements.

5.2.1 Records Research.

Perform a thorough review of construction data and history, design considerations, specifications, testing methods and results, as-built drawings, and maintenance history. Weather records and the most complete traffic history available are also part of a usable records file. Review the data in the current Pavement Management Program (PMP) developed in accordance with AC 150/5380-7.

5.2.2 Site Inspection.

The site should be visited and the condition of the pavements noted by visual inspection. This should include, in addition to the inspection of the pavements, an examination of the existing drainage conditions and drainage structures at the site. Note any evidence of the adverse effects of frost action, swelling soils, reactive aggregates, etc. Refer to Chapter 2 and AC 150/5320-5, *Surface Drainage Design,* for additional information on soil, frost, and drainage, respectively. Refer to ASTM D 5340, *Standard Test Method for Airport Pavement Condition Index Surveys,* on conducting a visual survey of pavements.

5.2.3 Pavement Condition Index.

The Pavement Condition Index (PCI) is a useful tool for evaluating airport pavements. The PCI is a numerical rating of the surface condition of a pavement and indicates functional performance with implications of structural performance. PCI values range from 100 for a pavement with no defects to 0 for a pavement with no remaining functional life. The index can serve as a common basis for describing pavement distresses and comparing pavements. ASTM D 5340 provides recommendations on conducting a PCI survey. Computer pavement management programs such as MicroPAVER or FAA PAVEAIR can be utilized to calculate a PCI.

5.2.4 Sampling and Testing.

The site inspection, records search, and reason for evaluation will determine the need for physical tests and materials analyses. A material evaluation for the design of an individual project will require more sampling and testing than an evaluation performed for a network analysis of the pavements at an airport. Sampling and testing provides information on the thickness, quality, and general condition of the existing pavement structure and materials.

5.2.4.1 **Direct Sampling.**

The basic evaluations consist of visual inspections with supplemental sampling and testing as needed. For a relatively new pavement constructed to FAA standards with no visible sign of wear or stress, information may be based on data as shown on the as-built sections for the most recent project.

5.2.4.2 **Grade and Roughness Assessment.**

An assessment of the pavements' roughness level is a reflection of its serviceability. Profile measurements that capture the profile of the pavement, including all grade changes, allow for a variety of roughness assessment methods. Pavement profiles may be evaluated with programs such as ProFAA. The FAA is currently researching different measures for the evaluation of in-service pavement roughness. Upon completing this research, the FAA will update guidance on airport pavement roughness. See AC 150/5380-9, *Guidelines and Procedures for Measuring Airfield Pavement Roughness.*

5.2.4.3 **Nondestructive Testing (NDT) Using Falling Weight Deflectometer and Heavy Falling Weight Deflectometer.**

NDT refers to any test method that does not involve removal or destruction of pavement material. The major advantages of NDT are the pavement is tested in place under actual conditions of moisture, density, etc.; the disruption of traffic is minimal; and the need for destructive tests is minimized. The most common NDT tools available to assist the evaluator include the Falling Weight Deflectometer (FWD) and Heavy Falling Weight Deflectometer (HWD). NDT using FWD or HWD, consists of observing pavement response to a controlled dynamic load. Appendix C contains additional guidance on using these tools

5.2.4.4 **NDT – Other Methods.**

5.2.4.4.1 Ground Penetrating Radar.

Ground penetrating radar is a nondestructive testing procedure that can also be used to study subsurface conditions. Ground penetrating radar depends on differences in dielectric constants to discriminate between materials. The technique is sometimes used to locate voids or foreign objects, such as abandoned fuel tanks and tree stumps, under pavements and embankments.

5.2.4.4.2 Infrared Thermography.

Infrared thermography is a nondestructive testing procedure where differences in infrared emissions are observed to determine certain physical properties of the pavement.

5.2.5 Evaluation Report.

5.2.5.1 The analyses, findings, and test results should be incorporated into an evaluation report, which becomes a permanent record for future reference. Evaluation reports can be in any form, but the FAA recommends it include a drawing identifying limits of the evaluation. Analysis of information should culminate in the assignment of load carrying capacity to the pavement sections under consideration.

5.2.5.2 The evaluation should also consider any impacts frost action may have on the pavement structure. Frost evaluations include review of soil, moisture, and weather conditions conducive to detrimental frost action. Frost action may result in reduction in the load capacity of the pavement structure.

5.3 Flexible Pavements.

Evaluation of existing flexible pavement structures requires, at a minimum:

- the determination of the thickness of the component layers and
- the strength of the subgrade, expressed as CBR or modulus (*E*).

5.3.1 Layer Thicknesses.

Determine layer thicknesses from borings, or as-built drawings and records.

5.3.2 Subgrade CBR.

5.3.2.1 **Laboratory or Field CBR.**

Laboratory CBR tests should be performed on soaked specimens in accordance with ASTM D 1883, *Standard Test Method for California Bearing Ratio (CBR) of Laboratory-Compacted Soils.* Perform field CBRs in accordance with ASTM D 4429, *Standard Test Method for CBR (California Bearing Ratio) of Soils in Place.* Field CBR tests on existing pavements less than 3 years old may not be representative unless the subgrade moisture content has stabilized.

5.3.2.2 **Back-calculate Modulus from NDT.**

Where it is impractical to perform laboratory or field CBR tests, a back-calculated subgrade elastic modulus value may be obtained from NDT test results. Appendix C, paragraph C.12, gives the procedures for obtaining the back-calculated modulus value. The back-calculated modulus value should be input directly into FAARFIELD without manually converting to CBR.

5.3.3 Layer Properties.

The materials in FAARFIELD are designated by corresponding FAA specifications. For example, where an existing flexible pavement consists of an HMA surface on a high-quality crushed aggregate base meeting FAA Item P-209, the base layer should be input as P-209 Crushed Aggregate in FAARFIELD. Where the quality of materials in a pavement structure to be evaluated differ significantly from the assumptions for FAA standard materials as given in AC 150/5370-10, it may be necessary to use the “User-defined” or “variable” layer types in FAARFIELD to input an appropriate modulus value. FAARFIELD allows an unlimited number of layers beneath the HMA surface; however, evaluation of more than 5 layers is not recommended.

5.3.4 Example of Flexible Pavement Evaluation Procedures.

1. After establishing evaluation parameters for the existing flexible pavement, use an evaluation process that is essentially the reverse of the design procedure. FAARFIELD can be used to determine the structural life of the existing pavement for a given traffic mix or alternatively, the pavement structure that will produce a 20-year life for a given traffic mix. Required inputs are the subgrade CBR or modulus value, thicknesses of surfacing, base and subbase courses, and annual departure levels for all airplanes using the pavement.

Assume an existing taxiway pavement was constructed to FAA standards and consists of the following pavement structure (Figure 5-1):

Thickness	Pavement Structure
4 inches	P-401 HMA Surface Course
5 inches	P-401/403 Stabilized Base Course
12 inches	P-209 Crushed Aggregated Base Course
10 inches	P-154 Subbase Course
	Subgrade, CBR = 5

The taxiway will serve the following mix of airplanes:

Airplane	Gross Weight (lbs)	Annual Departures
B737-800	174,700	3,000
A321-200 opt	207,014	2,500
EMB -195 STD	107,916	4,500
Regional Jet - 700	72,500	3,500

2. FAARFIELD will be used to determine the available structural life based on the above traffic mixture. Both subgrade CDF and HMA CDF will be checked.

3. The following steps are used:

   a. In the Structure screen, enter the layer thickness and material type for each layer.

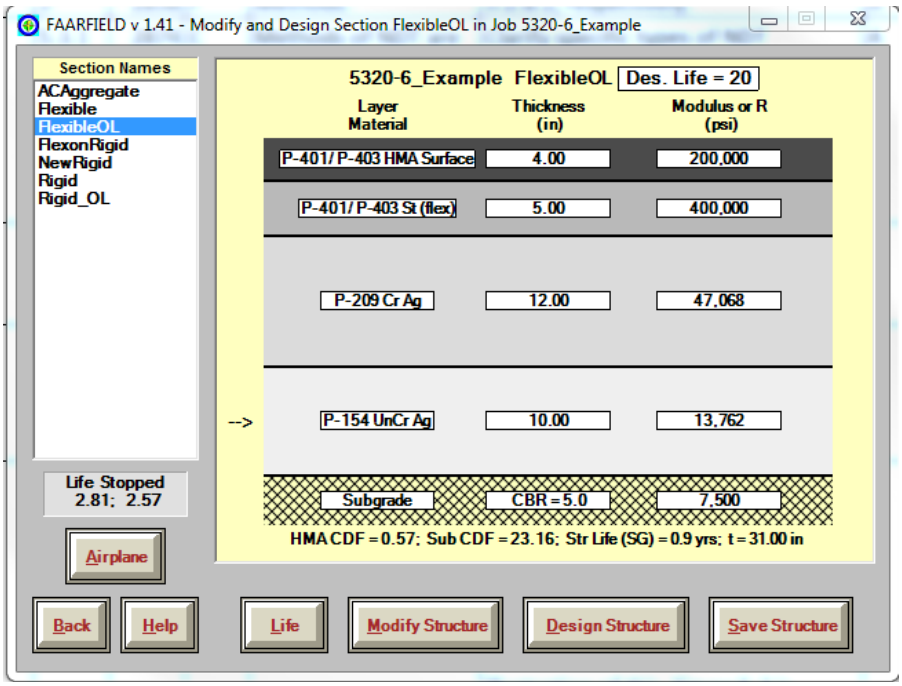
      Note: Since the pavement layers were constructed to FAA standards, the corresponding standard material types can be used for each layer.

   b. Enter the above airplane list using the Airplanes screen.

   c. On the Options screen, ensure the "HMA CDF" option is selected.

   d. Click the "Life" button.

Figure 5-1. Existing Taxiway Pavement Structure



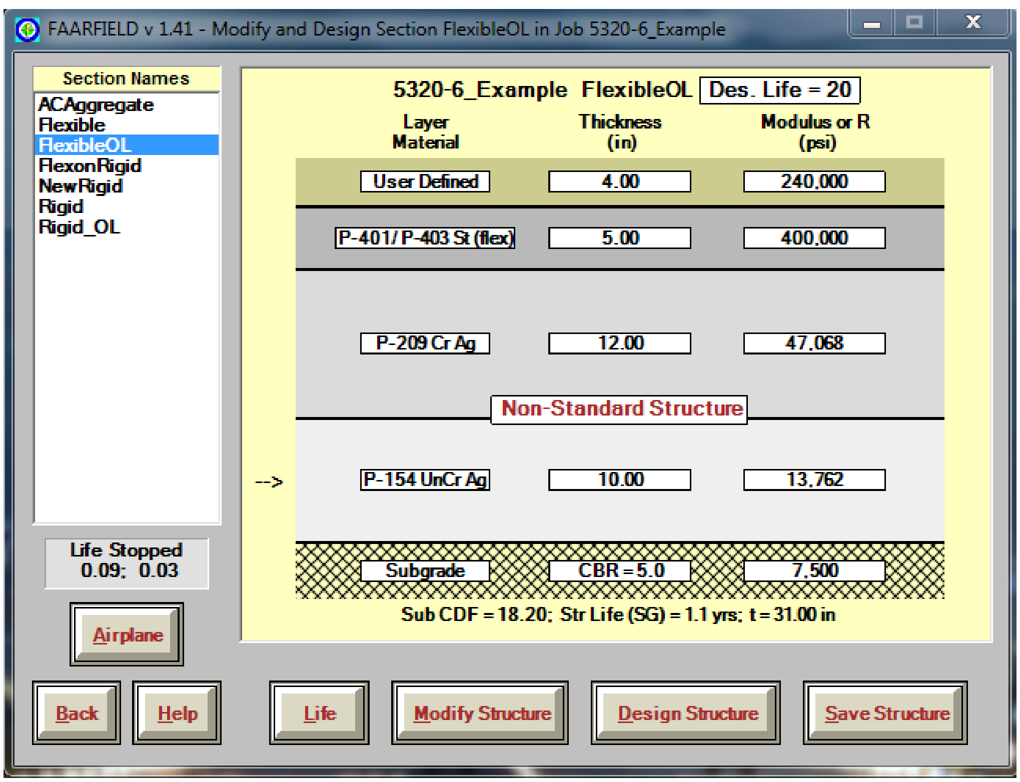
4. This evaluation indicates an overlay is recommended to support the given traffic mix. The computed value of subgrade CDF (Sub CDF) is 23.16, which is greater than 1 indicating that the structure has insufficient thickness to protect the subgrade for the given traffic for the design life being evaluated. The predicted structural fatigue life for the given structure and traffic loading is 0.9 years. This predicted structural life is based on the subgrade failure criteria. FAARFIELD also reports the HMA CDF value is 0.57. Although this value is less than 1.0, it is relatively high, indicating the HMA surface may be at risk of fatigue cracking. The procedures in Chapter 4 should be used to design the required overlay thickness.

5. The above example assumes that all layers were constructed to FAA standards. Often it is necessary to rely on NDT or other methods for layer characterization as

described in paragraph 5.2.4, since it is not known what materials were used to construct the pavement section. The User-Defined layer should be used to represent structural layers that deviate significantly from standard materials.

Note: Depending on the location of the layer being characterized, even large deviations from the standard material modulus values in FAARFIELD may have a relatively minor effect on the predicted structural life. As an illustration of this, Figure 5-2 is similar to Figure 5-1, except that the HMA surface has now been replaced with a User-Defined layer with an E = 240,000 psi (1,655 MPa). In this case increasing the modulus by 20 percent only slightly increases the predicted structural life, from 0.9 years to 1.1 years. Considering the variability inherent in the FAARFIELD design model, as well as the uncertainties associated with the other input data (future traffic levels, aircraft weights, subgrade CBR, etc.), this small increase in predicted life should not be considered significant.

Figure 5-2. Existing Taxiway Structure with User-Defined Surface Layer

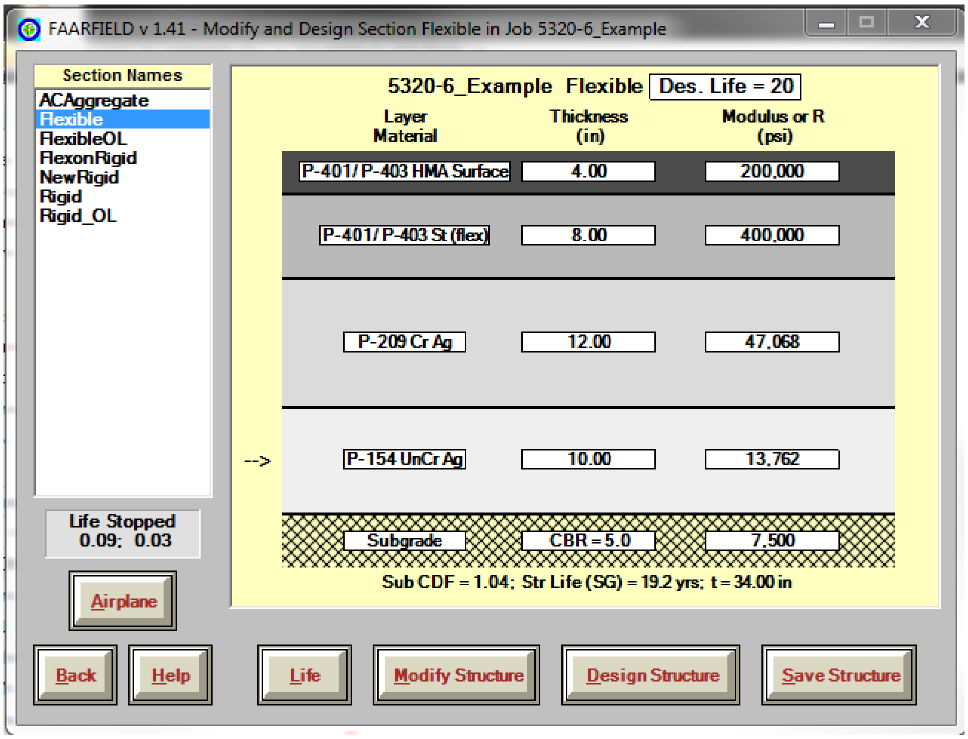


"

5.4 Overlay Requirement.

If an evaluation shows that the existing structure is deficient, typically the next step would be to determine how much additional surfacing is required to support the current traffic mix (an overlay design). Design of an overlay is an iterative process that considers various surface thicknesses. For example, milling 1 inch (25 mm) of the existing surface and adding 4 inches (100 mm) of P-401/403 will provide a structural fatigue life of 19.2 years (see Figure 5-3). Note in this example the remaining existing 3-inch (75-mm) surface course and 5-inch (125-mm) stabilized base is modeled as an 8-inch (200-mm) stabilized base layer. Information from NDT testing may be used to model the existing layers as user-defined layers in a FAARFIELD overlay design.

Figure 5-3. Flexible Pavement Evaluation



5.5 Rigid Pavements.

Evaluation of rigid pavements requires, at a minimum:

- the thickness of the component layers,
- the flexural strength of the concrete, and
- the modulus of the subbase and subgrade.

5.5.1 Layer Thicknesses.

Determine thicknesses from borings, cores, or as-built records of the pavement.

5.5.2 Concrete Flexural Strength.

5.5.2.1 Construction records or NDT data are typically used as the source for concrete flexural strength data. Construction strength data of the concrete may need to be adjusted upward to account for concrete strength gain with age. Correlations between flexural strength and other strength tests are approximate and considerable variations are likely.

5.5.2.2 ASTM C 496, *Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens*, provides an approximate relationship between concrete flexural strength and tensile splitting strengths, which is given by the following formula:

$$R = 1.02\ (T) + 117\ \text{psi} \quad \text{or} \quad 1.02\ (T) + 0.81\ \text{MPa}$$

R = flexural strength, psi (MPa)
T = tensile split strength, psi (MPa)

5.5.3 Subgrade Modulus.

5.5.3.1 Construction records or NDT data are typically used as for subgrade modulus. A back-calculated subgrade elastic modulus value may be obtained from NDT test results. Appendix C gives the procedures for obtaining the back calculated modulus value.

5.5.3.2 The modulus of subgrade reaction, k, can be determined by plate bearing tests performed on the subgrade in accordance with the procedures established in AASHTO T 222 but is more commonly obtained from NDT test procedures such as FWD or HWD. (See Appendix C.)

5.5.4 Back Calculated E Modulus Value or k Value in FAARFIELD.

5.5.4.1 The back-calculated E modulus value or k value can be input directly into FAARFIELD. If a back-calculated k-value is used, FAARFIELD will convert it to an E-modulus using the formula given in paragraph 3.14.4.

5.5.4.2 FAARFIELD materials are designated by corresponding FAA specifications. Where the quality of materials in a pavement structure to be evaluated differs significantly from the assumptions for FAA standard in AC 150/5370-10, it may be necessary to use the “undefined” or “variable” layer types in FAARFIELD to input an appropriate modulus value. In FAARFIELD, the number of structural layers above the subgrade for a rigid pavement is limited to 4, including the PCC surface. If the actual rigid pavement structure evaluated consists of more than 4 distinct layers, two or more of the lower layers must be combined to reduce the total number of layers to 4 or fewer for analysis. Rigid pavement evaluation is not highly sensitive to modulus properties of lower layers above the subgrade and the life computation should not be significantly affected.

5.5.5 Example of Rigid Pavement Evaluation Procedures.

5.5.5.1 FAARFIELD can be used to determine the remaining structural life of an existing pavement for a given traffic mix. A concrete-surfaced taxiway that was designed for a 20-year life called for the following pavement structure.

Pavement structure:

Layer Thickness	Pavement Structure
16.46 inches	P-501 PCC Surface Course(R = 650)
6 inches	P-304 Cement-treated Base Course
12 inches	P-209 Base Course
	Subgrade, E = 15,000 psi

For the following airplane traffic mix:

Airplane Name	Gross Weight (lbs)	Annual Departures
B737-800	174,700	3,000
A321-200 opt	207,014	2,500
EMB -195 STD	107,916	4,500
Regional Jet - 700	72,500	3,500

5.5.5.2 It is desired to estimate the remaining life considering the current increased traffic and as constructed layer properties. The subgrade was evaluated by NDT and found to have an *E*-modulus of approximately 7500 psi (52 MPa). Based upon cores taken on the taxiway the in place layer properties for the pavement structure are as follows:

Pavement structure:

Thickness	Pavement Structure
17.25 inches	P-501 PCC Surface Course(R = 685)
6 inches	P-304 Cement-treated Base Course
12 inches	P-209 Base Course
	Subgrade, E = 7500 psi

. The airplane traffic mix using the taxiway has changed, and now consists of the following traffic mix:

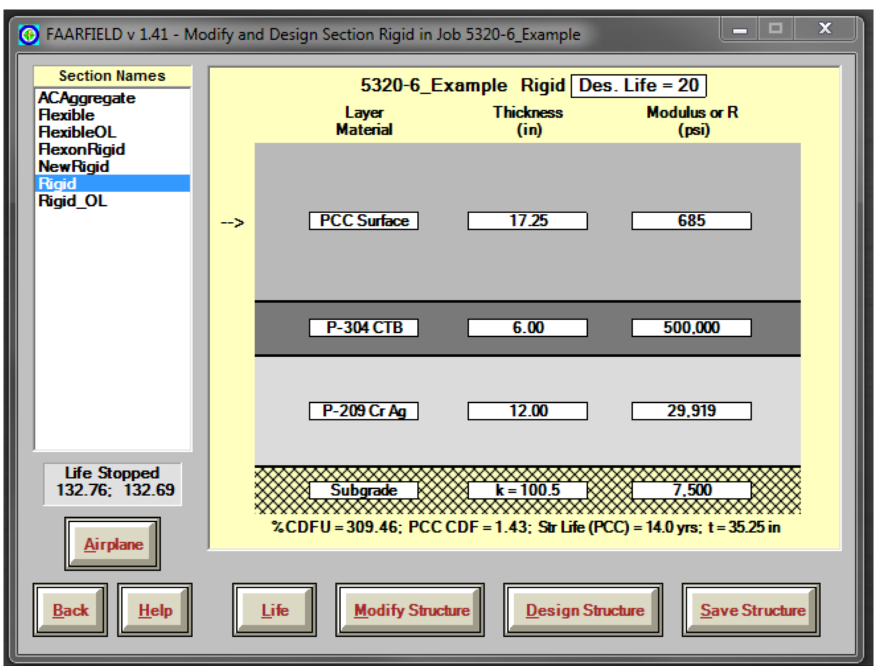
Airplane	Gross Weight (lbs)	Annual Departures
B737-800	174,700	3,000
A321-200 opt	207,014	2,500

Airplane	Gross Weight (lbs)	Annual Departures
EMB-195 STD	107,916	4,500
Regional Jet – 700	72,500	3,500
A380	1,238,998	50
B777-300 ER	777,000	50

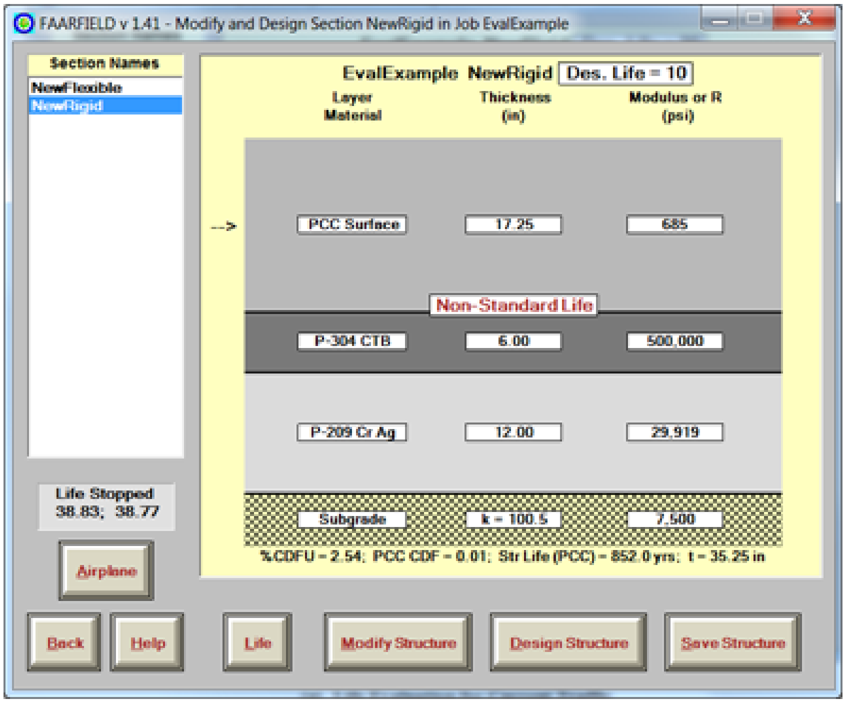
5.5.5.3 A life evaluation of the current pavement structure indicates a remaining structural fatigue life of 14.0 years with the new traffic mix (Figure 5-3). Strictly speaking this is the total life, not the remaining life, because the FAARFIELD Life calculation ignores any fatigue life consumed up to the point that the traffic changed. (In this example, a FAARFIELD Life analysis of the existing pavement with the original traffic indicates that after 10 years of service that % CDFU is only about 2.5% (Figure 5-4 b). This supports that it is reasonable to ignore the contribution of the earlier traffic, and that the total life computed by FAARFIELD can be considered the remaining life of the structure under the current traffic. Actual pavement performance will be impacted by any future changes in the airplane fleet mix composition and actual operating weights. The taxiway pavement should be monitored over time with regular pavement inspections.

Figure 5-4. Rigid Pavement Evaluation

a. Life Evaluation for Current Traffic



b. Life Evaluation for Original Traffic



5.6 **Use of Results.**

If the evaluation is being used for planning purposes and the existing pavement is found to be deficient relative to the design standards given in Chapter 3, the airport owner should be notified of the deficiency. The airport owner should then develop a corrective action plan of how it plans to address the deficiency (e.g., strengthen pavement and/or limit activity) and include this in the airport's capital improvement plan. If the evaluation is being used as part of a design for a project to reconstruct or upgrade the facility, the procedures given in Chapter 3 or Chapter 4 should be used to design the reconstruction or overlay project. In this case, the main concern is not the load-carrying capacity but the difference between the existing pavement structure and the new pavement structure needed to support the revised forecast traffic.

5.7 **Reporting Pavement Weight Bearing Strength.**

5.7.1 Aircraft Classification Number/Pavement Classification Number (ACN/PCN).

5.7.1.1 The International Civil Aviation Organization (ICAO) has a standardized method of reporting airport pavement weight bearing strength known as Aircraft Classification Number/Pavement Classification Number (ACN/PCN). This method of reporting is based on the concept of reporting strength in terms of a standardized equivalent single wheel load. The FAA has developed a software program, COMFAA, which may be used to compute PCN. AC 150/5335-5, *Standardized Method of*

Reporting Airport Pavement Strength – PCN, provides guidance on using the COMFAA software and on calculating and reporting PCN.

5.7.1.2 Report the PCN code to the appropriate regional FAA Airports Division, either in writing or as part of the annual update to the Airport Master Record, FAA Form 5010-l.

CHAPTER 6. PAVEMENT DESIGN FOR SHOULDERS

6.1 Purpose.

6.1.1 This chapter provides the FAA design procedure for paved airfield shoulders. Note blast pads and stopways may be designed following these same procedures.

6.1.2 Paved or surfaced shoulders provide resistance to erosion and debris generation from jet blast. Jet blast can cause erosion of unprotected soil immediately adjacent to airfield pavements. The shoulder must be capable of safely supporting the occasional passage of the most airplanes as well as emergency and maintenance vehicles.

6.1.3 Paved shoulders are required for runways, taxiways, taxilanes, and aprons accommodating Airplane Design Group (ADG) IV and higher aircraft and are recommended for runways accommodating ADG III aircraft. For shoulders adjacent to runways accommodating only ADG I and ADG II aircraft, the following surface types are recommended: turf, aggregate-turf, soil cement, lime, or bituminous stabilized soil. Refer to AC 150/5300-13 for standards and recommendations for airport design.

6.2 Shoulder Design.

6.2.1 Shoulders are designed to accommodate the most demanding of (1) a total of 15 fully loaded passes of the most demanding airplane or (2) anticipated traffic from airport maintenance vehicles. Minimum shoulder pavement layer thicknesses are given in Table 6-1. Shoulder pavement thicknesses are designed to allow safe operation of an airplane on an emergency basis across the paved shoulder area without damage to the airplane. Flexible shoulder pavement sections may experience noticeable vertical movements with each passage of an airplane and may require inspection and/or limited repair after each airplane operation. Rigid shoulder pavement sections may experience cracking after each airplane operation.

6.2.2 Drainage from the adjacent airfield pavement base and subbase must be considered when establishing the total thickness of the shoulder pavement section. A thicker shoulder section than structurally required and edge drains may be necessary to avoid trapping water under the airfield pavement. Typically this is accomplished by using minimum base/subbase on the outer edge and tapering back to match with the base/subbase under the adjacent runway pavement. AC 150/5320-5, *Airport Drainage Design*, provides additional guidance on drainage requirements.

6.2.3 Shoulder pavement thickness is determined using the FAARFIELD design software. Because the pavement is not intended to carry regular aircraft traffic, a complete traffic mixture is not considered. Instead the airplane requiring the thickest pavement section is used to determine the pavement shoulder thickness. As described in the procedure below, it is not necessary to perform a separate design for each airplane in the traffic mix. Rather, several airplanes with the largest contribution to the CDF should be evaluated to determine which is the most demanding for shoulder design. Aircraft

Rescue and Firefighting (ARFF), maintenance, and snow removal vehicles that operate on the shoulder should be considered separate of the aircraft in shoulder thickness pavement design.

6.2.4 The following steps are used for the shoulder design procedure:

Step 1: Create a new job file in FAARFIELD with the proposed pavement section for the shoulder design. Include all desired pavement layers, e.g., surface course, base course, stabilized course, subbase course, etc. Layer thickness should meet minimum thickness requirements for shoulder design.

Note: It may be necessary to use the User Defined pavement layer to represent the proposed shoulder pavement cross-section because of the minimum shoulder pavement layer thickness requirements.

Step 2: Input all airplanes from the traffic mixture and set annual departures to 1,200 annual departures. From the FAARFIELD Structure screen, click the “Life” button. Return to the airplane mixture, and scroll over to the column labeled “CDF Max for Airplanes”. In most instances, the airplane with the highest CDF Max value will be the most demanding airplane and will control the shoulder pavement design. However, the top few airplanes with high CDF max values should be evaluated because the thickness of the pavement section will influence which aircraft is the most demanding.

Step 3: Return to the FAARFIELD Airplane screen and clear the traffic mixture except for the most demanding airplane to be used to design the shoulder pavement thickness. Adjust operating weight as appropriate.

Step 4: Change annual departures to 1 departure.

Step 5: Return to the Structure screen and confirm the design period is 15 years. The intent is to design a pavement for 15 total departures of the most demanding airplane or vehicle.

Step 6: Confirm the composition and thickness of pavement layers and that the correct layer is designated for thickness iteration. The iteration layer will be shown with a small arrow along the left side.

Step 7: Click on the “Design Structure” button to design the minimum pavement section for the individual airplane.

Step 8: Repeat Steps 3-7 for all airplanes with significant CDF max contributions in the traffic mixture. The design for the shoulder pavement is the pavement section with the greatest thickness requirement.

Note: A thicker shoulder section than structurally required and edge drains may be necessary to provide drainage from the adjacent airfield pavement base and subbase to avoid trapping water under the airfield pavement.

Step 9: Check shoulder pavement thickness requirements for ARFF, snow removal, and maintenance vehicles that operate at the airport. Return to the FAARFIELD Airplane screen and clear all airplanes from the traffic mix. Add vehicles from the "Non-Airplane Vehicles" group in the FAARFIELD internal airplane library, and adjust the gross weights as necessary. In place of "Annual Departures" for non-airplane vehicles, enter the number of annual operations on the shoulder pavement. Use the number of operations that will be expected and do not limit to 15. After adding all non-airplane vehicles to be considered, return to the Structure screen and click on the "Design Structure" button to design the pavement section.

Step 10: In areas prone to frost, check frost protection requirements as discussed in paragraph 6.4.

Step 11: The final shoulder thickness design will be the greatest of the thickness requirements for the most demanding airplane (Steps 3-7), non-airplane vehicle traffic, minimum layer thickness required for frost protection, or the minimum shoulder pavement layer thickness (Table 6-1).

6.3 Shoulder Material Requirements.

6.3.1 Asphalt Surface Course Materials.

The material should be of high quality, similar to FAA Item P-401/P-403, and compacted to an average target density of 93 percent of maximum theoretical density. See AC 150/5370-10, Item P-401 and Item P-403.

6.3.2 Portland Cement Concrete Surface Course Materials.

The material should be of high quality, similar to FAA Item P-501, with a minimum design flexural strength of 600 psi (4.14 MPa). See AC 150/5370-10, Item P-501.

6.3.3 Base Course Materials.

Base course materials must be high quality materials, similar to FAA Items P-208, P-209, P-301, or P-304. See AC 150/5370-10, Item P2-208, P-209, P-301 or P-304.

6.3.4 Subbase Course Materials.

Place subbase course material in accordance with AC 150/5370-10, Item P-154.

6.3.5 Subgrade Materials.

Prepare subgrade materials in accordance with AC 150/5370-10, Item P-152.

6.4 Shoulders Areas Susceptible to Frost Heave.

In areas prone to frost heave, it may be necessary to increase the thickness of the shoulder pavement to avoid differential frost heave. Additional thickness of the pavement beyond that necessary for structural design may be achieved with any material suitable for pavement construction. The material should possess a CBR value higher than the subgrade and have non-frost susceptible properties. Place the additional layer immediately on the subgrade surface below all base and subbase layers. The FAA recommends limited subgrade frost protection in accordance with paragraph 3.12.17.

6.5 Reporting Paved Shoulder Design.

Include FAARFIELD analysis as part of the Engineer's Design Report on federally funded projects.

Table 6-1. Minimum Shoulder Pavement Layer Thickness

Layer Type	FAA Specification Item	Minimum Thickness, in (mm)
HMA Surface	P-401, P-403	4.0 (100)
PCC Surface	P-501	6.0 (150)
Aggregate Base Course	P-209, P-208,	6.0 (150)1
Subbase (if needed)	P-154	4.0 (100)

Note:
1. Minimum thickness of aggregate base

APPENDIX A. SOIL CHARACTERISTICS PERTINENT TO PAVEMENT FOUNDATIONS

Major Divisions		Letter	Name	Value as Foundation When Not Subject to Frost Action	Value as Base Directly under Wearing Surface	Potential Frost Action	Shrink and Swell	Drainage Characteristic	Unit Dry Weight (pcf)	CBR	Subgrade Modulus *k* (pci)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Coarse-gravelly soils	Gravel and gravelly soils	GW	Gravel or sandy gravel, well graded	Excellent	Good	None to very slight	Almost none	Excellent	125-140	60-80	300 or more
		GP	Gravel or sandy gravel, poorly graded	Good	Poor to fair	None to very slight	Almost none	Excellent	120-130	35-60	300 or more
		GU	Gravel or sandy gravel, uniformly graded	Good to excellent	Poor	None to very slight	Almost none	Excellent	115-125	25-50	300 or more
		GM	Silty gravel or silty sandy gravel	Good	Fair to good	Slight to medium	Very slight	Fair to poor	130-145	40-80	300 or more
		GC	Clayey gravel or clayey sandy gravel	Good to excellent	Poor	Slight to medium	Slight	Poor to practically impervious	120-140	20-40	200-300
	Sand and sandy soils	SW	Sand or gravelly sand, well graded	Good	Poor to not suitable	None to very slight	Almost none	Excellent	110-130	20-40	200-300
		SP	Sand or gravelly sand, poorly graded	Fair to good	Not suitable	None to very slight	Almost none	Excellent	105-120	15-25	200-300
		SU	Sand or gravelly sand, Poor uniformly Not suitablegraded	Fair to good	Poor	None to very slight	Almost none	Excellent	100-115	10-20	200-300
		SM	Silty sand or silty gravelly sand	Good	Not suitable	Slight to high	Very slight	Fair to poor	120-135	20-40	200-300
		SC	Clayey sand or clayey gravelly sand	Fair to good	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	105-130	10-20	200-300
Fine grained soils	Low compress ibility LL<50	ML	Silts, sandy silts, gravelly silts, or diatomaceous soils	Fair to good	Not suitable	Medium to very high	Slight to medium	Fair to poor	100-125	5-15	100-200
		CL	Lean clays, sandy clays, or gravelly clays	Fair to good	Not suitable	Medium to very high	Medium	Practically impervious	100-125	5-15	100-200
		OL	Organic silts or lean organic clays	Poor	Not suitable	Medium to very high	Medium to high	Poor	90-105	4-8	100-200
	High compress ibility LL<50	MH	Micaceous clays or diatomaceous soils	Poor	Not suitable	Medium to very high	High	Fair to poor	80-100	4-8	100-200
		CH	Fat clays	Poor to very poor	Not suitable	Medium	High	Practically impervious	90-110	3-5	50-100
		OH	Fat organic clays	Poor to very poor	Not suitable	Medium	High	Practically impervious	80-105	3-5	50-100
Peat and other fibrous organic soils		Pt	Peat, humus and other	Not suitable	Not suitable	Slight	Very high	Fair to poor	-	-	-

Page Intentionally Blank

APPENDIX B. DESIGN OF STRUCTURES

B.1 Background.

Airport structures such as culverts and bridges are usually designed to last for the foreseeable future of the airport. Information concerning the landing gear arrangement of future heavy airplanes is speculative. It may be assumed with sufficient confidence that strengthening of pavements to accommodate future airplanes can be performed without undue problems. Strengthening of structures, however, may prove to be extremely difficult, costly, and time-consuming. Point loadings on some structures may be increased; while on overpasses, the entire airplane weight may be imposed on a deck span, pier, or footing. The location of the structure on the airfield will determine whether the most demanding load will be an aircraft or a vehicle, e.g., fuel truck or snow removal equipment.

B.2 Recommended Design Parameters.

B.2.1 Structural Considerations.

For many structures the design is highly dependent upon the airplane landing gear configuration. Design for the largest and heaviest airplane or vehicle at maximum gross weight that could use the airport over the life of the airport. Structural loads and design requirements (including applicable seismic design requirements) should be determined with reference to AASHTO Load and Resistance Factor Design (LRFD). Refer to the following publication for more information: AASHTO LRFD Bridge Design Specifications (7th edition).

B.2.2 Foundation Design.

Foundation design will vary with soil type and depth. No departure from accepted methodology is anticipated; except that for shallow structures, such as inlets and culverts, the concentrated loads may require heavier and wider spread footings than those provided by the structural standards in current use. For buried structures, such as culverts, the following guidance is recommended.

1. When the depth of fill is less than 2 feet, the wheel loads will be treated as concentrated loads.
2. When the depth of fill is 2 feet or more, wheel loads will be considered as uniformly distributed over a square with sides equal to l.75 times the depth of the fill. When such areas from several concentrations overlap, the total load will be uniformly distributed over the area defined by the outside limits of the individual areas, but the total width of distribution will not exceed the total width of the supporting slab.
3. For maximum wheel loads exceeding 25,000 lbs. (11,400 kg), perform a structural analysis to determine the distribution of wheel loads at the top of the buried structure. Consider the maximum wheel loads, tire pressures, and gear configuration that will act on top of the buried structure. The load distributions in

Item 1 or 2 (as applicable) may be assumed conservatively in lieu of performing a detailed structural analysis.

B.2.3 Loads.

Note: All loads discussed herein are to be considered as dead load plus live loads. The design of structures subject to direct wheel loads should also anticipate braking loads as high as 0.7 × Gear Load. (Assumes no slip brakes)

B.2.4 Direct Loading.

1. Decks and covers subject to direct heavy airplane loadings such as manhole covers, inlet grates, utility tunnel roofs, bridges, etc., should be designed for the following loadings:

   a. Manhole covers for 100,000 lb. (45 000 kg) wheel loads with 250 psi (1.72 MPa) tire pressure. Higher tire pressures should be assumed if using aircraft will have tire pressures greater than 250 psi (1.72 MPa).

   b. For spans of 2 feet (0.6 m) or less in the least direction, a uniform live load of the larger of 250 psi (1.72 MPa) or the maximum tire pressure assumed for manhole cover design

   c. For spans of greater than 2 feet (0.6 m) in the least direction, the design will be based on the number of wheels which will fit the span. Design for the maximum wheel load anticipated at that location over the life of the structure. For example for a bridge at large hub airport it is conceivable that it should be designed considering a 1,500,000 pound (680,000 kg) aircraft.

2. Special consideration will be given to structures that will be required to support both in-line and diagonal traffic lanes, such as diagonal taxiways or apron taxi routes.

B.2.5 Pavement to Structure Joints.

Airport structures should be designed to support the design loads without assistance from adjacent pavements. Do not consider load transfer to pavement slabs when designing structures. It is recommended that isolation joints (Type A or A-1) be provided where concrete slabs abut structures. Reinforcement should be provided for all slabs with a penetration.

APPENDIX C. NONDESTRUCTIVE TESTING (NDT) USING FALLING-WEIGHT TYPE IMPULSE LOAD DEVICES IN THE EVALUATION OF AIRPORT PAVEMENTS

C.1 General.

Nondestructive testing (NDT) can make use of many types of data-collection equipment and methods of data analysis. The NDT data collected can be used to evaluate the load-carrying capacity of existing pavements; determine the material properties of in-situ pavement and subgrade layers for design of pavements; compare relative strength and/or condition within sections of a pavement system to each other; and provide structural performance data to supplement pavement condition index (PCI) survey data in an airport pavement management program (PMP). This appendix is restricted to just talking about NDT testing using falling weight type impulse load devices.

C.1.1 NDT Advantages.

C.1.1.1 There are several advantages to using NDT in lieu of or as a supplement to traditional destructive tests. A primary advantage is the capability to quickly gather data at several locations while keeping a runway, taxiway, or apron operational. Without NDT, structural data must be obtained from numerous cores, borings, and excavation pits on an existing airport pavement which can be very disruptive to airport operations.

C.1.1.2 Nondestructive tests are economical to perform and data can be collected at up to 250 locations per day. Heavy falling weight deflectometer (HWD) or falling weight deflectometer (FWD) equipment measures pavement surface response (i.e., deflections) from an applied dynamic load that simulates a moving wheel. The magnitude of the applied dynamic load can be varied so that it is similar to the load on a single wheel of the most demanding or design aircraft. Pavement deflections are recorded directly beneath the load plate and at typical radial offsets of 12 inches (30 cm), out to typical distances of 60 inches (150 cm) to 72 inches (180 cm).

C.1.1.3 The deflection data collected with HWD or FWD equipment can provide both qualitative and quantitative data about the strength of a pavement at the time of testing. The raw deflection data directly beneath the load plate sensor provides an indication of the strength of the entire pavement structure. Likewise, the raw deflection data from the outermost sensor provides an indication of subgrade strength.

C.1.1.4 In addition, when deflection or stiffness profile plots are constructed with deflection data from all test locations on a pavement facility, relatively strong and weak areas become readily apparent.

C.1.1.5 Quantitative data from HWD or FWD include material properties of each pavement and subgrade layer that engineers use with other physical properties, such as layer thicknesses and interface bonding conditions, to evaluate the structural performance of a pavement or investigate strengthening options. Most of the material property information is obtained using software programs that process and analyze raw HWD or FWD data. Once material properties, such as modulus of elasticity, *E*, and modulus of subgrade reaction, *k*, are computed, the engineer can conduct structural evaluations of existing pavements, design structural improvements, and develop reconstruction pavement cross-sections using subgrade strength data.

C.2 NDT Limitations.

C.2.1 NDT also has some limitations. NDT is a very good methodology for assessing the structural condition of an airfield pavement; however, engineers must use other methods to evaluate the functional condition of the pavement, for example, visual condition, smoothness, and friction characteristics. The visual condition is most frequently assessed using the PCI in accordance with ASTM International (ASTM) D5340, *Standard Test Method for Airport Pavement Condition Index Surveys*, and AC 150/5380-6, *Guidelines and Procedures for Maintenance of Airport Pavements.* Once the NDT-based structural and functional conditions are known, the engineer can assign an overall pavement condition rating.

C.2.2 The differentiation between structural and functional performance is important in developing requirements for pavement rehabilitation. For example, a pavement can have a low PCI due to environmental distress, yet the pavement has sufficient thickness to accommodate structural loading. The converse may also be true, where a pavement may be in good condition, but has a low structural life due to proposed heavier aircraft loading.

C.2.3 In addition, while NDT may provide excellent information about structural capacity, the engineer may still require other important engineering properties of the pavement layers, such as grain-size distribution of the subgrade, to determine swelling and heaving potential. For subsurface drainage evaluation and design, grain-size distribution and permeability tests may help assess the hydraulic capacity of the base, subbase, and subgrade.

C.2.4 It should also be noted that quantitative results obtained from raw NDT data are model dependent. The results depend on the structural models and software algorithms that are used by programs that process NDT data and perform a back-calculation of layer material properties.

C.2.5 Because of the model dependencies of NDT software analysis tools, the engineer should exercise caution when evaluating selected pavement types, such as continuously reinforced concrete pavement, post-tensioned concrete, and pre-tensioned concrete. The structural theory and performance models for these pavement types are significantly different than traditional pavements, which include Asphalt Cement Hot Mix Asphalt

(HMA), jointed plain Portland Cement Concrete (PCC), jointed reinforced PCC, HMA overlaid PCC, and PCC overlaid PCC.

C.2.6 Finally, FWD/HWD tests conducted at different times during the year may give different results due to climatic changes. For example, tests conducted during spring thaw or after extended dry periods may provide non-representative results or inaccurate conclusions on pavement at subgrade strength.

C.3 Use of Deflection Data.

There are many ways to use the deflection data to obtain pavement characteristics needed to identify the causes of pavement distresses, conduct a pavement evaluation, or perform a strengthening design. Engineers can evaluate the deflection data using qualitative and quantitative procedures. Subsequent sections present several methods that can be used to compute and evaluate such pavement characteristics as: ISM, DSM, and normalized deflections; back-calculated elastic modulus of pavement layers and subgrade; correlations to conventional characterizations (for example, California Bearing Ratio [CBR], k); crack and joint load transfer efficiency; void detection at PCC corners and joints. These derived pavement characteristics can then be used in the FAA's evaluation and design procedures.

C.4 NDT Equipment.

Nondestructive testing equipment includes both deflection and non-deflection testing equipment. Deflection measuring equipment for nondestructive testing of airport pavements can be broadly classified as static or dynamic loading devices. Dynamic loading equipment can be further classified according to the type of forcing function used, i.e., vibratory or impulse devices. Non-deflection measuring equipment that can supplement deflection testing includes ground-penetrating radar, infrared thermography, dynamic cone penetrometer, and devices that measure surface friction, roughness, and surface waves. The data collected from nondeflection measuring equipment often supplements deflection data or provides standalone information in pavement analysis work. Nondeflection measuring equipment includes the following:

C.4.1 <u>Friction Characteristics</u>.

Equipment is available to conduct surface friction tests on a pavement. The methods of testing and common types of friction testers for airports are addressed in AC 150/5320-12, *Measurement, Construction, and Maintenance of Skid Resistant Airport Pavement Surfaces.*

C.4.1.1 Smoothness Characteristics.

There are several types of equipment that are available to collect surface profile data and to determine how aircraft may respond during taxi, takeoff, and landing. AC 150/5380-9, *Guidelines and Procedures for Measuring Airfield Pavement Roughness,* provides procedures to evaluate a surface profile in terms of roughness and the impact pavement roughness may have on aircraft.

C.4.1.2 **Dynamic Cone Penetrometer (DCP).**
A DCP can be used to supplement NDT data. If cores are taken through the pavement to verify the thickness of an HMA or PCC layer, the DCP can help evaluate the stiffness of the base, subbase, and subgrade. Data is recorded in terms of the number of blows per inch required to drive the cone-shaped end of the rod through each of the layers. Plots of the data provide information about the changes in layer types and layer strengths. Refer to ASTM D 6951, *Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications,* for additional information.

C.4.1.3 **Ground-Penetrating Radar (GPR).**
The most common uses of GPR data include measuring pavement layer thicknesses, detecting the presence of excess water in a structure, locating underground utilities, investigating significant delamination between pavement layers, and potentially locating voids. Refer to ASTM D 6432, *Standard Guide for Using the Surface Ground Penetrating Radar Method for Subsurface Investigation,* for additional information.

C.4.1.4 **Infrared Thermography (IR).**
One of the most common uses of IR data is to determine if delamination has occurred between highway asphalt pavement layers.

C.5 Deflection Measuring Equipment.

C.5.1 There are several categories of deflection measuring equipment: static, steady state (for example, vibratory), and impulse load devices. A static device measures deflection at one point under a nonmoving load. Static tests are slow and labor intensive compared to the other devices. Vibratory devices induce a steady-state vibration to the pavement with a dynamic force generator. The dynamic force is then generated at a precomputed frequency that causes the pavement to respond (deflect). Impulse load devices, such as the FWD/HWD, impart an impulse load to the pavement with a free-falling weight that impacts a set of rubber springs. The magnitude of the dynamic load depends on the mass of the weight and the height from which the weight is dropped. The resultant deflections are typically measured with velocity transducers, accelerometers, or linear variable differential transducers (LVDT). The magnitude of the impulse load can be varied by changing the mass and/or drop height so that it is similar to that of a wheel load on the main gear of an aircraft. Table C-1 lists several ASTM standards that apply to deflection measuring equipment.

C.5.2 Using static or dynamic testing equipment has proven useful in providing data on the structural properties of pavement and subgrade layers. The data are typically used to detect patterns of variability in pavement support conditions or to estimate the strength of pavement and subgrade layers. With this information, the engineer can design rehabilitation overlays and new/reconstructed cross-sections, or optimize a rehabilitation option that is developed from a PMS.

C.5.3 This appendix focuses on nondestructive testing equipment that measures pavement surface deflections after applying a static or dynamic load to the pavement. NDT equipment that imparts dynamic loads creates surface deflections by applying a vibratory or impulse load to the pavement surface through a loading plate. For vibratory equipment, the dynamic load is typically generated hydraulically or by counter rotating masses. For impulse devices, such as the Falling Weight Deflectometer (FWD), the dynamic load is generated by a mass free falling onto a set of rubber springs, as shown in Figure C-1. The magnitude of the impulse load can be varied by changing the mass and/or drop height so that it is similar to that of a wheel load on the main gear of an aircraft.

Table C-1. ASTM Standards for Deflection Measuring Equipment

ASTM	**NDT Equipment Type**		
	Static	**Vibratory**	**Impulse**
D 1195, *Standard Test Method for Repetitive Static Plate Load Tests of Soils and Flexible Pavement Components, for Use in Evaluation and Design of Airport and Highway Pavements*	●		
D 1196, *Standard Test Method for Nonrepetitive Static Plate Load Tests of Soils and Flexible Pavement Components, for Use in Evaluation and Design of Airport and Highway Pavements*	●		
D 4602, *Standard Guide for Nondestructive Testing of Pavements Using Cyclic-Loading Dynamic Deflection Equipment*		●	
D 4694, *Standard Test Method for Deflections with A Falling-Weight-Type Impulse Load Device*			●
D 4695, *Standard Guide for General Pavement Deflection Measurements*	●	●	●
D 4748, *Standard Test Method for Determining the Thickness of Bound Pavement Layers Using Short-Pulse Radar*			●
D 5858, *Standard Guide for Calculating In Situ Equivalent Elastic Moduli of Pavement Materials Using Layered Elastic Theory*			●
E 2583, *Standard Test Method for Measuring Deflections with a Light Weight Deflectometer (LWD)*			●

ASTM	NDT Equipment Type		
	Static	**Vibratory**	**Impulse**
E 2835, *Standard Test Method for Measuring Deflections using a Portable Impulse Plate Load Test Device*			●

C.5.3.1 **Impulse Load Device.**

The most common type of equipment in use today is the impulse load device (i.e., FWD or HWD). ASTM D 4694, *Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load Device,* addresses key components of this device including instruments exposed to the elements, the force-generating device (for example, falling weight), the loading plate, the deflection sensor, the load cell, and the data processing and storage system. Typically, the HWD will be used for airport pavements.

1. **Load Plate Diameter.** Many impulse-loading equipment manufacturers offer the option of a 12-inch (30-cm) or an 18-inch (45-cm) diameter load plate. The 12-inch (30-cm) load plate is normally used when testing materials on airports.
2. **Sensor Spacing and Number.** The number of available sensors depends on the manufacturer and equipment model. As a result, the sensor spacing will depend on the number of available sensors and the length of the sensor bar. Although most equipment allows for the sensors to be repositioned for each pavement study, it is desirable to conduct work using the same configuration, regardless of the type of pavement structure.
3. In general, devices that have more sensors can more accurately measure the deflection basin that is produced by static or dynamic loads. Accurate measurement of the deflection basin is especially important when analyzing the deflection data to compute the elastic modulus of each pavement layer. It is also important to ensure that the magnitude of deflection in the outermost sensor is within the manufacturer's specifications for the sensors. The magnitude of the deflection in the outermost sensor depends primarily on the magnitude of the dynamic load, the thickness and stiffness of the pavement structure, and the depth to an underlying rock or stiff layer. The following sensor configuration is recommended:

Table C-1. Recommended Sensor Configuration

Sensor Distance from Center of Load Plate, inch (cm)						
Sensor 1	**Sensor 2**	**Sensor 3**	**Sensor 4**	**Sensor 5**	**Sensor 6**	**Sensor 7**
0 (00)	12 (30)	24 (60)	36 (90)	48 (120)	60 (150)	72 (180)

4. Pulse Duration. For impulse-load equipment, the force-pulse duration is the length of time between an initial rise in the dynamic load until it dissipates to near zero. Both the FAA and ASTM recognize a pulse duration in the range of 20 to 60 milliseconds as being typical for most impulse-load devices. Likewise, rise time is the time between an initial rise in the dynamic load and its peak before it begins to dissipate. Typical rise times for impulse-load devices are in the range of 10 to 30 milliseconds.
5. Load Linearity. For most pavement structures and testing conditions, traditional paving materials will behave in a linear elastic manner within the load range that the tests are conducted.

C.5.3.2 **Sensitivity Studies.**

1. Sensitivity studies at the National Airport Pavement Test Facility (NAPTF) and Denver International Airport (DIA) have shown there is little difference in the pavement response when the HWD impulse load is changed. Based on the results from the sensitivity studies, the amplitude of the impulse load is not critical provided the generated deflections are within the limits of all deflection sensors. The key factors that will determine the allowable range of impulse loads are pavement layer thicknesses and material types. Unless the pavement is a very thin PCC or HMA, HWD devices should be used for airport pavements.
2. Generally, the impulse load should range between 20,000 pounds (90 kN) and 55,000 pounds (250 kN) on pavements serving commercial air carrier aircraft, provided the maximum reliable displacement sensor is not exceeded. For thinner GA pavements, LWD may be used.

C.6 Pavement Stiffness and Sensor Response.

C.6.1 The load-response data that NDT equipment measures in the field provides valuable information on the strength of the pavement structure. Initial review of the deflection under the load plate and at the outermost sensor, sensors D1 and D7 in Figure C-1, respectively, is an indicator of pavement and subgrade stiffness. Although this information will not provide information about the strength of each pavement layer, it does provide a quick assessment of the pavement's overall strength and relative variability of strength within a particular facility (runway, taxiway, or apron).

C.6.2 Pavement stiffness is defined as the dynamic force divided by the pavement deflection at the center of the load plate. For both impulse and vibratory devices, the stiffness is defined as the load divided by the maximum deflection under the load plate. The Impulse Stiffness Modulus (ISM) and the Dynamic Stiffness Modulus (DSM) are defined as follows for impulse and vibratory NDT devices, respectively:

Equation C-1. Impulse and Dynamic Stiffness Modulus

$$I(D)SM = \left(\frac{L}{d_0}\right)$$

Where:

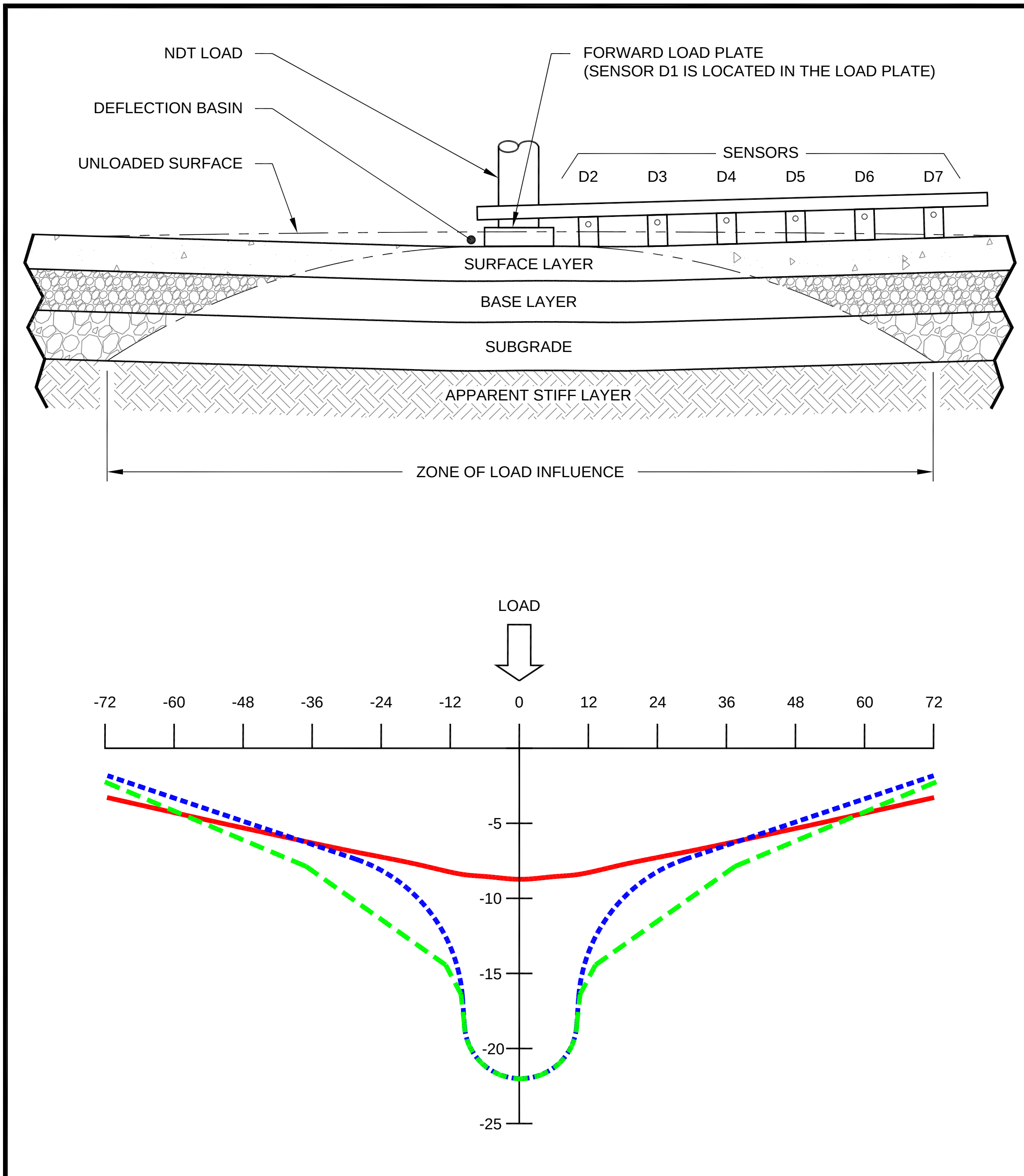
I(D)SM	=	Impulse and Dynamic Stiffness Modulus (kips/in)
L	=	Applied Load (kips)
d_o	=	Maximum Deflection of Load Plate (in)

C.7 Deflection Basin.

C.7.1 After the load is applied to the pavement surface, the sensors shown in Figure C-1 are used to measure the deflections that produce what is commonly referred to as a deflection basin. Figure C-1 also shows the zone of load influence that is created by a FWD and the relative location of the sensors that measure the deflection basin area. The deflection basin area can then be used to obtain additional information about the individual layers in the pavement structure that cannot be obtained by using deflection data from a single sensor.

C.7.2 The shape of the basin is determined by the response of the pavement to the applied load. The pavement deflection is the largest directly beneath the load and then decreases as the distance from the load increases. Generally, a weaker pavement will deflect more than a stronger pavement under the same load. However, the shape of the basin is related to the strengths of all the individual layers.

Figure C-1. Deflection Basin and Sensor Location



PAVEMENT 1	PAVEMENT 2	PAVEMENT 3
8" PCC E-4,000,000 PSI	4" HMA E-500,000 PSI	4" HMA E-500,000 PSI
6" AGG E-80,000 PSI	8" AGG E-20,000 PSI	8" AGG E-80,000 PSI
SG E-12,000 PSI	SG E-24,000 PSI	SG E-12,000 PSI

C.7.3 To illustrate the importance of measuring the deflection basin, Figure C-1, also shows a comparison of three pavements. Pavement 1 is PCC and pavements 2 and 3 are HMA. As expected, the PCC distributes the applied load over a larger area and has a smaller maximum deflection than the other two pavements. Although pavements 2 and 3 have the same cross-section and the same maximum deflection under the load plate, they would presumably perform differently under the same loading conditions because of the differences in base and subgrade strengths.

C.7.4 In addition to each layer's material properties, other factors can contribute to differences in the deflection basins. Underlying stiff or apparent stiff layers, the temperature of the HMA layer during testing, moisture contents in each of the layers, and PCC slab warping and curling can affect deflection basin shapes. An important component in the evaluation process, then, is analysis of the NDT data to estimate the expected structural performance of each pavement layer and subgrade.

C.8 NDT Test Planning.

C.8.1 Nondestructive testing combined with the analytical procedures described here can provide a direct indication of a pavement's structural performance. Visual condition surveys, such as the PCI procedure, provide excellent information regarding the functional condition of the pavement. However, visual distress data can only provide an indirect measure of the structural condition of the pavement structure. Once the airport operator and engineer decide to include NDT in their pavement study, they should focus on the number and types of tests that will be conducted. The total number of tests will depend primarily on the area of the pavements included in the study; the types of pavement; and whether the study is a project or network-level investigation.

C.8.2 Project-Level objectives include evaluation of the load-carrying capacity of existing pavements and to provide material properties of in-situ pavement layers for the design or rehabilitation of pavement structures. Network-Level objectives include collection of NDT data to supplement pavement condition index (PCI) survey data and generate Pavement Classification Numbers (PCN) for each airside facility in accordance with AC 150/5335-5, *Standard Method of Reporting Airport Pavement Strength-PCN.* Refer to AC 150/5380-7, *Airport Pavement Management Program (PMP),* for guidance on developing a PMP.

C.9 FWD/HWD Test Locations and Spacing.

C.9.1 There are several test scenarios that may be conducted during a pavement study. For all types of pavements, the most common is a center test. For jointed PCC and HMA overlaid PCC pavements, this is a test in the center of the PCC slab. For HMA pavements, this is a test in the center of the wheel path away from any cracks that may exist. The center test serves primarily to collect deflection data that form a deflection basin that can be used to estimate the strength of the pavement and subgrade layers.

C.9.2 For PCC and HMA overlaid PCC pavements, there are several tests that will help characterize the structure. These tests focus on the fact that most PCC pavements have joints and most HMA overlaid PCC pavements have surface cracks that have reflected up from PCC joints. FWD/HWD at various locations on the joints provides data regarding pavement response to aircraft loads and changes in climatic conditions.

C.9.3 Testing at longitudinal and transverse joints shows how much of an aircraft's main gear is transferred from the loaded slab to the unloaded slab. As the amount of load transfer is increased to the unloaded slab, the flexural stress in the loaded slab decreases and the pavement life is extended. The amount of load transfer depends on many factors, including pavement temperature, the use of dowel bars, and the use of a stabilized base beneath the PCC surface layer.

C.9.4 The corner is another common test location. This is an area where a loss of support beneath the PCC slab typically due to curling occurs more often than other areas in the slab. Conduct corner tests so the load plate is within 6 inches (15 cm) of the transverse and longitudinal joints. FWD/HWD in areas with lack of slab support could result in structural damage to the slab.

C.9.5 Often, concrete midslab, joint, and corner tests are performed on the same slab to evaluate the relative stiffness at different locations. If concrete slabs have corner breaks there is a possibility that voids exist.

C.9.6 The location and testing interval for each pavement facility should be sufficient to characterize the material properties. Center slab test locations and spacing should generally be in the wheel paths, spaced between 100 feet and 400 feet along the runway length. Additional testing for load transfer of PCC should include testing at transverse and longitudinal joints. For PCN surveys, FWD/HWD data should be collected randomly within the keel section of the runway. For both HMA and PCC pavements, FWD/HWD should not be conducted near cracks unless one of the test objectives is to measure load transfer efficiency across the crack. For HMA pavements, FWD/HWD passes should be made so that deflection data are at least 1.5 feet (0.5 m) to 3 feet (1 m) away from longitudinal construction joints. The total number of tests for each facility should be evenly distributed over the area tested with each adjacent FWD/HWD pass typically staggered to obtain comprehensive coverage. For testing of airside access roads, perimeter roads, and other landside pavement, refer to ASTM D 4695, *Standard Guide for General Pavement Deflection Measurements.*

C.10 Climate and Weather Affects.

Climate and weather affect FWD/HWD results. The engineer should select a test period that best represents the pavement conditions for a majority of the year. For PCC pavements, conduct FWD/HWD at a time when the temperature is relatively constant between the day and night.

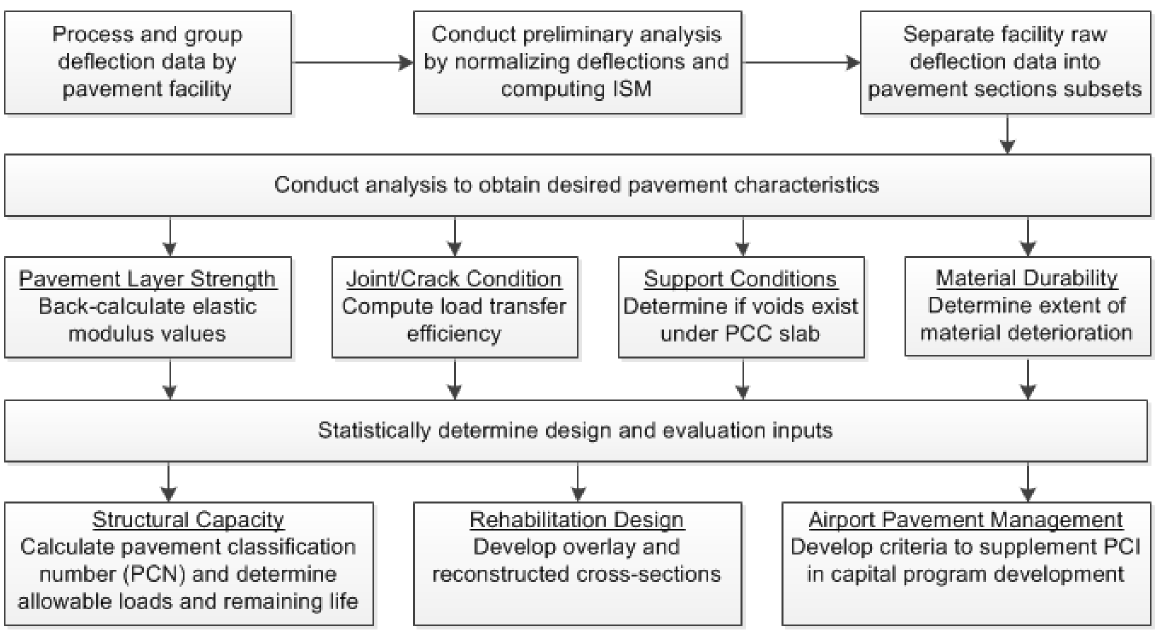
C.11 Mobilization.

Before mobilizing to the field site, the equipment operator must verify with airport management that a construction safety phasing plan has been prepared in accordance with AC 150/5370-2, *Operational Safety on Airports During Construction*, and that NOTAMs will be issued.

C.12 Deflection Data Analysis.

Figure C-2 provides an overview of the deflection data analysis process. There are several characteristics that are used to evaluate the structural condition of an existing pavement structure. The most common use of deflection data is to measure the strength of the structure as a whole and determine the individual layer properties within the structure. Because most PCC pavements are built using expansion, contraction, and construction joints, several additional characteristics are used to evaluate the condition of the concrete pavements. These discontinuities in the PCC create opportunities for the joint to deteriorate and transfer less load to the adjacent slab, lead to higher deflections at slab corners that may create voids beneath the slab, and provide opportunities for excessive moisture accumulation at the joints that may accelerate PCC material durability problems.

Figure C-2. Deflection Data Analysis and Design Flowchart



C.13 Process Raw Deflection Data.

C.13.1 The boundary limits of pavement sections within a facility should have already been defined in an airport pavement management program (PMP) or through a review of the construction history. In a PMP, a section is defined as an area of pavement that is expected to perform uniformly because of aircraft traffic levels, pavement age, or pavement cross-section. Deflection data can be used to define or refine the limits of all sections within a pavement facility.

C.13.1.1 The data file may contain several types of deflection data, such as PCC center, slab joint, and slab corner tests. The deflection data should be extracted from the file and organized by type and location of tests. The preliminary analysis of the center deflection data is routinely conducted by plotting either the ISM or normalized deflections along the length of an apron, taxiway, or runway.

C.13.1.2 The Impulse Stiffness Modulus (ISM) and the Dynamic Stiffness Modulus (DSM) are calculated as shown in Equation C-1.

C.13.1.3 Raw data deflections may be normalized by adjusting measured deflections to an airplane standard load.

Equation C-2. Normalized Deflection

$$d_{0n} = \left(\frac{L_{norm}}{L_{applied}}\right) d_0$$

Where:

d_{0n} = Normalized deflection

L_{norm} = Normalized load

$L_{applied}$ = Applied load

d_0 = Measured deflection at selected sensor location

C.13.1.4 When reviewing the profile plots of ISM values or normalized deflections, the engineer should look for patterns of uniformity and points of change identifying sections. The ISM values or normalized deflections under the load plate provide an indication of the overall strength of the entire pavement structure (i.e., pavement layers and subgrade) at each test location. For a given impulse load (for example, 40,000 pounds (180 kN)), increasing ISM values or decreasing normalized deflections indicate increasing pavement strength. Example profile plots of ISM and normalized deflects are as illustrated in Figures C-4 and C-5 respectively.

Figure C-3. ISM Plot Identifying Pavement Section Limits

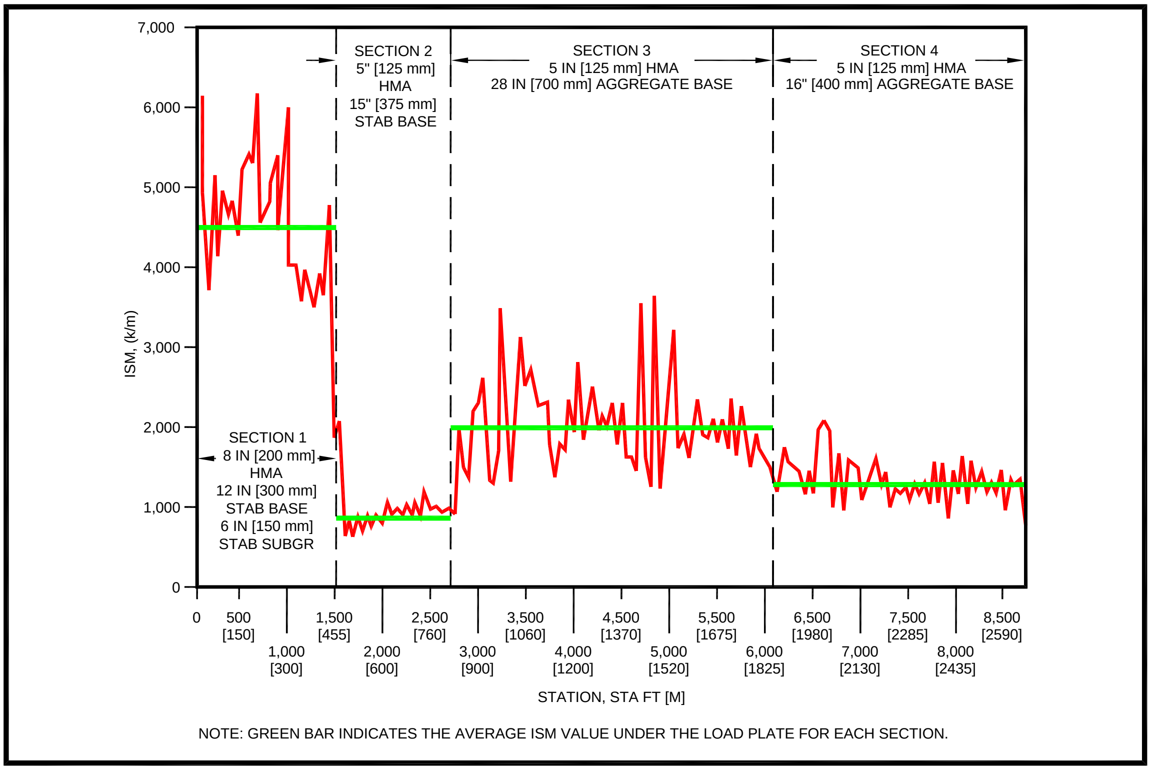
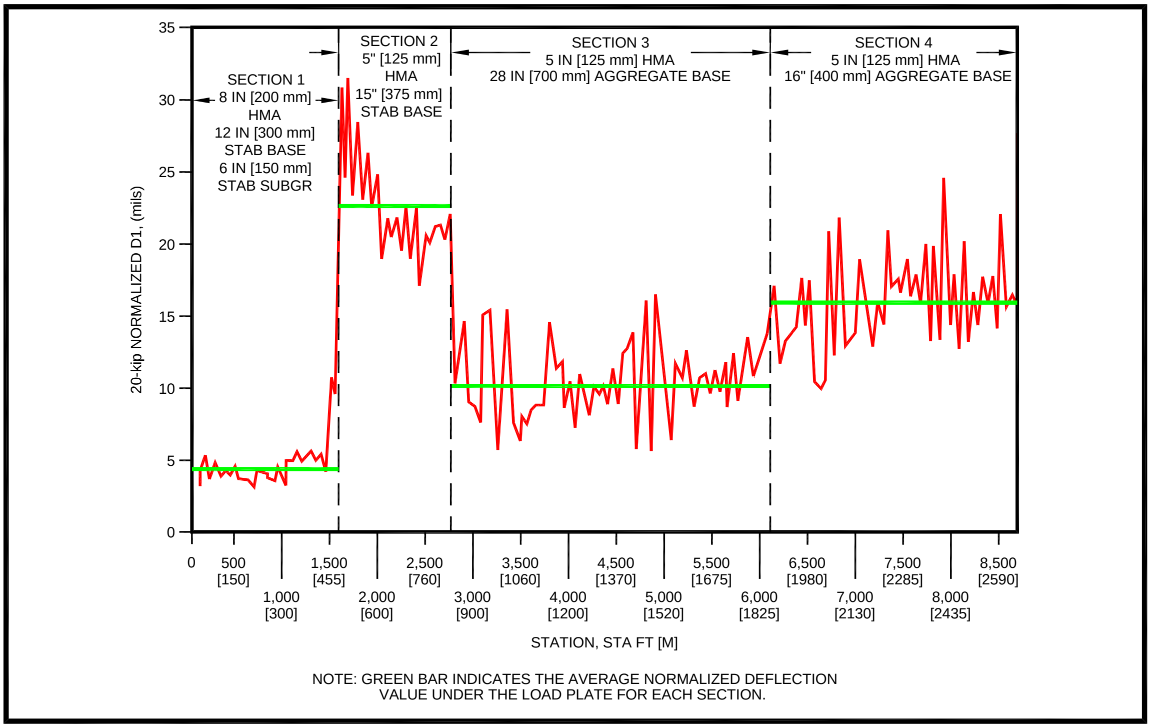


Figure C-4. Normalized Deflection Plot Identifying Pavement Section Limits



C.13.1.5 Figure C-3 illustrates how the ISM profile plots were used to identify four different pavement sections within this pavement facility. This figure shows that section 1 is the strongest of all four sections since its average ISM value is significantly higher than all other sections. Although the mean ISM values for sections 2, 3, and 4 are similar, ISM variability is much higher in section 3.

C.13.1.6 Likewise, section 2 may be the weakest of the sections because the HMA layer is less than 5 inches (13 cm) thick or the stabilized base may be very weak. Profile plots can identify locations where additional coring may be needed to provide information on layer thickness and strength.

C.13.1.7 Figure C-4 shows that normalized deflection profile plots can also be used to identify the limits of pavement sections within a particular facility. As these profile plots show, stronger pavement sections have lower normalized deflections. The engineer can use either normalized deflections or ISM values to identify section limits. ISM values are used more frequently and provide information independent of force.

C.13.1.8 Deflection data can also be used to identify variations in subgrade strength beneath a pavement. A sensor that is located a precomputed distance from the center of the load plate may provide a good estimate of the subgrade strength. The American Association of State Highway and Transportation Officials (AASHTO) 1993 design procedures provide guidance for the distance the sensor should be from the load plate to reflect the subgrade strength (for example, outside of the stress bulb at the subgrade-pavement interface).

C.13.1.9 Using the deflection test data separated by pavement sections and test type, the following may be determined; pavement layer strengths and material durability can be determined from center deflection data; joint condition and material durability can be determined from joint and crack deflection data; and support conditions and material durability can be determined from the PCC slab corner deflection data.

C.14 Software Tools.

Engineers have many choices regarding software tools for deflection data analysis. Back-calculation methods used for determination of layer properties should be consistent with the procedure used for structural evaluation and design. Although engineers have several choices regarding FAA software tools, they should select programs that have the same theoretical basis for a study. Stated differently, the back-calculation methods used should be consistent with the forward computational procedure that will be used for structural evaluation and design. FAA software tools such as FAARFIELD, COMFAA, and BAKFAA, are available at http://www.faa.gov/airports/engineering/design_software/.

C.15 Back-Calculation Analysis.

C.15.1 The engineer can use deflection basin data from flexible pavements and rigid center tests to compute the strength of pavement layers. The process used to conduct this analysis is referred to as back-calculation because the engineer normally does the opposite of traditional pavement design. Rather than determining the thickness of each pavement layer based on assumed layer strengths, back-calculation typically involves solving for pavement layer strengths based on assumed uniform layer thicknesses. Throughout the remainder of this section, layer strength is referred to in terms of Young's modulus of elasticity or simply the elastic modulus.

C.15.2 The types of loads that are applied through the use of NDT equipment fall into two general categories: static loads and dynamic loads. Dynamic loads include vibratory and impulse load devices. For both static and dynamic loads, the pavement can respond linearly or nonlinearly to the applied loads.

C.15.3 Back-calculation analysis work that falls in the static-linear category is typically conducted using two procedures. The first category allows the engineer to use closed-form procedures that directly compute the elastic modulus of each layer by using layer thicknesses and deflections from one or more sensors. The second category uses an iterative mechanistic process to solve for the elastic modulus by using layer thicknesses and deflections from at least four sensors.

C.15.4 Before conducting an analysis, the engineer should review the deflection tests that have been separated by pavement facility and section for back-calculation. Regardless of the software tool that will be used in the analysis, linear-elastic theory requires that pavement deflections decrease as the distance from the load plate increases. In addition, for typical sensor configurations, the deflections should gradually decrease from the load plate to the outermost sensor.

C.15.5 Deflection basin anomalies could occur for several reasons, including the presence of a crack near the load plate, a nonfunctioning sensor, sensor and equipment configuration error, sensors not properly calibrated, voids, loss of support, temperature curling or moisture warping of PCC slab, or several other reasons. The engineer should review the deflection data and remove data that have the following anomalies.

- **Type I Deflection Basin.** In this scenario, the deflections at one or more of the outer sensors are greater than the deflection under the load plate. This type of anomaly will produce the largest error during back-calculation analysis.
- **Type II Deflection Basin.** Another less obvious anomaly is an unusually large decrease in deflection between two adjacent sensors. While elastic layer theory requires deflections to decrease as the distance from the load plate increases, the amount of decrease should be gradual and relatively consistent between all sensors.
- **Type III Deflection Basin.** Similar to Type I, the deflection at the outermost sensor of two adjacent sensors is greater than the deflection at the sensor that is closest to the load plate.

C.15.6 For PCC pavement analysis, HMA overlays are considered to be thin if they are less than 4 inches (10 cm) thick and the PCC layer thickness is less than 10 inches (25 cm). The HMA overlay is also considered to be thin if it is less than 6 inches (15 cm) thick and the PCC layer is greater than 10 inches (25 cm) thick.

C.15.7 If the PCC structure does not contain a stabilized base, HMA overlay, or PCC overlay, the back-calculated dynamic effective modulus is the PCC modulus of elasticity. However, the back-calculated dynamic k-value must be adjusted to obtain a static k-value that is the basis for conventional FAA evaluation and design programs that use a k-value.

C.15.8 National Cooperative Highway Research Program (NCHRP) Report 372, Support Under Portland Cement Concrete Pavements, reported that the static k-value is equal to one-half of the dynamic k-value. The static-k value is the value that would be obtained by conducting plate bearing tests as described in AASHTO T 222.

C.15.9 If the PCC structure contains a stabilized base, thin HMA overlay, or PCC overlay, the back-calculated dynamic effective modulus may be used to compute two modulus values. Possible modulus scenarios are as follows: bonded or unbonded PCC overlay and PCC layer, thin HMA overlay and PCC layer, PCC layer and lean concrete or cement-treated base, or PCC layer and HMA stabilized base.

C.15.10 The results that are obtained through iterative back-calculation are influenced by many factors, such as Number of Layers, Layer Thicknesses, Layer Interface Condition, HMA Layer Temperature, environmental conditions, Adjacent Layer Modulus Ratios, Underlying Stiff Layer, Pavement Cracks, Sensor Errors, Non-uniform load plate contact, Pulse Duration, Frequency Duration, and Material Property Variability. Because so many factors impact the error level and results and, because there is no one unique solution, iterative elastic-layer back-calculation requires engineering judgment.

C.16 Rigid Pavement Considerations.

While it is important to know the strength of each layer in a pavement evaluation or design study, PCC pavements often require additional testing and evaluation of characteristics that are important for rigid pavements. These characteristics include joint and crack conditions, support conditions, and material durability.

C.16.1 <u>PCC Joint Analysis</u>.

C.16.1.1 The analysis of PCC joints or cracks is important because the amount of load that is transferred from one PCC slab to the adjacent slab can significantly impact the structural capacity of the pavement. NDT tests are conducted at joints and cracks to estimate what percentage of load is transferred from the loaded slab to the unloaded slab. As the amount of load transferred to the unloaded slab increases, the flexural stress in the loaded slab decreases and the pavement life is extended.

C.16.1.2 The amount of airplane load transfer depends on many factors, including gear configuration, tire contact area, pavement temperature, use of dowel bars, and use of a stabilized base beneath the PCC surface layer.

C.16.1.3 Deflection load transfer efficiency (LTE_Δ) is most frequently defined as shown in Equation C-3. If the LTE_Δ is being calculated for an HMA overlaid PCC at the joint reflective crack, compression of the HMA overlay may result in an inaccurate assessment of the load transfer.

Equation C-3. Load Transfer Efficiency.

$$LTE_\Delta = \left(\frac{\Delta_{unloaded\_slab}}{\Delta_{loaded\_slab}}\right) 100\%$$

Where:

LTE_Δ	=	Deflection load transfer efficiency, in percent
$\Delta_{unloaded\_slab}$	=	Deflection on loaded slab, normally under load plate, in mils
Δ_{loaded_slab}	=	Deflection on adjacent unloaded slab, in mils

C.16.1.4 Once LTE_Δ values are computed, they must be related to the stress load transfer efficiency (LTE_σ) to understand how load transfer will impact the structural capacity of a pavement section. This is necessary because the FAA design and evaluation procedures in AC 150/5320-6 assumes the amount of load transfer is sufficient to reduce the free edge flexural stress in a PCC slab by 25 percent. Since the relationship between LTE_Δ and LTE_σ is not linear, additional analysis work is required to compute if the stress load transfer efficiency is 25 percent. Equation C-4 shows how LTE_σ is defined.

Equation C-4. Stress Load Transfer Efficiency

$$LTE_\sigma = \left(\frac{\sigma_{unloaded\_slab}}{\sigma_{loaded\_slab}}\right) 100\%$$

Where:

LTE_σ	=	Stress load transfer efficiency, in percent
$\sigma_{unloaded\_slab}$	=	Stress on loaded slab, in psi
σ_{loaded_slab}	=	Stress on adjacent unloaded slab, in psi

C.16.2 PCC Void Analysis.

C.16.2.1 In addition to joint load transfer, another important characteristic of a PCC pavement is the slab support conditions. One of the assumptions made during PCC back-calculation is that the entire slab is in full contact with the foundation. The presence of surface distresses such as corner breaks, joint faulting, and slab cracking, indicates that a loss of support may exist in the pavement section. As with a joint condition analysis, the focus of the void analysis is near joints or slab corners.

C.16.2.2 A loss of support may exist because erosion may have occurred in the base, subbase, or subgrade; settlement beneath the PCC layer; or due to temperature curling or moisture warping.

C.16.3 PCC Durability Analysis.

C.16.3.1 The back-calculation analysis procedures assume that the PCC layer is homogenous and the results are based on center slab deflections and the condition of the slab in the interior. PCC pavements can experience durability problems as a result of poor mix designs, poor construction, reactive and nondurable aggregates, wet climates, and high numbers of freeze-thaw cycles. In general, durability problems are most severe along PCC joints and at slab corners because moisture levels are the highest at these locations.

C.16.3.2 Surface conditions may not be a good indicator of the severity several inches below the PCC surface and NDT deflection data may be very useful in assessing the severity of durability-related problems. This is especially true if a PCC pavement with durability problems has been overlaid with HMA. Often, the severity of the durability distresses increases after an HMA overlay has been constructed because more moisture is present at the HMA and PCC interface.

C.16.3.3 The extent of the durability problem can be assessed by evaluating the ISM (or DSM) obtained from the center of the slab and comparing it to the ISM (or DSM) at a transverse or longitudinal joint or at the slab corner. The ISM_{ratio} will not be equal to one for a perfect slab because slab deflections are highest at the slab corner and lowest at the slab center. If a joint load transfer or loss of support analysis has been conducted, the same raw deflection data can be used to compute the ISM_{ratio}.

Equation C-5. Impulse Stiffness Modulus Ratio.

$$ISM_{ratio} = K\left(\frac{ISM_{slab\_center}}{ISM_{slab\_corner}}\right)$$

or

$$ISM_{ratio} = K\left(\frac{ISM_{slab\_center}}{ISM_{slab\_joint}}\right)$$

Where:

ISM_{ratio} = Impulse stiffness modulus ratio

$ISM_{slab\ center}$ = Impulse stiffness modulus at slab center, in pounds/inch

$ISM_{slab\ corner}$ = Impulse stiffness modulus at slab corner, in pounds/inch

$ISM_{slab\ joint}$ = Impulse stiffness modulus at slab joint, in pounds/inch

C.16.3.4 An ISM_{ratio} greater than 3 may indicate that the PCC durability at the slab corner or joint is poor. If it is between 3 and 1.5, the durability is questionable. Finally, if the ratio is less than 1.5, the PCC is probably in good condition. These ranges are based on the assumption that the durability at the PCC interior is excellent. This assumption can be verified by reviewing the modulus values obtained from back-calculation analysis of the PCC layer.

C.16.3.5 Use of the ISM_{ratio} for HMA overlaid PCC pavements has the advantage of eliminating the "HMA compression" effect that occurs during NDT. Assuming that the HMA layer is the same thickness throughout the PCC slab and that its condition (for example, stiffness and extent of shrinkage cracks) is relatively constant throughout the slab, there should be approximately the same amount of HMA compression at the slab center, corner, and joint. The net effect is that the ISM_{ratio} will primarily reflect the durability of the PCC layer.

C.17 FWD/HWD Derived Evaluation and Design Inputs.

C.17.1 This section provides guidance on use of inputs developed from deflection data for structural evaluation and design in accordance with ACs 150/5320-6 and 150/5335-5. These inputs are used for pavement evaluation and design including; layer thickness, layer elastic moduli, CBR values, subgrade elastic moduli, and k-values. The engineer should know what evaluation or design program they will use when conducting back-calculation analyses.

C.17.2 For a more conservative evaluation or design approach, AC 150/5320-6 recommends that in general, the mean minus one standard deviation may be used for establishing evaluation and design inputs. If the coefficient of variation is large, i.e., greater than 20 percent, outliers should be removed to compute the mean minus one standard deviation. If outliers are not removed, this approach leads to the use of a pavement characteristic value (for example, ISM or elastic modulus) that is less than 85 percent of all section values for a normally distributed population. If the outliers are removed and the use of a mean minus one standard deviation continues to lead to unreasonable low input values, the engineer should consider division of the existing pavement section into two or more subsections.

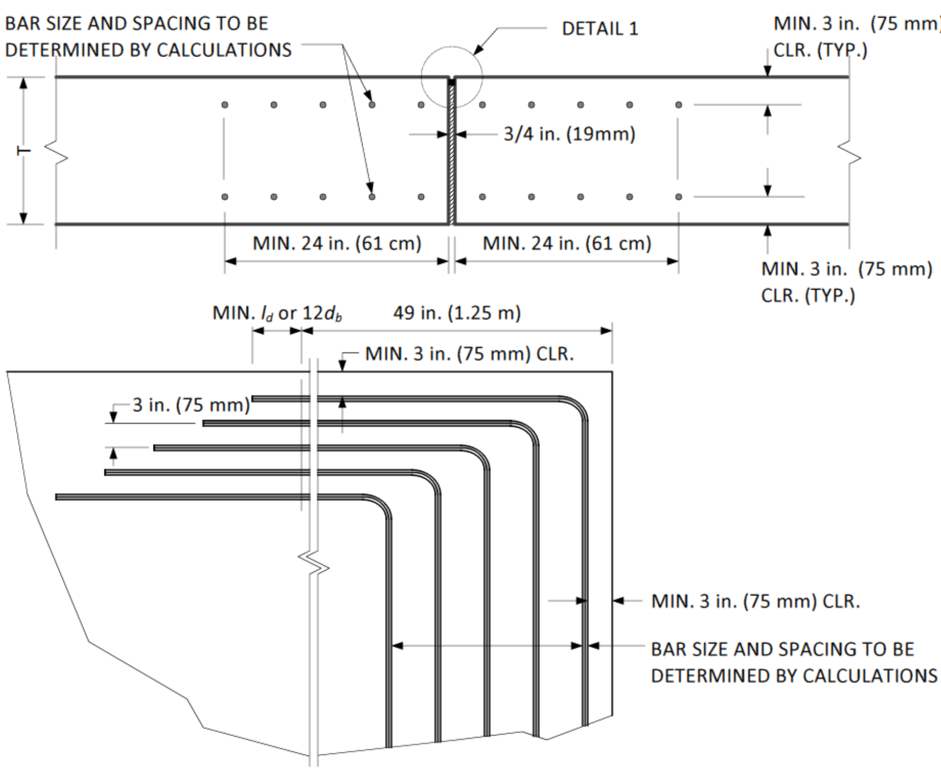
C.17.3 Use of Back-Calculated HMA and PCC Surface Moduli.

The allowable range of modulus values in FAARFIELD are given in Table 3-2. Existing pavement layers may need to be modeled as undefined or variable layers in FAARFIELD. The engineer should verify that the material layer data falls within these ranges. If the material layer data does not fall within the limits given, appropriate adjustments, either up or down must be made for the material layer. Do not go above the upper limit for the material. If the material layer data falls below the lower value, the engineer must adjust the layer type to reflect the lower value.

APPENDIX D. REINFORCED ISOLATION JOINT.

A reinforced isolation joint (Type A-1 in) can be used as an alternative to a thickened edge joint for PCC slabs that are greater than or equal to 9 inches. Sufficient steel reinforcement should be provided at the bottom of the slab for the reinforced concrete section to resist the maximum bending moment caused by the most demanding aircraft loading the free edge of the slab, assuming no load transfer, and application of the load factor (1.7 for live-load). The amount of steel provided must be supported by structural calculations. An equal amount of steel reinforcement must be placed at the top of the slab to resist negative moments that may arise at the slab corners. Any additional embedded steel used for crack control should conform to the requirements of paragraph 3.14.12.2. Where a reinforced isolation joint intersects another joint, the reinforcing steel should not be terminated abruptly, nor should it continue through the intersecting joint. At each intersecting joint, both top and bottom reinforcing bars should be bent 90 degrees in the horizontal plane and continue at least one bar development length (ld) or 12 bar diameters (12 db) beyond a point located a distance 49 inches (1.25 m) from the face of the isolation joint, as shown in Figure D-1. In all cases, a minimum of 3 inches (75 mm) clear cover shall be maintained on all reinforcing bars.

Figure D-1. Type A-1 Joint Detail



TYPE A-1 REINFORCED BAR TERMINATION DETAIL (TOP AND BOTTOM)

D.1 Design Example Reinforced Isolation Joint (Type A-1).

D.1.1 A new rigid pavement will be constructed for the following mix of airplanes: DC10-10, B747-200B Combi Mixed, and B777-200ER. An isolation joint will be provided at the location of planned future expansion. Because of the potential for trapped water, a reinforced isolation joint is selected. Assume that the concrete compressive strength f_c' = 4,000 psi (27.6 MPa). Using FAARFIELD, the PCC design thickness for a 20-year life was determined to be 15.0 inches (381 mm). The maximum stress to be used for the joint design is determined using FAARFIELD as follows:

1. In the Options screen, under "General Options," check the "Out Files" box.
2. Run a "Life" computation for the design section, using the design traffic mix. It is not necessary to run separate computations for each airplane.
3. For each airplane, obtain the computed PCC slab horizontal (tensile) edge stress from the output file NikePCC.out, in the FAARFIELD working directory. Note: The two stresses are reported for each airplane in the mix, the "Edge" stress and the "Interior" Stress. (The stress marked "PCC SLAB HOR STRESS" is simply the larger of the two values.) Disregard the "Interior" stress. Also note that stress values are in psi.
4. For the maximum "Edge" stress found in step 3, calculate the free edge stress by dividing the PCC slab horizontal stress by 0.75. (Dividing by 0.75 is necessary because the FAARFIELD edge stress has already been reduced by 25 percent to account for assumed joint load transfer.)

D.1.2 For this design example, the maximum PCC horizontal edge stress from the output file NikePCC.out was found to be 356.87 psi, for the B747-200B. Therefore, the maximum (working) free edge stress for the concrete section design is calculated as 356.87/0.75 = 475.83 psi.

D.1.3 The reinforced concrete section will be designed using the ultimate strength method. The dead load will be neglected.

1. Assuming a live load factor of 1.7, calculate the ultimate bending moment M_u:

$$M_u = 1.7 \times \frac{\sigma_{edge} \times I_g}{c} = 1.7 \times \frac{475.83\text{ psi} \times \left[\frac{(15.0\text{ in.})^3 \times 12\text{ in.}}{12}\right]}{7.5\text{ in.}} = 364{,}009\text{ lb.-in.} = 30.3\text{ kip-ft.}$$

where:

σ_{edge} = the maximum free edge stress based on FAARFIELD (step 4 above),

I_g = the gross moment of inertia calculated for a 1-foot strip of the concrete slab, and

c = the distance from the neutral axis to the extreme fiber, assumed to be one-half of the slab thickness.

2. Assume the bottom edge reinforcement will consist of No. 6 bars spaced at 6 inches at the bottom of the slab, as shown in figure 3-8. Neglecting the contribution of the top (compressive) steel to the moment resistance, calculate the flexural design strength using the following equation:

$$\phi M_n = \phi A_s f_y d\left[1-0.59\left(\rho\frac{f_y}{f_c'}\right)\right]$$

where:

ϕ = stress reduction factor (= 0.90 for flexure without axial loading)

A_s = steel area = 2 x 0.44 = 0.88 in^2 for 1-ft. width

f_y = steel yield stress (assume f_y= 60,000 psi)

f_c' = concrete compressive strength

d = depth to steel centroid

ρ = steel ratio $=\frac{A_s}{bd}$

b = section width = 12 in.

3. For the minimum 3 in. (76 mm) clear cover on No. 6 bars, d = 11.63 in. (295 mm). Using the above values, ϕM_n is calculated as 43.5 kip-ft. Since $M_u < \phi M_n$, the design is adequate for flexure.

4. A check should also be performed for minimum and maximum steel ratio. The minimum steel ratio is given by:

$$\rho_{\min} = \frac{200}{f_y}$$

where f_y is in psi. From the above values, obtain ρ_{min} = 0.0033.

The calculated steel ratio 0.0063 > 0.0033, hence the minimum steel ratio criterion is satisfied.

5. The maximum steel ratio is determined from the equation:

$$\rho_{\max} = 0.75\times\rho_b = 0.75\times\left[0.85\times\beta_1\frac{f_c'}{f_y}\frac{87000}{87000+f_y}\right] = 0.0213$$

where:

ρ_b = the balanced steel ratio,

β_1 = 0.85 (for f'_c = 4000 psi) and

f_y is in psi.

6. Since the calculated steel ratio ρ = 0.0060 < 0.0213, the maximum steel ratio criterion is also satisfied. For the final design, provide five (5) no. 6 bars spaced at 6 inches (152 mm) on centers.

APPENDIX E. VARIABLE SECTION RUNWAY.

E.1 Runways may be constructed with a transversely variable section. Variable sections permit a reduction in the quantity of materials required for the upper pavement layers of the runway. The following criteria should be considered when designing a variable section pavement.

E.2 Typically, the designer should specify full pavement thickness where departing traffic will be using the pavement. This typically includes the keel section of the runway, entrance taxiways, and aprons. The full-strength keel section is the center 50 feet (15 m) of a 150-foot wide runway.

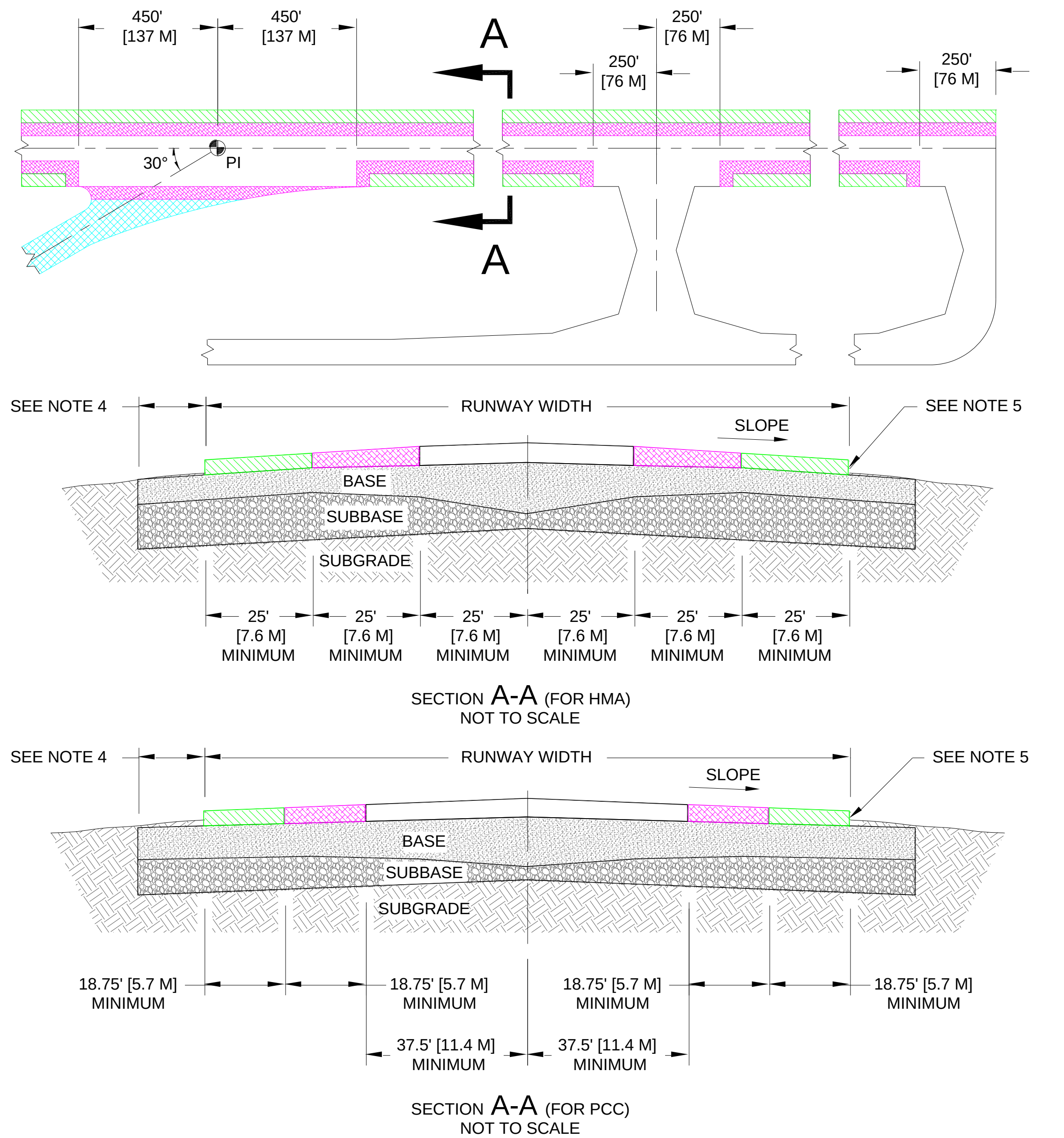
E.2.1 For high speed exits, the pavement thickness is designed using arrival weights and estimated frequency.

E.2.2 Along the extreme outer edges of the runway where pavement is required but traffic is unlikely, the pavement thickness is designed using the departure weights and 1 percent of estimated frequency.

E.2.3 Construction of variable sections is usually more costly due to the complex construction associated with variable sections and this may negate any savings realized from reduced material quantities.

E.3 For the variable thickness section of the thinned edge and transition section, the reduction applies to the concrete slab thickness. The change in thickness for the transitions should be accomplished over an entire slab length or width. In areas of variable slab thickness, the subbase thickness must be adjusted as necessary to provide surface drainage from the entire subgrade surface. Pavement thicknesses should be rounded to nearest 0.5 inch (1 cm). Typical plan and section drawings for transversely variable section runway pavements are shown in following figure.

Figure E-1. Variable Runway Cross-Section



NOTES:

1. RUNWAY AND TAXIWAY WIDTHS, TRANSVERSE SLOPES, ETC. PER AC 150/ 5300-13, AIRPORT DESIGN
2. SURFACE, BASE, PCC, ETC. THICKNESS PER AC 150/5320-6.
3. SECTIONS BASED ON 150 FT [46 M] RUNWAY WIDTH.
4. MINIMUM 12 INCHES [30 CM] UP TO 36 INCHES [90 CM] ALLOWABLE.
5. CONSTRUCT A 1.5 INCH [4 CM] DROP BETWEEN PAVED AND UNPAVED SURFACES.
6. WIDTH OF TAPERS AND TRANSITIONS ON RIGID PAVEMENTS TO BE AN EVEN MULTIPLE OF SLABS, MINIMUM ONE SLAB WIDTH.

LEGEND:

- FULL PAVEMENT THICKNESS (DESIGN USING 100% DEPARTURE TRAFFIC)
- PAVEMENT THICKNESS TAPERS TO REDUCED THICKNESS OF OUTER EDGE THICKNESS, FULL PAVEMENT THICKNESS AND/OR HIGH-SPEED TAXIWAY EXITS AND SIMILAR.
- OUTER EDGE THICKNESS (DESIGN USING 1% DEPARTURE TRAFFIC)
- HIGH-SPEED TAXIWAY EXITS AND SIMILAR (DESIGN USING ARRIVAL TRAFFIC)

APPENDIX F. RELATED READING MATERIAL

F.1 The following advisory circulars are available for download on the FAA website (http://www.faa.gov/airports/resources/advisory_circulars):

1. AC 150/5300-9, *Predesign, Prebid, and Preconstruction Conferences for Airport Grant Projects.*
2. AC 150/5300-13, *Airport Design.*
3. AC 150/5320-5, *Surface Drainage Design.*
4. AC 150/5320-12, *Measurement, Construction and Maintenance of Skid Resistance Airport Pavement Surfaces.*
5. AC 150/5320-17, *Airfield Pavement Surface Evaluation and Rating Manual.*
6. AC 150/5335-5, *Standardized Method of Reporting Airport Pavement Strength-PCN.*
7. AC 150/5340-30, *Design and Installation Details for Airport Visual Aids.*
8. AC 150/5370-10, *Standard for Specifying Construction of Airports.*
9. AC 150/5370-11, *Use of Nondestructive Testing Devices in the Evaluation of Airport Pavement.*
10. AC 150/5370-14, *Hot Mix Asphalt Paving Handbook.*
11. AC 150/5380-6, *Guidelines and Procedures for Maintenance of Airport Pavements.*
12. AC 150/5380-7, *Airport Pavement Management Program (PMP).*

F.2 The following order is available for download on the FAA website (http://www.faa.gov/airports/resources/publications/orders/)

1. Order 5100.38, *Airport Improvement Program Handbook.*
2. Order 5300.7, *Standard Naming Convention for Aircraft Landing Gear Configurations.*

F.3 Copies of the following technical reports may be obtained from the National Technical Information Service (http://www.ntis.gov):

1. DOT/FAA/AR-04/46, *Operational Life of Airport Pavements,* by Garg, Guo, and McQueen, December 2004.
2. FAA-RD-73-169, *Review of Soil Classification Systems Applicable to Airport Pavement Design,* by Yoder, May 1974; AD-783-190.
3. FAA-RD-73-198, Vol. 1, *Comparative Performance of Structural Layers in Pavement Systems. Volume I. Design, Construction, and Behavior under Traffic*

of Pavement Test Sections, by Burns, Rone, Brabston, and Ulery, June 1974; AD-0785-024.

4. FAA-RD-73-198, Vol. 3, *Comparative Performance of Structural Layers in Pavement Systems, Volume III: Design and Construction of MESL,* by Hammitt, December 1974; ADA-005-893.
5. FAA-RD-74-030, *Design of Civil Airfield Pavement for Seasonal Frost and Permafrost Conditions,* by Berg, October 1974; ADA-006-284.
6. FAA-RD-74-033, Vol. 3, *Continuously Reinforced Concrete Airfield Pavement. Volume III. Design Manual for Continuously Reinforced Concrete Pavement,* by Treybig, McCullough, and Hudson, May 1974; AD-0780-512.
7. FAA-RD-74-036, *Field Survey and Analysis of Aircraft Distribution on Airport Pavements,* by Ho Sang, February 1975; ADA-011-488.
8. FAA-RD-74-039, *Pavement Response to Aircraft Dynamic Loads. Volume II. Presentation and Analysis of Data,* by Ledbetter, September 1975, ADA-022-806.
9. FAA-RD-74-199, *Development of a Structural Design Procedure for Flexible Airport Pavements,* by Barker, and Brabston, September 1975; ADA-019-205.
10. FAA-RD-75-110, Vol. 2, *Methodology for Determining, Isolating, and Correcting Runway Roughness*, by Seeman, and Nielsen, June 1977; ADA-044-328.
11. FAA-RD-76-066, *Design and Construction of Airport Pavements on Expansive Soils,* by McKeen, June 1976; ADA-028-094.
12. FAA-RD-76-179, *Structural Design of Pavements for Light Aircraft, by Ladd, Parker, and Pereira,* December 1976; ADA-041-300.
13. FAA-RD-77-81, *Development of a Structural Design Procedure for Rigid Airport Pavements,* by Parker, Barker, Gunkel, and Odom, April 1979; ADA-069-548.
14. FAA-RD-81-078, *Economic Analysis of Airport Pavement Rehabilitation Alternatives – An Engineering Manual,* by Epps, and Wootan, October 1981; ADA-112-550.
15. FAA-PM-84/14, *Performance of Airport Pavements under High Traffic Intensities.*
16. DOT/FAA/PM-85115, *Validation of Procedures for Pavement Design on Expansive Soils,* by McKeen, July 1985; ADA-160-739.
17. FAA-PM-87/19, *Design of Overlays for Rigid Airport Pavements,* by Rollings, April 1988, ADA-194-331.

F.4 Copies of ASTM standards may be obtained from the ASTM International, 100 Barr Harbor Drive, PO Box C700, West Conshohocken, Pennsylvania, 19428-2959 or from the ASTM International website: http://www.astm.org/Standard/standards-and-publications.html.

F.5 Copies of Unified Facility Criteria (UFC) may be obtained from the National Institute of Building Sciences Whole Building Design Guide website: https://www.wbdg.org/.

F.6 Copies of Asphalt Institute publications are available from Asphalt Institute, 2696 Research Park Drive, Lexington, KY 40511-8480 or their website: http://www.asphaltinstitute.org/.

F.7 **Miscellaneous.**

1. Soil Cement Construction Handbook, Portland Cement Association, 5420 Old Orchard Road, Skokie, Illinois 60077, 1995. (www.cement.org)
2. Pavement Management for Airports, Roads and Parking Lots, M.Y. Shahin,2005
3. FHWA-HI-95-038, Geosynthetic Design and Construction Guidelines, 1995. (Development of Guidelines for Rubblization, Airfield Asphalt Pavement Technology Program (AAPTP) Report 04-01, by Buncher, M. (Principal Investigator), Fitts, G., Scullion, T., and McQueen, R., Draft Report, November 2007. (http://www.aaptp.us/reports.html)
4. Best Practices for Airport Concrete Pavement Construction, EB102, American Concrete Pavement Association, 9450 Bryn Mawr, STE 150, Rosemont, IL 60018Basic Asphalt Recycling Manual, Asphalt Recycling and Reclaimation Association, #3 Church Circle, PMB 250, Annapolis, Maryland 21401

Advisory Circular Feedback

If you find an error in this AC, have recommendations for improving it, or have suggestions for new items/subjects to be added, you may let us know by (1) mailing this form to:

Federal Aviation Administration
Airport Engineering Division (AAS-100)
800 Independence Avenue SW
Washington, DC 20591

or (2) faxing it to the attention of Manager, Airport Engineering Division (AAS-100), (202) 267-8663.

Subject: AC 150/5320-6F Date: ______________________

Please check all appropriate line items:

☐ An error (procedural or typographical) has been noted in paragraph ___________ on page ______________.

☐ Recommend paragraph _______________ on page _______________ be changed as follows:

____
____
____

☐ In a future change to this AC, please cover the following subject:
(Briefly describe what you want added.)

____
____
____

☐ Other comments:

____
____
____

☐ I would like to discuss the above. Please contact me at (phone number, email address).

Submitted by: _____________________________ Date: ______________________________