U.S. Department
of Transportation
Federal Aviation
Administration

Advisory Circular

Subject: Airport Pavement Design and Evaluation

Date: Draft (target 2020)
Initiated By: AAS-100

AC No: 150/5320-6G
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 150/5335-5D, *Standardized Method of Reporting Airport Pavement Strength – PCR*.

2 Cancellation.

This AC cancels AC 150/5320-6F, *Airport Pavement Design and Evaluation,* dated November 10, 2016.

3 Applicability.

The Federal Aviation Administration (FAA) recommends the use of the guidelines and standards in this AC for the design and evaluation of pavements at airports where aircraft operate. This AC does not constitute a regulation, is not mandatory, and is not legally binding. It will not be relied upon as a separate basis by the FAA for affirmative enforcement action or other administrative penalty. Conformity with this AC is voluntary, and nonconformity will not affect rights and obligations under existing statutes and regulations, however the following applies:

1. The use of this AC is mandatory for all projects funded under Federal grant assistance programs, including the Airport Improvement Program (AIP). See Grant Assurance No. 34, *Policies, Standards, and Specifications*.
2. This AC is mandatory, as required by regulation, for projects funded with the Passenger Facility Charge program. See PFC Assurance #9, *Standards and Specifications*.
3. This AC only applies to the design of pavements that are used by aircraft.

4 Principal Changes.

This AC contains the following changes:

1. Reformatted to comply with FAA Order 1320.46, *FAA Advisory Circular System*.

2. Added Chapter 2 discussion regarding subgrade stabilization.
3. Expanded Chapter 3 discussion of drainage layers. Revised text and design examples to incorporate changes in FAARFIELD v2.0 pavement design software. Minimum construction layer thickness adjusted. Rigid pavement joint spacing included option for technical analysis.
4. Pavement preservation included in Chapter 4 as an option for flexible pavements. Expanded discussion regarding reuse of existing pavement materials.
5. Updated pavement strength reporting reflecting changes in ICAO pavement strength reporting adopting new ICAO Aircraft Classification Rating/Pavement Classification Rating (ACR-PCR) protocol.
6. Expanded discussion in Appendix C of Nondestructive Testing.
7. Added Appendix D on Dynamic Cone Penetrometer.
8. Added Appendix E on Ground Penetrating Radar.
9. Added Appendix G with example of adding User Defined Vehicles to FAARFIELD.
10. Added Appendix H with all FAARFIELD examples.
11. Added Appendix I on Variable Section Runways.

5 Related Reading Material.

The publications listed in Appendix J 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.

John R. Dermody
Director of Airport Safety and Standards

CONTENTS

CONTENTS (CONTINUED)

CONTENTS (CONTINUED)

Paragraph **Page**

FIGURES

Number **Page**

TABLES

Number **Page**

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:

1. The subgrade (naturally occurring soil),
2. The paving materials (surface layer, base, and subbase),
3. The characteristics of applied loads (weight, tire pressure, location and frequency), and
4. Climate.

1.1.2 Airport pavements are designed and constructed to provide adequate support for the loads imposed by airplanes and to produce a surface that is:

1. firm,
2. stable,
3. smooth,
4. skid resistant,
5. year-round, all-weather surface,
6. free of debris or other particles that can be blown or picked up by propeller wash or jet blast.

1.1.3 To fulfill these performance requirements the pavement will need:

1. Structural capacity to support the imposed loads,
2. Sufficient inherent stability to withstand the abrasive action of traffic, adverse environmental conditions, and other deteriorating influences.
3. To be constructed properly using quality materials and workmanship and
4. To be maintained with regular and routine maintenance.

1.2 Pavement Design Standards.

1.2.1 Flexible Pavement.

The flexible pavement design guidance in this AC is based on layered elastic theory.

1.2.2 Rigid Pavement.

The rigid pavement design guidance in this AC is based on both layered elastic theory and three-dimensional finite element theory.

1.2.3 The failure curves have been calibrated with full-scale pavement tests at the FAA National Airport Pavement Test Facility (NAPTF).

1.3 FAA Pavement Design Program.

1.3.1 FAARFIELD.

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.

1.4 Evaluation of Existing Pavements.

This AC presents 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. Current pavement design methods may produce different pavement thicknesses than the methods used to design the original pavement. Use engineering judgment when evaluating results.

1.5 Construction Specifications and Geometric Standards.

1.5.1 Specifications.

Construction material specifications referenced by Item Number (e.g. P-401, Asphalt Mixture Pavements; P-501, Cement Concrete Pavement, etc.) are contained in AC 150/5370-10, *Standard Specifications for Construction of Airports.*

1.5.2 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.* Runway length requirements are discussed in AC 150/5325-4, *Runway Length Requirements for Airport Design.*

1.6 Airfield Pavements.

1.6.1 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.

1.6.1.1 Flexible pavements are those in which each structural layer is supported by the layer below and ultimately supported by the subgrade. Typically, the surface course for flexible pavements is asphalt mix, Item P-401.

1.6.1.2 Rigid pavements are those in which the principal load resistance is provided by the slab action of the surface concrete layer. Typically, the surface course for rigid pavements is cement concrete pavement, Item P-501.

1.6.2 Selection of Pavement Type.

1.6.2.1 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). See section 3.11 for factors to consider when evaluating pavement life. However, no pavement structure will perform for the desired service life without using quality materials installed and maintained with timely routine and preventative maintenance.

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

1.6.3 Cost Effectiveness Analysis.

1.6.3.1 When considering alternative pavement sections, assume 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. For real discount rates, refer to OMB Circular A-94, Appendix C, *Discount Rates for Cost-Effectiveness, Lease Purchase, and Related Analysis.* 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. Use engineer experience to establish the initial cost and life expectancy of the various alternatives, 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:

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

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, check with the FAA before using other analysis periods.
$\left(\frac{1}{1+r}\right)^{n}$		is commonly called the single payment present worth factor in most engineering economic textbooks

1.6.3.2 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.

1.6.3.3 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).

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.

1.6.3.4 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. Base salvage value 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.

1.6.3.5 Just because a life cycle cost analysis supports a pavement section does not ensure that funds will be available to support the initial construction.

1.6.3.6 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*.

1.6.4 Pavement Structure.

A 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 cement concrete and asphalt mixture.
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

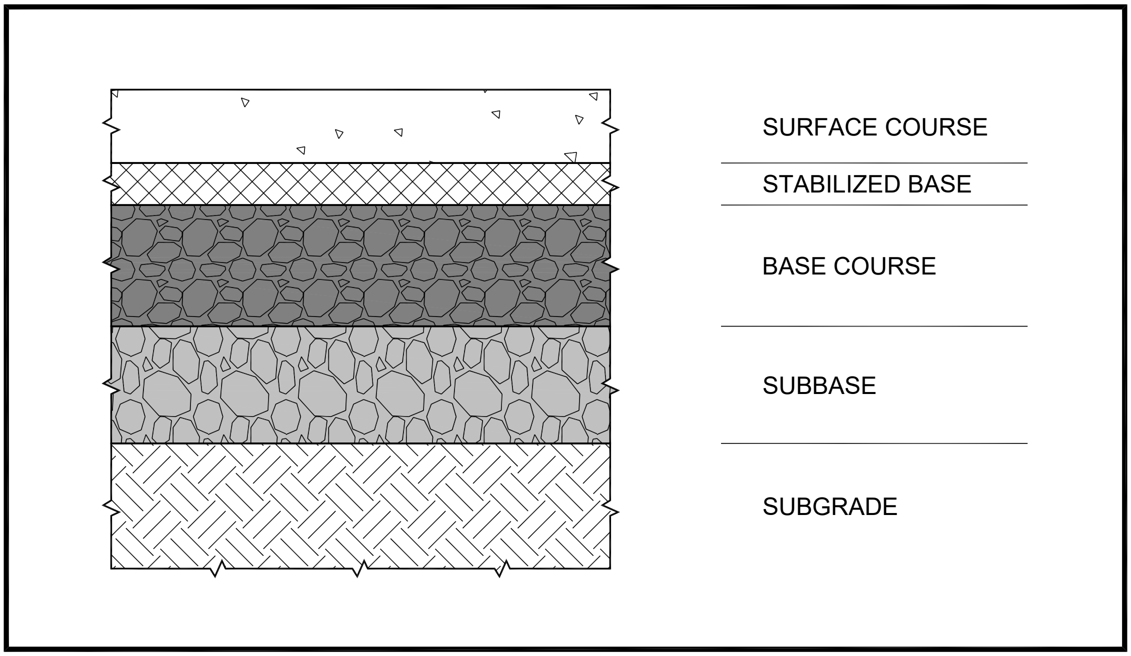


Table 1-1. Typical Pavement Specifications for Pavement Layers[1]

Pavement Layer	Pavement Specification
Surface Course	P-501/P-401[2]/P-403[2]
Stabilized Base Course	P-401/403 P-304[3] P-306[3] P-307[3]
Base Course	P-207[7] P-208[4] P-209[7] P-210 P-211[7] P-212 P-219[6] P-220[5]
Subbase Course	P-154 P-213[5]
Subgrade	P-152 P-155 P-156 P-157 P-158

Notes:

1. Refer to AC 150/5370-10, *Standard Specifications for Construction of Airports*, for individual specifications.
2. Use P-404 for locations that need a fuel resistant surface.
3. Use caution with P-304,P-306 or P307 all are susceptible to leading to reflective cracking.
4. 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.
5. Use of P-213 and 220 is not recommended where frost penetration into the subbase is anticipated.
6. P-219, Recycled Concrete Aggregate Base Course, quality of materials and gradation determine how P219 will perform.
7. P209/P211/P207 may be used as a stabilized base when geotechnical laboratory testing indicates that have CBR > 100.

1.7 Skid Resistance.

Airport pavements should provide a skid resistant surface that will provide good traction during all weather conditions. 49 USC 47101 f (2) recommends grooving or friction treatment of each primary and secondary runways at commercial service airports. Skid resistance is impacted by the combination of factors including: type of surface, aggregate size, texture, shape and gradation, mineralogy of coarse aggregate, and pavement grade, and smoothness. Refer to AC 150/5320-12, *Measurement, Construction, and Maintenance of Skid Resistant Airport Pavement Surfaces,* for information on construction and maintenance of skid resistant surfaces.

1.8 Staged Construction.

1.8.1 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.).

1.8.2 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.

1.8.3 Consider alignments of future development when selecting the longitudinal grades, cross-slope grade, stub-taxiway grades, etc.

1.8.4 Design each stage to safely accommodate the traffic using the pavement until the next stage is constructed.

1.8.5 Consider the future structural needs for the full-service life of the pavement when evaluating initial section to be constructed.

1.8.6 Design and construct the underlying layers and drainage facilities 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.

1.9 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.

The following sections highlight some of the more important aspects of soil mechanics that are important to the geotechnical and pavement engineers. Utilize a qualified professional geotechnical consultant to identify the type and properties of subgrade materials. Document geotechnical investigations and testing in the engineer's report on federally funded projects. Soil investigations is predominately applicable to construction of new pavements. Limited soil investigations required on rehabilitation projects.

2.1.1 <u>Soil.</u>

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

2.1.2 <u>Classification System.</u>

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. <u>Appendix A</u> provides a summary of general soil characteristics pertinent to pavements.

2.1.3 <u>Drainage.</u>

Soil conditions influence the size, extent, and nature of surface and subsurface drainage structures and facilities. See <u>Chapter 3</u> for general guidance on basic drainage layers. For detailed guidance on design of subsurface drainage layers, refer to <u>AC 150/5320-5</u>, *Airport Drainage Design, Appendix G.*

2.2 Soil Conditions.

2.2.1 <u>Site Investigation.</u>

Assess soil type and properties for all soils encountered on the project. Collect and identify representative samples of the various soils present to determine:

1. The distribution, profile, physical properties, location and arrangement of the various soils;

2. The site topography;
3. Location of the water table.
4. Climate data;
5. Availability and suitability of local aggregate materials for use in construction of the pavement structure;
6. Locations of possible additional borrow areas (if sufficient soils are not available within the boundaries of the airport).

2.2.2 Sampling and Identification Procedures.

See ASTM D 420, *Standard Guide to Site Characterization for Engineering Design and Construction Purposes,* for sampling and surveying procedures and techniques. This method is based on the soil profile. Follow ASTM D 2488, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedures),* 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. 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.

Aerial photography will assist in assessing relief, drainage, and soil patterns. 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 will assist in determining the soil or rock profile and its lateral extent. Due to variations at a site, the spacing of borings cannot always be definitively specified by rule or preconceived plan. Take sufficient borings to identify the extent of soils encountered.

2.3.1.2 Cores of existing pavement provide information about the existing pavement structure. Take color photographs of pavement cores and include with the geotechnical report.

2.3.2 Number of Borings, Locations, and Depths New Construction.

2.3.2.1 The locations, depths, and numbers of borings should be sufficient to determine and map existing soil conditions.

2.3.2.2 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.

2.3.2.3 See Table 2-1 for suggested criteria for the location, depth, and number of borings for new construction. These criteria vary depending upon the local conditions, e.g. number and type of subgrade materials or expected depth of embankment. Fewer borings are acceptable if soil conditions are uniform.

2.3.3 Number of Borings Rehabilitation Projects.

2.3.3.1 For rehabilitation projects, utilize the geotechnical reports and as built plans from previous projects. Supplement with NDT and minimally destructive testing to establish strength of existing materials.

2.3.3.2 When pavement rehabilitation or reconstruction is required due to subgrade failure, obtain sufficient borings to characterize the depth and extent of subgrade material that needs to be improved, or removed and replaced. Improvements may include re-compaction, chemical or mechanical stabilization, or replacement with suitable material.

2.3.3.3 See Chapter 4 for additional information on pavement rehabilitation projects.

Table 2-1. Typical Subsurface Boring Spacing and Depth for New Construction[1,2]

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.

Note 2: Follow geotechnical engineer recommendations for depth of borings under deep fills.

2.3.4 Soil Exploration Boring Log.

2.3.4.1 Summarize the results of the soil explorations in boring logs. A typical boring log includes:

1. Location of the boring,
2. Date performed,
3. Type of exploration,
4. Surface elevation,
5. Depth of materials,
6. Sample identification numbers,
7. Classification of the material,
8. Location of water table, and
9. Soil standard penetration resistance.

2.3.4.2 Refer to ASTM D 1586 *Standard Test Method for Standard Penetration Test (SPT) and Split Barrel Sampling of Soils.*

2.3.4.3 Obtain representative samples of the different soil layers encountered and perform laboratory tests to determine their physical and engineering properties. It is important that each sample tested be representative of a particular soil type and not be a mixture of several materials. Identification of soil properties from composite bag samples can lead to misleading representation of soil properties.

2.3.4.4 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.*

2.3.5 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.

2.3.6 Number of Cores.

Cores of existing pavement structure aid in the determination of the extent of rehabilitation and/or reconstruction required to correct the distress. Take sufficient cores to identify and evaluate condition of existing pavement structure and to help characterize extent and possible causes of distress.

2.3.7 Nondestructive and Minimally Destructive Testing.

Additional steps that may be taken to characterize the subsurface include nondestructive testing (NDT) such as Dynamic Cone Penetrometer (DCP) tests, or Ground Penetrating Radar (GPR).

2.3.7.1 NDT using falling weight deflectometer, 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.

2.3.7.2 Dynamic cone penetrometer (DCP) tests, per ASTM D 6951, *Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications,* can quickly provide useful information regarding relative strength of material. DCP testing is classified as a minimally destructive test. Perform DCP tests on each soil layer during soil borings or after taking pavement cores of existing pavements. DCP results can provide a quick estimate of subgrade strength utilizing correlations between DCP and CBR. In addition, plots of DCP results provide a graphical representation of the relative strength of subgrade layers. See Appendix D for additional information on DCP.

2.3.7.3 Ground Penetrating Radar (GPR) can provide a continuous profile of subsurface conditions. GPR has the potential to assist with identification of several subsurface conditions such as: providing a rough estimate of thickness of subsurface pavement layers; location of subsurface objects; help detect stripping or layer separation; detect subsurface moisture; identify any anomalies or changes in subsurface support. See Appendix E for additional information on GPR.

2.3.8 Soil Tests.

2.3.8.1 **Soil Testing Requirements.**

Identify the tests necessary to characterize the soil properties for the project in the geotechnical report. Subsurface evaluations typically 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 outlines how to prepare air dried samples for particle-size and plasticity tests.
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*.
4. The Unified Soil Classification System (ASTM D 2487) uses the plastic limit, liquid limit, and plasticity index of soils to classify soils.
5. 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 from a plastic to a liquid state, respectively.
6. 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.
7. These PL, LL and PI properties are used, either individually or combined with other soil properties, to correlate engineering behavior such as compressibility, permeability, compactibility, shrink-swell, and shear strength.

2.3.8.2 **Moisture-Density Relations of Soils.**

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

2.3.8.2.1 **Pavements Loads of 60,000 Pounds (27,200 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)).

2.3.8.2.2 **Pavement Loads Less than 60,000 Pounds (27,200 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)).

2.4 Soil Strength Tests.

2.4.1 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.

2.4.2 Characterize subgrade materials by a suitable strength or modulus parameter for pavement design and evaluation. 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.

2.4.3 Typically, CBR tests are used to establish the strength of subgrade for flexible pavements. The elastic modulus E can then be estimated for fine grained non expansive soils can be estimated from CBR using the following correlation: E (psi) = 1500 × CBR or E (MPa) = 10 × CBR. This correlation is an approximate relationship generally adequate for pavement design and analysis. Other correlations may be used such as, M_R=2,555 × CBR$^{0.64}$ from AASHTO 2002 Design Guide.

2.4.4 For rigid pavements, measure the strength of the subgrade with 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 × k$^{1.284}$ (k in pci). This correlation is an approximate relationship that is adequate for pavement design and analysis. If plate-load test data is not available, then estimate the elastic modulus E from CBR using the formula in paragraph 2.4.3.

2.4.5 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 heavy weight deflectometer (HWD) data or other nondestructive testing (NDT) using the methods described in Chapter 5 and Appendix C.

2.4.6 California Bearing Ratio (CBR).

The CBR test is a penetration test conducted at a uniform rate of strain. The force required to produce a given penetration in the material being tested 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.*

2.4.6.1 **Laboratory CBR.**

Conduct laboratory CBR tests on materials obtained from the site and remolded to the density that will be required 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 some time, typically this is what is referred to as a 'soaked' or 'saturated' CBR. The use of a soaked CBR design value also represents the time of year when the weakest subgrade is present, during periods of high moisture such as during spring thaw.

2.4.6.2 **CBR for 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. It is often necessary to use judgement and experience to assign CBR values to gravelly subgrade materials. The FAA pavement design procedure recommends a maximum subgrade *E* value of 50,000 psi (345 MPa) (CBR=33) for gravel and gravelly soils.

2.4.6.3 **Number of CBR Tests.**

The exact number of CBR tests required to establish a design value is dependent upon the number, type and nature of soils on the project site. 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.

2.4.7 Lime Rock Bearing Ratio.

When using the lime rock bearing ratio (LBR) to express soil strength, convert to CBR by multiplying the LBR by 0.8. (CBR 100 = LBR 125)

2.4.8 Plate Bearing Test.

2.4.8.1 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).

2.4.8.2 Perform plate bearing tests 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 non-repetitive static plate load tests on subgrade soils and flexible pavement components. It provides subgrade strength data for the evaluation and design of rigid and flexible-type airport and highway pavements.

2.4.8.3 **Plate Bearing Test Conditions.**

Conduct plate bearing tests in the field on test sections constructed to the design compaction and moisture conditions. Correct the k value if necessary to match the moisture conditions expected of the in-service pavement.

2.4.8.4 **Plate Size.**

Characterize subgrade strength with either elastic modulus (E) or resilient modulus (k value) for FAA rigid pavement design. Use a 30-inch (762mm) diameter plate to determine the k value. Using a smaller plate diameter may result in a higher k value.

2.4.8.5 **Number of Plate Bearing Tests.**

Plate bearing tests are expensive to perform limiting the number of tests that can be conducted to establish a design. Due to the limited number of tests, conservatively select the design *k* value.

2.4.8.6 When plate bearing test data is not available the *k* value may be estimated from available CBR data, see paragraph 3.14.4.

2.4.9 Additional Soil Strength Tests.

Other tests to assist in evaluating subgrade soils include:

1. ASTM D 3080, *Standard Test Method for Direct Shear Tests of Soils Under Consolidated Drained Conditions,*
2. ASTM D 2573, *Standard Test Method for Field Vane Shear Tests in Cohesive Soil, or*
3. ASTM D 2166, *Standard Test Method for Unconfined Compressive Strength of Cohesive Soil.*

2.4.10 Subgrade Support for Pavement Design.

2.4.10.1 The subgrade soil provides the ultimate support for both flexible and rigid pavements and the imposed loads. The pavement structure (surface, base and subbase) distributes the imposed loads to the subgrade over an area greater than the tire contact area.

2.4.10.2 Incorporate the available soils with the best engineering characteristics in the upper layer of the subgrade.

2.4.10.3 Conservatively select the value of subgrade support to use in the structural design. The value used for design should reflect the expected long-term subgrade support. The FAA recommends selecting a subgrade strength value for design that is one standard deviation below the mean.

2.4.10.4 Subbase and base layers are difficult to construct without adequate subgrade support. Constructability issues may require improvements to the subgrade to facilitate construction of the subgrade, subbase and base layers.

2.4.10.5 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.

2.4.10.6 When the design CBR is lower than 3, it is required to improve the subgrade through stabilization or other means. See paragraph 2.4.10.

2.4.10.7 Improving weak subgrades may be more cost effective than providing thicker layers of aggregate base and subbase.

2.5 Subgrade Stabilization.

2.5.1 Where the mean subgrade strength is lower than CBR 5, a modulus of 7,500 psi, it may be necessary to improve the subgrade chemically, mechanically, or by replacement with suitable subgrade material.

2.5.2 When the mean subgrade strength is less than a CBR 3, a modulus of 4,500 psi, it is necessary to improve the subgrade through stabilization or replacement with suitable subgrade material.

2.5.3 Consider subgrade stabilization if any of the following conditions exist: poor drainage, adverse surface drainage, frost, or the need to establish a stable working platform. Use chemical agents, mechanical or geosynthetic methods to stabilized subgrades. When it is not possible to create a stable subgrade with either chemical or mechanical stabilization, remove and replace the unsuitable material.

2.5.4 Consult a geotechnical engineer to determine what long-term strength to use in pavement design for stabilized layers. The FAA recommends using a very conservative estimate of the benefit unless test results are available to substantiate the long-term benefit.

2.5.5 Stabilize subgrade materials to a minimum depth of 12 in (300 mm), or to the depth recommended by the geotechnical engineer. To establish a stable working platform additional thickness may be required. When designing pavements that include a layer of stabilized material model this layer as a user-defined layer when performing pavement structural design in FAARFIELD (see Chapter 3).

2.5.6 Chemical Stabilization.

2.5.6.1 Chemical stabilization of subgrade soils can increase their strength, bearing capacity, improve their shrink/swell and freeze/thaw characteristics. Different soil types require different stabilizing agents for best results.

2.5.6.2 Cement can stabilize most soils. To facilitate even distribution of cement mix highly plastic clays prior to addition of cement.

2.5.6.3 Lime stabilization is most effective with plastic clayey soils.

2.5.6.4 Sandy soils with a pH < 5.3 or with organic content > 2% are classified as 'poorly reacting soils' and may not react normally with cement. If the existing soil has a low pH, chemical treatments using lime or cement will neutralize the soil and raise the pH. The cement used to neutralize the soil is in addition to the cement used for stabilization purposes.

2.5.6.5 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 Laboratory Handbook, Portland Cement Association; The Asphalt Institute Manual Series MS-19, Basic Asphalt Emulsion Manual; and AC 150/5370-10, Items P-155, P-156, P-157, and P-158. See paragraph 3.13.5.3 for information regarding how to model chemical stabilized layers in FAARFIELD.

2.5.6.6 Both cement and lime stabilization will increase the long term strength of soils. How much they will improve strength is dependent upon the type of soil, amount of cement or lime added as well as depth of treatment. Long term strength gains of 5 times or more of unstabilized strength are possible. Support expected strength of stabilized soil layers with laboratory testing in the geotechnical report. For additional information on cement stabilization see PCA RD125 *Comparative Performance of Portland Cement and Lime Stabilization of Moderate to High Plasticity Clay Soils.* For additional information on lime stabilization see National Lime Association, Bulletin 326, *Lime-Treated Soil Construction Manual.*

2.5.7 Mechanical Stabilization.

2.5.7.1 Not all subgrades can be stabilized with 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 underlying soils.

2.5.7.2 To facilitate construction of the pavement section, extremely soft soils may require bridging of the weak soils with a layer of rock or coarse sand. Bridging can be accomplished with the use of thick layers, 2-3 feet (600-900mm), of shot rock, cobbles or coarse sand. 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.

2.5.7.3 Geosynthetics may be used as the first layer of mechanical stabilization over soft fine-grained soils. The geosynthetic creates a working platform for the construction of the subsequent pavement layers.

2.5.8 Geosynthetics.

2.5.8.1 The term geosynthetics describes a range of manufactured synthetic products used to address geotechnical problems. Geosynthetics includes 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.

2.5.8.2 Include justification in the engineers report from the geotechnical engineer to support and justify what the geosynthetic will provide to the pavement structure. The most common use on airports is as a separation layer to prevent migration of fines, for example to keep fines from migrating into a non-frost susceptible base or subbase. Currently, the FAA does not consider any reductions in pavement structure for the use of any geosynthetics.

2.6 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.

2.6.1 For detrimental frost action, three conditions are needed:

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

2.6.2 Frost Susceptibility.

The size and distribution of voids in the soil mass is one element used to estimate the frost susceptibility of soils. Empirical relationships correlate the degree of frost susceptibility with the soil classification and the amount of material finer than 0.02 mm by weight. ASTM D422, *Standard Test Method for Particle-Size Analysis of Soils,* was withdrawn by ASTM in 2016, but the test method will provide an approximation of the percent material finer than 0.02mm.

2.6.3 For frost design purposes soils are categorized into four frost groups, frost group FG-1, FG-2, FG-3, and FG-4,as defined in Table 2-2. The higher the frost group number, the more frost susceptible the soil, i.e., soils in FG-4 are more frost susceptible than soils in frost groups 1, 2, or 3. Selection of the frost design group is a relative estimation of the potential for a soil to be susceptible to frost heave.

2.6.4 Soils with high liquid limits combined with high silt and clay content are more susceptible to frost heave than soils that have coarser gradation such as gravels or sands.

Table 2-2. Soil Frost Groups[1]

Frost Group	Kind of Soil	Percentage Finer than 0.02 mm by Weight[2]	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 15	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

Note 1: Determination of Frost Group is subjective,
Note 2: ASTM D422, *Standard Test Method for Particle-Size Analysis of Soils,* was withdrawn by ASTM in 2016, but the test method will provide an approximation of the percent material finer than 0.02mm.

2.6.5 Depth of Frost Penetration.

The depth of frost penetration is a function of the thermal properties of the pavement and soil, the surface temperature, the moisture content of the soil, and the temperature of the pavement and soil at the start of the freezing season. In determining the potential frost penetration depth, give consideration to local engineering and construction experience. The depth of frost penetration is dependent upon the moisture content and type of materials used. In general, the lower the moisture content of base and subbase materials, the deeper the frost penetration will be. 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.

2.6.6 Free Water Necessary for Frost Action.

Free water is needed in the soil mass for frost action (formation of ice lenses) to occur. 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. If the degree of saturation of the soil is 70 percent or greater, frost heave will probably occur. For any soil that may be susceptible to frost action, the designer should assume that sufficient water will be present to cause detrimental frost action.

2.6.7 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 and the depth of frost. Edge drain systems are most effective in removing free water when combined with a subsurface drainage layer. Limiting the amount of material retained on the No 200 sieve to less than 5% in base and subbase aggregate layers will help facilitate drainage of these layers. See paragraph 3.8, Drainage Layer, and AC 150/5320-5, *Airport Drainage Design.*

2.7 Frost Design.

2.7.1 See Chapter 3 for guidance on how to offset seasonal frost effects when designing pavements. A more rigorous evaluation for frost effects is necessary when designing for pavement service life greater than 20 years.

2.7.2 See Research Report No. FAA-RD-74-030, *Design of Civil Airfield Pavement for Seasonal Frost and Permafrost Conditions* for a discussion of frost action and its effects.

2.7.3 When economically feasible, it is always desirable to have uniform subgrade materials to minimize the potential for differential frost heave. In areas of significant frost and permafrost it may be necessary to remove and replace materials to a significant depth beneath the pavement.

2.7.4 Permafrost.

2.7.4.1 In arctic regions, it is common to encounter soils that are 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.

2.7.4.2 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.

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

2.7.4.4 Consider the depth of seasonal thaw when designing pavements in areas of permafrost. Base the thawing index for design (design thawing index) 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.

2.7.5 Muskeg.

2.7.5.1 Muskeg is a highly organic soil deposit encountered in arctic areas.

2.7.5.2 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.

2.7.5.3 If the thickness of muskeg is too thick to remove and replace, place a 5-foot (1.5 m) granular fill over the muskeg. This thickness is based on experience, however differential settlement will occur requiring considerable maintenance to maintain a smooth surface. Use a geosynthetic between the muskeg surface and the bottom of granular fill to prevent migration of the muskeg up into the granular fill.

Page Intentionally Blank

CHAPTER 3. PAVEMENT DESIGN

3.1 Design Considerations.

This chapter provides pavement design guidance for airfield pavements. Use the FAA computer program FAARFIELD for all pavement thickness designs regardless of aircraft gross weight. Consider the aircraft fleet that will utilize the pavement over its intended structural life when performing pavement design. Reality is that most pavement designs are controlled by the operations of the most demanding aircraft in the traffic mix, however it is still good practice to consider all aircraft when designing airfield pavements. At small GA airports often the most demanding load is that of maintenance and refueling vehicles. See Chapter 4 for procedures for overlay design, and Chapter 5 for procedures for evaluating pavements.

3.2 FAA Pavement Design.

3.2.1 The design of airport pavements is a complex engineering problem that involves the interaction of multiple variables. FAARFIELD uses layered elastic and three-dimensional finite element-based design procedures for new and overlay designs of flexible and rigid pavements respectively.

3.2.2 On federally funded projects, the structural design of airfield pavements must be based upon the use of FAARFIELD, and the engineers report must include a copy of the FAARFIELD pavement design report.

3.3 Flexible Pavements.

3.3.1 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.

3.3.2 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.

3.4.1 When all aircraft are less than 60,000 pounds (27,200 kg) full-depth asphalt pavements may be used.

3.4.2 FAARFIELD has the ability to analyze full depth asphalt pavements as a 2-layer structure consisting of only the asphalt surface layer and a subgrade layer. However, the preferred method of analyzing a full-depth asphalt pavement is to use a 3-layer structure consisting of an asphalt surface layer on top of an asphalt base (and a subgrade layer).

3.4.3 The Asphalt Institute 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*.

3.5 Rigid Pavements.

3.5.1 For rigid pavement design, FAARFIELD uses the horizontal stress at the bottom of the concrete 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.

3.5.2 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. FAARFIELD will check for minimum thicknesses of stabilized base, base and subbase.

3.6 Stabilized Base Course.

3.6.1 When 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.

3.6.2 Full scale performance tests have shown superior performance of both flexible and rigid pavements that include bases stabilized with asphalt or cement. Evaluate the potential reduction in long term performance 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).

3.6.3 Evaluate subsurface moisture conditions before considering substitution of an asphalt or cement base course with an unstabilized aggregate material. It is preferred to use a base course stabilized with asphalt or cement. Aggregate bases perform best when not saturated.

3.6.4 Materials that exhibit a remolded soaked CBR of 100 or greater and have proven performance under similar aircraft loadings and climatic conditions may be substituted for a stabilized base course. Lime rock must exhibit an LBR of 125 or greater.

3.6.5 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 concurrence during the review of engineer's report on federally funded projects.

3.6.6 Document in engineers report what stabilized base will be used, when pavement design includes aircraft over 100,000 pounds (45,359 kg).

3.7 Base or Subbase Contamination.

3.7.1 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. Contamination reduces the quality of the aggregate material, reducing its structural capacity.

3.7.2 Separation layers, either Geosynthetic separation materials or granular filter layers can be effectively used to reduce contamination from subgrade. In general, separation fabrics have potential for longer functional life than granular filter layers. Over time, granular filter layers become less effective when mixed with the adjacent layers. See paragraph 3.12.16.2 for information on parameters for granular filter layers.

3.7.3 To ensure long term performance of a subbase material needed for frost protection, include a separation layer of either geosynthetic separation material or a 4-inch granular filter layer.

3.8 Drainage Layer.

The use of drainage layers will protect pavements from moisture related subgrade, subbase and base failures. Drainage layers facilitate the quick removal of excess moisture from the pavement structure. General guidance on basic drainage layers is discussed below.

3.8.1 In non-frost areas, include provisions for subsurface drainage when subgrade soils have a coefficient of permeability less than 20 ft/day (6 m/day).

3.8.2 Pavements in frost areas constructed on FG2 or higher subgrade soils should include a subsurface drainage layer.

3.8.3 For rigid pavements, place the drainage layer immediately beneath the concrete slab.

3.8.4 For flexible pavements,

3.8.4.1 Place the drainage layer immediately above the subgrade, except as noted in paragraphs 3.8.7, 3.8.4.2, and 3.8.4.3.

3.8.4.2 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, place the drainage layer beneath the aggregate base and above the granular subbase.

3.8.4.3 When the total thickness of the pavement structure is less than 12 inches (300 mm), place the drainage layer directly beneath the surface layer using the drainage layer as a base.

3.8.5 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. Drainage layers that provide a permeability of 500 – 1500 feet per day may be used without calculations.

3.8.6 In the structural design of sections with drainage layers, model these layers in FAARFIELD as user defined layers. The modulus value assigned to the drainage layer depends upon the material used. The following modulus values may be used:

Asphalt-treated permeable base	150,000 psi
Cement-treated permeable base.	250,000 psi
Aggregate drainage layer (unstabilized)	15,000 – 30,000 psi

3.8.7 When the drainage layer is located beneath an unbound aggregate base, limit the material passing the No. 200 (0.075 mm) sieve in the aggregate base to less than 5 percent.

3.8.8 See EB 102 *Asphalt Treated Permeable Base* for sample specification. See AC 150/5370-10, Item P307, *Cement Treated Permeable Base Course*, for an example of a stabilized drainage layer. See IPRF-01-G-002-1(G) *Stabilized and Drainable Base for Rigid Pavement.*

3.8.9 For additional guidance on subsurface drainage layers, refer to AC 150/5320-5, *Airport Drainage Design, Appendix G, Design of Subsurface Drainage Systems.*

3.9 **Subgrade Compaction.**

3.9.1 FAARFIELD Compaction Depths.

3.9.1.1 The compaction requirements in FAARFIELD are based on the Compaction Index (CI) concept. Background information on this concept can 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).

3.9.1.2 In FAARFIELD, you must complete the thickness design analysis before computing the subgrade compaction requirements.

3.9.1.3 FAARFIELD determines compaction depths using 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.

3.9.1.4 FAARFIELD computes compaction requirements for the specific pavement design and traffic mixture and generates tables of required minimum density requirements for the subgrade beneath pavements. The values in these tables denote the minimum compaction requirements, more restrictive requirements may control on new embankments.

3.9.1.5 FAARFIELD generates two tables one for non-cohesive soil types and one for cohesive soil types. When determining the compaction requirement, non-cohesive soils have a plasticity index of less than 3.

3.9.2 New Embankments.

3.9.2.1 Compact cohesive fill under pavement to greater of depth calculated by FAARFIELD or 12" (300 mm), to 95 percent of maximum density. Compact embankments with cohesive soils outside of paved areas to at least 90 percent of maximum density.

3.9.2.2 Compact the top 6 inches (150 mm) of non-cohesive fill under pavement to 100 percent maximum density, and compact the remainder of the fill to 95 percent maximum density.

3.9.2.3 Adjust compaction requirements to address unique local soil conditions, when supported by a geotechnical engineer's report. When constructing deep fills, soils may require special compaction requirements as directed by the geotechnical engineer.

3.9.3 Cut Sections.

3.9.3.1 Subgrade densities in cut areas must be equal or greater than FAARFIELD compaction requirements.

3.9.3.2 When densities cannot be achieved by reworking and compaction of existing subgrade, remove and replace with suitable select material.

3.9.3.3 It is a good practice to rework and recompact at least the top 12 inches (300 mm) of subgrade in cut areas; however, depending upon the in-place densities, it may be necessary to rework and recompact additional 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.

3.10 Swelling Soils.

3.10.1 Swelling soils are clayey soils that exhibit a significant volume change caused by moisture variations. Pavements constructed on swelling soils are subject to differential movements causing surface roughness and cracking.

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, per ASTM D 1883 *Standard Test Method for California Bearing Ration (CBR) of Laboratory-Compacted Soils,* require treatment.

3.10.4 When swelling soils are present, incorporate methods to prevent or reduce the effects of soil volume changes. Treatment of swelling soils consists of removal and replacement, chemical stabilization, and compaction efforts in accordance with Table 3-1. Adequate drainage is important when dealing with swelling soils. When evaluating mitigation measures consider local experience with mitigation techniques and methods.

3.10.5 For additional information on identifying and handling swelling soils, see 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	Lime or cement stabilize soil to a depth of at least 6 in (150 mm)
Medium	6-10	Low	Lime or cement stabilize soil to a depth of at least 12 in (300 mm)
		High	Lime stabilize soil to a depth of at least 12 in (300 mm)
High	Over 10	Low	Lime or cement 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. Base the design subgrade strength on the moisture content and density used to control swelling.
3. Generally, lime stabilization works best on clay soils and cement on coarser soils with low clay/silt content. However, cement stabilization works on almost all soil types.
4. Use cement stabilization for soils with sulfate content above 3,000 ppm.
5. For lime stabilization, utilize 1-2% more lime than amount needed to increase the soil pH to > 12. Sufficient lime to increase the unconfined compressive strength of the soil at least 50 psi.
6. For cement stabilization, utilize 1-2% more than determined following the PCA method. See *PCA Soil Cement Construction Handbook* or UFC 3-250-11, *Soil Stabilization*.

3.11 Pavement Life.

3.11.1 Design Life in FAARFIELD refers to structural life, the total number of load cycles a pavement structure will carry before it fails structurally. Structural failure for rigid pavements occurs when one half the slabs have a structural (load related) crack. Structural failure for flexible pavements occurs when the subgrade is no longer protected from structural (load related) damage.

3.11.2 Functional or useful life, 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.

3.11.3 Functional life may be more or less than structural life.

3.11.4 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.

3.11.5 A number of factors influence the required thickness of pavement including:

1. The type of structural materials,
2. The magnitude and character of the airplane loads to be supported,
3. The volume and distribution of traffic,
4. The quality and type of materials that make up the pavement structure, and
5. The strength of the subgrade soils.

3.11.6 It is theoretically possible to perform a pavement structural design for any service period. To achieve the intended design life requires consideration of many interacting factors including:

1. Airplane mix,
2. Initial quality of materials and construction, and
3. Timely application of routine and preventative pavement maintenance.

3.11.7 Properly maintained pavements will have a longer functional life.

3.11.7.1 To maximize a flexible pavement's life, routine crack sealing and applications of pavement seal coats and small patches will be required.

3.11.7.2 To maximize a rigid pavements life crack sealing and joint sealant repair/replacement will be required as well as isolated slab replacement.

3.11.8 Due to deterioration from normal use and the environment, both flexible and rigid pavements may require rehabilitation of surface grades and renewal of surface characteristics. A mill and overlay may be required with flexible pavements and surface diamond grinding and isolated slab replacement with rigid pavements.

3.11.9 Design pavements on federally funded FAA projects for a 20-year structural life.

3.11.10 Obtain FAA approval during review of engineers report to use a structural design period other than 20 years on federally funded projects.

3.11.10.1 Phased projects may only require a temporary pavement for 1-2 years.

3.11.10.2 For example, a longer design life may be appropriate at a large hub airport when accurate forecasts of the future aircraft traffic are available and where the size and configuration of the airport 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 since the composition and frequency of future activity is unknown.

3.11.10.3 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).

3.11.10.4 A life cycle cost effectiveness analysis will help 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.

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. Design thicknesses in FAARFIELD are calculated using layered elastic and three-dimensional finite element-based structural analysis 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.

3.12.1.1 FAARFIELD currently does not take into account provisions for frost protection and permafrost discussed in paragraph 3.12.13. It is the responsibility of the user to check these provisions separately from FAARFIELD and to modify the thickness of the pavement structure to provide additional frost and or permafrost resistant materials.

3.12.1.2 Material or construction issues can lead to functional failures in pavements (e.g., excessive roughness, FOD, or surface deformations). These types of issues are not are not addressed directly by FAARFIELD.

3.12.1.3 FAARFIELD design assumes that all pavement layers meet the applicable requirements of AC 150/5370-10 for materials, construction, and quality control. User defined layers must be used when utilizing materials other than FAA standard materials.

3.12.2 Cumulative Damage Factor (CDF).

3.12.2.1 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.

3.12.2.2 Thickness designs using FAARFIELD use the entire 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.

3.12.2.3 Note, 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 2.0. Failure models used in FAARFIELD were calibrated using the most recent full-scale pavement tests at the FAA's National Airport Pavement Test Facility (NAPTF).

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 software is available for download at (https://www.faa.gov/airports/engineering/design_software/).

3.12.4 Overview of FAARFIELD Program.

FAARFIELD consists of a main program that calls several subprograms (libraries), as shown schematically in Figure 3-1. The main subprograms are:

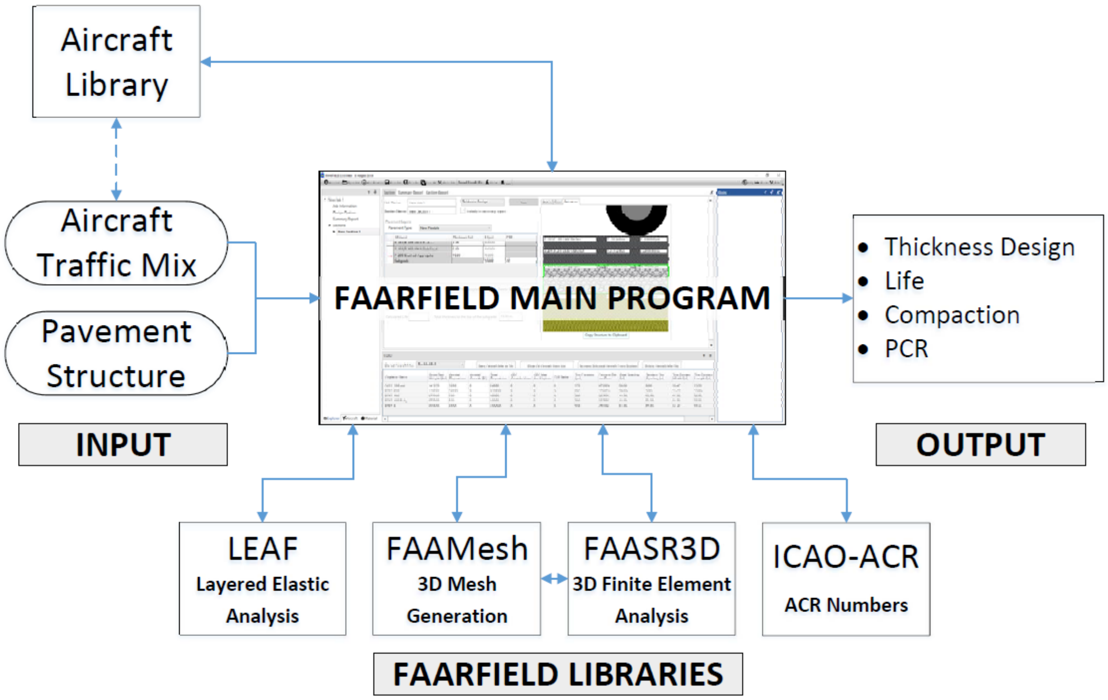
1. LEAF (layered elastic analysis); FAAMesh (three-dimensional mesh generation for finite element analysis);
2. FAASR3D (finite element processing); and
3. ICAO-ACR (ACR computation following the ICAO standard method).

The FAARFIEL D program operates either with U.S. customary or metric dimensions. The FAARFIELD program operates in four functional modes:

1. Thickness Design,
2. Life Computation;
3. Compaction Requirement; and

4. PCR Computation. See AC 150/5335-5 for discussion on the use of FAARFIELD to compute PCR values.

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, see Appendix H for FAARFIELD examples.) The basic FAARFIELD design steps include:

Step 1 After starting the program, select a pavement type.

Step 2 Modify the pavement structure by adding, deleting or changing layers as needed.

Step 3 Create a traffic mix by selecting a stored mix, or by picking aircraft from the aircraft library.

Step 4 If necessary, change the gross weight or number of departures of airplanes in the traffic mix.

Step 5 Run Thickness Design.

Step 6 [Optional] Run Compaction/Life to obtain subgrade compaction requirements.

Step 7 View or print the section design report.

3.12.6 Aircraft Traffic Considerations.

3.12.6.1 **Load.**

Design pavements using the maximum anticipated takeoff weights of the airplanes that will be regularly operating on the pavement. FAARFIELD provides manufacturer-recommended gross operating weights and load distribution, for many civil and military airplanes. For generic aircraft, the load is distributed to the landing gears with 95% to the main and 5% to the nose. Using the maximum anticipated takeoff weight provides a conservative design allowing for changes in operational use and traffic. 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.**

An airplanes gear type and configuration dictates how weight is distributed to a pavement Refer to FAA 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. Tire pressures and gross weight are linked in FAARFIELD. FAARFIELD maintains a constant contact area, therefore an increase in gross weight causes a proportional increase in tire pressure. Tire pressure has a more significant influence on strains in the asphalt surface layer than at the subgrade. Flexible pavements constructed with a highly-stability asphalt will accommodate tire pressures up to 254 psi (1.75 MPa). 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. Seasonal or other non-regular use aircraft may have significant impact on the pavement structure required. Perform a sensitivity analysis comparing 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, include sensitivity analysis and verification of actual activity in the engineers report.

3.12.6.5 Departure Traffic.

Generally, airfield pavements are designed considering only aircraft departures. The main reason for disregarding arrivals in design is that, typically, the arrival weights are much lighter than the departure weights (due to fuel consumption). If airport operations are such that most aircraft arrive and depart at essentially the same weight (for example, if refueling does not take place), then the number of departures in FAARFIELD should be adjusted to reflect the number of times the pavement is actually loaded at the operating weight (whether an arrival or departure). See paragraph 3.12.6.1 regarding thickness design of high-speed exit taxiways and other special cases.

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. Annual growth is calculated using the formula:

$$N = \left(1 + \frac{r \times L}{200}\right) \times N_A \times$$

Where: N is the total lifetime departures, N_A is the annual departures, L is the design life (typically 20 years), and r is the growth rate (percent). For example, FAARFIELD considers 225 annual departures at a 1% annual growth rate to be 4,950 total departures over a 20-year design life. It is not always necessary to include all aircraft that use a facility, but it is necessary to consider all of the heaviest aircraft that use a facility. When a few operations of a heavy aircraft control the design of the pavement structure, perform a sensitivity analysis to determine the impact of the additional operations of that heavy aircraft.

3.12.6.7 Airplane Traffic Mix.

Use the anticipated traffic mix of actual aircraft, for the design computations. Attempting to design for equivalent passes of a “design aircraft” instead of the actual aircraft mix can lead to erroneous results. If a particular aircraft that is part of the anticipated usage does not exist in the FAARFIELD aircraft library, the user can (a) substitute a close aircraft from the “generic” group; or (b) create a user-defined aircraft based on the aircraft gear characteristics. See Appendix G for additional information on building user-defined aircraft.

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, fueling equipment, passenger boarding bridges or ground service 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 common types of truck axles (single, dual, tandem, and dual-tandem). The included truck axles should be adequate for most light-duty pavement designs. See paragraph 3.18 for specific recommendations for passenger loading bridges and paragraph 3.19 for recommendations for ground service equipment.

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.

3.12.8.1 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 coverage of one full-load application.

3.12.8.2 The ratio of number of passes required to apply one coverage to a unit area of the pavement is expressed by the pass-to-coverage (P/C) ratio. The number of passes an airplane may make on a given pavement is easy to observe, but the number of coverages is mathematically derived in FAARFIELD.

3.12.8.3 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.

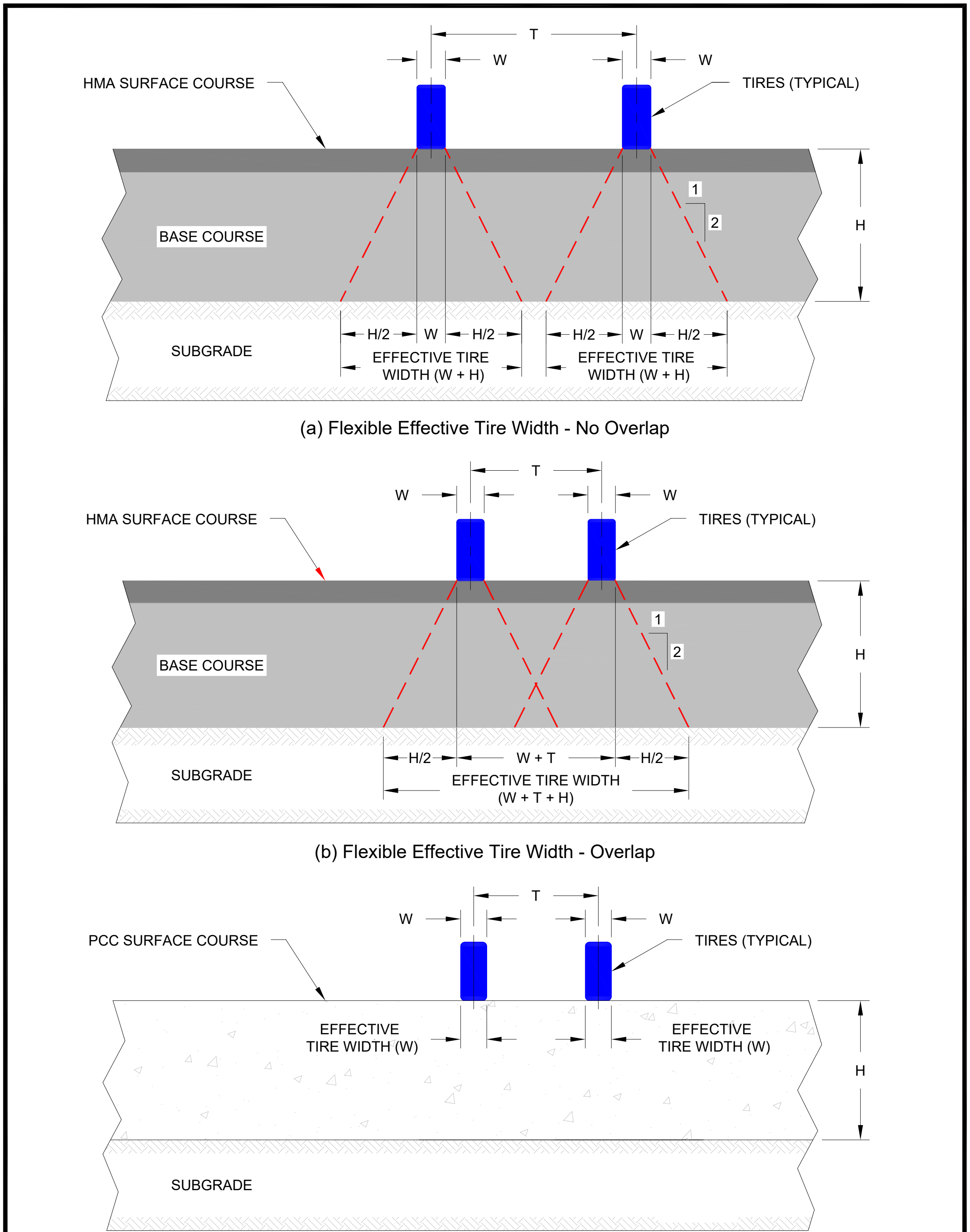
3.12.8.4 For flexible pavements, coverages are a measure of the number of repetitions of the maximum strain occurring at the top of subgrade.

3.12.8.5 For rigid pavements, coverages are a measure of repetitions of the maximum stress occurring at the bottom of the rigid layer (see Report No. FAA-RD-77-81, *Development of a Structural Design Procedure for Rigid Airport Pavements*).

3.12.8.6 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*).

3.12.8.7 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. Establish the flexible effective width by drawing "response lines" from the edges of the tire contact surface to the top of the subgrade at a slope of 1:2 slope. See Figure 3-2. Establish the effective width considering both tires in a landing gear when the response lines from the adjacent tires overlap. For rigid pavements, the effective tire width is equal to the nominal tire contact width at the surface of the pavement. FAARFIELD performs all effective tire width and P/C ratio calculations internally.

Figure 3-2. Effective Tire Width



(a) Flexible Effective Tire Width - No Overlap

(b) Flexible Effective Tire Width - Overlap

(c) Rigid Effective Tire Width

3.12.9 Cumulative Damage Factor.

3.12.9.1 Fatigue failure in FAARFIELD is expressed by a cumulative damage factor (CDF). The CDF is a form of Miner's rule, a cumulative damage model for fatigue failure. Using Miner's rule the total CDF is determined by summing the damage from each individual aircraft. The CDF is a number that represents the amount of structural fatigue life that has been used. Mathematically, CDF is the sum of *N* terms, where each term is the ratio of applied repetitions to allowable repetitions to failure for one of the *N* aircraft in the traffic mix. For a pavement design, the pavement structure thickness is adjusted until CDF = 1 for the given traffic mix and structural design life. For a single airplane ($N = 1$) and constant annual departures, CDF can be expressed as follows:

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

or

$$\text{CDF} = \frac{(\text{annual departures}) \times (\text{life in years})}{(P/C) \times (\text{coverages to failure})}$$

or

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

3.12.9.2 FAARFIELD calculates a CDF for each 10-inch (254-mm) wide strip along the pavement over a total width of 820 inches (20.8 m). FAARFIELD calculates a pass-to-coverage ratio for each strip assuming 75 percent of passes occur 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 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.

3.12.9.3 In FAARFIELD, the "CDF Graph" function displays plots of CDF versus lateral offset for each gear in the design mix, and a plot of total CDF for all airplanes in the mix. For a completed design the peak value of total CDF = 1.0. The offset at which the total CDF = 1.0 for a completed design is the critical offset. See Appendix H for example of CDF concept.

3.12.10 FAARFIELD Material Properties.

3.12.10.1 In FAARFIELD, pavement layers are assigned a thickness, elastic modulus, and Poisson's ratio. Flexible and rigid analysis utilize the same layer properties. FAARFIELD allows layer thicknesses to be varied, subject to minimum thickness requirements. Poisson's ratio is fixed for all materials however; the elastic moduli are dependent upon material type and are either fixed or variable (within a permissible range). Materials are identified in FAARFIELD by the designations as used in AC 150/5370-10; for example, crushed aggregate base course is Item P-209. Included in the list of materials is a user-defined layer with properties that can be set by the user. Table 3-2 lists the modulus values and Poisson's ratios used in FAARFIELD.

3.12.10.2 In a rigid analysis, FAARFIELD requires a minimum of 3 layers (surface, base and subgrade) but allows up to a total of five (5) layers. A flexible design may have an unlimited number of layers or as few as 2 layers (asphalt surface and subgrade).

3.12.10.3 When designing a new pavement, on federally funded projects, use FAA standard materials as specified in AC 150/5370-10 unless the use of other materials has been approved by the FAA as a modification to standards (see FAA Order 5300.1). 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	4,000,000 (30,000)	NA	0.15
	P-401/P-403/P-404 Asphalt Mixture	NA	200,000 (1,380)[1]	0.35
Stabilized Base and Subbase	P-401/P-403Asphalt Mixture	400,000 (3,000)		0.35
	P-306 Lean Concrete	700,000 (5,000)		0.20
	P-304 cement treated aggregate base	500,000 (3,500)		0.20
	P-220 Cement treated soil base	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
	P207 Recycled Asphalt aggregate base [2]	25,000-75,000		0.35
	P-154 uncrushed aggregate	Program Defined		0.35
Subgrade[3]	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 conservative modulus value corresponds to a pavement temperature of approximately 90°F (32°C).
2. The modulus of P207 is dependent upon the quality and if any additional stabilizing material incorporated, e.g. asphalt, cement, fly ash.
3. Model cement stabilized layer as a user-defined layer with a strength up to 50% greater than the subgrade. Model cement/lime kiln dust and fly ash as a user defined layer with a strength up to 20% greater than the subgrade. The use of higher values must be supported by laboratory testing.

3.12.11 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. The gross weight of the heaviest aircraft in the traffic mix determines minimum thickness requirements, regardless of traffic level. FAARFIELD automatically checks the minimum layer thickness requirements for standard materials based on the traffic mix entered, however the user must still verify that all thickness requirements have been met Use the larger of the values from Table 3-3 and Table 3-4 or the thickness as calculated by FAARFIELD rounded up to the nearest inch. Additional thickness may be required for frost protection.

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

Layer Type	FAA Specification Item	Maximum Airplane Gross Weight Operating on Pavement, lbs (kg)		
		<60,000 (27,215)	< 100,000 (45,360)	≥100,000 (45,360)
Asphalt Surface[1]	P-401	3 in (75 mm)	4 in (100 mm)	4 in (100 mm)
Stabilized Base[2]	P-401 or P-403; P-304; P-306[3]	Not Required	Not Required	5 in (125 mm)
Crushed Aggregate Base[4,5]	P-209, P-211	Not Required	6 in (150 mm)	6 in (150 mm)
Aggregate Base[4,5]	P-207, P-208, P-210, P-212, P-213, P-219	6 in (75 mm)	n/a	n/a
Drainable Base	P-307, ATPB[6]		6 in (150 mm) when used	6 in (150 mm) when used
Subbase[5,7]	P-154	6 in (125 mm) (if required)	6 in (125 mm) (If required)	6 in (125 mm) (if required)

Notes:

1. P-404-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-404 considered same as P-401.
2. See paragraph 3.6, Stabilized Base Course, for requirements and limitations.
3. Use of P-304 or P-306 requires measures to control potential for reflective cracking.
4. P-208, P-210, P-212, P-213, limited to pavements designed for gross loads of 60,000 pounds (27,215 kg) or less or for use as subbase.
5. P-207, P-219 require laboratory testing to establish if it will perform as a base or subbase. If CBR > 80 may be used in place of P209, CBR >60 in place of P-208. Both may be used as a subbase under stabilized base.
6. See EB 102 Asphalt Treated Permeable Base.
7. P154, when structural thickness of subbase required by FAARFIELD is less than 6 in, eliminate subbase in FAARFIELD and calculate thickness of 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)		
		<60,000 (27,215)	< 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]
Drainable Base	ATPB[4], P-307		6" (150 mm) when used	6" (150 mm) When used
Stabilized Base	P-401 or P-403; P-304; P-306	Not Required	Not Required	5 in (125 mm)
Base[3]	P-209, P-207, P-208, P-210, P-211, P-212, P-213, P-219, P-220	Not Required	6 in (150 mm)[2]	6 in (150 mm)
Subbase[2]	P-154	6 in (100 mm)	As needed for frost or to create working platform	As needed for frost or to create working platform

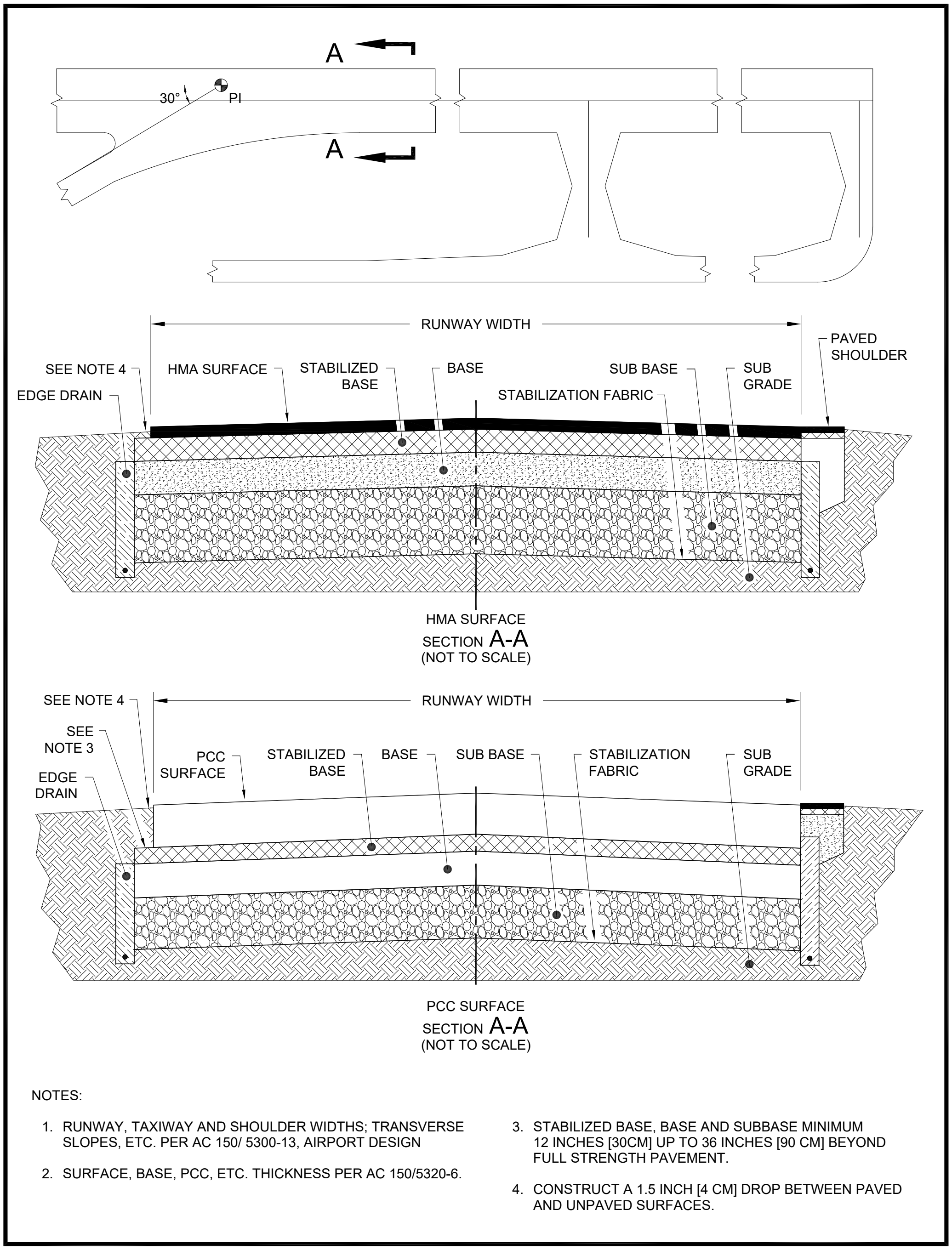
Notes:

1. Use maximum of FAARFIELD thickness to the nearest 0.5 inch (10 mm), or minimum layer thickness
2. Any base material may be used as a subbase.
3. P-207, P-219 require laboratory testing to establish if it will perform as a base or subbase. If CBR > 80 may be used in place of P209, CBR >60 in place of P-208. Both may be used as a subbase under stabilized base.
4. See EB102, Asphalt Treated Permeable Base Course.

3.12.12 Typical Pavement Sections.

3.12.12.1 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 and Figure 3-3.

Figure 3-3. Typical Plan and Sections for Pavements



3.12.12.2 Since traffic on runways is distributed with the majority of traffic on the center (keel) portion of the runway, 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 G).

3.12.13 Frost and Permafrost Design.

3.12.13.1 Consider the environmental conditions that will affect the pavement during its construction and service life when designing an airport pavement. 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.

3.12.13.2 For first few years after construction or rehabilitation of flexible pavement depth of thaw may increase.

3.12.13.3 It is important to keep cracks sealed to help prevent water from penetrating into base, subbase and subgrade.

3.12.13.4 To protect the non-frost susceptible base or subbase from contamination by subgrade material, include a geosynthetic separation fabric on top of subgrade.

3.12.14 Seasonal Frost.

3.12.14.1 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 any of three approaches: complete frost protection, limited frost protection, or reduced subgrade strength.

3.12.14.2 When constructing pavements in areas subject to seasonal frost it is important to provide uniform subgrade soils beneath the pavement. Avoid abrupt transitions between different subgrade materials.

3.12.14.3 Avoid abrupt changes in thickness of pavement structure.

3.12.14.4 The FAA considers base (P-209) material to be non-frost susceptible if less than 5% passes the No. 200 sieve, and less than 10% for subbase (P-154) material.

3.12.14.5 Note, studies with the Alaska Department of Transportation (AKDOT) have established that the percent passing the No. 200 sieve is approximately 2 times the amount of 0.02 mm material. Even though the 0.02 mm size is the critical opening size for frost action, since the No. 200 can be checked with a sieve analysis and the 0.02 mm material requires a hydrometer analysis, it is much quicker and easier to check the No. 200 material.

3.12.14.6 Support type and depth of frost protection in the engineer's report.

3.12.15 Complete Frost Protection.

3.12.15.1 Complete frost protection 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 should be sufficient to eliminate the adverse effects of frost penetration into the subgrade.

3.12.15.2 Complete frost protection is accomplished by providing a sufficient thickness of pavement and non-frost-susceptible material to contain frost penetration within the pavement structure.

3.12.15.3 The depth of frost penetration is determined by engineering analysis or by local codes and experience.

3.12.15.4 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 in the subbase or subgrade.

3.12.15.5 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.

3.12.15.6 Generally complete frost protection is only considered for runways and taxiways at large hub airports or in areas where frost penetration is minimal.

3.12.16 Limited Subgrade Frost Penetration.

3.12.16.1 The limited subgrade frost penetration method, based on engineering judgment and experience, limits frost heave to an acceptable level of maintenance, generally less than 1 inch (250 mm) of frost heave. Frost is allowed to penetrate to a limited degree into the underlying frost susceptible subgrade.

3.12.16.2 Non-frost susceptible materials are required for 65% of the depth of frost penetration, and a filter layer is required between the NFS subbase and the subgrade. (See paragraph 3.12.14.2.)

3.12.16.3 This method applies to soils in all frost groups when the functional requirements of the pavement permit a minor amount of frost heave.

3.12.16.4 After determining the thickness required for structural support, additional thickness of NFS subbase may be required to ensure that the NFS pavement structure is at least 65% of the depth of frost penetration.

3.12.16.5 Limiting frost heave and damage to pavements with limited subgrade protection, is a good solution for many airports.

3.12.17 Reduced Subgrade Strength.

3.12.17.1 The reduced subgrade strength method, is based on providing adequate pavement load carrying capacity during the critical frost melting period when the subgrade strength is reduced, ignoring the effects of frost heave. Airports should plan on annual maintenance to repair damage caused by frost heave.

3.12.17.2 To use the reduced subgrade strength method, the design assigns a subgrade strength rating close to what could be expected during the frost melting period, typically equal to 50% of the subgrade design strength.

3.12.17.3 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.

3.12.17.4 The required pavement thicknesses are determined using FAARFIELD, inputting -50% of the design subgrade strength. If the reduced subgrade strength is less than a CBR 3 it is recommended but not required to improve the subgrade. If the reduced subgrade strength is less than a CBR 5 it is recommended but not required to improve the subgrade. 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 or that determined from geotechnical investigations. Local experience on similar pavement projects may justify the use of reduced subgrade strength combined with spring load restrictions to control pavement distress.

3.12.18

3.12.18 Permafrost.

When designing pavements in permafrost regions consider the effects of seasonal thawing and refreezing, as well as the thermal effects of construction on the permafrost. New pavement construction 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 protection of permafrost should design the pavement structure. In the first few years after construction it is not unusual for the depth of thaw to increase due to the different thermal properties of the new pavement structure.

3.13 Flexible Pavement Design.

3.13.1 General

Flexible pavements consist of an asphalt mixture wearing surface placed on a base course and a subbase (if required) to protect the subgrade. In a flexible pavement structure, each pavement layer protects its supporting layer. A typical pavement structure is shown in Figure 1-1 and Figure 3-3. "Sandwich" construction, in which one or more pervious granular layers is located between two impervious layers, is not permitted. This is to prevent trapping water in the granular layer, which could result in a loss of pavement strength and performance.

3.13.2 Asphalt Mixture Surfacing.

3.13.2.1 The asphalt material 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. A dense-graded asphalt mixture, such as Item P-401, meets these requirements.

3.13.2.2 Use Item P-401 as the surface course for pavements serving aircraft weighing more than 30,000 pounds (13,600 kg). Item P-403 may be used as a surface course for pavements serving aircraft weighing 30,000 pounds (13,600 kg) or less. See AC 150/5370-10, Items P-401 and P-403, for additional discussion on asphalt pavement material specifications. See Table 3-3 for minimum requirements for asphalt mixture surface thickness.

3.13.2.3 In FAARFIELD, the asphalt 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 Asphalt Overlay type can be placed over asphalt or concrete surface types or user-defined layers. Refer to Table 3-2 for material properties used in FAARFIELD.

3.13.2.4 A solvent-resistant surface (such as P-404 or P-629) 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, physical properties, gradation, and compaction. A properly constructed base course will withstand the stresses produced and resist vertical pressures that may produce consolidation and distortion of the surface course, and resist volume changes caused by fluctuations in moisture content protecting the support layer from failing.

3.13.3.2 Base courses are classified as either stabilized or unstabilized. When aircraft in the design traffic mix have gross loads of 100,000 pounds (45,360 kg) or more a stabilized base is required (see paragraph 3.6). AC 150/5370-10, *Standard Specifications for 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-210, P-211, P-212, P-213, P-219). The use of Item P-208, P-210, P-212, P-213 *Aggregate Base Course*, as base course is limited to pavements designed for gross loads of 60,000 pounds (27,200 kg) or less. When supported with laboratory testing P-207 may be used as a base course.

3.13.3.3 P-207, P-219 require laboratory testing to establish performance as a base or subbase. If CBR > 80 may be used in place of P209, if CBR >60 in place of P-208. Both may be used as a subbase under stabilized base.

3.13.3.4 **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). Use variable stabilized flexible base to characterize a stabilized base which does not conform to the properties of P-401/P-403. Variable stabilized flexible has a modulus 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. Use appropriate measures to control the potential for reflective cracking when using rigid stabilized bases. Note: In AC 150/5370-10, Item P-304 and

Item P-306 both contain limits on strength of concrete, as well as provisions for control joints and / or use of bond breakers. The properties of the various stabilized base layer types used in FAARFIELD are summarized in Table 3-2. It is a best practice to offset stabilized bases 12 inches (300 mm) from the edge of the full strength pavement (see Figure 3-3).

3.13.3.5 Aggregate Base Course.

3.13.3.5.1 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.

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

3.13.3.5.3 Aggregate layers can be placed anywhere in the flexible pavement structure except at the surface or subgrade. Only two aggregate layers may be present in a structure, one crushed and one uncrushed., with the crushed layer above the uncrushed layer.

3.13.3.5.4 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.)

3.13.3.5.5 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).

3.13.3.6 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.

3.13.3.7 **Base Course Width.**

The base course may be offset 12 inches (300 mm) from the edge of the asphalt surface course. It is a good construction practice to construct the base course up to 12 inches wider than the asphalt surface course.

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.

3.13.4.2 The minimum thickness of subbase is 6 inches (150 mm), This minimum is recommended as a practical construction layer thickness for non-stabilized aggregate subbase. Additional thickness may be required to structurally protect subgrade or to provide frost protection to subgrade. If pavement structural design indicates a subbase thickness less than 6 inches, eliminate subbase and run FAARFIELD to calculate amount of structural base needed.

3.13.4.3 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, *Standard Specifications for Construction of Airports,* covers the quality of material, methods of construction, and acceptance of material.

3.13.4.4 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. See paragraph 3.9, Subgrade Compaction.

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:

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

3.13.5.2 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 2.4.6.

3.13.5.3 In cases where the top layer of subgrade is stabilized using a chemical stabilizing agent (cement, fly ash, etc.) per paragraph 2.5.6, 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:

Step 1 Enter a user-defined layer immediately above the subgrade.

Step 2 Set the design layer to the layer immediately above this user-defined layer. In FAARFIELD, this is done by highlighting the new design layer in the structure grid on the left side of the screen, and clicking the button "Select as the Design Layer." The new design layer will be indicated by the red arrow in the grid, and highlighted by a green border in the pavement section diagram to the right.

Step 3 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 FAA recommends conservative long term benefits of chemical stabilization, 50% for cement and 20% for fly ash).

Step 4 Enter the thickness of the user-defined material. The thickness should be equal to the depth of field stabilization.

Step 5 Enter the subgrade CBR. The CBR for the subgrade (lowest layer) should be equal to subgrade design strength of the natural (unstabilized) subgrade (see Chapter 2).

Step 6 After entering the appropriate traffic mix, select "Thickness Design" from the drop-down list and click "Run" to execute the design.

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 "Yes" for "Calculate HMA CDF" under FAARFIELD design options. 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.

3.14 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 a 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 concrete slab. The rigid pavement design model does not explicitly 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. However, FAARFIELD will not reduce the PCC thickness below the minimum allowable thickness of 6 inches (150 mm) (or 5 inches (125 mm), if all aircraft are less than 30,000 pounds (11,520 kg) gross weight). If minimum thickness is reached, the design process will abort with CDF < 1.0 and the design report will indicate: "Minimum layer thickness control, cdf analysis was not completed."

3.14.1.3 FAARFIELD uses a three-dimensional finite element model (FAASR3D) to compute the edge stresses in concrete slabs. The finite element-computed free edge stress is reduced by 25% to account for load transfer across joints. Critical stresses in rigid pavements normally occur at slab edges, but for certain aircraft gear configurations the critical stress may be located at the center of the slab. FAARFIELD uses a layered elastic analysis program (LEAF) to compute interior stress. The LEAF-computed stress is reduced by 5% to account for the effect of finite slab size. The design stress is the larger of: (a) 95% of the interior stress; or (b) 75% of the 3D-FEM computed free edge stress.

Note: FAARFIELD does not consider non-structural aspects of pavement thickness design, 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 provides a nonskid texture, minimizes the infiltration of surface water into the subgrade and provides structural support for airplane loading. The quality of the concrete, acceptance and control tests, methods of construction and handling, and quality of workmanship are covered in Item P-501Cement Concrete Pavement. See AC 150/5370-10, Item P-501 for additional discussion regarding concrete pavement specifications. See for minimum concrete 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 for minimum base thicknesses required under rigid pavements.

3.14.3.2 Stabilized base is required for base under pavements designed to serve airplanes over 100,000 pounds (see paragraph 3.6).

3.14.3.3 Two layers of base material may be used, e.g., a layer of P-306 over a layer of P-209. Avoid producing a sandwich section (granular layer between two stabilized layers) or placing a weaker layer over a stronger layer.

3.14.3.4 Subbase material may be substituted for aggregate base material in rigid pavements designed to serve only airplanes weighing 30,000 pounds (13,610 kg) or less.

3.14.3.5 Additional subbase may be needed for frost protection; or as a substitution for unsuitable subgrade material.

3.14.3.6 The following materials are acceptable for use under rigid pavements: stabilized base (P-401, P-403, P-307, P-306, P-304, P-220) and unstabilized base/subbase (P-209, P-208, P-219, P-211, P-154).

3.14.3.7 Best construction practice is to offset the first layer directly under the surface 12 to 36 inches from the edge of the concrete layer to create a solid path for the paver.

3.14.3.8 Up to three base/subbase layers can be added to the pavement structure in FAARFIELD for new rigid pavement design. 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. Refer to Table 3-4 for minimum layer thicknesses.

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

3.14.4.1 A value for 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. Use the subgrade strength as identified in the project geotechnical report for the pavement design. (See paragraph 2.4, Soil Strength Tests.) The subgrade modulus can be expressed either 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, FAARFIELD performs all structural computations using the elastic modulus *E*. If the foundation modulus is input as a *k*-value FAARFIELD will convert it automatically 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:

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

3.14.4.2 For existing pavements, the *E* modulus can be determined in the field from nondestructive testing (NDT). Generally, a heavy weight deflectometer (HWD) or dynamic cone penetrometer (DCP) 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.

For rigid pavements iin areas where conditions conducive to detrimental frost action exist, provide frost protection. Concrete slabs less than 9 in (230 mm) thick are more susceptible than slabs greater than 9 in (230 mm) to cracking from frost heave. Often, frost heave is most pronounced at the boundary between marked and unmarked areas on a runway, e.g. adjacent to the fixed distance marking and near edges of pavement. If complete frost protection is not provided, it is a best practice to reinforce concrete slabs. For slabs less than 9 in (230 mm), reinforce slabs with embedded steel providing no less than 0.050 percent steel in both directions. If not practical to reinforce all slabs, as a minimum reinforce slabs that include large areas of markings, (e.g., threshold bars, runway designation and fixed distance markings), for those slabs immediately adjacent to the markings and along edges of pavement where no paved shoulders. Refer to paragraph 2.6 for guidance on the determination of the depth of frost protection required.

3.14.6 FAARFIELD Calculation of Concrete Slab Thickness.

3.14.6.1 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. For maximum edge stress determination, the airplane gear may be positioned either parallel or perpendicular to the slab edge.

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

3.14.6.3 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 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. 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.

3.14.8.1 Variations in temperature and moisture content can cause volume changes and slab warping which may cause significant stresses. In general, smaller panels have better long-term performance.

3.14.8.2 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.

3.14.8.3 Slabs should be as nearly square as possible when no embedded steel is used.

3.14.8.4 Refer to Table 3-7 for recommended maximum joint spacing. Note that the slab thickness controls the joint spacing, not vice-versa. Table 3-7 is not intended to be used to establish slab thickness based on a predetermined joint spacing.

3.14.9 Joint Type Categories and Details

3.14.9.1 Pavement joints are categorized according to the function that the joint is intended to perform. Joint types are as described in Table 3-5 and below. Pavement joint details are shown in Figure 3-4, Figure 3-5, and Figure 3-6. The categories of joints are:

1. isolation,
2. contraction, and
3. construction joints.

All joints should be finished in a manner that permits the joint to be sealed.

3.14.9.2 Longitudinal joints should be designed to minimize pavement width changes.

1. All longitudinal construction joints should be doweled joints, unless the joint also serves as an isolation joint.
2. 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. The rationale is that the 'tension

ring' helps keep the joints closed thus helping assure that load is transferred through aggregate interlock.

3. 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.

3.14.9.3 Isolation Joints (Types A, A-1).

Isolation joints are needed:

1. Where the pavement abuts a structure; or
2. To isolate intersecting pavements where differences in direction of movement of the pavements may occur (e.g., between a connecting taxiway and a runway).
3. At locations to accommodate future expansion, for example where extensions or connections are planned. See paragraph 3.14.14.

3.14.9.3.1 Type A joints are created by increasing the thickness of the pavement along the edge of the slab (see Figure 3-4). This thickened edge will accommodate the load that otherwise would be transferred with dowels or by aggregate interlock in contraction and construction joints.

3.14.9.3.2 Type A-1 joints are reinforced to provide equivalent load carrying capacity as a thickened edge and may only be used for concrete 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.

3.14.9.4 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 and curling. Details for contraction joints are shown as Types B, C, and D in Figure 3-5. Details for joint sealant are shown in Figure 3-6.

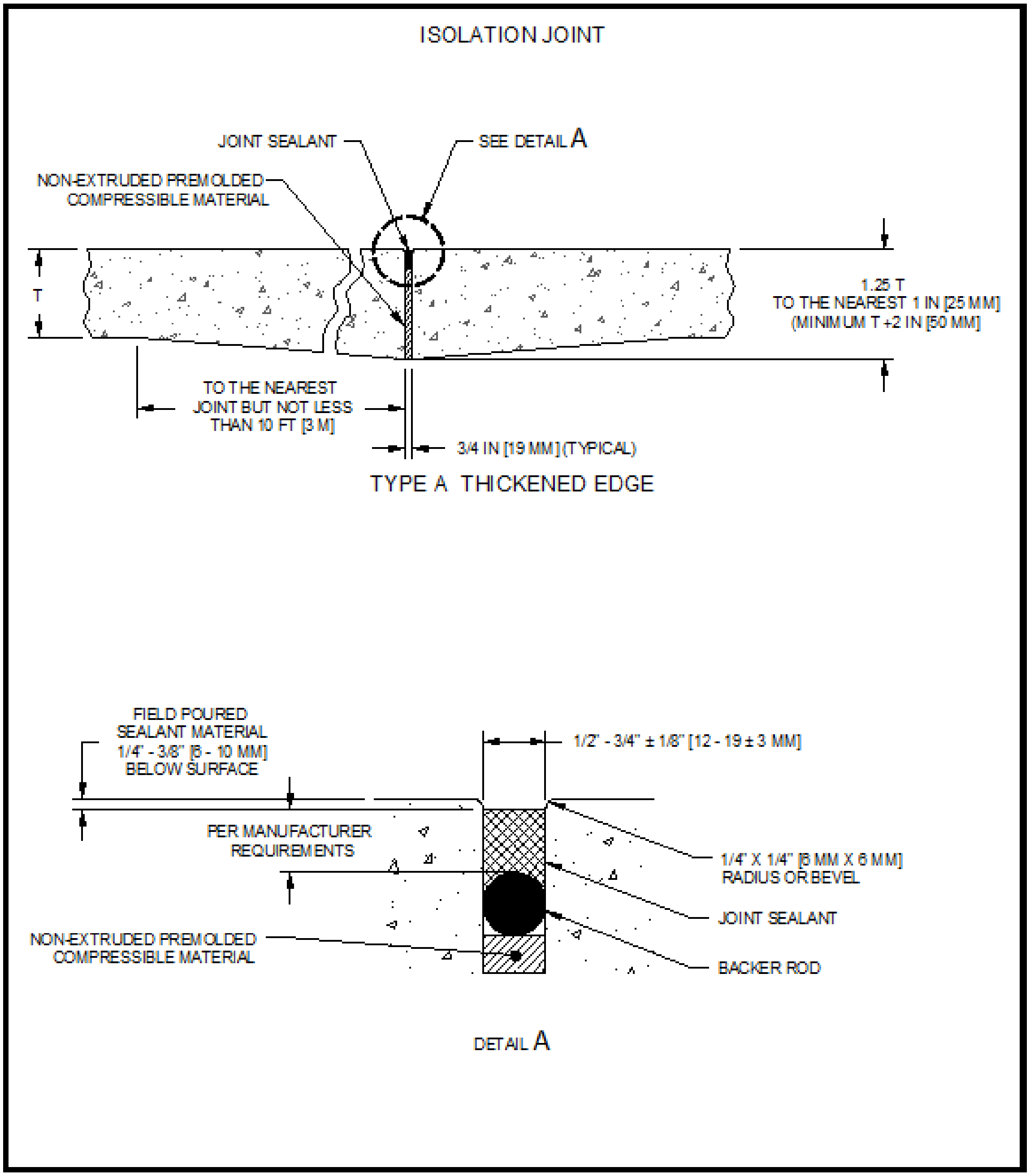
3.14.9.5 Construction Joints (Types E and F).

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 3-5.

Table 3-5. Pavement Joint Types

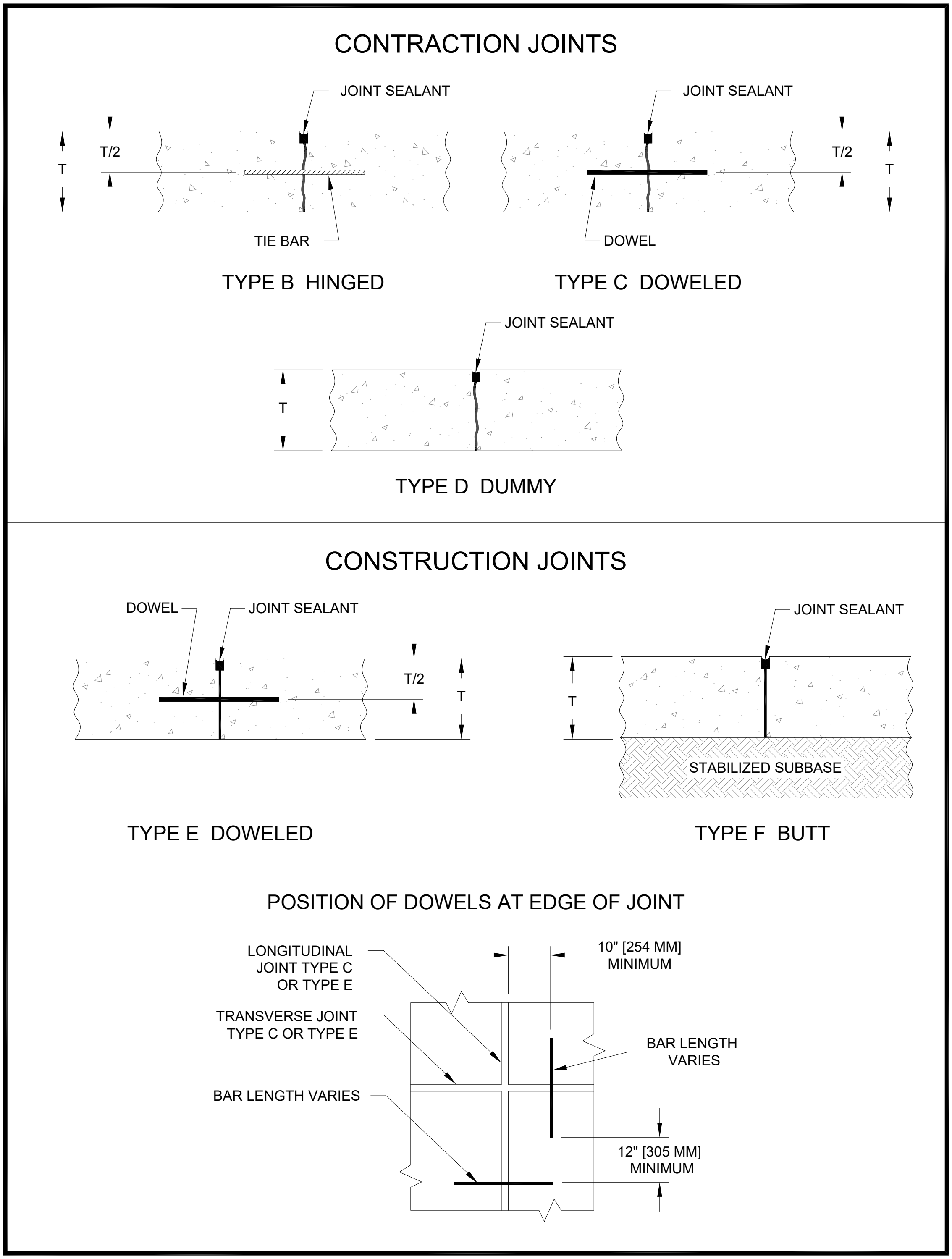
Type	Description	Longitudinal	Transverse
A	Thickened Edge Isolation Joint	Use at: -Pavement Intersections -Free edge that is location of future expansion -edge of structures	Use at: -pavement feature intersections when the pavement intersects at an angle. -free edge that is location of future expansion, -where pavement abuts a structure.
A-1	Reinforced Isolation Joint	For concrete slabs > 9 in (230 mm). Use at: -Pavement Intersections -Free edge that is location of future expansion - edge of structures	For concrete slabs > 9 in (230 mm). Use at: -Pavement Intersections -Free edge that is location of future expansion - edge of structures
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 at other locations with FAA approval, eg. at gate stands.	Use on the last three joints from a free edge, and for two or three joints on either side of isolation joints. Use at other locations with FAA approval, eg. at gate stands.
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 3-4. Rigid Pavement Isolation Joint



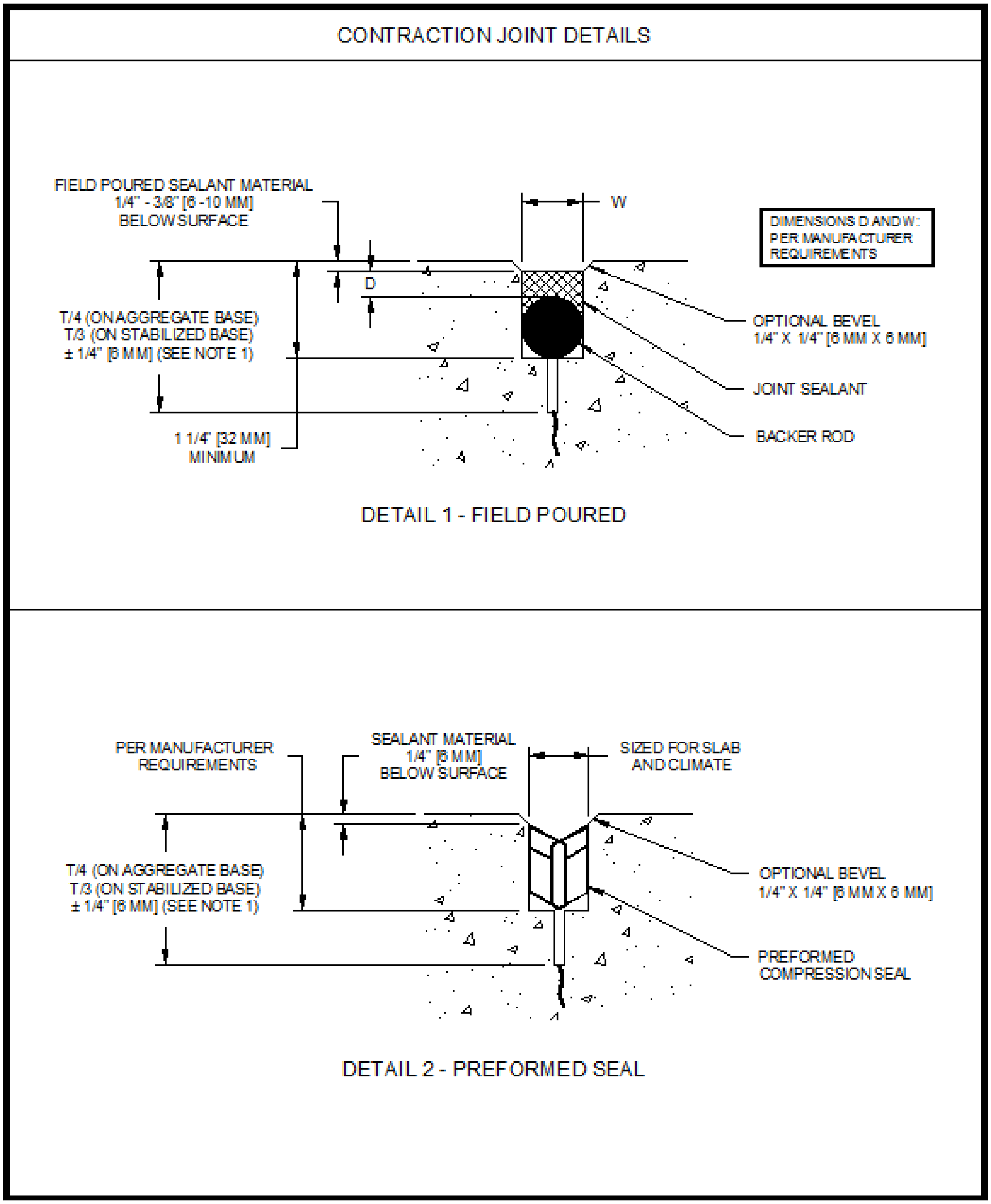
Note: When isolation joint is adjacent to a fillet, thicken fillet panels for minimum of 10 ft perpendicular to joint. At acute angle intersections transition from full thickened edge back to normal thickness over width of placement lane, perpendicular to isolation joint.

2038 **Figure 3-5. Rigid Pavement Contraction and Construction Joints**



2039

Figure 3-6. Rigid Pavement Joint Sealant Details



Notes:

1. Initial saw cut T/6 to T/5 (on stabilized base), when using early entry saw.
2. Size sealant reservoir to proper shape factor (depth (D): width (W)), based 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.
4. Beveled joints may help minimize sliver spalls due to snowplow damage.
5. Start first saw crew on transverse joints and second crew (if needed) on longitudinal joints following behind crew sawing transverse joints.

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), tie longitudinal contraction joints within 20 feet (6 m) of a free edge 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), use 20 inch long (510 mm) No.4 bars spaced at 36 inches (900 mm) on center for tie bars. For slabs 6 inches or greater (150 mm), use 30 inch long (762 mm), No. 5 bars spaced at 30 inches on center as tie bars. 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. Provide dowels in the last three transverse joints from a free edge. Justify use of additional dowels in engineers report. Research indicates that when stabilized base is included in the pavement section, the stabilized base will provide slab support assisting with load transfer. There is little benefit to providing more than minimum of dowels in last three joints from a free edge when the pavement section includes a stabilized base.

3.14.10.2.1 Size Length and Spacing of Dowels.

Size dowels to resist the shearing and bending stresses produced by the loads on the pavement. Dowel length and spacing sufficient to prevent failure of the concrete slab due to the bearing stresses exerted on the concrete. Table 3-6 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. To hold transverse dowels in position utilize a wire cage or basket firmly anchored to the base.or apaving machine equipped with an automated dowel bar inserter.

Table 3-6. 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.

3.14.11.1 Premolded compressible filler is used in isolation joints to accommodate movement of the slabs, and sealant is applied above the filler to prevent infiltration of water and foreign material.

3.14.11.2 The depth (D) and width (W) of the joint sealant reservoir is a function of the type of sealant material used. Construct the joint reservoir and install the joint sealant material in accordance with the joint sealant manufacturer's recommendations for the type of sealant used. 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 3-6 for typical joint reservoir details. Use backer rod material that is compatible with the type of sealant used and sized to provide the desired shape factor.

3.14.11.3 Standard specifications for joint sealants can be found 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.

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 3-7 shows a typical jointing plan for a runway end, parallel taxiway, and connector. Figure 3-8 shows a typical jointing plan for pavement for a 75-foot (23-m) wide runway. For sample concrete pavement Joint plans, see https://www.faa.gov/airports/engineering/pavement_design/.

3.14.12.1 **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.

3.14.12.2 **Odd-Shaped Slabs, Slabs with Structures, or Other Embedments.**

Cracks tend to form in slabs with odd or irregular shapes and in slabs that include structures and other embedment's. To minimize potential for cracking slabs that are nearly square or rectangular in shape have better long term performance.

3.14.12.2.1 Provide a minimum of 0.050 percent of the slab cross-sectional area in reinforcement in both directions, when the length-to-width ratio of slabs exceeds 1.25, or when slabs are irregular in shape (e.g. trapezoidal).

3.14.12.2.2 In addition, place embedded steel around the perimeter of embedded structures.

3.14.12.2.3 Steel does not prevent cracking. However, 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, this minimizes the infiltration of debris into the cracks.

3.14.12.2.4 Steel either may be 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). For slabs less than 9" place the steel approximately in the middle of the slab for slabs greater than 9" place the steel in the upper 1/3 of the slab.

3.14.12.2.5 Thin Slabs (<9") in areas subject to Freeze-Thaw.

Provide minimum temperature steel at mid-depth of the slab in areas subject to freeze-thaw. The embedded steel should consist of no less than

0.050 percent of the gross cross-sectional area of the slab in both directions.

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

3.14.13 Joint Spacing.

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/curling stresses. Shorter joint spacing generally provides better long-term in-service performance. Shorter joint spacing provides better performance in areas of freeze thaw. See Table 3-7 for recommended maximum joint spacing.

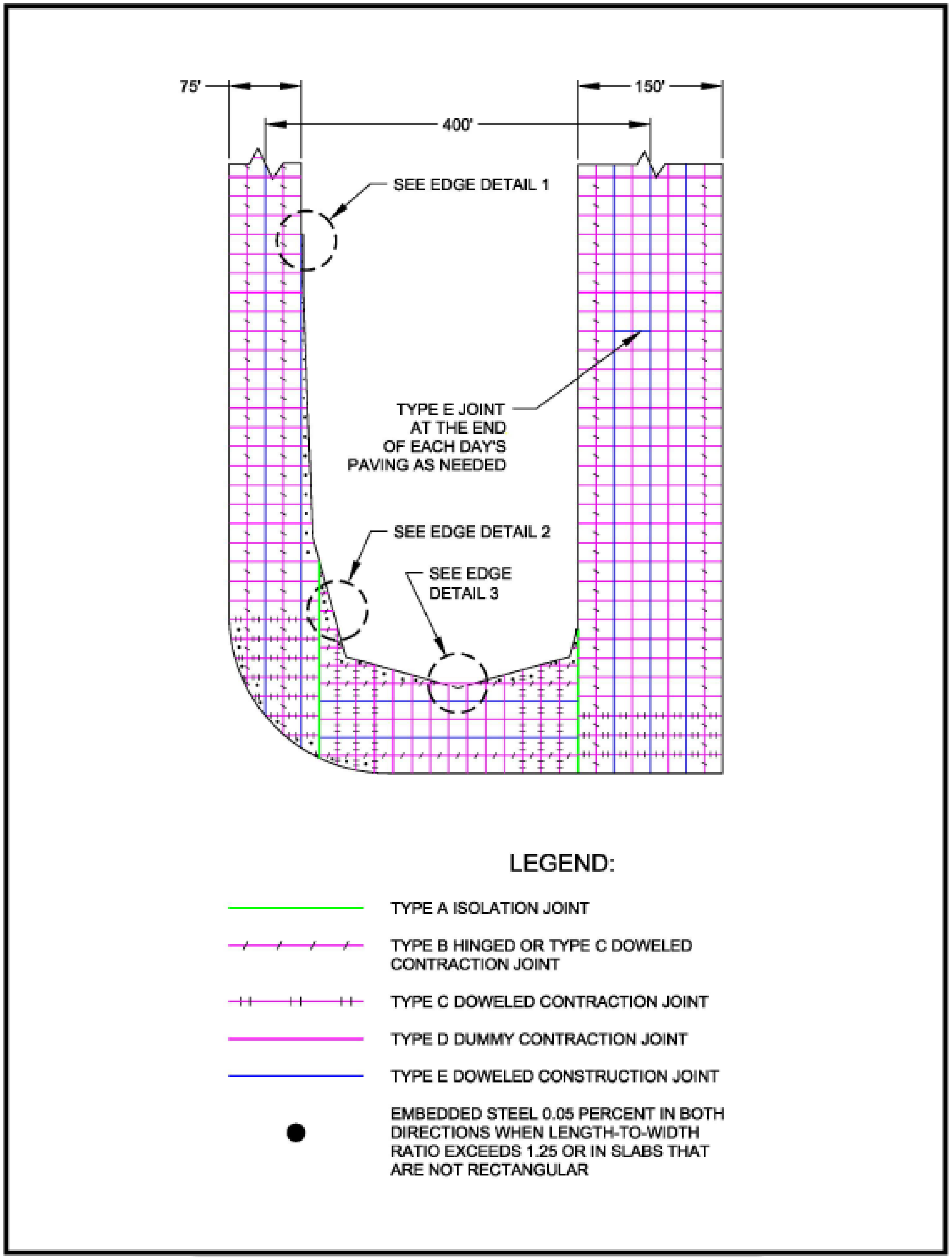
3.14.13.1 **Without Stabilized Base.**

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 3-7 requires technical analysis documented in engineers report that slab size in inches does not exceed 5 × radius of relative stiffness, in inches.

3.14.13.2 **With Stabilized Base.**

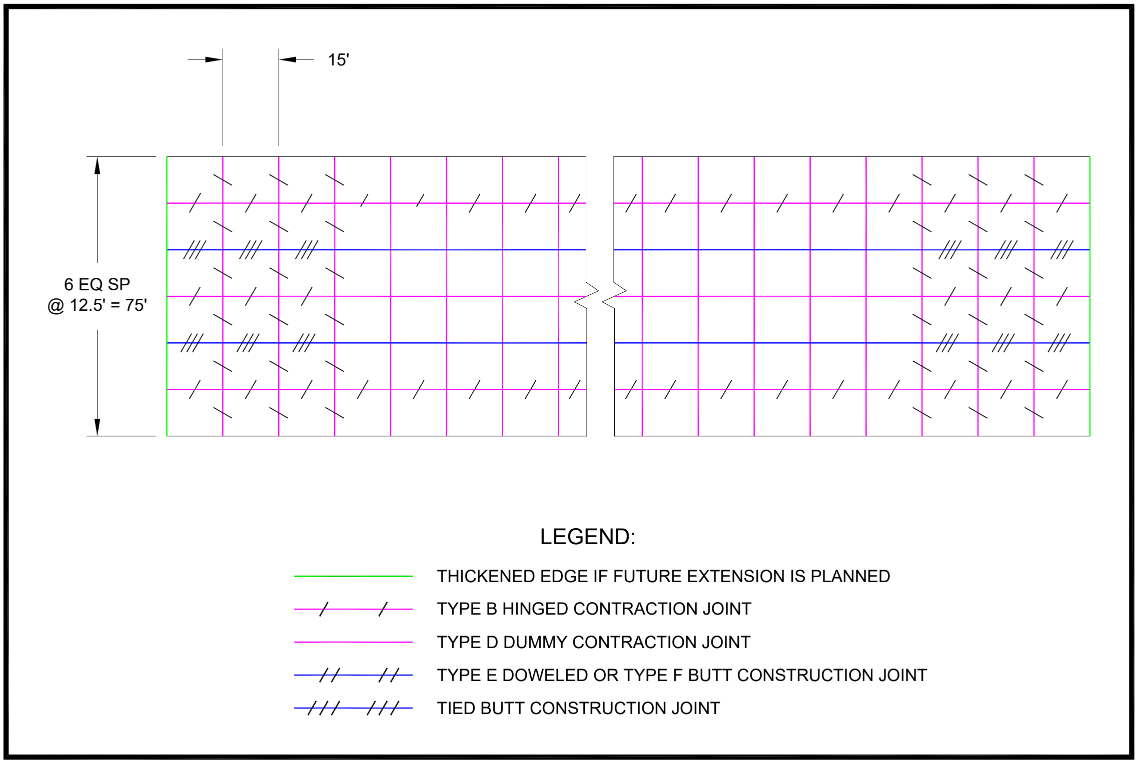
Rigid pavements supported on stabilized base are subject to higher warping and curling stresses than those supported on unstabilized base. A maximum spacing of 20 feet (6.1 m) is recommended for slabs equal to or thicker than 16 inches (406 mm). On federally funded projects exceeding the spacing as shown in Table 3-7 requires technical analysis in engineers report that slab size in inches does not exceed 5 × radius of relative stiffness, in inches.

2182 **Figure 3-7. Typical Joint Layout Pattern for Runway, Parallel Taxiway and**
2183 **Connector**



2184

Figure 3-8. (Optional) Joint Layout Concrete 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 3-7. Recommended Maximum Joint Spacing - Rigid Pavement[1,2,3]

a. Without Stabilized base

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, 3]

b. With Stabilized base

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)[2,3]

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. Spacing greater than 20 feet must be supported with technical analysis in engineers report that slab size in inches does not exceed 5 × radius of relative stiffness, in inches.

$$l = [\, E_{pcc} h_{pcc}^3 / (12 \times (1-\mu^2) \times k)]^{1/4}$$

where:

l = radius of relative stiffness, inches,

E_{pcc}=modulus of elasticity of concrete, psi,

h_{pcc} = slab thickness, inches,

μ=Poisson's ratio for concrete, usually 0.15,

k = modulus of subgrade reaction, lb/in^3

3.14.14 Jointing Considerations for Future Pavement Expansion.

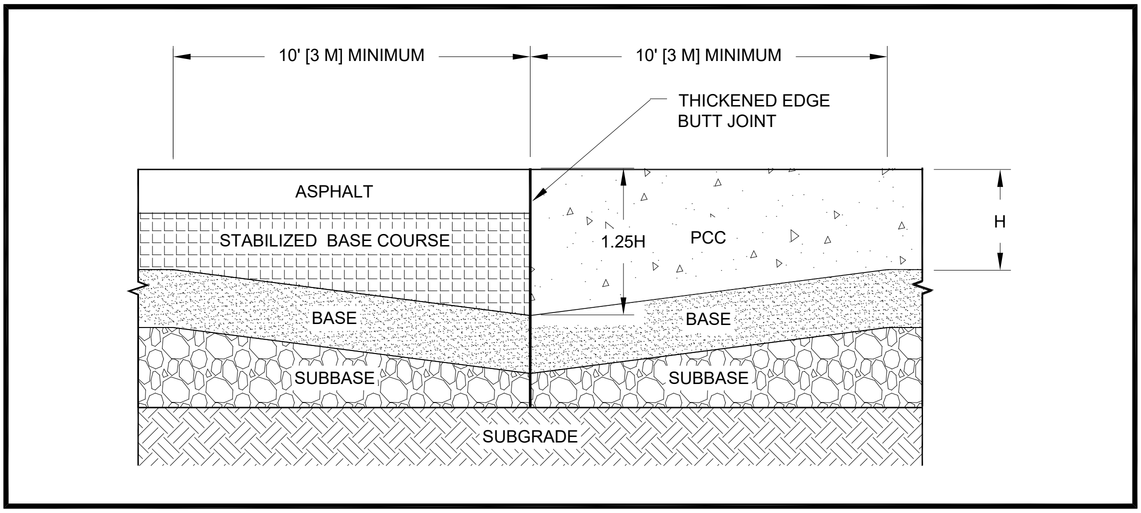
When a runway or taxiway is likely to be extended, an isolation joint should be provided at the location where the extension will begin. (for Type A - thickened edge joint, see Figure 3-4). In addition, at locations where there may be a need to accommodate a future connecting taxiway or apron entrance, a thickened or reinforced 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 Concrete and Asphalt.

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 3-9. See an example in paragraph H.3.

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. Note this only applies to where taxiway or runways transition from rigid to flexible and does not apply to transition on taxiway and runway shoulders.

Figure 3-9. Transition between Rigid and Flexible Pavement Sections



Dimension	Description
H	Design thickness of PCC pavement
B	Thickness of base
T	Design thickness of flexible 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

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

Pre-stressed, precast, structurally reinforced, and continuously reinforced concrete pavements (CRCP) have been used to a limited extent in airport applications. The main advantages of pre-stressed pavements and CRCP are that both allow for thinner slabs and greater distances between joints than jointed plain concrete pavement (JPCP). (In pre-stressed concrete pavement, prestressing tendons keep the entire section in compression, while reinforced concrete and CRCP depend on reinforcing steel to resist tensile stresses and control cracks.) Precast slabs, which can be fabricated offsite, may be considered when there is a short working window for individual slab replacements, or when normal concrete cure times would conflict with runway opening requirements. In addition to high construction costs compared to JPCP, there are a number of technical challenges that historically have limited the use of these materials on airports:

1. It is difficult to establish load transfer between precast panels and regular PCC.
2. Repair of PCC and retrofitting of in-pavement fixtures can be challenging with prestressed, precast and CRCP.
3. Structural design of prestressed, precast, reinforced, and CRCP pavements requires specialized procedures. FAARFIELD rigid pavement thickness design does not apply to these materials. The critical loads on precast slabs may occur during lifting and placement operations, not in service.

Use of pre-stressed, precast, structurally reinforced concrete, and continuously reinforced concrete airport pavements on federally funded projects requires approval from FAA AAS-100. Support request with: (1) Why is this a better solution than plain concrete, including an analysis of schedule and cost of all alternatives considered; (2) Technical analysis of slab design; (3) Construction details and specifications.

3.16 Aggregate Turf Pavements.

Aggregate-turf pavements may be appropriate 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. 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 support aircraft traffic, will not soften appreciably during wet weather and has sufficient soil to promote the growth of grass. See an example in paragraph H.1.

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.

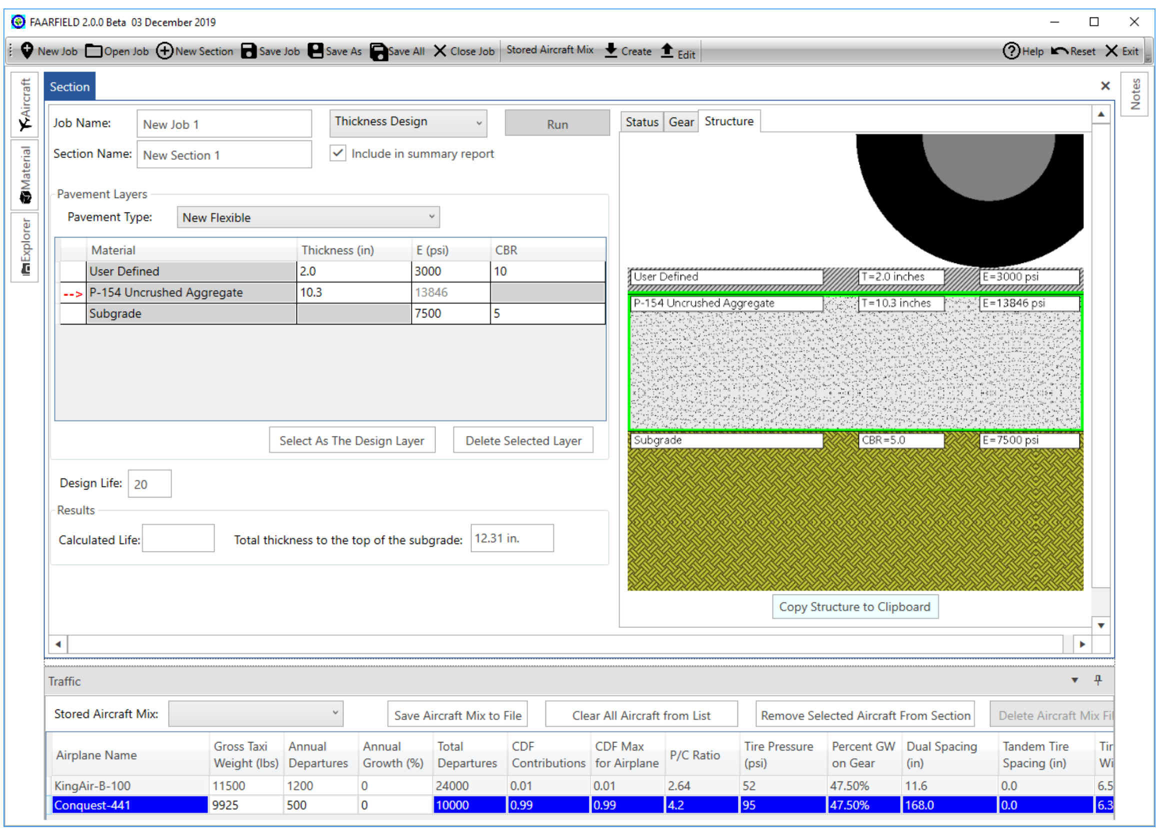
Assume that the airplane mix consists of the following:

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 3-10).

Figure 3-10. 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.

3.18.1 Design of the passenger loading bridge operating area is separate from the design of the adjacent aircraft apron. Due to the large range of potential loads, verify the actual loads and contact tire pressure with the manufacturer of the passenger loading bridge.

3.18.2 Loads of passenger loading bridges range from 40,000 – 100,000 pounds supported on two semi-solid tires with tire contact pressures ranging from up to 600-700 psi per tire.

3.18.3 Use the FAA recommends rigid pavement where the passenger loading bridge will operate. The FAA recommends verifying the wheel loads of the loading bridge.

3.18.4 Do not locate drainage structures or fuel hydrants in the jet bridge operation area.

3.18.5 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.

3.19 Ground Handling Equipment.

3.19.1 Design of pavement that is only utilized by ground servicing equipment should consider the loads used to move aircraft at the gate stand.

3.19.2 The loads for the tugs used to handle large aircraft can be significant, up to 65,000 pounds, generally distributed between 4 wheels. Tugs that can accommodate Boeing 737 and Airbus A320 type aircraft are generally weigh between 35,000-40,000-pounds.

Page Intentionally Blank

CHAPTER 4. PAVEMENT MAINTENANCE and REHABILITATION

4.1 General.

4.1.1 Pavement maintenance and rehabilitation are most effective when implemented as part of an overall Pavement Management Program (PMP). See AC 150/5380-7 *Airport Pavement Management Program (PMP)* for more information on development and implementation of a PMP.

4.1.2 Lower project costs and greater long-term benefits are achieved the earlier that maintenance or rehabilitation techniques are implemented. The condition of the pavement at the time of project greatly affects how much the functional life of the pavement will be extended.

4.1.3 Include justification for need of maintenance, rehabilitation or reconstruction in the engineer's report.

4.2 Pavement Maintenance.

4.2.1 All pavements benefit from timely maintenance. Pavements with a pavement condition index (PCI) greater than 70 are candidates for some form of maintenance. It is always more cost effective to extend the life of a pavement in good condition than to rehabilitate or reconstruct a pavement in fair or poor condition.

4.2.2 Timely crack sealing and application of surface treatments on flexible pavements is a cost-effective method to extend a pavement's functional life. Surface treatments are more effective the sooner the treatment is applied. Surface treatments may be applied any time after initial construction but often the first surface treatment is applied 5 years after initial construction.

4.2.3 Timely resealing of joints on rigid pavement to keep water and incompressible material out of joints will extend the functional life of rigid pavements.

4.2.4 Include justification for method and timing of maintenance in engineers report.

4.3 Rehabilitation.

4.3.1 Rehabilitation is defined as the replacement of a portion of the pavement structural layers. It is generally more cost effective to rehabilitate a pavement than to reconstruct it.

4.3.2 Pavements with a PCI less than 70 and greater than 55 are candidates for rehabilitation. There are times when a rehabilitation strategy is justified on pavements with PCI greater than 70.

4.3.3 Pavements 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 repair of localized structural damage.

4.3.4 Rehabilitation of flexible pavement consists of removal and replacement of a portion or all of the wearing surface. A mill and overlay of a flexible pavement will often provide a significant additional functional and structural life. When a flexible pavement needs replacement of the wearing surface is dependent upon many factors. Factors include: initial quality of materials and construction, environmental conditions, was routine maintenance performed and composition and nature of traffic as compared to design traffic. A flexible pavement constructed with quality materials and quality construction that is maintained with timely crack sealing and surface treatments can last beyond the 20-year structural life.

4.3.5 Rehabilitation of rigid pavement consists of repairing or replacing isolated slabs, less than 30 percent.

4.3.6 Include justification for method and timing of rehabilitation in engineers report.

4.4 **Reconstruction.**

4.4.1 Reconstruction is the replacement of the main structural elements of the pavement.

4.4.1.1 The slab is the main structural element of a rigid pavement. Replacement of more than 30% of the slabs is reconstruction.

4.4.1.2 For flexible pavements all improved materials above the subgrade, sub-base, base, stabilized base and surface course, constitute the pavement structure.

4.4.2 Pavements that have a pavement condition index less than 55 may be candidates for reconstruction. There are times when it is necessary to reconstruct a pavement with a PCI greater than 55.

4.4.3 Partial reconstruction of just the areas that are severely distressed, e.g. in the center (keel) sections, may be a cost-effective alternative to total reconstruction.

Existing base and subbase materials in good condition can be reused in place.

4.5 **Design Considerations for Rehabilitation and Reconstruction**

4.5.1 Assessment of Existing Conditions.

4.5.1.1 PCI is just a visual rating of the surface condition of a pavement; additional investigations are required to identify the underlying reason for the distress.

4.5.1.2 Assess the existing pavement structure including an evaluation of the thickness, condition and strength of each layer.

4.5.1.3 Study distressed areas in the existing pavement to determine the cause of the distresses and to identify potential mitigation strategies.

4.5.1.4 Include an evaluation of surface and subsurface drainage conditions and note any areas of pavement distress attributed to poor drainage. Overlaying an existing pavement without correcting poor subsurface drainage usually results in poor overlay performance. Correcting subsurface drainage deficiencies may require reconstructing the entire pavement structure.

4.5.1.5 Non-destructive testing (NDT) is a valuable technique for assessing the structural condition of the existing pavement, (see Appendix C). NDT can be used to estimate foundation strength, measure load transfer across existing concrete joints, and possibly detect voids beneath existing pavements. NDT also can be used to determine structural capacity, assist with calculating pavement classification rating (PCR), and identify areas of localized weakness.

4.5.2 Structural Considerations.

4.5.2.1 If significant changes in composition or frequency of aircraft traffic a structural overlay, minimum 3 inches (75mm), may be required.

4.5.2.2 Structurally, reconstruction is no different than designing a new pavement structure. Refer to Chapter 3 when reconstruction of pavements is required. When reconstructing a pavement due to structural failures, correct all deficiencies that contributed to the structural failure, e.g. improve subgrade or correct drainage.

4.5.2.3 When correcting structural distress it is necessary to establish the quality, thickness, and in-situ strength and/or modulus of existing materials with laboratory and/or field tests. Perform sufficient number tests to ensure statistical accuracy of results. The overlay design procedures in this advisory circular assume that the base pavement structural materials to be overlaid have significant remaining structural integrity.

4.5.3 Materials.

4.5.3.1 When selecting the type of overlay material, take into account existing pavement type, available materials, available contractors and cost of materials and construction.

4.5.3.2 Both rehabilitation and reconstruction can make use of existing materials by reusing existing layers in place, or by using reusing/recycling materials for base and subbase layers.

4.5.3.3 AC 150/5370-10 includes specification items for In-place Full Depth Reclamation (FDR) Recycled Asphalt Aggregate Base Course (P-207) and Recycled Concrete Aggregate Base Course (P-219).

4.5.3.4 The strength of a recycled material depends on many factors, including the type and condition of the recycled material and the method of recycling.

4.5.3.5 Material recycled in place will perform differently than material that is removed, reprocessed and replaced.

4.5.3.6 Both recycled asphalt pavement and recycled concrete pavement may be processed to be acceptable for use as a subbase material meeting Item P-154.

4.5.3.7 On federally funded projects, the use of recycled materials other than those meeting Items P-154, P-207 or P-219 requires a Modification of Standards (MOS) in accordance with FAA Order 5300.1, *Modification of Agency Airport Design, Construction and Equipment Standards*.

4.6 Construction Considerations

4.6.1 Assessment of Construction Methods and Equipment. Perform on-site investigations to ensure that selected method of rehabilitation can be accomplished with available materials and equipment. Perform investigations before or during the design phase. Include imitations in the plans and specifications on the size, weight or type of construction equipment necessary to minimize damage to portions of the pavement structure that will be retained and reused.

4.6.2 Before constructing overlay, remove weathered, raveled, or otherwise distressed asphalt material by milling or other means. When removing areas of distressed asphalt mixture by milling, either remove the entire layer or leave at least 2 inches of asphalt mixture in place. Sufficient material must remain to support the milling equipment, and all other construction equipment required to construct the overlay.

4.6.3 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. Provide for load transfer from the new pavement to the existing. This may require the construction of thickened edges or the use of stabilized base.

4.7 Overlay Structural Design.

4.7.1 General.

An overlay consists of a new asphalt or concrete surface course placed on top of an existing pavement. FAARFIELD overlay design is based on layered elastic and three-dimensional finite element methods of analysis.

4.7.2 Design Life.

FAARFIELD designs the overlay thickness required to provide a 20-year (or other chosen) structural design life by meeting the limiting stress or strain criterion, subject to minimum thickness requirements. (Table 3-3). Design overlays for a 20-year structural life from the time of overlay. A design life less than 20 years may be considered if (a) the original pavement is more than 15 years old at the time of the overlay, and (b) the primary purpose of the overlay is functional rehabilitation of the pavement surface (i.e., where the underlying pavement retains considerable structural integrity). In no case should an overlay be designed for less than 10 years of life. Include justification in engineers report supporting the use of a design life other than 20 years.

4.7.3 Design Traffic.

Use the most recent traffic projections to design overlays Even for relatively new pavements actual aircraft traffic may differ from traffic used in the original design. Note that for non-structural flexible overlays where the original design traffic has not changed significantly, there is no need for FAARFIELD thickness calculations.

4.7.4 Types of Structural Overlays.

FAARFIELD includes four types of overlay pavements:

1. asphalt overlay of existing flexible pavement;
2. asphalt overlay of existing rigid pavement;
3. concrete overlay of existing flexible pavement; and
4. concrete overlay of existing rigid pavement.

4.7.4.1 **Overlays of Existing Flexible Pavements.**

Designing an overlay for an existing flexible pavement is similar to designing a new pavement, except the design layer is the overlay layer. Characterize the existing pavement structure, assigning the appropriate thicknesses and moduli of the existing layers. A flexible overlay requires consideration of many factors including the condition, thickness, and properties of each layer of the existing flexible pavement structure. Milling of the asphalt surface may be required to correct surface and grade deficiencies and/or remove deteriorated existing asphalt surface material. In FAARFIELD, enter the final milled thickness, not the original thickness, for the existing asphalt layer thickness. Internally, FAARFIED iterates on the thickness of the overlay until the CDF at the top of the subgrade equals 1.0. The minimum structural overlay thickness is 3 inches (75 mm). The design thickness of the overlay is the larger of (a) the minimum thickness; or (b) the thickness required to achieve a subgrade or asphalt material CDF of 1. See an example in paragraph H.4.

4.7.4.2 **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. Characterize the existing flexible pavement by assigning the appropriate thicknesses and moduli of the existing layers. A trial overlay thickness is selected and FAARFIELD iterates on the thickness of the concrete overlay until a CDF = 1 is reached. The design thickness is the larger of the minimum PCC thickness or the overlay thickness required to achieve a CDF = 1. FAARFIELD assumes a frictionless (unbonded) interface between the concrete overlay and the existing flexible surface. Do not place a non-stabilized (unbound) material between the overlay and existing structure this would result in a sandwich pavement. The use of a fine stone bond breaker, ¼ inch (5 mm) or less 'choke stone', is not considered a sandwich pavement. It is not required to include the choke stone layer or other bond breaker material in the FAARFIELD structural design. 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. See FAARFIELD concrete overlay example in Appendix H.5.

4.7.5 Overlays of Existing Rigid Pavements.

Consider the structural condition of the existing pavement when designing overlays of an existing rigid pavement. FAARFIELD uses three values to characterize the strength and condition of the existing concrete surface: the flexural strength (*R*) of the existing material, the Structural Condition Index (SCI) and the Cumulative Damage Factor Used (CDFU). Nondestructive testing (NDT), borings, or engineering judgment can help determine the flexural strength *R* of the existing concrete.

4.7.6 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 for a standard overlay because they would require extensive repairs prior to the overlay. For pavements with significant distress, concrete rubblization or similar methods of destroying slab action prior to overlay may be a better alternative (see paragraph 4.8).

4.7.6.1 **Structural Condition Index (SCI).**

The condition of the existing rigid pavement prior to an overlay is expressed by the structural condition index (SCI). The SCI considers only load-related distresses of the PCI. The SCI is reported on a scale of 0 to 100. A pavement with no visible distress would have an SCI of 100 and a pavement with complete structural failure (i.e. loss of all slab action) would have an SCI equal to 0. 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. Because SCI does not deduct for non-structural distresses, the value of SCI is always greater than or equal to the corresponding PCI for a given pavement feature. For additional guidance on PCI, see Chapter 5 and ASTM D 5340, *Standard Test Method for Airport Pavement Condition Index Survey*. The specific distresses considered in SCI are:

- Corner Break (all severities)

- Cracks; Longitudinal, Transverse, and Diagonal (all severities)
- Shattered Slab/Intersecting Cracks (all severities)
- Spalling (Longitudinal and Transverse Joint) (all severities)
- Spalling (Corner) (all severities)

PAVER or FAA PAVEAIR can automatically calculate SCI. When using these programs to calculate SCI, check to make sure the SCI is defined using the distresses noted above. For additional guidance on deriving an SCI, see the FAARFIELD help.

4.7.6.2 **Cumulative Damage Factor Used (CDFU).**

CDFU is used only for overlays on rigid pavements when the SCI of the existing pavement is 100 (i.e., there are no visible cracks or other structural distresses). In all other cases where SCI < 100, CDFU = 100. CDFU represents the estimated percentage of a pavement's fatigue life that has been consumed. This feature is useful in cases where the pavement to be overlaid is not brand new (i.e., has received some traffic), but does not yet have visible damage. Estimate CDFU for pavements constructed on an aggregate base that have had uniform traffic using the following relationship

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

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

Use FAARFIED to calculate CDFU for rigid pavements on stabilized bases.

When computing percent CDFU for a rigid pavement on stabilized base, FAARFIELD sets CDFU to its default value of 100, which will give the most conservative design. To calculate a CDFU other than 100 in FAARFIELD:

1. Set up the structure based on what was constructed.
2. Enter the traffic that has been applied to the pavement.
3. Set "Design Life" to the number of years the pavement will have been in operation up to the time of overlay.
4. Run Life.

When the Life computation is completed the percent CDFU will display. FAARFIELD may compute a value of percent CDFU greater than 100. For those cases, enter a design value of 100 for percent CDFU. Setting percent CDFU to 100 will always give the most conservative design. See an example in paragraph H.9.

4.7.6.3 Asphalt Overlays of Existing Rigid Pavements.

The design process for asphalt overlays of rigid pavements considers two possible conditions: (1) SCI of the existing pavement less than 100 and (2) SCI equal to 100. When the SCI of the concrete base pavement reaches a terminal value the pavement structure is assumed to have failed. Currently, FAARFIELD does not address reflection cracking of the asphalt overlay as a potential failure mode. After FAARFIELD assumes an initial overlay thickness, it then iterates on the overlay thickness until a 20-year life is predicted. The modulus of the overlaid concrete pavement deteriorates with traffic as a function of its reduced SCI. This computation is done automatically within FAARFIELD. See Report No. DOT-FAA-PM-87/19, *Design of Overlays for Rigid Airport Pavements*, for the equations for the reduction of modulus as a function of the SCI.

In general, thicker asphalt overlays perform better than thin asphalt material overlays. Thin asphalt overlays of rigid pavements may cause or exacerbate distresses such as reflection cracking, slippage, and rutting. The minimum thickness of asphalt overlays on existing rigid pavements is 3 inches (75 mm).

4.7.6.3.1 Case 1: SCI Less Than 100.

The most likely situation is one in which the existing pavement exhibits 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.

4.7.6.3.2 Case 2: SCI Equal to 100.

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 (paragraph 4.7.6.2). FAARFIELD assumes the base pavement will deteriorate at different rates before and after the SCI drops below 100. See an example in paragraph H.6.

4.7.6.4 Treatment of Thick Asphalt Overlays on Existing Rigid Pavements.

For flexible overlays on rigid pavements, FAARFIELD assumes the existing rigid pavement supports load through flexural (slab) action. As the overlay thickness increases, the existing rigid pavement will tend to act less like a slab and more like a stiff base material. When the overlay thickness exceeds the thickness of the concrete layer, it may be more economical to evaluate as a flexible pavement design. treating the existing concrete as if it were a high-quality base material. If the option "Allow Flexible Computation for Thick Overlays on Rigid" is set to "Yes" under Design Options, FAARFIELD will perform both computations and report out the thinner flexible overlay. The "Allow Flexible Computations" option has no effect for concrete-on-rigid overlays, or when the calculated asphalt overlay thickness is less than the thickness of the existing concrete layer. The default value for "Allow Flexible Computations" option is "Yes".

4.7.6.5 **Concrete Overlays of Existing Rigid Pavements.**

The design of a concrete overlay of an existing rigid pavement is the most complex type of overlay design. Consider the condition of the existing pavement and the degree of bond between the overlay and existing pavement when designing the overlay. FAARFIELD considers two possible degrees of bond: fully unbonded and fully bonded.

4.7.6.5.1 Fully Unbonded Concrete Overlays.

The design of fully unbonded concrete overlays of rigid pavements assumes no bond between the overlay and existing slab. A bond breaker may be either a thin layer of asphalt mixture or a geosynthetic fabric bond-breaker. FAARFIELD disregards the thickness of any asphalt interlayer or other bond-breaker in the design of the overlay. The minimum thickness for a fully unbonded concrete overlay is 6 inches (150 mm). The design procedure assumes that the existing slab and overlay slab act independently of each other may have different moduli and deteriorate at different rates. During the design procedure, FAARFIELD iterates on the overlay thickness until it finds a design thickness that produces SCI = 80 for the overlay at the end of the 20-year design life. In contrast to asphalt-on-rigid overlay design, there is no defined terminal SCI condition for the existing concrete layer.

4.7.6.5.2 Fully Bonded Concrete Overlays.

On federally funded projects, FAA approval is required for the use of a bonded overlay. Only consider bonded overlays when the existing rigid pavement is in good to excellent condition. Any 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. The new section behaves as a monolithic slab by bonding the concrete overlay to the existing rigid pavement. FAARFIELD treats bonded overlays as a single layer, combining the existing surface and the overlay. The flexural strength used in the FAARFIELD computation should be the strength of the existing concrete. The thickness of the bonded overlay is computed by subtracting the thickness of the existing pavement from the total thickness of the required slab as computed by FAARFIELD. See an example in paragraph H.7.

4.7.7 Jointing of Concrete Overlays.

4.7.7.1 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.

4.7.7.2 The following may be used as a guide in the design and layout of joints in concrete overlays.

1. The timing for sawing joints is extremely critical on concrete overlays to minimize the curling and warping stresses and prevent random cracking.
2. Place contraction joints in unbonded overlays approximately over but within 1 foot (0.3 m) of existing isolation, construction, or contraction joints. Additional intermediate contraction joints may be necessary to control cracking in the unbonded overlay slab. Keep the ratio of slab size to radius of relative stiffness to less than 5.
3. Include embedded crack control steel in overlay slabs longer or wider than 20 feet (6.1 m)., regardless of overlay thickness. Consider embedded crack control steel reinforcement any time that overlay joint spacing is different than the underlying existing slab joint spacing.

4.7.8 Rigid Pavement with Previous Flexible Overlay.

There are many factors to consider when evaluating a rigid pavement that has an existing asphalt overlay. Factors to consider include the condition and thickness of the existing asphalt material overlay. The surface may require partial or complete milling depending on the existing pavement grades and condition of the asphalt material. The condition of the existing overlay will assist in determining the condition of the underlying rigid pavement, however there is no definitive way to establish what SCI to use. Use an SCI of 80 unless there are records or NDT reports that support the use of a lower SCI. Analyze the pavement structure as if the existing asphalt overlay was not present, calculate the overlay thickness required, and then adjust the overlay thickness to compensate for the existing overlay. The designer must use engineering judgment to determine the condition of the rigid pavement.

4.8 Nonstructural Flexible 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 because minimum construction lift thickness or other non-structural design considerations control. The minimum nonstructural asphalt overlay thickness on an existing flexible pavement is 2 inches (50 mm); however, a thicker overlay typically performs better. The overlay thickness should be specified in 0.5-inch (13-mm) increments starting at 2 inches (50 mm) minimum. Prior to removing any existing surface material, it is imperative to take sufficient pavement cores to determine the thickness and condition of the existing surface. When removing existing surface course material by milling, remaining material must have sufficient structural capacity to withstand construction loads. Leaving less than 2 inches of surface course often results 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.

4.9 Alternatives for Rehabilitation of Existing Pavement.

4.9.1 General.

An evaluation of the condition of the existing pavement will assist in determining which rehabilitation alternatives to consider. For example, if the condition of the existing rigid pavement is very poor (e.g., extensive structural cracking, joint faulting, “D” cracking, etc.), rubblization may not be appropriate. Alternatives to overlaying an existing pavement include Full Depth Reclaimation, Rubbilzation and Crack and Seat.

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

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

4.9.2.2 At non primary general aviation airports, serving aircraft less than 30,000 pounds gross weight, it may be possible to place a surface layer of asphalt or concrete directly on the recycled base. However, at larger airports a crushed aggregate base and/or stabilized base may be required in addition to the layer of FDR material prior to placement of a new surface layer.

4.9.2.3 In FAARFIELD, model the FDR layer as a user-defined layer with recommended modulus values ranging from 25,000 to 50,000 psi. When supported with laboratory testing or in-place field tests, higher values may be used. Engineering judgment is required for the selection of an appropriate modulus value for the FDR layer.

4.9.2.4 For the standard construction specification, see AC 150/5370-10, Item P-207, Full Depth Reclamation (FDR) Recycled Aggregate Base Course.

4.9.3 Rubblization of Existing Rigid Pavement.

4.9.3.1 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 3inch (25 to 75-mm) pieces at the top and 3- to 15-inch (75- to 381-mm) pieces at the bottom. Rubblization is accomplished either through mechanical force (a pattern of hammer drops) or by a using a resonant frequency breaker head. The resulting rubblized concrete layer behaves as a tightly interlocked, high-density, non-stabilized base, which prevents the formation of reflective cracks in the overlay.

4.9.3.2 Rubblization of existing concrete pavement may be effective in mitigating reflective cracking. Design the section as a flexible pavement, treating the rubbizied pavement as a base course. Reflective cracking is reduced or eliminated.

4.9.3.3 The thickness design procedure for an overlay over a rubblized concrete base is similar to a new flexible or new rigid pavement design. In FAARFIELD, model the rubblized concrete pavement layer as a user-defined layer with recommended modulus values ranging from 100,000 to 400,000 psi.

4.9.3.4 Engineering judgment is required for the selection of an appropriate modulus value for the rubblized concrete pavement layer. Many factors influence the modulus of the rubbilized layer including the thickness, strength and particle size of the rubblized layer, the condition and type of base, subbase and subgrade materials. Refer to AAPTP Report 04-01, *Development of Guidelines for Rubblization,* and Engineering Brief 66, *Rubblized Portland Cement Concrete Base Course,* for further information.

4.9.3.5 The following are suggested ranges for the design modulus value of rubblized PCC on airfields:

- Slabs 6 to 8 inches thick: E = 100,000 to 135,000 psi
- Slabs 8 to 14 inches thick: E = 135,000 to 235,000 psi
- Slabs greater than 14 inches thick: E = 235,000 to 400,000 psi

4.9.3.6 Install subsurface drainage for rubblized layers prior to rubbilization. See AAPTP Report 04-01.

4.9.4 Crack and Seat.

The crack and seat process involves using a hammer to fracture a concrete pavement 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. Rubbilization techniques have almost completely replaced crack and seat methods. On federally funded projects coordinate with FAA during the design phase regarding the use of rubblization or crack and seat techniques.

4.9.5 Pavement Interlayers.

4.9.5.1 An interlayer is a material or mechanical system placed between the existing pavement and the overlay to improve overlay performance. Types of interlayers may include: aggregate-binder courses; double chip seal, stress absorbing membrane interlayers (SAMI); paving fabrics; grids; or a combination of the above. The use of interlayers does not eliminate the need to fill cracks in existing pavement.

4.9.5.2 Before including pavement interlayers to retard reflective cracking, compare the cost of the interlayer the cost of providing additional thickness of asphalt material.

4.9.5.3 Do not consider pavement interlayers when existing pavements (flexible or rigid) show evidence of excessive deflections, substantial thermal stresses, and/or poor drainage. Some interlayers may impede future rehabilitation or reconstruction. When material placed on top of fabric does not meet acceptance standards replace deficient material and any damaged fabric.

4.9.5.4 Paving fabrics provide waterproofing when overlaying full depth asphalt pavement minimizing the amount of water that can get into the subgrade. However, the fabric may trap water in the upper layers of the pavement structure leading to premature surface deterioration and/or stripping.

4.9.5.5 FAARFIELD does not attribute any structural benefits to pavement for any type of interlayers in flexible thickness design. Evaluate the cost and benefits of an interlayer versus additional thickness of asphalt surface material on federally funded projects.

4.9.5.6 The FAA does not support the use of interlayers unless documentation in engineering report supports why the use is justified and what benefit it will provide to cost and life of pavement structure.

4.10 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.

4.10.1 Flexible Pavements.

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

4.10.1.1 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. Correct subsurface deficiencies prior to installation of a patch. Replace the unstable subgrade material with a select subgrade soil or install subsurface drainage facilities. Place and compact the subbase, base, and surface courses after correction of the subgrade condition.

4.10.1.2 Profile Milling.

Correct surface irregularities and depressions, such as shoving, rutting, scattered areas of settlement, "birdbaths," and bleeding with profile milling and by leveling with suitable asphaltic material mixtures. The leveling course should consist of high-quality asphalt mixture. See AC 150/5370-10 P-401 or P-403.

4.10.1.3 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.

4.10.1.4 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.

4.10.1.5 Porous Friction Courses (PFC).

Remove existing PFCs prior to any overlay.

4.10.1.6 Paint and Surface Contaminants.

Remove or scarify paint prior to an asphalt overlay to ensure bonding of the overlay to the existing pavement. Remove surface contaminants that will prevent bonding of the surface overlay (e.g., rubber, oil spills, etc.) prior to an asphalt overlay.

4.10.2 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. No corrective measures are needed when the subgrade is stable and no pumping has occurred. If slabs have been pumping or rocking under aircraft traffic this can be mitigated with injection of chemicals or cement grout into voids in subgrade. Consult an experience pavement or geotechnical engineer before performing chemical or grouting. A dowel bar retrofit or reconstruction may be required if extensive areas of pumping or rocking.

4.10.2.1 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. When broken and unstable slabs are throughout entire area then steps such as crack and seat procedures, rubbilization or reconstruction will be required. Refer to AAPTP 05-04, *Techniques for Mitigation of Reflective Cracks,* for additional information.

4.10.2.2 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.

4.10.2.3 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.

4.10.2.4 Surface Cleaning.

Prior to placing the overlay sweep the pavement surface to remove all dirt, dust, and foreign material. Remove excess joint-sealing material from rigid pavements. Paint does not require removal prior to construction of an unbonded concrete overlay.

4.10.3 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. To facilitate an adequate surface to bond to clean and prepare surface by shot peening or cold milling. A bonding agent will 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 concrete overlay.

4.10.4 Materials and Methods.

AC 150/5370-10, *Standard Specifications for 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 concrete pavement (Item P-501) or appropriate asphalt mixture pavement (Item P-401, P-403, P-404) requires FAA approval of a modification to standards.

Page Intentionally Blank

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 process. All evaluation projects involve a similar process as described in more detail in the following paragraphs.

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. Use pavement management programs PAVER or FAA PAVEAIR, to calculate current PCI and develop pavement management scenarios.

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 relatively new pavements constructed to FAA standards with no visible sign of wear or stress, use information 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 pavement's 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. Evaluate pavement profiles with programs such as ProVal or 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 include 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 Reflectometer (FWD) and Heavy 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 **Nondestructive Testing and Minimally Destructive Testing– Methods other than FWD/HWD.**

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. Use GPR to locate voids or foreign objects, such as abandoned fuel tanks and tree stumps, under pavements and embankments.

5.2.4.4.2 Dynamic Cone Penetrometer.

One use of dynamic cone penetrometer (DCP) is to estimate mechanical properties of pavement materials or subgrade soils at shallow depths. The test involves driving a cone shaped tip of a rod using the impact of a falling mass. The dynamic penetration index (DPI), defined as the penetration of the cone for each drop of the mass can be correlated with many engineering properties such as California Bearing Ratio (CBR), the resilient modulus and the shear strength. Common correlations for all soils except CL soils below CBR10 and CH soils:

$CBR = 292 / DCP^{1.12}$ for DCP in mm/blow

$CBR = 292 / (DCP \times 25.4)^{1.12}$ for DCP in in/blow

or for:

CL soils with CBR < 10

$CBR = 1 / (0.017019 \times DCP)^2$ for DCP in mm/blow

$CBR -= 1 / (0.433383 \times DCP)^2$ for DCP in in/blow

For CH Soils:

$CBR = 1 / 0.002871 \times DCP$ for DCP in mm/blow

$CBR = 1 / (0.072923 \times DCP)$ for DCP in in /blow

See Appendix D for additional correlations.

5.2.4.4.3 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 Pavement Evaluation Report.

5.2.5.1 Incorporate the analyses, findings, and test results into an evaluation report, 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.

Perform laboratory CBR tests on soaked specimens in accordance with ASTM D 1883, *Standard Test Method for California Bearing Ratio (CBR) of Laboratory-Compacted Soils*. Where it is impractical to perform laboratory or field CBR tests, a use back calculated subgrade elastic modulus values 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 can be input directly into FAARFIELD without manually converting to CBR. However, the back calculated CBR will be representative of the subgrade moisture at the time of the NDT testing.

5.3.3 Layer Properties.

The materials in FAARFIELD are designated by item numbers corresponding to standard materials in AC 150/5370-10. For example, where an existing flexible pavement consists of an asphalt material surface on a high-quality crushed aggregate base meeting FAA Item P-209, input the base layer as P-209 Crushed Aggregate in FAARFIELD. For materials that differ significantly from standard materials, input an appropriate modulus value using either the “User-defined” or “variable” layer types. FAARFIELD allows an unlimited number of layers beneath the asphalt surface; however, it is not recommend to limit to no more than 5 layers.

5.3.4 Example of Flexible Pavement Evaluation Procedures.

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.

For this example, valuate a taxiway pavement constructed to FAA standards with the pavement structure shown below (Figure 5-1):

Thickness (inches)	Pavement Layer
4.0	P-401 Asphalt Mixture Surface Course
5.0	P-401/403 Stabilized Base Course
12.0	P-209 Crushed Aggregated Base Course

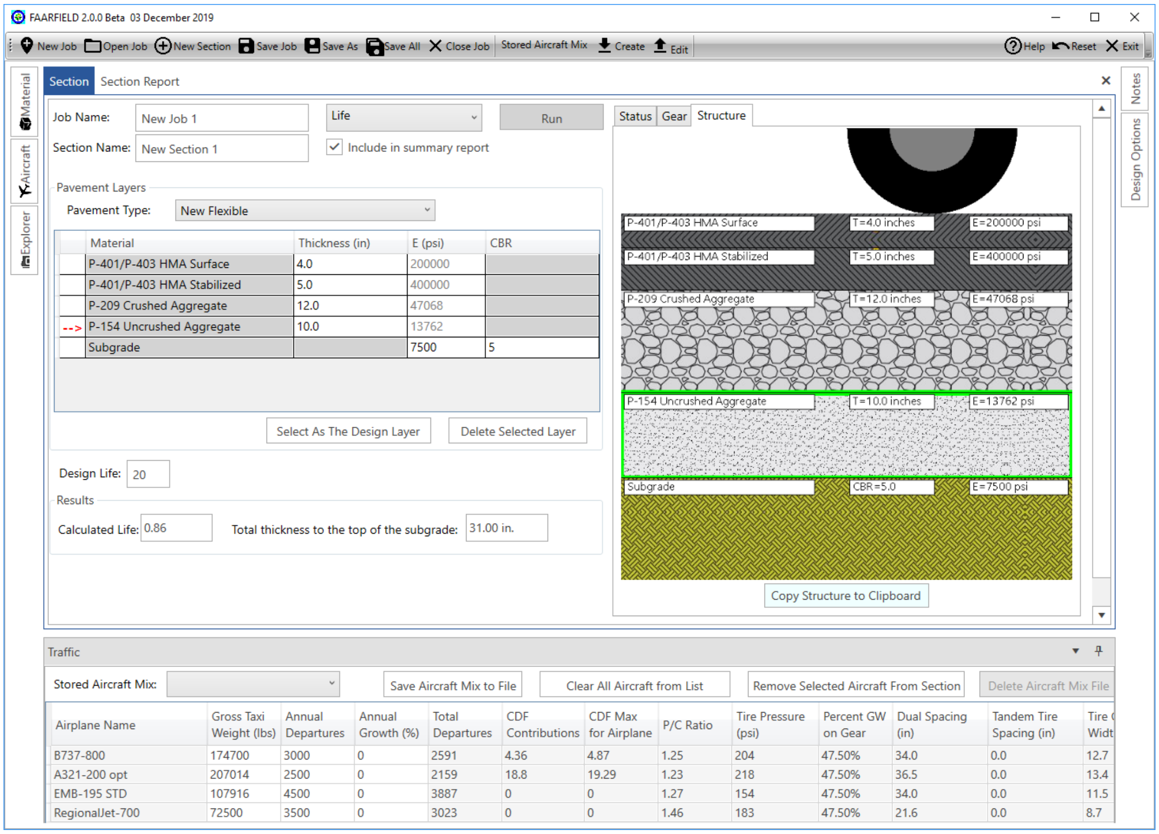
Thickness (inches)	Pavement Layer
10.0	P-154 Subbase Course
	Subgrade, CBR = 5

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

1. Using the traffic mixture shown above FAARFIELD can determine the available structural life, checking CDF of subgrade and asphalt.
2. The following steps are used:
    a. After opening FAARFIELD, begin by selecting pavement type “New Flexible” from the drop-down list. Adjust the layer thickness and material type for each layer, as necessary to match the existing pavement structure.
    b. Use standard material types to model each layer for pavements constructed following FAA standards. Enter the above airplane list from the FAARFIELD aircraft library. For each aircraft on the list, select the appropriate aircraft group, and aircraft name from the list on the left. The aircraft will appear on the “Traffic” list at the bottom of the screen. Modify gross weights and annual departures directly on the traffic list.
    c. On the Explorer tab, click “Design Options.” Ensure the “Calculate HMA CDF” option is set to “Yes.” Close or hide the Design Options.
    d. From the drop-down list at the top of the screen, select “Life.” Click “Run.” The FAARFIELD evaluation screen displays as shown in Figure 5-1.

Figure 5-1. Existing Taxiway Pavement Structure

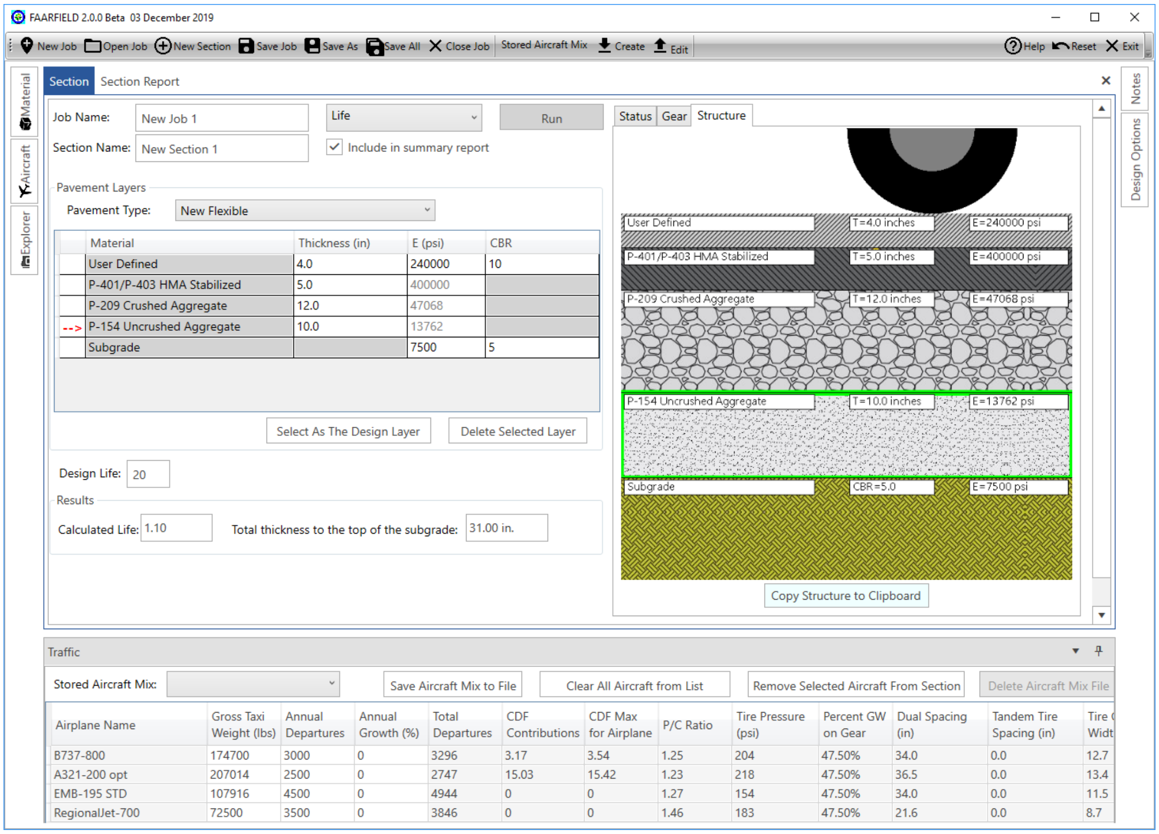


3. The computed value of subgrade CDF (Sub CDF) is 23.16, which is greater than 1.0, indicating that the structure has insufficient thickness to protect the subgrade for the given traffic for the design life. Based on the subgrade failure criteria, the predicted structural fatigue life for the given structure and traffic loading is 0.9 years. FAARFIELD also reports that 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. This evaluation indicates an overlay is needed to support the given traffic mix. The procedures in Chapter 4 should be used to design the required overlay thickness.

4. The above example assumes that all layers were constructed to FAA standards. When it is not known what standards were used for construction, use NDT to determine material properties. Use the user defined layer in FAARFIELD to model layers that deviate from standard materials.

Note: Deviations from the standard material modulus values in FAARFIELD may have a relatively minor effect on the predicted structural life, depending where the layer is in the pavement structure. As an illustration of this, Figure 5-2 is similar to Figure 5-1, except that the asphalt 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.

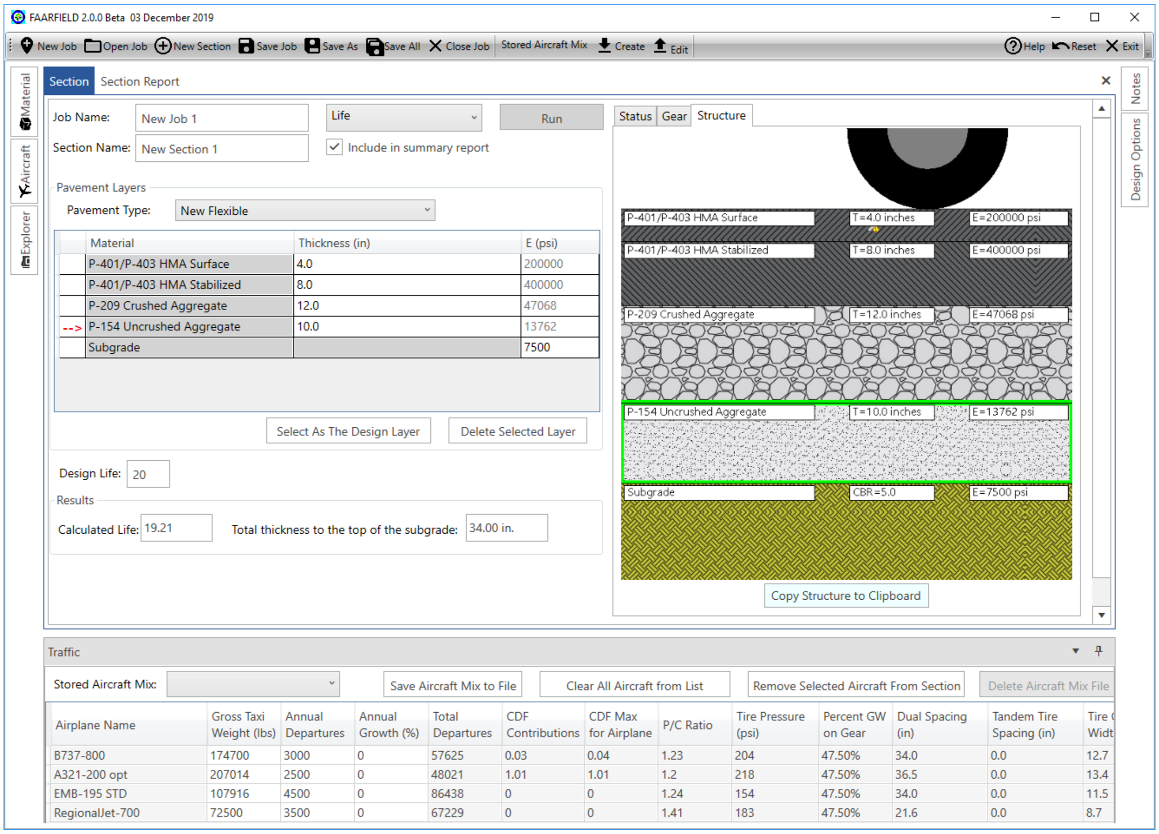
Figure 5-2. Existing Taxiway Structure with User-Defined Surface Layer



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). For this example, model the existing 3-inch (75-mm) surface course and 5-inch (125-mm) stabilized base as an 8-inch (200-mm) stabilized base layer. Use information available from NDT testing to model the existing layers as user-defined layers in a FAARFIELD overlay design.

Figure 5-3. Flexible Pavement Evaluation (with Overlay)



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 <u>Layer Thicknesses.</u>

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

5.5.2 <u>Concrete Flexural Strength.</u>

5.5.2.1 Use construction records or NDT data as the source for concrete flexural strength data. Construction strength data of the concrete strength may need to be adjusted upward to account for 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 to establish subgrade modulus. Use a back calculated subgrade elastic modulus value from NDT results with appropriate adjustments. When subgrade conditions at time of testing are not representative of average annual conditions, adjust NDT results as necessary. Appendix C gives a procedure for obtaining back calculated modulus values.

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 backcalculated E modulus value or k value can be input directly into FAARFIELD. If a backcalculated 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 Material types in FAARFIELD are designated by item numbers that correspond to standard materials in AC 150/5370-10. For example, a flexible pavement consisting of an asphalt material surface on a high-quality crushed aggregate base, in FAARFIELD input the base layer as P-209 Crushed Aggregate. Input an appropriate modulus valued using either the “User-defined” or “variable” layer types in FAARFIELD for materials that differ significantly from standard materials. In FAARFIELD, the number of structural layers above the subgrade for a rigid pavement is limited to 4, including the concrete surface layer. If the actual rigid pavement structure evaluated consists of more than 4 distinct layers, combine two or more of the lower layers to reduce the total number of layers to 4 or fewer for analysis. Rigid pavement life evaluation is not highly sensitive to modulus properties of layers above the subgrade.

5.5.5 Example of Rigid Pavement Evaluation Procedures.

5.5.5.1 Use FAARFIELD to determine the remaining structural life of an existing pavement for a given traffic mix. For this example, consider a concrete-surfaced taxiway designed for a 20 years structural life with the structure and traffic as shown below.

Pavement structure:

Layer Thickness (in)	Pavement Layer
16.1	P-501 Concrete Surface Course (R = 650 psi)
6.0	P-304 Cement-treated Base Course
12.0	P-209 Base Course
	Subgrade, E = 7,500 psi

Airplane traffic 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

5.5.5.2 After 10 years of use, the airplane traffic mix using the taxiway recently changed and now includes heavier aircraft. An evaluation of the subgrade using NDT provided a backcalculated E-modulus of 7500 psi (52 MPa). Cores taken on the taxiway tindicated he in-place layer properties for the pavement structure are as follows:

Layer Thickness (in)	Pavement Structure
17.25	P-501 PCC Surface Course (R = 685 psi)
6.0	P-304 Cement-treated Base Course
12.0	P-209 Base Course
	Subgrade, E = 7500 psi

The current traffic mix is as follows:

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
A380	1,238,998	1200

Airplane	Gross Weight (lbs)	Annual Departures
B777-300 ER	777,000	110

5.5.5.3 A life evaluation of the existing pavement structure indicates a remaining structural fatigue life of 15.5 years with the current 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 the percent CDFU is only about 2.5% after 10 years of service (Figure 5-5). Therefore, in this case it is reasonable to ignore the contribution of the earlier traffic and consider that the total life computed by FAARFIELD is the remaining life of the structure under the current traffic. Future changes in type of aircraft and actual operating weights will influence performance of pavement. Monitor the performance of the taxiway pavement with regular pavement inspections.

Figure 5-4. Rigid Pavement Evaluation - Life Evaluation for Current Traffic

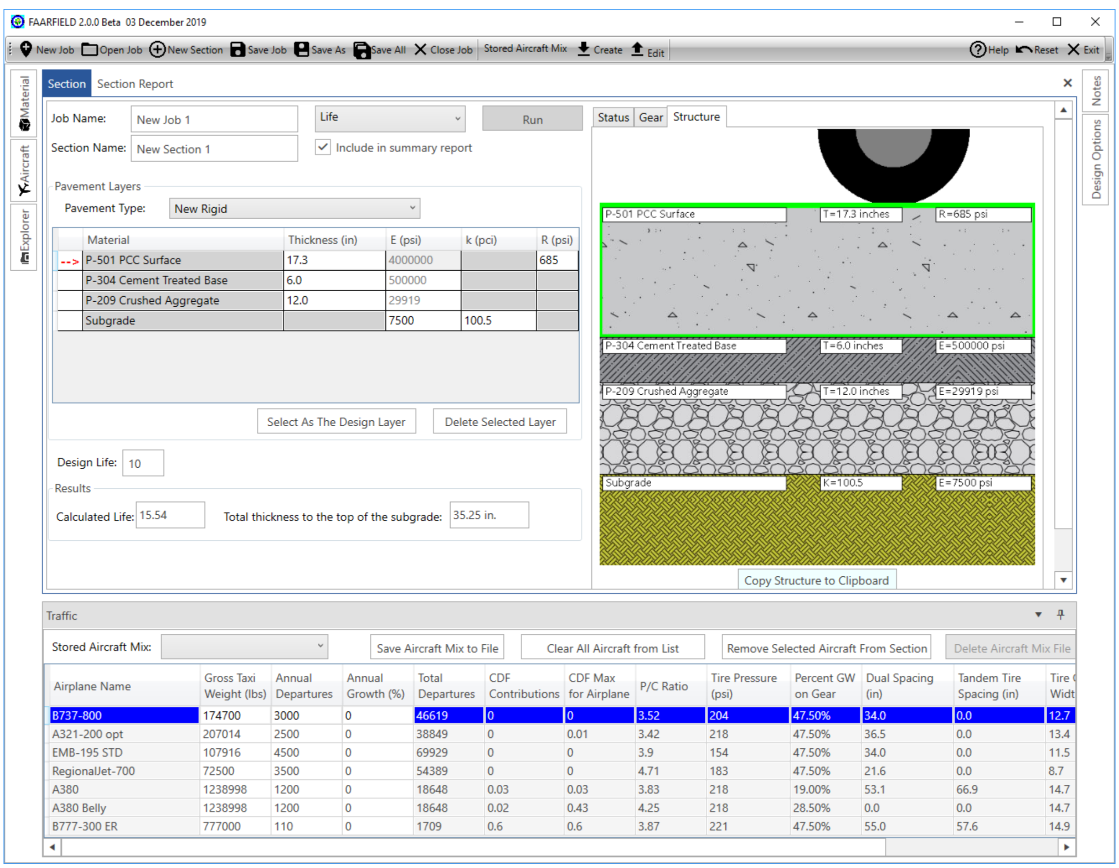
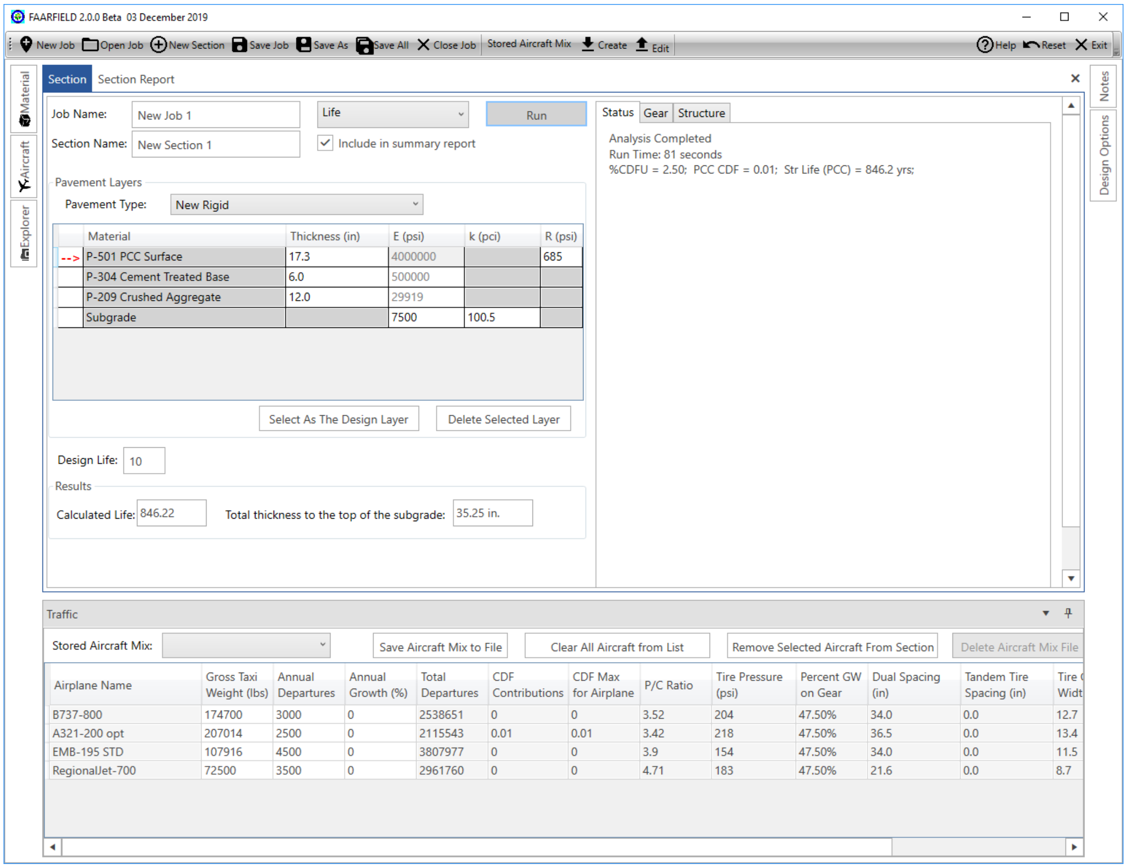


Figure 5-5. Rigid Pavement Evaluation - Life Evaluation for Original Traffic (After 10 Years of Traffic)



5.6 Use of Results.

Notify the airport when the existing pavement does not meet pavement design standards from Chapter 3. 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 main concern is not the load-carrying capacity but the difference between the existing pavement structure and the structure required to support the forecasted traffic.

5.7 Reporting Pavement Weight Bearing Strength.

5.7.1 Aircraft Classification Rating/Pavement Classification Rating (ACR/PCR).

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 Rating/Pavement Classification Rating (ACR/PCR). ACR-PCR reports strength relative to a derived equivalent single wheel load. FAARFIELD 2.0 includes an option to calculate ACR-

PCR. AC 150/5335-5, *Standardized Method of Reporting Airport Pavement Strength – PCR,* provides guidance on calculating and reporting PCR.

5.7.1.2 Report the PCR 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.

Page Intentionally Blank

CHAPTER 6. PAVEMENT DESIGN FOR SHOULDERS

6.1 Purpose.

6.1.1 This chapter provides the FAA design procedure for paved airfield shoulders. Note Design blast pads and stopways 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. Design shoulders to support the occasional passage of the most demanding airplane, emergency or maintenance vehicles.

6.1.3 Paved shoulders are required for all pavements for Airplane Design Group (ADG) IV and higher aircraft. For runways designed for ADG III aircraft, paved shoulders are recommended. For runways designed for ADG-1 or ADG-2 aircraft, stabilized soil shoulders are recommended. Suitable stabilizers include turf, aggregate-turf, soil cement, lime or bituminous material. Refer to AC 150/5300-13 for standards and recommendations for airport design.

6.2 Shoulder Design.

6.2.1 Design shoulders to accommodate the most demanding of (1) a total of 15 passes of the most demanding airplane or (2) anticipated traffic from airport maintenance vehicles. See Table 6-1 for minimum layer thicknesses for shoulder pavement. Design shoulder pavements to accommodate safe emergency operation of an 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 Consider drainage from the adjacent airfield pavement base and subbase when establishing the total thickness of the shoulder pavement section. A thicker shoulder section than is structurally required and edge drains may be necessary to avoid trapping water under the airfield pavement. Slope base, subbase and subgrade to match adjacent RW 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. The most demanding aircraft is generally the aircraft with the largest contribution to CDF. It is not necessary to perform a separate design for each airplane in the traffic mix, rather just those with the largest contributions to the CDF. Perform a separated analysis of vehicles and equipment that also may operate on the shoulder. Vehicles to consider include Aircraft Rescue and Firefighting (ARFF), maintenance, and snow removal vehicles.

6.2.4 Use the following steps 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: Utilize User Defined pavement layer to represent the proposed shoulder pavement cross-section when layer thicknesses exceed the minimum layer thickness requirements due to constructability need to match adjacent layers.

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 is 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, 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.

Use high quality base course materials, similar to FAA Items P-208, P-209, P-301, or P-304. See AC 150/5370-10 for specifications for Item P208, 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 with addition of non-frost susceptible material to avoid differential frost heave. The non-frost suceptable material should possess a CBR value higher than the subgrade 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 3.12.16.

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) Aircraft < 60,000 lbs (27,215kg)	Minimum Thickness, in (mm) Aircraft >60,000 lbs (27,215kg)
Asphalt Surface	P-401, P-403	3.0 (75)	4.0 (100)
Concrete	P-501	5.0 (125)	6.0 (150)
Aggregate Base Course	P-209, P-208,	6.0 (150)	6.0 (150)
Subbase (if needed)	P-154	4.0 (100)	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 STRUCTURE

B.1 Background.

Design airport structures such as culverts and bridges designed to last for the foreseeable future of the airport. Information concerning the landing gear arrangement of future heavy airplanes is speculative. Pavements can be strengthened as necessary to accommodate future loads. It is difficult, costly and time-consuming to strengthening structures. 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 (seventh edition).

B.2.2 Foundation Design.

Foundation design will vary with soil type and depth. Design footings for shallow structures considering the concentrated loads of aircraft.in addition to load cases required by structural design standards.

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, consider wheel loads as uniformly distributed over a square with sides equal to 1.75 times the depth of the fill. When loads from multiple wheel overlap, distribute the load uniformly over the area defined by the outside limits of the individual wheels.
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: Treat all loads 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, or highest of using aircraft.

   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, base the design on the number of wheels that will fit the span. Design for the maximum wheel load anticipated at that location over the life of the structure. Design loads at large hub airports should consider future aircraft. It is conceivable that the design loads include a 1,500,000-pound (680,000 kg) aircraft.

2. Consider both in line and skewed loadings for structures that accommodate diagonal taxiway or aprons.

B.2.5 Pavement to Structure Joints.

Design airport structures to support the design loads without assistance from adjacent pavements. Do not consider load transfer to pavement slabs when designing structures. Provide isolation joints (Type A or A-1) be provided where concrete slabs abut structures. For slabs with penetrations, provide a minimum of 0.050 percent of the slab cross-sectional area in reinforcement in both directions.

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

C.1 General.

Nondestructive testing (NDT) makes use of many types of data-collection equipment and methods of data analysis. Engineers use the NDT data to evaluate the load-carrying capacity of existing pavements to calculate remaining life; calculate crack and joint load transfer efficiency; void detection at rigid pavement corners and joints; determine the material properties of in-situ pavement layers and the subgrade layer for design of overlay thickness requirements of pavements; compare relative material stiffness and/or condition within sections of a pavement system to each other; correlate to conventional characterizations (i.e., California Bearing Ratio, k-value); and provide structural performance data to supplement visual survey data in an airport pavement management program (PMP). NDT will also have an increasing role in airport pavement construction quality control and quality acceptance. This appendix is restricted to only NDT deflections with a falling-weight-type impulse load device.

C.2 NDT Using Falling-Weight-Type Impulse Load Devices.

NDT equipment includes both deflection and non-deflection testing equipment. There are several categories of deflection measuring equipment: static, steady state, 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 deflect. Impulse load devices impart an impulse load to the pavement with a freefalling 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 measured with deflection sensors. 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 the aircraft. Deflection measuring equipment for NDT of airport pavements include falling weight deflectometer (FWD), heavy weight deflectometer (HWD), and light weight deflectometer (LWD). Table C-1 lists several ASTM standards that apply to deflection measuring equipment.

C.2.1 FWD imposes dynamic loading on the pavement surface using a load cell and measures surface deflections with sensors. Load levels of the FWD are often not adequate for evaluating thicker airfield pavement structure but may have applications for thinner airfield pavement structures. FWD is typically used on flexible asphalt, rigid concrete, or composite pavements. For more information, refer to ASTM D4694, *Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load Device*.

C.2.2 HWD is commonly used in airfield pavement evaluation and uses the similar principle with FWD, while using greater load levels of nearly 70 kips. HWD is typically used on flexible asphalt, rigid concrete, or composite pavements. For more information, refer to

ASTM D4694, *Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load Device.*

C.2.3 LWD is a portable version of the FWD using a load cell and deflection measuring sensors. The LWD data can be used to calculate material stiffness of airport pavement layers but is limited to unbound materials such as aggregate (base layers) and soil (subgrade) applications due to load cell limitations. Plots of the layer modulus data provide information about changes in layer types and layer stiffness to help quality control of base, subbase, and subgrade layers during construction. For more information, refer to ASTM E2583, *Standard Test Method for Measuring Deflections with Light Weight Deflectometer (LWD).*

Table C-1. ASTM Standards for Deflection Measuring Equipment

ASTM	Deflection Measuring Category		
	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)*			●
E 2835, *Standard Test Method for Measuring Deflections using a Portable Impulse Plate Load Test Device*			●

C.3 NDT Using Falling-Weight-Type Impulse Load Devices Advantages.

C.3.1 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 accurately and quickly measure data at several locations while keeping a runway, taxiway, or apron operational. The use of NDT to collect structural data minimizes any disruptions to airport operations.

C.3.2 Collecting NDT data is economical to perform at up to 250 locations per day using a HWD. HWD equipment measures pavement surface response (i.e., deflections) from an applied dynamic load that simulates a moving wheel. Engineers can vary the magnitude of the applied dynamic load to simulate the single wheel load of the most demanding or design aircraft. Deflection sensors record pavement deflections directly beneath the load plate and at transverse and longitudinal offsets. Typical longitudinal offsets for airport pavement structures are 2 inches (30 cm), out to typical distance of 72 inches (180 cm).

C.3.3 The deflection data collected with HWD equipment provides both qualitative and quantitative data about the stiffness of an entire pavement structure at the time of testing. The raw deflection data directly beneath the load plate sensor provides an indication of the material stiffness of the entire pavement structure. The raw deflection data from the outermost sensor provides an indication of subgrade stiffness.

C.3.4 In addition, deflection or stiffness profile plots of deflection data along an entire pavement facility show relatively strong and weak locations.

C.3.5 Quantitative data derived from HWD include material properties for flexible, rigid, or composite pavement layers and the subgrade layer. Engineers use the HWD derived material properties (e.g., modulus of elasticity, modulus of subgrade reaction) and other physical properties (e.g., layer thicknesses, interface bonding conditions) to evaluate the structural remaining life of a pavement or investigate rehabilitation options. BAKFAA is the FAA software to perform backcalculation of pavement material properties using HWD data.

C.3.6 LWD provides material properties of unbound aggregate and subgrade layers to use for quality control and quality assurance during construction. Modulus of elasticity is more useful for pavement evaluation and design than conventional methods of construction quality control and quality assurance.

C.4 NDT Using Falling-Weight-Type Impulse Load Devices Limitations.

C.4.1 NDT has some limitations. NDT is a very good methodology for assessing the structural condition of an airfield pavement; however, other methods are necessary to evaluate the functional condition of the pavement (e.g., visual condition, roughness, and friction). The visual condition is most frequently assessed 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.* The roughness is most frequently assessed in accordance with AC 150/5380-9, *Guidelines and Procedures for Measuring Airfield Pavement Roughness.* Friction is most frequently assessed in accordance with AC 150/5320-12, *Measurement* *of Skid-Resistant Airport Pavement Surfaces.* Once the NDT-based structural and functional conditions are known, the engineer can assign an overall pavement condition rating.

C.4.2 The differentiation between structural and functional performance is important in developing requirements for pavement rehabilitation. For example, a pavement may have a low PCI primarily caused by environmental distresses, yet the pavement has sufficient structure to accommodate fleet mix loading.

C.4.3 NDT may provide excellent information about structural capacity to evaluate an in place pavement structure, but the equipment is not sensitive enough to evaluate other important engineering properties of the pavement layers (e.g., grain-size distribution aggregate particles, swelling and heaving potential, permeability).

C.4.4 Material property results derived from raw NDT data are model dependent. The backcalculated layer material property results depend on the structural models and software algorithms that process NDT data. For flexible pavements, static HWD backcalculation models for elastic modulus results have been known to overestimate the actual base aggregate, subbase aggregate, and subgrade elastic modulus values.

C.4.5 The structural theory and models for continuously reinforced concrete pavement, post-tensioned concrete, and pre-tensioned concrete are significantly different from traditional pavements. Most NDT software only evaluates Asphalt, jointed plain Concrete, jointed reinforced concrete, asphalt overlaid concrete, and concrete overlaid concrete.

C.4.6 HWD results are time and temperature sensitive. Testing conducted at different climatic conditions during the year may give different results. For example, tests conducted during spring thaw or after extended dry periods may provide non-representative results or inaccurate conclusions on pavement subgrade stiffness.

C.4.7 Due to the load cell size of an LWD, applications are limited to unbound materials or thin asphalt pavement layers.

C.5 NDT Test Planning.

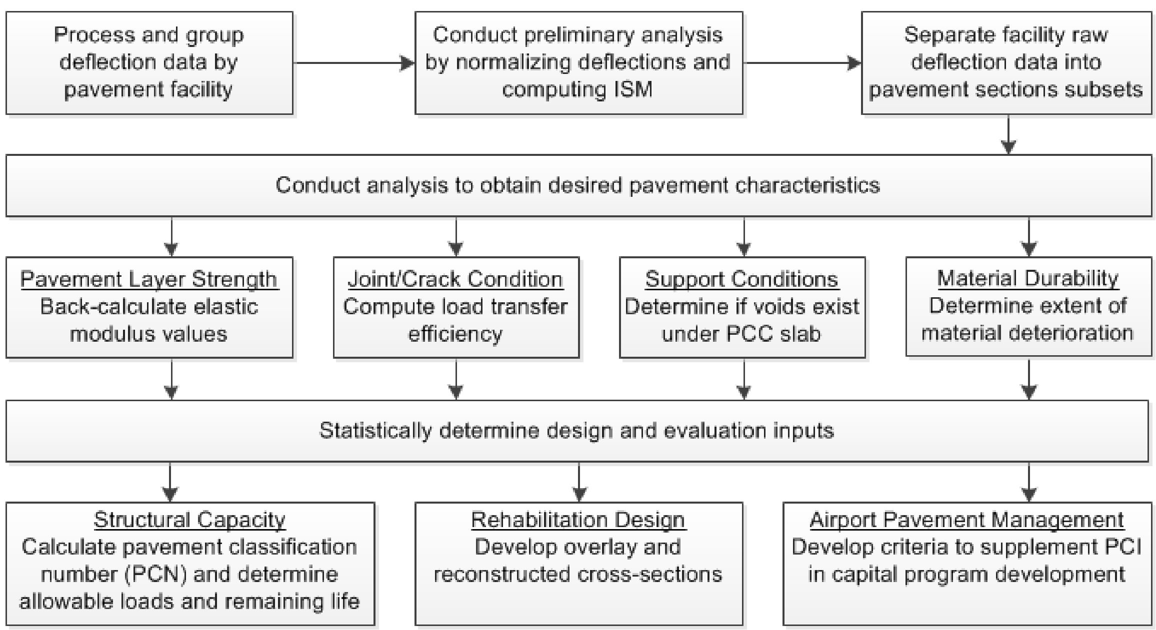
C.5.1 NDT combined with the analytical procedures described here can provide a direct indication of a pavement's structural performance. Visual condition surveys, such as the PCI procedure, provide excellent information regarding the functional condition of the pavement. However, visual distress data 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 to conduct. The total number of tests will depend primarily on the area of the pavement included in the study; the types of pavement; and whether the study is a project or network-level investigation.

C.5.2 Project-level evaluation objectives focus on load-carrying capacity of existing pavements or provide material properties of in-situ pavement layers for rehabilitation design. Network-level objectives include collection of NDT data to supplement pavement condition index (PCI) survey data and generate Pavement Classification

Ratings (PCR) for each airside facility in accordance with AC 150/5335-5, *Standardized Method of Reporting Airport Pavement Strength – PCR.* Refer to AC 150/5380-7, *Airport Pavement Management Program (PMP),* for guidance on developing a PMP.

C.5.3 Several methods evaluate the structural condition of an existing pavement structure using deflection data. The most common use of deflection data is to backcalculate the material stiffness of the structure from the measured deflection basin to determine the individual layer properties within the structure. Typically, airport concrete pavements use expansion, contraction, and construction joints. Joint deterioration and decreasing load transfer efficiency lead to higher deflections at slab corners that may create voids beneath the slab. The voids allow excessive moisture accumulation at the joints causing accelerated concrete material durability deterioration. Figure C-1 provides an overview of the process for using deflection data to evaluate the structural condition of an existing pavement structure.

Figure C-1. Flowchart for Using Deflection Data



C.6 Climate and Weather Affects.

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

C.7 Mobilization.

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, prior to mobilizing equipment.

C.8 HWD Test Locations and Spacing.

C.8.1 For all types of pavements, the most common is a center test. For jointed concrete and asphalt overlaid concrete pavements, this is a test in the center of the concrete pavement slab. For asphalt pavements, this is a test in the center of the wheel path. Avoid cracking between the load plate and deflection sensors. The center test primarily collects deflection data to measure a deflection basin.

C.8.2 For concrete and asphalt overlaid rigid pavements, HWD at various locations along the joints reflection cracking through the overlay provides data regarding pavement response to aircraft loading and changes due to climatic conditions.

C.8.3 HWD testing at longitudinal and transverse concrete joints measures load transfer of an aircraft's main gear from the loaded slab to the unloaded slab. Pavement life extends when load transfer increases to the unloaded slab, because the flexural stress in the loaded slab decreases. Effective load transfer depends on many factors including: pavement temperature; the use of dowel bars; and the use of a stabilized base beneath the concrete pavement layer.

C.8.4 HWD testing at the corner of a concrete slab is another common test location. The corner of a concrete slab is an area where loss of support beneath the concrete slab occurs more often than other areas in the slab. Corner testing is performed with the load plate within 6 inches (15 cm) of the transverse and longitudinal joints.

C.8.5 Center, joint, and corner of concrete tests are performed on the same slab to evaluate the relative stiffness at different locations.

C.8.6 The location and testing interval for each pavement facility should be sufficient to characterize the material properties. Center slab HWD test locations and spacing should be in the wheel paths, spaced between 100 feet and 400 feet along the runway length. Additional testing for load transfer of concrete should include testing at transverse and longitudinal joints. For PCR calculation, randomly test the keel section of the runway within the wheel path of the critical aircraft in the fleet mix. For flexible, rigid, or composite pavements, do not conduct testing near cracking unless one of the test objectives is to calculate load transfer efficiency across the cracking. For asphalt pavements, HWD testing should be at least 1.5 feet (0.5 m) to 3 feet (1 m) away from longitudinal construction joints. Evenly distribute the total number of tests over the evaluation area. Typically, each adjacent HWD pass is 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.9 Deflection Measuring Parameters.

C.9.1 The most common type of equipment in use is the falling-weight-type impulse load device. 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, the loading plate, the deflection sensors, the load cell, the data processing, and storage system.

C.9.1.1 Load Plate Diameter.

Many falling-weight-type impulse load equipment manufacturers offer the option of a 5.91-inch (15-cm) or an 8.86-inch (22.5-cm) radius load plate. Typically, airport pavement evaluation requires the 5.91-inch (15-cm) radius load plate.

C.9.1.2 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. In general, devices that have more sensors can more accurately measure the deflection basin. Accurate measurement of the deflection basin is critical when backcalculating the elastic modulus of individual pavement layers. Most equipment allows repositioning of sensors, but there are benefits to using the same configuration, regardless of the type of pavement structure. Table C-2 shows the FAA's recommended sensor configuration.

Table C-2. Recommended Sensor Configuration

Deflection Sensor Distance from Center of Load Plate, inch (cm)						
d_o	d_{12}	d_{24}	d_{36}	d_{48}	d_{60}	d_{72}
0 (00)	12 (30)	24 (60)	36 (90)	48 (120)	60 (150)	72 (180)

C.9.1.3 Pulse Duration.

For falling-weight-type 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.

C.9.1.4 Load Linearity.

For most pavement structures and testing conditions, engineers assume traditional paving materials will behave in a linear elastic manner within the load range testing.

C.9.2 Sensitivity studies at the FAA's National Airport Pavement Test Facility (NAPTF) and Denver International Airport (DIA) have shown there is little difference in the pavement response under varied HWD impulse loading. Generally, the impulse load should range between 20 kips (90 kN) and 55 kips (250 kN) on pavements serving commercial air carrier aircraft. The amplitude of the impulse load is not critical provided the pavement deflections are within the operational limits of each deflection sensor. The key factors that will determine the allowable range of impulse loads are pavement layer thicknesses, layer stiffness, and layer material types. FWD and LWD may provide an impulse load adequate to evaluate thinner pavements serving general aviation aircraft.

C.10 Pavement Stiffness and Sensor Response.

C.10.1 The load-response data that falling-weight-type impulse load equipment measures in the field provides valuable information on the material stiffness of the pavement structure. Initial review of the deflection under the load plate (d_0) is an indicator of pavement stiffness. The deflection under the outermost sensor (d_{72}) is an indicator of subgrade stiffness. The load-response data does not provide the stiffness of each pavement layer, but it does provide a quick assessment of the pavement's overall stiffness and relative variability of stiffness within a particular airport facility (e.g. runway, taxiway, apron).

C.10.2 Pavement stiffness is the dynamic force divided by the pavement deflection at the center of the load plate. The Impulse Stiffness Modulus (ISM) is defined as follows for falling-weight-type impulse load equipment, respectively:

Equation C-1. Impulse Stiffness Modulus

$$ISM = \left(\frac{L}{d_0}\right)$$

Where:

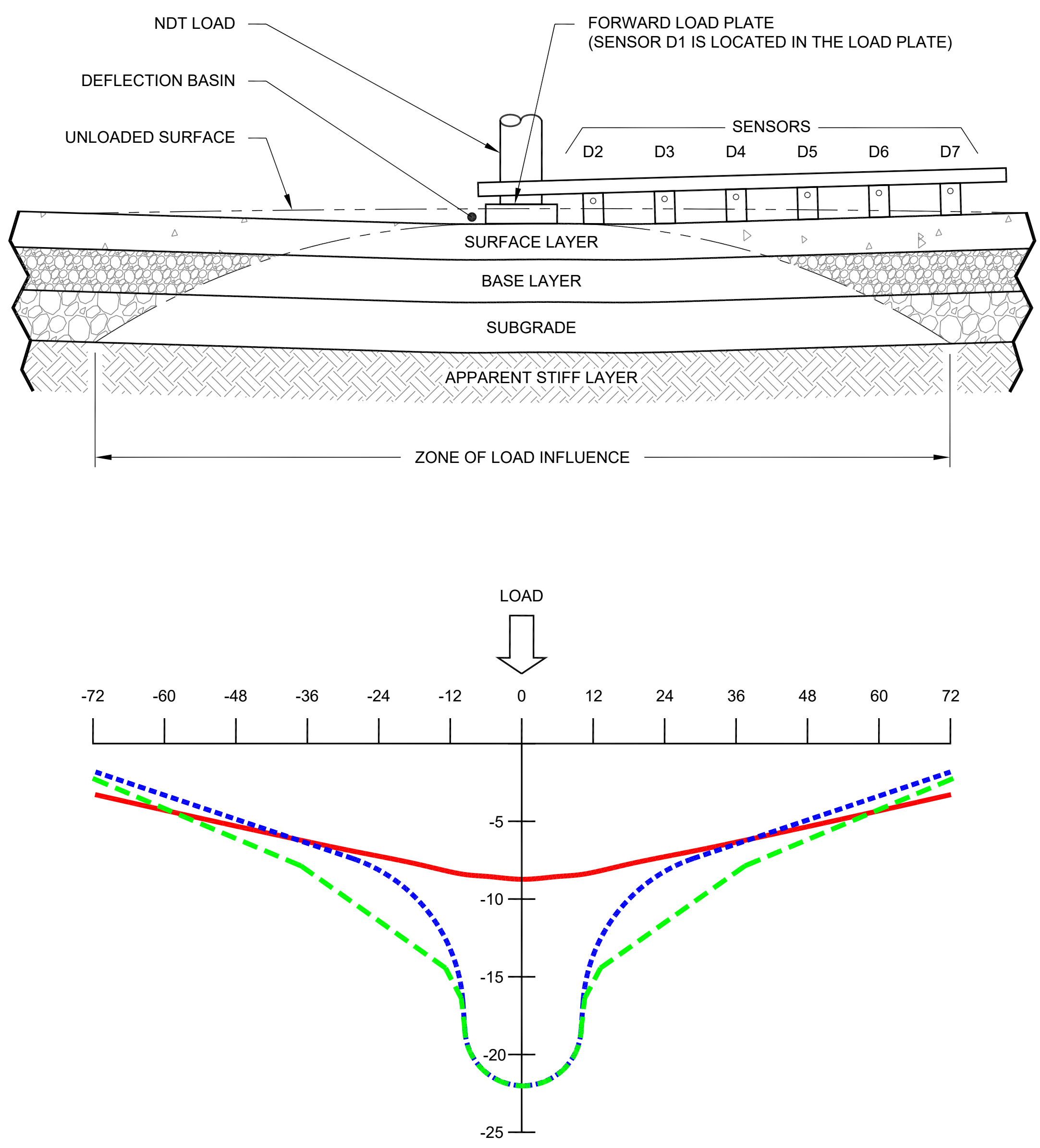
ISM	=	Impulse Stiffness Modulus, kips/in
L	=	Applied Load, kips
d_o	=	Maximum Deflection of Load Plate, in

C.11 Deflection Basin.

C.11.1 After the load is applied to the pavement surface, the deflection sensors measure the deflection basin. Figure C-2 is a schematic showing the zone of load influence created by a HWD and the relative location of the sensors that measure the deflection basin. The deflection basin can then be used to backcalculate the individual pavement material layer properties.

C.11.2 The response of the pavement to the applied load creates the shape of the deflection basin based on the thickness, stiffness, and material type of all the individual layers. The pavement deflection should be the largest directly beneath the load and then decrease as the distance from the load increases. Generally, a softer pavement will deflect more than a stiffer pavement under the same applied load.

Figure C-2. 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.11.3 To illustrate the importance of measuring the deflection basin, Figure C-2, also shows a comparison of three pavements. Pavement 1 is concrete and pavements 2 and 3 are asphalt. As expected, the rigid concrete pavement distributes the applied load over a larger area and has a smaller maximum deflection than flexible pavements 2 and 3. Although flexible 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 stiffness.

C.11.4 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 asphalt layer during testing, moisture contents in each of the layers, and concrete 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.12 Process Raw Deflection Data.

C.12.1 The boundary limits of pavement sections within a facility are defined in an airport pavement management program (PMP). In a PMP, a section is defined as an area of pavement that is expected to perform uniformly with similar aircraft traffic levels, pavement age, condition, or pavement cross-section. Deflection data can be used to define or refine the limits of all sections within a pavement facility.

C.12.2 A raw deflection data file may contain several types of deflection data, such as center, slab joint, and slab corner tests. The deflection data must 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.12.3 Raw data deflections may be normalized by adjusting measured deflections to an airplane standard load or the critical aircraft in the fleet mix.

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

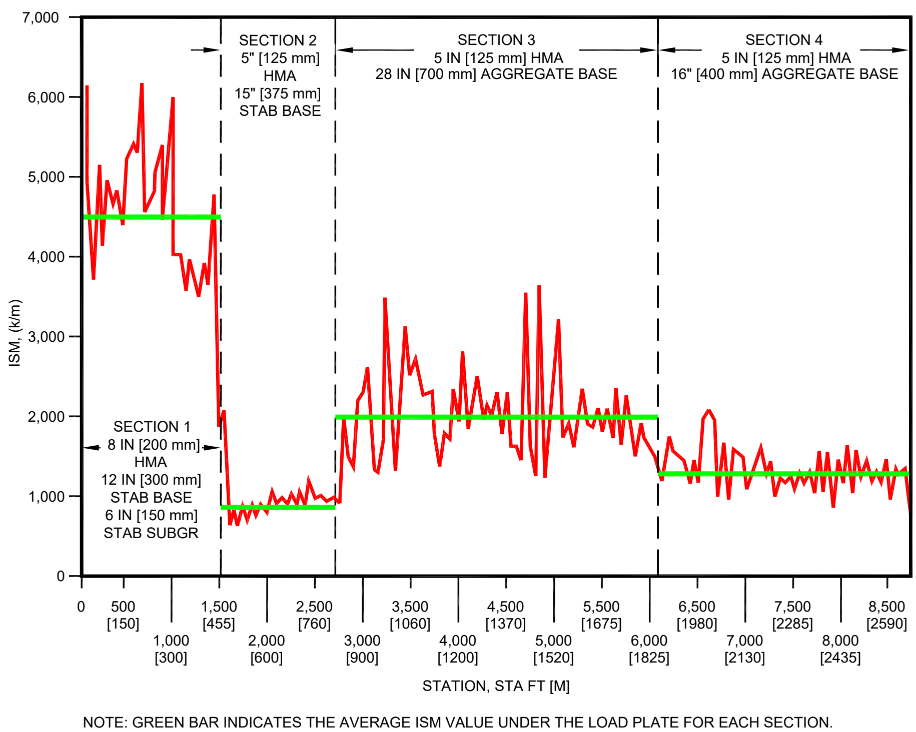
$L_{applied}$ = Applied load

d_0 = Measured deflection at selected sensor location

C.12.4 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

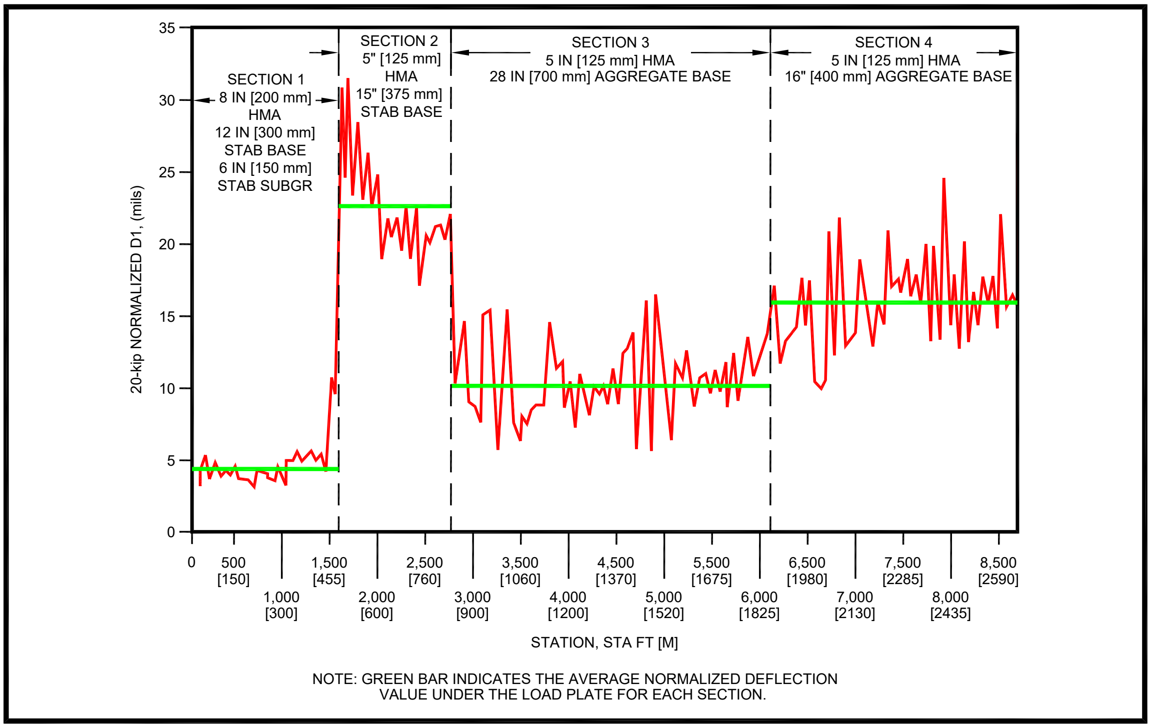
3693 ISM values or normalized deflections under the load plate provide an indication of the
3694 overall stiffness of the entire pavement structure (i.e., pavement layers and subgrade) at
3695 each test location. For a given impulse load (i.e., 40 kips (180 kN)), increasing ISM
3696 values or decreasing normalized deflections indicate increasing pavement stiffness.
3697 Example profile plots of ISM and normalized deflects are as illustrated in Figure C-2
3698 and Figure C-3, respectively.

3699 **Figure C-3. ISM Plot Identifying Pavement Section Limits**



3700

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



C.12.5 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.12.6 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 stiffness.

C.12.7 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.12.8 Deflection data can also be used to identify variations in subgrade stiffness 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 stiffness. 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 stiffness (for example, outside of the stress bulb at the subgrade-pavement interface).

C.12.9 Using the deflection test data separated by pavement sections and test type, the following may be determined; pavement layer stiffness 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.13 Software Tools.

Backcalculation 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 backcalculation methods should be consistent with the forward computational procedure that is used for structural evaluation and design. FAA software tools such as FAARFIELD and BAKFAA, are available at https://www.faa.gov/airports/engineering/design_software/.

C.14 Backcalculation Analysis.

C.14.1 The engineer can use deflection basin data from flexible pavements and rigid center tests to compute the stiffness of pavement layers. The process used to conduct this analysis is referred to as backcalculation because the engineer normally does the opposite of traditional pavement design. Rather than determining the thickness of each pavement layer based on assumed layer stiffness, backcalculation typically involves solving for pavement layer stiffness based on assumed uniform layer thicknesses. Throughout the remainder of this section, layer stiffness is referred to in terms of Young's modulus or simply the elastic modulus (E).

C.14.2 Backcalculation 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.14.3 Before conducting an analysis, the engineer should review the deflection tests that have been separated by pavement facility and section for backcalculation. Regardless of the analysis software tool, 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.14.4 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 concrete slab, or several other reasons. The engineer should review the deflection data and remove data that have the following anomalies.

C.14.4.1 **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 backcalculation analysis.

C.14.4.2 **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.

C.14.4.3 **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.14.5 For rigid pavement analysis, asphalt overlays are considered to be thin if they are less than 4 inches (10 cm) thick and the concrete layer thickness is less than 10 inches (25 cm). The asphalt overlay is also considered to be thin if it is less than 6 inches (15 cm) thick and the concrete layer is greater than 10 inches (25 cm) thick.

C.14.6 If the rigid pavement structure does not contain a stabilized base, asphalt overlay, or concrete overlay, the backcalculated dynamic effective modulus is the rigid pavement modulus of elasticity (E). The backcalculated dynamic k-value will need to be adjusted to obtain a static k-value for use in conventional FAA evaluation and design programs that use a k-value.

C.14.7 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.14.8 If the rigid pavement structure contains a stabilized base, thin asphalt overlay, or concrete overlay, the backcalculated dynamic effective modulus may be used to compute two modulus values. Possible modulus scenarios are as follows: bonded or unbonded concrete overlay and rigid pavement layer, thin asphalt overlay and rigid pavement layer, concrete layer and lean concrete or cement-treated base, or rigid layer and asphalt stabilized base.

C.14.9 The results that are obtained through iterative backcalculation 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 backcalculation requires engineering judgment.

C.15 Rigid Pavement Considerations.

While it is important to know the stiffness 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.15.1 Joint Analysis.

C.15.1.1 The analysis of joints or cracks in rigid pavements is important because the amount of load that is transferred from one slab to the adjacent slab can significantly impact the structural capacity of the pavement.

C.15.1.2 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 surface layer.

C.15.1.3 Deflection load transfer efficiency (LTE_Δ) is most frequently defined as shown in Equation C-3. If the LTE_Δ is being calculated at a reflective crack in in the asphalt overlay of a rigid pavement, compression of the asphalt 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.15.1.4 Relate computed LTE_Δ values, 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 this AC assumes the amount of load transfer is sufficient to reduce the free edge flexural stress in a concrete pavement 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.15.2 PCC Void Analysis.

C.15.2.1 In addition to joint load transfer, another important characteristic of a rigid pavement is the slab support conditions. One of the assumptions made during rigid pavement backcalculation 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.15.2.2 A loss of support may exist because erosion may have occurred in the base, subbase, or subgrade; settlement beneath the rigid pavement layer; or due to temperature curling or moisture warping.

C.15.3 Concrete Pavement Durability Analysis.

C.15.3.1 The backcalculation analysis procedures assume that the concrete pavement layer is homogenous and the results are based on center slab deflections and the condition of the slab in the interior. Concrete 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 joints and at slab corners because moisture levels are the highest at these locations.

C.15.3.2 Surface conditions may not be a good indicator of the severity several inches below the surface and NDT deflection data may be very useful in assessing the severity of durability-related problems. This is especially true if a concrete pavement with durability problems has been overlaid with asphalt. Often, the severity of the durability distresses increases after an asphalt overlay has been constructed because more moisture is trapped at the interface of the asphalt and concrete.

C.15.3.3 The extent of the durability problem can be assessed by evaluating the ISM obtained from the center of the slab and comparing it to the ISM 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.15.3.4 An ISM_{ratio} greater than 3 may indicate that the pavement 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 pavement is probably in good condition. These ranges are based on the assumption that the durability at the interior is excellent. This assumption can be verified by reviewing the modulus values obtained from backcalculation analysis of the rigid pavement layer.

C.15.3.5 Use of the ISM_{ratio} for asphalt overlays of concrete pavements has the advantage of eliminating the "HMA compression" effect that occurs during NDT. Assuming that the HMA layer is the same thickness and that its condition (for example, stiffness and extent of shrinkage cracks) is relatively constant, there should be approximately the same amount of compression in the asphalt layer at the slab center, corner, and joint. The net effect is that the ISM_{ratio} will primarily reflect the durability of the concrete layer.

C.16 HWD Derived Evaluation and Design Inputs.

C.16.1 This section provides guidance on use of inputs developed from deflection data for structural evaluation and design in accordance with this AC and AC 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 backcalculation analyses.

C.16.2 For a more conservative evaluation or design approach, the FAA 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 (i.e., 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.16.3 Use of Backcalculated 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, make appropriate adjustments, either up or down for the material layer. Do not go above the upper limit for the material. If the material layer data falls below the lower value, adjust the layer type to reflect the lower value.

APPENDIX D. DYNAMIC CONE PENETROMETER (DCP)

D.1 Dynamic Cone Penetrometer (DCP)

The dynamic cone penetrometer (DCP) is a tool that measures the penetration rate of a cone to estimate the mechanical properties of compacted pavement materials or undisturbed subgrade soils at shallow depths. Operation of the DCP make it a useful tool for site investigations based on simplicity, portability, and relative low cost of equipment. If cores are taken through the pavement to verify the thickness of a flexible or rigid 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 stiffness. Refer to ASTM D 6951, *Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications,* for additional information.

D.1.1 The DCP consists of two or more 5/8 inch (16 mm) shafts connected for desired depth. The lower drive rod contains a pointed tip, which is driven into the pavement material or subgrade. A sliding 10.1-lb (4.6-kg) or 17.6-lb (8-kg) hammer contained on the upper rod drives the tip. The penetration of the drive rod into the material after each hammer drop is recorded. This value recorded is known as the DCP index measured in inches (mm) per blow. The DCP index is plotted versus depth to identify thicknesses and stiffness of the different pavement layers. The DCP index can be correlated to other material properties such as the California Bearing Ratio (CBR), soil stiffness, or even soil density if moisture content is known. Table D-1 shows basic DCP correlations. Figure D-1 and Figure D-2 show schematic of DCP and the DCP in use, respectively.

Table D-1. Basic DCP Correlations

Soil Classification	Correlation	Source
All soils, except CL soils CBR < 10 and CH	$CBR = 292 / DCP^{1.12}$, DCP mm/blow $CBR = 292/ (DCP\times25.4)^{1.12}$, DCP in/blow	ASTM D6951
CL soils with CBR <10	$CBR = 1/(0.017019\times DCP)^2$, DCP mm/blow $CBR=1/(0.0432283\times DCP)^2$, DCP in/blow	ASTM D6951
CH	$CBR = 1/(0.002871\times DCP)$, DCP mm/blow $CBR=1/(0.072923\times DCP)$, DCP in/blow	ASTM D6951
All cohesive soils	$Log(E) = -0.45\times Log(DCP) + 2.52$, DCP mm/blow	Boutet 2007
All granular soils	$Log(E)= -0.62\times Log(DCP) + 2.56$, DCP mm/blow	Boutet 2007

Figure D-1. Schematic of DCP (ASTM D6951-09

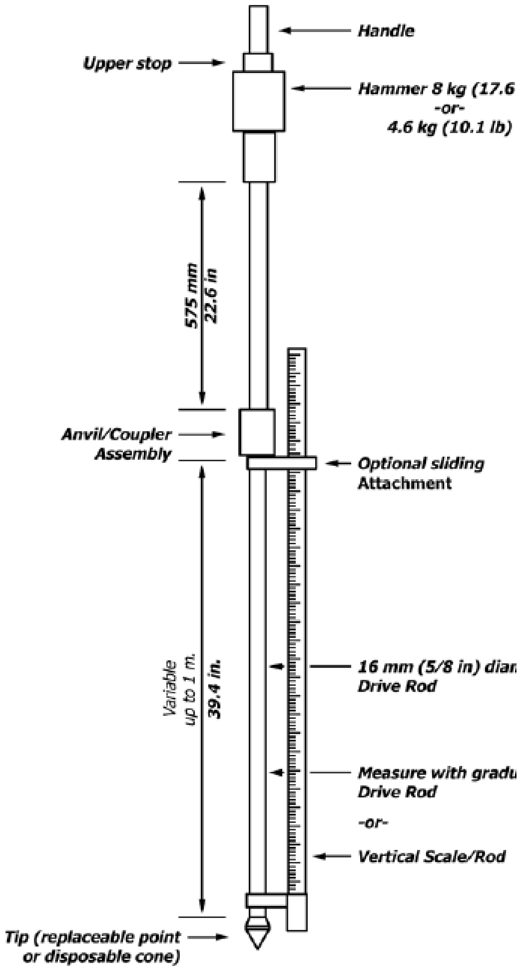


Figure D-2. DCP in Use (NAPTF)

APPENDIX E. GROUND PENETRATING RADAR

E.1 Ground Penetrating Radar (GPR).

Ground penetrating radar (GPR) measures portions of a beam of radar energy reflected as it strikes multiple interfaces between materials of different dielectric constants. This NDT uses electromagnetic radiation in the microwave band (UHF/VHF frequencies) of the radio spectrum. The electromagnetic wave pulse emitted into the pavement by an air-coupled or ground-coupled antenna. A second antenna records the reflected waves. The quality of the reflected signal is highly dependent on the sharpness of the contrast between adjacent layers or objects. The time between two echoes is a function of the distance traveled between two reflectors. Varying the frequency of the transmitted signal produces different results. High frequency waves will provide resolution at shallow depth, while low frequency waves will reach greater depths but with decreased resolution. GPR can be very effective in coarse-grained soils, ice, and frozen ground. GPR has limited effectiveness in fine-grained soils (silt or clay). The most common uses of GPR data include measuring pavement layer thicknesses, detecting the presence of excess water in a structure, locating underground utilities or rebar in concrete, 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. Figure E-1 and Figure E-2 show a vehicle based GPR and cart based GPR, respectively. Figure E-3 and Figure E-4 show a plot of GPR results for asphalt and concrete, respectively.

Figure E-1. Vehicle based Air-Coupled GPR (NAPTF)

Figure E-2. Cart based GPR (NAPTF)

Figure E-3. GPR Results for Asphalt (NAPTF)

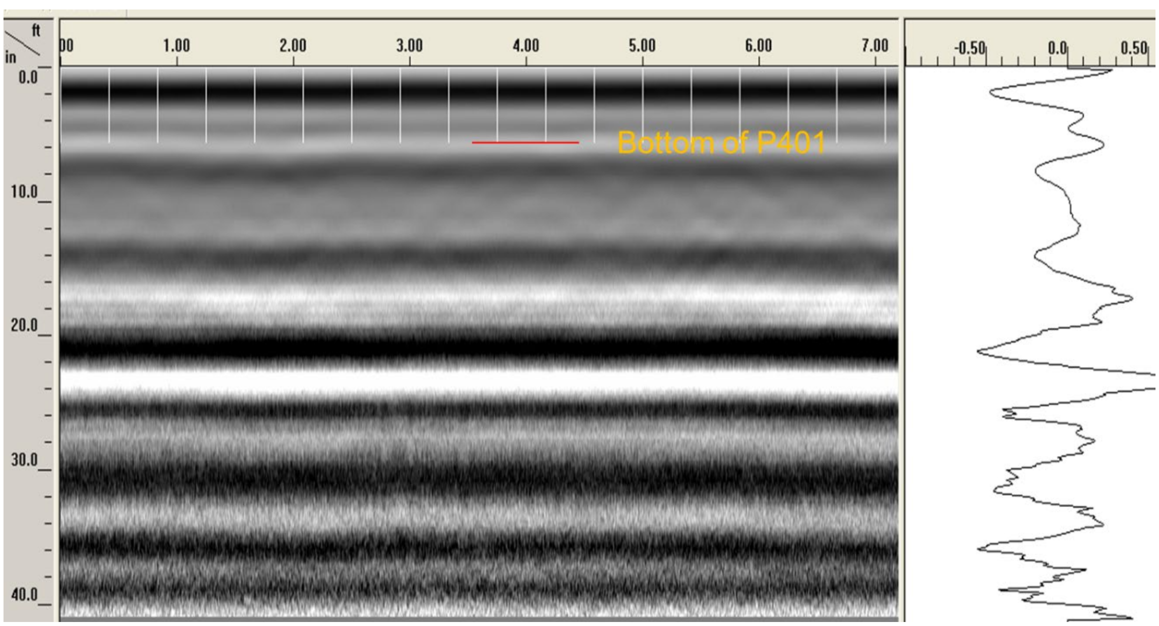
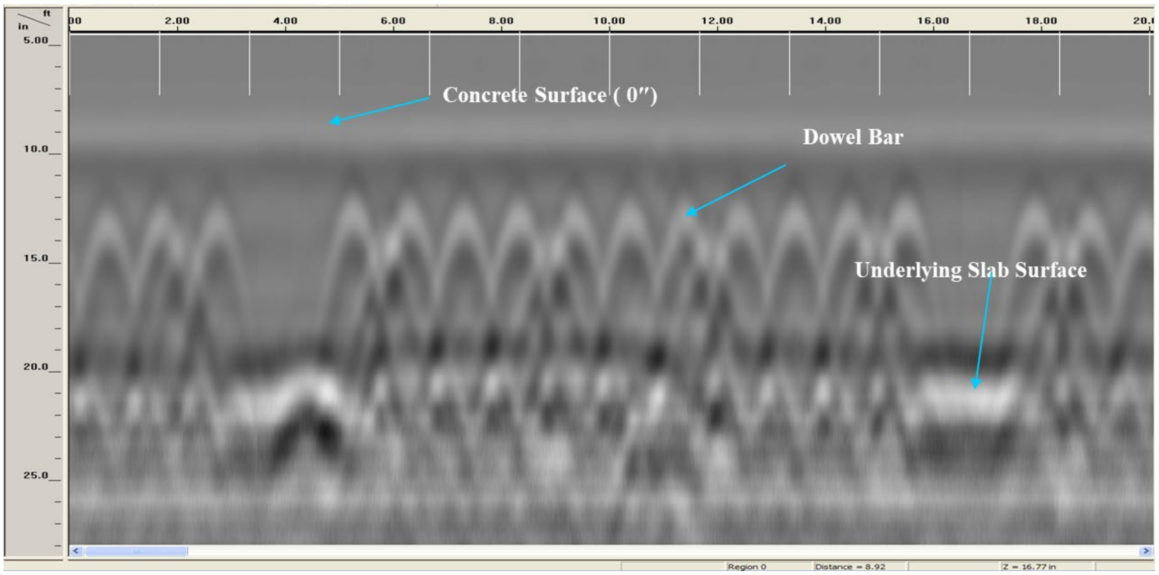


Figure E-4. GPR Results for Concrete (NAPTF)



Page Intentionally Blank

APPENDIX F. REINFORCED ISOLATION JOINT.

F.1 Reinforced Isolation Joint Description.

F.1.1 A reinforced isolation joint (Type A-1) can be used as an alternative to a thickened edge joint for PCC slabs that are greater than or equal to 9 inches, that occur where pavement centerlines intersect at approximately 90 degrees. When intersecting pavements are at acute angles which results in small irregularly shaped slabs on one side of the isolation joint it may not be possible to install the reinforcement steel.

F.1.2 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). Provide the amount of steel as supported by structural calculations.

F.1.3 Place an equal amount of steel reinforcement at the top of the slab to resist negative moments that may arise at the slab corners.

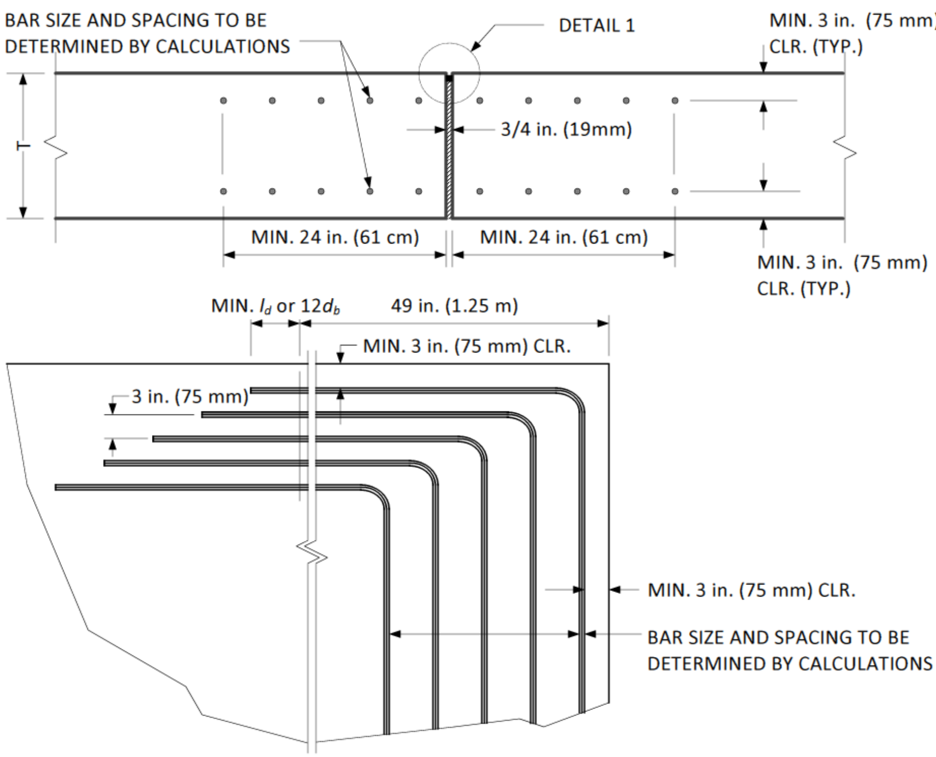
F.1.4 Any additional embedded steel used for crack control should conform to the requirements of paragraph 3.14.12.1.

F.1.5 Where a reinforced isolation joint intersects another joint, the reinforcing steel should not be terminated abruptly, nor should it continue through the intersecting joint.

F.1.6 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 F-1.

F.1.7 A minimum of 3 inches (75 mm) clear cover shall be maintained on all reinforcing bars.

Figure F-1. Type A-1 Joint Detail



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

F.2 Design Example Reinforced Isolation Joint (Type A-1).

F.2.1 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. On the Explorer tab, click “Design Options.” Set the “Output File” option to “Yes.” Close or hide the Design Options screen.
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 file *Output-Max Stress.txt* in the “Documents\FAARFIELD\PrintOut-” 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.)

F.2.2 For this design example, the maximum PCC horizontal edge stress from the output file *Output-Max Stress.txt* 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.

F.2.3 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 H-5. 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

$\rho =$ 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:

$\rho_b =$ the balanced steel ratio,

$\beta_1 =$ 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 G. USER-DEFINED VEHICLE IN FAARFIELD

FAARFIELD has an internal aircraft library containing most of the common aircraft in commercial service. Occasionally, it may be necessary to include aircraft in the traffic mix that do not appear in the internal library. FAARFIELD allows users to define and edit aircraft gears from the user interface. These user-defined vehicles are treated just like internal library aircraft in the design. However, they are identified by “(UD)” following the name.

G.1 Creating a User Defined Vehicle in FAARFIELD.

The following example shows how to create a user defined vehicle in FAARFIELD. Consider the flexible pavement design example shown in Figure G-1. To add to the current traffic mix, select Create New User Defined Vehicle from the menu bar at top of the screen.

Figure G-1. Select “Create New User Defined Vehicle”

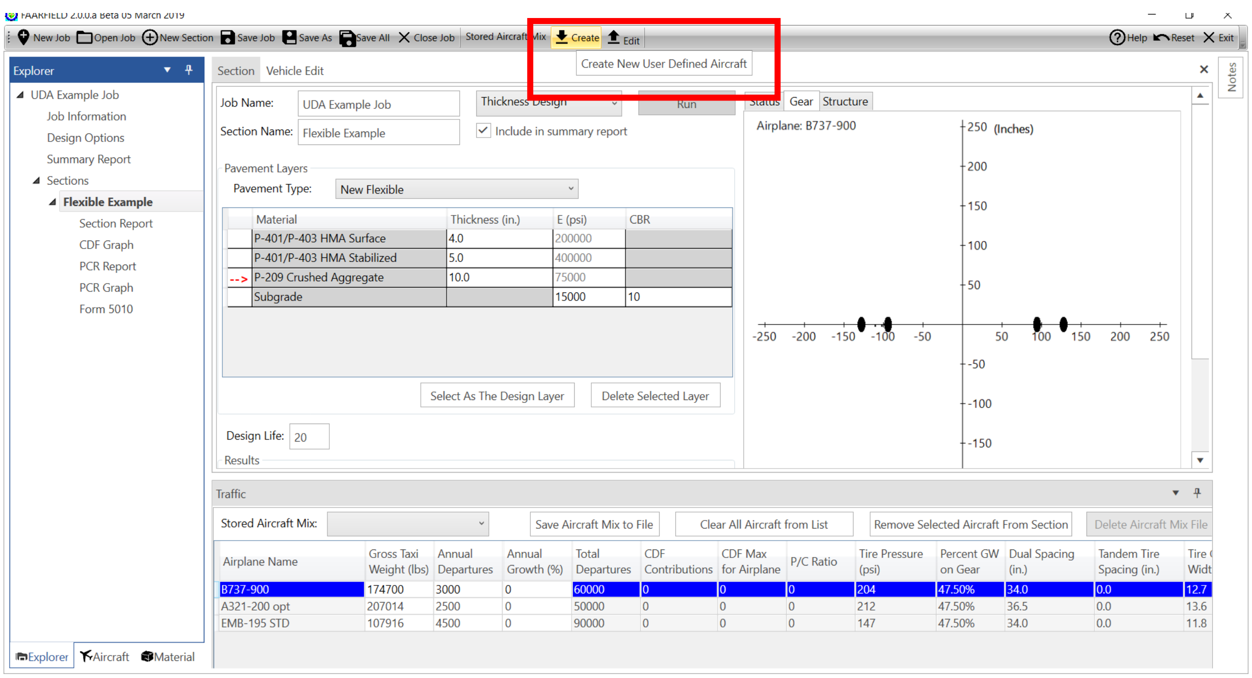


Figure G-2 shows the Vehicle Edit screen. Enter all the following data in the appropriate fields:

G.1.1 New User Defined Vehicle.

Enter a name

G.1.2 Gross Taxi Weight.

Enter the gross weight of the vehicle.

G.1.3 Percent Gross Weight on Whole Main Gear.

Enter the value as a decimal number between 0 and 1.0. In most cases, the value 0.95 is assumed for thickness design.

G.1.4 PCR Percent Gross Weight on Gear.

Enter the value of percent gross weight on the main gear to be used for ACR-PCR computations, as a decimal number between 0 and 1.0. This value, which is usually less than 0.95, may be obtained from the Aircraft Characteristics for Airport Planning manual published by the aircraft manufacturer. If the information is unknown or unavailable, enter 0.95 in this field.

G.1.5 Tire Coordinates.

Enter the horizontal coordinates of the tires in one main gear truck. The transverse (X) coordinate is defined with reference to the aircraft centerline. The tires will be reflected automatically on the other side of the aircraft longitudinal axis. The longitudinal (Y) coordinate origin is arbitrary but is typically at the center of the gear. It is not necessary to enter the dual tire spacing, tandem spacing or track spacing separately.

G.1.6 Evaluation Points.

Evaluation points define the horizontal locations where FAARFIELD evaluates the layered elastic response. It is necessary to define at least one evaluation point, but there is no upper limit. Typically, evaluation points are distributed on a point locus capturing the maximum subgrade strain for a particular gear geometry. This is necessary because the location of maximum strain can change from directly under the center of the tire for thin pavements to directly under the center of the gear for very thick pavements. For S, D and 3D gear types, the locus is relatively simple due to symmetry of the wheels. For 2D gears, the locus is more complex. The FAARFIELD internal library uses a bilinear locus as shown in Figure G-3, where the diagonal leg is defined by:

$$A/T = 0.561(D/T) - 0.264$$

Where:

A = distance to inflection point

T = tandem wheel spacing

D = dual wheel spacing

The example in Fig. G-2 shows evaluation points distributed on the above locus, with 6 points distributed on the diagonal leg, and three points distributed on the longitudinal leg. (One point is common to both legs, for a total of eight evaluation points.) It is only necessary to enter evaluation points for one gear, as shown in the example.

Figure G-2. Vehicle Edit Screen

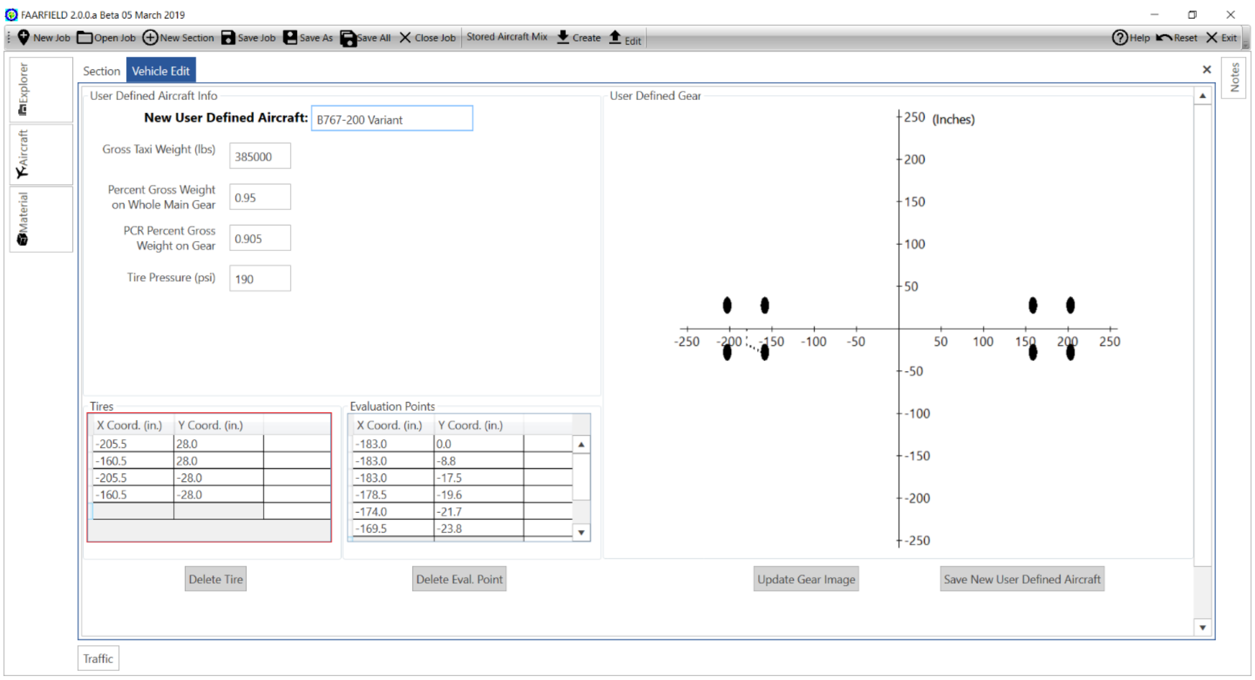
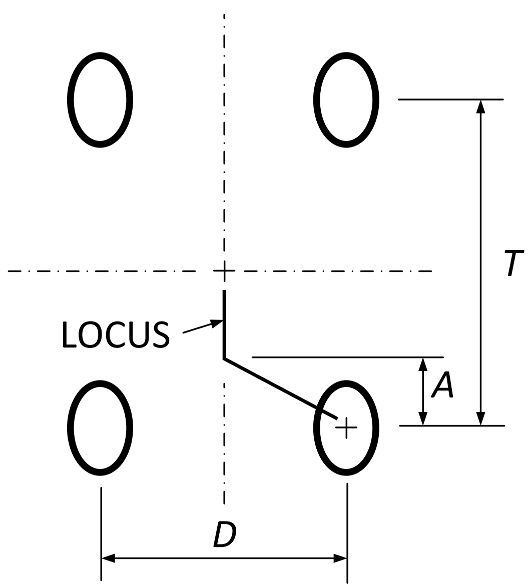
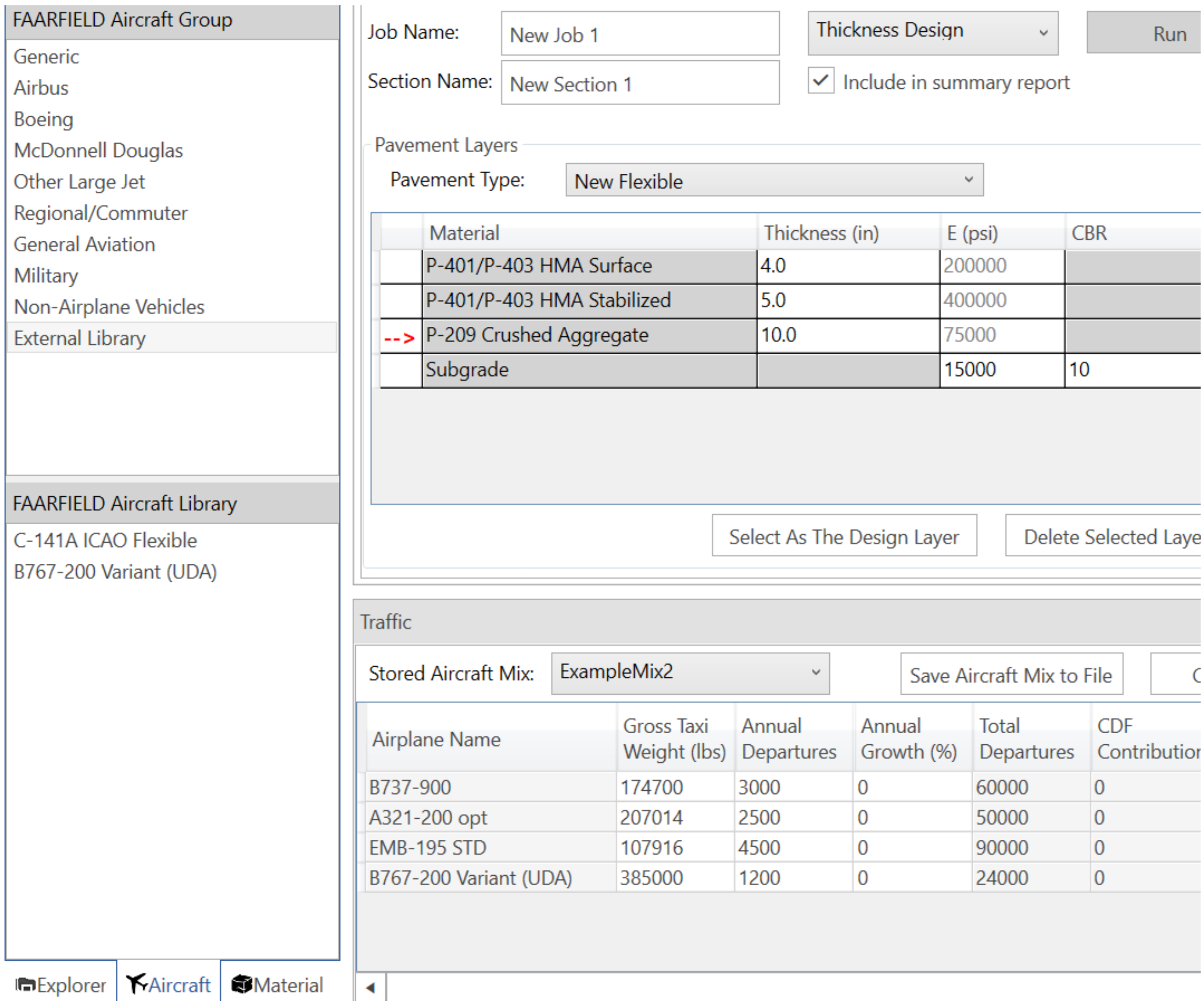


Figure G-3. Evaluation Point Locus for 2D Gear



As wheel and evaluation point coordinates are entered, the gear image on the right side of the screen will update automatically. Once all data have been entered, click "Save New User Defined Aircraft." The created UDA now appears in the FAARFIELD aircraft library under the "External Library" group and can be added to the aircraft mix (Figure G-4). The suffix "(UD)" indicates that the aircraft is user-defined.

Figure G-4. FAARFIELD Aircraft Library (External Library Group)



G.2 Editing a User Defined Vehicle in FAARFIELD.

To edit an existing user defined vehicle in the FAARFIELD external library, select "Edit User Defined Vehicle" from the menu bar at the top of the screen. This will bring up the Vehicle Edit screen. Select the vehicle to be edited from the drop-down list. Make any changes to the information on the screen, and to save changes, click "Update User Defined Vehicle".

Figure G-5. Select "Edit New User Defined Aircraft"

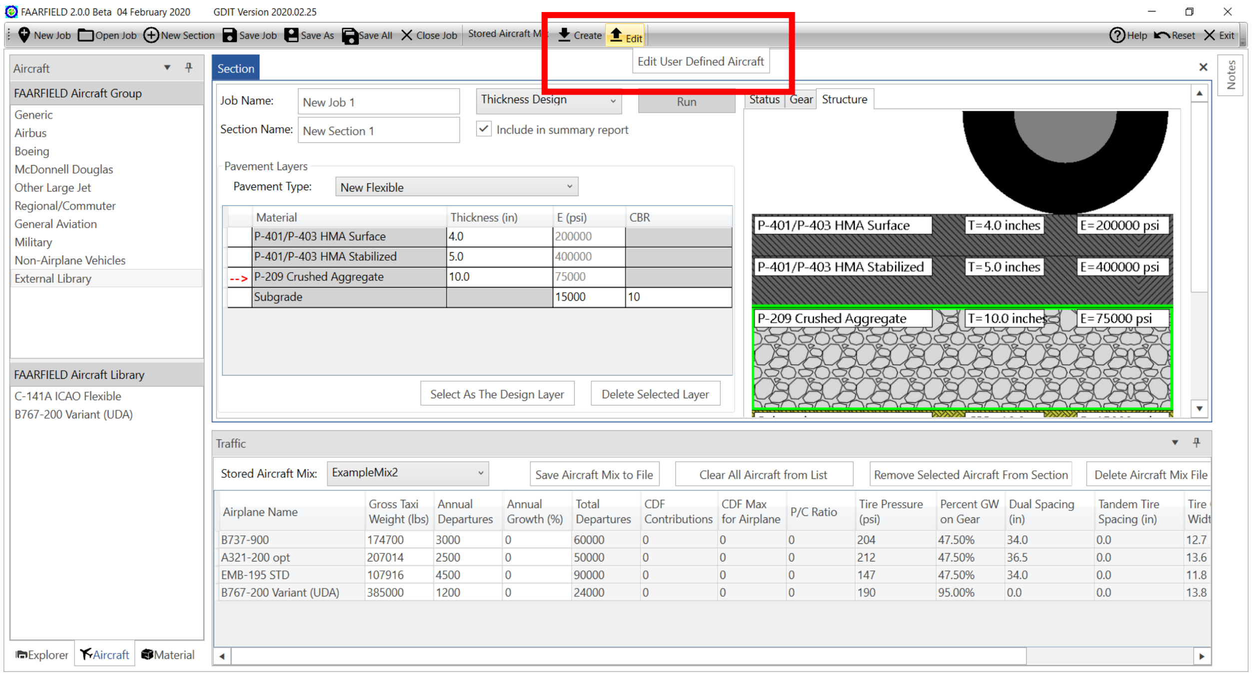
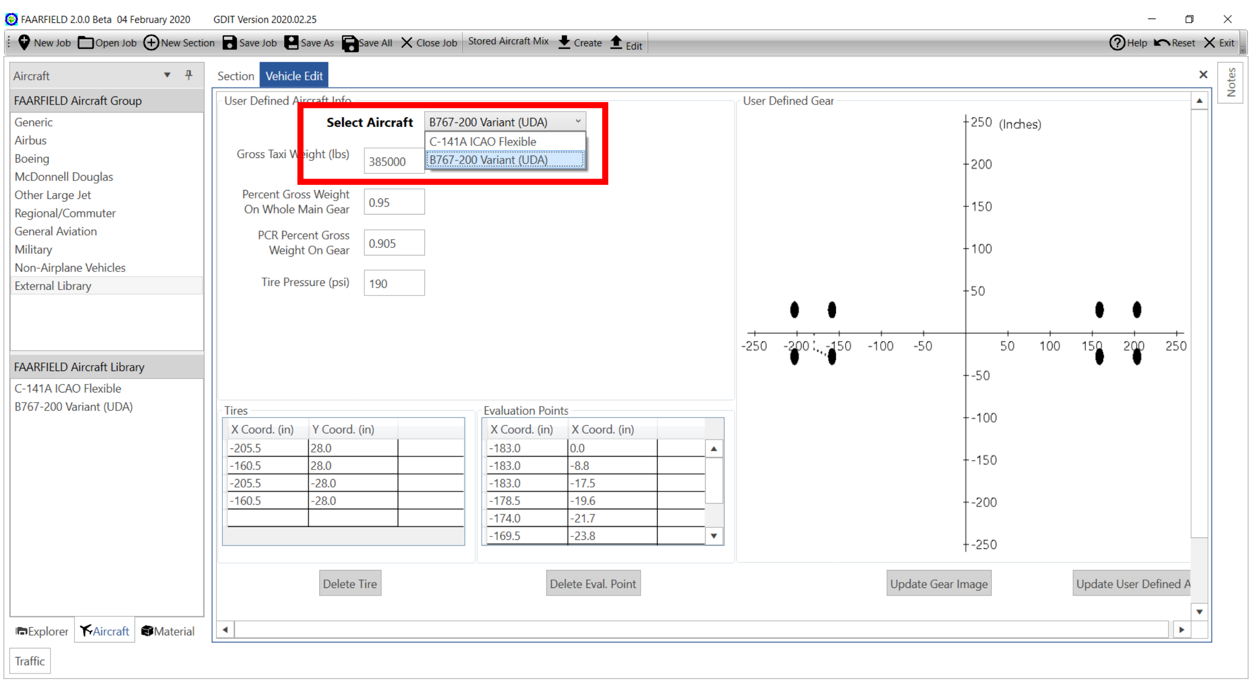


Figure G-6. Select UDA for Editing from Drop-Down List



G.3 UDA Data Files.

FAARFIELD automatically saves UDA data to files with an *.XML extension. A separate file is created for each UDA in the external library. Files are saved to the user's hard drive in the directory C:\Users\[user name]\Documents\My FAARFIELD\User Defined Aircraft. In addition, when a job is created that has UDAs in the traffic mix, the UDA data are stored in the job file. This allows FAARFIELD to open and run a job containing one or more UDAs even if the UDAs do not exist in the local external library.

APPENDIX H. FAARFIELD EXAMPLES

H.1 Example CDF Concept.

H.1.1 The following example illustrates the concept.
Given the following pavement structure:

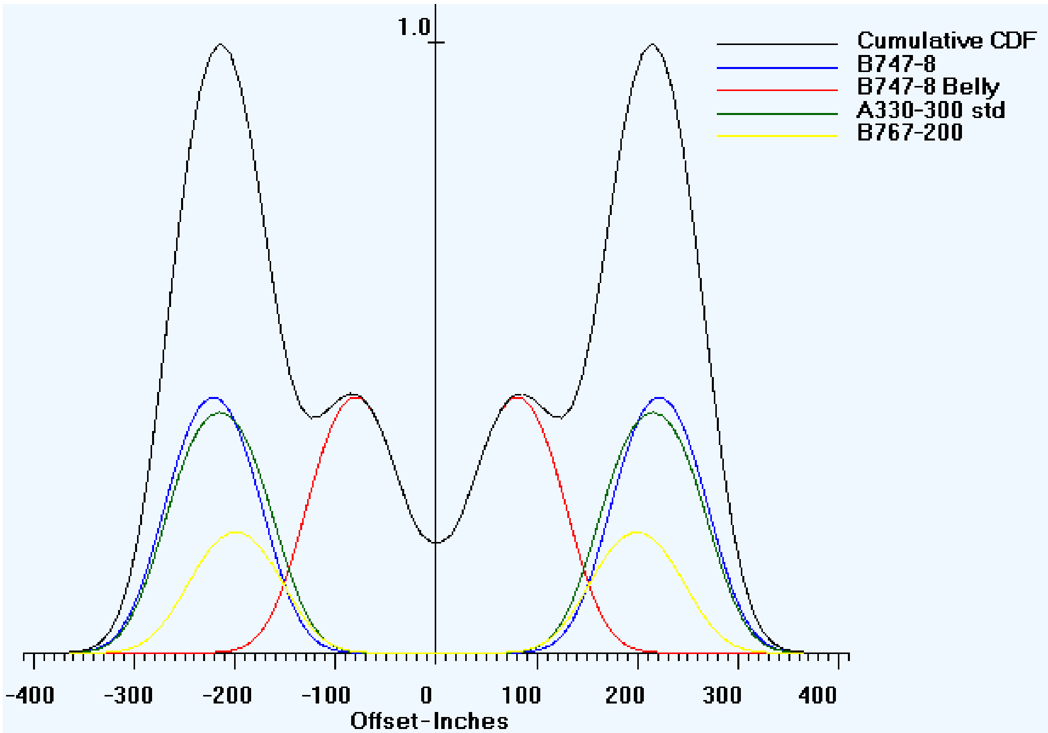
Thickness	Pavement Structure
4 inches	P-401 Asphalt Surface Course
8 inches	P-403 Stabilized Base Course
12 inches	P-209 Crushed Aggregate Base Course
13 inches	P-154 Aggregate Base Course
	Subgrade CBR 5 (7,500 psi Modulus)

Designed for the following airplane traffic:

Airplane	Gross Weight (lbs)	Annual Departures
B747-8	990,000	50
A330-300 std	509,000	500
B767-200	361,000	3000

H.1.2 To view the graph after the design is complete, select CDF Graph from the explorer on the left side of the screen. This action will display a graph depicting the contribution of each aircraft, as well as the combined CDF, as a function of lateral distance (offset) from the centerline. In the example shown in Figure H-1, the critical offset for CDF is located between the main gears for the evaluation aircraft. In this example, the B747 belly gear has a **large** individual CDF, but does not contribute to CDF at the critical offset.

Figure H-1. CDF Contribution for Airplane Mix



H.2 Example Flexible Pavement Design.

H.2.1 Flexible Design Example.

The design of a pavement structure is an iterative process in FAARFIELD. The user enters the pavement structure and airplane traffic for 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 After opening FAARFIELD, begin by selecting pavement type "New Flexible" from the drop-down list. The program displays the screen shown in Figure H-2.

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

Thickness	Pavement Structure
4 inches	P-401 Asphalt Surface Course
5 inches	P-401/P-403 Stabilized Base Course
6 inches	P-209 Crushed Aggregate Base Course
12 inches	P-154 Aggregate Base Course
	Subgrade, CBR=5 (E = 7500 psi)

Modify the default structure in Figure H-2 to match the above values. This example requires the following modifications:

1. Add a new layer under P-209 by clicking on the "P-209 Crushed Aggregate" label. Then in the dialog box select "P-154" and "Add layer below."
2. Click on the layer thickness of P-209 and enter 6 inches in the dialog box. Click OK.
3. Click on the layer thickness of P-154 and enter 12 inches in the dialog box. Click OK.
4. Click on the CBR label and enter 5 in the dialog box. Click OK.

The program now displays the screen shown in Figure H-3.

Step 3 For this example, assume the following 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

Enter the design traffic. Airplanes are selected from the airplane library at the left of the screen. (Display the aircraft library by selecting the "Aircraft" tab. Selected aircraft will appear in the Traffic list at the bottom of the screen. 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. The program displays the airplane list on the screen shown in Figure H-4.

Figure H-2. Flexible Design Example Step 1 (Select Pavement Type)

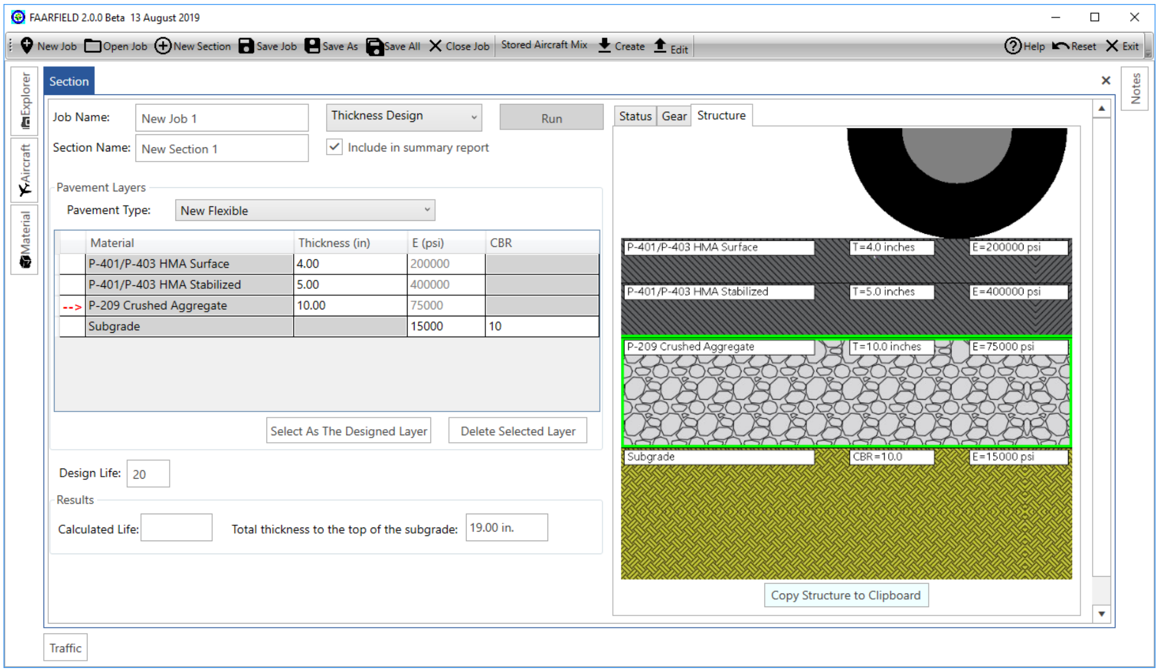


Figure H-3. Flexible Design Example Step 2 (Structure)

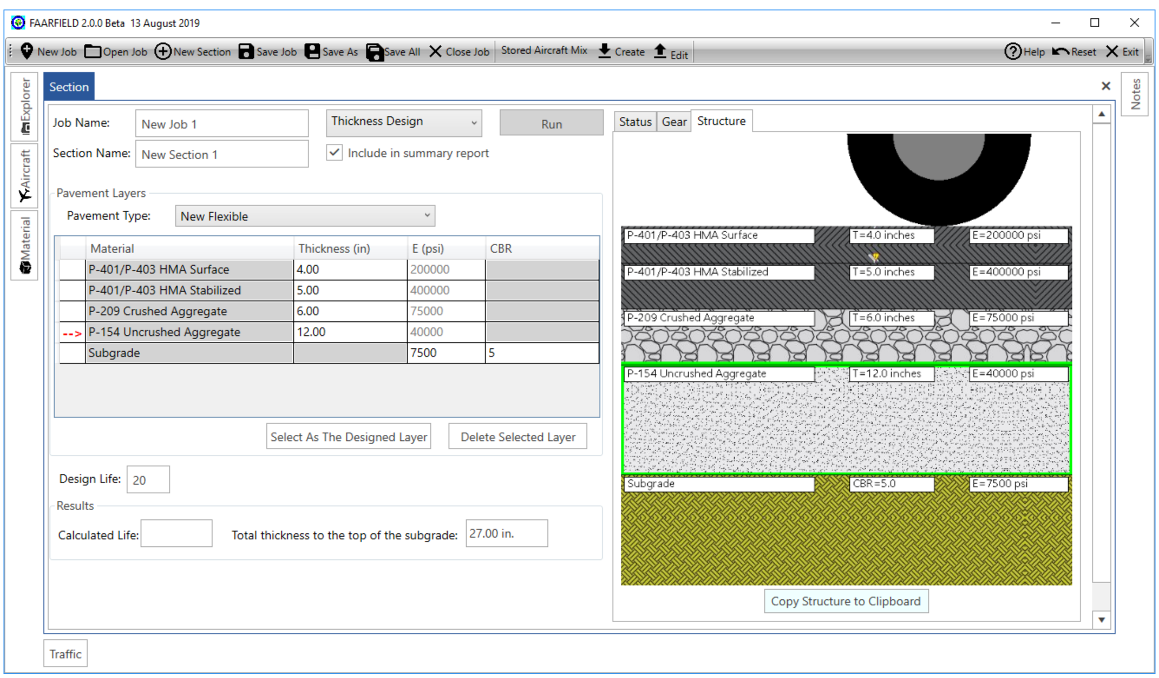


Figure H-4. Flexible Design Example Step 3 (Traffic)

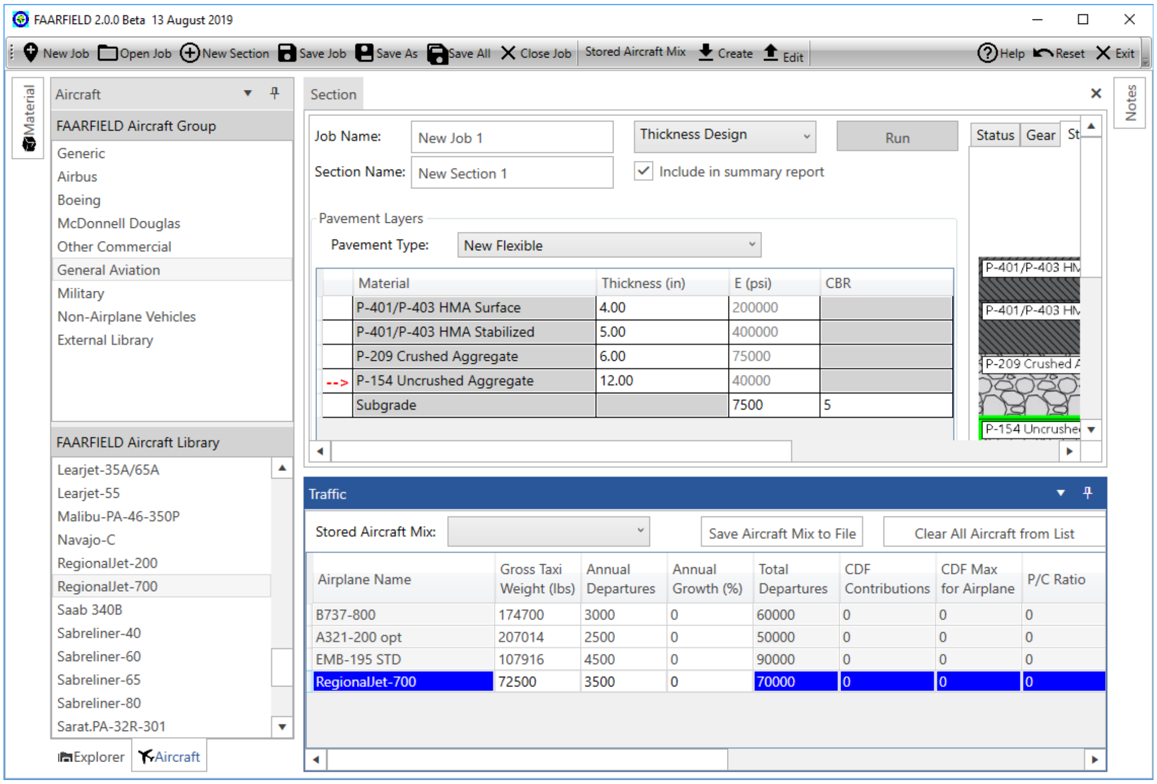
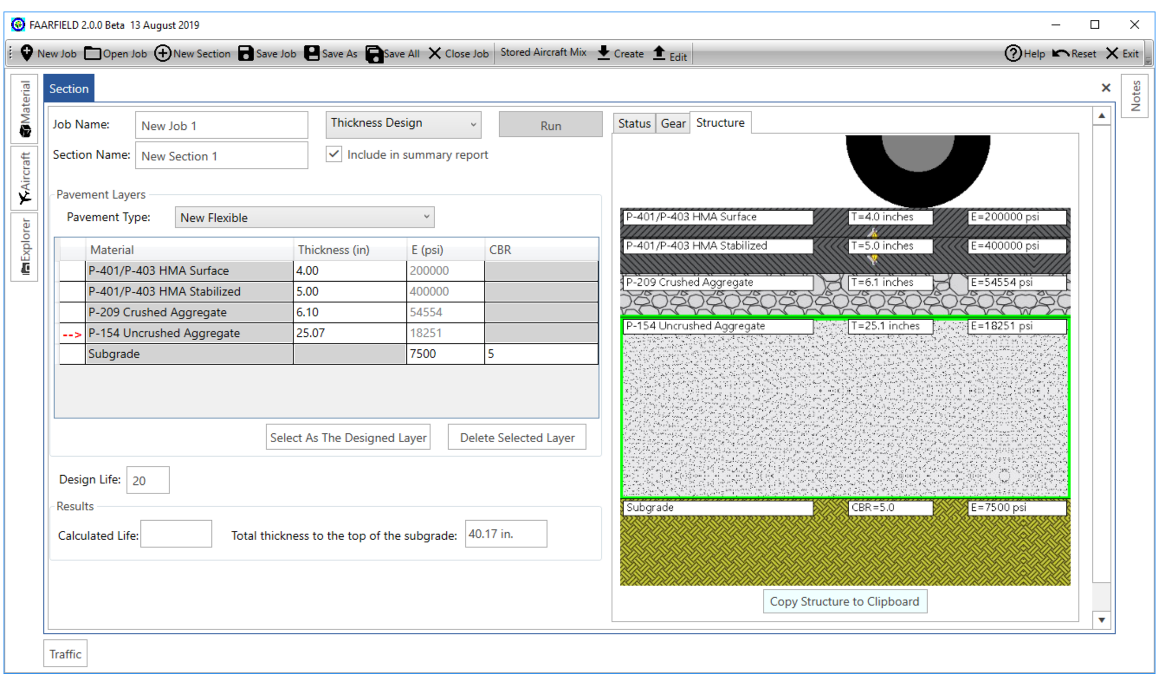


Figure H-5. Flexible Design Example Step 4 (Thickness Design)



Step 4 Click "Run" button to execute the thickness design. During the design process, FAARFIELD checks the P-209 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 red arrow at the left of the table. The results of the design are shown in Figure H-5.

Figure H-6. Flexible Design Example Step 5 (Settings for Final Design)

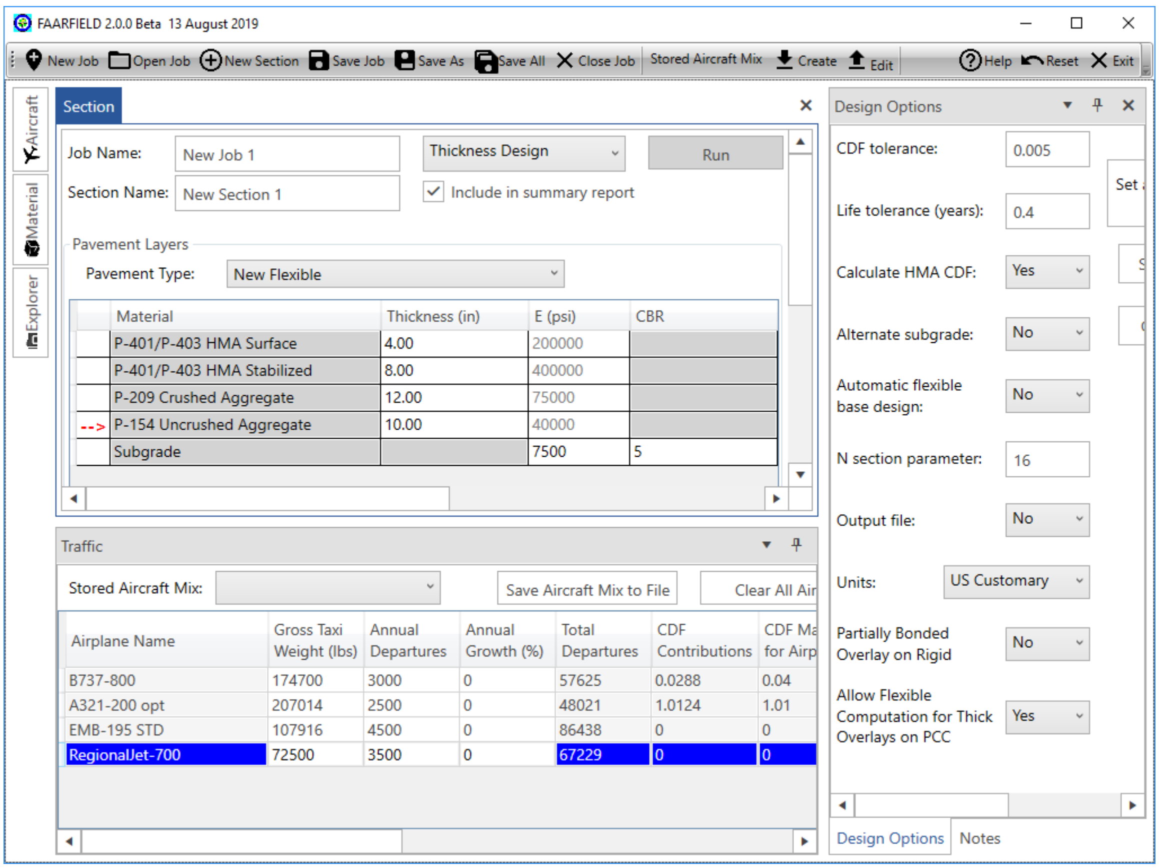
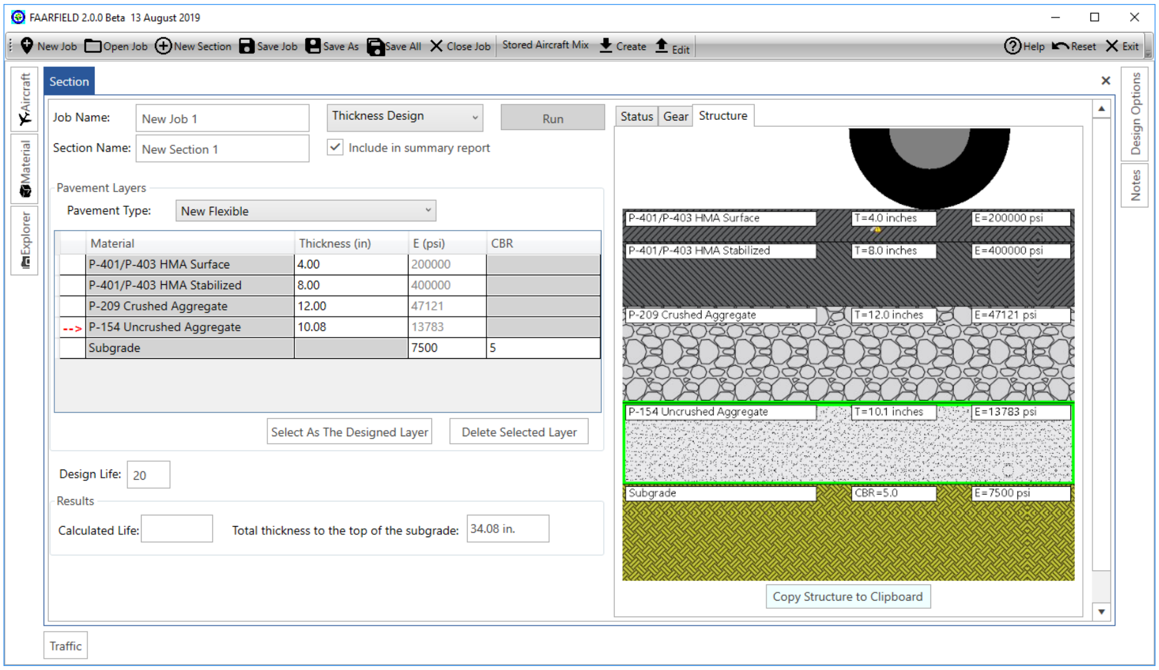


Figure H-7. Flexible Design Example Step 5 (Final Thickness Design)



Step 5 As indicated in Step 4, FAARFIELD automatically computes a minimum required thickness for the P-209 base layer. For practical reasons, the design base layer thicknesses will be higher than the minimum. To design the final (adjusted) structure:

1. Turn off automatic base design by selecting "No" for "Automatic Flexible Base Design' under FAARFIELD options. The Design Options box is at the right of the screen as shown in Figure H-6.
2. As stated in paragraph 3.13.6, it is good practice to perform a check for fatigue cracking in the final design. Select "Yes" for "Calculate HMA CDF" under FAARFIELD options.
3. Due to availability of material, performance of existing sections at airport, constructability issues (e.g. limiting number of different materials), frost protection requirements, it is often appropriate to adjust the pavement structure. There is no 'one' correct' solution to pavement structural design, there are many acceptable solutions. Provide justification of the final section chosen in the engineer's report.
4. Adjust the layers (surface, stabilized base and base) to reflect the final thickness to be constructed. For this example, assume the following pavement structure, which meets minimum layer thickness requirements: 4 inches P-401, 8 inches P-403, 12 inches P-209 and 10 inches P-154.
5. Click "Run" to perform the final thickness design. The results of the final design are shown in Figure H-7. The design

indicates 10.1 inches P-154 subbase, which will be rounded to 10 inches.

Figure H-8. Flexible Design Example Step 6 (Section Report)

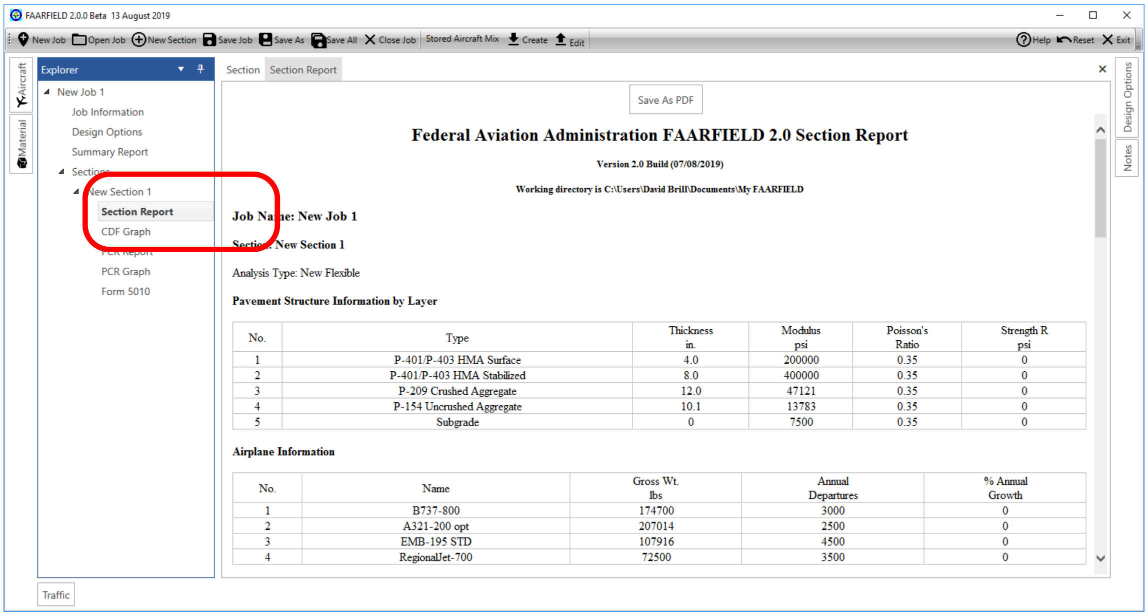
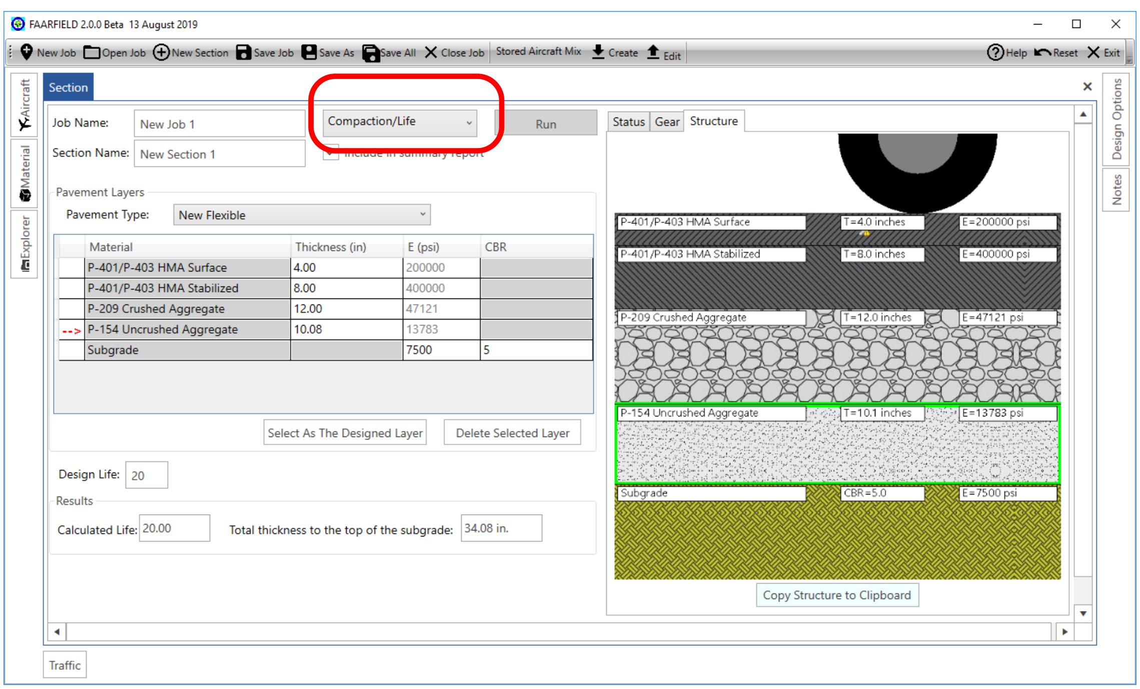


Figure H-9. Flexible Design Example Step 7 (Compaction/Life Evaluation)



Step 6 After the design is completed, the section report can be viewed by selecting "Section Report" in the explorer (Figure H-8). Save the section report to pdf format by clicking "Save as PDF" at the top of the screen. A Summary Report for all sections in the job is also available.

Step 7 FAARFIELD includes the ability to evaluate the depth of subgrade compaction required. After completing the thickness design, select "Compaction/Life" from the drop-down list at the top of the home screen (Figure H-9). After running "Compaction/Life," FAARFIELD adds two tables to the section report, containing subgrade compaction requirements for non-cohesive and cohesive soils, respectively. (Note: The compaction function will not be available if the design has not been completed.) Paragraph H.8 gives a detailed example of the compaction requirements computation in FAARFIELD. See paragraph 3.9 for additional discussion regarding subgrade compaction.

H.3 Example Rigid Pavement Design

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

Step 1 After opening FAARFIELD, begin by selecting pavement type "New Rigid" from the drop-down list. The program displays the screen shown in Figure H-11.

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

Pavement structure:

Thickness	Pavement Structure
14 inches (thickness to be determined by FAARFIELD)	P-501 Concrete Surface Course (*R* = 600 psi)
5 inches	P-401/P-403 Stabilized Base Course
12 inches	P-209 Crushed Aggregate Base Course
	Subgrade, *k*=100 pci (E = 7452 psi)

Modify the default structure in Figure H-11 to match the above values. The starting thickness for PCC is not critical, as FAARFIELD will generate a new approximate starting thickness based on layered elastic analysis before proceeding to finite element design. This example requires the following modifications:

1. In the structure image, click on the layer thickness of P-209 and enter 12 inches in the dialog box. Click OK. (Alternatively, enter 12.0 directly in the third line of the grid on the left.)
2. In the structure image, click on the subgrade layer *k*-value label and enter 100 in the dialog box. Click OK. (Alternatively, enter 100.0 pci directly in the last line of the grid on the left.)
3. In the structure image, click on the *R*-value label and enter 600 in the dialog box. Click OK. (Alternatively, enter 600 psi directly in the first line of the grid on the left.)

The program now displays the screen shown in Figure H-12.

Step 3 Enter the design airplane traffic. For this example, assume the following 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

Airplanes are selected from the airplane library at the left of the screen. Display the aircraft library by selecting the “Aircraft” tab. 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 on the airplane manufacturer. In addition there is a group of generic airplanes based on size and type of landing gear. In many cases, airplane models not in the airplane library can be represented adequately by a generic airplane. The program displays the airplane list on the screen shown in Figure H-13.

Step 4 Click the “Run” button to execute the thickness design. FAARFIELD iterates on the thickness of the concrete surface layer until a CDF of 1.0 is reached. FAARFIELD does not design the thickness of pavement layers other than the concrete 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. Under the "Status" tab, a clock displays the design progress. In this example, FAARFIELD gives a thickness of 17.14 inches (43 cm). The results of the completed design are shown in Figure H-14. For construction, the concrete layer design thickness in this example should be rounded to the nearest 0.5 inch (12.5 mm), or to 17.0 inches (425 mm).

Step 5 After the design is completed, the section report can be viewed by selecting "Section Report" in the explorer (Figure H-15). Save the section report to pdf format by clicking "Save as PDF" at the top of the screen. A Summary Report for all sections in the job is also available. For this example, the Section Report is reproduced in Figure H-18.

Step 6 To determine subgrade compaction requirements, select "Compaction/Life" from the drop-down menu and click "Run" (Figure H-16). Compaction requirements for the designed section will be displayed in the Section Report.

Figure H-10. Rigid Design Example Step 1

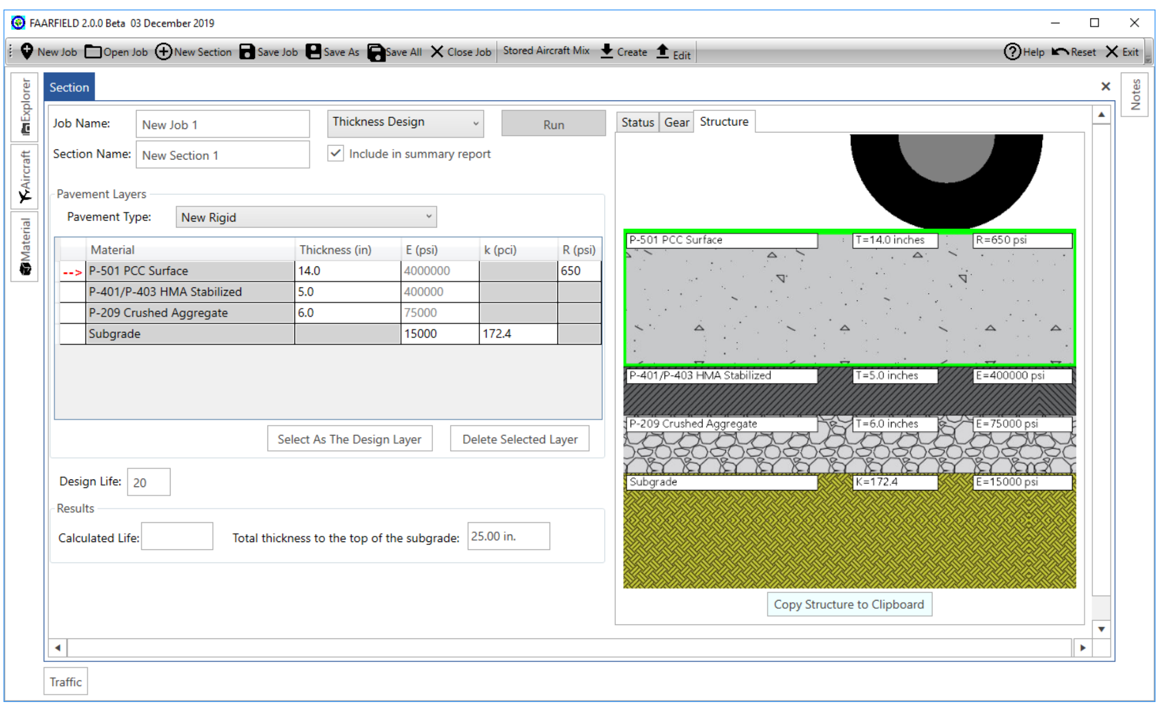


Figure H-11. Rigid Design Example Step 2 (Modify Structure Information)

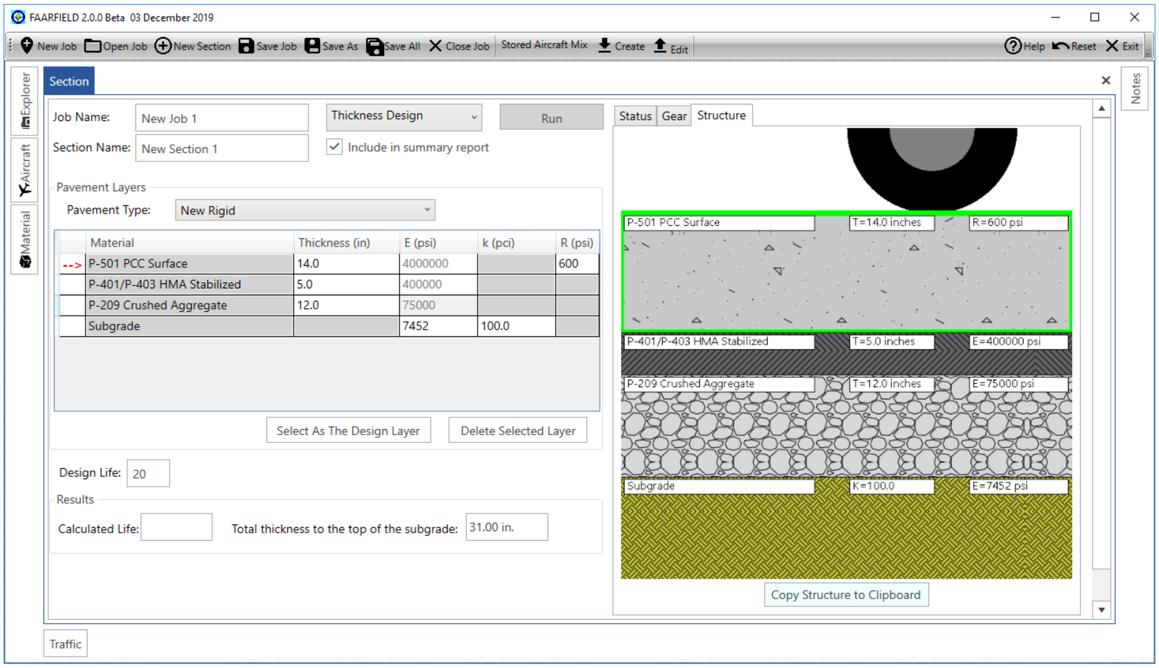


Figure H-12. Rigid Design Example Step 3 (Airplane Data)

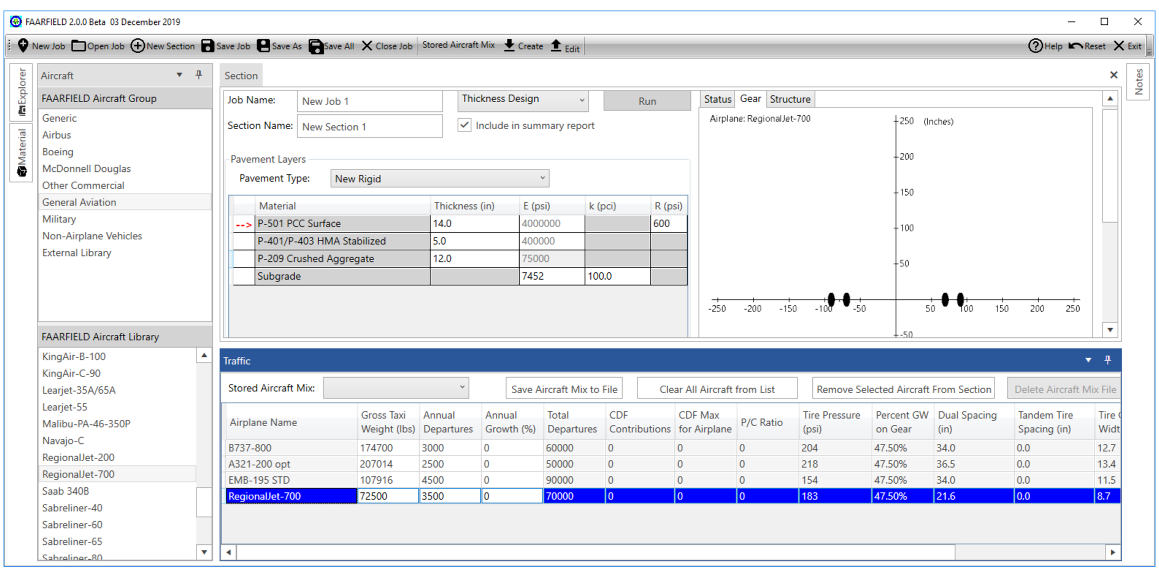


Figure H-13. Rigid Design Example Step 4 (Final Design)

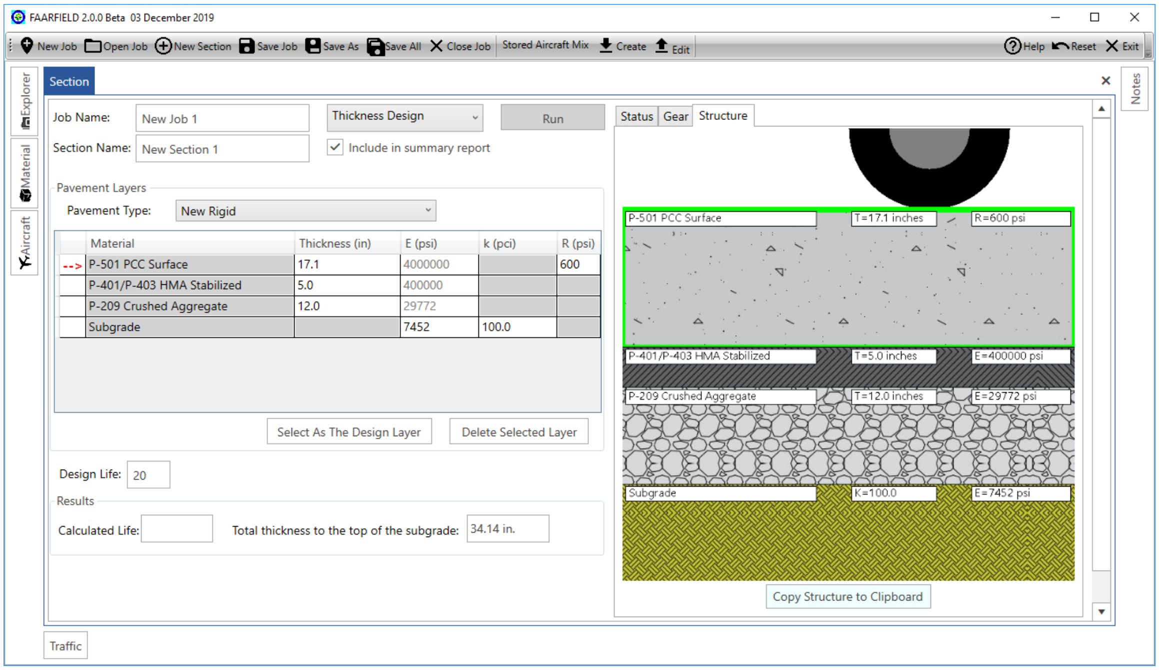
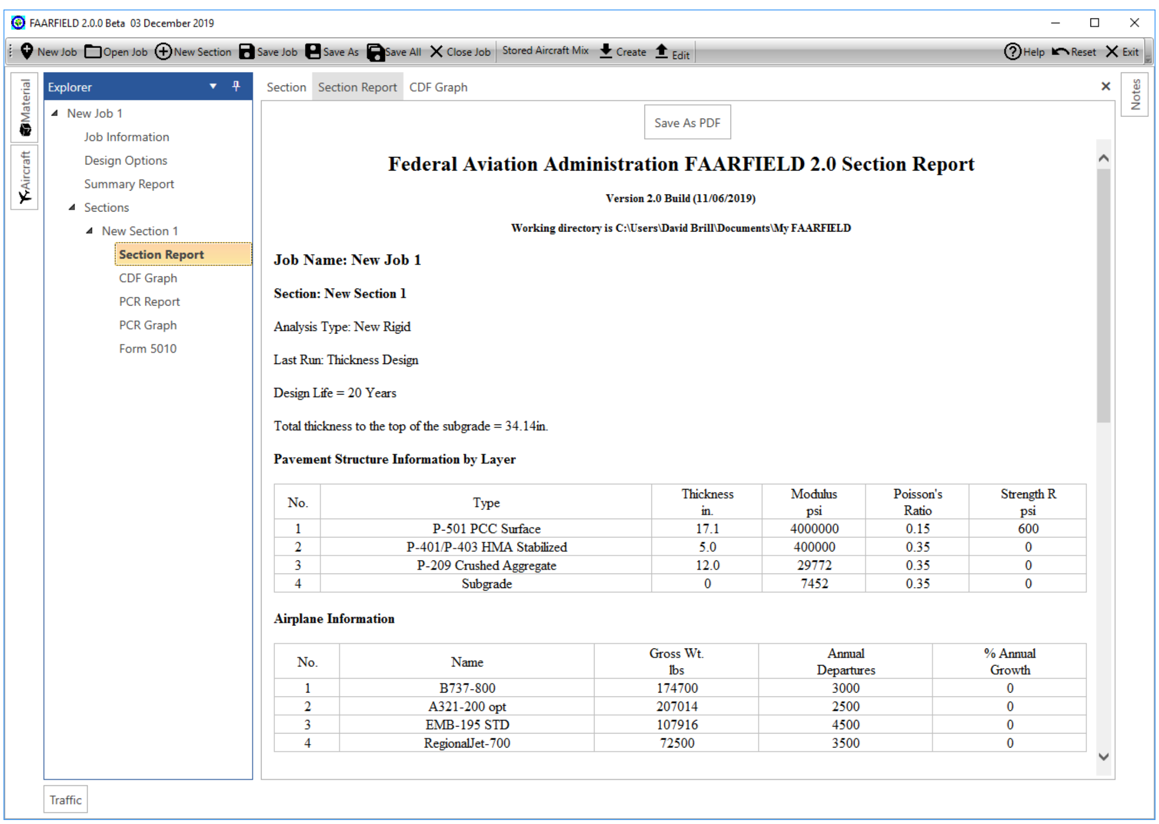
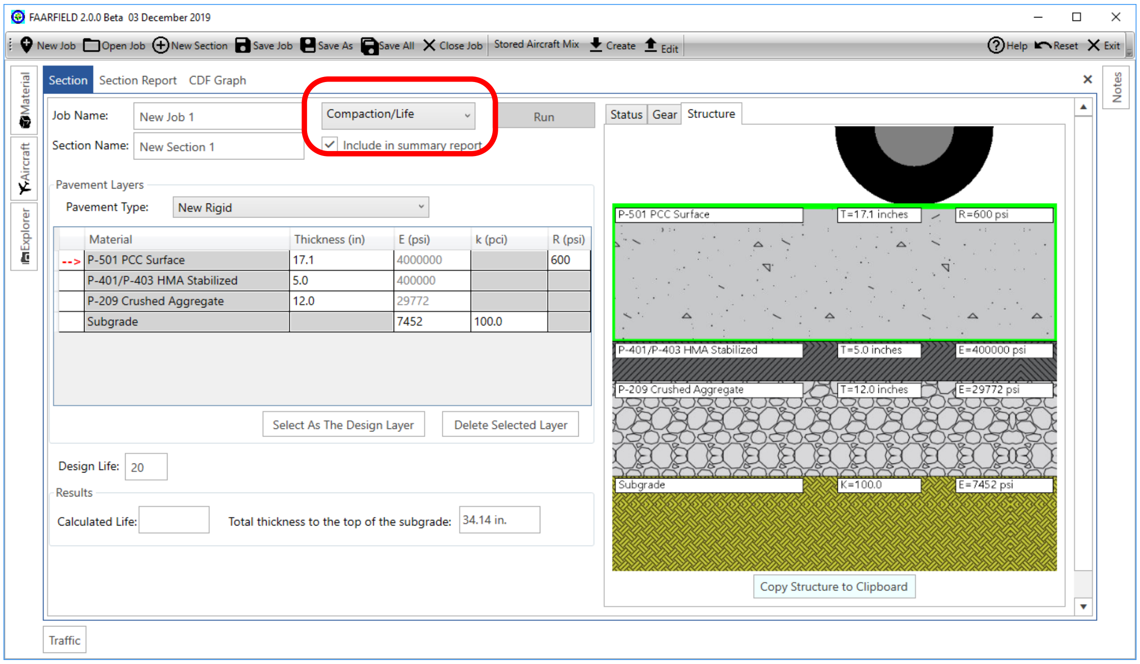


Figure H-14. Rigid Design Example Step 5 (Section Report)



4367 **Figure H-15. Rigid Design Example Step 6 (Compaction Requirements)**



4368

4369 **Figure H-16. (Not Used)**

Figure H-17. Section Report for Rigid Design Example

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 H-18. Section Report for Rigid Design Example (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.

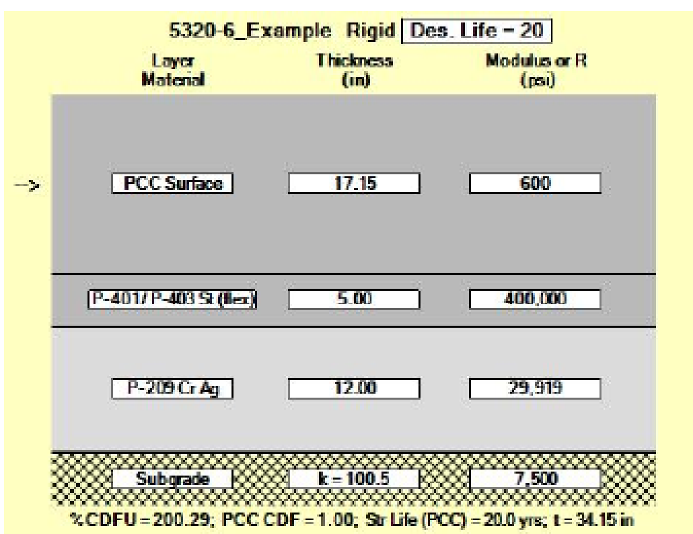
2.Tabulated values indicate depth ranges within which densities should equal or exceed the indicated percentage of the maximum dry density as specified in item P-152.

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.



H.4 Example Flexible Overlay of Flexible

H.4.1 Example - Asphalt Overlay on Existing Flexible Pavement.

An existing flexible taxiway has the following as-built pavement section:

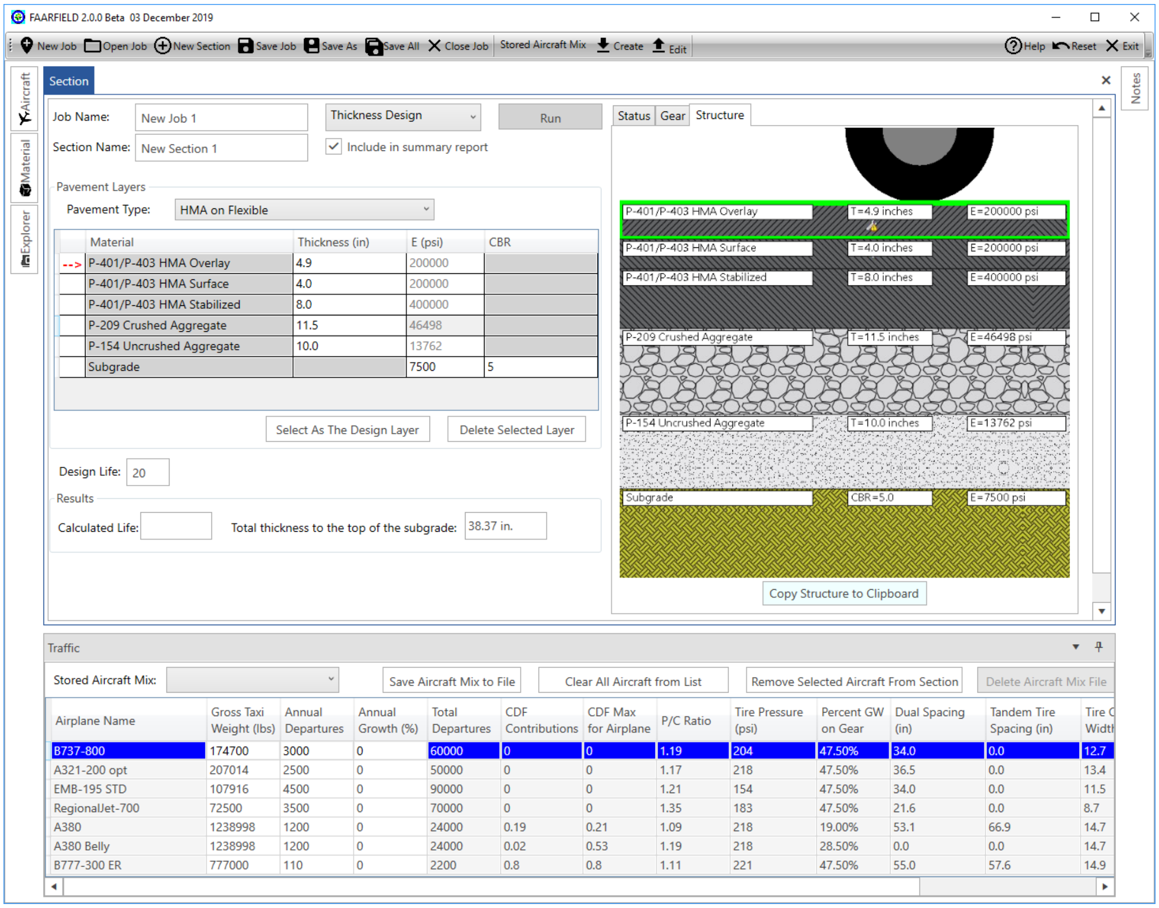
Thickness, inches	Layer Material
5.0	P-401 Asphalt Surface Course
8.0	P-403 Asphalt Stabilized Base Course
11.5	P-209 Crushed Aggregated Base Course
10.0	P-154 Aggregate Base Course
-	Subgrade CBR 5.0

The original section met the FAA standards for materials and construction in effect when constructed 17 years ago. The existing structure is in generally good condition. However, the most recent pavement inspection shows evidence of low-severity weathering and other non-structural distresses. Cores confirm that damage is confined to the top 1-inch (2.5 cm) of the asphalt surface. Traffic has increased, and an asphalt overlay is required to accommodate the following projected traffic 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
A380	1,238,998	1200
B777-300 ER	777,000	110

Figure 3-4 shows thee completed FAARFIELD design Note that the design life is 20 years. Prior to the overlay, the top 1-inch (2.54 mm) of the existing 5-inch (125 mm) surface will be milled. Therefore, in FAARFIELD the thickness of the P-401 surface layer is 4 inches (100 mm). Select the design type “HMA on Flexible”, enter the aircraft data, and edit the layer properties then select “Run” to execute the design. FAARFIELD indicates an overlay thickness of 4.9-inchs. Round this overlay thickness to 5 inches (125 mm) for construction. For an additional example of flexible pavement evaluation, refer to Chapter 5 and Appendix C.

Figure H-19. Example of Asphalt on Flexible Overlay Design in FAARFIELD

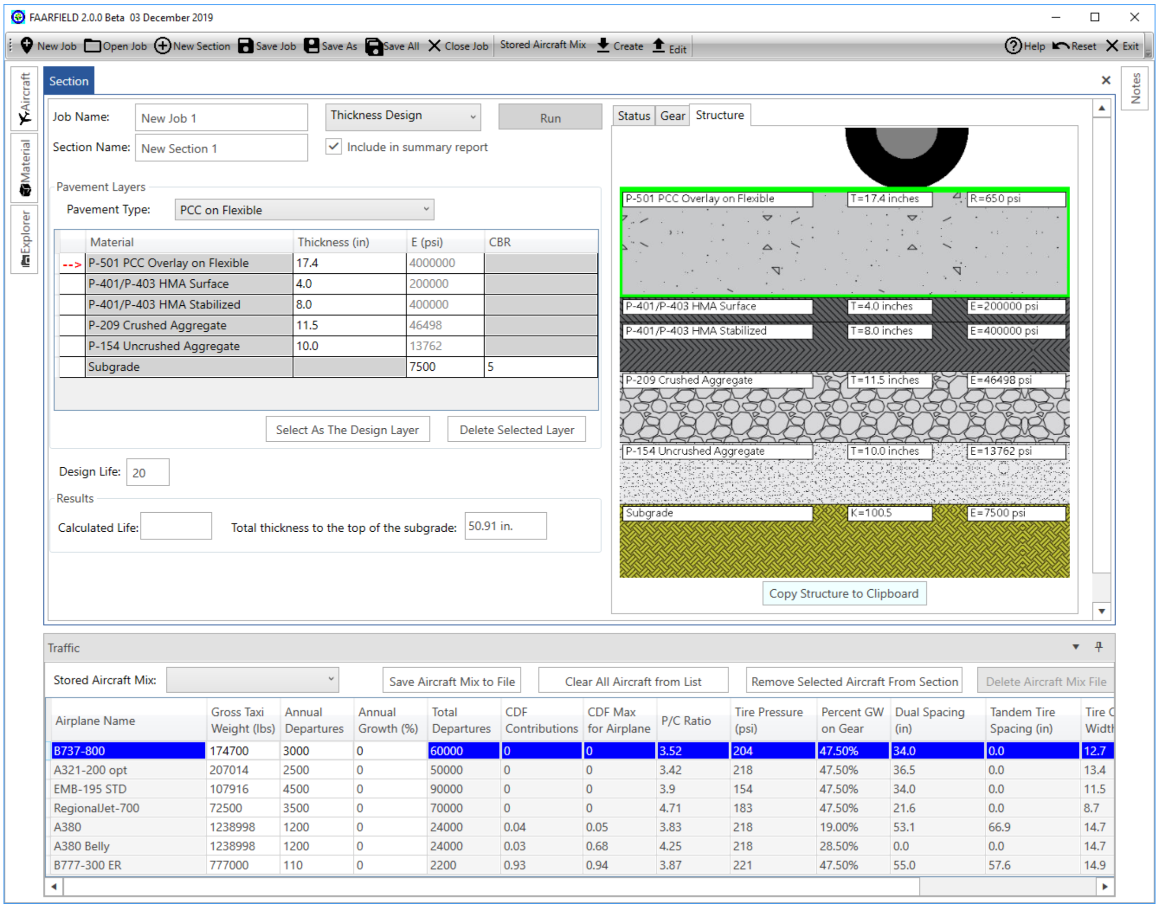


H.5 Example Rigid Overlay of Flexible.

H.5.1 Example - Concrete Overlay on Existing Flexible Pavement.

Assume a concrete overlay of the flexible section as identified in paragraph H.4.1 to accommodate the same aircraft traffic. In FAARFIELD, change the overlay material from P-401/P-403 HMA Overlay to P-501, PCC Overlay on Flexible, by clicking directly on the label (top left of the structure image) and using the dialog box that appears. FAARFIELD automatically changes the analysis type from "HMA on Flexible" to "PCC on Flexible." Assume a concrete flexural strength (*R*) of the overlay of 650 psi, and set the design life is 20 years. See Figure H-20. Click "Run" to execute the design. In this example, FAARFIELD requires a 17.4-inch overlay. Round to a 17.5-inch overlay. Figure 4-2 shows the FAARFIELD screen display.

Figure H-20. Example of Concrete on Flexible Overlay Design in FAARFIELD



H.6 Example Flexible Overlay of Rigid

H.6.1 Example - Asphalt Overlay on Existing Rigid Pavement.

Assume an existing taxiway pavement with the following section:

Thickness (in)	Pavement Structure
16.5	P-501 Concrete Surface Course (R = 625 psi)
5.0	P-401/P-403 Stabilized Base Course
12.0	P-209 Crushed Aggregate Base Course
-	Subgrade, k = 100.5 pci (E = 7500 psi)

The existing pavement will be strengthened to accommodate 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
A380	1,238,998	1200
B777-300 ER	777,000	110

Based on a visual survey, assign the existing pavement an SCI of 80. Estimate the existing concrete strength as 625 psi (4.5 MPa). Frost action is negligible. Perform the design in FAARFIELD using the following steps:

Step 1 In FAARFIELD, select pavement type "Asphalton Rigid" and enter all as-built layer properties and traffic as above. The initial overlay thickness is 12 inches (30 mm) by default. Enter 80 in the SCI box.

Step 2 Set the Design Life to 20 years.

Step 3 From the drop-down list at the top of the screen, select "Thickness Design." Click "Run" and allow the program to execute.

FAARFIELD calculates a required asphalt overlay thickness of 8.2 inches, which will be rounded to 8.5 inches for construction (Figure H-21).

Figure H-21. Example of Asphalt Overlay on Rigid Pavement in FAARFIELD

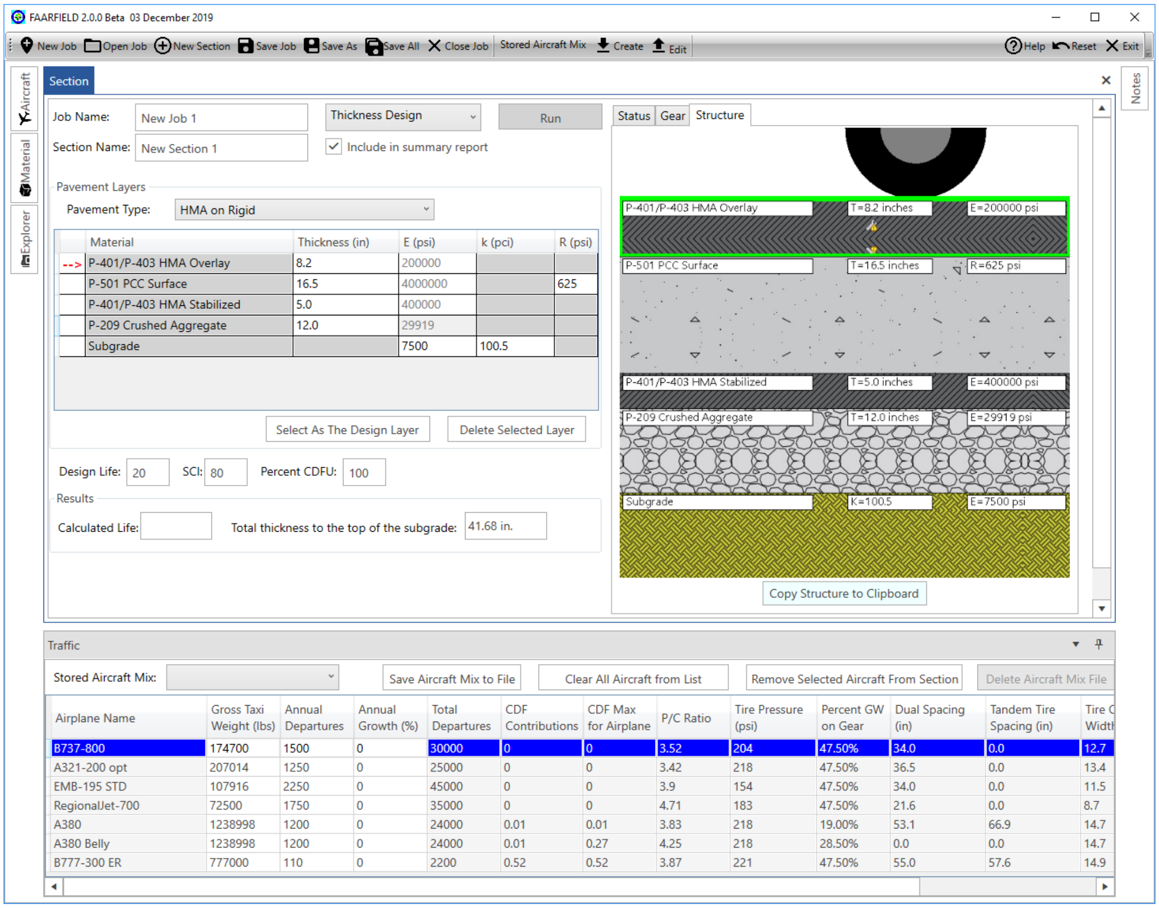


Figure H-22. (not used)

Figure H-23. (not used)

H.7 Example Rigid Overlay of Rigid

H.7.1 Example – Fully Unbonded Concrete Overlay on Existing Rigid Pavement.

Using the same pavement section and traffic as in the example in paragraph 4.7.6.4 evaluate an unbonded concrete overlay. Assume the concrete strength of the new concrete is 650 psi.

Step 1 In FAARFIELD, select pavement type "Unbonded on Rigid" and enter all as-built layer properties and traffic as above. Enter R = 650 psi for the P-501 PCC Surface layer (existing slabs) and R =

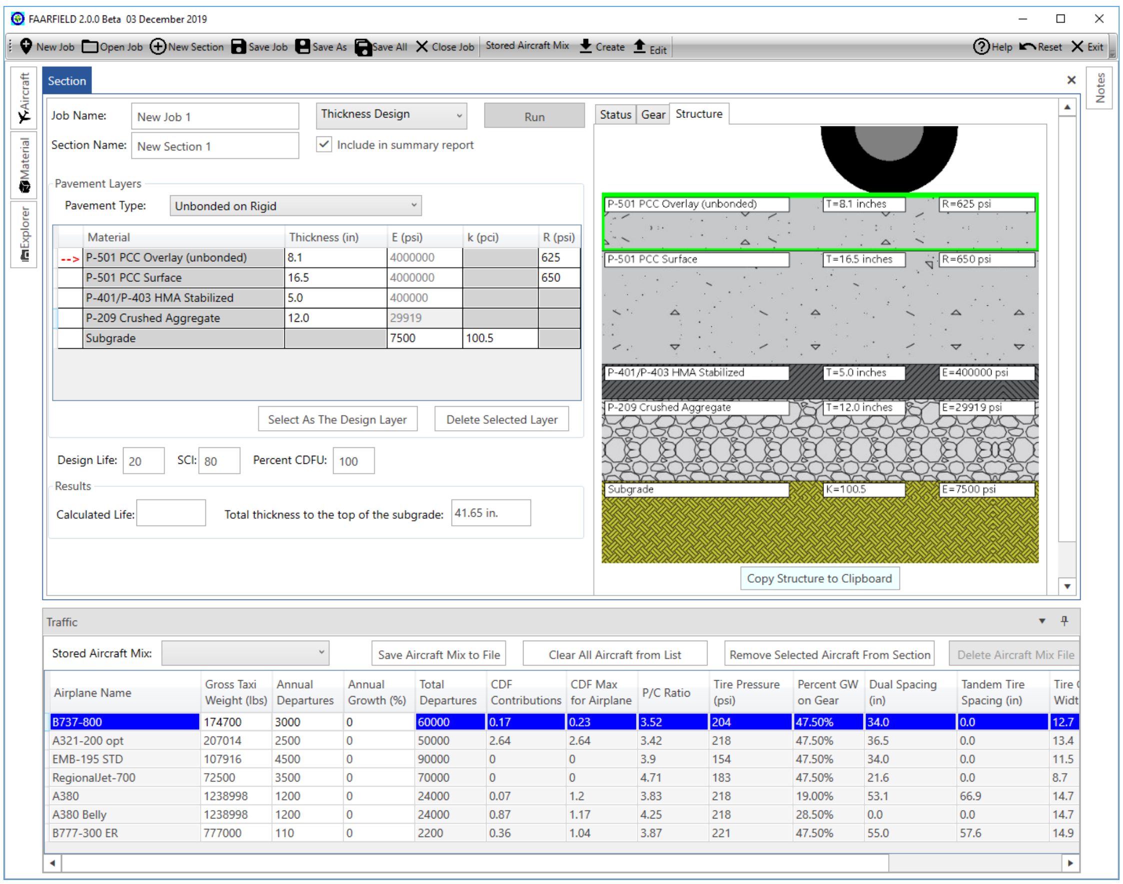
625 psi for the P-501 PCC Overlay. The initial overlay thickness is 12 inches (30 mm) by default. Enter 80 in the SCI box.

Step 2 Set the Design Life to 20 years.

Step 3 From the drop-down list at the top of the screen, select "Thickness Design." Click "Run" and allow the program to execute.

FAARFIELD calculates a concrete overlay thickness of 8.1 inches, which will be rounded down to the nearest 0.5 inches (8.0 inches) for construction (Figure H-24).

Figure H-24. Example of Unbonded Concrete Overlay on Rigid Pavement in FAARFIELD



H.8 Example FAARFIELD Compaction.

H.8.1 Detailed Example FAARFIELD Compaction Table.

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

Table H-1. 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:

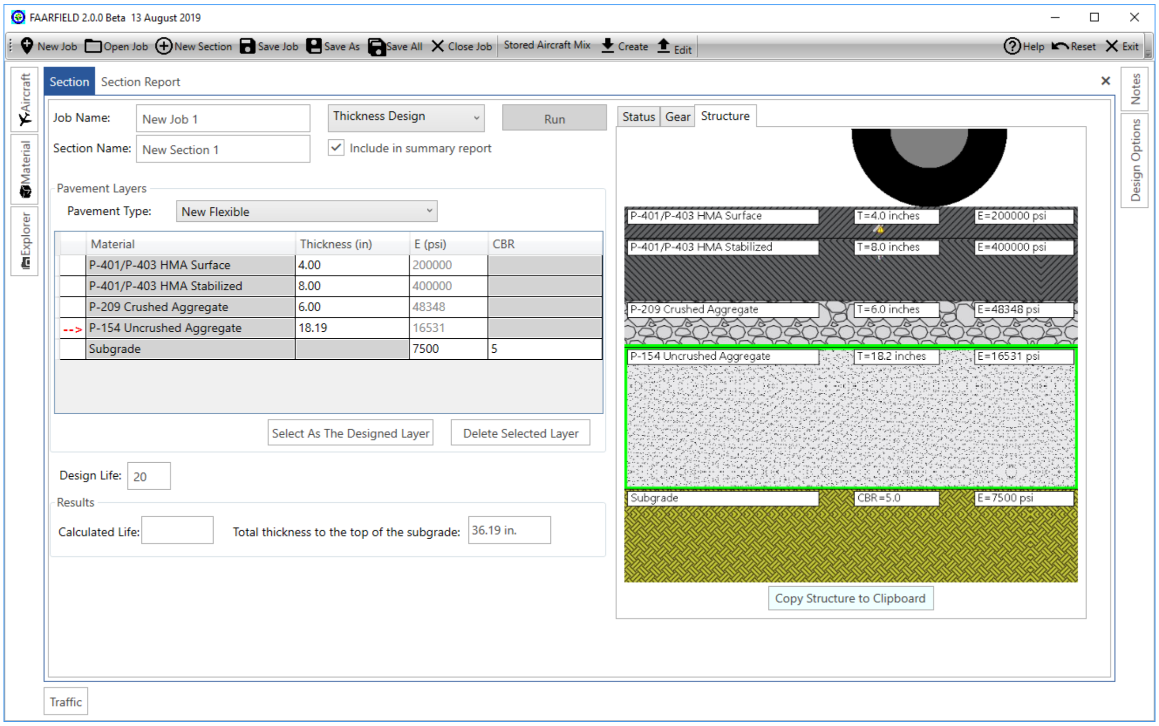
Table H-2. Depths and Densities

Depth Below Existing Grade	In-Place Density[1]
1 ft (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 are determined in accordance with ASTM D 1557 since the aircraft mix includes aircraft greater than 60,000 pounds (27,200 kg) gross weight per paragraph 2.4.2.

4. Run "Thickness Design." The FAARFIELD flexible pavement thickness design results in the following pavement structure (Figure H-25): 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 H-25. FAARFIELD Pavement Structure for Compaction Example



5. In the FAARFIELD home screen, select "Compaction/Life" from the drop-down menu and click "Run." Compaction requirements for the designed section will be displayed in the Section Report. Select "Section Report" in the explorer and scroll to the bottom of the page. For this example, the computed compaction requirements for cohesive soils are shown in Table H-3. For this example, assume that the top of the subgrade will be 20 inches below the top of the existing grade. Figure H-26 shows that the first four inches (10 cm) of subgrade will need to be compacted to meet the 90 percent maximum dry density requirement (red cross-hatched area). Below that level, Figure H-26 shows that the existing densities are greater than the compaction requirements calculated by FAARFIELD, hence no additional compaction is needed.

Table H-3. Computed Compaction Requirements for the Example Section

Cohesive Soil

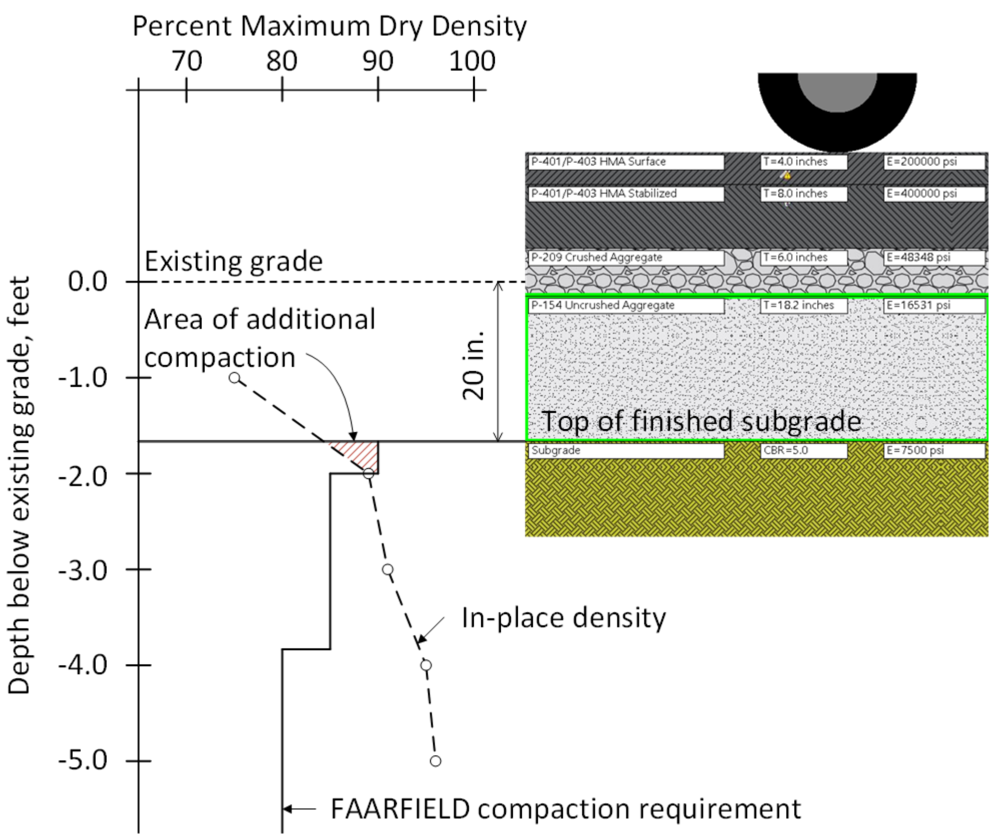
Percent Maximum Dry Density (%)	Depth of compaction from pavement surface (in)	Depth of compaction from top of subgrade (in)	Critical Airplane for Compaction
95	0 – 22	--	A321-200 opt
90	22 – 40	0 – 4	A321-200 opt
85	40 – 62	4 – 26	A321-200 opt

Percent Maximum Dry Density (%)	Depth of compaction from pavement surface (in)	Depth of compaction from top of subgrade (in)	Critical Airplane for Compaction
80	62 – 85	26 – 49	A321-200 opt

Notes:

1. Compaction requirements are given with reference to both pavement surface (finished grade) and finished top of subgrade. Values may not agree exactly due to rounding.
2. The critical airplane for compaction (last column in Table H-1) is the most demanding aircraft for compaction from the design aircraft list. It should not be confused with the critical or design aircraft as used in the CBR method of thickness design. In this example, the A321-200 opt had the most severe compaction requirement at all levels. However, in other cases there may be different critical airplanes for different density levels.
3. The specific compaction requirements in Table H-1 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.

Figure H-26. Subgrade Compaction Requirements for the Example Section



H.9 Example CDFU.

H.9.1 CDFU Example.

The following steps illustrate the procedure for calculating CDFU.

Consider the following existing rigid pavement structure:

Thickness, inches	Pavement Structure
17.5	P-501 Concrete Surface Course (*R* = 625 psi)
5.0	P-401/403 Stabilized Base Course
12.0	P-209 Crushed Aggregate Base Course
-	Subgrade: *k* = 100.5 pci (E = 7500 psi)

Originally, the above pavement was designed to accommodate 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

However, the as-built thickness is 17.5 inches.

The concrete surface does not currently exhibit structural distresses; i.e., SCI = 100. In preparation for an overlay design, we wish to determine the value of CDFU. Assume that the pavement has been in service for 12 years, and 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

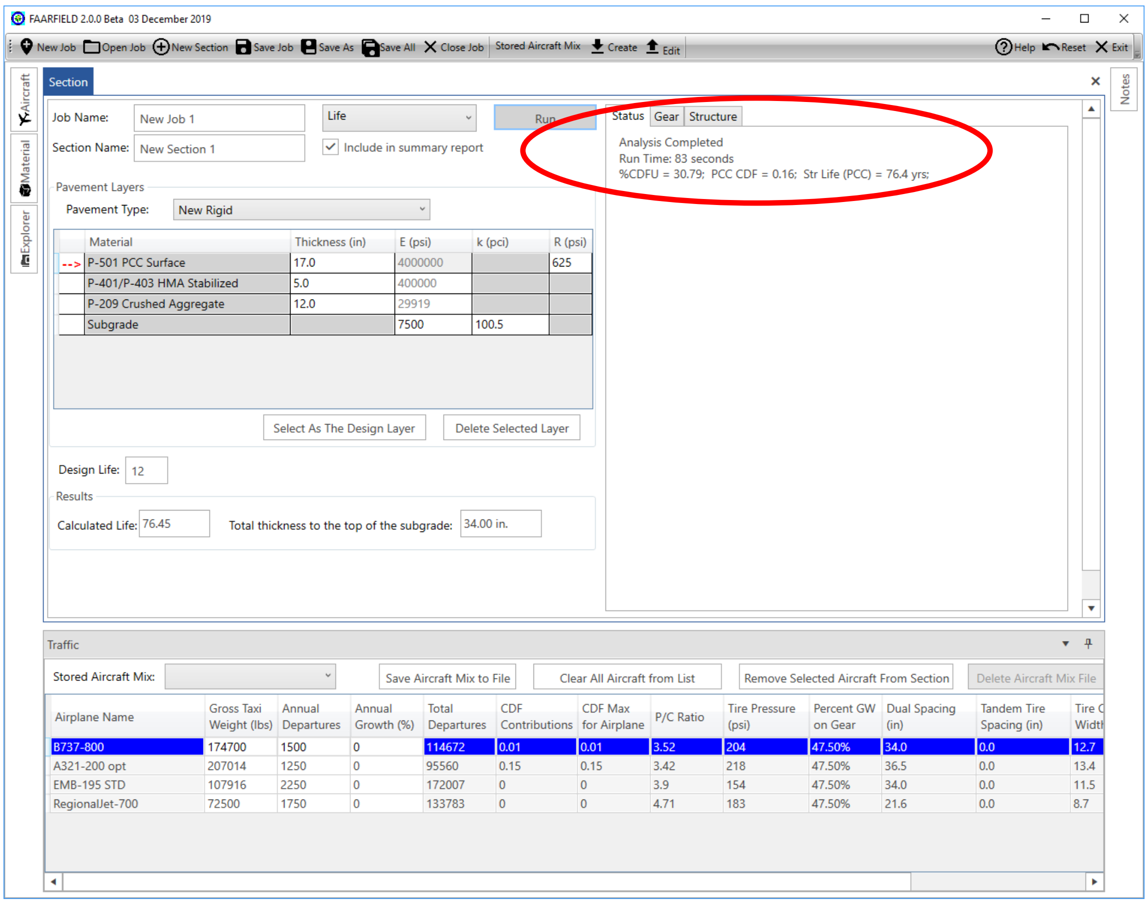
Step 1 In FAARFIELD, select pavement type “New Rigid” and enter all as-built layer properties and traffic as above. Use the actual number of annual departures for each aircraft in the traffic list.

Step 2 Set the Design Life to the number of years the pavement has been in operation (12 years). A message “The standard design life is 20 years (1 to 50 allowed)” will display, indicating that a life equal to other than 20 years has been selected. Click OK to dismiss the message.

Step 3 From the drop-down list at the top of the screen, select “Life.” Click “Run” and allow the program to execute. After execution is complete, the calculated percent CDFU will display under the status tab, at the upper right of the screen.

For the above case, FAARFIELD calculates percent CDFU equal to 30.79. For overlay design, the value CDFU = 31 percent would be used.

Figure H-27. Rigid Overlay Percent CDFU



One potential source of confusion is that the value percent CDFU = 31 does not mean that 31 percent of the original structural design life has been used up. This value should be interpreted as indicating that, the pavement will have received 31 percent of the number of traffic passes predicted to result in a first full structural crack (i.e., 31 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.

Page Intentionally Blank

APPENDIX I. VARIABLE SECTION RUNWAY

I.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.

I.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.

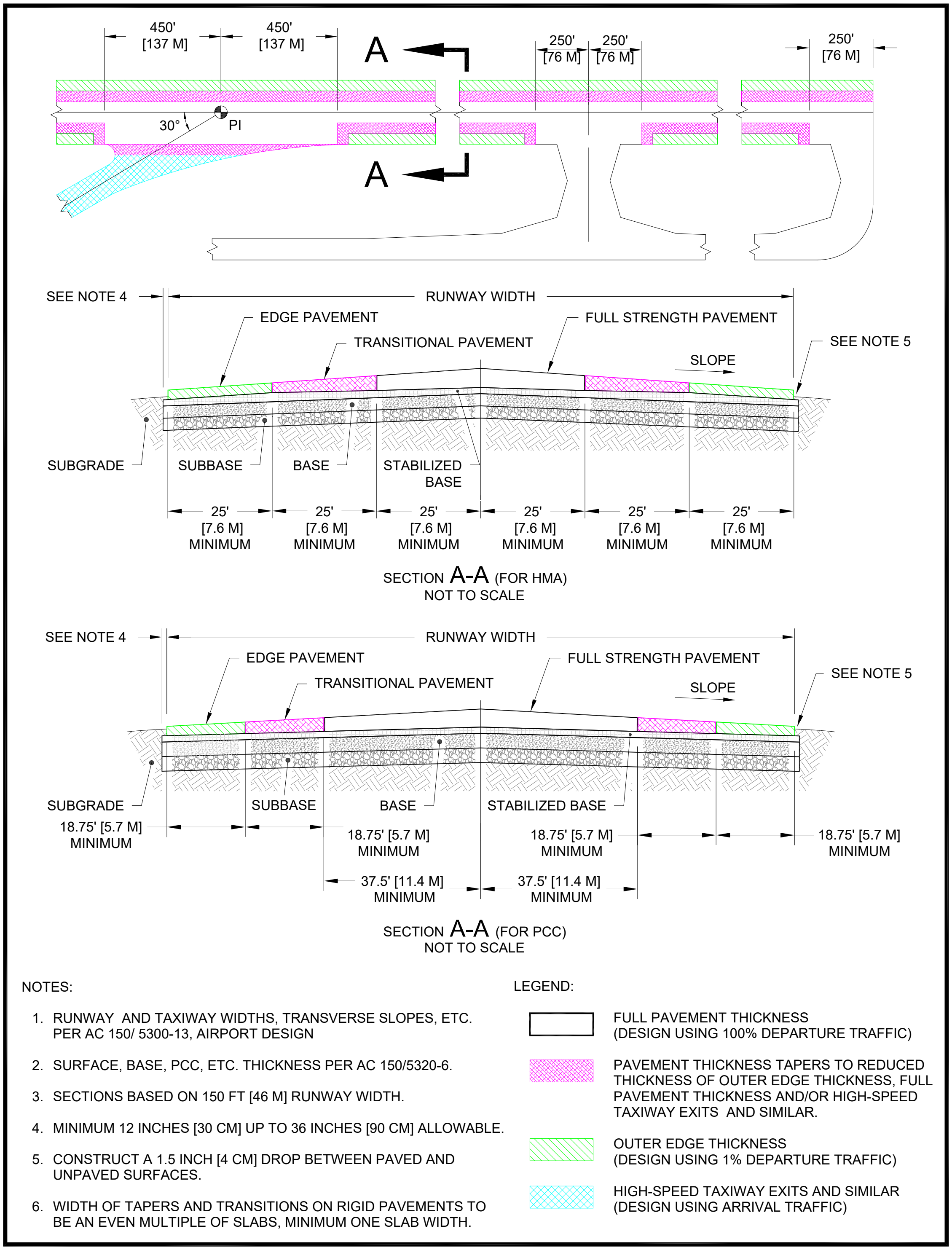
I.2.1 For high speed exits, the pavement thickness is designed using arrival weights and estimated frequency.

I.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.

I.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.

I.3 For rigid pavements 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, adjust the subbase thickness d 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 I-1. Variable Runway Cross-Section



APPENDIX J. RELATED READING MATERIAL

J.1 The following advisory circulars are available for download on the FAA website (https://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 150/5325-4, *Runway Length Requirements for Airport Design.*
7. AC 150/5335-5, *Standardized Method of Reporting Airport Pavement Strength-PCR.*
8. AC 150/5340-30, *Design and Installation Details for Airport Visual Aids.*
9. AC 150/5370-2, *Operational Safety on Airports During Construction*
10. AC 150/5370-10, *Standard for Specifying Construction of Airports.*
11. AC 150/5370-11, *Use of Nondestructive Testing Devices in the Evaluation of Airport Pavement.*
12. AC 150/5370-14, *Hot Mix Asphalt Paving Handbook.*
13. AC 150/5380-6, *Guidelines and Procedures for Maintenance of Airport Pavements.*
14. AC 150/5380-7, *Airport Pavement Management Program (PMP).*
15. AC 150/5380-9, *Guidelines and Procedures for Measuring Airfield Pavement Roughness.*
16. AC 150/5390-2, *Heliport Design.*

J.2 The following orders are available for download on the FAA website (https://www.faa.gov/airports/resources/publications/orders/):

1. FAA Order 5100.38, *Airport Improvement Program Handbook.*
2. FAA Order 5300.1, *Modification of Agency Airport Design, Construction and Equipment Standards*
3. FAA Order 5300.7, *Standard Naming Convention for Aircraft Landing Gear Configurations.*

J.3 Copies of the following technical reports may be obtained from the National Technical Information Service (https://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.

J.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: https://www.astm.org/Standard/standards-and-publications.html.

J.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/.

J.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/.

J.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.

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 Reclaiming Association, #3 Church Circle, PMB 250, Annapolis, Maryland 21401. (https://www.arra.org)

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 150/5320-6G 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: ______________________________