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UNIFIED FACILITIES CRITERIA (UFC)

AIRFIELD PAVEMENT EVALUATION

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

NAVAL FACILITIES ENGINEERING COMMAND

AIR FORCE CIVIL ENGINEERING CENTER

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RECORD OF CHANGES FOR UFC 3-260-03

Chapter 1, *Introduction.* Minor formatting

Chapter 2, *Evaluation Concepts*. Revised Tables 2-1, 2-2, 2-3 and 2-5.

Chapter 3, *Data Collection*. Implemented Linear Segmentation concept. Added Dynamic Cone Penetrometer, Automated Dynamic Cone Penetrometer, Coring Equipment, Split Tensile Tester, and Visual Surveys.

Chapter 4. *Pavement Evaluation Using Nondestructive Testing*. Delete portion of evaluation procedure that conflicts with new segmentation concept. Revised definition of design passes. Revised Table 4-2 to delete outdated aircraft and include new aircraft.

Chapter 5, *Evaluation of Flexible Pavement Using Direct Sampling.* Revised and updated examples. Added new flexible pavement evaluation curves.

Chapter 6, *Evaluation of Rigid Pavement Using Direct Sampling*. Revised and updated examples. Added new rigid pavement evaluation curves. Added new procedure for determining effective K.

Chapter 7, *Pavement Evaluation for Frost Conditions*. Added definitions of FASSI and FAIR. Revised examples. Added example for Reduced Moduli. Added paragraph on evaluation of permafrost.

Chapter 8, *Standardized Method for Reporting Airfield Pavement Strength. Revised* to correspond more closely with FAA procedure outlined in AC 150/5335-5A.

Chapter 9, *Computer Programs for Pavement Evaluation*. Deleted, incorporated into Appendix H.

Revised Appendices:
Appendix A, *References*—Updated reference titles
Appendix B, *Sampling and Testing*-Minor format changes
Appendix H-PCASE Computer Program for Pavement Evaluation--Combined and

revised Chapter 9, Computer Programs for Pavement Evaluation and Appendix C, How to Install Evaluation Computer Program (LEEP)

Added new Appendices:
-Appendix C-Detection of Voids Under Airfield Pavements
-Appendix D-Aircraft Gear Configuration Nomenclature
-Appendix E-Evaluation Procedure for Aged Asphalt (AC) Surfaces
-Appendix F-Structural Evaluation Procedure for Stabilized Soil Surfaced Airfields
-Appendix G-Inspection and Testing of Trim Pad Anchoring Systems

FOREWORD

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

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CHAPTER 1

INTRODUCTION

1-1. **PURPOSE**. This document presents criteria for evaluation of the load-carrying capability of pavements used (or to be used) to support aircraft. An evaluation is conducted to assess the allowable traffic that a pavement can sustain for given loading conditions or the allowable load for a given amount of traffic, without producing unexpected or uncontrolled distress.

1-2. **SCOPE**. This document is for use in evaluating Army, Air Force, Navy, and Marine Corps Airfields and Heliports and is applicable to conventional-type pavements. The procedures presented include direct sampling and nondestructive testing techniques. The document also describes computer programs that can be used for pavement evaluation.

1-3. **REFERENCES**. Appendix A contains a list of references used in this manual.

1-4. **UNITS OF MEASUREMENT**. The unit of measurement in this document is the inch-pound (IP). In some cases, International System of Units (SI) measurements may be the governing critical values because of applicable codes, accepted standards, industry practices, or other considerations.

1-5. **TYPES OF PAVEMENT**. The types of pavement considered in this manual are:

a.Flexible Pavement. A pavement with a bituminous surface course and one or more supporting base or subbase courses, placed over a prepared subgrade.

b. Plain Concrete Pavement. A single thickness of nonreinforced portland cement concrete resting directly on a prepared subgrade, granular base course, or stabilized layer.

c.Rigid Overlay on Rigid Pavement. A rigid overlay pavement that has been placed on an existing rigid pavement. In the construction of the rigid overlay, a bond-breaking course may or may not have been placed on the existing rigid pavement before the overlay was placed. If the bond-breaking course between the two rigid pavements is 4 inches or more in thickness, the entire pavement is considered to be a composite pavement (subparagraph f below).

d. Nonrigid Overlay on Rigid Pavement. A bituminous concrete or combination of bituminous concrete and granular base course that has been placed on an existing rigid pavement.

e. Rigid Overlay on Nonrigid Pavement. A rigid overlay pavement that has been placed on an existing nonrigid pavement.

f. Composite Pavement. A “sandwich pavement” consisting of a rigid overlay placed on an existing pavement that consists of a nonrigid overlay on a rigid pavement. The nonrigid overlay may be bituminous concrete for its full depth or a combination of bituminous concrete and granular base course. When the thickness of the nonrigid overlay is less than four inches, the entire pavement will be treated as a rigid overlay on rigid pavement and the nonrigid material will be considered to be a bond-breaking course.

g. Reinforced Concrete Pavement. A concrete pavement that has been reinforced with steel deformed-bar mats or welded-wire fabrics.

H. Fiber Reinforced Concrete. A concrete pavement that has been reinforced with fibers. Previous evaluation manuals contained curves for evaluating concrete pavements with steel fibers. These curves have been deleted because there are currently no airfield pavements in DoD with steel fibers, due to the fibers causing surface problems. Contact the appropriate MAJCOM Pavements Engineer, USACE /TSC or NAVFAC before using steel fibers. Other types of fibers are used in pavements, but do not reduce the pavement thickness requirement.

CHAPTER 2

EVALUATION CONCEPTS

2-1 **RELATIONSHIP OF DESIGN TO EVALUATION**. The design of a pavement requires selecting materials with the necessary strength and placing them at the proper thickness, density, and depth, so that the pavement will be capable of carrying an anticipated number of passes of a given load. Because of variations in materials and placement conditions, the as-constructed pavement may have strengths and thicknesses of layers greater or less than those required in the design process. Also, with time, usage, and environmental impacts, the elements of a pavement contributing to its strength can change. Thus, an evaluation determines the physical properties of a pavement as actually constructed or in its current condition and establishes there-from the traffic/load-supporting capability of the pavement.

2-2 **CONCEPTS**. The primary function of a pavement is to spread and distribute the wheel loads placed on it. Each airfield or landing strip has its own natural soil and environmental conditions, and the in-situ soils must ultimately sustain the stresses resulting from loads applied to the pavement. Since the strengths of native soils can vary widely from site to site, the ability to support loads also varies widely. Except in special cases, aircraft tire loads cannot be satisfactorily sustained directly on the native soils.

a. Pavement Structure. Pavement design and evaluation are concerned with determining the capability of the pavement structure to reduce the load intensity to a magnitude the airfield site soils can sustain. The larger the load on the surface or the higher the contact pressure, the stronger the pavement structure must be to distribute load and reduce load intensity (pressure or stress) to that which the native soil can accept. Layered flexible pavements distribute load by broadening the effective area supporting the load, from the tire contact area on the surface to a wider area on the base, to a still wider area on the subbase, and so on. Each layer must be of a quality to sustain the load intensity or stress it must accept, and each must be thick enough to broaden or distribute the load and reduce intensity to that which its supporting layer can sustain. Rigid pavements are stiffer and have a “beam action” or flexural capability that spreads or distributes load more widely, so these pavements can be much thinner than flexible pavements. However, thickness, flexural strength, and other quality aspects must be assessed during the evaluation process.

b. Loadings. Early aircraft were primarily supported on two main landing gear wheels, referred to as “single” wheels. With the large increases in aircraft gross weights, landing gears have changed to twin (2 per strut) wheel loadings, to twin-tandem (4-wheel) loadings, and to more complex (16 and 24 main gear wheels, extra “belly” gear) wheel support systems. The two main wheels of single-wheel aircraft are generally spaced far enough apart that there is no significant overlap of the distributed loads for even very thick pavement structures protecting weak subgrades. For twin wheels, however, and closely spaced tandem wheels or complex wheel groups, the patterns of distributed surface loadings at and near the bottom of pavement structures overlap so that the intensities (pressures or stresses) combine between adjacent wheels. This combining effect of load intensities is greater as the adjacent wheels become closer. Appendix D implements a new aircraft gear configuration nomenclature for the Services.

c.Tire Pressure. The intensity of stress at a given point in a flexible pavement is affected by the tire contact pressure, which, for large aircraft tires, is roughly equivalent to the inflation pressures. The major difference in stress intensities caused by variation in tire pressure occurs near the surface; consequently, the pavement surfacing and upper base-course layers are most seriously affected by high tire pressures. Evaluation curves in this document are based on constant tire pressure; previous curves were based on constant contact area.

d. Load Repetitions.

(1.) Repetition of load (aircraft passes or coverages) is an aspect of structural capability. A pavement capable of sustaining a certain aircraft loading on a regular repeating basis for some design life of the facility (commonly 20 years for Navy and Marine Corps , Army, and Air Force airfields) can sustain repeated application of a larger loading, but for a reduced pavement life (less number of passes). Passes are defined as the number of aircraft movements across an imaginary transverse line placed within 500 feet of the end of the runway. Since touch-and-go aircraft operations will not pass this line, they will not be counted. For taxiways and aprons, passes are determined by the number of aircraft movements across a line on the primary taxiway that connects the runway and the parking apron. At single-runway airfields, the pass levels for the runway, taxiway, and apron could be the same. Repetitions of load are also quantified as coverages, a term used to define the number of maximum stress repetitions that occur in a pavement as a result of aircraft operations. For flexible pavement, a coverage occurs when every point on the pavement surface within the traffic lane has been subjected to one application of maximum stress by operating aircraft. For rigid pavement, a coverage occurs when each point in the pavement within the limits of the traffic lane has been subjected to a maximum stress by operating aircraft. Maximum stress is the stress induced in the pavement by the aircraft wheels when the aircraft is operating at its maximum gross weight. Coverage is a function of gear and tire width. Evaluation curves in this manual use the Pass/Coverage Ratio. Pass/Coverage Ratios for rigid and flexible pavements are shown in Table 6-2.

(2) It follows that an evaluation of the structural capability of a pavement may determine not only a maximum allowable number of passes for a specific loading, but also a maximum allowable loading for a given number of passes of traffic.

(3) This pattern of load and repetitions implies that a single application of a given load can be considered to represent a number of applications of a load of a lower magnitude. The number of applications can therefore be taken as the equivalent applications of one load to another. These equivalent applications or equivalencies will normally be uneven or fractional numbers. For example, one application of a load which is 20 percent heavier than another, when applied to a pavement, may be considered equivalent to 6.5 applications of the smaller load, or one application of the lighter load may be considered equivalent to 0.15 applications of the larger load.

(4) Extension of this concept permits the reduction of an array of loadings and the repetitions of each to an equivalent number of repetitions of a single selected load. By stating each loading in the array as equivalent applications of a selected basic load, multiplying each by its actual number of repetitions, and accumulating the total, then the total applied traffic can be stated as equivalent repetitions (or applications) of the selected basic loading. This methodology is an important adjunct to evaluation, since it permits comparisons of cumulative past traffic, design traffic, traffic associated with load evaluation, and increments of pavement life associated with overloading.

2-**3 EVALUATION PROCEDURE**.

a.Steps in the Procedure. Evaluation is the assessment of pavement strength and condition and the computation of the load-carrying capability. The following steps are generally used in pavement evaluations:

(1)Thorough study of all existing information regarding design, construction, maintenance, traffic history of the pavements, results of physical-property tests of the pavements, and weather records for the vicinity.

(2) Determination of pavement condition by formal Pavement Condition Index (PCI) method as delineated in American Society for Testing and Materials (ASTM) D 5340 wherever possible, but as a minimum, by direct visual or cursory inspection.

(3) Segmentation of an airfield (Network) into Branches (Runway, Overrun, Taxiway, Apron, Helipad) and subdividing Branches into pavement Sections, as described in Chapter 3.

(4) Determination of the scope, validity of available data, and need for additional information or tests.

(5) Determination of pavement element characteristics and/or pavement response to loading for input to the evaluation method, using one or a combination of the following procedures:

(a)Selection of strength, thickness, and other behavioral values considered representative of the flexible or rigid pavement surfacing, base course, subbase course, and subgrade from available data.

(b) Opening test pits in selected representative locations for determination of material characteristics, layer thicknesses, soil strengths, and moisture-density conditions (seldom used due to impact on operations).

(c) Using the dynamic cone penetrometer (DCP) or automated DCP to determine soil strengths and layer thickness.

(d) Nondestructive testing that provides data for determining a stiffness modulus (dynamic or impulse) of the overall pavement Section for use as a basis for evaluation.

(e) Nondestructive methods that measure the deflection basin response to loading and determine the pavement layer moduli by matching the deflection basin with an elastic layer model.

(f) Nondestructive testing systems, such as the Portable Seismic Pavement Analyzer (PSPA), using wave propagation and elastic theory for determining layer stiffness moduli as a basis for evaluation.

(6) Determination of load-carrying capability and Pavement Classification Number (PCN) for each Section of the airfield pavements through the application of the evaluation criteria, using representative pavement properties. Load-carrying capability implies allowable load for selected repetitions or allowable repetitions for selected loadings.

(7) Assignment of a PCN for each airfield Section. The PCN for each runway is also reported, as required by ICAO, based on the load-carrying capability of the weakest pavement Section in the first 1000 feet from each runway end (full width) and the center 75 feet of the interior portion of the runway.

B. Additional Tests. The decision as to the necessity for obtaining additional test data at the time of the evaluation or as to the means of evaluation to be employed rests with the evaluating engineer. In many cases, and particularly when relatively new pavements are being considered, design and construction control data are sufficient for the evaluation. However, in these instances, the engineer must be satisfied that the data are representative and valid and that future changes in condition and strength have been considered. For older pavements or in cases where the applicability of available test results is in doubt, additional tests are desirable. Where circumstances preclude conducting these additional tests, physical property values should be assigned on the most realistic basis possible, with comments by the evaluating engineer on the limitations associated with the values used.

2-4 **SITE DATA**. In addition to test data on the physical properties of the pavement elements, it is desirable to obtain general information regarding the site. Much of the information can be obtained from records of preliminary investigations and from the design analysis. General types of information that should be obtained are as follows:

a.Geographical Location. The geographical location of the airfield can be determined using existing engineering data, normally furnished by the using agency.

b. Geology. The general geology of the vicinity will be determined as it applies to the soils at the airfield. The general type of soil deposition (e.g., alluvial, residual), the parent rock from which the soil is derived, and other pertinent information will be identified. Aerial photographs showing pertinent features of the area should be obtained when available. Information can be obtained from U.S. Geological Survey publications and from state geological departments, subsurface exploration companies, and similar organizations. Soil types can be determined from such sources as Department of Agriculture soil maps, state highway departments, and well logs.

c. Drainage and Ground-Water Conditions. First, the general surface-drainage system for the area should be ascertained. The natural drainage pattern can be established from contour maps published by the U.S. Geological Survey, the National Oceanic and Atmospheric Administration, or the National Geospacial-Intelligence Agency (NGA). Detailed information will be collected concerning drainage at the airfield, including descriptions of any drainage structures and shoulder slopes, and whether excessive vegetation or soil has built up along the pavement edges sufficiently to pond water on the pavements. The depths to ground-water tables in the vicinity and at the airfield property should be determined, and the presence of any perched water tables in the airfield subgrade will be noted. Information concerning ground-water tables can be obtained from well logs, cuts, or borings in the vicinity, and the location of springs and seeps. Subsurface drainage systems must also be identified and evaluated.

d. Climatic Data. Information on climatic data can be extracted from routine National Weather Service publications and from records of the airfield weather station. For the period of record, the climatic data should include average daily maximum and minimum temperatures for each month, average annual rainfall, freezing index, average humidity, and description of the prevailing winds.

e. Maintenance. Detailed information should be obtained on the maintenance performed on each facility. The dates when application of such items as overlays, seal coats, surface treatments, joint seals, and patches should be ascertained, and the reason for performing the work should be explained in all possible detail. Files of the Facilities Engineer, Base Civil Engineer, or responsible construction office should contain this information.

f. Current Condition of Pavements. A detailed survey should be made of the pavement surface on all Branches and Sections. Procedures for condition surveys of existing pavements are presented in ASTM D 5340. If a current survey is available, a cursory survey should be conducted for validation.

g. Airfield Traffic Data. For a pavement evaluation to be meaningful, it is essential to have some measure of normal and expected traffic in terms of repetitions and loading characteristics. Thus, the traffic data collected must include the type of aircraft, gross weight, and typical operating weights of each type aircraft regularly using the airfield on a day-to-day basis.

2-5 **OPERATIONAL CONSIDERATIONS**.

a.Intensity and Repetition of Load. The primary factors influencing the load-carrying capability of an airfield pavement are the thickness and strength of the pavement layers, distribution of the induced loading (gear configuration and tire pressure), and number of repetitions of loads by the aircraft. Airfield pavements may be evaluated to:

(1)Determine the number of repetitions of an aircraft that can use a pavement at a designated gross weight.

(2) Determine the allowable gross weight of an aircraft that can use a pavement for a given number of repetitions.

(3) Determine the effect past aircraft operations have had on pavement life.

(4) Determine the PCN for the day-to-day traffic or for specified standard traffic.

b. Aircraft Grouping for Air Force Evaluation. To reduce calculations and simplify the evaluation procedure, aircraft have been placed in 14 aircraft groups designated by an Aircraft Group Index, as shown in Table 2-1. The table contains a listing of aircraft that may use Air Force airfields. A controlling aircraft (aircraft having the most severe gear) was selected for each group with more than one aircraft. In addition, Table 2-1 includes the lowest and highest gross weight of aircraft in each group. The evaluation uses the critical gear and the full range of weights in each group, from lowest to highest. A description of the landing gear assembly on the aircraft is shown in Table 2-2. New gear configuration nomenclature from Appendix D and points for calculating stress are included.

Aircraft Group Index

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

Note: * Denotes Controlling Landing Gear Configuration in Group

Pass Intensity Levels (in Passes)

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

Gross Weight Ranges for Aircraft Groups (in KIPs)

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

Table 2-1. Aircraft Group Index and Pass Intensity Levels

Aircraft Group Index: Gear Types

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

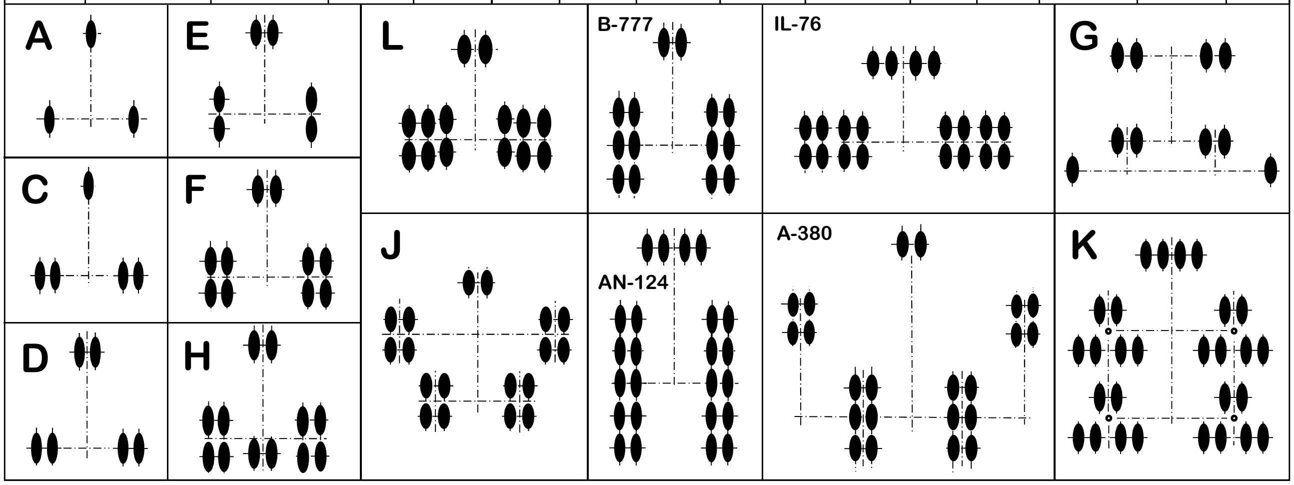


Table 2-2. Gear Types for USAF Evaluations

c. Aircraft for Army Evaluations. The Army airfield commander is responsible for providing for each runway, taxiway, and apron system, a traffic report of all aircraft using the airfield. Rotary wing aircraft should be included. The traffic report will include the following:

(1)Aircraft Type.

(2) Actual Weights.

d. Aircraft for Navy and Marine Corps Evaluations. The Airfield Commander will provide for each runway, taxiway, and apron system a traffic report of all aircraft using the airfield. The traffic report will include the aircraft type, actual weights, and number of aircraft passes. Navy aircraft can be gathered into five groups as indicated in Table 2-3. Evaluation results are expressed in terms of the five groups, or a subset of the five groups which encompasses the actual traffic at the activity. Each group is represented by one aircraft: F-18 for single tricycle, and P-8A for dual-tricycle, C-130 for single tandem tricycle, C-17 for triple/dual tandem tricycle, and C-5A for twin delta tandem (designated by [1)]. Special aircraft not included in these groups can be studied separately (e.g. KC-10 or small aircraft for the case of outlying landing fields). Navy and Marine Corps airfield are evaluated for the Peace Time Maximum Takeoff Weight. Table 2-4 contains the Peace-time Maximum Takeoff Weight and Design Maximum Landing Weight of various aircraft.

Single Tricycle	Dual Tricycle	Single-Tandem Tricycle	Triple/Dual-Tandem Tricycle	Twin Delta Tandem
F/A-18[1]	P-8A[1]	C-130[1]	C-17[1]	C-5A[1]
F-35C	C-40A		KC-135	
EA-18G	P-3		B-707	
F-15	CH-46		B-757	
T-1	CH-53		B-767	
T-2C	B-727		E-3A	
T-39A	B-737		E-6B	
A-6E				
S-3A				
E-2D				
T-34				
T-45				

Table 2-3. Navy Aircraft Groups

Aircraft	Peace-Time Max Take-off Weight (kips)	Design Max Landing Weight (kips)	Aircraft	Peace-Time Max Take-off Weight (kips)	Design Max Landing Weight (kips)	Aircraft Designation	Peace-Tim Max Take Weight (kip
F-15A/ B/C/D	68.0	68.0	Lockheed 1329	42.0	35.0	McDonnell Douglas DC-9-15	90.7
F-15E	81.0	81.0	Lockheed 1649A	156.0	123.0	McDonnell Douglas DC-9-15F	90.7
F-16A	34.6	35.4	Lockheed 749A	107.0	89.5	McDonnell Douglas DC-9-21	98.0
F-16B	34.7	35.4	Lockheed L-1011-1, Tristar	430.0	358.0	McDonnell Douglas DC-9-32	108.0
F-16C	36.3	37.5	Lockheed L-1011-100, Tristar	466.0	368.0	McDonnell Douglas DC-9-41	114.0
F-16D	35.6	37.5	Lockheed L-1011-200 Tristar	466.0	368.0	McDonnell Douglas DC-9-51	121.0
F-35B	40.4	51.0	Lockheed L-1011-500	496.0	368.0	McDonnell Douglas MD-81	140.0
F-35C	45.3	64.3	Lockheed L-1011-500	496.0	368.0	McDonnell Douglas MD-	149.5

			Tristar			82	
			Lockheed L-188	116.0	95.7	McDonnell Douglas MD-83	160.0
			Lockheed L-382 (L-100-20)	155.0	130.0	McDonnell Douglas MD-87	149.5
			Lockheed L-382 (L-100-30)	155.0	135.0	McDonnell Douglas MD-88	149.5

Aircraft	Peace-Time Max Take-off Weight (kips)	Design Max Landing Weight (kips)	Aircraft	Peace-Time Max Take-off Weight (kips)	Design Max Landing Weight (kips)	Aircraft	Peace-Time Max Take-off Weight (kips)
F/A-18 A/B/C/D	51.9	39.0	Martin 404	42.8	41.0	McDonnell Douglas DC-9-33F	108.0
Fairchild F-27	42.0	40.0	MC-130E/H	155.0	175.0	Nihon/N.A.M.C. YS-11A	55.1
Fairchild FH-227	45.5	45.0	McDonnell Douglas DC-10-10	440.0	363.5	P-3C	135.0
Fokker F-28	62.0	54.0	McDonnell Douglas DC-8-72F	335.0	250.0	RC-135S/W	301.6
Gen. Dyn./ Convair 880	193.0	155.0	McDonnell Douglas DC-10-30	555.0	421.0	RC-135U/V	301.6

Gen. Dyn./Convair 990	253.0	202.0	McDonnell Douglas DC-10-40	555.0	421.0	RF-4C	52.8
Grumman Gulfstream I	35.1	33.6	McDonnell Douglas DC-8-43	315.0	207.0	S-3A	52.5
Grumman Gulfstream II	65.5	58.5	McDonnell Douglas DC-8-55	325.0	217.0	TC-130Q	155.0
Hawker Siddeley HS-748	44.5	43.0	McDonnell Douglas DC-8-61	325.0	240.0	TC-4C	36.0
Ilyushin IL-62	357.2	231.5	McDonnell Douglas DC-8-61F	328.0	258.0	TR-1A	40.0
KC-10A	590.0	436.0	McDonnell Douglas DC-8-62	350.0	240.0	VC-25A	836.0
KC-130F	155.0	173.4	McDonnell Douglas DC-8-62F	350.0	250.0	WC-130E/H	155.0
KC-130R	155.0	173.4	McDonnell Douglas DC-8-63	355.0	258.0	WC-135B	301.6
P-8A	165.0	149.8	McDonnell Douglas DC-8-63F	355.0	275.0		
			McDonnell Douglas DC-8-71	325.0	240.0		

Table 2-4. Aircraft Peace-time Maximum Take-off and Design Maximum Landing Weight

2-6 **EVALUATION TESTING METHODS**. There are two basic testing methods used to evaluate Army, Navy, Marine and Air Force airfield pavements. These are nondestructive testing techniques and direct sampling techniques. The most commonly used method is the nondestructive testing method. The evaluation procedure using nondestructive testing is presented in Chapter 4, the procedure using direct sampling for flexible pavements is presented in Chapter 5, and the procedure using direct sampling for rigid pavements and overlays is presented in Chapter 6. Evaluation procedures in areas subject to seasonal frost are presented in Chapter 7.

2-7 **AIRCRAFT/PAVEMENT CLASSIFICATION NUMBERS (ACN/PCN).** The ACN/PCN is a reporting method for weight-bearing capability and not an evaluation procedure. NGA publishes weight bearing limits in terms of ACN/PCN in a Flight Information Publication for civil and international use. The intent is to provide planning information for individual flights or multi-flight missions which will avoid either overloading of pavement facilities or refused landing permission.

a.The International Civil Aviation Organization (ICAO) (DOC 9157-AN/901 and Amendment number 35 to Annex 14) devised the ACN/PCN method as an effective, simple, and readily comprehensible means for reporting aircraft weight-bearing capability of runways. The United States, as a cooperating ICAO nation, has agreed to report airfield weight-bearing limits by this method, and the airfield weight-bearing limits will be included in evaluation reports.

b. The ACN and PCN are defined as follows:

(1)ACN is a number that expresses the relative structural effect of an aircraft on different pavement types for specified standard subgrade strengths in terms of a standard single-wheel load.

(2) PCN is a number that expresses the relative load-carrying capability of a pavement in terms of a standard single-wheel load.

c. The system is structured so that a pavement with a particular PCN value can support, without weight restrictions, an aircraft that has an ACN value equal to or less than the pavement’s PCN value.

ACN values can be provided by the aircraft manufacturers, Army and Air Force Engineering Technical letters, or computed using PCASE. ACN charts and values found in this UFC were determined using PCASE. The ACN has been developed for two types of pavements, flexible and rigid, and for four levels of subgrade strength. Table 2-5 contains ACN values for the representative aircraft in the Navy Aircraft Groups at various evaluation weights.

Aircraft	Load	Peace -Time Take Off Weight (lbs)[1]	A K> 400 E>57022	B K ≤ 400 K > 200 E≤57022 E>23416	C K ≤ 200 K > 100 E≤23416 E>9616	D K≤ 100 E≤9616	A CBR>13 E>19500	B CBR>13 CBR>8 E≤19500 E>1200
F-18	Full	66,000	29	29	29	29	28	28
	Evaluation	43,900	26	26	26	26	25	25
	Half	39,700	22	22	22	22	21	21
	Empty	32,795	15	15	15	15	14	14
P-8	Full	188,200	41	43	45	46	36	39
	Evaluation	165,000	36	38	39	40	31	33
	Half	155,600	29	31	32	33	25	27
	Empty	141,800	17	18	19	20	15	16
C-130	Full	155,000	27	30	33	35	24	28
	Evaluation	143,000	25	28	30	32	22	26
	Half	113,500	19	21	23	25	17	20
	Empty	72,000	12	13	14	14	10	12
C-17	Full	585,000	52	50	54	66	50	57
	Evaluation	525,000	45	45	48	58	44	50
	Half	432,500	36	37	38	45	35	39
	Empty	280,000	21	24	24	23	20	21
C-5A	Full	769,000	26	31	40	50	33	36
	Evaluation	693,000	24	28	35	43	29	32
	Half	543,500	18	20	25	31	21	23
	Empty	318,000	10	10	10	12	10	10

Table 2-5. ACN Values for Representative Navy Aircraft

e. The PCN numerical value for a particular pavement is determined from the allowable load-carrying capability of the pavement for a given number of passes and a given landing gear configuration. Once the allowable load is established, the determination of the PCN value is a process of converting that load to a standard relative value. The allowable load to use for Army, Navy, and Marine Corps evaluations is the maximum allowable load of the most critical aircraft that can use the pavement for the number of equivalent passes expected to be applied for the remaining life. The allowable load to use for Air Force evaluations is based on 50,000 passes of the C-17 aircraft.

f. The PCN value is for reporting pavement strength only. The PCN value expresses the results of pavement evaluation in relative terms and cannot be used for pavement design or as a substitute for evaluation.

2-8 **EVALUATION OF ARMY AIRFIELDS AND HELIPORTS.** An evaluation indicating the allowable pass/load relationship and PCN will be made for each aircraft using the airfield. The PCN will be based on the critical aircraft and a design life of 20 years. The U.S. Army, as a result of its evaluations, requires that preliminary overlay thickness requirements be determined for planning purposes and included in the evaluation report along with maintenance requirements for day-to-day traffic. Design requirements for Army airfields are contained in UFC 3-260-02, Pavement Design for Airfields. A more thorough investigation should be completed for the selection of final overlay design thicknesses.

2-9 **EVALUATION OF AIR FORCE AIRFIELDS**. Evaluations indicating the allowable pass/load relationship will be made for each Aircraft Group Index (Table 2-1) for each airfield Section. The allowable load for Air Force airfields will be determined for four pass intensity levels based upon the aircraft group index as shown in Table 2-1. Pass intensity levels are for normal conditions and frost melting periods. PCN values will be determined for each Section based on 50,000 passes of the C-17 aircraft.

2-10 **EVALUATION OF NAVY AND MARINE CORPS AIRFIELDS.**

a.Navy and Marine Corps Air Stations are evaluated for 20-year life expectancy. The projected aircraft traffic for the next 20 years is first determined. Using aircraft equivalencies (e.g., from ICAO Aerodrome Design Manual) a design critical aircraft can be found for each Section, together with its critical passes. The design critical aircraft at this level of passes is equivalent to the whole traffic mix. Following the FAA definition (FAA AC 150/5320-6D and ICAO section 4.4.11), the design critical aircraft "...should be selected on the basis of the one requiring the greatest pavement thickness". For evaluation, the Navy has selected aircraft using the evaluation loads shown in Table 2-4.

b.In the Navy procedure, the whole traffic is converted to passes of a fully loaded F-18 (single tricycle category), then to passes of fully loaded P-8 (dual tricycle category), and so on for all the existing categories at the airfield (typically five or less). In each case a tentative PCN can be calculated. These tentative PCNs can be used to impose weight restrictions on each separate category to ensure that the pavements will last 20 years. The design critical PCN coincides with the tentative PCN of the aircraft which would require the greatest pavement thickness - this is used for determining the color structural condition map and overlays. The FLIP chart PCN is also one on these tentative PCNs, but may or may not coincide with the design critical PCN - the FLIP chart PCN is used for limiting airfield access to excessively damaging aircraft, as explained below.

c.It is necessary to prevent the use of the pavement by excessively large aircraft that would generate unacceptable amounts of damage, while avoiding, as much as possible, restricting day-to-day operations. This is done via the FLIP chart PCN and ICAOs ACN/PCN method. If the design critical aircraft PCN defined earlier is chosen for the FLIP (Flight Information Publication) this will restrict day-to-day operations of the large aircraft. Alternatively, the highest tentative PCN from each of the aircraft categories regularly using the base can be chosen as the FLIP PCN. This ensures both control over the most damaging aircraft and little interference with operations and is the recommended alternative. In some cases, e.g. when the pavement is very weak but the traffic is very low, it is possible that the FLIP PCN reaches a high value. This only indicates that the pavement would last a long time; however, it could have the unintended consequence of allowing the landing of large aircraft that could result in excessive pavement damage or even exceed the pavement capacity. To prevent this, the FLIP PCN shall be limited to the largest ACN (using peace-time full-loading) of the aircraft regularly using the airfield.

d.One objective of the evaluation is to assess capability of the pavement to carry out its mission for the next 20 years. If the pavement is not up to par, only part of the 10-year mission will be completed. For pavement purposes, this mission consists of three components: aircraft weights to be supported, aircraft passes, and desired pavement life. Hence the reduction in mission can be accomplished in three ways: by reducing the aircraft weights (and keeping passes and expected life constant), by reducing the aircraft passes (and keeping the other two constant), or by realizing that at the current weight and passes the expected life will be shorter. The Navy decided that this last option was most adequate since it would not restrict day-to-day operations, hence results are typically shown in terms of pavement life expectancy, and urgency of repair for each inadequate feature. This information can be conveyed simply via a color structural condition map.

e.The airfield life pavement expectancy can be reported in form of a four-color structural condition map, where the colors represent:

B (BLUE) - Expected pavement life greater than 10 years
G (GREEN) - Expected pavement life less than 10 years
Y (YELLOW) - Pavement in need of structural repair/upgrade

R (RED) - Very weak or failed pavement, no aircraft recommended

Alternatively the colors can be interpreted as indicating the weight restrictions necessary (at the original level of passes) to ensure that the feature will last the projected 20 years:

B (BLUE) - No weight restriction
G (GREEN) - To be used only by half-loaded aircraft
Y (YELLOW) - To be used only by unloaded aircraft
R (RED) - Not recommended for aircraft traffic until upgrade.

Alternatively the colors could be interpreted as indicating the pass level restrictions (at the original weight) necessary to ensure that the Section will last the projected 10 years. Note that increases in pass levels up to 50 percent could typically be accommodated by blue areas without significantly affecting the pavement life.

f.The color structural condition map is found as follows. First, the PCN of the design critical aircraft is found. This PCN is then compared to the ACN values in Table 2-5. For the design critical aircraft, and the given pavement and subgrade type, the PCN can be compared to three ACN values corresponding to a loaded, half-loaded, and unloaded aircraft. Colors are determined from the comparison:

·If $ACN_{\text{fully loaded}} \leq PCN$, the color is blue
·If $ACN_{\text{half-loaded}} \leq PCN \leq ACN_{\text{fully loaded,}}$ the color is green
·If $ACN_{\text{empty}} \leq PCN \leq ACN_{\text{half-loaded,}}$ the color is yellow
·If $PCN \leq ACN_{\text{empty,}}$ the color is red

g.It should be noted that any airfield pavement evaluation can be viewed as a life expectancy prediction. As such, it will depend on both the current pavement status, and the projected traffic. If the actual traffic later varies significantly from the projected traffic, a new evaluation will be necessary. Within blue areas, small traffic increases are acceptable.

2-11 **SUMMARY OF ARMY EVALUATION REQUIREMENTS.** The required elements of an Army evaluation are as follows:

a. Conduct a condition survey and assign PCI values to each Section

b. Collect necessary data.

c. Determine allowable load for each Section based on using aircraft and day-to-day traffic.

d. Determine PCN values for each Section.

e. Assign overall PCN value for the airfield based on critical aircraft.

f. Recommend maintenance alternatives.

g. Where needed, calculate overlay thickness for planning purposes.

2-12 **SUMMARY OF AIR FORCE EVALUATION REQUIREMENTS**. The required elements of an Air Force evaluation are as follows:

a. Conduct a general or cursory survey of the airfield pavements and assign a qualitative rating to each Section. The cursory survey will validate a more detailed PCI survey conducted by the base, MAJCOM, or by contract.

b. Collect necessary data using NDT and/or direct sampling techniques.

c. Determine allowable load for each Section based upon the 14 aircraft groups and standard pass levels.

d. Determine PCN values for each Section based upon 50,000 passes of the C-17 aircraft.

e. Assign a PCN value to the airfield based on the 50,000 passes of the C-17 aircraft and the weakest runway Section in the first 1000 feet from each runway end (full width) or the runway keel (center 75 feet) of the interior portion of the runway.

2-13 **SUMMARY OF NAVY AND MARINE CORPS EVALUATION REQUIREMENTS.** The required elements of a Navy and Marine Corps evaluation are as follows:

a. Conduct a condition survey of the airfield pavement and assign a PCI rating to each Section. Alternatively, obtain most recent PCI survey from the corresponding Navy Engineering Field Division, Facility Engineering Command (FEC) or NAVFAC Atlantic.

b. Collect construction history data, previous core and boring data.

c. Conduct NDT of each Section using a FWD or HWD.

d. Determine actual traffic using the airfield and projected traffic for the next 20 years.

e. Determine tentative PCNs for each Section (one for each aircraft category present at the airfield).

f. Determine the structural condition color map.

g. Determine the design critical aircraft and required overlays.

h. Determine the FLIP PCN for each runway Section.

i. For each runway, the FLIP PCN is the lowest of the FLIP PCNs for each Section of that runway.

2-14 **FROST-CONDITION EVALUATION.** If the existing soil, water, and temperature conditions are conducive to detrimental frost effects in the base, subbase, or subgrade materials, then during a portion of the year the supporting capability of a pavement will be less than if the same conditions of soil and water existed in a nonfreezing environment. Where such conditions exist, follow the criteria and procedures for the evaluation of airfield pavements in seasonal frost areas presented in Chapter 7.

CHAPTER 3

DATA COLLECTION

3-1 **GENERAL**. The selection of representative physical characteristics of a pavement requires a thorough study of all existing information and, in most cases, will require additional tests at the time of evaluation. The evaluation may be based on design and construction control data, when these data are considered representative of existing conditions. This fact is especially true for relatively new pavements; however, additional tests are desirable for the evaluation of older pavements, or when there is reason to doubt the validity of the existing information. Tests required when construction data are not available and the sampling and testing methods for conducting these tests are discussed in Appendix B.

3-2 **SEGMENTATION**. Evaluating an airfield pavement can be complicated because there may be several different pavement types, complex pavement structures, a mix of traffic, etc., on the same airfield. To simplify the evaluation, the airfield is broken into segments. In the past, these segments were called “features”, basic units with generally the same characteristics. Because of an increased emphasis on managing assets, the Department of Defense issued *Department of Defense Guide for Segmenting Types of Linear Structures,* which provides guidelines for segmenting airfield pavements. These guidelines provide a common framework that enables pavement evaluation data to be shared with real property, asset management, and geospatial information systems. This will standardize data reporting so that Geographic Information Systems, Real Property Inventory Systems and Asset Management Systems can share data. Pavement Evaluation/ PAVER now uses Network, Facility, Branch, and Section to segment an airfield. Network equates to the entire airfield, Branch is associated with a facility, such as Runway, Taxiway, Apron, or Helipad. A Branch is further segmented into Sections based on type pavement, use, structure, construction history, traffic area, and condition. A PAVER/Evaluation segmentation hierarchy and schema is shown in Figure 3-1 and 3-2. Details on the pavements/real property relationships are contained in AFI 32-1041.

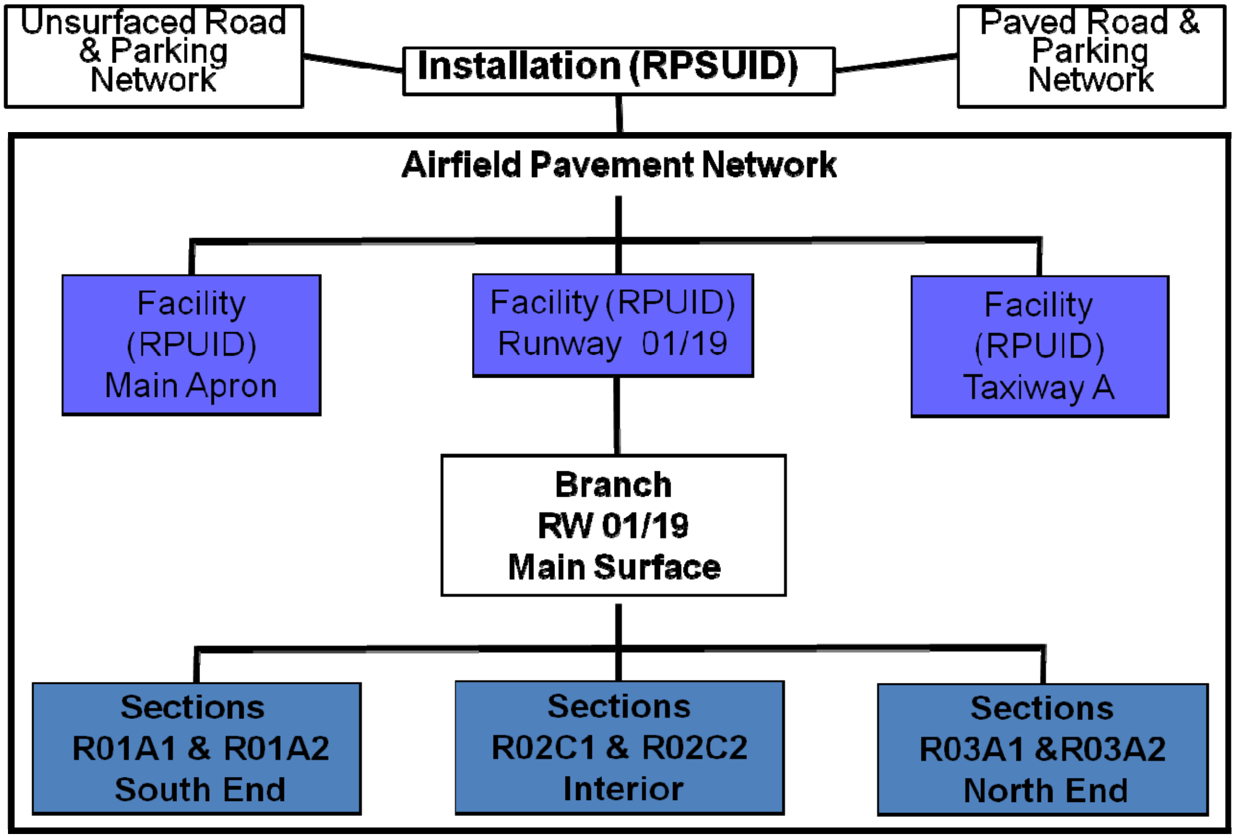
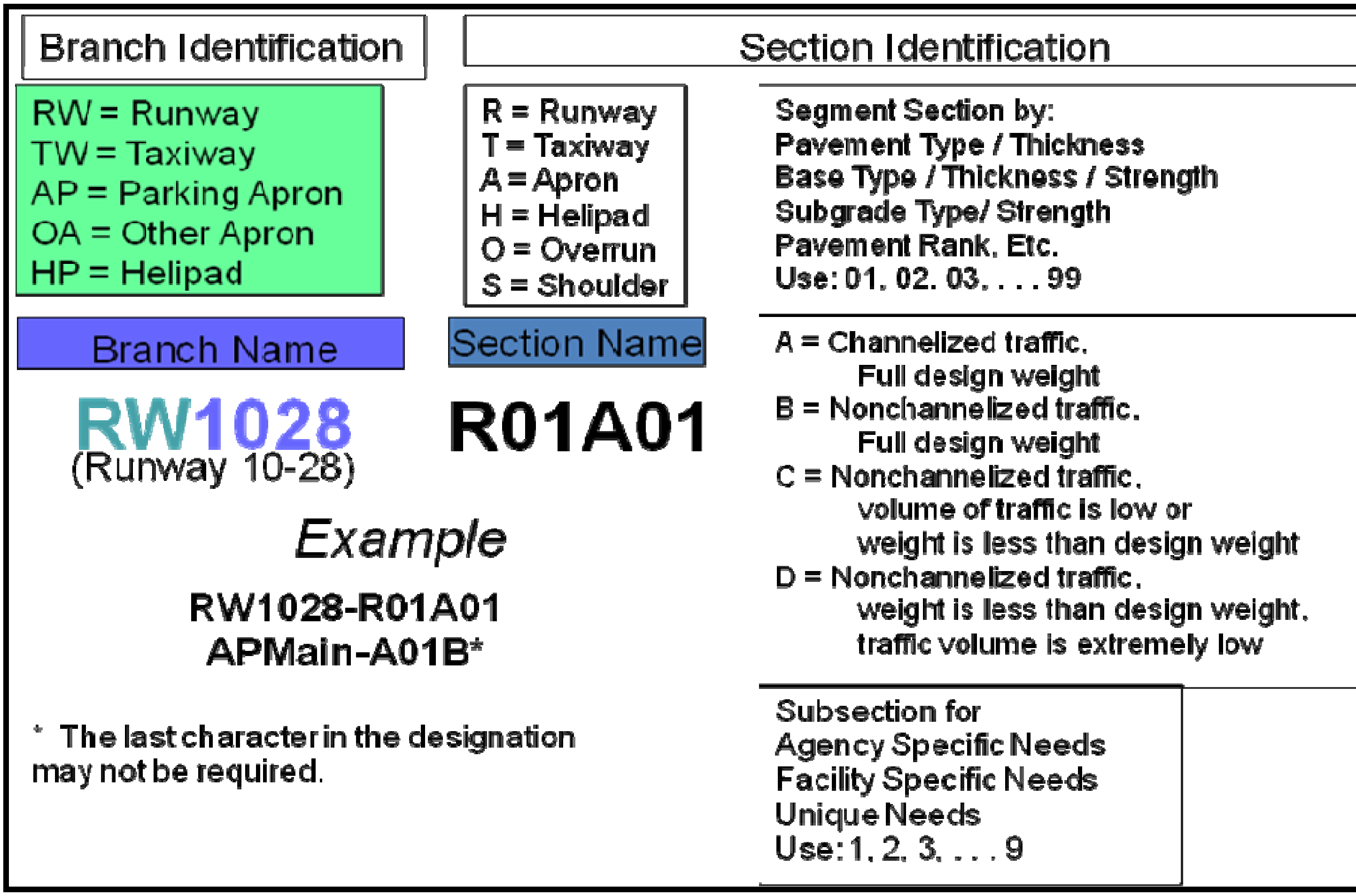


Figure 3-1. Segmentation Hierarchy



Branch Identification

RW = Runway
TW = Taxiway
AP = Parking Apron
OA = Other Apron
HP = Helipad

Branch Name

RW1028
(Runway 10-28)

Section Identification

R = Runway
T = Taxiway
A = Apron
H = Helipad
O = Overrun
S = Shoulder

Section Name

R01A01

Segment Section by:
Pavement Type / Thickness
Base Type / Thickness / Strength
Subgrade Type/ Strength
Pavement Rank, Etc.
Use: 01, 02, 03, . . . 99

A = Channelized traffic, Full design weight
B = Nonchannelized traffic, Full design weight
C = Nonchannelized traffic, volume of traffic is low or weight is less than design weight
D = Nonchannelized traffic, weight is less than design weight, traffic volume is extremely low

Example

RW1028-R01A01
APMain-A01B*

* The last character in the designation may not be required.

Subsection for
Agency Specific Needs
Facility Specific Needs
Unique Needs
Use: 1, 2, 3, . . . 9

Figure 3-2. Segmentation Schema

The following should be considered when identifying Branches and Sections:

Pavement Types. There are many pavement types: flexible, rigid, rigid or flexible overlay on rigid, flexible overlay on flexible, composite, reinforced rigid, and unsurfaced. A specific Section contains only one pavement type.

Pavement Use**.** Airfield pavements consist of runways, taxiways, aprons, shoulders, and overruns. Both a Branch and a Section are confined to a single pavement use. For example, a taxiway with an alpha designation that passes through an apron is a Branch separate and distinct from the apron pavements Branch. A taxi lane on an apron is considered part of the apron Branch although, if the taxi lane varies in thickness or other attribute, it may be broken out as a separate Section of the apron Branch. See UFC 3-260-01, *Airfield and Heliport Planning and Design*, for definitions.

Pavement Thickness**.** The actual pavement thickness usually varies considerably throughout an airfield system; however, each discrete pavement section has a constant nominal thickness that evaluators use as a representative thickness for the Section.

Construction History. In most cases, different contractors, using different materials and techniques, construct and maintain various portions of an airfield pavement system at different times. All pavements included in a specific Section have a consistent construction history. Subdividing sections that contain random replaced slabs or replaced asphalt areas involves engineering judgment. Generally, a section should be subdivided only when the replaced pavement is contiguous and comprises 25 percent or more of the existing section.

Pavement Rank. Pavement branches are assigned a rank; Primary, Secondary, Tertiary, or Unused to help define the importance of the structure in the asset management system (see definitions below). In instances where a facility has a portion that is a Primary pavement and a portion that is Secondary or Tertiary, break out branches according the rank. Aprons provide a good example. The main apron is a Facility. A portion of the apron supports the assigned flying mission, but half the apron is only used occasionally for air shows or overflow transient aircraft. In this case, the portion of the apron that supports the active mission is Primary and the remaining portion of the apron will have a separate branch that will be Tertiary. See definitions below:

Primary Pavements**.** Primary pavements are mission-essential pavements such as runways, parallel taxiways, main parking aprons, arm-disarm pads, alert aircraft pavements, and overruns (when used as a taxiway or for takeoff). In general only pavements that have aircraft use on a daily basis or frequently used transient taxiways and parking areas are considered primary.

Secondary Pavements**.** Secondary pavements are mission-essential but occasional-use airfield pavements, including ladder taxiways, infrequently used or transient taxiway and parking areas, overflow parking areas, and overruns (when used to test aircraft arresting gear). In general any pavements that do not have daily use by aircraft are secondary.

Tertiary Pavements**.** Tertiary pavements include pavements used by towed or light aircraft, such as maintenance hangar access aprons, aero club parking, wash racks, and overruns (when not used as a taxiway or to test aircraft arresting gear). Paved shoulders are classified as tertiary. In general any pavement that does not support aircraft taxiing under their own power or is used only intermittently is considered a tertiary pavement.

Unused Pavements**.** Unused pavements include any pavements that are abandoned or scheduled for demolition.

Traffic Areas**.** Airfield pavements are divided into traffic areas based on the lateral distribution of aircraft traffic and effective gross aircraft load. These areas are designated types A, B, C, and D, as described in UFC 3-260-02. Type D traffic areas are used in design, but generally not in evaluation. A Section will have only one traffic area designation. Branches may have multiple traffic designations. For example, the thresholds of a runway will be A traffic while the interior will typically be C traffic. As another example, the taxiway section to a wash rack may have a C traffic designation and the wash rack section itself would have a B traffic designation, even though they are both part of the same Branch (wash rack).

Pavement Condition. A discrete pavement area usually has consistent characteristics for each of the attributes. However, sometimes the condition of the pavement in a Section varies considerably. In this situation, the discrete pavement area can be subdivided into separate pavement Sections based on the surface condition of the pavement or the anticipated traffic the section may see. An example of this is the runway. The keel section of the runway sees a much higher volume of traffic than the outer portions. The keel section of the runway may be a Section separate from the outer portions. In general, only subdivide an apron or taxiway due to condition if the area involved is 25% or more of the total Section area and the PCI difference between the areas is at least 15 points. The objective is to only assign new Sections that significantly impact the results of the evaluation. Don’t subdivide small Sections.

Figure 3-3 illustrates a typical airfield segmented into Sections.

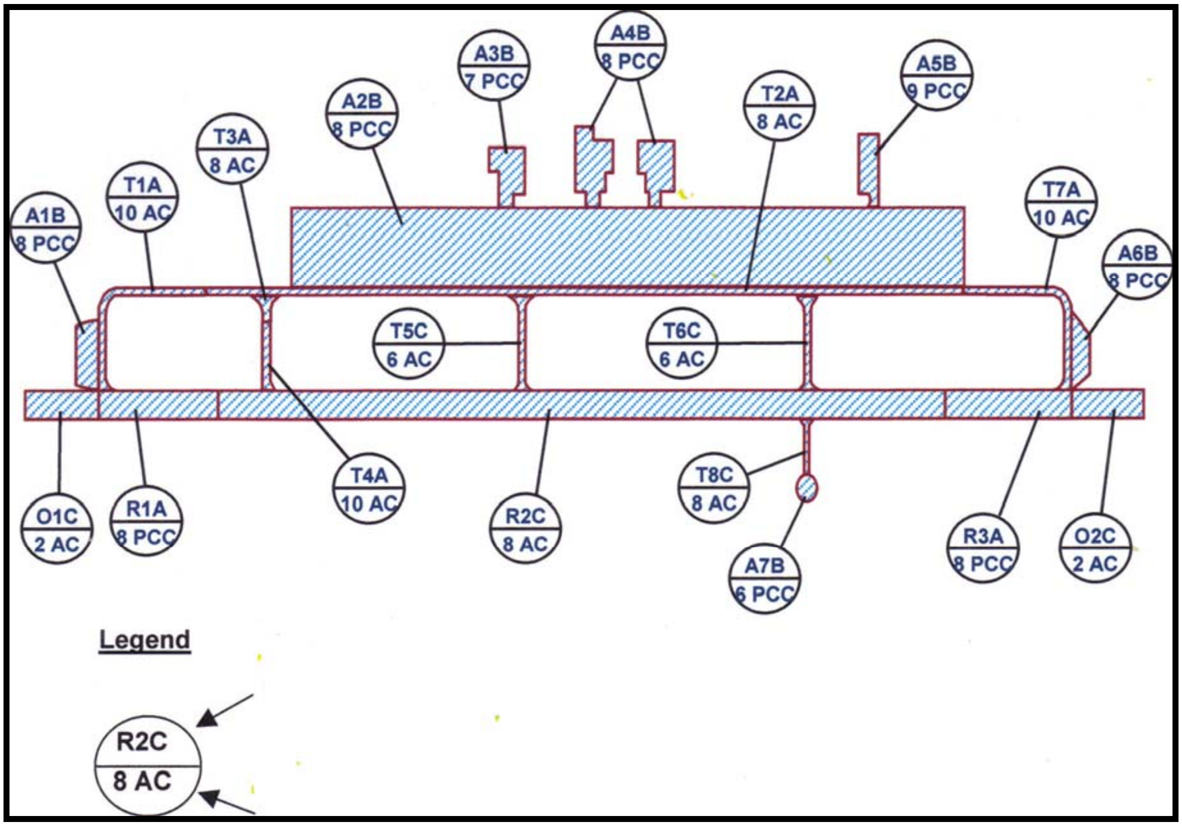


Figure 3-3. Typical Airfield Segmented into Sections

The following “Business Rules” should be followed to assist with standardization of airfield pavements:

Network. This is the name of the airfield. In most cases, it is the name of the base, but in some cases, there are multiple airfields on a base. Designating the Network is the responsibility of the base/command.

Branch. A Branch is a Runway, Taxiway, Apron, or Helipad. Designations are as follows:
Runway—RW (Runway 10/28 would be designated as RW1028. A Runway includes associated Overruns and paved Shoulders)

Taxiway—TW (includes associated paved Shoulders)

Parking Apron—AP (Includes associated paved Shoulders)

Other Apron—OA (Includes wash racks, arm-dearm pads, compass roses, power check pads, etc, and their associated paved Shoulders)

Helipad—HP (Includes associated paved Shoulders)

Sections. A Branch can be further segmented into Sections based on pavement type, thickness, strength, traffic area (A, B, or C), construction history, and condition. The Section can be subdivided based on unique user needs. Section designations are as follows:

Runway—R
Taxiway—T
Apron—A
Overrun—O
Shoulder—S
Helipad--H

Intersecting Pavements. When pavements intersect, the pavement surface is part of the highest in order of precedence. The order is:

Runways
Taxiways
Aprons
Helipads

For example, when runways cross, the resulting Section will be part of the Primary Runway. When a runway and taxiway intersect, the resulting Section will be part of the runway.

Dimensions and Areas. Dimensions and areas of each Section are recorded. Many areas will consist of length x width; others are more complicated and will have to be determined in the field.

3-3 **STUDY OF EXISTING DATA.** Existing data may be used to accomplish the evaluation or to supplement new data. In either case, all data available from previous tests made in connection with design, construction, repair, or earlier evaluations should be thoroughly studied. The performance of the pavement should be analyzed by means of traffic records, weather data, and the results of any previous condition surveys. In many instances, the existing data will indicate the uniformity of the material encountered and thus enable the scope of a test program to be established. The type of data that should be assembled and studied for this phase of the evaluation is discussed below. Where data are not available, outdated, or inadequate, testing will be required.

a. Subgrade, Subbase, and Base-course Strength. In many instances, it may be found that subgrade and base-course strength determinations were made for the pavement Sections (formerly features) during the initial construction period and that data may also be available from later tests. However, these tests may not be meaningful, since the strength will change with time. The exact locations of the tests should be determined by the evaluating engineer to properly assess the value of the information.

b. Pavement Thickness. Construction plans generally show pavement cross-sections for the various features of the airfield, including thickness, thickened edges, types of joints, and load-transfer devices. Surface thickness is confirmed by extracting and measuring cores from the existing pavement. Base and Subbase thickness and strength can be measured by the Dynamic Cone Penetrometer.

c. Concrete Flexural Strength (R). Construction control strength measurements can, in many instances, give a realistic picture of the uniformity or relative quality of the concrete in the various pavement Sections. Tests conducted during previous evaluation studies, when correlated with the construction-control tests, may also yield information of value, particularly in regard to strength change with time. Studies of this type may materially reduce the number of field tests necessary to establish the existing flexural strength on which the evaluation is to be based. Flexural strength can be obtained from concrete cores using the tensile splitting test and the correlation in Appendix B.

d. Condition of Existing Pavement. In most instances, recent condition-survey reports made in connection with special investigations can be obtained from the Geotechnical Laboratory, U.S. Army Engineer Research and Development Center, Vicksburg, MS, from the Air Force Civil Engineer Center(AFCEC/CO), Tyndall AFB, Florida, the appropriate regional Naval Facility Engineering Command (NAVFAC FEC) or NAVFAC Atlantic. Up-to-date maintenance records should be obtained for all pavements.

e. Subgrade, Subbase, and Base Course Physical Properties. Construction records generally contain soil profiles of the finished runway, taxiway, and apron sections and may also include results of soil-classification tests, moisture contents, moisture-density curves, and the seasonal position of the groundwater table for the subgrade soils. Modulus of elasticity in flexure of stabilized materials meeting the requirements outlined in UFC 3-250-11 *Soil Stabilization for Pavements* may also be found in construction records.

f. Physical Properties of Concrete. Results of field and laboratory tests to determine the physical properties such as slump, aggregate gradation, mix design, temperature, and curing of the concrete are generally available in construction records. Cores extracted to determine thickness and strength should be visually examined to determine any defects, such as ASR.

g. Physical Properties of Bituminous Pavements. Results of field and laboratory tests to determine the physical properties of bituminous pavements are generally available in construction records. Data should include aggregate test results including gradation, specific gravity and absorption, Los Angeles abrasion, soundness, percent natural sand, percent particles with two or more fractured faces in coarse aggregate, percent flat or elongated particles in coarse aggregate, fine aggregate angularity, and percent clay. Binder properties should include asphalt performance, viscosity, or penetration grade, type and percent of binder modifiers (if any), and specific gravity. Asphalt mixture properties should include compaction effort (number of blows of Marshall hammer or number of gyrations of Superpave compactor), percent binder by weight, density, air void content, voids in mineral aggregate, percent voids filled with binder, theoretical maximum density, tensile strength ratio, and wet tensile strength. Mixtures designed according to the Marshall method should include test results for Marshall stability and flow.

h. Nondestructive Test Data. Nondestructive test (NDT) data required include location of tests, deflection basins (the applied force and surface deflections at offset distances from the load) obtained utilizing NDT equipment and test procedures, and joint deflection data on rigid (and in some instances, composite) pavements.

i. Temperature Data. Temperature data are required for flexible pavements and pavements with a flexible overlay at the surface to include 5-day mean air temperature for the 5 days prior to testing, surface temperature at the time of testing, and average daily maximum and average daily mean air temperature for each month. For the evaluation of airfield pavements in seasonal frost areas, it would also be helpful to have monthly mean temperatures and monthly daily maximum and minimum temperatures for determining the thaw season.

3-4 COLLECTION OF NDT DATA.

a. Equipment. The NDT procedure evaluates response of a pavement system to an applied loading. Seven sensors are preferred. The number of layer moduli to be calculated from measured deflections cannot exceed the number of sensors. The outermost sensor (farthest from the load) shall be no less than 48 inches from the load, with the preferred minimum distance being 72 inches. Of the remaining sensors, one should be located at the center of the loaded area and the others at approximately 1-foot intervals from that point. The applied loading must be measured and must be accurate to at least plus or minus 2 percent of the expected load. Deflections must be determined at points on the pavement to describe a representative basin and must be accurate to at least plus or minus 2 percent. Most deflection measurement devices have four or more sensors to measure the deflection basin. Similarly, most deflection measurement devices have Sensor 1 at the center of load and the other sensors either at 1-foot intervals from that point or adjustable to any spacing out to a distance of 6 feet or more. The NDT device recommended for evaluation of military airfields is an impulse loading device commonly called a falling weight deflectometer (FWD) or Heavy Weight Deflectometer (HWD). The load on the pavement (impulse force) from an FWD or HWD is created by dropping weights from different heights onto a rubber or spring buffer system. The standard loading plates used to transmit the applied force to the pavement are either 12 inches or 18 inches in diameter. The drop height can be varied to produce an impact force up to 56,000 pounds depending on the model being used. The requirements for the test equipment and test procedures should be in accordance with ASTM D 4694. The FWD/HWD uses velocity transducers to measure the pavement response to the applied load. Deflections are obtained by integrating the surface velocity measured by the velocity transducers.

b. Testing. In this procedure, the response of a pavement system to an applied loading is characterized using deflection basin measurements. Since the time required to measure the deflection basin at each testing point is short (2 to 4 minutes), a large number of measurements can be made during the normal evaluation period. The various pavement configurations (Sections) and construction dates should be considered in the selection of NDT test locations. Thus, a thorough study of as-built pavement drawings and previous evaluation reports is particularly helpful in determining the testing program.

(1) Test Locations. On runways and taxiways, deflection basin measurements should be made every 100 feet on alternate sides of the center line along the main gear wheel paths. For flexible pavements, the offset is usually 10 to 12 feet from the center line. For rigid pavements, the tests should be performed at the center of the slab or largest unbroken piece. For apron areas, deflection basin measurements should be conducted in a grid pattern at 100- to 200-foot spacings. Additional tests should be made where wide variations in pavement response values are found. A minimum of three deflection basin measurements should be conducted on all pavement Sections. Figure 3-4 shows NDT test locations for a typical airfield.

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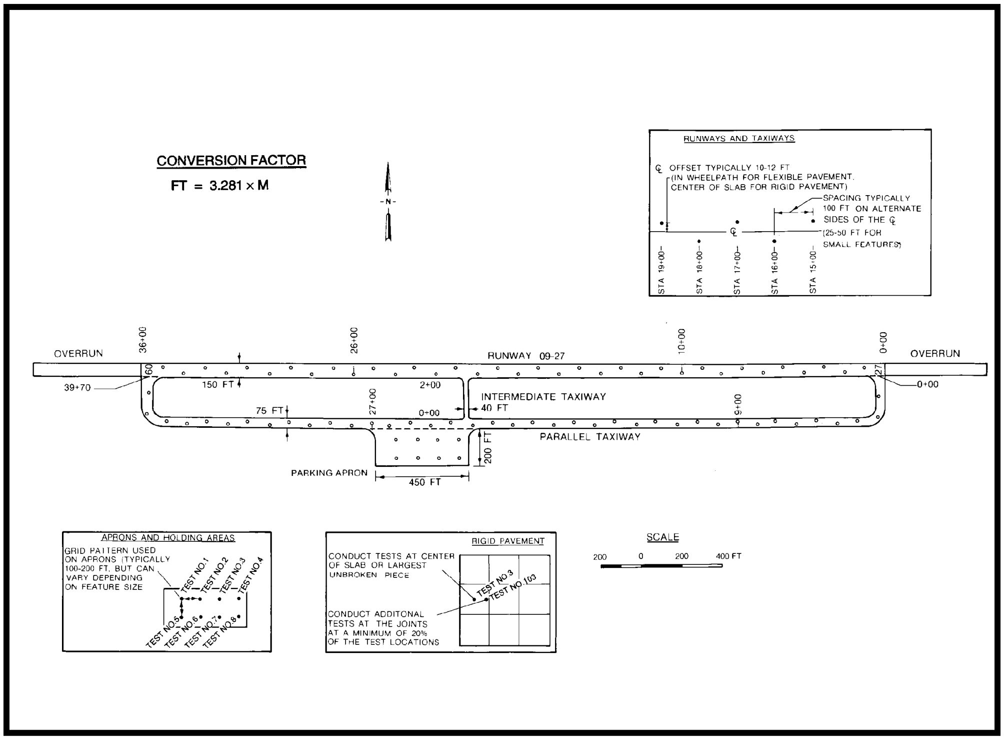


Figure 3-4. Typical NDT Locations

(2) Test Requirements. At each test location, the NDT equipment is positioned, a load is applied, and the resulting surface deflections at offset distances are determined. The magnitude of the loading will be largely dependent on the NDT equipment used, the type of aircraft for which the evaluation is being performed, and the pavement structure. The moduli of the subgrade and base-course materials are dependent on the applied stress level. NDT loading should be conducted at force levels near the single-wheel design load of the design aircraft. The decision to use the 12- or 18-inch load plate depends on the contact pressure produced by the design load. Tests should be performed with the plate that produces similar contact pressures as the design load. Only one deflection basin is required at each test location; however, for impulse devices, it is recommended that three repetitions be applied at a particular force level. The first loading is considered a seating load, and the results are disregarded. The second and third loadings should produce similar results. Results from the final loading should be used in the evaluation. If inconsistencies are observed in the third test sequence, the second load point can be used. Testing of flexible pavements with a thin asphalt surface is discussed in Appendix E.

(3) Joint Load Transfer. The ability of joints in PCC slabs to transfer load can be measured with an NDT device in the configuration shown in Figure 3-5. The ratio of deflections measured on each side of the joint is defined as the deflection ratio and is related to joint efficiency or load transfer. If there is a suspected problem

with load transfer, it is recommended that joint efficiency tests be performed on a transverse joint and the longitudinal joint nearest the wheel path at a minimum of 20 percent of the NDT test locations where PCC joint locations can be determined. Joint transfer tests should be performed early in the morning before the PCC slabs expand or a temperature gradient develops. Expansion, warping, and curling of PCC slabs due to changes in temperature can significantly affect the performance of joints. At low temperatures, the joint opening is presumably widest with less frictional resistance between slabs, and the load-transfer efficiency will be at a minimum. As the temperature rises, the joint tends to close or lock up, and the load-transfer capability approaches a maximum. If needed, reference point tests can be used to establish a relationship between air temperature and the deflection ratio from NDT such that adjustments can be made to test results collected over a wide range of temperatures. A reference slab can be selected within each Section to be tested on a given day. Joint tests can be conducted on each reference slab at 1- to 2-hour intervals throughout the testing period, or at closer intervals if the testing period is less than 4 hours on a given Section.

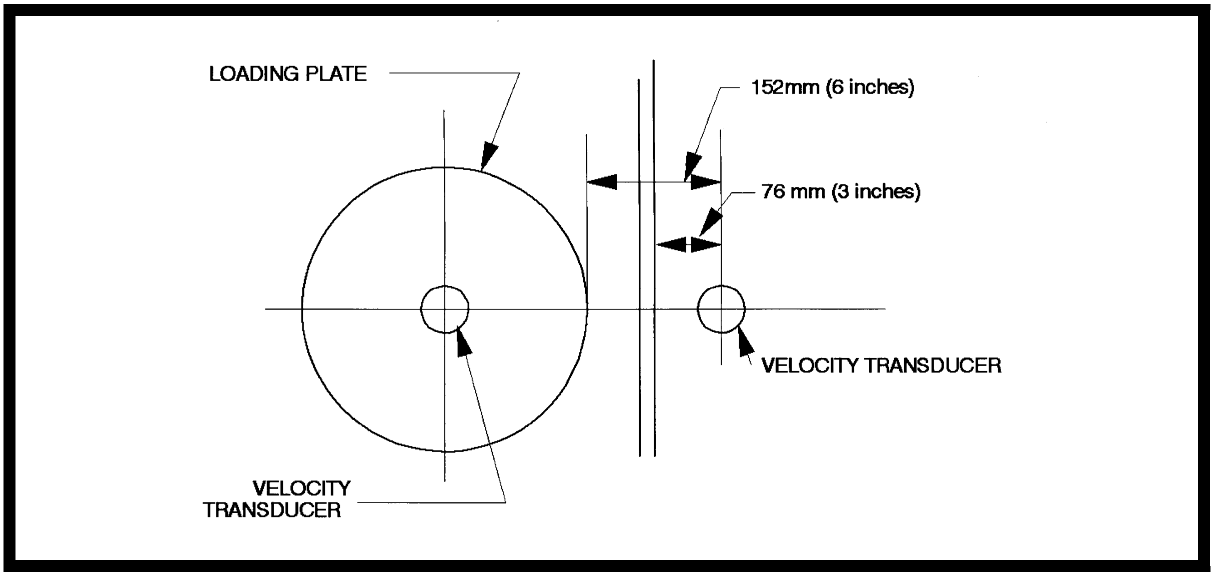


Figure 3-5. NDT Configuration for Determining Load Transfer

(4)Temperature Data, Bituminous Surface Layers. The modulus of bituminous concrete is temperature-dependent. The mean pavement temperature at the time of testing can be obtained by measuring the temperatures with thermometers installed 1 inch below the top, 1 inch above the bottom, and at mid-depth of the bituminous layer and averaging the values to obtain the mean pavement temperature. If actual temperature measurements are not available, the pavement temperature may be obtained by adding the measured pavement surface temperature at the time of test to the average (mean) air temperature for the 5-day period prior to the day of testing and obtaining the mean pavement temperature from Figure 3-6 or using the Climate module in the PCASE computer program (see paragraph 7-21 for instructions on using the Climate module). The latter is the more common practice and is recommended. The design air temperature is required for estimating a design pavement temperature and design modulus. The design air temperature for a particular locale is determined by averaging the average daily maximum temperature and the average daily mean temperature for the design month. Generally, the set of average temperatures will be necessary only for the hottest month indicated in the reporting period. Values based on records for the previous 20 to 30 years should be chosen, if available. These data can be obtained from records of the National Oceanographic and Atmospheric Administration for the particular locale or that nearest to it. With the design air temperature, the estimated design pavement temperature can be determined from Figure 3-7.

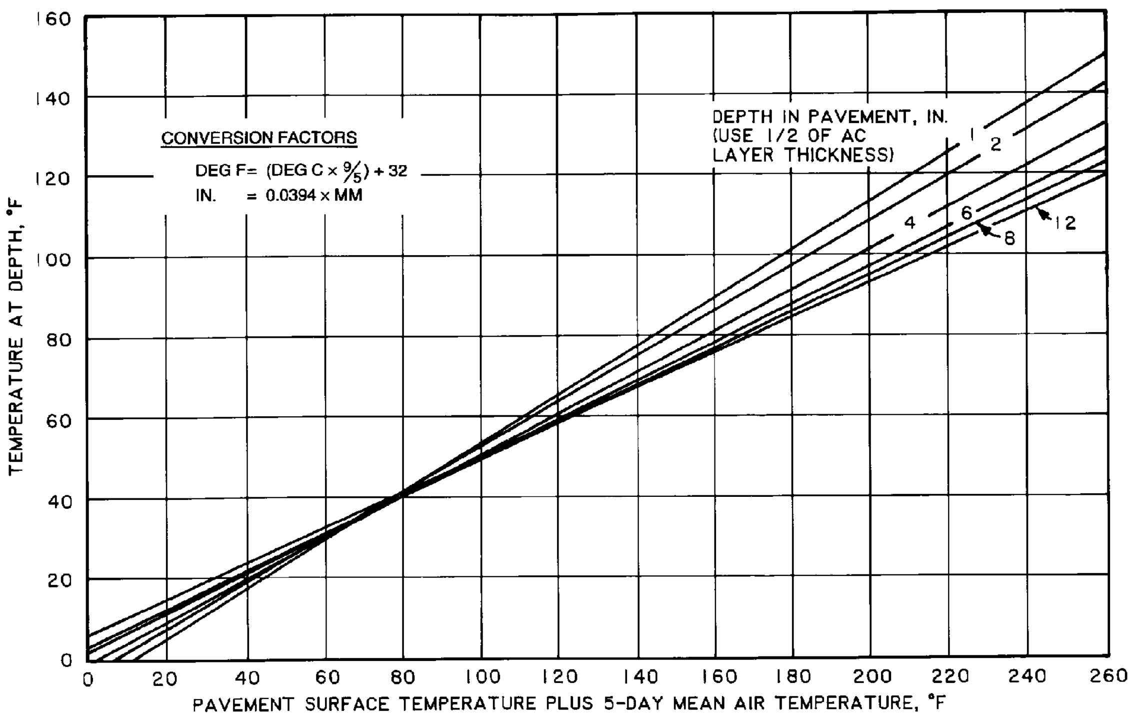


Figure 3-6. Procedure for Determining Mean Pavement Temperature

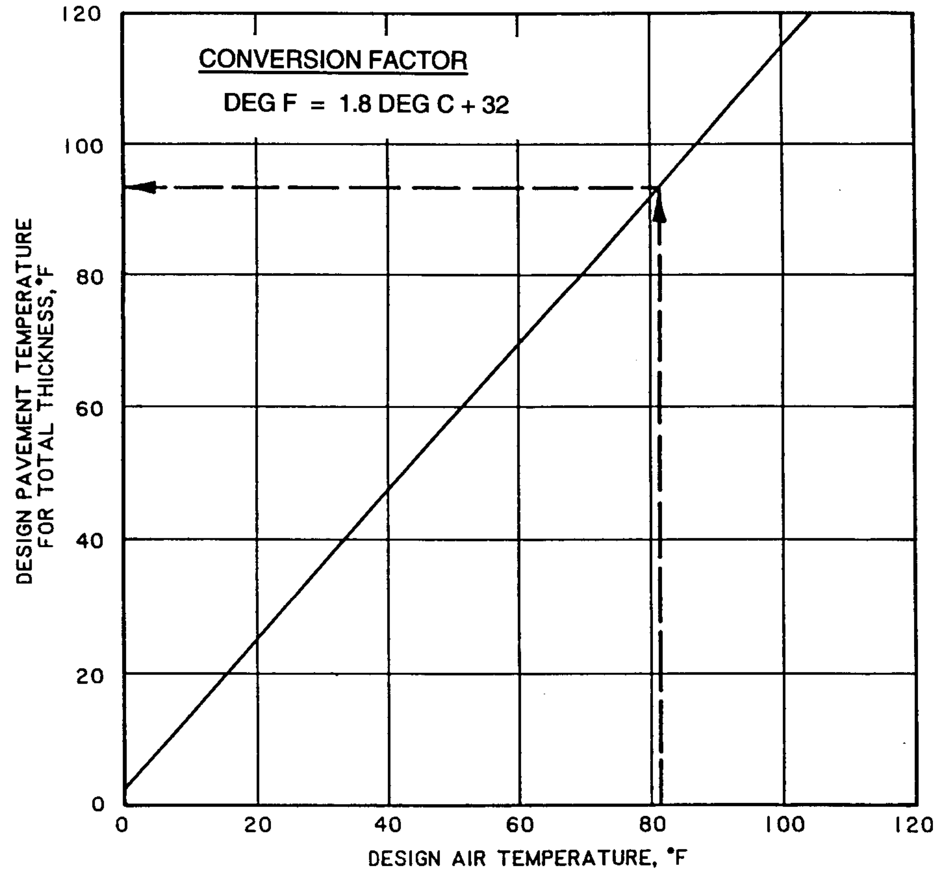


Figure 3-7. Procedure for Estimating Design Pavement Temperature

3-5 DATA COLLECTION USING DIRECT SAMPLING.

a. General. The type of data needed and the scope of the testing program to obtain these data depend on such factors as the amount and validity of existing data, the type of pavement being evaluated, and the condition of the pavement, and thus will be based largely on the judgment of the evaluating engineer. The condition survey is conducted, then test locations are selected, in-place tests made, samples for laboratory tests secured, and test holes back-filled. The laboratory tests are the final phase in the procurement of data. When NDT test data are obtained prior to direct sampling, the selection of the direct sampling locations will be tailored to match the results of the NDT data. Areas exhibiting a high degree of variation in the deflection measurements should be investigated as should areas exhibiting above average deflections.

b. Selection and Size of Test Areas. One of the first steps in the selection of sampling locations should be the establishment of longitudinal profiles along the runways, taxiways, and aprons to develop a general picture of subgrade, base, and pavement condition, so that test pits or core locations for collecting more detailed data can be located to the best possible advantage. Test pits are seldom used due to impact on the mission. Data for these profiles can be obtained by coring 4- or 6-inch- diameter holes in the pavement, through which thickness measurements can be made, samples of the foundation materials obtained, and DCP tests conducted. These samples should be classified in accordance with the Unified Soil Classification System as presented in ASTM D 2487. Usually, a spacing of 500 to 1,000 feet between these small holes will be sufficient, but occasionally when non-uniformity of pavement or foundation conditions exists, closer spacing may be necessary. From the information obtained, the pavements should be divided into Sections on the basis of pavement type, use, construction history, known strength, thickness, and foundation types.

(1) The preliminary sampling locations should enable test pits or Dynamic Core Penetration (DCP) tests to be placed in locations representing typical pavement and foundation conditions. In addition, the tests should be conducted in problem areas and areas that received intense traffic, that is, at or near the centers of runways, taxiways, or aprons instead of along the edge of the pavement.

(2) If pavement and foundation conditions are uniform throughout the airfield area, fewer tests will generally be required, if they are located to provide representative information for the entire system of airfield pavements. When the pavement or foundation conditions are not uniform, tests should be located to yield the necessary information for each type of pavement or foundation material. When failed areas or areas of excessive pavement distress are encountered, a sufficient number of tests must be located in the failed or distressed areas to determine the cause of the failure or distress.

(3) If test pits are used, the size of the test pits for rigid pavements will, in part, depend on the thickness of the pavement. Inasmuch as beams for flexural strength tests may be cut from the concrete specimen and removed from the slab, the length of the specimen must be greater than three times the pavement thickness, except when 6- by 6-inch beams are cut from the top and bottom of the slab for a three-point beam test. Since plate-bearing tests on the foundation materials will require the use of a 30-inch- diameter plate, test pits should be 4 by 5 feet to allow access to the foundation materials for testing and sampling. Tensile splitting tests are acceptable for computing flexural strengths and will require 6-inch-diameter core samples. An equation for calculating flexural strength from tensile splitting strength is presented in Appendix B.

(4) If used, test pits for flexible pavements (approximately 4 feet wide by 5 feet long) or core holes (up to 8 inches in diameter) are dug through the pavement to permit

the performance of in-place tests and to obtain samples for laboratory tests. CBR tests conducted in a core hole are referred to as small aperture testing. Core holes for DCP tests can be smaller. Core holes up to 5 inches in diameter do not create an operational problem for most aircraft, but a 4- by 5-foot test pit does. The same data are required for evaluation whether they are obtained from a test pit or from a core hole. A description of the general condition and a visual classification of materials from each test pit or core hole should be recorded. The thickness of the pavement should be measured to the nearest 1/4 inch and the total thickness of base and pavement to the nearest 1/2 inch. Several measurements should be made around the sides of the test pit or core hole to obtain representative thickness values. Each soil course should be described, giving color, in situ conditions, texture, and visual classification. References for testing and sampling procedures are given in Appendix B.

c. In-Place Tests for Rigid Pavements.

(1) Thickness Measurements. The thickness of all layers above the subgrade in all types of rigid pavements should be measured including base course, concrete slab, and all overlays. Thickness of the layers should be measured to the nearest 1/4 inch.

(2) Modulus of Soil Reaction.

(a) Rigid Pavements. The modulus of soil reaction of the subgrade or base course can be determined by the plate-bearing test as discussed in Appendix B. The plate-bearing test should normally be conducted on the surface of the material immediately beneath the pavement, that is, on the base course or on the subgrade, if there is no base course. When the plate-bearing test cannot be conducted, an approximate value of k can be determined by determining CBR values of each layer in the pavement structure using the DCP and determining the k value using the Effective k procedure outlined in Chapter 6. Subgrade or base-course materials that have been stabilized to the extent that they qualify as stabilized layers as outlined in UFC 3-260-02 require tests other than plate-bearing to determine their effect on the supporting value of the pavement structure.

(b) Rigid Overlay on a Flexible Pavement. When an evaluation is being made of a rigid overlay on a flexible pavement, the plate-bearing test will be performed on the surface of the flexible pavement, since the flexible pavement is considered to be a base course.

(c) Composite Pavements. When a composite pavement is being evaluated, the plate-bearing test will be performed on the surface of the nonrigid portion (bituminous concrete or flexible overlay) of the pavement provided the nonrigid portion of the pavement is 4 inches or more in thickness. In this case, the rigid base pavement and the nonrigid overlay pavement are considered to be base-course materials. When the plate-bearing test is performed on the surface of a flexible pavement or nonrigid-type overlay, both the test and k values are subject to certain limitations as discussed in the paragraph titled Rigid Overlay of Flexible Pavements in Chapter 6. If the nonrigid portion is less than 4 inches, the nonrigid portion and the base rigid pavement is removed and the plate bearing test is performed on the underlying base or subgrade.

(3) Percent Steel. For reinforced concrete pavements, the diameter and spacing of the steel in both the longitudinal and transverse directions should be measured.

(4) Field In-place CBR Tests. To evaluate a nonrigid overlay on rigid pavement, field in-place CBR tests may be required on the foundation materials in addition to plate-bearing tests. When the k value of the foundation material is greater than 200 pci or the concrete flexural strength is less than 400 psi, a higher load-carrying capability may be obtained for the nonrigid overlay or rigid pavement by using the flexible pavement evaluation procedure and assuming the rigid pavement to be a high-quality base-course material. When either of these conditions prevail, in-place CBR tests should be conducted on the foundation materials in addition to the plate-bearing tests. The in-place CBR tests must be conducted on both the base-course materials (if any) and on the subgrade in the same manner as in tests for the evaluation of flexible pavements

(5) Penetrometer Tests. Penetrometer tests can be used to determine the load-bearing capacity of subsurface pavement layers. The Dynamic Cone Penetrometer (DCP) is typically used on military airfields. Detailed test procedures and correlations for using the DCP and automated DCP are provided in Appendix B. The DCP or automated DCP is adequate for most pavement structures and is easy to deploy and implement. The DCP is a portable penetrometer device designed to penetrate soils to a depth of 39 inches. The DCP can be hand-held or automated. The 0.79-inch-diameter 60-degree cone is driven into the ground by raising and dropping a 17.6-lb hammer. Data is collected in terms of penetration per hammer blow, termed the DCP index value (mm/blow). The index can then be correlated to CBR using derived relationships. For testing rigid pavements, a 1-inch (DCP)-diameter hole is drilled through the portland cement concrete (PCC) until the top of the base or subgrade is encountered. A 4-6 inch hole is normally drilled for the automated DCP, using the on-board drill. The test device is then lowered to this point to begin the test sequence.

The ADCP equipment is identical to the standard DCP (Figure 3-8), with the addition of an automated mechanical hammer elevating system and an automatic data collection computer. The four main components of the DCP are the cone, the rod, the anvil, and the hammer. Energy is applied to the cone tip, through the rod, by dropping a 17.6-pound hammer a distance of 22.6 inches against the anvil. The diameter of the cone is 0.16 inch larger than that of the rod to ensure that only tip resistance is measured. By recording the number of hammer blows necessary to advance the cone into the soil, the soil strength is quantified in terms of a DCP index. The DCP index is the ratio of the depth of penetration to the number of blows of the hammer and has been empirically correlated to the California Bearing Ratio (CBR) and modulus of subgrade reaction (k). Figure 3-9 is a photo of the ADCP vehicle used for testing the underlying soils.

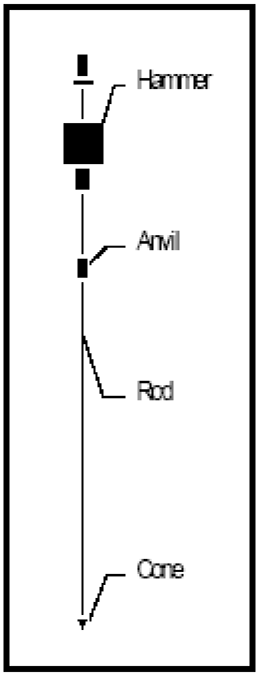


Figure 3-8. Schematic of DCP

Figure 3-9. Photo of ADCP

(6) Pavement Coring. Pavement coring is a vital part of the airfield pavement evaluation for three reasons. First, the cores are measured to verify the pavement thickness. Second, coring provides access to the subsurface layers for sampling and testing with equipment such as the ADCP described in this section. Last, extracted PCC cores are tested to determine the flexural strength of the pavement. Figure 3-10 shows a typical coring operation with a core drill. Six-inch or 4-inch diameter, diamond-tipped coring barrels are used to cut through both AC and PCC pavements. This type of pavement coring system is capable of cutting through pavements to depths of approximately 36 inches. All cores should be visually inspected in the field for evidence of defects.

Figure 3-10. Typical Coring Operation

(7) Field Density Tests. Density tests can be made on the base-course and subgrade materials. If the base course or subgrade is composed of granular materials, the most satisfactory methods of obtaining the density are by the sand-displacement or balloon methods, which are described in ASTM D 1556 and ASTM D 2167, respectively. If the subgrade is composed of a fine-grained cohesive material, the density can be best obtained either by drive-sampling (ASTM D 2937) or balloon methods (ASTM D 2167) or by the undisturbed sampling that may be required in connection with the plate-bearing test. All field density tests should be conducted adjacent to the area that was loaded during the plate-bearing test. When the overlay portion of a nonrigid overlay on rigid pavement is composed of a bituminous concrete and base course, density tests should be made on the base-course portion of the overlay.

d. In-place Tests for Flexible Pavements.

(1) Moisture-Content Determinations. The strength of base courses composed of substantial portions of fine materials is governed by the moisture content of the fine fraction. The fine fraction is that portion passing any of several sieve sizes ranging from No. 200 to No. 4. For the purposes of this document, material passing the No. 40 sieve has been selected as the critical portion. This is the same sieve on which separations are made for liquid and plastic limit determinations. The moisture content of both the material passing the No. 40 sieve and the total sample should be determined and shown in the tables of test data. If it is impractical to separate the material at the No. 40 sieve without affecting the moisture present, an absorption test following ASTM C 127 should be performed.

The percentage of absorption thus determined can be considered the moisture content of the coarse fraction, permitting arithmetic determination of the moisture content of the remainder (assuming all other moisture to be in this finer fraction). An indication of the stability of the base-course material can be obtained by comparing the moisture content of the material passing the No. 40 sieve with the liquid limit of the material. If the moisture content is near the liquid limit, the material can be considered unstable. Should the moisture content exceed the liquid limit, the base material will be very unstable if appreciable percentages of fines are present.

(2) CBR Tests. Considerable judgment must be used in selecting test locations in the test pit. In selecting test locations in the pit, the CBR piston should be placed so that the surface to be penetrated represents an average condition of the surface being tested and should not be set on unusually large pieces of aggregate or other unusual materials. It is also general practice to space the CBR tests in the pit so that the areas covered by the surcharge weights of the individual tests do not overlap. These tests should be performed on the surface and at each full 6-inch depth (especially if a strength problem is suspected) in the base and subbase courses, on the surface of the subgrade, and on underlying layers in the subgrade as needed. Density and moisture-content determinations should be made in the subgrade at 1-foot intervals to a total depth of 4 feet below the surface of the subgrade. The results of the density and moisture tests at these depths should be used to ascertain whether there is a need for additional CBR tests. The tests should be so located in the pit that the density determinations are performed between adjacent CBR tests. Three in-place CBR tests in test pits should be performed at each elevation tested. However, if the results of these three tests do not show reasonable agreement, three additional tests should be made. A reasonable agreement between three tests where the CBR is less than 10 permits a tolerance of 3; where the CBR is from 10 to 30, a tolerance of 5; and where the CBR is from 30 to 60, a tolerance of 10. Above a CBR of 60, variations in the individual readings are not as import. For example, actual test results of 6, 8, and 9 are reasonable, and their average is 8; results of 23, 18, and 20 are reasonable, and their average is 20. If the first three tests do not fall within this tolerance, then three additional tests are made at the same location, and the numerical average of the six tests is used as the CBR for that location. Generally, CBR values below about 20 are rounded off to the nearest point; those above 20 are rounded off to the nearest five points. A moisture-content sample should be obtained at the point of each penetration.

(3) Density Determinations. Three density determinations should be made at each elevation tested if samples of about 0.05-cubic-foot volume are taken; if somewhat larger samples are taken, the number of density determinations may be decreased to two. If a reasonable agreement is not found between the test results, two additional tests should be performed. A reasonable agreement is

considered to provide for a tolerance of about 5 pounds per cubic foot wet density. For example, test results of 108, 111, and 113 pounds per cubic foot wet density are in reasonable agreement, and their average is 111 pounds per cubic foot.

3-6 **SAMPLES**. Samples of the pavement, base course, subbase course, and subgrade materials are required for laboratory testing; the size of the samples depends on the type of laboratory tests to be made.

a. Rigid Pavement. All concrete cores obtained during the preliminary testing and all test specimens cut from the test pits should be retained for laboratory tests. The test specimens should be slightly more than three times as long and three times as wide as the pavement thickness, except when 6- by 6-inch beams are cut from the top and bottom of the specimens for three-point load beam tests. Flexural strength can be determined in the field by conducting tensile splitting tests on six-inch diameter cores.

b. Base and Subbase Courses Under Rigid Pavements. Bag samples of base and subbase courses underlying rigid pavements will be required for classification and compaction tests. The size of the sample will depend on the amount of large aggregate in the base course. In general, a 200-pound sample is sufficient. However, if laboratory CBR tests are necessary, which may be the case in the evaluation of a nonrigid overlay on rigid pavements, the size of the base-course sample should be about 600 pounds.

c. Flexible Pavement. Samples of typical pavement, base, subbase, and subgrade materials should be obtained for laboratory tests. The base and subgrade samples should be taken in a manner that will assure representative materials. Sampling methods are discussed in UFC 3-260-*02 Pavement Design for Airfields*. The samples to be obtained from the various materials are summarized in the following tabulation:

Material	Samples Per Pit	Remarks
Pavement	8 cores, 200 pounds of chunks	Chunks should be 8-10 inches in minimum dimension to permit separation of courses
Base and subbase Courses	600 pounds	Disturbed sample
	3 samples	Undisturbed cylinders to be taken of material with plastic fines where applicable
Subgrade	450 pounds	Disturbed sample. Increase to 600 pounds if much coarse material is present
	3 samples	Undisturbed cylinders

d. All-Bituminous Concrete and Flexible Overlays. Sampling of the bituminous concrete and base-course material in all-bituminous concrete and flexible overlays will be performed as described above for the pavement and base courses of flexible pavements. An exception is made when the all-bituminous concrete or flexible overlay exists between two thicknesses of rigid pavement (composite pavement). In this case, only one or two chunk samples of the bituminous concrete are needed from each test pit, since the only test necessary on the bituminous concrete portion of the overlay is an extraction test to determine the gradation of the aggregate and the bitumen content. Likewise, it will only be necessary to obtain a large enough sample of the base-course portion of the flexible overlay for a gradation test.

e. Subgrade. Bag samples and undisturbed samples of the subgrade may be required. If the subgrade is composed of a fine-grained material, a 100-pound bag sample will be sufficient; if the subgrade is composed of a granular material, a 200-pound bag sample should be obtained. However, if laboratory CBR tests are required, which may be the case in the evaluation of a nonrigid overlay on rigid pavements, the bag samples of subgrade material should be increased to 450 and 600 pounds for fine-grained and granular materials, respectively.

3-7 **LABORATORY TESTS REQUIRED**. Laboratory tests are necessary to classify the various pavement materials and establish their strength characteristics. These tests are outlined in the following subparagraphs and the test methods are presented in Appendix B. Laboratory test data may also be available from design and construction records.

a. Rigid Pavement. Normally, samples of the rigid pavement should be used to determine the flexural strength of beams or splitting-tensile strength of cores. Also, samples of the concrete should be visually examined to determine the type of aggregate and to estimate the maximum size of aggregate. This test can be performed in the field with a portable tester similar to the one described below:

Pavement evaluation teams can measure concrete flexural strength by correlation from a split-tensile test. In years past, this test was conducted in the laboratory. In recent years, the split-tensile test equipment has been modified and is now portable, allowing field testing of concrete core samples at the airfield. This, in turn, speeds the calculation of allowable loads. The split-tensile test is depicted in Figure 3-11. PCC cores are tested for strength by tensile splitting in accordance with standard practices.

Figure 3-11. Portable Split Tensile Tester

A vertical compressive load is applied to the side of the core sample. The cylinder is loaded at a constant rate until the specimen fails in tension across the diameter of the core sample from stresses induced by the compression load. The photo shows a failed specimen following a split-tensile test. The maximum load at failure is recorded, and measurements of the diameter and the length of the core are taken. The flexural strength is then calculated using this empirically developed relationship (WES, 1974):

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

In this equation, f is the flexural strength (pounds per square inch), p is the applied load (pound-force), and l and d are the length and diameter of the sample (inches), respectively.

b. Flexible Pavement.

(1) Where a flexible pavement consists of more than one course, the cores obtained for testing should be split at the interfaces of the various courses so that each course can be tested separately. The cores of each course should be tested in the laboratory for Marshall stability, flow, percentage of asphalt by weight, penetration of bitumen, aggregate type, shape and gradation, specific gravity of bitumen and aggregate, and density (CRD-C 649). If the pavement were designed according to Superpave criteria, the cores of each course should be

evaluated for percentage of asphalt by weight, aggregate gradation, and specific gravity according to AASHTO specifications which govern the placement of SHRP mixtures. The voids in the total mix and the percentage of voids filled with asphalt should be computed from the test results (CRD C-650, AASHTO Specifications for SHRP mixtures).

(2) Portions of the chunk samples should be used for determination of aggregate gradation, specific gravity of bitumen and aggregate, and penetration, ductility, and softening point of the bitumen. Other chunk samples should be recompacted as described in appendix B, and the recompacted specimens should be tested for Marshall stability, flow, and density. Their voids relations should also be computed. The stability of the cores cut from the pavement will often be lower than that of the recompacted sample. A part of this difference usually is due to differences in density, since the field cores seldom have density as high as the laboratory-compacted samples. The major part of this variation in stability is attributed to differences in the structure of the field and laboratory samples and also to the fact that the asphalt hardens some during reheating. Since the stability value is not the sole criterion for the evaluation of the mix, the lack of correlation between the stability of the field and laboratory samples is not particularly significant.

(3) No standard tests have been developed to determine resistance to spillage. However, a small amount of jet fuel should be spilled on one of the chunks from each test pit to see if the fuel penetrates the samples quickly or if it “puddles” on the surface.

(4) When the nonrigid overlay is between two thicknesses of rigid pavement, the only tests required are those to establish the gradation and bitumen content of the bituminous concrete and the gradation of the base-course material, if any.

c. Flexible Pavement Base Course, Subbase Course, and Subgrade. Classification data consisting of Atterberg limits, gradation, dry soil color, and specific gravity should be obtained from design and construction-control tests or from tests performed on samples of base course, subbase, and subgrade materials. Moisture-density and CBR relations should be determined from available data or from samples of base course, subbase, and subgrade materials remolded at three compaction efforts as described in CRD-C 653 and CRD-C 654.

d. Rigid Pavement Base Course and Subgrade. Classification data including gradation, Atterberg limits, specific-gravity and moisture-density relations should be established. For the evaluation of a nonrigid overlay, on rigid pavements, the moisture-density/CBR relation may be required. Undisturbed samples of the subgrade will be subjected to an adaptation of the consolidation test to determine the correction for saturation of the plate-bearing test results. The undisturbed samples may also be used for density determinations. For the evaluation of a nonrigid overlay on rigid pavement, soaked laboratory CBR tests on undisturbed samples of the subgrade material may be required.

3-8 **VISUAL SURVEY**. If possible, a detailed visual survey in accordance with ASTM D 5340, Airport Pavement Condition Index Surveys, should be performed before or during the evaluation. For the Army, AR 420-1 requires the condition survey data to be loaded into MicroPaver. If not possible to do a detailed survey, a cursory visual survey should be performed to validate condition. This visual assessment is not as detailed as outlined in the ASTM standard; however, the pavements are categorized in general terms based on this guidance. Pavement condition ratings range from 100 (new) to 0 (unsafe for aircraft operations). Simplified PCI ratings based on these PCI values are GOOD (PCI 100 to 71), FAIR (PCI 70 to 56), and POOR (PCI equal to or less than 55). A pavement surface may rate GOOD (PCI 70 to 100) but have underlying pavement or soil conditions that could result in pavement failure under the applied load of a given aircraft. On the other hand, a pavement may be structurally sound, but the surface condition may be hazardous for aircraft traffic (e.g., FOD). The pavement condition rating scale used in this type of analysis is shown in Figure 3-12 and is described in more detail in Table 3-1.

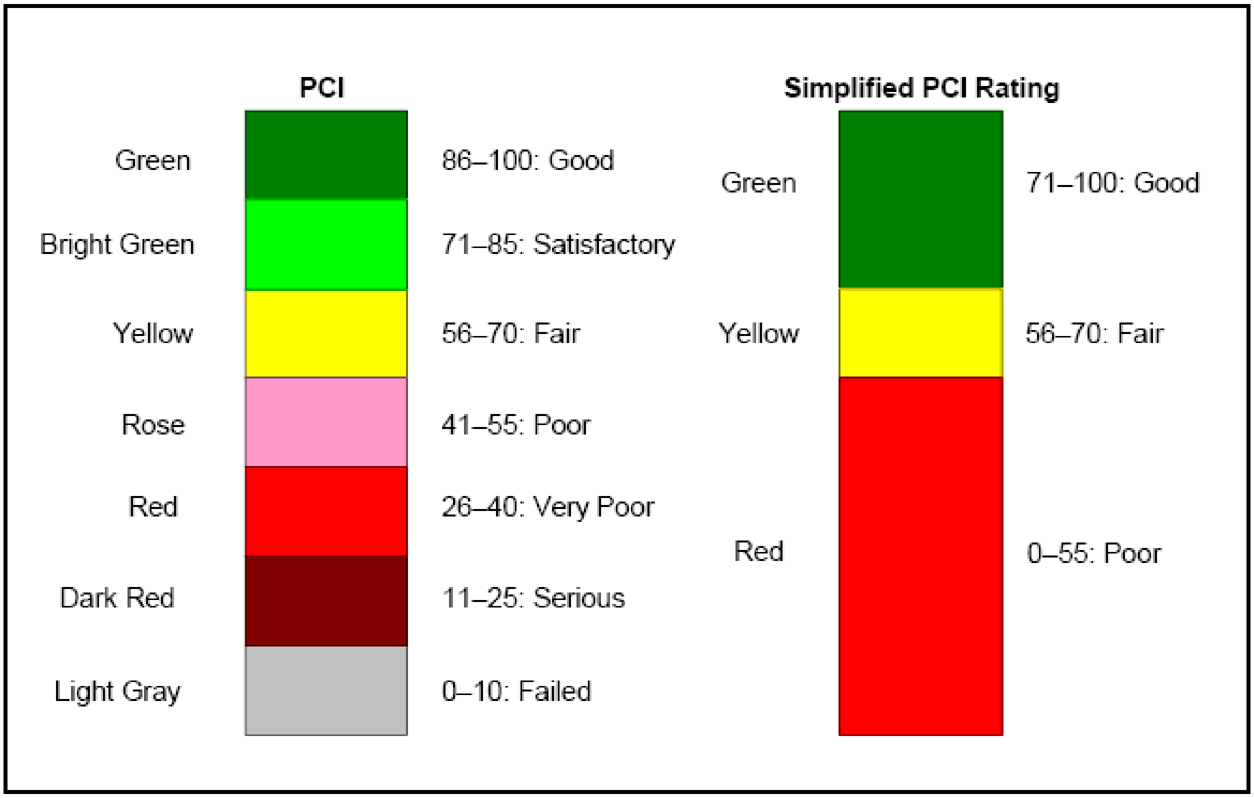


Figure 3-12. PCI Scales

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

Table 3-1. Definition of PCI Ratings

It is important to monitor and track the surface condition of pavements to identify pavement problems early and plan appropriate repairs. A continual evaluation program can also help determine the most cost effective maintenance and repair actions. Many pavement owners, such as cities, highway departments, and airports, use the PCI scale as a means to program maintenance and repair spending. The owners establish one PCI threshold that triggers maintenance action, a second PCI level that triggers repair, and possibly a third that triggers reconstruction. This is based on the theory that the rate of deterioration of the surface condition increases as the pavement ages. This is best shown in Figure 3-13, where the curve of PCI versus time drops rapidly as a pavement reaches over 50 percent of its original life span. By visualizing surface condition deterioration in this manner, one can see that the reported PCI indicates much more than a single number; it identifies the pavement's current stage in its life span. Maintenance activities are generally recommended for pavements that rate GOOD, where the cost is lower. If the owner waits until the pavement rates FAIR, the costs will far exceed those of routine maintenance, and some heavy repair may be required. This is obviously the more expensive option. Reconstruction is generally the only option to restore pavements rating POOR.

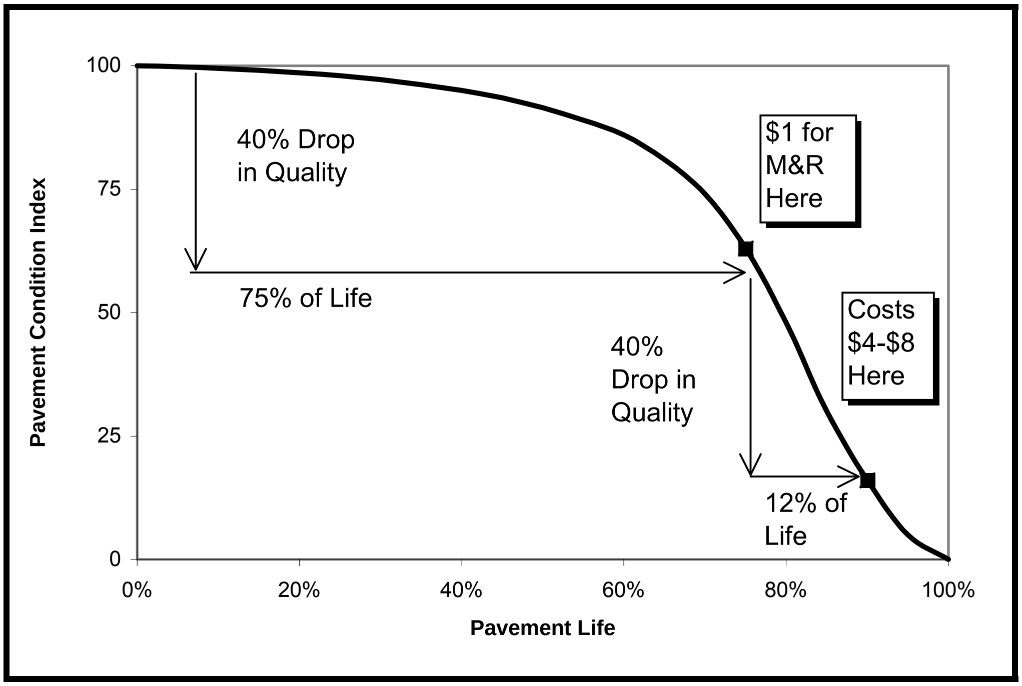


Figure 3-13. Typical Pavement Life Cycle (APWA,1983)

Of more direct impact, the value of completing the cursory PCI survey is threefold. First, it is a tool that helps identify potential structural problems. Second, for those pavements with a PCI of 40 or lower, reported AGLs are reduced by 25%; therefore, to complete the structural analysis, it must be determined whether any of the pavement features fall into those categories. Finally, in a subjective manner, the PCI survey can be used as a gauge to determine if the pavement is approaching the end of its life.

CHAPTER 4

PAVEMENT EVALUATION USING NONDESTRUCTIVE TESTING

4.1 EVALUATION PRINCIPLES.

a. Computer Program. The PCASE (Pavement-Transportation Computer Assisted Structural Engineering) desktop is a computer program used by the Services for design and evaluation of pavements, see Appendix H. The Evaluation module within the PCASE desktop is used to compute Allowable Loads, Allowable Passes, and Pavement Classification Numbers for evaluations.

b. Flexible Pavements. The structural deterioration of flexible pavements caused by traffic is normally evidenced by cracking of the asphalt concrete (AC) surface course and development of ruts in the wheel paths. The NDT evaluation procedure handles these two modes of structural deterioration through limiting values of the strain at the bottom of the AC layer and at the top of the subgrade. In the case of thin asphalt surfaces (less than 3 inches), the Services may consider only using strain at the top of the subgrade.

c. Rigid Pavements. Failure of rigid pavements due to the repeated application of loads (fatigue) is normally evidenced by cracking of the portland cement concrete (PCC) layer. Performance criteria for rigid pavements are based on limiting the tensile stress in the PCC slab to levels such that failure occurs only after the pavement has sustained a number of load repetitions. The stresses and strains used for entering the criteria are computed by the use of Burmister’s solution for multilayered elastic continua. The solution of Burmister’s equations for most pavement systems will require the use of computer programs and characterization of pavement materials by the thickness, modulus of elasticity, and Poisson’s ratio. Failure is based on a Structural Condition Index of 50 or 0, as discussed in paragraph 4.8.b (2).

4-2. PAVEMENT RESPONSE MODEL. The computer code used for computing pavement response is the five-layer linear elastic program WESLEA, which is a subroutine of the Layered-Elastic Evaluation Program (LEEP) used in the Evaluation Module of the PCASE desktop. When WESLEA is used, the following assumptions are made:

a. The pavement is a multilayered structure, and each layer is represented by the thickness, a modulus of elasticity, and Poisson’s ratio. Individual layers are assumed to be homogeneous, isotropic, and extending infinitely in the horizontal direction.

b. The interface between layers is continuous; i.e., the friction resistance between layers is greater than the developed shear force.

c. The bottom layer is located 20 feet from the surface and is of infinite thickness.

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

4-3.**PROCEDURE**. The procedure outlined in this chapter is applicable to flexible, plain concrete, plain concrete overlays, and nonrigid overlays on plain concrete pavements. Criteria are not available for reinforced or fibrous pavements. The procedure is based on a layered linear elastic model that characterizes multilayered pavement systems. The program uses layer strength parameters determined from field in situ measurements to compute allowable loads for a selected number of aircraft passes, allowable passes at a specified load, and the Pavement Classification Number (PCN). Strengthening requirements can then be determined for the design pass level and aircraft load. The evaluation will be valid for conditions existing at the time of test. PCASE is the computer program used by the Services to compute airfield capability. Information on how to obtain this program is in Appendix H.

4-4. **STEP 1, Segmentation**. Segmentation of airfield pavements using Network, Branch, and Section is described in Chapter 3. A typical airfield segmented into Sections is shown at Figure 3-3.

4-5. **STEP 2, Select Representative Deflection Sections.** NDT data is collected as described in Chapter 3. All basins, selected basins, or a representative deflection basin is selected for each Section to be evaluated. All basins or a representative basin can be analyzed for each Section. It is recommended that all basins be analyzed and the mean modulus value for each layer used for the pavement evaluation. Simply taking the average of each deflection reading is not acceptable because high or low values disturb the mean and change the shape of the basin used in the Evaluation Module of the PCASE desktop.

a. NDT data are grouped into areas of equivalent impulse stiffness modulus (ISM). ISM is defined as the force or load in kips divided by the deflection measured at the center of the load in inches. Although a pavement Section may supposedly be of the same type and construction, it should be subdivided when the strength characteristics measured in one area of the Section are greatly different from those in another area of the Section. An ISM is computed from the basin data to provide a qualitative stiffness comparison between test points and between pavement Sections. The current procedure is to plot the ISM values along the length of the Section and visually determine if a change in strength exists.

b. Measured deflections are normalized to a common load. In most cases, the NDT loading will vary slightly from test to test. To eliminate the effects of this variability, deflections are normalized with respect to load before the basins are compared. This is accomplished by multiplying each deflection by the load ratio (largest load measured within the Section divided by the load at which the deflection was obtained).

c. The geometric average deflection is computed for each sensor offset distance within a Section.

d. The area of each deflection basin is determined as illustrated in Figure 4-1. Only the hatched area (under the measured portion of the basin) is considered in this computation, and the area between two sensors is assumed trapezoidal.

e. Compute the average deflection basin area.

f. Although not used in determining the representative basin, an estimate of the modulus of subgrade reaction, k, beneath rigid and nonrigid overlay of rigid pavements can be determined by computing the volume of the deflection bowl as illustrated in Figure 4-2. The k value obtained in this manner is only an estimate, and it should be noted that a substantial portion of the area used in the computation is in the extrapolated range.

g. Compute an error function. An error function is computed as:

$$ERROR = \left(\frac{\overline{ISM} - ISM}{\overline{ISM}} \right)^2 + \sum_{1}^{ND} \left(\frac{\overline{DF} - DF}{\overline{DF}} \right)^2 + \left(\frac{\overline{AREA} - AREA}{\overline{AREA}} \right)^2 \quad \text{(eq 4-1)}$$

where

ISM =computed ISM
DF=measured deflection
AREA= computed area
ND=number of deflection sensors

$\overline{\mathrm{ISM}}$ =average ISM

$\overline{\mathrm{DF}}$ =average deflection

$\overline{\mathrm{AREA}}$ =average basin area

h. The deflection basin with the least error is selected as the representative basin for evaluating the Section.

I. The representative basin determined above is used whenever the coefficient of variation of the ISM from all basins in the feature is less than 15 percent. If the coefficient of variation is greater than 15 percent, then judgment is used to select an appropriate basin.

4-6. **STEP 3, Predict Layer Modulus Values.** The deflection basin produced by applying a load to the pavement with an NDT device gives input parameters to the system analysis that can be used to derive the relative strength parameters of the pavement layers. To determine modulus values, the pavement structure is modeled as a layered system similar to that illustrated in Figure 4-3. The computer program WESDEF was developed to determine a set of modulus values that provides the best fit between a measured and a computed deflection basin when given an initial estimate of the elastic modulus values, a range of modulus values, and a set of measured deflections. WESDEF calculations are contained within the Evaluation module of the PCASE desktop. To summarize the modulus back-calculation routine:

a. Consider the pavement system where:

(1) The modulus is unknown for a number of layers (NL).

(2) The deflection due to an NDT loading is measured at a number of deflection locations (ND).

(3) ND is greater than NL.

The objective is to determine the set of elastic modulus (E) values that will minimize the error between the computed deflection (CD) and the measured deflection (MD).

b. A set of E values is assumed, and the deflection is computed at the sensor location corresponding to the measured deflection. Each unknown E is varied individually, and a new set of deflections is computed for each variation. Figure 4-4 is a simplified description of how the deflection basins are matched. This illustration is for one deflection and one layer. For multiple deflections and layers, the solution is obtained by developing a set of equations that defines the slope and intercept for each deflection and each unknown layer modulus as follows:

$$Deflection_j = A_{ji} + S_{ji}\left(\log E_i\right) \quad \text{(eq 4-2)}$$

Where:

A = intercept

S = slope

j = 1 to the number of deflections

i = 1 to the number of layers with unknown modulus values

c. For WESDEF, a range of modulus values is input with an estimated initial modulus value for each layer for which modulus values are to be determined. The number of unknown modulus values cannot exceed the number of measured deflections. Best results are obtained when not more than three layers are computed in a single execution.

d. Default ranges and initial estimates for the modulus and Poisson's ratio of pavement materials are recommended in Table 4-1.

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

Table 4-1, WESDEF Default Modulus Values, (psi)

e. If the deflection basin includes a deflection measured at an offset distance of 72 inches, the initial subgrade modulus is estimated as follows:

$$E = 59{,}304.82\,(D72\ )^{-0.98737} \quad \text{(eq 4-3)}$$

Where:

E = subgrade modulus, pounds per square inch

D72 = deflection measured at a distance of 72 inches from an applied NDT loading normalized to 25,000 pounds

f. A range for the subgrade modulus is then established as the predicted value plus and minus 5,000 psi. This relationship is not valid for the case where bedrock is present near the pavement surface (<20 feet), and the default values should not be used if this situation is encountered. Use the depth to bedrockTypically, the modulus of any surface layer can and should be computed with WESDEF. However, in some instances it may be necessary to assign a modulus value to the AC or PCC layer (i.e., WESDEF yields unrealistic values or the surface layer is very thin). If assigned, the value will be based on the type of material or properties of the material at the time of testing. For flexible pavements, the surface temperature at the time of testing is added to the previous 5-day mean air temperature, and the mean pavement temperature is determined from Figure 4-5. The assigned AC modulus is obtained using Figure 4-6 and the loading frequency for the NDT device. The FWD or HWD device normally produces a load frequency at or near 20 Hz. The curves in Figure 4-6 are extrapolated from laboratory relationships for new AC mixes; therefore, predicted values may not always agree with actual field values. A modulus of 5,000,000 psi is recommended for a PCC layer in good condition. Modulus values developed from PSPA can also be used to establish the surface layer modulus.

g. WESDEF incorporates a layer of infinite thickness having a modulus of elasticity of 1,000,000 psi and Poisson's ratio of 0.5 below the subgrade layer. This stiff layer should be located at a depth of 20 feet unless soil profiles indicate the need for some other representation (i.e., shallow rock). See paragraph 4.6.e.

h. WESDEF is capable of handling both multiple loads and variable interface conditions. For a given layer (n) and underlying layer (n + 1), the interface value should be set at 1 for complete adhesion between the layers or 1,000 for almost frictionless slip between the layers. Values between 1 and 1,000 may be input to simulate varying degrees of friction. Almost frictionless slip is usually assumed at the bottom of a PCC layer and full adhesion is generally assumed for most other pavement materials.

i. WESDEF provides a tool with which modulus values can be predicted. Normally three iterations within the program produce a set of modulus values that yield a deflection basin that is within an average of 3 percent of each of the measured deflections. In analyzing the results from the WESDEF program, it is important to check the predicted modulus for each layer and determine if any of the predicted modulus values are against the limits. If the modulus is outside a limit, engineering judgment is required to select one of the following:

(1) Rerun WESDEF computing modulus values for fewer layers. Some options to be considered are as follows:

(a) Fix the modulus of an AC or PCC surface layer based on tests conducted with the PSPA or on material type and condition at the time of testing rather than computing the modulus.

(b) Combine base and subbase into one layer and compute a composite modulus or divide the base course into two layers.

(c) Fix the subgrade modulus based on results of a preliminary run or on the deflection of sensor #7. In some cases, subdividing the subgrade into two layers may be warranted.

(2) Rerun WESDEF with modified limits to include the predicted E disregarding boundary conditions (values outside default ranges may be unrealistic).
(3) Accept the results of the WESDEF run realizing that the predicted values are outside the typical range for a particular material.

j. The following guidelines may be helpful in determining layer modulus values using WESDEF:

(1) Do not attempt to compute the modulus values for more than three layers in a single WESDEF run. Limit the number of computed layer moduli to two if possible (particularly for rigid pavements).

(2) Do not attempt to compute the modulus of layers less than 3 inches thick. The modulus of a thin layer should be fixed based on material type, temperature, etc.; or else a thin layer should be combined with an adjacent layer and a composite modulus determined.

(3) When computing the modulus of a PCC layer, it may be necessary to combine a base or subbase layer with the subgrade layer and determine a composite modulus for the material beneath the PCC slab.

(4) Exercise caution when using modulus values outside the default ranges. Because the ranges are quite broad, values outside these limits may be unrealistic.

(5) For NDT devices with circular loaded areas, the offset distance to the first measured deflection is input to WESDEF as one-half the radius of the loading plate to approximate the deflection at one-half the radius of a uniformly distributed circular loaded area.

4-7. **STEP 4, Determine Design Traffic.**

a. The Army and Navy projects the total number of passes of each aircraft type that the pavement will be expected to support over its design life, usually 20 years. The Air Force evaluation is based on the 14 Aircraft Groups and the 4 Pass Intensity Levels in Table 2-1. For a given projected aircraft mixture, the critical aircraft must be determined for the evaluation. The critical aircraft is that aircraft from the mixture which requires the greatest pavement thickness to support its projected passes. The number of passes of the critical aircraft required to produce an equivalent effect on the pavement as the mixture of traffic is the design pass level. The TRAFFIC module within the PCASE desktop will determine the critical aircraft and compute equivalent passes of the critical aircraft. The procedures incorporated in TRAFFIC are as follows:

(1) Determine the total pavement thickness required for each individual aircraft at its projected pass level, using current criteria. Thicknesses should be computed using a representative subgrade modulus for the Section or Branch. The aircraft requiring the greatest thickness is designated as the critical aircraft.

(2) Determine the allowable number of passes for each individual aircraft for the maximum required thickness (thickness required by the critical aircraft).

(3) Determine the design passes in terms of the critical aircraft by multiplying the projected passes of the critical aircraft by the ratio of projected passes for each individual aircraft to allowable passes of each individual aircraft at the maximum thickness. The program outputs a traffic mix analysis showing how each individual aircraft contributes to the total design pass level and will identify the critical aircraft and design pass levels.

b. For Navy and Marine Corps evaluations, it is recommended to use the maximum peace-time take-off weight, and the maximum landing weight from the Navy Aircraft Characteristics supplement (see Table 2-4) and Transportation Systems Center Reports 13-2 and 13-3. If desired, the evaluation may use maximum war-time take-off weights, but these weights are unlikely, even during war-time. If desired, use more realistic, measured weights: in this case it is advised that the measured average weight plus one standard deviation be used for the 5 standard Navy aircraft; these values are shown as evaluation weights in Table 2-5.

c. As indicated, TRAFFIC will express the total traffic in terms of one critical aircraft and a corresponding design pass level. It is possible also to express the total traffic in terms of any other aircraft, in particular the aircraft representative of the five Navy categories. This is done by (1) dividing each aircraft equivalent passes by the total equivalent passes (design pass level) to obtain each aircraft participation, and (2) by dividing the actual aircraft passes by its participation. If this is done for each aircraft category, a tentative PCN can be found for each one.

4-8.**STEP 5, Compute Allowable Aircraft Loads, Allowable Passes, Required Overlay Thickness and PCN.** Allowable load-carrying capabilities and required overlay thicknesses are evaluated using the computer program WESPAVE. WESPAVE calculations are contained within the Evaluation module of the PCASE desktop. For a particular aircraft (gear configuration, load, pass intensity level, etc.), WESPAVE uses modulus values from WESDEF and computes stresses (rigid and rigid and nonrigid overlay on rigid pavement) and strains (flexible pavement) that will occur in the pavement system. WESPAVE then calculates the limiting stress or strain values from empirically developed layered- elastic values. Allowable load for the aircraft at the design pass level and allowable passes of the design aircraft at maximum load are determined by comparing the predicted stress or strain to the limiting value. Criteria and methodology incorporated in WESPAVE follows:

a. Passes/Coverages. Regarding the evaluation criteria, an important point that should be emphasized is that the surface criteria (AC and PCC) are based on coverages to failure, while the subgrade criteria are based on repetitions to failure. The lateral distribution of traffic has a greater effect on the number of maximum stress applications that occur at a point near the surface than for a point deep within the pavement structure (ERDC Miscellaneous Paper S-73-56). The incremental detriment to a pavement caused by a wheel of an aircraft at a particular location on the pavement is influenced by many factors such as number of tires on the aircraft, tire spacing, load on each tire, tire contact pressure, location of aircraft on the pavement, and previous loading history. As a result of different assumptions and development procedures used in analyzing results of traffic tests, the term coverage has different meanings for rigid and flexible pavements. For rigid pavements, coverage is a measure of the number of maximum stress applications that occur within the pavement due to the applied traffic. A coverage occurs when each point in the pavement within the limits of the traffic lane has been subjected to a maximum stress. For flexible pavements, coverage is a measure of the number of maximum stress applications that occur on the surface of the pavement due to the applied traffic. A coverage occurs when all points on the pavement surface within the traffic lane have been subjected to one application of maximum stress. Thus, a twin-tandem gear would produce two applications of stress on the surface of a flexible pavement, but it would produce only one maximum stress application within a rigid pavement if the tandem spacing was small and would produce two maximum stresses if the tandem spacing was large. The influence of the lateral distribution of aircraft traffic is expressed in terms of pass-to-coverage ratios derived for each aircraft. Pass/coverage ratios for individual aircraft are in Table 6-2 and on the aircraft curves in Chapters 5 and 6.

b. Limiting Stresses and Strains. WESPAVE determines the limiting values of stress/strain for a particular pavement type using the following:

(1) Flexible Pavements. Horizontal tensile strain at the bottom of the AC layer and vertical subgrade strain at the top of the subgrade are considered in the evaluation of flexible pavements. The limiting AC strain criterion (shown graphically in Figure 4-7) is as follows:

$$ALLOWABLE\ STRAIN_{AC} = 10^{-A} \quad \text{(eq 4-4)}$$

Where:

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

$$A = \frac{N + 2.665\ LOG_{10}\left(\frac{E_{AC}}{14.22}\right) + 0.392}{5}$$

Where:

$N = LOG_{10}$ (aircraft coverages)

E_{AC} = AC modulus, pounds per square inch

The allowable subgrade strain criterion (shown graphically in Figure 4-8) is as follows:

$$ALLOWABLE\ STRAIN_{SG} = \left(\frac{10{,}000}{N}\right)^{1/B} A \quad \text{(eq 4-5)}$$

Where:

ALLOWABLE $STRAIN_{SG}$ = allowable vertical strain at the top of the subgrade, inches/inches

N = aircraft repetitions (passes)

$A = 0.000247 + 0.000245\ LOG(E_{SG})$

$B = 0.0658\ (E_{SG})^{0.559}$

E_{SG} = subgrade modulus, pounds per square inch

(2) Rigid and Nonrigid Overlay on Rigid Pavements. WESPAVE assumes that an AC over PCC structure to be evaluated as a rigid pavement. If the modulus of the PCC layer determined using WESDEF is less than 1,000,000 psi, the pavement should

be evaluated as a flexible pavement. The evaluation of rigid and nonrigid overlay of rigid pavements is based on the tensile stress at the bottom of the slab. The criteria provide for prediction of pavement deterioration in terms of a structural condition index (SCI). The SCI is defined as follows:

$$SCI = 100 - A * (\text{ sum of structural deducts}) \quad \text{(eq 4-6)}$$

where A is an adjustment factor based on the number of distress types with deduct values in excess of five points determined from the condition survey, and the structural deducts are a function of distress types, severities, and densities associated with loads. The SCI prediction is based on a relationship between design factor and stress repetitions as related to crack formation in the PCC slabs due to load. An SCI of 50 corresponds fairly well to the formation of one or more cracks per slab in 50 percent of the trafficked slabs (first crack criteria) and is used as failure criteria by the Army and Navy. The Air Force uses SCI=0 for failure criteria. There can be considerable differences between SCI=0 and shattered slab criteria, but results are closer than previous criteria. The design factor DF is defined as the concrete flexural strength divided by the stress. The equation for the relationship given in Figure 4-9 is as follows:

$$DF = A + B \,\mathrm{LOG}\, C \quad \text{(eq 4-7)}$$

Where:

DF = design factor

A = 0.2967 + 0.002267 (SCI)

B = 0.3881 + 0.000039 (SCI)

C = coverage level at selected SCI

SCI = structural condition index

and

$$ALLOWABLE\ STRESS_{PCC} = \frac{R}{DF} \quad \text{(eq 4-8)}$$

Where:

ALLOWABLE $\text{STRESS}_{\text{PCC}}$ = allowable tensile stress at the bottom of the slab, pounds per square inch
R = PCC flexural strength, pounds per square inch

c. Maximum Stresses and Strains. Stresses/strains within a pavement system are computed using the controlling wheels of the design aircraft and the WES5 subroutine. The location of the maximum stress/strain value is influenced by factors such as pavement structure, wheel load, and wheel spacing. For a single-wheel aircraft, the maximum stress/strain will always occur directly underneath the wheel. For other more complicated gear configurations, stresses/strains must be computed at several positions to determine where the critical values occur. Gear configurations for various aircraft considered in evaluation are shown in Table 2-2 with controlling wheels and the recommended minimum number of stress/ strain evaluation positions indicated. The new gear nomenclature from Appendix D has been added.

d. Evaluation of Load Transfer. The deflection ratio from joint efficiency tests defined as

$$DEFLECTION\ RATIO = \frac{DEFLECTION\ OF\ UNLOADED\ SLAB}{DEFLECTION\ OF\ LOADED\ SLAB} \qquad \text{(eq 4-9)}$$

should be included in the evaluation of rigid and nonrigid overlays of rigid pavements which are evaluated as rigid. The allowable loads determined at the slab centers can be reduced for poor joint transfer using load reduction factors. These factors are a function of the deflection ratio. The procedure was developed by first relating the deflection ratios to the percent maximum edge stress. Finite element programs were used to compute edge stresses for a range of pavement thicknesses and subgrade moduli, k. The maximum edge stress condition is a free edge with no load transfer. The edge stress is reduced as more load is transferred across the joint. The 75 percent stress corresponds to a deflection ratio of 0.76, and this would be for 100 percent of the design load (load factor of 1.00). The condition of 100 percent maximum stress would occur at a deflection ratio of 0.0 (no load transfer) and would allow for only 75 percent of the design load (load reduction factor of 0.75). The allowable percent of design load was computed at different deflection ratios. Figure 4-9 then provides the procedure for reducing the allowable load determined at the slab center to account for the load-transfer capabilities at the joint. The load reduction factor falls between 0.75 and 1.00.

e. PCASE Output. A typical PCASE output is shown on Figure 4-10. The pavement evaluation is conducted for a specified number of passes of an aircraft. In this example, the evaluation is for 50,000 passes of a C-17. The Section is an A Traffic Area and the structure consists of 15 inches of PCC, with a flexural strength of 650 psi, on natural subgrade. Moduli values for the PCC and subgrade are 5,000,000 and 15,000 psi, respectively. Rigid pavement SCI at failure is set at 0 and the overlay condition factors (C_b and C_r) at 0.75. Results are as follows:

(1) Time. The timeframe for the evaluation is January-December.

(2) Design Passes. The evaluation is for 50,000 passes of the C-17.

(3) ACN. The ACN for a 585,000 pound C-17 is 54/R/C/W/T.

(4) PCN. The computed PCN for this Section is 43/R/C/W/T.

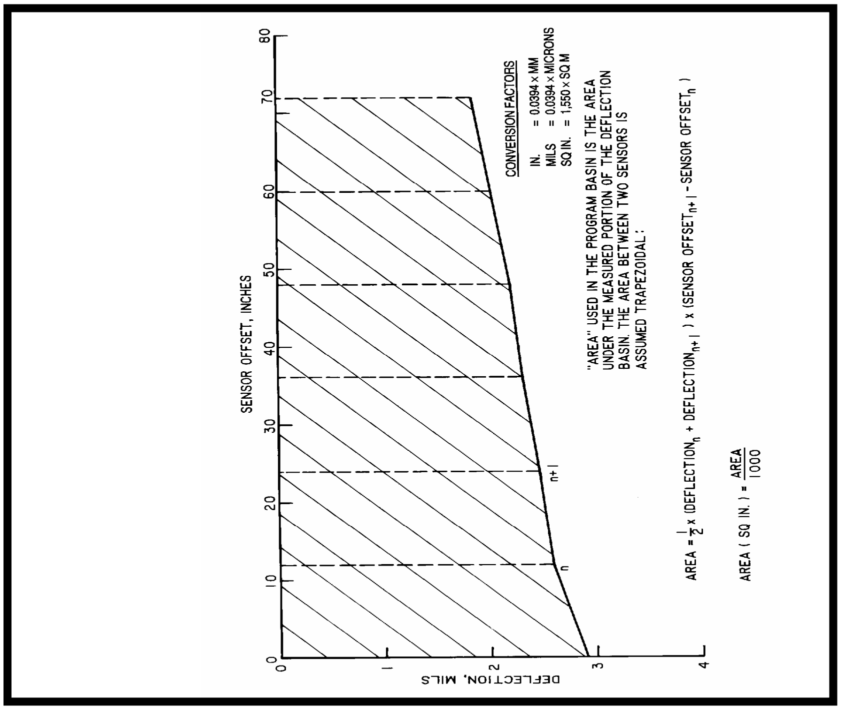
(5) Allowable Load. The Section will support a 483,800 pound C-17, for 50,000 passes.

(6) Allowable Passes. The Section will support a 585,000 pound C-17 for 6004 passes.

(7) AC. A 13.6 inch asphalt overlay is required to support 50,000 passes of the 585,000 pound C-17.

(8) PCCNB. A 12.0 inch non-bonded PCC overlay is required to support 50,000 passes of the 585,000 pound C-17.

(9) PCCPB. A 9.2 inch partially-bonded PCC overlay is required to support 50,000 passes of the 585,000 pound C-17.



CONVERSION FACTORS

IN. = 0.0394 x MM
MILS = 0.0394 x MICRONS
SQ IN. = 1,550 x SQ M

"AREA" USED IN THE PROGRAM BASIN IS THE AREA UNDER THE MEASURED PORTION OF THE DEFLECTION BASIN. THE AREA BETWEEN TWO SENSORS IS ASSUMED TRAPEZOIDAL:

$$\text{AREA} = \frac{1}{2} \times (\text{DEFLECTION}_n + \text{DEFLECTION}_{n+1}) \times (\text{SENSOR OFFSET}_{n+1} - \text{SENSOR OFFSET}_n)$$

$$\text{AREA (SQ IN.)} = \frac{\text{AREA}}{1000}$$

Figure 4-1. Determination of area beneath deflection basin

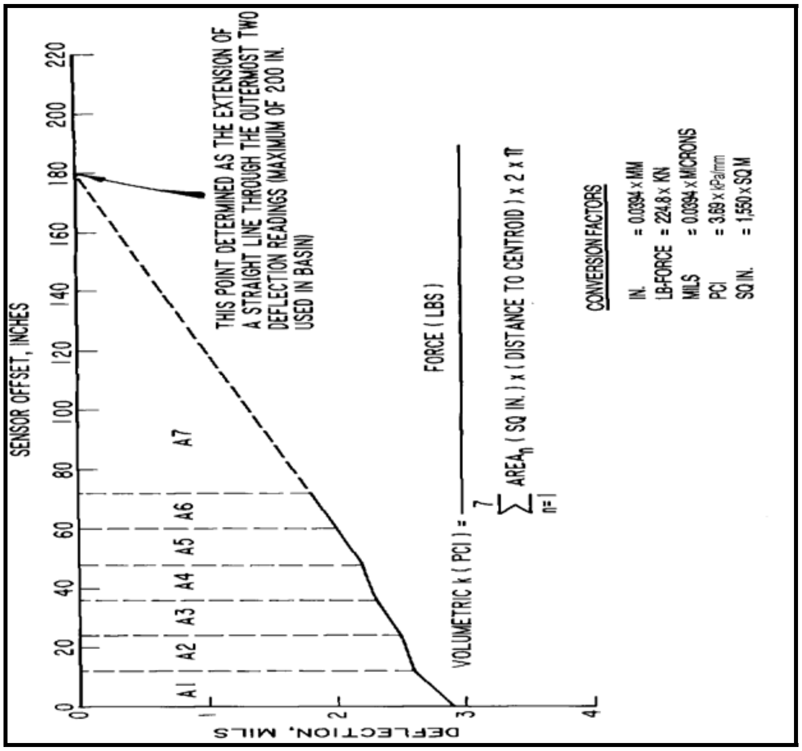


Figure 4-2. Procedure for determining the volumetric k value (an estimate of the modulus of subgrade reaction)

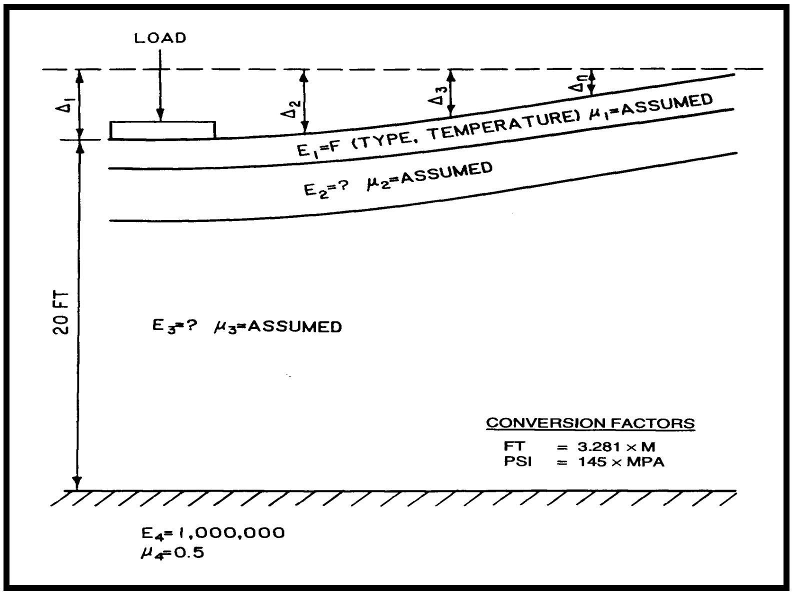


Figure 4-3. Illustration of a layered pavement structure

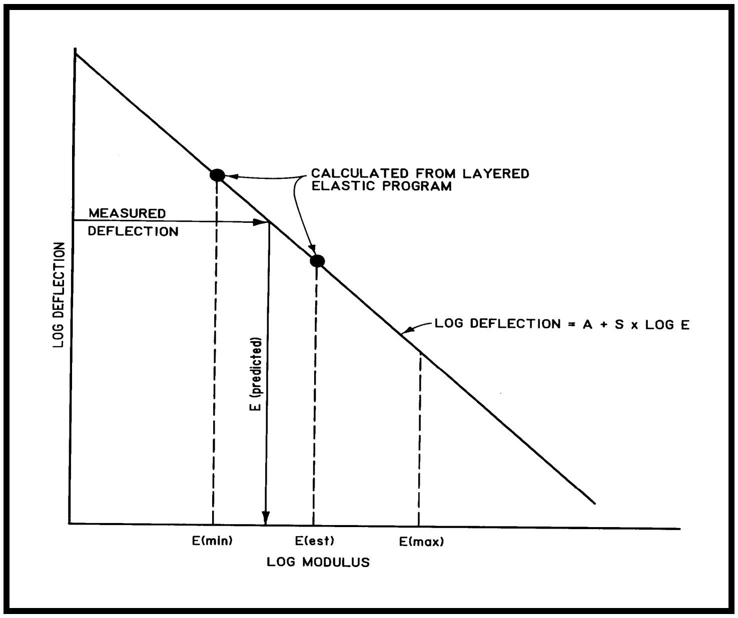


Figure 4-4. Simplified description of how deflection basins are matched in WESDEF (one deflection and one layer)

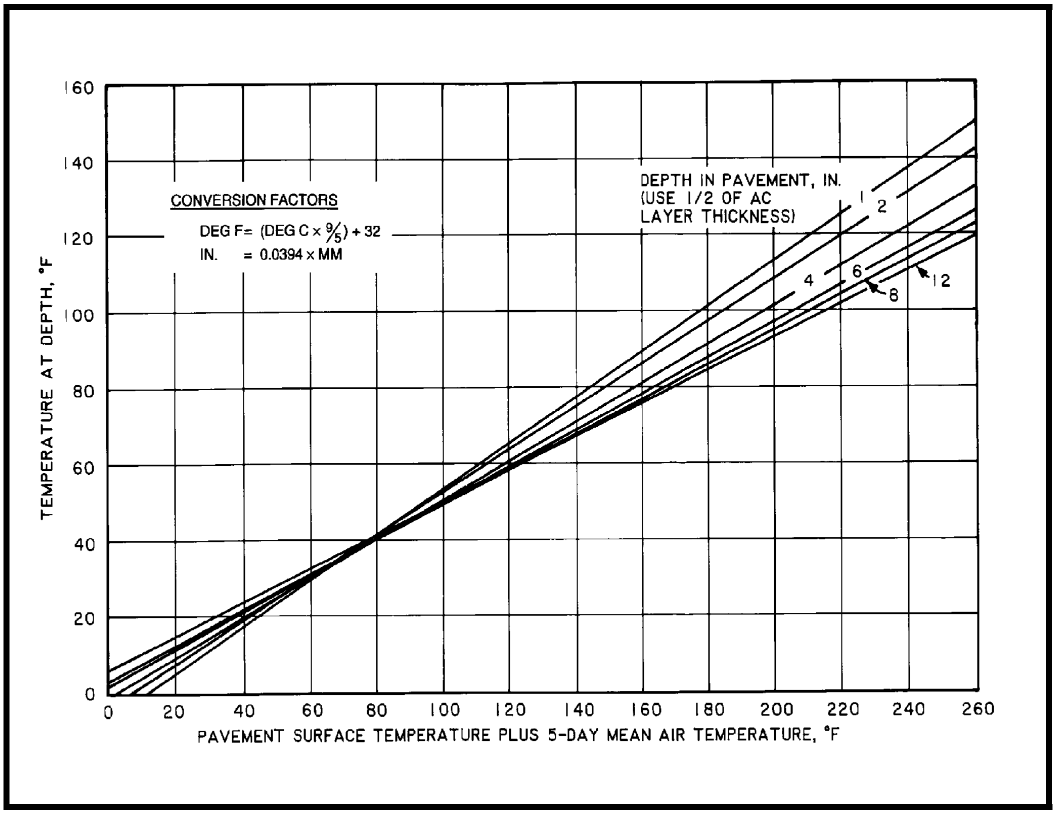


Figure 4-5. Determination of Mean Pavement Temperature

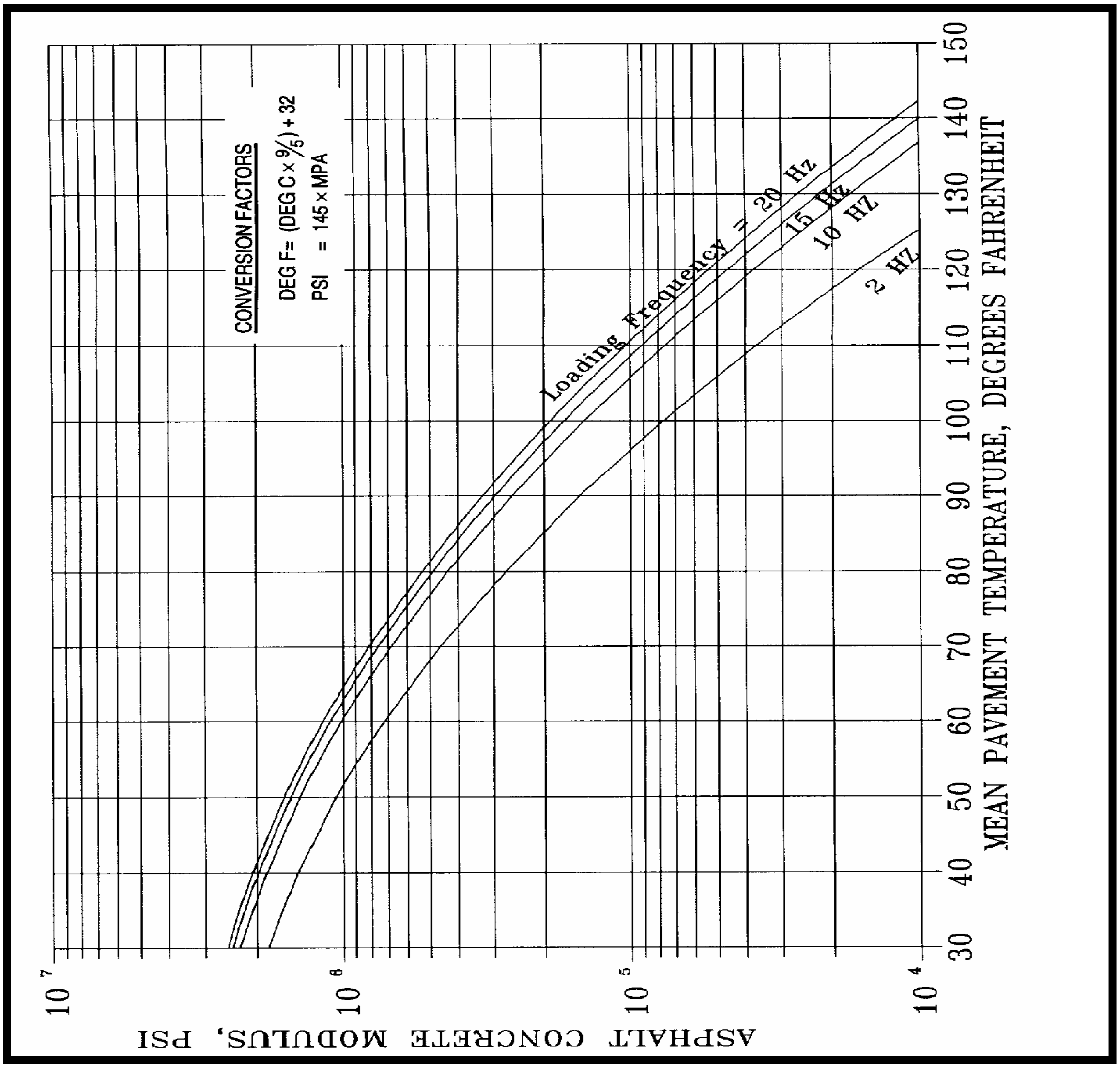


Figure 4-6. Prediction of AC modulus for bituminous layers

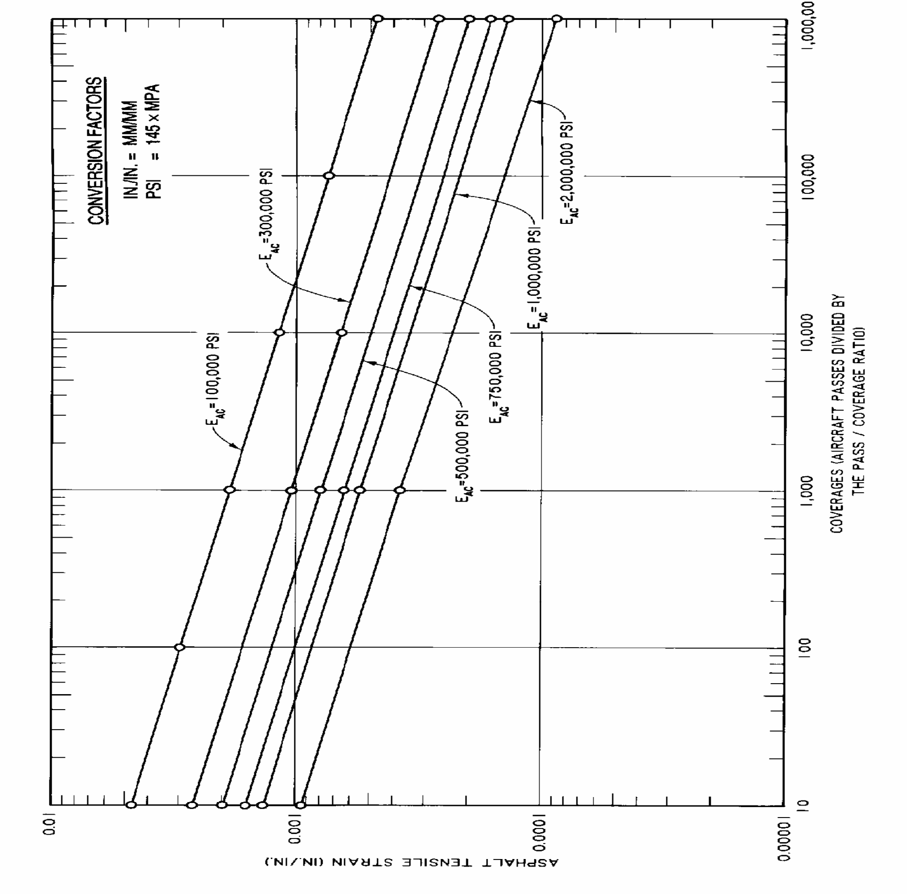


Figure 4-7. Limiting horizontal tensile strain criteria for an AC layer

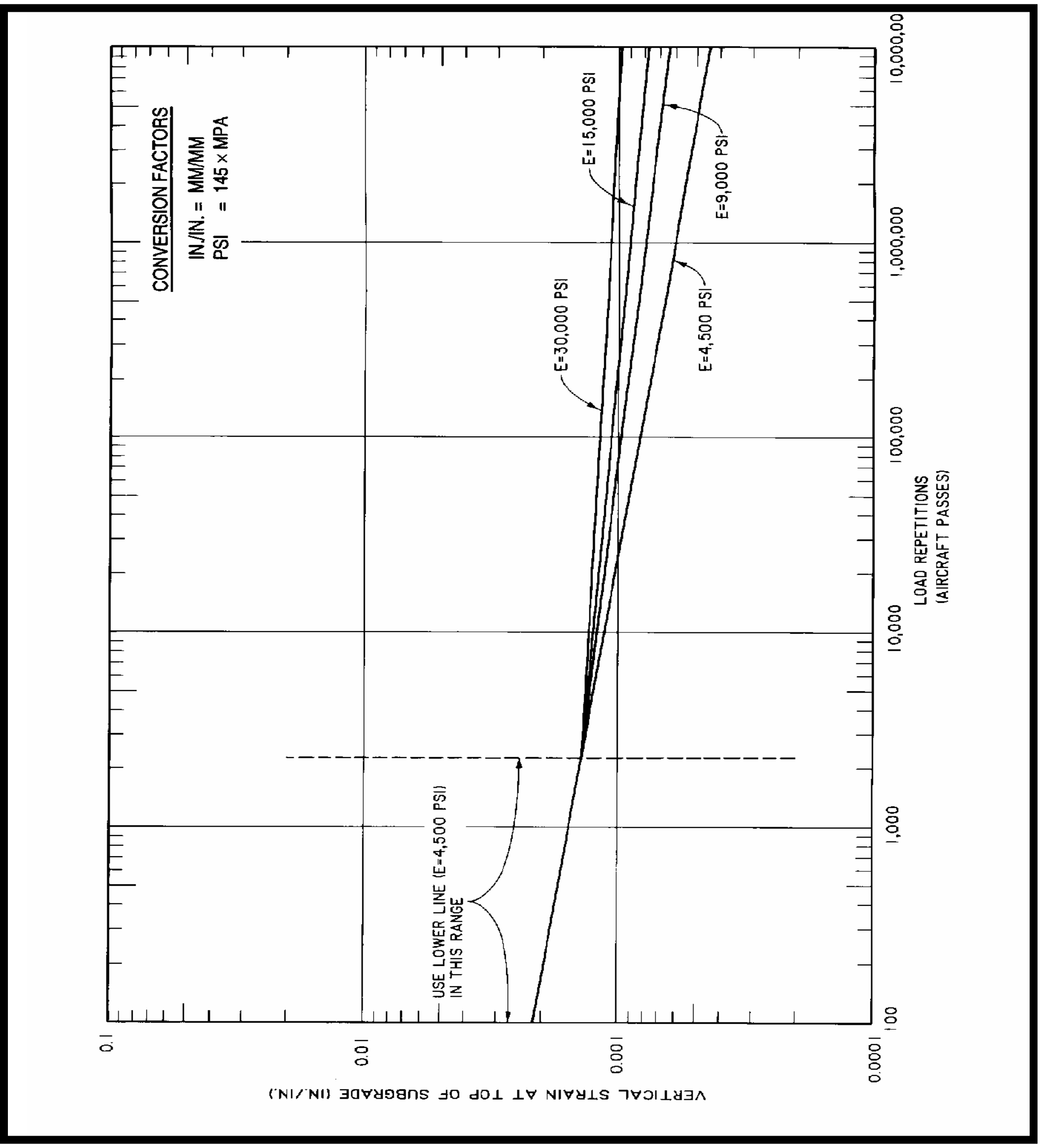


Figure 4-8. Limiting vertical subgrade strain criteria for flexible pavement

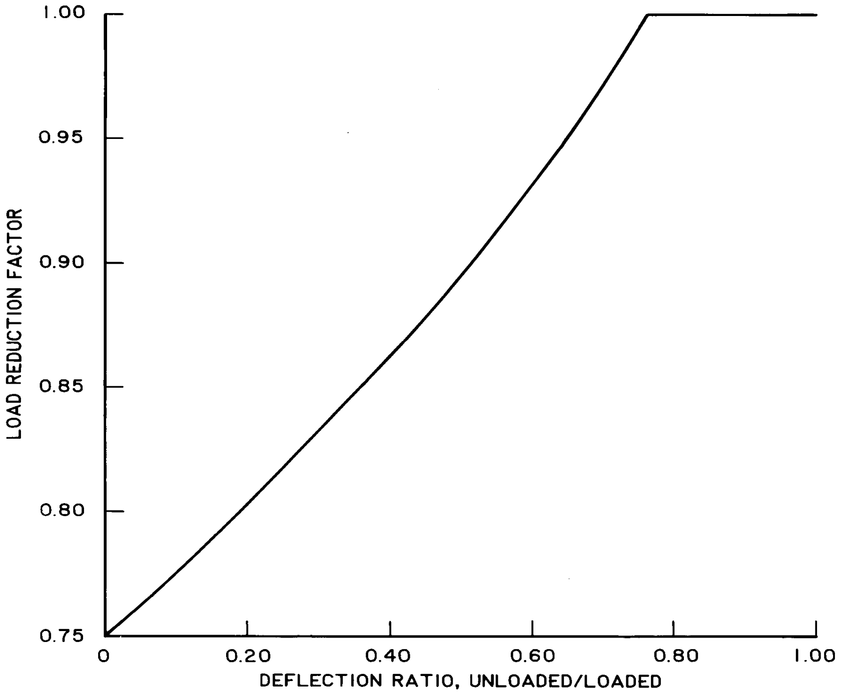
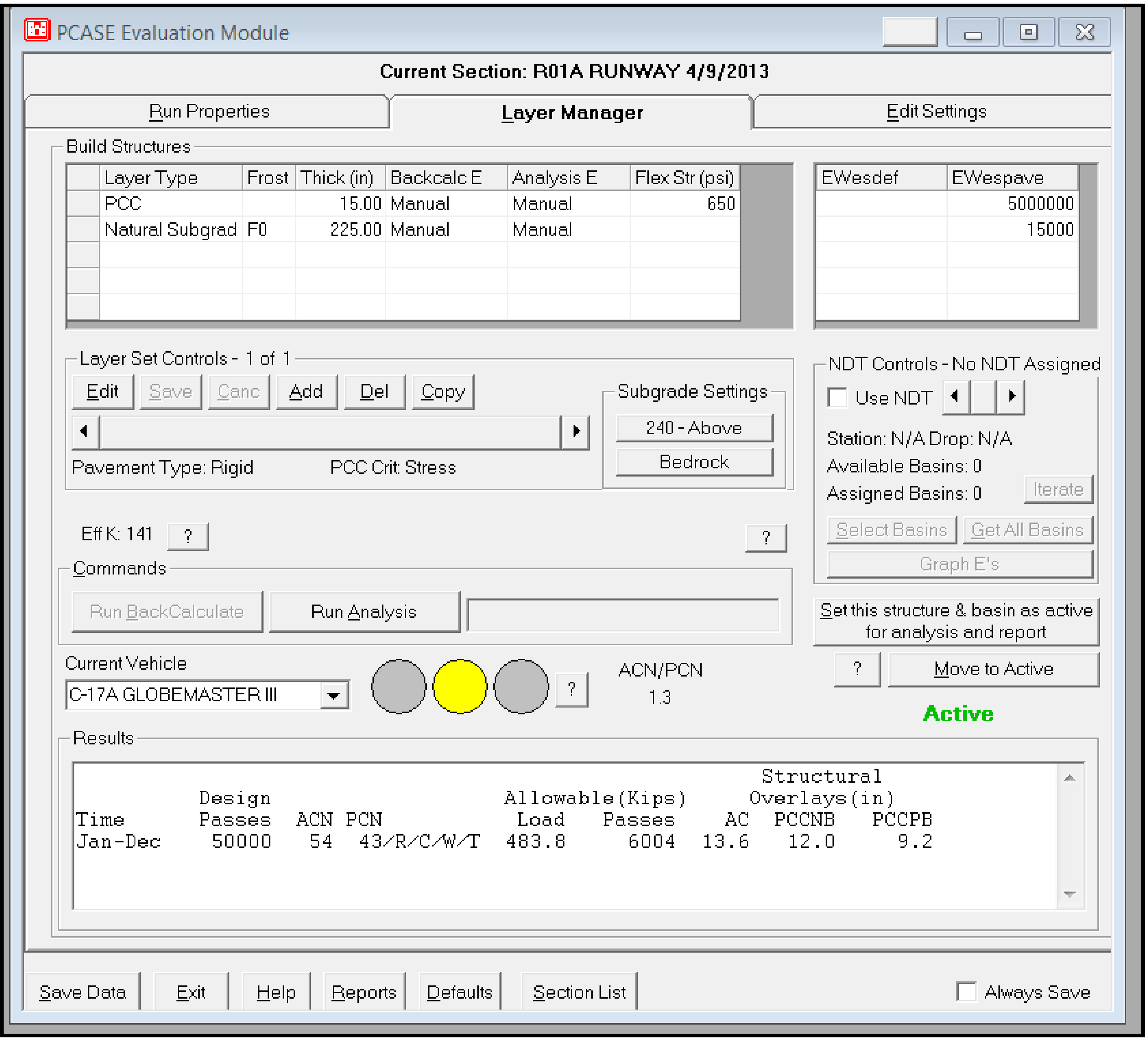


Figure 4-9. Load reduction factors for load-transfer analyses



Current Section: R01A RUNWAY 4/9/2013

Layer Type	Frost	Thick (in)	Backcalc E	Analysis E	Flex Str (psi)
PCC		15.00	Manual	Manual	650
Natural Subgrad	F0	225.00	Manual	Manual	

EWesdef	EWespave
	5000000
	15000

Pavement Type: Rigid

PCC Crit Stress

Station: N/A Drop: N/A

Available Basins: 0

Assigned Basins: 0

Eff K: 141

Current Vehicle

C-17A GLOBEMASTER III

ACN/PCN

1.3

Active

```
                                                   Structural
           Design                Allowable(Kips)    Overlays(in)
Time       Passes   ACN  PCN        Load   Passes    AC   PCCNB   PCCPB
Jan-Dec     50000    54  43/R/C/W/T 483.8    6004   13.6   12.0     9.2
```

Figure 4-10. PCASE results

CHAPTER 5

EVALUATION OF FLEXIBLE PAVEMENT USING DIRECT SAMPLING

5-1 **GENERAL**. This chapter presents criteria for evaluating flexible pavements using data from direct sampling. The data required for evaluation were presented in chapter 3. A computer program is also available for pavement evaluation and is discussed in Appendix H.

5-2 **FACTORS LIMITING LOAD-CARRYING CAPABILITY**. The load-carrying capability of a flexible pavement is limited by its critical or controlling layer, either the pavement surface, base, subbase, or subgrade. The ability of a given subsurface layer to withstand the loads imposed on it depends on the thickness and strength of material above it and its strength in its weakest condition. The critical or controlling layer is the layer that will support the least allowable load. Structural failure criteria is based on a 1-inch rut. To be realistic, an evaluation must take into account possible future changes in moisture content and density as well as the effects of freezing and thawing.

5-3 **SELECTION OF THICKNESS VALUES**. To evaluate an airfield facility, the pavements must be divided into traffic areas as described in UFC 3-260-02 Pavement Design for Airfields. A uniform thickness may be found for many pavements, and in these cases the traffic area types should be designated. For pavements designed in accordance with the traffic area concept, thickness differentials will occur between the various types of traffic areas. When the pavement has a uniform thickness for the entire width, the selected thickness for evaluation is no problem. When pavement thicknesses vary for a given Section, each thickness should be evaluated, but only the controlling evaluation for the facility should be reported. Segmentation of an Airfield Pavement Network into Branches and Sections is discussed in Chapter 3.

The in-place thicknesses of the asphaltic concrete and underlying layers are determined by actual measurement or from construction data. However, the measured thicknesses may need to be modified for use with the evaluation curves when the measured thickness exceeds the required minimum thickness. PCASE automatically adjusts for excess thickness. Minimum thickness requirements are contained in UFC 3-260-02 Pavement Design for Airfields. The excess thickness of asphalt is converted to an equivalent thickness of base course and added to the existing base thickness. Then, any excess base-course thickness is converted to an equivalent thickness of subbase and added to the subbase thickness. This adjusted section is then used for evaluation. The equivalencies are contained in Tables 5-2 and 5-3, and their use and an example using equivalency factors is in paragraph 5.5.

5-4 **SELECTION OF STRENGTH VALUES FOR SOIL LAYERS**. The strengths of the subgrade and overlying subbase and base courses are determined by means of CBR tests described in CRD-C 654, or DCP tests described in AF ETL 02-19. The quality of materials in the various layers of these courses can be determined by tests on the materials in place, by laboratory tests on samples of the materials, and from construction data. The CBR test results from an individual test pit will seldom be uniform, and the data must be carefully studied to arrive at reasonable values for use in the evaluation. No rules or formulas can be given by which to determine the number of values needed; rather, this is a matter of engineering judgment. A few guides are mentioned in the following paragraphs that may assist in applying this judgment.

a. When the material is uniform, strength values should be determined at a minimum of five locations.

b. When the uniformity of material and construction is not known, the number of test locations should be sufficient to indicate that the values obtained are indeed representative of the area being tested.

c. When materials and placement conditions are clearly nonuniform, a relatively large number of test locations will be required to obtain a representative value.

d. The study is usually accomplished by plotting test results on profiles or by arranging them in tabular form to show the range of the data. In most cases, the value selected for use in the evaluation should be on the conservative side. It should not be the lowest value in a range, but it should be a “low average.” When conditions are uniform, one method that may be used satisfactorily is that of taking the lower quartile value from a cumulative distribution plot. Where conditions are not uniform, the following example may be helpful.

e. Example: A subgrade material beneath a Branch varies in such a manner that the Branch may be divided into several rather large areas of different subgrade materials. The in-place CBR values for the entire facility, arranged in occurring order, are as follows: 7, 7, 8, 9, 9, 10, 14, 14, 15, 16, 20, 21, 21, 22, 28, 28, 28, 30, 30, and 31. A study of in-place conditions reveals that the degree of saturation of the subgrade is about the same for the entire Branch and that it is sufficiently high so that the in-place CBR values can be used for evaluation. Preliminary analysis of these data shows that the statistical distribution for the whole Branch is not good and that the values logically fall into four groups or Sections. Each Section is represented by one of the areas of different material. CBR values for the Sections are 7,7,8,9,and 10; 14,14,15, and 16; 20,21,21,and 22; and 28, 28 30, 30, and 31. Using the low average concept, values of 8, 14, 21, and 28 for the four Sections are reasonable for evaluation.

f. Regardless of the number of values available and the method of selecting the evaluation figure, the number of values and the analytical process used should be described and discussed in the evaluation report in sufficient detail to be easily followed at a later date.

g. Because of certain inherent difficulties in processing samples for laboratory tests and in performing in-place tests on base-course materials, it is advisable to assign CBR values to certain materials based on their service behavior, as shown in Table 5-1.

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

Table 5-1. Assigned CBR values for Base Course Materials

5-5. **THICKNESS EQUIVALENCIES**. When a pavement has a thickness of base or surface that exceeds the minimum thickness required for design, the excess thickness of asphalt is converted to an equivalent thickness of base course and then added to the existing thickness of base. Any resulting excess thickness of base above the minimum thickness is then converted to an equivalent thickness of subbase material which is then added to the subbase thickness for evaluation. The equivalency factors for converting asphalt surfacing to base and subbase are 1.15 and 2.3 respectively, and for converting base course to subbase is 2.0, as shown in Table 5-2. This means that 1 inch of asphalt is equal to 1.15 inches of base and 2.3 inches of subbase, and 1 inch of base course is equal to 2.0 inches of subbase. The following example illustrates use of

Equivalency Factors.

Material	Base Equivalency Factor	Subbase Equivalency Factor
Unbound Crushed Stone	1.0	2.0
Unbound Subbase	-[1]	1.0
Asphalt-Stabilized		
All-Bituminous Concrete	1.15	2.30
GW, GP, GM, GC		
SW, SP, SM, SC	1.0	2.0
	-[1]	1.5
Cement-Stabilized		
GW, GP, SW, SP	1.15	2.30
GC, GM	1.0	2.0
ML, MH, CL, CH	-[1]	1.7
SC, SM	-1	1.5
Lime-Stabilized		
ML, MH, CL, CH	-[1]	1.0
SC, SM, GC, GM	-1	1.1
Lime, Cement, Fly Ash Stabilized		
ML, MH, CL, CH	-[1]	1.30
SC, SM, GC, GM	-1	1.40
[1] Not used as base course.		

5-2. Equivalency Factors

Example: A runway touchdown section is to be evaluated for C-130 operations. The measured thickness of the pavement section and the equivalent thickness used to evaluate the pavement are shown in the following table. The C-130 requires a minimum surface thickness of 4 inches and a minimum base thickness of 6 inches. The base is unbound crushed stone.

Layer	Measured Thickness (in.)	Equivalent Thickness of Base (in.)	Equivalent Thickness of Subbase (in.)	Evaluation Thickness (in)
Asphalt Surface	5			4
Base	7	8.15 (7.0 +1.x1.15)		6
Subbase	10		14.30 (10+2.15x2)	14.3
Subgrade	-	-		

5-6. **QUALITY OF BITUMINOUS PAVEMENT.**

a. Ability to Support Traffic. The ability of a mix to support traffic of a given load depends on the type and gradation of the aggregate, the amount of bitumen in the mix, and the compaction of the mix. Mixes with rounded aggregates are less stable than those with crushed-face aggregates; mixes with aggregates of irregular gradings are less stable than those with well-graded aggregates. A deficiency in bitumen produces a pavement that may ravel, but too much bitumen produces a pavement that may rut and shove. The condition of bituminous pavement, either surface or binder course, at the time of sampling is evaluated by comparing the test data from the core samples with the design criteria given in UFC 3-260-02 Pavement Design for Airfields. Future behavior of the pavement under additional traffic is predicted by comparing the test data from the laboratory recompacted specimens with the design criteria. The following example shows the prediction of behavior from tests on cores and on laboratory recompacted surface course specimens. Assume that the thickness and aggregate gradation are satisfactory. The other test data are as follows:

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

*For shoulders and overruns

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

b. Ability to Withstand Fuel Spillage.

(1) Asphaltic cements are readily soluble in jet fuels. Maximum distress is caused to asphaltic concrete pavements by fuel dripping on a given area at frequent intervals, or by the pavement mix being sufficiently pervious to allow considerable penetration of the fuel. The voids in the total mix control the rate at which penetration can occur. Fuel will penetrate very little into pavements with about 3 percent voids but will rapidly penetrate pavements with high (over 7 percent) voids. Weathering appears to increase the pavement's resistance to penetration of jet fuels, and pavements about 1 year or older usually perform better in this respect than new pavements.

The surface course characteristics should be evaluated for resistance to jet fuel. The following tabulation will serve as a guide for evaluating asphalt pavements from the standpoint of fuel spillage for use in different areas of the field.

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

c. Ability to Withstand Jet Blast.

(1) Tests have shown that about 300°F is the critical temperature for asphaltic concrete. Poorly bonded thin layers should be noted. Field tests simulating pretakeoff checks at the ends of runways indicate that the maximum temperatures induced in the pavements when afterburners are not used are less

than 300°F. Maximum temperatures induced in pavement tests simulating maintenance checkups are 315°F. When afterburners are turned on after the aircraft has begun the takeoff run, little or no damage occurs.

(2) Thin-surface courses, not well bonded to the underlying layers, are subject to being eroded by a high-velocity blast, even though the binder is not melted. All jet aircraft currently in use are believed to produce blasts of sufficiently high velocity to flay such courses. Set-back distances for running-up engines have been established and are included in AF ETL 07-3. Surface layers less than 1 inch thick and poorly bonded are considered unsatisfactory for parking areas, and the 1,000-foot ends of runways and will be so reported in the narrative portion of the evaluation report for all aircraft.

(3) The United States and its allies are developing and starting to use aircraft with innovative thrust vectors that may impact airfield pavements, depending on operational usage. When these aircraft are present, additional investigation should be considered.

5-7 **EFFECTS OF TRAFFIC COMPACTION**.

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

b. Base Course and Subgrade.

(1) In the construction of airfield pavements, definite degrees of compaction are specified for the subgrade and base course to prevent excessive densification under traffic and the consequent development of surface roughness “birdbaths”

and loss of grade. The specification of definite degrees of compaction is also necessary because the design CBR values are based on assumed degrees of compaction.

(2) To evaluate the base, subbase, and subgrade from the standpoint of future compaction, it is necessary to compare the in-place densities, in percentage of ASTM D 1557 maximum density, with the design requirements for the various loads and gear configurations that the pavement is expected to support. If it is found that the in-place density of a layer is appreciably lower than that required, it must be assumed that traffic will densify the layer in time. Density requirements at various depths are discussed in UFC 3-260-02 Pavement Design for Airfields.

(3.) The effect of further compaction on strength of base and subgrade should also be considered. Some cohesive soils, when highly saturated, may develop pore pressures under traffic of heavy wheel loads and show serious loss of strength. A clue to the possibility of this happening can be obtained by comparing the in-place density and moisture contents with those of the laboratory compaction tests made at three compaction efforts to determine the line of optimums. This is illustrated in Figure 5-67 by a line drawn through the three optimum moisture contents. Pore pressure seldom develops unless the moisture and density are such that, when plotted on a diagram similar to that of Figure 5-67, the point falls to the right of the line of optimums. Therefore, the moisture and density of the soil being tested can be plotted on the laboratory chart and studied to determine if future compaction will produce pore pressures. For example, consider point A plotted in Figure 5-66 at a moisture content of 16 percent and a density of 103 pounds per cubic foot. Assume this represents a subgrade that should have 95 percent of ASTM D 1557 maximum density. If further compaction occurs, the density will increase to about 105 pounds per cubic foot (point A’ on the curve for 26-blow effort). Since this is to the left of the line of optimums, no pore pressures will develop. If the example had been a subgrade with a moisture content of 18 percent (point B), the increased compaction would cause the density to be plotted to the right of the line of optimums (B’), and pore pressures would result. The CBR that would develop under this condition could be estimated from laboratory CBR tests in which the material was compacted to the same density and moisture content.

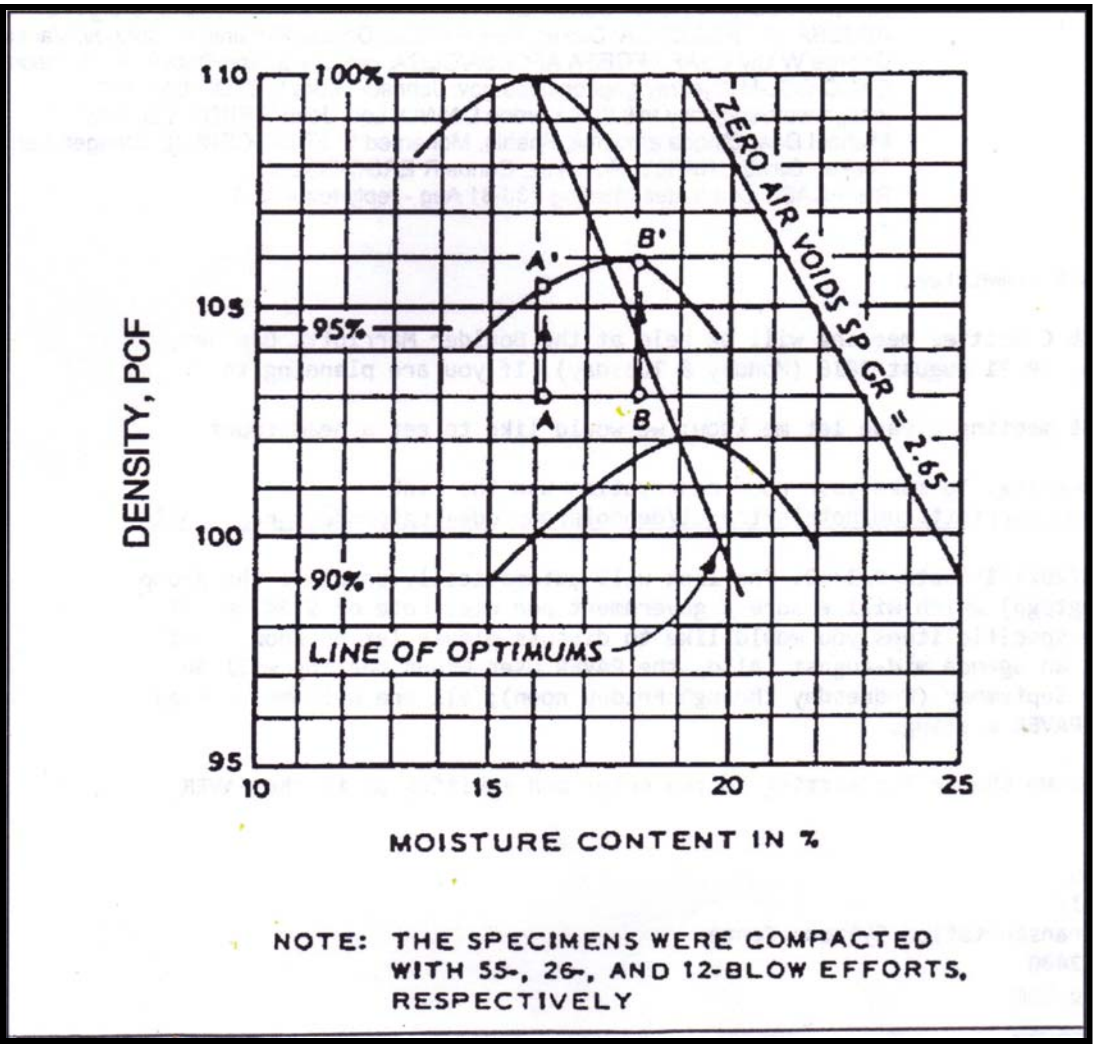


Figure 5-67. Line of Optimums

In an evaluation, lack of specified compaction will not make it necessary to lower the load-carrying capacity of the facility below that derived on the basis of thickness and CBR. However, if the measured densities are considerably less than those specified, this should be discussed in the evaluation report. It should be noted that materials of low density combined with low moisture content may not densify under traffic, but subsequent increases in moisture content will permit densification. Statements of possible amount of settlement due to densification should be included in the evaluation of pavements being subjected to channelized and heavy wheel-load traffic. In the case of cohesive materials that may develop pore pressures, a study of the possibility of loss in strength should be made and the lowest probable CBR estimated. This estimated value should be considered in selecting the evaluation CBR for the material.

5-8 **PASS/LOAD RELATIONSHIPS.**

a. **Evaluation Curves**. The evaluation of a flexible pavement to determine allowable gross load or passes requires knowing the thickness above the base, subbase(s), and subgrade courses and the selected CBR values for each of the layers. The aircraft curves in this UFC use the new stress-based CBR procedure and Constant Tire Pressure criteria. Evaluation curves previously published in the evaluation manuals were based on constant contact area assumption and the CBR procedure. Use of the newly approved criteria will result in curves or charts that may be considerably different from previous versions of this UFC.

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

Tire Contact Area =Load on Tire/Constant Tire Pressure

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

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

where

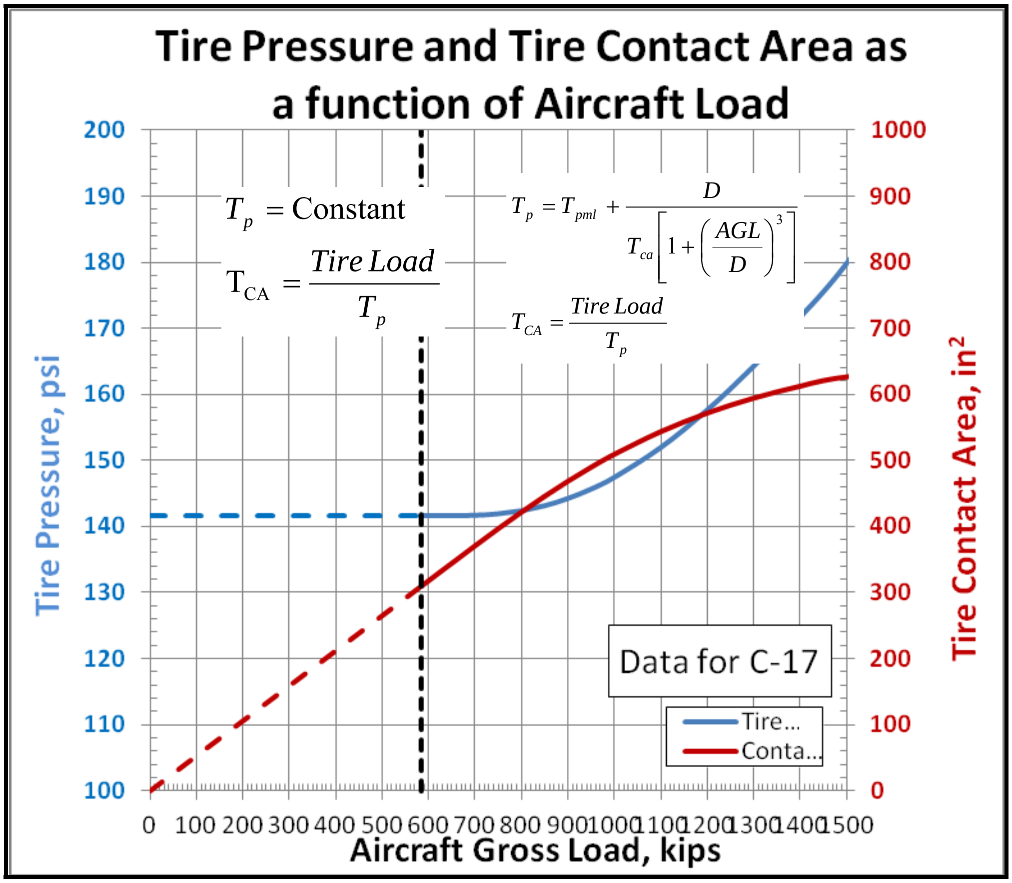
$T_p = Tire\ pressure\ used\ for\ calculations$

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

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

$D = AGL - MaxLoad$

An example of this relationship for the C-17 is shown below:

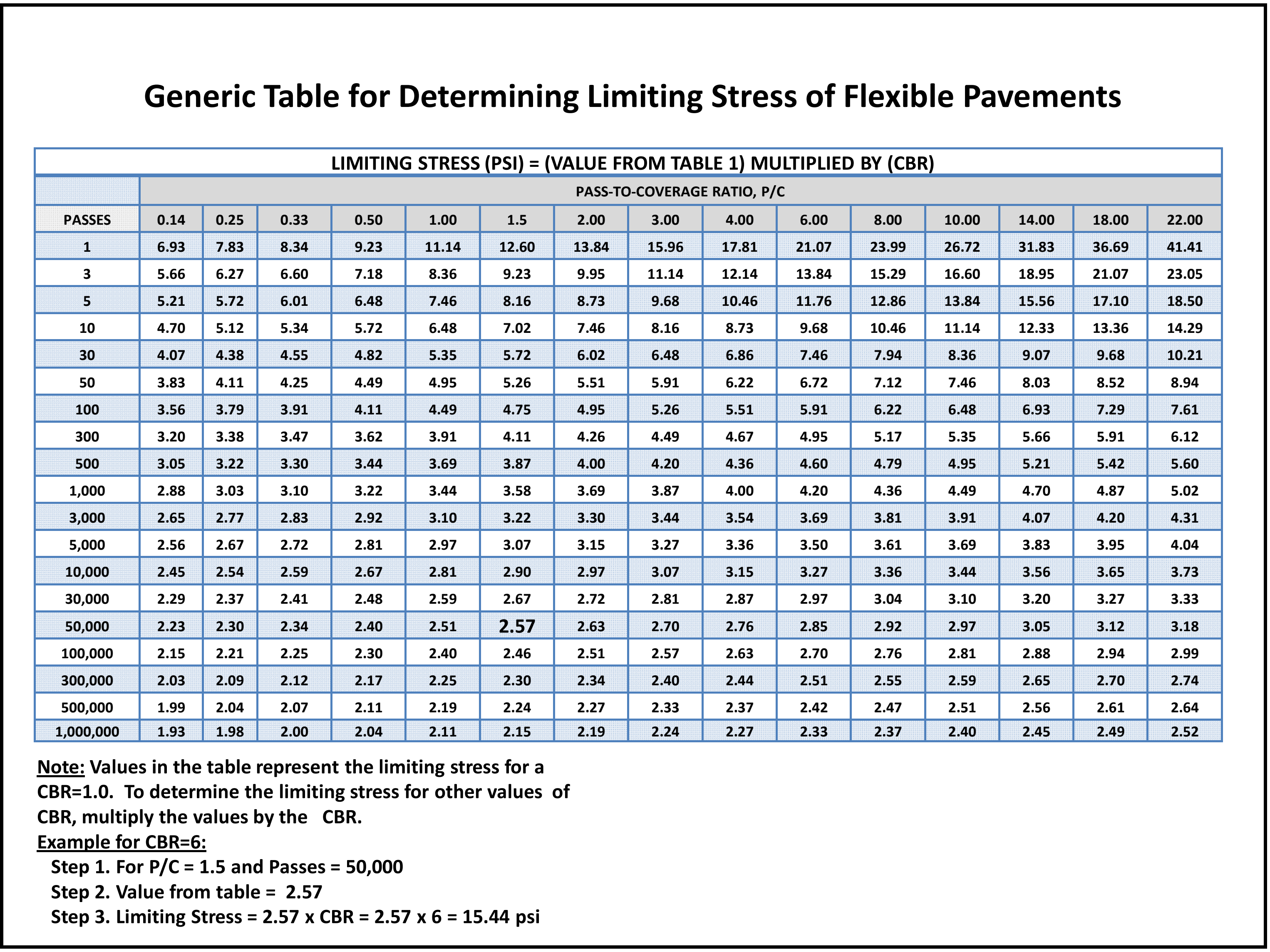


b. **Procedure for Determining Allowable Gross Load**. Figures 5-1 through 5-33 are used to determine the AGL for the 14 Air Force Groups and some specific aircraft. Use of these curves requires that the limiting (vertical) stress be determined for each layer in the pavement structure. The procedure for determining Allowable Gross Load is as follows:

Determine the limiting stress for each layer in the pavement structure (base, subbase(s), and subgrade) using Tables 5-3 or 5-4 and 5-5. Table 5-3 is a generic table that can be used for any aircraft for which the Pass- to- Coverage Ratio is known. Enter the table with the evaluation passes (vertical axis), then go horizontal to the Pass-to-Coverage Ratio for the evaluation aircraft. Multiply this value by the CBR for each layer to determine the limiting stress for that layer. This procedure may require interpolation. Then, use Figures 5-1 through 5-33 to determine the AGL.

Tables 5-4 and 5-5 provide the limiting stress for specific aircraft for a specific number of passes. Table 5-4 provides the limiting stress for “A” traffic areas and Table 5-5 the limiting stress for “B traffic areas. Multiply the published value by the CBR for each layer

to determine the limiting stress for each layer. The Allowable Load obtained for a "B" traffic area is multiplied by 1.33 to obtain the AGL for a "C" traffic area.
"D" traffic areas are not normally used in evaluation. Enter the Table with the evaluation aircraft, then horizontal to the evaluation passes.



Generic Table for Determining Limiting Stress of Flexible Pavements

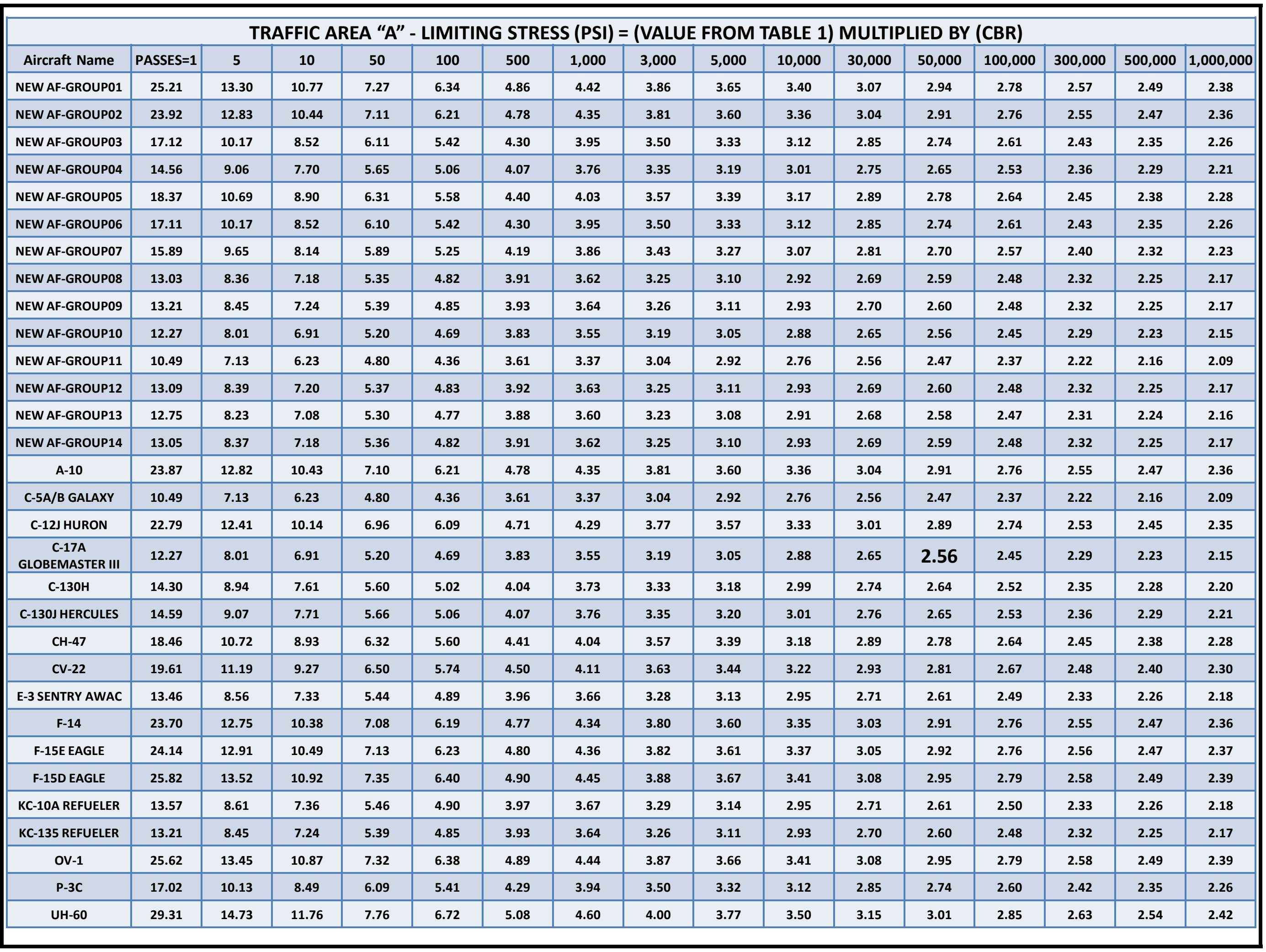
LIMITING STRESS (PSI) = (VALUE FROM TABLE 1) MULTIPLIED BY (CBR)														
	PASS-TO-COVERAGE RATIO, P/C													
PASSES	0.14	0.25	0.33	0.50	1.00	1.5	2.00	3.00	4.00	6.00	8.00	10.00	14.00	18.00	22.00
1	6.93	7.83	8.34	9.23	11.14	12.60	13.84	15.96	17.81	21.07	23.99	26.72	31.83	36.69	41.41
3	5.66	6.27	6.60	7.18	8.36	9.23	9.95	11.14	12.14	13.84	15.29	16.60	18.95	21.07	23.05
5	5.21	5.72	6.01	6.48	7.46	8.16	8.73	9.68	10.46	11.76	12.86	13.84	15.56	17.10	18.50
10	4.70	5.12	5.34	5.72	6.48	7.02	7.46	8.16	8.73	9.68	10.46	11.14	12.33	13.36	14.29
30	4.07	4.38	4.55	4.82	5.35	5.72	6.02	6.48	6.86	7.46	7.94	8.36	9.07	9.68	10.21
50	3.83	4.11	4.25	4.49	4.95	5.26	5.51	5.91	6.22	6.72	7.12	7.46	8.03	8.52	8.94
100	3.56	3.79	3.91	4.11	4.49	4.75	4.95	5.26	5.51	5.91	6.22	6.48	6.93	7.29	7.61
300	3.20	3.38	3.47	3.62	3.91	4.11	4.26	4.49	4.67	4.95	5.17	5.35	5.66	5.91	6.12
500	3.05	3.22	3.30	3.44	3.69	3.87	4.00	4.20	4.36	4.60	4.79	4.95	5.21	5.42	5.60
1,000	2.88	3.03	3.10	3.22	3.44	3.58	3.69	3.87	4.00	4.20	4.36	4.49	4.70	4.87	5.02
3,000	2.65	2.77	2.83	2.92	3.10	3.22	3.30	3.44	3.54	3.69	3.81	3.91	4.07	4.20	4.31
5,000	2.56	2.67	2.72	2.81	2.97	3.07	3.15	3.27	3.36	3.50	3.61	3.69	3.83	3.95	4.04
10,000	2.45	2.54	2.59	2.67	2.81	2.90	2.97	3.07	3.15	3.27	3.36	3.44	3.56	3.65	3.73
30,000	2.29	2.37	2.41	2.48	2.59	2.67	2.72	2.81	2.87	2.97	3.04	3.10	3.20	3.27	3.33
50,000	2.23	2.30	2.34	2.40	2.51	2.57	2.63	2.70	2.76	2.85	2.92	2.97	3.05	3.12	3.18
100,000	2.15	2.21	2.25	2.30	2.40	2.46	2.51	2.57	2.63	2.70	2.76	2.81	2.88	2.94	2.99
300,000	2.03	2.09	2.12	2.17	2.25	2.30	2.34	2.40	2.44	2.51	2.55	2.59	2.65	2.70	2.74
500,000	1.99	2.04	2.07	2.11	2.19	2.24	2.27	2.33	2.37	2.42	2.47	2.51	2.56	2.61	2.64
1,000,000	1.93	1.98	2.00	2.04	2.11	2.15	2.19	2.24	2.27	2.33	2.37	2.40	2.45	2.49	2.52

Note: Values in the table represent the limiting stress for a CBR=1.0. To determine the limiting stress for other values of CBR, multiply the values by the CBR.

Example for CBR=6:

Step 1. For P/C = 1.5 and Passes = 50,000
Step 2. Value from table = 2.57
Step 3. Limiting Stress = 2.57 x CBR = 2.57 x 6 = 15.44 psi

Table 5-3. Generic Table for Determining Limiting Stress of Flexible Pavements



TRAFFIC AREA “A” - LIMITING STRESS (PSI) = (VALUE FROM TABLE 1) MULTIPLIED BY (CBR)

Aircraft Name	PASSES=1	5	10	50	100	500	1,000	3,000	5,000	10,000	30,000	50,000	100,000	300,000	500,000	1,000,000
NEW AF-GROUP01	25.21	13.30	10.77	7.27	6.34	4.86	4.42	3.86	3.65	3.40	3.07	2.94	2.78	2.57	2.49	2.38
NEW AF-GROUP02	23.92	12.83	10.44	7.11	6.21	4.78	4.35	3.81	3.60	3.36	3.04	2.91	2.76	2.55	2.47	2.36
NEW AF-GROUP03	17.12	10.17	8.52	6.11	5.42	4.30	3.95	3.50	3.33	3.12	2.85	2.74	2.61	2.43	2.35	2.26
NEW AF-GROUP04	14.56	9.06	7.70	5.65	5.06	4.07	3.76	3.35	3.19	3.01	2.75	2.65	2.53	2.36	2.29	2.21
NEW AF-GROUP05	18.37	10.69	8.90	6.31	5.58	4.40	4.03	3.57	3.39	3.17	2.89	2.78	2.64	2.45	2.38	2.28
NEW AF-GROUP06	17.11	10.17	8.52	6.10	5.42	4.30	3.95	3.50	3.33	3.12	2.85	2.74	2.61	2.43	2.35	2.26
NEW AF-GROUP07	15.89	9.65	8.14	5.89	5.25	4.19	3.86	3.43	3.27	3.07	2.81	2.70	2.57	2.40	2.32	2.23
NEW AF-GROUP08	13.03	8.36	7.18	5.35	4.82	3.91	3.62	3.25	3.10	2.92	2.69	2.59	2.48	2.32	2.25	2.17
NEW AF-GROUP09	13.21	8.45	7.24	5.39	4.85	3.93	3.64	3.26	3.11	2.93	2.70	2.60	2.48	2.32	2.25	2.17
NEW AF-GROUP10	12.27	8.01	6.91	5.20	4.69	3.83	3.55	3.19	3.05	2.88	2.65	2.56	2.45	2.29	2.23	2.15
NEW AF-GROUP11	10.49	7.13	6.23	4.80	4.36	3.61	3.37	3.04	2.92	2.76	2.56	2.47	2.37	2.22	2.16	2.09
NEW AF-GROUP12	13.09	8.39	7.20	5.37	4.83	3.92	3.63	3.25	3.11	2.93	2.69	2.60	2.48	2.32	2.25	2.17
NEW AF-GROUP13	12.75	8.23	7.08	5.30	4.77	3.88	3.60	3.23	3.08	2.91	2.68	2.58	2.47	2.31	2.24	2.16
NEW AF-GROUP14	13.05	8.37	7.18	5.36	4.82	3.91	3.62	3.25	3.10	2.93	2.69	2.59	2.48	2.32	2.25	2.17
A-10	23.87	12.82	10.43	7.10	6.21	4.78	4.35	3.81	3.60	3.36	3.04	2.91	2.76	2.55	2.47	2.36
C-5A/B GALAXY	10.49	7.13	6.23	4.80	4.36	3.61	3.37	3.04	2.92	2.76	2.56	2.47	2.37	2.22	2.16	2.09
C-12J HURON	22.79	12.41	10.14	6.96	6.09	4.71	4.29	3.77	3.57	3.33	3.01	2.89	2.74	2.53	2.45	2.35
C-17A GLOBEMASTER III	12.27	8.01	6.91	5.20	4.69	3.83	3.55	3.19	3.05	2.88	2.65	**2.56**	2.45	2.29	2.23	2.15
C-130H	14.30	8.94	7.61	5.60	5.02	4.04	3.73	3.33	3.18	2.99	2.74	2.64	2.52	2.35	2.28	2.20
C-130J HERCULES	14.59	9.07	7.71	5.66	5.06	4.07	3.76	3.35	3.20	3.01	2.76	2.65	2.53	2.36	2.29	2.21
CH-47	18.46	10.72	8.93	6.32	5.60	4.41	4.04	3.57	3.39	3.18	2.89	2.78	2.64	2.45	2.38	2.28
CV-22	19.61	11.19	9.27	6.50	5.74	4.50	4.11	3.63	3.44	3.22	2.93	2.81	2.67	2.48	2.40	2.30
E-3 SENTRY AWAC	13.46	8.56	7.33	5.44	4.89	3.96	3.66	3.28	3.13	2.95	2.71	2.61	2.49	2.33	2.26	2.18
F-14	23.70	12.75	10.38	7.08	6.19	4.77	4.34	3.80	3.60	3.35	3.03	2.91	2.76	2.55	2.47	2.36
F-15E EAGLE	24.14	12.91	10.49	7.13	6.23	4.80	4.36	3.82	3.61	3.37	3.05	2.92	2.76	2.56	2.47	2.37
F-15D EAGLE	25.82	13.52	10.92	7.35	6.40	4.90	4.45	3.88	3.67	3.41	3.08	2.95	2.79	2.58	2.49	2.39
KC-10A REFUELER	13.57	8.61	7.36	5.46	4.90	3.97	3.67	3.29	3.14	2.95	2.71	2.61	2.50	2.33	2.26	2.18
KC-135 REFUELER	13.21	8.45	7.24	5.39	4.85	3.93	3.64	3.26	3.11	2.93	2.70	2.60	2.48	2.32	2.25	2.17
OV-1	25.62	13.45	10.87	7.32	6.38	4.89	4.44	3.87	3.66	3.41	3.08	2.95	2.79	2.58	2.49	2.39
P-3C	17.02	10.13	8.49	6.09	5.41	4.29	3.94	3.50	3.32	3.12	2.85	2.74	2.60	2.42	2.35	2.26
UH-60	29.31	14.73	11.76	7.76	6.72	5.08	4.60	4.00	3.77	3.50	3.15	3.01	2.85	2.63	2.54	2.42

Table 5-4. Limiting Stress for Specific Aircraft--A Traffic Area

Note: Values in the table represent the limiting stress for a CBR=1.0. To determine the limiting stress for other values of CBR, multiply the values obtained by the CBR.

Example for CBR=6:

Step 1. For 50,000 passes of the C-17
Step 2. Value from table = 2.56
Step 3. Limiting Stress = 2.56 x CBR = 2.56 x 6 = 15.36 psi

TRAFFIC AREA "B,C,D" - LIMITING STRESS (PSI) = (VALUE FROM TABLE 1) MULTIPLIED BY (CBR)

Aircraft Name	PASSES=1	5	10	50	100	500	1,000	3,000	5,000	10,000	30,000	50,000	100,000	300,000	500,000	1,000,000
NEW AF-GROUP01	31.24	15.37	12.20	7.97	6.88	5.18	4.68	4.05	3.82	3.54	3.19	3.05	2.88	2.65	2.56	2.44
NEW AF-GROUP02	29.01	14.63	11.69	7.72	6.69	5.07	4.59	3.99	3.76	3.49	3.15	3.01	2.84	2.62	2.53	2.42
NEW AF-GROUP03	22.25	12.21	10.00	6.88	6.04	4.68	4.26	3.74	3.55	3.31	3.00	2.88	2.73	2.53	2.44	2.34
NEW AF-GROUP04	18.68	10.81	9.00	6.36	5.62	4.43	4.05	3.58	3.40	3.19	2.90	2.79	2.65	2.46	2.38	2.29
NEW AF-GROUP05	24.20	12.93	10.51	7.14	6.24	4.80	4.37	3.82	3.61	3.37	3.05	2.92	2.77	2.56	2.47	2.37
NEW AF-GROUP06	22.06	12.14	9.95	6.86	6.02	4.67	4.26	3.74	3.54	3.30	2.99	2.87	2.72	2.52	2.44	2.34
NEW AF-GROUP07	20.31	11.46	9.46	6.60	5.82	4.55	4.15	3.66	3.47	3.25	2.95	2.83	2.69	2.49	2.41	2.31
NEW AF-GROUP08	15.87	9.64	8.13	5.89	5.25	4.19	3.86	3.43	3.27	3.07	2.81	2.70	2.57	2.40	2.32	2.23
NEW AF-GROUP09	15.58	9.51	8.04	5.84	5.21	4.16	3.84	3.41	3.25	3.06	2.79	2.69	2.56	2.39	2.32	2.23
NEW AF-GROUP10	13.60	8.63	7.38	5.47	4.91	3.97	3.67	3.29	3.14	2.96	2.71	2.62	2.50	2.33	2.27	2.18
NEW AF-GROUP11	11.25	7.51	6.52	4.97	4.51	3.71	3.45	3.11	2.98	2.82	2.60	2.51	2.40	2.25	2.19	2.11
NEW AF-GROUP12	14.97	9.24	7.84	5.73	5.12	4.11	3.79	3.38	3.22	3.03	2.77	2.67	2.54	2.37	2.30	2.21
NEW AF-GROUP13	14.28	8.93	7.61	5.60	5.02	4.04	3.73	3.33	3.18	2.99	2.74	2.64	2.52	2.35	2.28	2.20
NEW AF-GROUP14	13.72	8.68	7.42	5.49	4.93	3.98	3.68	3.30	3.14	2.96	2.72	2.62	2.50	2.34	2.27	2.18
A-10	33.95	16.24	12.79	8.25	7.09	5.30	4.78	4.13	3.89	3.60	3.23	3.09	2.91	2.68	2.58	2.47
C-5A/B GALAXY	11.24	7.51	6.52	4.97	4.51	3.71	3.45	3.11	2.98	2.82	2.60	2.51	2.40	2.25	2.19	2.11
C-12J HURON	31.87	15.58	12.34	8.04	6.93	5.21	4.70	4.07	3.84	3.56	3.20	3.06	2.88	2.65	2.56	2.45
C-17A GLOBEMASTER III	13.60	8.63	7.38	5.47	4.91	3.97	3.67	3.29	3.14	2.96	2.71	**2.62**	2.50	2.33	2.27	2.18
C-130H	18.29	10.65	8.88	6.30	5.57	4.39	4.03	3.56	3.38	3.17	2.89	2.78	2.64	2.45	2.37	2.28
C-130J HERCULES	18.72	10.83	9.01	6.36	5.63	4.43	4.06	3.59	3.40	3.19	2.90	2.79	2.65	2.46	2.38	2.29
CH-47	22.89	12.45	10.17	6.97	6.10	4.72	4.30	3.77	3.57	3.33	3.02	2.89	2.74	2.54	2.45	2.35
CV-22	20.28	11.45	9.46	6.60	5.82	4.54	4.15	3.66	3.47	3.24	2.95	2.83	2.69	2.49	2.41	2.31
E-3 SENTRY AWAC	16.45	9.89	8.32	5.99	5.33	4.24	3.90	3.47	3.30	3.09	2.83	2.72	2.59	2.41	2.34	2.25
F-14	33.58	16.13	12.71	8.21	7.06	5.29	4.76	4.12	3.88	3.59	3.22	3.08	2.91	2.67	2.58	2.46
F-15E EAGLE	29.31	14.73	11.76	7.76	6.71	5.08	4.60	4.00	3.77	3.50	3.15	3.01	2.85	2.63	2.54	2.42
F-15D EAGLE	31.60	15.49	12.28	8.01	6.91	5.20	4.69	4.06	3.83	3.55	3.19	3.05	2.88	2.65	2.56	2.45
KC-10A REFUELER	15.56	9.50	8.03	5.84	5.21	4.16	3.83	3.41	3.25	3.05	2.79	2.69	2.56	2.39	2.32	2.23
KC-135 REFUELER	16.10	9.73	8.20	5.93	5.28	4.21	3.88	3.44	3.28	3.08	2.81	2.71	2.58	2.40	2.33	2.24
OV-1	31.56	15.48	12.27	8.00	6.90	5.19	4.69	4.06	3.83	3.55	3.19	3.05	2.88	2.65	2.56	2.45
P-3C	22.00	12.11	9.93	6.85	6.01	4.66	4.25	3.73	3.54	3.30	2.99	2.87	2.72	2.52	2.44	2.34
UH-60	36.07	16.91	13.23	8.46	7.25	5.39	4.85	4.18	3.93	3.64	3.26	3.11	2.94	2.70	2.60	2.48

Table 5-5. Limiting Stress for Specific Aircraft-B Traffic Area

Note: Multiply B traffic area AGL x 1.33 to obtain C traffic area AGL.

Note: Values in Table 5-5 represent the limiting stress for a CBR=1.0. To determine the limiting stress for other values of CBR, multiply the values by the CBR.

Example for CBR=6:
Step 1. For 50,000 passes of the C-17
Step 2. Value from table = 2.62
Step 3. Limiting Stress = 2.62 x CBR = 2.62 x 6 = 15.72 psi

c. **Example use of Evaluation Curves to Determine AGL.** Evaluate an A Traffic

Area for 50,000 passes of a C-17 aircraft on a flexible pavement having a subgrade CBR of 6, a 13-inch subbase with CBR of 30, an 8-inch base with a CBR of 80, and a 3-inch asphalt surface course, for a total thickness of 24 inches. The combinations of CBR and thickness above the subgrade, subbase, and base course must be evaluated to determine the weakest combination. From Table 5-4, the value for 50,000 passes of a C-17 is 2.56. This value is multiplied by the CBR of each layer to determine the limiting stress of that layer. Using Figure 5-28, the layer having the least allowable load, the subgrade, controls the evaluation, as shown below.

Layer	Depth from Surface (in)	CBR Value	Limiting Stress	Allowable Load(pounds)
Base	3	80	80x 2.56=204.8	900,000 *
Subbase	11	30	30x2.56=76.8	650,000*
Subgrade	24	6	6x2.56=15.4	360,000

*Exceeds maximum weight of the C-17, 585,000 pounds

When evaluating for more than one aircraft, the weakest combination must be determined for all aircraft, since the same weak condition may not govern for all aircraft.

Assume the above pavement is a “C” traffic area. Determine the limiting stress from Table 5-5. For the C-17 at 50,000 passes, the value from Table 5-5 is 2.62. The limiting stress for the base, subbase, and subgrade is determined by multiplying 2.62 times the CBR of each layer. Each layer in the pavement is evaluated using Figure 5-28, as shown in the table below. The subgrade controls the evaluation. Note that the AGL determined from Table 5-5 is multiplied times 1.33 to obtain the AGL for a “C” traffic area.

Layer	Depth from Surface (in)	CBR Value	Limiting Stress	Allowable Load (Pounds)
Base	3	80	80 x 2.62 = 209.6	585,000*
Subbase	11	30	30 x 2.62 = 78.6	585,000*
Subgrade	24	6	6 x 2.62 = 15.7	365,000 x 1.33= 484,000

*Exceeds maximum weight of the C-17, 585,000 pounds

d**. Procedure for Determining Allowable Passes**:

-Enter the appropriate curve for the evaluation aircraft (Figures 5-1 to 5-33) with the evaluation load, CBR, and total thickness from the surface for each layer in the pavement structure to determine the Limiting Stress for that layer.

-Divide the Limiting Stress by the CBR

-Enter the appropriate evaluation curve (Figures 5-34 through 5-66) to determine the Allowable Passes.

e**. Example for determining Allowable Passes.** Determine the Allowable Passes for a 585,000 pound C-17 on the A and C traffic areas in the above example. Since the subgrade controls both evaluations, only consider the subgrade in this example.

Enter Figure 5-28 with the AGL=585,000, T=24 inches, and CBR= 6.This results in a Limiting Stress = 23.5 psi. Divide the Limiting Stress by the CBR of 6, 23.5/6=3.92. From Figure 5-61, Allowable Passes for an A traffic area=approximately 360 passes.

For the C Traffic area, divide 585,000 by 1.33=440,000 pounds. Enter Figure 5-28 with an AGL=440,000 pounds, T=24 inches and CBR=6 to obtain a Limiting Stress of 18.0 psi. Divide the Limiting Stress by the CBR=6, 18.0/6=3.0. Enter Figure 5-61 with Limiting Stress=3.0, go horizontal to the Traffic Area C curve and obtain approximately 6,000 passes.

5-9 **PAVEMENT CLASSIFICATION NUMBER**. In addition to evaluating airfield pavements for allowable load, using the above procedures (Paragraph 5-5), it is also necessary to report weight-bearing capacity of pavements in terms of the PCN. The PCN can then be compared with an Aircraft Classification Number (ACN) to determine if a pavement can support a particular aircraft. The PCN procedure is presented in Chapter 8.

5-10 **EVALUATIONS FOR ARID REGIONS**. The danger of saturation beneath flexible pavements is reduced when the annual rainfall is less than 15 inches, the water table (including perched water table) is at least 15 feet below the surface, and the water content of the subgrade will not increase above the optimum as determined by the ASTM D 1557 compaction test. Under such conditions, the total design thickness of the pavement, when based on a soaked CBR, can be reduced 20 percent. This reduction will be subtracted from the thickness of the select material or the subbase course having the lowest design CBR value. Therefore, when flexible pavements are evaluated using a soaked CBR value, the total thickness above the subgrade will be increased 25 percent before entering the evaluation curves. This increase in thickness will be added to the select material or the subbase course having the lowest CBR or to the same layer in which the reduction was made in the design analysis. **This increase in thickness would not apply for evaluations using in-place data.**

5-11 **EVALUATIONS FOR FROST CONDITIONS**. If the existing soil, water, and temperature conditions are conducive to detrimental frost effects in the base-course, subbase, or subgrade materials, then the pavement evaluation will be based on criteria for frost areas as given in Chapter 7 of this manual.

5-12 **EVALUATIONS FOR STABILIZED LAYERS**. Stabilized layers are incorporated in the design of pavement sections to make use of locally available materials that cannot otherwise meet the criteria for base or subbase courses. Materials must meet the requirements in UFC 3-250-11 Soil Stabilization for Pavements. When stabilized layers are used in design, equivalency factors assigned to the material result in a reduction in thickness requirements as compared with an unbound base course or subbase course. Therefore, for evaluating a stabilized layer, an equivalency factor is applied that results in an increase in thickness of the layer. Equivalency factors are determined from Table 5-2. If no information is available on the condition and strength of the stabilized layer, it should be treated as a high-quality granular layer. If DCP results indicate the layer is well stabilized (refusal for DCP), then the layer should be considered for the equivalency factors. As an example, assume that an Air Force pavement structure consists of a 4-inch asphaltic concrete, a 8-inch bituminous concrete base, and an 8-inch cement-stabilized gravelly clay subbase with an unconfined compressive strength of 700 pounds per square inch. From table 5-2 the 8-inch bituminous concrete base equivalency factor is 1.15 which would increase the thickness of the stabilized base for evaluation to 9.2 inches. Also from Table 5-2, the 8-inch cement-stabilized subbase equivalency factor is 2.0 which would increase the thickness of the stabilized subbase for evaluation to 16 inches.

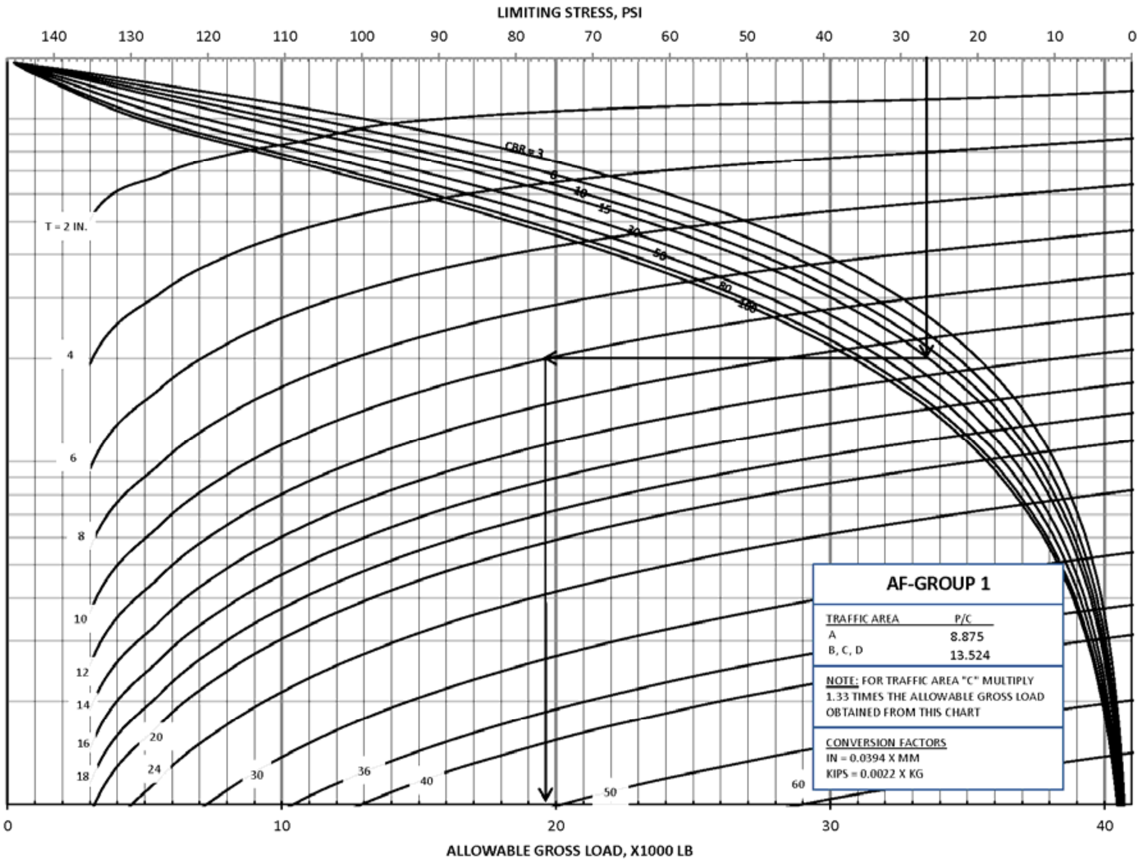


Figure 5-1. Flexible evaluation curve for Air Force Group 1 (AGL)

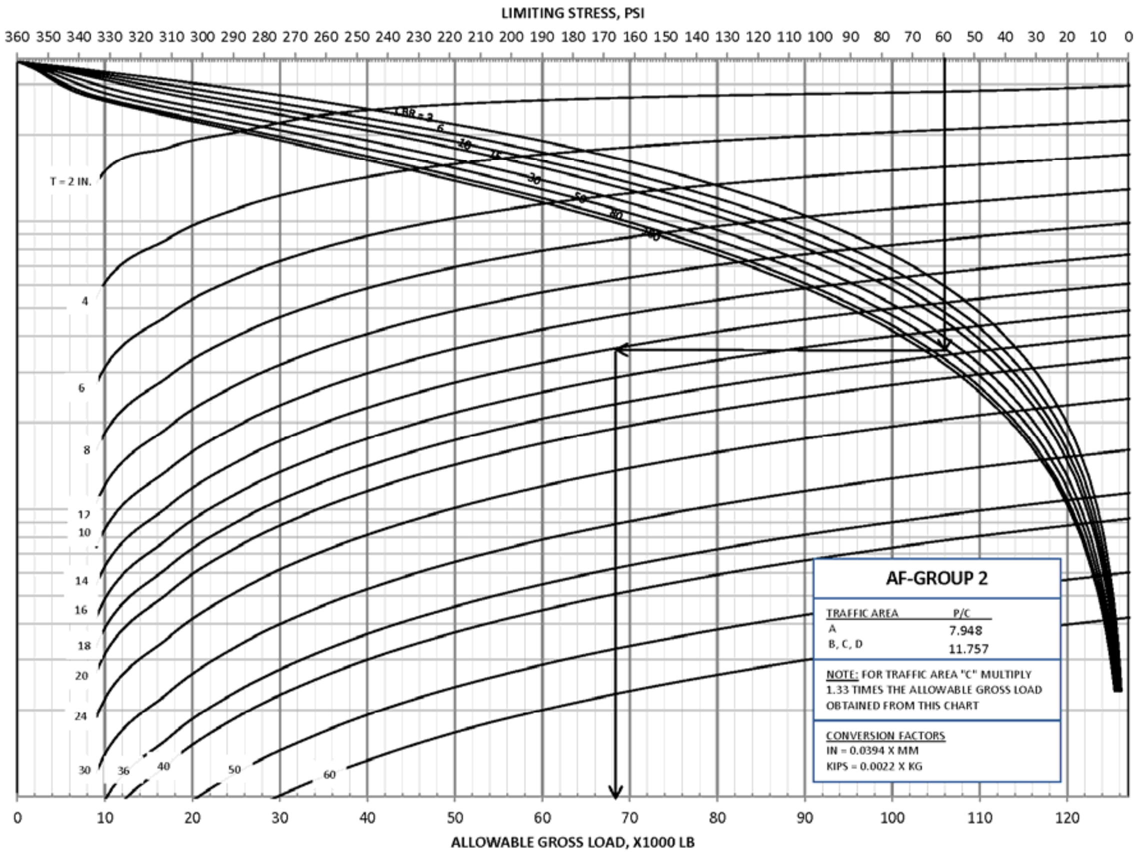


Figure 5-2. Flexible evaluation curve for Air Force Group 2 (AGL)

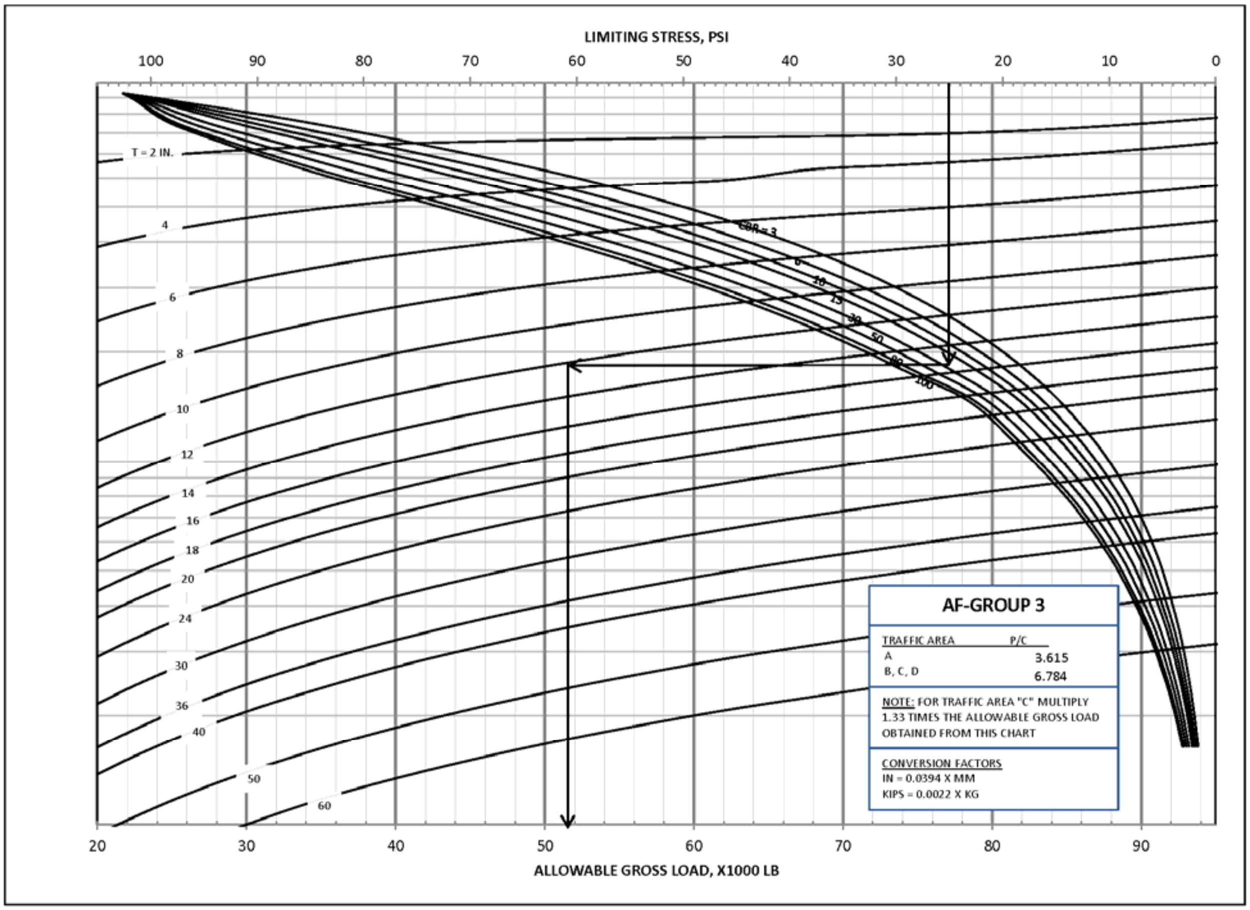
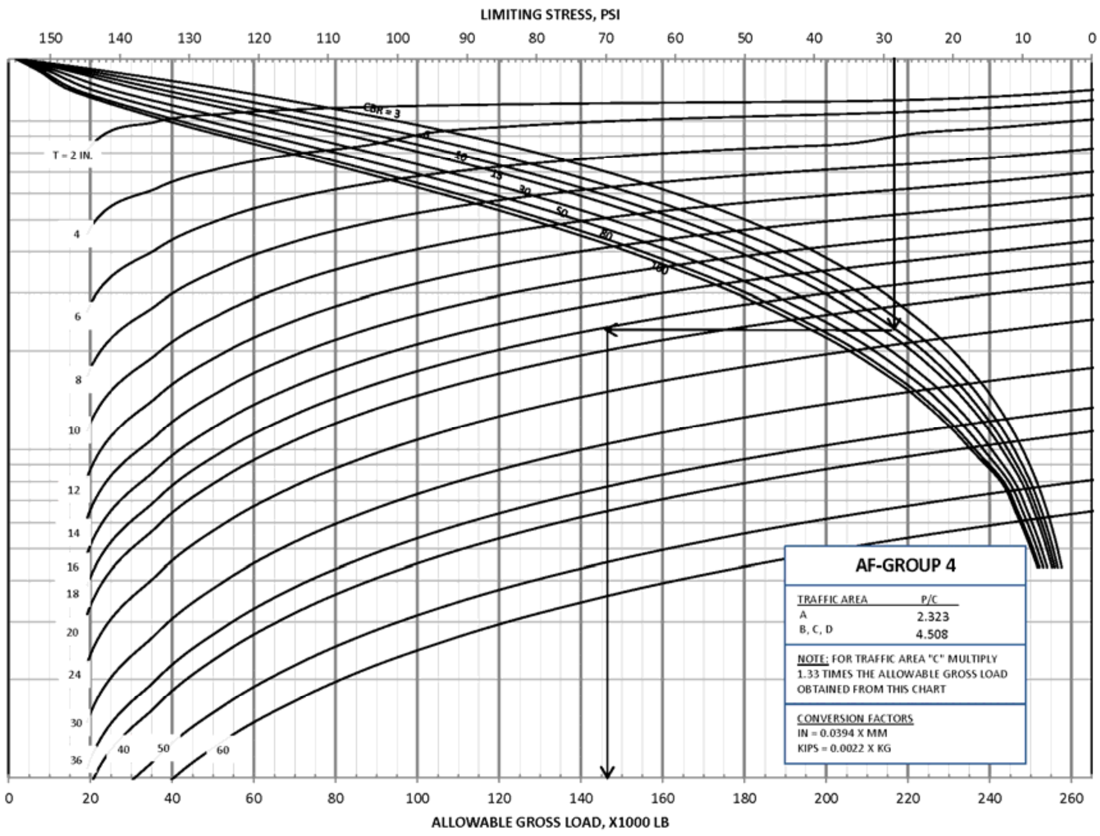


Figure 5-3. Flexible evaluation curve for Air Force Group 3 (AGL)



AF-GROUP 4

TRAFFIC AREA	P/C
A	2.323
B, C, D	4.508

NOTE: FOR TRAFFIC AREA "C" MULTIPLY 1.33 TIMES THE ALLOWABLE GROSS LOAD OBTAINED FROM THIS CHART

CONVERSION FACTORS
IN = 0.0394 X MM
KIPS = 0.0022 X KG

Figure 5-4. Flexible evaluation curve for Air Force Group 4 (AGL)

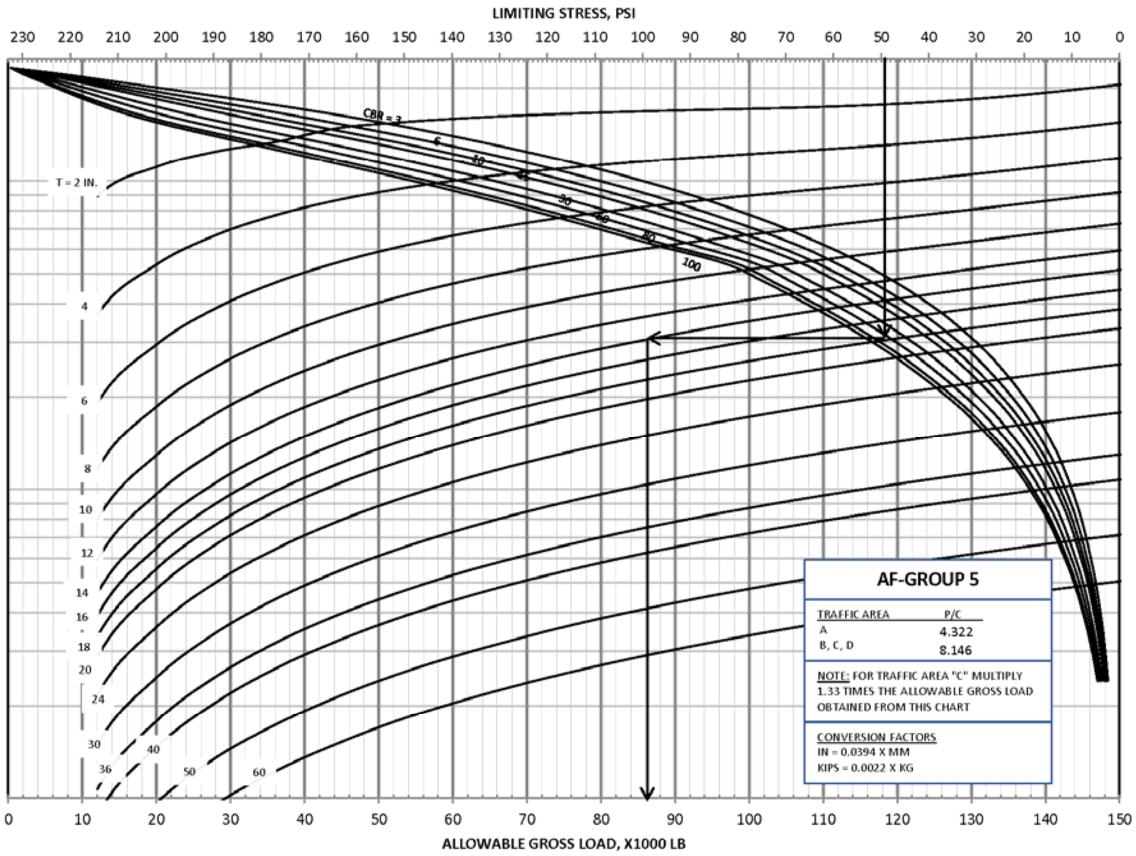
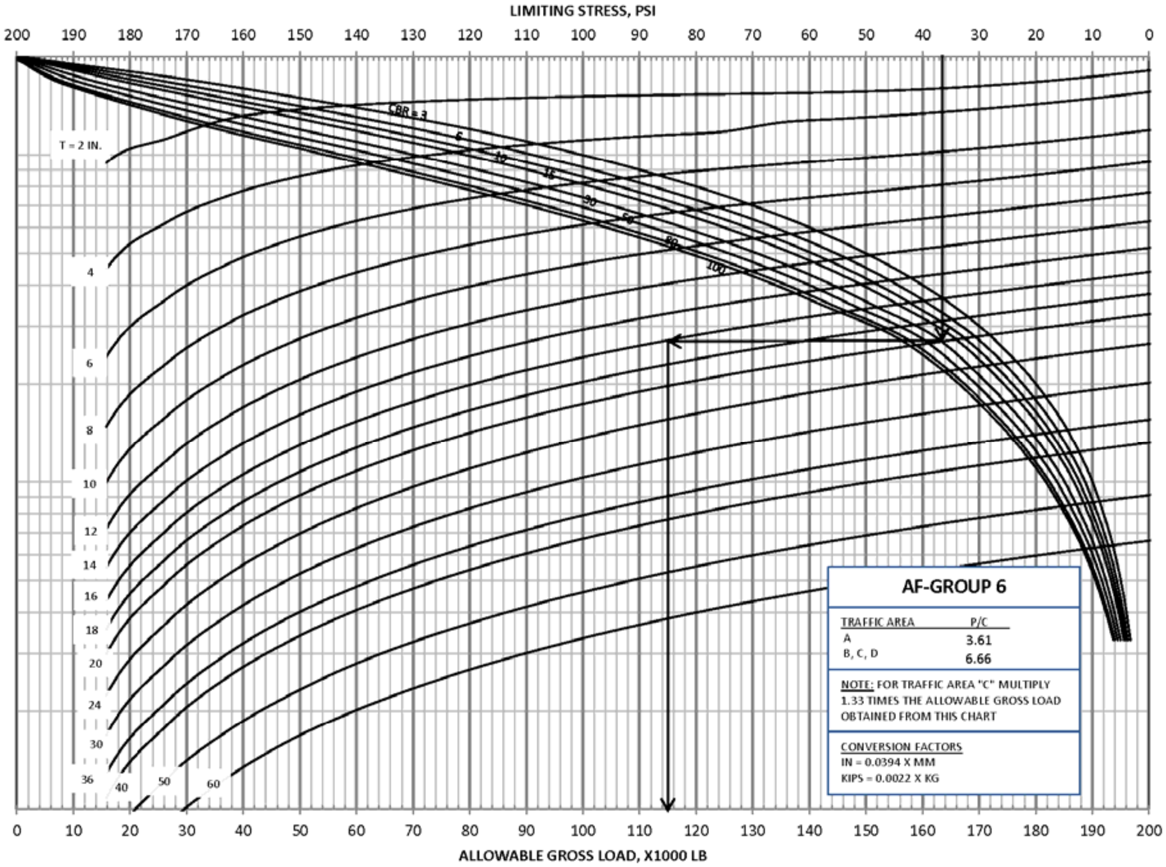


Figure 5-5. Flexible evaluation curve for Air Force Group 5 (AGL



AF-GROUP 6

TRAFFIC AREA	P/C
A	3.61
B, C, D	6.66

NOTE: FOR TRAFFIC AREA "C" MULTIPLY 1.33 TIMES THE ALLOWABLE GROSS LOAD OBTAINED FROM THIS CHART

CONVERSION FACTORS
IN = 0.0394 X MM
KIPS = 0.0022 X KG

Figure 5-6. Flexible evaluation curve for Air Force Group 6 (AGL)

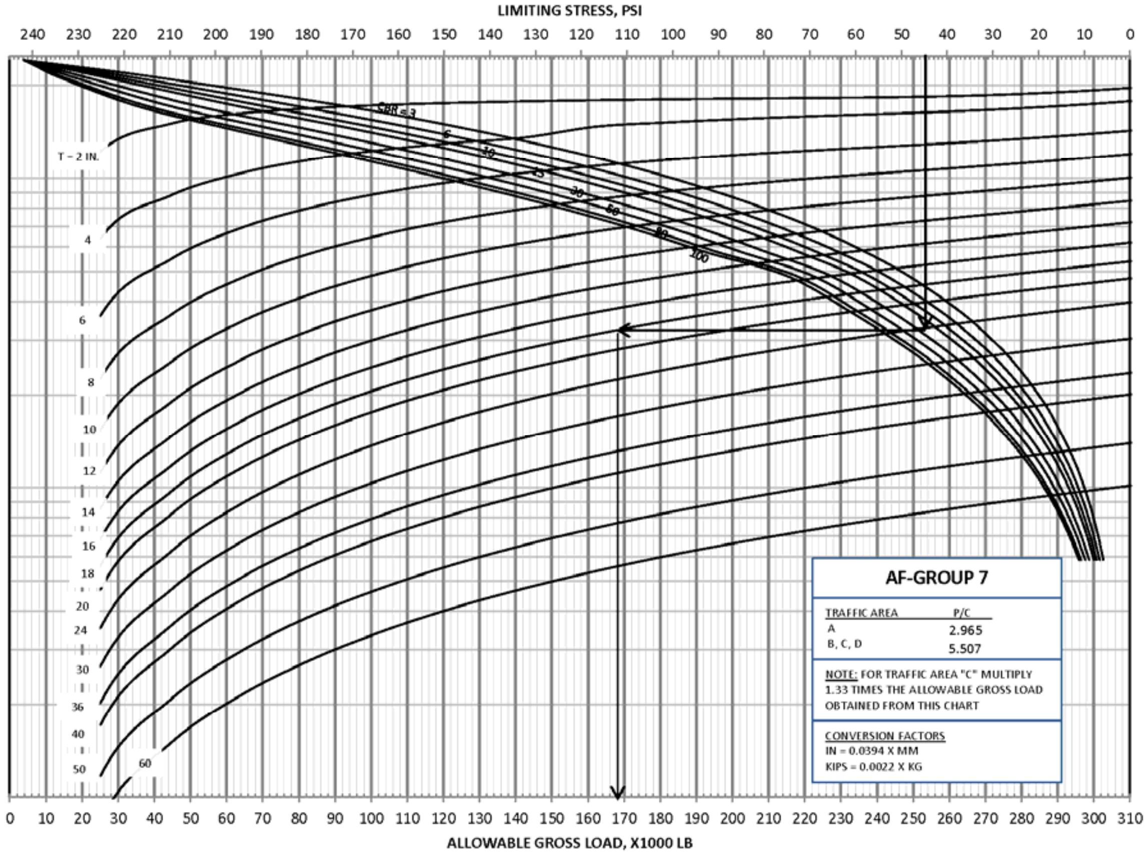


Figure 5-7. Flexible evaluation curve for Air Force Group 7 (AGL)

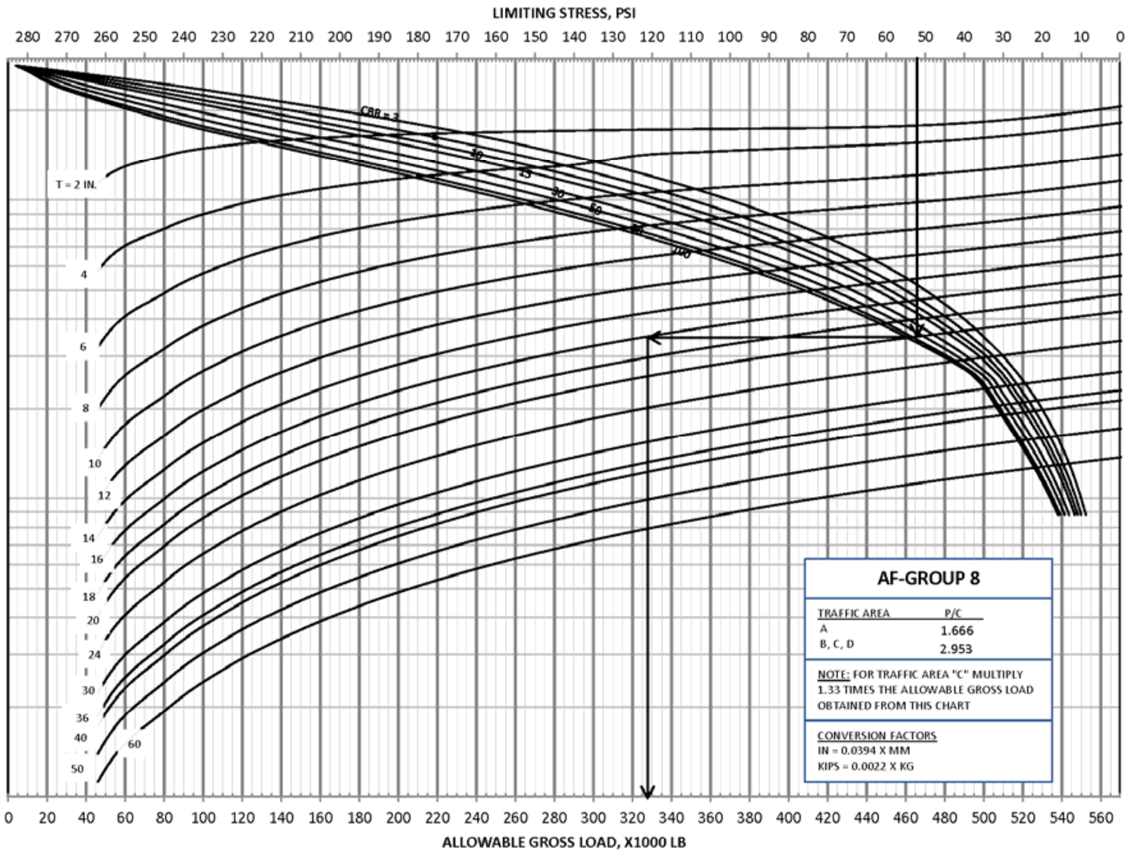


Figure 5-8. Flexible evaluation curve for Air Force Group 8 (AGL)

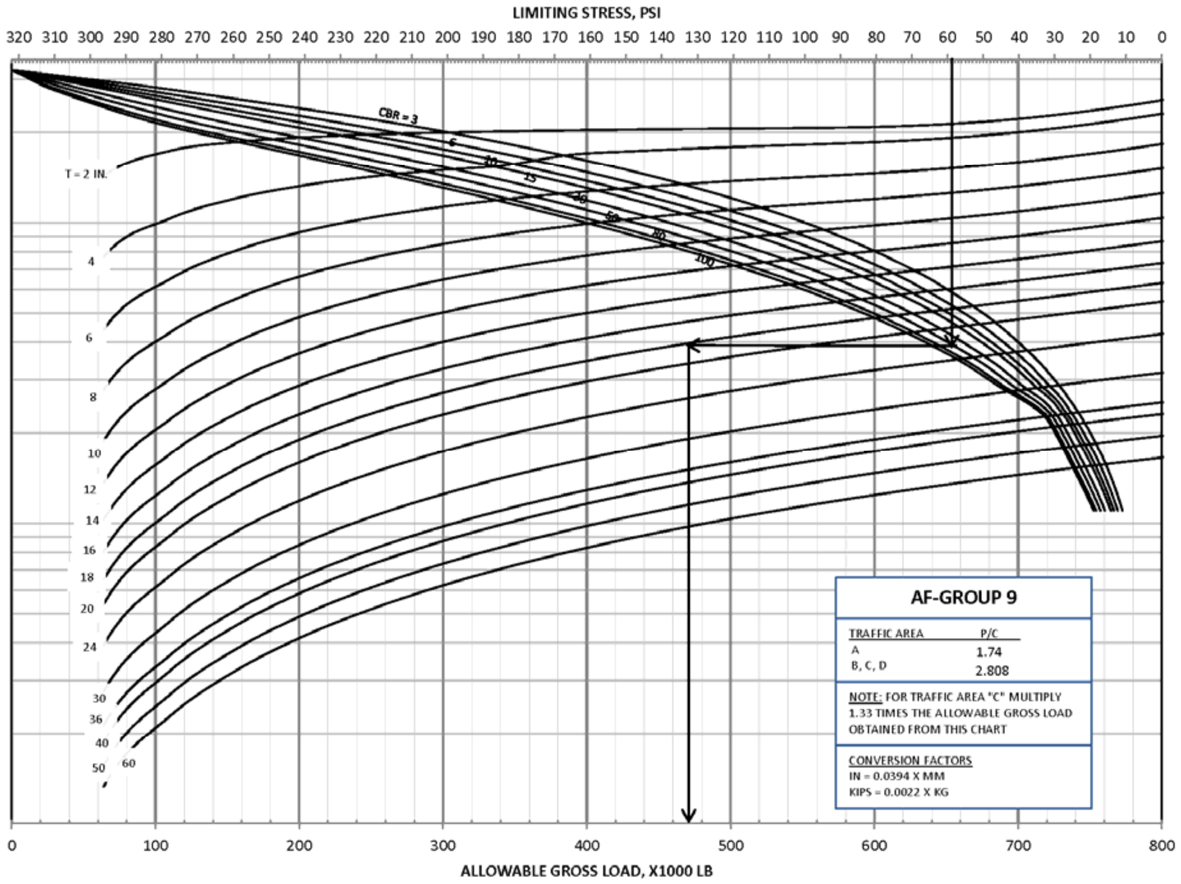


Figure 5-9. Flexible evaluation curve for Air Force Group 9 (AGL)

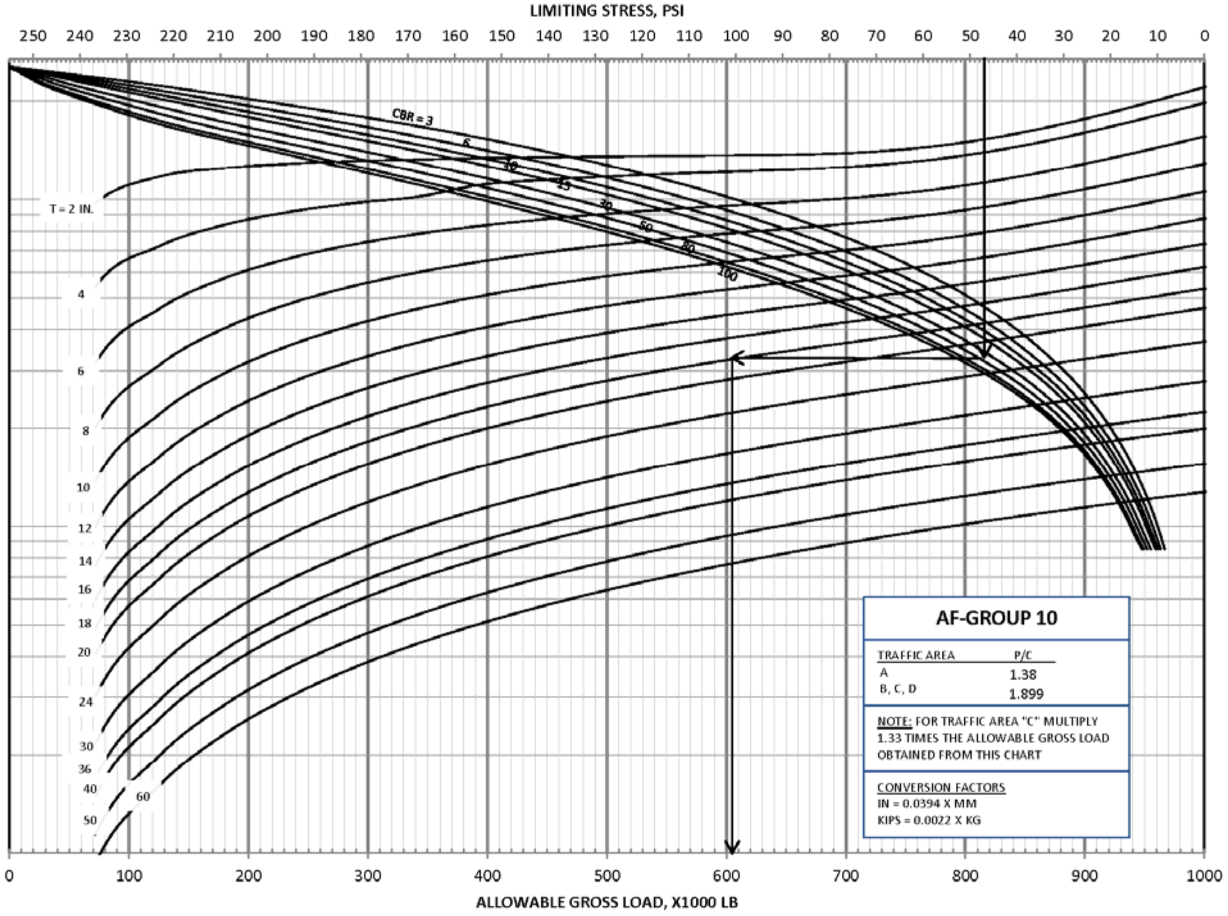


Figure 5-10. Flexible evaluation curve for Air Force Group 10 (AGL)

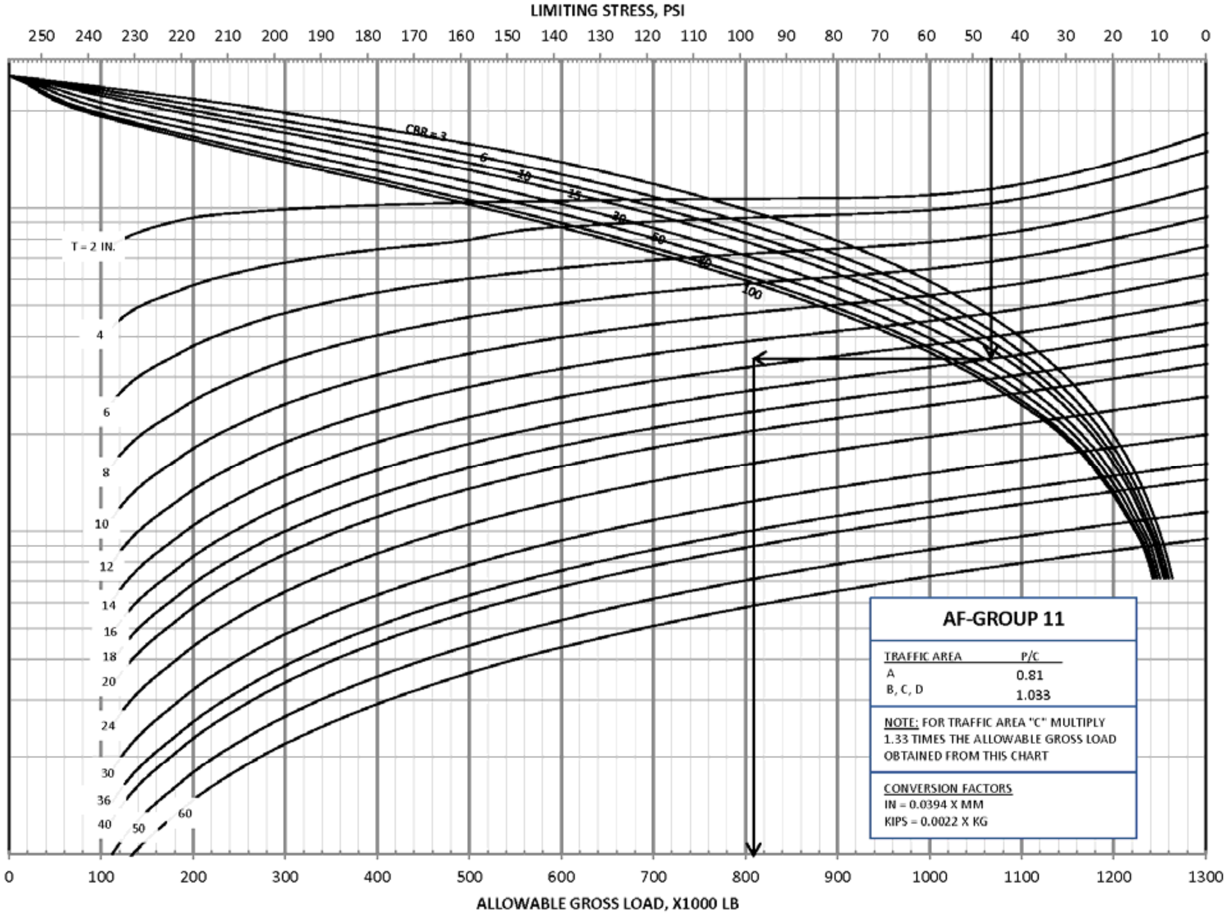


Figure 5-11. Flexible evaluation curve for Air Force Group 11 (AGL)

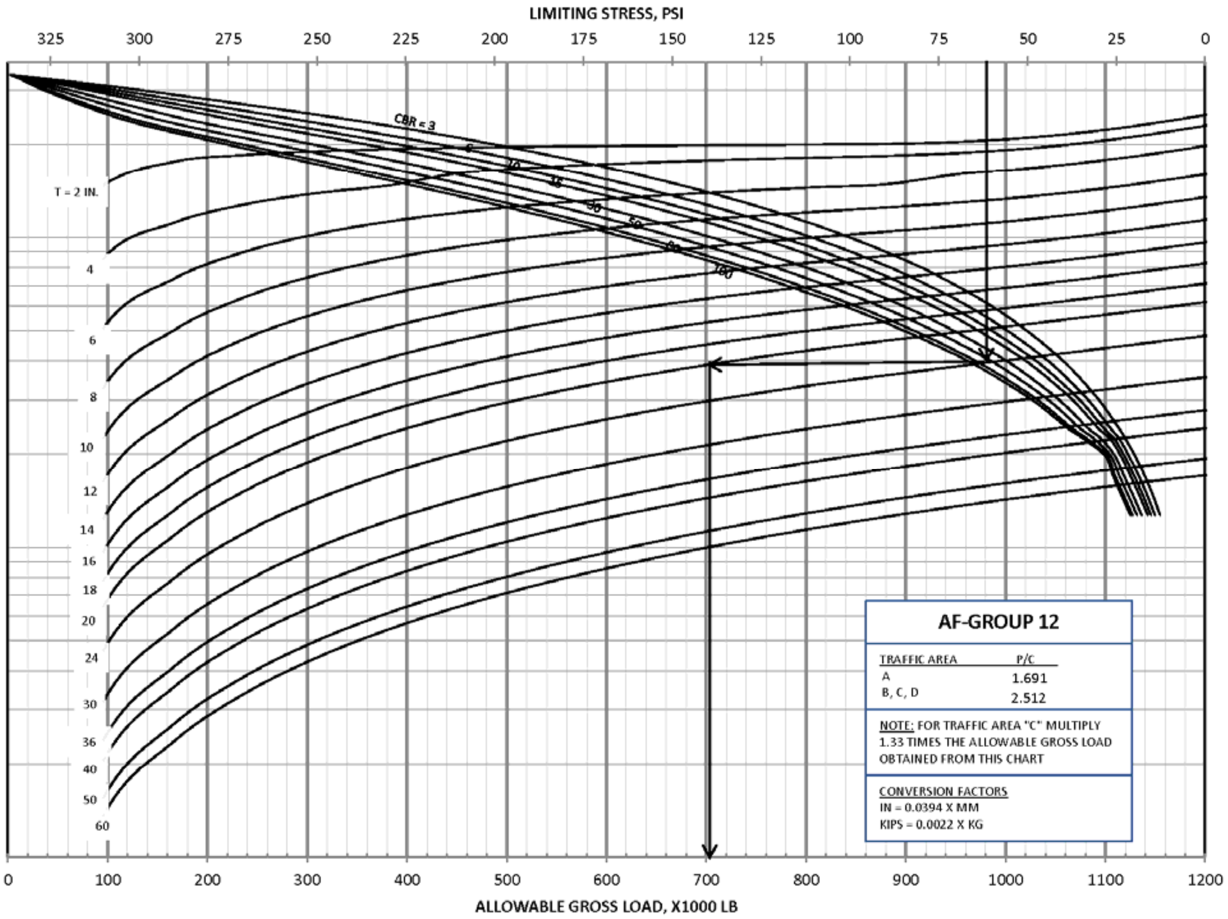
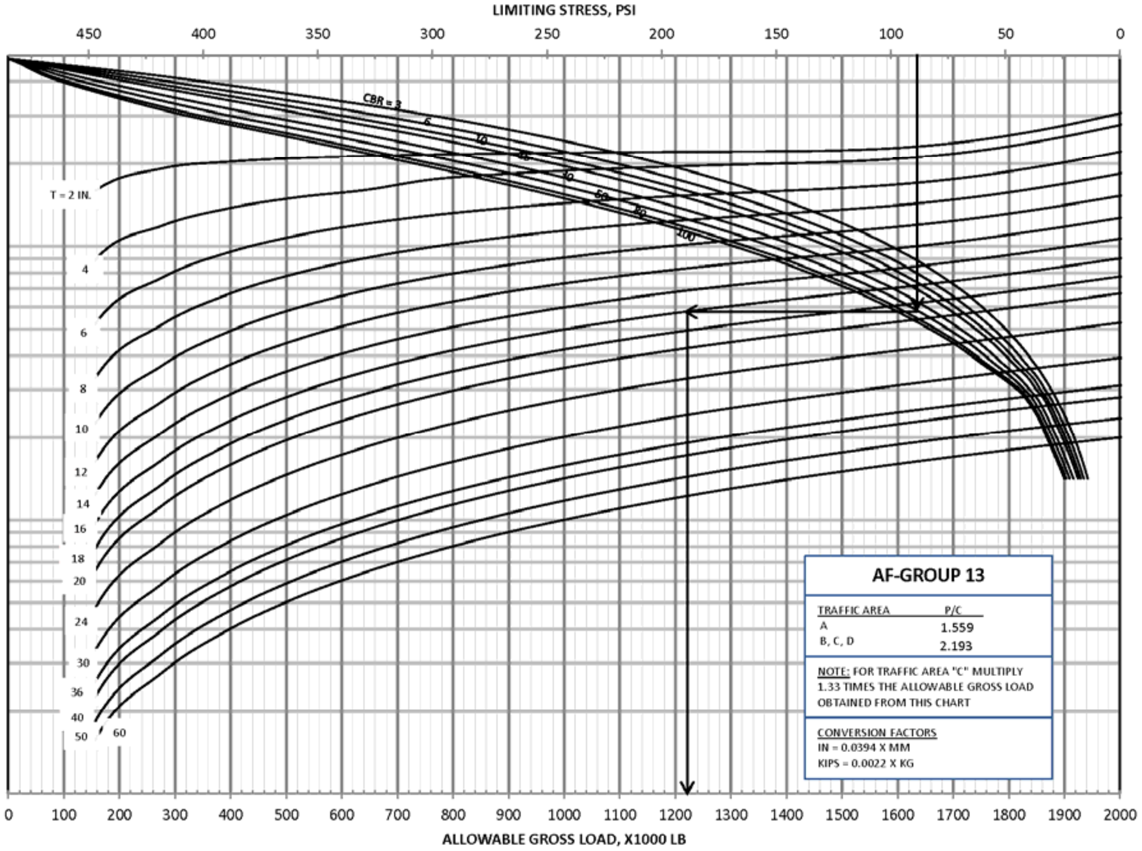


Figure 5-12. Flexible evaluation curve for Air Force Group 12 (AGL)



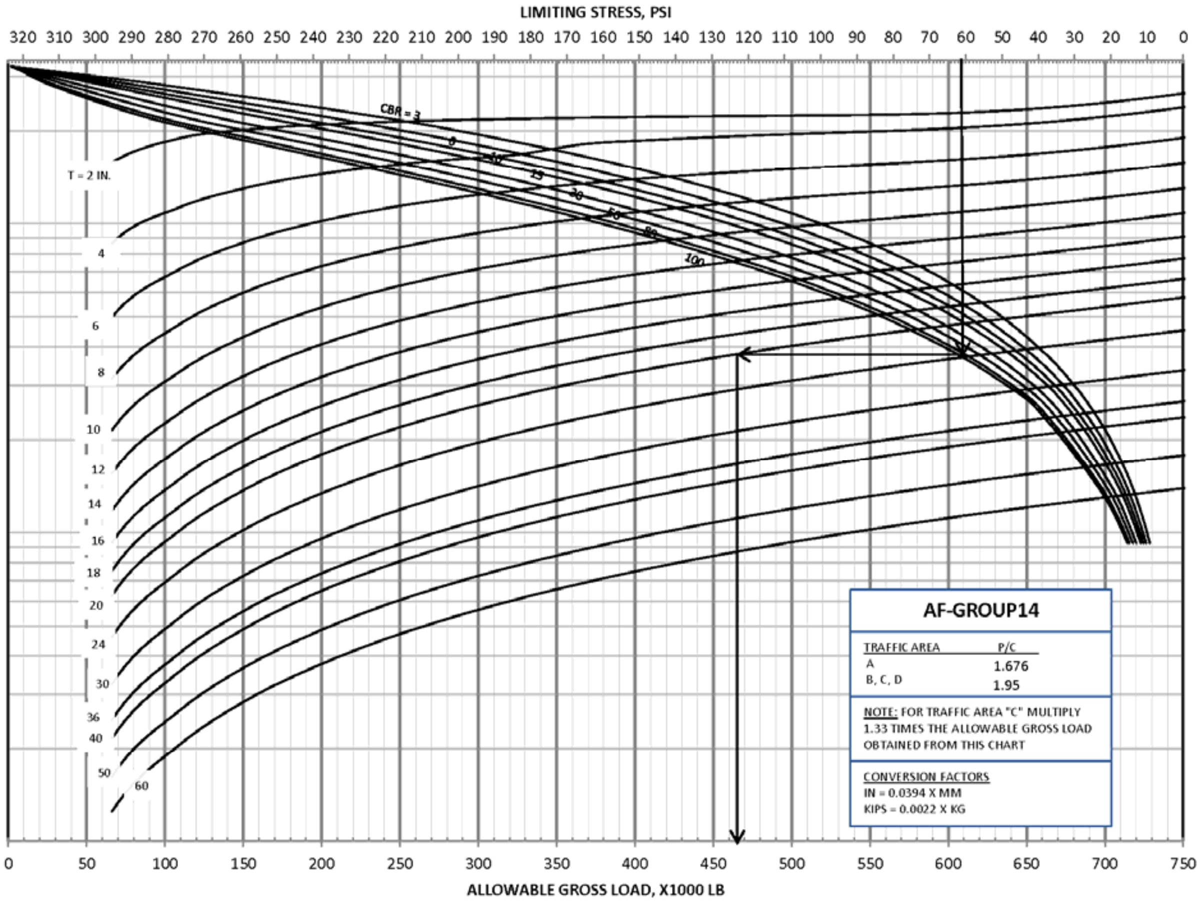
AF-GROUP 13

TRAFFIC AREA	P/C
A	1.559
B, C, D	2.193

NOTE: FOR TRAFFIC AREA "C" MULTIPLY 1.33 TIMES THE ALLOWABLE GROSS LOAD OBTAINED FROM THIS CHART

CONVERSION FACTORS
IN = 0.0394 X MM
KIPS = 0.0022 X KG

Figure 5-13. Flexible evaluation curve for Air Force Group 13 (AGL)



TRAFFIC AREA	P/C
A	1.676
B, C, D	1.95

NOTE: FOR TRAFFIC AREA "C" MULTIPLY 1.33 TIMES THE ALLOWABLE GROSS LOAD OBTAINED FROM THIS CHART

CONVERSION FACTORS
IN = 0.0394 X MM
KIPS = 0.0022 X KG

Figure 5-14. Flexible evaluation curve for Air Force Group 14 (AGL)

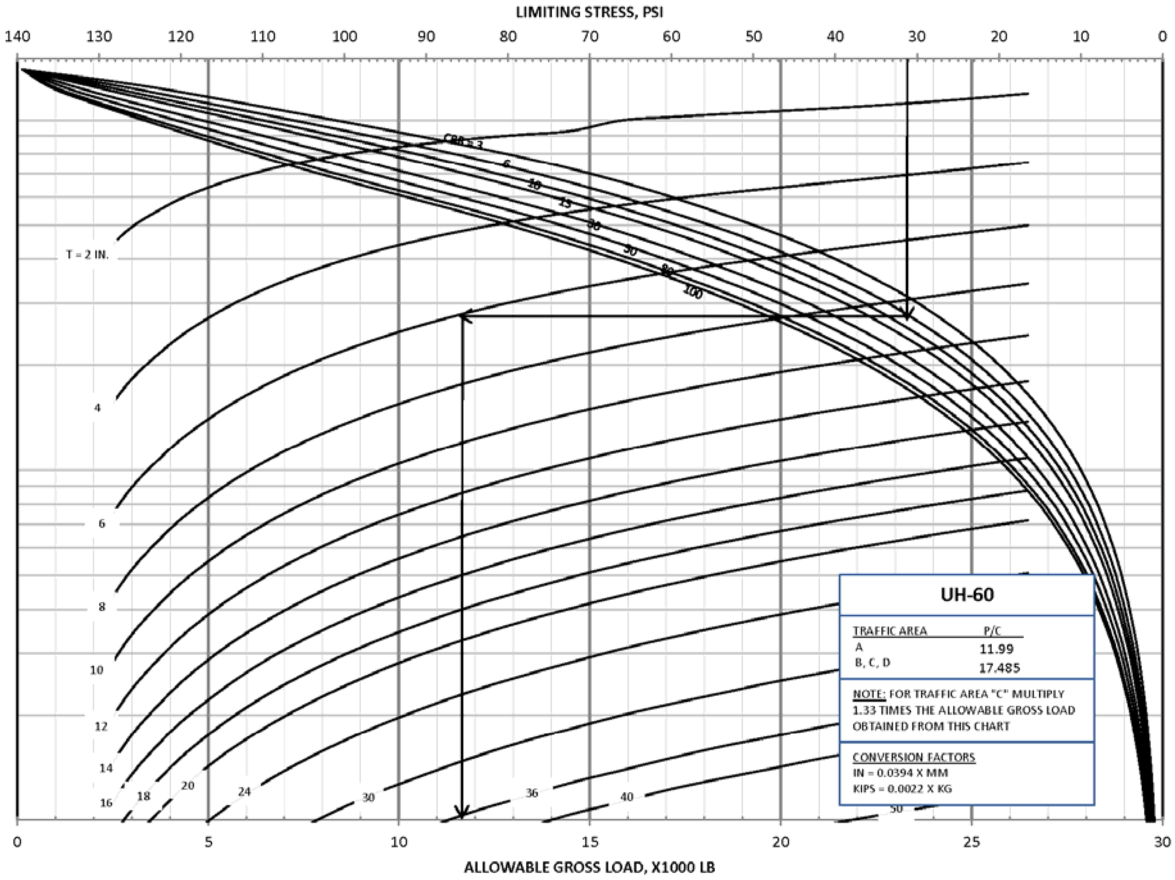
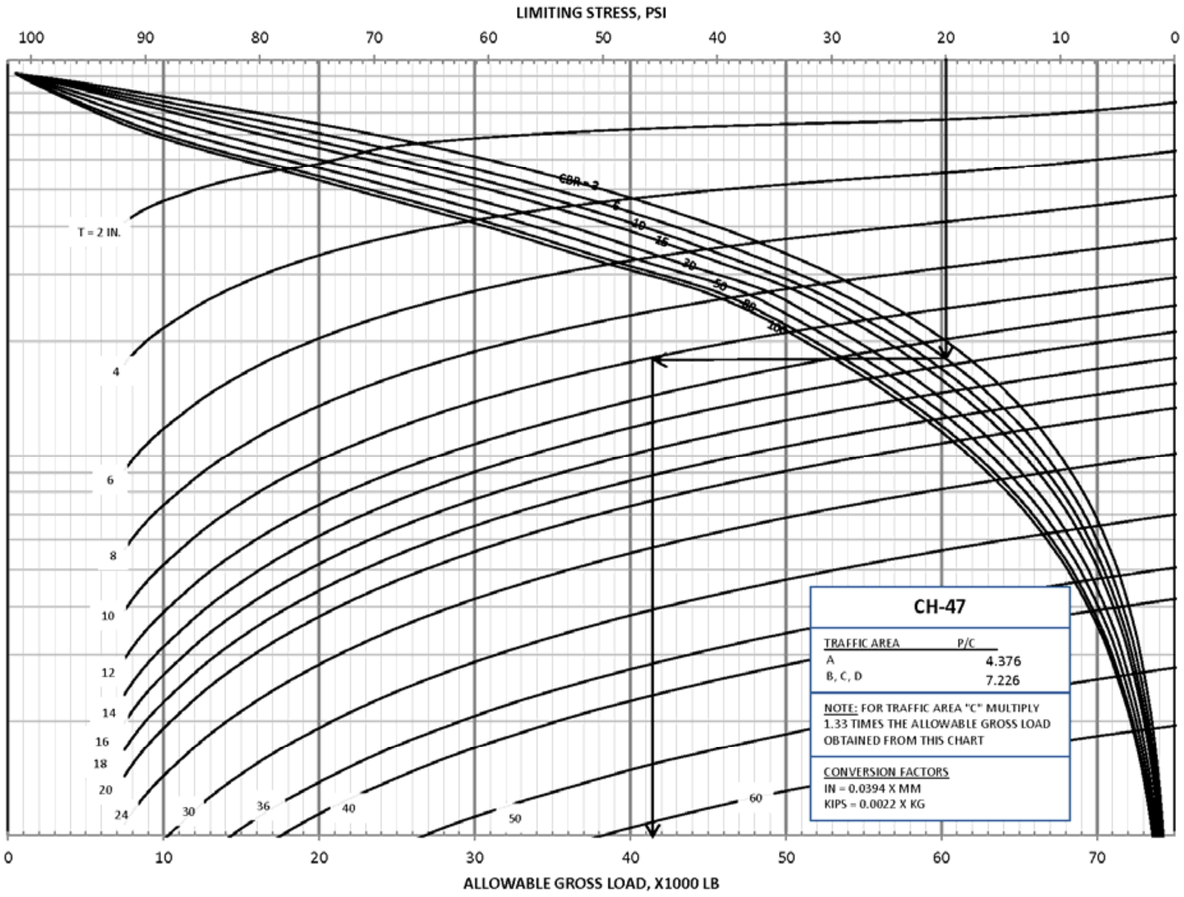


Figure 5-15. Flexible evaluation curve for UH-60 (AGL)



CH-47

TRAFFIC AREA	P/C
A	4.376
B, C, D	7.226

NOTE: FOR TRAFFIC AREA "C" MULTIPLY 1.33 TIMES THE ALLOWABLE GROSS LOAD OBTAINED FROM THIS CHART

CONVERSION FACTORS
IN = 0.0394 X MM
KIPS = 0.0022 X KG

Figure 5-16. Flexible evaluation curve for CH-47 (AGL)

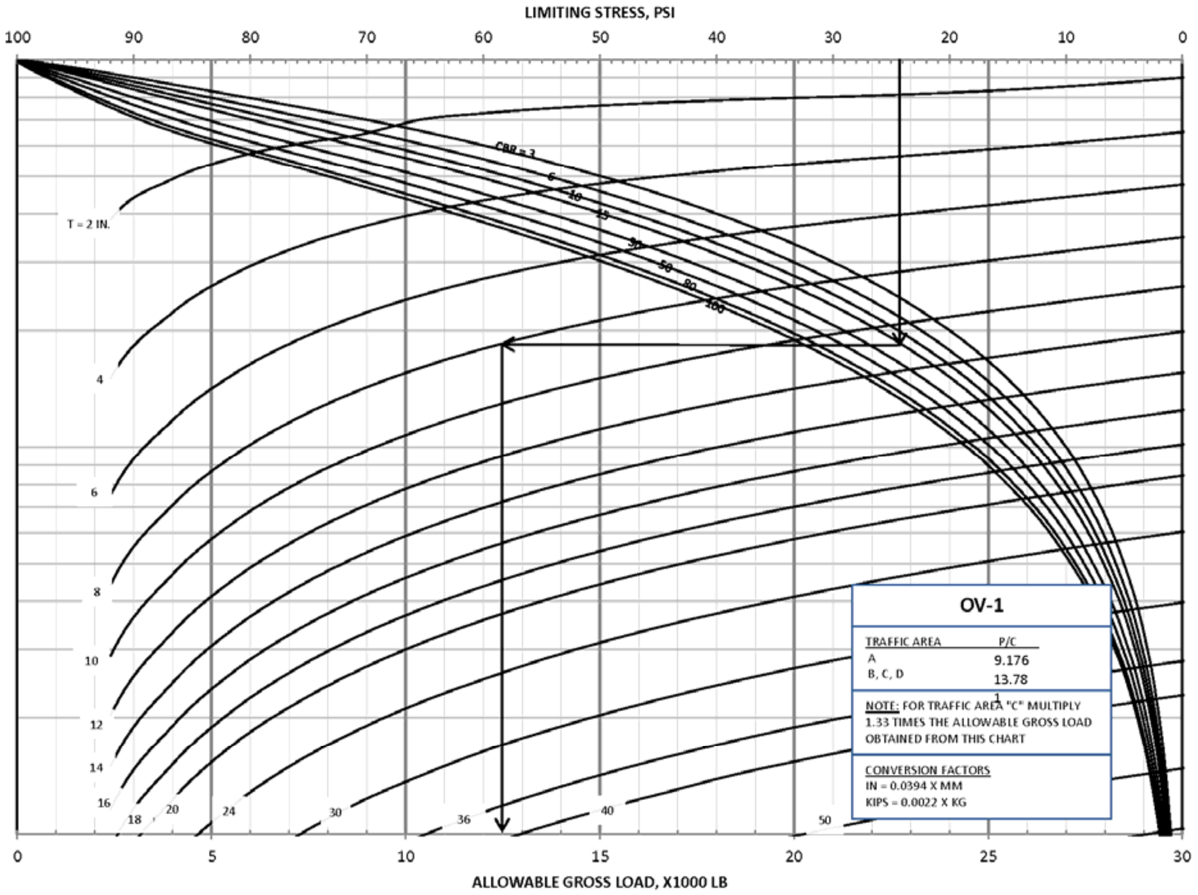
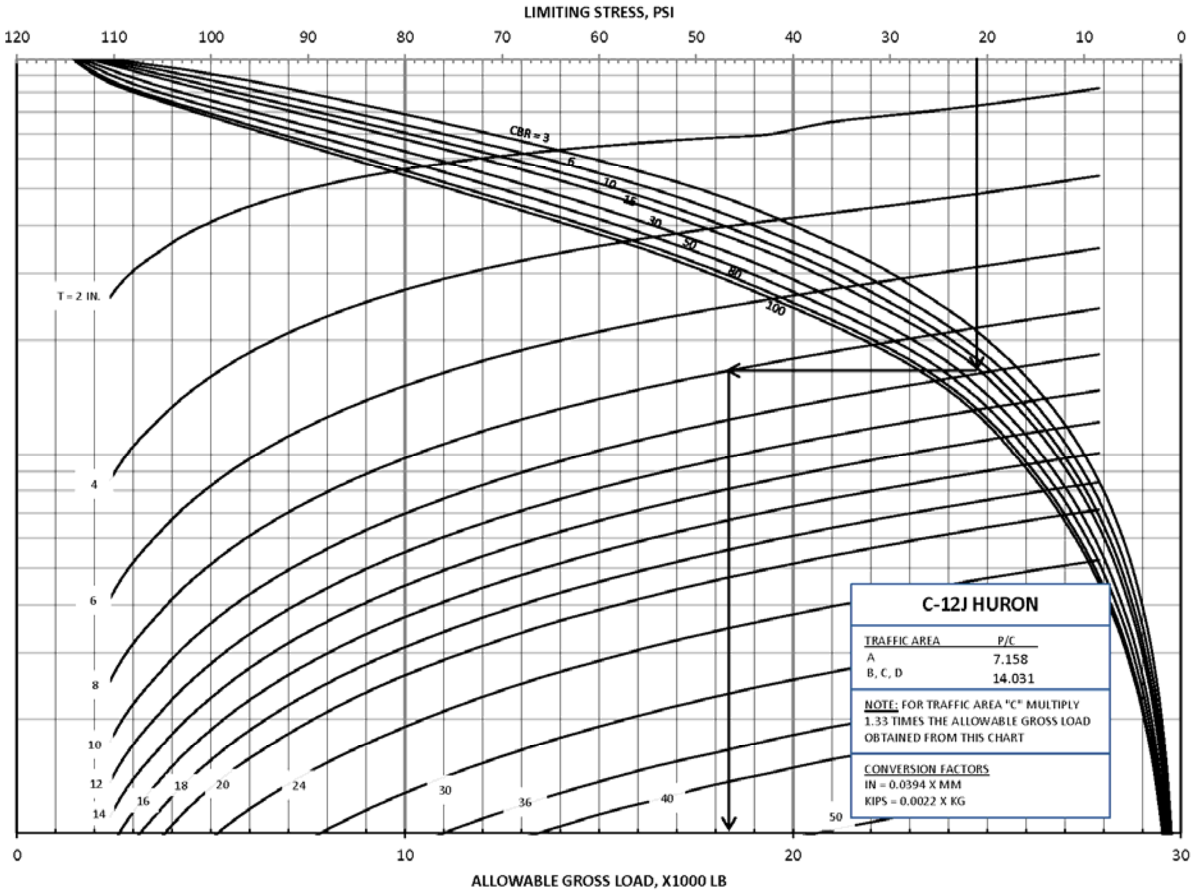


Figure 5-17. Flexible evaluation curve for OV-1 (AGL)



C-12J HURON

TRAFFIC AREA	P/C
A	7.158
B, C, D	14.031

NOTE: FOR TRAFFIC AREA "C" MULTIPLY 1.33 TIMES THE ALLOWABLE GROSS LOAD OBTAINED FROM THIS CHART

CONVERSION FACTORS
IN = 0.0394 X MM
KIPS = 0.0022 X KG

Figure 5-18. Flexible evaluation curve for C-12J (AGL)

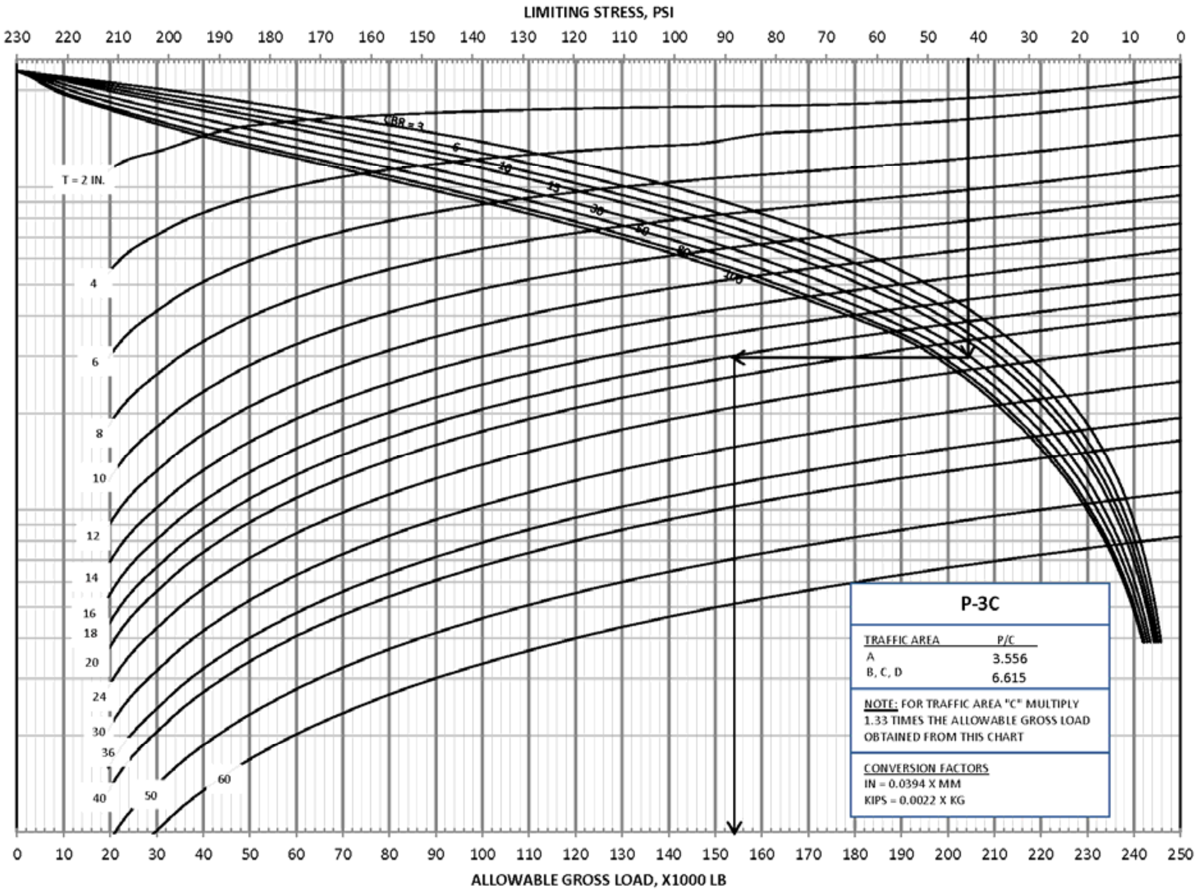


Figure 5-19. Flexible evaluation curve for P-3C (AGL)

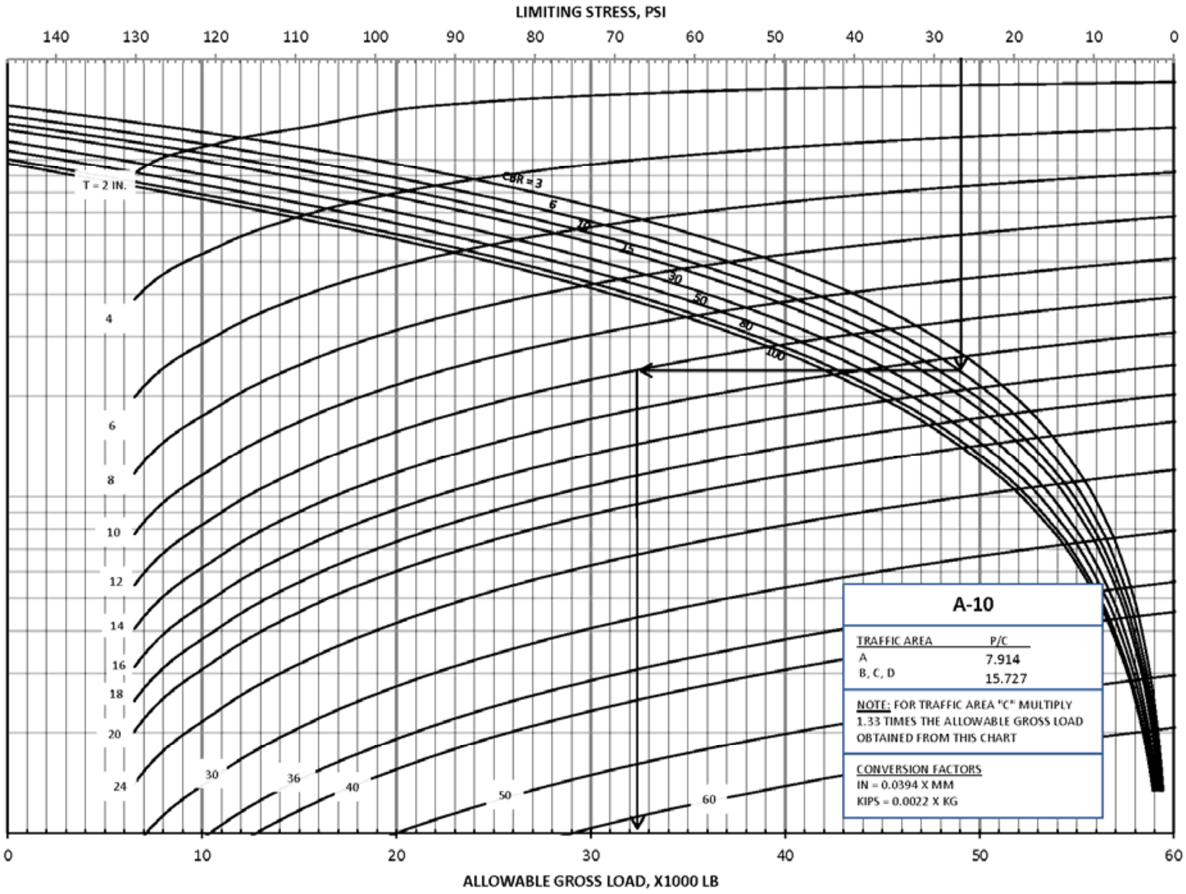


Figure 5-20. Flexible evaluation curve for A-10 (AGL)

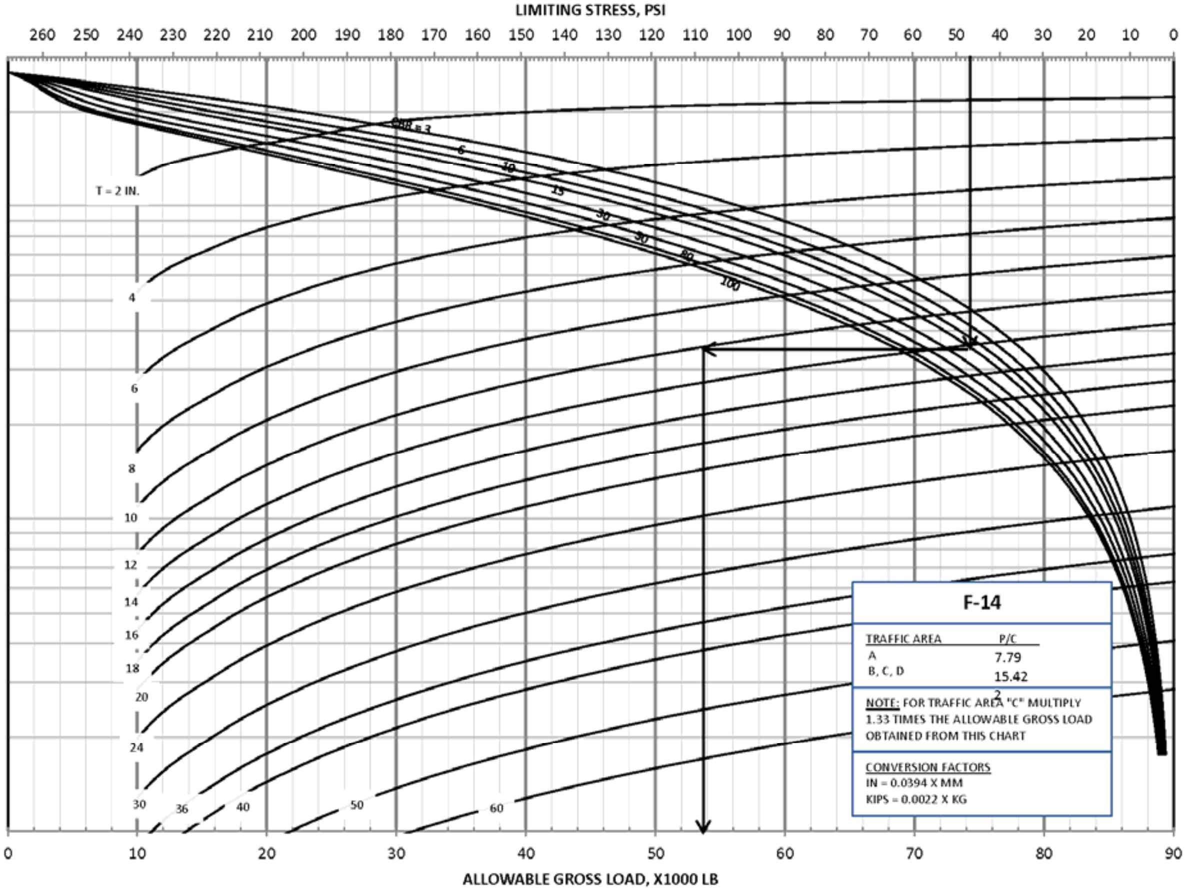


Figure 5-21. Flexible evaluation curve for F-14 (AGL)

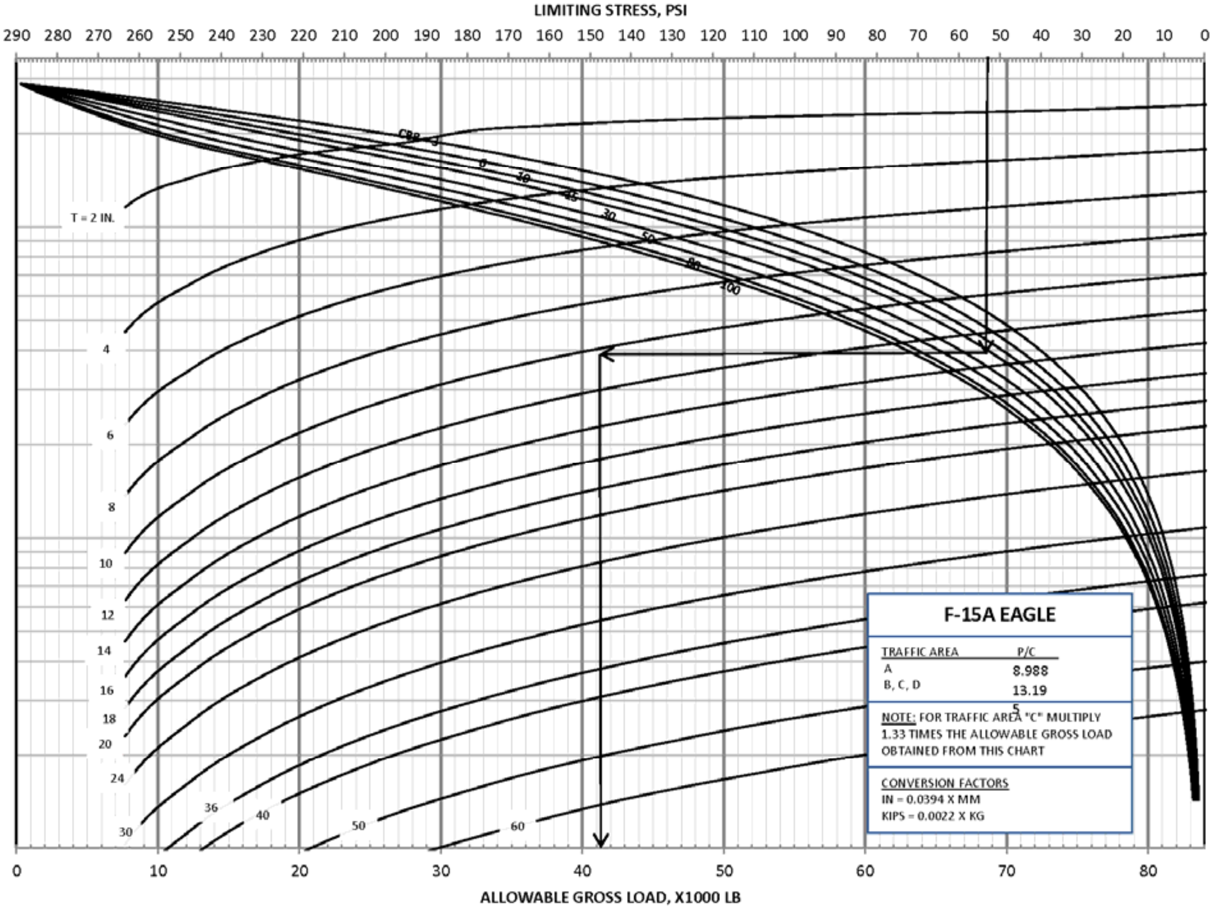


Figure 5-22. Flexible evaluation curve for F-15 C/D (AGL)

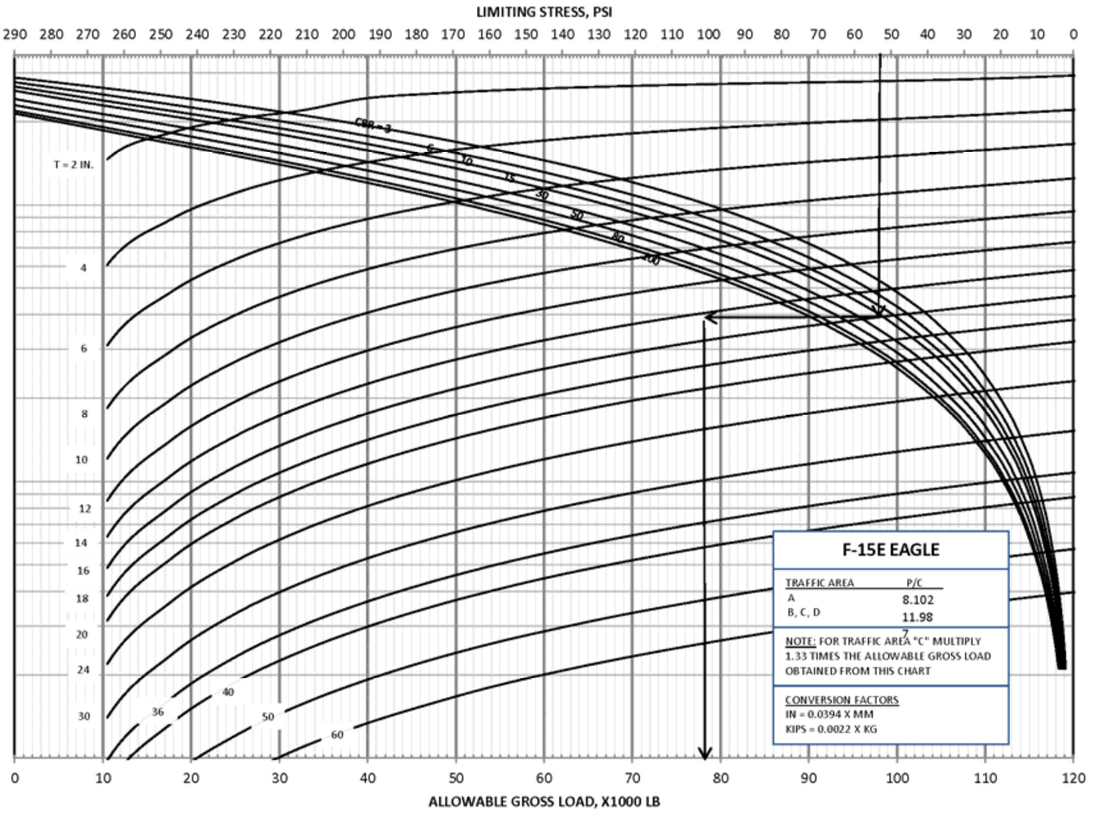


Figure 5-23. Flexible evaluation curve for F-15 E (AGL)

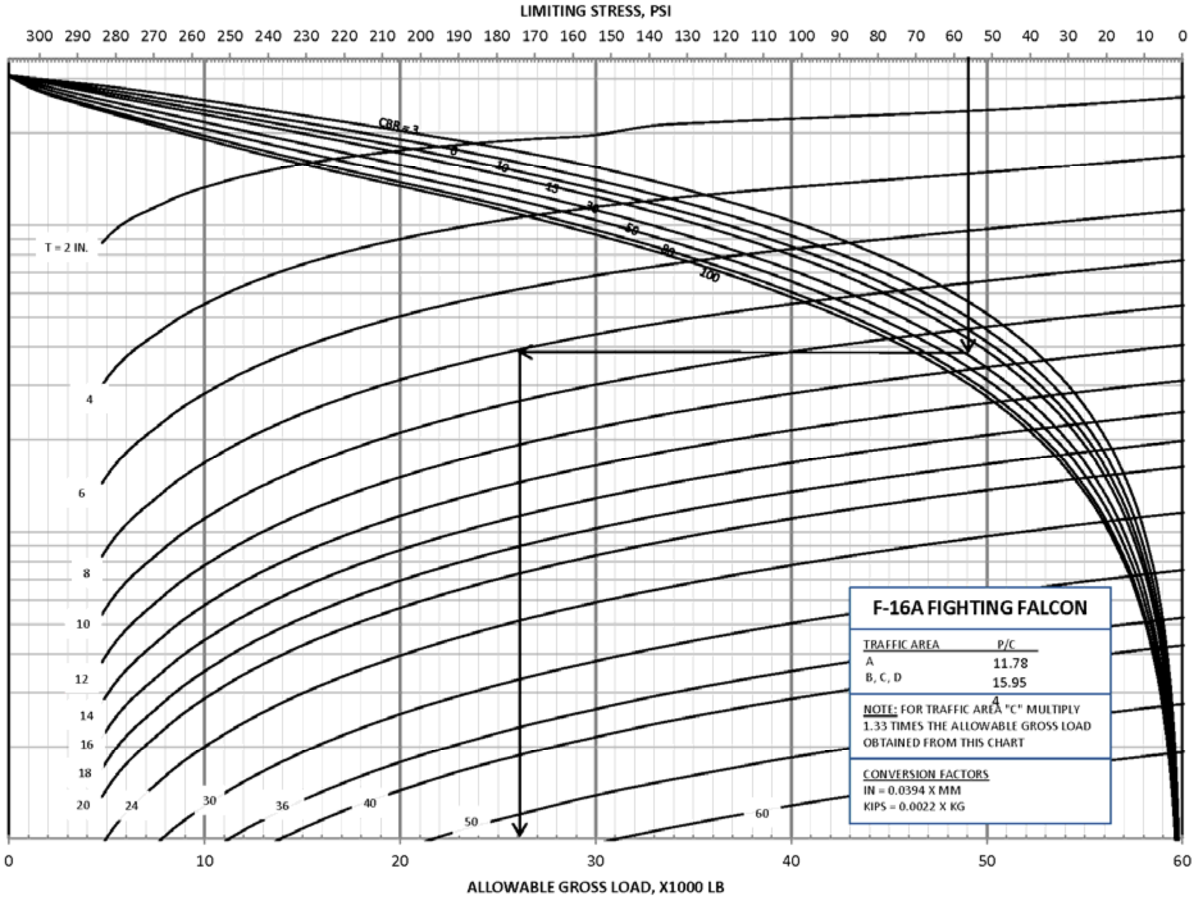


Figure 5-24. Flexible evaluation curve for F-16 (AGL)

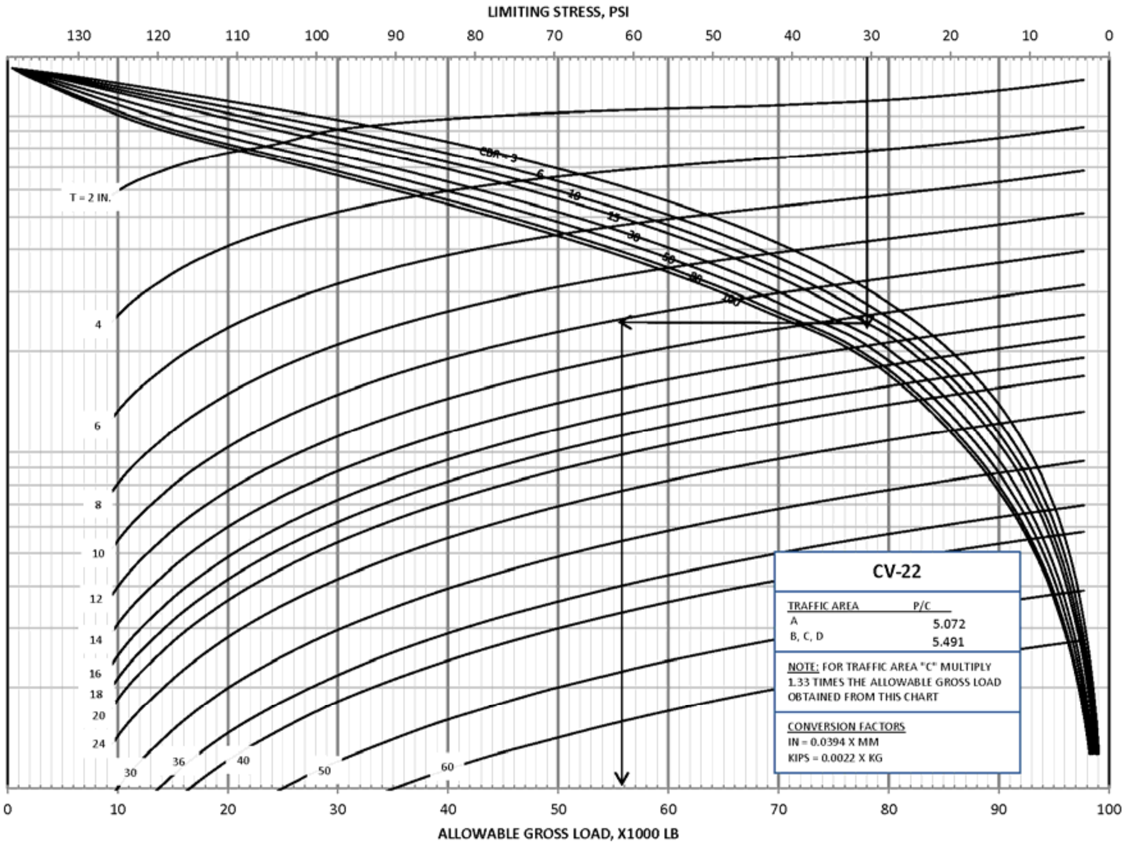


Figure 5-25. Flexible evaluation curve for CV-22 (AGL)

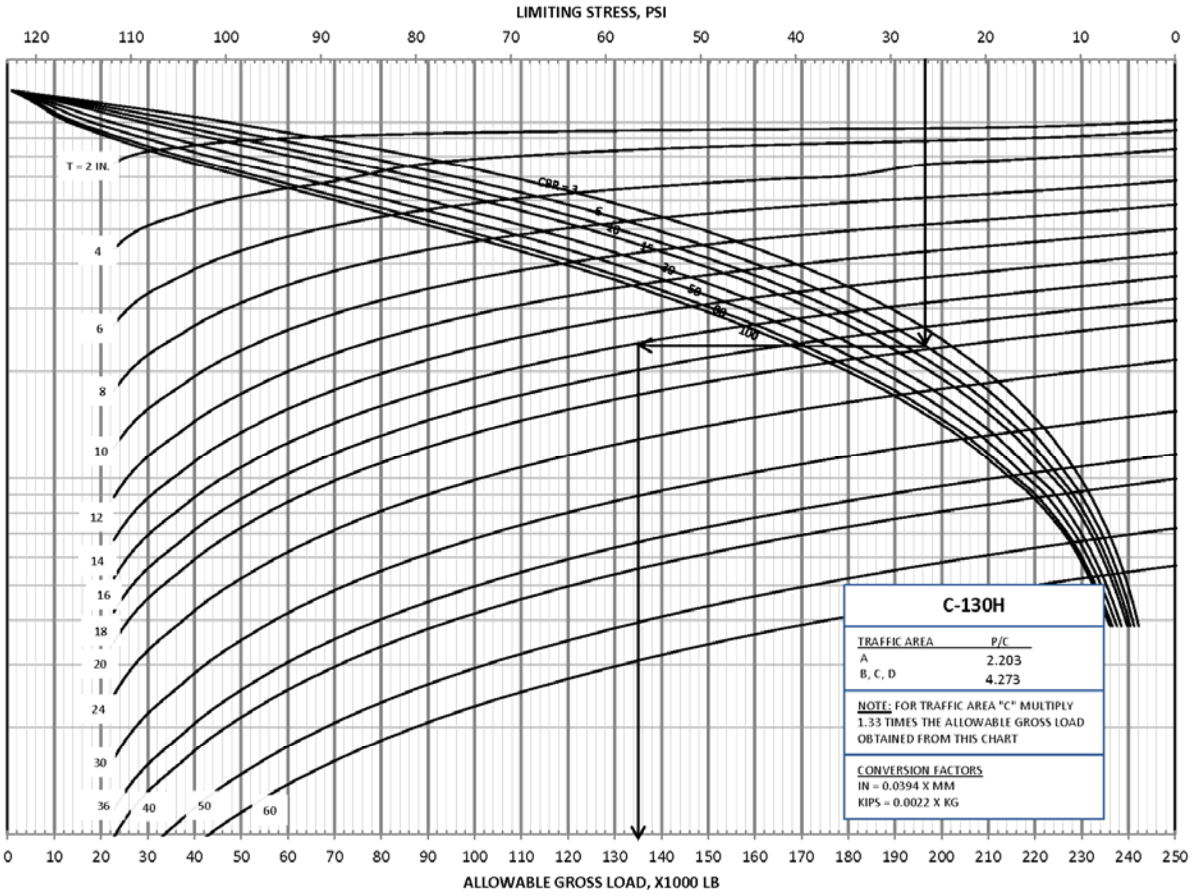


Figure 5-26. Flexible evaluation curve for C-130H (AGL)

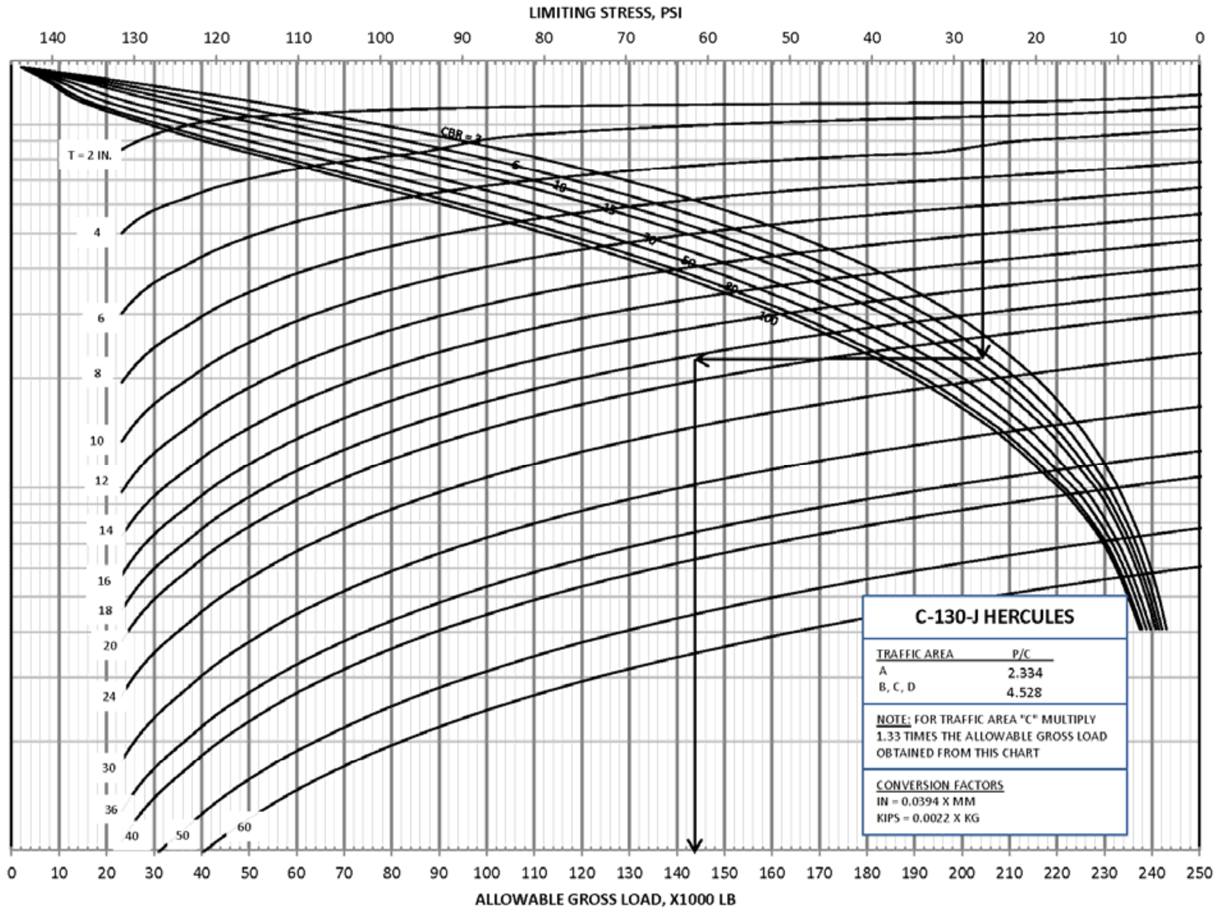


Figure 5-27. Flexible evaluation curve for C-130J (AGL)

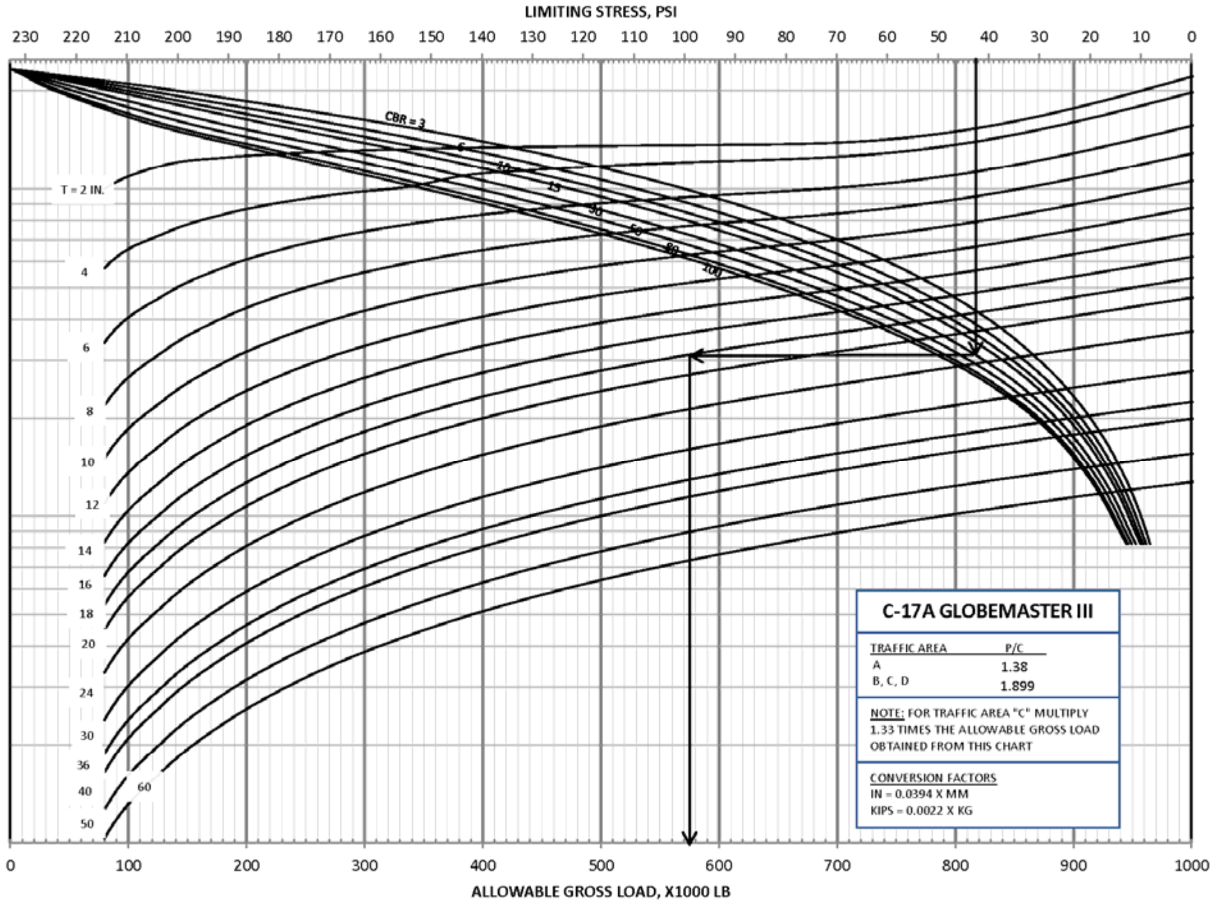


Figure 5-28. Flexible evaluation curve for C-17 (AGL)

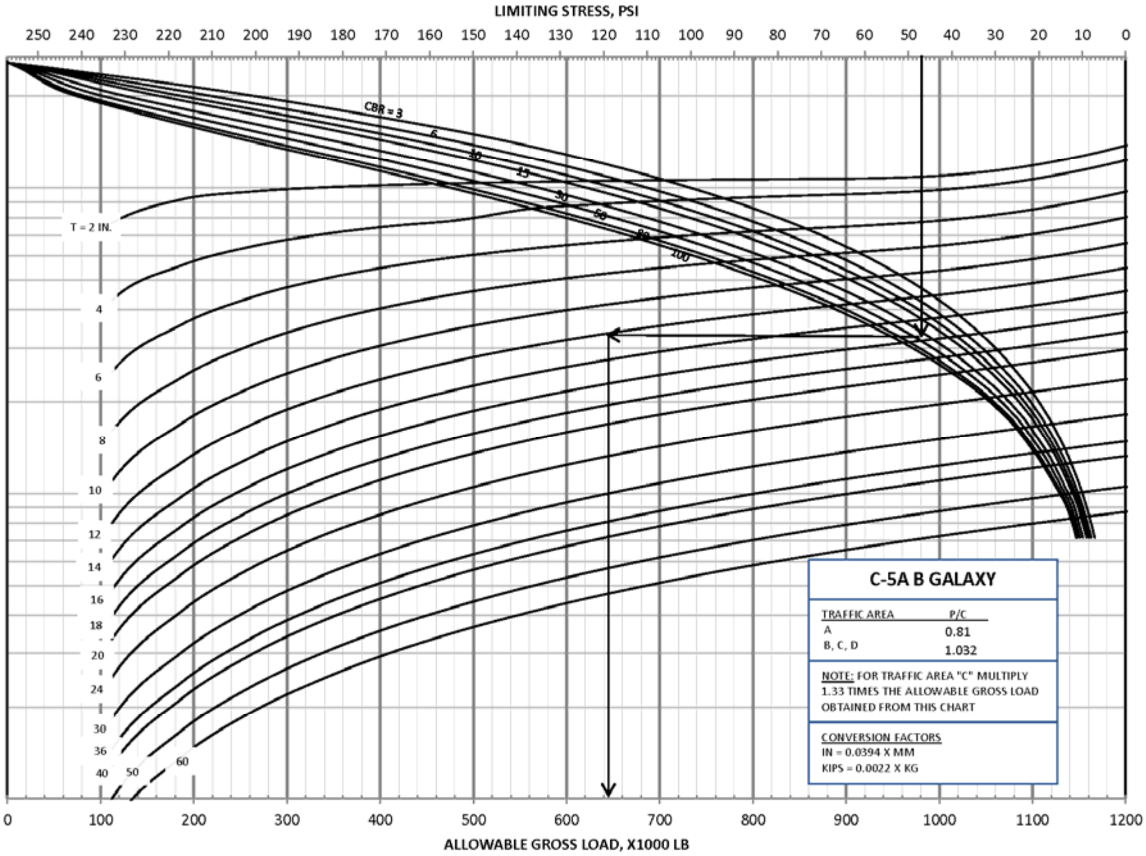
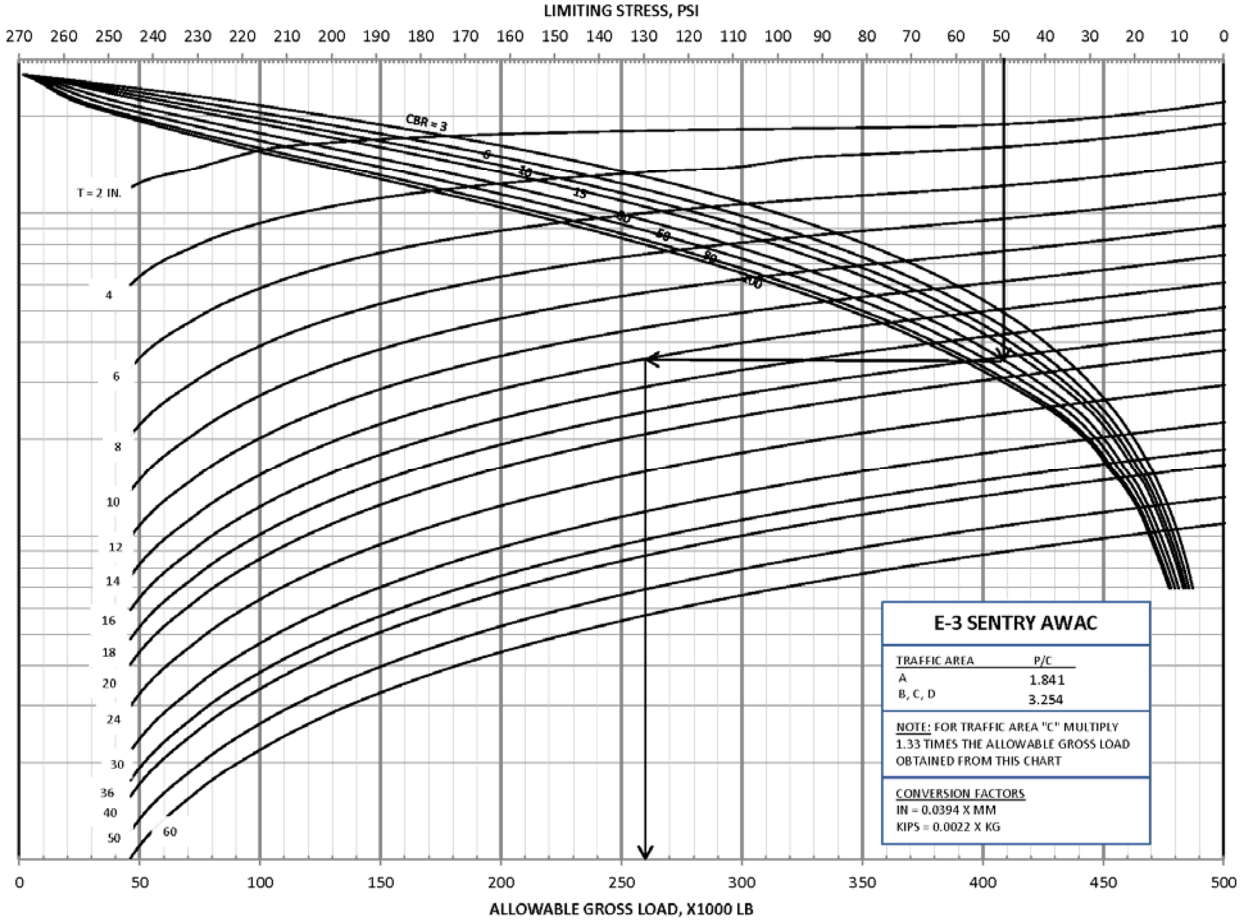


Figure 5-29. Flexible evaluation curve for C-5 (AGL)



E-3 SENTRY AWAC

TRAFFIC AREA	P/C
A	1.841
B, C, D	3.254

NOTE: FOR TRAFFIC AREA "C" MULTIPLY 1.33 TIMES THE ALLOWABLE GROSS LOAD OBTAINED FROM THIS CHART

CONVERSION FACTORS
IN = 0.0394 X MM
KIPS = 0.0022 X KG

Figure 5-30. Flexible evaluation curve for E-3 (AGL)

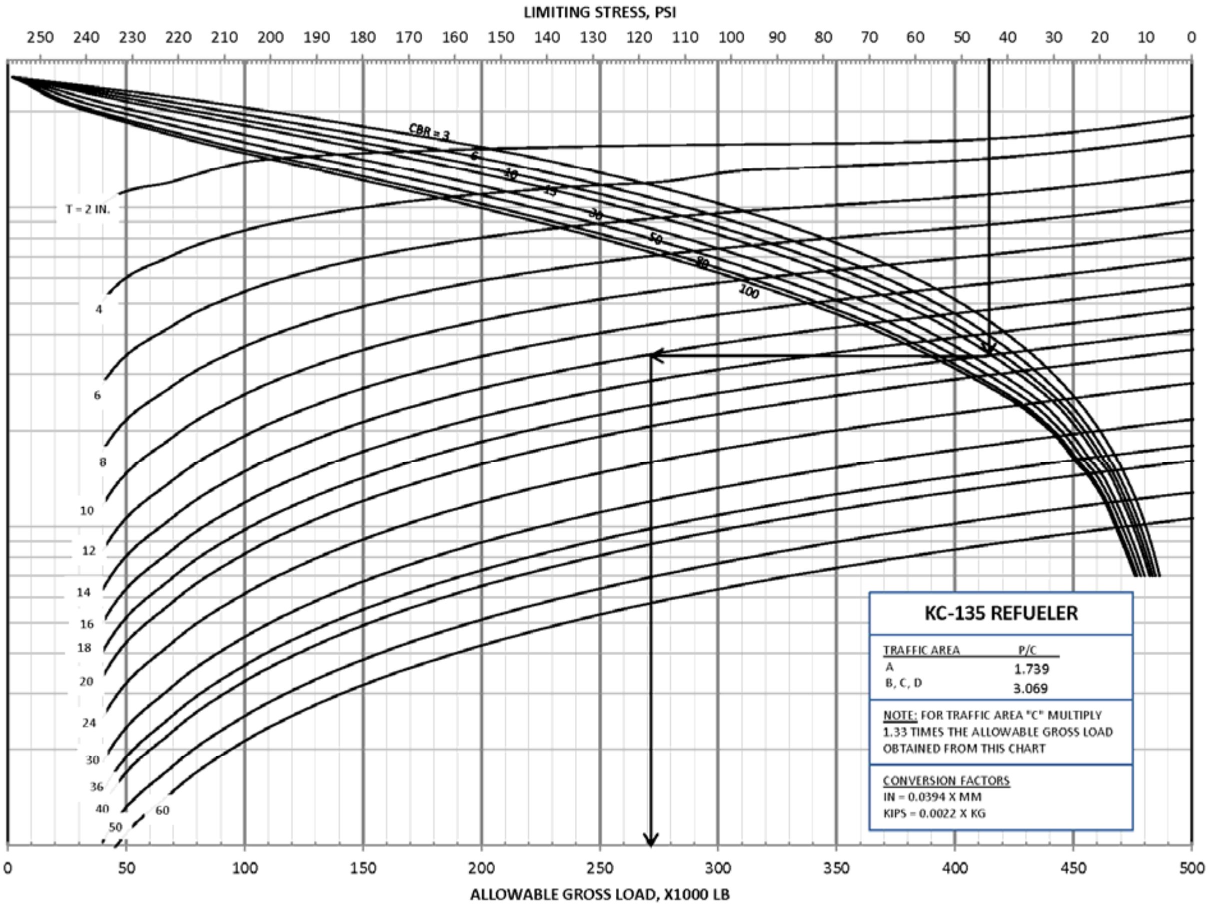


Figure 5-31. Flexible evaluation curve for KC-135 (AGL)

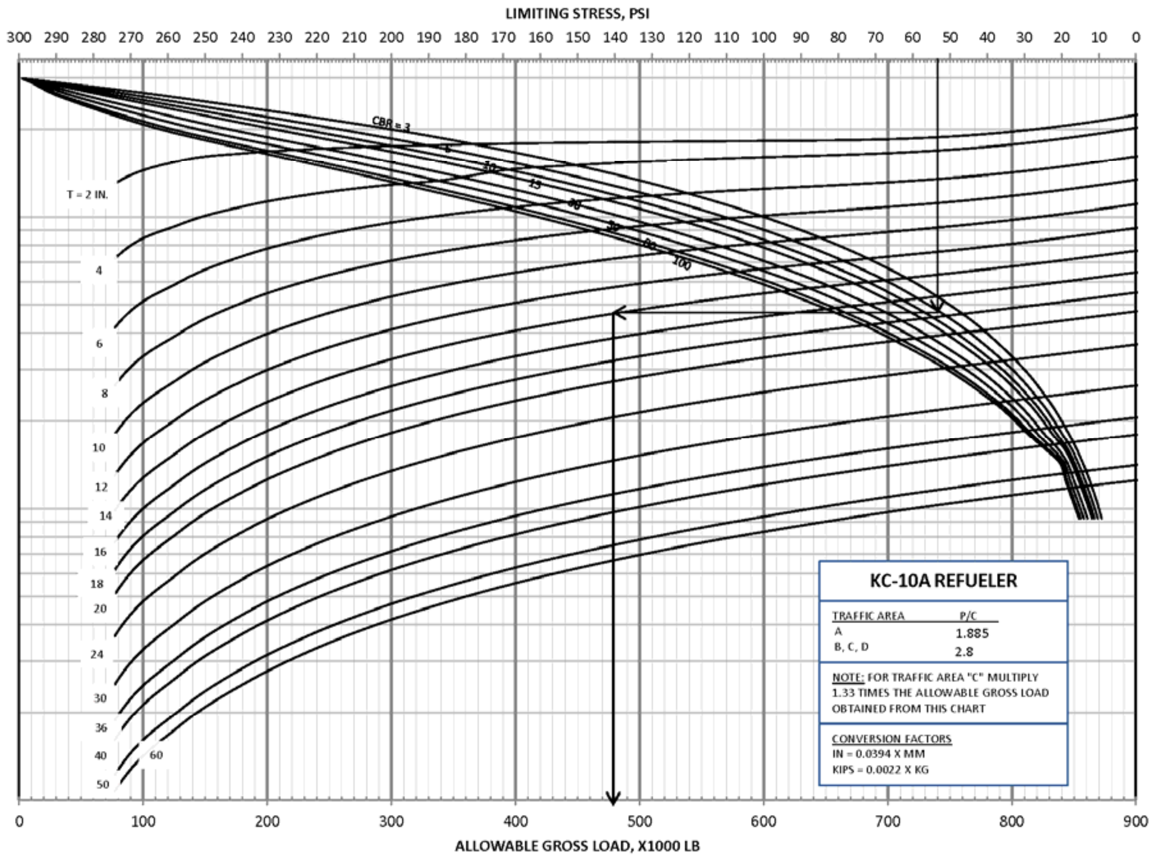


Figure 5-32. Flexible evaluation curve for KC-10 (AGL)

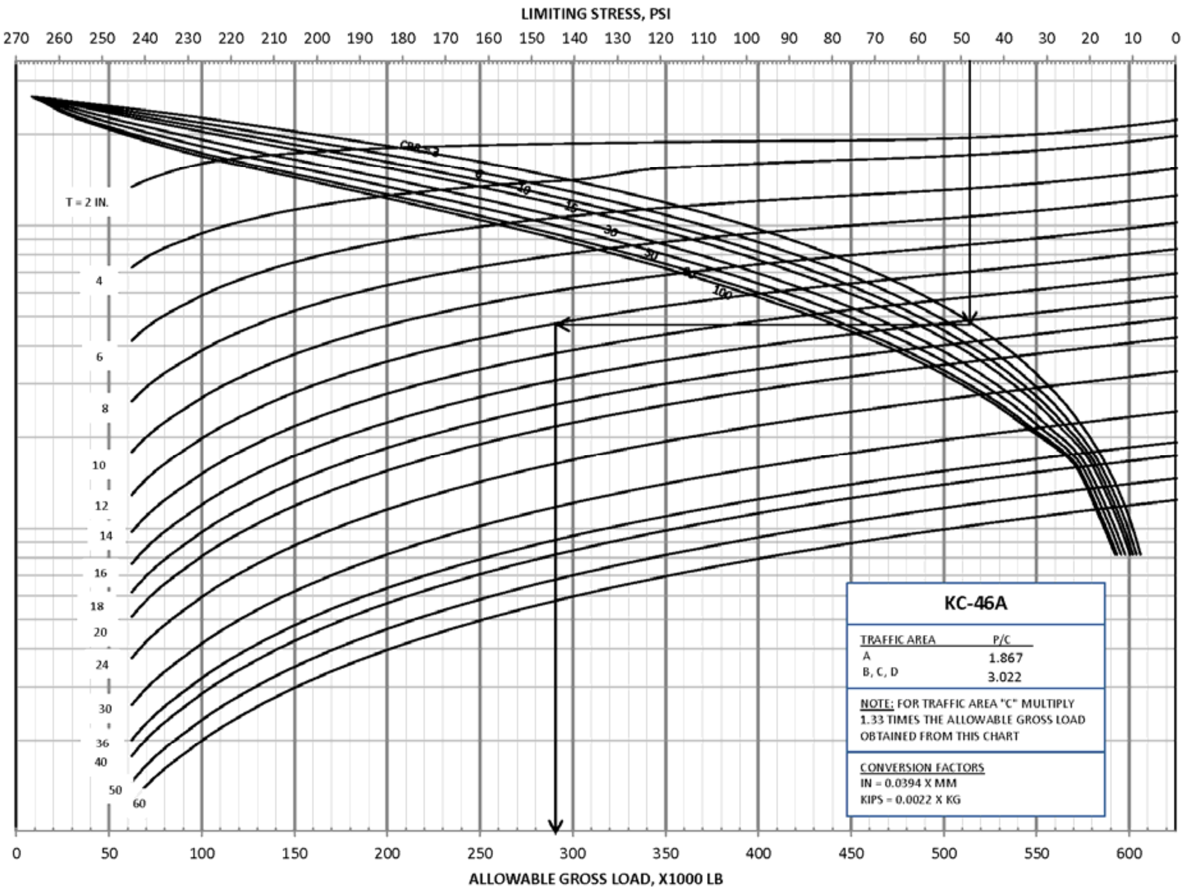


Figure 5-33. Flexible evaluation curve for KC-46A (AGL)

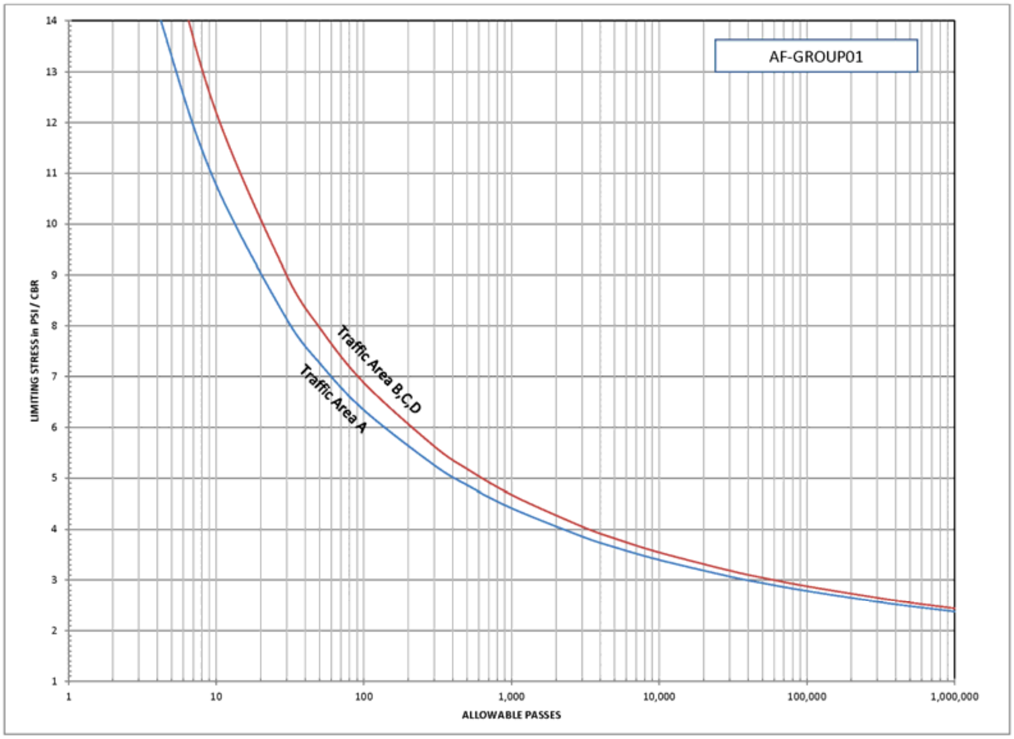


Figure 5-34—Flexible Evaluation Curve for Air Force Group 1 (Passes)

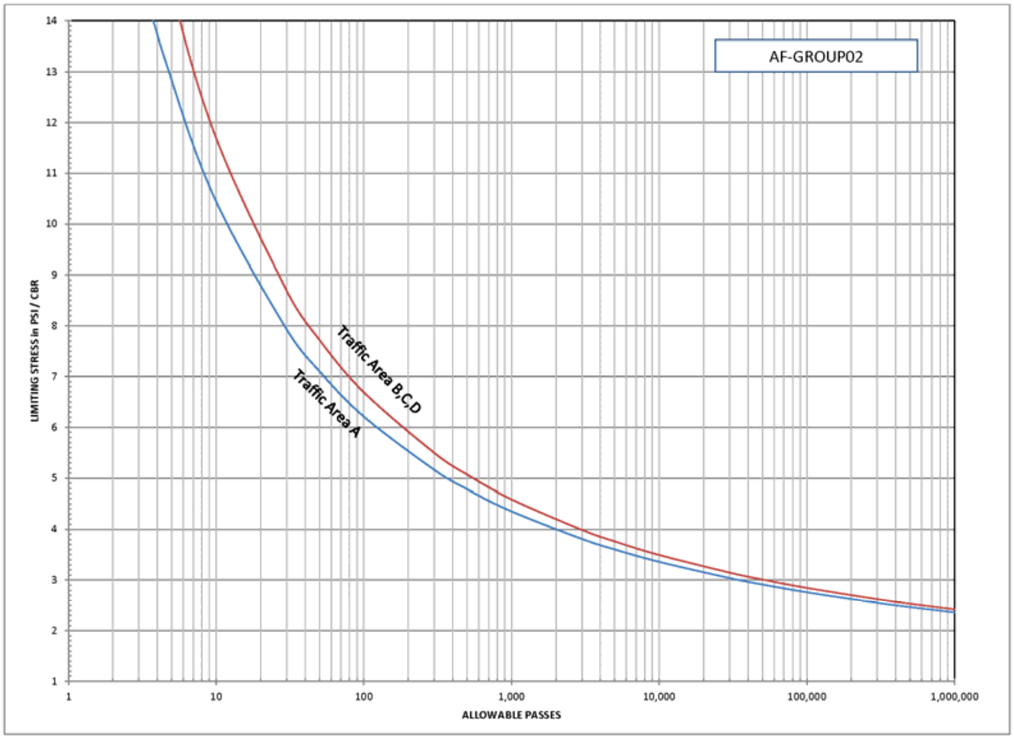


Figure 5-35—Flexible Evaluation Curve for Air Force Group 2 (Passes)

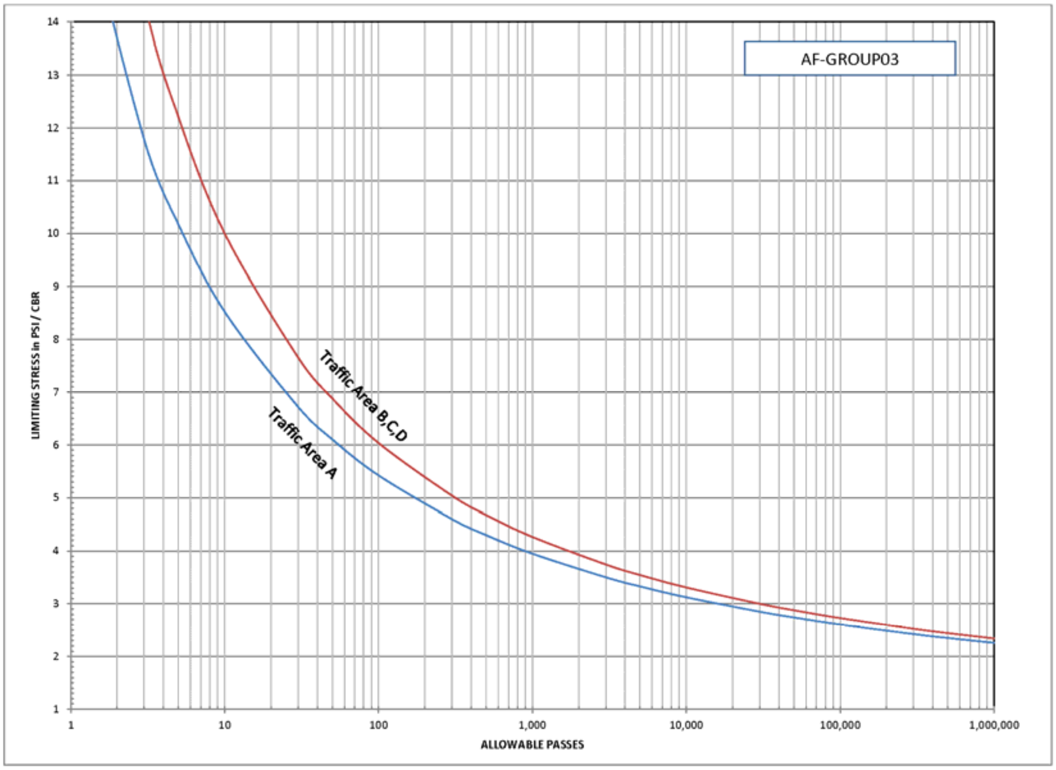


Figure 5-36—Flexible Evaluation Curve for Air Force Group 3 (Passes)

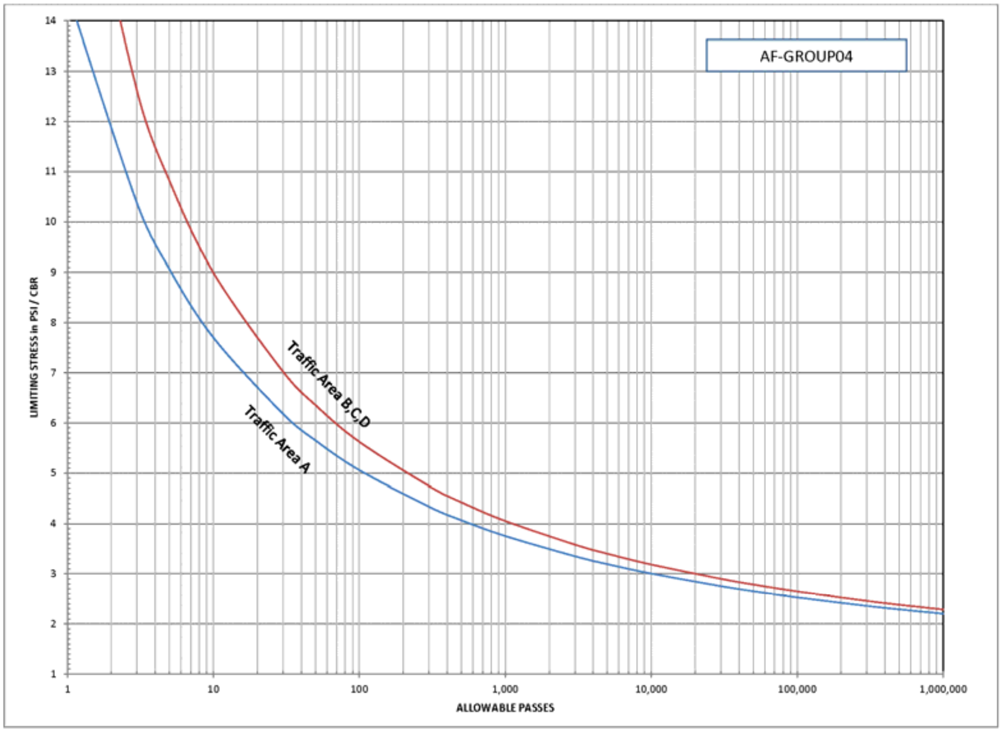


Figure 5-37—Flexible Evaluation Curve for Air Force Group 4 (Passes)

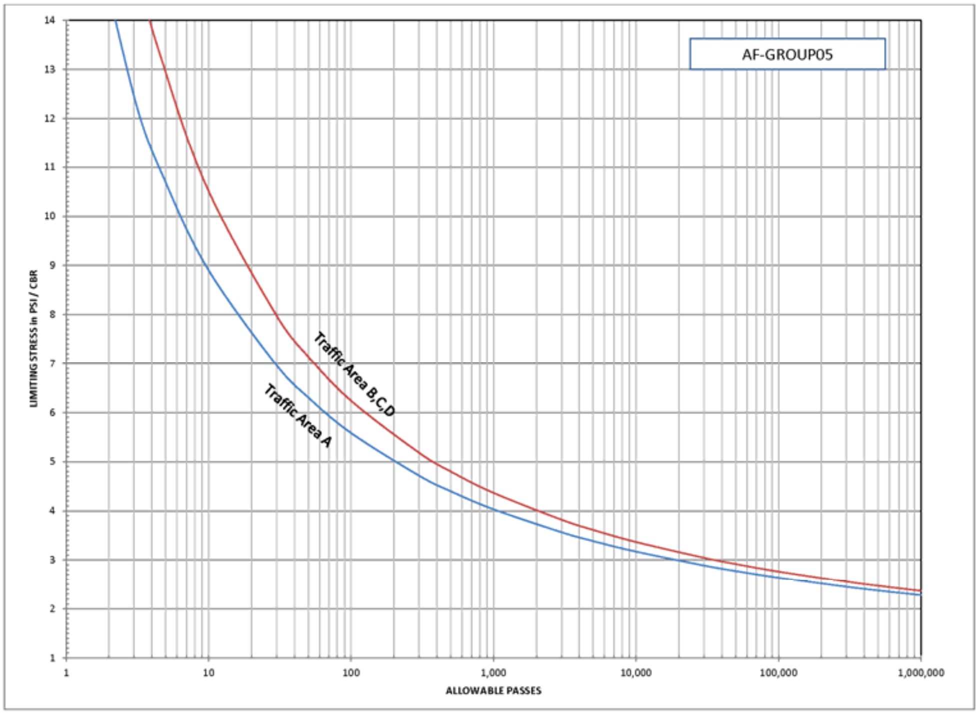


Figure 5-38—Flexible Evaluation Curve for Air Force Group 5 (Passes)

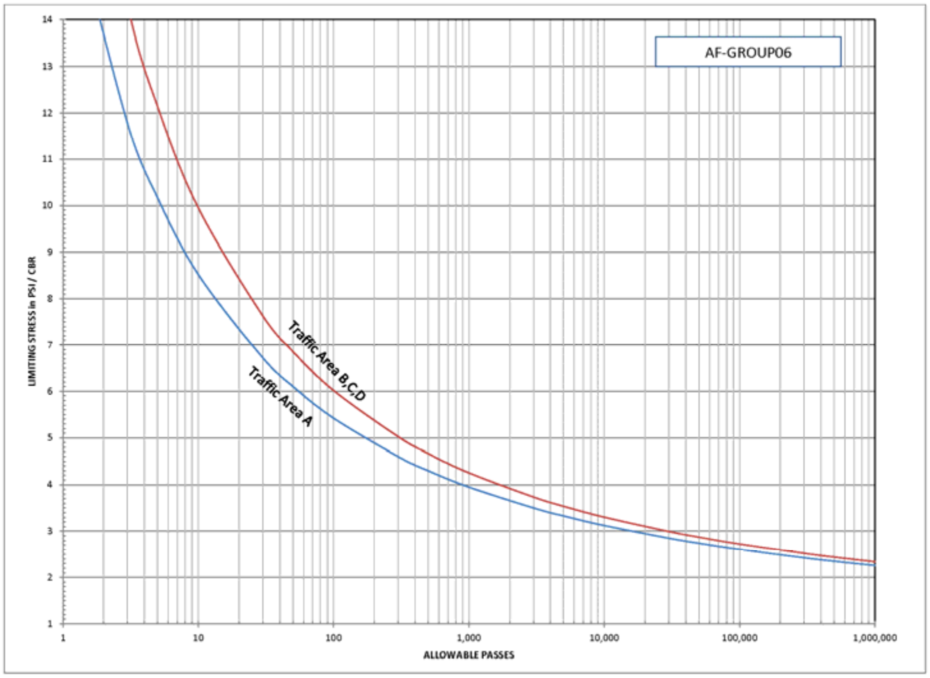


Figure 5-39—Flexible Evaluation Curve for Air Force Group 6 (Passes)

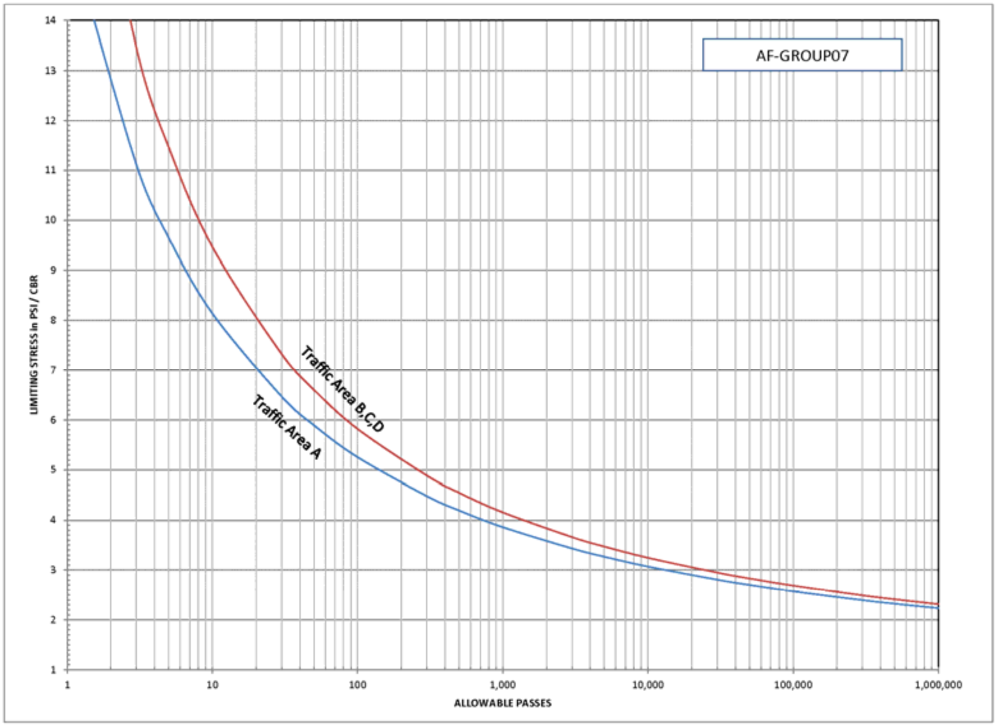


Figure 5-40—Flexible Evaluation Curve for Air Force Group 7 (Passes)

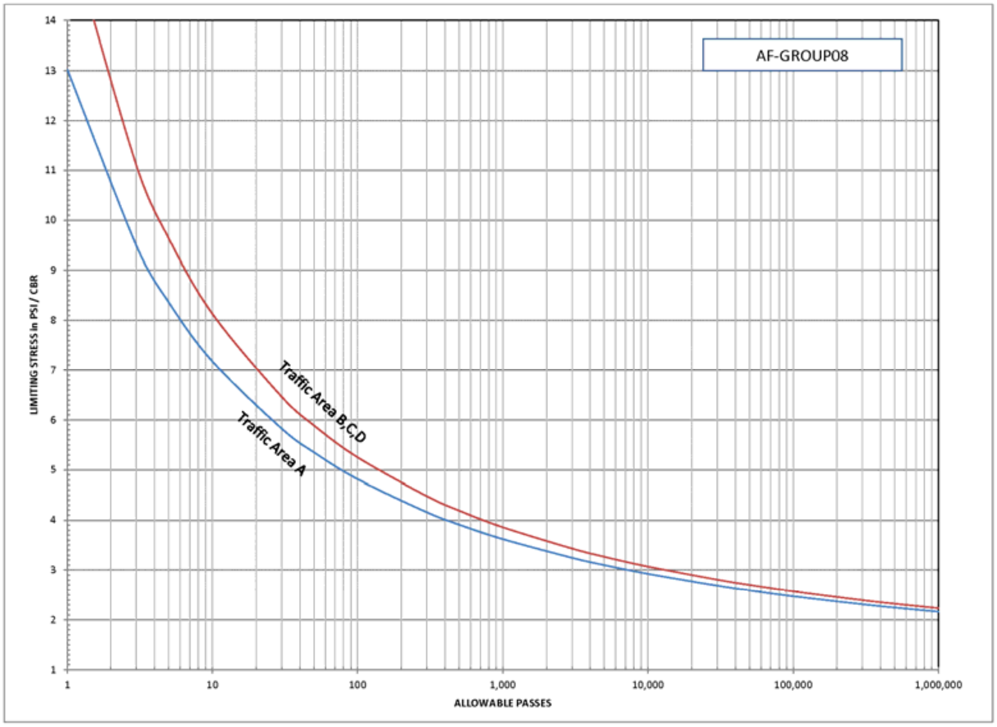


Figure 5-41—Flexible Evaluation Curve for Air Force Group 8 (Passes)

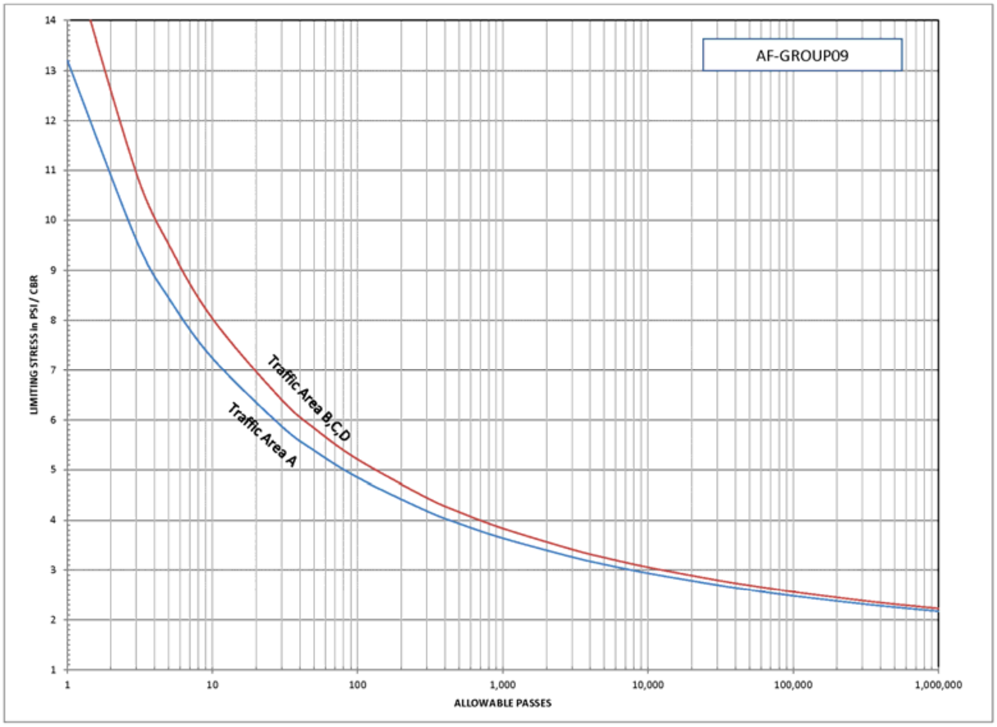


Figure 5-42—Flexible Evaluation Curve for Air Force Group 9 (Passes)

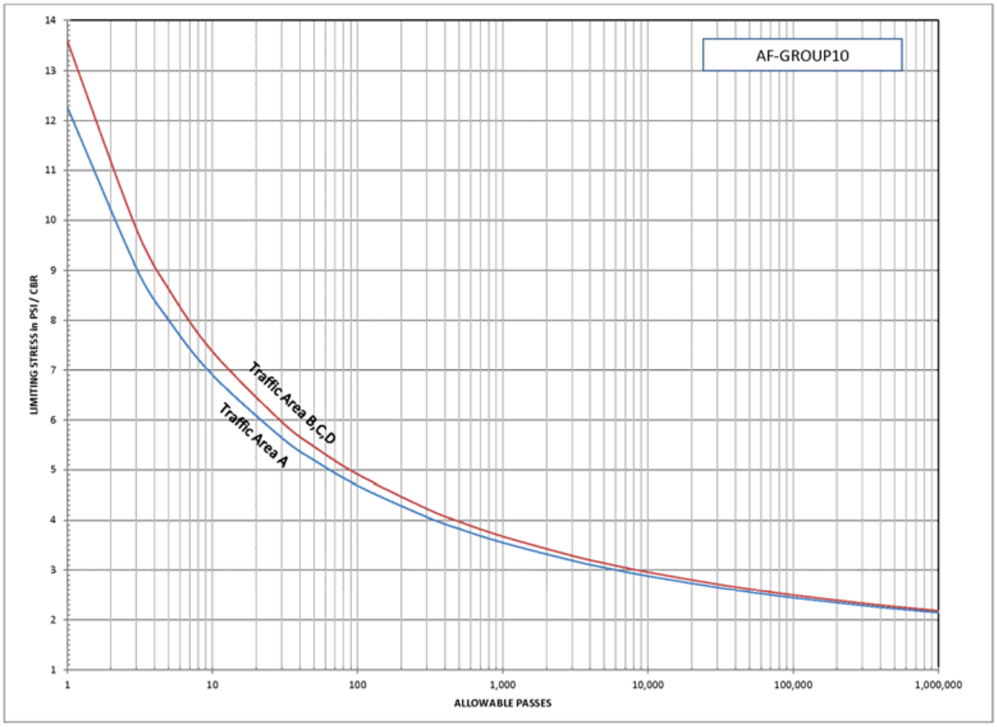


Figure 5-43—Flexible Evaluation Curve for Air Force Group 10 (Passes)

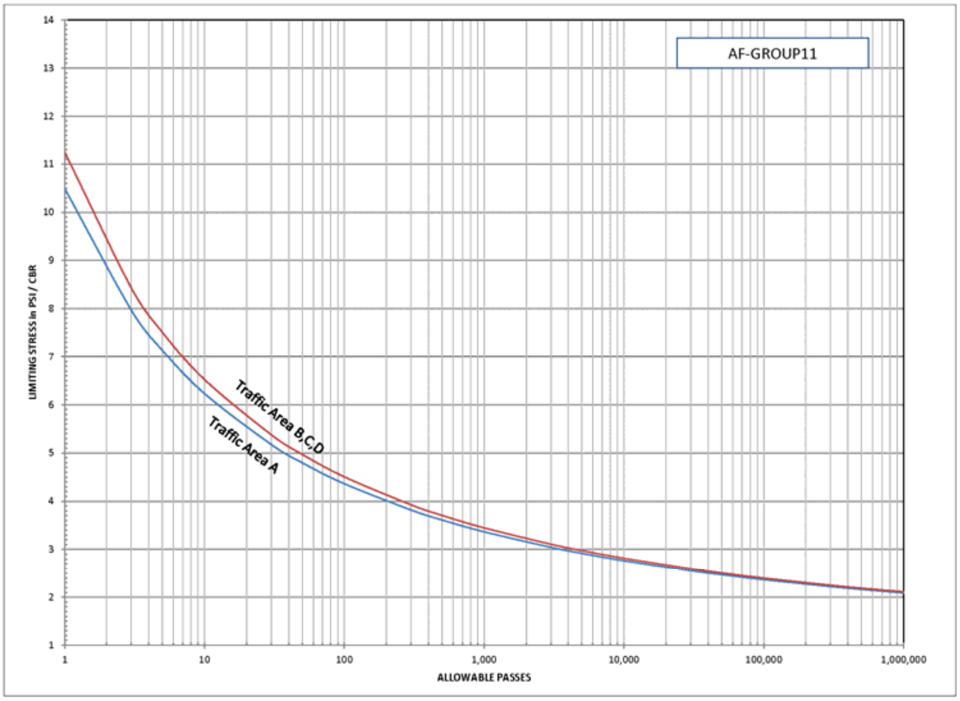


Figure 5-44—Flexible Evaluation Curve for Air Force Group 11 (Passes)

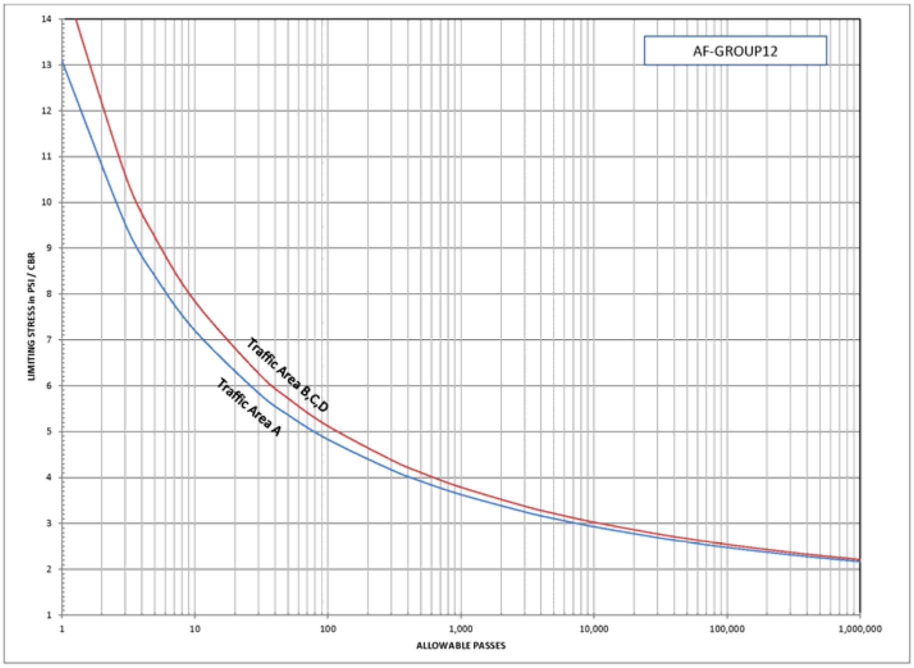


Figure 5-45—Flexible Evaluation Curve for Air Force Group 12 (Passes)

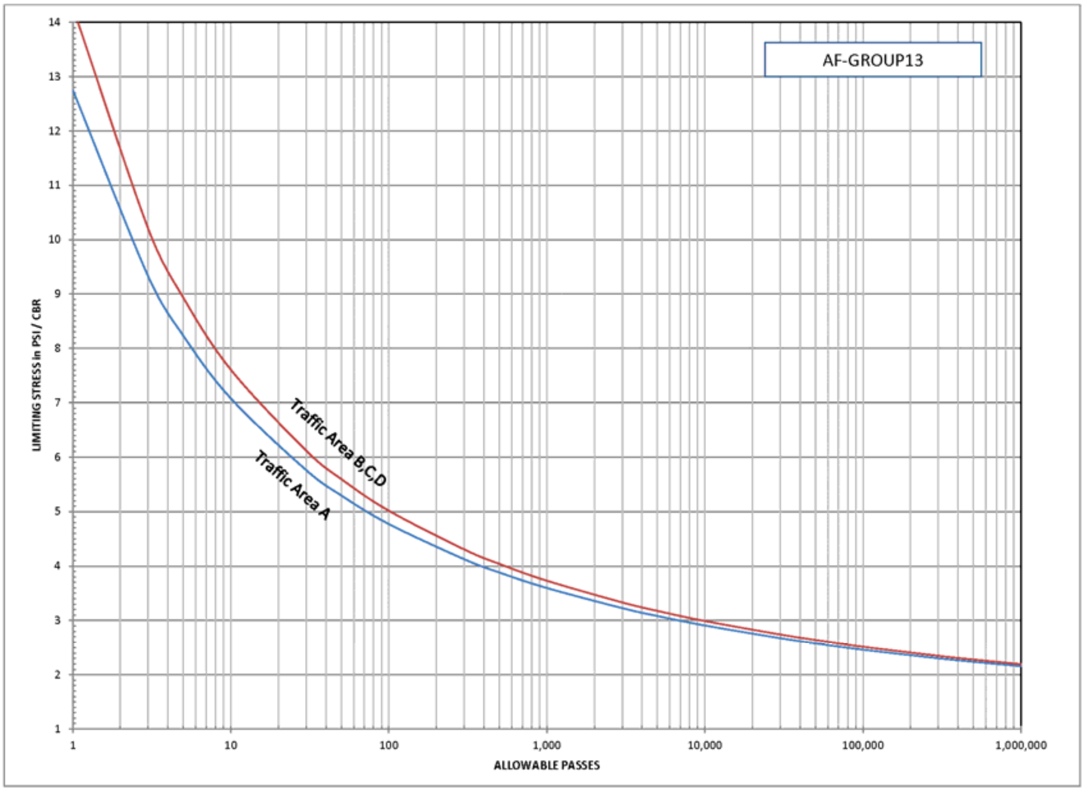


Figure 5-46—Flexible Evaluation Curve for Air Force Group 13 (Passes)

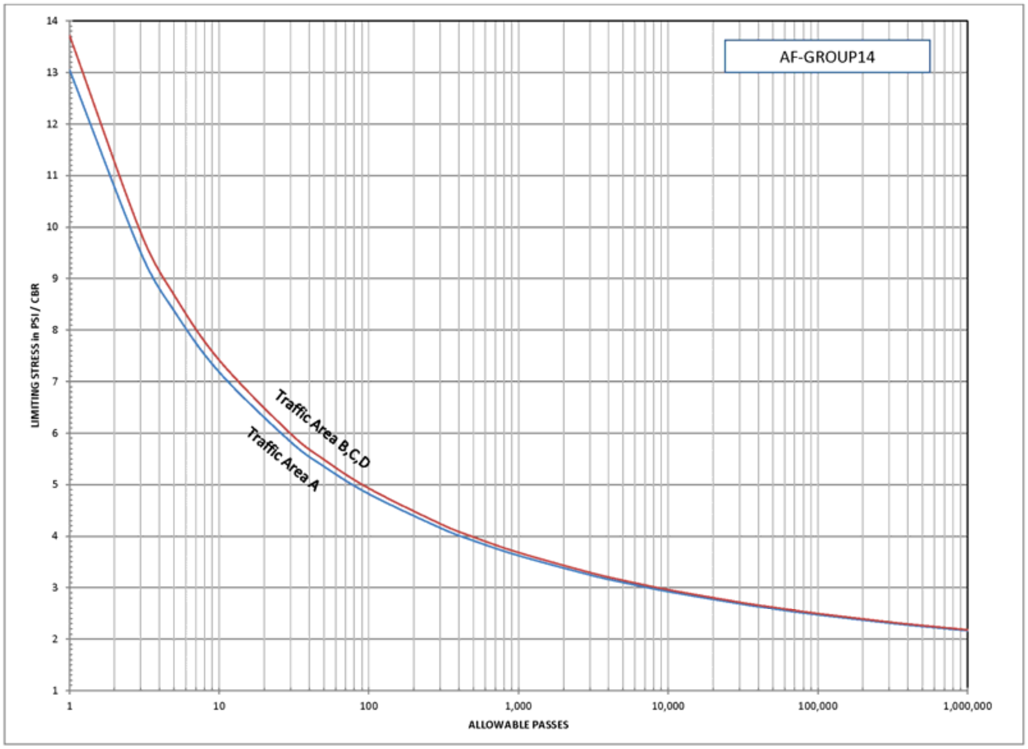


Figure 5-47—Flexible Evaluation Curve for Air Force Group 14 (Passes)

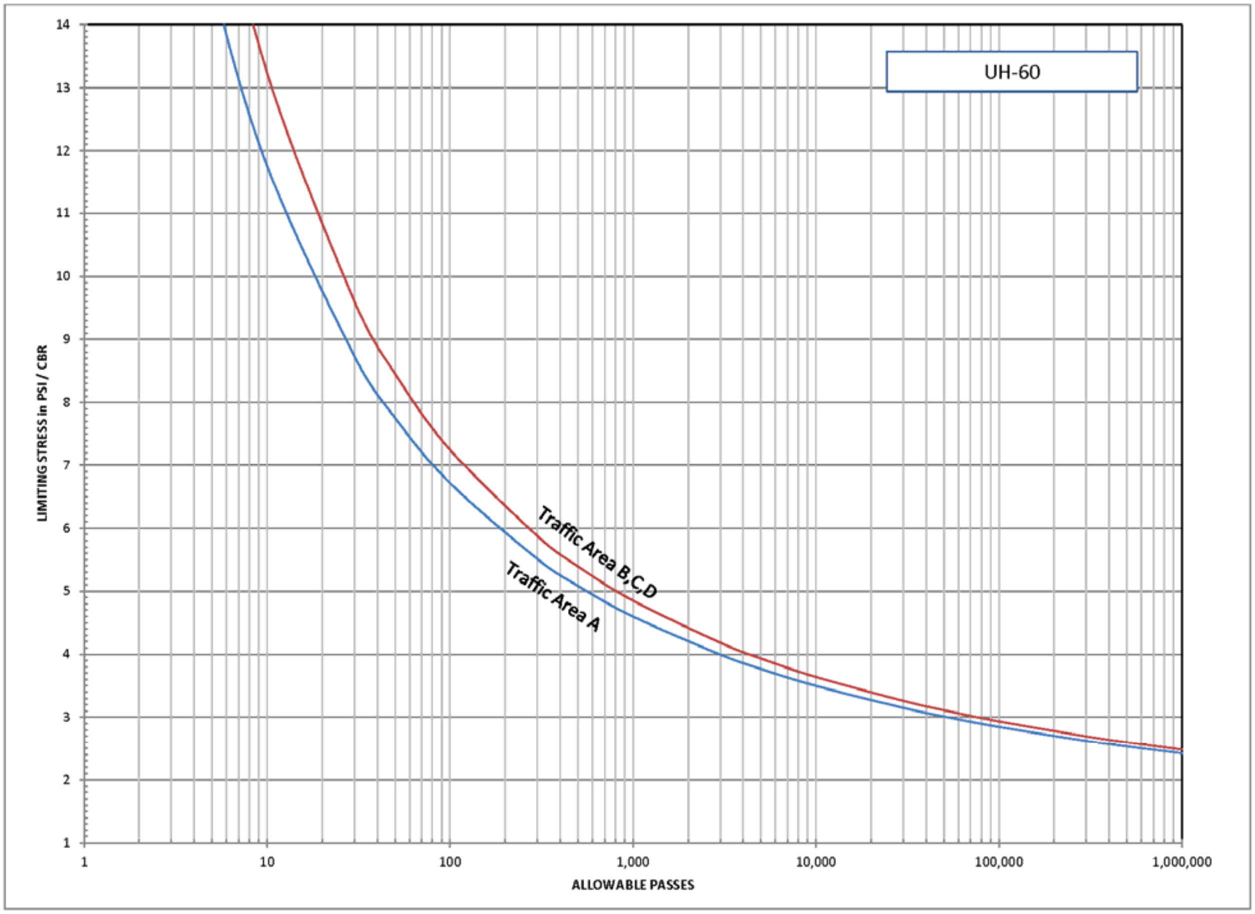


Figure 5-48—Flexible Evaluation Curve for UH-60 (Passes)

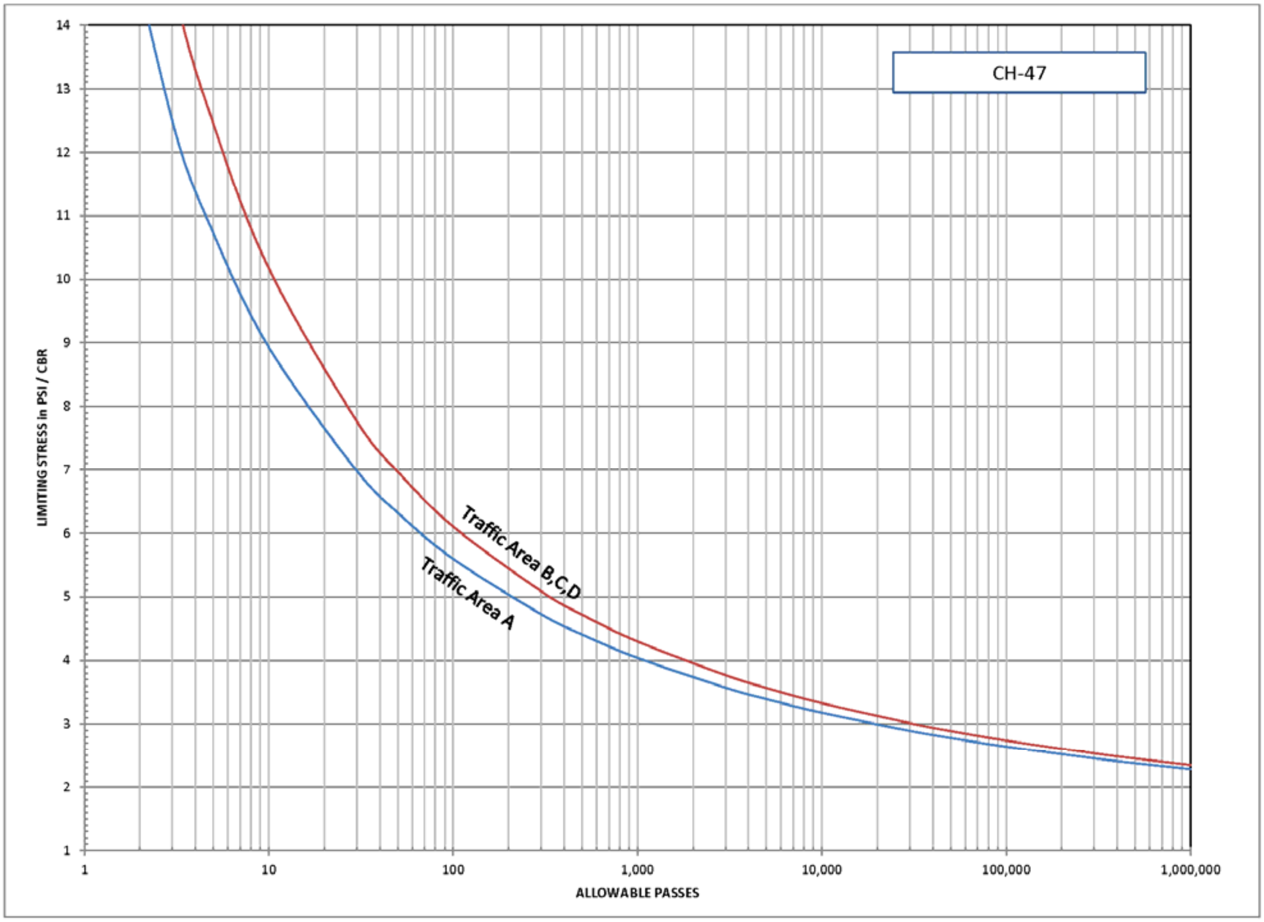


Figure 5-49—Flexible Evaluation Curve for CH-47 (Passes)

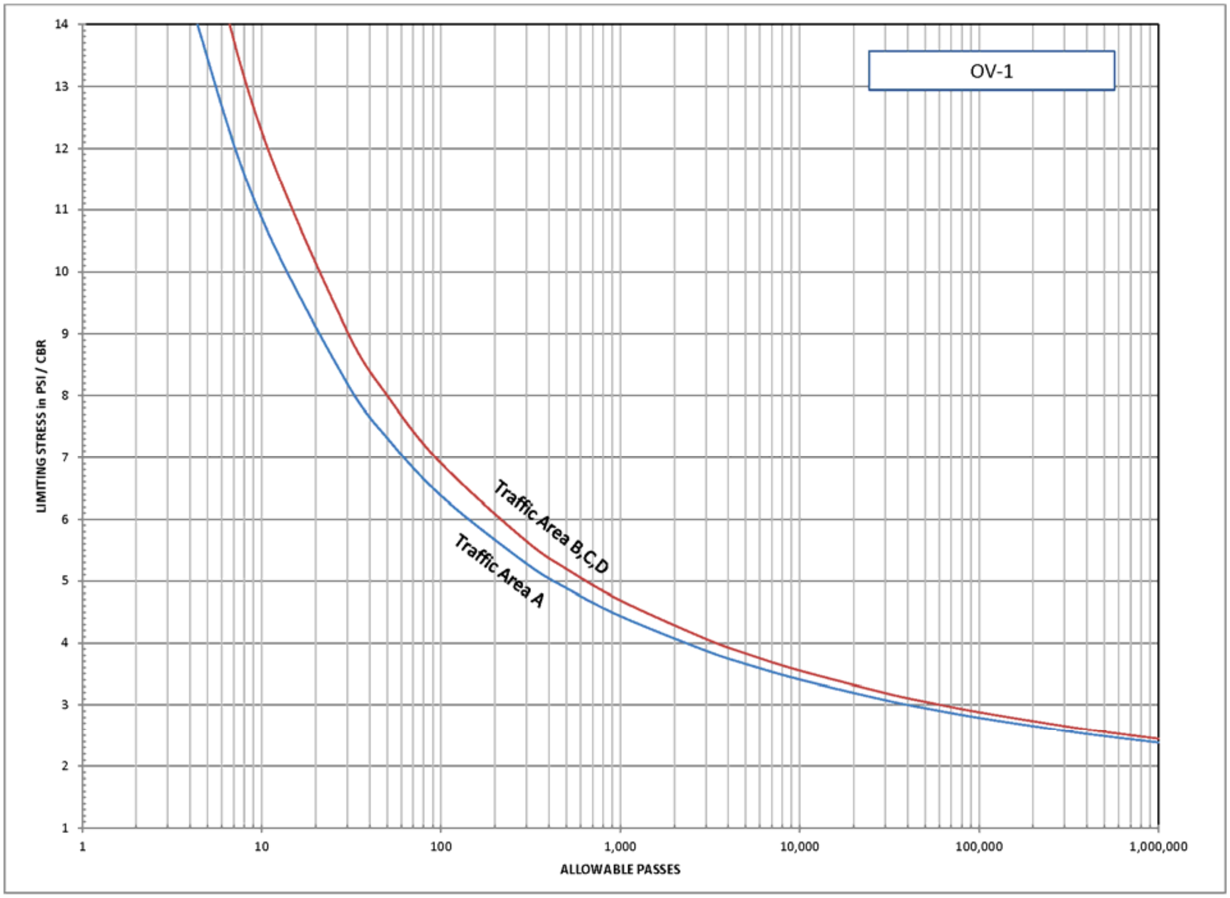


Figure 5-50—Flexible Evaluation Curve for OV-1 (Passes)

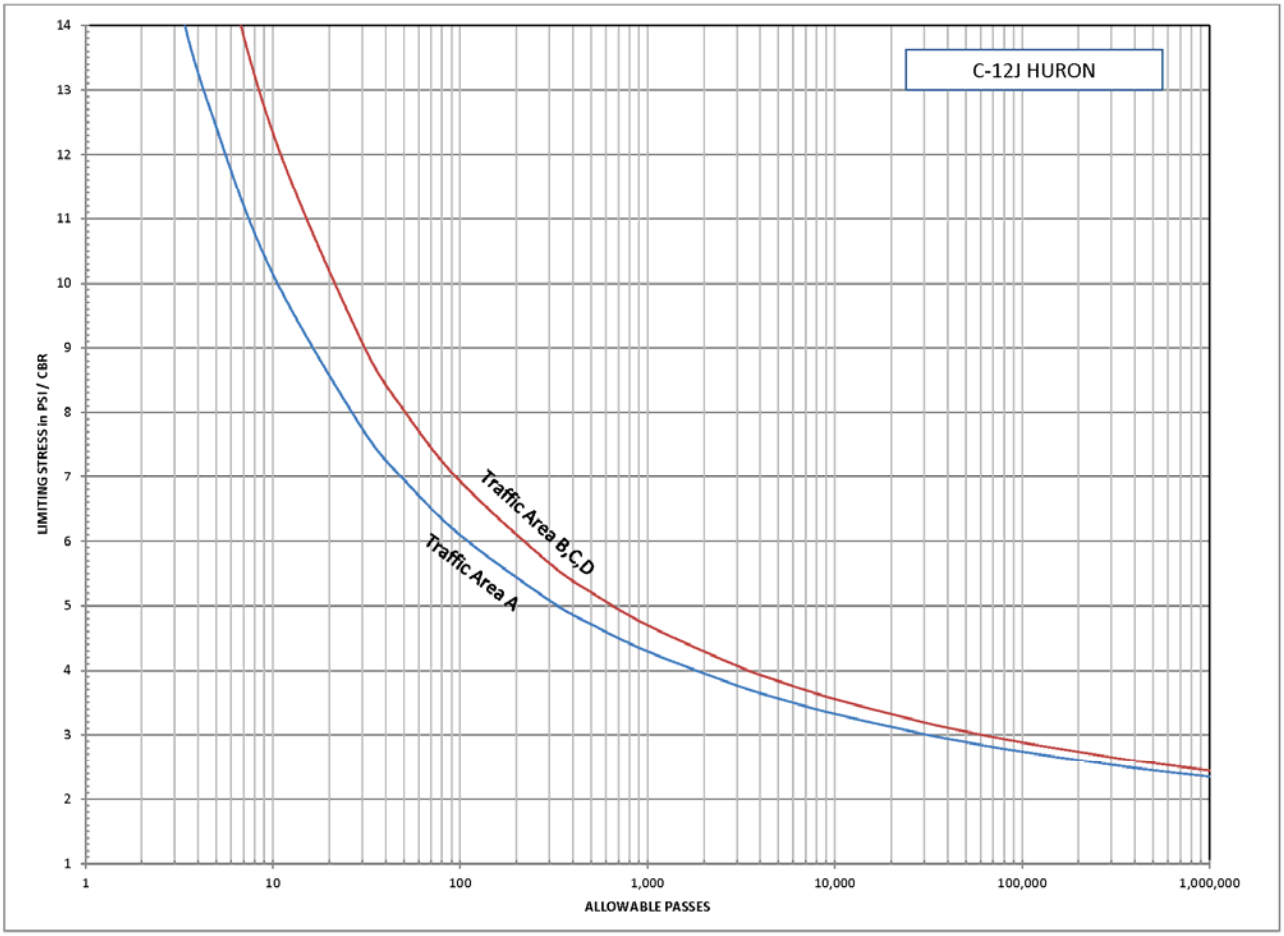


Figure 5-51—Flexible Evaluation Curve for C-12J (Passes)

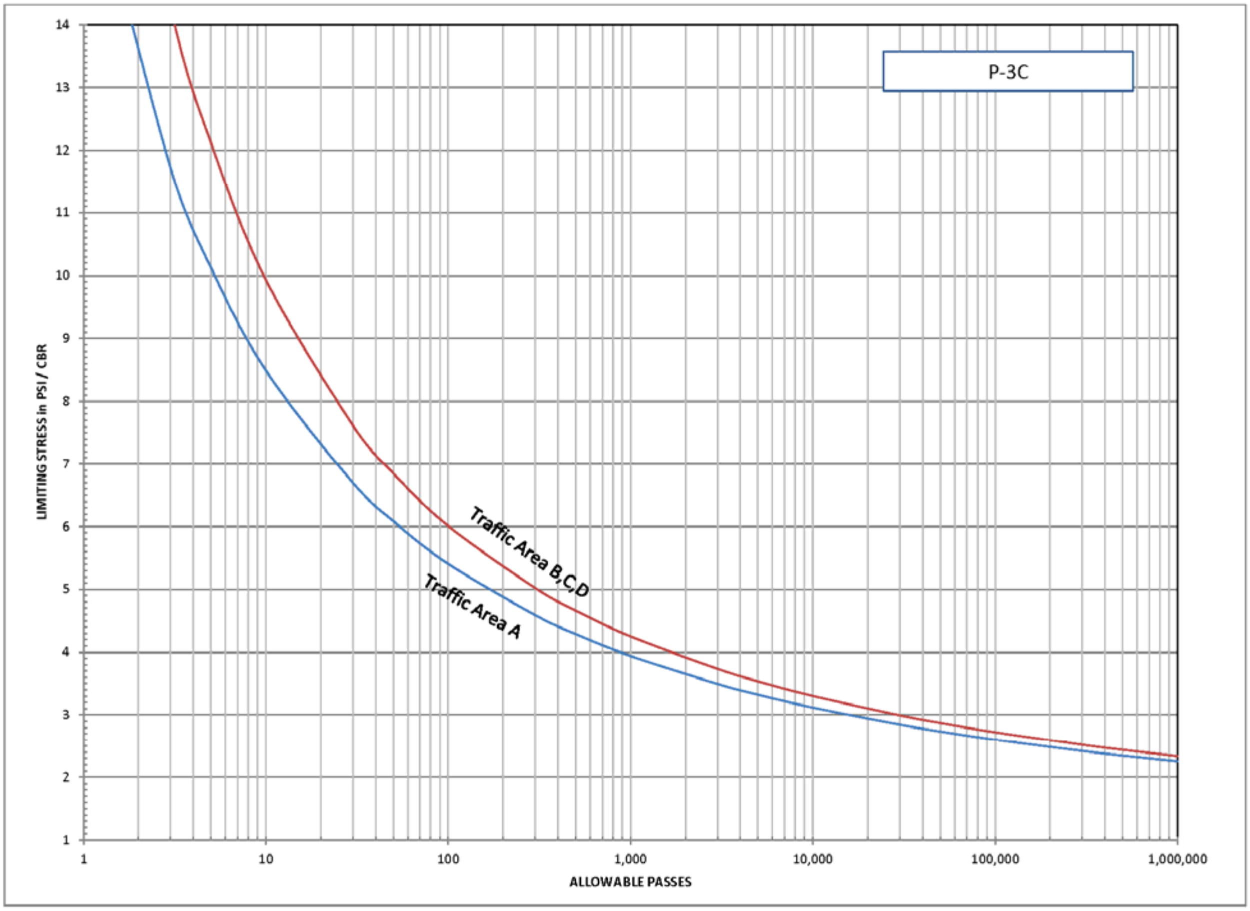


Figure 5-52—Flexible Evaluation Curve for P-3C (Passes)

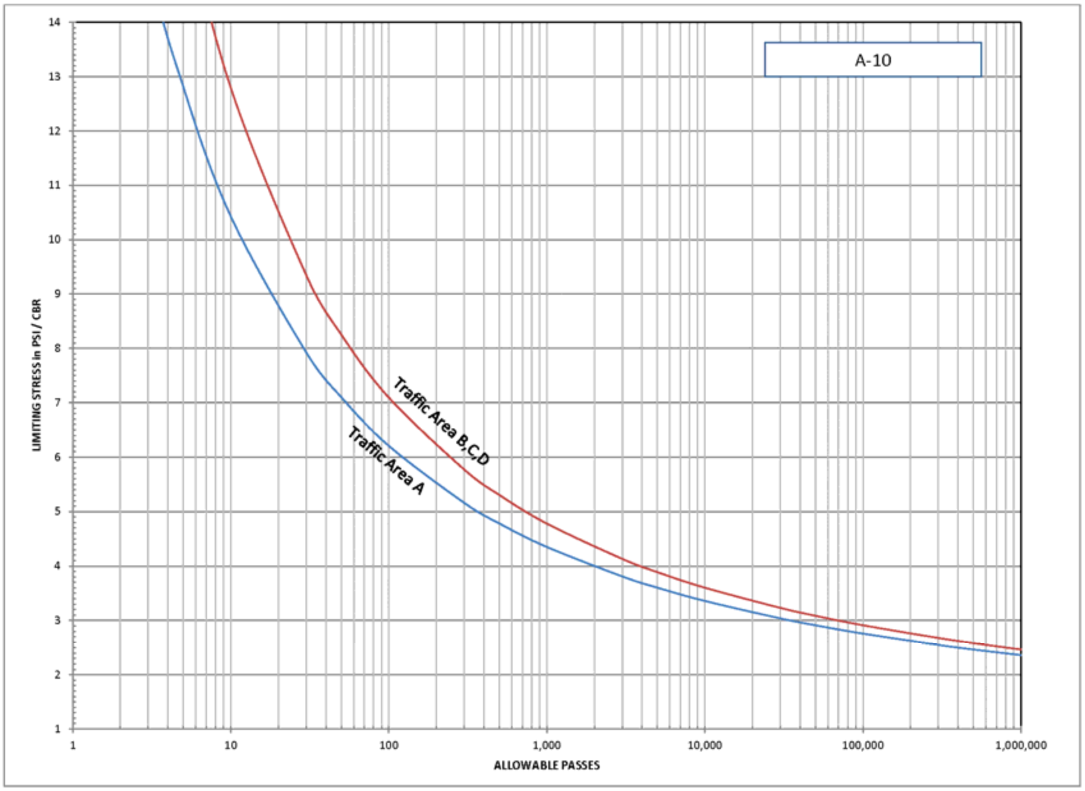


Figure 5-53—Flexible Evaluation Curve for A-10 (Passes)

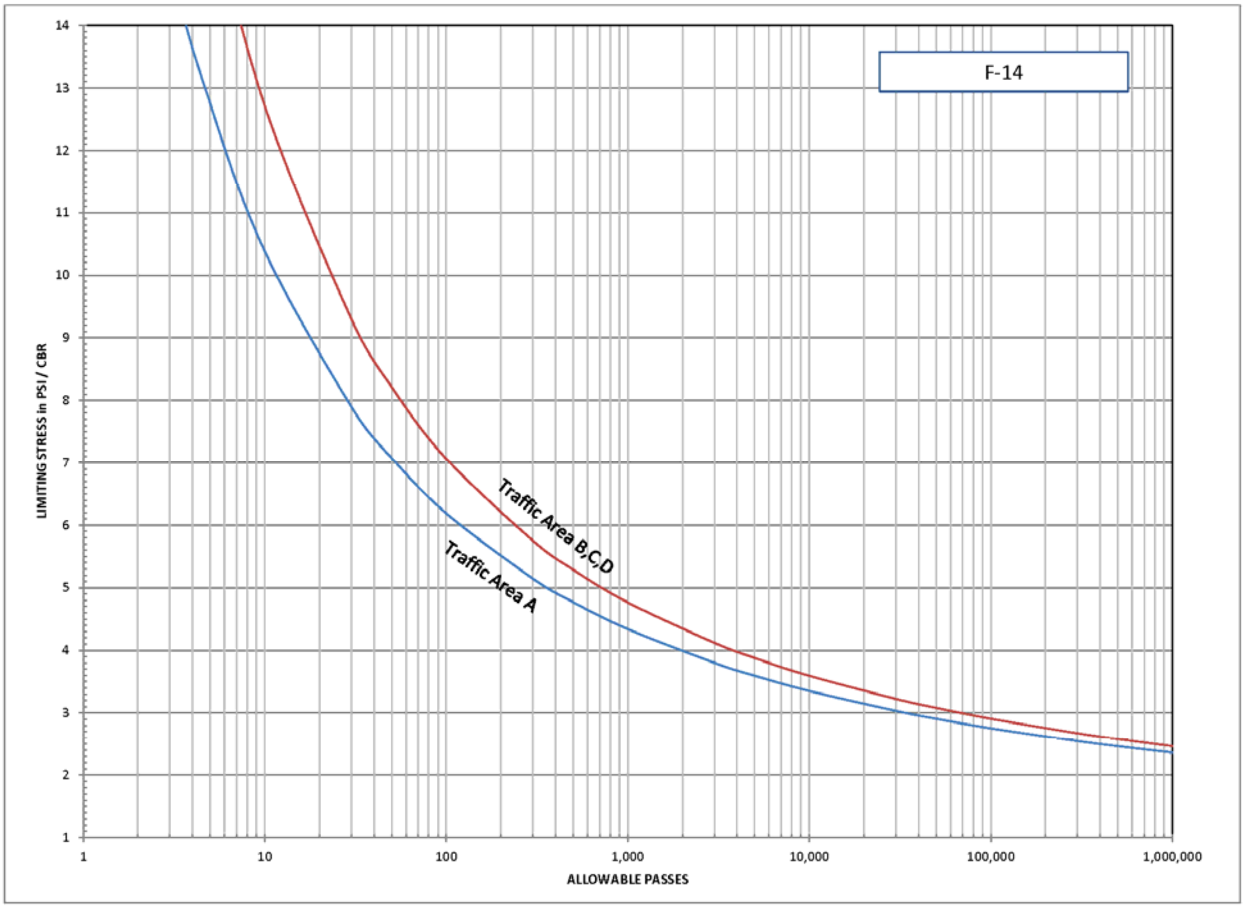


Figure 5-54—Flexible Evaluation Curve for F-14 (Passes)

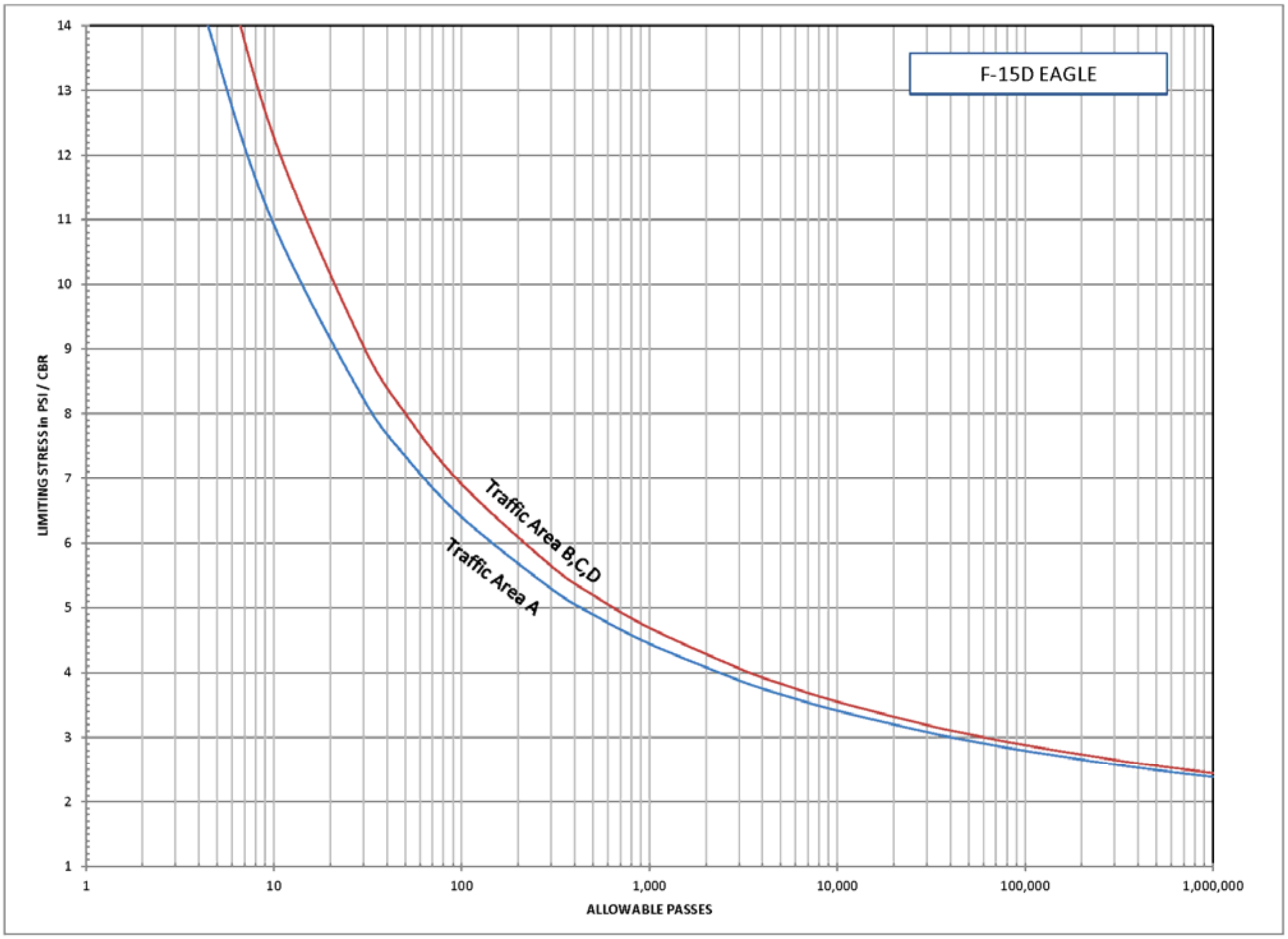


Figure 5-55—Flexible Evaluation Curve for F-15C/D (Passes)

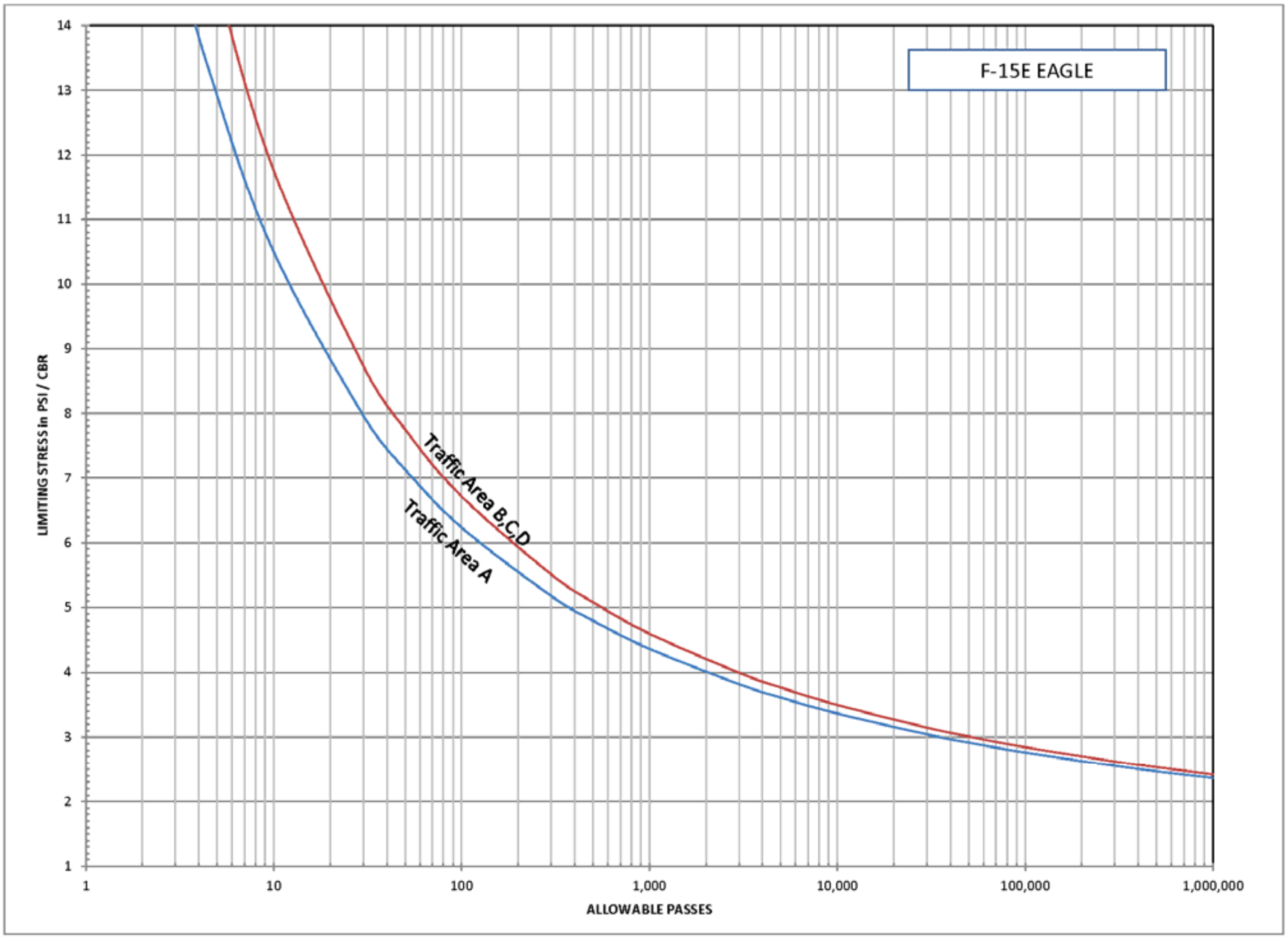


Figure 5-56—Flexible Evaluation Curve for F-15E (Passes)

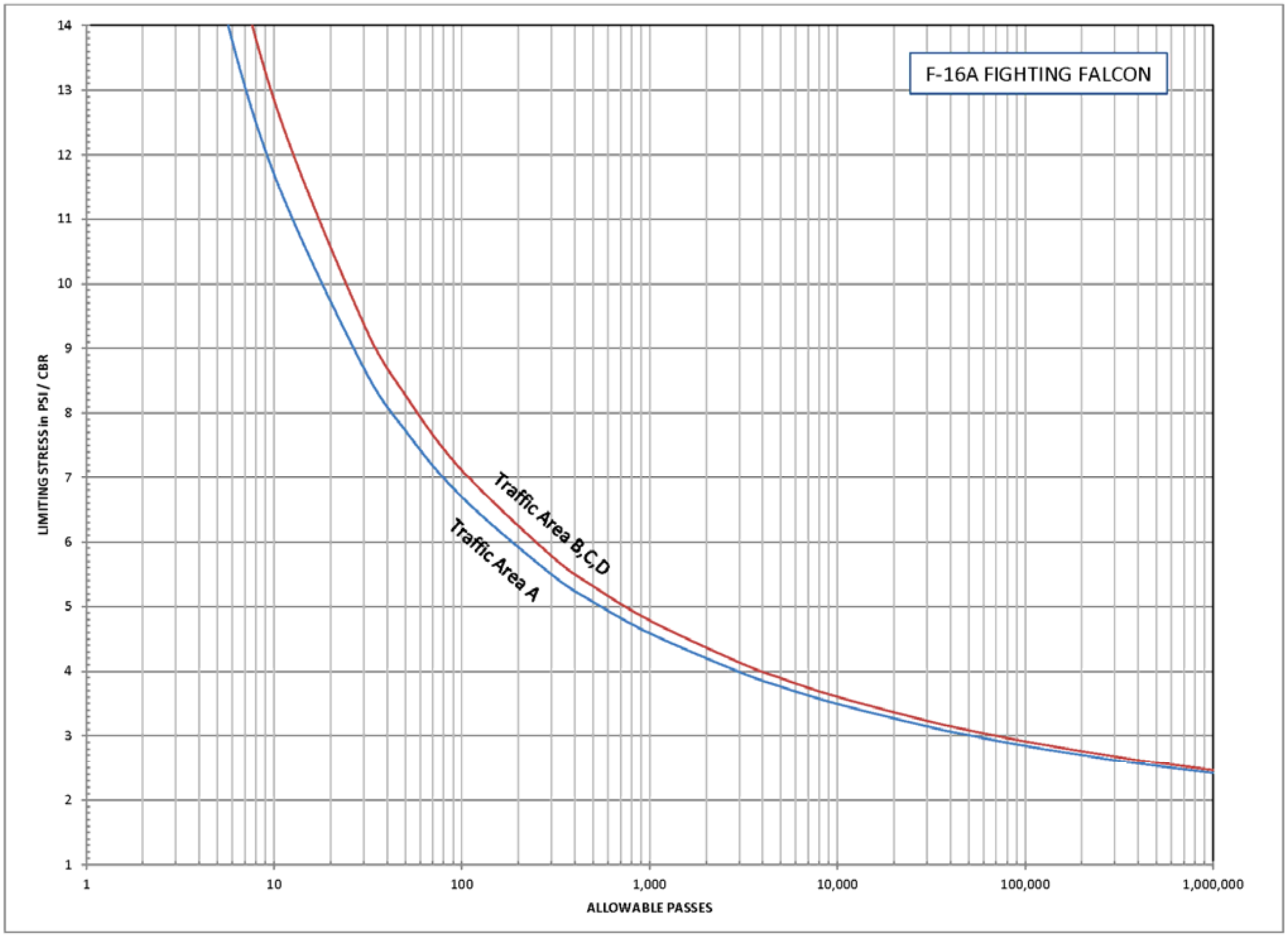


Figure 5-57—Flexible Evaluation Curve for F-16 (Passes)

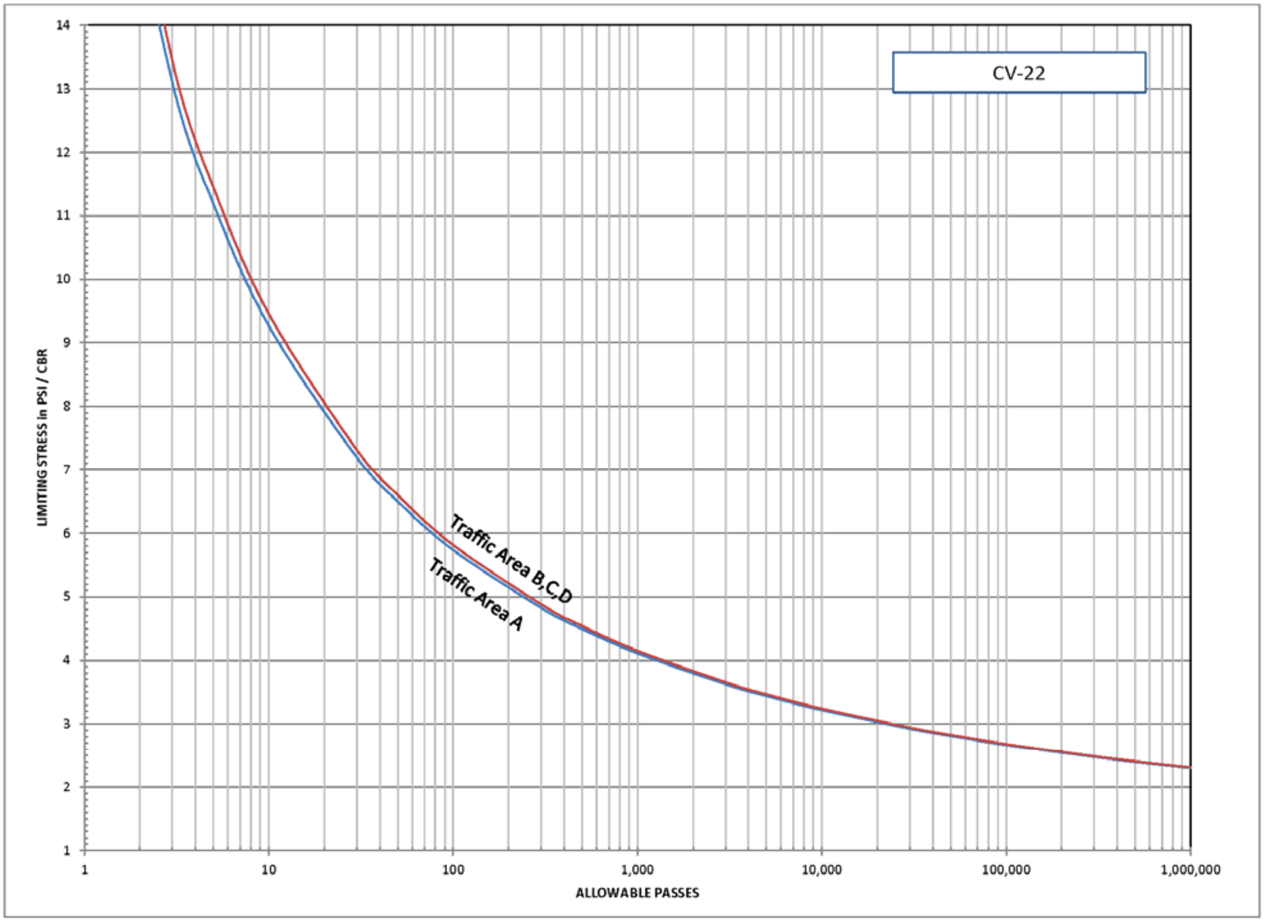


Figure 5-58—Flexible Evaluation Curve for CV-22 (Passes)

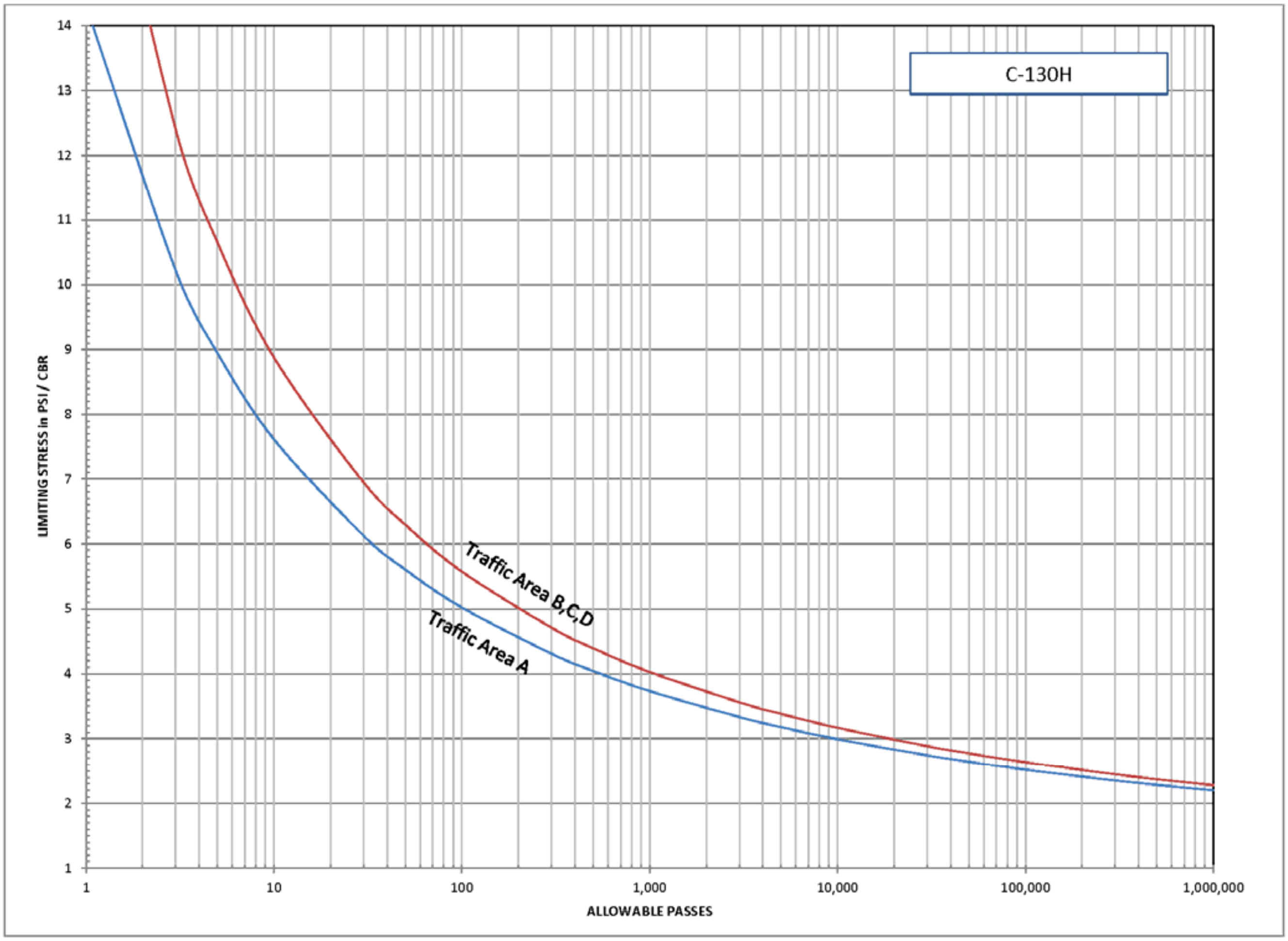


Figure 5-59—Flexible Evaluation Curve for C-130H (Passes)

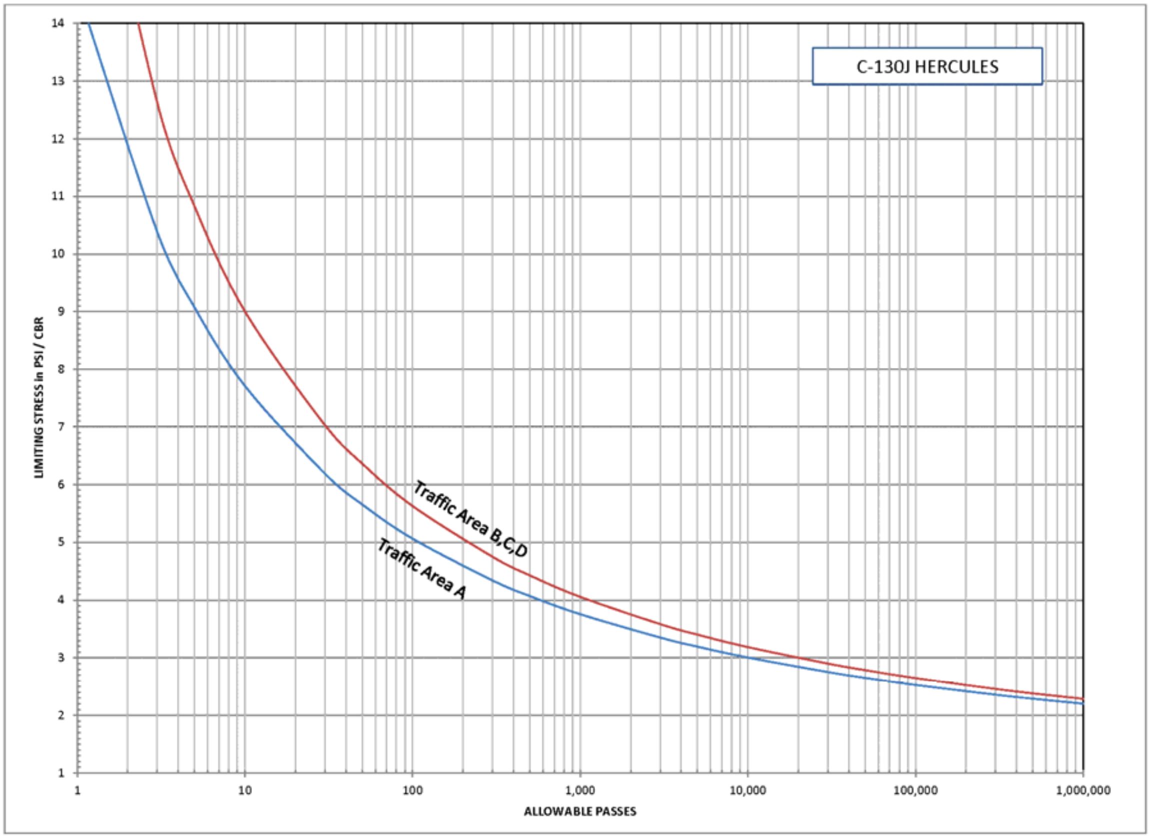


Figure 5-60—Flexible Evaluation Curve for C-130J (Passes)

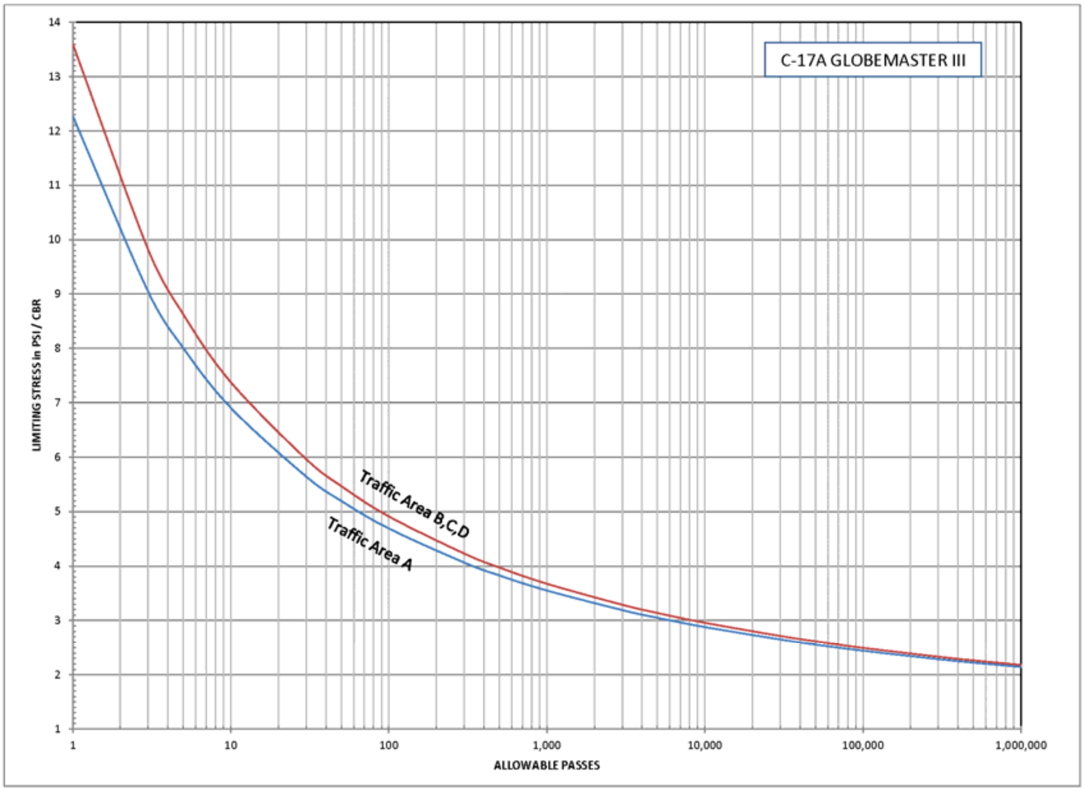


Figure 5-61—Flexible Evaluation Curve for C-17 (Passes)

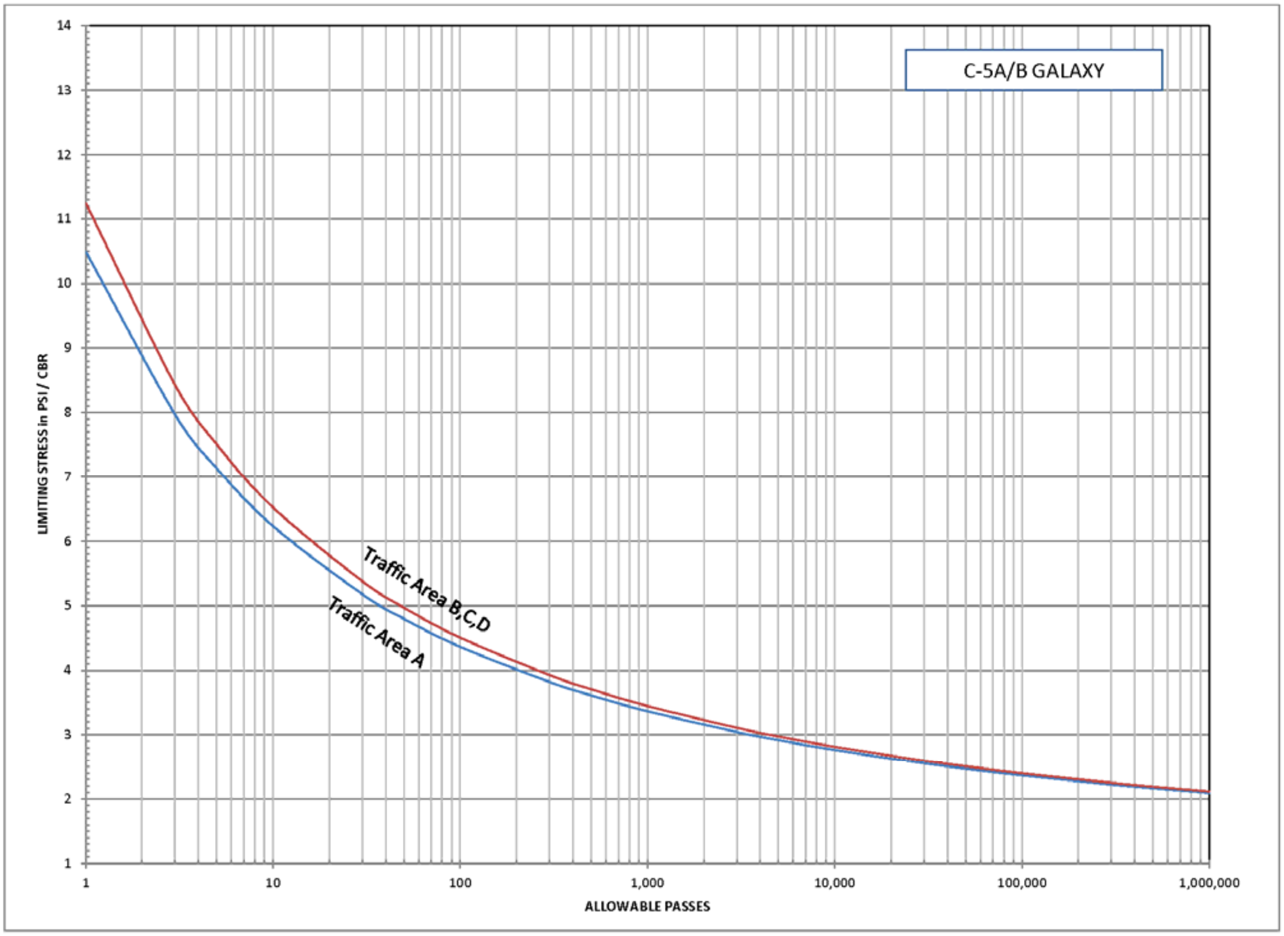


Figure 5-62—Flexible Evaluation Curve for C-5 (Passes)

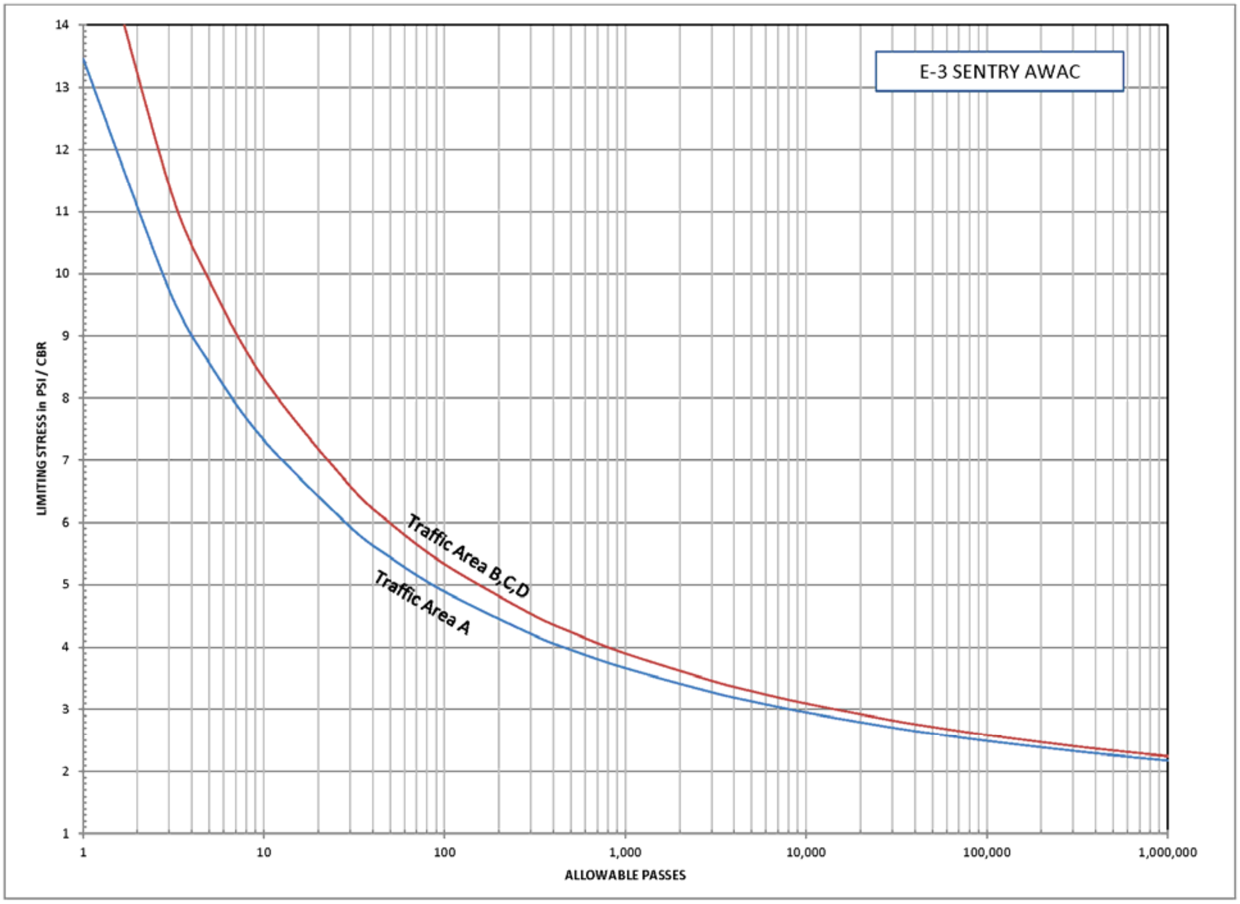


Figure 5-63—Flexible Evaluation Curve for E-3 (Passes)

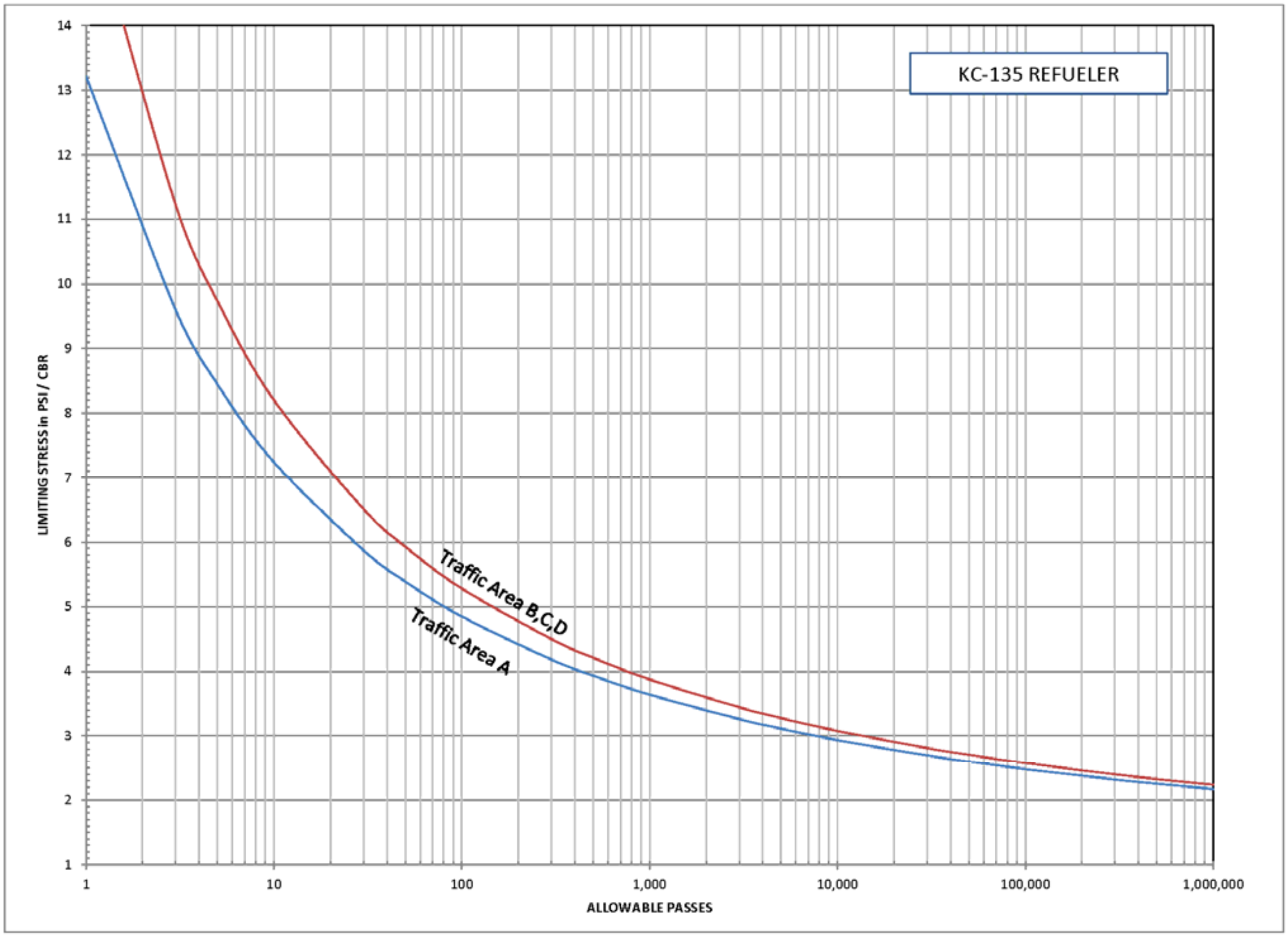


Figure 5-64—Flexible Evaluation Curve for KC-135 (Passes)

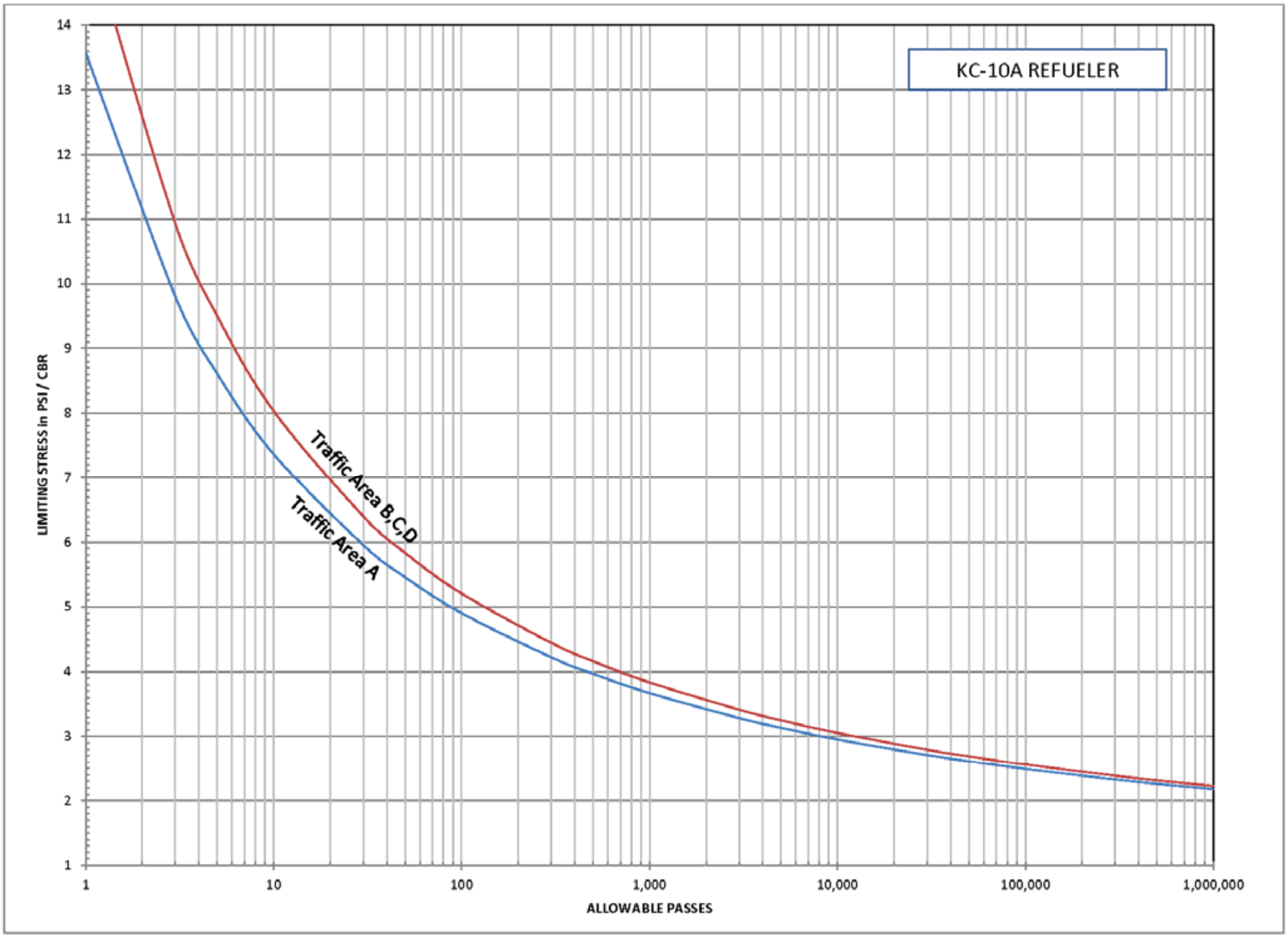


Figure 5-65—Flexible Evaluation Curve for KC-10 (Passes)

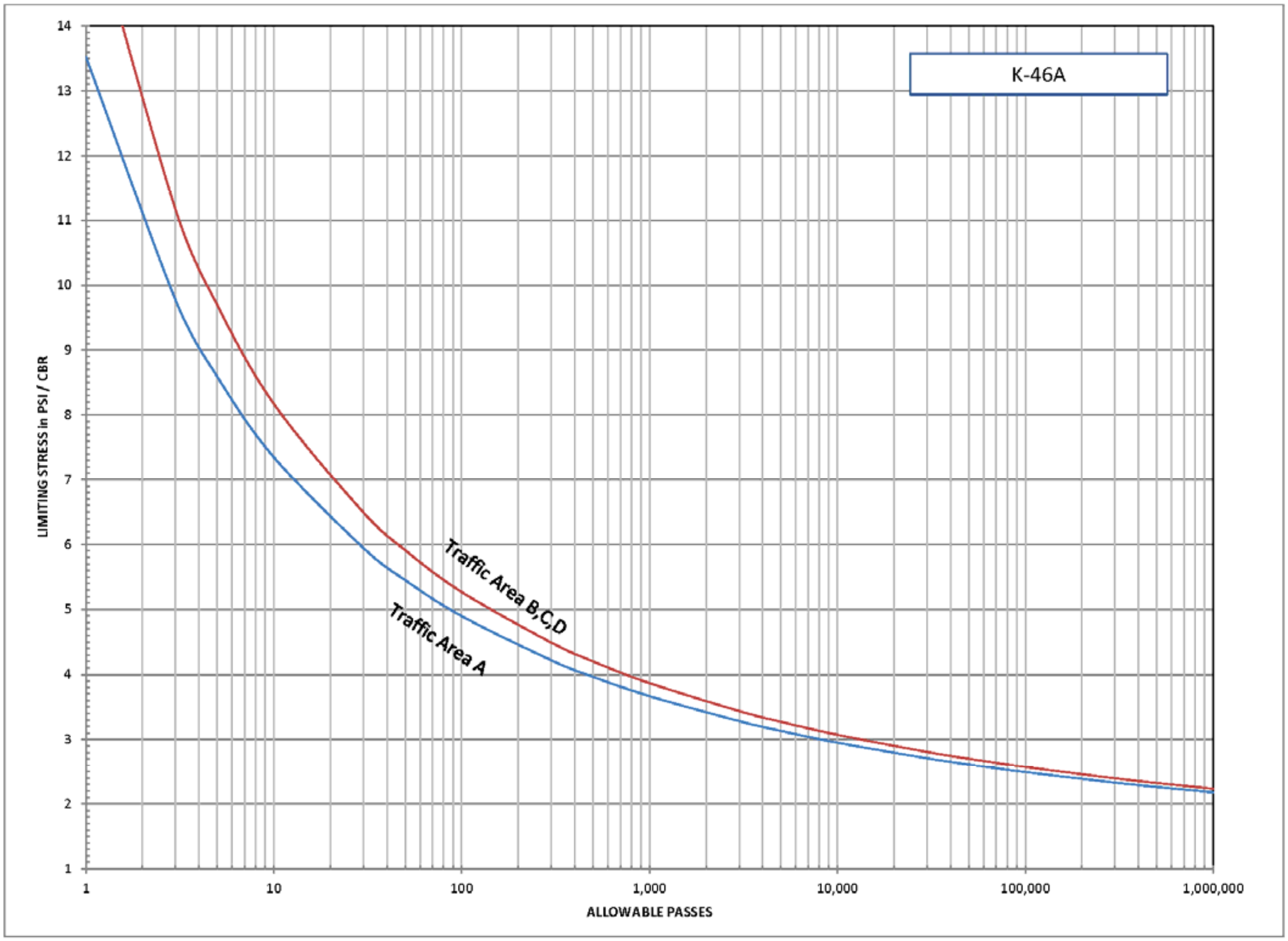


Figure 5-66—Flexible Evaluation Curve for KC-46A (Passes)

CHAPTER 6

EVALUATION OF RIGID PAVEMENTS USING DIRECT SAMPLING

6-1 **GENERAL**. This chapter presents criteria for evaluating rigid pavements and overlays using data from direct sampling. The data required for the evaluations were presented in chapter 3. A computer program is available to assist in a pavement evaluation and is discussed in Appendix H.

6-2 **FACTORS LIMITING LOAD-CARRYING CAPACITY**. The load-carrying capability of rigid pavements is limited by the strength of its weakest component—the portland cement concrete, base course, or subgrade. The ability of a subsurface layer to withstand the loads imposed on it depends on the thickness and strength of material above it and its strength in its weakest condition. An evaluation must also take into account possible future changes in moisture content and density as well as the effects of freezing and thawing where pertinent.

6-3 **SELECTION OF THICKNESS VALUES**. The in-place thicknesses of the portland cement concrete, base courses, and any overlays are determined from actual measurements, existing data, or from laboratory samples. Thicknesses should be measured to the nearest 1/4 inch.

6-4 SELECTION OF STRENGTH VALUES.

a. Concrete Flexural Strength, R.

(1) The R value to be used for each Section in the evaluation should be the arithmetical mean of all R values, except in special instances where, in the opinion of the evaluating engineer, a slightly lower or higher value is more representative of existing conditions. For the Physical Property Data and modeling, round the flexural strength to the nearest 5 psi and cap the flexural strength for individual tests at 850 psi and the average flexural strength flexural strength at 800 psi. The value used for each Section in the evaluation should be the arithmetical mean of all R values, except in special instances where, in the opinion of the evaluating engineer, a slightly lower value is more representative.

(2) When the evaluation is based on design and construction data, the representative R value should be the arithmetical mean of the R values obtained in the construction-control beam tests. Small changes in mix design that might have been necessary during construction to obtain the design strength should be disregarded when selecting representative R values. However, if there was a change in design strength that necessitated a change in mix design, this change should be considered and a representative R value obtained for each facility for which the design strength was changed.

(3) When the evaluation is based on the results of tests conducted at the time of evaluation or when tests are performed to check existing data, the amount of data available for arriving at a representative R value will generally be limited to a relatively few test results. The representative R value may be determined by using the results of tensile splitting tests and calculating the R value as presented in Appendix B, or by conducting flexural strength tests. The results of all tests from a Section should be used to compute an arithmetical mean. High or low results should not be discarded unless it is definitely established that erroneous results were obtained because the sample was defective or because incorrect test procedures were used.

b. Strength Values for Nonrigid Overlays.

(1) Rigid Pavement Procedure. For the evaluation of nonrigid overlay on rigid pavement using rigid pavement evaluation procedures, it is necessary to establish whether the nonrigid overlay portion meets the design requirements given in UFC 3-260-02 Pavement Design for Airfields. Should it not meet design requirements, early failure can be anticipated.

(2) Flexible Pavement Procedure. When a nonrigid overlay on rigid pavement is evaluated using the flexible pavement evaluation procedure, strength and thickness values should be selected in accordance with the procedures discussed in chapter 5 for flexible pavements.

c. Modulus of Soil Reaction, k.

(1) The selection of a representative k value can be made in much the same manner as that used in the selection of R values; however, generally less test data will be available. For evaluation purposes, the k value should be limited to 500 pci. An average k value is computed for each pavement Section. If the K value is more than 200 psi/inch, round down to the nearest 25 psi/inch. If the k value is less than 200 psi/inch, round down to the nearest 10 psi/inch. There will be instances where k values will be considerably higher or lower than the average of the majority of values, in which case a thorough study of foundation conditions should be made at this location to determine whether the test was erroneous or whether the foundation actually is nonuniform. If the test is found to be erroneous, the unusually high or low value should be discarded; if the foundation is actually nonuniform, a more extensive testing program may be needed to select a representative k value. Saturation correction will not be made for k values, since the material will have likely reached an equilibrium moisture content.

(2) A pavement can be investigated using a DCP to determine the bearing capacity

of a pavement structure at various depths. The DCP is a hand-held or automated device that drives a cone-tipped rod into the ground by repeatedly dropping a 17.6-lb hammer. Penetration measurements and hammer blow counts are typically made at 1-inch penetration intervals using the DCP. A correlation was derived between the rod penetration and resistance strength (CBR). Detailed test procedures and correlations for using the DCP device are provided in Appendix B. The CBR of the pavement layers can be converted to Young's Modulus by multiplying it by 1,500 or can be converted to k by using Figure 6-40. Each layer in the pavement structure (base, subbase, subgrade) must be examined to determine if the effective K is less than the k determined from Figure 6-40; use the lower value. The procedure for determining the effective k is outlined below.

(3.) After establishing the pavement structure with the DCP, the K for each layer is determined using Figure 6-40. The effective K for each layer is then determined by using Figures 6-41 through 6-44. The effective K for each layer is compared to the K for each layer determined by Figure 6-40 and the lowest value used for computing the effective K of the next layer.

Use:

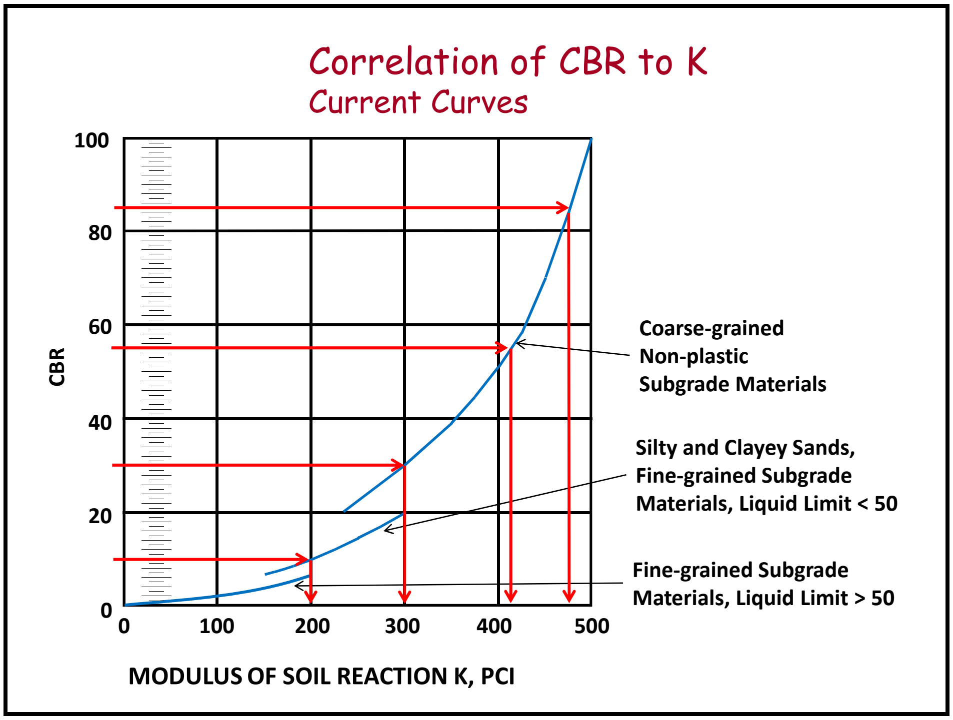
Figure 6-41 for CBR values from 90 - 100
Figure 6-42 for CBR values from 70- 89
Figure 6-43 for CBR values from 50-69
Figure 6-44 for CBR values below 50

Example. Determine the evaluation K for the following pavement section:

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

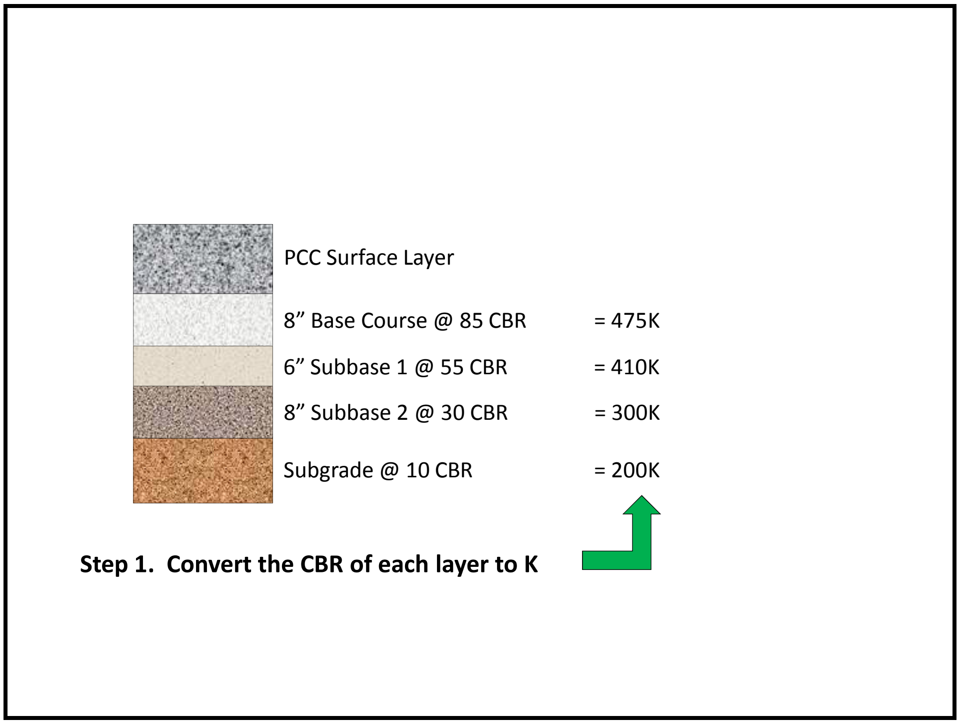
Procedure.

-Use Figure 6-40 to determine the k value for the subgrade, subbase 1, subbase 2, and base course:



Correlation of CBR to K

Current Curves



PCC Surface Layer

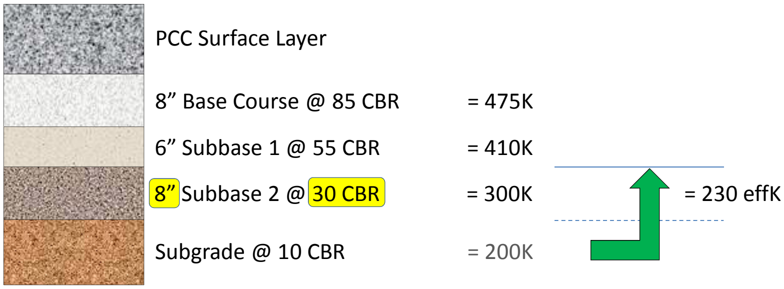
8” Base Course @ 85 CBR = 475K

6” Subbase 1 @ 55 CBR = 410K

8” Subbase 2 @ 30 CBR = 300K

Subgrade @ 10 CBR = 200K

Step 1. Convert the CBR of each layer to K



PCC Surface Layer

8” Base Course @ 85 CBR = 475K

6” Subbase 1 @ 55 CBR = 410K

8” Subbase 2 @ 30 CBR = 300K = 230 effK

Subgrade @ 10 CBR = 200K

Step 2. Start with bottom layer:

Determine the effective K (230) of this layer (SG) based upon the strength and thickness (8”) of the layer (SB 2) immediately above it

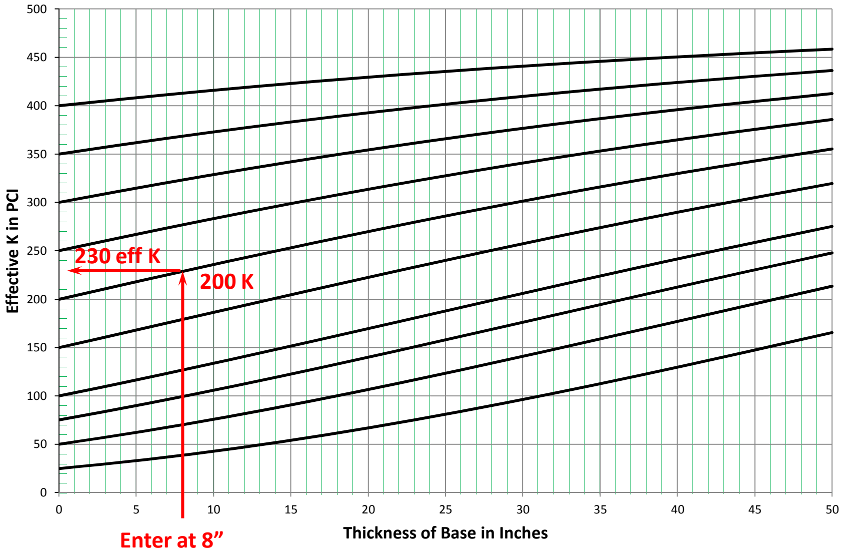
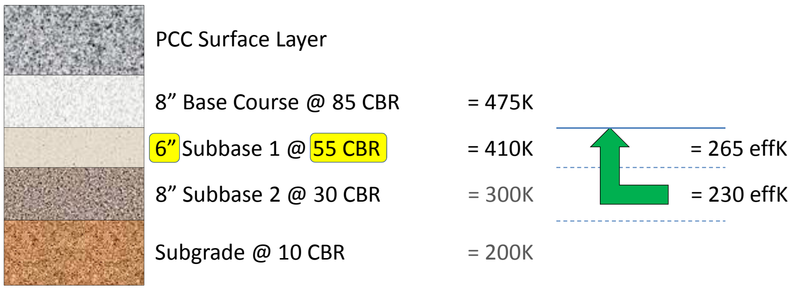


Figure 6-44---CBR < 50



PCC Surface Layer

8” Base Course @ 85 CBR = 475K

6” Subbase 1 @ 55 CBR = 410K = 265 effK

8” Subbase 2 @ 30 CBR = 300K = 230 effK

Subgrade @ 10 CBR = 200K

Step 3. Compare the k values (300 and 230) at the top of SB 2 and continue upward with the lowest one (230):

Step 4. Determine the effective K (265) of this layer based upon the strength and thickness (6”) of the layer (SB 1) above it

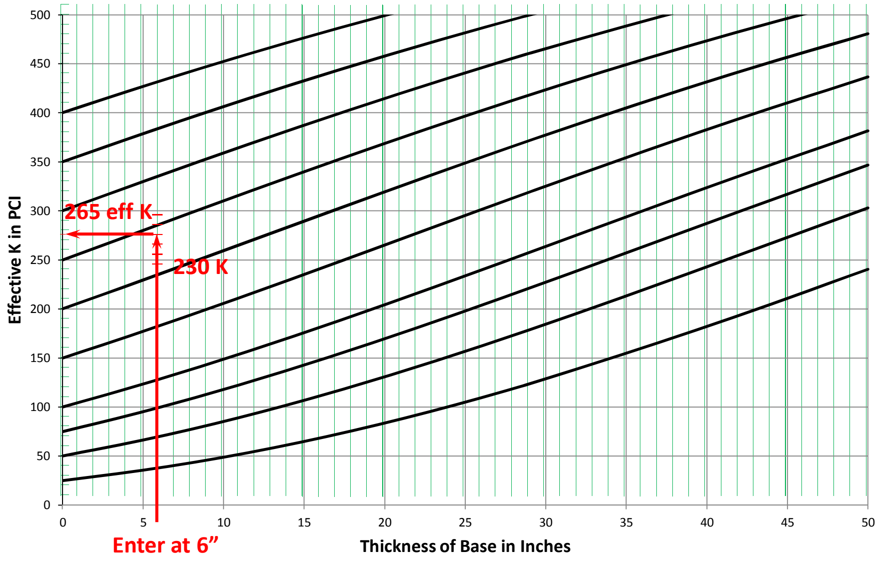


Figure 6-43 –50≤CBR<70

PCC Surface Layer

8” Base Course @ 85 CBR = 475K = 335 eff K

6” Subbase 1 @ 55 CBR = 410K = 265 eff K

8” Subbase 2 @ 30 CBR = 300K = 230 eff K

Subgrade @ 10 CBR = 200K

Step 5. Compare the K values (410 and 265) at the top of SB 1 and continue upward with the lowest one (265):

Step 6. Determine the effective K (335) of this layer based upon the strength and thickness (8”) of the layer (BC) above it

Step 7. Compare the eff K values of the layers immediately beneath the PCC (in this case 475 and 335) and use the lowest eff K (335) to evaluate the PCC

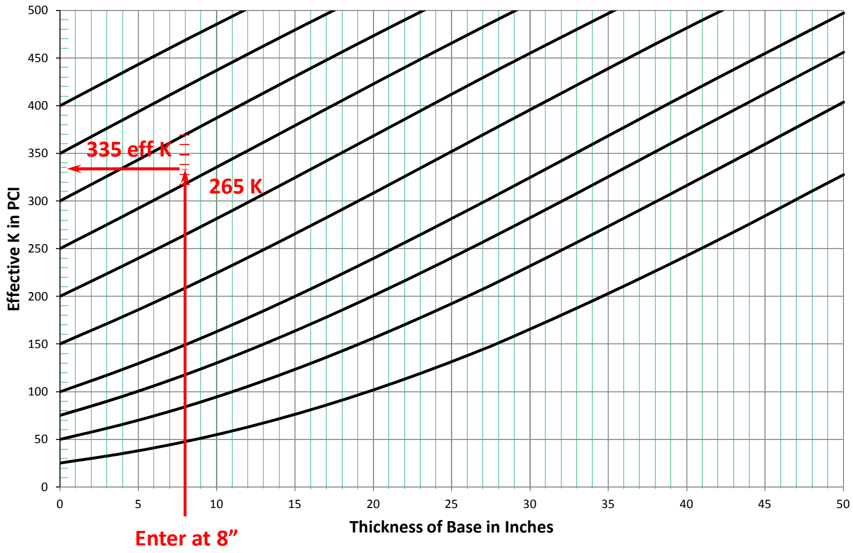


Figure 6-42-- 70 ≤ CBR < 90

Summary:

-From Figure 6-40, k of the subgrade =200 pci

-From Figure 6-44, k on top of Subbase 2=230 pci
-From Figure 6-43, k on top of Subbase 1=265 pci
-From Figure 6-41, k on top of Base =335 pci, Effective K

c. Limiting Conditions.

(1) When conditions do not indicate concrete or soil of normal physical properties, the evaluation must be modified accordingly. Ideal conditions seldom exist, and full consideration should be given to the probable influence of factors such as those outlined below. The narrative portion of the evaluation report should contain a discussion of the effect that any of the following factors might have on the evaluation of the pavement:

(a) High moisture absorption and shrinkage of the concrete.

(b) Extremely high daily variation in temperature.

(c) Wide variation in the flexural strength within a given pavement Section.

(d) Heterogeneous subgrade, base, or moisture conditions resulting in wide variations in modulus of soil reaction values.

(e) Nonrigid overlays (bituminous concrete and flexible overlay) that do not meet design requirements for flexible pavements.

(f) Unsatisfactory load transfer at the joints.

(2) No set method has been established for reducing the allowable loading for conditions such as those outlined above. Nonrigid overlays not meeting design requirements might be susceptible to rutting or raveling. If it can be determined that inadequate load transfer conditions exist at the joints, a reduction of up to 25 percent in the allowable load could be justified. When a PCI survey results in ratings of Very Poor, Serious, or Failed (i.e., PCI≤ 40) due primarily to structural cracking, the pavement is assumed to have inadequate load transfer. Any reduction in the allowable loading will be a matter of judgment, and the engineer must explore all possible sources of information consistent with the job conditions and perform such tests as are feasible to obtain factual data useful in determining the amount of reduction necessary.

6-5 **PLAIN CONCRETE PAVEMENTS**. Plain concrete pavements are evaluated using stresses due to load at the edge of a slab.

a. Edge Loading Condition. There are two basic evaluation criteria for plain concrete pavements. These two criteria are the standard evaluation and the extended life evaluation. Army and Navy airfield pass/load relationships are reported for both criteria. Air Force evaluations are reported using the extended life criteria.

(1) Standard Evaluation. The standard evaluation criteria are essentially the reverse of design and are based upon criteria where 50 percent of the slabs are cracked into two or three pieces at the end of traffic (sometimes referred to as initial failure or first crack failure).

(2) Extended Life Evaluation. The extended life evaluation is based upon a criterion where 50 percent of the slabs are cracked into approximately six pieces at the end of traffic (sometimes referred to as shattered slab failure). A slab cracked into 4 pieces is considered shattered if cracks are medium or high severity.

b. Data Required. The data required for evaluation of plain concrete pavements are presented in Chapter 3. In addition, if the pavement structure contains a stabilized layer, it will be necessary to obtain the modulus of elasticity and thickness of the stabilized layer. The stabilized layer is considered as a low-strength base pavement, and the following equation will be used to determine an equivalent thickness of the combined pavement:

$$h_E = \sqrt[1.4]{(h_e)^{1.4} + \left(\sqrt[3]{\frac{E_s}{E_c}}\, h_s \right)^{1.4}} \qquad \text{(Eq 6-1)}$$

Where:

h_E = thickness of plain concrete equivalent to the combined pavement and stabilized layer thickness, inches

h_e = thickness of concrete pavement, inches

h_s = thickness of stabilized layer, inches

E_c = modulus of elasticity of concrete. The curves in this UFC were developed using 4,000,000 psi. The modulus values used in PCASE can be modified, based on engineering judgment. However, the UFC and PCASE should be consistent, unless there is evidence to suggest otherwise.

E_s = flexure modulus of elasticity of the stabilized layers, psi. May be determined from Table 6-1 or calculated using deflections resulting from ASTM D 1635.

With this h_E value, the evaluation is made using the flexural strength of the pavement and the modulus of subgrade reaction of the material below the stabilized layer.

Compressive Strength, psi	**Modulus of Elasticity, psi**
500 – 750	500,000
750 - 1,000	800,000
1,000 - 1,500	1,200,000
1,500 - 2,000	1,600,000
Over 2,000	2,000,000

Table 6-1. Es Values for Stabilized Layers (Guide when Es not Available)

c. Evaluation Procedure for plain concrete pavements. After the existing thickness or equivalent thickness, flexural strength, and modulus of soil reaction have been determined, the evaluation of a pavement for Allowable Load or Passes is made using the following procedure.

(1) Design Factor for Standard or Extended Life evaluation. The Design Factor (DF) is defined as the ratio of PCC strength to edge stress. It is a function of the number of passes, the Pass/Coverage ratio of the aircraft for the appropriate traffic area from Table 6-2, and K or effective K immediately under the PCC pavement. The Design Factor for any aircraft is determined using Figure 6-1 for a Standard evaluation and Figure 6-2 for an Extended Life evaluation.

(2) Pass/Coverage Ratio. Rigid pavement evaluation curves require the use of a Pass/Coverage ratio, which is specific to each aircraft. The definition of the P/C ratio is in paragraph 2.2.d. Pass/Coverage ratios for most commercial and military aircraft for rigid and flexible pavements are in Table 6-2 and are also shown on the rigid pavement evaluation curves, Figures 6-4 to 6-35.

	RIGID P/C RATIO		FLEXIBLE P/C RATIO	
AIRCRAFT NAME	**TRAFFIC AREA A**	**TRAFFIC AREA B,C,D**	**TRAFFIC AREA A**	**TRAFFIC AREA B,C,D**
AF-GROUP01	8.875	13.524	8.875	13.524
AF-GROUP02	7.948	11.757	7.948	11.757
AF-GROUP03	3.615	6.784	3.615	6.784
AF-GROUP04	4.647	9.016	2.323	4.508
AF-GROUP05	4.322	8.146	4.322	8.146
AF-GROUP06	3.610	6.660	3.610	6.660
AF-GROUP07	2.965	5.507	2.965	5.507
AF-GROUP08	3.332	5.906	1.666	2.953
AF-GROUP09	3.480	5.617	1.740	2.808
AF-GROUP10	1.380	1.899	1.380	1.899
AF-GROUP11	1.400	1.644	0.810	1.033
AF-GROUP12	2.874	4.896	1.691	2.512
AF-GROUP13	3.118	4.387	1.559	2.193
AF-GROUP14	1.676	1.950	1.676	1.950
A-10	7.914	15.727	7.914	15.727
A-4 SKYHAWK	10.916	14.767	10.916	14.767
A-6 INTRUDER	7.291	14.479	7.291	14.479
A-7 CORSAIR II	8.723	13.481	8.723	13.481
AC-130H SPECTRE	4.648	9.023	2.324	4.511
AC-130U SPOOKY	4.648	9.023	2.324	4.511
AH-64 APACHE LONGBOW W/FCR	9.087	11.659	9.087	11.659
AIRBUS 400M	4.051	7.200	1.350	2.400
AIRBUS A319-100	3.870	6.746	3.870	6.746
AIRBUS A320-200	5.079	9.217	2.539	4.608
AIRBUS A330-200	1.916	2.822	1.916	2.822
AIRBUS A340-200	1.912	2.810	1.912	2.810
AIRBUS A380-800	3.875	4.981	1.443	1.823
AN-12 RUSSIAN	4.934	9.530	2.467	4.765
AN-124 RUSSIAN	3.224	5.490	0.645	1.098
AN-225 MRIYA COSSACK	3.270	5.572	0.467	0.796

AIRCRAFT NAME	**TRAFFIC AREA A**	**TRAFFIC AREA B,C,D**	**TRAFFIC AREA A**	**TRAFFIC AREA B,C,D**
ANDOVER C MK 1 & E MK 3	3.885	7.370	3.885	7.370
ANDOVER CC MK 2	3.963	7.619	3.963	7.619
ANDOVER CC MK 2 QUEENS FLIGHT	3.877	7.454	3.877	7.454
AO/A-10-A THUNDERBOLT II	7.800	15.500	7.800	15.500
AT-38B TALON	18.541	32.206	18.541	32.206
AV-8 HARRIER	3.194	6.272	3.194	6.272
B-1B LANCER	3.413	5.630	1.706	2.815
B-2 SPIRIT	3.920	6.806	1.960	3.403
B-52H STRATOFORTRESS	1.643	1.974	1.643	1.974
BAE (HS) HAWK T MK1	11.838	21.069	11.838	21.069
BAE BUCCANEER S MK 2A & 3B	8.444	15.566	8.444	15.566
BAE JETSTREAM T MK3	9.244	18.434	9.244	18.434
BAE NIMROD R MK1- MR MK2	4.325	8.305	2.146	4.145
BAE PEMBROKE C MK1	6.075	11.896	6.075	11.896
BAE146-100	4.062	7.390	4.062	7.390
BOEING 707-120B	3.835	6.803	1.918	3.401
BOEING 707-320B	3.550	6.268	1.775	3.134
BOEING 707-320C	3.489	6.160	1.745	3.080
BOEING 707-420	3.477	6.139	1.739	3.069
BOEING 717-200	3.610	6.660	3.610	6.660
BOEING 720	3.657	6.572	1.829	3.286
BOEING 720B	3.657	6.572	1.829	3.286
BOEING 727-100/100C	3.273	5.787	3.273	5.787
BOEING 727-200	3.298	5.834	3.298	5.834
BOEING 737-100	3.901	7.045	3.901	7.045
BOEING 737-200	3.846	6.945	3.846	6.945
BOEING 737-200 ADV /-200C/-200QC	3.892	7.028	3.892	7.028

AIRCRAFT NAME	TRAFFIC AREA A	TRAFFIC AREA B,C,D	TRAFFIC AREA A	TRAFFIC AREA B,C,D
BOEING 737-300	3.941	7.118	3.941	7.118
BOEING 737-400	3.752	6.774	3.752	6.774
BOEING 737-500	3.893	7.030	3.893	7.030
BOEING 737-600	4.015	7.108	4.015	7.108
BOEING 737-700/700C	3.883	6.873	3.883	6.873
BOEING 737-800	3.623	6.409	3.623	6.409
BOEING 737-900	3.622	6.409	3.622	6.409
BOEING 737-900ER	3.596	6.362	3.596	6.362
BOEING 737-BBJ	3.678	6.508	3.678	6.508
BOEING 737-BBJ2	3.614	6.395	3.614	6.395
BOEING 747-100B/300	3.769	5.310	1.885	2.655
BOEING 747-200B/-200B COMBI/-300	3.588	5.053	1.794	2.526
BOEING 747-200C/-200F	3.553	5.003	1.776	2.502
BOEING 747-300COMBI	3.653	5.146	1.827	2.573
BOEING 747-400	3.619	5.107	1.810	2.554
BOEING 747-8	3.612	4.931	1.806	2.466
BOEING 747-SP	4.115	5.848	2.046	2.923
BOEING 757-200PF	4.037	7.165	2.019	3.582
BOEING 757-300	4.027	7.146	2.014	3.573
BOEING 767-200	4.142	6.738	2.071	3.369
BOEING 767-200ER	3.731	6.068	1.866	3.034
BOEING 767-300	3.982	6.477	1.991	3.239
BOEING 767-300 FREIGHTER	3.716	6.043	1.858	3.022
BOEING 767-300ER	3.738	6.078	1.869	3.039
BOEING 767-400ER	3.692	5.961	1.846	2.980
BOEING 777-200	4.310	6.347	1.437	2.116
BOEING 777-300	4.197	6.181	1.399	2.060
C-12C/D HURON	8.181	15.910	8.181	15.910
C-12F HURON	7.469	14.524	7.469	14.524
C-12J HURON	7.158	14.031	7.158	14.031
C-130E HERCULES	4.412	8.555	2.206	4.278

AIRCRAFT NAME	TRAFFIC AREA A	TRAFFIC AREA B,C,D	TRAFFIC AREA A	TRAFFIC AREA B,C,D
C-130H	4.406	8.546	2.203	4.273
C-130-J HERCULES	4.667	9.056	2.334	4.528
C-130J-30 HERCULES	4.647	9.016	2.323	4.508
C-135A	3.450	6.030	1.725	3.015
C-135B	3.450	6.030	1.725	3.015
C-17A GLOBEMASTER III	1.380	1.899	1.380	1.899
C-20 A-B-C-D GULFSTREAM III	4.737	8.931	4.737	8.931
C-20 F-G-H GULFSTREAM IV	4.758	8.971	4.758	8.971
C-212 CASA C-41A	7.333	12.062	7.333	12.062
C-21A GATES LEARJET 35	8.351	16.362	8.351	16.362
C-22B	3.275	5.790	3.275	5.790
C-23	7.934	15.421	7.934	15.421
C-27	6.427	11.899	3.214	5.950
C-27J SPARTAN	5.409	10.027	2.704	5.014
C-295 CASA	7.133	13.666	3.567	6.833
C-2A	7.925	15.790	7.925	15.790
C-32A/B	4.038	7.166	2.019	3.583
C-37A GULFSTREAM V	4.480	8.442	4.480	8.442
C-38A COURIER	6.719	9.719	6.719	9.719
C-40A CLIPPER	3.864	6.839	3.864	6.839
C-40B/C	3.677	6.506	3.677	6.506
C-5A/B GALAXY	1.399	1.643	0.810	1.032
C-9 A/C NIGHTINGALE	3.728	6.924	3.728	6.924
CANBERRA B MK15-B MK16	6.628	13.084	6.628	13.084
CANBERRA B MK6-PR MK7-B MK8	6.560	12.950	6.560	12.950
CANBERRA PR MK3-T MKII- TT MK18	5.955	11.743	5.955	11.743
CANBERRA PR MK9	6.579	12.988	6.579	12.988

AIRCRAFT NAME	TRAFFIC AREA A	TRAFFIC AREA B,C,D	TRAFFIC AREA A	TRAFFIC AREA B,C,D
CANBERRA TMK4-T MK17	5.808	11.450	5.808	11.450
CH-47	4.376	7.226	4.376	7.226
CH-53	5.044	9.409	5.044	9.409
CH-53K	4.491	8.273	4.491	8.273
CHINOOK	4.004	6.564	4.004	6.564
CV-22	5.072	5.491	5.072	5.491
CV-580 CONAIR/CONVAIR	3.687	6.919	3.687	6.919
DASH 7	4.689	8.844	4.689	8.844
DC-10-40	3.005	5.127	1.836	2.727
DC-8-63F/73F	3.358	6.104	1.679	3.052
DC-9-51	3.724	6.884	3.724	6.884
E-2C	8.954	17.854	8.954	17.854
E-3 SENTRY AWAC	3.682	6.507	1.841	3.254
E-4B NATIONAL AIRBORNE OPERATIONS CENTER	3.715	5.238	1.857	2.619
E-8C JOINT STARS	3.477	6.138	1.739	3.069
EC-130E COMMANDO SOLO	4.282	8.297	2.141	4.149
EC-130H COMPASS CALL	4.366	8.464	2.183	4.232
EC-130J COMMANDO SOLO	4.668	9.058	2.334	4.529
EC-130J SUPER J	4.710	9.141	2.355	4.571
EC-135A	3.450	6.030	1.725	3.015
EC-135C	3.450	6.030	1.725	3.015
EC-135E	3.450	6.030	1.725	3.015
EC-135G	3.450	6.030	1.725	3.015
EC-135H	3.450	6.030	1.725	3.015
EC-135J	3.450	6.030	1.725	3.015
EC-135K	3.450	6.030	1.725	3.015
EC-135L	3.450	6.030	1.725	3.015
EC-135P	3.450	6.030	1.725	3.015
EC-135Y	3.611	6.314	1.806	3.157
F-111D-F	5.642	9.100	5.642	9.100

AIRCRAFT NAME	TRAFFIC AREA A	TRAFFIC AREA B,C,D	TRAFFIC AREA A	TRAFFIC AREA B,C,D
F-14	7.790	15.422	7.790	15.422
F-15A EAGLE	8.988	13.195	8.988	13.195
F-15B EAGLE	8.988	13.195	8.988	13.195
F-15C EAGLE	9.328	13.812	9.328	13.812
F-15D EAGLE	9.328	13.812	9.328	13.812
F-15E EAGLE	8.102	11.987	8.102	11.987
F-15E EAGLE W/CONFORMAL TANKS	8.102	11.987	8.102	11.987
F-16A FIGHTING FALCON	11.785	15.954	11.785	15.954
F-16B FIGHTING FALCON	11.785	15.954	11.785	15.954
F-16C FIGHTING FALCON	11.451	15.501	11.451	15.501
F-16D FIGHTING FALCON	11.451	15.501	11.451	15.501
F-22 RAPTOR	12.289	21.019	12.289	21.019
F-35A JOINT STRIKE FIGHTER CTOL	8.651	16.885	8.651	16.885
F-35B JOINT STRIKE FIGHTER STOVL	8.552	16.690	8.552	16.690
F-35C JOINT STRIKE FIGHTER CV	8.009	15.550	8.009	15.550
F-4C/D/E	8.721	17.362	8.721	17.362
FA-18F	7.500	12.713	7.500	12.713
FALCON 20 EW	11.743	15.897	11.743	15.897
FALCON DA-20	11.743	15.897	11.743	15.897
HARRIER GR MK3	3.622	7.103	3.622	7.103
HARRIER GR MK7-GR MK10	3.571	6.998	3.571	6.998
HC-130P/N COMBAT TANKER/COMBAT SHADOW	4.648	9.027	2.324	4.513
HERCULES C MK1	4.686	9.089	2.343	4.545

AIRCRAFT NAME	TRAFFIC AREA A	TRAFFIC AREA B,C,D	TRAFFIC AREA A	TRAFFIC AREA B,C,D
HH-60G PAVE HAWK	10.330	15.051	10.330	15.051
IL-76MD CANDID B	1.023	1.477	1.023	1.477
IL-76MF CANDID (STRETCHED)	0.973	1.405	0.973	1.405
IL-76T CANDID A	1.081	1.561	1.081	1.561
IL-76TD CANDID A	1.023	1.477	1.023	1.477
JAGUAR GR MK1-T MK2	5.475	7.280	5.475	7.280
KC-10A REFUELER	3.150	5.376	1.885	2.800
KC-46A	3.734	6.044	1.867	3.022
KC-135 REFUELER	3.477	6.139	1.739	3.069
KC-135A REFUELER	3.450	6.030	1.725	3.015
KC-135E STRATOTANKER	3.337	5.832	1.669	2.916
KC-135R/T STRATOTANKER	3.337	5.832	1.669	2.916
L-1011-500	3.577	5.437	1.788	2.718
LC-130H HERCULES	4.412	8.555	2.206	4.278
MC-130E COMBAT TALON I	4.648	9.023	2.324	4.511
MC-130H COMBAT TALON II	4.648	9.023	2.324	4.511
MC-130P COMBAT SHADOW	4.648	9.027	2.324	4.513
MD-11	3.064	5.230	1.872	2.781
MH-53J/M PAVE LOW	5.953	11.112	5.953	11.112
MHU-196 AIRFIELD	4.287	5.766	4.287	5.766
MIG-25 RUSSIAN	5.747	10.834	5.747	10.834
MIG-29 RUSSIAN	8.138	15.048	8.138	15.048
MV-22 OSPREY	4.540	8.328	4.540	8.328
OC-135B OPEN SKIES	3.477	6.077	1.738	3.039
OH-58A	16.162	20.706	8.081	10.353
OV-1	9.176	13.781	9.176	13.781
P-3C	3.556	6.615	3.556	6.615

AIRCRAFT NAME	TRAFFIC AREA A	TRAFFIC AREA B,C,D	TRAFFIC AREA A	TRAFFIC AREA B,C,D
P-3C ORION	3.580	6.661	3.580	6.661
P-3N ORION	3.580	6.661	3.580	6.661
P-8A POSEIDON	3.574	6.345	3.574	6.345
PHANTOM FG MK1	9.480	18.882	9.480	18.882
PREDATOR MQ-1	18.781	28.108	18.781	28.108
RC-135M	3.450	6.030	1.725	3.015
RC-135S COBRA BALL	3.450	6.030	1.725	3.015
RC-135U COMBAT SENT	3.450	6.030	1.725	3.015
RC-135V RIVET JOINT	3.450	6.030	1.725	3.015
RC-135W RIVET JOINT	3.450	6.030	1.725	3.015
RC-26B METROLINER	7.454	14.477	7.454	14.477
RQ-4A GLOBAL HAWK BLK 10	8.288	13.923	8.288	13.923
RQ-4B GLOBAL HAWK BLK 20	12.530	25.024	12.530	25.024
S3-B	9.244	17.937	9.244	17.937
SENTRY AEW MK1	3.550	6.292	1.775	3.146
SPACE SHUTTLE ORBITER	4.171	7.298	4.171	7.298
T-1A JAYHAWK	11.425	17.431	11.425	17.431
T-2C	13.362	26.659	13.362	26.659
T-34C	14.798	20.199	14.798	20.199
T-37B TWEET	16.574	32.341	16.574	32.341
T-38 A/C TALON	18.541	32.206	18.541	32.206
T-39A	11.634	15.205	11.634	15.205
T-43A	3.924	7.087	3.924	7.087
T-44A	9.743	18.518	9.743	18.518
T-45A	11.730	22.356	11.730	22.356
T-6A TEXAN II	24.415	34.354	24.415	34.354
TORNADO GR MK1	8.923	14.705	8.923	14.705
TORNADO GR MK2-F MK3	8.873	14.622	8.873	14.622
TRISTAR K MK1-KC	3.635	5.526	1.817	2.763

MK1				
AIRCRAFT NAME	**TRAFFIC AREA A**	**TRAFFIC AREA B,C,D**	**TRAFFIC AREA A**	**TRAFFIC AREA B,C,D**
TYPHOON (EUROFIGHTER)	10.256	19.779	10.256	19.779
U-2S	5.409	10.566	5.409	10.566
UAV SHADOW RQ-7B	21.187	37.946	21.187	37.946
UAV WARRIOR ERMD	16.202	26.071	16.202	26.071
UH-1	8.722	12.425	8.722	12.425
UH-1H/V HUEY	9.657	13.671	9.657	13.671
UH-1N TWIN HUEY	9.127	13.004	9.127	13.004
UH-46	8.009	15.105	8.009	15.105
UH-60	11.993	17.485	11.993	17.485
V-22	4.606	8.459	4.606	8.459
VC10 K MK2	3.138	5.557	1.569	2.779
VC10 K MK3	3.033	5.369	1.516	2.685
VC-25A AIR FORCE ONE	3.572	5.036	1.786	2.518
WC-130H HERCULES	4.648	9.023	2.324	4.511
WC-130J HERCULES	4.687	9.096	2.344	4.548
WC-135B	3.450	6.030	1.725	3.015
WC-135C CONSTANT PHOENIX	3.548	6.203	1.774	3.102
WC-135W CONSTANT PHOENIX	3.548	6.203	1.774	3.102

Table 6-2. Pass to Coverage Ratios for Evaluation Aircraft

(3) Radius of Relative Stiffness (L) in inches. Table 6-3 contains L values based on E= 4,000,000 psi and Mu=0.15. L is shown for 25 pci increments of K and 1 inch increments of PCC thickness. Intermediate L values can be interpolated. Table 6-4 contains L/T ratios, where T is the thickness of PCC.

(4) Evaluation Number. The Evaluation Number is a function of the L/H ratio, the flexural strength (R) of the PCC, and the Design Factor. L/T values for the L values in Table 6-3 are contained in Table 6-4. The Evaluation Number is determined from Figure 6-3.

(5) Evaluation procedure for determining the Allowable Load for plain concrete pavements. This is a 4 step

procedure:

Step 1-Determine the Design Factor from Figure 6-1 for a Standard evaluation or Figure 6-2 for an Extended Life evaluation. Note that the Pass/Coverage Ratio for each evaluation aircraft is required and is shown in Table 6-2 and on the appropriate figure for determining the Allowable Load (Figures 6-4 through 6-35).

Step 2-Determine L from Table 6-3 and the L/T ratio from Table 6-4.

Step 3. Determine the Evaluation Number from Figure 6-3.

Step 4. Determine the Allowable Load for the appropriate aircraft from Figures 6-4 through 6-35. These curves are based on Constant Tire Pressure. Previous curves were based on Constant Contact Area.

n)/	25	50	75	100	125	150	175	200	225	250	275	300	325	350	375	400	425	450	475	50
	10.81	9.09	8.21	7.64	7.23	6.91	6.64	6.43	6.24	6.08	5.93	5.81	5.69	5.59	5.49	5.40	5.32	5.25	5.18	5.1
	18.18	15.28	13.81	12.85	12.15	11.61	11.17	10.81	10.49	10.22	9.98	9.77	9.57	9.40	9.24	9.09	8.95	8.82	8.71	8.5
	24.63	20.72	18.72	17.42	16.47	15.74	15.15	14.65	14.22	13.85	13.53	13.24	12.97	12.74	12.52	12.32	12.13	11.96	11.80	11. 5
	30.57	25.70	23.23	21.61	20.44	19.53	18.79	18.18	17.65	17.19	16.78	16.42	16.10	15.80	15.53	15.28	15.05	14.84	14.64	14. 5
	36.14	30.39	27.46	25.55	24.17	23.09	22.22	21.49	20.86	20.32	19.84	19.42	19.03	18.68	18.36	18.07	17.80	17.54	17.31	17. 9
	41.43	34.84	31.48	29.30	27.71	26.47	25.47	24.63	23.92	23.30	22.75	22.26	21.82	21.42	21.05	20.72	20.40	20.11	19.84	19. 9
	46.51	39.11	35.34	32.89	31.10	29.72	28.59	27.65	26.85	26.15	25.54	24.99	24.49	24.04	23.63	23.25	22.90	22.58	22.28	21. 9
	51.41	43.23	39.06	36.35	34.38	32.85	31.60	30.57	29.68	28.91	28.23	27.62	27.07	26.58	26.12	25.70	25.32	24.96	24.62	24. 1
	56.15	47.22	42.67	39.71	37.55	35.88	34.52	33.39	32.42	31.58	30.83	30.17	29.57	29.03	28.53	28.08	27.66	27.26	26.90	26. 5
	60.77	51.10	46.18	42.97	40.64	38.83	37.36	36.14	35.09	34.17	33.37	32.65	32.01	31.42	30.88	30.39	29.93	29.50	29.11	28. 4
	65.28	54.89	49.60	46.16	43.65	41.71	40.13	38.81	37.69	36.71	35.84	35.07	34.38	33.75	33.17	32.64	32.15	31.69	31.27	30. 7
	69.68	58.59	52.94	49.27	46.60	44.52	42.84	41.43	40.23	39.18	38.26	37.44	36.69	36.02	35.41	34.84	34.31	33.83	33.37	32. 5
	73.99	62.22	56.22	52.32	49.48	47.27	45.49	43.99	42.72	41.61	40.63	39.75	38.97	38.25	37.60	36.99	36.44	35.92	35.44	34. 9
	78.22	65.77	59.43	55.31	52.31	49.98	48.09	46.51	45.16	43.98	42.95	42.02	41.19	40.44	39.74	39.11	38.52	37.97	37.46	36. 9
	82.37	69.27	62.59	58.25	55.08	52.63	50.64	48.98	47.56	46.32	45.23	44.26	43.38	42.58	41.86	41.19	40.57	39.99	39.45	38. 5
	86.46	72.70	65.69	61.13	57.82	55.24	53.15	51.41	49.92	48.62	47.47	46.45	45.53	44.70	43.93	43.23	42.58	41.97	41.41	40. 8
	90.48	76.08	68.75	63.98	60.51	57.81	55.62	53.80	52.24	50.88	49.68	48.61	47.65	46.77	45.97	45.24	44.56	43.93	43.34	42. 8
	94.44	79.41	71.76	66.78	63.16	60.34	58.06	56.15	54.53	53.11	51.86	50.74	49.74	48.82	47.99	47.22	46.51	45.85	45.23	44. 6
	98.35	82.70	74.73	69.54	65.77	62.84	60.46	58.48	56.78	55.31	54.00	52.84	51.79	50.84	49.97	49.17	48.43	47.75	47.11	46. 1
	102.21	85.94	77.66	72.27	68.35	65.30	62.84	60.77	59.01	57.47	56.12	54.91	53.83	52.84	51.93	51.10	50.33	49.62	48.95	48. 3
	106.02	89.15	80.55	74.96	70.90	67.74	65.18	63.04	61.21	59.62	58.21	56.96	55.83	54.81	53.87	53.01	52.21	51.47	50.78	50. 3
	109.78	92.31	83.41	77.63	73.41	70.14	67.49	65.28	63.38	61.73	60.28	58.98	57.81	56.75	55.78	54.89	54.06	53.30	52.58	51. 1
	113.50	95.44	86.24	80.26	75.90	72.52	69.78	67.49	65.53	63.83	62.32	60.98	59.77	58.68	57.67	56.75	55.90	55.10	54.36	53. 7
	117.18	98.54	89.04	82.86	78.36	74.87	72.04	69.68	67.66	65.90	64.35	62.96	61.71	60.58	59.54	58.59	57.71	56.89	56.13	55. 1
	120.83	101.60	91.81	85.44	80.80	77.20	74.28	71.84	69.76	67.95	66.35	64.92	63.63	62.46	61.40	60.41	59.50	58.66	57.87	57. 4
	124.43	104.64	94.55	87.99	83.21	79.51	76.50	73.99	71.84	69.97	68.33	66.86	65.53	64.33	63.23	62.22	61.28	60.41	59.60	58. 4
	128.01	107.64	97.26	90.51	85.60	81.79	78.70	76.11	73.90	71.98	70.29	68.78	67.41	66.18	65.04	64.00	63.04	62.15	61.31	60. 3
	131.54	110.62	99.95	93.02	87.97	84.05	80.87	78.22	75.95	73.97	72.23	70.68	69.28	68.01	66.84	65.77	64.78	63.86	63.01	62. 0
	135.05	113.57	102.62	95.50	90.32	86.29	83.03	80.30	77.97	75.95	74.16	72.56	71.12	69.82	68.62	67.53	66.51	65.57	64.69	63. 6
	138.53	116.49	105.26	97.96	92.64	88.51	85.17	82.37	79.98	77.90	76.07	74.43	72.96	71.62	70.39	69.27	68.22	67.26	66.35	65. 1

Table 6-3. Radius of Relative Stiffness (L) for E=4,000,000 psi and v=0.15

T(in)/ K(pci)	25	50	75	100	125	150	175	200	225	250	275	300	325	350	375	400	425	450	475	500
1	10.807	9.088	8.212	7.642	7.227	6.905	6.644	6.426	6.239	6.077	5.934	5.806	5.691	5.587	5.491	5.404	5.322	5.247	5.176	5.110
2	9.088	7.642	6.905	6.426	6.077	5.806	5.587	5.404	5.247	5.110	4.990	4.883	4.786	4.698	4.618	4.544	4.475	4.412	4.353	4.297
3	8.212	6.905	6.239	5.806	5.491	5.247	5.048	4.883	4.741	4.618	4.509	4.412	4.325	4.245	4.173	4.106	4.044	3.987	3.933	3.883
4	7.642	6.426	5.806	5.404	5.110	4.883	4.698	4.544	4.412	4.297	4.196	4.106	4.024	3.951	3.883	3.821	3.763	3.710	3.660	3.614
5	7.227	6.077	5.491	5.110	4.833	4.618	4.443	4.297	4.173	4.064	3.968	3.883	3.806	3.736	3.672	3.614	3.559	3.509	3.462	3.417
6	6.905	5.806	5.247	4.883	4.618	4.412	4.245	4.106	3.987	3.883	3.792	3.710	3.636	3.570	3.509	3.453	3.401	3.352	3.307	3.265
7	6.644	5.587	5.048	4.698	4.443	4.245	4.085	3.951	3.836	3.736	3.648	3.570	3.499	3.435	3.376	3.322	3.272	3.226	3.182	3.142
8	6.426	5.404	4.883	4.544	4.297	4.106	3.951	3.821	3.710	3.614	3.528	3.453	3.384	3.322	3.265	3.213	3.165	3.120	3.078	3.039
9	6.239	5.247	4.741	4.412	4.173	3.987	3.836	3.710	3.602	3.509	3.426	3.352	3.286	3.226	3.170	3.120	3.073	3.029	2.989	2.950
10	6.077	5.110	4.618	4.297	4.064	3.883	3.736	3.614	3.509	3.417	3.337	3.265	3.201	3.142	3.088	3.039	2.993	2.950	2.911	2.874
11	5.934	4.990	4.509	4.196	3.968	3.792	3.648	3.528	3.426	3.337	3.258	3.188	3.125	3.068	3.015	2.967	2.922	2.881	2.842	2.806
12	5.806	4.883	4.412	4.106	3.883	3.710	3.570	3.453	3.352	3.265	3.188	3.120	3.058	3.002	2.950	2.903	2.860	2.819	2.781	2.746
13	5.691	4.786	4.325	4.024	3.806	3.636	3.499	3.384	3.286	3.201	3.125	3.058	2.997	2.942	2.892	2.846	2.803	2.763	2.726	2.691
14	5.587	4.698	4.245	3.951	3.736	3.570	3.435	3.322	3.226	3.142	3.068	3.002	2.942	2.888	2.839	2.793	2.751	2.712	2.676	2.642
15	5.491	4.618	4.173	3.883	3.672	3.509	3.376	3.265	3.170	3.088	3.015	2.950	2.892	2.839	2.790	2.746	2.704	2.666	2.630	2.597
16	5.404	4.544	4.106	3.821	3.614	3.453	3.322	3.213	3.120	3.039	2.967	2.903	2.846	2.793	2.746	2.702	2.661	2.623	2.588	2.555
17	5.322	4.475	4.044	3.763	3.559	3.401	3.272	3.165	3.073	2.993	2.922	2.860	2.803	2.751	2.704	2.661	2.621	2.584	2.549	2.517
18	5.247	4.412	3.987	3.710	3.509	3.352	3.226	3.120	3.029	2.950	2.881	2.819	2.763	2.712	2.666	2.623	2.584	2.547	2.513	2.481
19	5.176	4.353	3.933	3.660	3.462	3.307	3.182	3.078	2.989	2.911	2.842	2.781	2.726	2.676	2.630	2.588	2.549	2.513	2.479	2.448
20	5.110	4.297	3.883	3.614	3.417	3.265	3.142	3.039	2.950	2.874	2.806	2.746	2.691	2.642	2.597	2.555	2.517	2.481	2.448	2.417
21	5.048	4.245	3.836	3.570	3.376	3.226	3.104	3.002	2.915	2.839	2.772	2.712	2.659	2.610	2.565	2.524	2.486	2.451	2.418	2.387
22	4.990	4.196	3.792	3.528	3.337	3.188	3.068	2.967	2.881	2.806	2.740	2.681	2.628	2.580	2.536	2.495	2.457	2.423	2.390	2.360
23	4.935	4.150	3.750	3.489	3.300	3.153	3.034	2.934	2.849	2.775	2.710	2.651	2.599	2.551	2.508	2.467	2.430	2.396	2.364	2.334
24	4.883	4.106	3.710	3.453	3.265	3.120	3.002	2.903	2.819	2.746	2.681	2.623	2.571	2.524	2.481	2.441	2.405	2.370	2.339	2.309
25	4.833	4.064	3.672	3.417	3.232	3.088	2.971	2.874	2.790	2.718	2.654	2.597	2.545	2.499	2.456	2.417	2.380	2.346	2.315	2.285
26	4.786	4.024	3.636	3.384	3.201	3.058	2.942	2.846	2.763	2.691	2.628	2.571	2.520	2.474	2.432	2.393	2.357	2.324	2.292	2.263
27	4.741	3.987	3.602	3.352	3.170	3.029	2.915	2.819	2.737	2.666	2.603	2.547	2.497	2.451	2.409	2.370	2.335	2.302	2.271	2.242
28	4.698	3.951	3.570	3.322	3.142	3.002	2.888	2.793	2.712	2.642	2.580	2.524	2.474	2.429	2.387	2.349	2.314	2.281	2.250	2.222
29	4.657	3.916	3.539	3.293	3.114	2.976	2.863	2.769	2.689	2.619	2.557	2.502	2.453	2.408	2.366	2.328	2.293	2.261	2.231	2.202
30	4.618	3.883	3.509	3.265	3.088	2.950	2.839	2.746	2.666	2.597	2.536	2.481	2.432	2.387	2.346	2.309	2.274	2.242	2.212	2.184

Table 6-4. Ratio of Radius of Relative Stiffness (L) to Thickness (T)

(6) Example for Determining Allowable Load for Plain Concrete Pavements. Determine the Allowable Load for 50,000 passes of a C-17 aircraft on an A traffic area using the Standard (first crack) evaluation procedure.

The PCC thickness is 14.0 inches, the k value immediately under the PCC surface is 200 psi/in, and the flexural strength = 600 psi.

Step 1. Determine the P/C ratio from Table 6-2 or Figure 6-30, then determine the Design Factor from Figure 6-1. From Table 6-2 or Figure 6-30, the P/C ratio for a C-17 on an A traffic area is 1.380. Enter Figure 6-1 with 50,000 passes, go horizontal to the P/C ratio (1.380), vertical to the k value (200 psi/in.), horizontal to the Design Factor of 1.66.

Step 2. Determine L from Table 6-3 and L/H from Table 6-4. For k=200 and T-14 inches, L =46.51inches. For k=200 and t=14 inches, L/T=3.32 (46.51/14).

Step 3. Determine the Evaluation Number from Figure 6-3. Enter Figure 6-3 with L/T=3.32, go horizontal to flexural strength=600 psi, vertical to DF=1.64, then horizontal to Evaluation Number =44.

Step 4. Determine the Allowable Load from Figure 6-30. Enter Figure 6-30 with EN=44, go vertical to L=46.51 inches, then horizontal to Allowable Load=480 kips.

Example for Determining Allowable Passes for Plain Concrete Pavement. Determine the Allowable Passes of a 500 kip C-17, for an Extended Life evaluation, using the pavement data in the previous example.
Given: T=14 inches, R=600 psi, k=200 psi/inch, P/C =1.38, L=46.51inches, L/T=3.32

Begin with Figure 6-30. Enter with the C-17 weight of 500 kips, go horizontal to L=46.51inches, then vertical to EN=44.

Next, enter Figure 6-3 with L/T=3.32, go horizontal to a flexural strength of 600 psi. Draw a vertical line. Then, extend a horizontal line from EN=44. The horizontal and vertical lines intersect at DF=1.64.

Enter Figure 6-2, Design Factor for Extended Life Evaluation, with a DF=1.64, go horizontal to k=200, vertical to P/C=1.38, then horizontal to approximately 400,000 passes.

d. Reinforced Concrete Pavements. The data required for the evaluation of reinforced concrete pavements and the selection of representative physical property values are essentially the same as those for plain concrete pavements presented in chapter 3, except that the percent steel is also required.

a. Reinforcing Steel. The reinforcing steel in a reinforced concrete pavement will normally be located at or above the neutral axis of the pavement section. If the steel is below the neutral axis, it would affect the determination of the flexural strength and the static modulus of elasticity in flexure. Therefore, when the reinforcing steel falls below the neutral axis in a test beam, the beam should be turned over and tested with the reinforcing steel above the neutral axis. The splitting tensile tests cannot be performed on a core of reinforced rigid pavement if any of the reinforcing steel is present in the core to be tested. It may be possible to obtain a core that contains none of the reinforcing steel, in which case the splitting tensile tests could be performed. However, if the pavement thickness is great enough, it may be possible to saw the core just below the reinforcing steel and perform the splitting tensile test on the lower, non-reinforced portion.

b. Method of Evaluation. Reinforced concrete pavements may be found on grade (single slab), as a part of an overlay system, or over stabilized layers. In either case, for Army and Air Force evaluations the thickness of the reinforced concrete pavement is converted to an equivalent thickness of plain concrete pavement, and the evaluation is made in the same manner as plain concrete.

(1) The first step in the evaluation of an Army or Air Force reinforced concrete pavement is to compute the thickness of a plain concrete pavement (equivalent thickness) having the same load-carrying capacity as the reinforced concrete pavement. This equivalent thickness h_E is determined from Figure 6-36, using the known thickness of the reinforced concrete pavement h_r and the percentage of steel reinforcement S per foot of pavement cross-sectional area. The percentage of steel is computed from equation 6-2:

$$S = \frac{A_s}{A_p} * 100 \qquad \text{(Eq 6-2)}$$

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

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

It is necessary to compute the percent steel in both the longitudinal and transverse directions. Normally it will be the same in both directions, but if there is a difference, the smaller value will be used. Next, enter Figure 6-36 with the known value of $h_{r,,}$ thickness of the reinforced slab. Make a vertical projection and extend it until it intersects the diagonal line representing the computed value of S. Then make a horizontal projection to the left until it intersects the scale line representing the values of h_E. The resulting value of h_E represents the equivalent thickness of the plain concrete pavement that would have the same load-carrying capacity as the reinforced concrete pavement.

(2) In determining the equivalent thickness from Figure 6-36, the effects of the reinforcing steel on the load-carrying capacity will be disregarded when S is less than 0.05 and h_E will simply equal h_r. Also, when S is greater than 0.5, the value of h_E will be determined using the diagonal line representing S = 0.5 percent.

(3) After the equivalent thickness has been determined, the method of evaluation will depend on whether the reinforced concrete pavement is on grade, in any overlay system, or over a stabilized layer. For reinforced concrete pavement on grade, the method of evaluation will be the same as for a plain concrete pavement except that the h_E value will be used instead of the reinforced concrete pavement thickness h_r . If the reinforced concrete pavement is part of an overlay system, the method of evaluation to be used will depend on the type of overlay system. If the reinforced concrete pavement is placed over a stabilized layer, it will be necessary to determine the equivalent thickness of plain concrete pavement to account for the effect of the stabilized layer. First, the equivalent thickness due to the reinforcing will be determined from Figure 6-36. Second, using the above equivalent thickness, the effect of the stabilized layer will be determined from equation 6-1. Using this thickness, $h_{E,}$ the evaluation will be determined as for plain concrete pavement. In any case, the thickness to be used will be the appropriate equivalent thickness, h_E, rather than the thickness

of the reinforced concrete pavement, h_r.

c. Evaluation Example for Reinforced Concrete Pavement. Assume:

(1) Runway interior = type C traffic area.

(2) Thickness of reinforced concrete pavement = 12 inches.

(3) Diameter of steel reinforcing bars, both longitudinal and transverse = 3/8 inch.

(4) Center-to-center spacing of reinforcing bars, both longitudinal and transverse = 6 inches.

(5) Flexural strength of concrete = 700 psi.

(6)The k value for the foundation material = 100 pci.

(7)The percentage of reinforcing steel in both the longitudinal and transverse directions is computed by substituting in equation 6-2:

$$S = \frac{A_s}{A_p} x\,100 = \frac{0.221}{144} x\,100 = 0.00153\ x\,100 = 0.153\ percent$$

where

$$A_s = \frac{(3.1416)(0.375\ )^2\ x\,(2)}{4} = 0.221\,square\,inches$$

$$A_p = 12\ x\,12 = 144\ square\ inches$$

Since h_r = 12 inches and S = 0.153 percent, Figure 6-36 shows the corresponding h_E value to be 14.4 inches. This h_E value is then used to determine the evaluation in the same manner as a plain concrete pavement.

6-7 **RIGID OVERLAY ON RIGID PAVEMENT.**

a.Data Required. The data required for the evaluation of a rigid overlay on rigid pavement does not differ greatly from those required for plain concrete pavements. The data needed for use with the evaluation curves are presented in Chapter 3. A study of the overlay design, construction records, and previous condition surveys must be made to determine the condition of the base pavement prior to the overlay. If the overlay pavement contains only a minimum of structural defects, then it can be assumed that very little "breakup" of the base pavement has occurred since it was overlaid, and the condition of the base pavement can be rated the same as it was immediately prior to the overlay. Methods for conducting the necessary tests are outlined in Appendix B.

b.Method of Evaluation. The first step in the evaluation of a rigid overlay(s) on a rigid pavement is the determination of the equivalent thickness of the combined pavement structure of the rigid overlay(s) and the rigid base pavement. The equivalent thickness is defined as a single thickness of plain concrete pavement having the same load-carrying capacity as the combined thickness of the rigid overlay(s) and the rigid base pavement. Start at the bottom of the structure and determine the equivalent thickness of the base pavement and overlay. If more than one overlay, use that equivalent thickness and the next overlay to determine the combined equivalent thickness. Continue this procedure with any remaining overlays. Use the average thickness of each layer in the Section structure to determine the equivalent thickness.

(1) Partial Bonded Overlay. If the overlay slab was cast directly on the base slab and no effort was made to break the bond between the overlay and the base pavement by means of a tack coat, sand, paper, bituminous concrete, or other materials placed between the overlay and the base pavement, then the equivalent thickness h_E of the combined overlay section can be computed from the following equation for partial bond between the overlay and the base pavement:

$$h_E = \sqrt[1.4]{(h_o)^{1.4} + Cr\,(h_b)^{1.4}} \qquad \text{(Eq 6-3)}$$

where

h_o = thickness of rigid overlay pavement, inches

C_r = coefficient representing condition of rigid base pavement

h_b = thickness of rigid base pavement, inches

(2) Unbonded Overlay.If a bond-breaker course was used between the rigid overlay and the rigid base pavement, the h_E value of the combined overlay section can be computed from the following equation for no bond between the overlay and the base pavement:

$$h_E = \sqrt{(h_o)^2 + Cr\,(h_b)^2} \qquad \text{(Eq 6-4)}$$

No credit is given to the thickness of the bond breaker if less than 4 inches. If the thickness of the bond breaker is greater than 4 inches, then the pavement will be evaluated as a composite pavement.

(a)The value of C_r in equations 6-3 and 6-4 depends on the condition of the existing rigid base pavement. The following C_r values are recommended, and are required in most contingency evaluations, where it is not possible to visually determine the condition of the existing base pavement.

C_r= 1.00 for base pavement in very good condition. There are no structural or reflective cracks in the rigid overlay. If the condition of the base pavement cannot be determined or is unknown, do not use this value.

C_r = 0.75 for base pavement in good condition. There are a few initial cracks due to loading, but no progressive cracks.

C_r = 0.35 for badly cracked base pavement. Approximately sixty percent of the slabs in the overlay contain Medium or High severity cracking or 50 percent of the slabs contain High severity cracks.

C_r values can also be determined using Figure 6-37, which is based on the Structural Condition Index. Both criteria are used in PCASE.

(b) After the h_E value of the combined section has been determined from equation 6-3 or 6-4, the method of evaluating a rigid overlay on a rigid base pavement is the same as for a plain concrete pavement. The flexural strength (R) to use for a Section Is the weighted average of the overlay and base pavement strengths, determined as follows:

$$R = \frac{h_o (R_o) + h_b (R_b)}{h_o + h_b} \qquad \text{(Eq 6-5)}$$

where

h_o = thickness of overlay

R_o = flexural strength of overlay

h_b = thickness of base slab

R_b = flexural strength of base slab

6-8. **NONRIGID OVERLAY ON RIGID PAVEMENTS**.

a. Data Required. The data required for the evaluation of a nonrigid overlay on rigid pavement are presented in Chapter 3. It is also necessary to determine the quality and strength of the nonrigid overlay material.

(1) For bituminous concrete overlays which consist of bituminous concrete for full depth, the data required will be the same as for the evaluation of the bituminous concrete portion of flexible pavements.

(2) For flexible overlays consisting of a granular base and a bituminous surface, the data required will be the same as for the evaluation of flexible pavements.

(3) The method of evaluation for nonrigid-type overlay pavements presented herein assumes that the bituminous concrete meets the design requirements set forth in

UFC 3-260-02, Pavement Design for Airfields, and that the base-course material of the overlay, if any, has a CBR of 80 or greater. Therefore, tests on the nonrigid overlay materials may be necessary to determine whether they meet design requirements. These tests should be made in accordance with concepts and procedures set forth in Chapter 3. Often the quality of the overlay materials can be determined from a study of construction records. If it can be ascertained that the overlay materials met design requirements during construction and there has been no deterioration of the overlay under traffic, the overlay materials may be assumed to be satisfactory, and no testing other than gradation of materials is required. When it is determined that the overlay materials (bituminous concrete or base-course materials) did not meet design requirements, the narrative portion of the evaluation report should discuss the consequences, such as rutting and raveling. Inadequacies of the nonrigid overlay can often be determined from surface conditions. Rutting or surface cracking are sometimes signs of inadequate strengths of the bituminous concrete and base course and should be investigated. However, in the case of thin overlays, care must be taken to determine whether surface cracking is the result of inadequate strength in the overlay or reflective cracking from joints and structural defects in the rigid base pavement.

b. Methods of Evaluation. The methods of evaluation for nonrigid overlay on rigid pavement are presented below. One method, designated as rigid pavement overlay evaluation, uses evaluation curves for plain concrete pavements discussed in this chapter. The other method, designated as flexible pavement evaluation, uses the flexible pavement evaluation curves presented in Chapter 5. Normally, the rigid overlay evaluation method yields the higher allowable gross weights at a selected pass level for these types of pavements and will be used. However, when the flexural strength of the rigid base pavement is less than 400 psi or the k value of the foundation is greater than 200 pci, the flexible pavement evaluation method will sometimes yield the higher allowable gross weight at a selected pass level, in which case this method should be used. Therefore, when the test results indicate that the flexural strength of the rigid base pavement is less than 400 psi or the k value is greater than 200 pci, it will be necessary to evaluate the nonrigid overlay on rigid pavement by both methods to determine which yields the higher allowable gross weight for a selected pass level.

(1) Rigid Pavement Evaluation Method. The first step in evaluating a nonrigid overlay using the rigid pavement evaluation method is to determine the equivalent thickness of the combined overlay section. The equivalent thickness, h_E , is defined as the thickness of a plain concrete pavement having the same load-carrying capacity as the combined overlay section and can be determined by the following equation:

$$h_E = \frac{1}{F}(0.33\,t + C_b\,h_b) \qquad \text{(Eq 6-6)}$$

Where:

t = thickness of nonrigid overlay pavement, inches

h_b = thickness of rigid base pavement, inches

C_b=Coefficient representing the condition of the rigid base.

F = a factor which controls the degree of cracking in the rigid base pavement, (Figure 6-39).

(a)The factor F in equation 6-6 is related to the controlled cracking in the rigid base pavement during the life of the pavement and is therefore dependent on the modulus of subgrade or base-course reaction k (measured or computed directly under the pavement) and traffic intensity in terms of passes. If a k value greater than 500 pci is established, the F value for a k of 500 pci should be used in computing the h_E value. For certain values of F, the equation will yield h_E greater than the combined thickness of h_b + t. When this occurs, use the value of h_b + t for h_E.

(b) For an evaluation, the equivalent thickness computed by means of equation 6-6, the concrete flexural strength, and modulus of subgrade or base-course reaction are used in conjunction with Figures 6-1 through 6-35 to determine the allowable gross weight at selected pass levels or the allowable number of passes for selected loads. However, determining allowable passes becomes an iterative procedure, since the F factor depends upon the traffic level.

Values for C_b range from 0.5 to 1.0, depending on condition of the base slab.
Since the base pavement has been overlaid, the condition of the base pavement is not normally known. C_b values can be determined from Figure 6-38 or the following can be used as guidance:

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

(2) Flexible Pavement Evaluation Method. The flexible pavement evaluation method considers the nonrigid overlay on rigid pavement to be a flexible pavement, with the rigid base pavement assumed to be a high-quality base course with a CBR of 100. The nonrigid overlay on rigid pavement is evaluated as a flexible pavement using the procedures presented in Chapter 5. Thus, when evaluating by the flexible pavement evaluation method, it will be necessary to determine the physical properties that are required for flexible pavement evaluations; that is, the quality of the asphaltic concrete portion of the overlay will have to be established, as well as the CBR values of the subgrade and base course beneath the rigid base pavement. As mentioned above, the rigid base pavement will be assumed to have a CBR of 100.

c. Evaluation Example. Perform a standard evaluation on a primary traffic area (A traffic area) pavement having a uniform thickness of a nonrigid overlay on a plain concrete pavement. The asphalt surface is in Excellent condition, with no reflective distresses. Condition of the base pavement is known to be good. The evaluation is to be accomplished for 100,000 passes of the F-14 aircraft. The pavement consists of a 4- inch bituminous overlay, a 6-inch plain concrete base pavement with a 650-psi flexural strength, and a subgrade with a modulus of subgrade reaction of 300 pci and a CBR of 20. The pavement will be evaluated using both the flexible and rigid evaluation methods.

(1) Rigid pavement evaluation. The following steps are followed:

(a) -From table 6-2, the P/C ratio for the F-14 operating on a rigid pavement A Traffic Area is 7.79.

-From figure 6-39, use 100,000 passes, P/C=7.79, and k=300 to determine F to be 0.81.

(b)Calculate the equivalent thickness by substituting in equation 6-6:

$$h_E = \frac{1}{0.81}[0.33(4) + 6] = 9.0 \text{ inches}$$

(d) Having determined the equivalent pavement thickness, the remainder of the evaluation will be accomplished in the same manner as a plain concrete thickness using the equivalent thickness as the existing thickness. Therefore, from Table 6-3, L=30.17 and from Table 6-4, L/H=3.35. From Table 6-2, P/C=7.790 and from Figure 6-1, DF=1.42. From Figure 6-3 EN=53 and from Figure 6-24 the Allowable Load =approximately 43 kips.

(3) Flexible pavement evaluation. The flexible pavement evaluation is conducted by considering the concrete pavement as a high-quality base course. It is then evaluated by considering the pavement as 4 inches of asphalt concrete, 6 inches

of 100 CBR base course, and a 20 CBR subgrade. From Table 5-4, the Limiting Stress for 100,000 passes of the F-14 on an A traffic area is 2.76 psi. Therefore, the Limiting Stress on the subgrade is 2.76 x 20=55.2 psi and the Limiting Stress on the base is 2.76 x 100=276 psi. Using Figure 5-21, the AGL for the subgrade is approximately 43 kips and the Allowable Load for the base is more than the maximum weight of the F-14.

(4) Controlling evaluation. In evaluating flexible overlays on rigid pavements, the larger of the controlling loads for the flexible (43 kips) and rigid (43 kips) evaluation controls the overall pavement evaluation. Since these evaluations produce the same results, the allowable load is 43 kips.

d. Determination of Additional Overlay Thickness. The following equation is used to determine the additional overlay thickness required to support aircraft operations.

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

Where:

t_{ao}=additional overlay required $_{E}$=equivalent thickness of existing base and overlay

F = a factor which controls the degree of cracking in the rigid base pavement,
See paragraph 6-8.6(1)(a) above. F factors can be determined from Figure 6-39.

6-9 **RIGID OVERLAY ON FLEXIBLE PAVEMENT**.

a. Data Required. When evaluating rigid overlay on flexible pavement, the flexible pavement (bituminous concrete, base course, and subbase course) is considered to be a base course for the rigid overlay. The data needed for use with the evaluation curves are presented in chapter 3. In the determination of the k value on the surface of the flexible pavement with the plate-bearing test, the following limitations are imposed:

(1) In no case will a k value greater than 500 pci be used.

(2) When the temperature of the existing bituminous pavement surface is above 75 degrees Fahrenheit, the asphaltic concrete pavement should be cut out and the test run on the base. When the temperature of the existing bituminous pavement surface is below 75 degrees Fahrenheit, run the tests on the asphaltic concrete pavement. Compare the value from the test with the value from Figures 6-41 through 6-44, then, select the smallest value to use. These figures may also be used as an alternative method for determining the k value on the flexible pavement. As noted earlier, each layer in the pavement structure must be examined in order to determine the layer controlling the effective k.

b. Method of Evaluation. Representative values must be selected for thickness of the rigid overlay, flexural strength of the rigid overlay, and modulus of reaction on the surface of the existing flexible pavement. The method of evaluating a rigid overlay on flexible pavement is the same as that used for a plain concrete pavement on a base course.

6-10 **COMPOSITE PAVEMENT**.

a. Data Required. The data required for the evaluation of a composite pavement are presented in Chapter 3 and depend, as does the method of evaluation, on the thickness of the nonrigid material between the two rigid pavements. When the thickness of the nonrigid material is less than 4 inches, the specific data required are equivalent thickness of the combined overlay section, flexural strength of the rigid overlay, and the k value of the foundation materials beneath the rigid base pavement. When the thickness of the nonrigid material between the rigid pavements is 4 inches or greater, the specific data required are thickness of the rigid overlay, flexural strength of the rigid overlay, and the k value on the surface of the nonrigid material beneath the rigid overlay.

(1) In the determination of the k value in a plate-bearing test on the surface of the nonrigid material between the rigid base and the rigid overlay pavement, the limitations imposed are the same as those on flexible pavement.

(2) Tests for the determination of the strength of the rigid base pavement are not required; however, the condition of the rigid base pavement must be known if the evaluation of the composite pavement is made using equation 6-6 to determine h_E. The condition of the base pavement must, of necessity, be determined from a study of previous design and construction records, previous condition surveys, and performance records of the pavements. If the rigid overlay pavement contains a minimum amount of structural defects, it can be assumed that the rigid base pavement has experienced little breakup since the overlay was placed, and the condition of the base pavement can be rated the same as it was immediately prior to the overlay.

b. Method of Evaluation. The two methods of evaluating a composite pavement, depending on the thickness of the nonrigid material between the rigid base pavement and the rigid overlay, are discussed below.

(1) If the thickness of the nonrigid material between the rigid base pavement and the rigid overlay is less than 4 inches, the composite pavement will be evaluated in the same manner as a rigid overlay on a rigid pavement, with the thickness of the nonrigid material assumed to be a bond-breaking course. The equivalent

thickness of the combined overlay section will be computed from equation 6-3 for partial bond between the overlay and the base pavement.

(2) If the thickness of the nonrigid material between the rigid base pavement and the rigid overlay is 4 inches or more, the composite pavement is evaluated in the same manner as a plain concrete pavement, with the nonrigid material and the rigid base pavement, assumed to be a base course. In the evaluation, the thickness of the rigid overlay and the concrete flexural strength of the rigid overlay will be used. The k value will be determined by a test performed on the surface of the existing nonrigid material.

6-11 **PAVEMENT CLASSIFICATION NUMBER**. In addition to evaluating airfield pavements for allowable loads or passes, it is necessary to report weight-bearing capacity of pavements in terms of the Pavement Classification Number. The PCN can then be compared with an ACN to determine if a pavement can support a particular pavement. The PCN is presented in Chapter 8.

6-12 **EVALUATIONS FOR FROST CONDITIONS**. If the existing soil, water, and temperature conditions are conducive to detrimental frost effects in the base or subgrade materials, the pavement evaluation will be based on criteria for seasonal frost areas, as given in Chapter 7 of this manual.

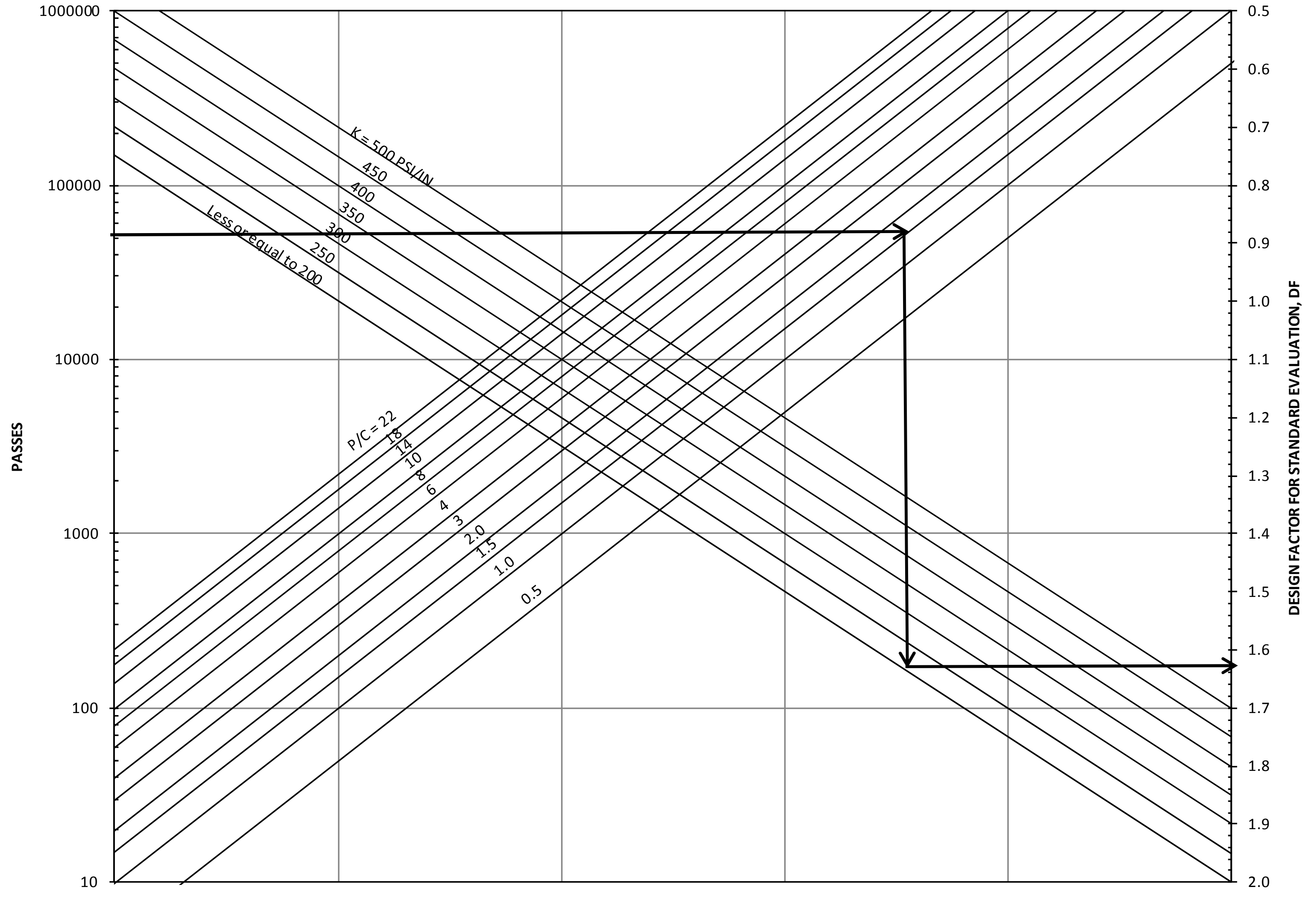


Figure 6-1. Design Factor for Standard Evaluation

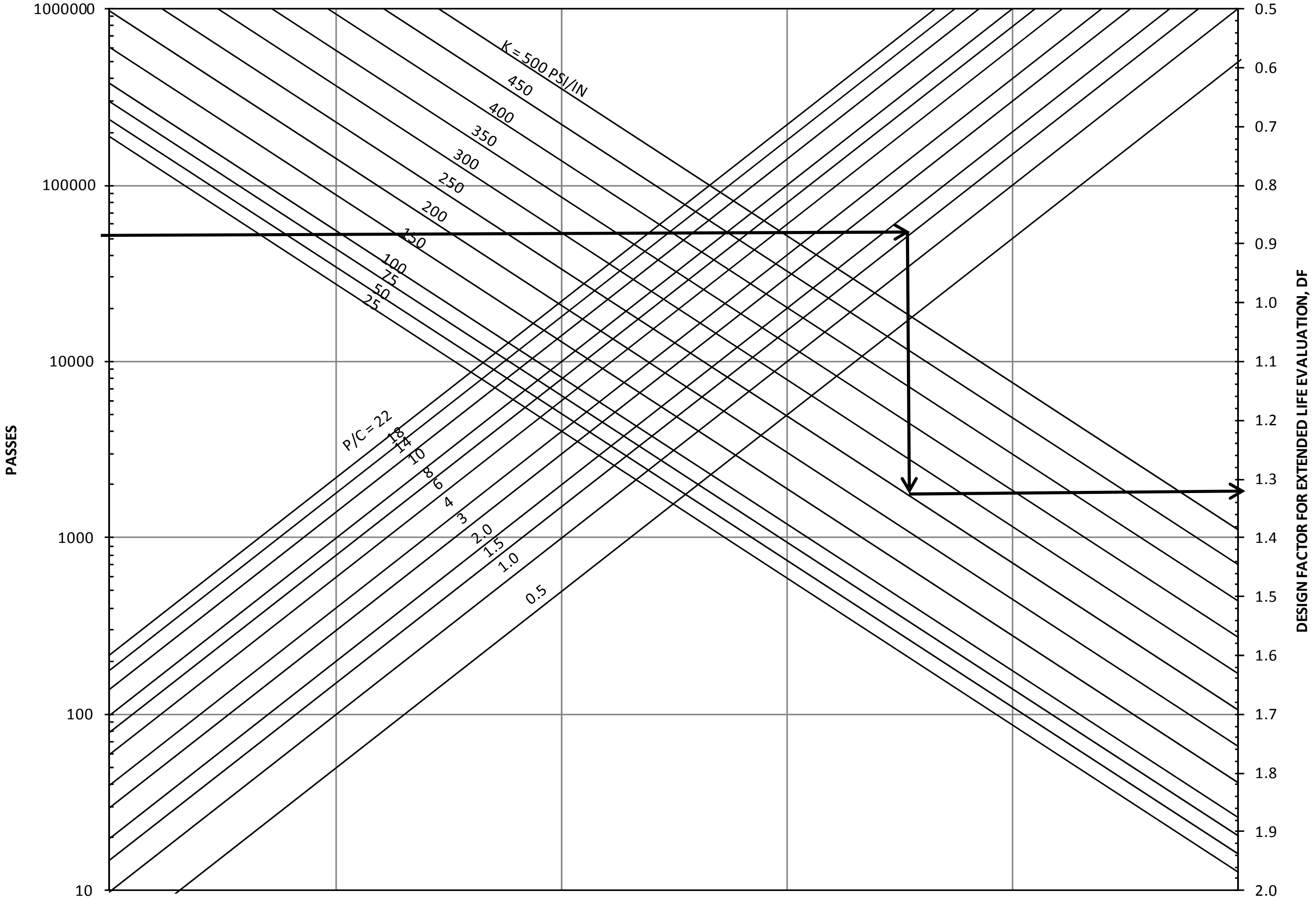


Figure 6-2. Design Factor for Extended Life Evaluation

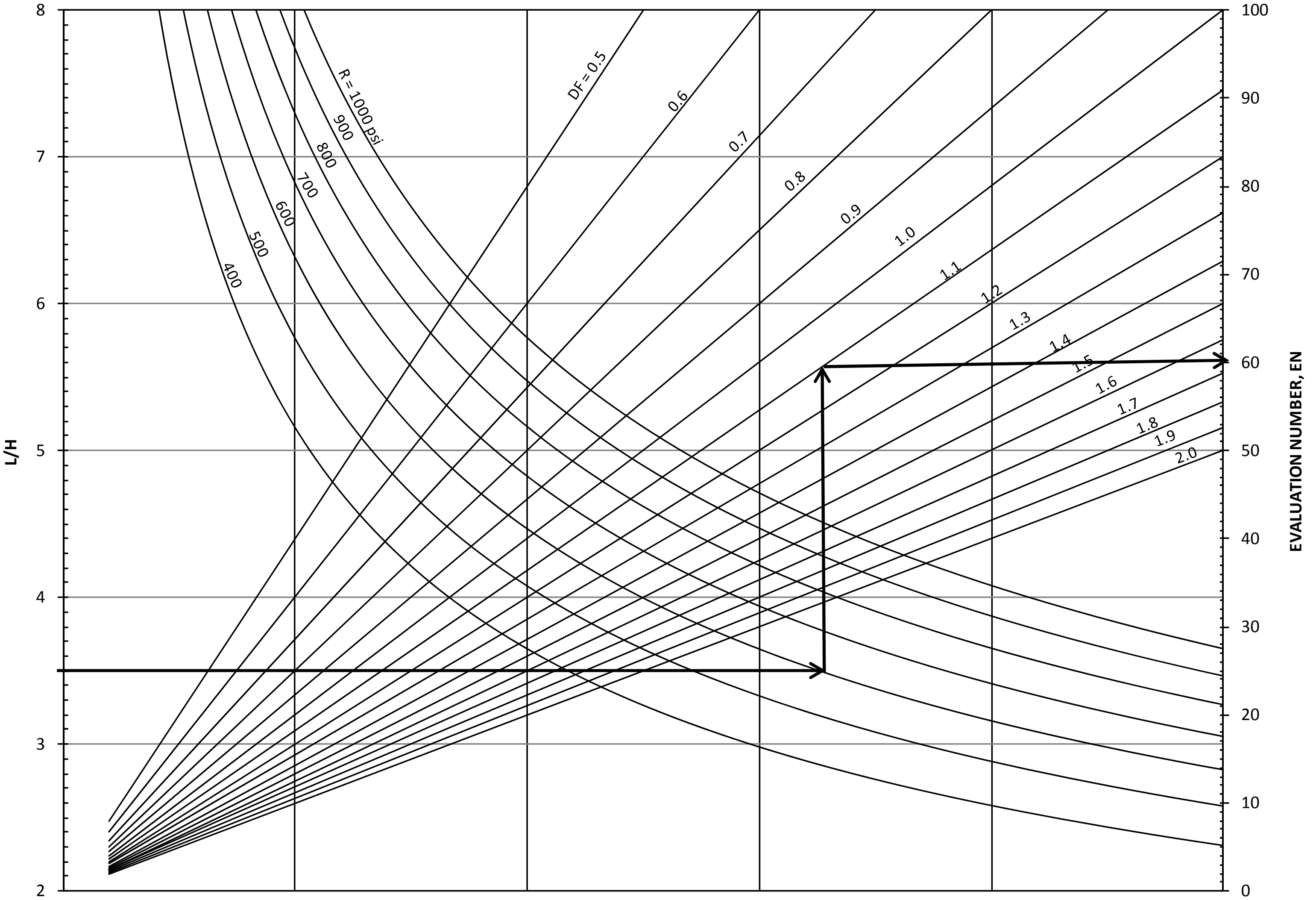


Figure 6-3. Evaluation Number for Rigid Pavement Evaluation

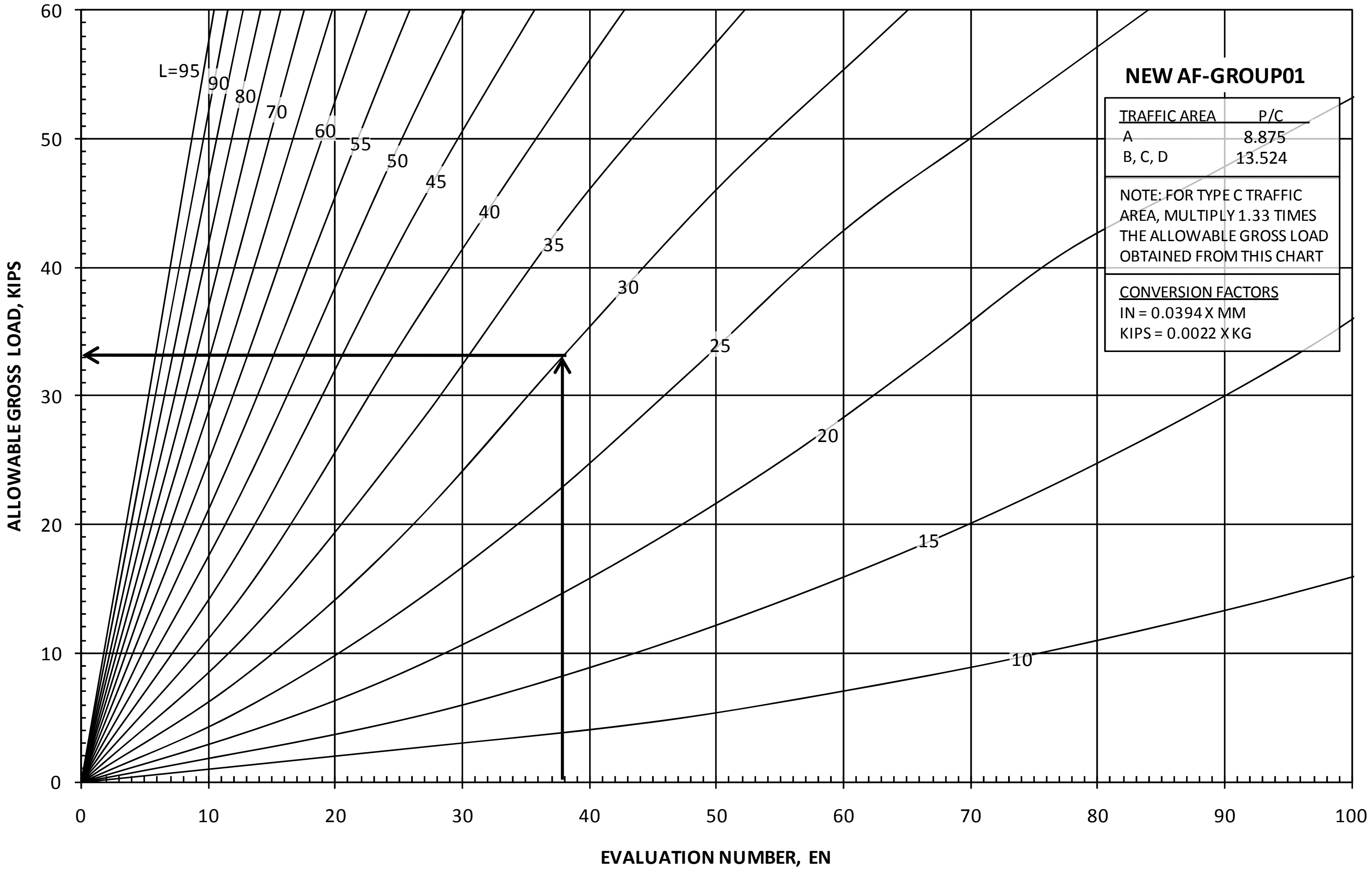


Figure 6-4. Rigid Pavement Evaluation Curve for Air Force Group 1

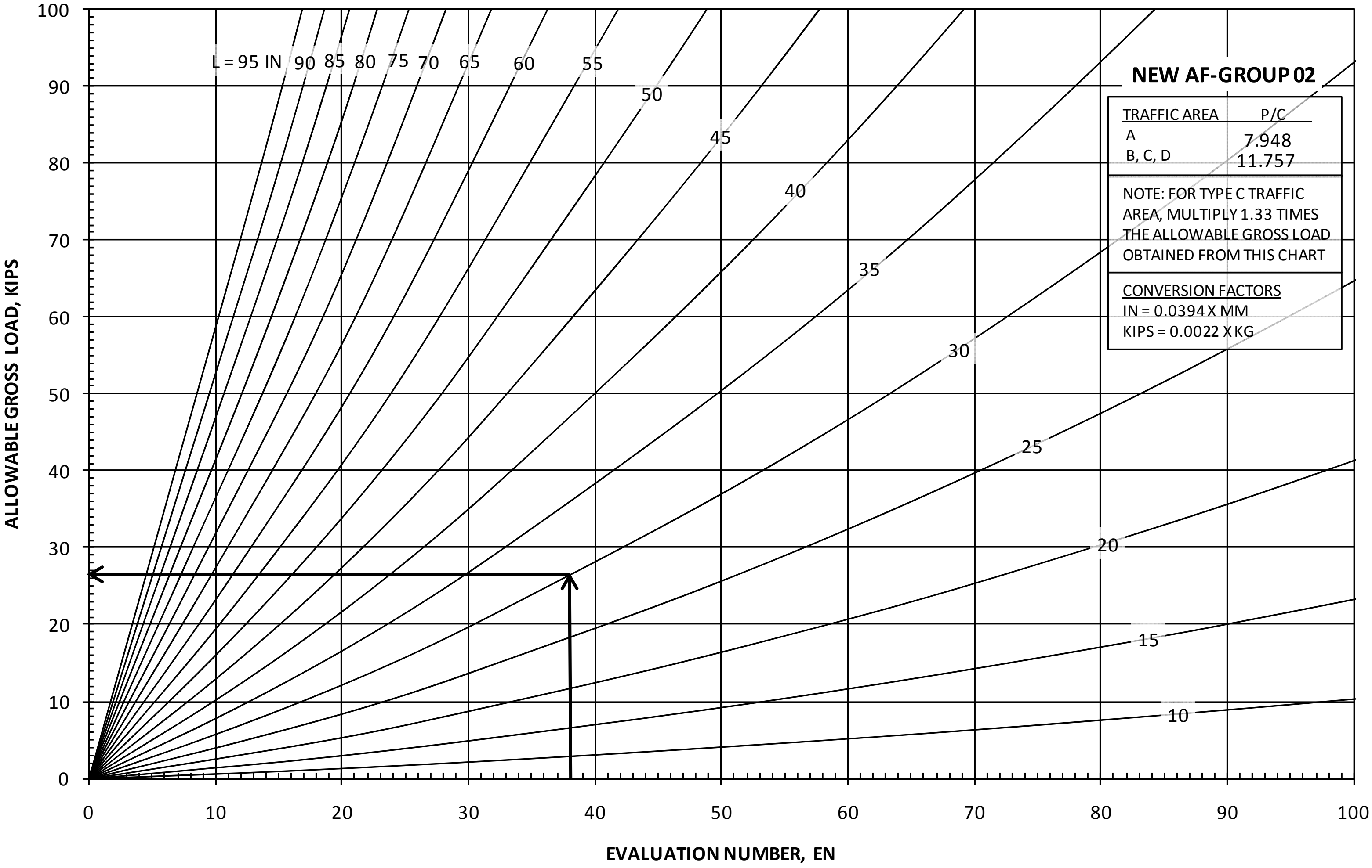


Figure 6-5. Rigid Pavement Evaluation Curve for Air Force Group 2

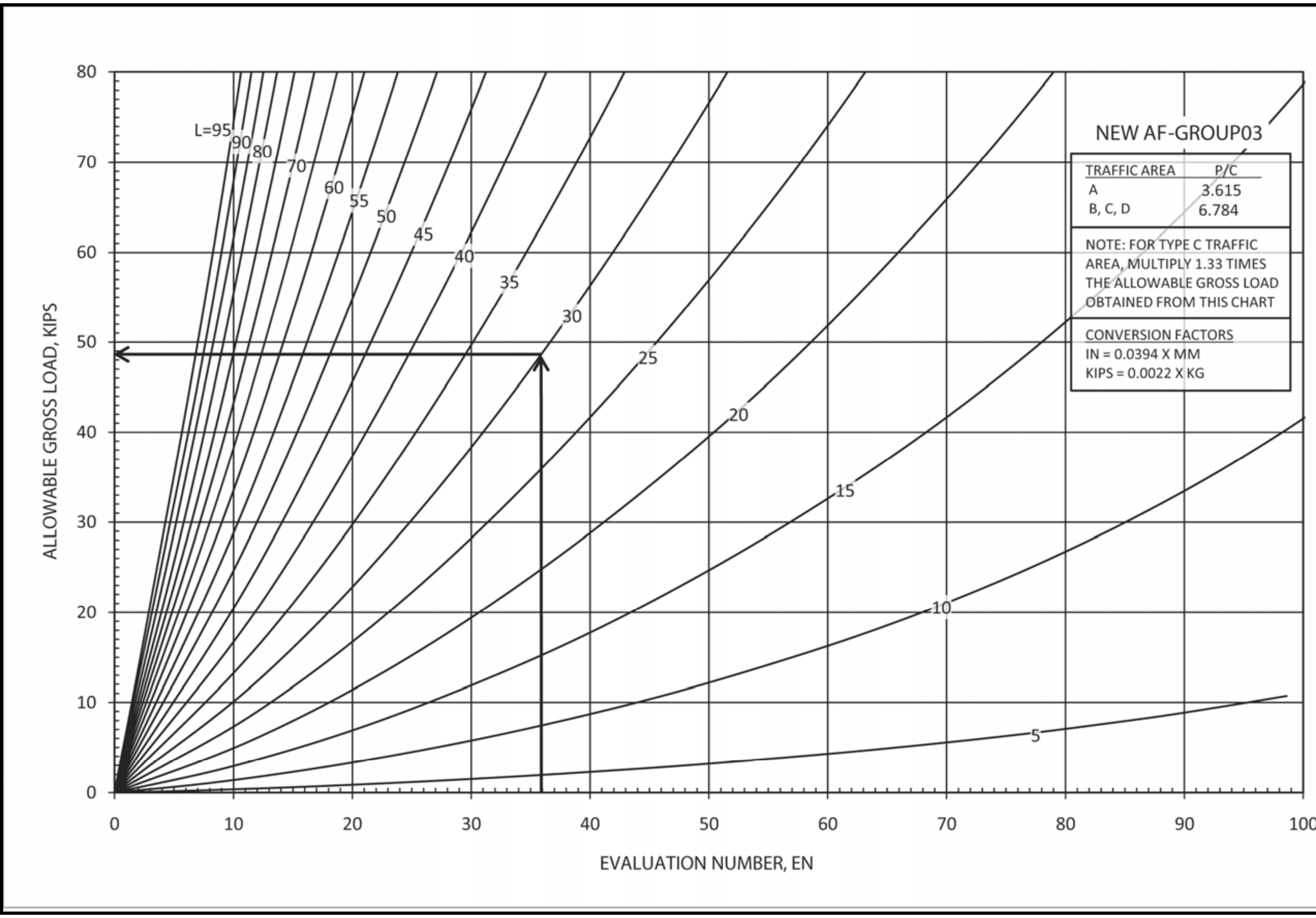


Figure 6-6. Rigid Pavement Evaluation Curve for Air Force Group 3

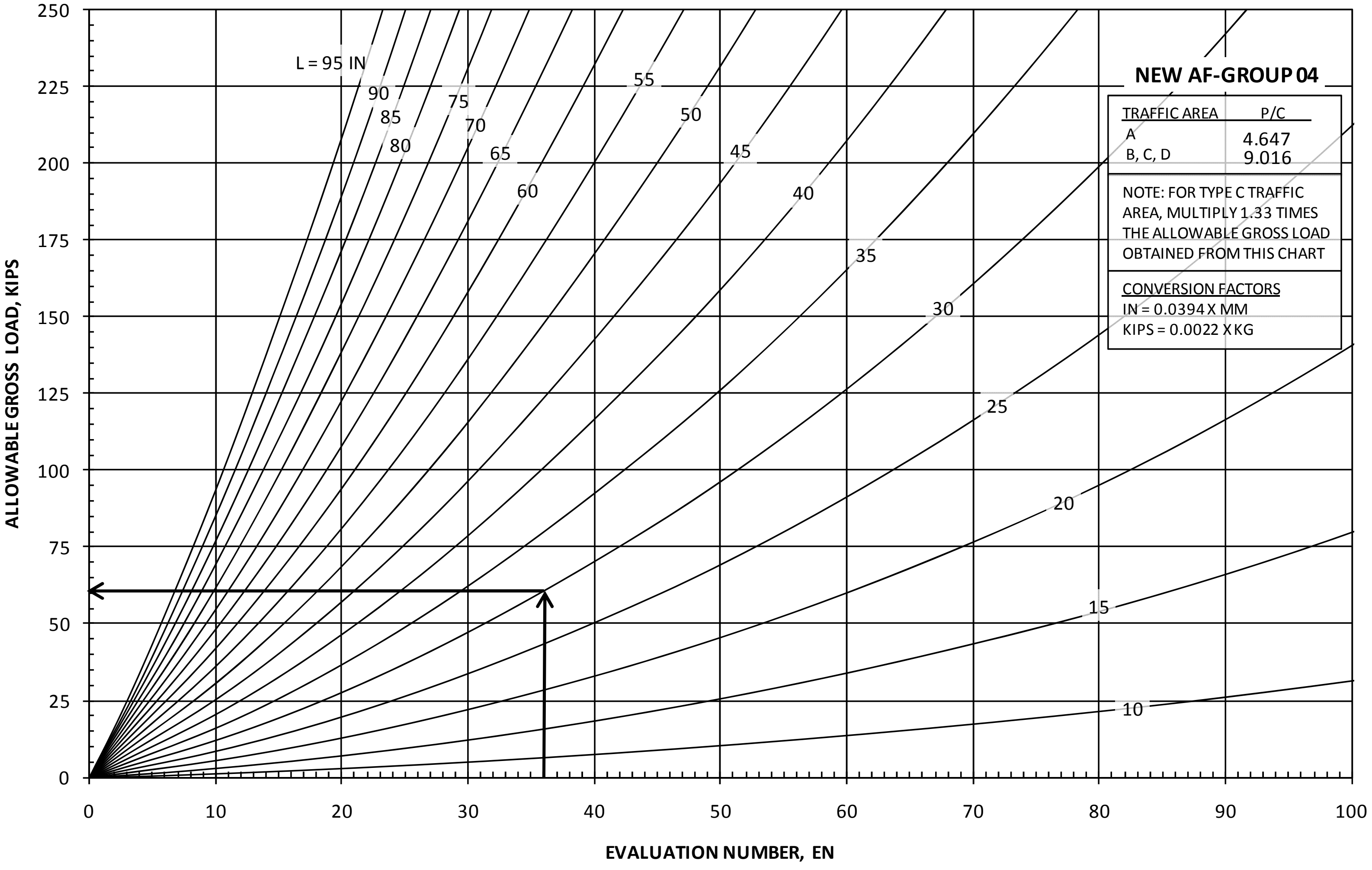


Figure 6-7. Rigid Pavement Evaluation Curve for Air Force Group 4

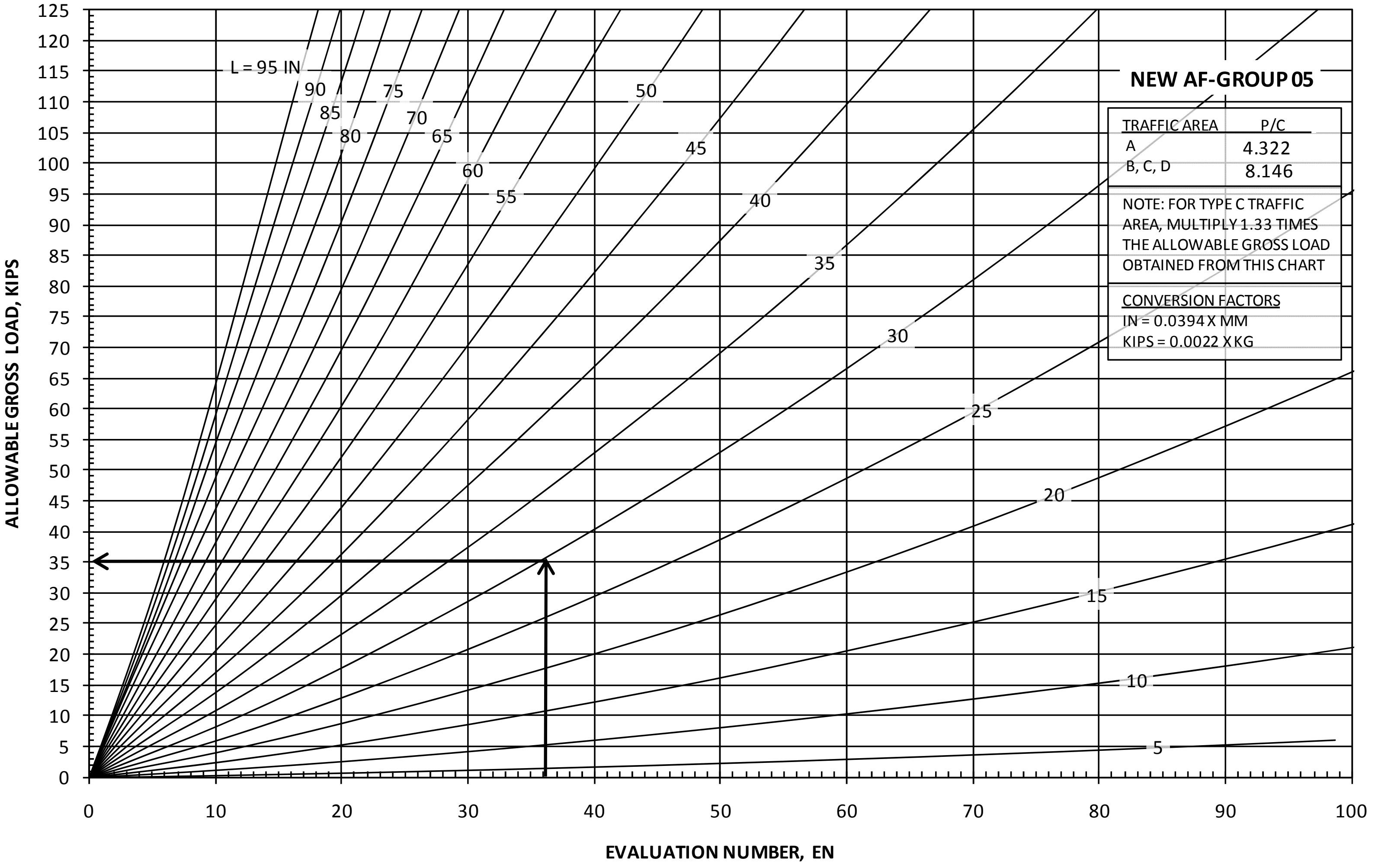


Figure 6-8. Rigid Pavement Evaluation Curve for Air Force Group 5

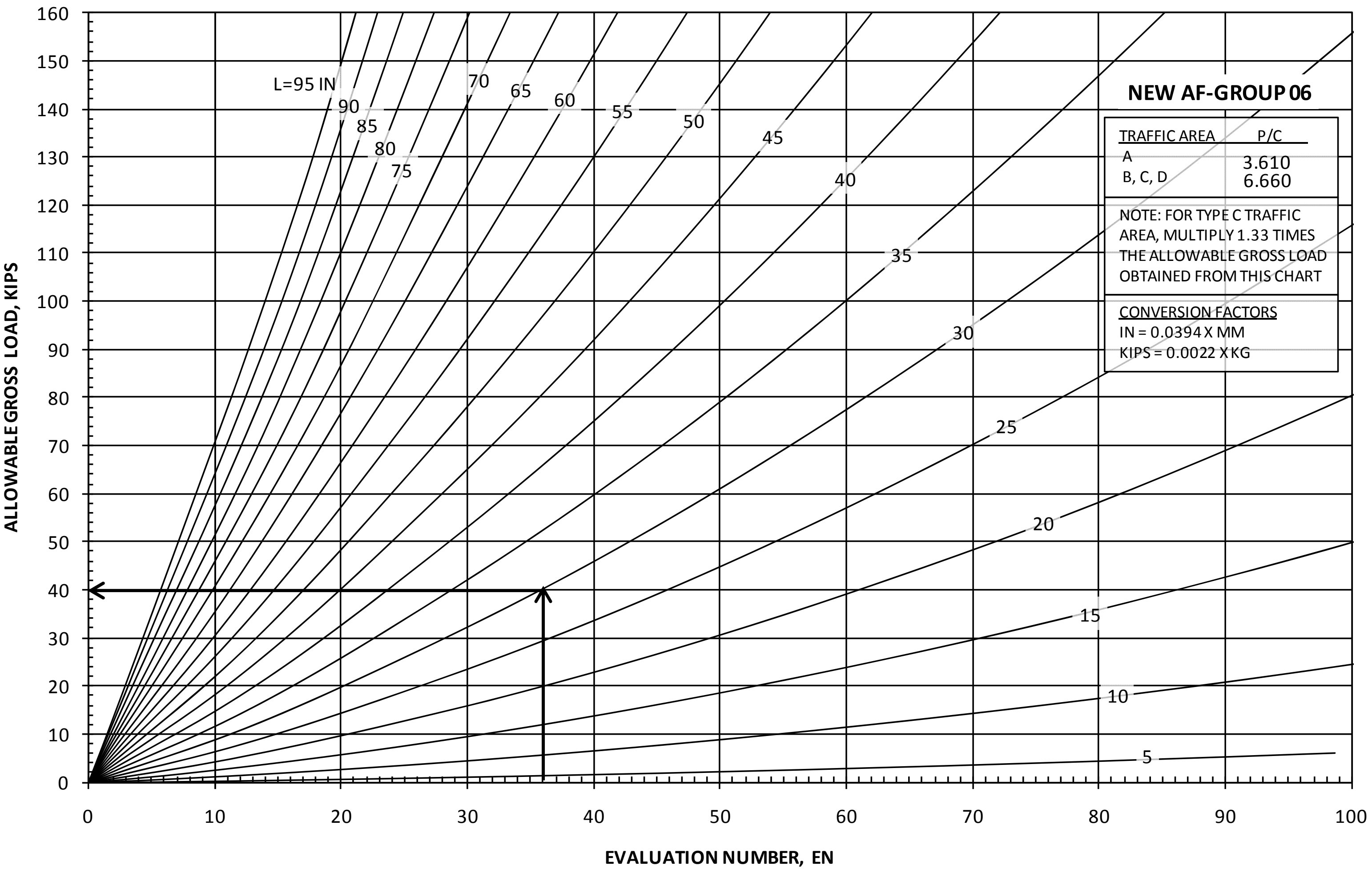
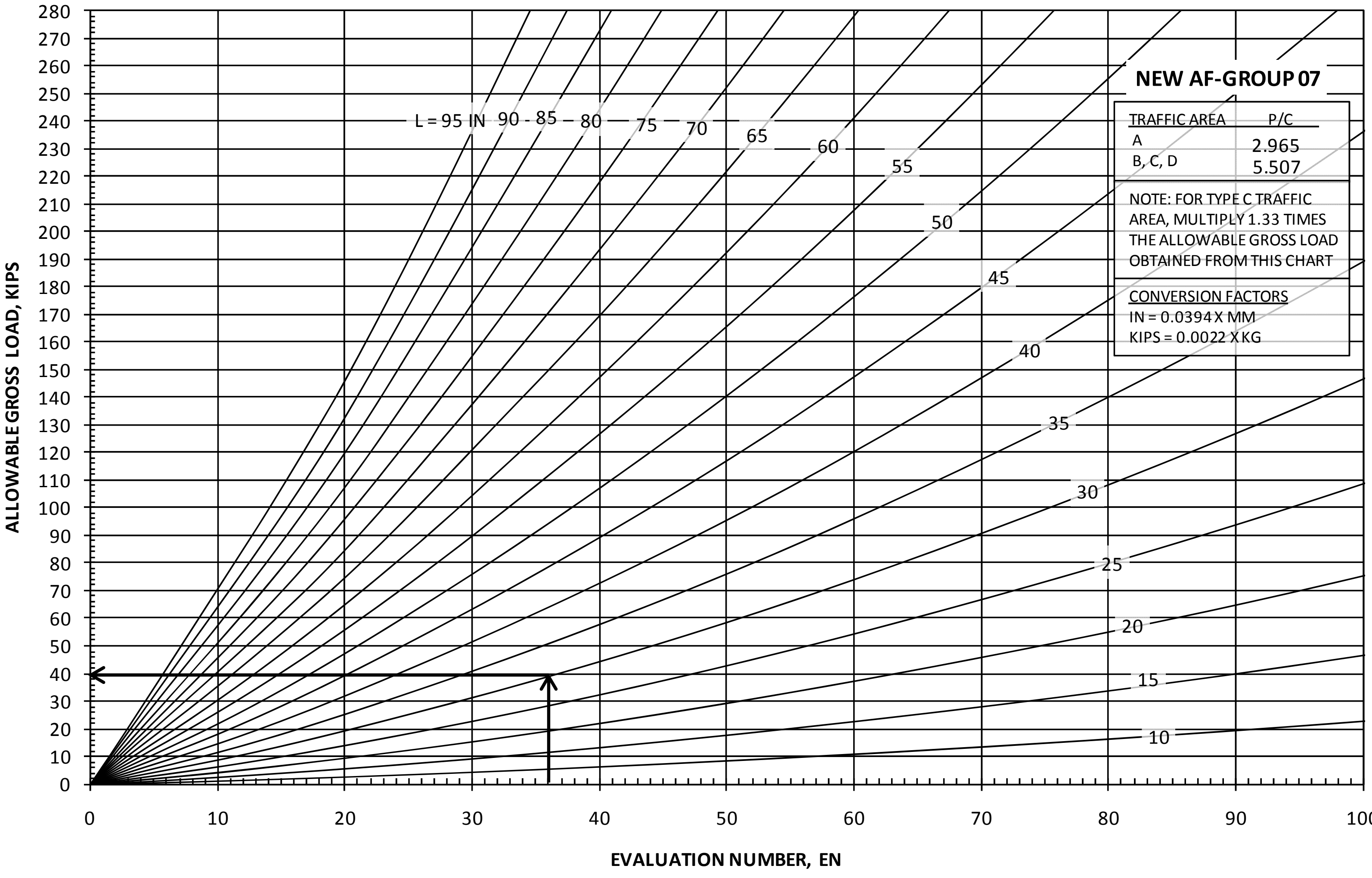


Figure 6-9. Rigid Pavement Evaluation Curve for Air Force Group 6



TRAFFIC AREA	P/C
A	2.965
B, C, D	5.507

NOTE: FOR TYPE C TRAFFIC AREA, MULTIPLY 1.33 TIMES THE ALLOWABLE GROSS LOAD OBTAINED FROM THIS CHART

CONVERSION FACTORS
IN = 0.0394 X MM
KIPS = 0.0022 X KG

Figure 6-10. Rigid Pavement Evaluation Curve for Air Force Group 7

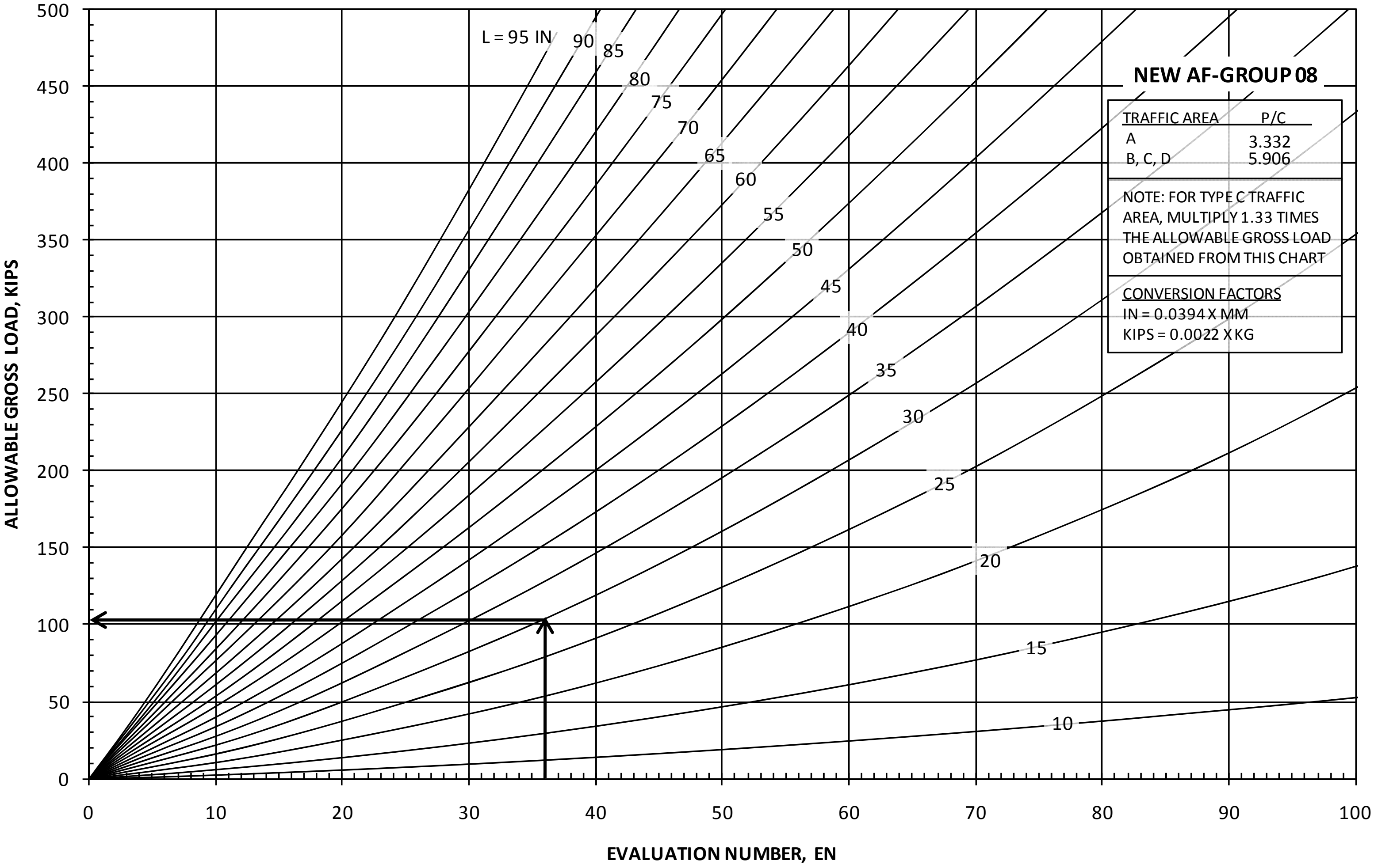


Figure 6-11. Rigid Pavement Evaluation Curve for Air Force Group 8

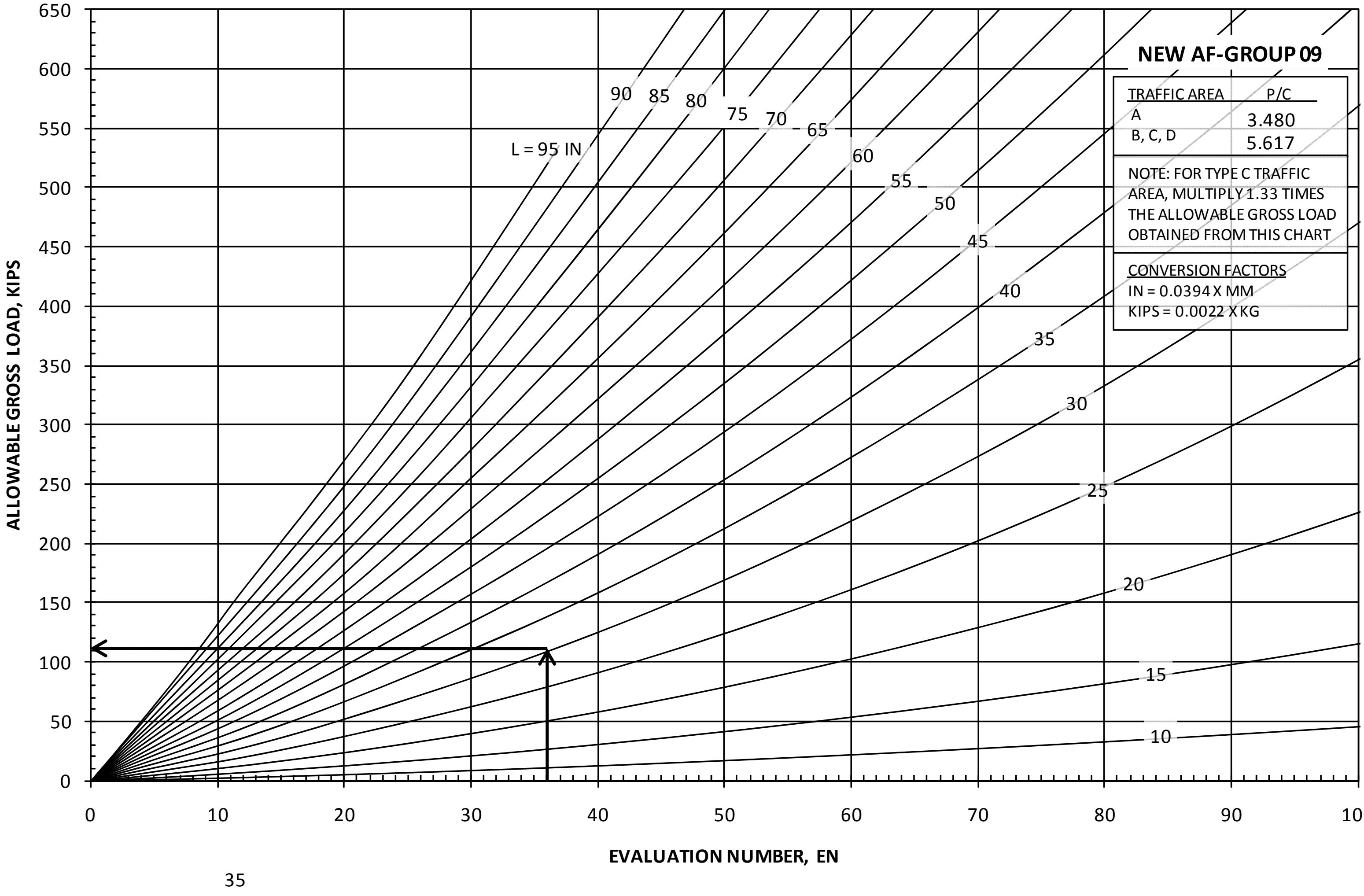


Figure 6-12. Rigid Pavement Evaluation Curve for Air Force Group 9

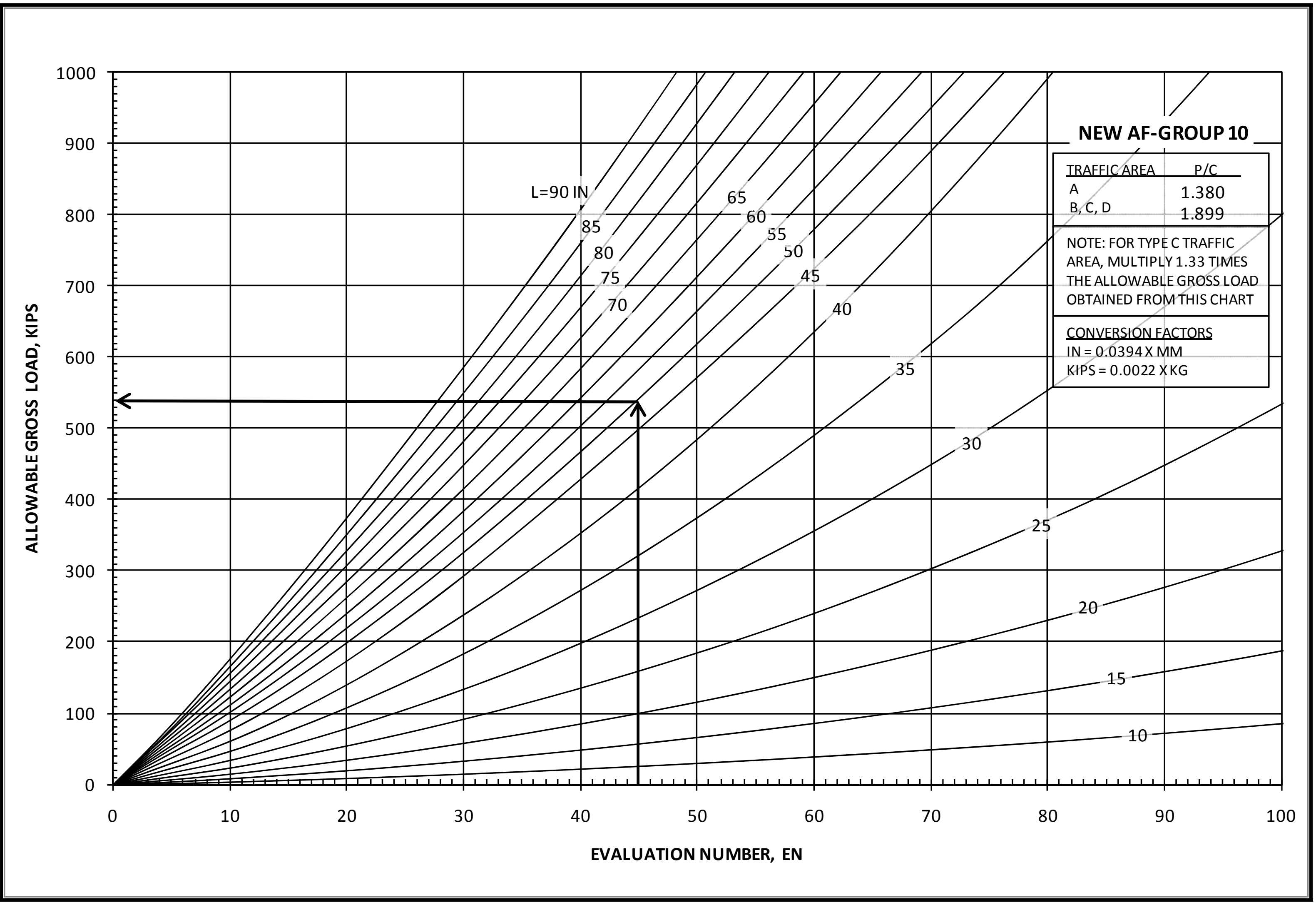


Figure 6-13. Rigid Pavement Evaluation Curve for Air Force Group 10

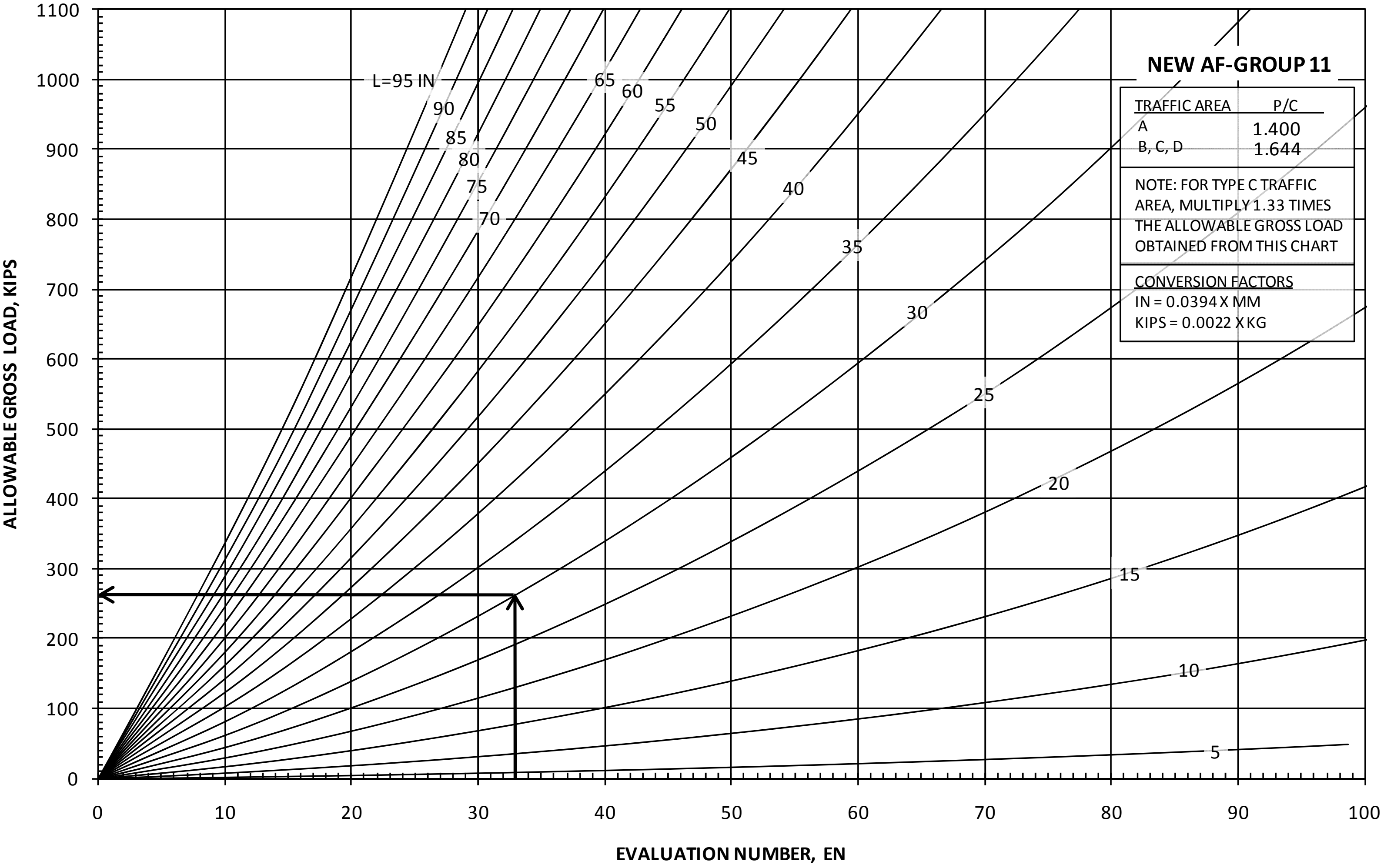


Figure 6-14. Rigid Pavement Evaluation Curve for Air Force Group 11

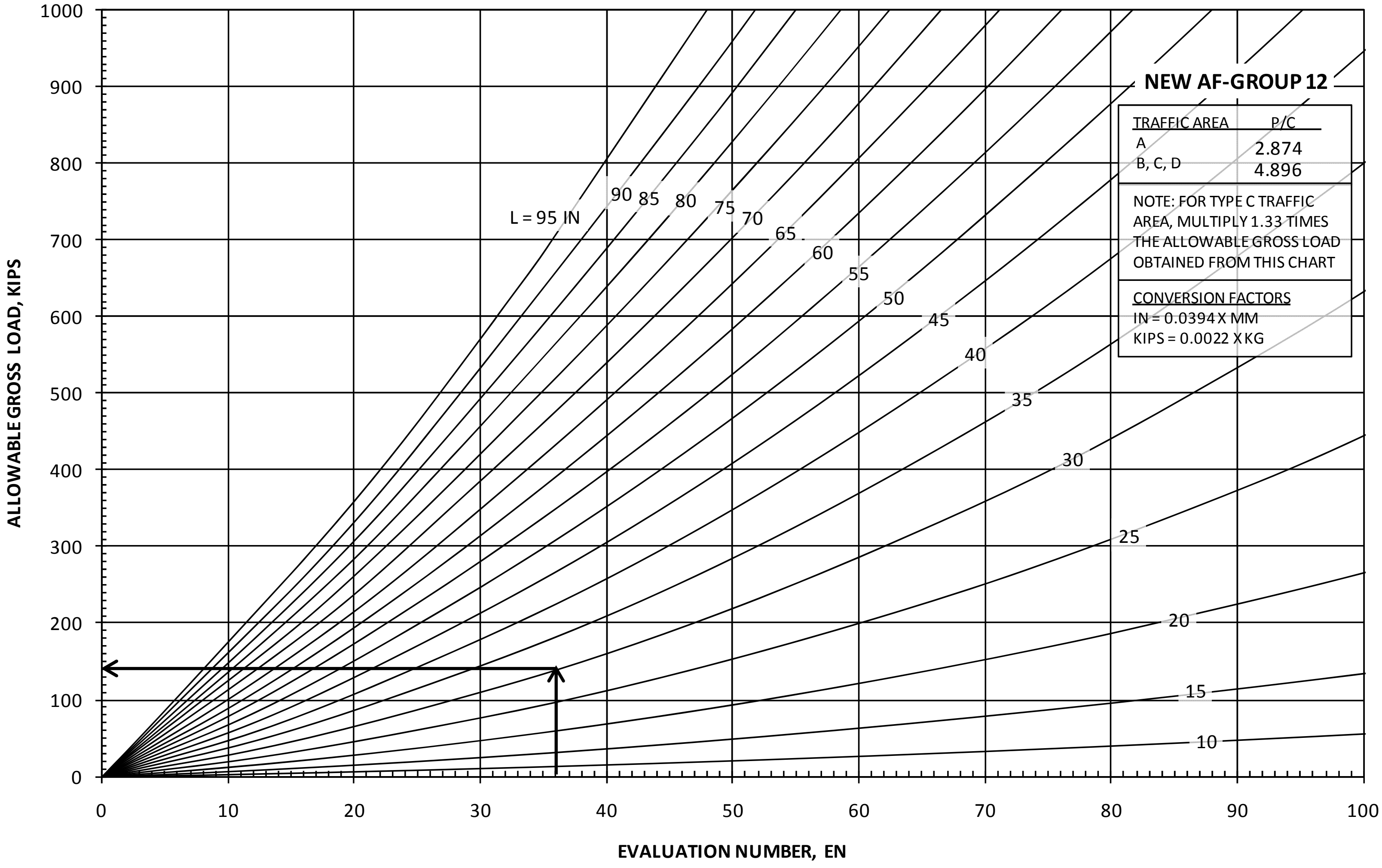


Figure 6-15. Rigid Pavement Evaluation Curve for Air Force Group 12

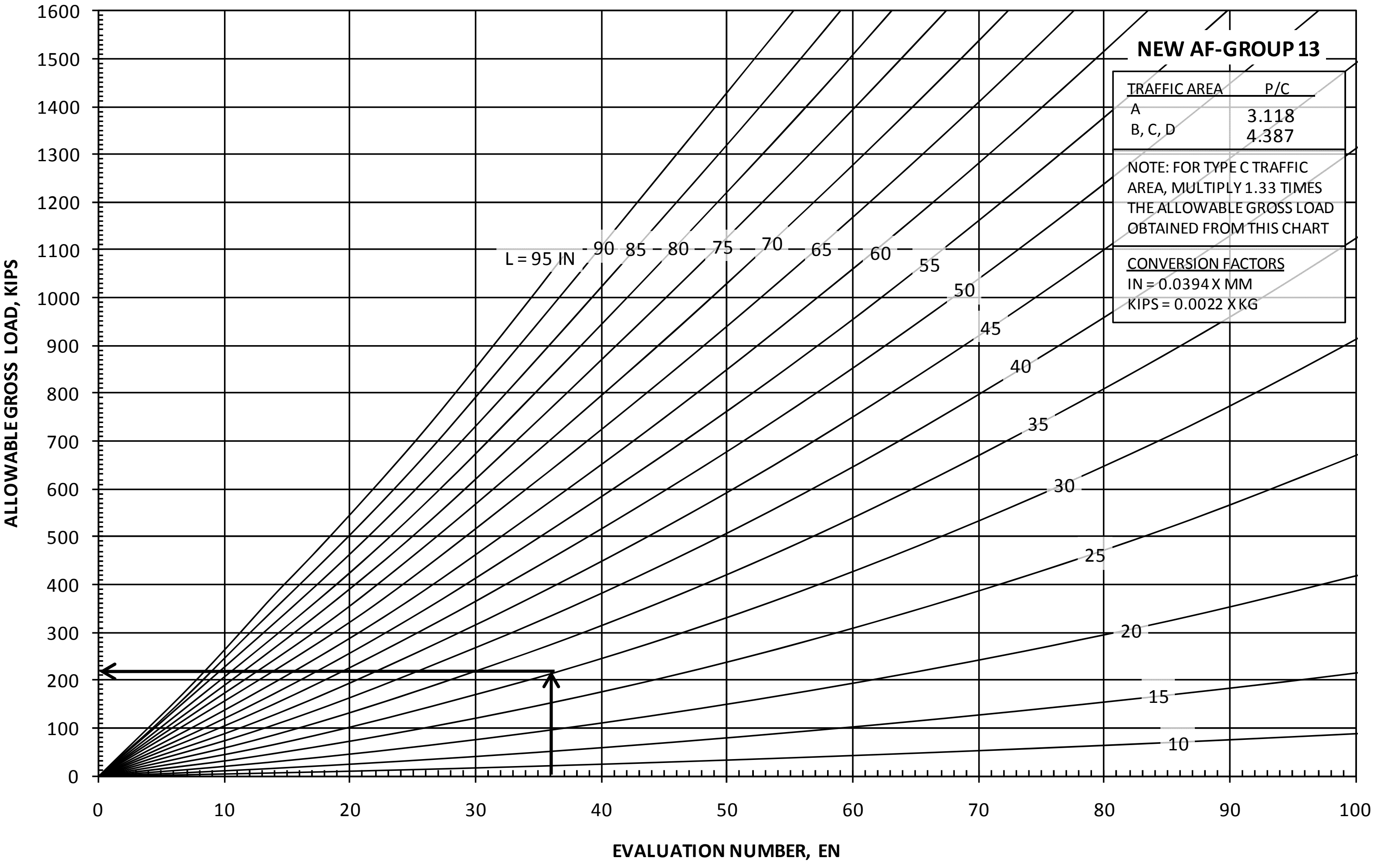


Figure 6-16. Rigid Pavement Evaluation Curve for Air Force Group 13

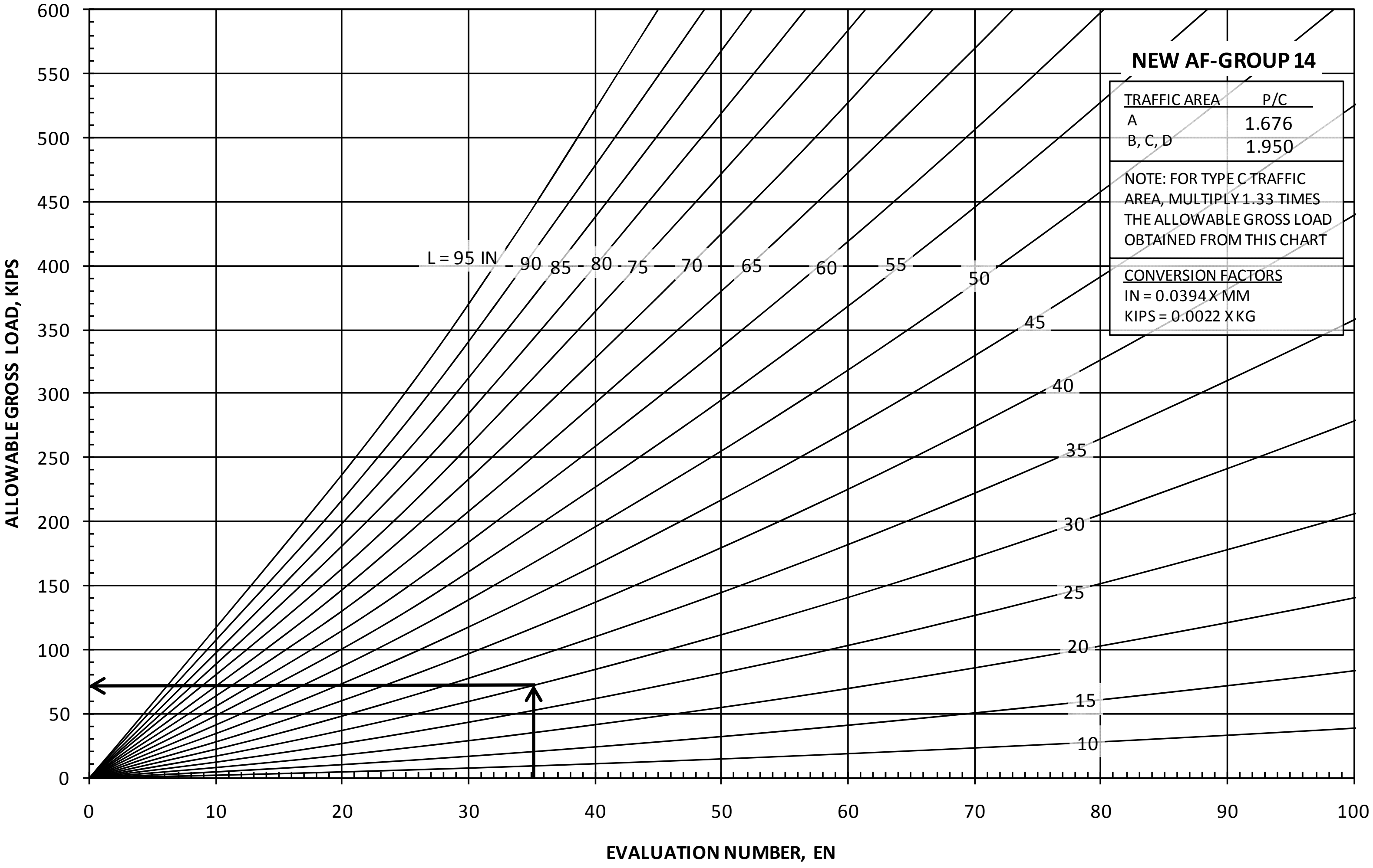


Figure 6-17. Rigid Pavement Evaluation Curve for Air Force Group 14

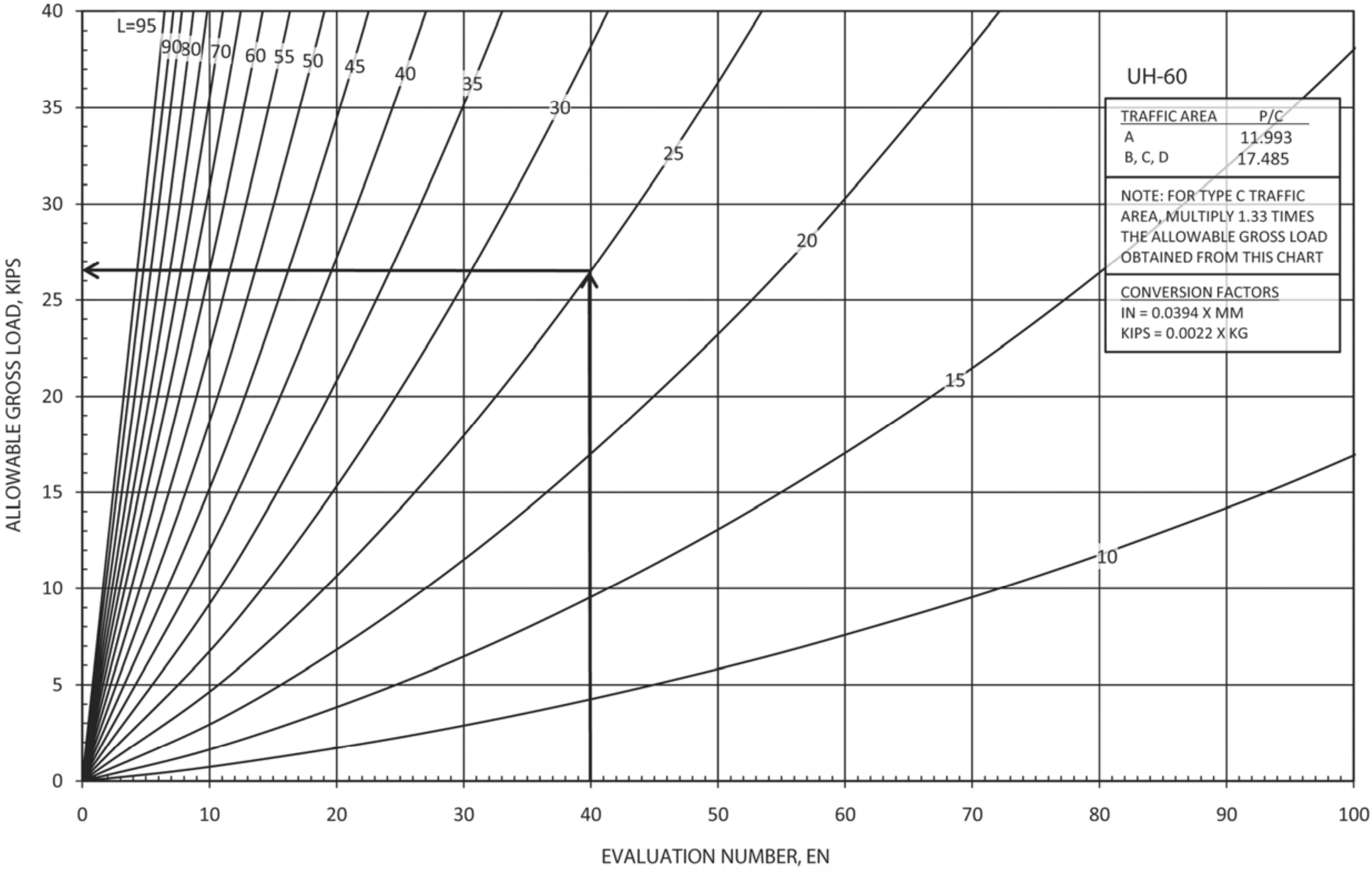
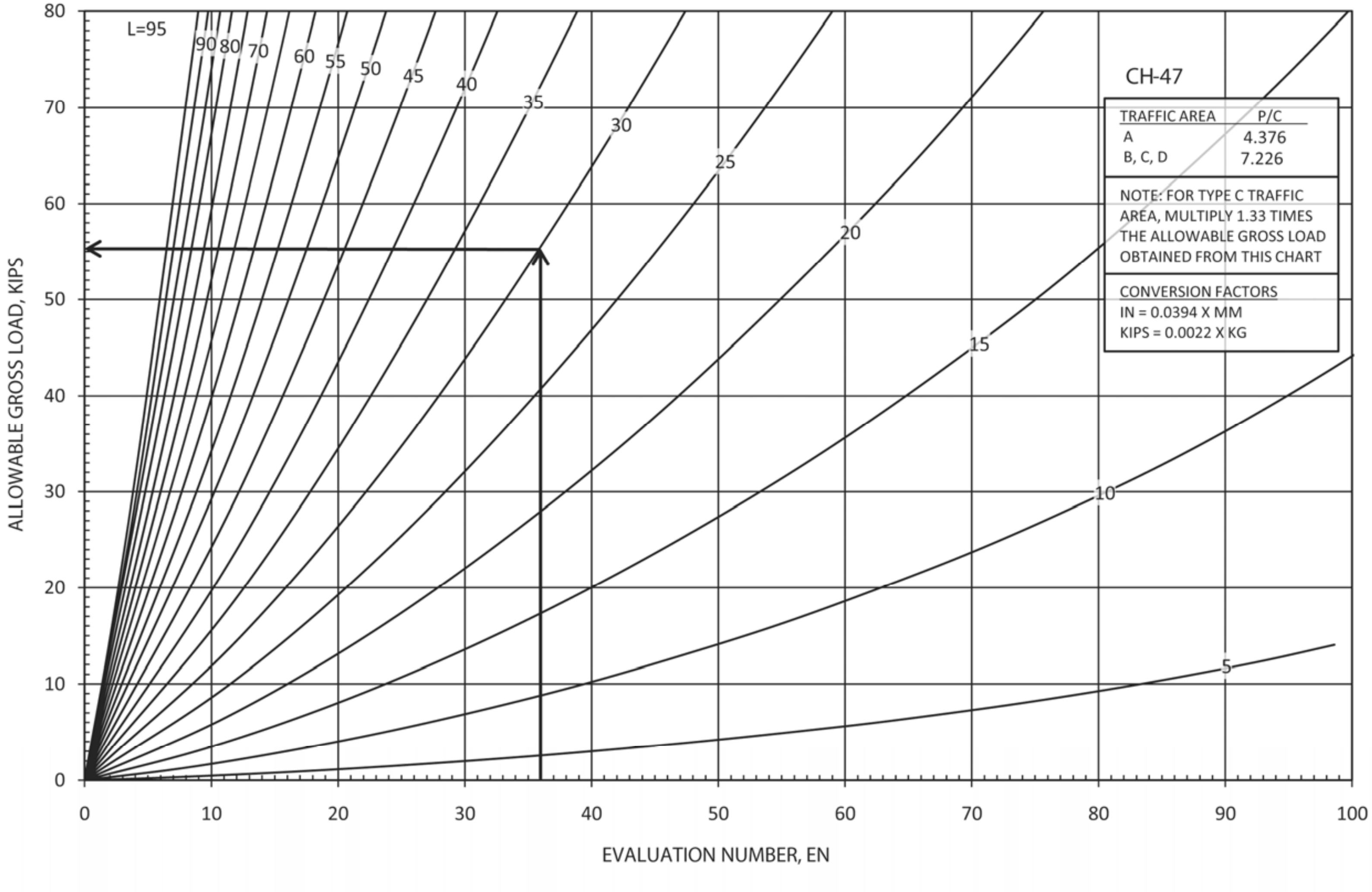


Figure 6-18. Rigid Pavement Evaluation Curve for UH-60

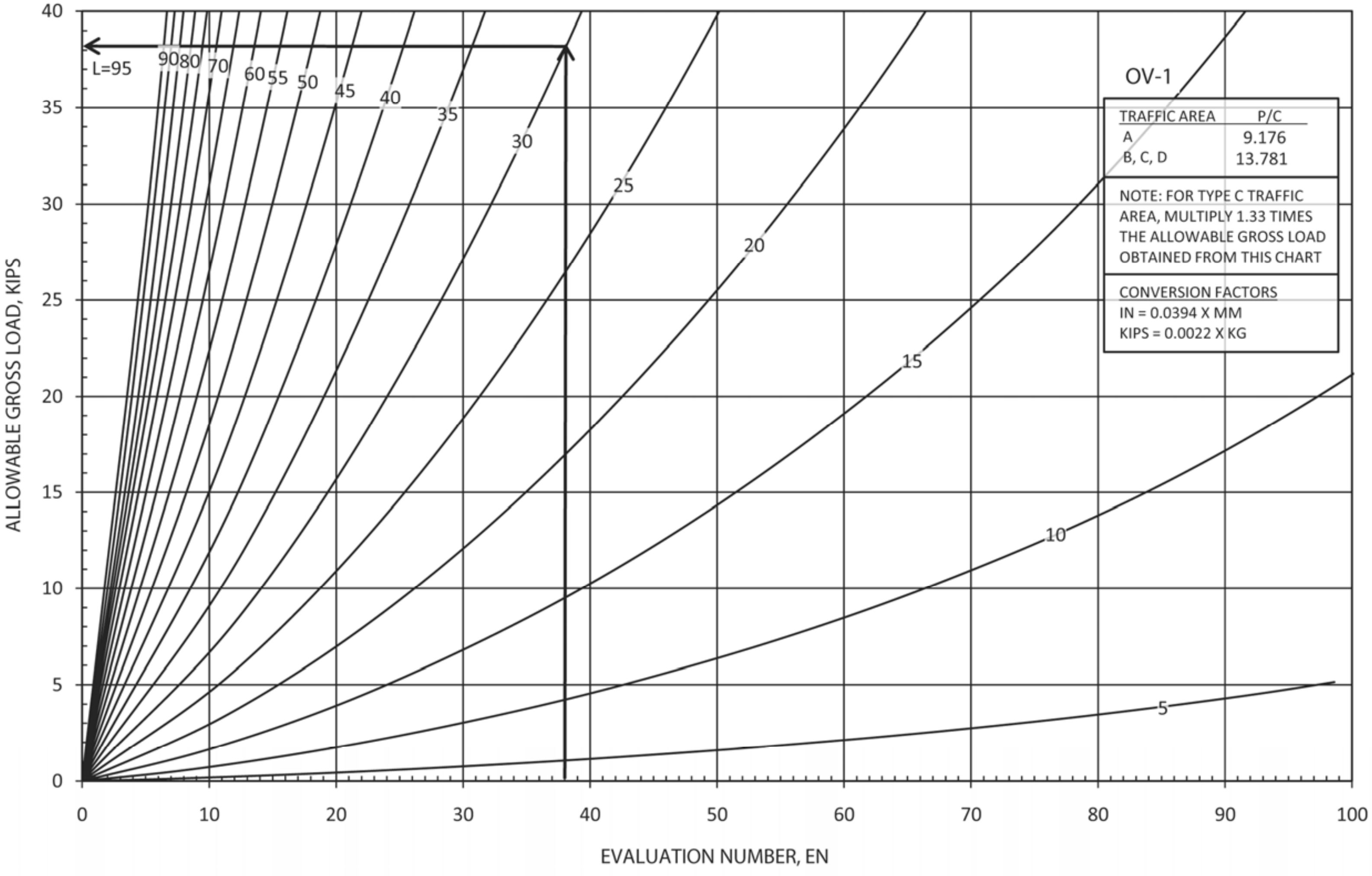


TRAFFIC AREA	P/C
A	4.376
B, C, D	7.226

NOTE: FOR TYPE C TRAFFIC AREA, MULTIPLY 1.33 TIMES THE ALLOWABLE GROSS LOAD OBTAINED FROM THIS CHART

CONVERSION FACTORS
IN = 0.0394 X MM
KIPS = 0.0022 X KG

Figure 6-19. Rigid Pavement Evaluation Curve for CH-47



TRAFFIC AREA	P/C
A	9.176
B, C, D	13.781

NOTE: FOR TYPE C TRAFFIC AREA, MULTIPLY 1.33 TIMES THE ALLOWABLE GROSS LOAD OBTAINED FROM THIS CHART

CONVERSION FACTORS
IN = 0.0394 X MM
KIPS = 0.0022 X KG

Figure 6-20. Rigid Pavement Evaluation Curve for OV-1

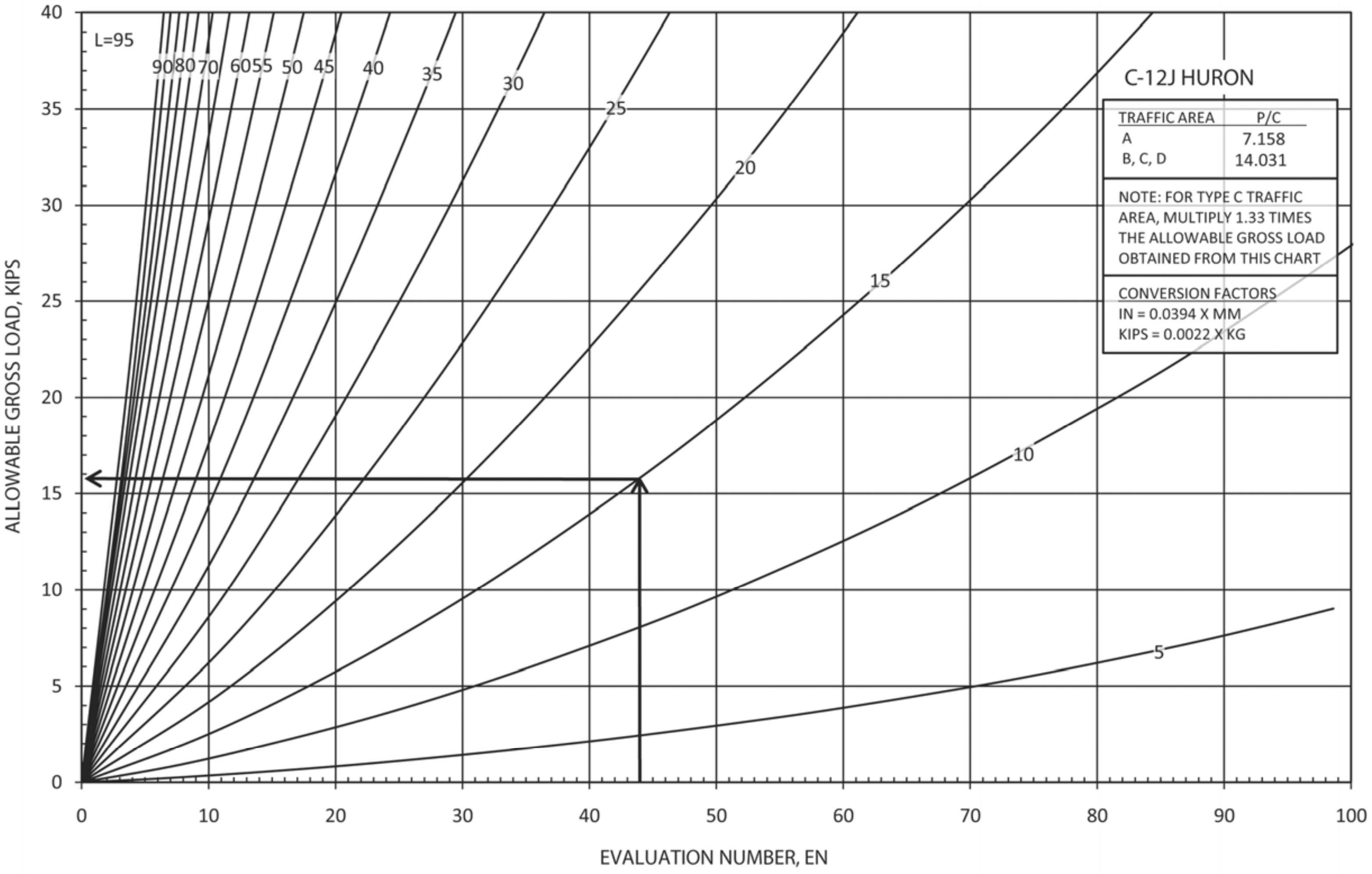


Figure 6-21. Rigid Pavement Evaluation Curve for C-12J

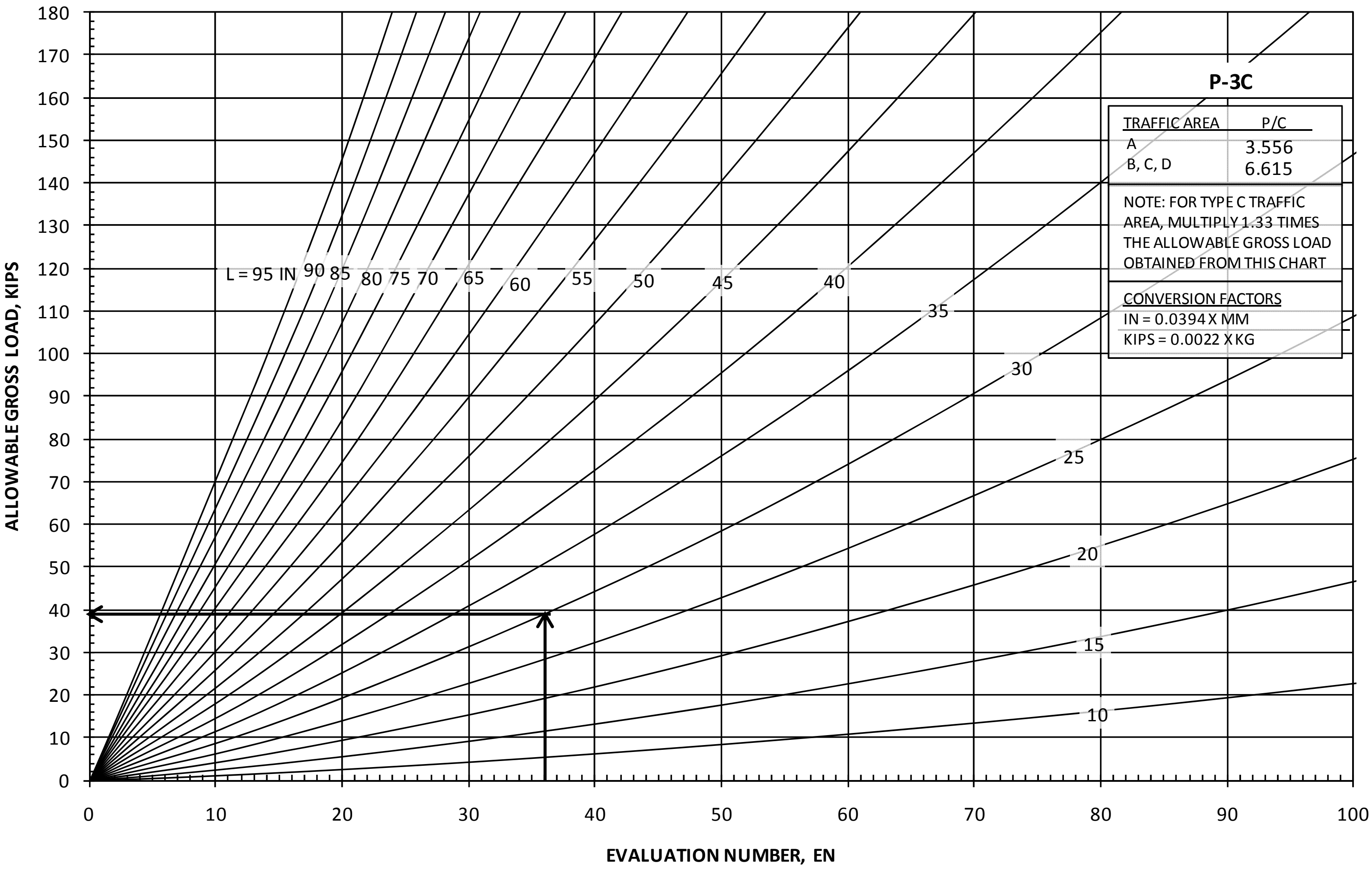


Figure 6-22. Rigid Pavement Evaluation Curve for P-3C

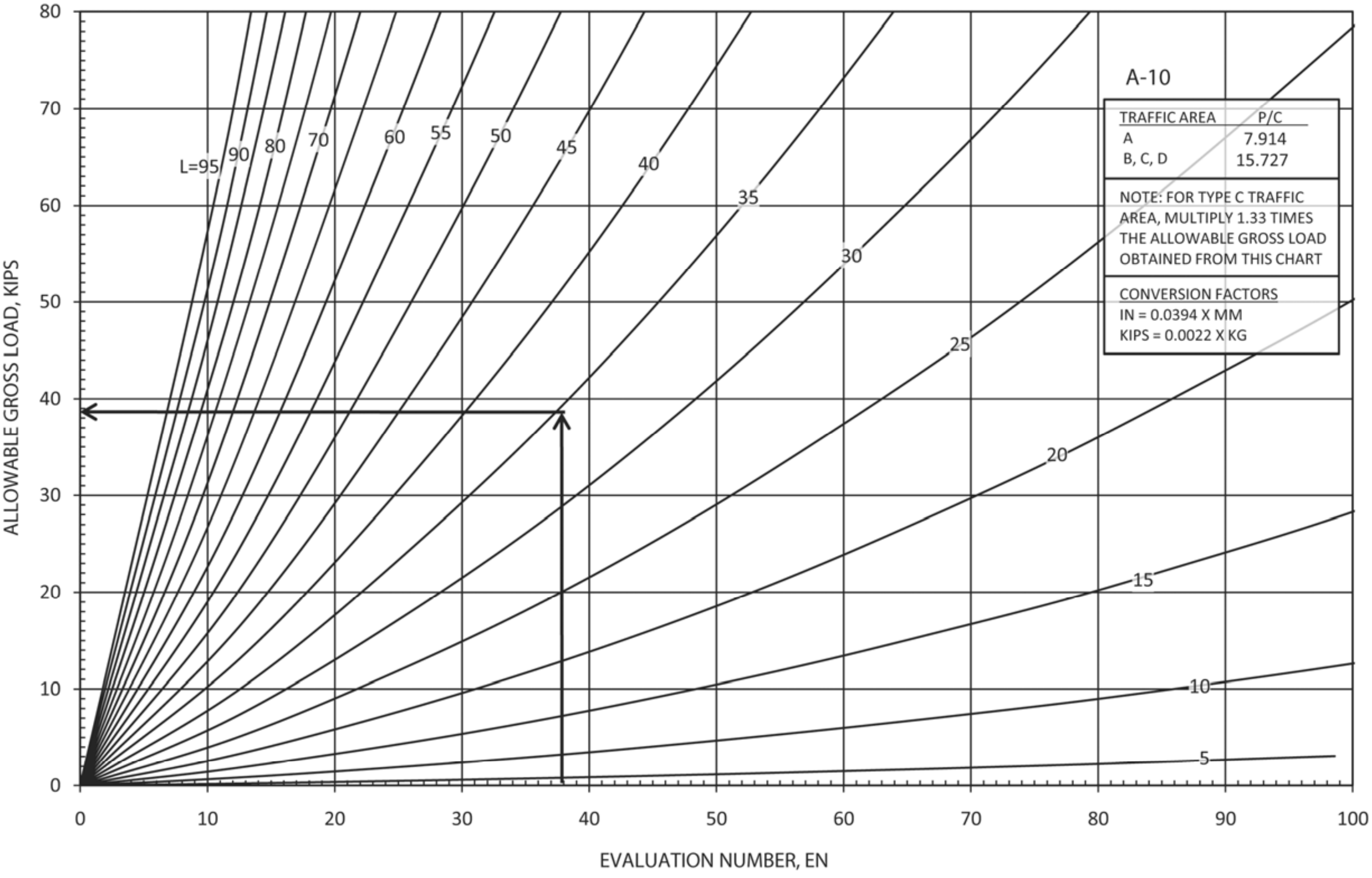
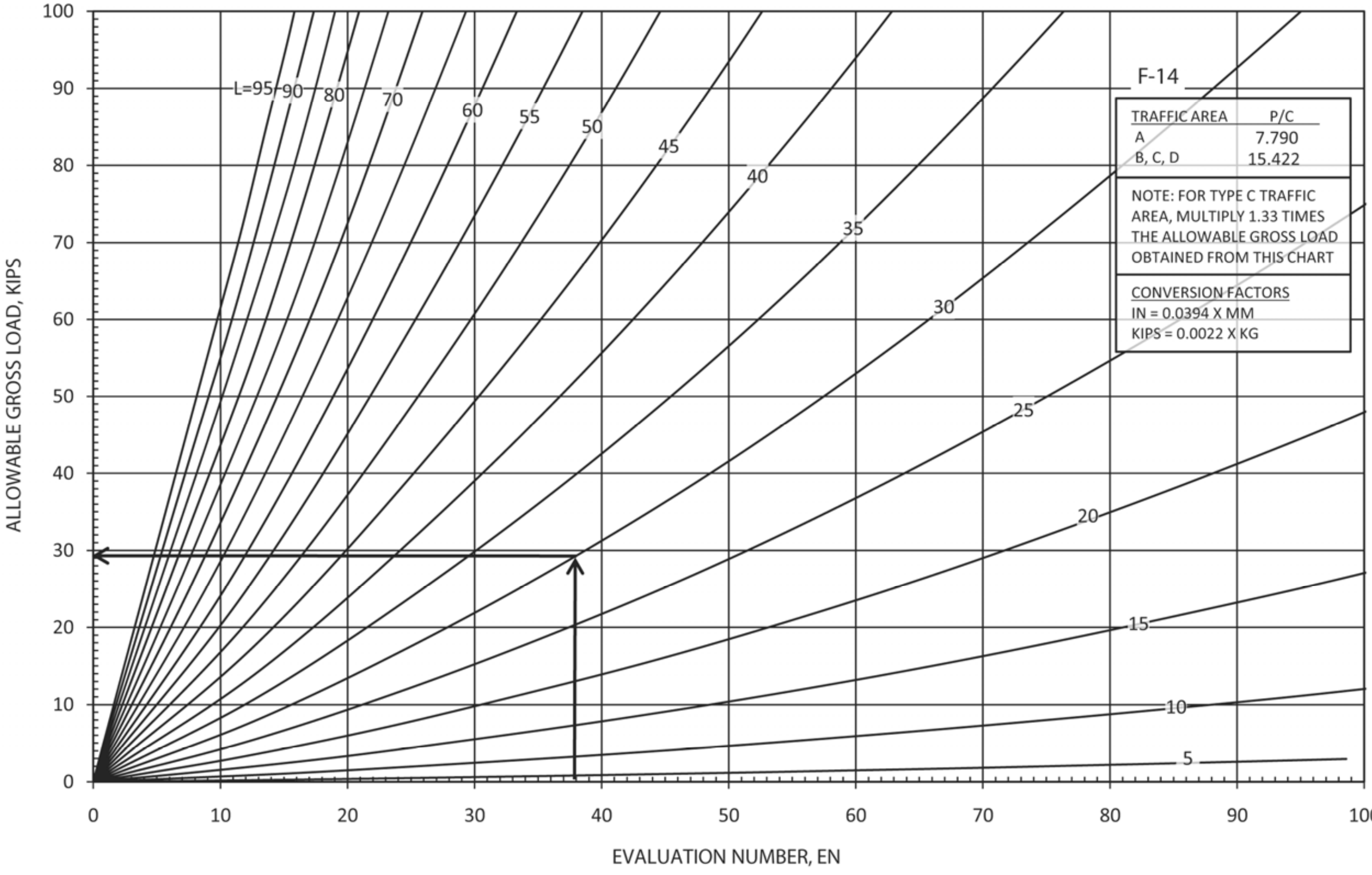


Figure 6-23. Rigid Pavement Evaluation Curve for A-10



TRAFFIC AREA	P/C
A	7.790
B, C, D	15.422

NOTE: FOR TYPE C TRAFFIC AREA, MULTIPLY 1.33 TIMES THE ALLOWABLE GROSS LOAD OBTAINED FROM THIS CHART

CONVERSION FACTORS
IN = 0.0394 X MM
KIPS = 0.0022 X KG

Figure 6-24. Rigid Pavement Evaluation Curve for F-14

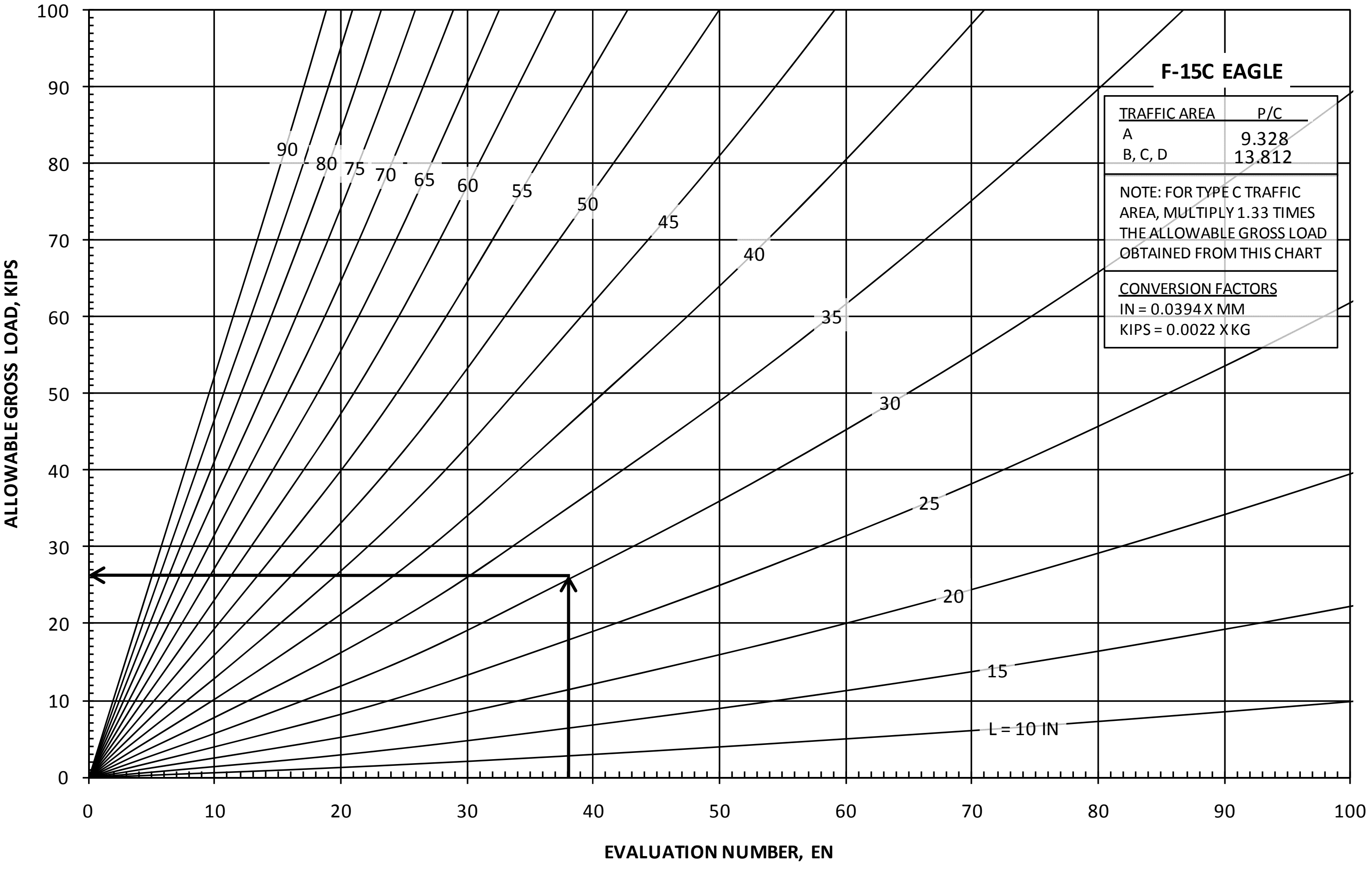
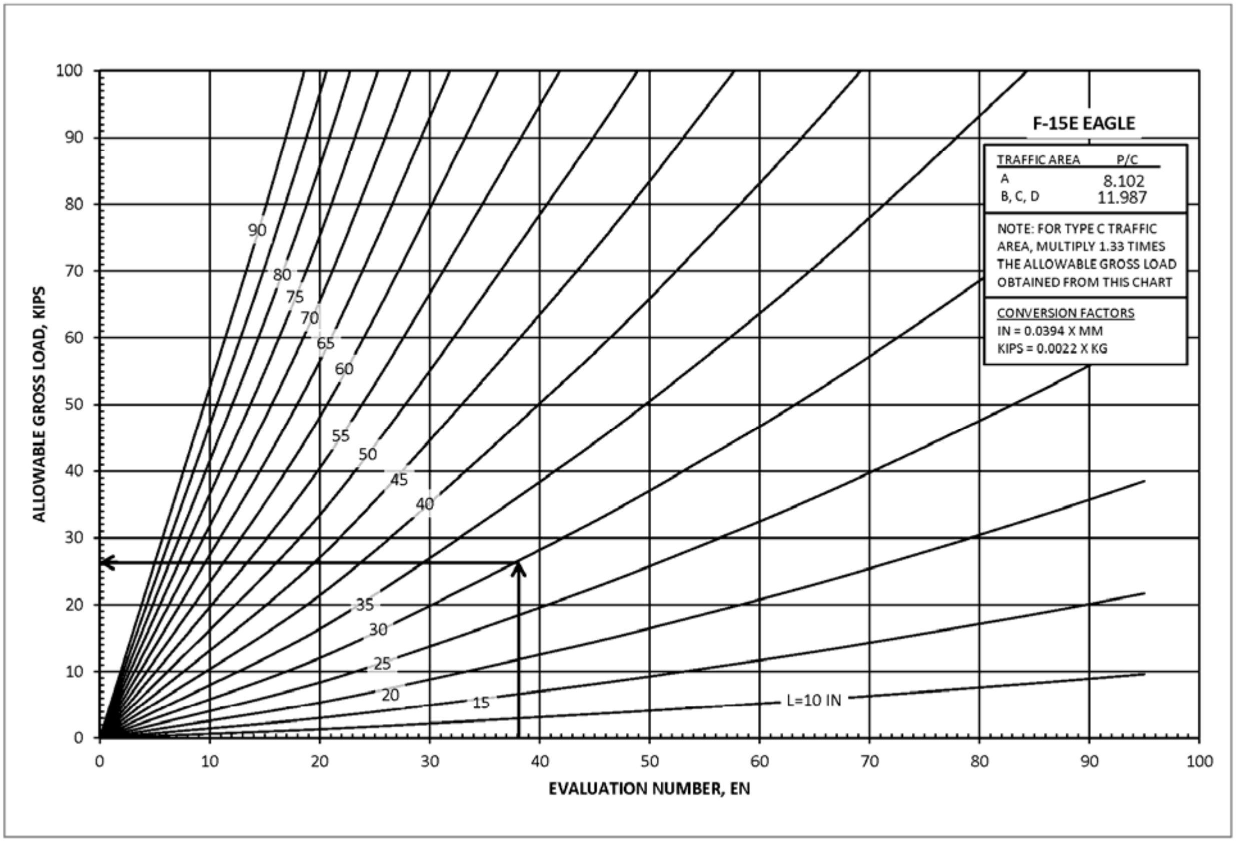


Figure 6-25. Rigid Pavement Evaluation Curve for F-15C/D



F-15E EAGLE

TRAFFIC AREA	P/C
A	8.102
B, C, D	11.987

NOTE: FOR TYPE C TRAFFIC AREA, MULTIPLY 1.33 TIMES THE ALLOWABLE GROSS LOAD OBTAINED FROM THIS CHART

CONVERSION FACTORS
IN = 0.0394 X MM
KIPS = 0.0022 X KG

Figure 6-25. Rigid Pavement Evaluation Curve for F-15E

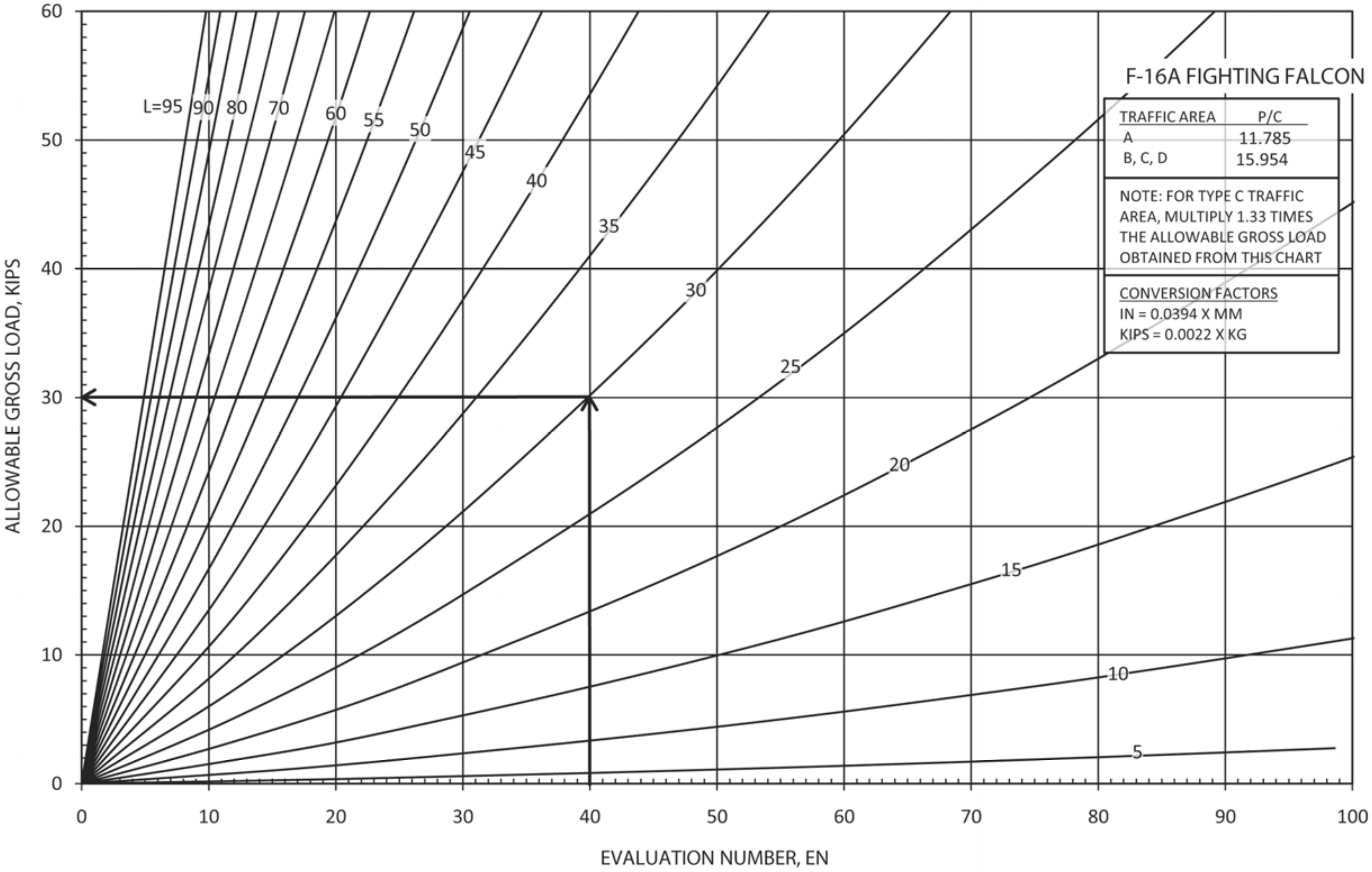


Figure 6-26. Rigid Pavement Evaluation Curve for F-16

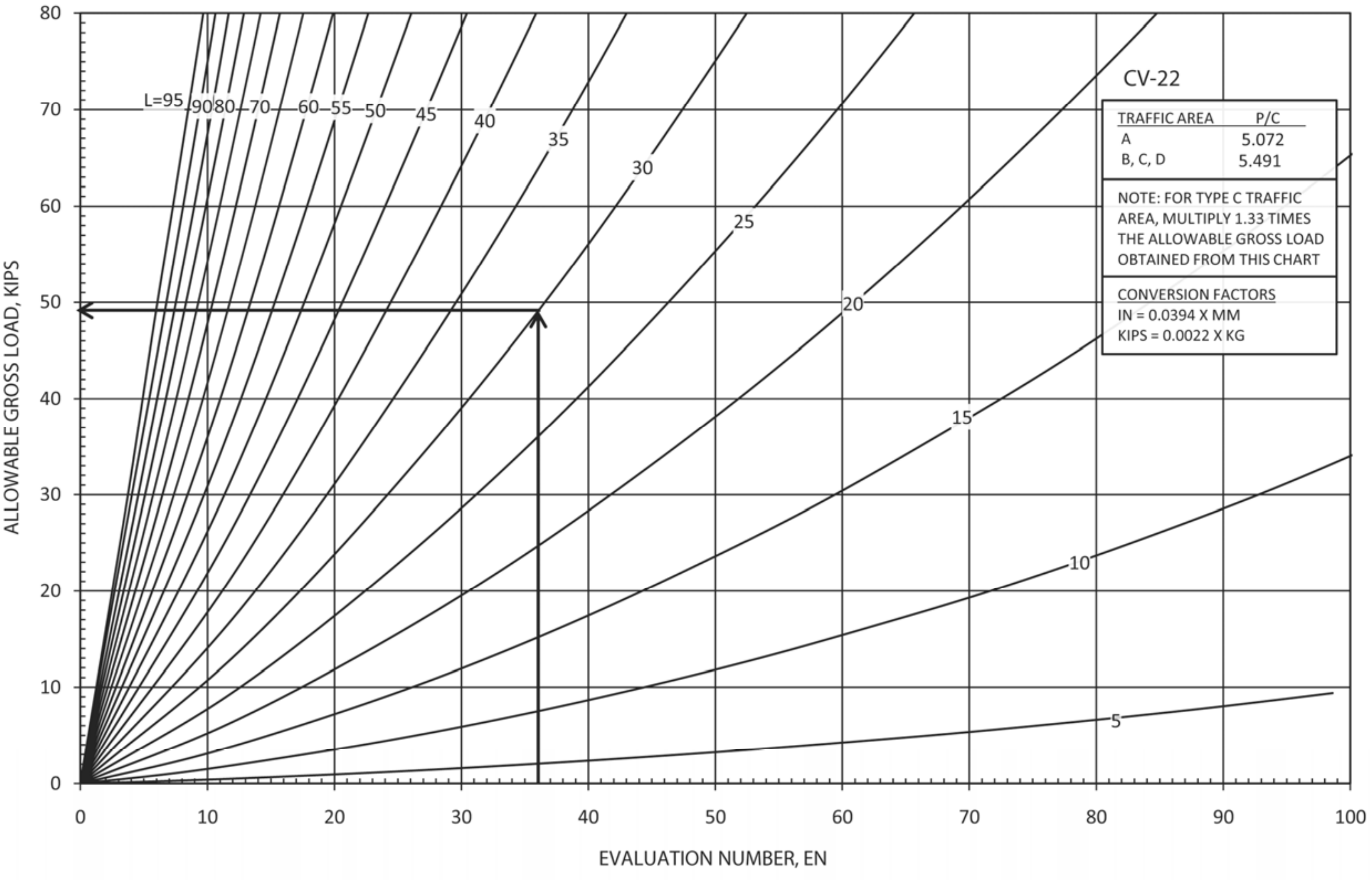


Figure 6-27. Rigid Pavement Evaluation Curve for CV-22

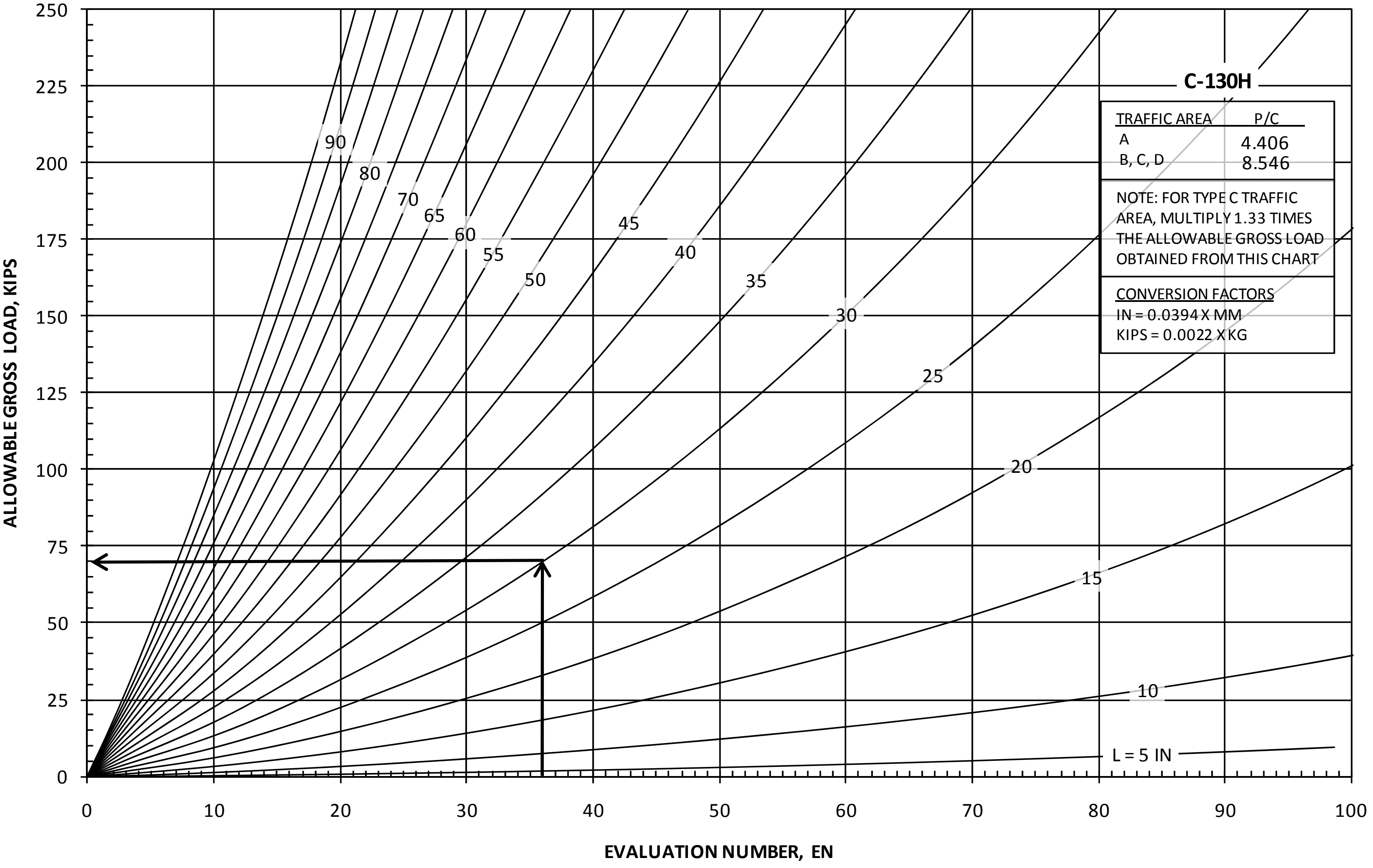


Figure 6-28. Rigid Pavement Evaluation Curve for C-130H

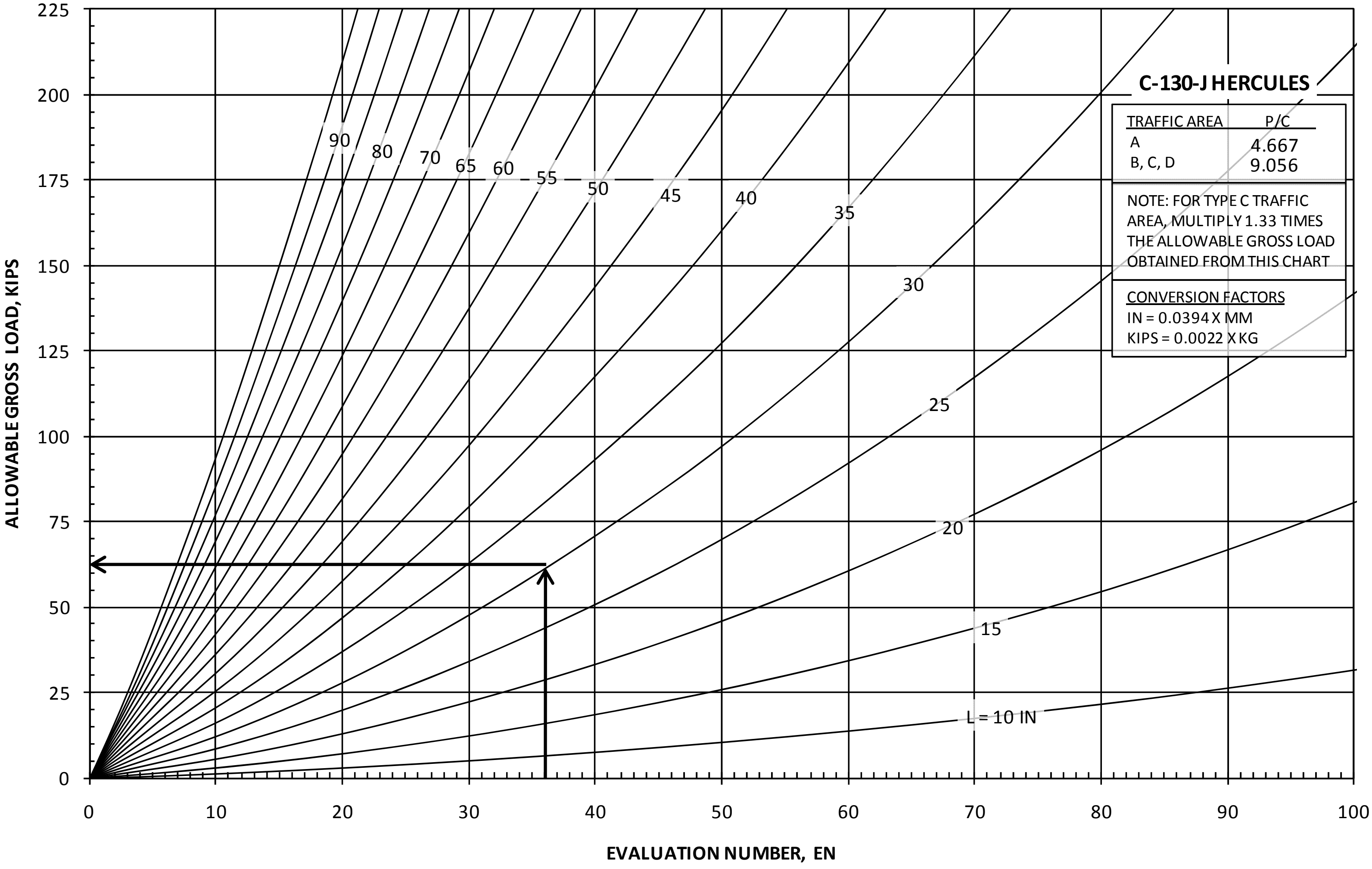


Figure 6-29. Rigid Pavement Evaluation Curve for C-130J

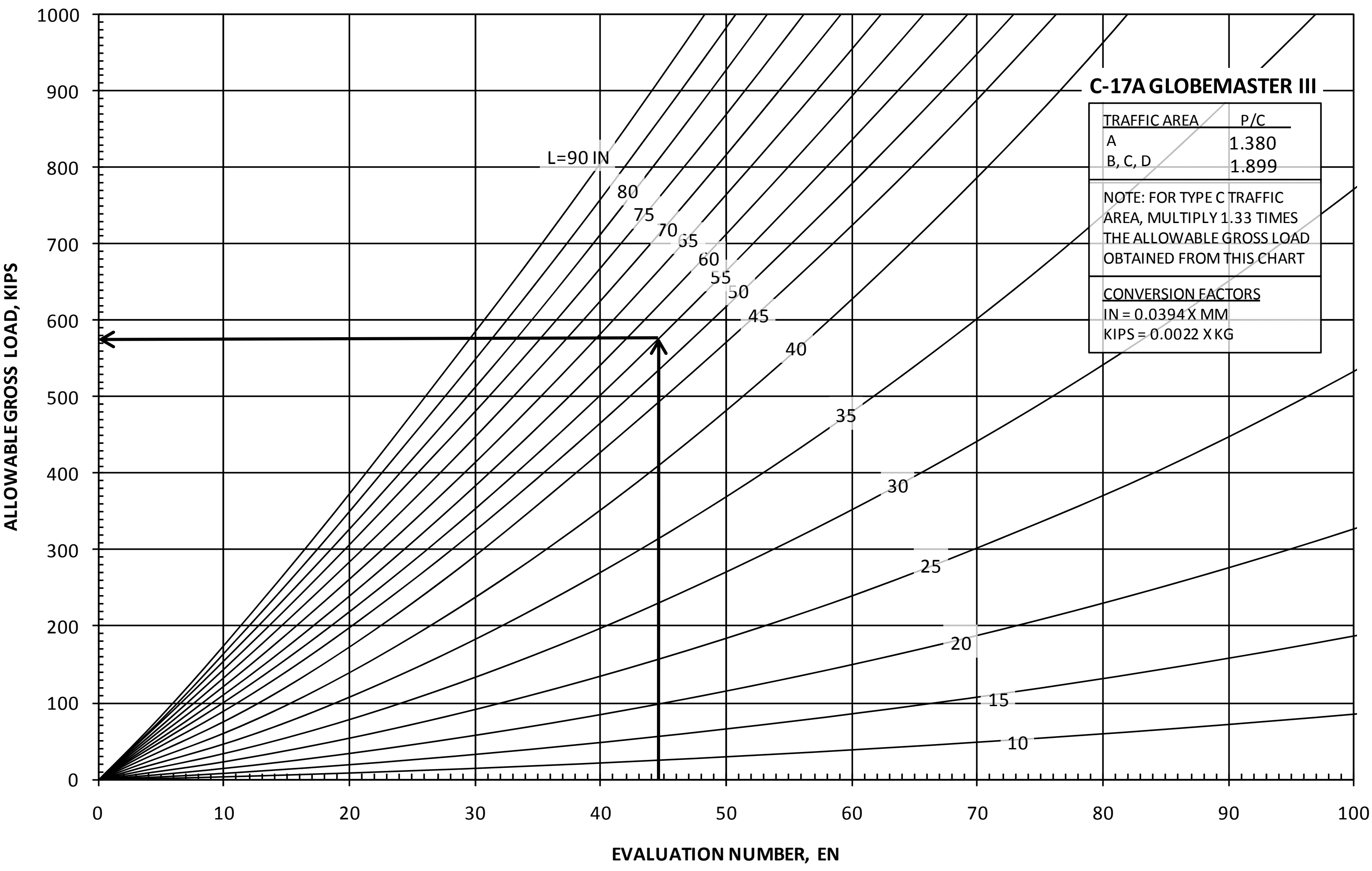


Figure 6-30. Rigid Pavement Evaluation Curve for C-17

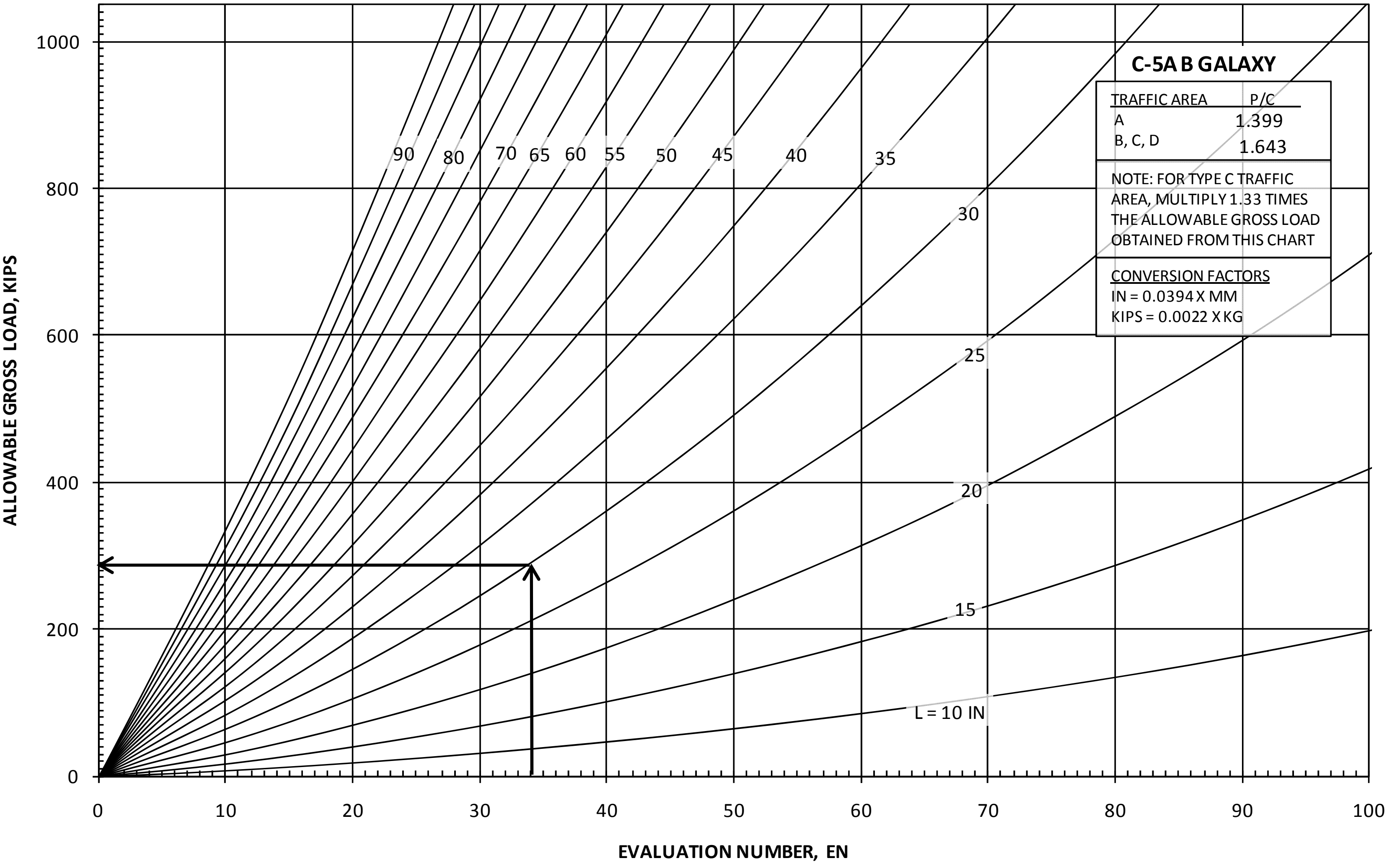


Figure 6-31. Rigid Pavement Evaluation Curve for C-5

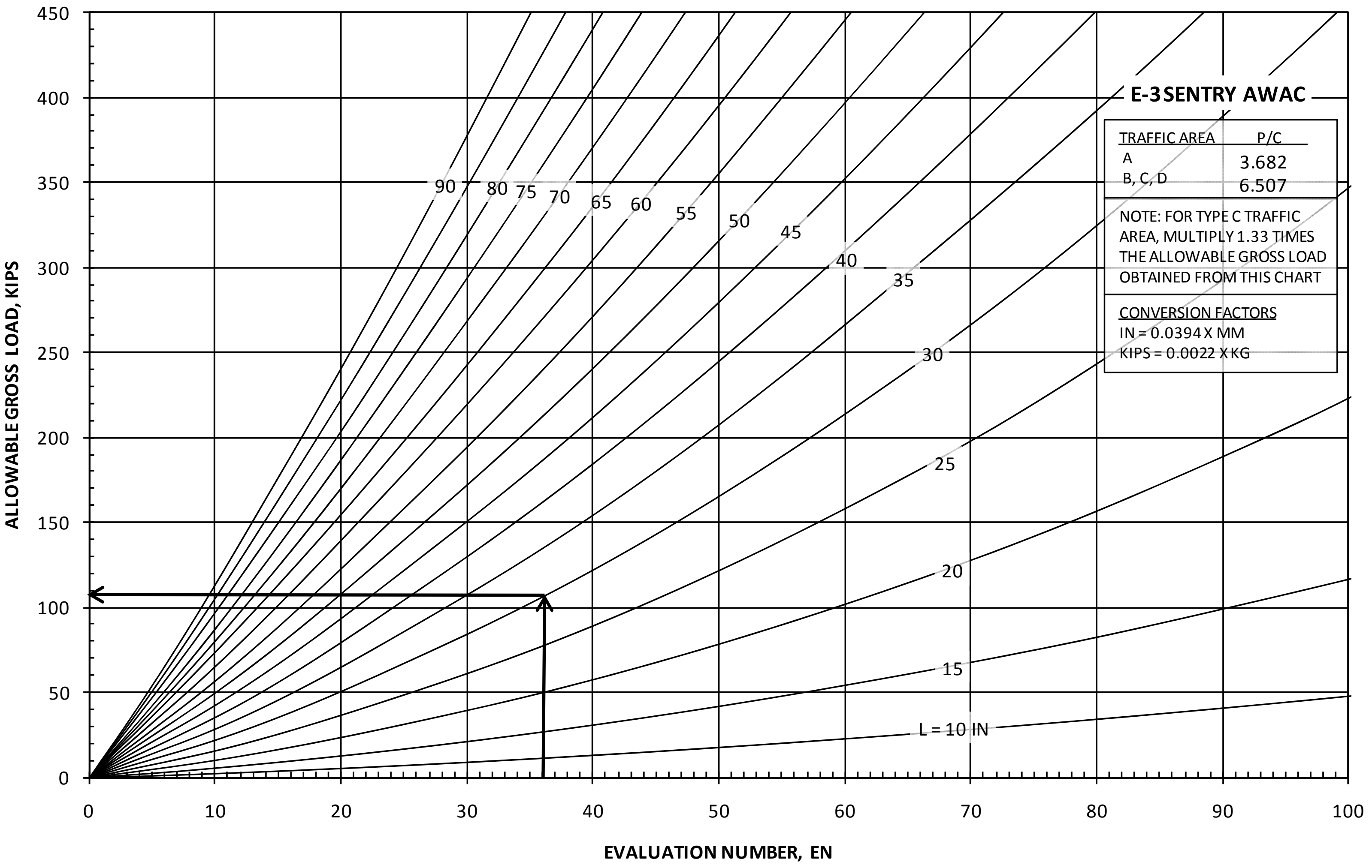


Figure 6-32. Rigid Pavement Evaluation Curve for E-3

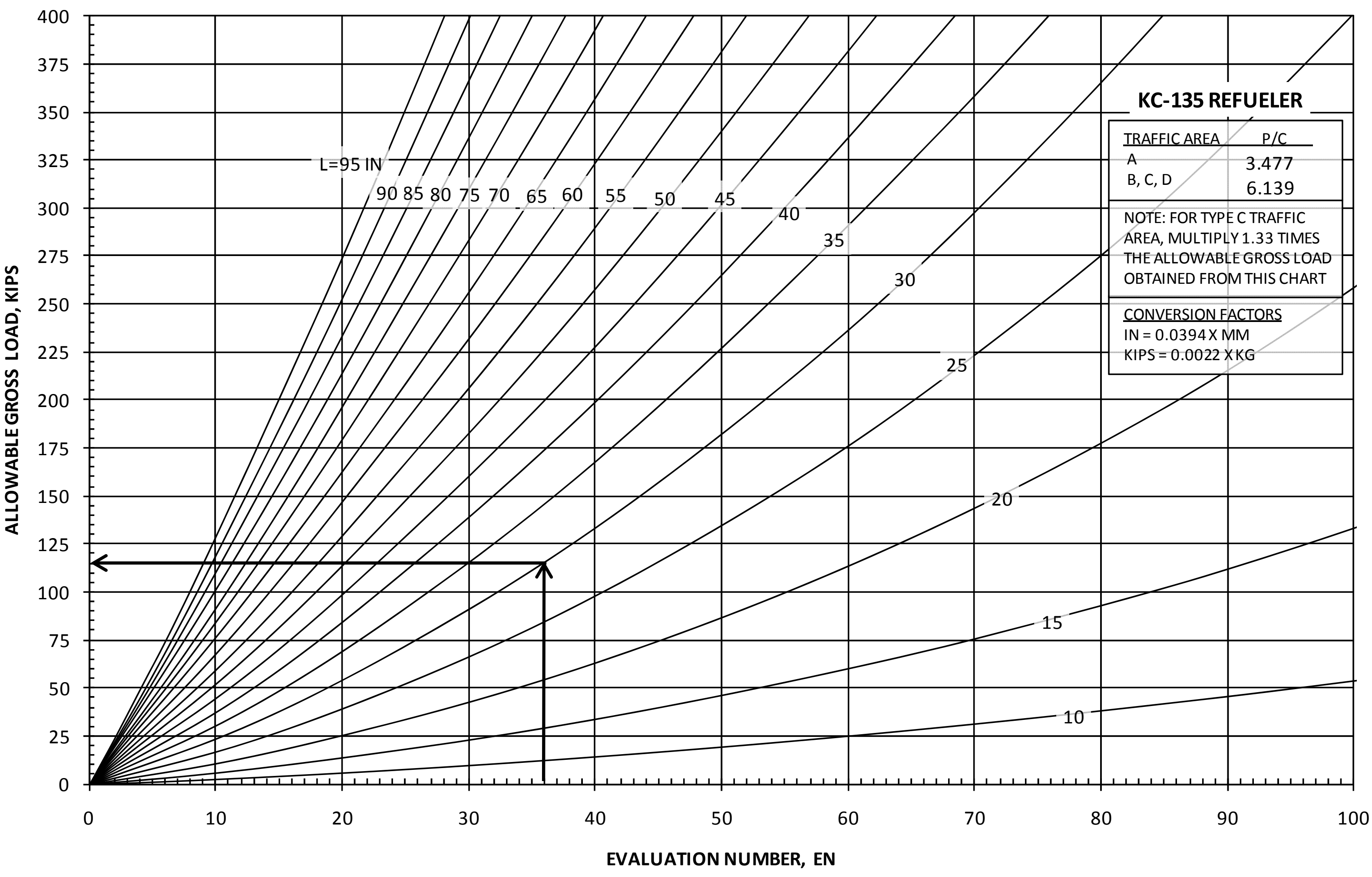


Figure 6-33. Rigid Pavement Evaluation Curve for KC-135

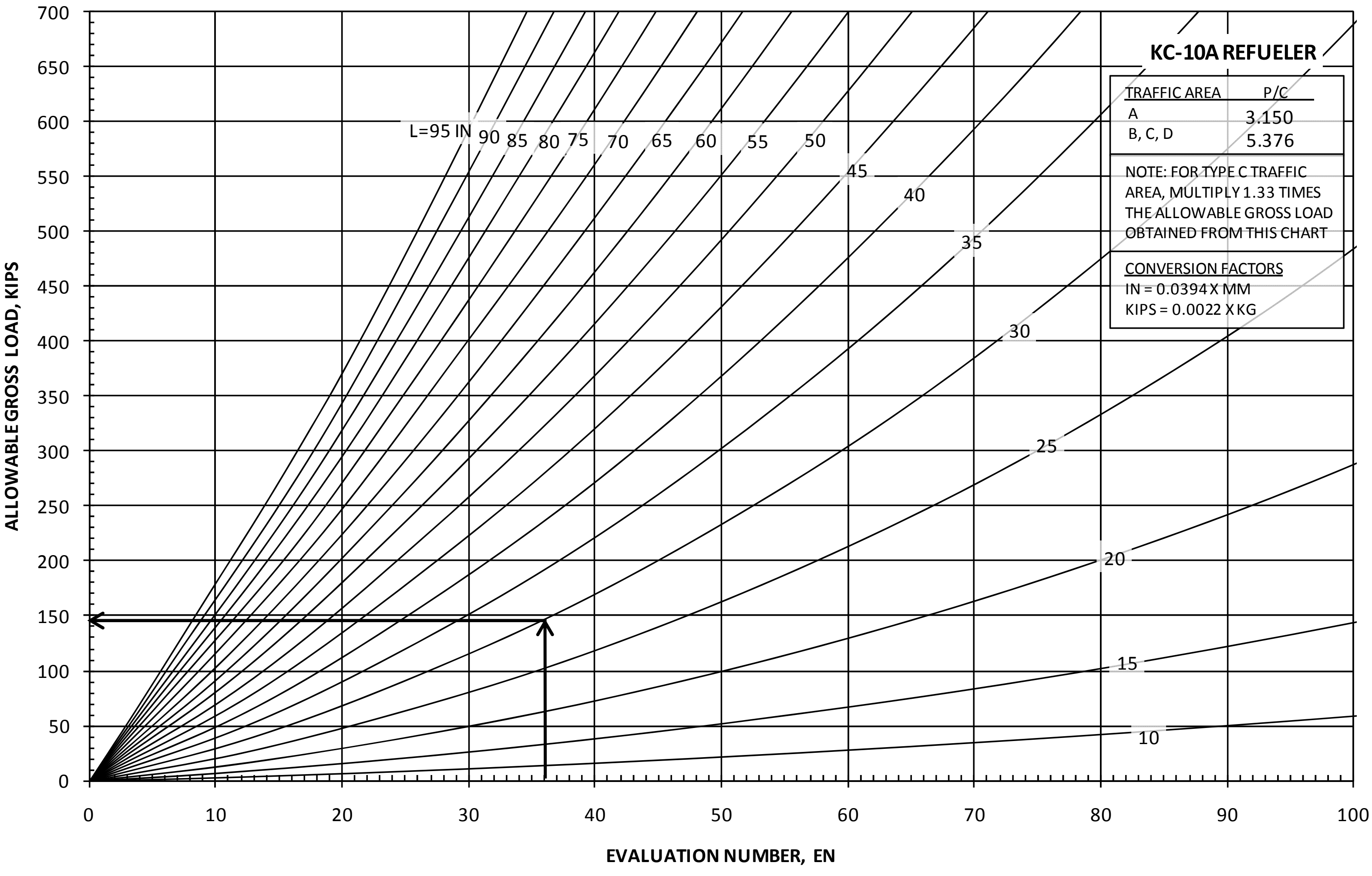


Figure 6-34. Rigid Pavement Evaluation Curve for KC-10

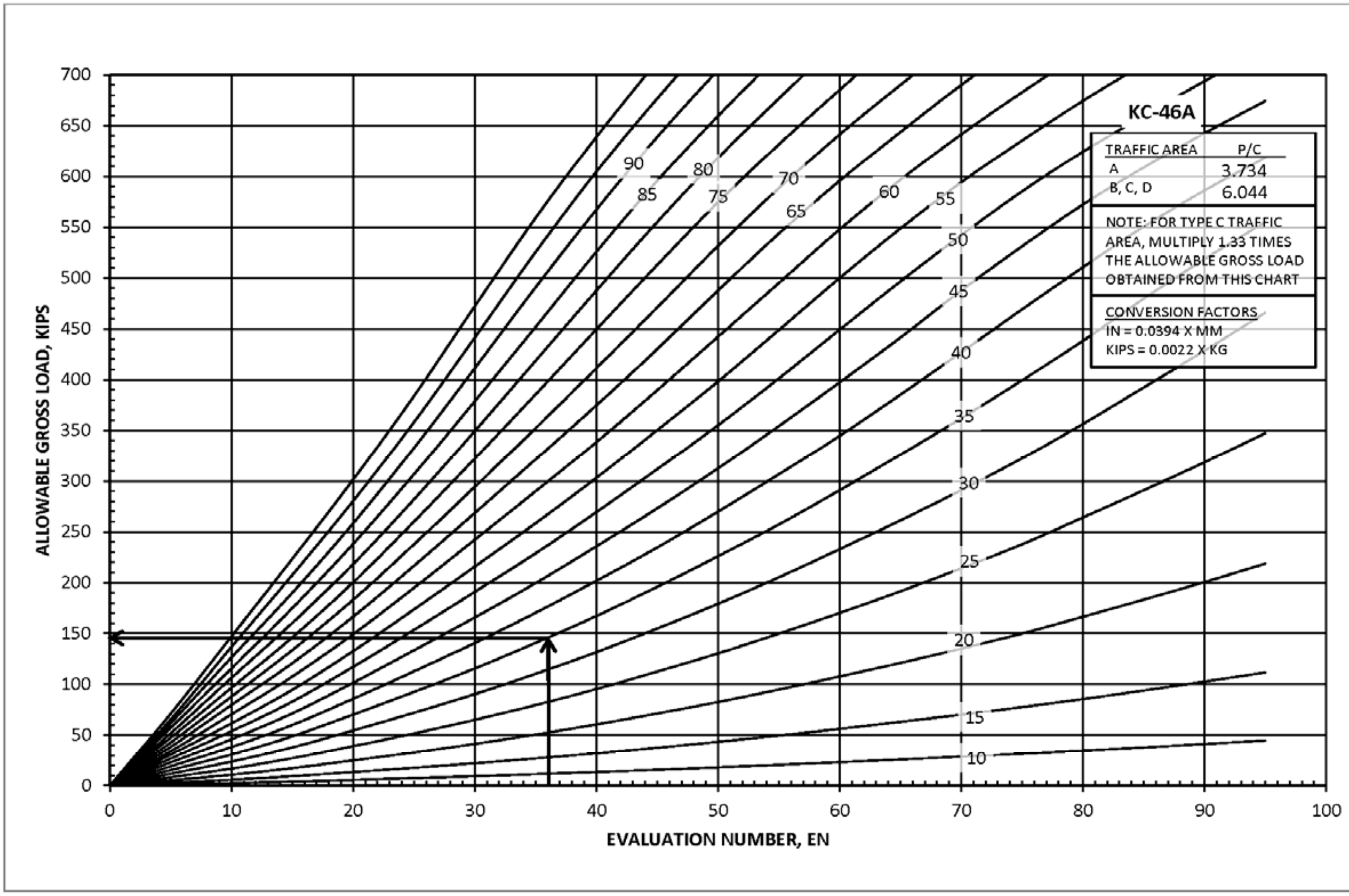


Figure 6-35. Rigid Pavement Evaluation Curve for KC-46A

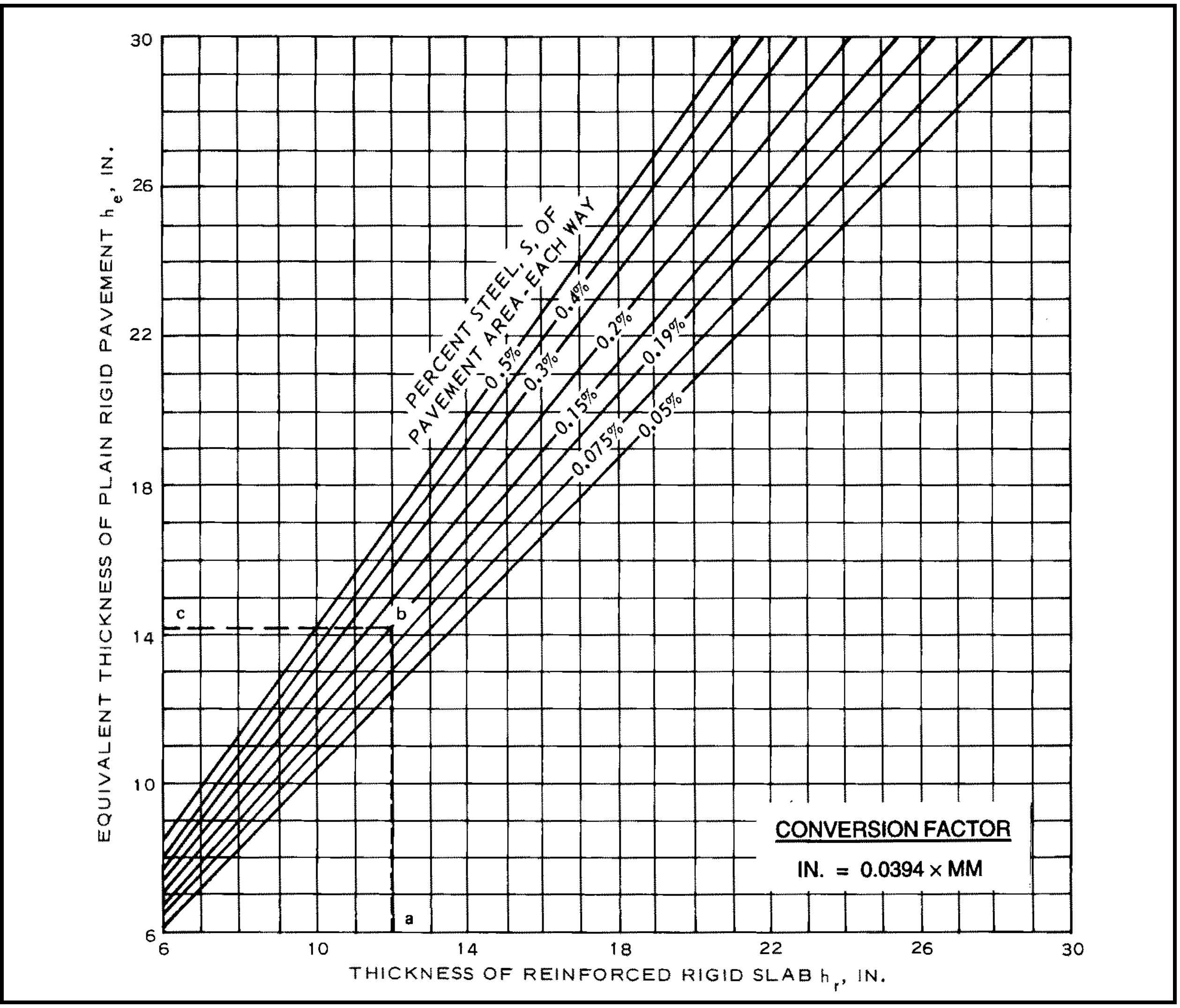


Figure 6-36. Equivalent Thickness of Reinforced Concrete Pavement

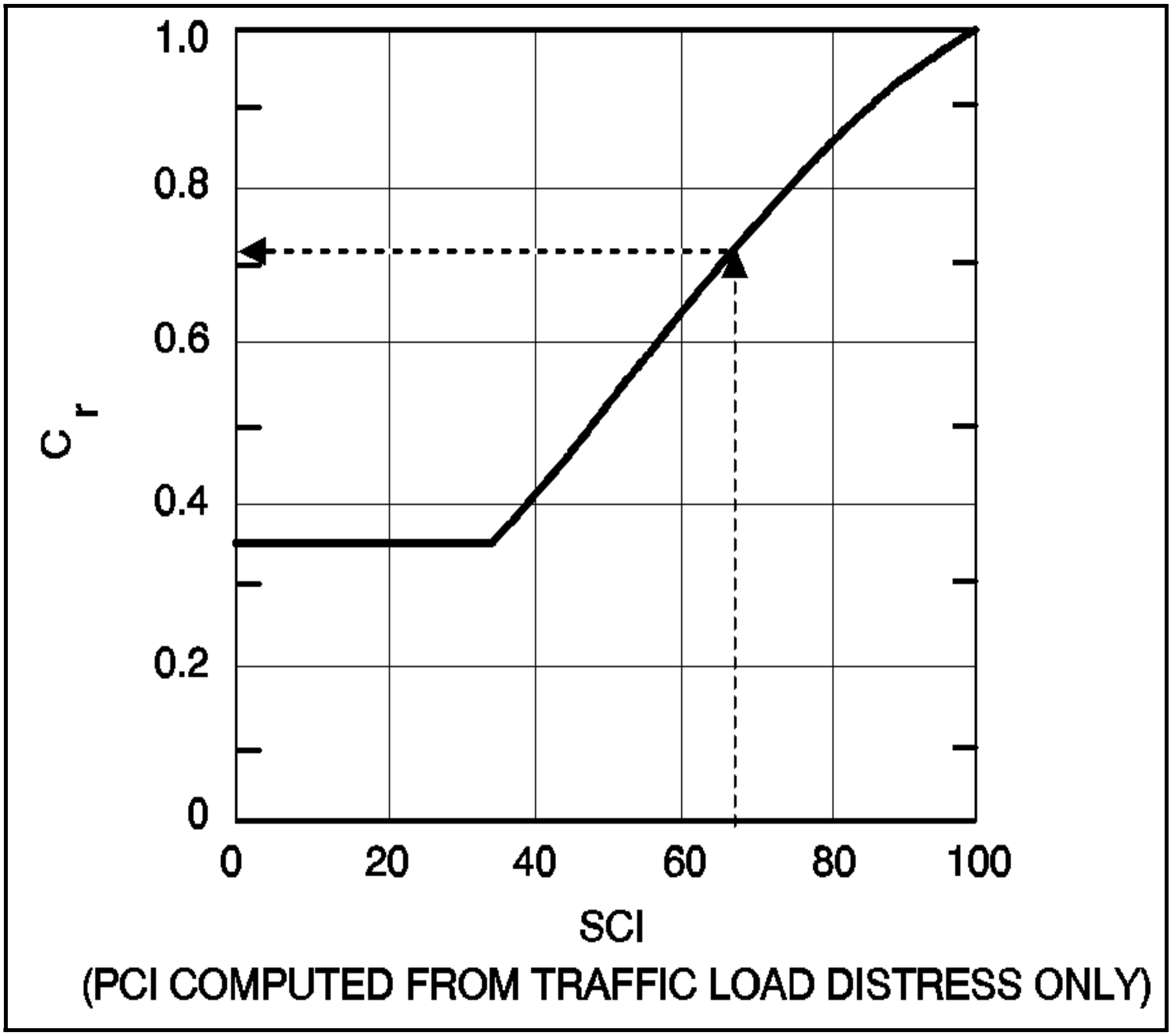


Figure 6-37. Condition Factor (C_b) for determining Equivalent Thickness of Nonrigid Overlays of Rigid Pavements

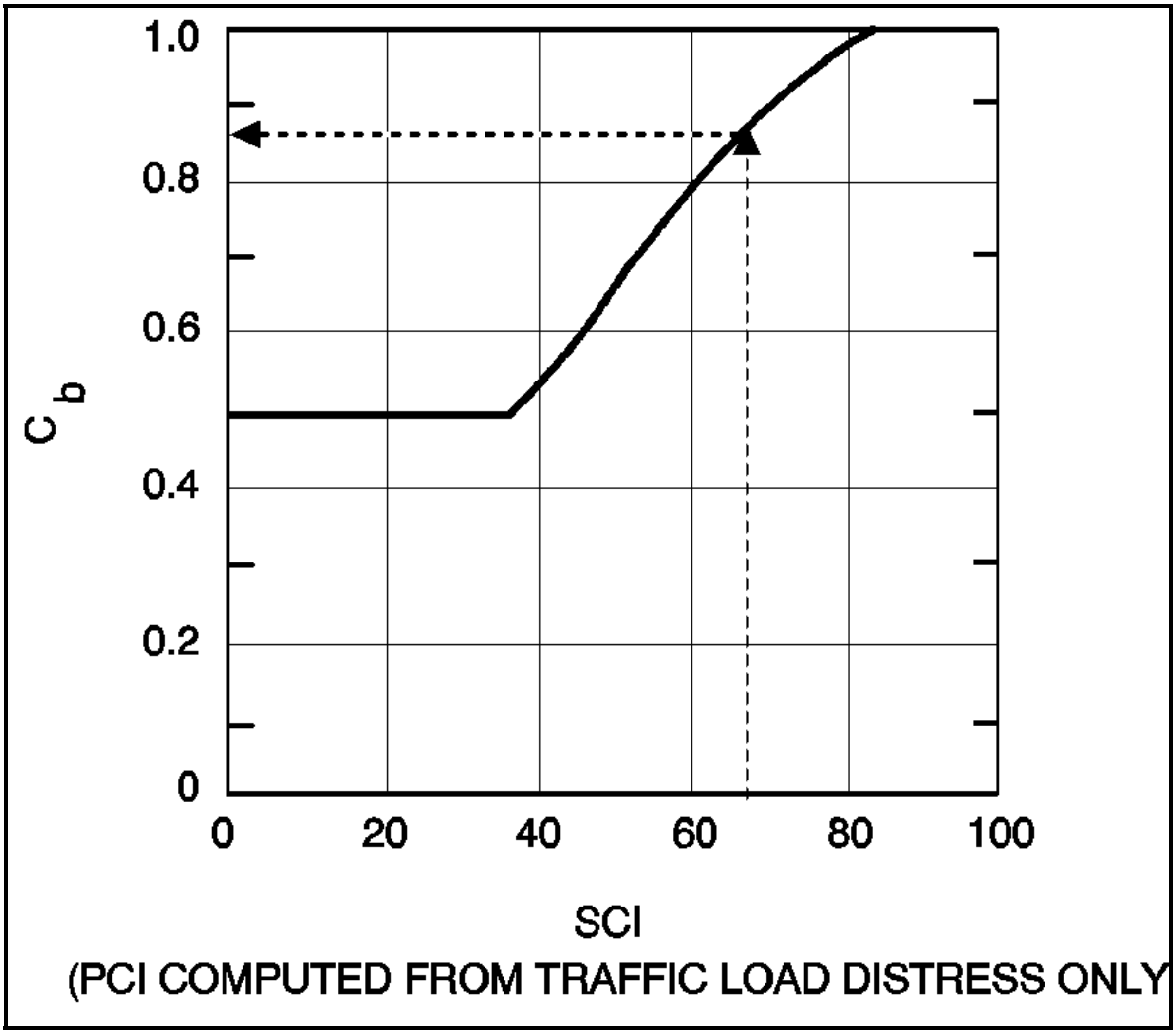


Figure 6-38. Condition Factor (C_r) for determining Equivalent Thickness of Rigid Overlays on Rigid Pavements

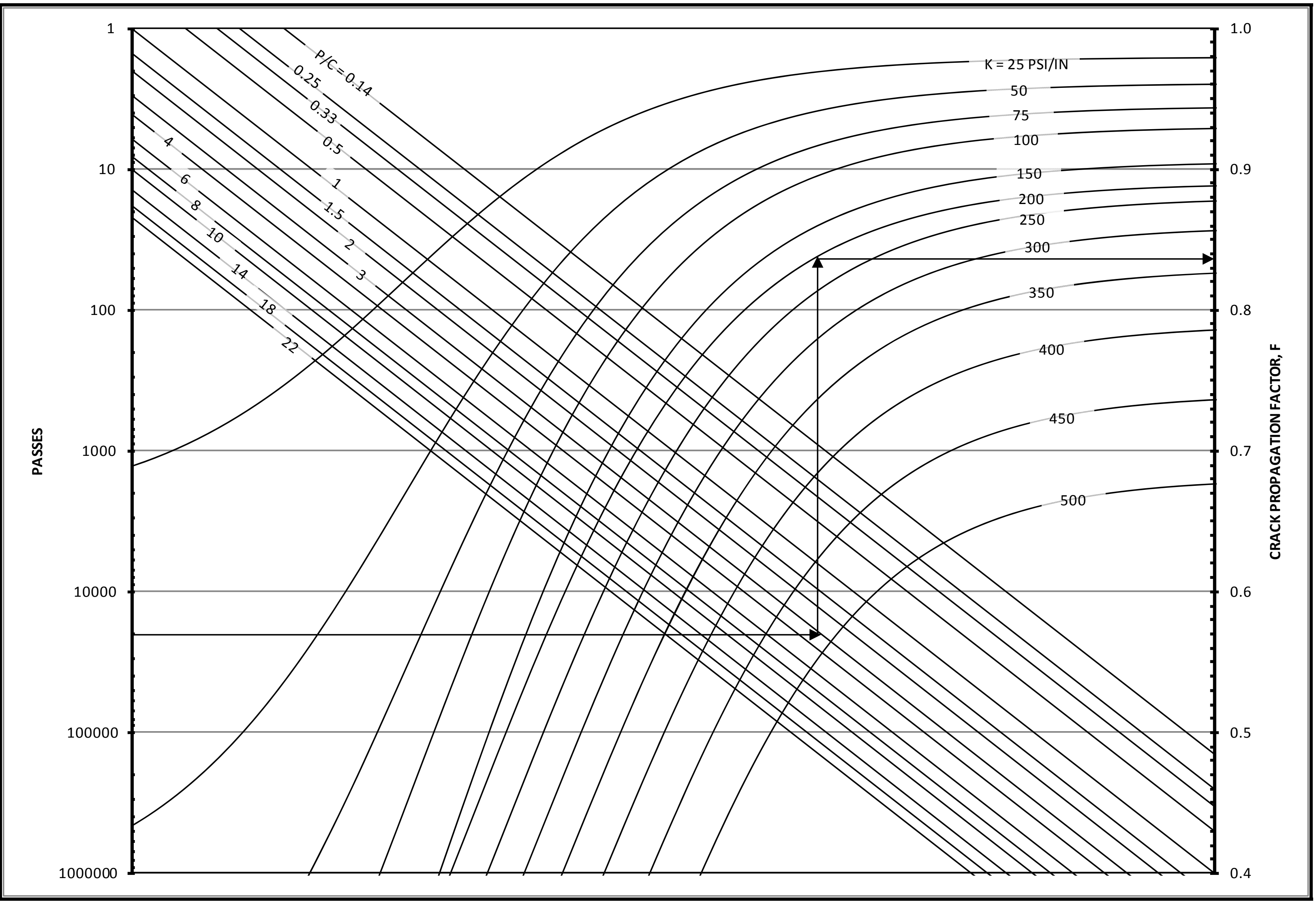


Figure 6-39. Crack Propagation Factor, F, for Nonrigid Overlays

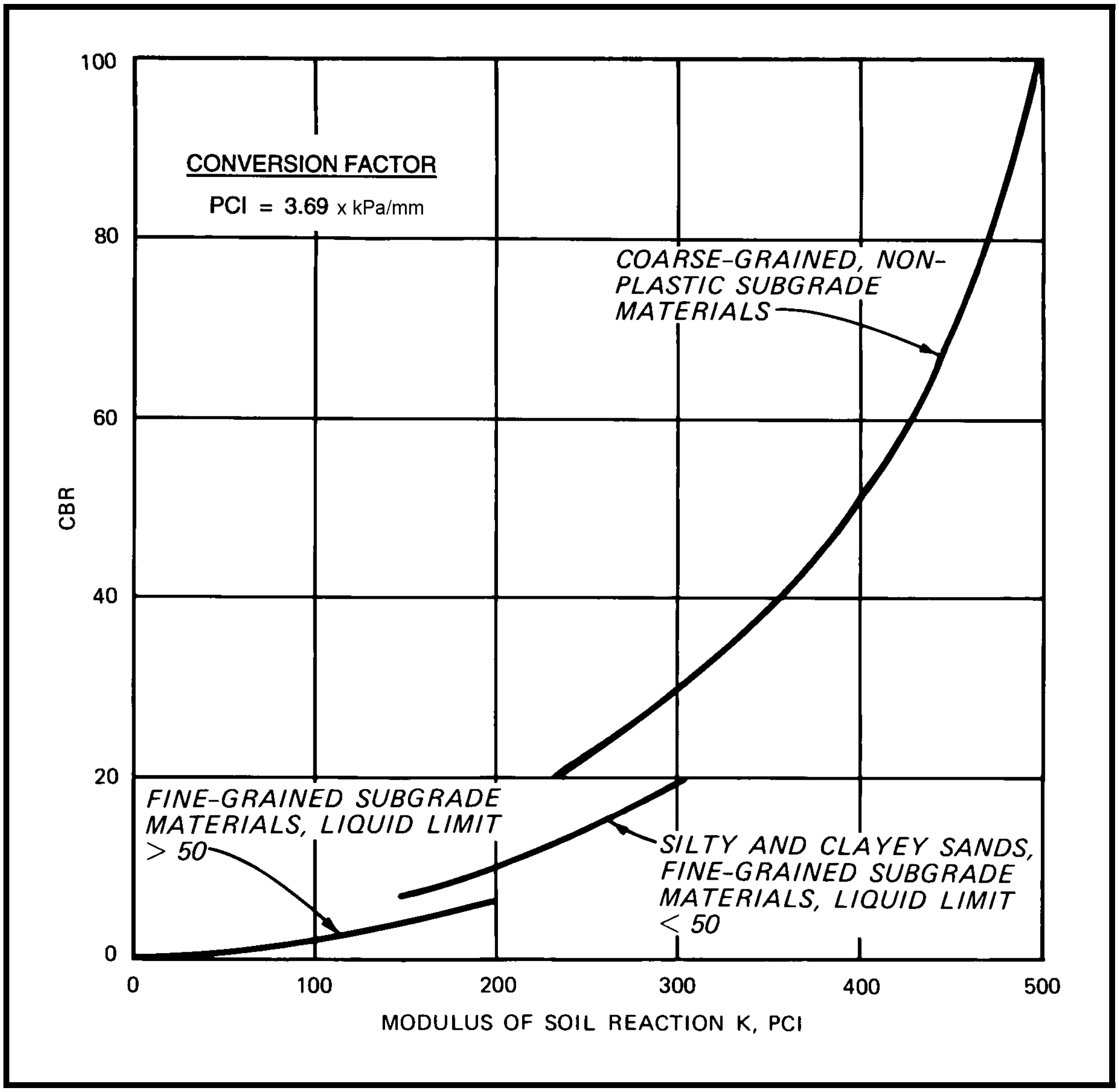


Figure 6-40. CBR/K Conversion

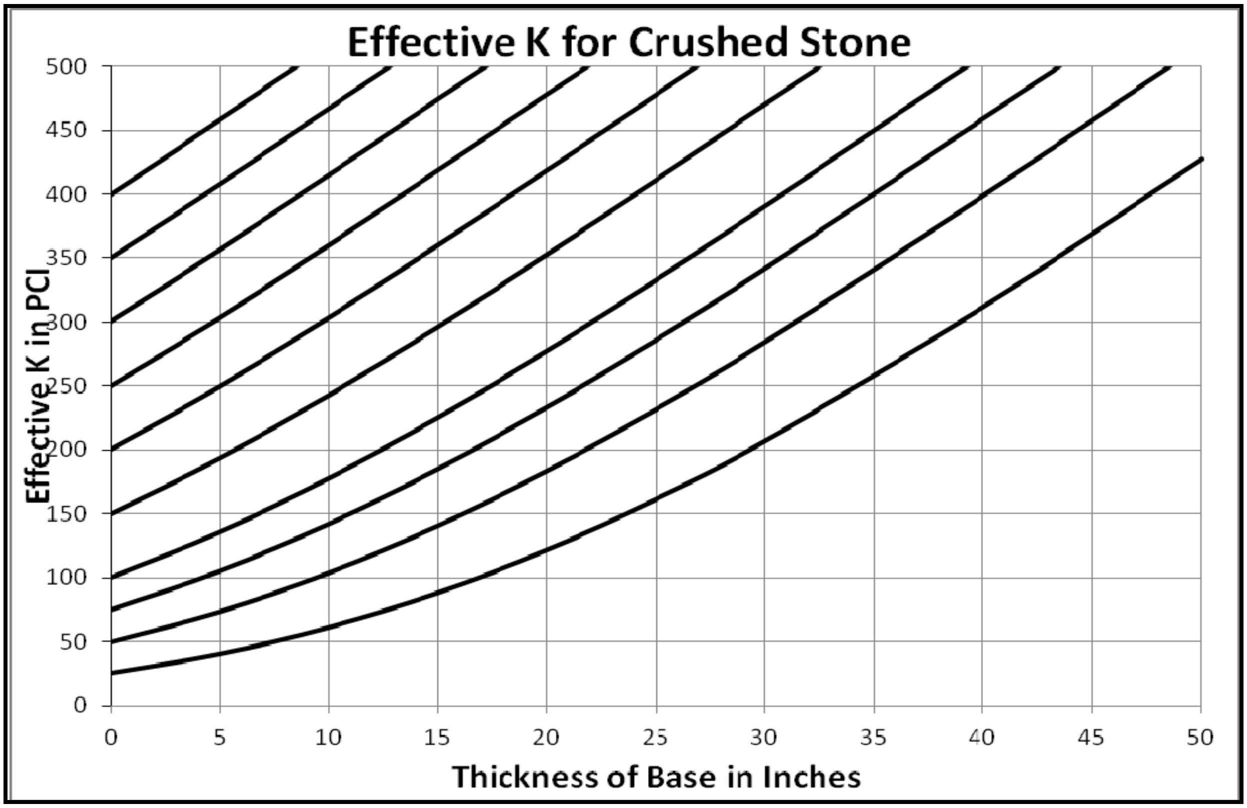


Figure 6-41. Effective K for Soil Layer Strength, CBR≥90

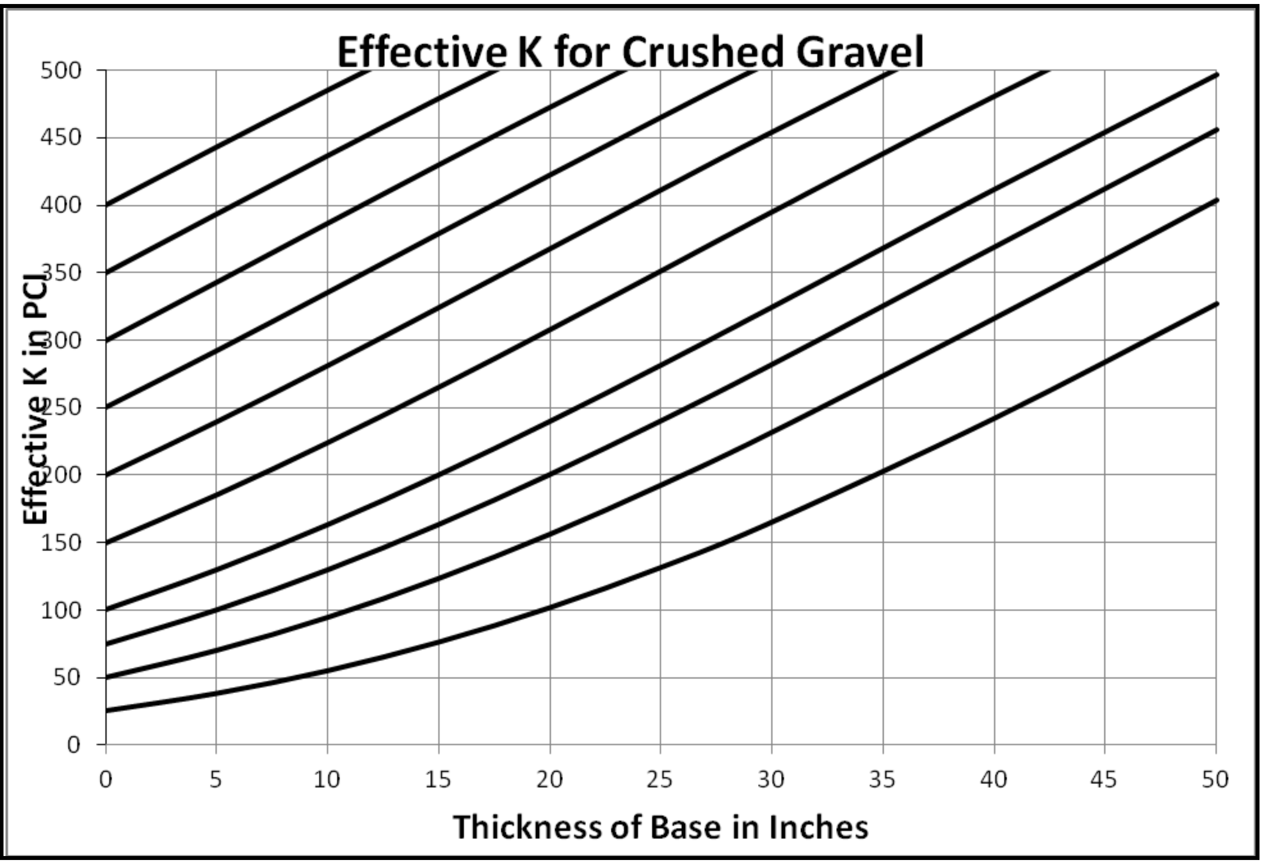


Figure 6-42. Effective K for Soil Layer Strength, 70≤CBR<90

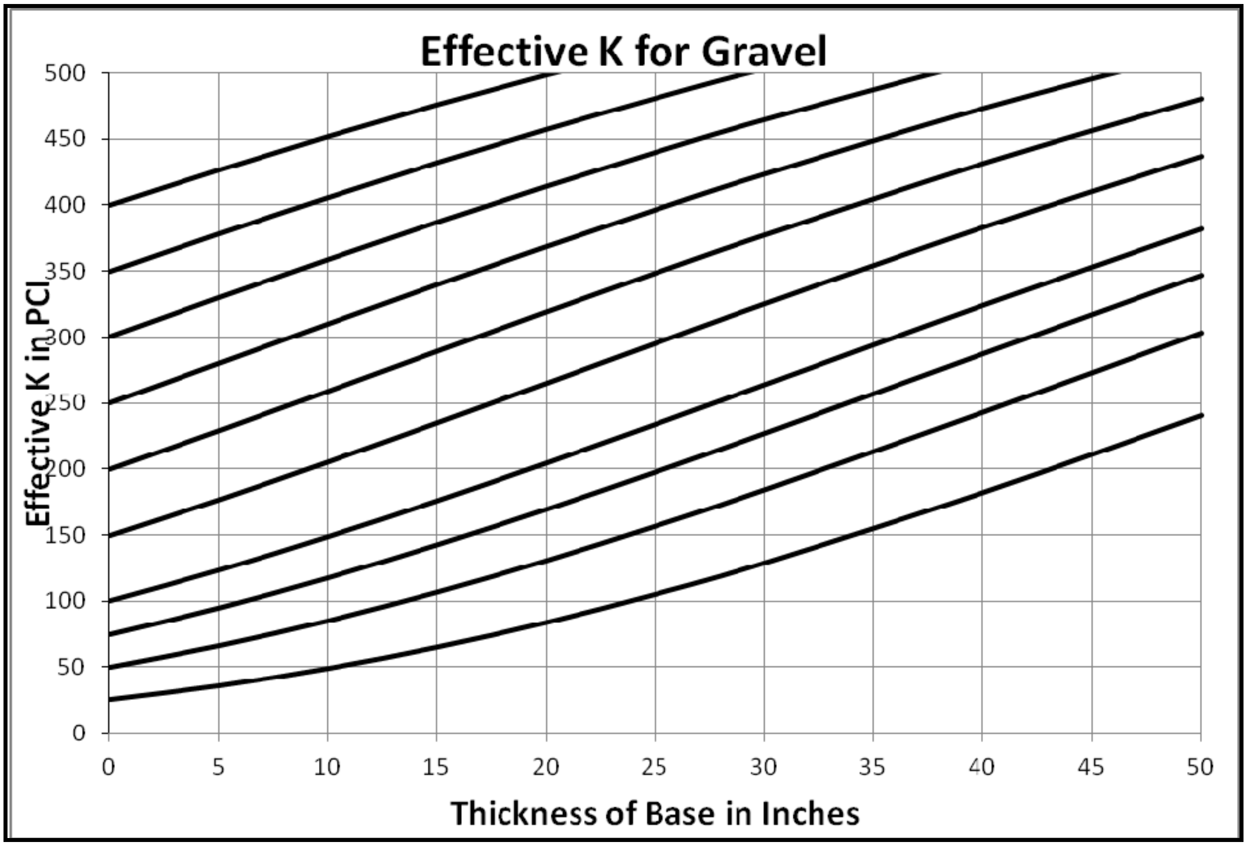


Figure 6-43. Effective K for Soil Layer Strength, 50≤CBR<70

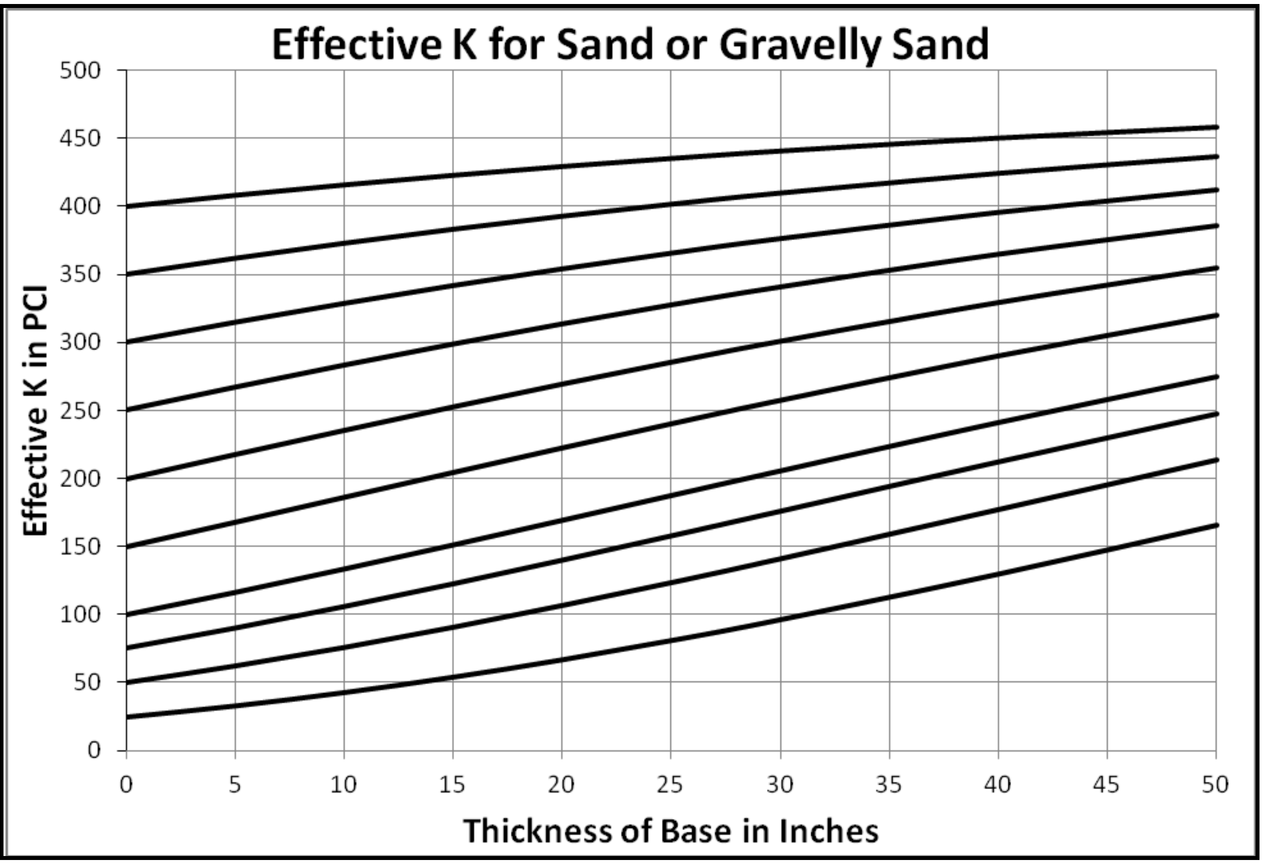


Figure 6-44. Effective K for Soil Layer Strength, < 50 CBR

CHAPTER 7

PAVEMENT EVALUATION FOR FROST CONDITIONS

7-1 **GENERAL**. This chapter presents criteria and procedures for the evaluation of airfield pavements in seasonal frost areas. If the existing base, subbase, and/or subgrade soils under the pavement structure are susceptible to detrimental frost action during part of the year, then the bearing capacity of the pavement structure will be less than if the same soil conditions existed in a nonfreezing environment. The conditions required for detrimental frost action are freezing temperatures, frost susceptible soils, and a source of water near the freezing front. The emphasis of the evaluation is in the reduction of the bearing capacity during thaw-weakening periods. The reduction in load-carrying capability develops as the soil structure changes and the melting of the ice releases an excess of water that does not readily drain or redistribute itself, thus softening the soil. Recovery from the softened condition comes about initially as a process of reconsolidation and dissipation of pore water pressure, followed by progressive desaturation and buildup of moisture tension, which stabilizes the soil. If such conditions conducive to detrimental frost effects exist, then the evaluation will be made up of two parts; normal period and period of weakening. The first will be based on normal, nonfreezing conditions and will be applicable to that period of the year during which the pavements are not affected by thawing of the base, subbase, or subgrade. The second, applicable to the thaw-weakening period, will be based on subgrade strengths using FASSI and FAIR values or reduced moduli values as prescribed in this chapter. Evaluations of airfields during thaw-weakening periods will use pass intensity levels identified in Chapter 2.

7-2 **FROST CONDITION TERMINOLOGY**. The following terms are used in this chapter.

a. Frost Action. A general term for freezing and thawing of moisture in materials and the resultant effects on these materials and on structures of which they are a part, or with which they are in contact.

b. Frost Susceptible Soil. Soil in which significant detrimental ice segregation will occur when the requisite moisture and freezing conditions are present. These soils will lose a substantial portion of their strength upon thawing.

c. Nonfrost Susceptible Materials. Cohesionless materials such as crushed rock, gravel, sand, slag, and cinders that do not experience significant detrimental ice segregation under normal freezing conditions. Cemented or stabilized materials that do not experience significant detrimental ice segregation, loss of strength upon thawing, and freeze thaw degradation are also considered to be nonfrost susceptible materials.

d. Frost Heave. The raising of the pavement surface due to formation of ice lenses in the underlying soil.

e. Frost-melting (Thawing) Periods. Intervals of the year when the ice in the base, subbase, and/ or subgrade returns to a liquid state. A period ends when all the ice in the ground has melted or when the previously frozen material is refrozen. In general, there may be several significant frost-melting periods during the winter months prior to the spring thaw.

f. Periods of Weakening (Thaw-weakening Periods). Intervals of the year when the base, subbase, and/or subgrade strength is below its normal summer values. These intervals correspond to frost- melting periods. The period ends when either the material is refrozen or when the subgrade strength has returned to the normal summer value at the end of the spring thaw-weakening period, Figure 7-1.

g. Critical Weakening Period. Interval during the period of thaw weakening when the base, subbase, and/or subgrade strength is at its lowest strength, Figure 7-1.

h. Recovery Period. Interval from the end of the critical weakening period to the beginning of the normal period. During this time the base, subbase, and/or subgrade strength is recovering to normal strength from lowest strength, Figure 7-1.

i. Normal Period. Interval during the year when the base, subbase, and/or subgrade strength is at its nonfrost strength, Figure 7-1.

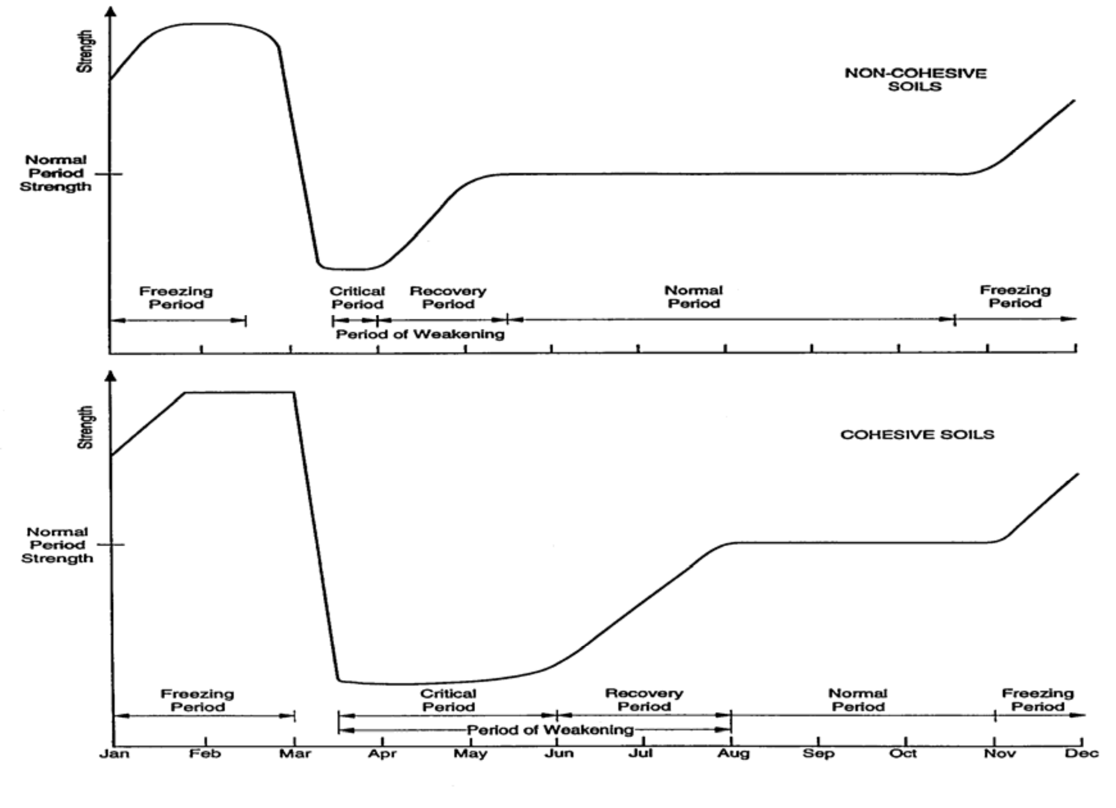


Figure 7-1. Illustration of Thaw-Weakening Period

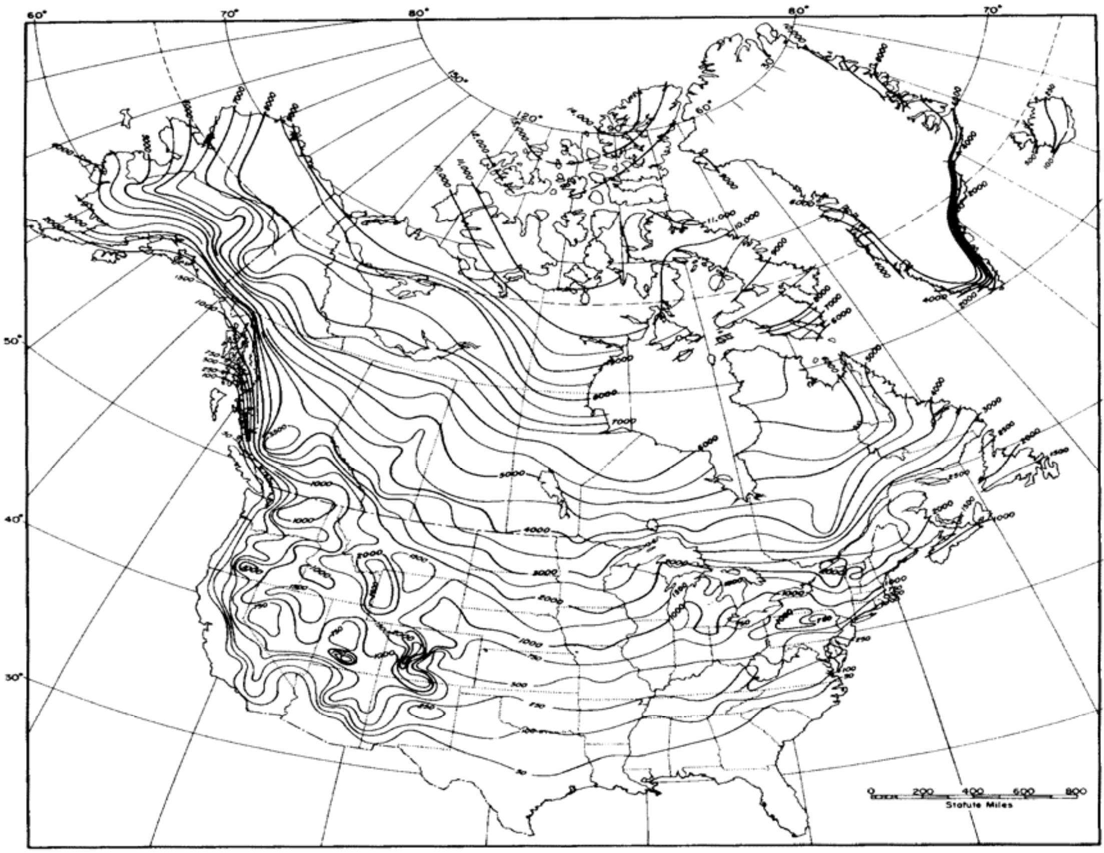
j. Average Daily Temperature. The average of the maximum and minimum temperatures for one day, or the average of several temperature readings taken at equal time intervals, generally hourly, during a day.

k. Mean Daily Temperature. The mean of the average daily temperatures for a given day, usually calculated over a period of several years.

l. Degree-Days. The Fahrenheit degree days for any given day equal the difference between the average daily air temperatures and 0 °C (32 °F). The Centigrade degree hours for any given day equal the average daily temperatures (°C) multiplied by 24 hours. The degree-days or degree-hours are negative when the average daily temperature is below 0 °C (32 °F) (freezing degree-days or hours) and positive when above (thawing degree-days or hours). Usually, the degree-days or hours are reported in terms of their absolute values and the distinction is made between freezing and thawing.

m. Freezing Index. The number of degree-days between the highest and lowest points on a curve of cumulative degree-days versus time for one freezing season. It is used as a measure of the combined duration and magnitude of below-freezing temperatures occurring during any given freezing season. The index is determined from air temperatures measured approximately 4.5 feet above the ground and is commonly designated as the air freezing index.

n. Design Freezing Index. The average air freezing index of the three coldest winters in the latest 30 years of record. If 30 years of record are not available, the air freezing index for the coldest winter in the latest 10-year period may be used. The design freezing index at a site need not be changed more than once in 5 years unless the more recent temperature records indicate a significant change in thickness requirements for frost protection. The design freezing indexes for North American locations are presented in Figure 7-2 in degree F-days.

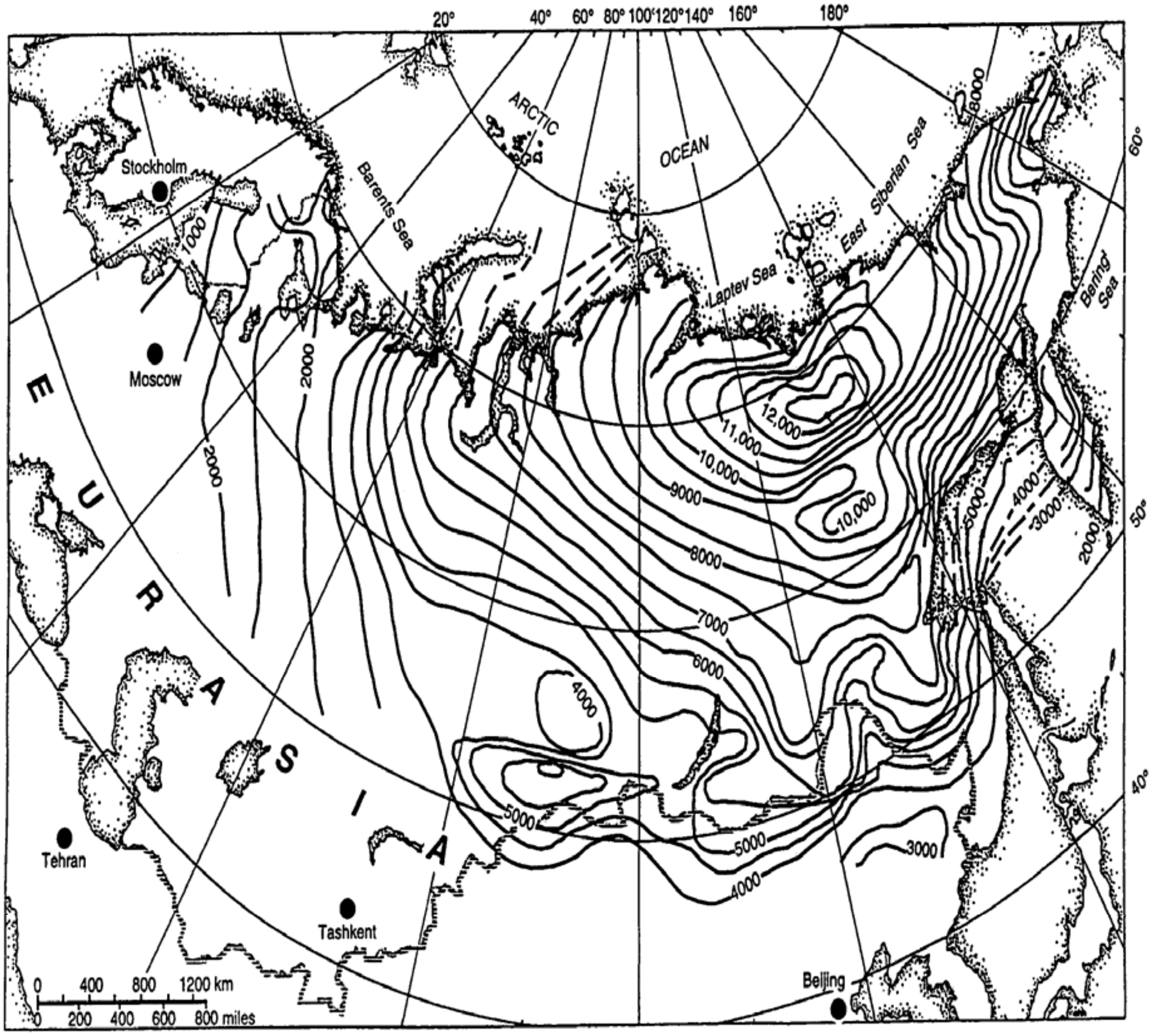


CONVERSION FACTOR

DEG F DAYS = 0.075 × DEG C HOURS

Figure 7-2. Distribution of Design Air Freezing Indices in North America

o. Mean Freezing Index. The freezing index determined on the basis of mean daily temperatures. The period of record over which average daily temperatures are averaged is usually a minimum of the latest 10 years, preferably 30. Mean freezing indexes for northern Eurasia are presented in Figure 7-3 in degree F-days. Design freezing indices are not available for Eurasia, but can be estimated. (See Paragraph 7-9).



CONVERSION FACTOR

DEG F DAYS = 0.075 × DEG C HOURS

Figure 7-3. Distribution of Mean Air Freezing Indices in Northern Eurasia

p. Combined Base Thickness. The combined thickness of base, subbase, drainage layer, and separation layer.

q. Frost Area Soil Support Indices (FASSI). The weighted average of CBR values for the annual cycle that is generally illustrated in Figure 7-1. These values are used in flexible pavement evaluation for the frost-melt period, as if they are true CBR values. FASSI values are shown in Table 7-3.

r. Frost Area Index of Reaction (FAIR). The weighted average of k values for the annual cycle. These values are used for rigid pavement evaluation for the frost-melt period, as if they are true k values. FAIR values are shown in Figure 7-6.

7-3 **FROST EFFECTS**. The detrimental effects of frost action are frost heave and thaw weakening. Frost heave, manifested by the raising of the pavement surface, is directly associated with ice segregation and is visible evidence on the surface that ice lenses have formed in the subgrade, subbase, and/ or base- course materials. Depending on variations in exposure to solar radiation or in the character of the soil and ground-water conditions underlying the pavement, heave can be uniform or nonuniform. Nonuniform heave results in unevenness or abrupt changes in grade at the pavement surface. If such conditions are noted by the evaluation team, or are reported by flight or other personnel, the location and description of the objectionable roughness will be included in the evaluation report.

a. When ice segregation has taken place in a frost susceptible soil, the soil is subsequently weakened during prolonged frost-melting periods, as during winter partial thaws and early in the spring. The melting of segregated ice leads to excess water in the base/subbase and/or subgrade that cannot drain through the still-frozen underlying soil. Drainage could also be restricted laterally at this time of the year; thus the period of severe weakening may last several weeks. Presence of drainage layers in the pavement structure should decrease this period of severe thaw weakening.

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

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

7-4 **CRITICAL WEAKENING PERIOD**. The critical weakening period comes during the early stages of frost-melting and may occur intermittently during the winter, when the segregated ice in the base, subbase and subgrade is melting. This critical period can last from 1 week to several months, depending on the soil type and environmental conditions. As the soil drains and reconsolidates, the pavement regains much of its lost strength. With the subsequent gradual desaturation and the corresponding buildup of moisture tension in the affected soils, the pavement gradually regains full normal-period bearing capacity. The length of the recovery period varies from a few weeks to several months, depending on the intensity of ice segregation, the depth of frost penetration, the rate of thawing, the permeability of the soil, the drainage conditions, precipitation, and atmospheric humidity. The performance of highways with a comparable subgrade in the vicinity of the airfield may be an indicator of the likely duration of the critical period, however since airfield pavements are wider and drainage paths longer, the thaw-weakened period is also likely to be longer.

7-5 **EFFECT OF FROST ACTION ON PAVEMENT SURFACE**. The most obvious structural effect of frost action on the pavement surface is random cracking and roughness as the result of differential frost heave. Studies of rigid pavements have shown that cracks may develop more rapidly during and immediately following the spring frost-melting period as a result of differential thaw than during the period of active heave. Deterioration and spalling of the edges of open cracks is a source of debris that is a potential cause of Foreign Object Damage (FOD) to aircraft engines. Cracks in flexible pavements may also be the result of contraction of the pavement during periods of extremely low temperatures. The effect of thaw weakening of subgrades and base courses may be more severe than cracks caused by frost heave or low-temperature contraction because it leads to destruction of the pavement, requiring reconstruction. Its effect is felt through a process of greatly accelerated cumulative damage to the pavement under successive traffic loads. Eventually, the accumulation of damage leads to visible surface cracking. This cracking may not become visible during frost melting. As a result, thaw weakening may not always be recognized as the dominant factor causing accelerated failure.

7-6 **MAGNITUDE OF SUBGRADE WEAKENING**. The load-bearing capacity of both flexible and rigid pavements can be severely reduced during critical weakening periods, however, the reduction is less critical for rigid than for flexible pavements. Rigid pavements experience a smaller reduction because the subgrade has less influence on the supporting capacity of rigid pavements than on that of flexible pavements. Subgrade soils under rigid pavements are subjected to less shearing deformation and remolding during critical weakening periods.

7-7 **RECOGNITION OF POTENTIAL FOR DETRIMENTAL FROST ACTION**. There are several ways to recognize either existing or potential frost action on pavements.

a. Visible surface effects associated with frost action include pavement heave and cracking during the freezing season and noticeable weakening or deflection during the frost-melting period. Pavements that are experiencing accelerated distress because of thaw weakening may also show alligator cracking or other load-associated cracking at an early age. Pumping may take place at cracks and joints. During pavement inspections, particular attention should be given to locations of transitions between cuts and fills and also at any boundaries of subgrade soils of varying frost susceptibility. D cracking is a common indication of freeze-thaw damage to PCC pavements, but is primarily associated with aggregates of poor quality in the concrete mixture. These are closely spaced crescent-shaped cracks that occur adjacent to longitudinal and transverse joints or free edges.

b. The construction, maintenance, and previous evaluation records of the airfields may help in confirming whether or not frost-susceptible conditions exist. Records of highway performance in the vicinity of the airfield that have similar subgrade conditions may provide a clue as to whether weakening occurs as a result of frost melting. In the analysis of highway performance records, the evaluator should carefully note and assess the many local influences that may affect frost action, such as variations in ground-water level, soil conditions, type of pavement surface, degree of shading, north versus south slope, frequency of snow plowing, position of underlying bedrock, etc.

c. Supplementary field and laboratory investigations to determine if detrimental ice segregation and thaw weakening are likely to occur in the base course, subbase course, or subgrade should be made, in addition to the basic investigations specified in chapter 3. With time, base and subbase materials can become degraded due to freeze-thaw cycles and traffic loads. The degradation may introduce additional fines, thus increasing its thaw-weakening potential. Before rehabilitation, the gradation and frost susceptibility of the base/subbase material should be determined and compared with the original as-constructed classifications. If any of the materials classify as possibly frost susceptible (PFS), a laboratory frost susceptibility test should be conducted to properly classify the material to estimate its strength during thawing periods. At the time of maximum heave, the surface roughness of pavements constructed over F4 subgrade soils, and in some instances over F3 soils, may be objectionable for aircraft with high landing and takeoff speeds. If experience indicates this is the case, it should be indicated in the evaluation report, and the report should include the locations and descriptions of the objectionable roughness. Surface elevations should be obtained at least once a month during the following winter to determine the magnitude of the detrimental heave.

7-8 **PAVEMENT EVALUATION-GENERAL**. The general procedure for pavement evaluation in cold regions is illustrated in Figure 7-4. Pavements in seasonal frost areas are evaluated using a stepwise procedure. The first step is to determine if the pavement structure is completely protected from frost action. If it is not, the second step is to determine if the thickness is adequate for limited subgrade frost penetration; if not, the third step is to apply the reduced subgrade strength procedure for the pavement evaluation or reduced modulii for NDT evaluation. The Services may vary the procedure based on their experience. Standard pavement evaluations conducted by DOD normally do not include step 2, limited frost penetration. If the pavement thickness is adequate for complete protection or limited subgrade frost penetration and no effects of frost action are apparent, the pavement is evaluated using nonfrost criteria. If any pavement feature evaluated at an airfield is adequately protected against frost action, a discussion to that effect will be included in the text of the report. Appropriate notes should also be included in tables of the report.

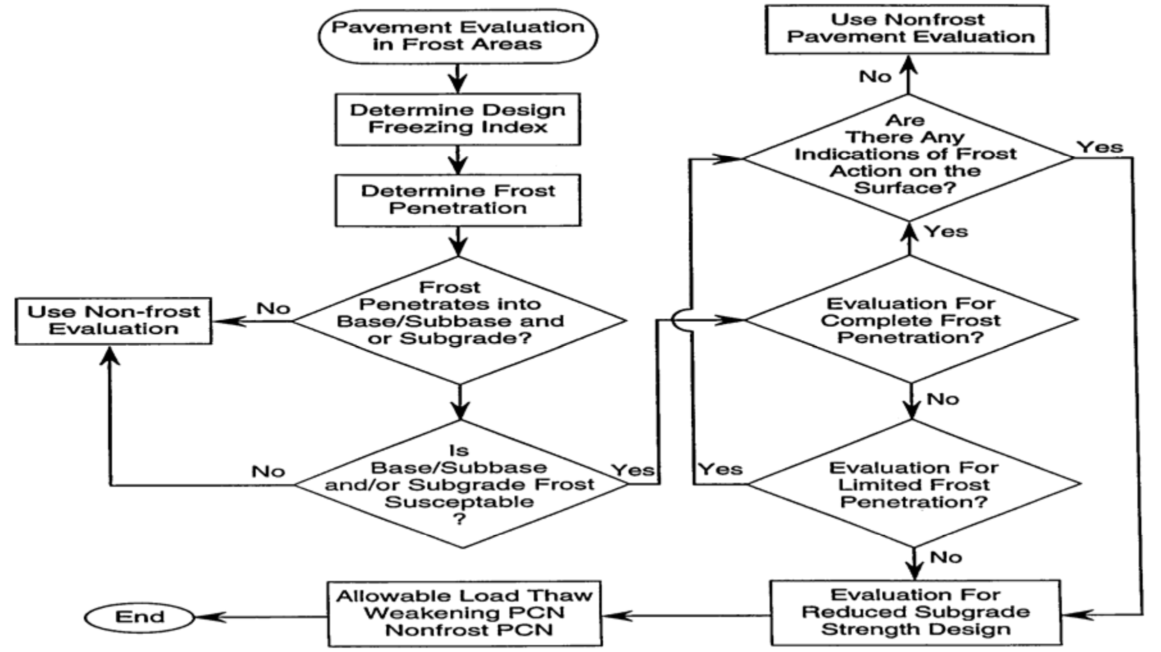


Figure 7-4. Pavement Evaluation in Frost Areas

Most permanent installations have considerable data on frost susceptibility. This data is thoroughly reviewed before the field evaluation. As indicated in Figure 7-4, If there is no indication of frost susceptibility in the reports and no visual evidence of frost action, the non-frost evaluation is used. If there is evidence of frost action, the reduced subgrade procedure or reduced moduli procedure is used for the frost melt period.

7-9 **DETERMINE DESIGN FREEZING INDEX (DFI)**. The DFI is the average air freezing index of the three coldest winters in the last 30 years of record. If 30 years of record are not available, the air freezing index for the coldest winter in the last 10-year period may be used. If either data sets are not available, an approximate freezing index may be obtained from the map in Figure 7-2 showing design air freezing indices for locations in North America. Special consideration will be necessary to compensate for local topographic conditions that will cause deviations from general freezing index values shown on this map; assistance for this adjustment can be obtained through Headquarters, U.S. Army Corps of Engineers (USACE/TSC), the appropriate Air Force Major Command, or the appropriate NAVFAC Headquarters. DFI for sites in Eurasia can be roughly estimated from the mean freezing indices in Figure 7-3 and using the following equation. The mean freezing index from Figure 7-3 must be multiplied by 13.33 to convert from °F days to °C hours. This equation could also be used at other sites where the mean freezing index is known. The Design Air Freezing Indices are also provided in the PCASE computer program within the depth of frost calculator, see paragraph 7-20 for details on using PCASE the desktop.

English Units

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

(Eq 7-1)

SI Units

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

7-10 **DETERMINE FROST SUSCEPTIBILITY OF BASE, SUBBASE, AND SUBGRADE LAYERS**. Determine if the base/subbase and/or subgrade is frost susceptible. Table 7-1, will be used to identify the frost susceptibility of the soil. Soils are listed in approximate order of increasing frost susceptibility and decreasing bearing capacity during periods of thaw.

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

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

Table 7-1. Frost Susceptibility Soil Classification

7-11 **EVALUATE PAVEMENT FOR COMPLETE FROST PROTECTION**. The total pavement thickness required to prevent freezing into the subgrade with respect to the design freezing index is determined from the PCASE computer program. Frost penetration depths determined from PCASE are measured from the pavement surface, which must be free of snow and ice during the winter. If the depth of frost penetration exceeds the thickness of surface and combined base and subbase, the pavement is not protected from frost and should be evaluated for frost effects. Instructions for using the depth of frost calculator in PCASE are shown in Examples 1, 2 and 3 in paragraphs 7-16, 17, and 18. Refer to Appendix H for information on obtaining the software.

$$c = d - p \qquad \text{(Eq 7-2)}$$

where:

c = thickness of combined base layers necessary for complete frost protection

d = thickness of pavement and combined base for complete frost protection (from PCASE)

p = thickness of surface layer

x = actual/existing thickness of combined base layers

a. Determine whether the combined base thickness (x) under the pavement being evaluated is sufficient to protect the subgrade from freezing. This is accomplished by comparing (x) with (c).

b. If ($x < c$), the evaluated pavement structure is inadequate for complete frost protection. If there are no indications of frost action, then evaluate the pavement structure for limited subgrade frost penetration. If there are indications of frost action, then evaluate the pavement structure with the reduced subgrade strength approach described below.

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

7-12 **EVALUATE PAVEMENT FOR LIMITED SUBGRADE FROST PENETRATION**. Determine if the combined base thickness under the evaluated pavement structure (x) is sufficient for limited frost penetration into the subgrade.

a. For limited frost penetration into the subgrade, estimate the average moisture content of the subgrade during nonfrost conditions. Compute water content ratio r. Use the same base-course water content as that assumed in frost penetration calculations.

$$r = \frac{\omega_{subgrade}}{\omega_{base}} \qquad \text{(Eq 7-3)}$$

b. If the computed r exceeds 2.0, use 2.0 for type A or primary B traffic areas. If r exceeds 3.0, use 3.0 for all pavements except those in type A, B, or primary traffic areas. Either use Figure 7-5, with c (equation 7-2) as the x coordinate and, at the applicable value of r, find the base/subbase (include drainage layer(s) thickness b for limited frost penetration into the subgrade or use equation 7-4.

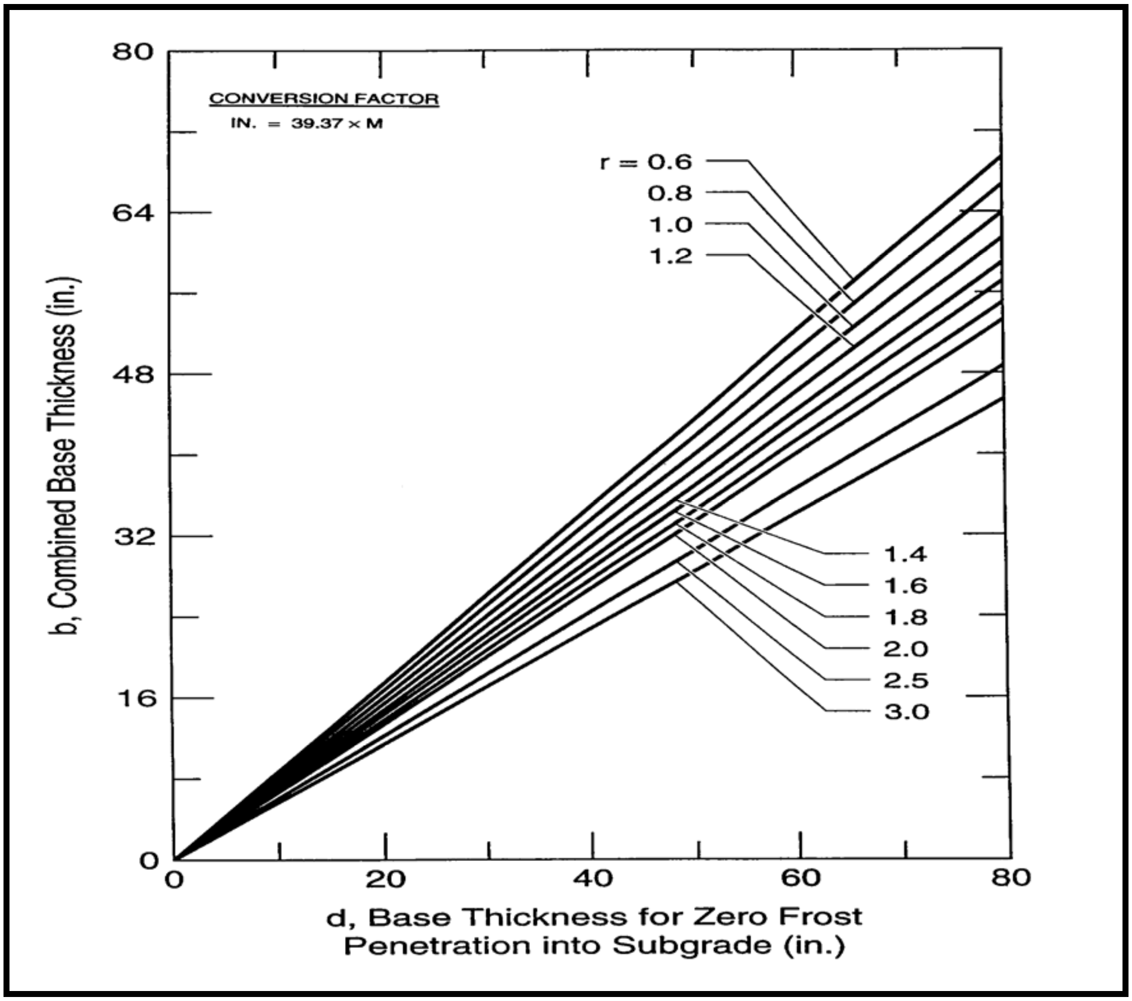


Figure 7-5. Estimation of Combined Base for Limited Subgrade Frost Penetration

If the base/ subbase thickness (x) at the evaluated site is $\geq$b or $\geq$ 60 inches minus the pavement thickness, the pavement is adequately protected against detrimental frost action.

$$b = c\,(f) \qquad \text{(Eq 7-4)}$$

where:

b = combined base thickness for limited subgrade frost penetration

f = factor from Table 7-2

c = thickness of combined base layers necessary for complete frost protection

Water Content Ratio (r)	**f**
0.6	0.881
0.8	0.850
1.0	0.806
1.2	0.781
1.4	0.756
1.6	0.725
1.8	0.706
2.0	0.644
2.5	0.613
3.0	0.550

Table 7-2. f Values for Different Water Content Ratios

c. Check the surface for any indications of frost action. If there are no indications of frost action, then use the nonfrost evaluation method. Otherwise evaluate the pavement structure with the reduced subgrade strength approach discussed below.

d. If all the pavements being evaluated at an airfield are adequately protected against frost action, or if the airfield is located where frost is not a problem, a note to that effect will be placed at the bottom of the summary.

7-13 **EVALUATE PAVEMENT FOR REDUCED SUBGRADE STRENGTH**. If determined that a pavement is not adequately protected against detrimental frost action, the procedures described below will be used in making frost evaluations. The frost evaluation will be based on the reduced strength of the subgrade, using FASSI or FAIR values as described below. Such evaluation will be modified, as appropriate, based on pavement performance history. At the time of maximum heave, the surface roughness of pavement constructed over F4 subgrade soils, and in some instances over F3 soils, may be objectionable for aircraft with high landing and takeoff speeds. If experience indicates this is the case, this fact should be indicated in the evaluation report, including the locations and descriptions of the objectionable roughness. Surface elevations should be obtained at least once a month during the following winter.

a. The allowable gross load allowed during thaw-weakening periods is based on the assumption that flight operations are continued at the same frequency in effect during the rest of the year. Allowable gross loads for flexible pavements during the thaw-weakening period are determined by using FASSI values with the evaluation curves in chapter 5 or the PCASE computer program. The applicable FASSI values for the various frost groups of subgrade soils are shown in Table 7-3. The FASSI values are used as if they were California Bearing Ratio (CBR) values with the evaluation curves; the term CBR is not applied to them, however, because being weighted average values for the annual cycle, their values cannot be determined by CBR tests.

Frost Group of Subgrade Soil	**F1**	**F2**	**F3 and F4**
Frost-Area Soil Support Index (FASSI)	9	6.5	3.5

Table 7-3. FASSI Values for Various Frost Susceptibility Soils

b. Allowable gross loads on rigid pavements during the thaw-weakening period are determined by using FAIR values with the evaluation curves in Chapter 6 or PCASE. FAIR values can be estimated from Figure 7-6. The curves in Figure 7-6 show the equivalent weighted average FAIR values for an annual cycle that includes a thaw-weakening period in relation to the thickness of the combined base.

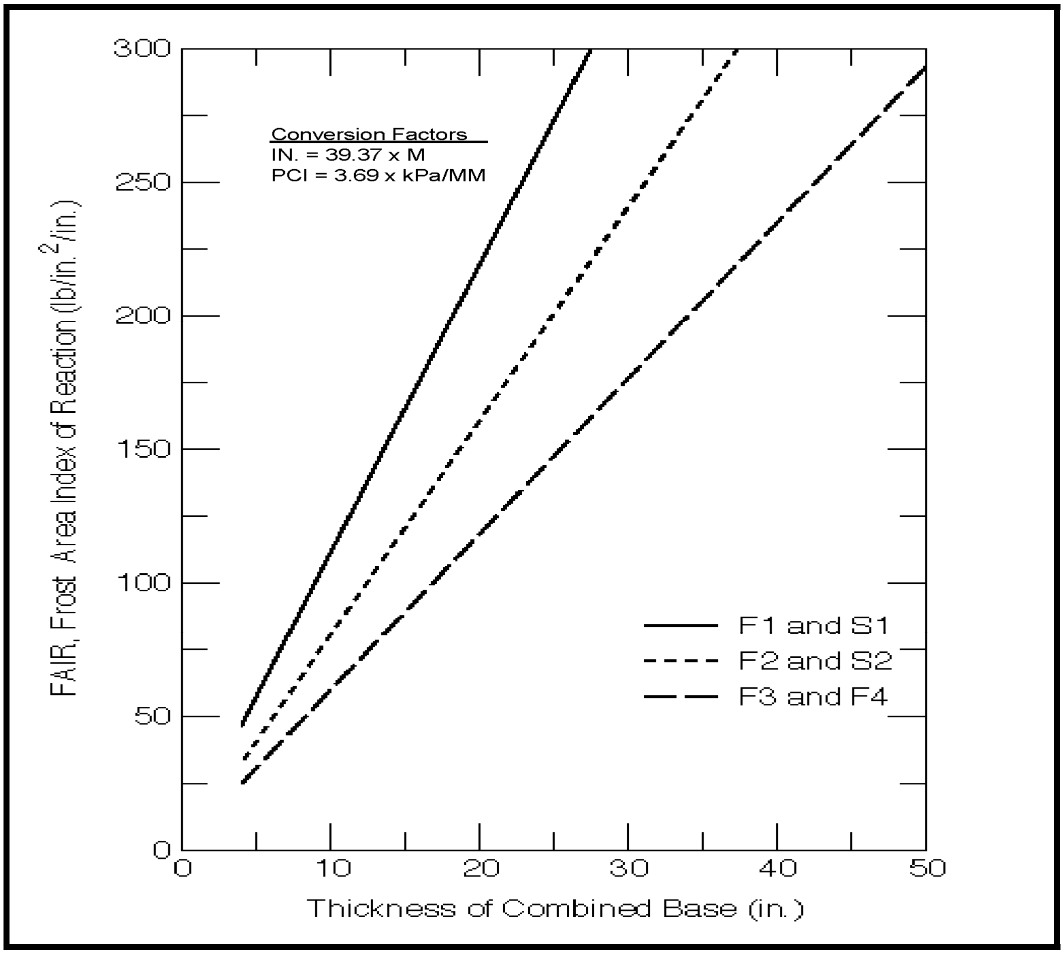


Figure 7-6. Determination of FAIR Value

The FAIR values can also be estimated from the following equations:

<u>English Units</u>

S1 or F1 material: *FAIR* ((*psi*/*in*.) = 4.2 + 10.8 × Combined *Base Course Thickness* (*inches*) (eq 7-5)

S2 or F2 material: *FAIR* ((*psi*/*in*.) = 1.3 + 8.0 × Combined *Base Course Thickness* (*inches*) eq 7-6)

F3 or F4 material: *FAIR* ((*psi*/*in*.) = 1.6 + 5.9 × Combined *Base Course Thickness* (*inches*) (eq 7-7)

FAIR values are used as if they were modulus of soil reaction values, k, and have the same units. The term modulus of soil reaction is not applied to them; however, because being weighted average values for an annual cycle, they cannot be determined by a plate-bearing test. If the modulus of soil reaction k, determined from tests on the equivalent base course and subgrade, but without frost melting, is numerically smaller then the FAIR value obtained from Figure 7-6, the test value should be used in the evaluation.

7-14 **REDUCTION FACTORS FOR NONDESTRUCTIVE TESTING**. The moduli of the subgrade during thaw periods are reduced modulus values obtained during the nonfrost period. The Air Force uses the Reduction Factors (RF) in Table 7-4. These reduction factors are to be used as guides. If subgrade modulus values are available for the thaw period, these values will be used.

$$Thaw\ Modulus = RF * Nonfrost\ Period\ Modulus$$

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

Table 7-4. Modulus Reduction Factors for Seasonal Frost Areas

The Army and Navy convert the FASSI values in Table 7-3 and the FAIR values in Figure 7-6 to modulus values and use these in lieu of Reduction Factors.

7-15 **EVALUATION METHODOLOGY**. The evaluation methodology requires the determination of allowable loads, allowable number of passes and PCN's to be reported for both thaw-weakened and normal periods. Using this dual reporting system, PCN's are reported for both the thaw-weakened and normal periods. The procedure utilizes the FASSI/FAIR or reduced modulus values for layer strengths during the thaw-weakened condition and measured material strengths during the normal period. Material properties for the normal period must be determined when the pavement has fully recovered from a thaw-weakened condition. Strengths of the pavement materials may be based on direct sampling or nondestructive testing. The evaluations are made for Pass Intensity Levels I and II for Air Force pavements. The PCN is determined for 50,000 passes of a C-17 for Air Force pavements and for a C-130, C-17, or the critical aircraft for Army and Navy Airfields. Substantial pavement overloads may be allowed during the period that the pavement is solidly frozen. The amount of overload and the period that the overload may be applied must be obtained from Headquarters, U.S. Army Corps of Engineers (HQUSACE/TSC) the appropriate Air Force Major Command, or the appropriate NAVFAC Headquarters.

a. Evaluation Periods. The duration of the period of weakening and the normal period must be determined and included in the evaluation report. The beginning and ending dates for each of the two periods must also be included. Since a number of frost-melting periods may occur during a typical winter period, it is essential that all periods of thaw weakening be included in the computation of the total period of weakening. The time required for strength recovery following a thaw will vary depending on local conditions. Principal factors affecting the recovery time are depth of frost penetration, type of frost-susceptible material, and subsurface drainage. Normally, the time for recovery will be from several weeks to several months. The thaw-weakened periods for different frost-susceptible soils are presented in Table 7-5. This table is to be used as a guide; the length of the thaw-weakened period can be changed based on local experience. The total period of weakening must also include frost-melting periods during the winter; the following will be used to establish those periods:

(1) If DFI≤ 1,000-°F days, one-half of the length of the freezing season will be included in the total period of weakening recommended in Table 7-5.

(2) If DFI > 1,000-°F days, a month will be added to the thaw-weakening period recommended in Table 7-5.

Frost-Susceptible Soil Classification	**End-of-Winter Thaw-Weakening Period (months)**
F1	1
F2	1
F3 and F4 (Noncohesive)	2
F3 and F4 (Cohesive)	3

Table 7-5. Length of End-of-Winter Thaw-Weakened Period

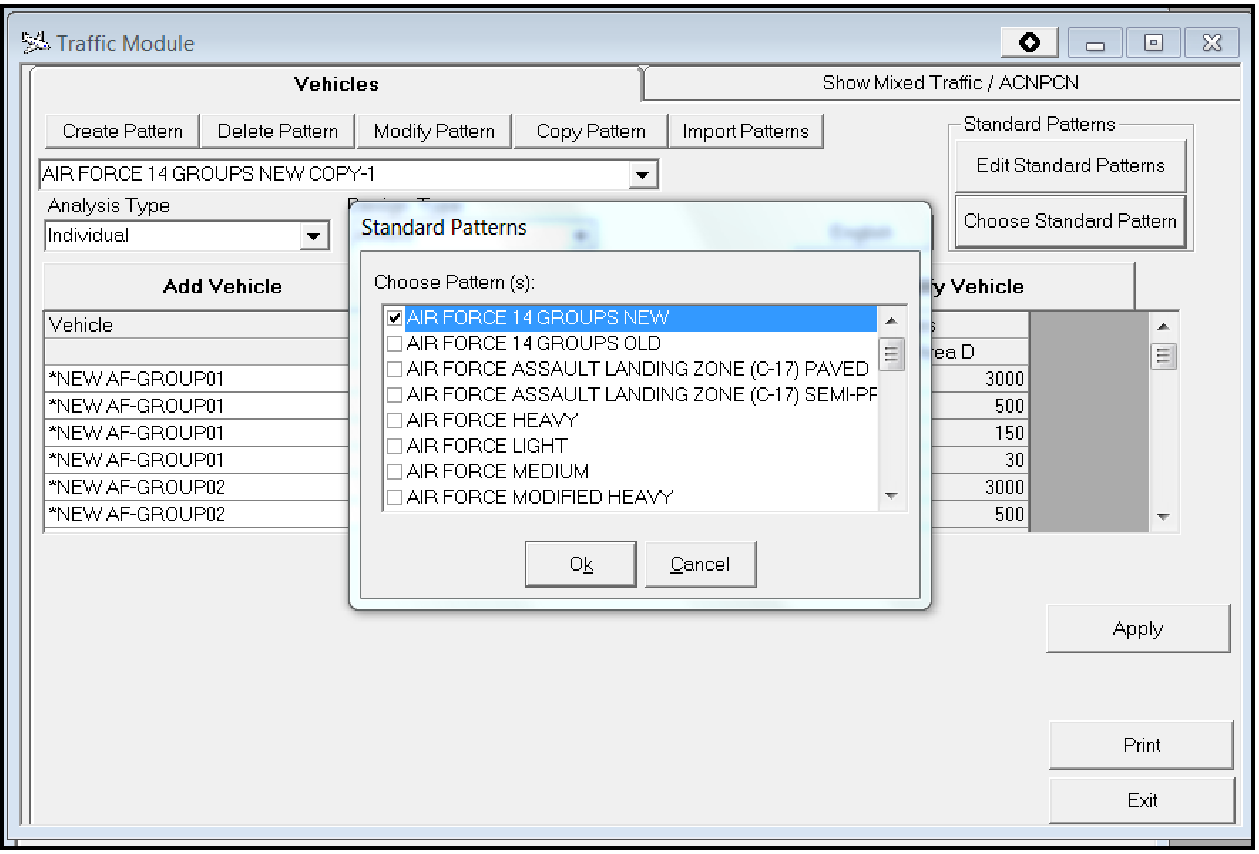
(3) The Climate module in the PCASE desktop can be used to determine the freezing season by inputting mean monthly and mean daily maximum and minimum temperatures.

b. Computations. Evaluation of airfield pavements in seasonal frost areas involves calculation of allowable aircraft loads for a given number of passes or allowable number of passes of a given load and PCN values that may be applied to a pavement during the normal and period of weakening. For calculating the allowable load during the normal and period of weakening, the design passes are divided between the normal season and thaw season based on the percentage of the year the location is in a thaw condition. For example, if the design passes are 50,000 and the thaw season is from March through May, the percent year the location is in a thaw condition is 25%. The passes are then divided based on the 25% resulting in 12,500 passes being applied in the thaw weakened period and 37,500 passes applied to the normal season.

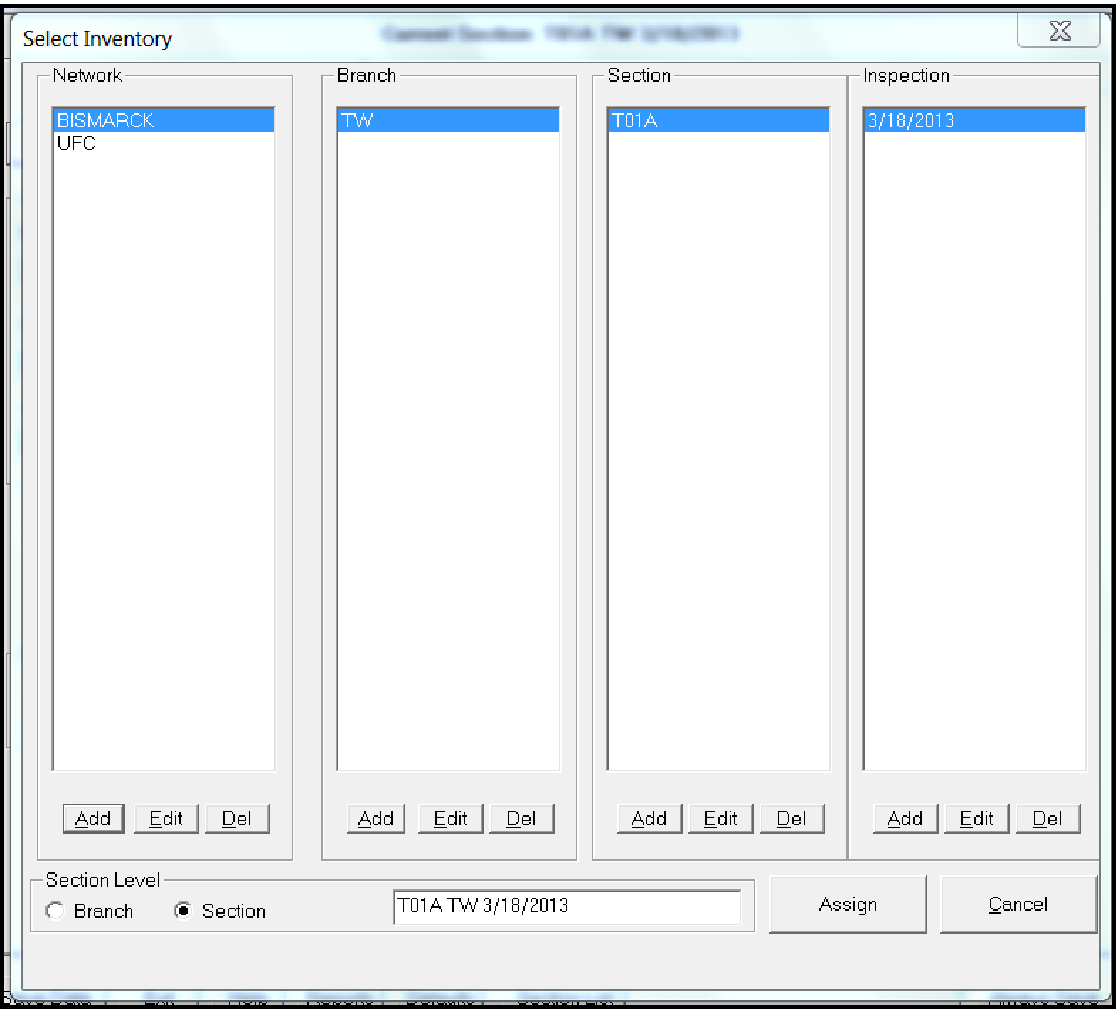
7-16 **EXAMPLE 1**. Evaluate an Air Force flexible pavement Type A traffic area consisting of 5 inches of asphalt concrete, 9 inches of crushed stone base (CBR = 100), and 12 inches of subbase (CBR = 30) over a silt subgrade. The pavement is to be evaluated for Pass Intensity Level I of the C-17 aircraft. The pavement surface is in good condition. The subgrade has dry density of 110 pounds per cubic foot and an average water content of 24 percent. The nonfrost CBR of the subgrade is 13. The base/ subbase-course material is a nonfrost-susceptible sandy gravel (GW) with an average dry unit weight of 135 pounds per cubic foot and average water content after drainage of 3 percent. The highest ground water is 2 feet below subgrade surface. For this example, the airfield is located in Bismarck, ND.

a. From PCASE

Open the Traffic module. Click on Choose Standard Pattern button. Check the box in front of “Air Force 14 Groups New”. Click on OK. Click the Apply button.



Open the Evaluation Module. On the Run Properties tab click on Create/Retrieve Section. On the Select Inventory screen click Add under the Network column and enter “Bismarck”. Click Add under the Branch column and enter “TW” for taxiway, enter a Section “T01A”, and enter an inspection date. Click Assign.



Network: BISMARCK, UFC

Branch: TW

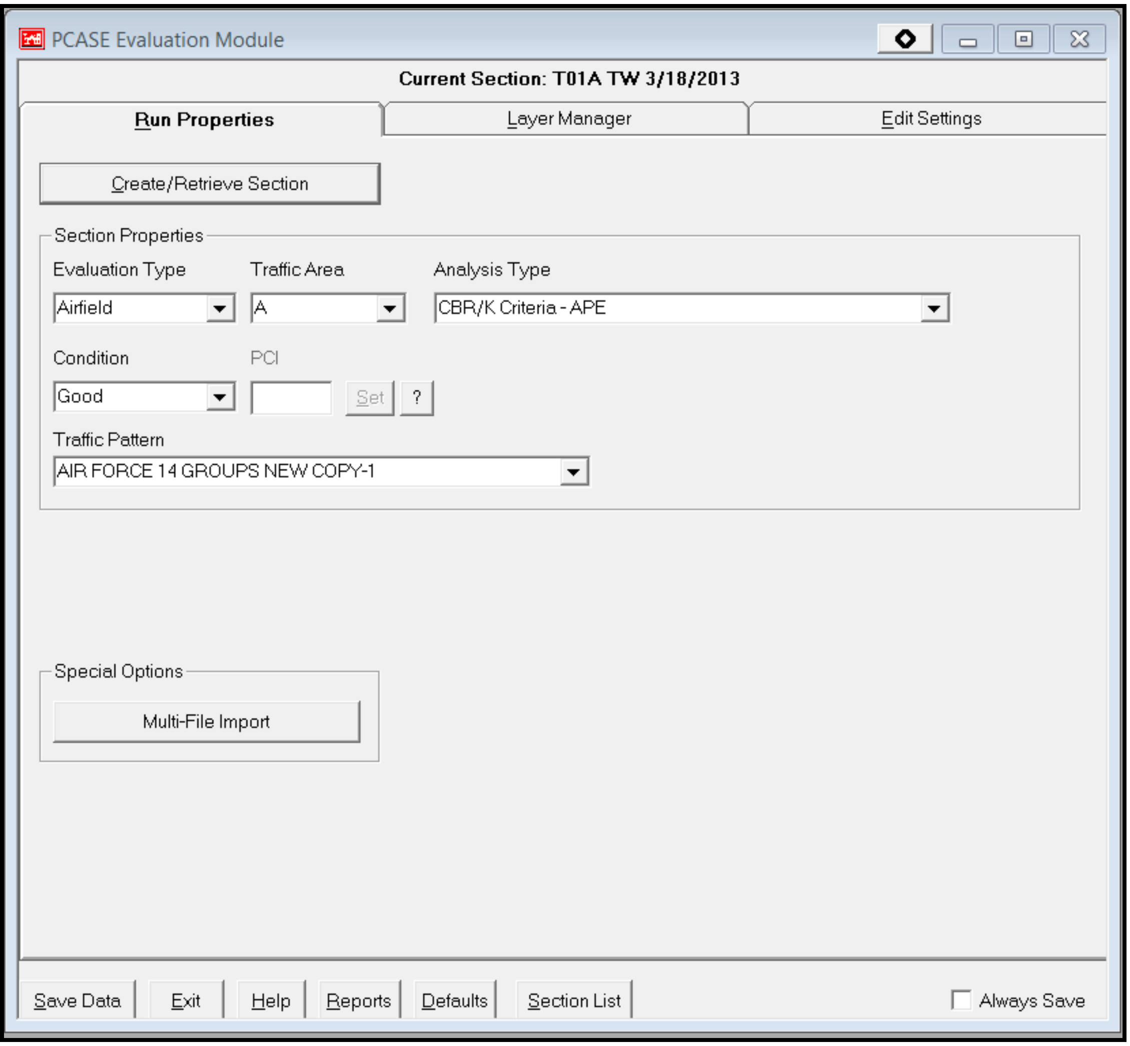
Section: T01A

Inspection: 3/18/2013

Section Level: ○ Branch ● Section

T01A TW 3/18/2013

On the Run Properties tab be sure the Traffic Area is set to “A”, Analysis Type is set to “CBR/K Criteria – APE”, Condition is Good, and choose the Traffic Pattern “Air Force 14 Groups New”



Current Section: T01A TW 3/18/2013

Evaluation Type: Airfield

Traffic Area: A

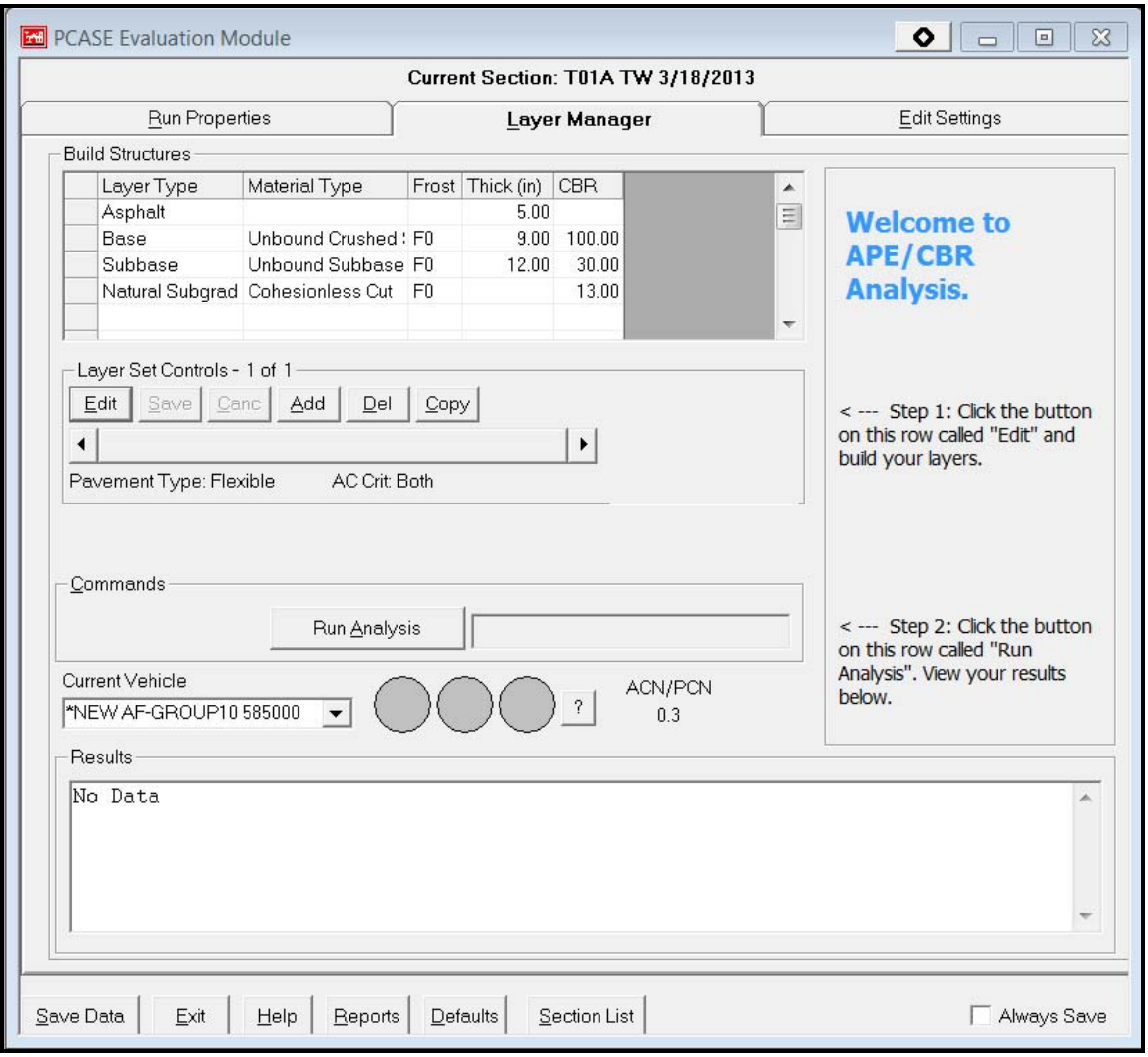
Analysis Type: CBR/K Criteria - APE

Condition: Good

PCI:

Traffic Pattern: AIR FORCE 14 GROUPS NEW COPY-1

Click on the Layer Manager Tab. Under the layer grid click on the Edit button and enter the layer information (layer types, material types, thicknesses, frost codes, and CBRs) as described above and shown below and click Save under the layer grid.



Current Section: T01A TW 3/18/2013

Layer Type	Material Type	Frost	Thick (in)	CBR
Asphalt			5.00	
Base	Unbound Crushed :	F0	9.00	100.00
Subbase	Unbound Subbase	F0	12.00	30.00
Natural Subgrad	Cohesionless Cut	F0		13.00

Layer Set Controls - 1 of 1

Pavement Type: Flexible AC Crit: Both

Current Vehicle: *NEW AF-GROUP10 585000

ACN/PCN 0.3

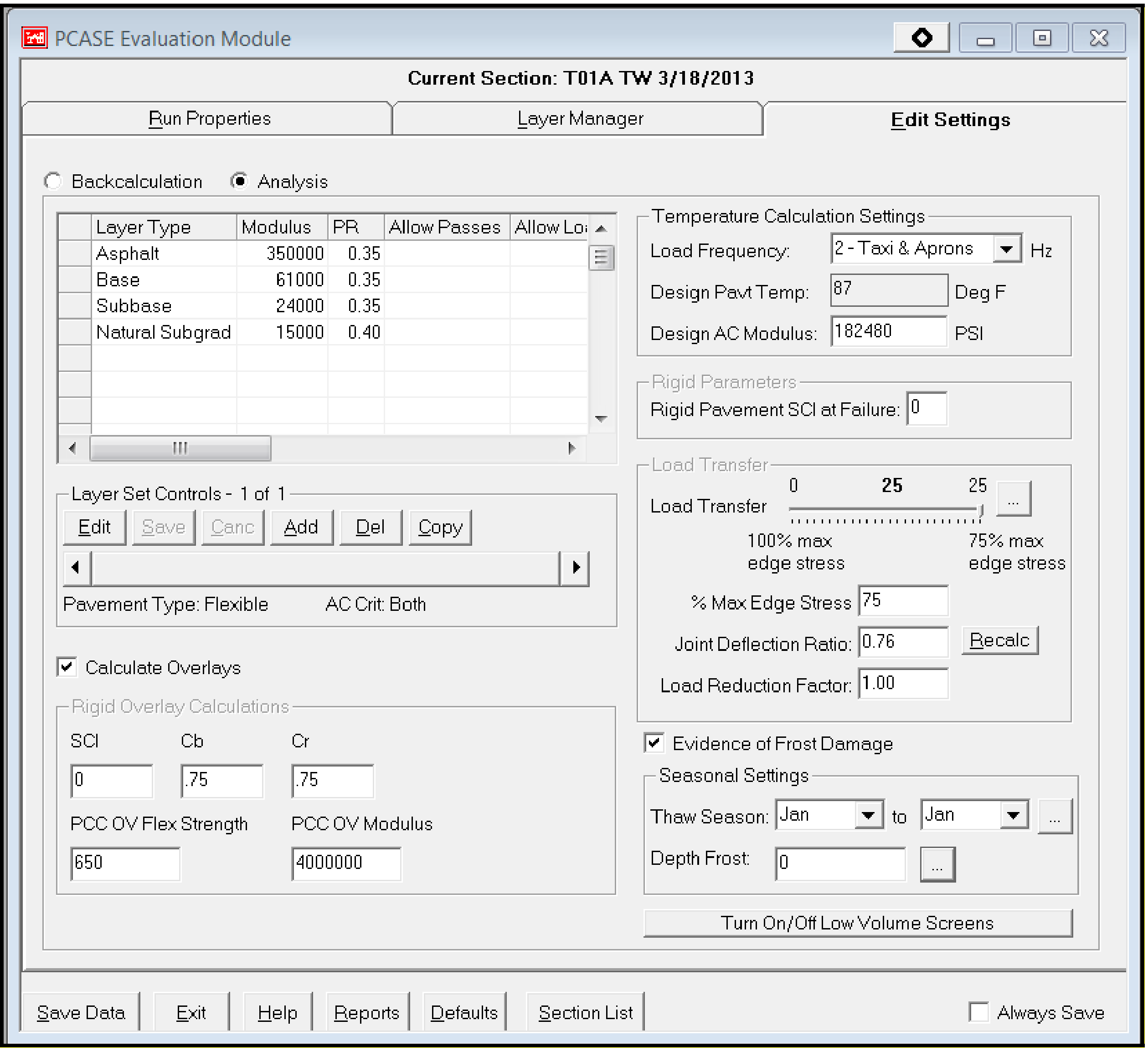
Results: No Data

Welcome to APE/CBR Analysis.

< --- Step 1: Click the button on this row called "Edit" and build your layers.

< --- Step 2: Click the button on this row called "Run Analysis". View your results below.

To open the depth of frost calculator click on the Edit Settings tab. Click on the Analysis button. Click the box in the bottom right of the screen labeled “Evidence of Frost Damage” and click the button with the “…” to the right of “Frost Damage”.



Current Section: T01A TW 3/18/2013

○ Backcalculation ● Analysis

Layer Type	Modulus	PR	Allow Passes	Allow Lo
Asphalt	350000	0.35		
Base	61000	0.35		
Subbase	24000	0.35		
Natural Subgrad	15000	0.40		

Layer Set Controls - 1 of 1

Pavement Type: Flexible AC Crit: Both

☑ Calculate Overlays

Rigid Overlay Calculations

SCI: 0
Cb: .75
Cr: .75
PCC OV Flex Strength: 650
PCC OV Modulus: 4000000

Temperature Calculation Settings

Load Frequency: 2 - Taxi & Aprons Hz
Design Pavt Temp: 87 Deg F
Design AC Modulus: 182480 PSI

Rigid Parameters

Rigid Pavement SCI at Failure: 0

Load Transfer

Load Transfer: 0 **25** 25
100% max edge stress 75% max edge stress
% Max Edge Stress: 75
Joint Deflection Ratio: 0.76
Load Reduction Factor: 1.00

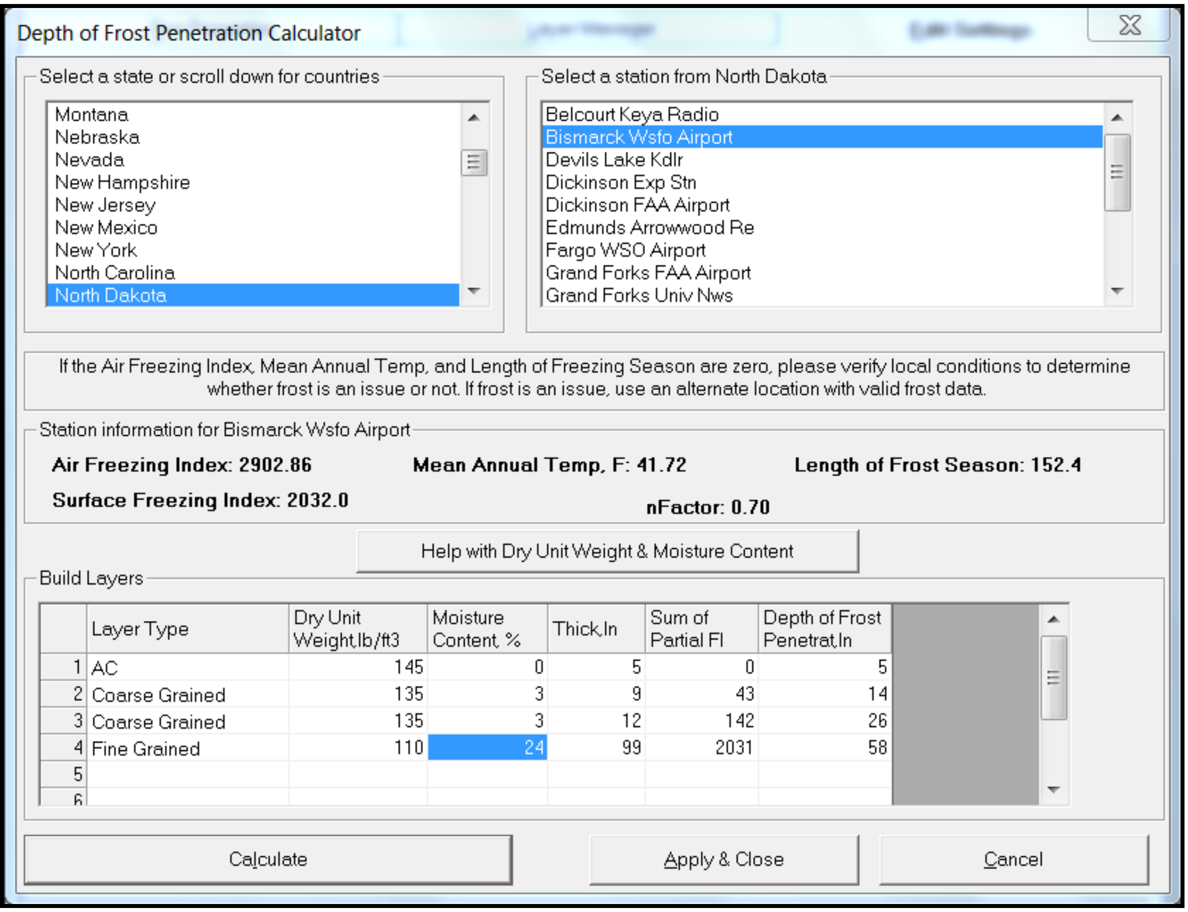
☑ Evidence of Frost Damage

Seasonal Settings

Thaw Season: Jan to Jan
Depth Frost: 0

☐ Always Save

In the Depth of Frost Penetration Calculator choose North Dakota for the State, and Bismarck Wsfo Airport for the station. For each layer enter the dry unit weights and moisture contents as given above and shown below. Click the calculate button. The depth of frost penetration for this example is 58 inches as indicated in the column label Depth of Frost Penetration for the Fine Grained layer. Also shown on this screen is the Design Air Freezing Index of 2903 degree F days and the design Length of Frost Season of 152 days. Click Apply & Close and the frost penetration depth will be imported to the Edit Settings screen.



If the Air Freezing Index, Mean Annual Temp, and Length of Freezing Season are zero, please verify local conditions to determine whether frost is an issue or not. If frost is an issue, use an alternate location with valid frost data.

Station information for Bismarck Wsfo Airport

Air Freezing Index: 2902.86
Mean Annual Temp, F: 41.72
Length of Frost Season: 152.4
Surface Freezing Index: 2032.0
nFactor: 0.70

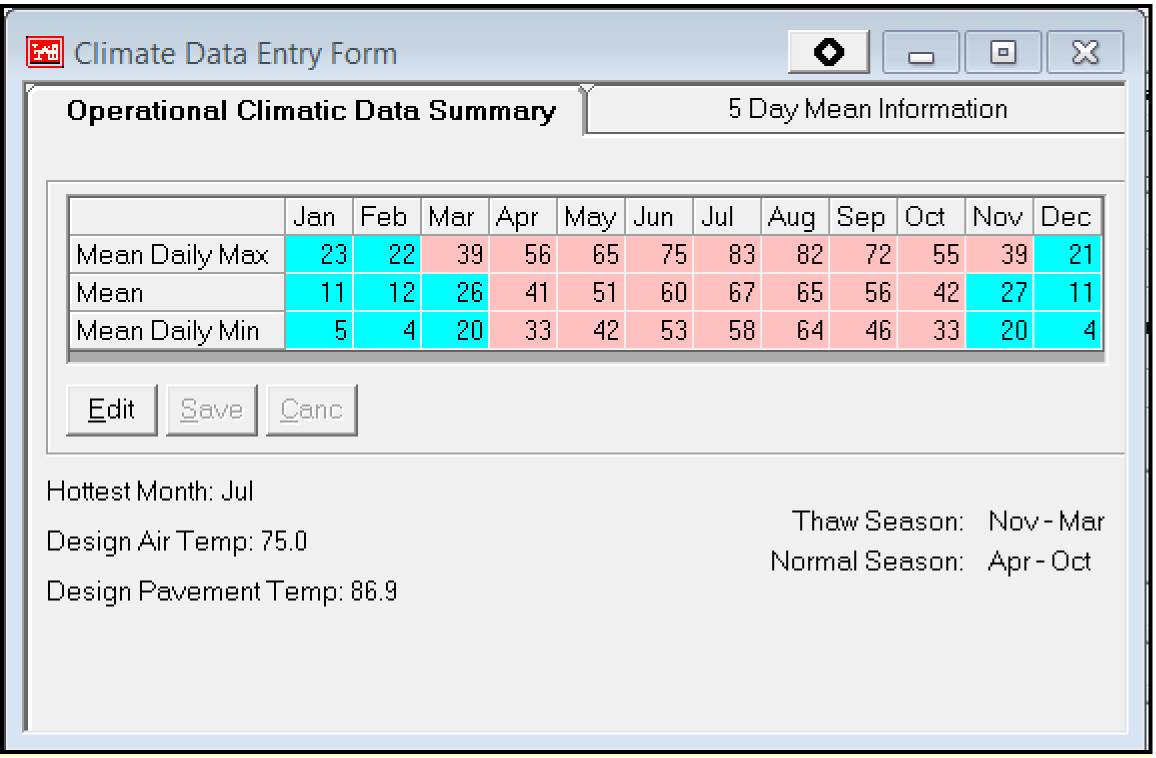
	Layer Type	Dry Unit Weight,lb/ft3	Moisture Content, %	Thick,In	Sum of Partial FI	Depth of Frost Penetrat,In
1	AC	145	0	5	0	5
2	Coarse Grained	135	3	9	43	14
3	Coarse Grained	135	3	12	142	26
4	Fine Grained	110	24	99	2031	58
5						
6						

Determine if the Base/Subbase and/or Subgrade is Frost-Susceptible. From Table 7-1, the subgrade is classified as an F4 frost-susceptible soil.

Evaluate for Complete Frost Penetration. With a 5-inch-thick pavement surface, the thickness of base course (c) for zero penetration of the subgrade is 58 - 5 = 53 inches. The thickness of the base and subbase layer (x) is 21 inches. Since x ≤ c, then the pavement structure was not designed for complete frost protection.

Evaluate for Limited Subgrade Frost Penetration. The ratio of subgrade to base-course water content r = 24/3 = 8. From Figure 7-5, using the maximum permissible ratio r of 2.0 applicable to traffic area A, the required total base thickness b that would hold subgrade frost penetration within the allowable limit is 33 inches. In this case, the 26-inch-thick section of pavement surface and base does not provide adequate protection against frost action, and evaluation for frost and nonfrost conditions are required.

Determine the period of thaw weakening. Open the PCASE Climate module. On the Operational Climatic Data Summary tab click the Edit button, enter the Mean Daily Maximum, Mean, and Mean Daily Minimum temperatures for each month for Bismarck, North Dakota, and click Save. Data may be obtained from various weather sources. On the Climate screen the Thaw Season is the actual Freezing Season. For this example the Freezing Season for Bismarck, ND is November through March. The thaw weakened period after the end of winter from Table 7-5 is estimated to be 2 months. Since the airfield is located in an area with a DFI of 2903 °F days, an additional 1 month (to cover for intermediate thaw periods during the freezing period) is added to the 2 months to obtain the total weakening period of 3 months. The thaw weakened period is from April to June.



	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Mean Daily Max	23	22	39	56	65	75	83	82	72	55	39	21
Mean	11	12	26	41	51	60	67	65	56	42	27	11
Mean Daily Min	5	4	20	33	42	53	58	64	46	33	20	4

Hottest Month: Jul

Design Air Temp: 75.0

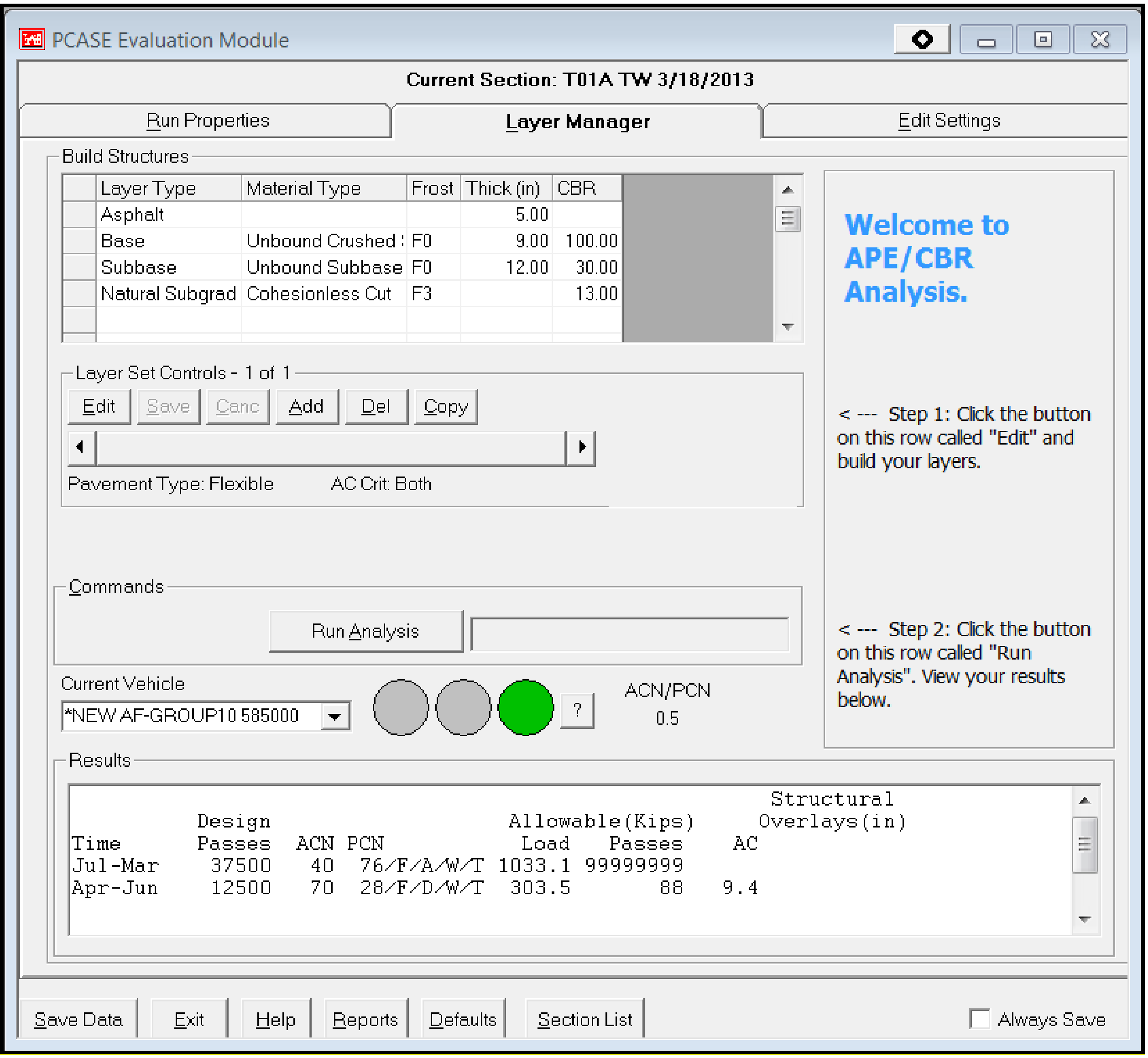
Design Pavement Temp: 86.9

Thaw Season: Nov - Mar

Normal Season: Apr - Oct

Evaluate for Reduced Subgrade Strength. Determine allowable load and PCN during thaw-weakened period. The soil is classified as a F4 frost-susceptible soil. From Table 7-3, the FASSI value is 3.5. Use either the procedure outlined in Chapter 5 or PCASE to determine the allowable load for the traffic area during the thaw-weakened period using the FASSI value of 3.5.

Using PCASE go back to the Evaluation Module. On the Edit Settings tab set the Thaw Period of April to June by pulling down the arrows to set each month. Click on the Layer Manager tab. Click the Edit button under the layer grid and change the Frost Code for the subgrade to F3/F4 and click Save under the layer grid. Click Run Analysis in the middle of the screen and the results will be given for the Air Force 14 groups at each pass intensity level. For results for the C-17 at 50,000 passes use the down arrow under Current Vehicle to scroll to Air Force Group 10 and the pass intensity level I. Results are shown below.



Current Section: T01A TW 3/18/2013

Layer Type	Material Type	Frost	Thick (in)	CBR
Asphalt			5.00	
Base	Unbound Crushed :	F0	9.00	100.00
Subbase	Unbound Subbase	F0	12.00	30.00
Natural Subgrad	Cohesionless Cut	F3		13.00

Pavement Type: Flexible AC Crit: Both

Current Vehicle: *NEW AF-GROUP10 585000

ACN/PCN 0.5

Welcome to APE/CBR Analysis.

< --- Step 1: Click the button on this row called "Edit" and build your layers.

< --- Step 2: Click the button on this row called "Run Analysis". View your results below.

Results

```
                                                        Structural
          Design                   Allowable(Kips)     Overlays(in)
Time      Passes  ACN PCN           Load   Passes    AC
Jul-Mar    37500   40  76/F/A/W/T 1033.1 99999999
Apr-Jun    12500   70  28/F/D/W/T  303.5       88    9.4
```

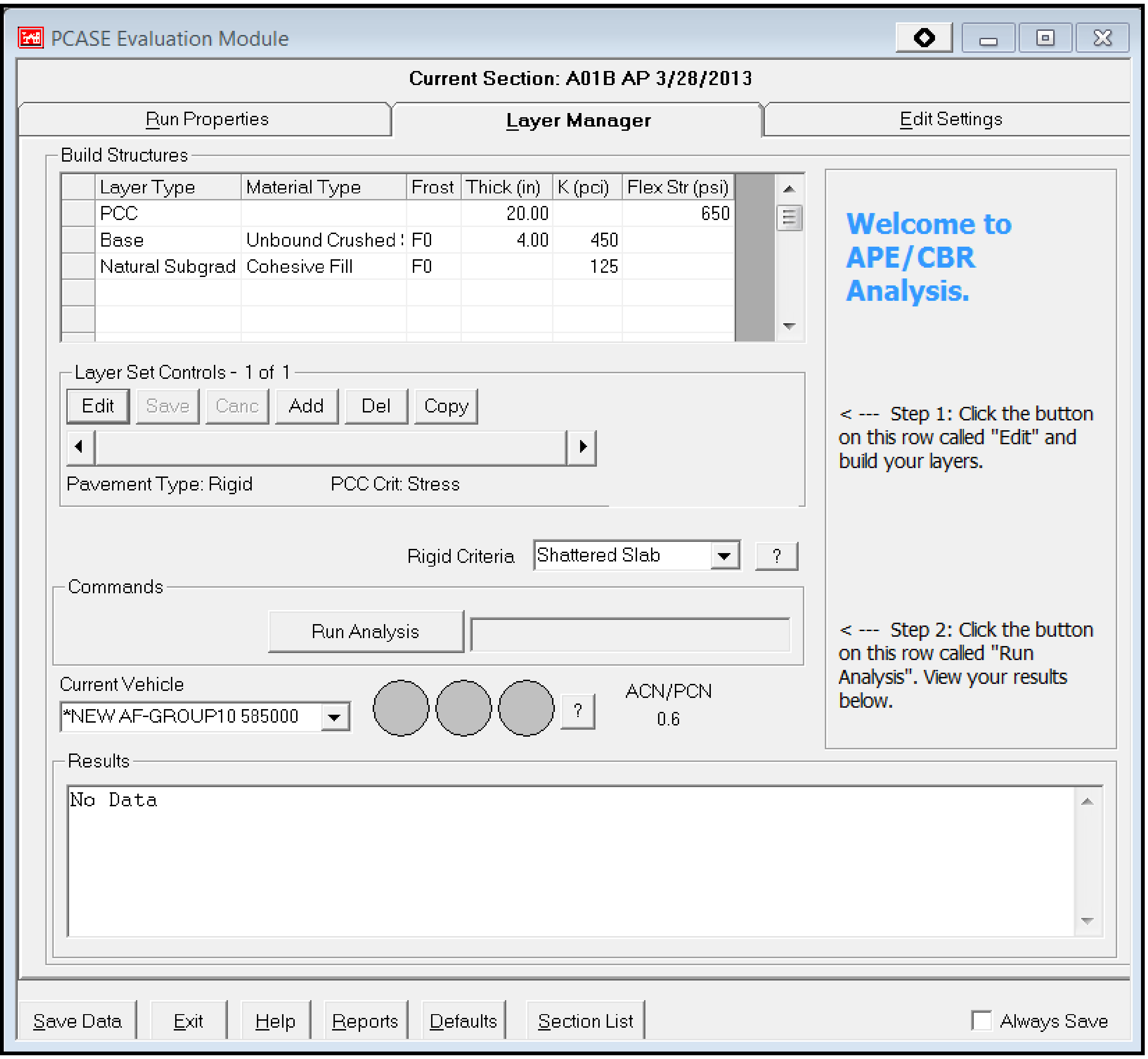
7-17 **EXAMPLE 2**. Evaluate an Air Force rigid pavement type B traffic area consisting of 20 inches of PCC and 4 inches of base on a clay subgrade. The flexural strength of the concrete is 650 psi. Visual inspection of the pavement shows it to be in good condition. The pavement is to be evaluated for Pass Intensity Levels I-IV of a 585 kip C-17, using extended life (shattered slab) criteria. The aircraft traffic is applied uniformly throughout the year. The subgrade is a clay with a PI of 10, a dry density of 100 pcf, and an average water content of 18 percent. From field tests, the subgrade k during the normal period was 125 pci. The base material is a nonfrost-susceptible sandy gravel (GW) with a normal k value of 450 pci. The average dry unit weight and average water content of the base layer are 135 pcf and 5 percent, respectively. The highest groundwater is at the subgrade surface. For this example, the airfield is located in Fairbanks, AK.

From PCASE:
In Example 1 the Air Force 14 Groups traffic was assigned and is available for use in the Evaluation module.

Open the Evaluation Module. On the Run Properties tab click on Create/Retrieve Section. On the Select Inventory screen click Add under the Network column and enter “Fairbanks”. Click Add under the Branch column and enter “AP” for apron, enter a Section “A01B”, and enter an inspection date. Click Assign.

On the Run Properties tab be sure the Traffic Area is set to “B”, Analysis Type is set to “CBR/K Criteria – APE”, Condition is Good, and choose the Traffic Pattern “Air Force 14 Groups New”

Click on the Layer Manager Tab. Under the layer grid click on the Edit button and enter the layer information (layer types, material types, thicknesses, frost codes, and K’s) as described above and shown below and click Save under the layer grid. Be sure the rigid criteria (in the middle of the screen) is set at “Shattered Slab”.



Current Section: A01B AP 3/28/2013

Layer Type	Material Type	Frost	Thick (in)	K (pci)	Flex Str (psi)
PCC			20.00		650
Base	Unbound Crushed	F0	4.00	450	
Natural Subgrad	Cohesive Fill	F0		125	

Pavement Type: Rigid

PCC Crit: Stress

Rigid Criteria: Shattered Slab

Current Vehicle: *NEW AF-GROUP10 585000

ACN/PCN: 0.6

Results:

No Data

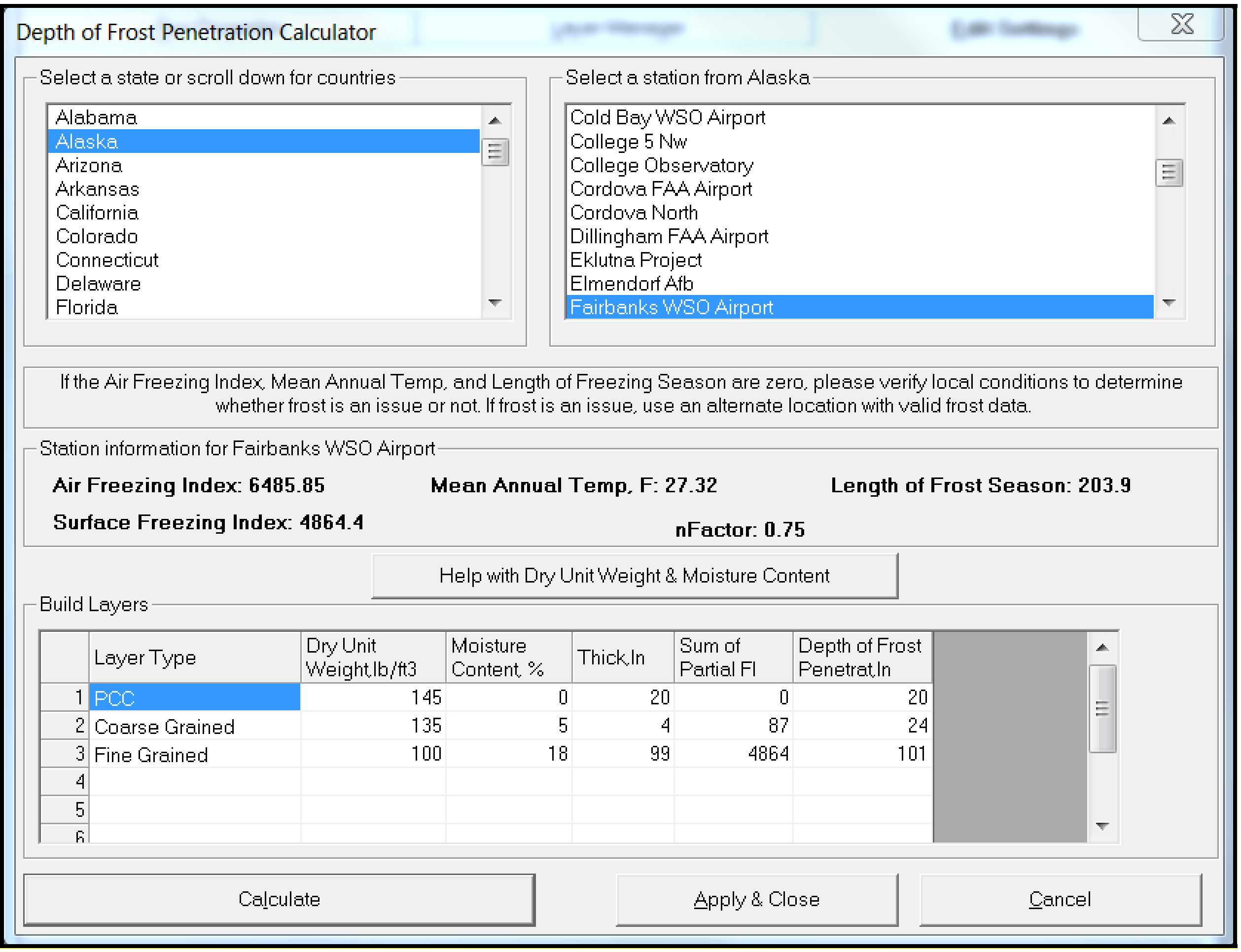
Welcome to APE/CBR Analysis.

< --- Step 1: Click the button on this row called "Edit" and build your layers.

< --- Step 2: Click the button on this row called "Run Analysis". View your results below.

To open the depth of frost calculator click on the Edit Settings tab. Click on the Analysis button. Click the box in the bottom right of the screen labeled “Evidence of Frost Damage” and click the button with the “…” to the right of “Frost Damage”.

In the Depth of Frost Penetration Calculator choose Alaska for the State, and Fairbanks WSO Airport for the station. For each layer enter the dry unit weights and moisture contents as given above and shown below. Click the calculate button. The depth of frost penetration for this example is 101 inches as indicated in the column label Depth of Frost Penetration for the Fine Grained layer. Also shown on this screen is the Design Air Freezing Index of 6486 degree F days and the design Length of Frost Season of 204 days. Click Apply & Close and the frost penetration depth will be imported to the Edit Settings screen.



If the Air Freezing Index, Mean Annual Temp, and Length of Freezing Season are zero, please verify local conditions to determine whether frost is an issue or not. If frost is an issue, use an alternate location with valid frost data.

Station information for Fairbanks WSO Airport

Air Freezing Index: 6485.85
Mean Annual Temp, F: 27.32
Length of Frost Season: 203.9
Surface Freezing Index: 4864.4
nFactor: 0.75

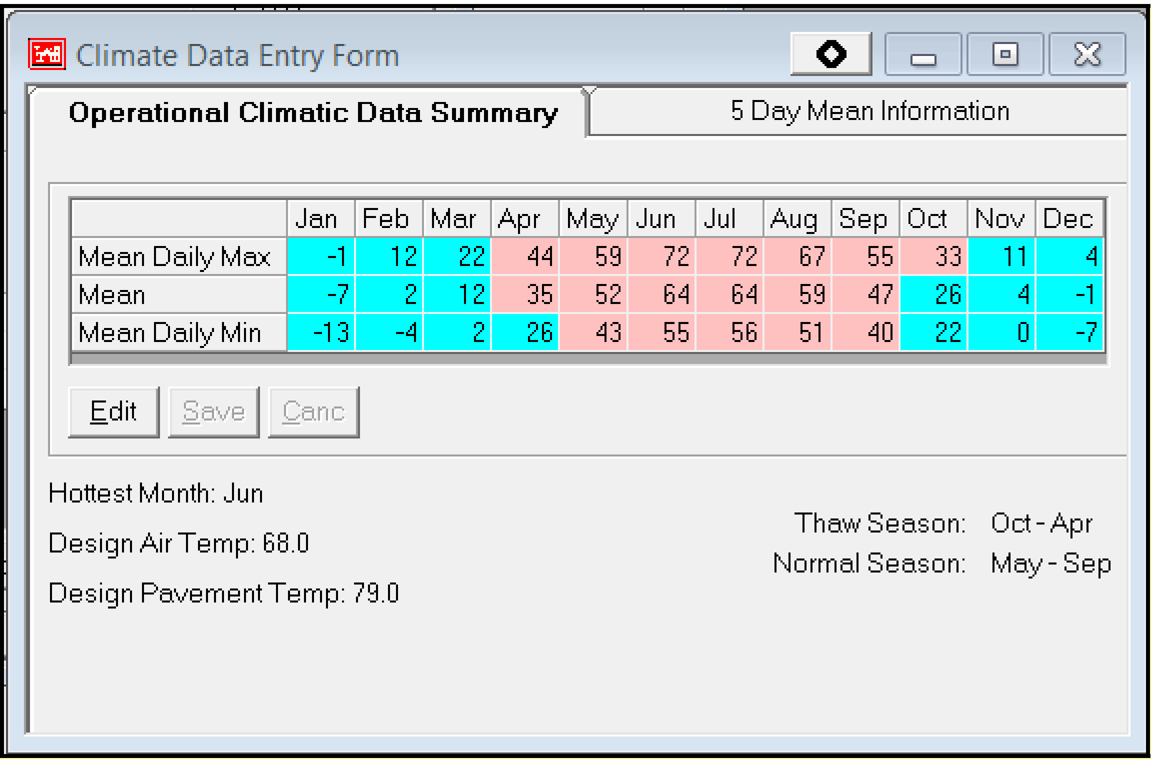
	Layer Type	Dry Unit Weight,lb/ft3	Moisture Content, %	Thick,In	Sum of Partial FI	Depth of Frost Penetrat,In
1	PCC	145	0	20	0	20
2	Coarse Grained	135	5	4	87	24
3	Fine Grained	100	18	99	4864	101
4						
5						
6						

Determine if the Base/Subbase and/or Subgrade is Frost-Susceptible. From Table 7-1, the subgrade is classified as an F4 frost-susceptible soil.

Evaluate for Complete Frost Penetration. With a 20-inch-thick pavement surface, the thickness of base course (c) for zero penetration of the subgrade is 101 - 20 = 81 inches. The thickness of the base layer (x) is 4 inches. Since x ≤ c, then the pavement structure was not designed for complete frost protection.

Evaluate for Limited Subgrade Frost Penetration. The ratio of subgrade to base-course water content r = 18/5 = 3.6. From Figure 7-5, using the maximum permissible ratio r of 2.0 applicable to type B traffic area, the required total base thickness b that would hold subgrade frost penetration within the allowable limit is 52 inches. In this case, the 24-inch- thick section of pavement and base does not provide adequate protection against frost action, and evaluation for thawing and normal conditions are required.

Determine the period of thaw weakening. Open the PCASE Climate module. On the Operational Climatic Data Summary tab click the Edit button, enter the Mean Daily Maximum, Mean, and Mean Daily Minimum temperatures for each month for Fairbanks, Alaska, and click Save. Data may be obtained from various weather sources. On the Climate screen the Thaw Season is the actual Freezing Season. For this example the Freezing Season for Fairbanks, AK is October through April. The thaw weakened period after the end of winter from Table 7-5 is estimated to be 3 months. Since the airfield is located in an area with a DFI of 4864 °F days, an additional 1 month (to cover for intermediate thaw periods during the freezing period) is added to the 3 months to obtain the total weakening period of 4 months. The thaw weakened period is from April to July.



	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Mean Daily Max	-1	12	22	44	59	72	72	67	55	33	11	4
Mean	-7	2	12	35	52	64	64	59	47	26	4	-1
Mean Daily Min	-13	-4	2	26	43	55	56	51	40	22	0	-7

Hottest Month: Jun

Design Air Temp: 68.0

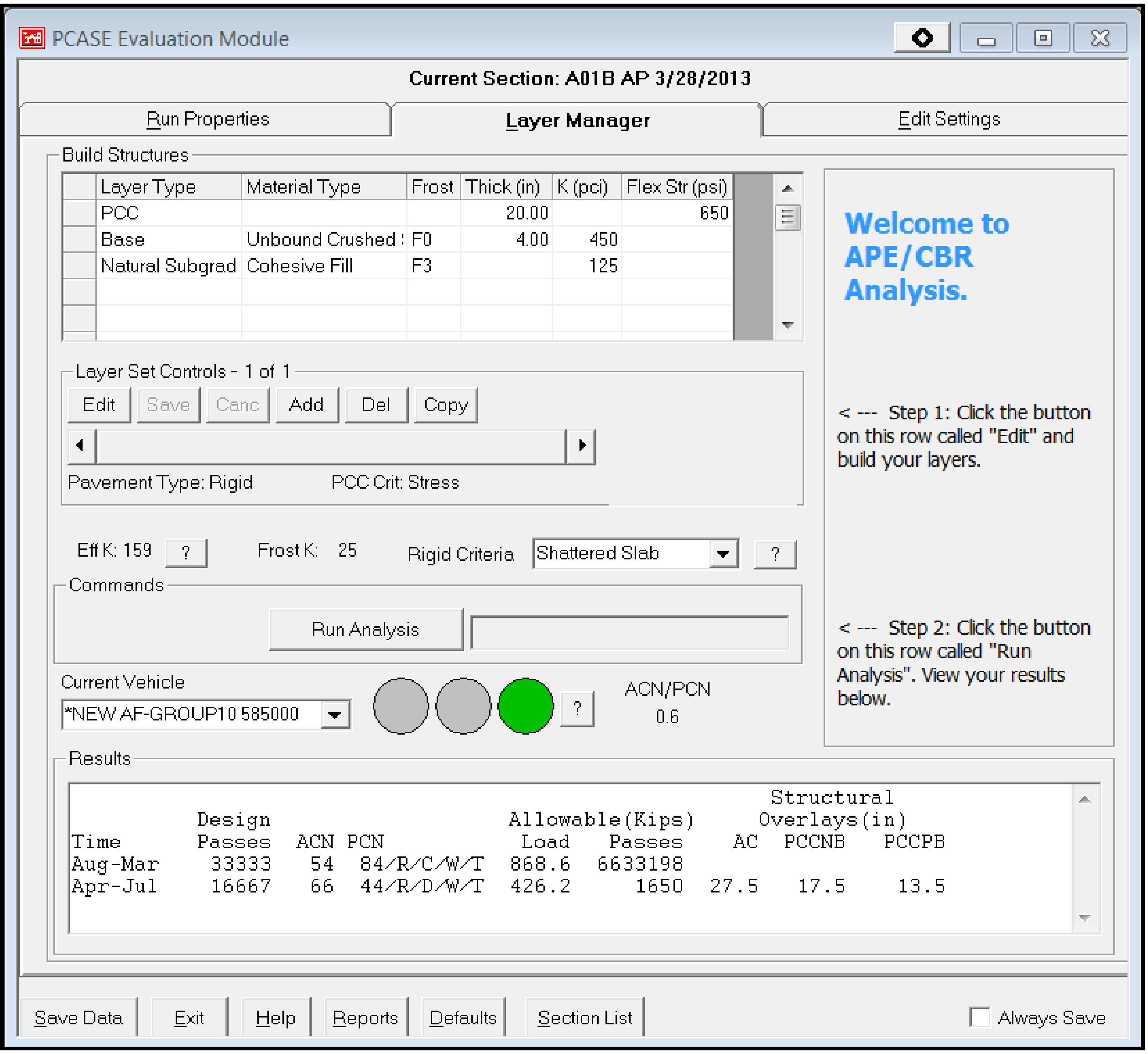
Design Pavement Temp: 79.0

Thaw Season: Oct - Apr

Normal Season: May - Sep

Evaluate for Reduced Subgrade Strength. Determine allowable load and PCN during thaw-weakened period. The soil is classified as a F4 frost-susceptible soil. From Figure 7-6, the FAIR value is 25. Use either the procedure outlined in Chapter 5 or PCASE to determine the allowable load for the traffic area during the thaw-weakened period using the FAIR value of 25.

Using PCASE, go back to the Evaluation Module. On the Edit Settings tab set the Thaw Period of April to July by pulling down the arrows to set each month. Click on the Layer Manager tab. Click the Edit button under the layer grid and change the Frost Code for the subgrade to F3/F4 and click Save under the layer grid. Click Run Analysis in the middle of the screen and the results will be given for the Air Force 14 groups at each pass intensity level. For results for the C-17 at 50,000 passes use the down arrow under Current Vehicle to scroll to Air Force Group 10 and the pass intensity level I. Results are shown below.



Current Section: A01B AP 3/28/2013

	Layer Type	Material Type	Frost	Thick (in)	K (pci)	Flex Str (psi)
	PCC			20.00		650
	Base	Unbound Crushed :	F0	4.00	450	
	Natural Subgrad	Cohesive Fill	F3		125	

Pavement Type: Rigid   PCC Crit: Stress

Eff K: 159   Frost K: 25   Rigid Criteria: Shattered Slab

Current Vehicle: *NEW AF-GROUP10 585000

ACN/PCN 0.6

Welcome to APE/CBR Analysis.

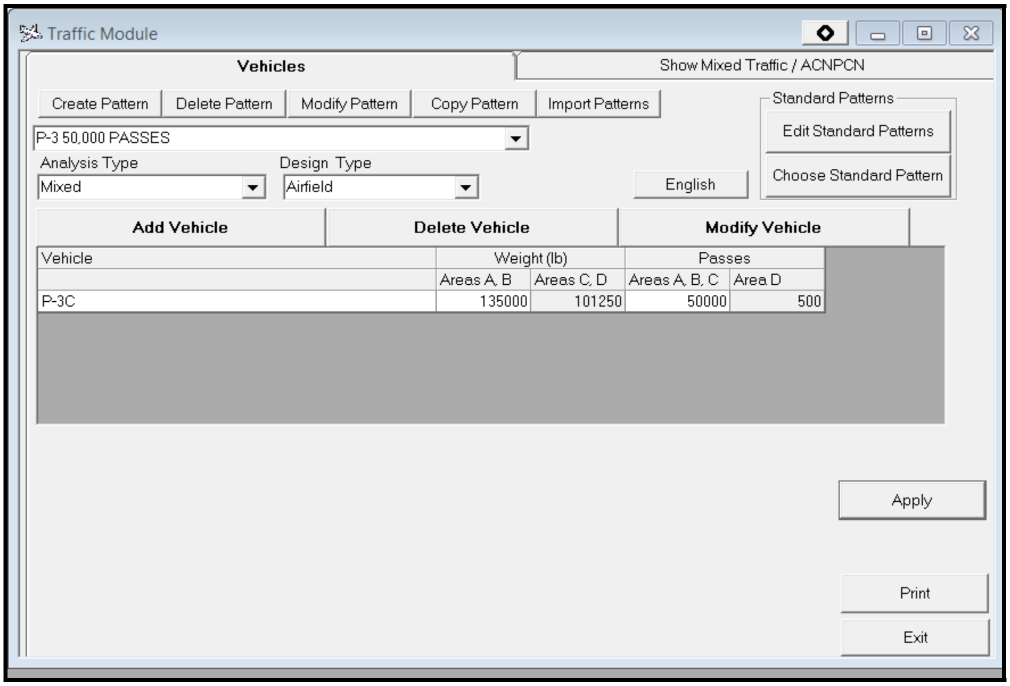
< --- Step 1: Click the button on this row called "Edit" and build your layers.

< --- Step 2: Click the button on this row called "Run Analysis". View your results below.

```
                                                   Structural
         Design                  Allowable(Kips)  Overlays(in)
Time     Passes  ACN  PCN          Load   Passes    AC   PCCNB   PCCPB
Aug-Mar   33333   54  84/R/C/W/T  868.6  6633198
Apr-Jul   16667   66  44/R/D/W/T  426.2     1650  27.5   17.5    13.5
```

7-18 **EXAMPLE 3, (REDUCED MODULI)**. An A Traffic Area in Fairbanks, Alaska was evaluated for the Navy in the summer, using a Heavyweight Deflectometer. Weather Data from Example 2 are applicable. The pavement section consists of 4 inches of AC, a 6 inch base course, a 15 inch subbase course, and a clay subgrade. Moduli values at the time of the evaluation were 300,000 psi for the asphalt, 75,000 psi for the base, 35,000 psi for the subbase, and 20,000 psi for the subgrade. The base was an NFS material, the subbase was an F1 material, and the subgrade was an F4 material. The critical aircraft is the P3, with an estimated 50,000 passes during the pavement life. What is the allowable load and PCN during the frost melt period?

The PCASE inputs and outputs are shown below:

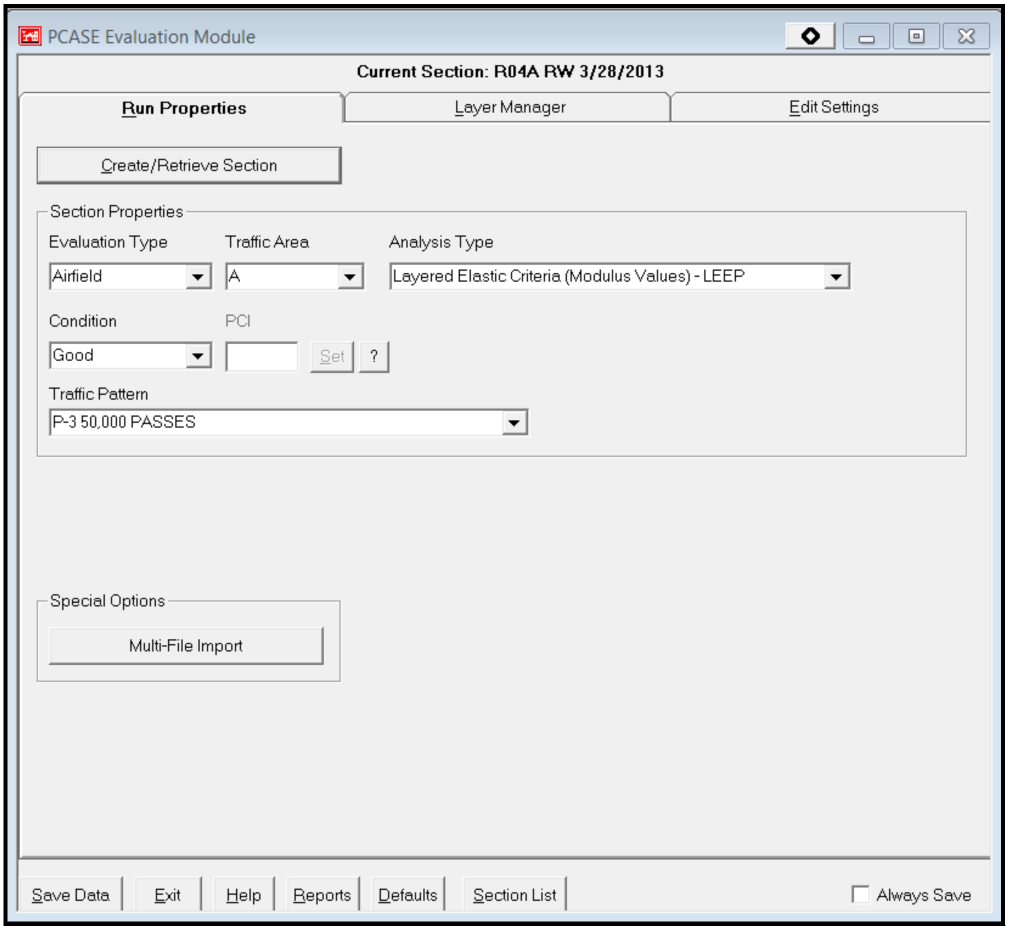


P-3 50,000 PASSES

Analysis Type: Mixed

Design Type: Airfield

Vehicle	Weight (lb)		Passes	
	Areas A, B	Areas C, D	Areas A, B, C	Area D
P-3C	135000	101250	50000	500



Current Section: R04A RW 3/28/2013

Section Properties

Evaluation Type: Airfield

Traffic Area: A

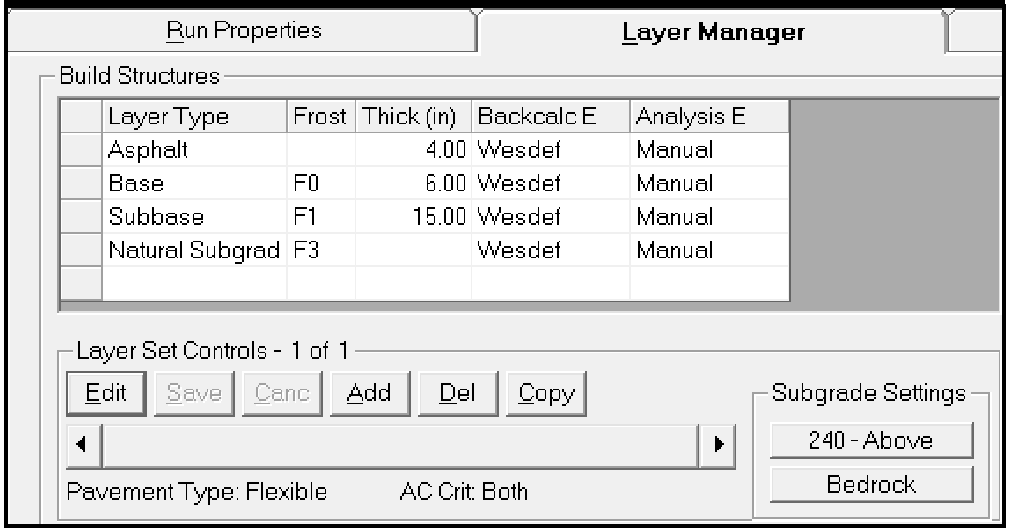
Analysis Type: Layered Elastic Criteria (Modulus Values) - LEEP

Condition: Good

PCI:

Traffic Pattern: P-3 50,000 PASSES

☐ Always Save

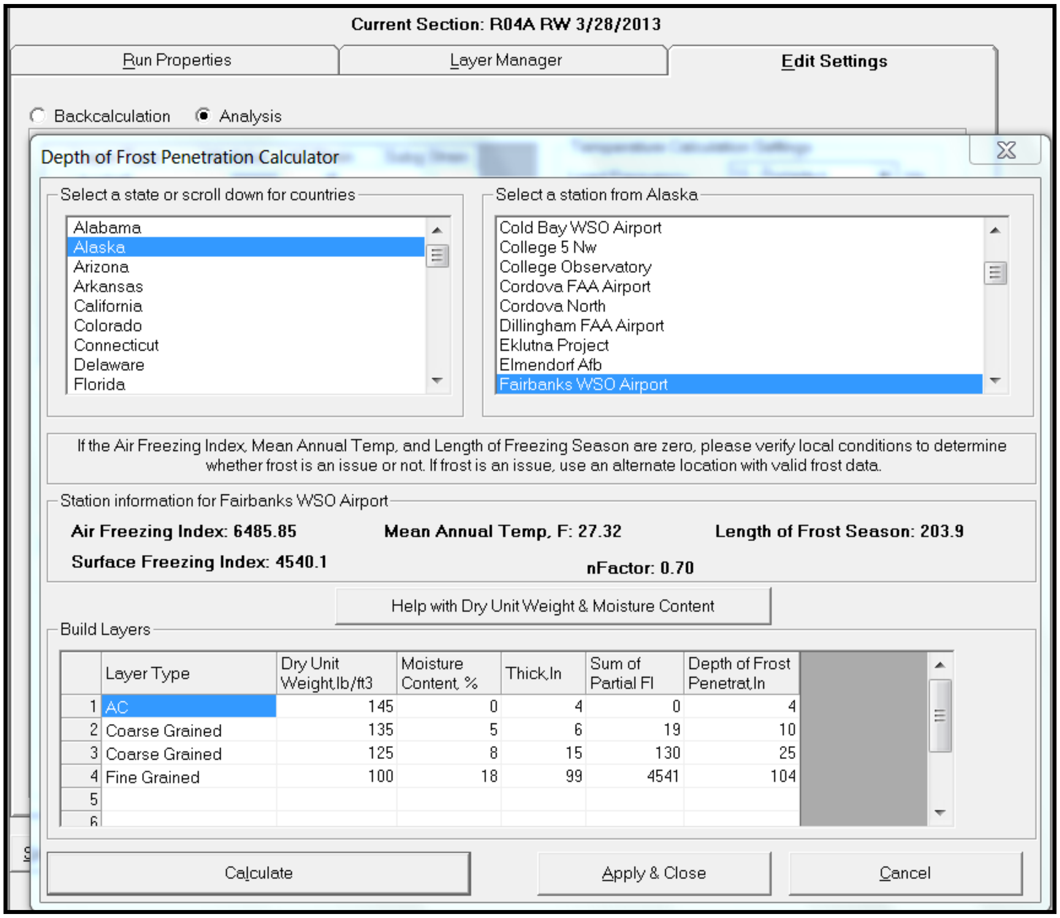


Build Structures

Layer Type	Frost	Thick (in)	Backcalc E	Analysis E
Asphalt		4.00	Wesdef	Manual
Base	F0	6.00	Wesdef	Manual
Subbase	F1	15.00	Wesdef	Manual
Natural Subgrad	F3		Wesdef	Manual

Layer Set Controls - 1 of 1

Pavement Type: Flexible AC Crit: Both



Current Section: R04A RW 3/28/2013

○ Backcalculation ● Analysis

Depth of Frost Penetration Calculator

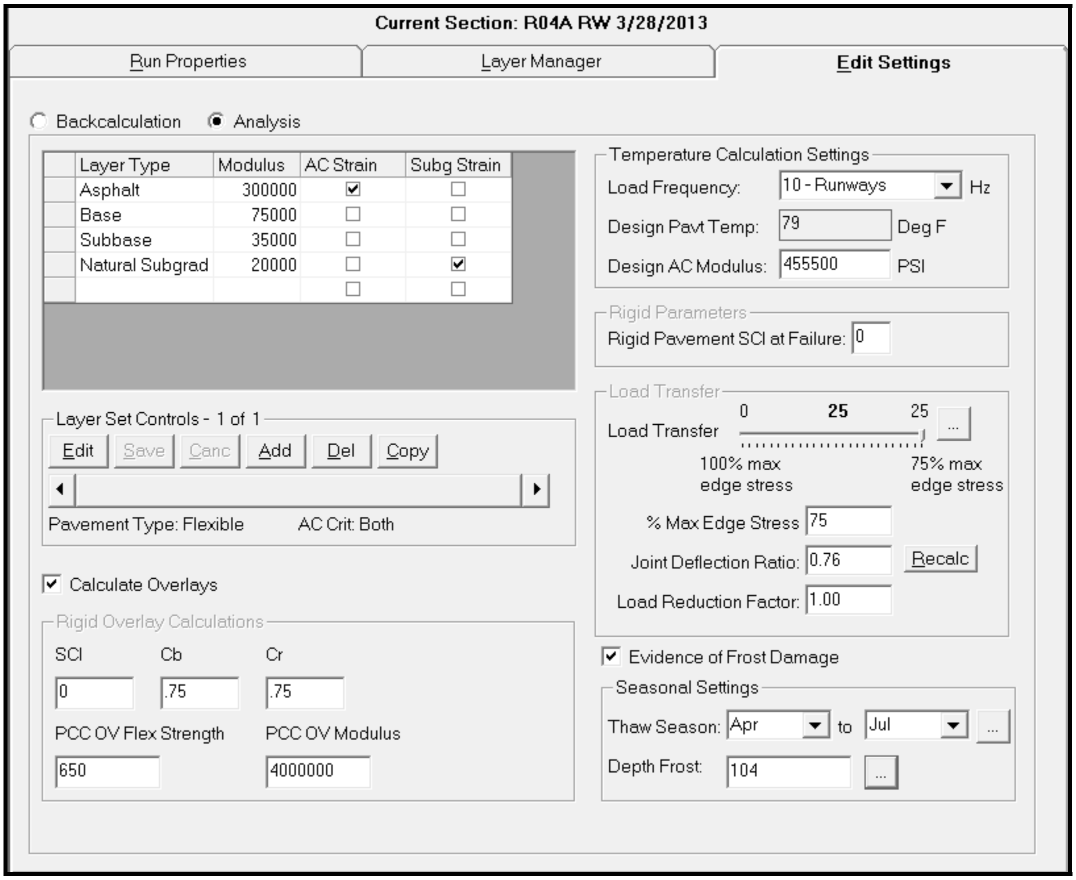
If the Air Freezing Index, Mean Annual Temp, and Length of Freezing Season are zero, please verify local conditions to determine whether frost is an issue or not. If frost is an issue, use an alternate location with valid frost data.

Station information for Fairbanks WSO Airport

Air Freezing Index: 6485.85 **Mean Annual Temp, F: 27.32** **Length of Frost Season: 203.9**

Surface Freezing Index: 4540.1 **nFactor: 0.70**

	Layer Type	Dry Unit Weight,lb/ft3	Moisture Content, %	Thick,In	Sum of Partial FI	Depth of Frost Penetrat,In
1	AC	145	0	4	0	4
2	Coarse Grained	135	5	6	19	10
3	Coarse Grained	125	8	15	130	25
4	Fine Grained	100	18	99	4541	104
5						
6						



Current Section: R04A RW 3/28/2013

○ Backcalculation ● Analysis

	Layer Type	Modulus	AC Strain	Subg Strain
	Asphalt	300000	☑	☐
	Base	75000	☐	☐
	Subbase	35000	☐	☐
	Natural Subgrad	20000	☐	☑
			☐	☐

Layer Set Controls - 1 of 1

Pavement Type: Flexible AC Crit: Both

☑ Calculate Overlays

Rigid Overlay Calculations

SCI: 0
Cb: .75
Cr: .75
PCC OV Flex Strength: 650
PCC OV Modulus: 4000000

Temperature Calculation Settings

Load Frequency: 10 - Runways Hz
Design Pavt Temp: 79 Deg F
Design AC Modulus: 455500 PSI

Rigid Parameters

Rigid Pavement SCI at Failure: 0

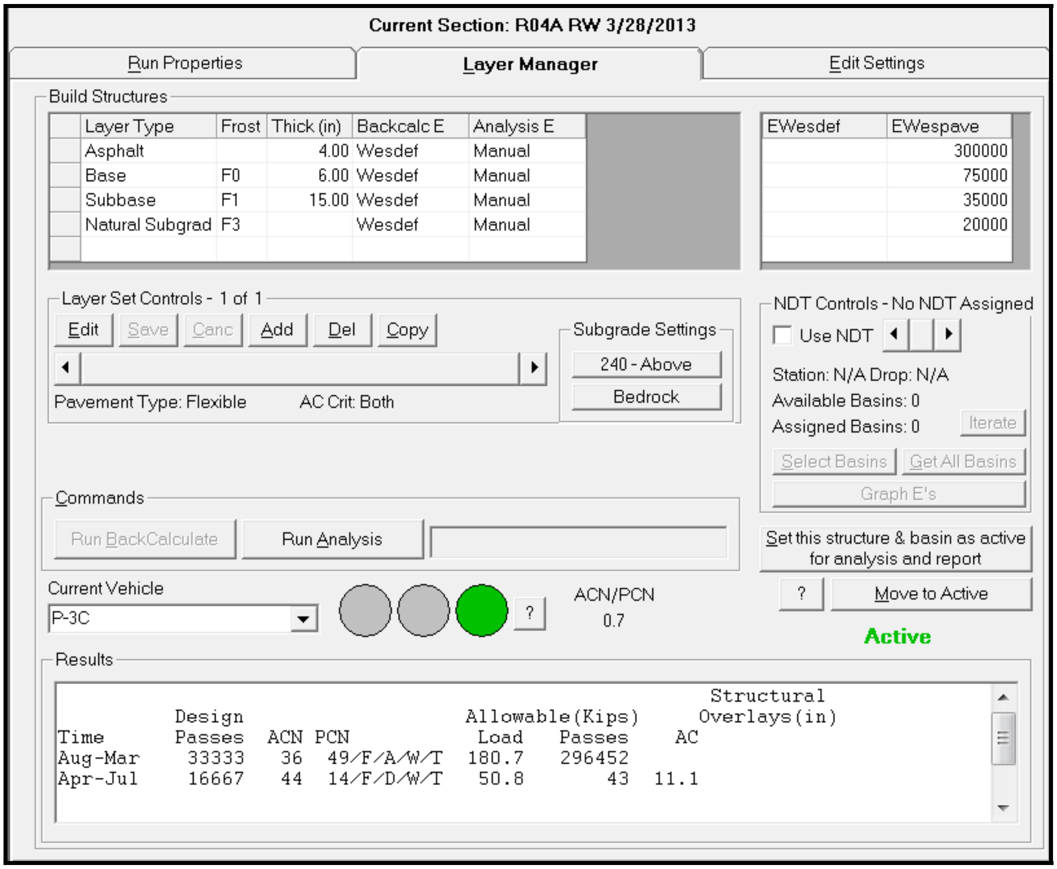
Load Transfer

Load Transfer 0 **25** 25
100% max edge stress
75% max edge stress
% Max Edge Stress: 75
Joint Deflection Ratio: 0.76
Load Reduction Factor: 1.00

☑ Evidence of Frost Damage

Seasonal Settings

Thaw Season: Apr to Jul
Depth Frost: 104



Current Section: R04A RW 3/28/2013

Layer Type	Frost	Thick (in)	Backcalc E	Analysis E
Asphalt		4.00	Wesdef	Manual
Base	F0	6.00	Wesdef	Manual
Subbase	F1	15.00	Wesdef	Manual
Natural Subgrad	F3		Wesdef	Manual

EWesdef	EWespave
	300000
	75000
	35000
	20000

Pavement Type: Flexible AC Crit: Both

☐ Use NDT

Station: N/A Drop: N/A
Available Basins: 0
Assigned Basins: 0

Current Vehicle: P-3C

ACN/PCN
0.7

```
                                                        Structural
          Design                   Allowable(Kips)    Overlays(in)
Time      Passes  ACN PCN           Load   Passes     AC
Aug-Mar    33333   36  49/F/A/W/T  180.7   296452
Apr-Jul    16667   44  14/F/D/W/T   50.8       43   11.1
```

7-19 **EVALUATION OF PAVEMENTS LOCATED ON PERMAFROST**. Typically, pavements located on permafrost are in their weakest condition during the summer. The permafrost melts from the top down and provides excess water that cannot drain because of the underlying frozen permafrost. Pavement evaluations are performed during the weakened state, which may only last a few months. This is essentially opposite of the evaluation procedures previously discussed in this chapter. The concept of reduced strength (FASSI, FAIR, reduced modulus values) does not apply in this situation. The following outlines a procedure that has been used to evaluate pavements in a permafrost area.

The pavement evaluation was conducted during the Summer with a Heavy Weight Deflectometer, when the pavement was in a weakened condition. Modulus values were established for each layer, including the saturated thawed layers and the frozen permafrost. This established the basis for the AGLs and PCNs published for the Summer period.

For the Winter period, the completely frozen period, it was assumed that the previously thawed layers (base, subbase or subgrade) would have the same modulus values when frozen as those established for the frozen permafrost during the Summer evaluation. It was also assumed that modulus values for the asphalt surface would be the same as established during the summer. These modulus values were used to determine the AGLs and PCNs during the Winter period.

CHAPTER 8

STANDARDIZED METHOD FOR REPORTING AIRFIELD PAVEMENT STRENGTH

8-1 **REFERENCES**.

a. Federal Aviation Administration Advisory Circular 150/5335-5A, Standardized Method of Reporting Airfield Pavement Strength-PCN. Background and basic procedures for determining ACN in this Chapter were extracted from this Advisory Circular.
b. International Civil Aviation Organization Aerodrome Design Manual, Part 3, Pavements.
c. Convention of International Civil Aviation-Aerodromes, Annex 14, Volume 1, Aerodrome Design and Operations.

8-2 **BACKGROUND**. The United States is a member of the International Civil Aviation Organization (ICAO) and is bound by treaty agreements to comply with the requirements of ICAO to the maximum extent practical. In 1977, ICAO established a Study Group to develop a single international method of reporting pavement strengths. The study group developed and ICAO adopted the Aircraft Classification Number - Pavement Classification Number (ACN-PCN) method. Using this method, it is possible to express the effect of an individual airplane on different pavements by a single unique number that varies according to airplane weight and configuration (e.g. tire pressure, gear geometry, etc.), pavement type, and subgrade strength. This number is the Aircraft Classification Number (ACN). Conversely, the load-carrying capacity of a pavement can be expressed by a single unique number, without specifying a particular airplane or detailed information about the pavement structure. This number is the Pavement Classification Number (PCN). ICAO only requires reporting the PCN for runways.

a. Definition of ACN**.** ACN is a number that expresses the relative effect of an airplane, at a given weight, on a pavement structure for a specified standard subgrade strength.

Definition of PCN**.** PCN is a number that expresses the bearing strength (load-carrying capability) of a pavement. USAF uses 50,000 passes of the C-17 to compute the PCN. The Army and Navy use the projected number of passes or equivalent passes the critical aircraft will make in the next 20 years.

c. System Methodology**.** The ACN-PCN system is structured so a pavement with a particular PCN value can support, without weight restrictions, an airplane that has an ACN value equal to or less than the pavement's PCN value. This is possible because ACN and PCN values are computed using the same technical basis.

Application. The use of the standardized method of reporting pavement strength applies only to pavements with bearing strengths of 12,500 pounds (5,700 kg) or greater.

2.2. Limitations of the ACN-PCN System. The ACN-PCN system is only intended as a method of reporting relative pavement strength so airport operators can evaluate acceptable operations of airplanes. It is not intended as a pavement design or pavement evaluation procedure, nor does it restrict the methodology used to design or evaluate a pavement structure. Operators should use the Allowable Loads or Allowable Passes contained in each Service's Pavement Evaluation Reports to manage day-to-day operations.

8-3 **DETERMINATION OF THE ACN**. Computation of the ACN requires detailed information on the operational characteristics of the airplane such as maximum aft center of gravity, maximum weight, wheel spacing, tire pressure, and other factors. ACN values can be obtained from the aircraft manufacturers, Transportation Systems Reports 13-2 and 13-3, or can be computed by PCASE.

8-3.1 Subgrade Category. The ACN-PCN method adopts four standard levels of subgrade strength for rigid pavements and four standard levels of subgrade strength for flexible pavements. These standard support conditions are used to represent a range of subgrade conditions as shown in Tables 8-1 and 8-2. Modulus values (E) for use in Layered Elastic analysis are shown in Tables 8-3 and 8-4. E values in Table 8-3 were obtained using $k=.07906(E^{0.7788})$. E values in Table 8-4 were obtained using E=1500CBR.

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

Table 8-1. Standard Subgrade Support Conditions for Rigid Pavement ACN and PCN Calculation

Subgrade Strength Category	CBR Value	Represents	Code Designation
High	15	$CBR \geq 13$	A
Medium	10	$8 < CBR < 13$	B
Low	6	$4 < CBR \leq 8$	C
Ultra Low	3	$CBR \leq 4$	D

Table 8-2. Standard Subgrade Support Conditions for Flexible Pavement ACN and PCN Calculation

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

Table 8-3. Standard Subgrade Support Conditions for Rigid Pavement ACN and PCN Calculation, Layered Elastic Procedure

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

Table 8-4. Standard Subgrade Support Conditions for Flexible Pavement ACN and PCN Calculation, Layered Elastic Procedure

8-3.2 Operational Frequency**.** Operational frequency is defined in terms of coverages that represent a full-load application on a point in the pavement. Coverages must not be confused with other common terminology used to reference movement of airplanes. As an airplane moves along a pavement section it seldom travels in a perfectly straight path or along the exact same path as before. This movement is known as airplane wander and is assumed to be modeled by a statistically normal distribution. As the airplane moves along a taxiway or runway, it may take several passes along the pavement for a specific point on the pavement to receive a full-load application. It is easy to observe the number of passes an airplane makes on a given pavement, but the number of coverages must be mathematically derived based upon the established pass-to-coverage ratio for each airplane.

8-3.3 Rigid Pavement ACN. For rigid pavements, the airplane landing gear flotation requirements are determined by the Westergaard solution for a loaded elastic plate on a Winkler foundation (interior load case), assuming a concrete working stress of 399 psi .

8-3.4 Flexible Pavement ACN. For flexible pavements, airplane landing gear flotation requirements are determined by the California Bearing Ratio (CBR) method for each subgrade support category. The CBR method uses a Boussinesq solution for stresses and displacements in a homogeneous, isotropic elastic half-space. To standardize the ACN calculation and to remove operational frequency from the relative rating scale, ACN values are determined for 10,000 coverages.

8-3.5 ACN Calculation. Using the parameters defined for each type of pavement, a mathematically derived single wheel load is calculated to define the landing gear/pavement interaction. The derived single wheel load implies equal stress to the pavement structure and eliminates the need to specify pavement thickness for comparative purposes. This is achieved by equating the thickness derived for a given airplane landing gear to the thickness derived for a single wheel load at a standard tire pressure of 181 psi The ACN is defined as two times the derived single wheel load (expressed in thousands of kilograms). The procedure for determining ACN is outlined in Reference 1.b.

8-3.6 Variables Involved in Determining ACN Values. Because airplanes can be operated at various weight and center of gravity combinations, ICAO adopted standard operating conditions for determining ACN values. The ACN is to be determined at the weight and center of gravity combination that creates the maximum ACN value. Tire pressures are assumed to be those recommended by the manufacturer for the noted conditions. Airplane manufacturers publish maximum weight and center of gravity information in their Airplane Characteristics for Airport Planning (ACAP) manuals.

8-3.7 Example Determination of ACN. From Figure 8-1, from PCASE**,** a C-130H aircraft operating at a weight of 120 kips on a rigid pavement with a subgrade k of 185 pci has an ACN of 24/R/C. The same aircraft operating on a flexible pavement with a subgrade CBR of 10 has an ACN of 21/F/B.

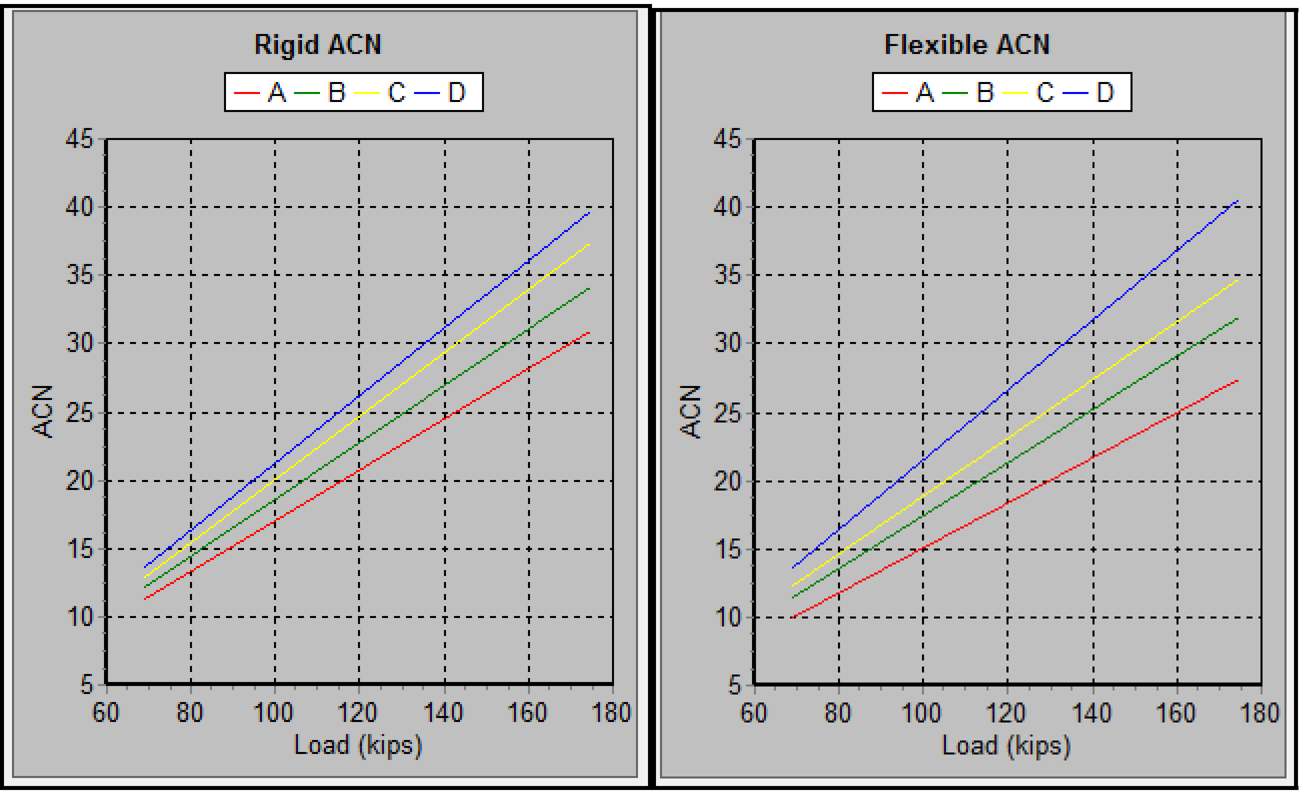


Figure 8-1. Rigid and Flexible ACN Values for C-130H Aircraft

8-4 **PCN CONCEPT**. The determination of a pavement rating in terms of PCN is a process of determining the ACN for the selected or most critical airplane and reporting the ACN value as the PCN for the pavement structure. Under these conditions, any airplane with an ACN equal to or less than the reported PCN value can safely operate on the pavement, subject to limitations on tire pressure. The Air Force selected the C-17 as the critical aircraft and a pavement life of 50,000 passes as the basis for calculating PCN. This allows the Air Force to compare capability of bases, since the basis of computing the PCN is the same at every base. ACN values for the maximum weight of the C-17 for the four subgrade strengths are shown in Table 8-3. The Army and Navy determine the critical aircraft for each base and project the estimated passes or equivalent for a 20-year period. They are then better able to manage day-to-day operations, using the PCN. The Air Force uses Allowable Loads or Allowable Passes to manage day-to-day operations.

RIGID	FLEXIBLE
52/R/A	40/F/A
50/R/B	45/F/B
54/R/C	53/F/C
66/R/D	70/F/D

Table 8-5. ACNs for C-17 Operating at 585 kips

8-4.1 Determining Numerical PCN Value. Determination of the numerical PCN value for a particular pavement can be based upon one of two procedures. The procedures are known as the “using” airplane method and the “technical” evaluation method. ICAO procedures permit member states to determine how PCN values will be determined based upon internally developed pavement evaluation procedures. DoD PCN values are based on the “technical” method, if at all possible.

8-4.2 Using Airplane Method. The using airplane method is a simple procedure where ACN values for all airplanes currently permitted to use the pavement facility are determined and the largest ACN value is reported as the PCN. An underlying assumption is that the pavement structure has the structural capacity to accommodate all airplanes in the traffic mix and that each airplane is capable of operating on the pavement structure without restriction. Significant over-estimation of the pavement capacity can result if an excessively damaging airplane, which uses the pavement on a very infrequent basis, is used to determine the PCN. Likewise, significant under-estimation of the pavement capacity can prevent acceptable traffic from operating. Use of the using airplane method is discouraged due to the above concerns.

8-4.3 Technical Evaluation Method. The accuracy of a technical evaluation is better than the using airplane procedure, but requires more time and resources. Pavement evaluation may require a combination of on-site inspections, load-bearing tests and engineering judgment. For DOD, the PCN numerical value is determined from an Allowable Load determined by a technical Pavement Evaluation conducted in accordance with this UFC. Once the Allowable Load is established, the determination of the PCN value is a simple process of determining the ACN of the airplane representing the Allowable Load and reporting the value as the PCN. The PCN can be determined from applicable ACN curves or computed by PCASE. **NOTE: When selecting the critical PCN for a runway, it is important to examine the entire PCN code, not just the numerical value. A lower numerical value may not be the critical PCN; it depends on the subgrade category. Examine the AGL when values with different subgrade categories are close and use the lower AGL.**

8-4.4 Limitations of the PCN. The PCN value is for reporting relative pavement strength only and should not be used for pavement design or as a substitute for evaluation. Pavement design and evaluation are complex engineering problems that require detailed analyses. They cannot be reduced to a single number.

8-4.5 Reporting PCN. The PCN system uses a coded format to maximize the amount of information contained in a minimum number of characters and to facilitate computerization. The PCN for a pavement is reported as a five-part number where the following codes are ordered and separated by forward slashes.

-Numerical PCN value
-Pavement type,
-Subgrade category,
-Allowable tire pressure, and
-Method used to determine the PCN.

8-4.6 Example PCN Reporting. An example of a PCN code is 80/R/B/W/T—with 80 expressing the PCN numerical value, R for rigid pavement, B for medium strength subgrade, W for high allowable tire pressure, and T for a PCN value obtained by a technical evaluation.

a. Numerical PCN Value. The PCN numerical value should be reported in whole numbers, rounding off any fractional parts to the nearest whole number. For pavements of diverse strengths, the controlling PCN numerical value for the weakest Section of the pavement should normally be reported as the strength of the pavement. Engineering judgment may be required in that, if the weakest Section is not in the most heavily used part of the runway, another representative Section may be more appropriate to determine the PCN.

b. Pavement Type. For the purpose of reporting PCN values, pavement types are considered to be either flexible or rigid structures. Table 8-4 lists the pavement codes for the purposes of reporting PCN.

Pavement Type	**Pavement Code**
Flexible	F
Rigid	R

Table 8-6. Pavement Codes for Reporting PCN

i) Flexible Pavement. Flexible pavements support loads through bearing rather than flexural action. They are normally comprised of several layers of selected materials designed to gradually distribute loads from the surface to the layers beneath. Each layer in the pavement structure is evaluated to determine structural capacity. The layer that produces the lowest Allowable Load is the controlling layer.

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

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

c. Subgrade Strength Category**.** As discussed above, there are four standard subgrade strengths identified for calculating and reporting ACN or PCN values. The standard values for rigid and flexible pavements are reported in Tables 8-1 and 8-2.

d. Allowable Tire Pressure. Table 8-5 lists the allowable tire pressure categories used in the ACN-PCN system. The tire pressure codes apply equally to rigid or flexible pavement sections; however, the application of the allowable tire pressure differs substantially for rigid and flexible pavements.

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

Table 8-7. Tire Pressure Codes for Reporting PCN

(i) Tire Pressures on Rigid Pavements. Tire pressure has little effect on pavements with Portland cement concrete surfaces. Rigid pavements are inherently strong enough to resist high tire pressures and can usually be rated as code W. However, when the rigid layer is very thin (less than 4 inches) or is thoroughly shattered (pieces less than about 2 feet wide), the tire pressure code should be reduced.

(ii) Tire Pressures on Flexible Pavements**.** Tire pressures may be restricted on asphaltic concrete, depending upon the quality of the asphalt mixture, climatic conditions, or thickness and condition of the surface. Tire pressure effects on an asphalt layer relate to the stability of the mix in resisting shearing or densification. A poorly constructed asphalt pavement can be subject to rutting due to consolidation under load. A properly prepared and placed mixture that conforms to DOD specifications can withstand tire pressures in excess of 254 psi. Pavements that are thinner than the minimum required by UFC 3-260-02 should not be rated above Code Y; pavements of poorer quality asphalt, or aged or severely cracked pavements should not be rated above 100 PSI.

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

8-4.7 Reporting the PCN Value. Once a PCN value and the coded entries are determined, the PCN code should be reported to:

NATIONAL Geospatial-Intelligence Agency
Attn: Air Information Library, L27
3838 Vogel Rd.
Arnold MO 63010

An airplane's ACN can then be compared with the published PCN to determine if the airplane can safely operate on the airfield’s runways, subject to any limitation on tire pressure.

8-5. **PAVEMENT OVERLOAD**. Overloading of pavements can result from loads too large or a substantially increased application rate, or both. Loads larger than the defined design or evaluation load shorten the design life, while smaller loads extend it. With the exception of massive overloading, pavements are not subject to a particular limiting load above which they suddenly or catastrophically fail. The structural behavior of pavements is such that a pavement can sustain a definable load for an expected number of repetitions during its design life. As a result, occasional overloading is acceptable, when expedient, with only a limited loss in pavement life expectancy and a relatively small acceleration of pavement deterioration anticipated. Examples of situations where operators may decide that it is acceptable to overload a pavement are emergency landings, short-term contingencies, exercises, and air shows. In the ACN/PCN methodology, a pavement can support operations of an aircraft if the PCN is equal to or greater than the ACN (i.e., ACN/PCN ≤ 1.0. For those operations in which the magnitude of load and/or the frequency of use do not justify a detailed analysis using the AGL/pass level methodology presented previously, ICAO suggests the following criteria as a “quick” approach:

a) For flexible pavements, occasional movements by aircraft with ACNs not exceeding the reported PCN by more than ten (10) percent (i.e., 1.0 ≤ ACN/PCN ≤ 1.1) should not adversely affect the pavement.
b) For rigid or composite pavements, in which a rigid pavement layer provides a primary element of the structure, occasional movements by aircraft with ACNs not exceeding the reported PCN by more than five (5) percent (i.e., 1.0 ≤ ACN/PCN ≤ 1.05) should not adversely affect the pavement.
c) If the pavement structure details are unknown, the five (5) percent limitation should apply (i.e., 1.0 ≤ ACN/PCN ≤ 1.05).
d) The annual number of movements by aircraft exceeding an ACN/PCN ratio of 1.0 should not exceed five (5) percent of the total annual aircraft movements.

NOTE: Movements by aircraft exceeding an ACN/PCN ratio of 1.0 should not normally be permitted on pavements exhibiting substantial signs of distress or failure. Furthermore, during any periods of thaw-weakening following frost penetration or when the strength of the pavement or its subgrade could be weakened by the presence of water, analysis should be performed using PCNs determined based on the criteria contained in Chapter 7.

NOTE: For the Air Force, ACN/PCN ratios exceeding the values presented above (i.e., 1.1 for flexible and 1.05 for rigid/composite/unknown structure pavements), the AGL/pass level methodology must be used to determine airfield structural capability.

8-6 **ADJUSTED ACN DUE TO INCREASE/DECREASE IN TIRE PRESSURE**. Tire pressure is a secondary factor in determining an ACN; however, ICAO procedures can determine the increase/decrease in ACN if the aircraft is operating at a tire pressure different than the one used to determine the ACN. The adjusted ACN can be used if conditions, such as a thin asphalt surface or a weak upper pavement layer, exist. Figures 8-3 and 8-4 are used to adjust the ACN for flexible pavements and Figure 8-5 is used for rigid pavements. On flexible pavements, it is assumed that adjustments will only be one category. Figures 8-3 and 8-4 were developed by ERDC based on the equation in Reference 2.3.

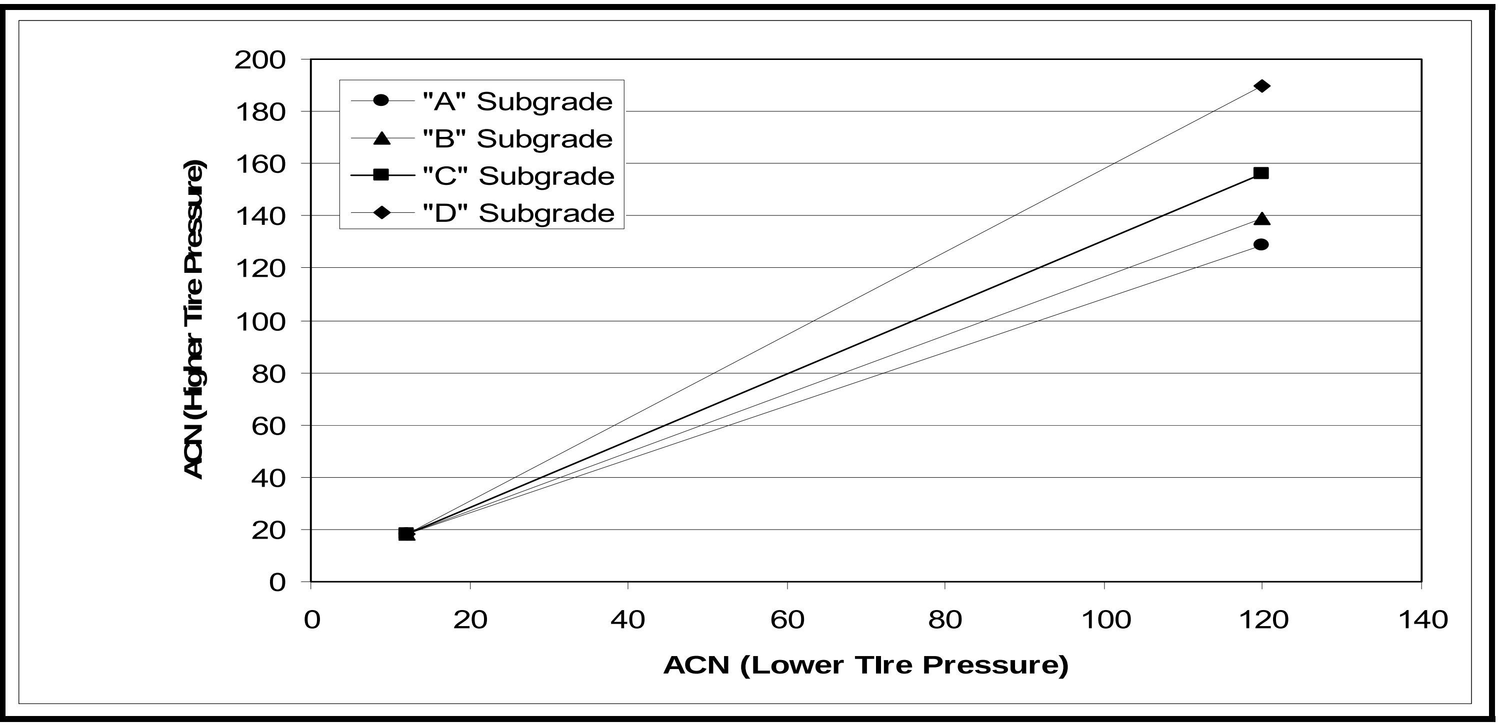


Figure 8-2. Adjusting Flexible Pavement ACN Due to Increase/Decrease in Tire Pressure (Z to Y or Y to Z)

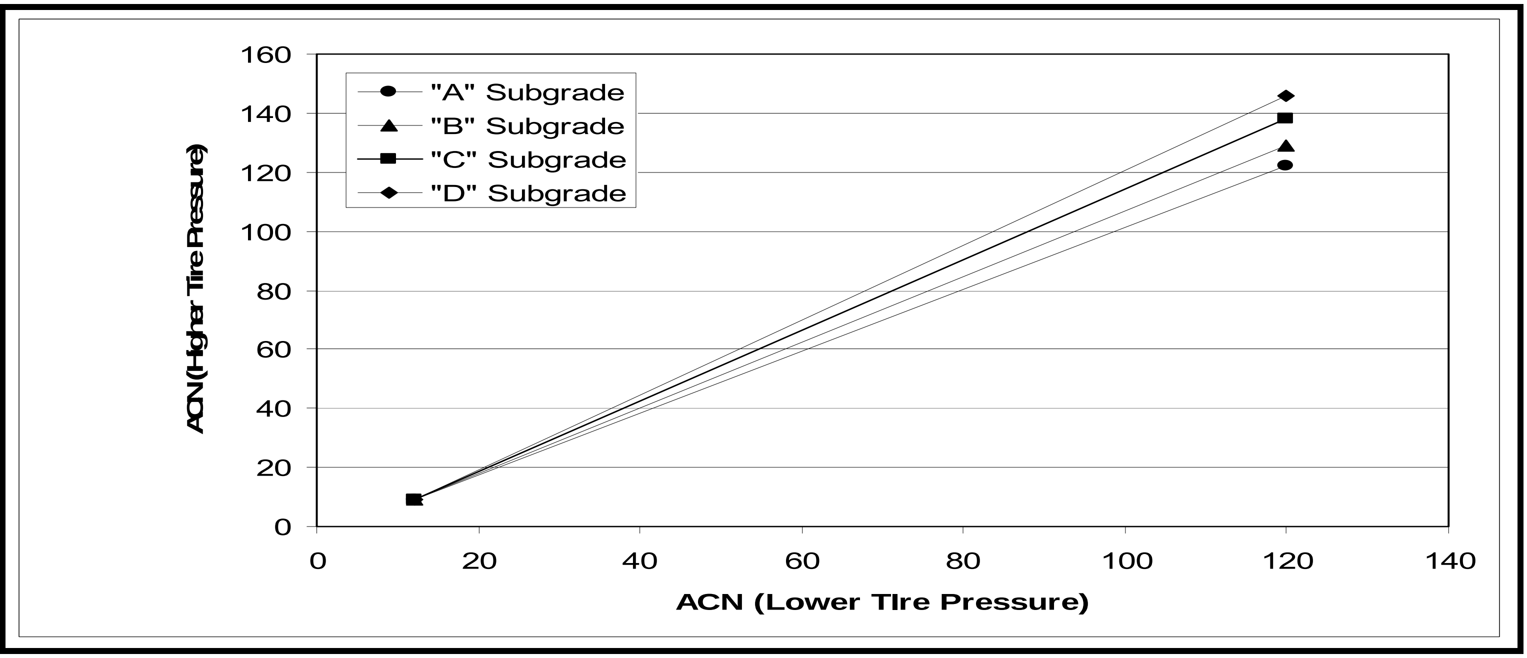


Figure 8-3. Adjusting Flexible Pavement ACN Due to Increase/Decrease in Tire Pressure (Y to X, X to Y, X to W, W to X)

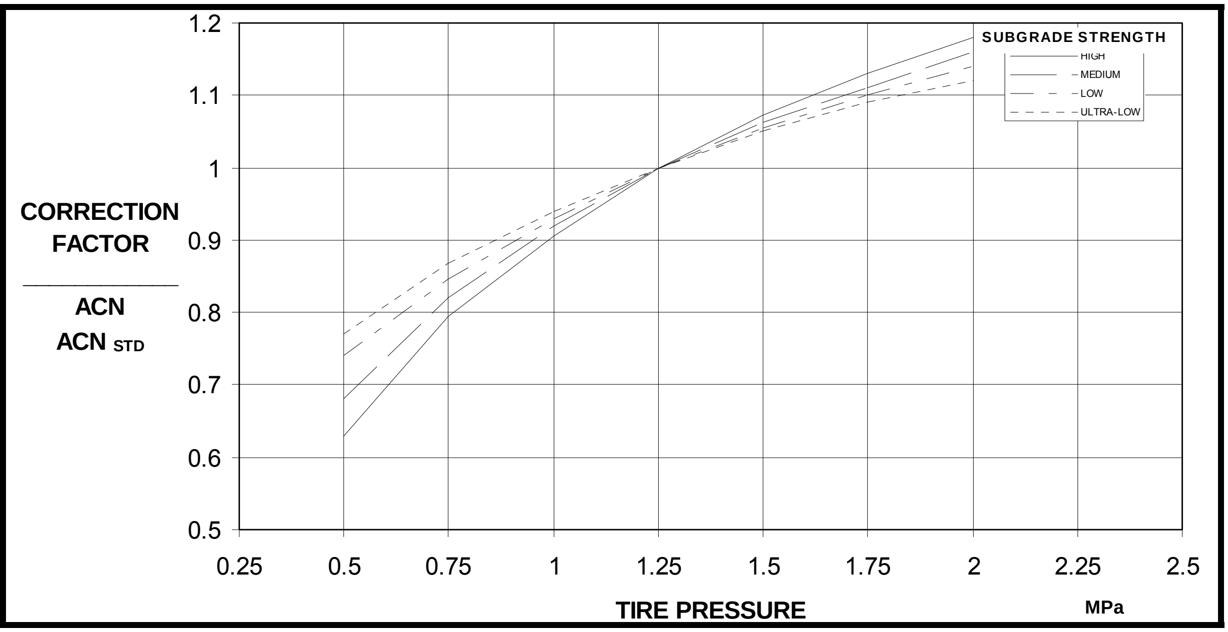


Figure 8-4. Adjusting Rigid Pavement ACN for Changes in Tire Pressure

a. Examples of Adjustment in ACN due to Tire Pressure.

(1) Flexible Pavement. The ACN for an aircraft was determined to be 60/F/D using a tire pressure of 160 psi. Using Figure 8-3, the ACN will be 55/F/D, if the tire pressure is reduced to 140 psi.

(2) Rigid Pavement. An aircraft operating at a tire pressure of 180 psi on a medium strength subgrade has an ACN of 50/R/B. The ACN will be 53/R/B if the tire pressure is increased to 215 psi (Enter Figure 8-5 with a tire pressure of 1.5 MPA and proceed vertically until the medium subgrade is intercepted. Proceed horizontally and read Correction Factor of 1.06. (50 x 1.06 = 53).

APPENDIX A

REFERENCES

A-1 GOVERNMENT PUBLICATIONS

1.1 Departments of the Army, the Navy, and Air Force

AFI 32-1041 Airfield Pavement Evaluation Program

UFC 3-260-02 Pavement Design for Airfields

TM 5-803-4 Planning of Army Aviation Facilities

UFC 3-250-11 Soil Stabilization for Pavements

1.2. Corps of Engineers (COE)

CRD-C 649 Standard Test Method for Unit Weight, Marshall Stability and Flow of Bituminous Materials

CRD-C 650 Standard Test method for Density and Percent Voids of Compacted Bituminous Paving Mixtures

CRD-C 653 Moisture Density Relations of Soils

CRD-C 654 California Bearing Ratio of Soils

CRD-C 655 Modulus of Soil Reaction

CRD-C 656 California Bearing Ratio and Pavement Sampling by the Small Aperture Procedure

ERDC Misc. Report S-73-56 Lateral Distribution of Aircraft Traffic

Transportation Systems Center TSC Report 13-2, Aircraft Characteristics for Airfield Pavement Design and Evaluation, Air Force and Army Aircraft

Transportation Systems Center TSC Report 13-3, Aircraft Characteristics for Airfield Pavement Design and Evaluation, Selective Commercial Aircraft

A-2 NONGOVERNMENT PUBLICATIONS

2.1. American Society for Testing and Materials (ASTM), 1916 Race Street, Philadelphia, PA 19103

C 39 Compressive Strengths of Cylindrical Concrete Specimens
C 42 Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
C 78 Flexural Strength of Concrete
C 127 Specific Gravity and Absorption of Coarse Aggregates
C 128 Specific Gravity and Absorption of Fine Aggregate
C 136 Sieve Analysis of Fine and Coarse Aggregate
C 496 Splitting Tensile Strength of Cylindrical Concrete Specimens
C 642 Density, Absorption, and Voids in Hardened Concrete
D 4 Bitumen Content
D 5 Penetration of Bituminous Materials
D 36 Softening Point of Bitumen (Ring-and-Ball Apparatus)
D 113 Ductility of Bituminous Materials
D 128 Analysis of Lubricating Grease
D 422 Particle Size Analysis of Soils
D 854 Specific Gravity of Soils
D 1556 Density and Unit Weight of Soil In-Place by the Sand Cone Method
D 1557 Laboratory Compaction Characteristics of Soil Using Modified Effort
D 1633 Test Method for Compressive Strength of Molded Soil-Cement Cylinders
D 1635 Test Method for Flexural Strength of Soil-Cement Using Simple Beam with Third Point Loading
D 1856 Recovery of Asphalt from Solution by Abson Method
D 2167 Density and Unit Weight of Soil In-Place by the Rubber-Balloon Method
D 2172 Quantitative Extraction of Bitumen from Bituminous Paving Mixtures
D 2216 Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil Aggregate Mixtures
D 2487 Classification of Soils for Engineering Purpose
D 2937 Density of Soil in Place by the Drive-Cylinder Method
D 4318 Liquid Limit, Plastic Limit, and Plasticity Index of Soils
D 4694 Test Method for Deflections with a Falling Weight-Type Impulse Load Device
D 5340 Airport Pavement Condition Index Surveys

2.2. American Concrete Institute (ACI), P.O. Box 9094, Farmington Hills, MI 48333-9094
ACI 544.2R-78 (R-83) Measurement of Properties of Fiber Reinforced Concrete

2.3. International Civil Aviation Organization, P.O. Box 400, Montreal, Quebec, Canada H3A2R2

Amendment Number 35 to the International Standards and Recommended Practices, Aerodromes, Annex 14 to the Convention of International Civil Aviation, March 1981.
Aerodrome Design Manual, Part 3

APPENDIX B

SAMPLING AND TESTING METHODS

B-1. **INTRODUCTION.** The following tabulation lists the sampling and testing normally performed in evaluating pavements. Many of these are standard, published methods, and the tabulation indicates the publication in which each standard method may be found. Some of the methods used are not described in readily available publications and therefore are described in subsequent paragraph herein.

Sampling Bituminous Paving Mixtures	ASTM D 979
Pavement cores	TM 5-825-2/AFM 88-6, Chap. 2
Unit weight, Marshall stability,and flow of bituminous mixtures	CRD-C 649
Density and percent voids of compacted bituminous paving mixtures	CRD-C 650
In-place density, sand cone method	ASTM D-1556
In-place (field) CBR	CRD-C 654
Laboratory CBR relations of soils	CRD-C 654
Moisture-density relations of soils	CRD-C 653
Sieve analysis	
Particle size analysis	
Specific gravity of soils	
Specific gravity and absorption of coarse aggregate	ASTM C 127
Specific gravity and absorption of fine aggregate	ASTM C 128
Moisture content of soil or aggregate (total sample)	ASTM D 2216
In-place density, drive cylinder method	ASTM D 2937
Liquid limit, plastic limit, and plasticity of soils	ASTM D 4318
Recovery of asphalt from solution by Abson method	ASTM D 1856
Extraction of bitumen from bituminous paving mixtures	ASTM D 2172
Recompaction of asphaltic concrete	Described below
Penetration of bituminous materials	ASTM D 5
Ductility of bituminous materials	ASTM D 113
Softening point of asphalt and tar materials	ASTM D 36
Test for bitumen	
Soils Sampling	
Plate-bearing tests	
Classification tests	
Sampling and preparation of test specimens	ASTM C 42

Flexural strength of concrete	ASTM C 78 as modified below
Compressive strength tests	ASTM C 39
Splitting tensile strength tests	ASTM C 496
Specific gravity of concrete	
Absorption by concrete	
Voids in concrete	ASTM C 642
Flexural strength of soil-cement	ASTM D 1635
Deep, quasi-static, cone and friction-cone penetration tests of soils	ASTM D 3441
Description and application of dual-mass dynamic cone penetrometer	FM 5-430-00-2/AFJPAM 32-8013, Vol II, Appendix J

Note: ASTM is the designation of standards and test methods issued by the American Society for Testing and Materials, 1916 Race Street, Philadelphia, PA 19103.

B-2. **RECOMPACTION OF ASPHALTIC CONCRETE**. Samples of existing pavements may be recompacted in the laboratory for comparison with the in-place conditions. The samples of pavement should be in the form of chunks of about 254-millimeter (10-inch) maximum dimension so that the various layers or course can be identified. If the pavement consists of more than one course, the courses should be separated and treated individually. The courses may be separated by heating the pieces of pavement and driving a hot knife between the layers or by other similar methods. After a course has been separated, it should be broken into small pieces and heated to a temperature of 115° to 127°C (240° to 260°F). The material should be thoroughly mixed during heating. Heating should be accomplished as rapidly as possible and should be performed in an oven or on a hotplate with constant stirring to ensure uniform heating. The hot mixture should be compacted in accordance with the standard procedures for the Marshall method. Compaction efforts of 50 and 75 blows on each side of the specimen should be used for comparison with criteria for tire pressures of 0.7 MPa and 1.4 MPa (100 and 200 pounds per square inch), respectively. Six or eight specimens should be compacted with each effort and tested in accordance with standard procedures for the Marshall method. In analyzing the test data, it should be recognized that reheating produces a hardening of the asphalt cement. This hardening causes somewhat higher stability values but has little effect on the other test values.

B-3. **SOILS SAMPLING**.

a. Disturbed Sampling. Two types of disturbed sampling will normally be required during an airfield pavement evaluation.

(1) Samples of the foundation materials will be needed for developing soil profiles, and

the most suitable method of obtaining these samples is by auger borings. These borings can be made into the foundation materials to the desired depth either in test pits or through small 102-millimeter or 152-millimeter- (4- or 6-inch-) diameter holes cored through the pavement. Samples of the foundation materials should be taken for each 152-millimeter (6-inch) vertical increment to a depth of 610 millimeter (2 feet) and for each 305-millimeter (12-inch) increment thereafter to the desired depth. Additional samples should be taken whenever there is a change in materials or moisture conditions. The samples should be sealed in jars and clearly marked before transportation to the laboratory, where they will be subjected to classification tests and moisture-content determinations.

(2) Samples of the foundation materials will also be required for compaction tests. Normally, these will be bag samples obtained from test pits. Samples of each type of material encountered should be obtained. The size of the bag samples required will depend on the type of material and the type of test to be performed. Generally, if the material is fine-grained, a 45-kilogram (100-pound) sample will be sufficient for the moisture-density determination; when the moisture-density-CBR relations are to be developed, a 204-kilogram (450-pound) sample should be obtained. If the material is granular, the size of the sample should be increased to 90 kilograms (200 pounds) for the moisture-density tests and 272 kilograms (600 pounds) for the moisture-density-CBR tests.

b. Undisturbed Sampling. If the subgrade is composed of a fine-grained cohesive material, undisturbed samples may be required for laboratory California Bearing Ratio (CBR) tests to evaluate a nonrigid overlay on rigid pavement. When laboratory CBR tests are required, an additional undisturbed sample will be needed. There is no prescribed method for obtaining undisturbed samples of the subgrade material. Any method that will provide enough material and maintain it in its existing condition is satisfactory. The method most widely used for undisturbed sampling is to trim a sample by hand to fit into a split cylinder of galvanized metal approximately 203 millimeters (8 inches) in diameter and at least 305 millimeters (12 inches) high. The sample should then be sealed at the sides and ends with paraffin to prevent moisture loss.

B-4. **PLATE-BEARING TESTS.** When the plate-bearing test is used to determine the k value on the surface of a pavement, such as required for the evaluation of a composite pavement or a rigid overlay on flexible pavement, the load reaction must be placed far enough away from the plates so that the stresses created by the load reaction will not influence the results of the plate-bearing tests. In general, the load reactions should be located on slabs adjacent to the slab on which the test is being performed and not less than 3.8 meters (12.5 feet) from the bearing plate. When the plate-bearing tests are performed on the surface on a pavement, the limitation outlined in chapter 3 of this manual will apply.

B-5. **MOISTURE-DENSITY-CBR RELATIONS.** The moisture-density-CBR relationships of the foundation materials may be required to evaluate a nonrigid overlay on rigid pavement and this should be developed as outlined in UFC 3-260-02.

B-6. **FLEXURAL STRENGTH TEST.** The flexural strength of the rigid pavement will be determined by the third-point loading procedure set forth in ASTM C 78 with the following modifications.

a. Test Specimens. For pavement thicknesses up to and including 305 millimeters (12 inches), the test specimens should have a square section with the width and thickness equal to the pavement thickness. For thicker pavement, either a square section with width and thickness equal to the pavement thickness can be used, or 152- by 152-millimeter (6- by 6-inch) beams can be cut from the top and bottom of the slab and tested with the results averaged to obtain a strength representative of the full section. With the 152- by 152-millimeter (6- by 6-inch) beams cut from the top and bottom of the slab, the slab required from the pavement may be much smaller than that required when the width and thickness of the specimen must equal the pavement thickness. The length of the specimen should be three times the thickness of the specimen plus approximately 152 millimeters (6 inches).

b. Procedure. The specimen shall be placed in the third-point loading apparatus and tested in its as-cast position. That is, the load shall be applied at the third points on the surface of the beam, which represents the pavement surface, and the load reaction will be located on the bottom of the beam, which represents the bottom of the pavement.

B-7. **SPLITTING TENSILE STRENGTH TESTS.** The splitting tensile strength test has been standardized by American Society Testing and Materials (ASTM). The procedures for conducting the test and calculating the splitting tensile strength of concrete cores are outlined in ASTM C 496. Essentially, the method consists of laying a concrete core with its longitudinal axis horizontal and then loading it along the longitudinal axis with a line load until the core splits along its diameter. The splitting tensile strength T is then computed from the equation:

$$T = \frac{2P}{\pi l d} \qquad \text{(eq B-1)}$$

where

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

l = length of core, millimeters (inches)

d = diameter of core, millimeters (inches)

A correlation should be established between the splitting tensile strength from 152-millimeter- (6-inch-) diameter cores and the beam flexural strength for each pavement where records indicate there is a difference in the properties of the concrete. If it is not possible to obtain samples for flexural beam tests, splitting tensile strengths for 152-millimeter (6-inch) diameter cores can be used with the following equation to obtain values of flexural strength for use in the evaluation. For 6-inch-diameter cores:

$$F = 1.02T + 210 \qquad \text{(eq B-2)}$$

where

F = flexure strength in psi

T = tensile splitting strength in psi

APPENDIX C

DETECTION OF VOIDS UNDER AIRFIELD PAVEMENTS

C-1 **GENERAL**. This Appendix provides technical guidance for detecting subsurface voids under airfield pavements. This assessment is being routinely applied at all Navy and Marine Corps airfields, and can be used at Air Force and Army airfields. The objective is to provide cost-effective and reliable methods to minimize the potential for accidental airfield pavement failure due to subsurface voids. For airfield pavements, the term void will be applied to either actual voids, voids filled with water, or simply pockets of very loose subgrade with low bearing capacity, since, ultimately, it is the effect on the load carrying capacity that is of importance.

C-2 **BACKGROUND**. Pavement failure due to subsurface voids has resulted in aircraft accidents and pavement failures at Navy and Air Force airfields, causing concerns for potential accidents and threats to life safety in the future, as facilities age and resources for maintenance and repair become scarce. In 1999, a pavement failure under the front gear of a trainer aircraft at NAS Pensacola spurred the evaluation of available technology and the development of a methodology to detect such subsurface weaknesses. The current optimum approach uses a combination of visual, non-destructive, and destructive testing. While the optimum detection protocol devised particularly targets pavements above drainpipe crossings, the method can be applied elsewhere.

C-3 **VOID DETECTION**.

3.1. Visual Inspection. Visual inspection of the airfield pavements should be performed with frequency sufficient to locate potential problem areas and satisfy the airfield manager of their operational safety. Such inspections should monitor pavements for conditions that may affect aircraft movement (FOD, depressions, pavement deterioration, etc.). Frequency should be determined by local physical conditions and operational tempo as to minimize the hazards. In particular, depressions and cracking can be indicative of subsurface deterioration. In flexible pavements, depressions are evident after a rainfall, or by the concentric marks left by the evaporated water. In rigid pavements, standard Navy 12½ by 15-ft concrete slabs cracked into two or more pieces, or larger slabs cracked into three or more pieces, as well as slabs that exhibit faulting at joints, may indicate underlying soft spots or voids. In particular, areas above drainpipe crossings should be carefully inspected since most problems appear above, or near these pipes. Problems observed in unpaved areas above a pipe are early warning signs of problems in nearby paved areas above the same pipe. Depressed pavement or shattered slabs surrounding drainage structures (catch basins) indicate infiltration of soil materials into the structure or pipe. Visual inspections can also follow Pavement Condition Index (PCI) guidelines, as detailed in ASTM Standards (such as ASTM D 5340 and D 6433).

3.2. Heavy Weight Deflectometer testing. If visual inspection suggests concern, further evaluation using a Heavy Weight Deflectometer (HWD) should be performed. The HWD investigation would cover all pipe crossings and additional suspect areas.

3.2.1. Data collection. For pipes under asphalt pavements, the data collection procedure is as follows:

-Identify the location of each pipe and mark it on the pavement.

-Follow each pipe but 10 feet offset to the left, and test every 10 feet (line A).

-Follow each pipe in the same direction, every 10 feet, but testing just above the pipe (line B).

-Follow each pipe in the same direction, every 10 feet, but offset to the right by 10 feet (line C).

For concrete pavements, the procedure is the same, except that the readings are taken at the center of each slab, typically 12.5 by 15 feet in dimension for the Navy, and often larger for the Air Force. Hence, typically three sets of readings are obtained for each distance along the pipe (except for the case where the pipe falls just in between 2 rows of concrete slabs and only 2 sets of readings are needed). The 10-foot distance was chosen because it is expected that the HWD cannot sense pavement deficiencies beyond a 5-foot radius. Since this methodology is primarily based on comparing successive drops at adjacent locations, a single drop at each location is typically sufficient. With a typical deflectometer, at each location a set of at least seven deflections is obtained, denominated D1 though D7, where D1 is the deflection under the load point, and D2 through D7 are typically at 15, 24, 36, 48, 60, and 72 inches from D1, respectively. Once the data are gathered, the impact stiffness modulus (ISM) can be used to assess the pavement relative strength at each drop location. This ISM (or ISM1) reflects the local pavement stiffness under the load point, and is found by dividing the load by D1. Similarly, the load can be divided by the other deflections, to give ISM2 = Load/D2, and so on, up to ISM7 = Load/D7. This is of interest since D1 mostly reflects the state of the pavement itself, whereas D7 mostly reflects the state of the subgrade. Using D1 alone is not sufficient to successfully detect voids under the pavement. The ISM1 through ISM7 plots along the pipes can be plotted and analyzed. They can also be normalized (by dividing each plot by the highest value in the plot) to determine relative effects of pavement weaknesses on each sensor.

3.2.2 (2) Data evaluation. Once the ISM plots are completed, the following rules can be followed to determine potentially weak areas:

-For asphalt pavements, an absolute ISM1 value below about 300 kips/inch is of concern (or represents a weak pavement)

-For concrete pavements, an absolute ISM1 value below 1000 kips/inch is of concern

-A relative ISM decay indicates an unexpected weakness

-A relative weakness in ISM1 indicates it is shallow

-A relative weakness in ISM7 indicates it is deep (about 3 to 20 feet)

-A relative weakness in both ISM1 and ISM7 indicates a general lack of support.

It is recommended that data evaluation be completed during data acquisition, so that weak areas can be marked immediately for later verification by penetrometer testing, as indicated below.

3.2.3 Load carrying capacity. The HWD will establish the effect of any subgrade weakness (or void) on the load-carrying capacity of the pavement.

3.2.4 Frequency. Periodic testing with a HWD is recommended at all pipe crossings, at least for airfields with a history of pipe problems. This HWD testing can be completed at the same time as the standard Pavement Classification Number (PCN) structural evaluation cycle, typically 10 years for Navy airfields.

3.2.5 Other areas. When coverage of large areas is required (e.g. where karst formations are prevalent), the current technology may not always be able to provide a cost-efficient solution. A risk analysis study indicated that for runways, two lines, 10 feet, on either side of the centerline, can be covered with the HWD and be cost effective. Along each line, for asphalt pavements, longitudinal testing should be completed at 10- or 20-foot spacing, and for Portland cement concrete pavements longitudinal testing should be completed at each slab center (e.g. 15-foot spacing for Navy airfields). For composite pavements, if the AC overlay is thick and the overlaid joints are not visible, they should be treated as asphalt (10- or 20-foot, spacing); if the AC overlay is thin (e.g. 2 inches) and the overlaid concrete joints are visible, then they should be treated as concrete (data should be obtained at the center of the overlaid slabs, i.e. at the center of the reflected cracks). The remainder of the runway is less likely to be used, and can be assessed, if deemed necessary, using a less reliable, but faster complementary technique, such as ground penetrating radar (GPR). Anomalies found with this complementary technique should be verified with the HWD.

3.3. Penetrometer testing. Weak areas revealed by the HWD (or the GPR and the HWD) should be further tested to determine the depth of the weakness, in order to identify the type of repair needed. This testing can be completed using

either a Dynamic Cone Penetrometer (DCP), or Standard Penetration Test (SPT) (see also Chapter 3, Section 4c(5) Penetrometer Tests, and Appendix B):
The Dynamic Cone Penetrometer (DCP or Automated DCP), in which the rod is pounded down using a 17.6-pound hammer dropped from a constant height of 22.6 inches. This system is portable, and its most recent version only needs a single operator. This system is designed to reach a depth of only 39 inches, but in testing weak areas for voids, this is typically sufficient.

The Standard Penetration Test (SPT) also called the Split-Spoon test because of the split-barrel used for soil sampling. This test is covered in ASTM D 1586. It consists of driving a split-barrel sampler to obtain both a representative soil sample and a measure of the soil resistance to penetration. The sampler is driven by dropping a 140-pound mass from a 30-inch height. The sampler is driven at 6-inch increments into the ground. For each increment the number of blows is recorded and is assumed to be representative of the soil strength. Typically the DCP and ECP have been easier to conduct than the SPT.

Data should be presented as CBR (California bearing ratio), or k (modulus of soil reaction) versus depth, or blows versus depth. A low CBR value (less than 3), a low k (less than 75), or a low number of blows, is indicative of a weak layer or an actual void. In some cases, the detected void will actually represent a separation of the concrete pavement and the underlying base. In these cases, after coring the pavement, the core may drop by a height representative of the void height. If the pavement is drilled instead, this separation will be more difficult to observe, and a bore scope may be necessary to assess the existing void. Prior to testing, it will be necessary to insure that no other buried utilities are present.

3.4. Video taping. Video taping the interior of pipe crossings is recommended when testing and/or visible failure is evident near or around pipe crossings, for example in the infield. It will help pinpoint the location of potential problem areas and define the need for maintenance and repair. Special attention should be paid to assessing pipe joints. Accumulations of fines near joints or other penetrations are a good indicator of a loss of subgrade material and possibly subgrade strength.

3.5. Alternate non-destructive techniques were evaluated for void detection, but are not believed to be as effective as the aforementioned tools in determining the existence of voids. Some could, however, provide useful complementary information, in particular:

-Ground Penetrating Radar (GPR) could not be used as a reliable tool to predict weak areas and should not be used as a primary tool for void detection at this time. However, a portable GPR is generally successful in locating the actual location of drainpipes and thickness of pavement layers, and potentially could be used to verify the extent of known voids, or to assess large areas with low traffic. Antenna frequencies around 250 to 500 MHz will allow assessing the subgrade at sufficient depth, assuming that neither clays nor water are present. Frequencies around 1 to 2 GHz will allow for pavement thickness determination but will not allow for the detection of any potential voids in the subgrade.

-Acoustic reflection sounding has been used by ASTM D 4580 to find delaminations in concrete bridge decks. Similarly, in some instances, a person walking alongside the HWD could hear a difference in the sound of the pavement spanning a shallow void, in particular with thin concrete pavements. This should be noted to bring attention to the potential of a void just under the slab during coring or drilling. Once coring or drilling has been completed, the use of a borescope has also been helpful in assessing the existence of such voids.

C-4 **VOID REPAIR AND PREVENTION**. Repair methods include pressure grouting, polymer injection, and removal and replacement. Prior to proceeding with repair, further investigation should be completed to determine if an actual void (or very loose area) is present, or if a deep layer of weak material is responsible for the readings. Coring or drilling can provide an opportunity to verify voids existing directly beneath the pavement surface, and penetration testing and probing can help to identify weak or void areas at greater depths. Pressure grouting and polymer injection may successfully fill a void (or compact a locally loose area), but may only have very limited success in compacting a deep layer of weak material. In the latter case, the injection may simply create polymer (or grout) lenses (i.e. thin layers) that will lift the pavement, but may not be able to provide additional compaction because there is typically no overburden on the pavement top to provide the required reaction. Note that where pavement surface integrity is sound and load carrying capacity is adequate, pavement lifting can be used to re-establish ride quality. If no void is present, and a weak subgrade is undermining the load carrying capacity, removal and replacement of the weak layer should be considered. Note that if the weak layer is under the water table, removal and replacement can become very difficult.

If an actual void is present, then lightweight polymer or grout injections may be preferred to removal and replacement because of the minimal impact on aircraft operations. Once set, grout provides a stiff material typically usable for any type of subgrade, and can also be used to fill gaps just under the slab. Lightweight polymer injection has some advantages over grout injection: 1) in case of soft subgrades and large voids, less weight is added, 2) a properly mixed polymer typically reaches most of its strength in a few minutes, and 3) the quick-setting polymer can seal large drain pipe cracks and deep sinkholes, while grout could flow down the sinkhole and proceed into the pipes. If polymer injection is used, the modulus of elasticity of the polymer needs to exceed the stiffness of the layer where it is injected, and therefore it should typically only be injected into the subgrade. Even then, tests on some limited data indicate that this requires a minimum density of:

-6 pcf for subgrades with elastic modulus of 6,000 psi

-10 pcf for subgrades with elastic modulus of 15,000 psi

-15 pcf for subgrades with elastic modulus of 25,000 psi

If lightweight polymer or grout injection is not available, then pavement and base removal and replacement can be considered, down to the prescribed depth. When pipe deterioration is extensive, internal pipe repair, jacketing, or pipe replacement should be considered.

APPENDIX D

AIRCRAFT GEAR CONFIGURATION NOMENCLATURE

D-1 **PURPOSE**. This Appendix establishes a standard convention for naming and characterizing aircraft landing gear configurations. Although it is primarily directed at fixed wing airplanes, it is applicable to any aircraft using wheels for landing purposes. Data in this Appendix was extracted from FAA Order 5300.7, Standard Naming Convention for Aircraft Landing Gear Configurations.

D-2. **BACKGROUND**. Landing gear configuration and aircraft gross weight are an integral part of airfield pavement design and are often used to characterize pavement strength. Historically, most aircraft used relatively simple gear geometries such as a single wheel per strut or two wheels side by side on a landing strut. As aircraft became larger and heavier, they required additional wheels to prevent individual wheel loads from introducing excessively high stresses into the pavement structure. For economy and efficiency reasons, aircraft manufacturers added more wheels per landing strut whenever possible. This often led to groups of wheels placed side-by-side and in tandem configurations.

2.1.Typical Gear Configurations. Up until the late 1980s, the majority of civilian and military aircraft used three basic gear configurations: the “single wheel” (one wheel per strut), the “dual wheel” (two wheels side by side on a strut), and the “dual tandem” (two wheels side by side followed by two additional side-by-side wheels). As aircraft continued to increase in gross weight, manufacturers attempted to limit the damage imparted to pavements by increasing the total number of wheels. This was typically done by adding additional landing struts to the aircraft. For example, McDonnell Douglas originally manufactured the DC-10 with two landing struts using the dual tandem gear configuration. When the company produced the heavier DC-10-30 variation of the aircraft, it added an additional landing strut, using a dual wheel configuration, to the center of the aircraft. Another example is the Boeing 747 aircraft. To reduce the impact to airfield pavements, Boeing used four landing struts with dual tandem configurations on the B-747.

2.2.Complex Gear Configurations. The increasingly complex gear arrangements quickly outgrew the simple single, dual, and dual tandem descriptions. Additionally, other aircraft were developed with gear configurations that used numerous wheels in arrangements that could not be described by the three simple gear configurations. As the number and complexity of gear arrangements increased and with no coordinated effort to provide a uniform naming convention, the FAA, U.S. Air Force, and U.S. Navy developed different naming systems that were not easily cross-referenced.

D-3 **DEFINITIONS**.

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

3.2.Body/Belly Gear. “Body/belly gear” refers to an additional landing gear or gears in the center portion of the aircraft between the main gears. Body/belly gears may be of a different type than the main gear and may be nonsymmetrical.

D-4 **INTENDED AREAS OF USE.** The naming convention shown in Figure 1 is intended for use in all civilian and military applications. All FAA pavement design guidance and FAA databases and database publications, e.g. 5010 Master Record, Airport/Facilities Directory, etc., use the described aircraft gear naming convention. This Appendix adopts this system for DOD.

D-5 AIRCRAFT GEAR GEOMETRY NAMING CONVENTION.

5.1.Basic Name for Aircraft Gear Geometry. Under the naming convention, abbreviated aircraft gear designations may include up to three variables: the main gear configuration, the body/belly gear configuration if body/belly gears are present, and an optional tire pressure code described below. Figure 1 illustrates the two primary variables.

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

5.3.Basic Gear Codes. This naming convention uses the following codes for gear designation purposes (see Figure 2):

S	Single
D	Dual
T	Triple
Q	Quadruple

5.4.Use of Historical Tandem Designation**.** Although the verbal description continues to use the term “tandem” to describe tandem gear configurations, the tandem designation “T” no longer appears in the gear name. “T” now indicates triple wheels.

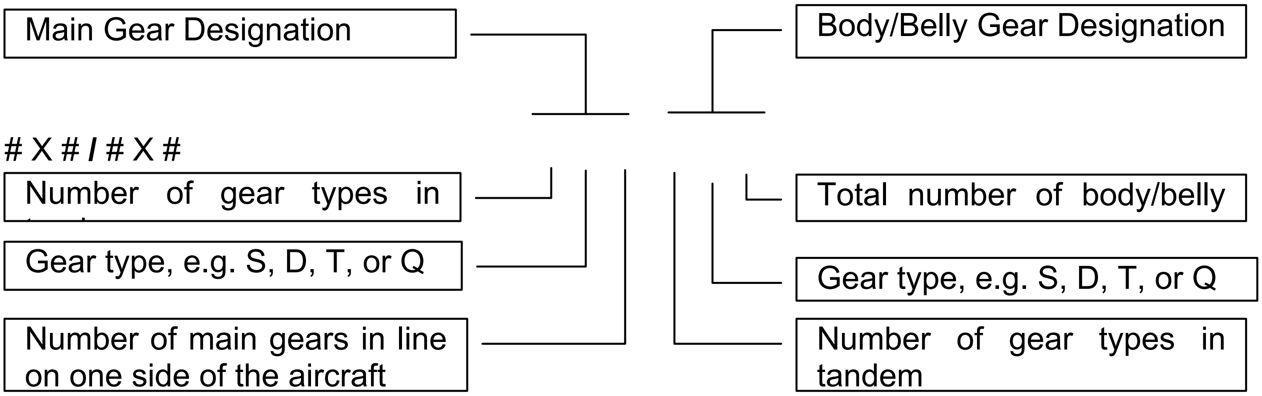


Figure 1. Aircraft Gear Naming Convention

5.5. Main Gear Portion of Gear Designation. The first portion of the aircraft gear name comprises the main gear designation. This portion may consist of up to three characters. The first character indicates the number of tandem sets or wheels in tandem, e.g. 3D = three dual gears in tandem. (If a tandem configuration is not present, the leading value of “1” is omitted.) Typical names are S = Single, 2D = two dual wheels in Tandem, 5D = five dual wheels in tandem, and 2T = two triple wheels in tandem.

5.5.1 The second character of the gear designation indicates the gear code, e.g. S, D, T, or Q.

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

5.6. Body/Belly Gear Portion of Gear Designation. The second portion of the aircraft gear name is used when body/belly gears are present. If body/belly gears are present, the main gear designation is followed by a forward slash (/), then the body/belly gear designation. For example, the B-747 aircraft has a two dual wheels in tandem main gear and two dual wheels in tandem body/belly gears. The full gear designation for this aircraft is 2D/2D2. The body/belly gear designation is similar to the main gear designation except that the trailing numeric value denotes the total number of body/belly gears present, e.g. 2D1 = one dual tandem body/belly gear; 2D2 = two dual tandem body/belly gears.

Because body/belly gear arrangement may not be symmetrical, the gear code must identify the total number of gears present, and a value of 1 is not omitted if only one gear exists.

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

5.8. Unique Gear Configurations. The Lockheed C-5 Galaxy has a unique gear type and is difficult to name using the proposed method. This aircraft will not be classified using the new naming convention and will continue to be referred to directly as the C5. Gear configurations such as those on the Boeing C-17, Antonov AN-124, and Ilusyin IL-76 might also cause some confusion; see Figures 8, 14, and 20, respectively. In these cases, it is important to observe the number of landing struts and the proximity of the struts. In the case of the AN-124, it is more advantageous to address the multiple landing struts as one gear, i.e. 5D or five duals in tandem, rather than use D5 or dual wheel gears with five sets per side of the aircraft. Due to wheel proximity, the C-17 gear is more appropriately called a 2T as it appears to have triple wheels in tandem. In contrast, the IL-76 has considerable spacing between the struts and should be designated as a Q2.

5.9. Examples of Gear Geometry Naming Convention**.** Figure 2 provides examples of generic gear types in individual and multiple tandem configurations. Figures 3 through 20 provide examples of known gear configurations.

5.10. Comparison of Naming Convention to Historical Procedures. Table 3 demonstrates the proposed naming convention and references the historic FAA, U.S. Air Force, and U.S. Navy methods. The historic Air Force methodology also addresses the configuration of the aircraft nose gear. Due to the insignificance of the pavement load imposed by the nose gear, the proposed method does not address nose gear configuration.

5.11. Inclusion of Tire Pressure Information. In addition to specifying gear geometry, the aircraft gear designation can also indicate the tire pressures at which the aircraft operates. Although tire pressure effects on airfield pavements are secondary to aircraft load and wheel spacing, they can have a significant impact on the ability of the pavement to accommodate a specific aircraft.

5.11.1 The Aircraft Classification Number (ACN) and the Pavement Classification Number (PCN) system created by the International Civil Aviation Organization (ICAO) has defined and categorized aircraft tire pressures into four groups for reporting purposes. Table 1 lists these groups and their assigned codes.

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

Table 1. Standard Tire Pressure Categories

5.11.2 To allow for the reporting of tire pressure, the gear naming convention includes a third variable. Using the codes identified by the International Civil Aviation Organization (ICAO), the tire pressure can be included in parentheses after the standard gear nomenclature. Table 2 provides sample gear names with and without the additional tire pressure code.

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

Table 2. Sample Gear Names With and Without Tire Pressure Codes

		Historic FAA Designations					U.S. Air Force Designations				U.S. NAVY Designations			
PROPOSED NOMENCLATURE	**Reference Figure**	**FAA Name**	**Main Gear**	**Belly Gear**	**# Belly Gear**	**Total # Wheels, Excluding Nose**	**Air Force Designation**	**Air Force Types**	**Air Force Name**	**NOSE GEAR**	**Navy Name**	**Navy Designation**	**DOD Flight Information**	
S	3	Single Wheel	SW			2	S	A	Single, Tricycle	Single Wheel	Single Tricycle	ST	S	F-1
S	4	Single Wheel	SW			2	S	B	Single, Tricycle	Dual wheel				
D	5	Dual wheel	DW			4	T	C	Twin, Tricycle	Single Wheel				Be
D	6	Dual wheel	DW			4	T	D	Twin, Tricycle	Dual wheel	Dual Tricycle	DT	T	B-
2S	7	Single Tandem				4	S-TA	E	Single, Tandem Tricycle	Dual wheel	Single Tandem Tricycle	STT	ST	C-
2T	8					12	TR-TA	L	Twin-Tandem, Tricycle	Dual wheel	Triple Tandem	TRT	TRT	C-
2D	9	Dual Tandem	DT			8	T-TA	F	Twin-Tandem, Tricycle	Dual wheel	Dual Tandem Tricycle	DTT	TT	B7 C1
2D/D1	10	Dual tandem	DT	DW	1	10	T-TA	H	Twin-Tandem, Tricycle	Dual wheel	Single Belly Twin Tandem	SBTT	SBTT	L1
2D/2D1	11	Dual Tandem	DT	DT	1	12				Dual wheel				A3
2D/2D2	12	Double Dual Tandem	DT	DT	2	16	T-TA	J	Twin-Tandem, Tricycle	Dual wheel	Double Dual Tandem	DDT	DDT	B-
3D	13	Triple dual Tandem	TDT			12				Dual wheel				B-
5D	14					20				4 across				An
7D	15					28				4 across				An
2D/3D2	16		DT	TDT	2	20				Dual wheel				A3
C5	17					24	T-D-TA	K	Twin-Delta-Tandem, Tricycle	4 across	Twin Delta Tandem	TDT	TDT	C-
D2	18					8	T-T	G	Twin-Twin, Bicycle	No Nose Gear - single outrigger	Twin Twin Tricycle	TT	TT	B-
Q	19					8								HS
Q2	20					16								IL-

Table 3. Proposed Naming Convention with Historical FAA, U.S. Air Force, and U.S. Navy Nomenclatures

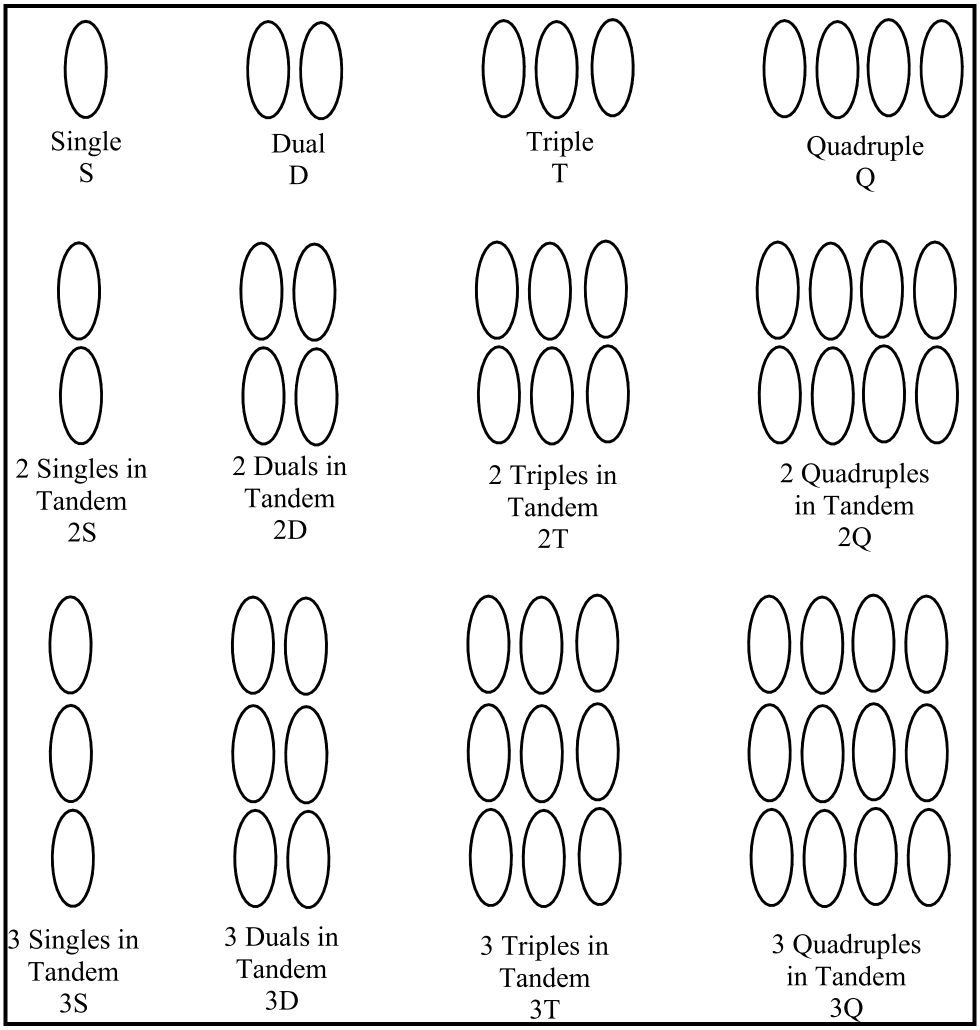


Figure 2. Generic Gear Configurations (Increase Numeric Value for Additional Tandem Axles)

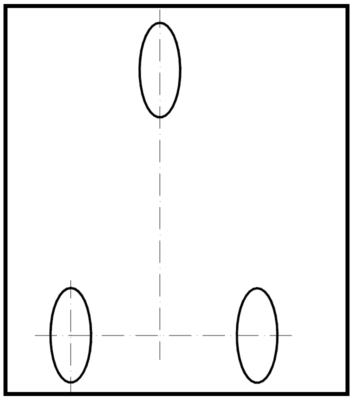


Figure 3. S - Single Wheel Main Gear with Single Wheel Nose Gear

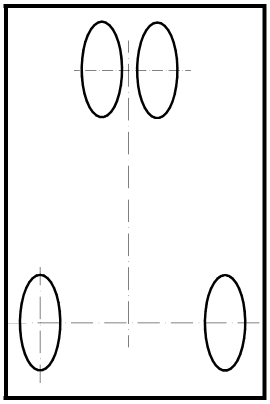


Figure 4. S - Single Wheel Main Gear with Dual Wheel Nose Gear

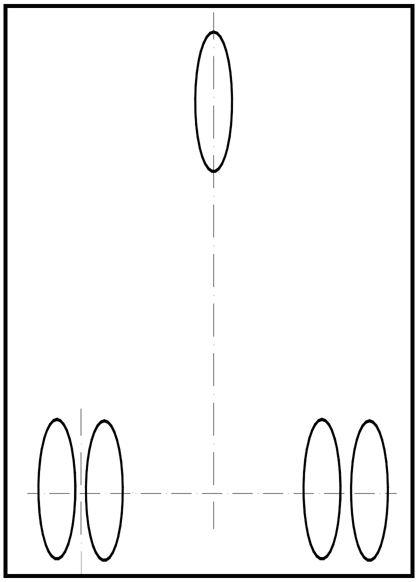


Figure 5. D - Dual Wheel Main Gear with Single Wheel Nose Gear

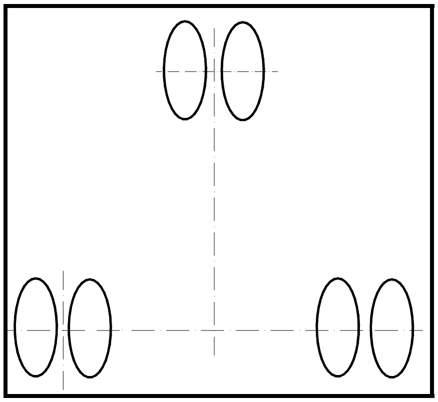


Figure 6. D - Dual Wheel Main Gear with Dual Wheel Nose Gear

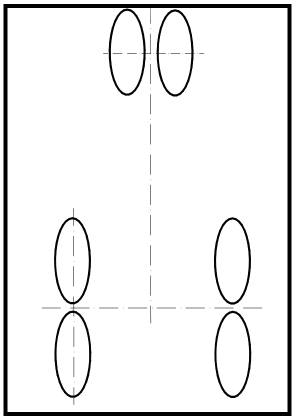


Figure 7. 2S - Two Single Wheels in Tandem Main Gear with Dual Wheel Nose Gear, Lockheed C-130

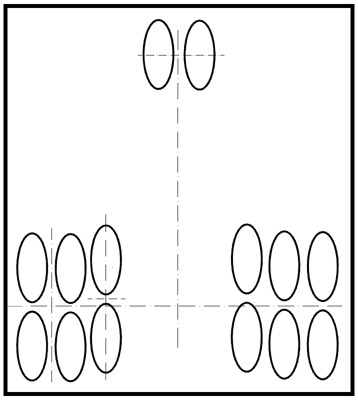


Figure 8. 2T - Two Triple wheels in Tandem Main Gear with Dual Wheel Nose Gear, Boeing C-17

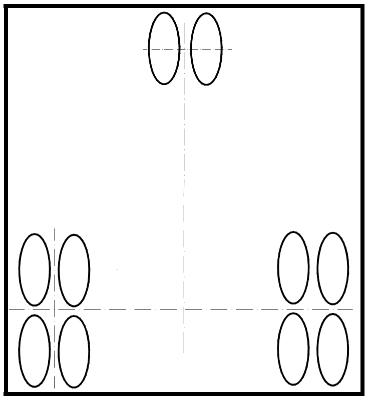


Figure 9. 2D - Two Dual Wheels in Tandum Main Gear with Dual Wheel Nose Gear

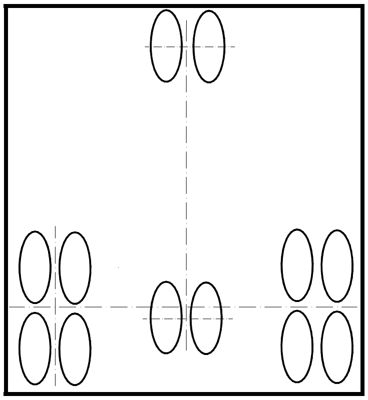


Figure 10. 2D/D1 - Two Dual Wheels in Tandum Main Gear/Dual Wheel Body Gear with Dual Wheel Nose Gear, McDonnell Douglas DC-10, Lockheed L-1011

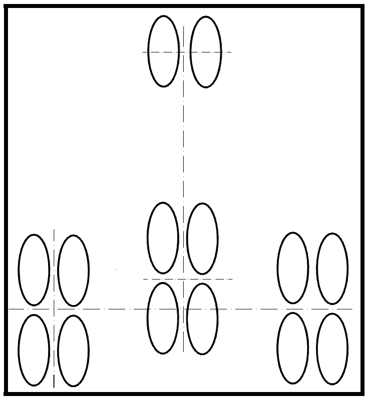


Figure 11. 2D/2D1 Two Dual Wheels in Tandem Main Gear/Two Dual Wheels in Tandum Body Gear with Dual Wheel Nose Gear, Airbus A340-600

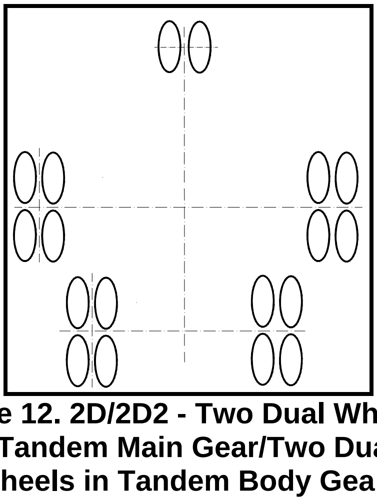


Figure 12. 2D/2D2 - Two Dual Wheels in Tandem Main Gear/Two Dual Wheels in Tandem Body Gear with Dual Wheel Nose Gear, Boeing B-747

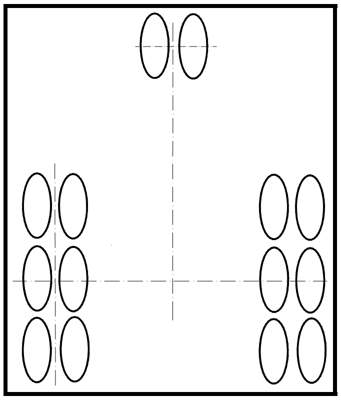


Figure 13. 3D - Three Dual Wheels In Tandem Main Gear with Dual Wheel Nose Gear, Boeing B-777

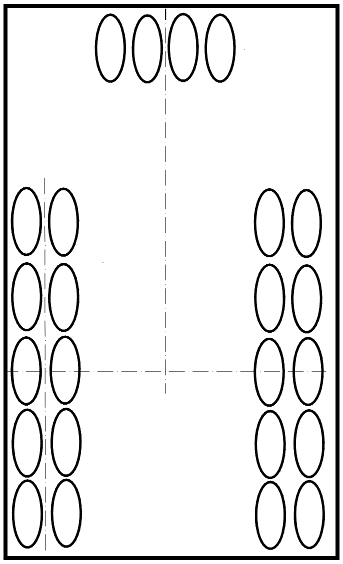


Figure 14. 5D - Five Dual Wheels in Tandum Main Gear with Quadruple Wheel Nose Gear, Antonov AN-124

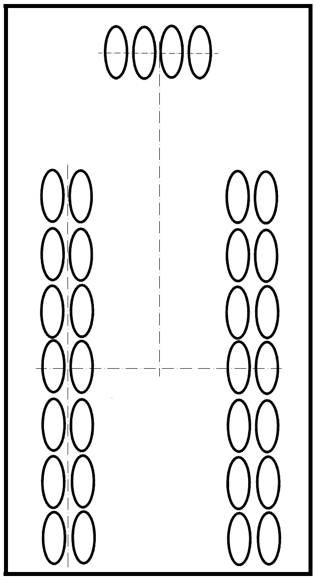


Figure 15. 7D - Seven Dual Wheels in Tandem Main Gear with Quadruple Nose Gear, AN-225

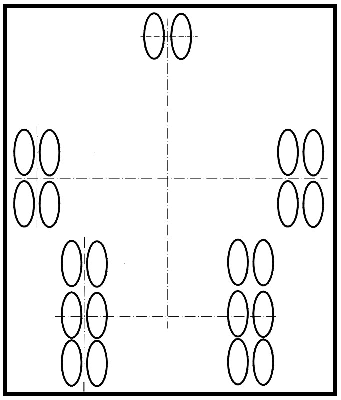


Figure 16. 2D/3D2 - Two Dual Wheels In Tandem Main Gear/Three Dual Wheels in Tandem Body Gear with Dual Wheel Nose Gear, Airbus A380

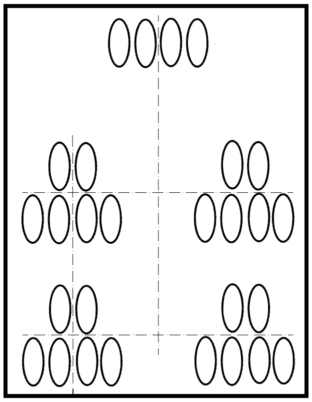


Figure 17.C5-Complex Gear Comprised of Dual Wheel and Quadruple Wheel Combination with Quadruple Wheel Nose Gear, Lockheed C5 Galaxy

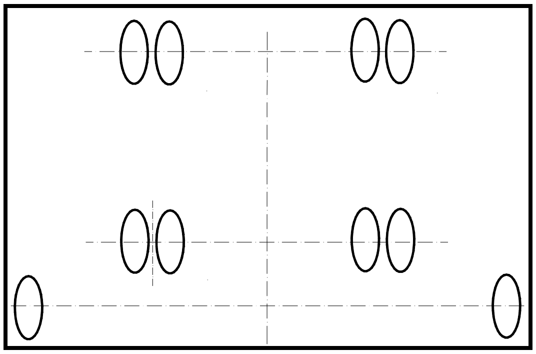


Figure 18. D2 - Dual Wheel Gear Two Struts per Side Main Gear with No Separate Nose Gear (note that single wheel outriggers are ignored), Boeing, B-52 Bomber

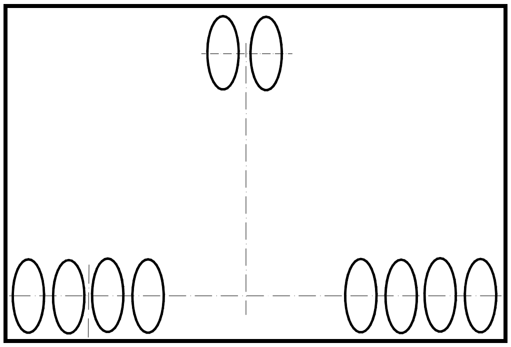


Figure 19. Q - Quadruple Wheel Main Gear with Dual Wheel Nose Gear, Hawker Siddeley HS-121 Trident

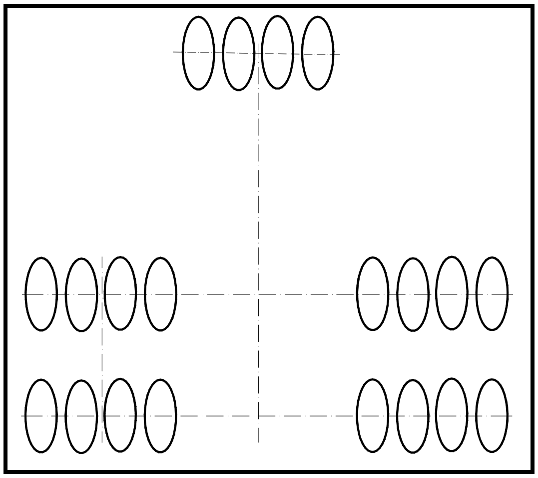


Figure 20. Q2 - Quadruple Wheels Two Struts per Side with Quadruple Nose Gear, Ilyushin IL-76

APPENDIX E

EVALUATION PROCEDURE FOR AGED ASPHALT CONCRETE (AC) SURFACES

E-1 **PURPOSE**. This Appendix provides guidance and methods for evaluating aged (i.e., three years or older) asphalt concrete (AC) surfaces in the field.

E-2 **PREFACE.** It is often necessary for military operations to use existing airfields that consist of aged or brittle AC surfaces. The ability to select suitable operating surfaces in the theater of operations is limited by the standard practices of airfield pavement evaluations, which have failed to identify problems caused by the use of aged AC pavements. Military missions may be severely impacted without the ability to predict the performance of aged AC pavements. Field and laboratory tests of aged and unaged AC conducted by the U.S. Army Engineer Research and Development Center concluded that the current DOD criterion for AC fatigue life has difficulty predicting fatigue failure for aged AC surfaces (Bell and Freeman 2007). The testing protocol presented herein is new criteria for pavement performance predictions at or close to 77^{o}F and an adjustment to the current DOD criterion at any other pavement temperature. The new criteria and the adjusted criterion are necessary for DOD organizations in order to improve fatigue life predictions of aged AC pavements (Bell, et. al., 2007).

E-3 **RELEVANT STANDARD TEST METHODS**:
ASTM D 6931-07, Standard Test Method for Indirect Tensile (IDT) Strength of Bituminous Mixtures

E-4 **AGED ASPHALT CONCRETE PERFROMANCE PREDICTION METHODS.**

4.1. Description of Tests.

4.1.1 Portable Seismic Pavement Analyzer (PSPA) Tests. The PSPA is a nondestructive testing device that rapidly measures Young’s modulus via ultrasonic surface waves. The PSPA is used to estimate the in situ seismic modulus of near surface pavement layers and determine relative strength parameters for use in pavement evaluations. The device is operated with a laptop computer, which is connected to an electronics box by a cable that transmits power to the receivers and the source. The source impacts the pavement surface, generating surface waves that are detected by the receivers. The measured signals are returned to the data acquisition board in the computer. The velocity at which the surface waves propagate is determined and the modulus is computed. PSPA tests can be completed within a few seconds and should be conducted at least three times in the same location so that an average modulus can be determined. At least ten PSPA measurements should be obtained for each airfield feature (Bell, 2006).

The modulus of AC pavements is dependent upon temperature; therefore, a design modulus must be used to standardize the PSPA results for purposes of predicting pavement performance. The AC design modulus is used to adjust the modulus measured by the PSPA at the field temperature, to a temperature of 77°F (25°C) and a design frequency of 15 Hz using equation 1.

$$E_{77^o F} = \frac{E_{PSPA}}{\left[(-0.0109 * ((T-32) * \frac{5}{9}) + 1.2627) * (3.2)\right]} \qquad \text{(Eq 1)}$$

E_{77^oF} = AC design modulus, ksi

E_{PSPA} = modulus measured from the PSPA, ksi

T = AC pavement temperature, °F

The AC design modulus is incorporated with the test results during the data analysis phase. E_{77^oF} is the adjusted AC modulus value to use for recording or analyzing data (Bell, 2006).

Indirect Tensile Strength (ITS) Tests. ITS tests are used to determine the tensile strength or stress of a cored sample. The tests are performed by loading a cylindrical specimen on a machine where a compressive load at a controlled deformation rate of 2 inches per minute is applied. The peak load at failure is recorded and used to calculate the ITS peak stress of the cored specimen. The ITS test procedure is presented in ASTM D 6931-07. Equation 2 is used to calculate the ITS peak stress of a sample.

$$S = \frac{2 * P}{\pi * t * D} \qquad \text{(Eq 2)}$$

S = ITS peak stress, psi

P = maximum load, lb

t = specimen height before test, in.

D = specimen diameter, in.

4.2 Testing Protocol. If predicting pavement performance (passes to failure), pavement evaluations should include either PSPA tests to determine the AC design modulus (at 15 Hz and 77°F) or ITS tests on a 4-in. diameter cores to determine the peak stress.

4.2.1. PSPA Modulus. If the PSPA is included in a pavement evaluation, to indicate AC integrity in terms of elastic modulus, use Equation 3 to predict the fatigue life of aged, field AC at or around 77°F. The PSPA can be used on an AC surface at any pavement temperature; however, the field measured modulus must be adjusted to the AC design modulus using Equation 1 as shown in section 4.1.1. The corrected modulus at 77°F should then be used with Equation 3 to predict the fatigue life of the pavement. The estimated tensile strain at the bottom of the AC layer should be found by layered elastic analysis (i.e., WinJULEA) using the AC modulus (E) or estimates can be found in Tables 1 and 2.

$$Log_{10}(\varepsilon_{ra}) = 7.94 - \left[\frac{LN\left(S_A x10^6\right)}{2.61}\right]^2 + \frac{E}{438{,}000\text{ psi}} \qquad \text{(EQ. 3)}$$

ε_{ra} = allowable strain repetitions for aged, field AC

S_A = tensile strain of AC, in./in.

E = AC design modulus, psi

4.2.2. ITS Test. If ITS tests are used in a pavement evaluation, to indicate AC integrity in terms of peak ITS stress, use Equation 4 to predict the fatigue life of aged, field AC at 77°F. The ITS tests are performed in a laboratory at room temperature (around 77°F). An estimate for AC modulus is needed to calculate the AC tensile strain by layered elastic analysis. Methods of estimating AC modulus are presented in this UFC.

$$Log_{10}(\varepsilon_{ra}) = 8.36 - \left[\frac{LN\left(S_A x10^6\right)}{2.62}\right]^2 + \frac{ITS\ Peak\ Stress,\ psi}{264\text{ psi}}$$

NOTE: If the strain at the bottom of the AC is unknown, then refer to the Tables 1 and 2 below for an estimate. The tables give typical values of S_A for some DOD and civil aircraft according to AC pavement thickness, AC modulus, base and subbase thickness, and base and subgrade modulus values Table 1 is based on an AC modulus of 350 ksi and Table 2 is for an AC modulus of 700 ksi.

NOTE: The strain at the bottom of the AC can also be computed using a design analysis within WinJULEA, which is available under the Help/Utilities menu of the PCASE (Pavement-Transportation Computer Assisted Structural Engineering) desktop computer program (available at www.pcase.com).

EXAMPLE: Determine the number of passes (stress repetitions) a C-17 can make on a 3-in. thick, brittle asphalt surface. Modulus was measured with the PSPA and determined to be 700 ksi when the asphalt surface temperature was 92^oF. The pavement structure is as follows:

3-in. asphalt concrete surface; E = 700,000 psi

11-in. base course; E = 75,000 psi

Subgrade; E = 30,000 psi

Solution:

Convert PSPA-measured modulus to AC Design Modulus, E_{77^oF}.

NOTE: The conversion from PSPA-measured modulus to AC Design Modulus accounts for the differences in both temperature and impulse frequency. In this example, the conversion to a lower frequency overrides the decrease in temperature, resulting in an overall decrease in the estimated asphalt modulus.

$$E_{77^\circ F} = \frac{E_{PSPA}}{[(-0.0109*((T-32)*\frac{5}{9})+1.2627)*3.2]}$$

$$= \frac{700}{[(-0.0109*((92-32)*\frac{5}{9})+1.2627)*3.2]}$$

$$= 243.226\,\text{ksi}$$

$$E_{77^\circ F} = 243{,}226\,\text{psi}$$

Estimate strain, S_A, at the bottom of the asphalt surface layer using a layered-elastic analysis program (i.e., WinJULEA). See Table 3, *Aircraft Characteristics*, for the single wheel load, tire pressure, and contact area for various aircraft.

To use WinJULEA for calculating horizontal tensile strain at the bottom of asphalt layers, under single-wheel loads:
-Open PCASE.
-Click on Help/Utilities.
-Click on WinJULEA.
-Type in the thickness, modulus, Poisson's Ratio (PR), and slip for each layer (slip = "0" for the common assumption of full bond).
-Type in the x-coordinate, y-coordinate, load, and contact area.
-Type in the Evaluation Points (x-coord. = 0; y-coord. = 0).
-Type in the Calculation Depth(s).
-Click Calculate.
-Strain_x and Strain_y are the horizontal strains, S_A, at the bottom of the asphalt surface layer.

For this example:

Layer 1: Thickness = 3; E-Modulus = 243,226; PR = 0.35; Slip = 0

Layer 2: Thickness = 11; E-Modulus = 75,000; PR = 0.35; Slip = 0

Layer 3: Thickness = 0; E-Modulus = 30,000; PR = 0.40; Slip = 0

x-coord. = 0; y-coord. = 0; Load = 44,850; Contact Area = 316

Depth 1 = 3

$$S_A = 1.36 \text{ x } 10^{-4} \ {}^{\text{in}}\!/_{\text{in.}} = 136 \ \mu\varepsilon$$

Determine allowable strain repetitions, ε_{ra}, for in situ asphalt at approximately 77°F (i.e., remaining pavement life).
Note: The repetitions (aircraft passes) are assumed equivalent to coverages, so the pass to coverage ratio (P/C ratio) is equal to one. A P/C ratio = 1 ignores the effects of aircraft wander and therefore provides for a conservative prediction of remaining life.

$$\text{Log}_{10}(\varepsilon_{\text{ra}}) = 7.94 - \left[\frac{\text{LN}(\text{S}_\text{A}\text{x}10^6)}{2.61}\right]^2 + \frac{\text{E}_{77^\text{o}\text{F}}}{438{,}000}$$

$$= 7.94 - \left[\frac{LN(136)}{2.61}\right]^2 + \frac{243{,}226}{438{,}000}$$

$$= 4.9525$$

$$\varepsilon_{ra} = 10^{4.9525}$$

$$\boxed{\varepsilon_{ra} = 89{,}600 \text{ aircraft passes}}$$

NOTE: Compare the above aged asphalt passes to the passes computed by PCASE (E=700,000), using strain at the bottom of the AC only and then using strain in the top of the subgrade and report the lower. In this case, PCASE computes passes of 283,245 passes and 22,238 passes respectively for an A traffic area.

These passes are converted to a Pass/Coverage of 1, used by the aged asphalt procedure, by dividing by the P/C ratio of 1.3861. Since the PCASE procedure using subgrade strain results in the lower number of passes, 22,238/1.3861=16, 044 passes are reported. PCASE is continuously updated and improved. Subsequent versions may result in different pass levels.

NOTE: For aged asphalt, the estimate for remaining life assumes a 'representative' pavement temperature of 77°F. Changes in pavement temperature, as would normally occur, are ignored. This decision was based on recognizing the realistic precision of fatigue life prediction models. Given all the uncertainties involved with estimating in situ asphalt moduli and load-induced strains, as well as the inherent variability of laboratory fatigue testing, the consideration of pavement temperature effects in fatigue life predictions was deemed unwarranted.

Aircraft	AC Thickness (in.)	Base Thickness (in.)	Base Modulus (ksi)	Subgrade Modulus (ksi)	Maximum S_A at Bottom of AC (in./in.)
C-17	3	19	50	15	2.95E-04
	3	23	50	15	2.94E-04
	3	12	75	30	1.74E-04
	3	14.5	75	30	1.70E-04
	4	17	50	15	3.98E-04
	4	21.5	50	15	3.90E-04
	5	15.5	50	15	4.36E-04
	5	20	50	15	4.22E-04
	10.5	20	75	30	2.43E-04
C-130	3	13.5	50	15	1.83E-04
	3	18.5	50	15	1.73E-04
	3	7.5	75	30	1.21E-04
	3	10	75	30	1.03E-04
	4	15.5	50	15	2.66E-04
	4	17	50	15	2.62E-04
	4	18.5	50	15	2.59E-04
	5	10	50	15	3.41E-04
	5	15	50	15	3.05E-04
	5	15.5	50	7.5	3.09E-04
	5	20	75	30	1.93E-04
F-15	3	16.5	50	15	8.92E-04
	3	18	50	15	8.79E-04
	5	20	50	15	7.73E-04
	7.5	20	50	15	5.58E-04
	9	20	75	30	3.66E-04
Analysis based on an AC Poisson's ratio of 0.35, base Poisson's ratio of 0.35, and subgrade Poisson's ratio of 0.40.					

Table 1. Pavement Designs and Calculated AC Strains for a 350 ksi AC Design Modulus.

Aircraft	AC Thickness (in.)	Base Thickness (in.)	Base Modulus (ksi)	Subgrade Modulus (ksi)	Maximum S_A at Bottom of AC (in./in.)
C-17	3	18	50	15	3.40E-04
	3	22	50	15	3.33E-04
	3	11	75	30	2.35E-04
	3	12.5	75	30	2.29E-04
	4	16	50	15	3.85E-04
	4	19	50	15	3.75E-04
	4	9	75	30	2.98E-04
	5	13.5	50	15	3.90E-04
	5	16.5	50	15	3.75E-04
	12.5	20	75	30	1.52E-04
C-130	3	13	50	15	2.39E-04
	3	19.5	50	15	2.18E-04
	3	6.5	75	30	1.86E-04
	3	9	75	30	1.61E-04
	4	8	50	15	3.35E-04
	4	14	50	15	2.79E-04
	4	15.5	50	15	2.73E-04
	4	6.5	75	30	2.30E-04
	5	7.5	50	15	3.38E-04
	5	13	50	15	2.89E-04
	5	13.5	50	7.5	3.07E-04
	5	20	75	30	1.89E-04
F-15	3	15.5	50	15	7.64E-04
	5.5	20	50	15	5.28E-04
	7.5	20	50	15	3.86E-04
	9.5	20	75	30	2.41E-04
Analysis based on an AC Poisson's ratio of 0.35, base Poisson's ratio of 0.35, and subgrade Poisson's ratio of 0.40.					

Table 2. Pavement Designs and Calculated AC Strains for a 700 ksi AC Design Modulus

Aircraft	Load (lb)	Tire Pressure (psi)	Contact (sq in.)
A-10	22,500	185	124
A320-200	17,652	186	95
A330-200	60,132	206	292
A340-200	60,484	206	292
AN-124	41,655	149	280
B-1	55,475	220	252
B-52	62,400	234	267
B-707	35,490	163	218
B-727	49,233	205	237
B-737	35,175	202	174
B-747	50,808	208	245
B-767	47,035	200	235
B-777-200	49,597	215	231
B-777-300	52,298	215	243
C-5	32,785	115	285
C-9	24,300	147	165
C-12	3,735	95	39
C-17	44,468	142	314
C-20	15,682	175	90
C-23	11,070	91.5	121
C-27	14,815	80	183
C-40	39,325	204	193
C-41	7,936	56	142
C-130	39,375	98	400
E-4	45,885	187	245
F-15C/D	29,580	355	83
F-15E	35,235	305	115
F-16C/D	16,875	312	54
F-22	35,204	360	98
F-18	27,100	200	135
KC-10	54,575	165	331
KC-135	37,694	155	218
P-3	34,438	190	179

Table 3. Aircraft Characteristics

APPENDIX F

STRUCTURAL EVALUATION PROCEDURE FOR STABILIZED SOIL SURFACED AIRFIELDS

F-1 **PURPOSE**. This Appendix outlines an approach for determining the remaining operational capability of stabilized soil surfaced airfields. A proven technique for modeling the complex mechanical behavior of stabilized soils does not currently exist. The successful development of a stabilized soil performance model will require additional research, but a promising approach has been established. This guidance presents a method for evaluating stabilized soil surfaced airfields in-situ using a linear elastic modeling approach commonly used for rigid and flexible pavements. Portland cement stabilized soil airfields were evaluated to develop this approach and the applicability to other stabilization methods is not yet known. In this method, portable, light weight strength measuring devices are used to evaluate the surface and subgrade, providing a rapid and easily deployable method for assessing the current condition of stabilized soil surfaced airfields.

F-2 **PREFACE**. The U.S. Army Corps of Engineers' Engineer Research and Development Center (ERDC) developed a method for assessing the remaining operations capability of stabilized soil surfaced airfields. Stabilized soil airfields are commonly used as alternative launch and recovery surfaces (ALRS) and also as contingency training facilities. Stabilized soil surfacing is a cost effective alternative to both Portland cement concrete and asphalt concrete surfaces and should be more durable than plain aggregate surfaced airfields. The mechanical behavior of stabilized materials is non-uniform and the performance is stress and time dependent. Fresh stabilized materials perform similarly to weak rigid pavement layers. However, after cracking occurs, the performance characteristics more closely resemble firm flexible pavement layers. Determination of the failure limits of stabilized materials is equally complex. The stabilized materials maintain a considerable amount of the original strength after cracking. Failure of the surface due to foreign object damage (FOD) potential, caused by the delamination of thin surface layers, occurs much earlier than structural failure characterized by severe cracking and rutting. Therefore, this guidance assesses the stabilized surface layer using a combination of rigid and flexible pavement evaluation approaches.

F-3 **BACKGROUND**.

3.1. Soil Stabilization. The stabilization of soils is accomplished by blending natural soils with supplementary materials in order to improve the engineering properties of the natural soils. Commonly used additive materials include Portland cement, lime, fly ash, asphalt cement, polymers, and fibers. The long term performance of stabilized soils is influenced by the characteristics of the parent soil, type and quantity of stabilization additive, construction practices, frequency and magnitude of loading, and environment of placement.

3.2. Multilayer Elastic Analysis. Multilayer linear elastic analysis is an analytical method of calculating the mechanistic responses (stress, strain, and deflection) of a pavement as the result of the application of an external load. Burmister's solutions are used to determine the stresses and strains in the pavement system. The magnitudes of the responses are used to determine the occurrence and severity of distresses developed in the pavement using an empirical approach. A number of assumptions are made in the modeling of the pavement system in order to conduct multilayer elastic analysis, including: the material properties of each layer are isotropic and homogeneous; the layers are characterized by elastic modulus (E) and Poisson's ratio (ν); each layer has a finite thickness, with the exception of the subgrade which is assumed to be infinite; each layer extends infinitely in the horizontal direction; and the loading is static and applied uniformly over a circular area. The mechanical responses are determined for the critical locations within the pavement system and the controlling location and responses are identified. Using established failure conditions, the maximum allowable aircraft coverages and loading are determined in the analysis.

3.3. Pavement - Transportation Computer Assisted Structural Engineering (PCASE). PCASE is a pavement design and evaluation computer application. The software was developed to provide engineers with a tool capable of handling all of the processes of pavement design and evaluation in a single interface. The evaluation protocol used in the program is based upon the standards set forth in UFC 3-260-02 and this UFC for airfield pavement design and evaluation, respectively. The PCASE program contains the linear elastic modeling subroutine WESLEA. WESLEA is a 5-layer linear elastic model which is used to conduct the mechanistic analysis. The modes of failure and indicating responses are different for rigid and flexible pavements when analyzed using multilayer elastic methods. It should be noted that PCASE is continuously updated and improved, therefore, analysis results using subsequent versions may vary from the current version.

3.3.1.<u>Rigid Pavements</u>. A uniform circular vertical load is applied to the surface of a rigid pavement with known flexural strength (R) and thickness (T). A rigid pavement slab responds to perpendicular loading by curling, that is, the top of the slab goes into compression and the bottom of the slab goes into tension. Rigid pavements are strong in compression, but relatively weak in tension. Therefore, failure occurs in rigid pavements at the bottom of the slab when the load induced tensile stress is in excess of the flexural strength of the stabilized material. PCASE calculates the tensile stress at the bottom of the rigid pavement layer due to a defined loading using the WESLEA subroutine. The number of allowable coverages is then determined when the tensile stress is not in excess of the flexural strength for a given surface condition index (SCI). Failure in rigid pavements is defined by cracking. Cracking occurs first at the bottom of the slab and then propagates upward toward the surface.

3.3.2. Flexible Pavements. When modeling flexible pavements, a uniform perpendicular circular load is applied to the pavement surface. All of the layers in the flexible pavement system are characterized by “E”, “ν”, and “T”, except for the subgrade, which is modeled with an infinite thickness. The flexible pavement layers respond to loading by undergoing shear deformation. The WESLEA subroutine calculates the load induced strain within the critical locations of the pavement layers. Failure in flexible pavements is defined by the permanent deformation of the pavement layers known as rutting. Rutting occurs when the load induced deformation in a pavement layer is in excess of the recoverable deformation of the material. PCASE uses the critical strain determined using WESLEA to determine the maximum allowable coverages at a given loading when the load induced strain is not in excess of the recoverable strain of the material. Rutting can occur in any of the pavement layers, but is generally primarily found in the subgrade layer.

3.4. Field Determination of Pavement Properties. The determination of the material properties of stabilized soil materials is possible by sampling and returning the material to the laboratory for testing. The transportation and testing of the samples is time consuming and cost ineffective. Therefore, a method of determining the properties of the material in-situ is preferred.

3.4.1. Material Properties.

3.4.1.1. The ability of a material to resist deflection due to an imparted force defines the elastic modulus (E). When considering pavement layers, the stiffness of the material determines the magnitudes of displacement and strain experienced as a result of being loaded. Stiffness is used interchangeably with the terms elastic modulus, Young’s modulus, and resilient modulus (M_R). The elastic modulus is used to describe the stiffness of the pavement layer, although it is more accurately a description of the resistance to deflection of the constituent materials within the layer.

3.4.1.2. The ratio of horizontal strain to axial strain in a material as it is loaded is known as the Poisson’s ratio (**ν**). The Poisson’s ratio defines the magnitude of deformation normal to the load. The deformation occurs as a result of inherent resistance to change in volume. The Poisson’s ratio is a material property and commonly ranges from 0.0 to 0.5, although some materials, such as foams, possess negative “**ν**” values.

3.4.1.3. The ability of a solid to resist fracture in bending is the flexural strength of the material and is also known as the modulus of rupture (R). With respect to pavements, and in particular rigid pavements, the flexural strength determines the amount of bending stress the slab can endure before cracking develops.

3.4.1.4. The physical depth or the distance from the surface to the bottom of a pavement layer is defined as the thickness (T) of that layer. The thickness of the stabilized pavement layer bears great influence on the exhibited performance characteristics.

3.4.2. In-Situ Testing Devices.

3.4.2.1. The properties of the stabilized surface layer may be determined using the Portable Seismic Property Analyzer (PSPA). The PSPA was developed by Geomedia Research and Development, as a portable device with the ability to nondestructively evaluate concrete, asphalt, and prepared subgrade materials. The device consists of an electronics box, extension rods, a wave generation source, and two receivers. The system is controlled by a laptop computer which also records the data. The PSPA (Figure 1) generates ultrasonic surface waves (USW), the speeds of which are measured by the two receivers. The velocity of the USW, along with the Poisson's ratio and mass density of the tested material, are used to calculate the Young's modulus.

Figure 1. Portable Seismic Property Analyzer (PSPA)

3.4.2.2. The strength of the subgrade is determined using the Dynamic Cone Penetrometer (DCP). The DCP is used extensively in both military and civilian applications. The DCP is designed to be a portable device capable of determining the in-situ strength of soils. The DCP is intended for use on horizontal construction applications, including fine and coarse grained soils, granular construction materials, and weak stabilized or modified materials. Materials underlying a bound surface layer can be tested by first drilling or coring an access hole (Figure 2). The DCP is composed of a handle, two rods, either a 10.1 lb or 17.6 lb hammer, an anvil, and a conical tip (Figure 3). The data output of the DCP is the DCP index. The DCP index is a measure of the penetration rate, or the depth of penetration, of the conical tip with each blow of the hammer. A number of published correlations exist relating the DCP index to California bearing ratio (CBR) and resilient modulus (M_R).

Figure 2. Drilling Through Overlying Stabilized Surface

Figure 3. Using a DCP to Determine Subgrade Soil Strength

F-4 **EVALUATION PROCEDURE**.

4.1. Assessing a Test Site. The evaluation of a stabilized soil airfield should include the critical locations such as the runway ends and taxiways, in addition to any parking aprons involved in the proposed operations. A minimum of 10 test locations should be established and focused on areas within the expected wheel path of the evaluation aircraft. An example of typical test location layout for a stabilized airfield runway is shown in Figure 4. The PSPA should be used to evaluate the stabilized material surface layer and the DCP to determine the level of subgrade support. Several factors should be noted and taken into consideration when evaluating the airfield. Of particular importance are the surface condition (current distress levels) and environment (moisture conditions). The influence of the conditions at the time of testing should be considered when reviewing the results of the analysis.

4.2. Measurement of Properties.

4.2.1. The PSPA should be used in accordance with the procedures outlined in the guidance developed by the ERDC and available in ERDC/GSL SR-06-9. Three tests should be run at each test location and used to determine the average PSPA modulus (E_{PSPA}) of that location. Testing with the PSPA should not be conducted on segments of the surface layer that are heavily cracked. If the width of cracks in the surface is in excess of 0.25 inch, the "E" value determined for the subgrade should also be used for the evaluation of the stabilized surface. When assessment using the PSPA is possible, Equation 1 should be used to modify "E_{PSPA}" to yield the elastic modulus of the stabilized material. Equation 1 was developed by conducting a regression analysis using PSPA measurements in the field and backcalculated elastic modulus (E) values determined using a Falling or Heavy Weight Deflectometer (FWD or HWD) in accordance with ASTM D 4694. The coefficient of determination for the regression relationship is 0.72. The correlation shown in Equation 2 should be used to establish the modulus of rupture of the surface layer. Equation 2 was developed by conducting a regression analysis using PSPA measurements in the field and samples returned to ERDC for laboratory testing. The testing was conducted in accordance with ASTM D 1635 and the coefficient of determination of the relationship is 0.77. It should be noted that "E_{PSPA}" is reported in ksi or 1.0 x 10^3 psi. The elastic modulus reported using the PSPA should be converted to psi before being used to calculate Young's modulus.

$$\boxed{E\left(psi\right)=353{,}753\times\ln\left(E_{PSPA}\right)-4{,}237{,}000 \qquad (1)}$$

$$\boxed{R\left(psi\right)=0.14\times10^{-3}\left(E_{PSPA}\right)+21.00 \qquad (2)}$$

4.2.2. A hammer drill or other boring device will be needed to penetrate the stabilized surface to access the subgrade material. Upon complete penetration of the surface layer, a measurement of the thickness (T) of the surface layer should be made. Care should be taken to accurately measure the thickness to +/- 0.25 inch. The DCP should then be run in accordance with the standard procedures outlined in ASTM D 6951 to determine the DCP index of the subgrade. The established correlation provided in Equation 3 will be used to determine the in-situ CBR of all subgrade soils except CL (low plasticity clay) soils with a CBR less than 10 and CH (high plasticity clay) soils. The in-situ CBR of the exception soils should be determined using Equations 4 and 5. The relationship presented in Equation 6 should be used to establish the elastic modulus (E) of the subgrade based upon the calculated CBR.

$$CBR(\%) = 292 \Big/ (DCPIndex)^{1.12} \tag{3}$$

$$CL\ Soils\ CBR < 10 : CBR(\%) = 1 \Big/ (0.017019 \times DCPIndex)^{2} \tag{4}$$

$$CH\ \ Soils : CBR(\%) = 1 \Big/ (0.002871 \times DCPIndex) \tag{5}$$

$$E(psi) = 1500 \times CBR \tag{6}$$

4.2.3. Poisson's ratio (ν) is a material property that cannot be measured in the field and, therefore, a common value for Portland cement stabilized soils, 0.20, should be used for evaluation.

4.3. Determination of Operational Limits. The evaluation module of the PCASE 2.08 software package may be used to determine the operational capacity of the stabilized soil surfaced airfield using the material characterization data collected in-situ. Due to the complex performance characteristics of stabilized materials, a combination of rigid and flexible analyses should be run to evaluate the stabilized surface. Users should note that the airfield and test locations must be established using the inventory module and the projected traffic input using the traffic module of the software before any analysis can be conducted. More information is available within the Help utility provided within the program.

4.3.1. New stabilized soil pavement layers and those exhibiting no structural cracking should be evaluated using both the rigid and flexible approaches. The occurrence of shrinkage cracking is expected in stabilized soil layers and should not be interpreted as structural cracking. Cracked pavements should be evaluated with the flexible approach only. For the evaluation, the stabilized surface will be modeled as either weak Portland cement concrete or stiff asphalt concrete in the rigid and flexible analyses, respectively.

4.3.2. Rigid Analysis. For the rigid analysis, the "R" and "E" for the surface layer will have to be entered, in addition to the "E" for the subgrade layer. Common values of Poisson's ratio should be used for the surface and subgrade if the true values are unknown -- 0.20 and 0.40, respectively. Due to the lack of joints capable of transferring loads in a stabilized surface, the load transfer should be set to 0.0 percent. For the Army and Navy, the structural condition index (SCI) should be set to 50, which is related to the appearance of 50% shattered slabs in a rigid pavement. A shattered slab is a rigid pavement unit that is broken into four or more pieces by intersecting cracks. This is defined as the failure point of rigid pavements. The Air Force uses a SCI of 0 to represent failure.It should be noted that a level of shrinkage cracking is expected in stabilized materials and should not be interpreted as failure of the surface. The analysis should be run and the allowable loading and passes to failure determined for the surface.

Example: An uncracked Portland cement stabilized surface on a contingency airfield is comprised of 9.0 inches of stabilized surface over compacted silty-sand subgrade at the runway ends. Evaluation yields E_{PSPA} = 1,350,000 psi for the surface and a DCP Index = 18 mm/blow for the subgrade. How many passes of a C-17 Globemaster, at a weight of 486,000 lbs, should be allowed on the pavement?

From the equations:

$$\begin{aligned} E\left(psi\right) &= 353{,}753 \times \ln\left(E_{PSPA}\right) - 4{,}237{,}000 \\ &= 353{,}753 \times \ln\left(1{,}350{,}000\right) - 4{,}237{,}000 = 756{,}441 \ psi \rightarrow 756{,}400 \, psi \end{aligned}$$

$$R\left(psi\right) = 0.14 \times 10^{-3}\left(E_{PSPA}\right) + 21.00 = 0.14 \times 10^{-3}\left(1{,}350{,}000\right) + 21.00 = 210 \ psi$$

$$CBR\left(\%\right) = {}^{292}\!\big/_{\left(DCPIndex\right)^{1.12}} = {}^{292}\!\big/_{\left(18.0\right)^{1.12}} = 11.5\%$$

$$E\left(psi\right) = 1500 \times CBR = 1500\left(11.5\right) = 17{,}250 \, psi$$

- Open the PCASE desktop computer program
- Select the Traffic tab, select Create Pattern, and name the pattern “C-17”, click “Ok”
- Select the Add Vehicle tab, click the box for the C-17 on the dropdown menu, click Add, set the Traffic Area Weight (lbs) to “486,000” for “Areas A/B”, then hit Apply
- Select the Evaluation Module tab, in the Run Properties tab click the Create/Retrieve Section button, add a Network, Branch, Section, and inspection date, and click Assign
- Set the Evaluation Type to “Airfield”, set the Traffic Area to “A” for the runway ends, and set the Condition to “Good”
- Set the Analysis Type to “LEEP” and select the Traffic Pattern “C-17” from the dropdown menu
- Select the Layer Manager tab, enter the properties for the stabilized layer: click “edit” under grid, set the surface to “PCC” from the dropdown menu by clicking on the first cell under Layer Type, enter the thickness as “9.0”, set the Analysis E to “Manual”, and enter the Flex Strength as “210”

- Select “Compacted Subgrade” from the dropdown menu for the next layer, enter the thickness as “231.0” or leave blank and it will calculate the thickness based on a depth to bedrock of 240 inches, set the Analysis E to “Manual”, then click Save under the layer grid
- Select the Edit Settings tab, click Backcalculation, click Edit, set Poisson’s ratios (PR) equal to “0.20” and “0.40” for the PCC and Compacted Subgrade respectively, set the Slip to “1.0” for both layers, then click Save
- In the Edit Settings tab, click Analysis, click Edit in the Layer Set Controls, enter the modulus value for each layer (756,400 psi for the PCC and 17,250 psi for the Compacted Subgrade), then click Save
- Set the rigid pavement SCI at failure (on right side of screen) to “50” and Load Transfer equal to “0.0%”
- Return to the Layer Manager tab and click Run Analysis
- Computations indicate that 3 passes of a C-17 at the specified load of 486,000 lbs are allowable

4.3.3. Flexible Analysis. The flexible analysis is run similarly to the rigid analysis. The top layer type should be set to asphalt and the “E” values used for analysis set to manual. The elastic modulus values calculated using the PSPA measurements should be input into the analysis settings and the allowable passes to failure again determined.

Example: Same pavement structure and operations scenario as the previous example.

- Within the Layer Manager tab, enter the layer properties for the flexible analysis: click Edit under grid, set the surface layer to “Asphalt” enter the thickness as “9.0”, and set the Analysis E to “Manual”
- Select “Compacted Subgrade” for next layer, enter the thickness as “231.0” or leave blank and it will calculate the thickness based on a depth to bedrock of 240 inches, set the Analysis E to “Manual”, and click Save
- Select the Edit Settings tab, click Backcalculation, set the Poisson’s ratios (PR) equal to “0.20” and “0.40” for the Asphalt and Compacted Subgrade respectively, set the Slip to “1.0” for both layers, then click Save
- Click Analysis, enter the modulus value for each layer (756,400 psi for the Asphalt, 17,250 psi for the Compacted Subgrade), then click Save
- Return to the Layer Manager tab and click Run Analysis
- Computations indicate that 25,778 passes of a C-17 at the specified load of 486,000 lbs are allowable

4.3.4. For new and uncracked stabilized layers, the rigid analysis will indicate the number of passes and allowable loading before structural cracking occurs. Once the initial cracking has occurred, the material performance transitions from that resembling a weak rigid layer to that of a stiff flexible layer. The flexible analysis will give an estimate of the number of passes and allowable loading before the complete failure of the stabilized soil surface layer. All of the test locations of interest on the airfield should be evaluated using the procedures outlined above. The lowest number of passes, or the maximum loading for a given number of passes, determined for all airfield features, should be used to establish the controlling condition for the airfield. Further guidance on the use of the software can be found in Appendix H of this UFC or the Help utility of the software.

Example: Using the previous rigid and flexible pavement analysis examples, only 3 passes of a C-17 at 486,000 lbs should be allowed to prevent structural cracking, while 25,778 passes may be allowed before complete failure of the stabilized soil surface occurs. After the stabilized layer cracks, it is no longer considered to be a rigid pavement and is analyzed as a flexible pavement.

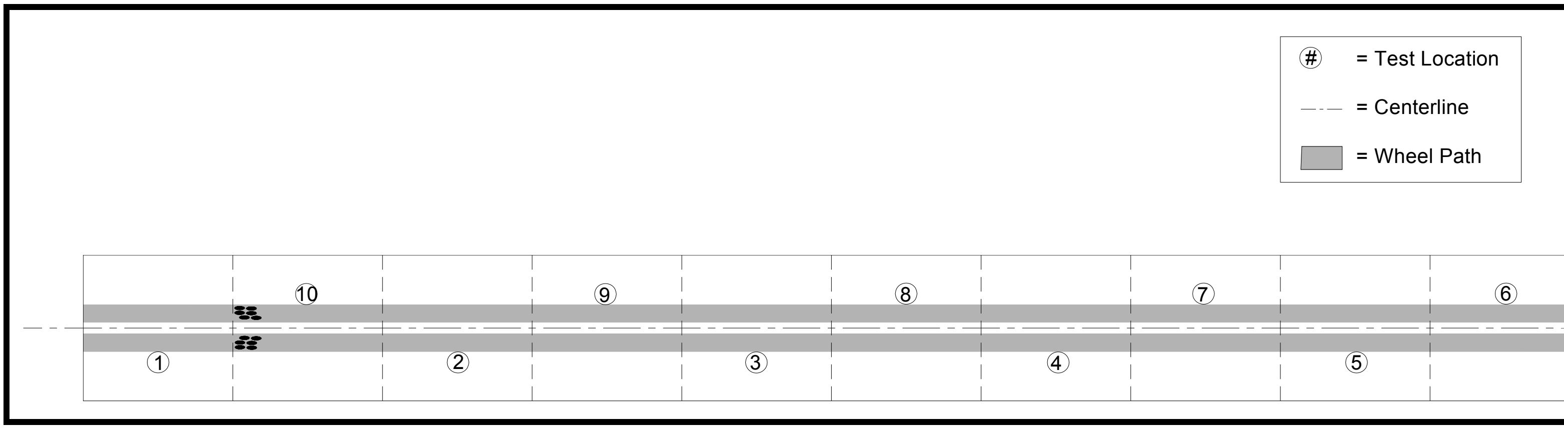
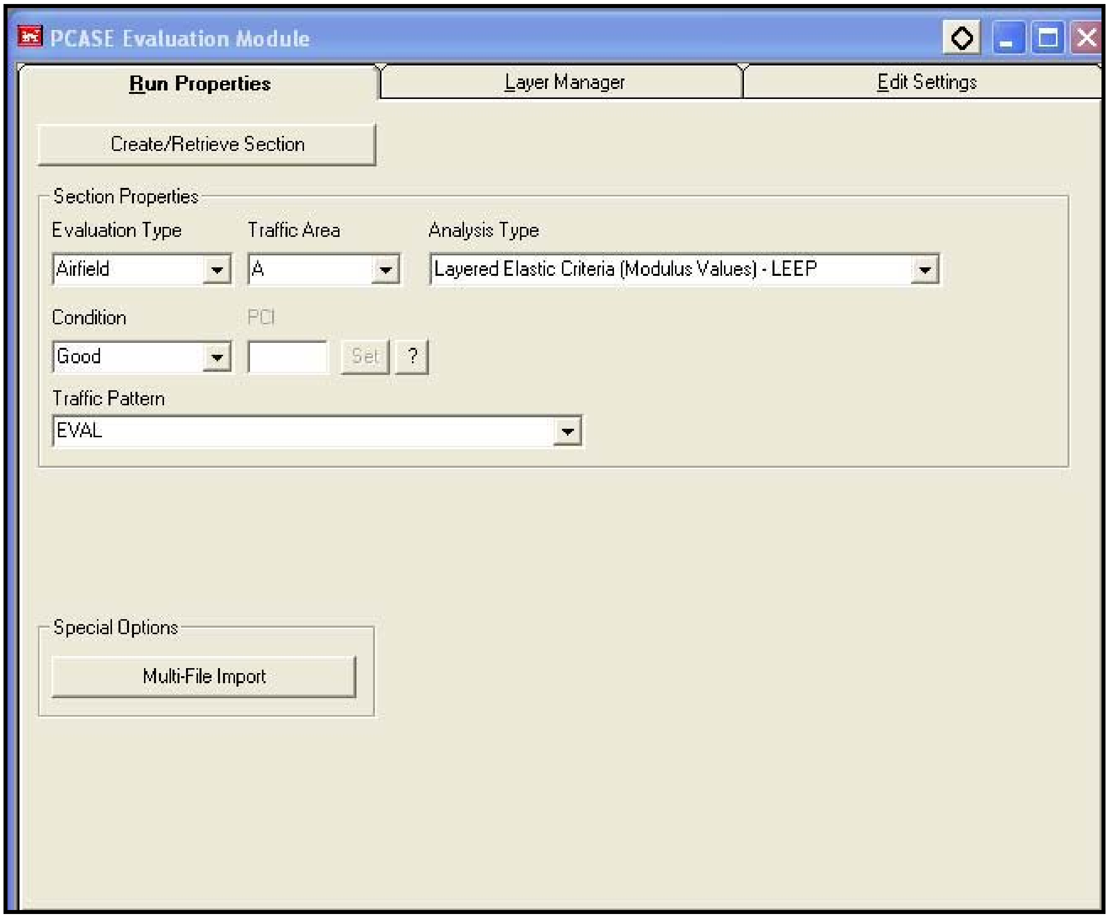


Figure 4. Test Location Layout for Stabilized Runway



Section Properties

Evaluation Type: Airfield

Traffic Area: A

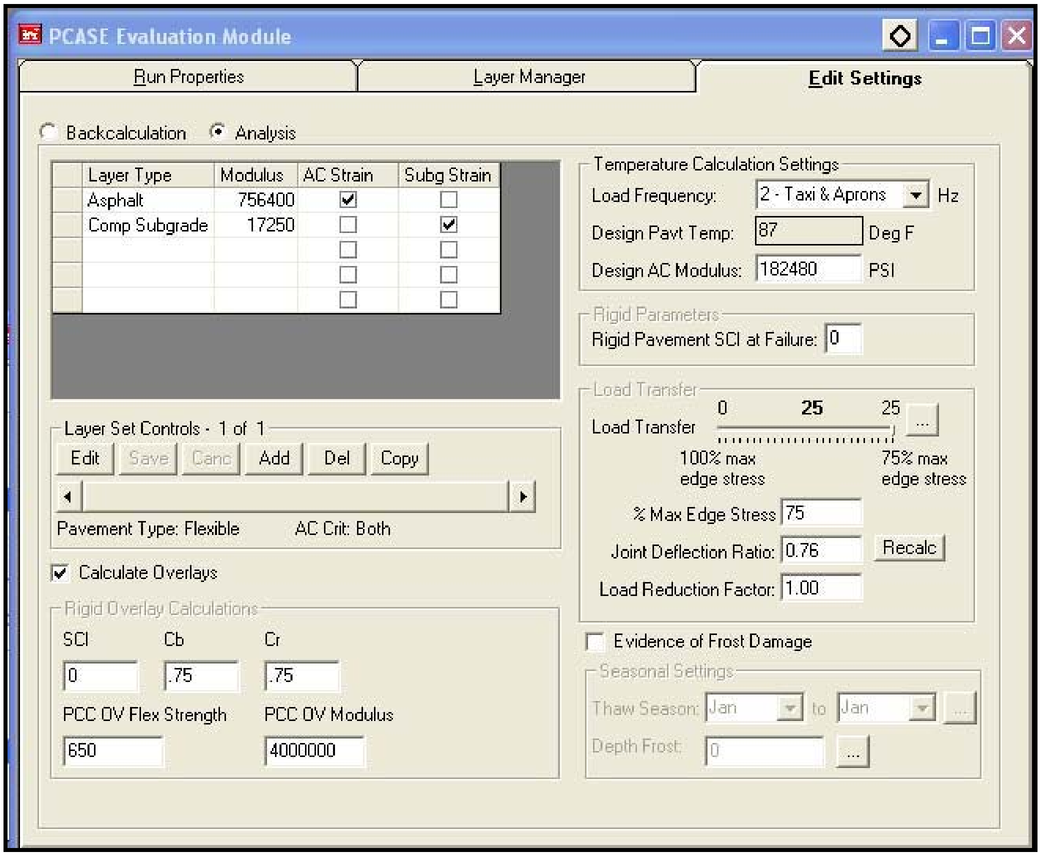
Analysis Type: Layered Elastic Criteria (Modulus Values) - LEEP

Condition: Good

PCI:

Traffic Pattern: EVAL

Figure 5. Evaluation Module Run Properties Tab in PCASE 2.09



○ Backcalculation ● Analysis

Layer Type	Modulus	AC Strain	Subg Strain
Asphalt	756400	☑	☐
Comp Subgrade	17250	☐	☑
		☐	☐
		☐	☐
		☐	☐

Pavement Type: Flexible AC Crit: Both

☑ Calculate Overlays

Rigid Overlay Calculations

SCI	Cb	Cr
0	.75	.75

PCC OV Flex Strength: 650

PCC OV Modulus: 4000000

Temperature Calculation Settings

Load Frequency: 2 - Taxi & Aprons Hz

Design Pavt Temp: 87 Deg F

Design AC Modulus: 182480 PSI

Rigid Parameters

Rigid Pavement SCI at Failure: 0

Load Transfer

Load Transfer 0 **25** 25

100% max edge stress 75% max edge stress

% Max Edge Stress 75

Joint Deflection Ratio: 0.76

Load Reduction Factor: 1.00

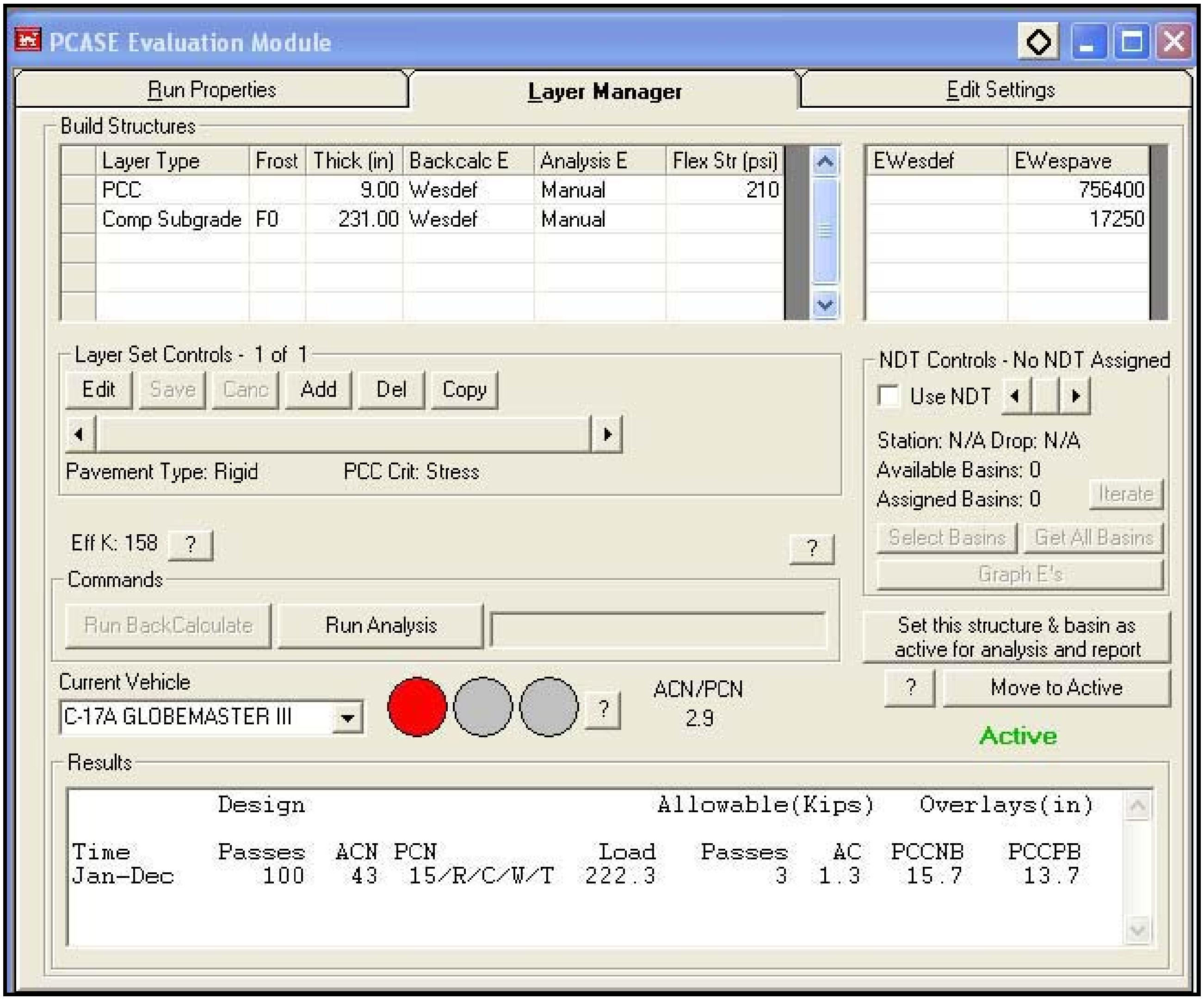
☐ Evidence of Frost Damage

Seasonal Settings

Thaw Season: Jan to Jan

Depth Frost: 0

Figure 6. Evaluation Module Edit Settings Tab in PCASE (flexible analysis) in PCASE
2.09



	Layer Type	Frost	Thick (in)	Backcalc E	Analysis E	Flex Str (psi)
	PCC		9.00	Wesdef	Manual	210
	Comp Subgrade	F0	231.00	Wesdef	Manual	

EWesdef	EWespave
	756400
	17250

Layer Set Controls - 1 of 1

Pavement Type: Rigid

PCC Crit: Stress

Eff K: 158

NDT Controls - No NDT Assigned

☐ Use NDT

Station: N/A Drop: N/A

Available Basins: 0

Assigned Basins: 0

Current Vehicle

C-17A GLOBEMASTER III

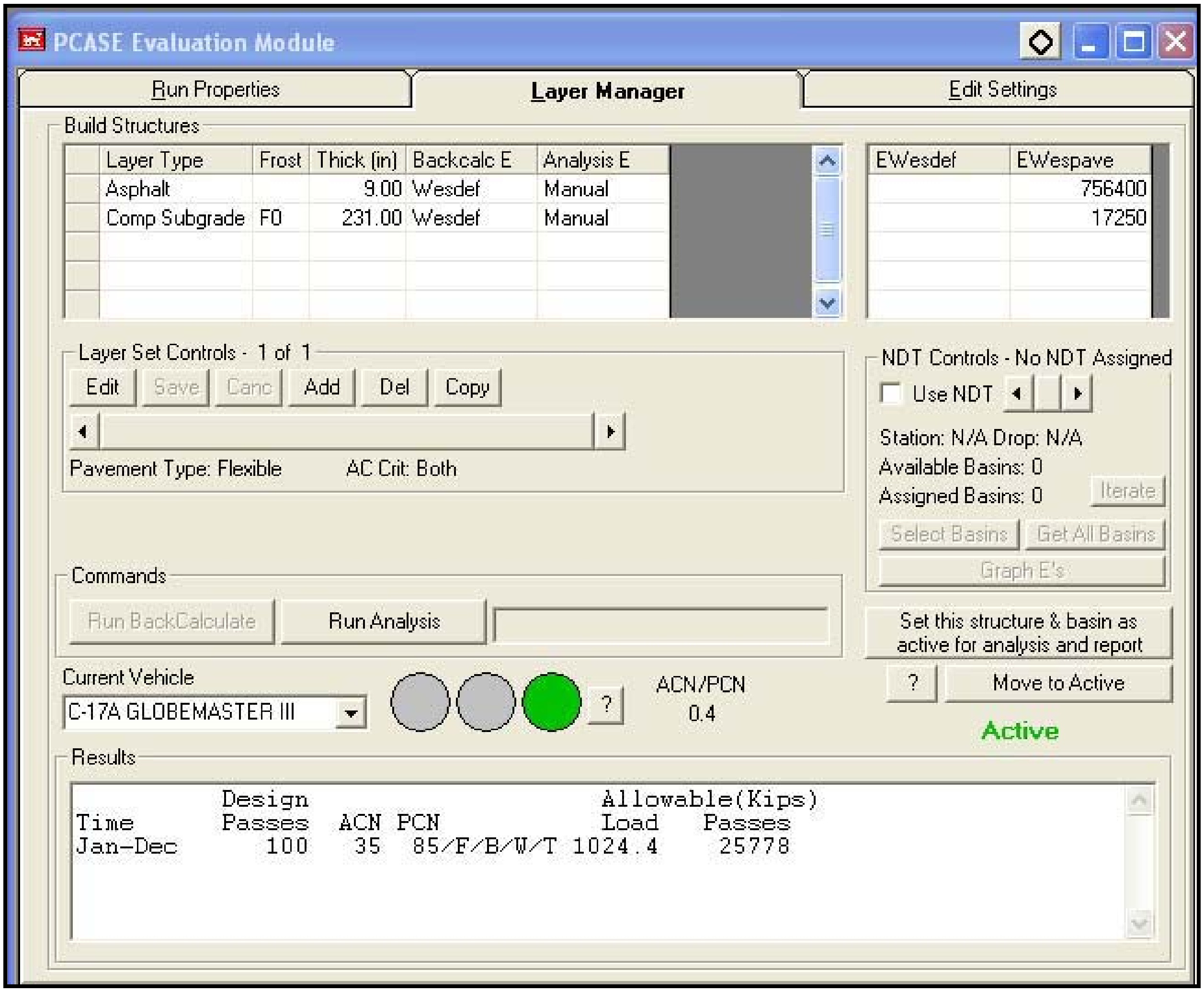
ACN/PCN

2.9

```
          Design                         Allowable(Kips)   Overlays(in)

Time      Passes   ACN  PCN              Load    Passes   AC  PCCNB    PCCPB
Jan-Dec      100    43   15/R/C/W/T     222.3         3  1.3   15.7     13.7
```

Figure 7. Evaluation Module Layer Manager Tab (Rigid Analysis) in PCASE 2.09



Layer Type	Frost	Thick (in)	Backcalc E	Analysis E
Asphalt		9.00	Wesdef	Manual
Comp Subgrade	F0	231.00	Wesdef	Manual

EWesdef	EWespave
	756400
	17250

Pavement Type: Flexible AC Crit: Both

☐ Use NDT

Station: N/A Drop: N/A
Available Basins: 0
Assigned Basins: 0

Current Vehicle
C-17A GLOBEMASTER III

ACN/PCN
0.4

```
         Design                     Allowable(Kips)
Time     Passes   ACN  PCN           Load    Passes
Jan-Dec     100    35   85/F/B/W/T 1024.4     25778
```

Tab (Flexible Analysis) in PCASE 2.09 Figure 8. Evaluation Module Layer Manager

APPENDIX G

INSPECTION AND TESTING OF TRIM PAD ANCHORING SYSTEMS

G-1 **BACKGROUND**. Most Air Force fighter aircraft use aircraft anchor blocks during power checks and routine maintenance procedures. Many existing aircraft anchor blocks were designed to withstand loads associated with F-4 operations, but are being used to support the operation of aircraft with higher thrusts, such as the F-15, F-22, and F-35. Table 1 contains thrust values for specific fighter aircraft. While catastrophic failure of an anchor block has not been reported, the stability of some legacy existing anchor block designs, under increased thrust from newer aircraft, has been questioned. Therefore, inspection and testing of suspect legacy anchor blocks is recommended. It is also recommended that only one engine at a time be tested.

Aircraft	Nominal Thrust lbs
F-16	29,000
F-18	32,000
F-15	25,000 per engine
F-22	35,000 per engine
F-35	40,000

Table 1. Nominal Values of Thrust for Various Aircraft

G-2 **ANCHOR BLOCK DESCRIPTIONS**. The Air Force uses two types of anchor blocks, Omni directional and bi-directional. Original designs were based on an applied load of 60,000 pounds. More recently, design of the bidirectional anchor block was modified for the F-22, to support a thrust of 100,000 pounds. Detailed description of the anchors and test results, including Safety Factors, are contained in AF ETL 00-2, Inspection and Testing of Trim Pad Anchoring Systems and AF ETL 01-10, Design and Construction of High-Capacity Trim Pad Anchoring Systems.

2.1. Omni-directional Anchor. This design comprises a steel rod, threaded at the top to accept a nut, embedded in a concrete block. The steel rod is 5 inches in diameter; the concrete block is 10 feet on each side and 3 feet thick. A steel collar, held in place by three washers and the nut, connects the aircraft to the anchor rod. Because the collar is free to rotate 360 degrees on the anchor rod, this type of anchor is omni-directional, and the aircraft can be connected at any orientation to the anchor block.

2.2. Bi-directional Anchor. This design is bi-directional; i.e., the aircraft can pull only in one of two directions, which are 180 degrees apart (opposite).

a. 60,000 pound thrust anchor. The nominal dimensions of the concrete block are the same as the omnidirectional anchor, 10 feet by 10 feet by 3 feet. The steel portion of the anchor consists of a built-up beam section embedded in the concrete. A 2.5-inch diameter rod that bends 180 degrees at its center forms a loop with two legs that extend approximately 3 feet. A 6-inch wide, 1-inch thick steel plate is welded between the two legs to form the web, and two 4-inch wide, 1-inch thick plates are welded to the outside edge of each leg to form the flanges. Although the concrete block can be either square or octagonal, the anchor itself is still bidirectional due to the orientation of the anchor loop.

b. 100,000 pound thrust anchor. This anchor is similar to the 60,000 pound thrust anchor, except it has been strengthened. The diameter of the steel rod was increased to 3-inches. A high strength steel alloy, with a yield strength of 100,000 psi, was used. Additional steel, including bearing plates, steel bars and larger reinforcement, were used. The concrete block remained the same size, 10-feet x 10-feet x 3-feet.

3. Failure Modes Analysis. Table 2 shows the failure modes that were analyzed during laboratory testing.

Site	**Cause**
Connecting Hardware	Shear Bearing Tensile Yielding

Steel Anchor Components	Shear Bending
Concrete Anchor Block	Bearing
Concrete Slabs	Compression Buckling
Steel-Concrete Interface	Pullout (Shear Failure)
Anchor-Slab Interface	Rotation (Shear Failure)

Table 2. Failure Modes

3.1. Failure of the aircraft anchors can result from material failure in the steel connecting hardware (anchor-to-aircraft), the steel components that transfer the load to the concrete anchor, the concrete anchor itself, or the adjoining concrete slabs. In the connecting hardware, failure could result from tensile yielding, shear, or bearing failure. The steel anchor components could fail in shear or bending, or combined shear and bending. The concrete anchor could fail in bearing as a result of the compressive stress imparted by the steel anchor components. The adjoining slabs could fail in compression or by buckling when loaded along the edge by the anchor block.

3.2. In addition to material failure in the individual components, the entire anchor block could be unstable and fail as a unit, by rotation or horizontal translation. A shear failure at the material interfaces may also occur. At the steel-concrete interface, this failure could result in pullout of the steel anchor component, leaving the concrete anchor block in place. At the anchor block-slab interface, the failure could result in rotation of the entire anchor block unit.

G-4 **INSPECTION AND TESTING**.

4.1. Safety Precautions

4.1.1. All components of this test setup are heavy and cumbersome. Ensure personnel are briefed on proper lifting techniques. Pinching and cuts caused by exposed metal surfaces are also hazards. As a minimum, personnel should use hearing protection, hardhats, safety glasses, shoes, and gloves.

4.1.2. All aircraft and equipment must be removed from the area before testing. Establish a 50-foot radius clear zone. Place warning flags, chains, or cones to establish a minimum radius of 50 feet from any of the components under tension.

4.1.3. All components in the test setup have been rated for at least 100,000 pounds of load. A 100,000 pound load can be applied, however, as a precaution, the system is not typically loaded beyond 70,000 pounds.

4.2. Inspection. Check the steel parts for rust, deformation, cracks, or anything that reduces the cross-sectional area. This could significantly change the factor of safety. Check the concrete for spalling around the anchor bolt and cracks through the slab. Check the dimensions of the slab to ensure it meets design size.

4.3. Testing.

4.3.1. Procedure:

(a) Move equipment storage containers and anchor plate to test area.

(b) Fill out checklist (Attachment 1). Use 1 new checklist per anchor.

(c) Position anchor plate (due to excessive weight, a forklift must be used).

(d) Lay out the slings. Use 2 (minimum) different length slings, so that the load cell is not in the center of the distance between the anchor and anchor plate. The center point is used as the lifting point. Attach the shorter sling to the anchor, so the load cell is located at the closest point to the anchor.

(e) Connect sling to anchor with shackle, as shown in Figure 1.

Figure 1. Connect Sling to Anchor, Using Shackle

(f.) Connect load cell with two shackles as shown in Figure 2. Locate load cell at furthest connection point from center lifting point, but do not connect/attach immediately to anchor or plate.

Figure 2. Load Cell Connected to Slings with 2 Shackles

(g) Connect slings together with shackle. See Figure 3.

Figure 3. Slings Connected by Shackle

(h) Position steel anchor plate (Figure 4 shows details of the anchor plate) using forklift and connect sling to steel anchor plate (Figure 5).

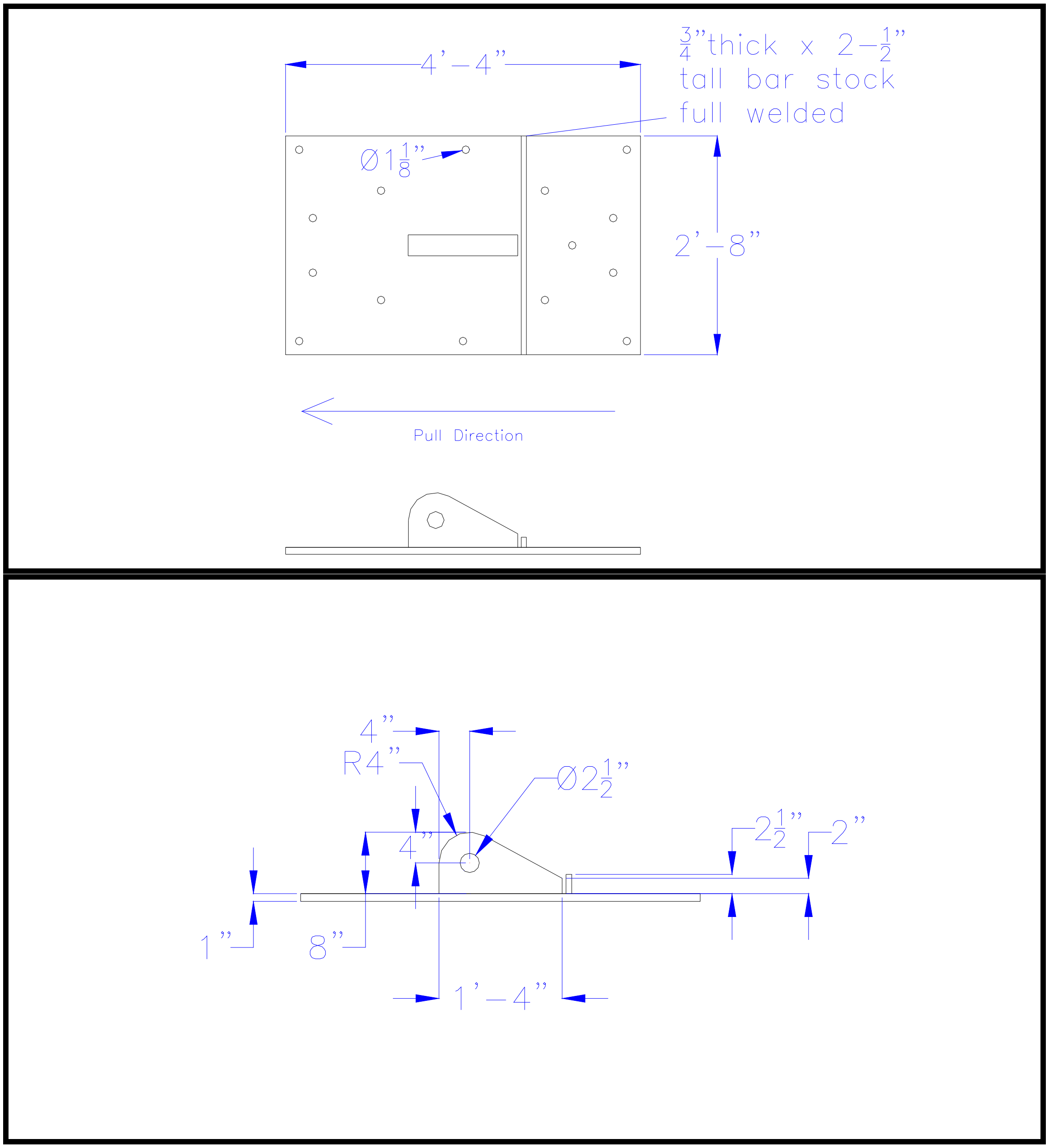


Figure 4. Detail of Anchor Plate

Figure 5. Position Anchor Plate and Connect Sling

(i) Ensure tell-tales & fiber optic lines on slings are visible, Figure 6. Check the fiber optic line with a flashlight and replace, if broken or damaged.

Figure 6. Check Tell-Tales and Fiber Optic Lines.

(j) Straddle forklift over sling assembly; push plate with forklift tines to apply between 600 to 2000 pounds of sling tension (2000 pounds preferred). Push on steel lip, not the front of the plate. Read tension from the remote or directly from the load cell. Ensure load cell has been zeroed prior to applying tension. Ensure fork tips are on a downward angle. (Figure 7)

Figure 7. Positioning Anchor Plate with Forklift.

(k) Assemble anchor bolts, washers, and nuts, Figure 8. Use two nuts, one to be pounded with the sledge and one to stay on the anchor bolt. Ensure top nut is flush with top of bolt to eliminate cross threading. Anchor bolts are 1 inch in diameter and 9 inches long (MKT Sup-R-stud grade 5 steel).

Figure 8. Assemble Anchor Bolts

(l)

Drill hole for anchor bolt, Figure 9. Ensure hammer drill is plumb and drill straight down through the pavement to full potential of bit or reaching base course (whichever occurs first). This ensures the bolts can be hammered below the top of the pavement upon completion of testing. Drill hole in increments, remove bit and debris, then re-start drilling. This will aid in preventing the bit from binding in the pavement.

Figure 9. Drilling Holes for Anchor Bolts

(m) Drive one bolt into the hole and secure nut. Install and secure each anchor bolt before moving to next holes. This ensures holes stay aligned and eliminates plate movement. Secure all 7 bolts in the back of the plate, as shown in Figure 10.

Figure 10. Securing Nuts on Anchor Plate

(n) Remove forklift and check sling tension on load cell. Record tension reading for future reference. Pretensioning the slings is required to limit the sling angle. Drill and put in the rest of the 8 bolts (minimum of 8, for 70K, or 10 for 100K, bolts required if unable to drill all 15). See Figure 11.

Figure 11. Complete Installation of Bolts

(o) Measure from center of anchor to center of anchor plate to find the center point of sling, Figure 12.

Figure 12. Measuring Distance from Anchor to Anchor Plate

(p) Position crane or forklift and put slings on crane (preferred) or forklift. Use protective sheath on slings at contact points with crane or forklift. Figures 13 shows a close-up of the setup; Figure 14 shows crane hooked to slings.

Figure 13. Using Protective Sheath to Protect Slings

Figure 14. View of Crane Applying Load to Slings

(q) Connect digital indicator to magnetic base, using a metal ammo box. Mount digital indicator to anchor at 45 degree angle, Figure 15. Zero out with the gauge rod depressed half way.

Figure 15. Mount Digital Indicator to Anchor at 45 Degree Angle

(r) Using crane or forklift, pull to 10,000 pounds on load cell (Ensure all ground personnel are at a stand-off distance, 1.5 times the total length of utilized slings). Use magnetic Inclinometer mounted on load cell to read angle of pull. Do not exceed 11 degree sling angle, the maximum angle at which aircraft engines are tested. See Figure 16.

Figure 16. Magnetic Inclinometer Mounted to Load Cell

(s) Maintain load at 10,000 lbs. Start stop watch, at 30 seconds take load cell reading and record. Take and record additional readings at 1:00, 1:30, and 2:00 minutes. Read and record inclinometer guage reading.

(t) Raise load to 20,000 pounds and repeat above timed steps and record. Continue in 10,000 pound increments until reaching 100,000 pounds or desired maximum load. At maximum load, repeat timed steps and record. Hold for 10 minutes.

(u) Lower and remove slings from crane or forklift. Ensure tell-tales on slings are still visible. If not visible, try to pull them out, so they are still visible. If they cannot be pulled out so they are visible, the sling is considered to be unserviceable and requires calibration. See Figure 17.

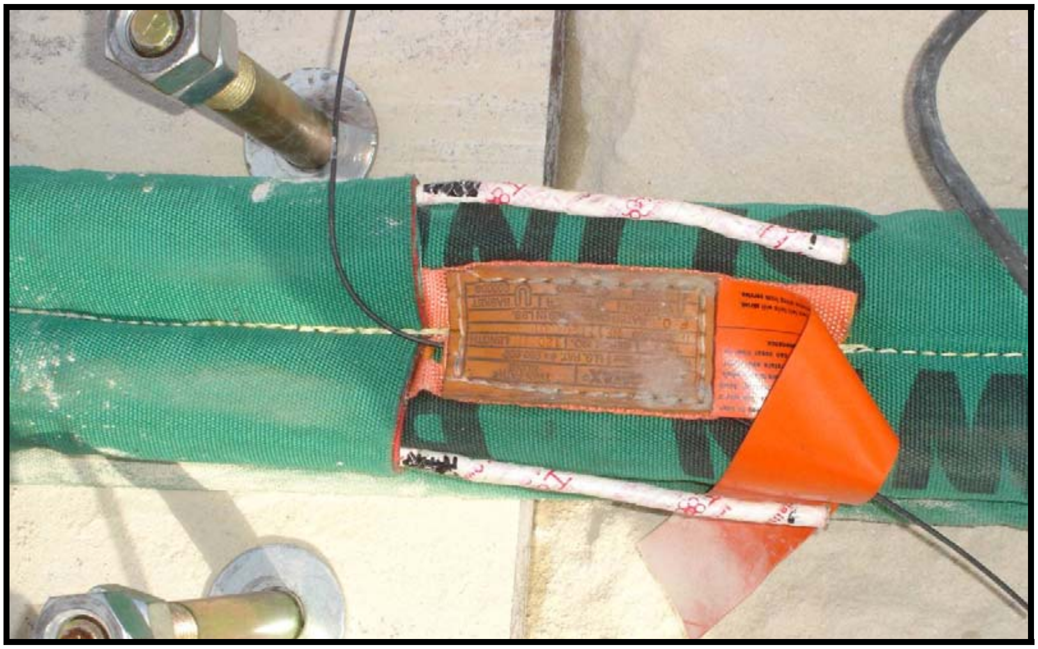


Figure 17. Inspect Slings to Determine if Tell-Tales are Visible

(v) Disconnect load cell immediately upon test completion. The slings are at a relaxed tension at this point, but will regain shape/tension quickly. Remove all slings and shackles, except the one attached to the anchor. Record the final displacement reading on the digital indicator attached to the anchor. The reading should take place not later than 1 minute after test completion. The digital indicator and sling can then be removed. Remove the anchor plate and countersink the anchor bolts to allow holes to be filled with a sealant material compatible with existing pavement. Clean and properly pack all slings, hardware, tools, etc. in storage containers.

G-5 **INSTALLATION OF ANCHOR PLATE IN ASPHALT CONCRETE**. The procedure outlined above assumes that the anchor plate is located in Portland cement concrete (PCC) pavement. There may be cases, although rare, when the anchor plate will have to be located in asphalt pavement. If this happens, use the modified anchor system and procedure described in Engineering Technical Letter 07-2, Anchoring a Fiberglass Mat Assembly in Asphalt Concrete (AC) Pavement.

G-6 **REPORT**. A report should be provided to the appropriate base, command, and AFCEC. The report should outline test procedures, observations, and results.

APPENDIX H

PCASE COMPUTER PROGRAM FOR PAVEMENT EVALUATION

H-1 **BACKGROUND**. The Services use PCASE (Pavements-Transportation Computer Assisted Structural Engineering) to design and evaluate airfield pavements. The program was developed and is continuously updated, expanded, and improved by USACE//ERDC/TSC. USACE/TSC manages the tri-service PCASE program and provides assistance, consulting services, and training. Attending PCASE training is highly encouraged in order to ensure the latest criteria and technology is used to design and evaluate pavements. A Tri-Services Committee provides direction and oversight of the program. TSC contact information:

U.S. Army Corps of Engineers
Transportation Systems Center
1616 Capitol Ave.
Omaha, NE 68102-4901
Telephone: 402-995-2400/2406

H-2 **USING PCASE**. Details on installation and use of PCASE for design and evaluation are contained in *PCASE User Manual.* The manual and latest version of the program is available at:

www.pcase.com or

https://transportation.erdc.dren.mil/pcase/