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UNIFIED FACILITIES CRITERIA (UFC)

AIRFIELD PAVEMENT EVALUATION

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AIRFIELD PAVEMENT EVALUATION

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

NAVAL FACILITIES ENGINEERING COMMAND

AIR FORCE CIVIL ENGINEER SUPPORT AGENCY

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

Change No.	Date	Location

This UFC supersedes TM 5-826-1/AFM 88-24, Chap 1, dated August 1988; TM 5-826-2/AFM 88-24, Chap. 2, dated December 1990; TM 5-826-3/AFM 88-24, Chap. 3, dated December 1990; TM 5-826-4, dated February 1980; TI 826-01/AFMAN 32-1121V1(I)/NAVFAC DM 21.7, dated August 1999; and AFJMAN 32-1036, dated August 1988.

FOREWORD

\1\
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AUTHORIZED BY:

DONALD L. BASHAM, P.E.
Chief, Engineering and Construction
U.S. Army Corps of Engineers

DR. JAMES W WRIGHT, P.E.
Chief Engineer
Naval Facilities Engineering Command

Kathleen I. Ferguson
KATHLEEN I. FERGUSON, P.E.
The Deputy Civil Engineer
DCS/Installations & Logistics
Department of the Air Force

Dr. GET W. MOY, P.E.
Director, Installations Requirements and Management
Office of the Deputy Under Secretary of Defense (Installations and Environment)

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CHAPTER 1

INTRODUCTION

1. PURPOSE. This document presents criteria for evaluation of the load-carrying capacity of pavements used (or to be used) for the support of 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.

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.

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

4. UNITS OF MEASUREMENT. The unit of measurement in this document is the International System of Units (SI). In some cases, inch-pound (IP) measurements may be the governing critical values because of applicable codes, accepted standards, industry practices, or other considerations. Where the IP measurements govern, the IP value may be shown in parenthesis following a comparative SI value or the IP values may be shown without a corresponding SI value. Chapter 4 contains several regression equations which are not available in SI units, and therefore the units for the equations in that chapter remain as English units.

5. TYPES OF PAVEMENT. The types of pavement considered in this manual are as follows:

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 102 millimeters (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 102 millimeters (4 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 steel fibers.

CHAPTER 2

EVALUATION CONCEPTS

1. RELATION 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 be subject to some change. Thus, an evaluation determines the physical properties of a pavement as actually built or in its current condition and establishes therefrom the traffic/load-supporting capacity of the pavement.

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.

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.

d. Load Repetitions.

(1) Repetitions of load or aircraft passes is an aspect of structural capacity. A pavement capable of sustaining a certain aircraft loading on a regular repeating basis for some design life of the facility (commonly 10 years for Navy and Marine Corps Airfields) can sustain repeated application of a larger loading, but for a reduced pavement life (less number of passes).

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

3. EVALUATION PROCEDURE.

a. Steps in the Procedure. Evaluation is the assessment of pavement strength and condition and the computation of the load-carrying capacity. 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 inspection.

(3) Designating pavement facilities and subdividing into pavement features where a facility is a part of an airfield or heliport used by aircraft such as a runway, taxiway, apron, etc., and where features are segments of a pavement with consistent structural thickness and materials were constructed at the same time, subject to approximately the same traffic, and have a uniform condition.

(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.

(c) Using the electronic cone penetrometer and the dynamic cone penetrometer 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 using wave propagation and elastic theory for determination of layer stiffness moduli as a basis for evaluation.

(6) Determination of load-carrying capacity and pavement classification number of the airfield pavements through the application of the evaluation criteria using representative pavement properties. In this regard, load-carrying capacity implies allowable load for selected repetitions or allowable repetitions for selected loadings.

(7) Assignment of an overall field evaluation based on the load-carrying capacity of the weakest pavement facility considered essential to the operation of the airfield.

b. Decision Regarding 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.

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 secured 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 Imagery and Mapping Agency (NIMA). Detailed information will be collected concerning drainage at the airfield, including descriptions of any drainage installations 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 seal coats, surface treatments, and patches should will 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 facilities. Procedures for condition surveys of existing pavements are presented in ASTM D 5340.

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.

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 what effect past aircraft operations have had on pavement life.

(4) Determine 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, operational aircraft have been divided into 14 aircraft groups designated by an Aircraft Group Index for Air Force evaluations as shown in table 2-1. As noted, the table contains a listing of all appropriate operational aircraft that may be expected to use Air Force airfields for various purposes. A controlling aircraft (aircraft having the most severe loading) was selected for each landing assembly configuration where more than one aircraft was involved as indicated in table 2-1. A description of the landing gear assembly on the controlling aircraft is shown in table 2-2 for Air Force aircraft.

Table 2-1
Air Force Aircraft Group Index

1	2	3	4	5	6	7
C-23[1]	F-15[1]	F-111[1]	C-130[1]	C-9[1]	T-43[1]	B-727[1]
C-12	A-7	F-117		C-7	737	C-22
C-21	A-10			DC-9		P-3
C-27	C-20			C-140		
A-37	F-4					
	F-5					
	F-14					
	F-16					
	F-22					
	F-100					
	F-101					
	F-102					
	F-105					
	F-106					
	T-1A					
	T-33					
	T-38					
	T-39					
8	9	10	11	12	13	14
E-3[1]	C-141[1]	C-17[1]	C-5[1]	KC-10[1]	E-4[1]	B-52[1]
707E-8	B-1			DC-10	747	
C-135	B-757			L-1011	VC-25	
KC-135	B-2					
VC-137						
DC-8						
EC-18						
A-300						
B-767						

[1] Controlling aircraft.

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, except for UH-60, CH-47, AH-64, and H-35 should not be included. The traffic report will include the following:

(1) Aircraft Type.

(2) Actual Weights.

(3) Aircraft Passes. Passes are defined as the number of aircraft movements across an imaginary transverse line placed within 152 meters (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

Table 2-2
Characteristics of Controlling Aircraft Landing Gear Assembly

Aircraft Group Index	Controlling Aircraft	Landing Assembly
1	C-23	Single-wheel, tricycle, 0.7-MPa (100-psi) tire pressure
2	F15	Single-wheel, tricycle, 0.06-square-meter (86-square-inch) contact area
3	F-111	Single-wheel, tricycle, 0.155 square-meter (241-square-inch) contact area
4	C-130	Single-tandem-wheel assembly, tricycle, spacing 1.5 meters (60 inches), 0.26-square-meter (400-square-inch) contact area
5	C-9	Twin-wheel assembly, tricycle, spacing 0.66 meter (26 inches), 0.106-square meter (165-square-inch) contact area
6	T-43	Twin-wheel assembly, tricycle, spacing 0.77 meter (30.5 inches), 0.11-square-meter (174-square-inch) contact area
7	B-727	Twin-wheel assembly, tricycle, spacing 0.86 meter (34 inches), 0.153-square-meter (237-square-inch) contact area
8	E-3	Twin-tandem-wheel assembly, tricycle, spacing 0.88- by 1.42-meters (34.5- by 56-inches, 0.14-square-meter (218-square-inch) contact area
9	C-141	Twin-tandem-wheel assembly, tricycle, spacing 0.82- by 1.22-meters (32.5- by 48-inches), 0.134-square-meter (208-square-inch) contact area
10	C-17	Tri-tandem-wheel assembly, tricycle, spacing 1.02- by 1.08- by 2.46-meters (41- by 43- by 97-inches), inside tandem offset 0.29 meters (11.5-inches), 0.206-square-meter (320-square-inch) contact area
11	C-5	Twin-delta-tandem-wheel assembly, tricycle, spacing 0.86- by 1.35- by 1.65-meters (34- by 53- by 65-inches), 0.184-square-meter (285-square-inch) contact area
12	KC-10	Twin-tandem-wheel assembly, tricycle, spacing 1.37- by 1.63 meters (54- by 64-inches), 0.190-square-meter (294-square-inch) contact area
13	E-4	Twin-tandem-wheel assembly, tricycle, spacing 1.12- by 1.47-meters (44- by 58-inches), 0.158-square-meter (245-square-inch) contact area
14	B-52	Twin-twin-wheel assembly, bicycle, spacing 0.94 by 1.57- by 0.94-meter (37-by 62-by 37-inches), 0.172-square-meter (267-square-inch) contact area

runway and the parking apron. At single-runway airfields, the pass level for the runway, taxiway, and apron will be the same.

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. The number of passes for each facility will be determined as discussed above for Army evaluations. Navy aircraft can be gathered into five groups as indicated in Table 2-3. Evaluations 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-14 for single tricycle, and P-3 for dual-tricycle, C-130 for single tandem tricycle, C-141

Table 2-3
Navy Aircraft Groups

Single Tricycle	Dual Tricycle	Single-Tandem Tricycle	Dual-Tandem Tricycle	Twin Delta Tandem
F-14[1]	P-3[1]	C-130[1]	C-141[1]	C-5A[1]
C-23	C-119		KC-135	
F-4E	C-124		DC-8	
F-8E	C-131		DC-10-10	
F-15	UH-46		DC-10-10CF	
F/A-18	DC-9		L-1011	
F-111	CH-53		B-707	
T-1	CH-54		B-757	
T-2C	B-727		B-767	
T-39A	B-737		E-3A	
A-3B	T-43		E-6A	
A-4M	C-7			
A-5	C-9B			
A-6E	C-118A			
A-7K	C-121			
P-2	C-140			
RA-5	C-22			
S-3A				
E-2C				
T-28D				
C-117				
T-34				
T-45				

[1] Representative aircraft.

for dual tandem tricycle, and C-5A for twin delta tandem. Special aircraft not included in these groups can be studied separately (e.g. KC-10 and DC-10-30, or C-17, or small aircraft for the case of outlying landing fields).

6. EVALUATION TESTING METHODS. There are two basic testing methods used to evaluate Army 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.

7. AIRCRAFT/PAVEMENT CLASSIFICATION NUMBERS (ACN/PCN). The ACN/PCN is a reporting method for weight-bearing capacity and not an evaluation procedure. The National Imagery and Mapping Agency 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 multiflight 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 capacity of airfields. 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 capacity 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.

d. ACN values will normally be provided by the aircraft manufacturers. The ACN has been developed for two types of pavements, flexible and rigid, and for four levels of subgrade strength.

e. The PCN numerical value for a particular pavement is determined from the allowable load-carrying capacity of the pavement. 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 to be based on 50,000 passes of the C-17 aircraft. Criteria for converting allowable loads to PCN values are presented in chapter 8.

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.

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 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 TI 825-01/AFM 32-1124(I)/NAVFAC DM 21.10. A more thorough investigation should be completed for the selection of final overlay design thicknesses.

9. EVALUATION OF AIR FORCE AIRFIELDS. Evaluations indicating the allowable pass/load relationship will be made for each aircraft group index (table 2-1). 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 2-4. Pass intensity levels are for normal conditions and frost melting periods.

Table 2-4
Pass Levels for Air Force Evaluation and Frost or Nonfrost Conditions

	Number of Passes for Aircraft Group Index		
Pass Intensity Levels	1-3	4-11	12-14
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

10. EVALUATION OF NAVY AND MARINE CORPS AIRFIELDS.

a. Navy and Marine Corps Air Stations are evaluated for 10-year life expectancy. The projected aircraft traffic for the next 10 years is first determined. Using aircraft equivalencies (e.g., from ICAO Aerodrome Design Manual) a design critical aircraft can be found for each feature, 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 5-2.

b. In the Navy procedure, the whole traffic is converted to passes to fully loaded F-14 (single tricycle category), then to passes of fully loaded P-3 (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 10 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.

d. One objective of the evaluation is to assess capability of the pavement to carry out its mission for the next 10 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 10 years:

B (BLUE) - No weight restriction
G (GREEN) - To be used only by half-loaded aircraft
Y (YELLOW) - To be used only by half-loaded 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 feature 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.

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 feature.

Table 2-5
ACN Values for Five Aircraft Types and Four Load Configurations (Peace Time)

Aircraft	Load	Peace Time Take Off Weight (Pounds)[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>12000	C CBR≤8 CBR>4 E≤12000 E>6000	D CBR≤4 E≤6000
F-14	Full	72,000	29	29	29	29	28	28	28	28
	Eval	65,000	26	26	26	26	25	25	25	25
	Half	54,350	22	22	22	22	21	21	21	21
	Empty	36,700	15	15	15	15	14	14	14	14
P-3	Full	135,000	40	43	44	45	36	39	42	46
	Eval	119,000	35	37	38	39	31	33	36	39
	Half	100,600	29	30	32	32	25	27	30	32
	Empty	66,200	17	18	19	20	15	15	17	20
C-130	Full	155,000	27	29	32	34	24	28	31	36
	Eval	143,000	24	27	29	31	22	26	28	32
	Half	113,500	19	21	23	24	17	20	22	25
	Empty	72,000	12	13	13	14	10	12	13	14
C-141	Full	325,000	42	50	59	65	44	50	62	75
	Eval	296,000	37	45	52	58	39	45	55	67
	Half	232,500	27	33	38	43	29	33	40	49
	Empty	140,000	13	15	18	21	15	16	18	23
C-5A	Full	769,000	26	31	40	49	33	36	44	61
	Eval	693,000	23	27	34	43	29	32	39	53
	Half	543,500	18	20	25	30	21	23	28	38
	Empty	318,000	10	10	10	12	10	10	12	15

[1] Conversion Factor
Kilograms = 0.453 × Pounds.

b. Collect necessary data.

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

d. Determine PCN values for each feature.

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.

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

a. Conduct a general survey of the airfield pavements and assign a qualitative rating to each feature.

b. Collect necessary data.

c. Determine allowable load for each feature based upon aircraft groups and standard pass levels.

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

e. Assign an overall PCN value to the airfield based on the C-17 aircraft and the weakest primary runway feature.

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 feature. Alternatively obtain most recent PCI survey from the corresponding Navy Engineering Field Division.

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

c. Conduct NDT of each feature using a FWD.

d. Determine actual traffic using the airfield and projected traffic for the next 10 years.

e. Determine tentative PCNs for each feature (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 feature.

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

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 capacity of a pavement will be less than if the same conditions of soil and water existed in a nonfreezing environment. Where such conditions exist, the rigid pavement evaluation will be

based on frost area indices of reaction (FAIR) and the flexible pavement evaluation on frost area soil support indices (FASSI) as given in chapter 7.

15. EMERGENCY CONSTRUCTION EVALUATION. Airfields or heliports constructed according to the emergency construction manuals will be evaluated using the criteria and procedures presented herein, except that they will be evaluated using 100, 1,000, and 10,000 passes.

CHAPTER 3

DATA COLLECTION

1. GENERAL. The selection of representative physical characteristics of a pavement requires a thorough study of all existing information and may 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.

2. STUDY OF EXISTING DATA. Existing data may be used to make 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, testing will be required.

a. Subgrade and Base-course Strength. In many instances, it may be found that subgrade and base-course strength determinations were made for the pavement 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 sections for the various features of the airfield, including thickness, thickened edges, types of joints, and load-transfer devices.

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 features. 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.

d. Condition of Existing Pavement. In some instances, recent condition-survey reports made in connection with special investigations can be obtained from the Geotechnical Laboratory, U.S. Army Engineer Waterways Experiment Station (WES), Vicksburg, MS, from the Air Force Civil Engineer Support Agency (AFCESA/CESC), Tyndall AFB, Florida, or the appropriate Navy Engineering Field Division Office, or the Naval Facilities Engineering Service Center. Up-to-date maintenance records should be obtained for all pavements.

e. Subgrade 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 ground-water table for the subgrade soils. Modulus of elasticity in flexure of stabilized materials meeting the requirements outlined in TM 5-822-14/AFJMAN 32-1019 for Army and Air Force and in MIL-HDBK-1021/4 for Navy and Marine Corps 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.

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 results of tests for Marshall stability, flow, percent bitumen by weight, density, voids relationships, aggregate gradation, specific gravity of bitumen and aggregate, and penetration (or viscosity) and ductility of bitumen. If the pavement were designed according to Strategic Highway Research Program (SHRP) criteria, the procedures and protocols governing the asphalt mixture design are available in the American Association of State Highway Transportation Officials (AASHTO) specifications. The results of test data should include: all asphalt binder testing to yield the SHRP performance grade (PG) used in the construction project including techniques such as Brookfield viscosity, flash point, dynamic shear rheometry, bending beam rheometry, and direct tension; all testing of modified asphalts that may include elastic recovery, force ductility, and phase separation potential; all aggregate testing including gradation, specific gravity, abrasion, soundness, crushed faces (fine and coarse aggregate fractions) thin, elongated particles, and clay content; all testing of the asphalt-aggregate mixture including gyratory compaction, specific gravity, water susceptibility, wheel tracking, and SHRP mixture analysis techniques. All of the previously mentioned testing should be in strict accordance with AASHTO specifications where appropriate.

h. Nondestructive Test Data. Nondestructive test (NDT) data required include 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.

3. COLLECTION OF NDT DATA.

a. Equipment. The NDT procedure evaluates response of a pavement system to an applied loading. An acceptable NDT device must provide an output containing a minimum of four deflections as follows: the near one is measured at the center of the applied load, the far one is at a distance of at least 1.22 meters (48 inches) from the applied load, and the other two deflections are spaced equidistance in between. 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 1.22 meters (48 inches) from the load, with the preferred minimum distance being 1.83 meters (72 inches). Of the remaining sensors, one should be located at the center of the loaded area and the others at approximately 305-millimeter (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 305-millimeter (1-foot) intervals from that point or adjustable to any spacing out to a distance of 1.83 meters (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). The load on the pavement (impulse force) from an FWD is created by dropping weights from different heights onto a rubber or spring buffer system. The standard loading plates used to transmit the applied force to the pavement are either 305 millimeters (12 inches) or 457 millimeters (18 inches) in diameter. The drop height can be varied to produce an impact force up to 224 kilonewtons

(50,000 pounds) depending on the FWD model being used. The requirements for the FWD test equipment and test procedures should be in accordance with ASTM D 4694. The FWD uses velocity transducers to measure the pavement response to the applied load. The number of velocity transducers depends on the manufacturer and ranges from a minimum of four to a maximum of nine. 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 is particularly helpful in determining the testing program.

(1) Test Locations. On runways and taxiways, deflection basin measurements should be made every 31 meters (100 feet) on alternate sides of the center line along the main gear wheel paths. For flexible pavements, the offset is usually 3 to 3.5 meters (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 31- to 61-meter (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 features. Figure 3-1 shows NDT test locations for a typical airfield.

(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 modulus of 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 305- or 457-millimeter (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.

(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 3-2. 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. Joint efficiency tests should 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. Reference point tests should 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 should be selected within each feature to be tested on a given day. Joint tests should 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 feature.

c. 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 25 millimeters (1 inch) below the top, 25 millimeters (1 inch) above the bottom, and at middepth 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 3-3. 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 3-4.

4. 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 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 for collecting more detailed data can be located to the best possible advantage. Data for these profiles can be obtained by coring 100- or 150-millimeter- (4- or 6-inch-) diameter holes in the pavement, through which thickness measurements can be made and samples of the foundation materials obtained. These samples should be classified in accordance with the Unified Soil Classification System as presented in ASTM D 2487. Usually, a spacing of 152 to 305 meters (500 to 1,000 feet) between these small holes will be sufficient, but occasionally when nonuniformity of pavement or foundation conditions exists, closer spacings may be necessary. From the information obtained, the pavements should be divided into features on the basis of pavement type, construction history, known strength, thickness, and foundation types.

(1) The preliminary sampling locations should enable test pits to be placed in locations representing typical pavement and foundation conditions. In addition, the test pits should be placed in 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, a nominal number of test pits (five or six) will generally be sufficient if they are located to provide representative information for the entire system of airfield pavements. When the pavement or foundation conditions are not uniform, test pits 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 test pits must be located in the failed or distressed areas to determine the cause of the failure or distress.

(3) 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 must 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 152 by 152 millimeters (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 762-millimeter- (30-inch-) diameter plate, test pits should be 1.22 by 1.52 meters (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 152-millimeter- (6-inch-) diameter core samples. An equation for calculating flexural strength from tensile splitting strength is presented in appendix B.

(4) Test pits for flexible pavements (approximately 1.22 meters (4 feet) wide by 1.52 meters (5 feet) long) or core holes (up to 200 millimeters (8 inches) in diameter) are dug through the pavement to permit the performance of in-place tests and to obtain samples for laboratory tests. Tests conducted in a core hole are referred to as small aperture testing. Core holes up to 200 millimeters (8 inches) in diameter do not create an operational problem for most aircraft but a 1.22- by 1.52-meter (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 6 millimeters (1/4 inch) and the total thickness of base and pavement to the nearest 13 millimeters (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 6 millimeters (1/4 inch).

(2) Modulus of Soil Reaction.

(a) All Rigid Pavements. The modulus of soil reaction on the subgrade or base course should be determined by the plate-bearing test as discussed in appendix B. In those instances when the plate-bearing test cannot be conducted, an approximate value of k can be determined by taking California Bearing Ratio (CBR) readings on the subgrade in 152-millimeter (6-inch) core holes (small aperture procedure) and determining the k value from the curve in figure 3-5. 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. The relationship between the thickness of base or subbase and the effective k of the base or subbase may be determined using figure 3-6. With subgrades or base courses that have been modified, the k value will be determined from figure 3-5 as previously noted. Subgrade or base-course materials that have been stabilized to the extent that they qualify as stabilized layers as outlined in TM 5-822-14/AFJMAN 32-1019 require tests other than plate-bearing to determine their effect on the supporting value of the pavement structure. Plate-bearing tests are also required in other areas as indicated in the following paragraphs.

(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 102 millimeters (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 Overlays of Flexible Pavements in chapter 6.

(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 54 MN/cubic meter (200 pci) or the concrete flexural strength is less than 2.758 MPa (400 psi), a higher load-carrying capacity 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. There are two basic types of penetrometers that can be used to evaluate pavements: the Electric Cone Penetrometer (ECP) and the Dynamic Cone Penetrometer (DCP). The ECP is mounted in a C-130 transportable vehicle and measures the shear strengths of the various subsurface material layers. The ECP uses a standard 35.8-millimeter (1.41-inch)-diameter cone with a 60-degree conical tip. The cone point is hydraulicly pushed through the pavement structure typically to a depth of 1.52 meters (5 feet) at a rate of 20.3 millimeters (0.8 inches)/second. The ECP can provide valuable information pertaining to the pavement structure including bearing strength (correlated to CBR), layer thicknesses, and material classification. The DCP is a hand-held portable penetrometer device designed to penetrate soils to depth of 0.99 meters (39 inches). The 20.3-millimeter (0.79-inch)-diameter 60-degree cone is driven into the ground by raising and dropping a 7.97-kilogram (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 50.8-millimeter (2-inch) (ECP) or 25.4-millimeter (1-inch) (DCP)-diameter hole is drilled through the portland cement concrete (PCC) until the top of the base subgrade is encountered. The test device is then lowered to this point to begin the test sequence. Detailed test procedures and correlations for using the ECP and DCP are provided in Appendix B. The ECP is typically used for tests requiring greater penetration depths. The DCP is adequate for most pavement structures and is considered easier to deploy and implement.

(6) Field Density Tests. Density tests must 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. The nuclear density meter may also be used to determine densities, but special care must be taken because of the influence of the sides of the test pits on test results. 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 0.075 to 4.75 millimeters (No. 200 to No. 4). For the purposes of this document, material passing the 0.42-millimeter (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 0.42-millimeter (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 0.42-millimeter (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 0.42-millimeter (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 152-millimeter (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 305-millimeter (1-foot) intervals to a total depth of 1.2 meters (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 of particular importance. 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) Penetrometer Tests. The two basic types of penetrometer tests that can be used to evaluate pavements are the ECP and the DCP. The ECP is located within a C-130 transportable vehicle and measures the shear strengths of the various subsurface material layers. The ECP cone point is hydraulicly pushed through the pavement structure typically to a depth of 1.52 meters (5 feet) at a rate of 20.3 millimeters (0.8 inches)/second. The ECP can provide valuable information pertaining to the pavement structure including bearing strength (correlated to CBR), layer thicknesses, and material classification. The DCP is a hand-held portable penetrometer device designed to penetrate soils to a depth of 0.99 meters (39 inches). The cone tipped rod is driven into the ground by raising an dropping a 7.97-kilogram (17.6-lb) hammer. Data are collected in terms of a DCP index value (mm of penetration/ blow). The index can then be correlated to CBR using derived relationships. Pavement layer thickness can be determined by examining zones of uniform strength 25.4 millimeter (1 inch) (DCP) diameter hole is drilled through the asphalt concrete (AC) wearing surface to the top of the base layer. The test device is then lowered to this point to begin the test sequence. Detailed test procedures and correlations for using the ECP and DCP

are provided in Appendix B. The ECP is typically used for tests requiring greater penetration depths. The DCP is adequate for most pavement structures and is considerably easier to deploy and implement.

(4) Density Determinations. Three density determinations should be made at each elevation tested if samples of about 0.0014-cubic-meter (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 80 kilograms/cubic meter (5 pounds per cubic foot) wet density. For example, test results of 1730, 1,777, and 1,810 kilograms/cubic meter (108, 111, and 113 pounds per cubic foot) wet density are in reasonable agreement, and their average is 1,777 kilograms/cubic meter (111 pounds per cubic foot). A nuclear density device is used for density determinations in the small aperture test method.

5. 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 specimens should be slightly more than three times as long and three times as wide as the pavement thickness, except when 152- by 152-millimeter (6- by 6-inch) beams are cut from the top and bottom of the specimens for three-point load beam tests.

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 91-kilogram (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 272 kilograms (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 TM 825-01/ AFM 32-1124(I)/NAVFAC DM 21.10. The samples to be obtained from the various materials are summarized in the following tabulation:

Material	**Samples Per Pit**	**Remarks**
Pavement	8 cores, 91 kilograms (200 pounds) of chunks	Chunks should be 203-254 millimeters (8-10 inches) in minimum dimension to permit separation of courses
Base and subbase courses	272 kilograms (600 pounds)	Disturbed sample
	3 samples	Undisturbed cylinders to be taken of material with plastic fines where applicable
Subgrade	204 kilograms (450 pounds)	Disturbed sample. Increase to 272 kilograms (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 45-kilogram (100-pound) bag sample will be sufficient; if the subgrade is composed of a granular material, a 91-kilogram (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 204 and 272 kilograms (450 and 600 pounds) for fine-grained and granular materials, respectively.

6. 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.

b. Flexible Pavement and Nonrigid Overlays.

(1) Where a 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 SHRP 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 void in the total mix and the percentage of voids filled with asphalt should be computed from the test results (CRD C-650, AASHTO Specifications from 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 non-rigid 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.

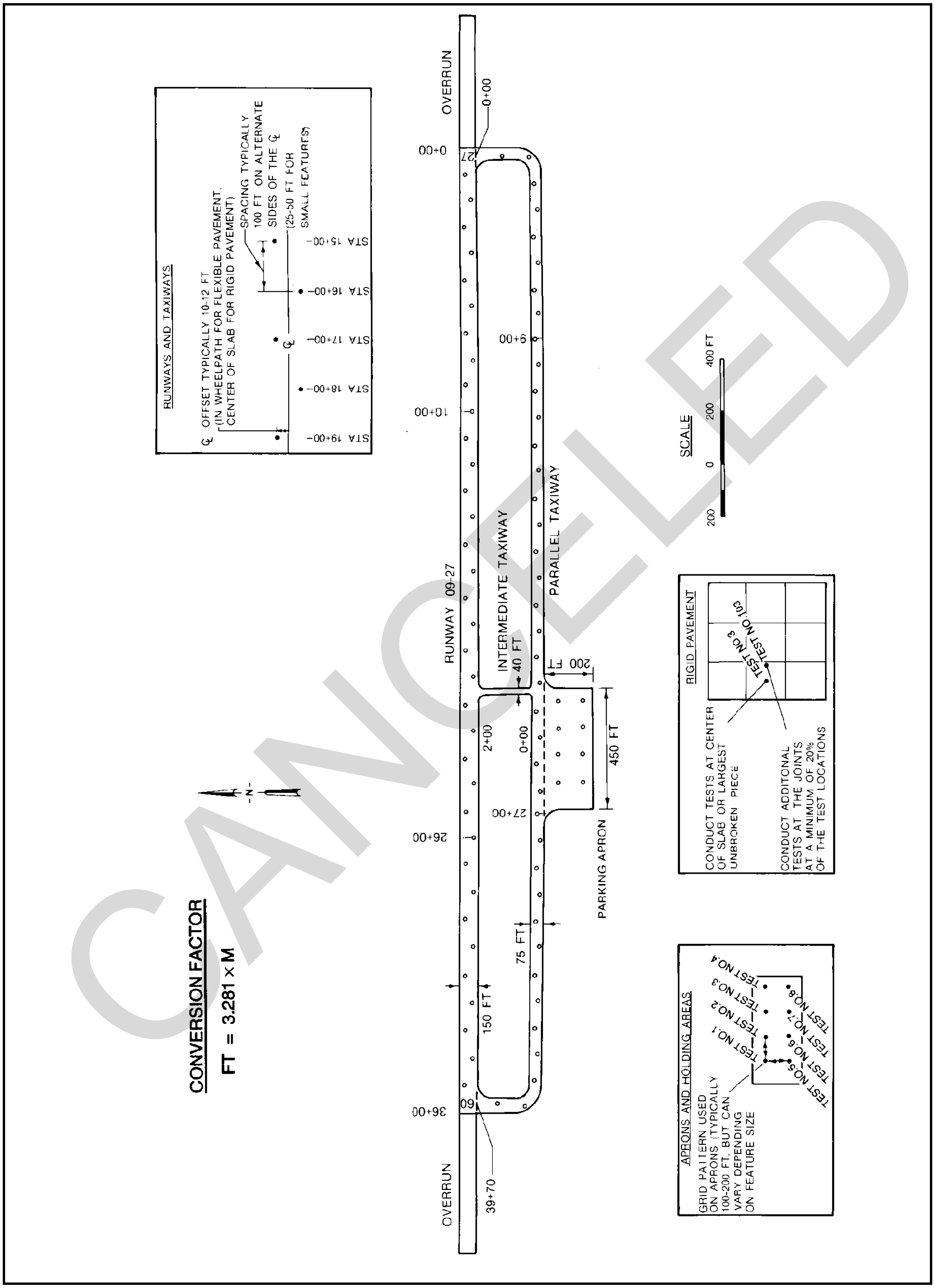


Figure 3-1. Typical airfield layout showing recommended NDT testing frequencies

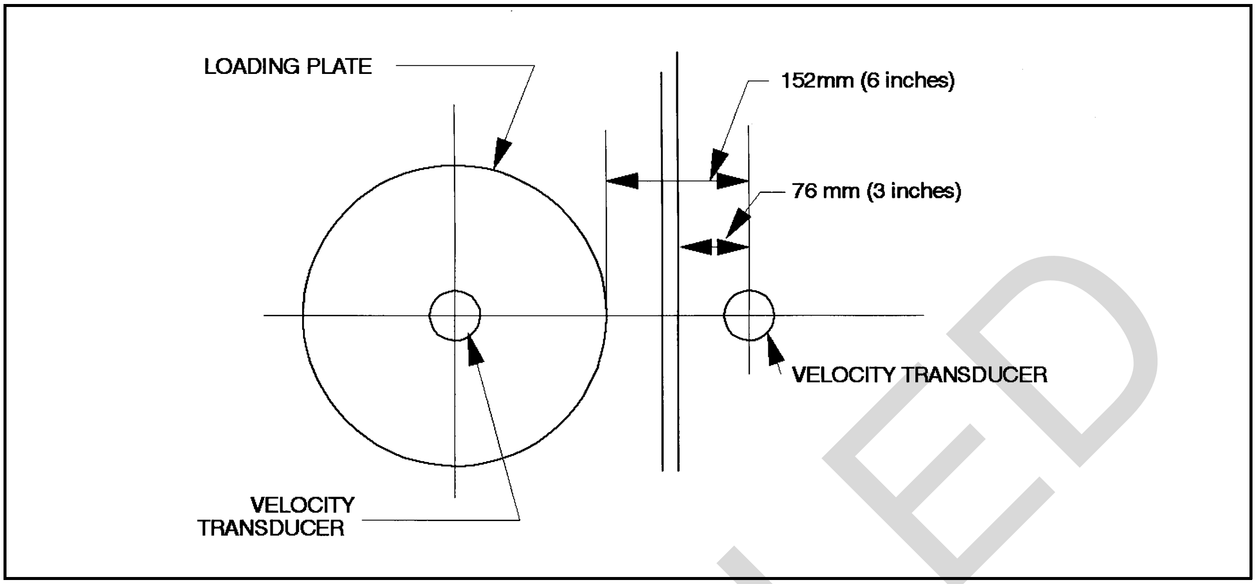


Figure 3-2. NDT equipment configuration for joint load-transfer measurements

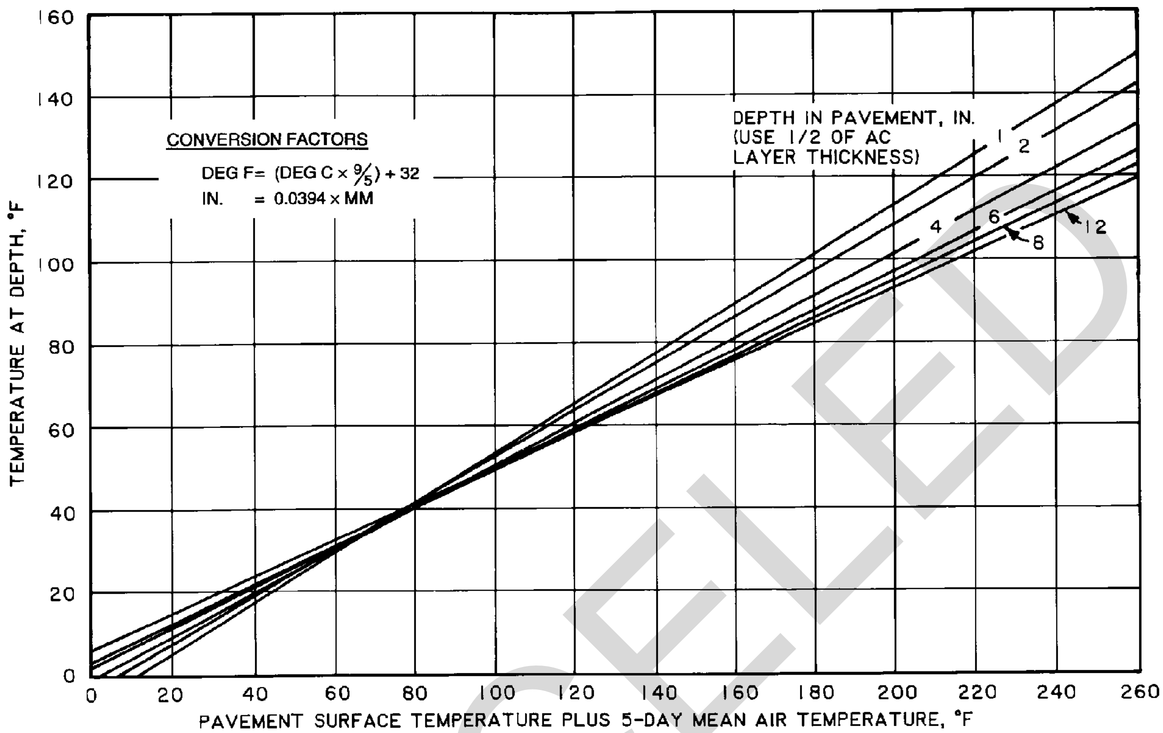


Figure 3-3. Prediction of pavement temperature for bituminous layers

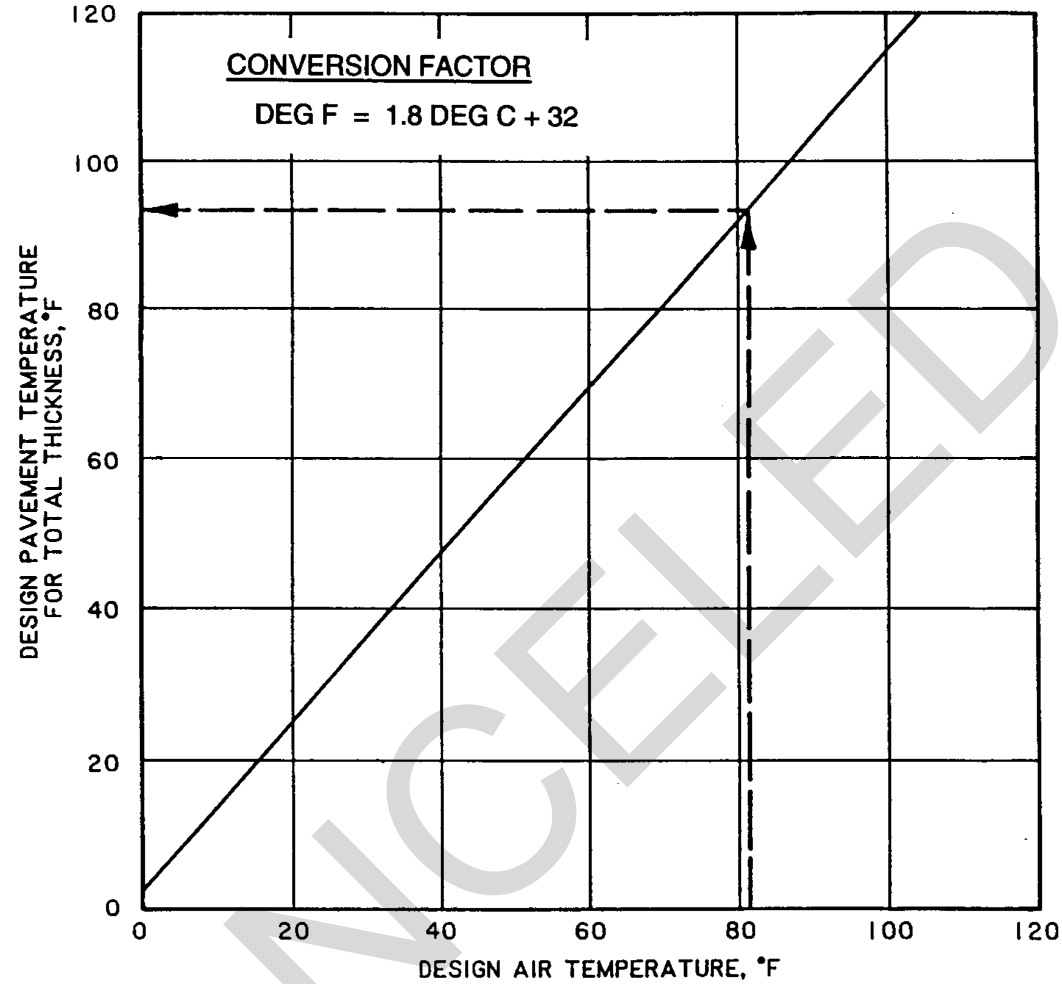


Figure 3-4. Relationship between design pavement temperature and design air temperature

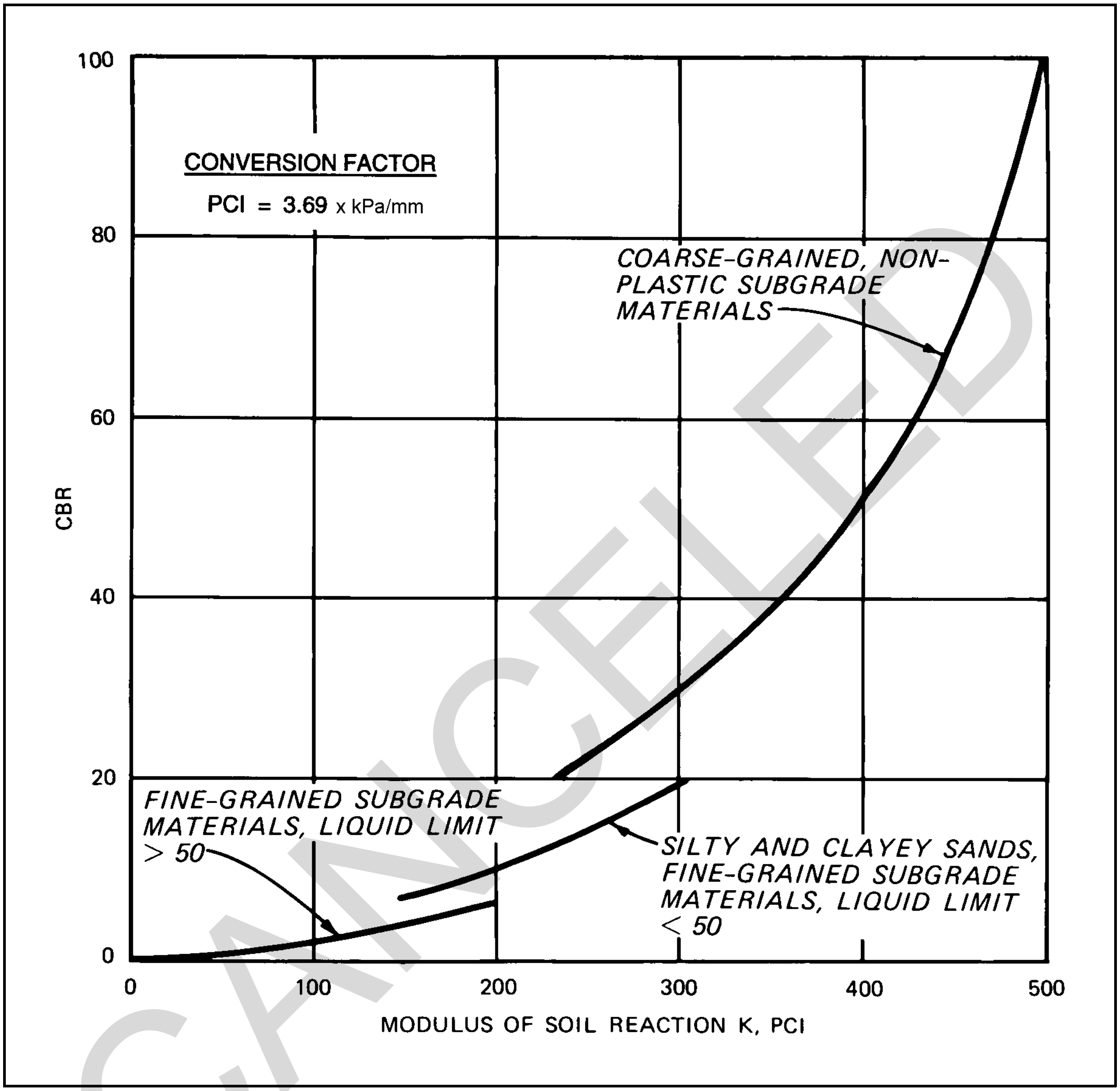


Figure 3-5. General relationship between CBR and modulus of subgrade or base-course reaction

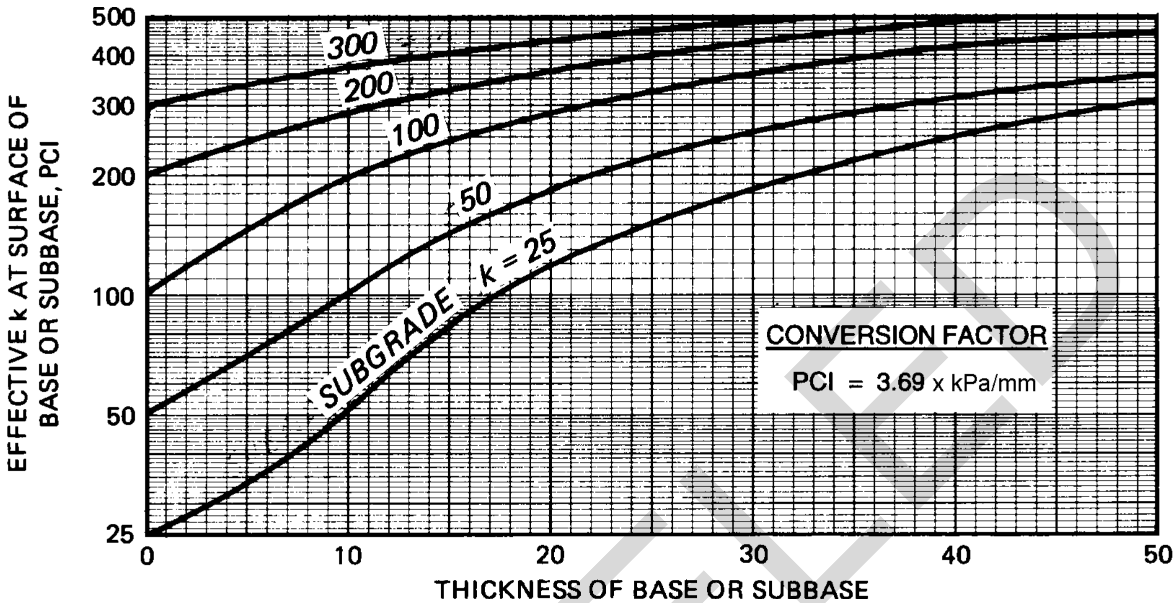


Figure 3-6. Curves for determining the effective k value

CHAPTER 4

PAVEMENT EVALUATION USING NONDESTRUCTIVE TESTING

1. EVALUATION PRINCIPLES. 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. 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.

2. PAVEMENT RESPONSE MODEL. The computer code recommended for computing pavement response is the five-layer linear elastic program WESLEA, which is a subroutine of the Layered-Elastic Evaluation Program (LEEP). 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 of infinite thickness.

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

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 yet available for reinforced or fibrous pavements. The procedure outlined in this section 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. The evaluation procedure is computer based, and installation guidelines for these programs are given in appendix C. Computer programs needed for the evaluation may be obtained as discussed in chapter 9.

4. STEP 1 (FEATURE IDENTIFICATION). Pavement facilities are divided into features according to type of traffic area, pavement type, and/or construction.

a. Traffic Areas. Air Force airfield pavements are categorized by traffic area as a function of traffic distribution and aircraft weight. The Air Force defines traffic areas in four categories (types A, B, C, and D) as described in TI 825-01/AFM 32-1124(I)/NAVFAC DM 21.10. The Army defines traffic areas in three categories (types A, B, and C). The Navy and Marine Corps define traffic areas as primary and secondary. For evaluation purposes, the Navy and Marine Corps also use the Army defined A, B, and C traffic areas. "A" is used for channelization traffic typically found on aprons. The terms "primary" and "secondary" refer to the pavement rank which is assigned as part of the condition survey. A primary pavement could have either “A” or “B” traffic depending on its use. Location of traffic areas depends on the airfield class or type as defined in TI 825-01/AFM 32-1124(I)/NAVFAC DM 21.10, for the Army and Air Force and in MIL-HDBK-1021/2 for the Navy and Marine Corps.

b. Pavement Type and Construction. After the pavements have been categorized by traffic area, further subdivide each area, if necessary, into features having the same pavement type and construction. Each area should be assigned a feature designation denoting the feature type (runway, taxiway, or apron), the feature number (numerical identification within a given feature type), and the type of traffic area. Figure 4-1 illustrates proper feature identification and designation for a typical airfield.

5. STEP 2 (SELECT REPRESENTATIVE DEFLECTION SECTIONS). Either all basins, selected basins, or a representative deflection basin is selected for each pavement feature to be evaluated. Depending on the speed of the computer systems used, all basins or a representative basin may be analyzed for each pavement section. For faster computer systems it is recommended that all basins be analyzed and the mean modulus value for each layer will be 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. The computations which are made by the computer program BASIN are as follows:

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 feature may supposedly be of the same type and construction, it should be treated as more than one pavement group when the strength characteristics measured in one section of the feature are greatly different from those in another 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 feature 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 feature divided by the load at which the deflection was obtained).

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

d. The area of each deflection basin is determined as illustrated in figure 4-2. 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 4-3. 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:

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

where

ISM = computed ISM

DF = measured deflection

AREA = computed area

ND = number of deflection sensors

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

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

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

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

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.

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 4-4. 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. To summarize the modulus backcalculation 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 4-5 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:

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

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.

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

Material	Range		Initial Estimate	Poisson's Ratio
	Minimum	Maximum		
Asphalt concrete	689 (100,000)	13,780 (2,000,000)	2,411 (350,000)	0.35
Portland cement concrete	17,222 (2,500,000)	48,230 (7,000,000)	24,115 (3,500,000)	0.15
Resin Modified Pavement*	4,823 (700,000)	20,669 (3,000,000)	11,713 (1,700,000)	0.27
High-quality stabilized base	3,445 (500,000)	17,222 (2,500,000)	6,890 (1,000,000)	0.20
Base-subbase, stabilized	689 (100,000)	6,890 (1,000,000)	2,067 (300,000)	0.25
Base-subbase, unstabilized	34 (5,000)	1,033 (150,000)	207 (30,000)	0.35
Subgrade	6.9 (1,000)	344 (50,000)	103 (15,000)	0.40

* To be added to WESDEF later.

e. If the deflection basin includes a deflection measured at an offset distance of 1.83 meters (72 inches), the initial subgrade modulus is estimated as follows:

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

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

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 be used if this situation is encountered.

f. Typically, 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 3-2. The assigned AC modulus is obtained using figure 4-6 and the loading frequency for the NDT device. The FWD 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.

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).

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 is 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 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. 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.

7. STEP 4 (DETERMINE DESIGN TRAFFIC).

a. The total number of passes of each aircraft type that the pavement will be expected to support over its design life must be projected. The normal design life for airfield pavements is 20 years. The Navy and Marine Corps use 10 years for evaluation purposes. For a runway, passes are determined by the number of aircraft movements across an imaginary transverse line placed within 500 feet of the end of the runway. Touch-and-go aircraft operations are typically not counted as passes. In some cases, and for Navy and Marine Corps evaluations, they may be counted for large aircraft (which may produce significant damage to the pavement), or for the case of outlying airfields (which receive essentially touch-and-go operations). 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. The designer should strive to obtain data for a specific airfield facility under evaluation to forecast aircraft traffic operations over the design life of the pavement. For a given projected aircraft mixture, the critical aircraft and design pass level 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 computer program TRAFFIC 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 airfield or pavement feature. 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.

(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, and to determine equivalent traffic, it is recommended to use the maximum peace-time take-off weight, and the maximum design landing weight from the Army ETL 1110-3-394 and the Navy Aircraft Characteristics supplement (see Table 4-2). If desired, the analyst may use maximum war-time take-off weights, but these weights are unlikely, even during war-time. If desired, the analyst may also use more realistic, measured weights (in this case it is advised that the measured average weight plus one standard deviation be used).

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 existing (typically five or less) 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 each actual aircraft passes by its participation. If this is done for each aircraft category, a tentative PCN can be found for each one.

8. STEP 5 (COMPUTE ALLOWABLE AIRCRAFT LOADS, ALLOWABLE PASSES, REQUIRED OVERLAY THICKNESS, AND PCN). Allowable load-carrying capacities and required overlay thicknesses are evaluated using the computer program WESPAVE. For a particular aircraft (gear configuration, load, pass intensity level, etc.), WESPAVE uses modulus values from WESDEF and computes stresses (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 are presented in the remainder of this section.

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 (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.

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

Aircraft Designation	Peace-Time Maximum Take-off Weight (kips)	Design Maximum Landing Weight (kips)	Aircraft Designation	Peace-Time Maximum Take-off Weight (kips)	Design Maximum Landing Weight (kips)	Aircraft Designation	Peace-Time Maximum Take-off Weight (kips)	Design Maximum Landing Weight (kips)
A-10A	50.0	33.0	Boeing 727-200	209.5	161.0	C-22B	170.5	142.5
A-6E	60.4	50.0	Boeing 737-100	110.0	99.0	C-2A	60.0	57.5
A-7D/K	42.0	37.1	Boeing 737-200	128.1	107.0	C-5A/B	769.0	769.0
AC-130A	124.2	124.2	Boeing 737-300	135.0	114.0	C-9A	108.0	99.0
AC-130H/U	155.0	175.0	Boeing 737-400	150.0	124.0	C-9B	110.0	99.0
SE-210 Caravelle	110.2	105.0	Boeing 747-100B	750.0	585.0	CH-53A/D	42.0	42.0
Airbus A300-600	363.8	304.2	Boeing 747-200B-Passeng.	833.0	630.0	Convair 240	42.5	39.8
Airbus A300B2	313.1	295.4	Boeing 747-200C-Cargo	833.0	630.0	Convair 340	47.0	46.5
Airbus A300B4	363.8	295.4	Boeing 747-200C-Passeng.	800.0	630.0	Convair 440	49.1	47.7
Airbus A310-300	346.1	273.4	Boeing 747-200F-Cargo	833.0	630.0	Convair 580	54.6	52.0
Airbus A320	158.7	138.9	Boeing 747-300-Passeng.	833.0	574.0	Convair 600	46.2	44.0
AV-8B/TAV-8B	32.0	26.0	Boeing 747-400	870.0	630.0	Convair 640	55.0	52.0
B-1B	440.0	346.5	Boeing 747SP	696.0	450.0	Douglas DC-4	73.0	63.5
B-2	336.5							
B-52G	488.0	450.0	Boeing 757-200	240.0	198.0	Douglas DC-6	106.0	88.2
B-52H	488.0	450.0	Boeing 767-200	315.0	272.0	Douglas DC-7	143.0	109.0
B.A.C. One-Eleven 200	80.0	71.0	Boeing 767-300	351.0	300.0	E-2C	53.0	45.0
B.A.C. One-Eleven 400	89.5	79.0	Boeing 767-300ER	380.0	300.0	E-3A	325.0	250.0

Table 4-2 (Continued)

Aircraft Designation	Peace-Time Maximum Take-off Weight (kips)	Design Maximum Landing Weight (kips)	Aircraft Designation	Peace-Time Maximum Take-off Weight (kips)	Design Maximum Landing Weight (kips)	Aircraft Designation	Peace-Time Maximum Take-off Weight (kips)	Design Maximum Landing Weight (kips)
B.A.C. One-Eleven 500	104.5	87.0	Boeing-767-200ER	351.0	278.0	E-3B	325.0	250.0
B.A.C./SNIAS Concorde	408.0	245	British Aero. 146-Model 100	84.0	77.5	E-3C	325.0	250.0
B.A.C./Vickers VC-10 1100	312.0	216.0	British Aero. 146-Model 200	93.0	81.0	E-4B	798.0	630.0
B.A.C./Vickers VC-10 1150	335.0	237.0	British Aero. 146-Model 300	93.0	83.0	E-6A	110.0	99.0
B.A.C./Vickers Viscount 745	64.5	57.5	C-130B	135.0	135.0	EA-6B	61.5	50.0
B.A.C./Vickers Viscount 810	72.5	64.0	C-130E/H	155.0	175.0	EC-130E/H	1555.0	175.0
Boeing 707-120B	257.3	190.0	C-130F	155.0	173.4	EC-135A/G/L	300.8	297.0
Boeing 707-320	312.0	207.0	C-130T	155.0	173.4	EC-135C/J	301.6	297.0
Boeing 707-320B	333.6	215.0	C-135B	301.6	297.0	E-135E/H/K/P/Y	301.6	297.0
Boeing 707-320C	333.6	247.0	C-137B	258.0	190.0	EC-18B	336.0	247.0
Boeing 707-420	312.0	207.0	C-137C	326.0	247.0	EF-111A	89.2	82.5
Boeing 720	229.3	175.0	C-140A/B	40.9	35.0	EP-3E	142.0	114.0
Boeing 720B	234.3	175.0	C-141B	325.0	323.0	F-100	41.5	41.5
Boeing 727-100C	160.0	142.5	C-17A	580.0	580.0	F-101B	52.4	35.5
Boeing 727-100C	160.0	142.5	C-20A/B	69.7	58.5	F-104G	29.0	17.2
F-105G	54.6	51.7	KC-130T	155.0	173.4	McDonnell Douglas DC-8-71F	328.0	258.0
F-106A/B	41.0	42.5	KC-135R	322.5	322.5	McDonnell Douglas DC-8-72	335.0	240.0
F-117A	52.5							

Table 4-2 (Continued)

Aircraft Designation	Peace-Time Maximum Take-off Weight (kips)	Design Maximum Landing Weight (kips)	Aircraft Designation	Peace-Time Maximum Take-off Weight (kips)	Design Maximum Landing Weight (kips)	Aircraft Designation	Peace-Time Maximum Take-off Weight (kips)	Design Maximum Landing Weight (kips)
F-111A/E	91.3	72.0	LC-130F	155.0	173.4	McDonnell Douglas DC-8-73	355.0	258.0
F-111D	97.5	82.5	LC-130R	155.0	173.4	McDonnell Douglas DC-8-73F	355.0	275.0
F-111F	97.0	82.5	Lockheed 1049	135.4	122.0	McDonnell Douglas DC-8-SSF	325.0	240.0
F-14A	72.0	60.0	Lockheed 1329	42.0	35.0	McDonnell Douglas DC-9-15	90.7	81.7
F-14B	72.0	60.0	Lockheed 1649A	156.0	123.0	McDonnell Douglas DC-9-15F	90.7	81.7
F-14D	72.0	60.0	Lockheed 749A	107.0	89.5	McDonnell Douglas DC-9-21	98.0	95.3
F-15A/B/C/D	68.0	68.0	Lockheed L-1011-1, Tristar	430.0	358.0	McDonnell Douglas DC-9-32	108.0	99.0
F-15E	81.0	81.0	Lockheed L-1011-100, Tristar	466.0	368.0	McDonnell Douglas DC-9-41	114.0	102.0
F-16A	34.6	35.4	Lockheed L-1011-200 Tristar	466.0	368.0	McDonnell Douglas DC-9-51	121.0	110.0
F-16B	34.7	35.4	Lockheed L-1011-500	496.0	368.0	McDonnell Douglas MD-81	140.0	128.0
F-16C	36.3	37.5	Lockheed L-1011-500 Tristar	496.0	368.0	McDonnell Douglas MD-82	149.5	130.0
F-16D	35.6	37.5	Lockheed L-188	116.0	95.7	McDonnell Douglas MD-83	160.0	139.5
F-4C	51.7	46.0	Lockheed L-382 (L-100-20)	155.0	130.0	McDonnell Douglas MD-87	149.5	130.0
F-4D	51.5	46.0	Lockheed L-382 (L-100-30)	155.0	135.0	McDonnell Douglas MD-88	149.5	130.0

Table 4-2 (Concluded)

Aircraft Designation	Peace-Time Maximum Take-off Weight (kips)	Design Maximum Landing Weight (kips)	Aircraft Designation	Peace-Time Maximum Take-off Weight (kips)	Design Maximum Landing Weight (kips)	Aircraft Designation	Peace-Time Maximum Take-off Weight (kips)	Design Maximum Landing Weight (kips)
F-4E	53.8	46.0	Martin 404	42.8	41.0	McDonnell Douglas DC-9-33F	108.0	99.0
F/A-18A/B/C/D	51.9	39.0	MC-130E/H	155.0	175.0	Nihon/N.A.M.C. YS-11A	55.1	54.0
Fairchild F-27	42.0	40.0	McDonnell Douglas DC-10-10	440.0	363.5	P-3C	135.0	114.0
Fairchild FH-227	45.5	45.0	McDonnell Douglas DC-8-72F	335.0	250.0	RC-135S/W	301.6	297.0
FB-111A	114.3	114.3	McDonnell Douglas DC-10-30	555.0	421.0	RC-135U/V	301.6	297.0
Fokker F-28	62.0	54.0	McDonnell Douglas DC-10-40	555.0	421.0	RF-4C	52.8	46.0
Gen. Dyn./Convair 880	193.0	155.0	McDonnell Douglas DC-8-43	315.0	207.0	S-3A	52.5	45.9
Gen. Dyn./Convair 990	253.0	202.0	McDonnell Douglas DC-8-55	325.0	217.0	SR-71A	170.0	??
Grumman Gulfstream I	35.1	33.6	McDonnell Douglas DC-8-61	325.0	240.0	TC-130Q	155.0	173.4
Grumman Gulfstream II	65.5	58.5	McDonnell Douglas DC-8-61F	328.0	258.0	TC-4C	36.0	34.3
Hawker Siddeley HS-748	44.5	43.0	McDonnell Douglas DC-8-62	350.0	240.0	TR-1A	40.0	??
Ilyushin IL-62	357.2	231.5	McDonnell Douglas DC-8-62F	350.0	250.0	VC-25A	836.0	630.0
KC-10A	590.0	436.0	McDonnell Douglas DC-8-63	355.0	258.0	WC-130E/H	155.0	175.0
KC-130F	155.0	173.4	McDonnell Douglas DC-8-63F	355.0	275.0	WC-135B	301.6	297.0
KC-130R	155.0	173.4	McDonnell Douglas DC-8-71	325.0	240.0			

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 strains at the bottom of the AC layer and vertical subgrade strains are considered in the evaluation of flexible pavements. The limiting AC strain criterion (shown graphically in figure 4-7) is as follows:

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

where

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

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

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

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

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

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

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:

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

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 80 corresponds to the formation of one or more cracks per slab in 50 percent of the trafficked slabs. However, experience has indicated that an SCI of 80 is somewhat conservative, and a value of SCI = 50 is recommended. Figure 4-9 shows the relationship of SCI in terms of design factor versus coverages. 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:

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

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

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

where

$ALLOWABLE\ STRESS_{PCC}$ = 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 figure 4-10 with controlling wheels and the recommended minimum number of stress/strain evaluation positions indicated. The computer program LEEP has a data file (NEWFILE.DAT) which contains the number and location of controlling wheels and evaluation positions.

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 4-11 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. Interpretation of WESPAVE Output. A typical WESPAVE output is shown on figure 4-12. The pavement evaluation is conducted for a specified number of passes of an aircraft. Following are the results from WESPAVE which must be fully understood and correctly interpreted:

(1) Allowable Load. The load on the critical aircraft that can be supported by the pavement for the desired number of passes.

(2) Allowable Passes. The number of passes of the critical aircraft at the load that can be applied to the pavement.

(3) Overlays. The additional pavement thickness required to adequately support the critical aircraft at a given load for a given number of passes. These thicknesses are to be used for planning only. A more thorough investigation is required for actual design. For flexible pavements, the overlay is the required thickness of AC. For rigid pavements, overlays are given for AC, partially bonded PCC (PCC placed directly on an existing slab), and unbonded PCC (PCC placed on a leveling course or bond breaker). For composite pavements, the existing AC overlay is considered a bond breaker, and the partially bonded case is not considered. Overlays are determined using criteria in TI 825-01/AFM 32-1124(I)/NAVFAC DM 21.10, chapter 3, or TM 5-825-2-1/AFM 88-6, chapter 2, section A, and TM 5-825-3-1/AFM 88-6, chapter 3, section A, for Army and Air Force pavements and in MIL-HDBK-1021/4 for Navy and Marine Corps pavements.

(4) PCN. PCN is computed for a critical aircraft using the allowable gross aircraft weight and subgrade strength determined from the evaluation. PCN for Air Force pavements is determined for 50,000 passes of a C-17 Aircraft.

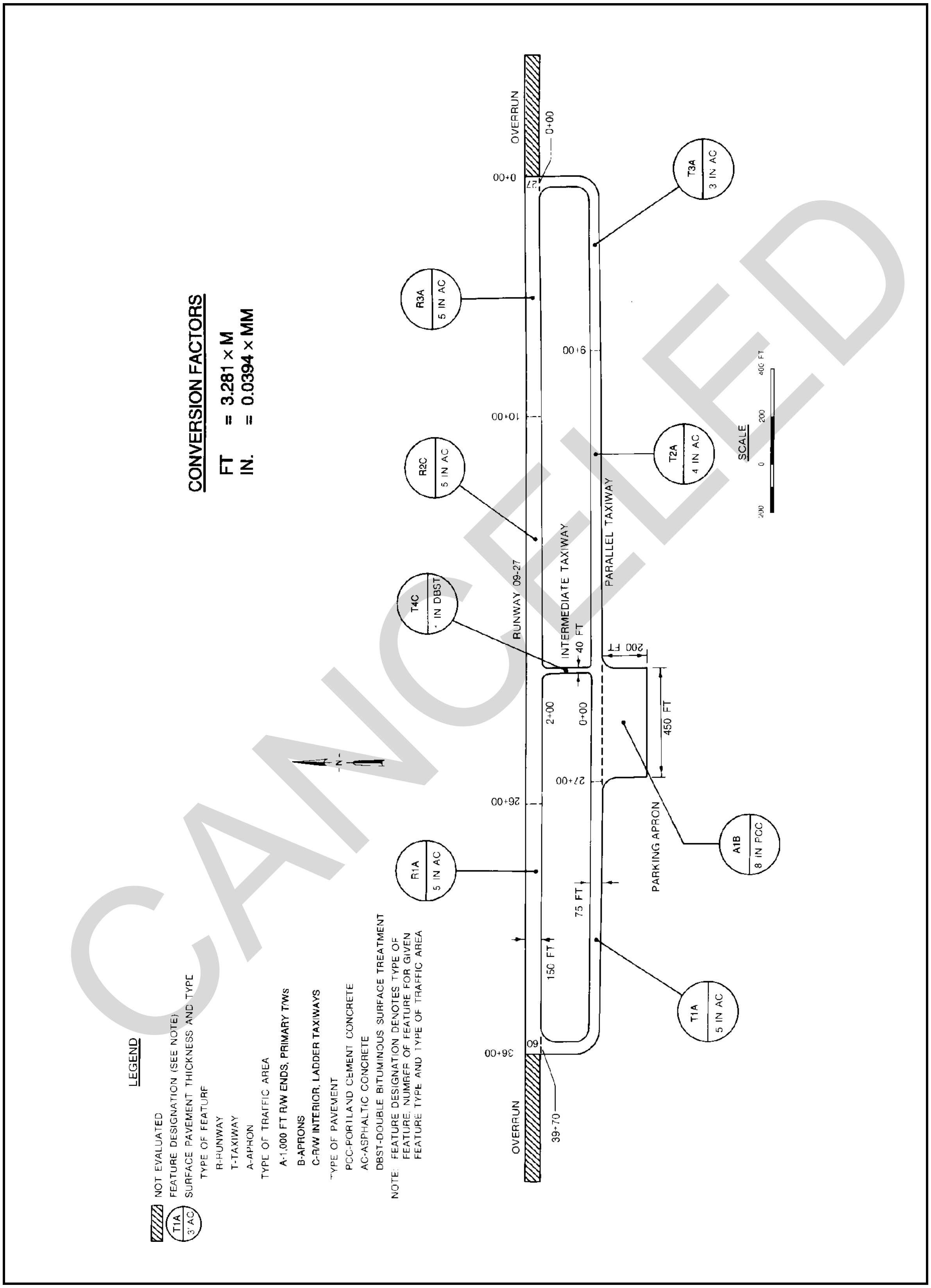


Figure 4-1. Typical Army airfield layout showing feature identification

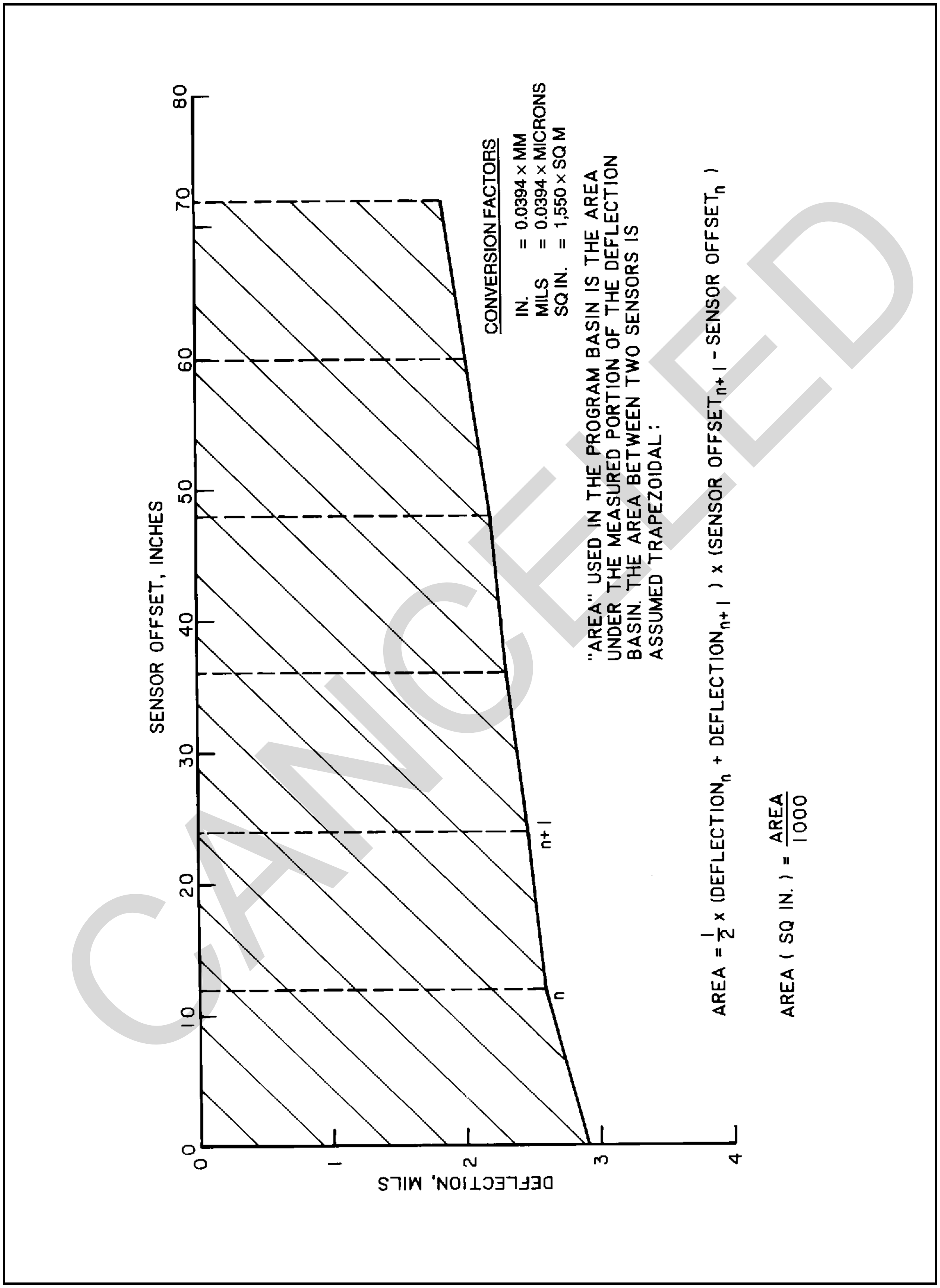


Figure 4-2. Determination of area beneath deflection basin

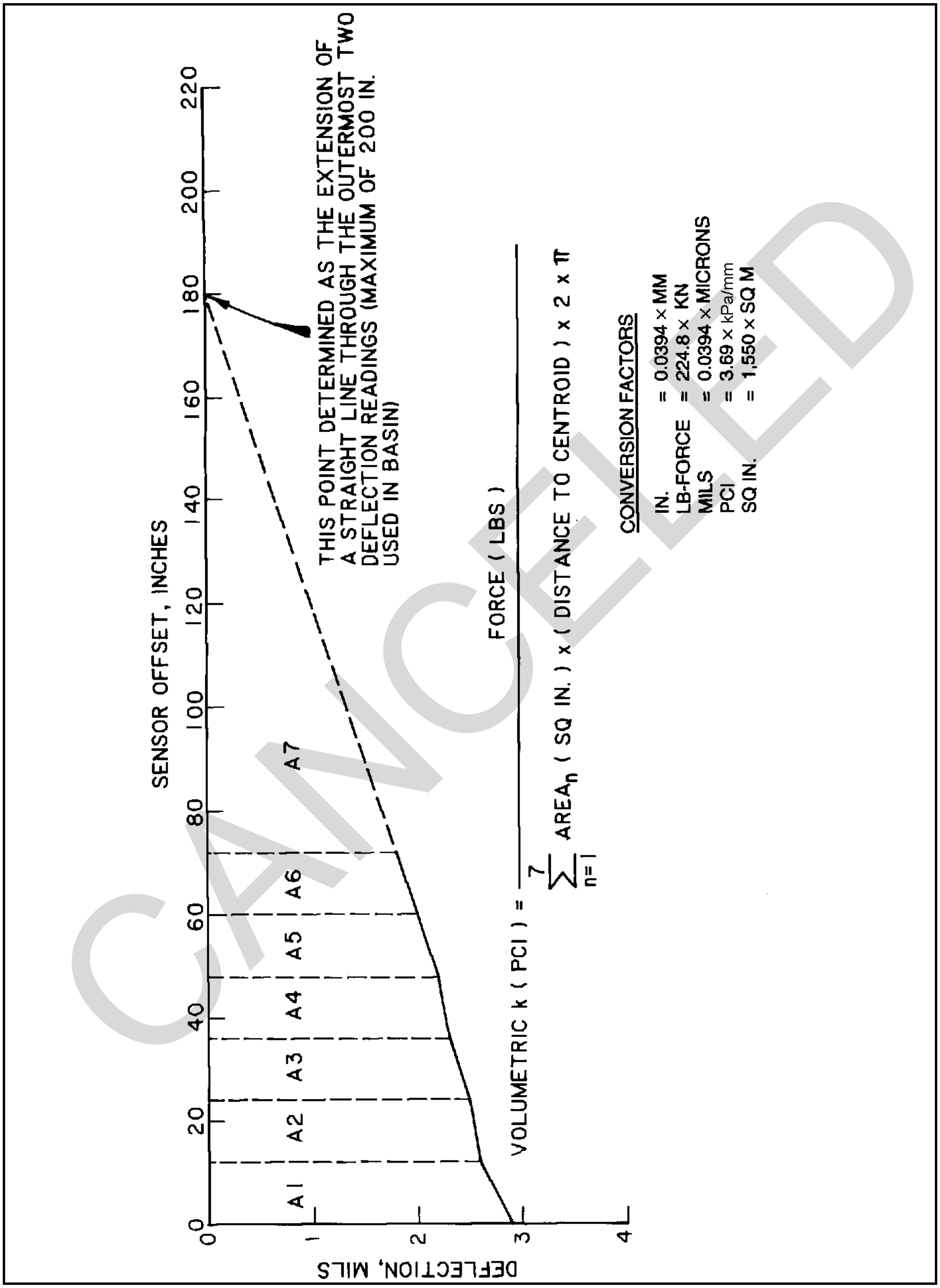


Figure 4-3. Procedure for determining the volumetric k value (an estimate of the modulus of subgrade reaction)

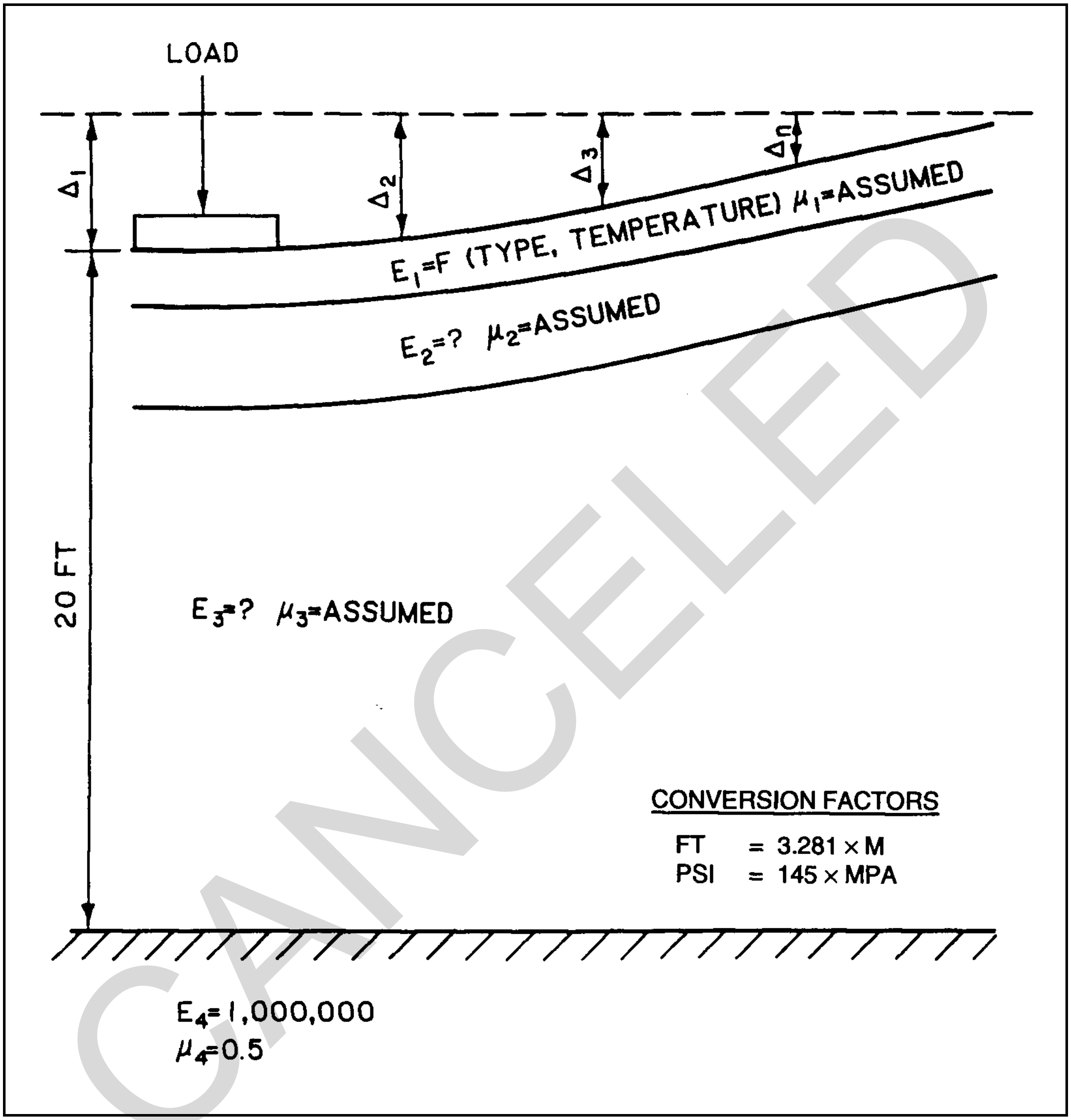


Figure 4-4. Illustration of a layered pavement structure

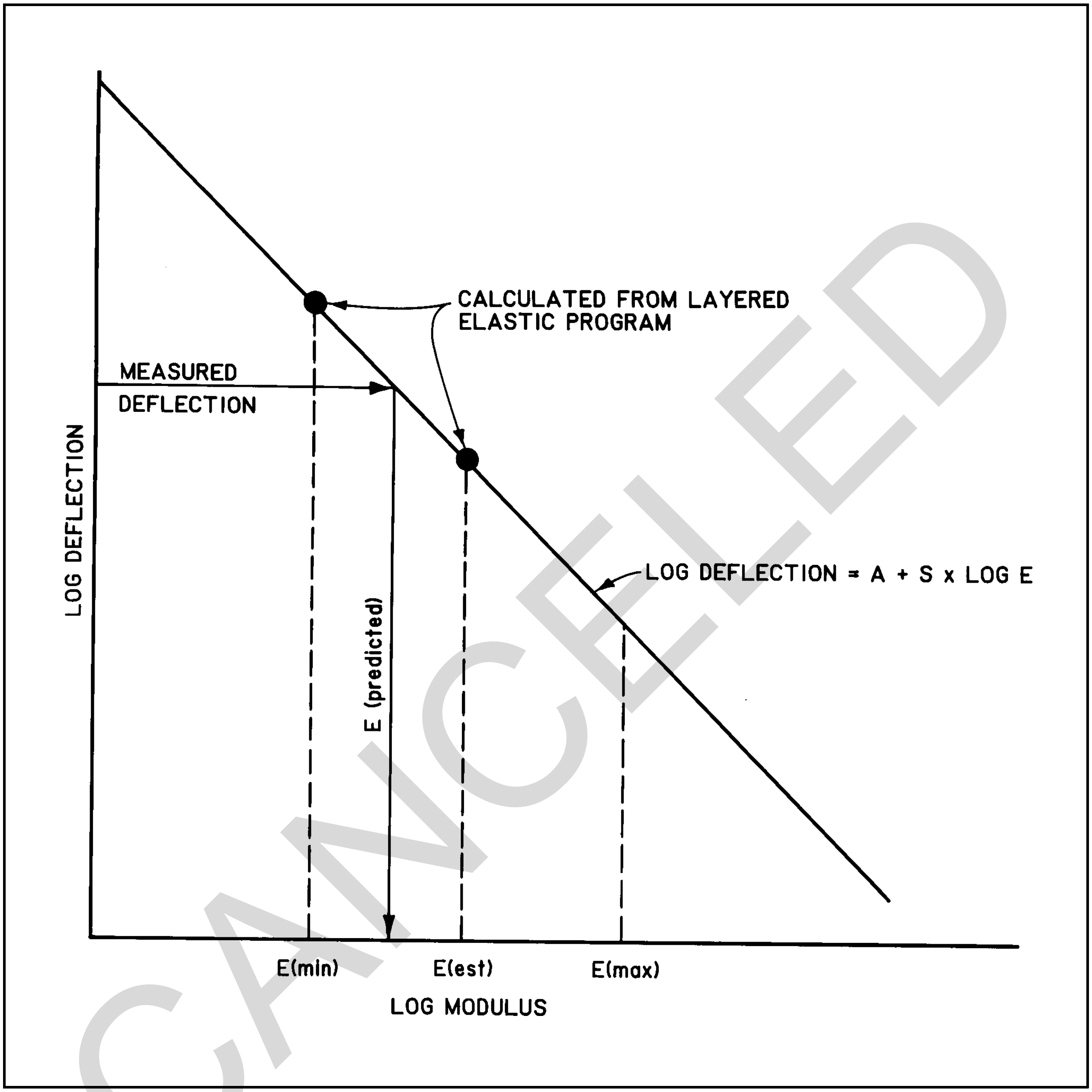


Figure 4-5. Simplified description of how deflection basins are matched in WESDEF (one deflection and one layer)

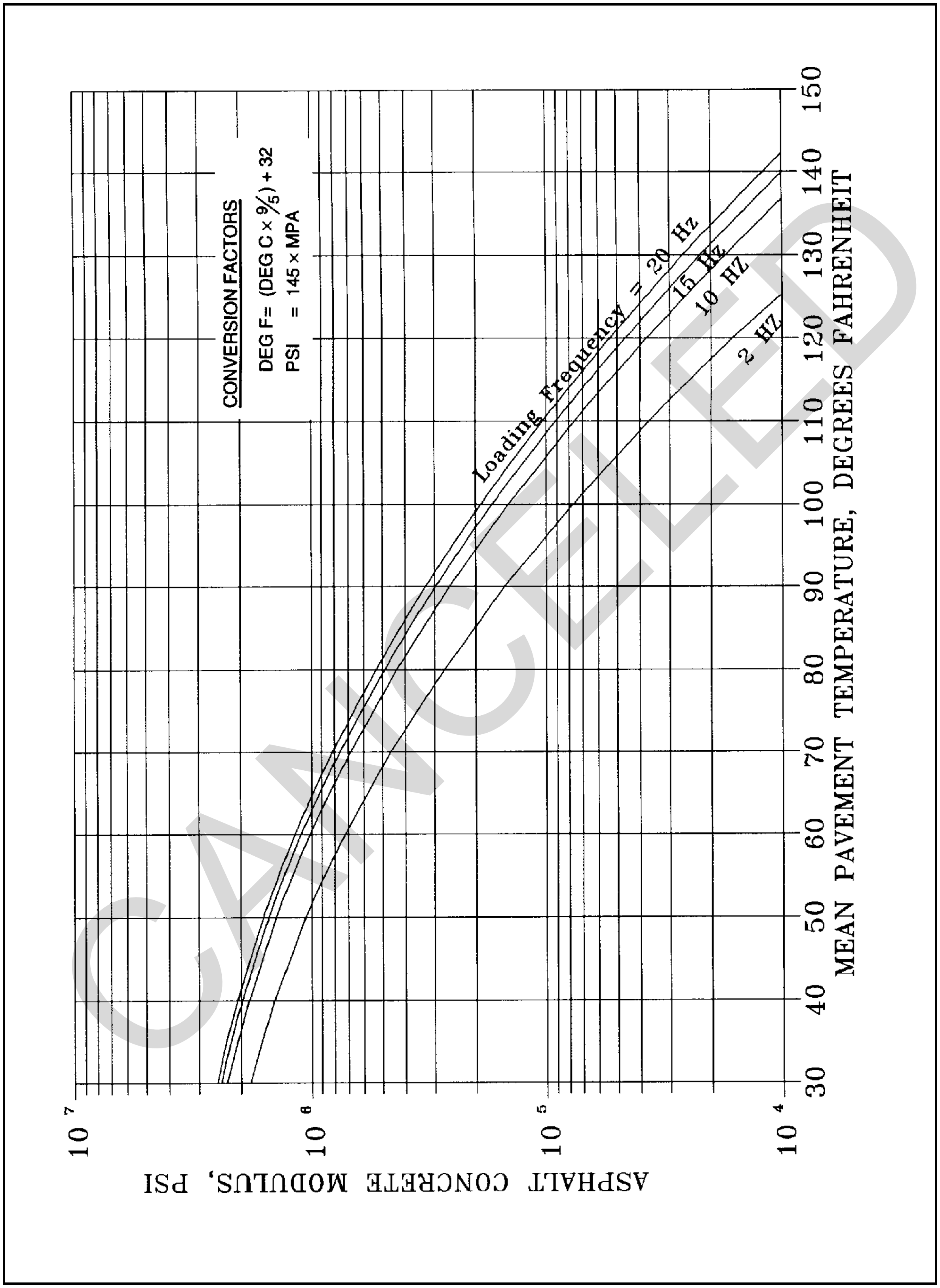


Figure 4-6. Prediction of AC modulus for bituminous layers

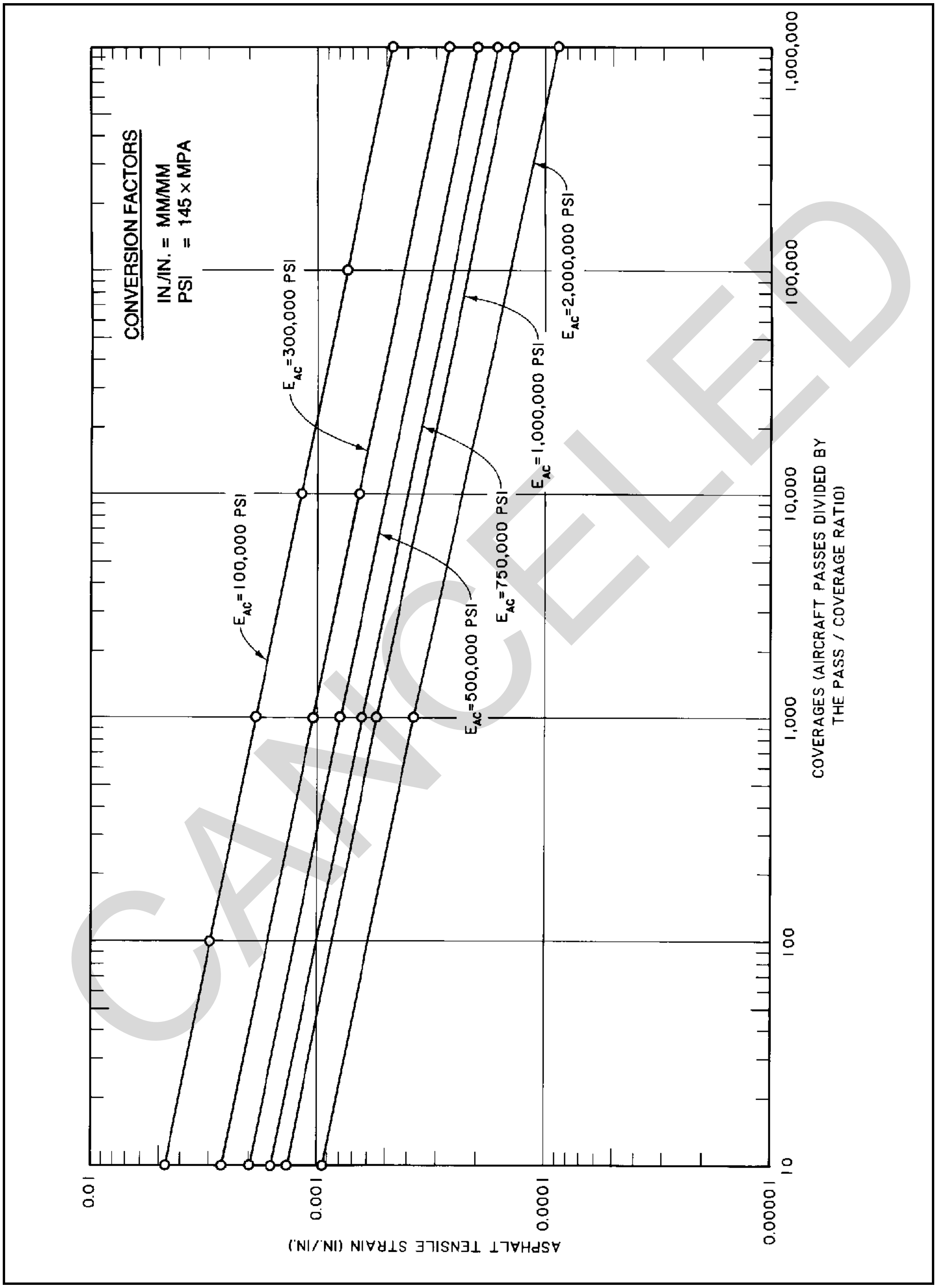


Figure 4-7. Limiting horizontal tensile strain criteria for an AC layer in a flexible pavement evaluation

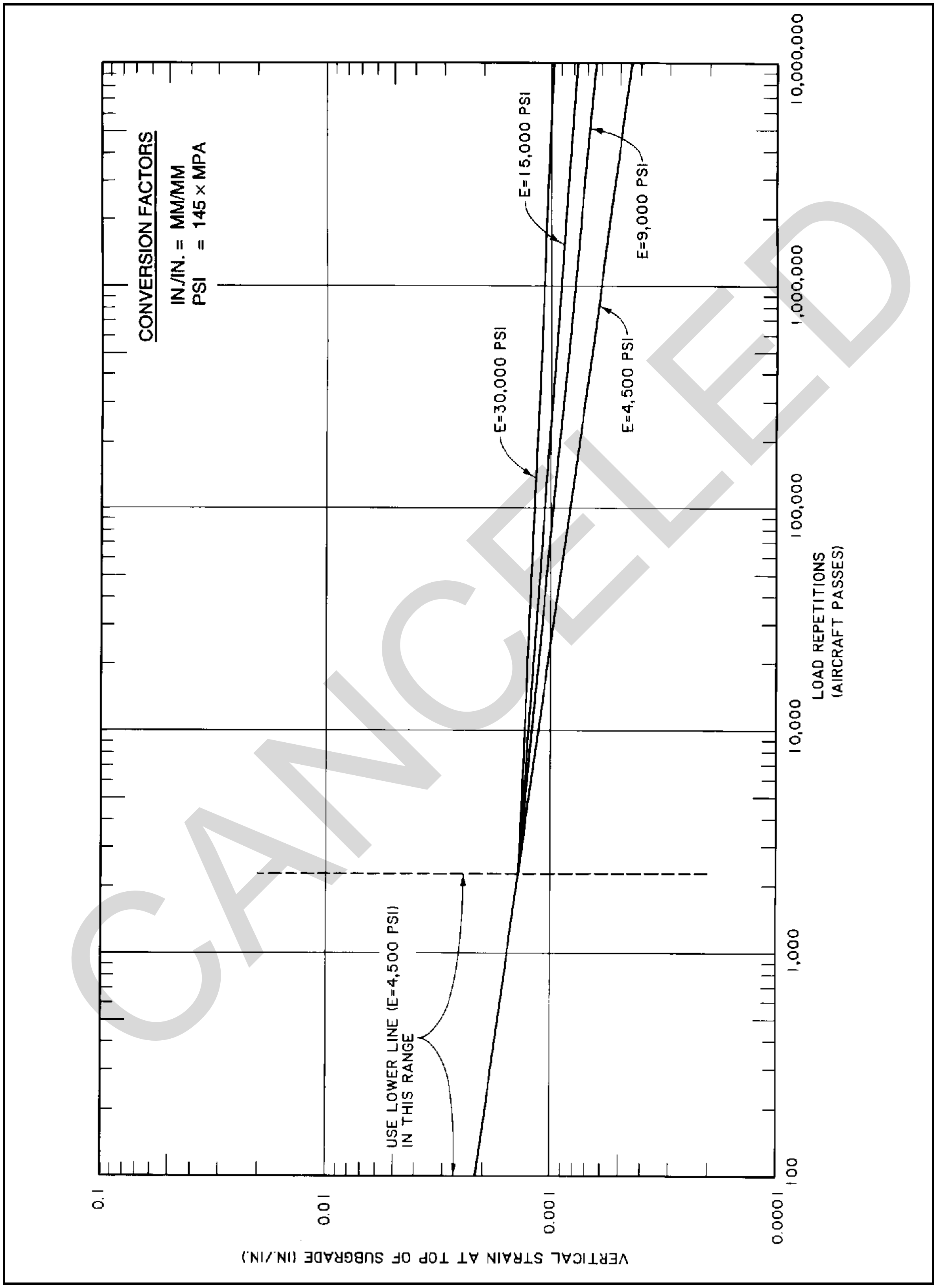


Figure 4-8. Limiting vertical subgrade strain criteria for flexible pavement evaluation

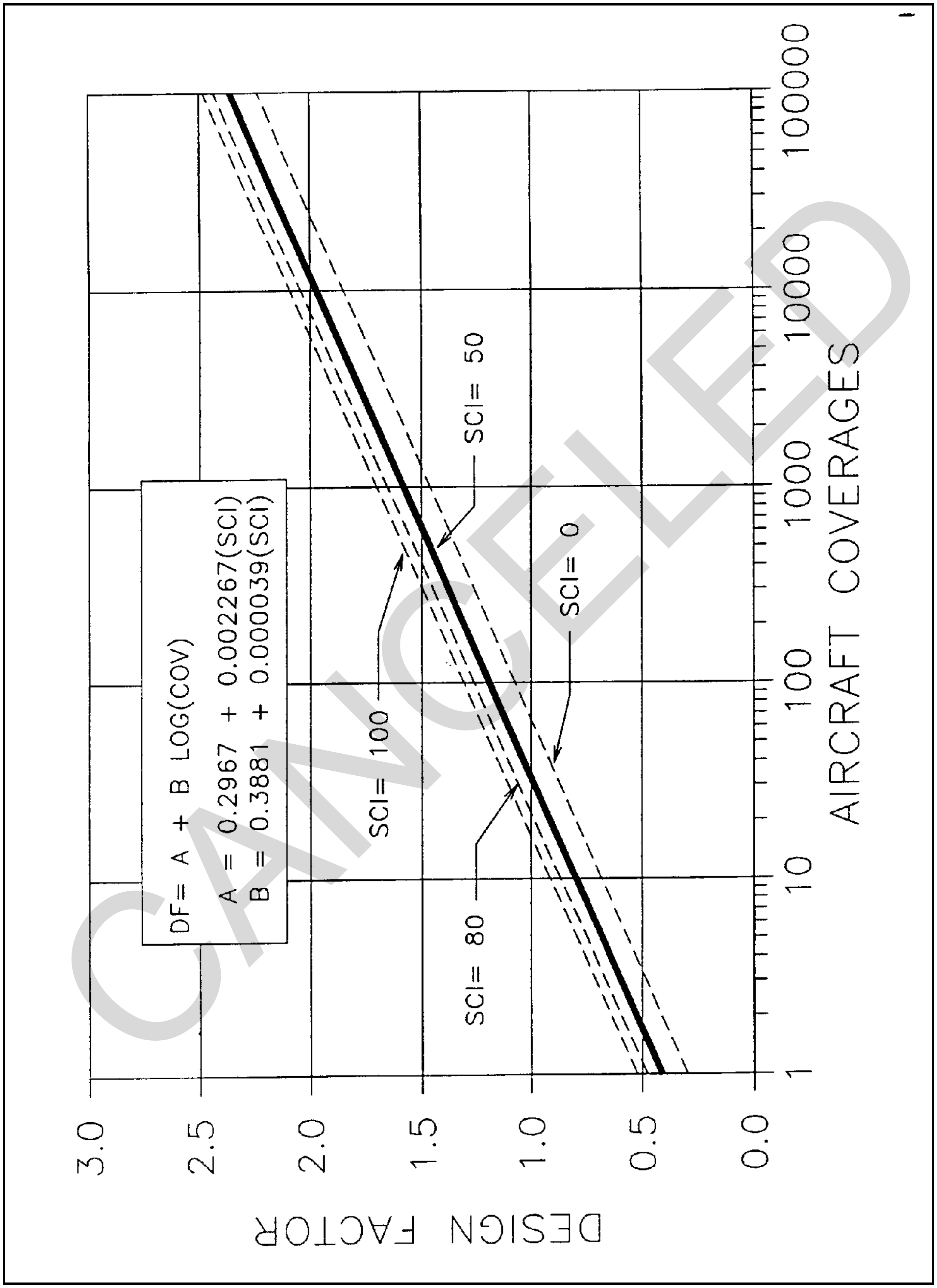


Figure 4-9. Design factor versus coverages for rigid pavement

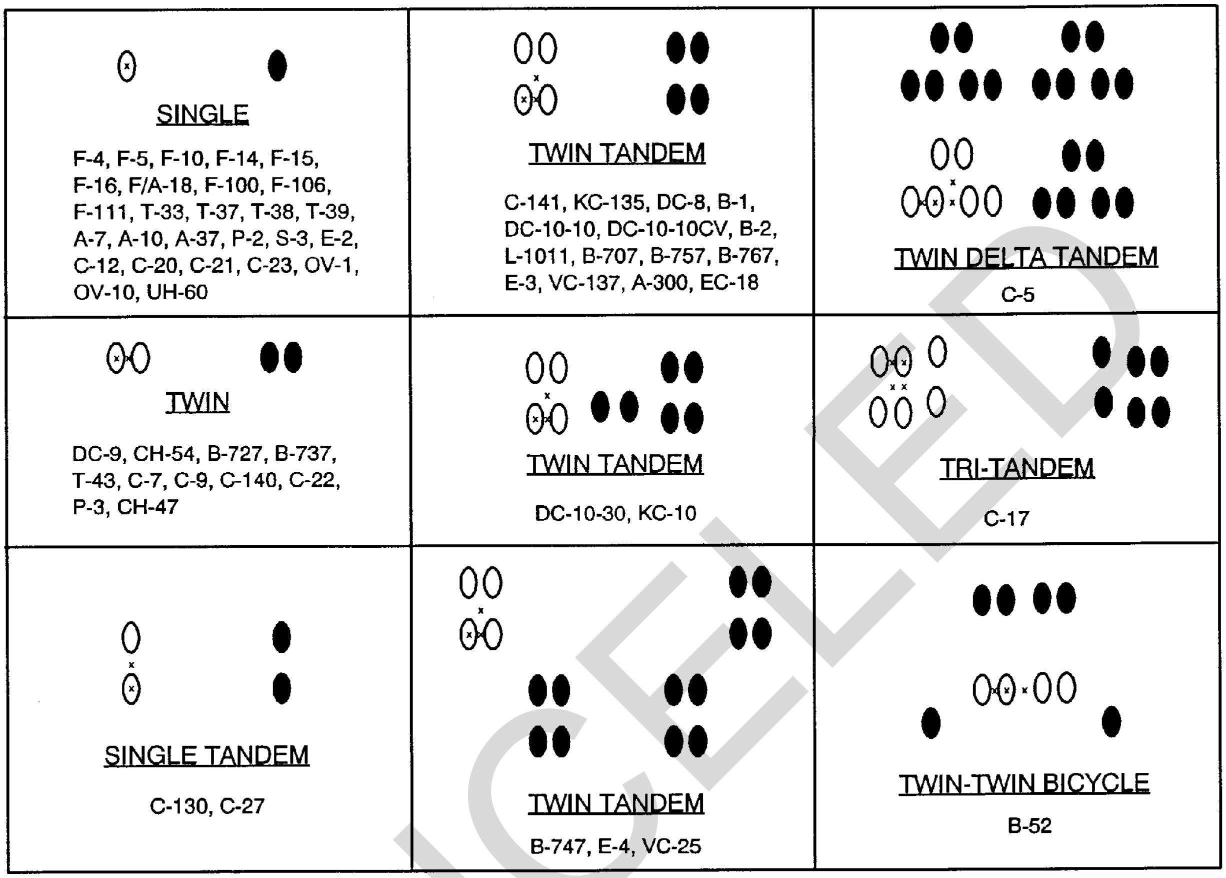


Figure 4-10. Gear configurations for several typical aircraft with controlling wheels and recommended evaluation positions indicated

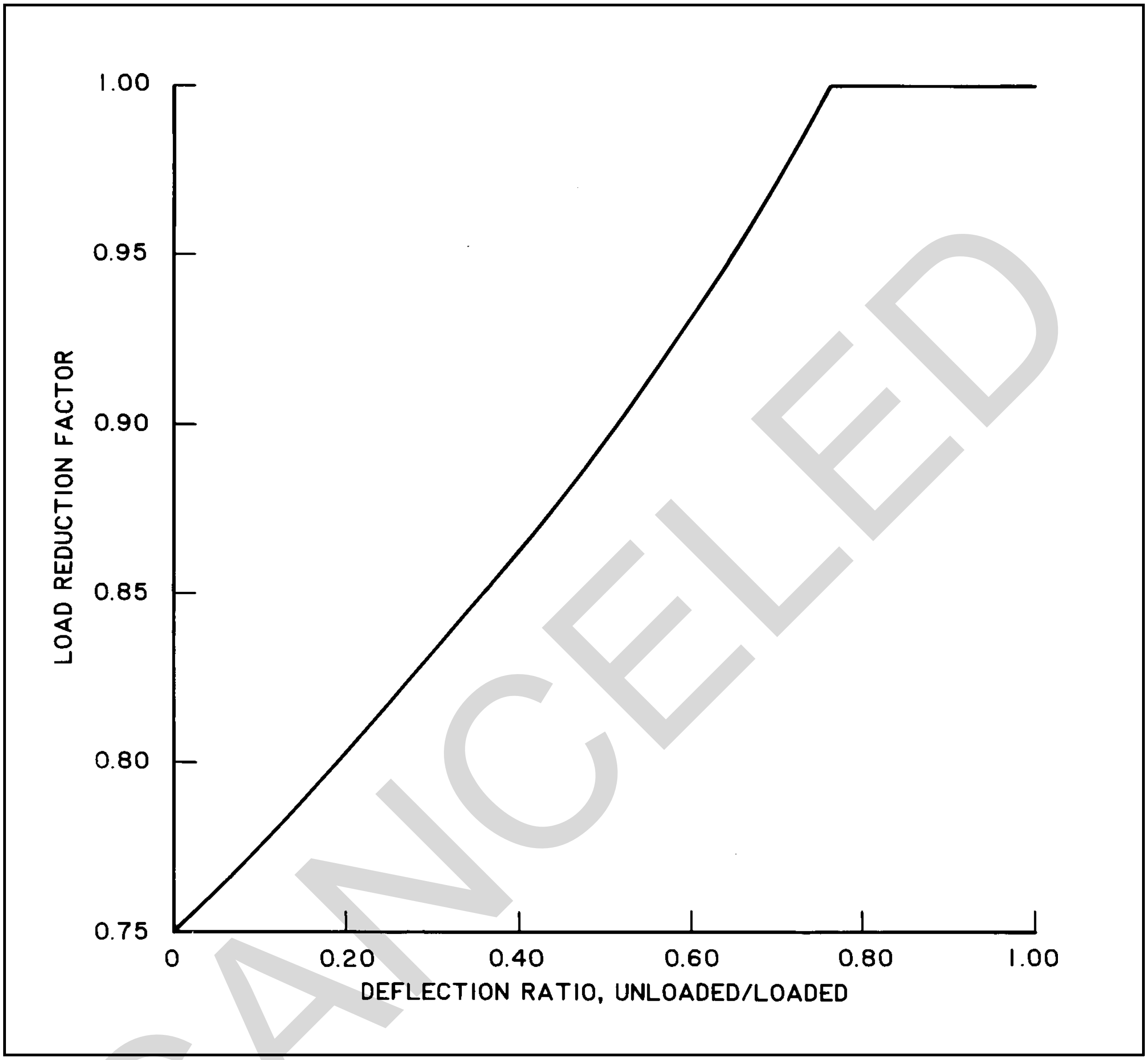


Figure 4-11. Load reduction factors for load-transfer analyses

PAVEMENT EVALUATION SUMMARY

DESIGN AIRCRAFT	DESIGN LOAD, KIPS	DESIGN PASS LEVEL	ALLOW-ABLE LOAD, KIPS	ALLOWABLE PASSES OF DESIGN AIRCRAFT	REQ'D OVERLAY, IN. AC	REQ'D OVERLAY, IN. PCC	REQ'D OVERLAY, IN. PCC, NO BOND	PCN
********************	******	********	******	*********	****	****	****	***
C-141	350.0	50000.	294.3	5921.	4.9	8.5	11.1	54

THIS PAVEMENT IS TO BE EVALUATED FOR 50,000 PASSES OF A 350-KIP C-141 AIRCRAFT

MAXIMUM C-141 GROSS LOAD THAT CAN BE SUSTAINED BY THIS PAVEMENT FOR THE DESIGN NUMBER OF PASSES (50,000)

NUMBER OF PASSES THAT THIS PAVEMENT CAN SUSTAIN THE C-141 AT THE DESIGN GROSS LOAD (350 KIPS)

AMOUNT OF OVERLAY REQUIRED TO INCREASE THE STRUCTURAL CAPACITY OF THIS PAVEMENT SUCH THAT IT CAN SUSTAIN THE DESIGN AIRCRAFT (C-141) AT THE DESIGN GROSS LOAD (350 KIPS) FOR THE DESIGN NUMBER OF PASSES (50,000)

FROM TIME TO TIME, AIRCRAFT NOT INCLUDED IN THE TRAFFIC MIX UPON WHICH THE EVALUATION WAS BASED MAY REQUEST PERMISSION TO OPERATE ON THE AIRFIELD. FOR OCCASIONAL USE OF THE AIRFIELD BY THESE "OTHER" AIRCRAFT, THE DECISION REGARDING SUCH A REQUEST WILL BE EVALUATED USING THE PCN. IF THE ACN OF THE AIRCRAFT IS LESS THAN OR EQUAL TO THE PCN, THE PAVEMENT CAN SUPPORT AN OCCASIONAL OPERATION OF THE AIRCRAFT WITHOUT WEIGHT RESTRICTION

NOTE: IN. = 0.0394 × MM
KIPS = 0.0022 × KG

Figure 4-12. Typical WESPAV output

CHAPTER 5

EVALUATION OF FLEXIBLE PAVEMENT USING DIRECT SAMPLING

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. Computer programs are also available for pavement evaluation and are discussed in chapter 9.

2. FACTORS LIMITING LOAD-CARRYING CAPACITY. The load-carrying capacity of a flexible pavement is limited by the strength of its weakest component, either the bituminous pavement, 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. 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 where pertinent.

3. SELECTION OF THICKNESS VALUES. 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 be modified for use with the evaluation curves when the measured thickness exceeds the required minimum thickness. Minimum thickness requirements are contained in TI 825-01/ AFM 32-1124 (I) NAVFAC DM 21.10. 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 and their use are presented in paragraph titled "Evaluation of Stabilized Layers."

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, ECP tests described in ASTM D 3441, or DCP tests described in FM 5-430-00-2 (AFJPAM 32-8013, Vol 2). 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 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. Consider a subgrade material beneath a facility being evaluated that varies in such a manner that the facility may be divided into several rather large areas of different subgrade materials. The in-place CBR values for the entire facility, arranged in ascending 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 area covered by the facility 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 facility is not good and that the values logically fall into four groups. Each group represents one of the areas of different material; thus, the most critical area is that represented by the range of values from 7 to 10, and the evaluation should be based on this area. Since the range is narrow, a formal statistical analysis is not necessary, and a visual inspection of the figures indicates that a value of 8 or 9 should be selected.

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 below.

Type of base-course material	**Evaluation CBR**
Graded crushed aggregate (100 CBR)	100
Water-bound macadam	100
Dry-bound macadam	100
Bituminous course, central plant, hot mix	100
Limerock	80
Bituminous macadam	80
Graded crushed aggregate (80 CBR)	80
Soil cement	80
Sand asphalt	80
Sand shell or shell	80
Open-graded (stabilized or unstabilized)	80

h. The above CBR values should be used when the material meets the quality requirements of the guide specifications or construction specifications. When the evaluation tests are made less than 3 years after construction and indicate plasticity index values greater than 5, in-place CBR values should be considered, but no value greater than 50 should be assigned. When tests are made 3 years or more after construction and indicate plasticity index values greater than 5, the in-place values should be used.

i. When evaluation tests on subbase materials are made less than 3 years after construction and the tested materials meet the suggested requirements in TI 825-01/AFM 32-1124 (I)/NAVFAC DM 21.10, the in-place CBR values should be considered, but no value greater than 50 should be assigned. When tests are made 3 years or more after construction, the in-place values should be used. Cases may occur where the CBR tests tend to underrate certain cohesionless, nonplastic materials. If records show adequate performance and service behavior for these materials, judgment may be used for the arbitrary assignment of a CBR value for evaluation.

5. PASS/LOAD RELATIONSHIPS.

a. Evaluation Curves. The evaluation of a flexible pavement with respect to thickness above the subgrade and/or base courses and the selected CBR values can be made for airfields using figures 5-1 to 5-47. These figures may be used to evaluate for specific aircraft and/or Air Force evaluation groups as indicated in the figures. These curves will be entered with the given thickness, CBR, and number of passes and determine the allowable aircraft gross weight that can use the pavement. The range of gross weights shown on the curves bracket the maximum and minimum weights of the aircraft in a particular group or class. Curves are presented for various traffic areas as defined in TI 825-01/AFM 32-1124 (I)/NAVFAC DM 21.10, for Army and Air Force and MIL-HDBK-1021/2 for Navy and Marine Corps.

b. Example Use of Evaluation Curves. Assume an evaluation is desired for 3,000 passes of the C-141 aircraft on a pavement having a subgrade CBR of 4, a 510-millimeter (20-inch) subbase with CBR of 30, a 152-millimeter (6-inch) base with CBR of 100, and a 102-millimeter (4-inch) surface course for a total thickness of 762 millimeters (30 inches). The pavement is an Air Force or Army type A traffic area or a Navy primary traffic area. When making an evaluation, the combinations of CBR and thickness above a given layer must be evaluated for the subgrade, subbase, and base course to determine the weakest combination. Enter figure 5-25 with the thickness of cover over a layer, the CBR of that layer, and the number of passes being evaluated and determine the allowable load for that layer. The layer having the least allowable load controls the evaluation. In this example, the base with a CBR of 100 and a cover layer 102 millimeters (4 inches) has an allowable load of 217,700 kilograms (480 kips), the subbase with a CBR of 30 and a cover of 255 millimeters (10 inches) has an allowable load of 165,560 kilograms (365 kips), and the subgrade with a CBR of 4 and a cover of 760 millimeters (30 inches) has an allowable load of 96,160 kilograms (212 kips). Therefore, the pavement is capable of sustaining 3,000 passes of the C-141 at 96,160 kilograms (212 kips). 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.

c. Thickness Selection. To evaluate an airfield facility, the pavements must be divided into traffic areas as described in TI 825-01/AFM 32-1124 (I)/NAVFAC DM 21.10, for Army and Air Force and MIL-HDBK-1021/2 for Navy and Marine Corps. 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 feature, each thickness should be evaluated but only the controlling evaluation for the facility should be reported.

d. 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 is then converted to an equivalent thickness of subbase material which is then added to the subbase thickness for evaluation. The equivalency factor used by the Army and Air Force for asphalt surfacing is 2.3 and for base course is 2.0. This means that 25 millimeters (1 inch) of asphalt is equal to 58 millimeters (2.3 inches) of subbase, and 25 millimeters (1 inch) of base course is equal to 50 millimeters (2.0 inches) of subbase. To illustrate use of these equivalencies, assume that a pavement has a total thickness of 508 millimeters (20 inches), consisting of 76 millimeters (3 inches) of asphaltic concrete, 203 millimeters (8 inches) of base course, and 228 millimeters (9 inches) of subbase. If the minimum thickness of asphaltic concrete is 76 millimeters (3 inches) and base is 152 millimeters (6 inches), then the existing base course has 50 millimeters (2 inches) of base not needed to meet minimum design requirements. The 50 millimeters (2 inches) is converted to subbase by multiplying the equivalency factor of 2.0 times the excessive thickness of 50 millimeters (2.0 inches) and then added to the subbase thickness. Therefore, the thicknesses to use for evaluation are 76 millimeters (3.0 inches) for asphaltic concrete, 152 millimeters (6.0 inches) for base, and 330 millimeters (13 inches) for subbase. All equivalency factors used by the Army, Navy, Marine Corps, and Air Force are shown in paragraph titled “Evaluations for Stabilized Layers.”

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 TI 825-01/AFM 32-1124 (I)/NAVFAC DM 21.10. 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:

	Field Cores	Recompacted Samples	
		50-blows	**75-blows**
Unit weight (density), kilograms/cubic meter (pounds per cubic foot)	2,308 (144.2)	2,396 (149.7)	2,415 (150.9)
Unit weight, percent of 50-blow laboratory compaction	96	---	---
Unit weight, percent of 75-blow laboratory compaction	95	---	---
Stability, Newtons (pounds)	8,375 (1,883)	13,028 (2,929)	14,571 (3,276)
Flow, in millimeters (1/100 inch)	3.81 (15)	4.06 (16)	4.06 (16)
Voids total mix, percent	8.5	4.5	3.7
Voids filled, percent	57.2	72.1	75.8

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 1.38 MPa (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, but tars are not. 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.

(2) The type of binder in the surface course should be determined and the surface course characteristics evaluated for resistance to jet fuel. The following tabulation will serve as a guide for evaluating the various types of bituminous 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
Tar and rubberized-tar concrete	Dense	All areas other than refueling pit areas

Note: Rubberized-tar concrete is authorized only in maintenance of existing rubberized-tar concrete pavements; it is not allowed in new construction.

c. Ability to Withstand Jet Blast.

(1) Tests have shown that about 149°C (300°F) is the critical temperature for asphaltic concrete and rubberized-tar concrete, while the critical temperature for tar concrete is about 121°C (250°F). 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 149°C (300°F). Maximum temperatures induced in pavement tests simulating maintenance checkups are 157°C (315°F). Rubberized-tar concretes will usually withstand these temperatures. None of the bituminous pavements will resist erosion when afterburners are turned on with the aircraft standing still. 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. Surface courses less than 25 millimeters (1 inch) thick and poorly bonded will be considered unsatisfactory for parking areas, and the 305-meter (1,000-foot) ends of runways will be so reported in the narrative portion of the evaluation report for all aircraft.

7. EFFECTS OF TRAFFIC COMPACTION.

a. Paving Mixes. Traffic tends to densify 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 (TI 825-01/AFM 32-1124 (I)/NAVFAC DM 21.10), the pavement must be considered to be in a critical condition. 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 TI 825-01/AFM 32-1124 (I)/NAVFAC DM 21.10.

(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 5-47 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 5-47, 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 5-47 at a moisture content of 16 percent and a density of 1,649 kilograms/cubic meter (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 1,681 kilograms/cubic meter (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.

(4) 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.

8. PAVEMENT CLASSIFICATION NUMBER. In addition to evaluating airfield pavements for allowable load, using the above procedures, 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. Values of ACN for given aircraft can be obtained from the aircraft manufacturer or from the locations presented in chapter 9. The PCN is presented in chapter 8.

9. EVALUATIONS FOR ARID REGIONS. The danger of saturation beneath flexible pavements is reduced when the annual rainfall is less than 381 millimeters (15 inches), the water table (including perched water table) is at least 4.6 meters (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.

10. EVALUATIONS FOR FROST CONDITIONS. If the existing soil, water, and temperature conditions are conducive to detrimental frost effects in the base-course or subgrade materials, then the pavement evaluation will be based on frost-area soil support indices as given in chapter 7 of this manual.

11. 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 TM 5-822-14/AFJMAN 32-1019. 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 5-1 for the Army and Air Force and from table 5-2 for the Navy and Marine Corps. 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 ECP or 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 determined from a test pit consisted of a 101-millimeter (4-inch) asphaltic concrete, a 203-millimeter (8-inch) bituminous concrete base, and an 203-millimeter (8-inch) cement-stabilized gravelly clay subbase with an unconfined compressive strength of 48 MPa (700 pounds per square inch). From table 5-1, the 203-millimeter (8-inch) bituminous concrete base equivalency factor is 1.15 which would increase the thickness of the stabilized base for evaluation to 233 millimeters (9.2 inches). From table 5-1, the 203-millimeter (8-inch) cement-stabilized subbase equivalency factor is 2.0 which would increase the thickness of the stabilized subbase for evaluation to 406 millimeters (16 inches).

Table 5-1
Equivalency Factors for Army and Air Force Pavements

Material	Equivalency Factors: Base	Equivalency Factors: Subbase
Unbound Crushed Stone	1.00	2.00
Unbound Subbase	-[1]	1.00
Asphalt-Stabilized		
All-Bituminous Concrete	1.15	2.30
GW, GP, GM, GC	1.00	2.00
SW, SP, SM, SC	-[1]	1.50
Cement-Stabilized		
GW, GP, SW, SP	1.15	2.30
GC, GM	1.00	2.00
ML, MH, CL, CH	-[1]	1.70
SC, SM	-[1]	1.50
Lime-Stabilized		
ML, MH, CL, CH	-[1]	1.00
SC, SM, GC, GM	-[1]	1.10
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.

Table 5-2
Equivalency Factors for Navy and Marine Corps Pavements

Material	Equivalency Factors
25 millimeters (1 inch) of lime-stabilized subbase may be substituted for	30 millimeters (1.2 inches) of unstabilized subbase course
25 millimeters (1 inch) of cement-stabilized subbase may be substituted for	30 millimeters (1.2 inches) of unstabilized subbase course
25 millimeters (1 inch) of cement-stabilized base may be substituted for	38 millimeters (1.5 inches) of unstabilized base course
25 millimeters (1 inch) of bituminous base may be substituted for	38 millimeters (1.5 inches) of unstabilized base course

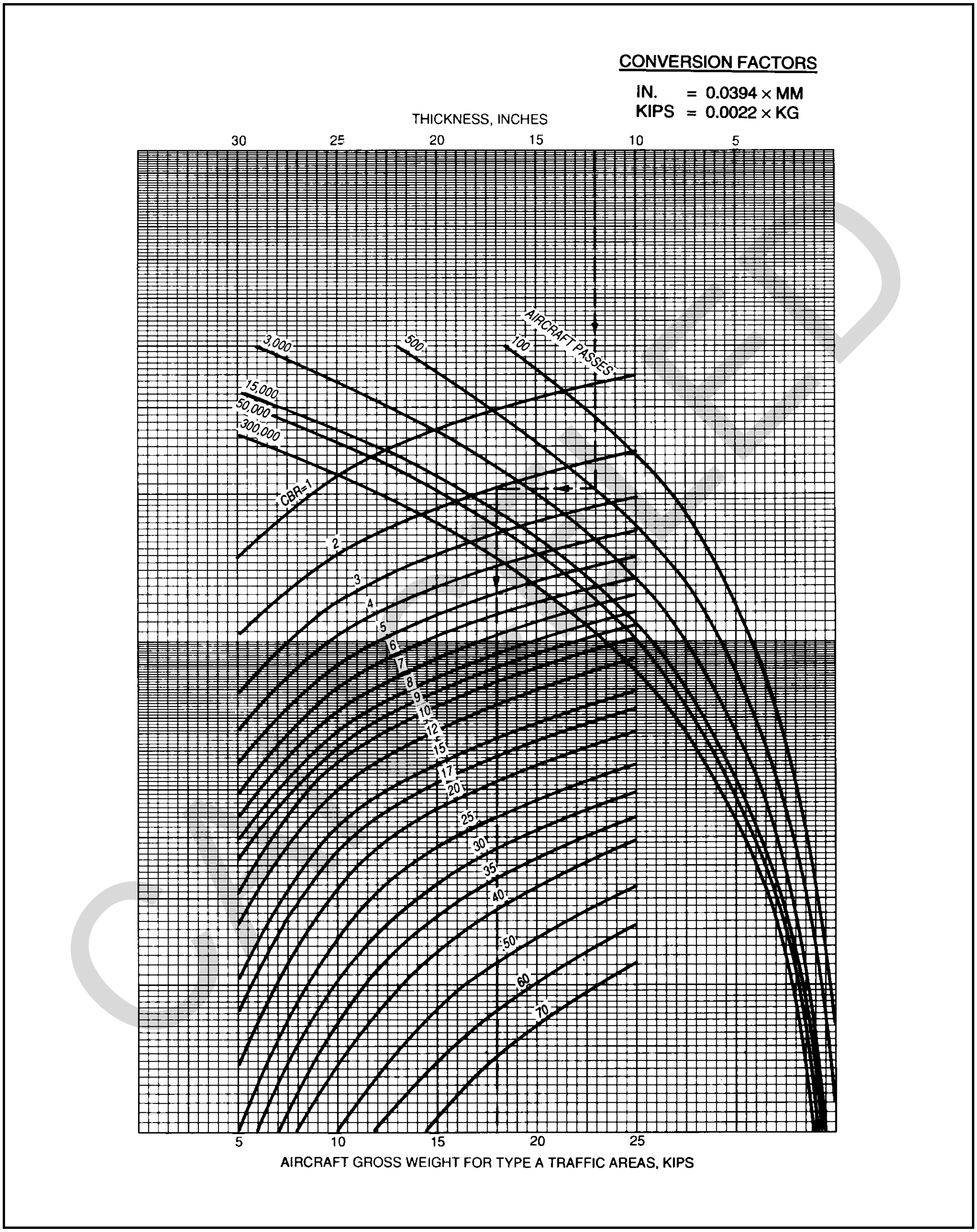


Figure 5-1. Flexible pavement evaluation curve for UH-60, Type A traffic area

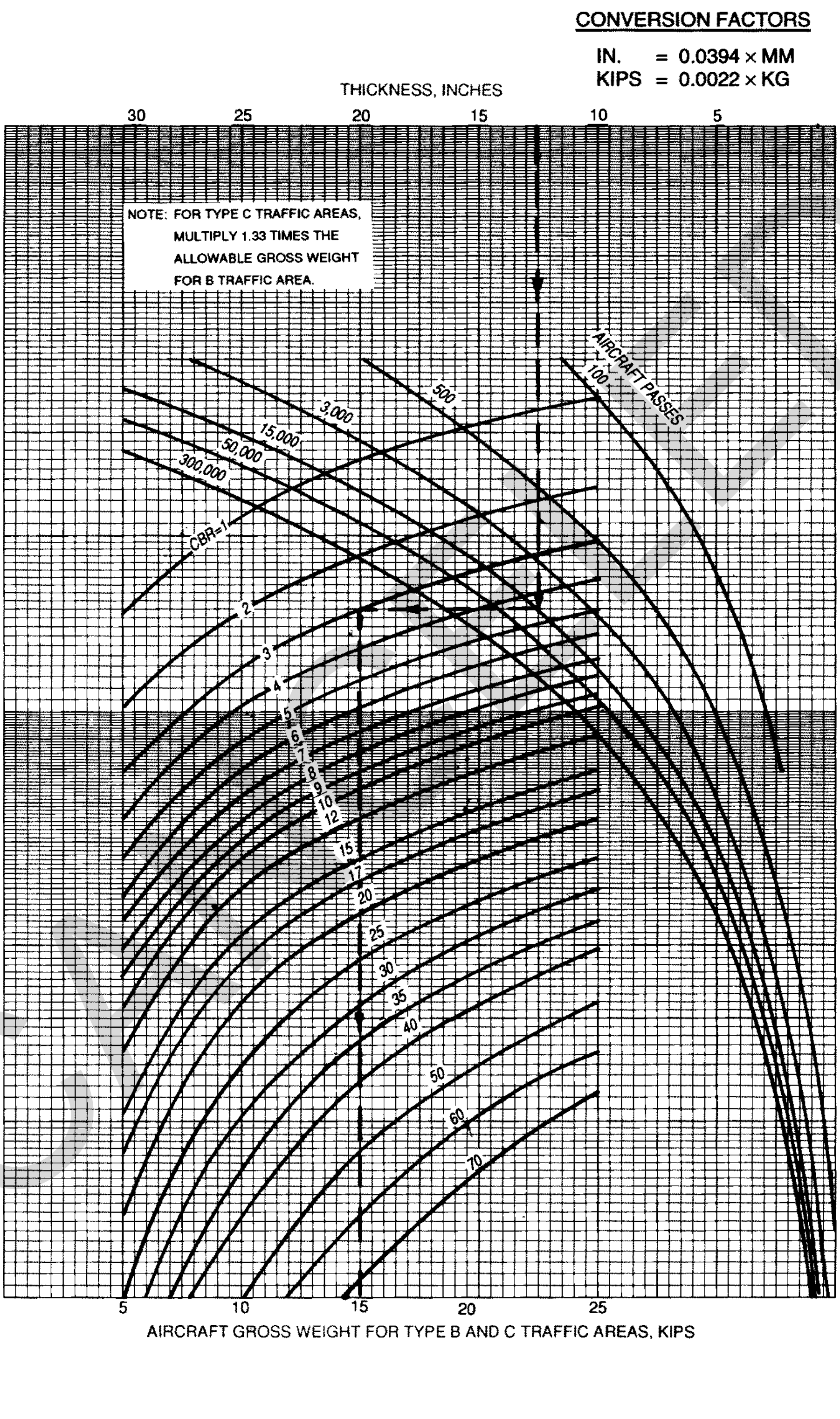


Figure 5-2. Flexible pavement evaluation curve for UH-60, Types B and C traffic areas

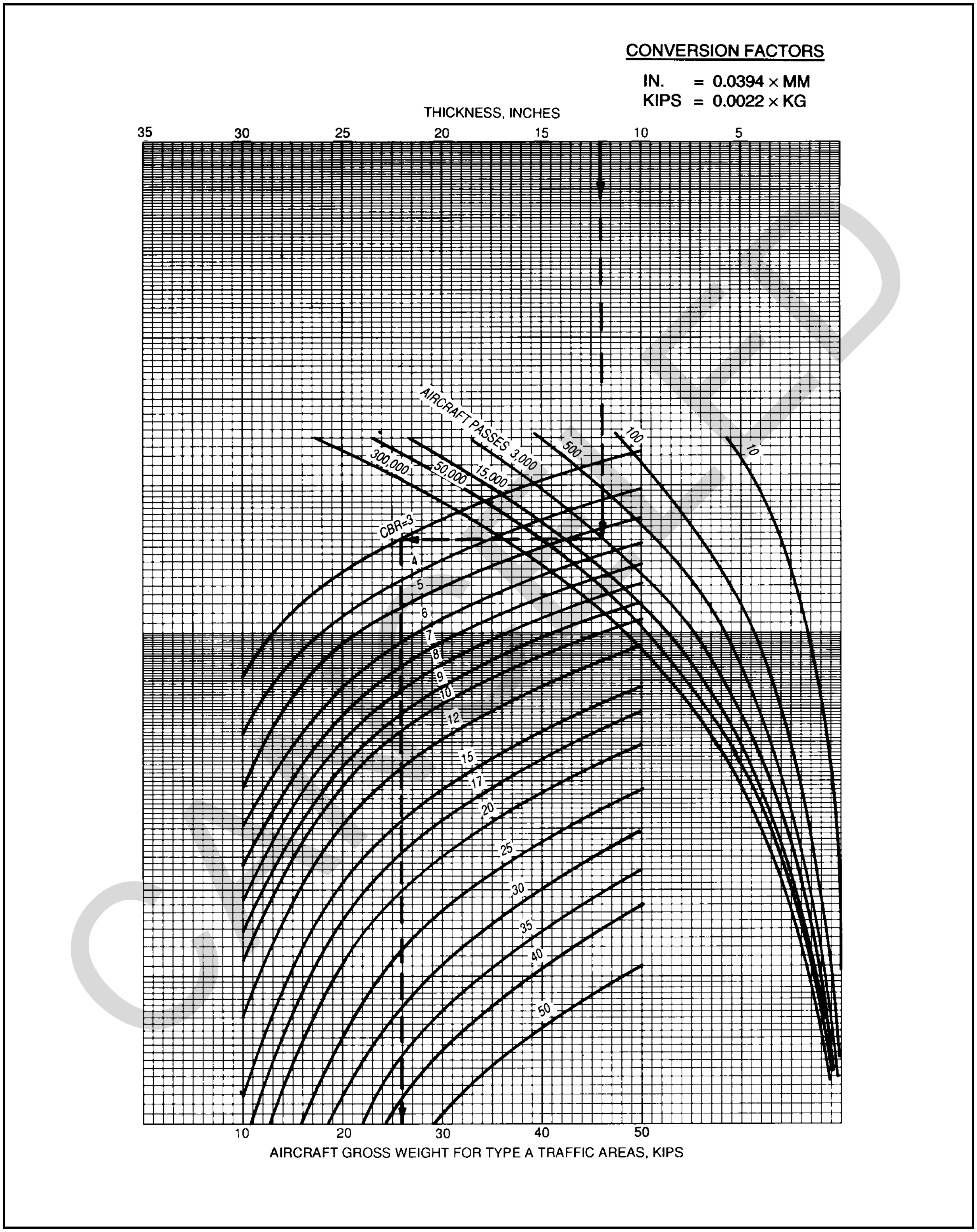


Figure 5-3. Flexible pavement evaluation curves for CH-47, Type A traffic area

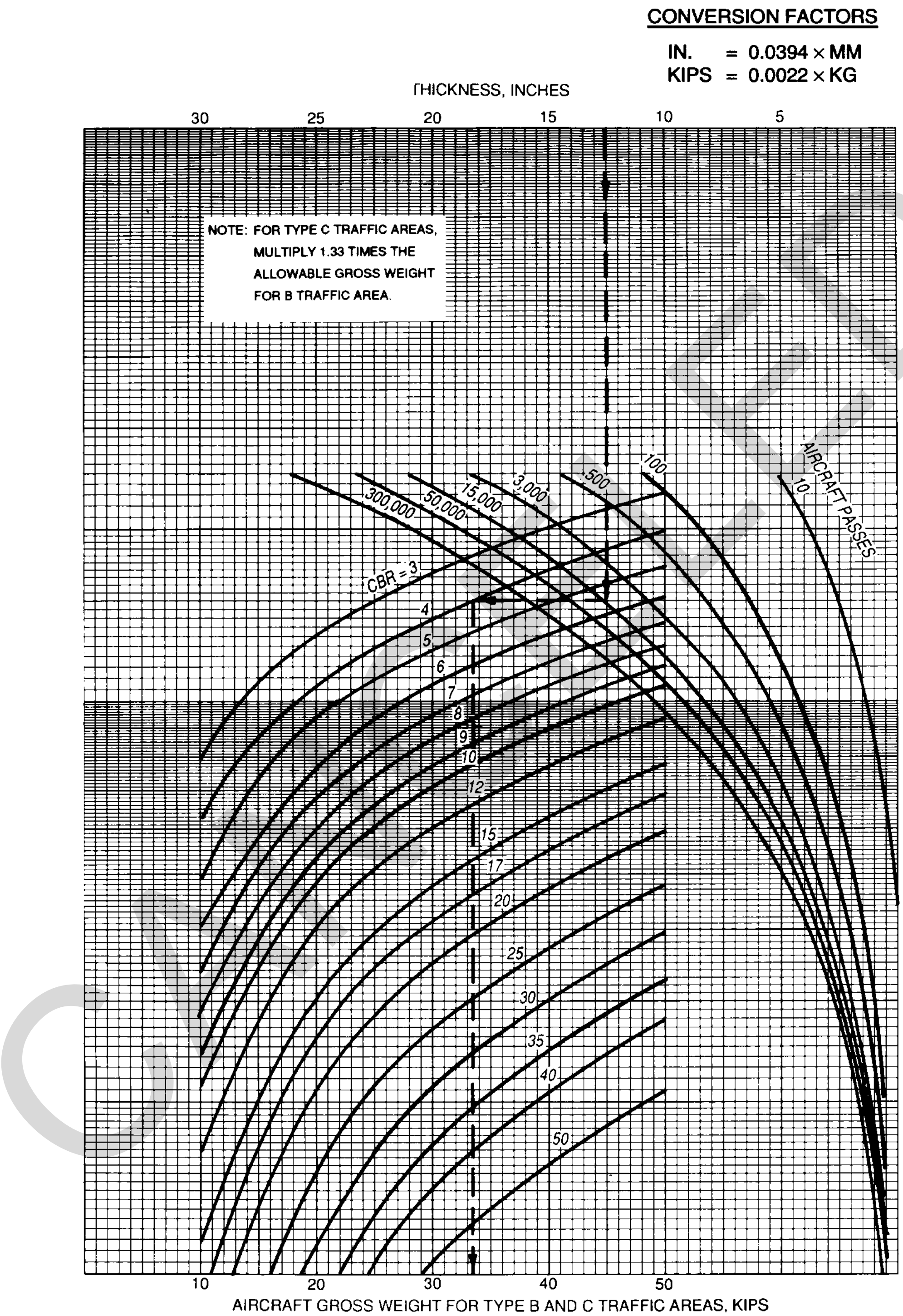


Figure 5-4. Flexible pavement evaluation curve for CH-47, Types B and C traffic areas

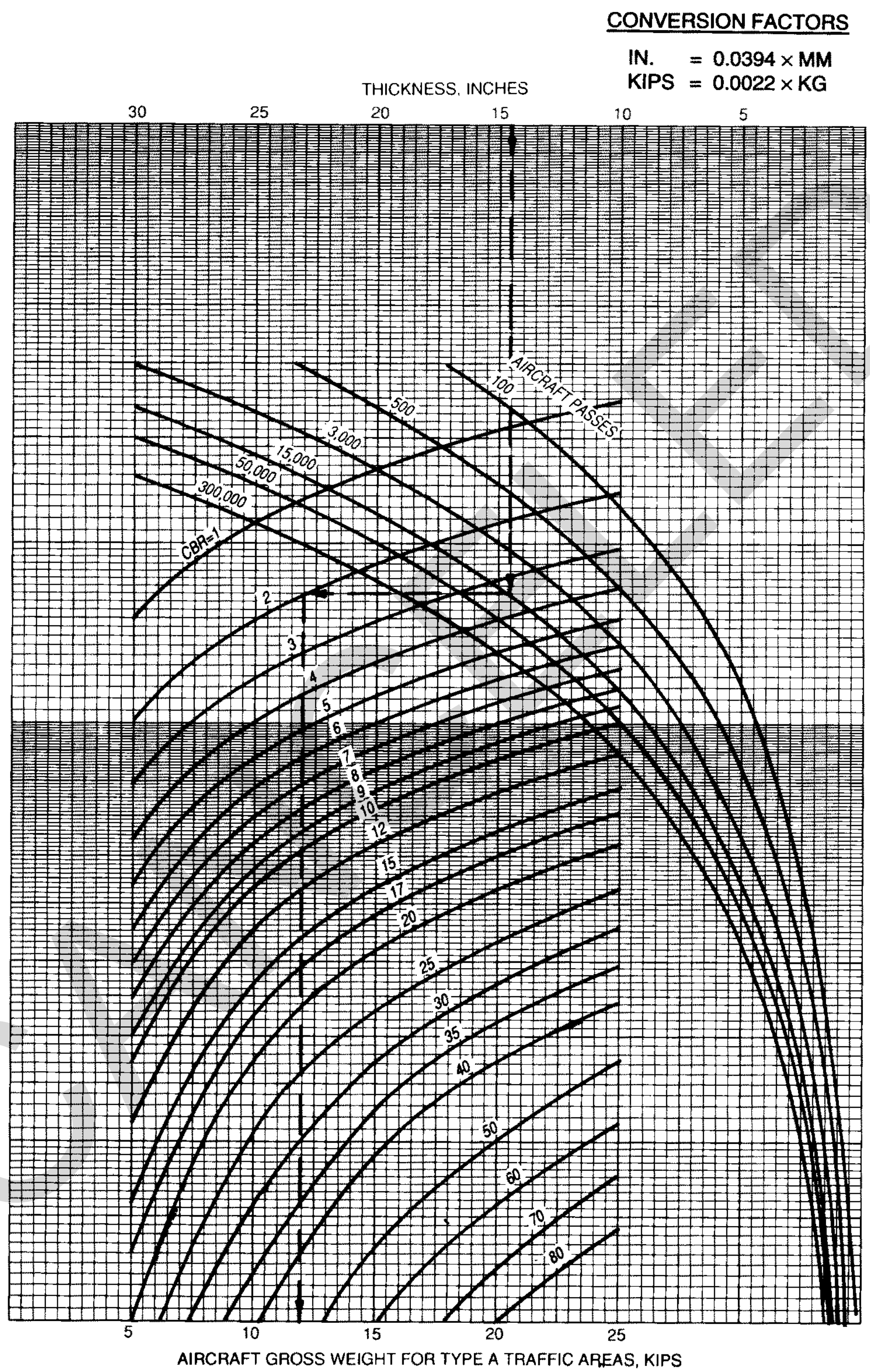


Figure 5-5. Flexible pavement evaluation curve for OV-1, Type A traffic area

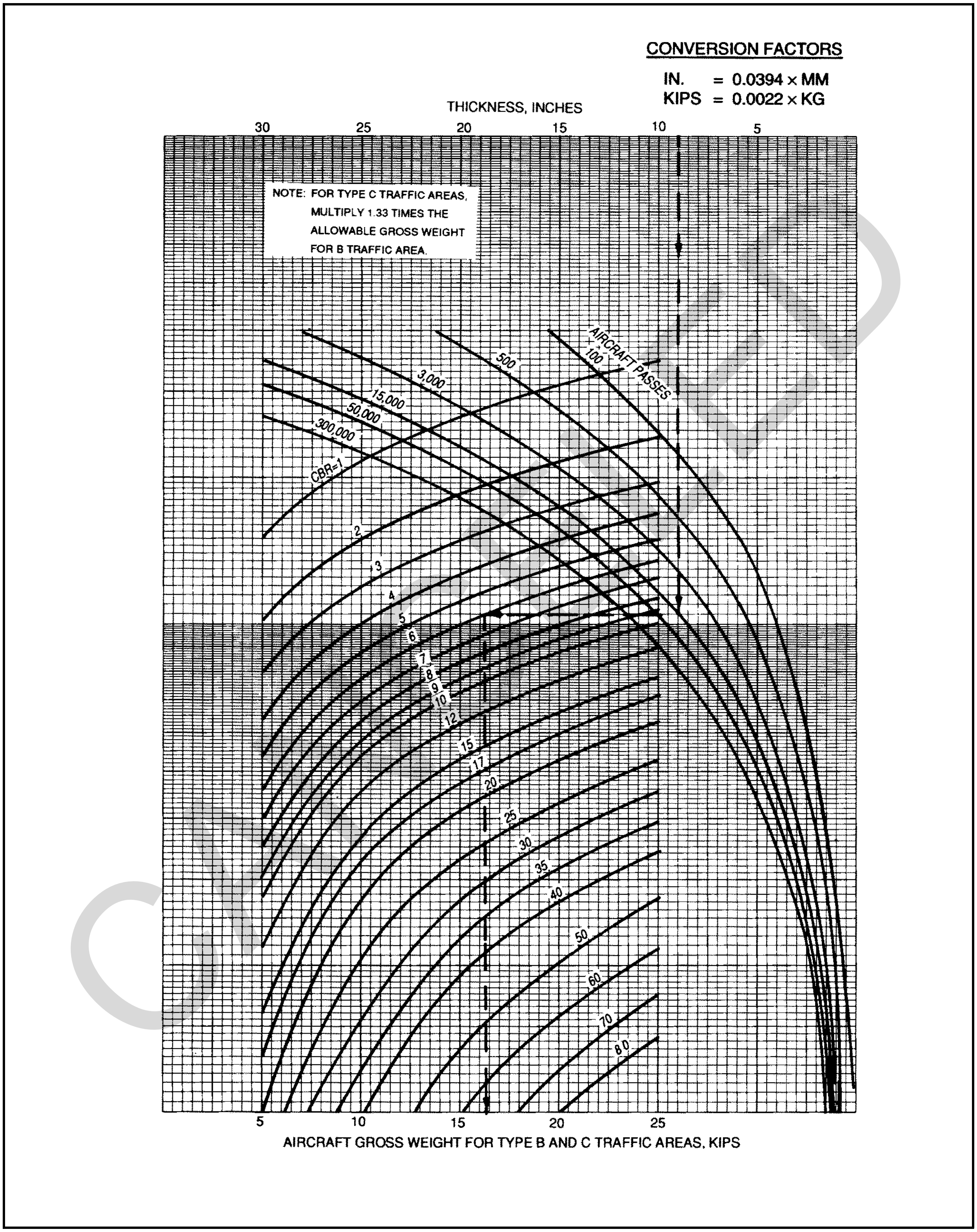


Figure 5-6. Flexible pavement evaluation curve for OV-1, Types B and C traffic areas

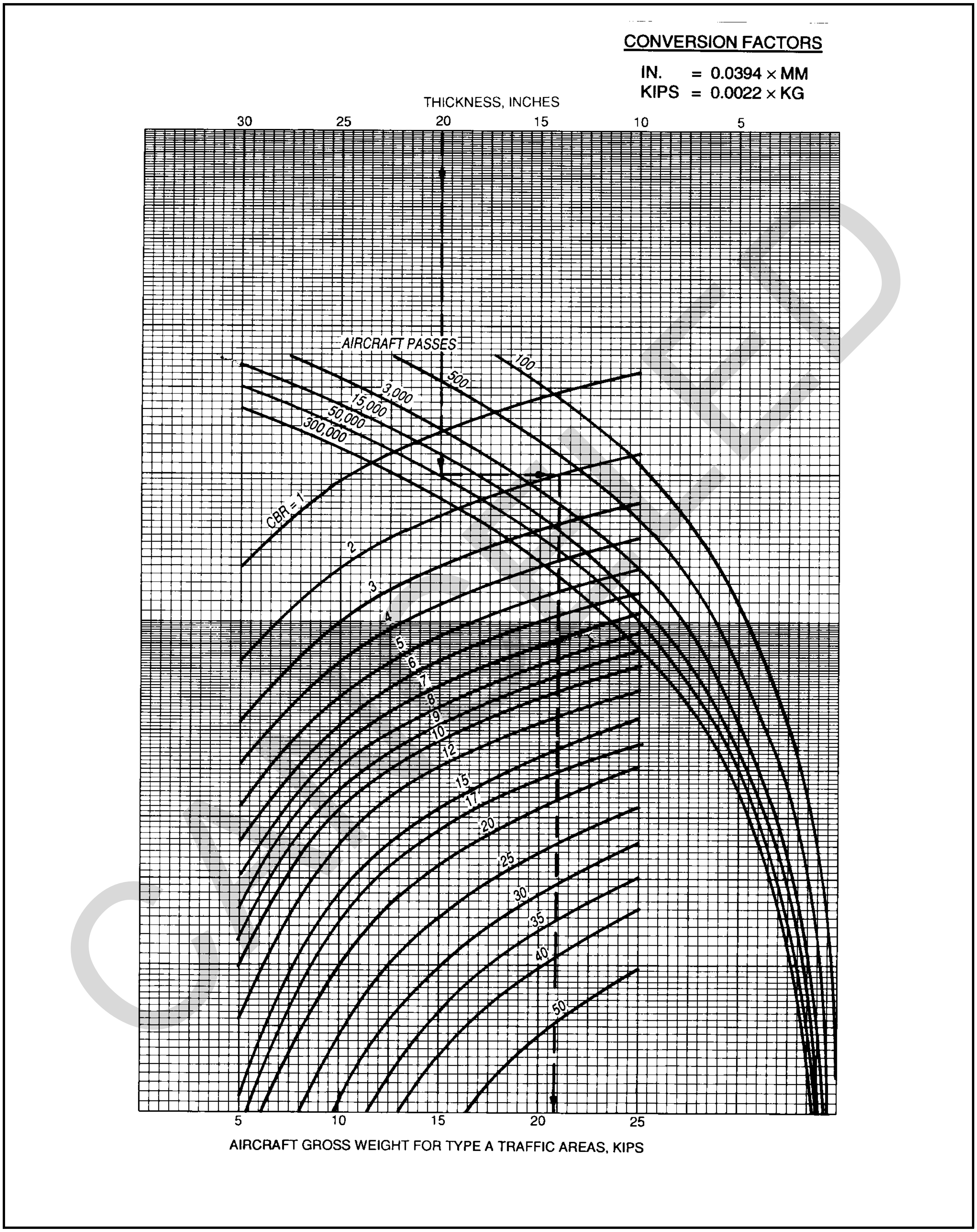


Figure 5-7. Flexible pavement evaluation curve for C-12, Type A traffic area

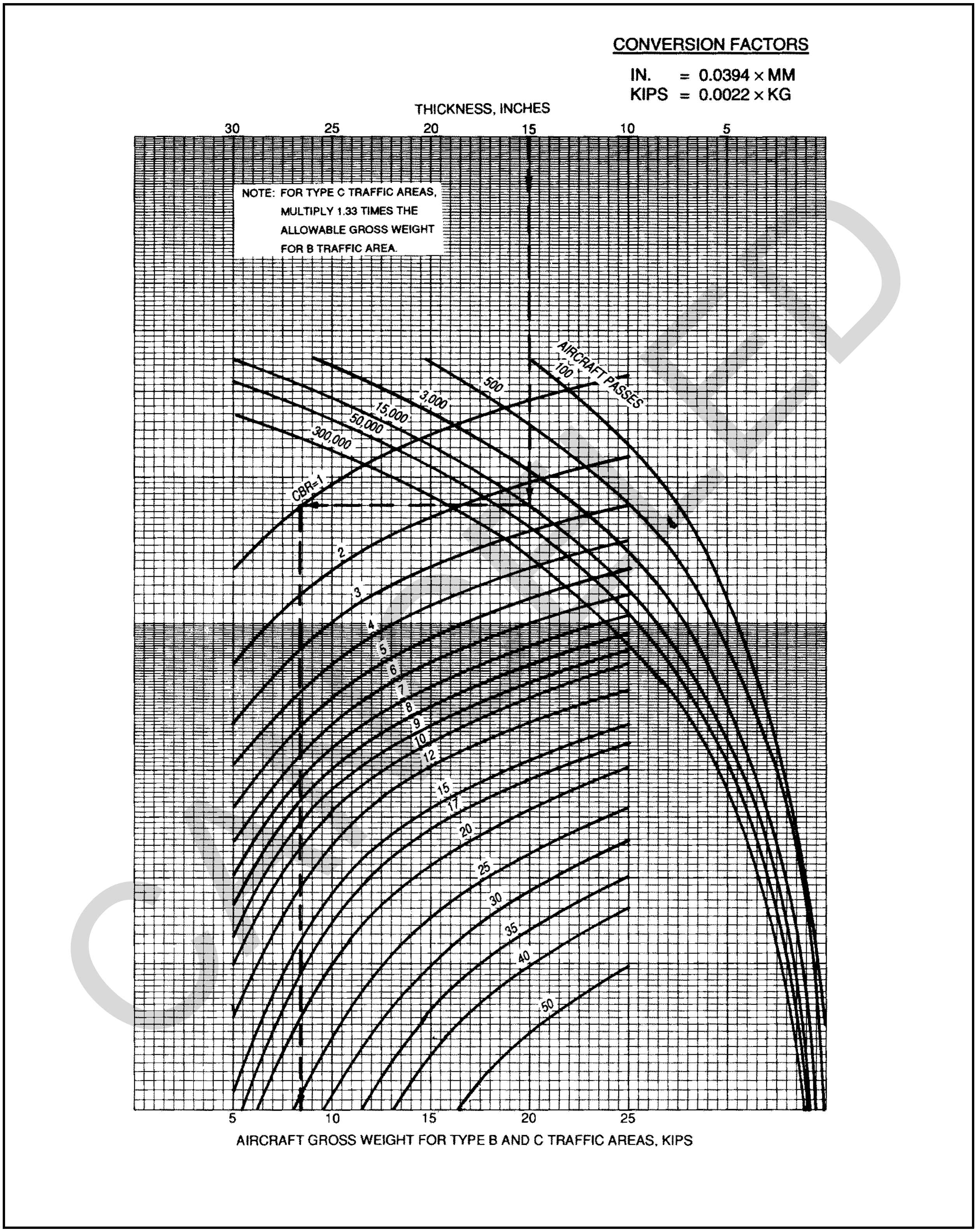


Figure 5-8. Flexible pavement evaluation curve for C-12, Types B and C traffic areas

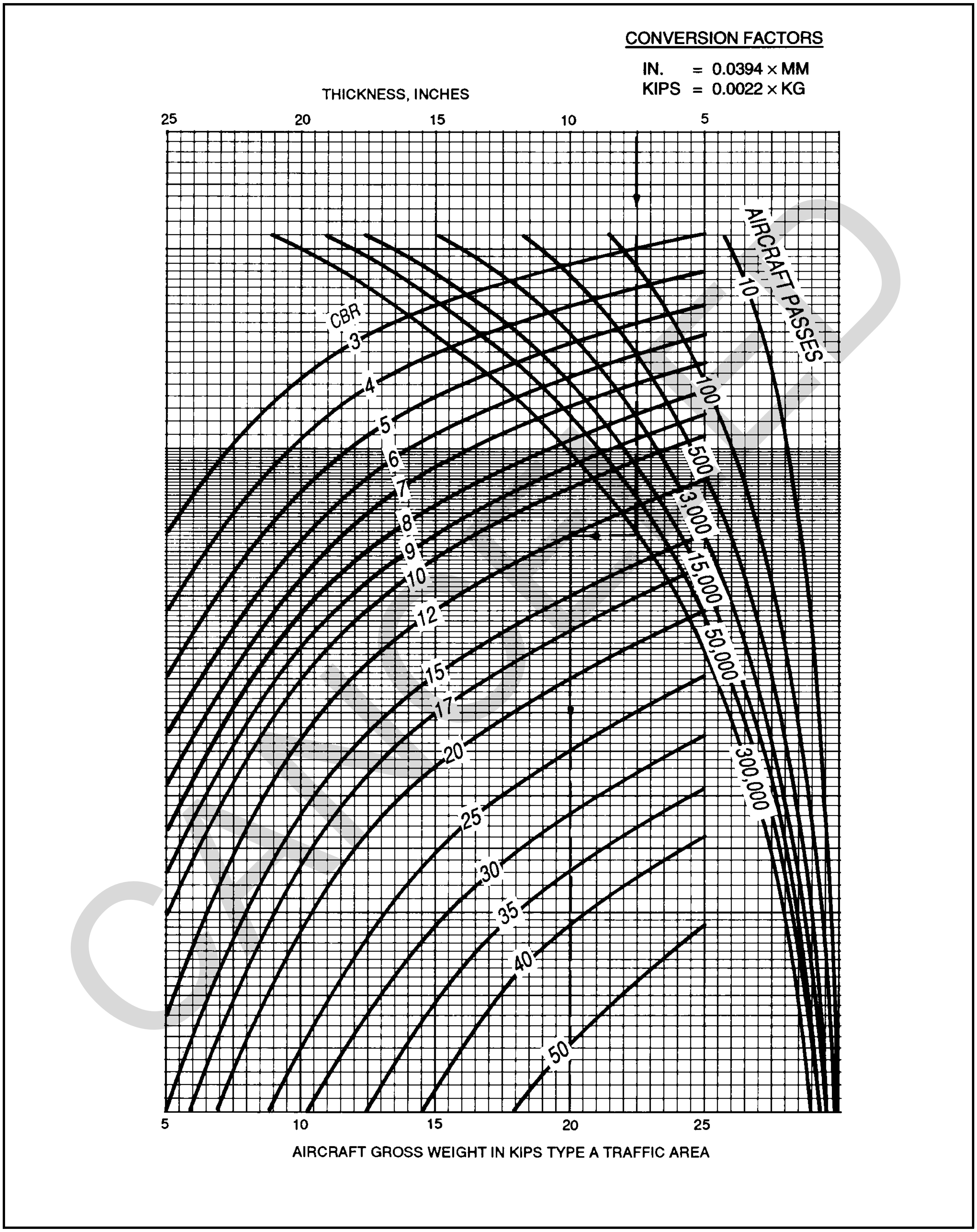


Figure 5-9. Flexible pavement evaluation curve for C-23 (Air Force Group Index 1), Type A traffic area

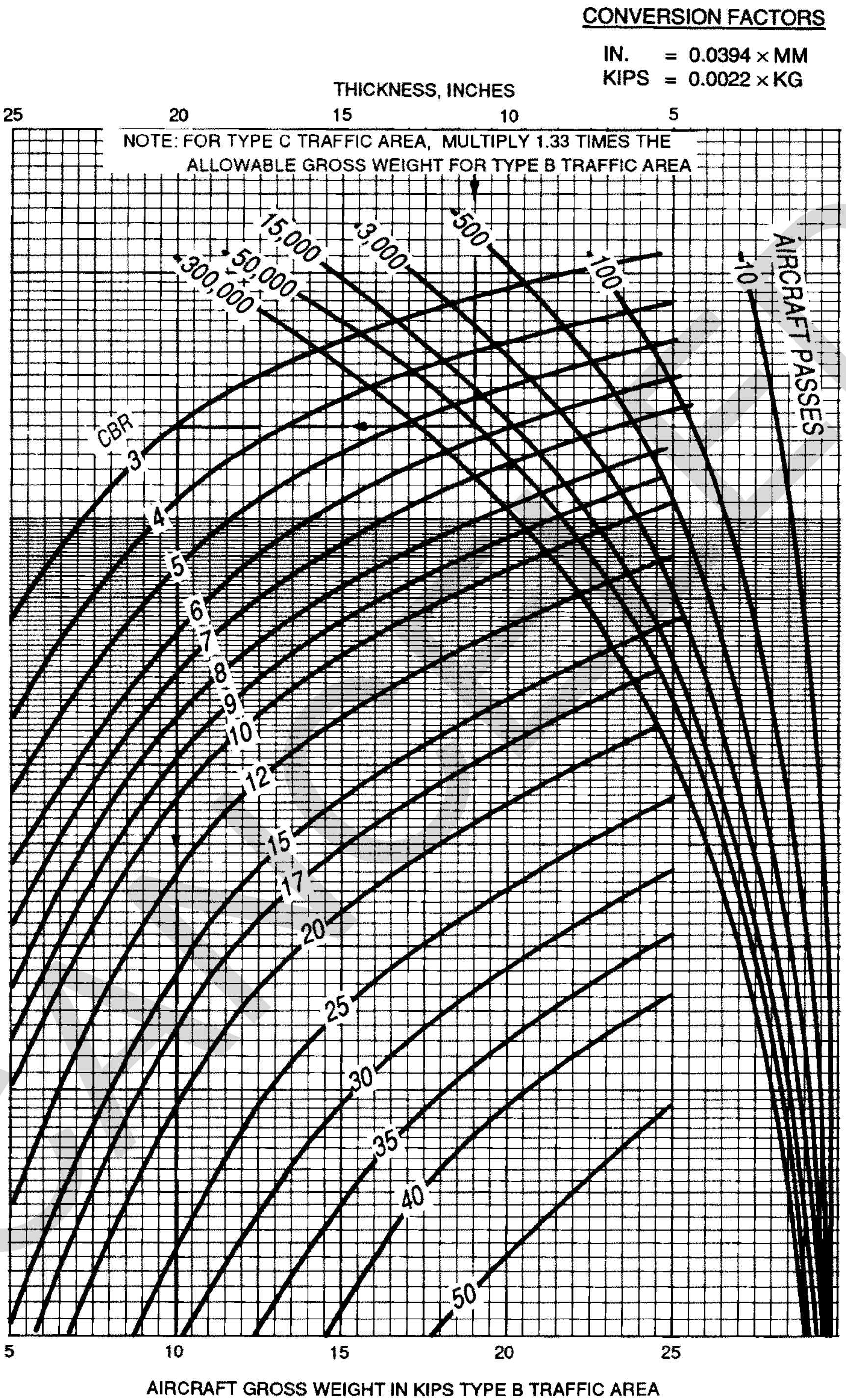


Figure 5-10. Flexible pavement evaluation curve for C-23 (Air Force Group Index 1), Types B and C traffic areas

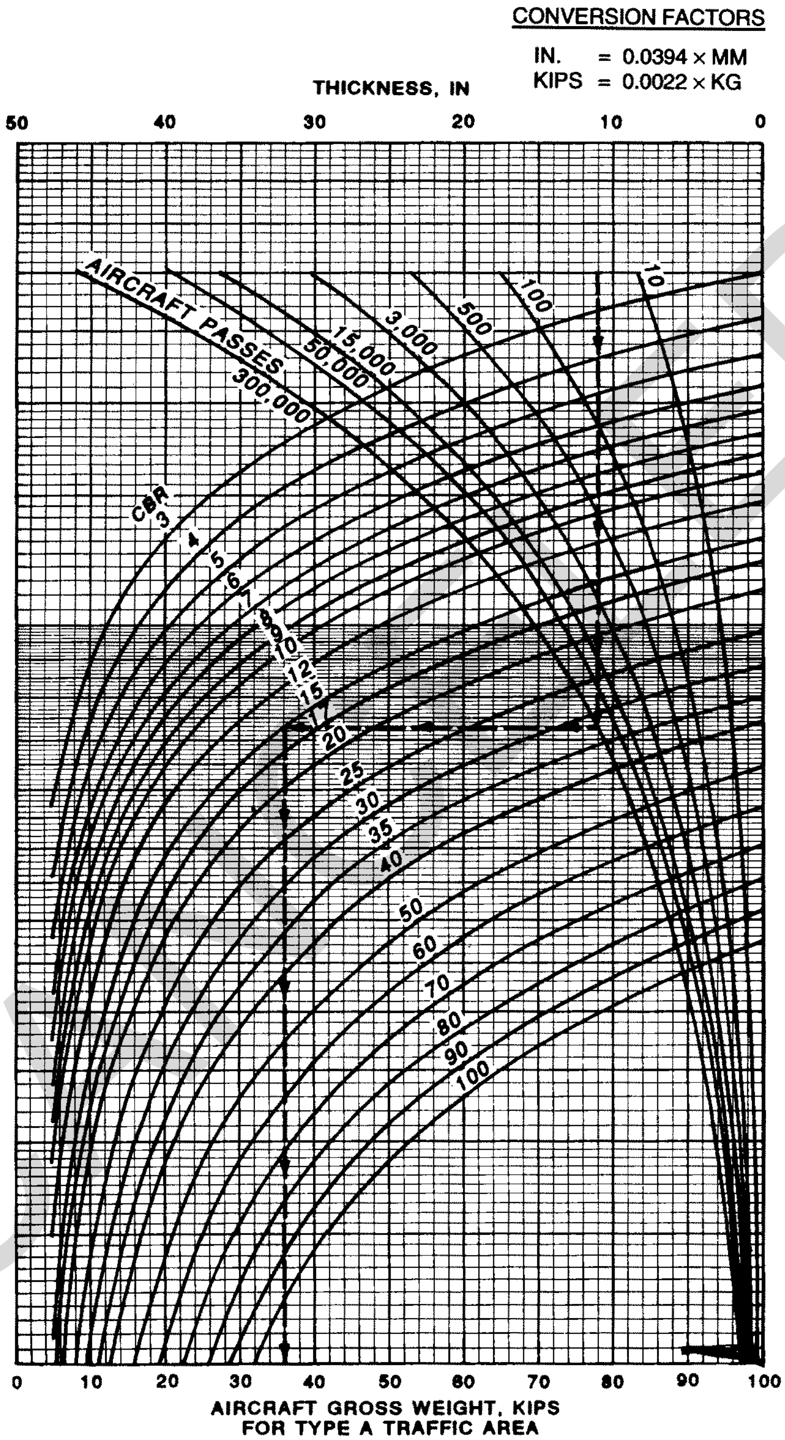


Figure 5-11. Flexible pavement evaluation curve for F-15 (Air Force Group Index 2), Type A traffic area

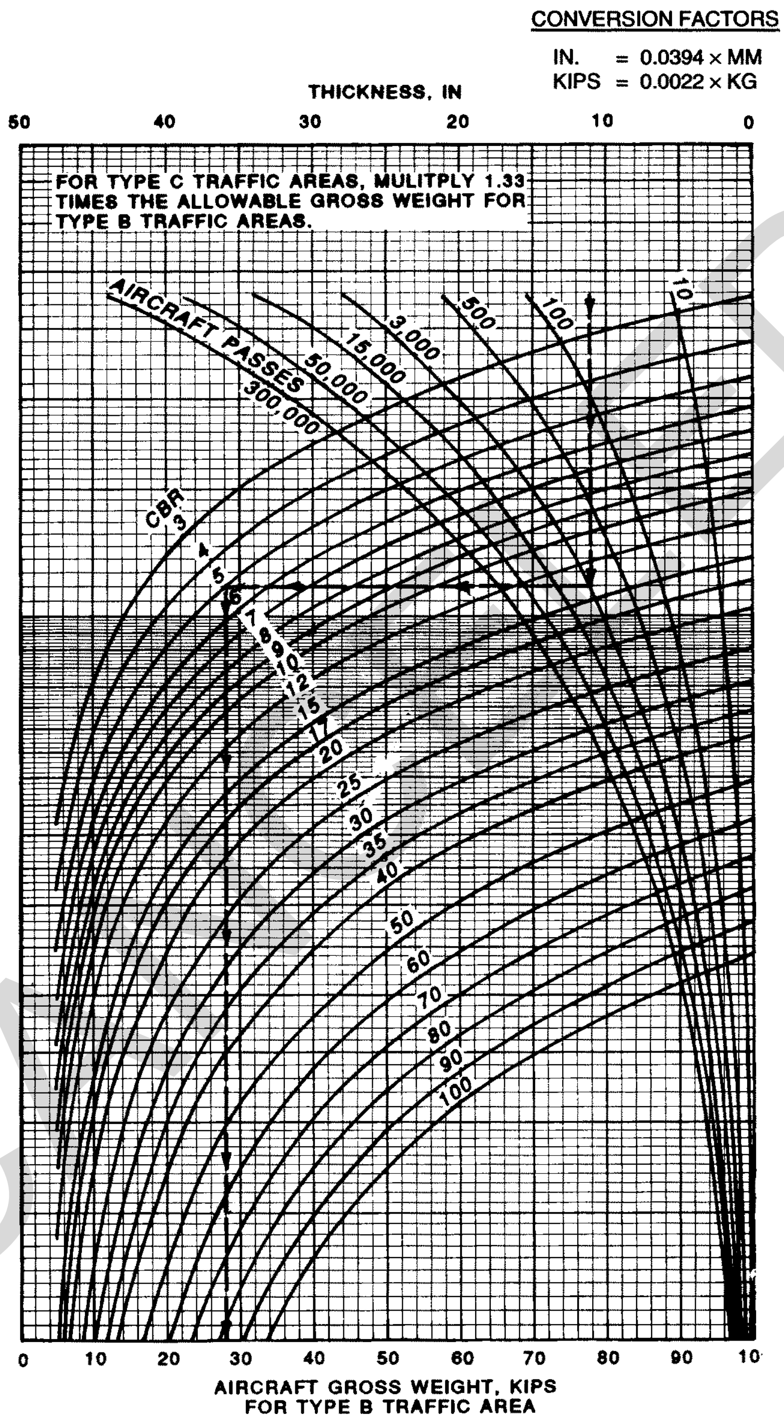


Figure 5-12. Flexible pavement evaluation curve for F-15 (Air Force Group Index 2), Types B and C traffic areas

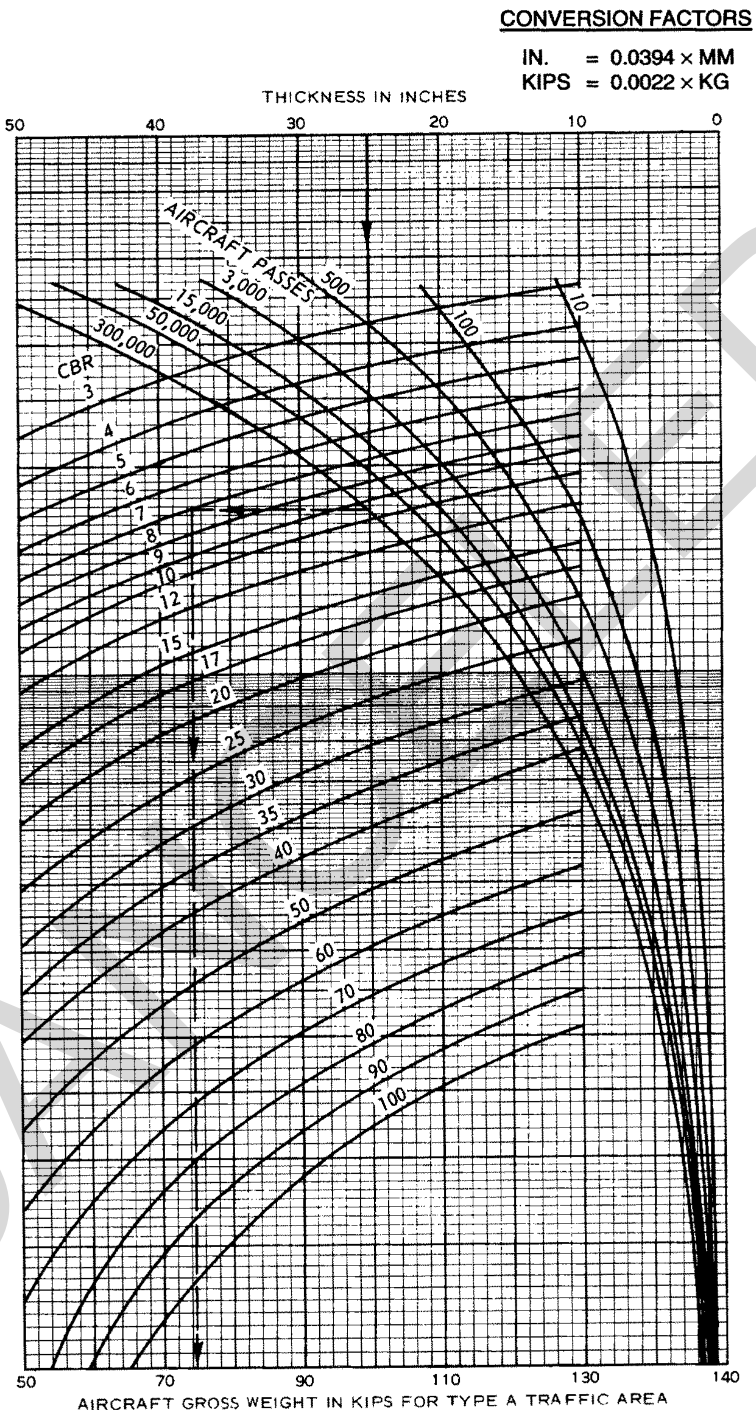


Figure 5-13. Flexible pavement evaluation curve for F-111 (Air Force Group Index 3), Type A traffic area

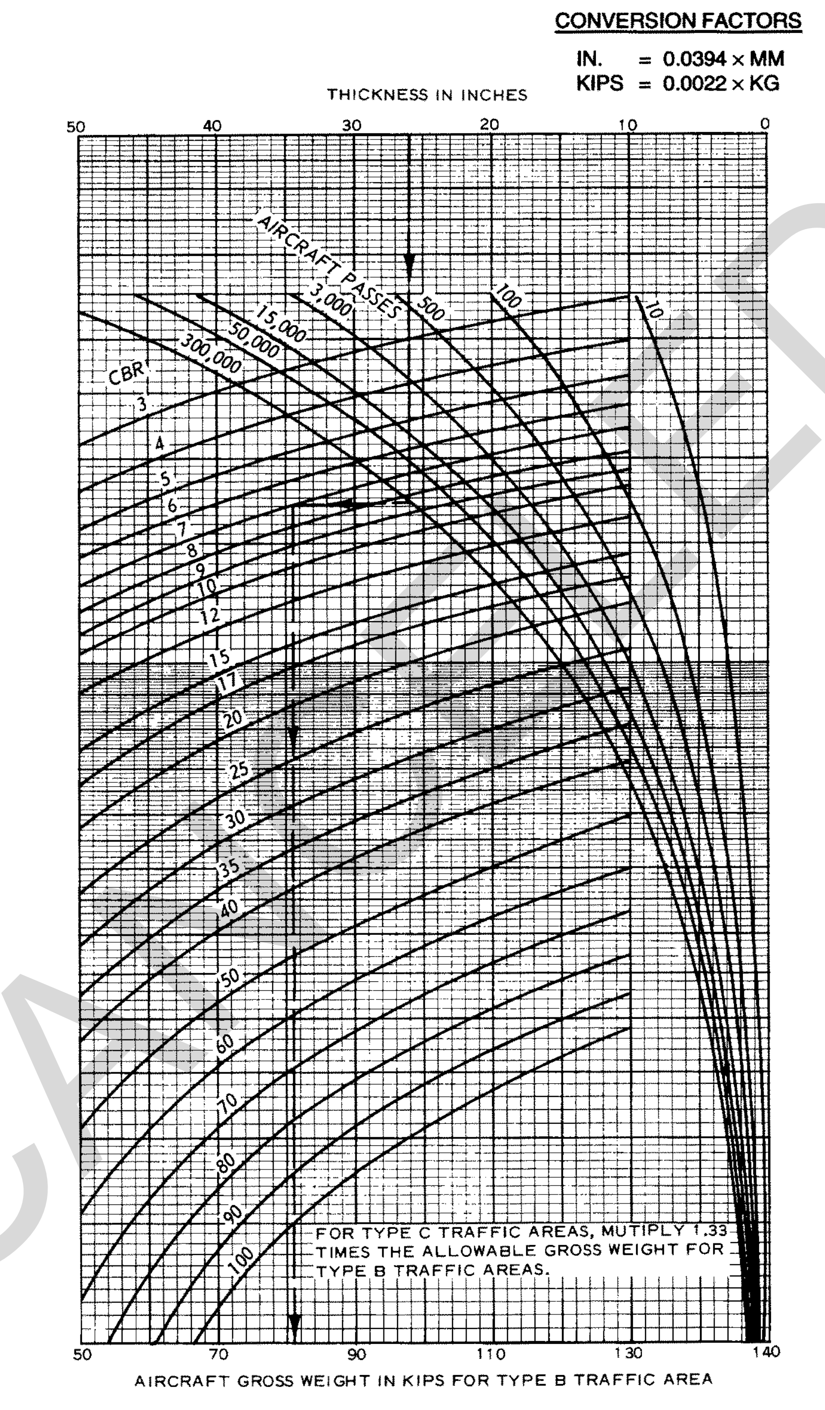


Figure 5-14. Flexible pavement evaluation curve for F-111 (Air Force Group Index 3), Types B and C traffic areas

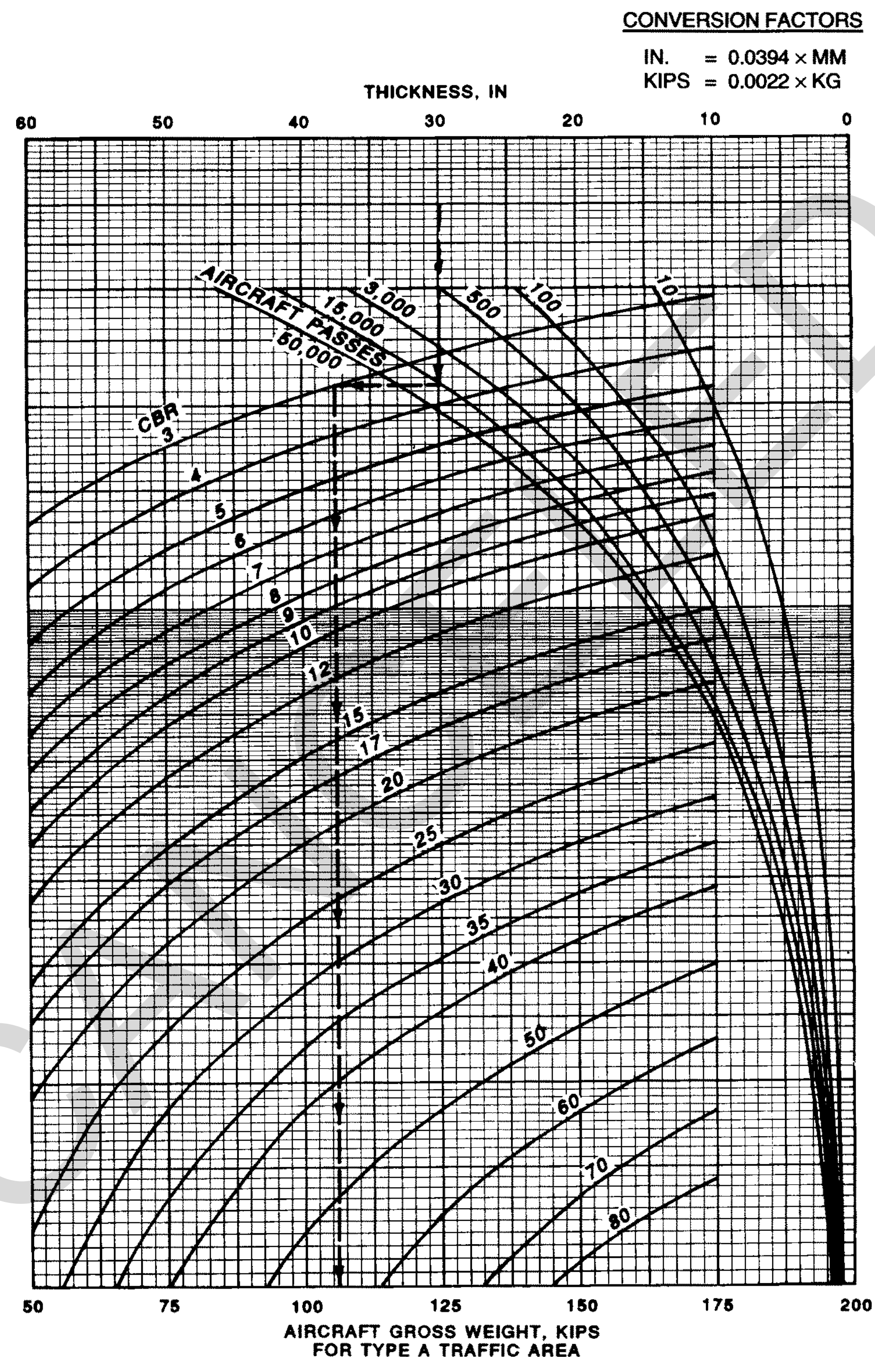


Figure 5-15. Flexible pavement evaluation curve for C-130 (Air Force Group Index 4), Type A traffic area

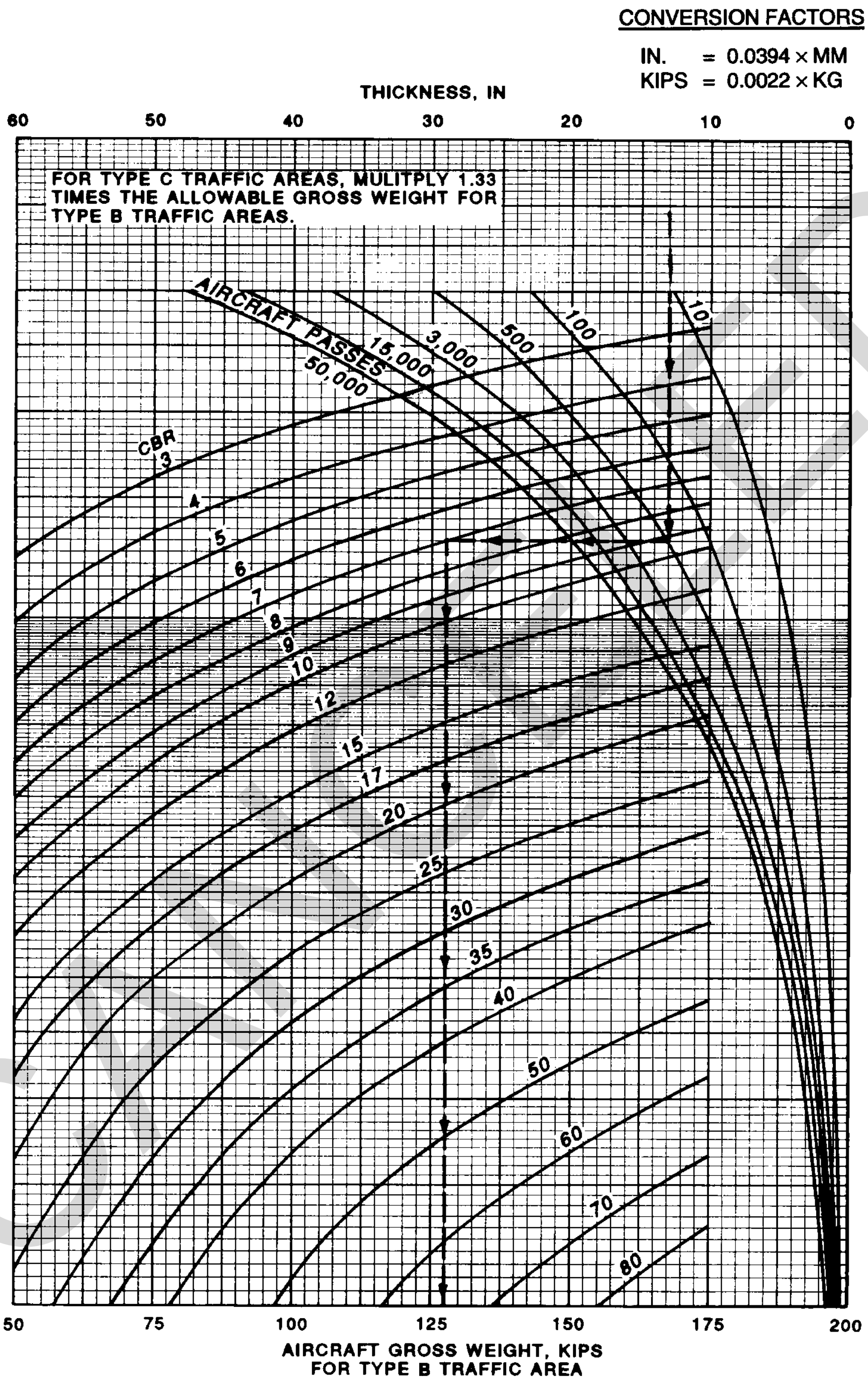


Figure 5-16. Flexible pavement evaluation curve for C-130 (Air Force Group Index 4), Types B and C traffic areas

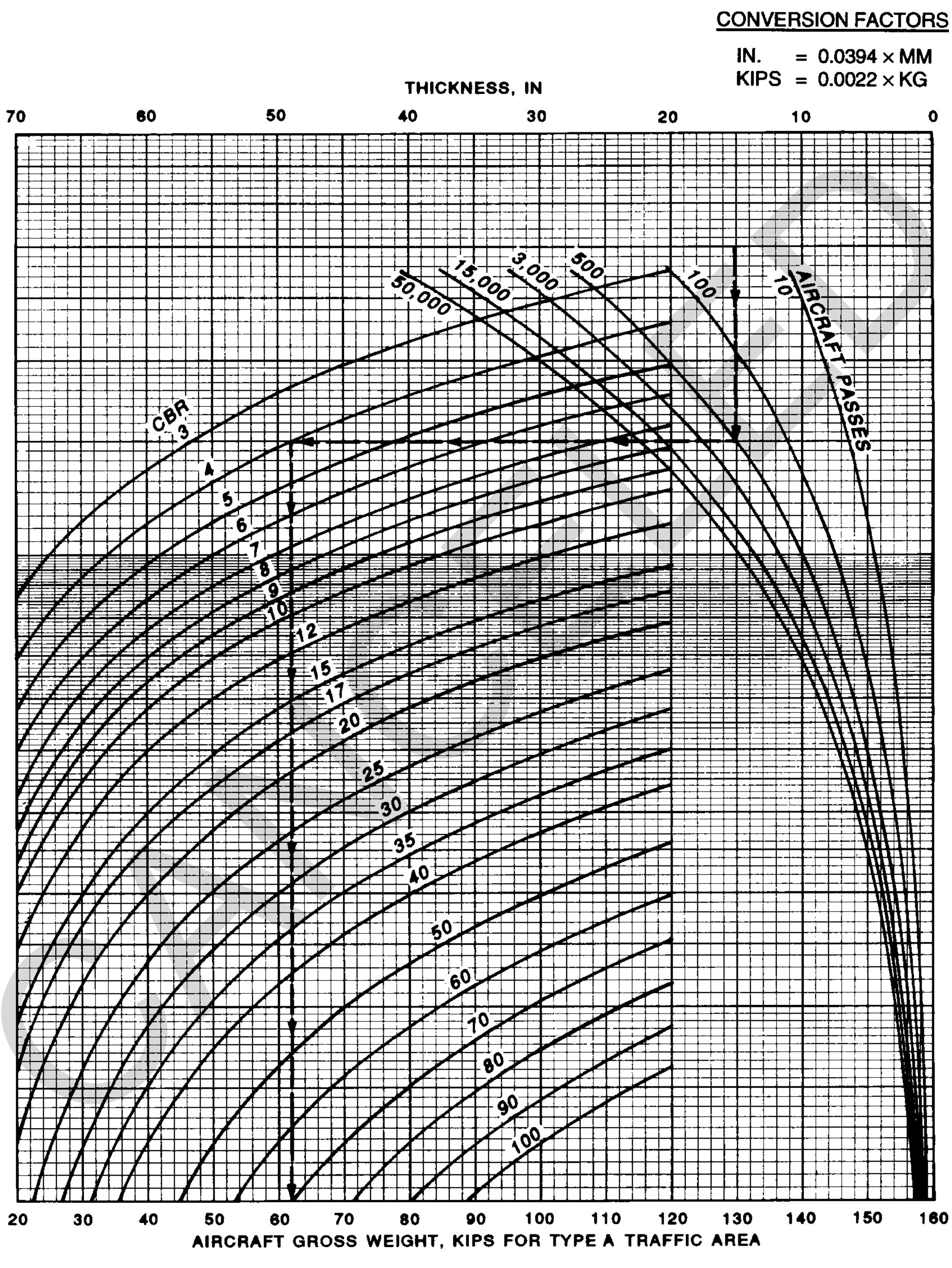


Figure 5-17. Flexible pavement evaluation curve for C-9 (Air Force Group Index 5), Type A traffic area

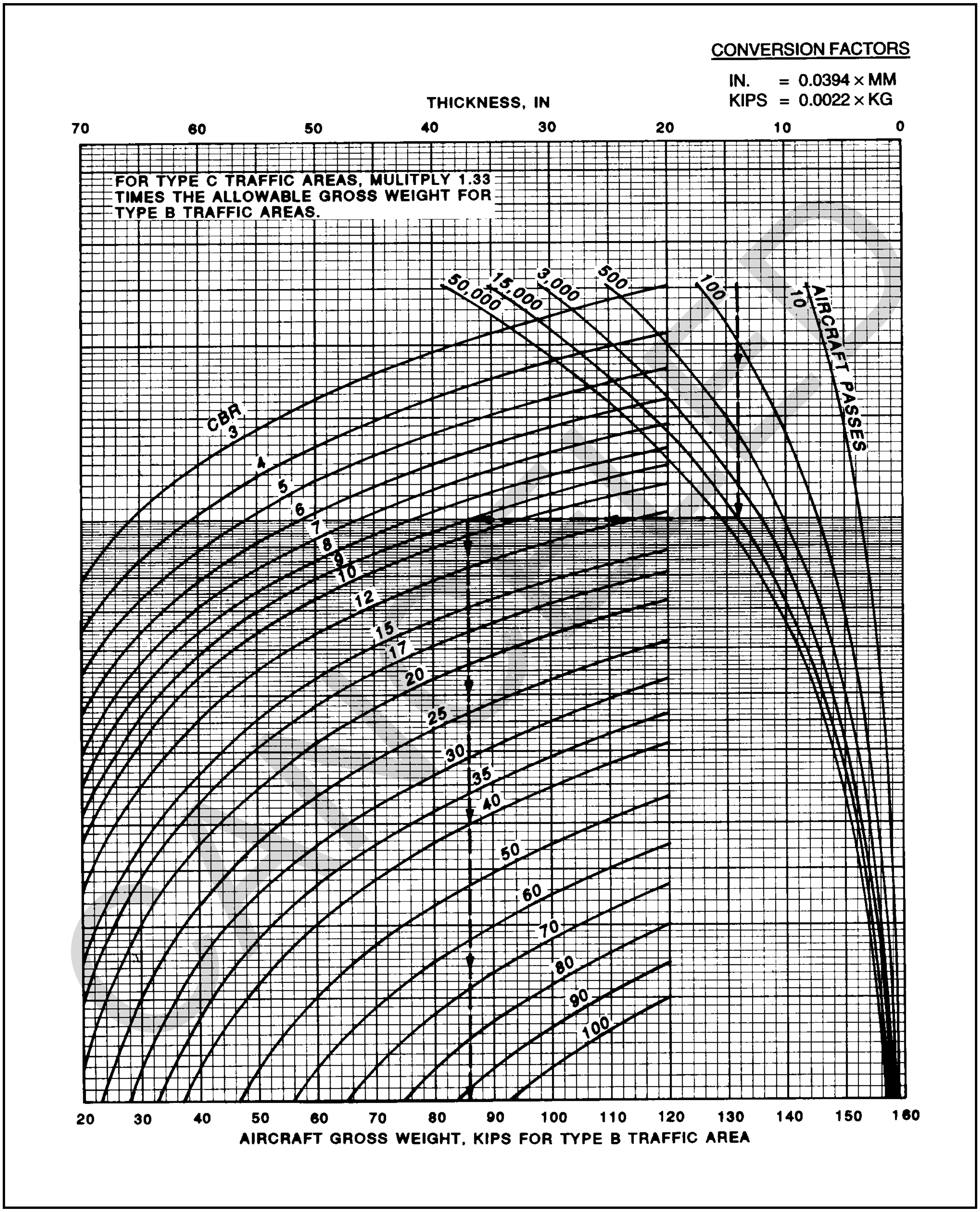


Figure 5-18. Flexible pavement evaluation curve for C-9 (Air Force Group Index 5), Types B and C traffic areas

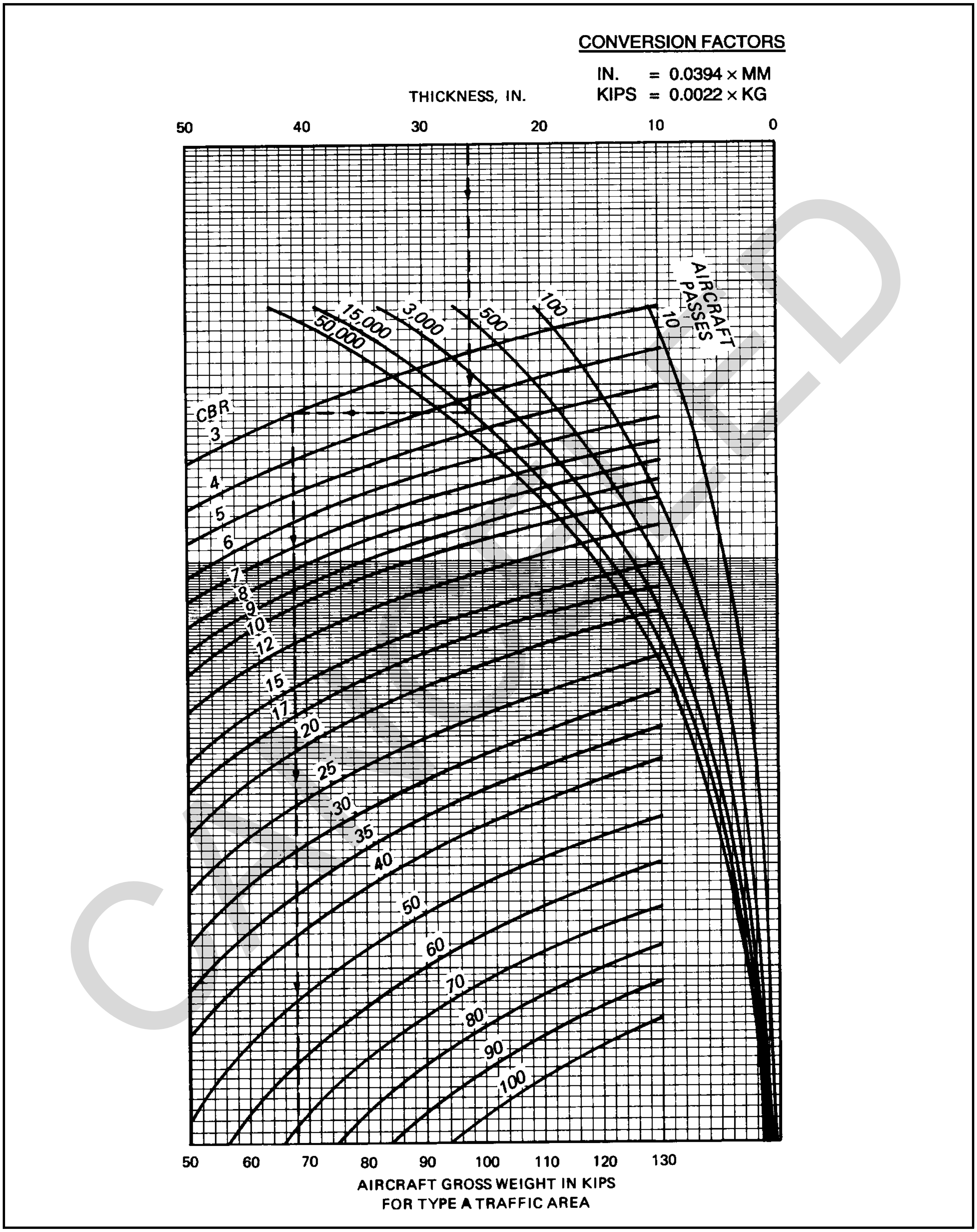


Figure 5-19. Flexible pavement evaluation curve for T-43 (Air Force Group Index 6), Type A traffic area

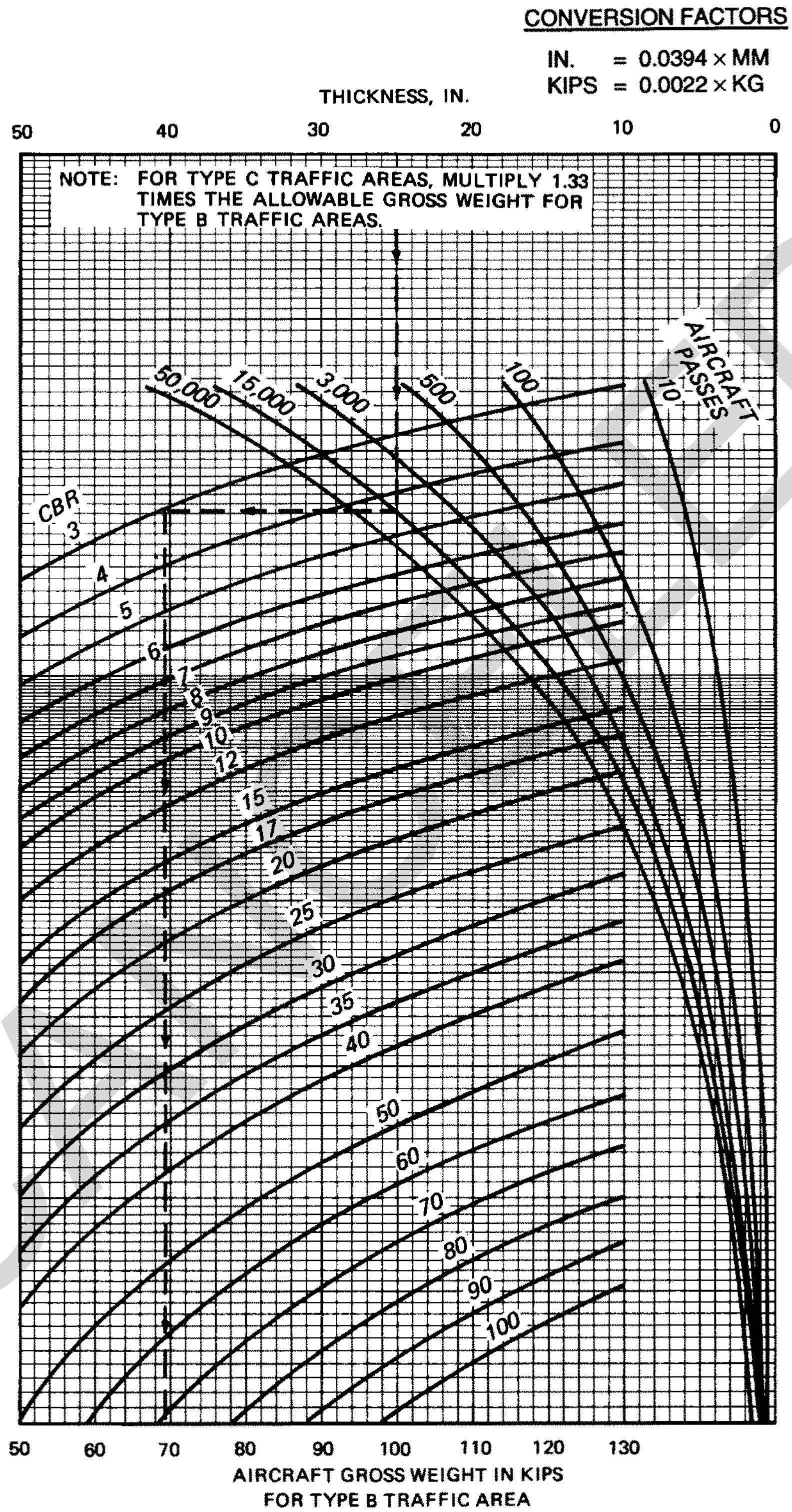


Figure 5-20. Flexible pavement evaluation curve for T-43 (Air Force Group Index 6), Types B and C traffic areas

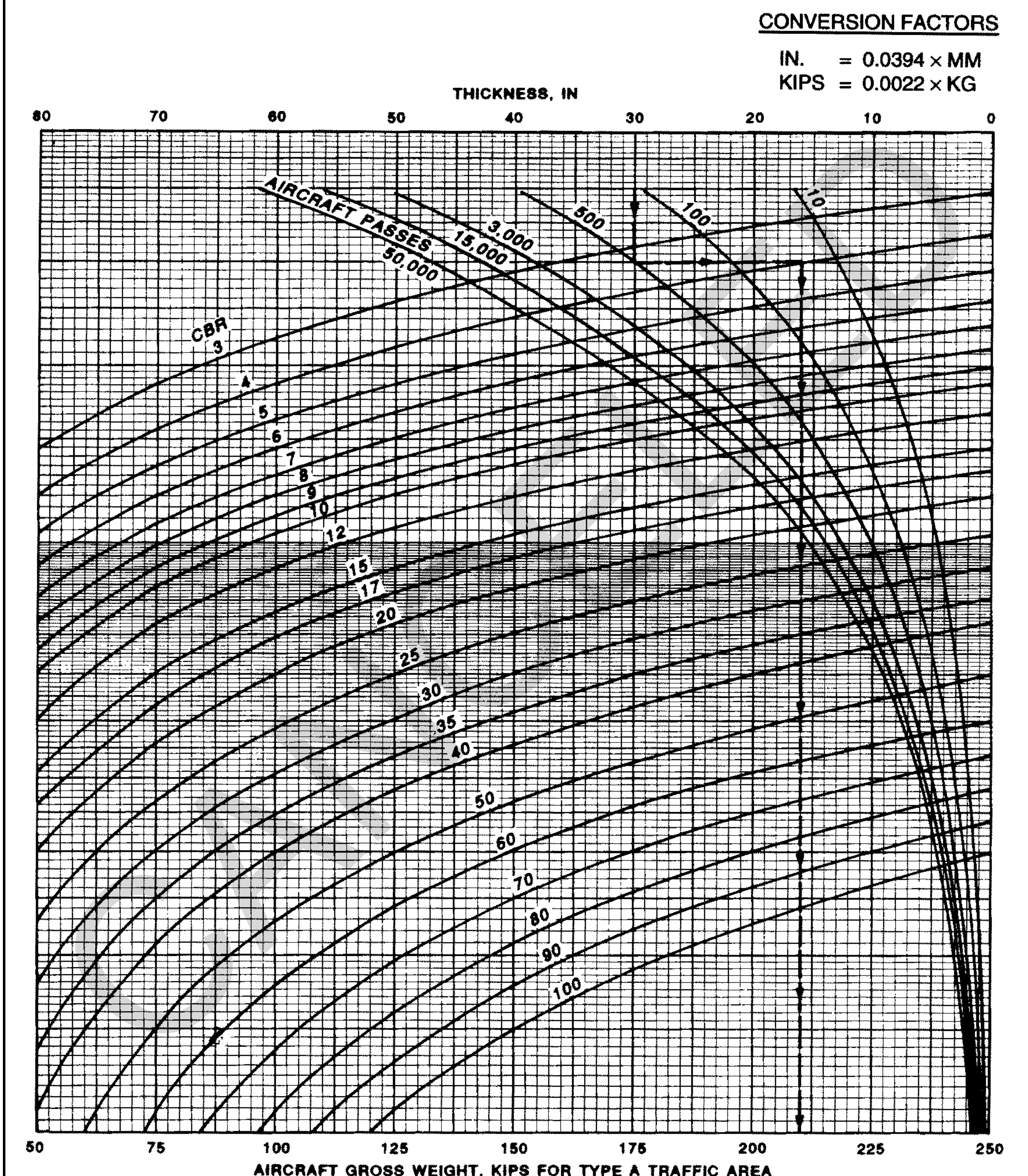


Figure 5-21. Flexible pavement evaluation curve for B-727 (Air Force Group Index 7), Type A traffic area

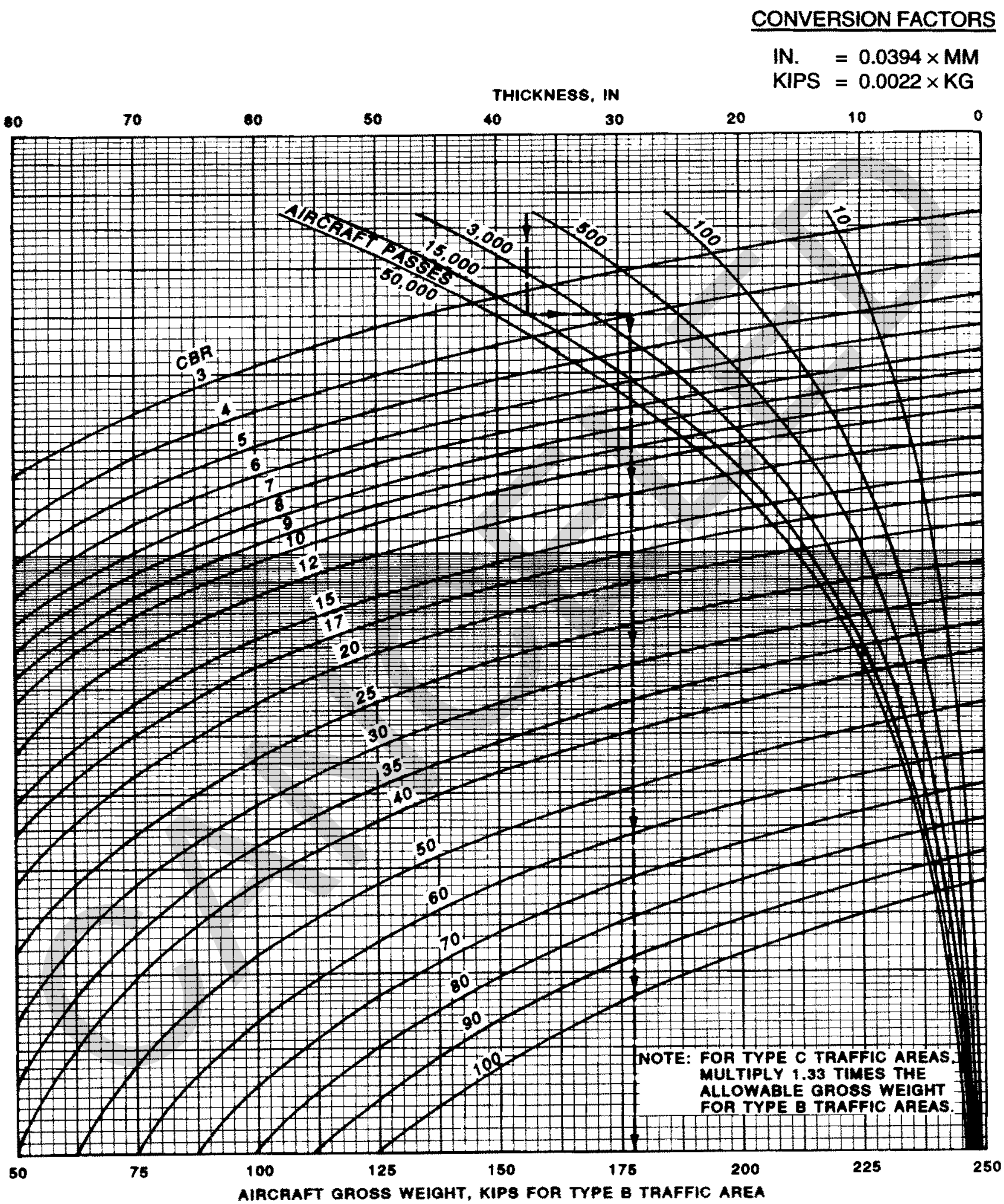


Figure 5-22. Flexible pavement evaluation curve for B-727 (Air Force Group Index 7), Types B and C traffic areas

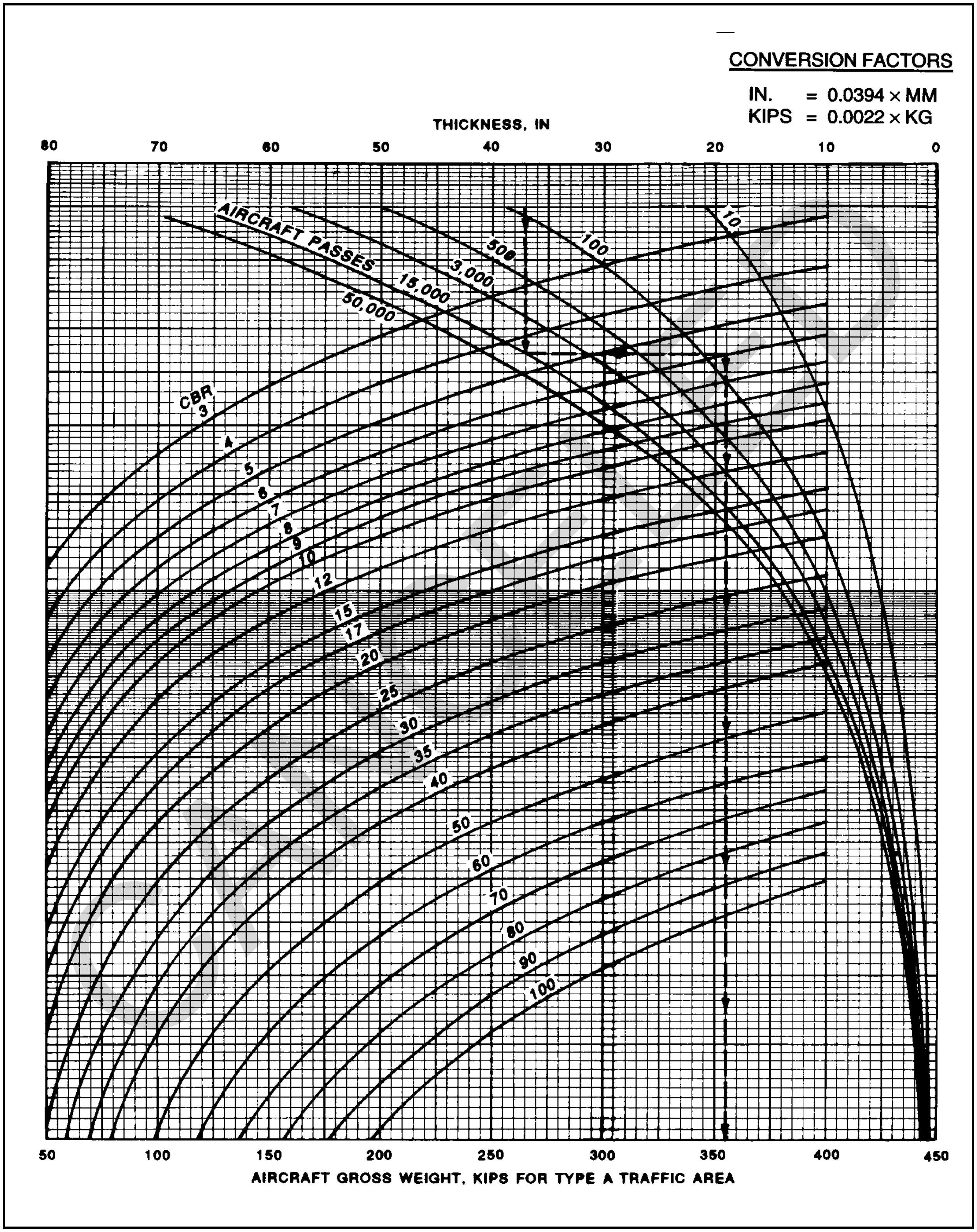


Figure 5-23. Flexible pavement evaluation curve for E-3 (Air Force Group Index 8), Type A traffic area

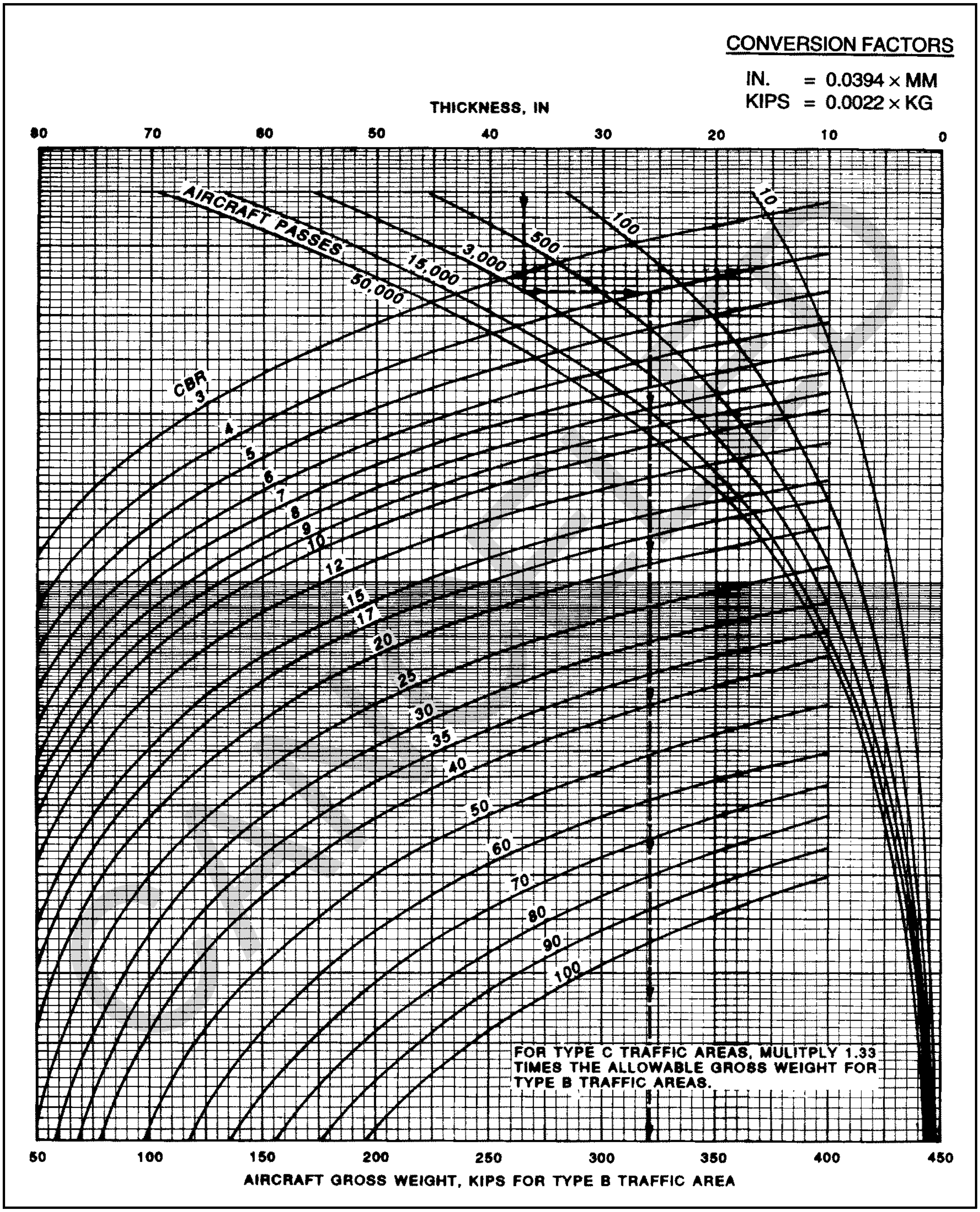


Figure 5-24. Flexible pavement evaluation curve for E-3 (Air Force Group Index 8), Types B and C traffic areas

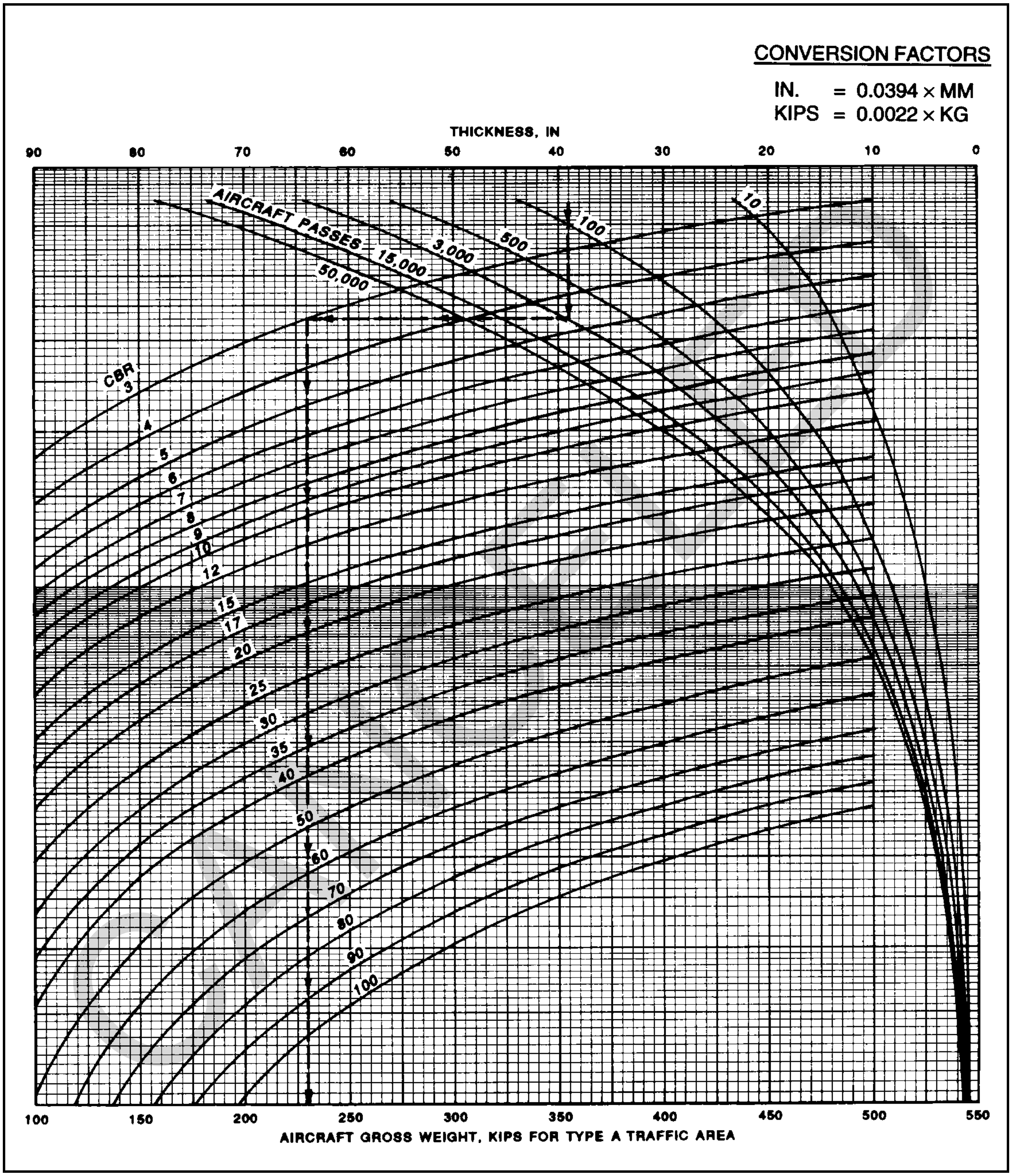


Figure 5-25. Flexible pavement evaluation curve for C-141 (Air Force Group Index 9), Type A traffic area

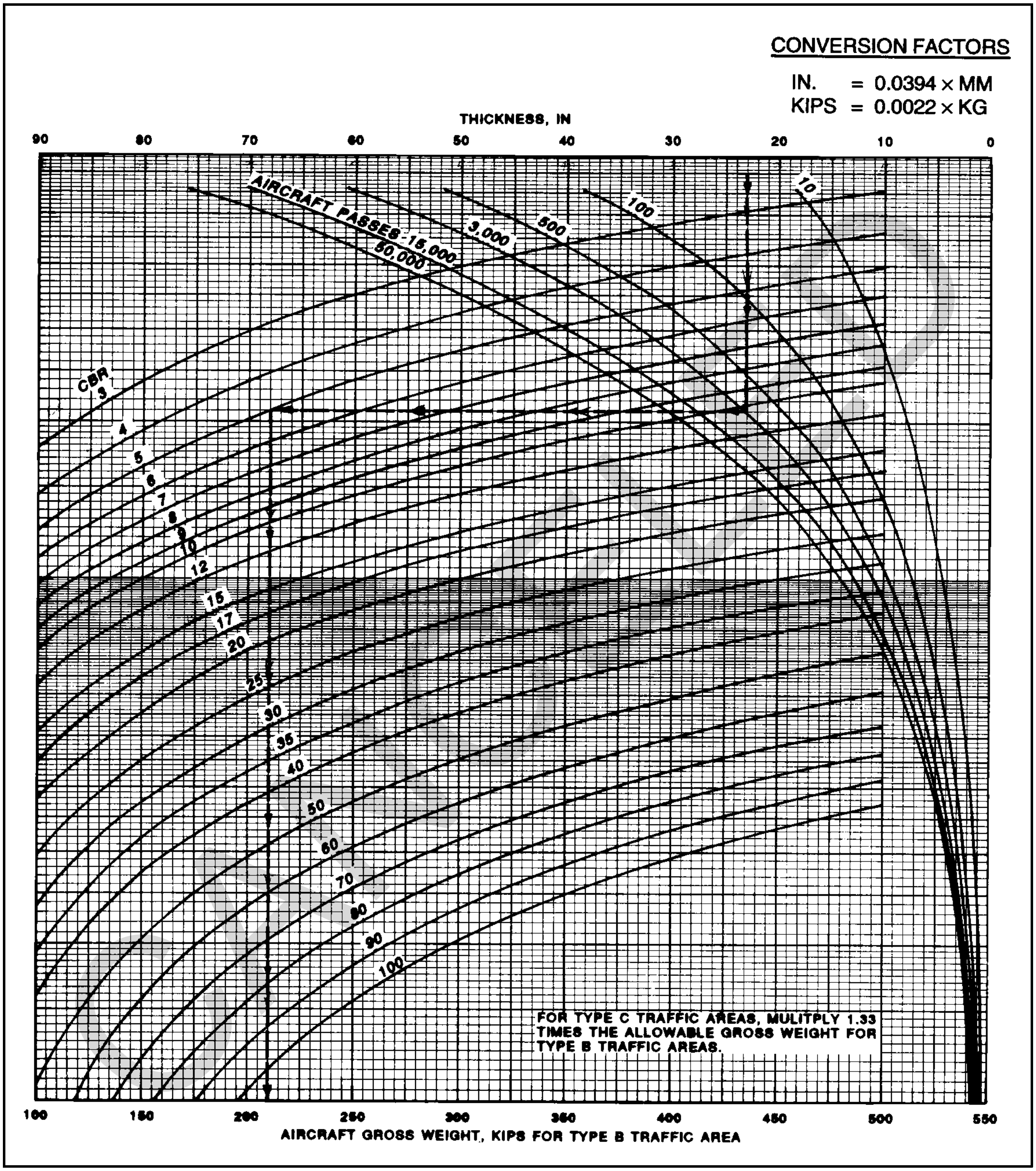


Figure 5-26. Flexible pavement evaluation curve for C-141 (Air Force Group Index 9), Types B and C traffic areas

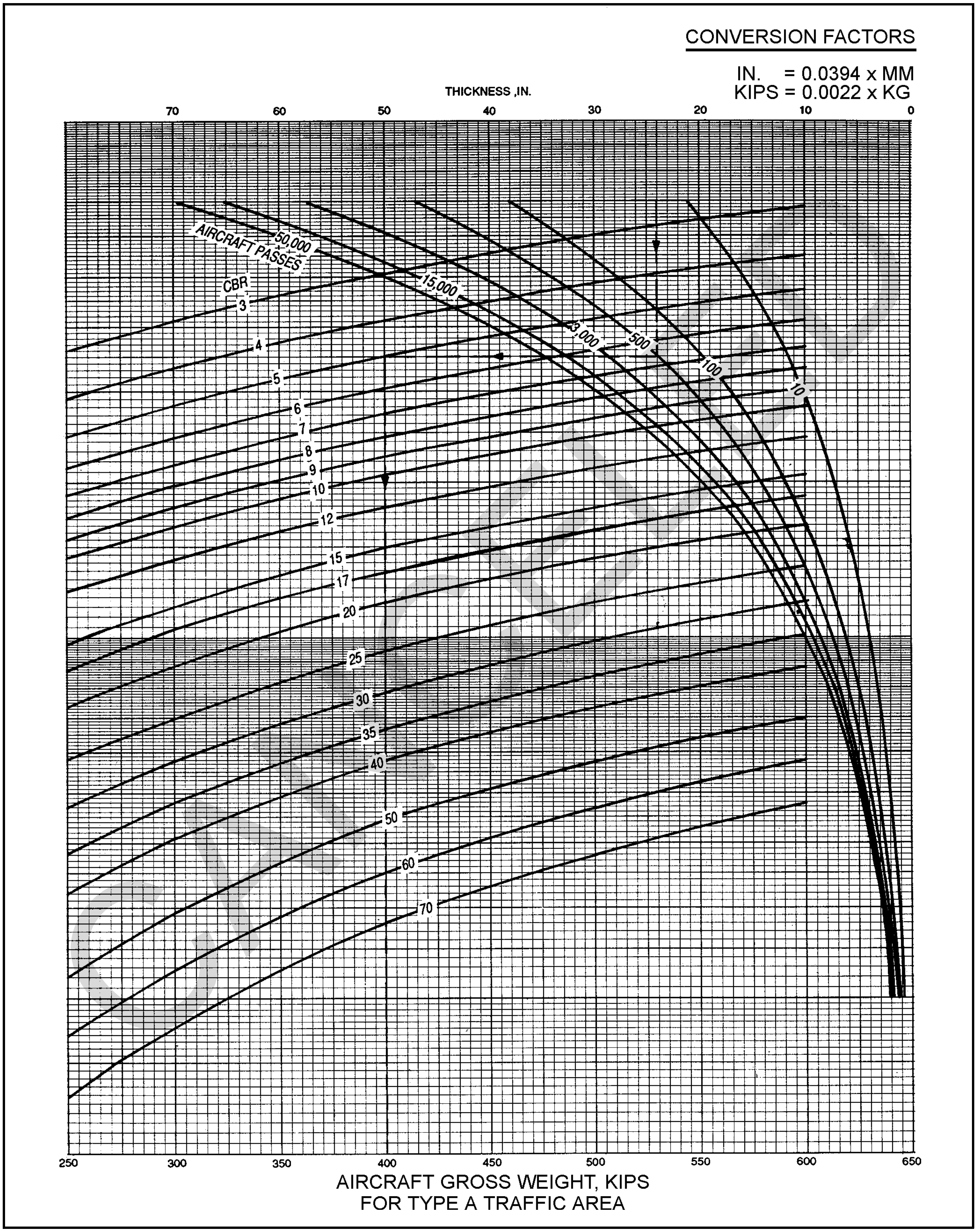


Figure 5-27. Flexible pavement evaluation curve for C-17 (Air Force Group Index 10), Type A traffic area

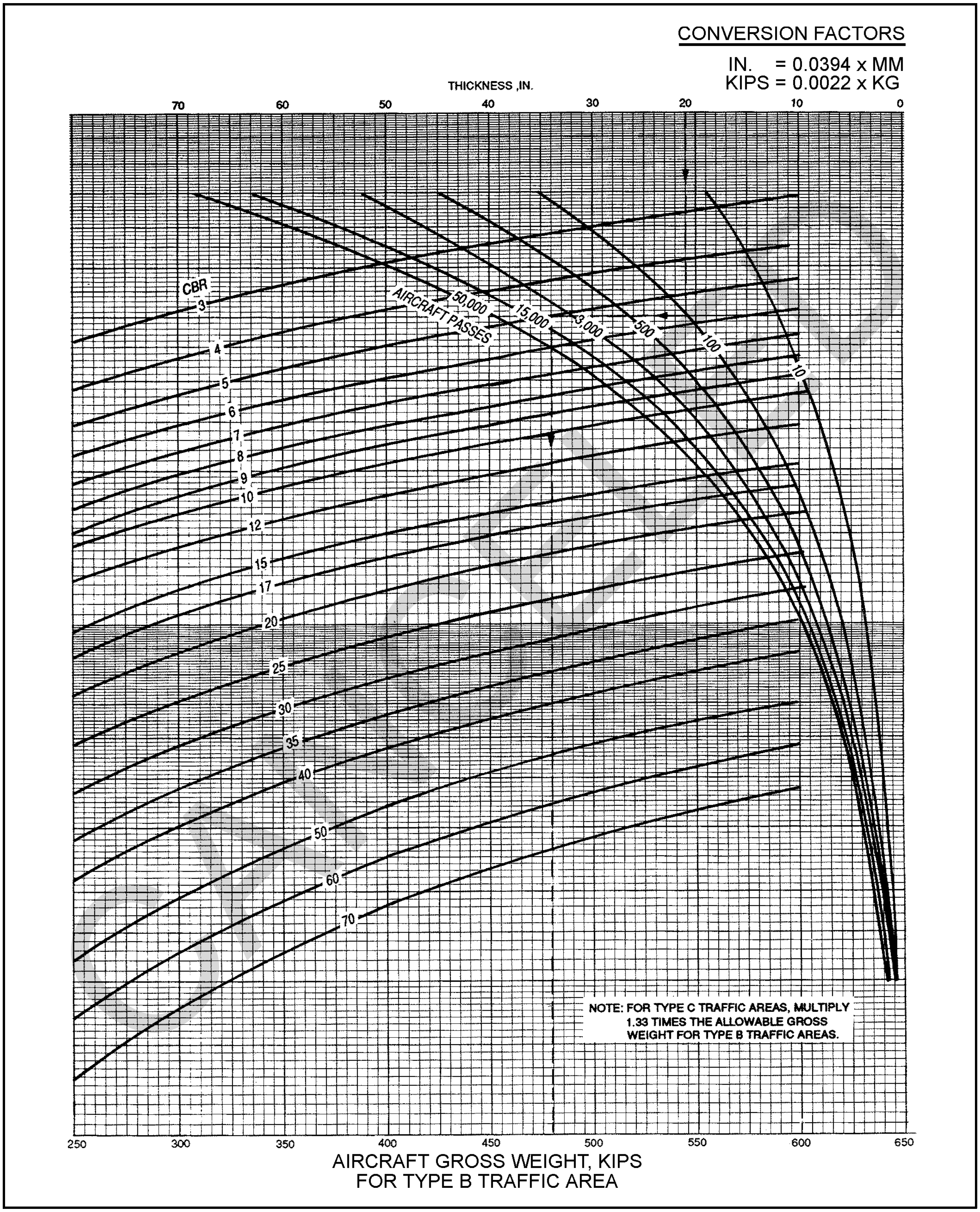


Figure 5-28. Flexible pavement evaluation curve for C-17 (Air Force Group Index 10), Type B and C traffic areas

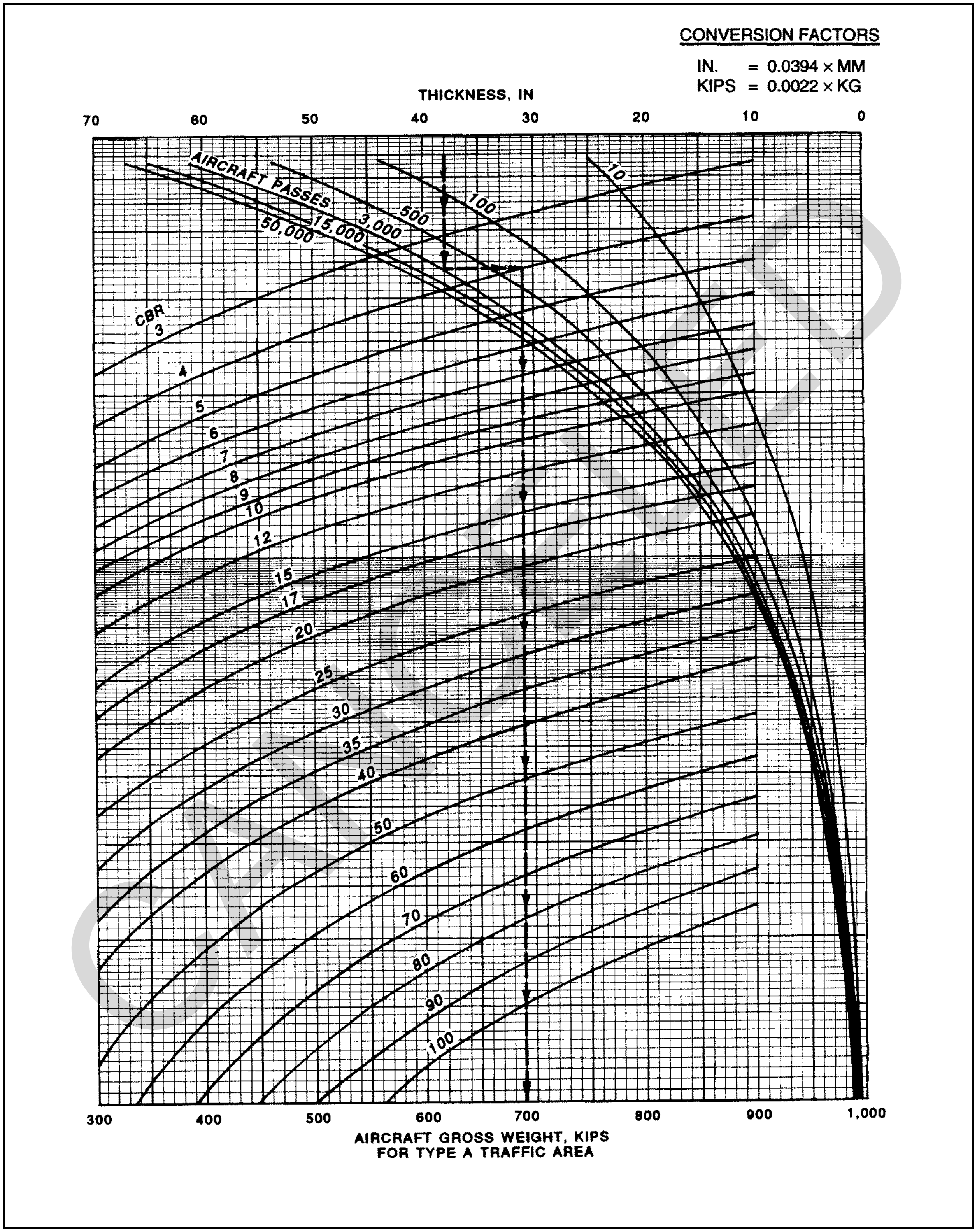


Figure 5-29. Flexible pavement evaluation curve for C-5 (Air Force Group Index 11), Type A traffic area

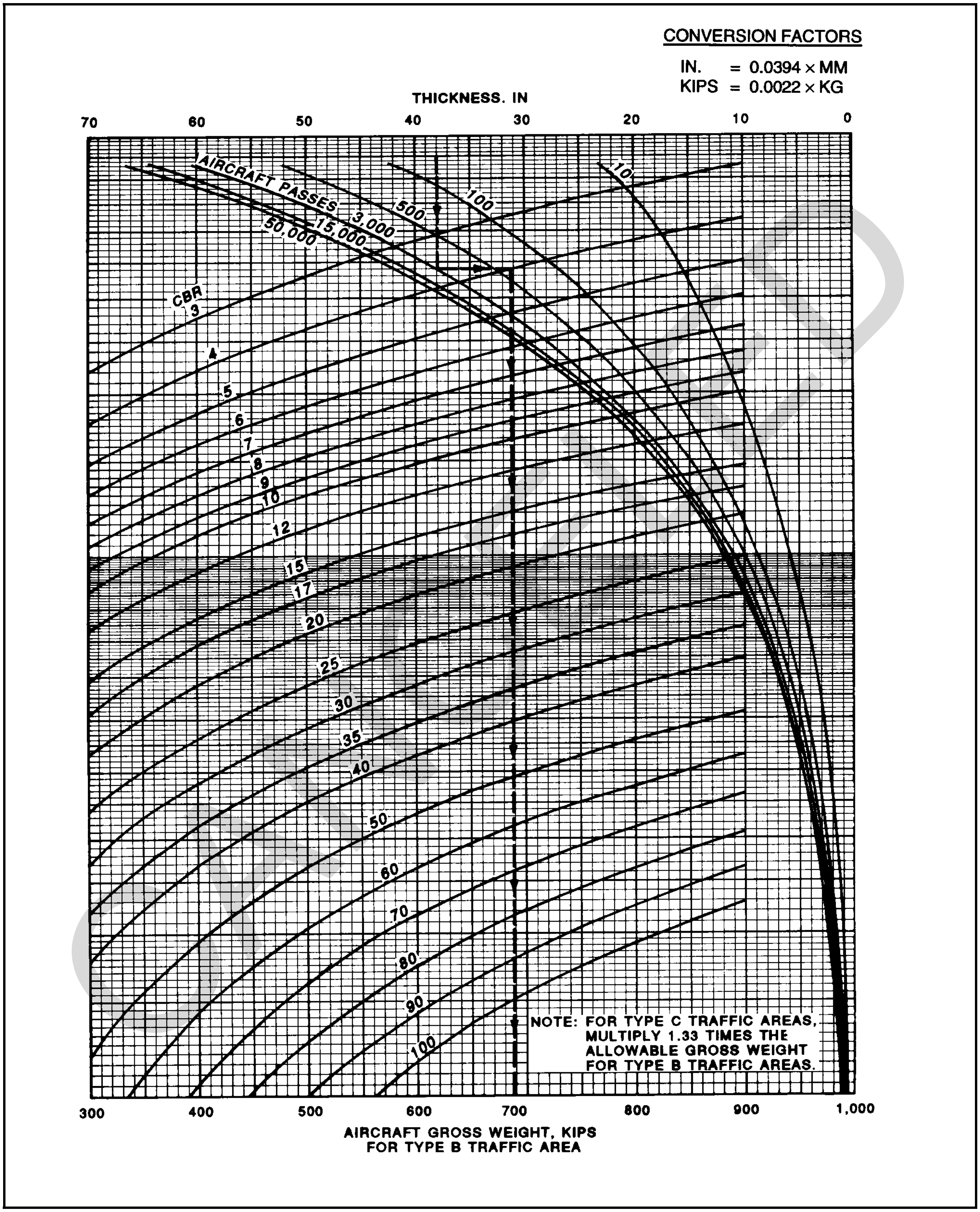


Figure 5-30. Flexible pavement evaluation curve for C-5 (Air Force Group Index 11), Type B and C traffic areas

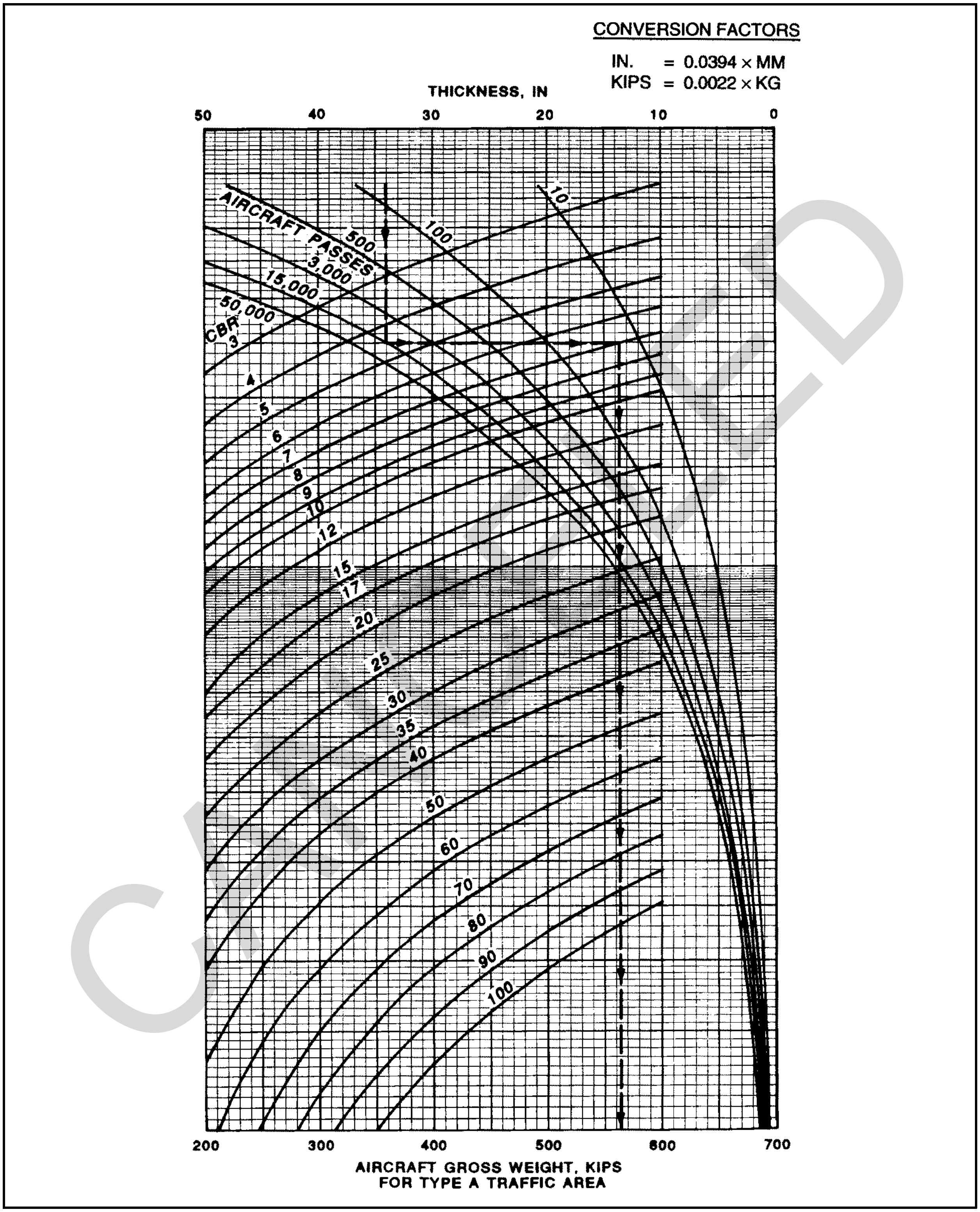


Figure 5-31. Flexible pavement evaluation curve for KC-10 (Air Force Group Index 12), Type A traffic area

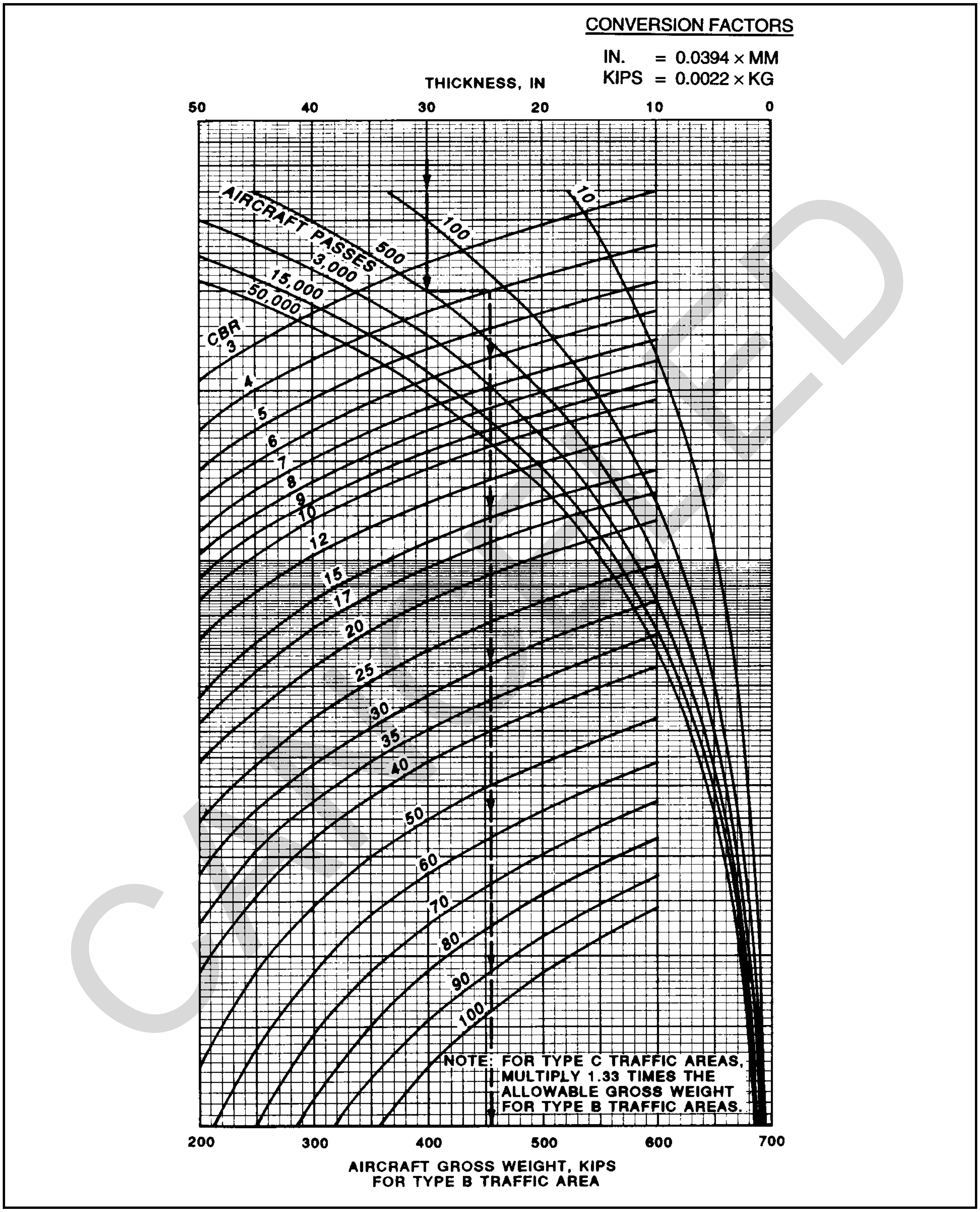


Figure 5-32. Flexible pavement evaluation curve for KC-10 (Air Force Group Index 12), Types B and C traffic areas

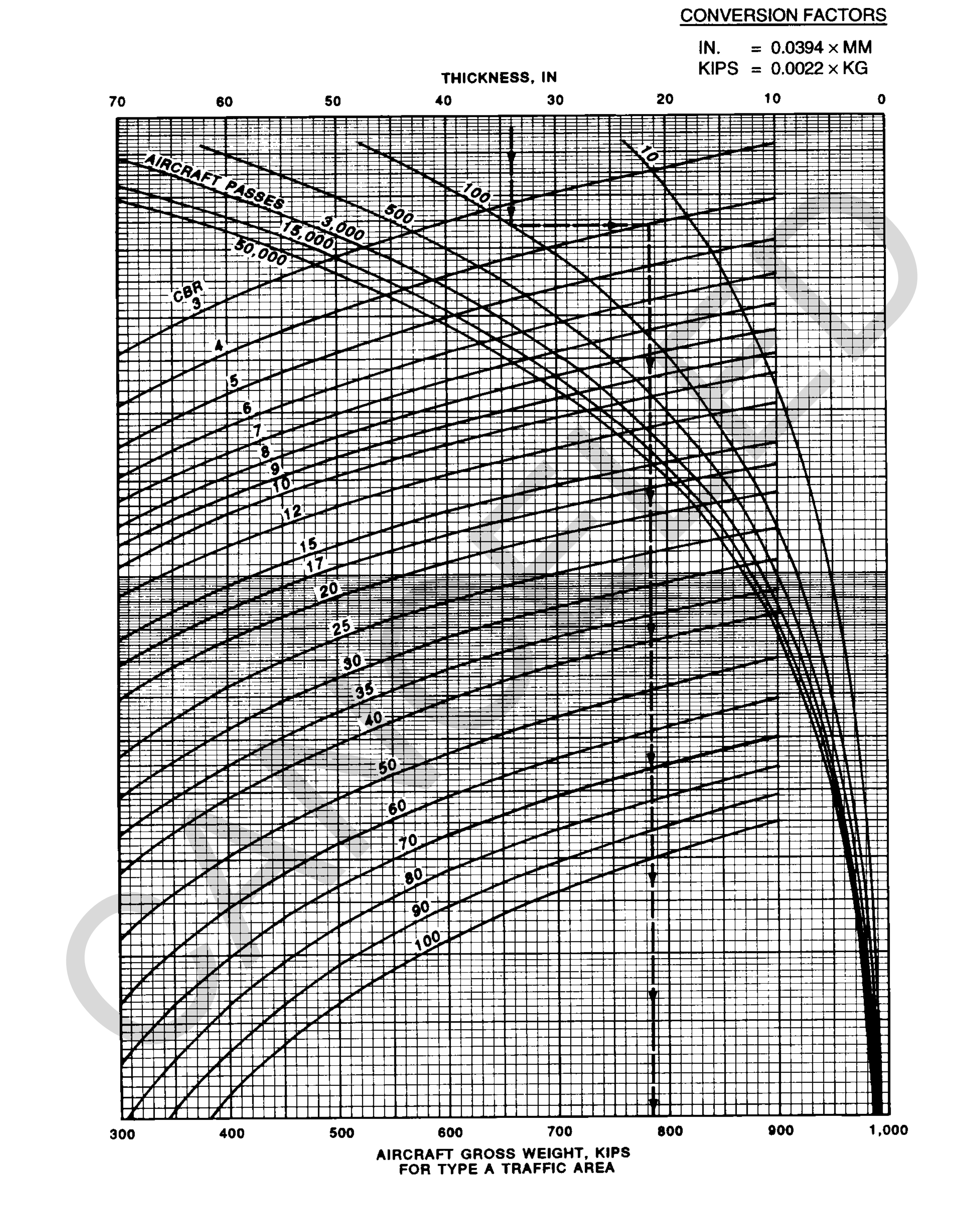


Figure 5-33. Flexible pavement evaluation curve for E-4 (Air Force Group Index 13), Type A traffic area

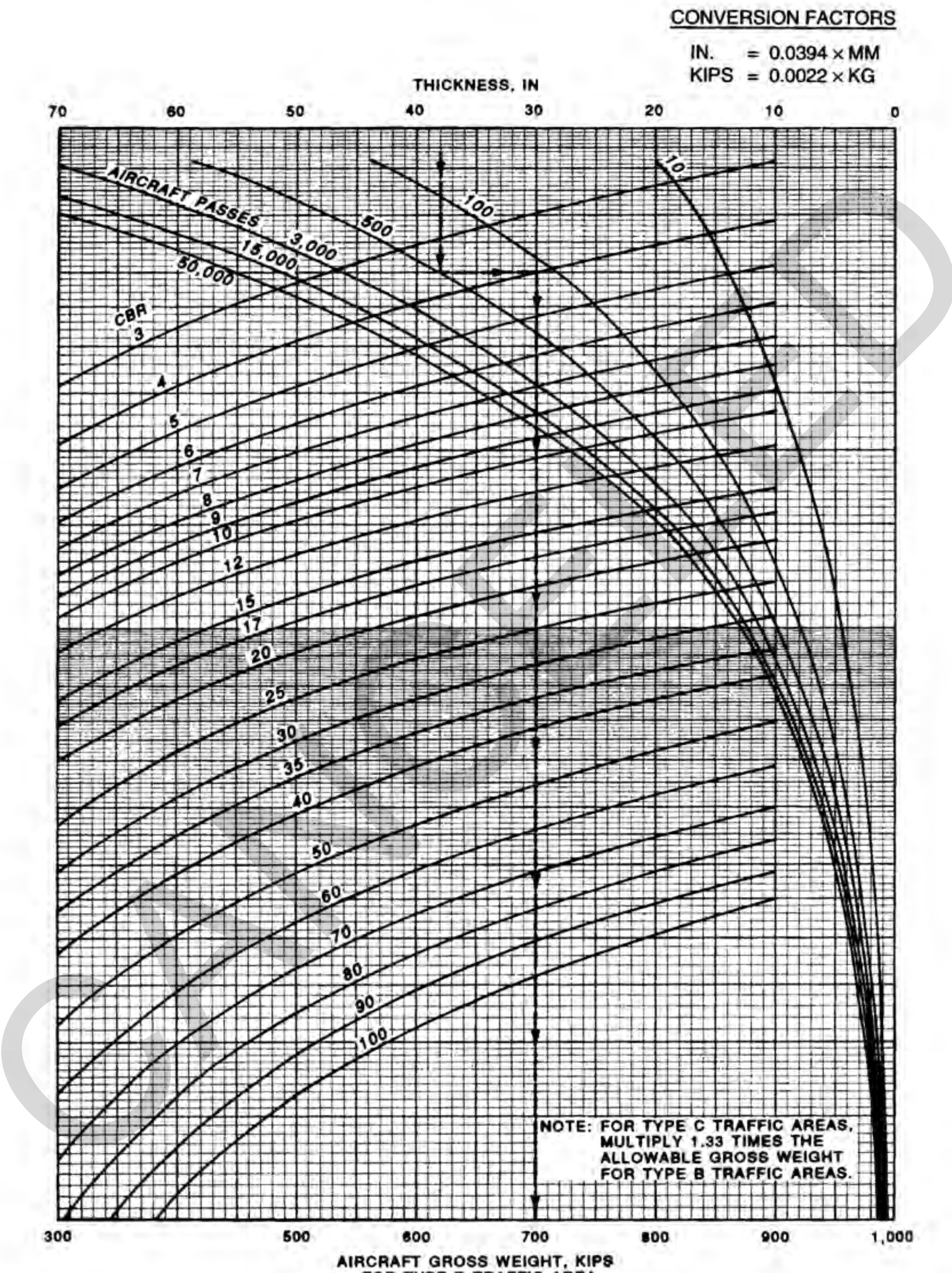


Figure 5-34. Flexible pavement evaluation curve for E-4 (Air Force Group Index 13), Types B and C traffic areas

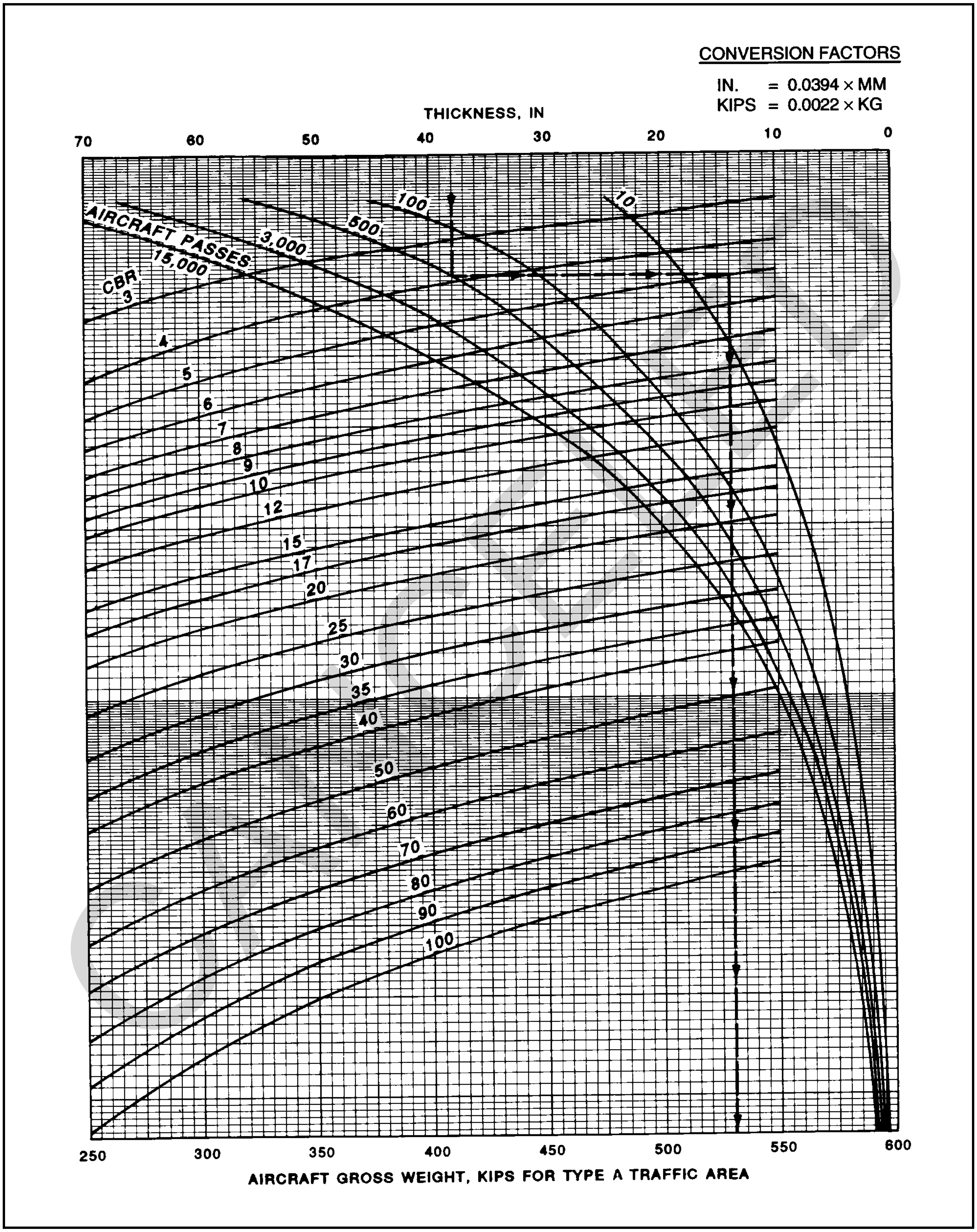


Figure 5-35. Flexible pavement evaluation curve for B-52 (Air Force Group Index 14), Type A traffic area

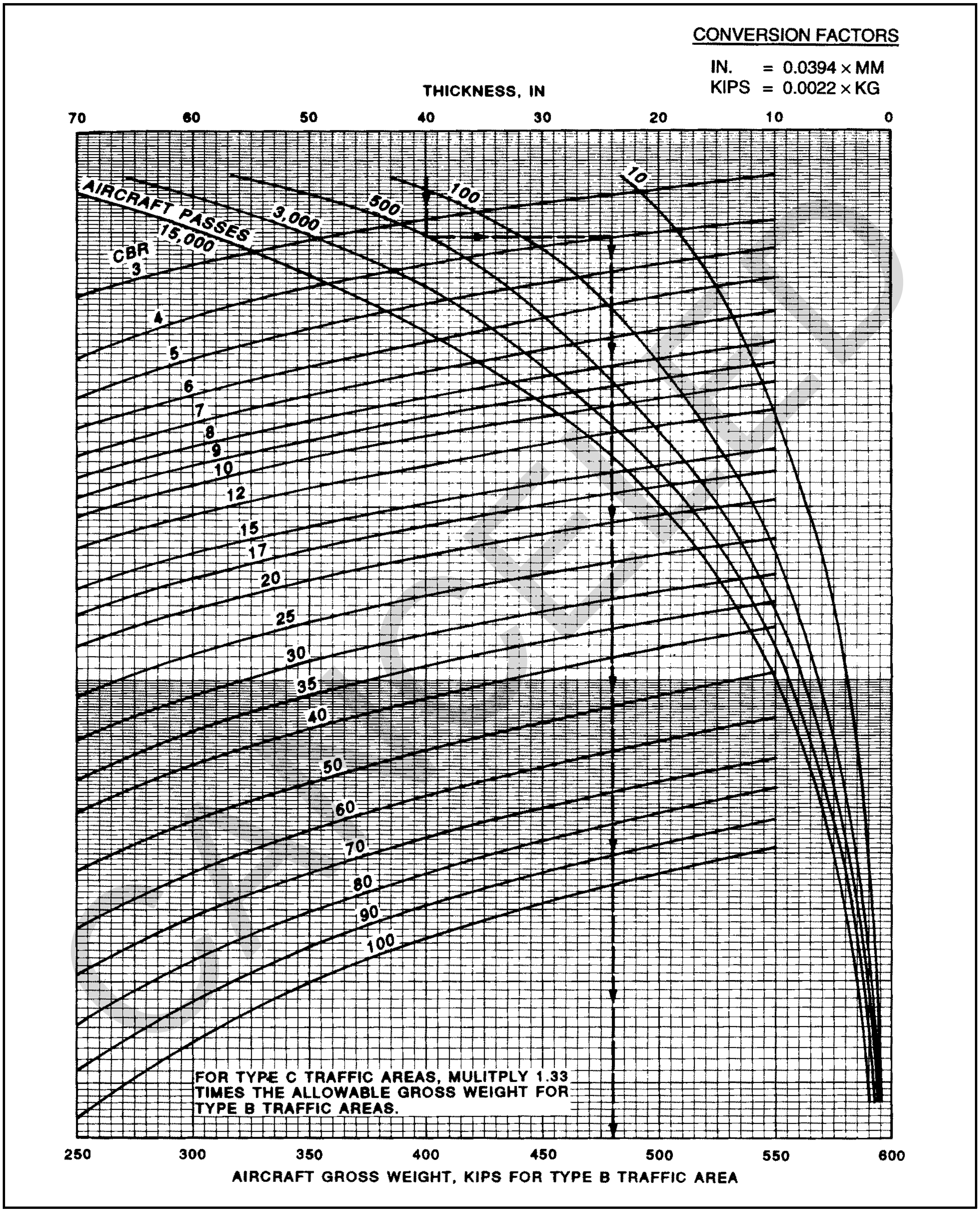


Figure 5-36. Flexible pavement evaluation curve for B-52 (Air Force Group Index 14), Types B and C traffic areas

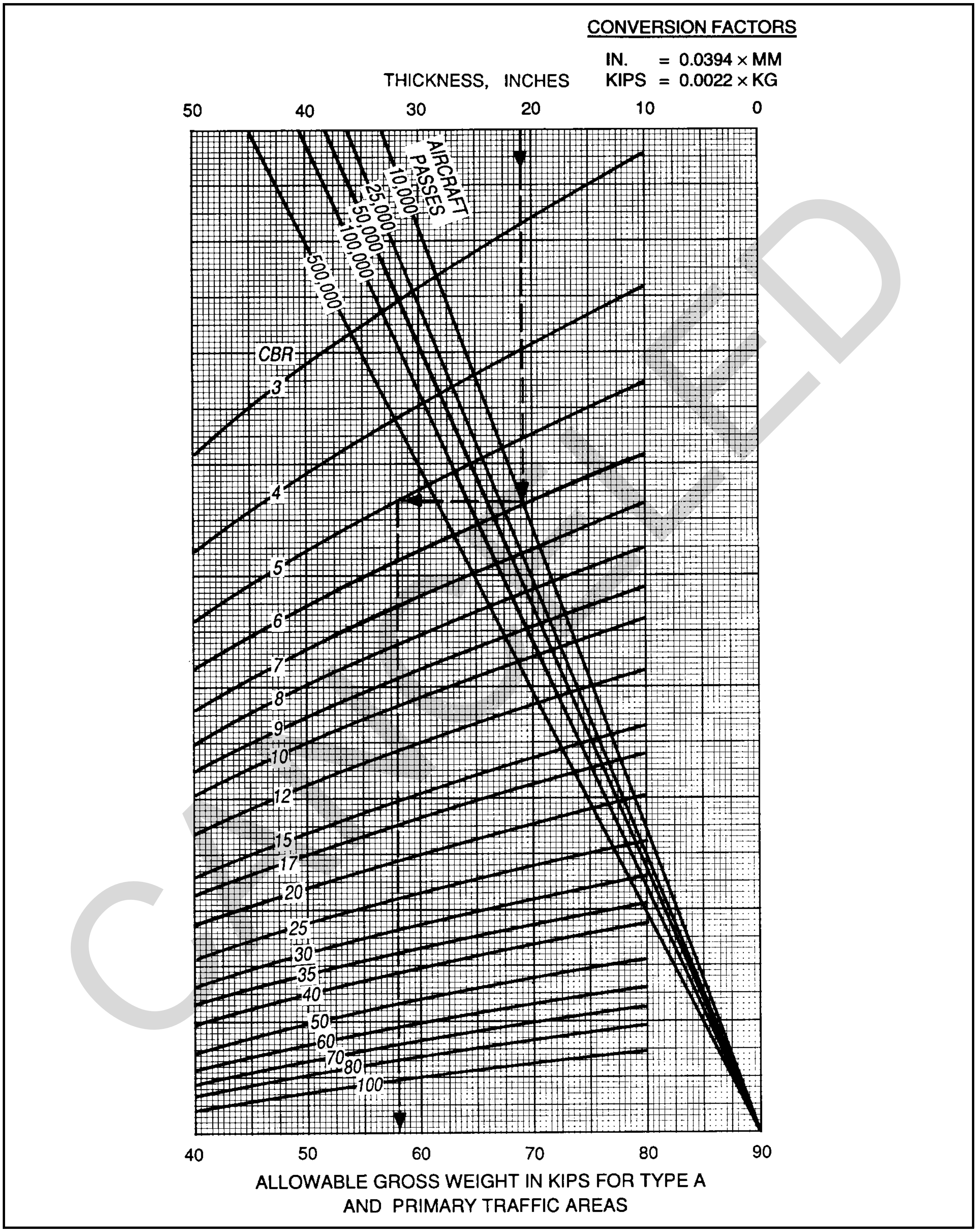


Figure 5-37. Flexible pavement evaluation curve for F-14, primary traffic area

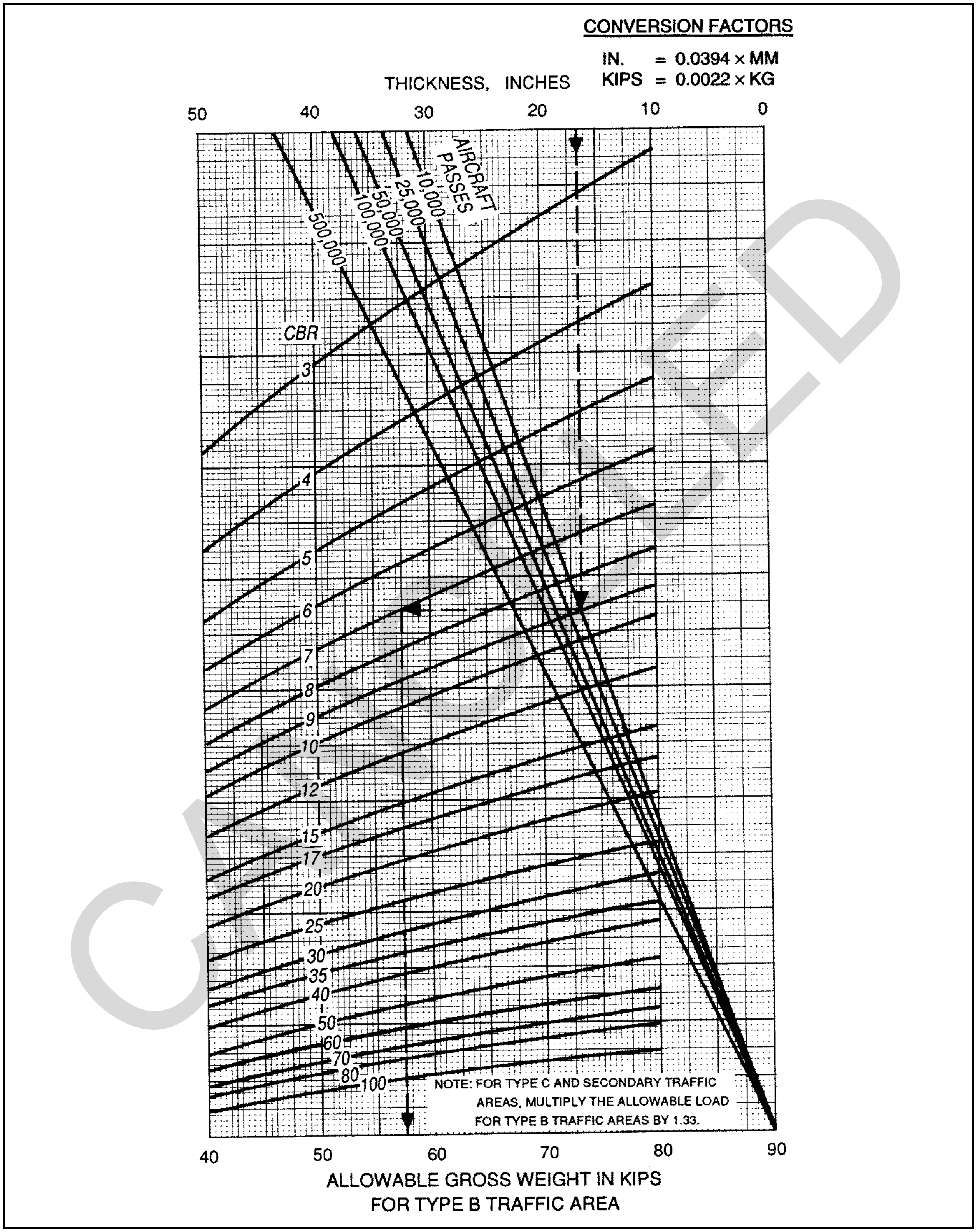


Figure 5-38. Flexible pavement evaluation curve for F-14, secondary traffic area

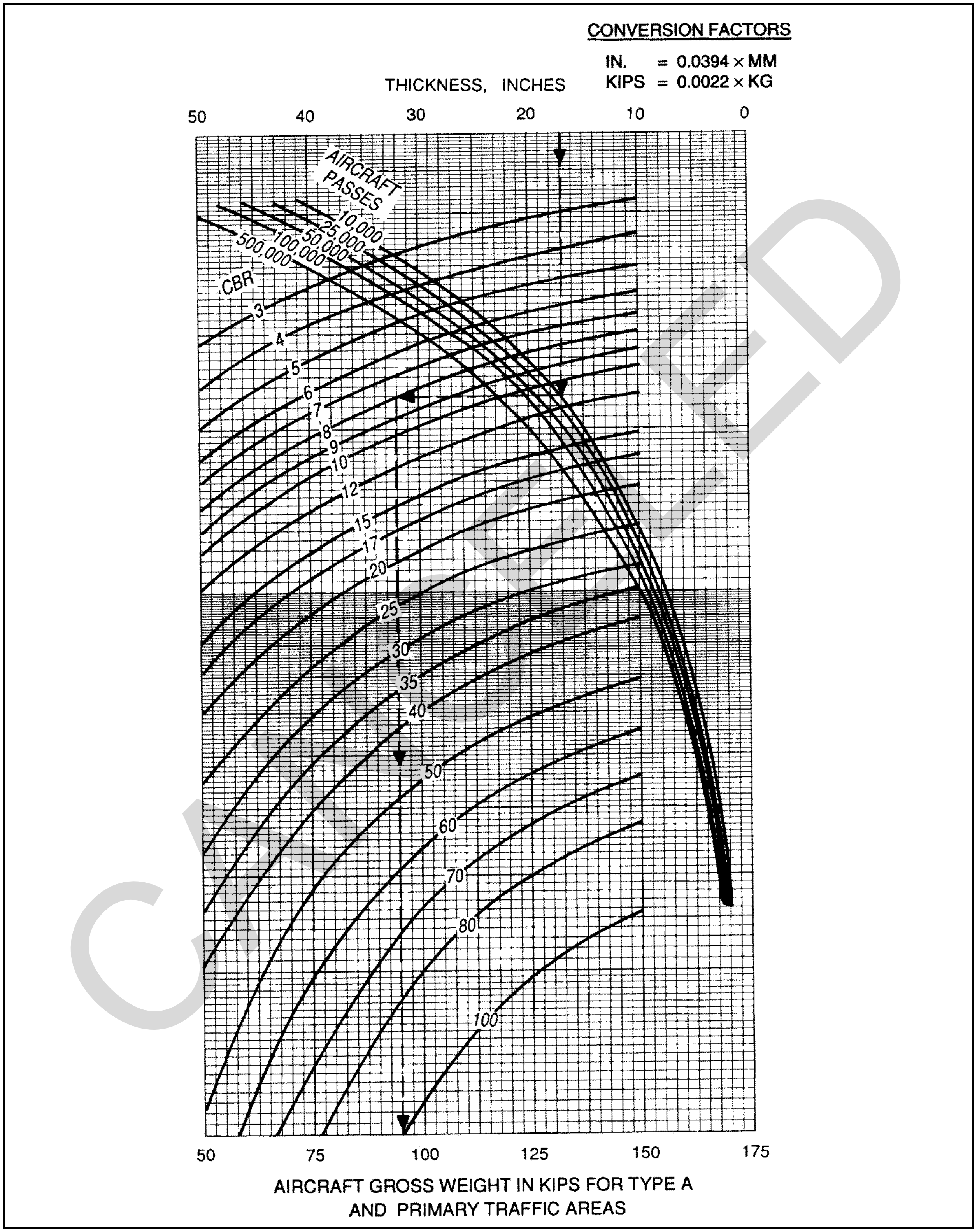


Figure 5-39. Flexible pavement evaluation curve for P-3, primary traffic area

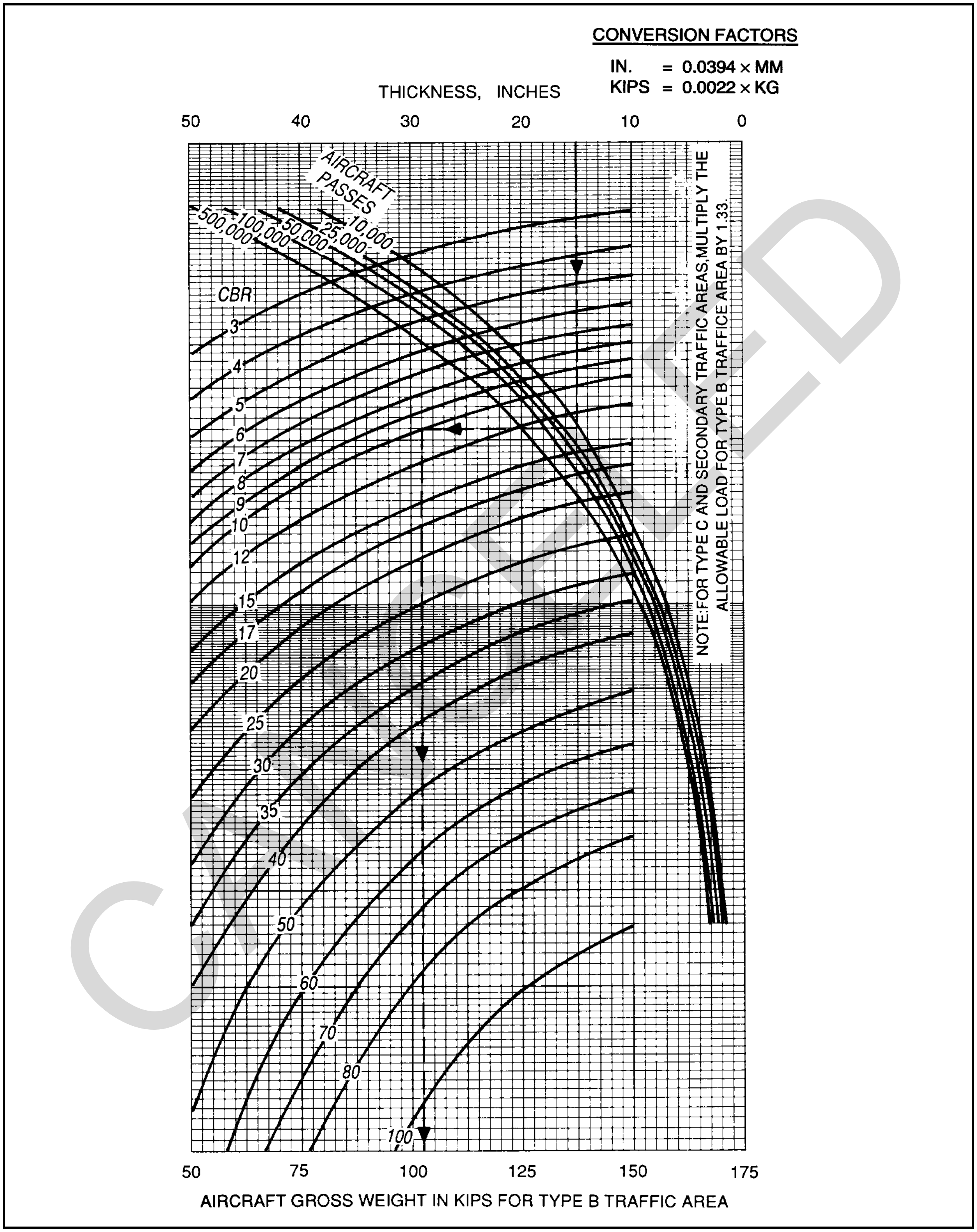


Figure 5-40. Flexible pavement evaluation curve for P-3, secondary traffic area

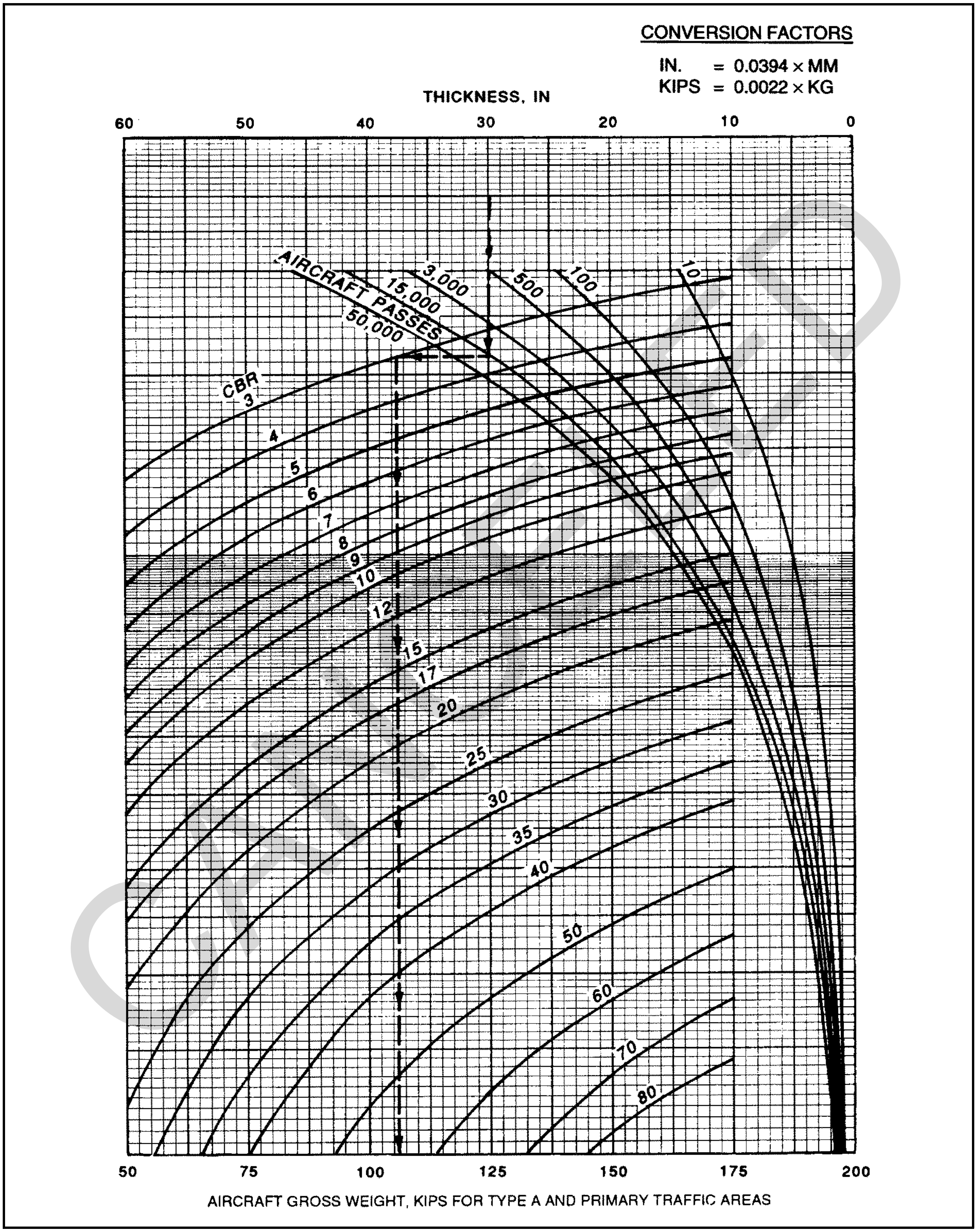


Figure 5-41. Flexible pavement evaluation curve for C-130, primary traffic area

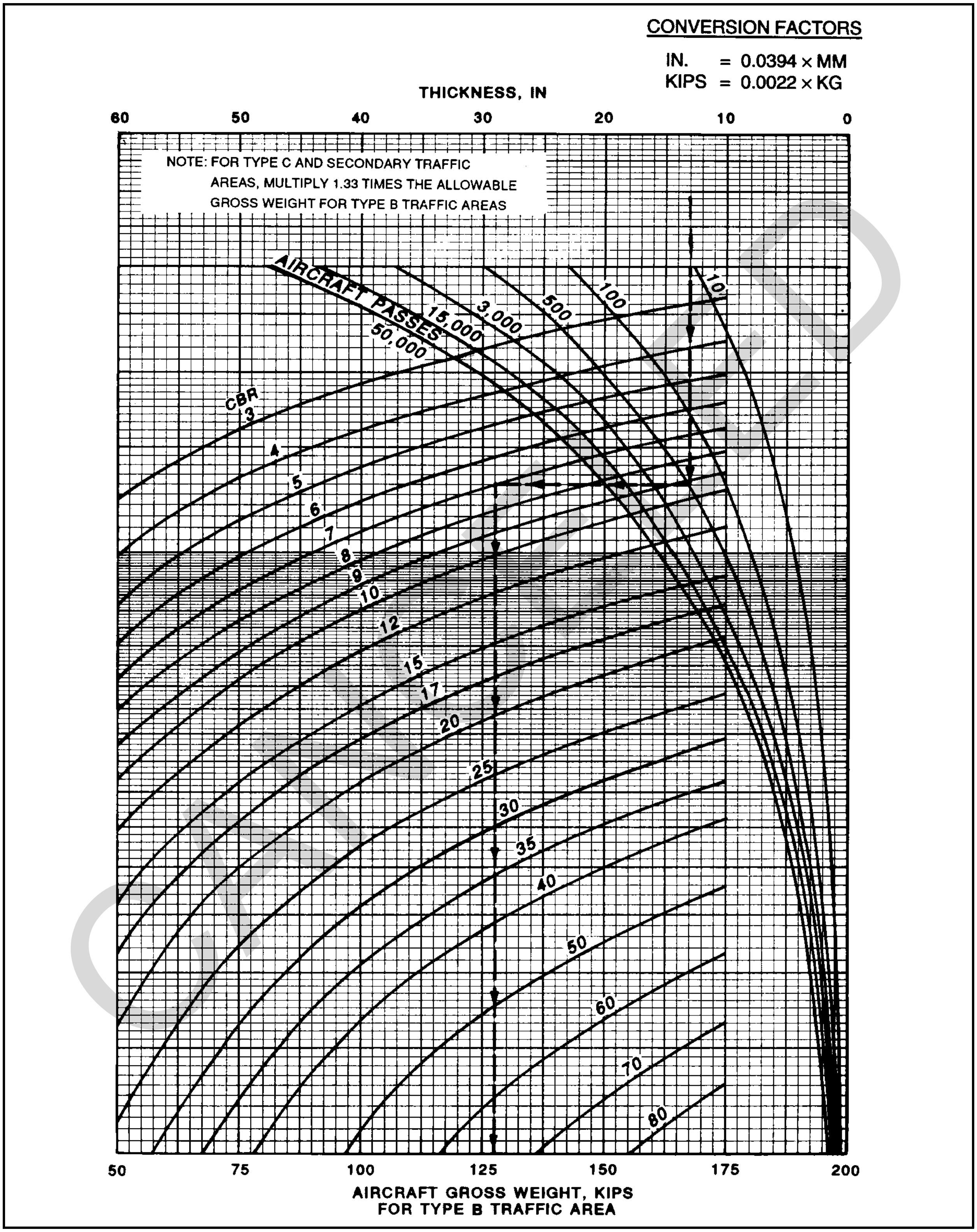


Figure 5-42. Flexible pavement evaluation curve for C-130, secondary traffic area

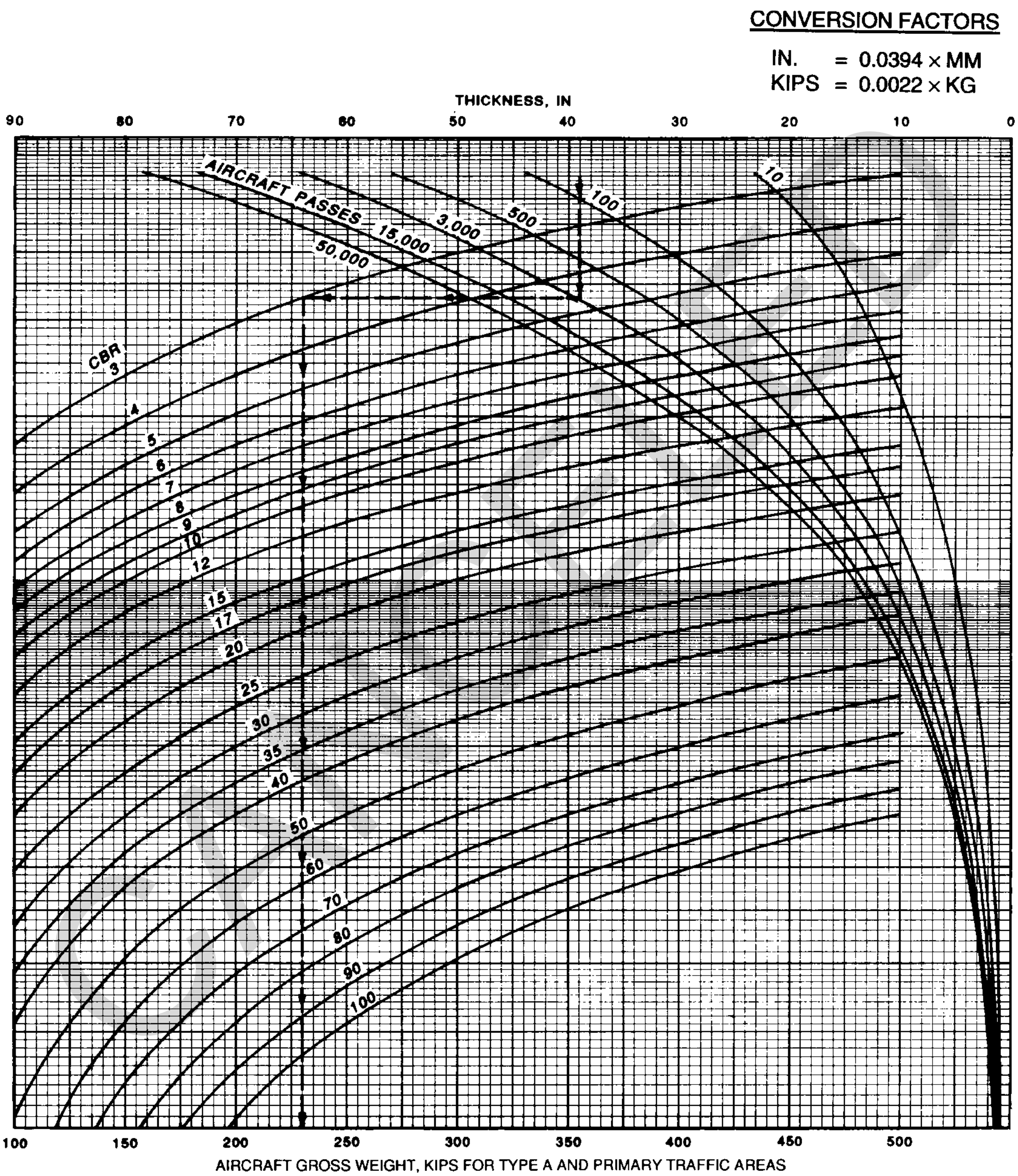


Figure 5-43. Flexible pavement evaluation curve for C-141, primary traffic area

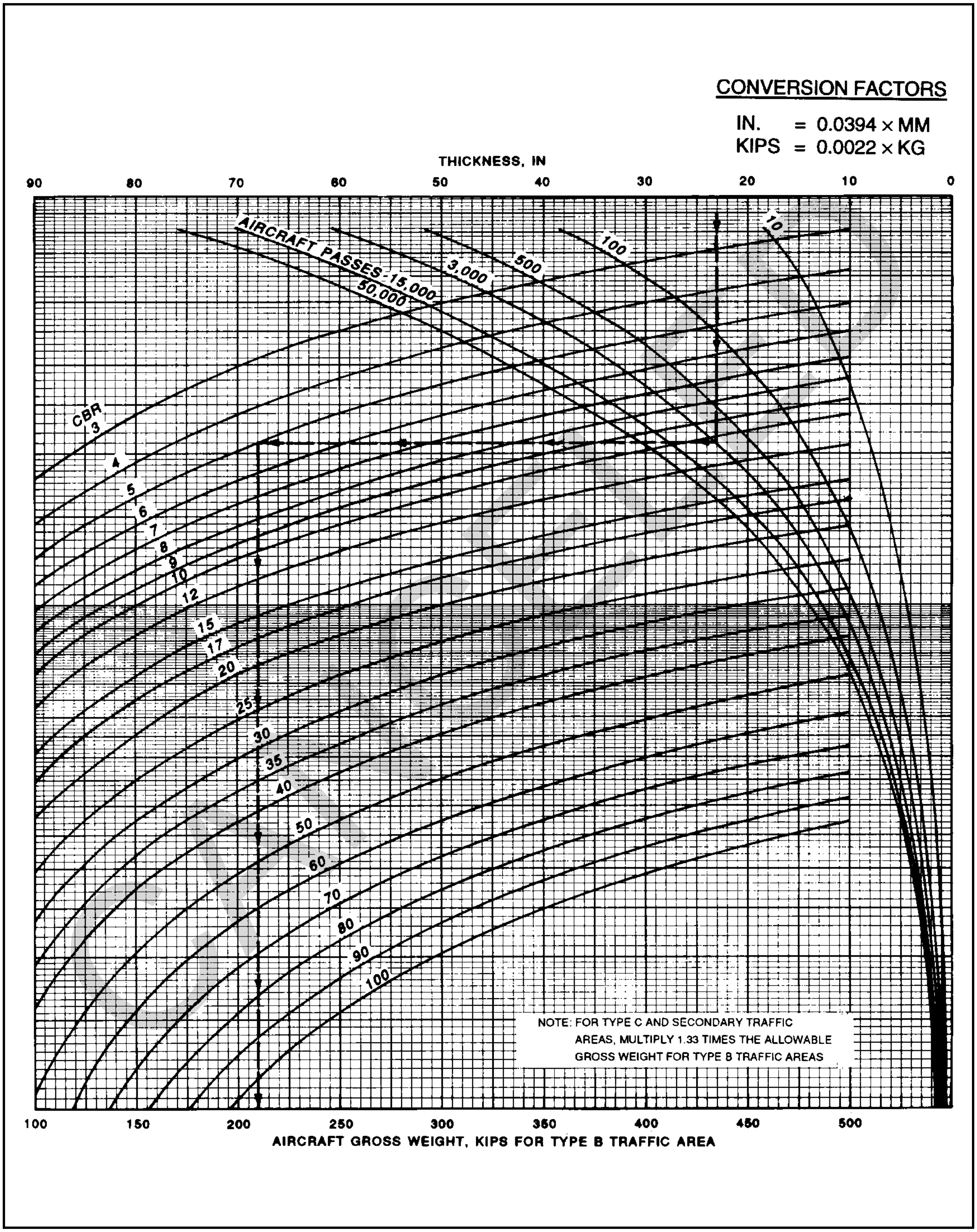


Figure 5-44. Flexible pavement evaluation curve for C-141, secondary traffic area

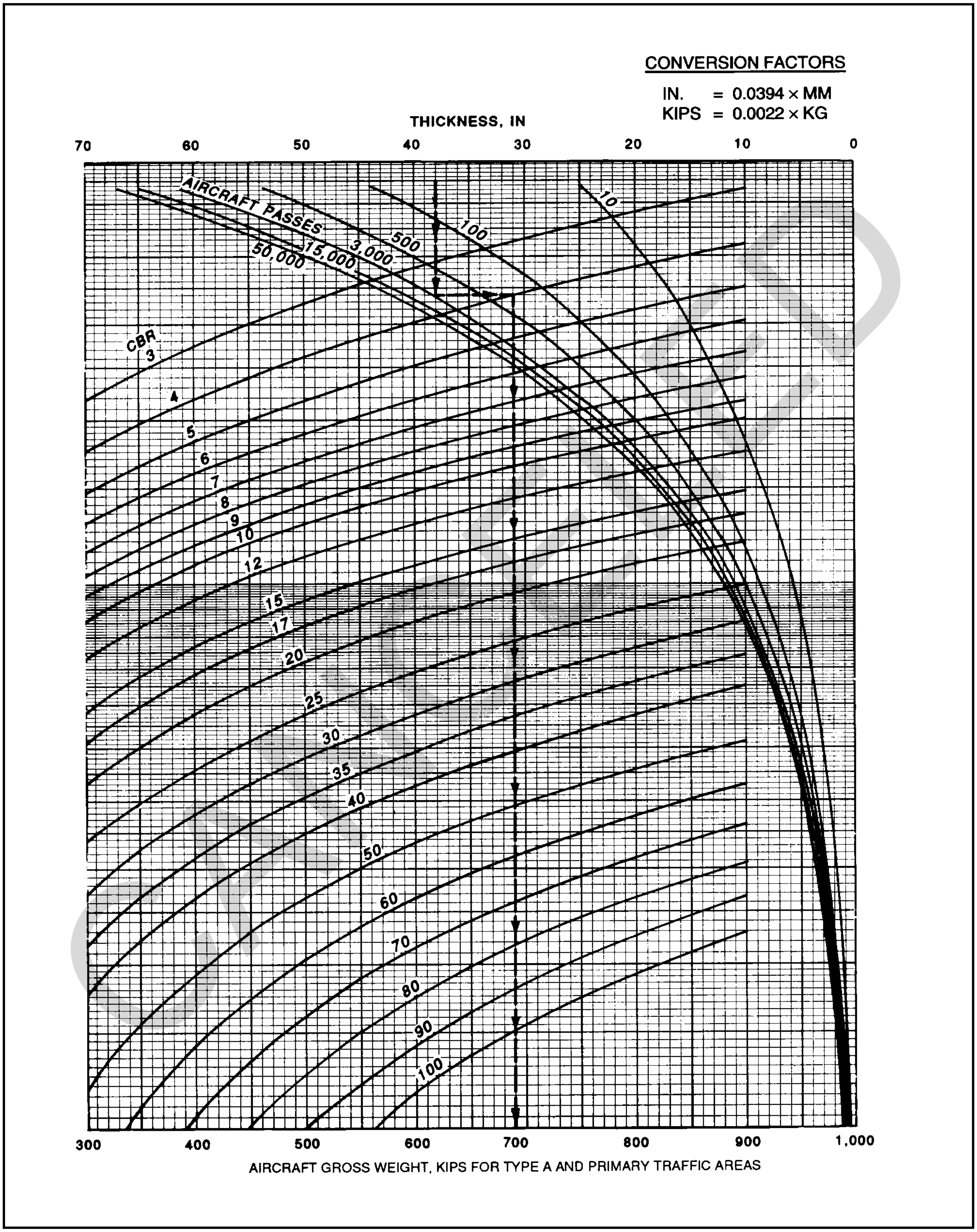


Figure 5-45. Flexible pavement evaluation curve for C-5, primary traffic area

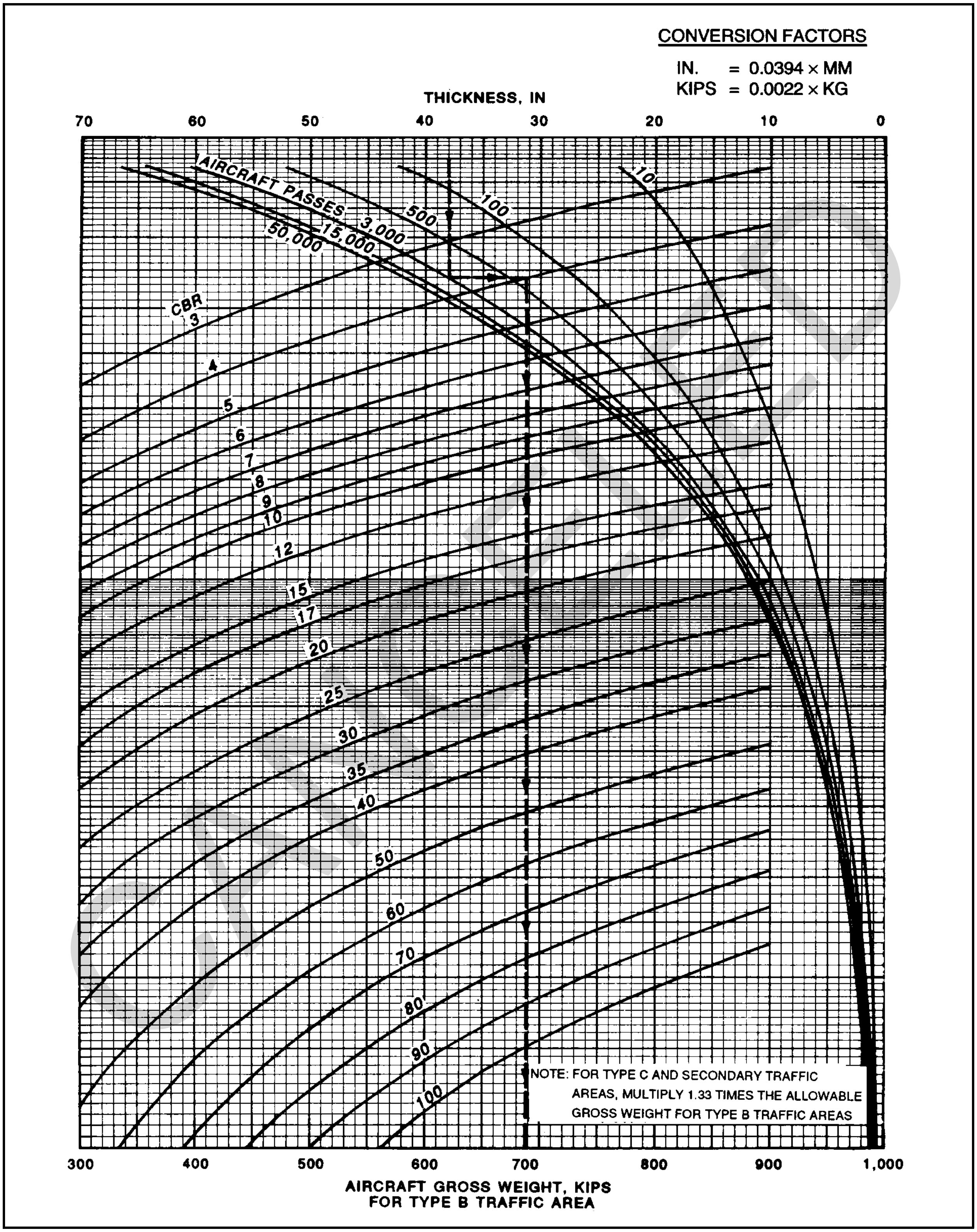
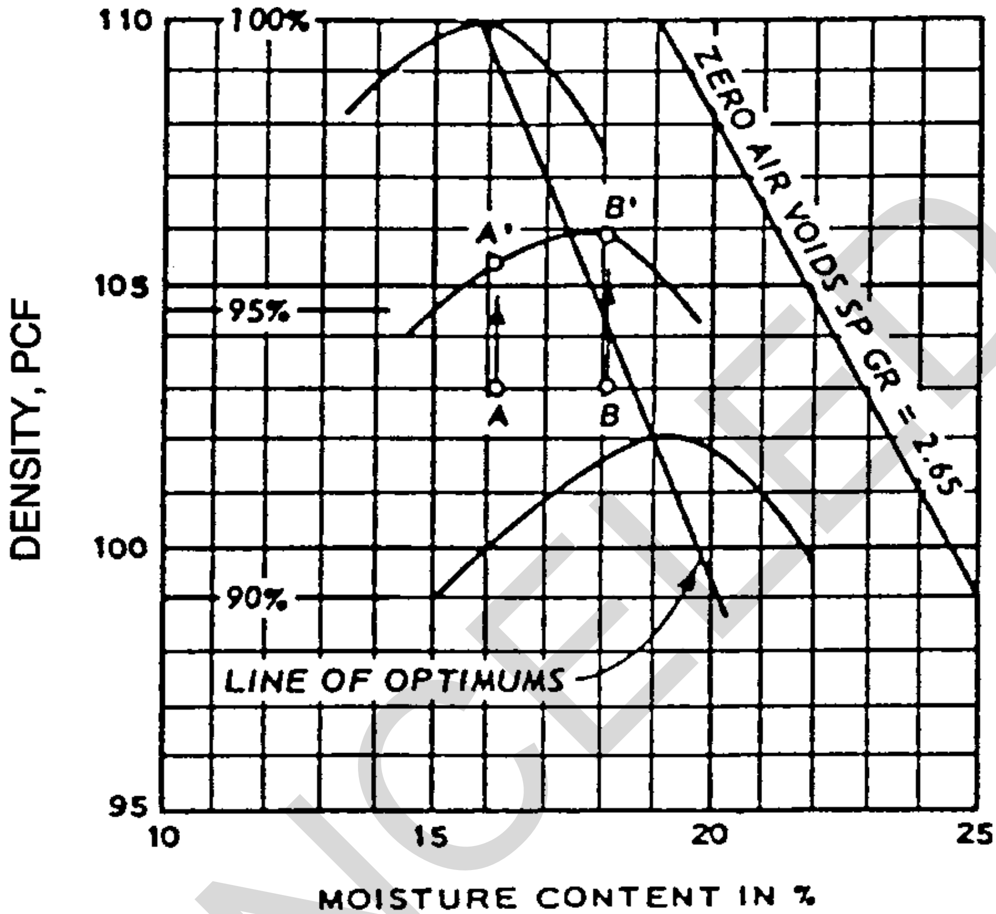


Figure 5-46. Flexible pavement evaluation curve for C-5, secondary traffic area



NOTE: THE SPECIMENS WERE COMPACTED WITH 55-, 26-, AND 12-BLOW EFFORTS, RESPECTIVELY

CONVERSION FACTOR

PCF = 0.0625 × KG/M^3

Figure 5-47. Comparison of field and laboratory densities and moisture contents

CHAPTER 6

EVALUATION OF RIGID PAVEMENTS USING DIRECT SAMPLING

1. GENERAL. This chapter presents criteria for evaluating all types of rigid pavements and overlays using data from direct sampling. The data required for the evaluations were presented in chapter 3. Computer programs are available to assist in a pavement evaluation and are discussed in chapter 9.

2. FACTORS LIMITING LOAD-CARRYING CAPACITY. The load-carrying capacity 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.

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 6.5 millimeters (1/4 inch).

4. SELECTION OF STRENGTH VALUES.

a. Concrete Flexural Strength, R.

(1) The R value to be used for each feature 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.

(2) When the evaluation is being 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 being based on the results of tests conducted at the time of evaluation or when tests are being 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 feature 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 TI 825-01/AFM 32-1124 (I)/NAVFAC DM 21.10, for Army and Air Force and MIL-HDBK-1021/4 for Navy and Marine Corps. 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 of 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 feature. 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 and equilibrium moisture content.

(2) The pavement foundation can be investigated using an ECP or a DCP. Each test device can be used to determine the bearing capacity of a pavement structure at various depths. The ECP device is mounted in a test van and consists of a cone-tipped rod that is hydraulically pushed into the ground at a rate of 20.3 millimeters (0.8 inches)/sec. The DCP is a hand-held device that drives a cone-tipped rod into the ground by repeatedly dropping a 7.97-kilogram (17.6-lb) hammer. Penetration measurements and hammer blow counts are typically made at 25.4-millimeter (1-inch) penetration intervals using the DCP. The ECP device automatically records the penetration data into a computer system. Both devices correlate the rod penetration to resistance strength (CBR) by using derived correlations. The CBR of the pavement layers can be converted to Young's Modulus by multiplying it by 1500 or can be converted to k by using Figure 3-5. Detailed test procedures and correlations for using the ECP and DCP devices are provided in Appendix B.
.

d. 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 of facility.

(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 or failed 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.

5. PLAIN CONCRETE PAVEMENTS. Plain concrete pavements may be evaluated using stresses due to load at the edge of a slab (used by the Army and Air Force) or stresses due to load at the interior of a slab (used by the Navy and Marine Corps).

a. Edge Loading Condition. When using the 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 airfield pass/load relationships are to be reported for both criteria. Air Force evaluations are to be reported using the extended life criteria.

(1) Standard Evaluation. The standard evaluation criteria are essentially the reverse of design and are based upon a 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).

b. Interior Load Condition. The interior load condition consists of only one criterion that is used as the basis of Navy and Marine Corps evaluations.

c. 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, millimeters (inches)

h_e = thickness of concrete pavement, millimeters (inches)

h_s = thickness of stabilized layer, millimeters (inches)

E_c = modulus of elasticity of concrete, usually taken to be 27,580 MPa (4,000,000 psi)

E_s = flexure modulus of elasticity of the stabilized layers, MPa (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.

d. Method of Evaluation. After the existing thickness or equivalent thickness, flexural strength, and modulus of soil reaction have been determined, the evaluation of a pavement using edge loading criteria (Army and Air Force) is made from figures 6-1 through 6-18 along with figures 6-19 through 6-36 for standard evaluations and figures 6-37 through 6-54 for extended life evaluations. Evaluation of a pavement using interior load criteria (Navy and Marine Corps) is made from figures 6-55 to 6-59.

e. Army or Air Force Evaluation Example for Plain Concrete. This example uses an Air Force group for evaluation, but the procedures are applicable to Army and Air Force. Assume:

(1) Airfield runway and parking apron having uniform thickness h_e = 305 millimeters (12 inches).

Table 6-1
Suggested E_s Values for Stabilized Layers (Use as a guide when values are not available.)

Compressive Strength, MPa (psi)	Modulus of Elasticity, MPa (psi)
3.5 - 5.2 (500 - 750)	3,448 (50,000)
5.2 - 6.9 (750 - 1,000)	5,515 (800,000)
6.9 - 10.3 (1,000 - 1,500)	8,275 (1,200,000)
10.3 - 13.8 (1,500 - 2,000)	11,032 (1,600,000)
Over 13.8 (Over 2,000)	13,790 (2,000,000)

(2) Concrete flexural strength R = 4.8 MPa (700 psi).

(3) Subgrade modulus of soil reaction k = 81.3 kPa/mm (300 pci).

(4) Condition of pavement = excellent with adequate load transfer at the joints.

The runway evaluation is based on the thickness of the pavement in the center 23-meter (75-foot) width of the runway, and the runway is divided into two traffic areas. The first 305 meters (1,000 feet) at the ends of the runway are type A traffic areas, the runway interior is a type C traffic area, and the parking apron is a type B traffic area.

(a) Problem 1. Determine the extended life evaluation for Air Force Group Index 14 on the types A, B, and C traffic areas in terms of the allowable gross weight. The allowable gross weight is determined as illustrated in the following tabulation:

Pass Levels	Design Factor[1] Type A Traffic	Type B and C Traffic	Load Factor[2] 1,000 kg (Kips)	Allowable Gross Weight 1,000 kg (Kips)[3] Type A Traffic	Type B Traffic	Type C Traffic
100	0.578	0.553	181(400)	313(690)	329(725)	438(965)
500	0.752	0.728	181(400)	240(530)	250(550)	333(735)
3,000	0.946	0.922	181(400)	193(425)	197(435)	263(580)
15,000	1.120	1.099	181(400)	163(360)	166(365)	220(485)

[1]From figure 6-54.
[2]From figure 6-18.
[3]Divide load factor by design factor for type A and B traffic areas. For type C traffic area, multiply allowable load for type B traffic by 1.33.

(b) Problem 2. Determine the standard evaluation for Air Force Group 14, using the above conditions, in terms of allowable passes. The allowable number of passes for several gross weights are determined as shown in the following tabulation:

Aircraft Gross Weight 1,000 kg (Kips)[1]	Weight for Evaluating Traffic Area C 1,000 kg (Kips)[2]	Load 1,000 kg (Kips) Factor[3]	Design Factor[4] Traffic Area A and B	Traffic Area C	Allowable Passes[5] Traffic Area A	Traffic Area B	Traffic Area C
218(480)	163(360)	181(400)	0.833	1.111	90	110	1,400
200(440)	150(330)	181(400)	0.909	1.212	170	235	3,810
181(400)	136(300)	181(400)	1.000	1.333	400	541	11,630
163(360)	122(270)	181(400)	1.111	1.481	1,105	1,502	45,515
136(300)	102(225)	181(400)	1.333	1.777	8,550	11,630	697,150

[1]Gross weight for which the evaluation is being made.
[2]Weight to be used in evaluating type C traffic areas is the aircraft gross weight times 0.75.
[3]From figure 6-18.
[4]Load factor divided by aircraft weight.
[5]From figure 6-36.

f. Navy and Marine Corps Evaluation Example for Plain Concrete.

(1) Airfield runway having a uniform thickness of 229 millimeters (9 inches).

(2) Concrete flexural strength of 4.8 MPa (700 psi).

(3) Subgrade modulus of reaction of 81 kPa/mm (300 pci).

(4) 100,000 passes.

(5) Condition of pavement is excellent.

The evaluation is based on the thickness of the pavement in the center 23-meter (75-foot) width of the runway and the runway is divided into two traffic areas. The first 305 meters (1,000 feet) at the ends are primary traffic areas and the runway interior is a secondary traffic area. The evaluation will be made using figure 6-55. Enter the figure with the given parameters as shown by the arrows and read an allowable gross weight of 27,700 kilograms (61,000 pounds) for the primary traffic area and 29,500 kilograms (65,000 pounds) for the secondary traffic area.

6. REINFORCED CONCRETE PAVEMENT. 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, nonreinforced 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. However, for Navy and Marine Corps pavements, the thickness of reinforced concrete is treated as if it was not reinforced and is evaluated as a plain concrete pavement.

(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 6-60, 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:

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

where

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

A_p = cross-sectional area of pavement per foot of pavement width or length, square millimeters (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 6-60 with the known value of h_r. 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 6-60, 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 6-60. 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 = 305 millimeters (12 inches).

(3) Diameter of steel reinforcing bars, both longitudinal and transverse = 9.5 millimeters (3/8 inch).

(4) Center-to-center spacing of reinforcing bars, both longitudinal and transverse = 152 millimeters (6 inches).

(5) Flexural strength of concrete = 4.8 MPa (700 psi).

(6) The k value for the foundation material = 27 kPa/mm (100 pci).

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

$$S = \frac{A_s}{A_p} \times 100 = \frac{142}{93{,}025} = 0.153 \textit{ percent, using SI units}$$

$$S = \frac{A_s}{A_p} \times 100 = \frac{0.221}{144} \times 100 = 0.00153 \times 100 = 0.153 \textit{ percent using IP units}$$

where

$$A_s = \frac{3.1416(9.5)^2 \times (2)}{4} = 142 \textit{ square millimeters, using SI units}$$

$$A_s = \frac{(3.1416)\ (0.375)^2 \times (2)}{4} = 0.221 \textit{ square inches using IP units}$$

$$A_p = 305 \times 305 = 93{,}025 \textit{ square millimeters, using SI units}$$

$$A_p = 12 \times 12 = 144 \textit{ square inches, using IP units}$$

Since h_r = 305 millimeters (12 inches) and S = 0.153 percent, figure 6-55 shows the corresponding h_E value to be 366 millimeters (14.4 inches). This h_E value is then used to determine the evaluation in the same manner as a plain concrete pavement.

7. RIGID OVERLAY ON RIGID PAVEMENT.

a. Data Required. The data required for the evaluation of a rigid overlay on rigid pavement do 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 or referenced in appendix B.

b. Method of Evaluation. The first step in the evaluation of a rigid overlay on a rigid pavement is the determination of the equivalent thickness of the combined section of the rigid overlay and the rigid base pavement. The equivalent thickness, which is defined as a single thickness of plain concrete pavement having the same load-carrying capacity as the combined thickness of the rigid overlay and the rigid base pavement, can be determined as follows:

(1) 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:

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

where

h_o = thickness of rigid overlay pavement, millimeters (inches)

C = coefficient representing condition of rigid base pavement

h_b = thickness of rigid base pavement, millimeters (inches)

(2) 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:

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

No credit is given to the thickness of the bond breaker if less than 102 millimeters (4 inches). If the thickness of the bond breaker is greater than 102 millimeters (4 inches), then the pavement will be evaluated as a composite pavement.

(a) The value of C in equations 6-3 and 6-4 depends on the condition of the existing rigid base pavement. The following C values are recommended:

C = 1.00 for base pavement in good condition

C = 0.75 for base pavement having a few initial cracks due to loading, but no progressive cracks

C = 0.35 for badly cracked base pavement

Other values for C can be used; however, since guidance is not provided, engineering judgment must be applied when selecting values other than those listed above.

(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 would be the weighted average of the overlay and base pavement strengths, determined as follows:

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

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

c. Evaluation Example. Determine an extended life evaluation in terms of passes for the C-130 aircraft at a gross weight of 61,236 kilograms (135 kips) on a type B traffic area consisting of a 152-millimeter (6-inch) base pavement and a 152-millimeter (6-inch) rigid overlay with no bond breaker.

The weighted average flexural strength is 4.13 MPa (600 psi), and the k value is 13.6 kPa/mm (50 pci). Since the base pavement contains a few initial cracks, the C factor is 0.75. With these data, the evaluation is made in the following manner.

(1) Step 1. Calculate the equivalent thickness, h_E ,using equation 6-3:

$$h_E = \sqrt[1.4]{(152)^{1.4} + C\ (152)^{1.4}} = 225 \ \textit{millimeters, using SI units}$$

$$h_E = \sqrt[1.4]{(6)^{1.4} + C\ (6)^{1.4}} = 8.93 \ \textit{inches using English units}$$

(2) Step 2. Enter figure 6-8 with the k = 13.6 kPa/mm (50 pci), R = 4.13 MPa (600 psi) and equivalent thickness of 8.93 inches and determine the load factor of 57,600 kilograms (127 kips).

(3) Step 3. Divide the load factor by 61,236 kilograms (135 kips) to obtain the design factor of 0.94.

(4) Step 4. Enter figure 6-44 with the design factor of 0.94 and the k = 13.6 kPa/mm (50 pci) for type B traffic area and determine the allowable passes of 850.

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 TI 825-01/AFM 32-1124 (I)/NAVFAC DM 21.10, 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 chapter 6. 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 2.76 MPa (400 psi) or the k value of the foundation is greater than 54 kPa/mm (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 2.76 MPa (400 psi) or the k value is greater than 54 kPa/mm (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:

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

where

t = thickness of nonrigid overlay pavement, millimeters (inches)

h_b = thickness of rigid base pavement, millimeters (inches)

F = a factor which controls the degree of cracking in the rigid base pavement. (figures 6-61 through 6-80)

(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 and traffic intensity in terms of passes. If a k value greater than 135 kPa/mm (500 pci) is established, the F value for a k of 135 kPa/mm (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 6-60 to determine the allowable gross weight at selected pass levels or the allowable number of passes for selected loads. However, the determining of allowable number of passes becomes an iterative procedure since the F factor depends upon the traffic level.

(c) If a condition factor (C) for base pavement is known, then the thickness of the rigid base pavement (h_b) would be multiplied by the condition factor to determine the equivalent thickness. Since the base pavement has been overlaid, the condition of the base pavement would not normally be known.

(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 for Army and Air Force. Evaluate a type A traffic area pavement having a uniform thickness of a nonrigid overlay on a rigid pavement. A standard evaluation is to be accomplished for 50,000 passes of the C-141 aircraft. The pavement consists of a 152-millimeter (6-inch) bituminous overlay, a 152-millimeter (6-inch) rigid base pavement with a 4.1-MPa (600-psi) flexural strength, an 203-millimeter (8-inch) base course having a k of 81 kPa/mm (300 pci) and a CBR of 30, a subgrade with a CBR of 10. Since the k value under the rigid pavement exceeds 54 MN/cubic meter (200 pci), it is necessary to evaluate the pavement by both the nonrigid and the flexible evaluation methods to obtain the highest allowable gross weight at the selected pass level.

(1) Rigid Pavement Evaluation Method. The following steps are followed:

(a) From figure 6-73, determine F to be 0.803.

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

$$h_E = \frac{1}{0.803}\left[0.33(152) + 152\right] = 252 \textit{ millimeters, using SI units}$$

$$h_E = \frac{1}{0.803}\left[0.33(6) + 6\right] = 9.94 \textit{ inches, using IP units}$$

(c) 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, using the k on the base course and figures 6-13 and 6-31, the allowable load would be 100,000 kilograms (220 kips).

(2) Flexible Pavement Evaluation Method. Since the existing thickness of asphalt exceeds the minimum thickness of 102 millimeters (4.0 inches) by 51 millimeters (2.0 inches), the thickness equivalencies of paragraph titled "Selection of Thickness Values" must be applied. The resulting section for the purpose of evaluation of the subgrade is then 102 millimeters (4.0 inches) of asphalt, 152 millimeters (6.0 inches) of 100 CBR base and 320 millimeters (12.6 inches) of 30 CBR subbase on a 10 CBR subgrade. The allowable aircraft gross weight on the subgrade for this condition is 122,000 kilograms (270 kips). To evaluate the subbase, the excessive asphalt is converted to an equivalent thickness of base course resulting in a section above the subbase of 102 millimeters (4.0 inches) of asphalt and 211 millimeters (8.3 inches) of base course. This would result in an allowable load of 212,000 kilogram (467 kips) for the subbase. The 152 millimeters (6.0 inches) of asphalt on the 100 CBR base course would evaluate for 263,000 kilograms (580 kips).

(3) Controlling Evaluation. Regardless of the procedure used, the higher controlling weight of either the flexible or rigid method would be used for the pavement evaluation. For this example, the 122,000 kilograms (270 kips) determined for the subgrade using the flexible pavement method would be the controlling evaluation.

d. Evaluation Example for Navy and Marine Corps. Evaluate a primary traffic area pavement having a uniform thickness of a nonrigid overlay on a plain concrete pavement. The evaluation is to be accomplished for 100,000 passes of the F-14 aircraft. The pavement consists of a 102-millimeter (4-inch) bituminous overlay, a 152-millimeter (6-inch) plain concrete base pavement with a 4.5-MPa (650-psi) flexural strength, and a subgrade with a modulus of subgrade reaction of 81 kPa/mm (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 figure 6-79, determine F to be 0.793.

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

$$h_E = \frac{1}{0.793}\left[0.33(101) + 152\right] = 234 \ \textit{millimeter, using SI units}$$

$$h_E = \frac{1}{0.793}\left[0.33(4) + 6\right] = 9.2 \ \textit{inches, using IP units}$$

(c) 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, using the k on the subgrade and figure 6-55, the allowable load would be 26,400 kilograms (58.2 kips).

(2) 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 102 millimeters (4 inches) of asphalt concrete, 152 millimeters (6 inches) of 100 CBR base course, and a 20 CBR subgrade. Using figure 5-35, the allowable load for the subgrade is 22,200 kilograms (49 kips) and for the base course is 40,800 kilograms (90 kips). The controlling load for the flexible pavement is therefore 22,200 kilograms (49 kips).

(3) Controlling evaluation. In evaluating flexible overlays on rigid pavements, the larger of the controlling loads for the flexible and rigid evaluation controls the overall pavement evaluation and is therefore 26,400 kilograms (58.2 kips).

9. RIGID OVERLAY OF 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 135 kPa/mm (500 pci) be used.

(2) When the temperature of the existing bituminous pavement surface is above 24 degrees Centigrade (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 24 degrees centigrade (75 degrees Fahrenheit), run the tests on the asphaltic concrete pavement. Compare the value from the test with the value from figure 3-6, then select the smallest value to use. Figure 3-6 may also be used as an alternative method for determining the k value on the flexible pavement.

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.

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 102 millimeters (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 102 millimeters (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 102 millimeters (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 101 millimeters (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.

11. FIBROUS REINFORCED CONCRETE PAVEMENTS.

a. Data Required. The data required for the evaluation of fibrous reinforced concrete pavements as presented in chapter 3 do not differ greatly from that required for plain rigid pavements. Generally, fibrous reinforced pavements are used for overlays because of the thin sections of pavements that can be used, and the evaluation would be the same as that outlined for a rigid overlay over rigid pavements in this chapter. The determination of the flexural strength value of fibrous concrete (ACI 544.2 R) is slightly different than that for plain concrete. The flexural strength value is normally higher on the stress-strain curve than the value selected for plain concrete specimens.

b. Method of Evaluation. Fibrous concrete slabs on grade will be evaluated in the same manner as plain concrete slabs. A load factor is determined from figures 6-1 to 6-18. The load factor is then divided by the aircraft weight in kips to determine a design factor. The design factor is then used with figures 6-81 to 6-90 to determine the allowable number of passes. The allowable gross weight in kips is determined by dividing the load factor from figures 6-1 to 6-18 by the design factor from figures 6-81 to 6-90. A fibrous concrete overlay pavement will be evaluated by determining an equivalent thickness of concrete pavement according to equation 6-3 or 6-4 and then evaluating as a slab on grade.

12. PAVEMENT CLASSIFICATION NUMBER. In addition to evaluating airfield pavements for allowable load, 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. Values of ACN for given aircraft can be calculated from the computer program ACN that can be obtained from the Army Corps of Engineers Transportation System Center, as discussed in chapter 9. The PCN is presented in chapter 8.

13. 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 frost area index of reaction as given in chapter 7 of this manual.

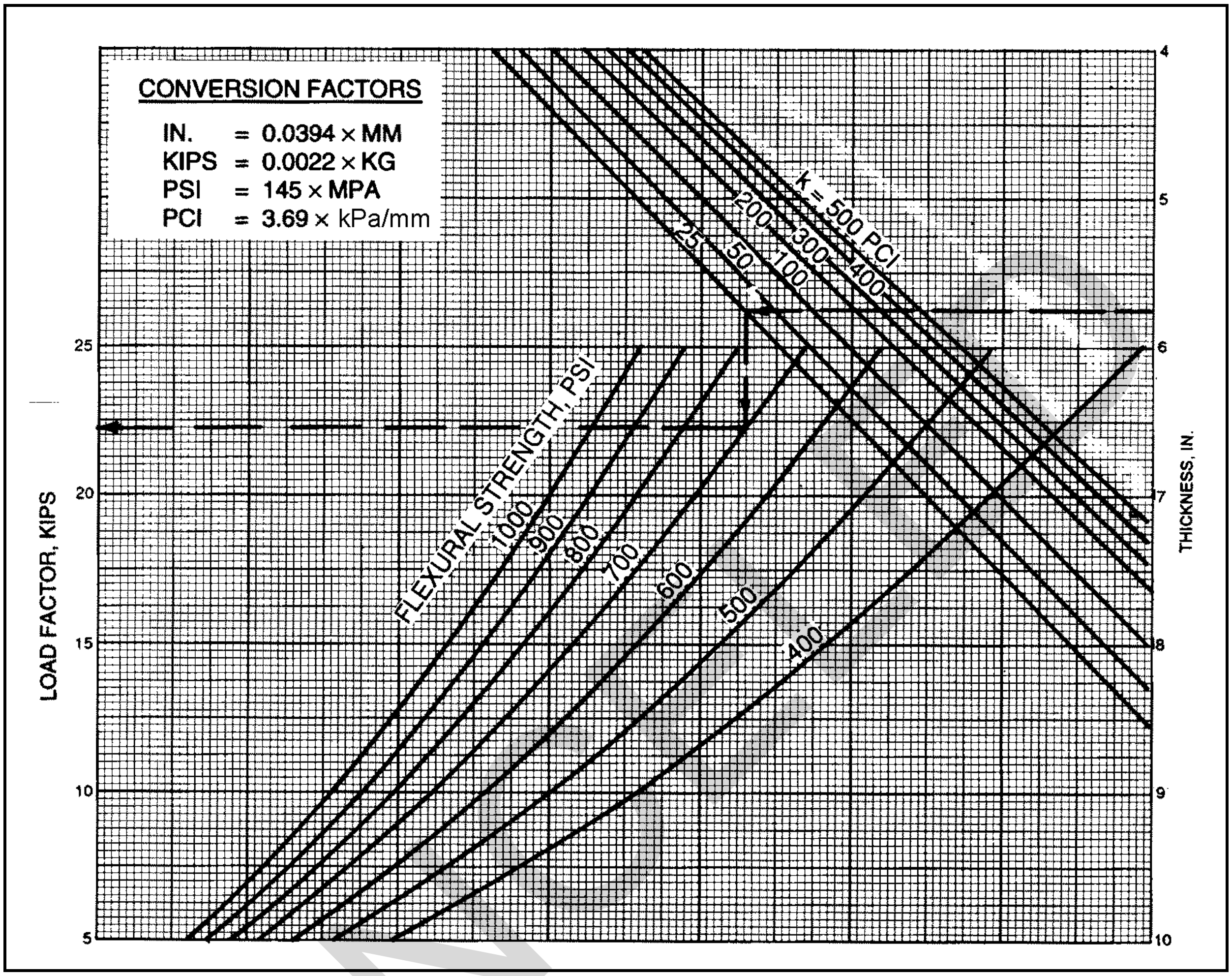


Figure 6-1. Load factor curves for rigid pavement evaluation, UH-60

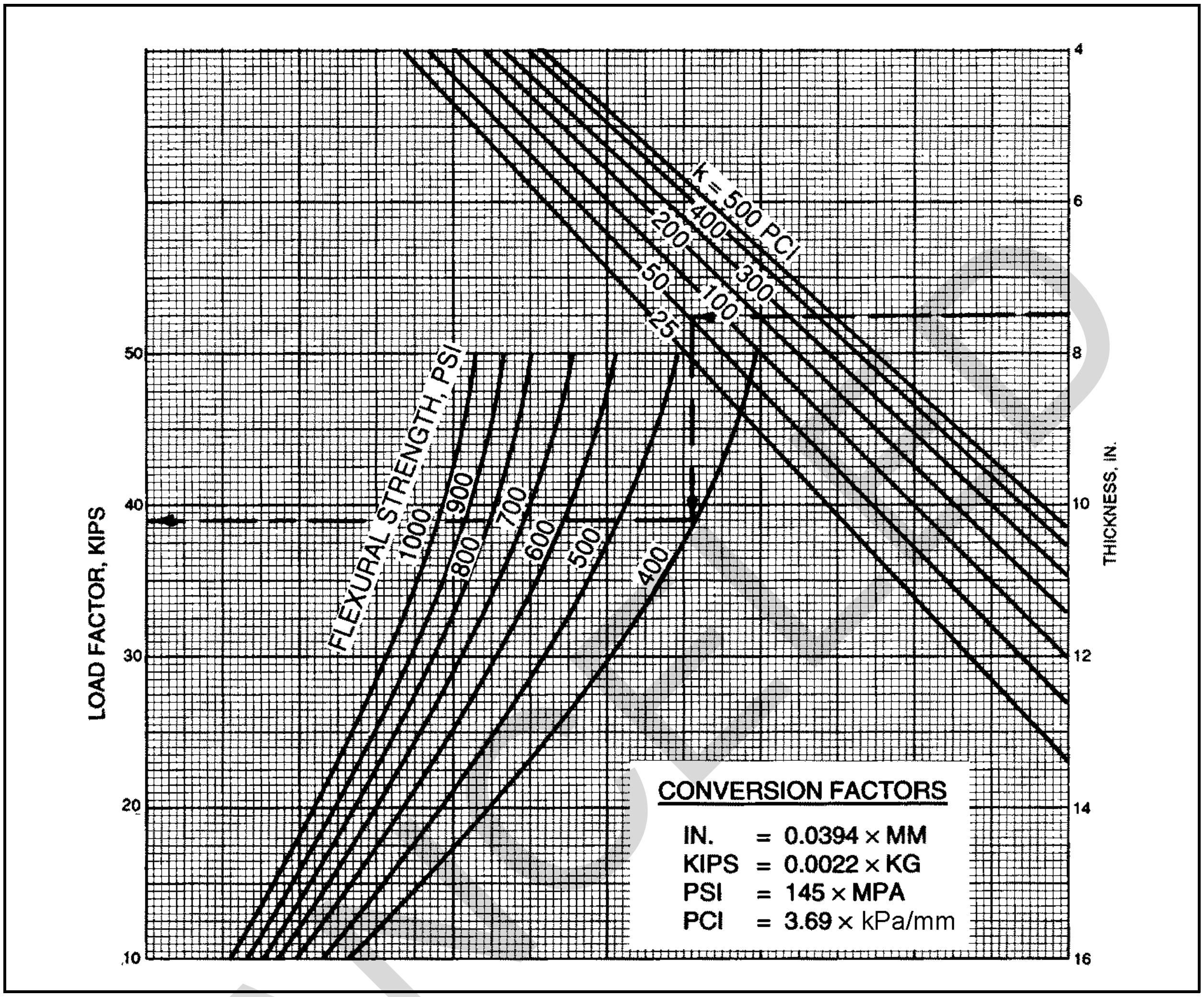


Figure 6-2. Load factor curves for rigid pavement evaluation, CH-47

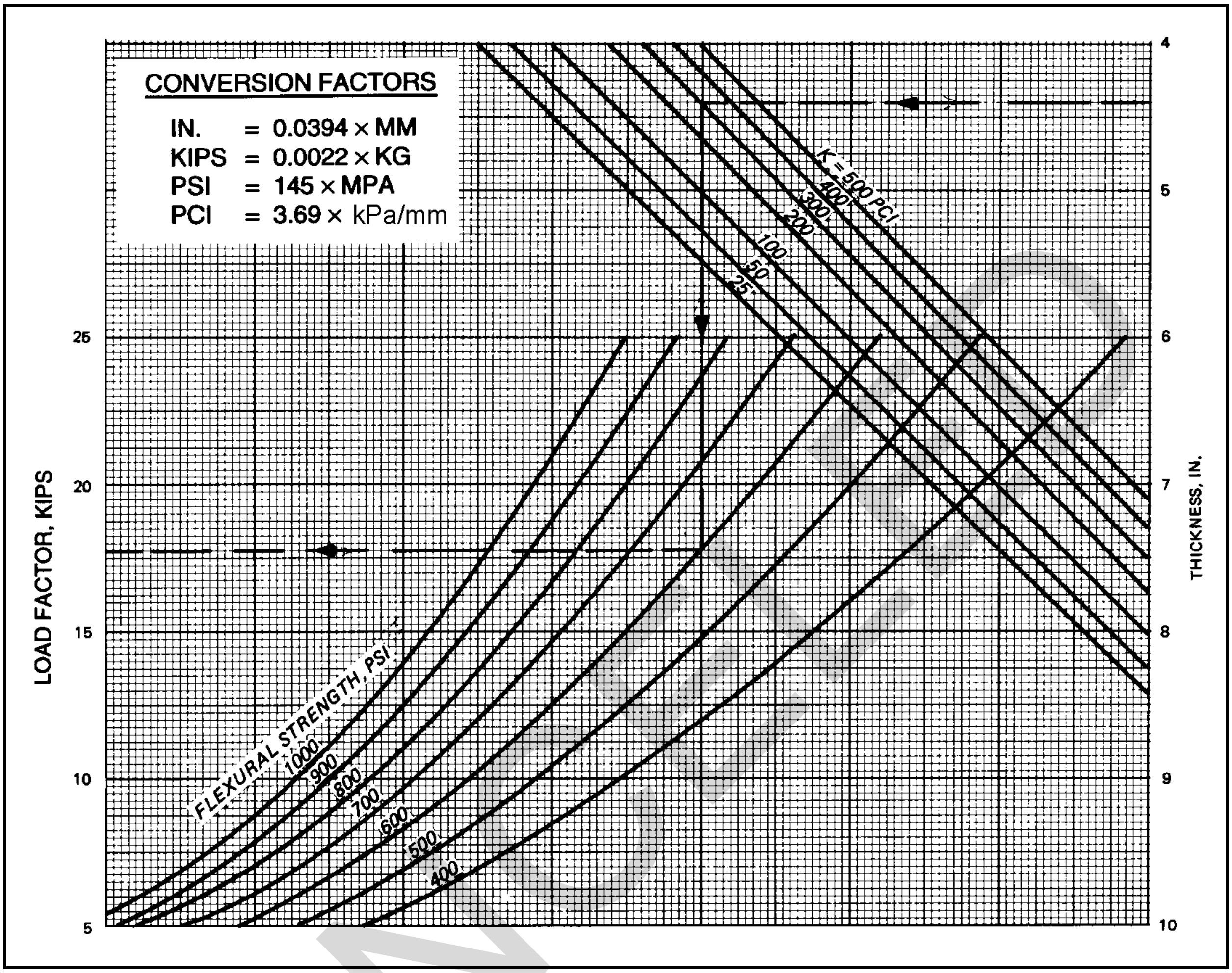


Figure 6-3. Load factor curves for rigid pavement evaluations, OV-1

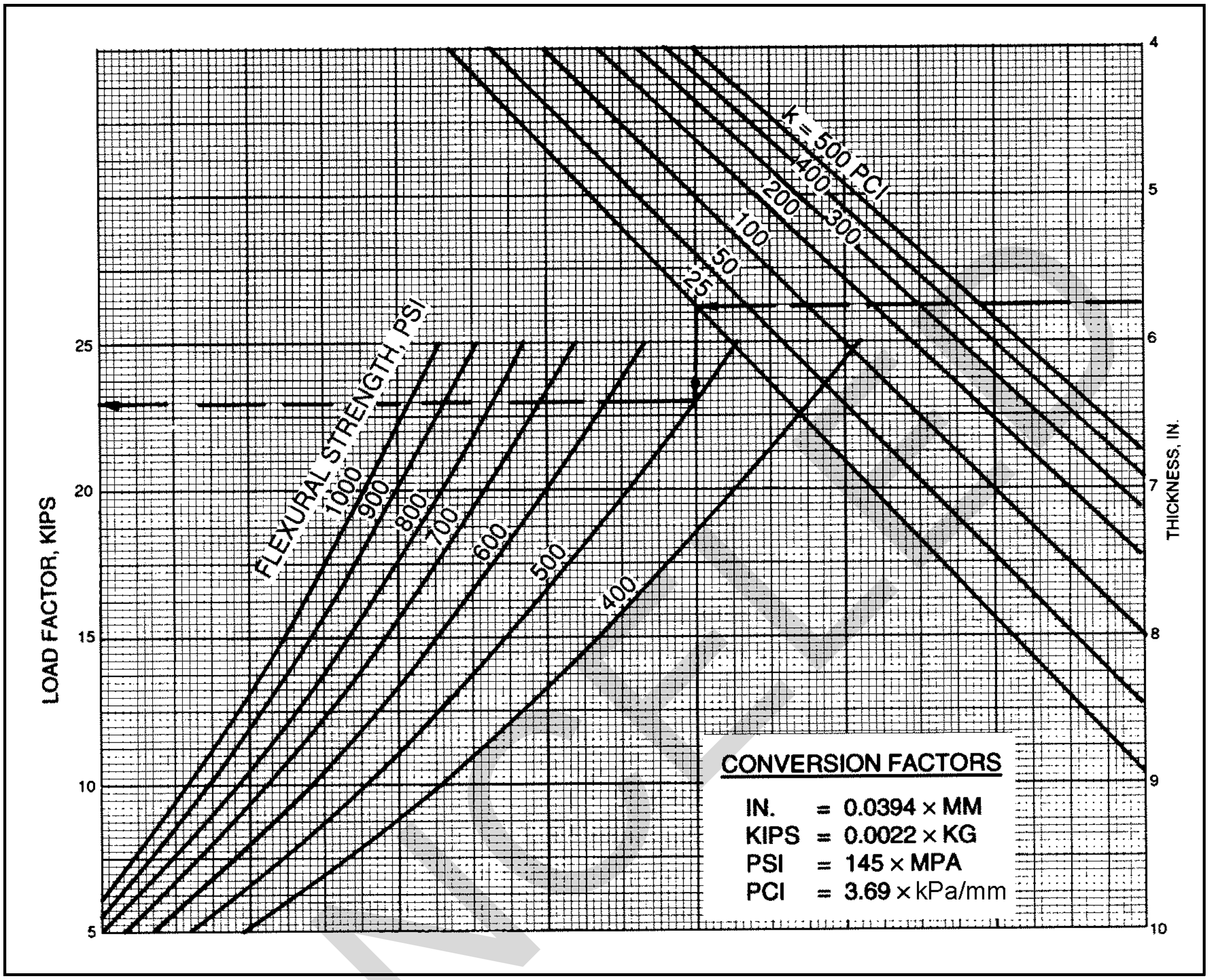


Figure 6-4. Load factor curves for rigid pavement evaluation, C-12

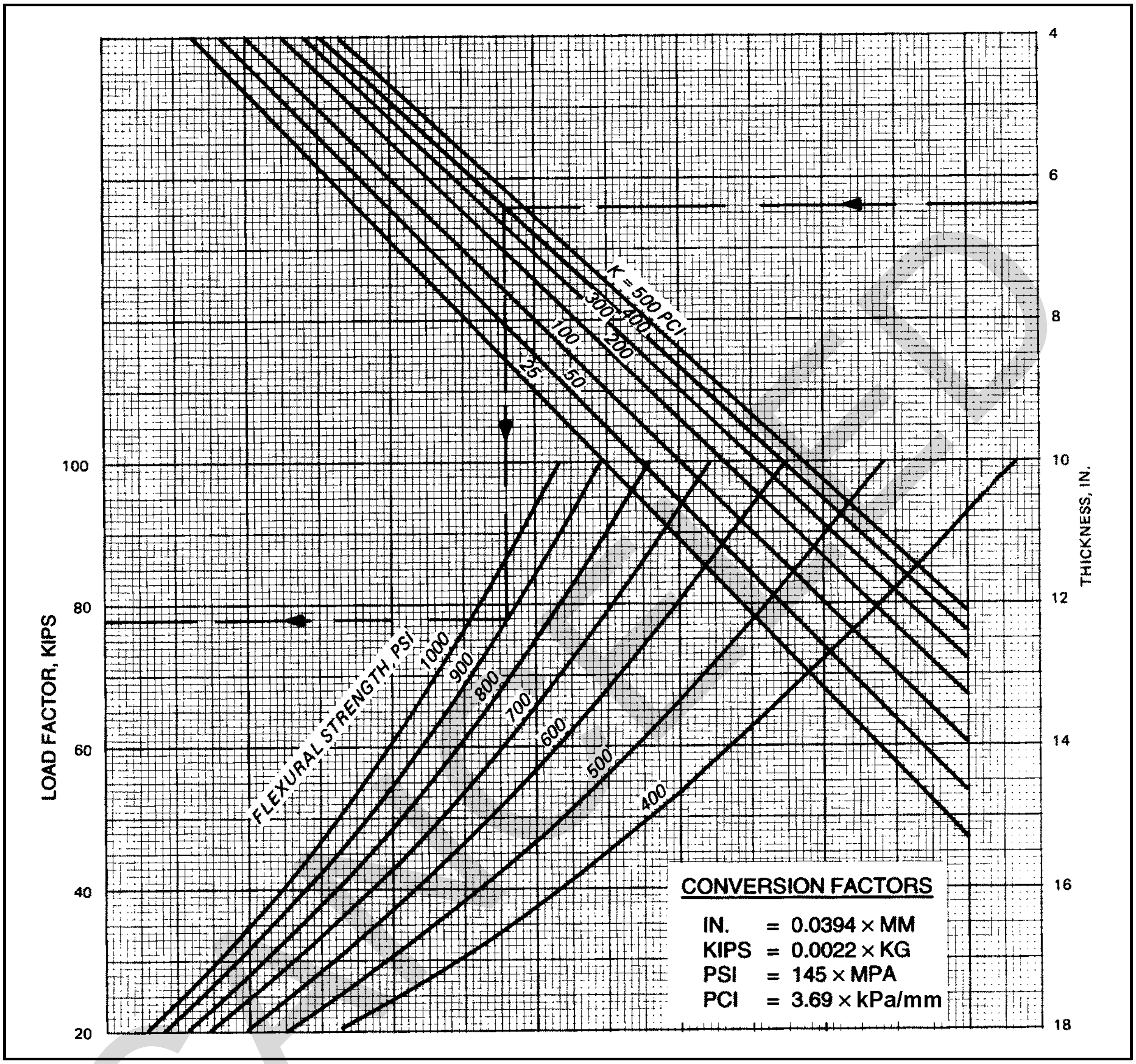


Figure 6-5. Load factor curves for rigid pavement evaluation, C-23 (Air Force Group Index 1)

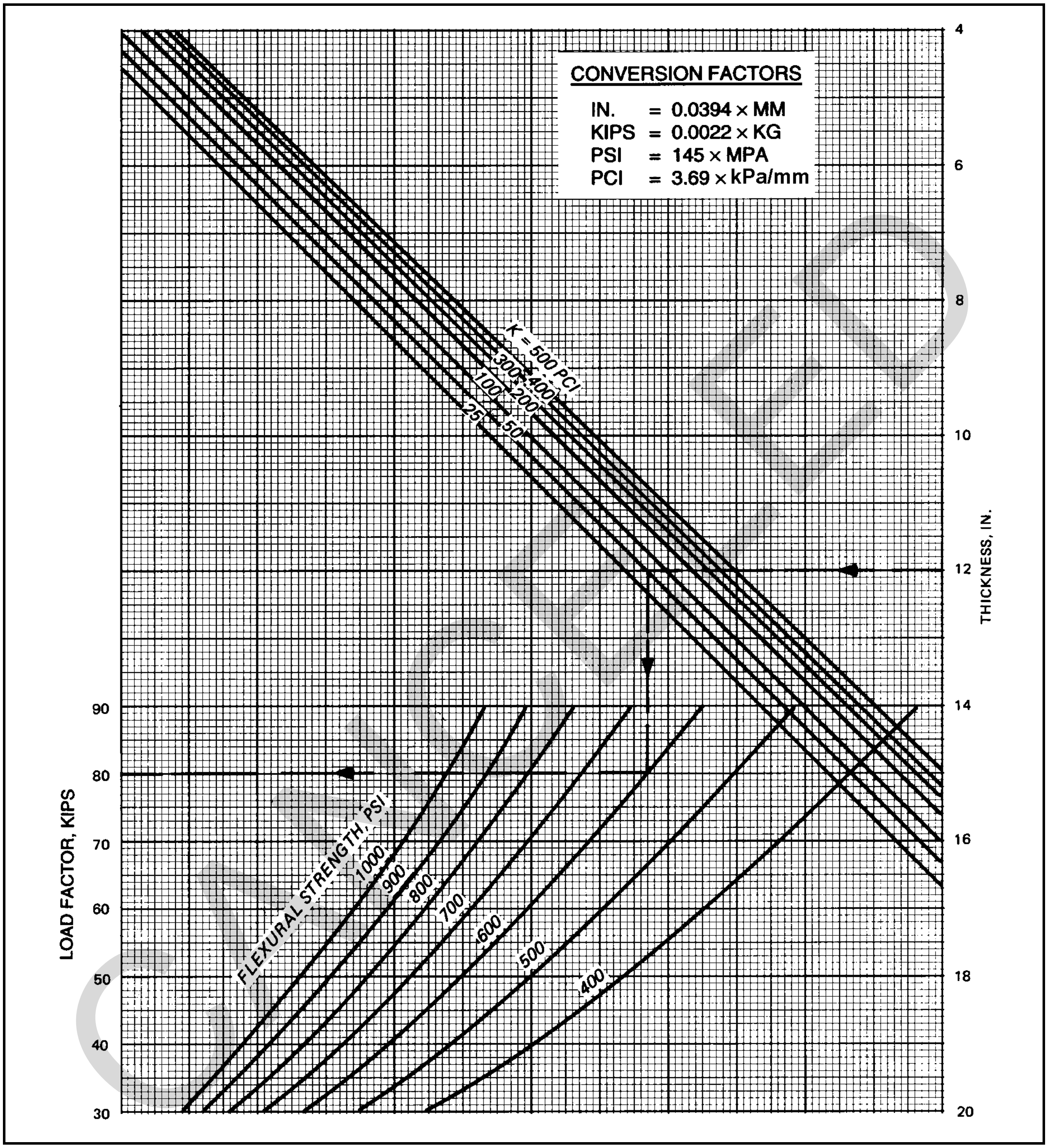


Figure 6-6. Load factor curves for rigid pavement evaluation, F-4 (Air Force Group Index 2)

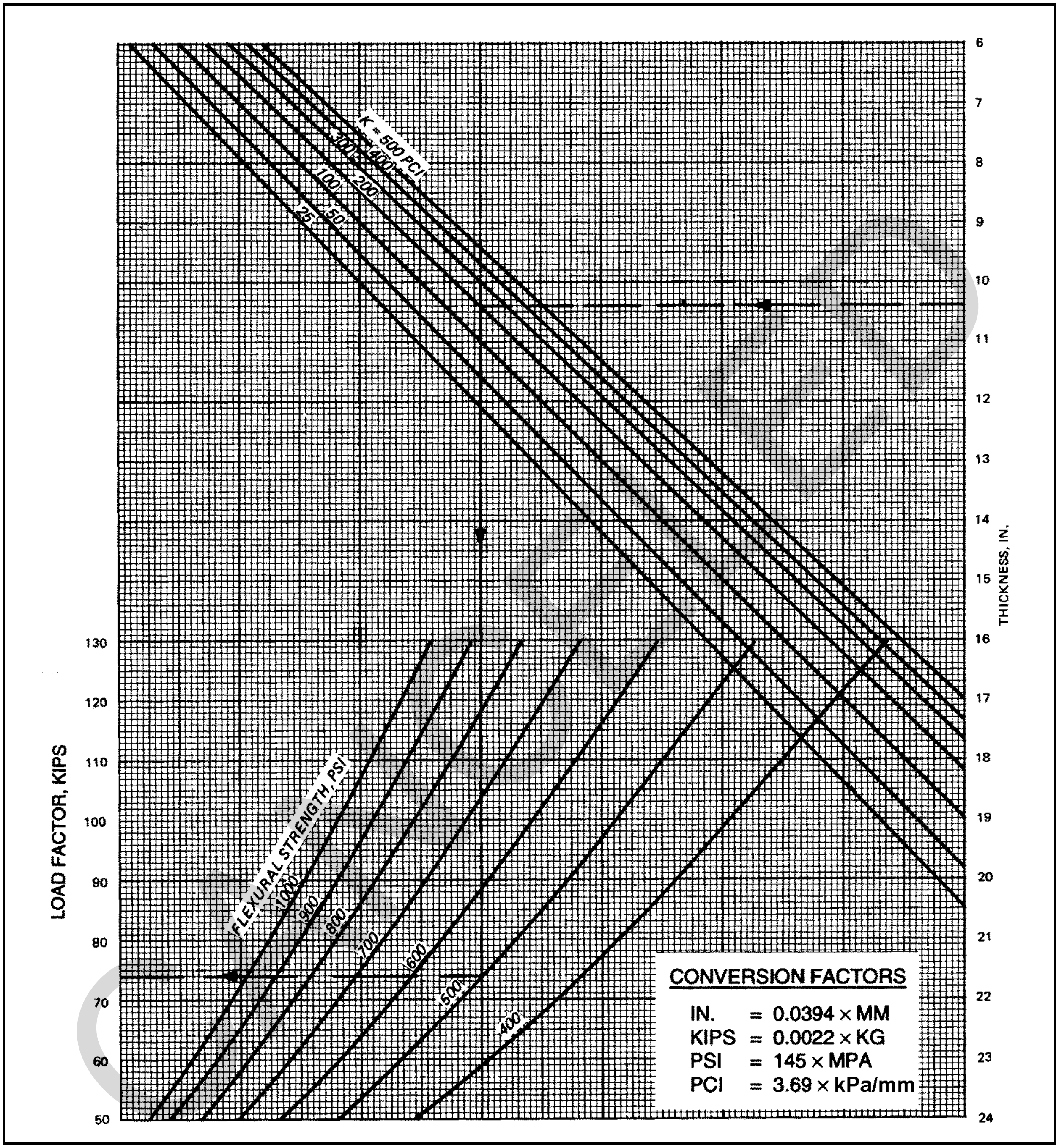


Figure 6-7. Load factor curves for rigid pavement evaluation, F-111 (Air Force Group Index 3)

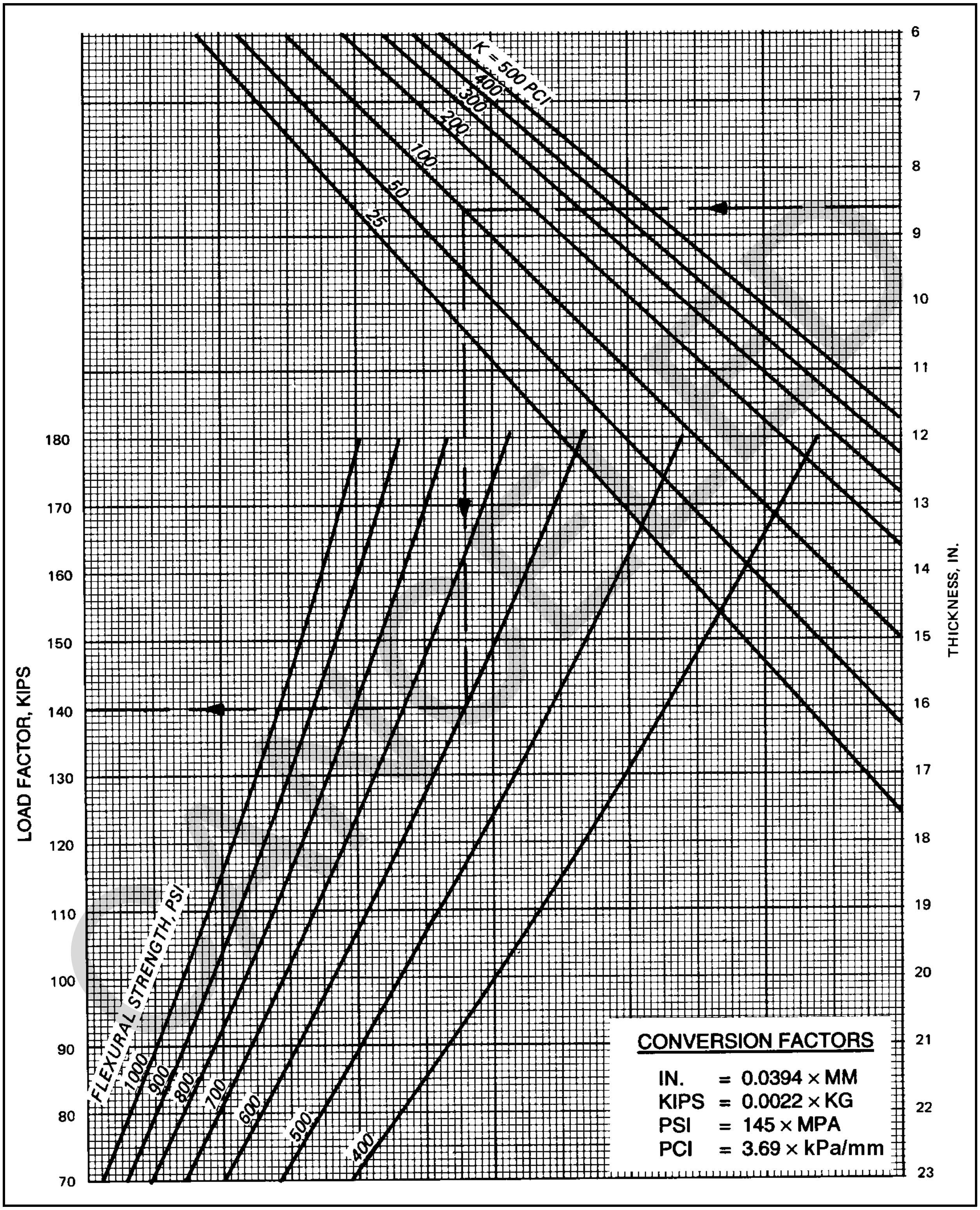


Figure 6-8. Load factor curves for rigid pavement evaluation, C-130 (Air Force Group Index 4)

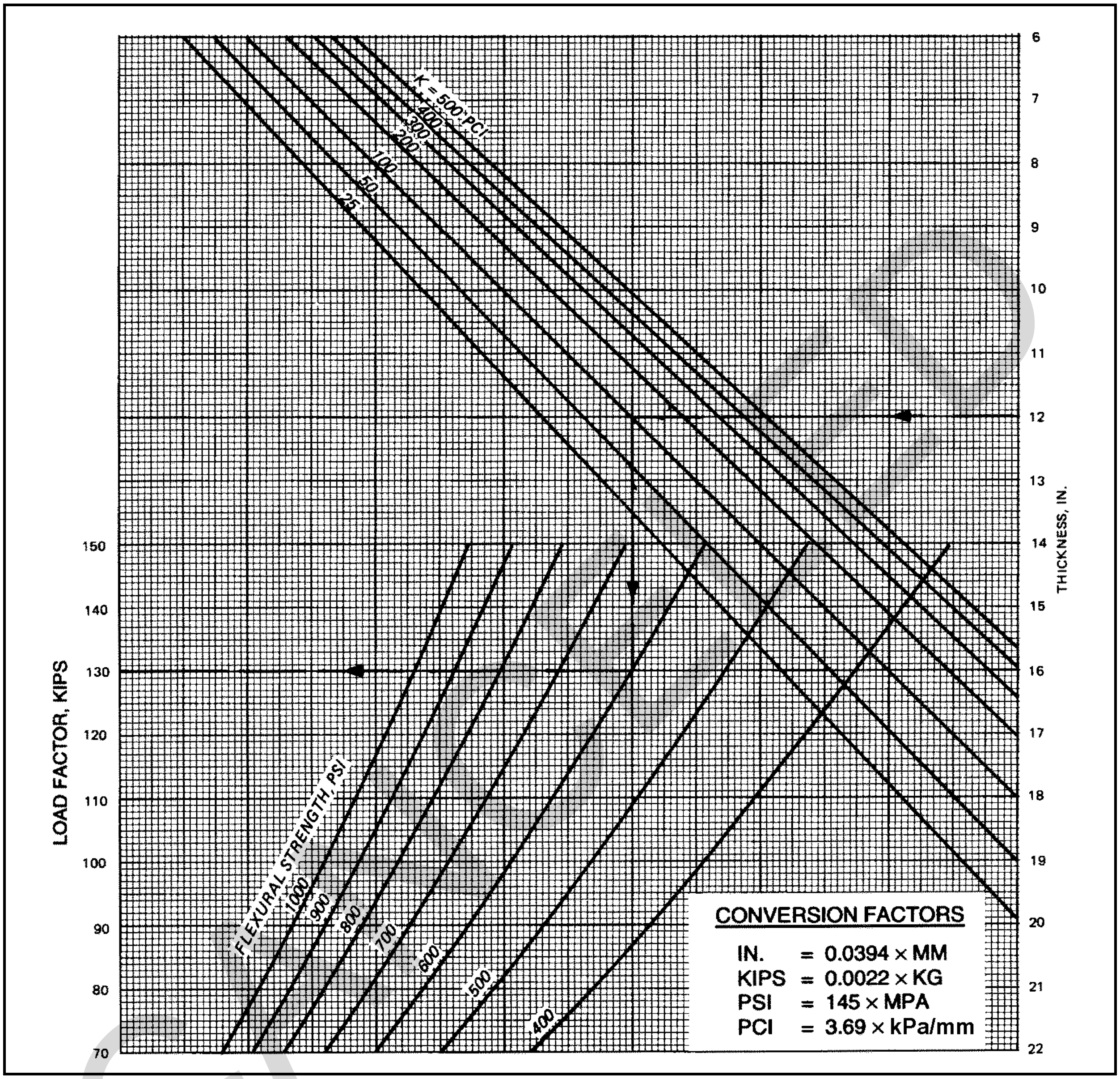


Figure 6-9. Load factor curves for rigid pavement evaluation, C-9 (Air Force Group Index 5)

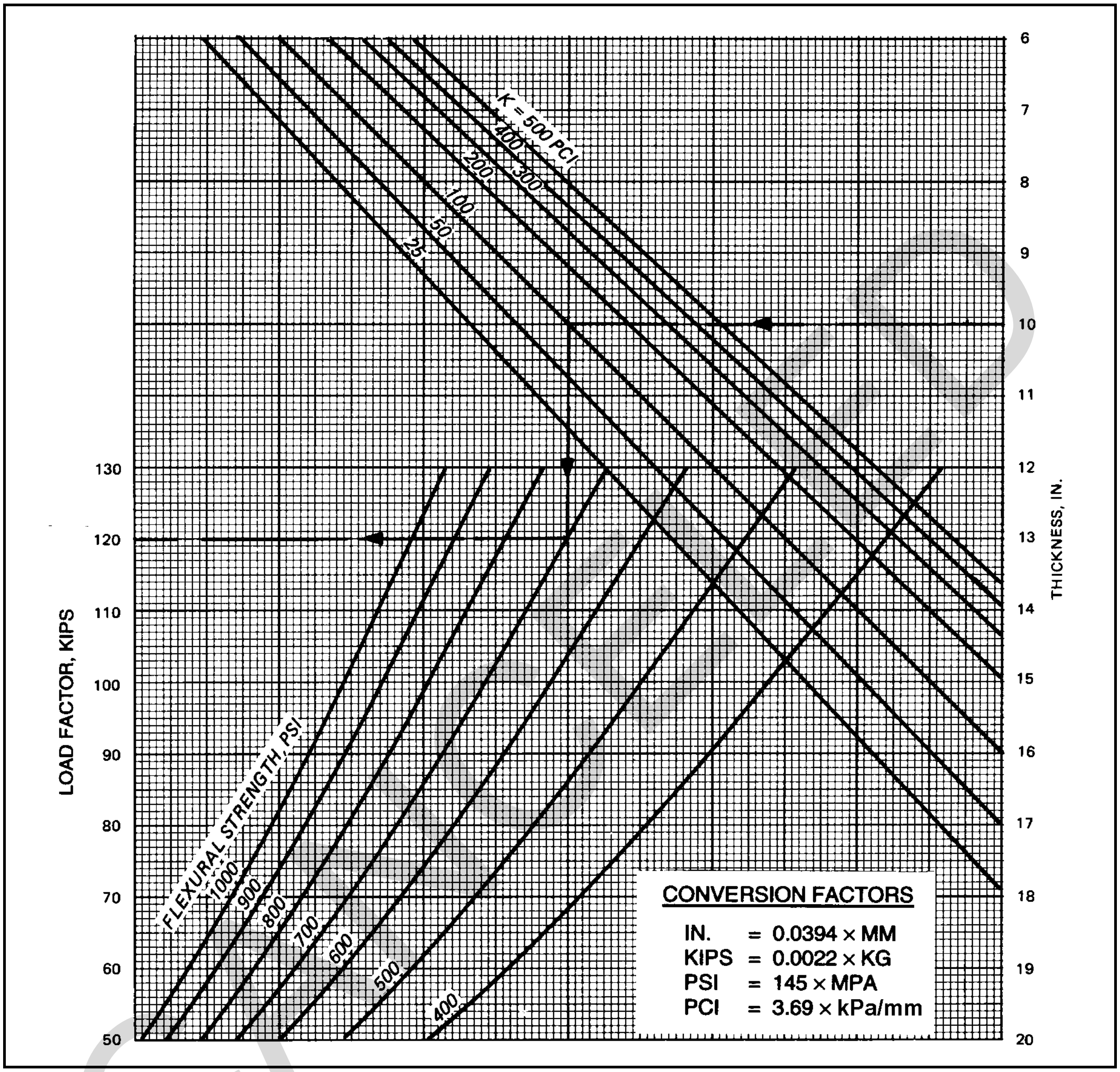


Figure 6-10. Load factor curves for rigid pavement evaluation, T-43 (Air Force Group Index 6)

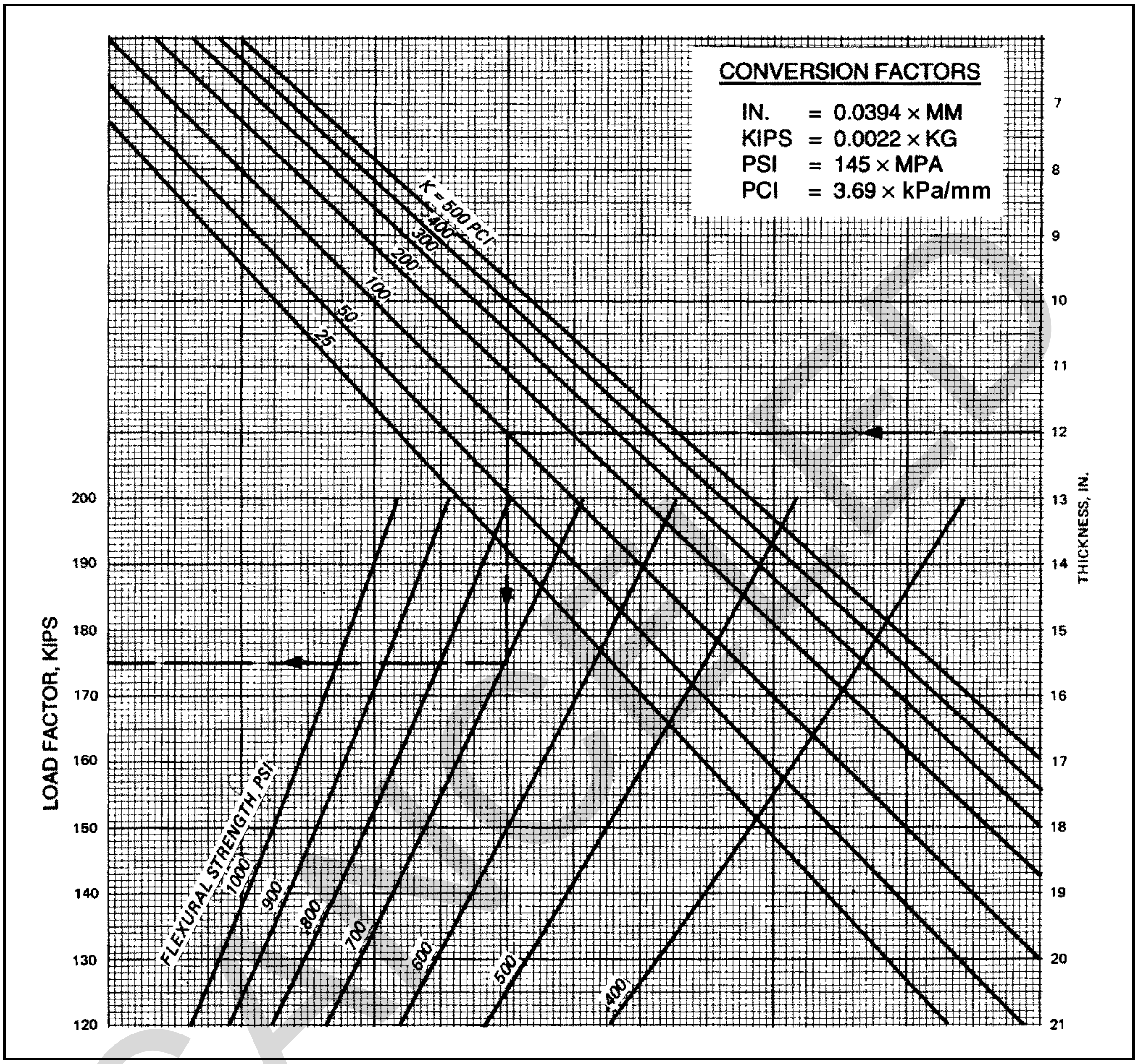


Figure 6-11. Load factor curves for rigid pavement evaluation, B-727 (Air Force Group Index 7)

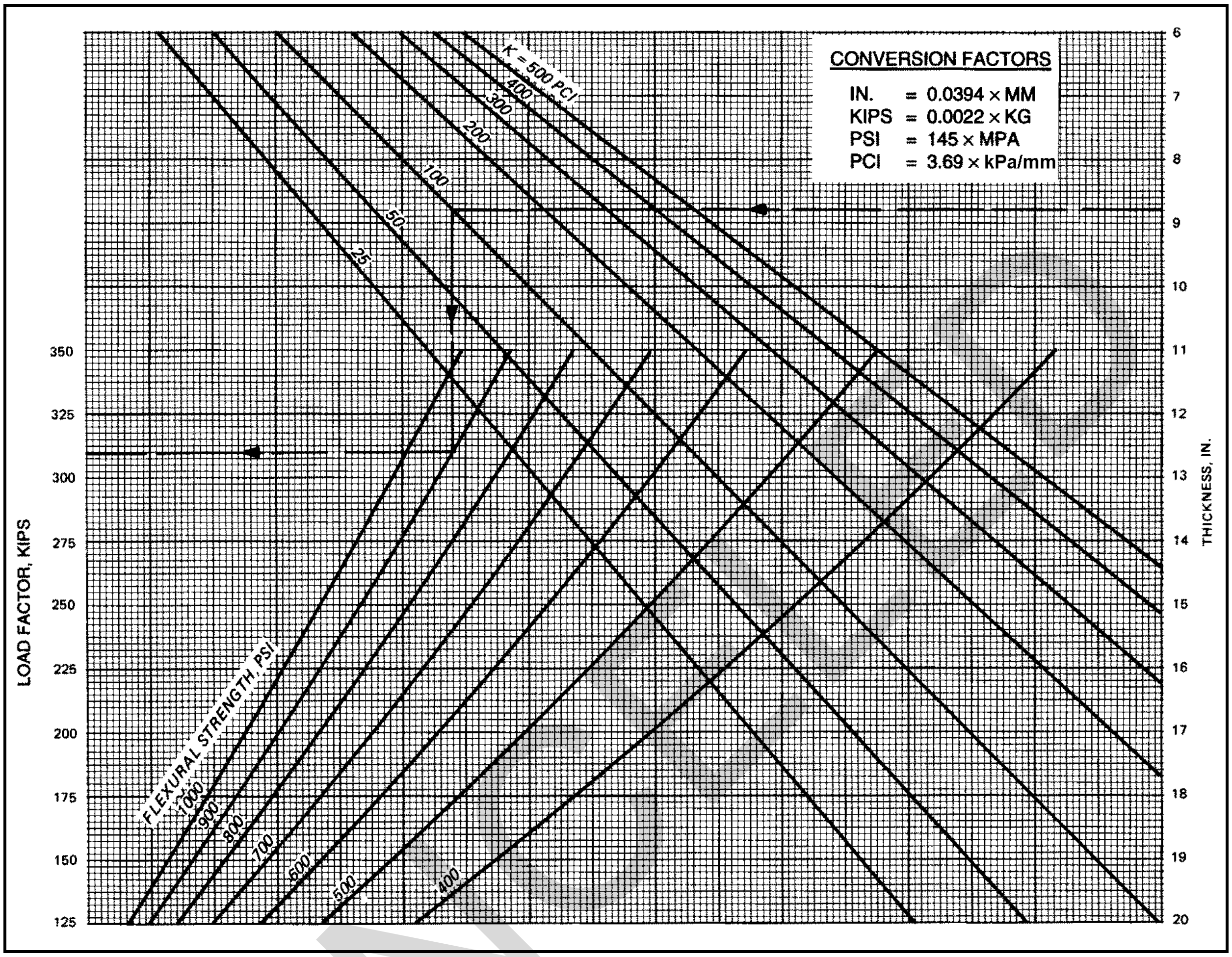


Figure 6-12. Load factor curves for rigid pavement evaluation, E-3 (Air Force Group Index 8)

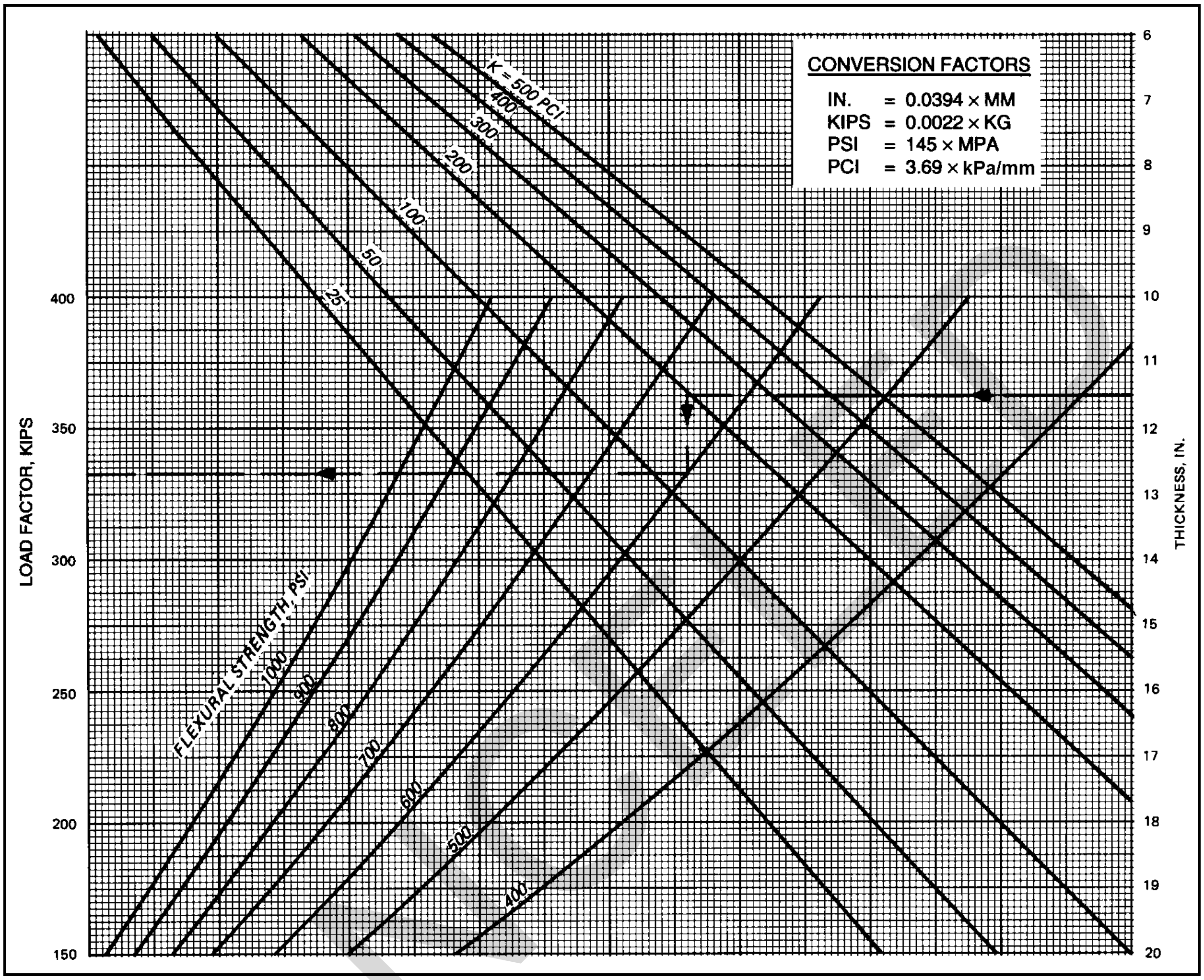


Figure 6-13. Load factor curves for rigid pavement evaluation, C-141 (Air Force Group Index 9)

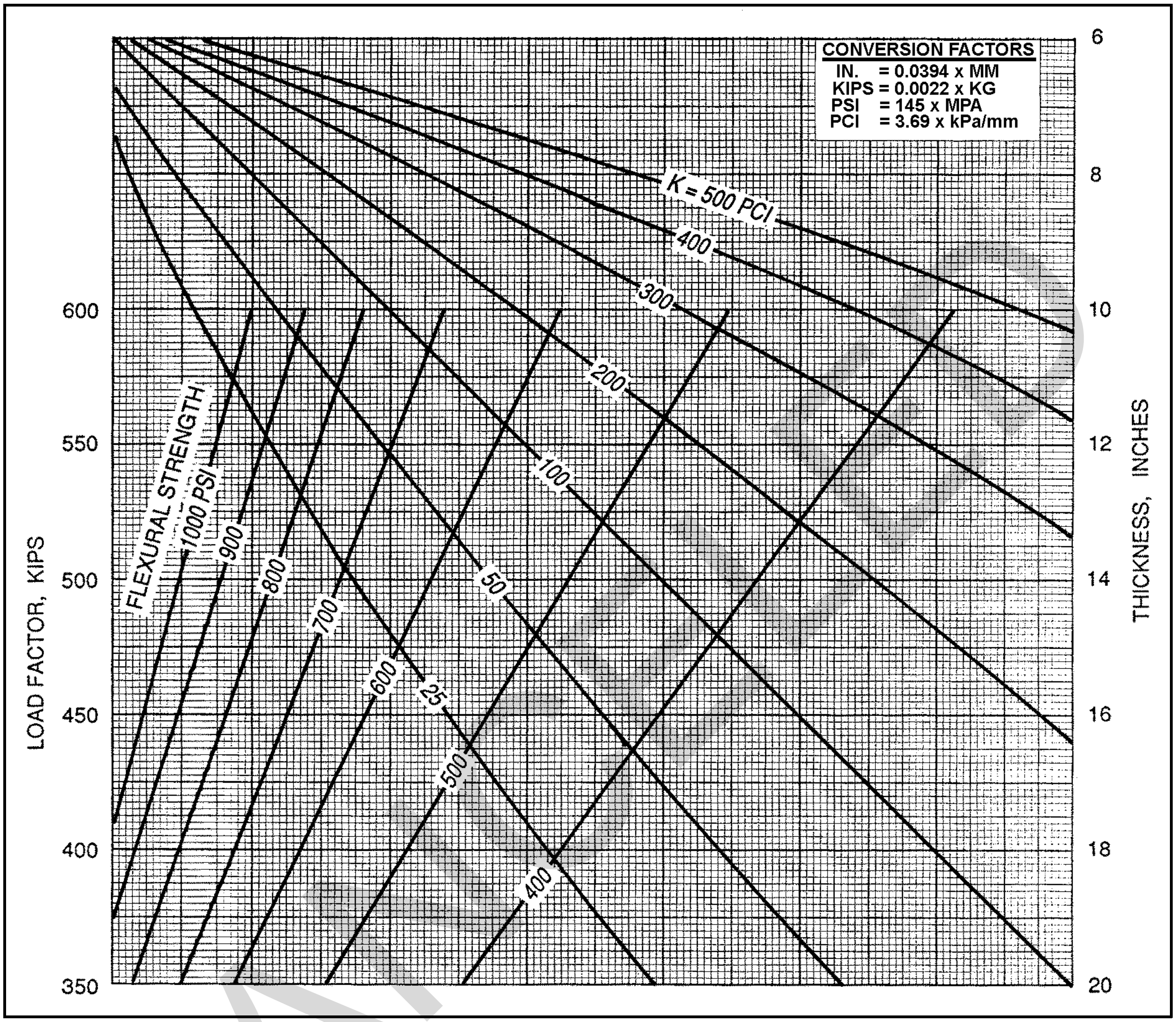


Figure 6-14. Load factor curves for rigid pavement evaluation, C-17 (Air Force Group Index 10)

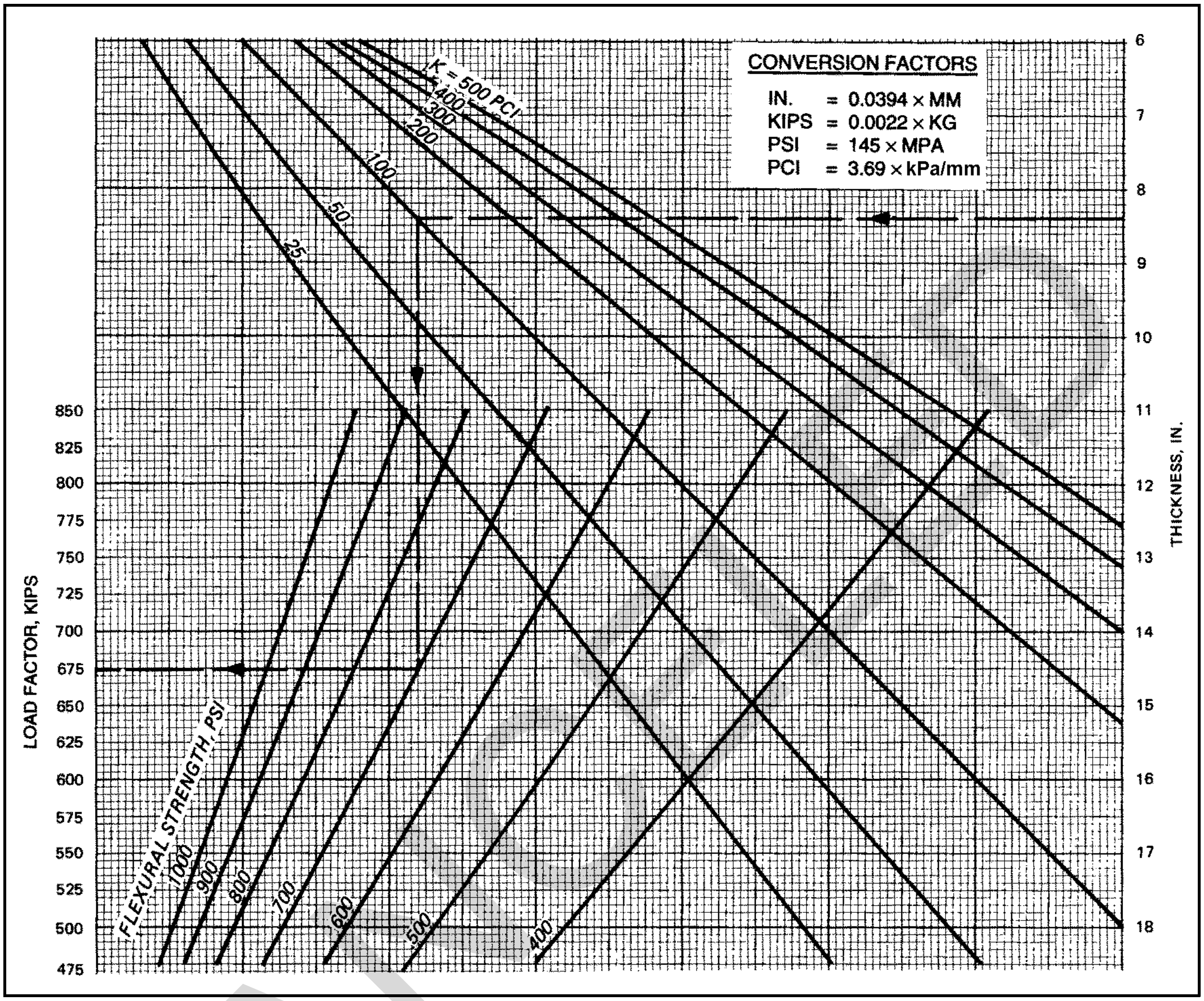


Figure 6-15. Load factor curves for rigid pavement evaluation, C-5 (Air Force Group Index 11)

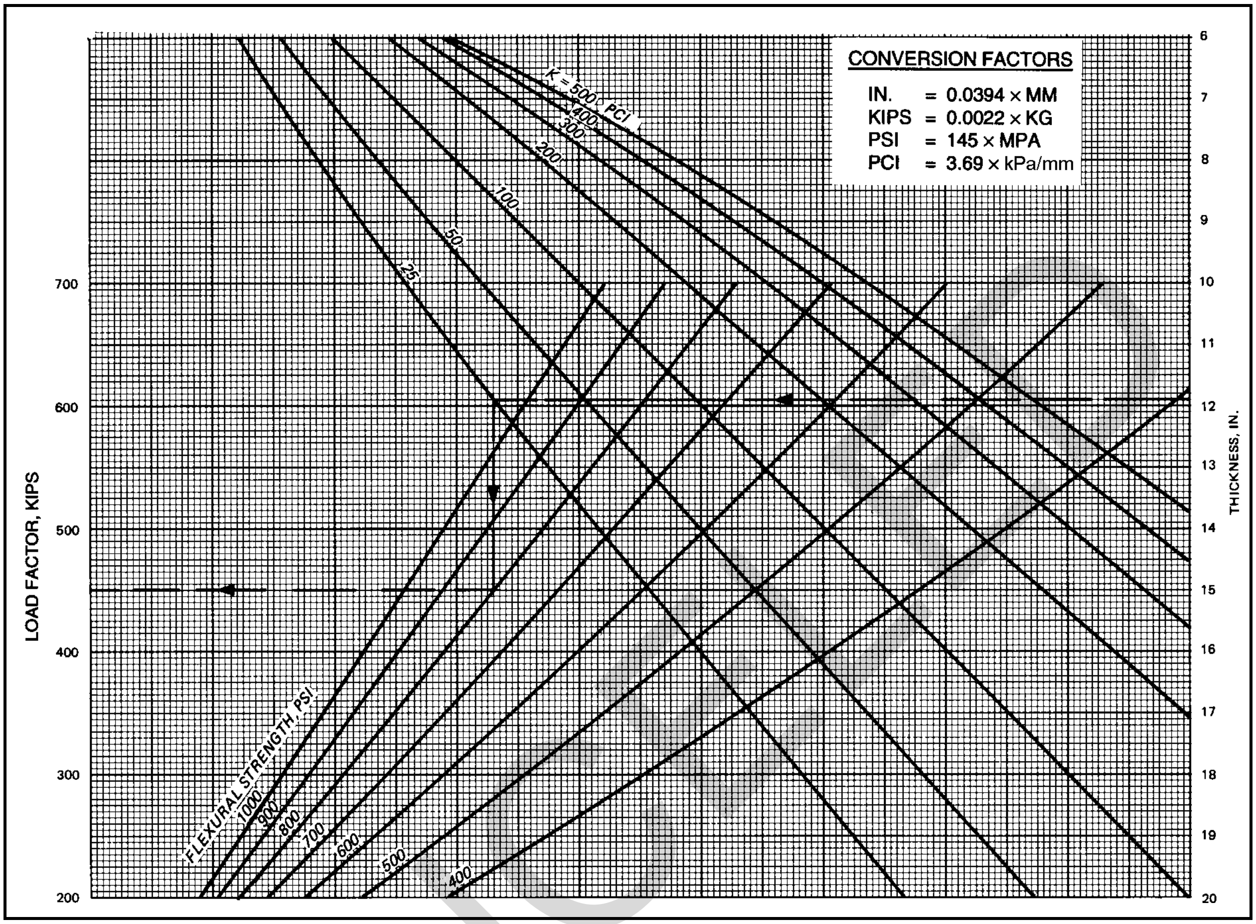


Figure 6-16. Load factor curves for rigid pavement evaluation, KC-10 (Air Force Group Index 12)

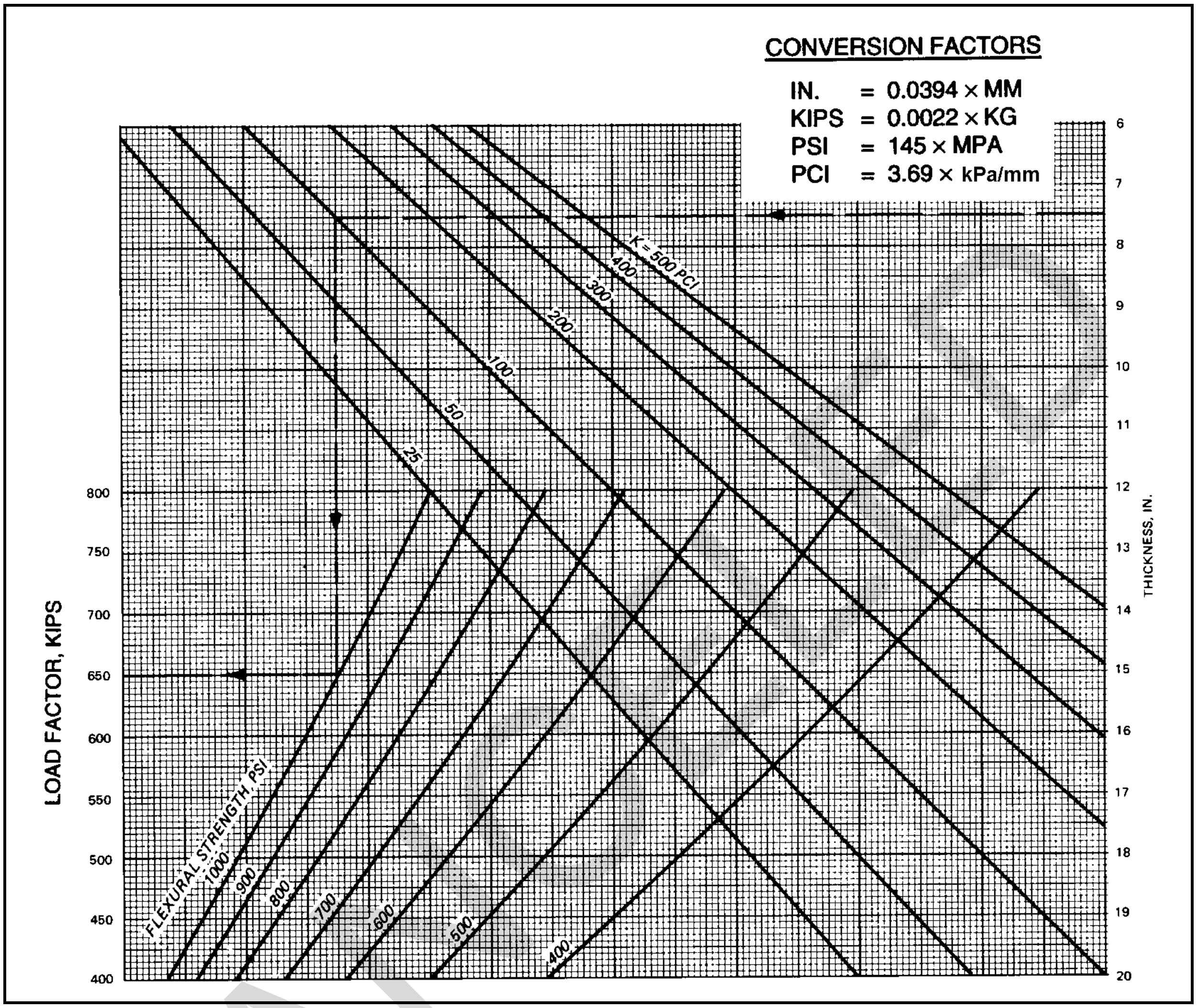


Figure 6-17. Load factor curves for rigid pavement evaluation, E-4 (Air Force Group Index 13)

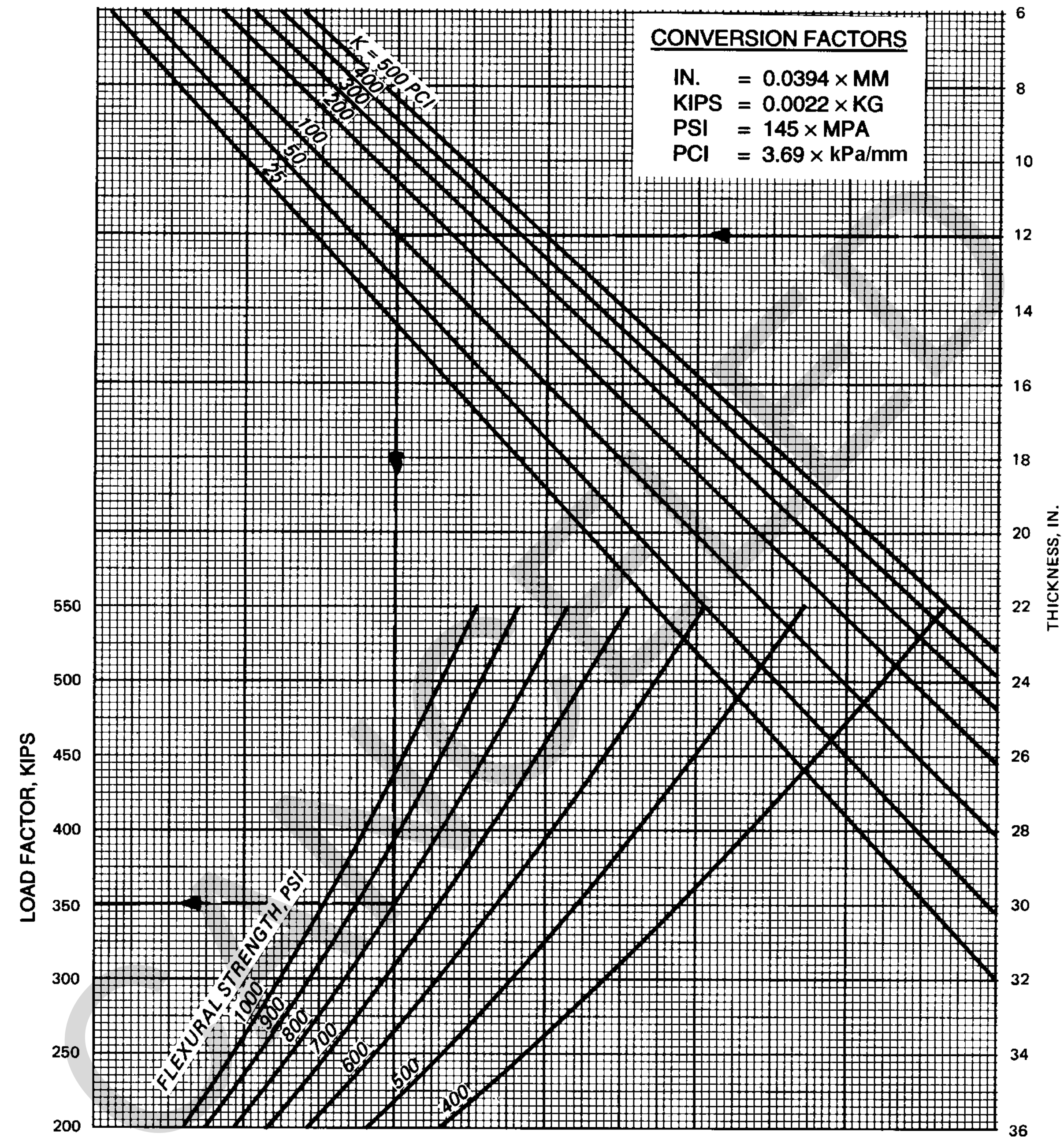


Figure 6-18. Load factor curves for rigid pavement evaluation, B-52 (Air Force Group Index 14)

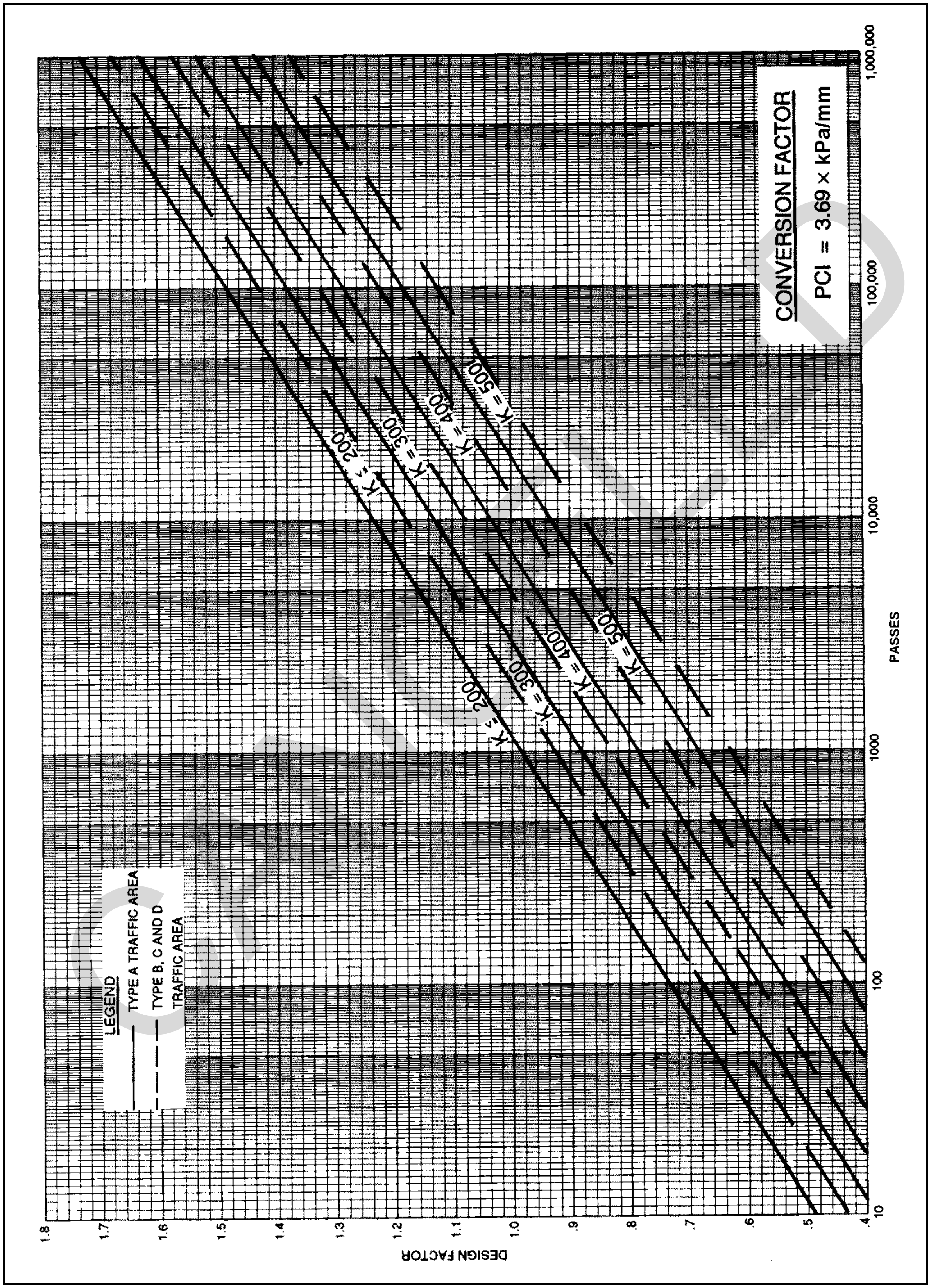


Figure 6-19. Design factors for standard evaluation, UH-60

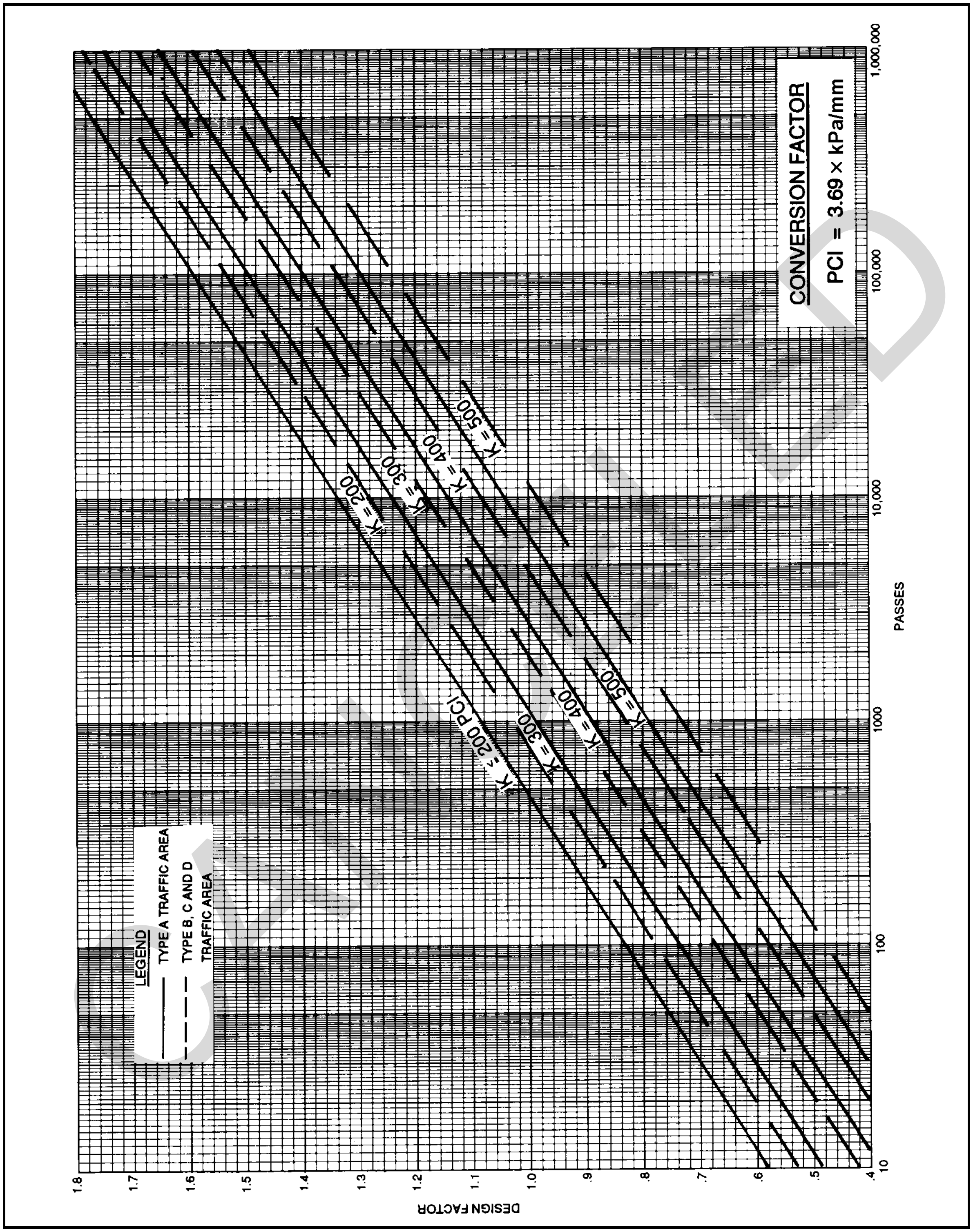


Figure 6-20. Design factors for standard evaluation, CH-47

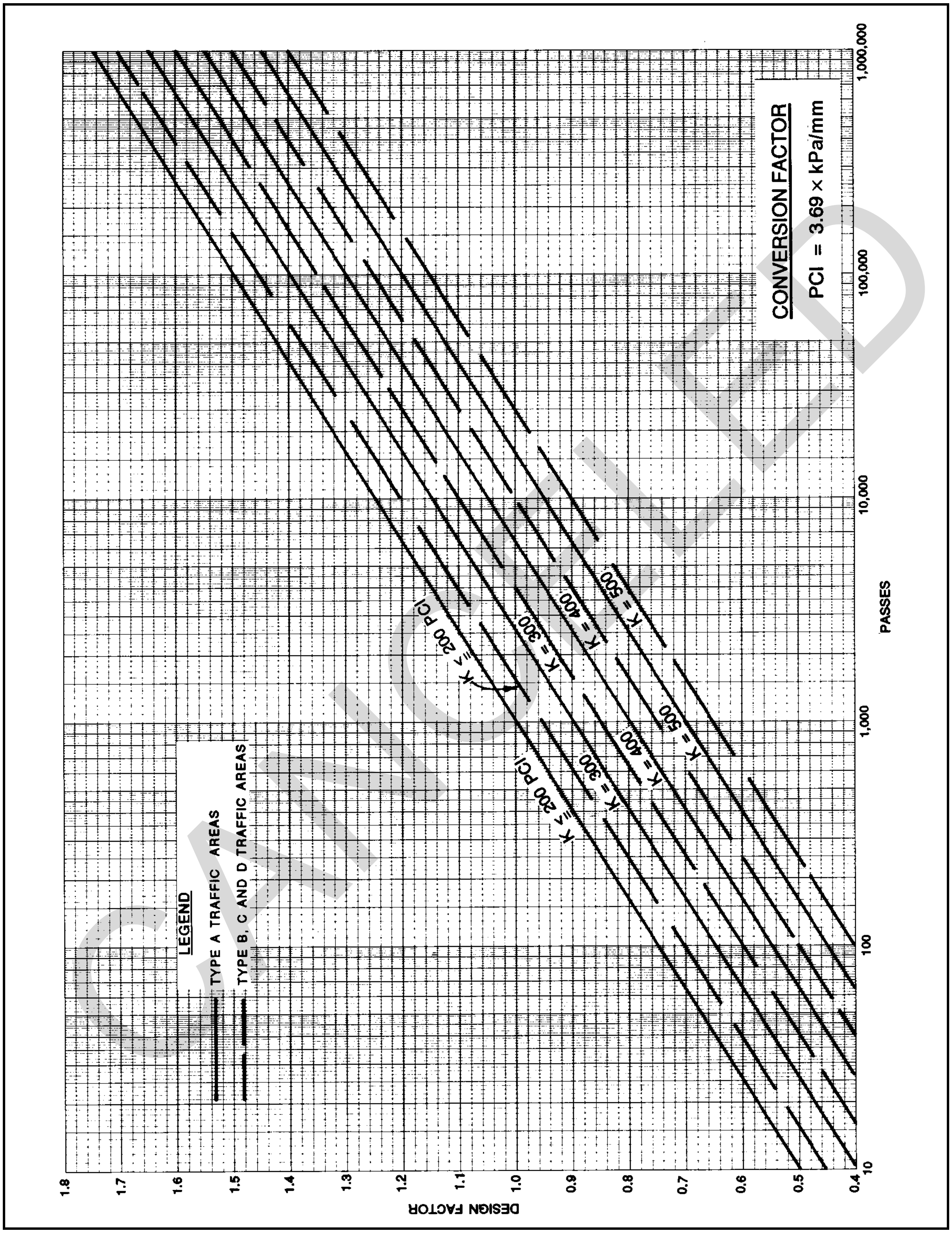


Figure 6-21. Design factors for standard evaluation, OV-1

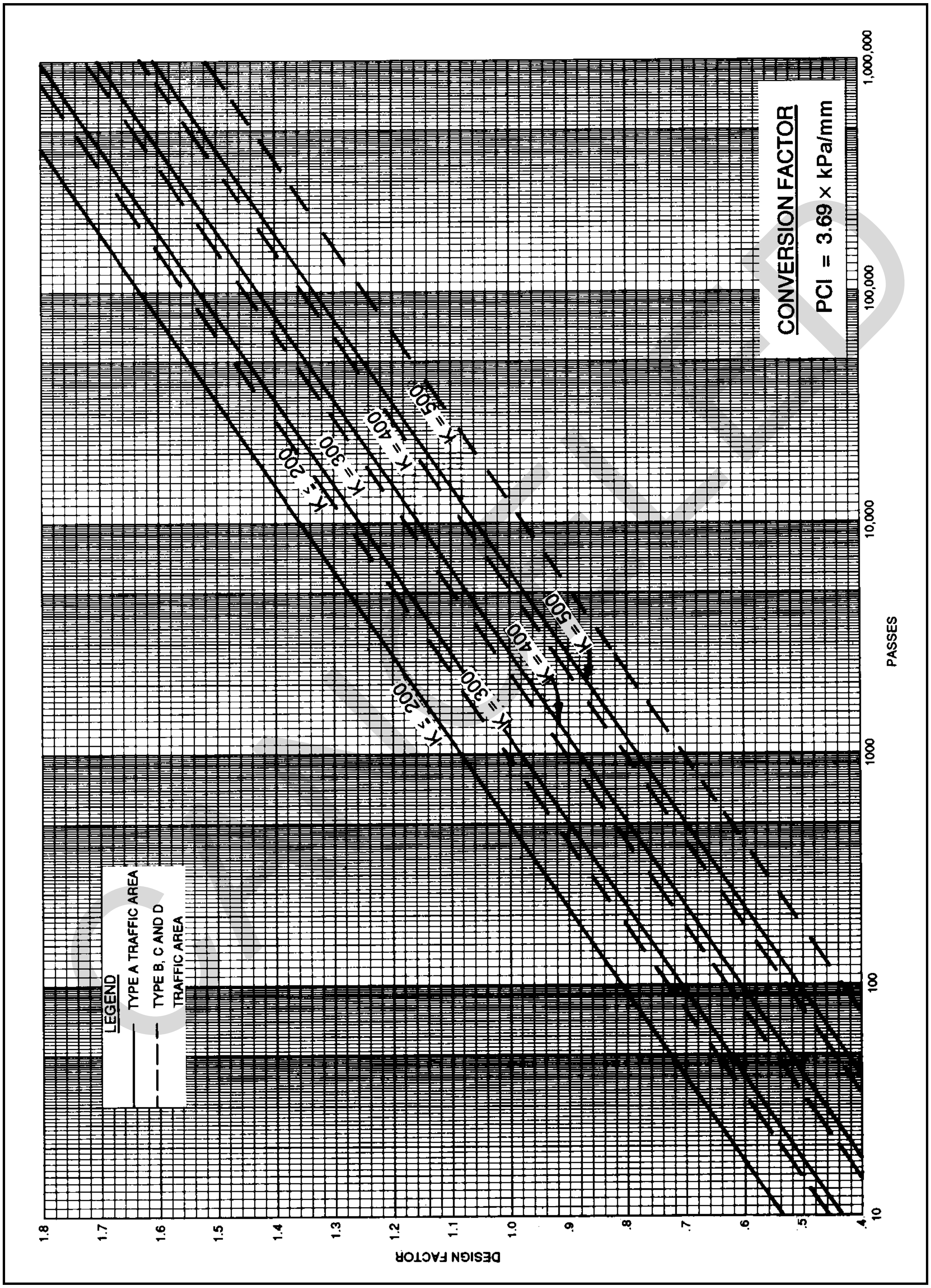


Figure 6-22. Design factors for standard evaluation, C-12

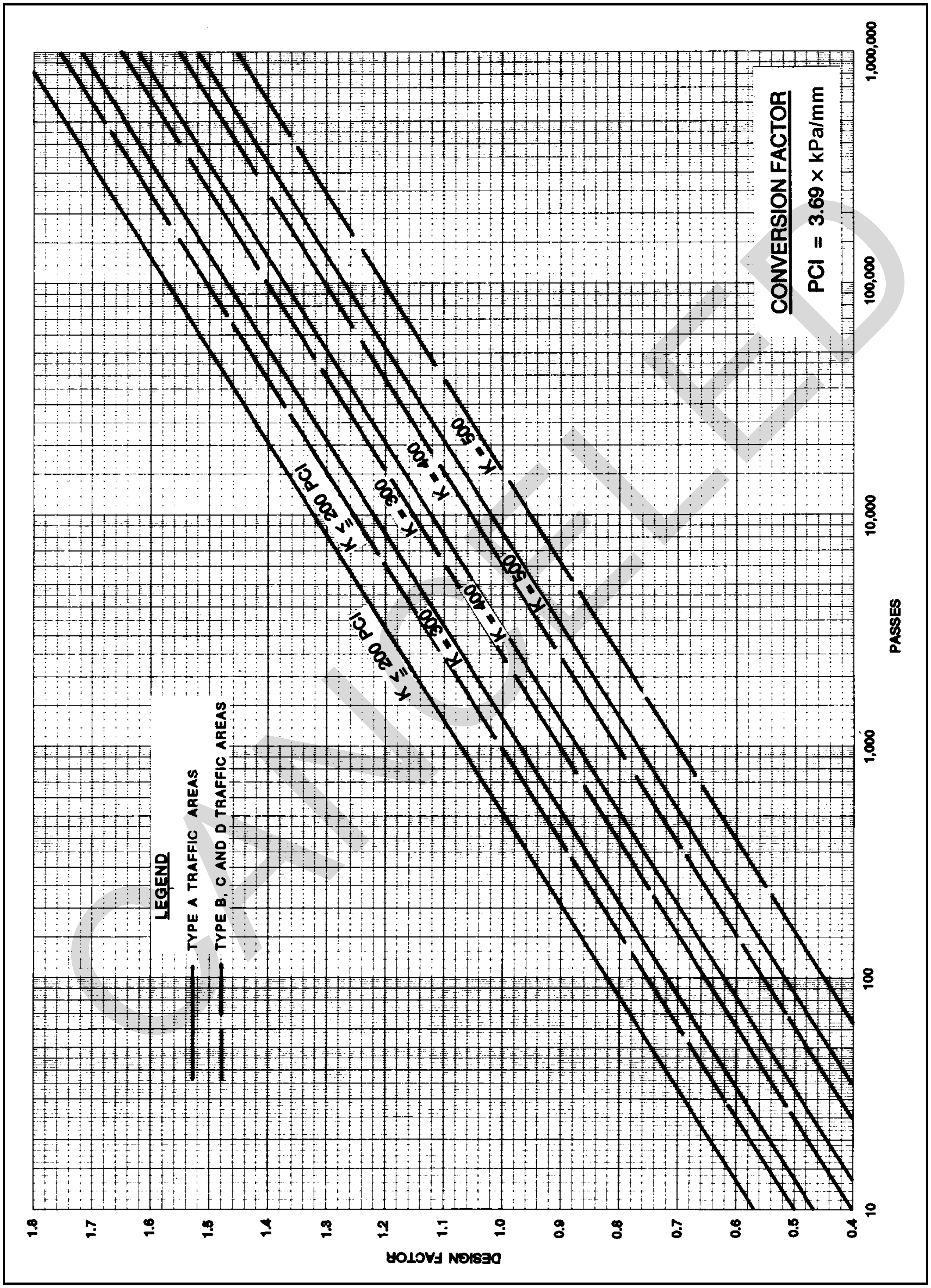


Figure 6-23. Design factors for standard evaluation, C-23 (Air Force Group Index 1)

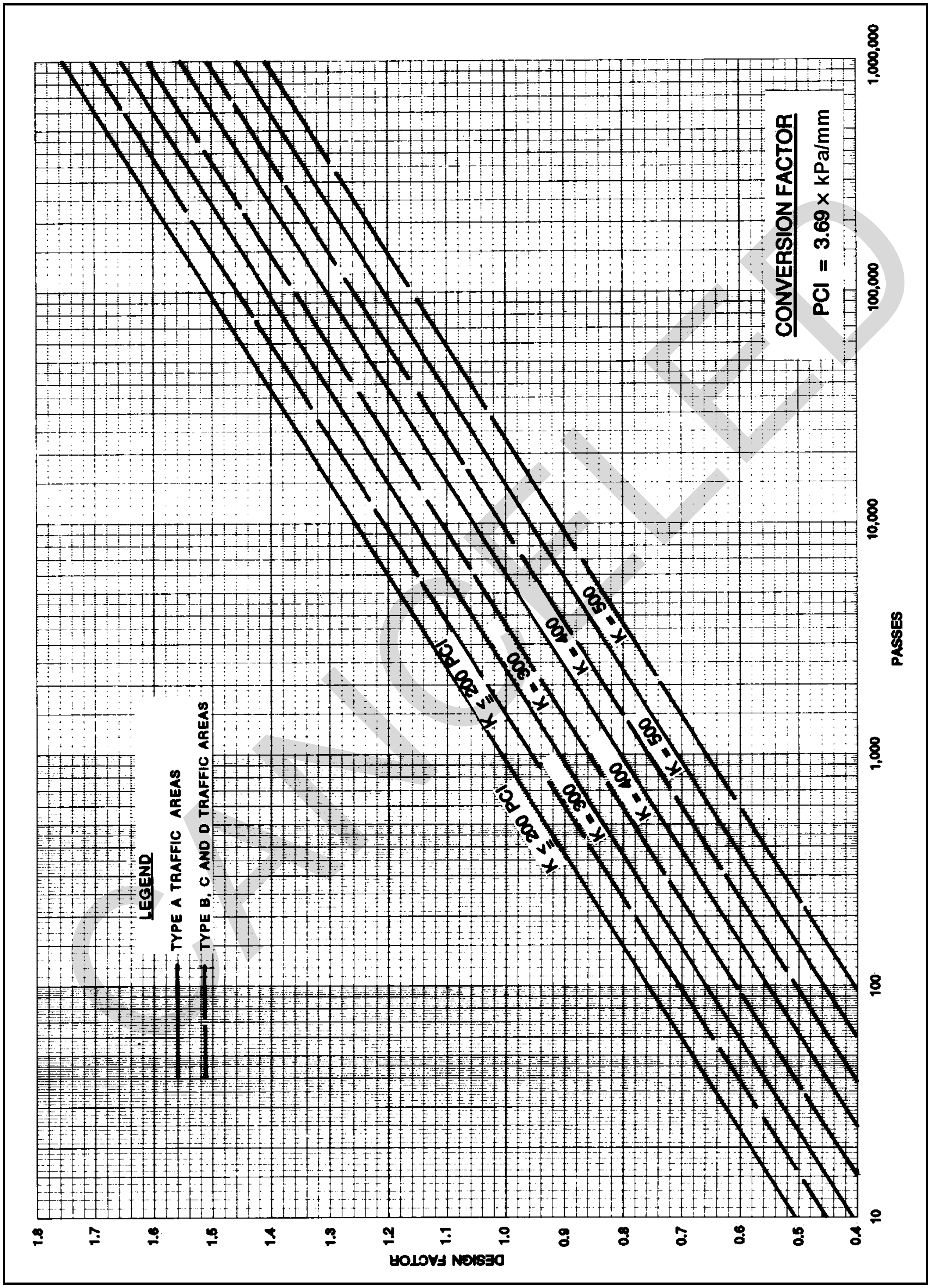


Figure 6-24. Design factors for standard evaluation, F-4 (Air Force Group Index 2)

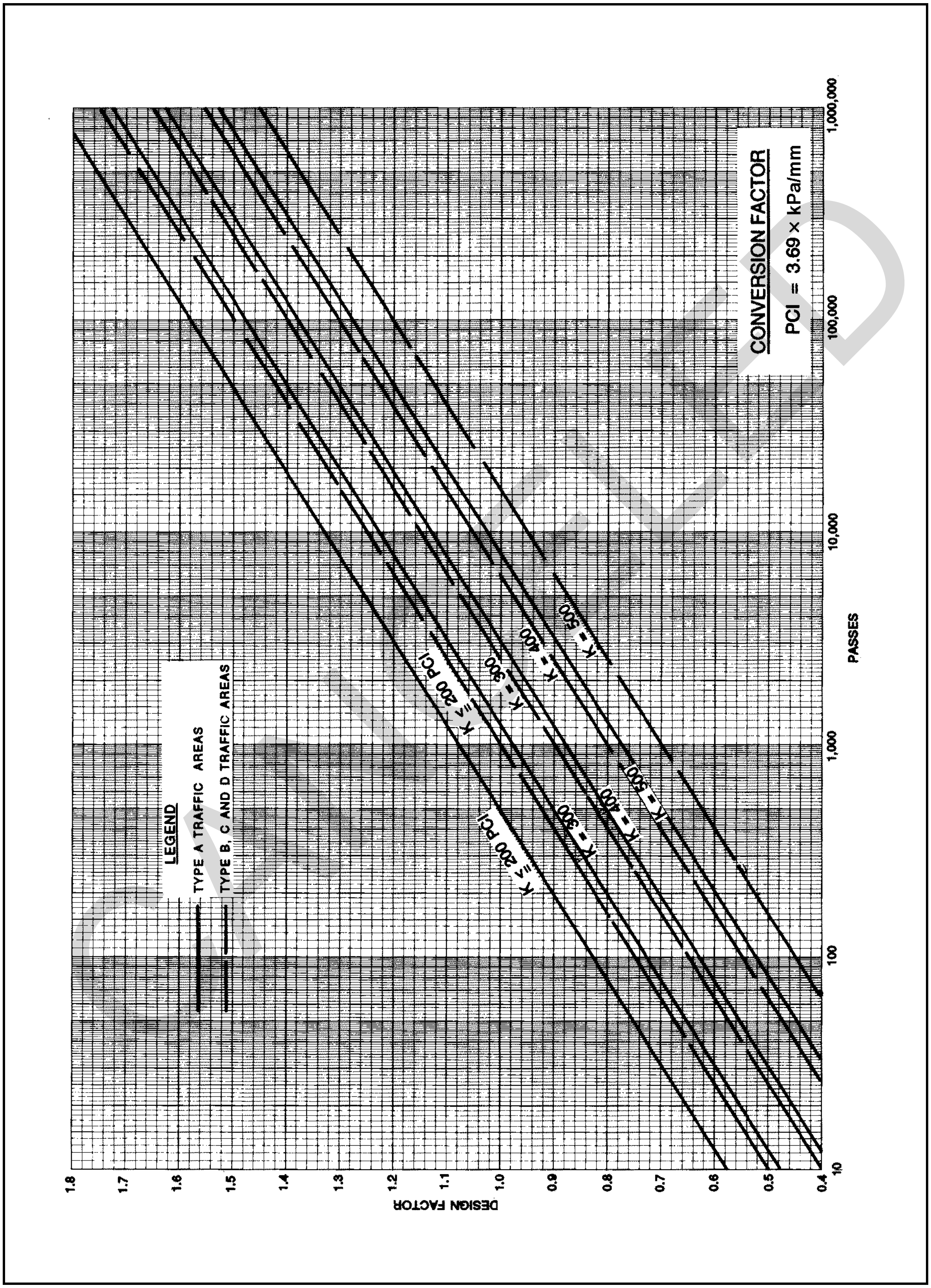


Figure 6-25. Design factors for standard evaluation, F-111 (Air Force Group Index 3)

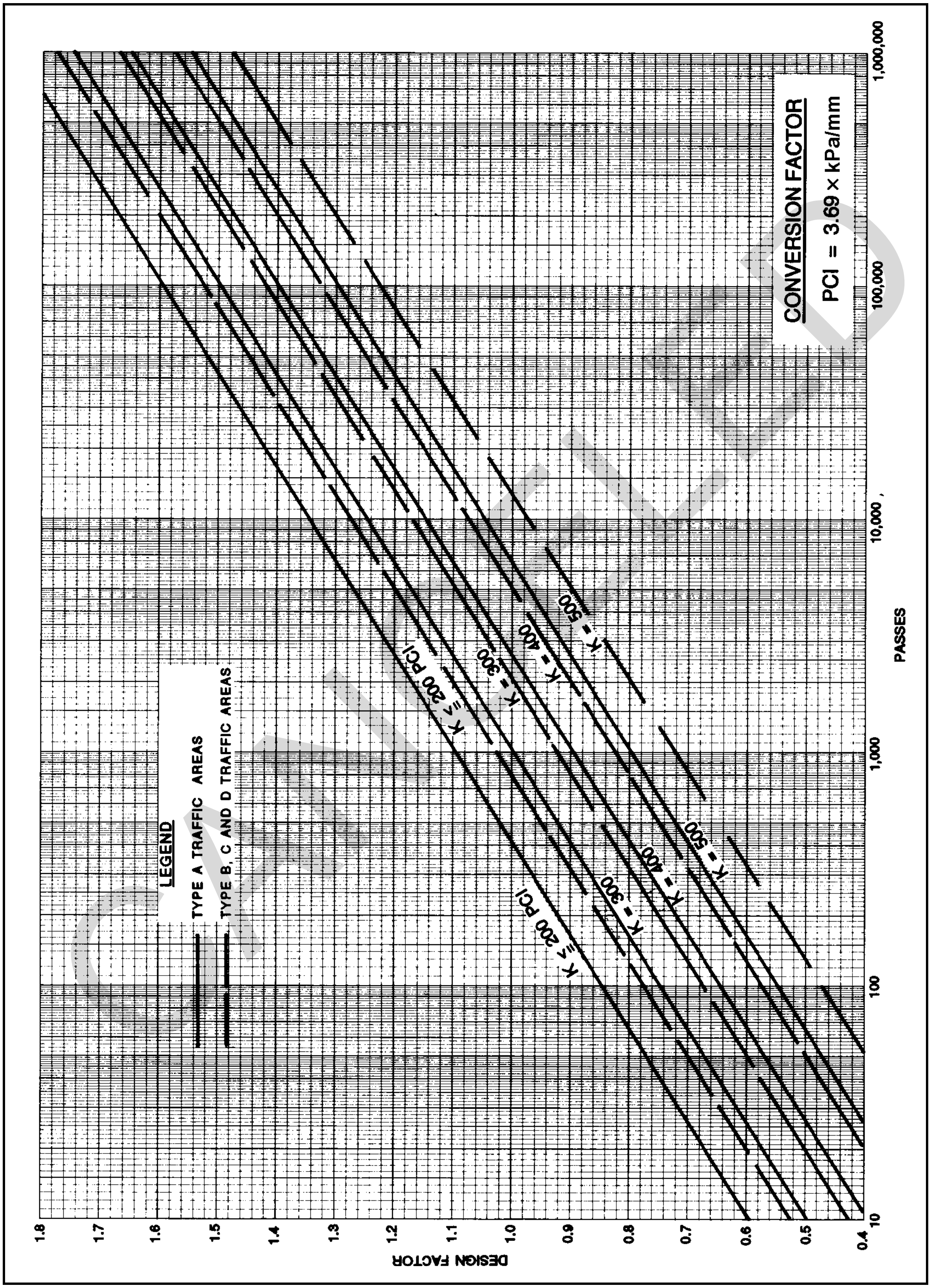


Figure 6-26. Design factors for standard evaluation, C-130 (Air Force Group Index 4)

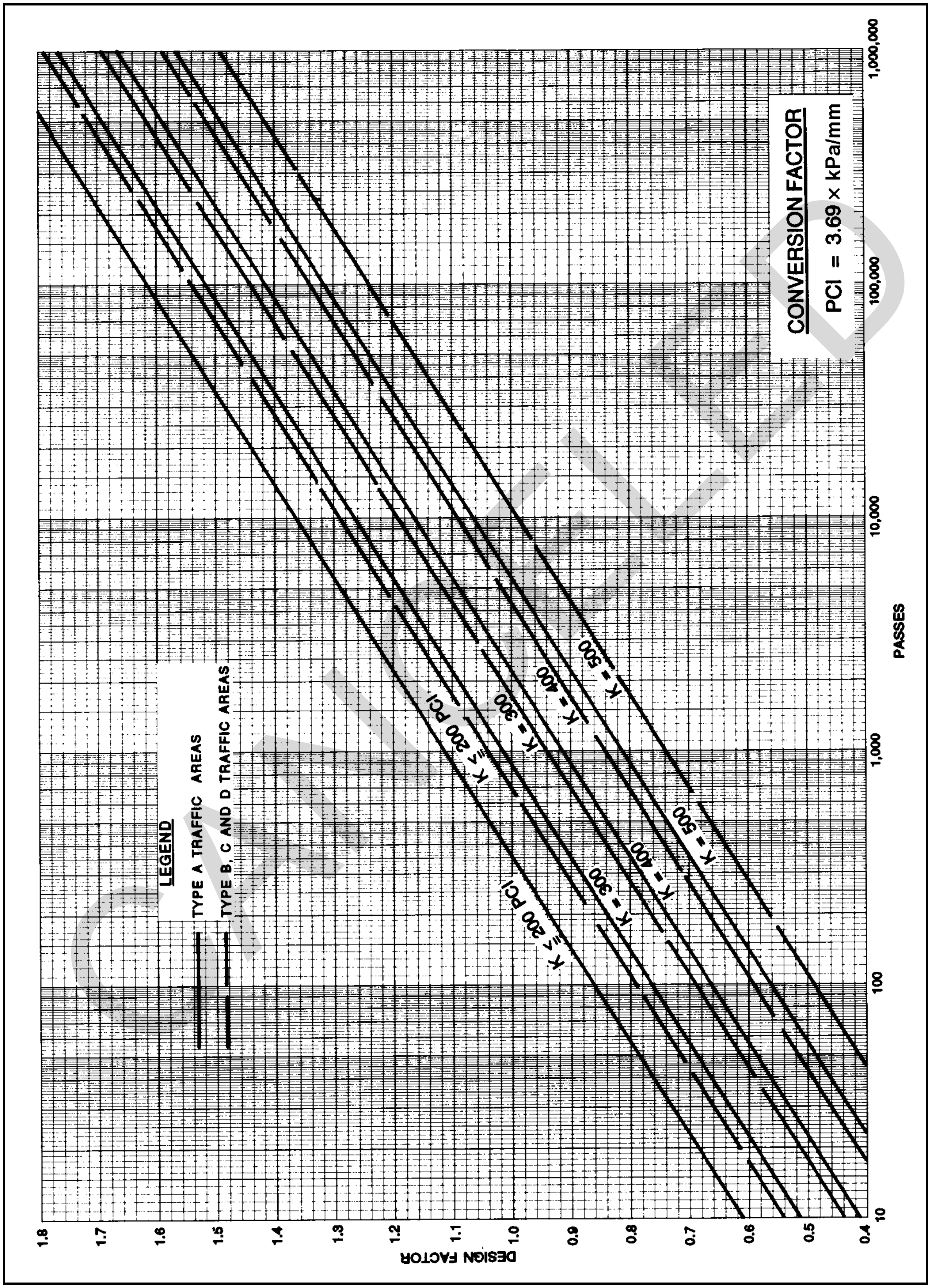


Figure 6-27. Design factors for standard evaluation, C-9 (Air Force Group Index 5)

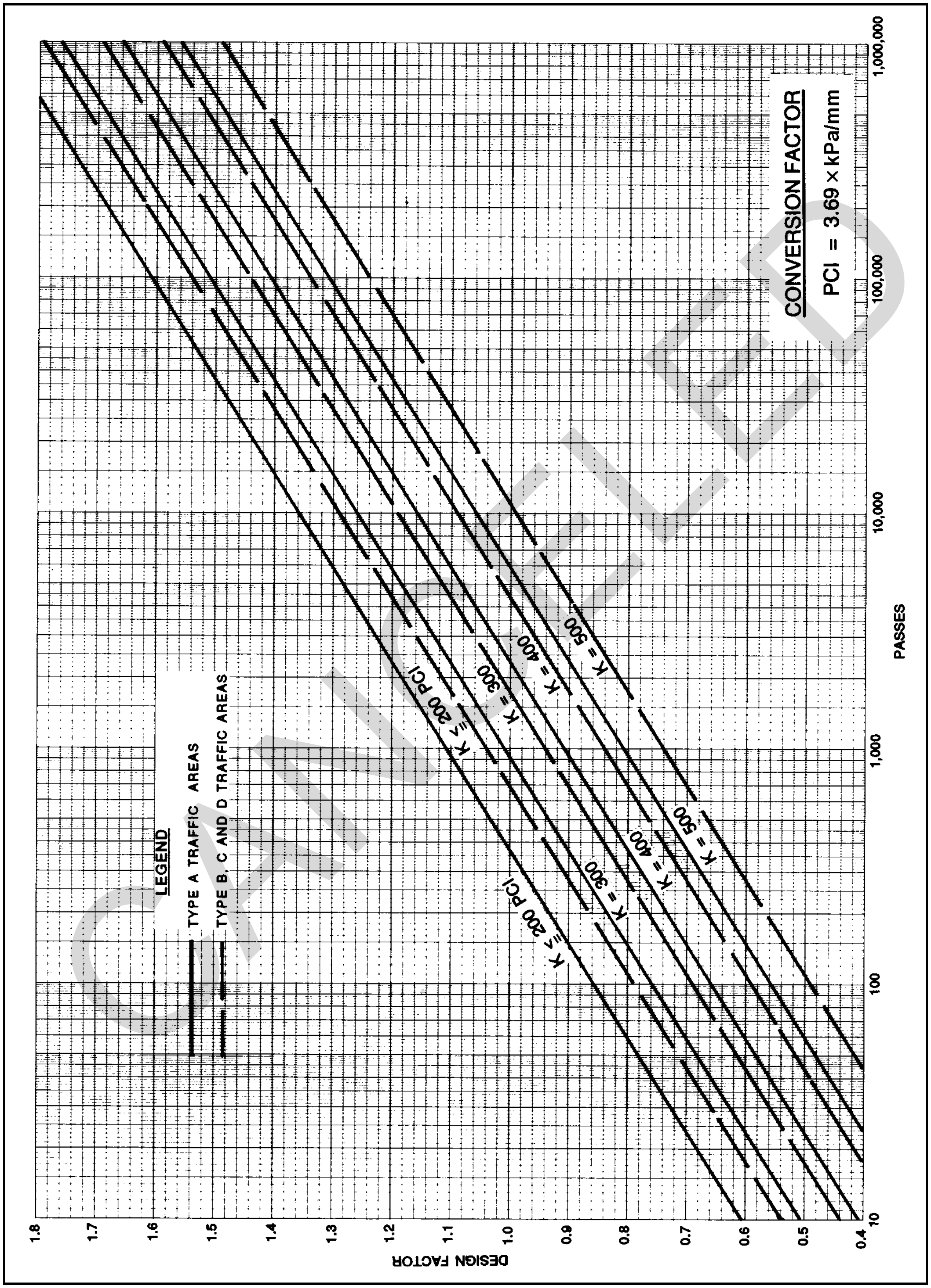


Figure 6-28. Design factors for standard evaluation, T-43 (Air Force Group Index 6)

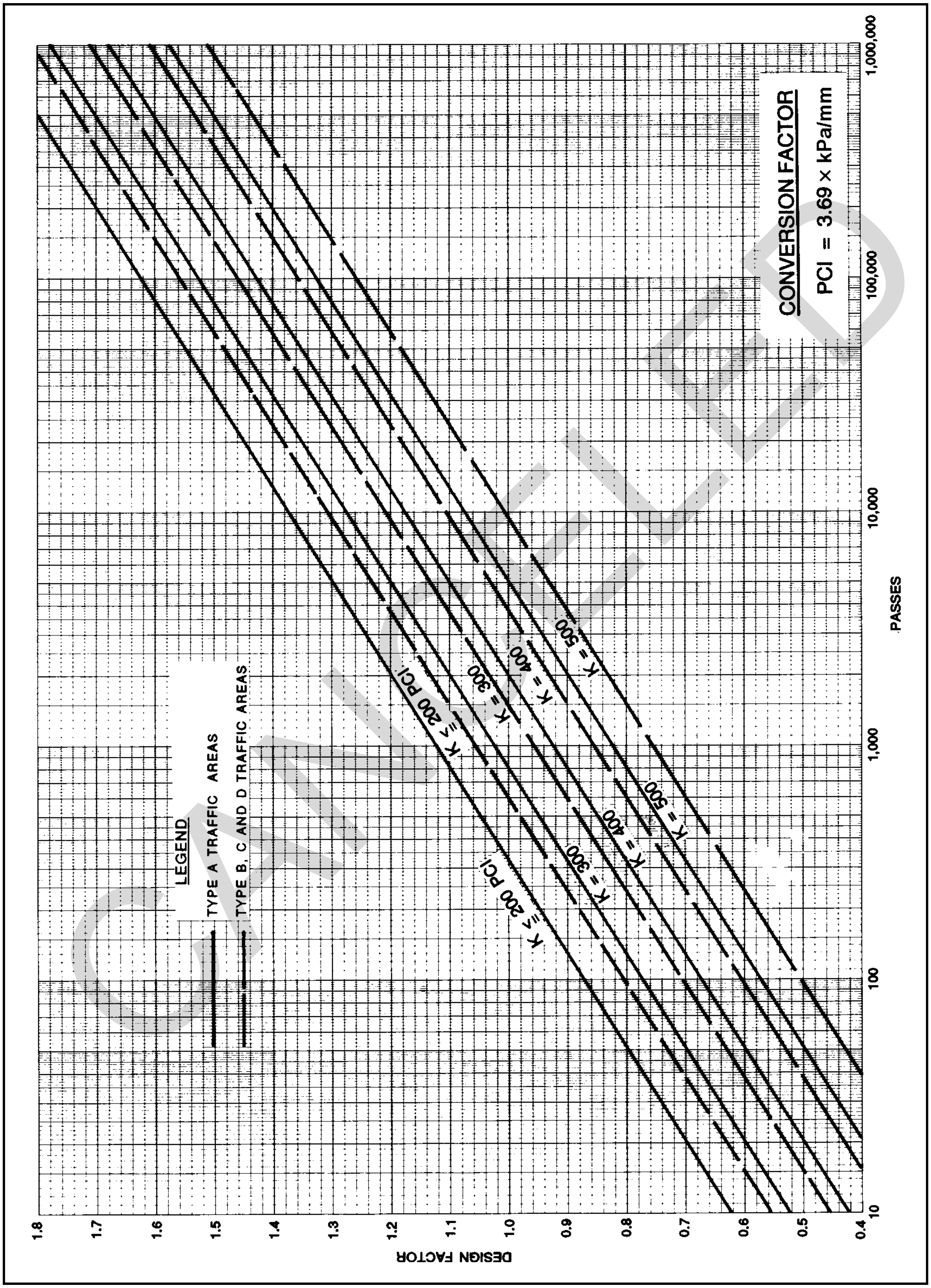


Figure 6-29. Design factors for standard evaluation, B-727 (Air Force Group Index 7)

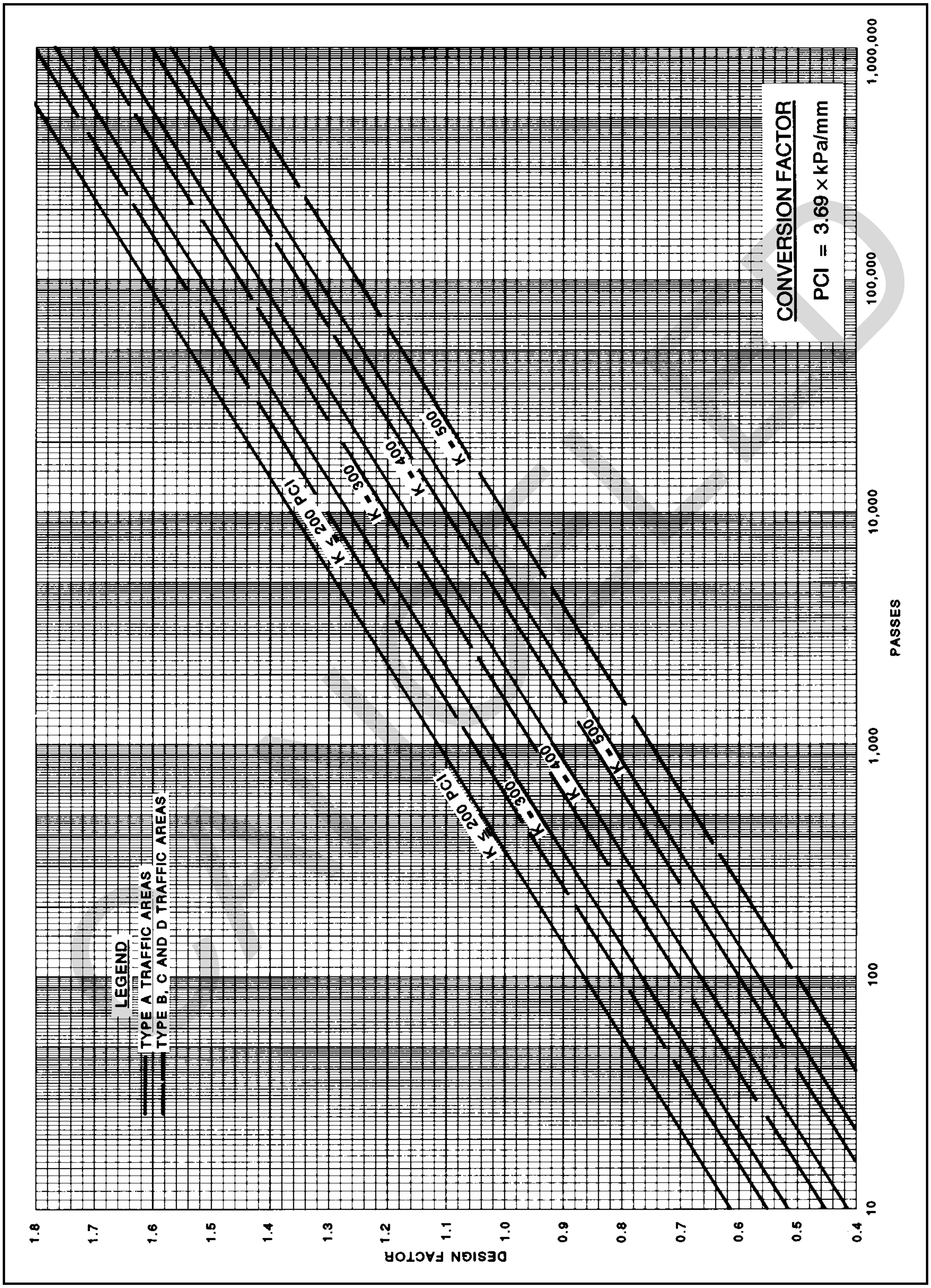


Figure 6-30. Design factors for standard evaluation, E-3 (Air Force Group Index 8)

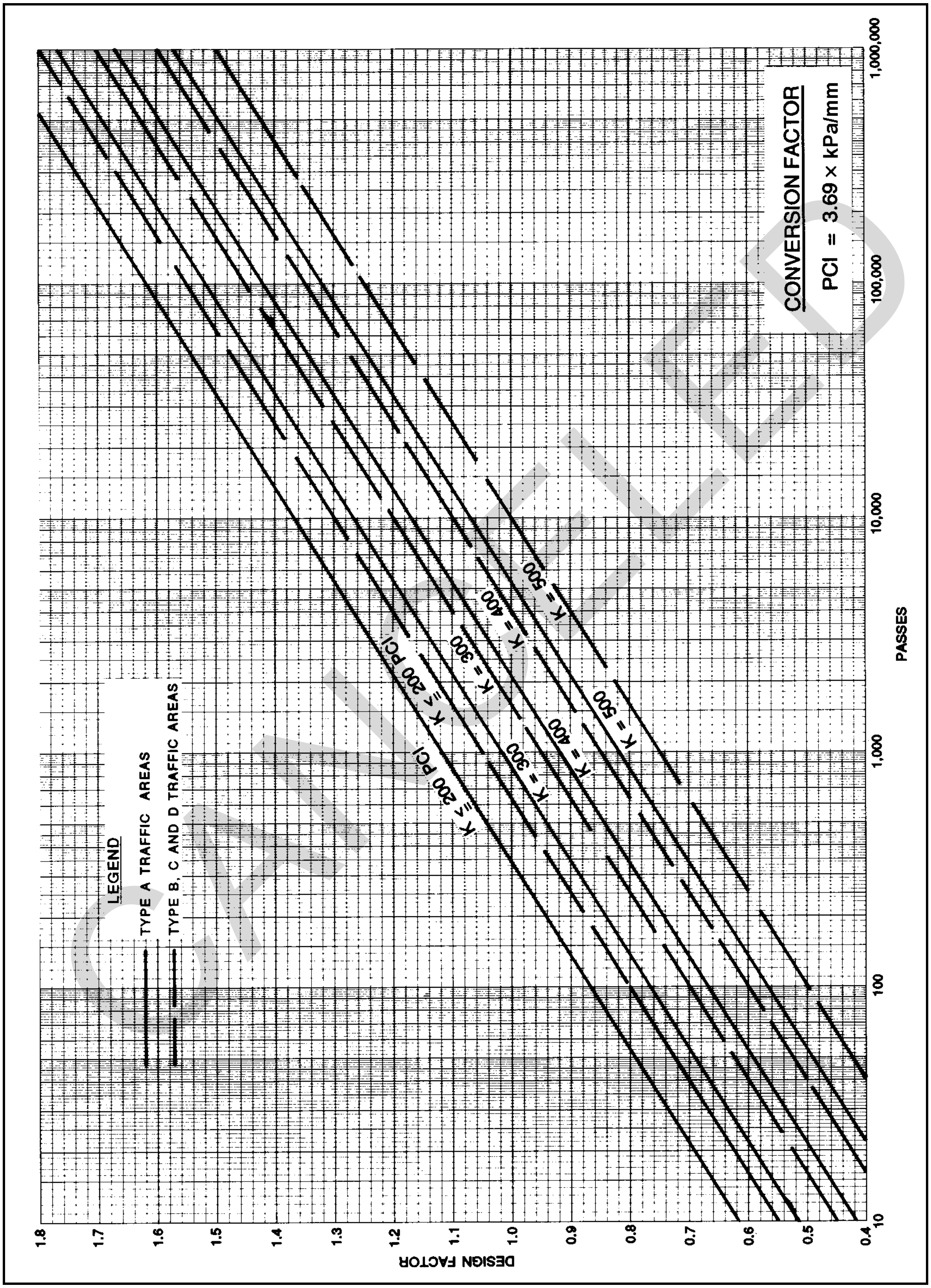


Figure 6-31. Design factors for standard evaluation, C-141 (Air Force Group Index 9)

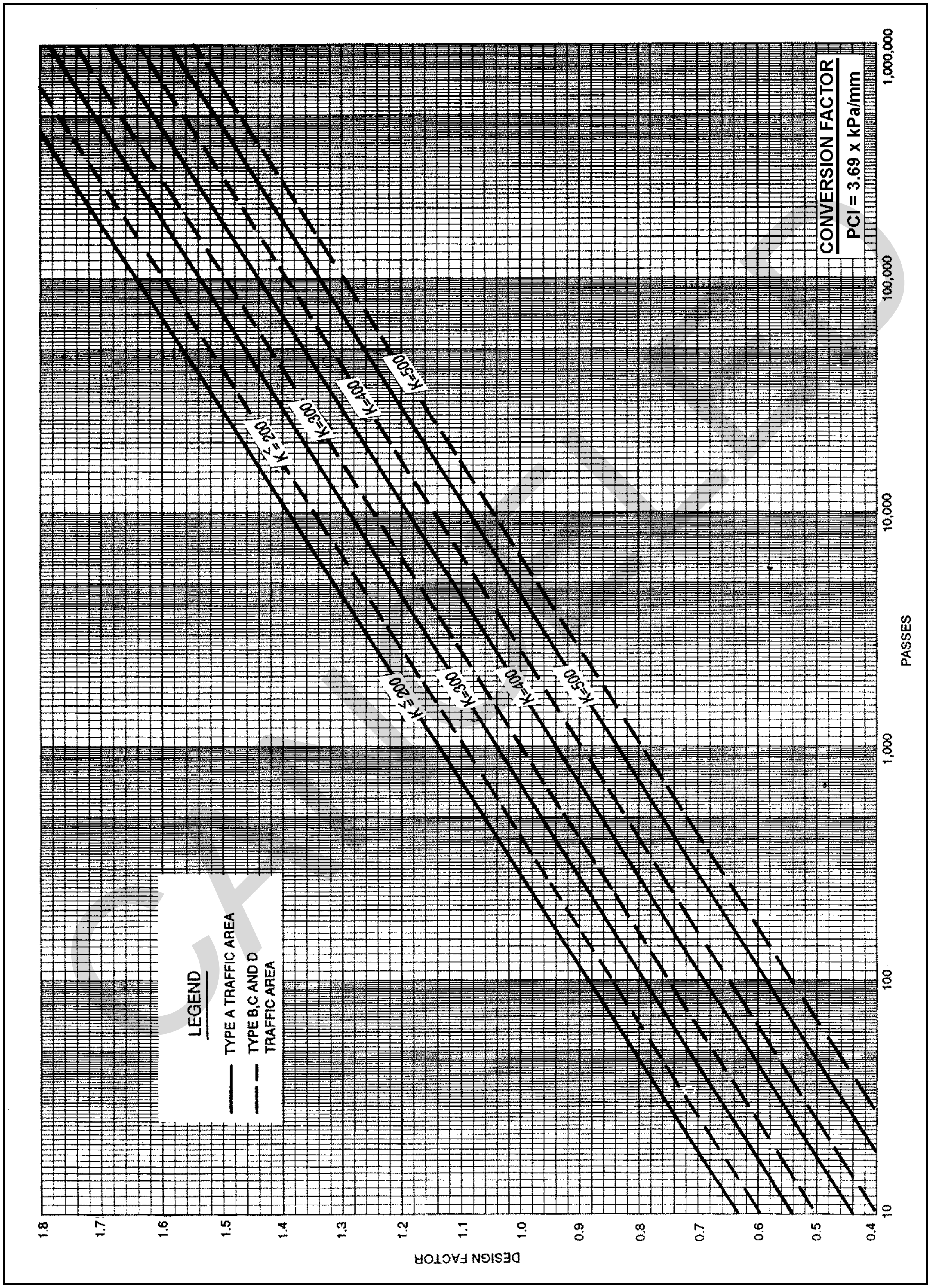


Figure 6-32. Design factors for standard evaluation, C-17 (Air Force Group Index 10)

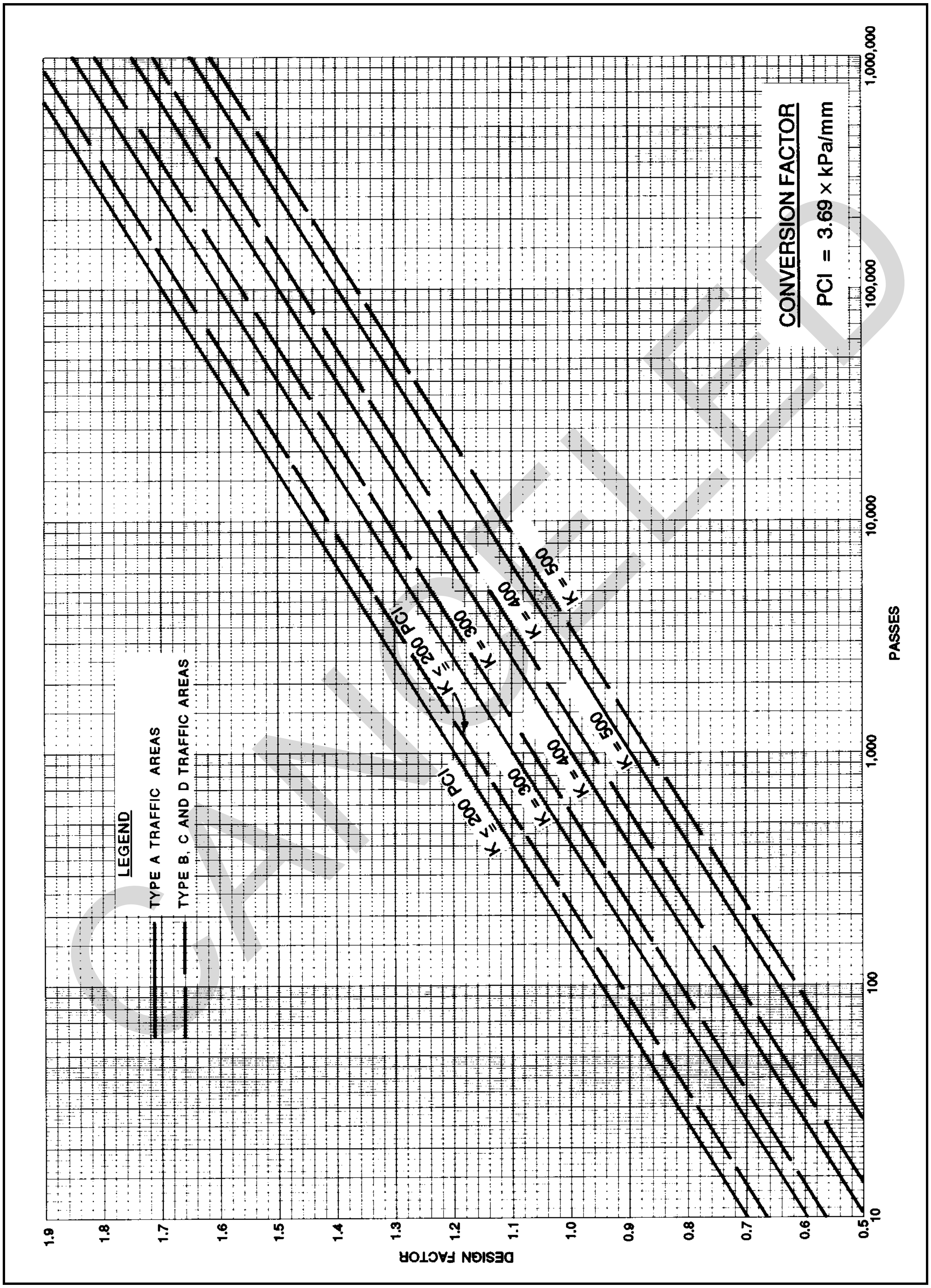


Figure 6-33. Design factors for standard evaluation, C-5 (Air Force Group Index 11)

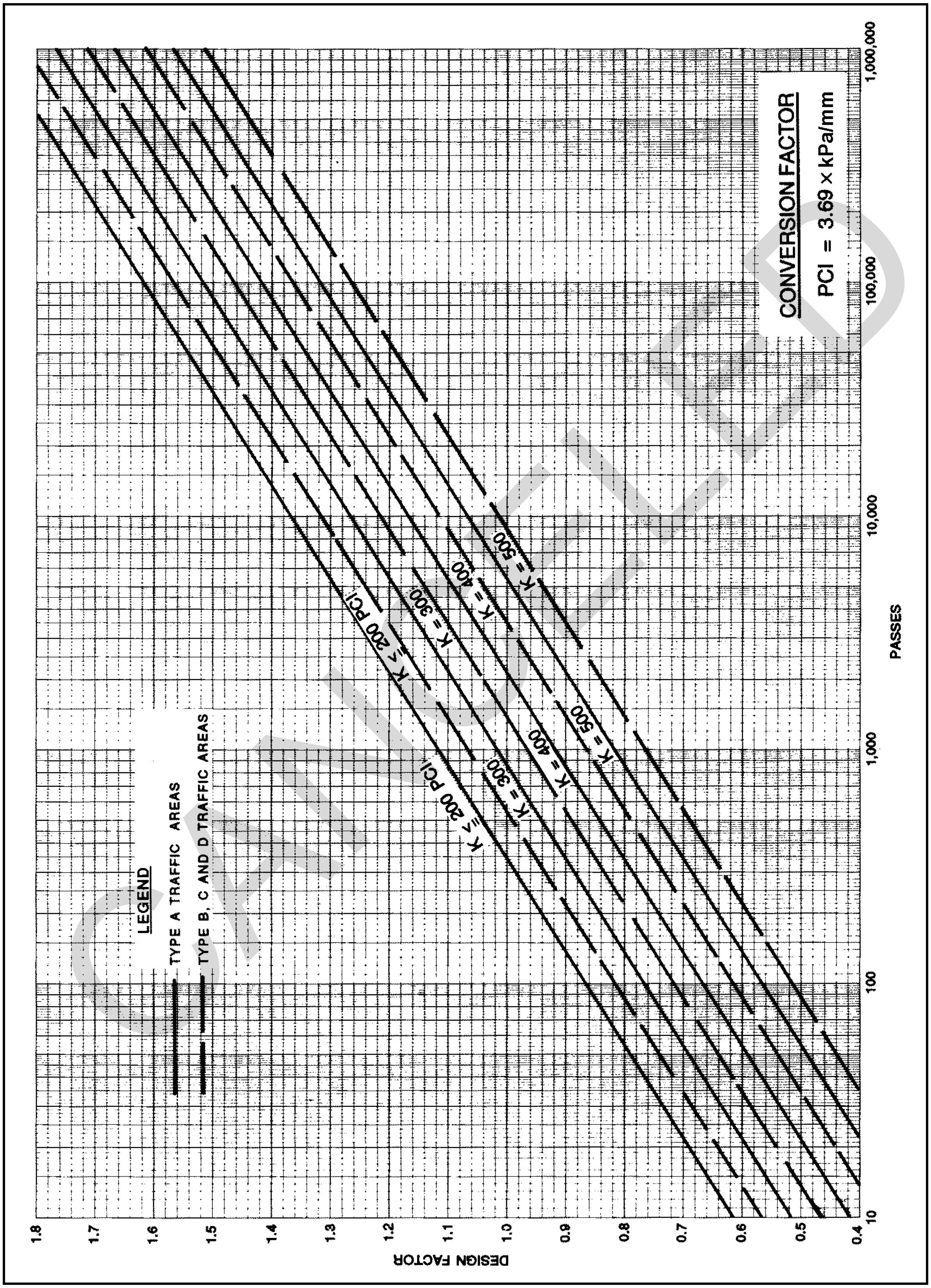


Figure 6-34. Design factors for standard evaluation, KC-10 (Air Force Group Index 12)

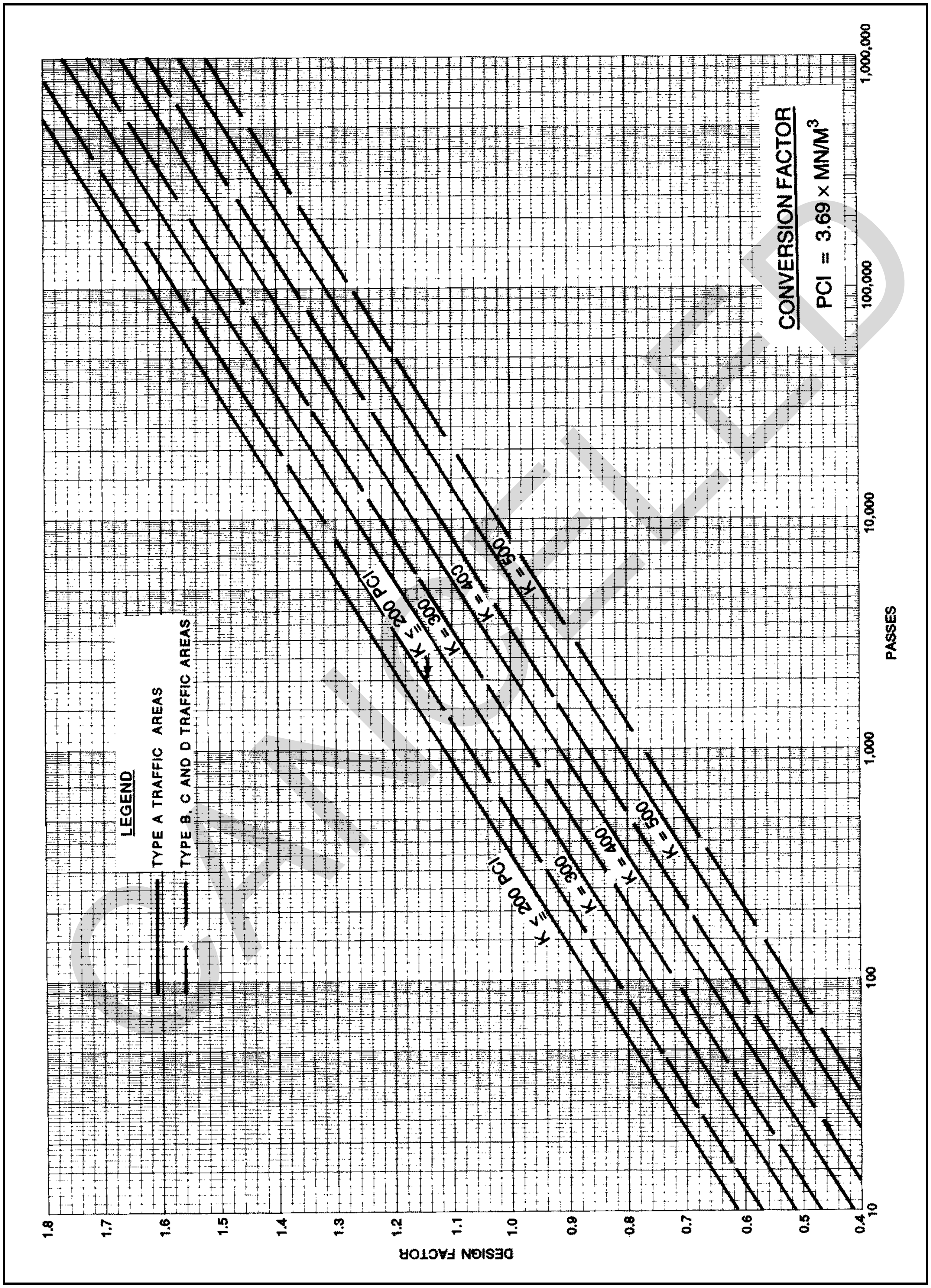


Figure 6-35. Design factors for standard evaluation, E-4 (Air Force Group Index 13)

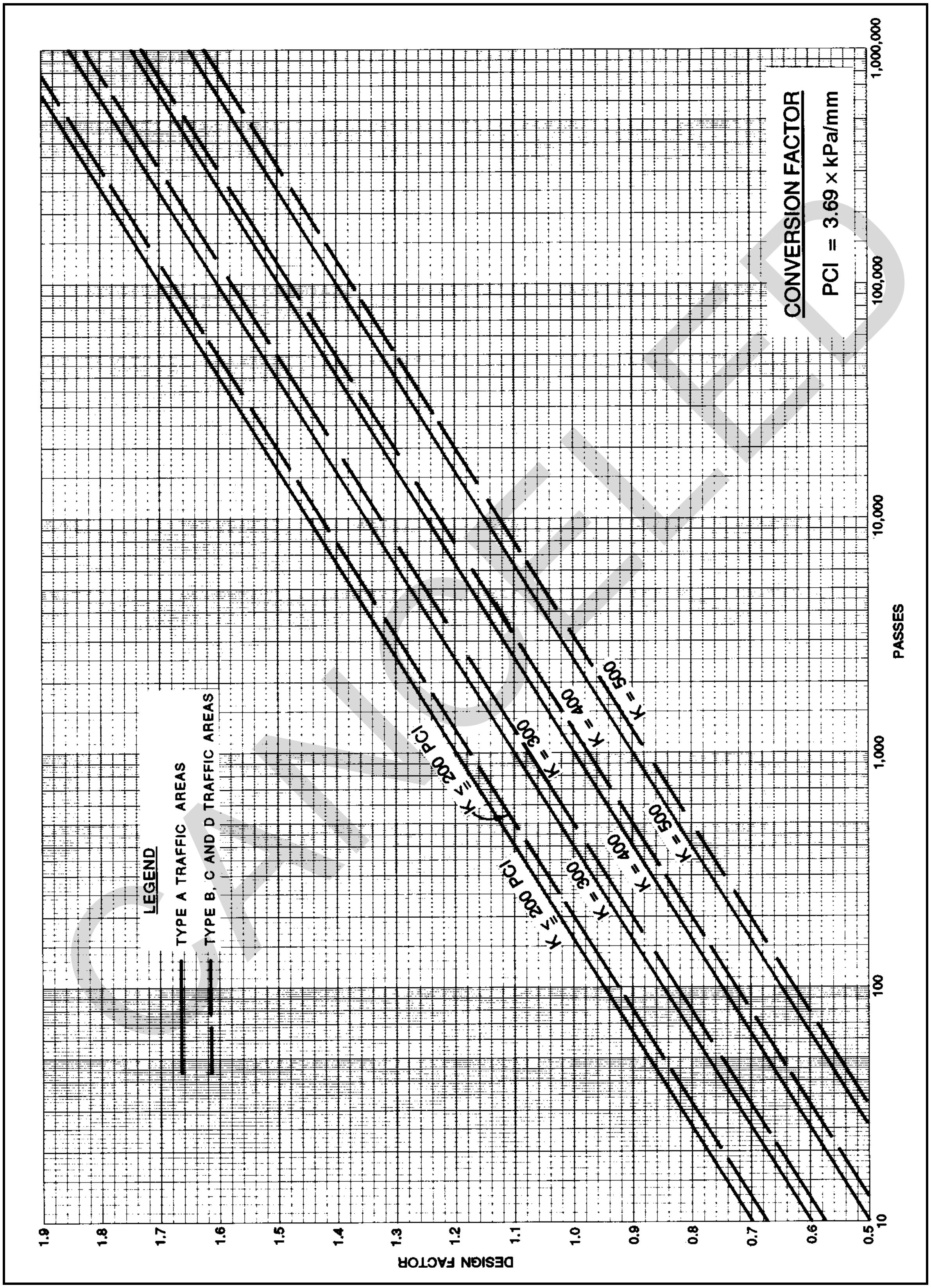


Figure 6-36. Design factors for standard evaluation, B-52 (Air Force Group Index 13)

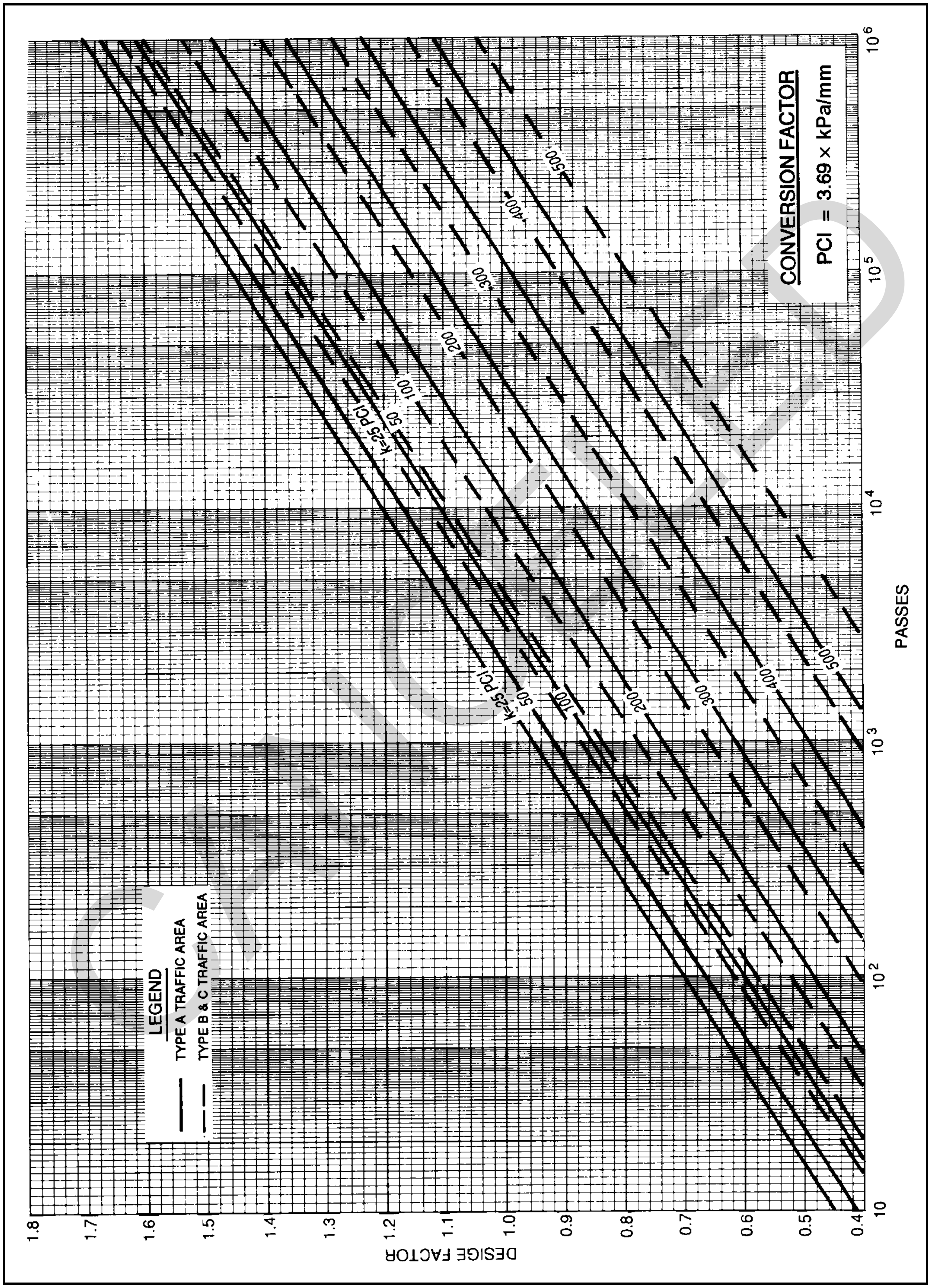


Figure 6-37. Design factors for extended life evaluation, UH-60

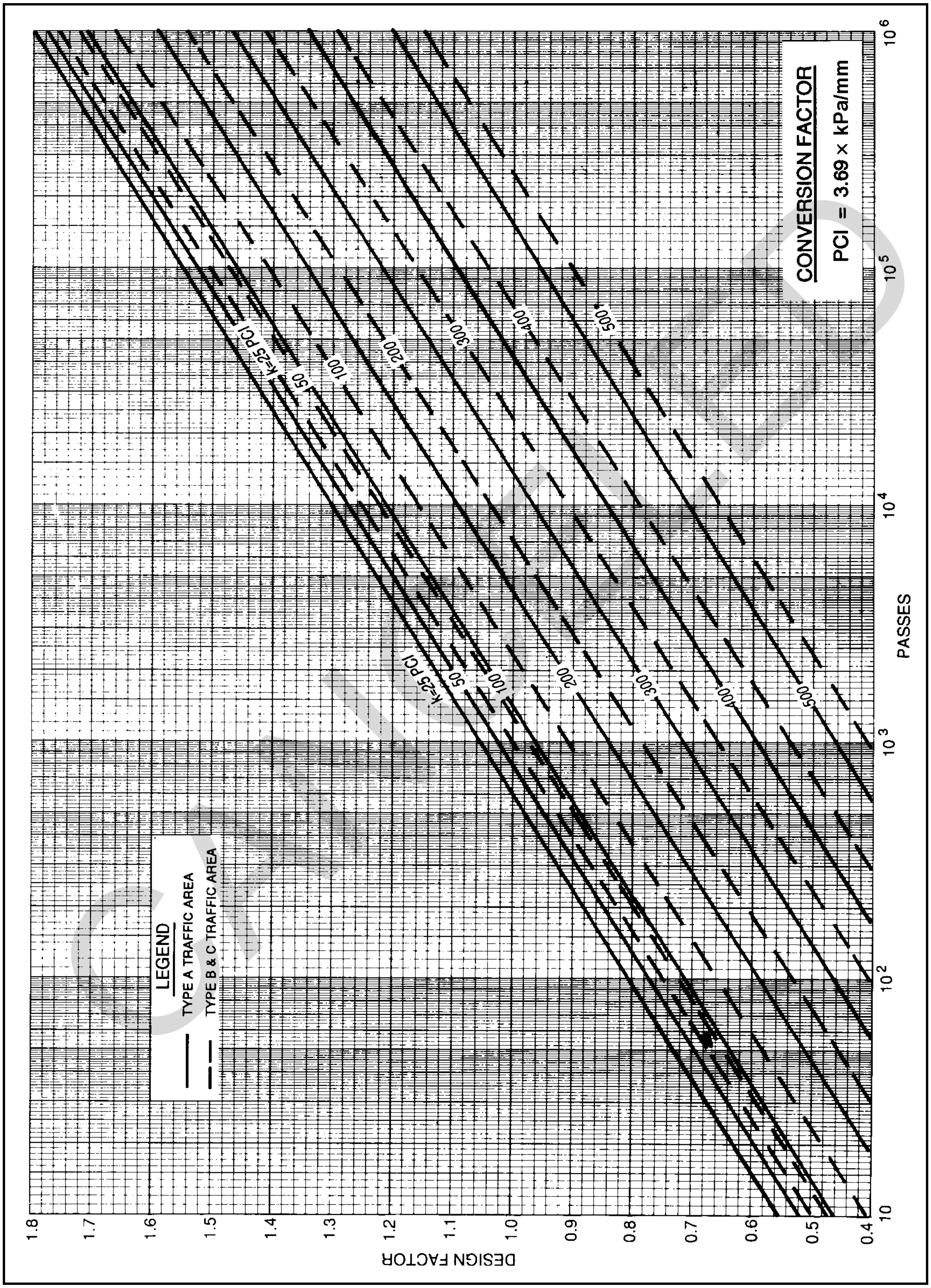


Figure 6-38. Design factors for extended life evaluation, CH-47

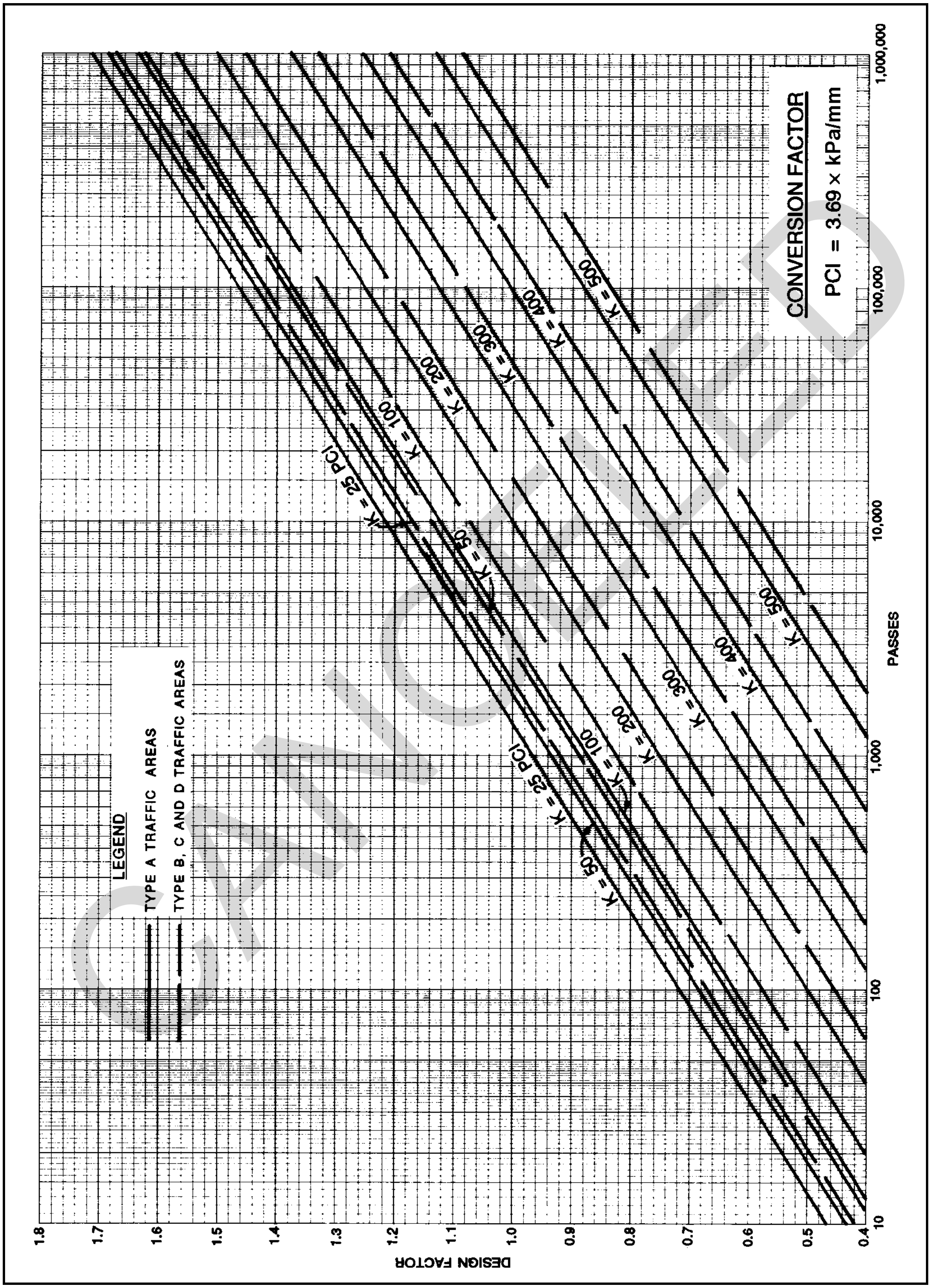


Figure 6-39. Design factors for extended life evaluation, OV-1

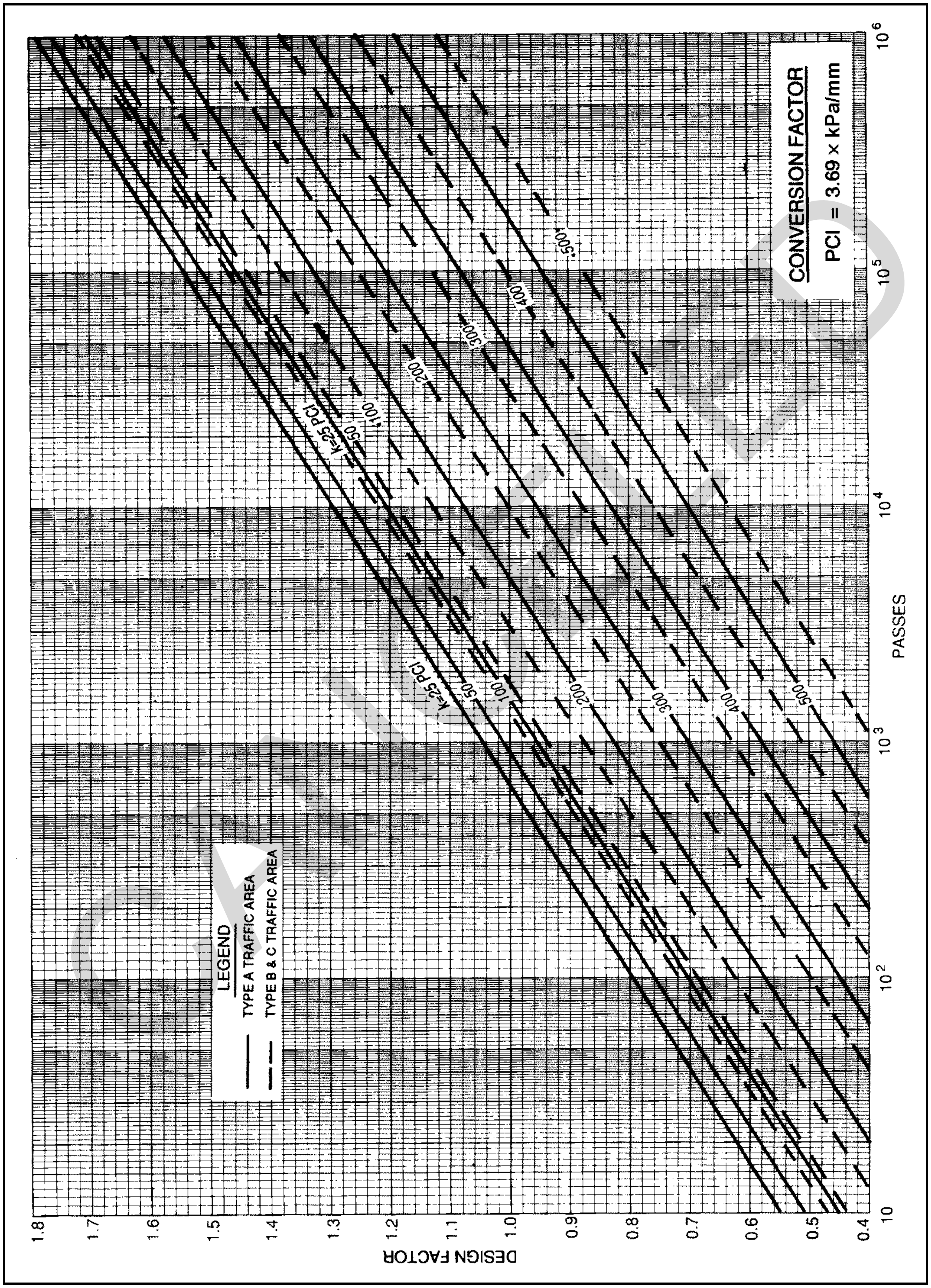


Figure 6-40. Design factors for extended life evaluation, C-12

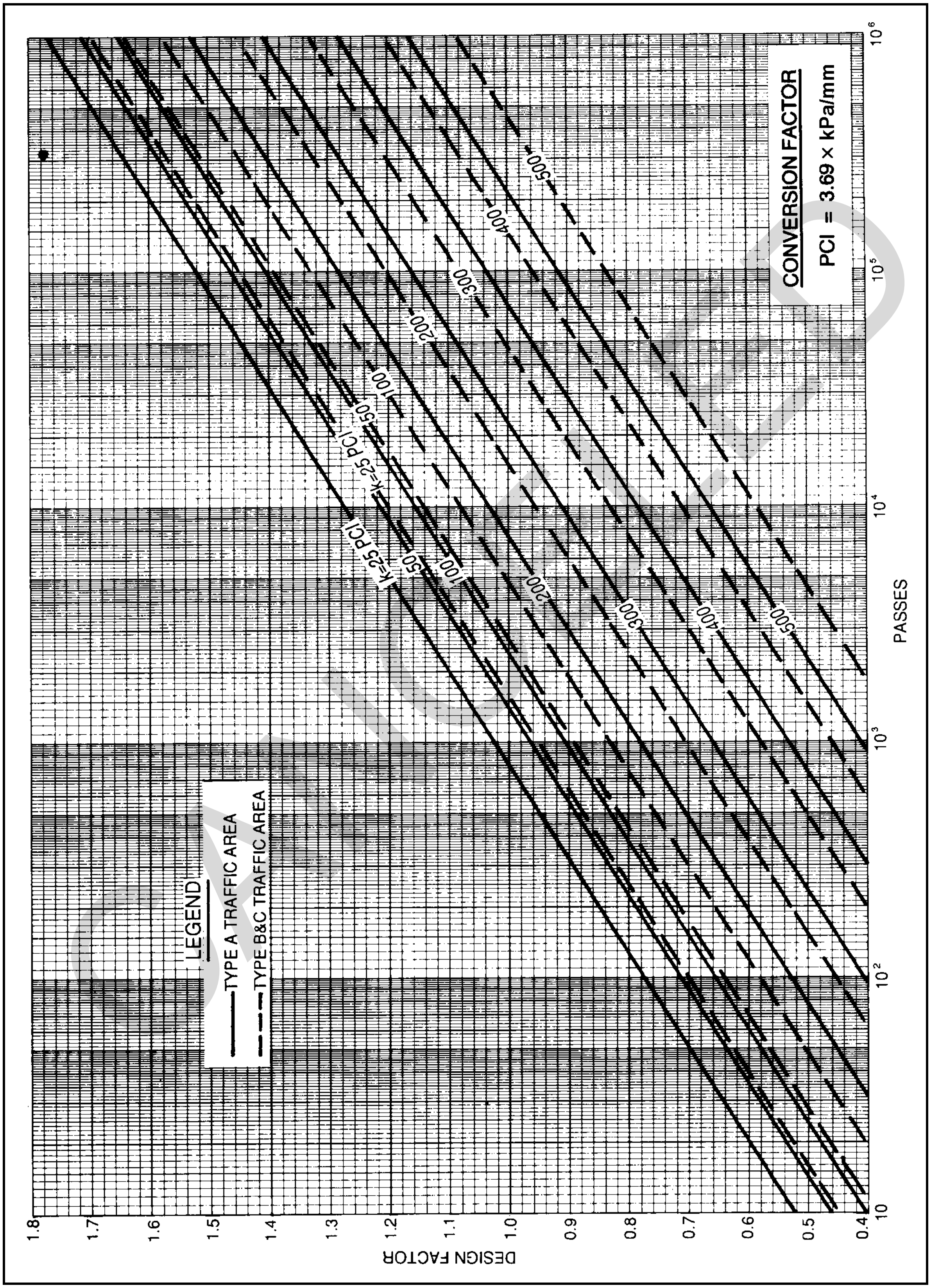


Figure 6-41. Design factors for extended life evaluation, C-23 (Air Force Group Index 1)

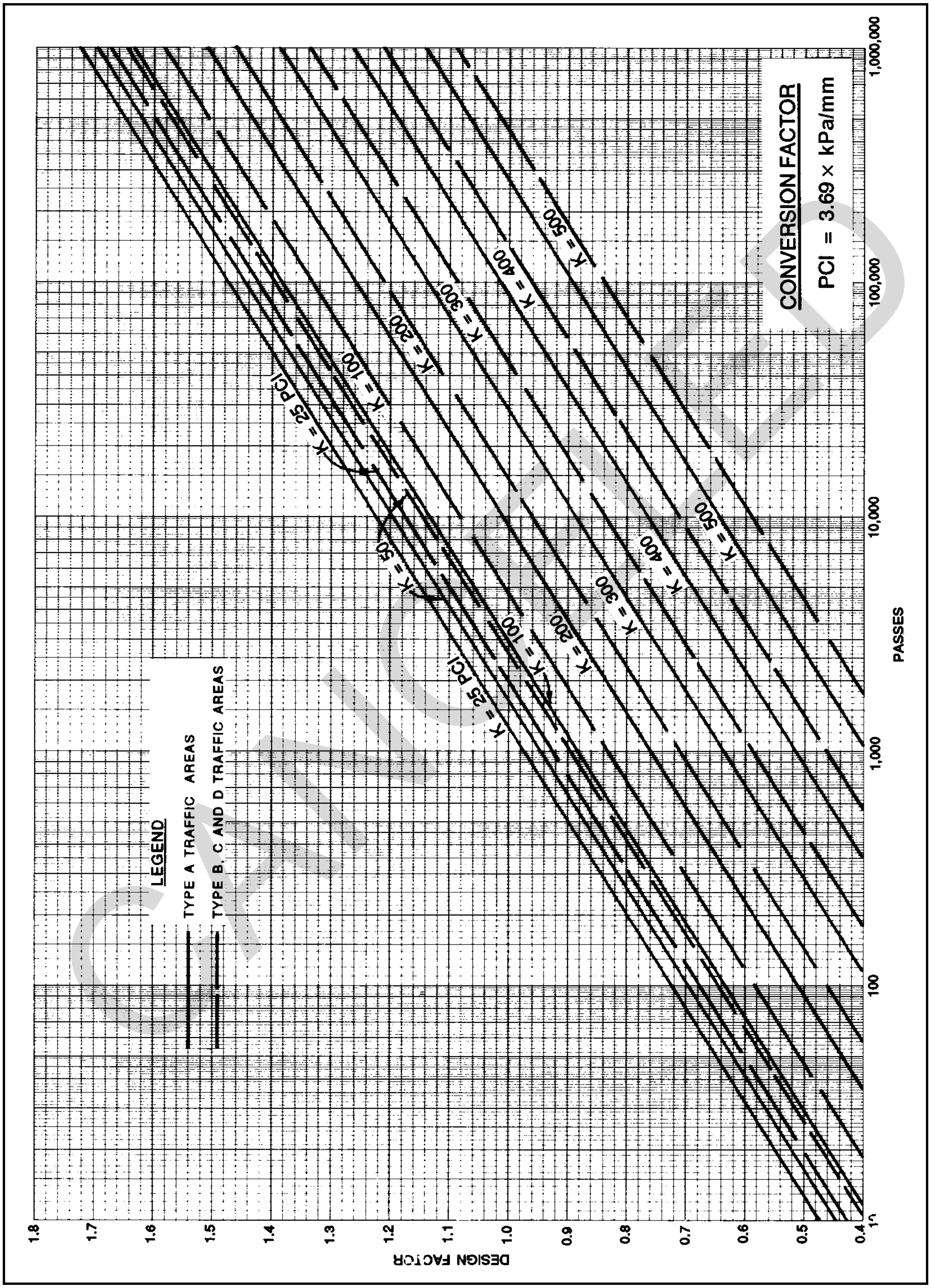


Figure 6-42. Design factors for extended life evaluation, F-4 (Air Force Group Index 2)

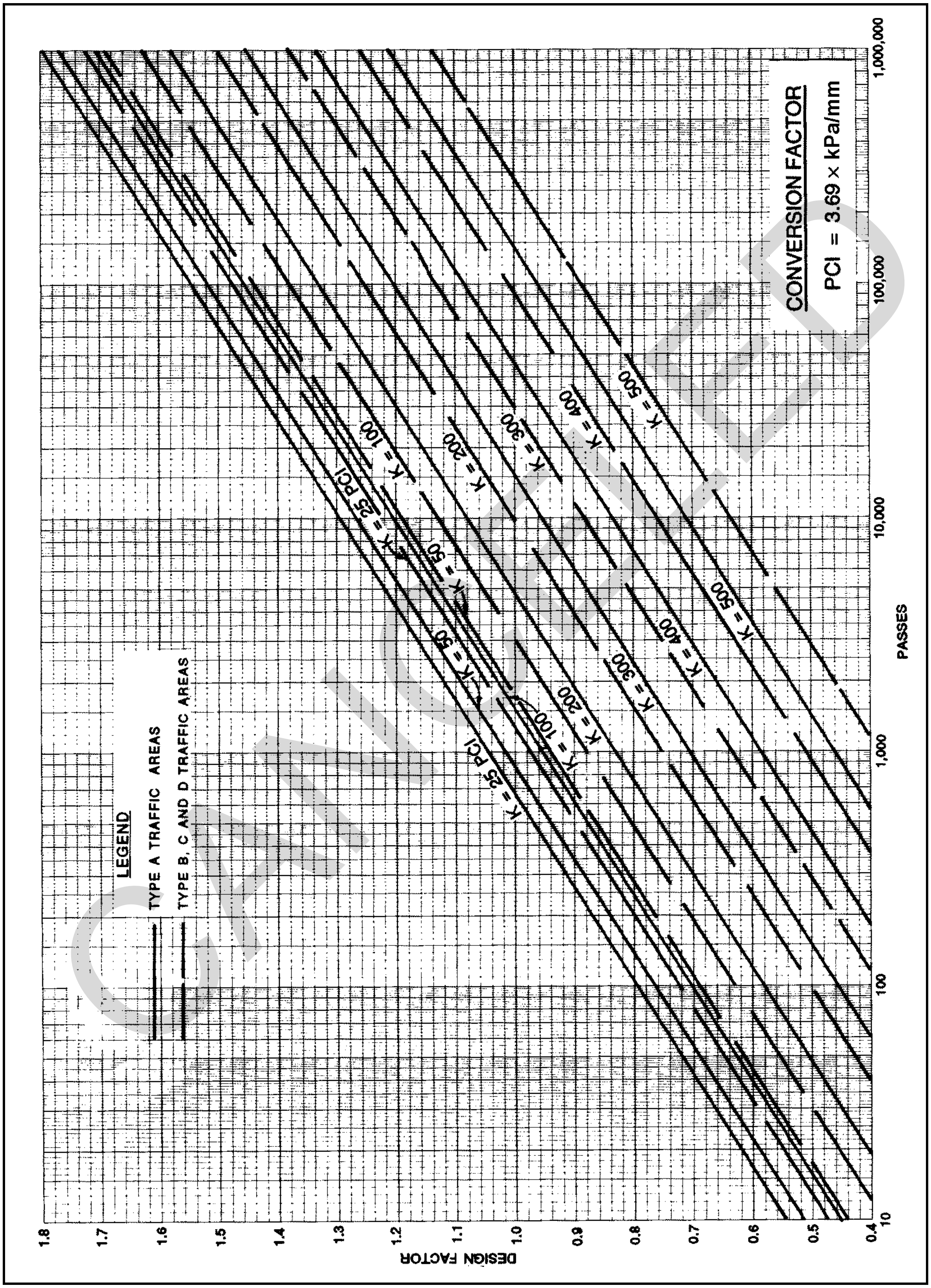


Figure 6-43. Design factors for extended life evaluation, F-111 (Air Force Group Index 3)

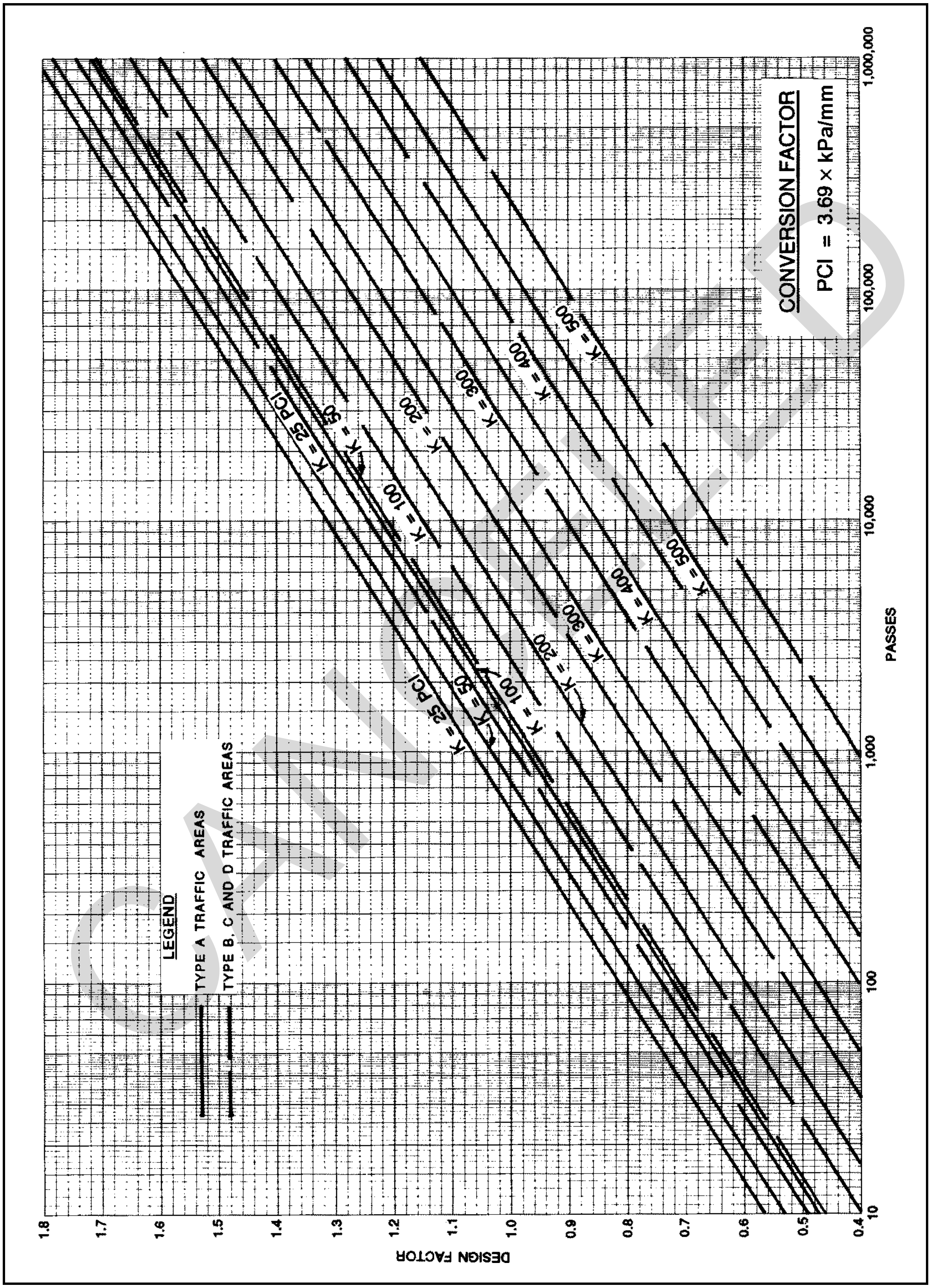


Figure 6-44. Design factors for extended life evaluation, C-130 (Air Force Group Index 4)

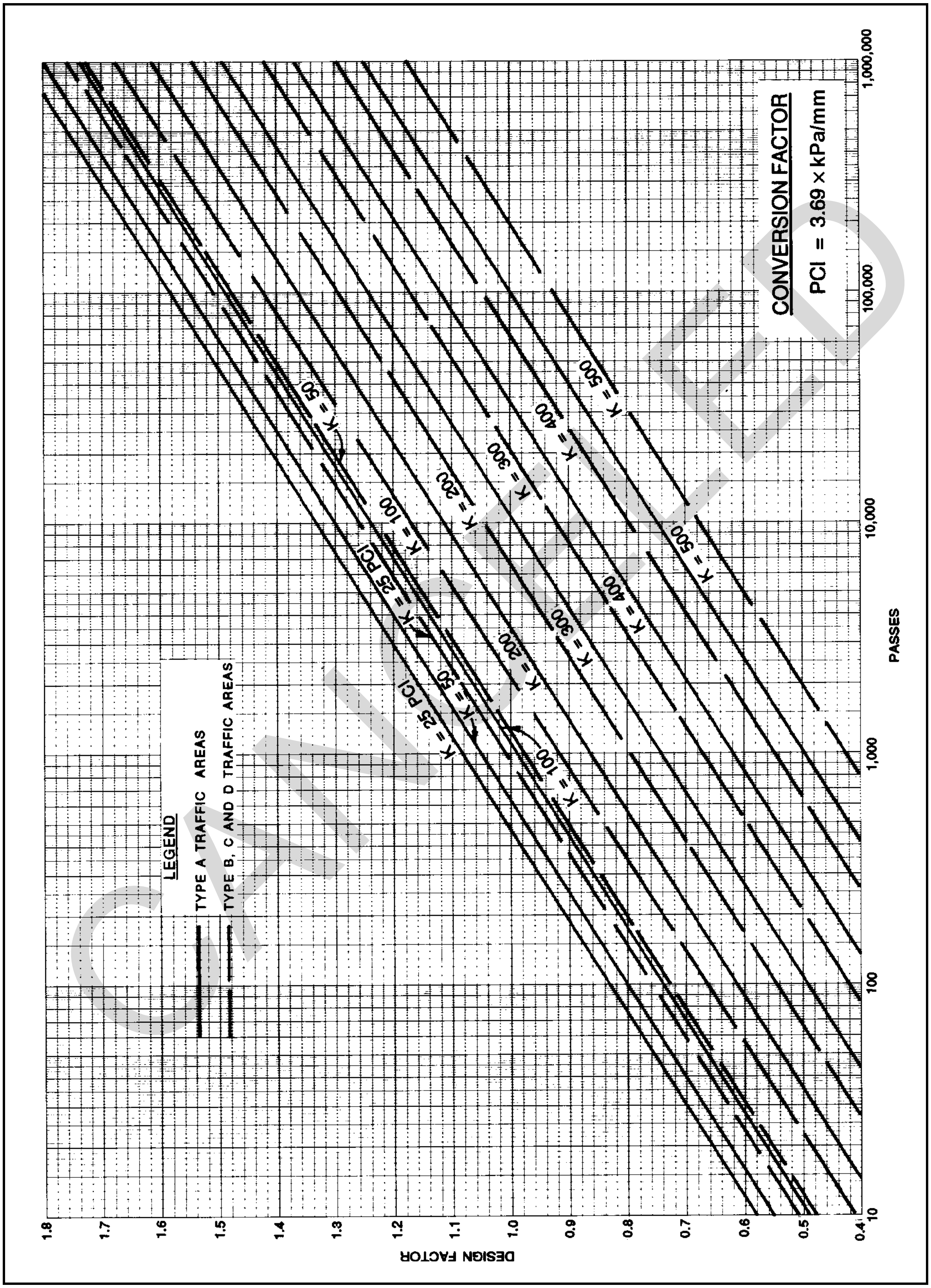


Figure 6-45. Design factors for extended life evaluation, C-9 (Air Force Group Index 5)

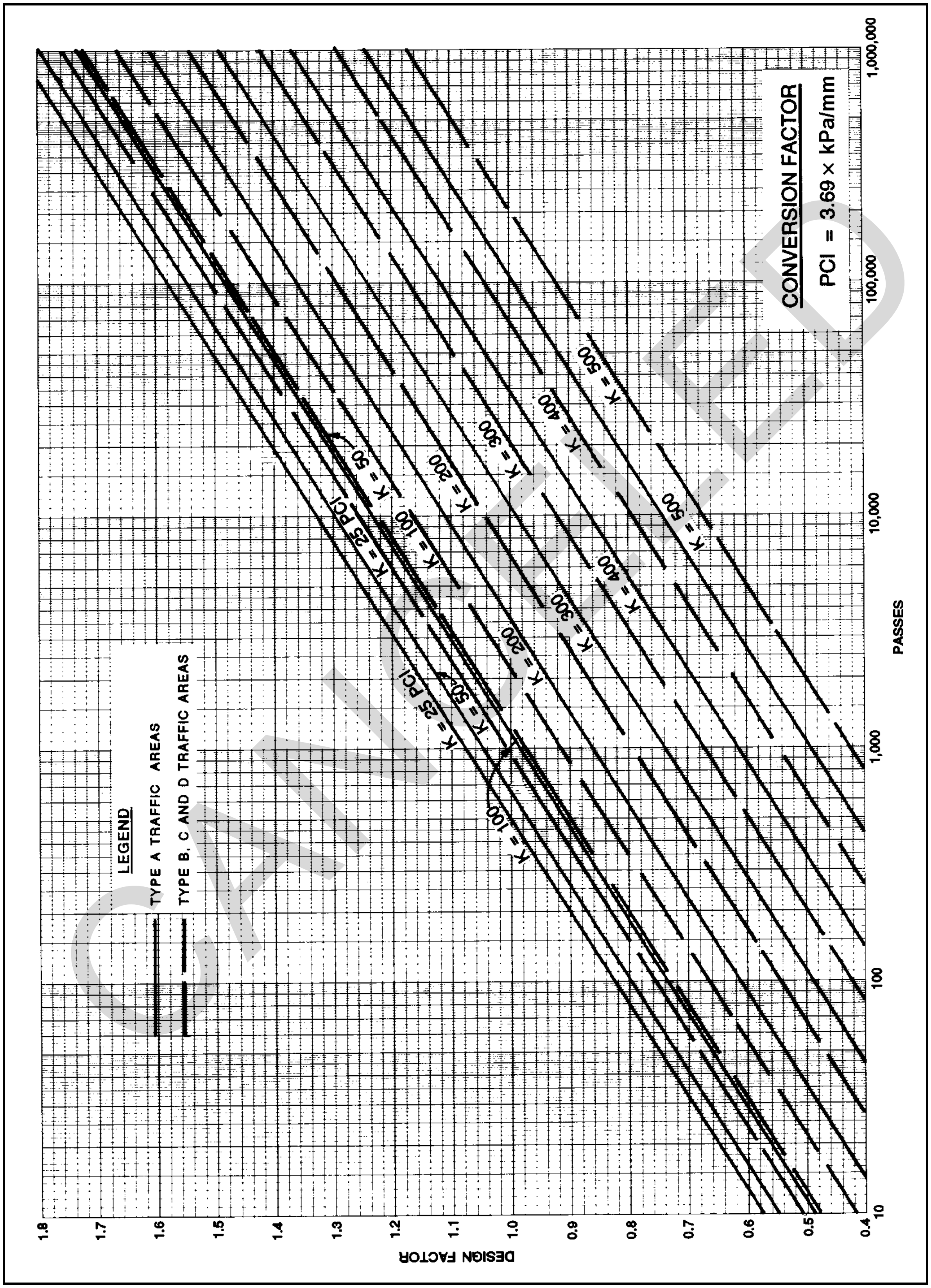


Figure 6-46. Design factors for extended life evaluation, T-43 (Air Force Group Index 6)

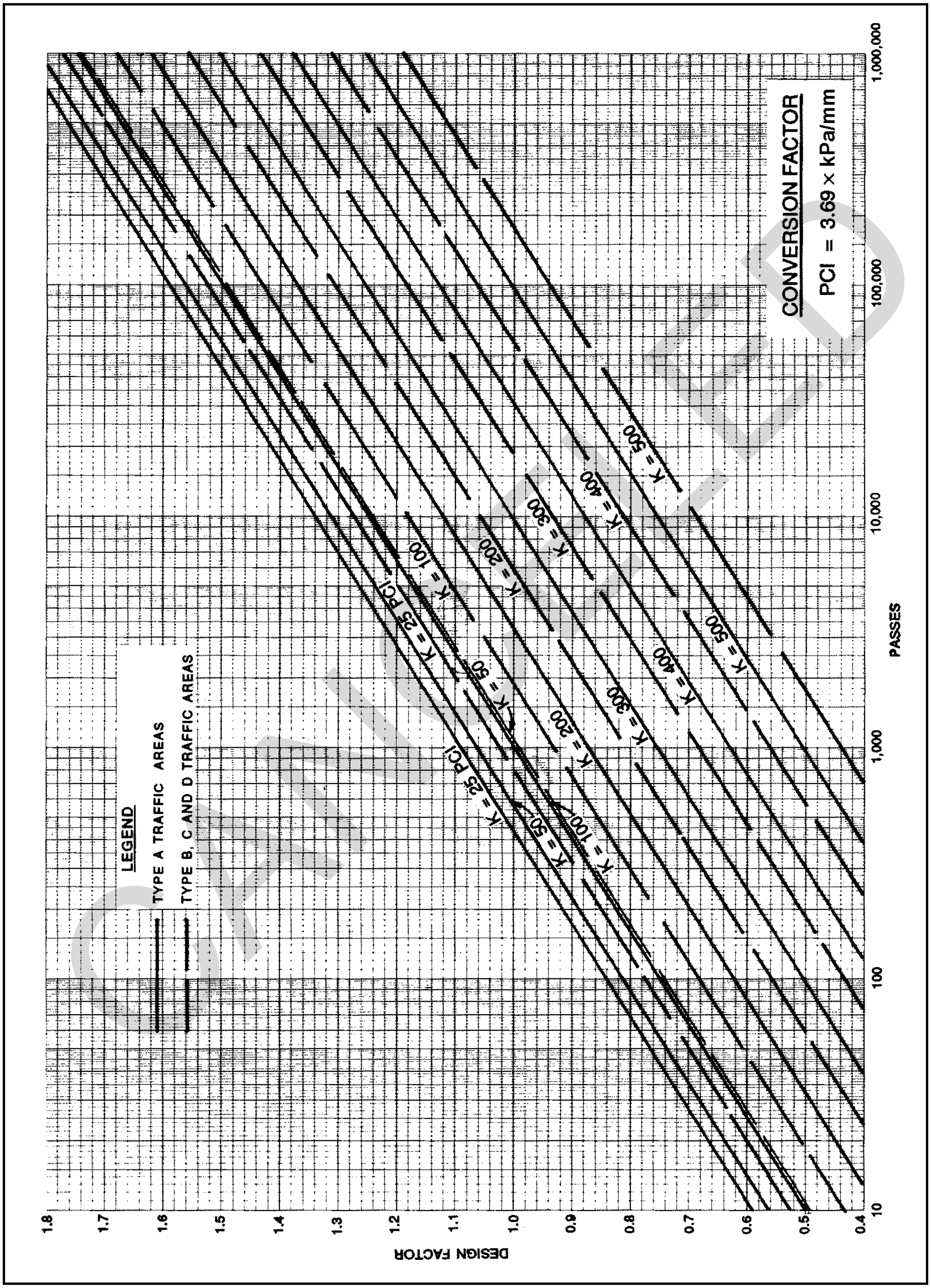


Figure 6-47. Design factors for extended life evaluation, B-727 (Air Force Group Index 7)

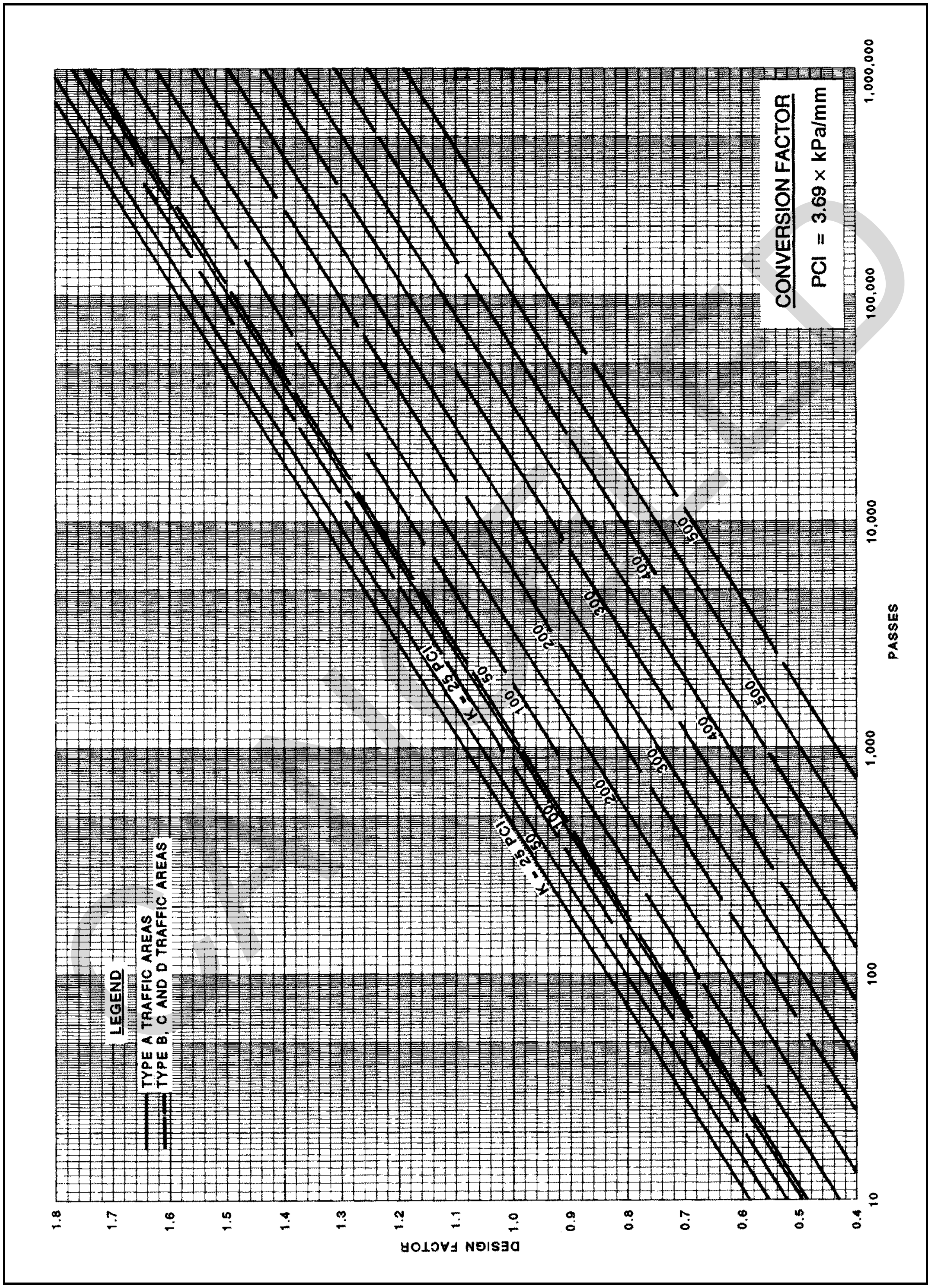


Figure 6-48. Design factors for extended life evaluation, E-3 (Air Force Group Index 8)

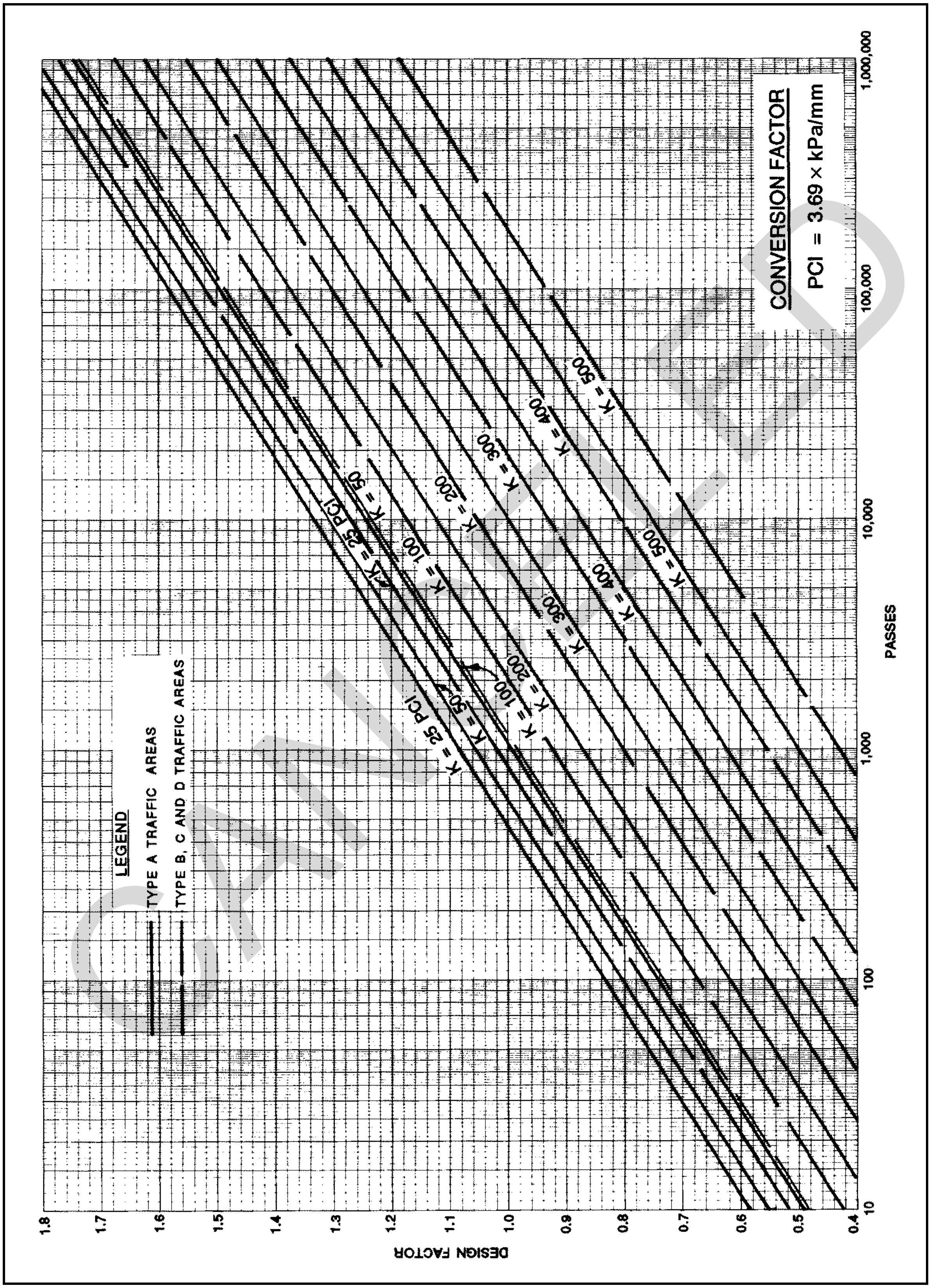


Figure 6-49. Design factors for extended life evaluation, C-141 (Air Force Group Index 9)

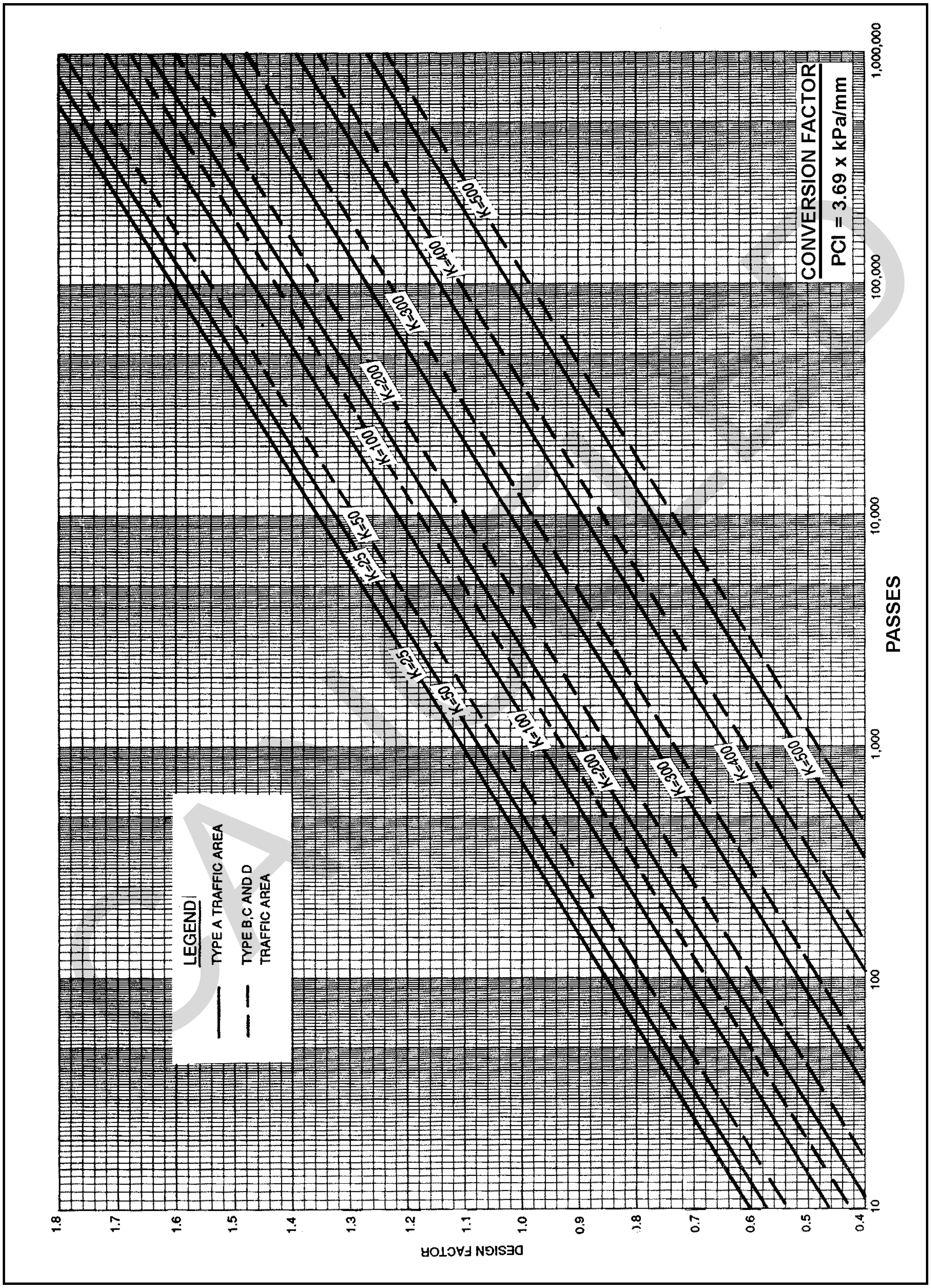


Figure 50. Design factors for extended life evaluation, C-17 (Air Force Group Index 10)

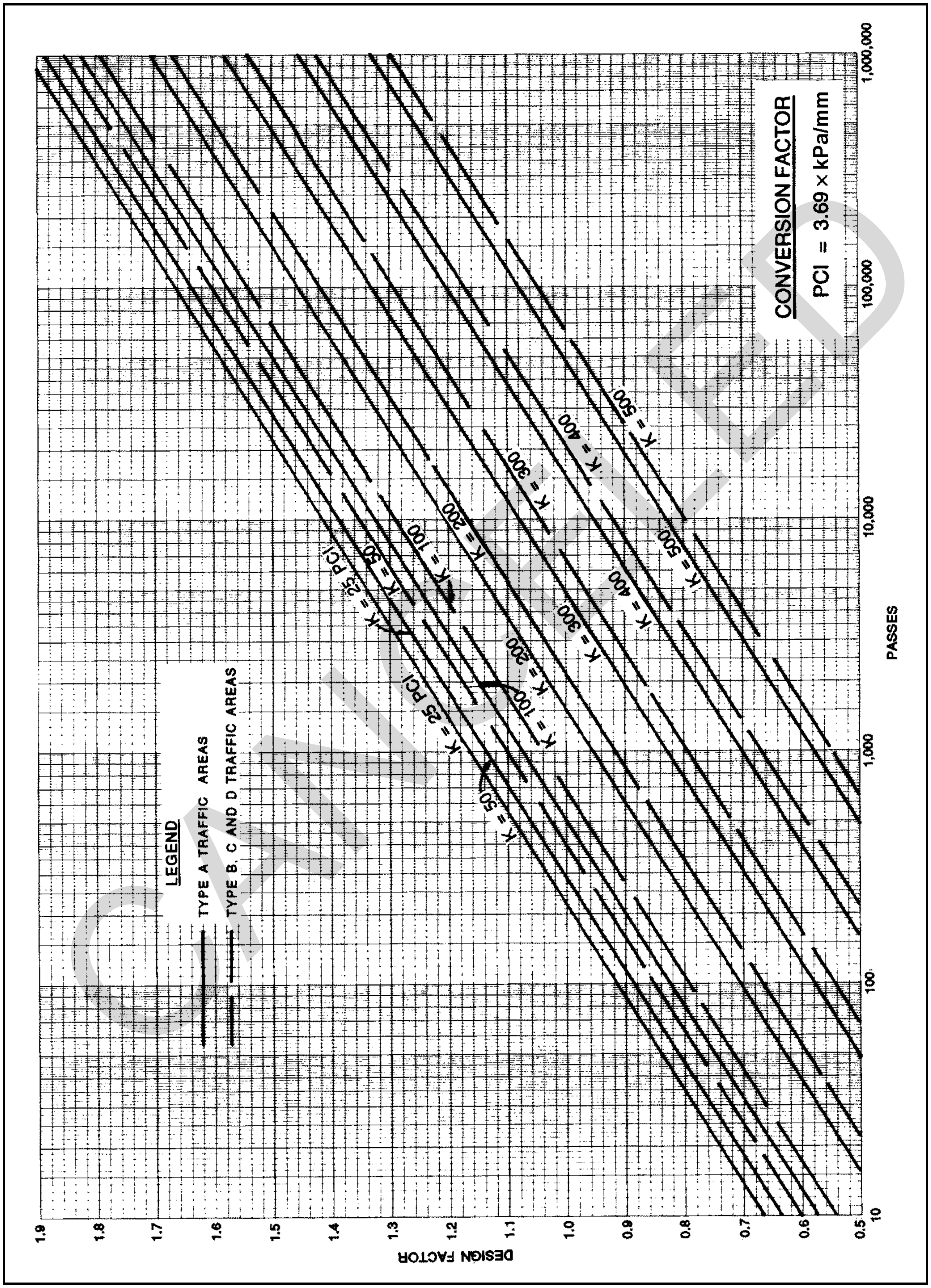


Figure 6-51. Design factors for extended life evaluation, C-5 (Air Force Group Index 11)

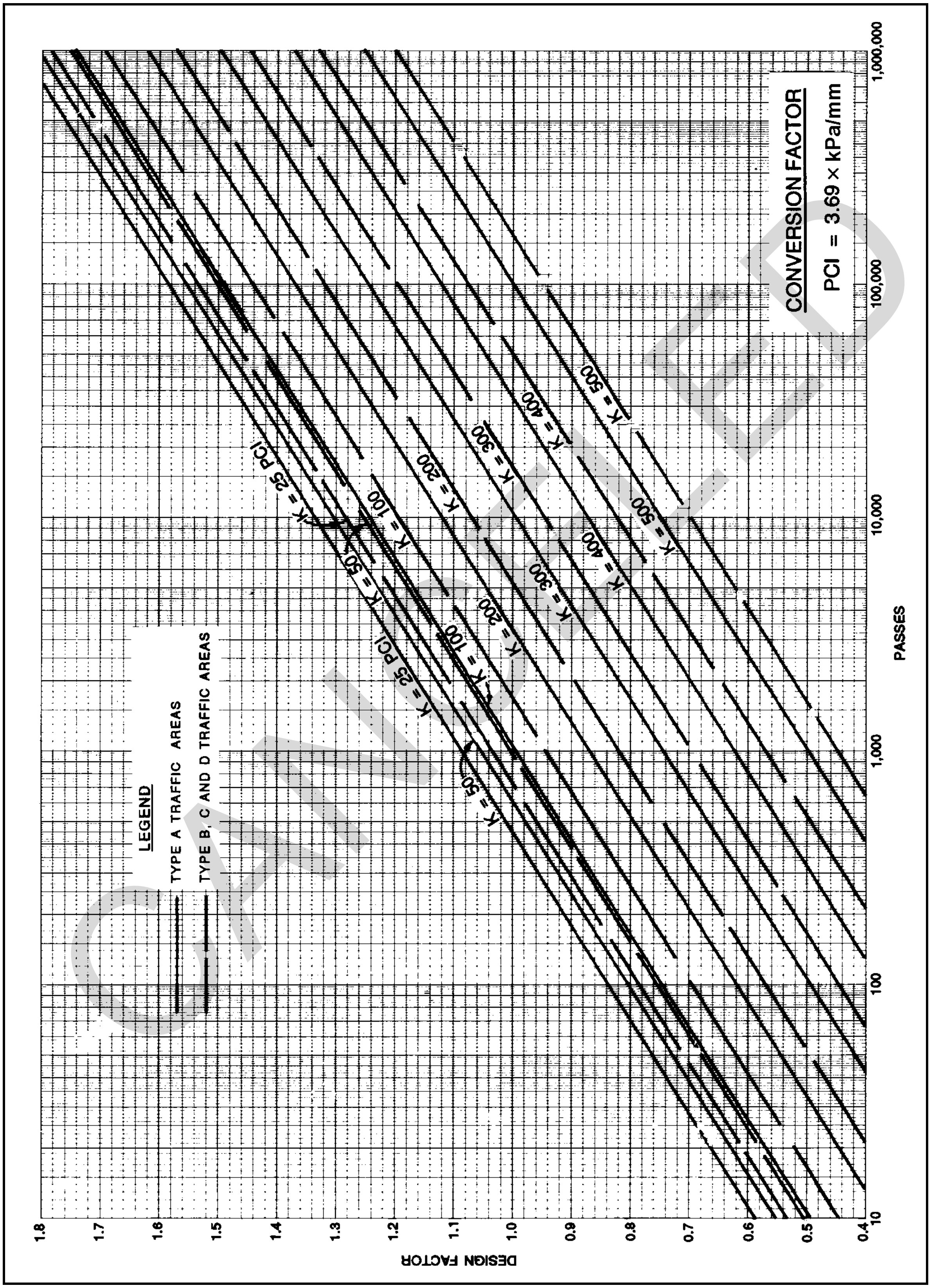


Figure 6-52. Design factors for extended life evaluation, KC-10 (Air Force Group Index 12)

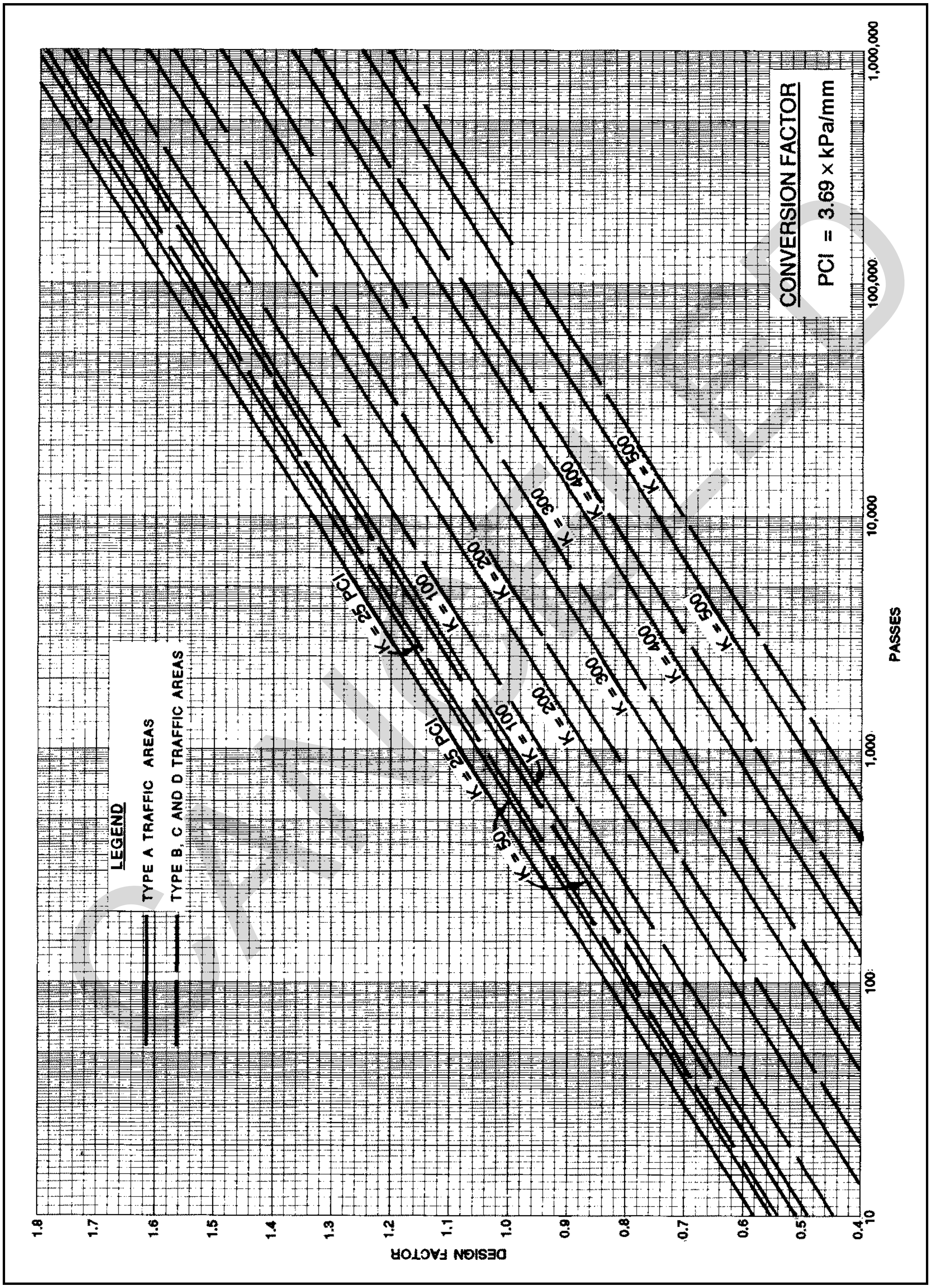


Figure 6-53. Design factors for extended life evaluation, E-4 (Air Force Group Index 13)

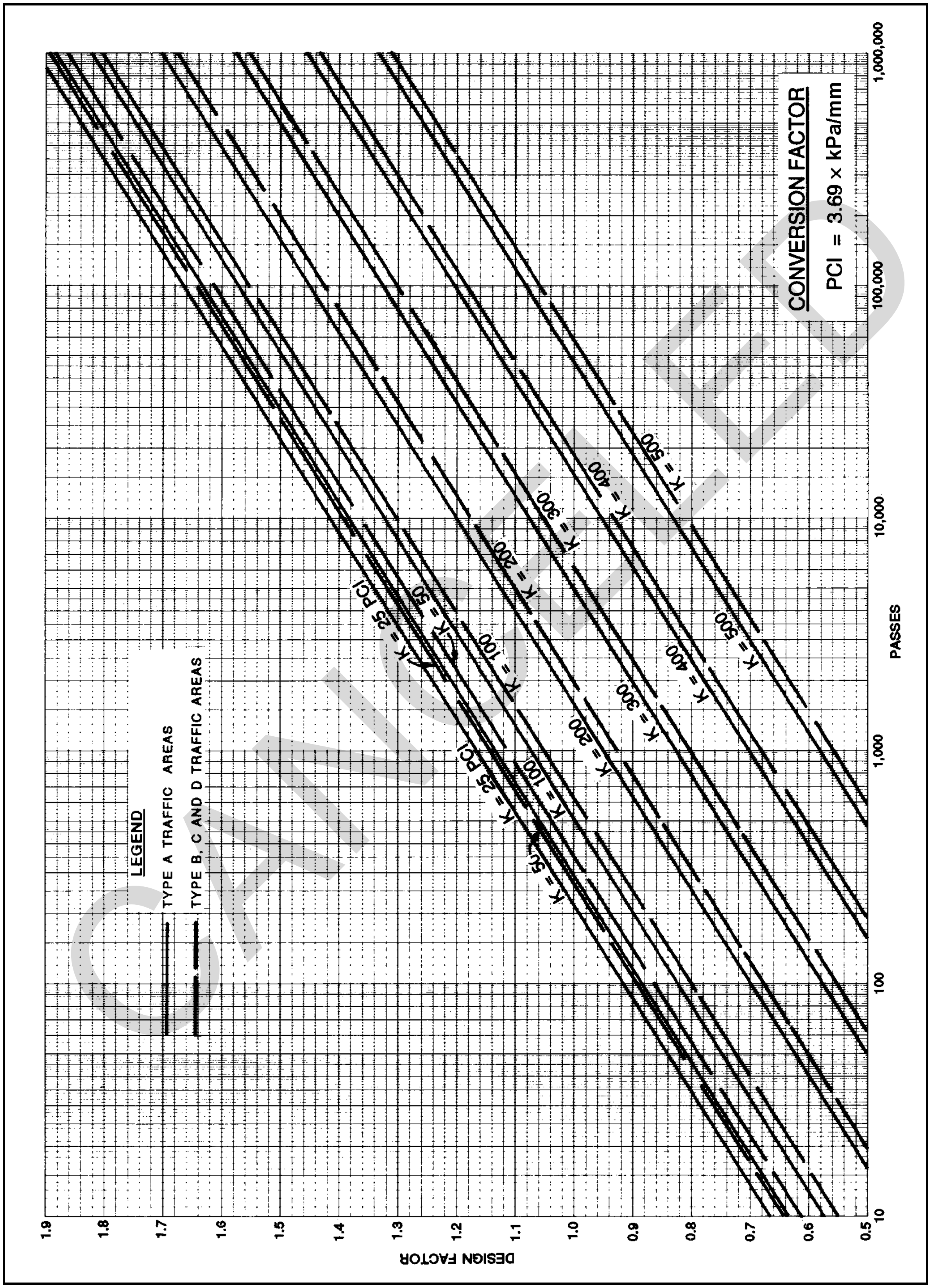


Figure 6-54. Design factors for extended life evaluation, B-52 (Air Force Group Index 14)

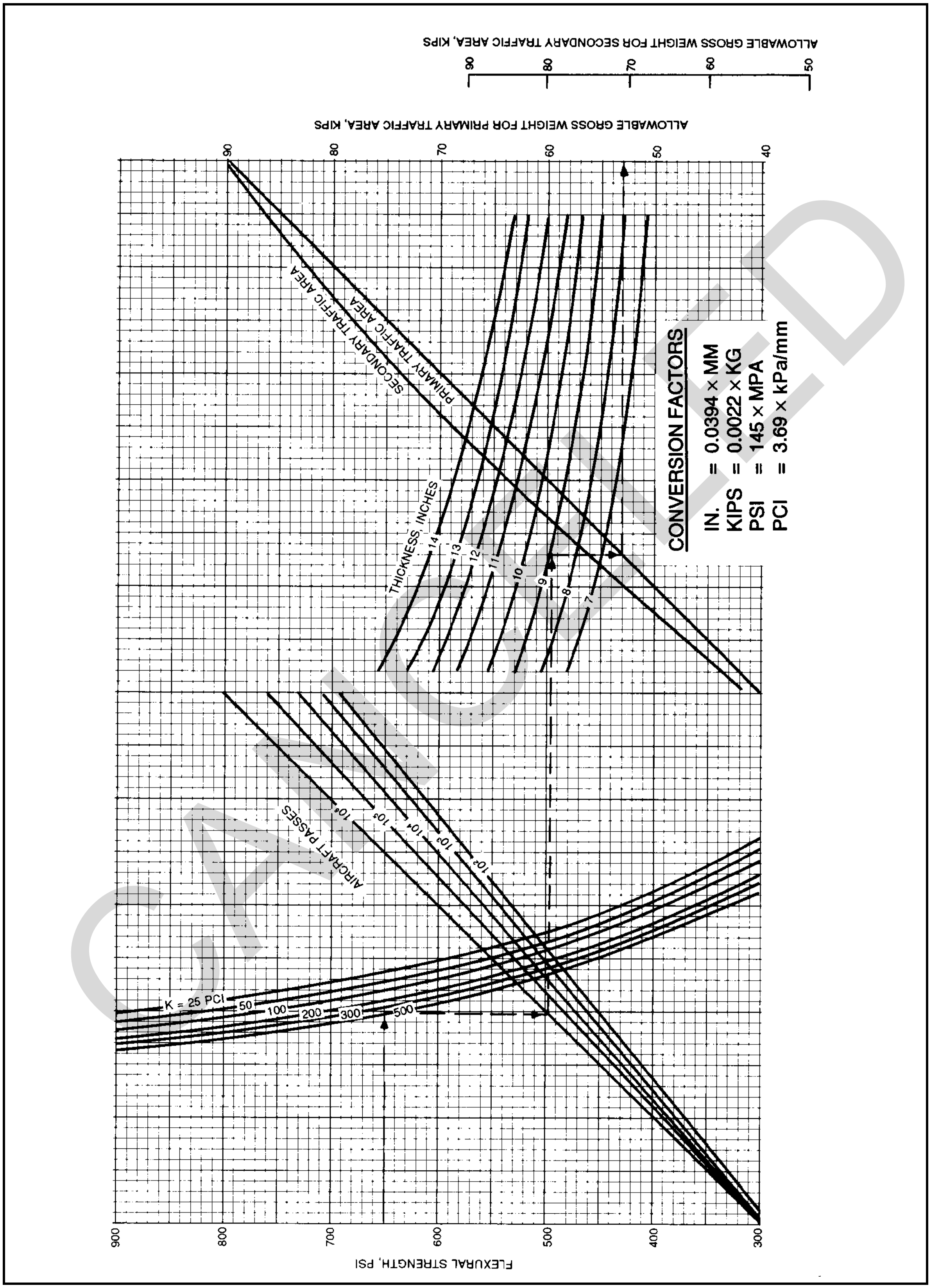


Figure 6-55. Rigid pavement evaluation curves, F-14 (interior load criteria)

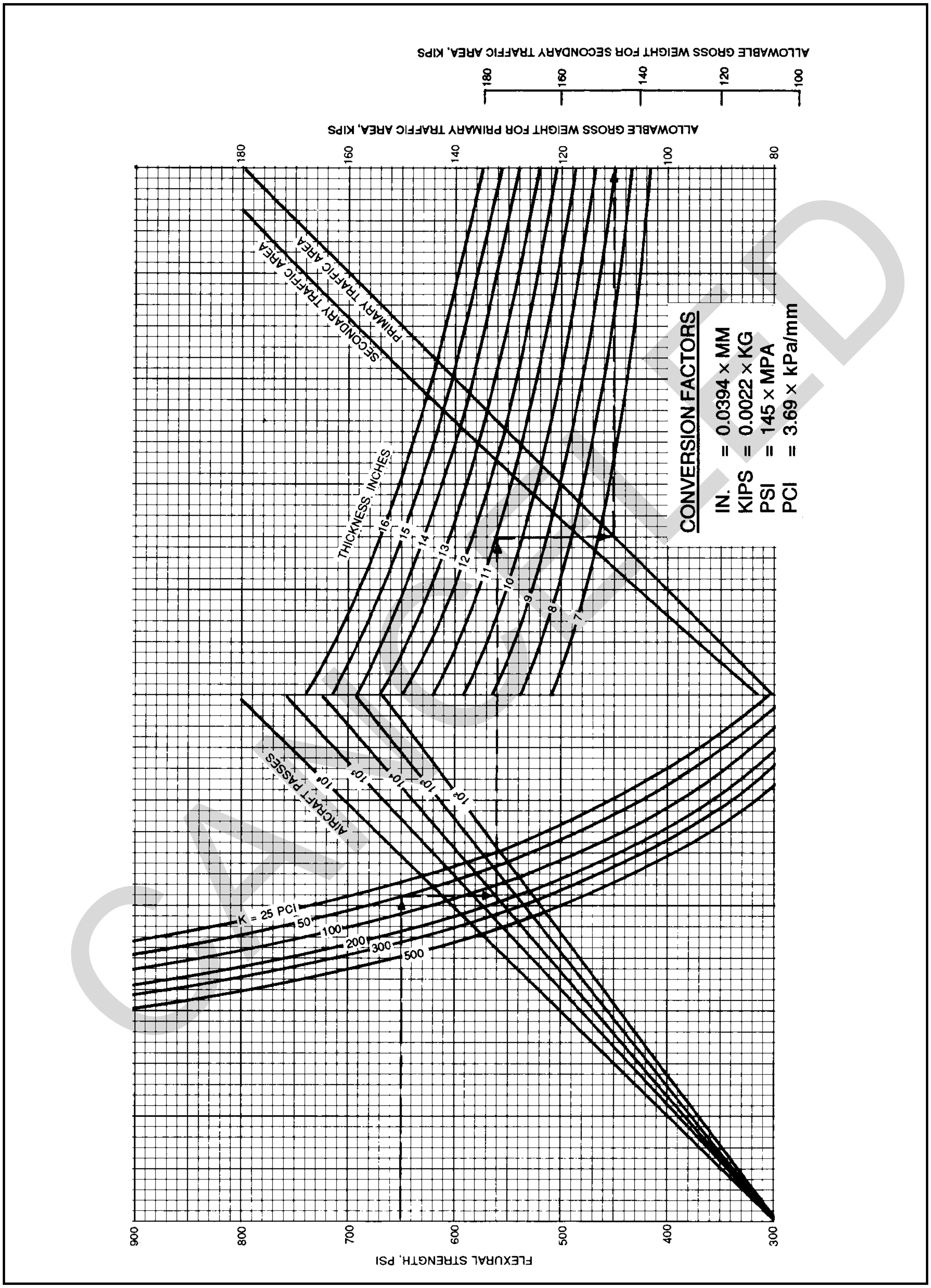


Figure 6-56. Rigid pavement evaluation curves, P-3 (interior load criteria)

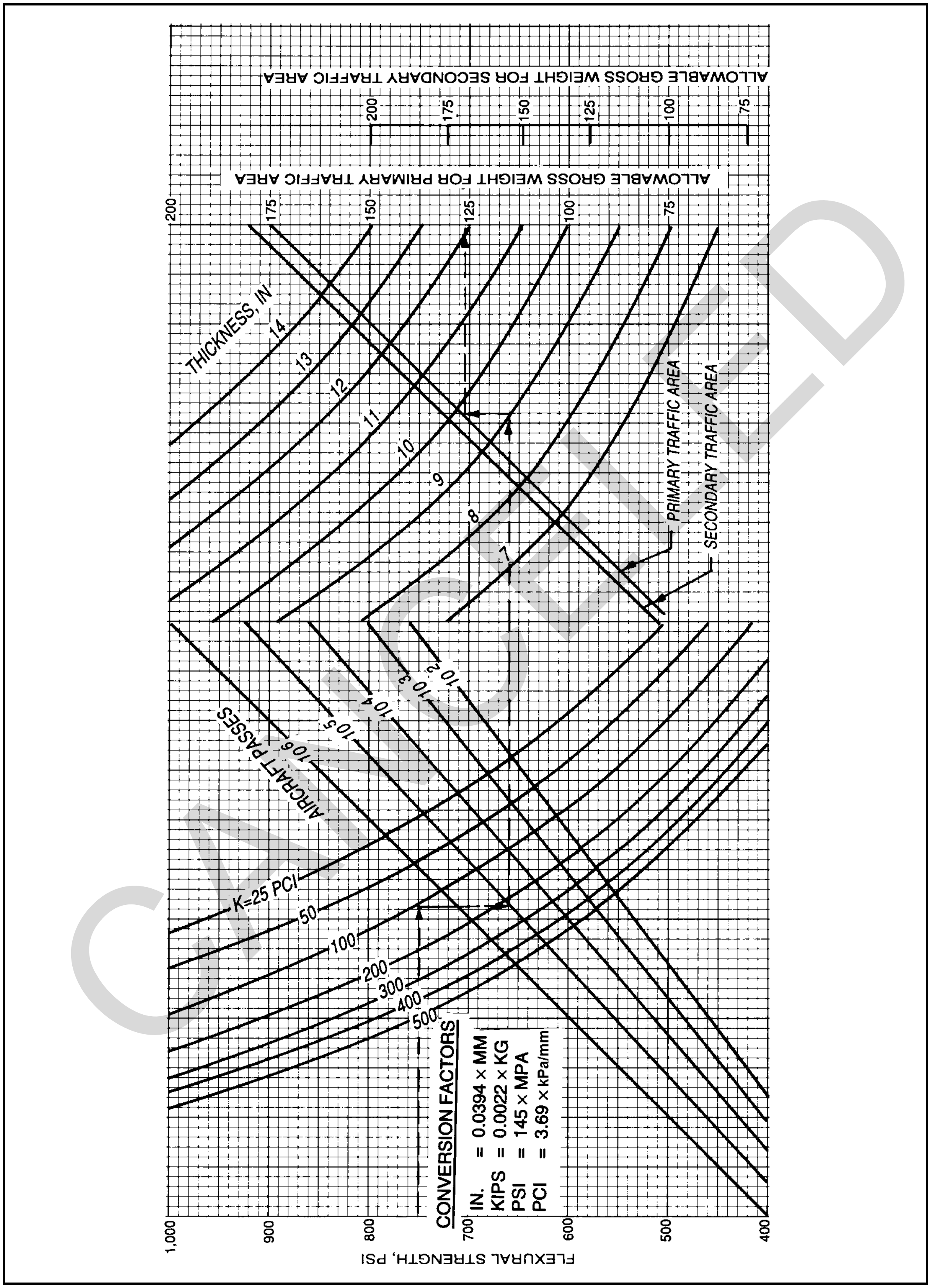


Figure 6-57. Rigid pavement evaluation curves, C-130 (interior load criteria)

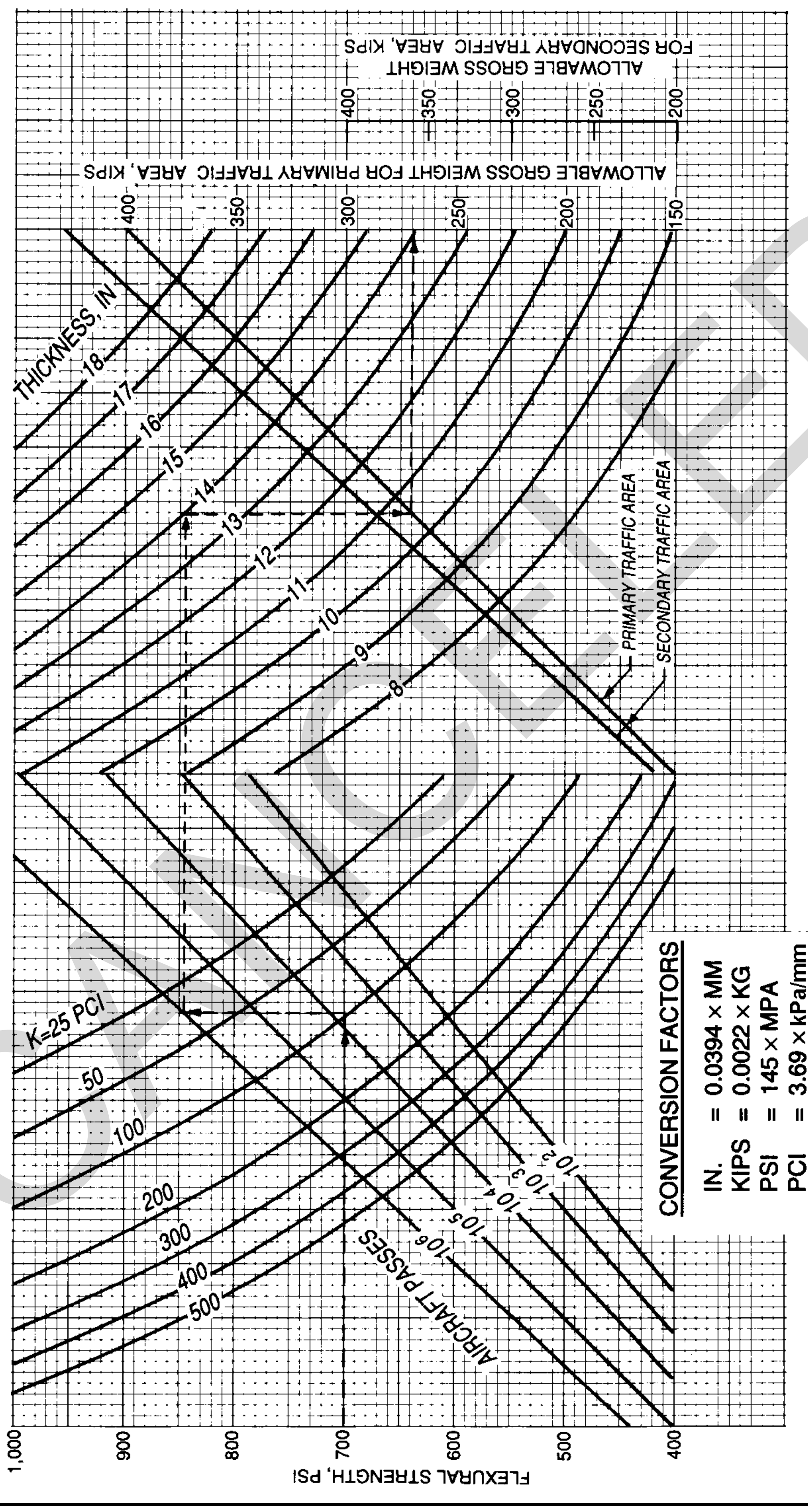


Figure 6-58. Rigid pavement evaluation curves, C-141 (interior load criteria)

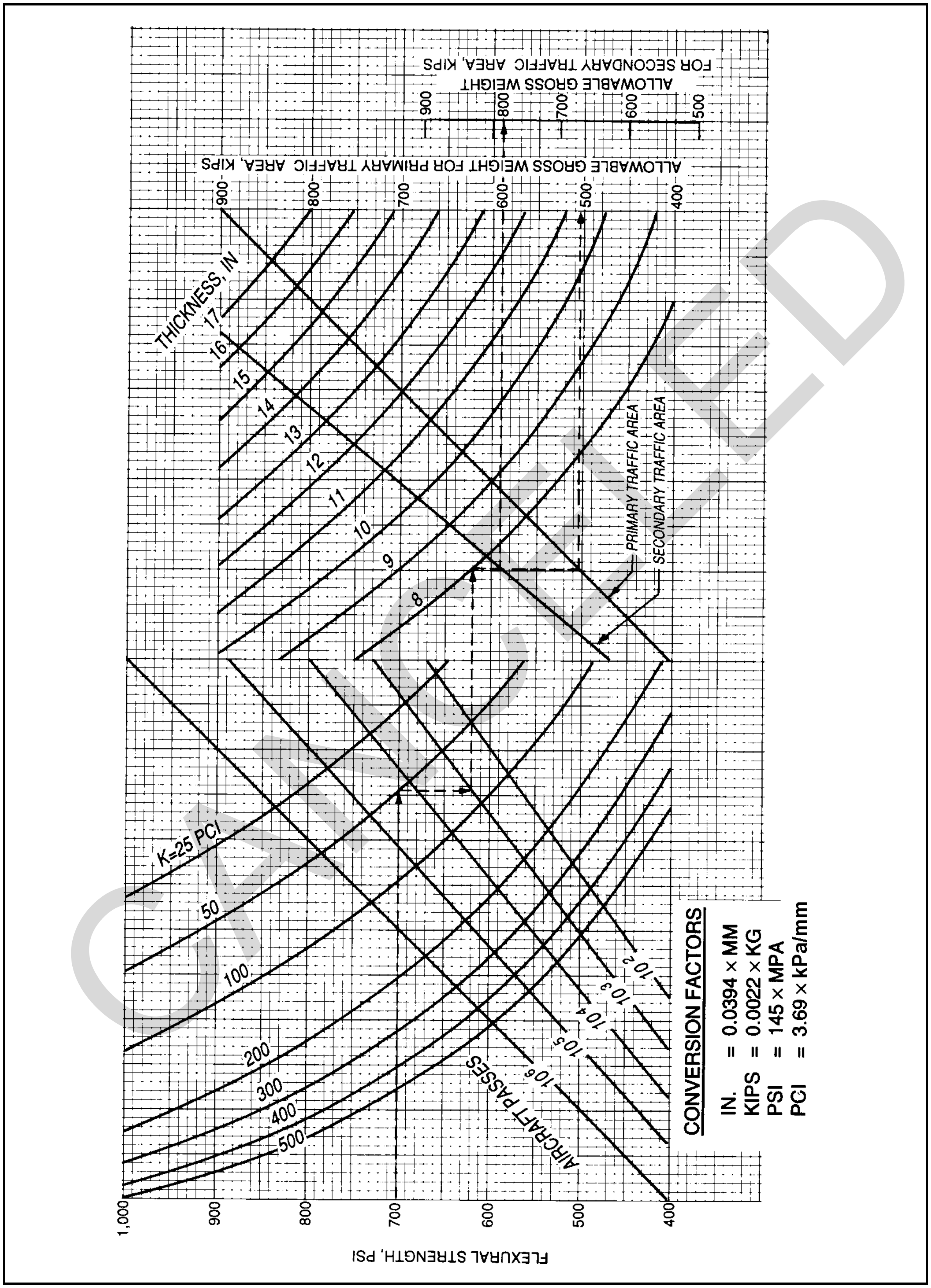


Figure 6-59. Rigid pavement evaluation curves, C-5 (interior load criteria)

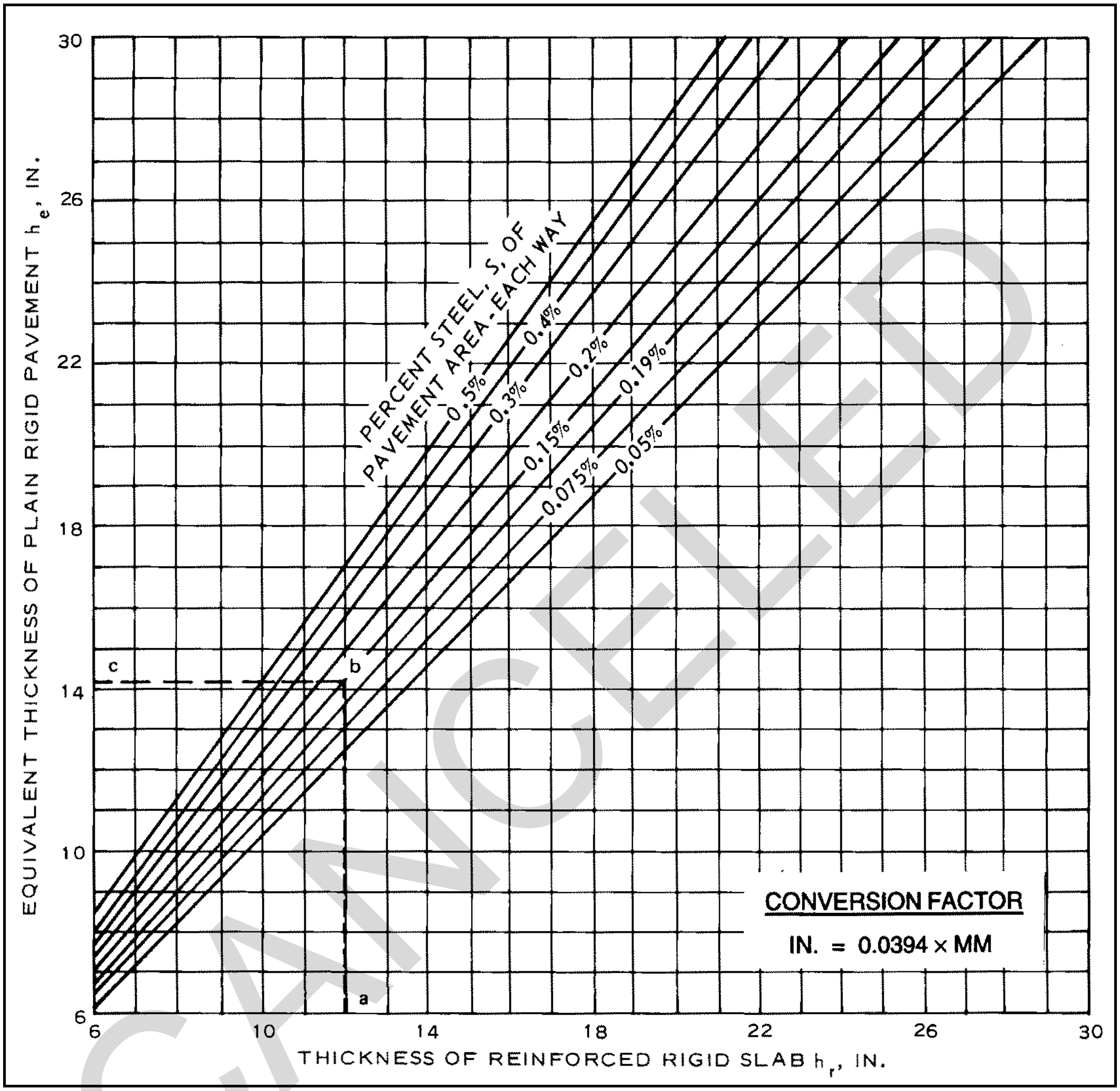


Figure 6-60. Equivalent thickness of reinforced concrete pavement

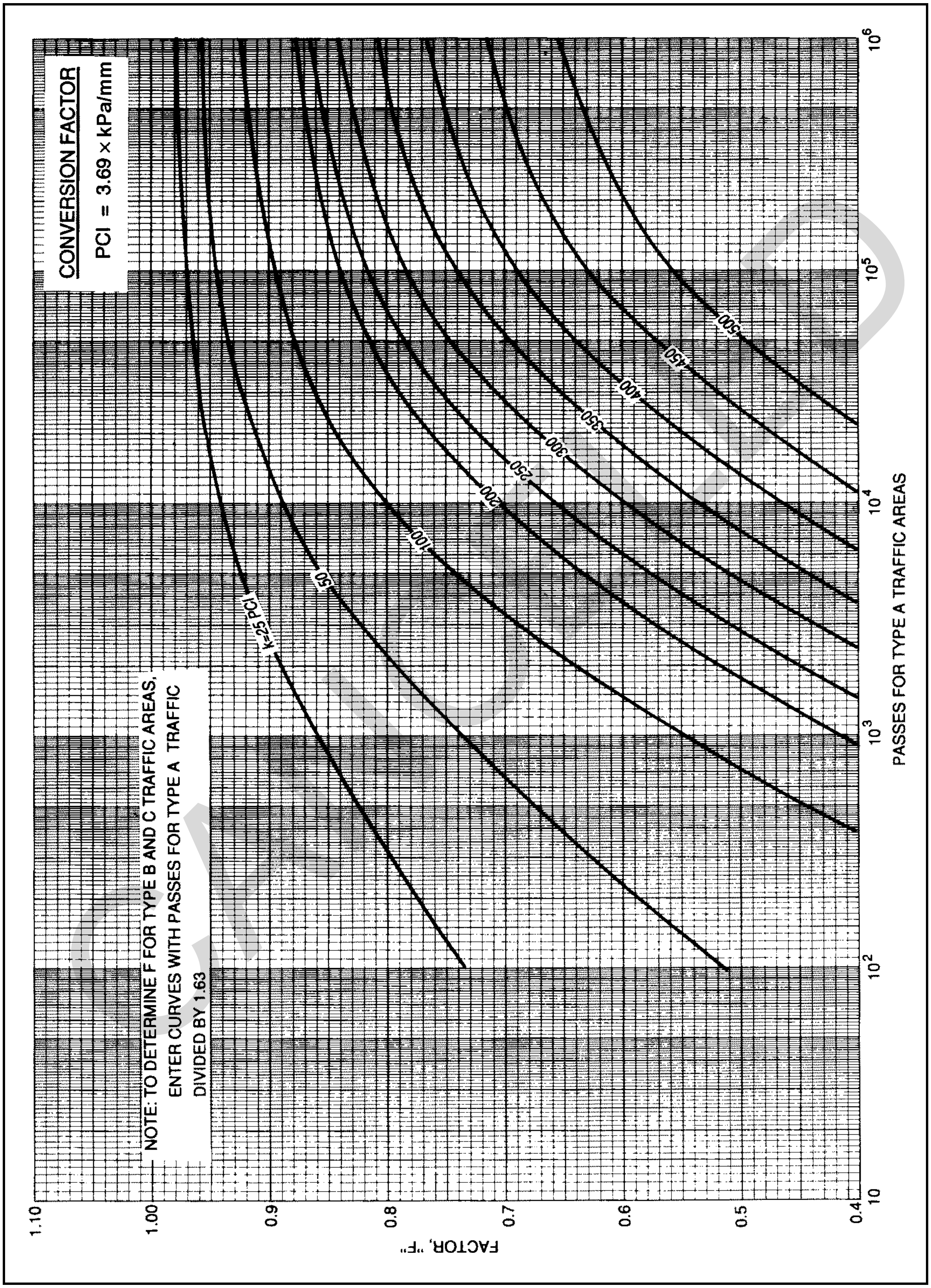


Figure 6-61. Factor for determining equivalent thickness of nonrigid overlay, UH-60

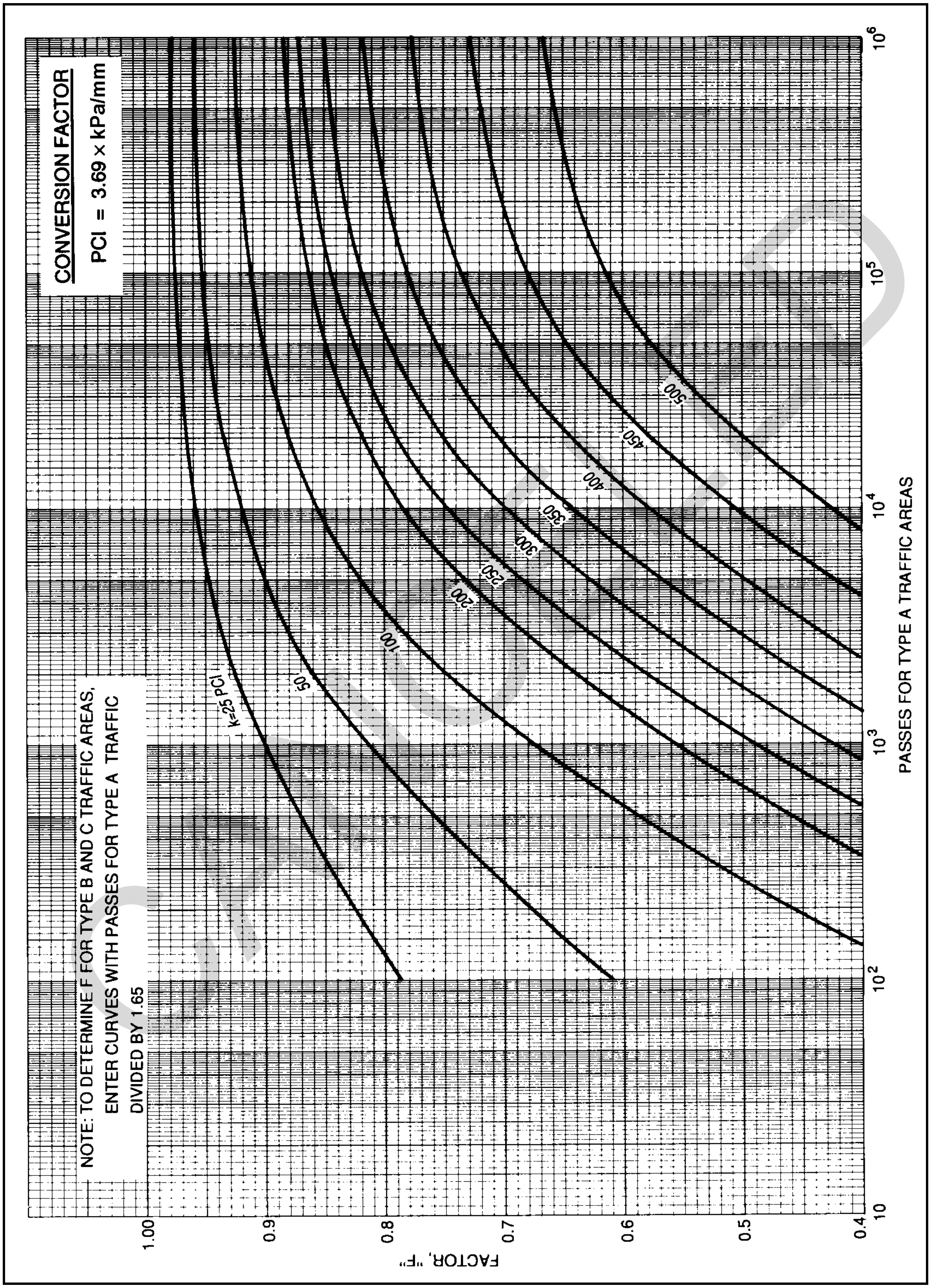


Figure 6-62. Factor for determining equivalent thickness of nonrigid overlay, CH-47

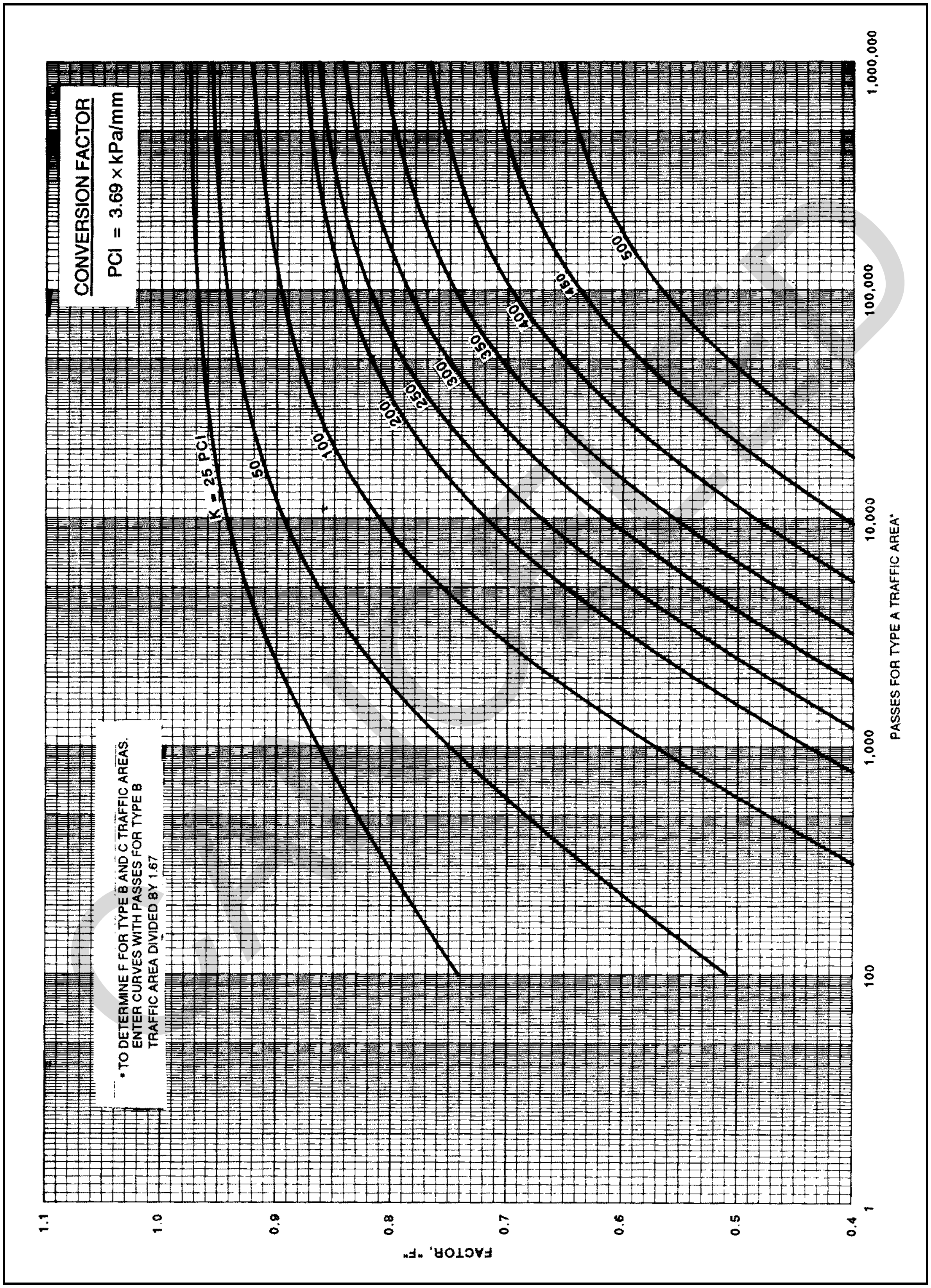


Figure 6-63. Factor for determining equivalent thickness of nonrigid overlay, OV-1

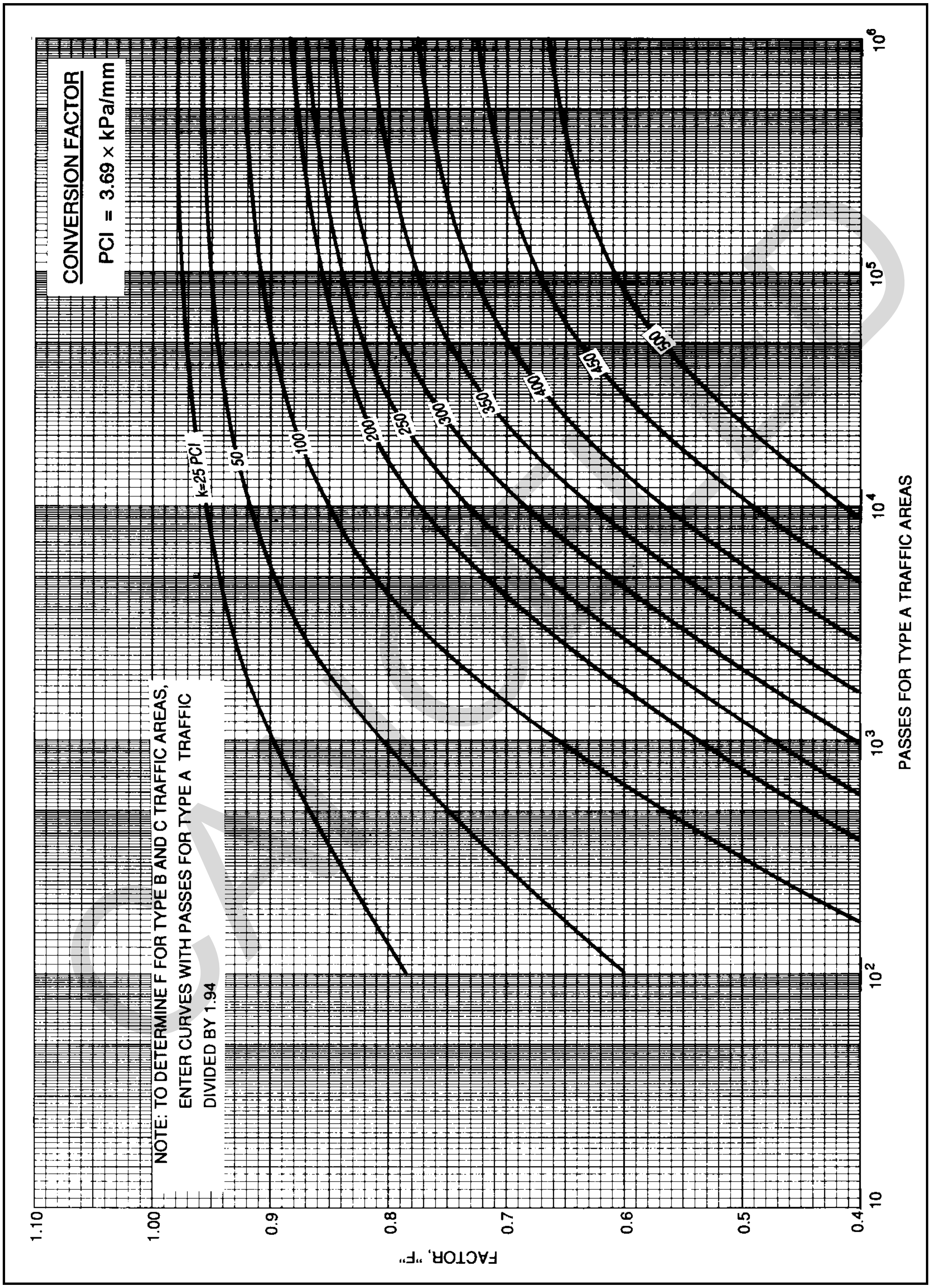


Figure 6-64. Factor for determining equivalent thickness of nonrigid overlay, C-12

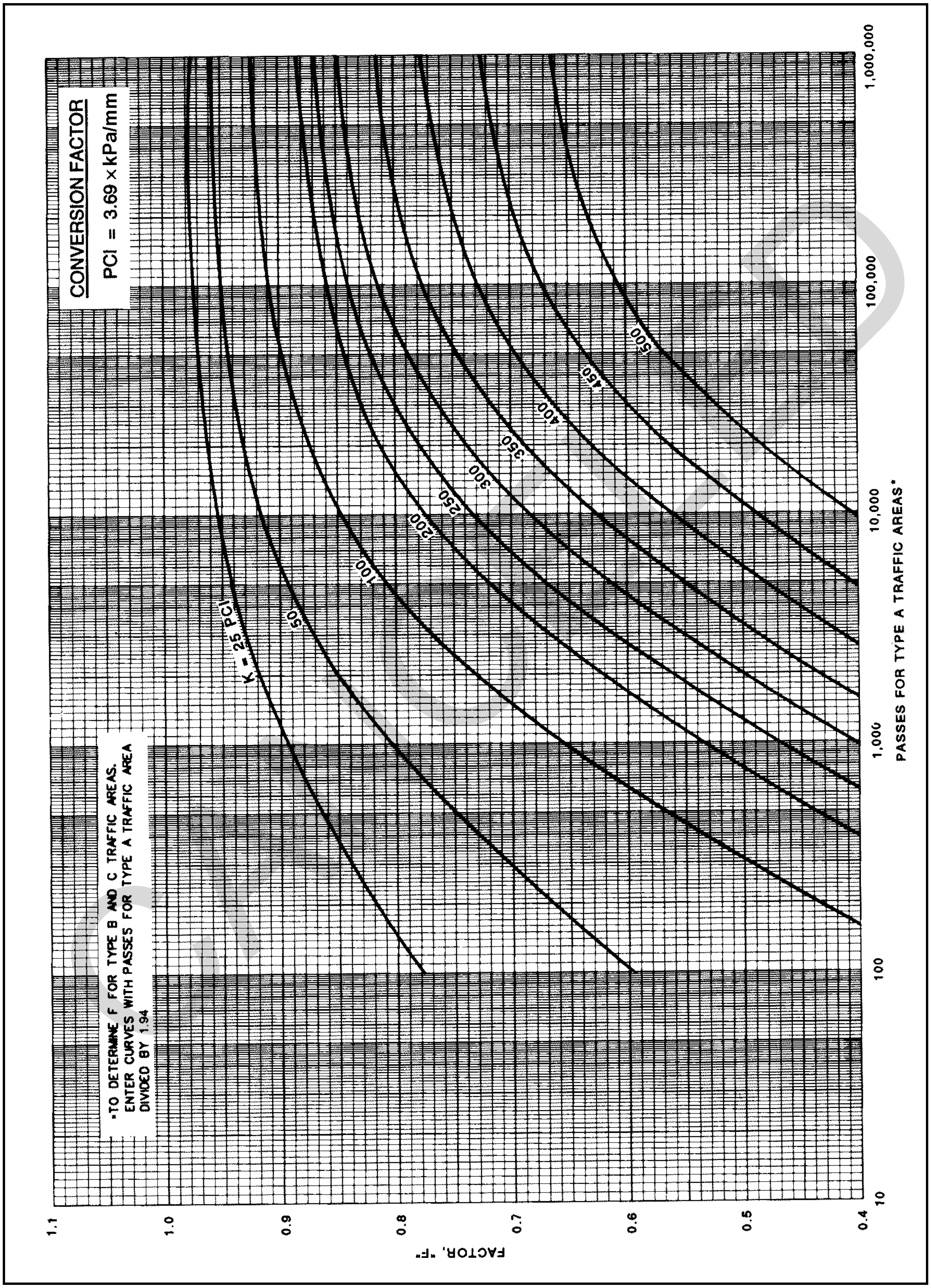


Figure 6-65. Factor for determining equivalent thickness of nonrigid overlay, C-23 (Air Force Group Index 1)

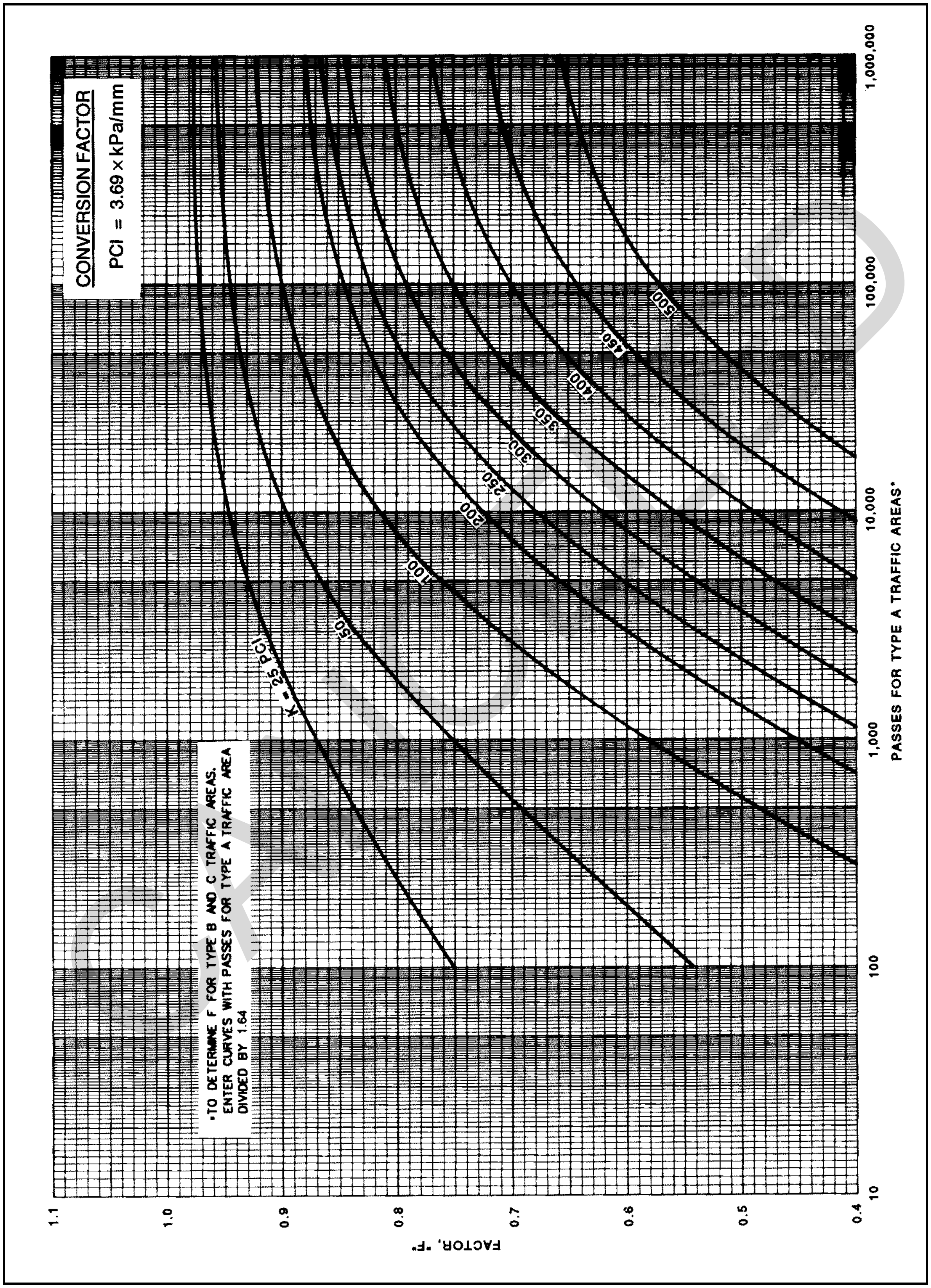


Figure 6-66. Factor for determining equivalent thickness of nonrigid overlay, F-4 (Air Force Group Index 2)

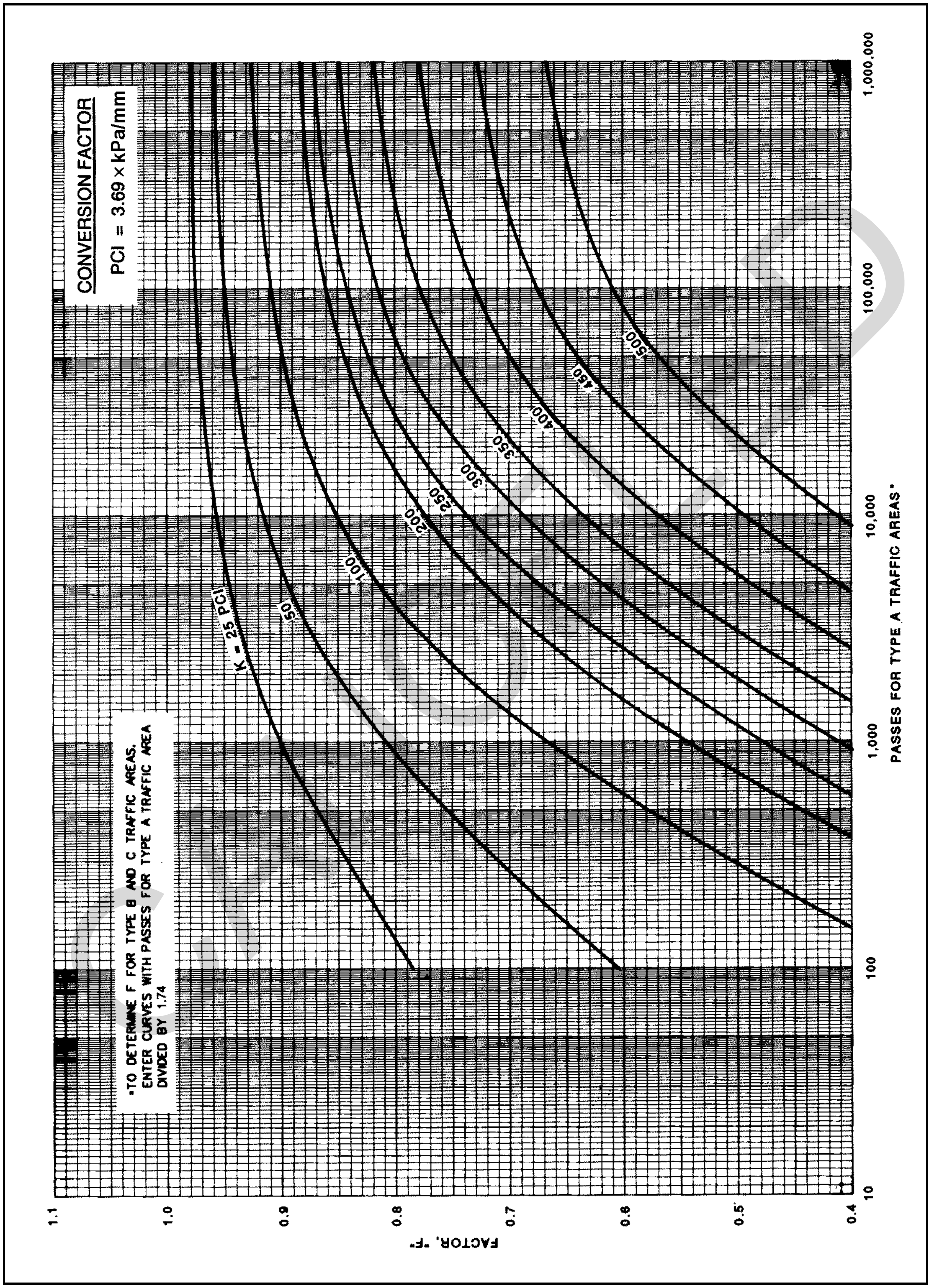


Figure 6-67. Factor for determining equivalent thickness of nonrigid overlay, F-111 (Air Force Group Index 3)

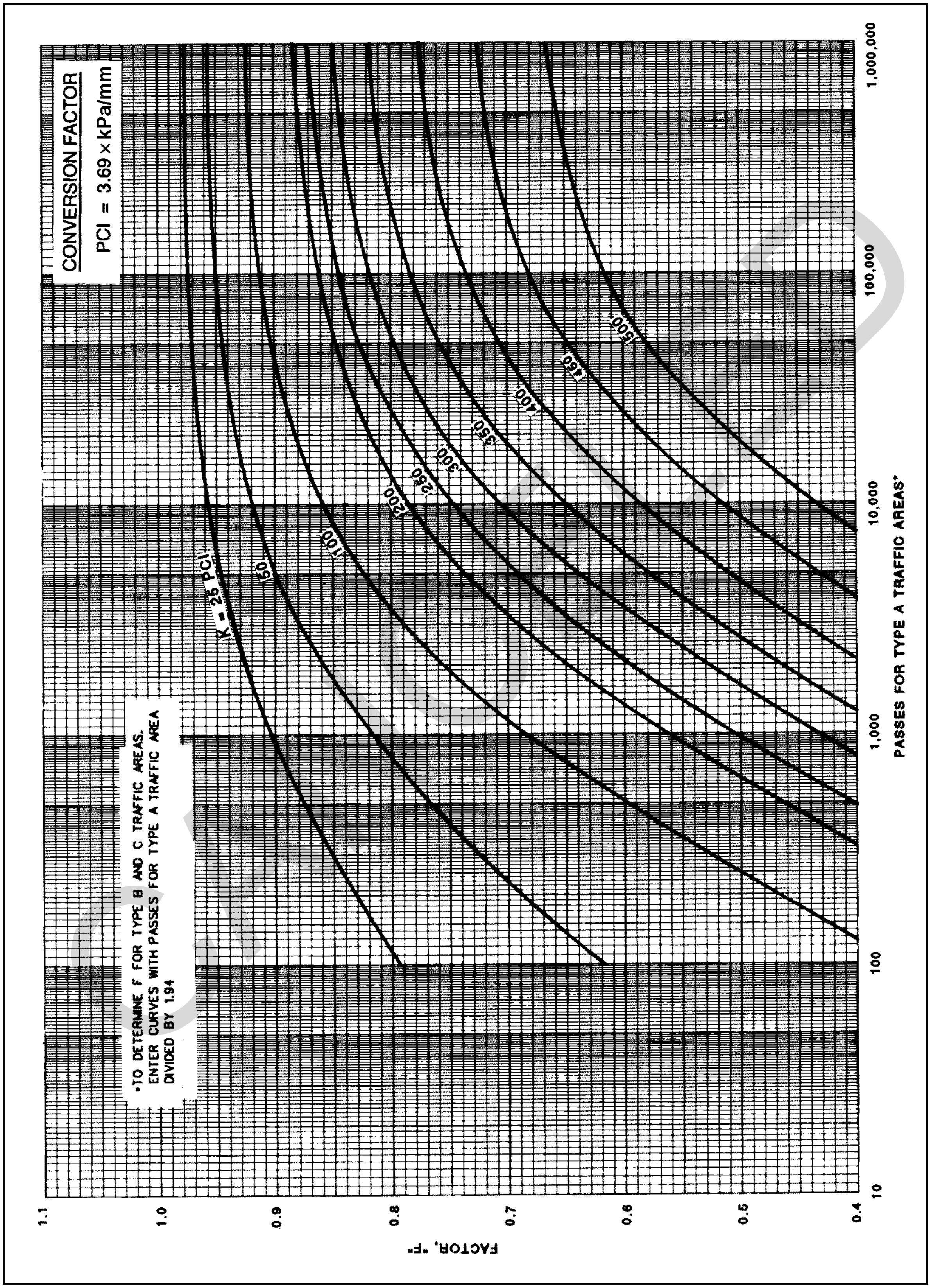


Figure 6-68. Factor for determining equivalent thickness of nonrigid overlay, C-130 (Air Force Group Index 4)

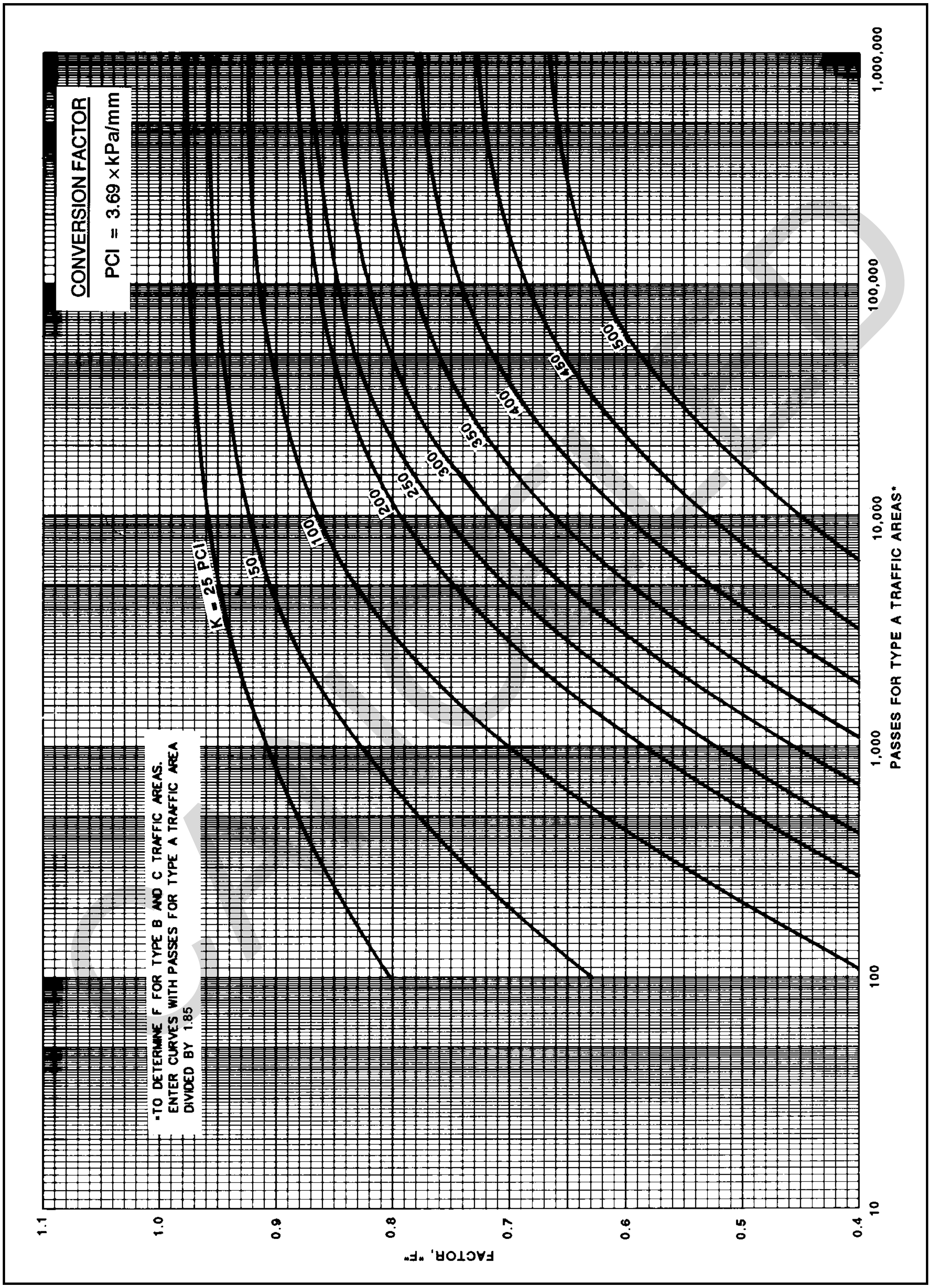


Figure 6-69. Factor for determining equivalent thickness of nonrigid overlay, C-9 (Air Force Group Index 5)

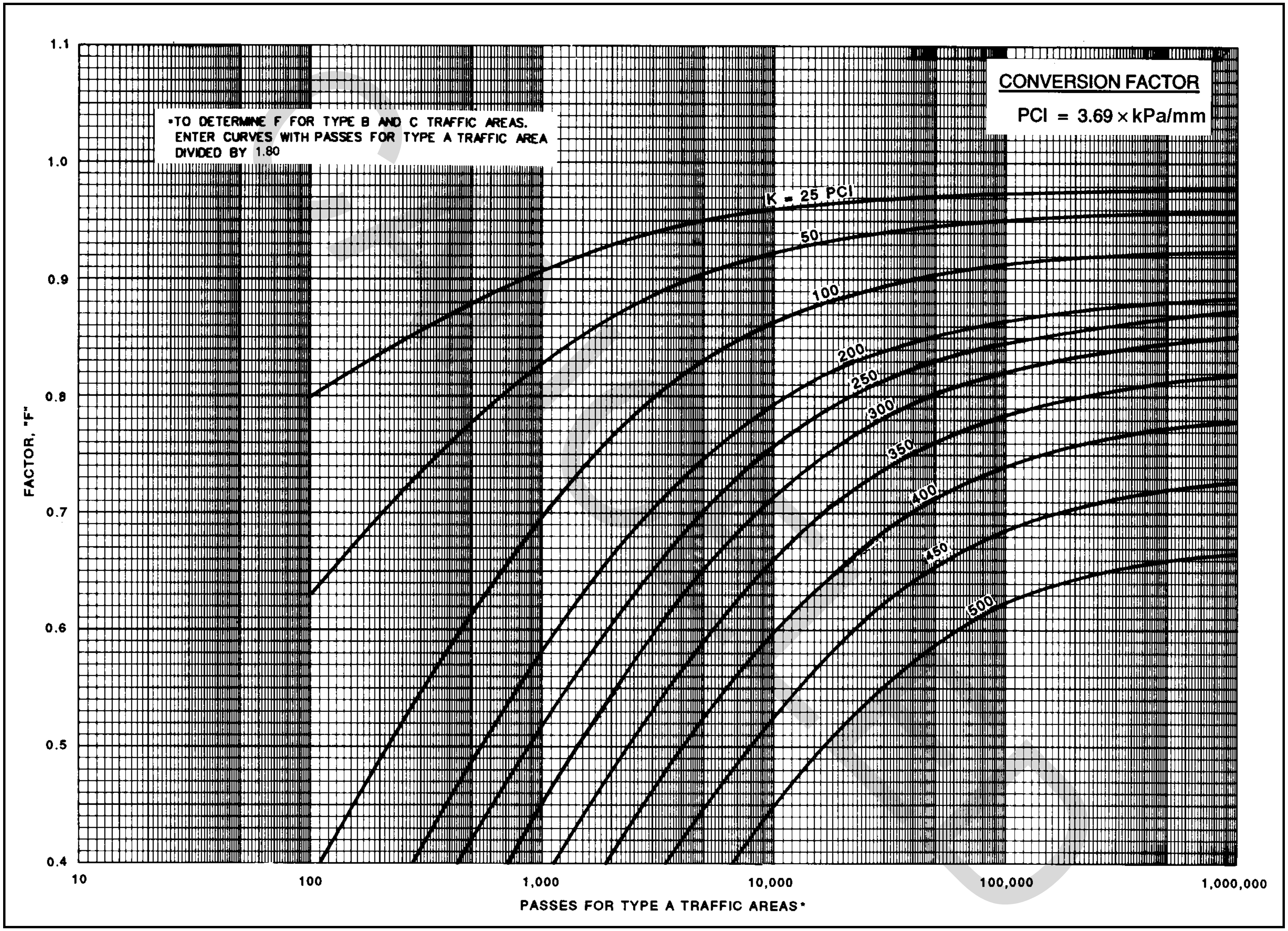


Figure 6-70. Factor for determining equivalent thickness of nonrigid overlay, T-43 (Air Force Group Index 6)

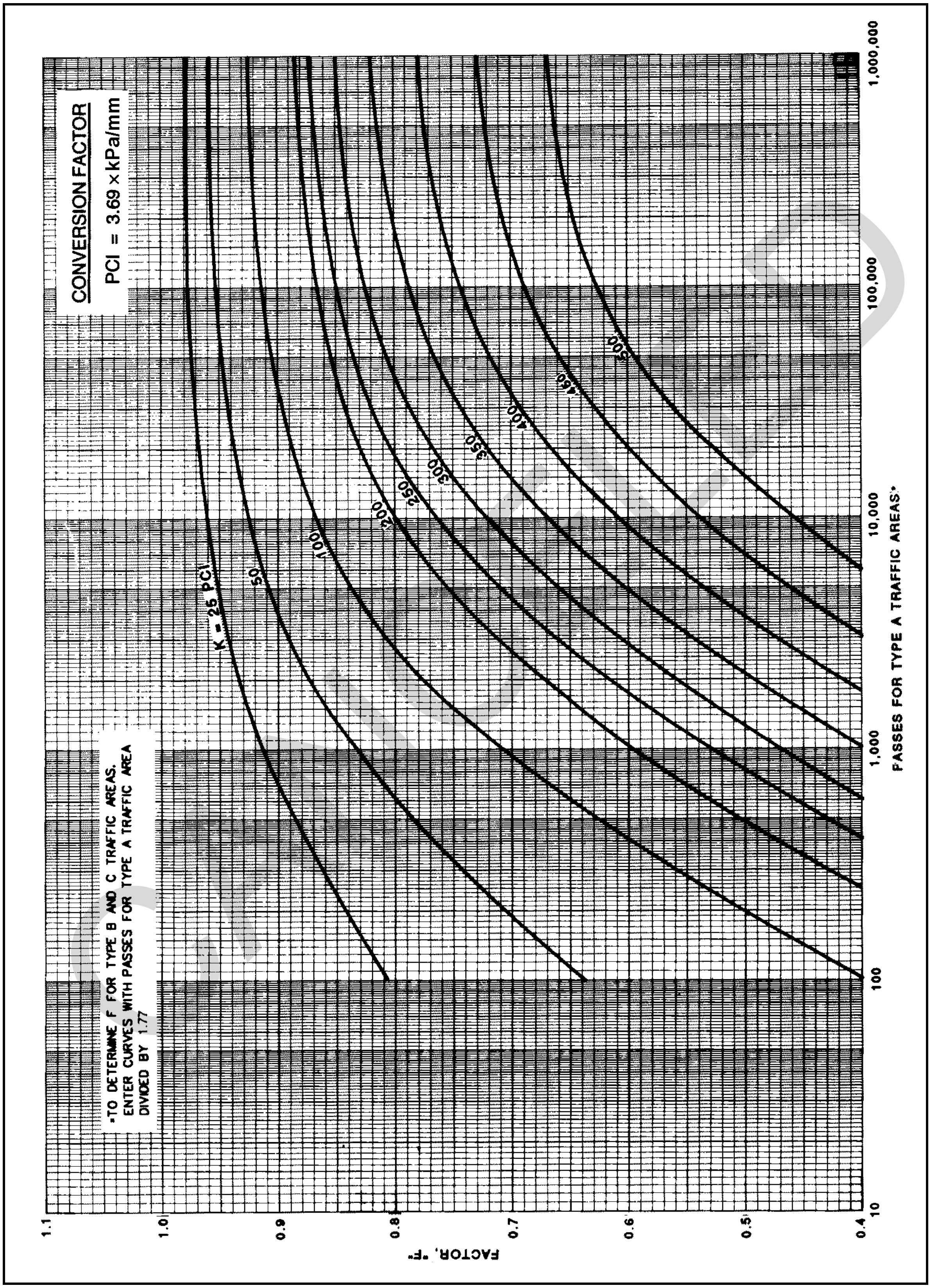


Figure 6-71. Factor for determining equivalent thickness of nonrigid overlay, B-727 (Air Force Group Index 7)

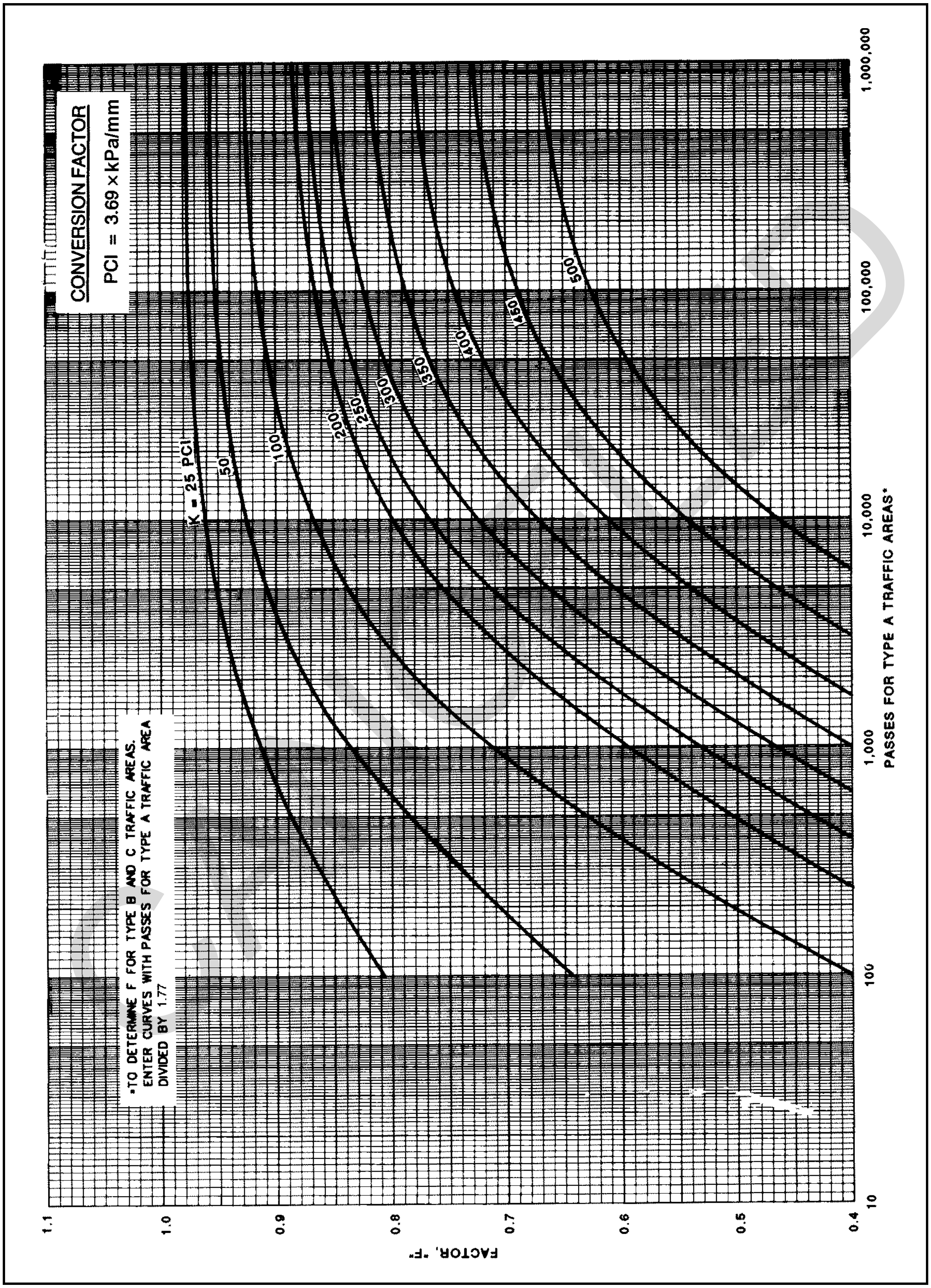


Figure 6-72. Factor for determining equivalent thickness of nonrigid overlay, E-3 (Air Force Group Index 8)

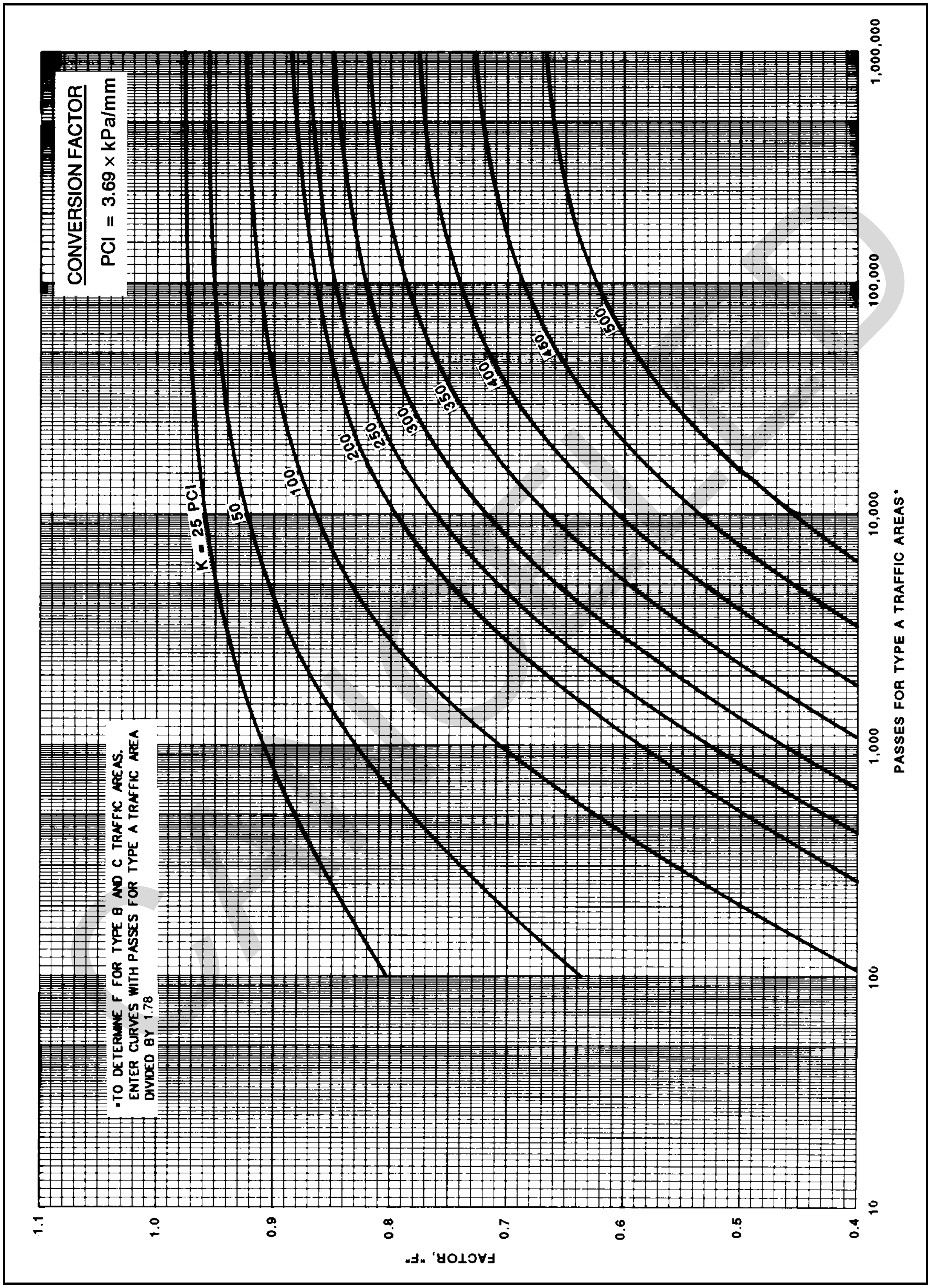


Figure 6-73. Factor for determining equivalent thickness of nonrigid overlay, C-141 (Air Force Group Index 9)

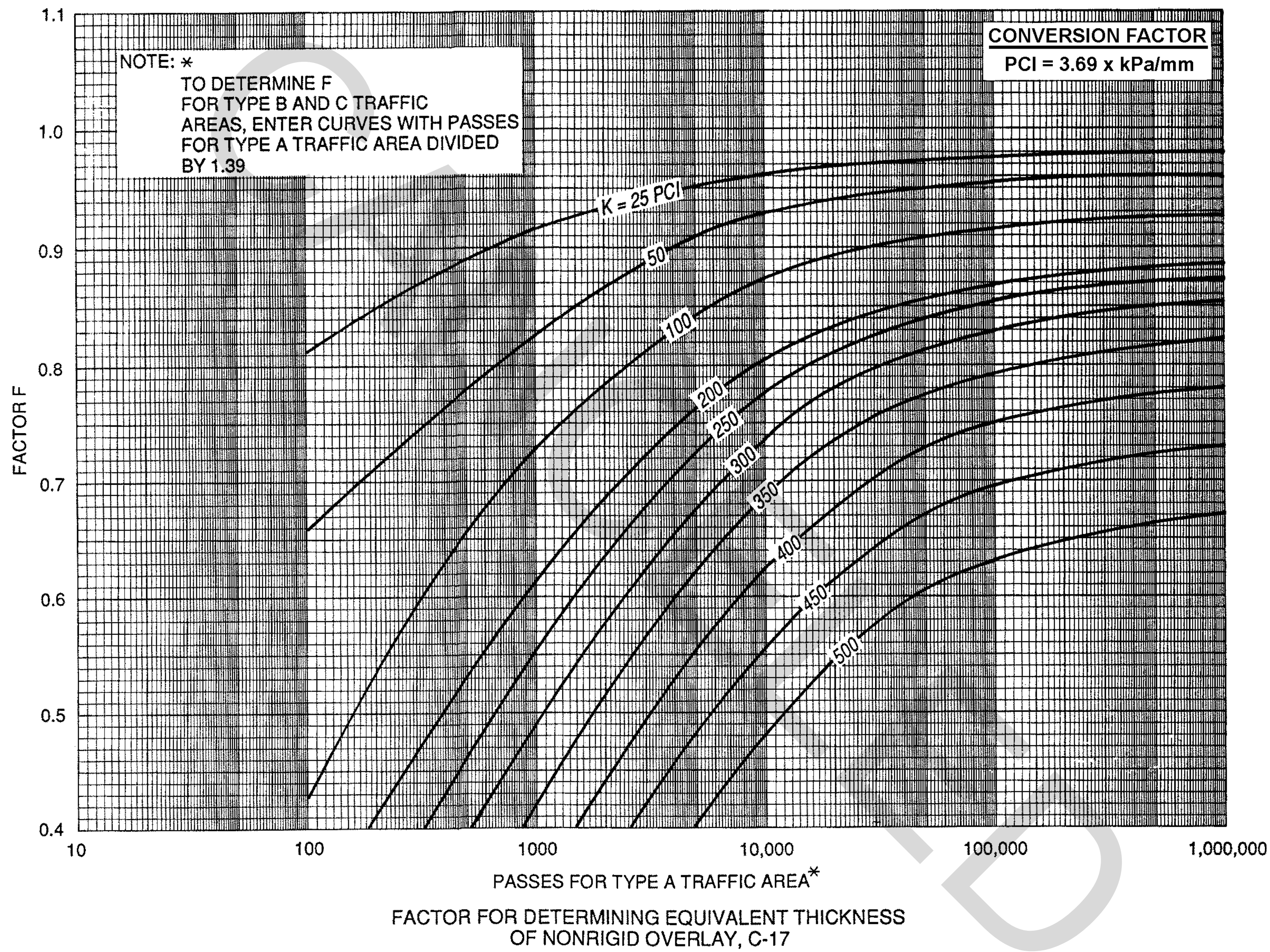


Figure 6-74. Factor for determining equivalent thickness of nonrigid overlay, C-17 (Air Force Group Index 10)

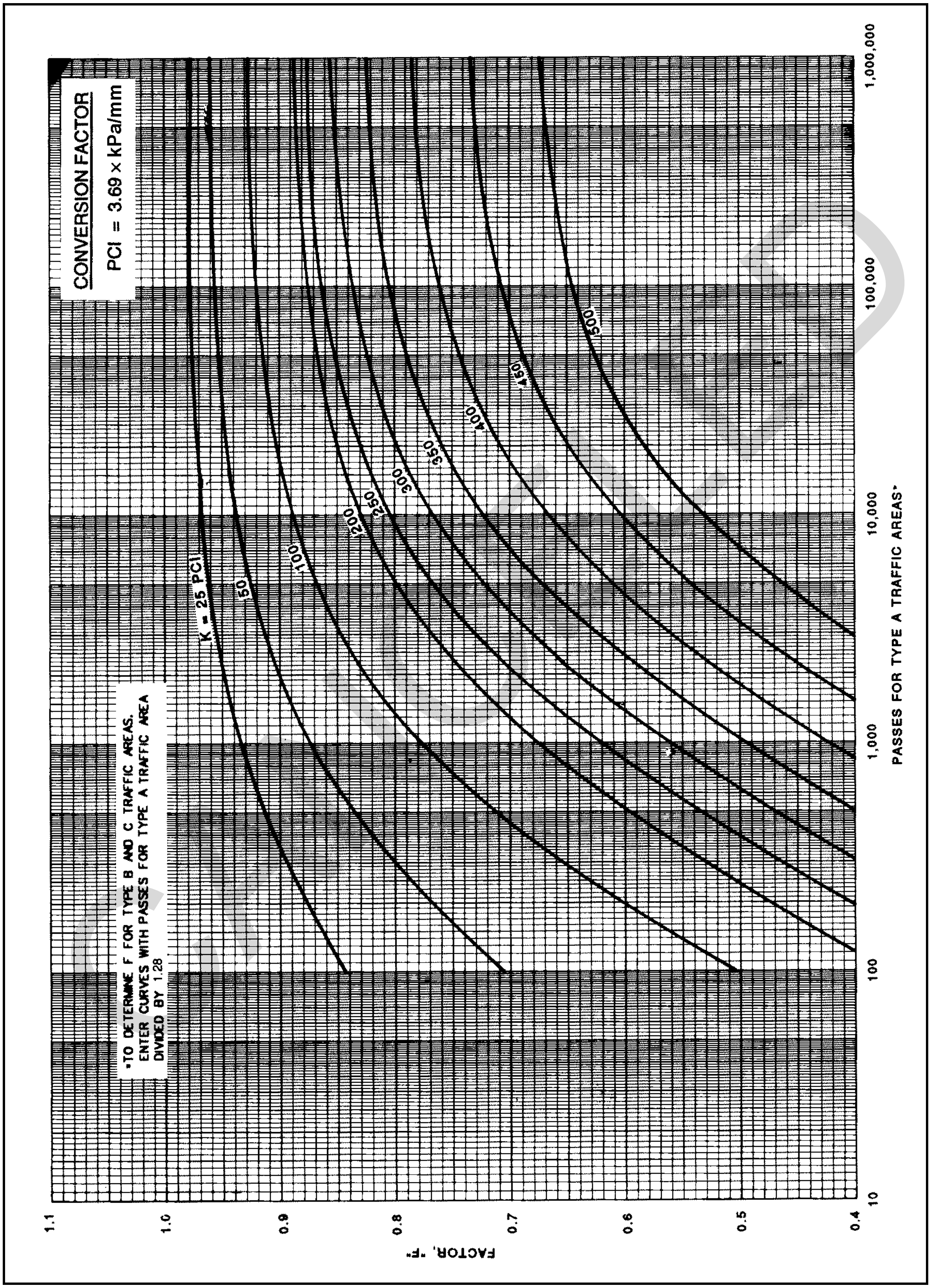


Figure 6-75. Factor for determining equivalent thickness of nonrigid overlay, C-5 (Air Force Group Index 11)

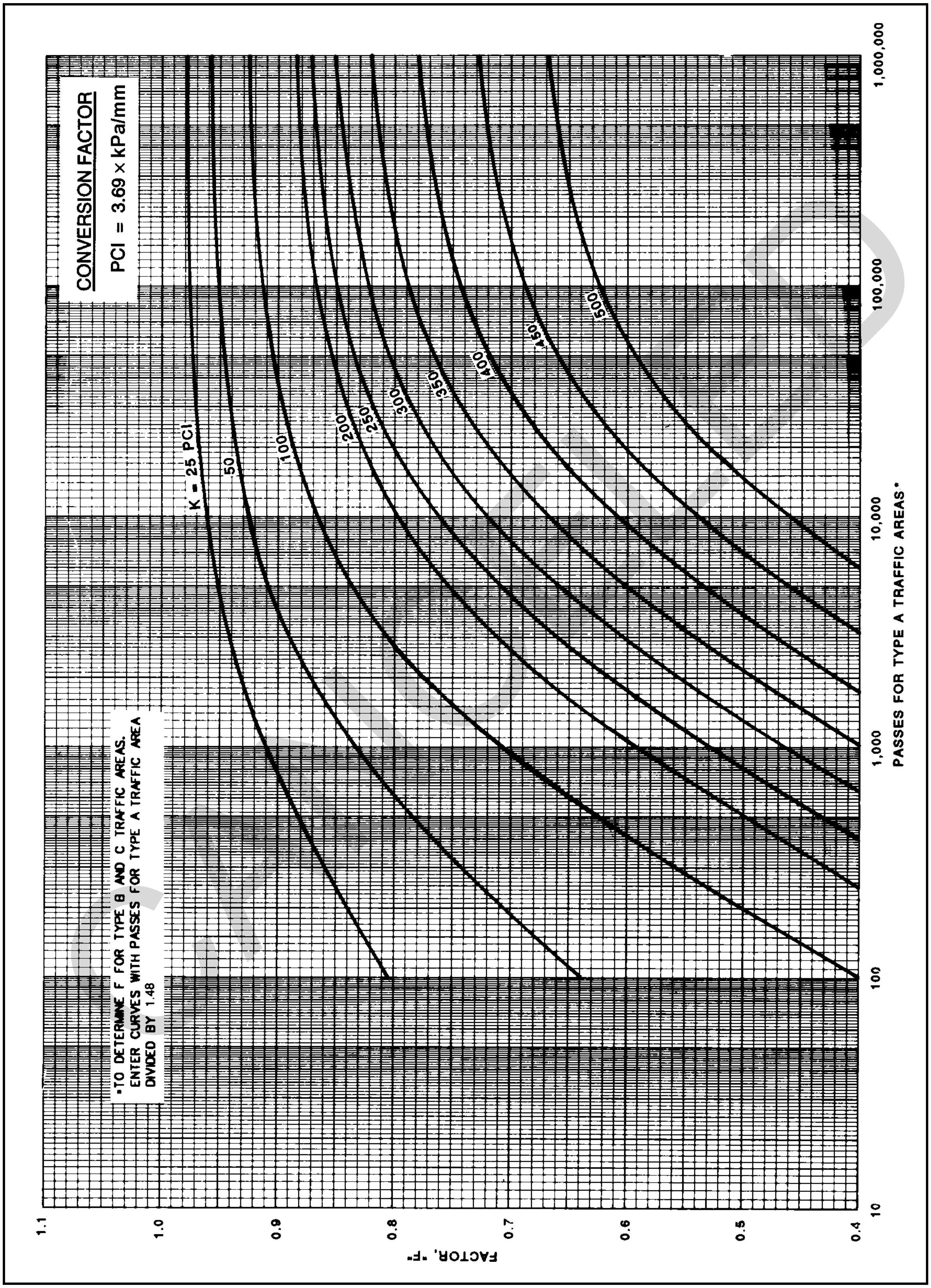


Figure 6-76. Factor for determining equivalent thickness of nonrigid overlay, KC-10 (Air Force Group Index 12)

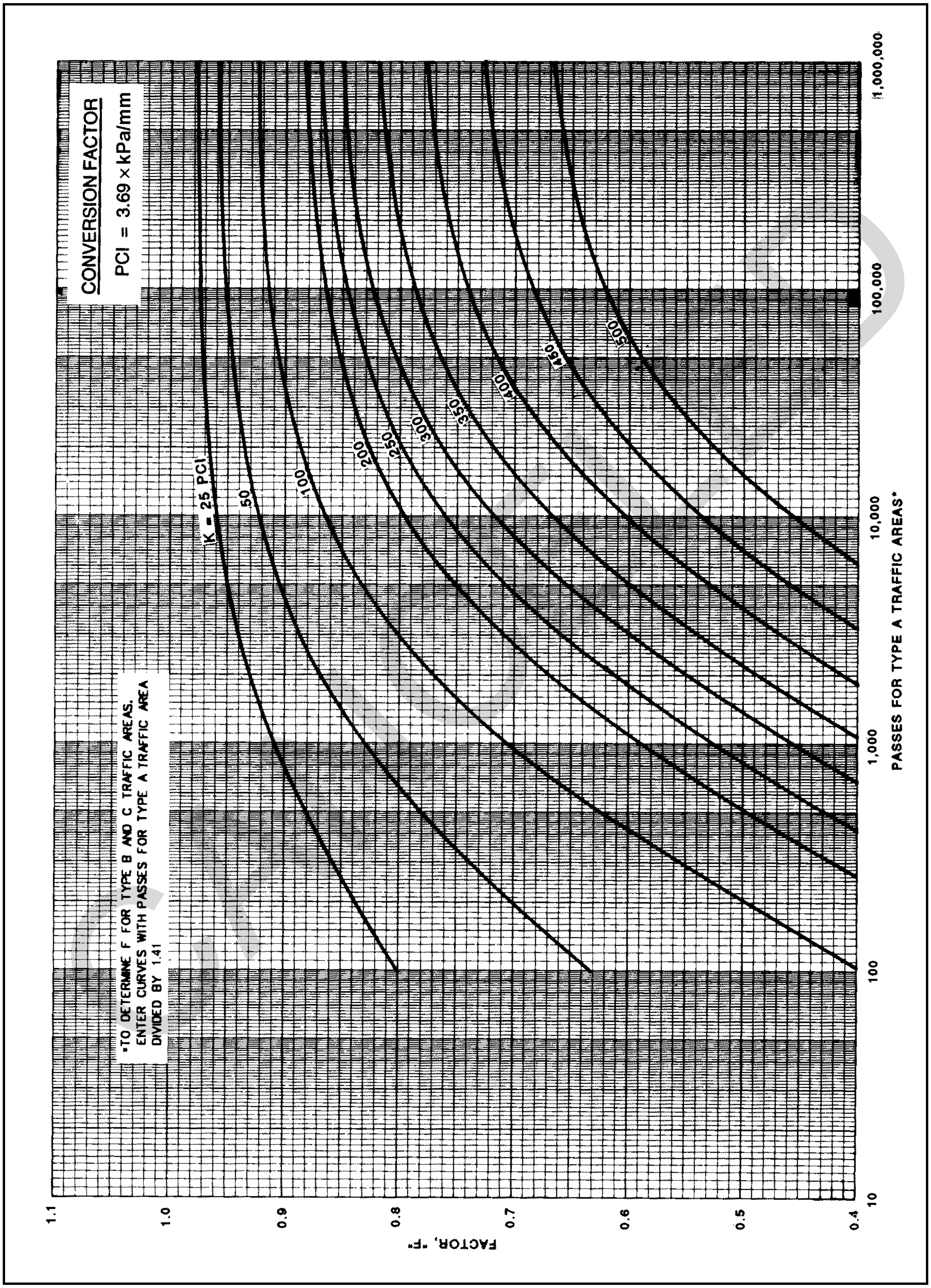


Figure 6-77. Factor for determining equivalent thickness of nonrigid overlay, B-707 (Air Force Group Index 13)

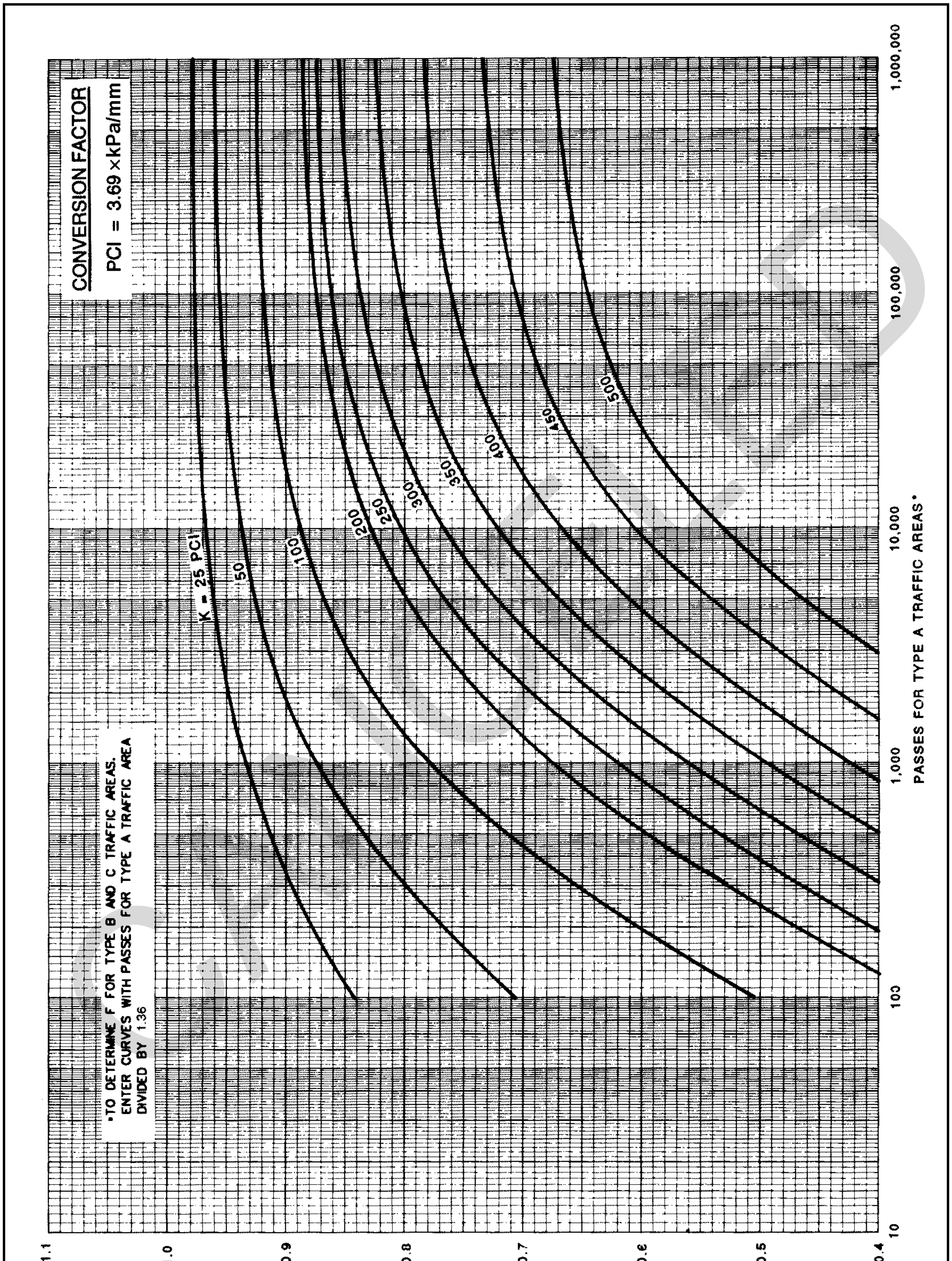


Figure 6-78. Factor for determining equivalent thickness of nonrigid overlay, B-52 (Air Force Group Index 14)

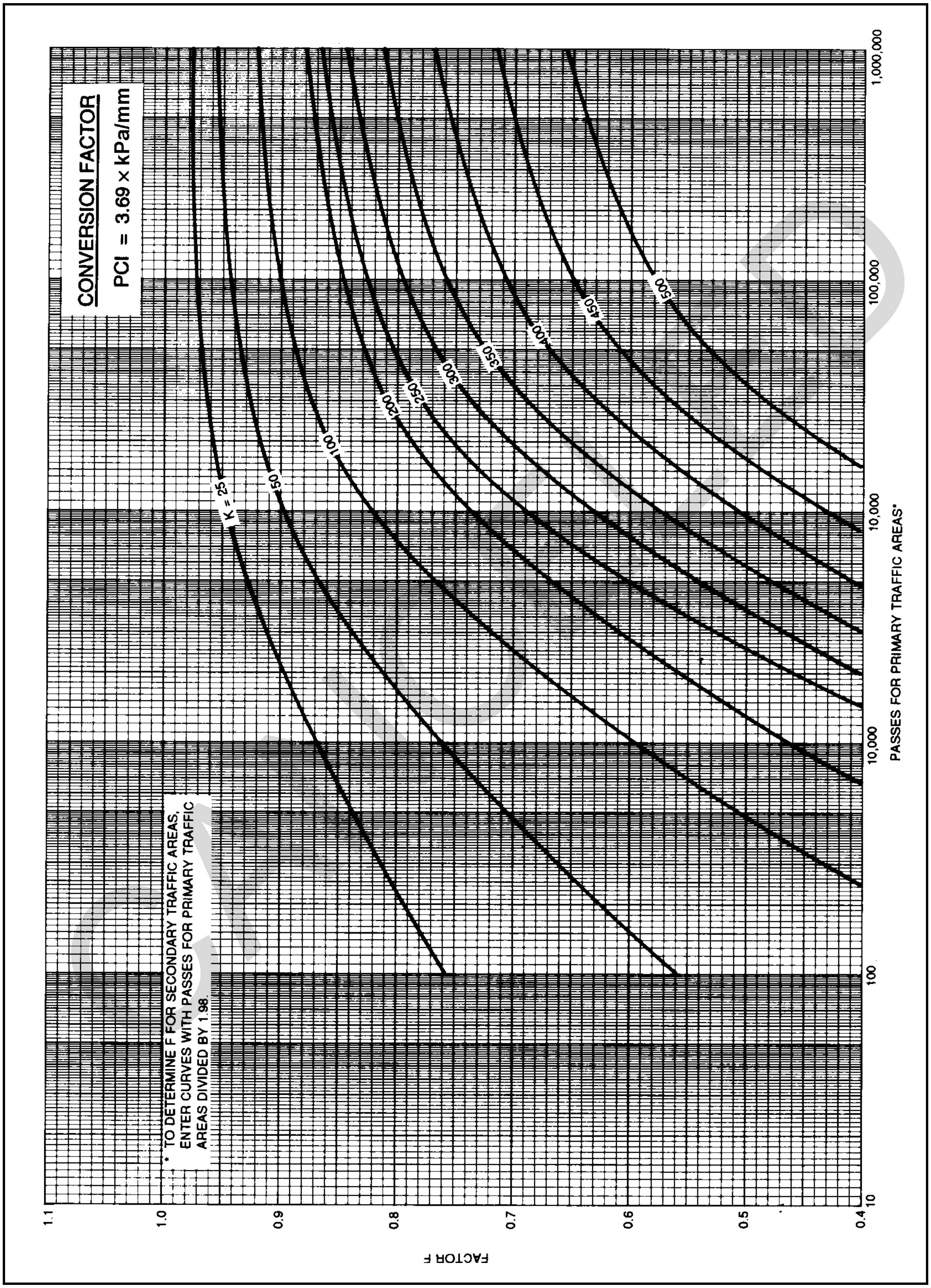


Figure 6-79. Factor for determining equivalent thickness of nonrigid overlay, F-14

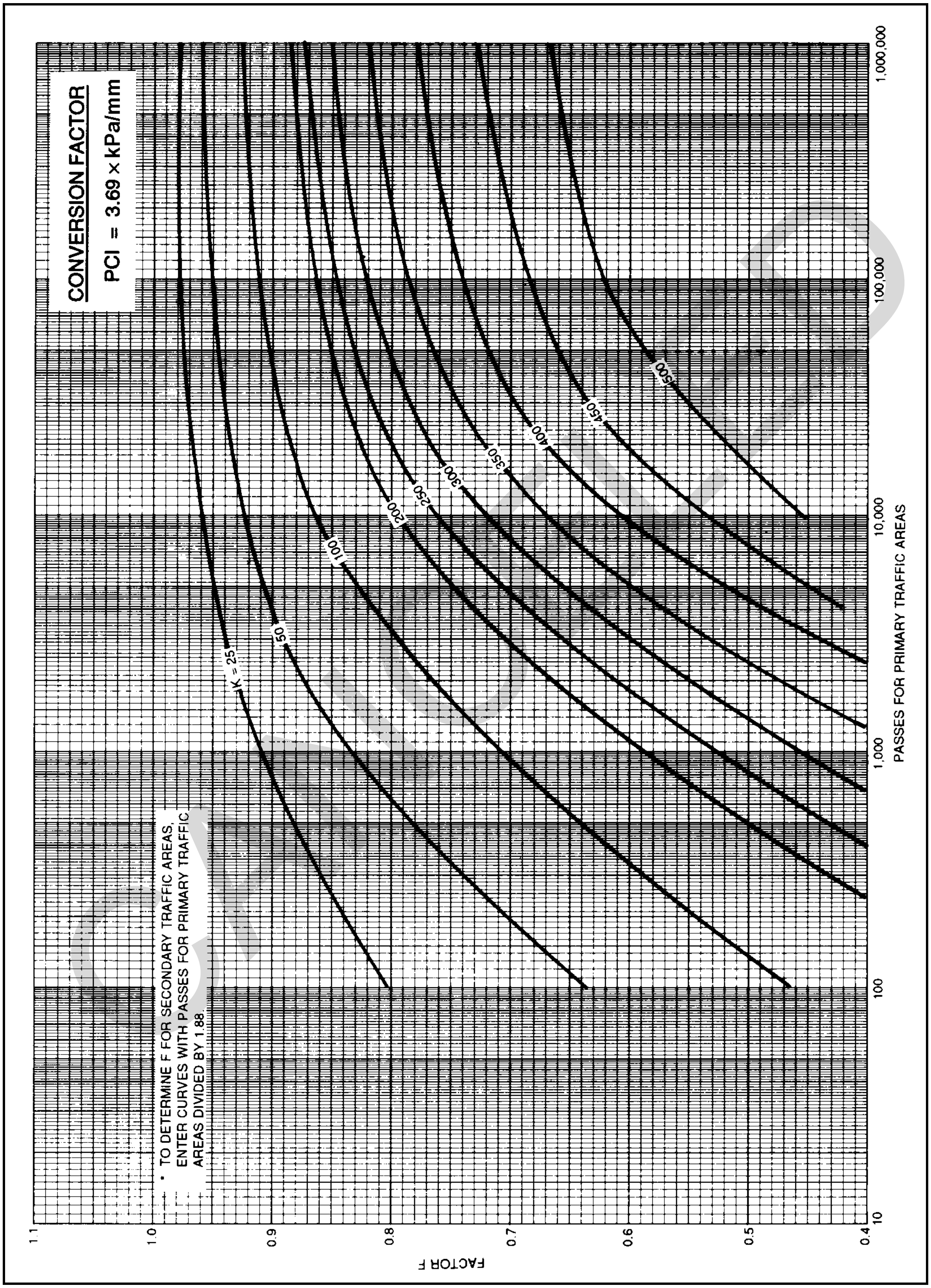


Figure 6-80. Factor for determining equivalent thickness of nonrigid overlay, P-3

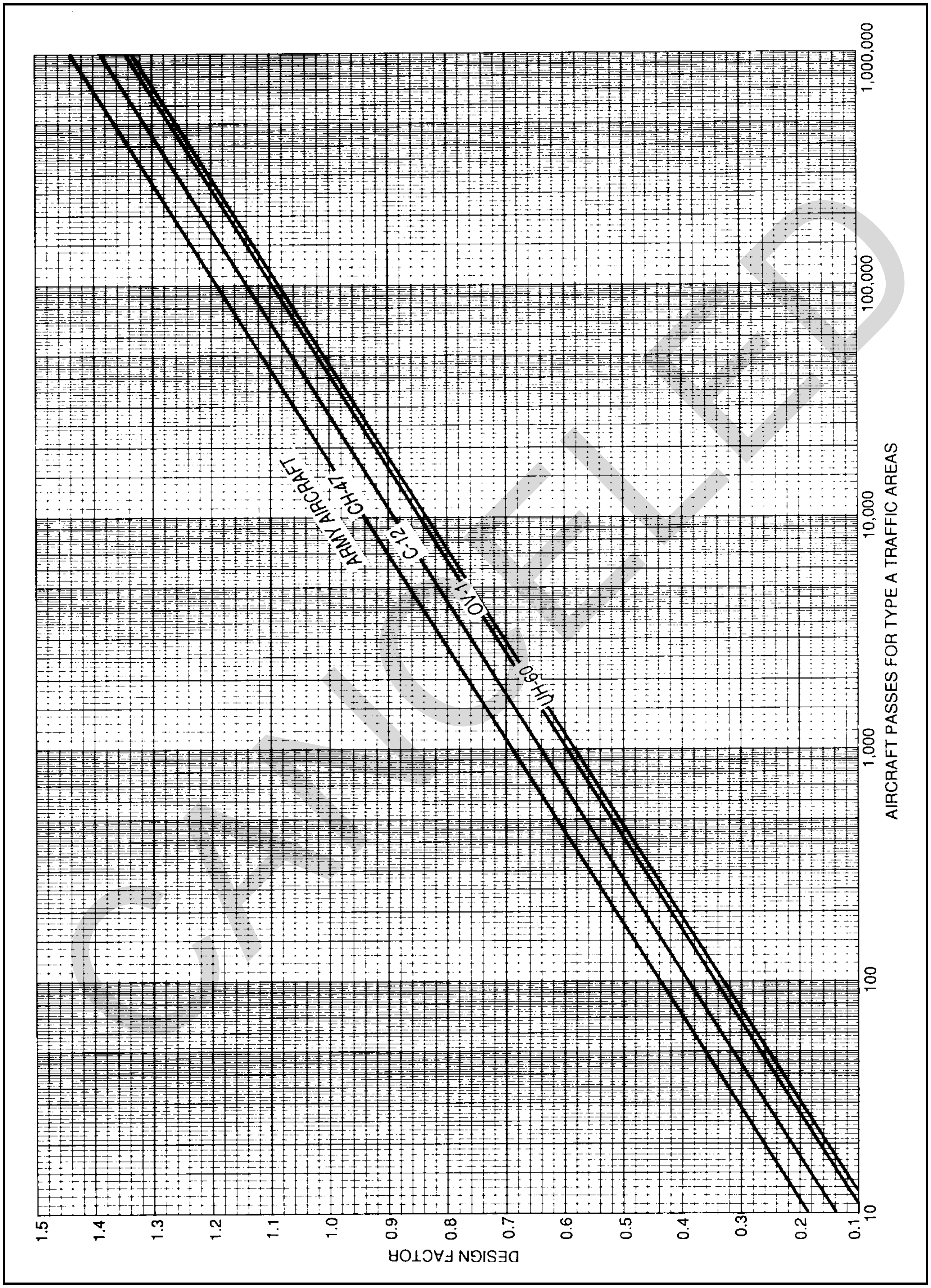


Figure 6-81. Fibrous concrete design factors for UH-60, CH-47, OV-1, and C-12, Type A traffic areas

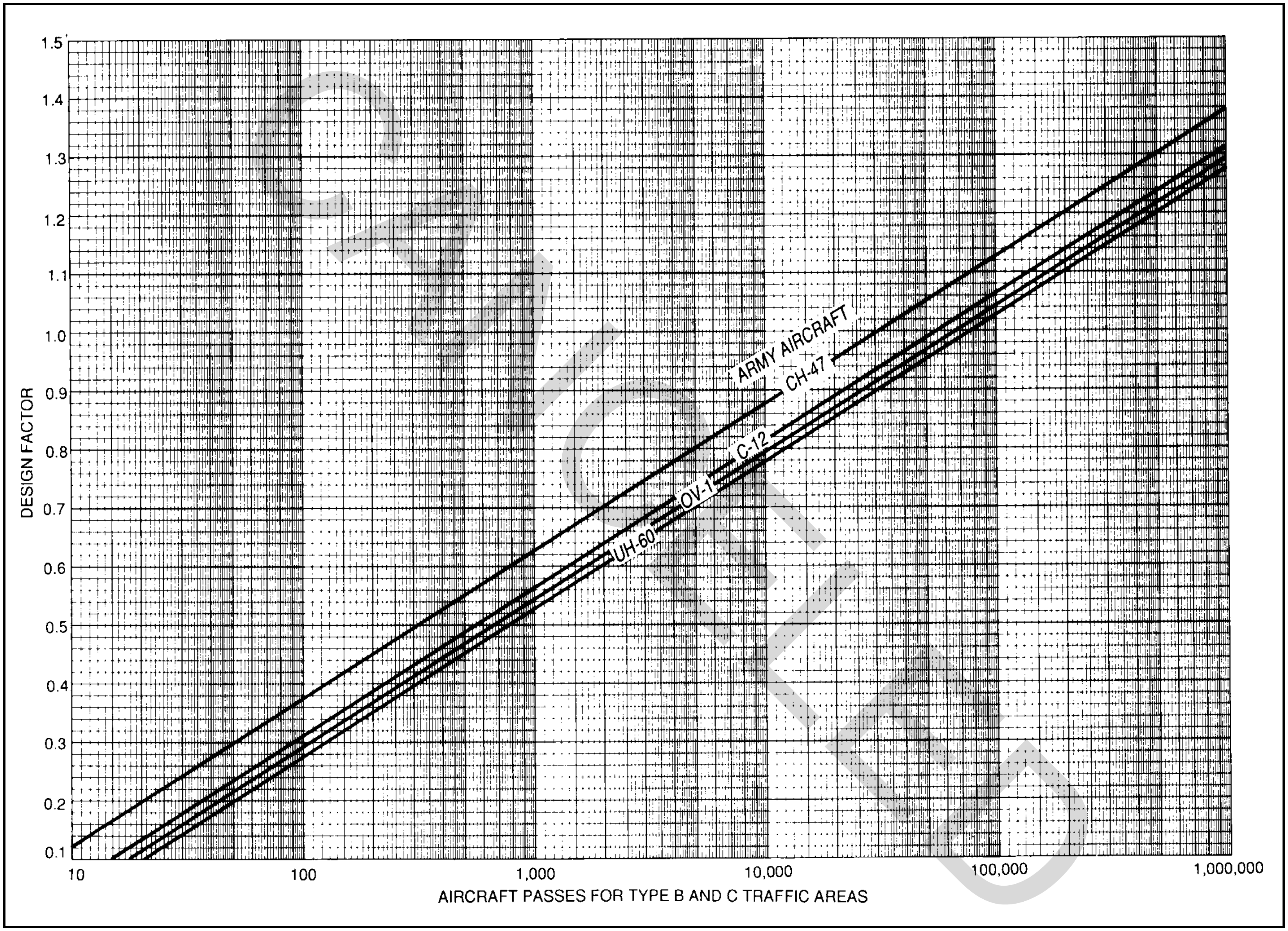


Figure 6-82. Fibrous concrete design factors for UH-60, CH-47, OV-1, and C-12, Types B and C traffic areas

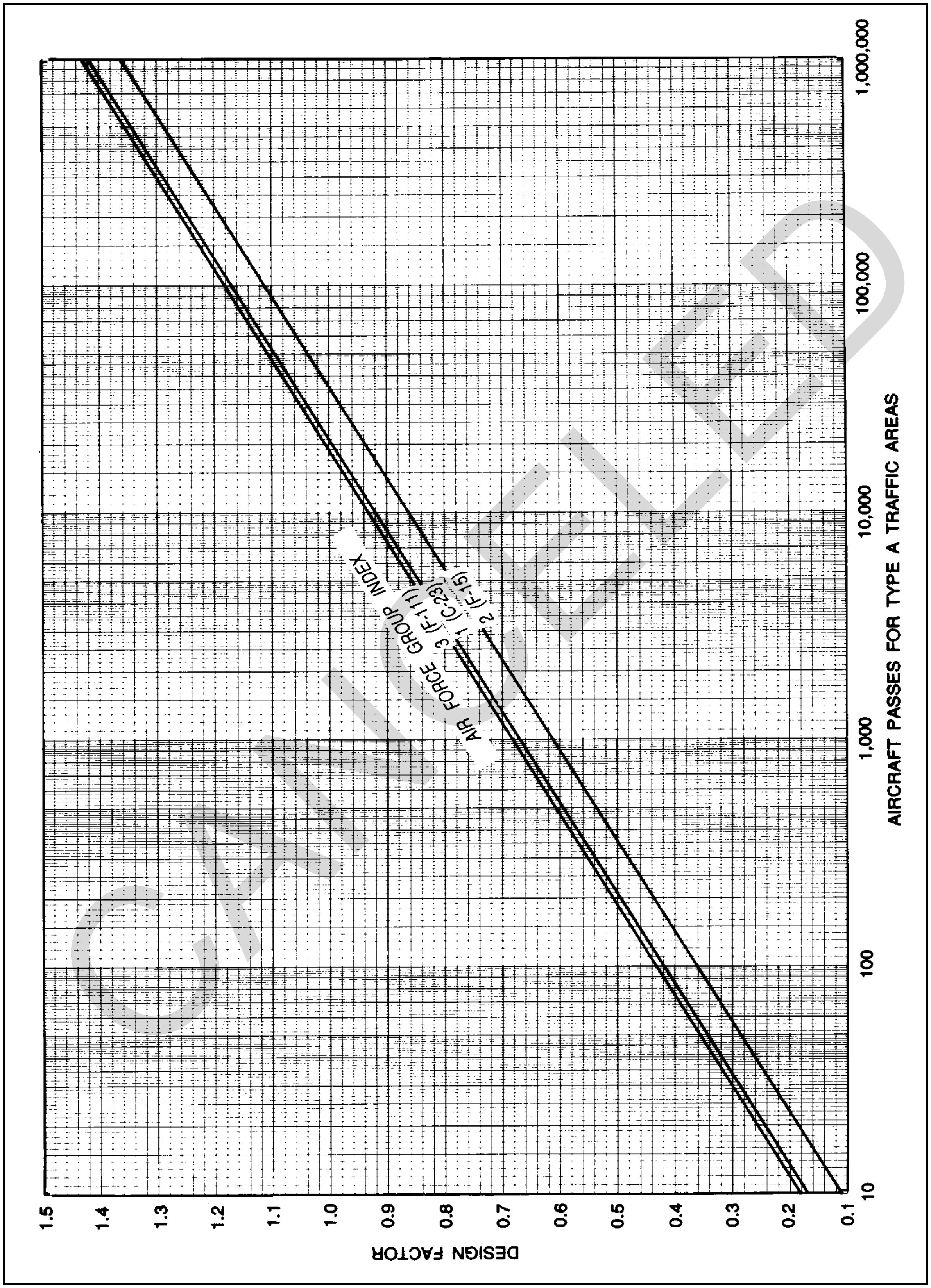


Figure 6-83. Fibrous concrete design factors for Air Force Group Indices 1, 2, and 3, Type A traffic areas

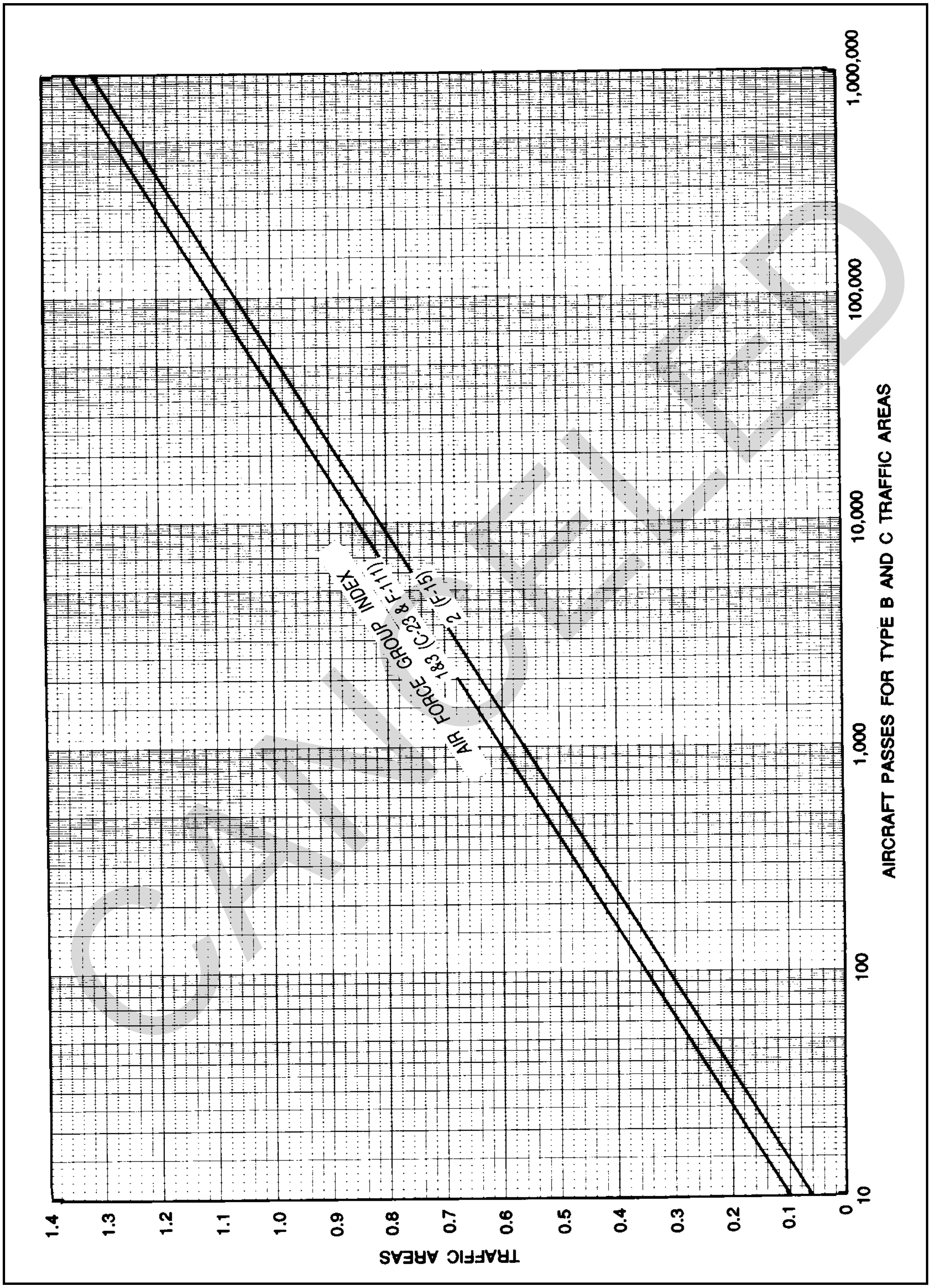


Figure 6-84. Fibrous concrete design factors for Air Force Group Indices 1, 2, and 3, Types B and C traffic areas

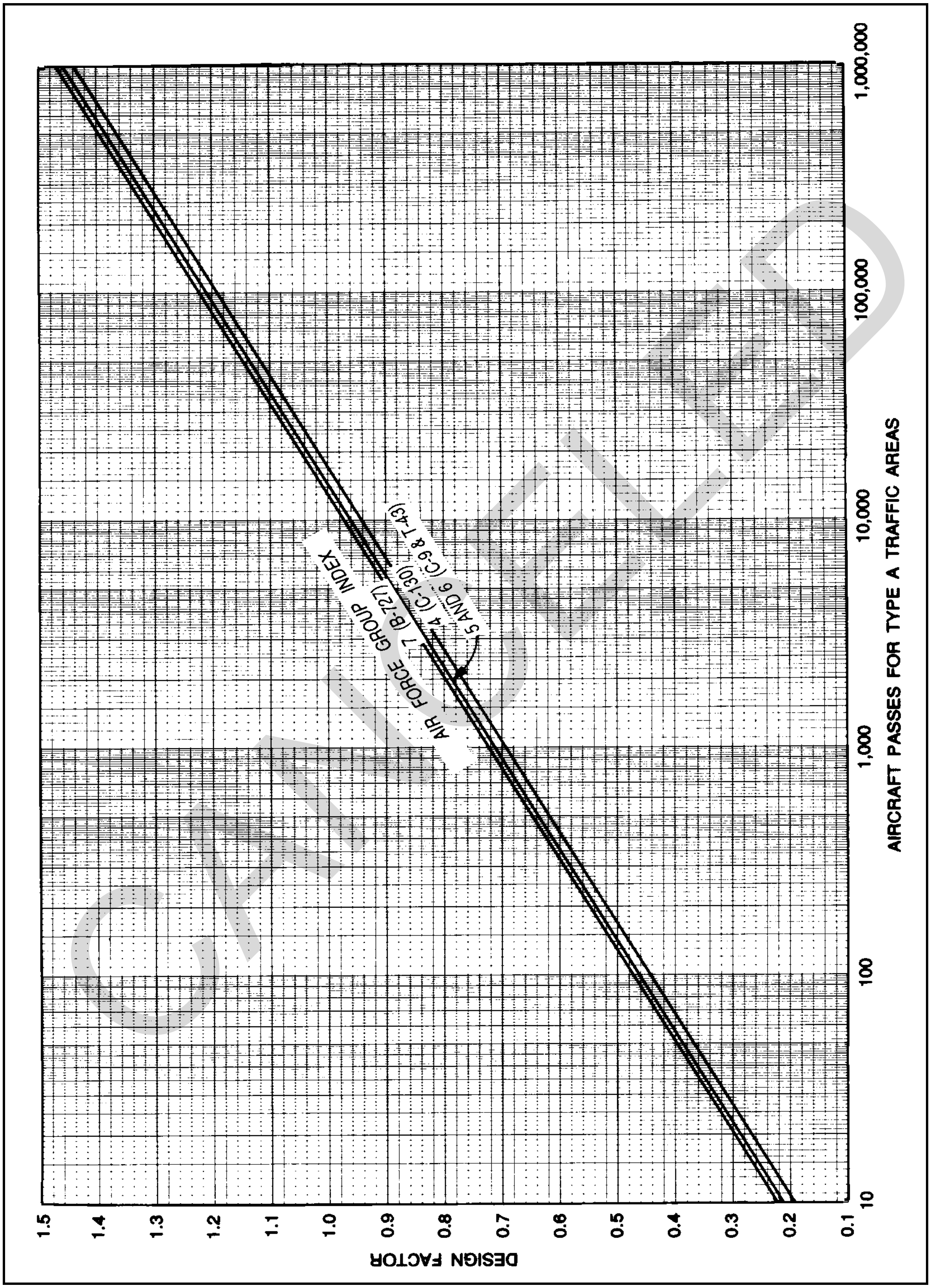


Figure 6-85. Fibrous concrete design factors for Air Force Group Indices 4, 5, 6, and 7, Type A traffic areas

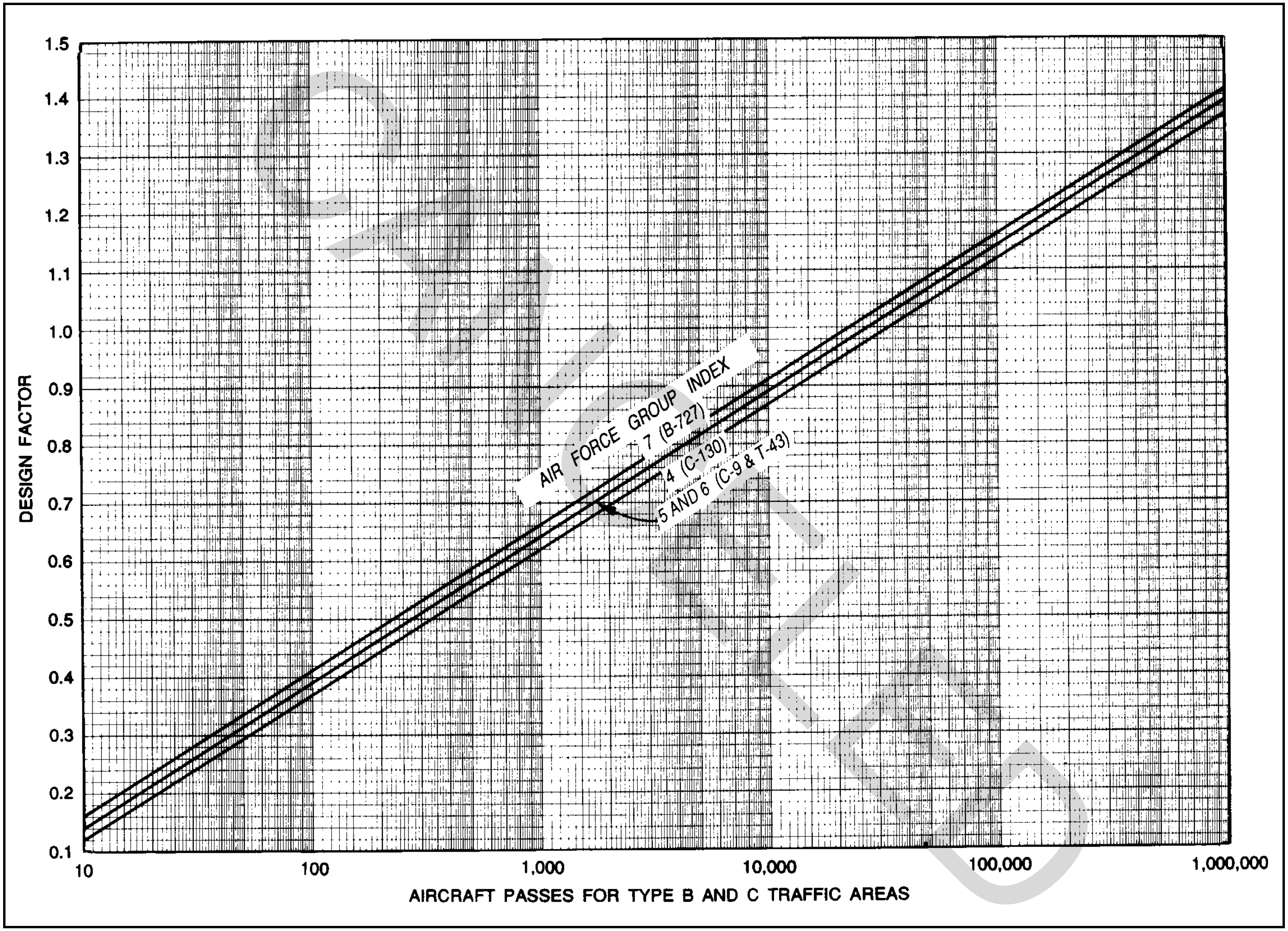


Figure 6-86. Fibrous concrete design factors for Air Force Group Indices 4, 5, 6, and 7, Types B and C traffic areas

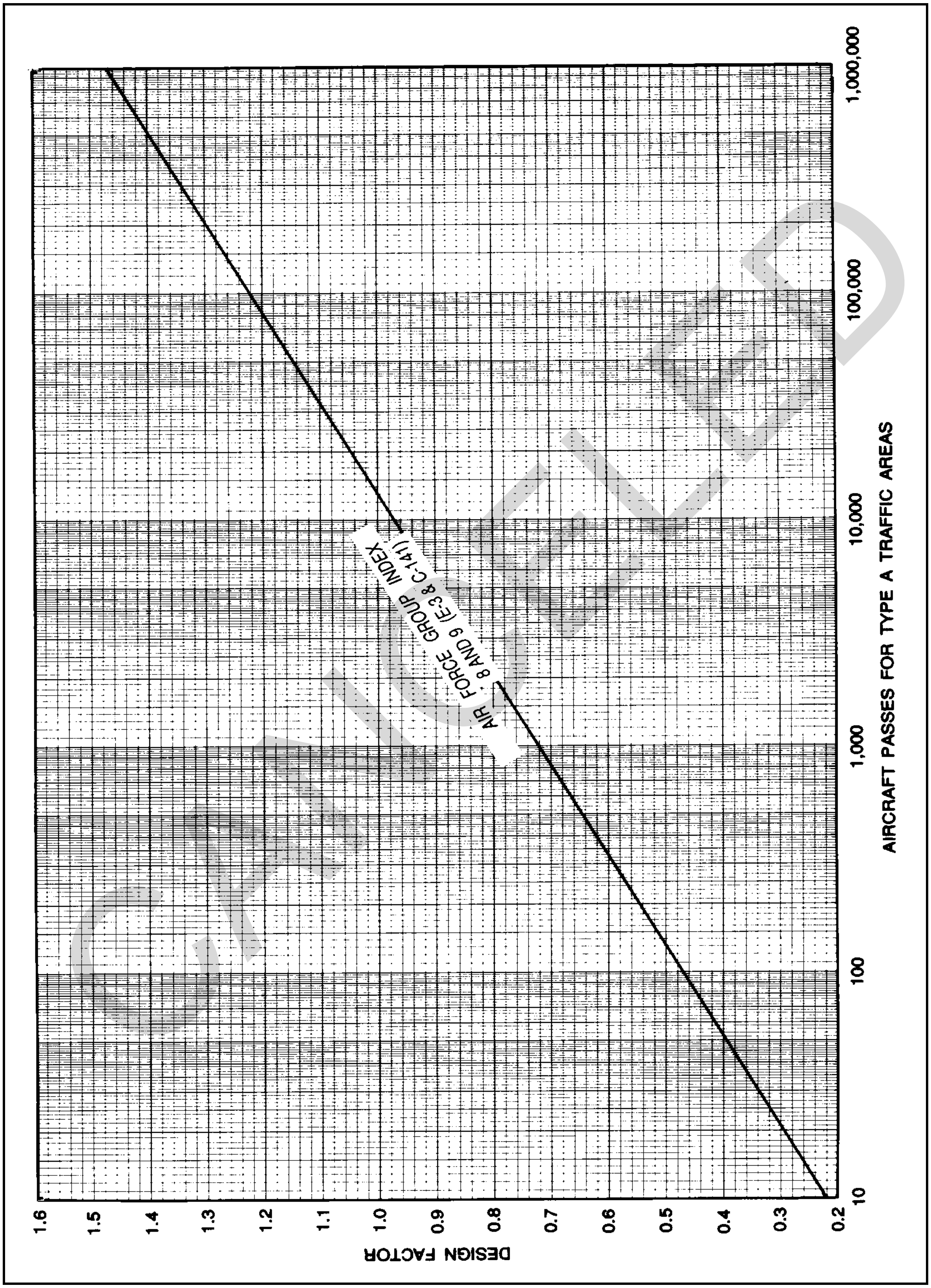


Figure 6-87. Fibrous concrete design factors for Air Force Group Indices 8 and 9, Type A traffic areas

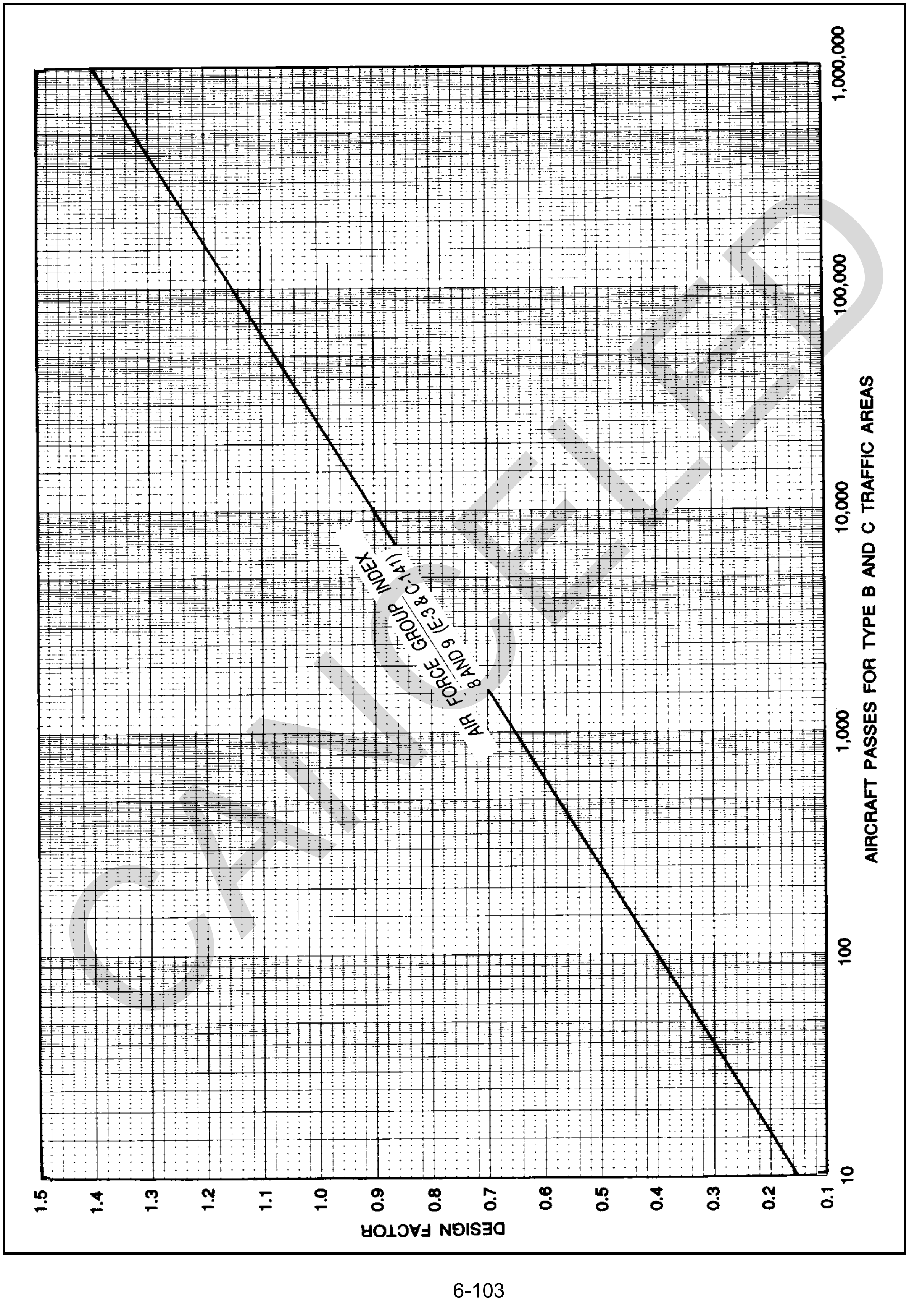


Figure 6-88. Fibrous concrete design factors for Air Force Group Indices 8 and 9, Types B and C traffic areas

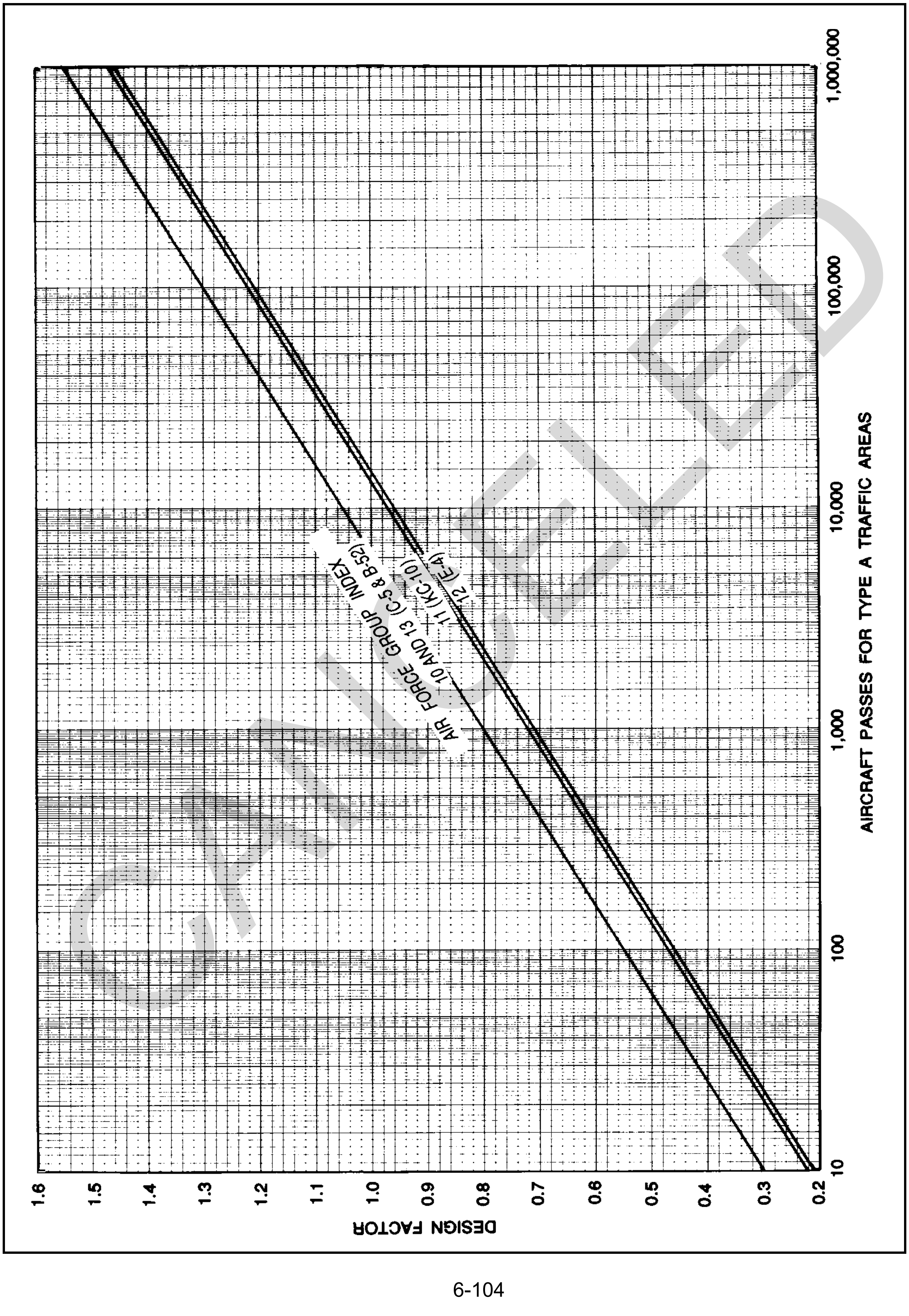


Figure 6-89. Fibrous concrete design factors for Air Force Group Indices 10, 11, 12, 13, and 14, Types A traffic areas

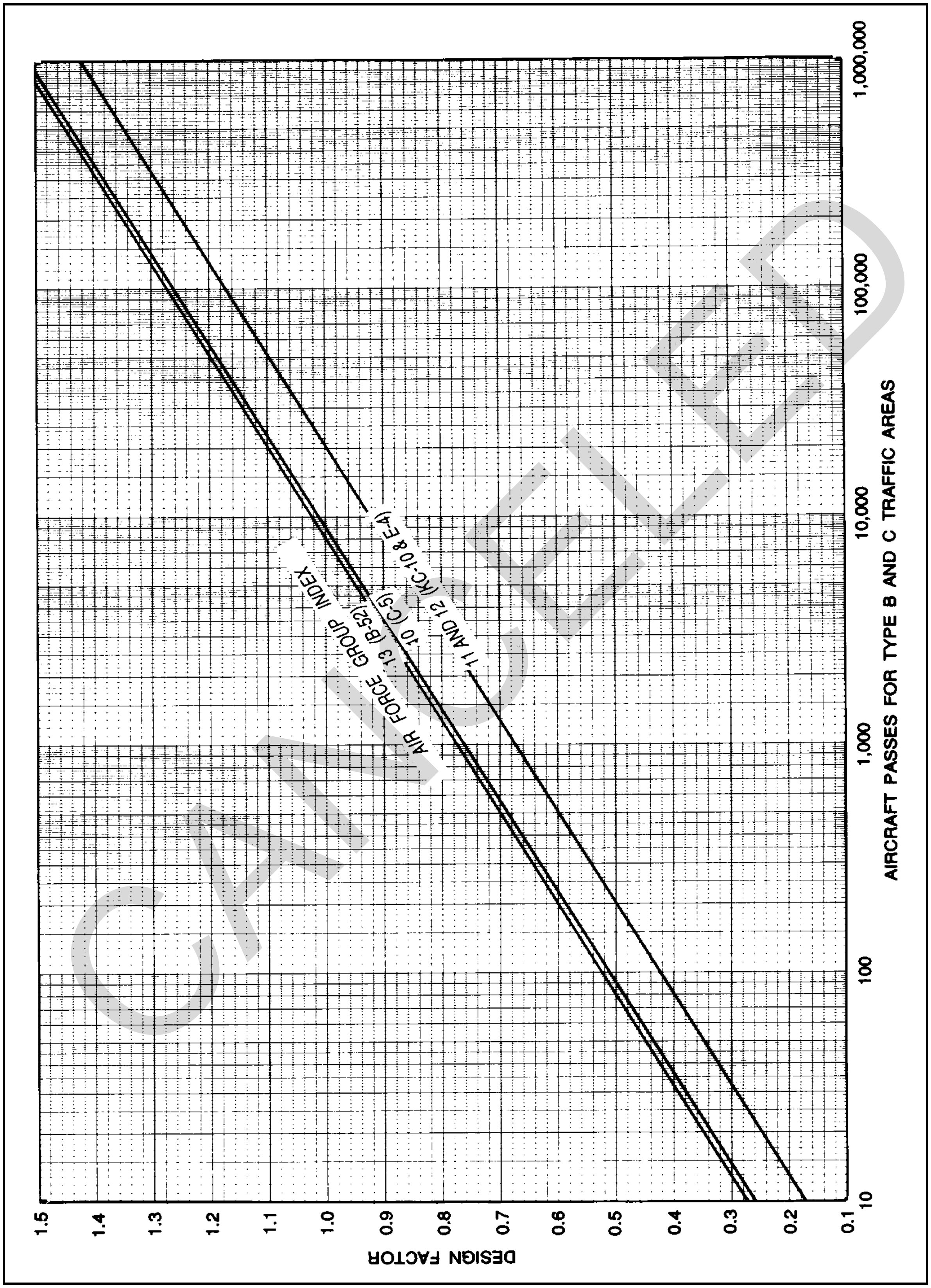


Figure 6-90. Fibrous concrete design factors for Air Force Group Indices 10, 11, 12, 13, and 14, Types B and C traffic areas

CHAPTER 7

PAVEMENT EVALUATIONS FOR FROST CONDITIONS

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 capacity 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 as prescribed in this chapter. Evaluations of airfields during thaw-weakening periods will use pass intensity levels identified in chapter 2.

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.

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 1.37 meters (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.

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. Design freezing indices are not available for Eurasia.

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

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 and 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.

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. 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 installation 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.

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.

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. 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. One common indication of freeze-thaw damage to PCC pavements is the appearance of D (Durability) cracking. These are closely spaced cresent-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.

8. PAVEMENT EVALUATION-GENERAL. The 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. Agencies 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.

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 (CEMP-ET), 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. These data are available in WORLDINDEX.

English Units

$$(DFI) = 429 + 1.143 \times mean\ freezing\ index\ (°F\ days)$$

(eq 7-1)

SI Units

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

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.

11. EVALUATE PAVEMENT FOR COMPLETE FROST PROTECTION. The combined base thickness required to prevent freezing into the subgrade with respect to the design freezing index is determined from the computer program MODBERG (available through the PCASE bulletin board). Frost penetration depths determined from MODBERG 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.

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

where

c = design thickness of combined base for complete frost protection (from MODBERG)

d = thickness of pavement and combined base for complete frost protection

p = thickness of surface layer

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 ≥ c) or (x ≥ 1,524 - 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.

Table 7-1
Frost Susceptibility Soil Classification

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

[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 design soil classification.

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

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

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 abscissa 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. If the base/ subbase thickness (x) at the evaluated site is equal to or greater than b or equal to or greater than 1524 mm (60 inches) minus the pavement thickness, the pavement is adequately protected against detrimental frost action.

$$b = c \times f \qquad \text{(eq 7-4)}$$

where

b = combined base thickness for limited subgrade frost penetration

f = factor from table 7-2

c = design thickness of combined base for complete frost protection (from MODBERG computer program)

Table 7-2
Values for Different Water Content Ratios

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

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.

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 computer program APE. 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.

Table 7-3
FASSI Values for Various Frost Susceptibility Soils

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

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 the computer program APE. 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. The FAIR values can also be estimated from the following equations:

<u>English Units</u>

$$S1 \text{ or } F1 \text{ material: } FAIR\ ((psi/in.) = 4.2 + 10.8 \times base\ Course\ Thickness\ (inches) \qquad \text{(eq 7-5)}$$

$$S2 \text{ or } F2 \text{ material: } FAIR\ ((psi/in.) = 1.3 + 8.0 \times base\ Course\ Thickness\ (inches) \qquad \text{(eq 7-6)}$$

$$F3 \text{ or } F4 \text{ material: } FAIR\ ((psi/in.) = 1.6 + 5.9 \times base\ Course\ Thickness\ (inches) \qquad \text{(eq 7-7)}$$

<u>SI Units</u>

$$S1 \text{ or } F1 \text{ material: } FAIR\ ((MN/m2/m) = 1.13 + 116.4 \times base\ Course\ Thickness\ (meters)$$

$$S2 \text{ or } F2 \text{ material: } FAIR\ ((MN/m2)/m) = 0.36 + 85.6 \times base\ Course\ Thickness\ (meters)$$

$$F3 \text{ or } F4 \text{ material: } FAIR\ ((MN/m2)/m) = 0.44 + 63 \times base\ Course\ Thickness\ (meters)$$

FAIR values are used as if they were moduli of soil reaction, 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.

14. REDUCTION FACTORS FOR NONDESTRUCTIVE TESTING. The moduli of the subgrade during thaw periods are reduced modulus values obtained during the nonfrost period. Reduction factors (RF) are

given 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.

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

Table 7-4
Modulus Reduction Factors for use in Seasonal Frost Areas

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

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 either a C-130 or C-141 for Army 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) (CEMP-ET), 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 7-4. 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 ≤ 13,330 °C hours (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 > 13,300 °C hours (1,000-°F days), a month will be added to the thaw-weakening period recommended in table 7-5.

Table 7-5
Length of End-of-Winter Thaw-Weakened Period for Evaluation Purposes

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

(3) The MODBERG program and the WORLDINDEX database (both programs are available from the PCASE Bulletin Board) contain information on the length of the freezing season (winter) and the mean date of the start of the freezing season and can be used to estimate these dates and length of the freezing season.

(4) A more accurate estimate of the total period of weakening can be obtained by applying the FROST program using measured air temperatures from, or near, the site for a 10- to 30-year period. The CRREL technical staff can provide additional guidance on the use of this program.

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 total period of weakening. For calculating the allowable load during the total period of weakening, it is assumed that the thaw-weakening period is over a 1-year period. The allowable traffic over this time period is the design traffic divided by the design year. Using the APE program or LEEP for reduced modulus determines the allowable load and PCN for the controlling aircraft for the pavement profile and properties at the site during the total period of weakening.

16. EXAMPLE 1. Evaluate an Air Force flexible pavement type A traffic area consisting of 127 millimeters (5 inches) of asphalt concrete, 229 millimeters (9 inches) of crushed stone base (CBR = 100), and 305 millimeters (12 inches) of subbase (CBR = 30) over a silt subgrade. The pavement is to be evaluated for the C-17 aircraft. The pavement surface is in good condition. The subgrade has dry density of 1,762 kilogram/cubic meter (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: 2,163 kilograms/meter (135 pounds per cubic foot) and average water content after drainage of 3 percent. The highest ground water is 610 millimeters (2 feet) below subgrade surface. For this example, the airfield is located in Bismarck, ND.

a. From WORLDINDEX:

DFI = 38,712 °C hours (2,903 °F days)
Mean annual temperature = 5°C (41°F)

Mean freezing length = 129 days
Mean start of freezing occurs on November 11.
Mean end of freezing occurs on March 19

b. Determine the Depth of Frost Penetration. For complete frost protection the depth of frost penetration (d) estimated from MODBERG program is 1,422 mm (56 inches).

c. Determine if the Base/Subbase and/or Subgrade is Frost-Susceptible. From table 7-1, the subgrade is classified as a F3 frost-susceptible soil.

d. Evaluate for Complete Frost Penetration. With a 127-millimeter- (5-inch-) thick pavement, the thickness of base course (c) for zero penetration of the subgrade is 1,422 - 127 = 1,295 millimeters (56 - 5 = 51 inches). The thickness of the pavement, base, and subbase layer (x) is 660 millimeters (26 inches). Since $x \le c$, then the pavement structure was not designed for complete frost protection.

e. 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 813 millimeters (32 inches). In this case, the 660-millimeter- (26-inch-) thick section of pavement and base does not provide adequate protection against frost action, and evaluation for frost and nonfrost conditions are required.

f. Evaluate for Reduced Subgrade Strength.

(1) Determine allowable load and PCN during thaw-weakened period. The soil is classified as a F3 frost-susceptible soil. From table 7-3, the FASSI value is 3.5. Use either figure 5-27 and the procedure outlined in chapter 5 or the computer program Airfield Pavement Evaluation (APE) to determine the allowable load for the traffic area during the thaw-weakened period using the FASSI value of 3.5. The allowable loads for pass intensity levels I to IV and the respective PCN are tabulated below.

Pass Intensity Level for C-17 Aircraft	No. of Passes	Allowable Load kg (lb)	PCN
I	15,000	97,296 (214,500)	19/F/D/W/T
II	3,000	114,623 (252,700)	27/F/D/W/T
III	500	146,692 (323,400)	40/F/D/W/T
IV	100	201,667 (444,600)	64/F/D/W/T

(2) Determine the Period of Thaw Weakening. From WORLDINDEX, the design beginning of freezing is Nov 11th and ends in March 19th. The average length of the freezing season is 129 days. The thaw weakened period after the end of winter from table 7-4 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 mean thaw weakened period is from Feb 17th to May 18th.

17. EXAMPLE 2. Evaluate an Air Force rigid pavement type B traffic area consisting of 508 millimeters (20 inches) of PCC and 102 mm (4 inches) of base on a clay subgrade. The flexural strength of the concrete is 4.5 MPa (650 psi). Visual inspection of the pavement shows it to be in good condition. The pavement is to be evaluated for the C-17. The aircraft traffic is applied uniformly throughout the year. The subgrade is a clay with a PI of 10, a dry density of 1,602 kilograms per cubic meter (100 pcf), and an average water content of 18 percent. From field tests, the subgrade k during the normal period was 33 KPa/mm (125 pci). The base material is a nonfrost-susceptible sandy gravel (GW) with a normal k value of 122 kPa/mm (450 pci). The average dry unit weight and average water content of the subbase layer are 2,163 kilograms/cubic meter (135 pcf) of 5 percent, respectively. The highest groundwater is at the subgrade surface. For this example, the airfield is located in Fairbanks, AK.

a. From WORLDINDEX:

DFI = 86,496 °C hours (6,487 °F days)
Mean annual temperature = -2.6 °C (27.3 °F)
Mean freezing length = 188 days
Mean start of freezing occurs on October 11
Mean end of freezing occurs on April 16

b. Determine if Base/Subbase and/or Subgrade is Frost Susceptible. From table 7-1, the subgrade is classified as a F4 frost-susceptible soil.

c. Determine Depth of Frost Penetration. The depth of frost penetration (d) estimated from the MODBERG program, assuming coarse grained material below the PCC layer is 2,565 mm (101 inches).

d. Evaluate for Complete Frost Penetration. The combined base thickness of pavement and base to prevent any freezing of the subgrade in the design freezing index year (complete protection) is 2,565 millimeters (101 inches). With a 508-millimeter- (20-inch-) thick pavement, the thickness of base course (c) for zero penetration of the subgrade is 2,057 millimeters (81 inches). The thickness of the pavement, base, and subbase layer (x) is 1,016 millimeters (40 inches). Since $x \leq c$, then the pavement structure was not designed for complete frost protection.

e. 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 1,321 millimeters (52 inches). In this case, the 610-millimeter- (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.

f. Evaluate for Reduced Subgrade Strength.

(1) Determine allowable load and PCN during thaw-weakened period. The soil is classified as a F4 frost-susceptible soil. From figure 7-7, the FAIR value is 50 kPa/mm (185 pci). The design load for the C-17 is 263,084 kilograms (580,000 pounds). Using figure 6-17, the procedure outlined in chapter 6, or the computer program APE, evaluate the allowable load for the traffic area during the thaw-weakened period using the FAIR value of 50 kPa/mm (185 pci). The allowable loads for pass intensity levels I to IV, and the respective PCN are tabulated below.

Pass Intensity Level	No. of Passes	Allowable Weight Kg (lb)	PCN
I	15,000	263,084 (580,000)	86/R/D/W/T
II	3,000	263,084 (580,000)	102/R/D/W/T
III	500	263,084 (580,000)	126/R/D/W/T
IV	100	263,084 (580,000)	159/R/D/W/T

(2) Determine the period of thaw weakening. The maximum load of 263,084 kg (580,000 lb) can be applied throughout the year.

18. EXAMPLE 3. Evaluate an Army Class IV airfield pavement taxiway consisting of 102 millimeters (4 inches) of asphalt concrete 152 millimeters (6 inches) of crushed aggregate base, and 508 millimeters (20 inches) of subbase on a silty sand subgrade. The pavement is in fair condition. The evaluation will be for the C-130 aircraft at a design pass level of 26,000. The aircraft traffic is applied uniformly throughout the year. The subgrade has a PI of 10, a dry density of 1,602 kilograms/cubic meter (100 pcf) and an average water content of 18 percent. From field tests, the subgrade CBR during the normal period was 7. The base was a nonfrost-susceptible well-graded gravel (GW) with a normal CBR value of 80, and the subbase was a nonfrost-susceptible sandy gravel (GP) and exhibited a normal period CBR value of 50. The average dry unit weight and average water content of the base/subbase layer are 2,163 kilograms/meters (135 pcf) and 3 percent, respectively. The highest ground water is at the sub-grade surface. For this example, this airfield is in Fairbanks, AK.

a. From WORLDINDEX:

DFI = 86,496 °C hours (6,487 °F days)
Mean annual temperature = -3 °C (26.6 °F)
Mean freezing length = 188 days
Mean start of freezing occurs on October 11
Mean end of freezing occurs on April 16

b. Determine if Base/Subbase and/or Subgrade is Frost Susceptible. From table 7-1, the subgrade is classified as a F4 frost-susceptible soil.

c. Determine Depth of Frost Penetration. The depth of frost penetration (d) estimated from the MODBERG program is 2.5 meters (100 inches).

d. Evaluate for Complete Frost Penetration. The combined base thickness of pavement and base to prevent any freezing of the subgrade in the design freezing index year (complete protection) is 2.5 meters (100 inches). With a 102-millimeter-(4-inch-) thick pavement, the thickness of base course (c) for zero penetration of the subgrade is 2.2 meters (96 inches). The thickness of the pavement, base, and subbase layer (x) is 76 millimeters (30 inches). Since $x \leq c$, then the pavement structure was not designed for complete frost protection.

e. Evaluate for Limited Subgrade Frost Penetration. The ratio of subgrade to base course water content r = 18/3 = 6. From figure 7-5, using the maximum permissible ratio r of 2.0 applicable to traffic area B, the required total base thickness b that would hold subgrade frost penetration within the allowable limit is 1.4 meters (55 inches). In this case, the 76-millimeter- (30-inch-) thick section of pavement and base does not provide adequate protection against frost action, and evaluation for frost and nonfrost conditions are required.

f. Evaluate for Reduced Subgrade Strength.

(1) Determine the allowable load and PCN during thaw-weakened period. The soil is classified as F4 frost susceptible soil. From table 7-3, the FASSI value is 3.5. The figure 5-16 or from APE, for 26,000 passes, the allowable load during the thaw-weakened period is 186,800 pounds. The PCN is 39/F/D/W/T.

(2) Determine the period of Thaw Weakening. From WORLDINDEX, the design beginning of freezing is Oct 11th and ends in April 16th. The average length of the freezing season is 188 days. The thaw weakened period after the end of winter from table 7-4 is estimated to be 3 months. Since the airfield is located in an area with a DFI of 6487 °F days, an additional 1 month (to cover for intermediate thaw period during the freezing period) is added to the 3 months to obtain the total weakening period of 4 months. The mean thaw-weakened period is from March 17th to July 15th.

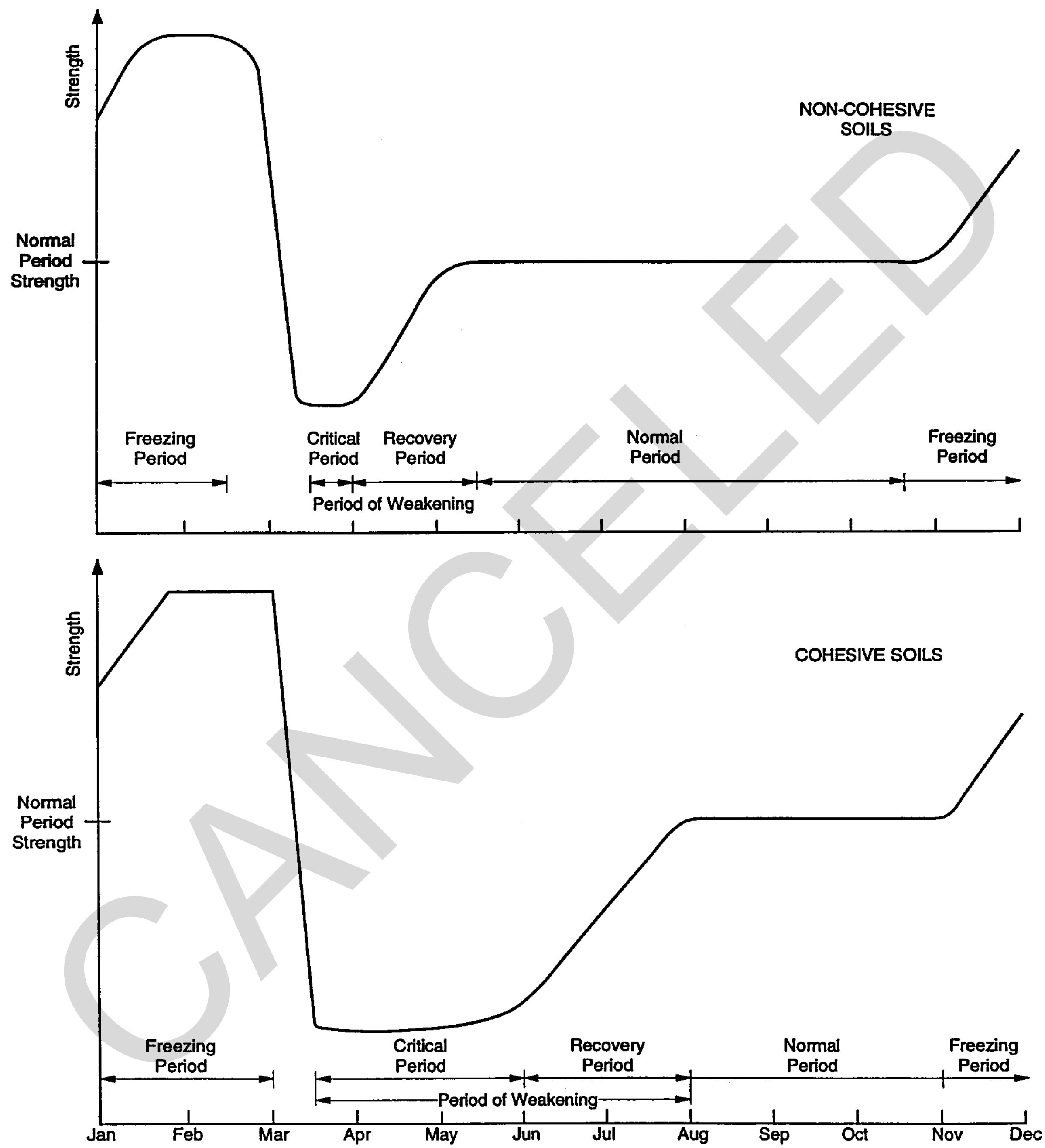
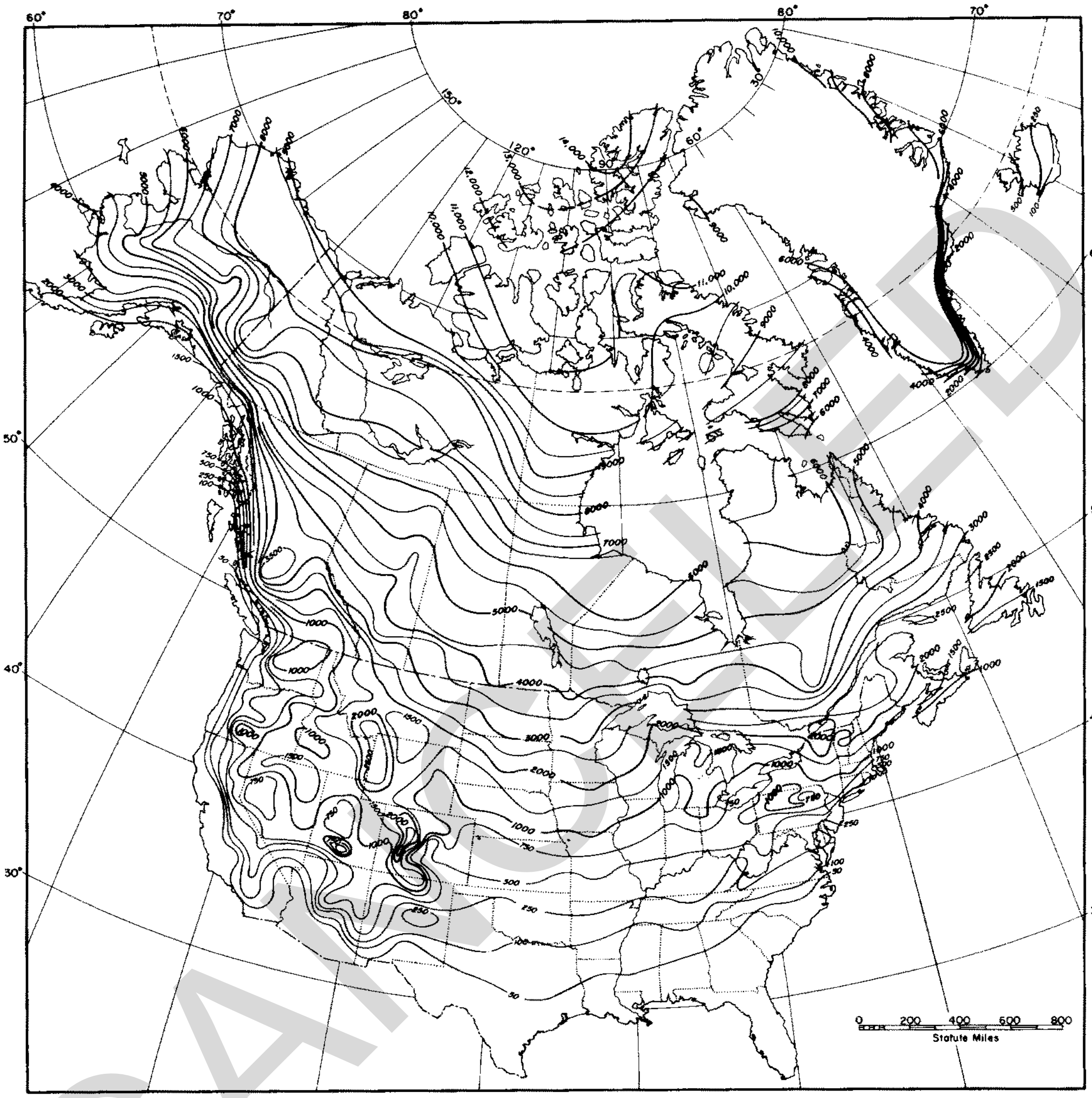


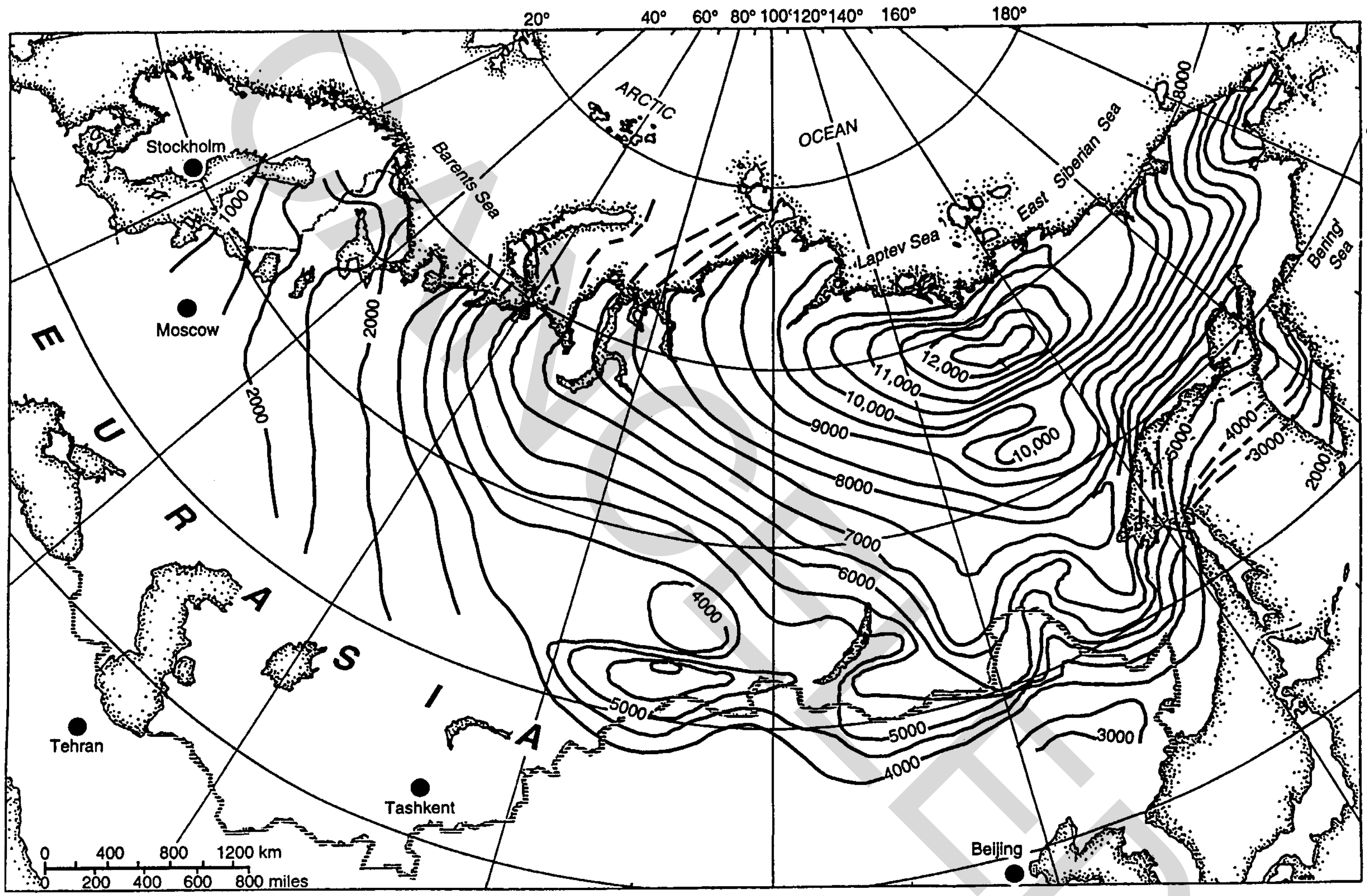
Figure 7-1. Illustration of thaw-weakening period



CONVERSION FACTOR

DEG F DAYS = 0.075 × DEG C HOURS

Figure 7-2. Distribution of design air freezing indices in North America



CONVERSION FACTOR

DEG F DAYS = $0.075 \times$ DEG C HOURS

Figure 7-3. Distribution of mean air freezing indices in northern Eurasia

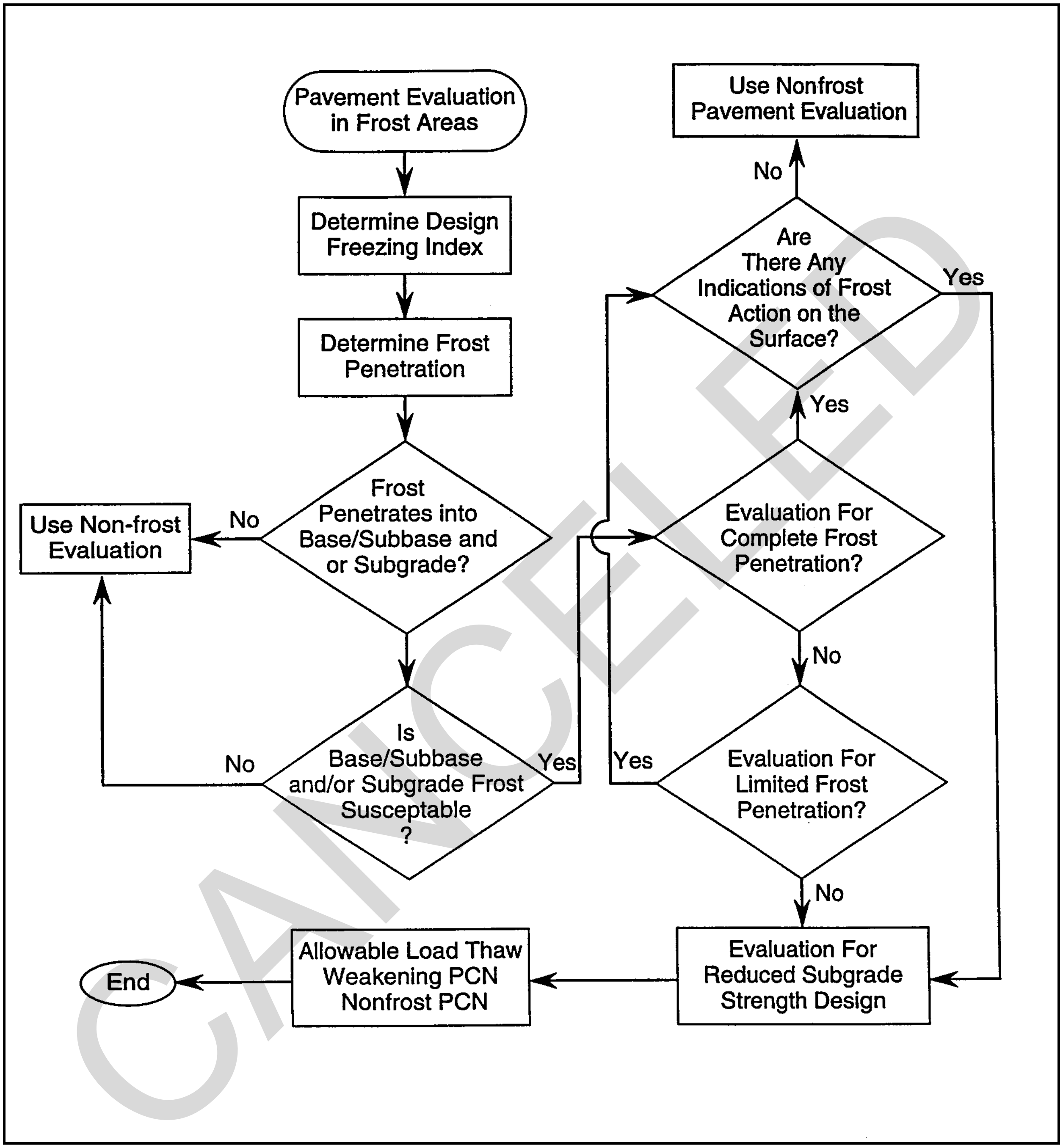


Figure 7-4. Pavement evaluation in frost areas

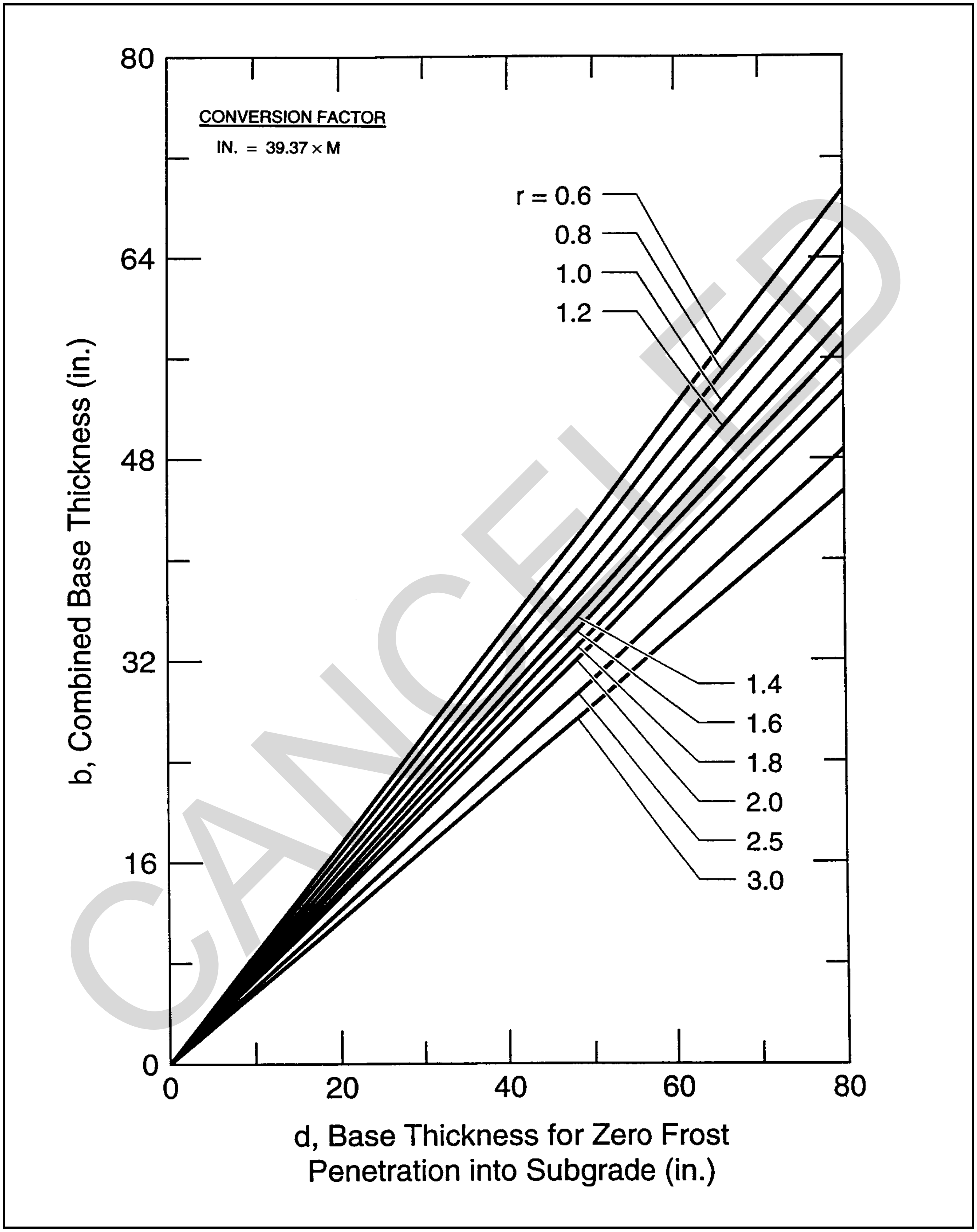


Figure 7-5. Estimation of combined base for limited subgrade frost penetration

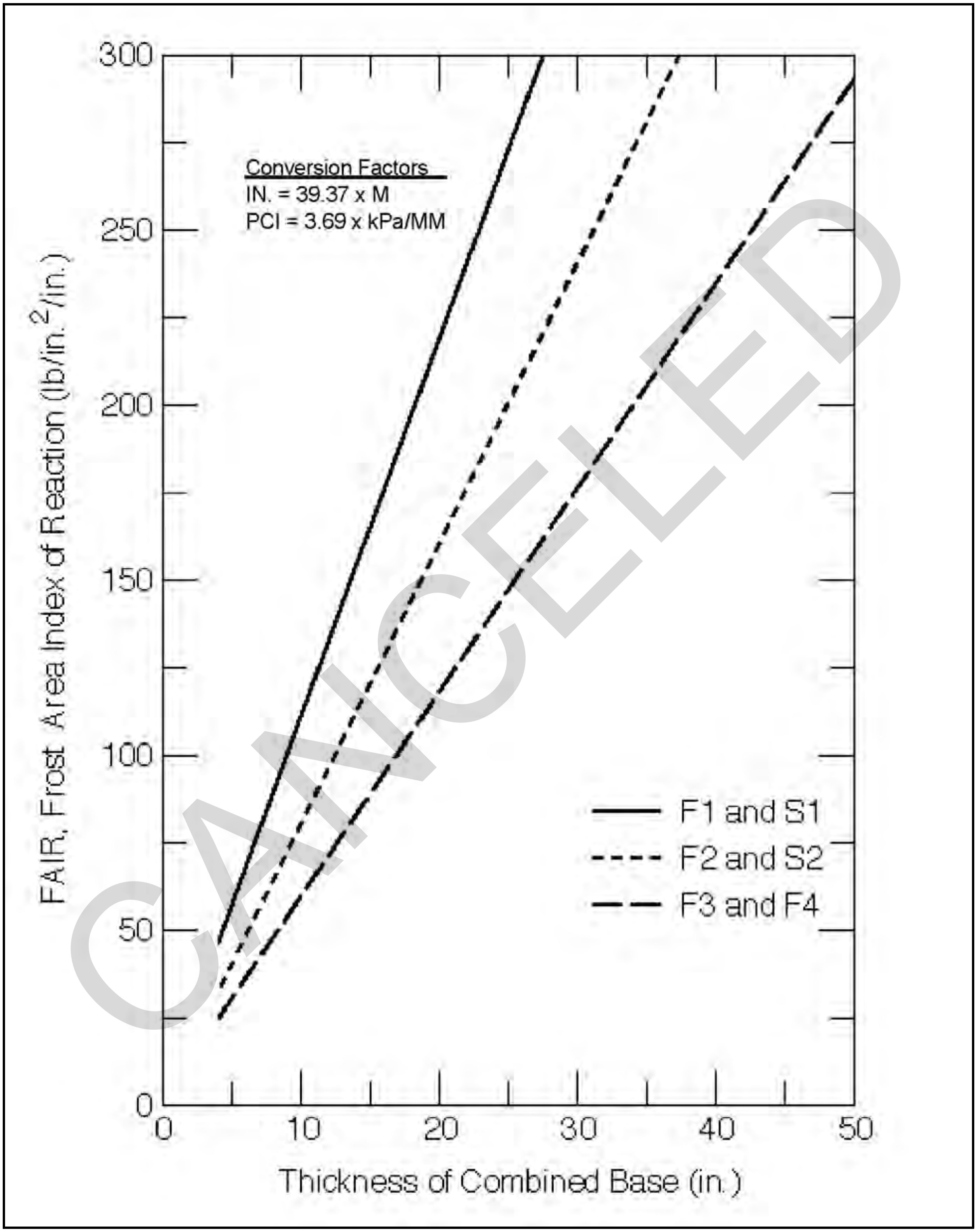


Figure 7-6. Determination of FAIR values

CHAPTER 8

REPORTING ALLOWABLE WEIGHT BEARING USING THE ACN/PCN PROCEDURE

1. GENERAL DESCRIPTION.

a. The PCN is an index number representing the largest load on an established standard single wheel which could be permitted to use a particular pavement without special reservations. Likewise, the ACN is an index number representing the load on the same established standard single wheel that is equivalent to an actual aircraft at a particular weight. The ACN therefore represents the equivalent loading an aircraft will apply to a pavement, and PCN represents the equivalent loading a pavement can withstand. A comparison of these two values will indicate whether or not an airfield pavement can support a particular aircraft.

b. Reasonably unique relations have been developed between ACN or PCN and aircraft weight for any aircraft for limited ranges of subgrade strength and for pavement types. The ACN/PCN method establishes four subgrade strength categories for each of two pavement types--flexible and rigid where rigid includes plain concrete, plain concrete overlays, and nonrigid overlays on plain concrete. Any individual pavement will have characteristics which place the pavement in only one of the subgrade and pavement type categories.

c. The establishment of a PCN for a particular pavement will require relations between PCN and aircraft weight for pertinent use aircraft and for the applicable pavement type and subgrade strength category. These have been determined using established procedures and standard parameters of the ACN/PCN method and are shown in figures 8-1 through 8-11.

d. Determination of the PCN for reporting weight-bearing capacities of pavements proceeds from the results of the pavement evaluation. The Army, Navy, and Marine Corps evaluations should determine the maximum allowable gross weight of the most critical aircraft that can use the pavement for the number of equivalent passes expected to be applied for the following 20-year period (10-year period for Navy and Marine Corps). The Air Force evaluation should determine the maximum allowable gross weight for 50,000 passes of the Group 10 (C-17) aircraft. The use of these maximum weights with figures 8-1 through 8-11 will allow the determination of the numerical PCN value. The evaluation should also provide sufficient information to permit selection of the pavement type and subgrade strength category.

2. PCN DETERMINATION.

a. To proceed with PCN determination, it is first necessary to establish the pavement type and sub grade strength category. The pavement type is selected as either rigid (Code R) or flexible (Code F). If the pavement is PCC or has PCC as a primary structural element, and is neither 102 millimeters (4 inches) or less in thickness or completely shattered, it should be considered a rigid pavement. Virtually all other pavement should be considered flexible.

b. Subgrade strength category should be determined from the following tabulation using characteristics of the pavement being rated.

		Subgrade Strength Category				Characteristic Value	
		Upper Limit		Lower Limit			
Code	Rating	K[1]	CBR	K[1]	CBR	K[1]	CBR
A	High strength	--	--	108 (400)	13	135 (500)	15
B	Medium strength	108 (400)	13	54 (200)	8	80 (300)	10
C	Low strength	54 (200)	8	27 (100)	4	40 (150)	6
D	Ultra-low strength	27 (100)	4	--	--	20 (75)	3

[1] K in MN/cubic meter (pounds per cubic inch).

c. For Army, Navy, and Marine Corps airfield pavements, the PCN is based upon the maximum loading of the most critical aircraft which can be allowed to use the airfield for the following 20-year period. This loading is based upon the average day-to-day traffic that has been converted to equivalent passes of the critical aircraft and extrapolated for the next 20 years. It should be noted that for airfield pavements limited to aircraft weighing less than 5,670 kilograms (12,500 pounds), the PCN is not applied and weight-bearing limits are reported directly in terms of maximum allowable gross weight of aircraft.

d. The PCN value assigned for Air Force airfield pavements will be based upon the allowable loading for 50,000 passes of the Group 10 (C-17) aircraft. This requires each pavement feature to be evaluated for the Group 10 (C-17) at 50,000 passes and a PCN assigned for that allowable loading from figure 8-11.

e. When the critical aircraft and allowable gross weight have been established, the pertinent relation between gross weight and PCN must be attained (for the proper pavement type and subgrade class). As mentioned earlier, the gross weight versus PCN relations can be computed using the standard methods or can be obtained from figures 8-1 through 8-11. By entering the proper figure with the allowable gross weight of the critical aircraft, the limiting PCN can be determined.

f. By the same process, a separate PCN for the frost-melting period can be determined and reported. Some guidance needs to be included, when reporting for a frost-melting period, to indicate during which period the more limiting value must be applied.

3. TIRE PRESSURE LIMITATION. An aspect of ACN/PCN reporting is limitation of tire pressure through application of categories for reporting in accordance with the following tabulation.

		Pressure Limited to:	
Code	Rating	MPa	PSI
W	High	No limit	No limit
X	Medium	1.50	217
Y	Low	1.00	145
Z	Very low	0.50	73

Rigid or rigid-overlay pavement can sustain the High (W) category except where the rigid layer is very thin (less than 102 millimeters (4 inches)) or is thoroughly shattered (pieces less than about 0.6 meters (2 feet wide)).

4. EVALUATION METHOD. The ACN/PCN system also requires a reporting of the general basis of evaluation. Code T will indicate a technical evaluation of the type prescribed herein. Thus, any evaluation following this manual will be reported as a technical evaluation (T). Where the reported PCN must be based only on knowledge of the heaviest aircraft using a facility and without a specific evaluation, it will be reported as a "using aircraft" evaluation (Code U).

5. CODED REPORTING IN FLIGHT INFORMATION PUBLICATION (FLIP) OR AIRFIELD INFORMATION PUBLICATION (AIP) DOCUMENTS. The coding indicated permits a greatly abbreviated reporting of the PCN and related information for use in FLIP or AIP type documents as used by the Defense Department (NIMA) or the civil (FAA) and international (ICAO) communities. Following is an example of coded reporting with explanatory notes.

Example PCN Code = 39/F/C/X/T where:

39 = PCN value established
F = Flexible pavement
C = Low-strength subgrade (between 4 and 8 CBR)
X = Limited to medium tire pressures (less than 217 psi)
T = Technical evaluation

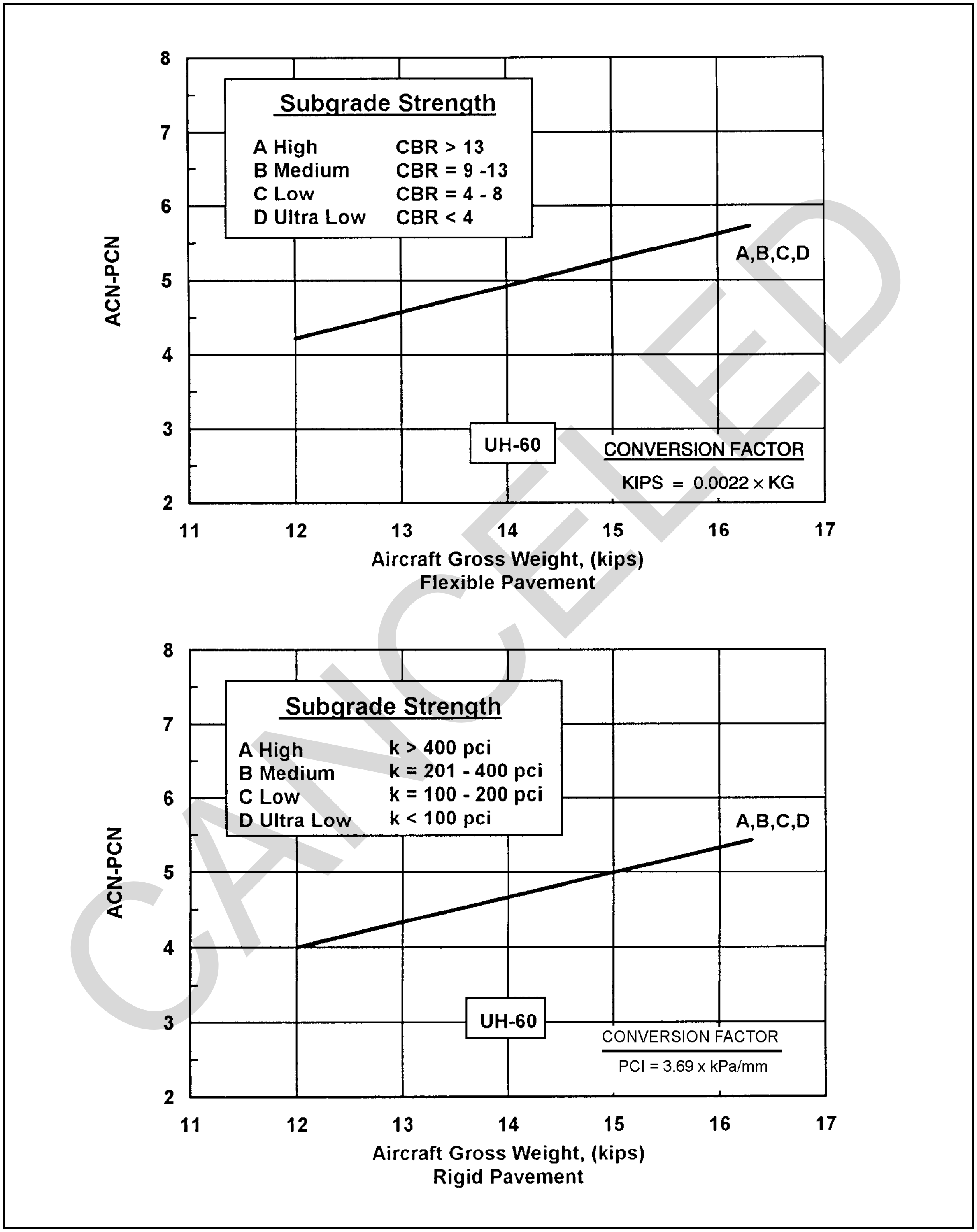


Figure 8-1. PCN curves for UH-60

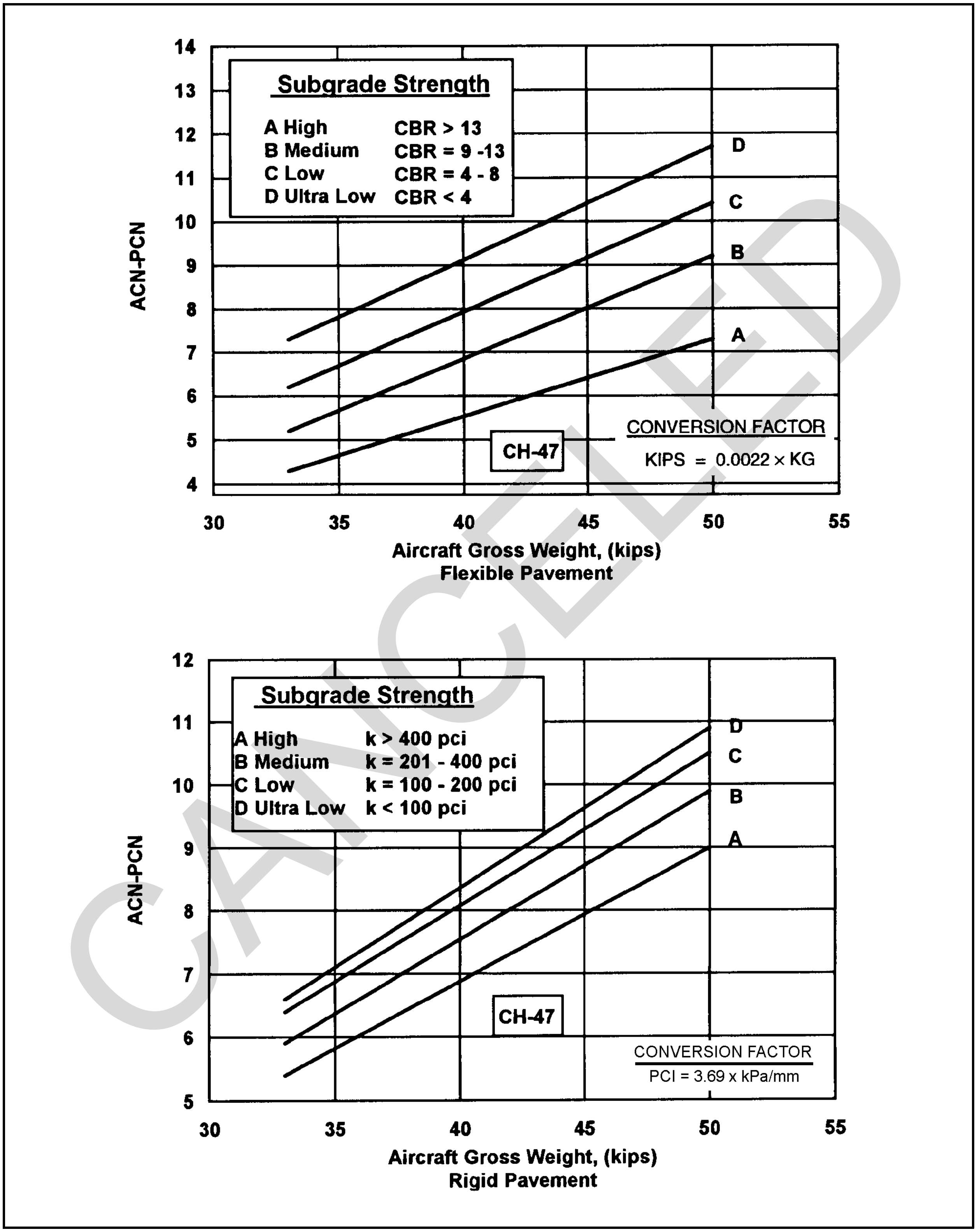


Figure 8-2. PCN curves for CH-47

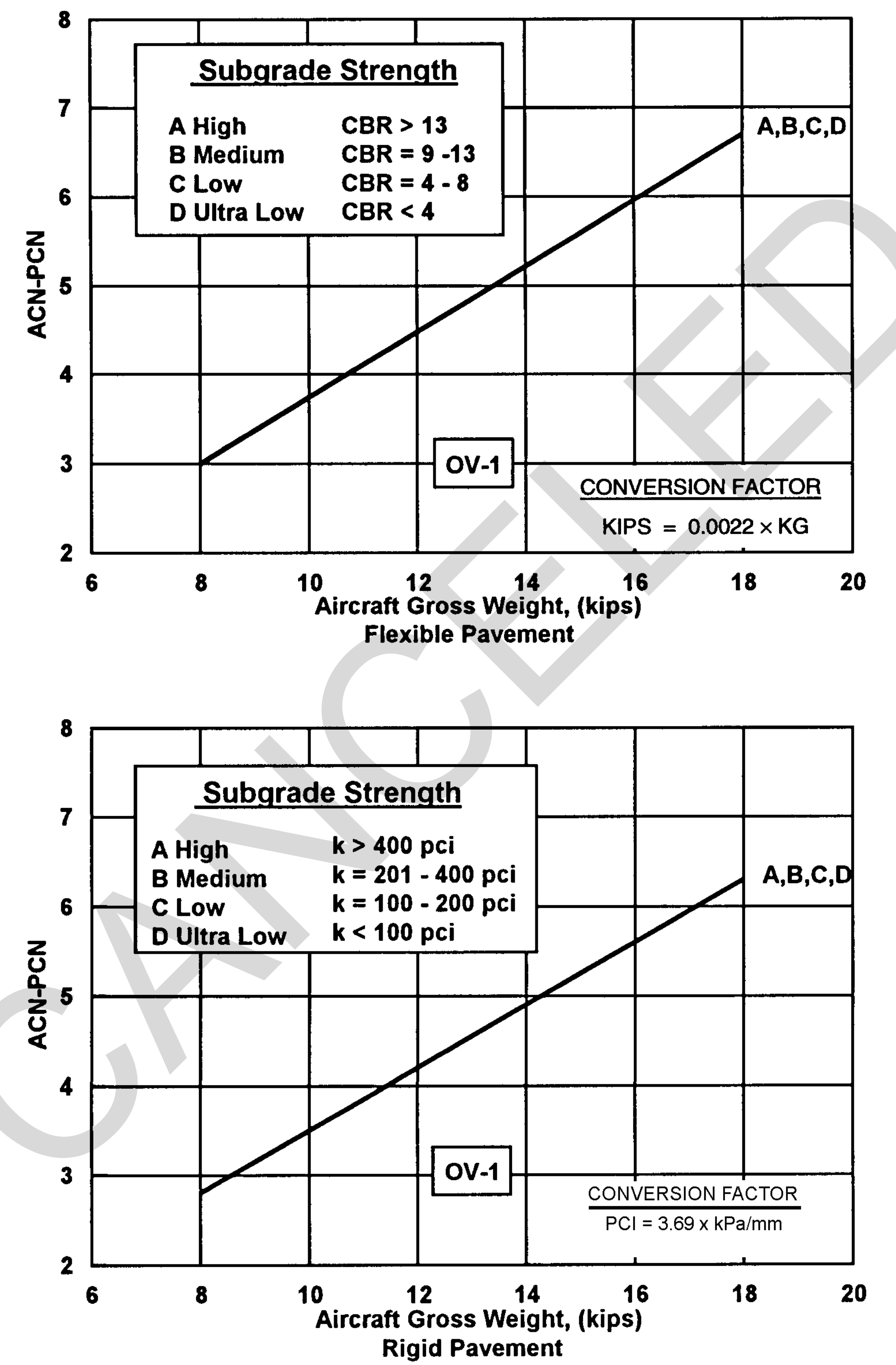


Figure 8-3. PCN curves for OV-1

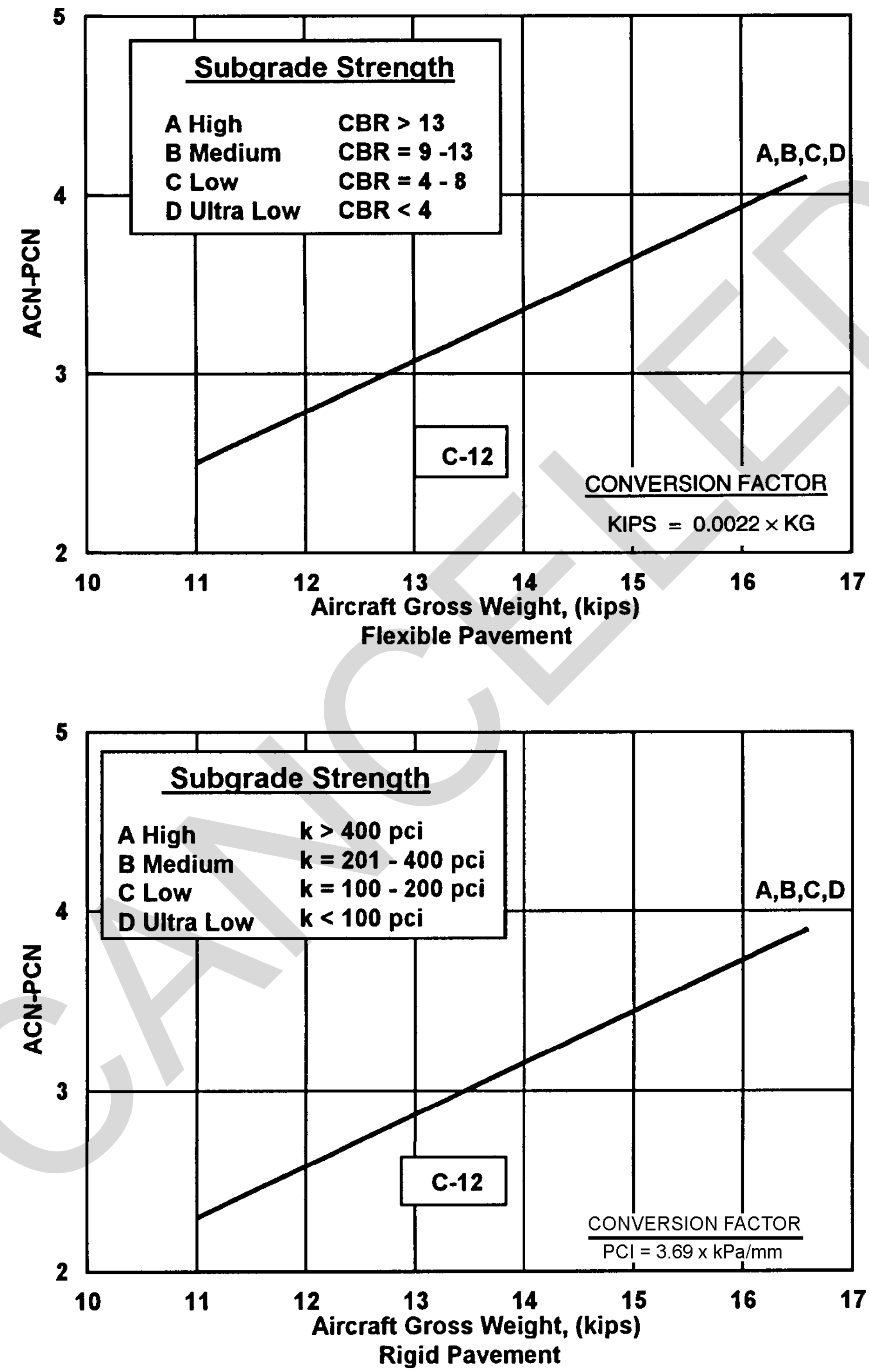


Figure 8-4. PCN curves for C-12

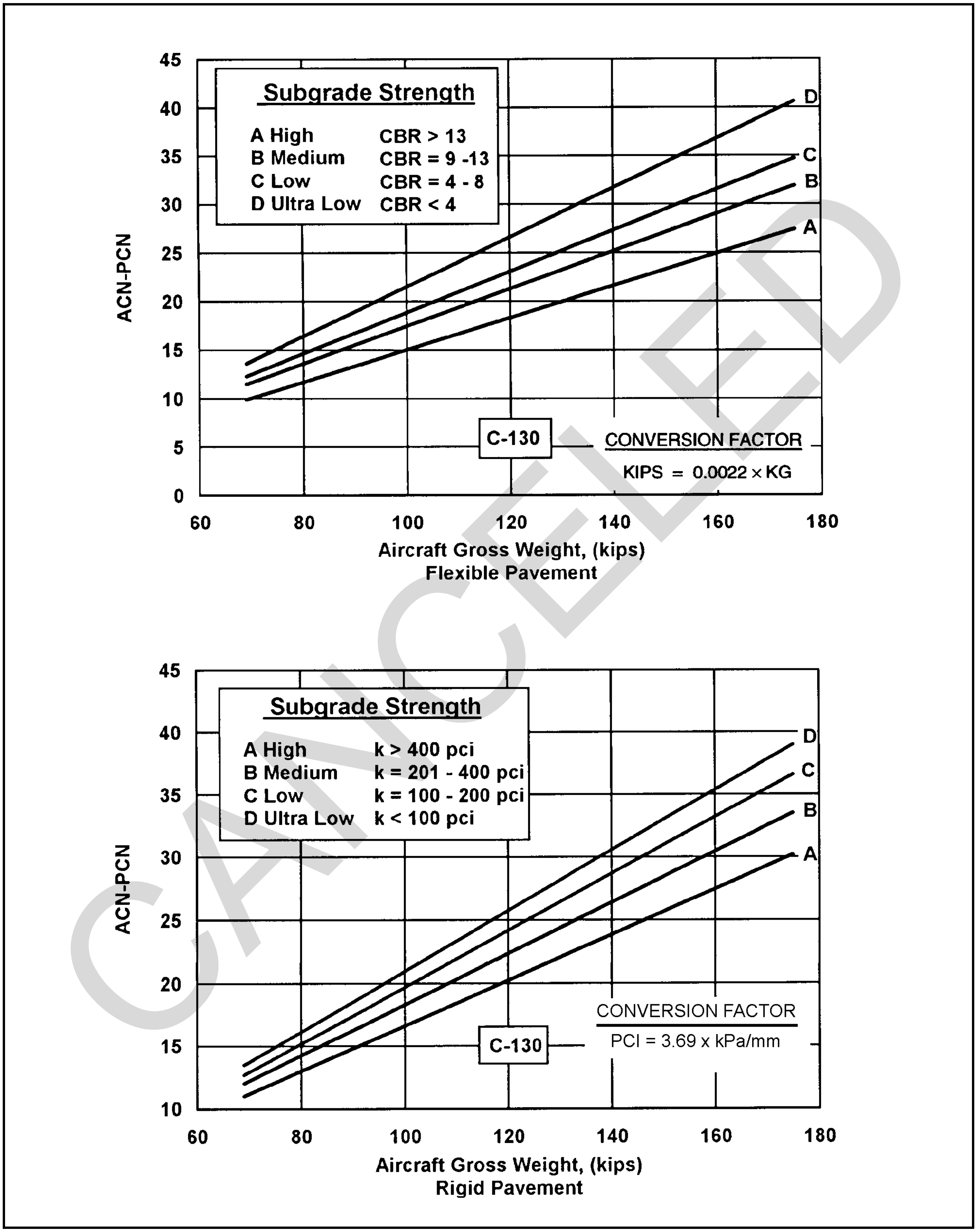
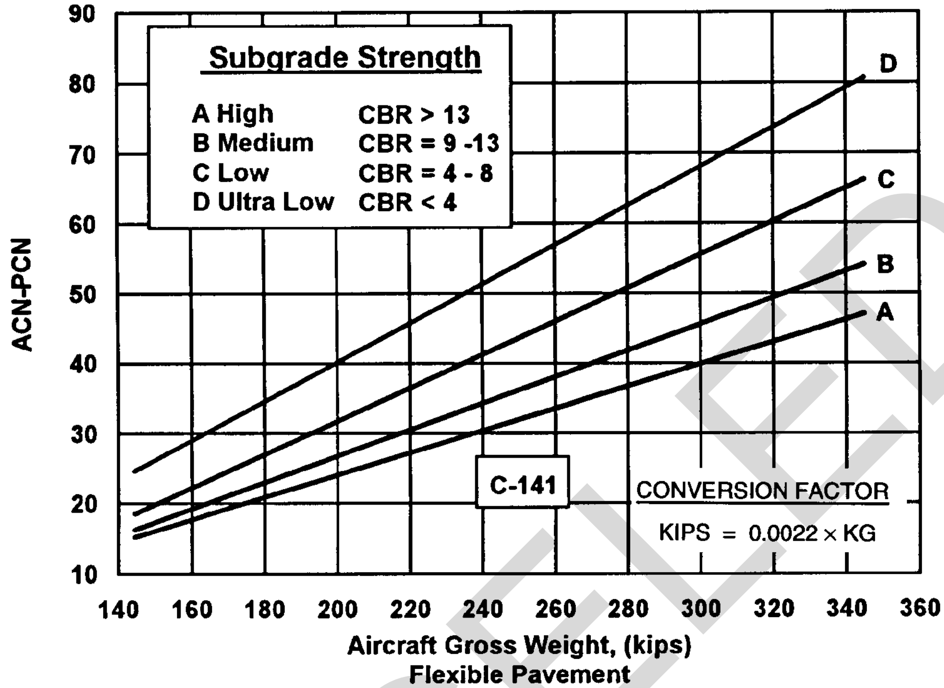


Figure 8-5. PCN curves for C-130



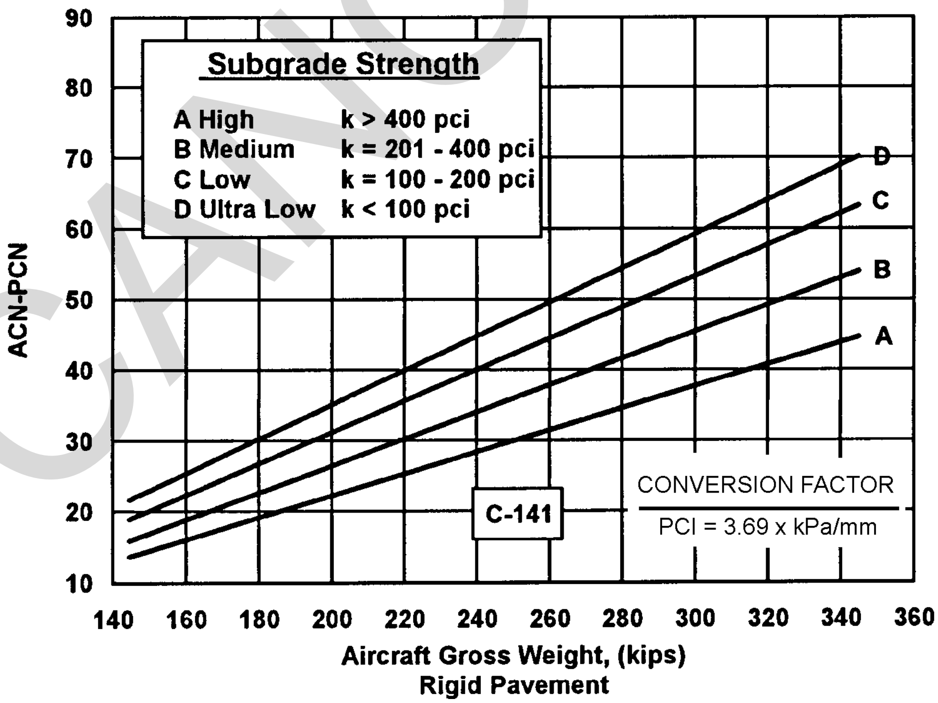
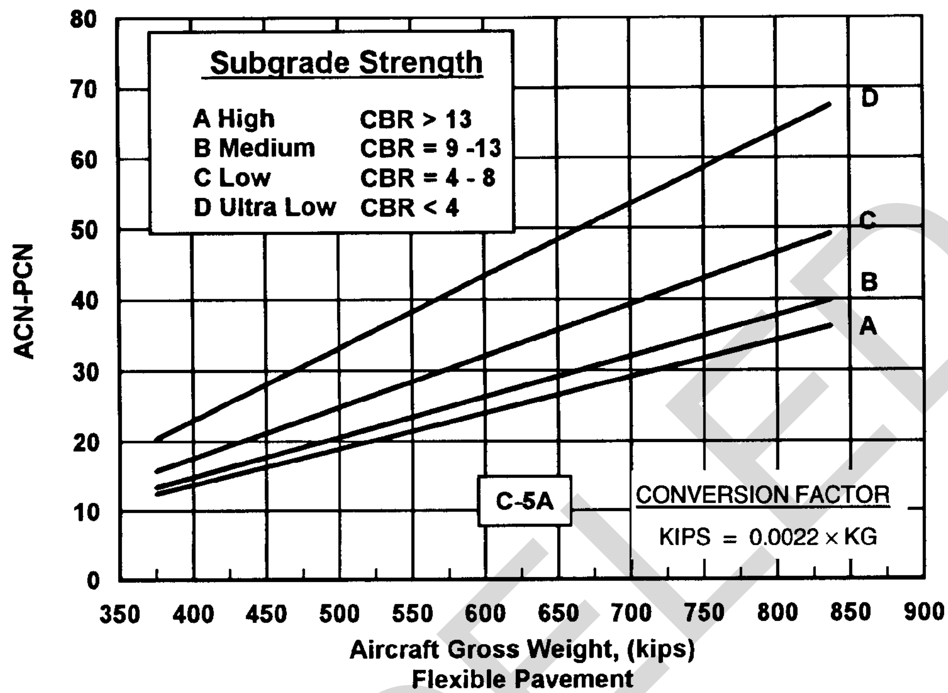


Figure 8-6. PCN curves for C-141



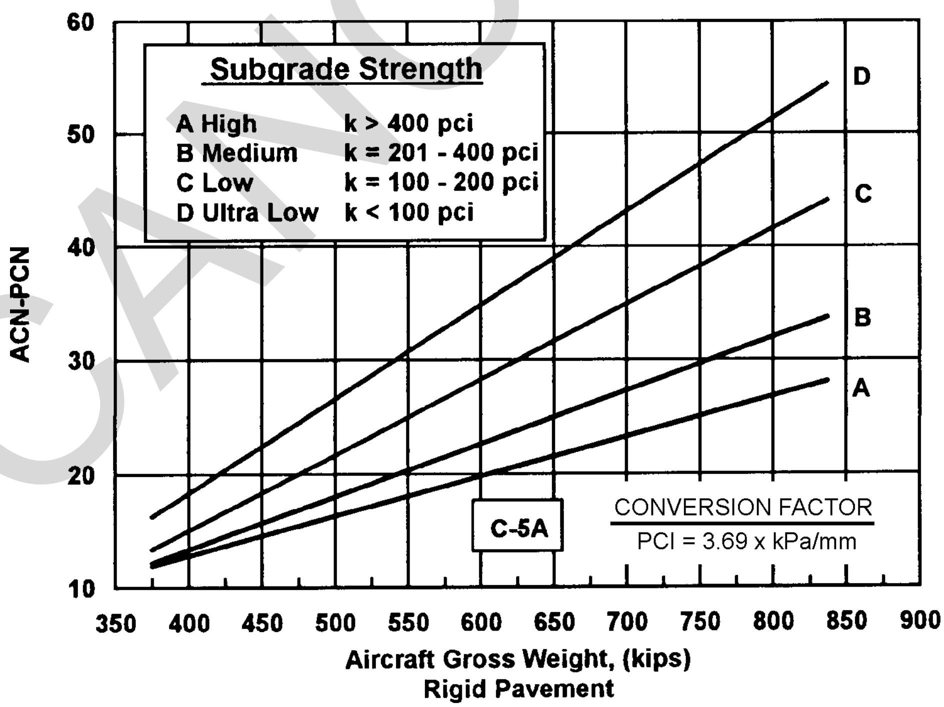


Figure 8-7. PCN curves for C-5

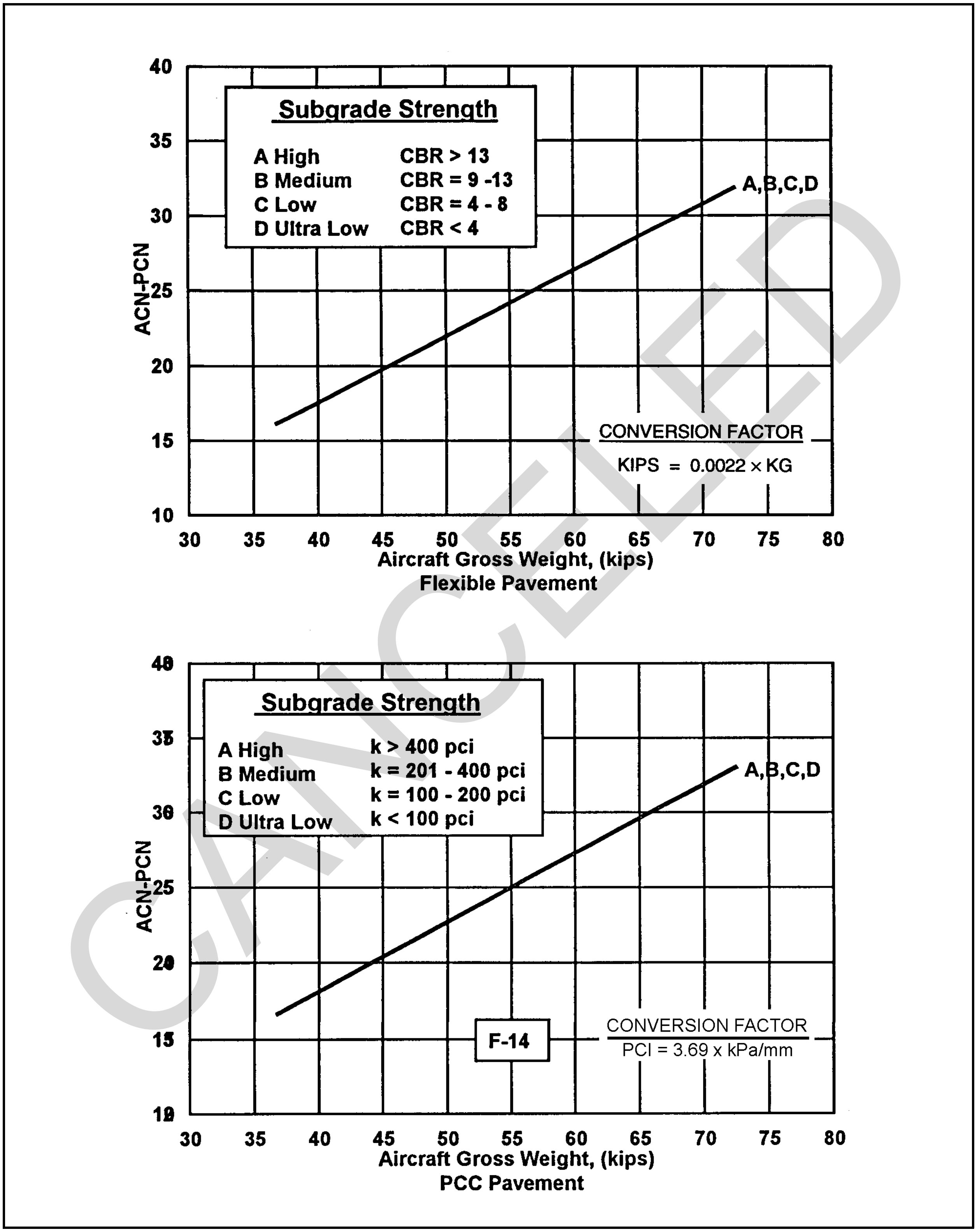


Figure 8-8. PCN curves for F-14

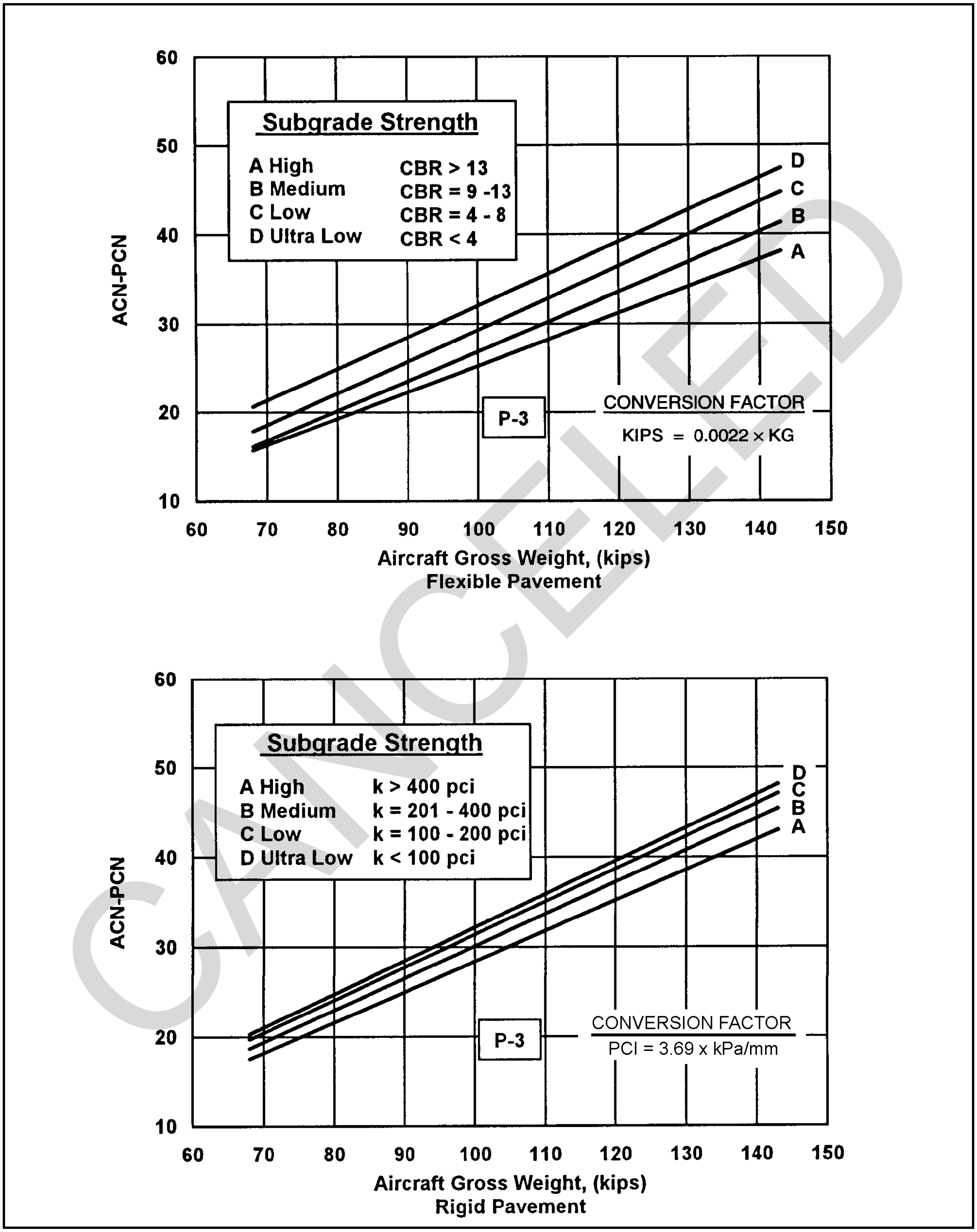
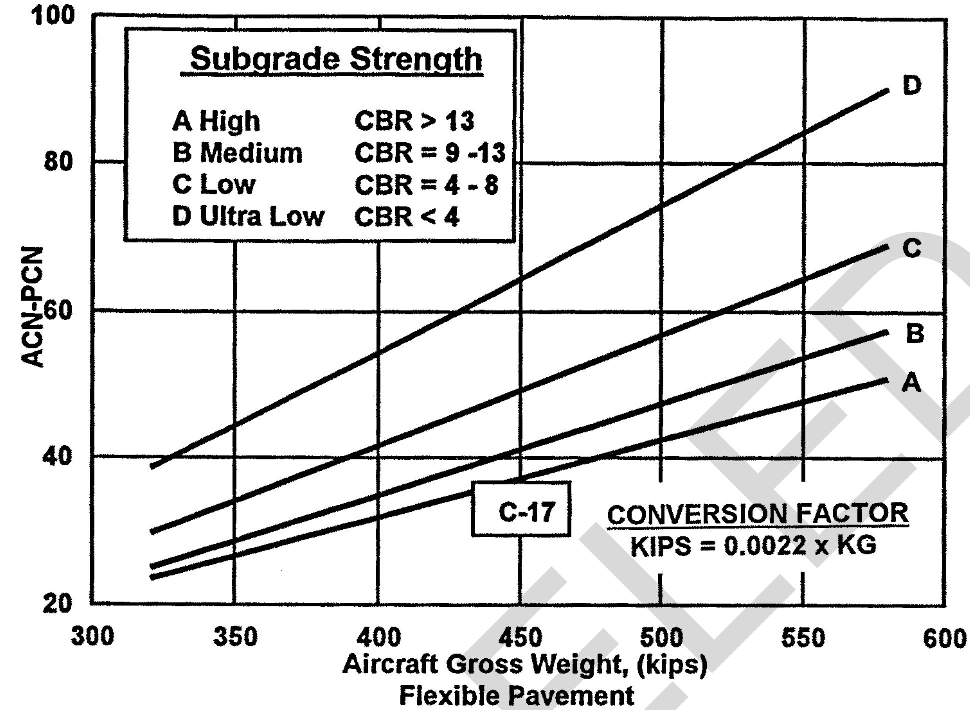


Figure 8-9. PCN curves for P-3



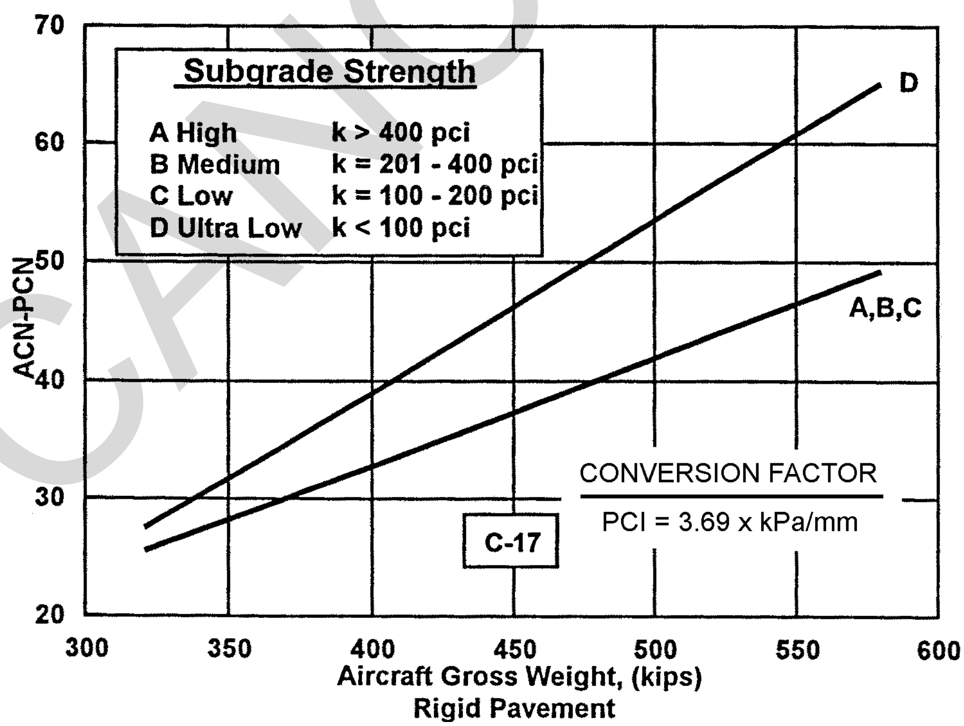


Figure 8-10. PCN curves for C-17

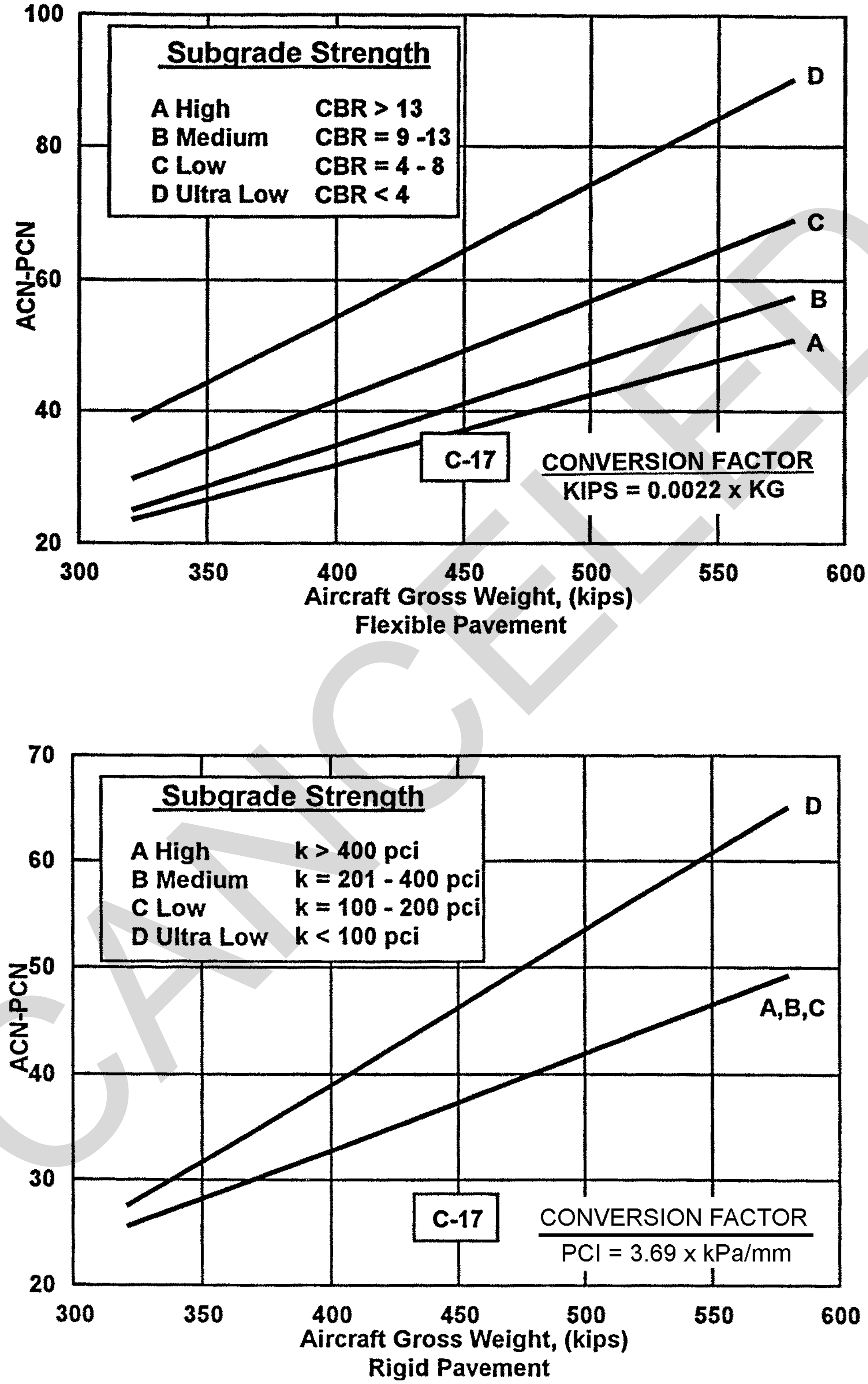


Figure 8-11. PCN curves for all Air Force pavements

CHAPTER 9

COMPUTER PROGRAMS FOR PAVEMENT EVALUATION

1. DEVELOPMENT OF COMPUTER PROGRAMS. Computer programs have been developed to aid in the evaluation of airfield pavements. One program titled Airfield Pavement Evaluation (APE) is for evaluating pavements and calculating PCN using data from direct sampling. Another program, titled Layered Elastic Evaluation Program (LEEP) is for evaluating pavements using data from nondestructive testing. There is also a program titled ACN which is for calculating Aircraft Classification Numbers. All programs have been run on IBM compatible microcomputers containing a minimum of 512K RAM. Installation of the LEEP Program is discussed in appendix C.

2. PROGRAM NAME. The computer programs names consist of alpha numeric identifiers. The letters APE for Airfield Pavement Evaluation, LEEP for Layer Elastic Evaluation Program, and ACN for Aircraft Classification Number represent the program name. The number, e.g., 1.0 or 1.1, represents the version number of the programs.

3. OBTAINING PROGRAMS. Current evaluation programs for rigid and flexible pavements may be obtained electronically from the following:

FTP Anonymous Site: pavement.wes.army.mil
World Wide WEB (WWW): http://pavement.wes.army.mil/pcase.html
or disks may be obtained from the Transportation Systems Center,
12565 West Center Road, Omaha, NE 68144-3869.

4. USING PROGRAMS. In developing the computer programs, an effort was made to provide a user friendly program requiring no external instructions for use of the programs. Aside from instructions for initiating execution, which is standard for any executable program, the user is led through the design procedure by a series of questions and informational screens. The input data required for pavement evaluation by the program is identical to the data required by evaluation criteria in this manual, and the results obtained from the program should be close to the results obtained from the evaluation curves. Because the computer program recalculates data and approximates certain empirical data, there may be some minor differences in results from the program and from the manual. If significant differences are found, contact the Transportation Systems Center.

APPENDIX A

REFERENCES

GOVERNMENT PUBLICATIONS

MIL-HDBK-1021/4	Rigid Pavement Design for Airfields
MIL-HDBK-1021/2	General Concepts for Airfield Pavement Design

Departments of the Army, the Navy, and Air Force

AFR 93-13	Air Force Pavement Evaluation Program
TI 825-01/AFM 32-1124 (I)/NAVFAC DM 21.10	Pavement Design for Airfields
TM 5-803-4	Planning of Army Aviation Facilities
TM 5-822-14/AFJMAN 32-1019	Soil Stabilization for Pavements
TM 5-825-2-1/AFM 88-6, Chap 2, Sec A	Flexible Pavement Design for Airfields, Elastic Layered Method
TM 5-825-3/AFM 88-6, Chap 3	Rigid Pavements for Airfields
TM 5-825-3-1/AFM 88-6, Chap 3	Rigid Pavement Design for Airfields, Elastic Layered Method
TM 5-826-4	Engineer and Deisgn Army Airfield/Heliport Pavement Reports

Corps of Engineers (COE)

CRD-C 649	(1995) Standard Test Method for Unit Weight, Marshall Stability and Flow of Bituminous Materials
CRD-C 650	(1995) 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 Aperature Procedure
S-73-56	Lateral Distribution of Aircraft Traffic

NONGOVERNMENT PUBLICATIONS

American Society for Testing and Materials (ASTM), 1916 Race Street, Philadelphia, PA 19103

C 39	(199_) Compressive Strengths of Cylindrical Concrete Specimens
C 42	(1994) Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
C 78	(1994) Flexural Strength of Concrete
C 127	(1988; R 1993) Specific Gravity and Absorption of Coarse Aggregates
C 128	(1993) Specific Gravity and Absorption of Fine Aggregate
C 136	(1996a) Sieve Analysis of Fine and Coarse Aggregate
C 496	(1996) Splitting Tensile Strength of Cylindrical Concrete Specimens
C 642	(1997) Density, Absorption, and Voids in Hardened Concrete
D 4	(1986); R 1993) Bitumen Content
D 5	(1995) Penetration of Bituminous Materials
D 366	(1995) Softening Point of Bitumen (Ring-and-Ball Apparatus)
D 113	(1986; R 1992) Ductility of Bituminous Materials
D 128	Analysis of Lubricating Grease
D 422	(1963; R 1990) Particle Size Analysis of Soils
D 854	(1992) Specific Gravity of Soils
D 1556	(1990; R 1996) Density and Unit Weight of Soil In-Place by the Sand Cone Method
D 1557	(1991) Laboratory Compaction Characteristics of Soil Using Modified Effort
D 1633	(1996) 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	(1995a) Recovery of Asphalt from Solution by Abson Method
D 2167	(1994) Density and Unit Weight of Soil In-Place by the Rubber-Balloon Method
D 2172	(1995) Quantitative Extraction of Bitumen from Bituminous Paving Mixtures
D 2216	(1992) Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil Aggregate Mixtures
D 2487	(1993) Classification of Soils for Engineering Purposes
D 2937	(1994) Density of Soil in Place by the Drive-Cylinder Method
D 4318	(1995a) Liquid Limit, Plastic Limit, and Plasticity Index of Soils
D 4694	(1987) Test Method for Deflections with a Falling Weight-Type Impulse Load Device
D 5340	(1993) Airport Pavement Condition Index Surveys

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

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 Pavements, Doc 9157-AN/901 Second edition, 1983.

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.

Samples or Test	Publication
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	ASTM C 136
Particle size analysis	ASTM D 422
Specific gravity of soils	ASTM D 854
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	ASTM D 4
Soils Sampling	Described below
Plate-bearing tests	CRD-C 655
Classification tests	ASTM D 2487
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	ASTM C 642
Absorption by concrete	ASTM C 642
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 TI 825-01/AFM 32-1124 (I)/NAVFAC DM 21.10.

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:

$$R = 1.02T + 1.45 \ (SI \ Units)$$
$$R = 1.02T + 210.5 \ (English \ Units) \qquad \text{(eq B-2)}$$

where

R = flexure strength in MPa (psi)

T = tensile splitting strength in MPa (psi)

APPENDIX C

HOW TO INSTALL EVALUATION COMPUTER PROGRAMS (LEEP)

C-1. DEVELOPMENT OF COMPUTER PROGRAMS. Computer programs have been developed to aid in the evaluation of airfield pavements. One program entitled Airfield Pavement Evaluation (APE) is for evaluating pavements and calculation the Pavement Classification number (PCN) using data from direct sampling. Another program, titled Layered elastic Evaluation Program (LEEP) is for evaluating pavements using data from nondestructive testing. Other programs include ACN that is used for calculating Aircraft classification Numbers, DCP that is used for reducing the data obtained from dynamic cone penetrometer test, and ECP that is used for reducing the data obtained from electronic cone penetrometer tests. All programs are designed for an IBM compatible PC, running Windows 95 or Windows NT.

C-2. OBTAINING PROGRAMS. Current evaluation programs for rigid and flexible pavements may be obtained electronically from the following:

- World Wide Web (WWW): hppt://pavement.wes.army.mil/ Click on the “Software” option.

C-3. INSTALLING/USING PROGRAMS. The APE and LEEP programs install just like any Microsoft Windows application. The user must download the program file and Click Start/Run and enter setup.exe. After the setup program has completed, the user will notice a new icon in the Start/Program tool bar named “LEEPWIN” or APEWIN”. Clicking this icon will begin the execution of the program. Once the program is executed, the help files will explain how to use the program.