Guidance Material

Airport Pavement

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GLOSSARY

Terms which are defined in the ICAO *Lexicon* Volume I (Doc 9110) are used in accordance with the meanings and usages given therein. A wide variety of terms is in use throughout the world to describe soils, construction materials, and components of airport pavements. As far as possible the terms used in this document are those which have the widest international use. However, for the convenience of the reader a short list of preferred terms and secondary terms which are considered to be their equivalent, and their definitions, is given below.

Preferred Term	Secondary Term	Definition
Aggregate		General term for the mineral fragments or particles which, through the agency of a suitable binder, can be combined into a solid mass, e.g., to form a pavement.
Aircraft Classification Number (ACN)		A number expressing the relative effect of an aircraft on a pavement for specified standard subgrade strength.
Asphaltic concrete	Bitumen concrete	A graded mixture of aggregate, and filler with asphalt or bitumen, placed hot or cold, and rolled.
Base Course	Base	The layer or layers of specified or selected material of designed thickness placed on a sub-base or subgrade to support surface course.
Bearing strength	Bearing capacity pavement strength	The measure of the ability of a pavement to sustain the applied load.
CBR	California Bearing Ratio	The bearing ratio of soil determined by comparing the penetration load of the soil to that of a standard material (see ASTM DI883). The method covers evaluation of the relative quality of subgrade soils but is applicable to sub-base and some base course materials.
Composite pavement		A pavement consisting of both flexible and rigid layers with and

		without separating granular layers.
Flexible Pavement		A pavement structure that maintains intimate contact with and distributes loads to the subgrade and depends on aggregate interlock, particle friction, and cohesion for stability.
Overlay		An additional surface course placed on existing pavement either with or without intermediate base or sub-base courses, usually to strengthen the pavement or restore the profile of the surface.
Pavement Classification Number (PCN)		A number expressing the bearing strength of a pavement for unrestricted operations.
Pavement Structure	Pavement	The combination of sub-base, base course, and surface course placed on a subgrade to support the traffic load and distribute it to the subgrade.
Portland cement concrete	Concrete	A mixture of graded aggregate with Portland cement and water.
Rigid Pavement		A pavement structure that distributes loads to the subgrade having as its surface course a Portland cement concrete slab of relatively high bending resistance.
Sub-base course	Sub-base	The layer or layers of specified selected material of designed thickness placed on a subgrade to support a base course.
Subgrade	Formation foundation	The upper part of the soil, natural or constructed, which supports the loads transmitted by the pavement.
Surface Course	Wearing course	The top course of a pavement structure.

FOREWORD

This supporting document on Runway Pavement contains guidance on the design of pavements including their characteristics and on evaluation and reporting of their bearing strength. The material included herein is closely associated with the specifications contained in CAR-14, Part I–Aerodromes Design and Operations.

The main objective of this guidance material is to assist proper design and construction of Runway Pavement with uniform application of those specifications for the safety and regularity of civil aviation.

Director General

Civil Aviation Authority of Nepal 2019
Babar Mahal, Kathmandu, Nepal

CHAPTER 1:-PROCEDURES FOR REPORTING AERODROME PAVEMENT STRENGTH

1.1 Procedure for pavements meant for heavy aircraft (ACN-PCN method)

1.1.1 Introduction

1.1.1.1 CAR-14, Part I, 2.6.2 specifies that the bearing strength of a pavement intended for aircraft of mass greater than 5700 kg shall be made available using the aircraft classification number - pavement classification number (ACN-PCN) method. To facilitate a proper understanding and usage of the CAN-PCN method the following material explains:

a) the concept of the method; and

b) how the ACNs of an aircraft are determined.

1.1.2 Concept of the ACN-PCN method

1.1.2.1 CAR-14, Part I defines ACN and PCN as follows:

ACN- A number expressing the relative effect of an aircraft on a pavement for specified standard subgrade strength.

PCN - A number expressing the bearing strength of a pavement for unrestricted operations.

At the outset, it needs to be noted that the ACN-PCN method is meant only for publication of pavement strength data in the Aeronautical Information Publications (AIPS). It is not intended for design or evaluation of pavements, nor does it contemplate the use of a specific method by the airport authority either for the design or evaluation of pavements In fact, the ACN-PCN method does permit States to use any design/evaluation method of their choice. To this end, the method shifts the emphasis from evaluation of pavements to evaluation of load rating of aircraft (ACN) and includes a standard procedure for evaluation of the load rating of aircraft. The strength of a pavement is reported under the method in terms of the load rating of the aircraft which the pavement can accept on an unrestricted basis. The airport authority can use any method of his choice to determine the load rating of his pavement. If, in the absence of technical evaluation, he chooses to go on the basis of the using aircraft experience, then he would compute the ACN of the most critical aircraft using one of the procedures described below, convert this figure into an equivalent PCN and publish it in the AIP as the load rating of his pavement. The PCN so reported would indicate that an aircraft with an ACN equal to or less than that figure can operate on the pavement subject to any limitation on the tire pressure.

1.1.2.2 The ACN-PCN method contemplates the reporting of pavement strengths on a continuous scale. The lower end of the scale is zero and there is no upper end. Additionally, the same scale is used to measure the load ratings of both aircraft and pavements.

1.1.2.3 To facilitate the use of the method, aircraft manufacturers will publish, in the documents detailing the characteristics of their aircraft, ACNs computed at two different masses: maximum apron mass, and a representative operating mass empty, both on rigid and flexible pavements and for the four standard subgrade strength categories. Nevertheless, for the sake of convenience CAR-14, Part I, Attachment A and Appendix 5 hereto include a table showing the ACNs of a number of aircraft. It is to be noted that the mass used in the ACN calculation is a "static" mass and that no allowance is made for an increase in loading through dynamic effects.

1.1.2.4 The ACN-PCN method also envisages the reporting of the following information in respect of each pavement:

a) pavement type;

b) subgrade category;

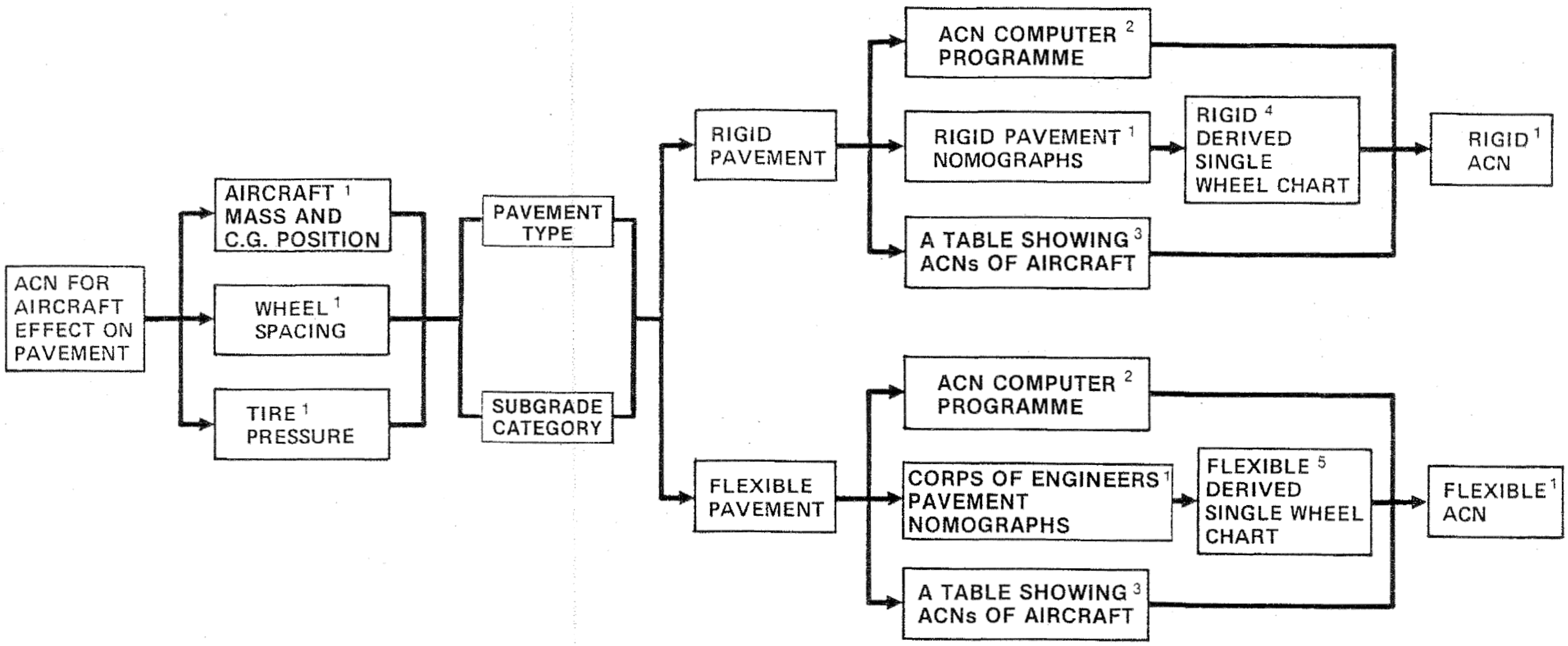
c) maximum tire pressure allowable; and

d) pavement evaluation method used.

The above data are primarily intended to enable aircraft operators to determine the permissible aircraft types and operating masses, and the aircraft manufacturers to ensure compatibility between airport pavements and aircraft under development. There is, however, no need to report the actual subgrade strength or the maximum tire pressure allowable. Consequently, the subgrade strengths and tire pressures normally encountered have been grouped into categories as indicated in 1.1.3.2 below. It would be sufficient if the airport authority identifies the categories appropriate to his pavement. (See also the examples included under CAR-14, Part I, 2.6.6.)

1.1.3 <u>How ACNs are determined</u>

1.1.3.1 The flow chart, below, briefly explains how the ACNs of aircraft are computed under the ACN-PCN method.



Relevant Documents

1. AIRPLANE CHARACTERISTICS FOR AIRPORT PLANNING (published by the aircraft manufacturer).
2. Appendix 2 of this manual.
3. Annex 14, Attachment B, Table B-1 and Appendix 5 of this manual
4. Figure 1-4 of this manual.
5. Figure 1-5 of this manual.

FLOW CHART

1.1.3.2 Standard values used in the method description of the various terms.

a) Subgrade category. In the ACN-PCN method eight standard subgrade values (i.e., four rigid pavement k values and four flexible pavement CBR values) are used, rather than a continuous scale of subgrade strengths. The grouping of subgrade with a standard value at the mid-range of each group is considered to be entirely adequate for reporting, The subgrade strength categories are identified as high, medium, low and ultra low and assigned the following numerical values:

Subgrade strength category

High strength; characterized by k* = 150 MN/m^3 and representing all k values above 120 MN/m^3 for rigid pavements, and by CBR 15 and representing all CBR values above 13 for flexible pavements.

Medium strength; characterized by k = 80 $MN/m^3$and representing a range in k of 60 to 12.0 MN/m^3 for rigid pavements, and by CBR 10 and representing a range in CBR of 8 to 13 for flexible pavements.

Low strength; characterized by k = 40 MN/m^3 and representing a range in k of 25 to 60 MN/m^3 for rigid pavements, and by CBR 6 and representing a range in CBR of 4 to 8 for flexible pavements.

Ultra low strength; characterized by k = 20 MN/m^3 and representing all k values below 25 MN/m^3 for rigid pavements, and by CBR = 3 and representing all CBR values below 4 for flexible pavements.

b) Concrete working stress for rigid pavements. For rigid pavements, a standard stress for reporting purposes is stipulated (σ = 2.75 MPa) only as a means of ensuring uniform reporting. The working stress to be used for the design and/or evaluation of pavements has no relationship to the standard stress for reporting.

c) Tire pressure. The results of pavement research and re-evaluation of old test results reaffirm that except for unusual pavement construction (i.e. flexible pavements with a thin asphaltic concrete cover or weak upper layers), tire pressure effects are secondary to load and wheel spacing, and may therefore be categorized in four groups for reporting purposes as: high, medium, low and very low and assigned the following numerical values:

Unlimited - No pressure limit

High - Pressure limited to 1.75 MPa

Medium - Pressure limited to 1.25MPa

Low - Pressure limited to 0.50 MPa

* Values determined using a 75 cm diameter plate.

d) Mathematically derived single wheel load: The concept of a mathematically derived single wheel load has been employed in the ACN-PCN method as a means to define the landing gear/pavement interaction without specifying pavement thickness as an ACN parameter. This is done by equating the thickness given by the mathematical model for an aircraft landing gear to the thickness for a single wheel at a standard tire pressure of 1.25 MPa. The single wheel load so obtained is then used without further reference to thickness; this is so because the essential significance is attached to the fact of having equal thicknesses, implying "same applied stress to the pavement", rather than the magnitude of, the thickness. The foregoing is in accord with the objective of the ACN-PCN method to evaluate the relative loading effect of an aircraft on a pavement.

e) Aircraft classification number (ACN). The ACN of an aircraft is numerically defined as two times the derived single wheel load, where the derived single wheel load is expressed in thousands of kilograms. As noted previously, the single wheel tire pressure is standardized at1.25 MPa. Additionally, the derived single wheel load is a function of the subgrade strength. The aircraft classification number (ACN) is defined only for the four subgrade categories (i.e., high, medium, low, and ultra low strength). The "two" (2) factor in the numerical definition of the ACN is used to achieve a suitable ACN vs. gross mass scale so that whole number ACNs may be used with reasonable accuracy.

f) Because an aircraft operates at various mass and centre of gravity conditions the following conventions have been used in ACN computations (see Figure 1-1).

1) The maximum ACN of an aircraft is calculated at the mass and e.g. that produces the highest main gear loading on the pavement, usually the maximum ramp mass and corresponding aft e.g. The aircraft tires are considered as inflated to the manufacturers 'recommendation for the condition;

2) Relative aircraft ACN charts and tables show the ACN as a function of aircraft gross mass with the aircraft e.g. at a constant value corresponding to the maximum ACN value (i.e., usually, the aft e.g., for max ramp mass) and at the max ramp mass tire pressure; and

3) Specific condition ACN values are those ACN values that are adjusted for the effects of tire pressure and/or e.g. location, at a specified gross mass for the aircraft.

1.1.3.3 Abbreviations

a) Aircraft parameters

MRGM - Maximum ramp gross mass in kilograms

b) Pavement and subgrade parameters

σ - Standard working stress for reporting, 2.75 MPa

t - Pavement thickness in centimeters

Thickness of slab for rigid pavements, or Total thickness of pavement structural system (surface to subgrade) for flexible pavements (see Figure 1-2).

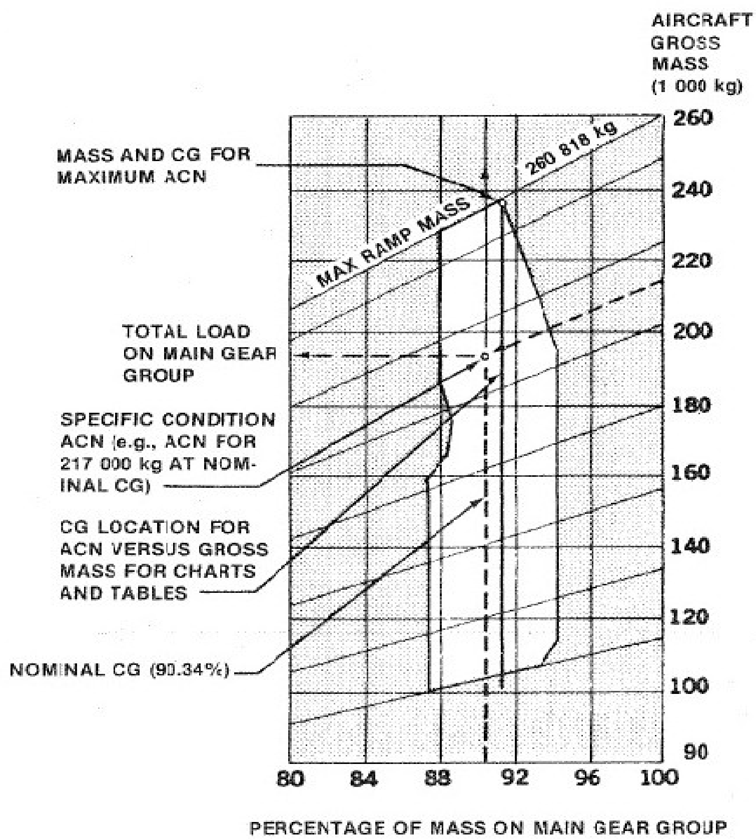


Figure 1-1 Landing gear loading on pavement Model DC-10 Series 30, 30CF, 40 and 40CF

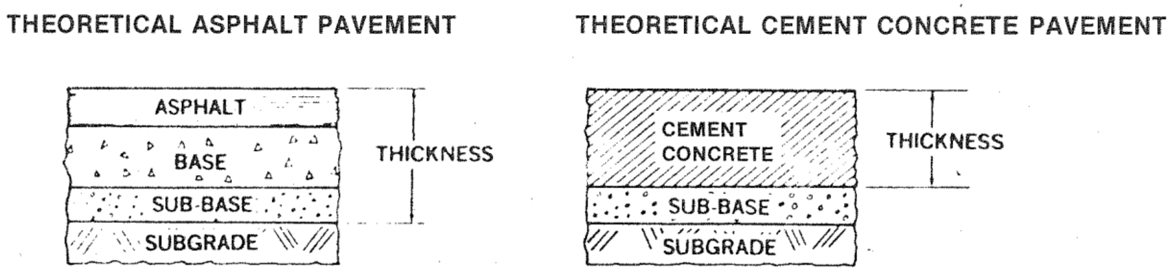


Figure 1-2

k - Westergaard's modulus of subgrade reaction in MN/m^3

ℓ - Westergaard's radius of relative stiffness in centimetres. This is computed using the following equation (see Figure 1-3).

$$\ell = \sqrt[4]{\frac{E\ t^3}{12\ (1 - \mu^2)k}}$$

E is modulus of elasticity

μ is Poisson's ratio (μ = 0.15)

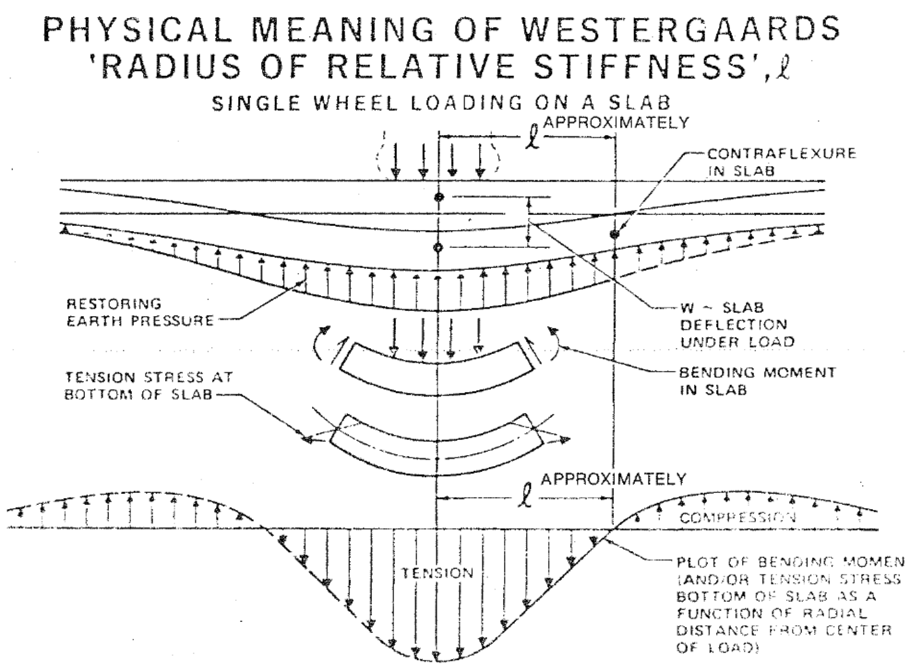


Figure 1-3

CBR - California Bearing Ratio in per cent
Tire Pressures
P_s - Tire pressure f or derived single wheel load - 1.25 MPa
P_q - Tire pressure for aircraft maximum ramp mass condition

1.1.3.4 Mathematical models. Two mathematical models are used in the ACN-PCN method: the Westergaard solution for a loaded elastic plate on a Winkler foundation (interior load case) for rigid pavements, and the Boussinesq solution for stress and displacements in a homogeneous isotropic elastic half-space under surface loading for flexible pavements. The use of these two, widely used, models permits the maximum correlation to world-wide pavement design methodologies, with a minimum need for pavement parameter values (i.e., only approximately subgrade k or CBR values are required).

1.1.3.5 Computer programmes. The two computer programmes developed using these mathematical models are reproduced in Appendix 2. The programme for evaluating aircraft on rigid pavements is based on the programme developed by Mr. R.G. Packard* of Portland Cement Association, Illinois, USA and that for evaluating aircraft on flexible pavements is based on the US Army Engineer Waterways Experiment Station Instruction Report S-77-1, entitled "Procedures for Development of CBR Deign Curves". It may, however, be noted that the aircraft classification tables included in CAR-14, Part I, Attachment A and in Appendix 5 of this Manual completely eliminate the need to use these programmes in respect of most of the aircraft currently in use.

1.1.3.6 Graphical procedures. Aircraft for which pavement thickness requirement charts have been published by the manufactures can also be evaluated using the graphical procedures described below.

1.1.3.7 Rigid pavements. This procedure uses the conversion chat shown in Figure 1-4 and the pavement thickness requirement charts published by the aircraft manufactures. The Portland Cement Association computer programme referred to in 1.1.3.5 was used in developing Figure 1-4. This figure related the derived single wheel load at a constant tire pressure of 1.25 MPa to a reference pavement thickness. It takes into account the four standard subgrade k values detailed in 1.1.3.2.a) above, and a standard concrete stress of 2.75MPa. The figure also includes an ACN scale which permits the ACN scale which permits the ACN to be read directly. The following steps are used to determine the ACN of an aircraft:

a) Using the pavement requirement chart published by the manufacturer obtain the reference thickness for the given aircraft mass, k value of the subgrade, and the standard concrete stress for reporting, i.e., 2.75 MPa;
b) Using the above reference thickness and Figure 1-4, obtain a derived single wheel load for the selected subgrade; and

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* Refer to document entitled "Design of Concrete Airport Pavement" by R.G. Packard, Portland Cement Association, Skokie, Illinois, 60076, dated 1973.

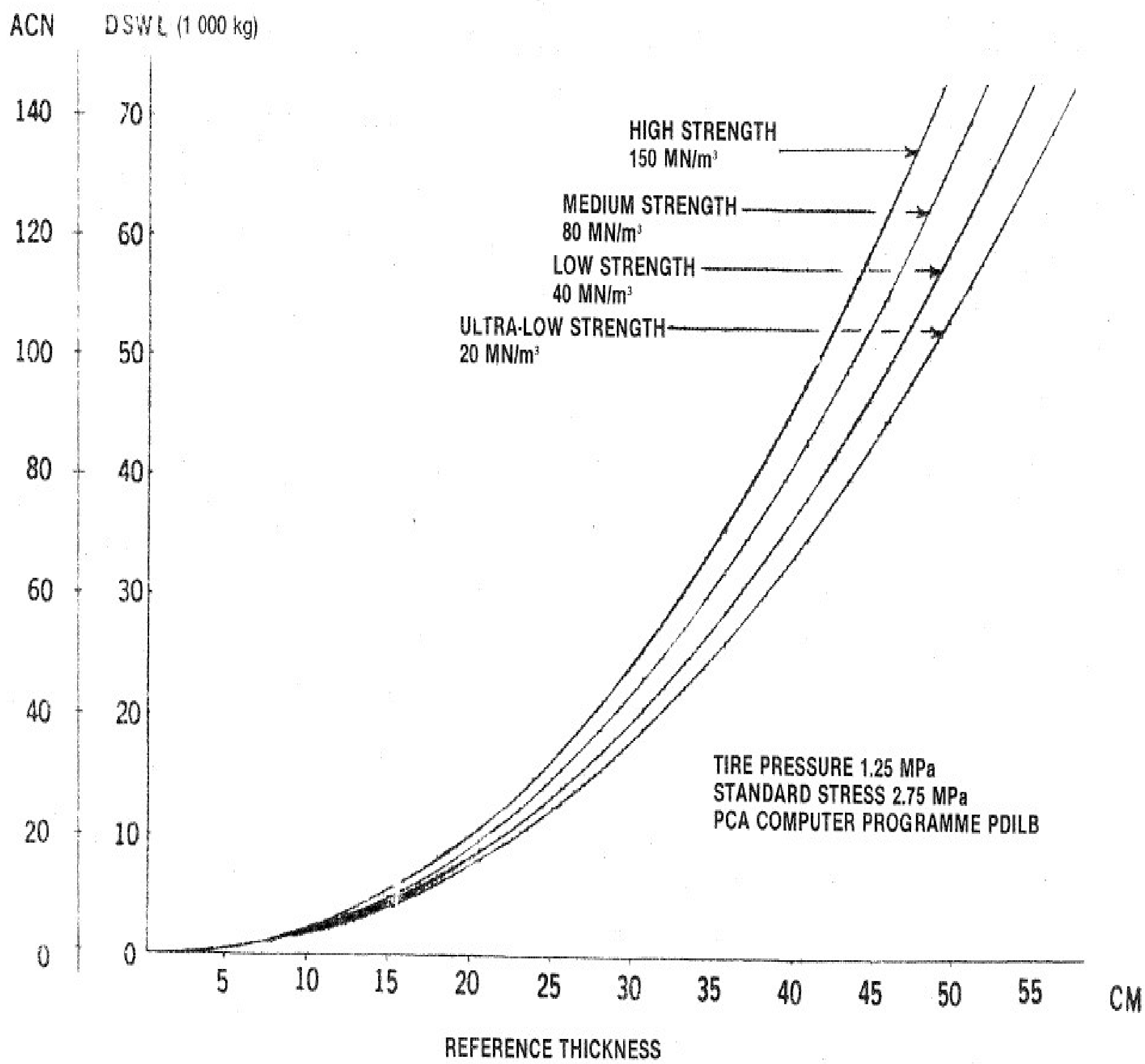


Figure 1-4. ACN Rigid Pavement Conversion Chart

c) The aircraft classification number, at the selected mass and subgrade k value, is two times the derived single wheel load in 1000 kg. Note that the ACN can also be read directly from the chart. Note further that tire pressure corrections are not needed when the above procedure is used

1.1.3.8 Flexible pavements. This procedure uses the conversion chart shown in Figure 1-5 and the pavement thickness requirement charts published by the aircraft manufacturers based on the United States Army Engineers CBR procedure. The former chart has been developed using the following expression:

$$t = \sqrt{\frac{DSWL}{C_1 \; CBR} - \frac{DSWL}{C_2 P_s}}$$

Where t = reference thickness in cm.

DSWL = a single wheel load with 1 .25 MPa tire pressure

P_s = 1. 25 MPa

CBR = standard subgrade (Note that the chart uses four standard values 3, 6, 10 and 15)

C_1 = 0.5695 C_2 =32.035

The reason for using the latter charts is to obtain the equivalency between the "group of landing gear wheels effect" to a derived single wheel load by means of Boussinesq Deflection Factors. The following steps are used to determine the ACN of an aircraft:

a) using the pavement requirement chart published by the manufacturer determine the reference thickness for the given aircraft mass, subgrade category, and 10000 coverages;

b) enter Figure 1-5 with the reference thickness determined in step a) and the CBR corresponding to the subgrade category and read the derived single wheel load; and

c) The ACN at the selected mass and subgrade category is two times the derived single wheel load in 1000 kg. Note that the ACN can also be read directly from the chart. Note further that tire pressure corrections are not needed when the above procedure is used.

1.1.3.9 Tire pressure adjustment to ACN. Aircraft normally have their tires inflated to the pressure corresponding to the maximum gross mass and maintain this pressure regardless of the variations in take-off masses. There are times, however, when operations at reduced masses and reduced tire pressures are productive and reduced ACNs need to be calculated. To do this for rigid pavements, a chart has been prepared by the use of the PCA computer programme PDILB and is given in Figure 1-6. The example included in the chart itself explains how the chart is used.

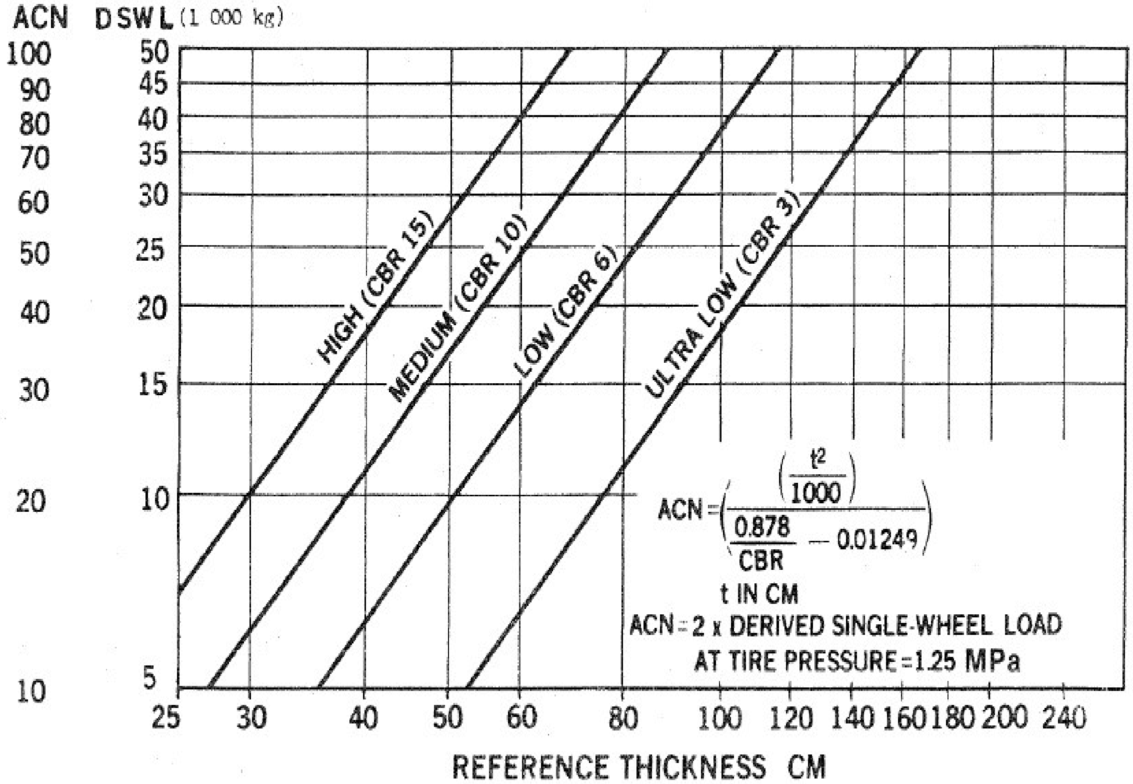


Figure 1-5. ACN Flexible Pavement Conversion Chart

1.1.3.10 For flexible pavements, the CBR $t = \sqrt{\dfrac{DSWL}{C_1 \quad CBR} - \dfrac{DSWL}{C_2 p_s}}$ equation

Was used to equate thickness and solve for the reduced pressure ACN in terms of the maximum tire pressure ACN at the reduced mass giving the following expression:

$$\text{ACN}_{\text{Reduced pressure}} = \text{ACN}_{\text{Maximum pressure}} \left[\frac{\dfrac{1}{C_1 \quad CBR} - \dfrac{1}{C_2 p_{red}}}{\dfrac{1}{C_1 \quad CBR} - \dfrac{1}{C_2 p_{max}}} \right]$$

(For values of C_1 and C_2 see 1.1.3.8)

1.1.3.11 Worked examples

Example 1: Find the ACN of B727-200 Standard at 78500 kg on a rigid pavement resting on a medium strength subgrade (i.e., k = 80 MN/m3). The tire pressure of the main wheels is 1.15MPa.

Solution: The ACN of the aircraft from the table in Appendix 5 of this Manual is 48.
It is also possible to determine the ACN of the aircraft using Figure 1-4 and the pavement requirement chart for the aircraft in Figure 1-7. This method involves the following operations:
a) from Figure 1-7 read the thickness of concrete needed for the aircraft mass of 78500 kg, the subgrade k value of 80 MN/m^3,and the standard concrete stress of 2.75 MPa as 31.75 cm; and
b) Enter Figure 1-4 with this thickness and read the ACN of the aircraft for the medium strength subgrade as48.

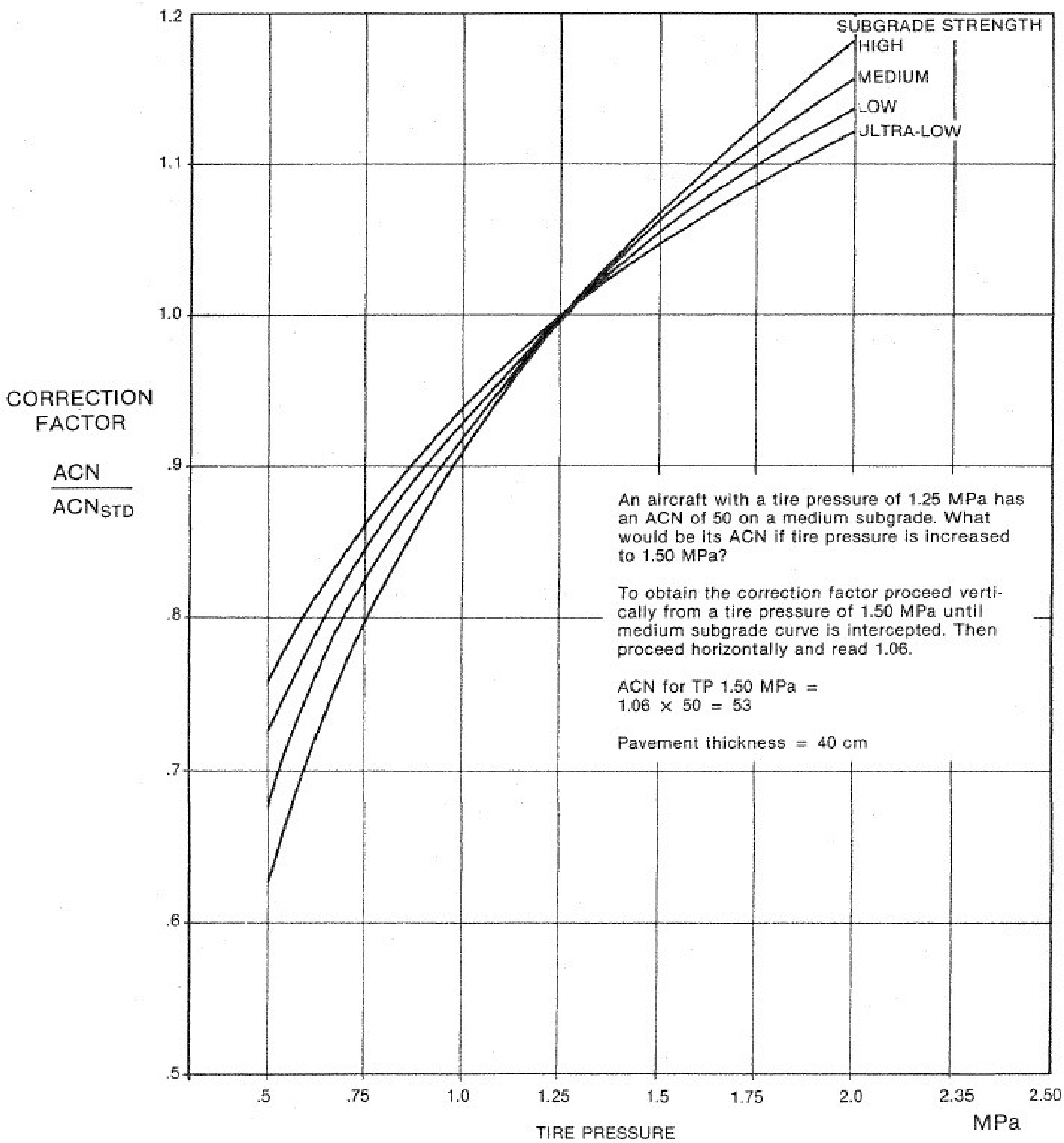


Figure 1-6. ACN tire pressure adjustment – rigid pavements only

Example 2: An AIP contains the following information related to a runway pavement:

PCN of the pavement = 80
Pavement type = rigid
Subgrade category = medium strength
Tire pressure limitation = none

Determine whether the pavement can accept the following aircraft at the indicated operating masses and tire pressures:

		Mass	Tire pressure
Airbus A 300 Model B2	at	142000 kg	1.23 MPa
B747-100	at	334751 kg	1.55 MPa
Concorde	at	185066 kg	1.26 MPa
DC-10-40	at	253105 kg	1.17 MPa

Solution: ACNs of these aircraft from Appendix 5 of this Manual are 44, 51, 71 and 53, respectively. Since the pavement in question has a PCN of 80, it can accept all of these aircraft.

Example 3: Find the ACN of DC-10-10 at 157400kg on a flexible pavement resting on a medium strength subgrade (CBR 10). The tire pressure of the main wheels is 1.28 MPa.

Solution: The ACN of the aircraft from Appendix 5 of this Manual is

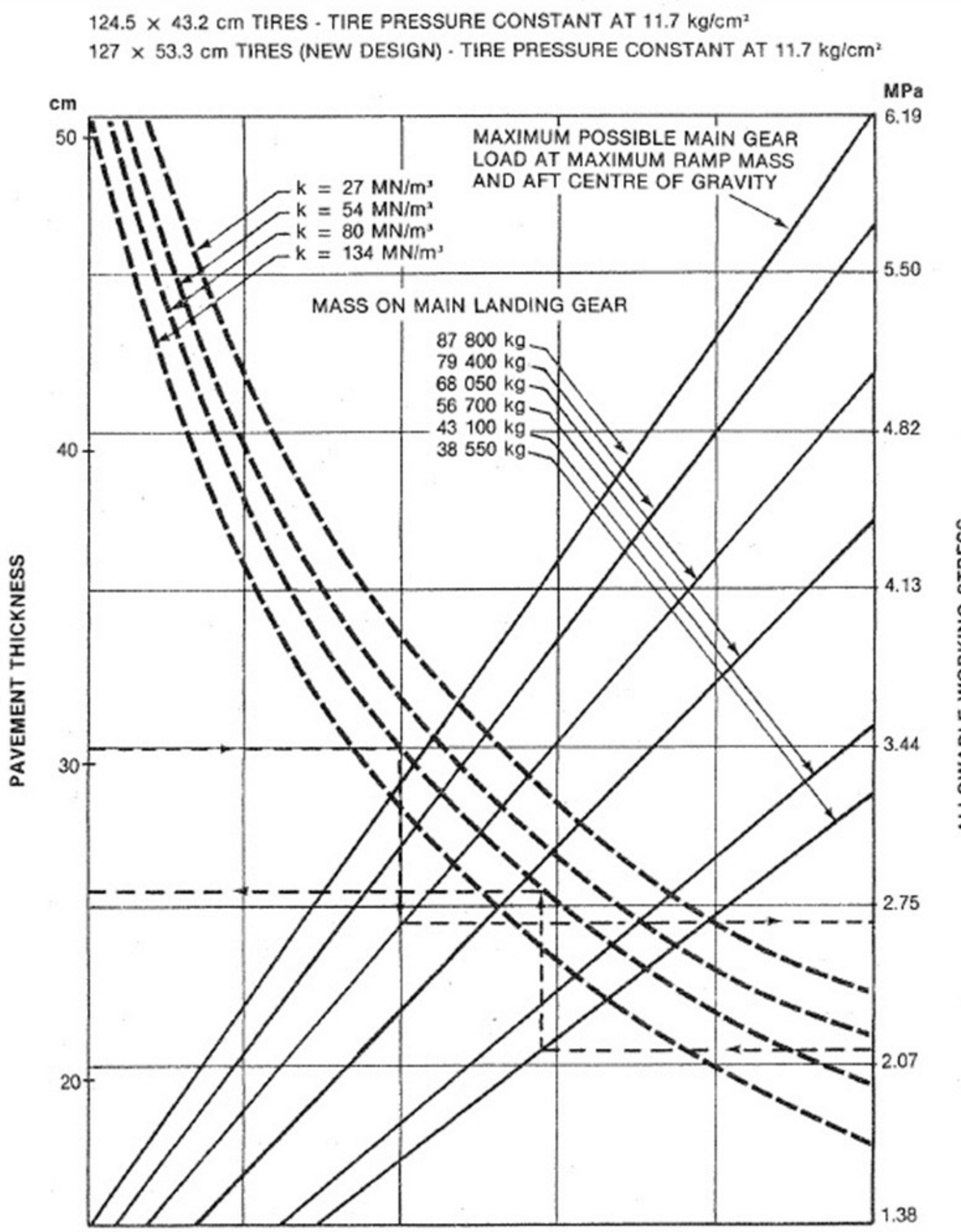
$$57 - \frac{(196\,406 - 157\,400)}{(196\,406 - 108\,940)} \times (57 - 27)$$

$$= 57 - \frac{39\,006}{87\,466} \times 30$$

$$= 57 - 13.4 = \underline{43.6} \text{ or } \underline{44}$$

It is also possible to determine the ACN of the aircraft using Figure 1-5 and the pavement requirement chart in Figure 1-8. This method involves the following operations:

a) from Figure 1-8 read the thickness of pavement needed for the aircraft mass of 157400 kg and the subgrade CBR of 10 as <u>57</u> cm; and
b) Enter Figure 1-5 with this thickness and read the ACN of aircraft for the subgrade CBR of 10 is <u>44</u>.



NOTE: THE VALUES OBTAINED BY USING THE MAXIMUM LOAD REFERENCE LINE AND ANY VALUE OF k ARE EXACT. FOR LOADS LESS THAN MAXIMUM, THE CURVES ARE EXACT FOR k = 80 BUT DEVIATE SLIGHTLY FOR OTHER VALUES OF k.

REFERENCES:
"DESIGN OF CONCRETE AIRPORT PAVEMENT" AND "COMPUTER PROGRAMME FOR AIRPORT PAVEMENT DESIGN - PROGAMME PDILB."
PORTLAND CEMENT ASSN.

RIGID PAVEMENT REQUIREMENTS—
PORTLAND CEMENT ASSOCIATION DESIGN METHOD
MODELS 727-100, -100C AT 77 200 kg; 727-200 STANDARD AT 78 500 kg,
ADVANCED 727-200 AT 89 800 kg AND 95 300 kg MAXIMUM RAMP MASS.

Figure 1-7

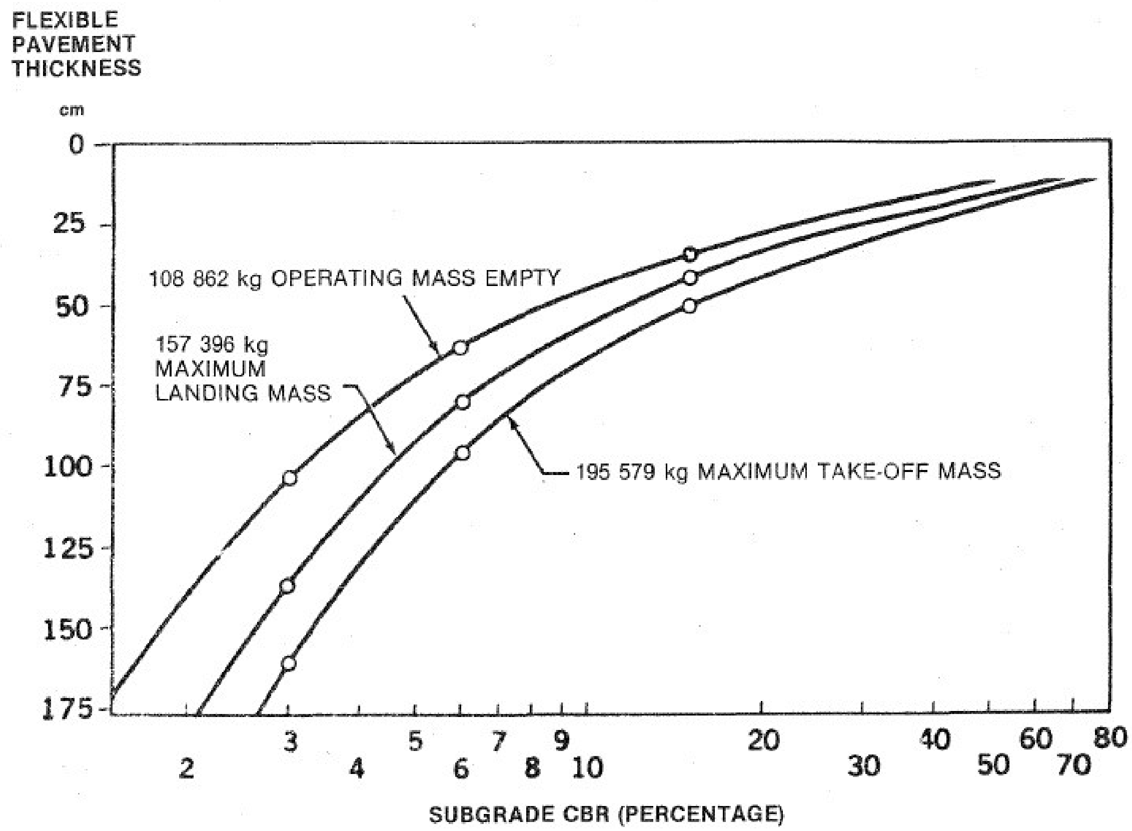


Figure 1-8. DC 10-10 Flexible Pavement Requirements 10000 Coverages aft c.g

1.2 Procedure for pavements meant for light aircraft

1.2.1 The ACN-PCN method described in 1.1 is not intended for reporting strength of pavements meant for light aircraft, i.e., those with mass less than 5700 kg. CAR-14, Part I specifies a simple procedure for such pavements. This procedure envisages the reporting of only two elements: max min allowable aircraft mass and maximum allowable tire pressure. It is important to note that the tire pressure categories of the ACN-PCN method (1.1.3.2, c) are not used for reporting maximum allowable tire pressure. Instead, actual tire pressure limits are reported as indicated in the following example:

Example: 4000 kg/0.50 MPa

CHAPTER 2: - GUIDANCE ON OVERLOAD OPERATIONS

2.1 Criteria suggested in CAR-14, Part I, Attachment A

2.1.1 Overloading of pavements can result either from loads too large or from a substantially increased application rate or both. Loads larger than the defined (design or evaluation) load shorten the design life whilst smaller loads extend it. With the exception of massive overloading, pavements in their structural behaviour are not subject to a particular limiting load above which they suddenly or catastrophically fail. Behaviour is such that a pavement can sustain a definable load for an expected number of repetitions during its design life. As a result, occasional minor overloading is acceptable, when expedient, with only limited loss in pavement life expectancy and relatively small acceleration of pavement deterioration. For those operations in which magnitude of overload and/or the frequency of use do not justify a detailed analysis the following criteria are suggested:

(a) For flexible pavements occasional movements by aircraft with ACN not exceeding 10 per cent above the reported PCN should not adversely affect the pavement;

(b) for rigid or composite pavements, in which a rigid pavement layer provides a primary element of the structure, occasional movements by aircraft with ACN not exceeding 5 per cent above the reported PCN should not adversely affect the pavement;

(c) if the pavement structure is unknown the 5 per cent limitation should apply; and

(d) The annual number of overload movements should not exceed approximately 5 per cent of the total annual aircraft movements.

2.1.2 Such overload movements should not normally be permitted on pavements exhibiting signs of distress or failure. Furthermore, overloading should be avoided during any periods of thaw following frost penetration or when the strength of the pavement or its subgrade could be weakened by water. Where overload operations are conducted, the appropriate authority should review the relevant pavement condition regularly and should also review the criteria for overload operations periodically since excessive repetition of overloads can cause severe shortening of pavement life or require major rehabilitation of pavement.

CHAPTER 3: - EVALUATION OF PAVEMENTS

3.1. General

3.1.1. The purpose of this chapter is to present guidance on the evaluation of pavements to those responsible for evaluating and reporting pavement bearing strength. Recognizing that responsible individuals may range from experienced pavement engineers to airfield managers not enjoying the direct staff support of pavement behavior experts, information will be included which attempts to serve the various levels of need.

3.2. Elements of pavement evaluation

3.2.1. The behaviour of any pavement depends upon the native materials of the site, which after leveling and preparation is called the subgrade, its structure including all layers up through the surfacing, and the mass and frequency of using aircraft. Each of these three elements must be considered when evaluating a pavement.

3.2.2. <u>The subgrade</u>. The subgrade is the layer of material immediately below the pavement structure which is prepared during construction to support the loads transmitted by the pavement. It is prepared by stripping vegetation, leveling or bringing to planned grade by cut and fill operations, and compacting to the needed density. Strength of the subgrade is a significant element and this must be characterized for evaluation or design of a pavement facility or for each section of a facility evaluated or designed separately, Soil strength and therefore subgrade strength is very dependent on soil moisture and must be evaluated for the condition it is expected to attain *in situ* beneath the pavement structure. Except in cases with high water tables, unusual drainage, or extremely porous or cracked pavement conditions soil moisture will tend to stabilize under wide pavements to something above 90 per cent of full saturation. Seasonal variation (excepting frost penetration of susceptible materials) is normally small to none and higher soil moisture conditions are possible even in quite arid areas. Because materials can vary widely in type the subgrade strength established for a particular pavement may fall anywhere within the range indicated by the four subgrade strength categories used in the ACN-PCN method, See Chapter 1 of this Manual and CAR-14, Part I, Chapter 2.

3.2.3. <u>The pavement structure.</u> The terms "rigid" and “flexible" have come into use for identification of the two principal types of pavements. The terms attempt to characterize the response of each type to loading. The primary element of a rigid pavement is a layer or slab of Portland cement concrete (PCC), plain or reinforced in any of several ways. It is often underlain by a granular layer which contributes to the structure both directly and by facilitating the drainage of water. A rigid pavement responds "stiffly" to surface loads and distributes the loads by bending or beam action to wide areas of the subgrade. The strength of the pavement depends on the thickness and strength of the PCC and any underlying layers above the subgrade. The pavement must be adequate to distribute surface loads so that the pressure on the subgrade does not exceed its evaluated strength.

A flexible pavement consists of a series of layers increasing in strength from the subgrade to the surface layer. A series such as select material, lower sub-base, sub-base, base and wearing course is commonly used. However, the lower layers may not be present in a particular pavement. The pavements meant for heavy aircraft usually have a bituminous bound wearing course. A flexible pavement yields more under surface loading merely accomplishing a widening of the 1oaded area and consequent reduction of pressure layer by layer. At each level from the surface to subgrade, the layers must have strength sufficient to tolerate the pressures at their level. The pavement thus depends on its thickness over the subgrade for reduction of the surface pressure to a value which the subgrade can accept. A flexible pavement must also have thickness of structure above each layer to reduce the pressure to a level acceptable by the layer. In addition, the wearing course must be sufficient in strength to accept without distress tire pressures of using aircraft.

3.2.4. Aircraft loading. The aircraft mass is transmitted to the pavement through the undercarriage of the aircraft. The number of wheels, their spacing, tire pressure and size determine the distribution of aircraft load to the pavement. In general, the pavement must be strong enough to support the loads applied by the individual wheels, not only at the surface and the subgrade but also at intermediate levels. For the closely spaced wheels of dual and dual-tandem legs and even for adjacent legs of aircraft with complex undercarriages the effects of distributed loads from adjacent wheels overlap at the subgrade (and intermediate) level. In such cases, the effective pressures are those combined from two or more wheels and must be attenuated sufficiently by the pavement structure. Since the distribution of load by a pavement structure is over a much narrower area on a high strength subgrade than on a low strength subgrade, the combining effects of adjacent wheels is much less for pavements on high strength than on low strength subgrades. This is the reason why the relative effects of two aircraft types are not the same for pavements of equivalent design strength, and this is the basis for reporting pavement bearing strength by sub-grade strength category. Within subgrade strength category the relative effects of two aircraft types on pavements can be uniquely stated with good accuracy.

3.2.5. Load repetitions and composition of traffic. It is not sufficient to consider the magnitude of loading alone. There is a fatigue or repetitions of load factor which should also be considered. Thus magnitude and repetitions must be treated together, and a pavement which is designed to support one magnitude of load at a defined number of repetitions can support a larger load at fewer repetitions and a smaller load for a greater number of repetitions. It is thus possible to establish the effect of one aircraft mass in terms of equivalent repetitions of another aircraft mass (and type).Application of this concept permits the determination of a single (selected) magnitude of load and repetitions level to represent the effect of the mixture of aircraft using a pavement.

3.2.6. Pavement condition survey. A particularly important adjunct to or part of evaluation is a careful condition survey. The pavement should be closely examined for evidences of deterioration, movement, or change of any kind. Any observable pavement change provides information on effects of traffic or the environment on the pavement.

Observable effects of traffic along with an assessment of the magnitude and composition of that traffic can provide an excellent basis for defining the bearing capacity of a pavement.

3.3. Elements of the ACN-PCN method

3.3.1. Pavement classification number. The pavement classification number (PCN)is an index rating (1/500th) of the mass which an evaluation shows can be borne by the pavement when applied by a standard (l.25 MPa tire pressure)single-wheel. The PCN rating established for a pavement indicates that the pavement is capable of supporting aircraft having an ACN (aircraft classification number) of equal or lower magnitude. The ACN for comparison to the PCN must be the aircraft ACN established for the particular pavement type and subgrade category of the rated pavement as well as for the particular aircraft mass and characteristics.

3.3.2. Pavement type. For purposes of reporting pavement strength, pavements must be classified as either rigid or flexible. A rigid pavement is that employing a Portland cement concrete (PCC) slab whether plain, reinforced, or prestressed and with or without intermediate layers between the slab and subgrade. A flexible pavement is that consisting of a series of layers increasing in strength from the subgrade to the wearing surface. Composite pavements resulting from a PCC overlay on a flexible pavement or an asphaltic concrete overlay on a rigid pavement or those incorporating chemically (cement) stabilized layers of particularly good integrity require care in classification. If the "rigid" element remains the predominant structural element of the pavement and is not severely distressed by closely spaced cracking the pavement should be classified as rigid. Otherwise the flexible classification should apply. Where classification remains doubtful, designation as flexible pavement will generally be conservative. Unpaved surfaces (compacted earth, gravel, laterite, coral, etc.) should be classified as flexible for reporting. Similarly, pavements built with bricks, or blocks should be classified as flexible. Large pre-cast slabs which require crane handling for placement can be classified as rigid when used in pavements. Pavements covered with landing mat and membrane surfaced pavements should be classified as flexible.

3.3.3. Subgrade Category. Since the effectiveness of aircraft undercarriages using multiple-wheels is greater on pavements founded on strong subgrades compared to those on weak subgrades, the problem of reporting bearing strength is complicated. To simplify the reporting and permit the use of index values for pavement and aircraft classification numbers (PCN and ACN) the ACN-PCN method uses four subgrade strength categories. These are termed: high, medium, low and ultra low with prescribed ranges for the categories. It follows that for a reported evaluation (PCN) to be useful the subgrade category to which the subgrade of the reported pavement belongs must be established and reported. Normally subgrade strength will have been evaluated in connection with original design of a pavement or later rehabilitation or strengthening. Where this information is not available the subgrade strength should be determined as part of pavement evaluation. Subgrade strength evaluation should be based on testing wherever possible. Where evaluation based on testing is not feasible a representative subgrade strength category must be selected based on soil characteristics, soil classification, local experience, or judgement. Commonly one subgrade category may be appropriate for an aerodrome. However, where pavement facilities are scattered over a large area and soil conditions differ from location to location several categories may apply and should be assessed and so reported. The subgrade strength evaluated must be that *in situ* beneath the pavement. The subgrade beneath an aerodrome pavement will normally reach and retain a fairly constant moisture and strength despite seasonal variations. However, in the case of severely cracked surfacing, porous paving, high ground water, or poor local drainage, the subgrade strength can reduce substantially during wet periods. Gravel and compact soil surfaces will be especially subject to moisture change. And in areas of seasonal frost, a lower reduced subgrade strength can be expected during the thaw period where frost susceptible materials are involved.

3.3.4. Tire pressure category. Directly at the surface the tire contact pressure is the most critical element of loading with little relation to other aspects of pavement strength. This is the reason for reporting permissible tire pressure in terms of tire pressure categories. Except for rare cases of spalling joints and unusual surface deficiencies, rigid pavements do not require tire pressure restrictions. However, pavements categorized as rigid which have overlays of flexible or bituminous construction must be treated as flexible pavements for reporting permissible tire pressure. Flexible pavements which are classified in the highest tire pressure category must be of very good quality and integrity, while those classified in the lowest category need only be capable of accepting casual highway traffic. While tests of bituminous mixes and extracted cores for quality of the bituminous surfacing will be most helpful in selecting the tire pressure category, no specific relations have been developed between test behaviour and acceptable tire pressure. It will usually be adequate, except where limitations are obvious, to establish category limits only when experience with high tire pressures indicates pavement distress.

3.3.5. Evaluation method. Wherever possible reported pavement strength should be based on a "technical evaluation". Commonly, evaluation is an inversion of a design method. Design begins with the aircraft loading to be sustained and the subgrade strength resulting from preparation of the local soil, then provides the necessary thicknesses and quality of materials for the needed pavement structure. Evaluation inverts this process. It begins with the existing subgrade strength, finds thickness and quality of each component of the pavement structure, and uses a design procedure pattern to determine the aircraft loading which the pavement can support. Where available the design, testing, and construction record data for the subgrade and components of the pavement structure can often be used to make the evaluation. Or, test pits can be opened to determine the thicknesses of layers, their strengths, and subgrade strength for the purpose of evaluation. A technical evaluation also can be made based on measurement of the response of pavement to load. Deflexion of a pavement under static plate or tire load can be used to predict its behaviour. Also there are various devices for applying dynamic loads to a pavement, observing its response, and using this to predict its behaviour. When for economic or other reasons a technical evaluation is not feasible, evaluation can be based on experience with "using aircraft". A pavement satisfactorily supporting aircraft using it can accept other aircraft if they are no more demanding than the using aircraft. This can be the basis for an evaluation.

3.3.6. Pavements for light aircraft. Light aircraft are those having a mass of 5700kg or less. These aircraft have pavement requirements less than that of many highway trucks, Technical evaluations of those pavements can, of course, be made, but an evaluation based on using aircraft is satisfactory. It is worth noting that at some airports service vehicles such as fire trucks, fuel trucks, or snow ploughs may be more critical than aircraft. Since nearly all light aircraft have single-wheel undercarriage legs there is no need for reporting subgrade categories. However, since some helicopters and military trainer aeroplanes within this mass range have quite high tire pressures limited quality pavements may need to have tire pressure limits established.

3.4. Assessing the magnitude and composition of traffic

3.4.1. General. Pavement bearing strength evaluations should address not merely an allowable load but a repetitions use level for that load. A pavement which can sustain many repetitions of one load can sustain a larger load but for fewer repetitions. Observable effects of traffic, even those involving careful measurements or on samples in controlled laboratory tests, unfortunately do not (unless Physical damage is apparent*) permit a determination of the portion of pavement's repetitions life that has been used or, conversely, is remaining. Thus an evaluation leading to bearing capacity determination is an assessment of pavement's total expected repetitions (traffic/load) life. Any projection of remaining useful life of the pavement will depend on a determination of all traffic sustained since construction or reconstruction.

* In the case of evident physical damage a pavement will already be in the last stages of its useful life.

3.4.2. Mixed loadings. Normally, it will be necessary to consider a mixture of loadings at their respective repetitions use levels. There is a strong tendency to rate pavement: bearing strength in terms of some selected loading for the allowable repetitions use level, and to rate each loading applied to a pavement in terms of its equivalent number of this basic loading. To do this, a relation is first established between loading and repetitions to produce failure. Such relations are variously established using combinations of theory or design methods and experience behaviour patterns or laboratory fatigue curves for the principal structural element of the pavement. Obviously, not all relations are the same,* but the repetitions parameter is not subtly effective. It needs only to be established in general magnitude and not in specific value. Thus fairly large variations can exist in the loading-repetitions relation without serious differences in evaluation resulting.

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*See Chapter 4, Figure 4-29 (French practice) and 4.4.12.1 (United States practice).

3.4.3. Using the curve for loading versus repetitions to failure, the failure repetitions for each loading can be determined and compared to that for the basic selected loading. From these comparisons, the equivalent number of the basic selected loading for single applications of any loading are determined, i.e., factors greater than 1 for larger loadings and less than 1 for smaller loadings. An explanatory example of this process follows:

a) Relate loading to failure repetitions, as illustrated in Figure 3-1;

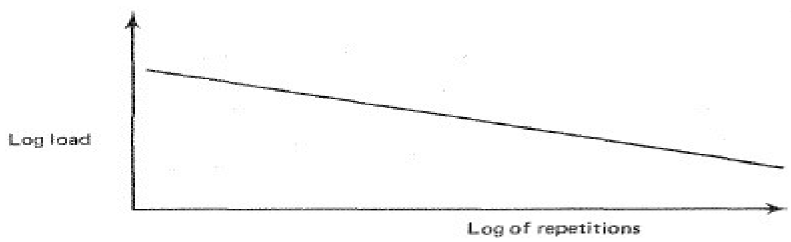

Figure 3-1

b) For selected loads L, read repetitions r from curve

$L_1 - r_1$

$L_2 - r_2$

$L_3 - r_3$

$L_4 - r_4$

c) choose L_3 as the basic load; and
d) compute equivalent repetitions factor f for each load

Load	Equivalent Repetitions Factor	
L_1	$f_1 = \frac{r_3}{r_1}$	(a value less than 1)
L_2	$f_2 = \frac{r_3}{r_2}$	(a value less than 1)
L_3	$f_3 = \frac{r_3}{r_3} = 1$	
L_4	$f_4 = \frac{r_3}{r_4}$	(a value greater than 1)

By use of these factors, the accumulated effect of any combination of loads experienced or contemplated can be compared to the bearing strength evaluation in terms of a selected loading at its evaluated allowable repetitions use level.

3.5. Techniques for “using aircraft” evaluation

3.5.1. While technical evaluation should be accomplished wherever possible, it is recognized that financial and circumstantial constraints will occasionally prevent it. Since it is most important to have completely reported bearing strength information and since the using aircraft evaluation is reasonably direct and readily comprehensible it is being presented first.

3.5.2. Heaviest using aircraft. A pavement satisfactorily sustaining its using traffic can be considered capable of supporting the heaviest aircraft regularly using it, and any other aircraft which has no greater pavement strength requirements. Thus to begin an evaluation based on using aircraft, the types and masses of aircraft and number of times each operates in a given period must be examined. Emphasis here should be on the heaviest aircraft regularly using the pavement. Support of a particularly heavy load, but only, does not necessarily establish a capability to support equivalent loads on a regular repetitive basis (see 3.4).

3.5.3. Pavement condition and behaviour. There must next be a careful examination of what effect the traffic of using aircraft is having on the pavement. The condition of the pavement in relation to any cracking, distortion or wear, and the experience with needed maintenance are of first importance. Age must be considered since overload effects on a new pavement may not yet be evident while some accumulated indications of distress may normally be evident in a very old pavement. In general, however, a pavement in good condition can be considered to be satisfactorily carrying the using traffic, while indications of advancing distress show the pavement is being overloaded. The Condition examination should take note of relative pavement behavior in areas of intense versus low usage such as in and out of wheel paths or most and least used taxiways, zones subject to maximum braking, e.g., taxiway turn-off, etc. Note should also be taken of behaviour of any known or observable weak or critical areas such as low points of pavement grade, old stream crossings, pipe crossings where initial compaction was poor, structurally weak sections, etc. These will help to predict the rate of deterioration under extant traffic and thereby indicate the degree of overloading or of under loading. The condition examinations should also focus on any damage resulting from tire pressures of using aircraft and the need for tire pressure limitations.

3.5.4. Reference aircraft. Study of the types and masses of aircraft will indicate those which must be of concern in establishing a reference aircraft and the condition survey findings will indicate whether the load of the reference aircraft should be less than that being applied or might be somewhat greater. Since load distribution to the subgrade depends somewhat on pavement type and subgrade strength, the particular reference aircraft and its mass cannot be selected until those elements of the ACN-PCN method which are reported in addition to the PCN have been established (see 3. 3.2 and 3. 3. 3)

3.5.5. Determination of the pavement type, subgrade strength and tire pressure categories. The pavement type must be established as rigid or flexible. If the pavement includes a Portland cement concrete slab as the primary structural element it should be classified as rigid even though it may have a bituminous overlay resurfacing (see 3.3.2). If the pavement includes no such load-distributing slab it should be classified as flexible.

3.5.6. The subgrade category must be determined as high, medium, low, or ultra low strength. If CBR or plate bearing test data are available for the subgrade these can be used directly to select the subgrade category. Such data, however, must represent *in situ* subgrade conditions. Similar data from any surrounding structures on the same type of soil and in similar topography can also be used. Soil strength data in almost any other form can be used to project an equivalent CBR or modules of subgrade reaction k for use in selecting the subgrade category. Information on subgrade soil strength may be obtainable form local road or highways agencies or local agricultural agencies. A direct, though somewhat crude or appropriate, determination of subgrade strength can be made from classification* of the subgrade material and reference to any of many published correlations such as that shown in Figure 3-2. (Also see 3.3.3 and 3.2.2.)

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*ASTM D2487, D3282, and D2488.

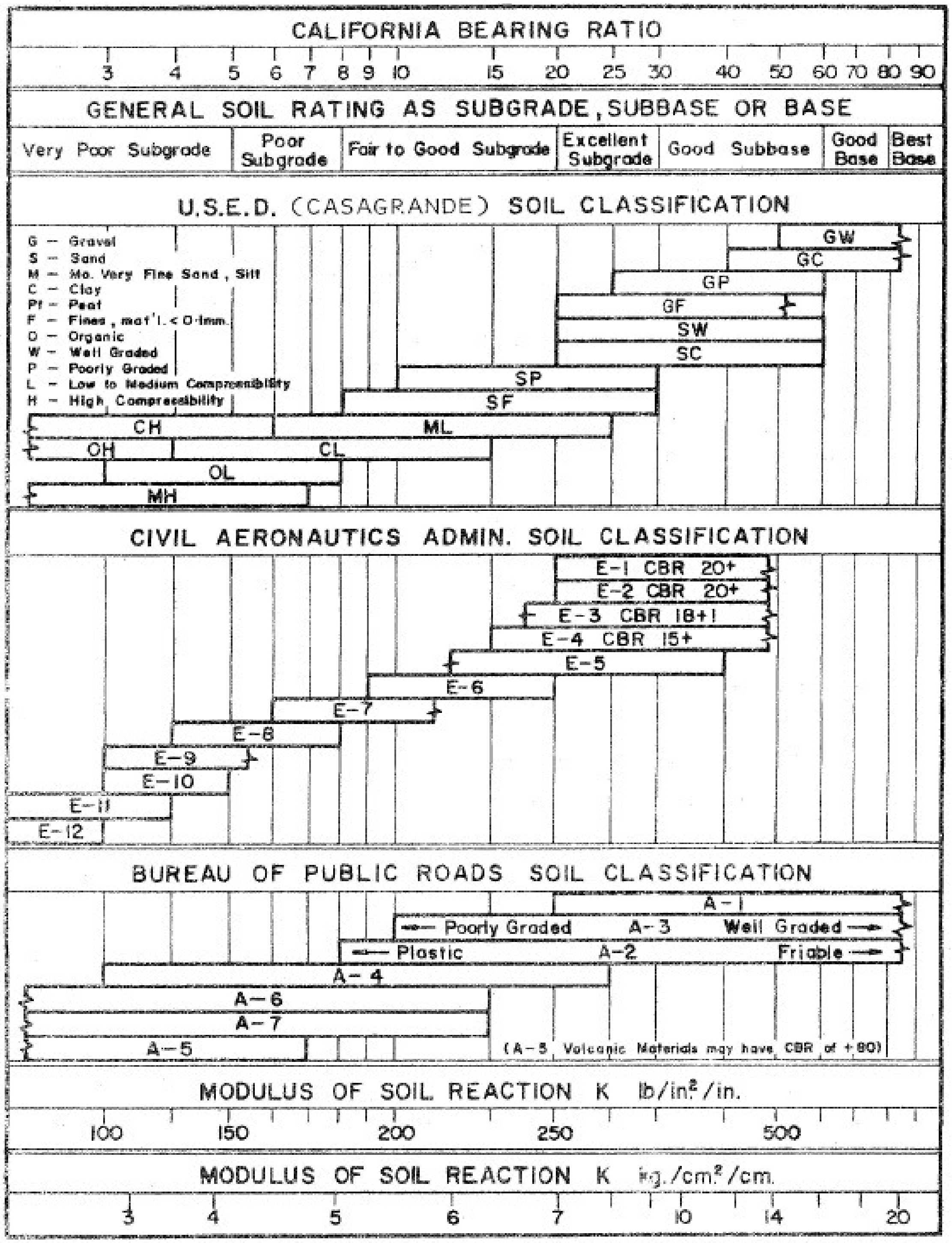


CHART TAKEN FROM "Design of Concrete Airport Pavement" PORTLAND CEMENT ASSOCIATION.

N.B. All interrelationships are very approximate. Actual tests are required to determine CBR, K, etc.

Figure 3-2 Interrelationships of soil classification, California Bearing Ratio and K values

3.5.7. The tire pressure category must be determined as high, medium, low or very low. Portland cement concrete surfacing and good to excellent quality bituminous surfacing can sustain the tire pressures commonly encountered and should be classified as high pressure category with no limit on pressure. Bituminous surfacing of inferior quality and aggregate or earth surfacings will require the limitation of lower categories (see 3.3.4). The applicable pressure category should normally be selected based on experience with using aircraft. The highest tire pressure being applied, other than rarely, by using aircraft, without producing observable distress should be the basis for determining the tire pressure category.

3.5.8. The most significant element of the using aircraft evaluation is determination of the critical aircraft and the equivalent pavement classification number (PCN) for reporting purposes. Having determined the pavement type and the subgrade category the next step would be the determination of the ACNs of aircraft using the pavement. For this purpose, the aircraft classification table presented in Appendix 5 or the relevant aircraft characteristics document published by the manufacturer should be used. Comparison of aircraft regularly using the pavements – at their operating masses - with the above-mentioned table or the relevant aircraft characteristics documents will permit determination of the most critical aircraft using the pavement. If the using aircraft are sat satisfactorily being sustained by the pavement and there are no known factors which indicate that substantially heavier aircraft could be supported, the ACN of the most critical aircraft should be reported as the PCN of the pavement. Thus any aircraft having an ACN no higher than this PCN can use the pavement facility at a use rate (as repetitions per month) no greater than that of presently supported aircraft without shortening the use- life of the pavement,

3.5.9. In arriving at the critical aircraft only aircraft using the pavement on continuing basis without unacceptable pavement distress should be considered. The occasional use of the pavement by a more demanding aircraft is not sufficient to ensure it continued support even if no pavement distress is apparent.

3.5.10. As indicated, a PCN directly selected based on the evaluated critical aircraft loading contemplates an aircraft use intensity in the future similar to that at the time of evaluation, if a substantial increase in use (wheel load repetitions) is expected, the PCN should be adjusted downward to accommodate the increase. A basis for the adjustment, which relates load magnitude to load repetitions, is presented in 3.4.

3.5.11. Pavements for light aircraft. In evaluating pavements meant for light aircraft - 5700 kg mass and less - it is unnecessary to consider the geometry of the undercarriage of aircraft or how the aircraft load is distributed among the wheels. Thus subgrade class and pavement type need not be reported, and only the maximum allowable aircraft mass and maximum allowable tire pressure need be determined and reported. For these the foregoing guidance on techniques for "using aircraft" evaluation should be followed.

3.5.12. Because the 5700 kg limit for light aircraft represents pavement loads only two-thirds or less of common highway leads, the assessment of traffic using pavements should extend to consideration of heavy ground vehicles such as fuel trucks, fire trucks , snow ploughs, service vehicles and the like. These must also be controlled in relation to load limited pavements.

3.6 Techniques and equipment for "technical" evaluation

3.6.1. Technical evaluation is the process of defining or quantifying the bearing capacity of a pavement through measurement and study of the characteristics of the pavement and its behaviour under load. This can be done either by an inversion of the design process, using design parameters and methods, but reversing the process to determine allowable load from existing pavement characteristics, or by a direct determination of response of the pavement to load by one of several means.

3.6.2. Pavement behaviour concepts for design and evaluation. Concepts of behaviour developed into analytical means by which pavements can be designed to accommodate specific site and aircraft traffic conditions are commonly referred to as design methods. There are a variety of concepts and many specific design methods. For example, several design and evaluation methods are explained in some detail in Chapter 4 of this Manual.

3.6.2.1. The early methods. The early methods for design and evaluation of flexible pavements were experience based and theory extended. They made use of index type tests to assess the strength of the subgrade and commonly to also assess the strength or contributing strength of base and sub-base layers. These were tests such as the CBR, plate bearing, and many others, especially in highway design. These early methods, extensively developed, are still the methods in primary use for aerodrome pavement design. The CBR method adopted for ACN determinations as mentioned in Chapter 1 and Appendix 2 of this Manual is an excellent example, and the French and Canadian methods described in Chapter 4 are further examples of CBR and plate loading methods, respectively.

3.6.2.2. Early methods for design and evaluation of rigid pavements virtually all made use of the Westergaard model (elastic plate on a Winkler foundation) but included various extensions to treat fatigue, ratio of design stress to ultimate stress, strengthening effects of subbase (or base) layers, etc. Westergaard developed methods for two cases: loading at the centre of a pavement slab (width unlimited) and loading at the edge of a slab (otherwise unlimited). While most rigid pavement methods use the centre slab load condition, some use the edge condition. These consider load transfer to the adjacent slab but means of treating the transfer vary. Plate bearing tests aroused to characterize subgrade (or subgrade and sub-base) support which is an essential element of these design methods. Here again the early methods, further developed, remain the primary basis for aerodrome pavement design. The method adopted for ACN determination (see Chapter 1 and Appendix 2) is an excellent example of these methods, and several other examples are presented in Chapter 4.

3.6.2.3. The newer - more fundamental - methods. Continuing efforts to base pavement design on more fundamental principles has led to the development of methods using the stress-strain response of materials and rational theoretical models. The advances in computer technology have made these previously intractable methods practical and led to computer oriented developments not otherwise possible.

3.6.2.4. The most popular theoretical model for the newer design methods is the elastic layered system. Layers are of finite thickness and infinite extent laterally except that the lowest layer (subgrade) is also of infinite extent downward. Response of each layer is characterized by its modulus of elasticity and Poisson's ratio. Values for these parameters are variously determined by laboratory tests of several types, by field tests of several types with correlations or calculated derivations, or merely by estimating values where magnitudes are not critical. These methods permit the stresses, strains, and deflexions from imposed loads to be computed. Multiple loads can be treated by superimposition of single loads. Commonly, the magnitude of strain at critical points (top of subgrade beneath load, bottom of surface layer, etc) is correlated with intended pavement performance for use in design or evaluation. While these methods have been applied mostly to flexible pavements there have also been applications to design of rigid pavements.

3.6.2.5. While the elastic layered models are currently popular it is recognized that the stress-strain response of pavement materials is non-linear. The layering permits variation of elastic modulus magnitude from layer to layer, but not laterally within each layer. There are developments which establish a stress dependence of the modulus of elasticity and use this dependence in finite element models of the pavement, through iterative computational means, to establish the effective modulus - element by element in the grid - and thereby produce a more satisfactory model. Here also strains calculated for critical locations and compared with correlations to expected behaviour. Finite element models are also being used to better model specific geometric aspects of rigid pavements but these remain largely research applications.

3.6.2.6. Direct load response methods. Theories applied earlier to pavement behaviour indicated proportionality between load and deflexion, thus implying that deflexion should be an indicator of capacity of a pavement to support load. This also implied that pavement deflexion determined for a particular applied load could be adjusted proportionately to predict the deflexion which would result from other loads. These were a basis for pavement evaluation. Field verification both from experience and research soon showed strong trends relating pavement behaviour to load magnitude and deflexion and led to the establishment of limiting deflexions for evaluation. There have since been many controlled tests and much carefully analyzed field experience which confirm a strong relation between pavement deflexion and the expected load repetitions (to failure) life of the pavement subject to the load which caused that deflexion. However, this relation, though strong, is not well represented by a single line or curve. It is a somewhat broad band within which many secondary factors appear to be impacting.

3.6.2.7. This established strong relation has been and is being used as the basis for pavement evaluation, but predominantly - until recently - applications have been to flexible pavements. Methods based on plate tests have been most common and the standard 762 mm diameter plate preferred. The LCN method and long used Canadian method are examples (see Chapter 4). Deflexions under actual wheel loads (or between the duals of two and four wheel gear) are the basis of some expedient methods which closely parallel the plate methods. The Benkleman Beam methods, as well as other highway methods, are applicable to evaluation of light aircraft pavements (see the Canadian practice in Chapter 4).

3.6.2.8. There are a number of reasons why dynamic pavement loading equipment became popular. Static plate loads of wheel load magnitude are neither transportable nor easily repositioned. Dynamic loading applies a pulse load much more like the pulse induced by a passing wheel. Repeated dynamic loading better represents the repeated loading of wheel traffic. But most important was the development of sensors which could merely be positioned on the pavement or load plate and would measure deflexion (vertical displacement). As a result, a variety of dynamic load equipment has been developed. Initially there were devices for highway applications and later heavier devices for aerodrome pavements. These range from light devices including loads of less than1000 kg to the heavy device described later in this chapter in connection with the United States FAA non-destructive evaluation methods (see 3.6.5). All of these earlier devices involved reciprocating masses capable of producing peak-to-peak pulse loads of up to nearly twice the static load. The pulse loads are essentially sinusoidal and steady state. Some devices can vary frequency and Load (but not static load except for surcharging). Some later dynamic devices - apparently quickly being popular involve a falling mass. These can apply loads in excess of twice the static mass and can vary force magnitude by controlling the height of fall. Pluses induced are repetitive but not steady, and the frequency is that which is normal for the device and pavement combination. The dynamic devices are applied in much the same manner as the static methods discussed in 3.6.2.7. Some can also be used to generate data on the stress-strain response of the pavement materials, as will be discussed later.

3.6.2.9. Essential inputs to pavement design methods. The parameters which define behavior of elements (layers) of a particular pavement within the model upon which its design is based vary from the CBR and other index type tests of the earlier flexible pavement methods and plate load tests of Westergaard rigid pavement and some flexible pavement method to the stress-strain, modulus values employed in the newer more fundamental methods.

3.6.2.10. CBR tests for determining the strengths of subgrades and of other unbound pavement layers for use in design or evaluation should be as described in the particular method employed (French. United States/ FAA, other), but generally will be as covered ASTM

D1883, "Bearing Ratio of Laboratory Compacted Soil for Laboratory Test Determinations". Commonly, field in-place CBR tests are preferable laboratory tests whenever possible, and such tests should be conducted in accordance with the following guidance (from United States Military Standard 621A).

3.6.2.11. Field in-place CBR tests

a) These tests are used for design under any one of the following conditions:

(1) when the in-place density and water content are such that the degree of saturation (percentage of voids filled with water) is 80 per cent or greater;

(2) when the material is coarse-grained and cohesion less so that it is not affected by changes in water content; or

(3) When construction was completed several years before. In the last-named case, the water content does not actually become constant but appears to fluctuate within rather narrow ranges, and the field in-place test is considered a satisfactory indicator of the load-carrying capacity. The time required for the water content to become stabilized cannot be stated definitely, but the minimum time is approximately three years.

b) Penetration. Level the surface to be tested, and remove all loose material. Then follow the procedure described in ASTM D-1883.

c) Number of tests. Three in-place CBR tests should be performed at each elevation tested in the base course and at the surface of the subgrade. However, if the results of the three tests in any group do not show reasonable agreement, additional tests should be made at the same location. 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. For CBRs example, actual test results of 6, 8 and 9 are reasonable and can be averaged as 8; results of 23, 18, and 20 are reasonable and can be averaged as 20. If the first three same location, and the numerical average of the six tests is used as the CBR at that location.

d) Moisture content and density. After completion of the CBR test, a sample shall be obtained at the point of penetration for moisture-content determination, and 10 to 15 cm away from the point of penetration for density determination.

3.6.2.12. Plate load tests for determination of the modulus of subgrade reaction (k) for Westergaard analysis in evaluation or design should be made in accordance with procedures of the method employed, or can be as presented in ASTM D1196, "Non-Repetitive Static Plate Load Tests of Soils and Flexible Pavement Components, for use in Evaluation and Design of Airport and Highway Pavements" or in ASTM D1196, "Repetitive Static Plate Load Tests of Soils and Flexible Components, for Use in Evaluation and Design of Airport and Highway Pavements". The procedures also relate to flexible pavement design, as indicated by ASTM standards titles. The Canadian practice, as described in Chapter 4, makes use of the ASTM method. The Canadian practice also covers use of other static or dynamic tests with non-standard plate sizes for either determination of subgrade coefficient values or for direct use in pavement evaluations.

3.6.2.13. Conventional methods and values pertaining to determination of modulus of elasticity, E, and Poisson's ratio, μ, are used in depicting structural behavior of the concrete layer in Westergaard analyses of rigid pavement. Commonly, μ is taken to be 0.15. The modulus, E, should be determined by test of the concrete and will normally be in the range of 25000 to 30000 MPa.

3.6.2.14. Modulus of elasticity and Poisson's ratio values are needed for each layer of an elastic layered system, and these can be determined in a variety of ways. Poisson's ratio is not a sensitive parameter and is commonly taken to be 0.3 to 0.33 for aggregate materials and 0.4 to 0.5 for fine grained or plastic materials. Since mean of determining modulus of elasticity vary and since the stress-strain response of soil and aggregate materials is non-linear (not proportional) the values found for a particular material, by the various means, are not the same singular quantity which ideal theoretical considerations would lead one to expect. Following are some of the ways in which modulus of elasticity values can be determined for use in theoretical models (such as elastic layered) of pavement behavior.

a) Modulus of elasticity values for subgrade materials particularly, but for other pavement layers as well – excepting bituminous or cemented materials – can be determined from correlations with index type strength tests. Most common has been correlation with CBR where:
E = 10 CBR MPa

b) Stress-strain response (modulus) can be determined by direct test of prepared or field sampled specimens, but these are nearly always unsatisfactory. Response is too greatly affected by either preparation or sampling disturbance to be representative.

c) It has been found that prepared specimens, and in some case specimens from field samples, can be subjected to repeated loading to provide - after several to many load cycles - a reasonably representative modulus or stress-strain response curve. Modulus of elasticity so determined is referred to as resilient modulus and is currently strongly favoured - in some form - for layered elastic analyses. Tests can be conducted as triaxial tests, indirect tensile tests, even unconfined compression test, and there may be others. Loadings can be regular wave forms (sinusoidal, etc) but are commonly of a selected load pulse shape with delays between pulses to better represent passing wheels. Resilient modulus can be determined for bituminous materials by some of these tests and other similar tests, but temperature is most significant both for testing and application of the modulus for bituminous layers. Moduli for the various pavement layers are taken from these type tests and used directly in layered system analyses, but there are frequently problems or questions of validity.

d) When dynamic plate load testing is carried out on existing pavements it is possible to instrument to measure the velocity of propagation of stress waves within the pavements. Means have been developed for deducing the modulus of elasticity of each layer - generally excepting the top layer or layers - of the pavement from these velocity measurements. While moduli so determined are sometimes used directly in layered analyses the determinations are for such small strains that values represent tangent moduli for curved stress-strain relations while the moduli for higher (working strain) stress levels should be lower. Determinations by this means adjusted by judgement or some established analytical means are used.

e) The subgrade modulus is the most significant parameter and some analyses use one of the above methods to determine a modulus for the subgrade and choose the moduli of other layers either directly on a judgement basis or by some simple numerical process (such as twice the underlying layer modulus or one-half the overlying layer modulus) since precise values are not critical.

f) By using selected or simplistically derived moduli for all layers except the subgrade, it is possible to compute a value for subgrade modulus using elastic layered analysis and plate or wheel load deflexions. This is done for some analyses.

g) There is rear interest currently in using elastic layered theory and using field determined deflexions from dynamic load pavement tests for points beneath the centre of load and at several offset positions from the load centre. By iterative computer means the moduli of the subgrade and several overlying layers can be computed. Such computed moduli are then used in the layered model to compute strains at critical locations as predicators of pavement performance.

3.6.2.15. Finite element methods permit formulation of pavement models which not only can provide for layering but can treat non-linear (curved) stress-strain response found for most pavement materials. Here again there is a requirement for Poisson's ratios and moduli of elasticity but these must now be determined for each pavement layer as a function of the load or stress condition existing at any point in the model (on any finite element). Moduli relations are established from laboratory tests and most commonly by repeated triaxial load tests. Generally, these are of the following form but there are variants.

a) For granular materials:

$$M_r = E = k_1 \theta^{k_2}$$

or

$$M_r = E = k_3 \sigma_3^{k_4}$$

b) For fine-grained soils:

$$M_r = E = k_5 \sigma_d^{k_6}$$

Where:

M_r- resilient modulus

E - modulus of elasticity

θ - bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$ or $\sigma_x + \sigma_y + \sigma_z$ (sum of 3 mutually perpendicular normal stresses at a point)

$\sigma_1, \sigma_2, \sigma_3$ - principal stresses
σ_3 - confining stress on the triaxial specimen
σ_d - deviator stress = $\sigma_1 - \sigma_3$
$k_1, k_2, k_3, k_4, k_5, k_6$ - constants found by test

3.6.3. Evaluation by inversion of design. To design a pavement one must select a design method. Then determine the thicknesses and acceptable characteristics of materials for each layer and the wearing surface taking into account the subgrade upon which the pavement will rest and the magnitude and intensity of traffic loading which must be supported. For evaluation, the process must be inverted since the pavement is already in existence. Character of the subgrade and thickness and character of each structural layer including the surfacing must be established, from which the maximum allowable magnitude and frequency of allowable aircraft loading can be determined by using a chosen design method in reverse. It is not necessary that the design method selected for evaluation be the method by which the pavement was designed, but the essential parameters, which characterize behaviour of the various materials (layers) must be those which the chosen design method employed.

3.6.3.1. The method and elements of deal. The design method to be inverted for evaluation must first be chosen. Next the elements of design inherent in the existing pavement must be evaluated in accordance with the selected design method.

a) Thickness of each layer must be determined. This may be possible from construction records or may require the drilling of core holes or opening of test pits to permit measuring thickness.

b) Subgrade strength and character must be determined. Here also construction records may supply the needed information either directly or by a translation of the information to the form needed for the selected design method. Otherwise it will be necessary to obtain the needed information from field studies. Reference to 3.6.2.9 to 3.6.2.1 will show the wide variety of ways in which subgrade behaviour is treated in the various design methods. Test pits may be necessary to permit penetration or plate testing or sampling of subgrade material for laboratory testing. Sampling or penetration testing in core holes may be possible. Dynamic or static surface load deflexion or wave propagation testing may be required. Specific guidance must be gained from details of the design method chosen for use in evaluation.
c) The strength and character of layers between the subgrade and surface must also be determined. Problems are much the same as for the subgrade (see b above) and guidance must come from the chosen design method.
d) Most procedures for the design of rigid pavements require a modulus of elasticity and limiting flexural stress for the concrete. If these are not available from construction records they should be determined by test on specimens extracted from the pavement (see DSTM C 469 - modulus of elasticity and ASTM C683 - flexural strength). For reinforced or pre-stressed concrete layers dependence must he placed on details of the individual selected design method.
e) Bituminous surfacing (or overlay) layers must be characterized to suit the selected design method and to permit determination of any tire pressure limitation which might apply. Construction records may provide the needed information otherwise testing will be required. Pavement temperature data may be required to help assess the stress-strain response or tire pressure effects on the bituminous layer.
f) Any special consideration of frost effects by the selected design method or for the climate of the area need to be treated and the impact upon the evaluation determined.
g) The cumulative load repetitions to which the pavement is subject is an important element of design and both past traffic sustained and future traffic expected may be factors in evaluation. See 3.4 in relation to assessing traffic. For some design methods it is sufficient to consider that the traffic being sustained adequately represents future traffic and the limiting load established by evaluation is for this intensity of traffic. This assumption is inherent in the translations between aircraft mass and ACN (or the reverse) of the ACN-PCN method. Many methods, however, require a load or stress repetitions magnitude as a basis for selection of a limiting deflexion or strain which is needed for load limit evaluation.

From the chosen design method and established quantities for the design elements, limiting load or mass can be established for any aircraft expected to use the pavement.

3.6.4. Direct or non-destructive evaluation. Direct evaluation involves loading a pavement, measuring its response, (usually in terms of deflexion under the load and sometimes also at points offset from the load to show deflexion basin shape), and inferring expected load support capacity from the measurements. Concepts were discussed in 3.6.2.6, 3.6.2.7, and 3.6.2.8.

3.6.4.1. Static methods. Static methods involve positioning plates or wheels, applying load, and measuring deflexions. Plate loads require a reaction against which to work in applying load while wheels can be rolled into position and then away. The original LCN for flexible pavements, developed by the United Kingdom but used by many, is an excellent example of the direct static methods. The Canadian method for flexible or rigid pavements uses plate load and deflexion but less directly (see Chapter 4). These direct methods depend on a correlation between pavement performance and deflexion resulting from loading of the type indicated in Figure 3-3. A warning comment may be needed here, since such correlations can be misinterpreted. They do not indicate the deflexion which will be measured under the load after it has been applied for some number of repetitions as might be interpreted. Deflexions of a pavement are essentially the same when measured early or late (following initial adjustment and before terminal deterioration) in its life. These correlations indicate the number of repetitions that can be applied to the pavement by the load which caused the deflection before failure of the pavement. Correlations are established by measuring the deflexions of satisfactory pavements and establishing their traffic history. The expeditious deflexion methods for evaluation described below are a good example of static methods.

3.6.4.2. Expeditious deflexion methods. Studies and observations by many researchers have shown a strong general correlation between the deflexion of a pavement under a wheel load and the number of traffic applications (repetitions) of that wheel load which will result in severe deterioration (failure) of the pavement (see Figure 3-3). These provide the basis for a simple expeditious means of evaluating pavement strength. References to several of these curves are listed below:

Transport and Road Research Laboratory Report LR 375 (British);

California Highway Research Report 633128;

Paper presented by Gschwendt and Poliacek at the Third International Conference on Structural Design of Asphalt Pavements; and

Paper presented by Joshep and Hali also at the Third International Conference on Structural Design of Asphalt Pavements.

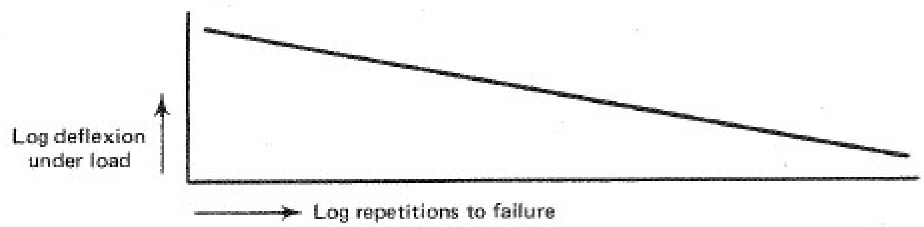


Figure 3- 3

3.6.4.3. While the pattern of these relations is quite strong, the scatter of specific points is considerable. Thus either the conservatisms of a limiting curve or the low confidence engendered by the broad scatter of points or some combination must be accepted in using these relations for expeditious pavement evaluations. They do provide a simple relatively inexpensive means of evaluation. The procedure for such evaluation is as follows:

a) Measure deflexion under a substantial wheel load in a selected critical pavement location. Single or multiple wheel configurations can be used.

   1) position aircraft wheel in critical area;

   2) mark points along pavement for measurement as indicated in Figure 3-4 a);

   3) using "line of sight" method, take rod readings at each point;

   4) move aircraft away and repeat rod readings;

   5) Plot difference in rod readings as deflexions. See Figure 3-4 b);and

   6) Connect points to gain an estimate of maximum deflexion beneath tire.

b) Plot load versus maximum deflexion as illustrated in Figure 3-4 c).

c) Combine the deflexion versus failure repetitions curve with the above curve to provide an evaluation of pavement bearing strength for the gear used to determine deflexion.

   1) determine the repetitions of load (or equivalent repetitions as explained in 3.4) which it is intended must use the pavement before failure;

   2) from a correlation of the type shown in Figure 3-3 determine the deflexion for the repetitions to failure; and

3) From the established relation of load to deflexion of the type shown in Figure 3-4 determine the pavement bearing strength in terms of the magnitude of load allowable on the wheel used for the deflexion measurements.

d) Use the procedure described in Chapter 1 to find how the evaluated pavement bearing strength relates to the PCN. Aircraft with ACN no greater than this PCN can use the pavement without overloading it. See Appendix 5 for ACN versus mass information.

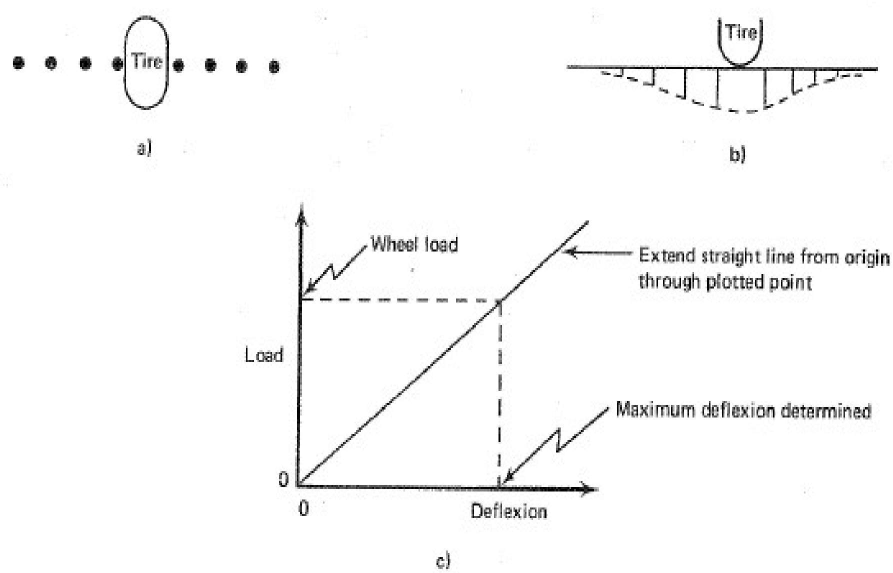


Figure 3-4

3.6.4.4. A similar procedure can be followed using a jack and loading plate working beneath a jacking point of an aircraft wing or some equally suitable reaction load. The complete pattern of load versus deflexion can be determined and dial gauges mounted on a long reference beam call be used instead of optical survey methods. With provision of a suitable access aperture the deflexion directly beneath the centre of the load can be measured. Results can be treated on the save lines as those for a single wheel load.

3.6.4.5. Methods used for highway load deflexion measurements, such as the Benkleman Beam methods, can be used to develop deflexion versus load patterns. Results are treated as indicated in Figure 3-4 to extrapolate loads to those of aircraft single-wheel loads, which with a relation as in Figure 3-3, permits evaluation of pavement bearing strength for single-wheel loads. From this the limiting aircraft mass on pavements for light aircraft can be determined directly and reported in accordance with Chapter 1, 1.2. If unusually large loading plate or tire pressures are involved it may be necessary to adjust between the single load characteristics used in the determination of the type indicated in Figure 3-4 (3.6.4.3a) and the reported limiting aircraft mass allowable or critical vehicle loads being compared to the limiting mass. Such adjustments can follow the procedures in Appendix 2 or a selected pavement design method. Limits on pavements for heavier aircraft can be determined as indicated in 3.6.4.3d). It should be noted that recent findings indicate extrapolation of load deflexion relations (as in Figure 3-4 c))from small load data taken on high strength pavements do not give good results, Unfortunately, the limits of extrapolation for good result are not established.

3.6.4.6. Dynamic methods. These methods involve a dynamic loading device which is mounted for travel on a vehicle or trailer and which is lowered, in position, onto the pavement. Devices make use of counter rotating masses, hydraulically actuated reciprocating masses, or falling weights (masses) to apply a series of pulses either in steady state by the reciprocating or rotating masses or attenuating by the falling mass. Most apply the load through a loading plate but some smaller devices use rigid wheels or pads. All methods make use of inertial instruments (sensors) which when placed on the pavement surface or on the loading plate can measure vertical displacement (deflexion).The dynamic loading is determined, usually by a load cell through which the load is passed on to the load plate. Comparison of the load applied and displacements measured provide load-deflexion relations for the pavement tested. Displacements are always measured directly under the load but are also measured at several additional points at specific distances from the centre of the load. Thus load-deflexion relations are determined not only for the load axis (point of maximum deflexion) but also at offset points which indicate the curvature or shape (slope) of the deflexion basin. The devices vary in size from some highly developed, highway oriented, units which apply loadings of less than 1000 kg to the large unit described in the United States FAA non-destructive test method presented in 3.6.5. Some of the counter-rotating and reciprocating mass systems can vary the frequency of dynamic loading and some of these and the falling weight units can vary the applied load.

3.6.4.7. It is possible to measure the time for stress waves induced by the dynamic loading to travel from one sensor to the next, and to compute the velocity from this time and distance between sensors. Some dynamic methods make use of these velocity measurements to evaluate the strength or stress-strain response of the subgrade and overlying pavement layers for use in various design methods. Shear wave velocity, v, is related to Modulus of Elasticity, E, by the relation:

$$v = \frac{1}{2}\left(\frac{E}{(1+\mu)\rho}\right)$$

(See Barkan's "Dynamics of Bases and Foundations")

Where Poisson's Ratio, μ, can satisfactorily be estimated (see 3.6.2.13 and 3 6.2.14),and density, ρ, of the subgrade or pavement layer (sub-base-base) can be determined by measurement or satisfactorily estimated. Modulus values thus determined are used, either directly or with modification, in theoretical design models, or they are used with correlations to project subgrade and other layer strengths in terms of CBR, subgrade coefficient k, and similar strength index quantities. Sensors used in the velocity measurements may need to be located at greater distances from the load than when used to determine deflection basin shape. Also, the dynamic device must be capable of frequency variation since the various pavement layers respond at preferred frequencies and these must be found and dynamic load energy induced at the preferred frequency for determination of each layer's velocity of wave energy propagation.

3.6.4.8. Application of dynamic methods measurements. The central and offset positions deflexions and stress-wave velocities variously determined by the variety of dynamic equipment and methods in use are being applied for pavement evaluation in a number of ways.

a) Direct correlations are made between the load-deflexion in response of pavement to dynamic loading and pavement behaviour. The correlations are developed from dynamic load testing of pavements for which behaviour can be established. The United States FAA nondestructive evaluation methodology presented in 3.6. is an excellent example.

b) Measurements from dynamic methods, either directly or with extrapolation, can provide plate load information. This can serve as input - with suitable plate size or other conversions - to methods such as the LCN or Canadian procedures. Used directly on subgrades or on other layers with established correlations subgrade coefficients can be determined for Westergaard analyses.

c) Shape of the deflection basin established from sensors placed at offsets from the load axis are used in some methods - especially for highways - to reflect overall stiffness, and thereby load distributing character, of the pavement structure. But direct use in establishing evaluation of load capacity has not found success,

d) Measured deflection under dynamic load is used to establish the effective modulus of elasticity of the subgrade in theoretical pavement models. The elastic constants (modulus and Poisson's ratio) for other layers are established by assumption or test and the subgrade modulus calculated using the load, the deflection measured, and the pavement model, commonly the elastic layered theory.

e) More recent developments involve the use of the elastic layered computer programmes. With an appropriate load applied, deflections are measured in the centre and at several offset locations. Then iterative computation means are used to establish elastic moduli for all layers of the pavement modeled.

f) Theoretical models with elastic constants as in d) and e) above are used to calculate strain in flexure of the top layer beneath the load or vertical strain at the top of subgrade beneath the load; which locations are considered critical for flexible pavements. Stress or strain in flexure of a rigid pavement slab can be similarly calculated. These are compared to values of strain (or stress) from established correlations with pavement performance. The literature provides many examples of these correlations.

1. 1977 International Air Transportation Conference, ASCE Proceedings - paper by Monismith.

2. The Design and Performance of Road Pavements by D. Croney -Transport and Road Research Laboratory, United Kingdom – Chapters 13 and 15

3. Fatigue of Compacted Bituminous Aggregate Mixtures, ASTM - STP508.

4. Symposium on Nondestructive Test and Evaluation of Airport Pavement– Nov 1975, Vicksburg, Miss., published May 1976 by U.S. Army Engineer - WES paper by Nielsen and Baird.

5. Other examples should be easily found in the pavement literatures.

g) Stress-wave velocity measurements are used to establish pavement layer characteristics without sampling. Moduli of elasticity of pavement layers are derived from these measurements and used directly in theoretical models or adjusted to better represent moduli at larger strains and used in the models. CBR values are derived from correlations between CBR and derived elastic moduli, commonly form E = 10 CBR in MPa. Modulus of subgrade reaction, k, and other such strength values could be similar derived.

3.6.4.9. Pavement strength reporting. For reporting information on pavement bearing strength the four elements specified in CAR-14, Part I and the PCN must be established.

a) Pavement type. The pavement will be considered rigid (code-R) if its primary load distribution capability is provided by a plain, reinforced, or pre-stressed Portland cement concrete (PCC) layer, and this layer is not so shattered that it can no longer perform as a load distributing slab. A pavement which makes primary use of a thick and strongly stabilized layer and which, as a result, is substantially thinner than an equivalent flexible pavement using no stabilized layer (such as the LCF structures at Newark) might also be considered rigid. All other pavements should be reported as flexible (code -F). This includes aggregate or earth-surfaced strips and expedient surfacing of military landing mat.

b) Subgrade strength. The subgrade strength category must be evaluated as high strength (A), medium strength (B), low strength (C), or ultra low strength (D). If CBR or coefficients of subgrade reaction are directly involved, selection of category can be made directly from the prescribed limits in CAR-14, Part I. Otherwise the category must be determined from a correlation between the subgrade strength parameter used for evaluation and CBR or subgrade coefficient, or it must be determined directly by judgment. For subgrade strengths on the borderline between categories, selection of the lower (weaker) strength category will generally be more conservative in relation to protection of the pavement from overload.

c) Tire pressure. The tire pressure category must be evaluated as high (W), medium (X), low (Y) or very low (Z). Where a surfacing is PCC the high category is virtually always pertinent. High quality bituminous surfacing or overlays should readily accept hi h category tire pressures while the very low category need only be able to sustain normal truck tire pressures. The medium and low categories fall below and above these two limits respectively. Some design methods set minimum bituminous layer thicknesses in relation to tire pressures (s e the Canadian method in Chapter 4) and these may help in selecting the tire pressure category. Some methods prescribe tire pressure directly in relation to surfacing characteristics and these can be directly applied or category selection. Otherwise selection must depend on experience and judgment in relation to surfacing characteristics, tire pressures of using aircraft, and condition surveys of pavements.

d) Evaluation method. This will be a technical evaluation reported as code T.

e) Reported PCN The PCN to be reported can be determined from the aircraft loads (masses) which the evaluation has established as maximum allowable or the pavement. By using the evaluation load for one of the heaviest type aircraft using the pavement and information shown in Appendix, and interpolating as necessary, the PCN can be found. This can be done for a selected representative aircraft or for several aircraft for which evaluation of allowable load has been made. All such determinations should yield the same PCN value, or very nearly so. If there are large differences it would be well to recheck both the translation from the evaluation load and the evaluation. If differences are small an average or lower range value should be selected for reporting. If needed information is not provided in Appendix 5 they can be obtained from the aircraft manufacturer, ICAO, or by analysis using the prescribed ACN-PCN methods (see Appendix 2).

3.6.4.10. Reporting strength of pavements meant for light aircraft. The pavement type, subgrade strength category, and type of evaluation are not required for light aircraft pavements, so only the limiting aircraft mass and tire pressure need be reported. The foregoing methods for load and tire pressure limitation determinations apply to pavements meant for light aircraft as well. Highway evaluation or design methods might also be used. All the precautionary measures discussed in 3.5.7 are equally applicable here.

3.6.5. United States Federal Aviation Administration non-destructive evaluation method*

3.6.5.1. Introduction. This report describes a procedure for the determination of the load-carrying capacity of airport pavement *systems* using non-destructive testing (NDT) techniques. The equipment and procedures have been developed by the United States Corps of Engineers in response to a need of the Federal Aviation Administration (FAA) and United States Army for making rapid evaluations of pavement systems with a minimum of interference to normal airport operations.

3.6.5.2. Little research was conducted in the field of NDT until about the mid-1950s when Royal Dutch Shell Laboratory researchers began a study of vibratory loading devices to evaluate flexible pavements. Many other agencies have since investigated the use of NDT techniques to evaluate pavements. The United States Army Engineer Waterways Experiment Station (WES) conducted minimal research using various types of vibratory equipment during the l95Os and 19608. Much of the early WES work emphasized attempts to measure the elastic properties of the various layers of pavement materials using wave propagation measurements. The basic approach involved use of these elastic constants along with multilayered theory for computation of allowable aircraft loadings. In 1970, an improved vibratory loading device was developed by the Army, and, in 1972, ES began a study for the FAA to develop an NDT evaluation procedure. To meet the FAA time frame, the primary effort has been directed" toward developing a procedure based upon measuring the dynamic stiffness modulus (DSM) of the pavement system and relating this value to pavement performance data. Work is continuing on the development of a methodology for measuring the elastic constants of the various layers using NDT techniques; however, this method has not yet been developed to an acceptable level of confidence.

3.6.5.3. Applications. The NDT evaluation procedure reported herein is applicable only to conventional rigid and flexible pavement systems. A conventional rigid pavement consists of a non-reinforced concrete surfacing layer on non-stabilized base and/or subgrade materials. A conventional flexible pavement consists of a thin (15 cm (6 in) or less) bituminous surfacing layer on non-stabilized layers of base, sub-base, and subgrade materials. Work is currently under way to extend the NDT procedure to other types of pavement systems which incorporate such other variables as thick bituminous surfacing and stabilized layers.

3.6.5.4. Equipment. The evaluation procedure contained herein requires the determination of the response of the pavement system to a specific steady state vibratory loading, Inasmuch as the response of materials making up the pavement system to loading is generally non-linear, the determination of the pavement response of use in the evaluation procedure contained herein requires a specific loading system. The loading device must exert a static load of 16 kips**on the pavement and be capable of producing 0 to l5-kippeak vibratory loads at a frequency of 15 Hz. The load is applied to the pavement surface through a 45 cm (18 in) diameter steel load plate. The vibratory load is monitored by means of three load cells mounted between the actuator and the load plate, and the pavement response is measured by means of velocity transducers mounted on the load plate. Automatic data recording and processing equipment is a necessity. The loading device must be readily transportable to accomplish a large number of tests in a minimum amount of time, thus avoiding interference with normal airport operations. The WES NDT equipment is mounted in a tractor-trailer unit as shown in Figure 3-5.

* The material included in this section was taken from the Federal Aviation Administration United States, Airport Pavement Bulletin No. FAA-74-1 of September 1974.
** 1 kip = 454 kg (1000 lb).

3.6.5.5. Data collection. In the evaluation procedure, the response of the pavement system to vibratory loading is expressed in terms of the DSM. Since the time required to measure a DSM at each testing point is short (2 to min), a large number of DSM measurements can be made during the normal evaluation period. On runways and primary and high-speed taxiways, DSM tests should be made at least every 75 m (250 ft) on alternate sides of t he facility centre line along the main gear wheel paths. For secondary taxiway systems or lesser used runways, DSM tests should be made about every 150 m (500 ft) on alternate sides of the centre line. For apron areas, DSM tests should be conducted in a grid pattern with spacing between 75 m and 150 m (250 ft and 500 ft). Additional tests should be made where wide variations in DSM values are found, depending upon the desired thoroughness of the evaluation. DSM measurements for rigid pavements must be made in the interior (near the centre) of the slab. The layout of DSM test sites and selection of DSM values for evaluation must consider the various pavement types, pavement sections, and construction dates. Thus, a thorough study of as-built pavement drawings is particularly helpful in designing the testing programme. After the DSM tests have been performed and grouped according to pavement type and construction, a representative DSM value should "be selected (as described below) for computation of the allowable loading.

3.6.5.6. At each test site, the loading equipment is positioned, and the dynamic force is varied from 0 to 15 kips at 2-kip intervals at a constant frequency of 15 Hz. The deflection of the pavement surface, measured by the velocity transducers, is plotted versus the applied load as shown in Figure 3- 6. The DSM (corrected as described below) is the inverse of the slope of the deflection versus load plot (see Figure 3-6).

3.6.5.7. In addition to the DSM measurement, it is necessary to know the pavement type (rigid or flexible) and the thicknesses and material classifications of each layer making up the pavement section These parameters can be determined from the construction (as-built) drawings or by drilling small-diameter holes through the pavement.

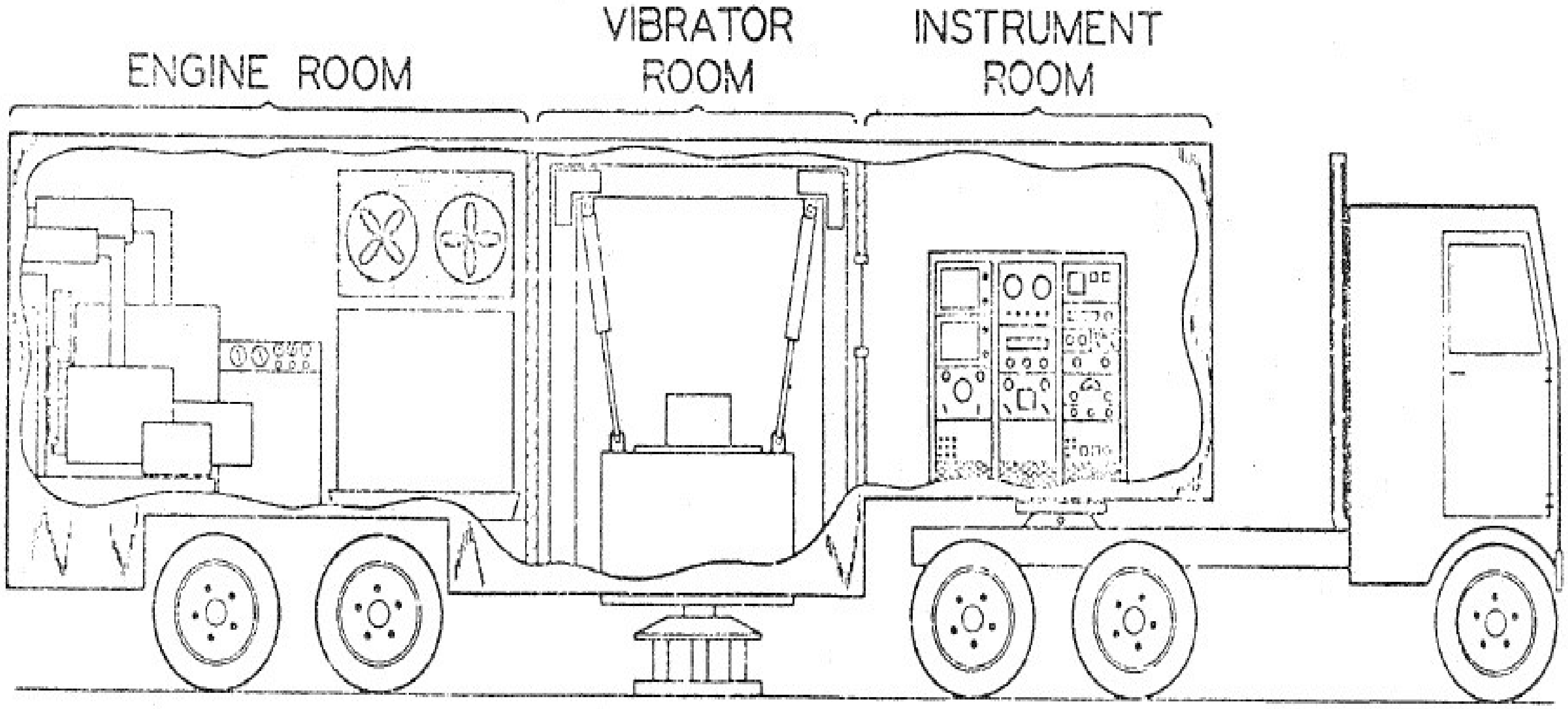


Figure 3-5. Waterways Experiment Station non-destructive testing equipment

3.6.5.8. When the evaluation is for flexible pavement, the temperature of the bituminous material must be determined at the time of test. This can be determined by directly measuring the temperatures with thermometers installed 2.5 cm (1 in) below the top, 2.5 cm (1 in) above the bottom, and at the mid-depth of the bituminous layer and averaging the values to obtain the mean pavement: temperature or by measuring the pavement surface and air temperatures and using Figure 3-7 to estimate the mean pavement temperature.

3.6.5.9. Data correction. The load- deflection response of many pavements, particularly flexible pavements, is non-linear at the" lower force levels but becomes more linear at the higher force levels (12 to 15 kips). In such cases, a correction is applied to the load- deflection curve so that the DSM is obtained from the linear portion of the curve (see Figure 3-6).

3.6.5.10. The modulus of bituminous materials is highly dependent upon temperature, so an adjustment in the measured DSM must be made if the temperature of the bituminous material at the time of test is other than 21^{o}C (70^{o}F). The correction is made by entering Figure -8 with the measured or calculated mean pavement temperature and determining the DSM temperature adjustment factor by which the measured DSM should be multiplied.

3.6.5.11. The DSM and load-carrying capacity of a pavement system can be significantly changed by the freezing and thawing of the materials, especially when frost penetrates a frost-susceptible layer of material. Correction factors to account for these conditions have not been developed. Therefore, the evaluation should be based on the normal temperature range, and, if a frost evaluation is desired, the DSM should be determined during the frost melting period.

3.6.5.12. A representative DSM value must be selected for each pavement group to be evaluated. Although a section of pavement may supposedly be of the same type and construction, it should be treated as more than one pavement group when the DSM values measured in one section of the pavement are greatly different from those in another section. The DSM value to be assigned to a pavement group for evaluation purposes will be determined by subtracting one standard deviation from the statistical mean.

3.6.5.13. Determination of allowable aircraft load. After determination and correction of the measurement of the DSM, the evaluation procedure depends upon the type of pavement, rigid or flexible.

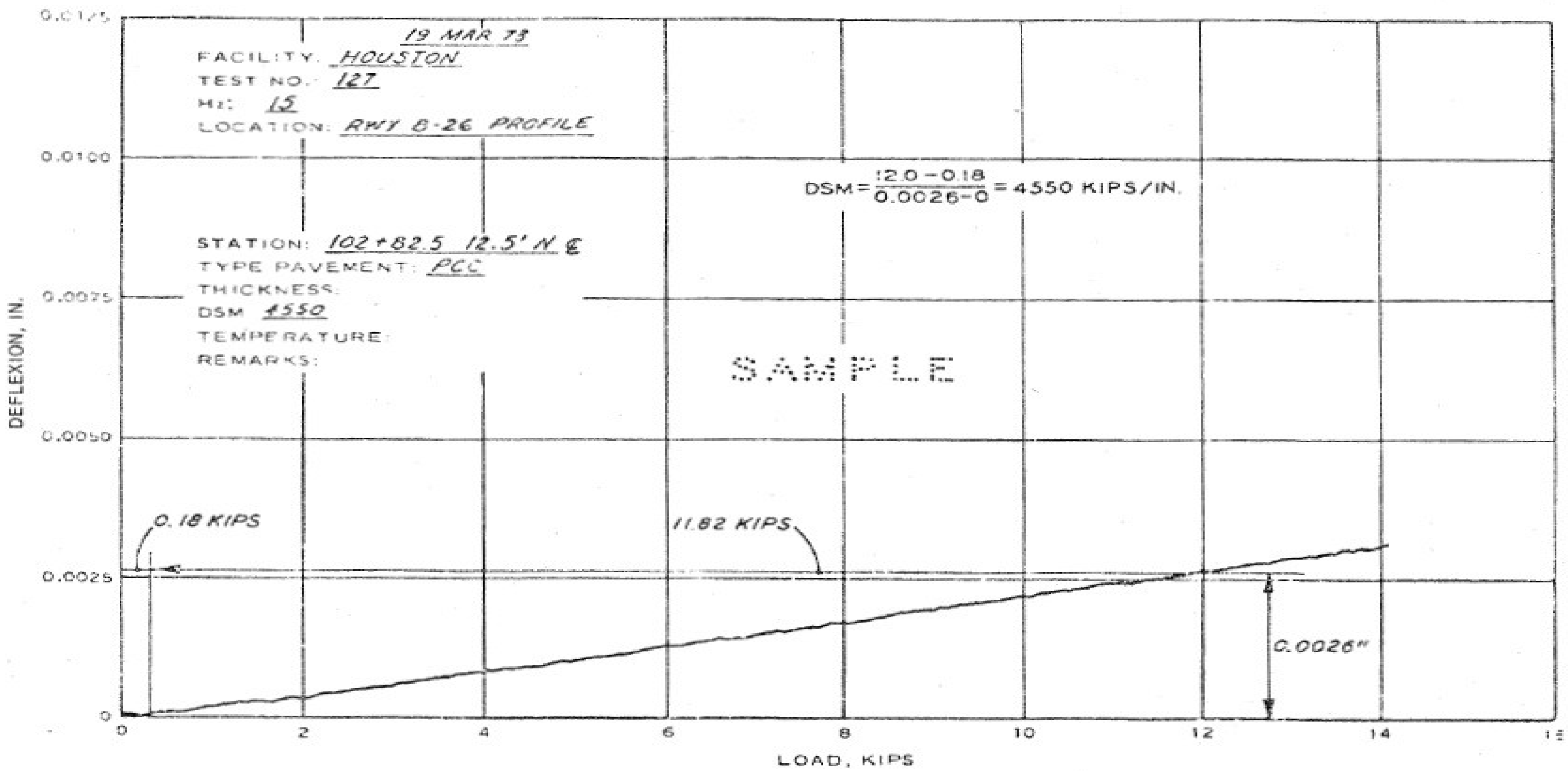


Figure 3-6. Deflexion versus load (sample plot)

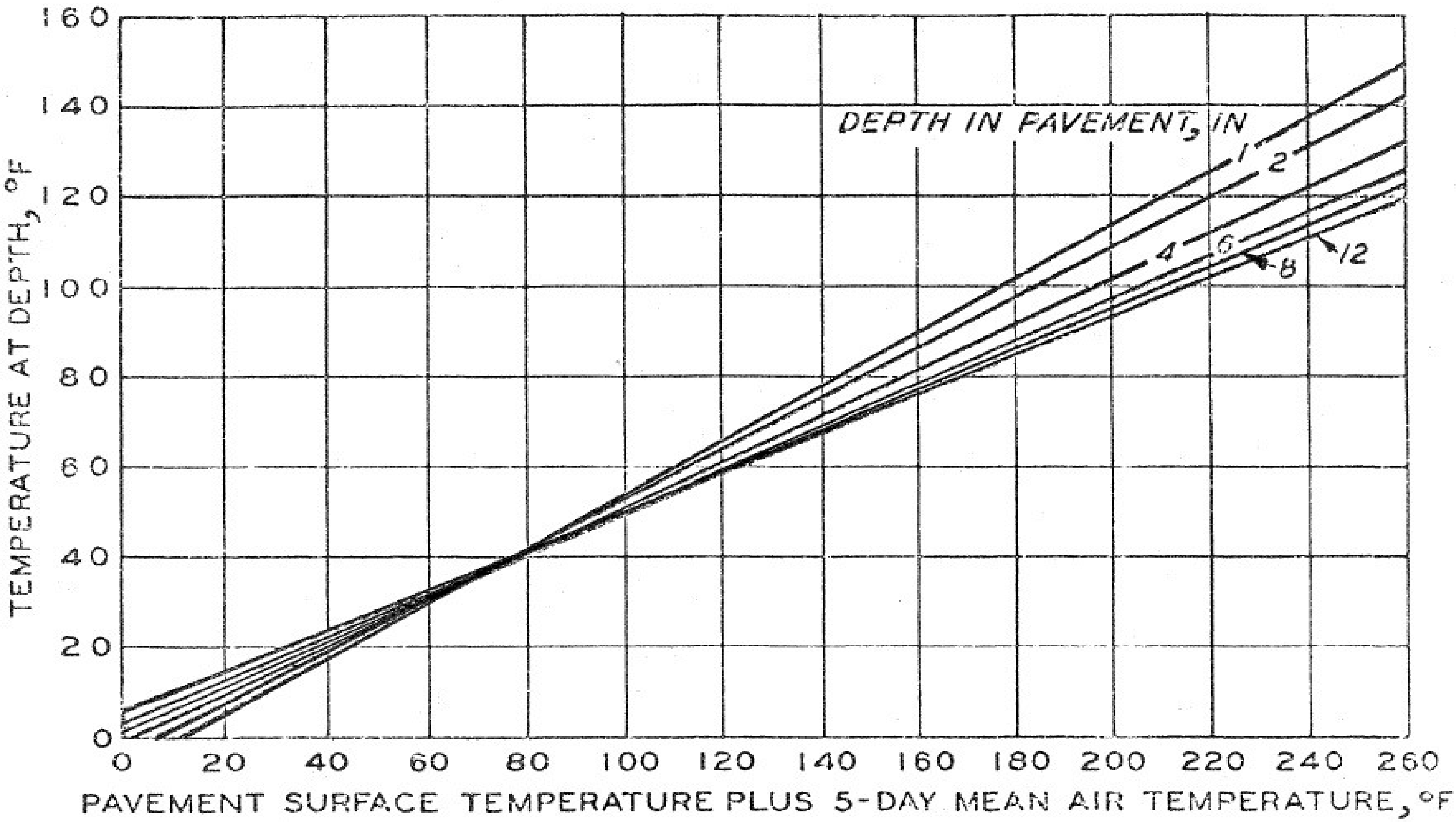


Figure 3-7. Prediction of flexible pavement temperatures

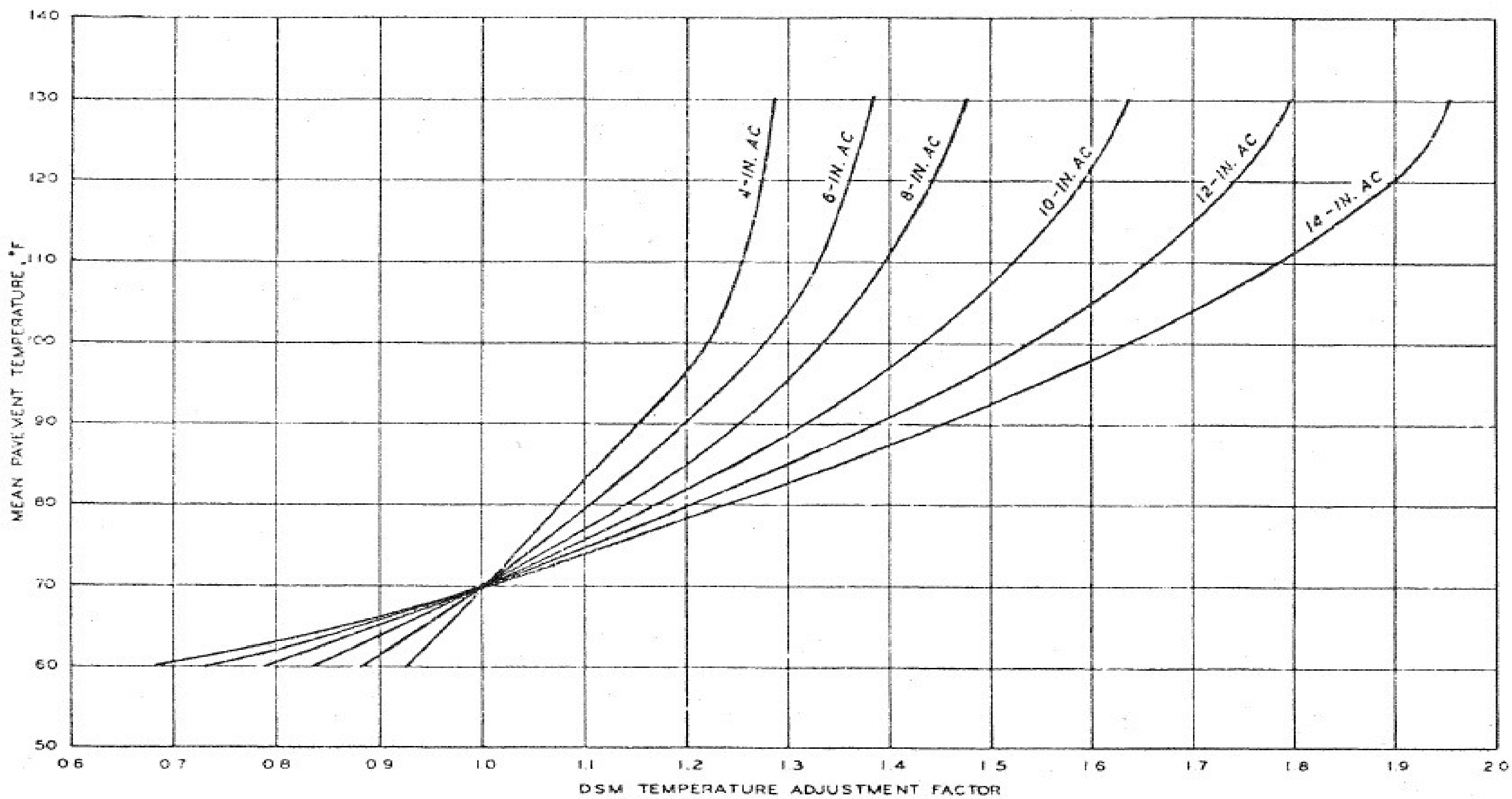


Figure 3-8. DSM temperature adjustment curves

3.6.5.14. Rigid Pavement evaluation.

Step 1

The corrected DSM is used to enter Figure 3-9 and determine the allowable single-wheel load.

Step 2

The radius of relative stiffness is computed as

$$\ell = 24.2 \sqrt[4]{\left(\frac{h^3}{F_F}\right)}$$

Where

h = thickness of the concrete slab, in.
F_F = foundation strength factor determined from Figure 3-10 using the FAA subgrade soil group classification

Step 3

Using, determine the load factor FL from Figure 3-II, 3>12, 3-13 or 3-14 depending upon the gear configuration of the aircraft for which the evaluation is being made.

Step 4

Multiply the allowable single-wheel load from Step 1 by the FL value determined from Step 3 to obtain the gross aircraft loading.

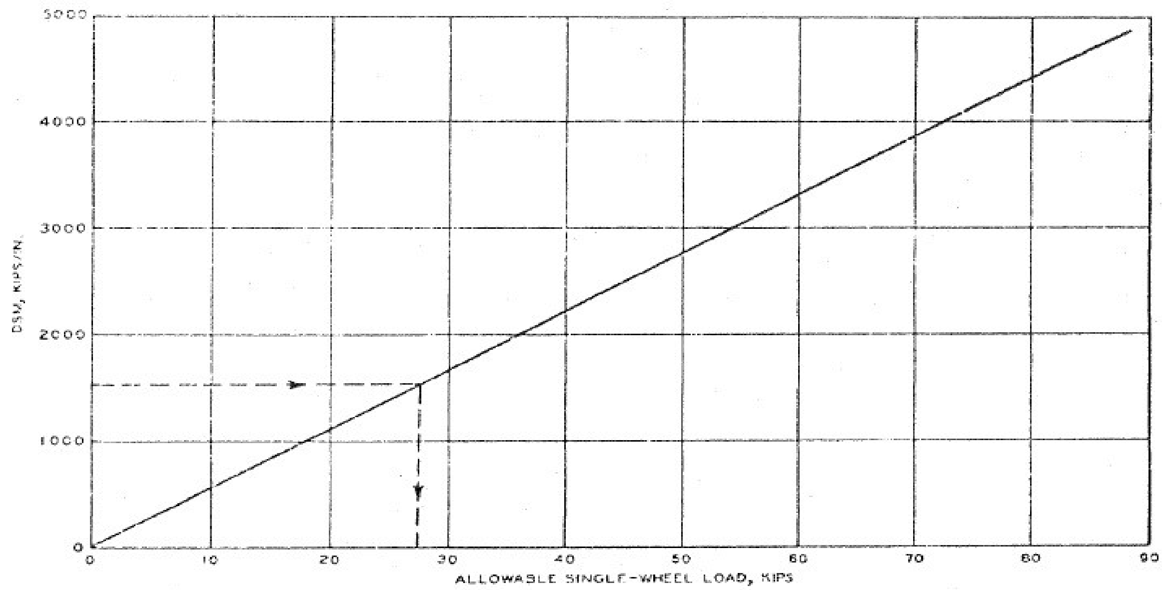


Figure 3-9. Evaluation curve for rigid pavement

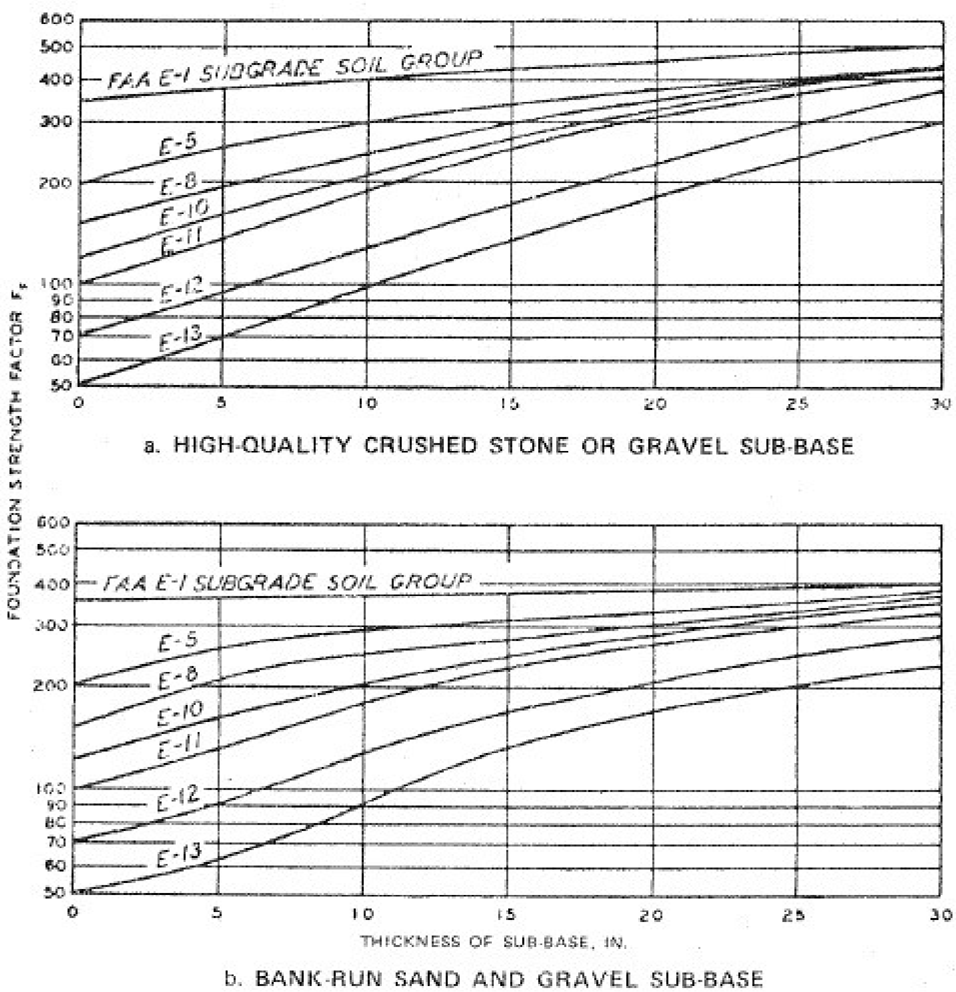


Figure 3-10. $F_F$versus sub-base thickness

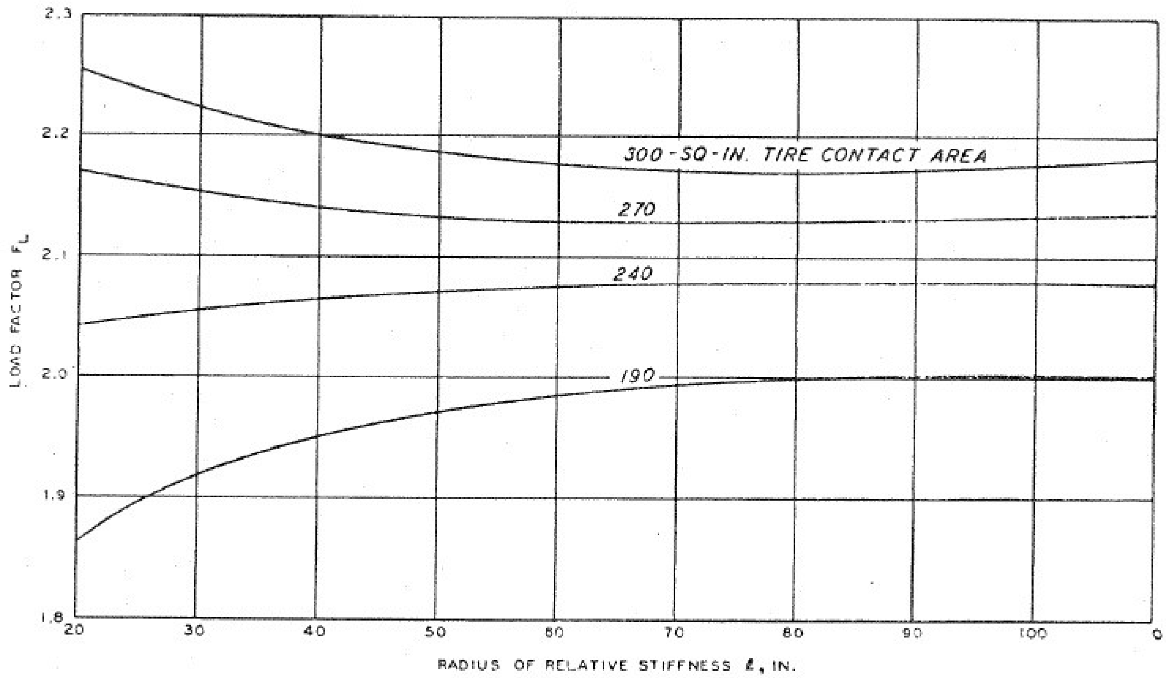


Figure 3-11. F_L versus l for single-wheel aircraft on rigid pavement

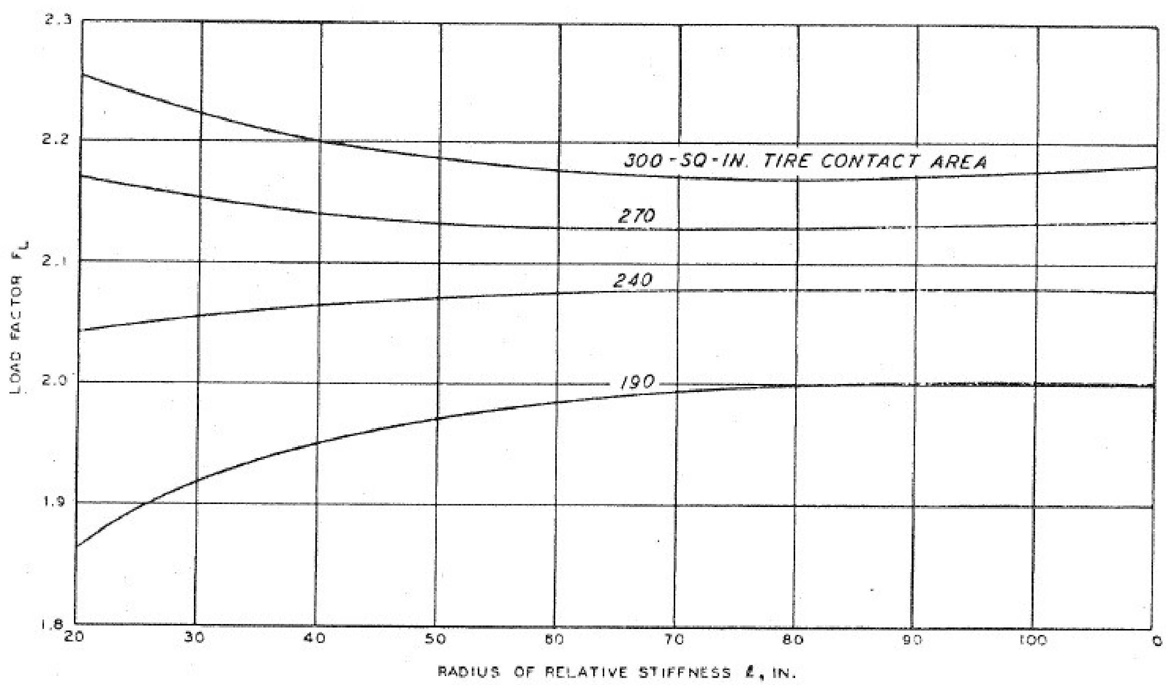


Figure 3-11. F_L versus l for dual wheel aircraft on rigid pavement

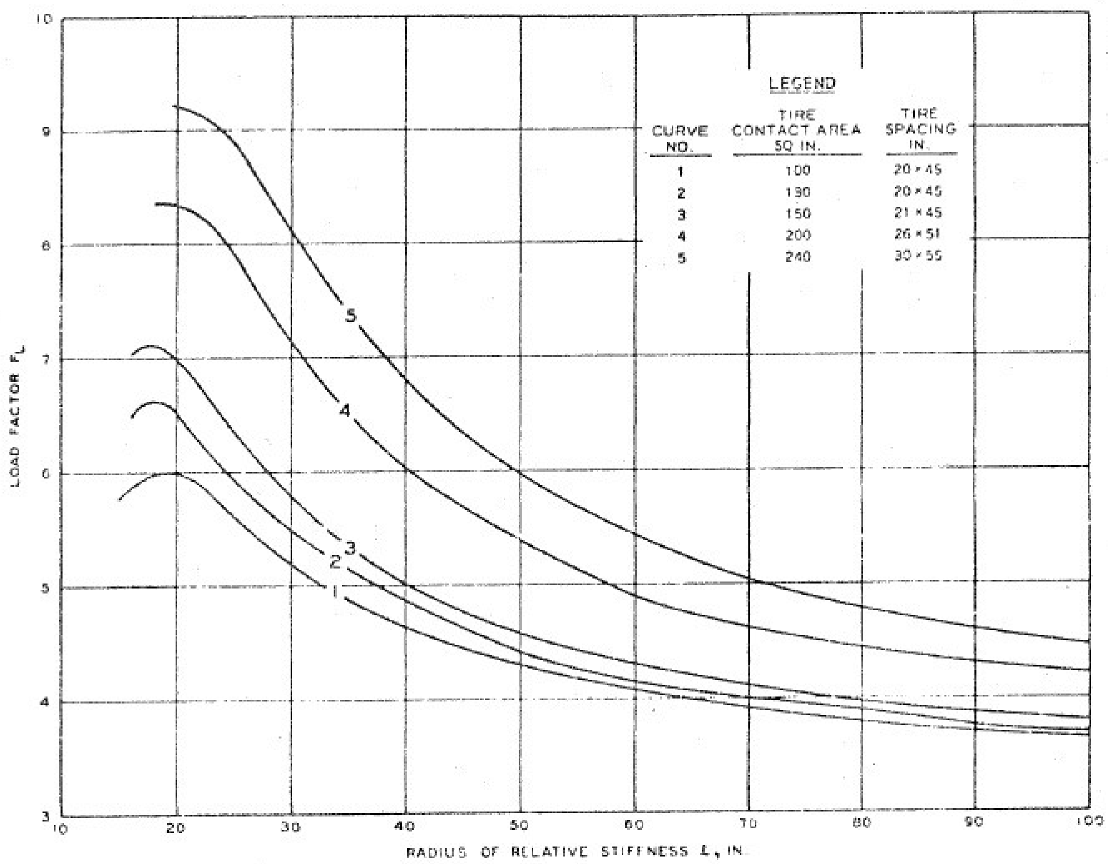


Figure 3-13. F_L versus l for dual tandem aircraft on rigid pavement

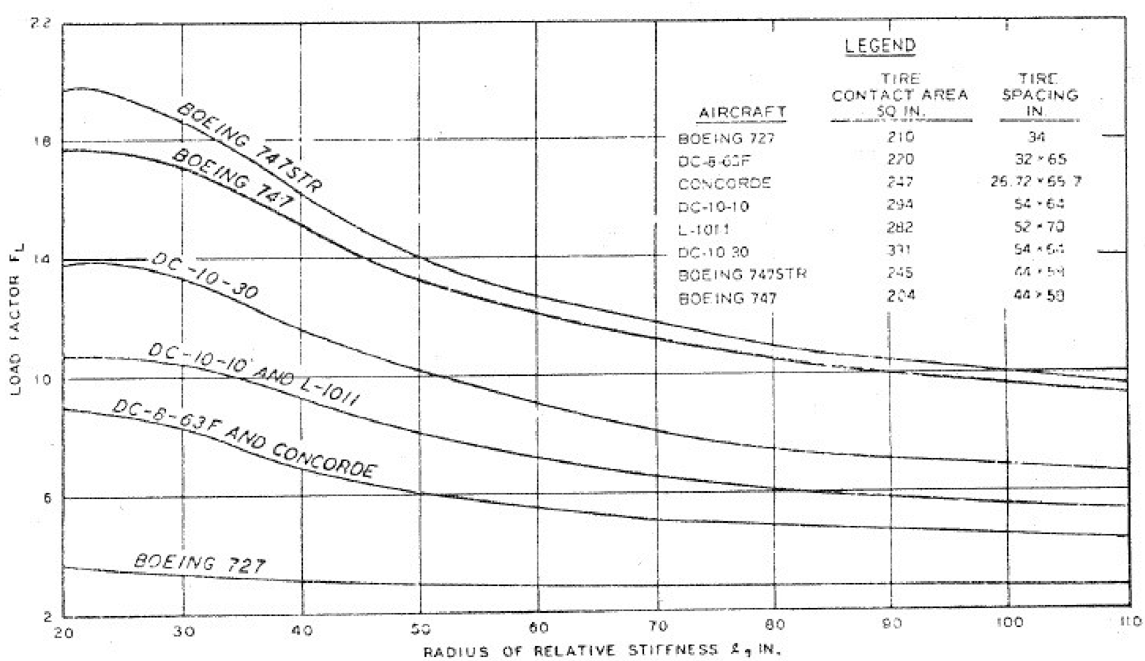


Figure 3-14. F_L versus l for various jet aircraft on rigid pavement

Step 5

Multiply the gross aircraft loading from Step by the appropriate traffic factor from Table 3-1 to obtain the allowable aircraft gross loading for critical areas for the pavement being evaluated. For the case of high-speed exit taxiways, the computed allowable gross load should be increased by multiplying by a factor of 1.18.

Step 6

The allowable loading obtained from Step 5 assumes that the rigid pavement being evaluated is structurally sound and functionally safe. The computed allowable loading should be reduced if one or more of the following conditions exist at the time of the evaluation:

1) the allowable load should be reduced by 10 per cent if 25 per cent or more of the slabs show evidence of pumping;
2) the allowable load should be reduced by 25 per cent if 30 to 50 per cent of the slabs have structural cracking associated with load (as opposed to shrinkage cracking, uncontrolled contraction cracking, frost heave, swelling soil, etc.). If more than 50 per cent of the slabs show load-induced cracking, the pavement should be considered failed;
3) the allowable loading should be reduced by 25 per cent if there is evidence of excessive joint distress such as continuous spalling along longitudinal joints, which would denote loss of the load-transfer mechanism.

3.6.5.15. Flexible pavement evaluation

Step 1

Using the DSM corrected for non-linear effects and adjusted to the standard temperature, determine the pavement system strength index Sp from Figure 3-15,

Step 2

Using the total thickness t of flexible pavement above the subgrade, compute the factor F_t for critical pavements as

$$F_t = 0.067t$$

or for high-speed taxiways as

$$F_t = 0.074t$$

Step 3

Using F_t determined in Step 2, enter Figure 3-16 and determine the ratio of the subgrade strength factor SSF to the pavement system strength index S_p

Step 4

Compute the subgrade strength factor SSF by multiplying SSF/ S_p by the value of S_p determined in step 1.

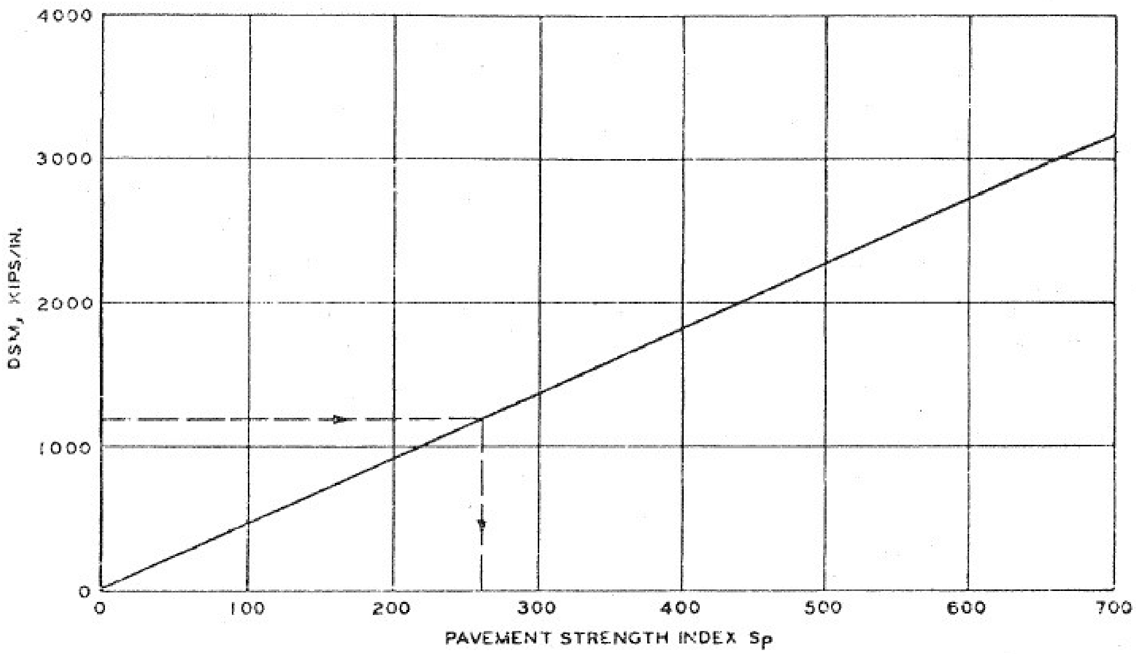


Figure 3-15. Evaluation curve for flexible pavement

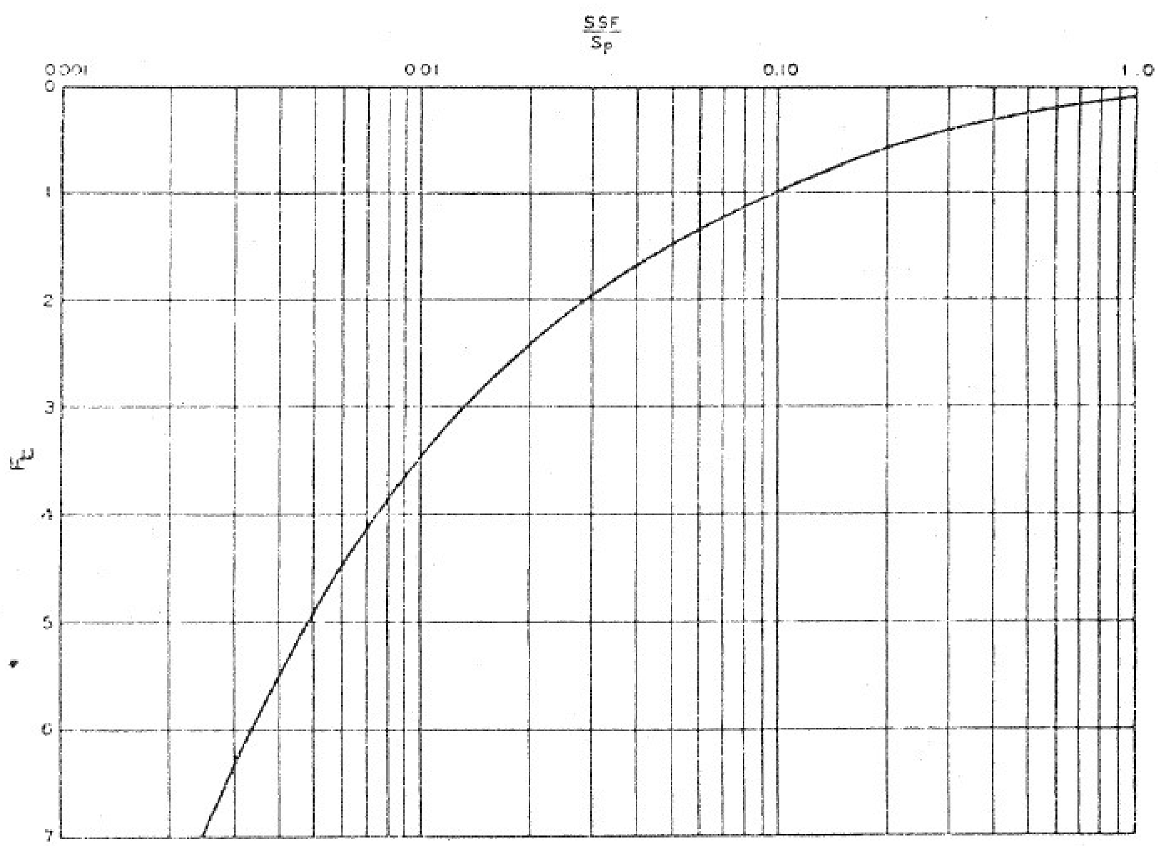


Figure 3-16. F_t versus $\frac{SSF}{S_P}$

Step 5

Evaluate the pavement for any aircraft desired as follows:

1) select the aircraft or aircraft main gear configuration for which the evaluation is being made and determine the tire contact area A of one wheel of the main landing gear (see Table 3-2);

2) select the annual departure level for each aircraft for which the evaluation is being made and determine the traffic factor a for each aircraft from Table 3-1;

3) compute the factor F_t for each aircraft for which the evaluation is being made for critical pavements as

$$F_t = \frac{t}{\alpha\sqrt{A}}$$

Or for high speed taxiways as

$$F_t = \frac{t}{0.9\ \alpha\sqrt{A}}$$

4) enter Figure 3-16 with Ft and determine SSF/Sp;

5) compute the pavement system strength index Sp for the aircraft being evaluated by dividing SS determined in Step by the ratio SSF/Sp determined in Sub step 4) above;

6) multiply Sp by the tire contact area A from Table 3G2 to obtain the equivalent single-wheel load (ES) of each aircraft for which the evaluation is being made;

7) enter Figure 3-17, 3-18, or 3-19 with the total pavement thickness t and determine the percentage of ESWL for the controlling number of wheels of the aircraft for which the evaluation is being made, i.e., if the aircraft has a dual-wheel assembly with a dual spacing of 26 in, use Curve 4 in Figure 3-17 or, if the evaluation is for the Boeing 747 STR aircraft, use the Boeing 747 STR curve in Figure 3-19;

8) the allowable gross aircraft Dad for the pavement being evaluated and for the traffic volume selected is then obtained from

$$\text{Allowable gross aircraft load} = \frac{\text{ESWL}}{\text{Per Cent ESWL}} \times \frac{1}{W_c} \times \frac{W_M}{0.95}$$

Where

ESWL = determined by sub step 6)
Per cent ESWL = determined by sub step 7)

Table 3-1. Traffic Factor for Flexible and Rigid Pavements

Aircraft	Traffic factor for cited annual departure level for 20-Year design life									
	1 200		3 000		6 000		15 000		25 000	
	Flexible	Rigid	Flexible	Rigid	Flexible	Rigid	Flexible	Rigid	Flexible	Rigid
30-kip single wheel	0.94	1.00	1.01	0.93	1.05	0.86	1.11	0.79	1.14	0.75
45-kip single wheel	0.94	1.00	1.01	0.92	1.05	0.85	1.11	0.78	1.14	0.75
60-kip single wheel	0.94	1.00	1.01	0.91	1.05	0.85	1.11	0.78	1.14	0.74
75-kip single wheel	0.94	1.00	1.01	0.91	1.05	0.84	1.11	0.77	1.14	0.74
50-kip dual wheel	0.84	0.97	0.87	0.88	0.89	0.82	0.91	0.75	0.92	0.72
75-kip dual wheel	0.84	0.96	0.87	0.87	0.89	0.82	0.91	0.75	0.92	0.72
100-kip dual wheel	0.84	0.96	0.87	0.87	0.89	0.81	0.91	0.75	0.92	0.72
150-kip dual wheel	0.84	0.95	0.87	0.86	0.89	0.81	0.91	0.74	0.92	0.71
200-kip dual wheel	0.84	0.95	0.87	0.86	0.89	0.81	0.91	0.74	0.92	0.71
100-kip dual tandem	0.78	0.99	0.79	0.89	0.80	0.83	0.81	0.77	0.82	0.73
150-kip dual tandem	0.78	0.98	0.79	0.88	0.80	0.82	0.81	0.76	0.82	0.73
200-kip dual tandem	0.78	0.97	0.79	0.88	0.80	0.82	0.81	0.75	0.82	0.72
300-kip dual tandem	0.78	0.95	0.79	0.87	0.80	0.81	0.81	0.75	0.82	0.72
400-kip dual tandem	0.78	0.95	0.79	0.86	0.80	0.81	0.81	0.74	0.82	0.71
Boeing 727	0.84	0.95	0.87	0.87	0.89	0.81	0.91	0.75	0.92	0.71
DC-8-63F	0.78	0.95	0.79	0.87	0.80	0.81	0.81	0.74	0.82	0.71
Boeing 747	0.70	0.97	0.70	0.88	0.705	0.82	0.71	0.75	0.71	0.72
DC-10-10	0.78	0.96	0.79	0.88	0.80	0.82	0.81	0.75	0.82	0.72
DC-10-30	0.78	0.96	0.79	0.87	0.80	0.82	0.81	0.75	0.82	0.72
L-1011	0.78	0.96	0.79	0.88	0.80	0.82	0.81	0.75	0.82	0.72
Concorde	0.78	0.94	0.79	0.86	0.80	0.80	0.81	0.74	0.82	0.71

W_C = number of controlling wheels used to determine the per cent

W_M = total number of wheels on all main gears of the aircraft (see Table 3-2) for which the evaluation is being made (does not include wheel on nose gear).

3.6.5.16. Summary. The evaluation procedure presented herein is what must be referred to as a first generation procedure. That is, further work is under way to extend the applicability of this procedure, and it will be updated as appropriate. In addition, research is under way which will establish the NDT evaluation procedure on a more theoretical basis and thus further enhance its applicability. The allowable loadings determined using the procedure presented herein are within acceptable limits of accuracy as compared with those determined using other recognized evaluation procedures. This procedure has the added advantages of being less costly, presenting less interference to normal airport operations, and providing the evaluating engineer with much more data on which to base his decisions. Also, in addition to their utility for arriving at allowable aircraft loading, the DSM values are useful for qualitative comparisons between one pavement area and another (DSM values on flexible pavements should not be compared with those on rigid pavements) and for locating areas which may show early distress and which may warrant further investigation. As more experience is gained with the NDT techniques and interpretation of data, it is envisioned that many other uses of the concept will emerge.

Table 3-2. Aircraft tire contact areas and total number of main gear wheels

Aircraft	Tire Contact Area cm^2	Tire Contact Area in^2	Total Number of Main Gear Wheels	Aircraft	Tire Contact Area cm^2	Tire Contact Area in^2	Total Number of Main Gear Wheels
30 kip single wheel	1 226	190	2	100 kip dual tandem	645	100	8
45 kip single wheel	1 548	240	2	150 kip dual tandem	839	130	8
60 kip single wheel	1 741	270	2	200 kip dual tandem	968	150	8
75 kip single wheel	1 935	300	2	300 kip dual tandem	1 290	200	8
50 kip dual wheel	968	150	4	400 kip dual tandem	1 548	240	8
75 kip dual wheel	1 032	160	4	Boeing 727	1 355	210	4
100 kip dual wheel	1 097	170	4	DC-8-63F	1 419	220	8
150 kip dual wheel	1 419	220	4	Boeing 747	1 316	204	16
200 kip dual wheel	1 677	260	4	Boeing 747 STR	1 580	245	16
				DC-10-10	1 897	294	8
				DC-10-3	2 135	331	10
				L-1011	1 819	282	8
				Concorde	1 593	247	8

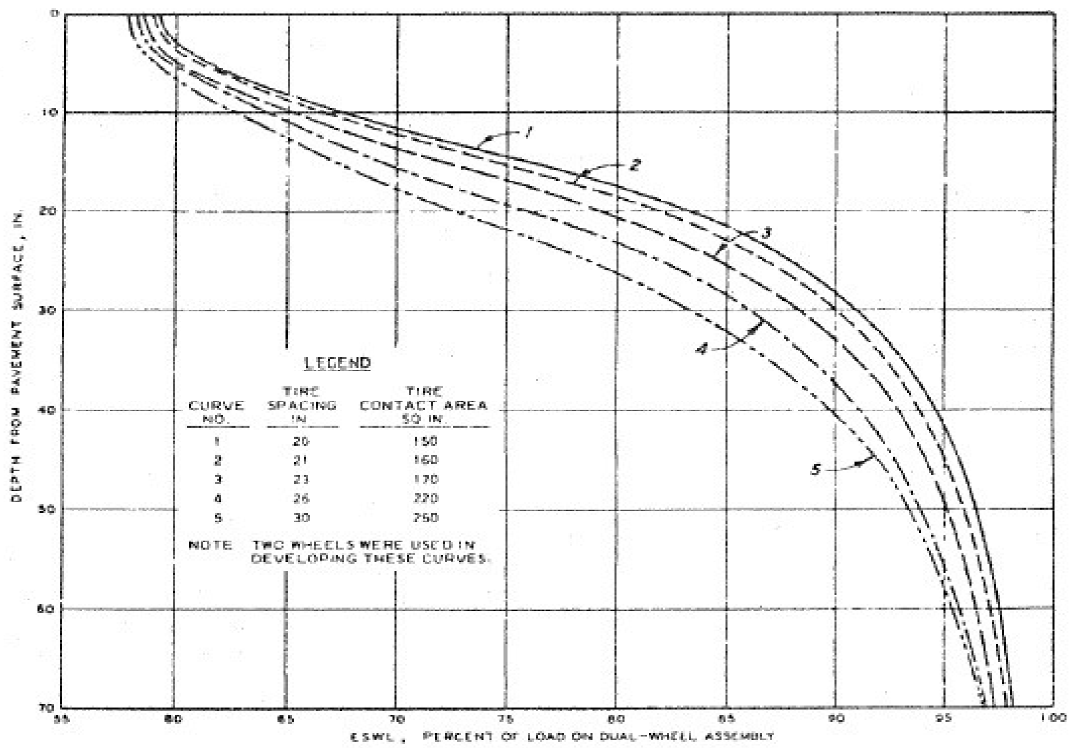


Figure 3-17. ESWL curves for dual wheel aircraft on flexible pavement

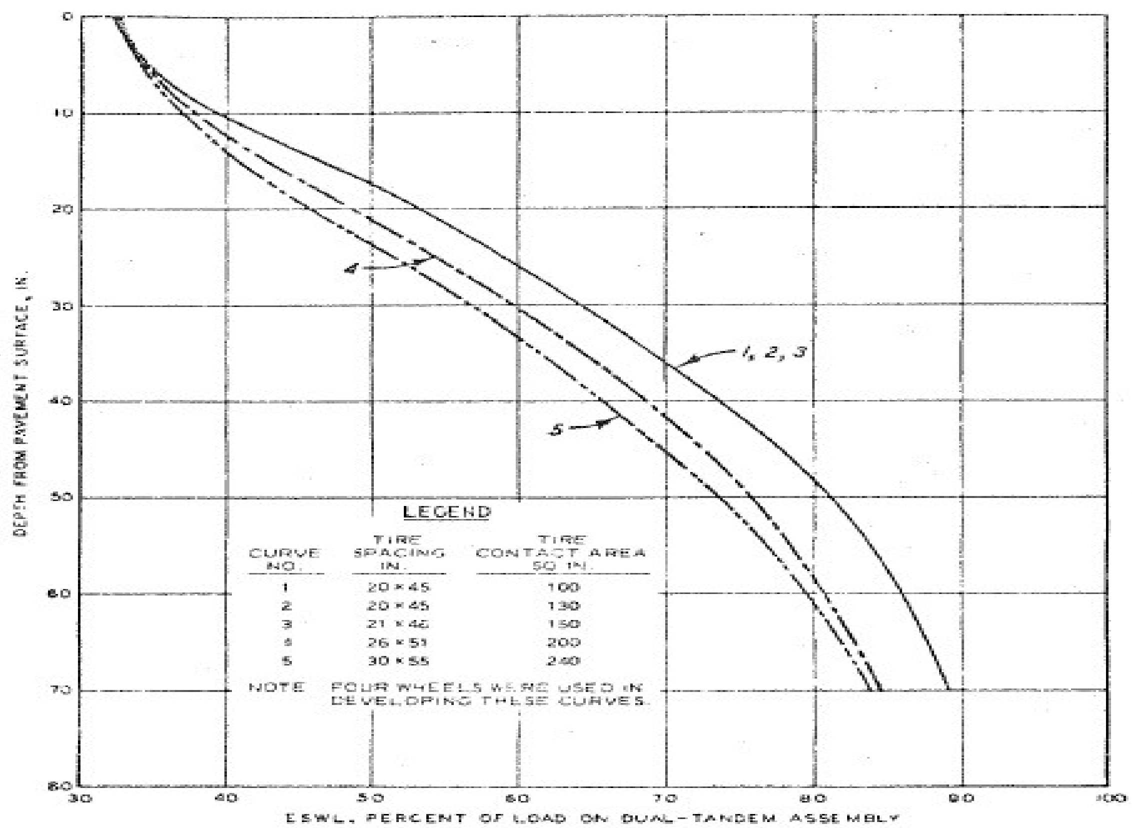


Figure 3-18. ESWL curves for dual tandem aircraft on flexible pavement

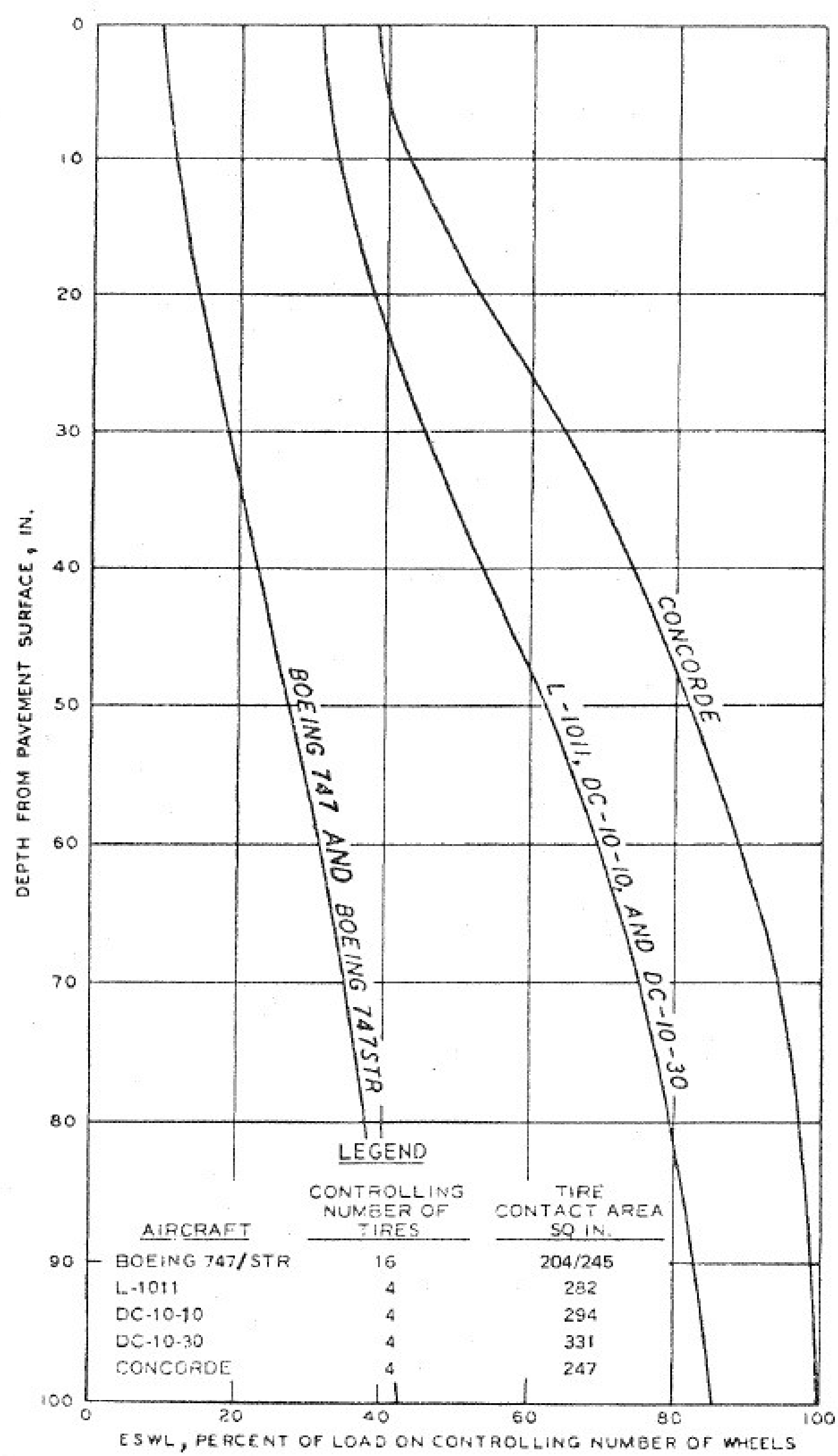


Figure 3-19. ESWL curves for various jet aircraft on flexible pavement

CHAPTER 4: - STATE PRACTICES FOR DESIGN AND EVALUATION OF PAVEMENTS

4.1. Canadian Practice

4.1.1. Scope

4.1.1.1. This section briefly outlines Transport Canada practices for the design and evaluation of airport pavements. Further details are available in Transport Canada's technical manual series. The practices described have evolved from Transport Canada's experience as the operator of all major civil airports in Canada. Most airport sites in Canada are subject to seasonal frost penetration and the design and evaluation practices described are oriented to this type of environment. The practices described do not apply to pavements constructed in permafrost regions where special design considerations are required. The practices outlined do not cover several topics which are associated with and essential to the design of pavement structures. Included in this category are pre-engineering studies such as soils, materials and topographic surveys, and design considerations such as pavement embankment stability and drainage. It should also be noted that the design of pavement structures is often greatly influenced by considerations related to cost, construction feasibility and airport operations.

4.1.2. Pavement design practices

Partial frost protection

4.1.2.1.1. Unless otherwise justified by a life cycle cost analysis, the thickness of pavements constructed on frost susceptible subgrades must not be less than the partial frost protection requirement given in Figure 4-1. The frost susceptibility of subgrades is assessed on the basis of subgrade soil gradation as shown in Figure 4-2. The partial frost protection requirement given in Figure 4-1 is a function of site freezing index. For a given winter period, this index in ^{o}c -days is calculated as the sum of average daily temperatures in ^{o}c, for each day over the freezing season, with below 0^{o}c temperatures taken as positive and above 0^{o}c temperatures taken as negative. The site freezing index used in Figure 4-1 is a ten-year average. The thickness requirements of Figure 4-1 are not sufficient to prevent excessive differential frost heaving when highly frost susceptible soils exist in pockets in an otherwise non-frost susceptible subgrade. This situation requires additional design measures, such as excavation of the frost susceptible soil to a suitable depth and replacement with material similar to the surrounding subgrade.

Flexible pavement design curves

4.1.2.2. A flexible pavement design curve for a given aircraft is a plot of pavement thickness required to support the aircraft loading as a function of subgrade bearing strength. The equation utilized to generate this design curve is:

$$S = (ESWL)\quad(c_1 10^{-c_2 t})$$

Where:

S = subgrade bearing strength (kN) as discussed in 4.1.3.3

ESWL = equivalent single wheel load of the design aircraft loading (kN)

t = pavement equivalent granular thickness (cm) as discussed in 4.1.3.1

c_1, c_2 = factors depending on contact area of ESWL,given in Figure 4-3.

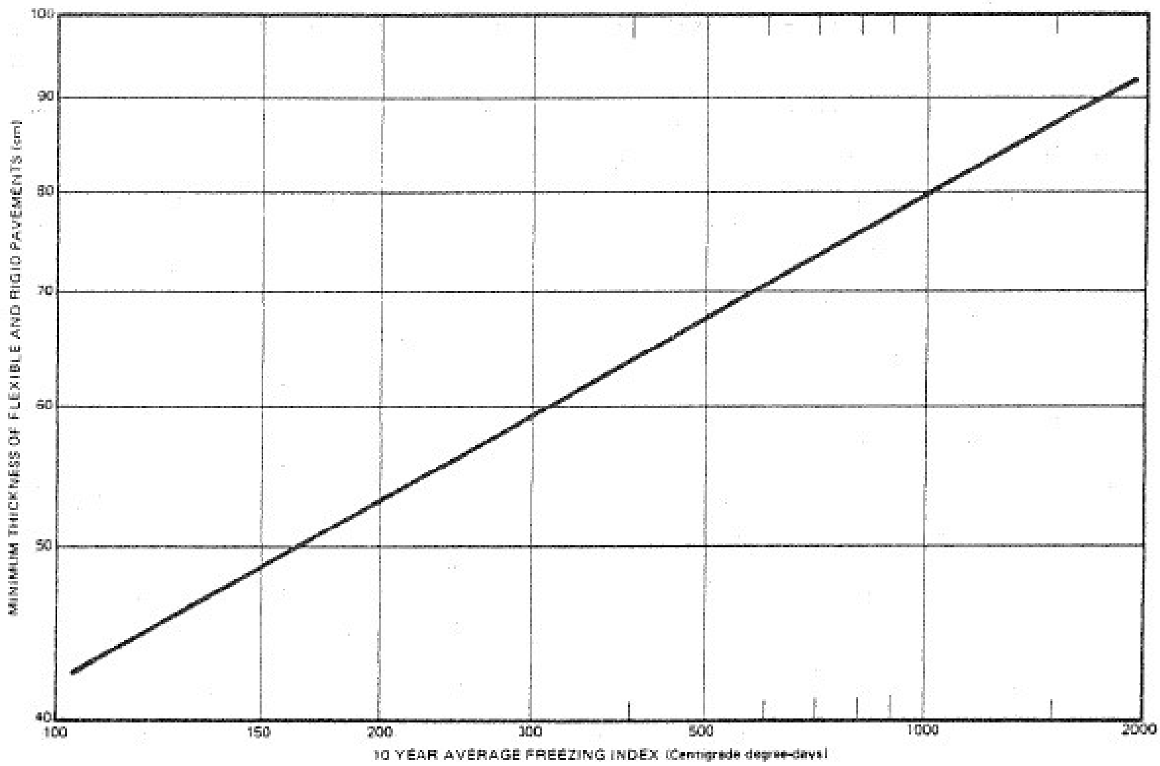


Figure 4-1. Partial frost protection requirements

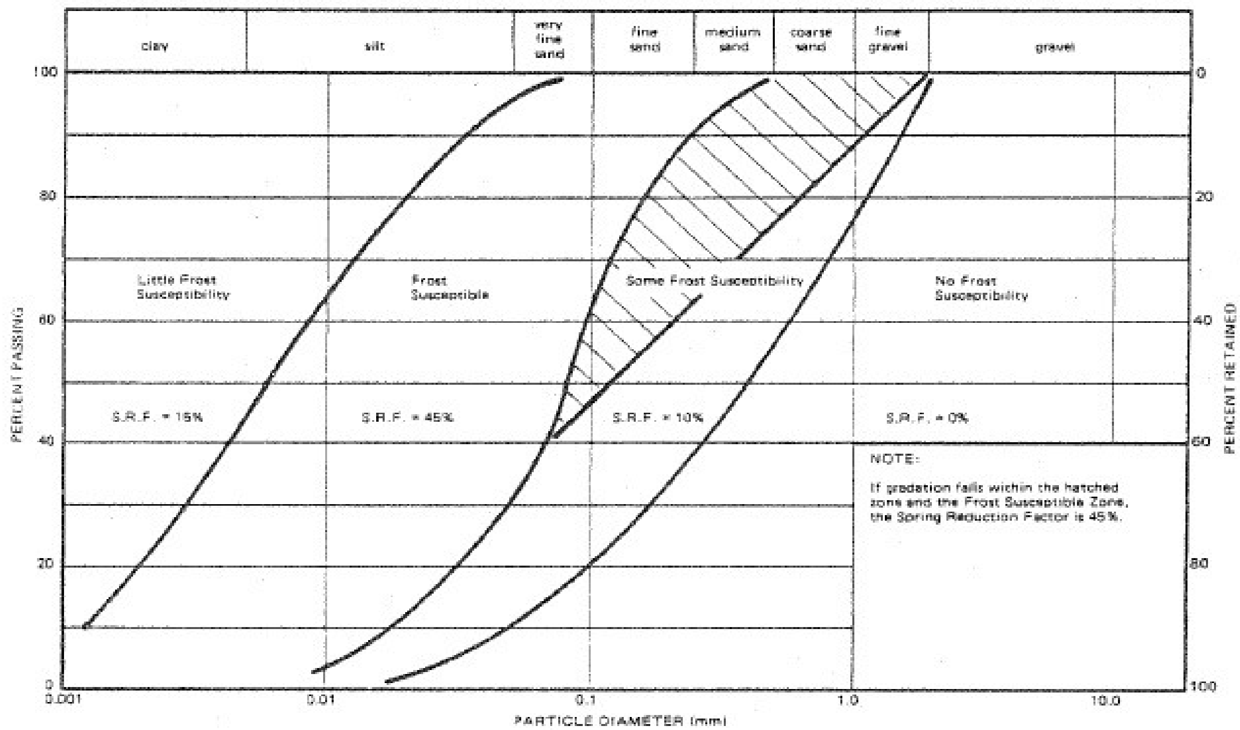


Figure 4-2. Subgrade frost susceptibility and spring reduction factor (S.R.F)

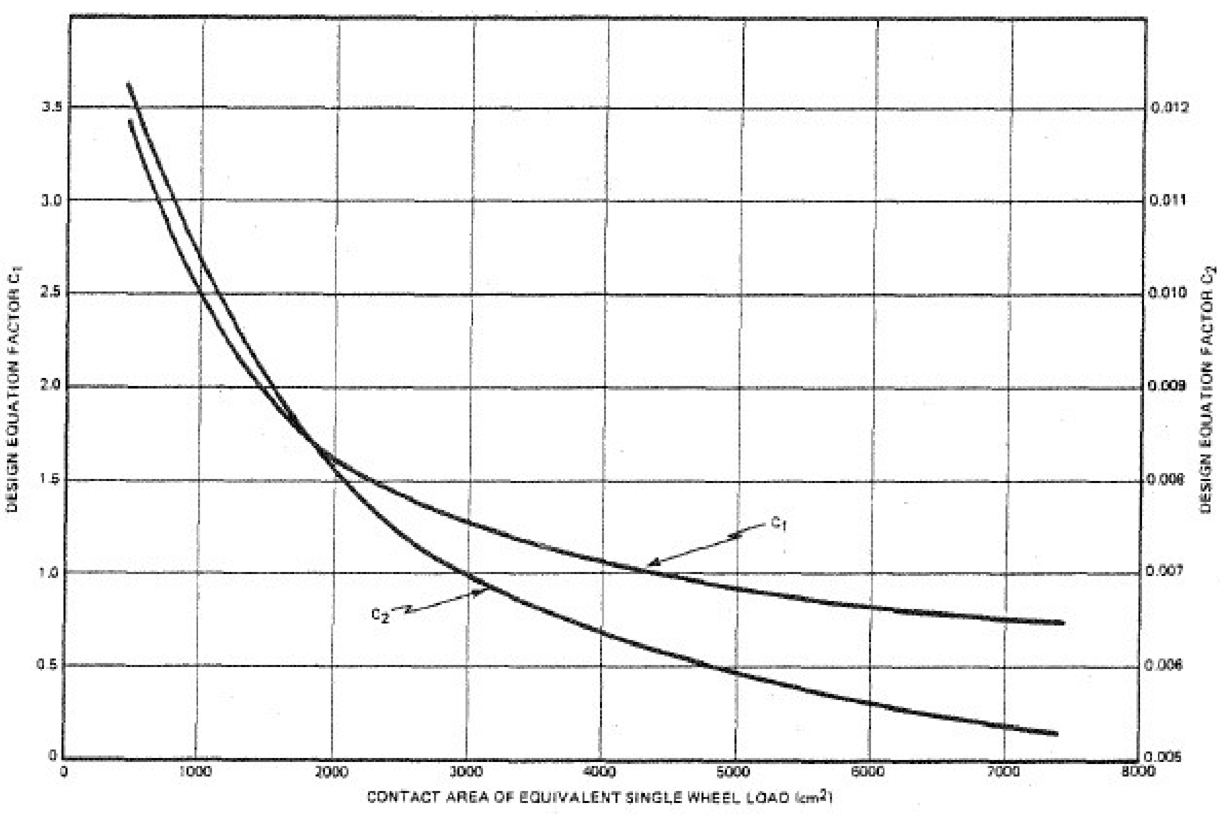


Figure 4-3. Design equation factors C_1 and C_2

Rigid Pavement design curves

4.1.2.2.1. A rigid pavement design curve for a given aircraft is a plot of concrete slab thickness required to support the aircraft loading as a function of bearing modulus of the surface on which the slab rests. Slab thickness required to support an aircraft loading is based on limiting to 2.75MPa the flexural stress occurring at the bottom of the slab directly under the centre of one tire of the aircraft gear. The stress calculations are carried out according to the Westergaard analysis for interior slab loading conditions using a computer programme similar to the one in Appendix 2.

Design curves for standard gear loadings

4.1.2.2.2. Airport pavements are usually designed for a group of aircraft having similar loading characteristics rather than for a particular aircraft. For this purpose a series of 12 standard gear loadings were defined to span the range of current aircraft loadings. Flexible and rigid pavement design curves for these standard gear loadings are given in Figures 4-5 and 4 -6. To compare the loading of a particular aircraft to the standard gear loadings, the flexible and rigid pavement design curves for the aircraft are superimposed over those for the standard gear loadings. Based on this method of comparison, Table 4-1 lists various aircraft and the standard gear loadings to which they are equivalent. The standard gear loading which is equivalent to a given aircraft loading is referred to as the "load rating" for that aircraft (ALR).

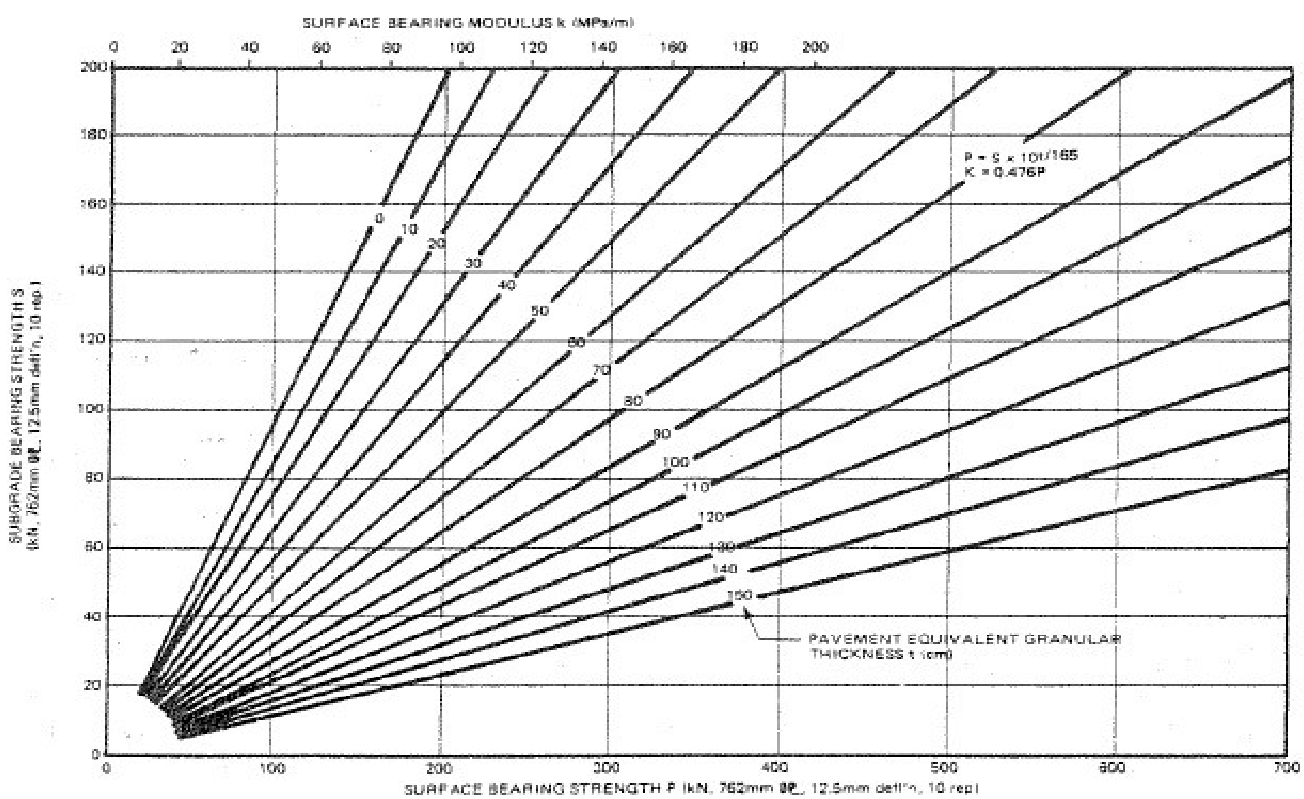


Figure 4-4. Surface bearing strength and bearing modulus as a function of subgrade bearing strength and pavement equivalent granular
thickness

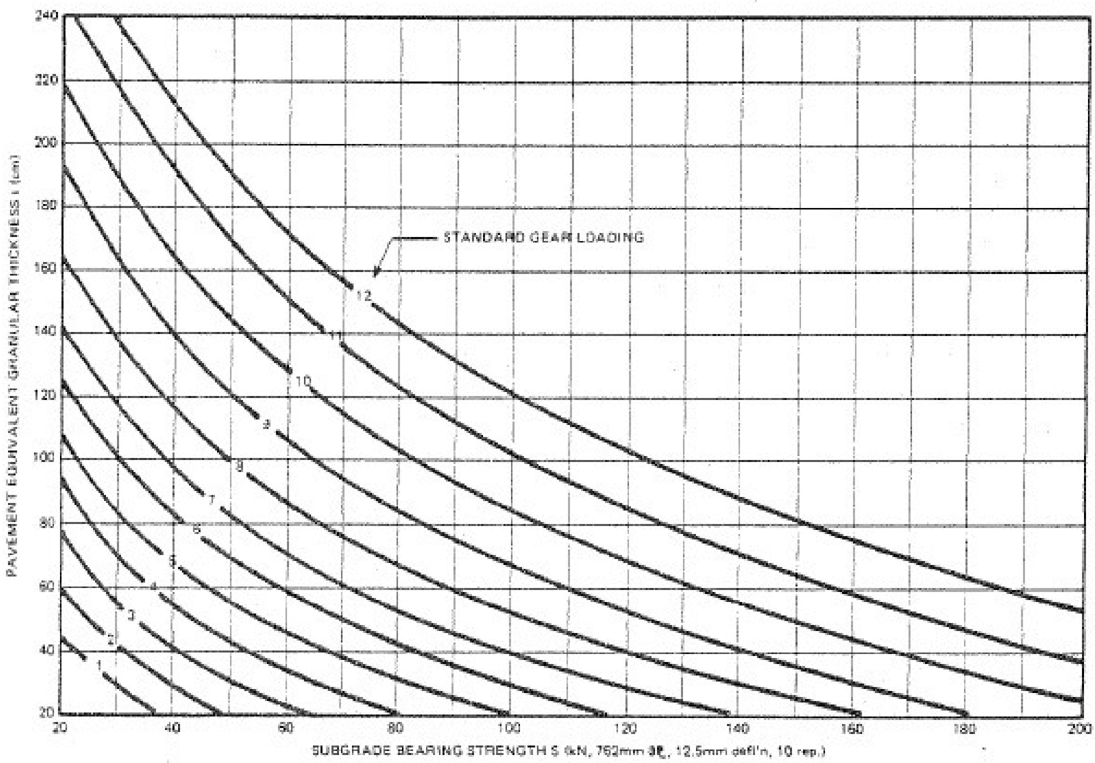


Figure 4-5. Flexible pavement design curves for standard gear loadings

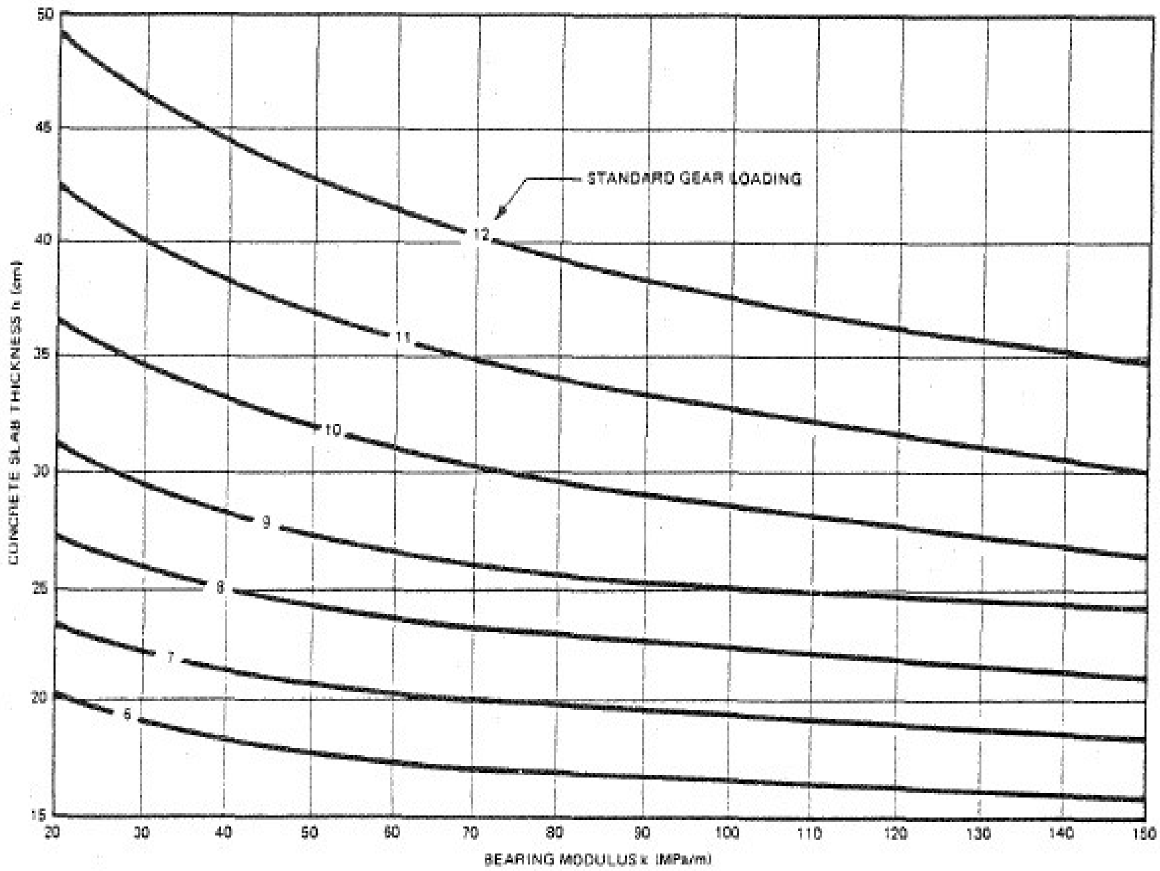


Figure 4-6. Rigid pavement design curves for standard gear loadings

4.1.2.2.3. The steps followed to determine asphalt pavement thickness requirements are

a. determine design loading (ALR) for the pavement on the basis of traffic studies and projections;

b. determine subgrade bearing strength as discussed in 4.1.3.3;

c. determine from Figure 4-5 pavement equivalent granular thickness requirement :for the design load rating;

d. determine the pavement thickness required for partial frost protection in accordance with 4.1.2.1; and

e. the pavement thickness provided will be as determined in c), or as determined in d), whichever is greater. In making the comparison, the equivalent granular thickness determined in c) must be converted to actual pavement thickness as discussed in 4.1.3.1.

Table 4-1 Aircraft load ratings

AIRCRAFT	TIRE PRESSURE (MPa)	WEIGHT (kN) (MAX) (MIN)	NOMINAL	AIRCRAFT LOAD RATINGS (ALR)							
				FLEXIBLE PAVEMENT AT S VALUE OF (kN)				RIGID PAVEMENT AT K VALUE OF (MPa/m)			
				50	90	130	180	20	40	80	150
B707-320	1.24	1500 800	10.7 7.6	10.7 7.3	10.7 7.2	10.5 7.2	10.4 7.2	10.5 7.6	10.3 7.6	10.2 7.2	10.1 7.1
B727-100-200	1.35	930 500	11.1 8.7	10.3 7.0	10.7 7.2	10.9 7.5	11.1 7.8	10.5 8.3	10.8 8.4	11.1 8.4	11.3 8.7
B737-100-200	1.02	450 300	8.0 6.5	7.5 5.8	7.5 6.0	7.5 6.4	7.5 –	7.8 6.2	7.7 6.4	7.9 6.4	8.0 6.5
B747-100-200	1.40	3600 2000	11.1 8.4	11.1 8.0	10.9 7.9	10.9 8.0	10.5 8.0	11.0 8.4	10.9 8.3	10.8 8.0	10.8 8.0
B767-200	1.20	1400 800	9.8 7.8	9.8 6.9	9.4 7.0	9.2 6.9	9.2 6.9	9.7 7.3	9.5 7.1	9.3 6.9	9.0 6.5
DC 6B	0.72	470 300	7.9 6.1	7.9 5.6	7.6 5.6	7.2 5.8	– –	7.9 6.0	7.9 6.1	7.9 6.1	7.9 6.1
DC-8-62-63	1.35	1600 800	11.2 7.9	11.0 7.5	11.0 7.6	11.1 7.7	11.2 7.7	10.9 7.9	10.9 7.9	10.9 7.7	10.9 7.5
DC-9-21-32	1.00	485 300	8.7 6.8	8.1 6.0	8.1 6.1	8.2 6.1	8.2 6.1	8.3 6.7	8.4 6.7	8.6 6.7	8.7 6.8
DC-10-20-30-40	1.21	1970 1200	11.0 7.8	11.0 7.0	10.9 6.6	10.8 6.3	10.7 6.0	11.0 7.8	11.0 7.7	10.8 7.5	10.8 7.2
A300-B2-B4	1.25	1480 1000	10.5 8.6	10.5 8.5	10.5 8.6	10.4 8.2	10.2 8.0	10.3 8.6	10.3 8.5	10.2 8.2	10.2 8.0
L1011-100-200	1.25	2080 1400	11.1 9.2	11.1 9.2	11.0 8.7	10.9 8.3	10.5 8.3	10.8 9.2	10.5 9.1	10.5 8.9	10.5 8.5
CONCORDE	1.27	1750 1000	11.8 9.0	11.4 9.0	11.6 8.9	11.7 8.7	11.8 8.5	11.3 9.0	11.3 9.0	11.4 8.7	11.4 8.4
HERCULES C-130	0.69	689 400	8.7 6.7	8.3 6.0	8.0 6.0	8.0 6.0	8.0 –	8.6 6.5	8.6 6.7	8.7 6.6	8.7 6.6
BAC-1-11-400	0.97	390 250	8.5 6.2	7.4 5.5	7.5 5.9	7.4 5.8	– –	8.0 6.0	8.1 6.2	8.3 6.2	8.5 6.2
CONVAIR 640	0.52	280 200	6.0 5.0	5.8 4.3	5.0 4.0	– –	– –	5.9 5.2	6.0 5.0	5.8 5.0	5.6 4.9

To determine aircraft load ratings at intermediate weights, interpolate linearly between the ALR values listed for minimum and maximum weights.

To determine aircraft load ratings at subgrade bearing strength (S) or bearing modulus (k) other than those listed, interpolate between the ALR values shown.

4.1.2.3. The thickness of pavement component layers will depend on tire pressure to be provided for, as outlined in the following table.

Pavement layer design thickness (cm)

Pavement layer	Design tire pressure (MPa)			
	Less than 0.4	0.4 to 0.7	0.7 to 1.0	Greater than 1.0
Asphaltic concrete	5.0	6.5	9.0	10.5
Cr Gravel or Cr Stone Base	15	23	23	30
Selected Granular Sub-base	As necessary to provide total thickness required			

Rigid pavement thickness requirements

4.1.2.4.The steps followed to determine rigid pavement thickness requirements are:

a) determine design loading (ALR) for the pavement on the basis of traffic studies and projections;

b) determine total pavement thickness required for partial frost protect on in accordance with 4.1.2.1;

c) estimate concrete slab thickness that will be required;

d) determine required base thickness by subtracting slab thickness from total pavement thickness determined in b);

e) determine bearing modulus at top of base course as discussed in 4.1.3.4;

f) determine concrete pavement slab thickness required for this bearing modulus from Figure 4-6; and

g) using the slab thickness determined in f) as a new estimate of requirements, repeat steps c) to f) until the slab thickness determined in f) equals that assumed in c).

4.1.2.4.1. The minimum base course layer provided is 15 cm, even if not required for frost protection. With pavements designed for a load rating of 12, the minimum base course normally provided is 20 cm of cement stabilized material. These minimum thicknesses are placed over selected granular sub-base material when thicker base layers are required for frost protection purposes.

4.1.2.4.2. The pavement design practices outlined above, and the evaluation practices outlined below, assume that the pavement is constructed to standard specifications governing the quality of pavement construction materials and workmanship. If standard specification requirements are not met, some adjustments based on engineering judgment may be required to the design and evaluation practices outlined. Tables 4-2, -3 and 4-provide some construction requirements considered essential to normal design and evaluation practices.

4.1.3. Pavement evaluation practices

Pavement thickness and equivalent granular thickness

4.1.3.1. The evaluation of pavement structures for aircraft loadings requires accurate information on the thickness of layers within the structure, and the physical properties of the materials in these layers. A bore hole survey is conducted to determine this information when it is not available from existing construction records. Equivalent granular thickness is a term applied to flexible pavement structures, and is the basis for comparing pavements constructed with different thicknesses of materials having different load distribution characteristics. The equivalent granular thickness is computed through the use of the granular equivalency factors for pavement construction materials listed in Table 4-5. The granular equivalency factor of a material is the depth of granular base in centimeters considered equivalent to one centimeter of the material on the basis of load distribution characteristics. The values given in Table 4-5 are conservative and actual granular equivalency factors are normally higher than the values listed. To determine the equivalent granular thickness of flexible pavement structure, the .depth of each layer in the structure is multiplied by the granular equivalency factor for the material in the layer. The pavement equivalent granular thickness is the sum of these converted layer thickness.

Table 4-2. Compaction requirements

Layer	Reference Density	Compaction Required % of Reference Density
Embankment Fill:		
cohesive soil	ASTM D 1557	90
non-cohesive soil	ASTM D 1557	95
Subgrade Surface: (1)		
cohesive soil	ASTM D 1557	93
non-cohesive soil	ASTM D 1557	98
Sub-Base	ASTM D 1557	98
Base Course	ASTM D 1557	100
Asphaltic Concrete	ASTM D 1559	98

Note: (1) Compaction of subgrade surface is specified 15mm deep in cohesive soil and 30mm deep in non-cohesive soil.

Table 4-3. Asphaltic and Portland cement concrete mix requirements

Property		Min.	Max.
ASPHALTIC CONCRETE			
Marshall Stability	(kN)	6.75	
Marshall Flow Index	(mm)	2	4
Air Voids	(%)	3	5
Voids in Mineral Aggregate:	(%)		
12.5mm max. sized aggregate		15	
25mm max. sized aggregate		13	
Immersion Loss	(%)		25
PORTLAND CEMENT CONCRETE			
Cement Content	(kg/m^3)	280	310
Water/Cement Ratio			0.45
Avg. 28 Day Flexural Strength	(MPa)	4.0	
Slump	(mm)	10	40
Entrained Air Content	(%)	4	6

Table 4-4. Aggregate requirements

Property			ASTM Test Method	Sub-Base	Base	Asphaltic Concrete Lower Course	Asphaltic Concrete Upper Course
Gradation (min-max)	% passing sieve size (mm)	75	C136	100			
		50	C136		100		
		38.1	C136		70-100		
		25	C136			100	
		19	C136		50-75		
		12.5	C136			70-85	100
		9.5	C136		40-65		
		4.75	C136		30-50	40-65	55-75
		2.00	C136			30-50	35-55
		0.425	C136	0-30	10-30	15-30	15-30
		0.180	C136			5-20	5-20
		0.075	C117	0-8	3-8	3-8	3-8
Crushed Content (% min)			—	—	60 (1)	60	60
Liquid Limit (% max)			D423	25	25	—	—
Plasticity Index (% max)			D424	6	6	—	—
Sand Equivalent (% min)			D2419	—	—	50	50
Abrasion Loss (% max)			C131 (2)	50	45	25	25
Soundness Loss (% max)			C88 (3)			12 coarse agg. 16 fine agg.	

NOTES:
(1) Crushed aggregate not necessary for bases under P.C.C. slab.
(2) Text method C131 - use gradation 'A' for base course and gradation 'B' for asphaltic concrete aggregate.
(3) Test method C38 - use magnesium sulphate.

Table 4-5. Granular equivalency factors

Pavement Material	Granular Equivalency Factor
Selected granular sub-base	1
Crushed gravel or stone base	1
Waterbound Macadam base	1-1/2
Bituminous stabilized base	1-1/2
Cement stabilized base	2
Asphaltic concrete (good condition)	2
Asphaltic concrete (poor condition)	1-1/2
Portland cement concrete (good condition)	3
Portland cement concrete (fair condition)	2-1/2
Portland cement concrete (poor condition)	2

Table 4-6. Typical subgrade bearing strengths

Subgrade Soil Type	Usual Spring Reduction %	Subgrade Bearing Strength (kN)		
		Fall Range	Design Value	
			Fall	Spring
GW - well graded gravel	0	290-400	290	290
GP - poorly graded gravel	10	180-335	220	200
GM - gravel with silty fines	25	135-335	180	135
GC - gravel with clay fines	25	110-245	145	110
SW - well graded sand	10	135-335	180	160
SP - poorly graded sand	20	110-200	135	110
SM - sand with silty fines	45	95-190	120	65
SC - sand with clay fines	25	65-155	85	65
ML - silt with low liquid limit	50	90-180	110	55
CL - clay with low liquid limit	25	65-135	85	65
MH - silt with high liquid limit	50	25-90	40	20
CH - clay with high liquid limit	45	25-90	55	30

<u>Pavement bearing strength measurements</u>

4.1.3.2. Transport Canada practice is to conduct measurements of bearing strength on the surface of flexible pavements. Testing is not conducted until at least two years after construction to permit subgrade moisture conditions to reach an equilibrium state. The bearing strength of rigid pavements is not normally measured, as strengths calculated on the basis of slab thickness and estimated bearing modulus are considered sufficiently accurate. The standard measure of bearing strength is the load in kilo Newton which will produce a deflexion of 12mm after 10 repetitions of loading, when the load is applied through a rigid circular plate 762 mm in diameter. This definition applies for subgrade bearing strength as well as for measurements conducted at the surface of a flexible pavement. In actual practice, a variety of test methods are employed to measure bearing strength. These methods include both repetitive and non-repetitive plate load test procedures in which a variety of bearing plate sizes may be used. Benkelman beam testing procedures may be employed in place of plate load testing at small airports intended to serve light aircraft only. Transport Canada document AK-68-31 "Pavement Evaluation - Bearing Strength" details the test methods which may be used, and provides correlations for converting the results of these test methods to the standard measure of bearing strength defined above.

<u>Subgrade bearing strength</u>

4.1.3.3. When a bearing strength measurement has been made on the surface of flexible pavement, and the equivalent granular thickness of the pavement structure is known, the subgrade bearing strength at that location may be estimated from Figure 4-4. Subgrade bearing strength varies from location to location throughout a pavement area. In pavements subject to seasonal frost penetration, variation also occurs with time of year, with the lowest values reached during the spring thaw period. The subgrade bearing strength used to characterize a pavement area is the lower quartile, spring reduced value. The lower quartile value of several bearing strength measurements made throughout a pavement area is that value for which 75 percent of the measurements are greater in magnitude. It is calculated as x - 0.675s, where x is the average of measurements made and s is their standard deviation. For pavements subject of seasonal frost penetration, spring thaw conditions are estimated by applying a reduction factor to lower quartile subgrade bearing strengths derived from summer and fall measurements. The reduction factor applied depends on gradation of the subgrade soil as shown in Figure 4-2, and typical spring reduction factors based on soil classification are listed in Table 4-6. When the ground water table is within 1 meter of the pavement surface, the spring reduction factors listed in Table 4-6 are increased by 10 for each soil type. Subgrade bearing strengths are normally established at existing airports through bearing strength measurement programmes. Subgrade bearing strength values derived from measurements are used when designing new pavement facilities at the airport provided subgrade soil conditions are similar throughout the site. when designing or evaluating pavements at an airport where strength measurements have not been made, a value of subgrade bearing strength is selected from Table 4 -6 on the basis of subgrade soil classification.

<u>Rigid Pavement bearing modulus</u>

4.1.3.4. Bearing modulus is based on the load in Mega Newton which will produce a deflection of 1.25 mm when the load is applied through a rigid circular plate 762 mm in diameter. This load is then divided by the volumetric displacement of the plate at this deflection (0.57×10^{-3} m^3) to compute bearing modulus in units of mega pascals per metre. Rigid pavement bearing modulus is the bearing modulus at the surface of the base course on which the concrete slab rests. It is rarely measured directly for pavement design or evaluation purposes. Instead, bearing modulus at the top of the base course is estimated from Figure 4-4 on the basis of a subgrade bearing strength determined as discussed in 4.1.3.3, and the equivalent granular thickness of sub-base and base course provided between subgrade and concrete slab.

Pavement strength reporting

4.1.3.5. The two parameters governing strength of flexible pavements are pavement equivalent granular thickness (t) as discussed in 4.1.3.1 and subgrade bearing strength (S) as discussed in 4.1.3.3. Pavement strength is reported in terms of the Pavement Load Rating (PLR) which is determined by plotting the point on Figure 4-5 using the pavement t and S values as coordinates. The load rating reported for the pavement is the numerical value of the standard gear loading whose design curve falls immediately above this point. The two parameters governing the strength of a rigid pavement are bearing modulus (k) as discussed in 4.1.3.4, and concrete slab thickness (h). These values are plotted on Figure 4-6 to determine the load rating of rigid pavements in a manner similar to that for flexible pavements. A tire pressure restriction may be applied to flexible pavements. The restriction applied is the tire pressure for which the pavement asphalt and base course thickness satisfy design requirements, as given in 4.1.2.6. No tire pressure restrictions are applied for concrete pavements. Aircraft having a load rating (ALR) and tire pressure equal to or less than the values reported for a pavement structure are authorized to operate on the pavement without restriction. Proposed operations by an aircraft with a load rating or tire pressure exceeding reported values must be referred to the airport operating authority for an engineering and management assessment.

Composite pavement structures

4.1.3.6. A composite pavement structure is created when an existing pavement structure is overlaid for strengthening or resurfacing purpose. Composite pavement structures are evaluated as flexible or rigid pavements are accordance with the procedures below:

a) Asphalt overlay on flexible pavement
A flexible pavement overlaid with additional asphalt pavement layers is evaluated as a flexible pavement having an equivalent granular thickness determined as outlined in 4.1.3.1.
b) Asphalt overlay on rigid pavement
A rigid pavement receiving an asphalt overlay less than 25 cm in thickness is evaluated as rigid pavement, with the concrete slab and asphalt overlay thickness is converted to an equivalent single slab thickness as given in Figure 4-7. A rigid pavement receiving as asphalt overlay greater than 25cm in thickness is evaluated as flexible pavement with an equivalent granular thickness determined as outlined in 4.1.3.1.

c) Concrete overlay on flexible pavement
A flexible pavement overlaid with a concrete slab is evaluated as a rigid pavement with the flexible pavement structure forming the base for the concrete slab.

d) Concrete overlay on rigid pavement
A rigid pavement overlaid by a concrete slab is evaluated as a rigid pavement with the two slabs converted to an equivalent slab thickness as given in Figure 4-

7, except when a separation course greater than 15 cm is placed between the two slabs. When a separation course greater than 15 cm in thickness is used, the upper slab is considered to act independently as a single slab with the lower slab forming part of` the base.

Surface condition evaluation

4.1.3.7. In addition to pavement bearing strength evaluation and reporting, airport pavements are subject to an evaluation of surface conditions yearly at international airports and biennially at other airports. The surface condition evaluation programme consists of a visually based structural conditions survey, and quantitative measurements of roughness and friction levels on runway surfaces.

4.1.3.8. Structural condition surveys are conducted by an experienced pavements engineer or technician who visually inspects the pavements and reports on the extent and severity of observed pavement defects and distress features. On the basis of traffic levels and observed defects and distress features, an estimate is also provided for the year in which pavement rehabilitation should be programmed. A typical Structural condition survey report is shown in Figure 4-8.

4.1.3.9. Runway roughness measurements are conducted with a Road meter, a device which records vertical movements in an automobile as the vehicle is driven along the runway at 80 km/h. Roadmeter readings are converted to a Riding Comfort Index on a scale of 0 to 10 and plotted as shown in Figure 4-9 to provide a record of runway roughness development with time. The runway roughness performance chart illustrated in Figure 4-9 is used to assess when excessive roughness levels requiring rehabilitation will be reached.

4.1.3.10. Runway surface friction measurements (normal wet state) are currently conducted with a SAAB Surface Friction Tester. Measurements are conducted at a vehicle speed of 65 km/h using a treaded measuring tire inflated to 0.21 MPa pressure. He runway surface friction profiles obtained from these measurements, as illustrated in Figure 4-10, are used to determine the need for surface texturing or rubber removal programmes.

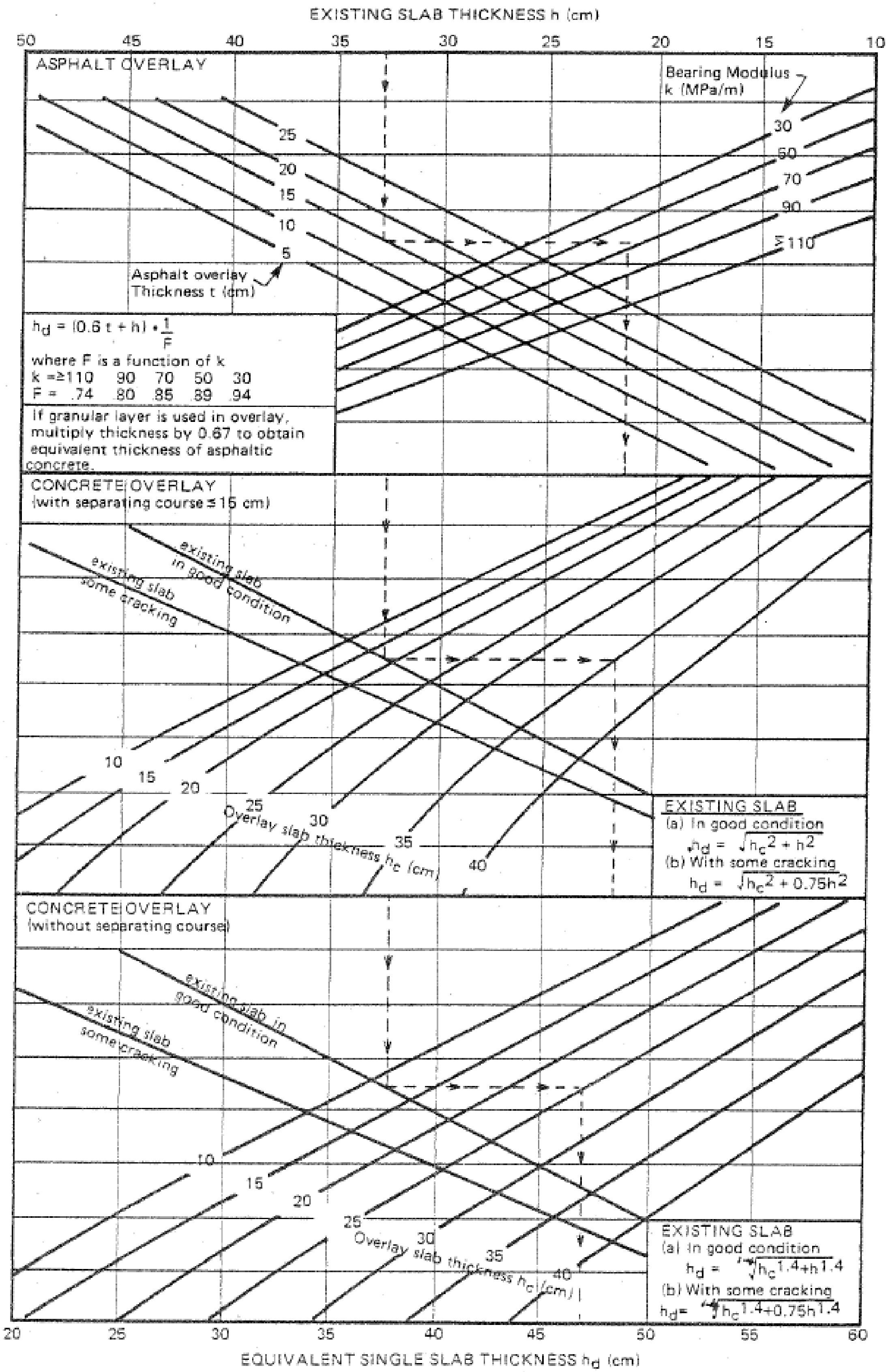


Figure 4-7. Equivalent single slab thickness of overlaid concrete slab

Airport: TERRACE, B.C.
Observer: D. JONES
Date: NOV. 22, 1979

Critical Aircraft Type: B 737
Operating Weight: 400 kN
Tire Pressure: 1.0 MPa A.L.R. ~ 9

PAVEMENT CONDITION SURVEY

Facility	Chainage From	Chainage To	Year Rehabilitation Reqd.	General Condition	Asphalt Surfaces: Alligator Cracking	Map Cracking	Transverse Cracking	Longitudinal Cracking	Rutting	Ravelling	P.C.C. Surfaces: Corner Cracking	Edge Cracking	Slab Cracking	Scaling	Surface Spalling	Joint/Crack Spalling	Joint Faulting	Joint Sealant Failure	All: Frost Heaving	Subgrade Settlement	Patching	Maintenance Needed
Rny 14-32	5+000	5+090	95	8							0	0	1/1	0/0	0/0	0/0	0/0	2	0/0	0/0	0	√
	5+090	6+739	90	7	0	0	1/1	1/1	0/0	1/1									0/0	0/0	1	
	6+739	6+829	95	8							0	1	0/0	0/0	0/0	0/0	0/0	2	0/0	0/0	0	√
Rny 08-26	5+000	5+060	85	5							1	2	3/1	1/1	0/0	1/3	3/1	3	0/0	1/1	2	√
	5+060	6+525	81	3	1	3	4/3	4/3	1/1	4/2									0/0	1/3	0	√
	6+525	6+585	85	5							1	2	3/2	1/1	0/0	1/3	3/1	3	0/0	1/1	2	√
Taxi A			81	3	4	2	4/2	4/3	1/1	4/1									0/0	1/1	2	√
Taxi B			90	7	0	0	1/1	1/1	0/0	1/1									0/0	0/0	0	
Taxi C			90	7	0	0	1/1	1/1	0/0	1/1									0/0	0/0	0	
Apron I			95	8							0	0	1/1	0/0	0/0	0/0	0/0	1	0/0	0/0	0	

General Condition Rating

10 9 8 7 6 5 4 3 2 1 0

very good | good | fair | poor | very poor

Pavement Defect Ratings

0: none 1: minor 2: moderate 3: major 4: extreme

Double Ratings

extent of defect / severity of defect

Figure 4-8. Pavement condition survey report

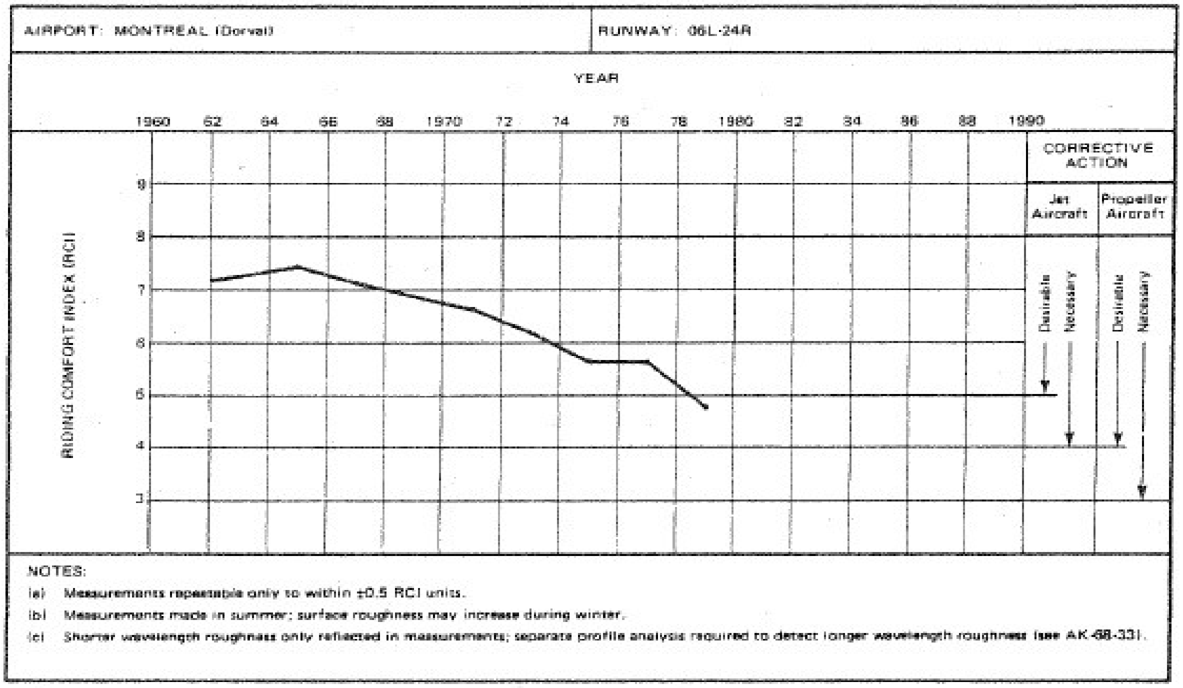


Figure 4-9. Runway roughness performance chart

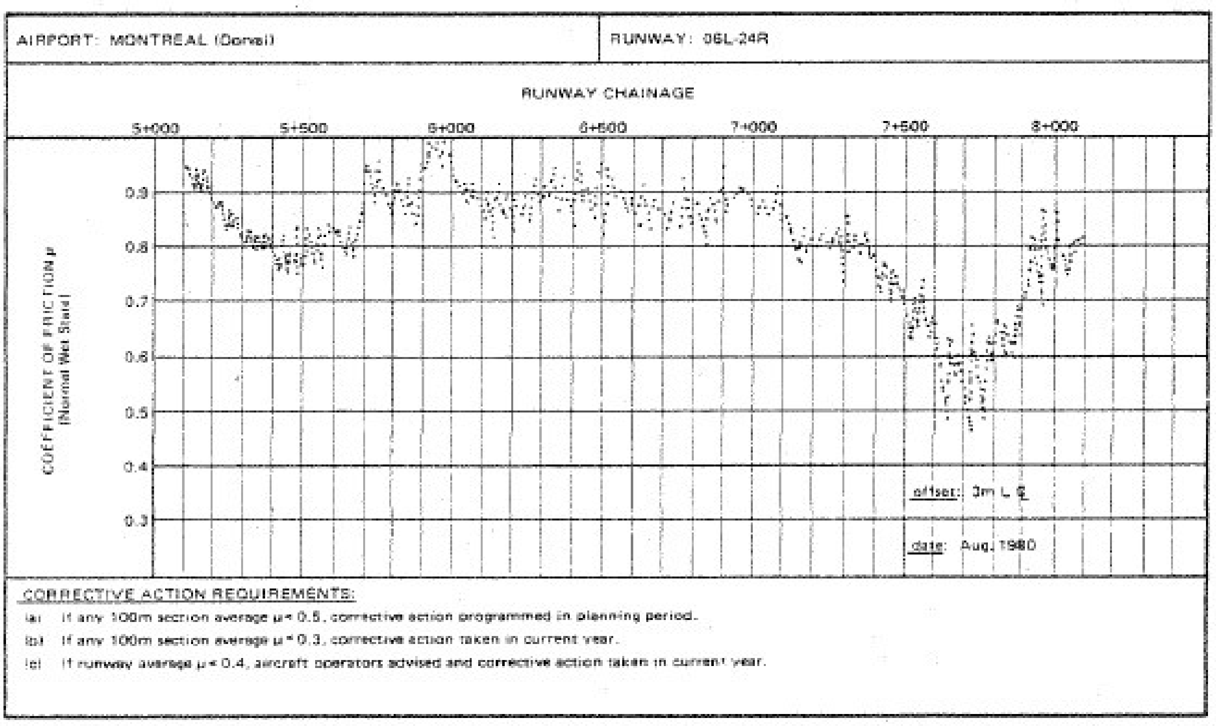


Figure 4-10. Runway coefficient of friction profile

4.2. French Practice

4.2.1. General

4.2.1.1. Definitions

a) Structure of pavement. A pavement normally comprises the following from top to bottom:
- a "surface layer" consisting of a "wearing course" and possibly a "binder course";
- a "base";
- a "sub-base"; and
- Possibly a lower sub-base or an improved subgrade.

b) Types of structures,

- A "flexible structure" consists only of courses of materials that have not been bound or treated with hydrocarbon binders.
- a "rigid structure" offers a wearing course made up of a Portland cement slab;
- a "semi-rigid structure" comprises a base treated with hydrocarbon binders; and
- a "composite (or mixed) structure" results from reinforcing a rigid structure with a flexible or semi-rigid structure.

c) Pavement types For the sake of simplification a distinction is made hereinafter only between the two major pavement types, referred to in general terms as follows:

- "flexible pavements" include flexible and semi-rigid structures, as well as certain types of composite structures (e.g., a formerly rigid, badly cracked pavement reinforced with material treated with hydro- carbon binders); and

- "rigid pavements" include rigid structures and certain types of composite structures (e.g., a rigid pavement renewed by applying a wearing course treated with hydrocarbon binders).

d) Bearing strength. The "bearing strength" or "bearing capacity" is the ability of a pavement to accept the loads imposed by aircraft while maintaining its structural integrity.

e) Pavement life. This is the period at the end of which the bearing strength of the pavement becomes inadequate to bear, without risk, the same traffic in the course of the following year, thus necessitating general reinforcement or reduction traffic. The "normal life" of a pavement is ten years and pavements are generally designed for that period, however, in the circumstances described later on in these guidelines, another value may be established for the life of a pavement.

f) Traffic

- One "movement (actual)" is the application to the pavement of a load by an actual undercarriage leg during one manoeuvre (take-off, landing, taxiing), The number of actual movements is generally higher than the number of movements accounted for by the operator (take-offs and landings),

- An "actual load P" is the load actually applied by an aircraft undercarriage leg.

- "Actual traffic" consists of different movements of varying actual loads applied by actual undercarriage legs of different categories.

- The "normal design load" is the load taken into account in formulas or graphs for the purpose of designing the pavement. It may be "weighted" or not, depending on the function of the pavement involved.

- "Normal traffic" is traffic consisting of ten movements per day by the aircraft producing the design load over an expected pavement life of at least ten years.

- The "allowable load Po" of a pavement is the load on an under- carriage leg (actual or fictitious) calculated according to the design concept as being allowable at the rate of ten movements per day over ten years.

- An "equivalent movement" is the application of a reference load by an undercarriage leg (actual or fictitious).

- "Equivalent traffic" corresponds to actual traffic reduced to a number of equivalent movements.

- The "potential" of a pavement on a given date is represented by the number of equivalent movements which it can accept during the residual life.

g) Types of Design

- "optimized design" (or optimized design method): design which takes into account all aircraft types having a significant effect on the pavement, This method is referable if sufficiently reliable and accurate traffic forecasts are available throughout the expected life of the pavement.

- "general design" (or general design method): design in terms of a reference load which the pavement must support. In practice, this method is mainly used at the level of preliminary studies or in the absence of accurate data. The reference load is evaluated in terms of the anticipated utilization of the aerodrome, the characteristics of aircraft in service or at the planning stage, and the specific role of the pavement in question.

4.2.2. Choice of the design load

4.2.2.1. Aircraft characteristics affecting the design

a) Aircraft mass. There is a need to list for each aircraft:

- in the case of the general design method: take-off mass
- in the case of the optimized design method: take-off mass, landing mass

Collection of data on the mass of the various aircraft to be considered in a design is a difficult task bearing in mind:

- the variations in payload

- the uncertainty of forecasting traffic composition (aircraft stages) and developments in regard to aircraft fleets.

For the purpose of studying an optimized design, one useful method consists of establishing mass histograms in respect of each aircraft. Selecting a category width of 1/20th of the maximum mass provides sufficient accuracy.

b) Undercarriage leg. Wheel assembly mounted on one leg. The complete set of undercarriage legs constitutes the undercarriage. A “typical undercarriage leg" which is representative of each of the three most widely used categories of undercarriages (single wheel, dual wheels, dual tandem wheels) is introduced The characteristics of the typical undercarriage legs are as follows:

Typical undercarriage leg	Track (cm)	Base (cm)	Tire pressure
Single Wheel	--	--	0.6 MPa
Dual wheels	70	--	0.9MPa
Dual tandem wheels	75	140	1.2MPa

c) Distribution of the mass over the undercarriage legs

1) Static distribution. The over-all distribution of the aircraft mass between the nose leg and the main undercarriage legs is dependent upon the load distribution of the aircraft (i.e., the position of the centre of gravity) and varies little. In the absence of data, one would assume that the distribution

is 10 per cent on the nose leg (maximum forward load distribution) 95 per cent on the main undercarriage legs (maximum reward load distribution) for conventional undercarriages.

2) Braking action. The effect of braking action is not taken into account in designing pavements. It plays a role only in specified studies (example: structures underneath the runway).

d) Loads used in the calculations. In the case of the undercarriages of current aircraft, the distance between the legs is such as to justify a separate study of the action of each undercarriage leg. The main undercarriage leg generally causes the greatest stress. In some cases, the secondary undercarriage leg may well be the most critical for the pavement (examples: nose leg of B-747, centre leg of DC-10- 30). The load is taken into account in the calculations in the form of a load per undercarriage leg. The graphs in respect of the main aircraft examined (Appendix 3) are produced in accordance with this concept. Those cases where the secondary undercarriage leg is likely to be more critical than the main undercarriage leg are identified and additional graphs provided.

4.2.2.2. Weighting of load according the function of the pavement. Each type of facility (runways, taxiway, aprons, maintenance areas, etc.) must be designed separately to take into account differing stress conditions. Although subjected to the same loads, some pavements may experience different fatigue conditions. For example:

a. Traffic is slow and concentrated on aprons and, conversely, rare and dispersed on shoulders and stopways; and

b. Consequences of dynamic effect. When an aircraft rolls at high speed (such as the middle part of the runway at take-off and the first 1000 m beyond the threshold during landing), the loading phenomenon is transient and thus less severe. In addition, the load is reduced by the lift of the wings. The loads listed in respect of each type of area are weighted to take into account the different fatigue conditions as shown on Figure 4-11. When studying a project, it is recommended to examine the savings that may be achieved by applying these concepts as well as the possible difficulties that may arise during construction or at the time when these areas may be used for a different purpose. Thus reductions in the thickness can be made whenever these will have real short and long term advantages. Such design concepts for reducing pavement thickness are commonly used in some countries. In France they have only been applied on a very limited scale up to now.

4.2.2.3. Loads other than those produced by aircraft. Some areas (such as those in front of airport: buildings) are not accessible to the undercarriage legs. on the other hand, aerodrome pavements do not only support aircraft, but also other vehicles and machinery (e.g., ground transportation vehicles - buses, trucks, baggage tow-trolleys, container carriers, fire fighting vehicles, aerobridges, etc.) which sometimes produce more critical loads (particularly on aprons). When stationary, these units have a considerable punching effect on the pavement producing concentrated stress, due to the fact that they are moving about in a limited space. The exceptional loads are taken into account in the following manner:

a) the affected areas are designed for these loads;

b) the surface of areas used by stress-producing vehicles or equipment must be limited (traffic rules, markings on the surface); and

c) special pavements may be studied (example: special coatings)

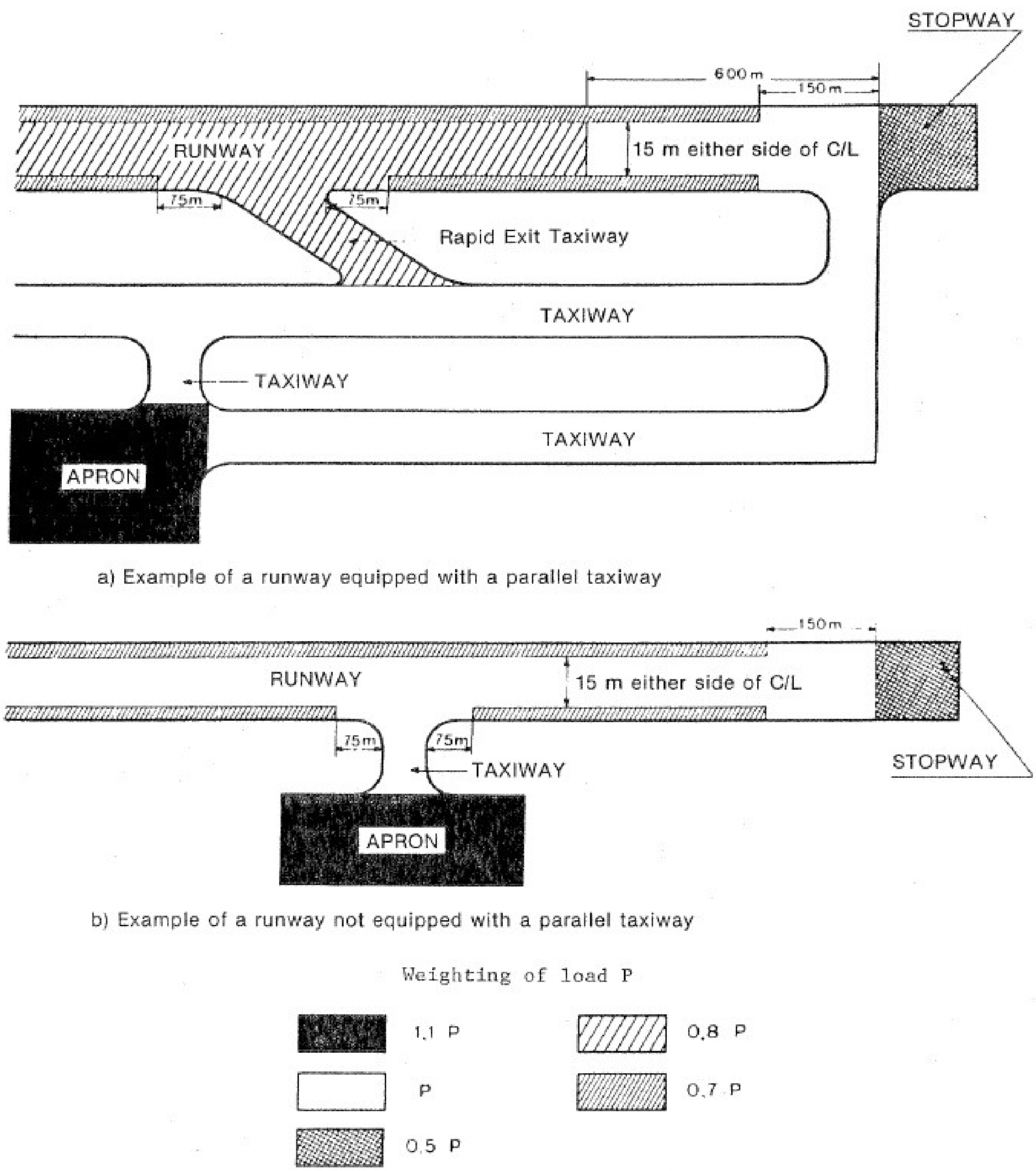


Figure 4-11. Weighing of load P

4.2.3. Designing flexible pavements

4.2.3.1. The design of a flexible pavement involves two stages:

a) Collection of data: - traffic (loads, movements)

- characteristics of the natural soil.

b) Calculation of the thickness, which also comprises two stages;

- the determination of an "equivalent pavement thickness"e using either the general design or optimized design methods.

- the selection of a pavement structure which provides an equivalent thickness corresponding to or greater than the thickness determined above.

4.2.3.2. Bearing strength of the subgrade

a) General case: The bearing strength of the subgrade is denoted by its *California Bearing Ratio* (CBR). The CBR value adopted is the lowest one obtained during the "test series in which the total number of samples is compacted to 95 per cent of Modified Proctor Optimum Density after having been immersed in water for four days.

b) Gravelly soils and pure sand: In the case of gravelly soils and pure sand, the CBR measurement is meaningless and general values will be adopted as shown in the following table:

Description of the soil	Measured CBR	Significant CBR
Pure well-graded gravel	40	20
Pure badly graded gravel	30	20
Gravel containing silt	>40 (PI < 7) > 20 (PI > 7)	20 (PI < 7) 10 (PI >7)
Gravel containing clay	20	10
Pure well-graded sand	20	10
Pure badly graded sand	20	6 to 8
		PI- Plasticity Index

c) Improved Subgrade. Where the pavement comprised an improved subgrade (considerable thickness of added material of average or non-homogeneous quality), this will be taken into account in the calculation in the following manner. Let it be assumed that the bearing strength of the untreated and improved subgrades are, respectively, CBR_1, and CBR_2 and that h_1 and h_2 , which will be calculated according to the design method selected (general or optimized) correspond to one of these CBRs. If h is the thickness of the improved subgrade, the required thickness of the pavement above this subgrade, i.e., e can be calculated by applying the formula:

$$e = h_1 - h \, \frac{CBR2 - CBR1}{CBR2 + CBR1}$$

providing exceeds or is at least equal to h2. Should e be less than h_2 than the thickness of the pavement is fixed at h_2. This also applies to cases where the natural soil comprises a substratum that is covered by a relatively thin soil layer of better bearing strength. This top layer may then be regarded as an improved subgrade so that the above method can still be used.

4.2.3.3. Calculating the equivalent pavement thickness

- General design – see 4.2.5
- Optimized design – see 4.2.6

4.2.3.4. Structure of the pavement. A flexible pavement is normally made up of three different courses of increasing quality from bottom to top: the sub-base, the base and the surface course. The concept of equivalent thickness is introduced to take into account the different mechanical qualities of each course. The equivalent thickness e of a course is equal to its actual thickness e_r multiplies by a numerical coefficient c or equivalence coefficient. The equivalent thickness of the pavement is equal to the sum of the equivalent thicknesses of its courses. The values shown in the table below may be used as a reference in the case of new materials:

New Materials	Equivalence Coefficient
Concrete-type dense bituminous mix	2
Sand-gravel mix bound with bitumen	1.5
Emulsion sand-gravel	1.2
Sand-gravel treated with hydraulic binders (cement, slag, fly-ash, lime)	1.5
Well-graded crushed gravel	1
Sand treated with hydraulic binders (cement, slag)	1
Pea gravel	0.75
Sand	0.5

In a properly constituted pavement, the equivalence coefficients of necessity increase from bottom to top.

4.2.3.5. Choice of a structure. The choice of a structure must take into account two general concepts:

a) Construction concepts which relate to the nature of the materials to be used, the quality and formulation of components, the minimum and maximum thicknesses involved, sound bonding of courses, etc,; and

b) Mechanical concepts which define the values of equivalence coefficients, proscribe or advise against the use of certain materials in the different courses, indicate the thicknesses of the treated materials needed for the normal mechanical behaviour of the pavement, etc. These directives have the following effect on the different courses:

- Surface course (wearing course and possibly binder course). The surface course must consist of bituminous concrete. (Some directives, especially as regards formulation and compactness to be achieved at the work site, differ considerably from those applicable to road pavements.)

- Base and sub-base. The choice of materials for the base and sub- base is subject to the applications specified in the. Following table:

Types of materials	Used in base	Used in sub-base	Remarks
Sand-gravel mix bound with hot hydrocarbon binders	Yes	No	Expensive materials.
Materials treated with hydraulic binders (coarse aggregated concrete, slag, fly-ash gravel, sand-based concrete)	No	Not advisable	Except with special dispensation following consultation of Administration.
Untreated gravel (crushed, well-graded)	Yes	Yes	--
Pea gravel	No	Yes	--
Materials treated with cold hydrocarbon binders (emulsion gravel)	Not advisable	Not advisable	The use of these materials calls for a technique which has not been sufficiently tested on aerodrome pavements.

Frequently, economic considerations make it necessary to envisage the use of materials that have been treated with hydraulic binders (coarse-aggregate concrete, slag based on sand-gravel mix, sand-gravel fly-ash mix, etc.) in the base or sub-base. However, the magnitude of the loads applied to aerodrome pavements creates much greater stresses than those produced on road pavements. The risks and consequences, among others, are:

- for the pavements: rapid signs of deterioration (cracks in wearing course, crumbling, tearing, pumping up of particles or re-appearance of fines of laitance);
- for aircraft: ingestion by jet engines of aggregate particles, evenness; and
- for management: higher maintenance costs (filling cracks).

Consequently, the use of materials treated with hydraulic binders is proscribed for the base and not advised for the sub base. In the case of the latter, an actual thickness measuring at least 20 cm of materials treated with hydrocarbon binders must cover the semi-rigid course. Any exception to these rules calls for a special study for which expert device of the Administration must be requested. Specifications for materials that may be used in the base or subbase are identical to those applied to road pavements.

4.2.3.6. Thickness of treated materials. An adequate thickness of treated materials is necessary to ensure an acceptable behaviour of the upper pavement layers. Figure 4-12 shows, for guidance, the optimum equivalent thickness of treated materials with respect to the total equivalent thickness of the pavement and the CBR of the natural soil.

4.2.3.7. Influence of climatic factors. In regions that are subject to significant seasonal climatic variations, possible changes in the bearing strength of the soil shall be taken into account. Despite the considerable influence which temperature has on bituminous mix pavements, no correction for thickness will be made to account for this parameter: the values indicated for the equivalence coefficients for the coating mixes suggested previously represent a weighted average. It is recommended that testing for frost-thaw be performed in accordance with the information contained in 4.2.7.

4.2.4. Designing rigid pavements

4.2.4.1. The design of rigid pavements involves the following two stages:

a) Collection of data:
    i. –Traffic (loads, movements)
    ii. Characteristics of the subgrade and of the hydraulic cement concrete; and

b) Calculation of the thickness of the concrete slab (only the most general case of non-reinforced and non-prestressed pavements is examined).

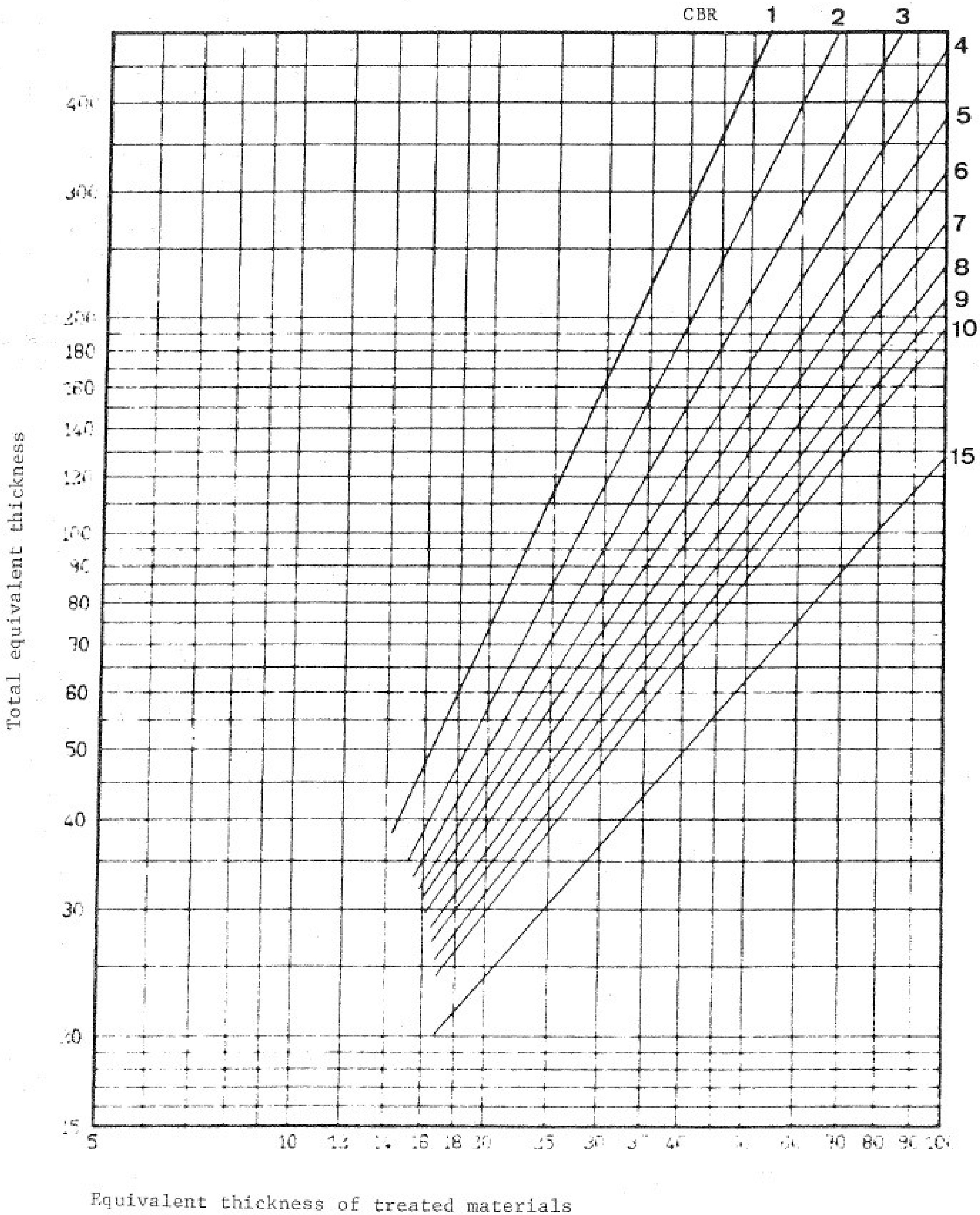


Figure 4-12. Flexible pavements: Optimum thickness of treated materials with regard to the equivalent thickness of treated materials to the total thickness of the pavement and to the CBR

4.2.4.2. Evaluation of the sub-base. A rigid pavement normally consists of two courses on top of the natural soil, i.e., a sub-base and hydraulic cement concrete slab. The bearing strength of the natural soil is expressed in the form of its modulus of reaction k_0. This is corrected in accordance with the equivalent thickness of the sub-base. The modulus thus corrected (i.e. modulus of sub-base reaction) makes it possible to account for the soil and sub-base as one single parameter in the calculations.

4.2.4.3. Bearing strength of natural soil (subgrade). The modulus of subgrade reaction k_0 of the soil is evaluated by means of a plate bearing test carried out on soil compacted to 95 per cent of the Modified Proctor Optimum density. It is desirable for a certain time to elapse between compacting and testing to allow the soil to regain its free moisture content. The number and distribution of test points must be such as to make the results meaningful.

4.2.4.4. Bearing strength of the sub-base. The modulus of subgrade reaction of natural soil is subsequently corrected in regard to the equivalent thickness of the sub- base. Figure 4-13 is used for this purpose. The definition of equivalent thickness is given in 4.2.3.4.

Important Note: The corrected k should be used in these calculations. Using the k measured at the top of the sub-base course would result in optimistic figures.

Although the sub-base affects the calculation only slightly (as a corrective term of modulus k which itself has only a minor impact), it has an important multiple role:

- it ensures a continuous support for the slab, particularly at its joints and participates in the transfer of loads;
- because of its weight it opposes a possible swelling of the sub-grade soil and protects it against frost;
- it offers a stable surface for subsequent concreting operations; and
- it prevents pumped up particles from rising at the joints.

4.2.4.5. Structure of the sub-base. It is important to have a high quality sub-base. The following rules must be applied:

- The sub-base course must be treated;
- The use of coarse aggregate concrete is advisable;
- lean cement concrete is not really recommended (higher risk of cracking);
- the actual thickness of the sub-base must be at least 15 cm to ensure an efficient use of the material; and

- the specifications for materials that may be used in a sub-base are Similar to those for road pavements.

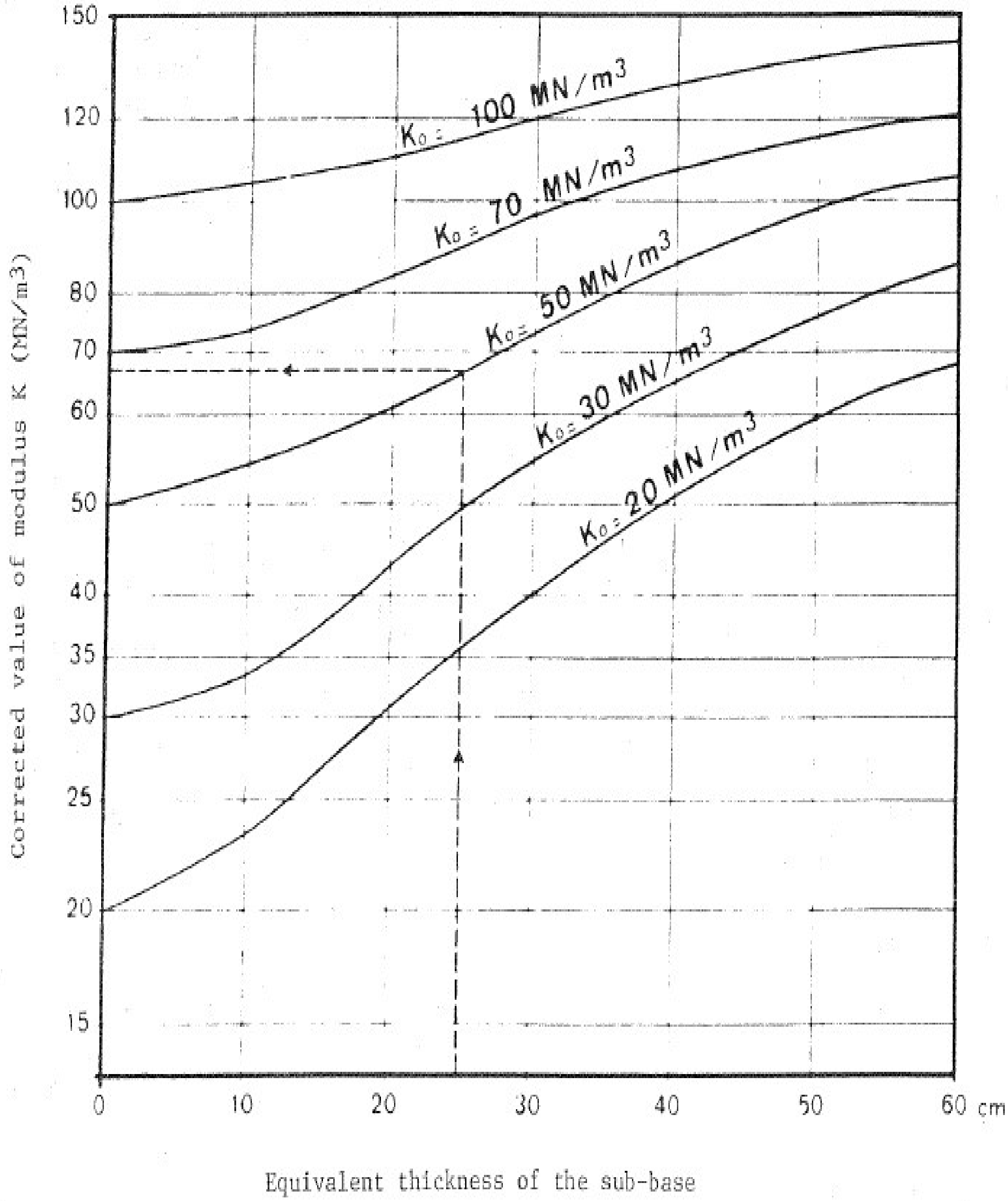


Figure 4-13. Modulus of reaction of the sub-base: Correction of the modulus of reaction of the subgrade on the basis of the equivalent thickness of the sub-base

The sub-base can rest on an improved subgrade which may or may not consist of stabilized materials. The total equivalent thickness of the two courses is subsequently taken into account to correct the modulus of subgrade reaction K. It is feasible to place a layer of porous concrete between the concrete slab and the treated sub-base in order to improve the drainage and to reduce the pumping effect.

4.2.4.6. Designing the thickness of the concrete slab. Due to the rigidity of the concrete, the vertical stresses applied to the subbase by a loaded concrete slab are always very low; the slab ensures the distribution of stresses due to loading by mobilizing its flexural strength. Consequently, contrary to what happens in the case of a flexible pavement, the design criterion for a rigid pavement is not maximum pressure at subgrade level, but permissible flexural moment of the slab. In the design, constant values are adopted to describe the concrete as follows:

modulus of elasticity: E = 30000 MPa

Poisson's ratio = 0.15

4.2.4.7. Stresses of concrete. Account is taken in the calculations of the permissible flexural stress on the concrete which equals the flexural breaking strength divided by a safety factor. The flexural breaking strength is measured on prismatic specimens after 90 days. The final value to be retained is the mean of the measured values reduced by a standard deviation which corresponds to the foreseeable scatter over the site (varying between a minimum of 10 per cent for a closely supervised construction site and 20 per cent). If the results of tests performed after 28 days' curing only are available, it may be assumed that the flexural strength of the concrete increases by 10 per cent between 28 and 90 days.

4.2.4.8. Safety factors. The safety factor depends on the type of joints used between the slabs of the pavement. It is established at 1.8 where joints are equipped with devices for the efficient transfer of loads and at 2.6 in other cases, as shown in the table below:

Type of device for transfer of loads across pavement construction joints	Other conditions	Safety factor
Without device	in all cases	2.6
Dowels	---	1.8
Tongue and groove joints	less than 3 unfavourable conditions (see below)	1.8
	at least 3 unfavourable conditions	2.6

(see below)

Unfavorable conditions

- poor subgrade (k 20 MN/m^3) or non-homogeneous or frost susceptible
- thin sub-base (e < 20 cm) or untreated
- heavy traffic consisting of wide-bodied aircraft (B -747, DC-10, etc.)
- significant daily temperature gradient
- absence of tie bars across joints

4.2.4.9. Construction rules- see 4.2.4.11

4.2.4.10. Thickness of concrete slab

- General design (see 4.2.5)
- Optimized design (see 4.2.6)

Comment: The general design method is generally adequate for studying rigid pavements.

4.2.4.11. Construction rules

a) Joints. A correctly designed rigid pavement must respect the main construction rules laid down in Figure 4-14.

b) Efficient transfer of loads. None of the devices described provides complete efficiency. The tongue and groove systems and the contraction-expansion joints are efficient only where the joints are not too open under the combined effect of hydraulic contraction (definitive) and thermic contraction (periodic); also, with time they lose some of this efficiency due to the fact that the two surfaces in contact show wear from the effects of traffic and the thermic cycles. The efficiency of dowelled joints is not closely linked to their openings. However, the transfer of loads is also likely to diminish with time, mainly due to the fact that the cylindrical cavity in which the dowel moves in a longitudinal direction becomes enlarged and more oval in shape. As pointed out, the sub-base may improve the transfer of loads, provided it is sufficiently rigid. However, its beneficial action also decreases with time, particularly because of surface erosion.

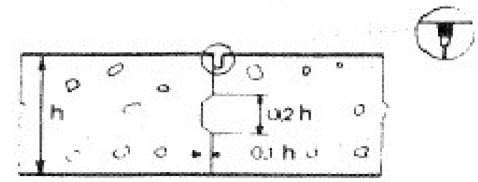
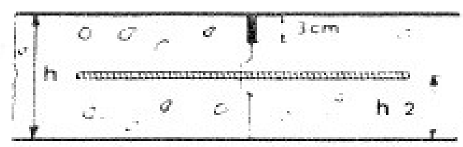
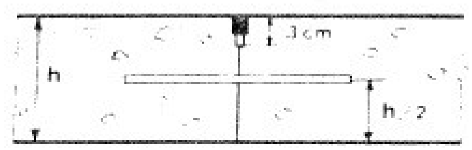
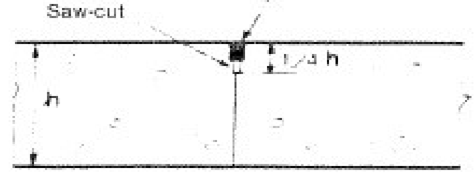
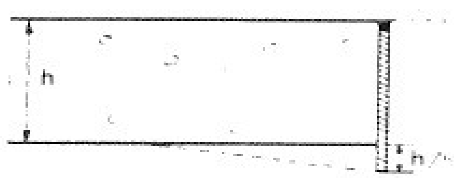
TYPE OF JOINT	DIAGRAM	L: Location C: Conditions for utilization
CONSTRUCTION	 Simple Tongue-and-Groove	L: Longitudinally: at the end of lanes. Transverally:where concreting is interrupted along the lane. C: tongue-and-groove only for slab thickness exceeding 20 cm dowels advised for pavements with heavy traffic of wide-bodied aircraft and on relatively poor soil tie bar only over longitudinal joints; the width of the link must not exceed 25 cm
	 Tongue-and-Groove with tie bar	
	 Dowel	
CONTRACTION	 	L: Longitudinally: where lane width exceeds 5 m Transversally: systematic installation at regular intervals
EXPANSION	 	L: at the junction of old and new work — between runways and taxiways — around substructures — along drains C: used to avoid undue stresses
	 Dowel	

Figure 4-14. Joints in cement concrete pavements

4.2.4.12. Influence of climatic factors

a) Factors of thermic orhygrometric origin. As a general rule it is accepted that, provided appropriate methods are used for the joints, stresses which have a thermic or hygrometric origin need not be takeninto account in the design. Flexural stresses produced by loads during use of the pavement are not the only tensile stresses to which the concrete may be subjected. Stresses may, first of all, result from .differential expansions between the top and bottom surfaces of the concrete because of differences between these two faces:

- in the temperature (temperature gradient)

- water content

Other stresses may also be caused by friction on the sub-base which resists a variation in length of the slab as a whole when a change in the temperature or in the water content occurs. These changes are assumed to be of a sufficient duration to enable the slab to achieve a state of hygrometric equilibrium. Consequently, they are changes that may be described as seasonal as opposed to those (daily) changes that are produced by hygrometric gradients in the slab. In all cases, the existence of joints which limit the lengths of the basic slabs has the effect of reducing the magnitude of the different types of stresses. Moreover, the stresses of the first category largely tend to compensate each other due to the fact that temperature gradients and water content are normally opposite characteristics. Finally, these different stresses do not appreciably increase the stresses imposed by loads.

b) Frost. An inspection for frost-thaw in accordance with the explanations contained in 4.2.7 is recommended.

4.2.5. General design

4.2.5.1. Principle. The general design method enables a pavement to be designed according to a reference load. For example:

- the maximum load of the aircraft considered to produce the greatest stress; and

- the desired load for a typical category of undercarriage.

The design is based on normal traffic conditions, i.e., ten movements per day over ten years at the design load. However, where the actual traffic clearly differs from this basic assumption, it is possible to apply a correction factor to take account of the actual traffic intensity. Examples of using the general design methods are:

- study of an aerodrome used for operations with an aircraft type that clearly produces greater stress than others;

- rigid pavements (the accuracy of the method is generally sufficient) ; and

- Preliminary studies in the absence of reliable traffic forecasts.

4.2.5.2. Determination of pavement thickness

Data required

- Normal design load P'

- CBR of the natural soil (flexible pavements)

- Modulus of subgrade reaction k and the permissible flexural stress of the concrete (rigid pavements)

Graphical method

Depending on the case under study, one uses either the graph for typical undercarriage (Figure 4-15 to 4-27) or the specific graph for the aircraft (Appendix 3).

Note. -If one intends to determine pavement thickness for an aircraft or, more generally, an undercarriage leg not included in the graphs in Appendix 3, it is possible to use the graphs for an aircraft whose main undercarriage leg (track, base) has characteristics that most closely resemble those of the aircraft under study.

4.2.5.3. Traffic intensity. Ten movements per day over 10 years represent an entirely reasonable and conservative assumption for the purpose of designing a new pavement. Nevertheless, it is conceivable that this figure is either clearly below the foreseeable traffic volume for the aerodrome (e.g. a major aerodrome) or considerably higher (e.g. an alternate aerodrome). It is necessary in those cases to take account of the actual traffic intensity appropriately adjusted. The correction is based on a relationship between the pairs (P, n), where P is the load and n the number of applications in movements/day and the pair (P', 10) where P' s the normal design load (by definition applied 10 times per day for):

$$P' = \frac{P}{C} \qquad [1]$$

The graph in Figure 4-28 translates relationship 1

with $C = 1.2 - 0.2 \log n$

Important Remark: Relationship [1] is only valid for a pavement life of ten years. For any other period, it would be appropriate to relate the figure to ten years (example: 4 movements/day over 20 years are equivalent to 8 movements/day over ten years). The value of factor C is limited to 1.2 at the top end of the scale (minimum assumption of 1 movement/day) and to 0.8 at the bottom end of the scale (maximum assumption of 100 movement/day).

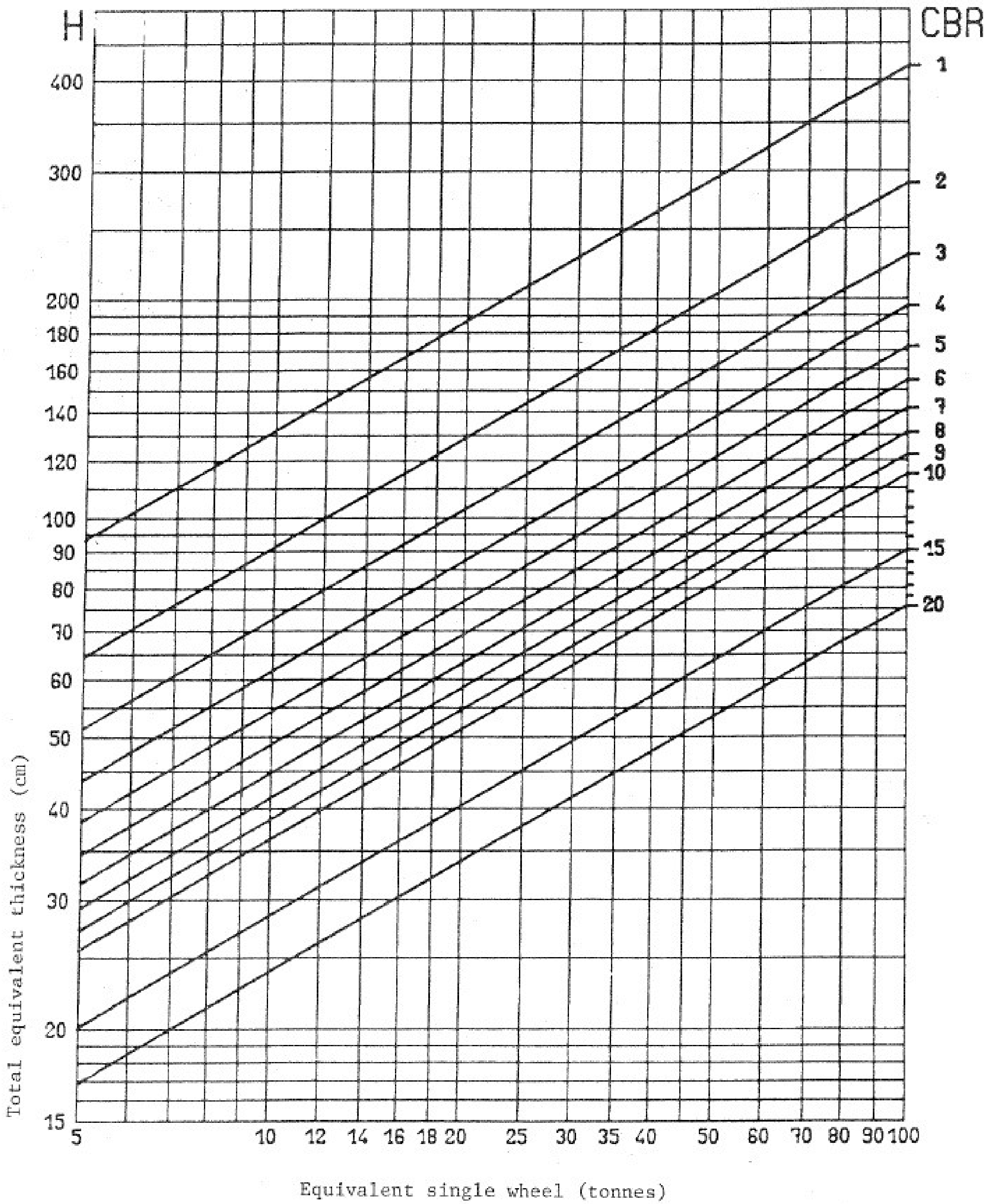


Figure 4-15. Flexible Pavement- typical undercarriage leg – single wheel

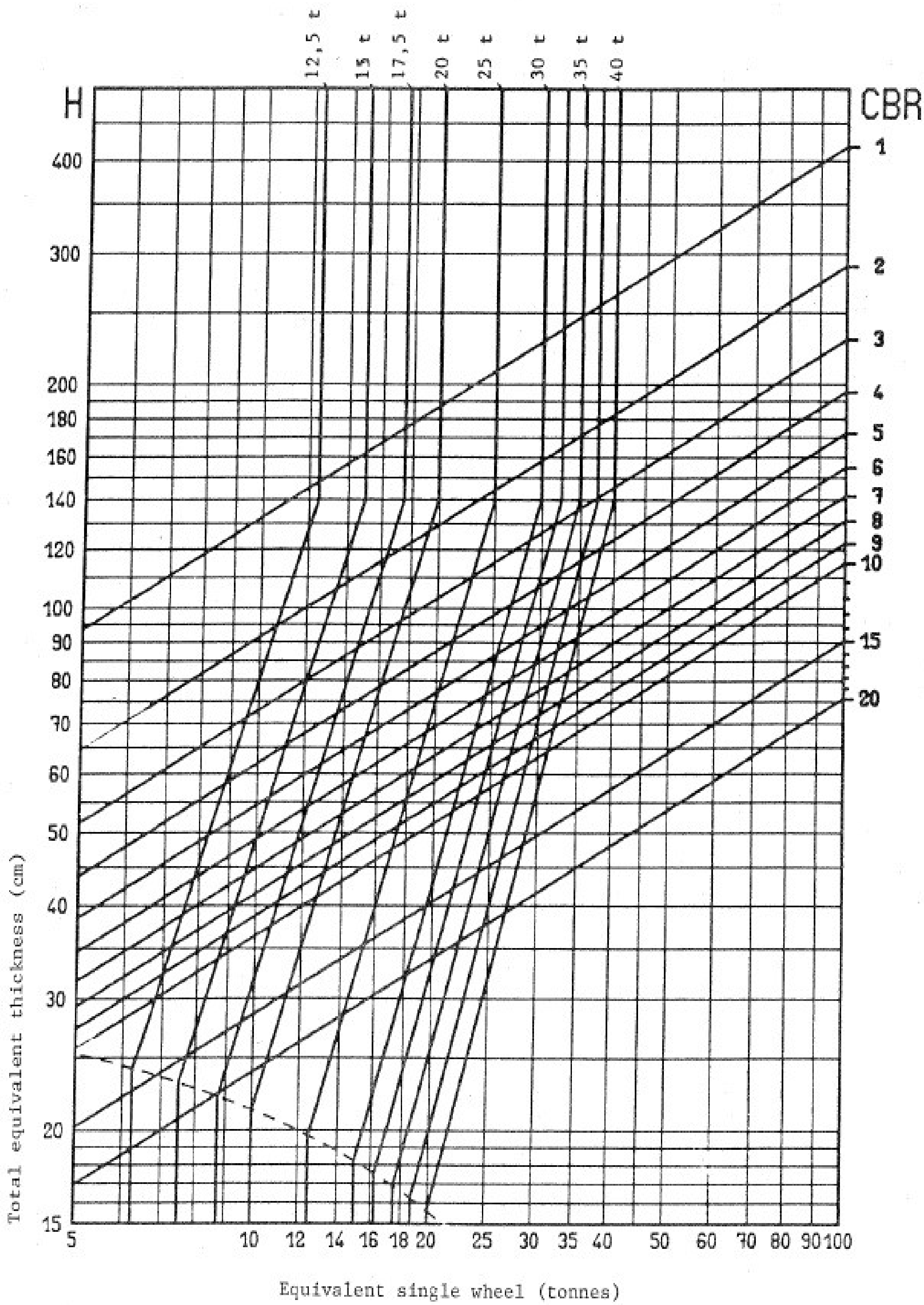


Figure 4-16. Flexible Pavement – typical undercarriage leg – dual wheels

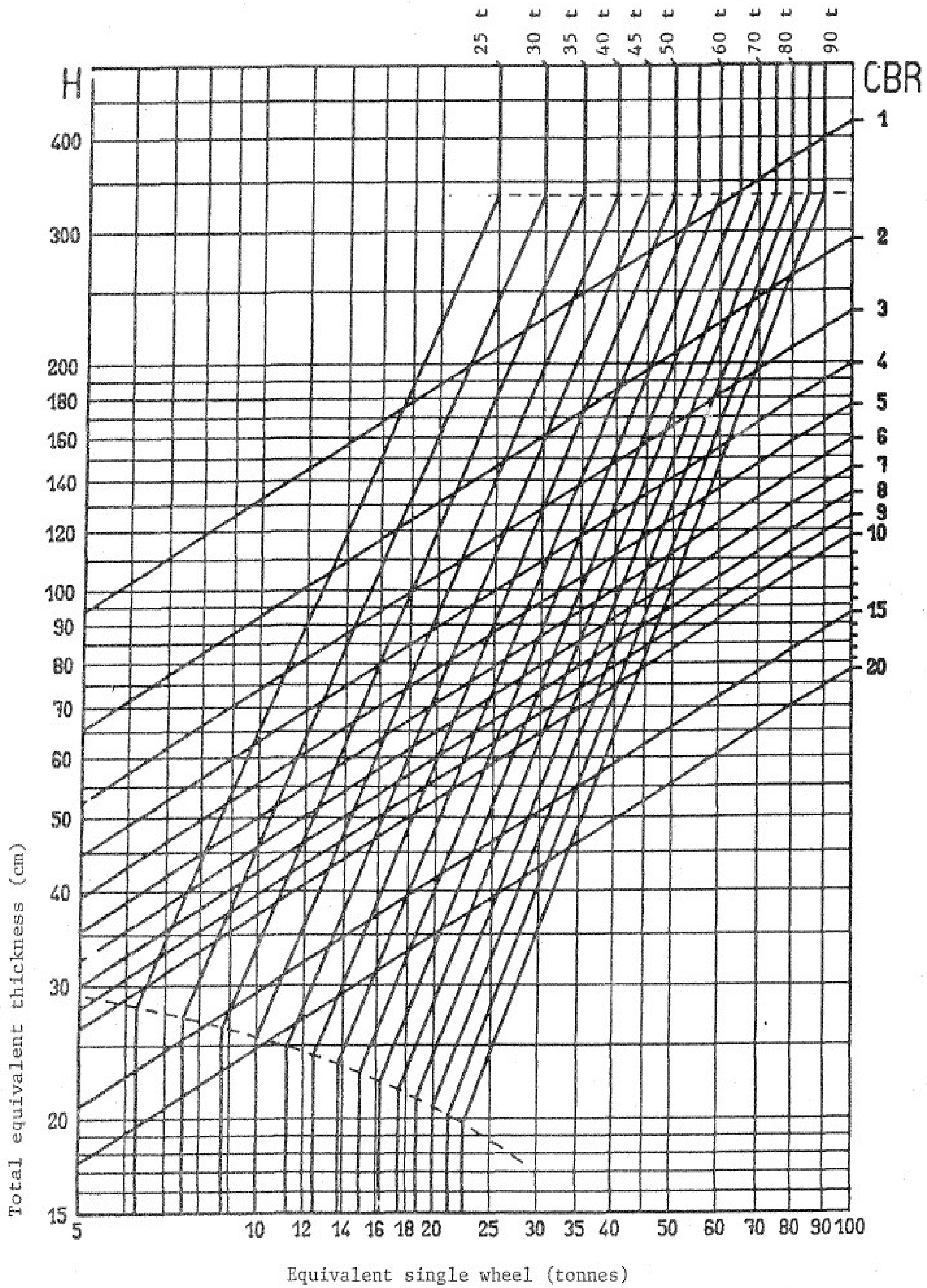


Figure 4-17. Flexible Pavement – typical undercarriage leg – dual tandem

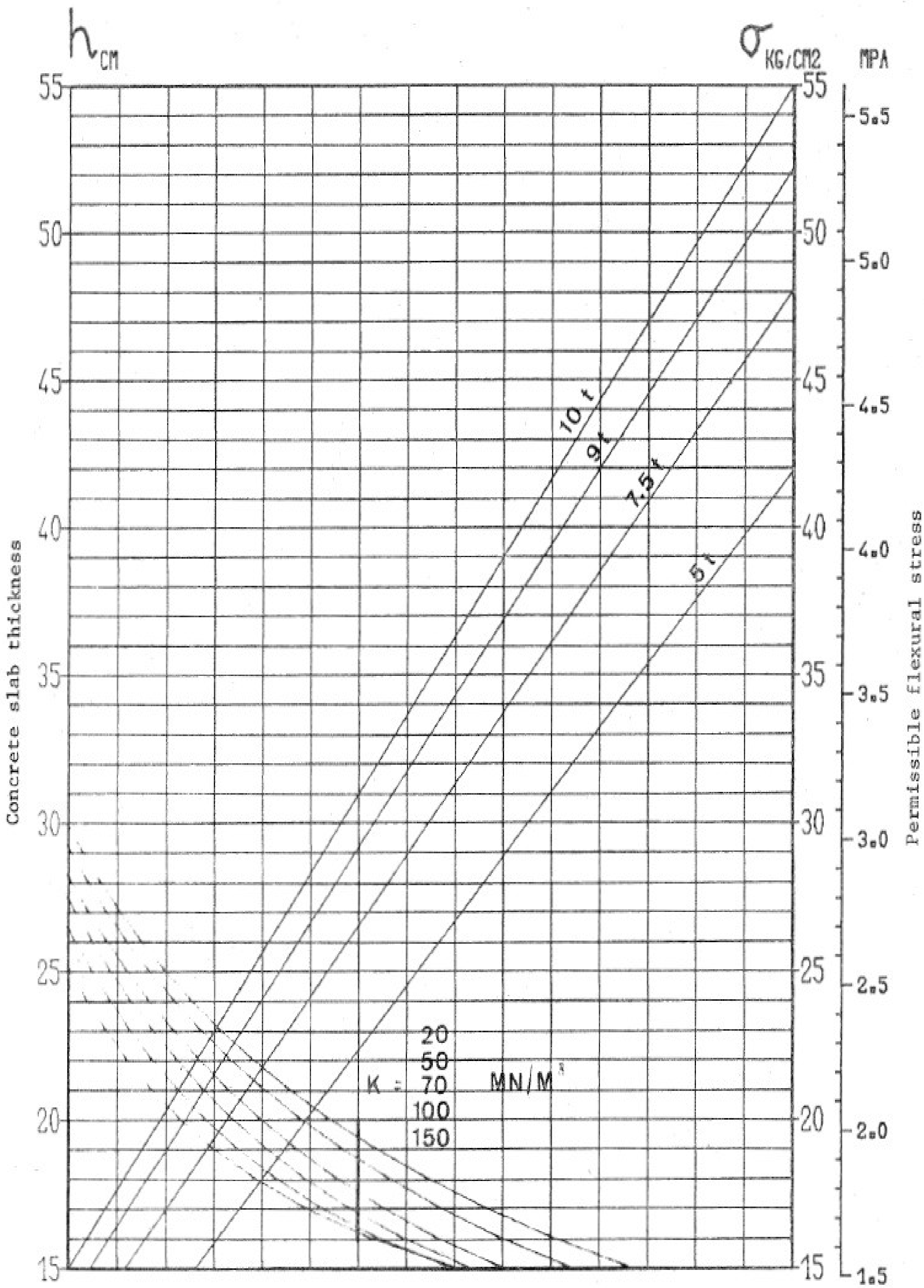


Figure 4-18. Rigid Pavement – typical undercarriage leg – single isolated wheel loads less than 10 tones

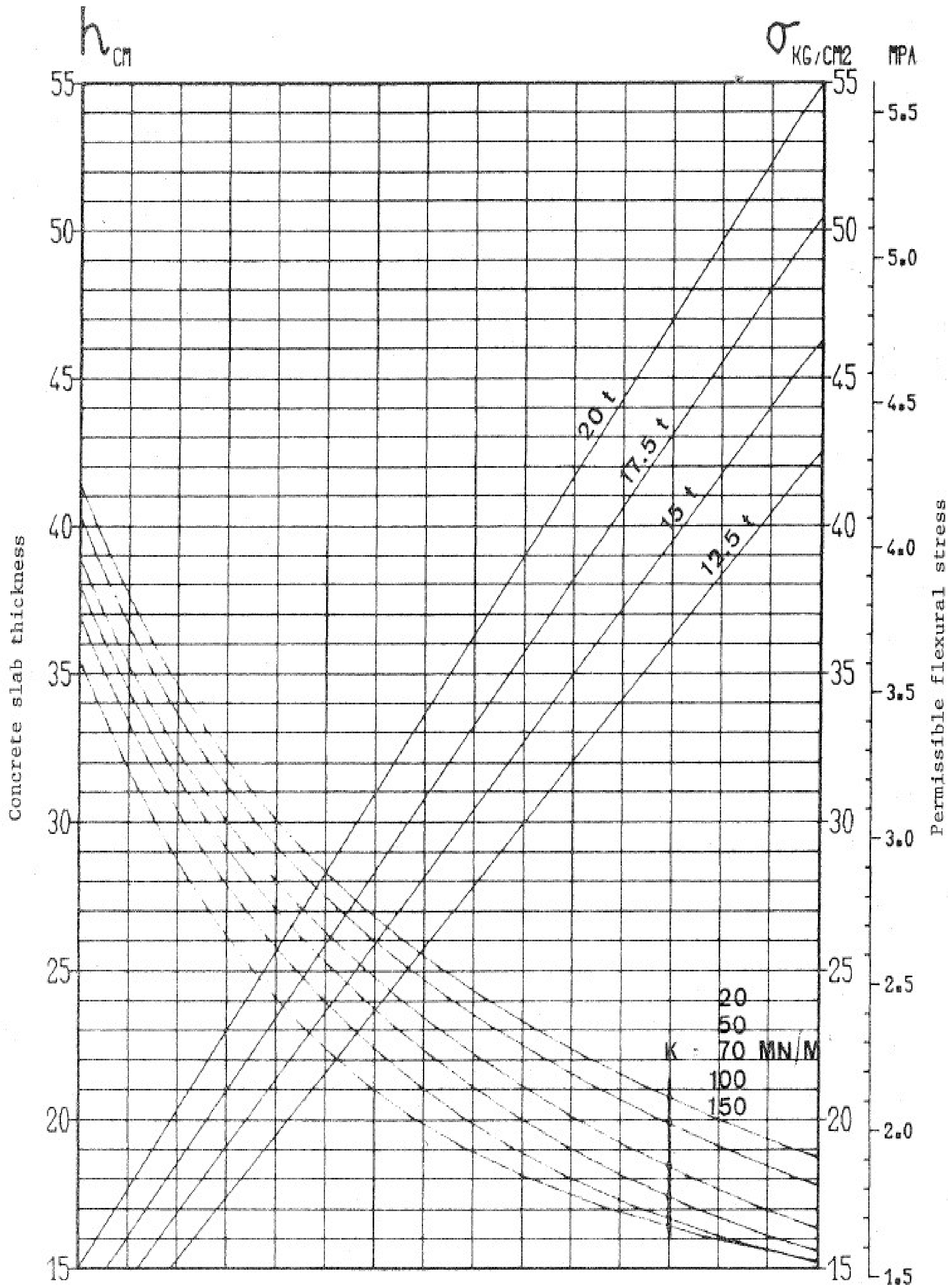


Figure 4-19. Rigid Pavement – typical undercarriage leg – single isolated wheel loads ranging from 10 to 25 tones

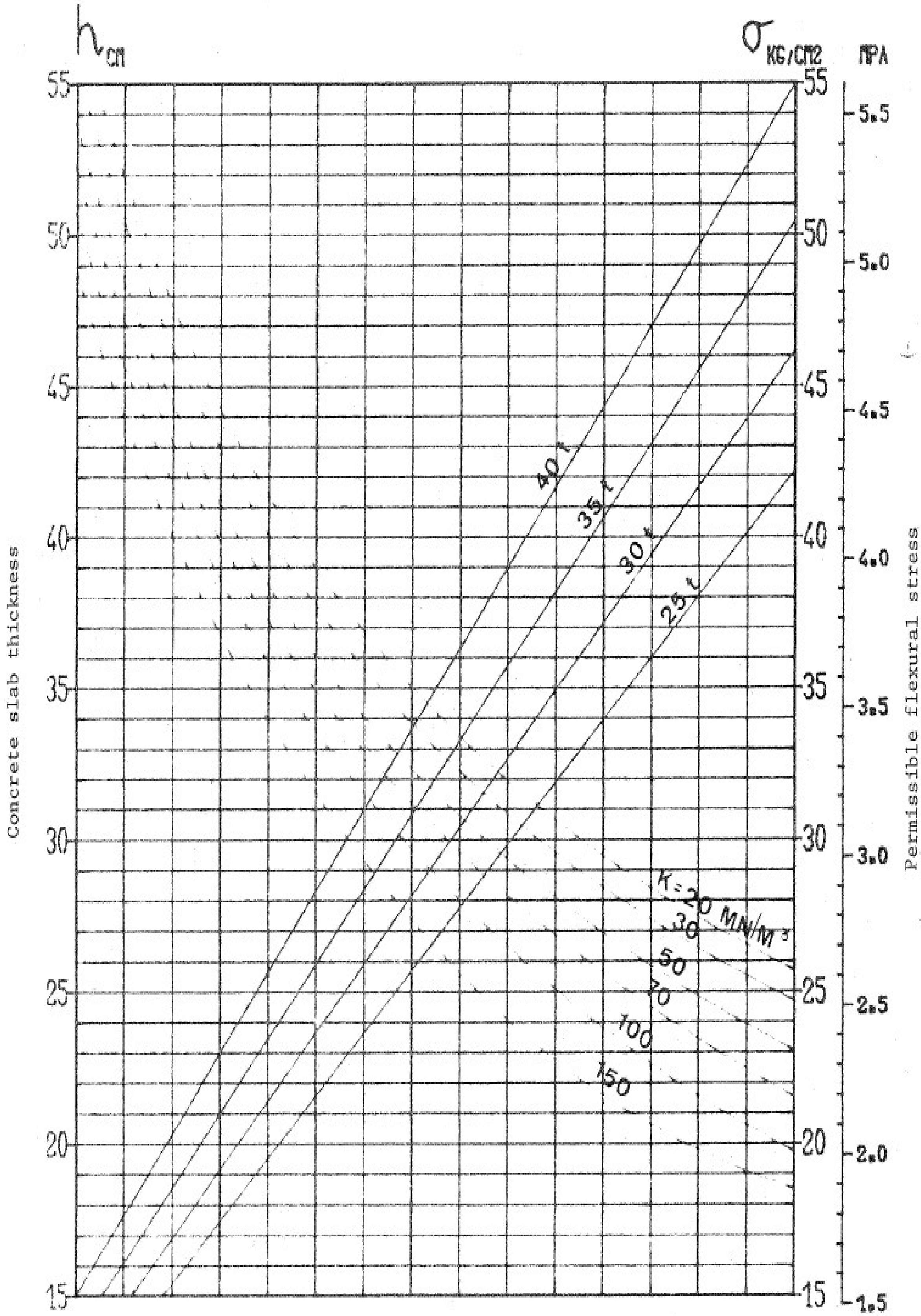


Figure 4-20. Rigid Pavement – typical undercarriage leg – single isolated wheel loads exceeding 25 tones

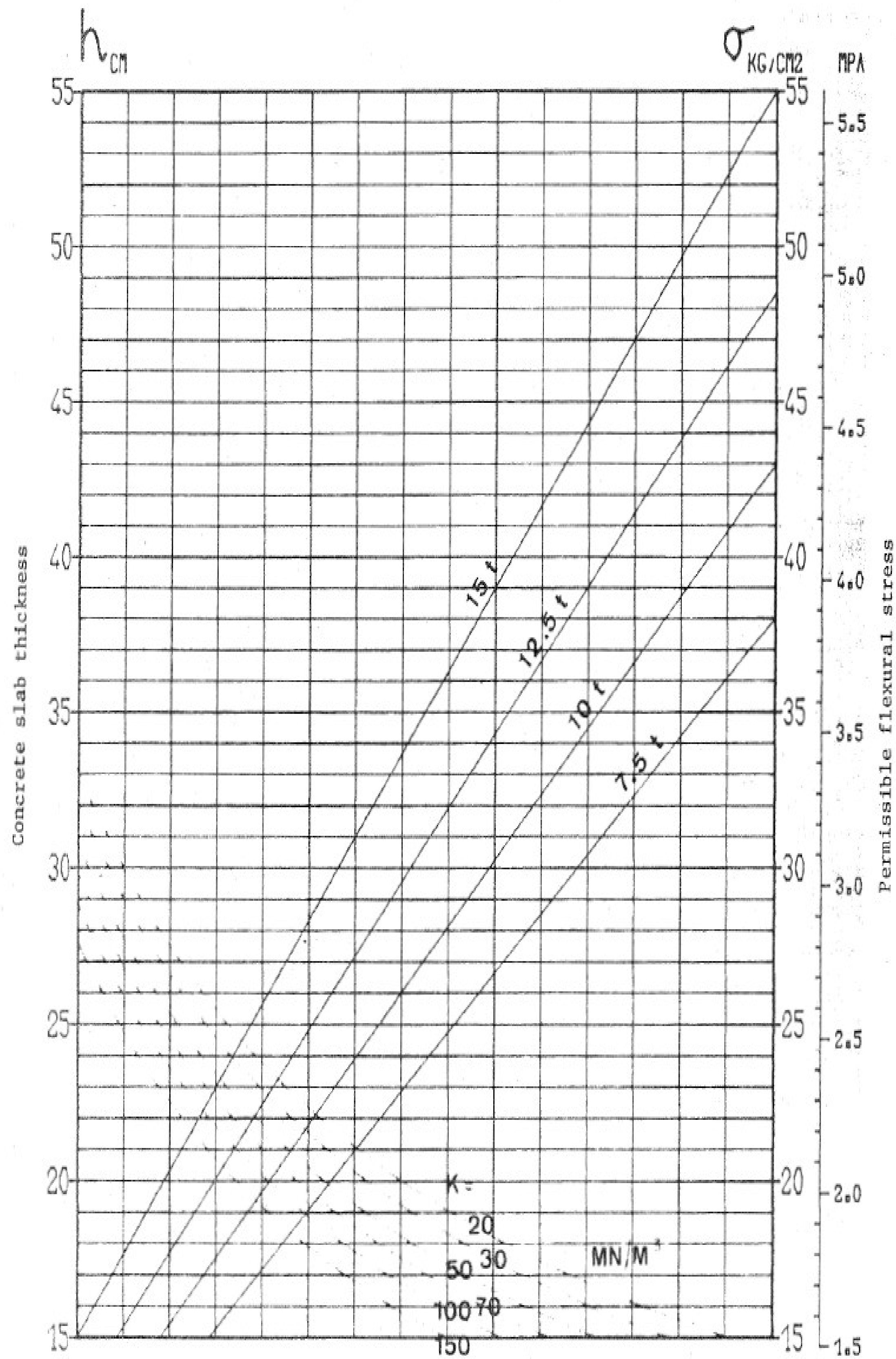


Figure 4-21. Rigid Pavement – typical undercarriage leg – dual wheels loads less than 15 tonnes

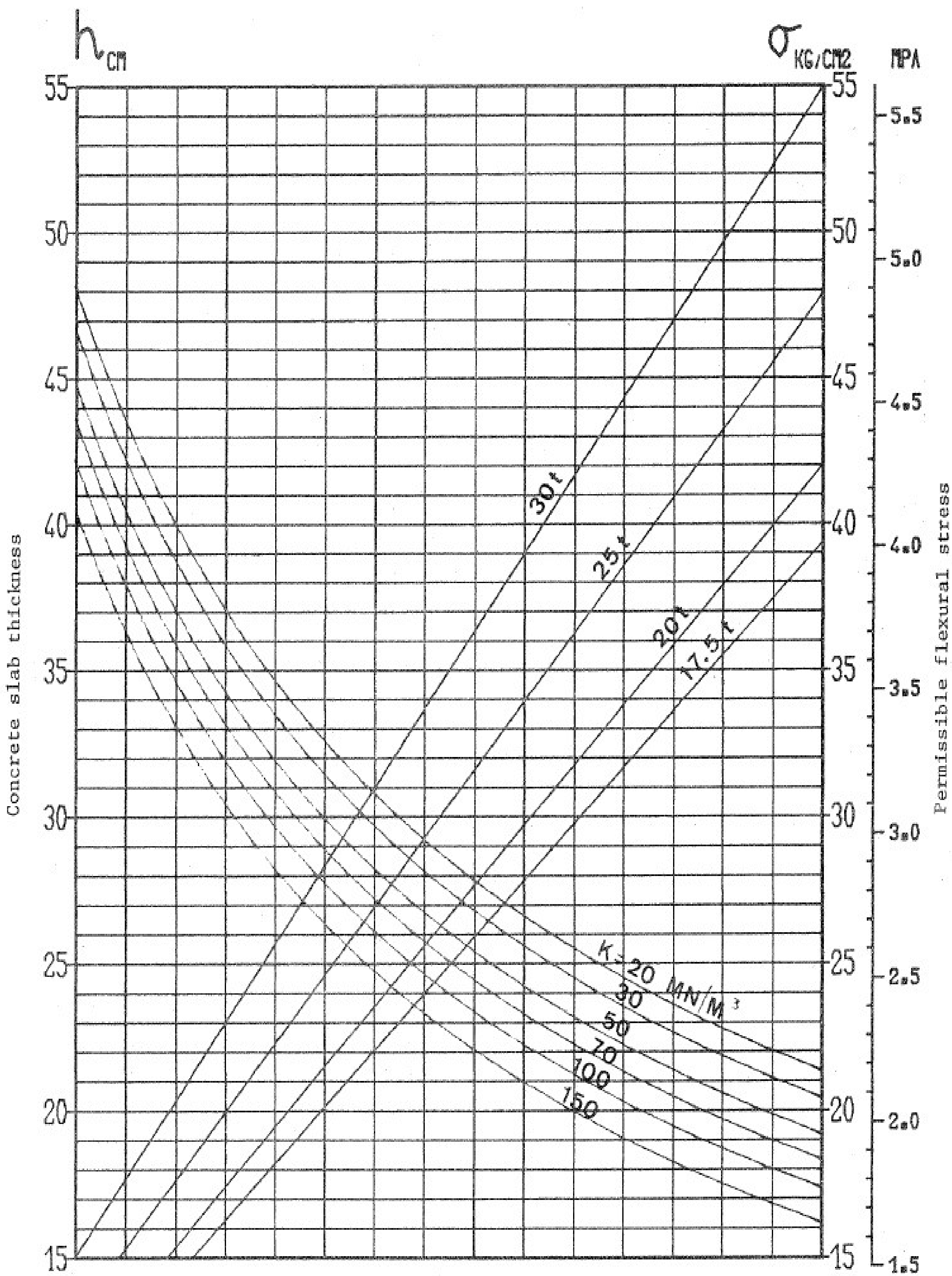


Figure 4-22. Rigid Pavement – typical undercarriage leg – dual wheels loads ranging from 15 to 32.5 tonnes

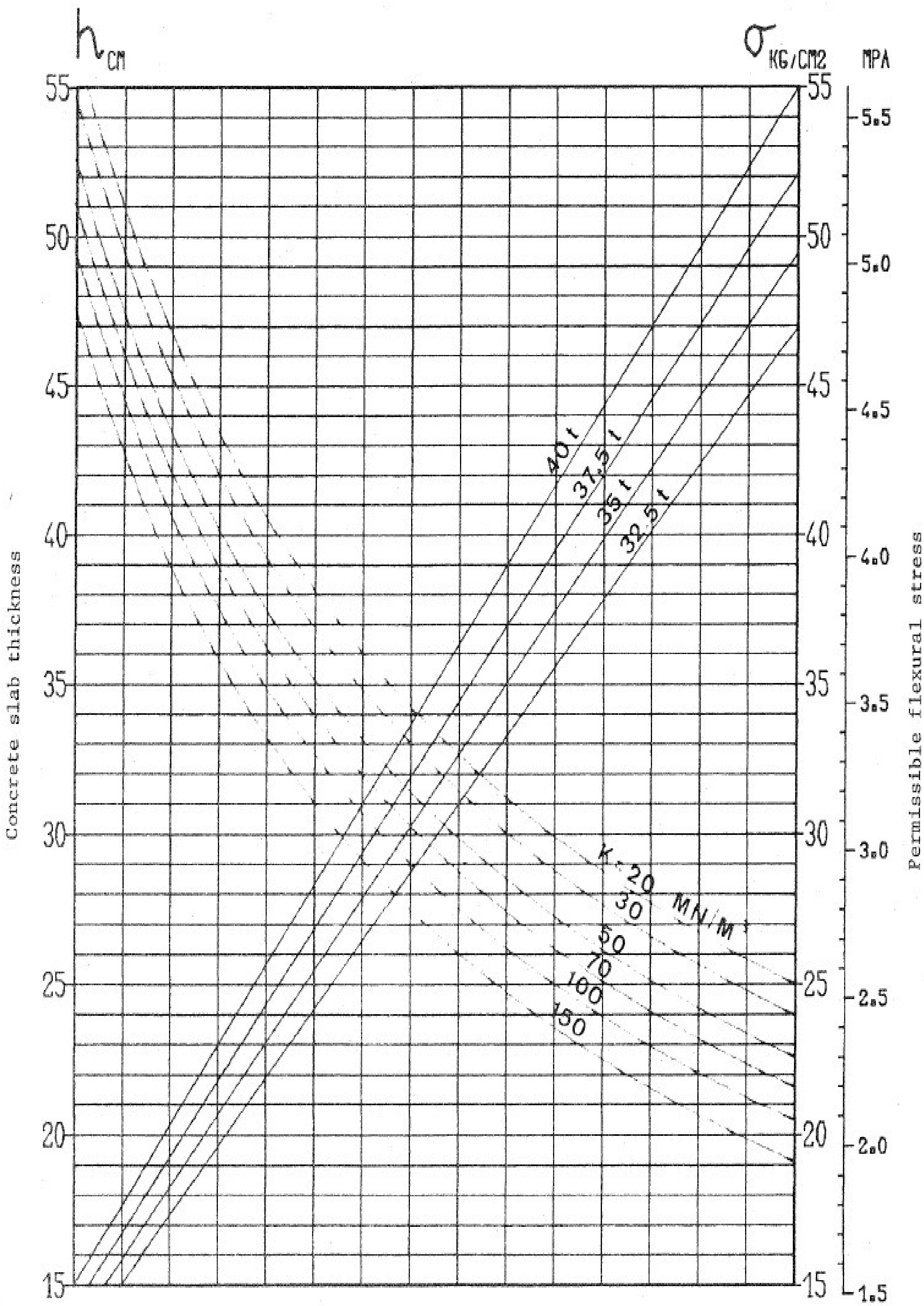


Figure 4-23. Rigid Pavement – typical undercarriage leg – dual wheels loads exceeding 32.5 tones

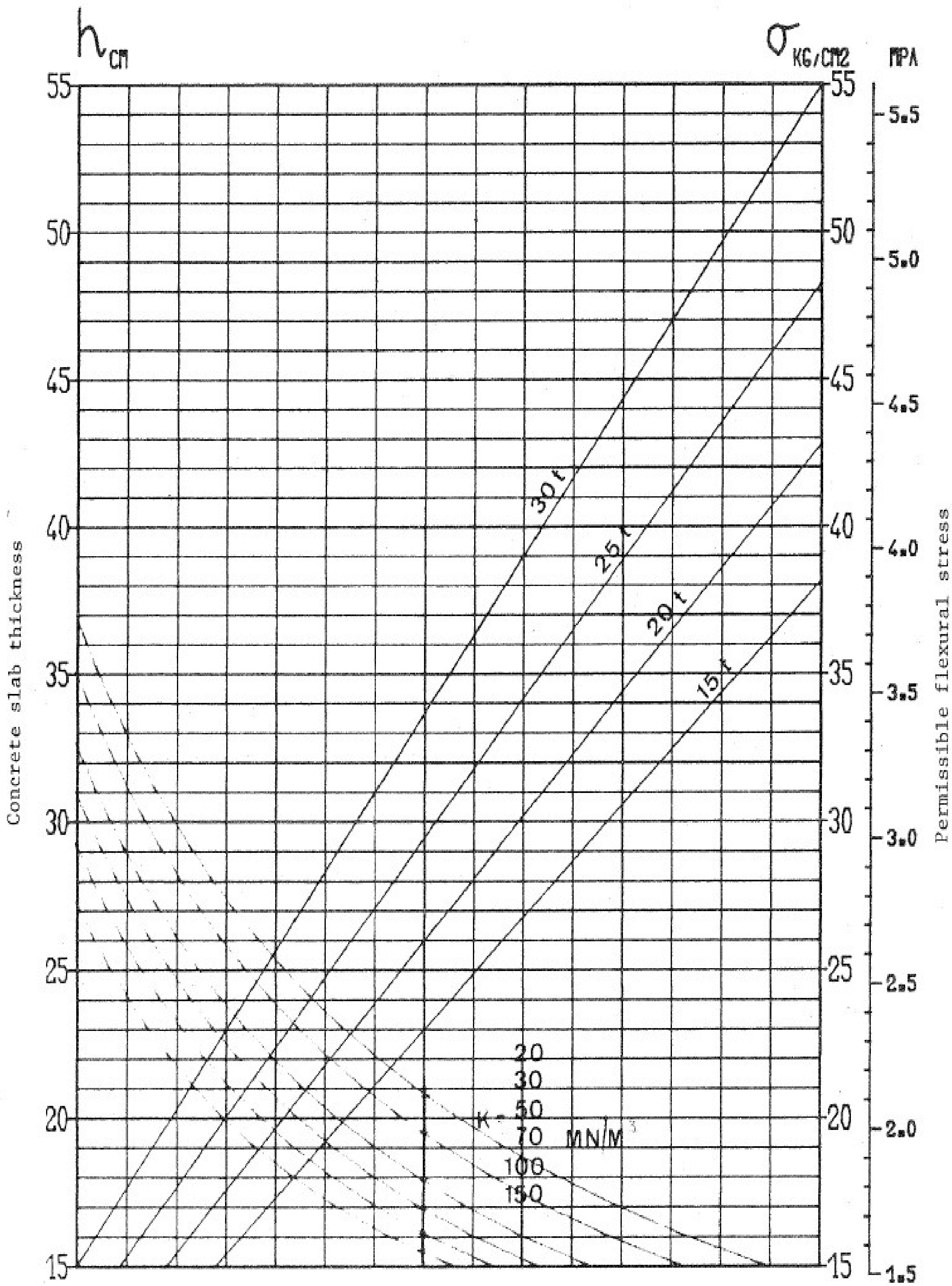


Figure 4-24. Rigid Pavement – typical undercarriage leg- dual tandem loads ranging from 15 to 30 tones

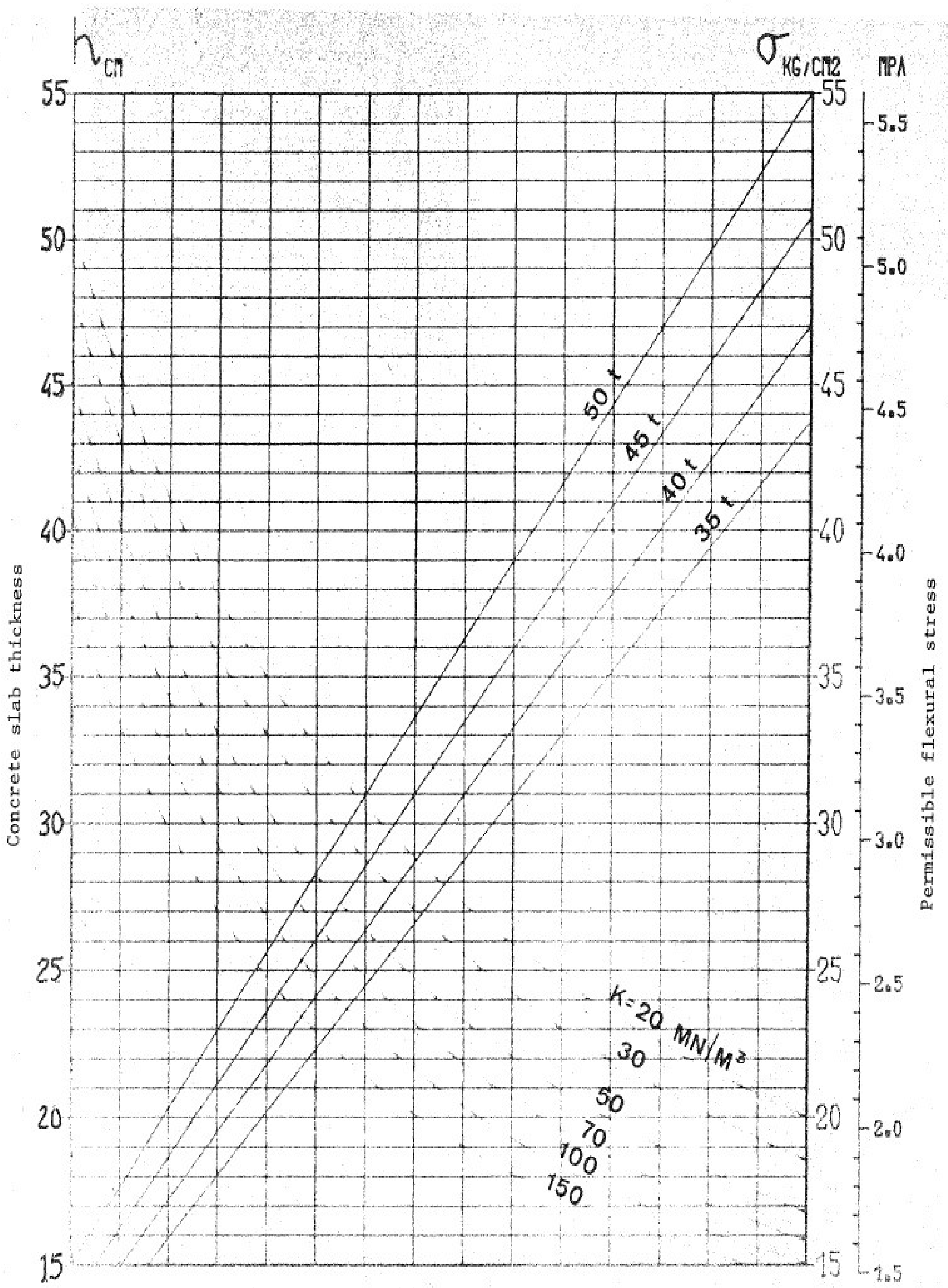


Figure 4-25. Rigid Pavement – typical undercarriage leg – dual tandem loads ranging from 30 to

55 tones

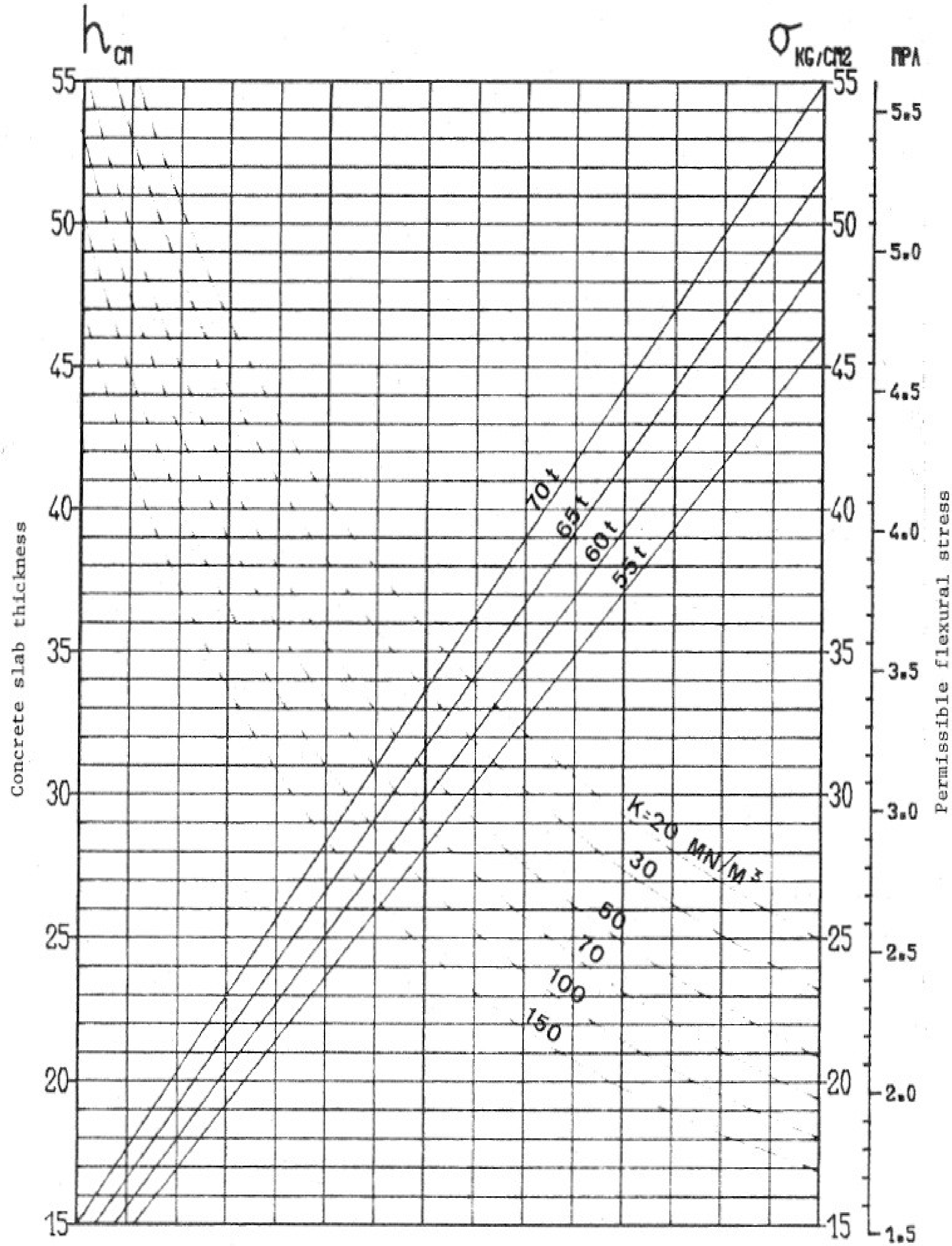


Figure 4-26. Rigid Pavement – typical undercarriage leg – dual tandem loads ranging from 55 to

75 tones

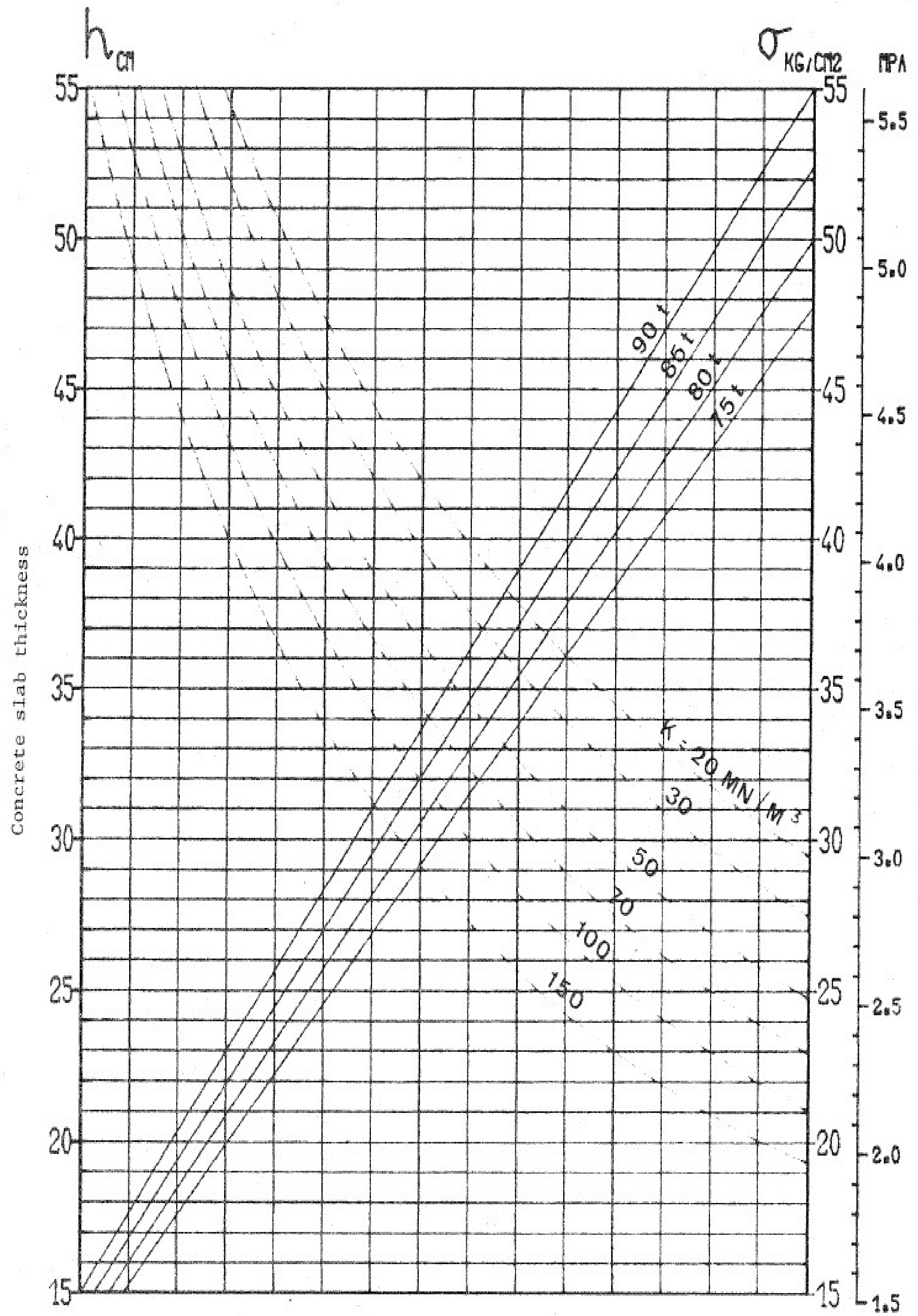


Figure 4-27. Rigid Pavement – typical undercarriage leg – dual tandem loads exceeding 75 tones

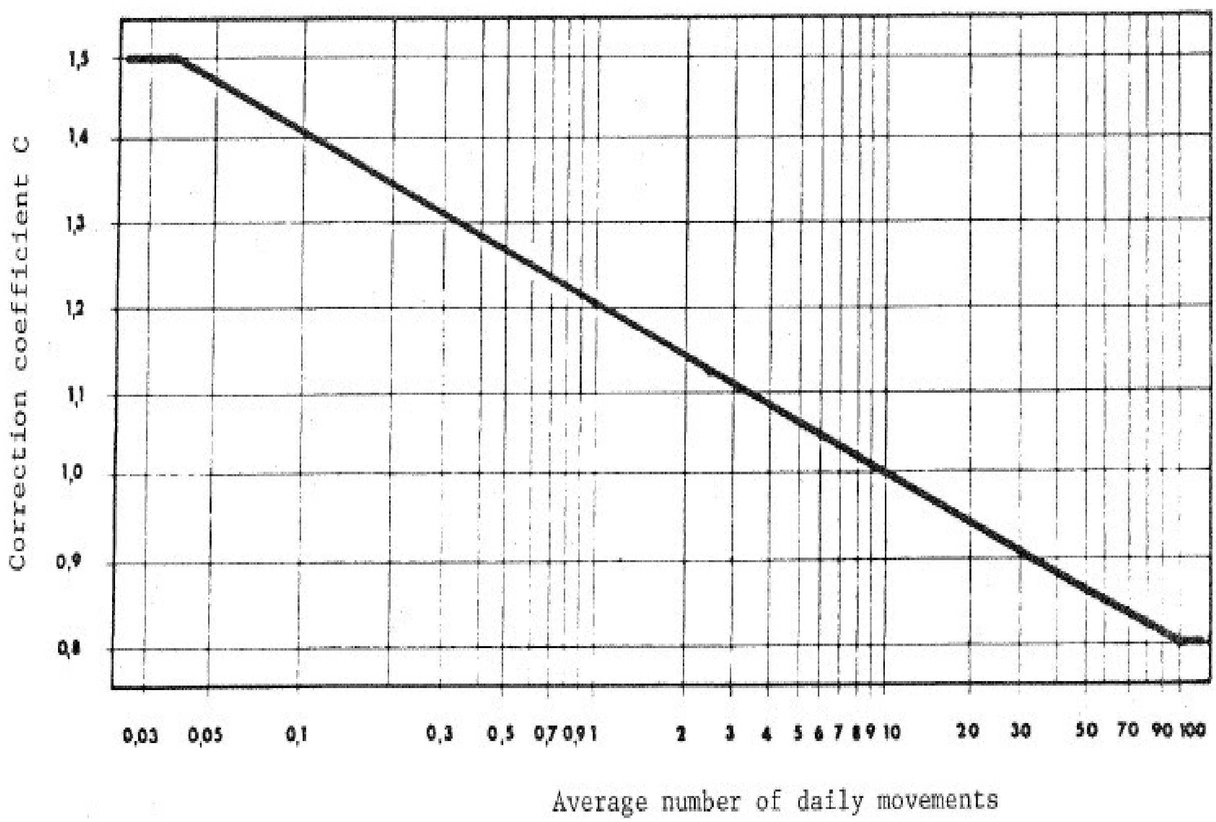


Figure 4-28. Correction of the design load with regard to the traffic intensity

$$\text{Corrected Load} = \frac{\text{Actual Load}}{C}$$

4.2.6. Optimized design

4.2.6.1. Principle. The optimized design method enables a pavement to be designed by taking into account several aircraft types at different frequencies. This method has the advantage that the actual movements of each actual load considered can be converted into equivalent movements of the same reference load. It is thus possible to compare the relative effect of different aircraft. In practice, therefore, the optimized design method is used when several types of aircraft producing approximately the save stresses must be considered (e.g. at major aerodromes), as well as for the purpose of granting concessions (see 2.2.2.2 and 4.2.8). Detailed traffic forecasts according to aircraft type serve as the basis for the design. Bearing in mind that it is sometimes difficult to establish accurate data (particularly for the actual loads), it is recommended that two calculations be made, i.e. one assuming a low traffic volume and the other a high one, with a view to assessing the sensitivity of the different parameters and the error margin for the calculation. Any pavement life may be selected (see 4.2.6.2). The optimized design takes into account the precise number of actual movements of each aircraft for the expected pavement life. Contrary to the general design method there is minimum assumption (1 movement/day or 3650 movements over ten years): the calculated pavement is more sensitive to traffic variations.

4.2.6.2. Pavement life. The life of a pavement (see definition in 4.2.1.1) is normally selected on the basis of the table below:

PAVEMENT LIFE		
Construction	- aerodromes with low traffic - Unreliable traffic forecasts	- aerodromes with heavy traffic - reliable traffic forecasts
Flexible	5 to 10 years	10 years
Rigid	rigid construction not advised	10 to 20 years

A period of ten yea s is normally adopted which correspond to the practice most widely used. The optimized design method takes into account a number of actual movements over a fixed pavement life. Any value may thus be chosen for the latter.

4.2.6.3. Determination of pavement thickness

a. Data required

- Traffic forecasts (for method used to establish these, see 4.2.1.1)
- CBR of natural soil (flexible pavements)
- Modulus of subgrade reaction k and the permissible flexural stress of the concrete (rigid pavements)

b. Calculation method. The calculation consists of applying an “iterative method” which permits the structural integrity under expected traffic to be checked in respect of successive thickness values:

Step 1 – An initial thickness is established.

Step 2 - the equivalent traffic of the expected actual traffic, equaling a number of equivalent movements of the allowable load Po of the structure being tested is calculated. The total number of operations constituting the equivalent traffic may be consolidated in one calculation along the lines of the example shown in Figure 4-29.

Step 3 - Depending on whether the result is less than or more than 36500 equivalent movements, steps 1 and 2 are repeated with a smaller or greater thickness respectively, until a thickness is found where the equivalent traffic is equal or as close as possible to the 36500 equivalent movements.

c. Practical calculation. In this way one can calculate for each air- craft considered as the most critical, the thickness required by its maximum expected mass, taking into account the number of actual movements anticipated at this mass and assuming that it would be the only aircraft using the pavement under study. The maximum thickness thus obtained, plus a few centimeters, usually produces an initial thickness that is fairly close to the final value. The effects of some aircraft quickly become negligible as the thickness is increased in the iterations (as soon as P/Po is less than 0.8). They can be deleted from the tables to simplify the calculations. The minimum increments in the iterations are generally 1 cm for rigid pavements and 1 to 2 cm for flexible pavements which represent the maximum accuracy that may be expected from an optimized design.

1	2	3	4	5	6	7
Aircraft	Actual loads	Allowable loads	$R = P/P_0$	C_p	Actual movements	Equivalent movements
Aircraft 1	$P_1,1$	Po,1	$R_1,1$	$C_{p1},1$	$N_1,1$	$N'_1,1$
	P_1,n_1		R_1,n_1	C_{p1},n_1	N_1,n_1	N'_1,n_1
Aircraft 2	$P_2,1$	Po,2	$R_2,1$	$C_{p2},1$	$N_2,1$	$N'_2,1$
	P_2,n_2		R_2,n_2	C_{p2},n_2	N_2,n_2	N'_2,n_2
. . .	. . .	. . .	. . .	. . .	. . .	. . .
Aircraft M	$P_m,1$	Po,m	$R_m,1$	$C_{pm},1$	$N_m,1$	$N'_m,1$
	P_m,n_m		R_m,n_m	C_{pm},n_m	N_m,n_m	N'_m,n_m
			Total equivalent traffic			$\sum N'$ i, j

Figure 4-29. Computation of total equivalent traffic

1) Subject aircraft. Two models of the same aircraft must be considered to be different if the characteristics of their undercarriage differ (number of wheels, size, and pressure).
2) Actual loads P, considered for each model.
3) Allowable loads Po, calculated by means of the graphs "Flexible pavement" and "Rigid Pavement", as applicable (see Appendix 3). If there is no graph for the subject aircraft, one uses the graph for the aircraft with characteristics closest to the aircraft under study.
4) Relationship R of the actual load P to the allowable load Po. This relationship must not exceed 1.2 for aprons and 1.5 for the other pavements (it is recommended, however, not to exceed 1.2).
5) Weighting coefficient C_p calculated either by means of Figure 4-30 or by applying the formula:

$$C_p = 10^{5(R-1)} \qquad [2]$$

6) Total number N of actual movements per aircraft over the anticipated pavement life.
7) Number N′ of equivalent movements to actual movements calculated by means of the formula:

$$N' = C_p \times N \qquad [3]$$

The total equivalent traffic is obtained by adding the number of equivalent movements in column (7).

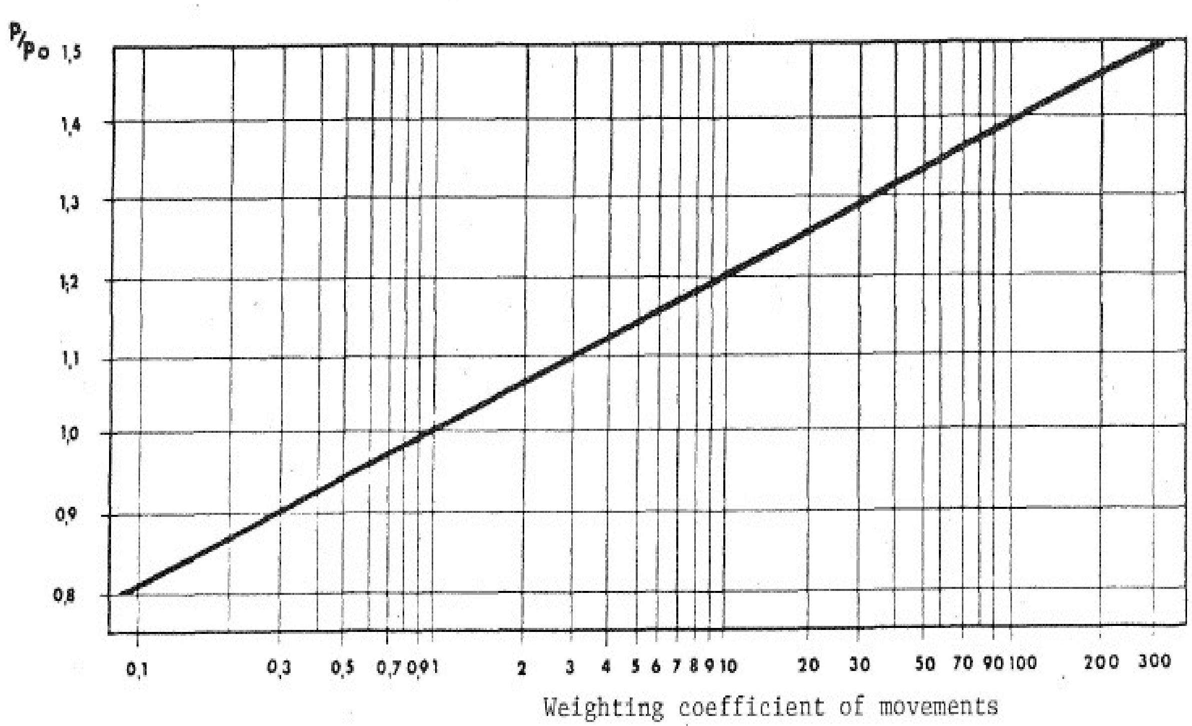


Figure 4-30. Equivalent traffic

Remark

The optimized design method can be used for purposes other than calculating thicknesses, e.g.

1 - Granting of concessions (see 2.2.2.2 and 4.2.8); and

2 - Potential of remaining pavement life (by comparing total and past traffic equivalents for an existing pavement).

4.2.7. Frost

4.2.7.1. It is recommended that structures be tested for the effects of frost-thaw as follows:

a) Classification of soil according to frost susceptibility. The classification of the LaboratorieCentral des Ponts et Chaussees* (Ministere des Transports, France) is used to express the frost susceptibility of soils.

b) Determination of frost penetration.Frost penetration is determined using the modified Berggren method adapted to the multi-layer case.The frost indices and thermic parameters are defined in the same manner as the LCPC.

c) Protecting pavement from frost. There are three feasible protection levels, as follows:

1) Total protection. Protection is calculated so as to ensure that the frost penetration determined for the exceptionally severe winter cannot reach soil layers that may be susceptible to frost.

2) High Protection. Same principle as total protection; however, the frost penetration is calculated for a not exceptionally severe winter.

3) Low protection. It is recognized that frost under severe winter conditions may penetrate a few centimeters into the courses or into frostsusceptible soil. The acceptable depth of penetration largely depends on the individual case and will be determined in consultation with the Administration. The table hereunder shows the recommended protection levels for information:

----

*Abbreviated as LCPC

AERODROME CATEGORY	NATURAL SOIL	
	Homogeneous	Non-homogeneous
LARGE AND MEDIUM AERODROMES*		
Runways and taxiways	H	T
Aprons	H	H
INTERMEDIATE AERODROMES**		
Runways and taxiways	H	H
Aprons	L	L
SMALL AERODROMES***		
Runways and taxiways	L	L
Aprons	L	L

Protection T = Total H = High L = Low

* = annual traffic exceeding 200 000 passengers

** = annual traffic from 50 000 to 200 000 passengers

*** = annual traffic less than 50 000 passengers

4.2.8. Allowable loads

4.2.8.1. Determining the allowable loads for existing pavements is a reciprocal Problem of the design process. Actually, three types of questions are covered by this heading, namely:

a) as regards a specific pavement, how to publish information on its bearing strength in terms of its characteristics;

b) conversely, how can the allowable load for every aircraft be determined from this information (which has been established in a synthetic manner); and

c) under what conditions should concessions be granted if the actual loads exceed the allowable loads

Moreover, in France two systems for the publication of information on runway bearing strength exist side by side, i.e.

- the method based on a typical undercarriage leg applied in France up to now; and
- the ACN - PCN method.

4.2.8.2. It is intended in this section to:

a) describe each of the two methods and the conditions in which they are used;

b) specify interim measures required as a result of using the two methods side by side; and

c) indicate the calculation process used in deciding when concessions should be granted.

4.2.8.3. <u>Publishing information of runway bearing strength</u>

a) <u>Method based on typical undercarriage leg.</u> Since practically all modern aircraft are equipped with undercarriages with single, dual or dual tandem wheel arrangements, the maximum load allowable on each pavement will have to be fixed for each of the three typical under carriages on the basis of ten movements per day over ten years.

<u>Example:</u> 20 t in respect of the single wheel, 35 t in respect of the dual wheel and 50 t in respect of the dual tandem wheel arrangements are expressed symbolically as follows:

The characteristics of the typical undercarriage legs are selected from the most critical landing gear characteristics of current aircraft (see 4.2.2). This method of fixing the allowable loads has the disadvantage of ignoring the variations which in fact exist within the same category of undercarriage. For example, if the track of the dual wheels or the tire pressure is different from that of the typical undercarriage, the effect on the pavement will differ considerably for the same mass of aircraft. Strictly speaking, therefore, an allowable load according to aircraft type should be established for a given pavement. Obviously, this method cannot be applied in practice. However, whenever such a precise calculation is justified (e.g., for the purpose of concessions), the exact landing gear characteristics are taken into account, so that this does not deprive certain aircraft of the advantages they derive from the design of their undercarriage.

20 T/SWL - 35 T/DW - 50 T/DTW

b) <u>ACN-PCN Method</u>

Note - This method is described in CAR-14, Part I and in Chapter 1 of this manual.

4.2.8.4. <u>Choice of a method</u>. The ACN-PCN method came into force for DIPs on 26 November 1981 and is gradually replacing the method based on a typical undercarriage leg.

a) Existing pavements

-A final PCN will be published following the complete evaluation of pavements under the conditions described in Section 4.2.9, and this will replace publications based on a typical undercarriage leg.

-An interim PCN will be published pending an evaluation, together with the existing method of reporting data based on a typical undercarriage leg.

b) Reinforced pavements

-A final PCN will be published following the complete reinforcement of a pavement; this will replace publications based on a typical undercarriage leg for the old pavement.

c) New pavements

-A final PCN will be published for new pavements.

Remark: In areas subject to pronounced seasonal climatic changes, the bearing strength of the subgrade can vary considerably in the course of the year. This may necessitate reporting two sets of PCN values, one for the dry and one for the wet season.

4.2.8.5. Calculating the value to be published

a) Required data. The data required for publishing information on pavement strength consist of:

- Total equivalent thickness and the CBR of the subgrade for flexible pavements.
- Thickness of the slab, permissible flexural stress, modulus of sub- grade reaction k for rigid pavements.

Such data are obtained in the case of:

- Old pavements: from an evaluation of bearing strength under conditions described in 4.2.9.
- Reinforced pavements: from the evaluation of the bearing strength prior to reinforcement and from the characteristics adopted in designing the reinforcement.
- New pavements: from the characteristics adopted for the design with possible corrections to take account of the actual construction.

b) Calculation

- Method based on a typical undercarriage leg. The permissible load Po for a typical undercarriage leg is obtained by using the reverse design method which consists of determining from graphs or formulas the load in terms of the characteristics of the subgrade and the pavement.

- ACN-PCN Method. Determining the PCN is a long and complex operation. The calculation involves the following successive steps:

  Step 1 – Establishing a list of aircraft using or likely to use the pavement under study.

  Step 2 – Calculating, with the aid of the reverse design method, the permissible Poi of the various aircraft in terms of the characteristics of the subgrade and the pavement.

  Step 3 – Calculating for each typical soil category the ACN which corresponds to the permissible load Poi. Subsequently, in each category one considers the PCN included between the maximum and minimum ACN values obtained. The PCN is expressed by two significant figures.

  Step 4 – Searching among the couples (soil category, PCN) for the value that will produce permissible load P′oi that are closest to Poi.

Usually the calculation results in a subgrade category that contains the CBR or modulus k value of the pavement under study. However, it is not unusual to obtain an adjacent subgrade category and the classification thus determined must be interpreted "within the meaning of the ACN-PCN method"

c) The four code letters which follow the PCN are selected in the following manner:

- Type of pavement: the classification is established according to the criteria 4.2.1.1.

- Category of subgrade strength: this is provided at the same time as the PCN by the calculation described above.

- Maximum allowable tire pressure: Code (no pressure limitation) will generally be adopted. Code X (pressure limited to 1.5MPa) is adopted where there is a proven risk of surface damage.

- Evaluation method: the PCN is calculated following a complete evaluation: Code T will normally be adopted. Code U can only be applied for an interim publication of the PCN of a pavement for which there are no reliable results obtained by detailed evaluation and whose behavior has been judged on the basis of its ability to accept existing traffic.

Remarks:

1) For a runway for which several homogeneous areas can be distinguished in regard to bearing strength, the values to be published are the lowest obtained over the entire pavement area.
2) If an area is amenable to a reduction in the normal design load (see 4.2.2.2), weighting is also used in calculating the allowable loads.

4.2.8.6. Using the published values

a) Determination of allowable loads:

1) ACN-PCN method. The allowable load Po of an aircraft is calculated on the basis of the published PCN by the relation:

$$Po = m + (M - m) \cdot \frac{PCN - \min ACN}{\max ACN - \min ACN} \qquad [4]$$

Max ACN: ACN value corresponding to the maximum mass*

Min ACN: ACN value corresponding to the minimum mass (operating mass empty)

2) Typical undercarriage leg method. The allowable load Po on the undercarriage leg of the aircraft under study is that which is published in respect of the corresponding typical undercarriage leg.

Remark: In the case of the pavement for which both the load per typical undercarriage leg and a PCN are published, one adopts the highest value obtained by using one or the other method.

b) Use of allowable loads:

-if the actual load P is less than the allowable load Po there is no restriction (load, number of movements) for the aircraft under study within the over-all fatigue limit of the pavement.

-if the actual load P exceeds load Po: a special study must be carried out which may have the following results for the subject aircraft:

-no restriction

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*See Appendix 5, Table 5-1.

-limited operation* (as regards mass or number of movements under a concession)

-refusal of access

Example

Determination of PCN of a flexible runway with the following characteristics:
total equivalent thickness e = 70 cm

CBR of subgrade CBR=8

The pavement receives traffic consisting almost exclusively of B-727-200, Standard, and Airbus A-300 B2, B-747-100.

Solution

Step 1.The subgrade may be classified in Category B (medium strength) as well as in category C (low strength). These two categories will then be tested in a subsequent calculation.

Aircraft	Load on each main undercarriage leg	Percentage of total mass on each main undercarriage leg	Total mass
A-300 B2	66 t	46.5	142 t
B-727-200 (Standard)	39 t	46.4	84 t
B-747-100	76 t	23.125	329 t

Step 3. Calculation of the ACN corresponding to the allowable load determined for each aircraft.

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* See 2.2.2.2 for guidance on this issue

Step 2. Calculation of allowable loads based on French practice (use of graphs in Appendix 3):

CATEGORY B

A-300 B2 $$ACN = 23^{*} + (45 - 23) \cdot \frac{142\,000 - 85\,690}{142\,000 - 85\,690} = 45$$

B-727-200 (Standard) $$ACN = 22 + (43 - 22) \cdot \frac{84\,000 - 44\,293}{78\,471 - 44\,293} = 46$$

B-747-100 $$ACN = 20 + (50 - 20) \cdot \frac{329\,000 - 162\,703}{334\,751 - 162\,703} = 49$$

CATEGORY C

A-300 B2 $$ACN = 26 + (55 - 26) \cdot \frac{142\,000 - 85\,690}{142\,000 - 85\,690} = 55$$

B-727-200 (Standard) $$ACN = 24 + (49 - 24) \cdot \frac{84\,000 - 44\,293}{78\,471 - 44\,293} = 53$$

B-747-100 $$ACN = 22 + (60 - 22) \cdot \frac{329\,000 - 162\,703}{334\,751 - 162\,703} = 59$$

Step 4. The PCN value to be determined ranges from 45 to 49 if one adopts Category B and between 53 and 59 for Category C. It is noted, however, that the B-727 is acceptable in both cases at a load exceeding the maximum all-up mass. When considering the A-300B2 and the B-747-100 only, the choice is limited within the range 55 to 59 for category C.

Step 5. The final choice is made between the mean values PCN= 47 and PCN = 57 obtained for Categories B and C respectively.

	Allowable load deduced from the PCN		"True" allowable load	Difference
Category B – PCN = 47	A-300 B2:	147.2 t	142 t	+ 5.2 t
	B-747-100:	317.5 t	329 t	-11.5 t
Category C – PCN = 57	A-300 B2:	145.8 t	142 t	+ 3.8 t
	B-747-100	321.2 t	329 t	- 7.8 t

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*See Appendix 5, Table 5-1

The difference between the allowable loads calculated by means of the two methods is less in the second case.

Step 6. Publication

PCN 57/F/C/W/T

4.2.9. Evaluation of pavements

4.2.9.1. General. Evaluation of existing pavements is an indispensable tool in ensuring efficient utilization of their potential. It fulfils three main objectives, as follows:

a) to determine when maintenance operations or more extensive work must be undertaken;
b) at the time such work has to be undertaken, to assess the residual qualities of the pavement with a view to enabling a technical and economic solution to be found and the design for a possible reinforcement to be determined; and
c) to determine, at any time, which aircraft types can use a particular pavement, and their mass and maximum movement frequency (allowable loads described in 4.2.8).

4.2.9.2. Pavement evaluation must take into account both the structural and functional characteristics of the pavements. The structural characteristics of the pavement/subgrade complex govern its bearing strength, i.e. its ability to bear loads imposed by aircraft while retaining its structural integrity during a certain life, The functional characteristics affect the state of the pavement surface and to what extent the pavement can be safely used by aircraft. They are:

a) the quality of the longitudinal profile and, in particular, the evenness which determine the degree of vibrations produced in aircraft during roll out;
b) slipperiness, which determines the degree of directional control and braking of the aircraft; and
c) quality of the surface (crumbling, breaking up of the asphalt, etc.), since defects can damage aircraft (ingestion of small stones by jet engines, tire bursts).

Moreover, the structural and functional characteristics are not independent: thus, the state of the surface can reveal possible structural defects and, conversely, a structure unsuited to the traffic causes deterioration of the surface.

4.2.9.3. Evaluation of pavements is a very complex procedure which calls for a synthesis by a specialist team of the following elements :

a) data on the design of the pavement and of the subsoil, as well as on possible subsequent work (maintenance, reinforcement, etc.);

b) study of the aerodrome site;

c) climatological data (hydrology, ground water, frost, etc);

d) visual inspections of the state of the pavement, surveying the deterioration and examining the drainage;

e) various measurements which enable certain parameters associated with the pavement characteristics (evenness, slipperiness, bearing strength) to be determined; and

f) measurement of the thickness and qualitative assessment of the pavement courses and the characteristics of the subgrade.

4.2.9.4. The following paragraphs deal only with the evaluation of the pavement bearing strength. The purpose of this evaluation is to assign the following representative structural parameters to an existing pavement to represent its current bearing strength which can be directly applied to determine the allowable load and any reinforcement required:

a) the CBR of the subgrade and total equivalent thickness for a flexible pavement; and

b) the modulus of reaction k of the subgrade, thickness of the concrete slab and the permissible flexural stress of the concrete in the case of a rigid pavement.

4.2.9.5. Two approaches may be used to determine these parameters, as follows:

a) by a procedure which is the exact reverse of the design process, the so-called "reverse design method"; and

b) by means of non-destructure plate loading tests on the surface of the pavement which indicate the actual allowable load in the case of a single wheel leg.

In practice, the evaluation of a pavement bearing strength must be made by synthesizing the results of these two complementary approaches.

4.2.9.6. Reverse design method. The purpose of the design method described previously which uses the subgrade data, is to determine a pavement structure that can bear a given traffic over a certain life, provided normal maintenance is performed. Conversely, once the characteristics of the subgrade and of the pavement structure are known, this method enables the traffic which can be accepted during a given time to be determined. The foregoing is the basis for evaluation bearing strength by means of the reverse design method. When this method is used by itself, however, considerable difficulties are encountered in determining the structural parameters that must be taken into account in evaluating an existing pavement and its subgrade. Even if records are available of the construction of the pavement, of any maintenance and reinforcement work performed in the past, and of the traffic accepted, this method requires many trial borings and testing of the pavement. Moreover, there will usually be some uncertainty concerning the results because of the difficulty of evaluating certain parameters (equivalence coefficients of the courses of a flexible pavement, load transfers between concrete slabs, etc.).

Remark: The reverse design method can only be used for a pavement that is correctly constituted (for flexible pavements, the courses must be of increasing quality from bottom to top and adhere closely).

4.2.9.7. Non-destructive plate tests. When interpreted by qualified personnel, non-destructive plate tests can directly provide the allowable load for a single wheel at a large number of points on a flexible pavement and the allowable load at the corners of slabs in the case of a rigid pavement. These tests are insufficient to determine the allowable load for aircraft with multiple wheel undercarriages or to serve as the basis for designing a reinforcement, in which case the reverse design method must be adopted. Nevertheless, the plate tests considerably reduce the number of destructive tests required in order to apply a reliable cross-check in the case of flexible pavements and enable the quality of the load transfer to be evaluated in the case of rigid pavements, as explained in the following paragraph.

4.2.9.8. Test programme to evaluate bearing strength. The amount of equipment required depends on the particular objective and how much is already known about the pavement:

a. If the pavement is old and little is known of its characteristics, all the equipment described below must be used.

b. If the pavement is of recent construction and adequate records are available or the pavement has already been the subject of a comprehensive evaluation of the type described above and changes in bearing strength only are to be determined, non destructive plate tests are usually adequate. This also applies to a pavement which has undergone a complete evaluation followed by reinforcement work, where the results of such work are to be checked.

The following paragraphs deal with the first case, i.e a complete study.

4.2.9.9. Delineation of homogeneous zones

a. The first phase of the study is intended to delineate the zones whose structure and state are identical and to assess their homogeneity in order to reduce the number of other tests needed to determine the pavement structure. To complete the information available from the records, a detailed visual inspection of the pavement must first be performed, including a survey and classification of its deterioration, as well as an inspection of the drainage system.

b. During a second stage, the following may be used:

For flexible pavements: either the Lacroix deflectograph of the LCPC, or the influograph of the STBA*.

For rigid pavements: the equipment for measuring vibration of slabs (DMBD) of the LCPC.

a. Finally, a relatively large number of non-destructive plate tests (from 80 to 100 on a medium-size aerodrome) are performed which not only enable the homogeneity of pavement behaviour to be assessed, as in the case of the above-mentioned equipment, but which also give the value of the allowable load for a single wheel at each of these points.

4.2.9.10. Description of the homogeneous zones. All the above-mentioned equipment is used to define the homogeneous zones on the basis of their structure and behaviour. Having determined the allowable load Po for each homogeneous zone, one or several borings must be performed to evaluate each zone. These borings are performed at one or several points at which plate tests were carried out producing a result Pi close to the allowable load Po adopted for that zone. Some borings are occasionally also performed at specific points (e.g. where the allowable load Pi is particularly low), As an order of magnitude, a total of 6 to 12 trial borings are usually sufficient for a medium size aerodrome, depending on the homogeneity of the pavements tested These trial borings must cover a surface area of approximately 1.5 m^2 and are performed:

a. to determine the structure of the pavement, particularly the thickness of the courses and to check the quality of the materials encountered, if necessary in the laboratory;

b. to undertake CBR tests or tests of the modulus of subgrade reaction whenever possible; and

c. to measure the moisture content and dry density of the subgrade and to take intact or treated samples for laboratory analysis and tests.

4.2.9.11. Interpretation and synthesis of the results. The results for each homogeneous zone are interpreted in the light of the data in respect of the pavement and traffic it has accepted, the surveys of its deterioration, the results of the inspection of the drainage system and all the measurements performed. This synthesis must be carried out by a specialist team, in practice the STBA. Cross-checking of the different measurement values permits making a final choice of the different characteristics required to calculate the allowable loads (see 4.2.8).

4.2.10. Reinforcement of pavements

4.2.10.1. General. The problem of reinforcement of aerodrome pavements can arise when maneuvering areas must be adapted to meet the future requirements of heavier aircraft or when pavements require strengthening to meet immediate needs of current traffic In practice, these two concerns are frequently confused Reinforcement is not the only solution, however, if a particular pavement is not suited to the present or future traffic:

- It may at times be preferable to build a new pavement somewhere else. This solution obviates the difficulty of maintaining the flow of traffic during the reinforcing work; it also allows for the introduction of an improved layout more adapted to new operating conditions.
-

*STBA: Service Technique des Bases Aeriennes, Ministere des Transports, France

- The "substitution" method could also be adopted. This consists of removing the existing pavement and rebuilding a new one at the same level. This solution, which in the case of a runway can be limited to15 v on either side of the centre line, avoids merging problems. However, of all the possible solutions, it is the most expensive one.

The text below deals with the actual reinforcement of pavements; it describes a method for determining the thickness of the reinforcement and deals with certain relevant problems encountered during construction.

4.2.10.2. Choice of solution. The reinforcement for a particular pavement (flexible or rigid) can be of the same type or different. The choice is governed by technical and economic considerations, by the restrictions imposed by the solution on the use of the aerodrome while the work is being carried out and by the bond between the reinforcement and the existing pavement.

4.2.10.3. Choice of the cross- sectional profile. Appreciable savings can be made in the cost of reinforcing a runway by reducing the thickness of the pavement outside a 30 m wide central strip and subject to compatibility with the geometrical standards of the cross-sectional profile. Apart from a saving in reinforcing material, the decrease in thickness of the reinforcement towards the edges of the runway, sometimes down- to nothing, also minimizes or even eliminates the need to raise the level of the shoulders.

4.2.10.4. The thickness of the flexible reinforcement may be obtained using the following relationship:

$$e = 3.75\ (Fh_t - h) \qquad [5]$$

- in this relationship, e is the equivalent thickness in accordance with the definition given in 4.2.3.4. It should be noted .that the materials used for a reinforcement must be a least equal in quality to those used for the sub-base course, i.e. the coefficient of equivalence must be at least 1;

- h is the thickness of the existing concrete slab;

- h_t is the theoretical thickness of the new slab less the existing slab. This thickness is calculated taking into account the allowable stress and the corrected k applicable to the existing slab;

- F is a coefficient of reduction of the thickness ht, the value of which is given in Figure 4-31 as a function of the modulus k already mentioned (the theoretical thickness of the concrete slab is reduced because it is assumed that the slab will crack to a certain extent in service, in contrast with the assumption made in connexion with the calculation for slabs used in the wearing course);

- The equivalent thickness of the reinforcement must not be less than 20 cm, unless special levelling courses are used to correct deformations. Because of the presence of joints and the movement of the slabs, the concrete will have to be covered with a layer of material of sufficient thickness to prevent the appearance of defects at the surface;

- Moreover, the relationship at [5] is applicable only to values resulting in an equivalent thickness e exceeding 20 cm.

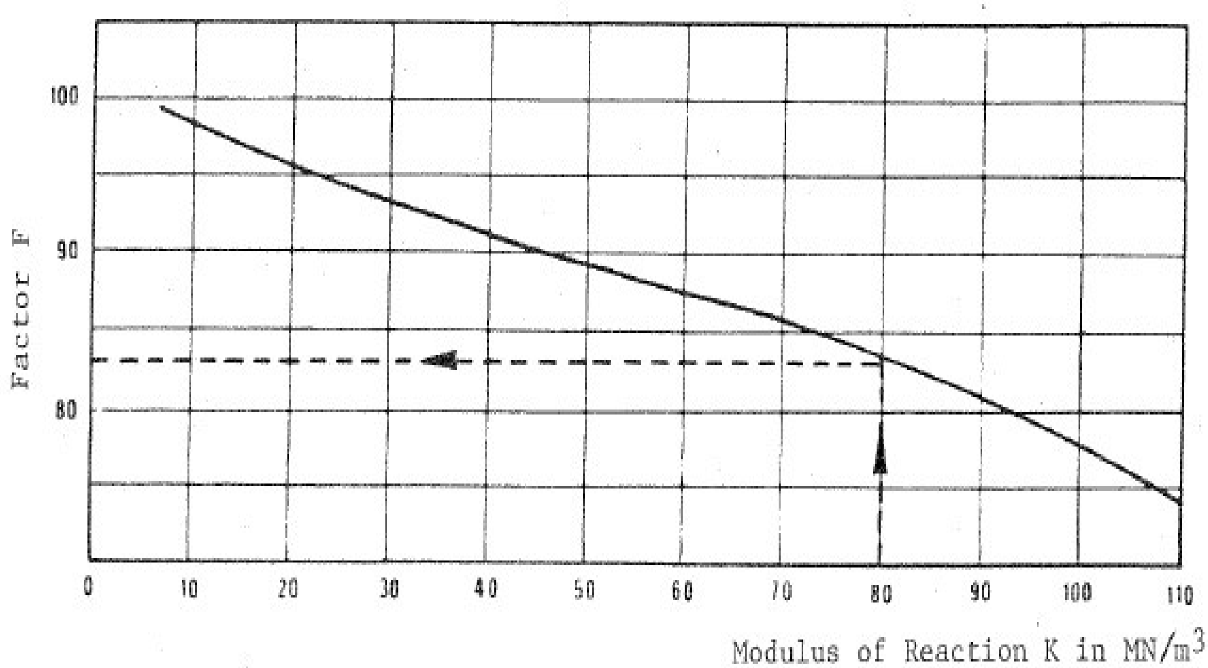


Figure 4-31. Flexible reinforcement on rigid pavement – Factor F

4.2.10.5. Construction rules. The most pressing problem – and one which has not yet been satisfactorily resolved – associated with the direct reinforcement of concrete with a bituminous mix is that of the reappearance of the joint in the rigid pavement at the surface of the reinforcement. Attempts are made to prevent this damage by reinforcing the pavement at these joints by means of metal lattices, plates, fabrics, etc., or at least by separating the course of bituminous mix from the slab over a certain distance on either side of the joint (e.g., by interposing a layer of sand). It is also possible to provide saw cut joints on the surface of the reinforcement to avoid irregular cracking. This solution facilitates maintenance, but reduces the bearing strength of the pavement.

4.2.10.6. Although seldom encountered, another possible difficulty is caused by the affinity of certain jointing compounds for the bitumen, which can result in swelling of the pavement at the joint of the reinforced slab. If in doubt, it will then be advisable to remove the jointing compound before the reinforcement is applied and to refill the joints with a mixture of sand and binder compatible with the one used in the reinforcing course. These rules cannot be applied in the case of reinforcement with concrete, unless the concrete is limited to the central portion of the runway and a "flexible" solution is adopted in the case of the lateral parts.

4.2.10.7. Preliminary Studies. An evaluation of the existing pavement is required (see 4.2.9). Of prime importance is a systematic boring of the pavement in view of the frequent discrepancies in thickness, constitution, etc. of the old pavements.

4.2.10.8. Reinforcement of flexible pavements

a) Flexible reinforcement. The thickness of the reinforcement is determined by the difference between the equivalent thickness required for a new pavement and that of the existing pavement. When .determining the latter, the following should be taken into account:

1) the equivalence coefficient have to be corrected according to the actual condition of the pavement courses; and
2) the equivalence coefficient of a pavement course at a given level cannot be greater than that of the course above it. For instance, if a bituminous mix in good condition (coefficient 2) is covered by coarse-aggregate cement (coefficient 1.5), the coefficient of the former also becomes 1.5.

b) Rigid reinforcement. When a flexible pavement is reinforced with a concrete slab, the former is only considered as a sub-base course in the calculations. The k value which is attributed to this course is determined by reference to Figure 4-13. The thickness of the slab is then established in accordance with 4.2.4, 4.2.5, and 4.2.6.

4.2.10.9. Reinforcement of rigid pavements

a) Flexible reinforcement. If the existing pavement is appreciably fragmented, it is advisable to consider it as a flexible pavement of the same thickness when computing the thickness of the reinforcement. It thus amounts to the same case as described above. The description below presupposes that the existing rigid pavement is still sound (in that case it is still possible to consider the existing rigid pavement as a flexible pavement of the same thickness if this is favourable to the calculations).

b) Rigid reinforcement. The thickness of the reinforcing slab is obtained by applying the formula:

$$h_r = \sqrt[1.4]{h_t^{1.4} - Ch^{1.4}} \qquad [6]$$

- h_t is the theoretical thickness of a new slab determined using the permissible stress in the new concrete and the corrected modulus of reaction for the existing subgrade.

- h is the thickness of the existing concrete slab.

- C is a coefficient introduced in order to take account of the quality of the existing pavement:

- C = 1 for a pavement in good condition,

- C = 0.75 for a pavement exhibiting some cracking at the corners, but not appreciably deteriorated,

- C = 0.35 for a badly fragmented pavement.

In practice one of the two latter values are generally applied.

The above relationship only applies if the reinforcing slab is laid directly on top of the existing pavement, If a layer of material (usually bituminous mix) is interposed between the two slabs, e.g. in order to alter the profile of the existing pavement, the formula for calculating the thickness of the reinforcement becomes:

$$h_r = \sqrt{h_t^2 - Ch^2} \qquad [7]$$

In this expression, the significance of the parameters and the values for coefficient C are the same as detailed previously. This formula results in slightly increased thick- nesses of the reinforcement.

4.2.10.10. Construction rules. To avoid the reappearance of the joints in the existing pavement in the form of cracks in the reinforcing slab, it is essential that the joints be superimposed as accurately as possible. Moreover, all the joints in the existing pavement must have new joints (of any type) above them. In particular, since the old slabs are generally smaller in width than those currently adopted, additional longitudinal contraction-expansion joints may be necessary in the reinforcing slab. E placement of the different reinforcing joints thus calls for a preliminary in-depth study if one wishes to void miscalculations.

4.2.11. Light pavements

4.2.11.1. Light pavements are intended exclusively for aircraft whose total mass does not exceed 5.7 tones. Figure 4-32 may be used to calculate the pavement thickness in relation to the CBR of the natural soil.

4.2.11.2. Allowable loads. The allowable load on a light pavement is 5700kg. The aircraft tire pressure must not exceed 0.6 MPa (approximately 6 kg/cm^2) to avoid any risk of punching. Consequently, the information to be published on pavement strength in accordance with the CAR-14, Part I provisions for light pavements will be 5 700 kg/ 0.6MPa.

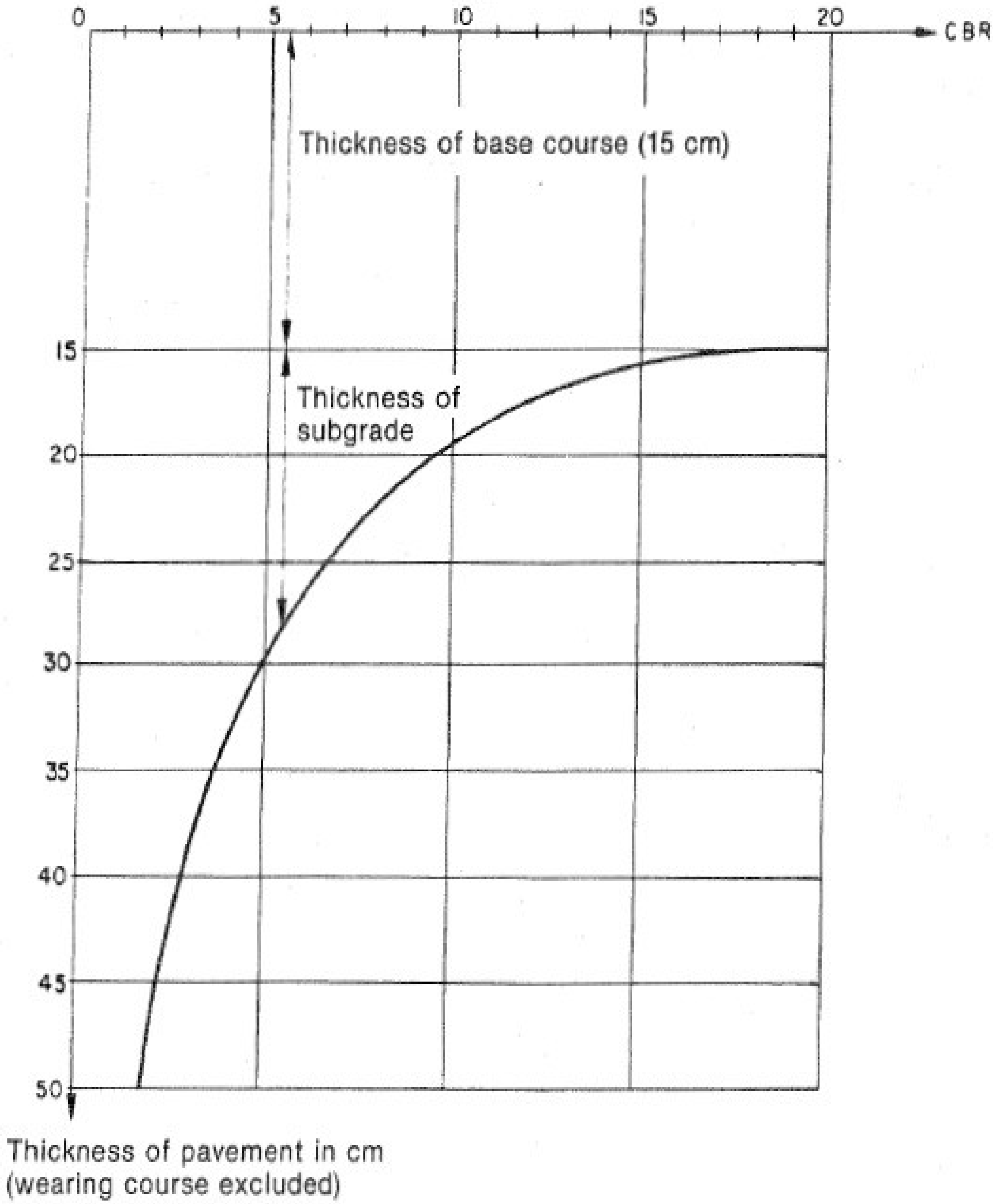


Figure 4-32. Designing a light pavement

4.3. United Kingdom practice

4.3.1. Design and evaluation of pavements

4.3.1.1. It is the United Kingdom practice to design for unlimited operational use by a given aircraft taking into account t e loading resulting from interaction of adjacent landing gear wheel assemblies where applicable. The aircraft is designated "the design aircraft" for the pavement. The support strength classification of the pavement is represented by the design aircraft's pavement classification number identifying its level of loading severity. All other aircraft ranked by the United Kingdom standards as less severe may anticipate unlimited use of the pavement though the final decision rests with the aerodrome authority.

4.3.1.2. While there are now available a number of computer programmes based on plate theory, multilayer elastic theory and finite element analysis, for those wishing to have readily available tabulated data for pavement design and evaluation, the Reference Construction Classification (RCC) system has been developed from the British load Classification Number (LCN) and Load Classification Group (LCG) systems. Pavements are identified as dividing broadly into rigid or flexible construction and analysed accordingly.

4.3.1.3. For the reaction of aircraft on rigid pavements, a simple two layer model is adopted. To establish an aircraft's theoretical depth of reference construction on a range of subgrade support values equating to the ICAO ACN/PCN reporting method, the model is analysed by Westergaard centre case theory. Account is taken of the effect of adjacent landing gear wheel assemblies up to a distance equal to three times the radius of relative stiffness. This is considered essential in any new system in view of the increasing mass of aircraft, complexity of landing gear layouts and the possible interaction of adjacent wheel assemblies on poor subgrades especially.

4.3.1.4. To resolve practical design and evaluation problems, a range of equivalency factors appropriate to the relative strengths of indigenous construction materials is adopted to convert between theoretical model reference construction depths and actual pavement thickness.

4.3.1.5. Aircraft reaction on flexible pavements follows the same basic pattern ado ted for rigid pavement design and evaluation. In this case a four pavement model is "analysed using the United States Corps of Engineers' development of the California Bearing Ratio (CBR) method. This includes Boussinesq deflection factors and takes into account interaction between ad scent landing gear wheel assemblies up to 20 radii distance. Practical design and evaluation problems are resolved using equivalency factors to relate materials and layer thicknesses to the theoretical model on which the reference construction depths for aircraft are assessed.

4.3.2. Reporting pavement strength

4.3.2.1. It is the United Kingdom practice to follow the ICAO ACN/PCN reporting method for aircraft pavements. The critical aircraft is identified as the one which impose a severity of loading condition closest to the maximum permitted on a given pavement for unlimited operational use. Using the critical aircraft's ACN individual aerodrome authorities decide on the PCN to be published for the pavement concerned.

4.3.2.2. Though not revealed by the ICAO ACN/PCN reporting method, when interaction between adjacent landing gear wheel assemblies affects the level of loading imposed by an aircraft, United Kingdom aerodrome authorities may impose restrictions on operations by a mass limitation or a reduction in the number of permitted movements. This is unlikely to occur, however, with aircraft currently in operational use except where subgrade support values are poor.

4.4. United States of America practice

Note.- The specifications in this section, and the calculations upon which they are based, were quoted to ICAO in inches and fractions thereof. Although metric equivalents are provided, in accordance with standard ICAO practice, they cannot be taken as being as precise as the figures quoted in inches.

4.4.1. Introduction

4.4.1.1 The United States Federal Aviation Administration method of designing and reporting airport pavement strength is in terms of gross aircraft weight for each type of landing gear. This permits the evaluation of a pavement with regard to its ability to support the various types and weights of aircraft. Comparison between the pavement strength (reported as gross weight for aircraft equipped with single wheel, dual wheel, and dual-tandem wheel undercarriages) and the actual gross weight of a specific aircraft will establish the pavement's ability to accommodate the aircraft. In 1978 the United States Federal Aviation Administration adopted the California Bearing Ratio (CBR) method of flexible pavement design, edge loading assumption for the design of rigid pavements and the Unified Soil Classification System. This section presents a detailed outline of current procedures and criteria which the United States Federal Aviation Administration has found necessary to follow in pavement design and in conducting a pavement strength evaluation.

4.4.2. Basic investigations and considerations

4.4.2.1 The United States is convinced that there is no quick or simple method of analysing a pavement's strength and that the services of a qualified engineer are essential to ensure a realistic evaluation. The thickness of the pavement and its components is but one of the factors to consider. Environmental features, both climatic and topographic, foundation conditions, quality of materials, and construction methods are all essential elements of any evaluation technique. The following basic investigations should be included in any meaningful evaluation:

a) Pavement condition surveys showing how the existing pavements are holding up under traffic must be conducted in detail. All areas of failure must be accurately mapped and causes of such failures ascertained. It is extremely important that failures due to traffic and load be differentiated from failures due to climate, drainage, and/or poor material, and workmanship;

b) a soil survey must be completed to disclose important variations in soil structure, changes in moisture content, water-bearing layers, water table, and similar determinations;

c) adequate tests, both field and laboratory, should be employed in evaluating the pavement foundation and the pavement's component parts;

d) drainage conditions at the site shall be analysed to ascertain the need for corrective measures prior to any rehabilitation work;

e) an analysis of the traffic history of the airport with regard to both weight of aircraft and number of operations associated with traffic density for the particular area under study must be undertaken and appropriately correlated with pavement performance; and

f) the quality of pavement materials and adequacy of construction methods and practices must be evaluated to determine the degree of conformance with required standards and specifications.

4.4.2.1. The soil survey is not confined to soils encountered in grading or necessarily to the area within the boundaries of the airport site. Possible sources of locally available material that may be used as borrow areas or aggregate sources should be investigated.

4.4.2.2. Samples representative of the different layers of the various soils encountered and various construction material discovered should be obtained and tested in the laboratory to determine their physical and engineering properties. Because the results of a test can only be as good as the sampling, it is of utmost importance that each sample be representative of a particular type of soil material and not be a careless and indiscriminate mixture of several materials.

4.4.2.3. Pits, open cuts, or both may be required for making inplace bearing tests, for the taking of undisturbed samples, for charting variable soil strata, etc. This type of supplemental soil investigation is recommended for situations which warrant a high degree of accuracy or when *in situ* conditions are complex and require extensive investigation.

4.4.3. Soil tests

4.4.3.1. Physical soil properties. To determine the physical properties of a soil and to provide an estimate of its behavior under various conditions, it is necessary to conduct certain soil tests. A number of field and laboratory tests have been developed and standardized. Detailed methods of performing soil tests are completely covered in publications of the American Society for Testing and Materials.

4.4.3.2. Testing requirements. Soil tests are usually identified by terms indicating the soil characteristics which the tests will reveal. Terms which identify the tests considered to be the minimum or basic requirement for airport pavement, with their ASTM designations and brief explanations, follow:

a) Dry preparation of soil samples for practicle-size analysis and determination of soil constants (ASTM D-421) or wet preparation of soil samples for grain-size analysis and determination of soil constants (ASTM D-2217). The dry method (D-421) should be used only for clean, cohesionless granular materials. The wet method (D-2217) should be used for all cohesive or borderline materials. In case of doubt, the wet method should be used.

b) Particle-size analysis of soils (ASTM C-422). This analysis provides a quantitative determination of the distribution of particle sizes in soils.

c) Plastic limit of soils (ASTM D-424). The plastic limit of a soil is defined as the lowest moisture content at which a soil will change from a semi-solid to a plastic state. At moisture contents above the plastic limit, there is a sharp drop in the stability of soils.

d) Liquid limit of soils (ASTM D-423). The liquid limit of a soil is defined as the lowest moisture content at which a soil passes from a plastic to a liquid state. The liquid state is defined as the condition in which the shear resistance of the soil is soslight that a small force will cause it to flow.

e) Plasticity index of soils (ASTM D-424). The plasticity index is the numerical difference between the plastic limit and the liquid limit. It indicates the range in moisture content over which a soil remains in a plastic state prior to changing into a liquid.

f) Moisture density relations of soils (ASTM D-698, D-1557). For purposes of compaction control during construction, tests to determine the moisture-density relations of the different types of soils should be performed.

1. For pavements designed to serve aircraft weighing 30 000 Ib (13000 kg) or more, use ASTM Method D-1557.

2. For pavements designed to serve aircraft weighing less than 30000 lb (13000 kg), use ASTM Method D-698.

4.4.3.3. Supplemental tests. In many cases additional soil tests will be required over those listed in 4.4.3.2 above. It is not possible to cover all the additional tests which may be required; however, a few examples are presented below. This list is not to be considered a complete list by any means.

a) Shrinkage factors of soils (AS D-427). This text may be required in areas where swelling soils might be encountered.

b) Permeability of granular soils (ASTM D-2434). This test may be needed to assist in the design of subsurface drainage.

c) Determination of organic material in soils by wet combustion (AASHTO T-194). This test may be needed in areas where deep pockets of organic material are encountered or suspected.

d) Bearing ratio of laboratory - compacted soils (ASTM D-1883).This test is used to assign a California Bearing Ratio (CBR) value to subgrade soils for use in the design of flexible pavements.

e) Modulus of soil reaction (AASHTOT 222). This test is used to determine the modulus of soil reaction, K, for use in the design of rigid pavements .

f) California bearing ratio, field in-place tests. Field bearing tests can be performed when the *in situ* conditions satisfy density and moisture conditions which will exist under the pavement being designed.

4.4.4. Unified soil classification system

4.4.4.1. The standard method of classifying soils for engineering purposes is ASTM DG2487, commonly called the Unified system. The change from the FAA system to the Unified system is based on the results of a research study which compared three different methods of soil classification. The research study concluded the Unified system is superior in detecting properties of soils which affect airport pavement performance. The primary purpose in determining the soil classification is to enable the engineer to predict probable field behaviour of soils. The soil constants in themselves also provide some guidance on which to base performance predictions. The Unified system classifies soils first on the basis of grain size, then further subgroups soils on the plasticity constants. Table 4-7 presents the classification of soils by the Unified system.

4.4.4.2. As indicated in Table 4-7, the initial division of soils is based on the separation of course and fine-grained soils and highly organic soils. The distinction between coarse and fine grained is determined by the amount of material retained on the No. 200 sieve. Coarse-grained soils are further subdivided into gravels and sands on the basis of the amount of material retained on the No. 4 sieve. Gravels and sands are then classed according to whether or not fine material is present. Fine-grained soils are "subdivided into two groups on the basis of liquid limit. A separate division of highly organic soils is established for materials which are not generally suitable for construction purposes. The final classification of soild subdivides materials into 15 different groupings. The group symbols and a brief description of each is given below:

a) GW - Well-graded gravels and gravel-sand mixtures, little or no fines.

b) GP - Poorly graded gravels and gravel-sand mixtures, little or no fines.

c) GM - Silty gravels, gravel-sand-silt mixtures.

d) GC - Clayed gravels, gravel-sand-clay mixtures.

e) SW - Well-graded sands and gravelly sands, little or no fines.

f) SP - Poorly graded sands and gravely sands, little or no fines.

g) SM - Silty sands, sand-silt mixtures.

h) SC - Clayed sands, sand-clay mixtures.

i) ML - Inorganic silts, very fine sands, rock flour, silty or clayey fine sands.

j) CL - Inorganic clays of low to medium plasticity, gravelly clays, silty clays, lean clays.

k) OL - Organic silts and organic silty clays of low plasticity

l) MH - Inorganic silts, micaceous or diatomaceous fine sands or silts, plastic silts.

m) CH - Inorganic clays or high plasticity, fat clays.

n) OH - Organic clays of medium to high plasticity.

o) PT - Peat, muck and other highly organic soils.

Table 4-7. Classification of soils for airport pavement applications

MAJOR DIVISIONS			Groups symbols
Coarse-grained soils more than 50% retained on No. 200 sieve 1/	Gravels 50% or more of coarse fraction retained on No. 4 sieve	Clean gravels	GW GP
		Gravels with fines	GM GC
	Sands less than 50% of coarse fraction retained on No. 4 sieve	Clean sands	SW SP
		Sands with fines	SM SC
Fine-grained soils 50% or less retained on No. 200 sieve 1/	Silts and clays liquid limit 50% or less		ML CL OL
	Silts and clays liquid limit greater than 50%		MH CH OH
Highly organic soils			PT

1/ Based on the material passing the 3 in (75 mm) sieve.

4.4.4.3. Determination of the final classification group requires other criteria in addition to those give in Table 4-7. These additional criteria are presented in Figure 4-33 and have application to both coarse and fine-grained soils.

4.4.4.4. A flow chart which outlines the soil classification process has been developed and is included as Figure 4-34. This flow chart indicates the steps necessary to classify soils in accordance with ASTM D-2487.

4.4.4.5. A major advantage of the ASTM D-2487 Unified system of classifying soils is that a simple, rapid method of field classification has also been developed; see ASTM D-2488, *Description of soils (Visual-manual procedure).* This procedure enables field personnel to classify soils rather accurately with a minimum of time and equipment.

CLASSIFICATION CRITERIA		
Classification on basis of percentage of fines Less than 5% Pass No. 200 sieve GW, GP, SW, SP More than 12% Pass No. 200 sieve GM, GC, SM, SC 5% to 12% Pass No. 200 sieve Borderline Classification requiring use of dual symbols	$C_u = D_{60}/D_{10}$ Greater than 4 $C_z = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
	Not meeting both criteria for GW	
	Atterberg limits plot below "A" line or plasticity index less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
	Atterberg limits plot above "A" line and plasticity index greater than 7	
	$C_u = D_{60}/D_{10}$ Greater than 6 $C_z = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
	Not meeting both criteria for SW	
	Atterberg limits plot below "A" line or plasticity index less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
	Atterberg limits plot above "A" line and plasticity index greater than 7	

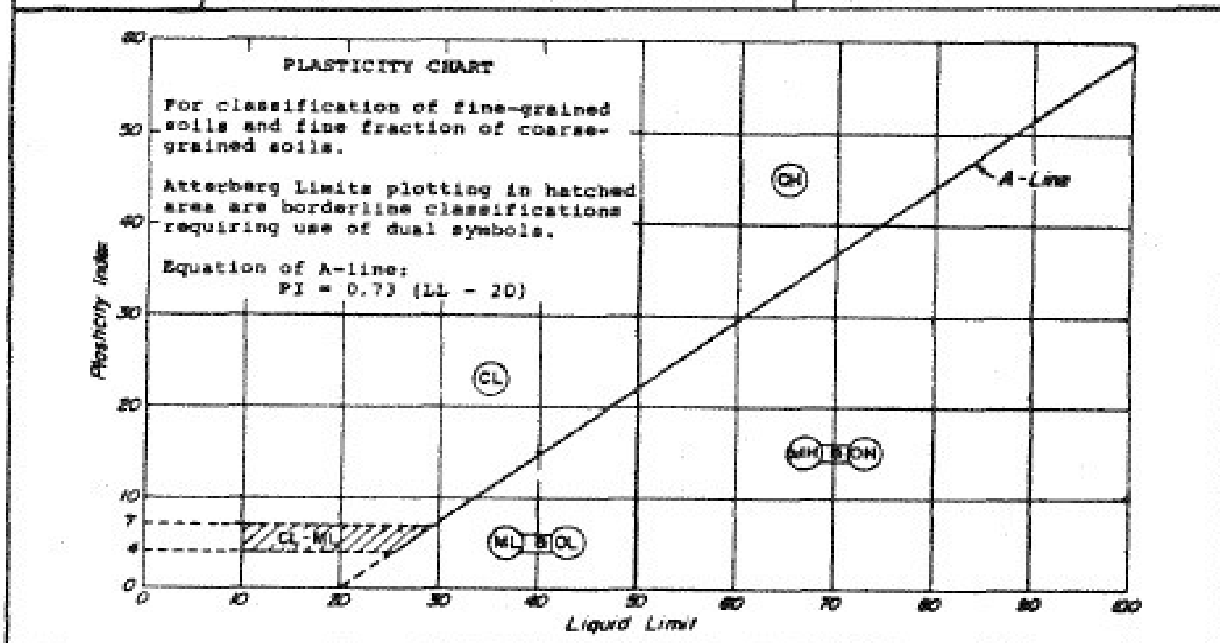


Figure 4-33. Soil Classification criteria

4.4.4.6. A table of pertinent characteristics of soils used for pavement foundations is presented in able 4-8. These characteristics are to be considered as approximate, and the values listed are generalizations which should not be used in lieu of testing.

4.4.5. Soil classification examples

4.4.5.1. The following examples illustrate the classification of soils by the Unified system. The classification process progresses through the flow chart shown in Figure 4-34.

<u>Example</u> 1

Assume a soil sample has the following properties and is to be classified in accordance with the Unified system.

Percentage passing No. 200 sieve - 98 per cent.

Liquid limit on minus 40 material - 30 per cent.

Plastic limit on minus 40 material - lo per cent.

<u>Solution</u>

See above "A" line, Figure 4-33. The soil would be classified as CL, lean clay of low to medium plasticity Table 4-8 indicates the material would be of fair to poor value as a foundation when not: subject to frost action. The potential for frost action is medium to high.

<u>Example</u> 2

Assume a soil sample with the following properties is to be classified by the Unified system.

Percentage passing No. 200 sieve – 48 per cent.

Percentage of coarse fraction retained on No. 4 sieve - 70 per cent.

Liquid limit on minus 40 fraction - 60 per cent.

Plastic 1imit on minus 40 f action - 20 per cent.

Table 4-8. Characteristics pertinent to pavement foundations

Major Divisions (1)	(2)	Letter (3)	Name (4)	Value as Foundation When Not Subject to Frost Action (5)	Value as Base Directly under Wearing Surface (6)	Potential Frost Action (7)	Compressibility and Expansion (8)	Drainage Characteristics (9)	Compaction Equipment (10)	Unit Dry Weight lb/ft³ (11)	Field CBR (12)	Subgrade Modulus & lb/in³ (13)
Coarse-grained soils	Gravel and gravelly soils	GW	Gravel or sandy gravel, well graded	Excellent	Good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment, steel-wheeled roller	125–140	60–80	300 or more
		GP	Gravel or sandy gravel, poorly graded	Good to excellent	Poor to fair	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment, steel-wheeled roller	120–130	35–60	300 or more
		GU	Gravel or sandy gravel, uniformly graded	Good	Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	115–125	25–50	300 or more
		GM	Silty gravel or silty sandy gravel	Good to excellent	Fair to good	Slight to medium	Very slight	Fair to poor	Rubber-tired equipment, sheepsfoot roller, close control of moisture	130–145	40–80	300 or more
		GC	Clayey gravel or clayey sandy gravel	Good	Poor	Slight to medium	Slight	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	120–140	20–40	200–300
	Sand and sandy soils	SW	Sand or gravelly sand, well graded	Good	Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	110–130	20–40	200–300
		SP	Sand or gravelly sand, poorly graded	Fair to good	Poor to not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	105–120	15–25	200–300
		SU	Sand or gravelly sand, uniformly graded	Fair to good	Not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	100–115	10–20	200–300
		SM	Silty sand or silty gravelly sand	Good	Poor	Slight to high	Very slight	Fair to poor	Rubber-tired equipment, sheepsfoot roller; close control of moisture	120–135	20–40	200–300
		SC	Clayey sand or clayey gravelly sand	Fair to good	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	105–130	10–20	200–300
Fine-grained soils	Low compressibility LL < 50	ML	Silts, sandy silts, gravelly silts, or diatomaceous soils	Fair to poor	Not suitable	Medium to very high	Slight to medium	Fair to poor	Rubber-tired equipment, sheepsfoot roller, close control of moisture	100–125	5–15	100–200
		CL	Lean clays, sandy clays, or gravelly clays	Fair to poor	Not suitable	Medium to high	Medium	Practically impervious	Rubber-tired equipment, sheepsfoot roller	100–125	5–15	100–200
		OL	Organic silts or lean organic clays	Poor	Not suitable	Medium to high	Medium to high	Poor	Rubber-tired equipment, sheepsfoot roller	90–105	4–8	100–200
	High compressibility LL > 50	MH	Micaceous clays or diatomaceous soils	Poor	Not suitable	Medium to very high	High	Fair to poor	Rubber-tired equipment, sheepsfoot roller	80–100	4–8	100–200
		CH	Fat clays	Poor to very poor	Not suitable	Medium	High	Practically impervious	Rubber-tired equipment, sheepsfoot roller	90–110	3–5	50–100
		OH	Fat organic clays	Poor to very poor	Not suitable	Medium	High	Practically impervious	Rubber-tired equipment, sheepsfoot roller	80–105	3–5	50–100
Peat and other fibrous organic soils		Pt	Peat, humus, and other	Not suitable	Not suitable	Slight	Very high	Fair to poor	Compaction not practical			

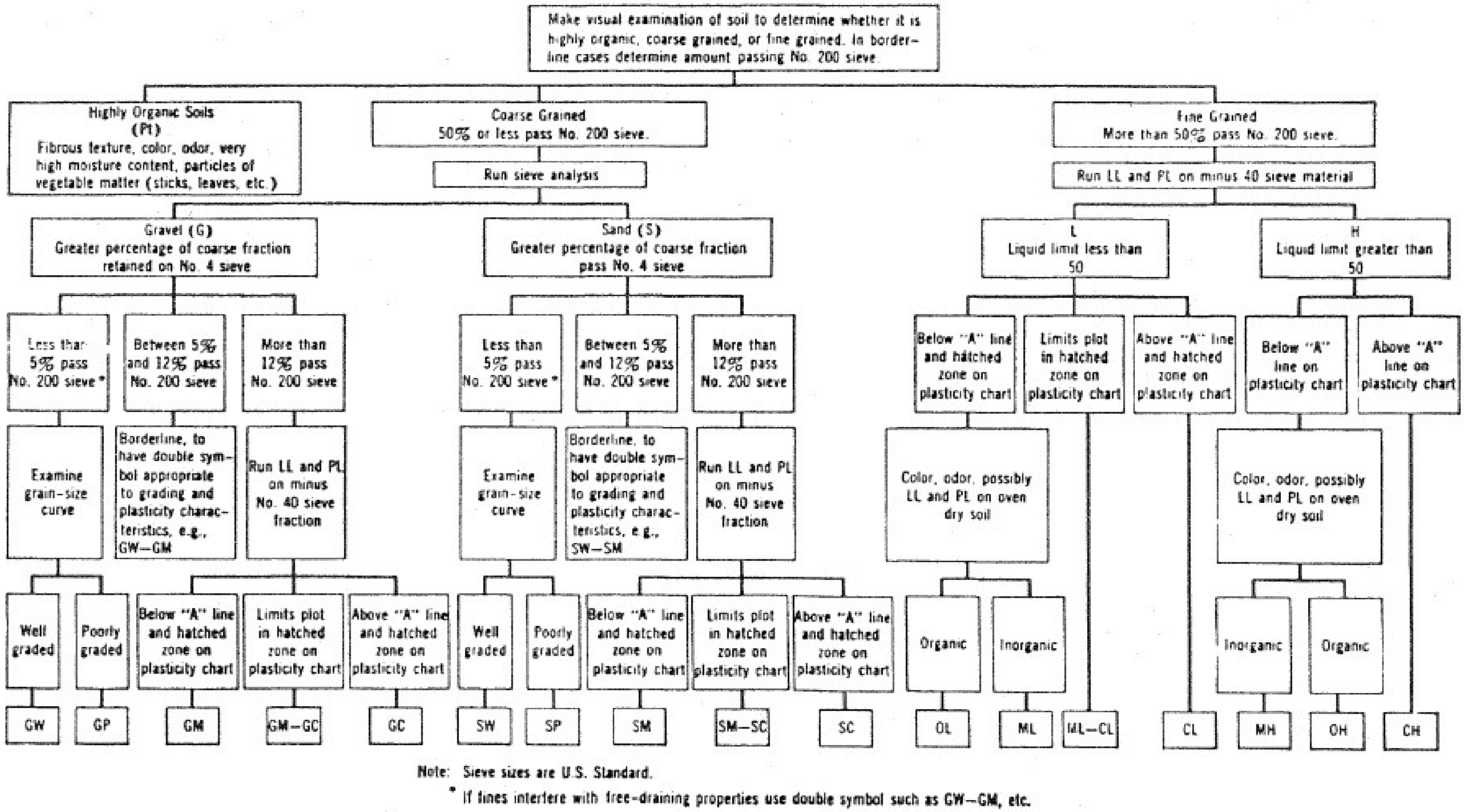


Figure 4-34. Flow chart for unified soil classification system

Solution

Compute plasticity index LL-PL - 40 per cent

See above "A" line , Figure 4-33 .

This sample is classified as GC, clayey gravel. Table 4-8 indicates the material is good for use as a pavement foundation when not subject to frost action. The potential for frost action is slight to medium.

4.4.6. Frost and permafrost

4.4.6.1. The design of pavements in areas subject to frost action or in areas of permafrost is a complex problem requiring detailed study. The detrimental effects of frost action may be manifested in frost heave or in loss of foundation support through frost melting.

4.4.6.2. The design of pavements for seasonal frost conditions can be accomplished in four different ways.

a) Complete protection method involves the removal of frost susceptible material to the depth of frost penetration and replacing the material with non frost susceptible material.

b) Limited subgrade frost penetration method allows the frost to penetrate a limited depth into the frost susceptible subgrade. This method holds deformations to small acceptable values.

c) Reduced subgrade strength method usually permits less "pavement thickness than the two methods discussed above and should be applied to pavements where aircraft speeds are low and the effects of frost heave are less objectionable. The primary aim of this method is to provide adequate structural capacity for the pavement during the frost melt period. Frost heave is not the primary consideration in this method.

d) Reduced subgrade frost protection method provides the designer method of statistically handling frost design. This method should only be used where aircraft speeds are low and some frost heave can be tolerated. The statistical approach allows the designer more latitude than the other three methods discussed above.

4.4.6.3. The design of pavements in permafrost areas requires efforts to restrict the depth of thaw. Thawing of the permafrost can result in loss of bearing strength. If thawed permafrost is refrozen, heaving can result and cause pavement roughness and cracking. Two methods of design are available for construction in permafrost areas, complete protection method and the reduced subgrade strength method. These methods are somewhat similar to the methods discussed under 4.4.6.2 for seasonal frost design.

4.4.6.4. The depth of frost penetration can be computed using the modified Berggren equation. The Berggren equation requires several inputs concerning 1ocal soil conditions and local temperature data. Utility companies near the site can also provide valuable data concerning frost depth. The designer should be cautioned that the depths of cover required to protect utility lines are conservative and generally exceed the depths of frost penetration.

4.4.6.5. The frost design procedures discussed herein can be found in FAA Research Report FAA-RD-74-30, *Design of civil airfield pavement for seasonal frost and permafrost conditions*. Another valuable reference for frost and permafrost design is United States Army Corps of Engineers Technical Manual TM 5-811-2, *Pavement design for frost conditions*.

4.4.7. Soil strength tests

4.4.7.1. Soil classification for engineering purposes provides an indication of the probable behaviour of the soil as a pavement subgrade. This indication of behaviour is, however, approximate. Performance different from that expected can occur due to a Variety of reasons such as degree of compaction, degree of saturation, height of overburden, etc. The possibility of incorrectly predicting subgrade behaviour can be largely eliminated by measuring soil strength. The strength of materials intended for use in flexible pavement structures is measured by the California Bearing Ratio (CBR) tests. Materials intended for use in rigid pavement structures are tested by the plate-bearing method of test. Each of these tests is discussed in greater detail in the subsequent paragraphs.

4.4.7.2. California bearing ratio. The CBR test is basically a penetration test conducted at a uniform rate of strain. The force required to produce a given penetration in the material under test is compared to the force required to produce the same penetration in a standard crushed limestone. The result is expressed as a ratio of the two forces. us a material with a CBR value of 15 means the material in question offers 15 per cent of the resistance to penetration that the standard crushed stone offers. Laboratory CBR tests should be performed in accordance with ASTM D-1883, *Bearing ratio of laboratory-compacted soils.* Field GER tests should be conducted in accordance with the procedures given in Manual Series No. 10 (MS-10) by The Asphalt Institute.

a) Laboratory CBR tests are conducted on materials which have been obtained from the site and remolded to the density which will be obtained during construction. Specimens are soaked for four days to allow the material to reach saturation. A saturated CBR test is used to simulate the conditions likely to occur in a pavement which has been in service for some time. Pavement foundations tend to reach nearly complete saturation after about three years. Seasonal moisture changes also dictate the use of a saturated CBR design value since traffic must be supported during periods of high moisture such as spring seasons.

b) Field GER tests can provide valuable information on foundations which have been in place for several years. The materials should have been in place for a sufficient time to. allow for the moisture to reach an equilibrium condition. An example of this condition is a fill which has been constructed and surcharged for a long period of time prior to pavement construction.

c) CBR tests on gravelly materials are difficult to interpret. Laboratory CBR tests on gravel often yield CBR results which are too high owing to the confining effects of the mould. The assignment of CBR values to gravelly subgrade materials may be based on judgement and experience. The information given in Table 4-8 may provide helpful guidance in selecting a design CBR value for a gravelly soil. Table 4-8 should not, however, be used indiscriminately as a sole source of data. It is recommended that the maximum CBR for unstabilized gravel subgrade be 50.

d) The number of CBR tests needed to properly establish a design value cannot be simply stated. Variability of the soil conditions encountered at the site will have the greatest influence on the number of tests needed. As an approximate "rule of thumb" three CBR tests on each different major soil type should be considered. The preliminary soil survey will reveal how many different soil types will be encountered. The design CBR value should be conservatively selected Common paving engineering practice is to select a value which is one standard deviation below the mean.

4.4.7.3. Plate bearing test. As the name indicates, the plate bearing test measures the bearing capacity of the pavement foundation. The plate bearing test result is expressed as a k value which has the units of pressure over length. The k value can be envisioned as the pressure required to produce a unit deformation of a bearing plate into the pavement foundation. Plate bearing tests should be performed in accordance with the procedures established in AASTO T 222.

a) Rigid pavement design is not too sensitive to the k value. An error in establishing a k value will not have a drastic impact on the design thickness of the rigid pavement. Plate bearing tests must be conducted in the field and are best performed on test sections which are constructed to the design compaction and moisture conditions. A correction to the k value for saturation is required to simulate the moisture conditions likely to be encountered by the in-service pavement.

b) Plate bearing tests are relatively expensive to perform and thus the number of tests which can be conducted to establish a design value is limited. Generally, only two or three tests can be performed for each pavement feature. The design k' value should be conservatively selected.

c) The rigid pavement design and evaluation curves presented in this material are based on a k value determined by a static plate load test using a 30 in (762 mm) diameter plate. Use of a plate of smaller diameter will result in a higher k value than is represented in the design and evaluation curves.

d) It is recommended that plate bearing tests be conducted on the subgrade and the results adjusted to account for the effect of sub-base. Figure 4-35 shows the increase in k value for various thicknesses of sub-base over a given subgrade k. Plate bearing tests conducted on top of sub-base courses can sometimes yield erroneous results since the depth of influence beneath a 30 in(762) bearing plate is not as great as the depth of influence beneath a slab loaded with an aircraft landing gear assembly. In this instance a sub-base layer can influence the response of a bearing plate more than the response of a loaded pavement.

e) The determination of k value for stabilized layers is a difficult problem. The k value normally has to be estimated. It is recommended that the k value be estimated as follows. The thickness of the stabilized layer should be multiplied by a factor ranging from 1.2 to 1.6 to determine the equivalent thickness of well-graded crushed aggregate. The actual value in the 1.2 to 1.6 range should be based on the quality of the stabilized layer and the thickness of the slab relative to the thickness of the stabilized layer. High-quality materials which are stabilized with high percentages of stabilizers should be assigned an equivalency factor which is higher than a lower-quality stabilized material. For a given rigid pavement thickness a thicker stabilized layer will influence pavement performance more than a thin stabilized layer and should thus be assigned a higher equivalency factor.

f) It is recommended that a design k value of 500 lb/in3 (136 MN/m^3) not be exceeded for any foundation. The information presented in Table 4-8 gives general guidance as to probable k values for various soil types.

4.4.8. Pavement design philosophy

4.4.8.1. The FAA policy of treating the design of aircraft landing gear and the design and evaluation of airport pavements as three separate entities is described in 4.4.1 of this Manual. The design of airport pavements is a complex engineering problem which involves a large number of interacting variables. The design curves presented in this Section are based on the CBR method of design for flexible pavements and a jointed edge stress analysis for rigid pavements. These procedures represent a change from prior FAA design methods and will result in slightly different pavement thicknesses. Because of thickness variations, the evaluation of existing pavements should be performed using the same method as was employed in the design. Details on how the new FAA methods of design were developed are as follows:

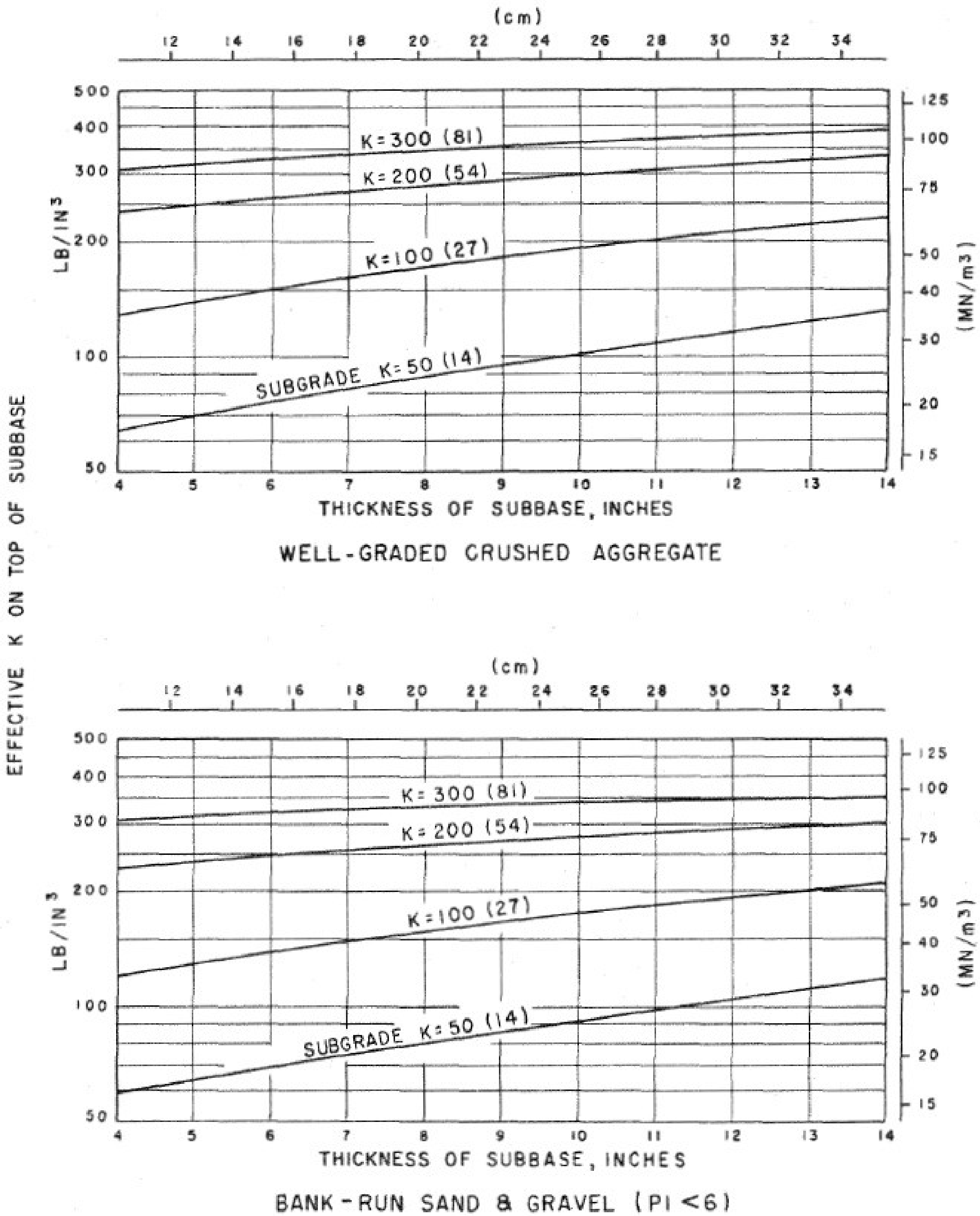


Figure 4-35. Effect of sub-base on modulus of subgrade reaction

4.4.8.2. Flexible pavements, The flexible pavement design curves presented in this Section are based on the California Bearing Ratio (CBR) method of design. The GER design method is basically empirical;

however, a great deal of research has been done with the method and reliable correlations have been developed. Gear configurations are related using theoretical concepts as well as empirically developed data. The design curves provide the required total thickness of flexible pavement (surface, base, and sub-base) needed to support a given weight of aircraft over a particular subgrade. The curves also show the required surface thickness. Minimum base course thicknesses are shown on a separate curve. A more detailed discussion of GER design is presented in Appendix 4.

4.4.8.3. Rigid pavements. The rigid pavement design curves in this Section are based on the Westergaard analysis of edge loading. The edge loading analysis has been modified to simulate a jointed edge condition. Design curves are furnished for areas where traffic will predominantly follow parallel to the joints and for areas where traffic is likely to cross joints at some acute angle. Previous FAA rigid pavement criteria were based on an interior loading assumption. Pavement stresses are higher at the jointed edge than at the slab interior. Test validations and field performance how practically all load induced cracks develop at the jointed edge and migrate towards the slab interior. For these reasons the basis of design was changed from interior to jointed edge. The design curves contain lines for five different annual traffic volumes. e thickness of pavement determined from the curves is for slab thickness only. Sub-base thicknesses are determined separately. A more detailed discussion of the basis for rigid pavement design is presented in Appendix 4.

4.4.9. Background

4.4.9.1. An airfield pavement and the operating aircraft represent an interactive system which must be recognized in the pavement design process. Design considerations associated with both the aircraft and the pavement must be satisfied in order to produce a satisfactory design. Careful construction control and some degree of maintenance will be required to produce a pavement which will achieve the intended design life. Pavements are designed to provide a finite life and fatigue failures are anticipated. Poor construction and lack of preventative maintenance will usually result in disappointing performance of even the best designed pavement.

4.4.9.2. The determination of pavement thickness requirements is a complex engineering problem. Pavements are subject to a wide variety of loadings and climatic effects. The design process involves a large number of interacting variables which are often difficult to quantify. Although a great deal of research work has been completed and more is underway, it has been impossible to arrive at a direct mathematical solution of thickness requirements. For this reason the determination of pavement thickness must be based on the theoretical anal sis of load distribution through pavements and soils, the analysis of experimental pavement data, and a study of the performance of pavements under actual service conditions. Pavement thickness curves presented in this Section have been developed through correlation of the data obtained from these sources. Pavements designed in accordance with these standards are intended to provide a structural life of 20 years that is free of major maintenance if no major changes in forecast traffic are encountered. It is likely that rehabilitation of surface grades and renewal of skid resistant properties will be needed before 20 years owing to destructive climatic effects and deteriorating effects of normal usage.

4.4.9.3. The structural design of airport pavements consists of determining both the overall pavement thickness and the thickness of the component parts of the pavement. There are a number of factors which influence the thickness of pavement required to provide satisfactory service. These include the magnitude and character of the aircraft loads to be supported, the volume of traffic, the concentration of traffic in certain areas, and the quality of the subgrade soil and materials comprising the pavement structure.

4.4.10. Aircraft considerations

4.4.10.1. Load. The pavement design method is based on the gross weight of the aircraft. For design purposes the pavement should be designed for the maximum take-off weight of the aircraft. The design procedure assumes 95 per cent of the gross weight is carried by the main landing gears and 5 per cent is carried by the nose gear. The maximum take-off weight should be used in calculating the pavement thickness required. Use of the maximum take-off weight is recommended to provide some degree of conservatism in the design and is justified by the fact that changes in operational use can often occur and recognition of the fact that forecast traffic is approximate at best. By ignoring arriving traffic some of the conservatism is offset.

4.4.10.2. Landing gear type and geometry

a) The gear type and configuration dictate how the aircraft weight is distributed to the pavement and determine pavement response to aircraft loadings. It would have been impractical to develop design curves for each type of aircraft. However, since the thickness of both rigid and flexible pavements is dependent upon the gear dimensions and the type of gear, separate design curves would be necessary unless some valid assumptions could be made to reduce the number of variables. Examination of gear configuration, tire contact areas, and tire pressure in common use indicated that these follow a definite trend related to aircraft gross weight. Reasonable assumptions could therefore be made and design curves constructed from the assumed data. These assumed data are as follows:
    1. Single gear aircraft. No special assumptions needed.
    2. Dual gear aircraft. A study of the spacing between dual wheels for these aircraft indicated that a dimension of 20 in (0.51 m) between the centerline of the tires appeared reasonable for the lighter aircraft and a dimension of 34 in (0.86 m) between the centerline of the tires appeared reasonable for the heavier aircraft.
    3. Dual tandem gear aircraft. The study indicated a dual wheel spacing of 20 in (0.51 m) and a tandem spacing of 45 in (1.14 m) for lighter aircraft, and a dual wheel spacing of 30 in (0.76 m) and a tandem spacing of 55 in (1.40 m) for "the heavier aircraft are appropriate design values.
    4. Wide body aircraft. Wide body aircraft, i.e.,B-747, DC-10, and L-l0l1 represent a radical departure from the geometry assumed for dual tandem aircraft described in 3 above. Owing to the large differences in gross weights and gear geometries, separate design curves have been prepared for the wide body aircraft

b) Tire pressure varies between 75 and 200 psi (0.52 to 1.38 MPa) depending on gear configuration and gross weight. It should be noted that tire pressure asserts less influence on pavement stresses as gross weight increases, and the assumed maximum of 200 psi (1.38 MPa) may be safely exceeded if other parameters are not exceeded.

4.4.10.3. Traffic volume. Forecasts of annual departures by aircraft type are needed for pavement design. Information on aircraft operations is available from Airport aster Plans, Terminal Area Forecasts, the National Airport System Plan, Airport Activity Statistics and FAA Air traffic Activity. These publications should be consulted in the development of forecasts of annual departures by aircraft type.

4.4.11. Determination of design aircraft.

4.4.11.1. The forecast of annual departures by aircraft type will result in a list of a number of different aircraft. e design aircraft should be selected on the basis of the one requiring the greatest pavement thickness. Each aircraft type in the forecast should be checked to determine the pavement thickness required by using the appropriate design curve with the forecast number of annual departures for that aircraft. The aircraft type which produces the greatest pavement thickness is the design aircraft. The design aircraft is not necessarily the heaviest aircraft in the forecast.

4.4.12. Determination of equivalent annual departures by the design aircraft

4.4.12.1. Since the traffic forecast is a mixture of a variety of aircraft having different landing gear types and different weights, the effects of all traffic must be accounted for in terms of the design aircraft. First, all aircraft must be converted to the same landing gear type as the design aircraft. The following conversion factors should be used to convert from one landing gear type to another:

To convert from	To	Multiply departures by
single wheel	dual wheel	0.8
single wheel	dual tandem	0.5
dual wheel	dual tandem	0.6
double dual tandem	dual tandem	1.0
dual tandem	single wheel	2.0
dual tandem	dual wheel	1.7
dual wheel	single wheel	1.3
double dual tandem	dual wheel	1.7

Secondly, after the aircraft have been grouped into the same landing gear configuration, the conversion to equivalent annual departures of the design aircraft should be determined by the following formula:

$$\log R_1 = \log R_2 \times \left(\frac{W_2}{W_1}\right)^{\frac{1}{2}}$$

Where R_1 = equivalent annual departures by the design aircraft

R_2 = annual departures expressed in design aircraft landing gear

W_1 = wheel load of the design aircraft

W_2 = wheel load of the aircraft in question

For this computation 95 per cent of the gross weight of the aircraft is assumed to be carried by the main landing gears. Wide body aircraft require special attention in this calculation. The procedure discussed above is a relative rating which compares different aircraft to a common design aircraft. Since wide body aircraft have radically different landing gear assemblies than other aircraft, special considerations are needed to maintain the relative effects. This is done by treating each wide body as a 300000 lb (136100 kg) dual tandem aircraft when computing equivalent annual departures. This should be done in every instance even when the design aircraft is a wide body. After the equivalent annual departures are determined, the design should proceed using the appropriate design curve for the design aircraft. For example, if a wide body isthe design aircraft, all equivalent departures should be calculated as described above, then the design curve for the wide body should be used with the calculated equivalent annual departures.

4.4.12.2. Example: Assume an airport pavement is to be designed for the following forecast traffic:

Aircraft	Gear type	Forecast annual departures	Maximum take-off weight (lb)	(kg)
727-100	dual	3 760	160 000	(72 600)
727-200	dual	9 080	190 500	(86 500)
707-320B	dual tandem	3 050	327 000	(148 500)
DC-9-30	dual	5 800	108 000	(49 000)
CV-880	dual tandem	400	184 500	(83 948)
737-200	dual	2 650	115 500	(52 440)
L-1011-100	dual tandem	1 710	450 000	(204 120)
747-100	double dual tandem	85	700 000	(317 800)

Solution

a) Determine design aircraft. A pavement thickness is determined for each aircraft in the forecast using the appropriate design curves. The pavement input data, CBR, k value, flexural strength, etc., should be the same for all aircraft. Aircraft weights and departure levels must correspond to the particular aircraft in the forecast. In this example the 727-200 requires the greatest pavement thickness and is thus the design aircraft.

b) Group for cast traffic into landing gear of design aircraft . In this example the design aircraft is equipped with a dual wheel landing gear so all traffic must be grouped into the dual wheel configuration.

c) Convert aircraft to equivalent annual departures of the design aircraft. After theaircraft mixture has been grouped into a common landing gear configuration, the equivalent annual departures of thedesign aircraft can be calculated.

Aircraft	Dual gear departures	Wheel load (lb)	(kg)	Wheel load of design aircraft (lb)	(kg)	Equivalent annual departures design aircraft
727-100	3 760	38 000	(17 240)	45 240	(20 520)	1 891
727-200	9 080	45 240	(20 520)	45 240	(20 520)	9 080
707-320B	5 185	38 830	(17 610)	45 240	(20 520)	2 764
DC-9-30	5 800	25 650	(11 630)	45 240	(20 520)	682
CV-880	680	21 910	(9 940)	45 240	(20 520)	94
737-200	2 650	27 430	(12 440)	45 240	(20 520)	463
747-100	145	35 625*	(16 160)	45 240	(20 520)	83
L-1011-100	2 907	35 625*	(16 160)	45 240	(20 520)	1 184
					Total	16 241

d) For this example the pavement would be designed for 16 000annual departures of a dual wheel aircraft weighing 190500 lb (86500 kg). The design should, however, provide for theheaviest aircraft in the traffic mixture when considering depth of compaction, thickness of asphalt surface, drainage structures, etc.

4.4.13. Designing the flexible pavement

4.4.13.1. Flexible pavements consist of a bituminous wearing surface placed on a base course and, when required by subgrade conditions, a sub-base. T he entire flexible pavement structure is ultimately supported by the subgrade. Definitions of the function of the various components are given in the following paragraphs. For some aircraft the base and sub-base have to be constructed of stabilized materials. The requirements for stabilized base and sub-base are also discussed in 4 .4.15.

4.4.13.2. Use of the design curves for flexible pavements requires a CBR value for the subgrade material, a GER value for the sub-base material, the gross weight of the design aircraft, and the number of annual departures of the design aircraft. The design curves presented in Figures 4-36 to 4-44 indicate the total pavement thickness required and the thickness of bituminous surfacing. Figure 4-45 indicates the minimum thickness of base course for given total pavement thicknesses and CBR values. or annual departures in excess of 25000 the total pavement thickness should be increased in accordance with 4.4.24 and the bituminous surfacing increased by 1 in (3 cm).

* Wheel lo.ads for wide body aircraft will be taken as the wheel load for a 300000 lb (136100 kg) aircraft for equivalent annual departure calculations.

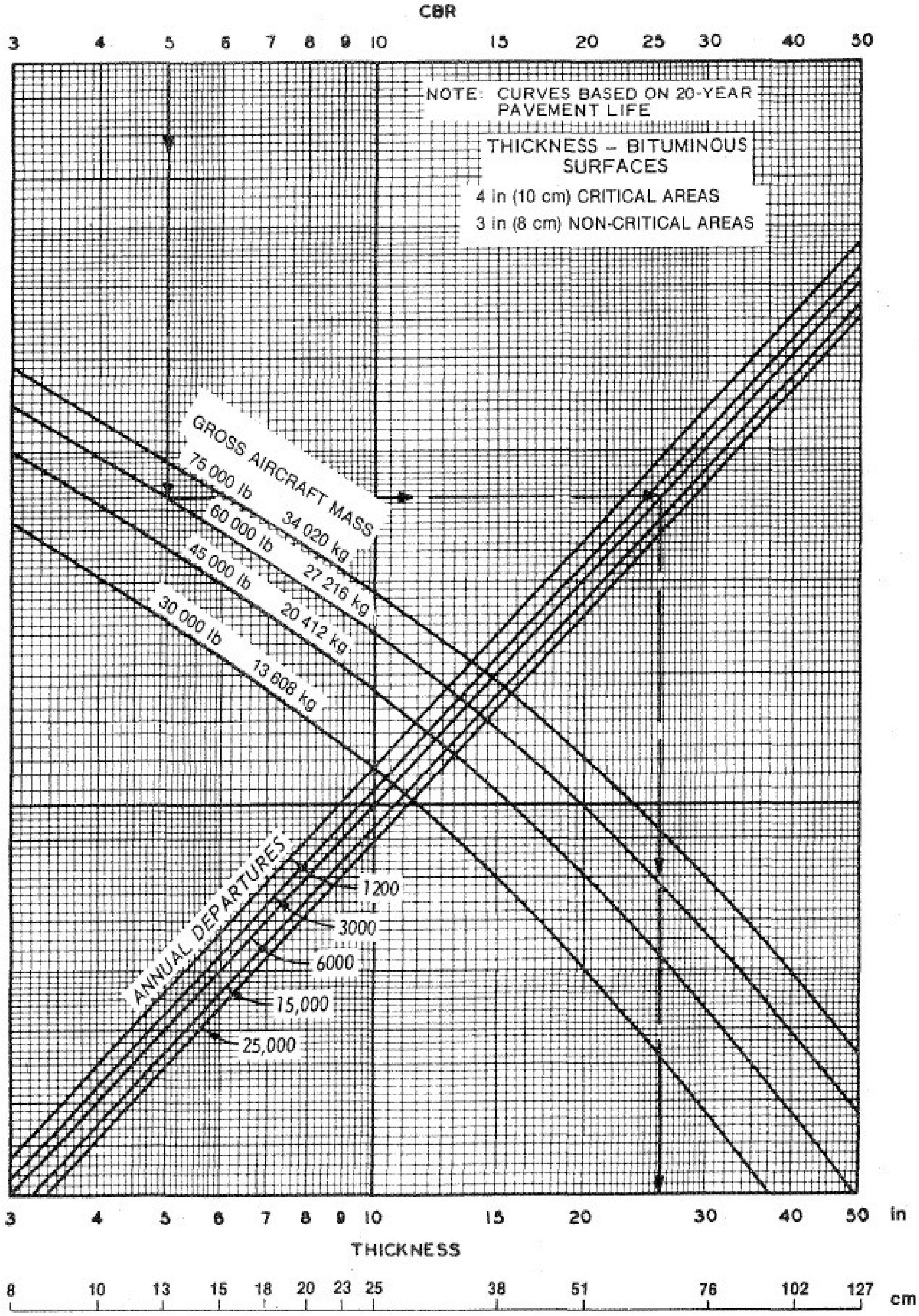


Figure 4-36. Flexible pavement design curves for critical areas, single wheel gear

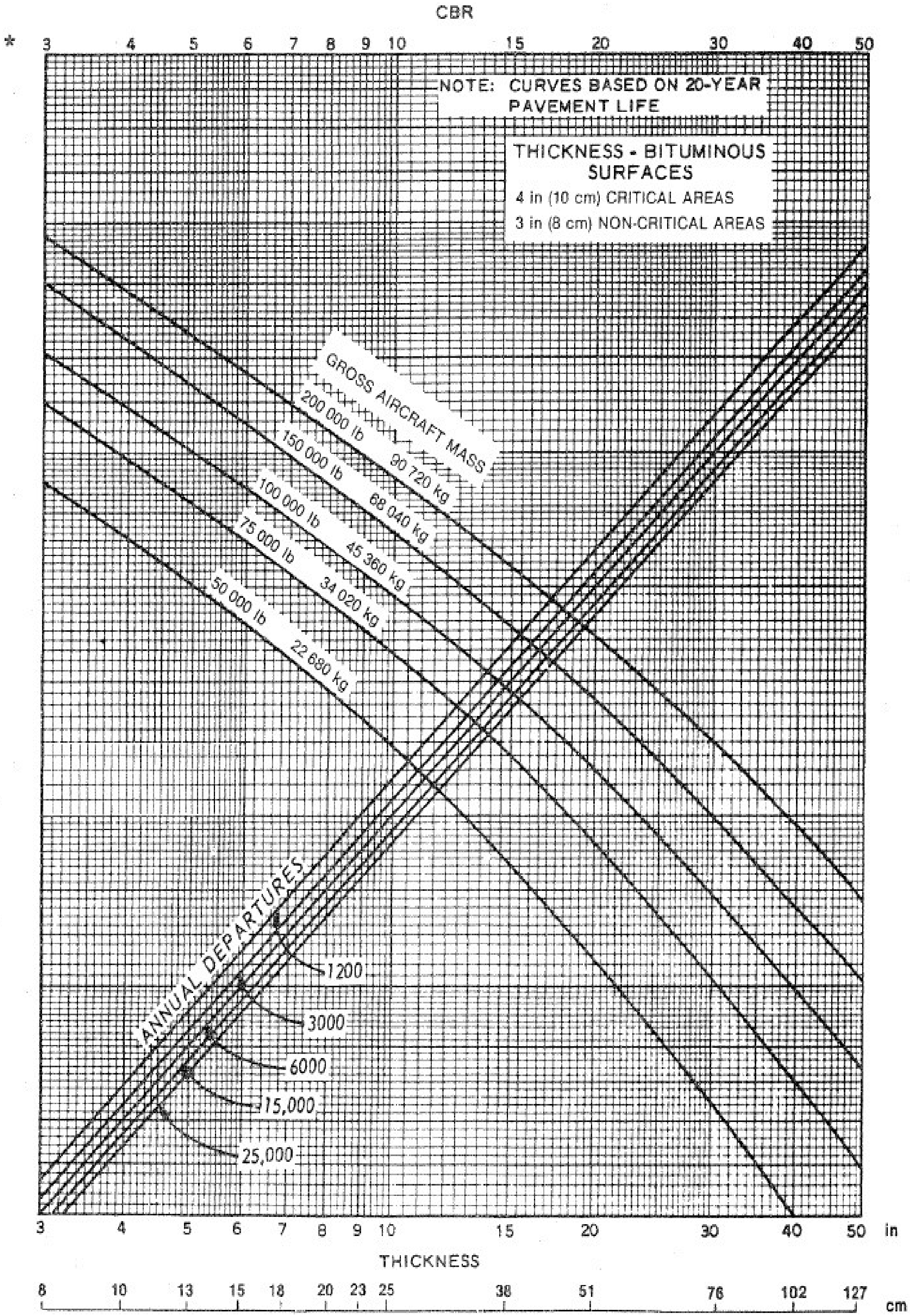

CBR
*
3 4 5 6 7 8 9 10 15 20 30 40 50
NOTE: CURVES BASED ON 20-YEAR PAVEMENT LIFE
THICKNESS - BITUMINOUS SURFACES
4 in (10 cm) CRITICAL AREAS
3 in (8 cm) NON-CRITICAL AREAS
GROSS AIRCRAFT MASS
200 000 lb 90 720 kg
150 000 lb 68 040 kg
100 000 lb 45 360 kg
75 000 lb 34 020 kg
50 000 lb 22 680 kg
ANNUAL DEPARTURES
1200
3000
6000
15,000
25,000
3 4 5 6 7 8 9 10 15 20 30 40 50 in
THICKNESS
8 10 13 15 18 20 23 25 38 51 76 102 127 cm

Figure 4-37. Flexible pavement design curves for critical areas, dual wheel gear

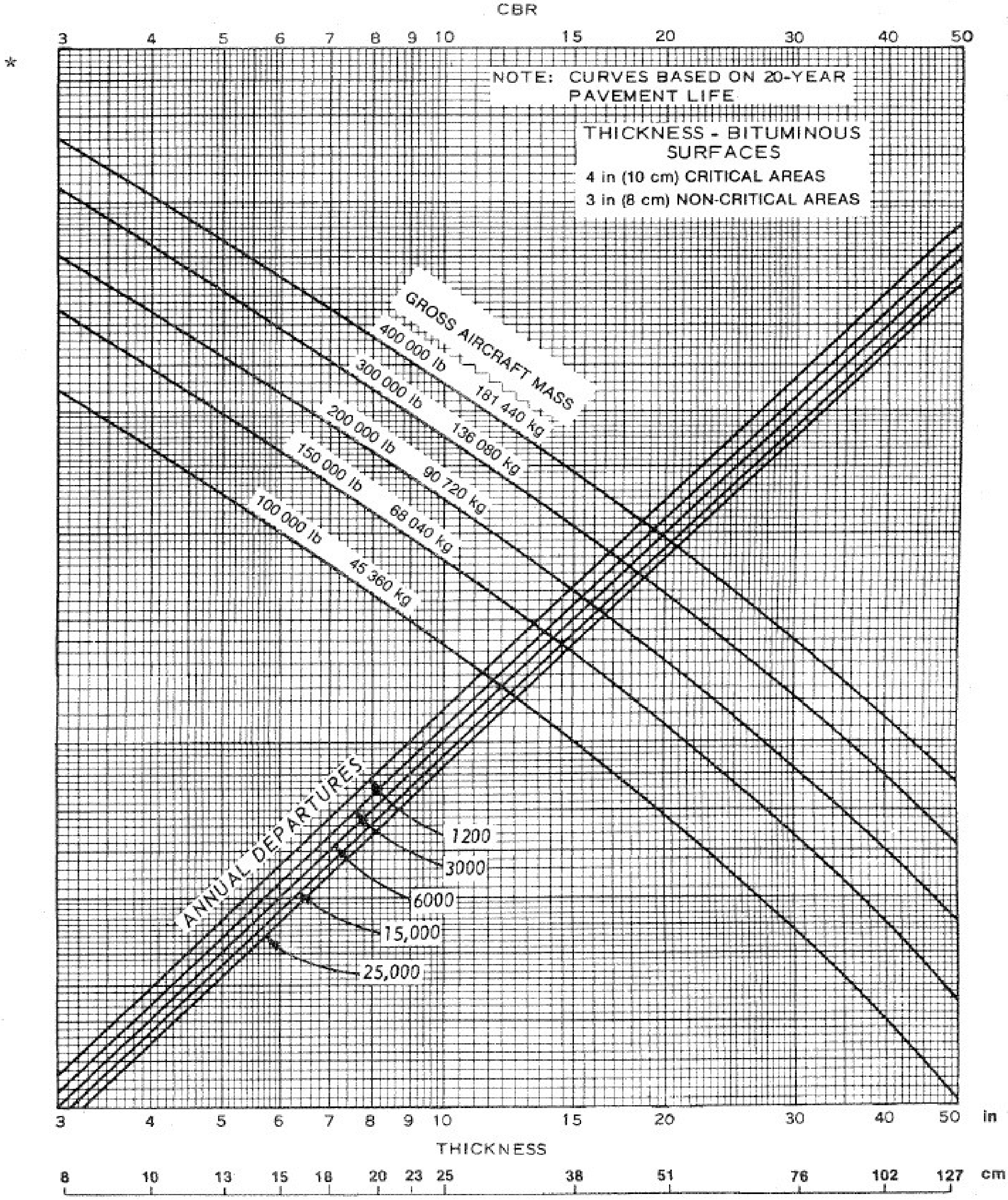


Figure 4-38. Flexible pavement design curves for critical areas, dual tandem gear

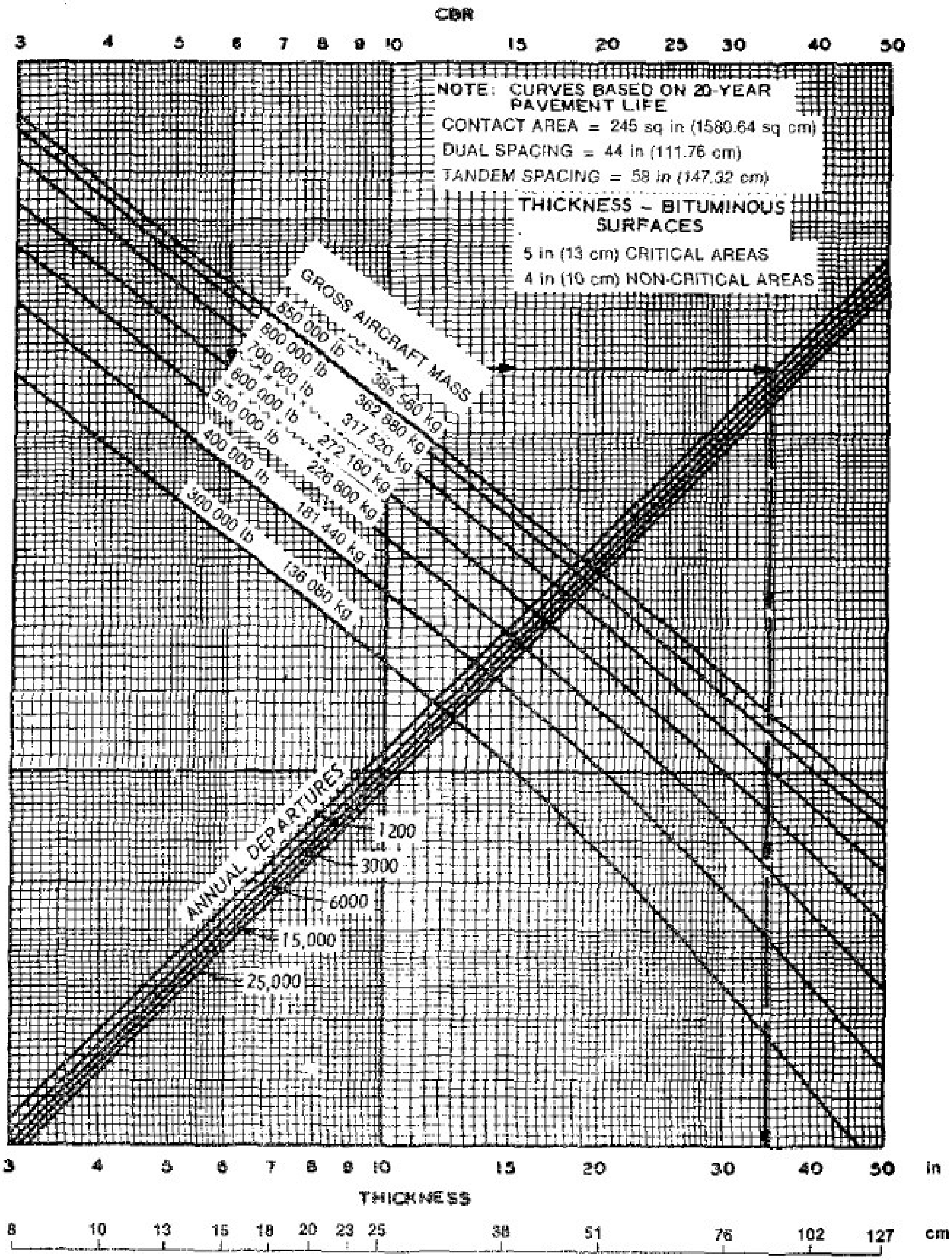


Figure 4-39. Flexible pavement design curves for critical areas, B747-100, SR, 200 B, C, F

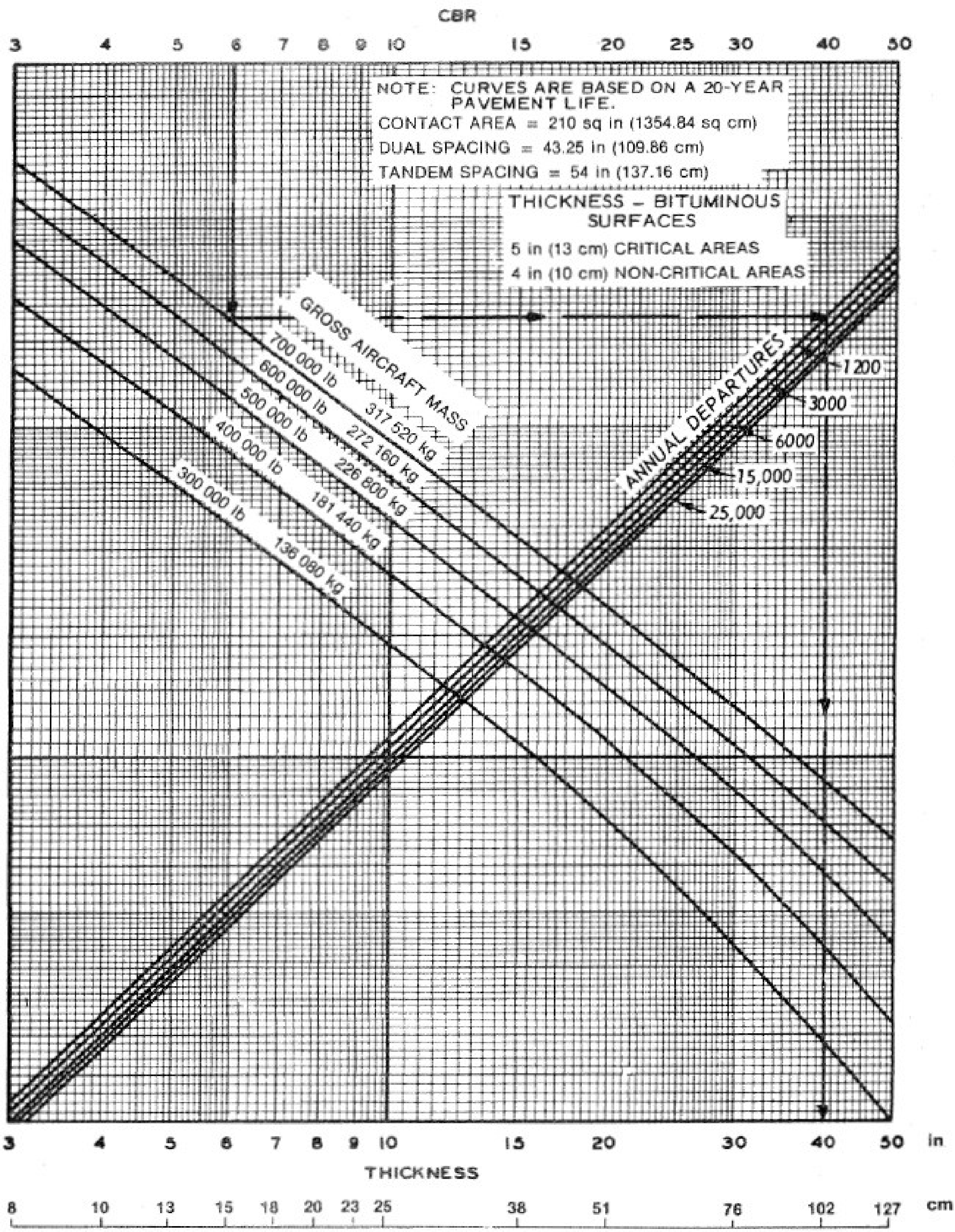


Figure 4-40. Flexible pavement design curves for critical areas, B747-SP

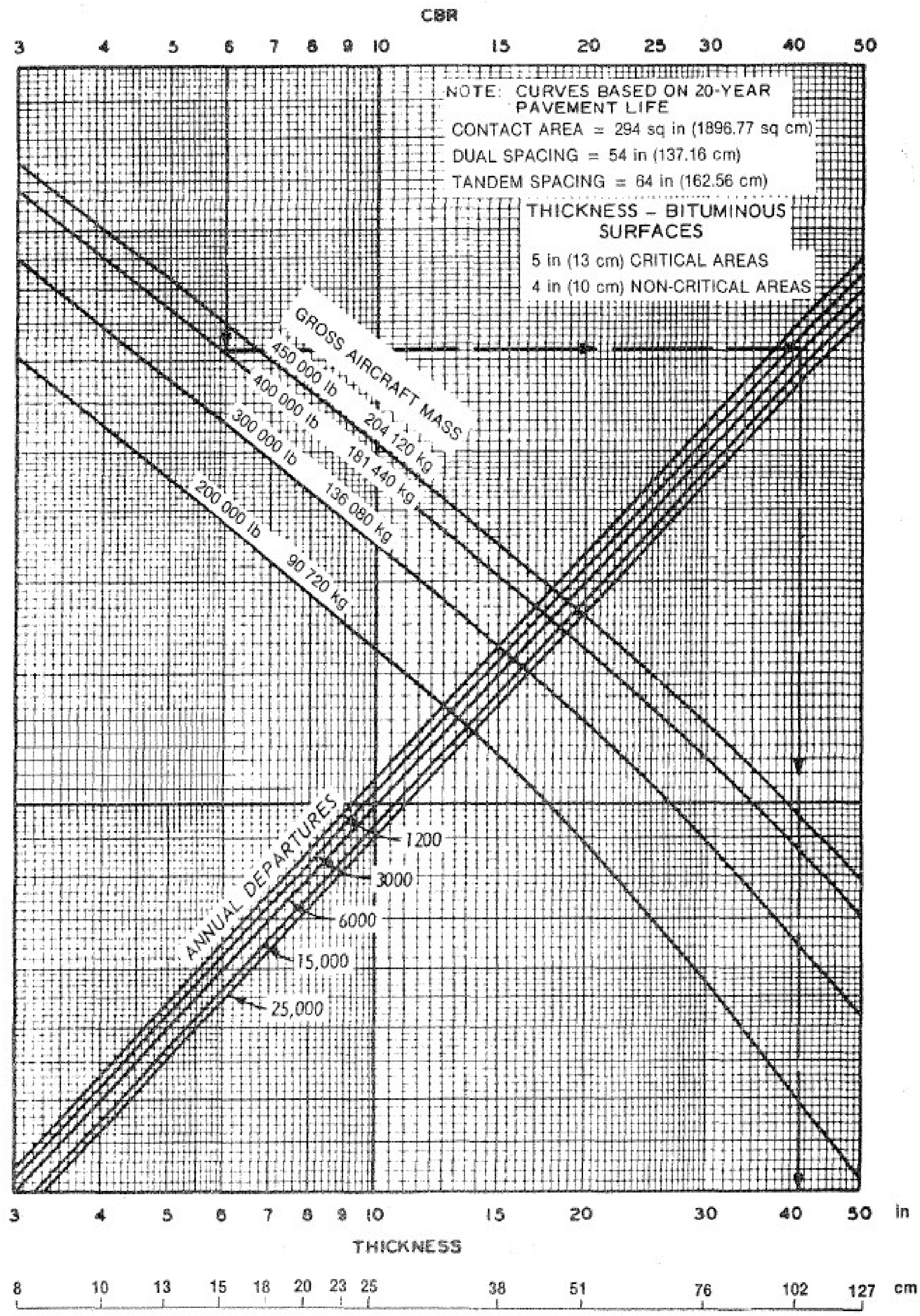


Figure 4-41. Flexible pavement design curves for critical areas, DC10-10, 10CF

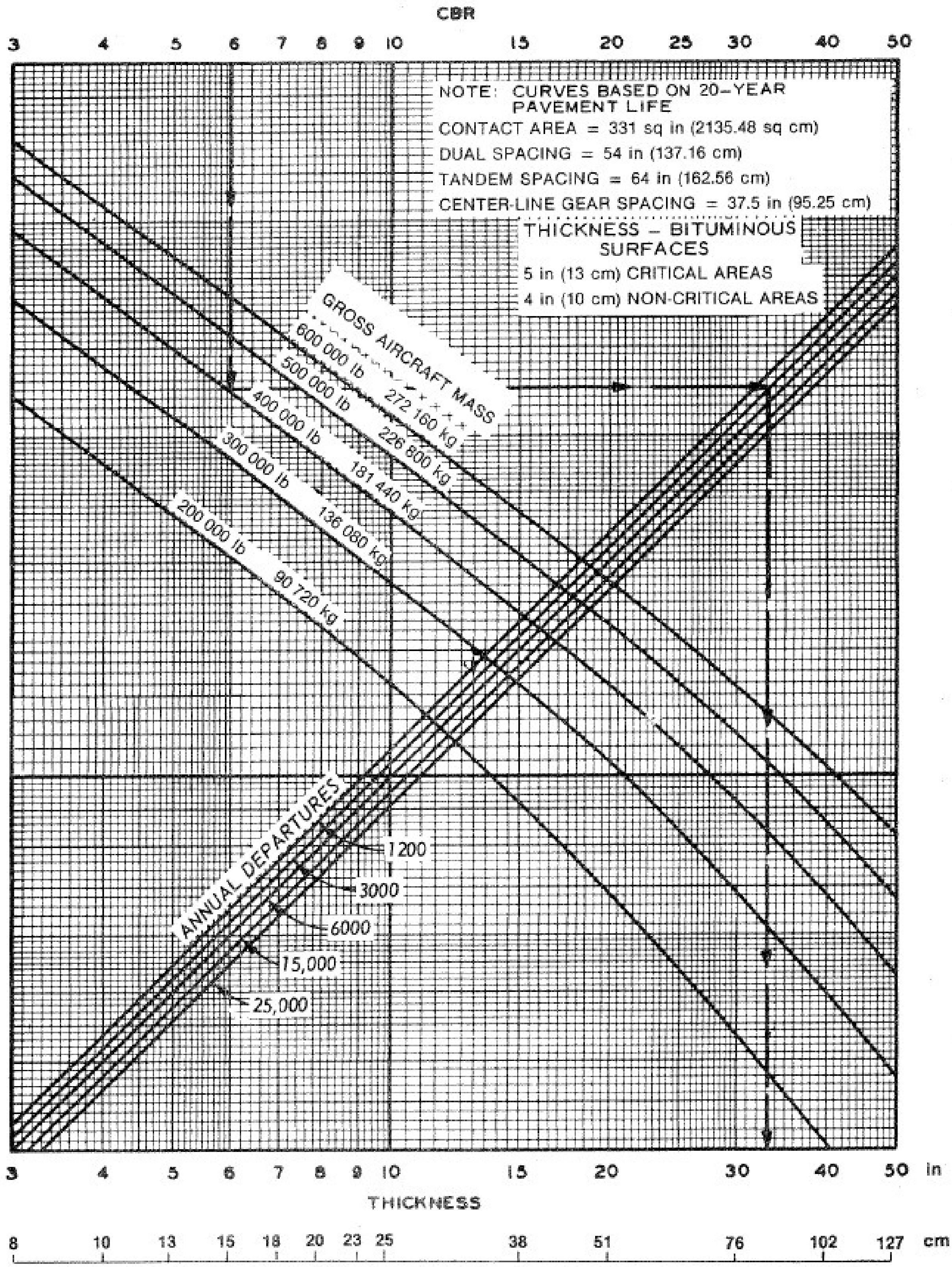


Figure 4-42. Flexible pavement design curves for critical areas, DC10-30, 30CF, 40, 40CF

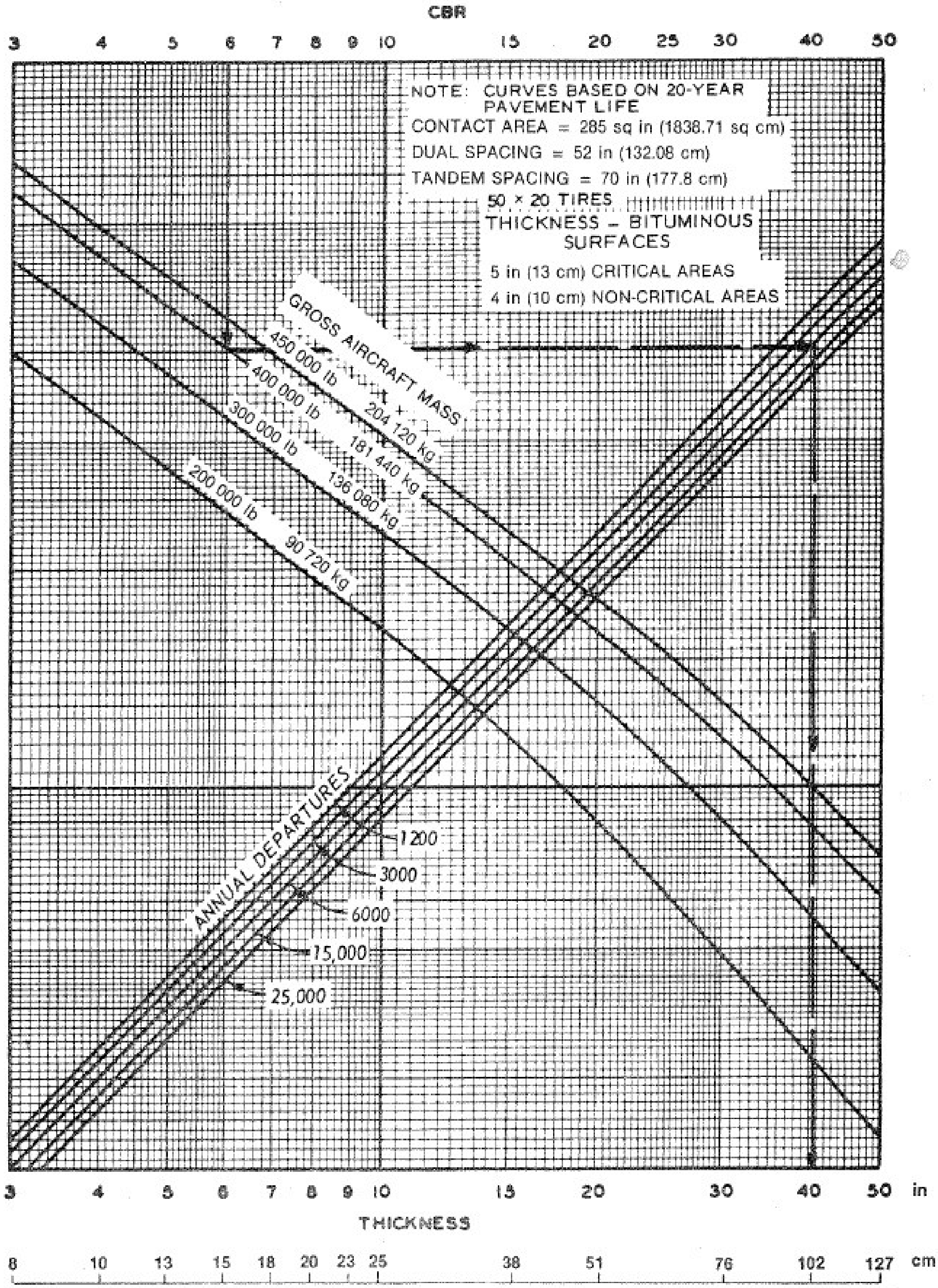


Figure 4-43. Flexible pavement design curve for critical areas, L-1011, 100

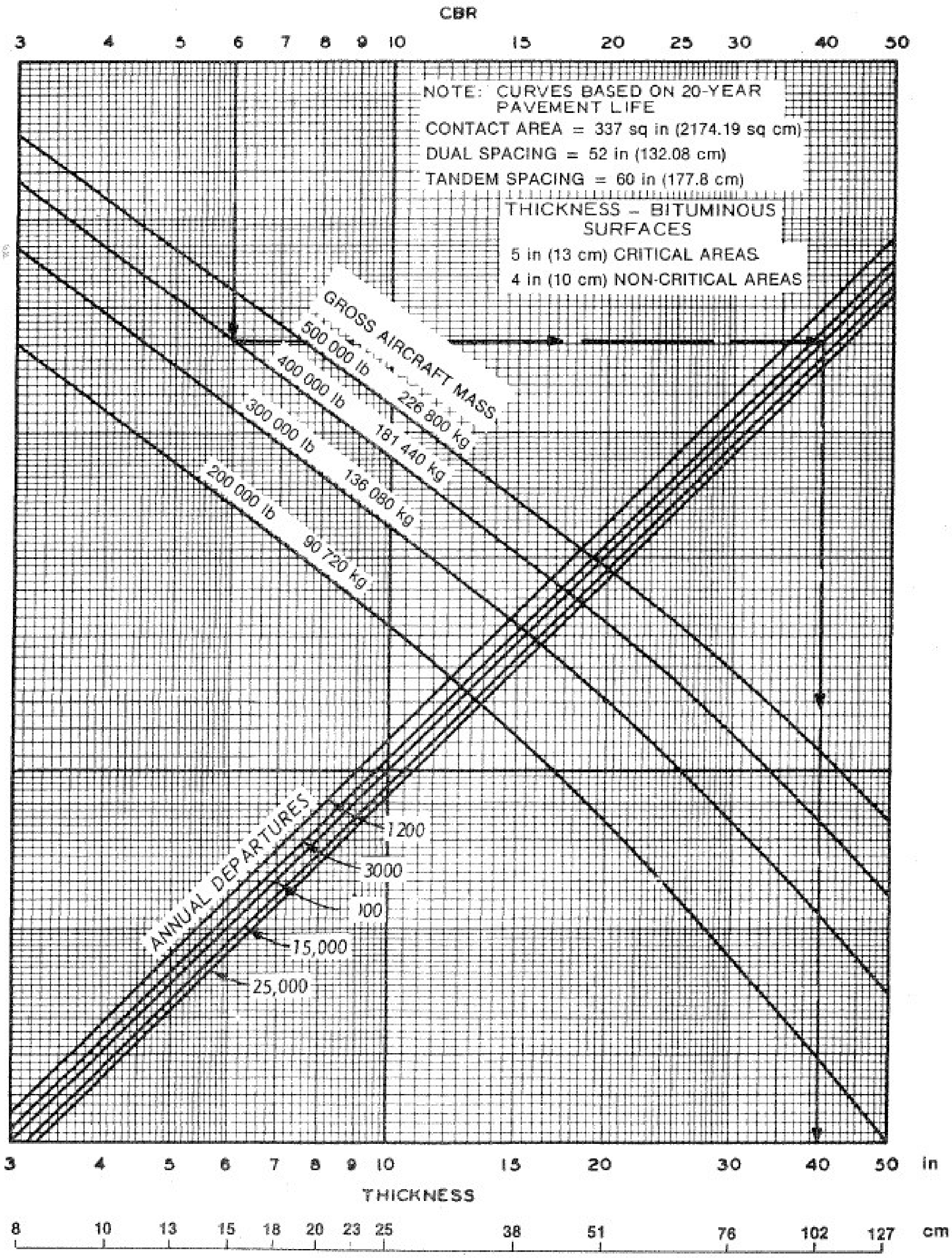


Figure 4-44. Flexible pavement design curves for critical areas, L-1011-100, 200

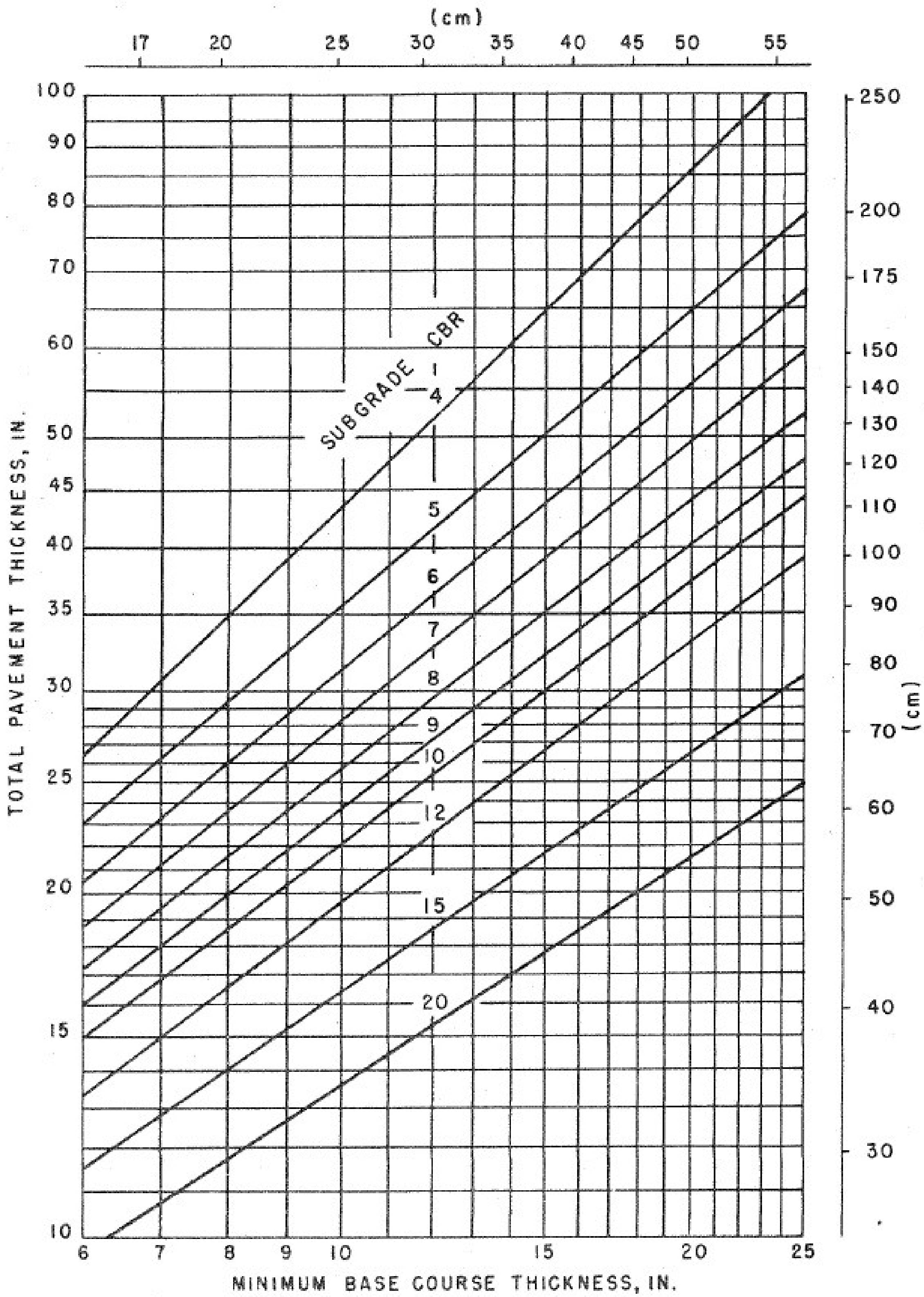


Figure 4-45. Minimum base course thickness requirement

4.4.14. Critical and non-critical areas_

4.4.14.1. The design curves, Figures 4-36 to 4-44, are used to determine the total critical pavement thickness, T and the surface course thickness requirements. The O.9T factor for the non-critical pavement applies to the base and sub-base courses; the surface course thickness is as noted on the design curves. For the variable section of the transition section and thinned edge, the reduction applies only to the base course. The 0.7T thickness for base shall be minimum permitted, and the sub-base thickness shall be increased or varied to provide positive surface drainage from the entire subgrade surface. For fractions of an inch of 0.5 or more, use the next higher whole number; for less than 0.5, use the next lower number.

4.4.15. Stabilized base and sub-base

4.4.15.1. Stabilized base and sub-base courses are necessary for 'new pavements designed to accommodate jet aircraft weighing 100000 lb (45350 kg) or more. These stabilized courses may be substituted for granular courses using the equivalency factors discussed in 4.4.16. A range of equivalency factors is given because the factor is sensitive to a number of variables such as layer thickness, stabilizing agent type and quantity, location of stabilized layer in the pavement structure, etc.

4.4.15.2. Exceptions to the policy requiring stabilized base and sub-base should be based on proven performance of a granular material. Proven performance in this instance means a history of satisfactory airport pavements using the materials. This history of satisfactory performance should be under aircraft loadings and climatic conditions comparable to those anticipated.

4.4.15.3. Other exceptions may be made on the basis of superior materials being available, such as '100 per cent crushed, hard, closely graded stone. These materials should exhibit a remoulded soaked CBR minimum of 100 for base and 35 for sub-base. In areas subject to frost penetration the materials should meet permeability and non-frost susceptibility tests in addition to the CBR requirements.

4.4.15.4. The minimum total pavement thickness should not be less than the total pavement thickness required by a 20 CBR subgrade on the appropriate design curve. Reflection cracking is sometimes encountered when cement treated base is used, The thickness of the bituminous surfacing course should be at least 4 in (10 cm) to minimize the chances of reflection cracking when cement treated base is used.

4.4.16. Stabilized sub-base and base equivalency factors

4.4.16.1. Stabilized sub-base courses off r some structural benefits to a flexible Pavement. The benefits can be expressed in the form of equivalency factors which indicate the substitution thickness ratios applicable to various stabilized layers. The thickness of stabilized material can be computed by dividing the granular sub-base thickness requirement by the equivalency factor. The equivalency factor ranges are presented in Table 4 -9 below.

Table 4-9. Recommended equivalency factor range stabilized sub-base

Material	Equivalency factor range
Bituminous surface course	1.7 -2.3

Bituminous base course	1.7-2.3
Cold laid bituminous base course	1.5- 1.7
Mixed in-place base course	1.5-1.7
Cement treated base course	1.6-2.3
Soil cement base course	1.5-2.0
Crushed aggregate base course	1.4-2.0
Gravel sub-base course	1.0

In establishing the equivalency factor shown above, the CBR of the gravel sub-base course was assumed to be 20.

4.4.16.2. Stabilized base course offer structural benefits to a flexible pavement in much the same manner as stabilized sub-base. The benefits are expressed as equivalency factors similar to those shown for stabilized sub-base. These ratios are used to compute the thickness of stabilized base by dividing the garanular base requirement by the equivalency factor. The equivalency factor ranges are presented in Table 4-10 below.

Table 4-10. Recommended equivalency factor range stabilized base

Material	Equivalency factor range
Bituminous surface course	1.2 -1.6
Bituminous base course	1.2-1.6
Cold laid bituminous base course	1.0- 1.2
Mixed in-place base course	1.0-1.2
Cement treated base course	1.2- 1.6
Soil cement base course	N/A
Crushed aggregate base course	1.0
sub-base course	N/A

The equivalency factors shown above assume a CBR value of 80 for crushed aggregated base course.

4.4.17. Design example

4.4.17.1. As an example of the use of the design curves, assume a flexible pavement is to be designed for a dual gear aircraft having a gross mass of 75000lb (34000kg) and 6000 annual equivalent departures of the design aircraft. Design CBR values for the sub-base and subgrade are 20 and 6, respectively.

4.4.17.2. Total pavement thickness. The total pavement thickness required is determined from Figure 4-37. Enter the upper abscissa with the subgrade CBR value, 6, Project vertically downward to the gross mass of the design aircraft, 75000 lb (34000 kg). At the point of intersection of the vertical projection and the aircraft gross weight, make a horizontal projection to the equivalent annual departures, 6 000, From the point of intersection of the horizontal projection and the annual departure level, make a vertical projection down to the lower abscissa and read the total pavement thickness; in this example - 21.3 in (51.2 cm).

4.4.17.3. Thickness of sub-base course. The thickness of the sub- se course is determined in a manner similar to the total pavement thickness Using Figure 4G37 enter the upper abscissa with the design CBR value for the sub-base, 20. The chart is used in the same manner as described in 4 4.17.2 above, i.e., vertical projection to aircraft gross weight, horizontal projection to annual departures, and vertical projection to lower abscissa. In this example the thickness obtained is 8.6 in (21.8 cm) This means that the combined thickness of bituminous surface and base course needed over a 20 CBR sub-base is 8.6 in (21.8 cm), thus having a sub base thickness of 21,3 - 8.6 = 12.7 in (32.2 cm).

4.4.17.4. Thickness of bituminous surface. As indicated by the Note in Figure 4- 37, the thickness of bituminous surface for critical areas is 4 in (10 cm) and for non-critical 3 in (8 cm).

4.4.17.5. Thickness of base course. The thickness of base course can be computed by subtracting the thickness of bituminous surface from the combined thickness of surface and base determined in 4.4.17.3 above; in this example 8.6 - 4.0 = 4.6 in (11.7 cm) of base course. The thickness of base course thus calculated should be compared with the minimum base course thickness required as shown in Figure 4- 45. Note that the minimum base course thickness is 6 in (15cm) for critical areas. Enter the left ordinate of Figure 4-45 with the total pavement thickness as determined in 4.4.17.2 above, in this example - 21.3 in (51.2 cm). Make a horizontal projection to the subgrade CBR line; in this example, 6. From the Intersection of the horizontal projection and the subgrade CBR line, make a vertical projection down to the lower abscissa and read the minimum base course thickness, in this example t minimum thickness of 6 in (15 cm) would be required The extra thickness of base required by Figure 4-45as opposed to the earlier calculation is taken out of the sub-base thickness not added to the total pavement thickness; in this example 12.7 - 1.4 - 11.3 in (28.7 cm).

4.4.17.6. Thickness of non-critical areas. The total pavement thickness for non- critical areas is obtained by taking 0.9 of the critical pavement base and sub-base thickness plus the required bituminous surface thickness given on the design charts. For the thinned edge portion of the critical and non-critical pavements, the 0.7T factor applies only to the base course because the sub-base should allow for transverse drainage.

4.4.17.7. Summary. The thickness calculated in the above paragraphs should be rounded off to even increments, If conditions for detrimental frost action exist, another analysis is required. The final design thicknesses for this example would be as follows:

Thickness Requirements

	Critical		Non-critical	
	in	(cm)	in	(cm)
Bituminous surface	4	(10)	3	(8)
Base course	6	(15)	5	(13)
Sub-base course	11	(28)	10	(25)
Transverse drainage	0	(0)	3	(8)

Since the design aircraft in this example weighs less than 100000lb (45300kg), stabilized base and sub-base are not required but could be used if desired.

4.4.18. Designing the rigid pavement

4.4.18.1. Design curves have been prepared for rigid pavements similar to those for flexible pavements; i.e., separate curves for single, dual, and dual tandem landing gear assemblies and separate design curves for wide-body jet aircraft. See Figures 4-46 to 4-54. These curves are based on a jointed edge loading assumption where the load is tangent to the joint. Use of the design curves requires four design input parameters: concrete flexural strength, subgrade modulus, gross weight of the design aircraft, and annual departure of the design aircraft. The rigid pavement design curves indicate the thickness of concrete only. Thicknesses of other components of the rigid pavement structure must be determined separately.

4.4.18.2. <u>Concrete flexural strength.</u> The required thickness of concrete pavement is related to the strength of the concrete used in the pavement. Concrete strength is assessed by the flexural strength method as the primary action of a concrete pavement slab is flexure. Concrete flexural strength should be determined by ASTM C-78 test method. Normally a 90-day flexural strength is used for design. The designer can safely assume the 90-day flexural strength of concrete will be 10 per cent higher than the 28 day strength.

4.4.18.3. <u>k value.</u> The k value is, in effect, a spring constant for the material supporting the rigid pavement and is indicative of the bearing value of the supporting material.

4.4.18.4. <u>Gross weight of aircraft.</u> The gross weight of the design aircraft is shown on each design curve. The design curves are grouped in. accordance with main landing gear assembly type except for wide body aircraft which are shown on separate curves. A wide range of gross weights is shown on all curves to assist in any interpolations which may be required. In all instances, the range of gross weights shown is adequate to cover weights of existing aircraft.

4.4.18.5. <u>Annual departure of design aircraft.</u> The fourth input parameter is annual departures of the design aircraft. The departures should be computed using the procedure explained in 4.4.12.

4.4.18.6. <u>Use of design curves.</u> The rigid pavement design curves are constructed such that the design inputs are entered in the same order as they are discussed above. Concrete flexural strength is the first input. The left ordinate of the design curve is entered with concrete flexural strength. A horizontal projection is made until it intersects with the appropriate foundation modulus line. A vertical projection is made from the intersection point to the appropriate gross weight of the design aircraft. A horizontal projection is made to the right ordinate showing annual departures. The pavement thickness is read from the appropriate annual departure line. The pavement thickness shown refers to the thickness of the concrete pavement only, exclusive of the sub-base.

4.4.19. Sub-base requirements

4.4.19.1. The purpose of a sub-base under a rigid pavement is to provide uniform stable support for the pavement slabs. A minimum thickness of 4 in (10 cm) of sub-base is required under all rigid pavements, except as shown in Table 4-11 below:

Table 4-11 Conditions where no sub-base is required

Soil classification	Good drainage		Poor drainage	
	No frost	Frost	No frost	Frost
GW	X	X	X	X
GP	X	X	X	
GM	X			
GC	X			
SW	X			

4.4.19.2. Sub-base thickness in excess of 4 in (10 cm) can be used to increase the modulus of soil reaction and reduce the required thickness of concrete needed, if economical. The cost of providing the additional thickness of sub-base should be weighed against the savings in concrete thickness. The materials suitable for sub-base courses under rigid pavements are listed below:

Gravel sub-base course

Bituminous base course

Aggregate base course

Crushed aggregate base course

Soil cement base course

Cement treated base course

4.4.19.3. Determination of k value for granular sub-base. The probable increase in k value associated with various thicknesses of different sub-base materials is shown in Figure 4-35. Figure 4-35 is intended for use when the sub-base is composed of unstabilized granular materials. Values shown in Figure 4-35 are to be considered guides and can be tempered by local experience.

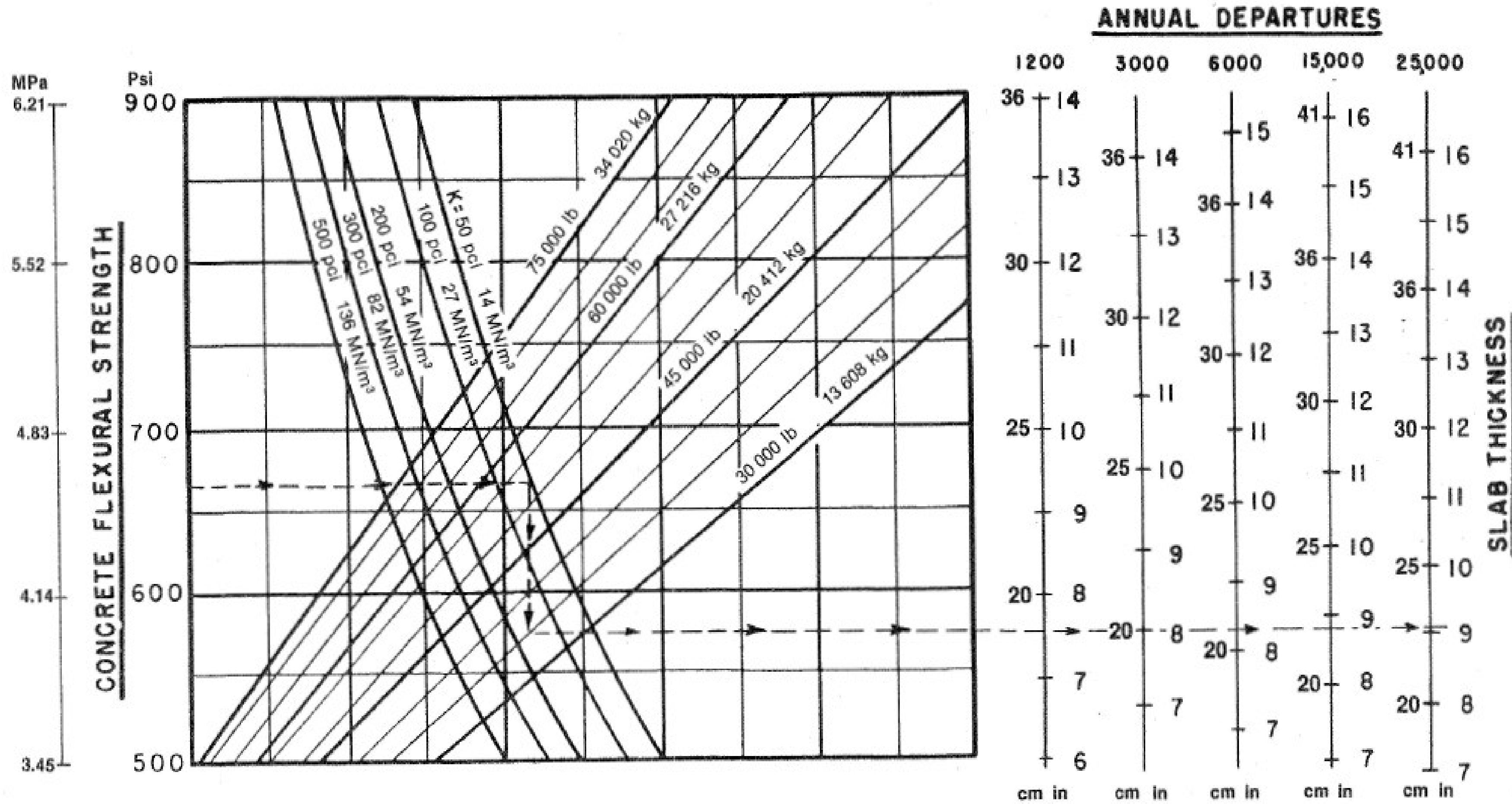


Figure 4-46. Rigid pavement design curves – single wheel gear

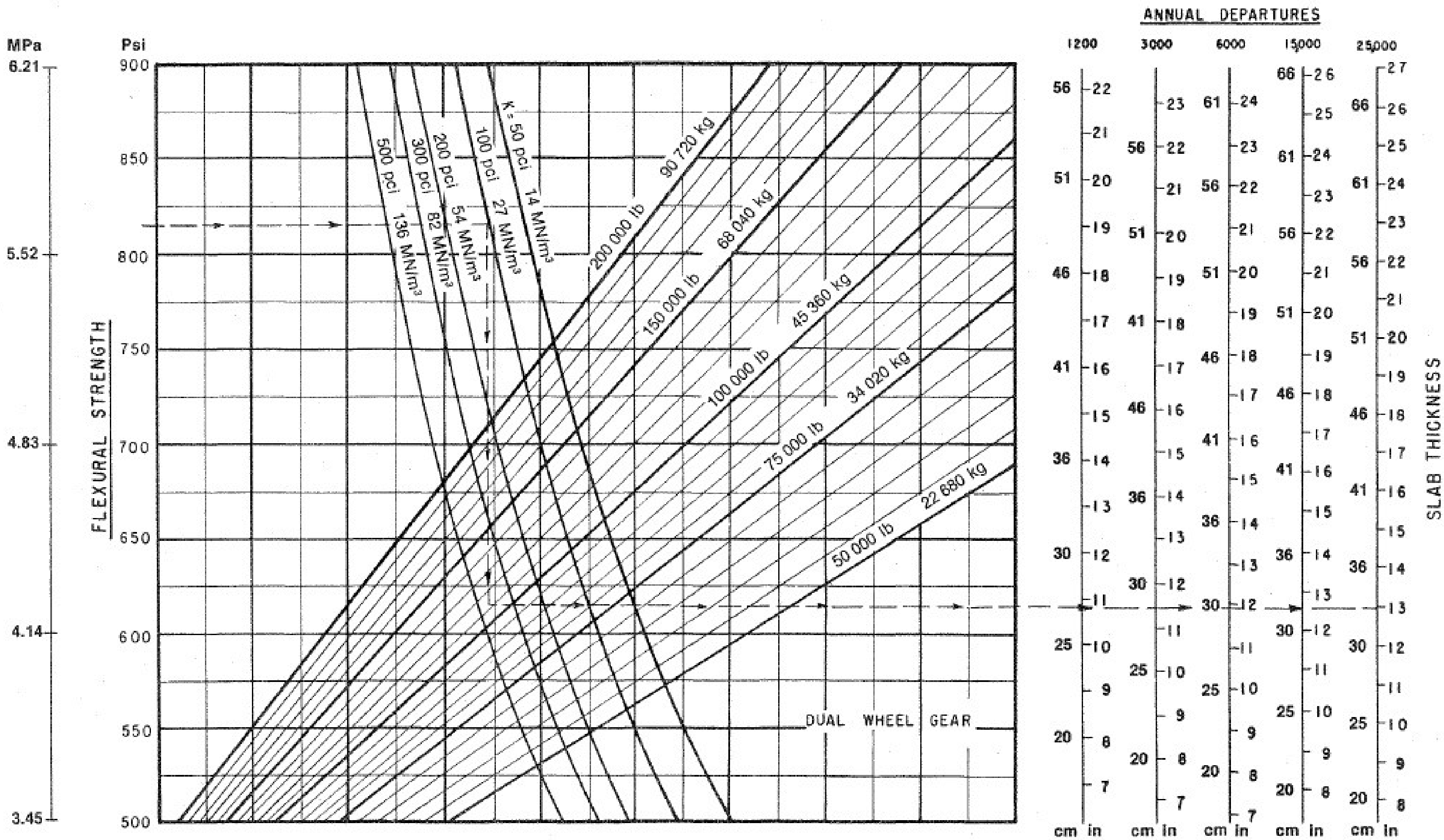


Figure 4-47. Rigid pavement design curves – dual wheel gear

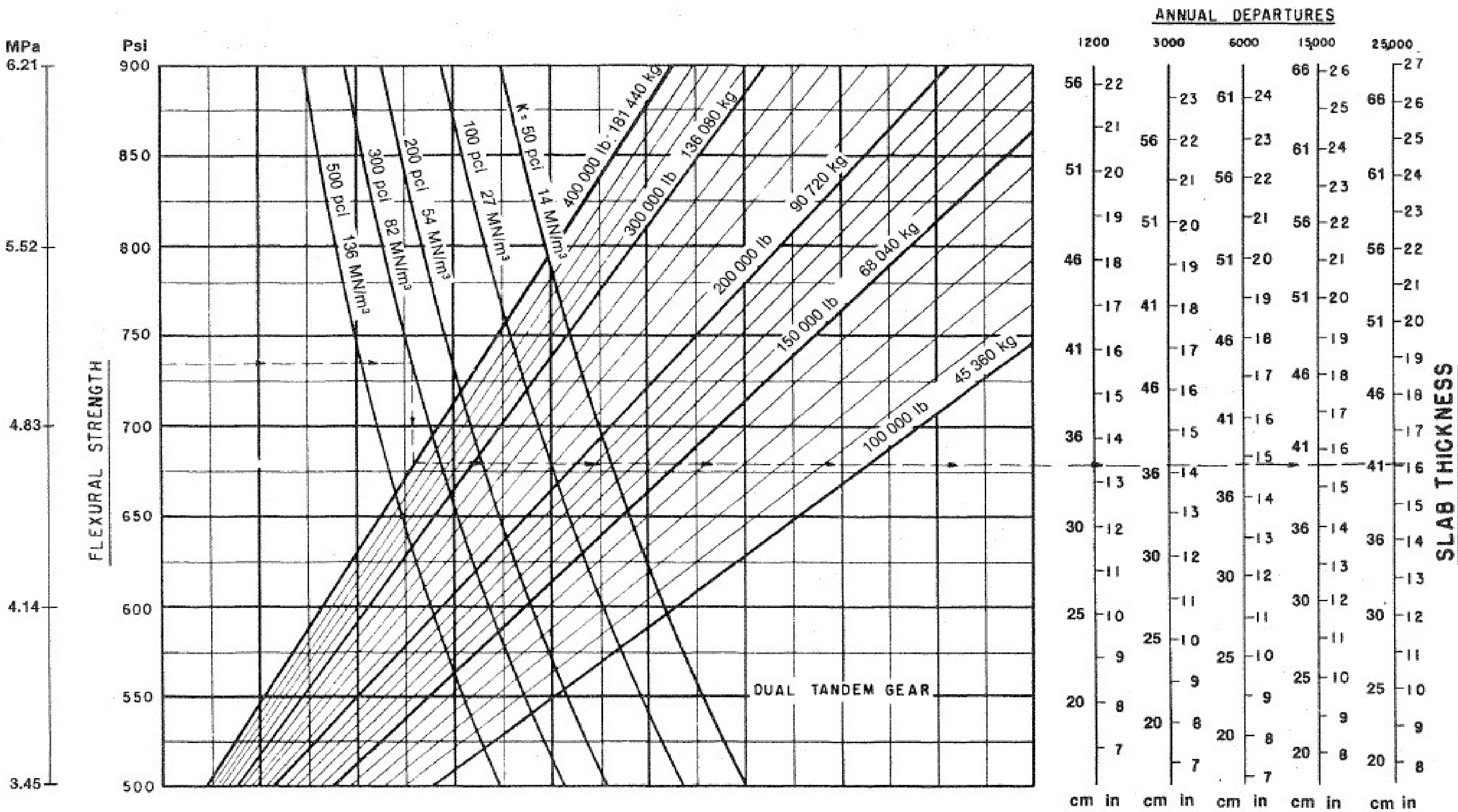


Figure 4-48. Rigid pavement design curves – dual tandem gear

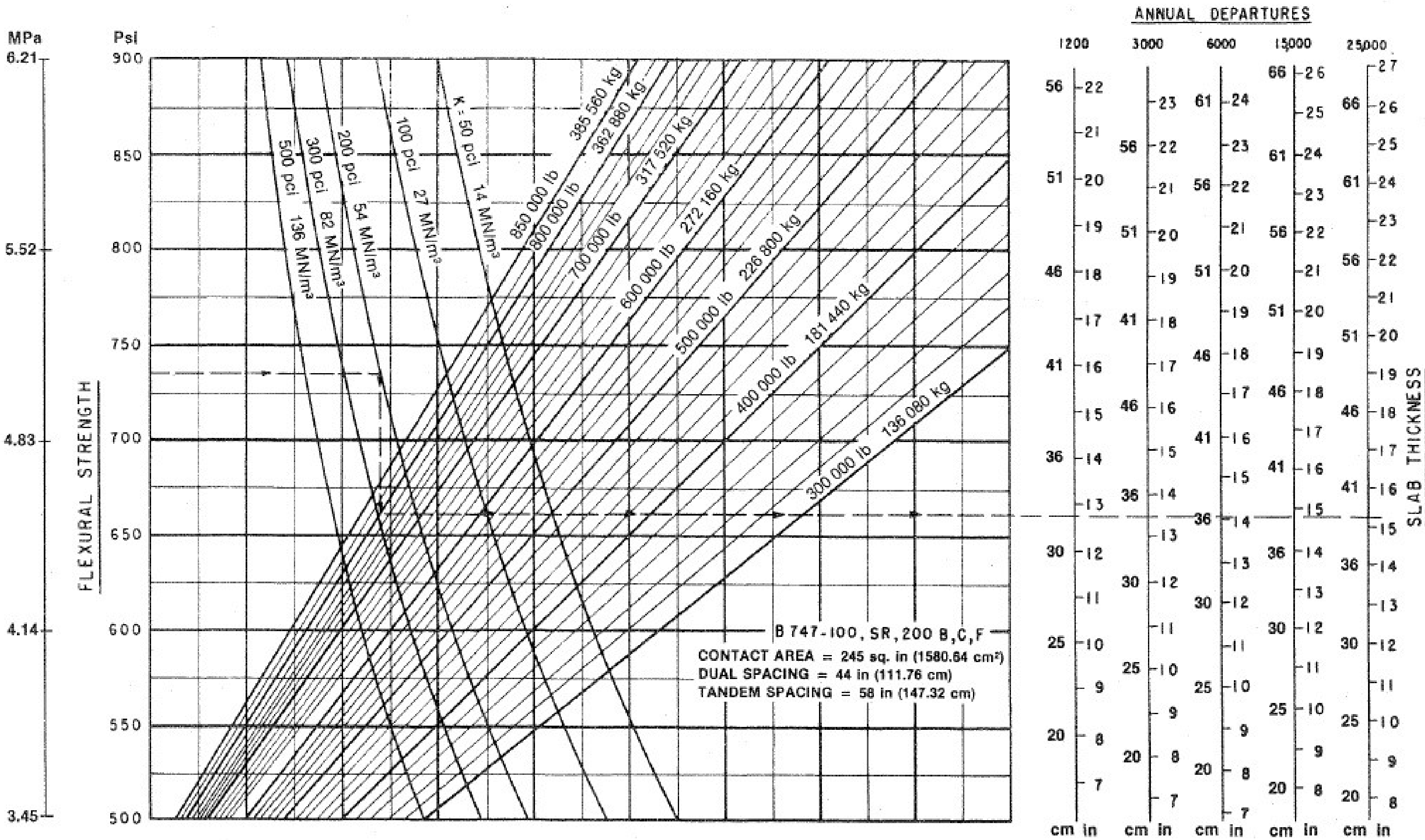


Figure 4-49. Rigid pavement design curves – B-747-100, SR, 200 B, C, F

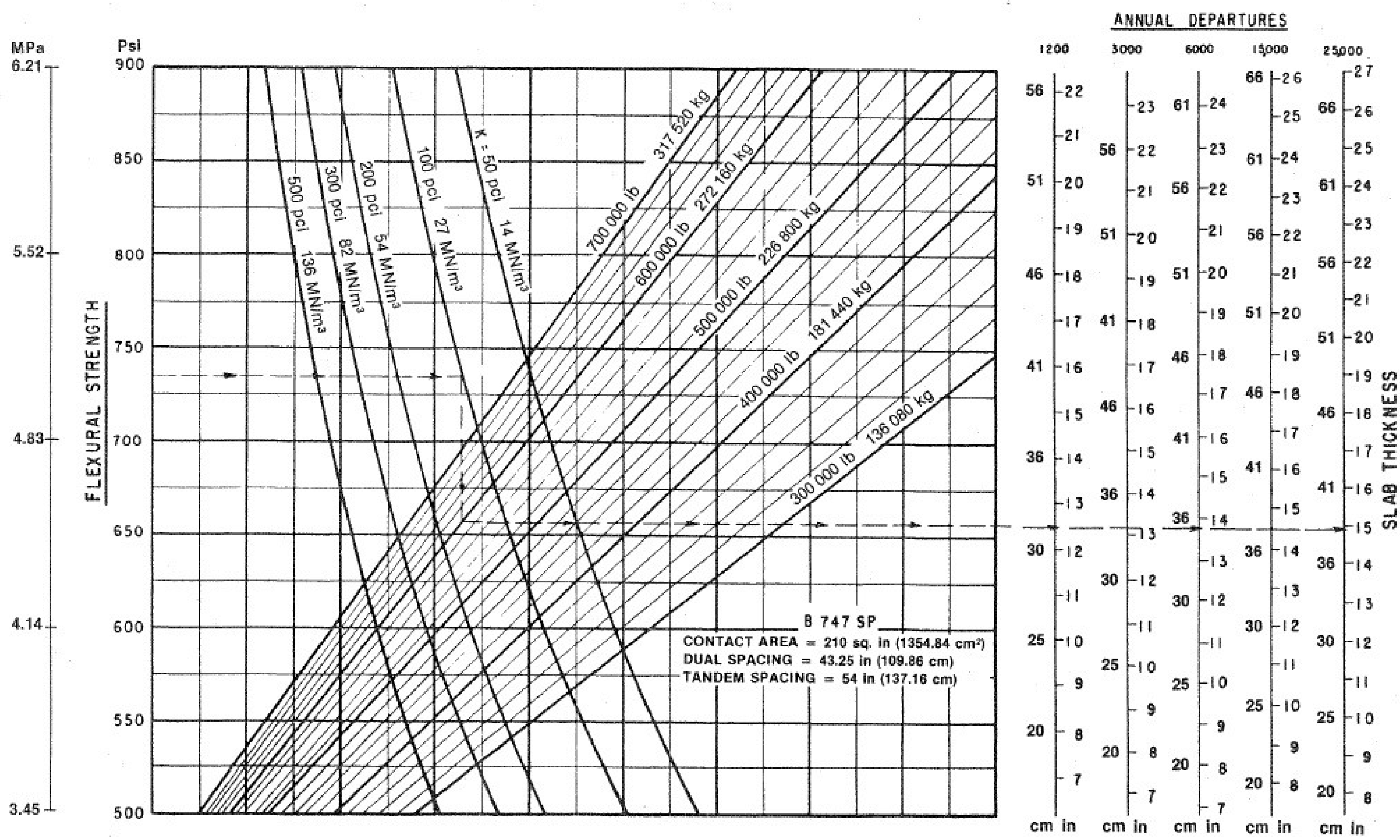


Figure 4-50. Rigid pavement design curves – B-747 – SP

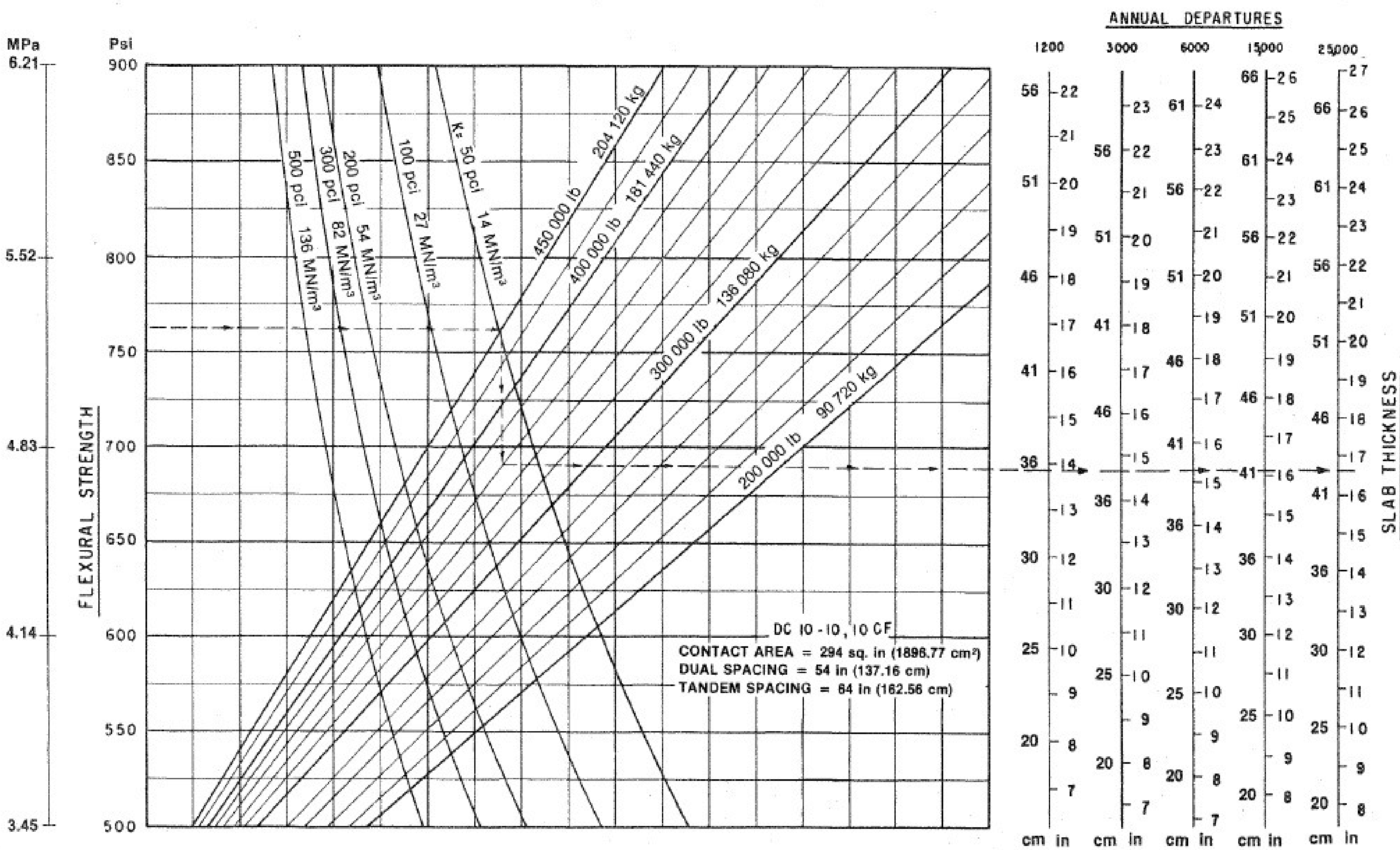


Figure 4-51. Rigid pavement design curves – DC 10-10, 10CF

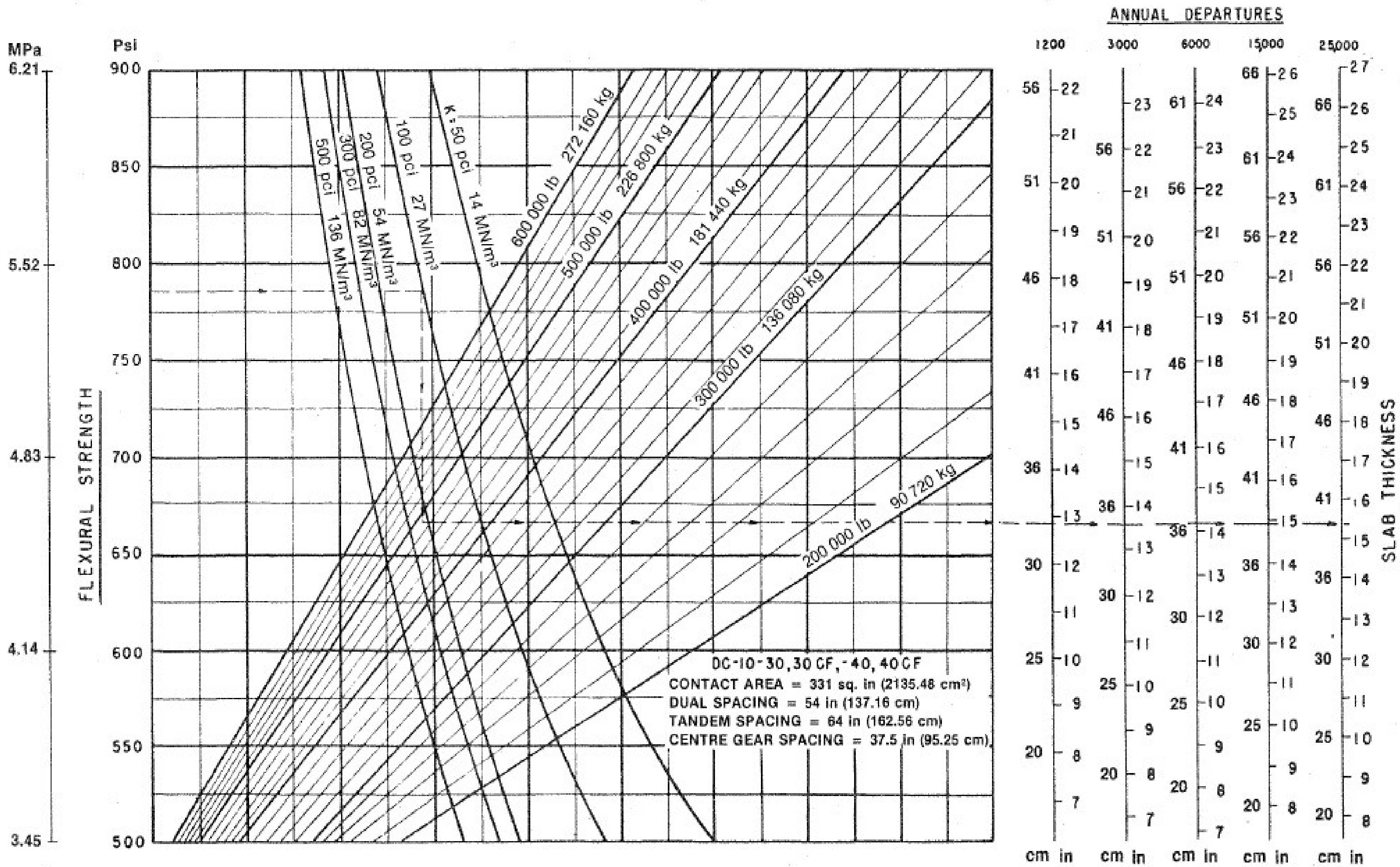


Figure 4-52. Rigid pavement design curves – DC 10-30, 30CF, 40, 40CF

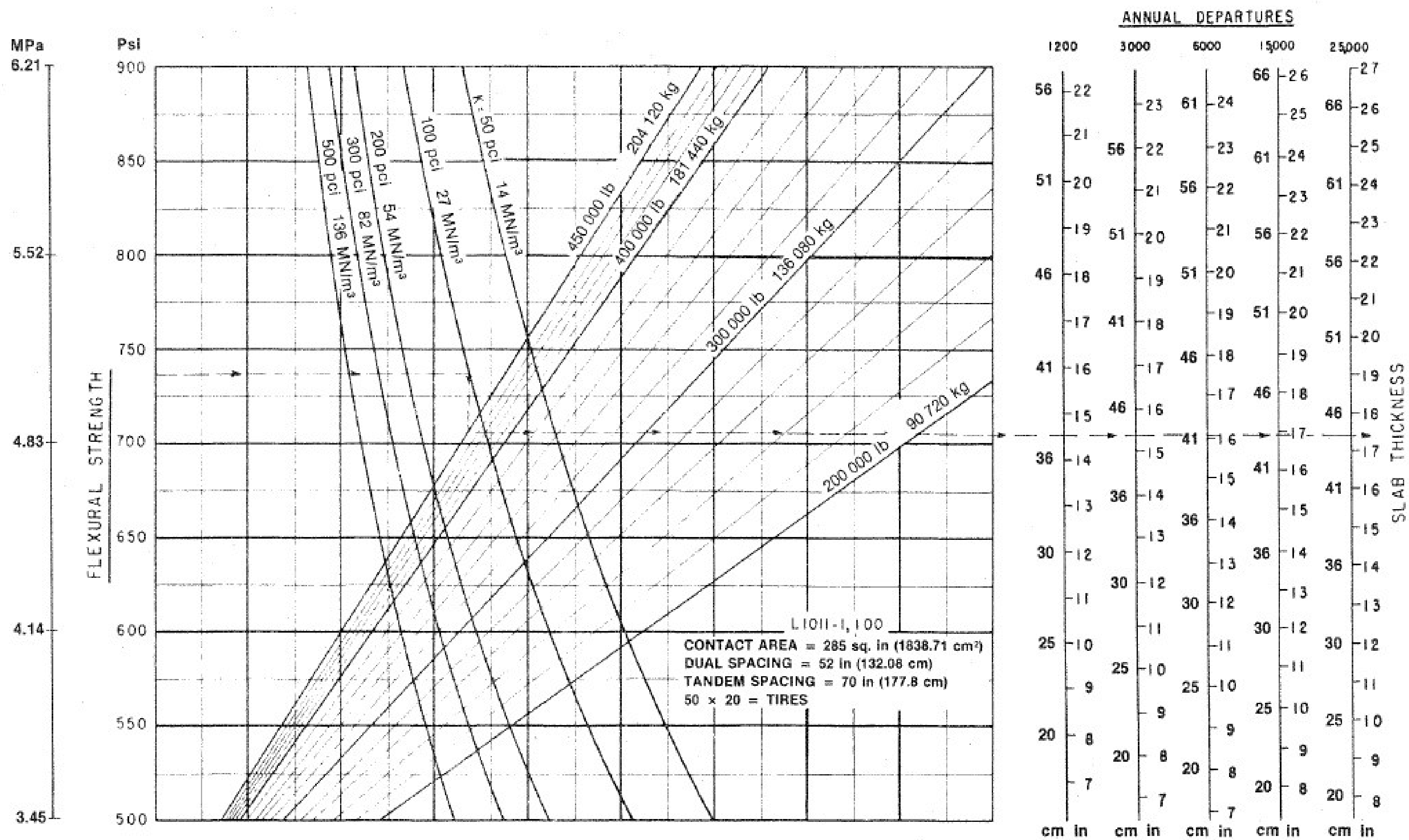


Figure 4-53. Rigid pavement design curves – L-1011-1, 100

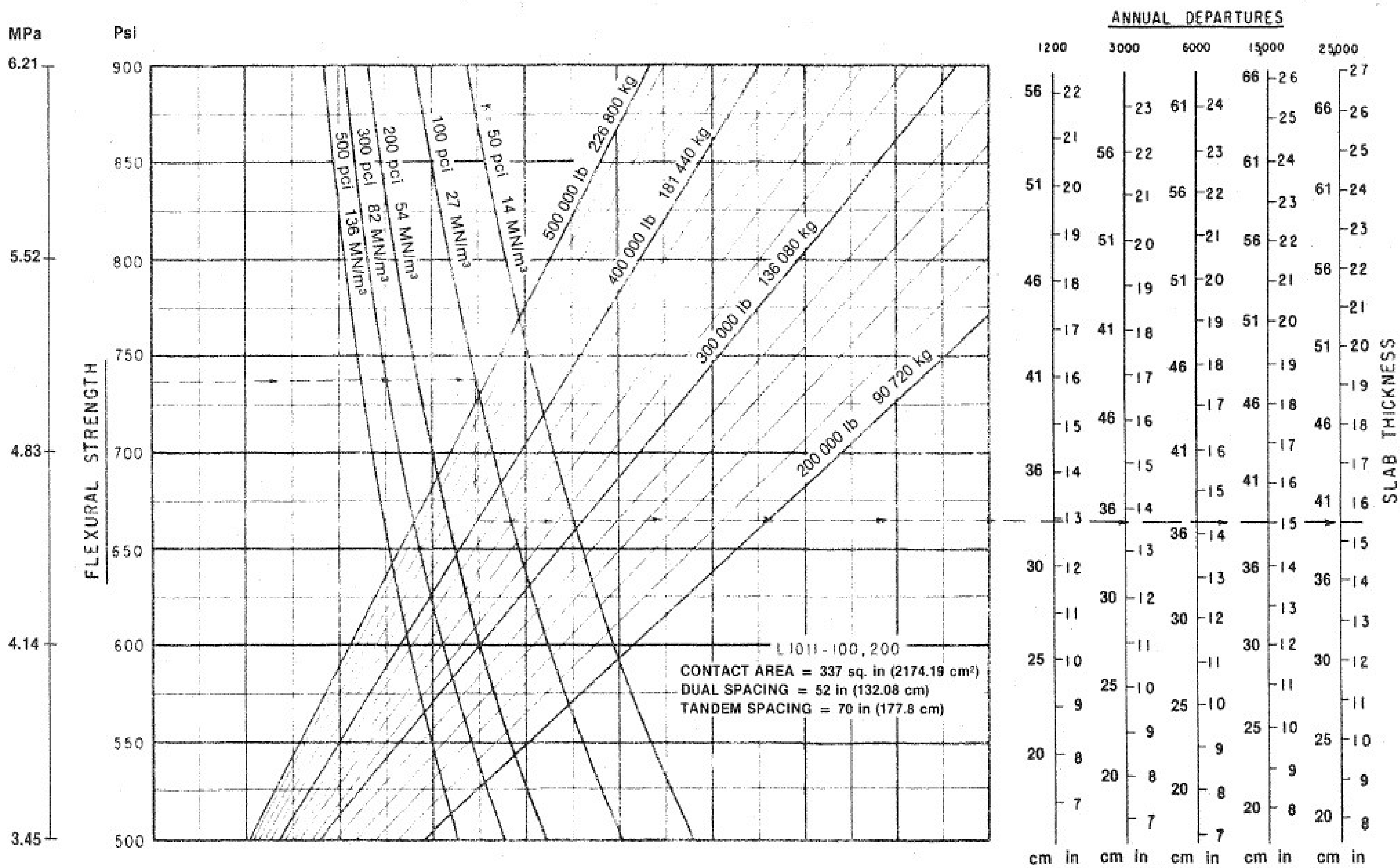


Figure 4-54. Rigid pavement design curves – L1011-100, 200

4.4.20. Critical and non-critical areas

4.4.20.1. The design curves, Figures 4-46 through 4-54 are used to determine the concrete slab thickness for the critical pavement areas. A 0.9T thickness for non- critical areas applies to the concrete slab thickness. For the variable thickness section of the thinned edge and transition section, the reduction applies to the concrete slab thickness. The change in thickness for transitions should be accomplished over an entire slab length or width. In areas of variable slab thickness, the sub-base thickness must be adjusted as necessary to provide surface drainage from the entire subgrade surface. For fractions of an inch of 0.5 or more, use the next higher whole number; for less than 0.5, use the next lower number.

4.4.21. Stabilized sub-base

4.4.21.1. Stabilized sub-base is to be required for all new rigid pavements designed to accommodate aircraft weighing 100000 lb (45400 kg) or more. The structural benefit imparted to a pavement section by a stabilized sub-base is reflected in the modulus of subgrade reaction assigned to the foundation. Exceptions to the policy of using stabilized sub-base are the same as given in 4.4.15.

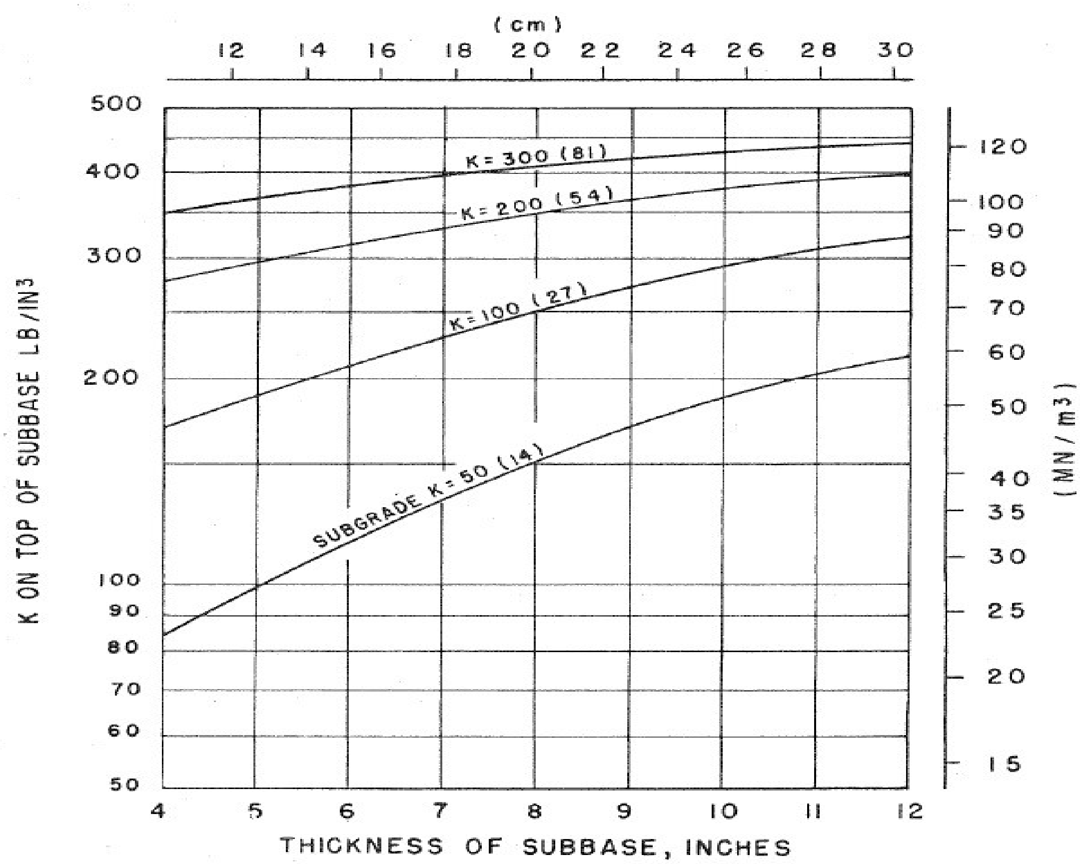


Figure 4-55. Effect of stabilized sub-base on subgrade modulus

4.4.21.2. <u>Determination of k value for stabilized sub-base.</u> The effect of stabilized sub-base is reflected in the foundation modulus. The difficulty in assigning a foundation modulus is that test data will not be available during the design phase. Figure 4-55shows the probable increase in k value with various thicknesses of stabilized sub-base located on subgrades of varying moduli. Figure 4-55 is applicable to cement stabilized and bituminous stabilized layers. Figure 4-55 was developed by assuming a stabilized layer is twice as effective as a well-graded crushed aggregate in increasing the subgrade modulus. Stabilized layers of lesser quality should be assigned somewhat lower k. values. After k value is assigned to the stabilized sub-base, the design procedure is the same as described in 4.4.18.

4.4.22. Design example

4.4.22.1. As an example of the use of the design curves, assume that a rigid pavement is to be designed for dual tandem aircraft having a gross weight of 350000 lb (160000 kg) and for 6000 annual equivalent departures of the design aircraft. The equivalent annual departure of 6000 includes 1200 annual departures of B-747 aircraft weighing 780000 lb (350000 kg) gross weight. The subgrade modulus of 100 Pei (25 MN/m^3) with poor drainage and frost penetration is 17 in (45 cm). The feature to be designed is a primary runway and requires 100 per cent frost protection. The subgrade soil is CL. Concrete mix designs indicate that a flexural strength of 650 psi (4.5 MN /m^2) can be readily produced with locally available aggregates.

4.4.22.2. The gross weight of the design aircraft dictates the use of a stabilized sub-base. Several thicknesses of stabilized sub-bases should be tried to determine the most economical section. Assume a cement stabilized sub-base will be used. Try a sub-base thickness of 6 in (15 cm). Using Figure 4-55, a 6 in (15 cm) thickness would likely increase the foundation modulus from 100 Pei (25 MNlm3) to 210 pci (57 MN/m3). Using Figure 4-48 dual tandem design curve, with the assumed design data, yields a concrete pavement thickness of 16.6 in (42 cm). This thickness would be rounded off 17 in (43 cm). Since the frost penetration is only 18 in (45 cm) and the combined thickness of concrete pavement and stabilized sub-base is 23 in (58 cm), no further frost protection is needed. Even though the wide body aircraft did not control the thickness of the slab, the wide bodies would have to be considered in the establishment of jointing requirements and design of drainage structures. Other stabilized sub-base thicknesses should be tried to determine the most economical section.

4.4.23. Optional rigid pavement design curves

4.4.23.1. When aircraft loadings are applied to a jointed edge, the angle of the landing gear relative to the jointed edge influences the magnitude of the stress in the slab. Figures 4-46 and 4-47, single wheel and dual wheel landing gear assemblies, are at the maximum stress when the gear is located parallel to the joint. Dual tandem assemblies do not produce the maximum stress when located parallel to the joint. Locating the dual tandem at an acute angle to the jointed edge will produce the maximum stress. Design curves, Figures 4-56 through 4-62, have been prepared for dual tandem gears located tangent to the jointed edge but rotated to the angle causing the maximum stress. These design curves can be used to design pavements in areas where aircraft are likely to cross the pavement joints at angles at low speeds such as runway holding aprons, runway ends, runway-taxiway intersections, aprons, etc. Use of Figures 4-56 to 4-62 is optional and should only be applied in areas where aircraft are likely to cross pavement joints at an angle and at low speeds.

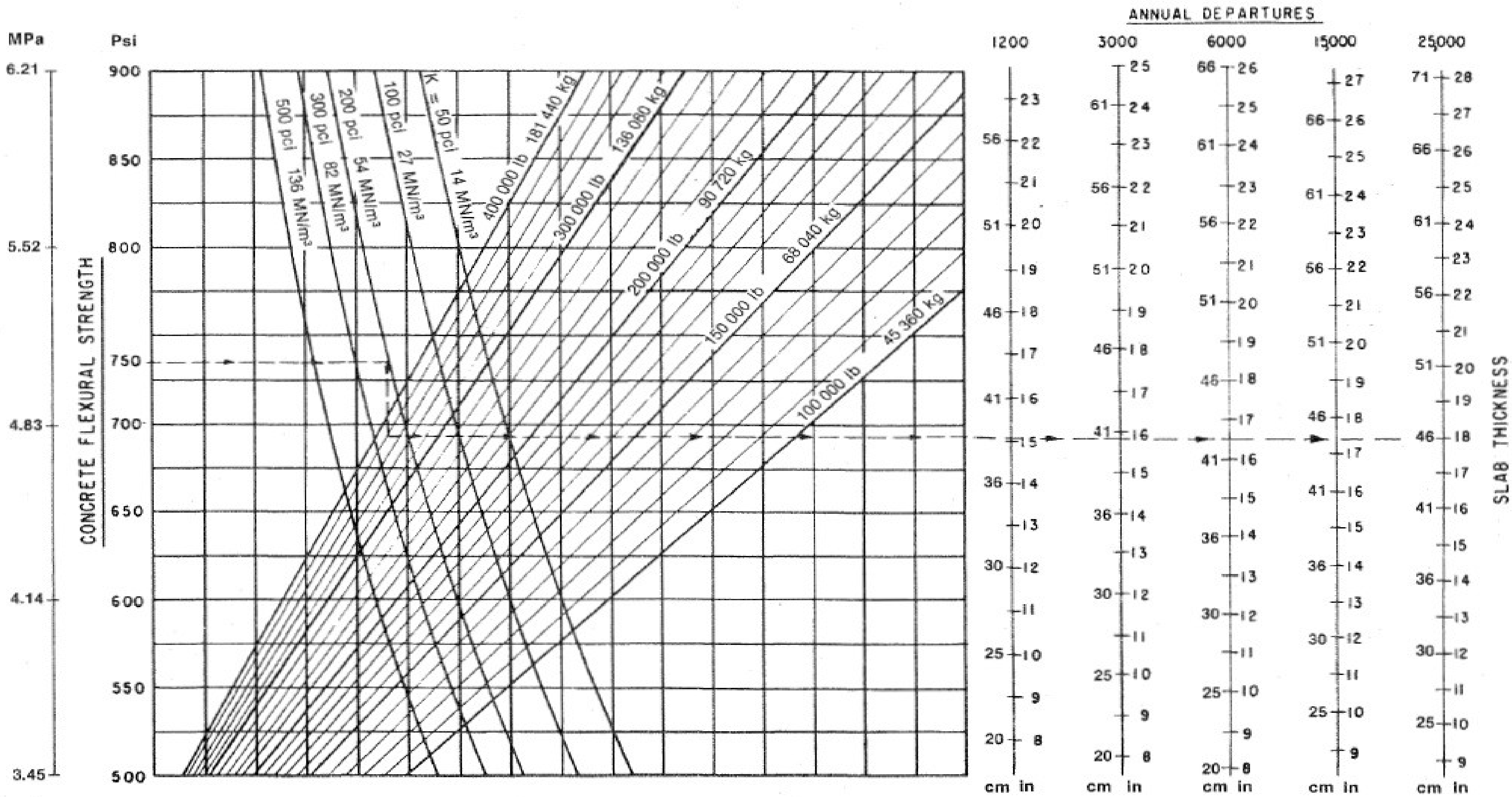


Figure 4-56. Optional rigid pavement design curves – dual tandem gear

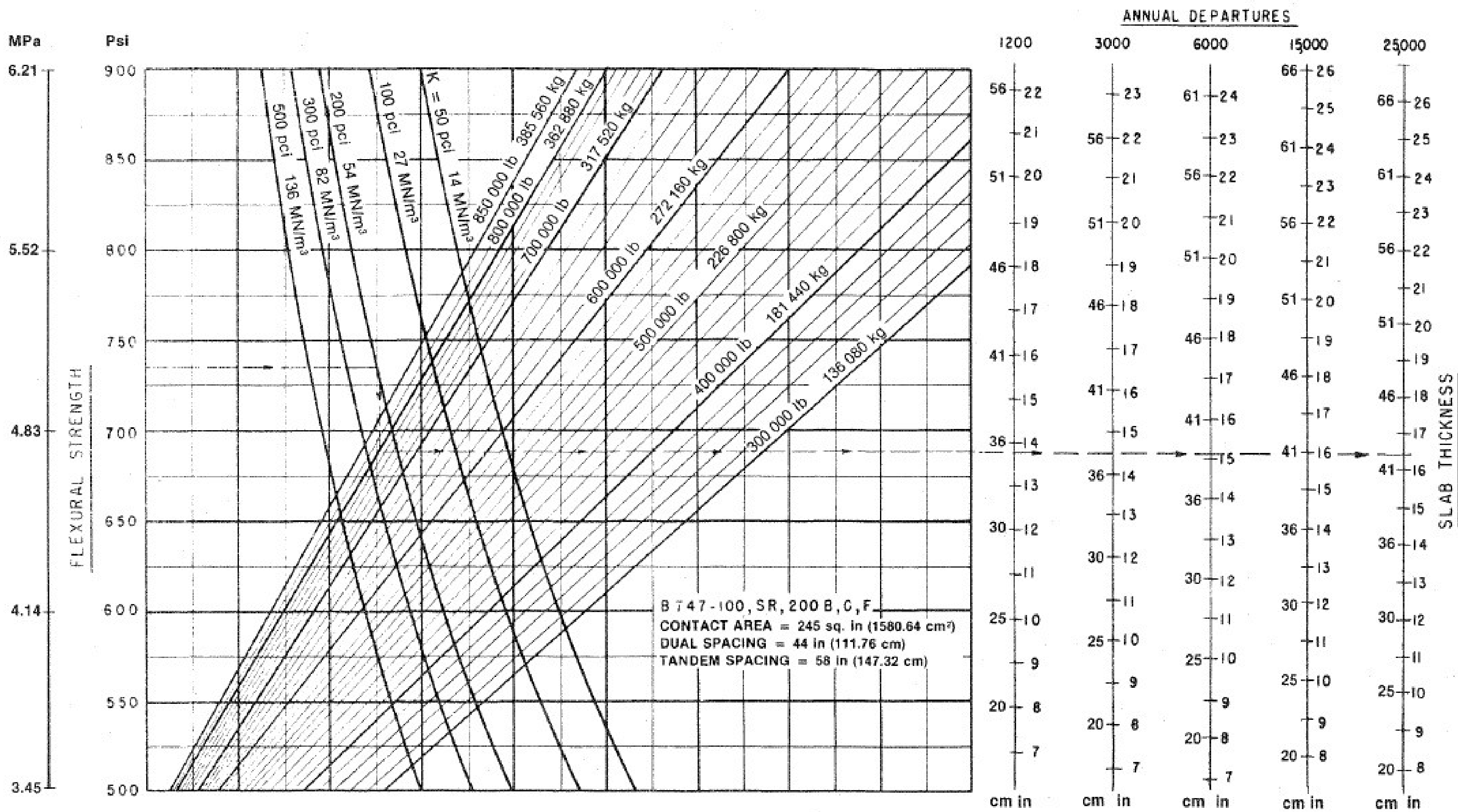


Figure 4-57. Optional rigid pavement design curves – B-747-100, SR, 200 B, C, F

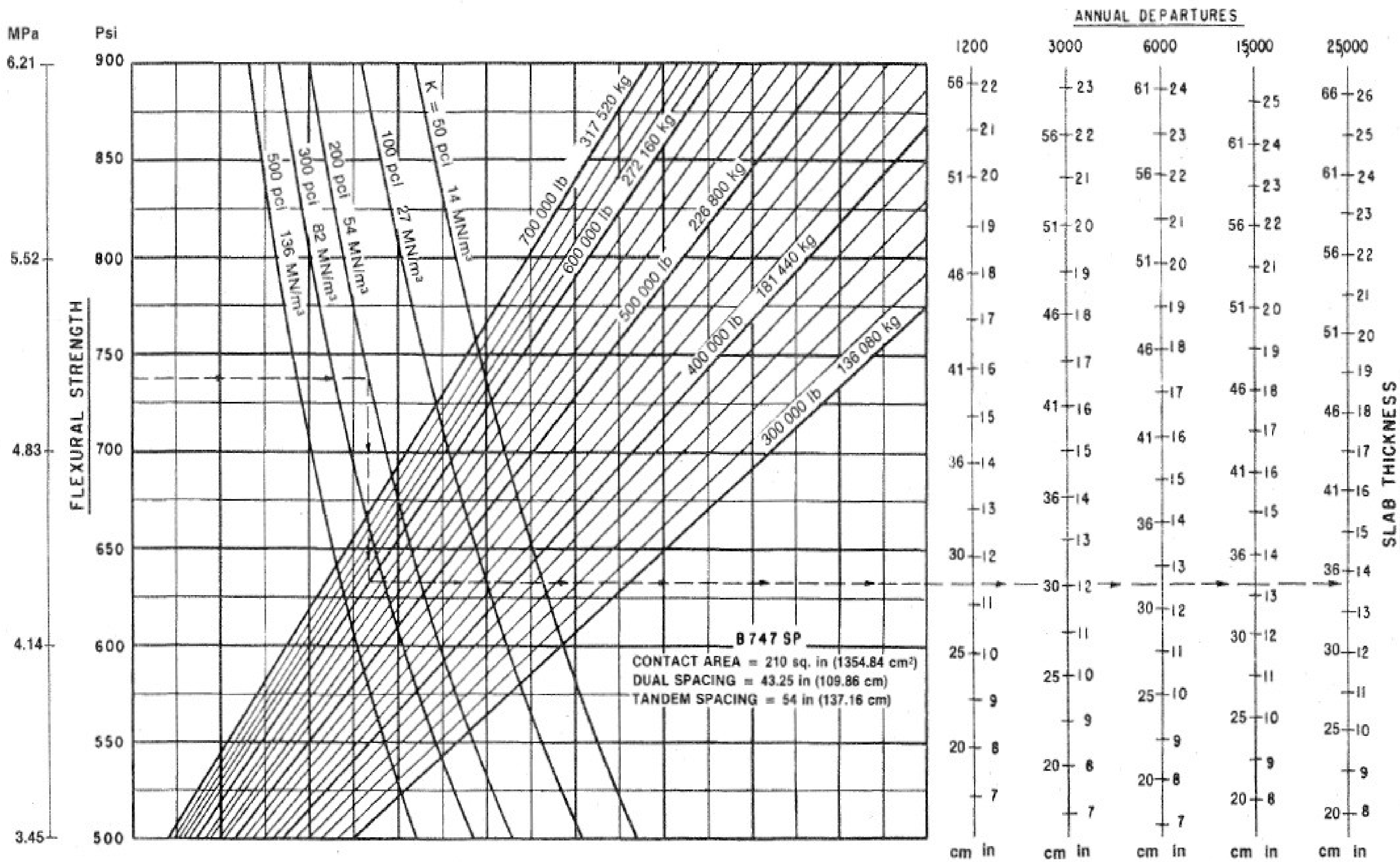


Figure 4-58. Optional rigid pavement design curve – B 747-SP

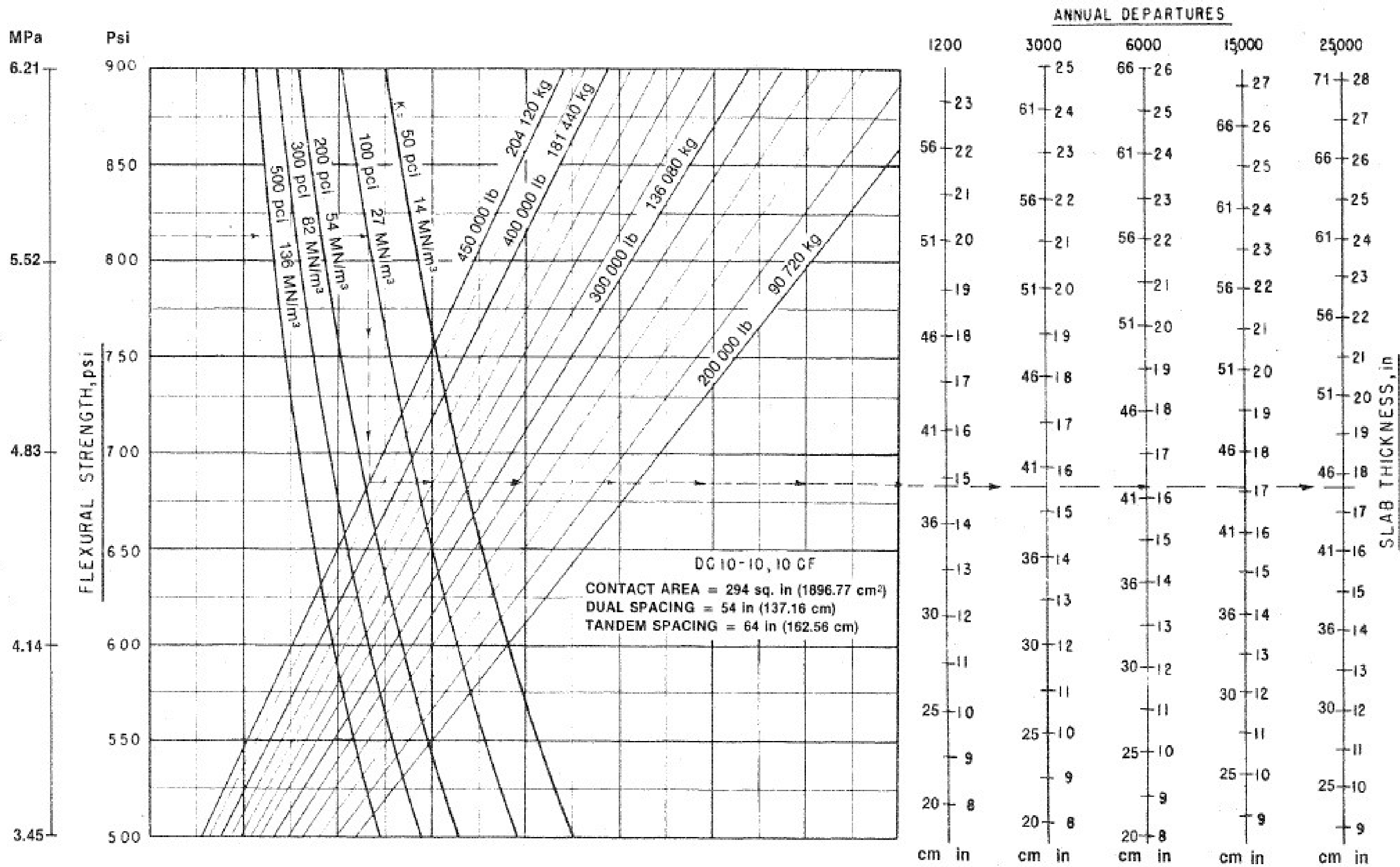


Figure 4-59. Optional rigid pavement design curves – DC 10-10, 10 CF

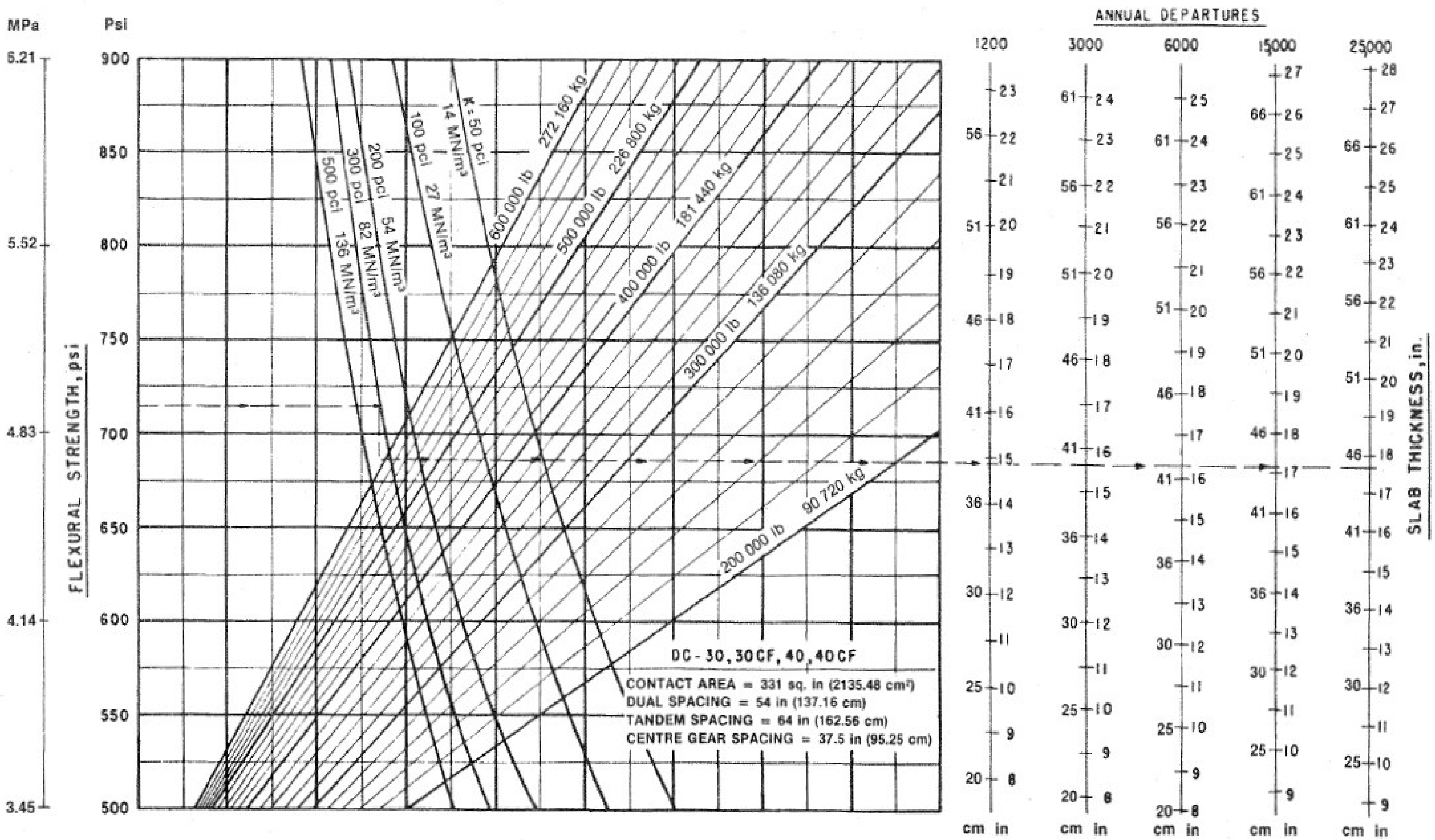


Figure 4-60. Optional rigid pavement design curves – DC 10-30, 30 CF, 40, 40CF

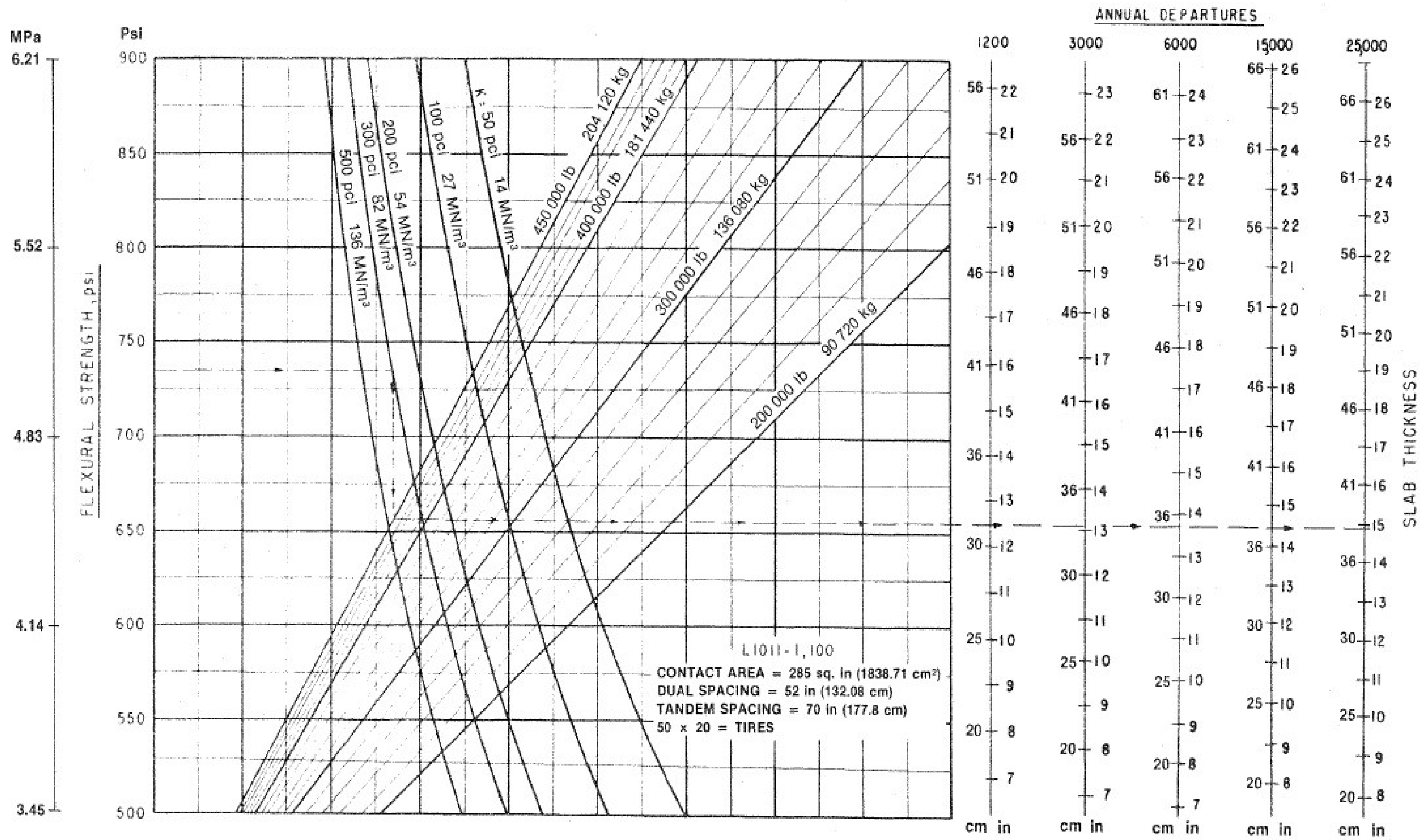


Figure 4-61. Optional rigid pavement design curves – L-1011-1, 100

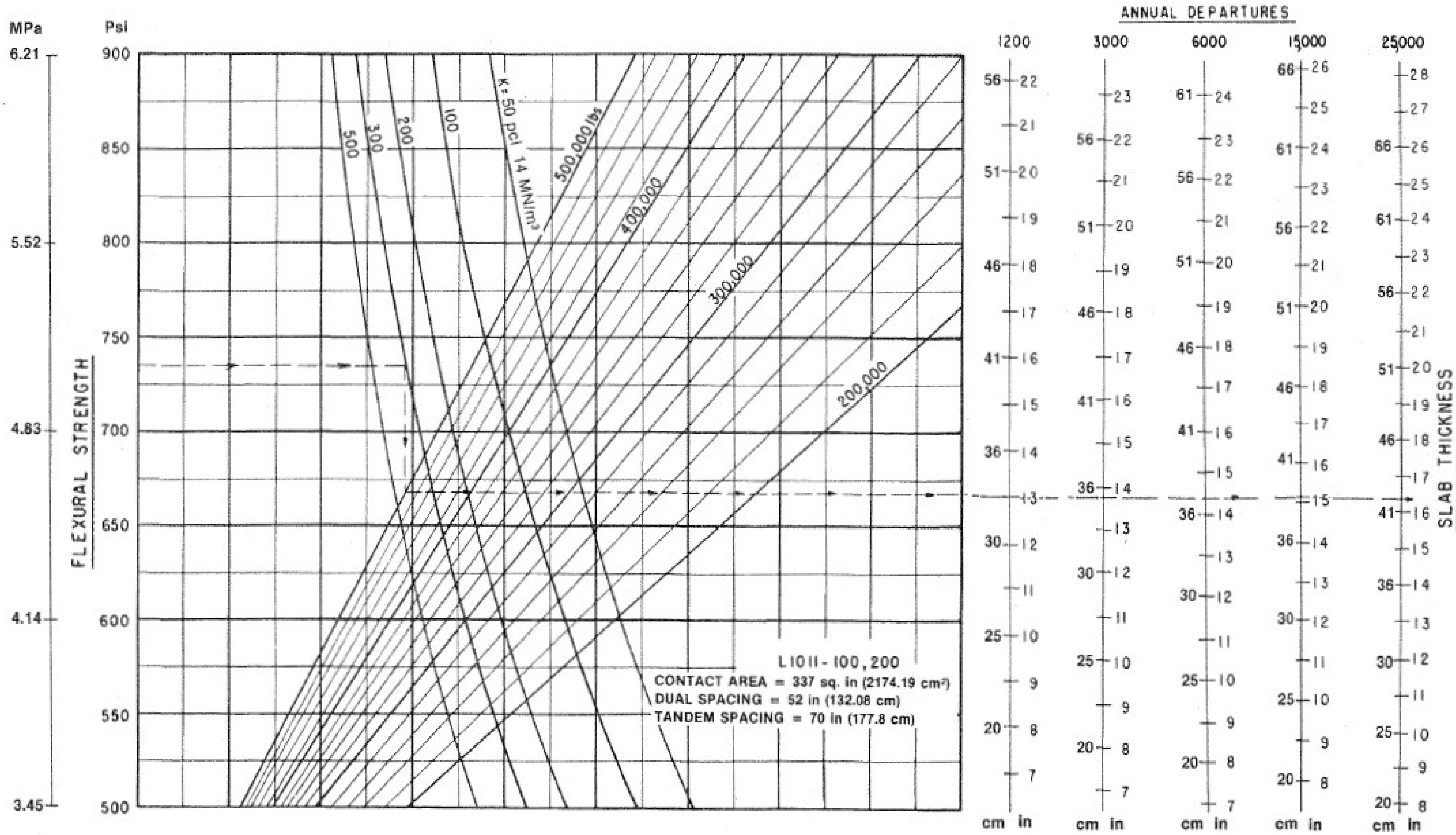


Figure 4-62. Optional rigid pavement design curve – L-1011-100, 200

4.4.24. High traffic volumes

4.4.24.1. There are a number of airports which experience traffic intensities far in excess of those indicated on the design curves. n these situations, maintenance is nearly impossible due to traffic intensity and makes initial construction even more important. Unfortunately, little information exists on the performance of airport pavements under high traffic intensities except for the experience gained through observation of in-service pavements. Rigid pavements designed to serve in situations where traffic intensity is high should reflect the following considerations.

4.4.24.2. Foundation. The foundation for the pavement provides the ultimate support to the structure. Every effort should be made to provide a stable foundation as problems arising later from an inadequate foundation cannot be practicably corrected after the pavement is constructed. The use of stabilized sub-base will aid greatly in providing a uniform, stable foundation. Generally speaking, the most efficient combination of rigid pavement thickness and stabilized sub-base thickness for structural capacity is a 1:1 ratio.

4.4.24.3. Thickness. Pavements subjected to traffic intensities greater than the25000 annual departure level shown on the design curves will require more thickness to accommodate the traffic volume. Additional thickness can be provided by increasing the pavement thickness in accordance with Table 4-12 shown below:

Table 4-12. Pavement thickness for high departure level expressed as a percentage of the 25 000 departure thickness

Annual departure level	Percentage of 25 000 departure thickness
50 000	104
100 000	108
150 000	110
200 000	112

The values given in Table 4-12 are based on extrapolations of research data and observations of in-service pavements. Table 4-12 was developed assuming logarithmic relationship between percentage of thickness and departures.

4.4.24.4. Panel size. Slab panels should be constructed to minimize joint movement. Small joint movement tends to provide for better load transfer across joints and reduces the elongation the joint sealant materials must accommodate when the slabs expand and contract. High-quality joint sealants should be specified to provide the best possible performance.

4.4.25. Reinforced concrete pavement

4.4.25.1. The main benefit of steel reinforcing is that, although it does not prevent cracking, it keeps the cracks that form tightly closed so that the interlock of the irregular faces provides structural integrity and usually improves pavement performance. By holding the cracks tightly closed, the steel minimizes the infiltration of debris into the cracks. The thickness requirements for reinforced concrete pavements are the same as plain concrete and are determined from the appropriate design curves. Steel reinforcement allows longer joint spacing, thus the cost benefits associated with fewer joints must be determined in the decision to use plain or reinforced concrete pavement.

4.4.25.2. Type and spacing reinforcement. Reinforcement may be either welded wire fabric or bar mats installed with end and side laps to provide complete reinforcement throughout the slab panel. End laps should be a minimum of 12 in (31cm) but not less than 30 times the diameter of the longitudinal wire or bar. Side laps should be a minimum of 6 in (15 cm) but not less than 20 times the diameter of the

$$A_s = \frac{3.7\ L\sqrt{Lt}}{f_s}$$

transverse wire or bar. End and side clearances should be a maximum of 6 in (15 cm) and a minimum of 2 in (5 cm) to allow for nearly complete reinforcement and yet achieve adequate concrete cover. Longitudinal members should be spaced not less than 4 in (10 cm) nor more than 12 in (31 cm) apart; transverse members should be spaced not less than 4 in (10 cm) nor more than 24 in (61 cm) apart.

4.4.25.3. Amount of reinforcement

a) The steel area required for a reinforced concrete pavement is determined from the subgrade drag formula and the coefficient of friction formula combined. The resultant formula is expressed as follows:

Where:

As = area of steel per foot of width or length, square inches

L - length or width of slab, feet

t = thickness of slab, inches

fs - allowable tensile stress in steel, psi

Note. – To determine the area of steel in metric units:

L = should be expressed in meters

t = should be expressed in millimeters

f_s = should be expressed in meganewtons per square meter

The constant 3.7 should be changed to 0.64

A_s = will then be in terms of square centimeter per meter

b) In this formula the slab weight is assumed to be 12.5 pounds per square foot, per inch of thickness (23.6 MN/m2). The allowable tensile stress in steel will vary with the type and grade of steel. It is recommended that allowable tensile stress be taken as two-thirds of the yield strength of the steel. Based on current specifications the yield strengths and corresponding design stresses (fs) are as listed in Table 4-13.

Table 4-13. Yield strengths of various grades of reinforcing steel

ASTM designation	Type and grade of steel	Yield strength psi (MN/m^2)	f_s psi	f_s (MN/m^2)
A 615	Deformed billet steel grade 40	40 000 (300)	27 000	(200)
A 616	Deformed rail steel, grade 50	50 000 (370)	33 000	(240)
A 616	Deformed rail steel, grade 60	60 000 (440)	40 000	(300)
A 615	Deformed billet steel, grade 60	60 000 (440)	40 000	(300)
A 185	Cold drawn welded steel wire fabric	65 000 (480)	43 000	(320)
A 497	Cold drawn welded deformed steel wire	70 000 (520)	47 000	(350)

c) The minimum percentage of steel reinforcement should be 0.05 per cent. The percentage of steel is computed by dividing the area of steel, A_S, by the area of concrete per unit of length (or width) and multiplying by 100. The minimum percentage of steel considered the least amount of steel which can be economically placed is 0.05 per cent. Steel reinforcement allows larger slab sizes and thus decreases the number of transverse contraction joints. The costs associated with providing a reinforced pavement must be compared with the savings realized in eliminating some of the transverse contraction joints to determine the most economical steel percentage. The maximum allowable s b length regardless of steel percentage is 75 ft (23 m)

4.4.26. Airport pavement overlays

4.4.26.1. General

a) Airport pavement overlays may be required for a variety of reasons. A pavement may have been damaged by overloading in such" a way that it cannot be maintained satisfactorily at a serviceable level. Similarly, a pavement in good condition may require strengthening to serve aircraft heavier than those for which the pavement was originally designed. A pavement" may al o require an overlay simply because the original pavement has served its design life and is "worn out". Generally, airport pavement overlays consist of either Portland cement concrete or bituminous concrete.

b) Definitions applicable to overlay pavements are as follows:

1) Overlay pavement. Pavement which is constructed on top of an existing pavement.

2) Bituminous overlay. Bituminous concrete pavement placed on an existing pavement.

3) Concrete overlay. Portland cement concrete pavement placed on an existing pavement.

4) Sandwich pavement. An over lay pavement containing granular separation course.

4.4.26.2. Design of bituminous overlays. Bituminous overlays can be applied to either flexible or rigid pavements. Certain criteria are applicable to the design of bituminous overlays whether they are to be placed over existing rigid or flexible pavements.

a) Overlay pavements which use a granular Separation course between the old and new surfaces are not allowed. Overlay pavements containing granular separate on courses are referred to as sandwich pavements. Sandwich pavements are not allowed because the separation course is likely to become saturated with water and provide rather unpredictable performance, Saturation of the separation course can be caused by the infiltration of surface water, ingress of ground or capillary water, or the condensation of water from the atmosphere. In any event, the water in the separation course usually cannot be adequately drained and drastically reduces the stability of the overlay.

b) Bituminous overlays for increasing strength should have a minimum thickness of 3 in (7.5 cm).

4.4.26.3. Bituminous overlays on existing flexible pavement

a) Use the appropriate basic flexible pavement curves to determine the thickness requirements for a flexible pavement for the desired load and number of equivalent design departures. A CBR value is required for the subgrade material and sub-base. Thicknesses of all pavement layers must be determined. The thickness of pavement required over the subgrade and sub-base and the minimum base course requirements must be compared with the existing pavement to determine the overlay requirements.

b) Adjustments to the various layers of the existing pavement may be necessary to complete the design. Bituminous surfacing may have to be converted to base, and base o sub-base conversion may be required A high-quality material may be converted to a lower-quality material, such as surfacing to base. A material may not be converted to a higher quality material. For example, excess sub-base cannot be converted to base. The equivalency factors shown in Tables 4 -9 and 4 -10 may be used as guidance in the conversion of layers. t must be recognized that the values shown are for new materials and t e assignment of factors for existing pavements must be based on judgement and experience. Surface cracking, high degree of oxidation, en deuce of low stability, etc. , are only a few of the considerations which would tend to reduce he equivalency factor, Any bituminous layer located between granular courses in the existing pavement should be evaluated inch for inch as granular base or sub-base course.

c) To illustrate the procedure of designing a bituminous overlay, assume an existing taxiway pavement composed of the following section. The subgrade CBR is 7, the bituminous surface course is 4 in (10 cm) thick, the base course is 6 in (15 cm) thick, the sub-base is 10 in (25 cm) thick, and the sub-base CBR is 15.Frost action is negligible. Assume the existing pavement is to be strengthened to accommodate a dual wheel aircraft weighing 100 s000 lb (45 000 kg) and an annual departure level of 3 000.The flexible pavement required for these conditions is:

Bituminous surface	4 in (10 cm)
Base	9 in (23 cm)
Sub-base	10 in (25 cm)
Total pavement thickness	23 in (58 cm)

The total pavement thickness must be 23 in (58 cm) in order to protect the CBR 7 subgrade. The combined thickness of surfacing and base must be 13 in (33 cm) to protect the CBR 15 sub-base. The existing pavement is thus 3 in (7.5 cm) deficient in total pavement thickness, all of which is due to base course. For the sake of illustration, assume the existing bituminous surface is in such a condition that surfacing can be substituted for base at an equivalency ratio of 1.3 to 1. Converting 2.5 in (6 cm) of surfacing to base yields a base course thickness of 9 2 in (23 cm) leaving 1.5 in (4 cm) of unconverted surfacing. A 2 5 in (6 cm) overlay would be required to achieve a 4 in (10 cm) thick surface. In this instance the minimum 3 in (7.5 cm) overlay thickness would control. A 3 in (7cm) overlay thickness would be required.

d) The most difficult part of designing bituminous overlays for flexible pavements is the determination of the CBR values for the subgrade and sub-base and conversion of layers. Subgrade and sub-base CBR values can best be determined by conducting field in-place CBR tests. The subgrade and sub-base must be at the equilibrium moisture content when field CBR tests are conducted. Normally a pavement which has been in place for at least 3 years will be in equilibrium. Layer conversions, i.e. converting base to sub-base, etc., are largely a matter of engineering judgement. When performing the conversions, it is recommended that any converted thicknesses never be rounded off.

4.4.26.4. Bituminous overlay on existing rigid pavement. To establish the required thickness of bituminous overlay for an existing rigid pavement, it is first necessary to determine the single thickness of rigid pavement required to satisfy the design" conditions. This thickness is then modified by a factor which controls the degree of cracking which will occur in the existing rigid pavement. The effective thickness of the existing rigid pavement is also adjusted by a condition factor C_b. The F and $C_b$factors perform two different functions in the bituminous overlay determination as discussed below:

a) The factor F which controls the degree of cracking "which will occur in the base pavement is a function of the amount of traffic and the subgrade strength. The F factor selected will dictate the final condition of the overlay and base pavement. The F factor in effect is indicating that the entire concrete single slab thickness determined from the design curves is not needed because a bituminous overlay pavement is allowed to crack and deflect more than a conventional rigid pavement. More cracking and deflection is allowable as the bituminous surfacing will not spall and can conform to greater deflections than a totally rigid pavement. Photographs of various overlay and base pavements shown in Figure 4-63 illustrate the meaning of the F factor. Figures 4-63 a), b) and c) show how the overlay and base pavements fail as more traffic is applied to a bituminous overlay on an existing rigid pavement . In the design of a bituminous overlay,the condition of the overlay and base pavement after the design life should be close to that shown in Figure 4-63 b), Figure 4-64 is a graph enabling the designer to select the appropriate value to yield a final condition close to that shown n Figure -63 b).

b) The condition factor C_b applies to the existing rigid pavement. The C_b factor is an assessment of the structural integrity of theexisting pavement. The determination of the proper C_b value is ajudgement decision for which only general guidelines can be provided. A C_b value of 1.0 should be used when the existing slabs contain nominal initial cracking and 0.75 when the slabs contain multiple cracking. The designer is cautioned that the range of C_b values used in bituminous overlay designs is different from the C_r values used in rigid overlay pavement design. The minimum C_b value is 0.75. A single C_b should be established for an entire area. The C_b value should not be .varied along a pavement feature.

c) After the F factor, condition factor C , and single thickness of rigid pavement have been established, the thickness of the bituminous overlay is computed from the following formula:

$$t= 2.5 (Fh - C_b h_e)$$

where t = thickness of bituminous overlay, inches

F = factor which controls the degree of cracking in the base pavement

h = single thickness of rigid pavement required for design conditions, inches. Use the exact value of h; do not round off.

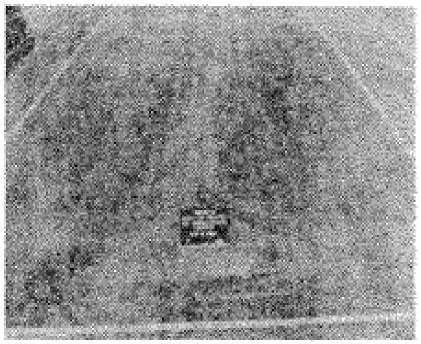
C_b= condition factor for base pavement ranging from 1.0 to 0.75

h_e = thickness of existing rigid pavement, inches

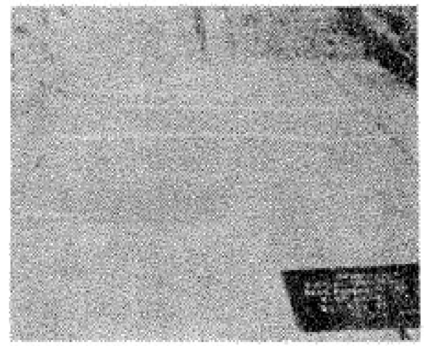
Calculation of bituminous overlay thickness in metric units should be performed using the formula below:

$$t= 2.5 (Fh - C_b h_e)$$

where t is in centimeters
h is in centimeters
h_e is in centimeters

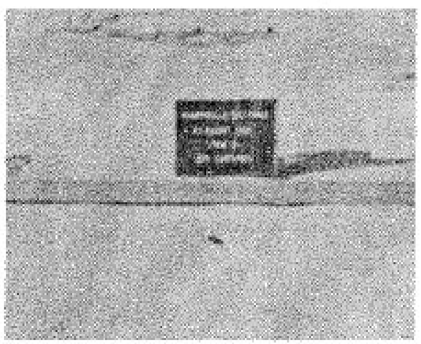


SURFACE OF OVERLAY

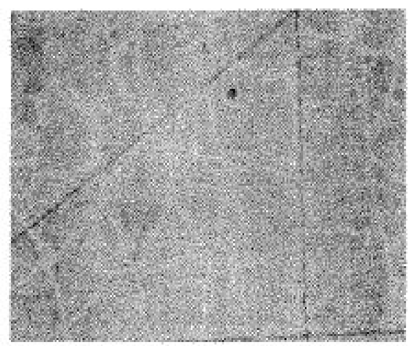


BASE PAVEMENT

(a)



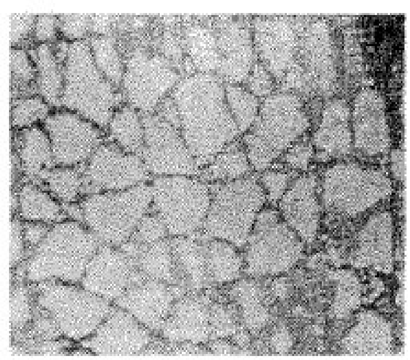
SURFACE OF OVERLAY



BASE PAVEMENT

(b)

SURFACE OF OVERLAY



BASE PAVEMENT

(c)

Figure 4-63. Illustration of various F factor for bituminous overlay design

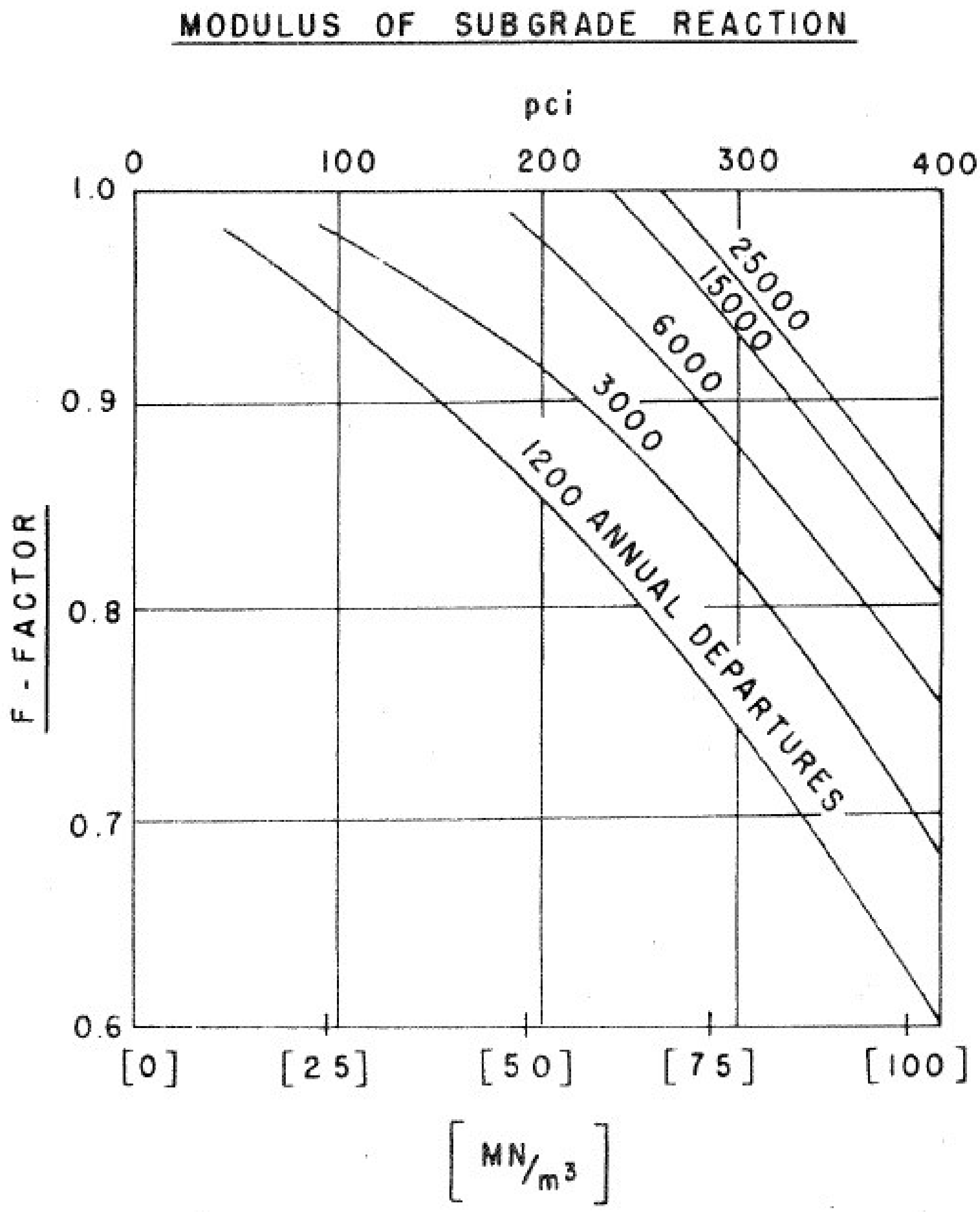


Figure 4-64. Graph of F factors vs. modulus of subgrade reaction for different traffic levels

d) The design of a bituminous overlay for a rigid pavement which has an existing bituminous overlay is slightly different. The designer should treat the problem as if the existing bituminous overlay were not present, calculate the overlay thickness required, and then adjust the calculated thickness to compensate for the existing overlay. If this procedure is not used, inconsistent results will often be produced.

1) An example of the procedure follows. Assume an existing pavement consists of a 10 in (25 cm) rigid pavement with a 3 in (7.5 cm) bituminous overlay. The existing pavement is to be strengthened to be equivalent to a single rigid pavement thickness of 14 in (36 cm). Assume an F factor of 0.9 and C_b of 0.9 are appropriate for the existing conditions.

2) Calculate the required thickness of bituminous overlay as if the existing 3 in (7.5 cm) overlay were not present.

$$t = 2.5\ (0.9 \times 14 - 0.9 \times 10)$$
$$t = 9 \text{ in } (23 \text{ cm})$$

3) An allowance is then made for the existing bituminous overlay. n this example assume the existing overlay is in such a condition that its effective thickness is only 2.5 in (6 cm). The required overlay thickness would then be 9 - 2.5 = 6.5 in (17cm). The determination of the effective thickness of the existing overlay is a matter of engineering judgment.

e) The formula for calculating the thickness of bituminous overlays on rigid pavements is limited in application to overlay thicknesses which are equal to or less than the thickness of the base rigid pavement. If the overlay thickness exceeds the thickness of the base pavement, the designer should consider designing the overlay as a flexible pavement and treating the existing rigid pavement as a high-quality base material. This limitation is based on the fact that the formula assumes the existing rigid pavement will support considerable load by flexural action. However, the flexural contribution becomes negligible for thick bituminous overlays.

4.4.26.5. Design of concrete overlays. Concrete overlays can be constructed on existing rigid or flexible pavements. The minimum allowable thickness for concrete overlays is 5 in (13 cm) when placed on a flexible pavement, directly on a rigid pavement, or on a leveling course. The minimum thickness of a concrete overlay which is bonded to an existing rigid pavement is 3 in (7.5 cm). The design of concrete overlays is predicated on equating the base and overlay section to a single slab thickness. The formulas presented were developed from research on test track pavements and observations of in-service pavements.

4.4.26.6. Concrete overlay on flexible pavement. The design of concrete overlays on existing flexible pavements is based on the design curves in 4.4.18.The existing flexible pavement is considered a foundation for the overlay slab.

a) For design of the rigid pavement, the existing flexible pavement shall be assigned a k value using Figure 4-35 or 4-55 or by conducting a plate bearing test on the existing flexible pavement. In either case the k value assigned should not exceed 500.

b) When frost conditions require additional thickness, the use of non-stabilized material is not allowed as this would result in a sandwich pavement. The frost protection must be provided by stabilized material.

4.4.26.7. Concrete overlay on rigid pavement. The design of concrete overlays on existing rigid pavements is also predicated on the rigid pavement design curves. The rigid pavement design curves indicate the thickness of concrete required to satisfy the design conditions for a single thickness of concrete pavement. Use of this method requires the designer to assign a k value to the existing foundation. The k value may be determined by field bearing tests conducted in test pits cut through the existing rigid pavement, or may be estimated from construction records for the existing pavement. The design of a concrete overlay on a rigid pavement requires an assessment of the structural integrity of the existing rigid pavement. The condition factor should be selected after a pavement condition survey. The selection of a condition factor is a matter of engineering judgment. The use of non-destructive testing (NOT) can be of considerable value in assessing the condition of an existing pavement. NDT can also be used to determine sites for test pits. In order to provide a more uniform assessment of condition factors, the following values are defined:

C_r = 1.0 for existing pavement in good condition - some minor cracking evident but no structural defects.

C_r = 0.75 for existing pavement containing initial corner cracks due to loading but no progressive cracking or joint faulting.

C_r = 0.35 for existing pavement in poor structural condition – badly cracked or crushed and faulted joints.

The three conditions discussed above are used to illustrate the condition factor rather than establish the only values available to the designer. Conditions at a particular location may require the use of an intermediate value of C_r within the recommended range.

a) Concrete overlay without leveling course.The thickness of the concrete overlay slab applied directly over the existing rigid pavement is computed by the following formula:

$$h_c = \sqrt[1.4]{h^{1.4} - C_r h_e^{1.4}}$$

h_c = required thickness of concrete overlay

h = required single slab thickness determined from design curves

h_e = thickness of existing rigid pavement

C_r = condition factor

Due to the inconvenient exponents in the above formula, graphic displays of the solution of the formula are given in Figures 4-65 and 4-66. These graphs were prepared for only two different condition factors, Cr = 1.0 and 0.75. The use of a concrete overlay pavement directly on an existing rigid pavement with a condition factor of less than 0.75 is not recommended because of the likelihood of reflection cracking.

b) Concrete overlay with leveling course. In some instances it may be necessary to apply a leveling course of bituminous concrete to an existing rigid pavement prior to the application of the concrete overlay. Under these conditions a different" formula for the computation of the overlay thickness is required. When the existing pavement and overlay pavement are separated, the slabs act more independently than when the slabs are in contact with each other. The formula for the thickness of an overlay slab when a leveling course is used is as follows:

$$h_c = \sqrt{h^2 - C_r h_e^2}$$

h_c = required thickness of concrete overlay

h = required single slab thickness determined from design curves

h_e = thickness of existing rigid pavement

C_r = condition factor

The leveling course must be constructed of highly stable bituminous concrete. A granular separation course is not allowed as this would constitute sandwich construction. Graphic solutions of the above equation are shown in Figures 4-67 and 4-68. These graphs were prepared for condition factors of 0.75 and 0.35. Other condition factors between these values can normally be computed to sufficient accuracy by interpolation.

c) Bonded concrete overlays. Concrete overlays which are bonded to existing rigid pavements are sometimes used under certain conditions. By bonding the concrete overlay to the existing rigid pavement the new section behaves as a monolithic slab. The thickness of bonded overlay required is computed by subtracting the thickness of the. Existing pavement from the thickness of the required slab thickness determined from design curves.

$$h_c = h - h_e$$

where:

h_c = required thickness of concrete overlay

h = required single slab thickness determined from design curves

h_e - thickness o f existing rigid pavement

Bonded overlays should be used only when the existing rigid pavement is in good condition. Defects in the existing pavement are more likely to reflect through a bonded overlay than other types of concrete overlays. The major problem likely to be encountered with bonded concrete overlays is achieving adequate bond. Elaborate surface preparation and exacting construction techniques are required to ensure bond.

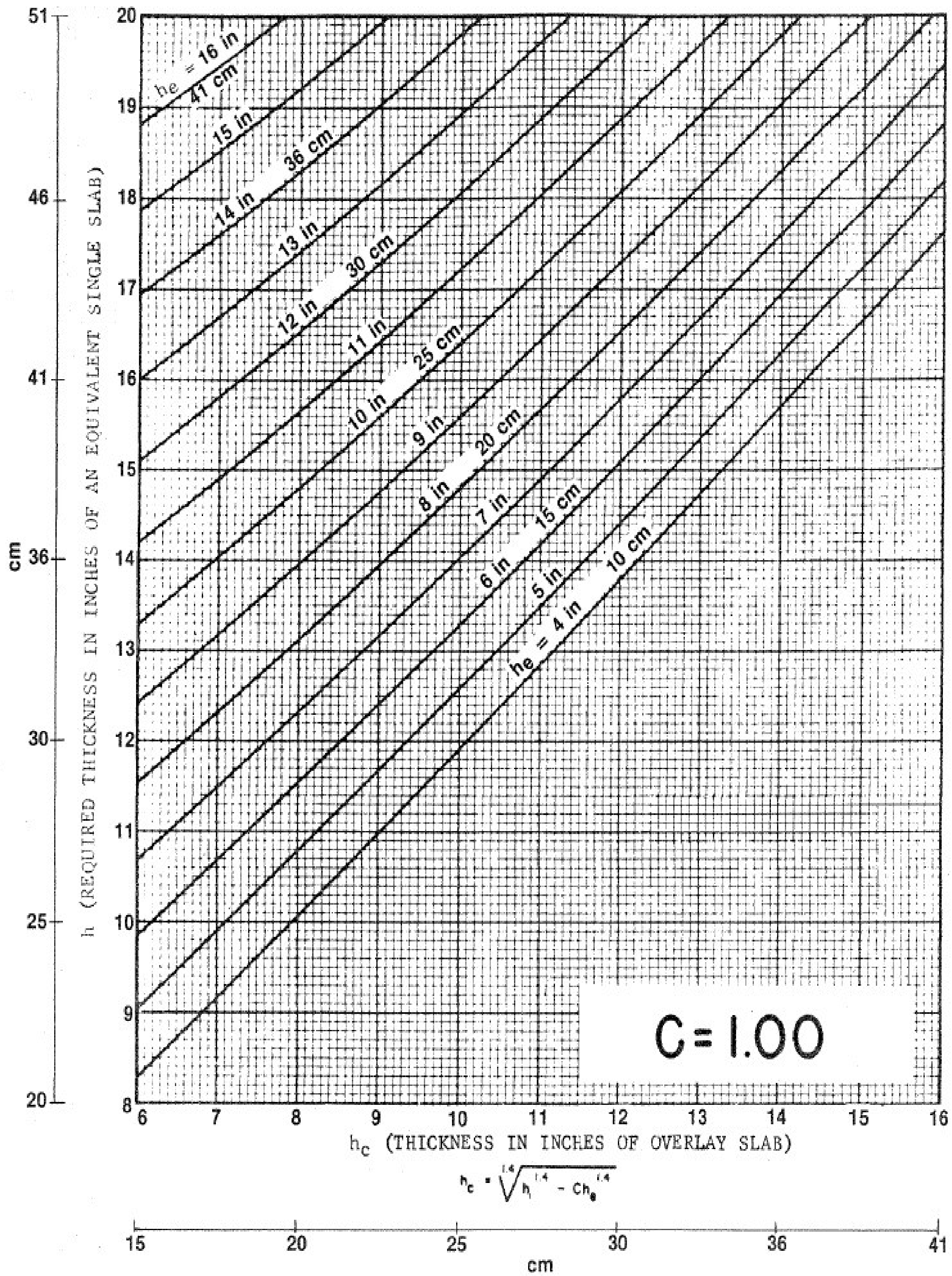


Figure 4-65. Concrete overlay on rigid pavement

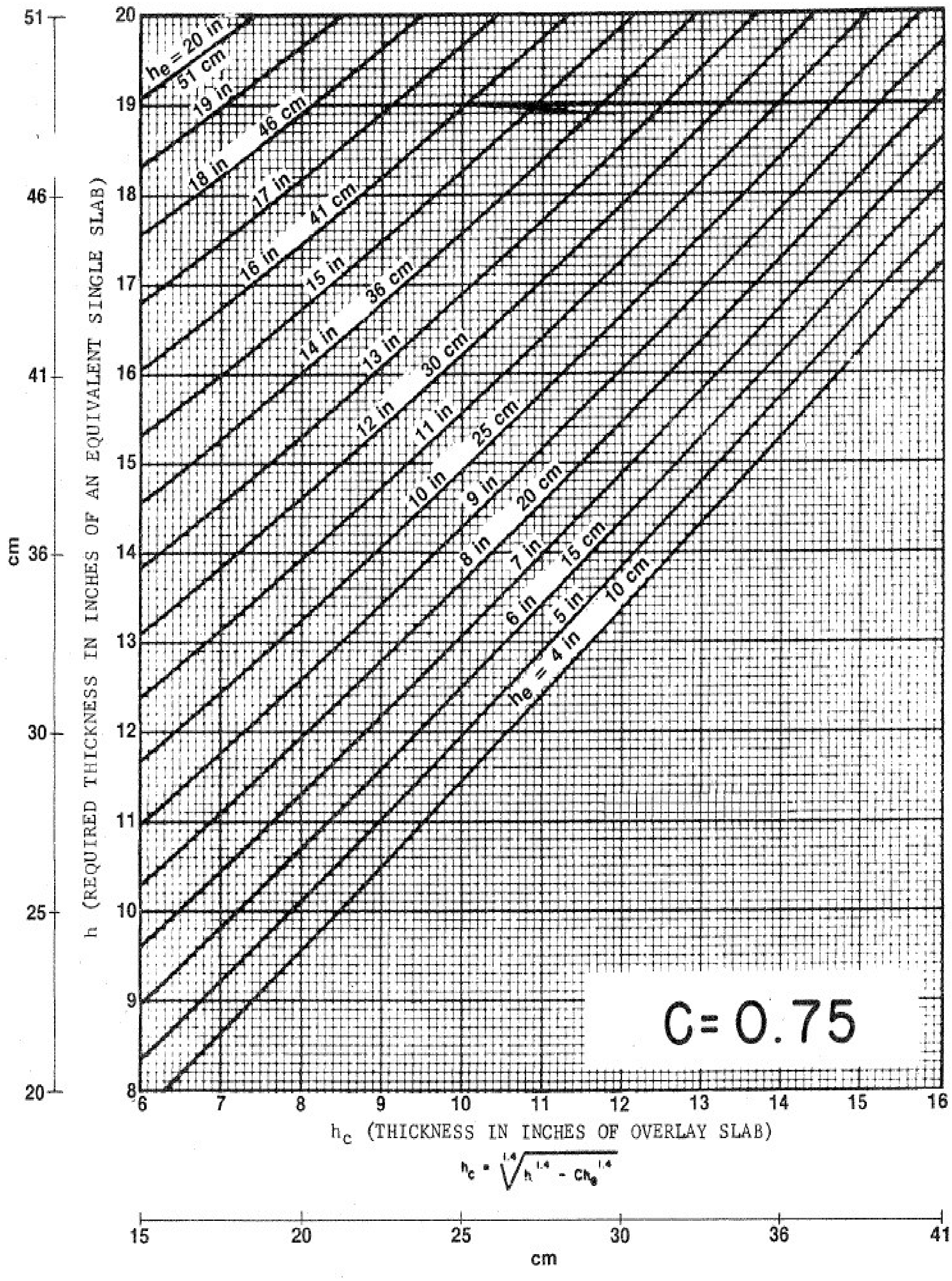


Figure 4-66. Concrete overlay on rigid pavement

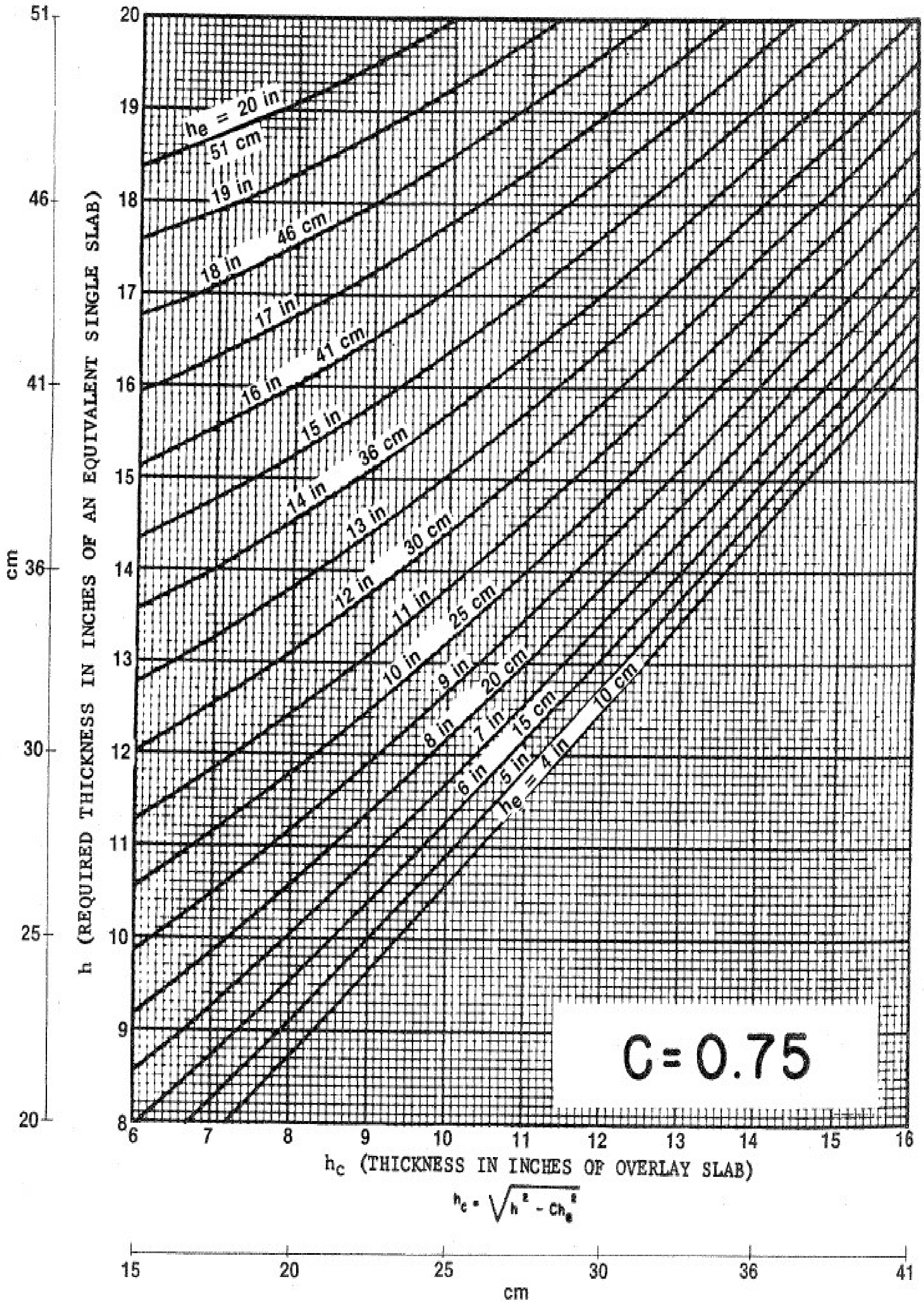


Figure 4-67. Concrete overlay on rigid pavement with leveling course

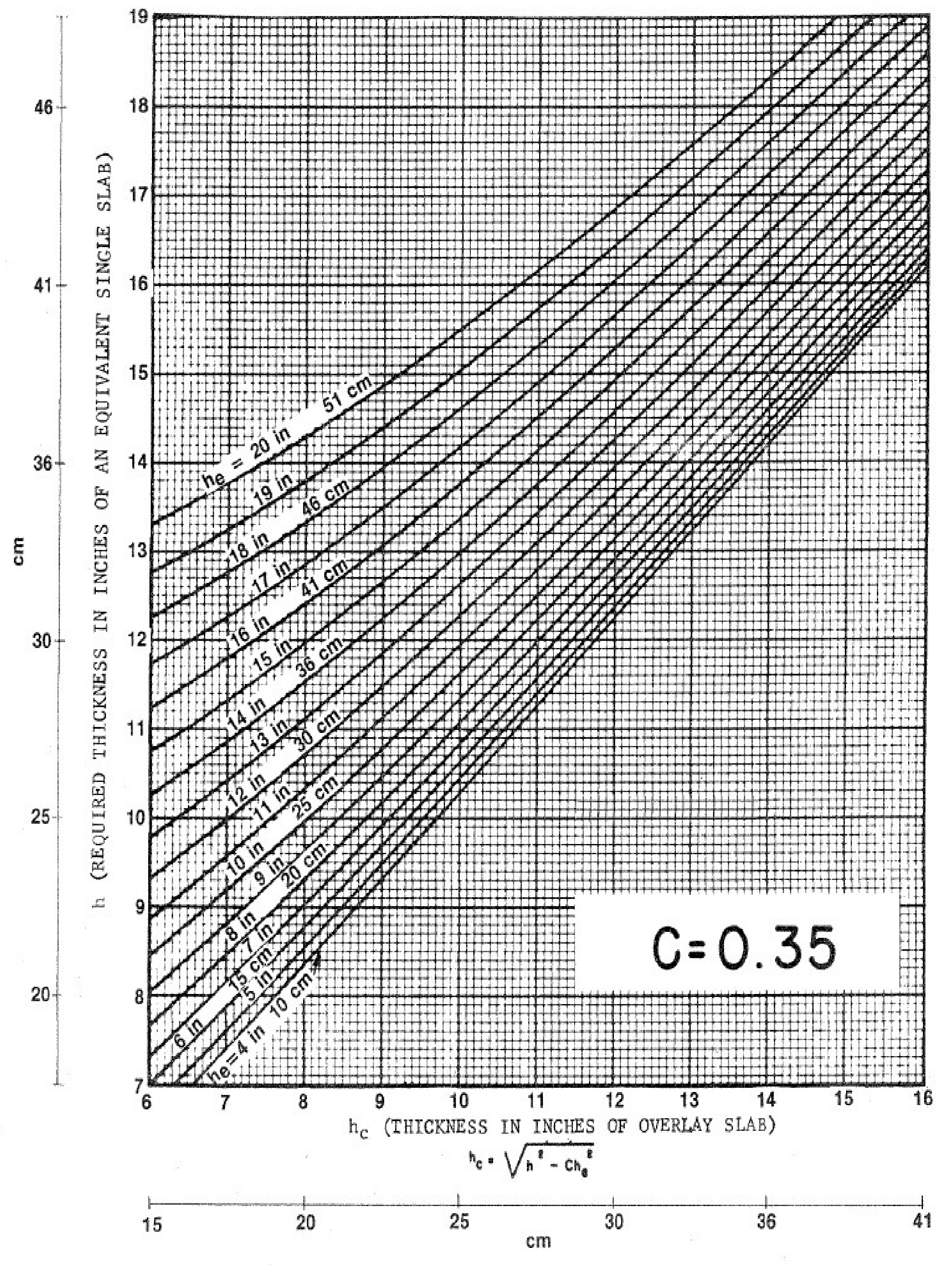


Figure 4-68. Concrete overlay on rigid pavement with leveling course

4.4.27. Pavement evaluation

4.4.27.1. Purposes of pavement evaluation

a) Airport pavements are evaluated for several reasons. Evaluations are needed to establish load carrying capacity for expected operations, to assess the ability of pavements to support significant changes from expected volumes or types of traffic, and to determine the condition of existing pavements for use in the planning or design of improvements which may be required to upgrade a facility.

b) Evaluation procedures are essentially the reversal of design procedures. Since the new FAA design methodology described in this Manna may result in slightly different thicknesses than other design methods it would be inappropriate to evaluate existing pavements by the new method unless they had a so been designed by that method, This could reduce allowable loads and penalize aircraft operators. To avoid this situation, pavements should be evaluated for the various conditions indicated in the following paragraphs.

4.4.27.2. Evaluations for expected operations. When airport pavements are subjected to the loads which were anticipated at the time of design, their evaluation should be based on that original design method. For example, if a pavement was designed by method X to serve certain aircraft for a 20-year life and the traffic using the pavement is essentially the same as was anticipated at the time of design, the pavement should be evaluated according to method X. The evaluator should recognize that some deterioration will occur over the 20 year design life. The load bearing strength of the pavement should not be reduced if the pavement is providing a safe operational surface. The prior evaluation curves are furnished in Appendix 4, to facilitate this evaluation policy, See Figures A4-8 to A -21.

4.4.27.3. Evaluations for changing traffic. Evaluations are sometimes required to determine the ability of an existing pavement to support substant1al changes in pavement loadings. This can be brought on by the introduction of different types of aircraft or changes in traffic volume. In these instances it is also recommended that existing pavements be evaluated according to the methods by which they were designed. The effect of changes in traffic volume is usually small and will not have a large impact on allowable loads. The effect of changes in aircraft types depends on the gear weight and gear configuration of the aircraft. The load carrying capacity of existing bridges, culverts, storm drains, and other structures should also be considered in these evaluations.

4.4.27.4. Evaluation for planning and design. Evaluations of existing pavements to be used in planning or designing improvements should be based on the method which will be used to design those improvements, The procedures to be followed in evaluating pavements according to the design criteria contained in this Manual are as follows:

a) Evaluation steps

1) Site inspection. This may include, in addition to the examination of the existing drainage conditions and drainage facilities of the site, consideration of the drainage area, outfall, water table, area development, etc . Evidence of frost action should be observed.

2) Records research and evaluation. This step may, at least in part, precede step 1) above. This step is accomplished by a thorough review of construction data and history, design considerations, specification, testing methods and results, as-built drawings, and maintenance history. Weather records and the most complete traffic history available are also parts of a usable records file. When soil, moisture, and weather conditions conducive to detrimental frost action exist, an adjustment to the evaluation may be required.

3) Sampling and testing. The need for and scope of physical tests and materials analyses will

be based on the findings made from the site inspection, records research, and type of evaluation. A complete evaluation for detailed design will require more sampling and testing than, for example, an evaluation intended for use in a master plan. Sampling and testing is intended to provide information on the thickness, quality and general condition of the pavement elements.

4) Evaluation report. Analysis of steps 1), 2) and 3) should culminate in the assignment of load carrying capacity to the pavement sections under consideration. The analyses, findings, and test results should be incorporated in a permanent record for future reference. While these need not be in any particular form, it is recommended that a drawing identifying area limit of specific pavement sections be included.

b) Direct Sampling procedure. The basic evaluation procedure for planning and design will be visual inspection and reference to the F design criteria, supplemented by the additional sampling, testing, and research which the evaluation processes may warrant. For relatively new pavement without visible signs of wear or stress, strength may be based on inspection of the as-constructed sections, with modification for any material variations or deficiencies of record. Where age or visible distress indicates the original strength no longer exists, further modification should be applied on the basis of judgement or a combination of judgment and supplemental physical testing. For pavements which consist of sections not readily comparable to AA design standards, evaluation should be based on FAA standards after materials comparison and equivalencies have been applied.

1) Flexible pavements. Laboratory or field CBR tests may be useful in supplementing soil classification tests. Figure 4 -69shows the approximate relationship between the subgrade classification formerly used by the FAA and CBR.

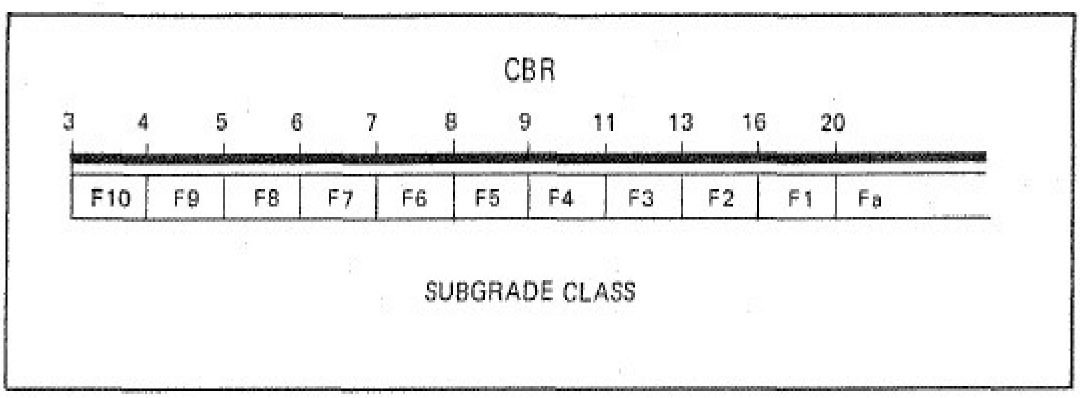


Figure 4-69. CBR –FAA subgrade class comparison

Conversion of F subgrade classification factors to CBR is permissible where CBR tests are not feasible. The thickness of the various layers in the flexible pavement structure must be known in order to evaluate the pavement. Thickness may be determined from borings or test pits As-built drawings and records can also be used to determine thicknesses if the records are sufficiently complete and accurate.

2) Rigid pavements. The evaluation requires the determination of the thickness of the component layers, the flexural strength of the concrete, and the modulus of subgrade

reaction.

a) The thickness of the component layers is usually available from construction records. Where information is not available or of questionable accuracy, thicknesses may be determined by borings or test pits in the pavement.

b) The flexural strength of the concrete is most accurately determined from test beams sawed from the existing pavement and tested in accordance with ASTM C-78. Sawed beams are expensive to obtain and costs incurred in obtaining sufficient numbers of beams to establish a representative sample may be prohibitive. Construction records may be used as a source of concrete flexural strength data, if available. The construction data will probably have to be adjusted for age as concrete strength increases with time. An approximate relationship between concrete compressive strength an flexural strength exists and can be computed by the following formula:

$$R = 9\sqrt{fc'}$$

where R = flexural strength

fc' = compressive strength

Tensile splitting tests (ATSM C-496) can be used to determine an approximate value of flexural strength. Tensile splitting strength should be multiplied by about 1.5 to approximate the flexural strength. It should be pointed out that the relationships between flexural strength and compressive strength or tensile splitting strength are approximate and considerable variations are likely.

c) The modulus of subgrade reaction is determined by plate bearing tests performed on the subgrade. These tests should be made in accordance with the procedures established in AASHTO T 222. An important part of the test procedure for determining the subgrade reaction modulus is the correction for soil saturation which is contained in the prescribed standard. The normal application utilizes a correction factor determined by the consolidation testing of samples at and saturated moisture content. For evaluation of older pavement, where evidence exists that the subgrade moisture has stabilized or varies through a limited range, the correction for saturation is not necessary. If a field plate bearing test is not practical, the modulus of subgrade reaction may be estimated by using Table 4-8.

d) Sub-bases will require an adjustment to the modulus of subgrade reaction. The thickness of the sub-base is required to calculate a k value for a sub-base The sub-base thickness can be determined from construction records or from borings. The guidance contained in 4.4.19 should be used in assigning a k value to a sub-base.

4.4.27.5. Flexible pavements. After all of the evaluation parameters of the existing flexible pavement have been established using the guidance given in the above paragraphs, the evaluation process is essentially the reverse of the design procedure. The design curves are used to determine the load carrying capacity of the existing pavement. Required inputs are subgrade and sub-base CBR values, thicknesses of surfacing, base and sub-base courses and an annual departure level. Several cheeks must be performed to determine the load carrying capacity of a flexible pavement. The calculation which yields the lowest allowable load will control the evaluation.

a) Total pavement thickness. Enter the lower abscissa of the appropriate design curve with the total pavement thickness of the existing pavement. Make a vertical projection to the annual departure. level line. At the point of intersection between the vertical projection and the departure level line make a horizontal projection across the design curve. Enter the upper abscissa with the CBR value of the subgrade. Make a vertical projection downward until it intersects the horizontal projection made previously. The point of intersection of these two projections will be in the vicinity of the load lines on the design curves. An allowable load is read by noting where the intersection point falls in relation to be load lines.

b) Thickness of surfacing and base. The combined thickness of surfacing and base must also be checked to establish the load carrying capacity of an existing flexible pavement. This calculation requires the C R of the sub-base, the combined thickness of surfacing and base and the annual departure level as inputs. The procedure is the same as that described in a) above, except that the sub-base CBR and combined thickness of surfacing and base are used to enter the design curves.

c) Deficiency in base course thickness. The_thickness of the existing base course should be compared with the minimum base con Se thicknesses shown in Figure 4-45. Inputs for use of this curve are to al pavement thickness and subgrade CBR. Enter the left ordinate of Figure 4-45with the total pavement thickness. Make a horizontal projection to the appropriate subgrade CBR line. At the point of intersection of the horizontal projection and the subgrade CBR line, make a vertical projection down to the lower abscissa and read the minimum base course thickness. Notice that the minimum base course thickness is 6 in (15 cm). If there is a deficiency in the thickness of the existing base course, the pavement should be closely monitored for signs of distress. The formulation of plans for overlaying the pavement to correct the deficiency should be considered.

d) Deficiency in surfacing thickness. e thickness of the existing surface course should be compared with that shown on the appropriate design curve. If the existing surface course is thinner than that given on the design curve, the pavement should be closely observed for surface failures. It is recommended that planning to correct the deficiency in surfacing thickness be considered.

4.4.27.6. Rigid pavements. The evaluation of rigid pavements for aircraft requires concrete flexural strength, k value of the foundation, slab thickness, and annual departure level as inputs. The rigid pavement design curves are used to establish load carrying capacity. The design curves are entered on the left ordinate with the flexural strength of the concrete. A horizontal projection is made to the k value of the foundation. At the point of intersection of the horizontal projection and the k line, a vertical projection is made into the vicinity of the load lines. The slab thickness is entered on the appropriate departure level scale on the right side of the chart. A horizontal projection is made from the thickness scale until it intersects the previous vertical projection. The point of intersection of these projections will be in the vicinity of the load lines. The load carrying capacity is read by noting where the intersection point falls in relation to the load lines.

CHAPTER 5: - METHODS FOR IMPROVING RUNWAY SURFACE TEXTURE

5.1. Purpose

5.1.1. CAR-14, Part I require that the surface of a paved runway be so constructed as to provide good friction characteristics when the runway is wet. Additional provisions contain minimum specifications for the configuration of runway surfaces and recognize in particular the need for some form of special surface treatment. The purpose of this chapter is to provide guidance on proved methods for improving runway surface texture. This includes essential engineering criteria for the design contraction and treatment of runway surfaces, the uniform and worldwide application of which is considered important to satisfy the relevant provisions of CAR-14, Part I.

5.2. Basic Considerations

5.2.1. Historical background

5.2.1.1. With the steady growth of aircraft mass and the associated significant increase in the take-off and landing speeds, a number of operational problems have become apparent with conventional types of runway surfaces. One of the most significant and potentially dangerous is the aquaplaning phenomenon which has been held responsible in a number of aircraft incidents and accidents.

5.2.1.2. Efforts to alleviate the aquaplaning problem have resulted in the development of new types of runway pavements of particular surface texture and of improved drainage characteristics. Experience has shown that these forms of surface finish, apart from successfully minimizing aquaplaning risks, provide a substantially higher friction level in all degrees of wetness, ie. from damp to a flooded surface.

5.2.1.3. It is now generally agreed that measuring and reporting wet friction conditions is not required to be done on a daily routine basis. This is the result of the development of a new philosophy of dealing with the wet runway problem. There is of course a need for a general improvement of the friction levels provided by runway surfaces in “normal” wet conditions and for the elimination of substandard surfaces in particular.

5.2.1.4. This has resulted in the definition of minimum acceptable wet friction levels for new and existing runways. Accordingly runways should be subject to periodic evaluation of the friction levels by using the techniques identified in Attachment A of technology for the finishing of surfaces which experience has proved effectively provides the wet friction requirement and minimizes aquaplaning.

5.2.2. Functional requirements

5.2.2.1. A runway pavement, considered as while, is supposed to fulfill the following three basic functions:

a) to provide adequate bearing strength;
b) to provide good riding qualities; and
c) to provide good surface friction characteristics.

The first criterion addresses the structure of the pavement, the second the geometric shape of the top of the pavement and the third the texture of the actual surface.

5.2.2.2. All three criteria are considered essential to achieve a pavement which will functionally satisfy the operational requirements. From the operational aspect, however, the third one is considered the most important because it has a direct impact on the safety of aircraft operations. Regularity and efficiency may also be affected. Thu s the friction criterion may become a decisive factor for the selection and the form of the most suitable finish of the pavement surface.

5.2.3. Problem identification

5.2.3.1. When in a dry and clean state, individual runways generally provide comparable friction characteristics with operationally insignificant differences in friction levels, regardless of the type of pavement (asphalt/cement concrete) and the configuration of the surface. Moreover, the friction level available is relatively unaffected by the speed of the aircraft, Hence, the operation in dry runway surfaces is satisfactorily consistent and no particular engineering criteria for surface friction are needed for this case.

5.2.3.2. In contrast, when the runway surface is affected by water to any degree of wetness (i.e. from a damp to a flooded state), the situation is entirely different. For this condition, the friction levels provided by individual runways drop significantly from the dry value and there is considerable disparity in the resulting friction level between different surface. This variance is due to differences in the type of pavement, the form of surfaces finish (texture) and the drainage characteristics (shape). Degradation of available friction (which) is particularly evident when aircraft operate at high speeds) can have serious implications on safely, regularity or efficiency of operations. The extent will depend on the friction actually required versus the friction provided.

5.2.3.3. The typical reduction of friction when a surface is wet and the reduction of friction as aircraft speed increases are explained by the combined effect of viscous and dynamic water pressures to which the tire/surface is subjected. The pressure causes a partial loss of “dry” contact the extent of which tends to increase with speed. There are conditions where the loss is practically total and the friction drops to negligible values. This is identified as viscous, dynamic or rubber-reverted aquaplaning. The manner in which there phenomenon affect different areas of the tire/surface interface and how they change in size with speed is illustrated in Figure 5-1.

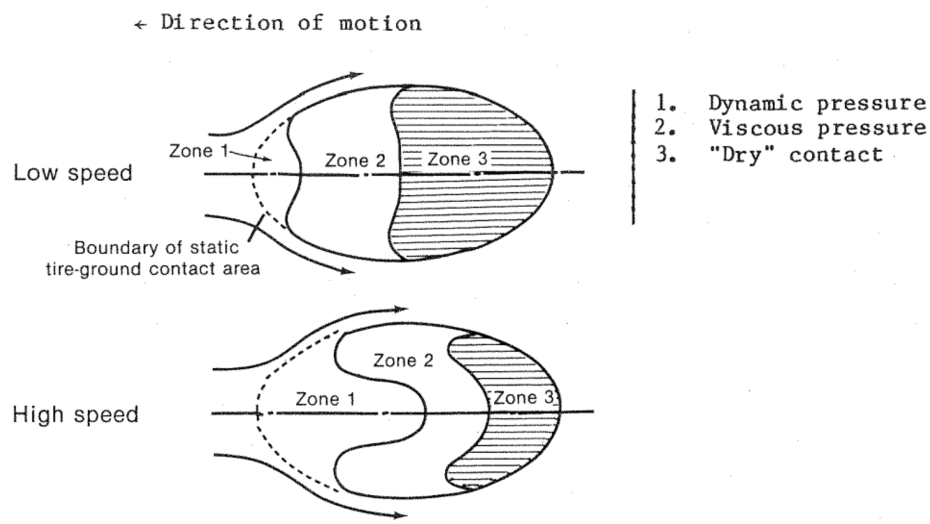


Figure 5-1. Areas of tire/surface interface

5.2.3.4. In the light of these considerations, it may be said that the wet runway case appears as a significant hazard and a potential threat to flight operations. Efforts to achieve a general improvement of the situation are, therefore, well justified. As mentioned earlier, the application of modern runway surface treatment is considered the most practical and effective technique to improve the friction characteristics of a wet runway.

5.2.4. Design objectives

5.2.4.1. In the light of the foregoing considerations, the objectives for runway pavement design, which are similarly applicable for maintenance, can be formulated as follows:

A runway pavement should be so designed and maintained as to provide a runway surface which meets adequately all functional requirements at all times throughout the anticipated lifetime of the pavement, in particular:

a. to provide in all anticipated conditions of wetness, high friction levels and uniform friction characteristics; and

b. to minimize the potential risk of all forms of aquaplaning, i.e. viscous, dynamic and rubber-reverted aquaplaning, Information on these types of aquaplaning is contained in the Airport Services Manual(Doc 9137,AN/898) Part 2, Pavement Surface Conditions.

5.2.4.2. As is outlined below, the provision of adequate wet runway friction is closely related to the drainage characteristics of the runway surface. The drainage demand in turn is determined by local precipitation rates. Drainage demand, therefore, is a local variable which will essentially determine the engineering efforts and associated investments/costs required to achieve the objective, In general, the higher the drainage demand, the more stringent the interpretation and application of the relevant engineering criteria will become.

5.2.5. Physical design criteria

5.2.5.1. General. The problem of friction on runway surfaces affected by water can in the light of the latest state-of-the-art be interpreted as a generalized drainage problem consisting of three distinct criteria:

a) surface drainage (surface shape);

b) tire/surface interface drainage (macrotexture); and

c) penetration drainage (microtexture).

The three criteria can significantly be influenced by engineering measures and it is important to note that all of them must be satisfied to achieve adequate friction in all possible conditions of wetness, i.e. from a damp to a flooded surface.

5.2.5.2. Surface drainage. Surface drainage is a basic requirement of utmost importance. It serves to minimize water depth on the surface, in particular in the area of. the wheel path. The objective is to drain water off the runway in the shortest path possible and particularly out of the area of the wheel path. Adequate surface drainage is provided primarily by an appropriately sloped surface (in both the longitudinal and transverse directions) and surface evenness. Drainage capability can, in addition, be enhanced by special surface treatments such as providing closely spaced transverse grooves or by draining water initially through the voids of a specially treated wearing course (porous friction course). The effectiveness of the drainage capability of modern types of surfaces is evident in that the surfaces when subjected to even high rainfall rates retain a rather damp appearance. It should be clearly understood, however, that special. surface treatment is not a substitute for poor runway shape, be it due to inadequate slopes or lack of surface evenness. This may be an important consideration when deciding on the most effective method for improving the wet friction characteristics of an existing runway surface.

5.2.5.3. Tire/surface interface drainage (macrotexture). The purpose of interface drainage (under a moving tire) is twofold:

a) to prevent as far as feasible residual surface bulkwater from intruding into the forward area of the interface; and
b) to drain intruding water to the outside of the interface.

The objective is to achieve high water discharge rates from under the tire with a minimum of dynamic pressure build-up. It has been established that this can only be achieved by providing a surface with an open rnacrotexture.

5.2.5.4. Interface drainage is actually a dynamic process, i.e., is highly susceptible to the square of speed. Macrotexture is therefore particularly important for the provision of adequate friction in the high speed range. From the operational aspect, this is most significant because it is in this speed range where lack of adequate friction is most critical with respect to stopping distance and directional control capability ,

5.2.5.5. In this context it is worthwhile to make a comparison between the textures applied in road construction and ru11ways. The smoother textures provided by road surfaces can achieve adequate drainage of the footprint of an automobile tire because of the patterned tire treads which significantly contribute to interface drainage. Aircraft tires, however, cannot be produced with similar patterned treads and have only a number of circumferential grooves which contribute substantially less to interface drainage. Their effectiveness diminishes relatively quickly with tire wear. The more vital factor, however, which dictates the macrotexture requirement is the substantially higher speed range in which aircraft operate. This may explain why some conventional runway surfaces which were built to specifications similar to road surfaces (relatively closed-textured) show a marked drop in wet friction with increasing speed and often a susceptibility to dynamic aquaplaning at comparatively small water depths.

5.2.5.6. Adequate macrotexture can be provided by either asphalt or cement concrete surfaces, though not with equal effort, stability or effectiveness. With cement concrete pavement surfaces, the required macrotexture may be achieved with transverse wire comb texturing when the surface is in the plastic stage or with closely spaced transverse grooves. With asphalt surfaces, the provision of macrotexture may be achieved by providing open graded surfaces.

5.2.5.7. A further design criteria calls for best possible uniformity of surface texture. This requirement is important to avoid undue fluctuations in available friction since these fluctuations would degrade antiskid braking efficiency or may cause tire damage.

5.2.5.8. The surface finish considered most effective from the standpoint of wet friction is grooving in the case of Portland cement concrete and the porous friction course in the case of asphalt. Their effectiveness can be explained by the fact that they not only provide good interface drainage, but also contribute significantly to bulk water drainage.

5.2.5.9. Penetration drainage (microtexture). The purpose of penetration drainage is to establish "dry" contact between the asperities of the surface and the tire tread in the presence of a thin viscous water film. The viscous pressures which increase with speed tend to prevent direct contact except at those locations of the surface where asperities prevail, penetrating the viscous film. This kind of roughness is defined as microtexture.

5.2.5.10. Microtexture refers to the fine-scale roughness of the individual aggregate of the surface and is hardly detectable by the eye, however, assessable by the touch. Accordingly, adequate microtexture can be provided by the appropriate selection of aggregates known to have a harsh surface. This excludes in particular all polishable aggregates.

5.2.5.11. Macro- and microtexture are both vital constituents for wet surface friction, i.e. both must adequately be provided to achieve acceptable friction characteristics in all different conditions of wetness. The combined effect of micro- and macrotexture of a surface on the resulting wet friction versus speed is illustrated in Figure 5-2 indicating also that the design objective formulated in 5.2.4 can be achieved by engineering means.

5.2.5.12. A major problem with microtexture is that it can change within short time periods (unlike macrotexture), without being easily detected. A typical example of this is the accumulation of rubber deposits in the touchdown area which will largely mask microtexture without necessarily reducing macrotexture. The result can be a considerable decrease in the wet friction level. This problem is catered for by periodic friction measurements which provide a measure of existing microtexture. If it is determined that low wet friction is caused by degraded surface microtexture, there are methods available to effectively restore adequate microtexture for existing runway surfaces (see 5.3).

5.2.6. Minimum specifications

5.2.6.1. The basic engineering specifications for the geometrical shape (longitudinal slope/trans verse slope/surface evenness) and for the texture (macrotexture) of a runway surface are contained in CAR-14, Part I.

5.2.6.2. Slopes. All new runways should be designed with uniform transverse profile in accordance with the value of transverse slope recommended in CAR-14, Part I and with a longitudinal profile as nearly level as possible. A cambered transverse section from a centre crown is preferable but if for any reason this cannot be provided then the single runway cross fall should be carefully related to prevailing wet winds to ensure that surface water drainage is not impeded by the wind blowing up the transverse slope. (In the case of single cross falls it may be necessary at certain sites to provide cut• off drainage along the higher edge to prevent water from the shoulder spilling over the runway surface.) Particular attention should be paid to the need for good drainage in the touchdown zone since aquaplaning induced at this early stage of the landing, once started, can be sustained by considerably shallower water deposits further along the runway.

5.2.6.3. If these ideal shape criteria are met, aquaplaning incidents will be reduced to a minimum, but departures from these ideals will result in an increase of aquaplaning probability, no matter how good· the friction characteristic of the runway surface may be. These comments hold true for major reconstruction projects and, in addition, when old runways become due for resurfacing the opportunity should be taken, wherever possible, to improve the levels to ass.ist surface drainage. Every improvement in shape helps, no matter how small.

5.2.6.4. Surface evenness. This is a constituent of runway shape which requires equally careful attention. Surface evenness is also important for the riding quality of high speed jet aircraft.

5.2.6.5. Requirements for surface evenness are described in CAR-14, Part I, Attachment A,5, and reflect good engineering practices. Failure to meet these minimum requirements can seriously degrade surface water drainage and lead to ponding. This can be the case with aging runways as a result of differential settlement and permanent deformation of the pavement surface. Evenness requirements apply not only for the construction of a new pavement but throughout the life of the pavement. The maximum tolerable deformation of the surface should be specified as a vital design criterion. This may have a significant impact on the determination of the most appropriate type of construction and type of pavement.

5.2.6.6. With respect to susceptibility to ponding when surface irregularities develop, runway shapes with maximum permissible transverse slopes are considerably less affected than those with marginal transverse slopes. Runways exhibiting ponding will normally require a resurfacing and reshaping to effectively alleviate the problem.

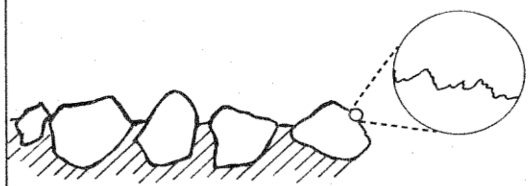
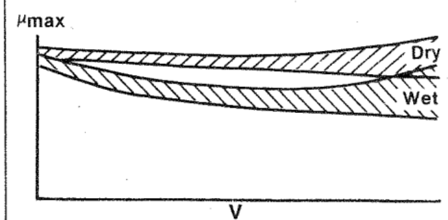
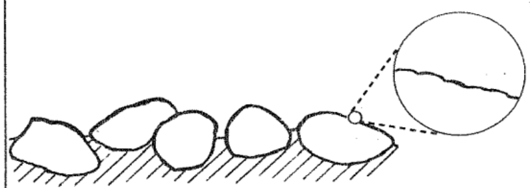
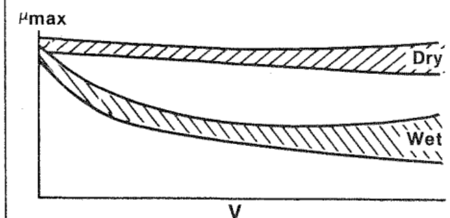
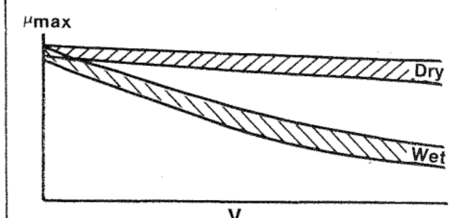
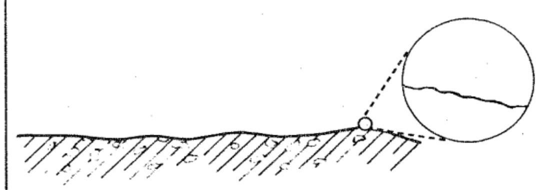
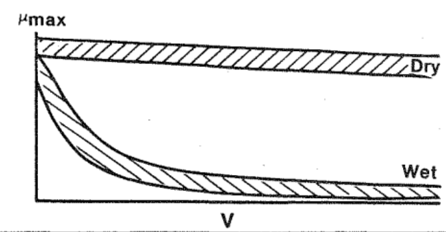
SURFACE				APROXIMATE TRENDS IN MAXIMUM TIRE-GROUND COEFFICIENT OF FRICTION FOR SMOOTH TREAD TIRE
No.	**MACRO-TEXTURE**		**MICRO-TEXTURE**	
I	OPEN macro-textured surfaces provide good bulk drainage of tire-ground contact area. In wet conditions μ decreases gradually with increase in V. Tread grooves have little effect. At high speeds μ may increase due to hysteresis effects.		HARSH micro-textured surfaces permit substantial penetration of thin fluid films; general level of friction is high.	
II			SMOOTH or POLISHED micro-textured surfaces have poor thin-film penetration properties and a generally low level of friction results.	
III	CLOSED macro-textured surfacces give poor contact area drainage. In wet conditions μ decreases rapidly with increase in V. Tread grooves are most effective on this type of surface.		HARSH micro-textured surfaces permit substantial penetration of thin fluid films; general level of friction is high.	
IV			SMOOTH or POLISHED micro-textured surfaces have poor thin-film penetration properties and a generally low level of friction results.	

Figure 5-2. Effect of surface texture on tire-surface coefficient of friction

5.2.6.7. Surface texture. Surface macrotexture requirements are specified in CAR-14, Part I in terms of average surface texture depth, which should not be less than 1 mm for new surfaces. It is also recognized that this provision will normally call for some form of special surface treatment. The minimum value for average texture depth has been empirically derived and reflects the absolute minimum required to provide adequate interface drainage. Higher values of average texture depth may be required where rainfall rates and intensities are a critical factor to satisfy interface drainage demand. Surfaces which fall short of the minimum requirement for average surface texture depth will show poor wet friction characteristics, particularly if the runway is used by aircraft with high landing speeds. Remedial action is, therefore, imperative. Methods for improving the wet friction characteristics of runways are described in 5. 3.

5.2.6.8. As outlined earlier, uniformity of the texture is also an important criterion. In this respect, there are several specific types of surfaces which meet this requirement (see 5.3). These surfaces will normally achieve average texture depths higher than 1 mm.

5.2.6.9. The macrotexture of a surface does not normally change considerably with time, except for the touchdown area as a result of rubber deposits. Therefore, periodic control of available average surface texture depth on the uncontaminated portion of the runway surface will only be required at long intervals.

5.2.6.10. With respect to microtexture there is no direct measure available to define the required fine scale roughness of the individual aggregate in engineering terms. Accordingly, there are no relevant specifications in CAR-14, Part I. However, from experience it is known that good aggregate must have a harsh surface and sharp edges to provide good water film penetration properties. It is also important that the aggregate be actually exposed to the surface and not coated entirely by a smooth material. Since microtexture is a vital constituent of wet friction regardless of speed, the adequacy of microtexture provided by a particular surface can be assessed generally by friction measurements. Lack of microtexture will result in a considerable drop in friction levels throughout the whole speed range. This will occur even with minor degrees of surface wetness (e.g. damp). This rather qualitative method may be adequate for detecting lack of microtexture in obvious cases.

5.2.6.11. Degradation of microtexture caused by traffic and weathering may occur, in contrast to macrotexture, within comparatively short time periods and can also change with the operational state of the surface. Accordingly, short-termed periodic checks by friction measurements are necessary, in particular with respect to the touchdown areas where rubber deposits quickly mask microtexture.

5.2.6.12. Runway surface friction calibration. CAR-14, Part I requires runway surfaces to be calibrated periodically to verify their friction characteristics when wet. These friction characteristics 1m.1st not fall below levels specified by the State for new construction (minimum design objective) and for maintenance. Wet friction levels, reflecting minimum acceptable limits for new construction and maintenance, which are in use in some States are given in Attachment A, 7 of CAR-14, Part I.

5.2.6.13. For the design of a new runway, the optimum application of the basic engineering criteria for runway shape and texture will normally provide a fair guarantee of achieving levels well in excess of the applicable specified minimum wet friction level. When large deviations from the basic specifications for shape or texture are planned, it will then be advisable to conduct wet friction measurements on different test surfaces in order to assess the relative influence of each parameter on wet friction, prior to deciding on the final design. Similar considerations apply for surface texture treatment of existing runways.

5.3. Surface treatment of runways

5.3.1. General

5.3.1.1. The methods described in this section are based on the experiences of several States. It is important that a full engineering appreciation of the existing pavement be made at each site before any particular method is considered, and that, once selected, the method is suitable for the types of aircraft operating. It should be noted that with respect to the improvement of the friction characteristics of existing runway pavements, a reshaping of the pavement may be required in certain cases prior to the application of special surface treatment in order to be effective.

5.3.2. Surface dressing of asphalt

5.3.2.1. Operational considerations. Aircraft with dual tandem undercarriage at tire pressure 1930 kPa and all-up masses exceeding 90 000 kg have been operating regularly for a number of years from runways which have been deliberately surface-dressed to improve friction. (Figure 5-3) There is no evidence of an increase in tire wear.

5.3.2.2. Consideration of existing pavement. The over-all shape and profile of the existing runway is not as important as it is with other treatments and, where a number of transverse and longitudinal slope changes occur in the runway length, surface dressing is probably the only suitable method short of expensive reshaping. In spite of the fact that the over-all shape need not be ideal, nevertheless, for a successful application of this treatment, the compacting equipment must be capable of following the minor surface irregularities to ensure a uniform adhesion of the chippings. Where this condition cannot be ensured, a new asphalt wearing course may be necessary before applying the surface dressing.

5.3.2.3. Effectiveness of treatment. A satisfactory surface dressing will initially raise the friction coefficient of the surface to a high value which, thereafter, depending on the intensity of traffic, will slowly decrease. Normally an effective life of up to five years can be expected.

5.3.2.4. Runway ends. Runway ends used for the start of take-off should not be treated. Aircraft will scuff in turning, both fuel spillage and heat will soften the binder, and blast will tend to loosen chippings.

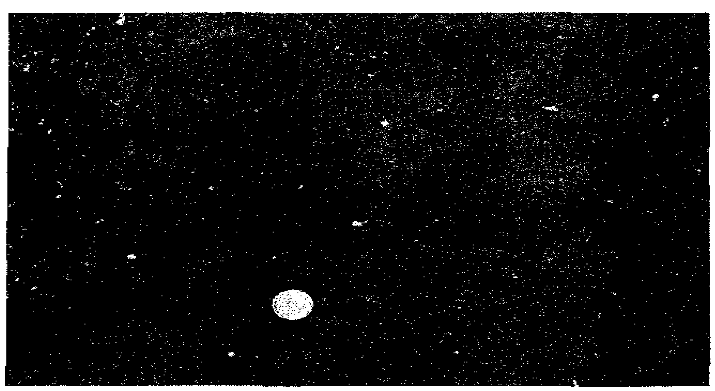


Figure 5-3. Surface dressing of asphalt

5.3.2.5. Chippings. 1be chippings may be from one of the following groups: Basalt, Gabbro, Granite, Gritstone, Hornfels, Porphyry or Quartzite.

5.3.2.6. Mechanical gritter. The chippings are distributed by a mechanical gritter of approved type incorporating a mechanical feed capable of ensuring that the selected rate of spread is rigidly maintained throughout the work.

5.3.2.7. Restrictions during bad weather. Work must not be carried out during periods of rain, snow or sleet or on frozen surfaces or on those on which water is lying, When weather conditions dictate, suitable protection must be afforded to the chippings during delivery.

5.3.2.8. Existing pit covers, gully gratings and aerodrome markings. These must be protected by masking, and the surface dressing finished neatly around them. When masking of the aerodrome markings is not indicated, they may be obliterated.

5.3.2.9. Preparation of the existing surfacing, Immediately before spraying the binder, the existing surfaces must be thoroughly cleaned by mechanical brooms, supplemented by hand brooming if necessary. All vegetation, loose materials, dust and debris, etc., must be removed as indicated.

5.3.2.10. Application of surface binder, 1be binder must be applied at the selected rate without variation and so that a film of uniform thickness results. Particular care must be taken to avoid dripping, spilling and creating areas of excessive thickness.

5.3.2.11. Application of coated chippings, The temperature of the chippings when applied to the sprayed surface binder must be not less than 83^{o}C when using bitumen binder and 72^{o}C when using tar binder. Before and during the rolling operation any bald patches must be covered with fresh chippings.

5.3.2.12. Rolling. The coated chippings must be rolled immediately after spreading and before loss of heat.

5.3.2.13. Final sweeping and rolling. Within three days of the gritting operation all loose chippings mu.st be swept from the surface with hand-brooms, loaded onto trucks and removed as directed. Then the entire surface nu.st again be thoroughly rolled at least three more times. All chippings must adhere firmly to the finished surface which should be of uniform texture and colour. The surface must be entirely free of irregularities due to scabbing, scraping, dragging, droppings, excessive overlapping, faulty lane or transverse junctions, or other defects, and it must be left clean and tidy. Under no circumstances should swept up chippings be re-used.

5.3.3. Grooving of pavements

5.3.3.1. Operational considerations. There are no operational objections to the grooving of existing surfaces. Experience of operating all types of aircraft from grooved surfaces over a number of years indicates that there is no limit within the foreseeable future to the aircraft size, loading or type for which such surfaces will be satisfactory. 1bere is inconclusive evidence of a slightly greater rate of tire wear under some operational conditions.

5.3.3.2. Methods of grooving include the sawing of grooves in existing or properly cured asphalt (Figure 5-4) or Portland cement concrete pavements, and the grooving or wire combing of Portland cement concrete while it is in the plastic condition. Based on current techniques, sawed grooves provide a more uniform width, depth, and alignment. This method is the most effective means of removing water from the pavement/tire interface and improves the pavement skid resistance. However, plastic grooving and wire combing are also effective in improving drainage and friction characteristics of pavement surfaces. They are cheaper to construct than the sawed grooves, particularly where very hard aggregates are used in pavements. Therefore the cost-benefit relationship should be considered in deciding which grooving technique should be used for a particular runway.

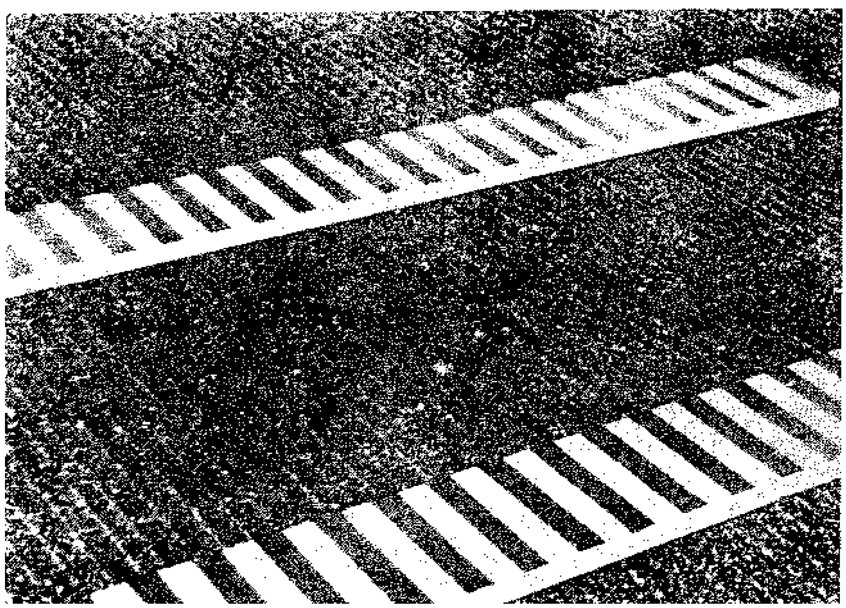


Figure 5-4. Grooving of asphalt surface
(Note.- Scale shows 2.5 cm divisions)

5.3.3.3. Factors to be considered. The following factors should be considered in justifying grooving of runways:

a) historical review of aircraft accidents/incidents related to aquaplaning at airport facility;
b) wetness frequency (review of annual rainfall rate and intensity);
c) transverse and longitudinal slopes, flat areas, depressions, mounds, or any other abnormalities that may affect water run-off;
d) surface texture quality as to slipperiness under dry or wet conditions, Polishing of aggregate, improper seal coating, inadequate microtexture/macrotexture, and contaminant buildup are some examples of conditions which may affect the loss of surface friction;
e) terrain limitations such as drop-offs at the ends of runway end safety areas;
f) adequacy of number and length of available runways;
g) cross-wind effects, particularly when low friction factors prevail; and
h) the strength and condition of existing runway pavements.

5.3.3.4. Evaluation of existing pavement. Asphalt surfaces must be examined to determine that the existing wearing course is dense, stable and well-compacted. If the surface exhibits fretting or where large particle fractions of coarse aggregate are exposed on the surface itself, then other methods will need to be considered, or resurfacing will have to be undertaken before grooving is put in hand. Rigid pavement mu s t be examined to ensure that the existing surface is sound, free of scaling or extensive spalls, or "working cracks". Apart from the condition of the surface itself, the ratio between transverse and longitudinal slopes becomes important. If the longituidinal slopes are such that the water run-off is directed along the runway instead of clearing quickly to the runway side drains, then a condition could arise when the grooves would fill with free water, fail to drain quickly and possibly encourage aquaplaning. For the same reason, surfaces with depressed areas should be repaired or replaced before grooving.

5.3.3.5. Effectiveness of treatment. Transverse grooving will always result in a measurable increase of the friction coefficient, though the extent of the improvement will be related to the quality of the existing surface. The duration of this improvement will depend 011 the properties of the asphalt wearing course, the climate and traffic. Experience has shown that grooving does not result in an increase of the rate of deterioration of the asphalt. The improvement also applies to rigid pavement surfaces as they are not adversely affected by the grooving. No grooves becoming clogged with dust, industrial waste, or other contaminants have been found although some minor rubber deposits have been observed.

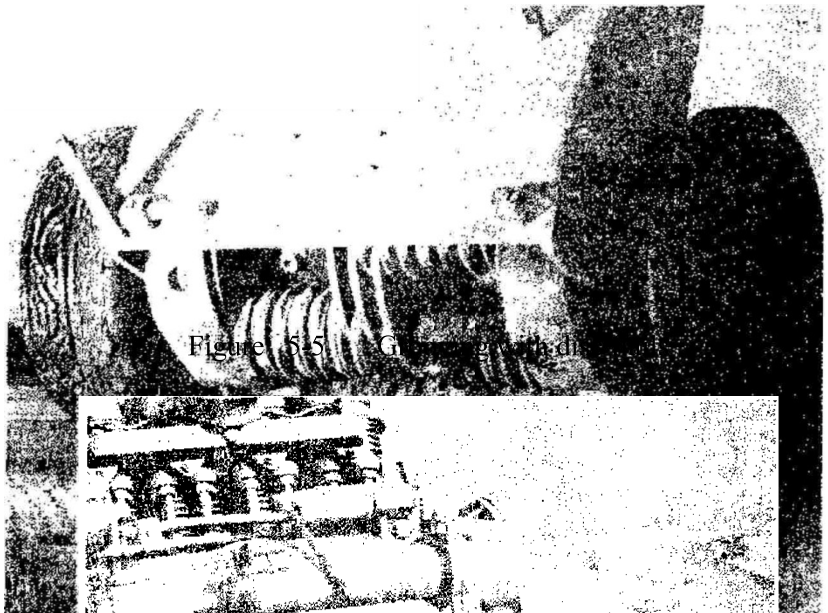


Figure 5-5. Grooving with di[illegible]

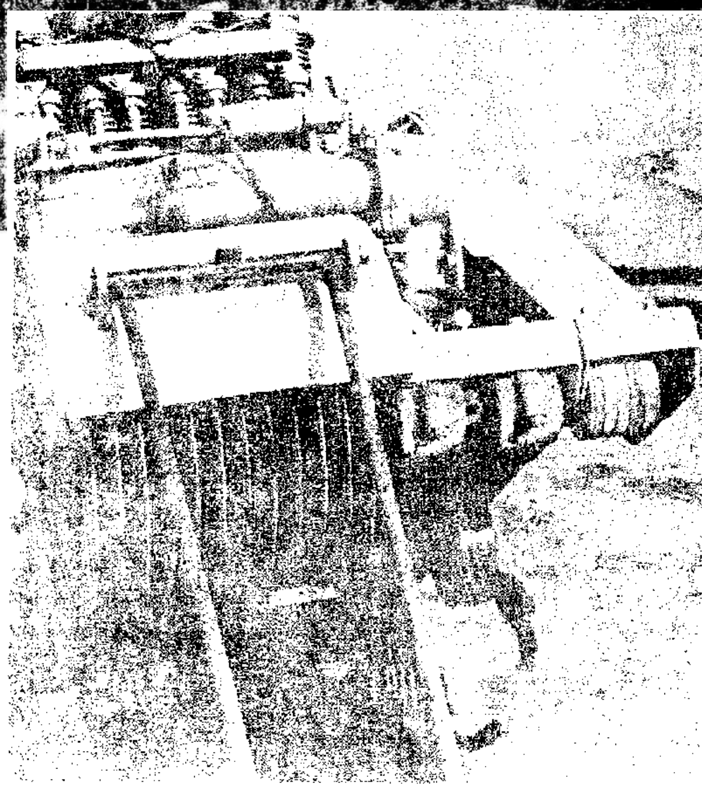


Figure 5-6. Grooving with saws

5.3.3.6. Technique. The surface is to be grooved across the runway at right angles to the runway edges or parallel to non-perpendicular transverse joints, where applicable, with grooves which follow across the runway in a continuous line without break. The machine for grooving will incorporate disc flails (Figure 5-5) or flail cutters or a sawing machine (Figure 5-6) incorporating a minimum of 12 blades. Sawing machines include water tanks and pressure sprays. Commonly used groove configurations are 3mm wi.de by 3 mm deep at approximately 25 mm centres, or 6 mm by 6 mm with a centre spacing of 31 mm.

5.3.3.7. The grooves may be terminated within 3 m of the runway pavement edge to allow adequate space for the operation of the grooving equipment. Tolerances should be established to define groove alignment, depth, width and spacing. Suggested tolerances are ± 40 mm in alignment for 22 m, and average depth or width ± 1. 5 mm. Grooves should not be cut closer than 75 mm to transverse joints. Diagonal or longitudinal saw kerfs where lighting cables are installed should be avoided. Grooves may be continued through longitudinal construction joints. Extreme care must be exercised when grooving near in-runway lighting fixtures and sub-surface wiring. A 60 cm easement on each side of the light fixture is recommended to avoid contact by the grooving machine. Contracts should specify the contractor's liability for damage to light fixtures and cable. Clean-up is extremely important and should be continuous throughout the grooving operation. The waste material collected during the grooving operation must be disposed of by flushing with water, sweeping, or vacuuming. If waste material is flushed, the specifications should state whether the airport owner or contractor is responsible for furnishing water for cleanup operations. Waste material collected during the grooving operation must not be allowed to enter the airport storm or sanitary sewer, as the material will eventually clog the system. Failure to remove the material can create conditions that will be hazardous to aircraft operations.

5.3.3.8. Plastic grooves and wire comb. Grooves can be constructed in new Portland cement concrete pavements while in the plastic condition . The "plastic grooving" or wire comb (see Figure 5-7) technique can be included as an integral part of the paving train operation. A test section should be constructed to demonstrate the performance of the plastic grooving or wire combing equipment and set a standard for acceptance of the complete product.

5.3.3.9. Technique. Tolerances for plastic grooving should be established to define groove alignment, depth, width, and spacing. Suggested tolerances are± 7 .5 mm in alignment for 22 m; minimum depth 3 mm, maximum depth 9.5 mm; minimum width3 mm, maximum width 9.5 mm; minimum spacing 28 mm, maximum spacing 50 mm centre to centre, Tolerances for wire combing should result in an average 3 mm x 3 mm x 12 mm configuration.

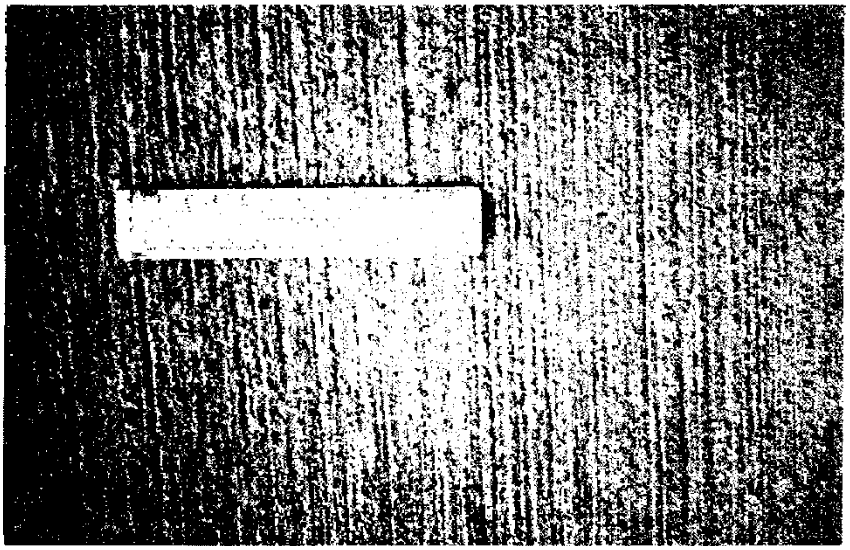


Figure 5-7. New concrete surfacing textured with wire comb

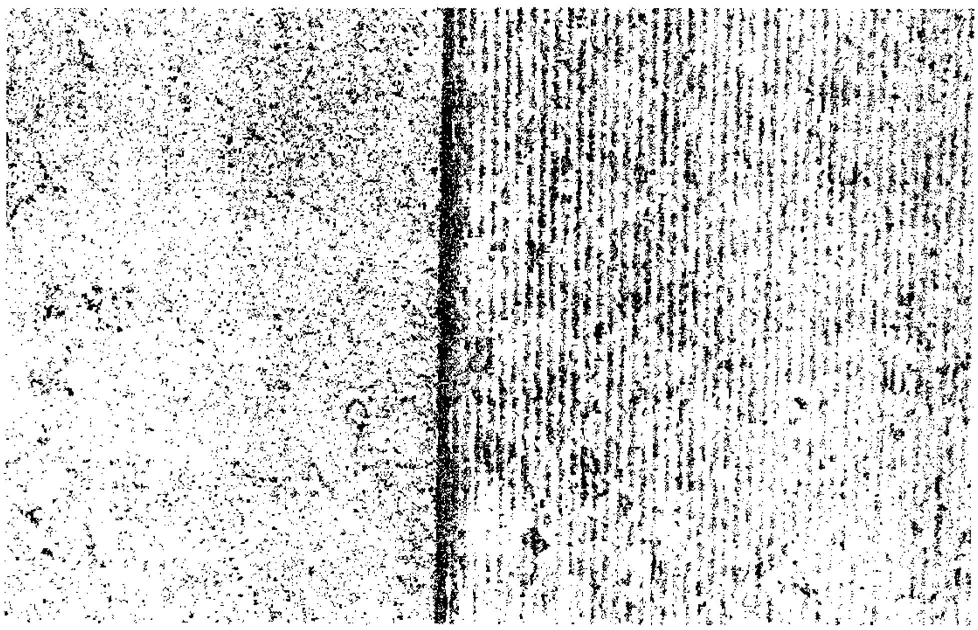


Figure 5-8. Existing Portland cement concrete before and after scoring

5.3.3.10. The junction of groove face and pavement surface should be squared or rounded or slightly chamfered. Hand-finishing tools, shaped to match the grooved surface, should be provided. The contractor should furnish a "bridge" for workmen to work from to repair any imperfect areas. The equipment should be designed and constructed so that it can be controlled to grade and be capable of producing the finish required. If pavement grinding is used to meet specified surface tolerances, it should be accomplished in a direction parallel to the, formed grooves.

<u>Grooving runway intersections</u>

5.3.3.11. General. Runway intersections require a decision as to which runway's continuous grooving is to be applied. The selection of the preferred runway will normally be dictated by surface drainage aspects, except that if this criterion does not favour either runway, consideration will be given to other relevant criteria.

5.3.3.12. Criteria. The main physical criterion is surface drainage. Where drainage characteristics are similar for the grooving pattern of either runway, consideration should be given to the following operational criteria:

- aircraft ground speed regime;
- touchdown area; and
- risk assessment.

5.3.3.13. Surface drainage. The primary purpose of grooving a runway surface is to enhance surface drainage. Hence, the preferred runway is the one on which grooves are aligned closest to the direction of the major down slope within the intersection area. The major down slope can be determined from a grade contour map.

5.3.3.14. The above aspect is essential because intersection areas involve, by design, rather flat grades (to satisfy the requirement to provide smooth transition to aircraft travelling at high speeds) and, therefore, are susceptible to water ponding.

5.3.3.15. Where appropriate, consideration may be given to additional drainage channels across the secondary runway where the groove pattern terminates in order to prevent water from this origin from affecting the intersection area.

5.3.3.16. Aircraft speed. Since grooving is particularly effective regarding wet surface friction characteristics in the high ground speed regime, preference should be given to that runway on which the higher ground speeds are frequently attained at the intersection.

5.3.3.17. Touchdown area. Provided the speed criterion does not apply, the runway on which the intersection forms part of the touchdown area should be preferred because grooving will provide rapid wheel spin-up on touchdown in particular when the surface is wet.

5.3.3.18. Risk assessments. Eventually, the selection of the primary runway can be based on an operational judgement of risks for overruns (rejected takeoff or landing) taking into account:

- runway use (take off/landing);
- runway lengths;
- available runway end safety areas;
- movement rates; and
- particular operating conditions.

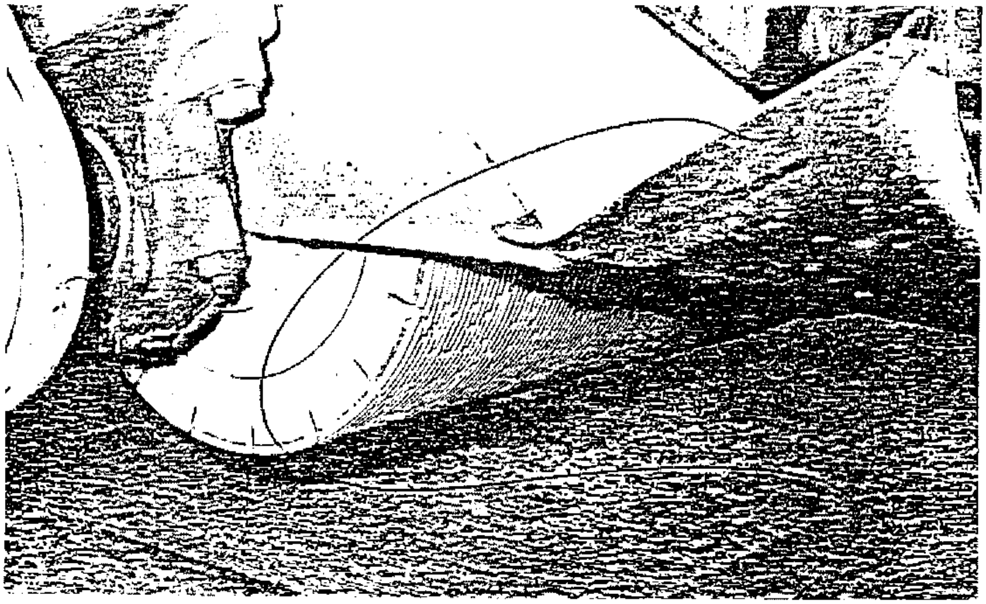


Figure 5-9. Scoring with diamond segmented cutting drum

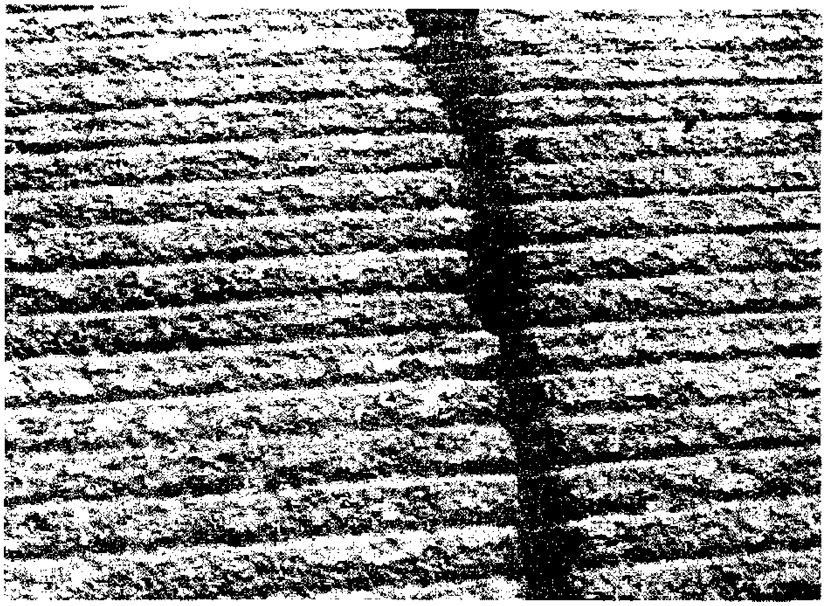


Figure 5-10. Reflex percussive technique - Portland cement concrete

5.3.4. Scoring of cement concrete

5.3.4.1. Operational considerations. There do not appear to be any operational objections to the scoring of existing Portland concrete surfaces (Figure 5-8), and this method of treatment seems to be suitable for all types of aircraft.

5.3.4.2. Consideration of existing pavement. It will be understood that it would be difficult to score uniformly concrete surfaces which are "rough". Pavements with damaged or poorly formed joints, or on which laitance has led to extensive spalling of the surface, would be equally difficult to score. If the existing surface is reasonably free of these defects, there are no other engineering limitations to scoring.

5.3.4.3. Effectiveness of treatment. Transverse scoring of concrete improves considerably the friction characteristics of pavements initially textured at the time of construction with bolts, burlap or brooms. The useful life of the treatment depends on the frequency of traffic but in general the scoring remains effective for the life of the concrete.

5.3.4.4. Runway ends. Runway ends should be left unscored to make it easier to wash down and clean off fuel and oil droppings. Moreover, engine blast can be more damaging on a scored than on an untexured surface. The directional control of an aircraft moving from the taxiway on to the runway can become reduced, presumably because of a tendency of the tires to track in the scores. In addition, a possibility of an increase in tire wear in turning cannot be totally discounted.

5.3.4.5. Technique. An acceptable "trial" area should be available for inspection and it is recommended that this be provided at the aerodrome to determine a precise texture depth requirement, as this will tend to vary with the quality of the concrete. The runway is to be scored transversely by a single pass of a cutting drum (Figure 5-9) incorporating not less than 50 circular segmented diamond saw blades per 30 cm width of drum. The drum is to be set at 3 mm setting on a multi-wheeled articulated frame with outrigger wheels, fixed to give a uniform depth of scoring over the entire surface of the runway to ensure the removal of all laitance and the exposure of the aggregate. It should be noted that scoring generates a great deal of dust during treatment and it is necessary to sweep and wash down the surface before operations re-start.

5.3.5. Reflex percussive technique

5.3.5.1. The reflex percussive technique is predominantly applied for grooving of existing runway surfaces and represents a cost-effective alternative to saw-cut grooving techniques. It has been successfully applied on various types of runway surfaces to provide adequate grooving. The technique can also effectively be used for other purposes, such as removal of rubber deposits in touchdown zone areas or for the restoration of micro/macrotexture of a degraded existing runway surface.

5.3.5.2. The reflex percussive technique uses star-shaped or pentagonal disk flails. The specification of the cross section and spacing of the grooves will be dictated primarily by the drainage requirements determined from local precipitation conditions and the slopes of the runway surface. For cement concrete surfaces, the pitch ranges normally from 42 mm to 48 mm and for asphalt surfaces from 42 mm to 56 mm, respectively. For either type of surface, however, local conditions may require closer spacings between two consecutive grooves to satisfy drainage demand, down to 32 mm. On the other hand, higher spacings are often used at runway ends where aircraft line up, in order to avoid high stresses on the treads of scrubbing aircraft ti.res. Typical cross sections for grooving cement concrete and asphalt surfaces are:

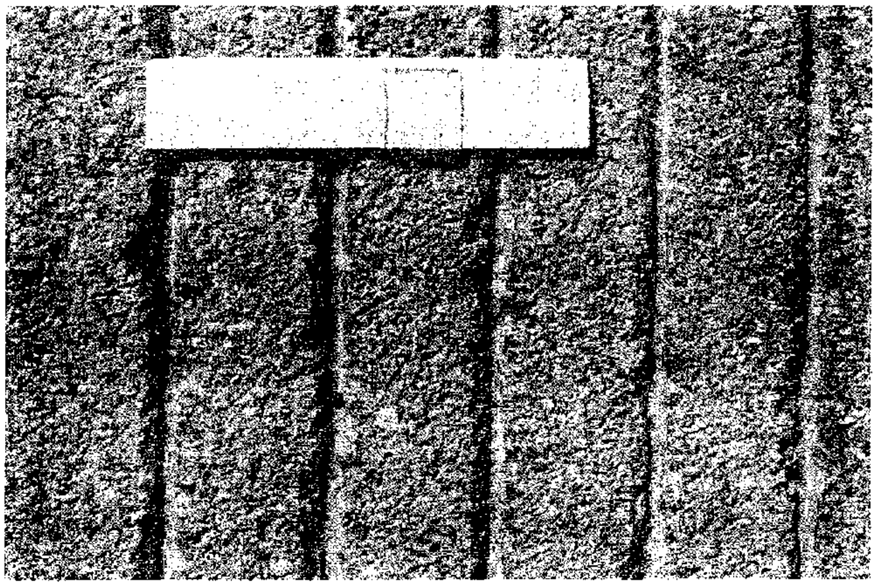


Figure 5-11. Reflex percussive technique - Asphalt surface

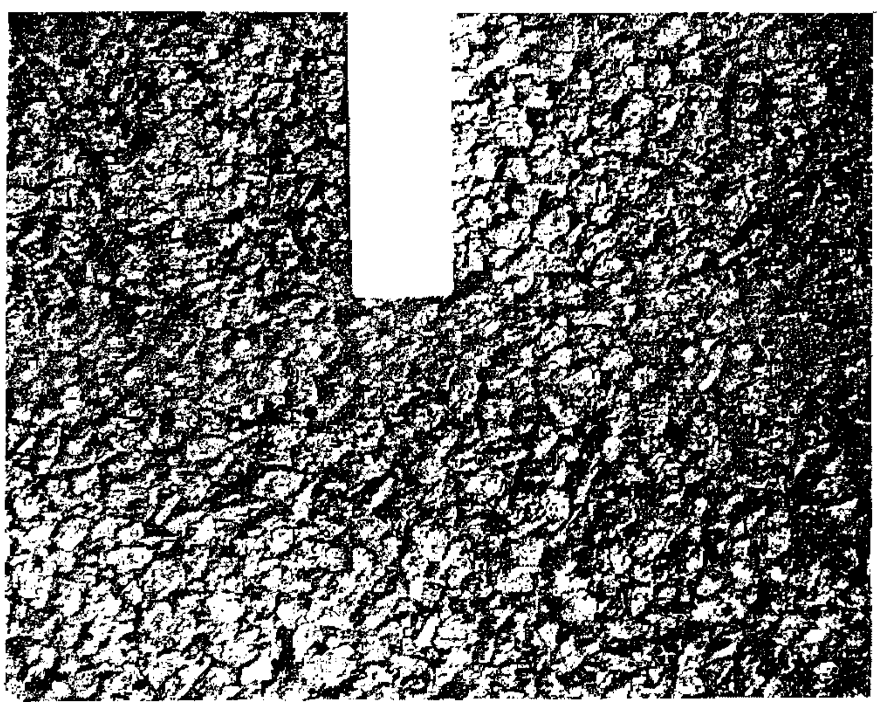


Figure 5-12. Porous friction course surfacing

Portland cement concrete:	Width/depth/pitch 10/3/27 mm, edges and trough rounded (see Figure 5-10)

Asphalt surface: Width/depth/pitch 9/3/58 mm,
edges and trough rounded (see Figure 5-11).

5.3.5.3. The surface of the Portland cement concrete or asphalt surface is to be grooved perpendicular to the runway centre line or parallel to non-perpendicular transverse joints, where applicable, in continuous uninterrupted lines terminating approximately 3 m before the edge of the runway. On concrete runways, a strip on both sides adjacent to each joint is to be left ungrooved to prevent weakening of the individual slab edges. After grooving, debris and all loose material are to be removed satisfactorily.

5.3.6. Porous friction course

5.3.6.1. The porous friction course consists of an open-graded, bituminous surface course composed of mineral aggregate and bituminous material, mixed in a central mixing plant, and placed on a prepared surface (Figure 5-12). This friction course is deliberately designed not only to improve the skid-resistance but to reduce aquaplaning incidence by providing a "honeycomb" material to ensure a quick drainage of water from the pavement surface direct to the underlying impervious asphalt. The porous friction course is able, because of its porosity and durability, to maintain over a long period a constant and relatively high wet friction value.

5.3.6.2. Limitations of porous friction course. Friction courses of this kind should only be laid on new runways of good shape, or on reshaped runways approaching the criteria expected for new runways. They must always be over densely graded impervious asphalt wearing courses of high stability. Both of these requirements are necessary to ensure a quick flow of the water below the friction course and over the impervious asphalt to the runway drainage channels.

5.3.6.3. Runway ends. The porous friction course is not recommended at the runway ends. Oi1 and fuel droppings would clog the interstices and soften the bitumen binder, and jet engine heat would soften the material which blast would then erode. Erosion would tend to be deeper than on a normal dense asphalt and the possibility of engine damage through ingestion of particles of runway material should not be discounted. Scuffing might occur in turning movements during the first few weeks after laying. For these reasons, it is recommended that runway ends be constructed of brushed or grooved concrete, or of a dense asphalt.

5.3.6.4. Aggregate. The aggregate consists of crushed stone, crushed gravel, or crushed slag with or without other inert finely divided mineral aggregate, The aggregate is composed of clean, sound, tough, durable particles, free from clay balls, organic matter, and other deleterious substances. The type and grade of bituminous material is to be based on geographical location and climatic conditions. The maximum mixing temperature and controlling specification is also to be specified.

5.3.6.5. Weather and seasonal limitations. The porous friction course is to be constructed only on a dry surface when the atmospheric temperature is 10° C and rising (at calm wind conditions) and when the weather is not foggy or rainy.

5.3.6.6. Preparation of existing surfaces. Rehabilitation of an existing pavement for the placement of a porous friction course includes: construction of bituminous overlay, joint sealing, crack repair, reconstruction of failed pavement and cleaning of grease, oil, and fuel spills. Immediately before placing the porous friction course, the underlying course is to be cleared of all loose or deleterious material with power blowers, power brooms, or hand brooms as directed. A tack coat is to be placed on those existing surfaces where a tack coat is necessary for bonding the porous friction course to the existing surface. If emulsified asphalt is used, placement of the porous friction course can be applied immediately. However, if cutback asphalt is used, placement of porous friction course must be delayed until the tack coat has properly aired.

5.3.7. Emulsified asphalt slurry seal

5.3.7.1. The emulsified asphalt slurry seal course consists of a mixture of emulsified asphalt, mineral aggregate, and water, properly proportioned, mixed, and spread evenly on a prepared underlying course of existing wearing course. The aggregate consists of sound and durable natural or manufactured sand, slag, crusher fines, crushed stone, or crushed stone and rock dust, or a combination thereof. The aggregate is to be clean and free from vegetable matter, dirt, dust, and other deleterious substances. The aggregate is to have a gradation within the limits shown below.

GRADATION OF AGGREGATES

Sieve Size	Percentage by weight passing sieves Type I	Type II	Type III
9.5 mm	---	100	100
4.75 mm	100	90-100	70-90
2.36 mm	90-100	65-90	45-70
1.18 mm	65-90	45-70	28-50
600 micro m	40-60	30-50	19-34
300 micro m	25-42	18-30	12-25
150 micro m	15-30	10-21	7-18
75 micro m	10-20	5-15	5-15
Residual asphalt content-percentage dry aggregate	10-16	7.5-13.5	6.5-12
Kilograms of aggregate per square metre	3.2-5.4	5.4-8.1	8.1-10.8

5.3.7.2. The Type I gradation is used for maximum crack penetration and is usually used in low density traffic areas where the primary objective is seali.ng, The Type 11 gradation is used to seal and improve skid resistance. The Type III gradation is used to correct surface conditions and provide skid resistance.

5.3.7.3. Mineral filler is only used if needed to improve the workability of the mix or to improve the gradation of the aggregate. The filler is considered as part of the blended aggregate.

5.3.7.4. Tack coat specified for the slurry. The tack coat is a diluted asphalt emulsion of the same type mix. The ratio of asphalt emulsion to water should be 1 to 3.

5.3.7.5. Weather limitations. The slurry seal is not applied if either the pavement or the air temperature is 13° C or below or when rain is imminent. Slurry placed at lower temperatures usually will not cure properly due to poor dehydration and poor asphalt coalescence.

5.3.7.6. Cleaning existing surface. Prior to placing the tack coat and slurry seal coat, unsatisfactory areas are to be repaired and the surface cleaned of dust, dirt, or other loose foreign matter, grease, oil, or any type of objectionable surface film, Any standard cleaning method is acceptable except that water flushing is permitted in areas where considerable cracks are present in the pavement surface. Any painted stripes or marking on the surface to be treated are to be removed before applying the tack coat. When the surface of the existing pavement or base is irregular or broken, it must be repaired or brought to uniform grade and cross section. Cracks wider than 10 mm must be sealed with compatible joint sealer prior to applying the slurry seal.

5.3.7.7. Application of bituminous tack coat. Following the preparation for sealing, application of the diluted emulsion tack coat is done by means of a pressure distributor in amounts between 0.23 to 0.68 L/m^2. The tack coat is to be applied at least two hours before the slurry seal, but within the same day.

5.3.7.8. The main items of design in emulsified asphalt slurry seals are aggregate gradation, emulsified asphalt content, and consistency of the mixture. The aggregates, emulsified asphalt, and water should form a creamy-textured slurry that, when spread, will flow in a wave ahead of the strike-off squeegee. This will allow the slurry to flow down into the cracks in the pavement and fill them before the strike-off passes over. The cured slurry is to have a homogeneous appearance, fill all cracks, adhere firmly to the surface, and have skid resistant texture.

CHAPTER 6: - PROTECTION OF ASPHALT PAVEMENTS

6.1. The problem

6.1.1. Since petroleum-base fuels and lubricants contain solvents for asphalt, their spillage on asphaltic pavements creates problems. Severity of problems is related to the degree of exposure to the penetrating solvents.

6.1.2. The highly volatile gasoline and high octane fuels of the past have been less of a problem since they quickly evaporated when spillage occurred and systems using these fuels have provided good containment. Massive and frequently repeated spillage can be a problem, of course, since such fuels are excellent solvents. Fuel spillage surfaced as a particular problem with the advent of turbine and jet engines. The kerosene and light oil jet fuels involved do not readily evaporate and early engine systems routinely spilled quantities of fuel on engine shutdown. Hydraulic fluids and lubricating oils, which evaporate or "cure out" even less rapidly than jet fuels, can also cause or contribute to problems.

6.1.3. Since the severity of adverse effects of spillage on asphalt pavements is related to exposure, concern must be for the number of times spillage is repeated in one location, the length of time the spilled fuel or oil remains on (or in) the pavement, and the location and extent of spillage on the pavement. It has been found that a single spillage of jet fuel, and even several spillages in the same location when there is time for evaporation and curing between spillages, do not normally have a significant adverse effect on the pavement. However, some staining and a tender pavement are to be expected during the curing period.

6.1.4. Spillages can result from routine operations such as engine shut-down, fuel tank sediment draining, consistent use of solvents for cleaning of engine or hydraulic system elements, etc. More commonly spillage is the result of fuel handling operations, of spilled oil or hydraulic fluid, or accumulated drippings from engine oil leakage or mishandling.

6.1.5. Thus locations of concern on pavements are those where aircraft are regularly fuelled, parked, or serviced. The broad areas of landing and taxiing operations will not be of concern, since even spillages attendant to aircraft accidents will be minimized by clean-up and represent only a single spillage which will cure without permanent damage. Even fuel burned on the asphalt surface will normally only leave a surface scar of no structural significance.

6.1.6. In areas where spillage occurs repeatedly or spilled fuel or oil remains for long periods on the pavement the solvent action softens the asphalt and reduces adhesion to the surface aggregate. While heat from the sun or warm air conditions help evaporate solvents and re-cure the asphalt, the elevated temperatures contribute to the asphalt softening. The result of the spillage, aggravated by heat, can be shoving of the asphalt mix, tire tread printing, tracking of asphalt to adjacent areas or production of loose material, and pavement abrasion also producing loose material on the pavement surface. In maintenance and work areas asphalt and grit picked up by tools, shoes, and clothing can be transferred to mechanical systems.

6.1.7. The surface texture and condition of pavements have a bearing on the severity of the problem. Open or porous pavements will be more readily penetrated by fuel or oil and will slow the evaporation and re-cure process. It has been found that rubber tire traffic, whether from rolling or traffic tends to close the surface and retard fuel penetration. Cracks and joints, not well sealed, are a particular source of trouble. These provide access for fuel to deeper zones within the pavement, provide greater surface areas for fuel intake, and retain fuel much longer thereby retarding evaporation and cure. Low areas which will retain or pond fluids, whether adjacent to cracks or joints or in central areas of pavement, will prolong exposure to spilled fuel.

6.2. Treatment of the problem

6.2.1. The best treatment is avoidance of spillage and this may be possible in many cases of operational spillage and some accidental spillage. Fuel tank sediment drainage can be caught and need not be allowed on the pavement. Drip pans can be used for oil drip locations and for bleeding or servicing of hydraulic systems. Trays may be practical to catch engine shut-down spillage or small quantities of refueling spillage.

6.2.2. Removal of the spilled fuel or oil and reduction of exposure through clean up is the next aspect of treatment. Spilled fuel or oil can be flushed off the pavement with water. Addition of detergents assists the process of separating the fuel and especially oil from the asphalt pavement. While this has been a common treatment there are beginning to be environmental complaints from effects of the run-off. A vacuuming process, with suitable equipment, can be used to remove spilled fuel and some fuel recovery is possible. Absorbent materials can also be used for fuel and oil pickup with suitable arrangement for disposal. Rolls, pads, and granular materials are all used and in some cases wringers are used for fuel recovery. There is another aspect of absorption by granular materials in spillage areas to consider. Accumulations of dust and sand, either blown or man placed, will absorb small spillages, oil drippings, etc., and form a mat which contains the spilled material and reduces its availability for soiling of personnel and equipment. While this temporarily facilitates movement of personnel it can greatly increase exposure of the pavement to effects of the fuel and oil.

6.2.3. Since problems are aggravated by repeated exposure to spillage, it is sometimes possible to relocate aircraft parking, fuelling, or servicing positions to ameliorate the deterioration.

6.2.4. Spillage problems cannot develop if spilled fuel or oil is not allowed to come in contact with the asphalt pavement. Protective coatings have accordingly been developed to provide a barrier between the fuel or oil and the pavement, which is then not affected by the spilled fuel or oil.

6.3. Protective coatings

6.3.1. Protective coating materials are generally liquids, some heated to become liquid, which when spread on the pavement cure or set to become a protective coating. These are commonly referred to as seal coats when common spray application and bituminous materials are involved. Most of the liquid materials can be applied in any of several ways including spraying using hand sprays or asphalt distributor equipment, pouring on the surface and spreading using squeezes, rolling onto the surface with paint rollers, and application or spreading using brushes. Single and multiple application are variously employed, and fine aggregate may be spread and embedded in the coating before setting or curing to improve wear or skid resistance.

6.3.2. Coating materials in emulsion form can be extended and premixed with fine aggregate to form a slurry and applied as a slurry seal.* Single or multiple applications can be used here also. Two layer applications are common.

6.3.3. Thin overlays of materials not affected by spillage can be applied to protect asphalt pavements. Conventional construction methods are applicable unless some very unconventional materials are employed.

6.4. Materials for protective coatings

6.4.1. Coal-tar pitch is only slightly soluble to insoluble in the light petroleum fractions (napthas) which are solvents for asphalts and can be employed, in much the same way as is asphalt, in pavement applications. Also, in many places, depending upon relative availability and economic circumstances, tar has been cost competitive with asphalt for spray applications and as a binder for pavements. Thus coal-tar pitch is used as a protective sealer**and is the basic ingredient in various commercially offered sealers for protective coating applications.

6.4.2. Because tar is more temperature sensitive than asphalt, means of adjusting the temperature response to one similar to asphalt were studied. It was found that addition of latex rubber would accomplish this purpose and it was subsequently found that the rubberized tar (commonly called tar-rubber) gave a somewhat better performance than unmodified tar. For these reasons the most favoured and some of the best performing protective coatings are rubberized coal-tar pitch emulsions. The United States FAA Engineering Brief No. 22, "Asphalt Rubber and Rubberized Coal Tar Pitch Emulsion" presents comments and a guide specification for "Rubberized Coal Tar Pitch Emulsion Seal Coat (For Bituminous Pavements)" which is representative of material quantities and characteristics as well as application methods which apply, In the United States the rubberized coal tar pitch emulsion costs two to three times as much as asphalt emulsion.

* ASTM D-3910 Standard Practice of Design, Testing, and Construction of Slurry Seal.
** ASTM D-3423 Standard Practice for Application of Emulsified Coal-Tar Pitch (Mineral Colloid Type)

6.4.3. Sealing materials are offered which employ epoxies and polymers of various types either alone or in a bituminous base, which can be tar or asphalt. While these have attributes which should make them effective, experience with their application in the field is limited. Therefore trial test applications are recommended to help assess effectiveness before broad applications are undertaken. These materials range in price in the United States up to 20 times that of liquid asphalts.

6.4.4. Tar-rubber binder materials and, in at least one instance, epoxy-asphalt binder of a type used for bridge deck protection, have been placed as overlays of asphalt pavements to provide protection from fuel spillage along with structural upgrading. These are effective so long as cracking can be controlled (prevented orcracks kept sealed), Cost of the tar-rubber binder is perhaps twice the cost of asphaltmix while the epoxy-asphalt may run to five times the cost of asphalt mix but can be placed as a very thin (20 mm) overlay.

6.5. Application

6.5.1. Surfaces to receive protective coatings must be thoroughly cleaned. Any surface films of oil need to be carefully removed. Areas of pavement which have become affected by prior fuel spillage and any badly cracked areas must be removed and replaced with sound pavement and these patches should be thoroughly cured (2 to 4 weeks) prior to the sealing. All but very narrow cracks must be cleaned arid filled with crack filler.

6.5.2. Methods of application should follow standard practice as recommended by airfield or highway authorities, trade associations, or the product manufacturer. Seal coat guidance can be found in ASTM D-3423 or the United States FAA Engineering Brief No.22, Appendix B. Slurry seal guidance will be found in ASTM D-3410.

6.5.3. Commonly, single applications of seal or slurry seal are such as to provide O. 3 to O. 5 kg/m2 of residual bitumen. Two and even three applications are usual. Surfaces should be moist but not wet for emulsion applications and temperatures should be favourable both for application and subsequent cure - 10°C to 27°C is desirable. A lower limit is 7°C and favourable temperatures should continue at least 4 hours after placement. Epoxy and polymeric seals should be applied and cured as recommended for the individual material, but commonly application rates are 0.3 to 0.4 kg/m2.

6.6. Protection gained

6.6.1. Durability and wear can vary with the materials and applications, the surface cleaning and preparation, maintenance of the protective coating, and of course exposure to spillage and traffic. Testing and experience have shown that good coatings, well applied to clean well prepared surfaces and properly maintained, will provide satisfactory protection in most cases. In areas of very severe exposure, as at central fuelling points, no protective coatings have been found to be entirely satisfactory.

6.6.2. In other than the most severe spillage locations unsatisfactory behaviour can be experienced when elements of good practice are ignored. Some material formulations and application methods, either individually or in concert, can result in imperfect coverage by the seal coating. Bubbles can exist at application (sometimes called fish eyes) and leave holes in the coating or bubbles can form beneath a coating after cure and on breaking leave holes, and coatings can shrink and crack. Improper surface cleaning can result in a poor bond and peeling of the coating. And cracks in the coated pavement will tend to come through the protective surface coating.

6.6.3. When fuel can gain access through holes or cracks in the seal coat, through peeled areas, or through cracks reflected from the lower pavement, or when fuel saturated pavement has not been removed and is covered by the seal coat, conditions are worsened rather than improved by the seal since, in addition to not preventing access of the spilled fuel or oil to the asphalt, the seal coat greatly inhibits the evaporation and cure-out of the spillage.

6.6.4. Overlays of tar-rubber binder give spillage protection and are not subject to bubble holes, peeling, or wear through. Tar-rubber overlays are subject to shrinkage, cracking and to crack reflection from underlying pavements. They must be properly compacted since pavements having voids of as much as 6 per cent will be porous enough to permit penetration of jet fuel.

6.7. Maintenance consideration

6.7.1. Maintenance includes clean-up of spills as discussed earlier under "treatment of the problem". Ponding must be prevented to avoid extending exposure from spillage. Other maintenance is concerned with maintaining integrity of the protective coating. Cracks must be kept sealed with a fuel resistant sealer. Retreatment must be employed when deterioration, wear through, or peeling leads to openings in the coating. Accidental scars must be closed. If asphalt patching is required then the surface, after suitable cure, needs to be coated against spillage effects.

6.8. Some related concerns

6.8.1. Some seal coats provide reduced skid resistance, and while fuel resistant coatings are not commonly employed on aerodromes in areas of severe skidding potential, the problem, should it intrude, can be treated through embedment of sand size aggregate in the seal coat before final cure.

6.8.2. As earlier mentioned there is developing concern for the flushing of spilled fuel and oil, and of chemicals employed to assist the removal of oils, into adjacent drains. Catchments and acceptable disposal practices may be required.

6.8.3. Spilled fuel which finds its way into subsurface drains and culverts can be a safety hazard. Such spillage can develop explosive fuel-air mixtures in the confined drains and a spark ignition will result in an explosion. The risk to life and property can be real and consequential.

6.8.4. There can be a question as to the desirability of rolling seal coats. Rolling can improve film adhesion, and, as earlier mentioned, close surface pores and reduce fuel penetration. Generally, therefore, rolling of bituminous seals using flat (no tread) rubber tire rollers should be beneficial, but whether the resulting improvement warrants the rolling effort has not been established. Steel wheel rolling would not be of benefit and may damage the coating. Any rolling of polymeric seals might be undesirable, and supplier recommendations should be followed.

CHAPTER 7: - STRUCTURAL CONCERNS FOR CULVERTS AND BRIDGES

7.1. Problem description

7.1.1. Subsurface structures for drainage or access must commonly be crossed by pavements which support aircraft. Such facilities are subject to the added loading imposed by the aircraft sometimes directly as in the case of bridges, subsurface terminal facilities, and the like, but more often indirectly as loading transmitted to buried pipes and culverts through the soil layer beneath the pavement.

7.1.2. These subsurface structures must be considered in connexion with evaluation of pavement strength. The patterns of stresses induced by surface wheel loads as they are transmitted downward are not the same on the subsurface structures as on the subgrade. This is not only because these structures are not at subgrade level but also because the presence of the structure distorts the patterns. Thus the considerations which permit use of the ACN-PCN method to limit pavement overloading are not necessarily adequate to protect subsurface structures. In some cases the subsurface structure can be the critical or limiting element thereby necessitating the reporting of a lower PCN for the pavement.

7.1.3. In the design of new facilities care must be given to the structural adequacy of pipes, culverts, and bridged crossings, not only for the contemplated design loadings but for possible future loadings to avoid a need for very costly corrective treatments made necessary by a growth in aircraft loadings.

7.2. Types of substructures

7.2.1. Probably the most common and least apparent buried structures at aerodromes are pipes facilitating drainage of surface or subsurface water. These can range in diameter from 100 mm to 4 or 5 m and in cover depth from 300 mm to 50 m and more in the case of high embankments, and they can be quite stiff in relation to the surrounding soil (rigid pipe) or quite easily deformed by vertical loading (flexible pipe). The most common rigid pipe is made of reinforced cement concrete but there are also pipes made of plain cement concrete or clay. The latter pipes are of necessity smaller in diameter. The most common flexible pipe is of corrugated steel but there are also corrugated aluminium pipes, several types of plastic pipes, bituminized fibre pipes and others. Pipe installations are designed taking into account such factors as the pipe type, the bedding, backfill, installation materials and conditions, the embankment depth and the load imposed by it, and surface live loads to be sustained.

7.2.2. Box culverts which are either square or rectangular in shape are commonly used for stream crossings beneath pavements. They are designed for the hydraulic flow and the loads to be supported. They are usually of cast in situ reinforced cement concrete. Span between side walls can vary from about 1 to 5 m. Smaller box drains are often used in wide apron areas directly beneath pavements as surface flow collectors.

7.2.3. Arches of structural metal plates, of the type used for constructing large diameter pipes are sometimes used in preference to short bridges to span stream or pavement crossings. In such cases, soil is placed beside and above the arch up to subgrade level and the pavement constructed thereon. In rare cases tunnels may pass beneath aerodrome pavements.

7.2.4. Bridges are used in a number of cases for highways to pass beneath taxiways and runways and, increasingly, subsurface terminal facilities are placed beneath aprons and taxiways. These are designed to support the using aircraft and structure dead loads. Also runway extensions over water are sometimes placed on bridges supported on piles and these must be designed to accommodate aircraft loads in addition to their dead weight.

7.3. Some Guiding Concepts

7.3.1. The discussion in Chapter 3, 3.2.4, on Aircraft Loading is pertinent to concepts of distribution of stresses from surface loads within embankments beneath pavements. High stress surface loads are distributed by the pavement structure and as the loads extend downward they are further distributed over wider areas with consequent reduction in stress magnitudes. As the pattern of stress goes deeper and extends over wider areas, the effects of adjacent wheels overlap leading to doubling or even greater multiplying of the stress induced by one wheel, The deeper the pattern extends, the farther apart individual wheels can be and still have interacting effects. These are the patterns of stresses introduced by the live loads (aircraft) into the ground beneath pavements, and along with the mass of the soil and pavement, represent the magnitudes of stresses or loading delivered to buried structures.

7.3.2. The presence of a buried structure (which does not act in the same manner as the soil it displaces) has a significant impact on the pattern of live and dead load stresses (ambient stresses) induced by the surface loads, pavement and backfill material. A concrete pipe, for instance, is much stiffer in the vertical direction than is the adjacent soil. Thus compression (vertical deflexion) of the soil under aircraft loading results in a relative upward thrust of the rigid pipe into the soil with a con• sequent accumulation of greater than ambient stress and loading, This is why some deep• ly buried rigid pipes are protected by soft (baled straw, loose soil, etc,) zones above the pipe, In such cases, the vertical stiffness of the pipe and soft zone is less than the stiffness of soil beside the pipe and stresses are accumulated more by the adjacent soil. This is also why the character and condition of bedding and backfill are very important.

7.3.3. Box culverts accumulate stresses in the same way as rigid pipes but the impact on the structure is not the same. The vertical sidewalls of box culverts while much stiffer than the soil are far stronger than necessary to sustain the accumulated stresses or loading, and the span between sidewalls is less stiff than the sidewalls and subject to reduced stress. It should be noted that these reductions are small, however, and are reduced from the higher stresses accumulated on the stiff box culvert.

7.3.4. Metal and other flexible pipes are generally less stiff vertically than adjacent soil and not subject to stress accumulations in the manner of rigid pipes. However, metal pipes are very stiff in circumference and some larger diameter pipes with deep corrugations and located near the surface can accumulate more than ambient loading. Large metal arches with fixed footings can also be relatively stiff structures.

7.4. Evaluation of subsurface structures

7.4.1. General

7.4.1.1. Every subsurface structure beneath a pavement must be considered in connexion with evaluation of the pavement. And while specific determinations would in each case require careful structural analysis, the likelihood that a particular structure would prove more critical than the pavement in limiting aircraft loads depends greatly on the type, size, and location of the structure. Accordingly, certain guidance can be suggested to assist in determining which structures can, at small risk, be considered not to be limiting, which ones are marginal and need to be carefully considered, and which require study and analysis to define load limitations or needed strengthening.

7.4.2. Deeply buried structures

7.4.2.1. The live load on deeply buried structures tends to be only a small fraction of the dead load so that pipes or culverts of moderate size and smaller, which do not accumulate an undue share of the live load, will not limit surface loadings. This will include pipe diameters or structure spans up to about one-third of the protective cover (distance between pavement surface and top of pipe or culvert). Table7-1 indicates the thickness of protective cover of soil and pavement structure above drainage structures of not too large span which will spread the load sufficiently, considering combining of effects from adjacent wheels, to reduce the pressure induced on the structure by aircraft (live) loads to less than 10 per cent of the earth (dead)load. It is not likely that an added 10 per cent of pressure will exceed the structural capacity of in-service pipes or culverts, Where aircraft to be supported have tire loads greater than 200 kN somewhat greater cover depths may be needed to attain the 10 per cent limitation on increased (live load) pressure.Table7-1. Protective cover needed over structures beneath aerodrome pavements.

Number of wheels*	Cover depth in metres
1	4
2	5
4	6
8	7.5
16	9.5

*Consider all wheels within or touching a circle whose diameter equals the depth of protective cover over the structure.

Pipes and culverts of the sizes indicated (about one-third of the depth of cover) and at depths equal to or greater than that shown in Table 7-1 should not require a separate load limitation of the overlying pavement.

7.4.2.2. Structures at shallower depths need more detailed examination. Whether load limitations beyond those for protection of the pavement may be needed will depend on rigidity of the pipe or culvert, bedding and backfill, pavement structure, and conservatism of the original design. Sufficient analysis should be made either to confirm that the buried structure does not require a more critical load limitation than the pavement or to establish appropriate load limitations.

7.4.2.3. Wide span structures; i.e., very large pipes, arches, and wide box culverts, even with substantial cover will tend to accumulate stress from surface loads (by soil arching) and may have to support virtually all of the aircraft (live) load as well as the earth (dead) load. Thus any structure whose span exceeds about one-third of the cover depth should be carefully analysed to establish surface load limits or possible need for strengthening,

7.4.3. Shallow pipes, conduits, subdrains, and culverts

7.4.3.1. The ACN-PCN method limits aircraft mass to prevent over-stress of the pavement subgrade and overlying layers. These same limits tend to protect shallow buried structures from over-stress, except for quite large (over 3 or 4 m diameter or span) structures, which may accumulate load on the same critical section from more than one landing gear leg. Beneath rigid pavements a minimum cover of about one-half metre between the slab and structure is commonly considered to provide adequate protection from any loading. Pipes and culverts beneath flexible pavements will be protected when their top surface (outer crown of pipe) is within about one-half metre of the top of the subgrade. At greater depths, while stresses from surface wheel loads or combined effects of several wheel loads attenuate and are less than the pavement subgrade can accept, the combined effect (stress) and for an aircraft multiple wheel load, though ACN-PCN limited, may be greater than were considered in the original pavement design. Therefore pipes, drains, culverts, etc., should be carefully examined for possible need for strengthening when the individual wheel load or the number of wheels of the using aircraft are expected to be increased.

7.4.3.2. Shallow structures of substantial span (over 3 or 4 m) will need analysis in connexion with any contemplated increases in wheel loads or gross aircraft masses.

7.4.4. At surface drains, conduits, and the like

7.4.4.1. Collector drains, box conduits (for lighting, wiring, fuel lines, etc.), and any similar pavement crossing installations, are sometimes placed directly at the pavement surface. These would rarely be so large that more than a single wheel would need to be supported by the installation at any time. Consequently, only single wheel loadings need be of concern for the design as well as evaluation.

7.4.5. Bridges supporting aerodrome pavements

7.4.5.1. Need for passage of highway and rail traffic beneath aerodrome pavements and the placement of terminal connexions and facilities beneath taxiway and apron pavements has required the use of bridges to support the pavements and using aircraft. Such structures receive little if any protection from pavement load limitations and must be separately considered in establishing safe loadings. The original design analyses will have established the type and magnitude of loads for which the bridges are adequate. If the intended usage has changed and pavements are likely to be used by markedly heavier aircraft or aircraft with different undercarriage configuration than considered in design, a new analysis will be needed to establish the suitability of the structure for such usage.

7.4.6. Pile supported structures

7.4.6.1. Sometimes runways and taxiways extend over water and these are placed on pile supported structures. These, as for bridges, will have been subject to design analyses to provide for the contemplated loads. Here again there will be a need for re-analysis if operations by heavier aircraft or aircraft with substantially different undercarriage layout are contemplated.

7.4.7. Tunnels under pavements

7.4.7.1. Tunnels behave in a manner similar to large diameter pipes and can be considered to respond in much the same manner. Thus shallower tunnels would require careful analysis of expected increased aircraft loads on overlying pavements. Deeply buried tunnels might require only casual examination if cover depths were sufficient to minimize induced live loads.

7.4.8. Treatment of severely limiting cases

7.4.8.1. Where structures beneath pavements limit aircraft loads beyond the PCN (which is assessed to protect the pavement) these limitations will need to be reported in terms of specific aircraft type and load (mass) as exceptions. Where multiple taxiways permit avoidance of the critical structures the problem can be handled by local routing of aircraft. If, however, all aircraft must cross the critical structure the limitation must be emphasized when reporting pavement strengths. Only very shallow structures and extreme overloading - except for bridges or pile supported pavements represent some hazard to aircraft, and aircraft safety will rarely if ever be compromised by overload of buried (earth covered) structures. Bridges and pile supported pavements receive the loading directly and must be structurally capable of supporting the imposed loadings.

7.4.8.2. Load limitations on critical structures can be eliminated either by special analyses which establish that larger than intended design loadings can be sustained, or by strengthening. Commonly, design conservatism, better-than-minimum installation, larger-than-needed safety factors and more searching design type analyses may result in larger allowable loadings. These can range from a simple restudy of the design data to extensive field study of the installation including study of surrounding backfill or measurement of strain or deflexion response of the structure under load. An example of such a study can be found in the April 1973 issue of Airport World under the title, "New Bridge or No ? ". This is a publication of the United States Aircraft Owners and Pilots Association and the article deals with a study undertaken in the 1970s to assess the suitability of an existing bridge at Chicago O'Hare International Airport for use by wide bodied aircraft.

7.4.8.3. The strengthening of a substructure can be accomplished using internal bands, struts, or liners to strengthen or reduce span in pipes, culverts, arches, e t c; , but these reduce the designed drainage capacity. Sometimes structures can be stiffened by grouting surrounding soil from the surface or from inside the structure. It may be possible to introduce compressible zones of soil or other material above pipes or culverts and reduce the transmission of pavement loads to the buried structure. Also, provision of load distributing pavement structures (buried slabs for instance) may reduce loads on pipes, culverts or drains. Of course, re-design and reconstruction is the obvious ultimate solution. Some bridges or pile-supported pavements may be strengthened by adding elements (beams, etc.) to the existing structure.

7.5. Considerations in design of new facilities

7.5.1. Structural concerns for drainage and similar structures in relation to the evaluation of pavements for load support capacity have been discussed earlier in this chapter. Patterns of behaviour in connexion with size, flexibility, live and dead loads, deep and shallow cover have been indicated, and these apply also to design considerations where new facilities are planned. This section will amplify some of the earlier discussions and treat aspects of structural behaviour of somewhat more direct concern for design.

7.5.2. Loads. Loads which must be considered in design of buried structures are those resulting from the weight of overlying soil and pavement structure (overburden) plus those induced by aircraft or other vehicles on the pavement above. Heavy construction loads passing over pipe before it has its full protective cover may also need to be considered. These loads produce the patterns of ambient stress present in embankments where they are not disrupted by the presence of pipe or other structures or by the pockets of loose, dense or other types of soil introduced by the installation of pipes, culverts, etc. It is the distortion of the ambient stress patterns by the character of the pipe or structure, the nature of the pipe bedding, any trench used during installation, and the type and compacted density of the backfill around the pipe which leads to larger or smaller than ambient stress loads on the buried structures. This too is what complicates the design problem and leads to established design methods which provide only nominal guidance.

7.5.3. Ambient overburden stresses are the result of the mass of overlying soil and pavement structure and can be directly determined. Stresses induced by aircraft tire loads can be calculated using the theory for a uniformly distributed circular load on the surface of a continuum. The theory for an elastic layered continuum, with suit able elastic constants (E, μ), should be preferred, but the theory for a single layer system (Boussinesq) will provide reasonable stress determinations for flexible pavements and deeper installations beneath rigid pavements. Plots or tabulations of single layer stresses can be found in references such as: the 1954 Highway Research Board Proceedings, HRB Bulletin 342 of 1962, Yoder's textbook on "Principles of Pavement Design" (United States), Croney's text "The Design and Performance of Road Pavements" TRRL (United Kingdom). Stresses for the combined effects of several wheels can be determined by superposition of the single wheel stresses at pertinent lateral spacing. Because of the time rate of response of soil to rapid loading it is not necessary to consider any added dynamic effects of the aircraft loading.

7.5.4. The ambient stresses which obtain at the various depths beneath the pavement are thus a combination of the overburden (dead load) stresses and the aircraft landing gear load (live load) stresses. It is these stresses modified by the existence and behaviour of a pipe or other buried structure and any distortions due to its installation that determine the loads which must be supported by the pipe or structure. In general, hard (stiff) elements or zones will accumulate stress from the adjacent embankment soil while soft elements or zones will shed stress to the adjacent soil. Thus the more rigid structures, such as box culverts, concrete pipe, and the like, will tend to be subject to greater stress and load than that implied by the ambient stress, while more flexible structures, such as steel, aluminum, and plastic pipe or rigid structures provided with an overlying zone of loose soil, straw, sawdust, etc. will tend to be subject to less than the ambient stress.

7.5.5. A most important consideration in the determination of loadings for design of buried structures is in providing for future upgrading of pavement facilities and growth in aircraft masses supported. Where upgrading is likely in the future the design of buried structures beneath pavements for the heavier loadings expected will commonly be far less costly during the original design and construction than when left for subsequent modification.

7.5.6. Pipes. Pipes are described generally in 7.2.1 and most types are covered by ASTM standards for the pipe characteristics and tests to determine pipe strength. Concrete, clay, asbestos-cement, solid wall plastic, and other geometrically similar types of pipe are made in a variety of wall thicknesses and/or reinforcements, as well as diameters to provide an array of strengths for use in design of installations. Steel, aluminum, and some plastic pipes are made in a variety of gauges (thicknesses of material) and corrugation configurations to provide an array of pipe stiffnesses and side-wall strengths for installation design purposes. While round pipes are most common there are elliptical pipes - used vertically for increased strength or horizontally for low head - and pipe arches having rounded crown and flattened invert for special application as access ways, utility ducts, etc.

7.5.7. Design limitations for rigid pipe are commonly established to control the progression of cracking at the crown and invert. Prevention of cracks wider than 0.4 mm is the usual practice. Earlier practice for flexible pipe installation design was to limit pipe deflection to 5 per cent of the pipe diameter, but current practice prefers to require competent backfill soil compaction (85 per cent of Standard Density ASTM D-698) and limit the buckling in ring compression.

7.5.8. Installation conditions. Bedding, backfill, and trench conditions of pipe installation can have significant effect on performance. Pipe can be placed on flat compacted earth, on a 60°, 90°, or 120° shaped bed, on a sand or fine gravel cushion, in a lean or competent concrete cradle, etc. Pipe can be placed in a narrow or wide trench, shallow or deep trench, vertical or sloping sidewall trench, or no trench. Backfill can be poorly compacted beneath (haunches) or beside the pipe and can be the same as adjacent embankment material or a select sand, gravel, or other superior material, or it can be a stabilized (cement or lime) soil. Rigid pipe can be insulated from its normal accumulation of greater than ambient stress by placing a soft zone of loose soil, straw, foamed plastic, leaves, or similar material above the pipe. All of these many variables can have an impact on the design loads to be considered.

7.5.9. Design. Because of the many variables in loading, pipe characteristics, and installation conditions design concepts, methods, and supporting methods for characterizing behaviour of materials are beyond what can be presented here. Design details can be found in some geotechnical textbooks, such as "Soil Mechanics" by Krynine (United States), “Soil Engineering" by Spangler (United States) and in trade literature, such as "Concrete Pipe Design Manual" of the American Concrete Pipe Association (United States Library of Congress Catalog No. 78-58624), "Handbook of Steel Drainage and Highway Construction Products" of the American Iron and Steel Institute (United States Library of Congress Catalog No. 78-174344) and in the many references to technical literature contained in these documents. Some specific design guidance for minimum protective cover beneath flexible or rigid pavement for several types of pipe recomputed based on selected (common) installation conditions can be found in the United States FAA manual on "Airport Drainage" AC 150/5320-5B, as well as in the two trade literature manuals referenced above.

7.5.10. Other structures. Design of bridges and pile supported extensions over water, which support aircraft loads directly, must follow accepted structural design practice. It will be most important to anticipate future aircraft growth loads to avoid very costly subsequent strengthening. Box culverts will be subject to the ambient stresses (7.5.3) increased by the up thrust of such stiff structures into the overlying embankment (7.5.4). The resulting load should be determined by careful analysis, but should fall between about 130 per cent and 170 per cent of the load due only to ambient stress depending upon span of the structure, magnitude and extent of surface load, protective cover depth, and soil stiffness adjacent to the culvert. Any large corrugated metal arches (over 5 m) with shallow soil cover should be subjected to careful geo-technical and structural design. Each will be a separate case and of a magnitude to warrant careful design analysis.

CHAPTER 8: - CONSTRUCTION OF ASPHALTIC OVERLAYS

8.1. Introduction

8.1.1. The volume and frequency of operations at many airports makes it virtually mandatory to overlay (resurface) runways portion by portion so that they may be returned to operational status during peak hours. The purpose of this chapter is to detail the procedures to be used by those associated with such overlaying, viz. the airport manager, project manager, designer and contractors to ensure that the work is carried out most efficiently and without loss of revenues, inconvenience to passengers or delays to the air traffic systems. A unique feature of such off-peak construction is that a temporary ramp (a transition surface between the overlay and the existing pavement) must be constructed at the end of each work session so that the runway can be used for aircraft operations once the work force clears the area. This chapter includes guidance on the design of such temporary ramps, however, it is not the intent of this chapter to deal with the design of overlays per se. For guidance on the latter subject, the reader should refer to Chapter 4.

8.2. Airport authority's role

8.2.1. Project co-ordination

8.2.1.1. Off-peak construction is, by its very nature, a highly visible project requiring close coordination with all elements of the airport during planning and design and virtually daily during construction. Once a runway paving project has been identified by the airport, it is important that the nominees of the airport authority, users and the Civil Aviation Authority of the State meet to discuss the manner in which construction is to be implemented. The following key personnel should be in attendance at all planning meetings: from the airport authority - the project manager, the operations, planning, engineering and maintenance directors; from the airlines - local station managers and head office representatives where appropriate; from the civil aviation authority - representatives from Air Traffic Services and Aeronautical Information Services. The agenda should include:

a. determination of working hours. Since time is of the essence in off-peak construction, the contractor should be given as much time as possible to overlay the pavement each work period. A minimum period of 8.5 hours is recommended. Work should be scheduled for a time period that will displace the least amount of scheduled flights. The selection of a specific time period should be developed and coordinated with airline and other representatives during the initial planning meetings. Early identification of the hours will allow the airlines to adjust future schedules, as needed, to meet construction demands. It is essential that the runway be opened and closed at the designated time without exception, as airline flight schedules, as well as the contractor's schedules, will be predicated on the availability of the runway at the designated time;

b. identification of operational factors during construction and establishment of acceptable criteria include:

1) designation of work areas;
2) aircraft operations;
3) affected navigation aids· (visual and non-visual aids);
4) security requirements and truck haul routes;
5) inspection and requirements to open the area for operational use;
6) placement and removal of construction barricades;
7) temporary aerodrome pavement marking and signing;

8) anticipated days of the week that construction will take place; and
9) issuance of NOTAM and advisories;

c. lines of communication and co-ordination elements. It is essential that the project manager be the only person to conduct co-ordination of the pavement project. The methods and lines of communication should be discussed for determining the availability of the runway at the start of each work period and the condition of the runway prior to opening it for operations;

d. special aspects of construction including temporary ramps and other details as described herein; and

e. contingency plan in case of abnormal failure or an unexpected disaster. Role of project manager

8.2.1.2. Project manager. It is essential that the airport authority select a qualified project manager to oversee all phases of the project, from planning through final inspection of the completed work. This individual should be experienced in design and management of aerodrome pavement construction projects and be familiar with the operation of the airport. The project manager should be the final authority on all technical aspects of the project and be responsible for its co-ordination with airport operations. All contact with any element of the airport authority should be made only by the project manager so as to ensure continuity and proper co-ordination with all elements of aerodrome operations. Responsibilities should include:

a) planning and design:
1) establishment of clear and concise lines of communications;
2) participation as a member of the design engineer's selection team
3) co-ordination of project design to meet applicable budget constraints;
4) co-ordination of airport and airlines with regards to design review, including designated working hours, aircraft operational requirements, technical review and establishment of procedures for coordinating all work; and
5) chairmanship of all meetings pertaining to the project; and

b) construction:
1) complete management of construction with adequate number of inspectors to observe and document work by the contractor;
2) checking with the weather bureau, airport operations and air traffic control prior to starting construction and confirming with the contractor's superintendent to verify if weather and air traffic conditions will allow work to proceed as scheduled;
3) conferring with the contractor's project superintendent daily and agreeing on how much work to attempt, to ensure the opening of the runway promptly at the specified time each morning. This is especially applicable in areas where pavement repair and replacement are to take place; and
4) conducting an inspection with airport operations of the work area before opening it to aircraft traffic to ensure that all pavement surfaces have been swept clean, temporary ramps are properly constructed and marking is available for aircraft to operate safely.

8.2.1.3. Resident engineer. The designation of a resident engineer, preferably a civil engineer, will be of great benefit to the project, and of great assistance to the project manager. Duties of the resident engineer should include:

a) preparation of documentation on the work executed during each work period;
b) ensuring all tests are performed and results obtained from each work period;
c) scheduling of inspection to occur each work period;
d) observing contract specifications compliance and reporting of any discrepancies to the project manager and the contractor; and
e) maintaining a construction diary.

8.2.2. Testing requirements

8.2.2.1. There is no requirement for additional tests for off-peak construction versus conventional construction. The only difference with off-peak construction is that it requires acceptance testing to be performed at the completion of each work period and prior to opening to operations and results reviewed before beginning work again. These procedures normally will require additional personnel to ensure that tests are performed correctly and on time.

8.2.3. Inspection requirements

8.2.3.1. One of the most important aspects of successful completion of any kind of paving project is the amount and quality of inspection performed. Since the airport accepts beneficial occupancy each time the runway is open to traffic, acceptance testing must take place each work period. In addition to the project manager and resident engineer, the following personnel are recommended as a minimum to observe compliance with specifications:

a) Asphalt plant inspector. A plant inspector with a helper whose primary duty it will be to perform quality control tests, including aggregate gradation, hot bin samples and Marshall tests.
b) Paving inspectors. There should be two paving inspectors with each paving machine. Their duties should include collection of delivery tickets, checking temperatures of delivered material, inspection of grade control methods, and inspection of asphalt lay-down techniques and joint construction smoothness.
c) Compaction inspector. The compaction inspector should be responsible for observing proper sequencing of rollers and for working with a field density meter to provide the contractor with optimum compaction information.
d) Survey crew. Finished grade information from each work period is essential to ensuring a quality job. An independent registered surveyor and crew should record levels of the completed pavement at intervals of at least 8 m longitudinally and 4 m transversely, and report the results to the project manager at the completion of each work period.
e) Pavement repair inspector. Shall be responsible for inspection of all pavement repairs and surface preparation prior to paving.
f) Electrical inspector. Ensures compliance with specifications.

8.3. Design considerations

8.3.1. General.

Plans and specifications for pavement repair and overlay during off-peak periods should be presented in such detail as to allow ready determination of the limits of pavement repair, finish grades and depths of overlay. Plans and specifications are to be used for each work period by the contractor and inspection personnel, and should be clear and precise in every detail.

8.3.2. Pavement survey

8.3.2.1. A complete system of bench marks should be set on the side of the runway or taxiway to permit a ready reference during cross-sectioning operations. The bench marks should be set at approximately 125 m intervals. Pavement cross-sectioning should be performed at 8 m intervals longitudinally, and 4 m intervals transversely. Extreme care should be exercised in level operations, since the elevations are to be used in determining the depth of asphalt overlay. The designer should not consider utilizing grade information from previous as-built drawings or surveys that were run during the winter months, as it has been shown that elevations can vary from one season to the next. This is especially critical for single lift asphalt overlays.

8.3.2.2. After finish grades and transverse slope of the runway are determined, a tabulation of grades should be included in the plans for the contractor to use in bidding the project and for establishment of erected stringline. The tabulation of grades should include a column showing existing runway elevation, a column showing finish overlay grade, and a column showing depth of overlay. Grades should be shown longitudinally every 8 m and transversely every 4 m. This item is considered essential in the preparation of plans for contracting off-peak construction.

8.3.3. Special details

8.3.3.1. Details pertaining to the following items should be included in the plans:

a) Temporary ramps. At the end of each hot mix asphalt concrete overlay work period, it will be necessary to construct a ramp to provide a transition from the new course of overlay to the existing pavement. The only exception to construction of a ramp is when the depth of the overlay is 4 cm or less. In multiple lift overlays, these transitions should be not closer than 150 m to one another. As far as possible, the overlay should proceed from one end of the runway toward the other end in the same direction as predominant aircraft operar1ons so that most aircraft encounter a downward ramp slope. In the event of continued operational change of direction, it would be advantageous for the overlaying to proceed upgrade since an upgrade ramp is shorter and avoids long thin tapers. The construction of the ramp is one of the most important features in the work period. A ramp that is too steep could cause possible structural damage to the operating aircraft or malfunction of the aircraft's instruments. A ramp that is too long may result in a ravelling of the pavement, and foreign object damage to aircraft engines, as well as taking excessive time to construct. The longitudinal slope of the temporary ramp shall be between 0.8 and 1.0 per cent, measured with reference to the existing runway surface or previous overlay course. The entire width of the runway should normally be overlaid during each work session. Exceptional circumstances, e.g. adverse weather conditions, equipment failure, etc. may not permit the overlaying of the full runway width during a work session. Should that be the case, the edges need to be merged with the old pavement surface to avoid a sudden level change in the event an aircraft veers off the overlaid portion. The maximum transverse slope of the temporary ramp should not exceed 2 per cent. A temporary ramp may be constructed in two ways, depending upon the type of equipment that is available. The most efficient way is to utilize a cold planing machine to heel-cut the pavement at the beginning and at the end of the work period overlay (refer to Figure 8-1). If cold planing equipment is not available, then a temporary ramp should be constructed as shown in Figure 8-2. In no case should a bond-breaking layer be placed under the ramp for easy removal during the next work period. Experience has shown that this bond-breaking layer almost always comes loose causing subsequent breaking-up of the pavement under aircraft operations.

b) In-pavement lighting. Details depicting the removal and re-installation of in-pavement lighting are to be included on the plans where applicable. The details should depict the removal of the light fixture and extension ring, placement of a target plate over the light base, filling the hole with hot mix dense graded asphalt until overlay operations are complete, accurate survey location information, core drilling with a 10 cm core to locate the centre of the target plate, and final coring with an appropriate sized core machine. The light and new extension ring can then be installed to the proper elevation.

c) Runway markings. During the course of off-peak construction of a runway overlay, it has been found acceptable, if properly covered by a NOTAM, to mark only the centre line stripes and the runway designation numbers on the new pavement until the final asphalt lift has been completed and final striping can then be performed. In some cases where cold planing of the surface or multiple lift overlays are used, as many as three consecutive centre line stripes may be omitted to enhance the bond between layers.

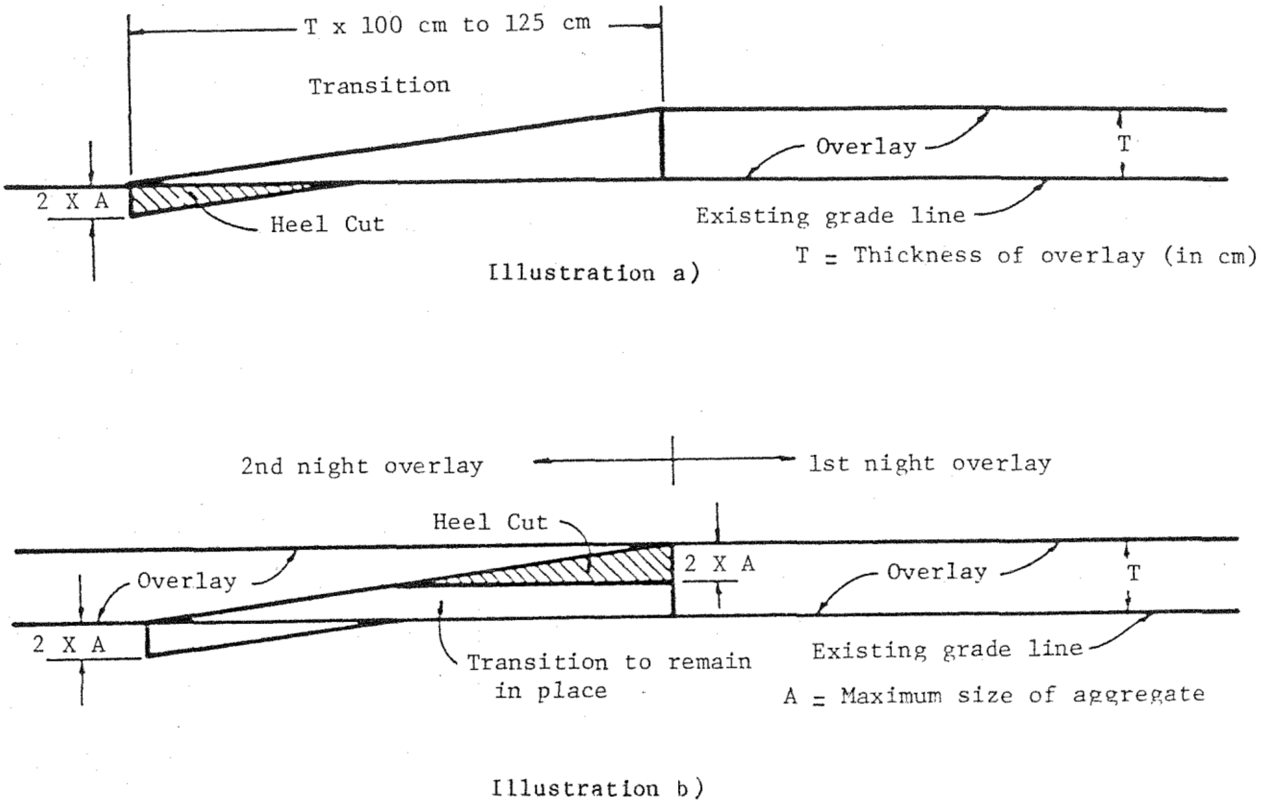


Figure 8-1. Temporary ramp construction with cold planing machine

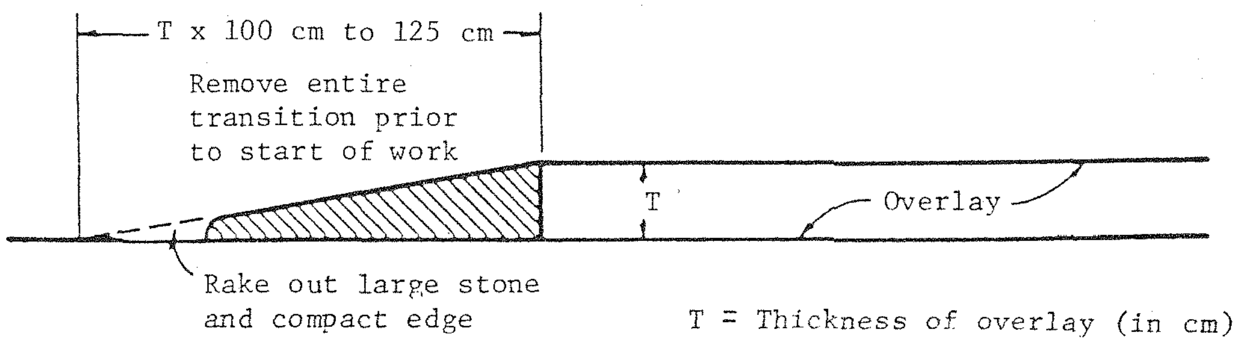


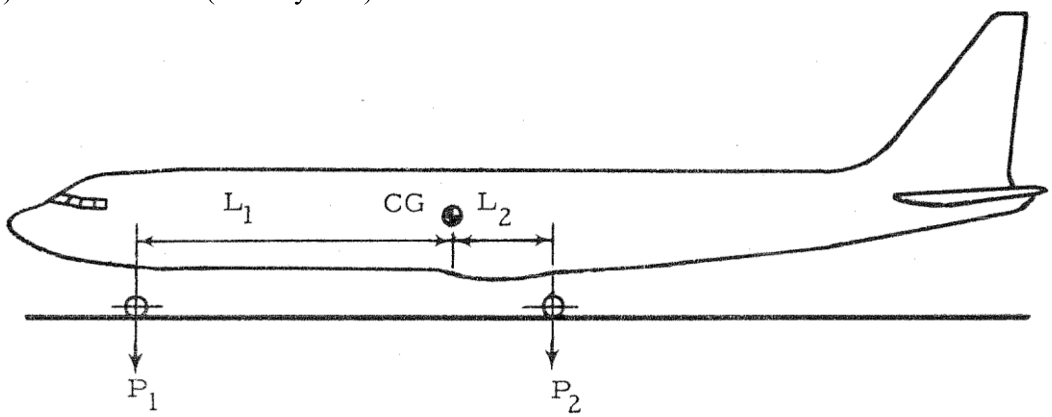
Figure 8-2. Temporary ramp construction without cold planing machine

APPENDIX 1: - AIRCRAFT CHARACTERISTICS AFFECTING PAVEMENT BEARING STRENGTH

1. General

1.1 This Appendix describes those characteristics of aircraft which affect pavement strength design, namely: aircraft weight, percentage load on nose wheel, wheel arrangement, main leg load, tire pressure and contact area of each tire. Table Al-1 contains these data for most of the commonly used aircraft.

1.2 Aircraft loads are transmitted to the pavement through the landing gear which normally consists of two main legs and an auxiliary leg, the latter being either near the nose (now the most frequent arrangement) or near the tail (older system).



1.3 The portion of the load imposed by each leg will depend on the position of the centre of gravity with reference to the three supporting points. The static distribution of the load by the different legs of a common tricycle landing gear may be illustrated as follows:

Where W is the aircraft weight; P1 the load transmitted by the auxiliary leg; $P_2$the load transmitted by both main legs; L_1 and L_2 the distance measured along the plane of symmetry from the centre of gravity to P_1 and P_2 respectively, then

$$W = P_1 + P_2 \qquad\qquad P_1 L_1 = P_2 L_2$$

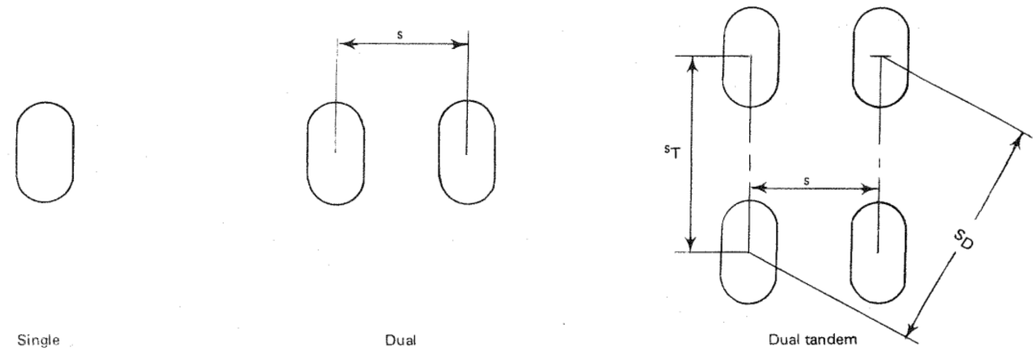
Therefore

$$P_2 = P_1 \frac{L_1}{L_2}$$

1.4 Usually the ratio L_1/L_2 is around 9, i.e. the auxiliary leg accounts for approximately 10 per cent of the aircraft gross weight. Therefore, each main leg imposes a load equal to about 45 per cent of that weight. Wheel base and track width have not been included, since these dimensions are such that there is no possibility of interaction of the stresses imposed by the different legs of the landing gear.

1.5 From the above considerations, it will be seen that the characteristics of each main leg provide sufficient information for assessing pavement strength requirements. Accordingly, the table confines itself to providing data thereon.

1.6 The load supported or several rubber-tired wheels. The main legs of landing gear of by each leg is transmitted to the pavement by one The following wheel arrangements will be found on civil aircraft at present in-service.



1.7 For pavement design and evaluation purposes the following wheel spacings are significant, and therefore listed in the table.
S - distance between centres of contact areas of dual wheels
ST - distance between axis of tandem wheels
SD - distance between centres of contact areas of diagonal wheels and is given by the expression

$$S_D = \sqrt{(S^2 + S_T^2)}$$

Tire pressures given are internal, or inflation pressures.

1.8 It should be noted that throughout the table figures refer to the aircraft at its maximum take-off weight. For lesser operational weights, figures quoted for "load on each leg", "tire-pressure" and/or "contact area" should be decreased proportionally.

Listof abbreviations used in TableAl-1

COM	- Complex
D	- Dual
DT	- Dual tandem
F	- Front
R	- Rear
S	- Distance between centres of contact areas of dual wheels
S_D	- Distance between centres of contact areas of diagonal wheels
S_T	- Distance between axis of tandem wheels
T	- Tandem
kg	- Kilogram
MPa	- Megapascal
cm	- Centimetre

Note on units

This table has been prepared in metric units. To convert from kilogram to newton multiply by 9.80665.

Table A1-1.- Aircraft characteristics for design and evaluation of pavements

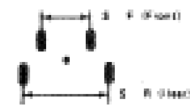
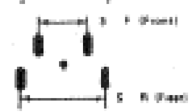
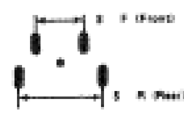
				MAIN LEGS OF LANDING GEAR					
Aircraft type	All-up mass (kg)	Load on one main gear leg (%)	Wheel arrangement	Load on each leg (kg)	Tire pressure (MPa)	Wheel spacing (cm) (S)	(S_T)	(S_D)	Additional data for complex wheel arrangement
A300 B2 Airbus	137 000	47.0	DT	64 390	1.2	89	140	165.9	
A300 B2 Airbus	142 000	47.0	DT	66 740	1.29	89	140	165.9	
A300 B4 Airbus	150 000	47.0	DT	70 500	1.39	93	140	168.1	
A300 B4 Airbus	157 000	47.0	DT	73 790	1.48	93	140	168.1	
A300 B4 Airbus	165 000	47.0	DT	77 550	1.29	93	140	168.1	
A300-600 Airbus	165 000	47.0	DT	77 550	1.29	93	140	168.1	
A300-600R Airbus	170 000	47.4	DT	80 580	1.35	93	140	168.1	
A300-600R Airbus	171 700	47.4	DT	81 390	1.35	93	140	168.1	
A310-200 Airbus	132 000	46.7	DT	61 640	1.23	93	140	168.1	
A310-200 Airbus	138 600	46.7	DT	64 730	1.3	93	140	168.1	
A310-200 Airbus	142 000	46.7	DT	66 310	1.33	93	140	168.1	
A310-300 Airbus	150 000	47.0	DT	70 500	1.42	93	140	168.1	
A310-300 Airbus	157 000	47.4	DT	74 420	1.49	93	140	168.1	

Aircraft type	All-up mass (kg)	Load on one main gear leg (%)	Wheel arrangement	MAIN LEGS OF LANDING GEAR: Load on each leg (kg)	Tire pressure (MPa)	Wheel spacing (cm) (S)	(S_T)	(S_D)	Additional data for complex wheel arrangement
A320-100 Airbus Dual	66 000	47.1	D	31 090	1.28	93	-	-	
A320-100 Airbus Dual	68 000	47.1	D	32 030	1.34	93	-	-	
A320-100 Airbus Dual tandem	68 000	47.1	DT	32 030	1.12	78	100	126.8	Option
A320-200 Airbus Dual	73 500	47.0	D	34 550	1.45	93	-	-	
A320-200 Airbus Dual Tandem	73 500	47.0	DT	34 550	1.21	78	100	126.8	Option
BAC 1-11 Series 400	39 690	47.5	D	18 853	0.93	53	-	-	
BAC 1-11 Series 475	44 679	47.5	D	21 223	0.57	62	-	-	
BAC 1-11 Series 500	47 400	47.5	D	22 515	1.08	53	-	-	
BAe 146 Series 100	37 308	46.0	D	17 162	0.80/0.52	71	-	-	
BAe 146 Series 200	40 600	47.1	D	19 123	0.88/0.61	71	-	-	
B707-120B	117 027	46.7	DT	54 652	1.17	86	142	166.0	
B707-320B	148 778	46.0	DT	68 438	1.24	88	142	167.1	
B707-320C Freighter	152 407	46.7	DT	71 174	1.24	88	142	167.1	

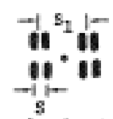
Aircraft type	All-up mass (kg)	Load on one main gear leg (%)	Wheel arrangement	MAIN LEGS OF LANDING GEAR: Load on each leg (kg)	Tire pressure (MPa)	Wheel spacing (cm) (S)	(S_T)	(S_D)	Additional data for complex wheel arrangement
B707-320C Convertible	152 407	46.7	DT	71 174	1.24	88	142	167.1	
B707-320/420	143 335	46.0	DT	65 934	1.24	88	142	167.1	
B720	104 326	47.4	DT	49 451	1.00	81	124	148.1	
B720B	106 594	46.4	DT	49 460	1.00	81	124	148.1	
B727-100C	73 028	47.8	D	34 907	1.09	86	-	-	
B727-100	77 110	47.6	D	36 704	1.14	86	-	-	
B727-200 Standard	78 471	48.5	D	38 058	1.15	86	-	-	
B727-200 Advanced	84 005	48.0	D	40 322	1.02	86	-	-	
B727-200 Advanced	86 636	47.7	D	41 325	1.06	86	-	-	
B727-200 Advanced	89 675	46.9	D	42 058	1.15	86	-	-	
B727-200 Advanced	95 254	46.5	D	44 293	1.19	86	-	-	
B737-100	44 361	46.2	D	20 495	0.95	78	-	-	
B737-200	45 722	46.4	D	21 215	0.97	78	-	-	
B737-200	52 616	45.5	D	23 940	1.14	78	-	-	
B737-200	52 616	45.5	D	23 940	0.66	78	-	-	
B737-200/200C Advanced	53 297	46.4	D	24 730	1.16	78	-	-	
B737-200/200C Advanced	56 699	46.3	D	26 252	1.23	78	-	-	

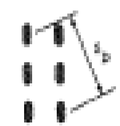
				MAIN LEGS OF LANDING GEAR					
Aircraft type	All-up mass (kg)	Load on one main gear leg (%)	Wheel arrangement	Load on each leg (kg)	Tire pressure (MPa)	Wheel spacing (cm) (S)	(S_T)	(S_D)	Additional data for complex wheel arrangement
B737-200 Advanced	58 332	46.0	D	26 833	1.25	78	-	-	
B737-300	61 462	45.9	D	28 211	1.34	78	-	-	
B737-300	61 462	45.9	D	28 211	1.14	78	-	-	
B737-400	64 864	46.9	D	30 421	1.44	78	-	-	
B737-500*	60 781	46.1	D	28 020	1.34	78	-	-	
B747-100	323 410	23.4	COM	75 678	1.50	112	147	184.8	Main U/C - 4 No. DT units Data based on equal load distribution
B747-100B (Passenger)	334 749	23.1	COM	77 327	1.56	112	147	184.8	Main U/C - 4 No. DT units Data based on equal load distribution
B747-100B	341 553	23.1	COM	78 899	1.32	112	147	184.8	Main U/C - 4 No. DT units Data based on equal load distribution
B747-100B SR	260 362	24.1	COM	62 747	1.04	112	147	184.8	Main U/C - 4 No. DT units Data based on equal load distribution
B747-SP	302 093	22.9	COM	69 179	1.30	110	137	175.7	Main U/C - 4 No. DT units Data based on equal load distribution
B747-SP	318 881	21.9	COM	69 835	1.40	110	137	175.7	Main U/C - 4 No. DT units Data based on equal load distribution
B747-200B	352 893	23.6	COM	83 283	1.37	112	147	184.8	Main U/C - 4 No. DT units Data based on equal load distribution

*Preliminary information

Aircraft type	All-up mass (kg)	Load on one main gear leg (%)	Wheel arrangement	MAIN LEGS OF LANDING GEAR: Load on each leg (kg)	Tire pressure (MPa)	Wheel spacing (cm) (S)	(S_T)	(S_D)	Additional data for complex wheel arrangement
B747-200C	373 305	23.1	COM	86 233	1.30	112	147	184.8	Main U/C - 4 No. DT units Data based on equal load distribution
B747-200F/300	379 201	23.2	COM	87 975	1.39	112	147	184.8	Main U/C - 4 No. DT units Data based on equal load distribution
B747-400	395 987	23.4	COM	92 661	1.41	112	147	184.8	Main U/C - 4 No. DT units Data based on equal load distribution
B757-200	109 316	45.2	DT	49 411	1.17	86	114	142.8	
B767-200	143 789	46.2	DT	66 431	1.31	114	142	182.1	
B767-200-ER	159 755	46.9	DT	74 925	1.21	114	142	182.1	
B767-300	159 665	47.5	DT	75 841	1.21	114	142	182.1	
B767-300-ER	172 819	46.9	DT	81 052	1.31	114	142	182.1	
B767-300-ER	185 520	46.0	DT	85 339	1.38	114	142	182.1	
Caravelle 10	52 000	46.1	COM	23 966	F0.75 R1.17	R40 F45	107	115.1	
Caravelle 12	55 960	46	COM	25 743	F0.69 R1.08	F38 R41	107	114.1	
Concorde	185 066	48.0	DT	88 803	1.26	68	167	180.3	
Canadair CL 44	95 708	47.5	COM	45 461	1.12	F51 R76	122	137.5	
CV 880 M	87 770	46.6	DT	40 901	1.03	55	114	126.6	
CV 990	115 666	48.5	DT	56 098	1.28	61	118	132.8	
DC-3	11 430	46.8	Sin	5 349	0.31	-	-	-	

Aircraft type	All-up mass (kg)	Load on one main gear leg (%)	Wheel arrangement	MAIN LEGS OF LANDING GEAR Load on each leg (kg)	Tire pressure (MPa)	Wheel spacing (cm) (S)	(S_T)	(S_D)	Additional data for complex wheel arrangement
DC-4	33 113	46.8	D	15 480	0.53	74	-	-	
DC-8-43	144 242	46.5	DT	67 073	1.22	76	140	159.3	
DC-8-55	148 778	47.0	DT	69 926	1.28	76	140	159.3	
DC-8-61/71	148 778	48.0	DT	71 413	1.30	76	140	159.3	
DC-8-62/72	160 121	46.5	DT	76 858	1.29	81	140	161.7	
DC-8-63/73	162 386	47.6	DT	77 296	1.30	81	140	161.7	
DC-9-15	41 504	46.2	D	19 175	0.90	61	-	-	
DC-9-21	45 813	47.2	D	21 624	0.98	64	-	-	
DC-9-32	49 442	46.2	D	22 842	1.07	64	-	-	
DC-9-41	52 163	46.7	D	24 334	1.10	66	-	-	
DC-9-51	55 338	47.0	D	26 009	1.17	66	-	-	
MD-81	63 957	47.8	D	30 539	1.17	71	-	-	
MD-82/88	68 266	47.6	D	32 460	1.27	71	-	-	
MD-83	73 023	47.4	D	34 613	1.34	71	-	-	
MD-87	68 266	47.4	D	32 358	1.17	71	-	-	
DC-10-10	200 942	46.9	DT	94 141	1.31	137	163	212.9	
DC-10-10	196 406	47.2	DT	92 605	1.28	137	163	212.9	
DC-10-15	207 746	46.7	DT	96 914	1.34	137	163	212.9	Loading based on wing DT. Main U/C includes central D.
DC-10-30/40	268 981	37.9	COM	101 944	1.24	137	163	212.9	Loading based on wing DT. Main U/C includes central D.
DC-10-30/40	253 105	37.7	COM	95 421	1.17	137	163	212.9	Loading based on wing DT. Main U/C includes central D.

Aircraft type	All-up mass (kg)	Load on one main gear leg (%)	Wheel arrangement	MAIN LEGS OF LANDING GEAR Load on each leg (kg)	Tire pressure (MPa)	Wheel spacing (cm) (S)	(S_T)	(S_D)	Additional data for complex wheel arrangement
DC-10-30/40	260 816	37.6	COM	98 069	1.21	137	163	212.9	Loading based on wing DT. Main U/C includes central D.
MD-11	274 650	39.2	COM	107 663	1.41	137	163	212.9	Loading based on wing DT. Main U/C includes central D.
Dash 7	19 867	46.8	D	9 228	0.74	42	-	-	
F27 Friendship Mk500	19 777	47.5	D	9 394	0.54	45	-	-	
Fokker 50 HTP	20 820	47.8	D	9 952	0.59/ 0.55	52	-	-	
Fokker 50 LTP	20 820	47.8	D	9 952	0.42	52	-	-	
F28 Fellowship Mk1000LTP	29 484	46.3	D	13 651	0.58	58	-	-	
F28 Fellowship Mk1000HTP	29 484	46.3	D	13 651	0.69	55	-	-	
Fokker 100	44 680	47.8	D	21 357	0.98	59	-	-	
HS125-400A -400B	10 600	45.5	D	4 824	0.77	32	-	-	
HS125-600A -600B	11 340	45.5	D	5 160	0.83	32	-	-	
HS748	21 092	43.6	D	9 196	0.59	48	-	-	
IL62	162 600	47.0	DT	76 910	1.08	80	165	188.4	
IL62M	168 000	47.0	DT	79 460	1.08	80	165	188.4	
IL76T	171 000	23.5	COM	38 730	0.59	-	258	-	 S 62, S_1 144
IL86	211 500	31.2	COM	64 390	0.88	125	149	194.5	Main U/C 3 DT units

Aircraft type	All-up mass (kg)	Load on one main gear leg (%)	Wheel arrangement	MAIN LEGS OF LANDING GEAR Load on each leg (kg)	Tire pressure (MPa)	Wheel spacing (cm) (S)	(S_T)	(S_D)	Additional data for complex wheel arrangement
L-100-20	70 670	48.2	T	17 031	0.72	-	154	-	Main wheels arranged in tandem on four separate legs.
L-100-30	70 670	48.4	T	17 102	0.72	-	154	-	Main wheels arranged in tandem on four separate legs.
L1011-1	195 952	47.4	DT	92 881	1.33	132	178	221.6	
L-1011-100/200	212 281	46.8	DT	99 348	1.21	132	178	221.6	
L-1011-500	225 889	46.2	DT	104 361	1.27	132	178	221.6	
Trident 1E	61 160	46.0	COM	28 196	1.03	-	-	-	 S_1 32 S_2 94
Trident 2E	65 998	47.0	COM	31 019	1.07	-	-	-	 S_1 30 S_2 95
Trident 3	68 266	45.5	COM	31 095	1.14	-	-	-	 S_1 30 S_2 95
TU134A	49 000	45.6	DT	22 690	0.83	56	99	113.7	
TU154B	98 000	45.1	COM	44 198	0.93	62	F103 R 98	223.6	
VC10-1150	151 953	48.3	DT	73 317	1.01	86	155	177.3	

APPENDIX 2: - PROCEDURES FOR DETERMINING THE AIRCRAFT CLASSIFICATION NUMBER OF AN AIRCRAFT

1. Rigid pavements

1.1 The ACN of an aircraft for operations on a rigid pavement shall be determined using Computer Programme No. 1.

Note.- Computer Programme No. 1 is based on programme PDILB developed by Mr. R.G. Packard of Portland Cement Association, Illinois, United States, for design of rigid pavements. For convenience, several aircraft types currently in use have been evaluated on rigid pavements founded on the four subgrade categories at CAR-14, Part I, Chapter 2, 2.5.6 b) and the results tabulated in Attachment A, Table B-1 of that Annex and Table A5-1 of Appendix 5 of this Manual.

2. Flexible pavements

2.1 The ACN of an aircraft for operations on a flexible pavement shall be determined using Computer Programme No. 2.

Note.- Computer Programme No. 2 is based on the United States Army Engineer's CBR method of design of flexible pavements (see United States Army Engineer Waterways Experiment Station Instruction Report S-77-1). For convenience, several aircraft types currently in use have been evaluated on flexible pavements founded on the four subgrade categories at CAR-14, Part I, Chapter 2, 2.5.6 b) and the results tabulated in Attachment A, Table B-1 of that Annex and Table A5-1 of Appendix 5 of this Manual.

Note: For detailed information about computer programme please refer Aerodrome Design Manual Part -3 ; Pavements

APPENDIX 3: - PAVEMENT DESIGN AND EVALUATION GRAPHS PROVIDED BY FRANCE

Notes:

1) The pavement design and evaluation graphs included in this Appendix are based on the same aircraft characteristics (track, wheel base, standard tire pressure) as those used to calculate the ACN.
2) The weights shown in the graphs represent static loads on the main undercarriage leg.
3) The rigid pavement graphs assume that the tire pressure remains constant at the value q^o shown in the graphs. Should the actual tire pressure q be different from q^o proceed as follows:

   a) If p is the weight of the undercarriage leg in question, find the weight p^1 producing the same contact area at the pressure q^o using the relationship:

   $$\frac{p1}{q^o} = \frac{p}{q}$$

   b) Consult the graph to determine stress σ^1 produced by the weight p^1 in the slab in question.

   c) The value σ required is then given by the relationship:

   $$\frac{\sigma}{\sigma 1} = \frac{q}{q^o}$$

4) The flexible pavement graphs assume that the tire pressure remains constant at the value q^o shown in the graphs. If the actual tire pressure q does not differ by more than± 0.3 MPa from q^o, it is accepted that the effects of the pressure may be disregarded.

   Conversely, a correction is made in accordance with the following:

$$h = h^o \frac{\left[\dfrac{1}{0.57\ CBR} - \dfrac{1}{32\ q^o}\right]^{1/2}}{\left[\dfrac{1}{0.57\ CBR} - \dfrac{1}{32\ q}\right]^{1/2}} \quad \text{(q expressed in MPa)}$$

Where h is the thickness sought for pressure q

h^o is the thickness read on the graph drawn up for pressure q^o.

5) Figures A3-1 to A3-10 are provided as examples.

Graphs for all types of aircraft are available on request from:
MINISTERE DES TRANSPORTS Direction Generale de l' Aviation Civile
Service Technique des Bases Aeriennes
246, rue Lecourbe - 75732 PARIS CEDEX 15 - FRANCE

FLEXIBLE PAVEMENT
A 300 B2
Main Leg
Tire pressure: 1.23 MPa

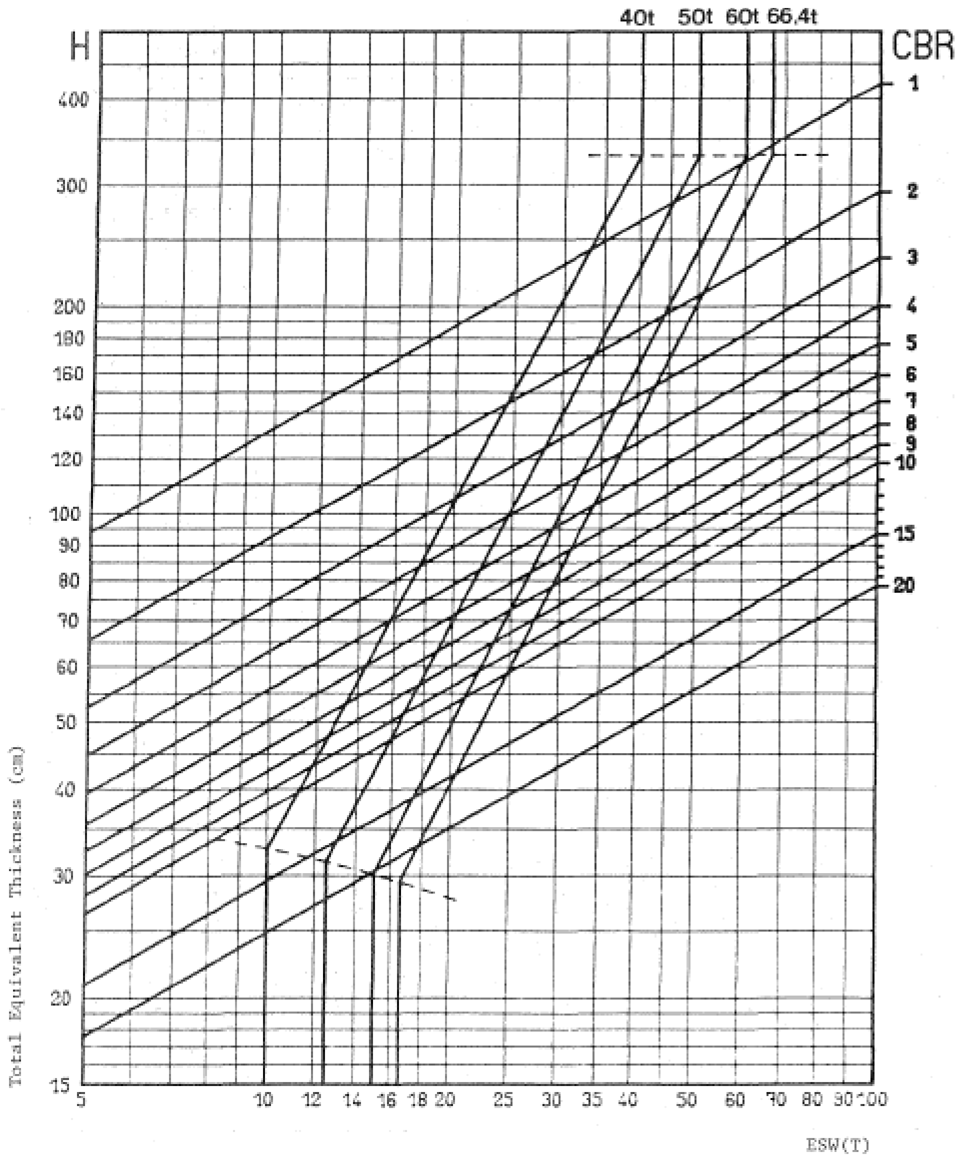


Figure A3-1

RIGID PAVEMENT
A 300 B2
Main Leg
Tire pressure: 1.23 MPa

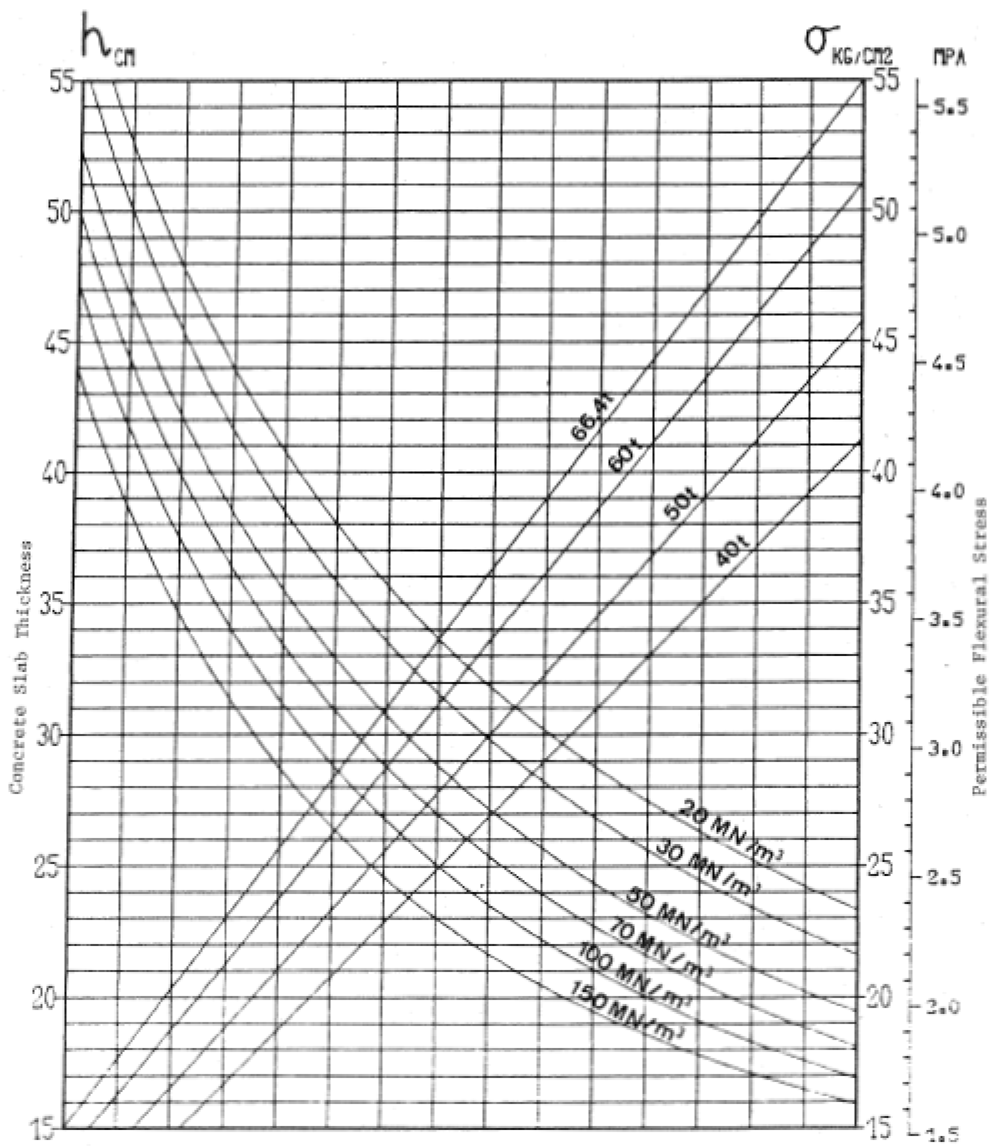


Figure A3-2

FLEXIBLE PAVEMENT A300 B4 - A310
Main Leg
Tire pressure: 1.41 MPa

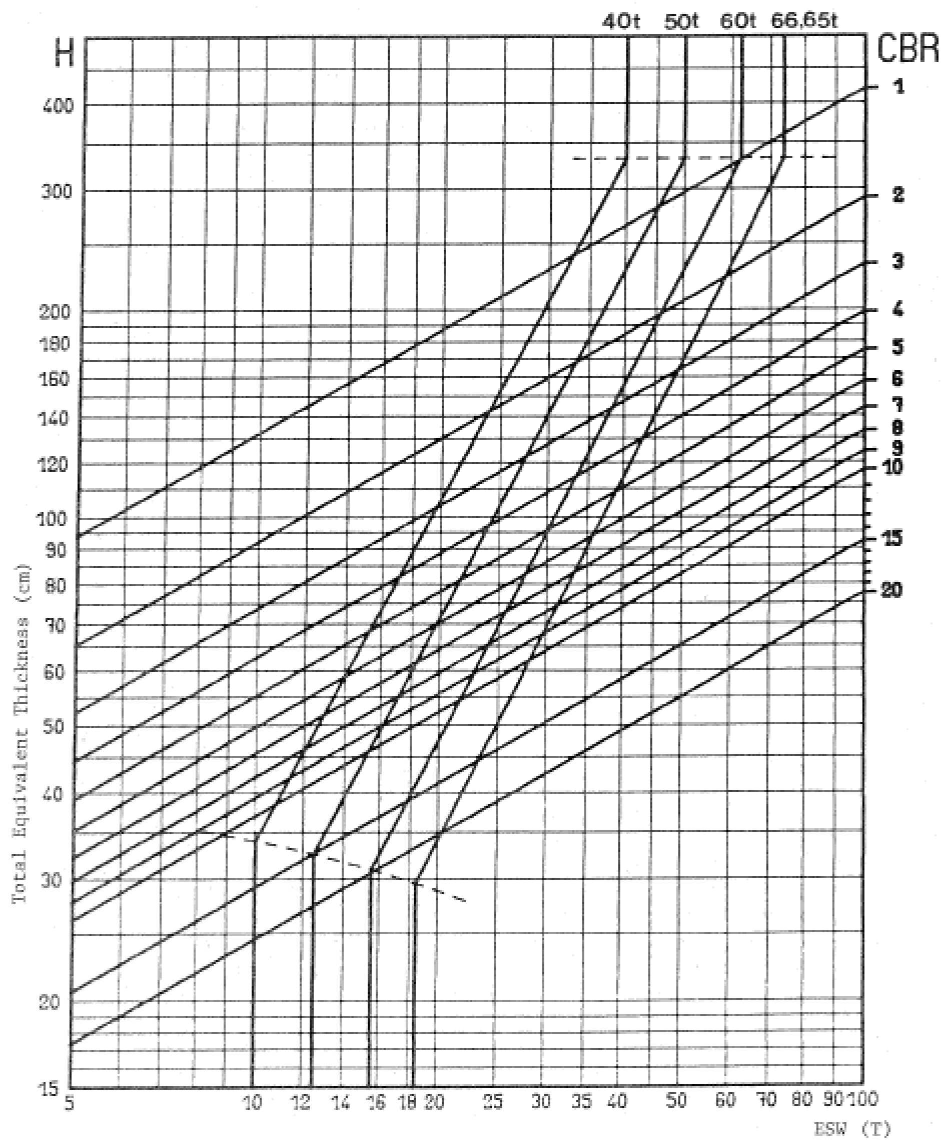


Figure A3-3

RIGID PAVEMENT
A 300 B4 - A310
Main Leg
Tire pressure: 1.41 MPa

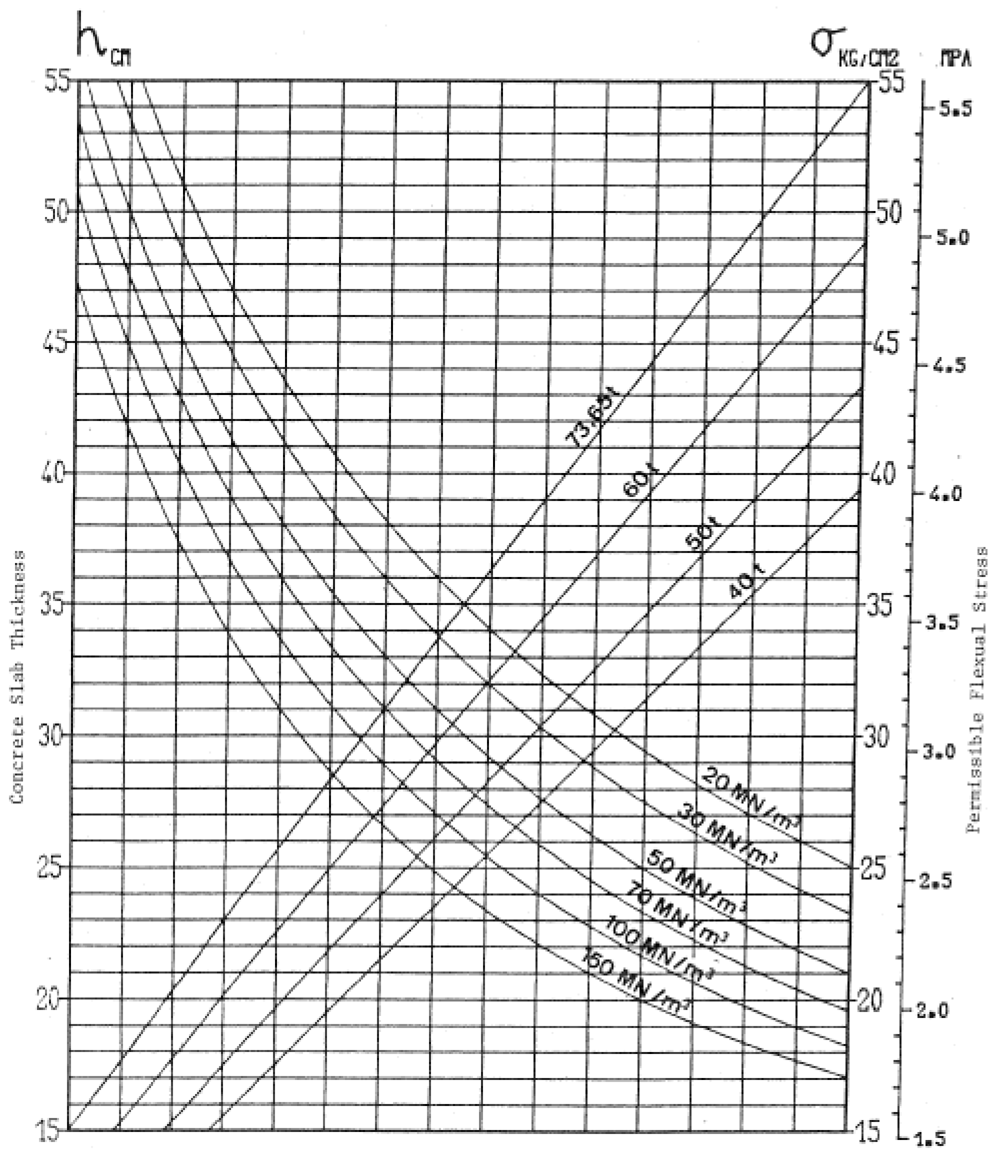


Figure A3-4

FLEXIBLE PAVEMENT
B 727 (all series)
Main Leg

Tire pressure: 1.09 MPa

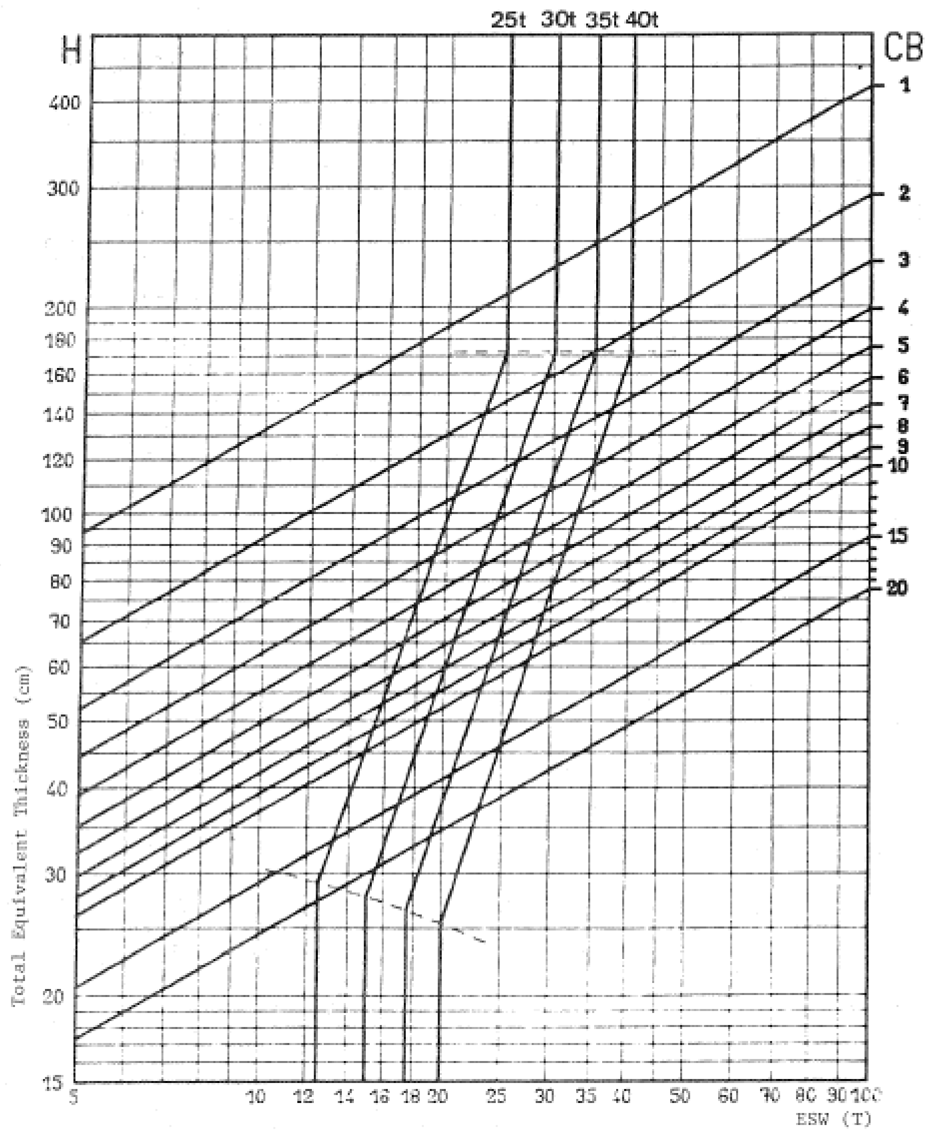


Figure A3-5

RIGID PAVEMENT
A 727 (all series)
Main Leg

Tire pressure: 1. 09 MPa

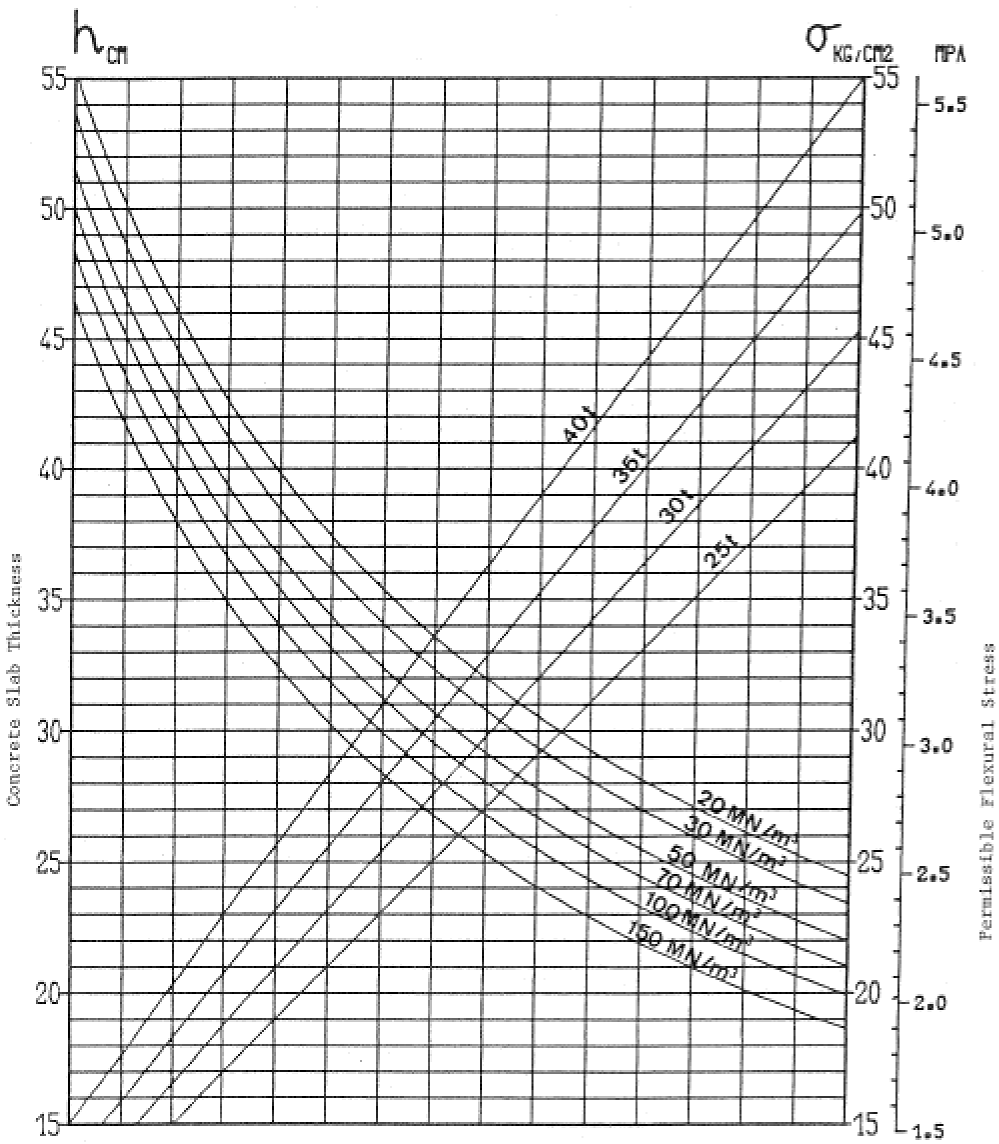


Figure A3-6

FLEXIBLE PAVEMENT
B 737 (all series)
Main Leg Tire pressure: 1.02 MPa

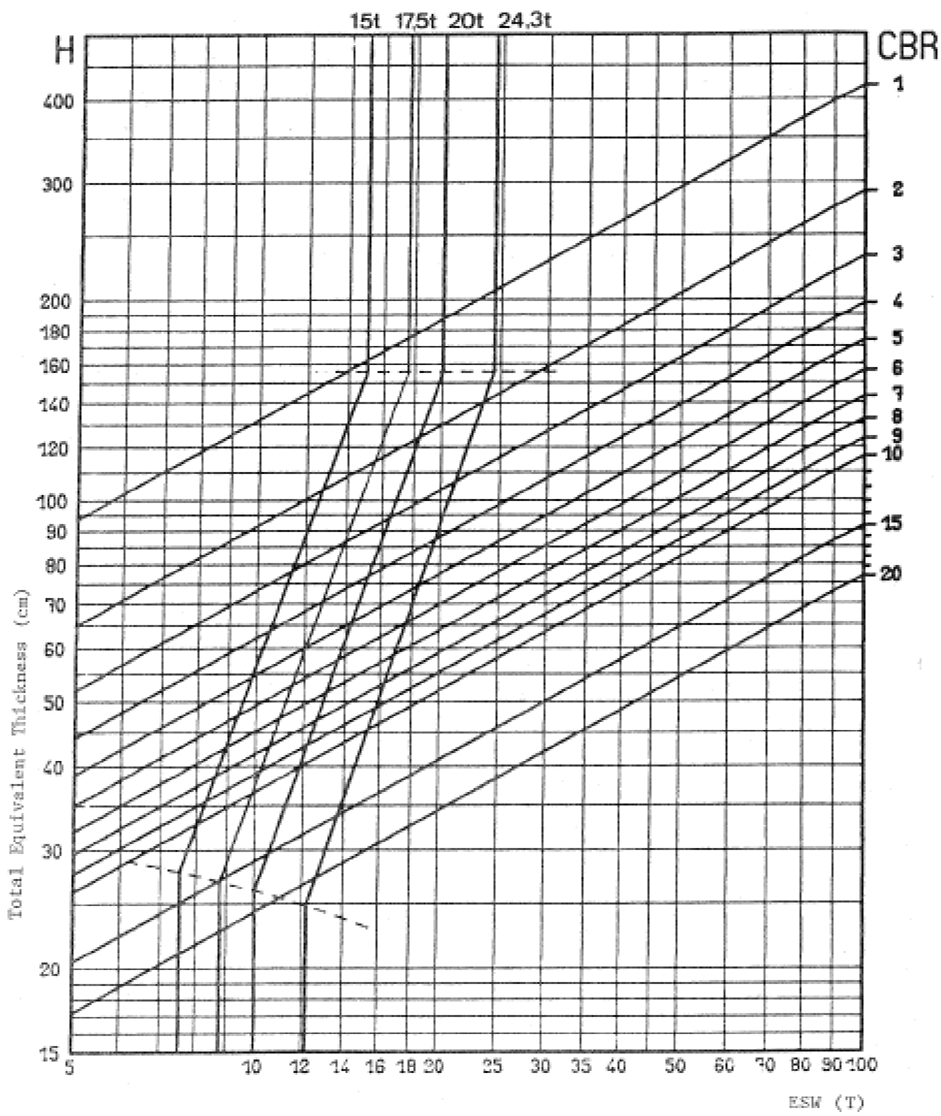


Figure A3-7

RIGID PAVEMENT
B 737 (all series)
Main Leg
Tire pressure: 1.02 MPa

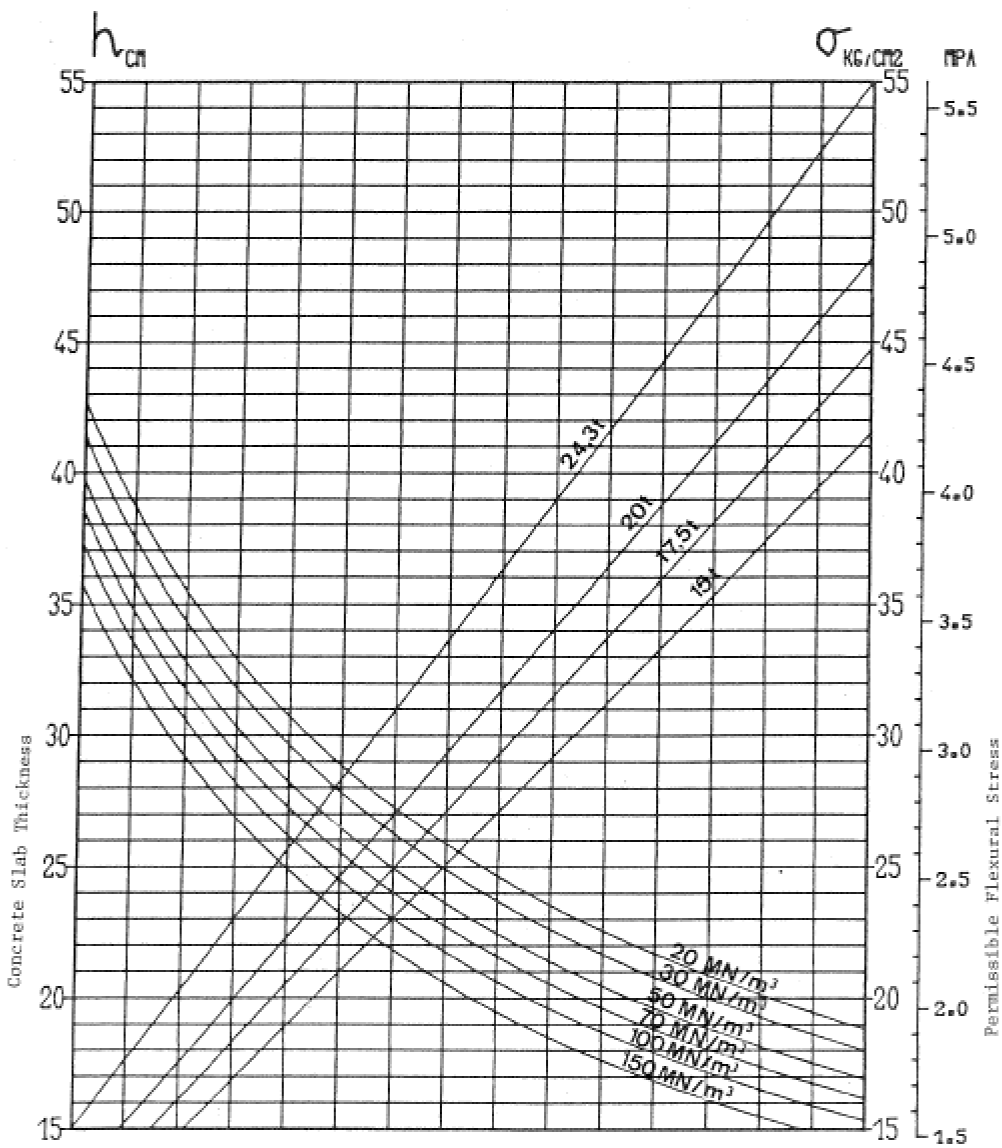


Figure A3-8

FLEXIBLE PAVEMENT
B 747 (series 100 - 200 - B, C, F - SR)
Main Leg

Tire pressure : 1.45 MPa

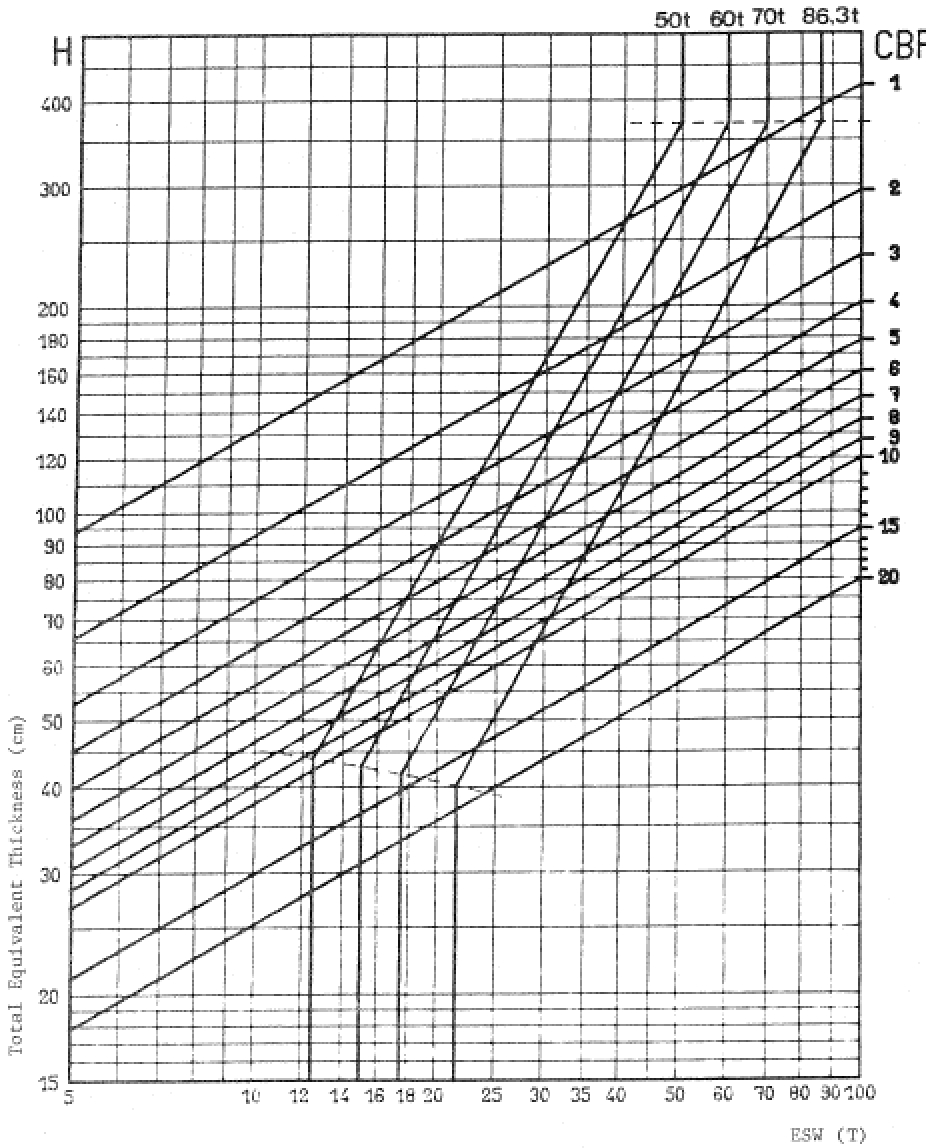


Figure A3-9

RIGID PAVEMENT
B 747 (series 100 - 200, B, C, F - SR)
Main Leg

Tirepressure: 1.45MPa

Tirepressure: 1.45MPa

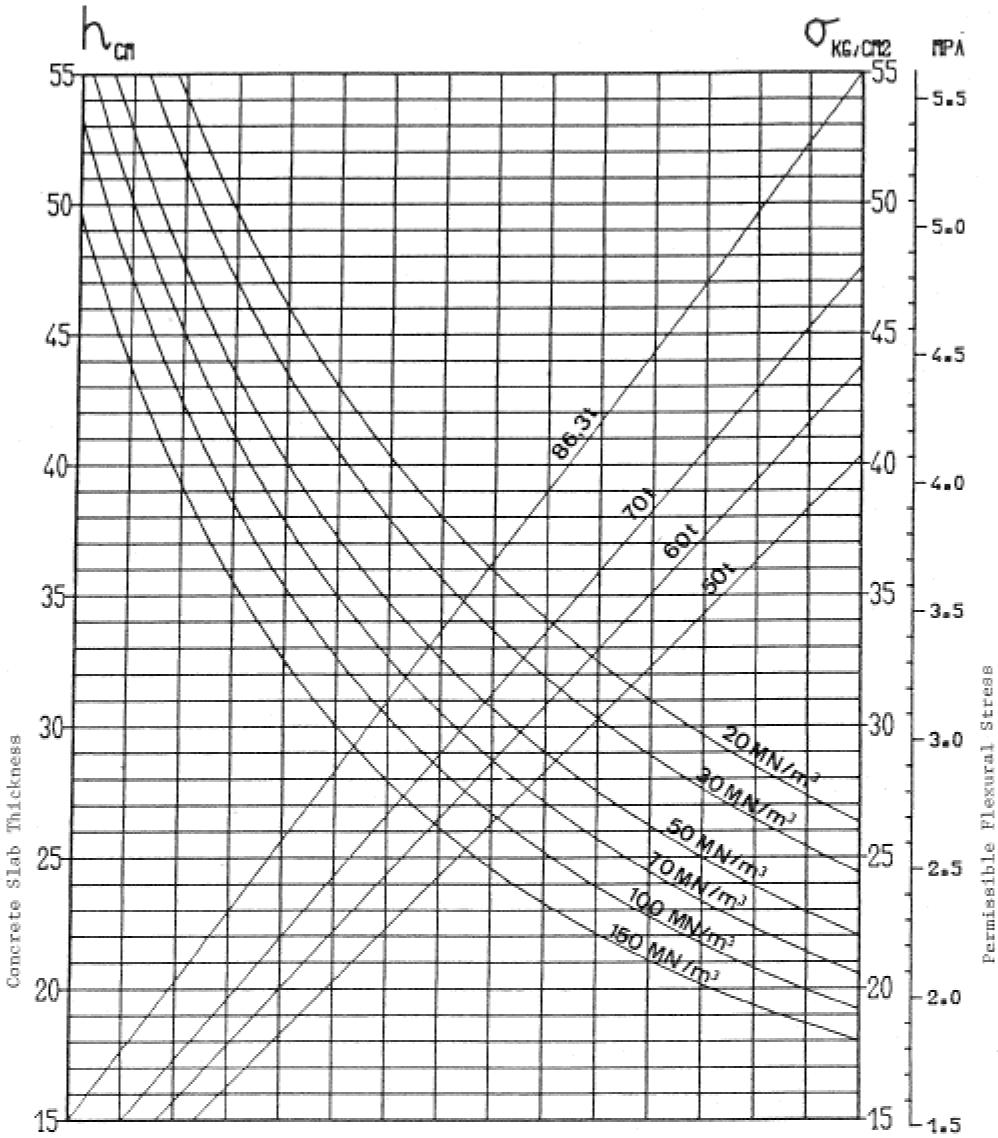


Figure A3-10

APPENDIX 4: - BACKGROUND INFORMATION ON THE UNITED STATES PRACTICE FOR THE DESIGN AND EVALUATION OF PAVEMENTS

1. Prior FAA method of soil classification

1.1 Background

The FAA method of soil classification which was used prior to the adoption of the Unified Soil Classification System is presented in this Appendix. The reason for including the method in this Appendix is that many past records contain references to the FAA method and this Appendix allows the reader to converse in the FAA classification method.

1.2 Soil classification

a) While the results of individual tests indicate certain physical properties of the soil, the principal value is derived from the fact that, through correlation of the data so obtained, it is possible to prepare an engineering classification of soils related to their field behaviour. Such a classification is presented in Figure A4-l.

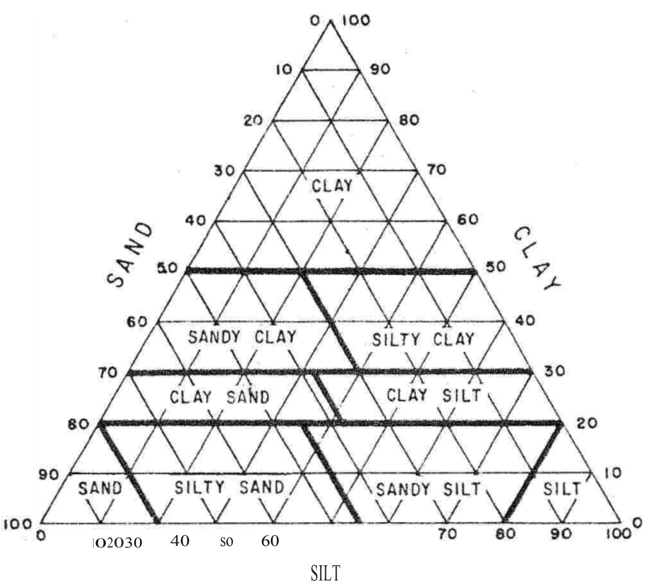
b) The soil classification requires basically the performance of three tests -- the mechanical analysis, determination of the liquid limit, and determination of the plastic limit. Tests for these properties have been utilized for many years as a means of evaluating soil for use in the construction of embankments and pavement subgrades. These tests identify a particular soil as having physical properties similar to those of a soil whose performance and behaviour are known. Therefore, the test soil can be expected to possess the same characteristics and degree of stability under like conditions of moisture and climate.

c) As can be discerned from Figure A4-l, the mechanical analyses provide the information to permit separation of the granular soils from the fine-grained soils, whereas the several groups are arranged in order of increasing values of liquid limit and plasticity index. The division between granular and fine grained soils is made upon the requirement that granular soils must have less than 35 per cent of silt and clay combined. Determination of the sand, silt, and clay fractions is made on that portion of the sample passing the No. 10 sieve because this is considered to be the critical portion with respect to changes in moisture and other climatic influences. The classification of the soils with respect to different percentages of sand, silt, and clay is shown in Figure A4-2.

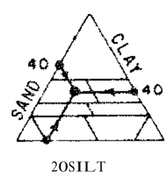
Soil group		Mechanical analysis				Liquid limit	Plasticity Index
		Material retained on No. 10 sieve (Percentage)*	Material finer than No. 10 sieve (Percentage)				
			Coarse sand, passing No. 10, retained on No. 40	Fine sand, passing No. 40, retained on No. 200	Combined silt and clay, passing No. 200		
Granular	E-1	0-45	40+	60-	15-	25-	6-
	E-2	0-45	15+	85-	25-	25-	6-
	E-3	0-45			25-	25-	6-
	E-4	0-45			35-	35-	10-
Fine grained	E-5	0-55			45-	40-	15-
	E-6	0-55			45+	40-	10-
	E-7	0-55			45+	50-	10-30
	E-8	0-55			45+	60-	15-40
	E-9	0-55			45+	40+	30-
	E-10	0-55			45+	70-	20-50
	E-11	0-55			45+	80-	30+
	E-12	0-55			45+	80+	-
	E-13	Muck and peat - field examination					

* If percentage of material retained on the No. 10 sieve exceeds that shown, the classification may be raised, provided such material is sound and fairly well graded.

Figure A4-l. Classification of soils for airport pavement construction



EXAMPLE: 20% Silt, 40% Sand and 40% Clay.



Therefore the sample is a Sandy Clay.

Figure A4-2. Textural classification of soils

1) Group E-1 includes well-graded, coarse, granular soils that are stable even under poor drainage conditions and are not generally subject to detrimental frost heave. Soils of this group may conform to well-graded sands and gravels with little or no fines. If frost is a factor, the soil should be checked to determine the percentage of the material less than 0.02 mm in diameter.

2) Group E-2 is similar to Group E-1 but has less coarse sand and may contain greater percentages of silt and clay. Soils of this group may become unstable when poorly drained as well as being subject to frost heave to a limited extent.

3) Groups E-3 and E-4 include the fine, sandy soils of inferior grading. They may consist of fine cohesion less sand or sand-clay types with a fair-to-good quality of binder. They are less stable than Group E-2 soils under adverse conditions of drainage and frost action.

4) Group E-5 comprises all poorly graded soils having more than35 percent but less than 45 per cent of silt and clay combined. This group also includes all soils with less than 45 percent of silt and clay but which have plasticity indices of 10 to 15. These soils are susceptible to frost action.

5) Group E-6 consists of the silts and sandy silts having zero-to-low plasticity. These soils are friable and quite stable when dry or at low moisture contents. They lose stability and become very spongy when wet and, for this reason, are difficult to compact unless the moisture content is carefully controlled. Capillary rise in the soils of this group is very rapid; and they, more than soils of any other group, are subject to detrimental frost heave.

6) Group E-7 includes the silty clay, sand clay, clayey sands, and clayey silts. They range from friable to hard consistency when dry and are plastic when wet. These soils are stiff and dense when compacted at the proper moisture content. Variations in moisture are apt to produce a detrimental volume change. Capillary forces acting in the soil are strong, but the rate of capillary rise is relatively slow and frost heave, while detrimental, is not as severe as in the E-6 soils.

7) Group E-8 soils are similar to the E-7 soils but the higher liquid limits indicate a greater degree of compressibility expansion, shrinkage, and lower stability under adverse moisture conditions.

8) Group E-9 comprises the silts and clays containing micaceous and diatomaceous materials. They are highly elastic and very difficult to compact. They have low stability in both the wet and dry state and are subject to frost heave.

9) Group E-10 includes the silty clay and clay soils that form hard clods when dry and are very plastic when wet. They are very compressible, possess the properties of expansion, shrinkage, and elasticity to a high degree and are subject to frost heave. Soils of this group are more difficult to compact than those of the E-7 or E-8 groups and require careful control of moisture to produce a dense, stable fill.

10) Group E-11 soils are similar to those of the E-10 group but have higher liquid limits. This group includes all soils with liquid limits between 70 and 80 and plasticity indices over 30.

11) Group E-12 comprises all soils having liquid limits over 80 regardless of their plasticity indices. They may be highly plastic clays that are extremely unstable in the presence of moisture, or they may be very elastic soils containing mica, diatoms, or organic matter in excessive amounts. Whatever the cause of their instability, they will require the maximum in corrective measures.

12) Group E-13 encompasses organic swamp soils such as muck and peat which are recognized by examination in the field. In their natural state, they are characterized by very low stability and density and very high moisture content.

1.3 Special conditions affecting fine-grained soils

a) A soil may possibly contain certain constituents that will give test results which would place it, according to Figure A4-1, in more than one group. This could happen with soils containing mica, diatoms, or a large proportion of colloidal material. Such overlapping can be avoided by the use of Figure A4-3 in conjunction with Figure A4-l, with exception of E-5 soils, which should be classified strictly by Figure A4-1.

b) Soils with plasticity indices higher than corresponding to the maximum liquid limit of the particular group are not of common occurrence. When encountered, they are placed in the higher numbered group as shown in Figure A4-3. This is justified by the fact that for equal liquid limits the higher the plasticity index, the lower the plastic limit (the plastic limit is the point when a slight increase in moisture causes the soil to rapidly lose stability).

1.4 Coarse material retained on No. 10 sieve

Only that portion of the sample passing the No. 10 sieve is considered in the above-described classification. Obviously, the presence of material retained on the No. 10 sieve should serve to improve the over-all stability of the soil. For this reason, upgrading the soil from 1 to 2 classes is permitted when the percentage of the total sample retained on the No. 10 sieve exceeds 45 per cent for soils of the E-1 to E-4 groups and 55 per cent for the others. This applies when the coarse fraction consists of reasonably sound material which is fairly well graded from the maximum size down to the No. 10 sieve size. Stones or rock fragments scattered through a soil should not be considered of sufficient benefit to warrant upgrading.

1.5 Subgrade classification

a) For each soil group, there are corresponding subgrade classes.
These classes are based on the performance of the particular soil as a subgrade for rigid or flexible pavements under different conditions of drainage and frost. The subgrade class is determined from the results of soil tests and the information obtained by means of the soil survey and a study of climatological and topographical data. The subgrade classes and their relationship to the soil groups are shown in Figure A4-4.The prefix "F" indicates sub grade classes for flexible pavements. These subgrade classes determine the total pavement thickness for a given aircraft load. A brief description of the classes will be presented here.

b) Subgrades classed as Fa furnish adequate subgrade support without the addition of sub-base material. The soil's value as a subgrade material decreases as the number increases.

c) Good and poor drainage refer to the subsurface soil drainage.

1) Poor drainage is defined for the purpose of this manual as soil that cannot be drained because of its composition or because of the conditions at the site. Soils primarily composed of silts and clay for all practical purposes are impervious; and as long as a water source is available, the soil's natural affinity for moisture will render these materials unstable. These fine-grained soils cannot be drained and are classified as poor drainage as indicated in Figure A4-4. A granular soil that would drain and remain stable except for conditions at the site, such as high water table, flat terrain, or impervious strata, should also be designated as poor drainage. In some cases, this condition may be corrected by the use of sub drains.

2) Good drainage is defined as a condition where the internal soil drainage characteristics are such that the material can and does remain well drained resulting in a stable subgrade material under all conditions.

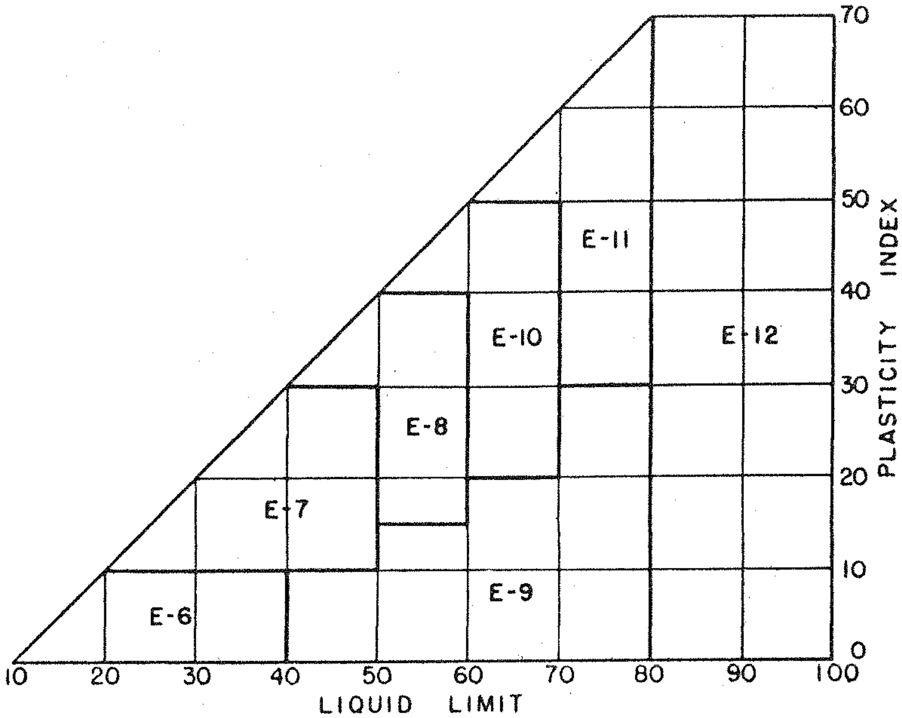


Figure A4-3. Classification chart for fine-grained soils

Soil Group	Subgrade Class		
	Good Drainage	Poor Drainage	
	No Frost or Frost	No Frost	Frost
E-1	Fa	Fa	F1
E-2	Fa	F1	F2
E-3	F1	F2	F3
E-4	F1	F2	F4
E-5		F3	F5
E-6		F4	F6
E-7		F5	F7
E-8		F6	F8
E-9		F7	F9
E-10		F8	F10
E-11		F9	F10
E-12		F10	F10
E-13	Not suitable for subgrade		

Figure A4-4. Airport paving subgrade classification

d) There is a tendency to overlook the detrimental effects of frost in pavement design. The effects of frost are widely known; however, experience shows that all too often pavements are damaged or destroyed by frost that was not properly taken into account in the design. Most inorganic soils containing 3per cent or more of grains finer than 0.02 mm in diameter, by weight, are frost susceptible for pavement design purposes. The sub grade soil should be classified either as "No Frost" or “Frost" depending on one of the two following conditions:

1) No frost should be used in the design when the average frost penetration anticipated is less than the thickness of the pavement section.

2) Frost should be used when the anticipated average frost penetration exceeds the pavement sections. The design should consider including non-frost susceptible material below the required sub-base to minimize or eliminate the detrimental frost effect on the subgrade. The extent of the subgrade protection needed depends on the soil and the surface and subsurface environment at the site.

2. Development of pavement design curves

2 .1 Background

a) The pavement design curves presented in Chapter 4, 4.4 of this manual were developed using the California Bearing Ratio (CBR) method for flexible pavements and the Westergaard edge loading analysis for rigid pavements. The curves are constructed for the gross weight of the aircraft assuming 95 per cent of the gross weight is carried on the main landing gear assembly and 5 per cent of the gross weight is carried on the nose gear assembly. Aircraft traffic is assumed to be normally distributed across the pavement in the transverse direction. Pavements are designed on the basis of static load analysis. Impact loads are not considered to increase the pavement thickness requirements.

b) Generalized design curves have been developed for single, dual, and dual tandem main landing gear assemblies. These generalized curves do not represent specific aircraft but are prepared for a range of aircraft characteristics which are representative of all civil aircraft except wide body. The aircraft characteristics assumed for each landing gear assembly are shown in Table A4-l, A4-2 andA4-3.

2.2 Flexible pavements

a) The design curves for flexible pavements are based on the CBR method of design. The CBR is the ratio of the load required to produce a specified penetration of a standard piston into the material in question to the load required to produce the same penetration in a standard well-graded, crushed limestone. Pavement thicknesses necessary to protect various CBR values from shear failure have been developed through test track studies and observations of in-service pavements. These thicknesses have been developed for single wheel loadings. Assemblies other than single wheel are designed by computing the equivalent single wheel load for the assembly based on deflection. Once the equivalent single wheel is established, the pavement section thickness can be determined from the relationships discussed above.

b) Load repetitions are indicated on the design curves in terms of annual departures. The annual departures are assumed to occur over a 20-year life. In the development of the design curves, departures are converted to coverage. 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. One coverage occurs when all points on the pavement surface within the traffic lane have been subjected to one application of maximum stress, assuming the stress is equal under the full tire print.
Each pass (departure) of an aircraft can be converted to coverages using a single pass-to-coverage ratio which is developed assuming a normal distribution and applying standard statistical techniques. The pass-to-coverage ratios used in developing the flexible pavement design curves are given in Table A4-4. Annual departures are converted to coverages by multiplying by 20 and dividing that product by the pass-to-coverage ratio given in Tables A4-4. Figure A4-5 shows the relationship between load repetition factor and coverages. The pavement section thickness determined in accordance with a) above is multiplied by the appropriate load repetition factor (Figure A4-5) to give the final pavement thickness required for various traffic levels.

Table A4-1. Single wheel assembly

Gross mass lb	(kg)	Tire pressure psi	(MN/m^2)
30000	(13600)	75	(0.52)
45000	(20400)	90	(0.62)
60000	(27200)	105	(0.72)
75000	(34000)	120	(0.83)

Table A4-2. Dual wheel assembly

Gross Mass lb	(kg)	Tire pressure psi	(MN/m^2)	Dual spacing in	(cm)
75000	(34000)	110	(0.76)	21	(53)
100000	(45400)	140	(0.97)	23	(58)
150000	(68000)	160	(1.10)	30	(76)
200000	(90700)	200	(1.38)	34	(86)

Table A4-3. Dual tandem assembly

Gross mass lb	(kg)	Tire pressure psi	(MN/m^2)	Dual spacing in	(cm)	Tandem spacing in	(cm)
100000	(45400)	120	(0.83)	20	(51)	45	(114)
150000	(68000)	140	(0.97)	20	(51)	45	(114)
200000	(90700)	160	(1.10)	21	(53)	46	(117)
300000	(136100)	180	(1.24)	26	(66)	51	(130)
400000	(181400)	200	(1.38)	30	(76)	55	(140)

Specific design curves are presented for wide body aircraft. The aircraft characteristics are shown on the design curves.

Table A4-4. Pass-to-coverage ratios for flexible pavements

Design curve	Pass-to-coverage ratio
Single wheel	5.18
Dual wheel	3.48
Dual tandem	1.84
B-747	1.85
DC 10-10	1.82
DC 10-30	1.69
L-1011	1.81

Figure A4-5. Load repetition factor vs. coverages

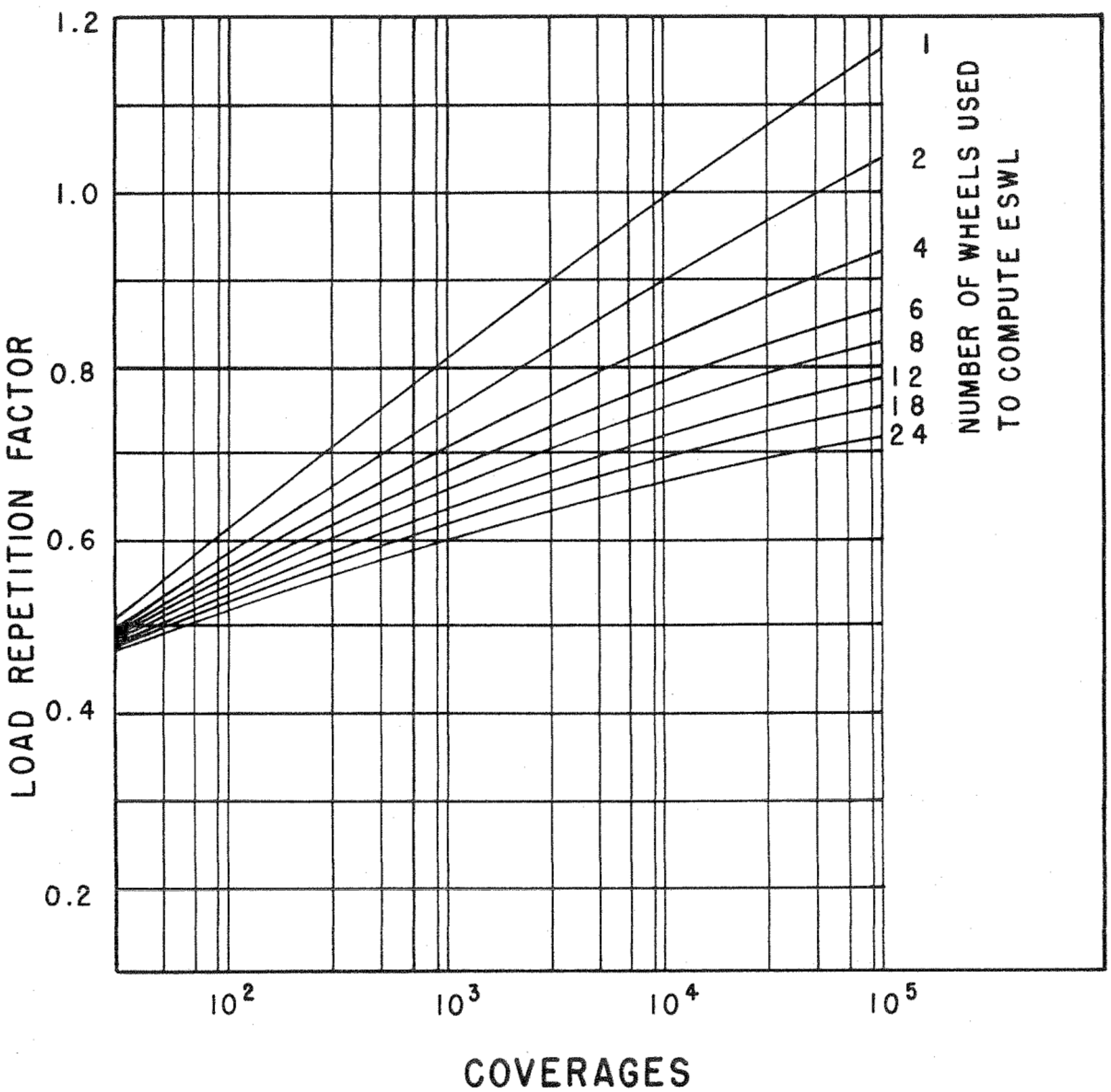
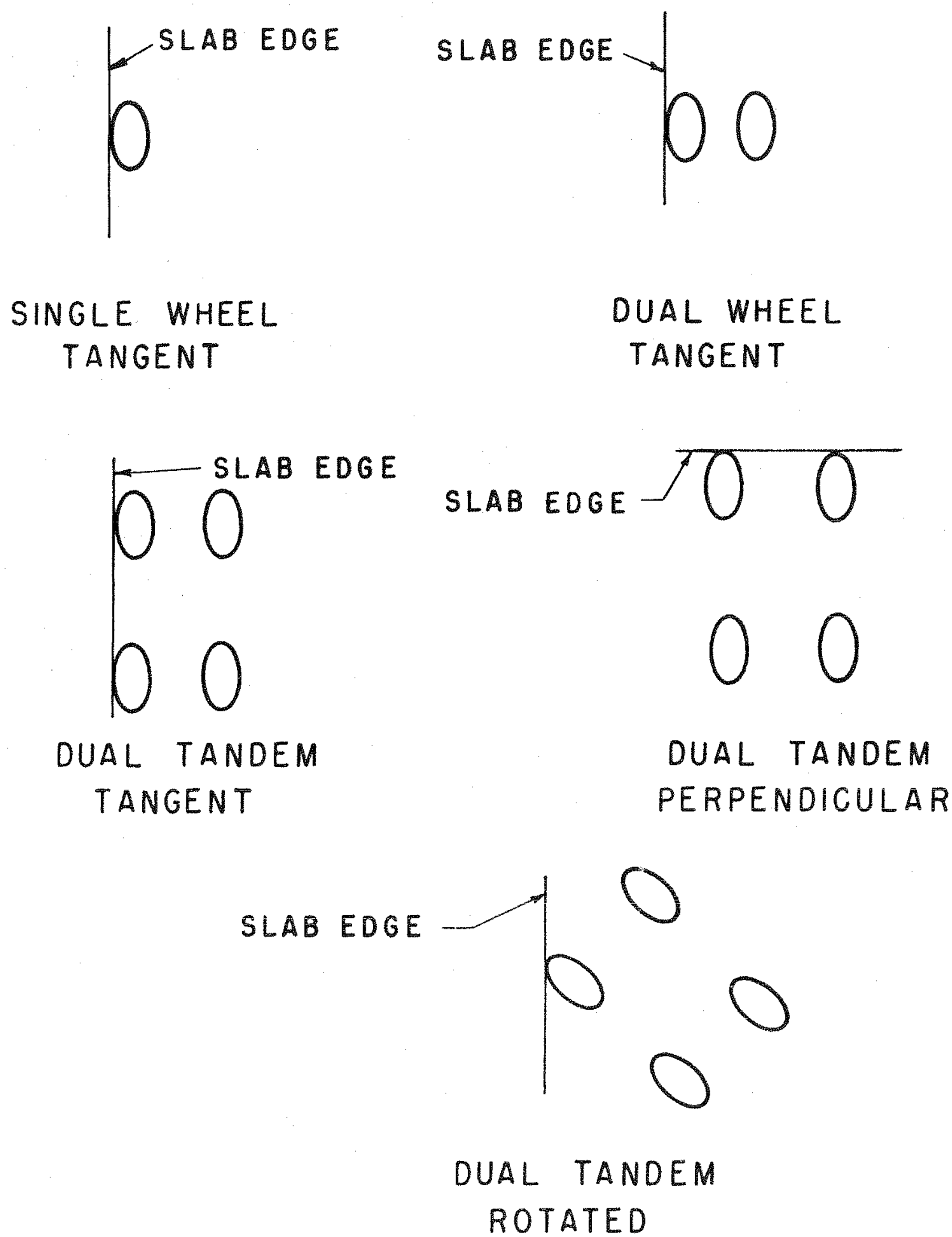


Figure A4-6. Assembly positions for rigid pavement analysis



2.3 Rigid pavements

a) The design of rigid airport pavements is based on the Westergaard analysis of an edge loaded slab resting on a dense liquid foundation. The edge loading stresses are reduced by 25 per cent to account for load transfer across joints. Two different cases of edge loading are covered by the design curves. Figures 4-46 to 4-54 of Chapter 4 assume the landing gear assembly is either tangent to a longitudinal joint or perpendicular to a transverse joint, whichever produces the largest stress. Figures 4-56 to 4-62 of the same chapter are for dual tandem assemblies and have been rotated through an angle to produce the maximum edge stress. Computer analyses were performed for angles from 0 to 90 degrees in 10-degree increments. Single and dual wheel assemblies were analysed for loadings tangent to the edge only as the stress is maximum in that position. Sketches of the various assembly positions are shown in Figure A4-6.

b) Fatigue effects are taken into consideration by converting traffic to coverages. The coverage concept provides a means of normalizing pavement performance data which can consist of a variety of wheel sizes, spacings and loads for pavements of different cross sections. For rigid pavements, coverage is a measure of the number of maximum stress applications occurring within the pavement slab due to the applied traffic. One coverage occurs when each point in the pavement within the limits of the traffic lane has experienced a maximum stress, assuming the stress is equal under the full tire print. Each pass (departure) of an aircraft can be converted to coverages using a single pass-to-coverage ratio which is developed assuming a normal distribution and applying standard statistical techniques. The pass-to-coverage ratios used in developing the rigid pavement design curves are given in Table A4-5. Annual departures are converted to coverages assuming a 20-yeardesign life. Coverages are determined by multiplying annual departures by 20 and dividing that product by the pass-to-coverage ratio shown in Table A4-5.

Table A4-5. Pass-to-coverage ratios for rigid pavements

Design curve	Pass-to-coverage ratio
Single wheel	5.18
Dual wheel	3.48
Dual tandem	3.68
B-747	3.70
DC 10-10	3.64
DC 10-30	3.38
L-1011	3.62

c) After the conversion of departures to coverages, the slab thickness is adjusted in accordance with the fatigue curve developed by the Corps of Engineers from test track data and observation of in-service pavements. The fatigue relationship is applicable to the pavement structure; i.e., the slab and foundation are both included in the relationship. The thickness of pavement required to sustain5000coverages of the design loading is considered to be 100 per cent thickness. Any coverage level could have been selected as the100 per cent thickness level as long as the relative thicknesses for other coverage levels shown in Figure A4-7 is maintained.

d) Pavement thickness requirements for 5 000 coverages were computed for various concrete strengths and subgrade moduli. Allowable concrete stress for 5000 coverages was computed by dividing the concrete flexural strength by 1.3 (analogous to a safety factor). The pavement thickness necessary to produce the allowable concrete stress for 5 000 coverages is then multiplied by the percentage thickness shown in Figure A4-7 for other coverage levels.

3. Prior FAA pavement evaluation curves

3.1 To facilitate the pavement evaluation policy described in Chapter 4, 4.4.27.2 the evaluation curves used by the FAA previously are reproduced as Figures A4-8 to A4-21of this Appendix.

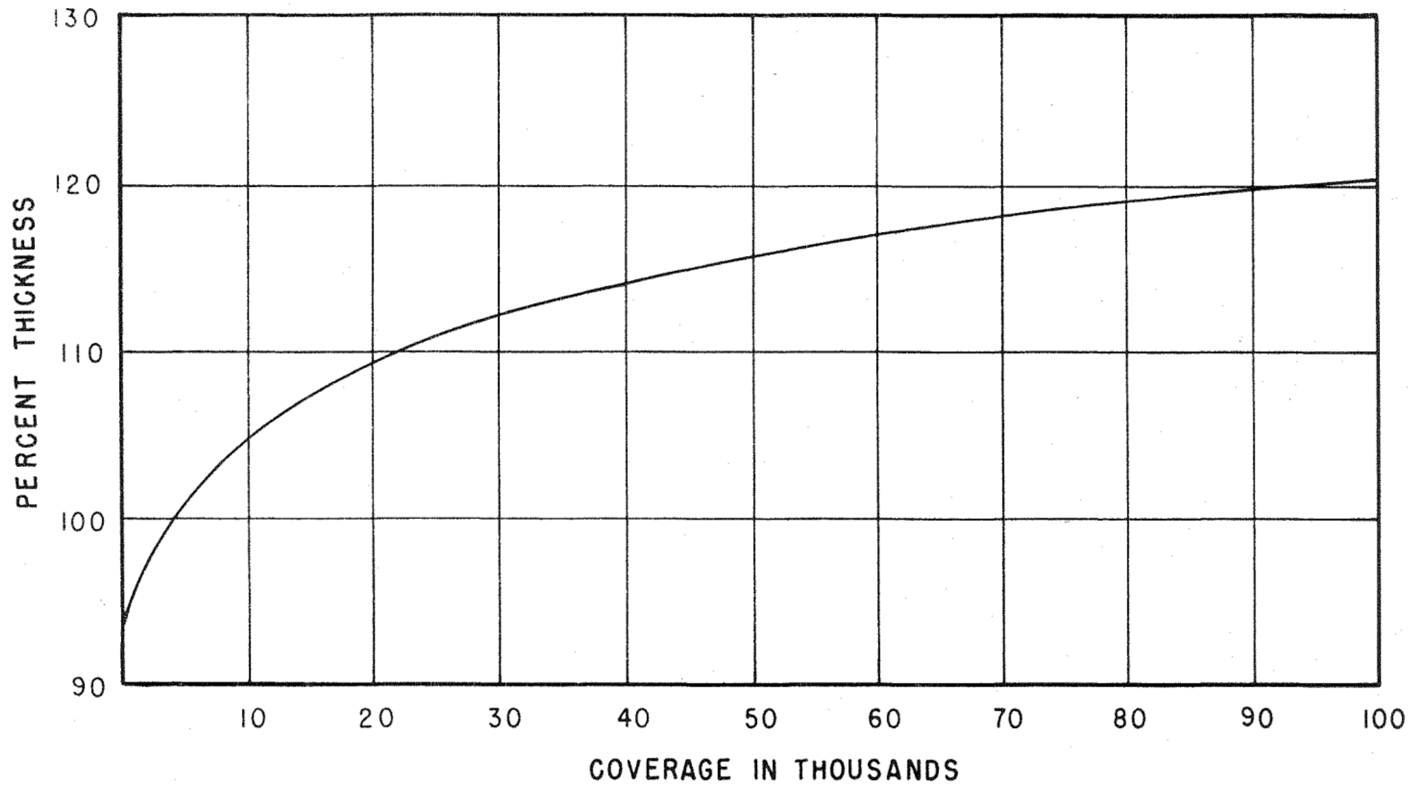


Figure A4-7.Percentage thickness vs. coverages

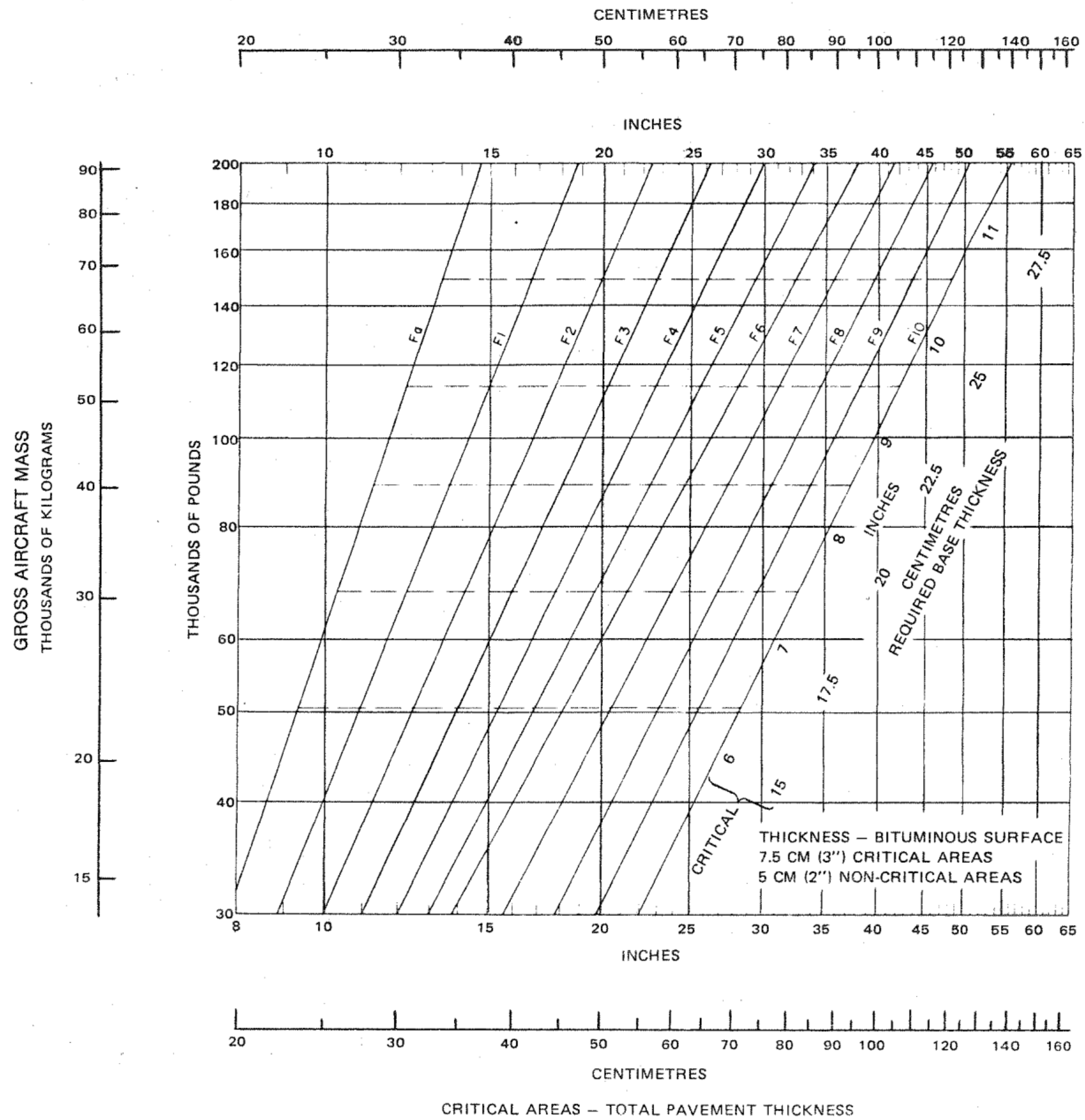


Figure A4-8. Flexible pavement evaluation curves - single wheel gear

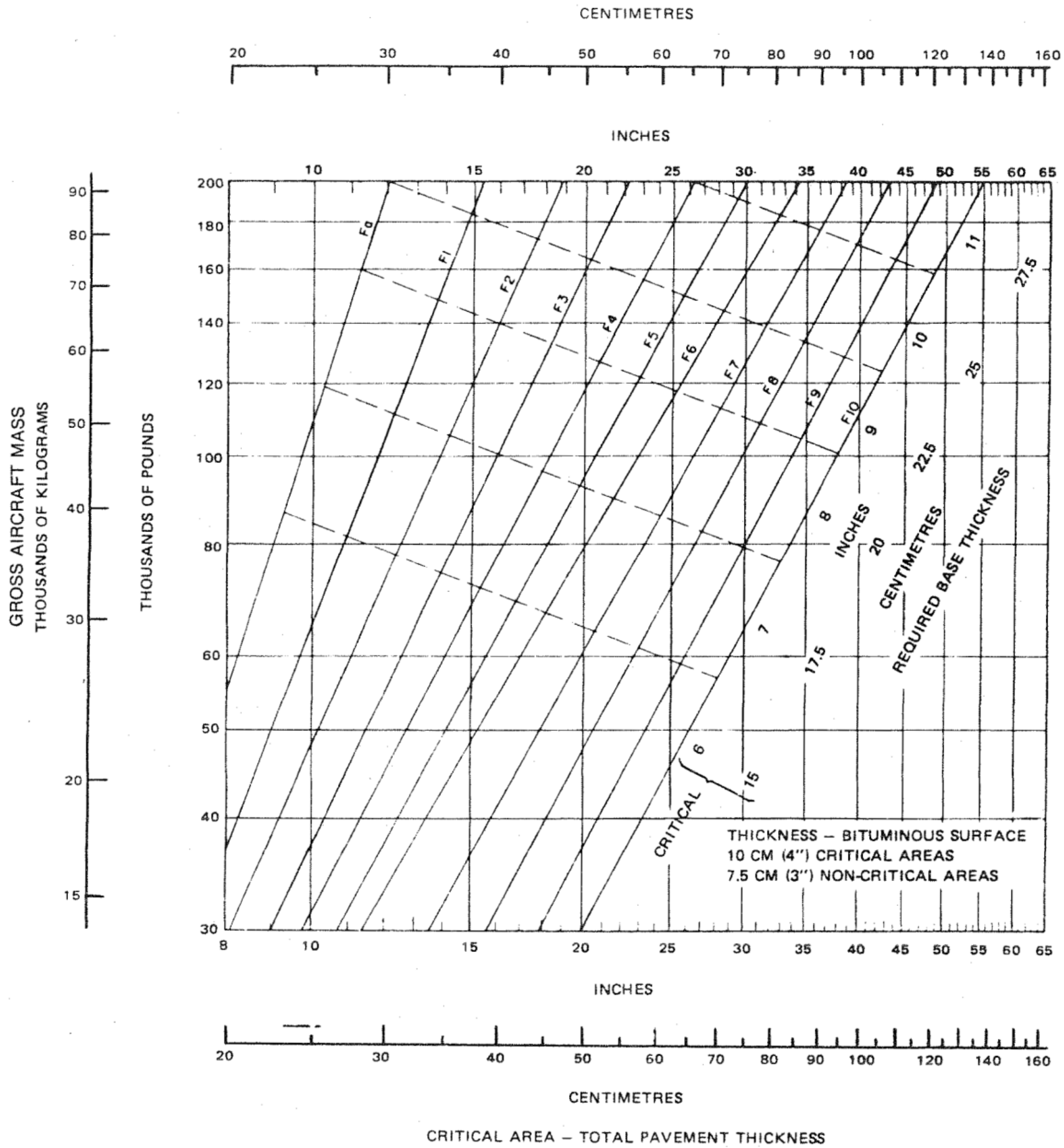


Figure A4-9. Flexible pavement evaluation curves - dual wheel gear

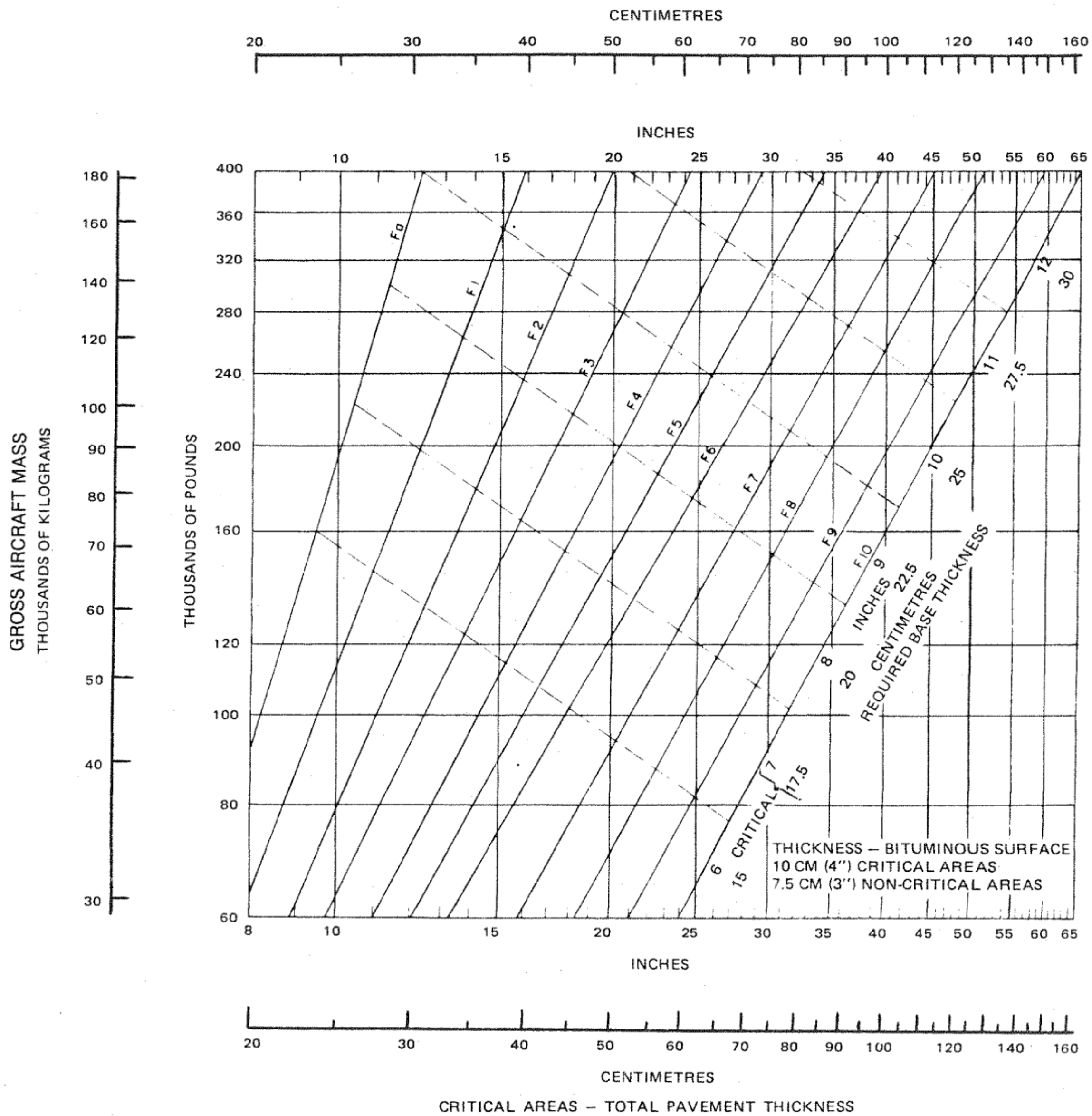


Figure A4-10. Flexible pavement evaluation curves - dual tandem gear

Figure A4-ll. Flexible pavement evaluation curves - B-747

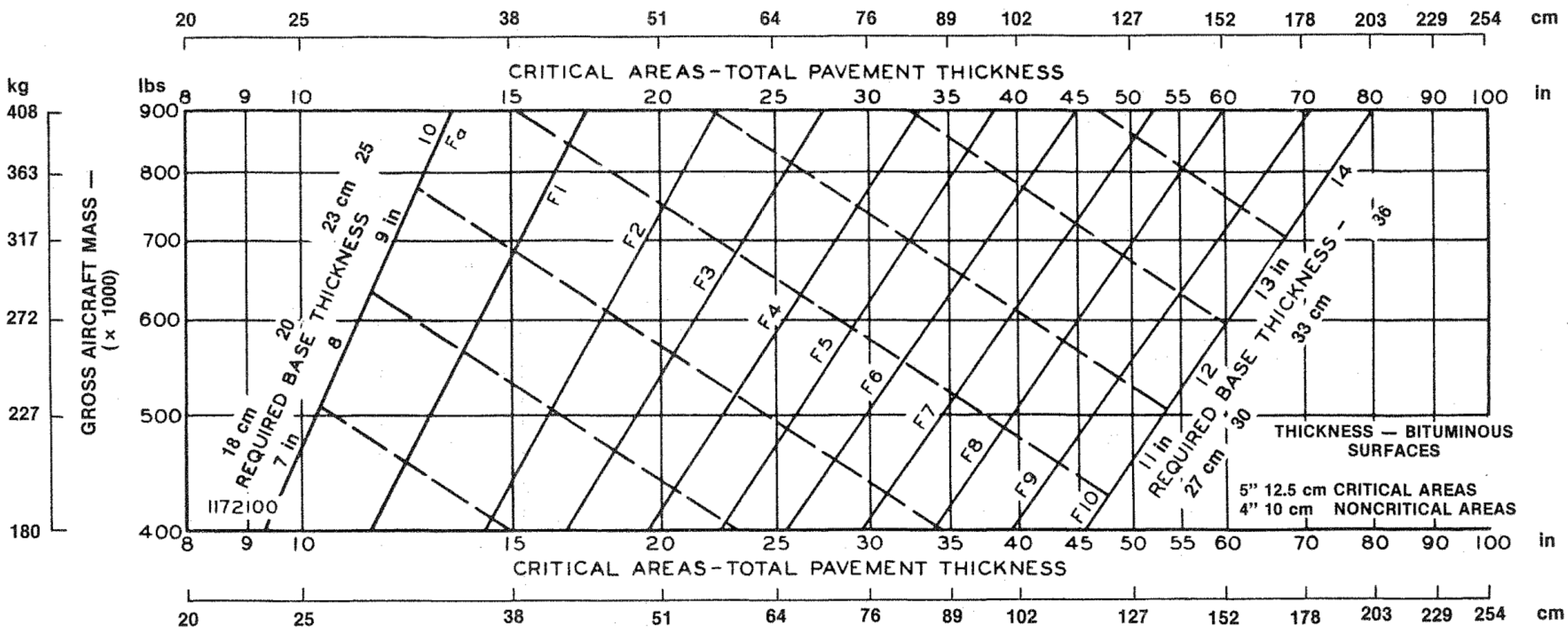
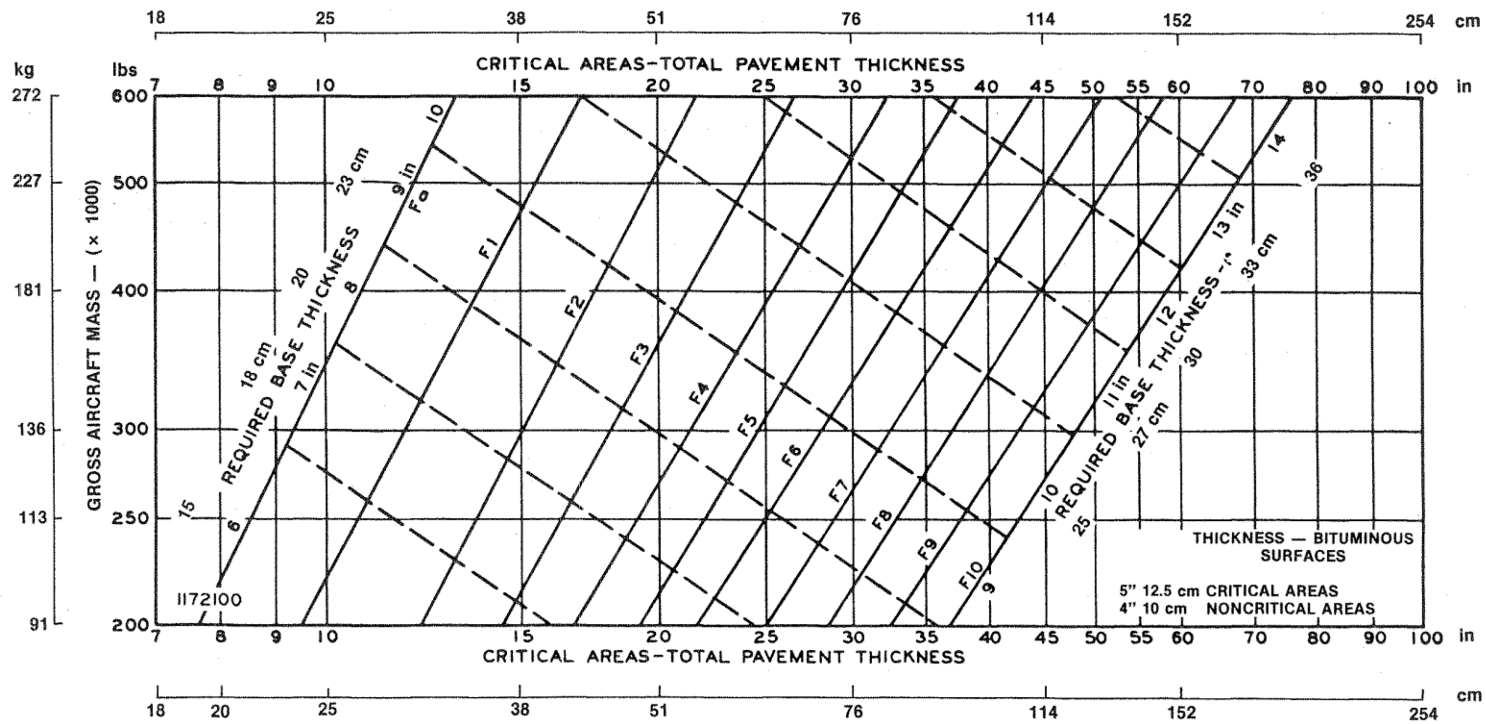


Figure A4-12. Flexible pavement evaluation curves - DCl0-10



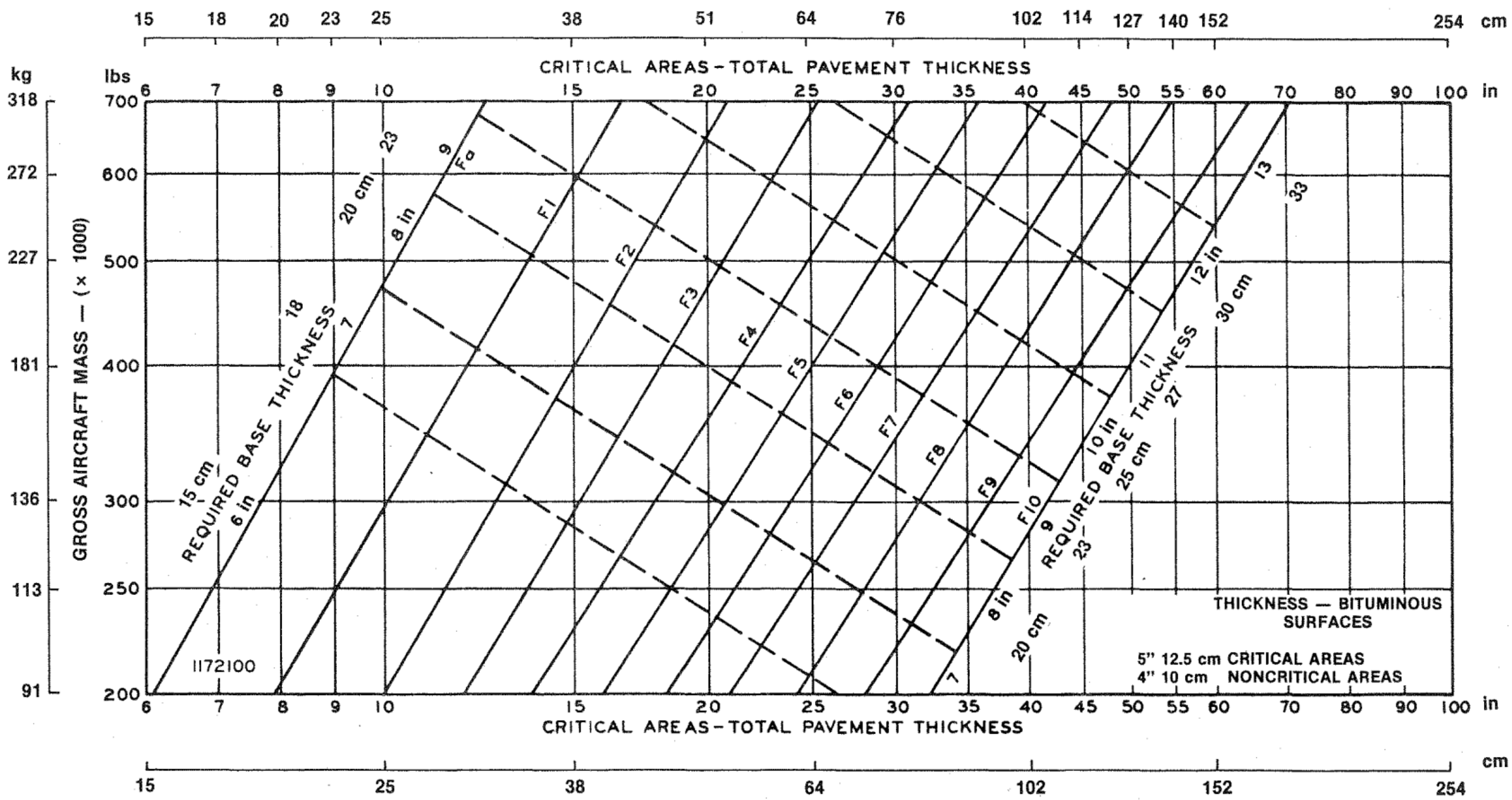


Figure A4-13. Flexible pavement evaluation curves - DCl0-10

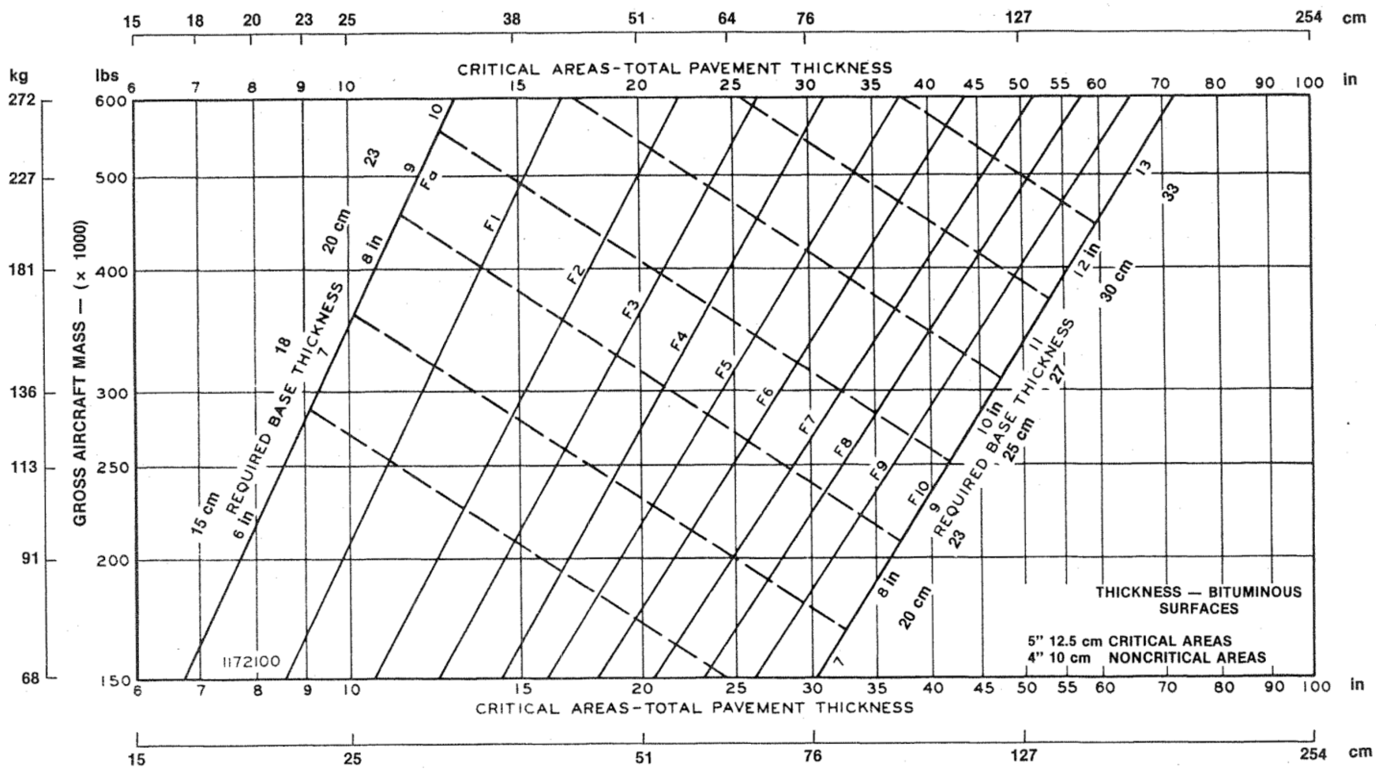
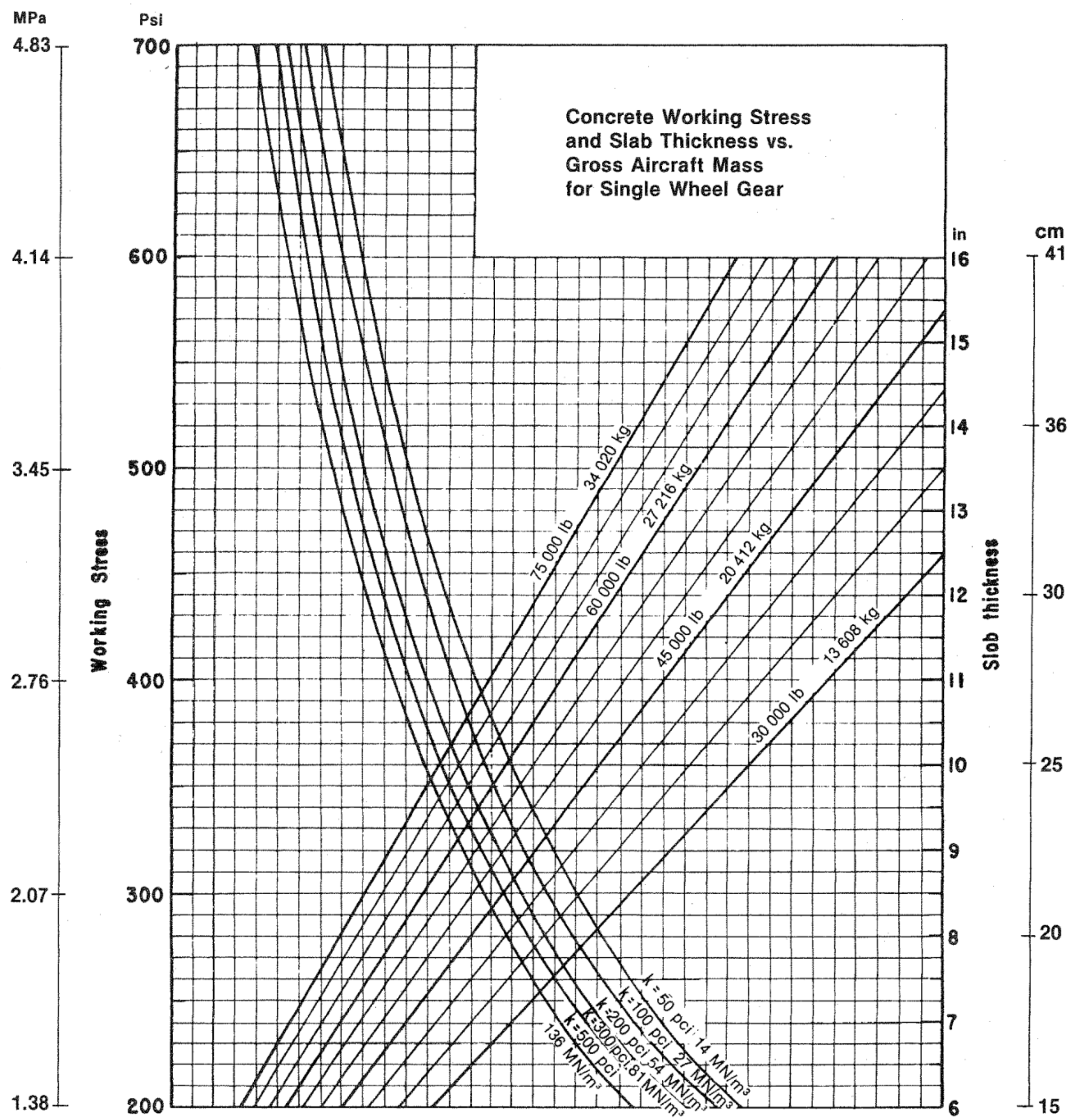


Figure A4-14. Flexible pavement evaluation curves - L-1011

Figure A4-15. Rigid pavement evaluation curves - single wheel gear



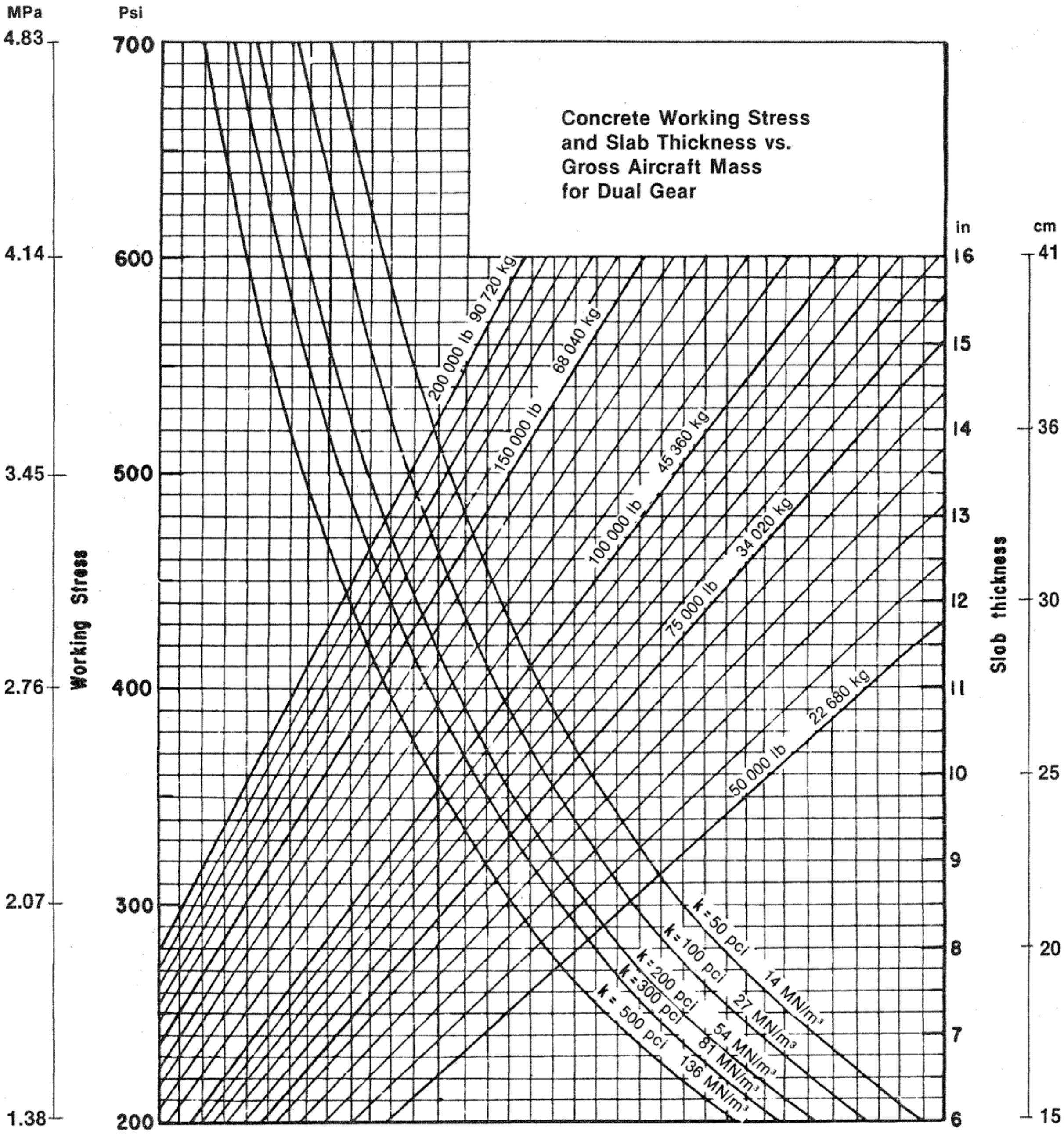


Figure A4-16. Rigid pavement evaluation curves - dual wheel gear

Figure A4-17. Rigid pavement evaluation curves - dual tandem gear

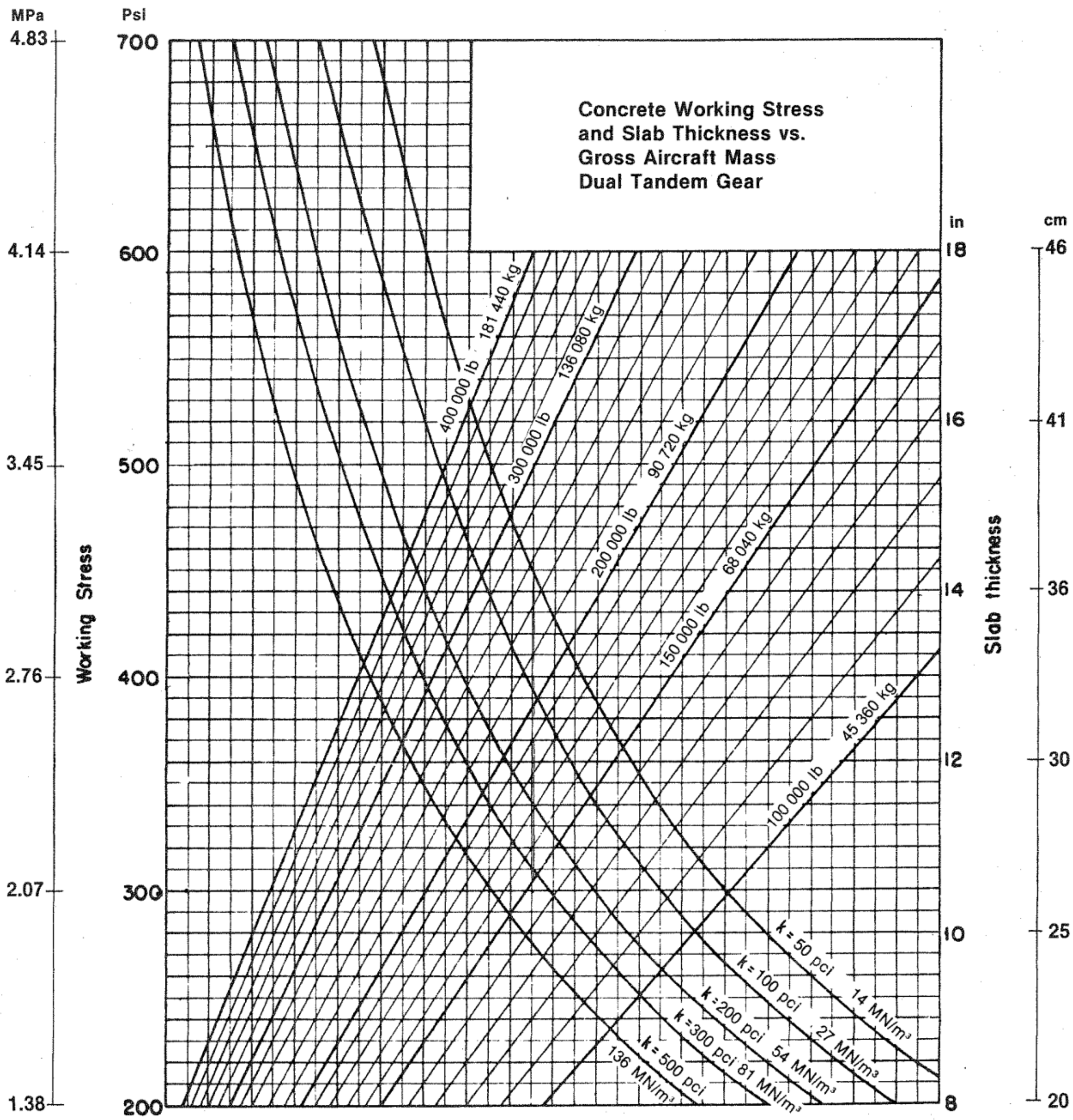
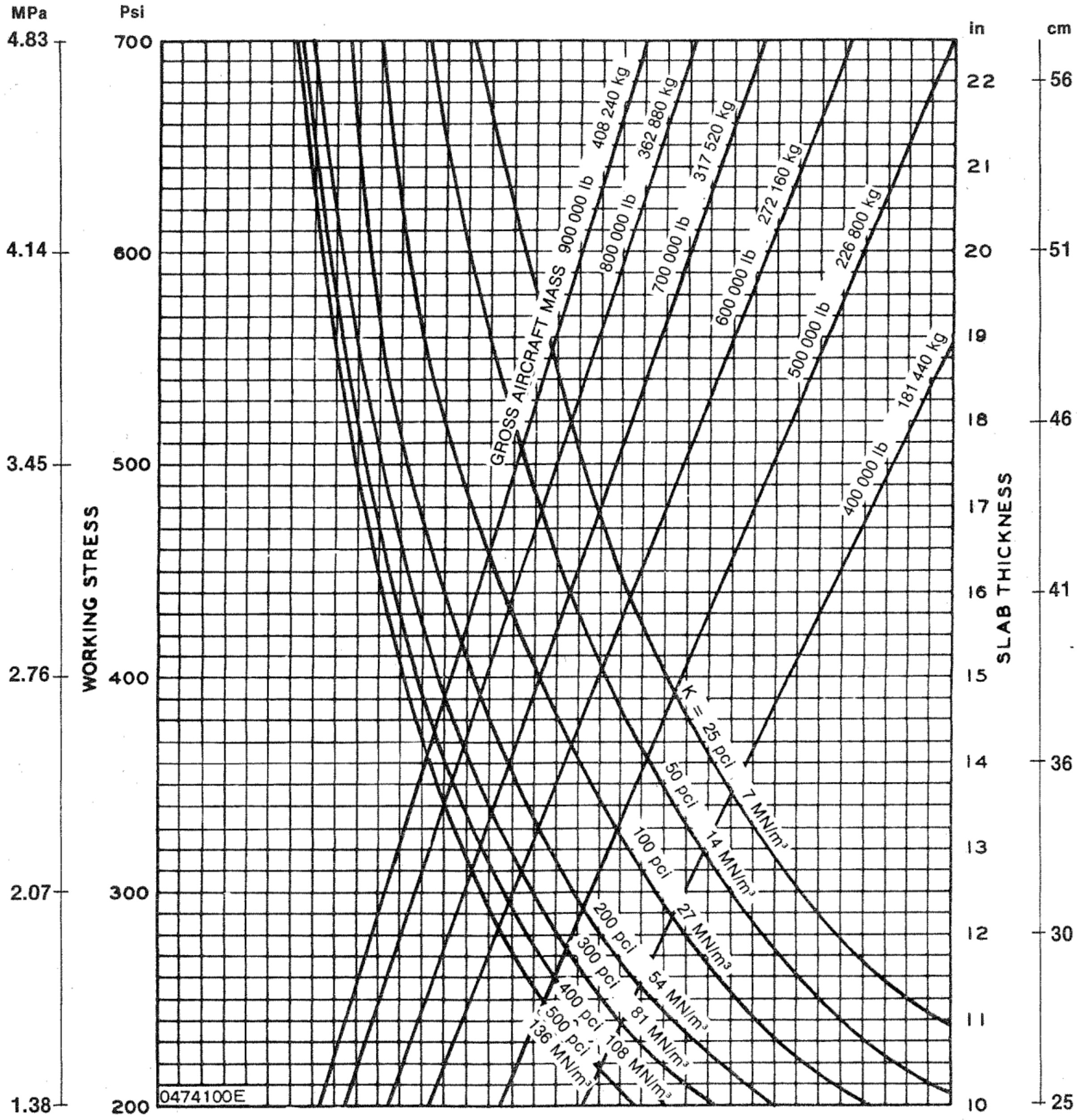


Figure A4-18. Rigid pavement evaluation curves - B-747



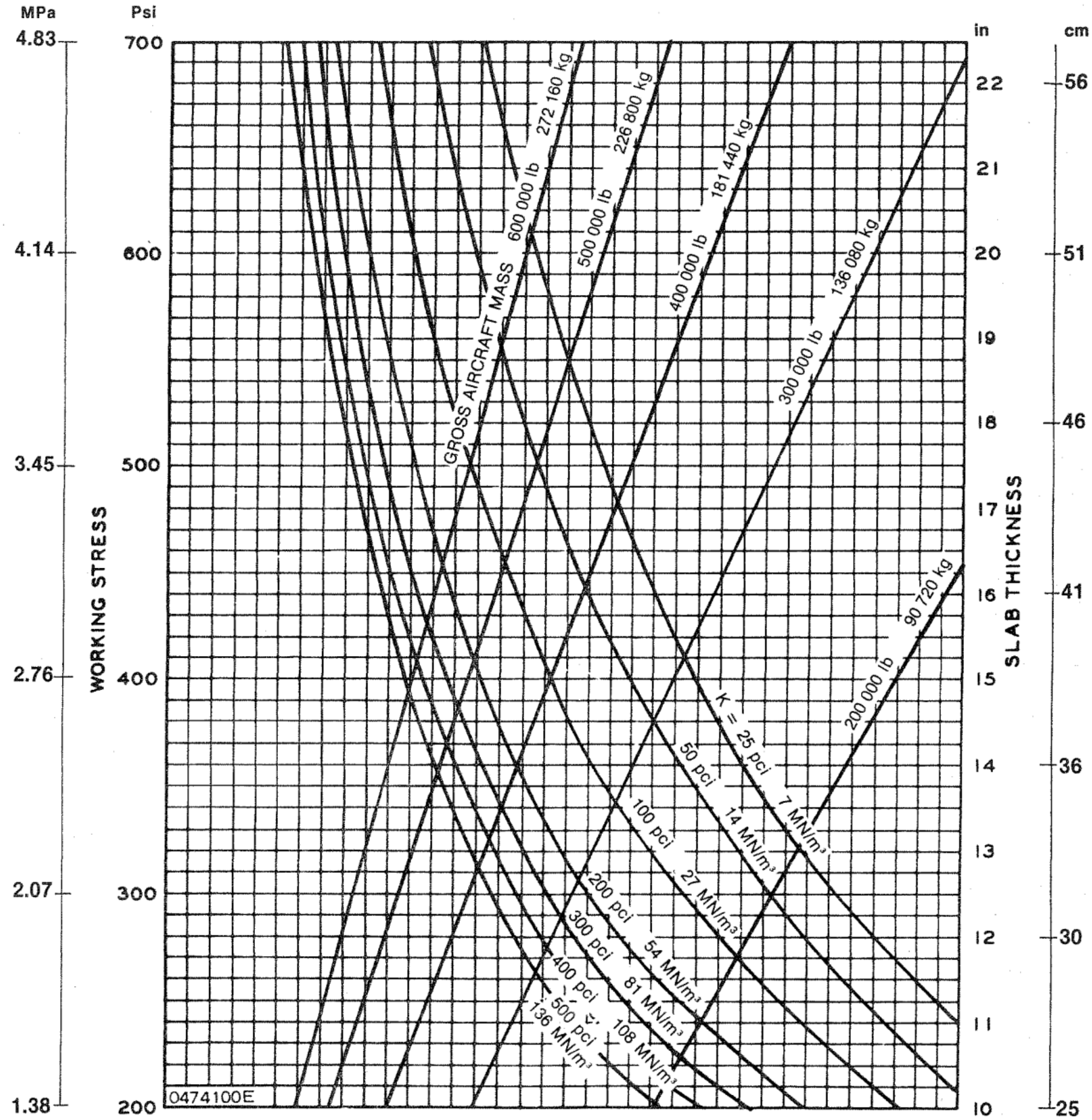


Figure A4-19. Rigid pavement evaluation curves - DC 10-10

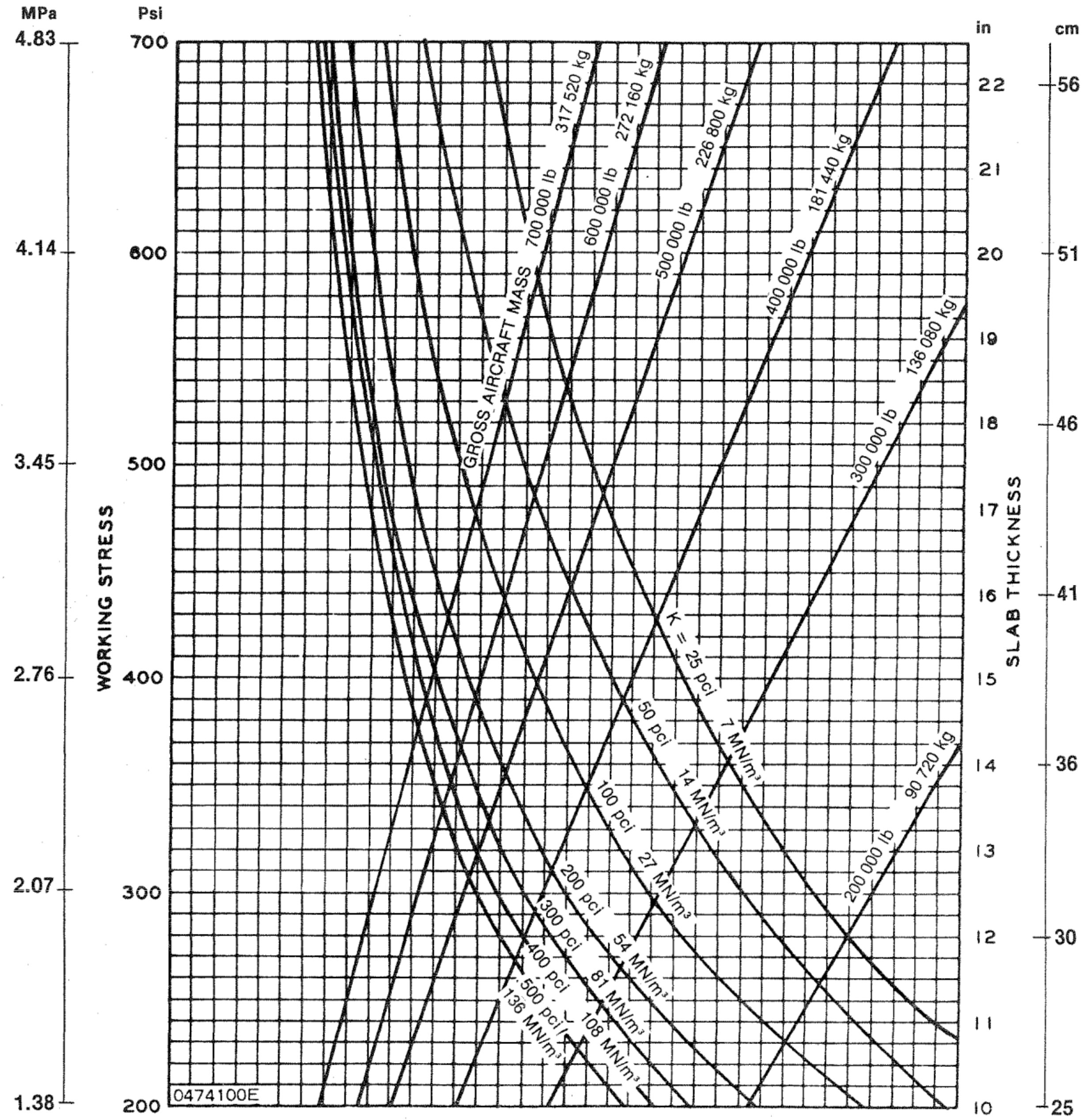


Figure A4-20. Rigid pavement evaluation curves - DC 10-30

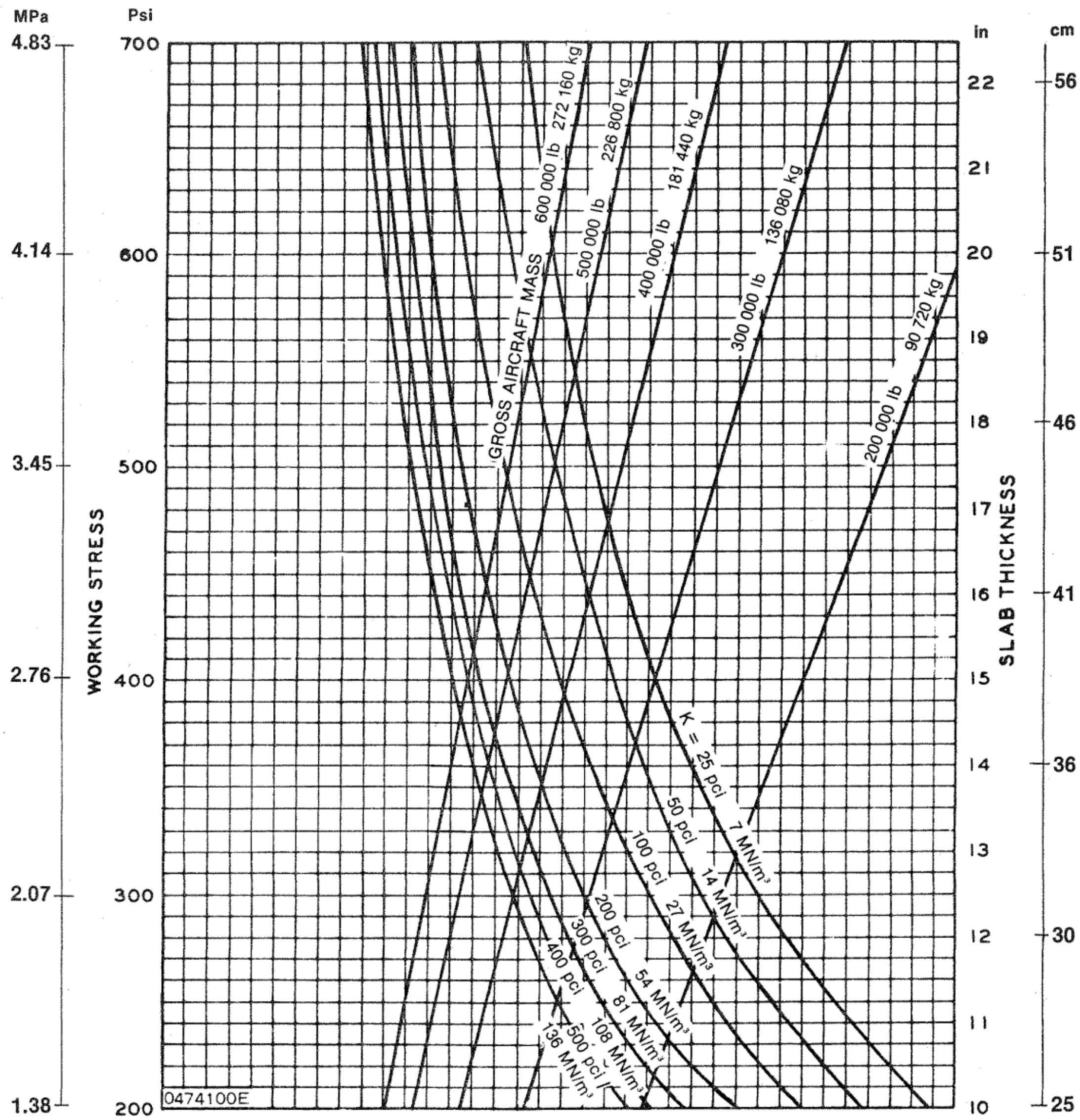


Figure A4-21. Rigid pavement evaluation curves - L-1011

APPENDIX 5: - ACNS FOR SEVERAL AIRCRAFT TYPES

1. Introduction

1.1 For convenience, several aircraft types currently in use have been evaluated on rigid and flexible pavements using the computer programmes in Appendix 2 and the results tabulated in Table A5-1. The two all-up masses shown in column 2 for each aircraft type are respectively the maximum apron (ramp) mass and a representative operating mass empty. To compute the ACN for any intermediate value, proceed on the assumption that the ACN varies linearly between the operating mass empty and the maximum apron mass.

Table A5.1. ACNs for several aircraft types on rigid and flexible pavements

Aircraft type	All-up mass (kg)	Load on one main gear leg (%)	Tire pressure (MPa)	ACN FOR RIGID PAVEMENT SUBGRADES – MN/m^3				ACN FOR FLEXIBLE PAVEMENT SUBGRADES – CBR			
				High 150	Medium 80	Low 40	Ultra-low 20	High 15	Medium 10	Low 6	Very low 3
1	2	3	4	5	6	7	8	9	10	11	12
A300 B2 Airbus	137 000/85 910	47.0	1.2	35/18	42/21	50/25	58/29	39/20	43/22	53/24	68/34
A300 B2 Airbus	142 000/85 910	47.0	1.29	35/19	45/22	53/26	61/30	40/21	45/22	55/25	71/34
A300 B4 Airbus	150 000/88 180	47.0	1.39	41/20	49/22	57/26	65/31	43/21	49/22	59/25	76/35
A300 B4 Airbus	157 000/88 330	47.0	1.48	45/20	53/22	62/26	70/31	46/21	52/22	63/25	80/36
A300 B4 Airbus	165 000/88 505	47.0	1.29	46/17	55/20	64/25	73/29	49/20	56/21	68/25	84/36
A300-600 Airbus	165 000/87 100	47.0	1.29	46/17	55/19	64/24	73/28	49/19	56/21	68/24	84/35
A300-600R Airbus	170 000/85 033	47.4	1.35	49/17	58/19	68/23	78/28	52/19	58/20	71/23	89/34
A300-600R Airbus	171 700/85 033	47.4	1.35	50/17	59/19	69/23	79/28	52/19	59/20	72/23	90/34
A310-200 Airbus	132 000/76 616	46.7	1.23	33/15	39/18	46/21	54/24	36/18	40/19	48/20	64/27
A310-200 Airbus	138 600/76 747	46.7	1.3	35/16	42/18	51/21	58/25	39/18	43/19	52/20	68/28
A310-200 Airbus	142 000/75 961	46.7	1.33	37/15	44/17	52/20	60/23	40/17	44/18	54/20	70/27
A310-300 Airbus	150 000/77 037	47.0	1.42	42/13	49/14	58/17	66/20	44/15	49/15	59/16	76/24
A310-300 Airbus	157 000/78 900	47.4	1.49	45/14	54/15	63/18	71/22	47/15	53/15	64/16	81/25
A320-100 Airbus Dual	66 000/37 203	47.1	1.28	37/19	40/20	42/21	44/23	33/18	34/18	38/19	44/22
A320-100 Airbus Dual	68 000/39 700	47.1	1.34	39/20	41/22	43/23	45/24	35/19	36/19	40/20	46/23
A320-100 Airbus Dual Tandem	68 000/40 243	47.1	1.12	18/9	21/10	24/12	28/14	18/9	19/10	23/11	32/14

Aircraft type	All-up mass (kg)	Load on one main gear leg (%)	Tire pressure (MPa)	ACN FOR RIGID PAVEMENT SUBGRADES – MN/m^3 High 150	Medium 80	Low 40	Ultra-low 20	ACN FOR FLEXIBLE PAVEMENT SUBGRADES – CBR High 15	Medium 10	Low 6	Very low 3
1	2	3	4	5	6	7	8	9	10	11	12
A320-200 Airbus Dual	73 500 / 39 748	47.0	1.45	44 / 20	46 / 22	48 / 23	50 / 25	38 / 19	40 / 19	44 / 20	50 / 24
A320-200 Airbus Dual Tandem	73 500 / 40 291	47.0	1.21	18 / 9	22 / 10	26 / 11	30 / 13	19 / 9	21 / 10	26 / 11	35 / 14
BAC 1-11 Series 400	39 690 / 22 498	47.5	0.93	25 / 13	26 / 13	28 / 14	29 / 15	22 / 11	24 / 12	27 / 13	29 / 15
BAC 1-11 Series 475	44 679 / 23 451	47.5	0.57	22 / 10	25 / 11	27 / 12	28 / 13	19 / 9	24 / 10	28 / 12	31 / 15
BAC 1-11 Series 500	47 400 / 24 757	47.5	1.08	32 / 15	34 / 16	35 / 16	36 / 17	29 / 13	30 / 13	33 / 15	35 / 17
BAe 146 Series 100	37 308 / 23 000	46.0	0.80	18 / 10	20 / 11	22 / 12	23 / 13	17 / 10	18 / 10	20 / 11	24 / 13
BAe 146 Series 100	37 308 / 23 000	46.0	0.52	16 / 9	18 / 10	19 / 11	21 / 12	13 / 8	16 / 9	19 / 11	23 / 13
BAe 146 Series 200	40 600 / 23 000	47.1	0.88	22 / 11	23 / 12	25 / 13	26 / 14	19 / 10	21 / 10	23 / 11	27 / 13
BAe 146 Series 200	40 600 / 23 000	47.1	0.61	19 / 10	21 / 11	23 / 12	24 / 12	16 / 8	20 / 10	22 / 11	27 / 13
B707-120B	117 027 / 57 833	46.7	1.17	28 / 12	33 / 12	39 / 15	46 / 17	31 / 13	34 / 14	41 / 15	54 / 20
B707-320B	148 778 / 64 764	46.0	1.24	38 / 13	46 / 14	54 / 17	62 / 20	42 / 15	47 / 15	57 / 17	72 / 22
B707-320C (Freighter)	152 407 / 61 463	46.7	1.24	40 / 13	48 / 14	57 / 16	66 / 19	44 / 14	49 / 15	60 / 17	76 / 21
B707-320C (Convertible)	152 407 / 67 269	46.7	1.24	40 / 14	48 / 15	57 / 18	66 / 21	44 / 16	49 / 17	60 / 19	76 / 24
B707-320/420	143 335 / 64 682	46.0	1.24	36 / 13	43 / 14	52 / 17	59 / 20	40 / 15	44 / 15	54 / 17	69 / 22
B720	104 326 / 50 258	47.4	1.00	25 / 10	30 / 11	37 / 13	42 / 16	29 / 11	31 / 12	39 / 14	51 / 18
B720 B	106 594 / 52 163	46.4	1.00	25 / 10	30 / 11	37 / 13	42 / 16	29 / 11	31 / 12	39 / 14	51 / 18
B727-100	77 110 / 41 322	47.6	1.14	46 / 22	48 / 23	51 / 25	53 / 26	41 / 20	43 / 20	49 / 22	54 / 26
B727-100C	73 028 / 41 322	47.8	1.09	43 / 22	45 / 23	48 / 25	50 / 26	39 / 20	40 / 21	46 / 22	51 / 26

Aircraft type	All-up mass (kg)	Load on one main gear leg (%)	Tire pressure (MPa)	ACN FOR RIGID PAVEMENT SUBGRADES – MN/m³ High 150	Medium 80	Low 40	Ultra-low 20	ACN FOR FLEXIBLE PAVEMENT SUBGRADES – CBR High 15	Medium 10	Low 6	Very low 3
1	2	3	4	5	6	7	8	9	10	11	12
B727-200 (Standard)	78 471 / 44 293	48.5	1.15	48/24	50/26	53/27	56/29	43/22	45/23	51/25	56/29
B727-200 (Advanced)	84 005 / 44 270	48.0	1.02	49/23	52/24	55/26	58/28	45/21	48/22	55/24	60/29
B727-200 (Advanced)	86 636 / 44 347	47.7	1.06	51/23	54/25	58/26	60/28	47/22	50/22	56/24	61/28
B727-200 (Advanced)	89 675 / 44 470	46.9	1.15	54/23	57/25	60/27	62/28	49/21	51/22	58/24	63/28
B727-200 (Advanced)	95 254 / 45 677	46.5	1.19	58/24	61/25	64/27	67/29	52/22	55/22	62/25	66/29
B737-100	44 361 / 26 581	46.2	0.95	23/12	24/13	26/14	27/15	20/12	22/12	24/13	28/15
B737-200	45 722 / 27 170	46.4	0.97	24/13	25/14	27/15	29/16	22/12	23/12	26/14	30/16
B737-200	52 616 / 27 125	45.5	1.14	29/13	31/14	32/15	34/16	26/12	27/12	30/13	34/15
B737-200	52 616 / 27 125	45.5	0.66	24/11	26/12	28/13	30/14	21/10	25/11	29/13	34/15
B737-200/200C (Advanced)	53 297 / 29 257	46.4	1.16	30/15	32/16	34/17	35/18	27/14	28/14	31/15	36/17
B737-200/200C (Advanced)	56 699 / 28 985	46.3	1.23	33/15	34/16	36/17	38/18	29/14	30/14	34/15	38/17
B737-200 (Advanced)	58 332 / 29 620	46.0	1.25	34/15	36/16	38/17	39/18	30/14	31/14	35/15	39/17
B737-300	61 462 / 32 904	45.9	1.34	37/18	39/18	41/20	42/21	32/16	33/16	37/17	41/20
B737-300	61 462 / 32 904	45.9	1.14	35/17	37/18	39/19	41/20	31/15	33/16	37/17	41/20
B737-400	64 864 / 33 643	46.9	1.44	41/19	43/20	45/21	47/22	35/16	37/17	41/18	45/21
B737-500*	60 781 / 31 312	46.1	1.34	37/17	38/17	40/19	42/19	32/15	33/15	37/16	41/19
B747-100	323 410 / 162 385	23.4	1.50	41/17	48/19	57/22	65/25	44/19	48/20	58/22	77/28
B747-100B	334 749 / 173 036	23.1	1.56	43/18	50/20	59/24	68/28	46/20	50/21	60/24	80/30

* Preliminary data

Aircraft type	All-up mass (kg)	Load on one main gear leg (%)	Tire pressure (MPa)	ACN FOR RIGID PAVEMENT SUBGRADES – MN/m³ High 150	Medium 80	Low 40	Ultra-low 20	ACN FOR FLEXIBLE PAVEMENT SUBGRADES – CBR High 15	Medium 10	Low 6	Very low 3
1	2	3	4	5	6	7	8	9	10	11	12
B747–100B	341 553	23.1	1.32	41	49	58	68	46	51	62	82
	171 870			17	19	22	26	20	21	23	30
B747–100B SR	260 362	24.1	1.04	27	32	40	47	33	36	43	59
	164 543			16	17	21	25	19	20	23	30
B747SP	302 093	22.9	1.30	35	42	51	59	40	44	52	71
	147 716			14	16	19	22	17	17	19	25
B747SP	318 881	21.9	1.40	37	44	52	60	41	45	54	72
	147 996			14	15	18	21	16	17	18	23
B747–200B	352 893	23.6	1.37	45	53	64	73	50	55	67	88
	172 886			18	20	24	28	21	22	24	31
B747–200C	373 305	23.1	1.30	46	55	66	76	52	57	70	92
	166 749			16	18	21	25	19	20	22	29
B747–200F/300	379 201	23.2	1.39	47	57	68	78	53	59	73	94
	156 642			16	17	20	24	18	19	21	26
B747–400	395 987	23.4	1.41	53	63	75	85	57	64	79	101
	178 459			19	21	25	29	21	22	25	32
B757–200	109 316	45.2	1.17	27	32	38	44	29	32	39	52
	60 260			12	14	17	19	14	14	17	22
B767–200	143 789	46.2	1.31	33	38	46	54	37	40	47	65
	78 976			15	17	20	24	18	19	21	26
B767–200–ER	159 755	46.9	1.21	37	44	54	63	43	47	57	77
	80 853			16	18	21	25	19	19	22	28
B767–300	159 665	47.5	1.21	38	45	54	63	43	48	58	78
	86 070			17	19	23	27	20	21	24	32
B767–300–ER	172 819	46.9	1.31	43	51	61	71	48	53	65	86
	87 926			18	20	24	28	21	22	24	32
B767–300–ER	185 520	46.0	1.38	47	56	66	76	51	57	70	92
	88 470			18	20	24	28	21	22	24	31
Caravelle Series 10	52 000	46.1	0.75	15	17	20	22	15	17	19	23
	29 034			7	8	9	10	7	7	9	11
Caravelle Series 12	55 960	46.0	0.88	16	19	22	25	17	19	21	26
	31 800			8	9	10	12	8	9	10	12
Concorde	185 066	48.0	1.26	61	71	82	91	65	72	81	98
	78 698			21	22	25	29	21	22	26	32
Canadair CL 44	95 708	47.5	1.12	25	30	35	40	27	30	36	47
	40 370			9	10	11	13	9	10	11	14

Aircraft type	All-up mass (kg)	Load on one main gear leg (%)	Tire pressure (MPa)	ACN FOR RIGID PAVEMENT SUBGRADES – MN/m³ High 150	Medium 80	Low 40	Ultra-low 20	ACN FOR FLEXIBLE PAVEMENT SUBGRADES – CBR High 15	Medium 10	Low 6	Very low 3
1	2	3	4	5	6	7	8	9	10	11	12
Convair 880 M	87 770 / 40 195	46.6	1.03	26 / 9	31 / 10	36 / 12	41 / 14	27 / 10	31 / 10	36 / 12	44 / 15
Convair 990	115 666 / 54 685	48.5	1.28	41 / 15	48 / 17	54 / 19	60 / 22	40 / 15	45 / 16	53 / 19	64 / 24
DC-3	11 430 / 7 767	46.8	0.31	6 / 4	7 / 5	7 / 5	7 / 5	4 / 3	6 / 4	8 / 5	9 / 6
DC-4	33 113 / 22 075	46.8	0.53	13 / 8	15 / 9	17 / 10	18 / 11	11 / 7	14 / 9	16 / 10	20 / 12
DC-8-43	144 242 / 61 919	46.5	1.22	41 / 15	49 / 16	57 / 18	65 / 21	43 / 15	49 / 16	59 / 18	74 / 23
DC-8-55	148 778 / 62 716	47.0	1.30	45 / 15	53 / 16	62 / 19	69 / 22	46 / 15	53 / 16	63 / 18	78 / 24
DC-8-61/71	148 778 / 68 992	48.0	1.30	46 / 17	54 / 19	63 / 22	71 / 25	48 / 18	54 / 19	64 / 21	80 / 28
DC-8-62/72	160 121 / 65 025	46.5	1.29	47 / 15	56 / 16	65 / 19	73 / 22	49 / 16	56 / 16	67 / 18	83 / 24
DC-8-63/73	162 386 / 72 002	47.6	1.34	50 / 17	60 / 19	69 / 23	78 / 26	52 / 18	59 / 19	71 / 22	87 / 29
DC-9-15	41 504 / 22 300	46.2	0.90	23 / 11	25 / 12	26 / 13	28 / 14	21 / 10	22 / 11	26 / 12	28 / 14
DC-9-21	45 813 / 23 879	47.2	0.98	27 / 12	29 / 13	30 / 14	32 / 15	24 / 11	26 / 12	29 / 13	32 / 15
DC-9-32	49 442 / 25 789	46.2	1.07	29 / 14	31 / 15	33 / 15	34 / 16	26 / 12	28 / 13	31 / 14	34 / 16
DC-9-41	52 163 / 27 821	46.7	1.10	32 / 15	34 / 16	35 / 17	37 / 18	28 / 13	30 / 14	33 / 15	37 / 18
DC-9-51	55 338 / 29 336	47.0	1.17	35 / 17	37 / 17	39 / 18	40 / 19	31 / 15	32 / 15	36 / 16	39 / 19
MD-81	63 957 / 35 571	47.8	1.17	41 / 20	43 / 21	45 / 23	46 / 24	36 / 18	38 / 19	43 / 71	46 / 24
MD-82/88	68 266 / 35 629	47.6	1.27	45 / 21	47 / 22	49 / 24	50 / 25	39 / 18	42 / 19	46 / 20	50 / 24
MD-83	73 023 / 36 230	47.4	1.34	49 / 21	51 / 22	53 / 24	55 / 25	42 / 18	46 / 19	50 / 21	54 / 24
MD-87	68 266 / 33 965	47.4	1.27	45 / 19	47 / 21	49 / 22	50 / 23	39 / 17	42 / 18	46 / 19	50 / 22

Aircraft type	All-up mass (kg)	Load on one main gear leg (%)	Tire pressure (MPa)	ACN FOR RIGID PAVEMENT SUBGRADES – MN/m^3 High 150	Medium 80	Low 40	Ultra-low 20	ACN FOR FLEXIBLE PAVEMENT SUBGRADES – CBR High 15	Medium 10	Low 6	Very low 3
1	2	3	4	5	6	7	8	9	10	11	12
DC-10-10	196 406 / 108 940	47.2	1.28	45/23	52/25	63/28	73/33	52/26	57/27	68/30	93/38
DC-10-10	200 942 / 105 279	46.9	1.31	46/22	54/24	64/27	75/31	54/24	58/25	69/28	96/36
DC-10-15	207 746 / 105 279	46.7	1.34	48/22	56/24	67/27	74/31	55/24	61/25	72/28	100/36
DC-10-30/40	253 105 / 120 742	37.7	1.17	44/20	53/21	64/24	75/28	53/22	59/23	70/25	97/32
DC-10-30/40	260 816 / 124 058	37.6	1.21	46/20	55/21	67/25	78/29	56/23	61/23	74/26	101/33
DC-10-30/40	268 981 / 124 058	37.9	1.24	49/20	59/21	71/25	83/29	59/23	64/23	78/26	106/33
MD-11	274 650 / 127 000	39.2	1.41	56/23	66/25	79/28	92/32	64/25	70/26	85/29	114/37
DCH 7 DASH 7	19 867 / 11 793	46.8	0.74	11/6	12/6	13/7	13/7	10/5	11/6	12/6	14/8
FOKKER 27 Mk500	19 777 / 11 879	47.5	0.54	10/5	11/6	12/6	12/7	8/4	10/5	12/6	13/7
FOKKER 50 HTP	20 820 / 12 649	47.8	0.59/ 0.55	10/6	11/6	12/7	13/7	8/5	10/5	12/6	14/8
FOKKER 50 LTP	20 820 / 12 649	47.8	0.41	9/5	10/5	11/6	12/7	6/4	9/5	11/6	14/8
FOKKER 28 Mk1000LTP	29 484 / 15 650	46.3	0.58	14/6	15/7	17/8	18/9	11/5	14/6	16/7	19/9
FOKKER 28 Mk1000HTP	29 484 / 16 550	46.3	0.69	15/8	16/8	18/9	18/10	13/6	15/7	17/8	20/10
FOKKER 100	44 680 / 24 375	47.8	0.98	28/13	29/14	31/15	32/16	25/12	27/13	30/14	32/16
HS125-400A -400B	10 600 / 5 683	45.5	0.77	6/3	6/3	7/6	7/3	5/2	5/3	6/3	7/3
HS125-600A -600B	11 340 / 5 683	45.5	0.83	7/3	7/3	7/3	8/3	5/2	6/3	7/3	8/3
HS748	21 092 / 12 183	43.6	0.59	10/5	11/5	11/6	12/6	8/4	9/5	11/6	13/7
IL-62	162 600 / 66 400	47.0	1.08	42/14	50/15	60/18	69/20	47/16	54/17	64/18	79/24

Aircraft type	All-up mass (kg)	Load on one main gear leg (%)	Tire pressure (MPa)	ACN FOR RIGID PAVEMENT SUBGRADES – MN/m^3 High 150	Medium 80	Low 40	Ultra-low 20	ACN FOR FLEXIBLE PAVEMENT SUBGRADES – CBR High 15	Medium 10	Low 6	Very low 3
1	2	3	4	5	6	7	8	9	10	11	12
IL-62M	168 000 / 71 400	47.0	1.08	43 / 16	52 / 17	62 / 19	71 / 22	50 / 17	57 / 18	67 / 20	83 / 26
IL-76T	171 000 / 83 800	23.5	0.64	38 / 11	38 / 14	38 / 16	39 / 16	37 / 15	40 / 16	45 / 18	53 / 22
IL-86	209 500 / 111 000	31.2	0.88	25 / 13	31 / 14	38 / 16	46 / 19	34 / 16	36 / 17	43 / 19	61 / 23
L-100-20	70 670 / 34 205	48.2	0.72	30 / 14	33 / 15	36 / 16	38 / 17	27 / 12	31 / 14	33 / 15	38 / 16
L-100-30	70 670 / 34 701	48.4	0.72	30 / 14	33 / 15	36 / 16	38 / 17	27 / 12	31 / 14	33 / 15	39 / 17
L-1011-1	195 952 / 108 862	47.4	1.33	45 / 24	52 / 25	62 / 28	73 / 33	52 / 25	56 / 27	66 / 29	91 / 38
L-1011 -100/200	212 281 / 110 986	46.8	1.21	46 / 23	55 / 24	66 / 28	78 / 32	56 / 25	61 / 26	73 / 30	100 / 38
L-1011 -500	225 889 / 108 924	46.2	1.27	50 / 23	59 / 24	72 / 27	84 / 31	60 / 25	65 / 26	79 / 28	107 / 36
Trident 1E	61 160 / 33 203	46.0	1.03	32 / 15	34 / 16	37 / 17	39 / 18	23 / 10	24 / 11	27 / 12	32 / 15
Trident 2E	65 998 / 33 980	47.0	1.07	37 / 16	39 / 17	42 / 18	44 / 19	26 / 11	28 / 12	31 / 13	36 / 16
Trident 3	68 266 / 39 060	45.5	1.14	37 / 18	40 / 19	42 / 21	44 / 22	26 / 13	28 / 14	31 / 15	36 / 18
TU-134A	47 600 / 29 350	45.6	0.83	11 / 7	13 / 8	16 / 9	19 / 10	12 / 7	13 / 8	16 / 9	21 / 12
TU-154B	98 000 / 53 500	45.1	0.93	19 / 8	25 / 10	32 / 13	38 / 17	20 / 10	24 / 11	30 / 13	38 / 18
VC10-1150	151 953 / 71 940	48.3	1.01	38 / 16	46 / 17	56 / 20	65 / 23	44 / 17	50 / 18	61 / 21	77 / 27