GM 14-43

CIVIL AVIATION AUTHORITY OF BANGLADESH

Guidance Manual

Standardized Method of Evaluating & Reporting Airport Pavement Strength - PCN

Version 2.0
21 August 2024

AERODROME STANDARD DIVISION

GM 14-43

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Guidance Manual
on

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Foreword

This Guidance Material on standardized method of reporting Airport Pavement Strength is the version 2.0 which has been prepared by Aerodrome Standard Division for the use and guidance of standardized method of reporting Airport Pavement Strength in the performances of their duties. All matters pertaining to aerodrome operator duties, responsibilities and procedures have been covered to the extent possible in this Guide.

Aerodrome Operators are expected to use good judgment in dealing with matters with where specific guidance is unavailable or be aware of changes in aviation technology, legislation and developments within the industry that may necessitate changes to requirements and the relevant procedures followed by CAAB.

The guidance contains the Standards, Policies & Procedures that pertain to Aerodrome Operator. The contents of the guidance material on standardized method of reporting Airport Pavement Strength shall not be deemed to supersede any instructions contained in the following documents: Aerodrome manual; CARs; ANOs; Rules & Regulations; AIP; AICs; Aerodrome Handbook; Standard Circulars; Aerodrome maintenance manual; Aerodrome emergency manual.

All the Aerodrome Operators are required to be fully conversant with the relevant contents of this book. The content of this guide is mainly extracted from Annexes, Documents, Aerodrome manuals; CARs; ANOs; Rules & Regulations; AIP; AICs; Aerodrome Handbook; Standard Circulars; Aerodrome maintenance manual; Aerodrome emergency manual.

The undersigned certifies that this aerodrome operator guidance on standardized method of reporting Airport Pavement Strength satisfies all the regulatory requirements. The responsibility to publish, make revisions and amendments and to control of the guidance shall be vested in and done according to the instructions and procedures described.

This guidance material on standardized method of reporting Airport Pavement Strength will be updated from time to time in relation to the changes in rules, regulations and or based on received suggestive ideas. Comments and recommendations are welcome and should be Forwarded to the undersigned.

21-8-2024

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GLOSSARY

Terms which are defined in the ICAO *Annex-14* Volume I 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.

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 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 andusage of the ACN-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 ACN and PCN as follows:

ACN- A number expressing the relative effect of an aircraft on a pavement for specified standard sub-grade 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. Attachment A and Appendix 3 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;

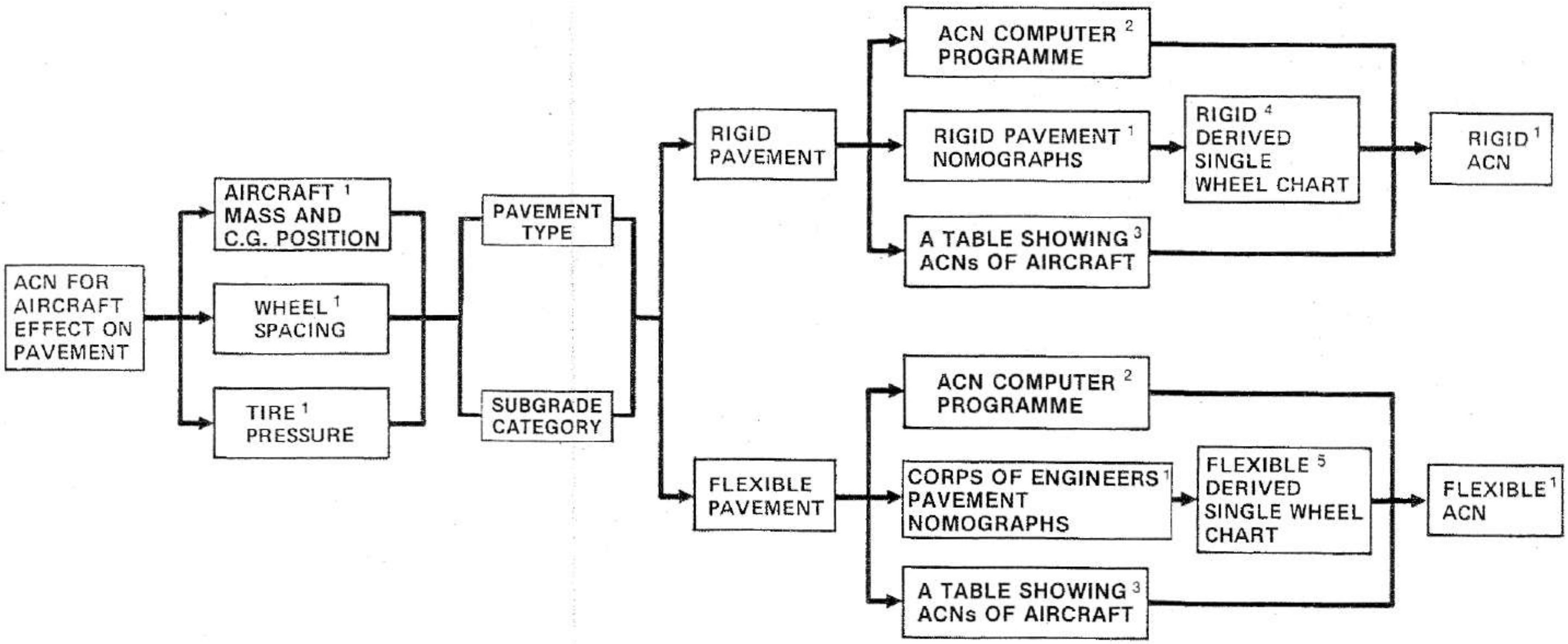
c) maximum tire pressure allowable; and

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.

1.1.3 How ACNs are determined

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) <u>Aircraft classification number (ACN).</u> 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).

c) 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.

d) Relative aircraft ACN charts and

e) 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

f) 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 <u>Abbreviations</u>

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).

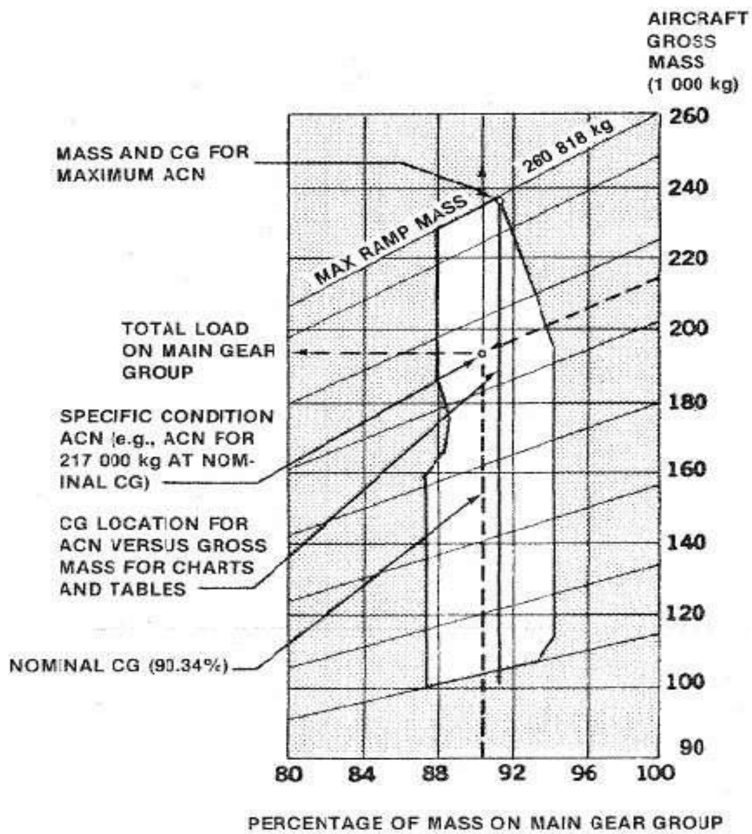


Figure 1-1 Landing gear loading on pavement Model DC-10 Series 30, 30CF, 40 and 40CF

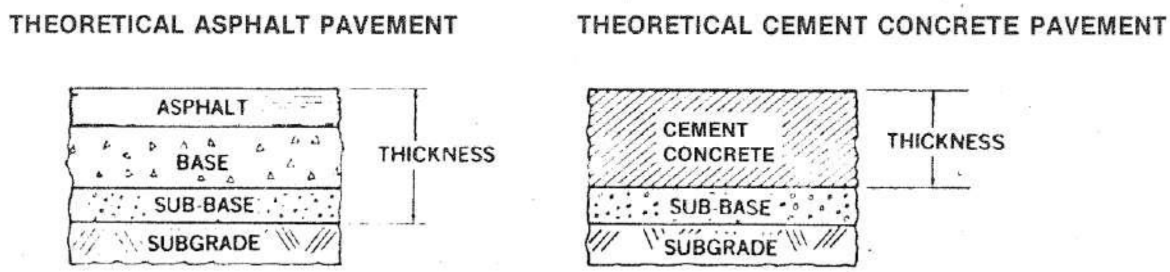


Figure 1-2

k - Westergaard's modulus of subgrade reaction in MN/m³

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)

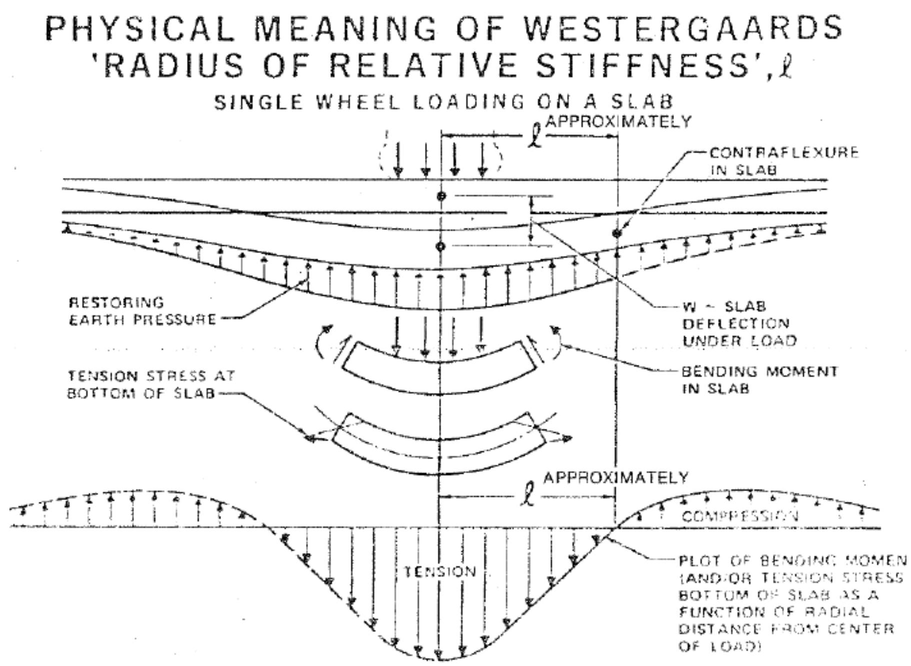


Figure 1-3

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". 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.

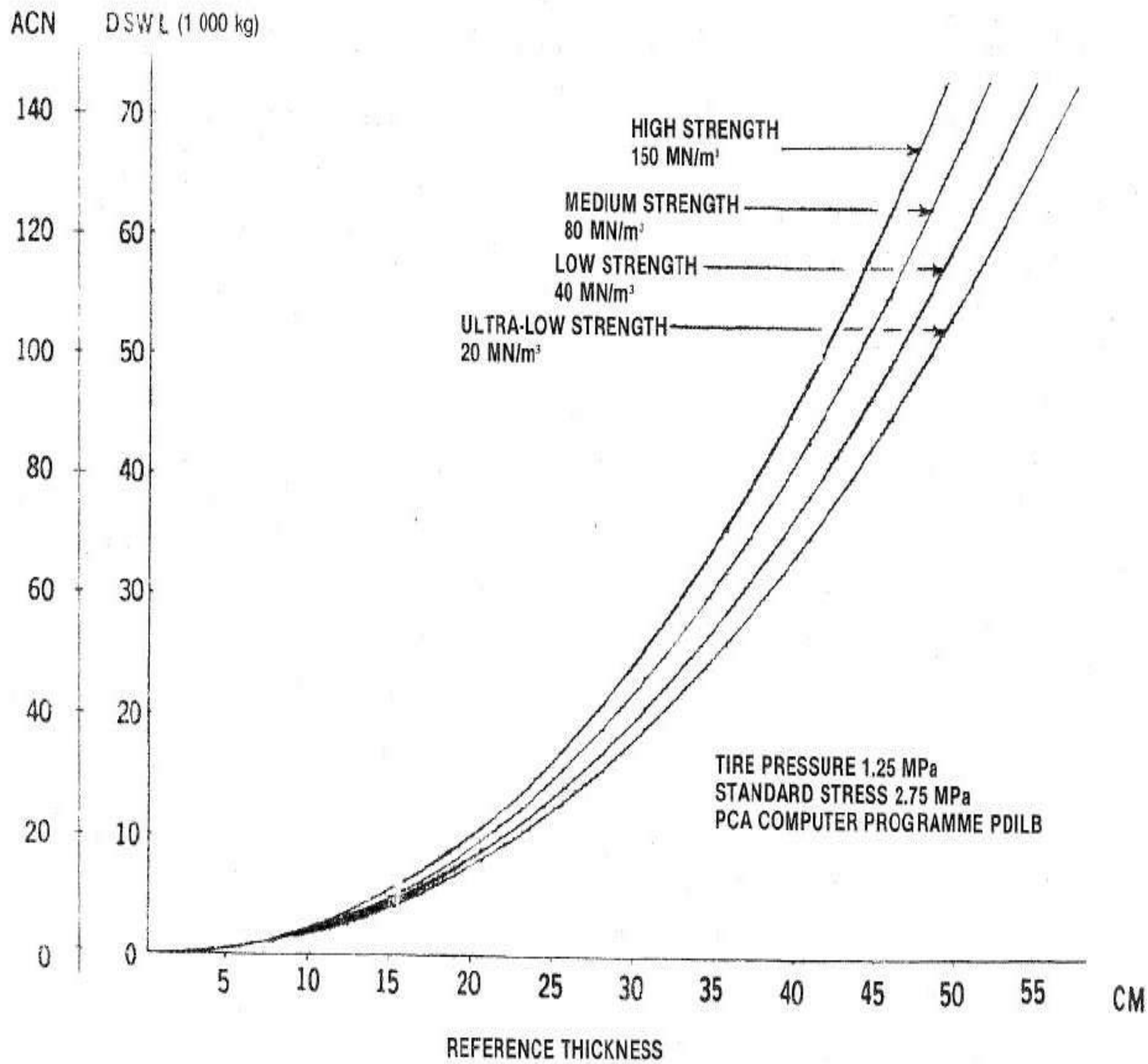


Figure 1-4. ACN Rigid Pavement Conversion Chart

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 <u>Flexible pavements.</u> 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:

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

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 <u>Tire pressure adjustment to ACN.</u> 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.

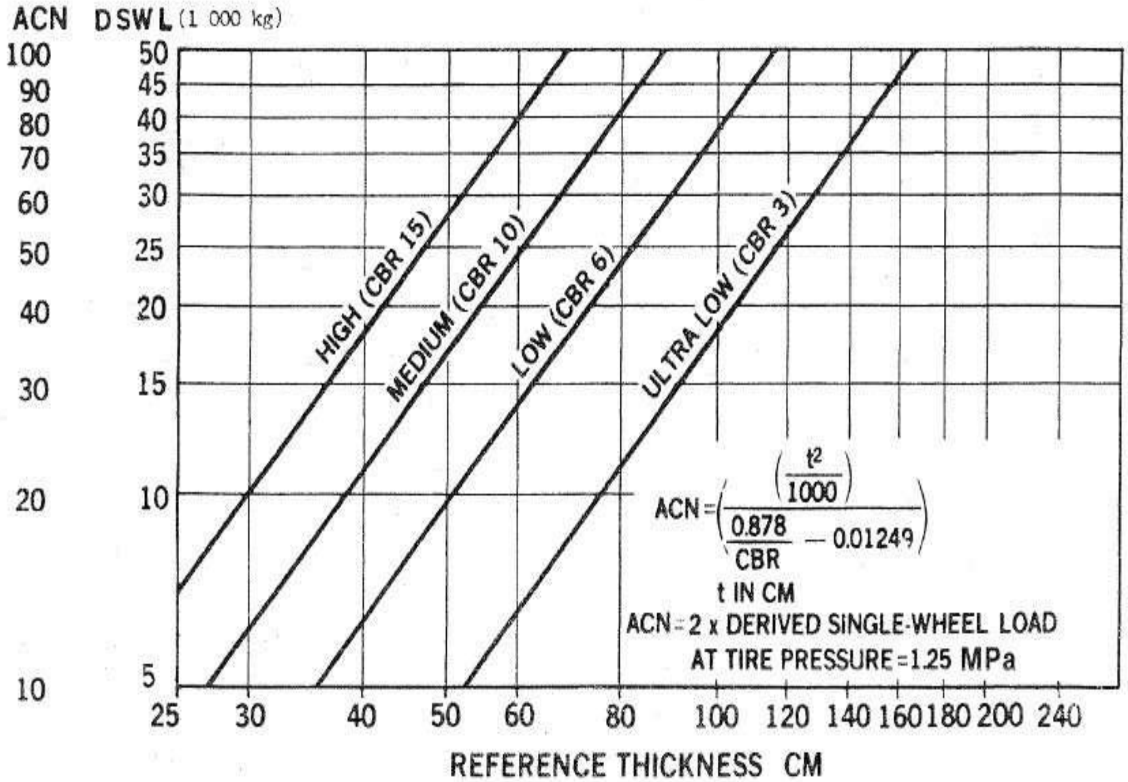


Figure 1-5. ACN Flexible Pavement Conversion Chart

1.1.3.10 For flexible pavements, the CBR $t = \sqrt{\frac{DSWL}{C_1 \; CBR} - \frac{DSWL}{C_2 p_s}}$ 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:

$$ACN_{\text{Reduced pressure}} = ACN_{\text{Maximum pressure}} \left[\frac{\frac{1}{C_1 \; CBR} - \frac{1}{C_2 p_{red}}}{\frac{1}{C_1 \; CBR} - \frac{1}{C_2 p_{max}}} \right]$$

(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.

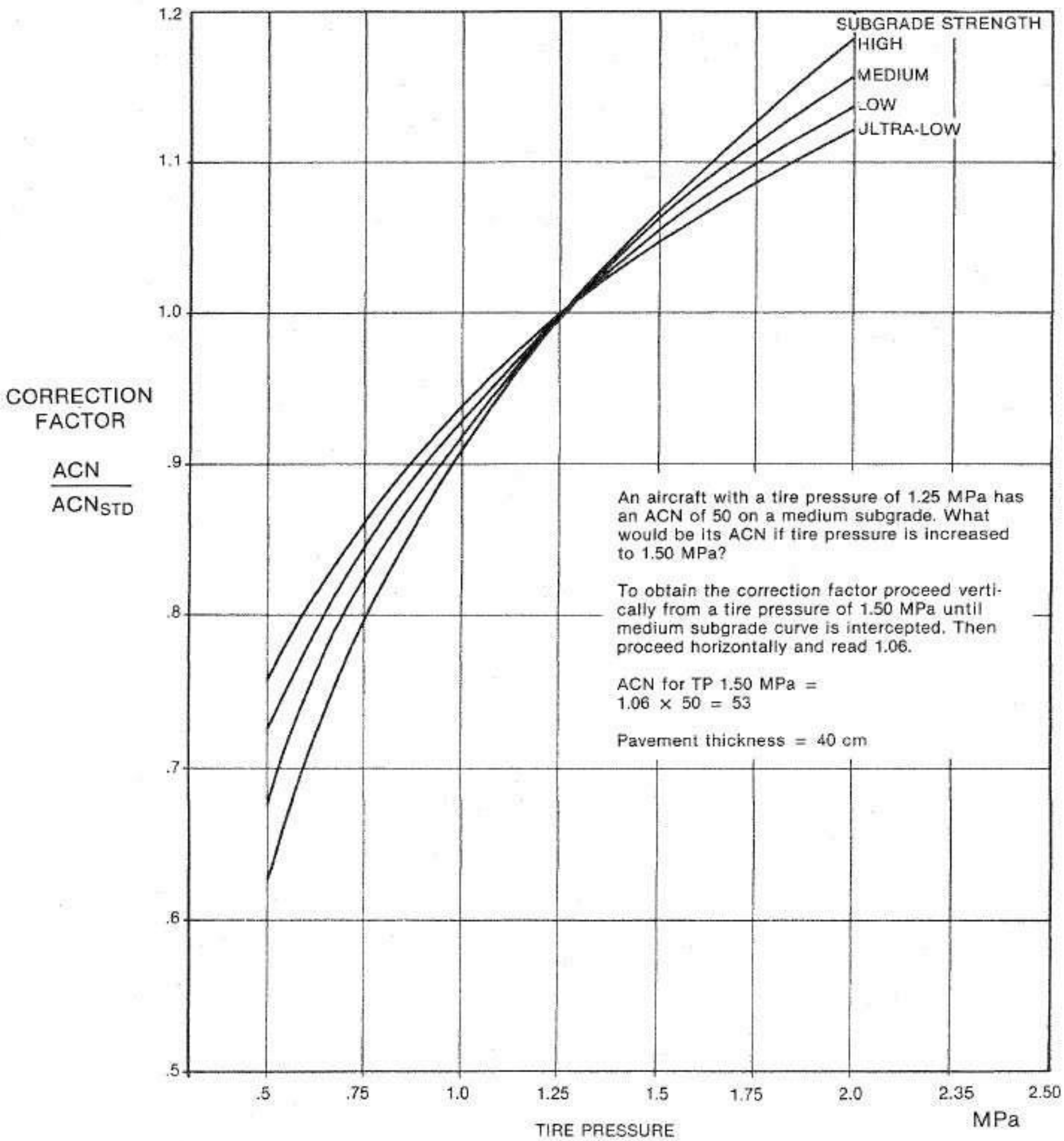


Figure 1-6. ACN tire pressure adjustment – rigid pavements only

<u>Example 2:</u> 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 3 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

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

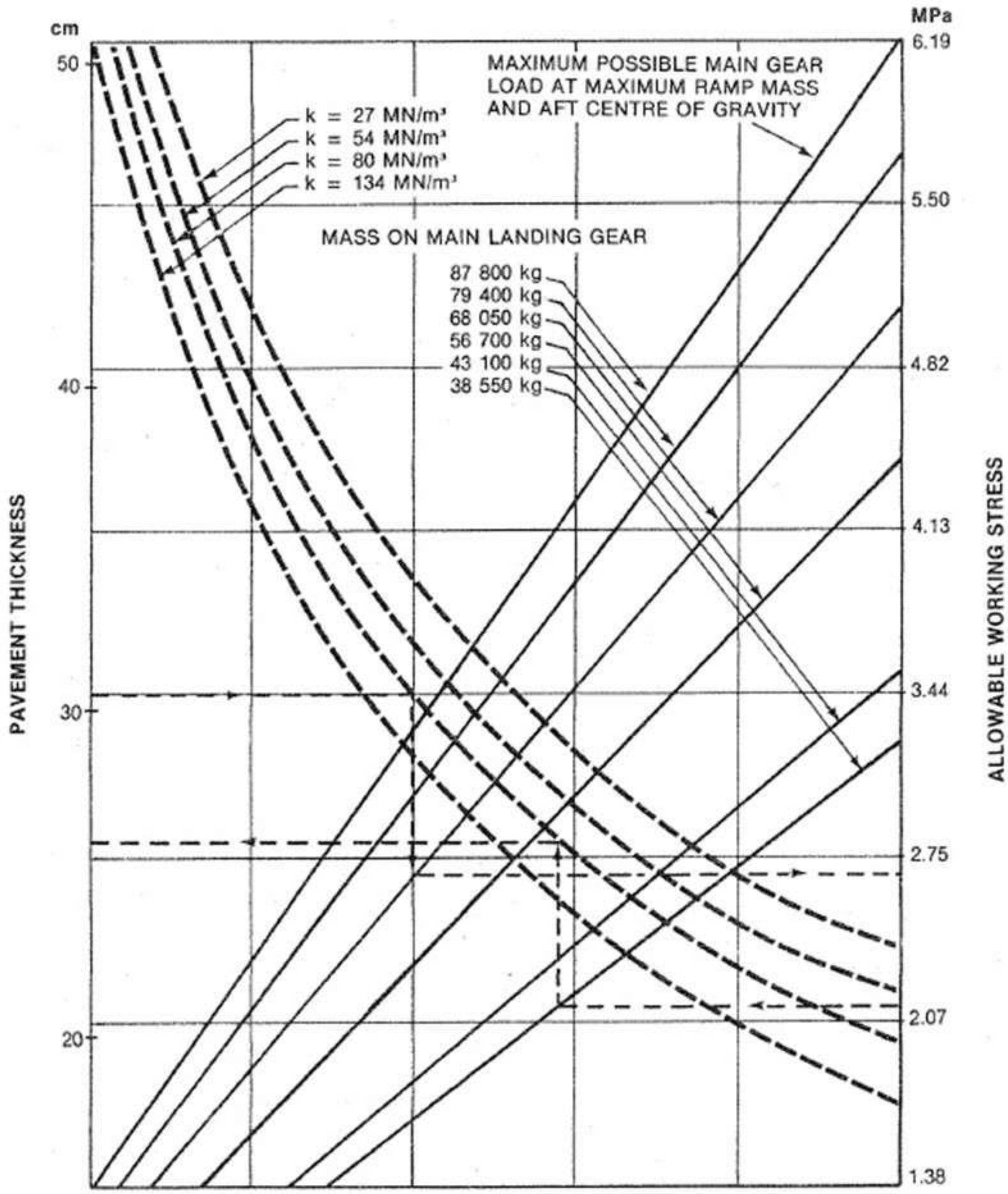
$$= 57 - \frac{39\ 006}{87\ 466} \times 30$$

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

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 57 cm; and
b) Enter Figure 1-5 with this thickness and read the ACN of aircraft for the subgrade CBR of 10 is 44.

124.5 × 43.2 cm TIRES - TIRE PRESSURE CONSTANT AT 11.7 kg/cm²
127 × 53.3 cm TIRES (NEW DESIGN) - TIRE PRESSURE CONSTANT AT 11.7 kg/cm²



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

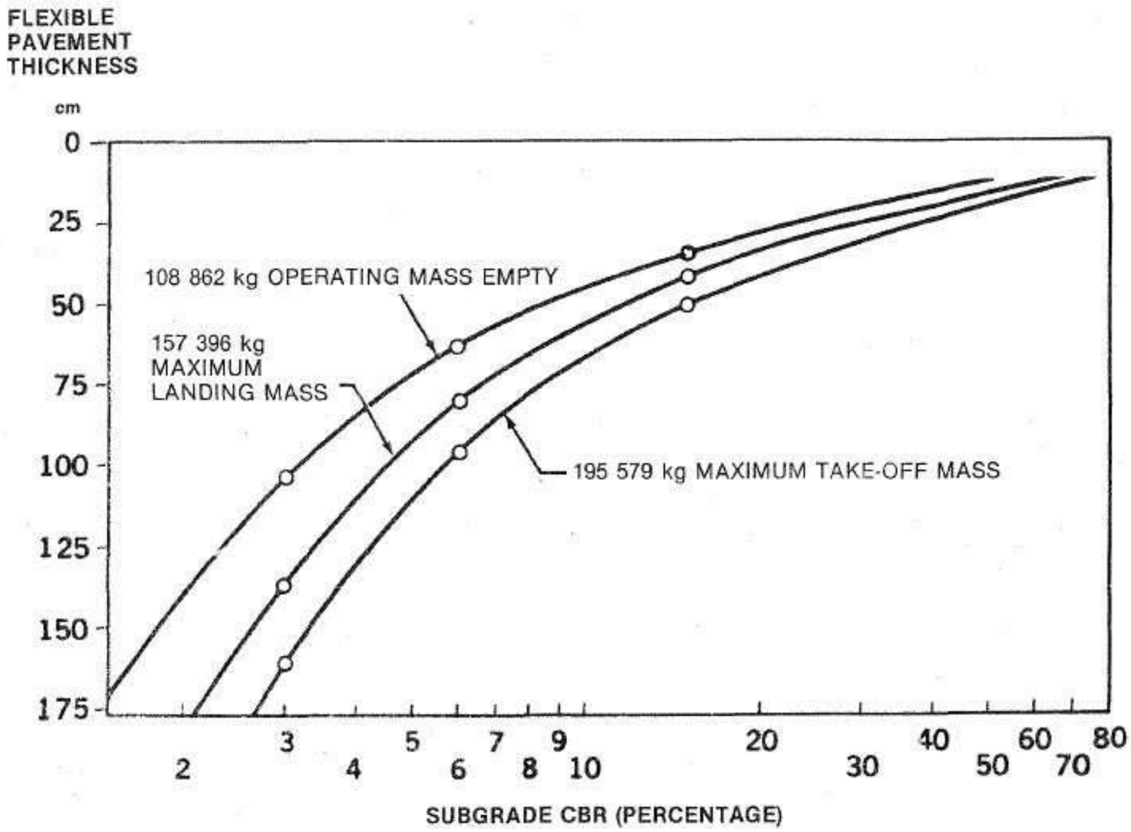


Figure 1-8. DC 10-10 Flexible Pavement Requirements 10000 Coverage aft c.g

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. 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 OVERLOADS OPERATIONS

2.1 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 behavior are not subject to a particular limiting load above which they suddenly or catastrophically fail. Behavior 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 behavior 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.

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 judgment. 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 aero planes within this mass range have quite high tire pressures limited qualitypavements 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.

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;

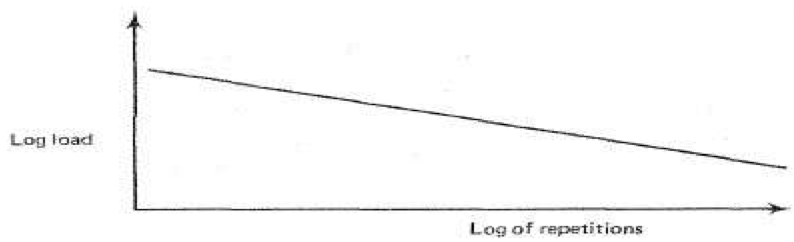


Figure 3-1

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.

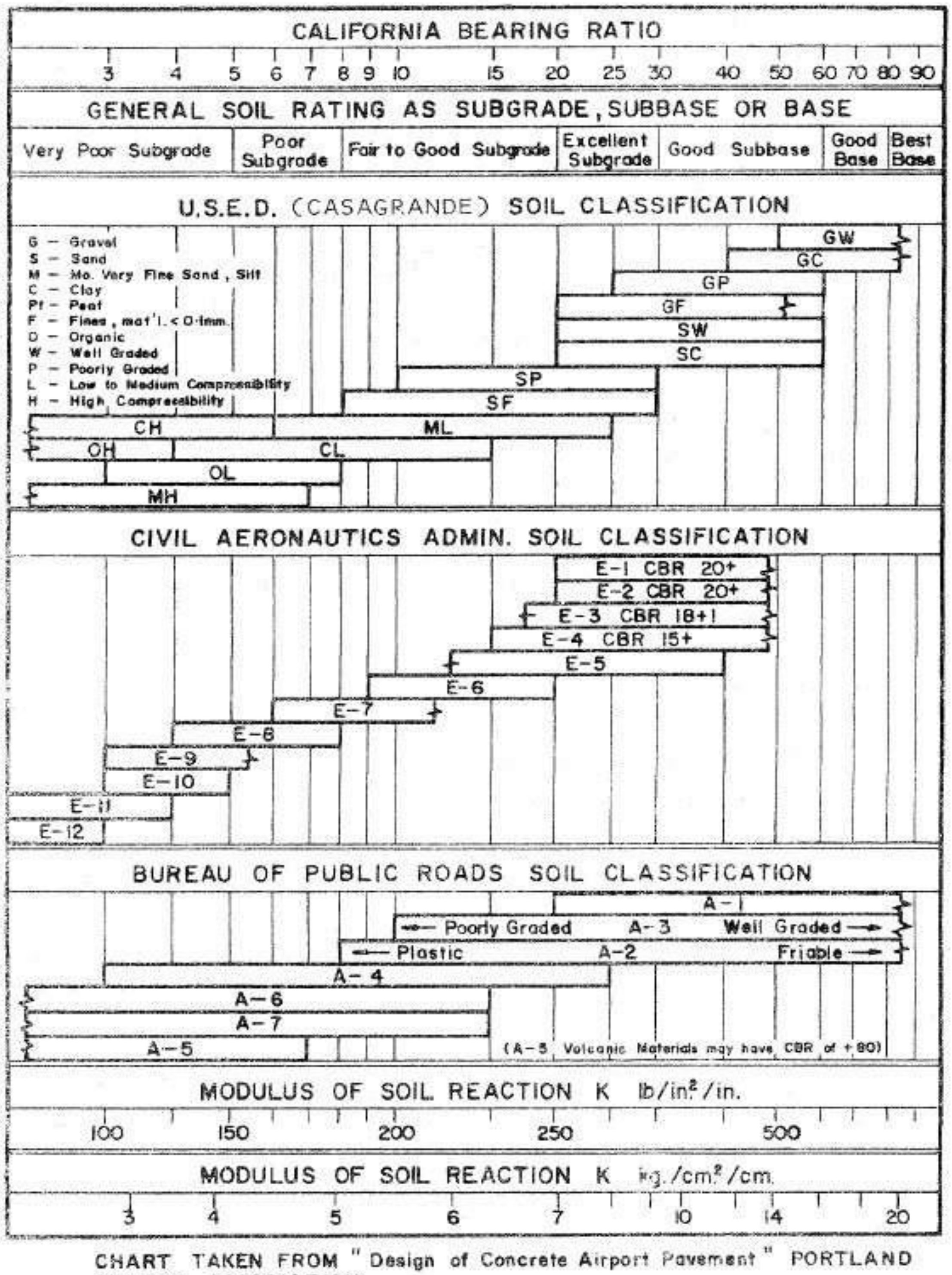


CHART TAKEN FROM " Design of Concrete Airport Pavement " PORTLAND CEMENT ASSOCIATION.

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. <u>Pavement behaviour concepts for design and evaluation.</u> 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.

3.6.2.1. <u>The early methods.</u> 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.

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.

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. 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. TheBenkleman Beam methods, as well as other highway methods, are applicable to evaluationof light aircraft pavements.

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 in the particular method (FAA), 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, 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. 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.

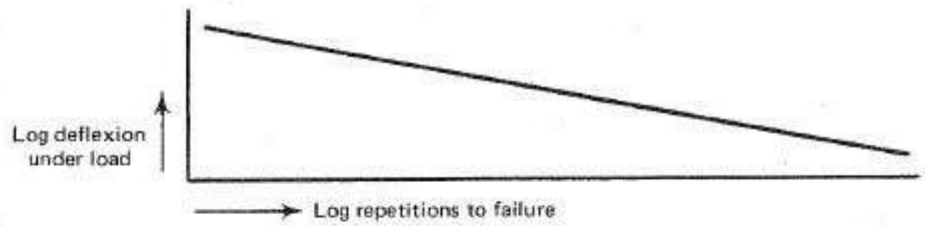


Figure 3- 3

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.

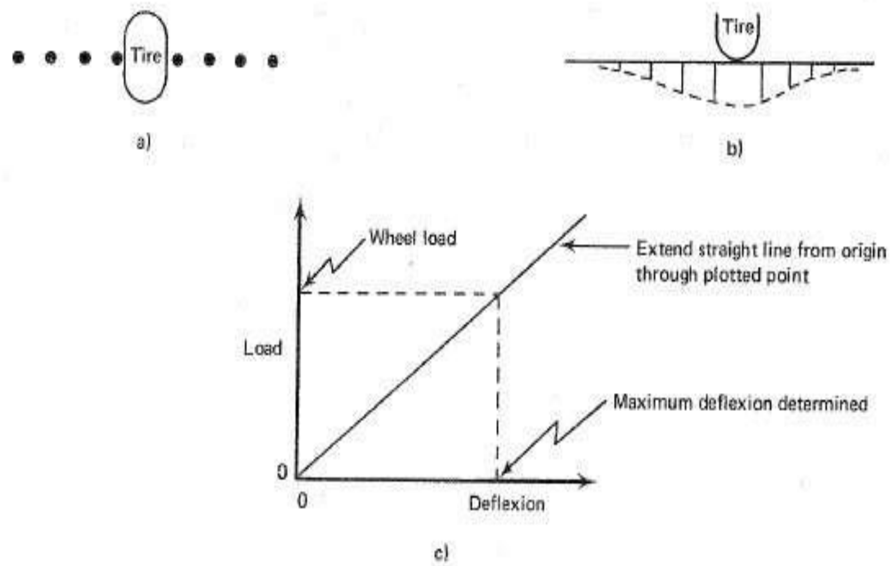


Figure 3-4

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 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 ANO-14. 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 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.

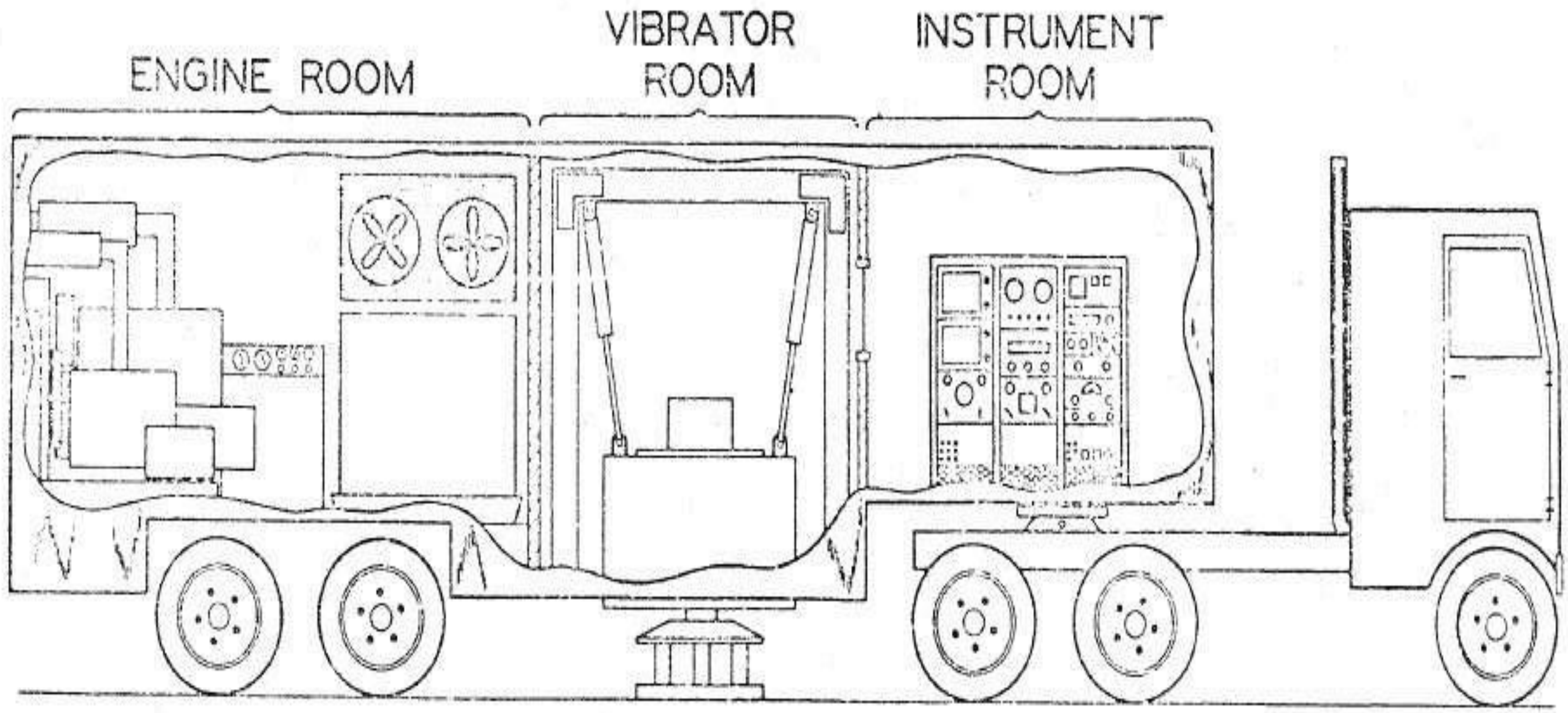


Figure 3-5. Waterways Experiment Station non-destructive testing equipment

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.

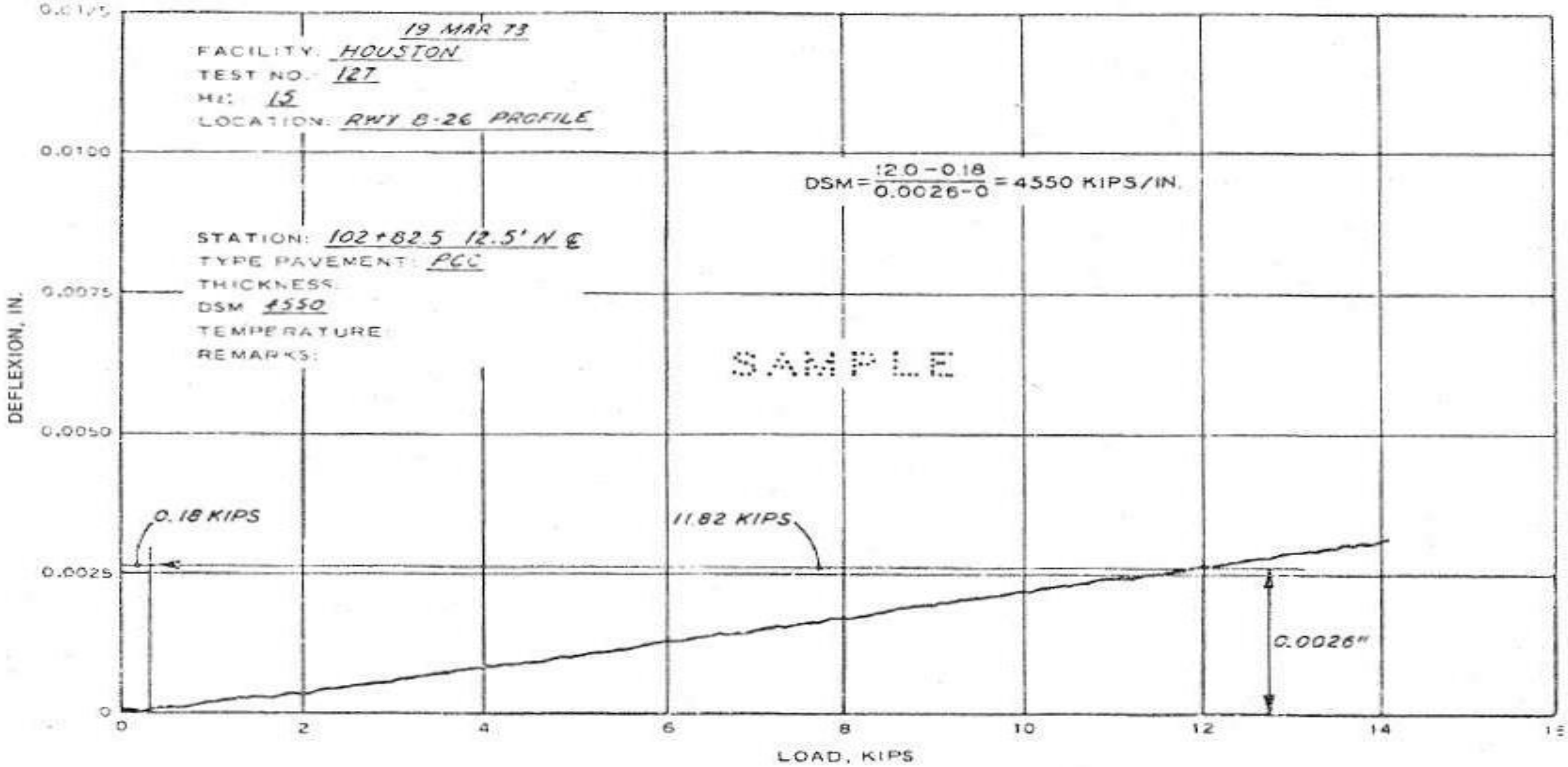


Figure 3-6. Deflexion versus load (sample plot)

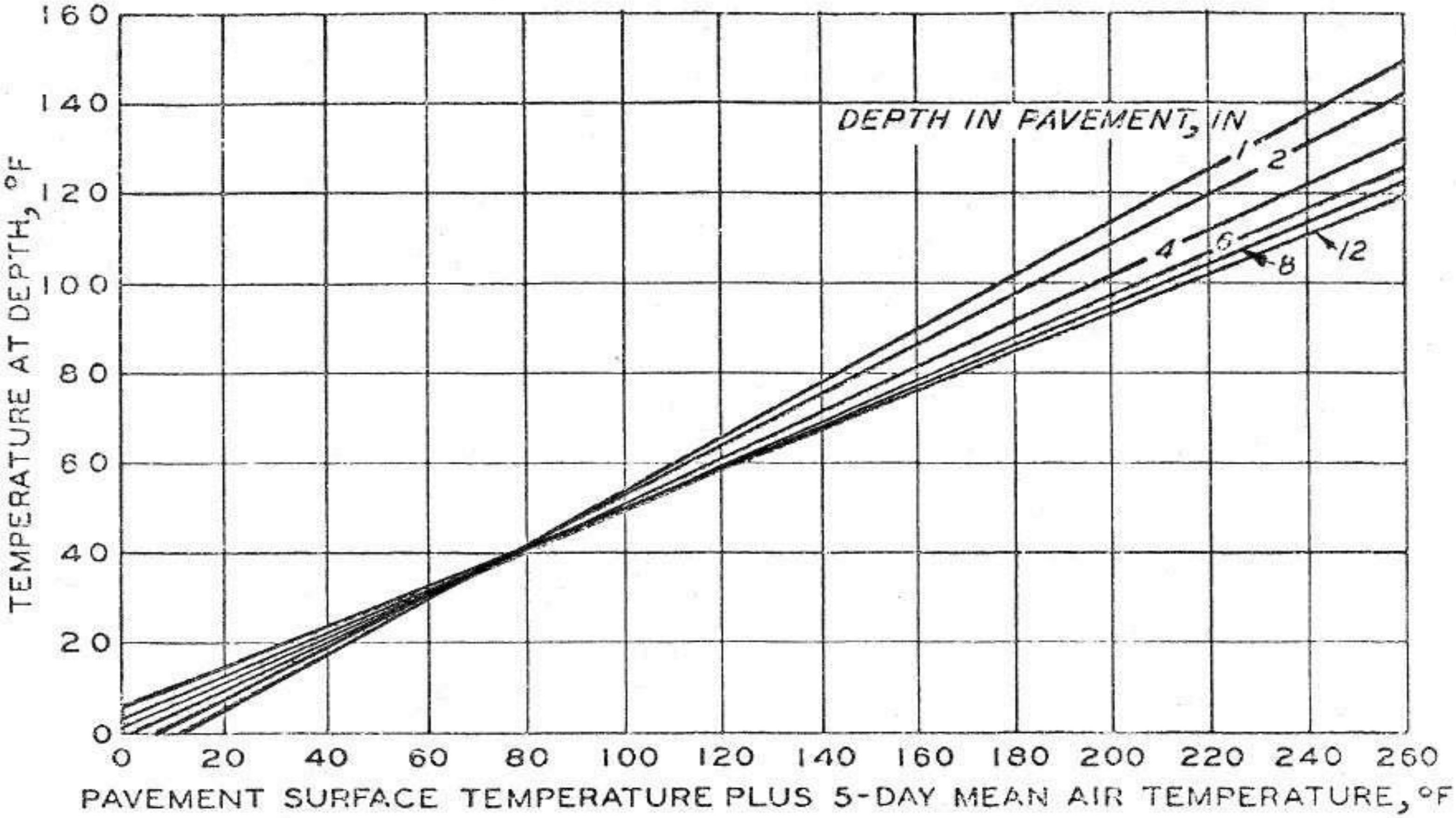


Figure 3-7. Prediction of flexible pavement temperatures

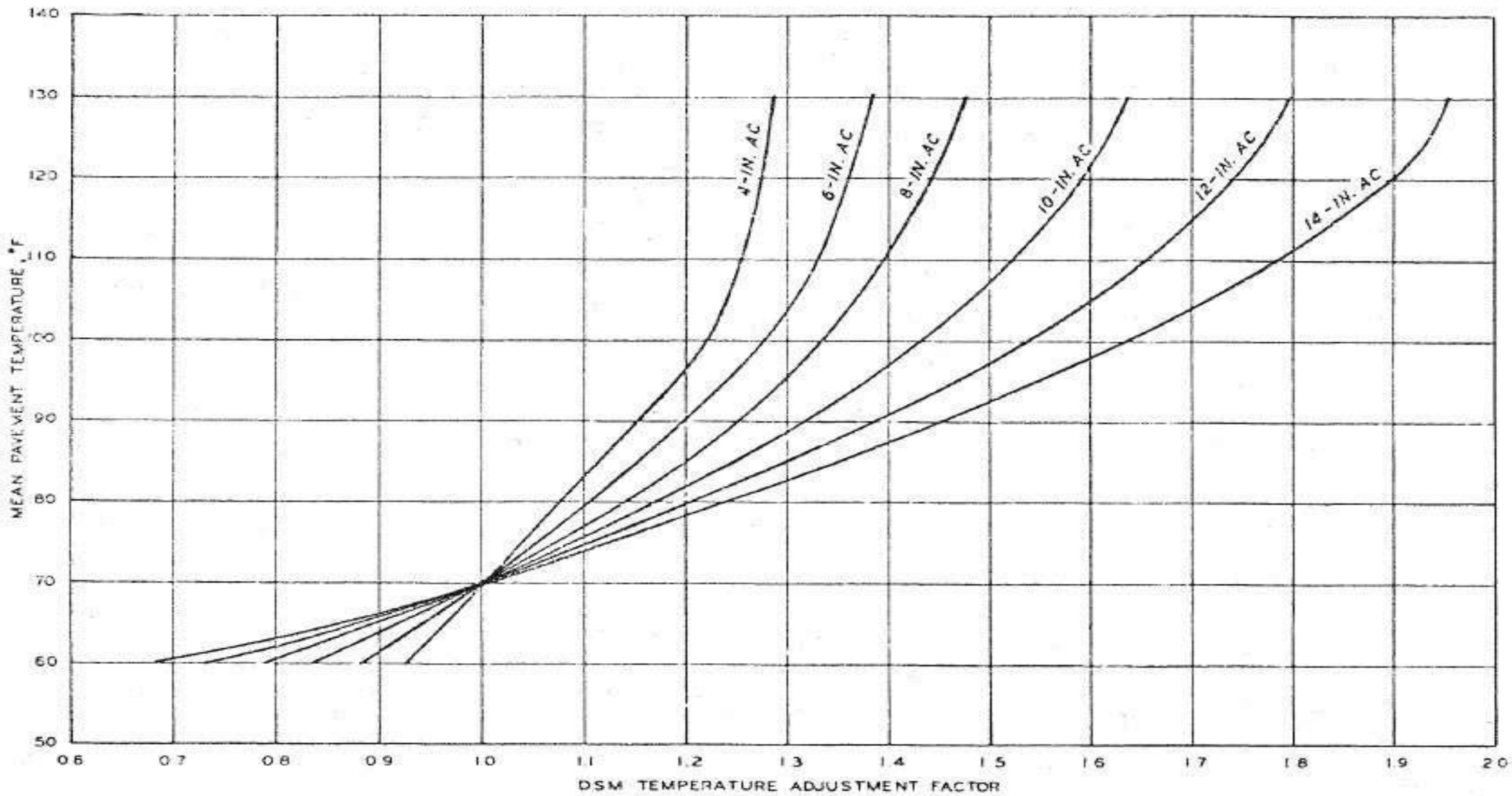


Figure 3-8. DSM temperature adjustment curves

3.6.5.14. <u>Rigid Pavement evaluation.</u>

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.

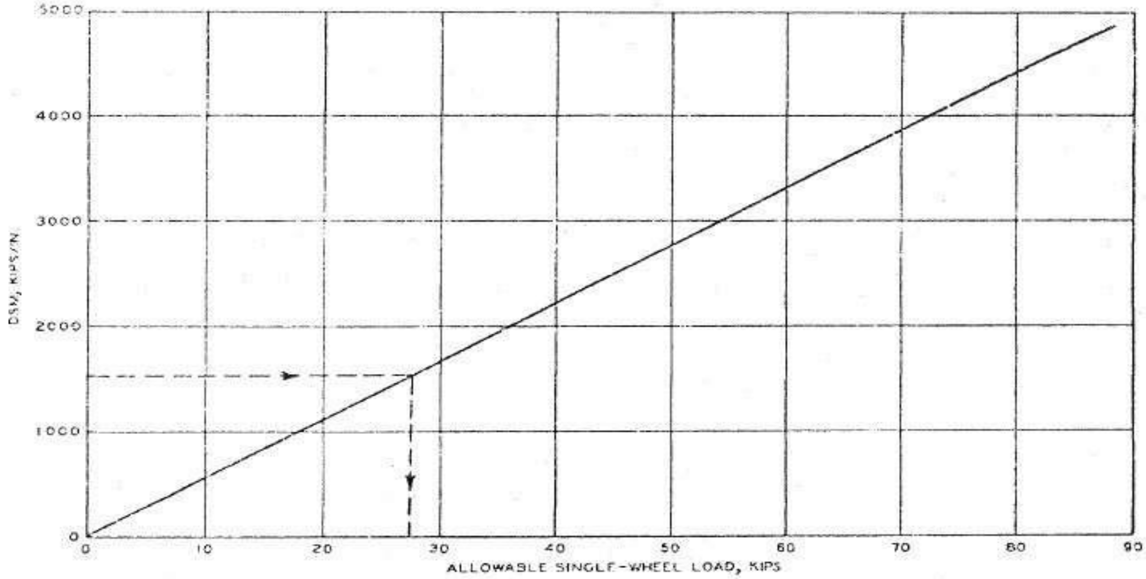


Figure 3-9. Evaluation curve for rigid pavement

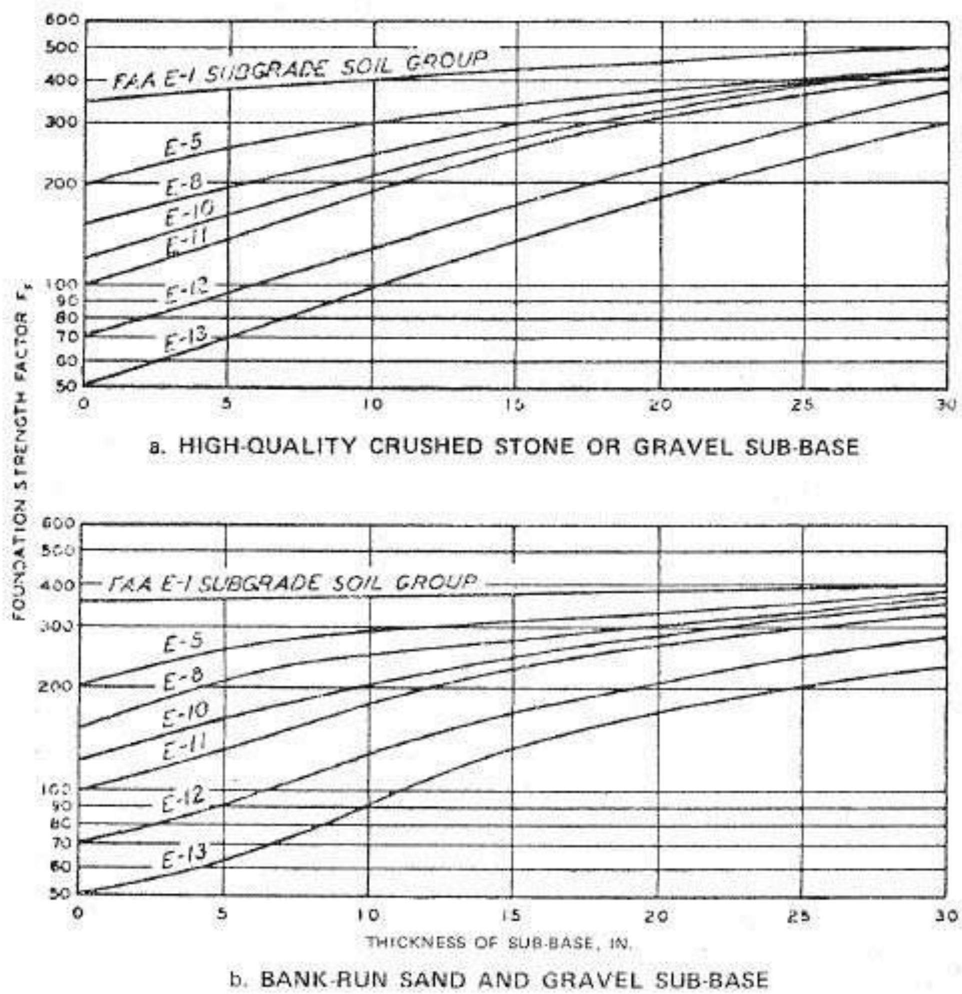


Figure 3-10. F_F versus sub-base thickness

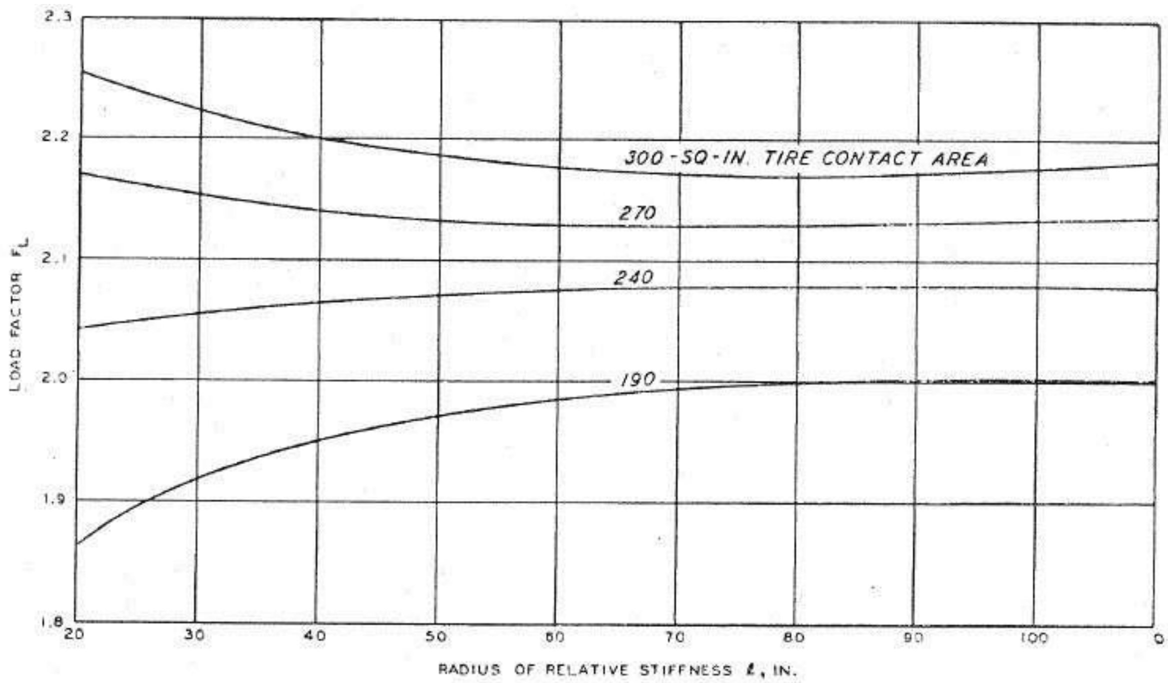


Figure 3-11. $F_L$versus l for single-wheel aircraft on rigid pavement

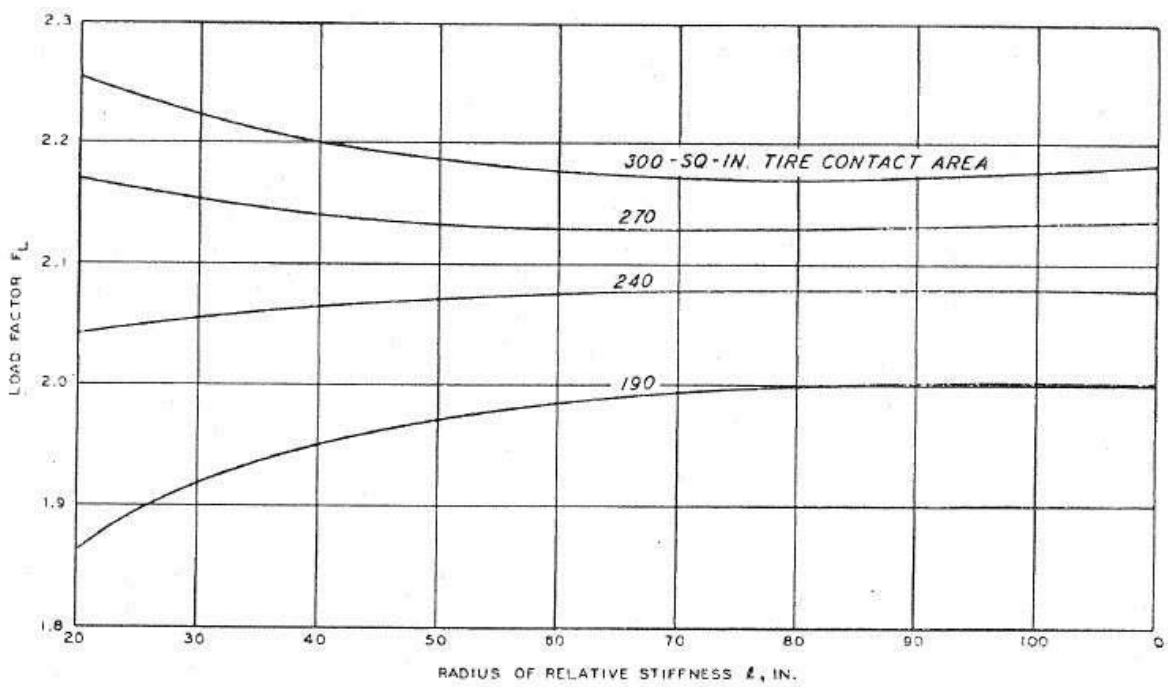


Figure 3-11. $F_L$versus l for dual wheel aircraft on rigid pavement

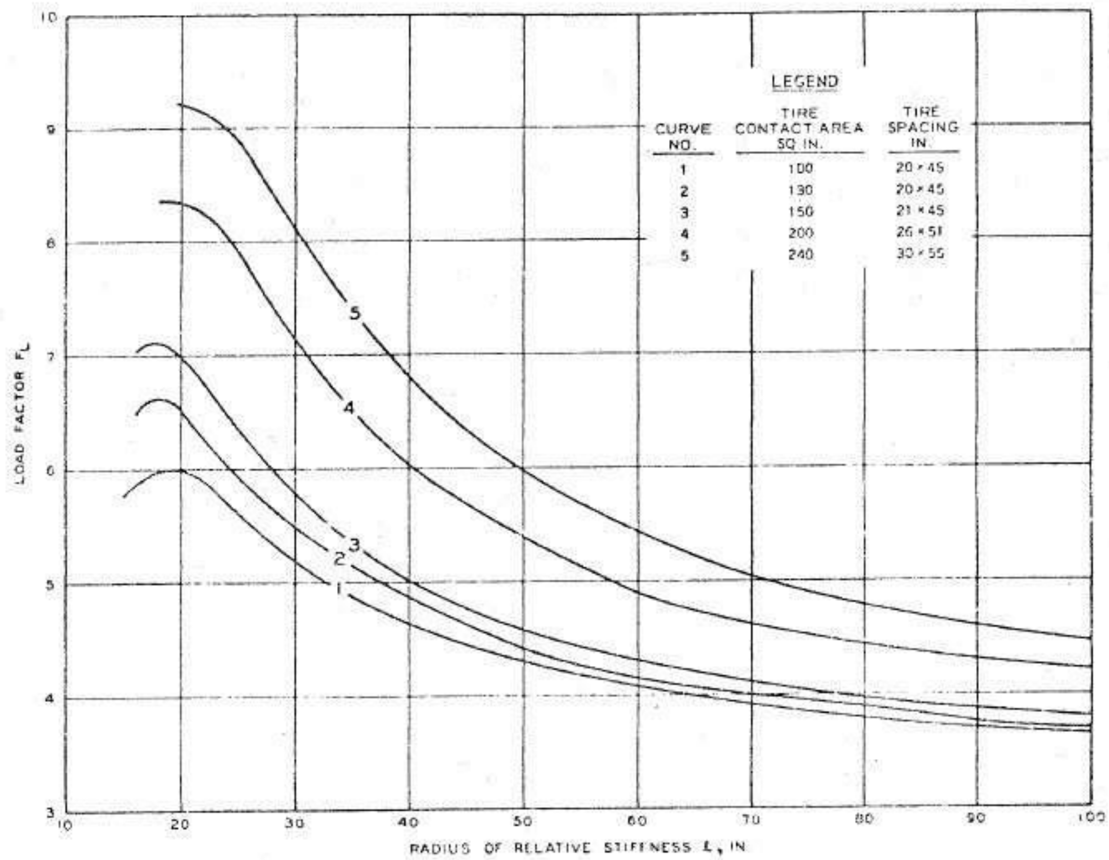


Figure 3-13. F_L versus l for dual tandem aircraft on rigid pavement

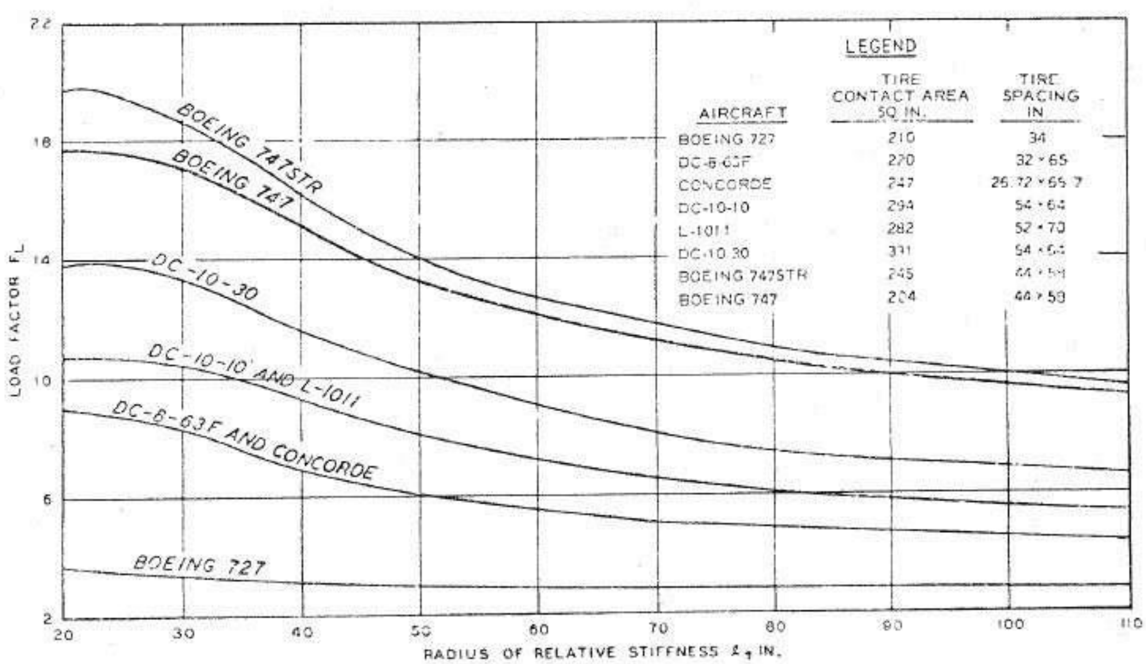


Figure 3-14. F_L versus l for various jet aircraft on rigid pavement

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.

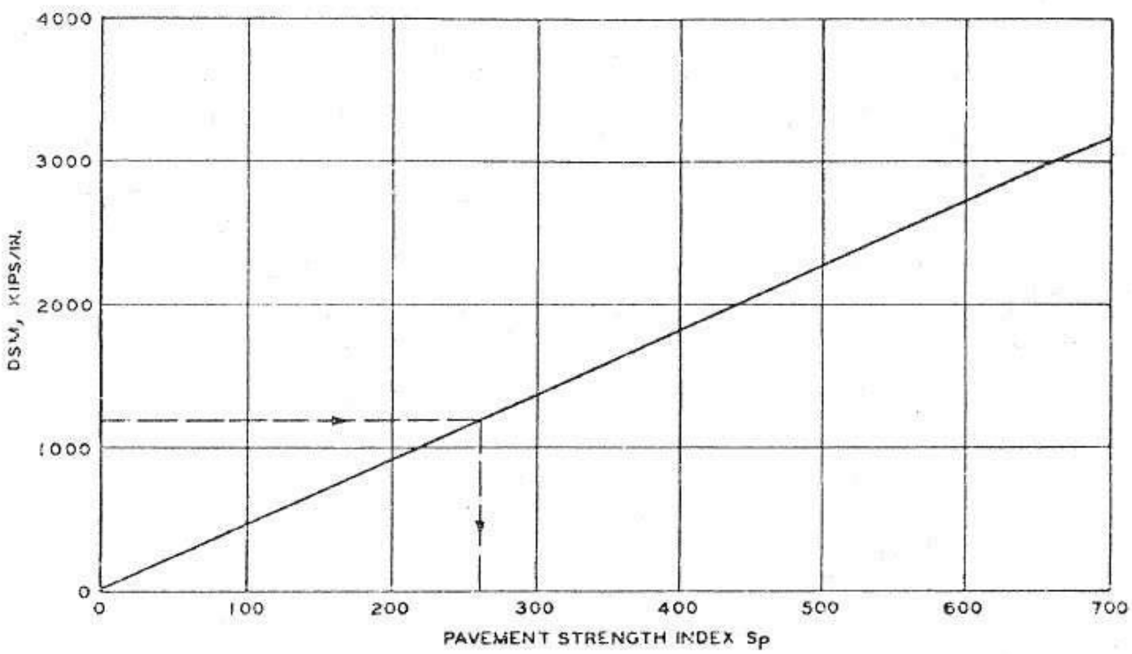


Figure 3-15. Evaluation curve for flexible pavement

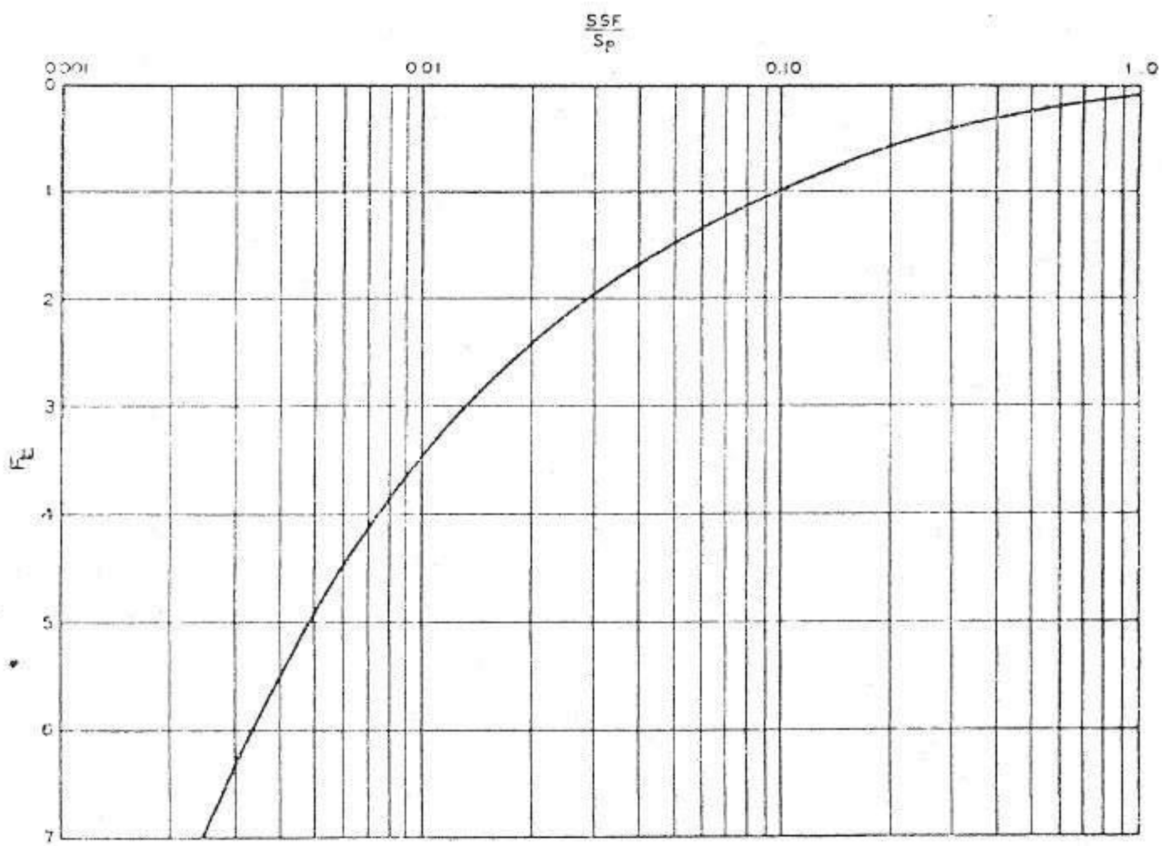


Figure 3-16. F_t versus $\frac{SSF}{S_P}$

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		Total Number of Main Gear Wheels	Aircraft	Tire Contact Area		Total Number of Main Gear Wheels
	cm^2	in^2			cm^2	in^2	
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

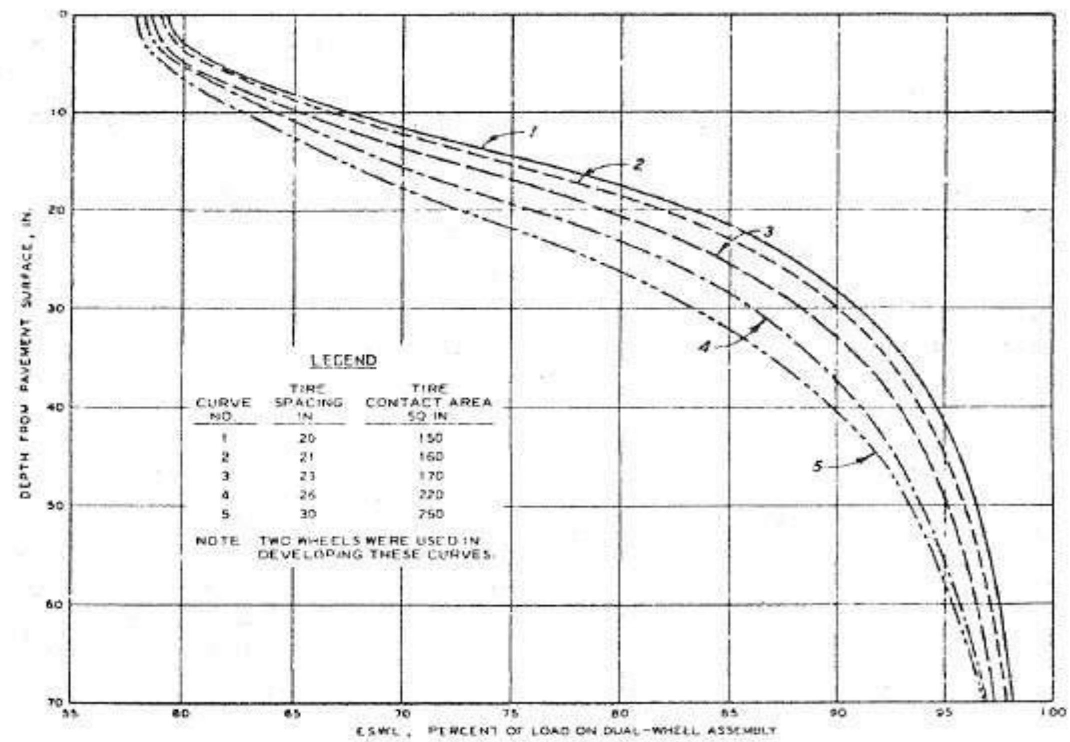


Figure 3-17. ESWL curves for dual wheel aircraft on flexible pavement

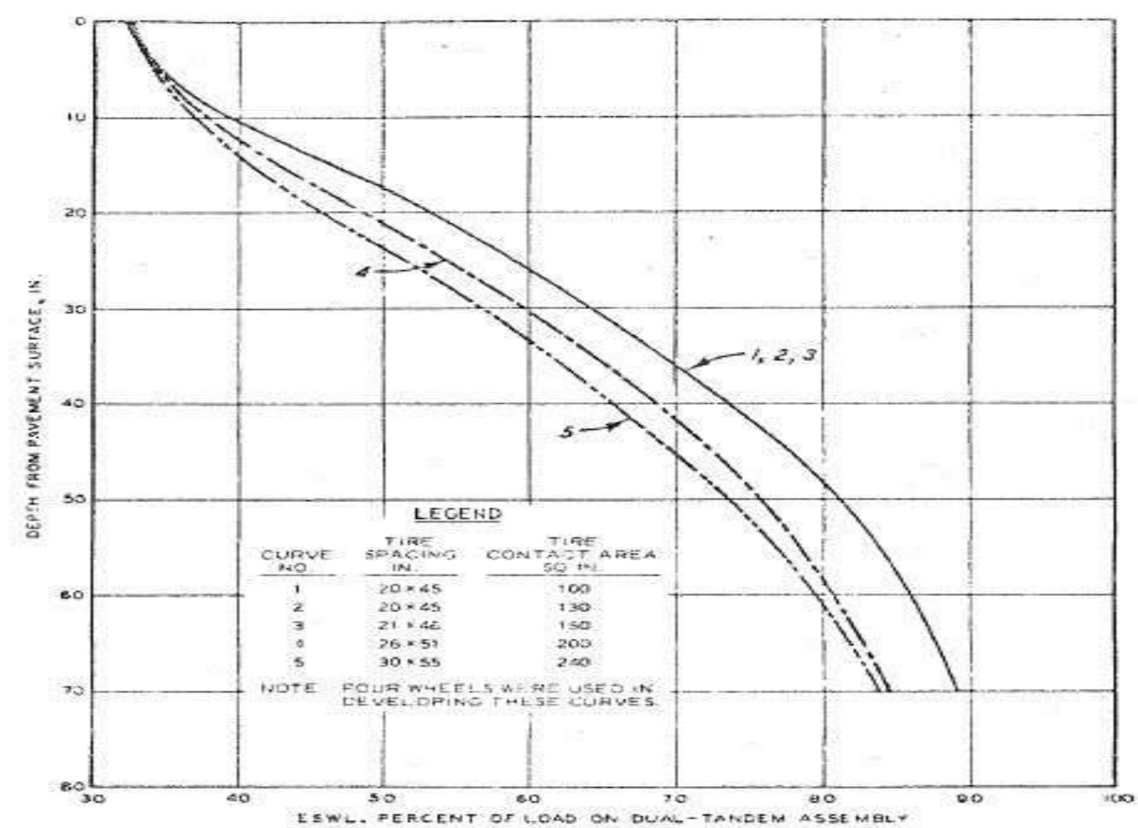


Figure 3-18. ESWL curves for dual tandem aircraft on flexible pavement

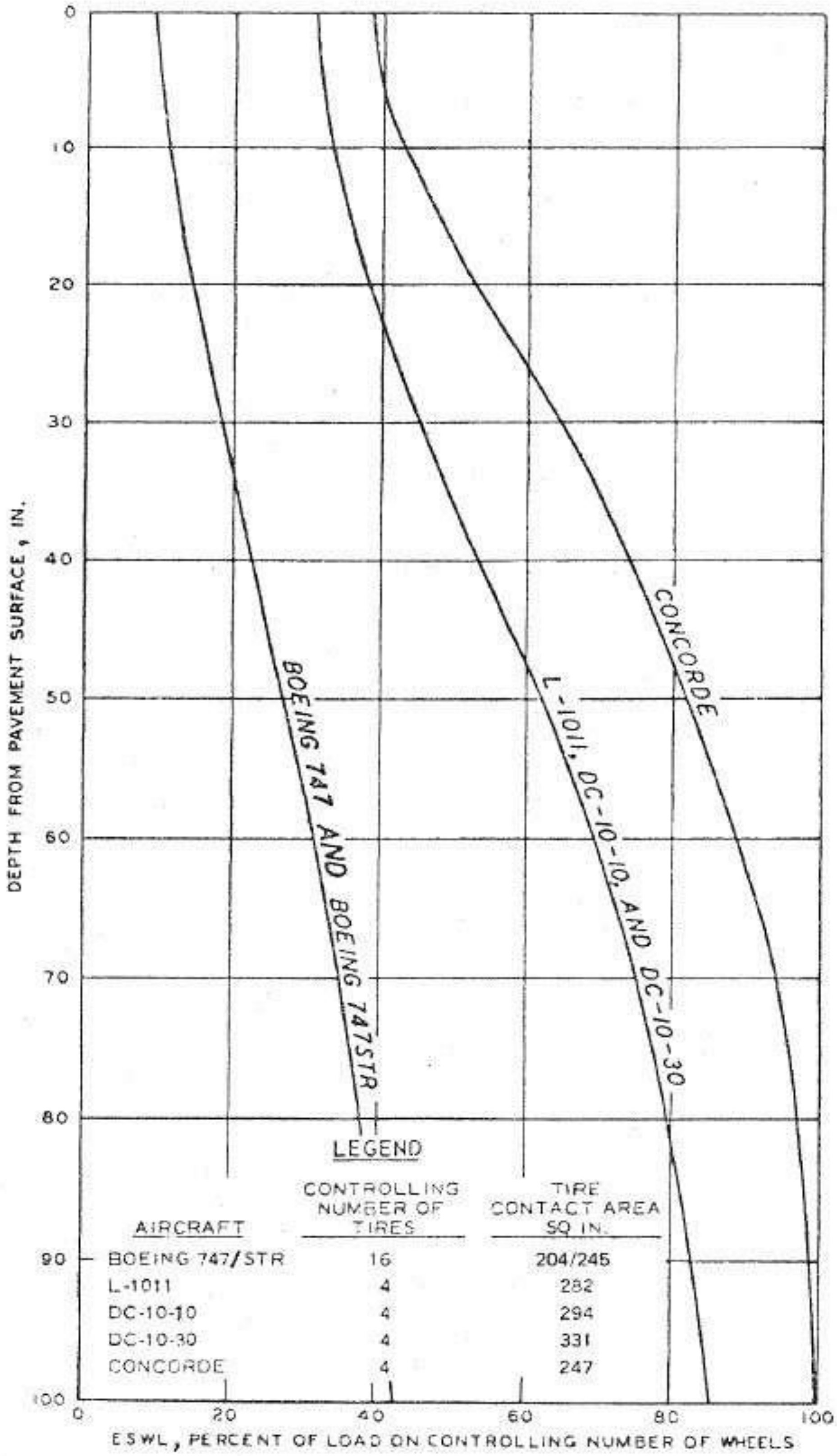


Figure 3-19. ESWL curves for various jet aircraft on flexible pavement

CHAPTER 4: - METHODS FOR IMPROVING RUNWAY SURFACE TEXTURE

4.1. Purpose

4.1.1. 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.

4.2. Basic Considerations

4.2.1. Historical background

4.2.1.1. 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.

4.2.1.2. 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.

4.2.1.3. 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.

4.2.1.4. 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.

4.2.2. Functional requirements

4.2.2.1. 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.

4.2.2.2. 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.

4.2.3. Problem identification

4.2.3.1. 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.

4.2.3.2. 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.

4.2.3.3. 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.

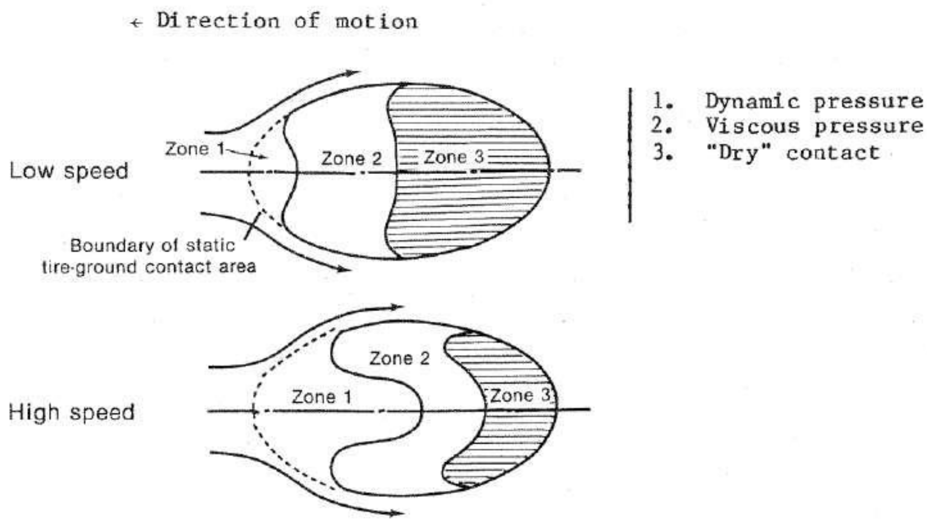


Figure 4-1. Areas of tire/surface interface

4.2.3.4. 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.

4.2.4. Design objectives

4.2.4.1. 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.

4.2.4.2. 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.

4.2.5. Physical design criteria

4.2.5.1. 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.

4.2.5.2. 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.

4.2.5.3. 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.

4.2.5.4. 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 ,

4.2.5.5. 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.

4.2.5.6. 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.

4.2.5.7. 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.

4.2.5.8. 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.

4.2.5.9. 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.

4.2.5.10. 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.

4.2.5.11. 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.

4.2.5.12. 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.

4.2.6. Minimum specifications

4.2.6.1. The basic engineering specifications for the geometrical shape (longitudinal slope/trans verse slope/surface evenness) and for the texture (macrotexture) of a runway surface .

4.2.6.2. Slopes. All new runways should be designed with uniform transverse profile in accordance with the value of transverse slope 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.

4.2.6.3. 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.

4.2.6.4. 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.

4.2.6.5. Failure to meet 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.

4.2.6.6. 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.

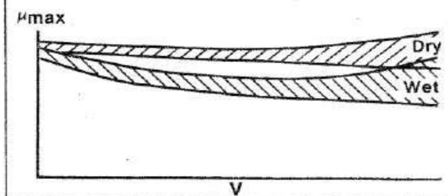
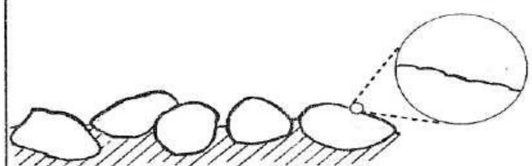
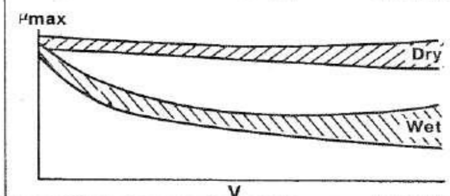
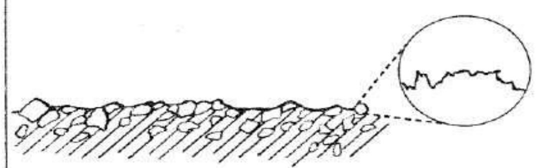
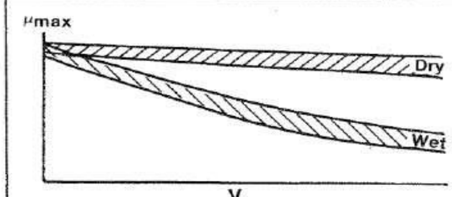
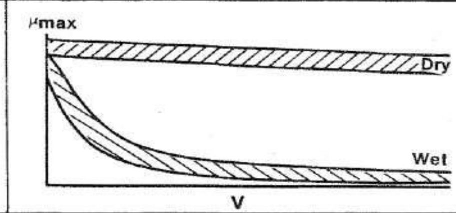
SURFACE				APROXIMATE TRENDS IN MAXIMUM TIRE-GROUND COEFFICIENT OF FRICTION FOR SMOOTH TREAD TIRE
No.	MACRO-TEXTURE		MICRO-TEXTURE	
I	OPEN macro-textured surfaces provide good bulk drainage of tire-ground contact area. In wet conditions μ decreases gradually with increase in V. Tread grooves have little 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 4-2. Effect of surface texture on tire-surface coefficient of friction

4.2.6.7. Surface texture 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.

4.2.6.8. 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.

4.2.6.9. 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.

4.2.6.10. With respect to microtexture there is no direct measure available to define the required fine scale roughness of the individual aggregate in engineering terms. 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.

4.2.6.11. 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.

4.2.6.12. Runway surface friction calibration. 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.

4.2.6.13. 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.

4.3. Surface treatment of runways

4.3.1. General

4.3.1.1. 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.

4.3.2. Surface dressing of asphalt

4.3.2.1. 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.

4.3.2.2. 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.

4.3.2.3. 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.

4.3.2.4. 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.

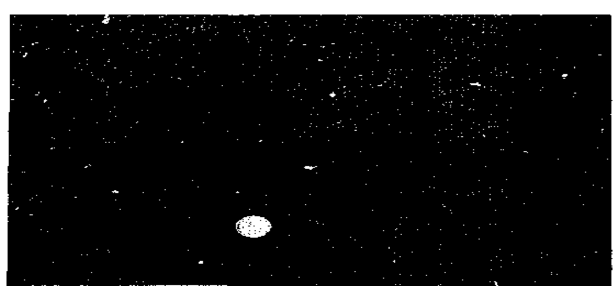


Figure 4-3. Surface dressing of asphalt

4.3.2.5. Chippings. 1be chippings may be from one of the following groups: Basalt, Gabbro, Granite, Gritstone, Hornfels, Porphyry or Quartzite.

4.3.2.6. 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.

4.3.2.7. 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.

4.3.2.8. 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.

4.3.2.9. 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.

4.3.2.10. 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.

4.3.2.11. Application of coated chippings, The temperature of the chippings when applied to the sprayed surface binder must be not less than 83ºC when using bitumen binder and 72ºC when using tar binder. Before and during the rolling operation any bald patches must be covered with fresh chippings.

4.3.2.12. Rolling. The coated chippings must be rolled immediately after spreading and before loss of heat.

4.3.2.13. 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.

4.3.3. Grooving of pavements

4.3.3.1. 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.

4.3.3.2. 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.

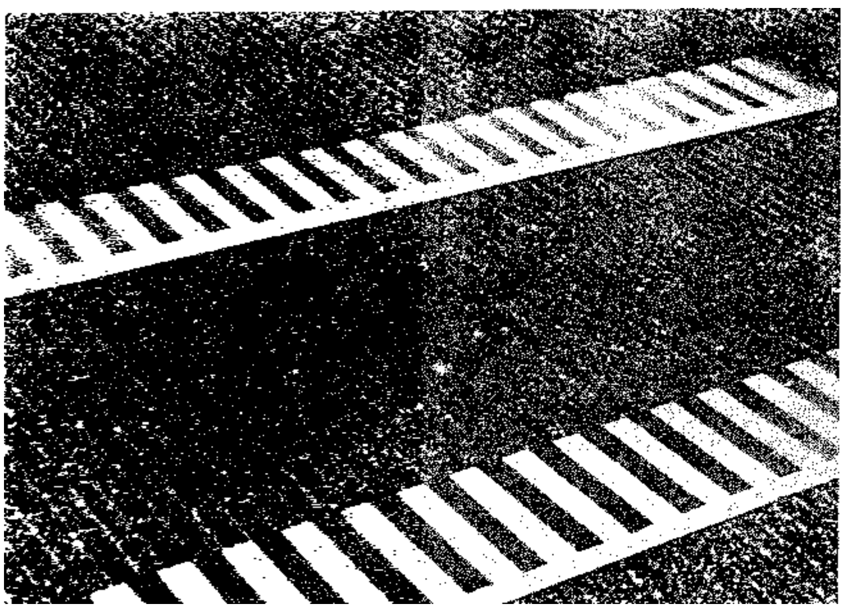


Figure 4-4. Grooving of asphalt surface
(Note.- Scale shows 2.5 cm divisions)

4.3.3.3. 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 micro texture/macro texture, 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.

4.3.3.4. 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 longitudinal 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.

4.3.3.5. 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.

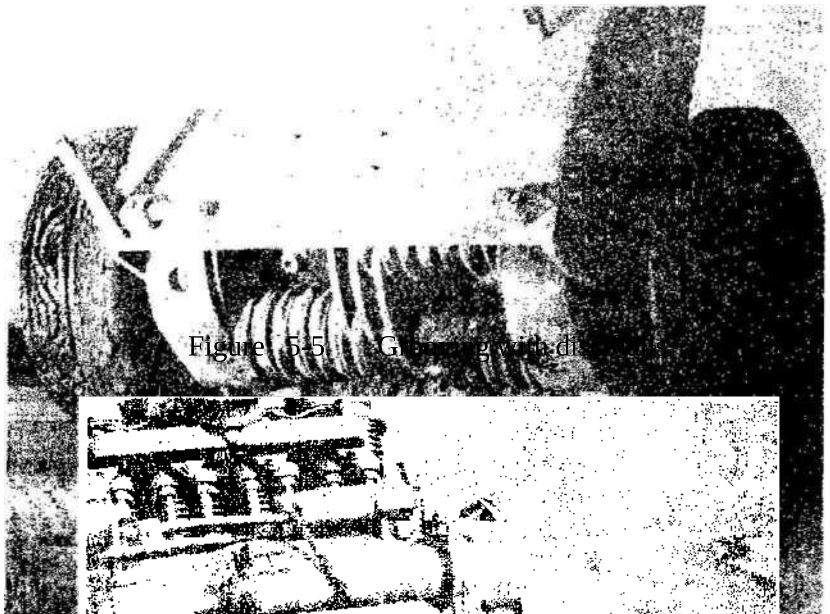


Figure 5-5. G[illegible]g with di[illegible]

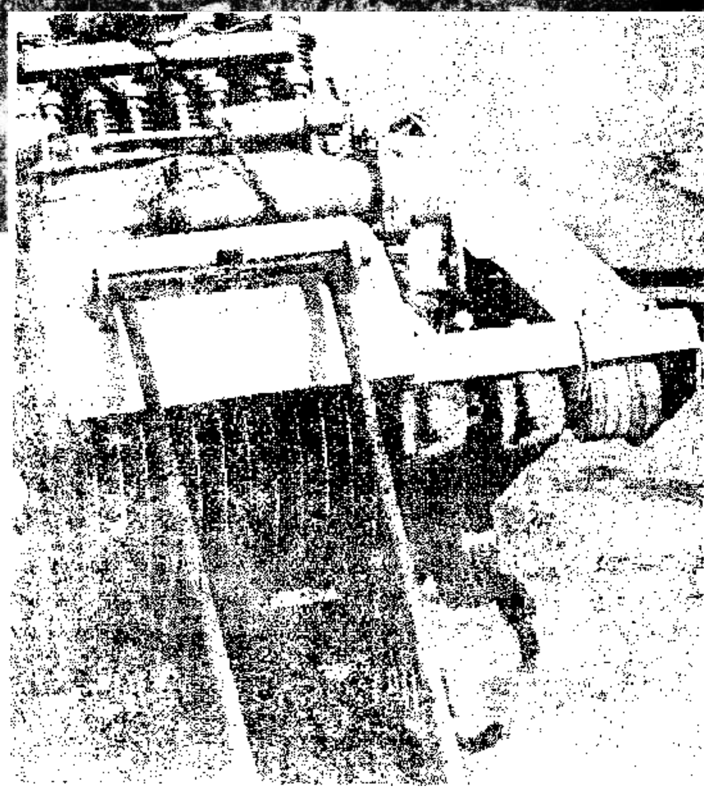


Figure 4-6. Grooving with saws

4.3.3.6. 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.

4.3.3.7. 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.

4.3.3.8. 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.

4.3.3.9. 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.

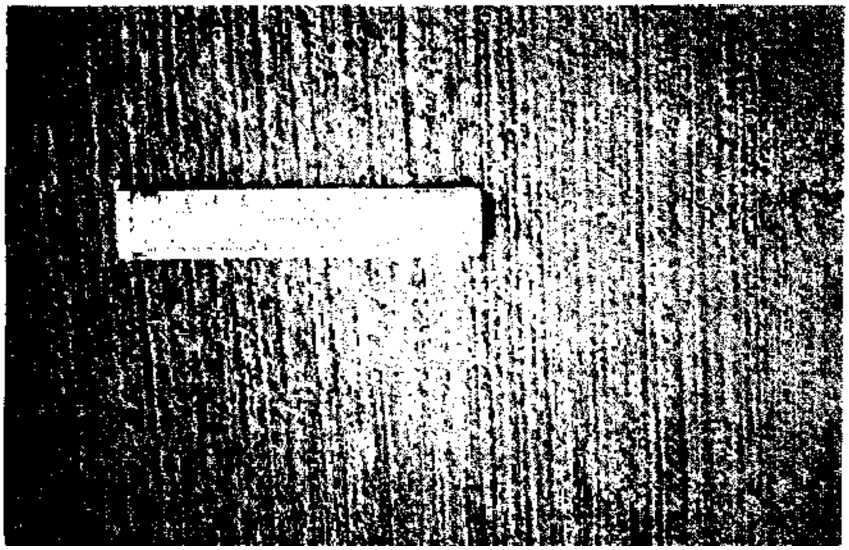


Figure 4-7. New concrete surfacing textured with wire comb

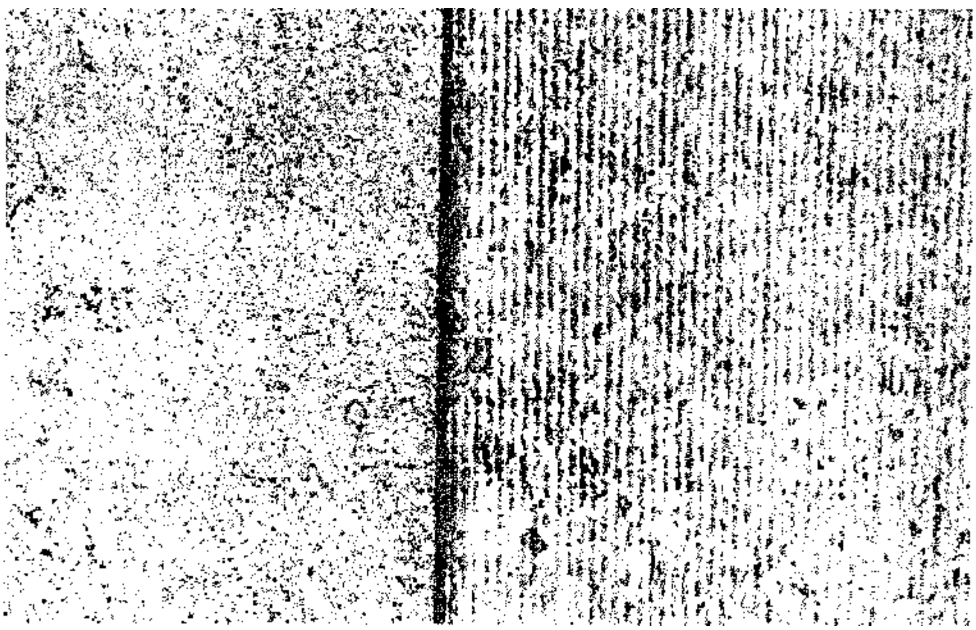


Figure 4-8. Existing Portland cement concrete before and after scoring

4.3.3.10. 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.

Grooving runway intersections

4.3.3.11. 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.

4.3.3.12. 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.

4.3.3.13. 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.

4.3.3.14. 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.

4.3.3.15. 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.

4.3.3.16. 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.

4.3.3.17. 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.

4.3.3.18. 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.

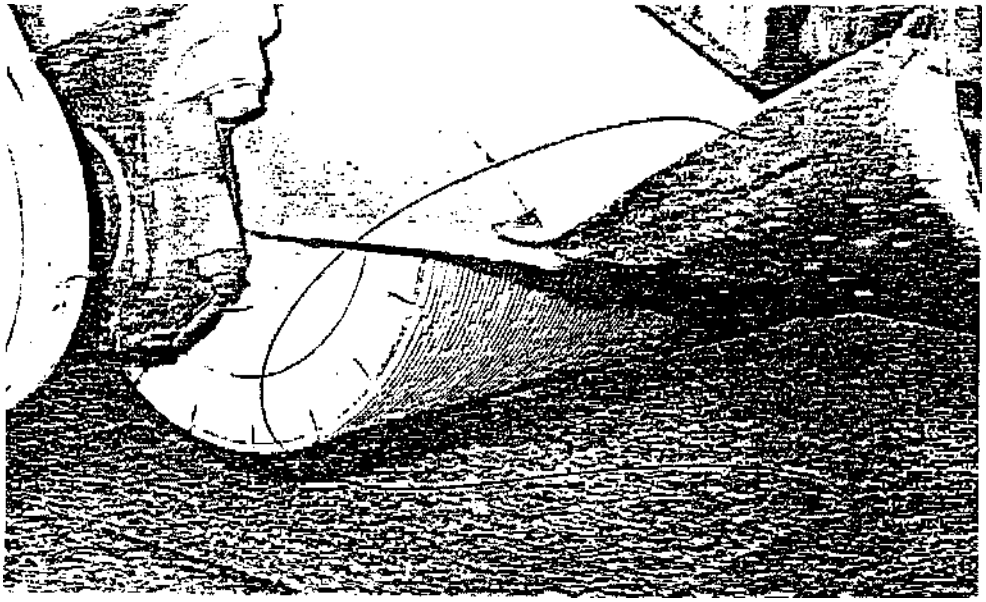


Figure 4-9. Scoring with diamond segmented cutting drum

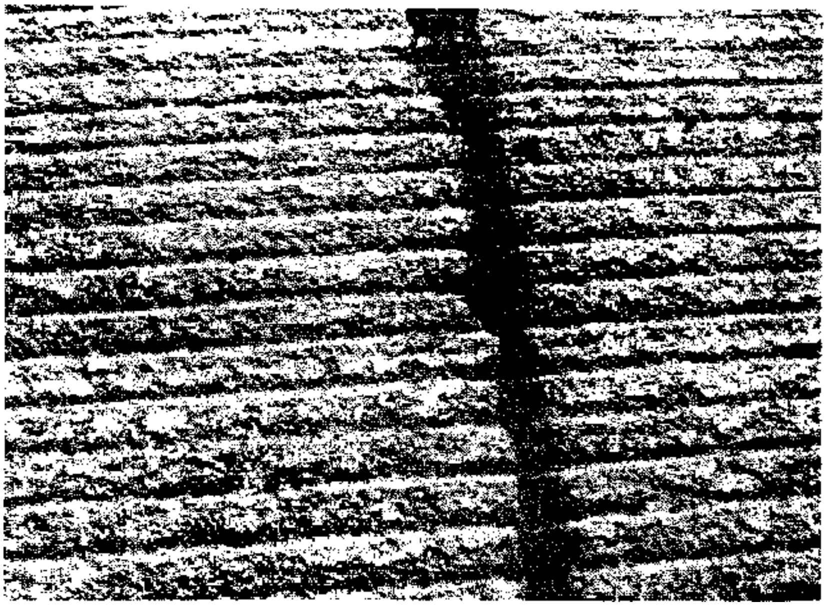


Figure 4-10. Reflex percussive technique - Portland cement concrete

4.3.4. Scoring of cement concrete

4.3.4.1. 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.

4.3.4.2. 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.

4.3.4.3. 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.

4.3.4.4. 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.

4.3.4.5. 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.

4.3.5. Reflex percussive technique

4.3.5.1. 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.

4.3.5.2. 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:

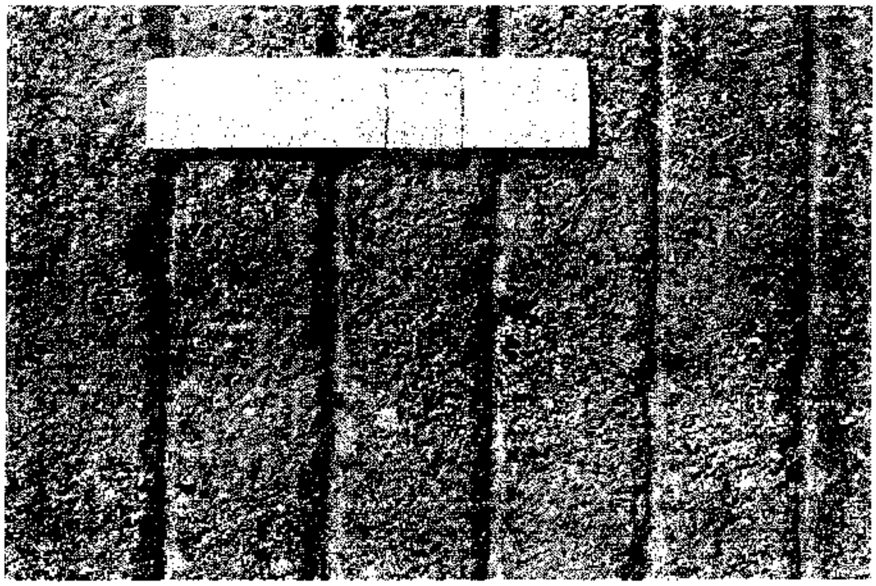


Figure 4-11. Reflex percussive technique - Asphalt surface

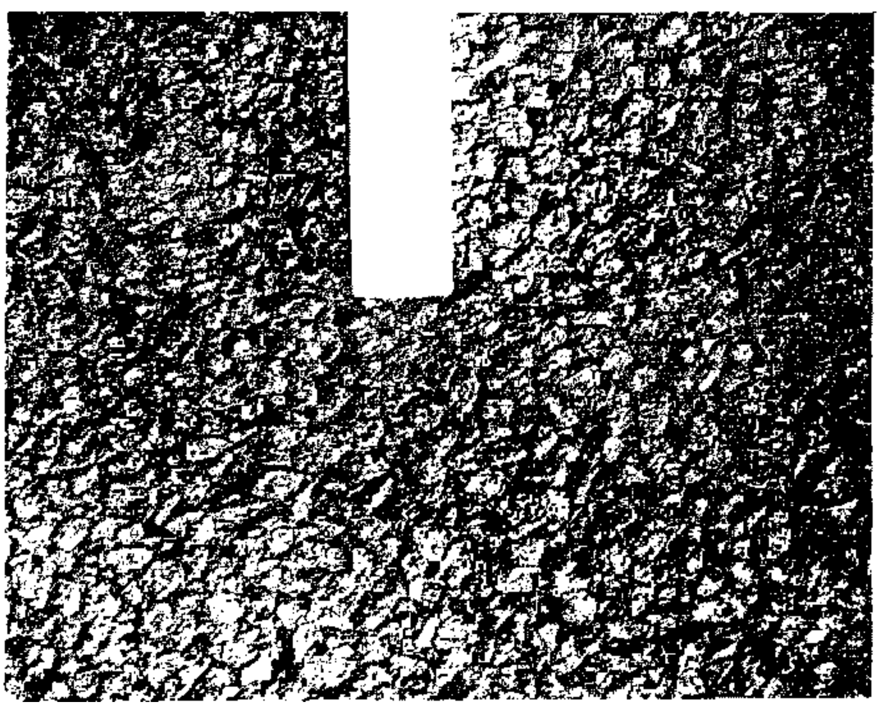


Figure 4-12. Porous friction course surfacing

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 4-11).

4.3.5.3. 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.

4.3.6. Porous friction course

4.3.6.1. 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.

4.3.6.2. 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.

4.3.6.3. 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.

4.3.6.4. 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.

4.3.6.5. 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.

4.3.6.6. 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.

4.3.7. Emulsified asphalt slurry seal

4.3.7.1. 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

4.3.7.2. 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.

4.3.7.3. 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.

4.3.7.4. 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.

4.3.7.5. 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.

4.3.7.6. 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.

4.3.7.7. 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.

4.3.7.8. 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 5: - PROTECTION OF ASPHALT PAVEMENTS

5.1. The problem

5.1.1. 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.

5.1.2. 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.

5.1.3. 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.

5.1.4. 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.

5.1.5. 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.

5.1.6. 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.

5.1.7. 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.

5.2. Treatment of the problem

5.2.1. 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.

5.2.2. 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.

5.2.3. 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.

5.2.4. 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.

5.3. Protective coatings

5.3.1. 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.

5.3.2. 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.

5.3.3. 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.

5.4. Materials for protective coatings

5.4.1. 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.

5.4.2. 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)

5.4.3. 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.

5.4.4. 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.

5.5. Application

5.5.1. 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.

5.5.2. 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.

5.5.3. 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.

5.6. Protection gained

5.6.1. 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.

5.6.2. 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.

5.6.3. 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.

5.6.4. 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.

5.7. Maintenance consideration

5.7.1. 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.

5.8. Some related concerns

5.8.1. 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.

5.8.2. 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.

5.8.3. 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.

5.8.4. 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 6: - STRUCTURAL CONCERNS FOR CULVERTS AND BRIDGES

6.1. Problem description

6.1.1. 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.

6.1.2. 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.

6.1.3. 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.

6.2. Types of substructures

6.2.1. 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.

6.2.2. 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.

6.2.3. 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.

6.2.4. 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.

6.3. Some Guiding Concepts

6.3.1. 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.

6.3.2. 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.

6.3.3. 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.

6.3.4. 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.

6.4. Evaluation of subsurface structures

6.4.1. General

6.4.1.1. 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.

6.4.2. Deeply buried structures

6.4.2.1. 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.

6.4.2.2. 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.

6.4.2.3. 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,

6.4.3. Shallow pipes, conduits, subdrains, and culverts

6.4.3.1. 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.

6.4.3.2. 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.

6.4.4. At surface drains, conduits, and the like

6.4.4.1. 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.

6.4.5. Bridges supporting aerodrome pavements

6.4.5.1. 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.

6.4.6. Pile supported structures

6.4.6.1. 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.

6.4.7. Tunnels under pavements

6.4.7.1. 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.

6.4.8. Treatment of severely limiting cases

6.4.8.1. 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.

6.4.8.2. 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.

6.4.8.3. 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.

6.5. Considerations in design of new facilities

6.5.1. 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.

6.5.2. 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.

6.5.3. 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.

6.5.4. 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.

6.5.5. 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.

6.5.6. 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.

6.5.7. 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.

6.5.8. 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.

6.5.9. 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.

6.5.10. 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 7: - CONSTRUCTION OF ASPHALTIC OVERLAYS

7.1. Introduction

7.1.1. 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.

7.2. Airport authority's role

7.2.1. Project co-ordination

7.2.1.1. 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

7.2.1.2. 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.

7.2.1.3. 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.

7.2.2. Testing requirements

7.2.2.1. 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.

7.2.3. Inspection requirements

7.2.3.1. 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.

7.3. Design considerations

7.3.1. 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.

7.3.2. Pavement survey

7.3.2.1. 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.

7.3.2.2. 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.

7.3.3. Special details

7.3.3.1. Details pertaining to the following items should be included in the plans:

a) <u>Temporary ramps</u>. 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 7-1). 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.

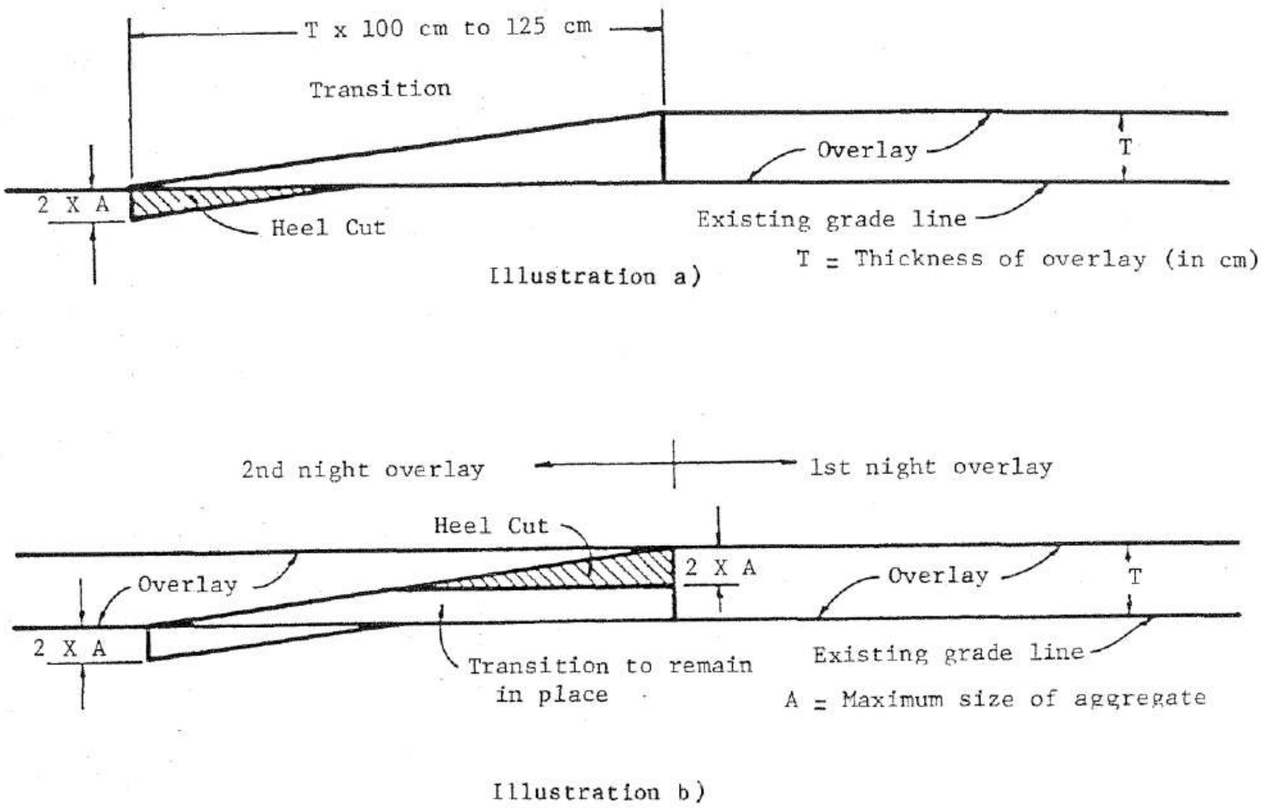


Figure 7-1. Temporary ramp construction with cold planing machine

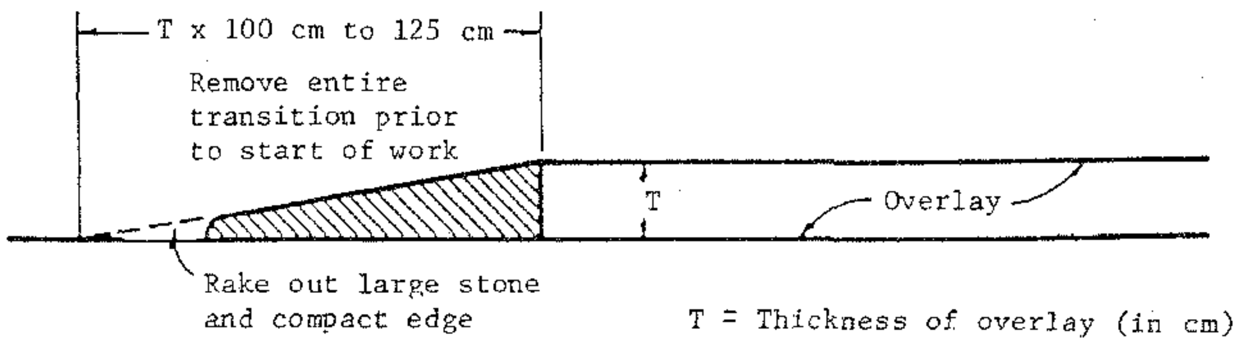


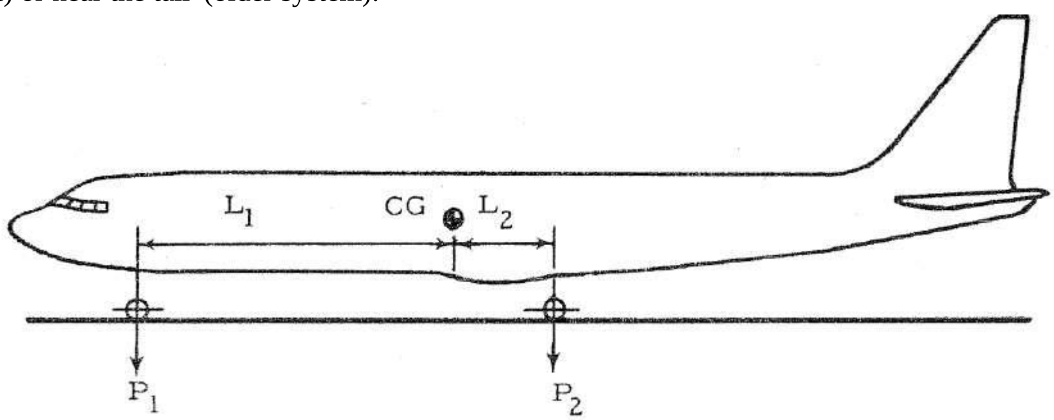
Figure 7-2. Temporary ramp construction without cold planing machine

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

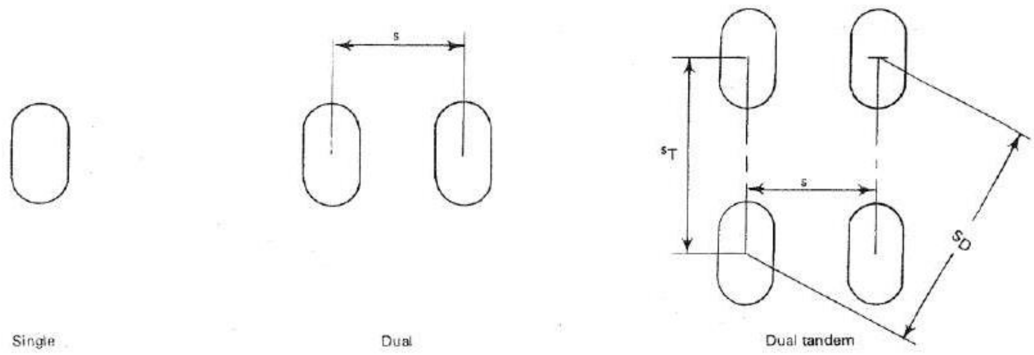
$$W = P_1 + P_2 \qquad P_1 L_1 = P_2 L_2$$

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.

List of abbreviations used in Table Al-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

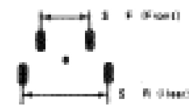
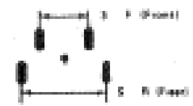
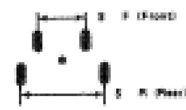
Aircraft type	All-up mass (kg)	Load on one main gear leg (%)	Wheel arrangement	MAIN LEGS OF LANDING GEAR: Load on each leg (kg)	Tire pressure (MPa)	Wheel spacing (cm) (S)	(S_T)	(S_D)	Additional data for complex wheel arrangement
A300 B2 Airbus	137 000	47.0	DT	64 390	1.2	89	140	165.9	
A300 B2 Airbus	142 000	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	

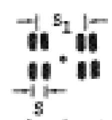
Aircraft type	All-up mass (kg)	Load on one main gear leg (%)	Wheel arrangement	MAIN LEGS OF LANDING GEAR: Load on each leg (kg)	Tire pressure (MPa)	Wheel spacing (cm) (S)	(S_T)	(S_D)	Additional data for complex wheel arrangement
B707-320C Convertible	152 407	46.7	DT	71 174	1.24	88	142	167.1	
B707-320/420	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	-	-	

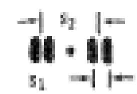
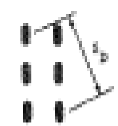
				MAIN LEGS OF LANDING GEAR					
Aircraft type	All-up mass (kg)	Load on one main gear leg (%)	Wheel arrangement	Load on each leg (kg)	Tire pressure (MPa)	Wheel spacing (cm) (S)	(S_T)	(S_D)	Additional data for complex wheel arrangement
B737-200 Advanced	58 332	46.0	D	26 833	1.25	78	-	-	
B737-300	61 462	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 ANO-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 3 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 ANO-14, Part I, 2.5.6 b) and the results tabulated in Attachment A, Table B-1 of that Annex and Table A5-1 of Appendix 3 of this Manual.

Note: For detailed information about computer programme please refer Aerodrome Design Manual Part -3; Pavements

APPENDIX 3: - 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/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
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				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
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/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
B747-100B	341 553/171 870	23.1	1.32	41/17	49/19	58/22	68/26	46/20	51/21	62/23	82/30
B747-100B SR	260 362/164 543	24.1	1.04	27/16	32/17	40/21	47/25	33/19	36/20	43/23	59/30
B747SP	302 093/147 716	22.9	1.30	35/14	42/16	51/19	59/22	40/17	44/17	52/19	71/25
B747SP	318 881/147 996	21.9	1.40	37/14	44/15	52/18	60/21	41/16	45/17	54/18	72/23
B747-200B	352 893/172 886	23.6	1.37	45/18	53/20	64/24	73/28	50/21	55/22	67/24	88/31
B747-200C	373 305/166 749	23.1	1.30	46/16	55/18	66/21	76/25	52/19	57/20	70/22	92/29
B747-200F/300	379 201/156 642	23.2	1.39	47/16	57/17	68/20	78/24	53/18	59/19	73/21	94/26
B747-400	395 987/178 459	23.4	1.41	53/19	63/21	75/25	85/29	57/21	64/22	79/25	101/32
B757-200	109 316/60 260	45.2	1.17	27/12	32/14	38/17	44/19	29/14	32/14	39/17	52/22
B767-200	143 789/78 976	46.2	1.31	33/15	38/17	46/20	54/24	37/18	40/19	47/21	65/26
B767-200-ER	159 755/80 853	46.9	1.21	37/16	44/18	54/21	63/25	43/19	47/19	57/22	77/28
B767-300	159 665/86 070	47.5	1.21	38/17	45/19	54/23	63/27	43/20	48/21	58/24	78/32
B767-300-ER	172 819/87 926	46.9	1.31	43/18	51/20	61/24	71/28	48/21	53/22	65/24	86/32
B767-300-ER	185 520/88 470	46.0	1.38	47/18	56/20	66/24	76/28	51/21	57/22	70/24	92/31
Caravelle Series 10	52 000/29 034	46.1	0.75	15/7	17/8	20/9	22/10	15/7	17/7	19/9	23/11
Caravelle Series 12	55 960/31 800	46.0	0.88	16/8	19/9	22/10	25/12	17/8	19/9	21/10	26/12
Concorde	185 066/78 698	48.0	1.26	61/21	71/22	82/25	91/29	65/21	72/22	81/26	98/32
Canadair CL 44	95 708/40 370	47.5	1.12	25/9	30/10	35/11	40/13	27/9	30/10	36/11	47/14

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