Superseded

CONSOLIDATED REPRINT

This consolidated reprint includes Changes 1 and 2

AC 150/5320-6C

DATE

ADVISORY CIRCULAR

AIRPORT PAVEMENT DESIGN AND EVALUATION

DEPARTMENT OF TRANSPORTATION
Federal Aviation Administration
Washington, D.C.

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ADVISORY CIRCULAR

DEPARTMENT OF TRANSPORTATION
FEDERAL AVIATION ADMINISTRATION

SUBJECT: AIRPORT PAVEMENT DESIGN AND EVALUATION

1. PURPOSE. This circular provides guidance to the public for the design and evaluation of pavements at civil airports.

2. CANCELLATION. AC 150/5320-6B, Airport Pavement Design and Evaluation, dated May 28, 1974, is cancelled.

3. REFERENCES. The publications listed in Appendix 4 provide further guidance and detailed information on the design and evaluation of airport pavements.

4. HOW TO OBTAIN THIS PUBLICATION. Additional copies of this AC 150/5320-6C, Airport Pavement Design and Evaluation, may be obtained free of charge from the Department of Transportation, Subsequent Distribution Unit, M-494.3, Washington, D.C. 20590. FAA field personnel may obtain copies from their respective regional Distribution Officers.

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FOREWORD

This advisory circular is intended to provide guidance on the structural design and evaluation of airport pavements.

Although aircraft landing gears are involved in airport pavement design and evaluation, this circular is not intended to dictate any facet of landing gear design. In 1958, the FAA adopted a policy of limiting maximum Federal participation in airport pavements to a pavement section designed to serve a 350,000-pound (159 000 kg) aircraft with a DC-8-50 series landing gear configuration. In addition, the intent of the policy was to insure that future aircraft were equipped with landing gears which would not stress pavements more than the referenced 350,000-pound (159 000 kg) aircraft. Aircraft manufacturers have accepted and followed the 1958 policy and have designed aircraft landing gear which conform to the policy even though aircraft gross weights have substantially exceeded 350,000 pounds (159 000 kg). This has been accomplished by increasing the number and spacing of landing gear wheels. This circular does not affect the 1958 policy with regard to landing gear design.

The pavement design guidance presented in Chapter 3 of this circular is based on methods of analysis which have resulted from experience and recent research. The change in methods was adopted to exploit these advances in pavement technology and thus provides better performing pavements and easier-to-use design curves. Generally speaking, the new design guidance will require somewhat thicker pavement sections than were required in the past. The cost of increased thickness is eligible under the Airport Development Aid Program (ADAP).

The pavement evaluation portion of this circular is presented in Chapter 6 and is related back to the previous FAA method of design to insure continuity. An aircraft operator could be penalized unfairly if an existing facility were evaluated using a method different from that employed in the original design. A slight change in pavement thickness can have a dramatic effect on the payload or range of an aircraft. Since the new pavement design methodology generally requires slightly greater pavement thicknesses, an evaluation of an existing pavement using the new methodology would likely reduce allowable loads and penalize operators. To avoid this situation the evaluation should be based on the same methodology as was used for design. The evaluation curves in Chapter 6 have been reproduced from AC 150/5320-6B, dated May 28, 1974, to facilitate the evaluation procedure.

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CHAPTER 1. AIRPORT PAVEMENTS - THEIR FUNCTION AND PURPOSES.

1. GENERAL. Airport pavements are constructed to provide adequate support for the loads imposed by aircraft using an airport and to produce a firm, stable, smooth, all-year, all-weather surface free from dust or other particles that may be blown or picked up by propeller wash or jet blast. In order to satisfactorily fulfill these requirements, the pavement must be of such quality and thickness that it will not fail under the load imposed. In addition, it must posses sufficient inherent stability to withstand, without damage, the abrasive action of traffic, adverse weather conditions, and other deteriorating influences. To produce such pavements requires a coordination of many factors of design, construction, and inspection to assure the best possible combination of available materials and a high standard of workmanship.

a. Types of Pavement. Pavements discussed in this circular are flexible, rigid, bituminous overlays, and rigid overlays. Various combinations of pavement types and stabilized layers can result in complex pavements which would be classified in between flexible and rigid. The design and evaluation guidance in this circular can be adapted to any pavement type.

b. Economic Analysis and Design Selection. When properly designed and constructed, any pavement type (rigid, flexible, composite, etc.) can provide a satisfactory pavement for any civil aircraft. However, some designs may be more economical than others and still provide satisfactory performance. The engineer is required to provide a rationale for the selected design in the engineer's report (see AC 150/5300-9). Often this rationale will be based on economic factors derived from evaluating several design alternatives. Life-cycle cost analysis should be used if the design selection is based on least cost. An example of a life-cycle cost analysis of alternatives for pavement rehabilitation is shown in Appendix 5. More details on life-cycle cost analysis can be found in research report DOT/FAA/RD-81/78 (see Appendix 4). Many new developments in construction have evolved in recent times which can significantly affect pavement costs, such as, recycling. In instances where no clear cost advantage can be established in the design process, alternate bids should be taken. Design selection is not always controlled by economic factors. Operational constraints, funding limitations, future expansion, etc., can override economic factors in the design selection. These considerations should be addressed in the engineer's report. *

c. Pavement Courses.

(1) Surface courses include portland cement concrete, bituminous concrete, sand-bituminous mixture, and sprayed bituminous surface treatments.

(2) Base courses consist of a variety of different materials which generally fall into two main classes, treated and untreated. The untreated bases consist of crushed or uncrushed aggregates. The treated bases normally consist of a crushed or uncrushed aggregate that has been mixed with a stabilizer such as cement, bitumen, etc.

(3) Subbase courses consist of a granular material, a stabilized granular material, or a stabilized soil.

2. STANDARDS AND SPECIFICATIONS.

a. Reference is made by Item Number throughout the text to construction material standards contained in AC 150/5370-10, Standards for Specifying Construction of Airports.

b. Geometric standards concerning pavement lengths, widths, grades, and slopes are presented in advisory circulars listed in Appendix 4.

3. SPECIAL CONSIDERATIONS. Airport pavements should provide a surface which is not slippery and will provide good traction during any weather conditions. AC 150/5320-12, Measurement, Construction and Maintenance of Skid Resistant Airport Pavement Surfaces, presents information on skid resistant surfaces.

4. STAGE CONSTRUCTION OF AIRPORT PAVEMENTS.

a. In some instances it may be necessary to construct the airport pavement in stages; that is, to build up the pavement profile, layer by layer, as the traffic using the facility increases in weight and number. Lateral staging, i.e., planning for future widening of pavements is sometimes advantageous to accommodate larger aircraft. If stage construction is to be undertaken, the need for sound planning cannot be overemphasized. The complete pavement should be designed prior to the start of any stage, and each stage must provide an operational surface.

The planning of a stage constructed pavement should recognize a number of considerations such as the following:

(1) Careful economic studies are required to determine if staged construction is warranted. Construction materials and labor costs follow inflationary trends and can be expected to increased as later stages are constructed. The costs and time involved in any pavement shutdown or diversion of traffic necessitated by the construction of any stage should be considered. The costs of mobilizing construction equipment several times should be compared with mobilizing once. The costs of maintaining an intermediate stage should be considered.

(2) Each stage should be designed to adequately accommodate the traffic which will use the pavement until the next stage is constructed.

(3) The underlying layers and drainage facilities of a stage constructed pavement should be built to the standards required for the final cross section. Providing the proper foundation and drainage facilities in the first stage is mandatory as the underlying layers will not be readily accessible for upgrading in the future.

(4) All parties concerned and, insofar as practicable, the general public should be informed that staged construction is planned. Staged construction sometimes draws unjust criticism when relatively new facilities are upgraded for the next stage.

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CHAPTER 2. SOIL INVESTIGATIONS AND EVALUATION.

5. GENERAL. The importance of accurate identification and evaluation of pavement foundations cannot be overemphasized. Although it is impossible to explore the entire field of soil mechanics in a publication such as this, the following text will highlight those aspects which are particularly important to the airport paving engineer.

a. A classification system for soils which are to be used in connection with design of airport pavements is set forth in this chapter. To avoid misunderstanding, certain terms employed are defined below:

(1) For engineering purposes, and particularly as it applies to airports, soil includes all natural deposits which, without requiring blasting under unfrozen conditions, can be moved with earthmoving equipment. Harder materials are considered to be rock.

(2) Soil conditions include such items as the elevation of the water table, the presence of water bearing strata, and the field properties of the soil. Field properties of the soil include the soil's density, moisture content, and frost penetration.

(3) The soil profile is the vertical arrangement of layers of soils, each of which possesses different physical properties from the adjacent layer.

(4) Subgrade soil is that soil which forms the foundation for the pavement. It is the soil directly beneath the pavement structure.

b. Soil conditions and the local prices of suitable construction materials are the most important items affecting the cost of construction of the landing areas and the pavements. Earthwork and grading costs are directly related to the difficulty with which excavation can be accomplished and compaction obtained.

c. It should be remembered that the subgrade soil supports the pavement and the loads imposed on the pavement surface. The pavement serves to distribute the imposed load to the subgrade over an area greater than that of the tire contact area. The greater the thickness of pavement, the greater is the area over which the load on the subgrade is distributed. It follows, therefore, that the more unstable the subgrade soil, the greater is the required area of load distribution and consequently the greater is the required thickness of pavement. The soils having the best engineering characteristics encountered in the grading and excavating operations should be incorporated in the upper layers of the subgrade by selective grading if economically feasible.

d. In addition to the relationship which soil conditions bear to grading and paving operations, they determine the necessity for underdrains and materially influence the amount of surface runoff. Thus, they have a resulting effect on the size and extent of other drainage structures and facilities. (See FAA publication, AC 150/5320-5, Airport Drainage.)

6. SOIL INVESTIGATIONS.

a. To provide essential information on the various types of soils, investigations should be made to determine their distribution and physical properties. The information so obtained, when combined with data on site topography and area climatic records, provides basic planning material essential to the logical and effective development of the airport. An investigation of soil conditions at an airport site will include:

(1) A soil survey to determine the arrangement of different layers of the soil profile with relation to the proposed subgrade elevation.

(2) Sampling of the layers of soil.

(3) Testing of samples to determine the physical properties of the various soil materials with respect to in-place density and subgrade support.

(4) A survey to determine the availability of materials for use in construction of the subgrade and pavement.

b. With respect to sampling and surveying procedures and techniques, ASTM D-420, Investigating and Sampling Soils and Rock for Engineering Purposes, is one of the most frequently used. This method is based entirely on the soil profile. In the field, the various layers that comprise the soil profile are identified by such characteristics as color, texture, structure, consistency, compactness, cementation, and to varying degrees, chemical composition.

(1) The use of Department of Agriculture soils maps, United States Geodetic Survey (USGS) geologic maps, and USGS engineering geology maps can prove valuable aids in the study of soils at and in the vicinity of the airport. Although the pedological classification, determined from these maps, does not treat soil as an engineering or construction material, data so obtained are extremely useful to the agronomist in connection with the development of turf areas on airports and to the engineer concerned with preliminary investigations of site selection, development costs, and alignment.

(2) The practice of determining data on soils by use of aerial photographs is established and commonly acceptable. Relief, drainage, and soil patterns may be determined from the photographs, and an experienced photo interpreter can define differences in characteristics of soils. By employing this method of investigation, it is possible to expedite soil studies and reduce the amount of effort required to gather data.

7. SURVEYING AND SAMPLING.

a. The initial step in an investigation of soil conditions is a soil survey to determine the quantity and extent of the different types of soil, the arrangement of soil layers, and the depth of any subsurface water. These profile borings are usually obtained with a soil auger or similar device. Washed borings are not recommended due to inaccuracies of depth determinations. The intent of the borings is to determine the soil or rock profile and its lateral extent. Inasmuch as each location presents its particular problems and variations, the spacing of borings cannot always be definitely specified by rule or preconceived plan. Suggested criteria for the location, depth, and number of borings are given in Table 2-1. Wide variations in these criteria can be expected due to local conditions.

b. Obviously, the locations, depths, and numbers of borings must be such that all important soil variations can be determined and mapped. Whenever past experience at the location in question has indicated that settlement or stability in deep fill areas may be a problem or, if in the opinion of the engineer, additional investigations are warranted, more or deeper borings may be required in order that the proper design, location, and construction procedures may be determined. Conversely, where uniform soil conditions are encountered, fewer borings may be acceptable.

TABLE 2-1. RECOMMENDED SOIL BORING SPACINGS AND DEPTHS

AREA	SPACING	DEPTH
Runways and Taxiways	Along Centerline, 200 Feet (68 m) on Centers	Cut Areas - 10' (3.5 m) Below Finished Grade Fill Areas - 10' (3.5 m) Below Existing Ground Surface 1/
Other Areas of Pavement	1 Boring per 10,000 Square Feet (930 sq m) of Area	Cut Areas - 10' (3.5 m) Below Finished Grade Fill Areas - 10' (3.5 m) Below Existing Ground Surface 1/
Borrow Areas	Sufficient Tests to Clearly Define the Borrow Material	To Depth of Proposed Excavation of Borrow

1/ For deep fills, boring depths shall be used as necessary to determine the extent of consolidation and slippage, which the fill to be placed may cause.

c. A graphic log of soil conditions can be of great value in assessing subgrade conditions. It is recommended that the graphic log be developed which summarizes the results of the soil explorations. A typical graphic log is included as Figure 2-1. The graphic log should include:

(1) Location

(2) Date performed

(3) Type of exploration

(4) Surface elevation

(5) Depth of materials

(6) Sample identification numbers

(7) Classification

(8) Water table

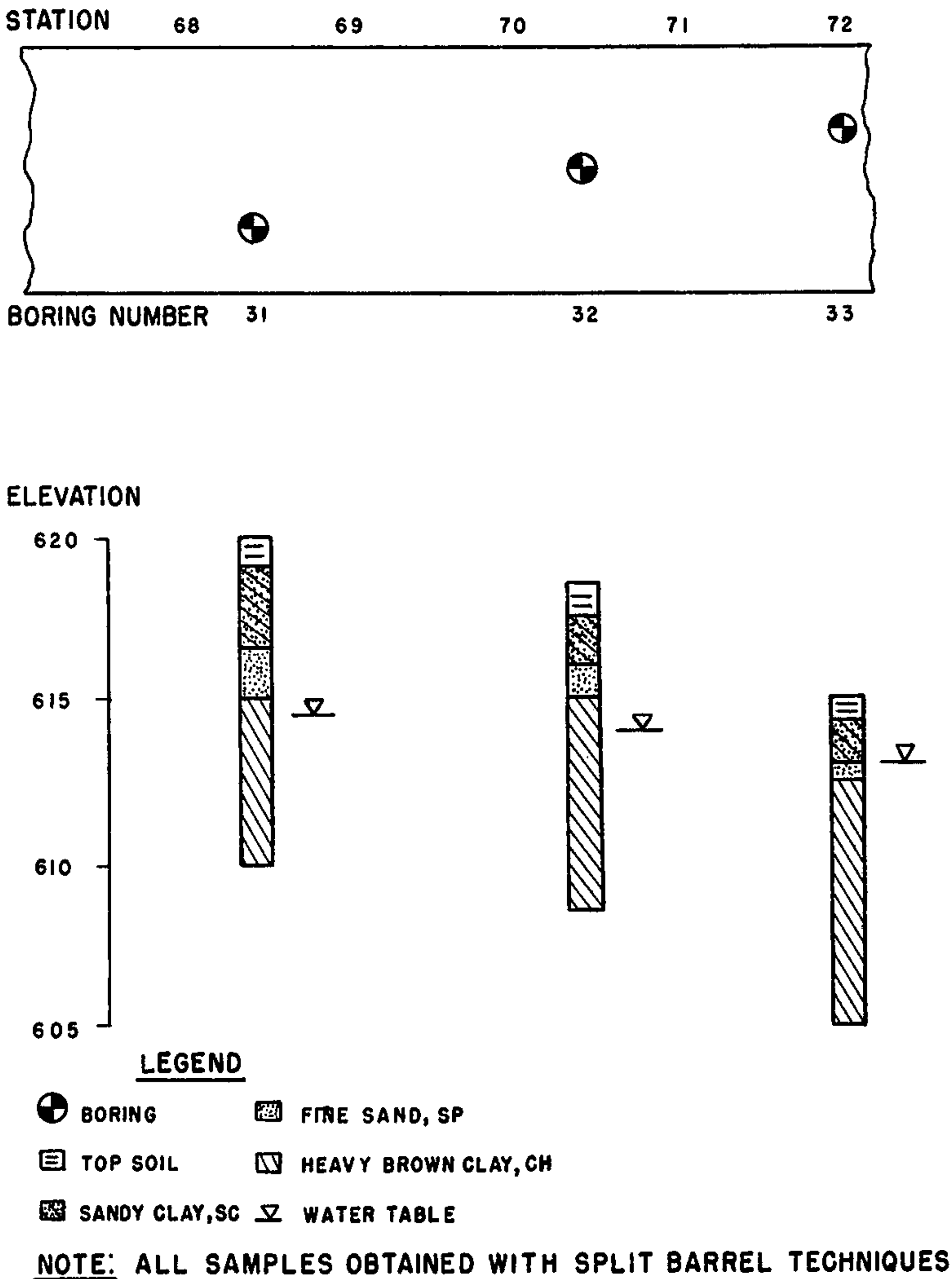


FIGURE 2-1. TYPICAL GRAPHIC BORING LOG

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

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

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

8. SOIL TESTS.

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

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

(1) Dry Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants (ASTM D-421) or Wet Preparation of Soil Samples for Grain-Size Analysis and Determination of Soil Constants (ASTM D-2217). The dry method (D-421) should be used only for clean, cohesionless granular materials. The wet method (D-2217) should be used for all cohesive or borderline materials. In case of doubt, the wet method should be used.

(2) Particle-Size Analysis of Soils (ASTM C-422). This analysis provides a quantitative determination of the distribution of particle sizes in soils.

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

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

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

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

(a) For pavements designed to serve aircraft weighing 30,000 pounds (13 000 kg) or more, use ASTM Method D-1557.

(b) For pavements designed to serve aircraft weighing less than 30,000 pounds (13 000 kg), use ASTM Method D-698.

c. Supplemental Tests. In many cases additional soil tests will be required over those listed in paragraph 8b above. It is not possible to cover all the additional tests which may be required; however, a few examples are presented below. This list is not to be considered a complete list by any means.

(1) Shrinkage Factors of Soils (ASTM D-427). This test may be required in areas where swelling soils might be encountered.

(2) Permeability of Granular Soils (ASTM D-2434). This test may be needed to assist in the design of subsurface drainage.

(3) Determination of Organic Material in Soils by Wet Combustion (AASHTO T-194). This test may be needed in areas where deep pockets of organic material are encountered or suspicioned.

(4) Bearing Ratio of Laboratory-Compacted Soils (ASTM D-1883). This test is used to assign a California Bearing Ratio (CBR) value to subgrade soils for use in the design of flexible pavements.

(5) Modulus of Soil Reaction (AASHTO T 222). This test is used to determine the modulus of soil reaction, K, for use in the design of rigid pavements.

(6) California Bearing Ratio, Field In-place Tests. Field bearing tests can be performed when the in situ conditions satisfy density and moisture conditions which will exist under the pavement being designed.

9. UNIFIED SOIL CLASSIFICATION SYSTEM.

a. The standard method of classifying soils for engineering purposes is ASTM D-2487, commonly called the Unified system. The change from the FAA system to the Unified system is based on the results of a research study which compared three different methods of soil classification. The research study concluded the Unified system is superior in detecting properties of soils which affect airport pavement performance. The primary purpose in determining the soil classification is to enable the engineer to predict probable field behavior of soils. The soil constants in themselves also provide some guidance on which to base performance predictions. The Unified system classifies soils first on the basis of grain size, then further subgroups soils on the plasticity constants. Table 2-2 presents the classification of soils by the Unified system.

b. As indicated in Table 2-2, the initial division of soils is based on the separation of coarse-and fine-grained soils and highly organic soils. The distinction between coarse and fine grained is determined by the amount of material retained on the No. 200 sieve. Coarse-grained soils are further subdivided into gravels and sands on the basis of the amount of material retained on the No. 4 sieve. Gravels and sands are then classed according to whether or not fine material is present. Fine-grained soils are subdivided into two groups on the basis of liquid limit. A separate division of highly organic soils is established for materials which are not generally suitable for construction purposes.

TABLE 2-2. CLASSIFICATION OF SOILS FOR AIRPORT PAVEMENT APPLICATIONS

MAJOR DIVISIONS			Group Symbols
Coarse-grained Soils more than 50% retained on No. 200 sieve 1/	Gravels 50% or more of coarse fraction retained on No. 4 sieve	Clean Gravels	GW GP
		Gravels with Fines	GM GC
	Sands less than 50% of coarse fraction retained on No. 4 sieve	Clean Sands	SW SP
		Sands with Fines	SM SC
Fine-grained Soils 50% or less retained on No. 200 sieve 1/	Silts and Clays Liquid Limit 50% or less		ML CL OL
	Silts and Clays Liquid Limit Greater than 50%		MH CH OH
Highly Organic Soils			PT

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

The final classification of soils subdivides materials into 15 different groupings. The group symbols and a brief description of each is given below:

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

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

(3) GM - Silty gravels, gravel-sand-silt mixtures.

(4) GC - Clayey gravels, gravel-sand-clay mixtures.

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

(6) SP - Poorly graded sands and gravelly sands, little or no fines.

(7) SM - Silty sands, sand-silt mixtures.

(8) SC - Clayey sands, sand-clay mixtures.

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

(10) CL - Inorganic clays of low to medium plasticity, gravelly clays, silty clays, lean clays,

(11) OL - Organic silts and organic silty clays of low plasticity.

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

(13) CH - Inorganic clays or high plasticity, fat clays.

(14) OH - Organic clays of medium to high plasticity.

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

c. Determination of the final classification group requires other criteria in addition to that given in Table 2-2. These additional criteria are presented in Figure 2-2 and have application to both coarse- and fine-grained soils.

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

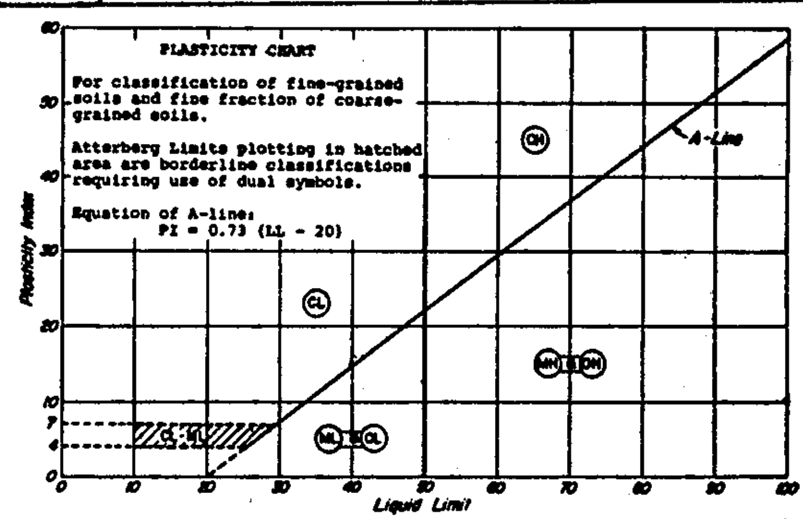


FIGURE 2-2. SOIL CLASSIFICATION CRITERIA

d. A flow chart which outlines the soil classification process has been developed and is included as Figure 2-3. This flow chart indicates the steps necessary to classify soils in accordance with ASTM D-2487.

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

f. A table of pertinent characteristics of soils used for pavement foundations is presented in Table 2-3. These characteristics are to be considered as approximate, and the values listed are generalizations which should not be used in lieu of testing.

10. EXAMPLES. The following examples illustrate the classification of soils by the Unified system. The classification process progresses through the flow chart shown in Figure 2-3.

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

(1) Percent passing No. 200 sieve - 98%.

(2) Liquid limit on minus 40 material - 30%.

(3) Plastic limit on minus 40 material - 10%.

(4) Above "A" line, see Figure 2-2. The soil would be classified as CL, lean clay of low to medium plasticity. Table 2-3 indicates the material would be of fair to poor value as a foundation when not subject to frost action. The potential for frost action is medium to high.

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

(1) Percent passing No. 200 sieve - 48%.

(2) Percent of coarse fraction retained on No. 4 sieve - 70%.

(3) Liquid limit on minus 40 fraction - 60%.

(4) Plastic limit on minus 40 fraction - 20%.

(5) Compute Plasticitv Index LL-PL - 40%.

(6) Above "A" line, see Figure 2-2.

(7) This sample is classified as GC, clayey gravel. Table 2-3 indicates the material is good for use as a pavement foundation when not subject to frost action. The potential for frost action is slight to medium.

c. A more detailed discussion of the Unified Soil Classification System can be found in Appendix 4.

11. FAA SOIL CLASSIFICATION SYSTEM. Since the adoption of the ASTM D-2487 method of soil classification represents a radical change in FAA standards, the previous method of soil classification is included in Appendix 1 to this advisory circular. This is considered necessary in order to allow for transitioning from the old system to the new.

12. FROST AND PERMAFROST. The design of pavements in areas subject to frost action or in areas of permafrost is a complex problem requiring detailed study. The detrimental effects of frost action may be manifested in frost heave or in loss of foundation support through frost melting. Refer to Chapter 3 for the application of frost and permafrost design guidance to pavement design examples.

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

(1) Complete protection method involves the removal of frost susceptible material to the depth of frost penetration and replacing the material with nonfrost susceptible material.

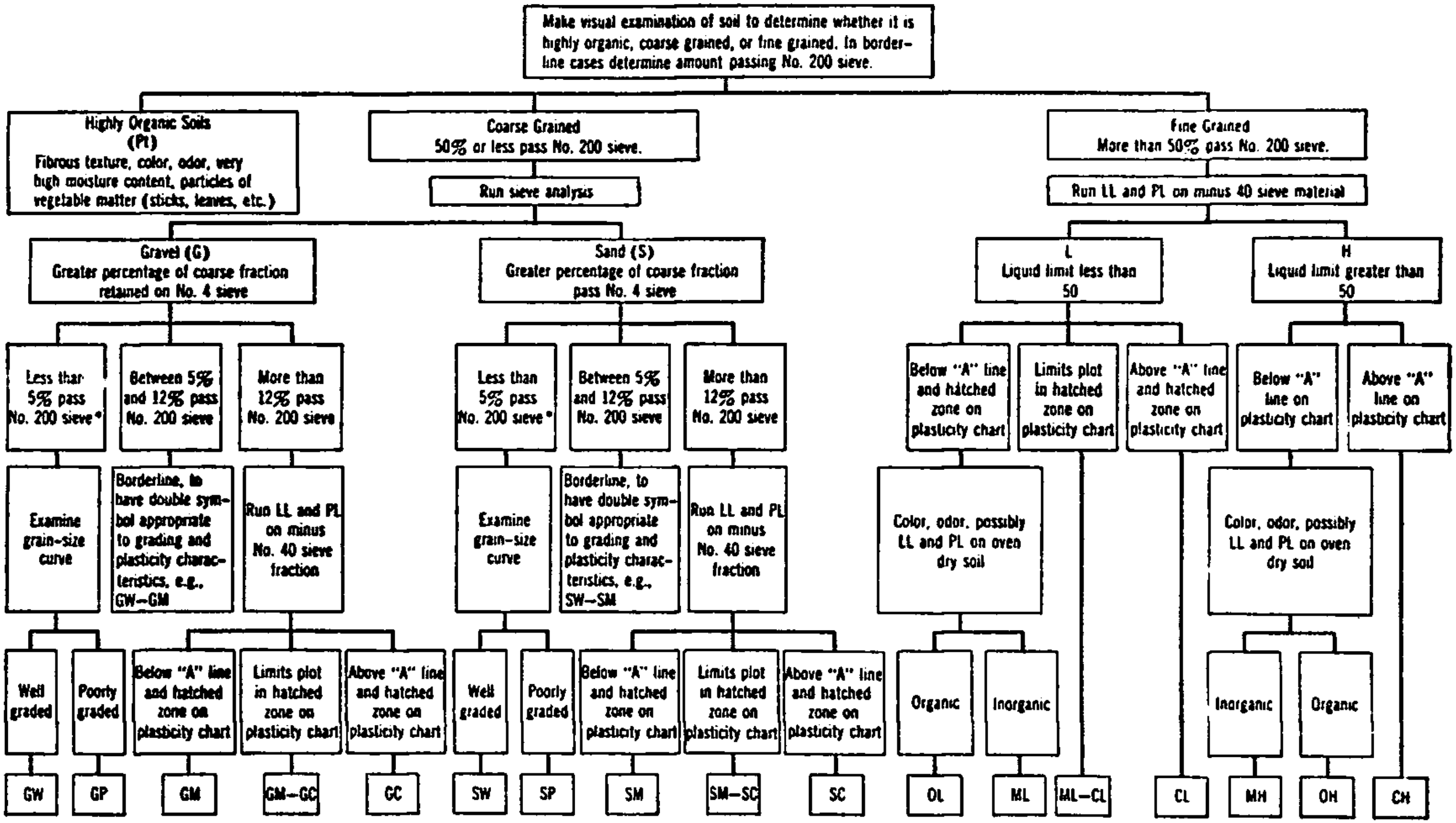
(2) Limited subgrade frost penetration method allows the frost to penetrate a limited depth into the frost susceptible subgrade. This method holds deformations to small acceptable values.

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

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

TABLE 2-3. CHARACTERISTICS PERTINENT TO PAVEMENT FOUNDATIONS

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



Note: Sieve sizes are U.S. Standard.

* If fines interfere with free-draining properties use double symbol such as GW-GM, etc.

FIGURE 2-3. FLOW CHART FOR UNIFIED SOIL CLASSIFICATION SYSTEM

b. The design of pavements in permafrost areas requires efforts to restrict the depth of thaw. Thawing of the permafrost can result in loss of bearing strength. If thawed permafrost is refrozen, heaving can result and cause pavement roughness and cracking. Two methods of design are available for construction in permafrost areas, complete protection method and the reduced subgrade strength method. These methods are somewhat similar to the methods discussed under paragraph 12a for seasonal frost design.

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

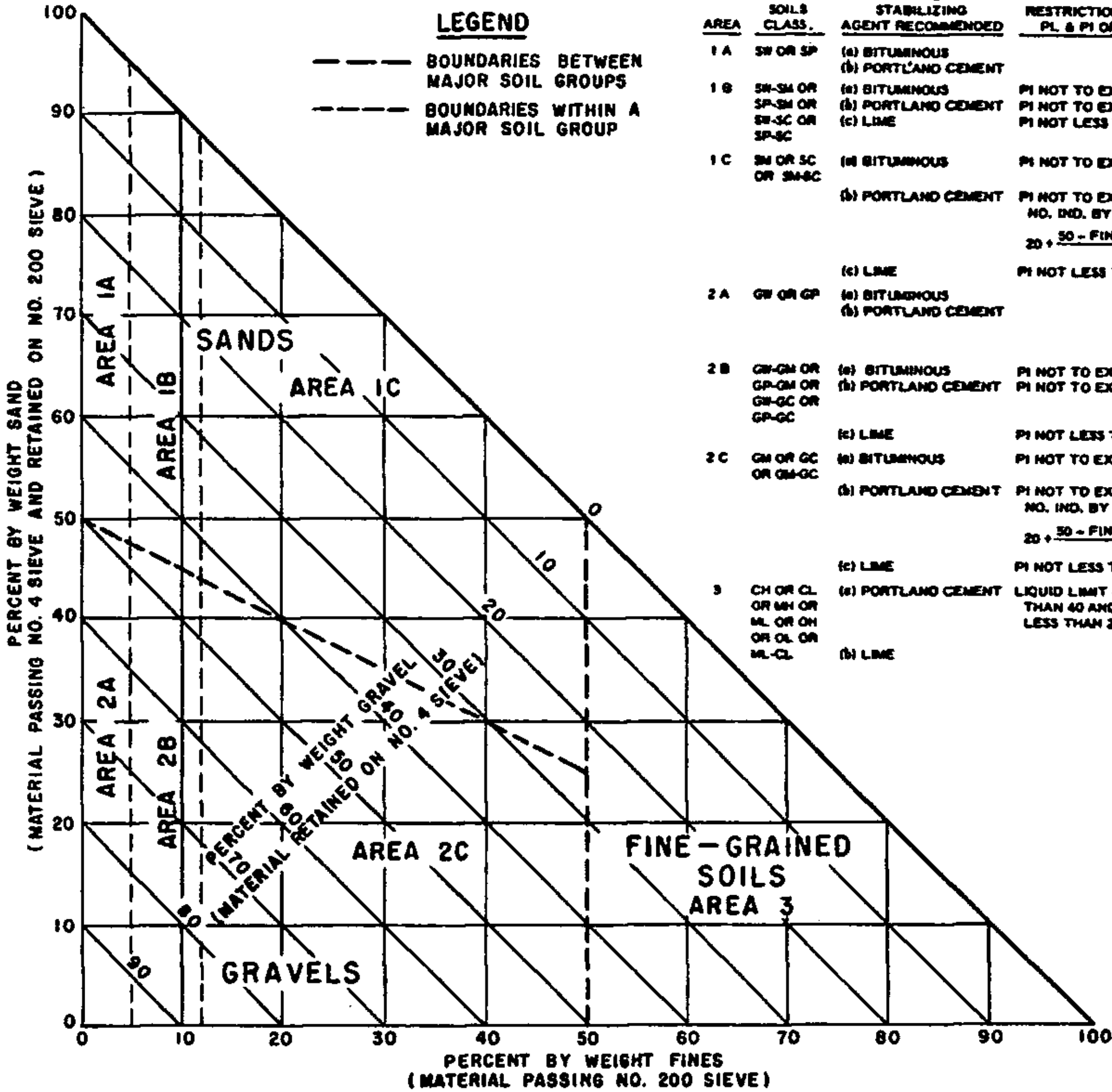
d. The frost design procedures discussed herein can be found in FAA Research Report FAA-RD-74-30, Design of Civil Airfield Pavement for Seasonal Frost and Permafrost Conditions. See Appendix 4. Another valuable reference for frost and permafrost design is U. S. Army Corps of Engineers Technical Manual TM 5-818-2, Pavement Design for Frost Conditions. See Appendix 4.

13. <u>SUBGRADE STABILIZATION</u>. Subgrade stabilization should be considered if one or more of the following conditions exist: poor drainage, adverse surface drainage, frost, or need for a stable working platform. Subgrade stabilization will also improve soil workability in certain instances. Different soil types require different stabilizing agents for best results. Figure 2-4 shows a method for the selection of stabilizer based on the properties of the soil to be stabilized. The selection is based primarily on gradation with restrictions given in terms of the Atterburg limits.

a. An example of the use of Figure 2-4 follows: Assume a soil has a gradation with 15% retained on the No. 4 sieve, 60% passing the No. 200 sieve and 25% passing the No. 4 and retained on the No. 200. The liquid limit of the soil is 60 and the plasticity index is 30.

(1) Enter the abscissa in Figure 2-4 with the percent of material passing the No. 200 sieve, 60%.

(2) Project vertically until the percentage of material passing the No. 4 sieve and retained on the No. 200 sieve agrees with the left ordinate. In this example the soil falls in Area 3.



AREA	SOILS CLASS.	TYPE STABILIZING AGENT RECOMMENDED	RESTRICTION ON LL, PL & PI OF SOIL	RESTRICTION ON (-200 MATL.)	REMARKS
1 A	SW OR SP	(a) BITUMINOUS (b) PORTLAND CEMENT			
1 B	SW-SM OR SP-SM OR SW-SC OR SP-SC	(a) BITUMINOUS (b) PORTLAND CEMENT (c) LIME	PI NOT TO EXCEED 10 PI NOT TO EXCEED 30 PI NOT LESS THAN 12		
1 C	SM OR SC OR SM-SC	(a) BITUMINOUS	PI NOT TO EXCEED 10	NOT TO EXCEED 30% BY WT	
		(b) PORTLAND CEMENT	PI NOT TO EXCEED NO. IND. BY EQ: $20 + \frac{50 - \text{FINES CONT.}}{4}$		
		(c) LIME	PI NOT LESS THAN 12		
2 A	GW OR GP	(a) BITUMINOUS (b) PORTLAND CEMENT			WELL-GRADED MATL ONLY MATL SHOULD CONTAIN AT LEAST 45% BY WT OF MATL PASSING NO. 4 SIEVE
2 B	GW-GM OR GP-GM OR GW-GC OR GP-GC	(a) BITUMINOUS (b) PORTLAND CEMENT	PI NOT TO EXCEED 10 PI NOT TO EXCEED 30		WELL-GRADED MATL ONLY MATL SHOULD CONTAIN AT LEAST 45% BY WT OF MATL PASSING NO. 4 SIEVE
		(c) LIME	PI NOT LESS THAN 12		
2 C	GM OR GC OR GM-GC	(a) BITUMINOUS	PI NOT TO EXCEED 10	NOT TO EXCEED 30% BY WT	WELL-GRADED MATL ONLY
		(b) PORTLAND CEMENT	PI NOT TO EXCEED NO. IND. BY EQ: $20 + \frac{50 - \text{FINES CONT.}}{4}$		MATL SHOULD CONTAIN AT LEAST 45% BY WT OF MATL PASSING NO. 4 SIEVE
		(c) LIME	PI NOT LESS THAN 12		
3	CH OR CL OR MH OR ML OR OH OR OL OR ML-CL	(a) PORTLAND CEMENT	LIQUID LIMIT LESS THAN 40 AND PI LESS THAN 20		ORGANIC AND STRONGLY ACID SOILS FALLING WITHIN THIS AREA ARE NOT SUSCEPTIBLE OF STABILIZATION BY ORDINARY MEANS
		(b) LIME			

FIGURE 2-4. GRADATION TRIANGLE FOR AID IN SELECTING A SOIL STABILIZER

(3) In the notes, Area 3 soils can be stabilized with either portland cement or lime. Stabilization by portland cement is restricted to materials with a liquid limit less than 40 and a plasticity index less than 20. (See restrictions on Atterburg limits)

(4) In this example lime would be the recommended stabilizer for use with this soil.

b. Figure 2-4 should be considered as a guide. The information cannot be expected to cover all possible situations. Some soils can be effectively stabilized by using a combination of agents. Local experience should receive a great deal of consideration in the selection of a stabilization method.

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

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

(1) Laboratory CBR tests are conducted on materials which have been obtained from the site and remolded to the density which will be obtained during construction. Specimens are soaked for 4 days to allow the material to reach saturation. A saturated CBR test is used to simulate the conditions likely

to occur in a pavement which has been in service for some time. Pavement foundations tend to reach nearly complete saturation after about 3 years. Seasonal moisture changes also dictate the use of a saturated CBR design value since traffic must be supported during periods of high moisture such as spring seasons.

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

(3) CBR tests on gravelly materials are difficult to interpret. Laboratory CBR tests on gravel often yield CBR results which are too high due to the confining effects of the mold. The assignment of CBR values to gravelly subgrade materials may be based on judgment and experience. The information given in Table 2-3 may provide helpful guidance in selecting a design CBR value for a gravelly soil. Table 2-3 should not, however, be used indiscriminately as a sole source of data. It is recommended that the maximum CBR for unstabilized gravel subgrade be 50.

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

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

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

(2) Plate bearing tests are relatively expensive to perform and thus the number of tests which can be conducted to establish a design value is limited. Generally only 2 or 3 tests can be performed for each pavement feature. The design k value should be conservatively selected.

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

(4) It is recommended that plate bearing tests be conducted on the subgrade and the results adjusted to account for the effect of subbase. Figure 2-5 shows the increase in k value for various thicknesses of subbase over a given subgrade k. Plate bearing tests conducted on top of subbase courses can sometimes yield erroneous results since the depth of influence beneath a 30-inch (762 mm) bearing plate is not as great as the depth of influence beneath a slab loaded with an aircraft landing gear assembly. In this instance a subbase layer can influence the response of a bearing plate more than the response of a loaded pavement.

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

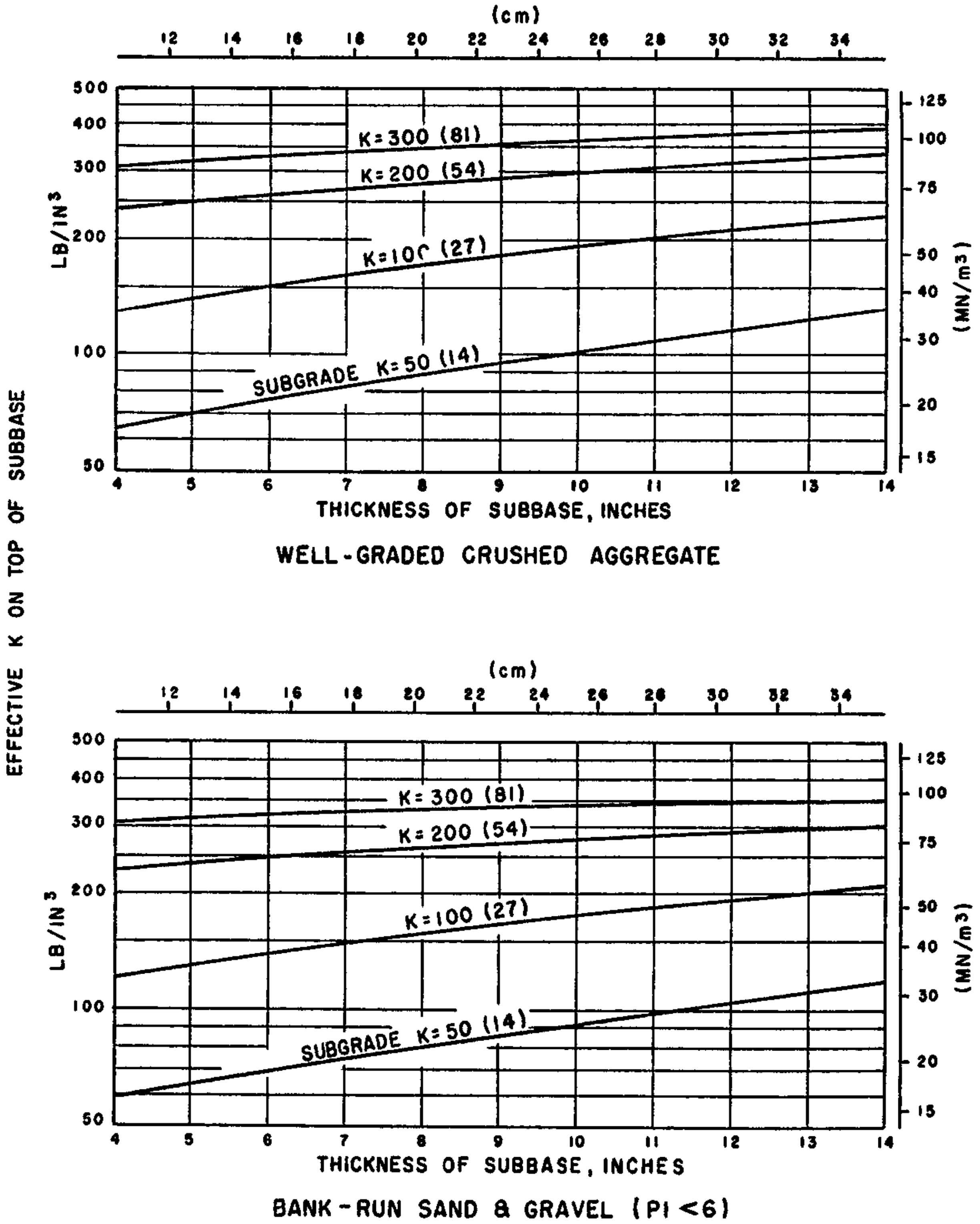


FIGURE 2-5. EFFECT OF SUBBASE ON MODULUS OF SUBGRADE REACTION

(6) It is recommended that a design k value of 500 lbs/in^3 (136 MN/m^3) not be exceeded for any foundation. The information presented in Table 2-3 gives general guidance as to probable k values for various soil types.

CHAPTER 3. PAVEMENT DESIGN.

SECTION 1. DESIGN CONSIDERATIONS

15. SCOPE. This chapter covers pavement design for airports serving aircraft with gross weights of 30,000 pounds (13 000 kg) or more. Chapter 5 is devoted to the design of pavements serving lighter aircraft with gross weights under 30,000 pounds (13 000 kg).

16. DESIGN PHILOSOPHY. The FAA policy of treating the design of aircraft landing gear and the design and evaluation of airport pavements as three separate entities is described in the Foreword to this advisory circular. The design of airport pavements is a complex engineering problem which involves a large number of interacting variables. The design curves presented in this chapter are based on the CBR method of design for flexible pavements and a jointed edge stress analysis for rigid pavements. These procedures represent a change from prior FAA design methods and will result in slightly different pavement thicknesses. The design curves in this chapter will satisfy the standards required by Section 16(a) of the Airport and Airway Development Act of 1970, as amended. Other design procedures such as those based on layered elastic analysis and those developed by The Asphalt Institute and the Portland Cement Association may be utilized to determine pavement thicknesses when approved by the FAA. These procedures will yield slightly different design thicknesses due to different basic assumptions. All pavement designs should be summarized on FAA Form 5100-1, Airport Pavement Design, (see AC 150/5100-3A) which is considered to be part of the Engineer's Report. Because of thickness variations, the evaluation of existing pavements should be performed using the same method as was employed in the design. Procedures to be used in evaluating pavements are described in detail in Chapter 6 of this advisory circular. Details on how the new FAA methods of design were developed are as follows:

   a. Flexible Pavements. The flexible pavement design curves presented in this chapter are based on the California Bearing Ratio (CBR) method of design. The CBR design method is basically empirical; however, a great deal of research has been done with the method and reliable correlations have been developed. Gear configurations are related using theoretical concepts as well as empirically developed data. The design curves provide the required total thickness of flexible pavement (surface, base, and subbase) needed to support a given weight of aircraft over a particular subgrade. The curves also show the required surface thickness. Minimum base course thicknesses are shown on a separate curve. A more detailed discussion of CBR design is presented in Appendix 2.

b. Rigid Pavements. The rigid pavement design curves in this chapter are based on the Westergaard analysis of edge loading. The edge loading analysis has been modified to simulate a jointed edge condition. Design curves are furnished for areas where traffic will predominantly follow parallel to the joints and for areas where traffic is likely to cross joints at some acute angle. Previous FAA rigid pavement criteria were based on an interior loading assumption. Pavement stresses are higher at the jointed edge than at the slab interior. Test validations and field performance show practically all load induced cracks develop at the jointed edge and migrate toward the slab interior. For these reasons the basis of design was changed from interior to jointed edge, as recommended by the U. S. Army Corps of Engineers, under a pavement research contract for the FAA. The design curves contain lines for five different annual traffic volumes. The thickness of pavement determined from the curves is for slab thickness only. Subbase thicknesses are determined separately. A more detailed discussion of the basis for rigid pavement design is presented in Appendix 2.

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

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

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

18. AIRCRAFT CONSIDERATIONS.

a. Load. The pavement design method is based on the gross weight of the aircraft. For design purposes the pavement should be designed for the maximum takeoff weight of the aircraft. The design procedure assumes 95 percent of the gross weight is carried by the main landing gears and 5 percent is carried by the nose gear. AC 150/5325-5, Aircraft Data, lists the weight of nearly all civil aircraft. The maximum takeoff weight should be used in calculating the pavement thickness required. Use of the maximum takeoff weight is recommended to provide some degree of conservatism in the design and is justified by the fact that changes in operational use can often occur and recognition of the fact that forecast traffic is approximate at best. By ignoring arriving traffic some of the conservatism is offset.

b. Landing Gear Type and Geometry.

(1) The gear type and configuration dictate how the aircraft weight is distributed to the pavement and determine pavement response to aircraft loadings. It would have been impractical to develop design curves for each type of aircraft. However, since the thickness of both rigid and flexible pavements is dependent upon the gear dimensions and the type of gear, separate design curves would be necessary unless some valid assumptions could be made to reduce the number of variables. Examination of gear configuration, tire contact areas, and tire pressure in common use indicated that these follow a definite trend related to aircraft gross weight. Reasonable assumptions could therefore be made and design curves constructed from the assumed data. These assumed data are as follows:

(a) Single Gear Aircraft. No special assumptions needed.

(b) Dual Gear Aircraft. A study of the spacing between dual wheels for these aircraft indicated that a dimension of 20 inches (0.51 m) between the centerline of the tires appeared reasonable for the lighter aircraft and a dimensi of 34 inches (0.86 m) between the centerline of the tires appeared reasonable for the heavier aircraft.

(c) Dual Tandem Gear Aircraft. The study indicated a dual wheel spacing of 20 inches (0.51 m) and a tandem spacing of 45 inches (1.14 m) for lighter aircraft, and a dual wheel spacing of 30 inches (0.76 m) and a tandem spacing of 55 inches (1.40 m) for the heavier aircraft are appropriate design values.

(d) Wide Body Aircraft. Wide body aircraft; i.e., B-747, DC-10, and L-1011 represent a radical departure from the geometry assumed for dual tandem aircraft described in paragraph (c) above. Due to the large differences in gross weights and gear geometries, separate design curves have been prepared for the wide body aircraft.

(2) Tire pressure varies between 75 and 200 psi (516 to 1 380 kPa) depending on gear configuration and gross weight. It should be noted that tire pressure asserts less influence on pavement stresses as gross weight increases, and the assumed maximum of 200 psi (1 380 kPa) may be safely exceeded if other parameters are not exceeded.

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

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

20. DETERMINATION OF EQUIVALENT ANNUAL DEPARTURES BY THE DESIGN AIRCRAFT.

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

To Convert From	To	Multiply Departures By
single wheel	dual wheel	0.8
single wheel	dual tandem	0.5
dual wheel	dual tandem	0.6
double dual tandem	dual tandem	1.0
dual tandem	single wheel	2.0
dual tandem	dual wheel	1.7
dual wheel	single wheel	1.3
double dual tandem	dual wheel	1.7

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

$$\log R_1 = \log R_2 \times \left(\frac{W2}{W1}\right)^{\frac{1}{2}}$$

where R_1 = equivalent annual departures by the design aircraft

R_2 = annual departures expressed in design aircraft landing gear

W_1 = wheel load of the design aircraft

W_2 = wheel load of the aircraft in question

For this computation 95 percent of the gross weight of the aircraft is assumed to be carried by the main landing gears. Wide body aircraft require special attention in this calculation. The procedure discussed above is a relative rating which compares different aircraft to a common design aircraft. Since wide body aircraft have radically different landing gear assemblies than other aircraft, special considerations are needed to maintain the relative effects. This is done by treating each wide body as a 300,000-pound (136 100 kg) dual tandem aircraft when computing equivalent annual departures. This should be done in every instance even when the design aircraft is a wide body. After the equivalent annual departures are determined, the design should proceed using the appropriate design curve for the design aircraft. For example if a wide body is the design aircraft, all equivalent departures should be calculated as described above; then the design curve for the wide body should be used with the calculated equivalent annual departures.

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

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

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

(2) Group Forecast Traffic into Landing Gear of Design Aircraft. In this example the design aircraft is equipped with a dual wheel landing gear so all traffic must be grouped into the dual wheel configuration.

(3) Convert Aircraft to Equivalent Annual Departures of the Design Aircraft. After the aircraft mixture has been grouped into a common landing gear configuration, the equivalent annual departures of the design aircraft can be calculated.

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

1/ Wheel loads for wide body aircraft will be taken as the wheel load for a 300,000-pound (136 100 kg) aircraft for equivalent annual departure calculations.

(4) For this example the pavement would be designed for 16,000 annual departures of a dual wheel aircraft weighing 190,500 pounds (86 500 kg). The design should, however, provide for the heaviest aircraft in the traffic mixture when considering depth of compaction, thickness of asphalt surface, drainage structures, etc.

21. TRAFFIC DISTRIBUTION.

a. Research studies have shown that aircraft traffic is distributed laterally across runways and taxiways according to statistically normal (bell shaped)distribution. FAA Report No. FAA-RD-36, Field Survey and Analysis of Aircraft Distribution on Airport Pavements, dated February 1975, contains the latest research information on traffic distribution. The design procedures presented in this circular incorporate the statistically normal distribution in the departure levels.

b. In addition to the lateral distribution of traffic across pavements, traffic distribution and nature of loadings are considered at runway ends, aprons, and high speed turnoffs.

22. TYPICAL SECTIONS. Typical plan and cross section drawings for runway pavements are shown in Figure 3-1. These typical sections are intended for runways to serve jet powered aircraft. Deviations from these typical sections will be common due to the change inherent in staged construction projects where runways are extended and the location of taxiways is uncertain. As a general rule-of-thumb the designer should specify full pavement thickness T where departing traffic will be using the pavement; pavement thickness of 0.9T will be specified where traffic will be arrivals such as high speed turnoffs; and pavement thickness of 0.7T will be specified where pavement is required but traffic is unlikely such as along the extreme outer edges of the runway. Note that the full-strength keel section has been reduced to 50 feet (15 m) on the basis of the research study discussed in paragraph 21.

23. CLIMATIC CONSIDERATIONS.

a. General. The design of an airport pavement must consider the climatic conditions which will act on the pavement during its construction and life. Most climatic effects such as protection of the pavement during curing, laydown temperatures, etc., are handled by construction specifications and local construction experience.

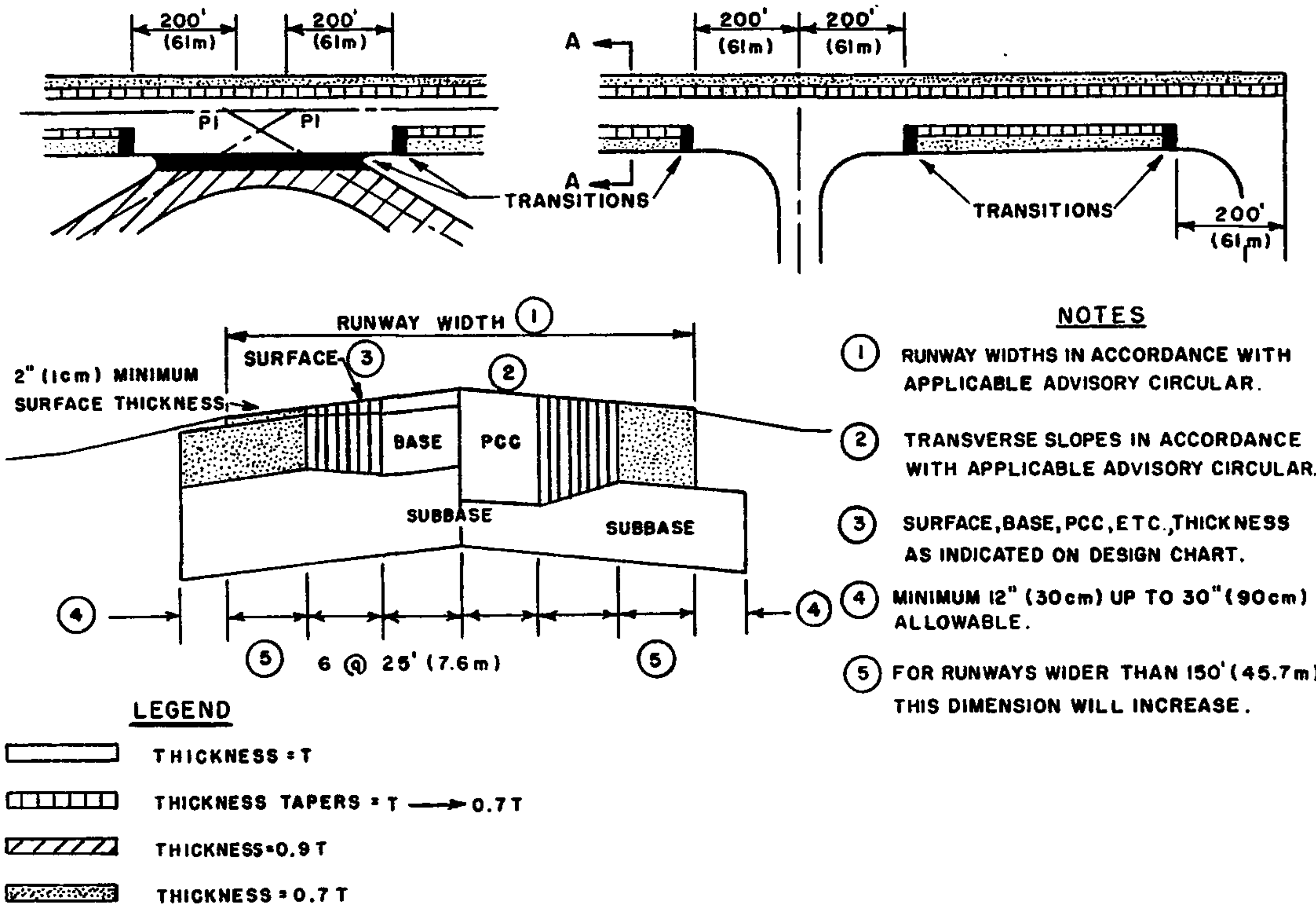


FIGURE 3-1. TYPICAL PLAN AND CROSS SECTION FOR RUNWAY PAVEMENTS

b. Frost Protection. The effects of frost have a direct bearing on the design of airport pavements and are treated separately from the structural design for aircraft loadings. In Chapter 2, paragraph 12, four different methods of designing for frost effects are presented. The degree of frost protection required is dictated by the soil conditions and by the usage that pavement will receive. The recommended frost design methods are presented in Table 3-1. The recommendations shown in Table 3-1 should be treated as approximations and should be used if more precise information based on local experience is not available for use in frost design.

c. Permafrost. Design considerations in permafrost areas should be predicated on local experience and practice.

TABLE 3-1. RECOMMENDED FROST DESIGN METHODS

Pavement Feature	Soil Conditions			
	Uniform	Slightly Variable	Variable	Extremely Variable
Large Hub				
Runway	LSP	LSP	LSP	CP
Taxiway	RSP	RSP	LSP	CP
Apron	RSS	RSP	LSP	LSP
Medium Hub				
Runway	LSP	LSP	LSP	LSP
Taxiway	RSP	RSP	LSP	LSP
Apron	RSS	RSP	RSP	RSP
Small Hub				
Runway	RSP	RSP	LSP	LSP
Taxiway	RSS	RSP	RSP	LSP
Apron	RSS	RSS	RSS	RSP

CP - Complete Protection

LSP - Limited Subgrade Protection

RSP - Reduced Subgrade Protection

RSS - Reduced Subgrade Strength

SECTION 2. FLEXIBLE PAVEMENT DESIGN.

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

25. BITUMINOUS SURFACING.

a. The bituminous surface or wearing course must prevent the penetration of surface water to the base course; provide a smooth, well-bonded surface free from loose particles which might endanger aircraft or persons; resist the shearing stresses occasioned by aircraft loads; and furnish a texture of nonskid qualities, yet not cause undue wear on tires. To successfully fulfill these requirements, the surface must be composed of mixtures of aggregates and bituminous binders which will produce a uniform surface of suitable texture possessing maximum stability and durability. Since control of the mixture is of paramount importance, these requirements can best be achieved by use of a central mixing plant where proper control can be most readily obtained. A dense-graded bituminous concrete such as Item P-401 produced in a central mixing plant will most satisfactorily meet all the above requirements.

b. Whenever a flexible pavement is to be subjected to concentrated fuel spillage or other solvents, as at aircraft fueling positions and maintenance areas, protection should be provided by a solvent resistant surface such as Item P-625, Tar Emulsion Protective Seal Coat. The military has had satisfactory service from rubberized tar hot mixes in areas of concentrated fuel spillage. Corps of Engineers Guide Specification CE-807.25 is the military specification for rubberized tar concrete.

26. BASE COURSE. The base course is the principal structural component of the flexible pavement. It has the major function of distributing the imposed wheel load pressures to the pavement foundation, the subbase and/or subgrade. The base course must be of such quality and thickness to prevent failure in the subgrade, withstand the stresses produced in the base itself, resist vertical pressures tending to produce consolidation and resulting in distortion of the surface course, and resist volume changes caused by fluctuations in its moisture content. In the development of pavement thickness requirements, a CBR value of 80 is assumed for the base course.

a. The quality of the base course depends upon composition, physical properties and compaction. Many materials and combinations thereof have proved satisfactory as base courses. They are composed of select, hard, and durable aggregates.

b. Specifications covering the quality of components, gradation, manipulation control, and preparation of various types of base courses for use on airports for design loads above 30,000 pounds (13 000 kg) aircraft gross weight are as follows:

(1) Item P-201 - Bituminous Base Course

(2) Item P-209 - Crushed Aggregate Base Course

(3) Item P-211 - Lime Rock Base Course

(4) Item P-214 - Penetration Macadam Base Course

(5) Item P-215 - Cold Laid Bituminous Base Course

(6) Item P-304 - Cement Treated Base Course

27. SUBBASE.

a. A subbase is included as an integral part of the flexible pavement structure in all pavements except those on subgrades classified as GW or GP. The function of the subbase is similar to that of the base course. However, since it is further removed from the surface and is subjected to lower loading intensities, the material requirements are not as strict as for the base course. Specification Item P-154, Subbase Course, covers the quality, gradation, control, and preparation of the standard subbase course. In the development of pavement thickness requirements the CBR value of the subbase course is a variable.

b. Certain materials that are permitted only for base courses for pavements serving aircraft with gross weights of less than 30,000 (13 000 kg) pounds may be used as subbase courses for the larger aircraft. They are:

(1) Item P-206 - Dry-Bound Macadam Base Course or Water-Bound Macadam Base Course

(2) Item P-208 - Aggregate Base Course

(3) Item P-210 - Caliche Base Course

(4) Item P-212 - Shell Base Course

(5) Item P-213 - Sand-Clay Base Course

(6) Item P-216 - Mixed In-Place Base Course

(7) Item P-301 - Soil Cement Base Course

c. When the material Items P-201 and P-304 are used as base courses, they may also be used as subbase.

28. SUBGRADE.

a. The subgrade soils are subjected to lower stresses than the surface, base, and subbase courses. Subgrade stresses decrease with depth, and the controlling subgrade stress is usually at the top of the subgrade unless unusual conditions exist. Unusual conditions such as a layered subgrade or sharply varying water contents or densities can change the location of the controlling stress. These conditions should be checked during the soils investigation. The ability of a particular soil to resist shear and deformation will vary with its density and moisture content.

(1) Specification Item P-152, Excavation and Embankment, covers the construction and density control of subgrade soils. Figure 3-2 shows depths below the subgrade surface to which compaction controls apply.

(2) Noncohesive soils, for the purpose of determining compaction control, are those with a plasticity index (P.I.) of less than 6.

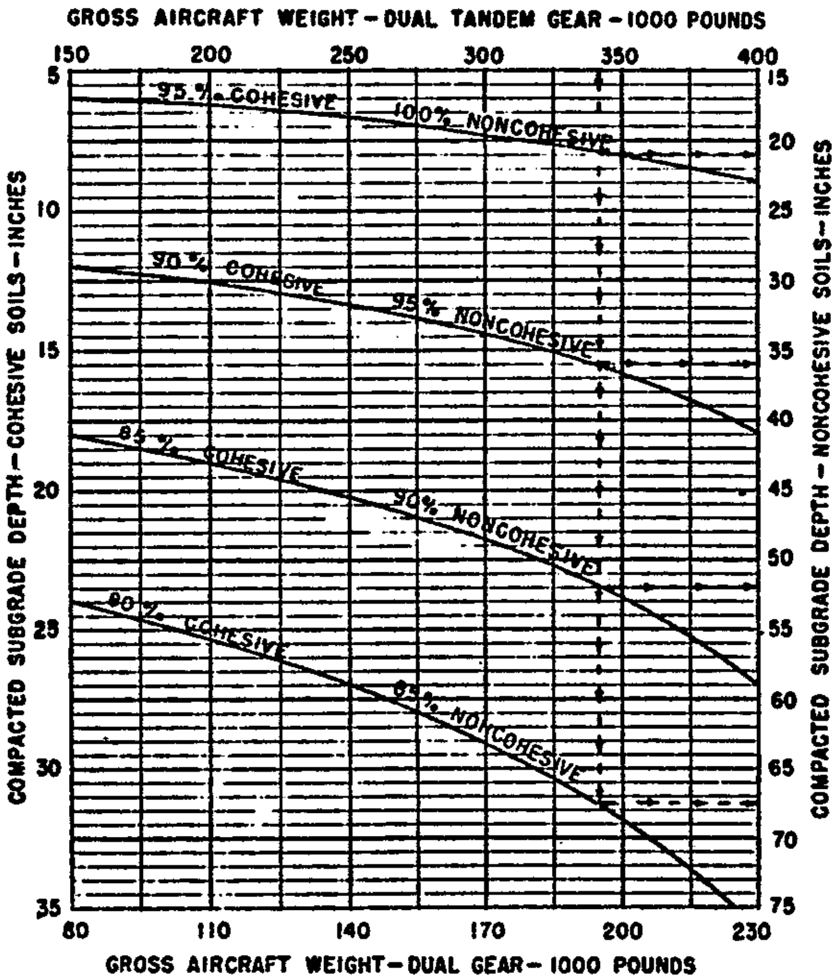
(3) Example: For an apron extension to accommodate a 340,000-pound (154 000 kg) dual tandem geared aircraft, a soils investigation has shown the subgrade will be noncohesive. In-place densities of the soils have been determined at even foot increments below the ground surface. Design calculations indicate that the top of subgrade in this area will be approximately 10 inches (0.3 m) below the existing grade. Depths and densities may be tabulated as follows:

Depth Below Existing Ground	Depth Below Finished Subgrade	In-place Density
1' (0.3 m)	2" (0.05 m)	70%
2' (0.6 m)	14" (0.35 m)	84%
3' (0.9 m)	26" (0.66 m)	86%
4' (1.2 m)	38" (0.97 m)	90%
5' (1.5 m)	50" (1.27 m)	93%

In Figure 3-2, project a line downward from 340,000 pounds (154 000 kg) on dual tandem scale; and from the point of intersection with the line representing each density requirement project a line to the noncohesive compacted subgrade depth scale.

Reference to the tabulation shows that for this example in-place density is satisfactory at a depth of 38 inches (0.97 m) being 90 percent and within the required 90 percent zone. It will be necessary to compact an additional 2 inches (0.05 m) at 90 percent, 15 inches (0.38 m) at 95 percent, and the top 21 inches (0.53 m) of subgrade at 100 percent density. With modern compaction equipment, these densities can usually be achieved from the surface in a noncohesive soil.

(4) For most soils, moisture-density curves show the water content at which the desired density can be most easily achieved. These soils when so compacted will provide a satisfactory level of in-place stability. Some soils, when compacted to optimum densities,will attract available water and swell. The swelling is accompanied by an extreme loss in bearing value. Soils of this type should be stabilized or modified where possible to the extent required to preclude the swelling. Where this is impractical, compaction with additional water to lower densities will minimize swelling, but will reduce bearing values. Methods of identifying and handling swelling soils are presented in FAA Report No. FAA-RD-76-66, Design and Construction of Airport Pavements on Expansive Soils, by R. Gordon McKeen dated June 1976. See Appendix 4.



NOTES:

1. Curved lines denote depths below the finished subgrade above which densities should equal or exceed the indicated percentage of the maximum density at optimum moisture as determined by the FAA compaction control T-611.

2. For embankment areas the charted criteria should be met except that the minimum density of soils placed in fill should be 90% for cohesive and 95% for noncohesive, and for the top nine inches in fill should be not less than 95% for cohesive and 100% for noncohesive, of the T-611 density.

3. The subgrade in cut areas shall have natural densities shown or should (a) be compacted from the surface to achieve the required densities, (b) should be removed and replaced in which case the minimum densities for fills apply, or (c) when economics and grades permit, be covered with sufficient select or subbase material so that the uncompacted subgrade is at a depth where the in-place densities are satisfactory.

4. For swelling soils, reduced densities may be used.

5. 1 inch = 2.54 cm
1 lb. = 0.454 kg

FIGURE 3-2. SUBGRADE COMPACTION REQUIREMENTS FOR FLEXIBLE PAVEMENTS

29. DESIGN REQUIREMENTS AS AFFECTED BY SUBGRADE PROFILE.

a. In some cases the upper level of the subgrade may exist as a clearly defined thin layer of soil of much better quality than the underlying soil. Obviously, to design on the basis of only the thin upper layer would be inadequate in many instances. However, it must be realized that the upper level of superior quality soil, even though thin in section, will provide some benefits which can be utilized in the pavement design. The following paragraphs illustrate the means by which these benefits may be realized. This procedure applies only when the layer thicknesses involved are 12 inches (30 cm) or greater. The procedure does not apply when the underlying soil is a swelling soil.

b. As an illustration, assume the upper level of soil is designated "A" and the underlying layer is designated "B." If the thickness of layer "A" is insufficient to reduce the stresses imposed on layer "B" to an acceptable level, then an increase in the thickness of subbase is necessary over that which would be required if the soil was composed entirely of layer "A" material. Conversely, the required subbase thickness would be less than the subbase thickness required to protect layer "B" because of some beneficial effect of layer "A."

c. The thickness of subbase required to fulfill design requirements for layer "B" should be determined by using the design curves with the CBR value for layer "B." The thickness of subbase required by using the CBR value of layer "A" should also be determined. Logically, the thickness of subbase should lie somewhere between the two thicknesses determined above. The required subbase thickness is determined by the following formula:

$$z = y - \frac{t(y-x)}{x+y}$$

where: z = required thickness of subbase

x = subbase thickness for layer "A" soil

y = subbase thickness for layer "B" soil

t = thickness of layer "A"

(1) It can be seen from the formula that "z" will be less than "x" if "t" is greater than "x" + "y." Therefore, if "t" is equal to or greater than the sum of "x" + "y," the subbase required for layer "A" should be used.

(2) For an example of the application of this formula, assume a subgrade condition is encountered where a 12-inch (30 cm) thick layer of high bearing capacity soil overlies a lower bearing soil. The higher bearing strength soil is designated as "A" and the lower strength is designated as "B." Assume the soil strengths are such that a pavement constructed on soil "A" would require a 3-inch (8 cm) thickness of subbase and a pavement constructed on soil "B" would require an 11-inch (28 cm) subbase.

$$Z = 11 - \frac{12\ (11-3)}{11+3}$$

$$Z = 4 \text{ inches } (10 \text{ cm})$$

(3) If "t" had been greater than 3 + 11 = 14 inches (36 cm), the subbase requirements would have been that as required for layer "A"; i.e., 3 inches (8 cm).

d. This illustrates the manner in which economic advantage may be gained by use of selective grading. If superior material is available on the site, it may be economical to remove inferior material, or a portion thereof, and replace it with the superior material thereby reducing the subbase thickness requirements.

30. INITIAL THICKNESS SELECTION. Due to the variations in stress distribution of single, dual, dual tandem, and wide body landing gear configurations as previously discussed, separate flexible pavement design curves for each of these gear configurations have been prepared and are presented in Figures 3-3 through 3-12, inclusive. The thicknesses determined from these design charts are for untreated granular materials and have to be adjusted for frost effects and stabilized materials as appropriate.

31. DESIGN INPUTS. Use of the design curves for flexible pavements requires a CBR value for the subgrade material, a CBR value for the subbase material, the gross weight of the design aircraft, and the number of annual departures of the design aircraft. The design curves presented in Figures 3-3 through 3-11 indicate the total pavement thickness required and the thickness of bituminous surfacing. Figure 3-12 indicates the minimum thickness of base course for given total pavement thicknesses and CBR values. For annual departures in excess of 25,000 the total pavement thickness should be increased in accordance with Table 3-5 and the bituminous surfacing increased by 1-inch (3 cm).

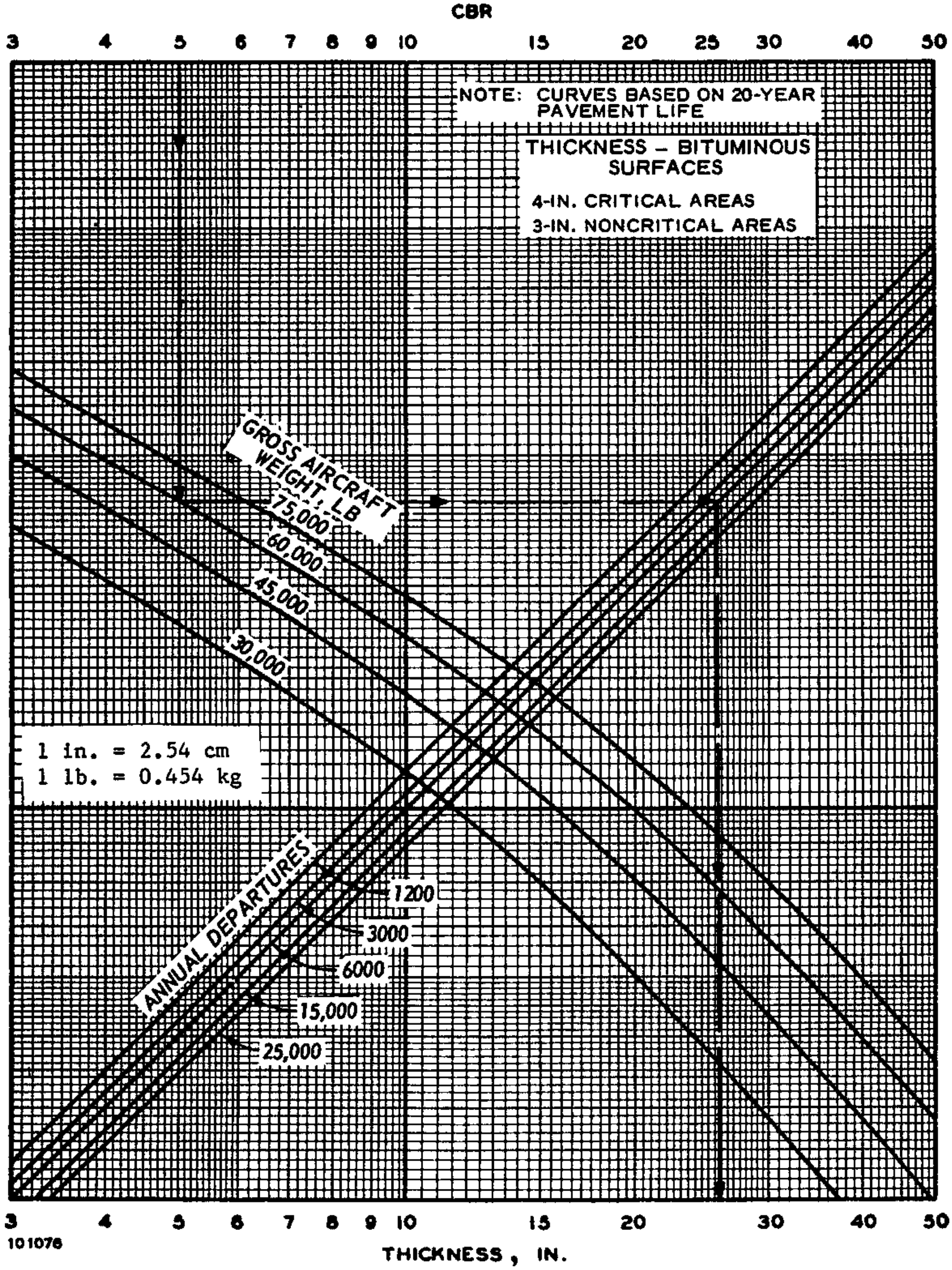


FIGURE 3-3. FLEXIBLE PAVEMENT DESIGN CURVES FOR CRITICAL AREAS, SINGLE WHEEL GEAR

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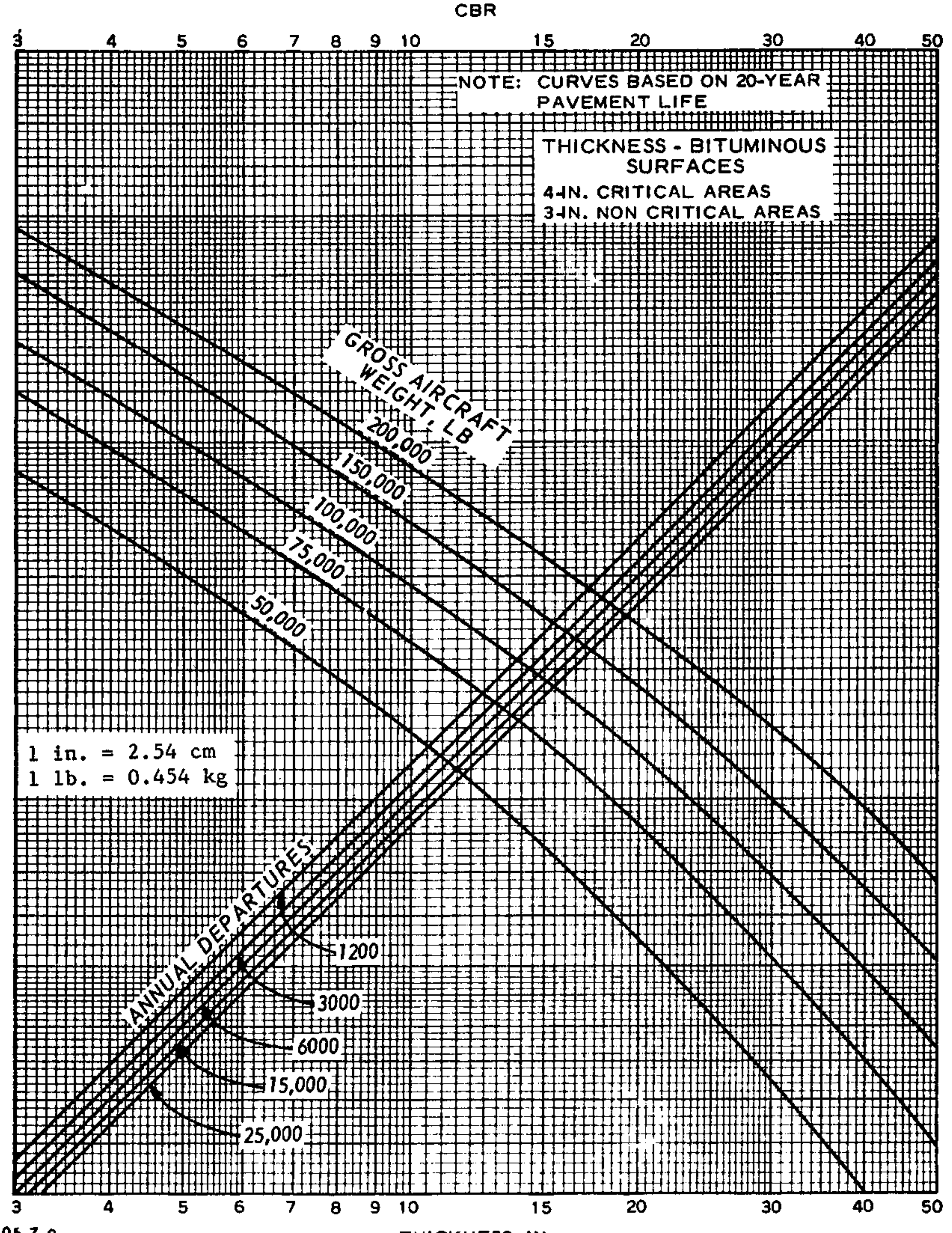

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FIGURE 3-4. FLEXIBLE PAVEMENT DESIGN CURVES FOR CRITICAL AREAS, DUAL WHEEL GEAR

*

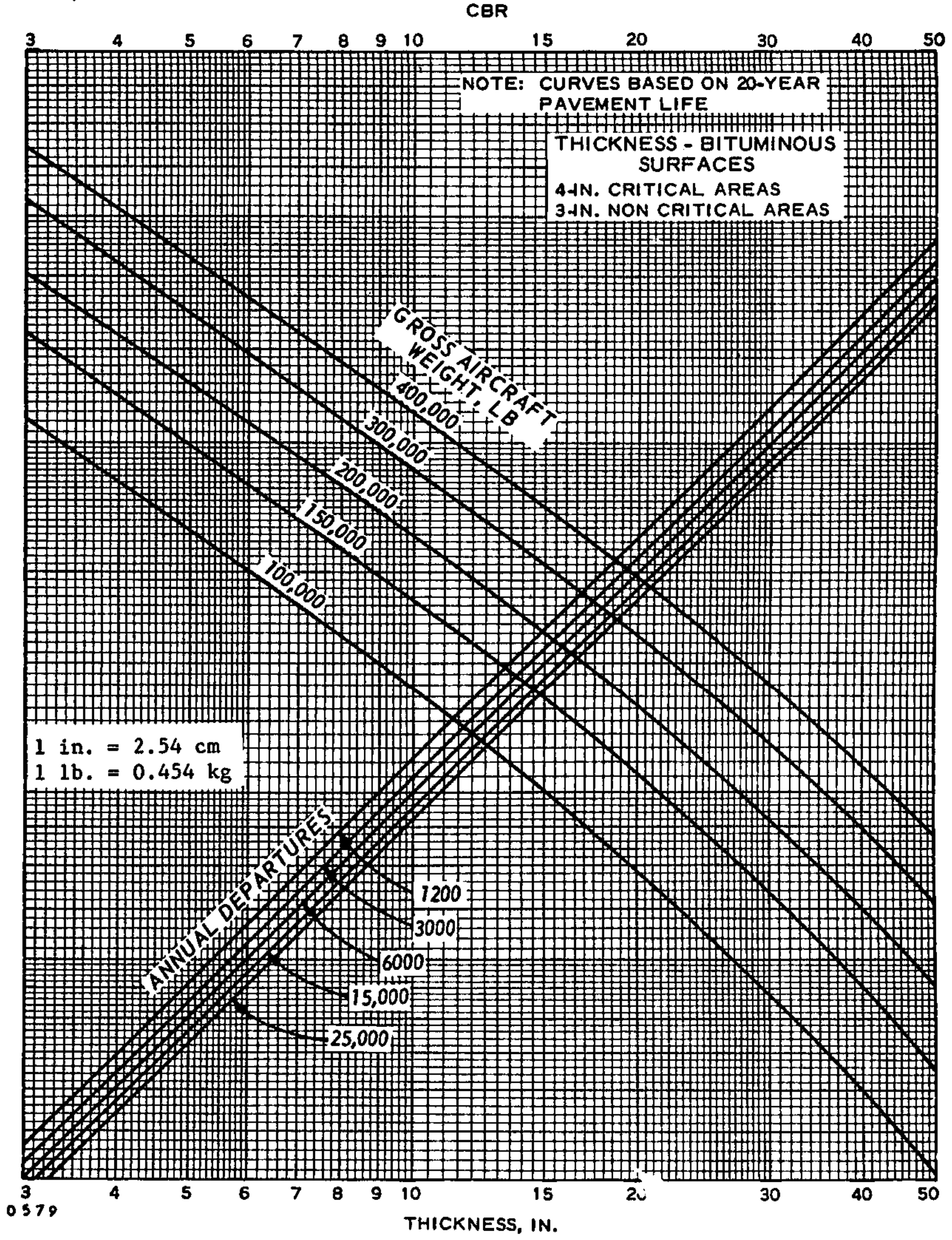


FIGURE 3-5. FLEXIBLE PAVEMENT DESIGN CURVES FOR CRITICAL AREAS, DUAL TANDEM GEAR

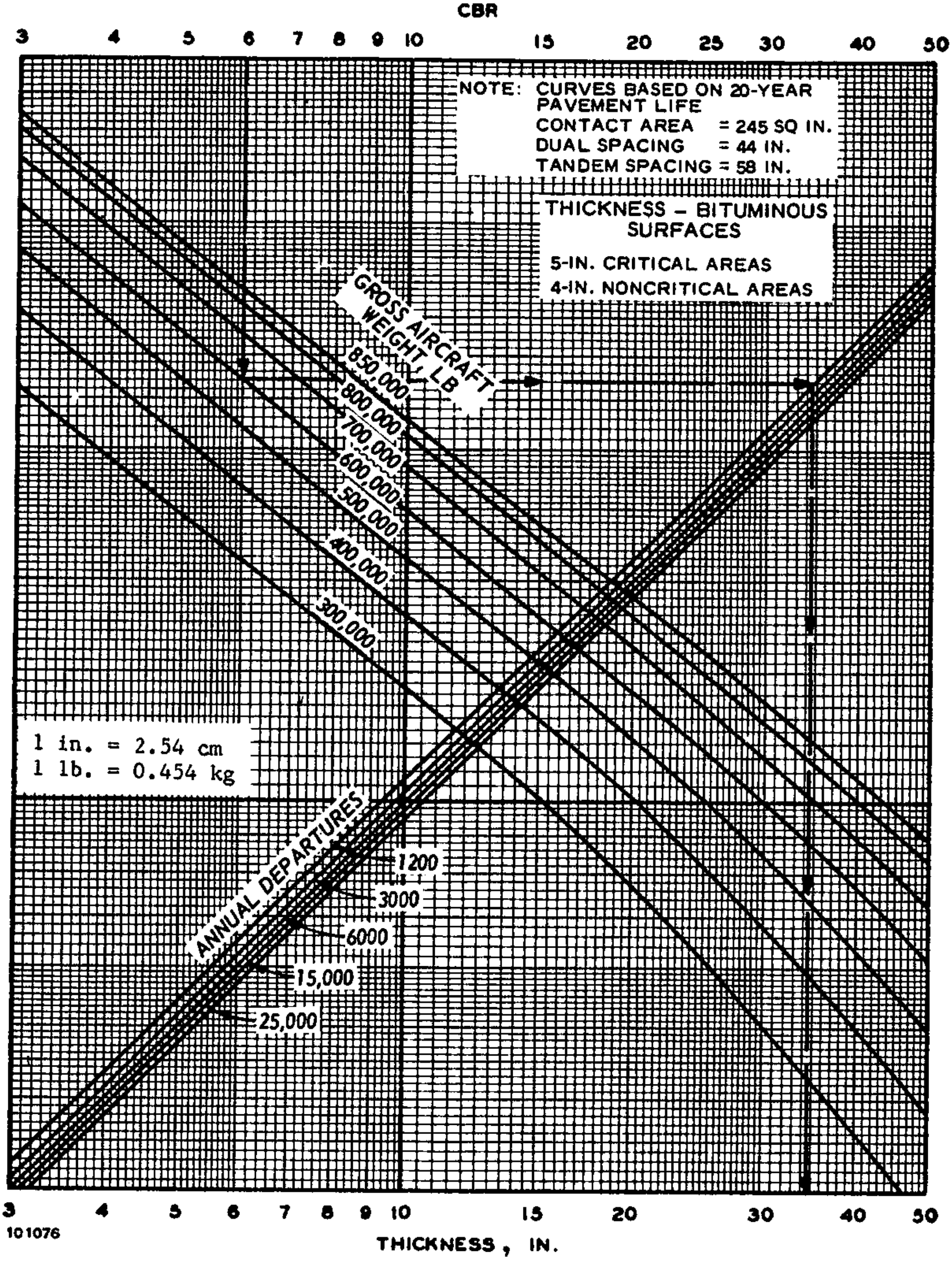


FIGURE 3-6. FLEXIBLE PAVEMENT DESIGN CURVES FOR CRITICAL AREAS, B-747-100, SR, 200 B, C, F

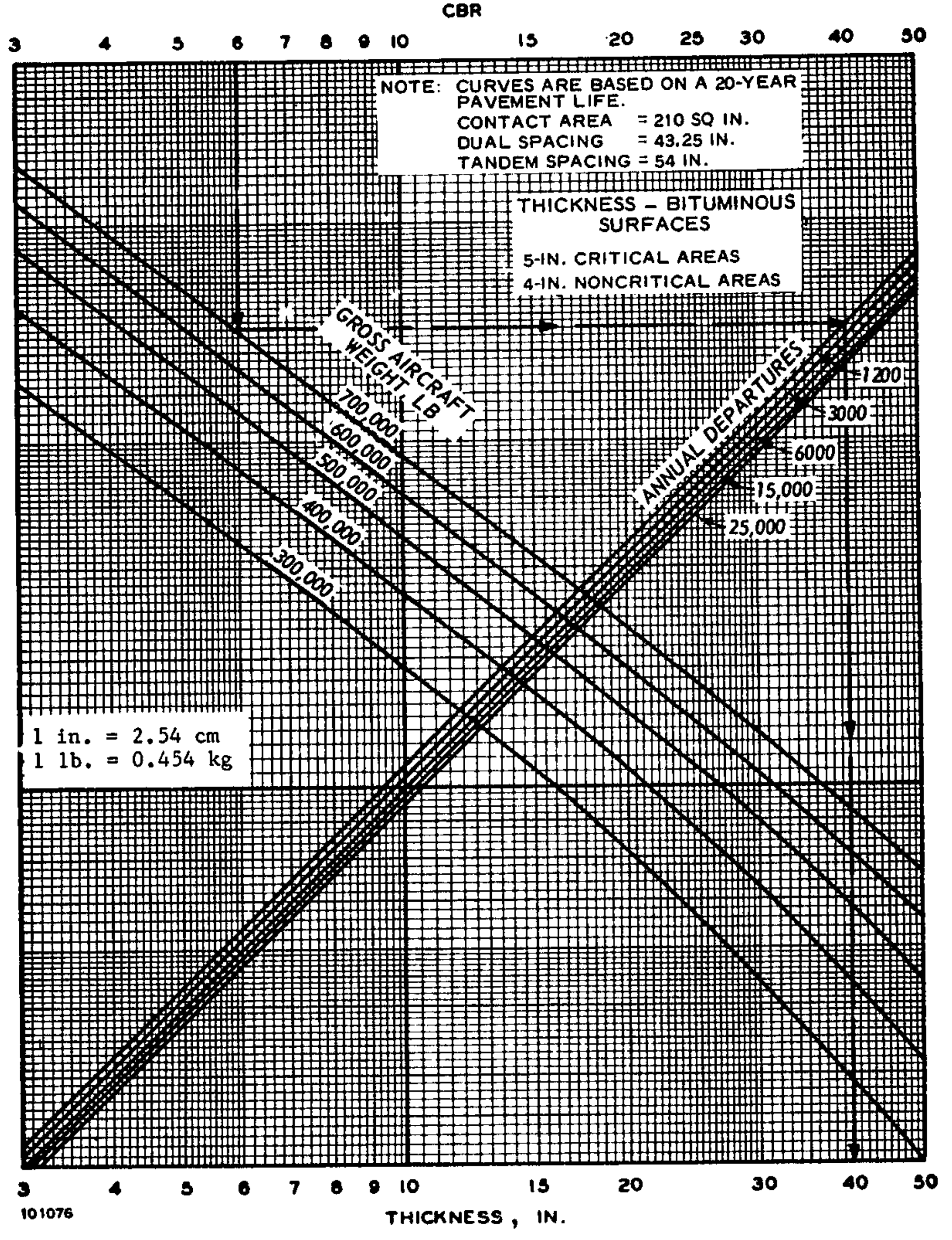


FIGURE 3-7. FLEXIBLE PAVEMENT DESIGN CURVES FOR CRITICAL AREAS, B-747-SP

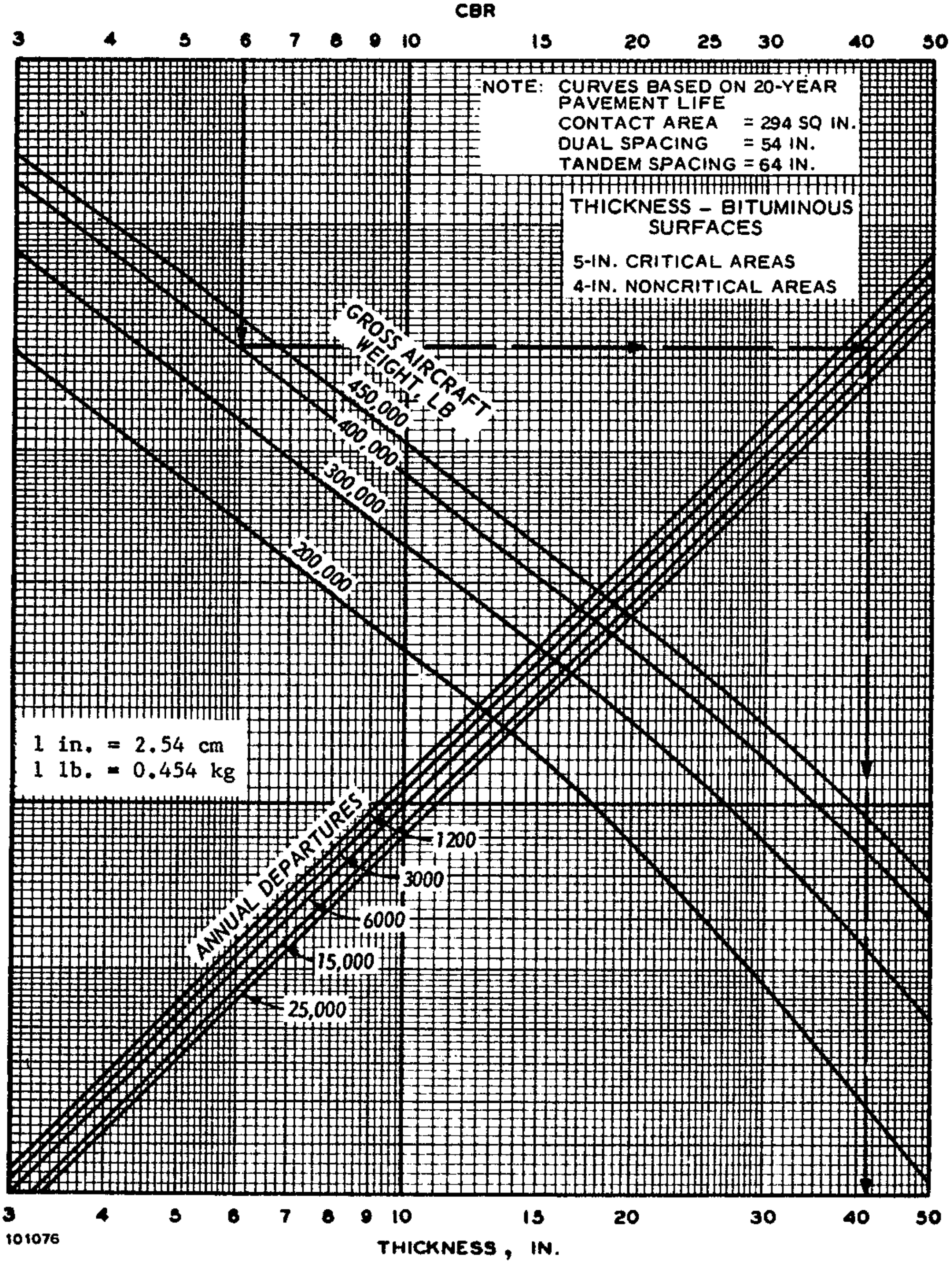


FIGURE 3-8. FLEXIBLE PAVEMENT DESIGN CURVES FOR CRITICAL AREAS, DC 10-10, 10CF

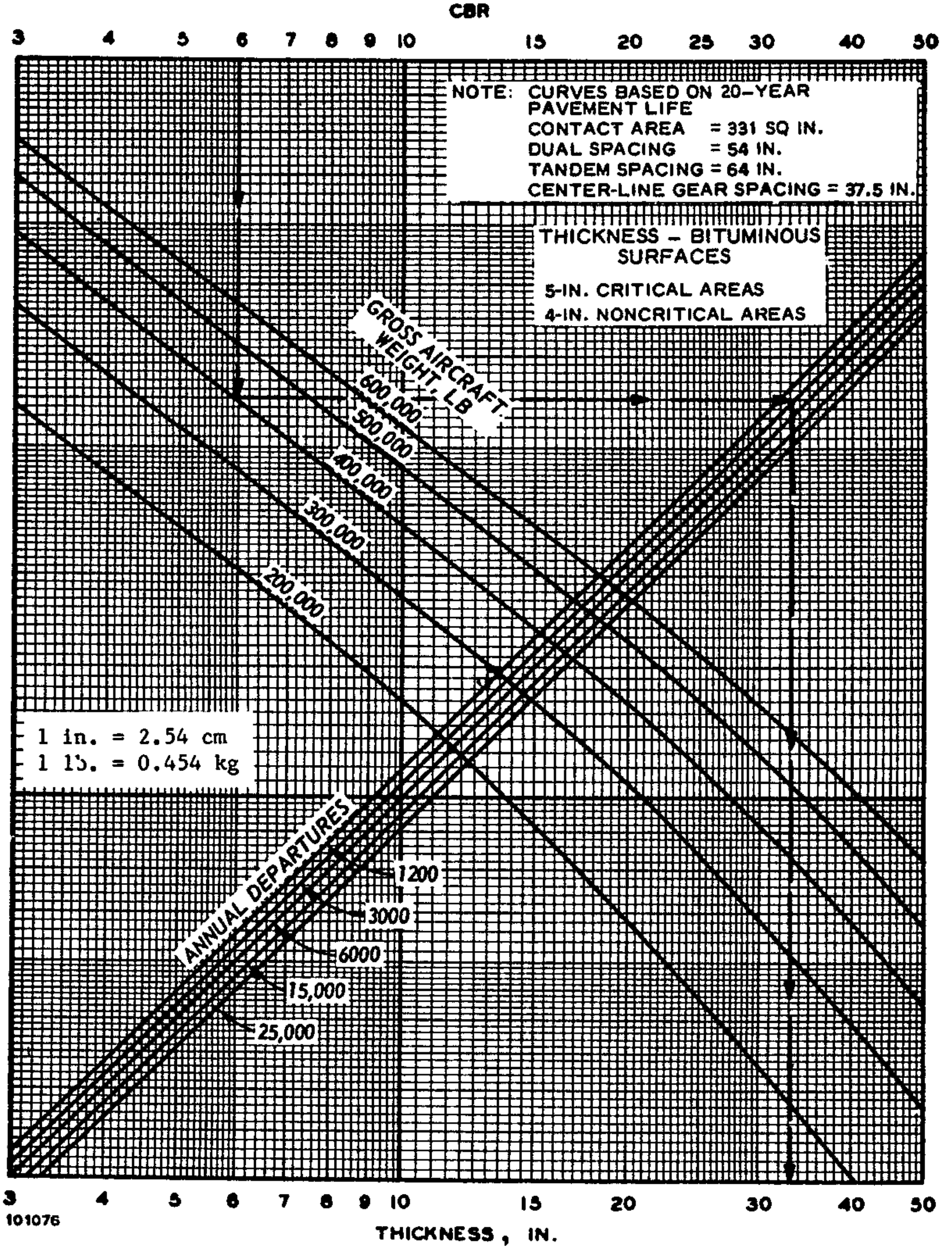


FIGURE 3-9. FLEXIBLE PAVEMENT DESIGN CURVES FOR CRITICAL AREAS, DC 10-30, 30CF, 40, 40CF

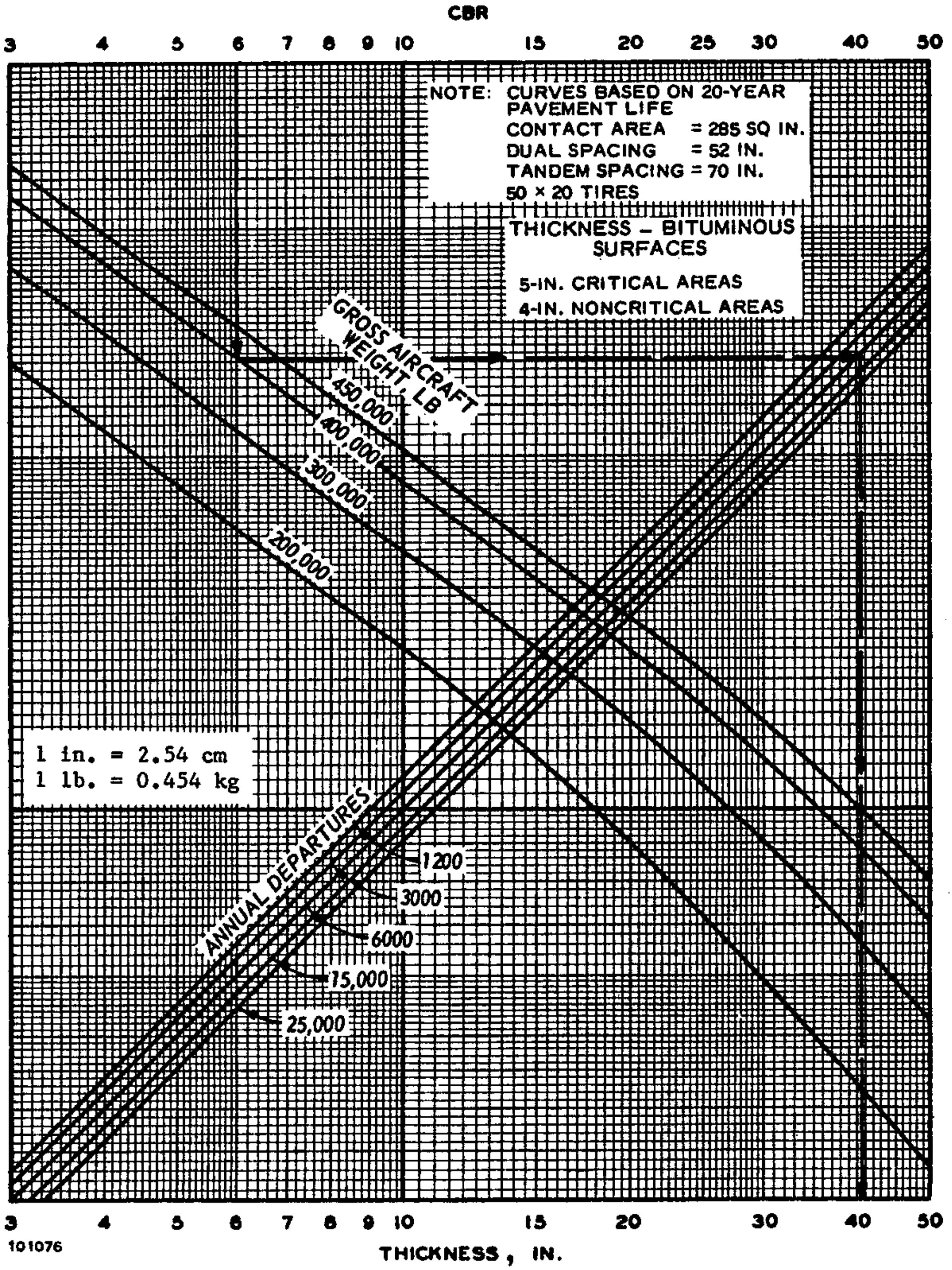


FIGURE 3-10. FLEXIBLE PAVEMENT DESIGN CURVES FOR CRITICAL AREAS, L-1011-1, 100

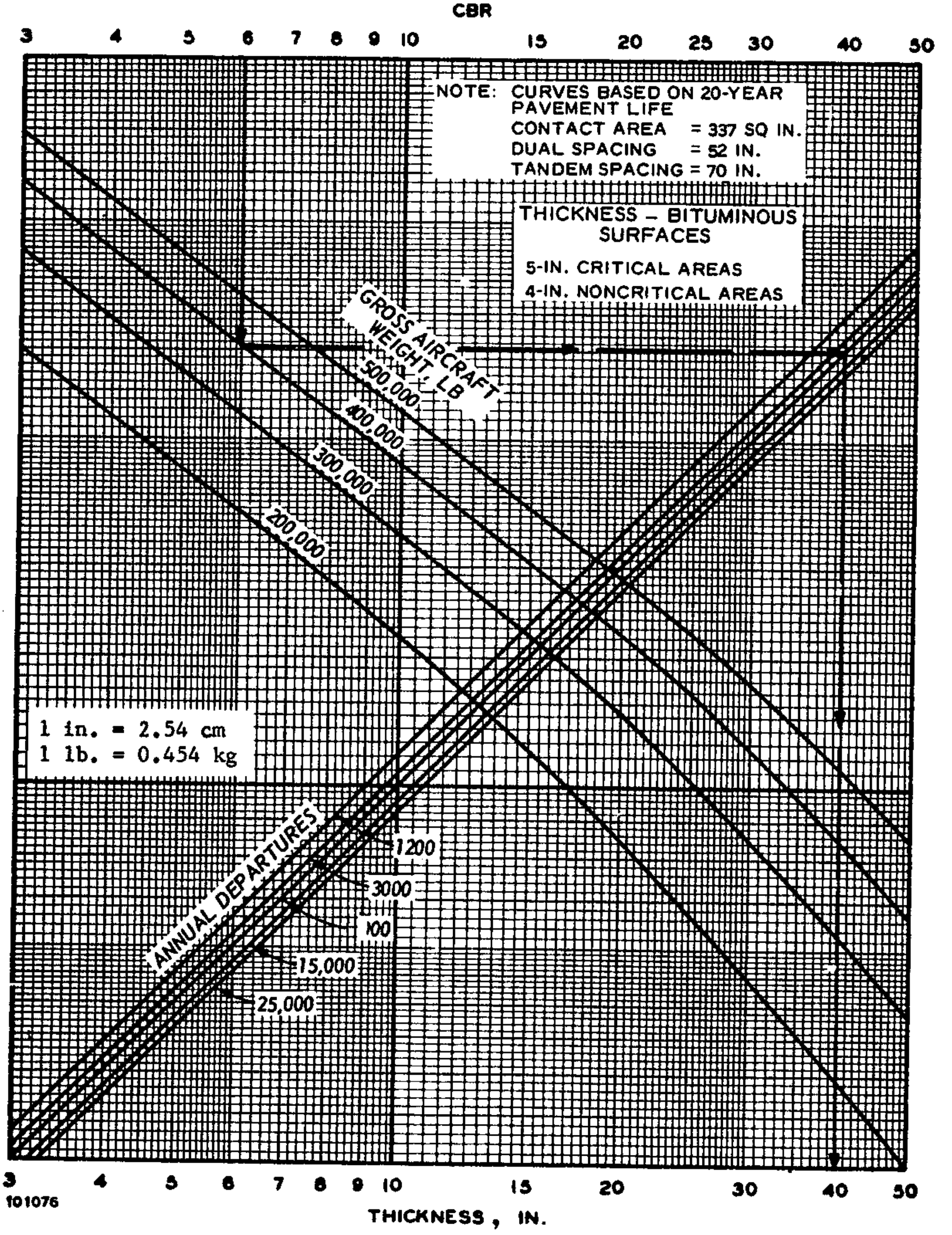


FIGURE 3-11. FLEXIBLE PAVEMENT DESIGN CURVES FOR CRITICAL AREAS, L-1011-100, 200

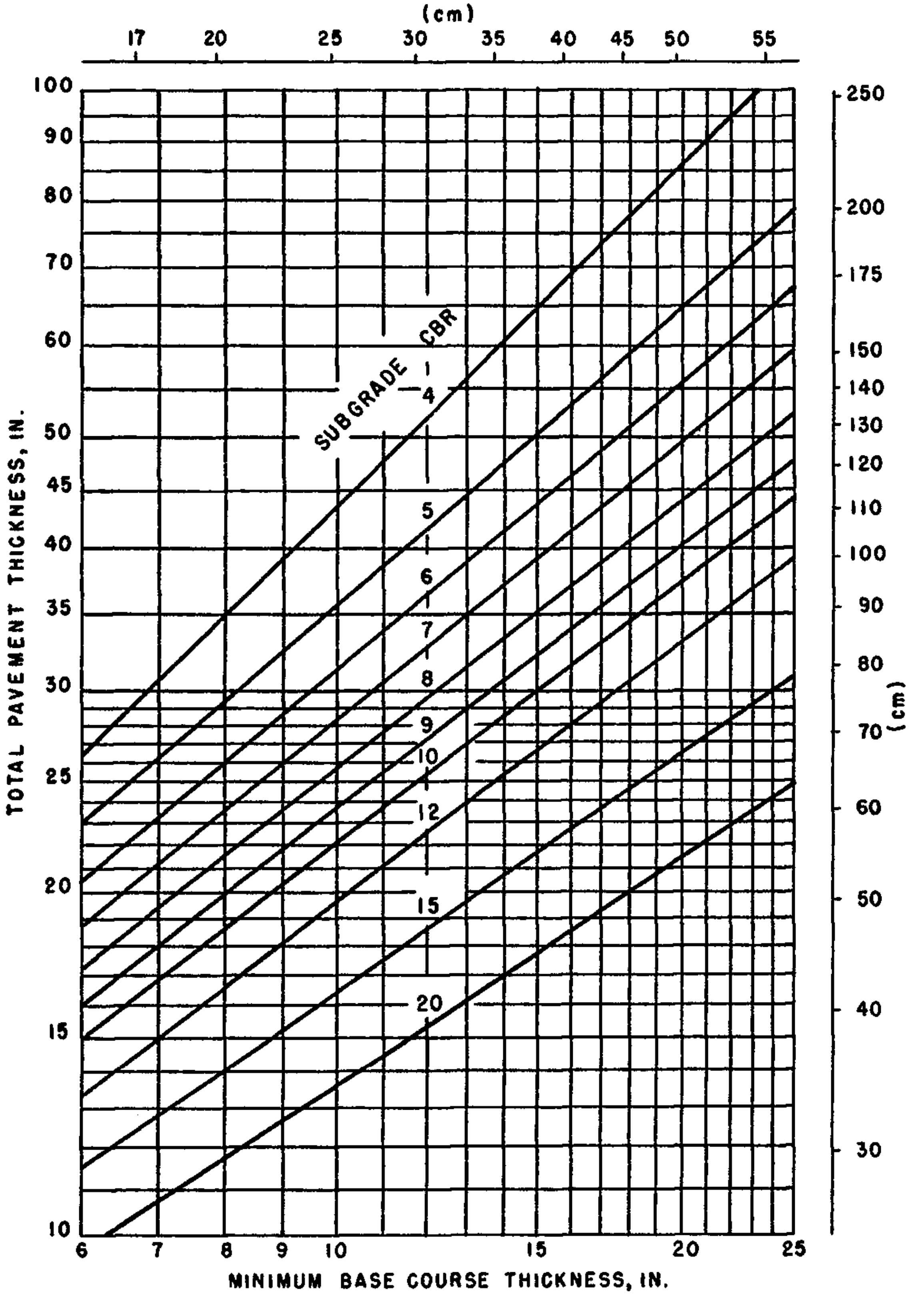


FIGURE 3-12. MINIMUM BASE COURSE THICKNESS REQUIREMENTS

32. CRITICAL AND NONCRITICAL AREAS. The design curves, Figures 3-3 through 3-11, are used to determine the total critical pavement thickness, "T," and the surface course thickness requirements. The 0.9T factor for the noncritical pavement applies to the base and subbase courses; the surface course thickness is as noted on the design curves. For the variable section of the transition section and thinned edge, the reduction applies only to the base course. The 0.7T thickness for base shall be minimum permitted, and the subbase thickness shall be increased or varied to provide positive surface drainage from the entire subgrade surface. Surface course thicknesses are as shown in Figure 3-1. For fractions of an inch of 0.5 or more, use the next higher whole number; for less than 0.5, use the next lower number.

33. STABILIZED BASE AND SUBBASE.

a. Stabilized base and subbase courses are necessary for new pavements designed to accommodate jet aircraft weighing 100,000 pounds (45 350 kg) or more. These stabilized courses may be substituted for granular courses using the equivalency factors discussed in paragraph 36. These equivalency factors are based on research studies which measured pavement performance. See FAA Report No. FAA-RD-73-198, Volumes I, II, and III, Comparative Performance of Structural Layers in Pavement Systems. See Appendix 4. A range of equivalency factors is given because the factor is sensitive to a number of variables such as layer thickness, stabilizing agent type and quantity, location of stabilized layer in the pavement structure, etc.

b. Exceptions to the policy requiring stabilized base and subbase should be based on proven performance of a granular material such as lime rock in the State of Florida. Proven performance in this instance means a history of satisfactory airport pavements using the materials. This history of satisfactory performance should be under aircraft loadings and climatic conditions comparable to those anticipated.

c. Other exceptions may be made on the basis of superior materials being available, such as 100 percent crushed, hard, closely graded stone. These materials should exhibit a remolded soaked CBR minimum of 100 for base and 35 for subbase. In areas subject to frost penetration the materials should meet permeability and nonfrost susceptibility tests in addition to the CBR requirements.

d. The minimum total pavement thickness should not be less than the total pavement thickness required by a 20 CBR subgrade on the appropriate design curve, nor shall P-201 and P-304 be less than 4 inches (10 cm) and 6 inches (15 cm) in thickness, respectively. Reflection cracking is sometimes encountered when P-304 is used. The thickness of the bituminous surfacing course should be at least 4 inches (10 cm) to minimize the chances of reflection cracking when P-304 is used.

34. FROST EFFECTS. Frost protection should be provided in areas where conditions conducive to detrimental frost action exist. Frost protection recommendations are given in paragraph 23b of this document. Frost considerations may result in thicker subbase courses than the thicknesses needed for structural support.

35. DESIGN EXAMPLE. As an example of the use of the design curves, assume a flexible pavement is to be designed for a dual gear aircraft having a gross weight of 75,000 pounds (34 000 kg) and 6,000 annual equivalent departures of the design aircraft. Design CBR values for the subbase and subgrade are 20 and 6, respectively.

a. Total Pavement Thickness. The total pavement thickness required is determined from Figure 3-4. Enter the upper abscissa with the subgrade CBR value, 6. Project vertically downward to the gross weight of the designing aircraft, 75,000 pounds (34 000 kg). At the point of intersection of the vertical projection and the aircraft gross weight, make a horizontal projection to the equivalent annual departures, 6000. From the point of intersection of the horizontal projection and the annual departure level, make a vertical projection down to the lower abscissa and read the total pavement thickness; in this example - 21.3 inches (51.2 cm).

b. Thickness of Subbase Course. The thickness of the subbase course is determined in a manner similar to the total pavement thickness. Using Figure 3-4, enter the upper abscissa with the design CBR value for the subbase, 20. The chart is used in the same manner as described in "a" above, i.e., vertical projection to aircraft gross weight, horizontal projection to annual departures, and vertical projection to lower abscissa. In this example the thickness obtained is 8.6 inches (21.8 cm). This means that the combined thickness of bituminous surface and base course needed over a 20 CBR subbase is 8.6 inches (21.8 cm), thus having a subbase thickness of 21.3 - 8.6 = 12.7 inches (32.2 cm).

c. Thickness of Bituminous Surface. As indicated by the note in Figure 3-4, the thickness of bituminous surface for critical areas is 4 inches (10 cm) and for noncritical, 3 inches (7.5 cm).

d. Thickness of Base Course. The thickness of base course can be computed by subtracting the thickness of bituminous surface from the combined thickness of surface and base determined in "b" above; in this example 8.6 - 4.0 = 4.6 inches (11.7 cm) of base course. The thickness of base course thus calculated should be compared with the minimum base course thickness required as shown in Figure 3-12. Note that the minimum base course thickness is 6 inches (15.2 cm) for critical areas. Enter the left ordinate of Figure 3-12 with the total pavement thickness as determined in "a" above, in this example - 21.3 inches (51.2 cm). Make a horizontal projection to the subgrade CBR line; in this example, 6. From the intersection of the horizontal projection and the subgrade CBR line, make a vertical projection down to the lower abscissa and read the minimum base course thickness, in this example the minimum thickness of 6 inches (15.2 cm) would be required. The extra thickness of base required by Figure 3-12 as opposed to the earlier calculation is taken out of the subbase thickness not added to the total pavement thickness; in this example 12.7 - 1.4 = 11.3 inches (28.7 cm).

e. Thickness of Noncritical Areas. The total pavement thickness for noncritical areas is obtained by taking 0.9 of the critical pavement base and subbase thicknesses plus the required bituminous surface thickness given on the design charts. For the thinned edge portion of the critical and noncritical pavements, the 0.7T factor applies only to the base course because the subbase should allow for transverse drainage. The transition section and surface course requirements are as noted in Figure 3-1.

f. Summary. The thickness calculated in the above paragraphs should be rounded off to even increments as discussed in paragraph 32. If conditions for detrimental frost action exist, another analysis is required. The final design thicknesses for this example would be as follows:

	Thickness Requirements					
	Critical		Noncritical		Edge	
	in.	(cm)	in.	(cm)	in.	(cm)
Bituminous Surface	4	(10)	3	(8)	2	(5)
Base Course	6	(15)	5	(13)	4	(10)
Subbase Course	11	(28)	10	(25)	8	(20)
Transverse Drainage	0	(0)	3	(8)	7	(18)

Since the design aircraft in this example weighs less than 100,000 pounds (45 300 kg) stabilized base and subbase are not required but could be used if desired.

36. STABILIZED SUBBASE AND BASE EQUIVALENCY FACTORS.

a. Stabilized subbase courses offer some structural benefits to a flexible pavement. The benefits can be expressed in the form of equivalency factors which indicate the substitution thickness ratios applicable to various stabilized layers. The thickness of stabilized material can be computed by dividing the granular subbase thickness requirement by the equivalency factor. The equivalency factor ranges are presented in Table 3-2 below:

TABLE 3-2. RECOMMENDED EQUIVALENCY FACTOR RANGE STABILIZED SUBBASE

Material	Equivalency Factor Range
P-401, Bituminous Surface Course	1.7-2.3
P-201, Bituminous Base Course	1.7-2.3
P-215, Cold Laid Bituminous Base Course	1.5-1.7
P-216, Mixed In-Place Base Course	1.5-1.7
P-304, Cement Treated Base Course	1.6-2.3
P-301, Soil Cement Base Course	1.5-2.0
P-209, Crushed Aggregate Base Course	1.4-2.0
P-154, Subbase Course	1.0

In establishing the equivalency factors shown above the CBR of the standard subbase, P-154 was assumed to be 20.

b. Stabilized base courses offer structural benefits to a flexible pavement in much the same manner as stabilized subbase. The benefits are expressed as equivalency factors similar to those shown for stabilized subbase. These ratios are used to compute the thickness of stabilized base by dividing the granular base requirement by the equivalency factor. The equivalency factor ranges are presented in Table 3-3 below:

TABLE 3-3. RECOMMENDED EQUIVALENCY FACTOR RANGE STABILIZED BASE

Material	Equivalency Factor Range
P-401, Bituminous Surface Course	1.2-1.6
P-201, Bituminous Base Course	1.2-1.6
P-215, Cold Laid Bituminous Base Course	1.0-1.2
P-216, Mixed In-Place Base Course	1.0-1.2
P-304, Cement Treated Base Course	1.2-1.6
P-301, Soil Cement Base Course	N/A
P-209, Crushed Aggregate Base Course	1.0
P-154, Subbase Course	N/A

The equivalency factors shown above assume a CBR value of 80 for P-209.

c. Ranges of equivalency factors are shown for the stabilized materials, as variations in the quality of materials, construction techniques, and control can influence the equivalency factor. In the selection of equivalency factors, consideration should be given to the traffic using the pavement, total pavement thickness, and the thickness of the individual stabilized layer. For example, a thin stabilized layer in a pavement structure subjected to heavy loads spread over large areas will result in an equivalency factor near the low end of the range. Conversely, light loads on thick layers will call for equivalency factors near the upper end of the ranges.

d. Restrictions on the substitution of stabilized layers for granular layers are necessary because of construction limitations and other considerations such as subgrade stresses. The minimum thicknesses of P-201 and P-304 should be 4 inches (10 cm) and 6 inches (15 cm), respectively. The minimum total thickness shall not be less than the total pavement thickness required by the CBR 20 line of the appropriate design curve. In order to prevent a saturated entrapped granular layer in the pavement structure, no unstabilized granular base course shall be placed on a stabilized subbase course. Frost protection should still be provided when stabilized layers are used.

e. As an example of the use of the equivalency factors assume a flexible pavement is required to serve a design aircraft weighing 300,000 pounds (91 000 kg) with a dual tandem landing gear. The equivalent annual departures are 15,000. The subgrade design CBR value is 7. Item P-216 will be used as the subbase course and Item P-201 will be used as the base course. Enter Figure 3-5 with the subgrade CBR of 7 and read a total pavement thickness of 40.2 inches (102 cm) using the reference value of 20 (see paragraph 34d) for subbase; enter Figure 3-5 again and read the bituminous surface plus base course thickness needed to protect a 20 CBR subbase - 19.0 inches (48 cm). The unstabilized section thus obtained would consist of 4 inches (10 cm) of bituminous surface, 15 inches (38 cm) of base and 21 inches (53 cm) of subbase. The stabilized subbase, P-216, will be assigned an equivalency factor of 1.5 (see Table 3-2) based on the location within the structure, size of loading involved, and local experience. This means that 1 inch (2.5 cm) of P-216 is equivalent to 1.5 inches (3.8 cm) of granular subbase with a CBR of 20. The thickness of stabilized subbase would therefore equal 21 inches (53 cm) divided by 1.5 or 14 inches (36 cm). The stabilized base, P-201, is assigned an equivalency factor of 1.6 (see Table 3-3) based on its high quality and record for good performances in the area. This means 1 inch (2.5 cm) of P-201 is equal to 1.6 inches (4.1 cm) of CBR 80 base material. The thickness of stabilized base would be determined by dividing 15 by 1.6, yielding 9.4 inches (24 cm). The stabilized base thickness would be rounded to 9 inches (23 cm). The final stabilized section would consist of 4 inches (10 cm) bituminous surface, 9 inches (24 cm) stabilized base, P-201, and 14 inches (36 cm) stabilized subbase, P-216. The total pavement thickness is 27 inches (69 cm) and should be compared with the total pavement thickness for CBR 20 which is 19 inches (48 cm). Since the total pavement thickness with stabilization, 27 inches (69 cm), is greater than that required for CBR 20, 19 inches (48 cm), the design is suitable. Had the total stabilized thickness been less than the CBR 20 thickness, the stabilized subbase thickness would have been increased to make up the difference.

37. FULL-DEPTH ASPHALT PAVEMENTS. Full-depth asphalt pavements contain asphaltic cement in all components above the prepared subgrade. The design of full-depth asphalt pavements can be accomplished using the equivalency factors presented in paragraph 36. Manual Series No. 11 prepared by The Asphalt Institute, dated January 1973, can also be used to design full-depth asphalt pavements when approved by the FAA. The designer is reminded that full-depth asphalt pavements also require consideration of frost effects.

SECTION 3. RIGID PAVEMENT DESIGN.

38. GENERAL. Rigid pavements for airports are composed of portland cement concrete placed upon a granular or treated subbase course that rests upon a compacted subgrade. Under certain conditions, a subbase is not required, see paragraph 40.

39. CONCRETE PAVEMENT. The concrete surface must provide an acceptable nonskid surface, prevent the infiltration of surface water, and provide structural support to the aircraft. The quality of the concrete, the mixes, the control tests, methods of construction and handling, and quality of workmanship are covered in detail in Item P-501, Cement Concrete Pavement.

40. SUBBASE. The purpose of a subbase under a rigid pavement is to provide uniform stable support for the pavement slabs. A minimum thickness of 4 inches (10 cm) of subbase is required under all rigid pavements, except as shown in Table 3-4 below:

TABLE 3-4. CONDITIONS WHERE NO SUBBASE IS REQUIRED

Soil Classification	Good Drainage		Poor Drainage	
	No Frost	Frost	No Frost	Frost
GW	X	X	X	X
GP	X	X	X	
GM	X			
GC	X			
SW	X			

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

Item P-154, Subbase Course

Item P-201, Bituminous Base Course

Item P-208, Aggregate Base Course

Item P-209, Crushed Aggregate Base Course

Item P-301, Soil Cement Base Course

Item P-304, Cement Treated Base Course

41. STABILIZED SUBBASE. Stabilized subbase is to be required for all new rigid pavements designed to accommodate aircraft weighing 100,000 pounds (45 400 kg) or more. The structural benefit imparted to a pavement section by a stabilized subbase is reflected in the modulus of subgrade reaction assigned to the foundation. Exceptions to the policy of using stabilized subbase are the same as given in paragraph 33.

42. SUBGRADE.

a. The subgrade materials under a rigid pavement should be compacted to provide adequate stability and uniform support as with flexible pavement; however, the compaction requirements for rigid pavements are not as stringent as flexible pavement due to the relatively lower subgrade stress. For cohesive soils used in fill sections, the entire fill shall be compacted to 90 percent maximum density. For cohesive soils in cut sections, the top 6 inches (15 cm) of the subgrade shall be compacted to 90% maximum density. For noncohesive soils used in fill sections, the top 6 inches (15 cm) of fill shall be compacted to 100 percent maximum density, and the remainder of the fill shall be compacted to 95 percent maximum density. For cut sections in noncohesive soils, the top 6 inches (15 cm) of subgrade shall be compacted to 100 percent maximum density and the next 18 inches (46 cm) of subgrade shall be compacted to 95 percent maximum density.

b. Swelling soils will require special considerations. FAA Report No. FAA-RD-76-66, Design and Construction of Airport Pavements on Expansive Soils, contains guidance on the identification and treatment of swelling soils. See Appendix 4.

43. <u>DETERMINATION OF FOUNDATION MODULUS (k VALUE) FOR RIGID PAVEMENT</u>. In addition to the soils survey and analysis and classification of subgrade conditions, the determination of the foundation modulus is required for rigid pavement design. The foundation modulus (k value) should be assigned to the material directly beneath the concrete pavement. However, it is recommended that a k value be established for the subgrade and then corrected to account for the effects of subbase.

a. <u>Determination of k Value for Subgrade</u>. The preferred method of determining the subgrade modulus is by testing a limited test section which has been constructed to the required specifications. The section should consist of the design subgrade material and utilization of plate bearing test procedures specified in AASHTO T 222 Nonrepetitive Static Plate Load Test of Soils and Flexible Pavement Components, for Use in Evaluation and Design of Airport and Highway Pavements. If the construction and testing of a test section is impractical, the values listed in Table 2-3 may be used. The designer is cautioned that the values in Table 2-3 are approximate and engineering judgment should be used in selecting a design value. Fortunately, rigid pavement is not too sensitive to k value and an error in estimating k will not have a large impact on rigid pavement thickness.

b. <u>Determination of k Value for Granular Subbase</u>. The probable increase in k value associated with various thicknesses of different subbase materials is shown in Figure 2-5. Figure 2-5 is intended for use when the subbase is composed of unstabilized granular materials such as P-154 or P-209. Values shown in Figure 2-5 are to be considered guides and can be tempered by local experience.

c. Determination of k Value for Stabilized Subbase. The effect of stabilized subbase is reflected in the foundation modulus. The difficulty in assigning a foundation modulus is that test data will not be available during the design phase. Figure 3-13 shows the probable increase in k value with various thicknesses of stabilized subbase located on subgrades of varying moduli. Figure 3-13 is applicable to cement stabilized, P-304, and bituminous stabilized, P-201, layers. Figure 3-13 was developed by assuming a stabilized layer is twice as effective as a well-graded crushed aggregate in increasing the subgrade modulus. Stabilized layers of lesser quality than P-304 or P-201 should be assigned somewhat lower k values. After k value is assigned to the stabilized subbase, the design procedure is the same as described in paragraph 45.

44. DETERMINATION OF CONCRETE SLAB THICKNESS. Design curves have been prepared for rigid pavements similar to those for flexible pavements; i.e., separate curves for single, dual, and dual tandem landing gear assemblies and separate design curves for wide body jet aircraft. See Figures 3-14 through 3-22. These curves are based on a jointed edge loading assumption where the load is tangent to the joint. Use of the design curves requires four design input parameters: concrete flexural strength, subgrade modulus, gross weight of the design aircraft, and annual departure of the design aircraft. The rigid pavement design curves indicate the thickness of concrete only. Thicknesses of other components of the rigid pavement structure must be determined separately.

a. Concrete Flexural Strength. The required thickness of concrete pavement is related to the strength of the concrete used in the pavement. Concrete strength is assessed by the flexural strength method as the primary action of a concrete pavement slab is flexure. Concrete flexural strength should be determined by ASTM C-78 test method. Normally a 90-day flexural strength is used for design. Item P-501 of AC 150/5370-10 is specified in terms of the 28-day flexural strength of the concrete. This is done to reduce the curing of the specimens to a practical time. The designer can safely assume the 90-day flexural strength of concrete will be 10% higher than the 28-day strength except when high early strength cement or pozzolanic admixtures are used. When either high early strength cement or pozzolanic admixtures are used, the 28-day flexural strength should be used for design.

b. k Value. The k value is, in effect, a spring constant for the material supporting the rigid pavement and is indicative of the bearing value of the supporting material.

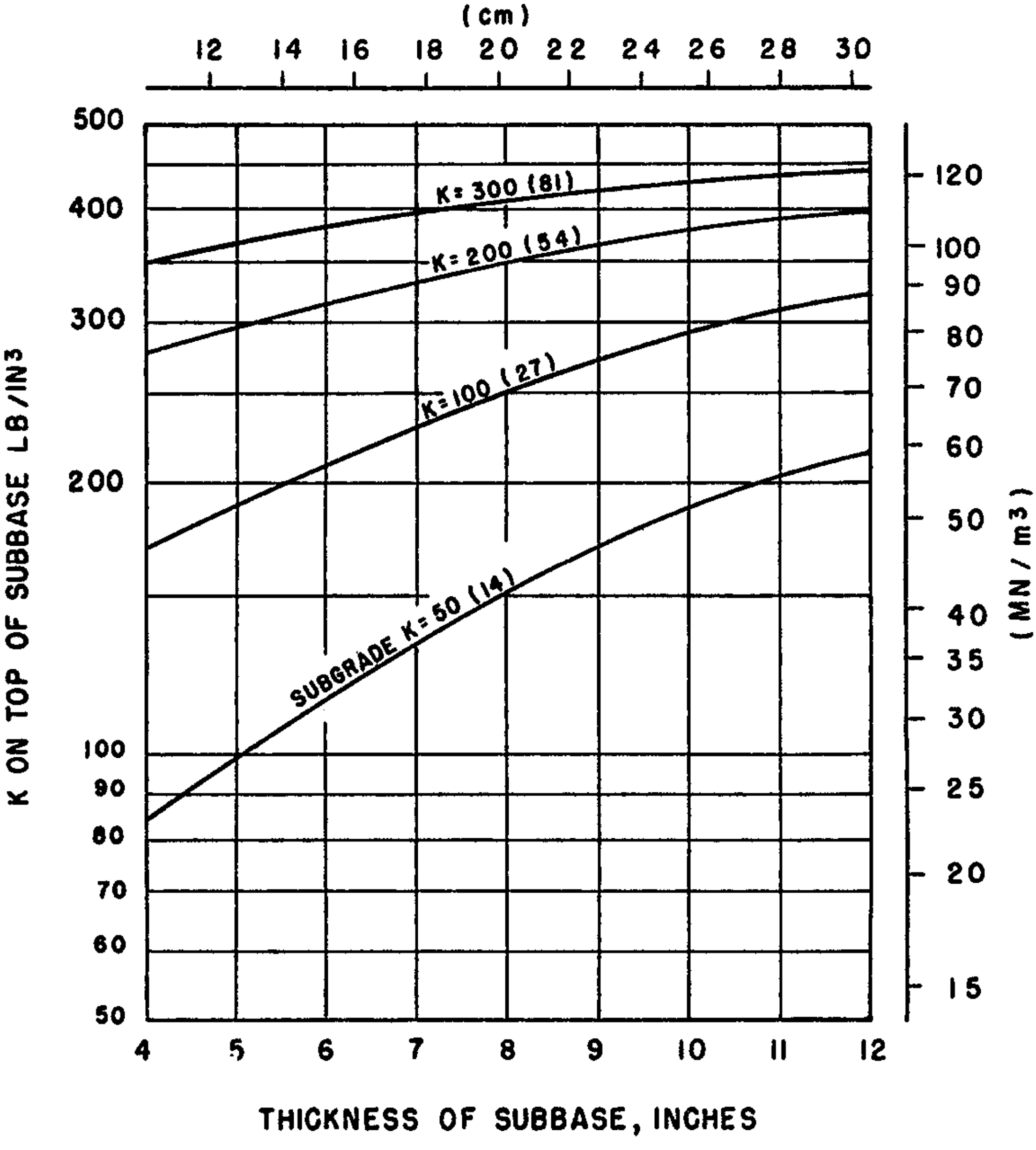
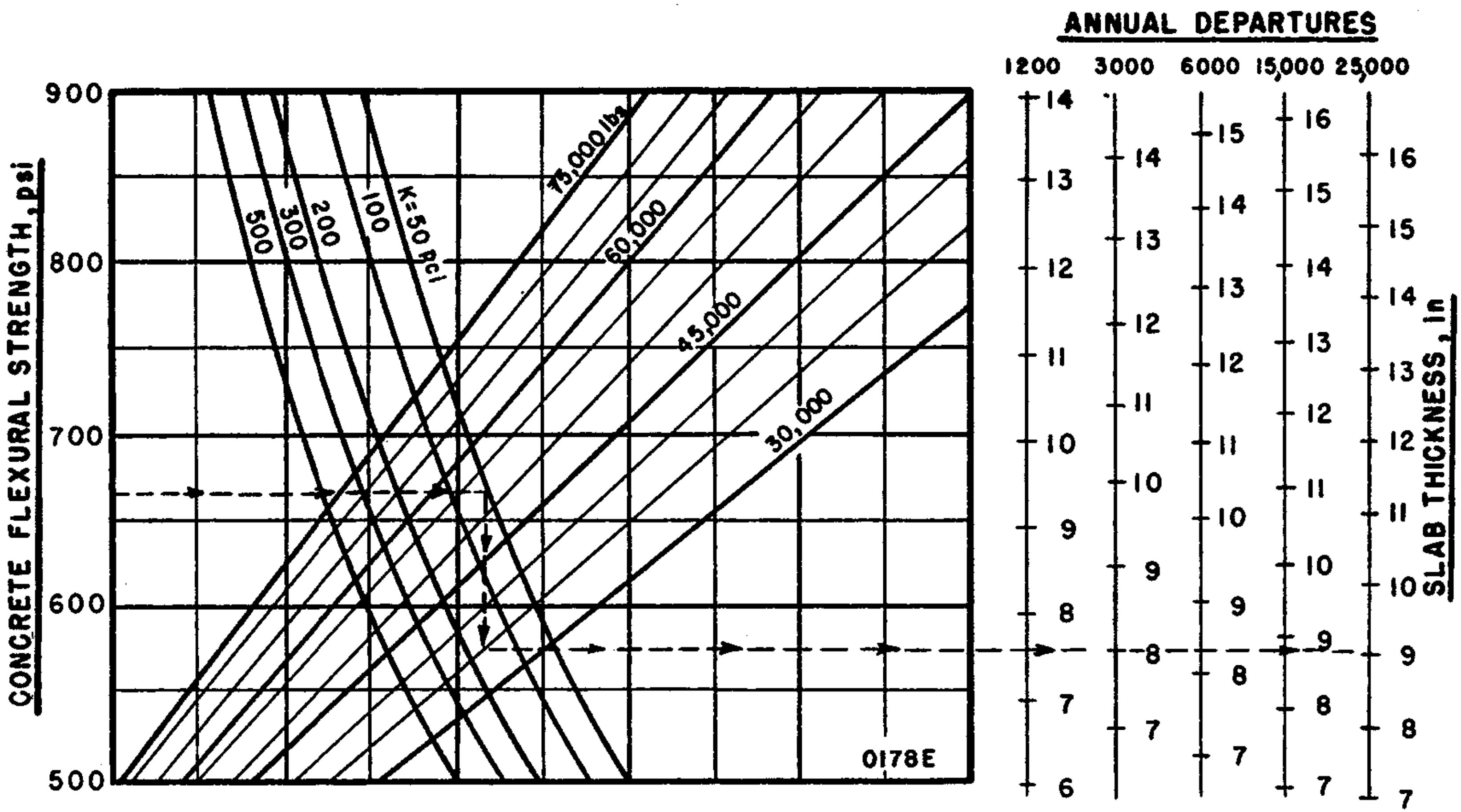


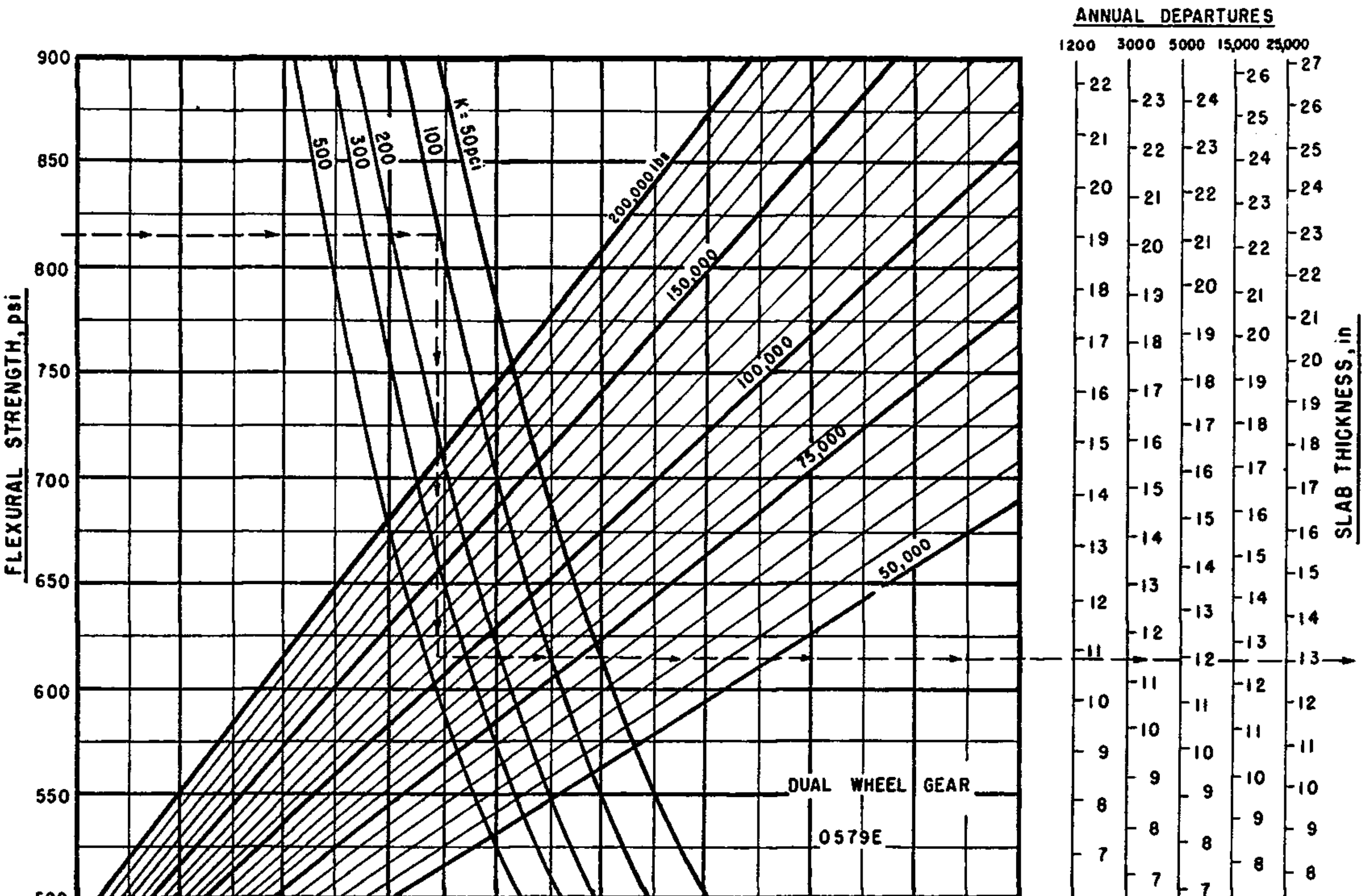
FIGURE 3-13. EFFECT OF STABILIZED SUBBASE ON SUBGRADE MODULUS



NOTE:

1 inch = 2.54 cm | psi = 0.0069 MN/m²
1 lb = 0.454 kg | pci = 0.272 MN/m³

FIGURE 3-14. RIGID PAVEMENT DESIGN CURVES - SINGLE WHEEL GEAR



NOTE:
1 inch = 2.54 cm 1 psi = 0.0069 MN/m²
1 lb = 0.454 kg 1 pci = 0.272 MN/m³

FIGURE 3-15. RIGID PAVEMENT DESIGN CURVES - DUAL WHEEL GEAR

*

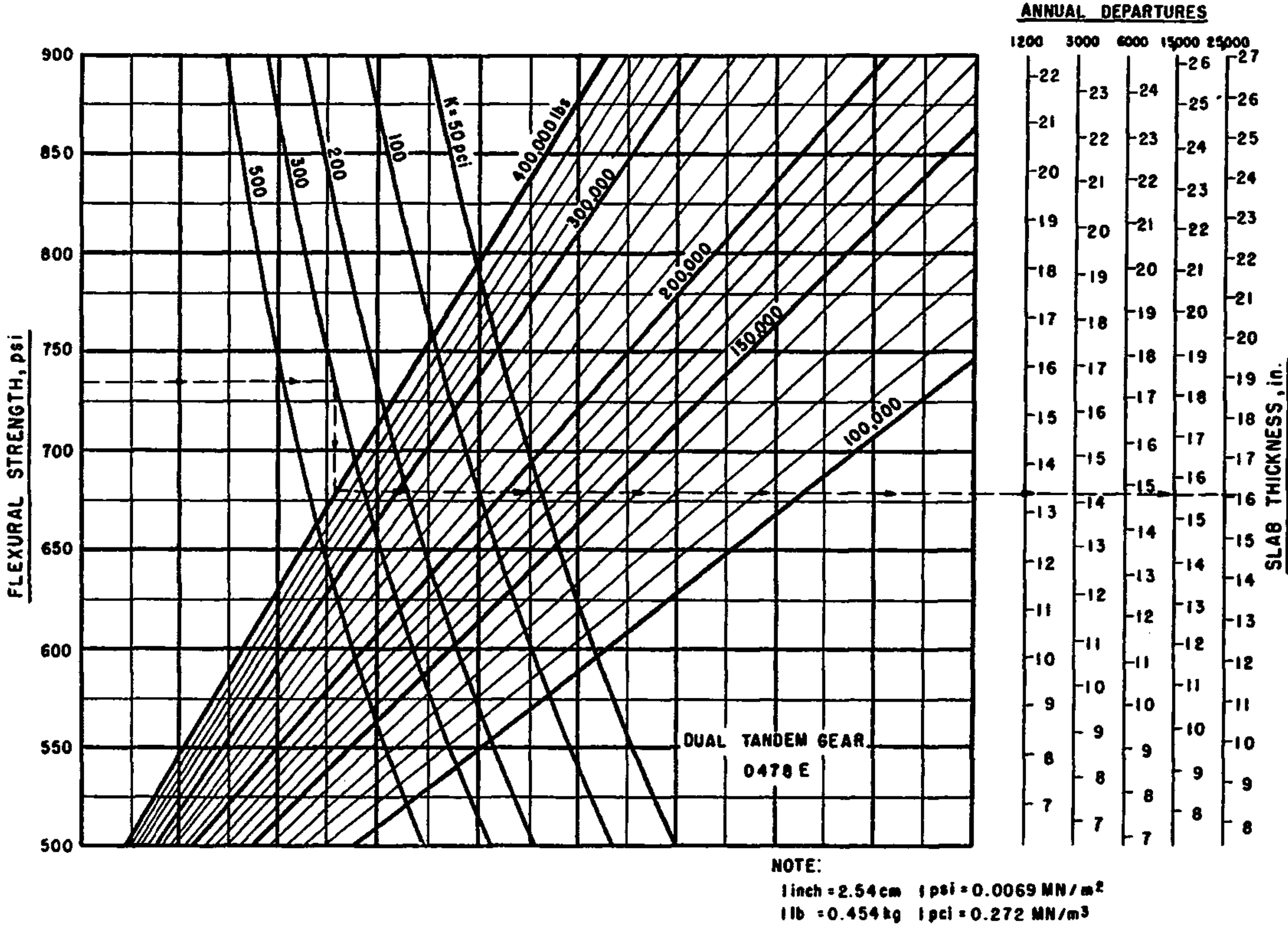


FIGURE 3-16. RIGID PAVEMENT DESIGN CURVES - DUAL TANDEM GEAR

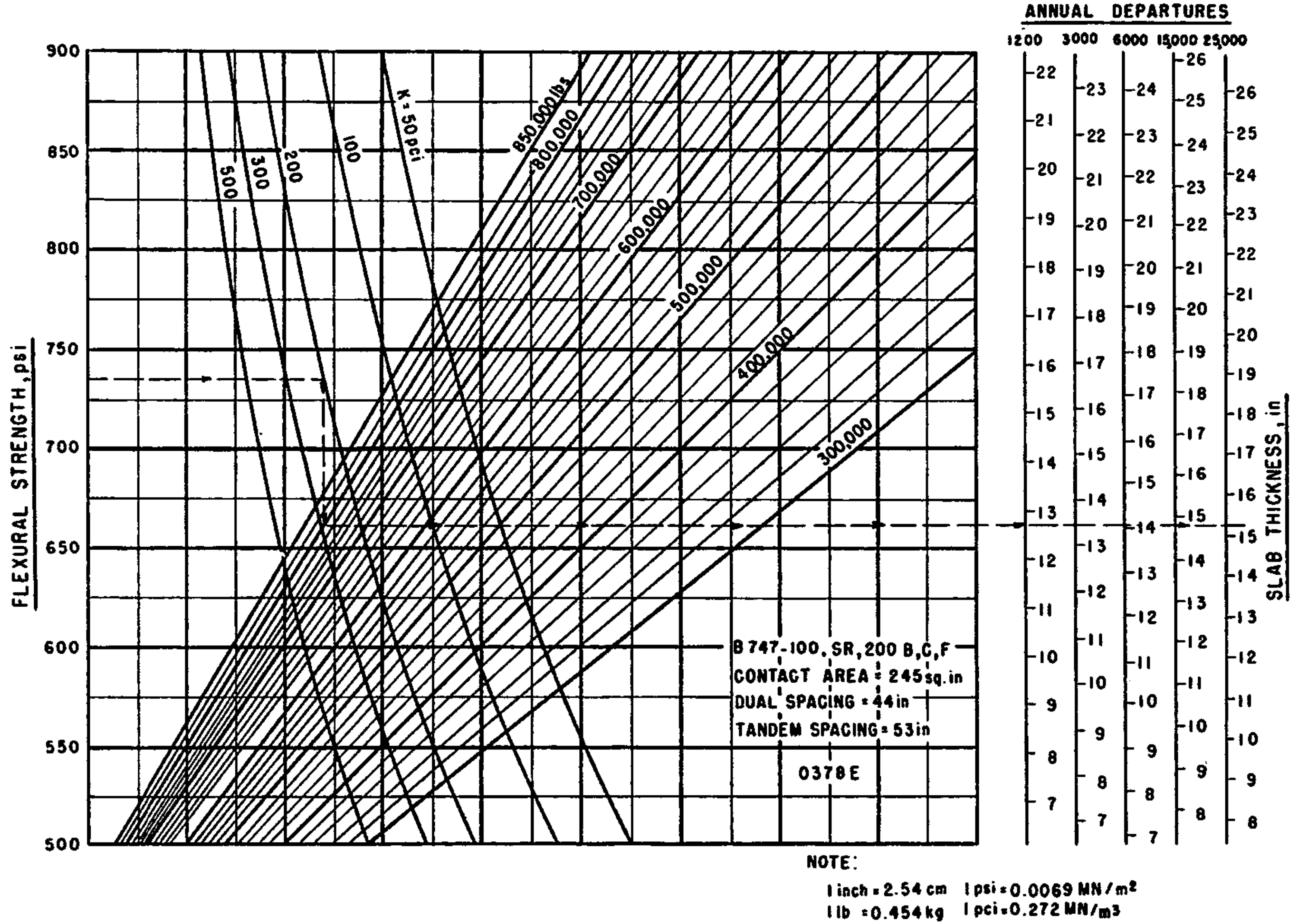
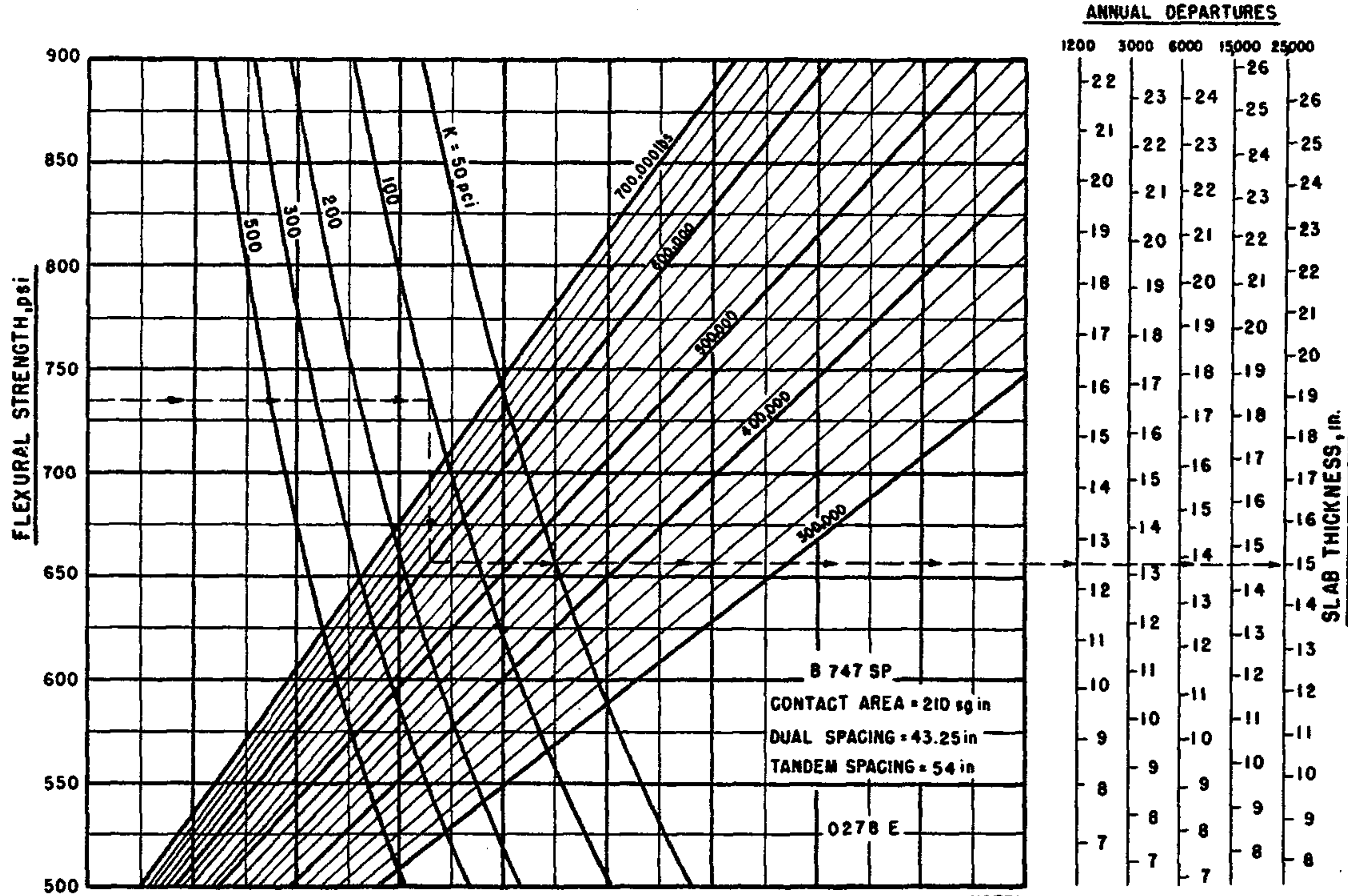


FIGURE 3-17. RIGID PAVEMENT DESIGN CURVES - B-747-100, SR, 200 B, C, F



NOTE:
1 inch = 2.54 cm   1 psi = 0.0069 MN/m²
1 lb = 0.454 kg   1 pci = 0.272 MN/m³

FIGURE 3-18. RIGID PAVEMENT DESIGN CURVES - B-747-SP

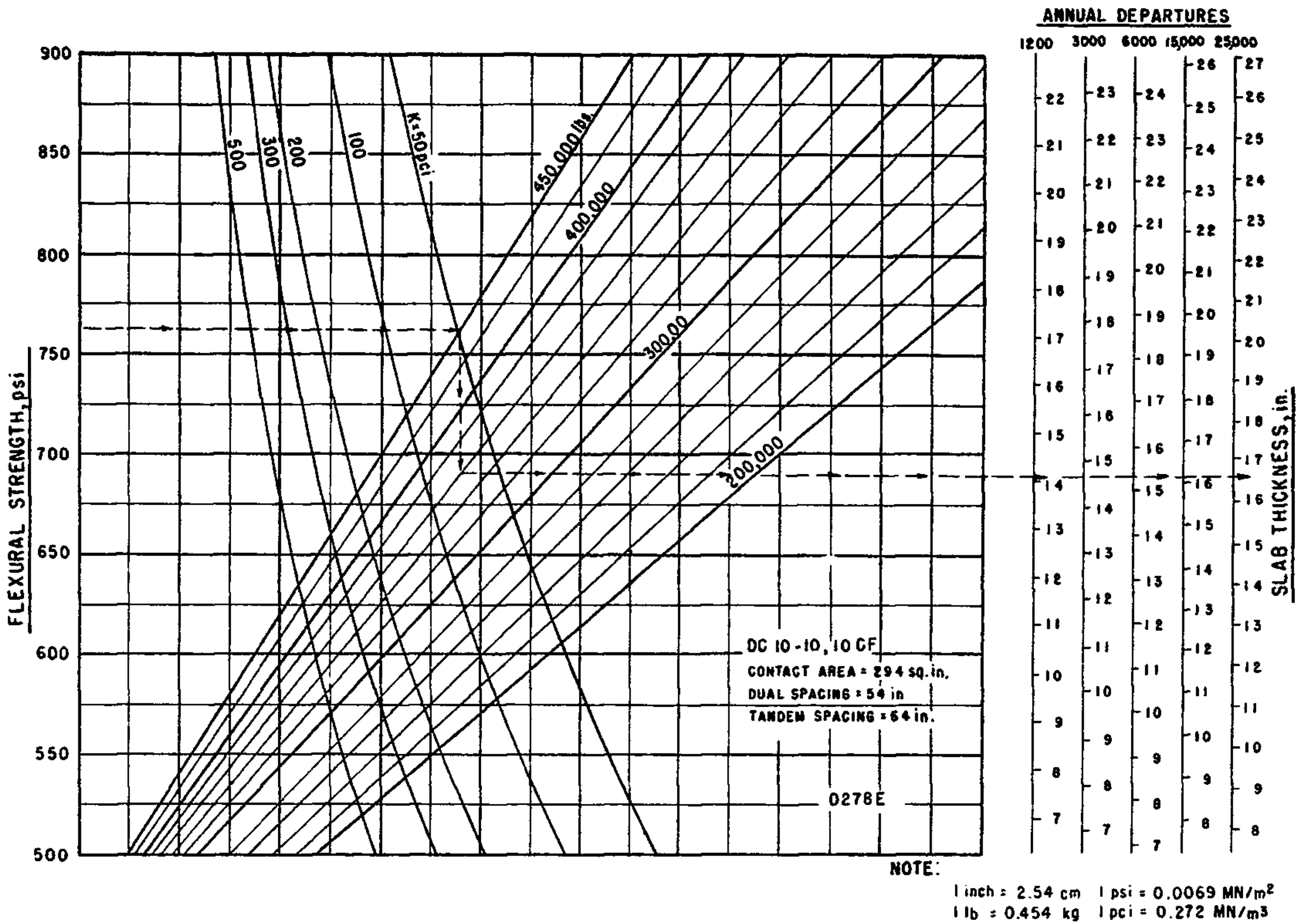


FIGURE 3-19. RIGID PAVEMENT DESIGN CURVES - DC 10-10, 10CF

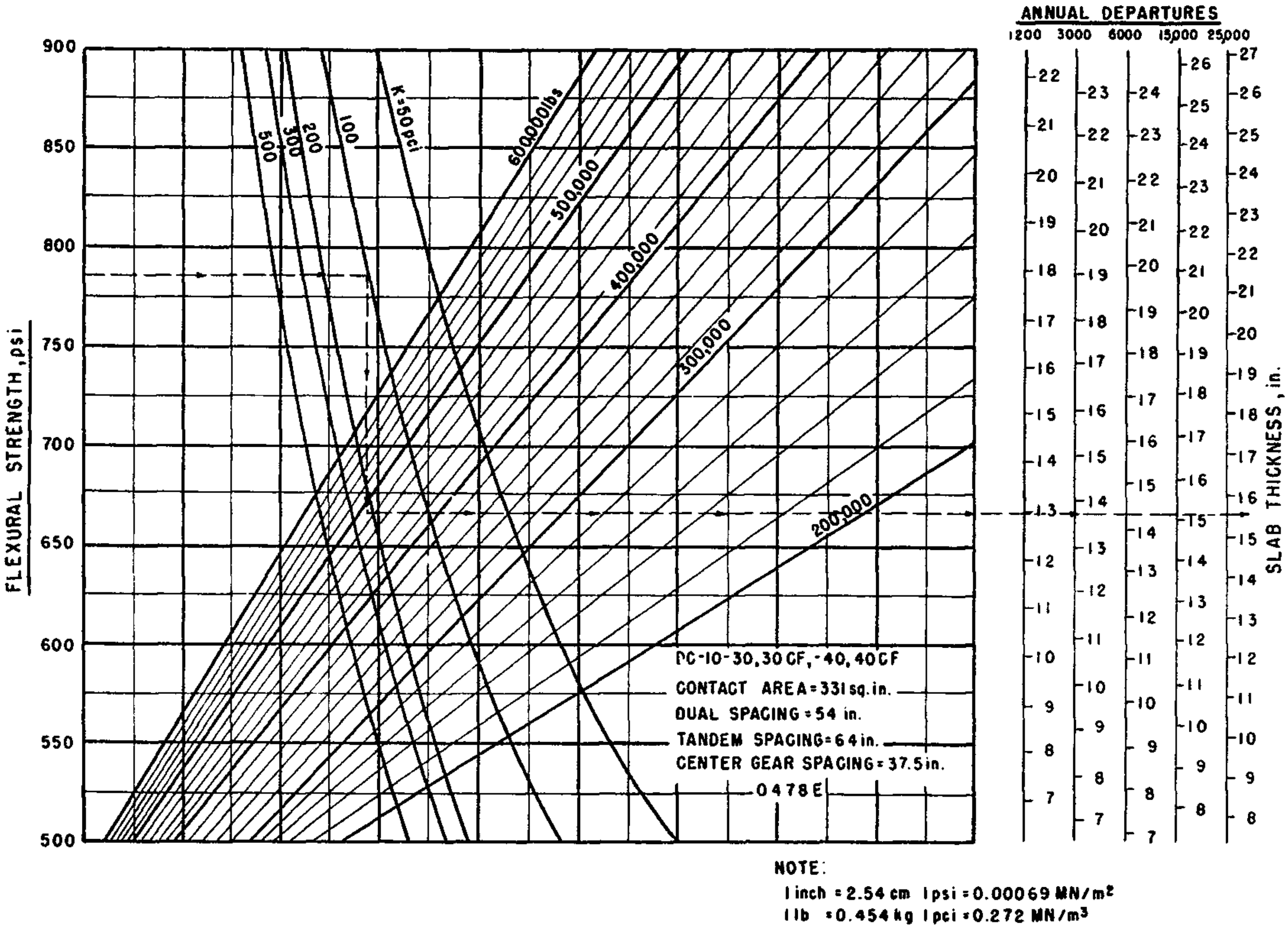
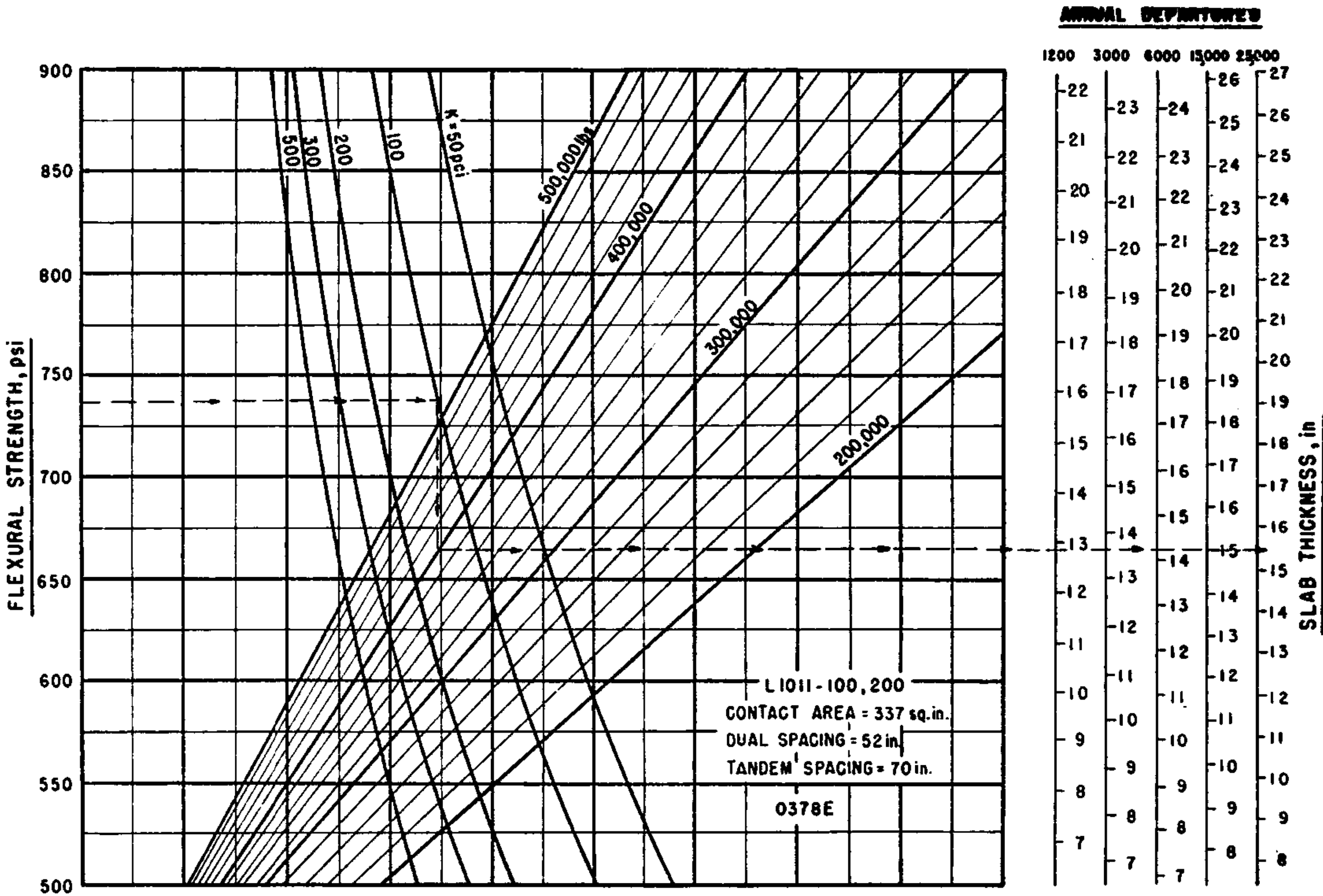


FIGURE 3-20. RIGID PAVEMENT DESIGN CURVES - DC 10-30, 30CF, 40, 40CF



NOTE:

1 inch = 2.54 cm 1psi = 0.0069 MN/m²

1lb = 0.454 kg 1pci = 0.272 MN/m³

FIGURE 3-22. RIGID PAVEMENT DESIGN CURVES - L-1011-100,200

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

d. Annual Departures of Design Aircraft. The fourth input parameter is annual departures of the design aircraft. The departures should be computed using the procedure explained in paragraph 20.

45. USE OF DESIGN CURVES. The rigid pavement design curves are constructed such that the design inputs are entered in the same order as they are discussed in paragraph 44. Concrete flexural strength is the first input. The left ordinate of the design curve is entered with concrete flexural strength. A horizontal projection is made until it intersects with the appropriate foundation modulus line. A vertical projection is made from the intersection point to the appropriate gross weight of the design aircraft. A horizontal projection is made to the right ordinate showing annual departures. The pavement thickness is read from the appropriate annual departure line. The pavement thickness shown refers to the thickness of the concrete pavement only, exclusive of the subbase. This thickness is that shown as "T" in Figure 3-1, sometimes referred to as the critical thickness.

46. FROST EFFECTS. As with flexible pavements, frost protection must be provided for rigid pavements in areas where conditions conducive to detrimental frost action exist. Frost protection considerations differ somewhat from those for flexible pavements and local experience should dictate the depth of frost protection required. In the absence of such experience, the frost protection requirements given in paragraph 23b apply.

47. CRITICAL AND NONCRITICAL AREAS.

a. The design curves, Figures 3-14 through 3-22, are used to determine the concrete slab thickness for the critical pavement areas shown as "T" in Figure 3-1. The 0.9T thickness for noncritical areas applies to the concrete slab thickness. For the variable thickness section of the thinned edge and transition section, the reduction applies to the concrete slab thickness. The change in thickness for the transitions should be accomplished over an entire slab length or width. In areas of variable slab thickness, the subbase thickness must be adjusted as necessary to provide surface drainage from the entire subgrade surface. For fractions of an inch of 0.5 or more, use the next higher whole number; for less than 0.5, use the next lower number.

b. When aircraft loadings are applied to a jointed edge, the angle of the landing gear relative to the jointed edge influences the magnitude of the stress in the slab. Figures 3-14 and 3-15, single wheel and dual wheel landing gear assemblies, are at the maximum stress when the gear is located parallel to the joint. Figures 3-16 through 3-22 apply to dual tandem landing gear assemblies. Dual tandem assemblies do not produce the maximum stress when located parallel to the joint. Locating the dual tandem at an acute angle to the jointed edge will produce the maximum stress. Design curves, Figures 3-23 through 3-29, have been prepared for dual tandem gears located tangent to the jointed edge but rotated to the angle causing the maximum stress. These design curves can be used to design pavements in areas where aircraft are likely to cross the pavement joints at angles at low speeds such as runway holding aprons, runway ends, runway-taxiway intersections, aprons, etc. Use of Figures 3-23 through 3-29 is optional and should only be applied in areas where aircraft are likely to cross pavement joints at an angle and at low speeds.

48. DESIGN EXAMPLE. As an example of the use of the design curves, assume that a rigid pavement is to be designed for dual tandem aircraft having a gross weight of 350,000 pounds (160 000 kg) and for 6,000 annual equivalent departures of the design aircraft. The equivalent annual departures of 6,000 include 1,200 annual departures of B-747 aircraft weighing 780,000 pounds (350 000 kg) gross weight. The subgrade modulus of 100 pci (25 MN/m3) with poor drainage and frost penetration is 18 inches (45 cm). The feature to be designed is a primary runway and requires 100 percent frost protection. The subgrade soil is CL. Concrete mix designs indicate a flexural strength of 650 psi (4.5 MN/m^2) can be readily produced with locally available aggregates. The gross weight of the design aircraft dictates the use of a stabilized subbase. Several thicknesses of stabilized subbases should be tried to determine the most economical section. Assume a stabilized subbase of P-304 will be used. Try a subbase thickness of 6 inches (15 cm). Using Figure 3-13, a 6-inch (15 cm) thickness of P-304 would likely increase the foundation modulus from 100 pci (25 MN/m^3) to 210 pci (57 MN/m^3). Using Figure 3-16, dual tandem design curve, with the assumed design
* data, yields a concrete pavement thickness of 16.6 inches (42 cm). This
thickness would be rounded off to 17 inches (43 cm). Since the frost *
penetration is only 18 inches (45 cm) and the combined thickness of
* concrete pavement and stabilized subbase is 23 inches (58 cm), no *
further frost protection is needed. Even though the wide body aircraft did not control the thickness of the slab, the wide bodies would have

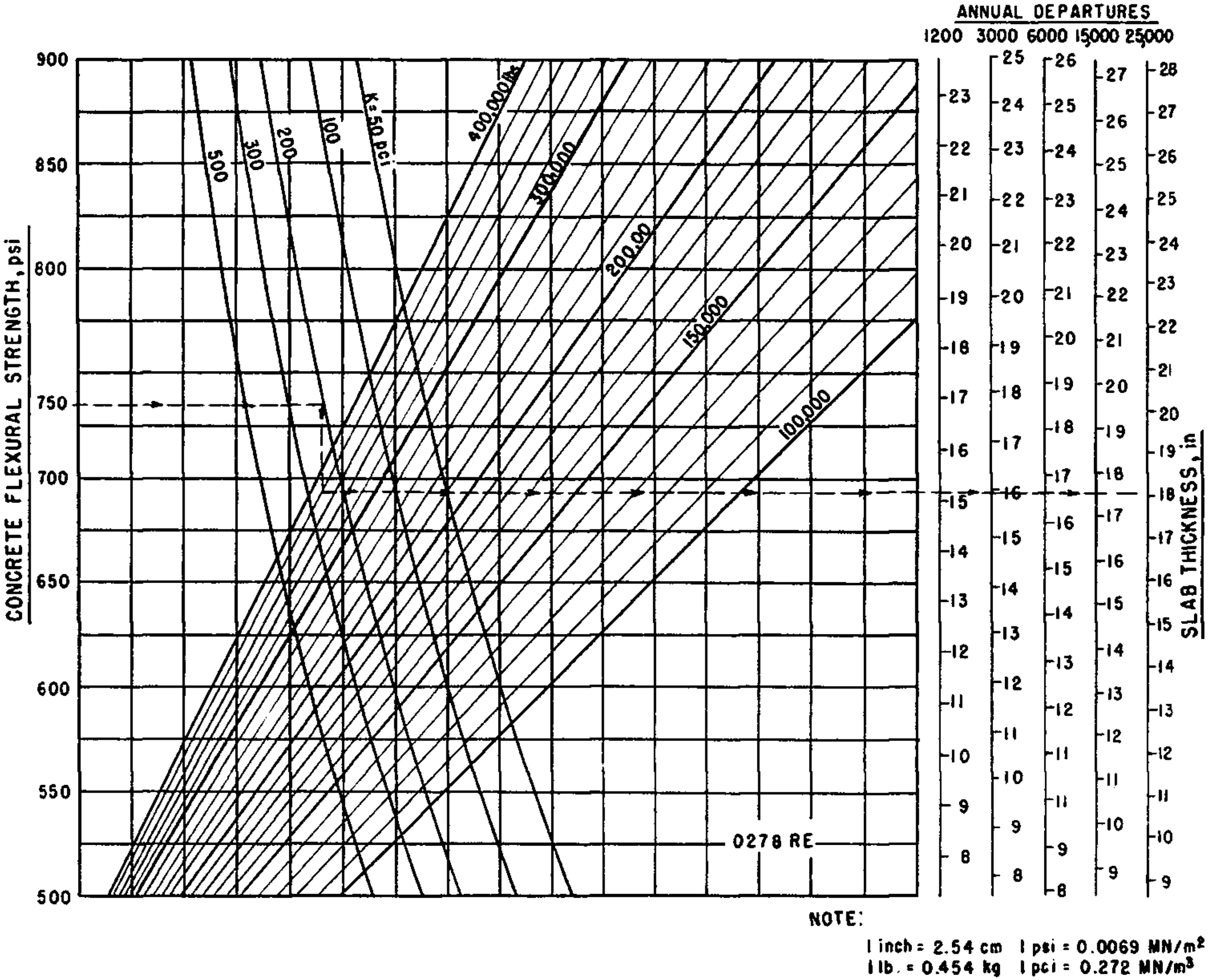


FIGURE 3-23. OPTIONAL RIGID PAVEMENT DESIGN CURVES - DUAL TANDEM GEAR

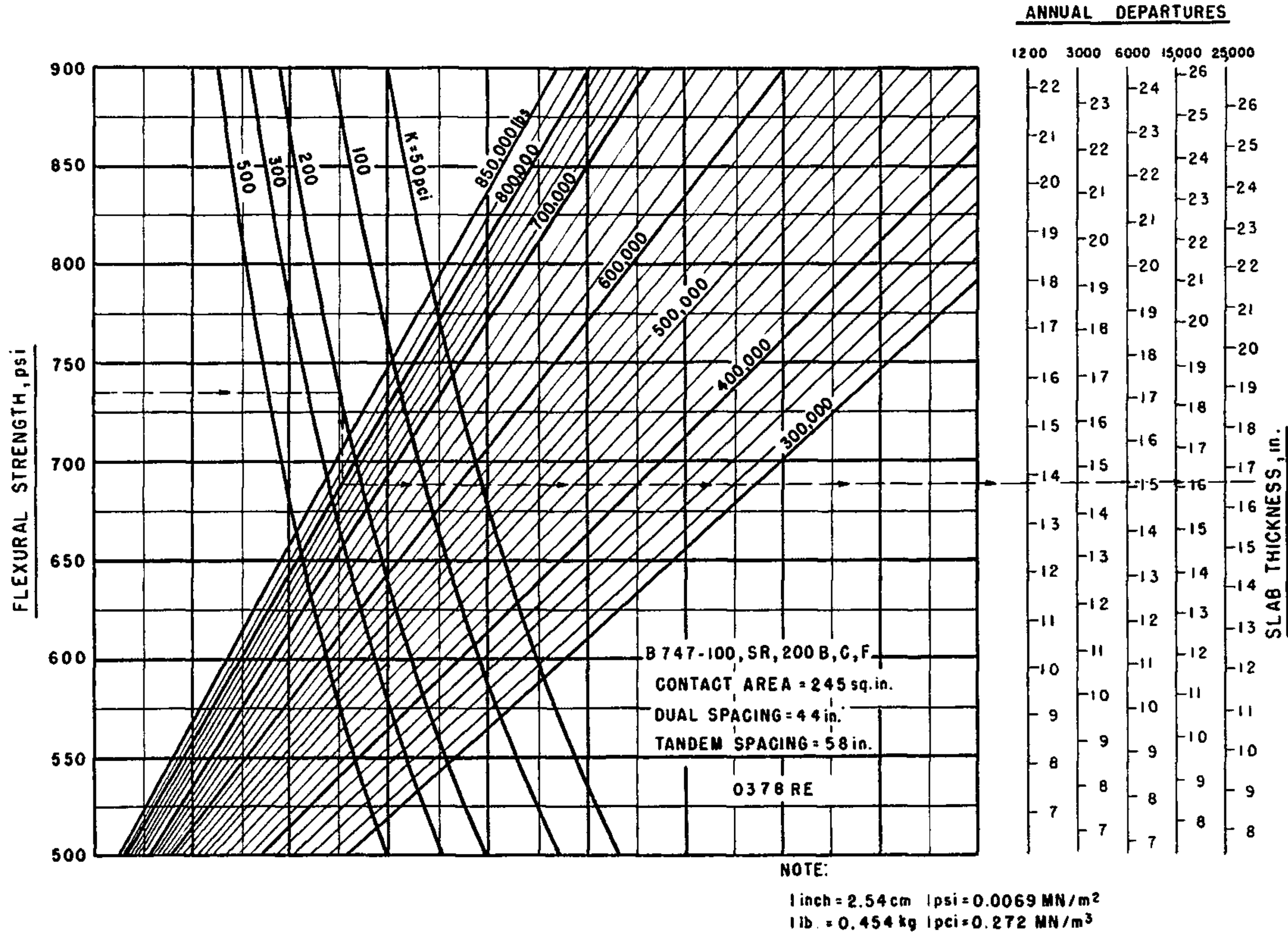


FIGURE 3-24. OPTIONAL RIGID PAVEMENT DESIGN CURVES - B-747-100, SR, 200 B, C, F

284-571 (R41)

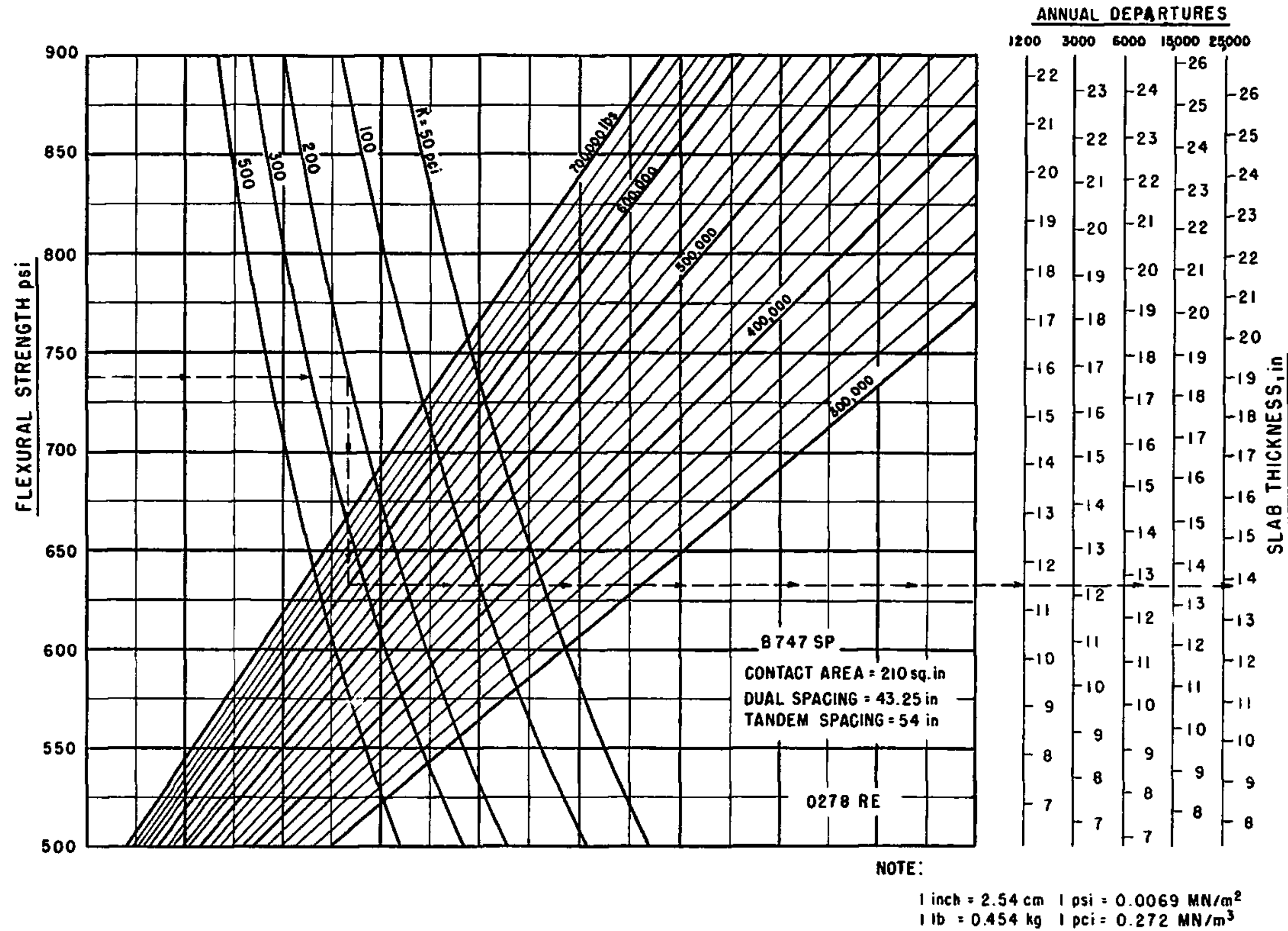


FIGURE 3-25. OPTIONAL RIGID PAVEMENT DESIGN CURVES - B-747-SP

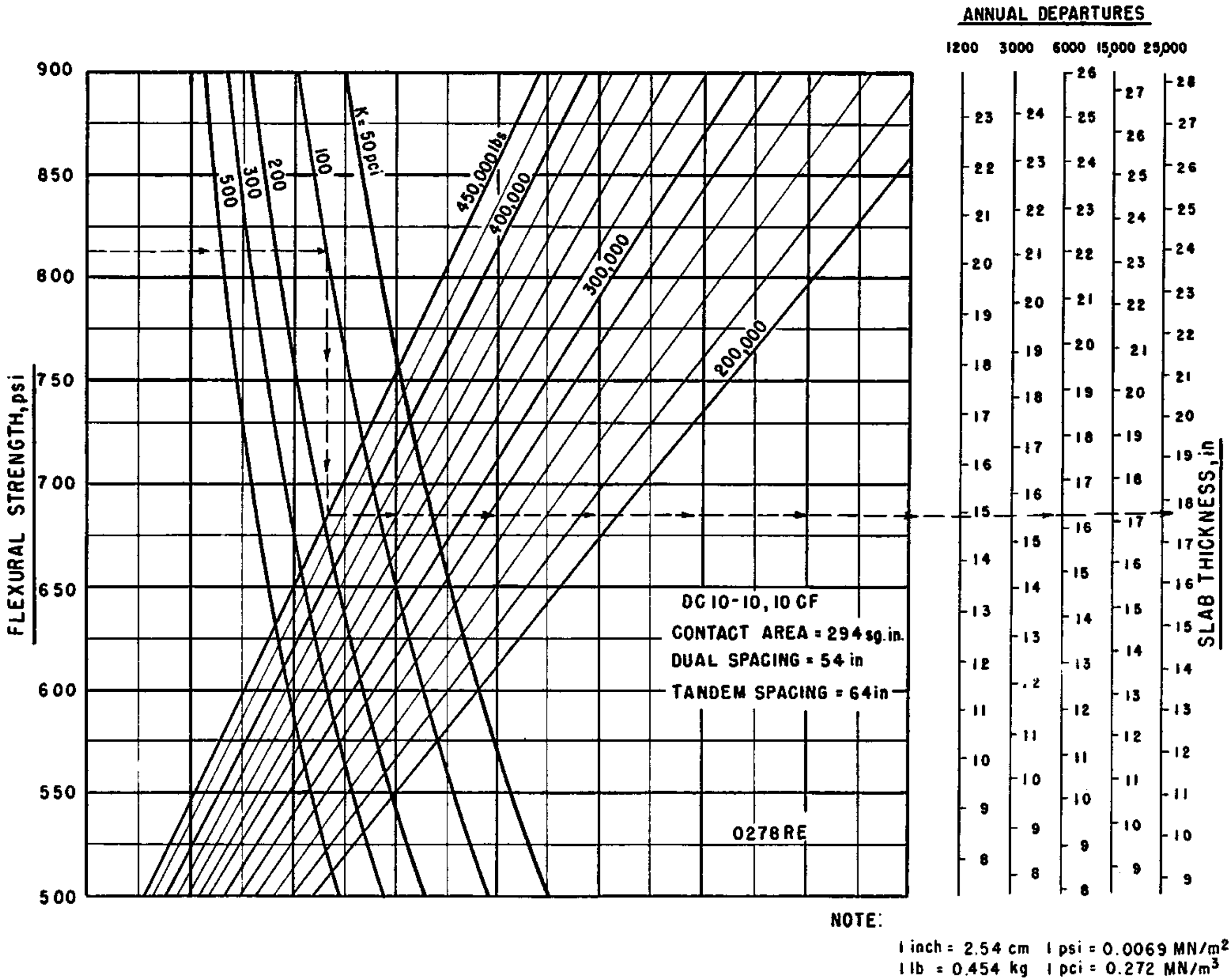


FIGURE 3-26. OPTIONAL RIGID PAVEMENT DESIGN CURVES - DC 10-10, 10CF

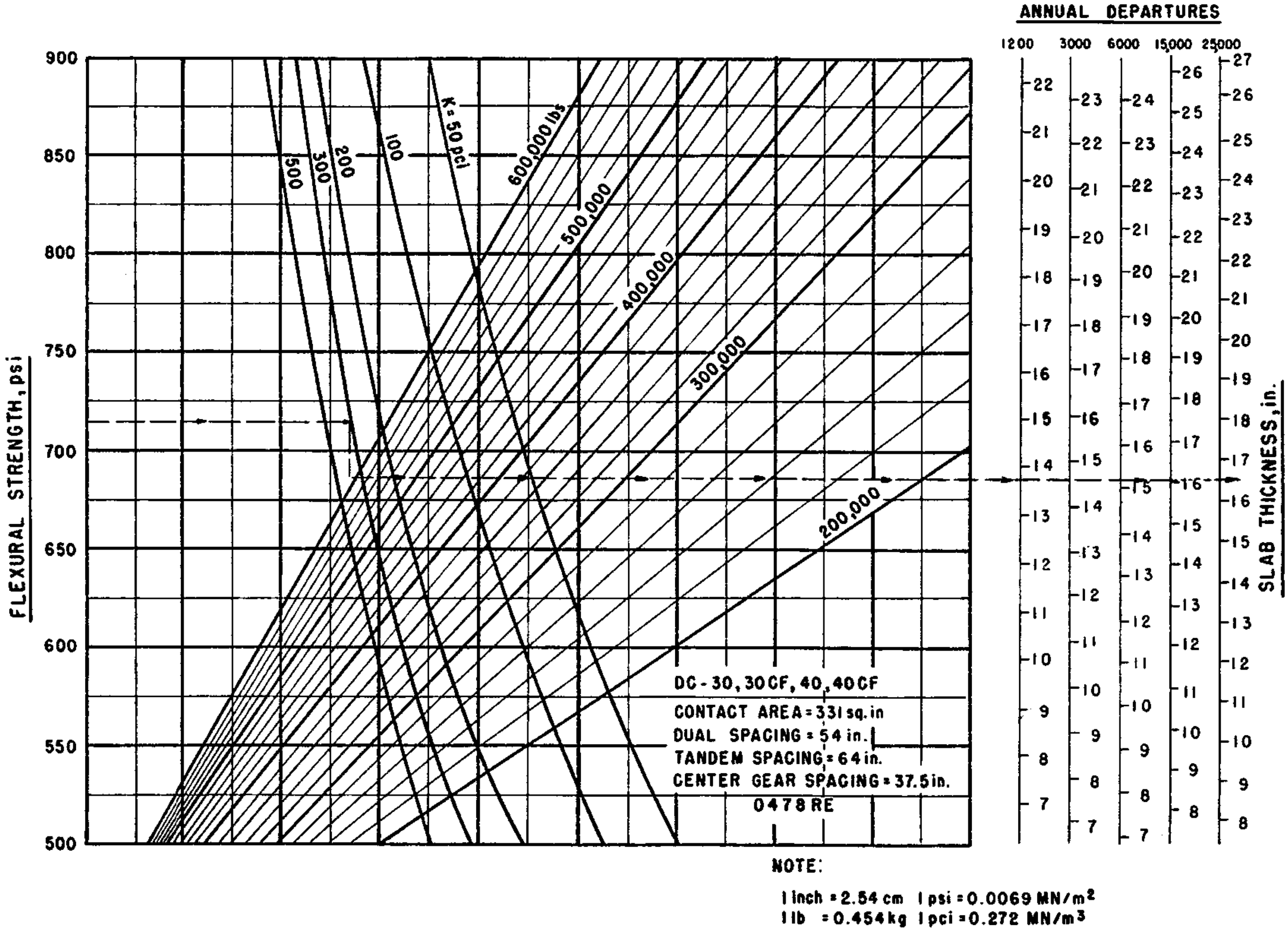
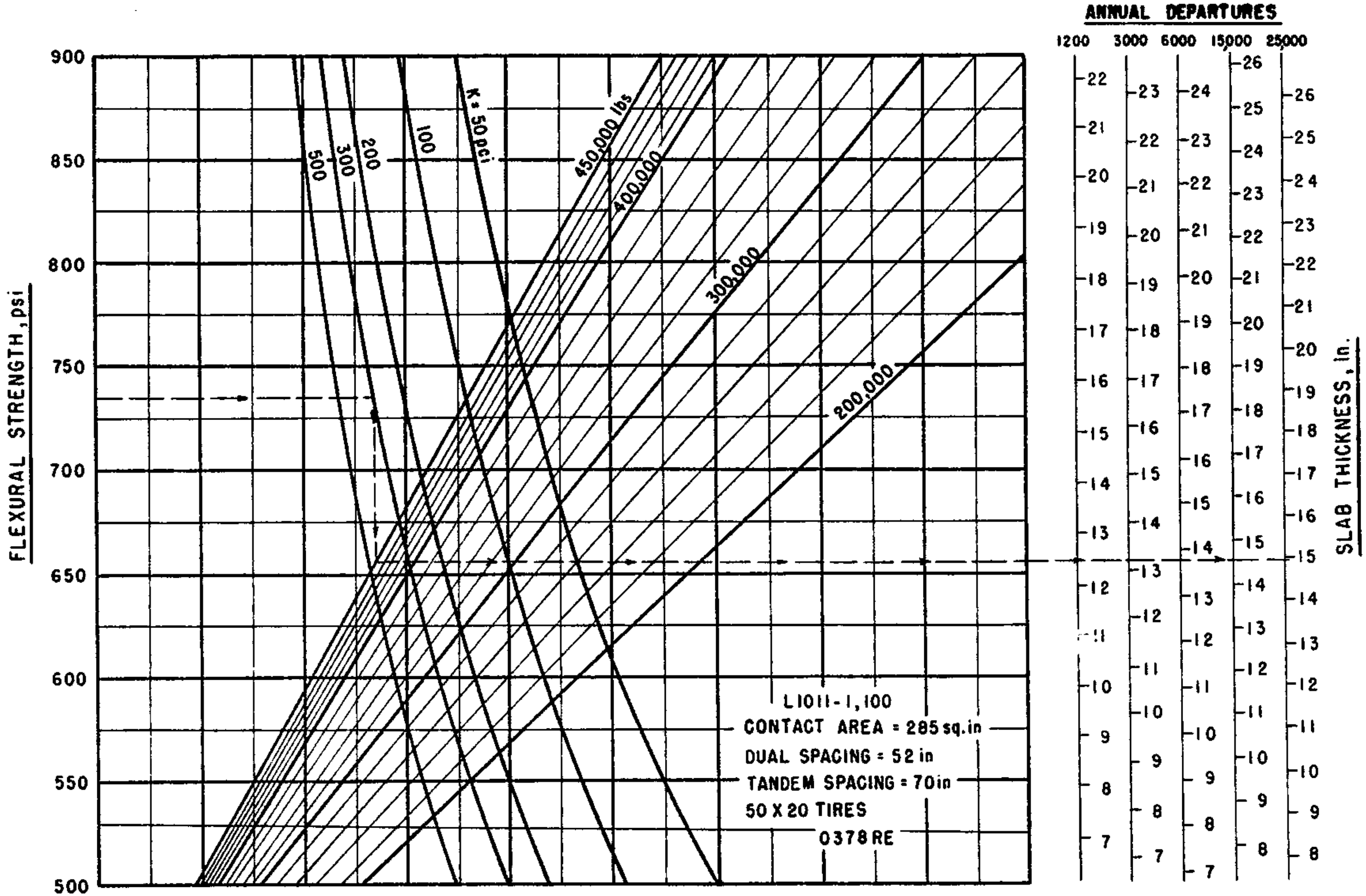


FIGURE 3-27. OPTIONAL RIGID PAVEMENT DESIGN CURVES - DC 10-30, 30CF, 40, 40CF

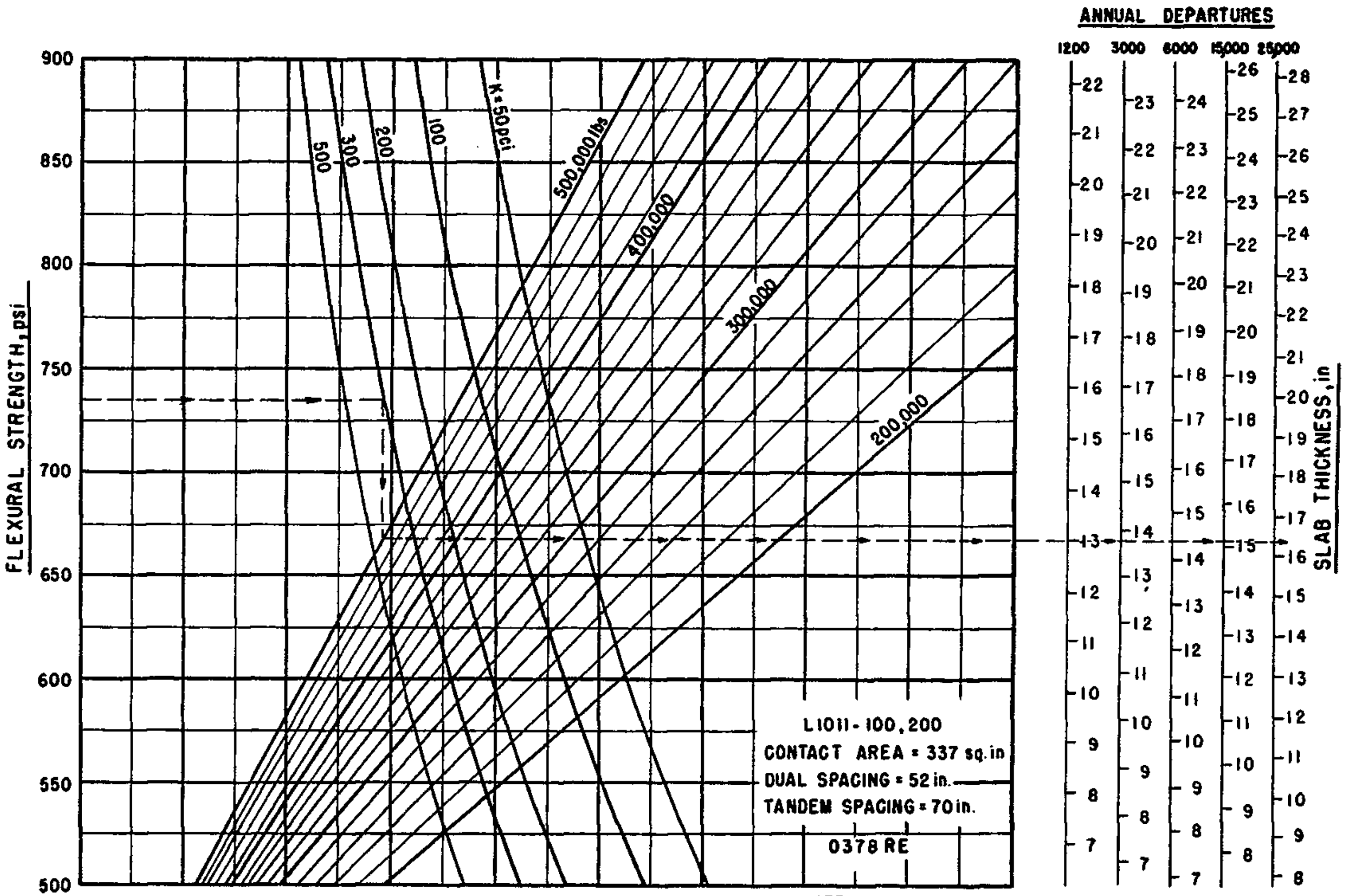


NOTE:

1 inch = 2.54 cm 1 psi = 0.0069 MN/m²

1 lb = 0.454 kg 1 pci = 0.272 MN/m³

FIGURE 3-28. OPTIONAL RIGID PAVEMENT DESIGN CURVES - L-1011-1, 100



NOTE:
1 inch = 2.54 cm   1 psi = 0.0069 MN/m²
1 lb = 0.454 kg   1 pci = 0.272 MN/m³

FIGURE 3-29. OPTIONAL RIGID PAVEMENT DESIGN CURVES - L-1011-100,200 to be considered in the establishment of jointing requirements and design of drainage structures. Other stabilized subbase thicknesses should be tried to determine the most economical section.

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

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

b. Thickness. Pavements subjected to traffic intensities greater than the 25,000 annual departure level shown on the design curves will require more thickness to accommodate the traffic volume. Additional thickness can be provided by increasing the pavement thickness in accordance with Table 3-5 shown below:

TABLE 3-5. PAVEMENT THICKNESS FOR HIGH DEPARTURE LEVEL EXPRESSED AS A PERCENT OF THE 25,000 DEPARTURE THICKNESS

Annual Departure Level	Percent of 25,000 Departure Thickness
50,000	104
100,000	108
150,000	110
200,000	112

The values given in Table 3-5 are based on extrapolations of research data and observations of in-service pavements. Table 3-5 was developed assuming a logarithmic relationship between percent of thickness and departures.

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

50. JOINTING OF CONCRETE PAVEMENTS. Variations in temperature and moisture content can cause volume changes and slab warping and produce stresses of significant magnitude. In order to reduce the detrimental effects of these stresses and to minimize random cracking, it is necessary to divide the pavement into a series of slabs of predetermined dimensions by means of joints. These slabs should be as nearly square as possible when no reinforcement is used.

a. Joint Categories. Pavement joints are categorized according to the function which the joint is intended to perform. The categories are expansion, contraction, and construction joints. All joints, regardless of type, should be finished in a manner which permits the joint to be sealed. Pavement joint details are shown in Figures 3-30 and 3-31 and are summarized in Table 3-6. These various joints are described as follows:

(1) Expansion Joints. The function of an expansion joint is to isolate intersecting pavements and to isolate structures from the pavement. There are two types of expansion joints.

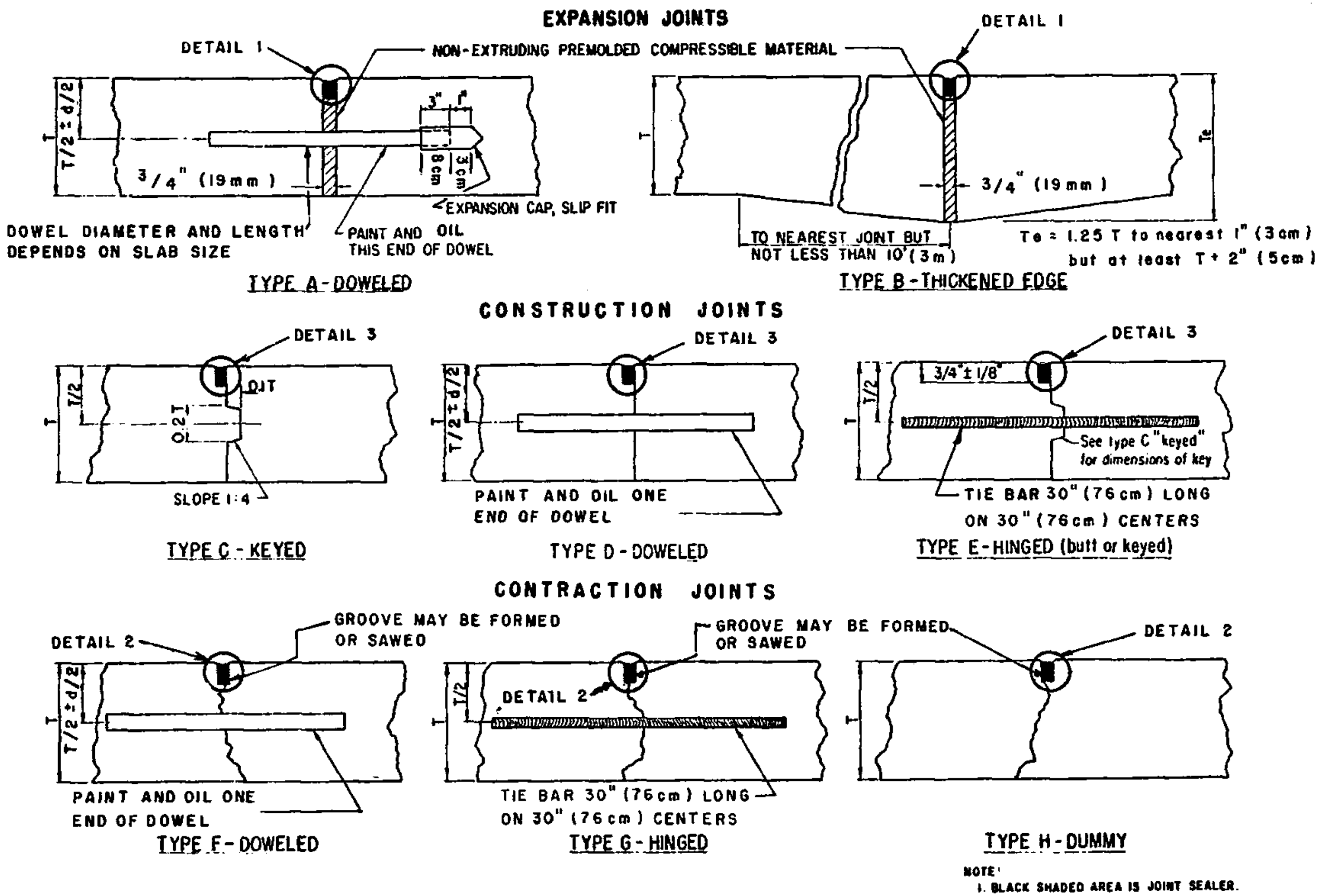
(a) Type A is used when load transfer across the joint is required. This joint contains a 3/4-inch (2 cm) nonextruding compressible material and is provided with dowel bars for load transfer.

(b) Type B is used when conditions preclude the use of a load transfer device which spans across the joint,such as where the pavement abuts a structure or where horizontal differences in movement of the pavements may occur. These joints are formed by increasing the thickness of the pavement along the edge of slab. No dowel bars are provided.

(2) Contraction Joints. The function of contraction joints is to provide controlled cracking of the pavement when the pavement contracts due to decrease in moisture content, or a temperature drop. Contraction joints also decrease stresses caused by slab warping. Details for contraction joints are shown as Types F, G, and H, in Figure 3-30.

TABLE 3-6. JOINT TYPES - DESCRIPTION AND USE

TYPE	DESCRIPTION	LONGITUDINAL	TRANSVERSE
A	Doweled expansion joint.		Use near intersections to isolate different pavement areas.
B	Thickened edge expansion joint.	Use at intersections where dowels are not suitable and where pavements abut structures.	Provide thickened edge (or keyway) where pavement extension is likely.
C or D	Keyed or doweled construction joint.	Use for all construction joints except where Type E is used.	Use Type D where paving operations are delayed or stopped.
E	Hinged construction joint.	Use for all contraction joints of the taxiway and for all other contraction joints placed 25' (8 m) or less from the pavement edge unless wide body aircraft are expected. See paragraph 51b for wide body aircraft requirements.	
F	Doweled contraction joint.		Use for contraction joints for a distance of at least 3 joints from a free edge, for the first two joints on each side of expansion joints and for all contraction joints in reinforced pavements.
G	Hinged contraction joint.	Use for all contraction joints of the taxiway and for all other contraction joints placed 25' (8 m) or less from the pavement edge unless wide body aircraft are expected. See paragraph 51b for wide body aircraft requirements.	
H	Dummy contraction joint.	Use for all other contraction joints in pavement.	Use for all remaining contraction joints in nonreinforced pavements.


EXPANSION JOINTS
DETAIL 1
NON-EXTRUDING PREMOLDED COMPRESSIBLE MATERIAL
T/2 ± d/2
T
3/4" (19mm)
3" 1"
8cm 3cm
EXPANSION CAP, SLIP FIT
DOWEL DIAMETER AND LENGTH DEPENDS ON SLAB SIZE
PAINT AND OIL THIS END OF DOWEL
TYPE A-DOWELED
DETAIL 1
Te
3/4" (19mm)
TO NEAREST JOINT BUT NOT LESS THAN 10' (3m)
Te = 1.25 T to nearest 1" (3cm) but at least T + 2" (5cm)
TYPE B-THICKENED EDGE
CONSTRUCTION JOINTS
DETAIL 3
T/2
0.1T
0.2T
SLOPE 1:4
TYPE C-KEYED
DETAIL 3
T/2 ± d/2
PAINT AND OIL ONE END OF DOWEL
TYPE D-DOWELED
DETAIL 3
3/4" ± 1/8"
See type C "keyed" for dimensions of key
TIE BAR 30" (76cm) LONG ON 30" (76cm) CENTERS
TYPE E-HINGED (butt or keyed)
CONTRACTION JOINTS
DETAIL 2
GROOVE MAY BE FORMED OR SAWED
T/2 ± d/2
PAINT AND OIL ONE END OF DOWEL
TYPE F-DOWELED
GROOVE MAY BE FORMED OR SAWED
DETAIL 2
TIE BAR 30" (76cm) LONG ON 30" (76cm) CENTERS
TYPE G-HINGED
DETAIL 2
TYPE H-DUMMY
NOTE:
1. BLACK SHADED AREA IS JOINT SEALER.

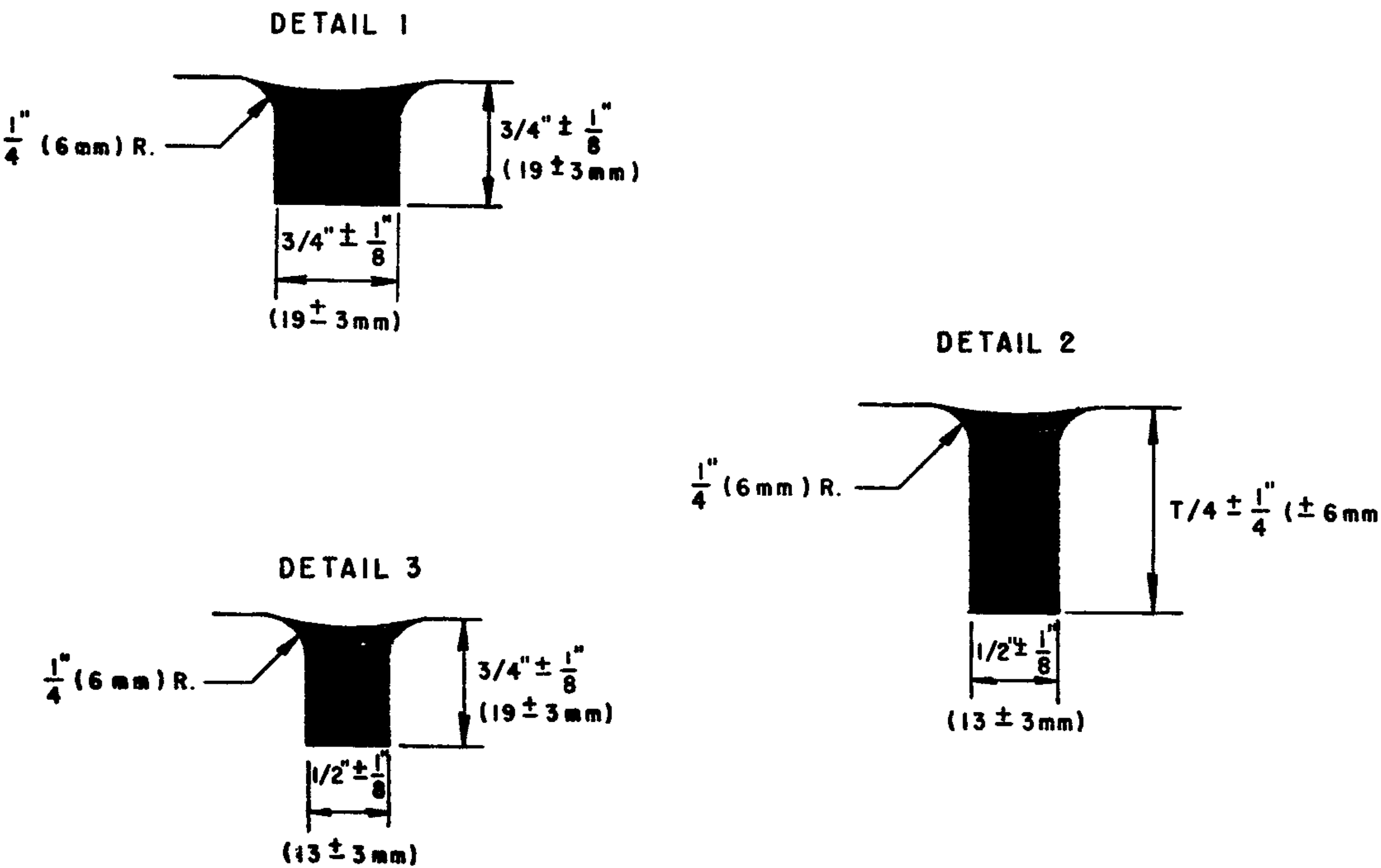
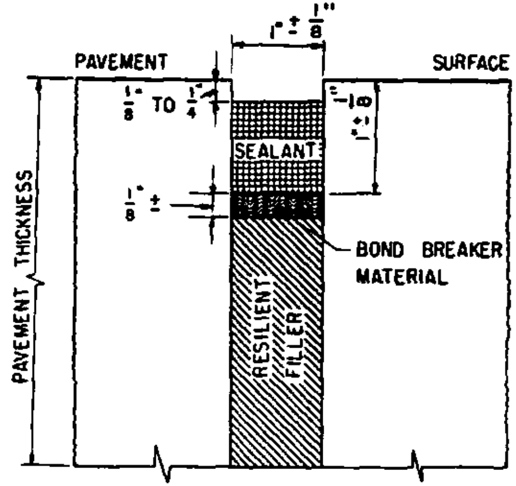
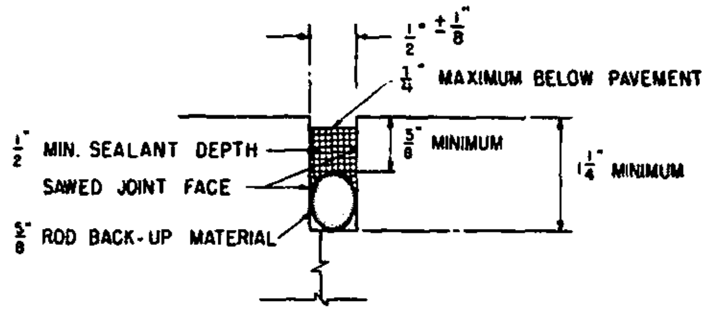


FIGURE 3-30. DETAILS OF JOINTS IN RIGID PAVEMENTS



EXPANSION JOINTS TYPE A AND B



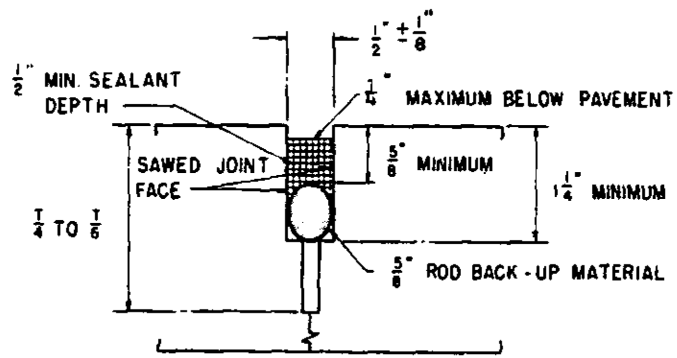
CONSTRUCTION JOINT TYPE C, D AND E

SYMBOLS

- SEALANT
- ROD BACK-UP MATERIAL 1/
- BOND BREAKER MATERIAL
- RESILIENT FILLER

1/ CLOSED-CELL RESILIENT FOAM OR SPONGE RUBBER

NOTE:
1 in. = 2.54 cm



CONTRACTION JOINT TYPE F, G AND H

FIGURE 3-31. DETAILS OF JOINTS IN RIGID PAVEMENTS FOR USE WITH TYPE VI SEALING MATERIAL

(3) Construction Joints. Construction joints are required when two abutting slabs are placed at different times such as at the end of a day's placement, or between paving lanes. Details for construction joints are shown as Types C, D, and E in Figure 3-30.

b. Joint Spacing. A rule-of-thumb for joint spacing given by the Portland Cement Association is applicable: "As a rough guide, the joint spacing (in feet) should not greatly exceed twice the slab thickness (in inches)." Table 3-7 shows the recommended maximum joint spacings. Shorter spacings may be more convenient in some instances. The ratio of slab length to slab width should not exceed 1.25 in unreinforced pavements.

TABLE 3-7. RECOMMENDED MAXIMUM JOINT SPACINGS

Slab Thickness Inches	Transverse	Longitudinal
less than 9 (23 cm)	15 feet (4.6 m)	12.5 feet (3.8 m)
9 to 12 (23 to 31 cm)	20 feet (6.1 m)	20 feet (6.1 m)
greater than 12 (31 cm)	25 feet (7.6 m)	25 feet (7.6 m)

51. SPECIAL JOINTING CONSIDERATIONS. A number of special considerations are required when designing the jointing system for a portland cement concrete pavement. Several considerations are discussed below.

a. Keyed Joints. Keyed construction joints should not be used for slabs less than 9 inches (23 cm) in thickness. Keyed joints in slabs of lesser thickness result in very small key-ways with limited strength. In some instances the key and key-way may be so small that the coarse aggregate cannot enter the key or key-way and segregation of material can result.

b. Jointing Systems for Wide Body Jet Aircraft. Experience gained through full-scale test track pavements indicates poor performance can be expected from keyed longitudinal construction joints supported on low-strength foundations when wide body aircraft loadings are encountered. Special jointing designs are required for different subgrade strengths as discussed below.

(1) For foundation moduli of 200 pci (54 MN/m^3) or less, a doweled or thickened edge construction joint, Type D or B, is recommended. Keyed joints should not be used as poor performance will likely result. In areas of low traffic usage, such as extreme outer lanes of runways and aprons, keyed joints, Type C, may be used.

(2) For foundation moduli between 200 pci (54 MN/m^3) and 400 pci (109 MN/m^3) hinged construction joints, Type E, may be used as well as doweled or thickened edge. The maximum width of pavement which can be tied together depends on several factors such as subgrade frictional restraints, pavement thickness, and climatic conditions. Normally, the maximum width of tied pavement should not exceed 75 feet (23 m). Type C joints may be used in low traffic areas.

(3) For foundation moduli of 400 pci (109 MN/m^3) or greater conventional keyed joints, Type C, may be used regardless of traffic usage. The designer is reminded, however, that the prohibition against keyed joints in pavements less than 9 inches (23 cm) thick shall still remain in effect.

c. Future Expansion. When a runway or taxiway is likely to be extended at some future date, it is recommended that a thickened edge joint be provided at that end of the runway or taxiway. Likewise, if any pavement is to be widened in the future, a key-way or thickened edge should be provided at the appropriate edge.

52. JOINTING STEEL.

a. Tie Bars. Tie bars are used across certain longitudinal contraction joints and keyed construction joints to hold the slab faces in close contact. The tie bars themselves do not act as load transfer devices. By preventing wide opening of the joint, load transfer is provided by the keyed joint or by aggregate interlock in the crack below the groove-type joint. Tie bars should be deformed bars conforming to the specifications given in Item P-501. The bars should be 5/8 inch (16 mm) in diameter and 30 inches (76 cm) long and spaced 30 inches (76 cm) on center.

b. Dowels. Dowels are used at joints to provide for transfer of load across the joint and to prevent relative vertical displacement of adjacent slab ends. Dowels permit longitudinal movement of adjacent slabs.

(1) Where Used. Provision for load transfer by dowels is provided at all transverse expansion joints and all butt-type construction joints. Dowels for contraction joints should be provided at least three joints from a free edge. Contraction joints in the interior of the pavement may be the dummy groove type.

(2) Size Length and Spacing. Dowels should be sized such that they will resist the shearing and bending stresses produced by the loads on the pavement. They should be of such length and spacing that the bearing stresses exerted on the concrete will not cause failure of the concrete slab. Table 3-8 indicates the dowel dimensions and spacing for various pavement thicknesses.

TABLE 3-8. DIMENSIONS AND SPACING OF STEEL DOWELS

Thickness of Slab	Diameter	Length	Spacing
6-7 in (15-18 cm)	3/4 in (20 mm)	18 in (46 cm)	12 in (31 cm)
8-12 in (21-31 cm)	1 in (25 mm)	19 in (46 cm)	12 in (31 cm)
13-16 in (33-41 cm)	1 1/4 in 1/ (30 mm)	20 in (51 cm)	15 in (38 cm)
17-20 in (43-51 cm)	1 1/2 in 1/ (40 mm)	20 in (51 cm)	18 in (46 cm)
21-24 in (54-61 cm)	2 in 1/ (50 mm)	24 in (61 cm)	18 in (46 cm)

1/Dowels noted may be a solid bar or high-strength pipe. High-strength pipe dowels must be plugged on each end with a tight fitting plastic cap or with bituminous or mortar mix.

(3) Dowel Positioning. The alignment and elevation of dowels is extremely important in obtaining a satisfactory joint. Transverse dowels will require the use of a fixture, usually a wire cage or basket firmly anchored to the subbase, to hold the dowels in position. During the concrete placement operation, it is advisable to place plastic concrete directly on the dowel assembly immediately prior to passage of the paver to prevent displacement of the assembly by the paving equipment. Some paving machines have a dowel placer which can also be used to accurately position dowels. The recommended tolerance for dowel alignment in either the horizontal or vertical plane is 2% or 1/4 inch per foot (21 mm per meter).

53. JOINT SEALANTS AND FILLERS. Sealants are used in all joints to prevent the ingress of water and foreign material in the joint. Premolded compressible fillers are used in expansion joints to permit expansion of the slabs. Joint sealants are applied above the filler in expansion joints to prevent infiltration of water and foreign material. In areas subject to fuel spillage, fuel resistant sealants should be used. Specifications for joint sealants are given in Item P-605.

54. JOINT LAYOUT NEAR PAVEMENT INTERSECTIONS.

a. Isolation Joints. Two intersecting pavements such as a taxiway and runway should be isolated to allow the pavements to move independently. Isolation can best be accomplished by using a Type B expansion joint between the two pavements. The expansion joint should be positioned such that the two pavements can expand and contract independently; normally this can be accomplished by using a Type B expansion joint where the two pavements abut. One isolation joint is normally sufficient to allow independent movement.

b. Odd-Shaped Slabs. Cracks tend to form in odd-shaped slabs; therefore, it is normally good practice to maintain sections which are nearly square or rectangular in shape. Pavement intersections which involve fillets are difficult to design with a few odd-shaped slabs. In instances where odd-shaped slabs cannot be avoided, steel reinforcement is recommended. Steel reinforcement should consist of 0.05% steel in both directions in slabs where the length-to-width ratio exceeds 1.25 or in slabs which are not rectangular in shape. The steel reinforcement should be placed in accordance with the recommendations given in Paragraph 55, Reinforced Concrete Pavement. Fillets may be defined by constructing slabs to the normal, full dimensions and painting out the unused portion of the slab with bitumen.

55. REINFORCED CONCRETE PAVEMENT. The main benefit of steel reinforcing is that, although it does not prevent cracking, it keeps the cracks that form tightly closed so that the interlock of the irregular faces provides structural integrity and usually improves pavement performance. By holding the cracks tightly closed, the steel minimizes the infiltration of debris into the cracks. The thickness requirements for reinforced concrete pavements are the same as plain concrete and are determined from the appropriate design curves, Figures 3-14 through 3-29. Steel reinforcement allows longer joint spacings, thus the cost benefits associated with fewer joints must be determined in the decision to use plain or reinforced concrete pavement.

56. TYPE AND SPACING OF REINFORCEMENT. Reinforcement may be either welded wire fabric or bar mats installed with end and side laps to provide complete reinforcement throughout the slab panel. End laps should be a minimum of 12 inches (31 cm) but not less than 30 times the diameter of the longitudinal wire or bar. Side laps should be a minimum of 6 inches (15 cm) but not less than 20 times the diameter of the transverse wire or bar. End and side clearances should be a maximum of 6 inches (15 cm) and a minimum of 2 inches (5 cm) to allow for nearly complete reinforcement and yet achieve adequate concrete cover. Longitudinal members should be spaced not less than 4 inches (10 cm) nor more than 12 inches (31 cm) apart; transverse members should be spaced not less than 4 inches (10 cm) nor more than 24 inches (61 cm) apart.

57. AMOUNT OF REINFORCEMENT.

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

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

where:

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

L = length of width of slab, feet

t = thickness of slab, inches

f_s = allowable tensile stress in steel, psi

NOTE: To determine the area of steel in metric units:

L should be expressed in meters

t should be expressed in millimeters

f_s should be expressed in mega newtons per square meter

The constant 3.7 should be changed to 0.64

A_s will then be in terms of square centimeters per meter

b. In this formula the slab weight is assumed to be 12.5 pounds per square foot, per inch of thickness (23.6 MN/m^2). The allowable tensile stress in steel will vary with the type and grade of steel. It is recommended that allowable tensile stress be taken as two-thirds of the yield strength of the steel. Based on current specifications the yield strengths and corresponding design stresses (f_s) are as listed in Table 3-9.

TABLE 3-9. YIELD STRENGTHS OF VARIOUS GRADES OF REINFORCING STEEL

ASTM Designation	Type & Grade of Steel	Yield Strength psi (MN/m^2)	f_s psi	f_s (MN/m^2)
A 615	Deformed Billet steel grade 40	40,000 (300)	27,000	(200)
A 616	Deformed Rail steel, grade 50	50,000 (370)	33,000	(240)
A 616	Deformed Rail steel, grade 60	60,000 (440)	40,000	(300)
A 615	Deformed Billet steel, grade 60	60,000 (440)	40,000	(300)
A 185	Cold drawn welded steel wire fabric	65,000 (480)	43,000	(320)
A 497	Cold drawn welded deformed steel wire	70,000 (520)	47,000	(350)

c. The minimum percentage of steel reinforcement should be 0.05%. The percentage of steel is computed by dividing the area of steel, A_s, by the area of concrete per unit of length (or width) and multiplying by 100. The minimum percentage of steel considered the least amount of steel which can be economically placed is 0.05%. Steel reinforcement allows larger slab sizes and thus decreases the number of transverse contraction joints. The costs associated with providing a reinforced pavement must be compared with the savings realized in eliminating some of the transverse contraction joints to determine the most economical steel percentage. The maximum allowable slab length regardless of steel percentage is 75 feet (23 m).

58. DIMENSIONS AND WEIGHTS OF REINFORCEMENT. Dimensions and unit weights of standard deformed reinforcing bars are given in Table 3-10, and wire size number, diameters, areas, and weights of wires used in welded wire fabric are given in Table 3-11.

TABLE 3-10. DIMENSIONS AND UNIT WEIGHTS OF DEFORMED STEEL REINFORCING BARS

	NOMINAL DIMENSIONS							
	Diameter		Area		Perimeter		Unit Weight,	
Number	in.	(mm)	sq. in.	(cm^2)	in.	(cm)	lbs. per ft.	kg/m
3	0.375	(9.5)	0.11	(0.71)	1.178	(3.0)	0.376	(0.56)
4	0.500	(12.7)	0.20	(1.29)	1.571	(4.0)	0.668	(1.00)
5	0.625	(15.9)	0.31	(2.00)	1.963	(5.0)	1.043	(1.57)
6	0.750	(19.1)	0.44	(2.84)	2.356	(6.0)	1.502	(2.26)
7	0.875	(22.2)	0.60	(3.86)	2.749	(7.0)	2.044	(3.07)

TABLE 3-11. SECTIONAL AREAS OF WELDED FABRIC

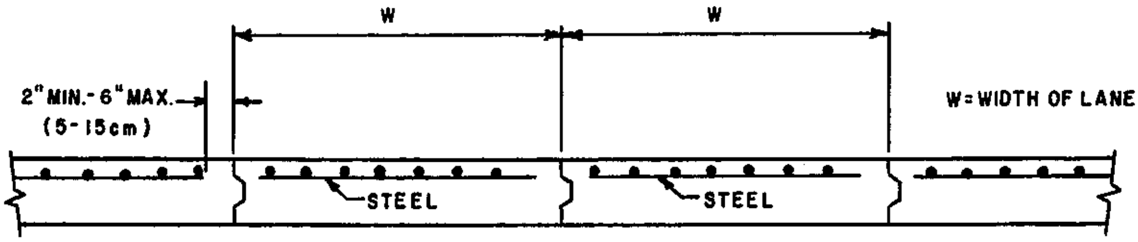
Wire Size Number Smooth	Wire Size Number Deformed	Nominal Diameter, Inches	Nominal Weight Lbs./Lin. Ft.	Center-to-Center Spacing 4"	6"	8"	10"	12"
W31	D31	0.628	1.054	.93	.62	.465	.372	.31
W30	D30	0.618	1.020	.90	.60	.45	.36	.30
W28	D28	0.597	.952	.84	.56	.42	.336	.28
W26	D26	0.575	.934	.78	.52	.39	.312	.26
W24	D24	0.553	.816	.72	.48	.36	.288	.24
W22	D22	0.529	.748	.66	.44	.33	.264	.22
W20	D20	0.504	.680	.60	.40	.30	.24	.20
W18	D18	0.478	.612	.54	.36	.27	.216	.18
W16	D16	0.451	.544	.48	.32	.24	.192	.16
W14	D14	0.422	.476	.42	.28	.21	.168	.14
W12	D12	0.390	.408	.36	.24	.18	.144	.12
W11	D11	0.374	.374	.33	.22	.165	.132	.11
W10.5		0.366	.357	.315	.21	.157	.126	.105
W10	D10	0.356	.340	.30	.20	.15	.12	.10
W9.5		0.348	.323	.285	.19	.142	.114	.095
W9	D9	0.338	.306	.27	.18	.135	.108	.09
W8.5		0.329	.289	.255	.17	.127	.102	.085
W8	D8	0.319	.272	.24	.16	.12	.096	.08
W7.5		0.309	.255	.225	.15	.112	.09	.075
W7	D7	0.298	.238	.21	.14	.105	.084	.07
W6.5		0.288	.221	.195	.13	.097	.078	.065
W6	D6	0.276	.204	.18	.12	.09	.072	.06
W5.5		0.264	.187	.165	.11	.082	.066	.055
W5	D5	0.252	.170	.15	.10	.075	.06	.05
W4.5		0.240	.153	.135	.09	.067	.054	.045
W4	D4	0.225	.136	.12	.08	.06	.048	.04

Note: 1 inch = 2.54 cm
1 lb./lin. ft. = 1.5 kg/m

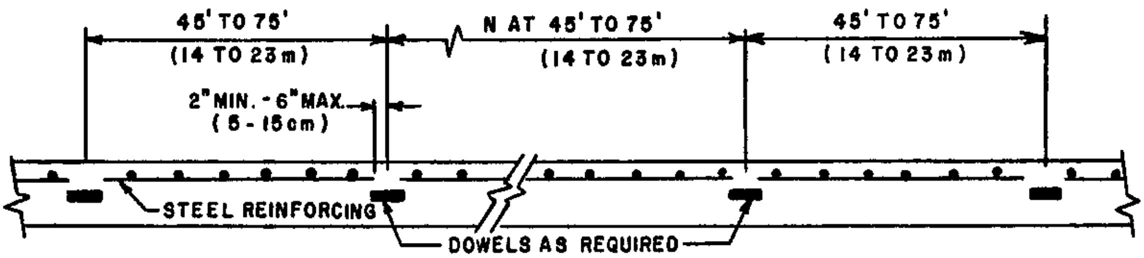
59. WELDED WIRE FABRIC. The use of welded wire fabric requires some special design considerations to achieve the most economical design. The use of smooth welded wire fabric or deformed welded wire fabric is the option of the designer. The choice should be based on the difference in allowable design stresses, the availability of the desired sizes (smooth wire fabric is available in a wider range of sizes), and the costs associated with each style of fabric. It is recommended that the minimum size of longitudinal wire be W5 or D5. The minimum transverse wire should be no smaller than W4 or D4. In addition, should calculated area of longitudinal steel be less than 0.05 percent of the cross-sectional area of slab, the size and spacing of the steel members (bars or wire) should be determined on the premise that the minimum area should not be less than 0.05 percent. This percentage applies in the case of steel having a yield strength of 65,000 psi (480 MN/m^2). If lower grades are used, the percentage should be revised proportionately upward. For example, Table 3-11 shows that W10 wires, spaced 10 inches (25 cm) apart, furnish an area of 0.12 square inches (0.77 cm^2) which satisfies the requirement for pavements up to 20 inches (51 cm) thick. Sizing of individual sheets of welded wire fabric is also important in providing an economical design. Not all fabricators supply all wire sizes in all spacings. While nearly any fabric style can be produced on special order, it is generally more economical to specify a standard production configuration. Sheet and roll widths in excess of 8 feet (2.5 m) can result in higher shipping costs.

60. JOINTING OF REINFORCED PAVEMENTS. Contraction joints in reinforced pavements may be spaced up to 75 feet (23 m) apart, and all joints should be provided with load transfer devices as shown in Figure 3- 32. Also, this figure presents other reinforcement details such as clearance at joints and edges of pavement and depth below the surface. The longer joint spacing allowed with reinforced pavements will result in larger joint openings. The joints must be sealed extremely carefully to accommodate the larger movements at the joints.

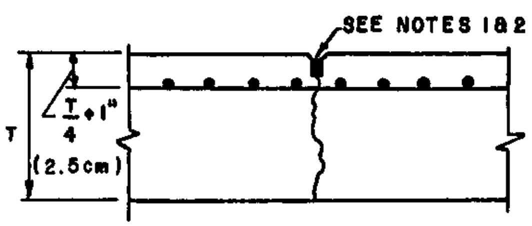
61. CONTINUOUSLY REINFORCED CONCRETE PAVEMENT. Continuously reinforced concrete pavement usually contains 0.5 to 1.0 percent steel reinforcing in the longitudinal direction and is constructed without transverse contraction joints. As with conventionally reinforced concrete pavement, the pavement is expected to crack when continuously reinforced, and the reinforcing steel is to hold the cracks tightly closed. Designs using continuously reinforced concrete must be approved on a case-to-case basis. The same thickness is required for continuously reinforced concrete pavement as that required for nonreinforced concrete pavement. Detailed design guidance can be found in FAA Report No. FAA-RD-73, Volume III, Design Manual for Continuously Reinforced Concrete Pavements. See Appendix 4.



TRANSVERSE CROSS SECTION OF PAVING LANES



LONGITUDINAL CROSS SECTION



LONGITUDINAL CONTRACTION JOINT
(SEE NOTE 3)

NOTES:

1. SEE FIGURES 3-30 & 3-31 FOR GROOVE DETAILS
2. JOINT DETAILS ARE SIMILAR TO FIGURES 3-30 & 3-31 EXCEPT FOR STEEL REINFORCING.
3. USE THIS JOINT WHEN THE SLAB THICKNESS IS 10 INCHES (25cm) OR LESS AND PAVING EXCEEDS $12\frac{1}{2}$ FEET (4m).

FIGURE 3-32. JOINTING OF REINFORCED RIGID PAVEMENTS

CHAPTER 4. AIRPORT PAVEMENT OVERLAYS.

62. GENERAL.

a. Airport pavement overlays may be required for a variety of reasons A pavement may have been damaged by overloading in such a way that it cannot be maintained satisfactorily at a serviceable level. Similarly, a pavement in good condition may require strengthening to serve heavier aircraft than those for which the pavement was originally designed. A pavement may also require an overlay simply because the original pavement has served its design life and is "worn out." Generally, airport pavement overlays consist of either portland cement concrete or bituminous concrete.

b. Definitions applicable to overlay pavements are as follows:

(1) Overlay pavement. Pavement which is constructed on top of an existing pavement.

(2) Bituminous overlay. Bituminous concrete pavement placed on an existing pavement.

(3) Concrete overlay. Portland cement concrete pavement placed on an existing pavement.

(4) Sandwich Pavement. An overlay pavement containing a granular separation course.

Typical overlay pavement cross sections are shown in Figure 4-1.

63. PRELIMINARY DESIGN DATA. Regardless of the type of overlay to be employed, several determinations should be made prior to the actual design. The following items will provide this essential information:

a. Determine the foundation conditions under the existing pavement. This determination should include the soil classification, drainage conditions, and some estimate of foundation strength (CBR or subgrade modulus).

b. Determine the actual thickness of each layer of the existing pavement, its condition and strength.

c. In accordance with the requirements for the particular type of overlay, determine the pavement thickness required for the loading under consideration by using the appropriate basic pavement design curves included in Chapter 3.

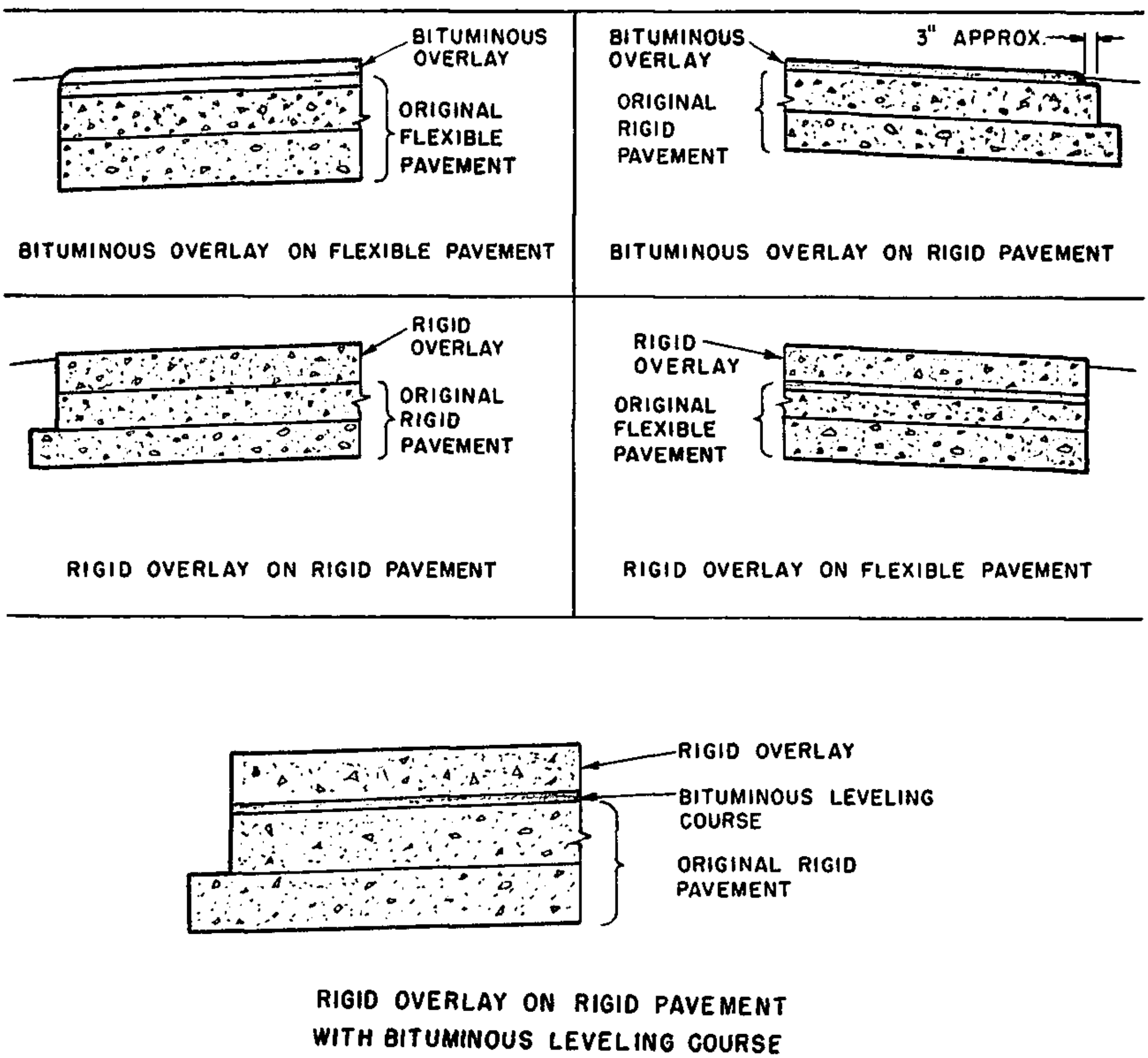


FIGURE 4-1. TYPICAL OVERLAY PAVEMENTS

d. Failed areas in the existing pavement should be carefully studied to determine the probable cause of failure.In some instances reconstruction or subsurface drainage is needed.

e. Techniques and equipment are now available to recycle old pavement materials and incorporate the material in a reconstructed section. Pavements which are severely distressed in the center portions can sometimes be economically reconstructed by building a keel section using recycled materials. Use of this method of reconstruction is essentially the same as building a new pavement.

64. MATERIAL SELECTION CONSIDERATIONS. Criteria are presented in this chapter for both bituminous and concrete overlay pavements. The selection of the overlay type should be made after careful consideration of many factors. The designer should consider the total life cycle cost of the overlay pavement. Life cycle costs should include initial construction and maintenance costs over the design life of the pavement. Other considerations such as allowable down time of the pavement and availability of alternate pavements to use during construction will have a significant impact on the overlay type selected.

65. DESIGN OF BITUMINOUS OVERLAYS. Bituminous overlays can be applied to either flexible or rigid pavements. Certain criteria are applicable to the design of bituminous overlays whether they are to be placed over existing rigid or flexible pavements.

a. Overlay pavements which use a granular separation course between the old and new surfaces are not allowed. Overlay pavements containing granular separation courses are referred to as sandwich pavements. Sandwich pavements are not allowed because the separation course is likely to become saturated with water and provide rather unpredictable performance. Saturation of the separation course can be caused by the infiltration of surface water, ingress of ground or capillary water, or the condensation of water from the atmosphere. In any event, the water in the separation course usually cannot be adequately drained and drastically reduces the stability of the overlay.

b. Bituminous overlays for increasing strength should have a minimum thickness of 3 inches (7.5 cm).

66. BITUMINOUS OVERLAYS ON EXISTING FLEXIBLE PAVEMENT.

a. Use the appropriate basic flexible pavement curves (Figures 3-3 through 3-11) to determine the thickness requirements for a flexible pavement for the desired load and number of equivalent design departures. A CBR value is required for the subgrade material and subbase. Thicknesses of all pavement layers must be determined. The thickness of pavement required over the subgrade and subbase and the minimum base course requirements must be compared with the existing pavement to determine the overlay requirements.

b. Adjustments to the various layers of the existing pavement may be necessary to complete the design. Bituminous surfacing may have to be converted to base, and base to subbase conversion may be required. A high quality material may be converted to a lower quality material, such as surfacing to base. A material may not be converted to a higher quality material. For example excess subbase cannot be converted to base. The equivalency factors shown in Tables 3-2 and 3-3 may be used as guidance in the conversion of layers. It must be recognized that the values shown in Tables 3-2 and 3-3 are for new materials and the assignment of factors for existing pavements must be based on judgment and experience. Surface cracking, high degree of oxidation, evidence of low stability, etc., are only a few of the considerations which would tend to reduce the equivalency factor. Any bituminous layer located between granular courses in the existing pavement should be evaluated inch for inch as granular base or subbase course.

c. To illustrate the procedure of designing a bituminous overlay, assume an existing taxiway pavement composed of the following section. The subgrade CBR is 7, the bituminous surface course is 4 inches (10 cm) thick, the base course is 6 inches (15 cm) thick, the subbase is 10 inches (25 cm) thick, and the subbase CBR is 15. Frost action is negligible. Assume the existing pavement is to be strengthened to accommodate a dual wheel aircraft weighing 100,000 pounds (45 000 kg) and an annual departure level of 3,000. The flexible pavement required (referring to Figure 3-4) for these conditions is·

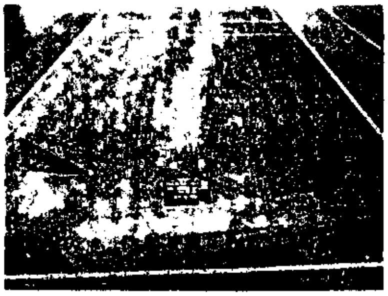
Bituminous Surface	4 inches	(10 cm)
Base	9 inches	(23 cm)
Subbase	10 inches	(25 cm)
Total pavement thickness	23 inches	(58 cm)

The total pavement thickness must be 23 inches (58 cm) in order to protect the CBR 7 subgrade. The combined thicknesses of surfacing and base must be 13 inches (33 cm) to protect the CBR 15 subbase. The existing pavement is thus 3 inches (7.5 cm) deficient in total pavement thickness, all of which is due to base course. For the sake of illustration, assume the existing bituminous surface is in such a condition that surfacing can be substituted for base at an equivalency ratio of 1.3 to 1. Converting 2.5 inches (6 cm) of surfacing to base yields a base course thickness of 9.2 inches (23 cm) leaving 1.5 inches (4 cm) of unconverted surfacing. A 2.5 inch (6 cm) overlay would be required to achieve a 4 inch (10 cm) thick surface. In this instance the minimum 3-inch (7.5 cm) overlay thickness would control. A 3-inch (7.5 cm) overlay thickness would be required.

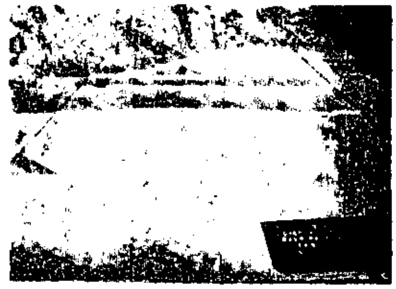
d. The most difficult part of designing bituminous overlays for flexible pavements is the determination of the CBR values for the subgrade and subbase and conversion of layers. Subgrade and subbase CBR values can best be determined by conducting field in-place CBR tests. Field CBR tests should be performed in accordance with the procedures given in Manual Series No. 10 (MS-10) by The Asphalt Institute. See Appendix 4. The subgrade and subbase must be at the equilibrium moisture content when field CBR tests are conducted. Normally a pavement which has been in place for at least 3 years will be in equilibrium. Layer conversions, i.e., converting base to subbase, etc., are largely a matter of engineering judgment. When performing the conversions, it is recommended that any converted thicknesses never be rounded off. To illustrate, if a converted thickness yields a value of 2.6 inches (7 cm), this value should not be rounded off to 3 inches (7.5 cm).

67. BITUMINOUS OVERLAY ON EXISTING RIGID PAVEMENT. To establish the required thickness of bituminous overlay for an existing rigid pavement, it is first necessary to determine the single thickness of rigid pavement required to satisfy the design conditions. This thickness is then modified by a factor "F" which controls the degree of cracking which will occur in the existing rigid pavement. The effective thickness of the existing rigid pavement is also adjusted by a condition factor "C_b." The "F" and "C_b" factors perform two different functions in the bituminous overlay determination as discussed below:

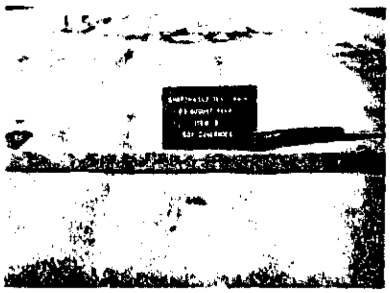
a. The factor "F" which controls the degree of cracking which will occur in the base pavement is a function of the amount of traffic and the subgrade strength. The "F" factor selected will dictate the final condition of the overlay and base pavement. The "F" factor in effect is indicating that the entire concrete single slab thickness determined from the design curves is not needed because a bituminous overlay pavement is allowed to crack and deflect more than a conventional rigid pavement. More cracking and deflection is allowable as the bituminous surfacing will not spall and can conform to greater deflections than a totally rigid pavement. Photographs of various overlay and base pavements shown in Figure 4-2 illustrate the meaning of the "F" factor. Figures 4-2a, b, and c show how the overlay and base pavements fail as more traffic is applied to a bituminous overlay on an existing rigid pavement. In



SURFACE OF OVERLAY



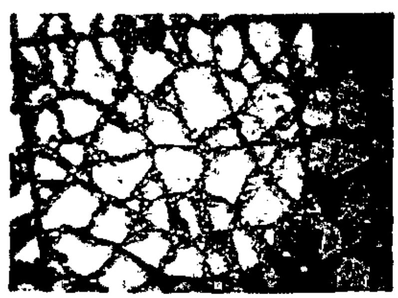
BASE PAVEMENT



SURFACE OF OVERLAY

BASE PAVEMENT

SURFACE OF OVERLAY



BASE PAVEMENT

FIGURE 4-2. ILLUSTRATION OF VARIOUS "F" FACTORS FOR BITUMINOUS OVERLAY DESIGN

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the design of a bituminous overlay, the condition of the overlay and base pavement after the design life should be close to that shown in Figure 4-2b. Figure 4-3 is a graph enabling the designer to select the appropriate "F" value to yield a final condition close to that shown in Figure 4-2b.

b. The condition factor "C_b" applies to the existing rigid pavement. The "C_b" factor is an assessment of the structural integrity of the existing pavement. The determination of the proper "C_b" value is a judgment decision for which only general guidelines can be provided. A "C_b" value of 1.0 should be used when the existing slabs contain nominal initial cracking and 0.75 when the slabs contain multiple cracking. The designer is cautioned that the range of "C_b" values used in bituminous overlay designs is different from the "C_r" values used in rigid overlay pavement design. The minimum "C_b" value is 0.75. A single "C_b" should be established for an entire area. The "C_b" value should not be varied along a pavement feature. Figures 4-4 and 4-5 illustrate "C_b" values of 1.0 and 0.75, respectively.

c. After the "F" factor, condition factor "C_b," and single thickness of rigid pavement have been established, the thickness of the bituminous overlay is computed from the following formula:

$$t = 2.5 (Fh - C_b h_e)$$

where t = thickness of bituminous overlay, inches

F = factor which controls the degree of cracking in the base pavement

h = single thickness of rigid pavement required for design conditions, inches. Use the exact value of h; do not round off.

C_b = condition factor for base pavement ranging from 1.0 to 0.75

h_e = thickness of existing rigid pavement, inches

Calculation of bituminous overlay thickness in metric units should be performed using the formula below:

$$t = 6.3 (Fh - C_b h_e)$$

where:

t is in centimeters
h is in centimeters
h_e is in centimeters

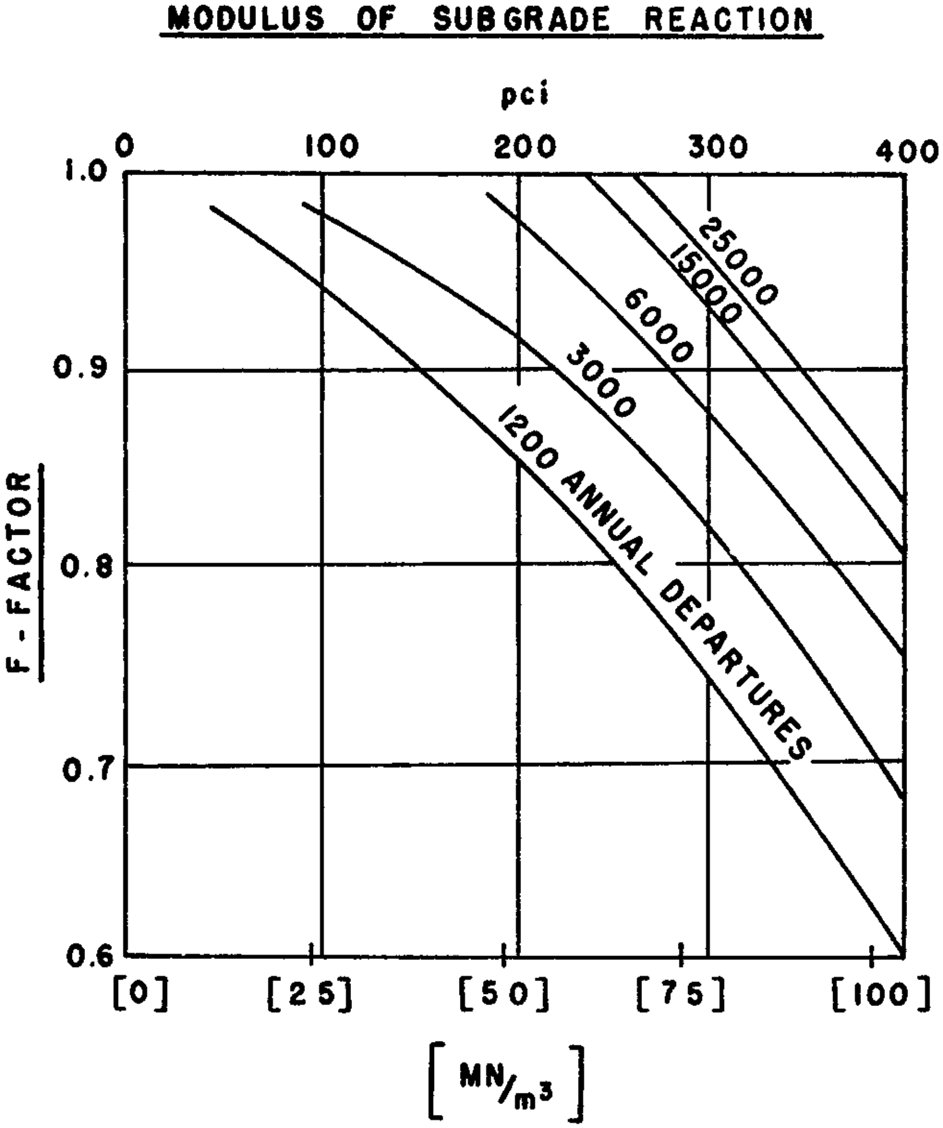


FIGURE 4-3. GRAPH OF "F" FACTOR VS. MODULUS OF SUBGRADE REACTION FOR DIFFERENT TRAFFIC LEVELS

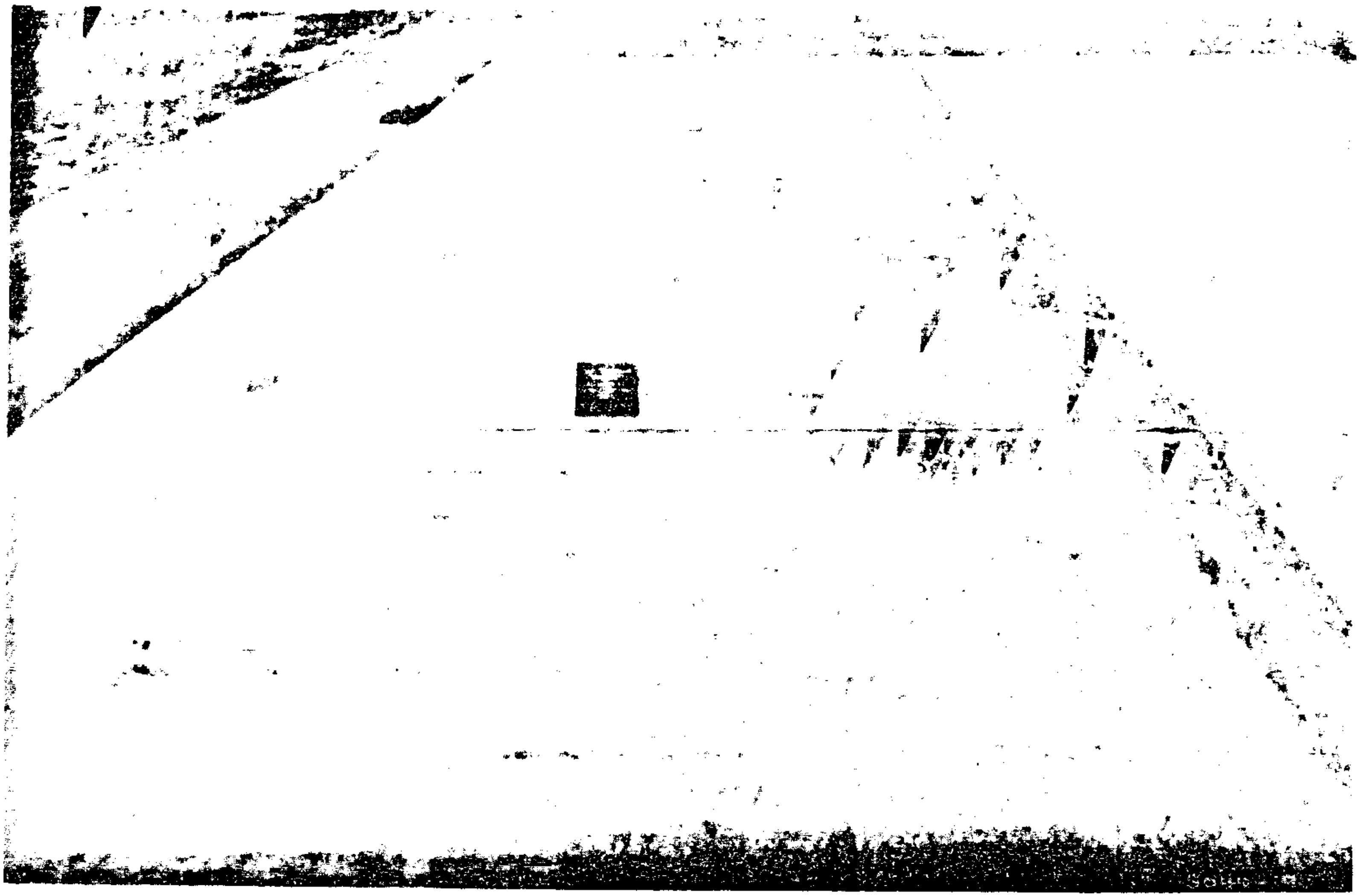


FIGURE 4-4. EXAMPLE OF A "C_b" FACTOR OF 1.0 FOR BITUMINOUS OVERLAY DESIGN

FIGURE 4-5. EXAMPLE OF A "C_b" FACTOR OF 0.75 FOR BITUMINOUS OVERLAY DESIGN

d. The design of a bituminous overlay for a rigid pavement which has an existing bituminous overlay is slightly different. The designer should treat the problem as if the existing bituminous overlay were not present, calculate the overlay thickness required, and then adjust the calculated thickness to compensate for the existing overlay. If this procedure is not used, inconsistent results will often be produced.

(1) An example of the procedure follows. Assume an existing pavement consists of a 10-inch (25 cm) rigid pavement with a 3-inch (7.5 cm) bituminous overlay. The existing pavement is to be strengthened to be equivalent to a single rigid pavement thickness of 14 inches (36 cm). Assume an "F" factor of 0.9 and "C_b" of 0.9 are appropriate for the existing conditions.

(2) Calculate the required thickness of bituminous overlay as if the existing 3-inch (7.5 cm) overlay were not present.

$$t = 2.5\ (0.9 \times 14 - 0.9 \times 10)$$

$$t = 9 \text{ inches } (23 \text{ cm})$$

(3) An allowance is then made for the existing bituminous overlay. In this example assume the existing overlay is in such a condition that its effective thickness is only 2.5 inches (6 cm). The required overlay thickness would then be 9 - 2.5 = 6.5 inches (17 cm). The determination of the effective thickness of the existing overlay is a matter of engineering judgment.

e. The formula for calculating the thickness of bituminous overlays on rigid pavements is limited in application to overlay thicknesses which are equal to or less than the thickness of the base rigid pavement. If the overlay thickness exceeds the thickness of the base pavement, the designer should consider designing the overlay as a flexible pavement and treating the existing rigid pavement as a high quality base material. This limitation is based on the fact that the formula assumes the existing rigid pavement will support considerable load by flexural action. However, the flexural contribution becomes negligible for thick bituminous overlays.

68. <u>NONSTRUCTURAL BITUMINOUS OVERLAYS.</u> In some instances overlays are required to correct nonstructural problems such as restoration of crown, improve ridability, etc. Thickness calculations are not required in these situations, as thickness is controlled by other design considerations or minimum practical overlay thickness. Information concerning runway roughness correction can be found in FAA Report No. FAA-RD-75-110, Methodology for Determining, Isolating and Correcting Runway Roughness. See Appendix 4.

69. REFLECTION CRACKING IN BITUMINOUS OVERLAYS. Reflection cracking in bituminous overlays has been and continues to be a problem which has not been completely solved. Numerous materials and techniques have been tried in attempts to solve the problem with varying degrees of success. Large numbers of research studies have been conducted which often have produced contradictory findings. At the present time the use of particular materials or techniques to prevent reflection cracking should be done on a case-by-case basis.

70. DESIGN OF CONCRETE OVERLAYS. Concrete overlays can be constructed on existing rigid or flexible pavements. The minimum allowable thickness for concrete overlays is 5 inches (13 cm) when placed on a flexible pavement, directly on a rigid pavement, or on a leveling course. The minimum thickness of a concrete overlay which is bonded to an existing rigid pavement is 3 inches (7.5 cm). The design of concrete overlays is predicated on equating the base and overlay section to a single slab thickness. The formulas presented were developed from research on test track pavements and observations of in-service pavements.

71. CONCRETE OVERLAY ON FLEXIBLE PAVEMENT. The design of concrete overlays on existing flexible pavements is based on the design curves in Figures 3-14 through 3-29. The existing flexible pavement is considered a foundation for the overlay slab.

a. For design of the rigid pavement, the existing flexible pavement shall be assigned a k value using Figure 2-5 or 3-13 or by conducting a plate bearing test on the existing flexible pavement. In either case the k value assigned should not exceed 500.

b. When frost conditions require additional thickness, the use of nonstabilized material is not allowed as this would result in a sandwich pavement. The frost protection must be provided by stabilized material.

72. CONCRETE OVERLAY ON RIGID PAVEMENT. The design of concrete overlays on existing rigid pavements is also predicated on the rigid pavement design curves, Figures 3-14 through 3-29. The rigid pavement design curves indicate the thickness of concrete required to satisfy the design conditions for a single thickness of concrete pavement. Use of this method requires the designer to assign a k value to the existing foundation. The k value may be determined by field bearing tests conducted in test pits cut through the existing rigid pavement, or may be estimated from construction records for the existing pavement. The design of a concrete overlay on a rigid pavement requires an assessment of the structural integrity of the existing rigid pavement. The condition factor should be selected after a pavement condition survey. The selection of a condition factor is a matter of engineering

judgment. The use of nondestructive testing (NDT) can be of considerable value in assessing the condition of an existing pavement. NDT can also be used to determine sites for test pits. NDT procedures are given in Advisory Circular 150/5370-11, Use of Nondestructive Testing Devices in the Evaluation of Airport Pavements. See Appendix 4. In order to provide a more uniform assessment of condition factors, the following values are defined:

C_r = 1.0 for existing pavement in good condition -- some minor cracking evident but no structural defects.

C_r = 0.75 for existing pavement containing initial corner cracks due to loading but no progressive cracking or joint faulting.

C_r = 0.35 for existing pavement in poor structural condition - badly cracked or crushed and faulted joints.

The three conditions discussed above are used to illustrate the condition factor rather than establish the only values available to the designer. Conditions at a particular location may require the use of an intermediate value of C_r within the recommended range. Photographs of three different values of C_r are shown in Figures 4-6, 4-7, and 4-8.

a. Concrete Overlay Without Leveling Course. The thickness of the concrete overlay slab applied directly over the existing rigid pavement is computed by the following formula:

$$h_c = \sqrt[1.4]{h^{1.4} - C_r h_e^{1.4}}$$

h_c = required thickness of concrete overlay

h = required single slab thickness determined from design curves

h_e = thickness of existing rigid pavement

C_r = condition factor

Due to the inconvenient exponents in the above formula, graphic displays of the solution of the formula are given in Figures 4-9 and 4-10. These graphs were prepared for only two different condition factors, C_r = 1.0 and 0.75. The use of a concrete overlay pavement directly on an existing rigid pavement with a condition factor of less than 0.75 is not recommended because of the likelihood of reflection cracking.

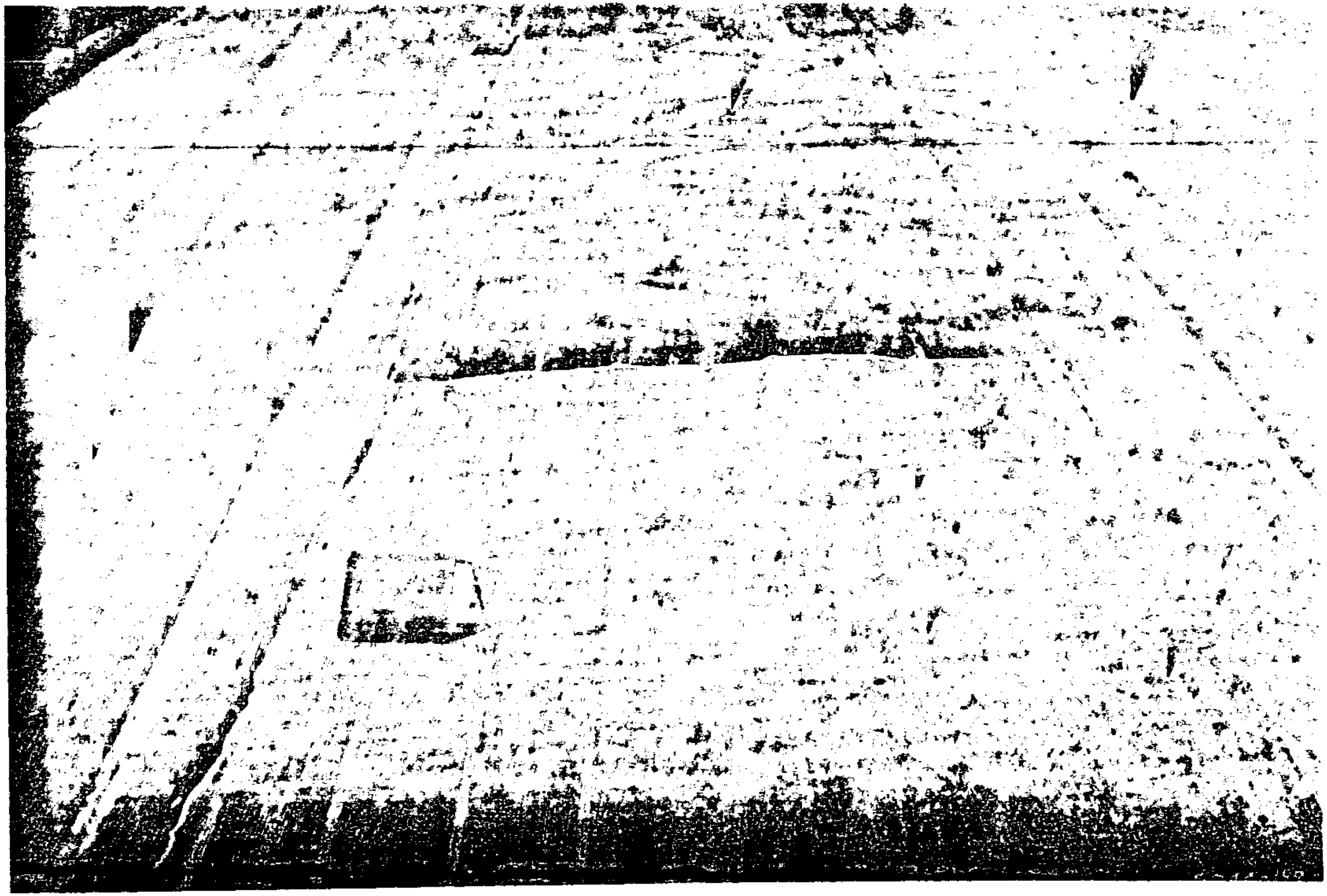


FIGURE 4-6. EXAMPLE OF A CONDITION FACTOR, "C_r" OF 0.85 USED FOR CONCRETE OVERLAY DESIGN PURPOSES

FIGURE 4-7. EXAMPLE OF A CONDITION FACTOR, "C_r" OF 0.5 USED FOR CONCRETE OVERLAY DESIGN PURPOSES

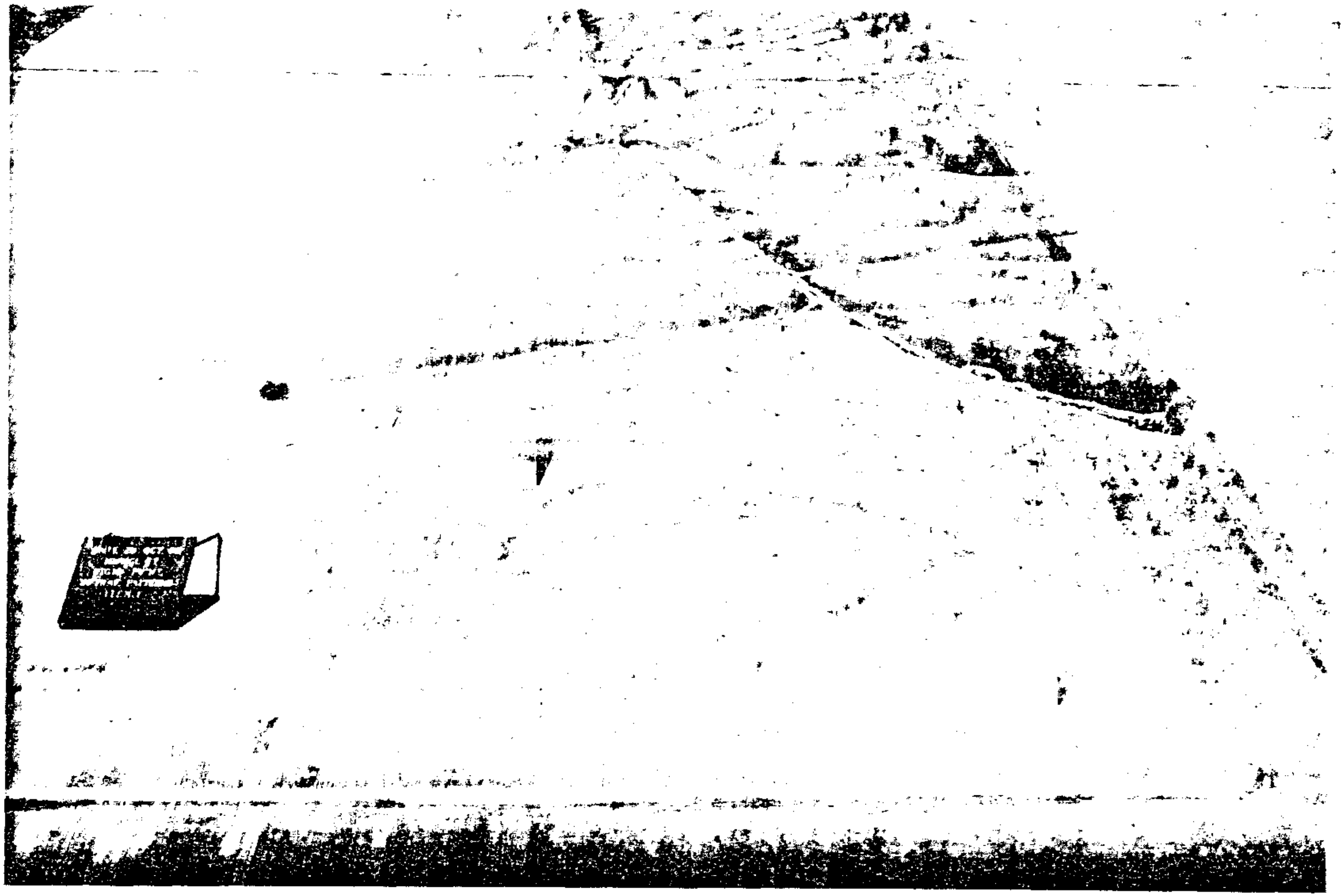
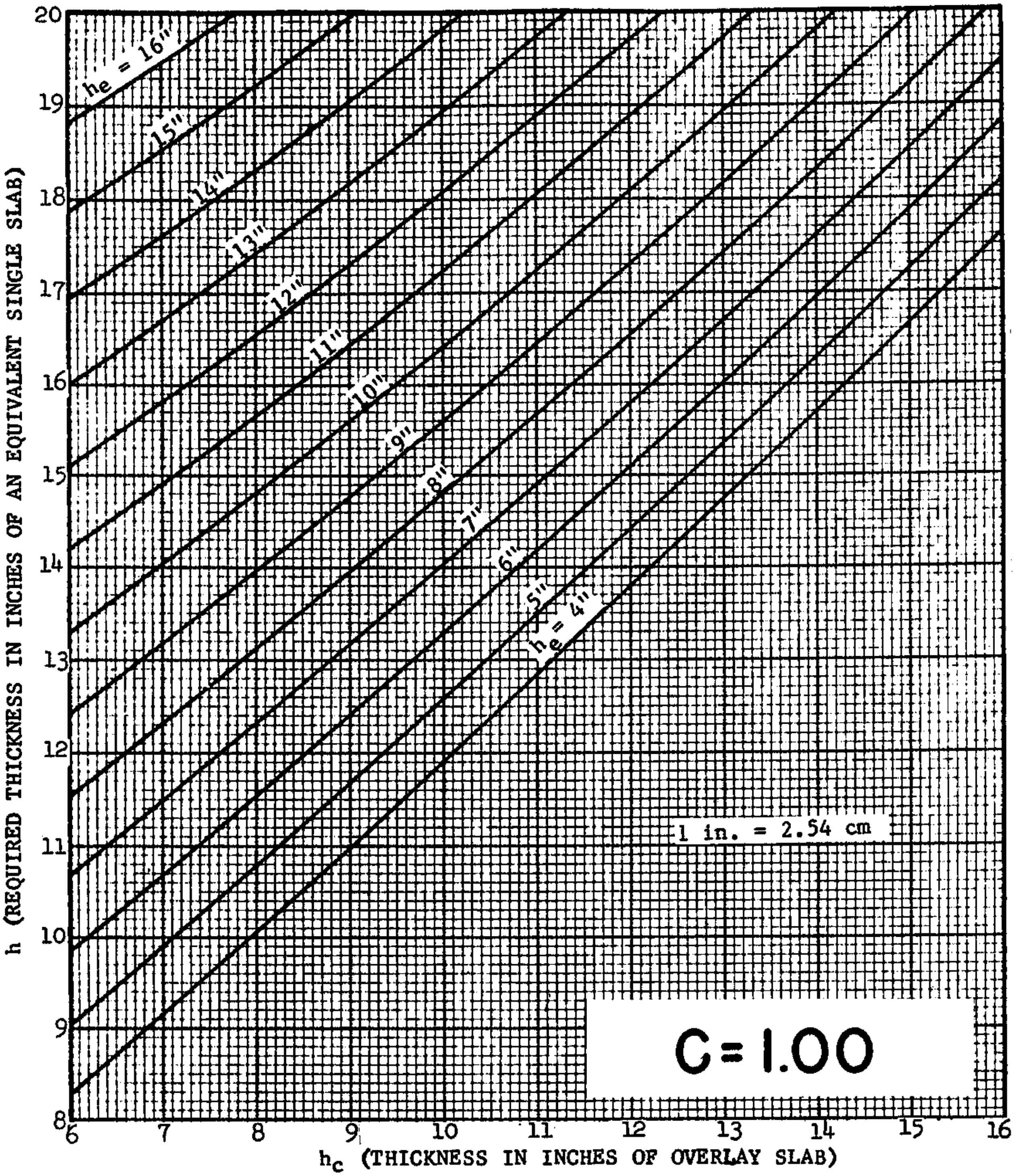


FIGURE 4-8. EXAMPLE OF A CONDITION FACTOR, "C_r" OF 0.35, USED FOR CONCRETE OVERLAY DESIGN PURPOSES



$$h_c = \sqrt[1.4]{h^{1.4} - Ch_e^{1.4}}$$

FIGURE 4-9 CONCRETE OVERLAY ON RIGID PAVEMENT

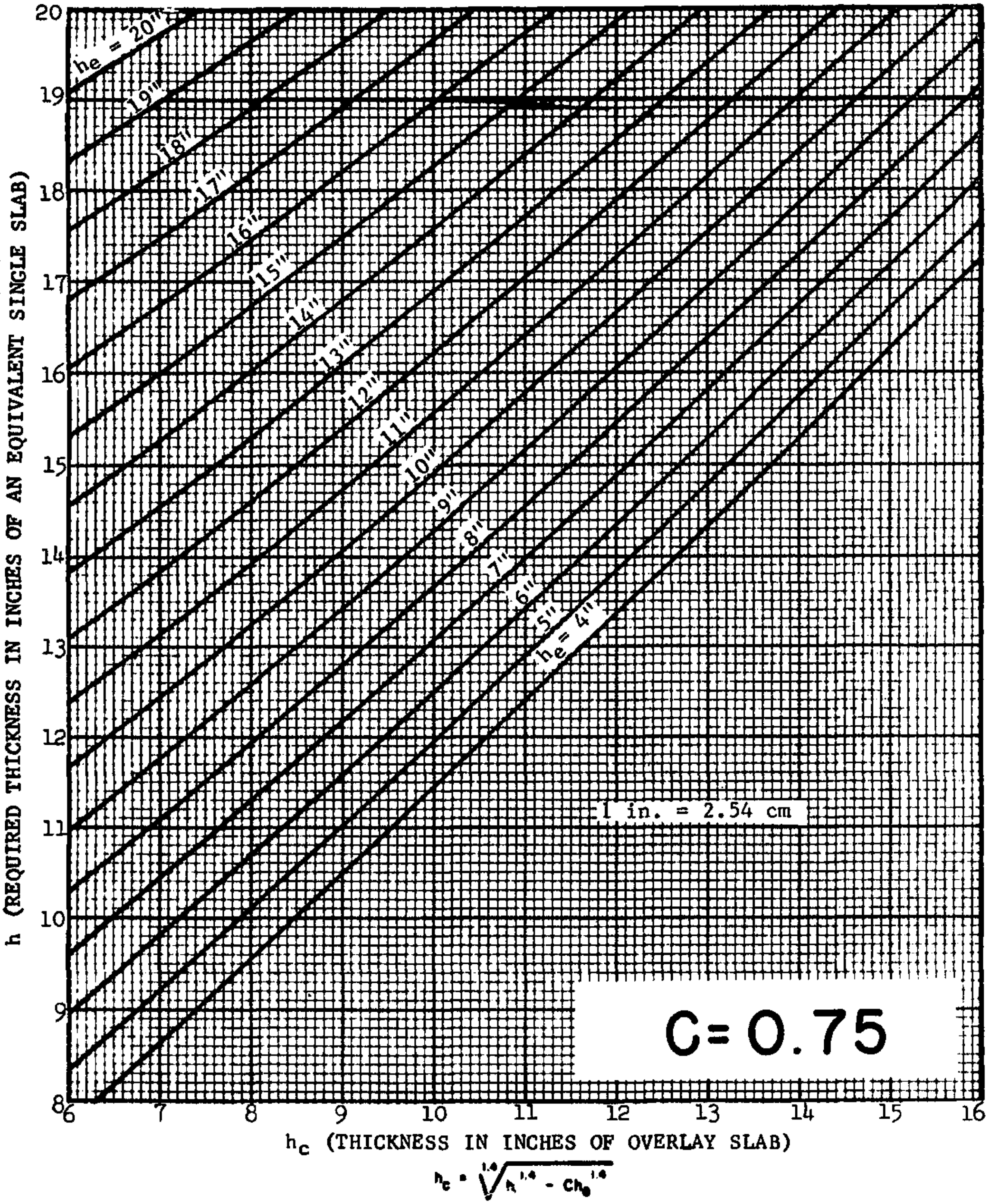


FIGURE 4-10. CONCRETE OVERLAY ON RIGID PAVEMENT

b. Concrete Overlay With Leveling Course. In some instances it may be necessary to apply a leveling course of bituminous concrete to an existing rigid pavement prior to the application of the concrete overlay. Under these conditions a different formula for the computation of the overlay thickness is required. When the existing pavement and overlay pavement are separated, the slabs act more independently than when the slabs are in contact with each other. The formula for the thickness of an overlay slab when a leveling course is used is as follows:

$$h_c = \sqrt{h^2 - C_r h_e^2}$$

h_c = required thickness of concrete overlay

h = required single slab thickness determined from design curves

h_e = thickness of existing rigid pavement

C_r = condition factor

The leveling course must be constructed of highly stable bituminous concrete. A granular separation course is not allowed as this would constitute sandwich construction. Graphic solutions of the above equation are shown in Figures 4-11 and 4-12. These graphs were prepared for condition factors of 0.75 and 0.35. Other condition factors between these values can normally be computed to sufficient accuracy by interpolation.

73. BONDED CONCRETE OVERLAYS. Concrete overlays which are bonded to existing rigid pavements are sometimes used under certain conditions. By bonding the concrete overlay to the existing rigid pavement the new section behaves as a monolithic slab. The thickness of bonded overlay required is computed by subtracting the thickness of the existing pavement from the thickness of the required slab thickness determined from design curves.

$$h_c = h - h_e$$

where:

h_c = required thickness of concrete overlay

h = required single slab thickness determined from design curves

h_e = thickness of existing rigid pavement

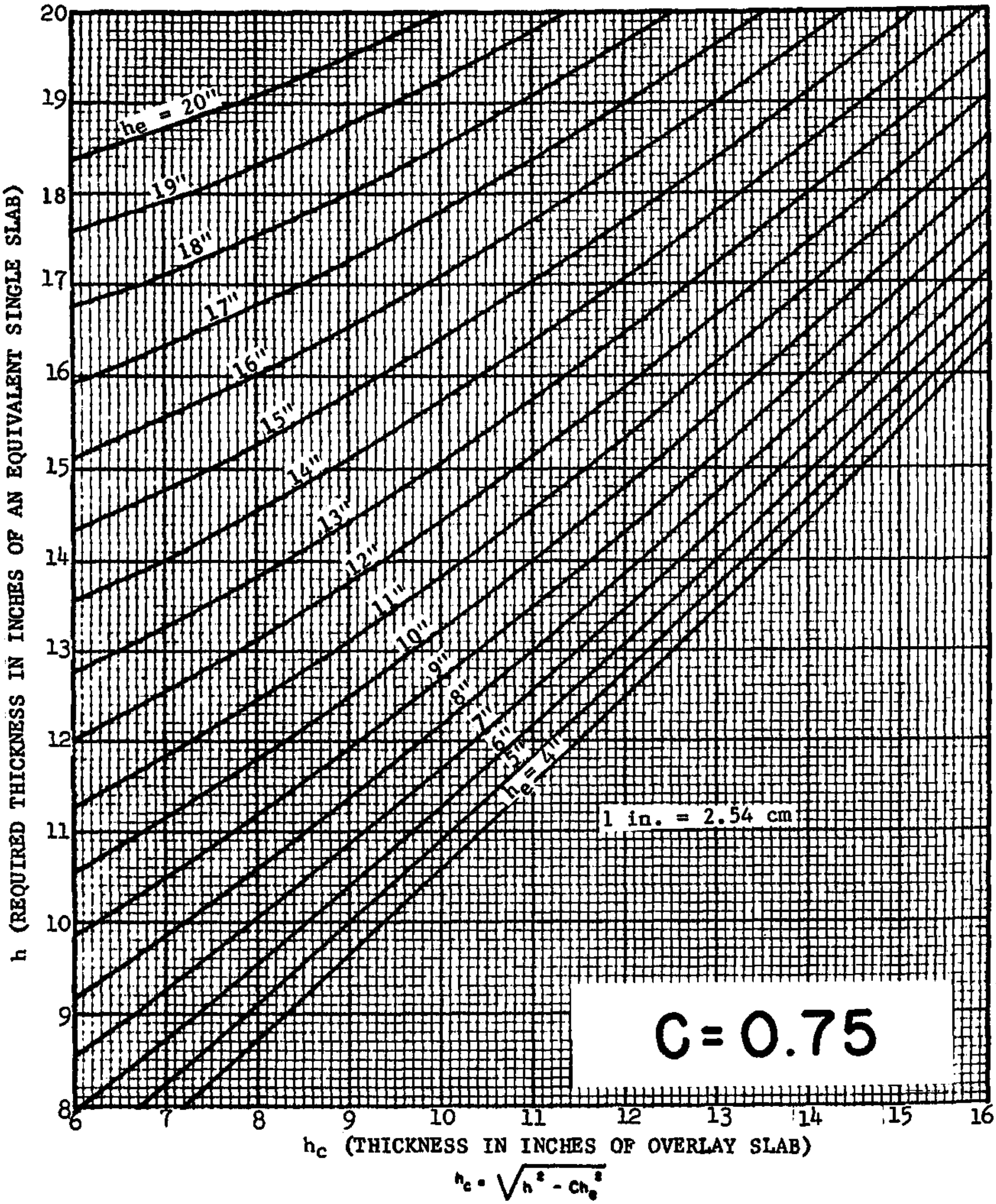


FIGURE 4-11. CONCRETE OVERLAY ON RIGID PAVEMENT WITH LEVELING COURSE

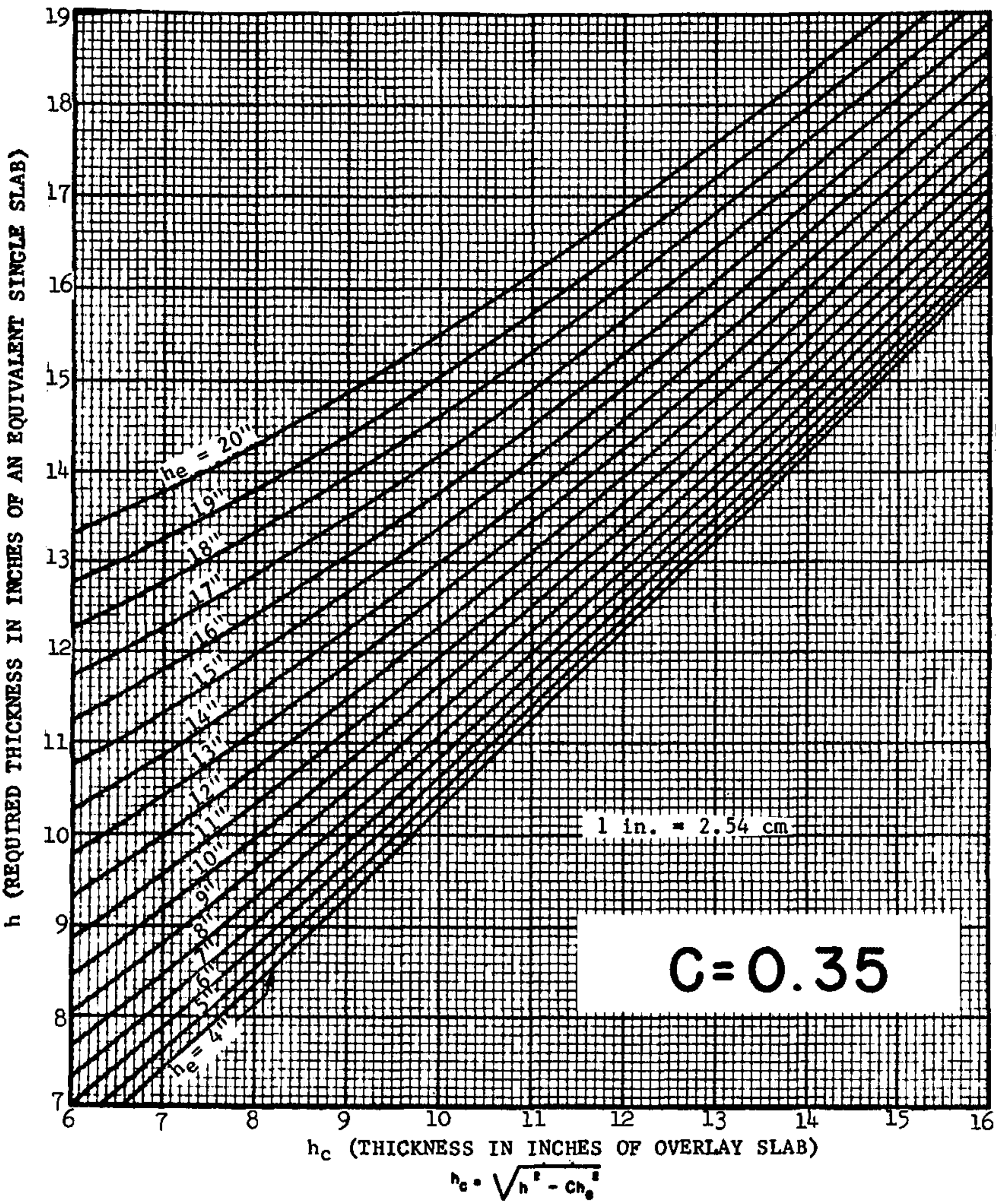


FIGURE 4-12. CONCRETE OVERLAY ON RIGID PAVEMENT WITH LEVELING COURSE

Bonded overlays should be used only when the existing rigid pavement is in good condition. Defects in the existing pavement are more likely to reflect through a bonded overlay than other types of concrete overlays. The major problem likely to be encountered with bonded concrete overlays is achieving adequate bond. Elaborate surface preparation and exacting construction techniques are required to insure bond.

74. JOINTING OF CONCRETE OVERLAYS. Where a rigid pavement is to receive the overlay, some modification to jointing criteria may be necessary because of the design and joint arrangement of the existing pavement. The following points may be used as guides in connection with the design and layout of joints in concrete overlays.

a. Joints need not be of the same type as in the old pavement.

b. It is not necessary to provide an expansion joint for each expansion joint in the old pavement; however, a saw cut or plane of weakness should be provided within 1 foot (0.3 m) of the existing expansion joint.

c. Contraction joints may be placed directly over or within 1 foot (0.3 m) of existing expansion, construction, or contraction joints. Should spacing result in slabs too long to control cracking, additional intermediate contraction joints may be necessary.

d. If a concrete overlay with a leveling course is used, the joint pattern in the overlay does not have to match the joint pattern in the existing pavement.

e. If slabs longer than 20 feet (6.1 m) are desired, distributed steel reinforcement should be provided regardless of overlay thickness.

75. PREPARATION OF THE EXISTING SURFACE FOR THE OVERLAY. Before proceeding with construction of the overlay, steps should be taken to correct all defective areas in the existing surface, base, subbase, and subgrade. Careful execution of this part of an overlay project is essential as a poorly prepared base pavement will result in an unsatisfactory overlay. Deficiencies in the base pavement will often be reflected in the overlay.

a. Failures in flexible pavements can take the form of pavement breakups, potholes and surface irregularities, and depressions.

(1) Localized areas of broken pavement will have to be removed and replaced with new pavement. This type of failure is usually encountered where the pavement is deficient in thickness, the subgrade consists of unstable material, or poor drainage has reduced the supporting power of the subgrade. To correct this condition, the subgrade material should be replaced with a

select subgrade soil or by installation of proper drainage facilities; this is the first operation to be undertaken in repairing this type of failure. Following the correction of the subgrade condition, the subbase, base, and surface courses of the required thickness should be placed. Each layer comprising the total repair should be thoroughly compacted before the next layer is placed.

(2) Surface irregularities and depressions, such as shoving, rutting, scattered areas of settlement, and occasional "birdbaths" should be leveled by rolling, where practical, or by filling with suitable bituminous mixtures. If the "birdbaths" and settlements are found to exist over extensive areas, a bituminous leveling course may be required as part of the overlay. The leveling course should consist of a high-quality bituminous concrete. Scattered areas requiring leveling or patching may be repaired with bituminous patch mixtures.

(3) A bleeding surface may detrimentally affect the stability of the overlay and for this reason any excess bituminous material accumulated on the surface should be bladed off if possible. In some instances, a light application of fine aggregates may blot up the excess material, or a combination of the two processes may be necessary.

(4) Cracks, and joints, 1/2 inch (1 cm) or more in width, should be filled with a lean mixture of sand and liquid bituminous material. This mixture should be well tamped in place, leveled with the pavement surface and any excess removed.

(5) Potholes should be thoroughly cleaned and filled with a suitable bituminous mixture and tamped in place.

b. In rigid pavements, narrow transverse, longitudinal, and corner cracks will need no special attention unless there is an appreciable amount of displacement and faulting of the separate slabs. If the subgrade is stable and no pumping has occurred, the low areas can be taken care of as part of the overlay and no other corrective measures are needed. On the other hand, if pumping has occurred at the slab ends or the slabs are subject to rocking under the movement of aircraft, subgrade support should be improved by pumping cement grout under the pavement to fill the voids that have developed. Pressure grouting requires considerable skill to avoid cracking slabs or providing uneven support for the overlay.

(1) If the pavement slabs are badly broken and subject to rocking because of uneven bearing on the subgrade, the rocking slabs can be broken into smaller slabs to obtain a more firm seating. Badly broken slabs that do not rock will not require repairs since the criteria make adjustments for such a condition in the pavement thickness. In some cases, it may be desirable to replace certain badly broken slabs with new slabs before starting construction of the overlay. The decision in such cases will have to be made according to the merits of the individual project.

(2) Where the existing pavement is rough due to slab distortion, faulting, or settlement, a provision should be made for a leveling course of bituminous concrete before the overlay is commenced.

(3) Cracks, and joints, 1/2 inch (1 cm) or more in width, should be filled with a lean mixture of sand and liquid bituminous material. This mixture should be tamped firmly in place, level with the pavement surface and any excess removed.

(4) After all repairs have been completed and prior to the placing of the overlay, the surface should be swept clean of all dirt, dust, and foreign material that may tend to break the bond between the overlay and the existing pavement. Any extruding joint-sealing material should be trimmed from rigid pavements.

(5) Bonded concrete overlays will require special attention to insure bond with the existing pavement. Acid etching or mechanical texturing is sometimes used to provide a surface which will allow bonding. A cement grout placed immediately ahead of the concrete overlay is recommended to promote bonding.

76. MATERIALS AND METHODS. With regard to quality of materials and mixes, control tests, methods of construction, and workmanship, the overlay pavement components are governed by AC 150/5370-10, Standards for Specifying Construction of Airports.

a. If a bituminous overlay is specified, the existing pavement should receive a light tack coat or fog coat immediately after cleaning. The overlay should not extend to the edges of the pavement but should be cut off approximately 3 inches (7.5 cm) from each edge.

b. After cleaning, existing concrete surfaces should be wetted prior to depositing the plastic concrete of a rigid overlay to insure as good a bond as possible.

c. Should the existing pavement require drilling to provide anchorage for the overlay pavement forms, the size and number of holes should be the minimum necessary to accomplish that purpose. Holes should not be located close to joints or cracks. Location of holes for form anchors should be such as to avoid causing additional cracking or spalling.

77. NEW OVERLAY MATERIALS. In recent years, some new pavement overlay materials have been used with varying degrees of success. These materials include fibrous concrete, asbestos asphalt, and rubberized asphalt. Use of materials other than conventional portland cement concrete or asphaltic concrete should be subject to case-by-case approval.

CHAPTER 5. PAVEMENTS FOR LIGHT AIRCRAFT.

78. GENERAL.

a. Pavements for light aircraft may be defined as landing facilities intended to accommodate personal aircraft or other small aircraft engaged in nonscheduled activities such as agricultural or instructional flying. These pavements will not be required to handle aircraft exceeding a gross weight of 30,000 pounds (13 000 kg), and in many cases these aircraft will not exceed 12,500 pounds (5 700 kg). The design for pavements which are to serve aircraft of 30,000 pounds (13 000 kg) gross weight or more should be based on the criteria contained in Chapter 3 of this publication.

b. Some airports may not require paved operational areas. Conditions at the site may be acceptable for development of a turf surface adequate for limited operations of these light aircraft. It may be possible to construct an aggregate-turf surface by improving the stability of a soil with the addition of aggregate prior to development of the turf. Aggregate-turf construction is covered in some detail in the latter part of this chapter.

c. In most areas, however, it is not possible to provide and maintain a stable turf surface because of adverse weather conditions or high density traffic. Under these conditions, construction of an all-weather pavement may be necessary.

d. Pavements designed to serve aircraft of less than 30,000 pounds (13 000 kg) gross weight may be flexible or rigid-type pavements.

79. TYPICAL SECTIONS. Typical cross sections for light aircraft pavements are shown in Figure 5-1. No distinction is made between critical and noncritical pavement sections for pavements serving light aircraft.

80. FLEXIBLE PAVEMENT THICKNESS.

a. The curves shown in Figure 5-2 give the pavement thickness requirements for aircraft weighing up to 30,000 pounds (13 000 kg) gross weight. For aircraft of 30,000 pounds (13 000 kg) and above the curves in Chapter 3 should be used. The pavement thickness determined from Figure 5-2 should be used on all areas of the airport pavement. No reduction in thickness should be made for "noncritical" areas of pavements.

b. Use of the curve requires a CBR value for the subgrade and the gross weight of the design aircraft. The preferred method of establishing the subgrade CBR is by testing. The testing procedures described in Chapter 3 should also be applied to light load pavements. In instances where CBR tests are not practical, the values listed in Table 2-3 may be used.

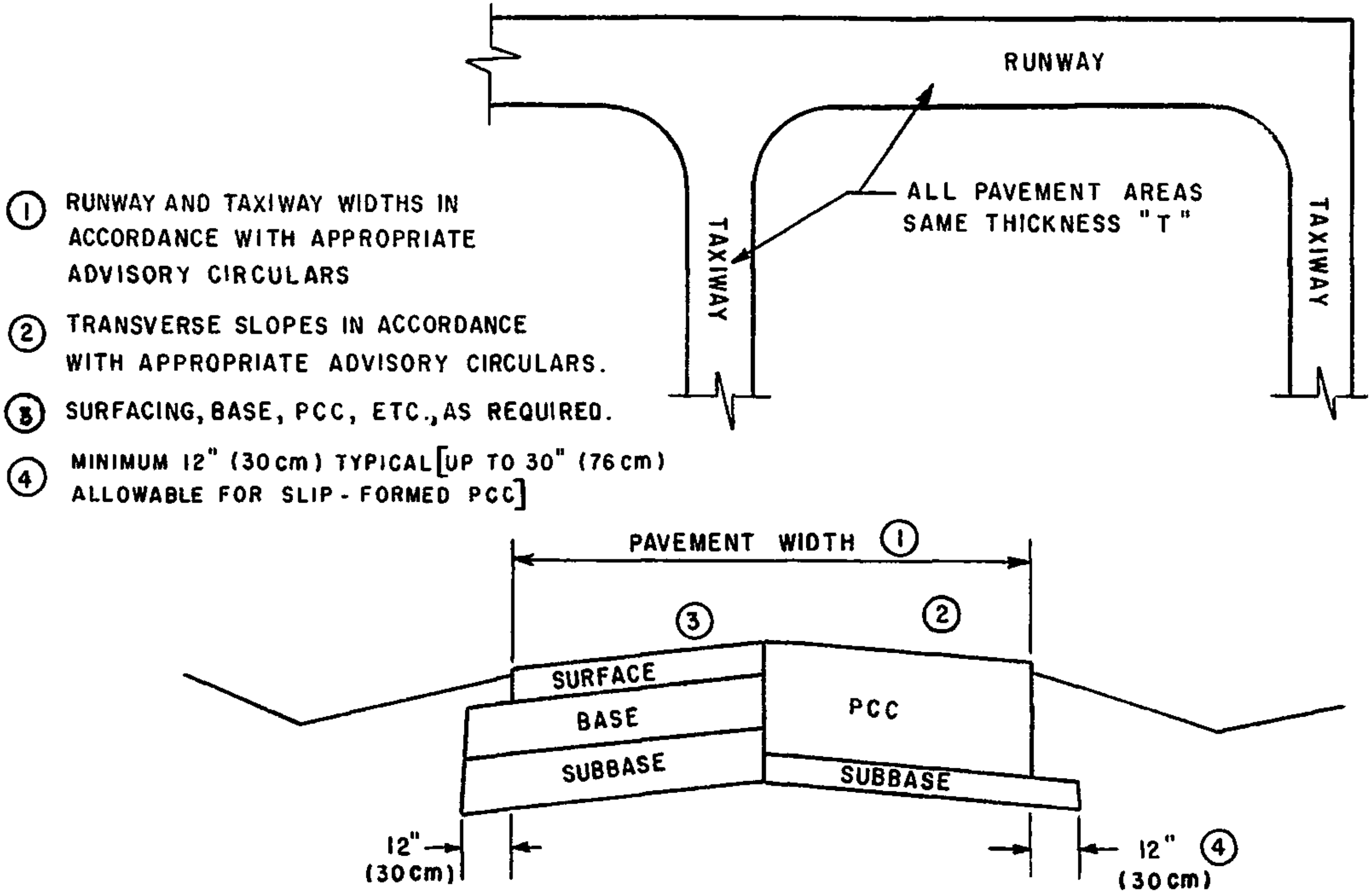


FIGURE 5-1. TYPICAL SECTIONS FOR LIGHT AIRCRAFT PAVEMENTS

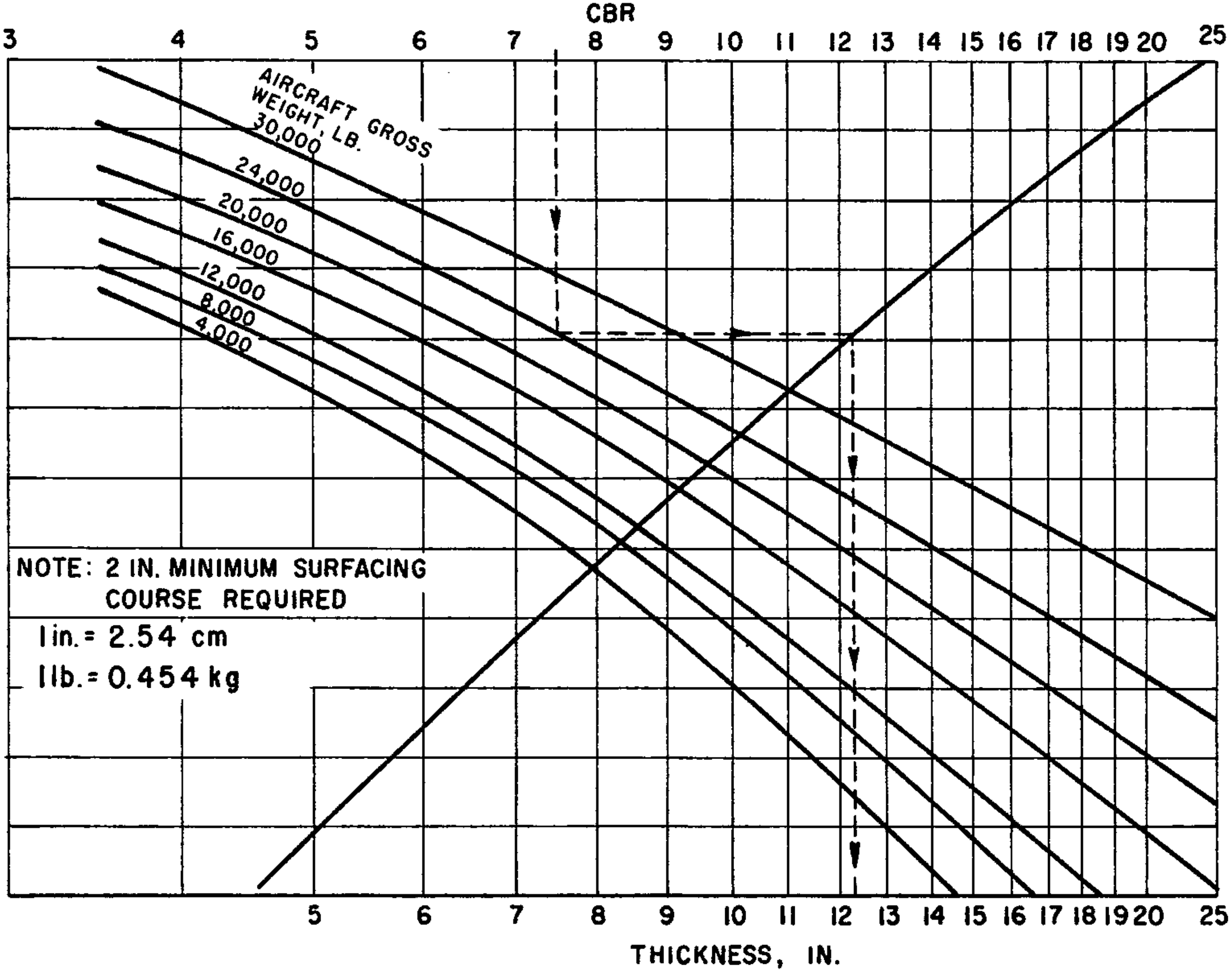


FIGURE 5-2. DESIGN CURVES FOR FLEXIBLE PAVEMENTS - LIGHT AIRCRAFT

c. The thickness of surfacing and base is determined by using the CBR-20 line. The difference between the total pavement thickness required and the CBR-20 line thickness, composed of surfacing and base, yields the thickness of subbase. Note that the minimum thickness of bituminous surfacing over a granular base is 2 inches (5 cm). The reason for the 2-inch (5 cm) minimum surfacing thickness is that thinner layers are difficult to lay down and compact on granular bases. Bituminous surfacing thickness of less than 2 inches (5 cm) is permissible on stabilized base materials if proper laydown and compaction can be achieved.

d. The base course thicknesses in Figure 5-2 range from 3 inches (7.5 cm) to 6 inches (15 cm) while the subbase thicknesses vary from 0-14 inches (0-36 cm). In some instances difficulties may be encountered in compacting thin bases or subbases. In these cases the base or subbase thicknesses may be increased to facilitate construction even though the additional thickness is not needed for structural capacity.

e. As an example of the use of Figure 5-2, assume a pavement is to be designed for the following conditions.

Aircraft gross weight = 20,000 lbs. (9100 kg)

Subgrade CBR = 7

(1) Enter the upper abscissa of Figure 5-2 with the subgrade CBR value of 7.

(2) Make a vertical projection downward to the aircraft gross weight line of 20,000 lbs. (9100 kg).

(3) At the point of intersection of the vertical projection and the aircraft gross weight line, make a horizontal projection to the pivot line.

(4) At the point of intersection of the horizontal projection and the pivot line, make a vertical projection down to the lower abscissa and read the total pavement thickness required, in this example 11.8 inches (30 cm).

(5) To determine the thickness of surfacing and base proceed as in steps (1) through (4) above using a CBR value of 20. In this example, a thickness of 5 inches (13 cm) is read on the lower abscissa. This represents the combined thickness of surfacing and base.

(6) The design section would thus consist of 2 inches (5 cm) of bituminous surfacing, 3 inches (7.5 cm) of base, and 7 inches (18 cm) of subbase. Should difficulties be anticipated in compacting the 3-inch (7.5 cm) base course, some of the subbase material should be replaced with base course material to ease construction. If base material is substituted for subbase material, the equivalency factors given in Table 3-2 may be used to determine the thickness ratios.

f. Under certain conditions, it may be desirable to utilize a bituminous surface treatment on a prepared base course in lieu of hot mix asphaltic concrete. In such instances the strength of the pavement is furnished by the base, subbase, and subgrade. Additional base course thickness will be necessary to make up for the missing surface course. Additional base should be provided at a ratio of 1.2 to 1.6 inches (3-4 cm) of base for each 1 inch (2.5 cm) of surfacing.

g. Since the base and subbase course materials discussed in Chapter 3 are more than adequate for light aircraft, full consideration should be given to the use of locally available, less expensive materials which are entirely satisfactory for these pavements. These materials may include locally available granular materials, soil aggregate mixtures, or soils stabilized with portland cement, bituminous materials, or lime. The designer is cautioned, however, if the ultimate design of the pavement is greater than 30,000 lbs. (13 000 kg), higher quality materials should be specified at the outset.

h. Since the loads which these pavements must support are much less than those accommodated by pavements designed for heavier aircraft, certain reductions can be made in the compaction requirements for the base and subbase materials. Compaction control for these pavements is based on the ASTM D-698 standard. The weight of maintenance equipment should also be taken into account in designing these pavements.

81. RIGID PAVEMENT THICKNESS.

a. Rigid pavements designed to serve aircraft weighing 12,500 pounds (5700 kg) or less should be 5 inches (13 cm) thick. Rigid pavements designed to serve aircraft weighing between 12,500 pounds (5700 kg) and 30,000 pounds (13 000 kg) should be 6 inches (15 cm) thick. No design curves for light duty rigid pavements are presented since there are only two thickness requirements.

b. Rigid pavements designed to serve aircraft weighing between 12,500 pounds (5 700 kg) and 30,000 pounds (13 000 kg) will require a minimum subbase thickness of 4 inches (10 cm) except as shown in Table 3-4 of Chapter 3. No subbase is required for designs intended to serve aircraft weighing 12,500 pounds (5 700 kg) or less, except when soil types OL, MH, CH or OH are encountered. When the above soil types are present, a minimum 4-inch (10 cm) subbase should be provided.

c. The spacing of joints for rigid pavements for light aircraft should be 12.5 feet (3.8 m) for longitudinal joints and 15 feet (4.6 m) for transverse joints. Jointing details are shown in Figure 5-3. Note that butt-type construction joints are permitted when bituminous or cement stabilized subbase is provided. The designer is reminded that keyed construction joints are not allowed on slabs less than 9 inches (23 cm) thick. Thickened edges are not required when the design is based on aircraft weighing 12,500 pounds (5 700 kg) or less. The last three transverse joints of a runway or taxiway should be doweled TYPE D contraction joints. The outer lanes of all pavement features should be tied to the inner lanes using a TYPE C hinged joint. Odd-shaped slabs should be reinforced with 0.05% steel in both directions. Odd-shaped slabs are defined as slabs which are not rectangular in shape or in which the slab length-to-width ratio exceeds 1.25.

d. Specifications concerning the quality and placement of portland cement concrete shall be in accordance with Item P-501, Portland Cement Concrete Pavement. The materials suitable for subbase courses are covered in Item P-154, Subbase Course.

82. SOIL STABILIZATION. Soil stabilization is the procedure whereby the properties of a soil are improved to the extent that it will meet the requirements for pavement bases or subbases. Stabilized soils are not intended to serve as a surface course and must be provided with a surface in order to resist the abrasive action of operating vehicles and weathering. To be effective, stabilization should provide a foundation which will furnish adequate support for the loads transmitted through the paved surface, and will eliminate or reduce to an appreciable extent the detrimental effects of volume changes occurring in the soil due to climate influences or moisture variations. Mechanical and chemical stabilization are the two general types currently employed.

a. Mechanical stabilization of soil for airport pavements follows standard practices developed over the years, and requirements regarding materials as well as construction methods are quite definitively established. Performance studies have disclosed that the success of a granular stabilized base course depends on the gradation of the mixture and the physical properties of the material passing the No. 40 sieve.

(1) The gradation for gravel or stone is available in Item P-208, Aggregate Base Course. Likewise, Item P-213, Sand-Clay Base Course, gives gradations for the coarse and fine types. In addition to the gradation requirements, there are certain other requirements common to all granular-type base courses. Among these are:

(a) The fraction passing the No. 200 sieve should not exceed one-half the fraction passing the No. 40 sieve.

(b) The liquid limit of the material passing the No. 40 sieve should not exceed 25 and the plasticity index should not exceed 6.

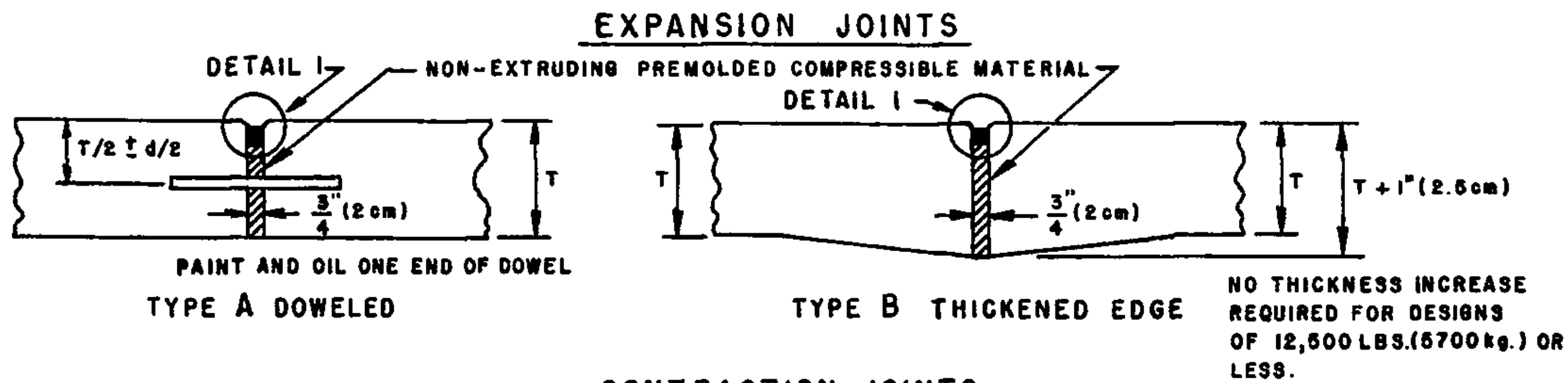
(c) For the fine aggregate type of sand-clay base, the plasticity index of the material passing the No. 40 sieve should not exceed 4.

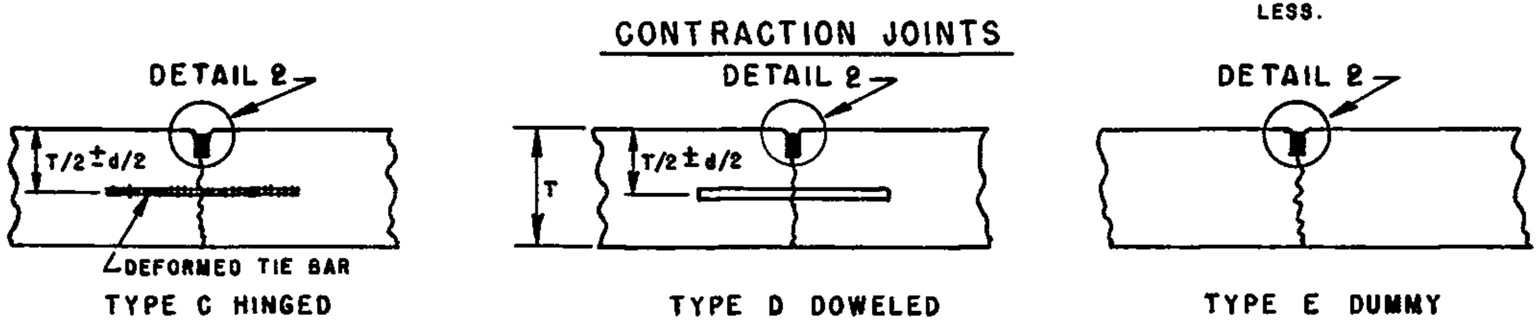
(2) Granular-type stabilized soil base courses meeting the requirements outlined above, when properly compacted, can give excellent service. It is emphasized, however, that the restriction placed upon the plasticity index must be rigidly adhered to if successful stabilization by these means is to be expected.

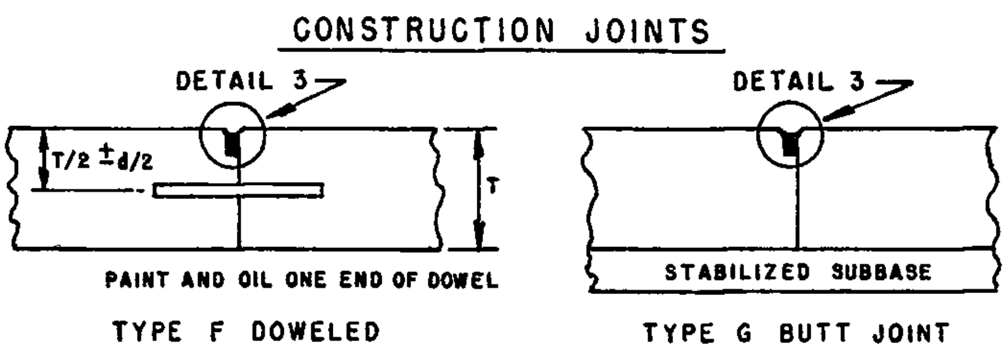
b. A relatively new form of mechanical stabilization, Membrane Encapsulated Soil Layer (MESL), was studied in the FAA pavement research program and showed promise as a potential subbase for light aircraft. The process involves encapsulating a compacted clay soil in a watertight membrane. Compacted clay is quite stable as long as it can be kept dry. The construction techniques are discussed in FAA Research Report No. FAA-RD-73-198-III, Comparative Performance of Structural Layers in Pavement Systems, Volume III, Design and Construction of MESL. See Appendix 4. Approval for the use of encapsulated soil layers should be on a case-by-case basis.

c. Chemical stabilization usually involves the addition of bituminous material, cement or lime to a soil or soil aggregate. Recommendations for the best type of chemical stabilization can be found in Chapter 2, paragraph 13 of this circular. Further discussion of chemical stabilization is presented below.

(1) Bituminous stabilization is the combining of bituminous material with soil, soil aggregate, or sand to produce the desired soil characteristics. Bituminous stabilizing agents include cutback asphalts, slow-curing asphalts or road oils, emulsified asphalts, and tars. Methods of construction vary with the type of equipment available but, regardless of the







NOTE:

1. FOR DETAILS 1, 2, & 3 SEE NEXT PAGE.
2. ALL DOWELS 3/4" (19mm) DIA, 18" (46 cm) LONG, SPACED 12" (30 cm) ON CENTERS.
3. ALL TIE BARS NO. 4 DEFORMED BARS 20" (51 cm) LONG, SPACED 36" (0.9 m) ON CENTERS.

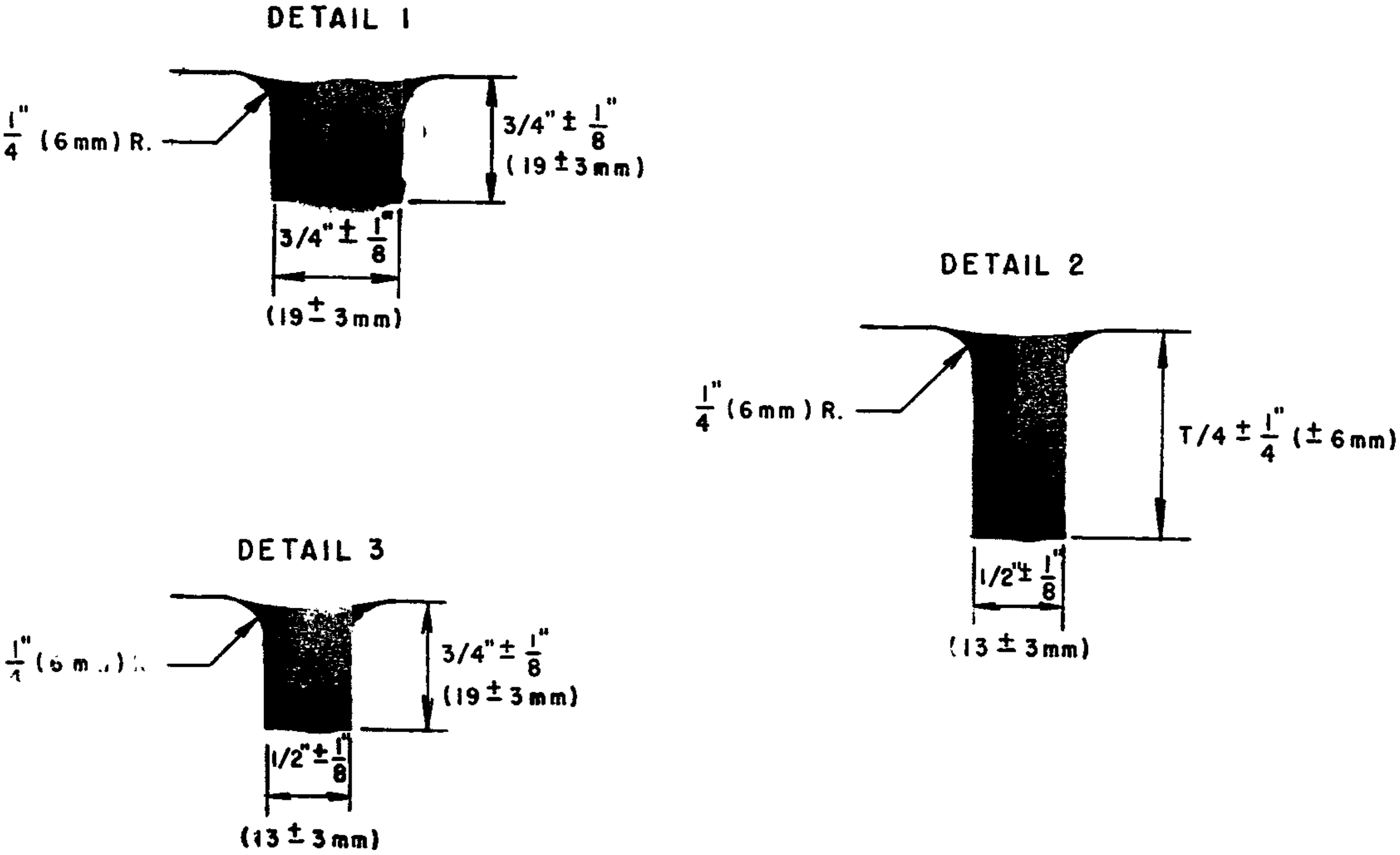


FIGURE 5-3. JOINTING DETAILS FOR LIGHT LOAD RIGID PAVEMENTS

equipment, the different steps consist essentially of soil preparation by scarifying and pulverizing, thorough and uniform mixing of the bituminous material with the soil, curing of the mixture to get rid of excess moisture and volatile constituents, and compaction to a predetermined density.

(a) A successful job depends on the proper execution of each one of these steps. Test methods for determining the type and amount of bituminous material vary considerably in different areas. Manufacturers' recommendations also differ in this respect. Engineering services along these lines are available from the producers of the particular materials selected for use. In this connection, D-915, Testing Soil-Bituminous Mixtures, has been adopted by the American Society for Testing and Materials.

(b) The type and grade of bituminous material to use depend on the characteristics of the soil, the climatic conditions, and the type of mixing equipment available. It is generally accepted that the best practice is to use the heaviest grade of bituminous material that can be readily mixed with the soil. Travel plants will permit the use of heavier grades of bitumens than will harrows and motor graders. The most commonly used grades of bituminous binders are:

Liquid Asphalts (Rapid Curing)
RC-70, RC-250, and RC-800

Liquid Asphalts (Medium Curing)
MC-70, MC-250, and MC-800

Liquid Asphalts (Slow Curing)
SC-70, SC-250, and MCSC-800

Tar RT-3 to TR-7

Emulsified Asphalt SS-1

(c) Bituminous stabilization will give satisfactory performance on airports when the mixtures are made with soils having the proper physical characteristics. On the other hand, serious failures can occur where the soils have high silt and clay contents, contain mica in appreciable amounts, mixtures are compacted before they have cured properly, or surfaces are placed on the stabilized base too soon, thus trapping the excess moisture and volatile materials.

(d) In general, Item P-216, Mixed In-Place Base Course, covers the methods of construction for this form of stabilization. This item must be modified by deleting the sections referring to job mix formula, materials, and composition of mixture and substituting requirements applicable to the material to be stabilized. Changes in construction procedures may be desirable in certain localities.

(2) By the addition of portland cement in the correct quantity, many types of soils and materials such as shale, gravel, sand, screenings, slag, and mine tailings can be stabilized. Construction of soil cement bases has been standardized to a large degree. Item P-301, Soil Cement Base Course, covers the construction of soil cement base courses. Item P-304, Cement Treated Base Course, covers cement-treated base courses.

(a) Where soil cement is to be employed, the minimum thickness of such stabilization should be 6 inches (15 cm). Plastic soils can be modified with lime to reduce the plasticity and allow the use of cement stabilization. This two-step method of stabilization is relatively more expensive than stabilizing in a single step and will be economical only in areas where aggregate costs are high.

(b) Portland cement can be used in the reconstruction of gravel base courses that have failed because of a high plasticity index of the soil binder. The reconstruction consists of scarifying and pulverizing the existing gravel base, adding and mixing the portland cement, and recompacting to a controlled density. The addition of the correct amount of cement can produce mixtures having plasticity indices well under the 6 percent maximum specification requirement.

(c) In instances where the minimum thickness of 6 inches (15 cm) of soil cement base course is greater than the thickness of base course determined by Figure 5-2, the excess thickness can be subtracted from the subbase thickness. The total pavement thickness of base and subbase should be as shown in Figure 5-2.

(3) Lime, in small percentages, has been added to base course materials containing appreciable amounts of soil in order to reduce the plasticity. Performance records of highway pavements indicate this reduction in plasticity accomplishes a marked improvement in the stability of the base course. Lime also reduces the tendency for a soil to swell.

(a) The amount of lime required for stabilization should be determined by means of laboratory tests. Various percentages of lime should be mixed with the soil and the percentage which results in reducing the plasticity to the desired level should be selected. In general, 2 or 3 percent of hydrated lime will serve to reduce the plasticity of pit-run gravel and similar base course materials to the extent that they meet specifications.

(b) Plastic soils should be treated with hydrated lime in amounts ranging from 3 to 10 percent. Investigations show that for each soil there is an optimum percentage of lime. An addition of lime in excess of this amount will not reduce the plasticity to any significant degree. The lowest percentage above which improvement is negligible is the most satisfactory for the particular soil. Soils stabilized with lime should not be considered for base course purposes,but they may be used as subbase material.

(c) With respect to construction procedures, lime-soil combinations are processed in a manner similar to soil-cement combinations which are covered in Item P-301, Soil Cement Base Course, except that the lime may be applied as a slurry or in the dry state.

(d) Other chemical stabilizers such as resins, plastics, metallic salts, and polymers have been used as a means of improving the stability of soils. These stabilizers are relatively expensive and their use should be restricted to case-by-case applications.

83. AGGREGATE TURF.

a. Aggregate-turf runways differ from the usual turf runway in that the stability of the soil has been increased by the addition of granular materials prior to establishment of the turf. The objective of this type of construction is to provide a landing area that will not soften appreciably during wet weather and yet will retain sufficient soil to promote the growing of turf. Such a runway is designed to serve aircraft having a gross weight not exceeding 12,500 pounds (5 700 kg), although under certain conditions aircraft considerably in excess of this loading might be accommodated.

b. In general, the material used in the aggregate-turf combination consists of whatever suitable supply is locally available to permit construction work to be accomplished as economically as possible. The gradation requirements of the mixture and the stabilizer aggregate are presented in Item P-217, Aggregate--Turf Pavement. The materials should be composed of natural or prepared mixtures of soil with gravel, stone, sand, or any other aggregate, and the aggregate retained on the No. 4 sieve should be reasonably sound and durable enough to resist weathering, abrasion, and crushing. Shales and similar materials that break up and weather rapidly should not be used.

c. Construction details and material requirements are covered in Item P-217, Aggregate--Turf Pavement. The proportion of aggregate to soil and the degree of compaction that is permissible from the standpoint of stability should be weighed against the requirements for the establishment of turf. Local climatic conditions exert a great influence on these two factors. Compaction from 80 to 90 percent of maximum density, as defined in ASTM D-698, is considered satisfactory for stability and will not interfere with the growth of grass.

d. The desirable thickness to be stabilized with the granular materials varies with the type of soil and the drainage and climatic conditions. The total thickness of aggregate stabilized soil should be determined from Figure 5-2 irrespective of the 2-inch (5 cm) surface.

84. OVERLAYS. Overlays of pavements intended to serve light aircraft are designed in the same manner as overlays for heavy aircraft. The main difference in requirements is that an overlay intended to increase strength should be a minimum of 2 inches (5 cm) thick rather than 3 inches (7.5 cm).

85. FULL-DEPTH ASPHALT PAVEMENTS. Pavements to serve light aircraft may be constructed of full-depth asphalt on a case-by-case approval basis. Criteria should be as specified in paragraph 37.

86. PAVEMENT RESEARCH FOR LIGHT AIRCRAFT. An FAA sponsored research study has been conducted on pavements for light aircraft. The results of the study are presented in Report No. FAA-RD-76-179, Structural Design of Pavements for Light Aircraft. Portions of this report were used in this circular.

CHAPTER 6. PAVEMENT EVALUATION.

87. SCOPE. This chapter covers the evaluation of pavements for all weights of aircraft.

88. PURPOSES OF PAVEMENT EVALUATION.

a. Airport pavements are evaluated for several reasons. Evaluations are needed to establish load carrying capacity for expected operations, to assess the ability of pavements to support significant changes from expected volumes or types of traffic, and to determine the condition of existing pavements for use in the planning or design of improvements which may be required to upgrade a facility.

b. Evaluation procedures are essentially the reversal of design procedures. Since the new FAA design methodology may result in slightly different thicknesses than other design methods, such as those published in earlier FAA advisory circulars, it would be inappropriate to evaluate existing pavements by the new method unless they had been designed by that method. This could reduce allowable loads and penalize aircraft operators. To avoid this situation, pavements should be evaluated for the various conditions indicated in the following paragraphs.

89. EVALUATIONS FOR EXPECTED OPERATIONS. When airport pavements are subjected to the loads which were anticipated at the time of design, their evaluation should be based on that original design method. For example, if a pavement was designed by method X to serve certain aircraft for a 20-year life and the traffic using the pavement is essentially the same as was anticipated at the time of design, the pavement should be evaluated according to method X. The evaluator should recognize that some deterioration will occur over the 20-year design life. The load bearing strength of the pavement should not be reduced if the pavement is providing a safe operational surface. The evaluation curves which were contained in AC 150/5320-6B, dated May 28, 1974, have been reproduced in this chapter to facilitate this evaluation policy. See Figures 6-1 through 6-15.

90. EVALUATIONS FOR CHANGING TRAFFIC. Evaluations are sometimes required to determine the ability of an existing pavement to support substantial changes in pavement loadings. This can be brought on by the introduction of different types of aircraft or changes in traffic volume. In these instances it is also recommended that existing pavements be evaluated according to the methods by which they were designed. The effect of changes in traffic volume are usually small and will not have a large impact on allowable loads. The effect of changes in aircraft types depends on the gear weight and gear configuration of the aircraft. The load carrying capacity of existing bridges, culverts, storm drains, and other structures should also be considered in these evaluations.

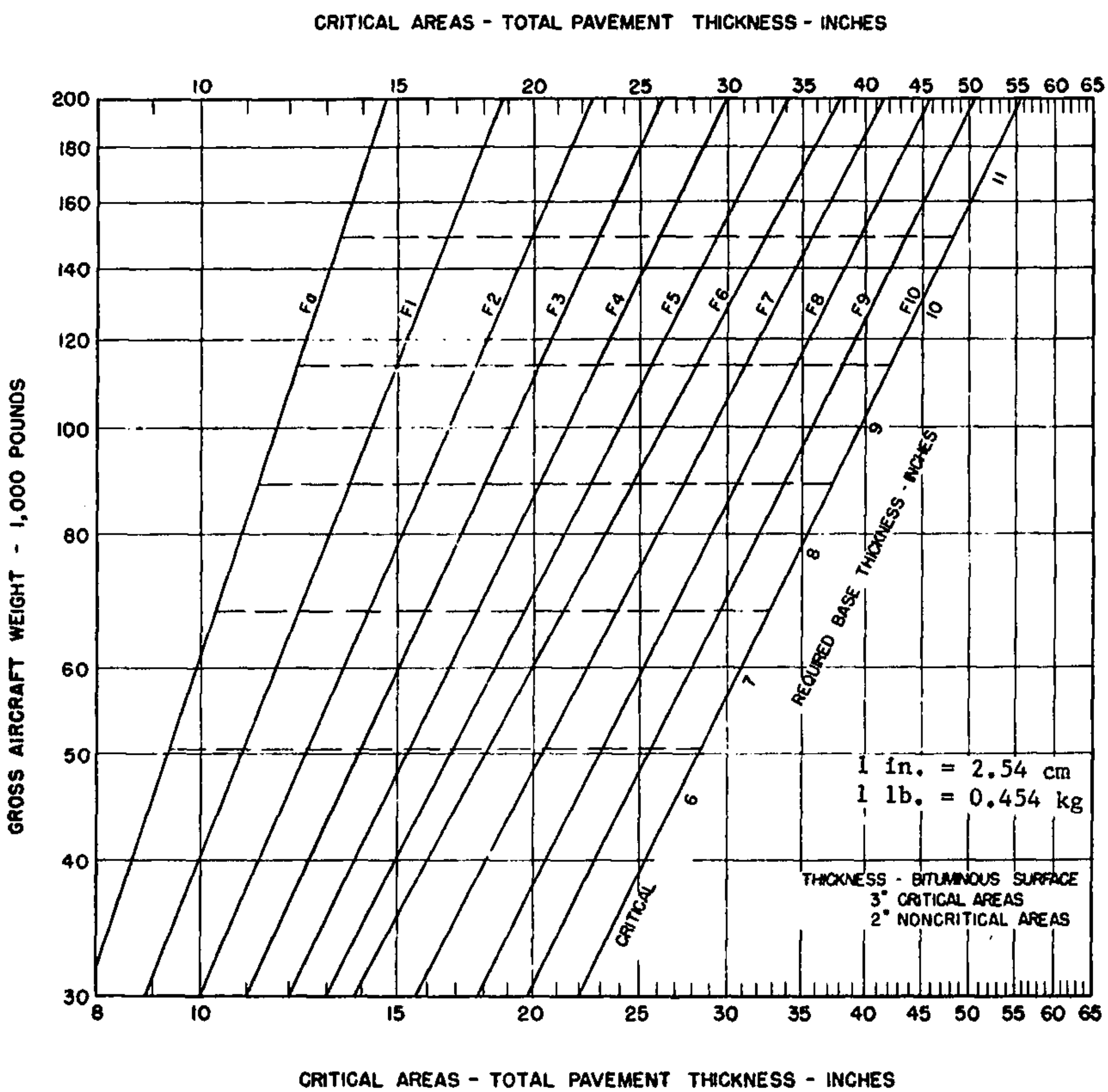


FIGURE 6-1. FLEXIBLE PAVEMENT-EVALUATION CURVES - SINGLE WHEEL GEAR

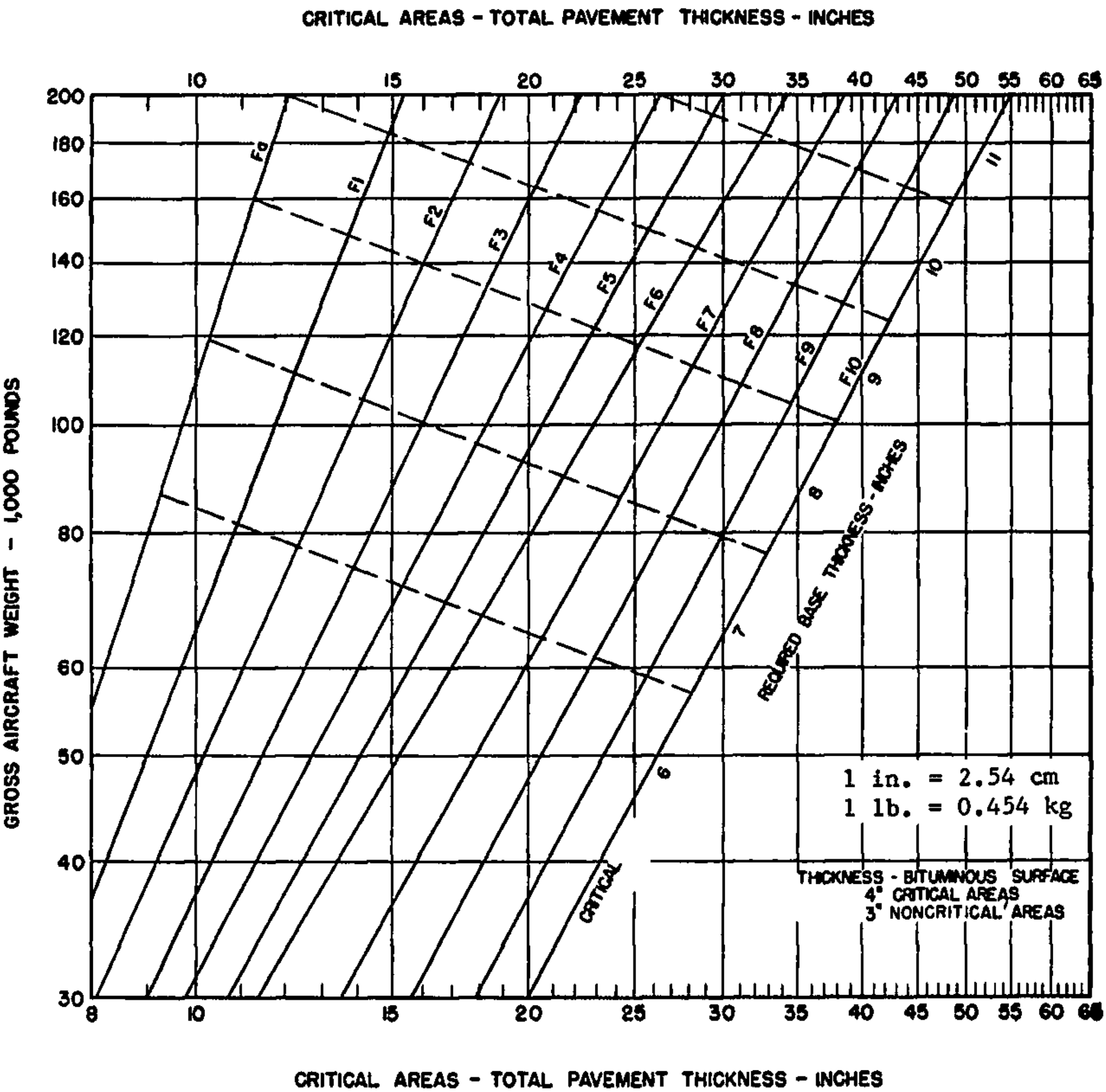


FIGURE 6-2. FLEXIBLE PAVEMENT - EVALUATION CURVES - DUAL WHEEL GEAR

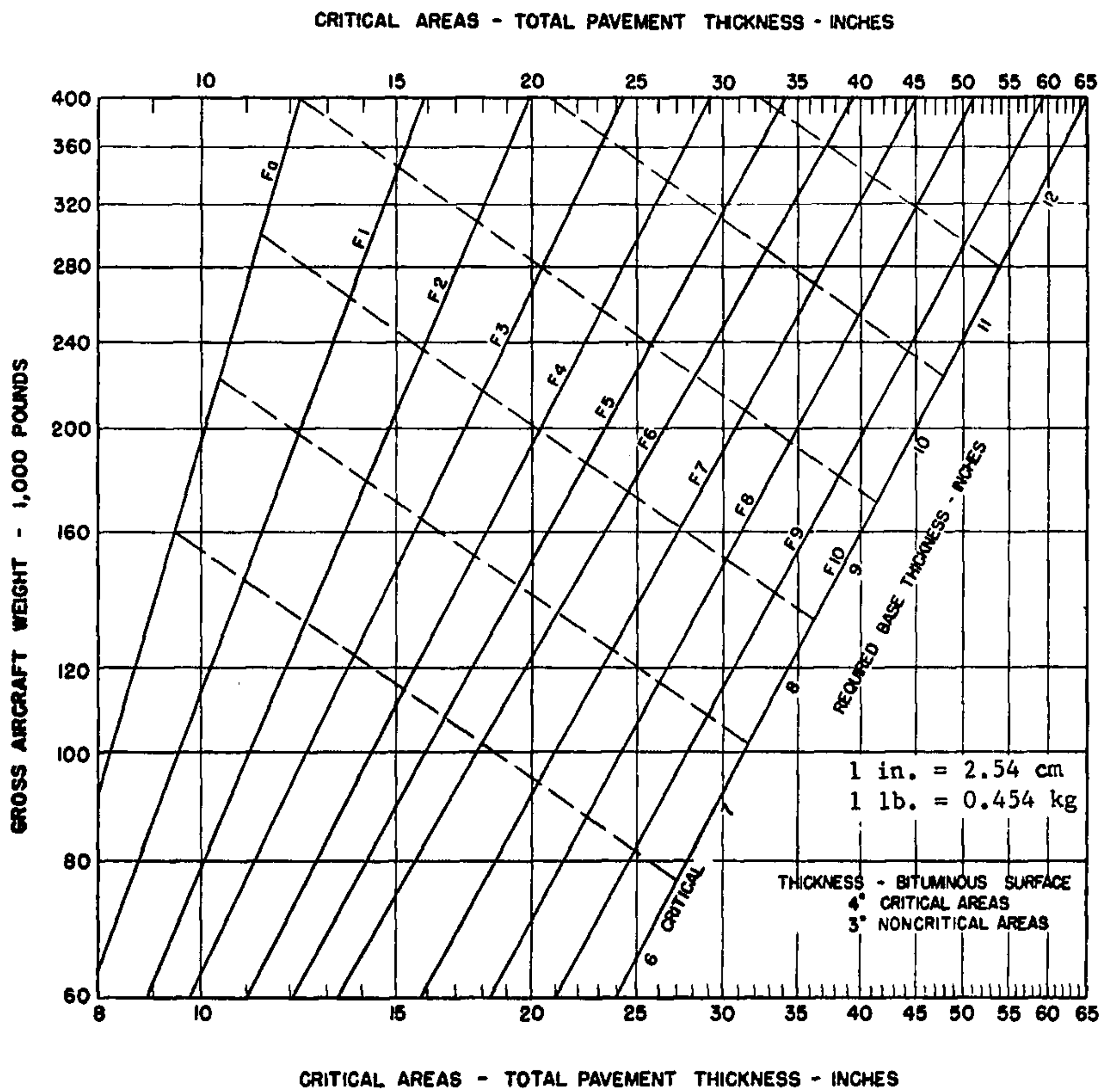
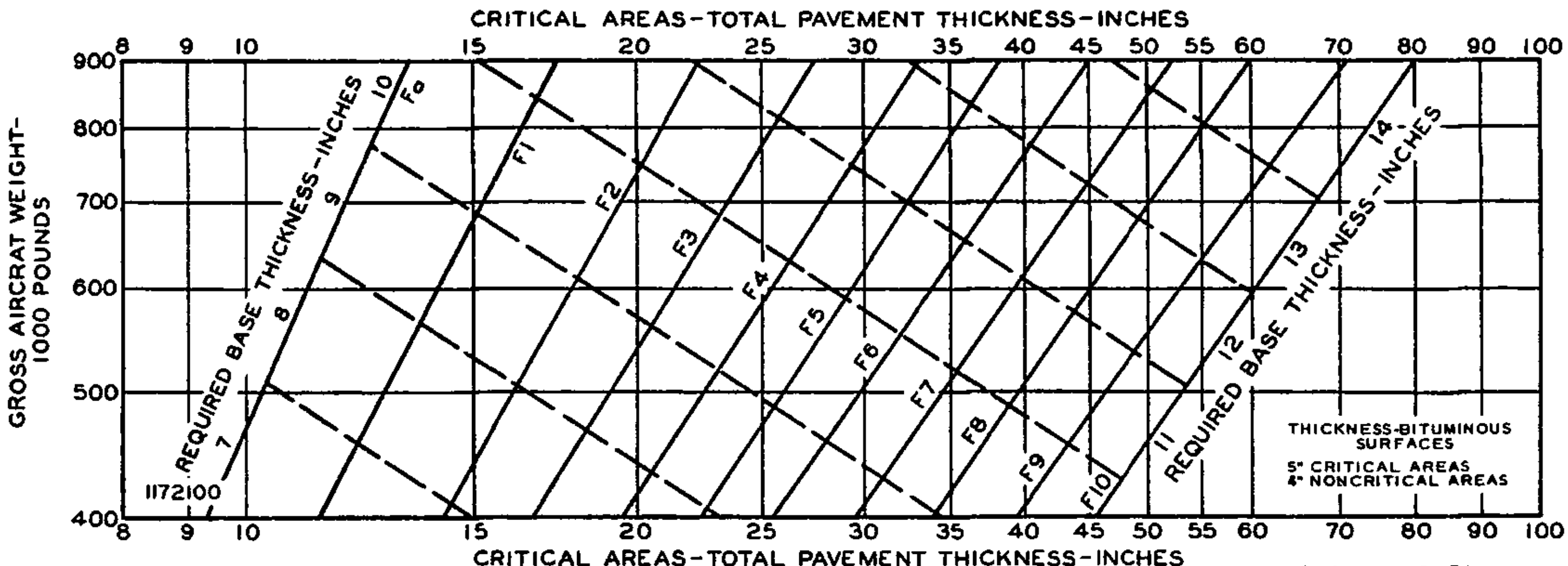
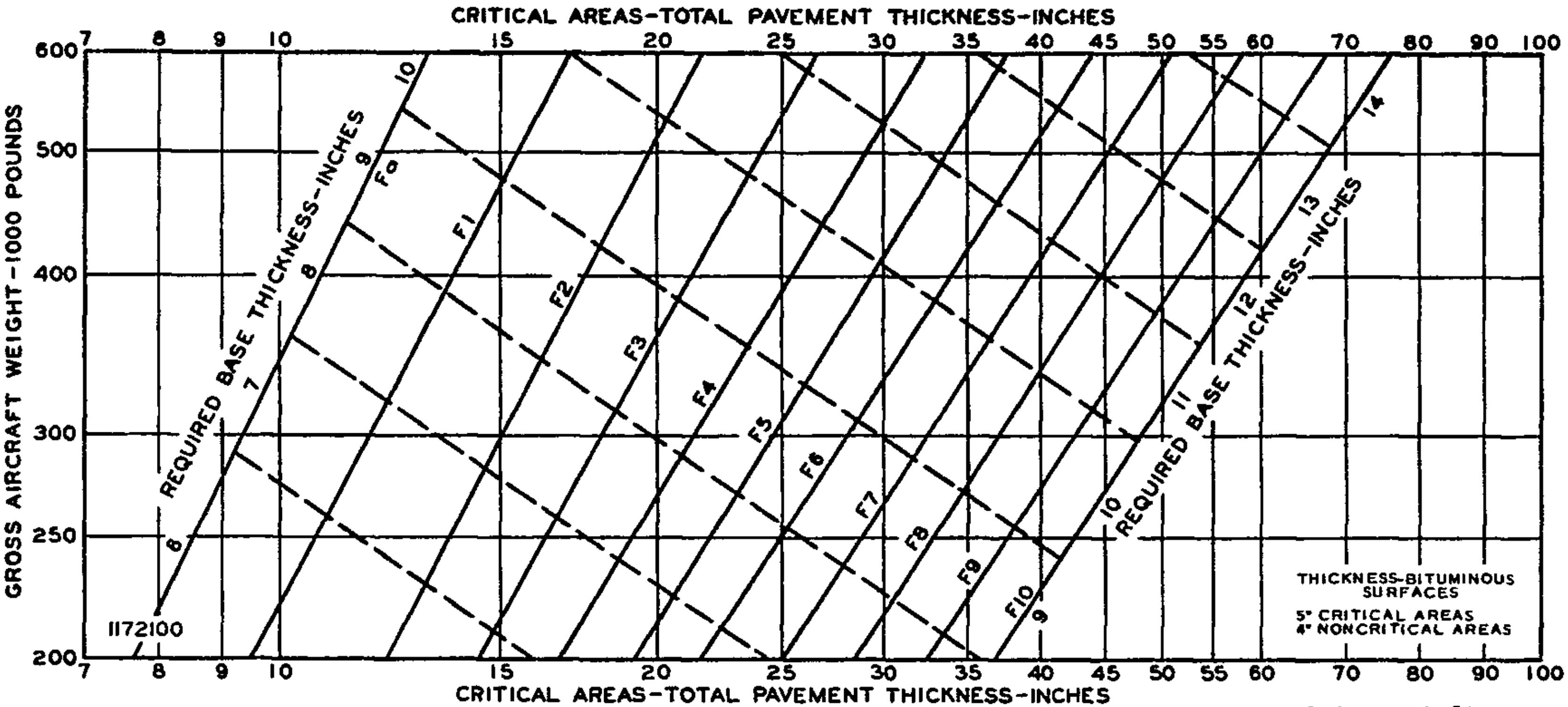


FIGURE 6-3. FLEXIBLE PAVEMENT - EVALUATION CURVES - DUAL TANDEM GEAR



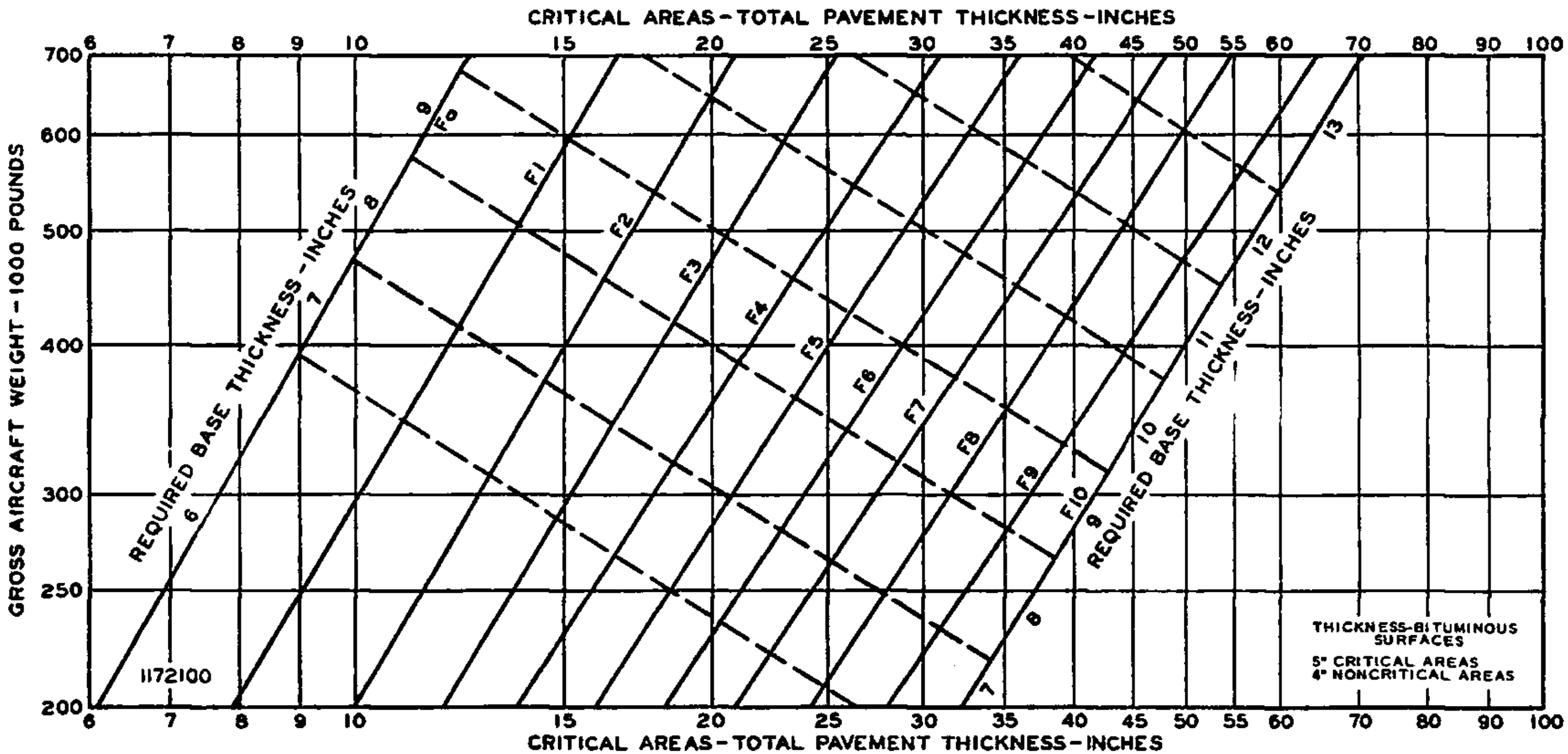
1 in. = 2.54 cm
1 lb. = 0.454 kg

FIGURE 6-4. FLEXIBLE PAVEMENT - EVALUATION CURVES - B-747



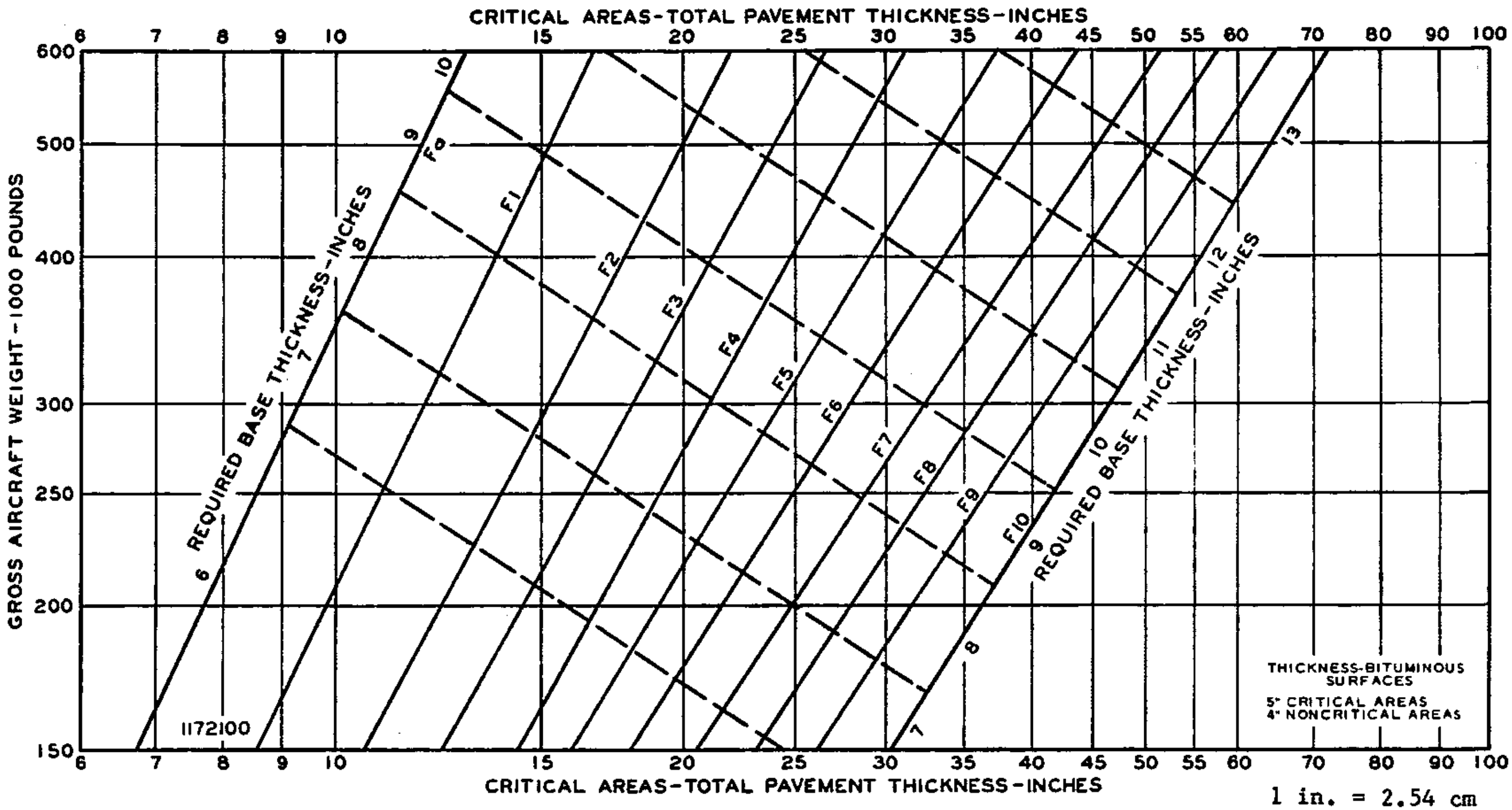
1 in. = 2.54 cm
1 lb. = 0.454 kg

FIGURE 6-5. FLEXIBLE PAVEMENT - EVALUATION CURVES - DC 10-10



1 in. = 2.54 cm
1 lb. = 0.454 kg

FIGURE 6-6. FLEXIBLE PAVEMENT - EVALUATION CURVES - DC 10-30



1 in. = 2.54 cm
1 lb. = 0.454 kg

FIGURE 6-7. FLEXIBLE PAVEMENT - EVALUATION CURVES - L-1011

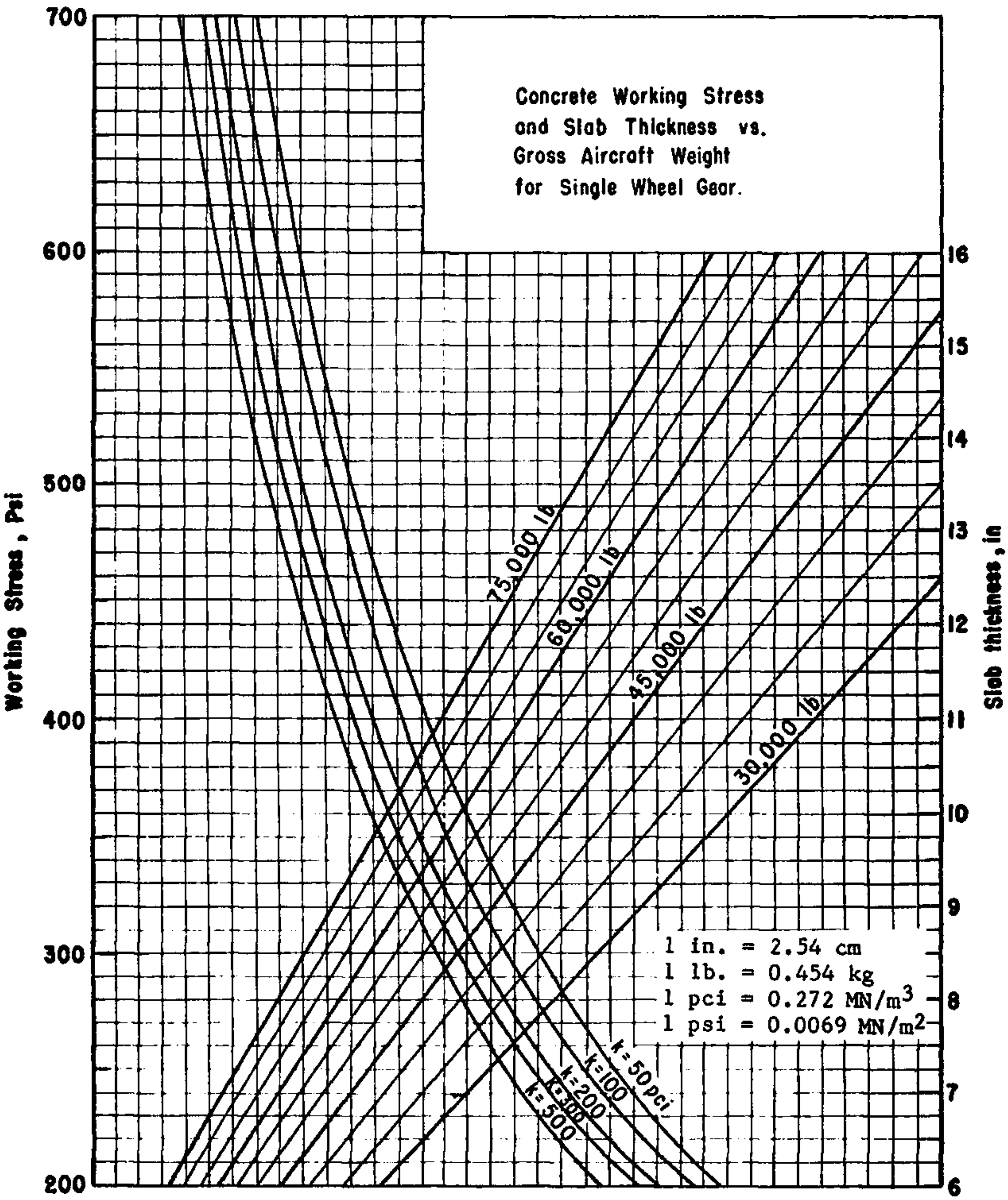


FIGURE 6-8. RIGID PAVEMENT - EVALUATION CURVES - SINGLE WHEEL GEAR

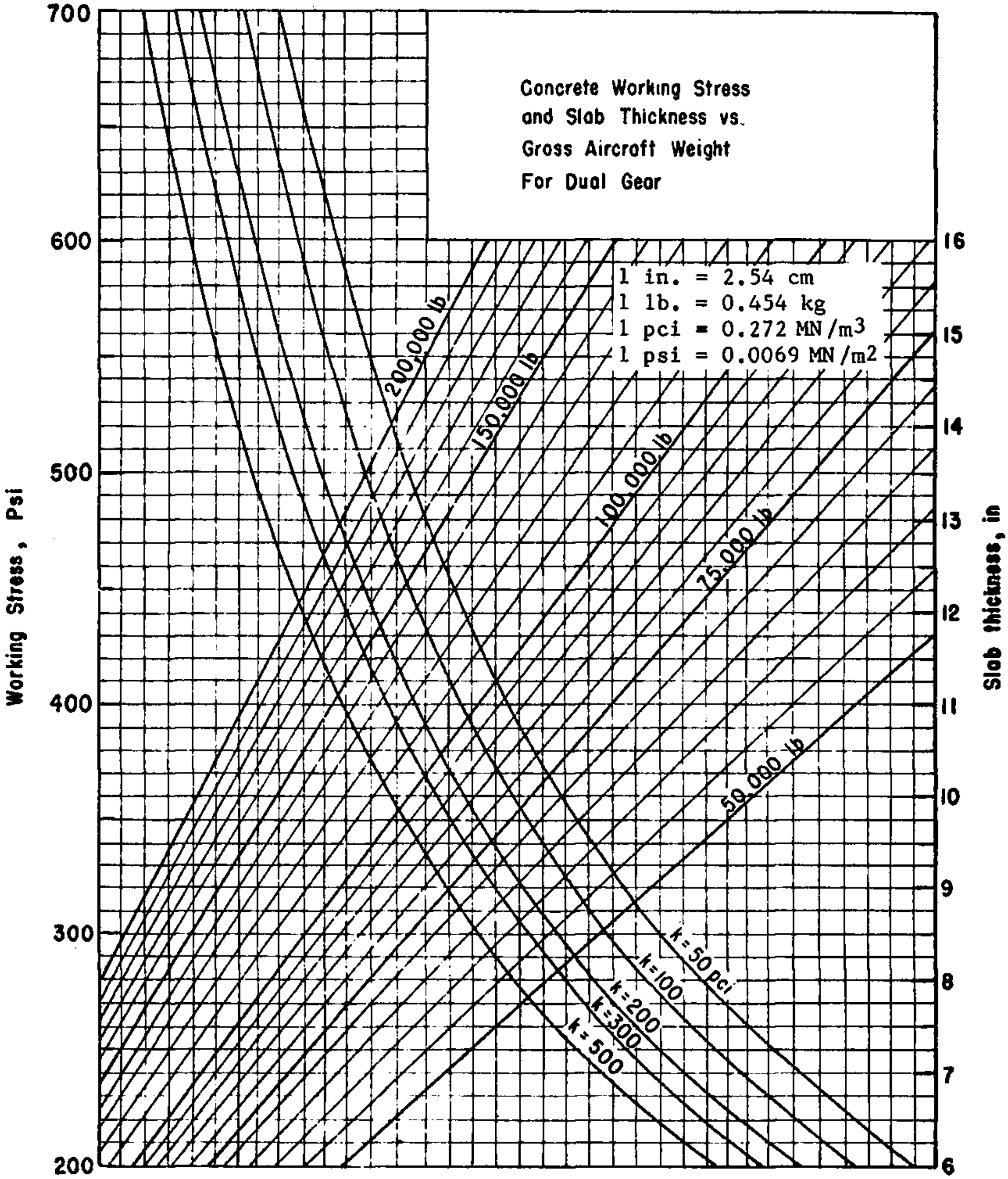


FIGURE 6-9. RIGID PAVEMENT - EVALUATION CURVES - DUAL WHEEL GEAR

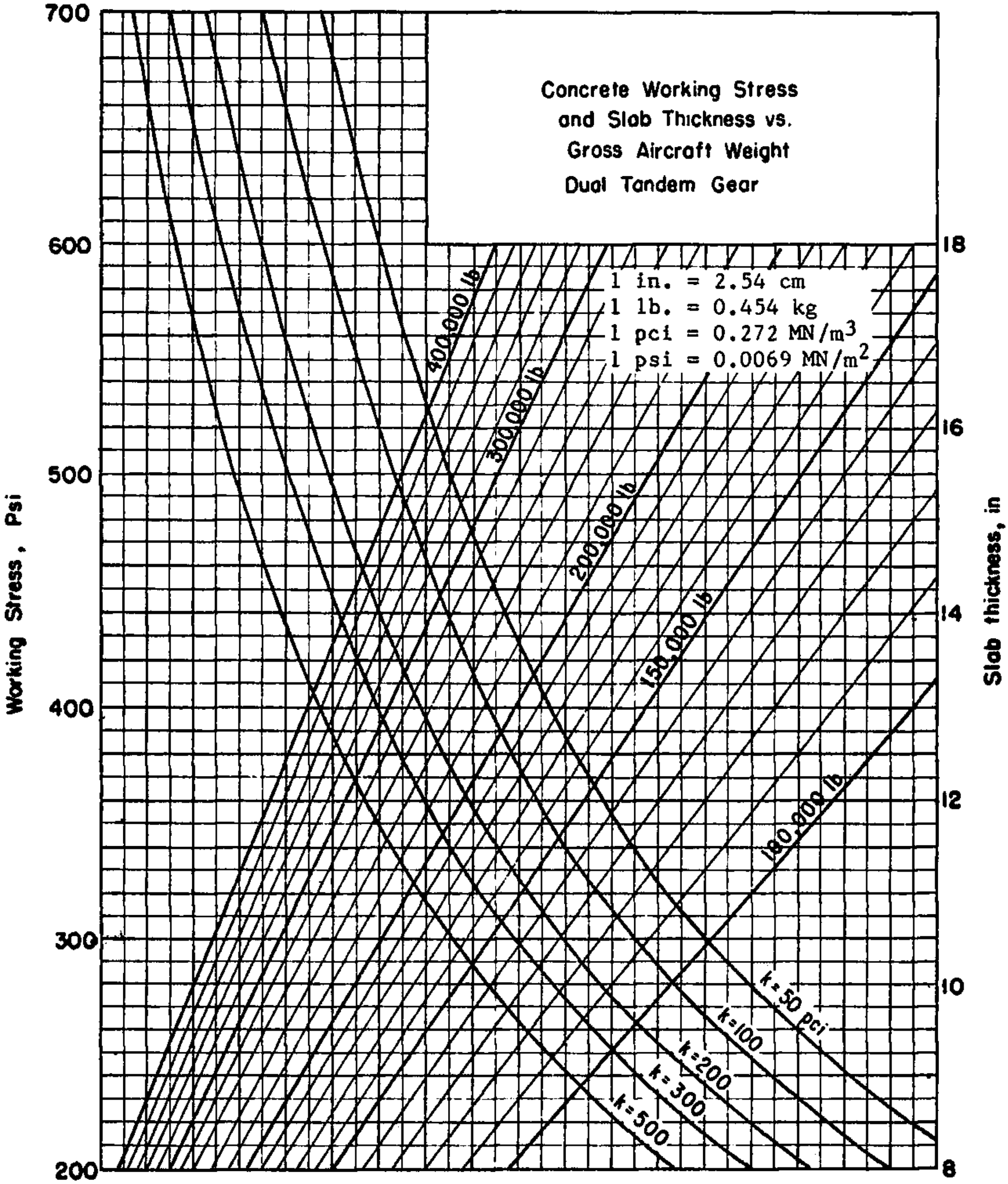


FIGURE 6-10. RIGID PAVEMENT - EVALUATION CURVES - DUAL TANDEM GEAR

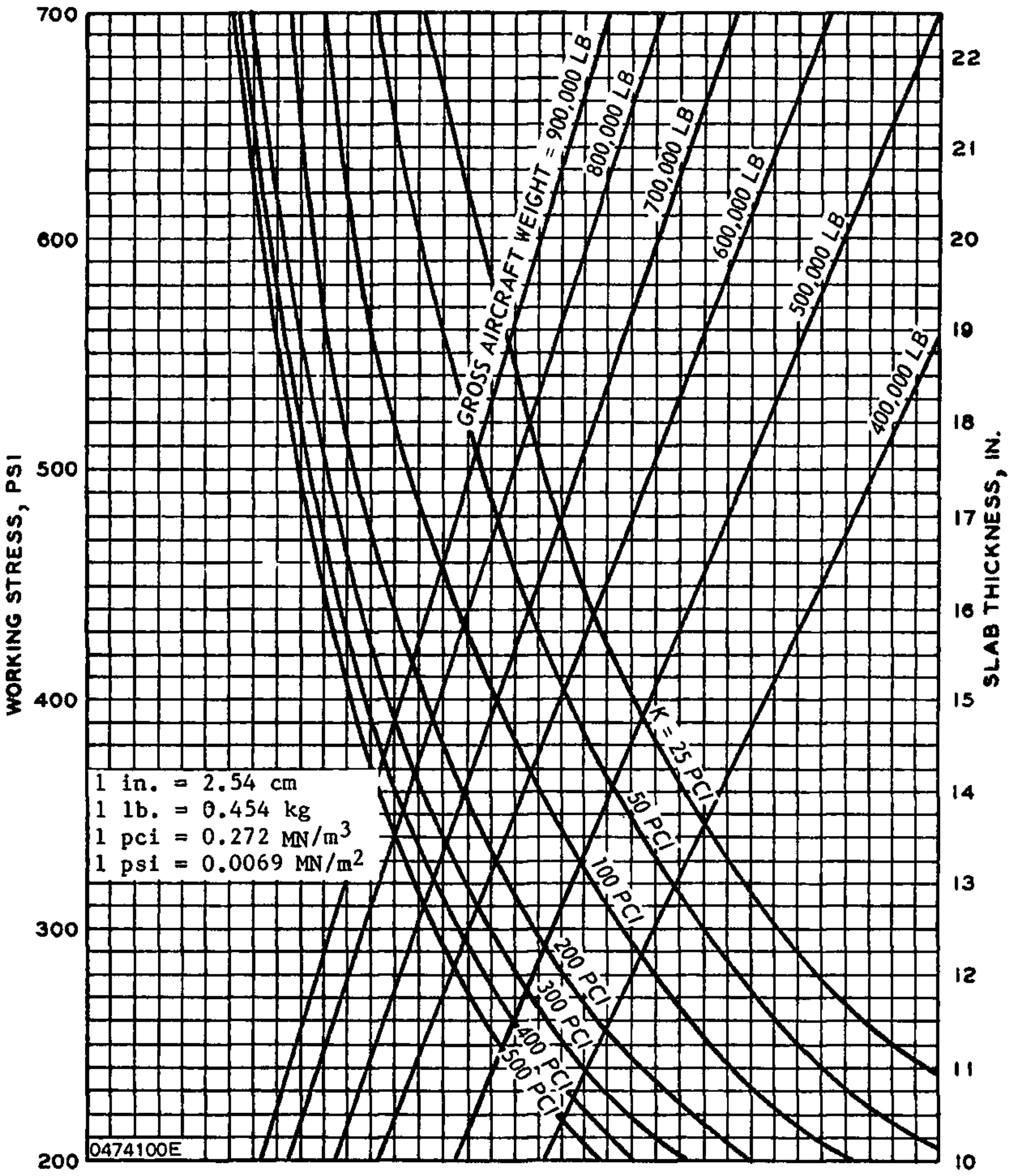


FIGURE 6-11. RIGID PAVEMENT - EVALUATION CURVES - B-747

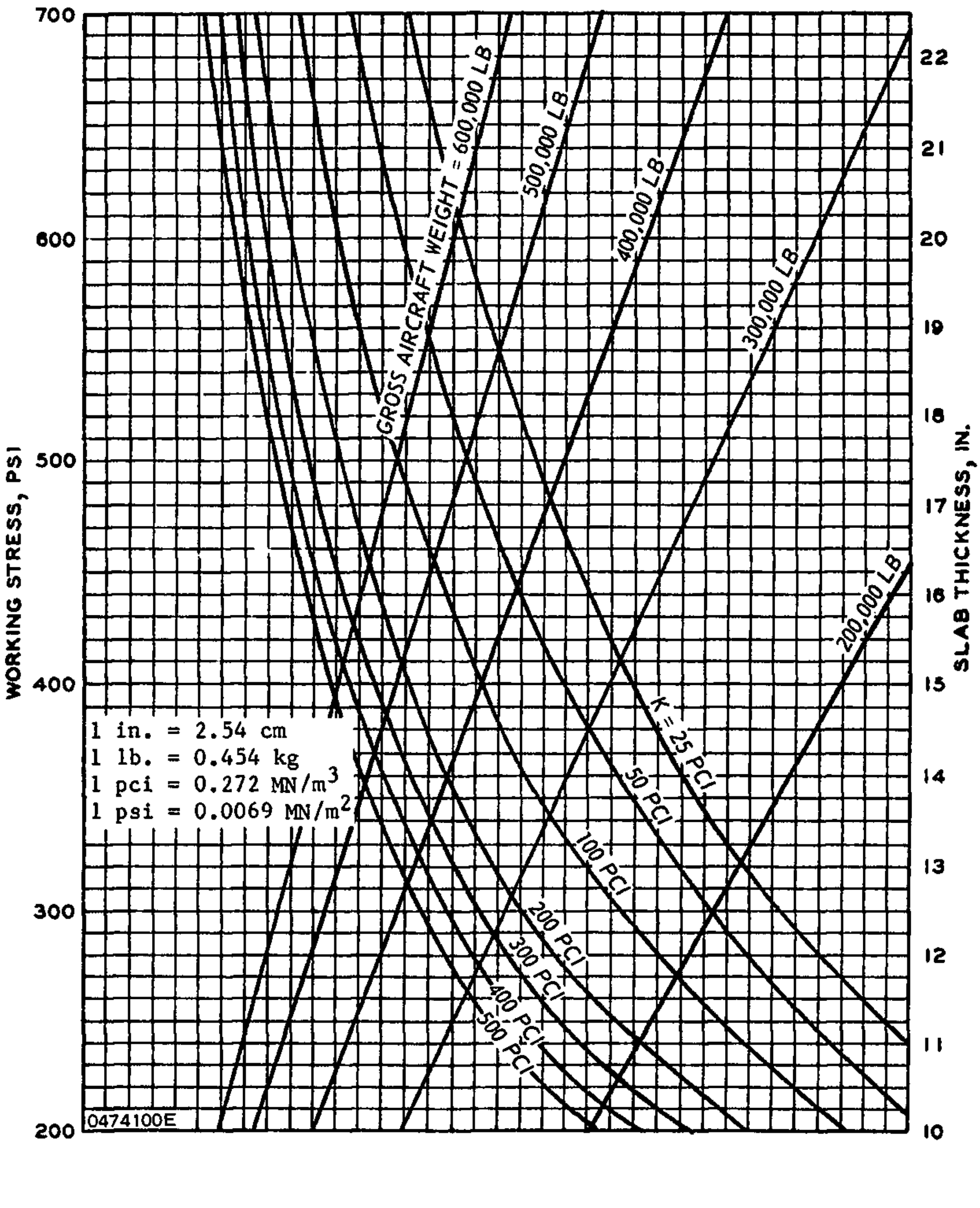


FIGURE 6-12. RIGID PAVEMENT - EVALUATION CURVES - DC 10-10

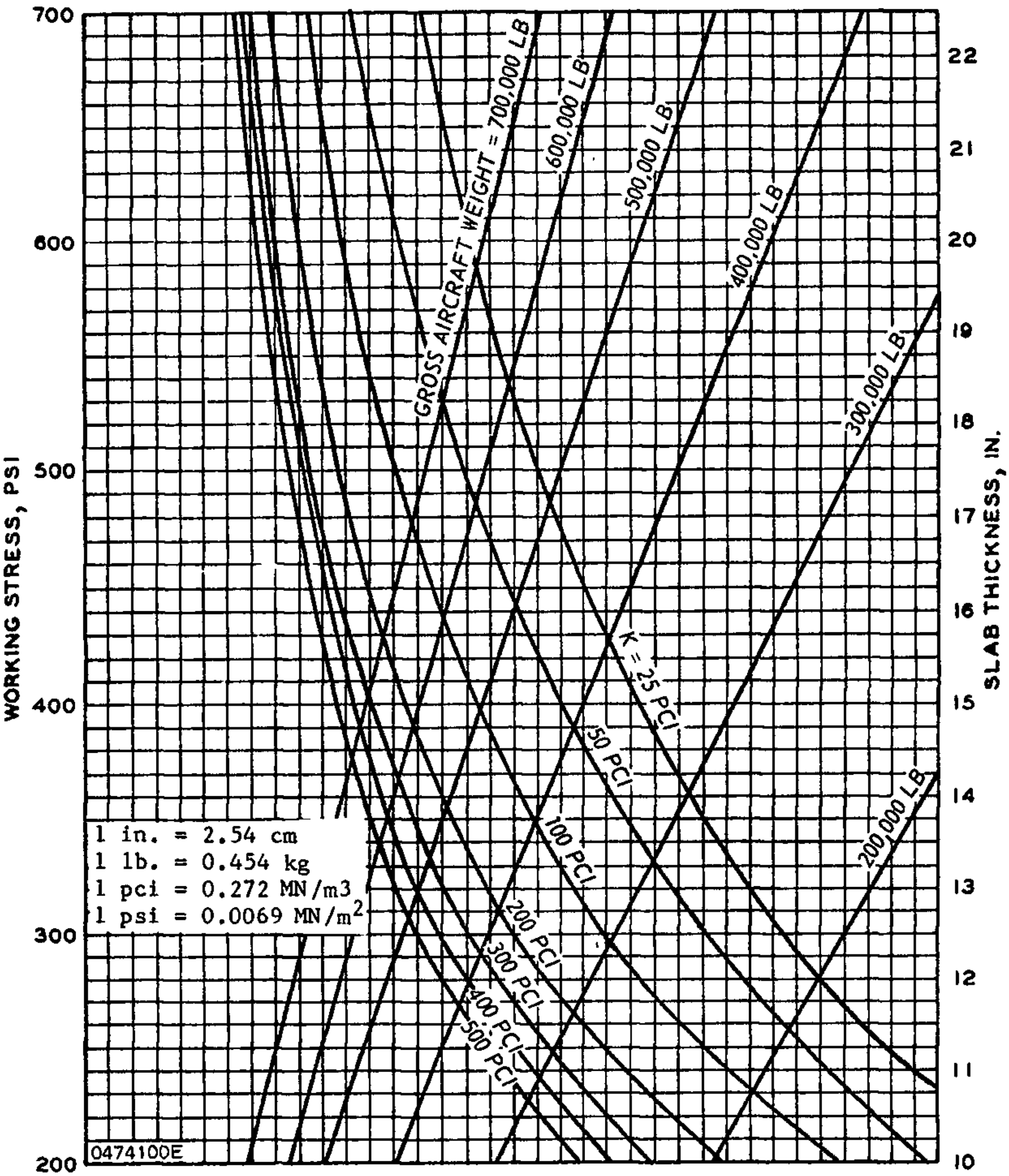


FIGURE 6-13. RIGID PAVEMENT - EVALUATION CURVES - DC 10-30

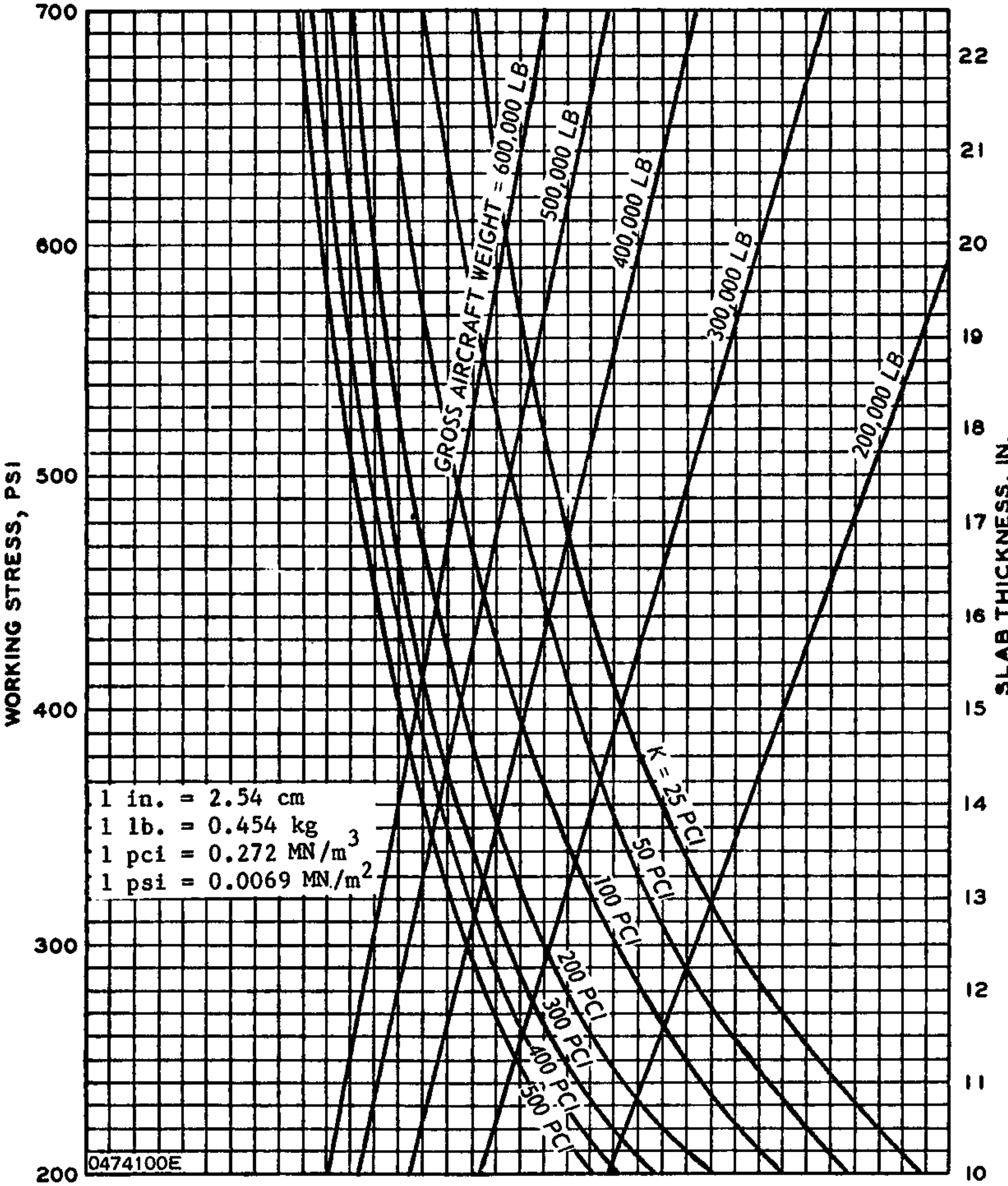


FIGURE 6-14. RIGID PAVEMENT - EVALUATION CURVES - L-1011

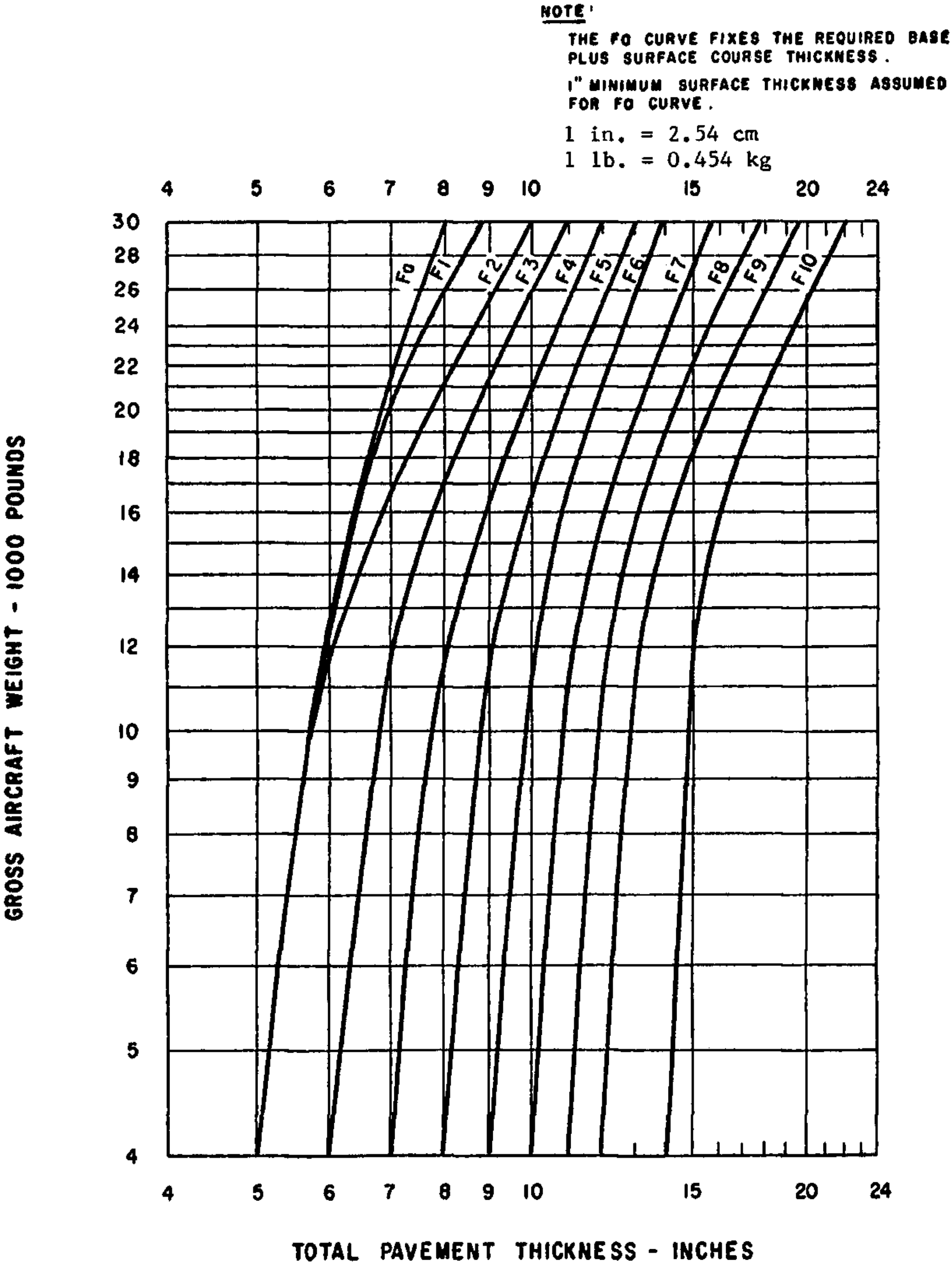


FIGURE 6-15. FLEXIBLE PAVEMENT - EVALUATION CURVES - LIGHT LOAD AIRCRAFT

91. EVALUATIONS FOR PLANNING AND DESIGN. Evaluations of existing pavements to be used in planning or designing improvements should be based on the method which will be used to design those improvements. The criteria presented in Chapters 3, 4, and 5, or other methods approved by the FAA may be used. The procedures to be followed in evaluating pavements according to the design criteria contained in this advisory are as follows.

a. Evaluation Steps.

(1) Site Inspection. This may include, in addition to the examination of the existing drainage conditions and drainage facilities of the site, consideration of the drainage area, outfall, water table, area development, etc. Evidence of frost action should also be observed. The principles set forth in Chapter 2 of this circular and in AC 150/5320-5, Airport Drainage, apply.

(2) Records Research and Evaluation. This step may, at least in part, precede step (1) above. This step is accomplished by a thorough review of construction data and history, design considerations, specifications, testing methods and results, as-built drawings, and maintenance history. Weather records and the most complete traffic history available are also parts of a usable records file. When soil, moisture, and weather conditions conducive to detrimental frost action exist, an adjustment to the evaluation may be required.

(3) Sampling and Testing. The need for and scope of physical tests and materials analyses will be based on the findings made from the site inspection, records research, and type of evaluation. A complete evaluation for detailed design will require more sampling and testing than, for example, an evaluation intended for use in a master plan. Sampling and testing is intended to provide information on the thickness, quality, and general condition of the pavement elements.

(4) Evaluation Report. Analysis of steps (1), (2), and (3), should culminate in the assignment of load carrying capacity to the pavement sections under consideration. The analyses, findings, and test results should be incorporated in a permanent record for future reference. While these need not be in any particular form, it is recommended that a drawing identifying area limits of specific pavement sections be included.

b. Direct Sampling Procedures. The basic evaluation procedure for planning and design will be visual inspection and reference to the FAA design criteria, supplemented by the additional sampling, testing, and research which the evaluation processes may warrant. For relatively new pavement constructed to FAA standards and without visible sign of wear or stress, strength may be based on inspection of the FAA Form 5100-1, Airport Pavement Design, and the as-constructed sections, with modification for any material variations or deficiencies of record. Where age or visible distress indicates the original strength no longer exists, further modification should be applied on the basis of judgment or a combination of judgment and supplemental physical testing. For pavements which consist of sections not readily comparable to FAA design standards, evaluation should be based on FAA standards after materials comparison and equivalencies have been applied.

(1) Flexible Pavements. Laboratory or field CBR tests may be useful in supplementing soil classification tests. Figure 6-16 shows the approximate relationship between the subgrade classification formerly used by the FAA and CBR.

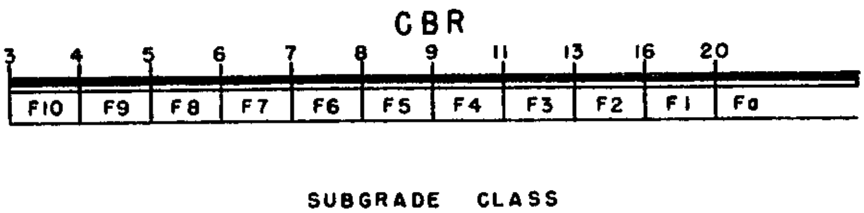


FIGURE 6-16. CBR - FAA SUBGRADE CLASS COMPARISONS

Laboratory CBR tests should be performed on soaked specimens in accordance with ASTM D-1883, Bearing Ratio of Laboratory-Compacted Soils. Field CBRs should be performed in accordance with the procedure given in The Asphalt Institute Manual Series 10 (MS-10), Soils Manual.

(a) Field CBR tests on existing pavements less than 3 years old may not be representative unless the subgrade moisture content has stabilized. The evaluation process assumes a soaked CBR is and will not give reliable results if the subgrade moisture content has not reached the ultimate in situ condition.

(b) Conversion of "F" subgrade classification factors to CBR is permissible where CBR tests are not feasible.

(c) The thickness of the various layers in the flexible pavement structure must be known in order to evaluate the pavement. Thicknesses may be determined from borings or test pits. As-built drawings and records can also be used to determine thicknesses if the records are sufficiently complete and accurate.

(2) Rigid Pavements. The evaluation requires the determination of the thickness of the component layers, the flexural strength of the concrete, and the modulus of subgrade reaction.

(a) The thickness of the component layers is usually available from construction records. Where information is not available or of questionable accuracy, thicknesses may be determined by borings or test pits in the pavement.

(b) The flexural strength of the concrete is most accurately determined from test beams sawed from the existing pavement and tested in accordance with ASTM C-78. Sawed beams are expensive to obtain and costs incurred in obtaining sufficient numbers of beams to establish a representative sample may be prohibitive. Construction records may be used as a source of concrete flexural strength data, if available. The construction data will probably have to be adjusted for age as concrete strength increases with time. Strength-age relationships can be found in Portland Cement Association, Engineering Bulletin, Design of Concrete Airport Pavement. An approximate relationship between concrete compressive strength and flexural strength exists and can be computed by the following formula:

$$R = 9\sqrt{f'_c}$$

where R = flexural strength

f'_c = compressive strength

Tensile splitting tests (ASTM C-496) can be used to determine an approximate value for flexural strength. Tensile splitting strength should be multiplied by about 1.5 to approximate the flexural strength. It should be pointed out that the relationships between flexural strength and compressive strength or tensile splitting strengths are approximate and considerable variations are likely.

(c) The modulus of subgrade reaction is determined by plate bearing tests performed on the subgrade. These tests should be made in accordance with the procedures established in AASHTO T 222. An important part of the test procedure for determining the subgrade reaction modulus is the correction for soil saturation which is contained in the prescribed standard. The normal application utilizes a correction factor determined by the consolidation testing of samples at in situ and saturated moisture content. For evaluation of older pavement, where evidence exists that the subgrade moisture has stabilized or varies through a limited range, the correction for saturation is not necessary. If a field plate bearing test is not practical, the modulus of subgrade reaction may be estimated by using Table 2-2 in Chapter 2 of this circular. Fortunately, pavement evaluation is not too sensitive to the modulus of subgrade reaction.

(d) Subbases will require an adjustment to the modulus of subgrade reaction. The thickness of the subbase is required to calculate a k value for a subbase. The subbase thickness can be determined from construction records or from borings. The guidance contained in Chapter 3, Section 3 should be used in assigning a k value to a subbase.

c. Materials Comparison and Equivalencies.

(1) For the purposes of design and evaluation, flexible pavements are assumed to be constructed of asphaltic concrete surfacing, granular base, and granular subbase courses of a predetermined quality. When the materials in a pavement structure to be evaluated are at variance with these assumptions, the materials will have to be compared and equated to a standard section. The nonstandard sections after conversion have to be checked for load carrying capacity based on two requirements.

(a) Surface plus base course thickness.

(b) Total pavement section thickness.

The requirement yielding the lesser strength will control the evaluation.

(2) Equivalency factor ranges shown in Tables 3-2 and 3-3 for subbase and base are recommended for evaluation purposes. The actual value selected will depend on the composition, quality, and condition of the layer.

(3) Broken bituminous surface course (shrinkage cracks due to age and weathering, without evidence of base failure) shall be evaluated as an equal thickness of nonstabilized base. A bituminous surface, with limited cracking and well maintained, may justify use of an equivalency between the limits noted. This may apply also to stabilized base, but in no event shall base course be assigned a higher equivalency or value than is assigned to a base or surface material which is above it in the pavement structure.

(4) Conversion of material to a higher classification, such as subbase to base, will not be permitted, except that where excess stabilized base course (P-201 or P-304) exists immediately under a flexible surface which is deficient in thickness the stabilized material may be counted as an equal thickness of surface.

(5) The above equivalencies are essentially in keeping with the FAA design system. The equivalencies are considered to be generally applicable and where area experience or physical test results show that other values are valid, they may be substituted for the values recommended here.

d. Application of Procedures.

(1) Flexible Pavements. After all of the evaluation parameters of the existing flexible pavement have been established using the guidance given in the above paragraphs, the evaluation process is essentially the reverse of the design procedure. The design curves presented in Chapter 3 or 5 are used to determine the load carrying capacity of the existing pavement. Required inputs are subgrade and subbase CBR values, thicknesses of surfacing, base and subbase courses and an annual departure level. Several checks must be performed to determine the load carrying capacity of a flexible pavement. The calculation which yields the lowest allowable load will control the evaluation.

(a) Total Pavement Thickness. Enter the lower abscissa of the appropriate design curve in Chapter 3 or 5 with the total pavement thickness of the existing pavement. Make a vertical projection to the annual departure level line. For light load pavements, Chapter 5, a single pivot line is used. At the point of intersection between the vertical projection and the departure level line, or single pivot line in the case of light load pavements, make a horizontal projection across the design curve. Enter the upper abscissa with the CBR value of the subgrade. Make a vertical projection downward until it intersects the horizontal projection made previously. The point of intersection of these two projections will be in the vicinity of the load lines on the design curves. An allowable load is read by noting where the intersection point falls in relation to the load lines.

(b) Thickness of Surfacing and Base. The combined thickness of surfacing and base must also be checked to establish the load carrying capacity of an existing flexible pavement. This calculation requires the CBR of the subbase, the combined thickness of surfacing and base and the annual departure level as inputs. The procedure is the same as that described in subparagraph (a) above, except that the subbase CBR and combined thickness of surfacing and base are used to enter the design curves.

(c) Deficiency in Base Course Thickness. The thickness of the existing base course should be compared with the minimum base course thicknesses shown in Figure 3-12. Inputs for use of this curve are total pavement thickness and subgrade CBR. Enter the left ordinate of Figure 3-12 with the total pavement thickness. Make a horizontal projection to the appropriate subgrade CBR line. At the point of intersection of the horizontal projection and the subgrade CBR line, make a vertical projection down to the lower abscissa and read the minimum base course thickness. Notice that the minimum base course thickness is 6 inches (15 cm). If there is a deficiency in the thickness of the existing base course, the pavement should be closely monitored for signs of distress. The formulation of plans for overlaying the pavement to correct the deficiency should be considered.

(d) Deficiency in Surfacing Thickness. The thickness of the existing surface course should be compared with that shown on the appropriate design curve. If the existing surface course is thinner than that given on the design curve, the pavement should be closely observed for surface failures. It is recommended that planning to correct the deficiency in surfacing thickness be considered.

(2) Rigid Pavements. The evaluation of rigid pavements for aircraft weighing in excess of 30,000 pounds (13 600 kg) requires concrete flexural strength, k value of the foundation, slab thickness, and annual departure level as inputs. The rigid pavement design curves in Chapter 3 are used to

establish load carrying capacity. The design curves are entered on the left ordinate with the flexural strength of the concrete. A horizontal projection is made to the k value of the foundation. At the point of intersection of the horizontal projection and the k line, a vertical projection is made into the vicinity of the load lines. The slab thickness is entered on the appropriate departure level scale on the right side of the chart. A horizontal projection is made from the thickness scale until it intersects the previous vertical projection. The point of intersection of these projections will be in the vicinity of the load lines. The load carrying capacity is read by noting where the intersection point falls in relation to the load lines.

(3) Use of Results. If the evaluation is being used for planning purposes and the existing pavement is found to be deficient in accordance with the design standards given in Chapter 3 or 5, the sponsor should be notified as to the deficiency and consideration should be given to planning corrective action. If the evaluation is being used as a part of the design for a project to reconstruct or upgrade the facility, the procedures given in Chapters 3, 4, or 5 should be used to design the reconstruction or overlay project. In this instance the main concern is not the load carrying capacity but rather the difference between the existing pavement structure and the section that is needed to support forecast traffic.

92. EVALUATIONS USING NONDESTRUCTIVE TESTING. Nondestructive testing (NDT) provides a means of evaluating pavements which tends to remove some of the subjective judgment needed in other evaluation procedures. FAA Advisory Circular 150/5370-11, Use of Nondestructive Testing Devices in the Evaluation of Airport Pavements, contains guidance on nondestructive testing. The major advantages of nondestructive testing are: the pavement is tested in place under actual conditions of moisture, density, etc.; the disruption of traffic is minimal; and the need for destructive tests is minimized. Research efforts are continuing in the area of nondestructive testing to broaden its application. Several different NDT procedures are available in addition to that described in AC 150/5370-11. These other procedures may be used when approved by FAA.

APPENDIX 1. FAA METHOD OF SOIL CLASSIFICATION

1. BACKGROUND. The FAA method of soil classification which was used prior to the adoption of the Unified Soil Classification System is presented in this appendix. The reason for including the method in this appendix is t allow a smooth transition between the FAA and Unified methods. Also many past records contain references to the FAA method and this appendix allows the reader to converse in the FAA classification method.

2. SOIL CLASSIFICATION.

a. While the results of individual tests indicate certain physical properties of the soil, the principal value is derived from the fact that, through correlation of the data so obtained, it is possible to prepare an engineering classification of soils related to their field behavior. Such a classification is presented in Figure 1.

b. The soil classification requires basically the performance of three tests -- the mechanical analysis, determination of the liquid limit, and determiniation of the plastic limit. Tests for these properties have been utilized for many years as a means of evaluating soil for use in the construction of embankments and pavement subgrades. These tests identify a particular soil as having physical properties similar to those of a soil whose performance and behavior are known. Therefore, the test soil can be expected to possess the same characteristics and degree of stability under like conditions of moisture and climate.

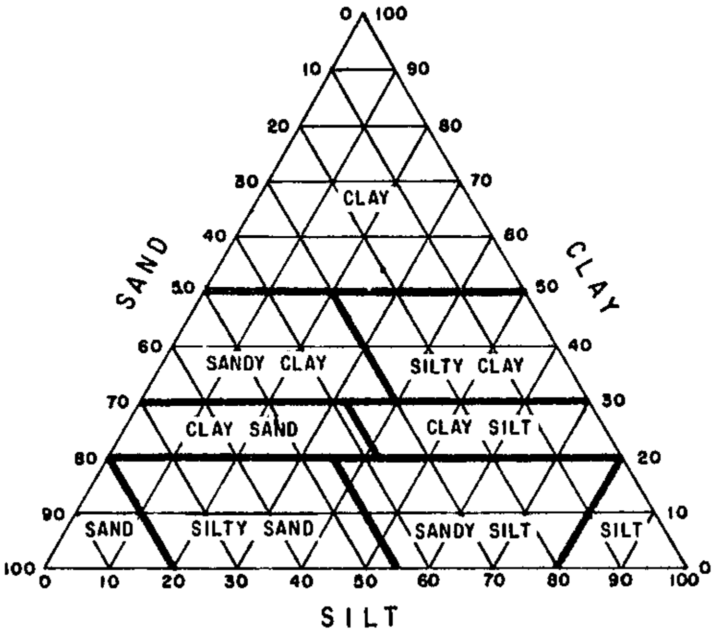
c. As can be discerned from Figure 1, the mechanical analyses provide the information to permit separation of the granular soils from the fine-grained soils; whereas, the several groups are arranged in order of increasing values of liquid limit and plasticity index. The division between granular and fine-grained soils is made upon the requirement that granular soils must have less than 35 percent of silt and clay combined. Determination of the sand, silt, and clay fractions is made on that portion of the sample passing the No. 10 sieve because this is considered to be the critical portion with respect to changes in moisture and other climatic influences. The classification of the soils with respect to different percentages of sand, silt, and clay is shown in Figure 2.

(1) Group E-1 includes well-graded, coarse, granular soils that are stable even under poor drainage conditions and are not generally subject to detrimental frost heave. Soils of this group may conform to well-graded sands and gravels with little or no fines. If frost is a factor, the soil should be checked to determine the percentage of the material less than 0.02 mm in diameter.

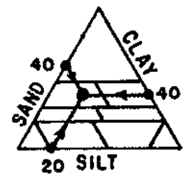
Soil group		Mechanical analysis: Material retained on No. 10 sieve—percent [1]	Material finer than No. 10 sieve—percent: Coarse sand, passing No. 10; retained on No. 40	Fine sand, passing No. 40 retained on No. 200	Combined silt and clay; passing No. 200	Liquid limit	Plasticity index
Granular	E-1	0-45	40+	60−	15−	25−	6−
Granular	E-2	0-45	15+	85−	25−	25−	6−
Granular	E-3	0-45	....	....	25−	25−	6−
Granular	E-4	0-45	....	....	35−	35−	10−
Fine grained	E-5	0-55	....	....	45−	40−	15−
Fine grained	E-6	0-55	....	....	45+	40−	10−
Fine grained	E-7	0-55	....	....	45+	50−	10-30
Fine grained	E-8	0-55	....	....	45+	60−	15-40
Fine grained	E-9	0-55	....	....	45+	40+	30−
Fine grained	E-10	0-55	....	....	45+	70−	20-50
Fine grained	E-11	0-55	....	....	45+	80−	30+
Fine grained	E-12	0-55	....	....	45+	80+	....
	E-13	Muck and peat—field examination					

[1] If percentage of material retained on the No. 10 sieve exceeds that shown, the classification may be raised, provided such material is sound and fairly well graded.

FIGURE 1. CLASSIFICATION OF SOILS FOR AIRPORT PAVEMENT CONSTRUCTION



EXAMPLE: 20% Silt, 40% Sand and 40% Clay.



Therefore the sample is a Sandy Clay.

FIGURE 2. TEXTURAL CLASSIFICATION OF SOILS

(2) Group E-2 is similar to Group E-1 but has less coarse sand and may contain greater percentages of silt and clay. Soils of this group may become unstable when poorly drained as well as being subject to frost heave to a limited extent.

(3) Groups E-3 and E-4 include the fine, sandy soils of inferior grading. They may consist of fine cohesionless sand or sand-clay types with a fair-to-good quality of binder. They are less stable than Group E-2 soils under adverse conditions of drainage and frost action.

(4) Group E-5 comprises all poorly graded soils having more than 35 percent but less than 45 percent of silt and clay combined. This group also includes all soils with less than 45 percent of silt and clay but which have plasticity indices of 10 to 15. These soils are susceptible to frost action.

(5) Group E-6 consists of the silts and sandy silts having zero-to-low plasticity. These soils are friable and quite stable when dry or at low moisture contents. They lose stability and become very spongy when wet and, for this reason, are difficult to compact unless the moisture content is carefully controlled. Capillary rise in the soils of this group is very rapid; and they, more than soils of any other group, are subject to detrimental frost heave.

(6) Group E-7 includes the silty clay, sand clay, clayey sands, and clayey silts. They range from friable to hard consistency when dry and are plastic when wet. These soils are stiff and dense when compacted at the proper moisture content. Variations in moisture are apt to produce a detrimental volume change. Capillary forces acting in the soil are strong, but the rate of capillary rise is relatively slow and frost heave, while detrimental, is not as severe as in the E-6 soils.

(7) Group E-8 soils are similar to the E-7 soils but the higher liquid limits indicate a greater degree of compressibility expansion, shrinkage, and lower stability under adverse moisture conditions.

(8) Group E-9 comprises the silts and clays containing micaceous and diatomaceous materials. They are highly elastic and very difficult to compact. They have low stability in both the wet and dry state and are subject to frost heave.

(9) Group E-10 includes the silty clay and clay soils that form hard clods when dry and are very plastic when wet. They are very compressible, possess the properties of expansion,

shrinkage, and elasticity to a high degree and are subject to frost heave. Soils of this group are more difficult to compact than those of the E-7 or E-8 groups and require careful control of moisture to produce a dense, stable fill.

(10) Group E-11 soils are similar to those of the E-10 group but have higher liquid limits. This group includes all soils with liquid limits between 70 and 80 and plasticity indices over 30.

(11) Group E-12 comprises all soils having liquid limits over 80 regardless of their plasticity indices. They may be highly plastic clays that are extremely unstable in the presence of moisture, or they may be very elastic soils containing mica, diatoms, or organic matter in excessive amounts. Whatever the cause of their instability, they will require the maximum in corrective measures.

(12) Group E-13 encompasses organic swamp soils such as muck and peat which are recognized by examination in the field. In their natural state, they are characterized by very low stability and density and very high moisture content.

3. SPECIAL CONDITIONS AFFECTING FINE-GRAINED SOILS.

a. A soil may possibly contain certain constituents that will give test results which would place it, according to Figure 1, in more than one group. This could happen with soils containing mica, diatoms, or a large proportion of colloidal material. Such overlapping can be avoided by the use of Figure 3 in conjunction with Figure 1, with exception of E-5 soils, which should be classified strictly by Figure 1 and paragraph 2c(4).

b. Soils with plasticity indices higher than corresponding to the maximum liquid limit of the particular group are not of common occurrence. When encountered, they are placed in the higher numbered group as shown in Figure 3. This is justified by the fact that for equal liquid limits the higher the plasticity index; the lower the plastic limit (the plastic limit is the point when a slight increase in moisture causes the soil to rapidly lose stability).

4. COARSE MATERIAL RETAINED ON NO. 10 SIEVE. Only that portion of the sample passing the No. 10 sieve is considered in the above-described classification. Obviously, the presence of material retained on the No. 10 sieve should serve to improve the overall stability of the soil. For this reason, upgrading the soil from 1 to 2 classes is permitted when the percentage of the total sample retained on the No. 10 sieve

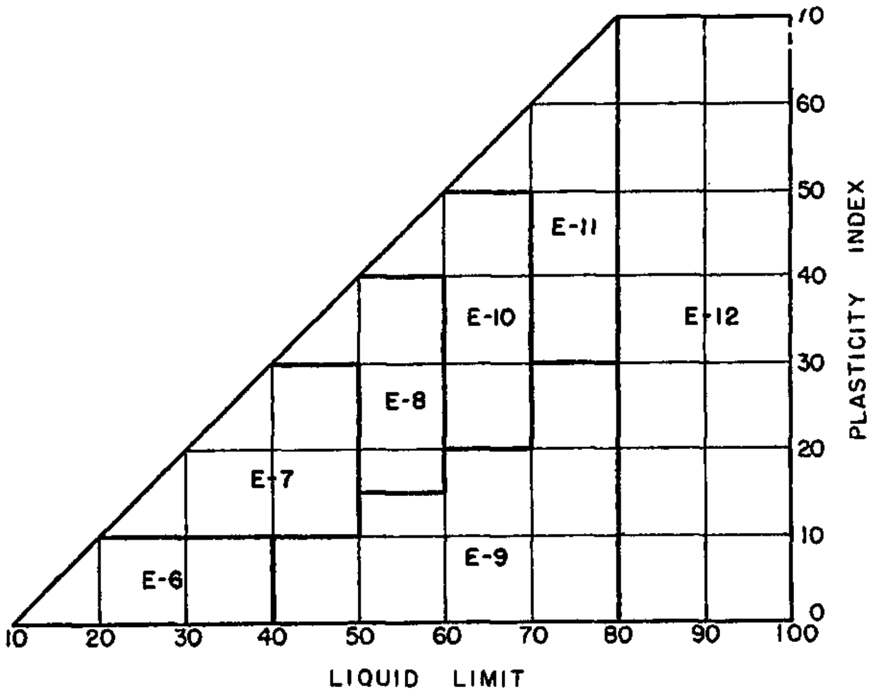


FIGURE 3. CLASSIFICATION CHART FOR FINE-GRAINED SOILS

exceeds 45 percent for soils of the E-1 to E-4 groups and 55 percent for the others. This applies when the coarse fraction consists of reasonably sound material which is fairly well graded from the maximum size down to the No. 10 sieve size. Stones or rock fragments scattered through a soil should not be considered of sufficient benefit to warrant upgrading.

5. SUBGRADE CLASSIFICATION.

a. For each soil group, there are corresponding subgrade classes. These classes are based on the performance of the particular soil as a subgrade for rigid or flexible pavements under different conditions of drainage and frost. The subgrade class is determined from the results of soil tests and the information obtained by means of the soil survey and a study of climatological and topographical data. The subgrade classes and their relationship to the soil groups are shown in Figure 4. The prefix "F" indicates subgrade classes for flexible pavements. These subgrade classes determine the total pavement thickness for a given aircraft load. A brief description of the classes will be presented here.

b. Subgrades classed as Fa furnish adequate subgrade support without the addition of subbase material. The soil's value as a subgrade material decreases as the number increases.

c. Good and poor drainage refer to the subsurface soil drainage.

(1) Poor drainage is defined for the purpose of this circular as soil that cannot be drained because of its composition or because of the conditions at the site. Soils primarily composed of silts and clay for all practical purposes are impervious; and as long as a water source is available, the soil's natural affinity for moisture will render these materials unstable. These fine-grained soils cannot be drained and are classified as poor drainage as indicated in Figure 4. A granular soil that would drain and remain stable except for conditions at the site, such as high water table, flat terrain, or impervious strata, should also be designated as poor drainage. In some cases, this condition may be corrected by the use of subdrains.

(2) Good drainage is defined as a condition where the internal soil drainage characteristics are such that the material can and does remain well drained resulting in a stable subgrade material under all conditions.

Soil Group	Subgrade Class		
	Good Drainage	Poor Drainage	
	No Frost or Frost	No Frost	Frost
E-1	Fa	Fa	F1
E-2	Fa	F1	F2
E-3	F1	F2	F3
E-4	F1	F2	F4
E-5		F3	F5
E-6		F4	F6
E-7		F5	F7
E-8		F6	F8
E-9		F7	F9
E-10		F8	F10
E-11		F9	F10
E-12		F10	F10
E-13	Not suitable for subgrade		

FIGURE 4. AIRPORT PAVING SUBGRADE CLASSIFICATION

d. There is a tendency to overlook the detrimental effects of frost in pavement design. The effects of frost are widely known; however, experience shows that all too often pavements are damaged or destroyed by frost that was not properly taken into account in the design. Most inorganic soils containing 3 percent or more of grains finer than 0.02 mm in diameter, by weight, are frost susceptible for pavement design purposes. The subgrade soil should be classified either as "No Frost" or "Frost" depending on one of the two following conditions:

(1) No frost should be used in the design when the average frost penetration anticipated is less than the thickness of the pavement section.

(2) Frost should be used when the anticipated average frost penetration exceeds the pavement sections. The design should consider including nonfrost susceptible material below the required subbase to minimize or eliminate the detrimental frost effect on the subgrade. The extent of the subgrade protection needed depends on the soil and the surface and subsurface environment at the site.

APPENDIX 2. DEVELOPMENT OF PAVEMENT DESIGN CURVES.

1. BACKGROUND.

a. The pavement design curves presented in this circular were developed using the California Bearing Ratio (CBR) method for flexible pavements and the Westergaard edge loading analysis for rigid pavements. The curves are constructed for the gross weight of the aircraft assuming 95% of the gross weight is carried on the main landing gear assembly and 5% of the gross weight is carried on the nose gear assembly. Aircraft traffic is assumed to be normally distributed across the pavement in the transverse direction. See FAA Research Report No. FAA-RD-74-36, Field Survey and Analysis of Aircraft Distribution on Airport Pavement. Pavements are designed on the basis of static load analysis. Impact loads are not considered to increase the pavement thickness requirements. See FAA Research Report No. FAA-RD-74-39, Pavement Response to Aircraft Dynamic Loads.

b. Generalized design curves are presented in Chapter 3 for single, dual, and dual tandem main landing gear assemblies. These generalized curves do not represent specific aircraft but are prepared for a range of aircraft characteristics which are representative of all civil aircraft except wide body. The aircraft characteristics assumed for each landing gear assembly are shown in Tables 1, 2, and 3.

2. RIGID PAVEMENTS.

a. The design of rigid airport pavements is based on the Westergaard analysis of an edge loaded slab resting on a dense liquid foundation. The edge loading stresses are reduced by 25 percent to account for load transfer across joints. Two different cases of edge loading are covered by the design curves. Figures 3-14 through 3-22 assume the landing gear assembly is either tangent to a longitudinal joint or perpendicular to a transverse joint, whichever produces the largest stress. Figures 3-23 through 3-29 are for dual tandem assemblies and have been rotated through an angle to produce the maximum edge stress. Computer analyses were performed for angles from 0 to 90 degrees in 10-degree increments. Single and dual wheel assemblies were analyzed for loadings tangent to the edge only as the stress is maximum in that position. Sketches of the various assembly positions are shown in Figure 1.

TABLE 1. SINGLE WHEEL ASSEMBLY

Gross Weight lbs.	(kg)	Tire Pressure psi	(MN/m^3)
30,000	(13 600)	75	(0.52)
45,000	(20 400)	90	(0.62)
60,000	(27 200)	105	(0.72)
75,000	(34 000)	120	(0.83)

TABLE 2. DUAL WHEEL ASSEMBLY

Gross Weight lbs.	(kg)	Tire Pressure psi	(MN/m^2)	Dual Spacing in.	(cm)
50,000	(22 700)	80	(0.55)	20	(51)
75,000	(34 000)	110	(0.76)	21	(53)
100,000	(45 400)	140	(0.97)	23	(58)
150,000	(68 000)	160	(1.10)	30	(76)
200,000	(90 700)	200	(1.38)	34	(86)

TABLE 3. DUAL TANDEM ASSEMBLY

Gross Weight lbs.	(kg)	Tire Pressure psi	(MN/m^2)	Dual Spacing in.	(cm)	Tandem Spacing in.	(cm)
100,000	(45 400)	120	(0.83)	20	(51)	45	(114)
150,000	(68 000)	140	(0.97)	20	(51)	45	(114)
200,000	(90 700)	160	(1.10)	21	(53)	46	(117)
300,000	(136 100)	180	(1.24)	26	(66)	51	(130)
400,000	(181 400)	200	(1.38	30	(76)	55	(140)

Specific design curves are presented for wide body aircraft. The aircraft characteristics are shown on the design curves.

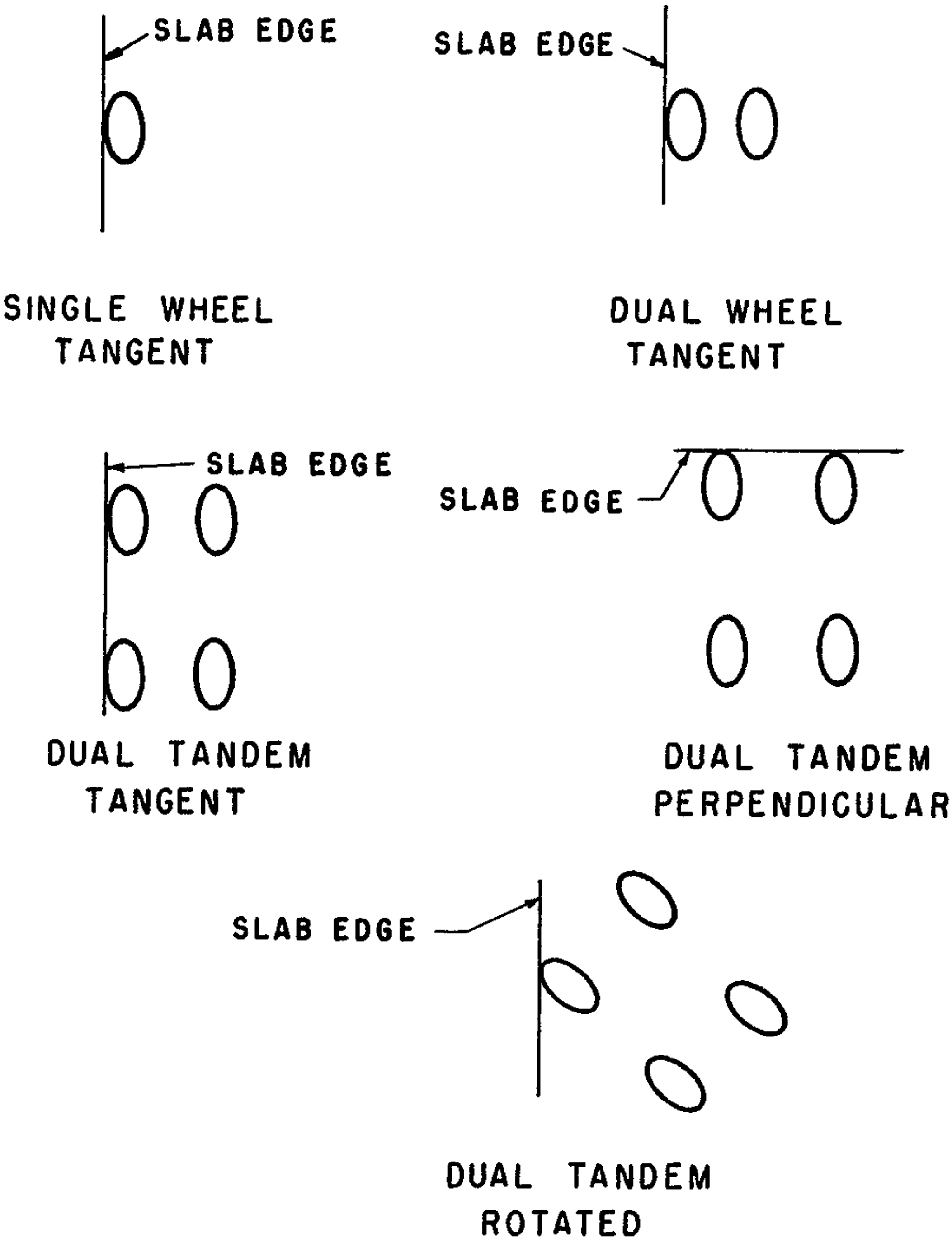


FIGURE 1. ASSEMBLY POSITIONS FOR RIGID PAVEMENT ANALYSIS

b. Fatigue effects are taken into consideration by converting traffic to coverages. The coverage concept provides a means of normalizing pavement performance data which can consist of a variety of wheel sizes, spacings and loads for pavements of different cross sections. For rigid pavements, coverage is a measure of the number of maximum stress applications occurring within the pavement slab due to the applied traffic. One coverage occurs when each point in the pavement within the limits of the traffic lane has experienced a maximum stress, assuming the stress is equal under the full tire print. Each pass (departure) of an aircraft can be converted to coverages using a single pass-to-coverage ratio which is developed assuming a normal distribution and applying standard statistical techniques. The pass-to-coverage ratios used in developing the rigid pavement design curves in Chapter 3 are given in Table 4. Annual departures are converted to coverages assuming a 20-year design life. Coverages are determined by multiplying annual departures by 20 and dividing that product by the pass-to-coverage ratio shown in Table 4.

TABLE 4. PASS-TO-COVERAGE RATIOS FOR RIGID PAVEMENTS

Design Curve	Pass-to-Coverage Ratio
Single Wheel	5.18
Dual Wheel	3.48
Dual Tandem	3.68
B-747	3.70
DC 10-10	3.64
DC 10-30	3.38
L-1011	3.62

c. After the conversion of departures to coverages, the slab thickness is adjusted in accordance with the fatigue curve developed by the Corps of Engineers from test track data and observation of in-service pavements. The fatigue relationship is applicable to the pavement structure; i.e., the slab and foundation are both included in the relationship. The thickness of pavement required to sustain 5,000 coverages of the design loading is considered to be 100 percent thickness. Any coverage level could have been selected as the 100 percent thickness level as long as the relative thicknesses for other coverage levels shown in Figure 2 is maintained.

d. Pavement thickness requirements for 5,000 coverages were computed for various concrete strengths and subgrade moduli. Allowable concrete stress for 5,000 coverages was computed by dividing the concrete flexural strength by 1.3 (analogous to a safety factor). The pavement thickness necessary to produce the allowable concrete stress for 5,000 coverages is then multiplied by the percent thickness shown in Figure 2 for other coverage levels.

3. FLEXIBLE PAVEMENTS.

a. The design curves for flexible pavements in Chapter 3 of this circular are based on the CBR method of design. The CBR is the ratio of the load required to produce a specified penetration of a standard piston into the material in question to the load required to produce the same penetration in a standard well-graded, crushed limestone. Pavement thicknesses necessary to protect various CBR values from shear failure have been developed through test track studies and observations of in-service pavements. These thicknesses have been developed for single wheel loadings. Assemblies other than single wheel are designed by computing the equivalent single wheel load for the assembly based on deflection. Once the equivalent single wheel is established, the pavement section thickness can be determined from the relationships discussed above.

b. Load repetitions are indicated on the design curves in terms of annual departures. The annual departures are assumed to occur over a 20-year life. In the development of the design curves, departures are converted to coverages. For flexible pavements, coverage is a measure of the number of maximum stress applications that occur on the surface of the pavement due to the applied traffic. One coverage occurs when all points on the pavement surface within the traffic lane have been subjected to one application of maximum stress, assuming the stress is equal under

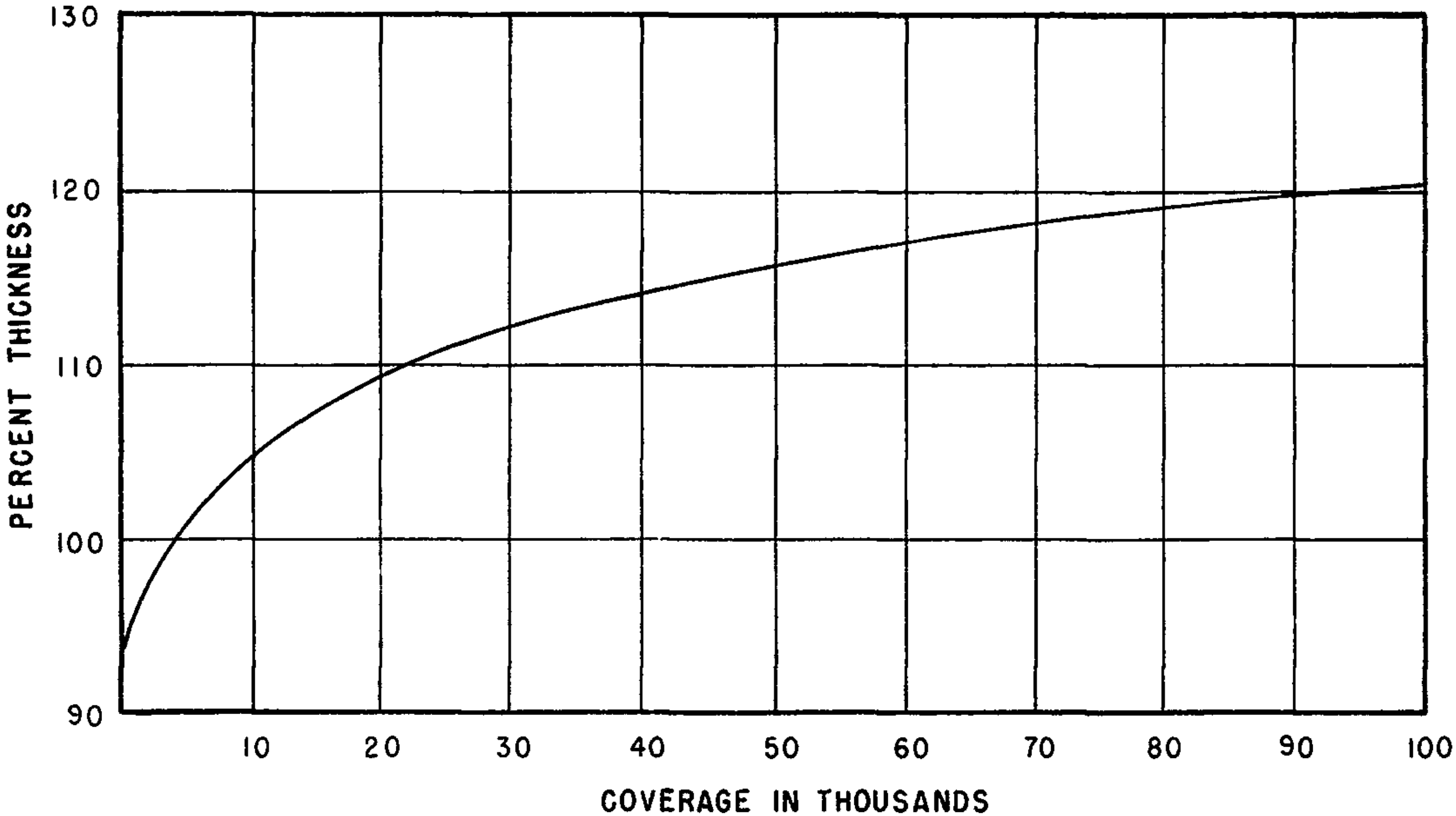


FIGURE 2. PERCENT THICKNESS VS. COVERAGES

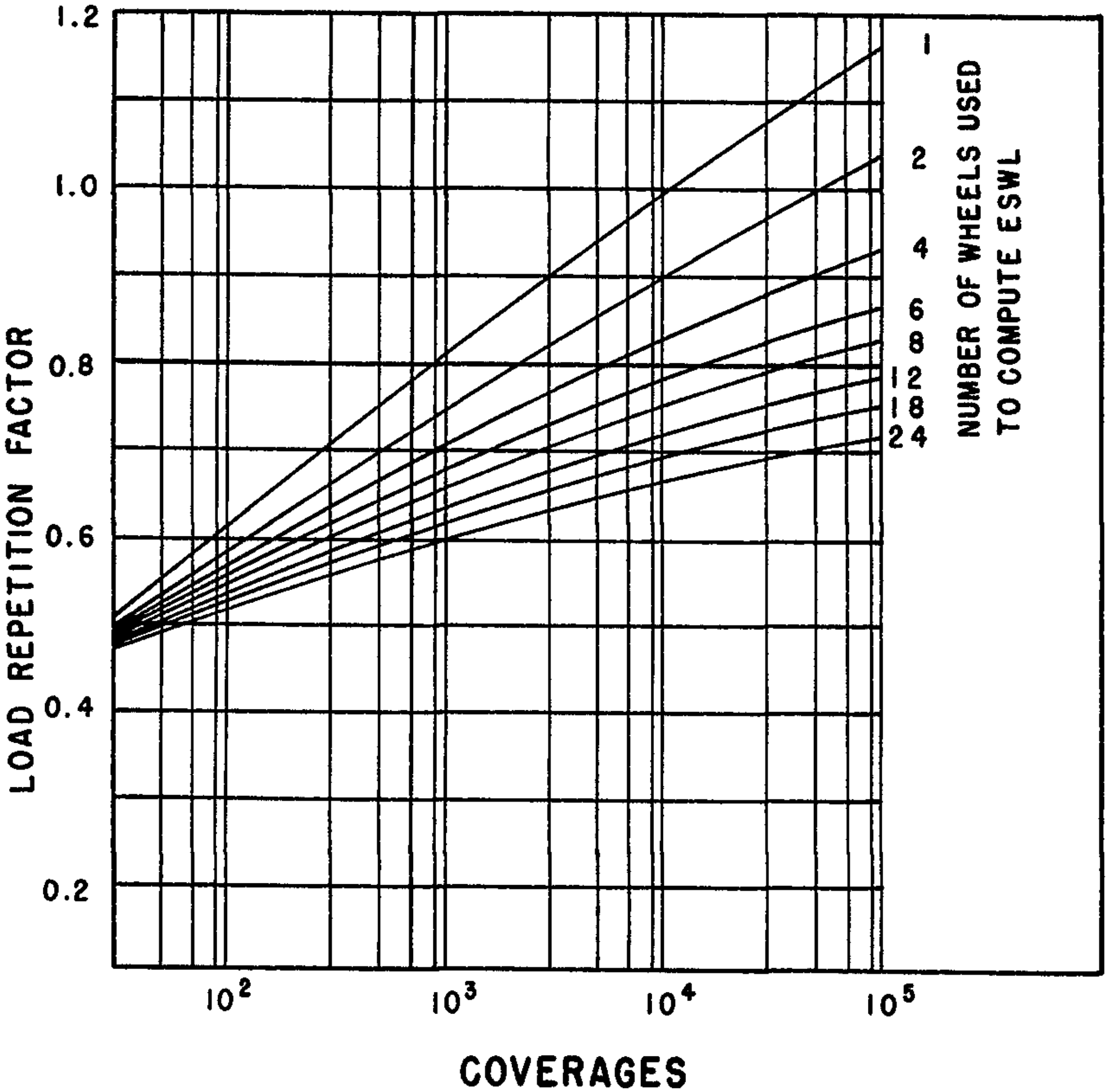


FIGURE 3. LOAD REPETITION FACTOR VS. COVERAGES

the full tire print. Each pass (departure) of an aircraft can be converted to coverages using a single pass-to-coverage ratio which is developed assuming a normal distribution and applying standard statistical techniques. The pass-to-coverage ratios used in developing the flexible pavement design curves in Chapter 3 are given in Table 5. Annual departures are converted to coverages by multiplying by 20 and dividing that product by the pass-to-coverage ratio given in Table 5. Figure 3 shows the relationship between load repetition factor and coverages. The pavement section thickness determined in accordance with paragraph a above is multiplied by the appropriate load repetition factor (Figure 3) to give the final pavement thickness required for various traffic levels.

TABLE 5. PASS-TO-COVERAGE RATIOS FOR FLEXIBLE PAVEMENTS

Design Curve	Pass-to-Coverage Ratio
Single Wheel	5.18
Dual Wheel	3.48
Dual Tandem	1.84
B-747	1.85
DC 10-10	1.82
DC 10-30	1.69
L-1011	1.81

APPENDIX 3. DESIGN OF STRUCTURES FOR HEAVY AIRCRAFT.

1. BACKGROUND. Airport structures such as culverts and bridges are usually designed to last for the foreseeable future of the airport. Information concerning the landing gear arrangement of future heavy aircraft is speculative. It may be assumed with sufficient confidence that strengthening of pavements to accommodate future aircraft can be performed without undue problems. Strengthening of structures, however, may prove to be extremely difficult, costly, and time-consuming. Point loadings on some structures may be increased; while on overpasses, the entire aircraft weight may be imposed on a deck span, pier,or footing.

2. RECOMMENDED DESIGN PARAMETERS.

   a. For structures with spans in excess of 10 feet (3 m), the slab or deck design, and to some degree the design of piers and foundations, becomes greatly dependent upon the aircraft gear configuration. Our assessment indicates that three basic configurations, shown in Figure 1, will, if all are considered in the design of the bridge components, provide sufficient support for any aircraft which may be forthcoming. These consist of two areas enclosing eight wheels each, or 16 wheels per aircraft comprising the main gear. Nose gears, as such, are not considered, except as they occur in the static load. The "area" dimensions are 6 to 8 feet by 20 feet (2-3 m by 6 m) each supporting half of the aircraft gross weight. Wheel prints are uniformly spaced within their respective areas.

   b. Foundation design will vary with depth and soil type. No departure from accepted methodology is anticipated; except that for shallow structures, such as inlets and culverts, the concentrated loads may require heavier and wider spread footings than those presently provided by the structural standards in current use.

   c. It should be noted that all loads discussed herein are to be considered as live loads, and that braking loads as high as 0.7G (for no-slip brakes) must be anticipated on bridge decks and considered for other structures subject to direct wheel loads.

   d. Decks and covers subject to direct heavy aircraft loadings such as manhole covers, inlet grates, utility tunnel roofs, bridges, etc., should be designed for the following loadings:

      (1) Manhole covers for 100,000-lb. (45 000 kg) wheel loads with 250 psi (1.7 MN/m^2) tire pressure.

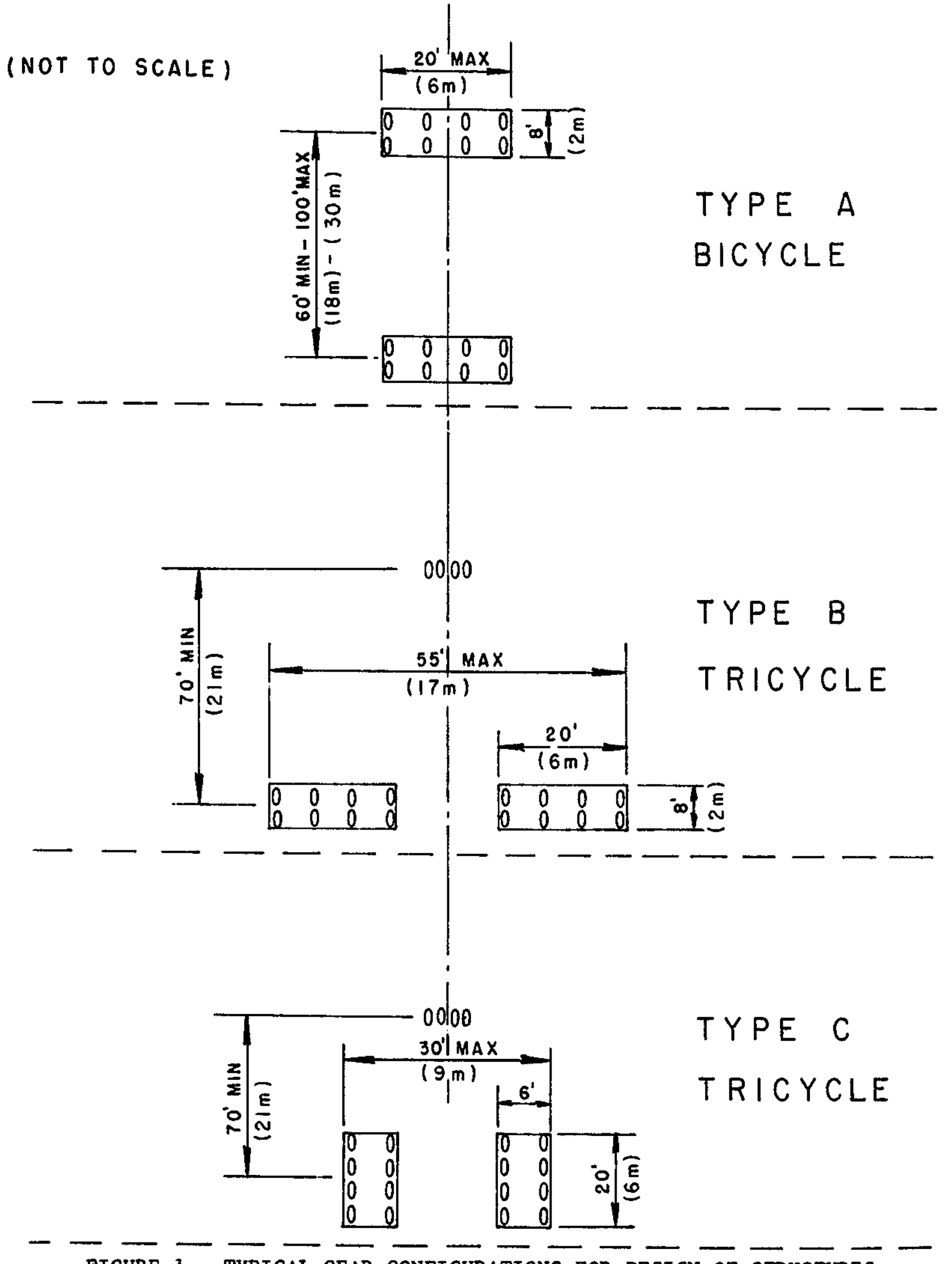


FIGURE 1. TYPICAL GEAR CONFIGURATIONS FOR DESIGN OF STRUCTURES

(2) For spans of 2 feet (0.6 m) or less in the least direction, a uniform live load of 250 psi (1.7 MN/m^2).

(3) For spans between 2 feet (0.6 m) and 10 feet in the least direction, a uniform live load varying between 250 psi (1.7 MN/m^2) and 50 psi, in inverse proportion to the span length

(4) For spans of 10 feet (3 m) or greater in the least direction, the design shall be based on the number of wheels which will fit the span. Wheel loads of 50,000 to 75,000 pounds (22 700 to 34 000 kg) should be considered.

(5) Special consideration shall be given to structures that will be required to support both in-line and diagonal traffic lanes, such as diagonal taxiways or apron taxi routes. If structures require expansion joints, load transfer may not be possible.

APPENDIX 4—RELATED READING MATERIAL

1. The latest issuance of the following free publications may be obtained from the Department of Transportation, Utilization and Storage Section, M-443.2, Washington, D.C. 20590. Advisory Circular 00-2, updated triannually, contains the listing of all current issuances of these circulars and changes thereto.

a. AC 00-2, Federal Register, Advisory Circular Checklist and Status of Federal Aviation Regulations.

b. AC 150/5320-12, Measurement, Construction, and Maintenance of Skid Resistant Airport Pavement Surfaces.

c. AC 150/5325-5, Aircraft Data.

d. AC 150/5335-2, Airport Aprons.

e. AC 150/5370-11, Use of Nondestructive Testing Devices in the Evaluation of Airport Pavements.

f. AC 150/5380-6, Guidelines and Procedures for Maintenance of Airport Pavements.

g. AC 150/5300-9, Predesign, Prebid, and Preconstruction Conferences for Airport Grant Projects.

h. AC 150/5300-12, Airport Design Standards--Transport Airports.

NOTE: AC 150/5300-12 cancelled AC 150/5325-2, AC 150/5325-6, AC 150/5335-1, and AC 150/5335-4. *

2. The following advisory circulars which can be found in AC 00-2 may be obtained from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402. Use the Superintendent of Documents stock number when ordering, along with the AC number and title. Send a check or money order in the amount listed for each document. No. c.o.d. orders are accepted.

a. AC 150/5320-5B, Airport Drainage, dated July 1, 1970.

b. AC 150/5370-10, Standards for Specifying Construction of Airports, dated October 24, 1974.

3. Copies of the following reports may be obtained from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

a. FAA-RD-73-169, Review of Soil Classification Systems Applicable to Airport Pavement Design, May 1974, by Yoder; AD-783-190.

b. FAA-RD-74-30, Design of Civil Airfield Pavement for Seasonal Frost and Permafrost Conditions, October 1974, by Berg; ADA-006-284.

c. FAA-RD-74-36, Field Survey and Analysis of Aircraft Distribution on Airport Pavements, February 1975, by Ho Sang; ADA-011-488.

d. FAA-RD-76-66, Design and Construction of Airport Pavements on Expansive Soils, January 1976, by McKeen; ADA-28-094.

e. FAA-RD-73-198-I, Design and Construction and Behavior Under Traffic of Pavement Test Sections, June 1974, by Burns, Rone, Brabston, Ulery; AD-785-024.

f. FAA-RD-74-33, III, Design Manual for Continuously Reinforced Concrete Pavements, May 1974, by Treybig, McCullough, Hudson; AD-780-512.

g. FAA-RD-75-110-II, Methodology for Determining, Isolating and Correcting Runway Roughness, June 1977, by Seeman, Nielsen; ADA-44-378.

h. FAA-RD-73-198-III, Design and Construction of MESL, December 1974 by Hammitt; AD-005-893.

i. FAA-RD-76-179, Structural Design of Pavements for Light Aircraft December 1976, by Ladd, Parker, Percira; ADA-041-300.

j. FAA-RD-74-39, Pavement Response to Aircraft Dynamic Loads, Volume II - Presentation and Analysis of Data, by Ledbetter; ADA-22-806.

* k. FAA-RD-81-78, Economic Analysis of Airport Pavement Rehabilitation Alternatives, October 1981, by Epps and Wootan. *

4. Copies of ASTM standards may be obtained from the American Society for Testing and Materials, 1916 Race Street, Philadelphia, Pennsylvania 19103.

5. Copies of AASHTO standards may be obtained from the American Association of State Highway and Transportation Officials, 314 National Press Building, Washington, D.C. 20004.

6. Copies of the following publications may be obtain from: Commander, U.S. Army, A.G. Publication Center, 1655 Woodson Road, St. Louis, Missouri 63114.

a. TM5-824-2, Flexible Airfield Pavements, Department of the Army Technical Manual, February 1969.

b. TM5-824-3, Rigid Pavements for Airfields other than Army, Departments of the Army and the Air Force, Technical Manual, December 1970.

c. TM5-818-2, Pavement Design for Frost Conditions, Department of the Army, Technical Manual, July 1965.

7. Copies of MS-11, Full Depth Asphalt Pavements for Air Carrier Airports, January 1973, IS-154, Full Depth Asphalt Pavements for General Aviation, January 1973, and MS-10, Soils Manual, February 1969, may be obtained from the Asphalt Institute, Asphalt Institute Building, College Park, Maryland 20740.

8. Copies of Engineering Bulletin, Design of Concrete Airport Pavement, by Robert G. Packard can be obtained from the Portland Cement Association, Old Orchard Road, Skokie, Illinois 60076.

APPENDIX 5—ECONOMIC ANALYSIS

1. BACKGROUND. The information presented in this appendix was developed from research report DOT/FAA/RD-81/78. The cost data used are probably not current, however, the principles and procedures are applicable. An example is given for illustrative purposes.

2. ANALYSIS METHOD.

a. Present worth or present value economic analyses are considered the best methods for evaluating airport pavement design or rehabilitation alternatives. A discount rate of 4 percent is suggested together with an analysis period of 20 years. Residual salvage values should be calculated on the straight-line depreciated value of the alternative at the end of the analysis period. The initial cost and life expectancy of the various alternatives should be based on the engineer's experience with consideration given to local materials, environmental factors and contractor capability.

b. The basic equation for determining present worth is shown below:

$$PW = C + M_1\left(\frac{1}{1+r}\right)^{n_1} + \ldots\ldots\ldots M_i\left(\frac{1}{1+r}\right)^{n_i} - S\left(\frac{1}{1+r}\right)^{z}$$

Where:

PW = Present worth
C = Present cost of initial design or rehabilitation activity
M_i = Cost of the ith maintenance or rehabilitation alternative in terms of present costs, i.e., constant dollars.
r = Discount rate (four percent suggested)
n_i = Number of years from the present of the ith maintenance or rehabilitation activity.
S = Salvage value at the end of the analysis period
z = Length of analysis period in years (20 years suggested)

The term

$$\left(\frac{1}{1+r}\right)^{n_i}$$

is commonly called the single payment present worth factor in most engineering economic textbooks. From a practical standpoint, if the difference in the present worth of costs between two design or rehabilitation alternatives is 10 percent or less, it is normally assumed to be insignificant and the present worth of the two alternatives can be assumed to be the same.

3. STEP BY STEP PROCEDURE. The information presented in this appendix is intended to demonstrate how to calculate cost comparisons for airport pavement alternatives using the present worth method. The following is a step by step procedure illustrating the analysis method.

a. Identify and record key project descriptions such as:

(1) Project Number and Location

(2) Type of Facility

(3) Design Aircraft

(4) Annual Departures of Design Aircraft

(5) Subgrade Strength

b. If appropriate, determine the condition of existing pavement and record data such as:

(1) Existing Pavement Layers (thicknesses, etc.)

(2) Condition of Pavement (description of distress, pavement condition index, P.C.I., [see AC 150/5380-6], etc.)

(3) Skid Resistance

(4) Required Thickness of New Pavement

c. Identify what feasible alternatives are available.

d. Determine costs associated with each feasible alternative in terms of present day costs.

(1) Initial Cost

(2) Maintenance

(3) Future Rehabilitation

e. Calculate life-cycle cost for each alternative to be evaluated.

f. Summarize life-cycle costs, length of time required to perform and the chance for success for each alternative.

g. Evaluate the most promising alternatives based on costs, time required, operational constraints, chance for success, etc.

h. If the selection cannot be narrowed to one alternative in the evaluation process, the most promising alternatives should each be bid and the selection made on the basis of the lowest bid.

4. **EXAMPLE PROBLEM.** An example problem is discussed below which illustrates the use of the present worth life-cycle costing techniques described above.

Example - Light-Load General Aviation Airport.

a. A general aviation airport runway is in need of rehabilitation. The existing pavement contains alligator, transverse, and longitudinal cracking. The design aircraft for the facility has a gross weight of 24,000 lbs (10 890 kg). Using the procedures in Chapter 5 of this circular, a 3 inch (76 mm) thick bituminous overlay is required to rehabilitate the pavement. Pertinent data are presented in the Project Summary.

PROJECT SUMMARY

Location - Muddville, TX
Number - A.I.P. 12-34-567

Type of Facility: General Aviation Runway
length = 3,200 ft. (75 m)
width = 75 ft. (23 m)

Design Aircraft: 24,000 lbs. (10,890 kg)

Annual Departures of Design Aircraft: 3,000

Subgrade Strength: CBR = 4

Existing Pavement:

Layer and Type	Thickness	Condition
AC Surface	4 in. (102 mm)	Poor
Untreated Base	10 in. (254 mm)	Good

Condition of Existing Pavement:

Condition Survey: Alligator cracking, moderate 15% of area
Trans. cracking, moderate, 350'/station
Long. cracking, moderate, 400'/station
P.C.I. = 35

Skid Resistance: Good

Req'd. Thickness New Pave. = 18 in. (457 mm) total
2 in. (51 mm) surf.
5 in. (127 mm) base
11 in. (279 mm) subbase

b. Seven rehabilitation alternatives including surface, in-place, and hot-mix recycling are considered feasible. The alternatives under consideration are:

(1) Asphalt-rubber chip seal to delay overlay.

(2) Full width 3-inch (76 mm) direct overlay

(3) Surface recycle 1-inch (25 mm) deep + 2-inch (51 mm) overlay

(4) Asphalt-rubber interlayer + 3-inch (76 mm) overlay

(5) Fabric interlayer + 3-inch (76 mm) overlay

(6) Cold recycle with asphalt emulsion 6-inch (152 mm) deep + 2-inch (51 mm) overlay

(7) Hot recycle and rework base

c. The present day costs of various activities associated with these alternatives are estimated as shown in Table 1.

TABLE 1. COSTS OF REHABILITATION ACTIVITIES

Rehabilitation Activity	Cost	
	$/sq. yd.	$/sq. m)
Asphalt-Rubber Chip Seal	1.25	(1.50)
Asphalt-Rubber Interlayer	1.25	(1.50)
Fabric Interlayer	1.20	(1.44)
Surface Recycling	0.90	(1.08)
Asphaltic Concrete - 1 in. (25 mm)	1.65	(1.97)
Cold Recycle + 2-in. (51 mm) Overlay	6.60	(7.89)
Hot Recycle + Rework Base	8.10	(9.69)

d. The life-cycle costs for each alternative are calculated. This example shows the calculation for only one alternative, the asphalt-rubber chip seal. The calculations are shown in Table 2. Some of the important aspects of this analysis are discussed further below.

TABLE 2. PRESENT WORTH LIFE-CYCLE COSTING

EXAMPLE 1. ALTERNATIVE 1 ASPHALT-RUBBER CHIP SEAL

Year		Cost, $/sq. yd.	Present Worth Factor, 4%	Present Worth Dollars
0	A-R Chip Ceal	1.25	1.0000	1.25
1			0.9615	
2			0.9246	
3	Maintenance	0.25	0.8890	0.22
4	3" Overlay	4.95	0.8548	4.23
5			0.8219	
6			0.7903	
7			0.7599	
8			0.7307	
9			0.7026	
10	Maintenance	0.10	0.6756	0.07
11	Maintenance	0.10	0.6496	0.06
12	Maintenance	0.10	0.6246	0.06
13	Maintenance	0.10	0.6006	0.09
14	Maintenance	0.25	0.5775	0.14
15	1½" Overlay	2.48	0.5553	1.38
16			0.5339	
17			0.5134	
18			0.4936	
19	Maintenance	0.10	0.4746	0.05
20	Maintenance	0.15	0.4564	0.07
Sub Total		9.88		
Salvage Value		-0.71	0.4564	-0.32
Total		9.17		7.300

Note: To convert from $/sq.yd. to $/sq. m, divide by 0.8361.

(1) The asphalt-rubber chip seal is estimated to delay the need for an overlay for 4 years. In the third year the asphalt-rubber chip seal will need maintenance costing $0.25/sq yd. ($0.29/sq m).

(2) In the fourth year a 3-inch (76 mm) overlay will be required. This overlay will require maintenance starting in the 10th year and will require progressively more maintenance as time goes on. In the 14th year maintenance will reach $0.25/sq. yd. ($0.29/sq. m).

(3) In the 15th year a 1.5-inch (38 mm) leveling course will be required. This leveling course will not require maintenance until the 19th year. Maintenance costs begin to escalate again as time goes on.

(4) The 20th year marks the end of the analysis period. The salvage value of the leveling course is: the ratio of the life remaining/to how long it will last; multiplied by its cost. The leveling course, constructed in the 15th year, is expected to have a life of 7 years. It was used for only 5 years during the analysis period. Thus, the leveling course had 2 years of life remaining at the end of the analysis period. The salvage value is 2/7 x $2.48 = $0.71. Discounting the salvage value to the 20th year yields a salvage value of $0.32. Since the salvage value is an asset rather than a cost, it is shown as a negative cost in Table 2. All other activities are assumed to have no salvage value since their useful lives have been exhausted during the analysis period. In this example, a discount rate of 4% was assumed. The present worth calculations for the other six alternatives should be calculated in a similar fashion.

e. A final summary of all alternatives considered in this example is shown in Table 3. This summary shows initial costs, life-cycle costs, construction times, and the probability for success in percent. This final summary is a convenient method of presenting all alternatives for evaluation. In this example a discount rate of 4% was used in all calculations. Maintenance and need for rehabilitation in future years are the engineer's estimates.

TABLE 3. SUMMARY OF ALTERNATIVES

	Alternatives	First Cost $/ sq.yd.	Present Worth Life Cycle $/sq.yd.	Time	Success Chance for %
#1	Asph-Rub Chip Seal	1.25	7.30	2 days	90
#2	3-in. Direct Overlay	4.95	7.29	5 days	95
#3	Surf. Recycle + Overlay	4.20	6.22	4 days	97
#4	A-R Layer + Overlay	6.20	7.39	4 days	97
#5	Fabric + Overlay	6.15	7.74	4 days	97
#6	Cold Recycle	6.60	7.41	6 days	97
#7	Hot Recycle	8.10	8.46	6 days	99

NOTE: To convert from $/sq.yd. to $/sq. m, divide by 0.8361.

f. Comparing and ranking the various alternatives shown in Table 3 yields the following results:

TABLE 4. COMPARATIVE RANKING OF ALTERNATIVES

First Cost	Life-Cycle Cost	Time	Chance for Success
#1	#3	#1	#7
#3	#2	#3	#3
#2	#1	#4	#4
#5	#4	#5	#5
#4	#6	#2	#6
#6	#5	#6	#2
#7	#7	#7	#1

The average life-cycle cost of all 7 alternatives is $7.40/sq. yd. ($8.85/sq m). Adding and subtracting 10% to the average life-cycle cost yields a range of $6.66/sq. yd. to $8.14/sq. yd. ($7.97/sq m to $9.74/sq m).

Alternative #3, surface recycling with an overlay, is lowest in life-cycle costs. Life-cycle costs for alternatives #1, 3, 4, 5, and 6 are within the 10% range of the average cost. Alternative #7 is the most costly and exceeds 10% of the average cost. Alternative #3 appears to be the most promising as it ranks high in three of the four categories considered. The decision to select alternative #3 must consider the availability of contractors capable of performing surface recycling and the time required for completion.

5. SUMMARY. This Appendix presents an economic procedure for evaluating a wide variety of airport pavement design strategies. While the design example addresses a rehabilitation project, the principles are applicable to designs of new pavements as well. Cost data used in the example are out of date and should be updated with more current local costs before individual evaluations leading to strategy selection are undertaken. Whenever possible, local costs should be used in all alternative analyses as local conditions sometimes vary considerably from broad overall averages.

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