UFC 3-260-02
30 June 2001

UNIFIED FACILITIES CRITERIA (UFC)

PAVEMENT DESIGN FOR AIRFIELDS

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UNIFIED FACILITIES CRITERIA (UFC)

PAVEMENT DESIGN FOR AIRFIELDS

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

NAVAL FACILITIES ENGINEERING COMMAND

AIR FORCE CIVIL ENGINEER SUPPORT AGENCY

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

Change No.	Date	Location

This UFC supersedes TM 5-818-2/AFM 88-6, Chap 4, dated January 1985; TM 5-822-2/AFM 88-7, Chap 5, dated July 1987; TM 5-822-13/AFJMAN 32-1018, dated October 1994; TM 5-822-5/AFM 88-7, Chap 1, dated June 1992; TM 5-5-824-1/AFM 88-6, Chap 1, dated June 1987; TM 5-825-2/AFM 88-6, Chap 2/DM 21.3, dated August 1978; TM 5-825-2-1/AFM 88-6, Chap 2, Sec A, dated November 1989; TM 5-825-3/AFM 88-6, Chap 3, dated August 1988; and TM 5-825-3-1/AFM 8-6, Chap 3, Sec A, dated September 1988.

FOREWORD

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PAVEMENT DESIGN FOR AIRFIELDS

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CHAPTER 1

INTRODUCTION

1. PURPOSE. This document establishes general concepts and criteria for the design of airfield pavements for the U.S. Army, Navy, Air Force, and Marine Corps.

2. SCOPE. This document prescribes procedures for determining the thickness, material, and density requirements for airfield pavements in nonfrost and frost areas. It includes criteria for the California Bearing Ratio (CBR) procedure and elastic layered analysis for flexible pavements and the Westergaard Analysis and elastic layered analysis for rigid pavements. The elastic layered analysis for rigid pavements covers only plain concrete, reinforced concrete, and concrete overlay pavements.

3. REFERENCES. Appendix A contains a list of references used in these instructions.

4. UNITS OF MEASUREMENT. The unit of measurement system in this document is the International System of Units (SI). In some cases inch-pound (IP) measurements may be the governing critical values because of applicable codes, accepted standards, industry practices, or other considerations. Where the IP measurements govern, the IP values may be shown in parenthesis following a comparative SI value or the IP values may be shown without a corresponding SI value.

5. PAVEMENT. A pavement as used in this document is a surfaced area designed to carry aircraft traffic and includes the entire pavement system structure above the subgrade. All slabs on grade required to support aircraft loadings, whether interior (hangar floors) or exterior, are to be considered airfield pavements.

a. Flexible Pavement. Flexible pavements are so designated due to their flexibility under load and their ability to withstand small degrees of deformation. The design of a flexible pavement structure is based on the requirement to limit the deflections under load and to reduce the stresses transmitted to the natural subsoil. The principal components of the pavement include a bituminous concrete surface, graded crushed aggregate base course, stabilized material, drainage layer, separation layer, and subbase courses. A bituminous concrete surface course is hot mixed bituminous concrete designed as a structural member with weather and abrasion resisting properties. It may consist of wearing and binder or intermediate course. Figure 1-1 illustrates the components and the terminology used in flexible pavements. Examples of all bituminous concrete pavements (ABC) and flexible pavements utilizing stabilized layers are shown in Figures 1-2 and 1-3. Not all layers shown in the figures are required in every pavement.

b. Rigid Pavement. A rigid pavement is considered to be any pavement system that contains portland cement concrete as one element. Rigid pavements transfer the load to the subgrade by bending or slab action through tensile forces as opposed to shear forces. The principal components of a rigid pavement are the concrete slab, base course, drainage layer, and separation layer. However, a stabilized layer may be required based on site conditions. Figure 1-4 illustrates the components of a rigid pavement. The drainage and separation layer will normally serve as the base course. The following pavements are considered to be rigid pavements:

(1) Plain concrete pavement is a nonreinforced jointed rigid pavement.

(2) Reinforced concrete pavement is a jointed rigid pavement that has been strengthened with deformed bars or welded wire fabric.

(3) Continuously reinforced concrete pavement is a rigid pavement that is constructed without joints and uses reinforcing steel to maintain structural integrity across contraction cracks that form in the pavement.

(4) Fibrous concrete pavement is a rigid pavement that has been strengthened by the introduction of randomly mixed, short, small-diameter steel fibers. Nonsteel fibers have been used in portland cement concrete (PCC) to control shrinkage cracking, but their use is not covered in this TI.

(5) Prestressed concrete pavement is a rigid pavement that has been strengthened by the application of a significant horizontally applied compressive stress during construction.

(6) Rigid overlay pavement is a rigid pavement used to strengthen an existing flexible or rigid pavement.

(7) Nonrigid overlay pavement is either all-bituminous or bituminous with base course used to strengthen an existing rigid pavement.

6. USE OF FLEXIBLE PAVEMENTS. The use of flexible pavements on airfields must be limited to those pavement areas not subjected to detrimental effects of fuel spillage, severe jet blast, or parked aircraft. Jet blast damages bituminous pavements when the intense heat is allowed to impinge in one area long enough to burn or soften the bitumen so that the blast erodes the pavement. Hot-mix asphaltic concretes generally will resist erosion at temperatures up to 150 degrees Celsius (300 degrees Fahrenheit). Temperatures of this magnitude are produced only when aircraft are standing and are operated for an extended time or with afterburners operating. Fuel spillage leaches out the asphalt cement in asphaltic pavements. In an area subject to casual minor spillage, the leaching is not serious, but where spillage is repeated in the same spot at frequent intervals, the leaching will expose loose aggregate. Flexible pavements are generally satisfactory for runway interiors, secondary taxiways, shoulders, paved portions of overruns, or other areas not specifically required to have a rigid pavement surfacing.

7. USE OF RIGID PAVEMENTS. The following pavements will be rigid pavement: all paved areas on which aircraft or helicopters are regularly parked, maintained, serviced, or preflight checked, on hangar floors and access aprons; on runway ends (305 meters (1,000 feet)) of a Class B runway; areas that may be used from the runway end to 90 meters (300 feet) past the barrier to control hook skip; primary taxiways for Class B runways; hazardous cargo, power check, compass calibration, warmup, alert, arm/disarm, holding, and washrack pads; and any other area where it can be documented that flexible pavement will be damaged by jet blast or by spillage of fuel or hydraulic fluid. Navy aircraft arresting gear pavement protection shall be designed in accordance with NAVFAC design definitive #1404521 and 1404522 shown in NAVFAC P-272. The 2 meters (6.56 feet) of pavement on both the approach and departure sides of the arresting gear pendent shall be PCC for Navy and Marine Corps. Rigid pavements shall also be used at pavement intersections where aircraft/vehicles have a history of distorting flexible pavements and where sustained operations of aircraft/vehicles with tire pressures in excess of 2.06 MPa (300 psi) occur. Continuously reinforced concrete pavement will be used in liquid oxygen (LOX) storage and handling areas to eliminate the use of any organic materials (joint sealers, asphalt pavement, etc.) In those areas. The type of pavement to be used on all other paved areas will be selected on the basis of life cycle costs.

8. SOIL STABILIZATION. Soils used in pavements may be stabilized or modified through the addition of chemicals or bitumens. A stabilized soil is one which has improved load-carrying and durability characteristics through the addition of admixtures. The principal benefits of stabilization include a reduction in pavement thickness, provision of a construction platform, decreased swell potential, and reduction of the susceptibility to pumping as well as the susceptibility to strength loss due to moisture. Lime, cement, and fly ash, or any combination of these, and bitumen are the commonly used additives for soil stabilization. A modified soil is one which has improved construction characteristics through the use of additives. However, the additives do not improve the strength and durability of the soil sufficiently to qualify as a stabilized soil with a subsequent reduction in thickness. Criteria for the design of stabilized soils is contained in TM 5-822-14/AFMAN 32-1019. Additional discussion of soil stabilization is found in TM 5-818-1/AFM 88-3, Chapter 7.

9. DESIGN ANALYSIS. The outlines in Appendixes B and C will be used to prepare design analyses for all projects under design. All pertinent items and computational details will be included showing how design results were obtained.

10. WAIVERS TO CRITERIA. Each DoD Service component is responsible for setting administrative procedures necessary to process and grant formal waivers. Waivers to the criteria contained in this manual will be processed in accordance with Appendix D.

11. COMPUTER PROGRAMS. Computer programs have been developed for the design of pavements. The computer programs may be obtained electronically from the following:

a. Word Wide Web (WWW) address: http://pcase.com.

b. FTP Anonymous Site: pavement.wes.army.mil.

Disks may also be obtained from the U.S. Army Corps of Engineers, Transportation Systems Center, 215 North 17th Street, Omeha, NE 68102-4978.

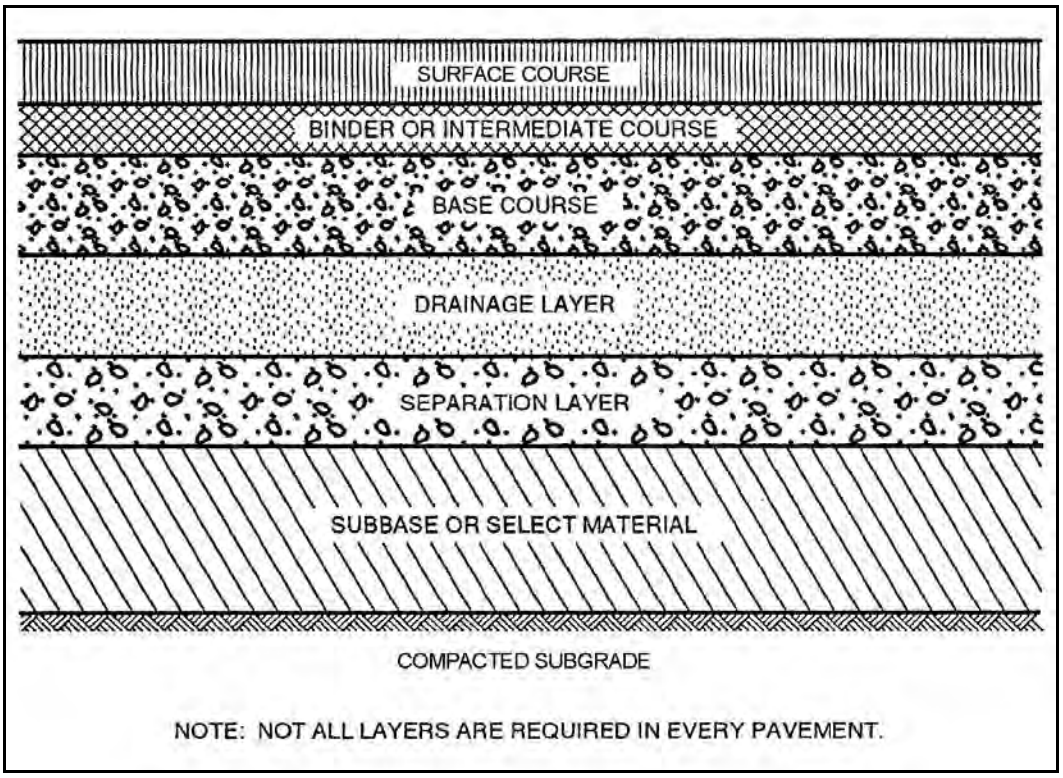


Figure 1-1. Typical flexible pavement structure

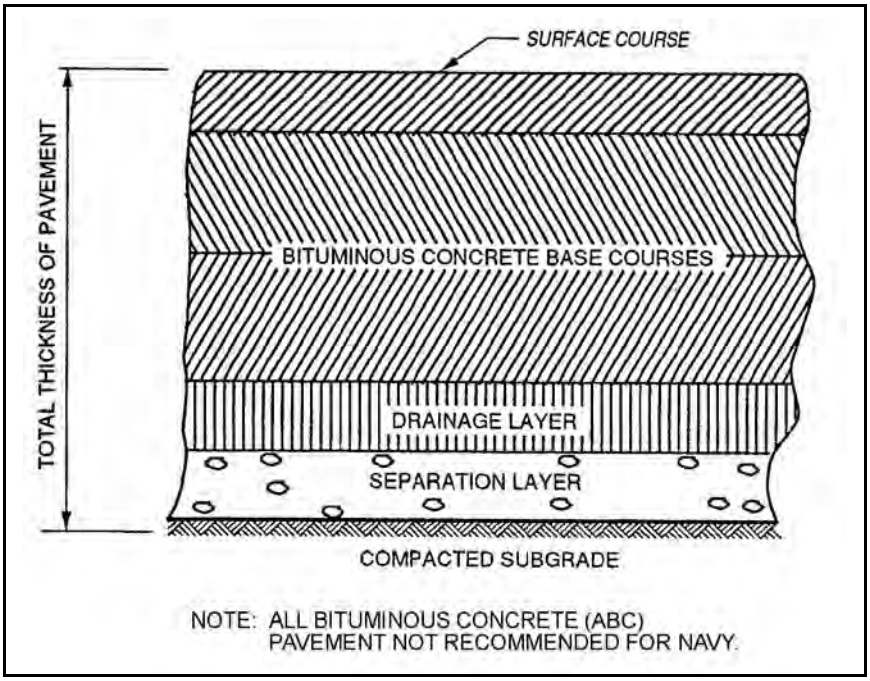


Figure 1-2. Typical all-bituminous concrete pavement

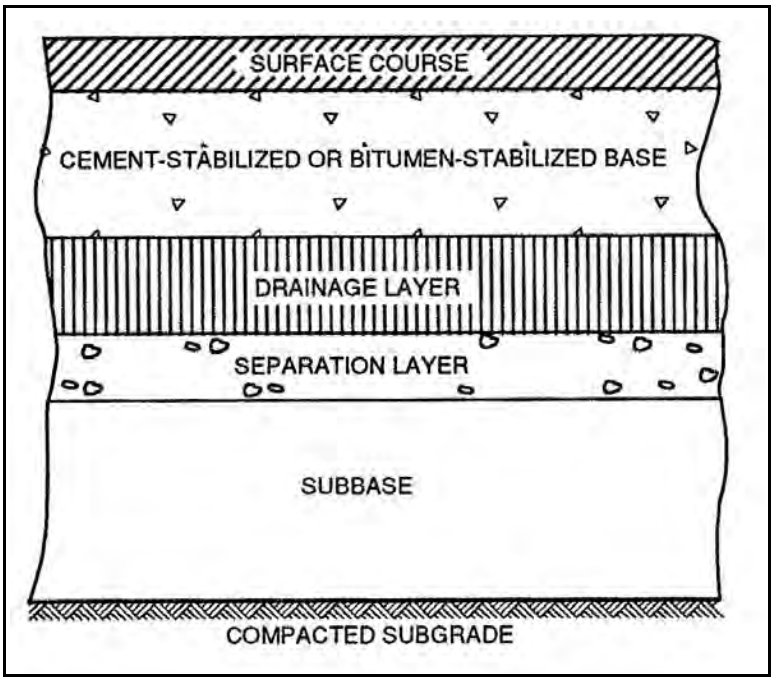


Figure 1-3. Typical flexible pavement with stabilized base

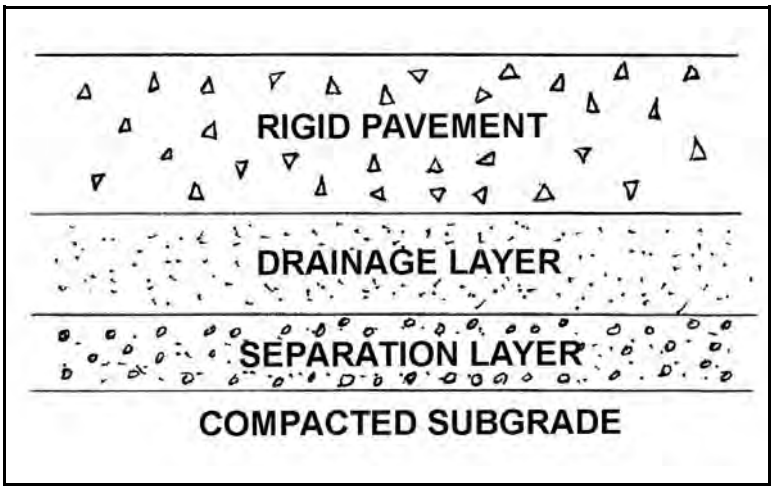


Figure 1-4. Typical rigid pavement structure

CHAPTER 2

ARMY AIRFIELD/HELIPORT REQUIREMENTS

1. ARMY AIRFIELD/HELIPORT CLASSES. Army airfields are divided into six classes referred to as Class I (heliports-helipads with aircraft 11,340 kilograms (25,000 pounds) or less), Class II (heliports-helipads with aircraft over 11,340 kilograms (25,000 pounds)), Class III (airfields with Class A runways), Class IV (airfields with Class B runways), Class V contingency (theater of operations) heliports or helipads supporting Army assault training missions, and Class VI assault landing zones for contingency (theater of operations) airfields supporting Army training missions.

2. ARMY AIRFIELD AND HELIPORT LAYOUT. The layout for all Class I, II, III, and IV Army airfields, heliports, and helipads will be designed in accordance with the tri-service manual UFC 3-260-01. All Class V and VI Army contingency (theater of operations) airfield, heliport, and helipad layouts shall be designed in accordance with FM 5-430-00-2/AFJPAM 32-8013, Vol. II. Class VI airfields used for Army contingency training missions shall be designed in accordance with AF ETL 98-5. Any deviations from these criteria must be submitted through the installation MACOM to the U.S. Army Aeronautical Services Agency (USAASA) for waiver approval.

3. TRAFFIC AREAS FOR ARMY AIRFIELD PAVEMENTS. Construction of primary taxiways, runways, and apron taxi lanes with keel sections (alternating variable thickness) as indicated by traffic will not be authorized for Army aircraft operational surfaces. Uniform pavement section thicknesses will be used.

a. Class I and II Heliports. These heliport classes have only one traffic area, Type B.

b. Class III Airfields. These airfields contain three traffic areas, Types A, B, and C. Type A traffic areas consist of the primary taxiways and the first 152 meters (500 feet) of runway ends. Type B traffic areas consist of parking aprons, warm-up pads, arm/disarm pads, compass calibration pads, power check pads, dangerous/ hazardous cargo pads, and taxiways connecting the primary taxiway to aprons and pads. Type C traffic areas consist of runway interiors between the 152-meter (500-foot) end sections, secondary (ladder) taxiways, hangar floors, washracks, and hangar access aprons. Type C traffic areas are designed using 75 percent of the aircraft gross weight and the same aircraft passes as Type A traffic areas. A typical layout of Army airfield traffic areas for Class III airfields is shown in Figure 2-1.

c. Class IV Airfields. These airfields contain three traffic areas, Types A, B, and C. Type A traffic areas consist of the primary taxiways and the first 305 meters (1,000 feet) of runway ends. Type B traffic areas consist of the parking aprons, warm-up pads, arm/disarm pads, power check pads, compass calibration pads, dangerous/hazardous cargo pads, and taxiways from the primary taxiway to aprons and pads. Type C traffic areas consist of runway interiors between the 305-meter (1,000-foot) end sections, secondary (ladder) taxiways (between runway and primary taxiway), hangar floors, hangar access aprons, and washracks. A typical layout of Army airfield traffic areas for Class IV airfields is shown in Figure 2-1.

d. Class V Heliports. This heliport has only one traffic area, Type B.

e. Class VI Airfields. This airfield has only one traffic area, Type A.

f. Exceptions. At facilities other than assault landing zones where a parallel taxiway is not provided, the runway shall be designed as Type A Traffic Area with double the required traffic.

4. ARMY AIRCRAFT DESIGN LOADS AND PASS LEVELS. Army airfield pavements will be designed according to mission requirements of each airfield, heliport, and helipad for a 20-year design life to include the military and civilian peacetime aircraft traffic plus all anticipated special operations and/or mobilization requirements defined by the Army installation and its MACOM. The total 20-year design aircraft traffic is based on specific aircraft types, their mission operational weights, and their projected pass levels. The airfield mission traffic used for design requires the approval of the MACOM and USAASA. Aircraft hangar floors or apron pavements shall not be designed for jacking loads as long as the foot print of the jack is equal to or greater than the contact area of the combined tires on the aircraft gear being elevated. Army aircraft operational pavements may consist of one or a combination of the following Army airfield-heliport classes:

a. Class I. Heliports and helipads with aircraft maximum operational weights equal to or less than 11,340 kilograms (25,000 pounds). The design of heliports and helipads will be based on the number of equivalent passes of the UH-60 aircraft at a 7,395-kilogram (16,300-pound) operational weight. The projected equivalent passes will be generated for the airfield mission traffic but shall not be less than 50,000 passes for a heliport nor less than 20,000 passes for a helipad.

b. Class II. Heliports that support aircraft with maximum operational weights over 11,340 kilograms (25,000 pounds). The design will be based on the number of equivalent passes of the CH-47 aircraft at a 22,680-kilogram (50,000-pound) operational weight. The projected equivalent passes will be generated for the airfield mission traffic but shall not be less than:

(1) 50,000 passes for visual flight rules (VFR) heliports.

(2) 20,000 passes for VFR helipads.

(3) 100,000 passes for instrument flight rules (IFR) heliports.

(4) 30,000 passes for IFR helipads.

c. Class III. Airfields that primarily support fixed wing aircraft requiring a Class A runway as defined in UFC 3-260-01. The design will be based on the projected number of aircraft operations but not less than 50,000 passes of a C-23 aircraft at an 11,200-kilogram (24,600-pound) operational weight plus 10,000 passes of a CH-47 aircraft at an operational weight of 22,680-kilograms (50,000-pounds).

d. Class IV. Airfields supporting aircraft requiring a Class B runway as defined in EI 02C013/ AFMAN 32-1013/NAVFAC P-971.

(1) The design for an airfield with its longest runway extending less than or equal to 1,525 meters (5,000 feet) will be based on the number of projected equivalent passes of the C-130 aircraft at a 70,310-kilogram (155,000-pound) or the C-17 aircraft at 263,100-kilograms (580,000-pound) operational weight. The projected equivalent passes will be generated for the airfield mission traffic but shall not be less than 75,000 passes for the C-130 or 50,000 passes for the C-17.

(2) The design for an airfield with its longest runway extending over 1,525 meters (5,000 feet) but less than or equal to 2,745 meters (9,000 feet) will be based on the number of projected equivalent

passes of the C-17 aircraft at a 263,100-kilogram (580,000-pound) operational weight. The projected equivalent passes will be generated for the airfield mission traffic but shall not be less than 75,000 passes.

(3) The design for an airfield with its longest runway extending over 2,745 meters (9,000 feet) will be based on the number of projected equivalent passes of the C-17 aircraft at a 263,100-kilogram (580,000-pound) operational weight. The projected equivalent passes will be generated for the airfield mission traffic but shall not be less than 100,000 passes.

e. Class V. Contingency (theater of operations) heliports or helipads supporting Army assault training missions. The design for the heliport or helipad will be based on the number of projected equivalent passes of the CH-47 aircraft at a 22,680-kilogram (50,000-pound) operational weight. The projected equivalent passes will be generated for the airfield mission traffic but shall not be less than 5,000 passes. Army assault heliport or helipad structural sections shall be designed in accordance with the criteria in this document with a bituminous surface or a military landing mat as described in FM5-430-00-2/AFJPAM 32-8013, Vol II.

f. Class VI. Assault landing zones for contingency (theater of operations) airfields or airstrips supporting Army training missions that have semi-prepared or paved surfaces. The design for airfields supporting Army training missions will be based on the number of equivalent passes of the C-130 aircraft at a 70,310-kilogram (155,000-pound) operational weight or the C-17 aircraft at a 263,100-kilogram (580,000-pound) operational weight. The equivalent passes will be not less than 10,000 passes for paved airfields. Army assault airfield or airstrip structural sections shall be designed in accordance with this manual. Army assault airfields with semi-prepared (unsurfaced) surfaces shall be designed in accordance with TM 5-822-12, TM 5-822-14, or Air Force ETL 98-2.

5. ROLLER-COMPACTED CONCRETE PAVEMENT. Roller-compacted concrete pavement (RCCP) is a rigid pavement and can be used as pavement except for runway and high-speed taxiway pavements for fixed-wing aircraft. RCCP can be used for all helipad and heliport pavements. RCCP shall be designed in accordance with ETL 1110-3-475.

6. RESIN MODIFIED PAVEMENT. Resin Modified Pavement (RMP) can be used as an Army pavement except for fixed-wing runways and high-speed taxiways. RMP can be used for helipads and heliport pavements and for both rotary-wing and fixed-wing parking aprons.

7. PAVED SHOULDERS.

a. Location. Paved shoulders should be provided for airfield and heliport construction as designated in UFC 3-260-01.

b. Structural Requirements. As a minimum, paved shoulders shall be designed to support 5,000 coverages of a load of 4,535 kilograms (10,000 pounds) imposed by a single wheel with a tire pressure of 0.69 MPa (100 psi). When shoulder pavements are to be used by support vehicles (snow removal equipment, fire trucks, fuel trucks, etc.), the shoulder should be designed accordingly for whichever governs.

8. SURFACE DRAINAGE. Design of surface drainage shall be in accordance with TM 5-820-1/ AFM 88-5, Chapter 1.

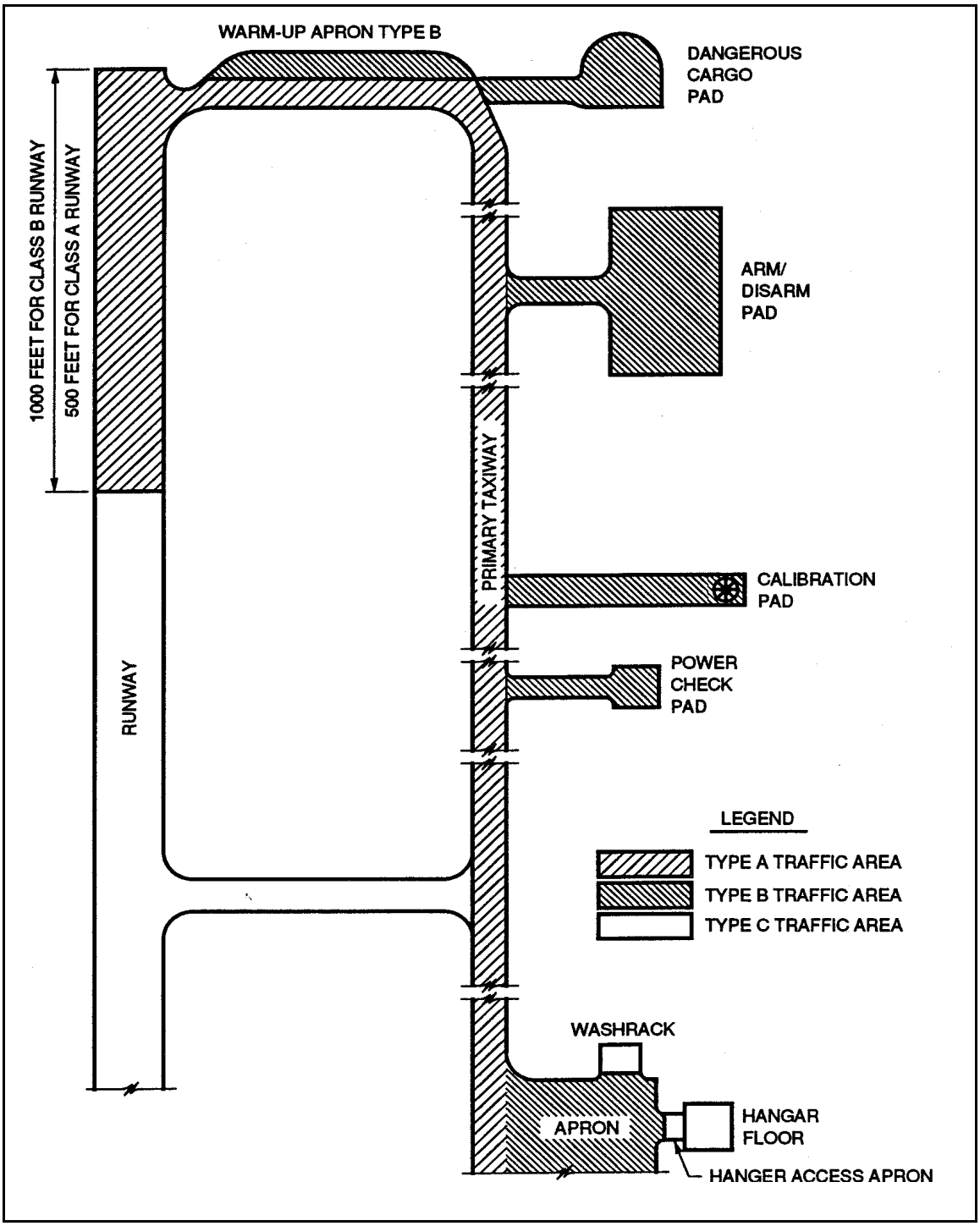


Figure 2-1. Typical layout of traffic areas for Army Class III and IV airfields

CHAPTER 3

AIR FORCE AIRFIELD AND AGGREGATE SURFACED HELICOPTER SLIDE AREAS AND HELIPORT REQUIREMENTS

1. AIR FORCE AIRFIELD TYPES. Airfield mission and operational procedures have resulted in the development of six types of Air Force airfields: light, medium, heavy, modified heavy, auxiliary, and assault landing zone. The decision on which airfield type to design for will be made by the appropriate Major Command (MAJCOM). Designs should generally be based upon medium load criteria with the following exceptions.

a. Air Training Command bases should be designed as light load. Auxiliary airfields at Air Training Command bases will be designed for the load and pass level selected by the Major Command.

b. For bases where B-52's are the critical missions, use heavy load criteria.

c. For bases where the B-1 and/or KC-10's are the critical mission, use modified heavy load criteria.

d. Assault landing zone criteria should be used to design runways for C-130 or C-17 training.

e. MAJCOMs should plan for future missions. For example, if the current mission uses KC-135 tankers but will use KC-10 aircraft in the future, the KC-10 should be the design aircraft.

f. In lieu of the above criteria, MAJCOMs have the option to design for specific aircraft and projected pass levels.

2. TRAFFIC AREAS FOR AIR FORCE AIRFIELDS. On normal operational airfields, the pavements can be grouped into four traffic areas designated as Types A, B, C, and D which are defined below and shown in Figures 3-1, 3-2, or 3-3 for each type airfield. A layout of the assault landing zone is not shown since all areas are Type A traffic areas. Modified heavy-load airfields will have the same traffic areas as medium-load airfields. Auxiliary airfields will have the same traffic areas as light-load airfields.

a. Type A Traffic Areas. Type A traffic areas are those pavement facilities that receive the channelized traffic and full design weight of aircraft. Aircraft with steerable gear, including fighter-type aircraft, operate within a relatively narrow taxilane producing sufficient coverages or stress repetition within the narrow lane to require special design treatment. Type A traffic areas for pavements are dictated by the operational patterns of aircraft. These traffic areas require a greater pavement thickness than those areas where the traffic is more evenly distributed. Pavement features considered to be Type A traffic areas on each airfield type are as follows:

(1) Heavy-load airfield.

(a) Portions of long straight sections of primary taxiways will be Type A traffic areas. Traffic channelization is limited to the center of the taxiway for aircraft with a bicycle-gear configuration. Therefore, the center 7.6-meter (25-foot) (minimum) of long straight sections will be designed as a Type A traffic area. The outside lanes will be designed as Type B traffic areas. An alternative design is to provide uniform thickness for the full width of the taxiway.

(b) Taxiways connecting runway ends and primary taxiways, short lengths of primary taxiway turns, and intersections of primary taxiways will be Type A traffic areas. The effects of traffic channelization on these areas cannot be well defined; therefore, these pavements will be designated as Type A traffic areas requiring a uniform pavement thickness for the full width of the taxiway.

(c) Through taxilanes or portions of through taxiways on aprons (7.6-meter (25-foot) minimum) will be designed as Type A traffic areas.

(d) Portions of the first 305 meters (1,000 feet) of runway ends will be Type A traffic areas. On these pavements, the effects of channelized traffic are generally confined to the center 23-meter (75-foot) width and the approach area from the connecting taxiway. These portions will be designed as Type A traffic areas and will require a uniform thickness. The dimensions of the approach area will correspond to the width of the connecting taxiway plus the taxiway fillets. An alternate design for the first 305 meters (1,000 feet) of runway ends is to provide a uniform thickness for the full width of the pavement. Design of the pavement for channelized traffic must include the lanes where the traffic of the design landing-gear type (bicycle or tricycle) is applied. For the present heavy-load pavement (bicycle-landing gear), the selection of a thickened center section or a uniform thickness for the full width of the facility will be determined on the basis of life cycle costs and projected future mission. In seasonal frost areas, it is often desirable to use a constant transverse section to preclude differential frost heave.

(2) Medium-load and modified heavy-load airfield.

(a) Primary taxiways will be designed as Type A traffic areas. The effects of channelized traffic are well defined on long straight sections. However, the channelization is not as confined as for a heavy-load pavement, and it is not practical to construct primary taxiways of alternating variable thicknesses as indicated by traffic requirements. Therefore, the primary taxiways for medium-load and modified heavy-load airfields will normally be constructed to provide a uniform thickness for the full width of pavement facility. The entire primary taxiway, including straight sections, turns, and intersections, will be designated as Type A traffic areas.

(b) Through taxilanes and portions of through taxiways on aprons (11-meter (35-foot) minimum) will be designed as Type A traffic areas.

(c) Portions of the first 305 meters (1,000 feet) of runway ends will be designed as Type A traffic areas. On these pavements, the effects of channelized traffic are generally confined to the center 23-meter (75-foot) width and the approach area from the connecting taxiway. These portions will be designed as Type A traffic areas and will require a uniform thickness. The dimensions of the approach area will correspond to the width of the connecting taxiway plus the taxiway fillets. An alternate design for the first 305 meters (1,000 feet) of runway ends would be to provide a uniform thickness for the full width of the pavement facility. The selection of a thickened center section or a uniform thickness for full width of the facility will be determined on the basis of life cycle costs unless mission requirements dictate a uniform thickness (an example is formation takeoffs). In frost areas, it is often desirable to use a uniform thickness to preclude differential frost heave.

(3) Light-load and auxiliary airfields. Primary taxiways and the first 305 meters (1,000 feet) of runway ends will be designed as Type A traffic areas. The effects of channelized traffic are reasonably well defined on long straight sections. However, it is not considered practical to construct primary taxiways and runway ends of alternating variable thicknesses for light-load and auxiliary airfields as indicated by traffic requirements. Therefore, the primary taxiways and the first 305 meters (1,000 feet) of runway ends for light-load and auxiliary airfields will normally be constructed to provide a uniform

thickness for the full width of pavement facility. The entire primary taxiway, including straight sections, turns, and intersections, will be designated as Type A traffic areas.

(4) Assault landing zone airfield. The type of aircraft operations conducted on these pavements will require the entire runway, the 91-meter (300-foot) overruns, and the short access taxiways to be designed as Type A traffic areas.

b. Type B Traffic Areas. Type B traffic areas are those in which the traffic is more evenly distributed over the full width of the pavement facility but which receive the full design weight of the aircraft during traffic operations. Inasmuch as there is a better distribution of the traffic on these pavements, the repetition of stress within any specific area is less than on Type A traffic areas; therefore, a reduction in required pavement thickness can be allowed. Pavement facilities considered to be Type B traffic areas on each airfield type are as follows:

(1) Heavy-load airfield. All aprons (except hangar access aprons), pads, and hardstands, and traffic lanes adjacent to the center lane on long straight sections of primary taxiways are designed as Type B traffic areas.

(2) Medium-load and modified heavy-load airfields. All aprons (except hangar access aprons), pads, and hardstands are Type B traffic areas.

(3) Light-load and auxiliary airfields. All aprons (except hangar access aprons), hardstands, and power check pads are Type B traffic areas.

(4) Assault landing zone. No Type B traffic area.

c. Type C Traffic Areas. Type C traffic areas are those in which the volume of traffic is low or the applied weight of the operating aircraft is generally less than the design weight. In the interior portion of runways, there is enough lift on the wings of the aircraft at the speed at which the aircraft passes over the pavements to reduce considerably the stresses applied to the pavements. Thus, the pavement thickness can be reduced in these portions of the runways. Therefore, all runway interiors, except shortfield, will be designated as Type C traffic areas regardless of type of design loadings. For the heavy, modified heavy, and medium-load airfields, the edges of the runway seldom receive a fully loaded aircraft; therefore, for these airfields, the Type C traffic areas are limited to the center 23-meter (75-foot) width of runway interior. However, in seasonal frost areas, it may be necessary to use a uniform thickness for the entire width of the runway to preclude frost heave. Pavement facilities at all airfields considered to be Type C traffic areas are as follows:

(1) Heavy-load airfields.

(a) Secondary (ladder) taxiways.

(b) The center 23-meter (75-foot) width of runway interior between the 305-meter (1,000-foot) runway ends and at runway edge adjacent to intersections with ladder taxiways.

(c) Main gear path area of hangar access aprons and floors and washrack pavements. (The pavement outside the main gear path area of hangar access aprons and floors and washracks are designed as a light-load Type C traffic area.)

(2) Medium-load and modified heavy-load airfields.

(a) Secondary (ladder) taxiways.

(b) The center 23-meter (75-foot) width of runway interior between 305-meter (1,000-foot) runway ends and at runway edges adjacent to intersections with ladder taxiways.

(c) Hangar access aprons and floors and washrack pavements. At Air Mobility Command Installations, hangar access aprons shall be designed as Medium Load Type C Traffic Area for the main gear plus 3 meters (10 feet) on each side. The remainder of the access apron shall be Light Load Type C Traffic Area.

(3) Light-load and auxiliary airfields.

(a) Full width of runway interior between the 305-meter (1,000-foot) runway ends and secondary (ladder) taxiways.

(b) Hangar access aprons and floors.

(c) Washrack pavements.

(4) Assault landing zone. No Type C traffic areas.

d. Type D Traffic Areas. Type D traffic areas are those in which the traffic volume is extremely low and/or the applied weight of operating aircraft is considerably lower than the design weight. The pavement facilities considered to be Type D traffic areas are the edges of runways that are designed for heavy-load, medium-load, and modified heavy-load airfields. Aircraft on heavy-, modified heavy-, or medium-load runways seldom, if ever, operate outside of the center 23-meter (75-foot) width of the runway interior, and the only traffic that will occur on the edges of the runway will be occasional heavy, medium, or modified heavy aircraft loads or frequent light aircraft loads. Therefore, a substantial reduction in required pavement thickness can be made. Pavement facilities considered to be Type D traffic areas are as follows:

(1) Heavy-load airfields. The outside edges of the entire length of runway, except for the approach and exit areas at taxiway intersections, are Type D traffic areas.

(2) Medium-load and modified heavy-load airfields. The outside edges of the entire length of runway except for the approach and exit areas at taxiway intersections are Type D traffic areas.

(3) Light-load and auxiliary airfields. There are no Type D traffic areas on light-load or auxiliary pavements.

(4) Assault landing zone. No Type D traffic areas.

3. AIRCRAFT DESIGN LOADS FOR AIR FORCE PAVEMENTS. The design loads for light, medium, heavy, modified heavy, auxiliary, and assault landing zone airfield pavements have been established by the Air Force and are shown in Table 3-1. The concept is to design each airfield type for a mixture of aircraft traffic at the loads shown. These loads represent the design gross weights for each type traffic area and overruns on the airfield. Aircraft hangar floors or apron pavements shall not be designed for jacking loads as long as the foot print of the jack is equal to or greater than the contact area of the combined tires on the aircraft gear being elevated.

4. DESIGN PASS LEVELS FOR AIR FORCE PAVEMENTS. Aircraft traffic data reports indicating type and frequency of aircraft traffic at selected Air Force bases have been analyzed to establish criteria to be used in the design of airfield pavements. These design pass levels are shown in Table 3-1 for the different traffic areas and aircraft types. Airfield pavements may be designed for alternate pass levels if dictated by the intended use of the facility and subject to the approval of the appropriate Air Force Major Command.

5. RESIN MODIFIED PAVEMENT. Resin Modified Pavement (RMP) can be used as an Air Force pavement except for runways and high-speed taxiways. RMP can be used for helipads and heliport pavements and for both rotary-wing and fixed-wing aprons.

6. PAVED SHOULDERS.

a. Location. Paved shoulders should be provided for airfield and heliport construction as designated in UFC 3-260-01.

b. Structural Requirements. As a minimum, paved shoulders shall be designed to support a load of 4,535 kilograms (10,000 pounds) imposed by a single wheel with a tire pressure of 0.69 MPa (100 psi). When shoulder pavements are to be used by support vehicles (snow removal equipment, fire trucks, fuel trucks, etc.), the shoulders should be signed accordingly for whichever governs.

7. AGGREGATE SURFACED HELICOPTER SLIDE AREAS AND HELIPORTS. Geometric and structural criteria for the design of aggregate surfaced helicopter slide areas and heliports are listed below. These criteria are applicable to all Air Force organizations with pavement design and construction responsibilities.

a. Geometric Criteria. Geometric criteria can be found in UFC 3-260-01.

b. Structural Criteria. Airfield structural design criteria are presented below.

(1) Thickness (Non-Frost Areas). Factors which determine thickness are the California Bearing Ratio (CBR) of the subgrade, helicopter weight, and passes. The minimum required thickness is 150 millimeters (6 inches). Use Figure 3-6 for design of aggregate surface thickness for helicopters. Enter Figure 3-4 with the subgrade CBR (see Chapter 6 for selection of subgrade CBR) to determine the thickness required for a given load and pass level. The thickness determined from the figure may be constructed of surface course material for the total depth over the natural subgrade; or in a layered system consisting of select material, subbase, and surface course over compacted subgrade for the same total depth. Check the layered section to ensure sufficient material protects the underlying layer, based upon the CBR of the underlying layer. The top 150 millimeters (6 inches) must meet the gradation requirements of Table 3-2.

(2) Select Materials and Subbases. Select design CBR values materials and subbases in accordance with Chapter 7, except as modified in Table 3-3.

(3) Thickness (Frost Areas). In areas where frost effects impact pavement design, there are additional considerations concerning thicknesses and required layers in the pavement structure. For frost design, soils are divided into eight groups as shown in Table 3-4. Only the non-frost-susceptible (NFS) group is suitable for base course. NFS, S1, or S2 soils may be used for subbase course, and any

Table 3-1
Design Gross Weights and Pass Levels for Airfield Pavements

		A Traffic Area		B Traffic Area		C Traffic Area[1]		D Traffic Area[1]		Overruns[1]		
Airfield Type	Design Aircraft	Weight Pounds	Passes	Weight Pounds	Passes	Weight Pounds	Passes	Weight Pounds	Passes	Weight Pounds	Passes	Shoulder
Light	F-15 C/D	68,000	400,000	68,000	400,000	51,000	400,000	NA	NA	51,000	4,000	Shoulders are designed to support 5,000 coverages of a 10,000 pound single-wheel load having a tire pressure of 100 psi.
	C-17	580,000	400	580,000	400	435,000	400			435,000	1	
Medium	F-15 E	81,000	100,000	81,000	100,000	60,750	100,000	60,750	1,000	60,750	1,000	
	C-17	580,000	400,000	580,000	400,000	435,000	400,000	435,000	4,000	435,000	4,000	
	B-52[2]	400,000	400	400,000	400	300,000	400	300,000	4	300,000	4	
Heavy	F-15 E	81,000	100,000	81,000	100,000	60,750	100,000	60,750	1,000	60,750	1,000	
	C-17	580,000	200,000	580,000	200,000	435,000	200,000	435,000	2,000	435,000	2,000	
	B-52	480,000	120,000	480,000	120,000	360,000	120,000	360,000	1,200	360,000	1,200	
Modified Heavy	F-15 E	81,000	100,000	81,000	100,000	60,750	100,000	60,750	1,000	60,750	1,000	
	C-17	580,000	200,000	580,000	200,000	435,000	200,000	435,000	2,000	435,000	2,000	
	B-1	480,000	120,000	480,000	120,000	360,000	120,000	360,000	1,200	360,000	1,200	
Assault Landing Zone	C-130	175,000	50,000 per squadron	NA	NA	NA	NA	NA	NA	175,000	50,000 per squadron	
	C-17	502,000	100,000	NA	NA	NA	NA	NA	NA	502,000	100,000	
Auxiliary	F-15	Design loads and passes are determined by the major command.										

[1] The design gross weights for Types C and D traffic areas and overruns are 75 percent of the design gross weights for Types A and B traffic areas. Pass levels for Type D traffic areas and overruns are one percent of the pass levels forType A traffic area. Assault landing zone overruns are designed the same as rest of pavement.
[2] B-52 aircraft will not be included in the mixed traffic design of medium load airfields with less than 200-foot-wide runways.

Conversion Factors
Kilograms = 0.453 × pounds
Megapascals = 0.006894 × psi
Meters = 0.3048 × feet

Table 3-2
Gradation for Aggregate Surface Courses (Percent Passing)

Sieve Designation	No. 1	No. 2	No. 3	No.4
25.0 mm (1”)	100	100	100	100
9.5 mm (3/8”)	50-85	60-100		
No. 4	35-65	50-85	55-100	70-100
No. 10	25.50	40.70	40-100	55-100
No. 40	15.30	24-45	20-50	30-70
No. 200	8-15	8-15	8-15	8-15

Note: The percent by weight finer than 0.02 millimeter (0.04 inch) shall not exceed 3 percent.

Table 3-3
Maximum Permissible Values for CBR and Gradation Requirements

Material	Maximum CBR	Maximum Size	Maximum % Passing		Maximum Liquid Limit*	Maximum Plasticity Index*
			#10	#200		
Subbase	50	50 mm (2”)	50	15	25	5
Subbase	40	50 mm (2”)	80	15	25	5
Subbase	30	50 mm (2”)	100	15	25	5
Select Material	20	75 mm (3”)	--	--	35	12

* ASTM D 4318.

Table 3-4
Frost Design Soil Classification

Frost Group	Type Soil	Percentage Finer Than 0.02 mm (0.04”) by Weight	Unified Soil Classification Soil Types***
NGS*	(a) Gravels Crushed Stone Crushed rock	0-1.5	GW, GP
	(b) Sands	0-3	SW, SP

(Continued)

Table 3-4 (Concluded)

Frost Group	Type Soil	Percentage Finer Than 0.02 mm (0.04") by Weight	Unified Soil Classification Soil Types***
PFS*	(a) Gravels Crushed Stone Crushed rock	1.5-3	GW, GP
	(b) Sands	3-10	SW, SP
S1	Gravelly soils	3-6	GW, GP, GW-GM, GP-GM
S2	Sandy soils	3-6	SW, SP, SW-SM, SP-SM
F1	Gravelly soils	6-10	GM, GW-GM, GP-GM
F2	(a) Gravelly soils	10-20	GM, GW-GM, GP-GM
	(b) Sands	6-15	SM, SW-SM, SP-SM
F3	(a) Gravelly soils	over 20	GM, GC
	(b) Sands, except very fine silty sands	over 15	SM, SC
	(c) Clays, PI 12	--	CL, CH
F4	(a) Gravelly soils	--	ML, MH
	(b) Sands, except very fine silty sands	over 15	SM
	(c) Clays, PI 12	--	CL, CL-ML
	(d) Verved clays and other fine grained banded sediments	--	CL, ML, SM and CH

* Nonfrost-susceptible.
** Possible frost-susceptible, but requires laboratory test to determine frost design soil classification.
*** Defined in AFM 89-3, *Materials Testing*.

of the eight groups may be found as subgrade soils. Soils are listed in approximate order of decreasing bearing capability during periods of thaw.

(a) Required Thickness. Where there are frost-susceptible subgrades, determine section thickness according to the reduced subgrade strength method. The reduced 3-5 subgrade strength method uses the frost area soil support indexes (FASSI) in Table 3-5. Use FASSI like CBR values. The term CBR is not applied, because FASSI are weighted average values for an annual cycle and their values cannot be determined by CBR tests. Enter Figure 3-4 with the soil support indexes (vice CBR values) to determine the required section thickness.

Table 3-5
Frost Area Soil Support Indices (FASSI) of Subgrade Soils

Frost Group	FASSI
F1 and S1	9.0
F2 and S2	6.5
F3 and F4	3.5

(b) Pavement Section Layers. When frost is a consideration, recommend the pavement section consist of layers that will ensure the stability of the system, particularly during thaw periods. The layered system may consist of a 150-millimeter- (6-inch-) thick minimum wearing surface of fine crushed stone, a coarse-graded base course, and/or a well-graded subbase of sand or gravely sand. To ensure the stability of the wearing surface, the width of the base course and subbase should exceed the final desired surface width by a minimum of 0.35 meter (1 foot) on each side.

(c) Wearing Surface. The wearing surface contains fines (material passing the #200 sieve) to provide stability in the aggregate surface. The presence of fines improves the layer's compaction characteristics and helps to provide a relatively smooth surface.

(d) Base Course. The coarse-graded base course is important in providing drainage of the granular fill. Base course should be non-frost-susceptible to retain strength during spring thaw periods.

(e) Subbase. A well-graded subbase provides additional bearing capacity over the frost-susceptible subgrade. It also provides a filter layer between the coarsegraded base course and the subgrade to prevent migration of the subgrade into the voids in the coarser material during periods of reduced subgrade strength. Therefore, the material must meet standard filter criteria. The subbase must be either non-frost susceptible or of low frost susceptibility (Sl or S2), The filter layer may or may not be necessary depending upon the type of subgrade material. If the subgrade consists principally of gravel or sand, the filter layer may not be necessary, and may be replaced by additional base course if the gradation of the base course meets filter criteria. For finer grained soils, the filter layer will be necessary. If using a geotextile, the sand subbase/filter layer may be omitted, as the fabric will be placed directly on the subgrade and acts as a filter.

(f) Compaction. The subgrade should be compacted to provide uniformity of conditions and a working platform for placement and compaction of subbase. Compaction will not change a subgrade's frost-area soil support index. However, because frost weakens the subgrade, compacted

subgrade in frost areas will not be considered part of the layered system of the airfield, which should be comprised of only the wearing, base, and subbase courses.

(g) Base Course and Filter Layer. Relative thicknesses of the base course and filter layer vary, and should be based on the required cover and economic considerations.

(h) Alternate Design. The reduced subgrade strength design provides a soil thickness above a frost-susceptible subgrade which minimizes frost heave. For a more economical design, a frost-susceptible select material or subbase may be used as a part of the total thickness above the frost-susceptible subgrade. However, thickness above the select material or subbase must be determined by using the FASSI of the select or subbase material. Frost-susceptible soils used as select materials or subbases must meet current specifications; the restriction on the allowable percent finer than 0.02 mm is waived.

(4) Surface Course. Materials requirements for construction of aggregate surfaced airfields depend upon whether frost is a factor in the design.

(a) Nonfrost Areas. Material used for airfields should be sufficiently cohesive to resist abrasive action. It should have a liquid limit no greater than 35 and a plasticity index between 4 and 9. It also should be graded for maximum density and minimum volume of voids to enhance optimum moisture retention while resisting excessive water intrusion. Gradation should consist of an optimal combination of coarse and fine aggregates to ensure minimum void ratios and maximum density. This material will exhibit cohesive strength as well as intergranular shear strength. Recommended gradations are shown in Table 3-6. If the fines fraction of the material does not meet plasticity characteristics, the material may be modified by adding chemicals. Chloride products can, in some cases, enhance moisture retention, and lime can be used to reduce excessive plasticity.

(b) Frost Areas. Where frost is a consideration, a layered system should be used. The percentage of fines should be restricted in all the layers to facilitate drainage and reduce the loss of stability and strength during thaw periods. Use gradation numbers 3 and 4 shown in Table 3-6 with caution, since they may be unstable in a freeze-thaw environment.

Table 3-6
Gradation for Aggregate Surface Courses (Percent Passing)

Sieve Designation	No. 1	No. 2	No. 3	No.4
25.0 mm (1”)	100	100	100	100
9.5 mm (3/8”)	50-85	60-100	--	--
No. 4	35-65	50-85	55-100	70-100
No. 10	25.50	40.70	40-100	55-100
No. 40	15.30	24-45	20-50	30-70
No. 200	8-15	8-15	8-15	8-15

Note: The percent by weight finer than 0.02 mm (0.04 in.) shall not exceed 3 percent.

(5) Compaction. Compaction requirements for the subgrade and granular layers are expressed as a percent of maximum CE 55 density as determined by using CRD-C653, *Standard Test Method for Determination of Moisture-Density Relations of Soils*. For granular layers, compact the material to 1 00 percent of maximum CE 55 density. Select materials and sub-qrades in fills must have densities equal to or greater than the values shown in Table 3-7, except that fills will be placed at no less than 95 percent compaction for cohesionless soils (PI - 5; LL 25) or 90 percent compaction for cohesive soils (PI > 5; LL > 25). Subgrades in cuts must have densities equal to or greater than the values shown in Table 3-7. Subgrades occurring in cut sections will be either compacted from the surface to meet the densities shown in Table 3-7 removed and replaced before applying the requirements for fills, or covered with sufficient material so that the uncompacted subgrade will be at a depth where the in-place densities are satisfactory. Depths in Table 3-7 are measured from the surface of the aggregate, and not the surface of the subgrade.

Table 3-7
Compaction Requirements for Helicopter Pads and Slide Areas

	Cohesive Soils					Cohesionless Soils			
Percent	100	95	90	85	80	100	95	90	85
Depth Below Pavement Surface, millimeters	100	150	200	250	300	150	250	325	400
(inches)	(4)	(6)	(8)	(10)	(12)	(6)	(10)	(13)	(16)

c. Drainage. Drainage is a critical factor in aggregate surface airfield design, construction, and maintenance. It should be considered prior to construction; and, when necessary, serve as a basis for site selection.

(1) Provide adequate surface drainage to minimize moisture damage. Quick removal of surface water reduces absorption and ensures more consistent strength and reduced maintenance. Drainage must not result in damage to the aggregate surfaced airfield through erosion of fines or erosion of the entire surface layer. Ensure changes to the drainage regime can be accommodated by the surrounding topography without damage to the environment, or the newly constructed slide area or pad.

(2) The surface geometry of an airfield should be designed so that drainage is provided at all points. Depending upon the surrounding terrain, surface drainage can be achieved by a continual cross slope, or by a series of two or more interconnecting cross slopes.

(3) Provide adequate drainage outside the airfield area to accommodate maximum flow. Use culverts sparingly, and only in areas where adequate cover of granular fill is provided over the culvert. Evaluate drainage for adjacent areas to determine if rerouting is needed to prevent water from other areas flowing across the airfield.

d. Maintenance. The two primary causes of deterioration of aggregate surfaced areas requiring frequent maintenance are the environment and traffic. Rain or water flow will wash fines from the aggregate surface; traffic action causes erosion of surface materials. Maintenance should be performed at least every six months, and more frequently if required. Frequency of maintenance will be high for the first few years of use, but will decrease over time to a constant value. Most of the maintenance will consist of grading to remove ruts and potholes and replacing fines. Occasionally, the surface layer may

have to be scarified, additional aggregate added to restore original thickness. and the wearing surface recompacted to the specified density.

e. Dust Control. A dust palliative prevents soil particles from becoming airborne as a result of wind or traffic. Dust palliatives used on traffic areas must withstand abrasion. An important factor limiting use of dust palliatives in traffic areas is the extent of surface rutting or abrasion that will occur under traffic. Some palliatives will tolerate deformations better than others, but ruts in excess of 13 millimeters (1/2 inch) will usually destroy any thin layer or shallow-depth penetration dust palliative treatment. A wide selection of materials for dust control is available, Several materials have been recommended for use and are discussed in AFJMAN 32-1019.

8. SURFACE DRAINAGE. Design of surface drainage shall be in accordance with TM 5-820-1/ AFM 88-5, Chapter 1.

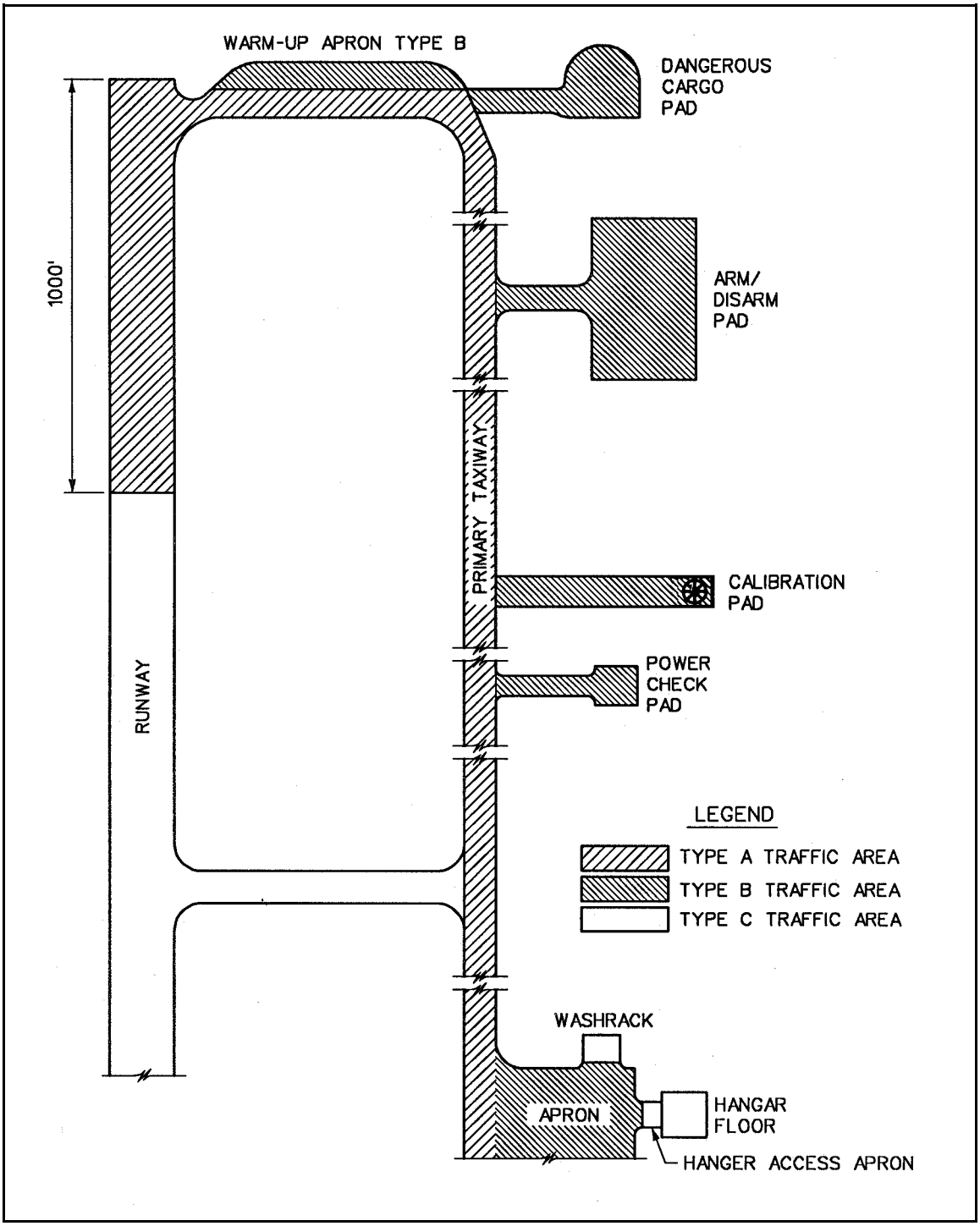


Figure 3-1. Typical layout of traffic areas for Air Force light-load and auxiliary airfield pavements

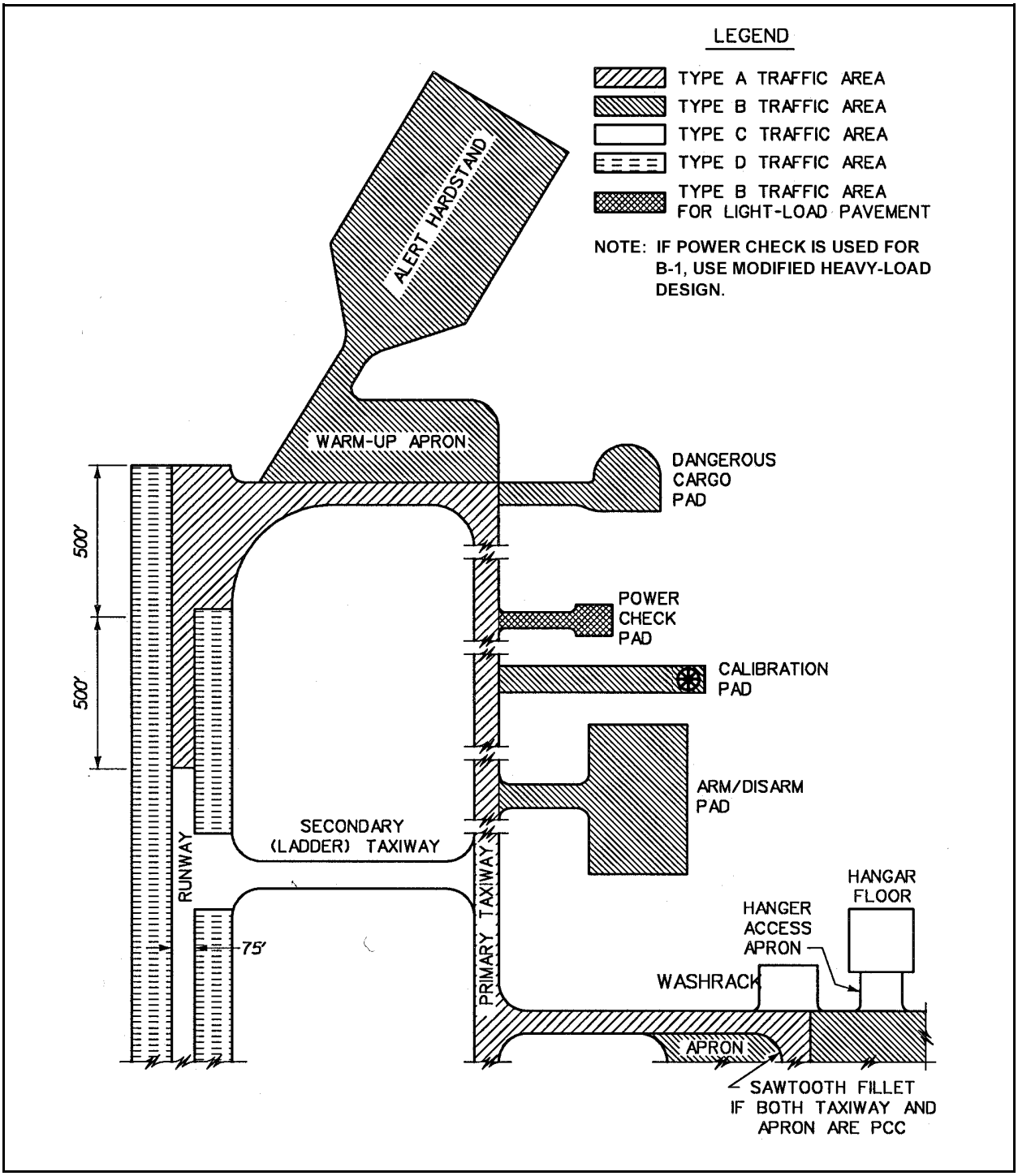


Figure 3-2. Typical layout of traffic areas for Air Force medium- and modified-heavy-load airfield pavements

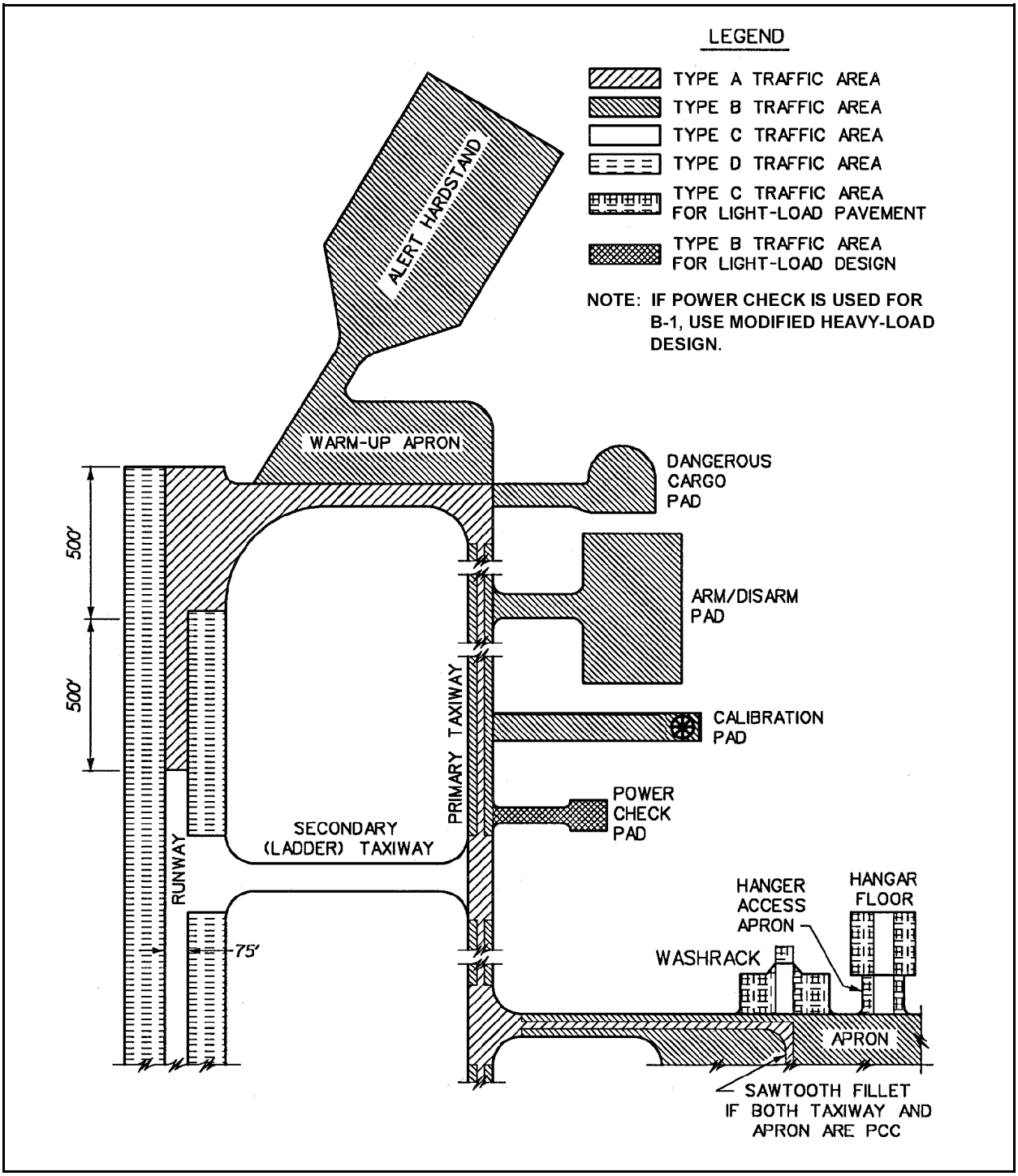
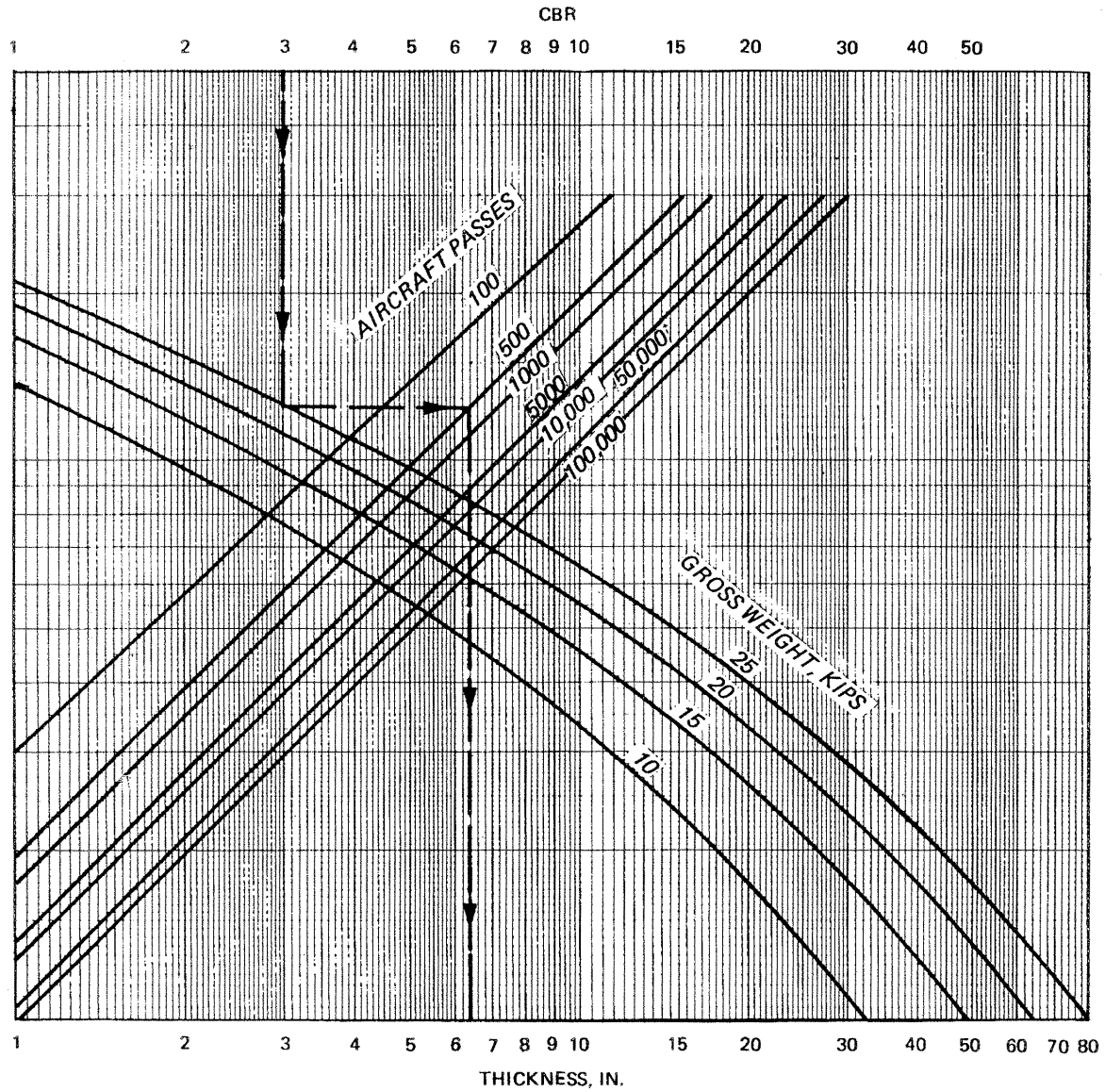


Figure 3-3. Typical layout of traffic areas for Air Force heavy-load airfield pavements



CONVERSION FACTORS
MILLIMETERS = 25.4 X INCHES
KILOGRAMS = 0.4535 X POUNDS

Figure 3-4. Aggregate surfaced design curves for helicopters

CHAPTER 4

NAVY AND MARINE CORPS AIRFIELD REQUIREMENTS

1. TRAFFIC. Traffic is an important input for pavement thickness design. An airfield pavement shall be designed to support a forecast number of loadings by one or more types of aircraft expected to use the facility over the design period. This requires information related to:

a. Aircraft types (gear configurations).

b. Maximum gross weight of each aircraft type.

c. Lateral wander associated with each aircraft type.

d. Predicted number of operations of each aircraft type over the design life of the pavement.

2.T RAFFIC AREAS. Airfield pavements are categorized by traffic area as a function of either lateral traffic distribution or aircraft weight or both. The three principal traffic areas recognized on Navy and Marine Corps air stations are primary, secondary, and supporting. For purposes of standardization and for preparation of the Tri-Service design criteria, a primary area corresponds to an Air Force B traffic area and a secondary traffic area corresponds to an Air Force C traffic area. These designated traffic areas for a typical airfield layout plan are shown in Figure 4-1.

a. Primary Traffic Areas. Primary traffic areas require high pavement strength due to the combination of high operating weights and channelized traffic. Primary traffic areas include:

(1) First 305 meters (1,000 feet) of runways.

(2) Primary taxiways.

(3) Holding areas.

(4) Aprons.

b. Secondary Traffic Areas. Secondary traffic areas are normally subjected to unchannelized traffic and aircraft operating at lower weights than primary traffic areas. Secondary traffic areas include:

(1) Runway interiors.

(2) Intermediate taxiway turnoffs.

c. Supporting Areas. Supporting areas are not intended for normal aircraft operations. They are designed to withstand occasional passes of aircraft on an emergency basis. Supporting traffic areas include:

(1) Inner 3 meters (10 feet) of runway shoulders.

(2) Stabilized portions of runway overruns.

(3) Blast protective pavement.

3.AIRCRAFT LOADINGS. Factors which must be considered in pavement thickness design are the landing gear configuration, weight distribution, gear loads, number of wheels, wheel spacing, tire width, and tire inflation pressure. These characteristics are different for each aircraft and will result in a different pavement response. All aircraft expected to use the facility over the design period shall be considered in the pavement thickness design.

a. Aircraft Types. A landing gear assembly shall consist of a single wheel for smaller aircraft, or dual and dual tandem wheels for larger aircraft. Figure 4-2 illustrates the various multiwheel landing gear assemblies and lists typical aircraft for each.

b. Design Weight. The maximum static gear loads are used for pavement thickness design. Table 4-1 presents the design gear loads and other characteristics for Navy and Marine Corps aircraft. To use the design curves herein, the design gear load must be converted to the design gross aircraft weight (typically, the maximum gross take-off weight) by assuming that 95 percent of the gross aircraft weight is carried by the main gears. The design gear loads given in Table 4-1 represent the maximum static gear loads expected to be applied to a pavement.

c. Use of Other Gear Loads in Design. Gear loads other than those listed in Table 4-1 may be used for design when required. Since certain areas of an airfield (e.g., runway shoulders, runway overruns) do not normally carry fully loaded aircraft, they do not need to be designed for the maximum gross weight.

d. Hangar Floors. Aircraft in hangars are not normally loaded with cargo, fuel, or armaments. Hangar floors shall be designed for the empty weight of the aircraft. When exact data are not available, 60 percent of the maximum gross weight of the aircraft shall be used. Aircraft hangar floors or apron pavements shall not be designed for jacking loads as long as the foot print of the jack is equal to or greater than the contact area of the combined tires on the aircraft gear being elevated.

e. Standard Design Aircraft. One aircraft in each gear assembly group has been designated the representative aircraft for that group. The tabulation below identifies these five standard aircraft types which are to be used as default values in the design of rigid and flexible pavements only when site-specific aircraft loadings are not available.

Standard Design Aircraft Types

Landing Gear Assembly	Representative Aircraft	Tire Pressure Mpa (psi)	Design Gear Load, kg (lb)
Single	F-14	1.65 (240)	13,608 (30,000)
Dual	P-3	1.31 (190)	30,845 (68,000)
Single Tandem	C-130	0.65 (95)	38,100 (84,000)
Dual Tandem	C-141	1.24 (180)	70,310 (155,000)
Twin Delta Tandem	C-5A	0.79 (115)	86,190 (190,000)

Table 4-1
Aircraft Characteristics and Design Loadings

Type	DOD Designation	Type of Loading Gear	Design Gear Load (lb)	Design Tire Pressure (psi)	Pass/Coverage[3] Chan.	Pass/Coverage[3] Unchan.	Empty Weight (lb)	Maximum Take-off Weight (lb)	Wing Span (ft)	Length (ft)	Wheel Base (in.)	Tread (in.)	Main Gear Tire Spacing A (in.)[4]	Main Gear Tire Spacing B (in.)[5]
Attack	A-3B	S	37,000	245	3.48	14.96		78,000	72.5	76.4			--	--
	A-4M	S	12,500	200	11.63	23.26	10,500	24,500	27.5	41.25	160.5	93.5	--	--
	A-5	S	29,500	300	9.27	18.54	38,000	80,000	53.3	76.5	264.0	150.5	--	--
	RA-5C	S	38,000	350	8.82	17.64	38,800	81,700	53.3	76.5	264.0	150.5	--	--
	A-6E	S	28,700	200	7.67	15.35	36,600	60,400	53.0	55.75	206.0	132.0	--	--
	A-7K	S	21,000	200	8.97	13.91	21,800	42,000	38.7	46.1	188.1	113.9	--	--
	AV-8B	Special	15,000	125	3.89	7.47	12,000	24,000	30.3	45.7	135.0			
Fighter	F-4E	S	22,500	300	13.70	27.39	31,800	58,000	38.4	58.3	279.0	215.0	--	--
	F-8E	S	18,000	265	13.69	27.39	19,700	34,300	85.7	54.5			--	--
	F-14	S	30,000	240	8.58	17.00	36,700	72,600	64.1	61.98	276.5	192.0	--	--
	F/A-18	S	21,000	200	8.22	16.44	30,000	51,900	40.4	56.0	213.7		--	--
Trainer	T-1	S	9,000	200	13.69	27.39							--	--
	T-2C	S	7,000	165	14.10	28.20	8,000	14,000	37.9	38.8	155.0	221.0	–	–
	TC-4C	T		123				36,000	78.3	67.9		290.0	–	–
	TA-4F/J	S		350				24,500	27.5	46.2			--	--
	T-39A	S	9,000	165	12.45	24.89	10,000	18,700	44.4	43.8	174.0	86.0	--	--
	T-28D	S	4,300	60	10.85	21.02	6,700	9,000	41.0	33.0	144.0	162.0	--	--
	T-34C	S	1,500	60			2,200	3,000	33.3	28.8			--	--
	T-44A	S	4,500	90	12.99	24.75	6,300	9,600	50.3	35.5	147.5	153.0	–	–
	T-45A	S		125	11.68	22.31		14,500	30.8	39.3	170.0	154.0	--	--
Patrol	P-3C	TT	68,000	190	3.45	6.49	66,200	143,000	99.7	116.8	357.0	374.0	26.0	--
	S-3A	S	19,000	245	10.43	20.87	26,864	46,000	68.7	53.3	225.0	165.0	--	--
Transport and Tanker	C-1A	S		142			20,640	26,800	69.7	42.3	106.9	222.0	–	–
	C-2A	S		235	7.91	15.69		60,000	80.6	56.8	278.4	234.0	–	–
	C-5A	TDT	190,000	115	0.83	1.05	318,000	837,000	222.7	247.8	765.1	449.5		
	C-17	TRT	260,000		1.37	1.9	279,000	580,000	2088.0	2038.0			--	--
	C-121	T	81,000	170	3.45	6.18			123.0	113.6	599.0	336.0	28.0	--
	C-130	ST	84,000	95	4.36	8.56	72,000	175,000	132.6	97.8	388.0	171.0	--	60.0
	KC-10	SBTT	212,000	181	3.77	5.59	271,000	599,000	165.3	182.3	869.0	416.0	--	--
	KC-135	TT	142,000	155	3.37	5.97	104,300	301,600	130.8	136.3	708.0	265.0	35.8	59.8
	C-141B	TT	155,000	180	3.49	6.25	140,000	344,900	160.0	145.0	678.7	251.0	32.5	48.0
	C-9B	T	51,300	152	3.85	7.18	62,000	108,000	93.3	119.3	638.5	196.0	25.0	--
	C-117	S	15,300	56	5.56	11.11		36,800	85.0	64.4	440.0	222.0	--	--
	C-118A	T	54,300	124	3.48	6.39	59,000	112,000	117.5	106.8	432.0	296.5	29.0	--
Bomber	B-52	TTB	250,000	240	1.58	2.15	230,000	480,000	185.0	162.0	597.0	136.0	62.0	--

(Continued)

S = Single Tricycle, T = Dual Tricycle, TDT = Twin Delta Tandem, ST = Single Tandem Tricycle, TT = Dual Tandem Tricycle

NOTES:
1. Blank spaces indicate data not readily available.
2. This data represents the best available figures at the time of publication. The user should update this information for later models of the design aircraft.
3. Values given are for rigid and flexible pavements. Pass to Coverage Ratios for flexible pavements for aircraft with Dual Tandem Tricycle Gear are equal to one-half the value shown. All Tandem Wheel Aircraft produce only one maximum stress for each pass of the gear for rigid pavements.
4. A represents the transverse tire spacing on one main gear.
5. B represents the longitudinal tire spacing on one main gear.

Table 4-1 (Concluded)

Type	DOD Designation	Type of Loading Gear	Design Gear Load (lb)	Design Tire Pressure (psi)	Pass/Coverage[3]		Empty Weight (lb)	Maximum Take-off Weight (lb)	Wing Span (ft)	Length (ft)	Wheel Base (in.)	Tread (in.)	Main Gear Tire Spacing	
					Chan.	Unchan.							A (in.)[4]	B (in.)[5]
Commercial	B-707	TT	157,000	180	3.30	5.87	146,400	333,600	145.8	152.9	708.0	265.0	34.5	56.0
	B-727	T	98,000	150	3.30	5.88	101,500	209,500	108.0	153.6	760.0	225.0	34.0	--
	B-737	T	54,000	150	3.20	5.80	60,500	125,000	93.0	100.0	447.0	206.0	30.5	--
	B-747	DDT	190,000	195	3.84	5.43	363,000	778,000	195.7	231.3	1,008.0	434.0	43.25	54.0
	B-757-200	TT	105,000	170	3.30	5.88	129,900	220,000	124.5	155.3				
	B-767-200	TT	143,000	183	3.71	6.05	180,540	300,000	156.3	159.1				
	DC-8	TT	172,000	196	3.19	5.82		350,000	148.5	187.4	930.0	250.0	30.0	55.0
	DC-9 Series 10	T	57,000	170	3.61	6.73	50,840	90,500	89.4	104.4	524.4	196.8	24.0	--
	DC-10 Series 30	TT	210,500	165	3.77	5.61	267,197	572,000	165.3	181.6	868.6	429.0	54.0	64.0
	(Center Dual)		91,100	140	2.63	3.96	248,485	466,000						
	L-1011-200	TT	219,000	165	3.66	5.57	249,100	450,000	155.3	177.8	840.0	432.0	52.0	70.0
Early Warning	E-1B	S		151				27,400	72.3	45.2			–	–
	E-2C	S	24,500	260	8.58	17.00	38,100	51,900	80.6	57.6	278.0	233.8	--	--
	E-3A	TT	155,000	180	3.30	5.87	88,000	325,000	145.8	152.9	708.0	265.0	34.5	56.0
	EA-6B	S		230				61,500	53.0	59.8			--	--
	EP-3E	T						142,000	99.7	105.9			--	--
	ES-3A	S		245			34,000	52,500	68.7	53.3	225.0	165.0	--	--
Reconnaissance	UC-12M	S		64				13,500	54.5	43.8	179.4	206.0	--	--
Rotary Wing	AH-1W						10,200	14,750	48.0	58.0	146.4	84.0	--	--
	CH-46E	T			8.01	15.22	16,000	24,300	51.0	84.3	297.6	176.4	20.0	–
	CH-53E	T	26,558	165			33,226	69,750	79.0	90.0	327.0	156.0		
	HH-3A	T						19,100	62.0	72.9	282.5	156.0	--	--
	HH-60H	S			11.94	19.49		21,880	53.7	64.8		104.0	--	--
	MH-53E	T			5.23	9.53	36,745	69,750	79.0	99.0		156.0	15.0	--
	RH-53D	T			5.23	9.53		42,000	72.2	88.6		156.0	15.0	--
	SH-3H	T						21,000	62.0	72.9	282.5	156.0	--	--
	SH-60F	S			11.94	19.49		21,880	53.7	64.9		104.0	–	--
	TH-57B/C							3,350	33.3	39.2	56.5	75.5	–	–
	UH-1N							10,500	48.0	57.3		109.0	–	–
	UH-3H							21,000	62.0	72.9	282.5		–	–
	UH-46E	T	9,800	150			12,550	22,800	51.0	84.4	298.0	176.4		
	VH-3A							19,100	62.0	72.9		156.0	--	--
VTOL	MV-22	T		117	4.72	8.66		57,000	1014.6	747.2	3000.0	156.0	--	--

4.T RAFFIC VOLUME. The traffic type, volume, and pavement design life are essential inputs to the pavement design procedure. Determine the total number of passes of each aircraft type that the pavement will be expected to support over its design life. The minimum design life for Navy and Marine Corps facilities is 20 years. Only aircraft departures are normally included as passes in pavement thickness design. The exception to this is in touchdown areas on runways where the impact due to aircraft performing touch-and-go operations will cause pavement damage. On pavements that are to be used for touch-and-go operations, add the expected number of touch-and-go operations over the design life to the number of departures to arrive at the design traffic. Obtain data for the specific Navy and Marine Corps airfield facility under design to forecast aircraft traffic operations over the design life of the pavement. When site-specific traffic projections are not available, the traffic pass levels listed below are the minimum pass levels to be used in design.

Aircraft	**Total Passes Over 20 Year Design Life**[1]
F-14	300,000
P-3	100,000
C-130	50,000
C-141	25,000
C-5A	25,000

[1] Departures at Maximum Gross Weight.

5.RO LLER-COMPACTED CONCRETE PAVEMENT. Roller-compacted concrete pavement (RCCP) is a rigid pavement and can be used as pavement except for runway and high-speed taxiway pavements for fixed-wing aircraft. RCCP can be used for all helipad and heliport pavements.

6.RESIN MODIFIED PAVEMENT. Resin Modified Pavement (RMP) can be used as an Navy pavement except for fixed-wing runways and high-speed taxiways. RMP can be used for helipads and heliport pavements and for both rotary-wing and fixed-wing parking aprons.

7.PAVED SHOULDERS.

a. Location. Paved shoulders should be provided for airfield and heliport construction as designated in EI 02C013/AFJMAN 32-1013/NAVFAC P-971.

b. Structural Requirements. As a minimum, paved shoulders shall be designed to support a load of 4,535 kilograms (10,000 pounds) imposed by a single wheel with a tire pressure of 0.69 MPa (100 psi).

8.PAVEMENT DESIGN POLICY. The Navy recognizes PCASE rigid and flexible pavement design programs and consensus industry standard programs in addition to the traditional Navy rigid pavement design program. Designers are encouraged to consider life cycle costs when designing new pavements. When the life of the pavement can be extended by more than 10 times, it is acceptable to increase the pavement thickness by 1 inch or less as determined by the Navy's traditional rigid pavement center

panel loading procedure. Use of the Army/Air Force edge loading condition is another way to provide for improved pavement life cycle costs. Designers shall complete a sensitivity analysis of the above mentioned programs and review with the senior airfield designer in their geographic area of responsibility.

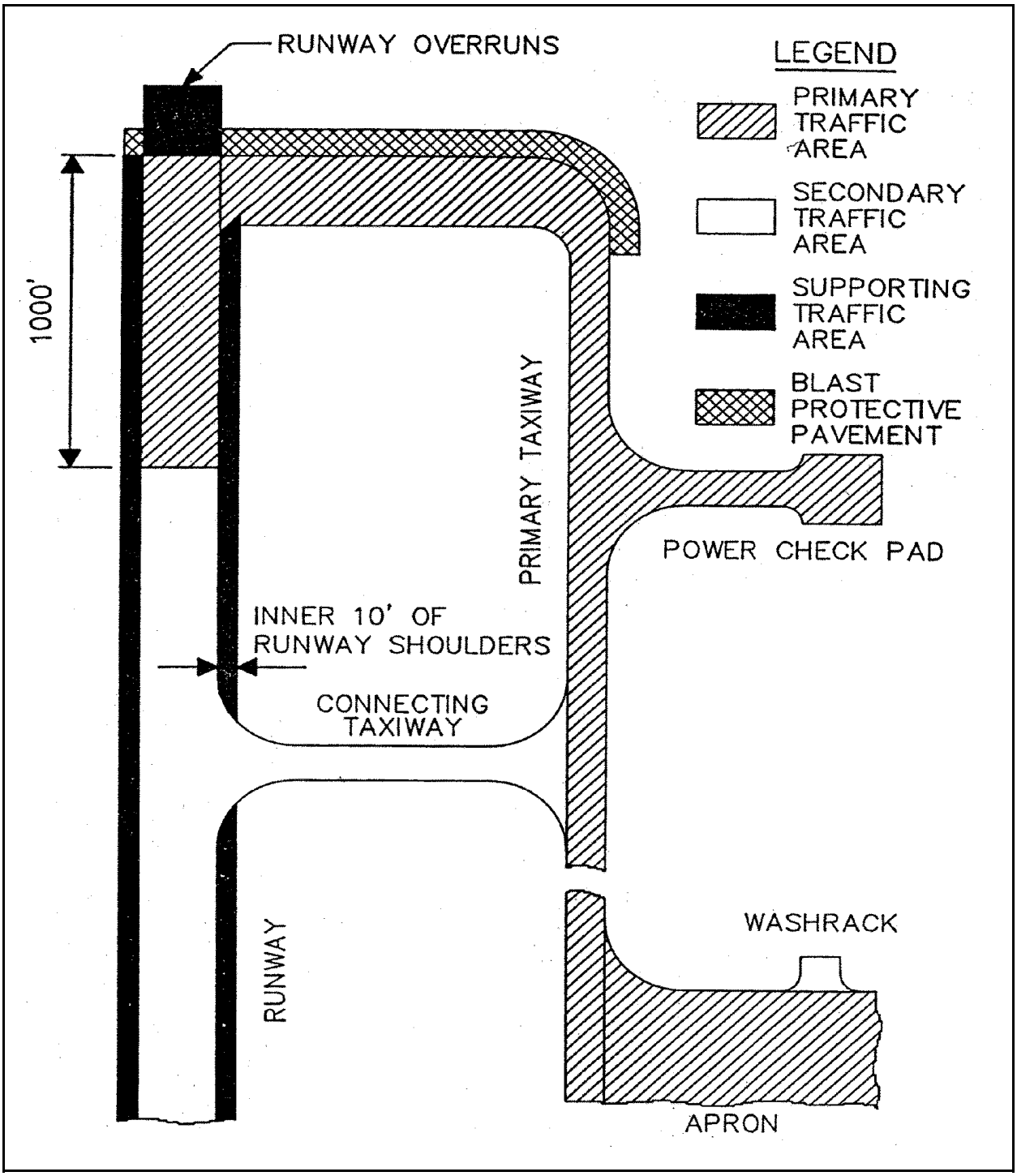


Figure 4-1. Primary, secondary, and supporting traffic areas for Navy and Marine Corps airfield pavements

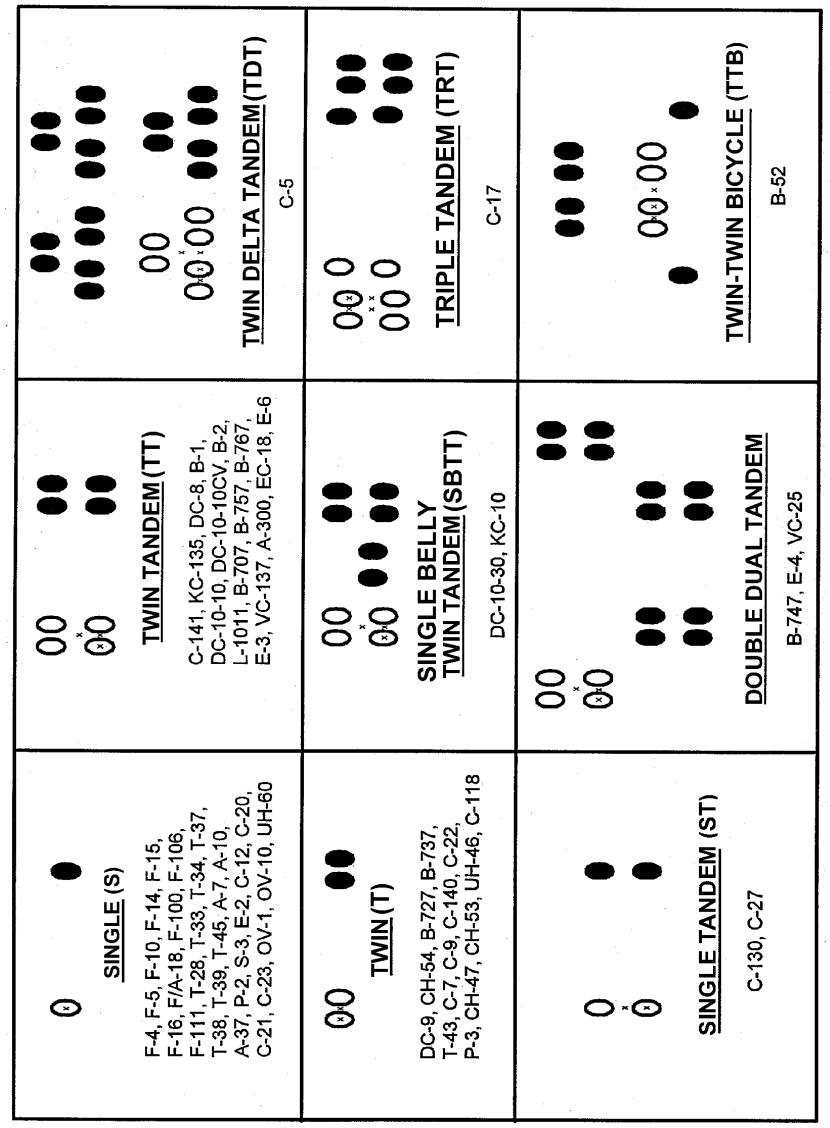


Figure 4-2. Landing gear assemblies

CHAPTER 5

SITE INVESTIGATIONS

1. GENERAL. The design of pavements must be based on a complete and thorough investigation of climatic conditions, topographic conditions, subgrade conditions, borrow areas, and sources of base course, subbase course paving, and other materials. These preliminary investigations will necessitate use of standard tests and all other available information such as aerial photographs, pavement evaluations, condition surveys, construction records, soil maps, geologic maps, topographic maps, and meteorological data. Table 5-1 lists sampling and testing standards used in soil investigations. Although previous investigations should be used to establish preliminary soil characteristics, additional investigations must be performed for final design.

2. SUBGRADE INVESTIGATIONS.

a. Field Reconnaissance. Conduct field reconnaissance with the available topographical, geographical, and soil maps; aerial photographs; meteorological data; previous investigations; and condition surveys and pavement evaluation reports. This step should precede an exploratory boring program.

b. Spacing of Preliminary Borings. The subgrade conditions in the area to be used for airfield pavement construction should be determined by exploratory borings. The recommended maximum spacing of borings should be as shown in the following tabulation, and should be supplemented with additional borings whenever variations in soil conditions or unusual features are encountered.

Item	Spacing of Borings
Runway and taxiways ≤ 61 meters (200 ft) wide	One boring every 61 to 152 meters (200 to 500 feet) longitudinally on alternating side of pavement centerline
Runways >61 meters (200 feet) wide	Two borings every 61 to 152 meters (200 to 500 feet longitudinally (one boring on each side of centerline)
Parking aprons and pads	One boring per 2,325-square-meter (25,000-square-foot) area

c. Depth of Borings. In cut sections, borings should extend to a minimum depth of 3 meters (10 feet) below the finished grade or to rock. In shallow fill sections, borings should extend to a minimum depth of 3 meters (10 feet) below the surface of the natural subgrade or to rock. Shallow fills are those where the effect of the weight of the fill on the natural subgrade is small compared to the weight of the design aircraft (generally 1.8 meters (6 feet) or less). In high-fill sections, borings should extend to a minimum depth of 15.2 meters (50 feet) below the surface of the natural subgrade or to rock. Results of borings will be used to develop boring logs as illustrated in Figure 5-1.

d. Samples. Soil samples should be obtained from the borings for classification purposes. After these samples are classified, soil profiles should be developed and representative soils selected for testing. A typical soil profile is shown in Figure 5-2. Test pits or large-diameter borings may be required to obtain the samples needed for CBR testing, or to permit in-place tests of the various soil layers. The types and number of samples required will depend on the characteristics of the subgrade soils. Subsoil

Table 5-1
Soil Sampling and Testing Standards

Category	Description	ASTM	CRD
Exploratory borings	Auger samples	D 1452	
	Split barrel sampling	D 1586	
	Thin walled sampling	D 1587	
Identification and classification tests	Liquid limit	D 4318	
	Plastic limit	D 4318	
	Sieve analysis	D 422	
	Finer than No. 200 Sieve	D 1140	
	Classification (Unified Soil Classification)	D 2487	
Laboratory tests	Moisture-density relations	D 1557	
	Remolded CBR	C-654	
	Moisture content	D 2216	
	Unconfined compression	D 2166	
	Permeability test	D 2434	
	Consolidation test	D 2435	
In-place tests	Density and moisture content:		
	Sand cone	D 1556	
	Drive cylinder	D 2937	
	Rubber balloon	D 2167	
	Nuclear method (density)	D 2922	
	Nuclear method (moisture content)	D 3017	
	In-place CBR	C-654	
	Dynamic Cone Penetrometer (DCP)	See Note (2)	
	CBR by small aperture		
	Modulus of soil reaction		C-655

Note: (1) Testing for Air Force and Army Pavements will be by ASTM or CRD.
(2) Description and application of the DCP is provided in FM 5-430-00-2/AFJPAM 32-8013, Vol II, Appendix J.

investigations in the areas of proposed pavement should include measurements of in-place water content, density, and strength to ascertain the presence of weak areas and soft layers in the subsoil.

e. Borrow Areas. Where material is to be borrowed, borings should be made in these areas to a depth of 0.6 to 1.2 meters (2 to 4 feet) below the anticipated depth of borrow. One boring should be made for each 930 square meters (10,000 square feet) with a minimum of three borings per borrow area. Samples from the borings should be classified and tested for water content, density, and strength.

f. Environmental Hazards. When conducting subsurface investigations, hazardous or toxic waste material may be located, and appropriate environmental actions will have to be taken. This may be true around fueling areas particularly if replacing an existing fueling apron where fuel has leaked through the pavement and contaminated the soil. There may also be buried materials that have to be dealt with in some areas.

3. SELECT MATERIAL AND SUBBASE FOR FLEXIBLE PAVEMENTS. Areas within the airfield site or within a reasonable haul distance from the site should be explored for possible sources of select material and subbase. Exploration procedures similar to those described for subgrades should be used. Test pits or large auger borings are required to obtain representative samples of gravelly materials.

4. BASE COURSES, DRAINAGE LAYERS, SEPARATION LAYERS, CONCRETE AND BITUMINOUS CONCRETE. Since these pavement layers are generally constructed using crushed and processed materials, a survey should be made of existing sources plus other possible sources in the general area. Significant savings may be made by developing possible quarry sites near the airfield location. This is particularly important in remote areas where no commercial producers are operating and in areas where commercial production is limited in quantity.

5. OTHER CONSTRUCTION MATERIALS. The availability and quality of bituminous materials and portland cement should be determined. The availability and type of lime and fly ash will also aid in the evaluation and applicability of stabilized layers. This information will be helpful in developing designs and alerting designers to local conditions and shortages.

6. SOIL CLASSIFICATION. All soils will be classified in accordance with the unified soil classification system (USCS) as given in American Society for Testing and Materials (ASTM) D 2487. Sufficient investigations will be performed at a particular site so that all soils to be used or removed during construction can be described in accordance with the USCS plus any additional description considered necessary. When classifying soils, be alert to the presence of problem soils such as:

a. Clays that Lose Strength When Remolded. The types of clays that show a decrease in strength when remolded are generally in the CH and OH groups. They are clays that have been consolidated to a very high degree, either under an overburden load or by alternate cycles of wetting and drying, or that have by other means developed a definite structure. They have a high strength in the undisturbed state. Scarifying, reworking, and rolling these soils in cut areas may produce a lower bearing value than that of the undisturbed soils.

b. Soils that Become “Quick” When Molded. Some soils deposits such as silts and very fine sands, (predominantly in classifications ML, SM, and SC) when compacted in the presence of a high water table, will pump water to the surface and become “quick” or “spongy” with a loss of practically all bearing value. The condition can also develop in most silts and poorly drained very fine sands if these materials are compacted at a moisture content higher than optimum. This is because compaction reduces the air voids so that the available water fills practically all the void space.

c. Soils With Expansive Characteristics. Expansive soils are generally those with a liquid limit more than 40 and a plasticity index more than 15. Soils with expansive characteristics give the most trouble when significant changes occur in moisture content of the subgrade during different seasons of the year. TM 5-818-7 may be helpful in identifying expansive soils.

7. SOIL COMPACTION TESTS. Soil compaction tests will be used to determine the compaction characteristics of soils. The degree of compaction required is expressed as a percentage of the maximum density obtained by the test procedure used. Table 5-1 shows test methods to be used for determining density. The laboratory compaction control tests should not be used on soil that contains particles easily broken under the blow of the hammer. Also, the unit weight of certain types of sands and gravels obtained by this method is sometimes lower than the unit weight that can be obtained by field methods. Density tests in these cases should be made under some variations of the test methods, such as vibration or tamping (alone or in combination) to obtain higher laboratory density. In some cases, it may be necessary to construct field test sections to establish compaction characteristics.

8. SOIL STRENGTH. Soil strength is measured by the CBR for use in designing flexible pavements and by the modulus of soil reaction (k) for the design of rigid pavements. Strength tests must be made on material that represents the field condition that will be most critical from a design standpoint. Details of the CBR test procedure are given in CRD-C 654 and details of the modulus of soil reaction test are given in CRD-C 655. Figure 5-3 shows approximate relationships between soil classifications and soil strength values. The relationships will not be used for design of pavements. They are given for checking and estimating, not as a substitute for testing. Guidance in determining soil strength values are presented in Chapters 6 through 8.

9. IN-PLACE SOIL STRENGTH TESTS. Test pits for in-place soil strength tests and associated moisture-density tests should be located at approximately 305-meter (1,000-foot) intervals for runways and taxiways. For parking aprons and pads, one test pit should be located for each 16,720 square meters (20,000 square yards). The number and spacing of test pits may be modified whenever variations in soil conditions or unusual features are encountered.

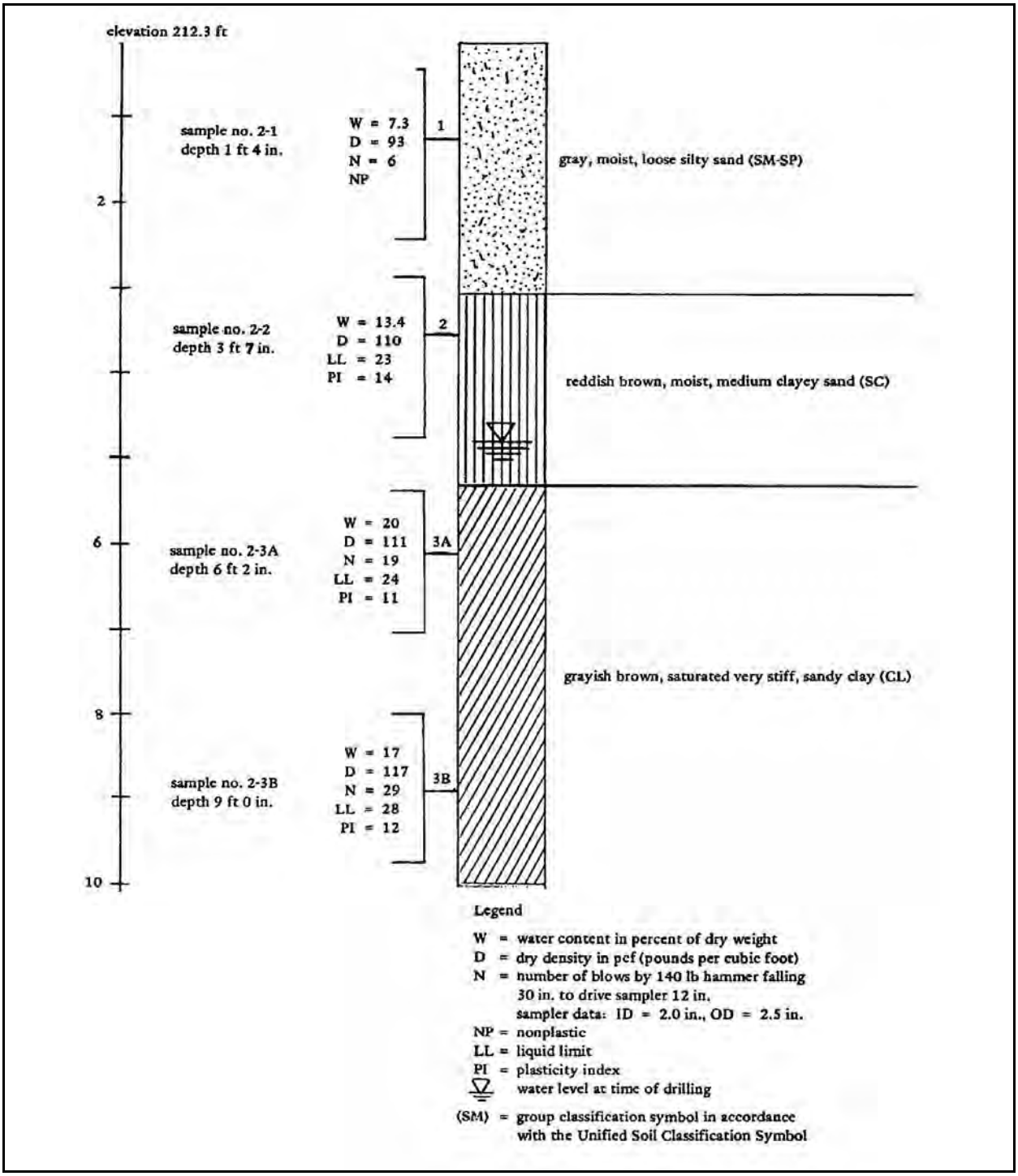


Figure 5-1. Typical boring log

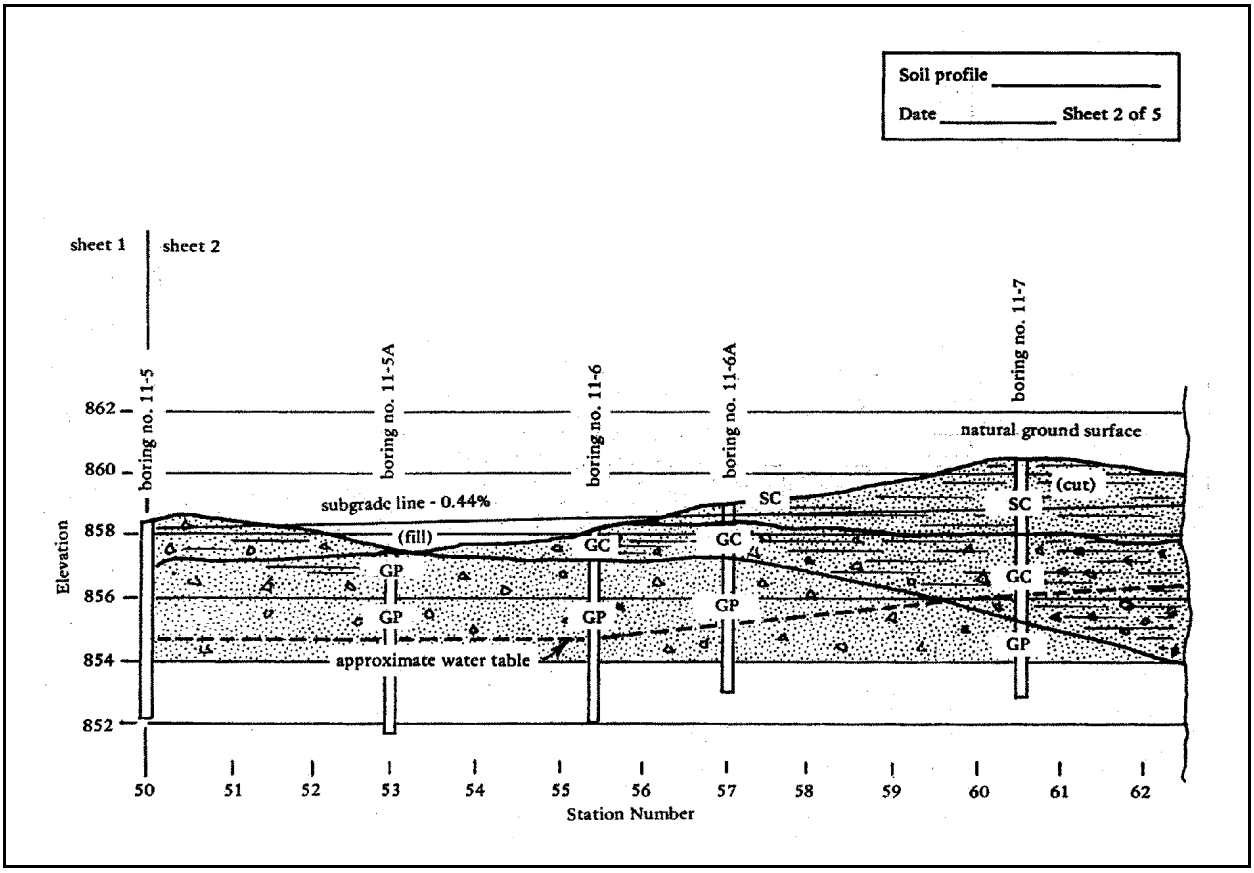
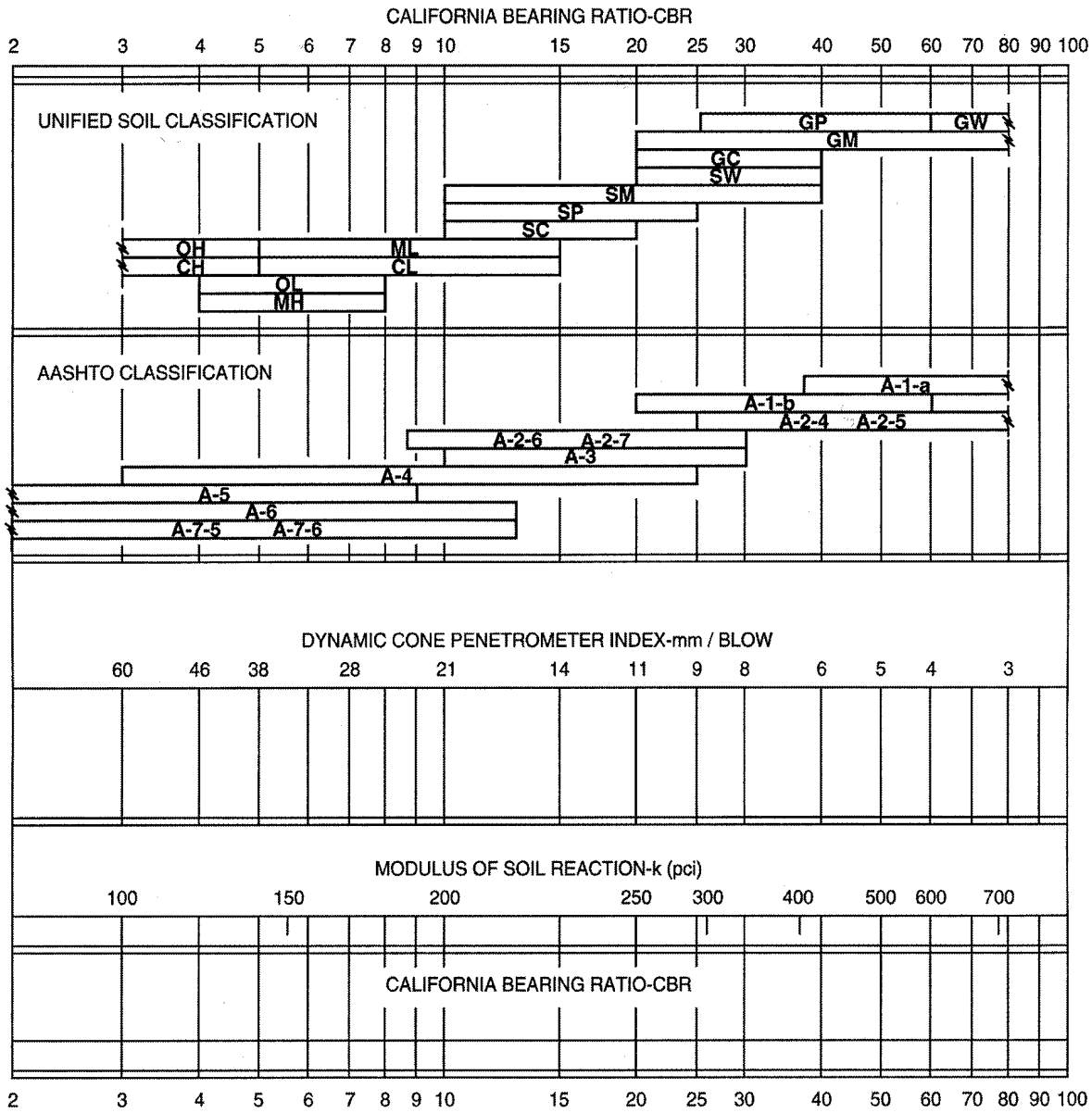


Figure 5-2. Typical soil profile



NOTE: CORRELATIONS NOT TO BE USED FOR DESIGN. AFTER PORTLAND CEMENT ASSOCIATION "THICKNESS DESIGN FOR CONCRETE HIGHWAY AND STREET PAVEMENTS" PACKARD, ROBERT G., 1984. DESIGN LIMIT FOR k IS 500.

Figure 5-3. Approximate relationships of soil classification and soil strength

CHAPTER 6

SUBGRADE

1. SUITABILITY OF SUBGRADE. The information obtained from the explorations and tests previously described should be adequate to enable full consideration of all factors affecting the suitability of the subgrade and subsoil. The primary factors are as follows:

a. The general characteristics of the subgrade soils.

b. Depth to bedrock.

c. Depth to water table (including perched water table).

d. The compaction that can be attained in the subgrade and the adequacy of the existing density in the layers below the zone of compaction requirements.

e. The strength that the compacted subgrade, uncompacted subgrade, and subsoil will have under local environmental conditions.

f. The presence of weak or soft layers in the subsoil.

g. Susceptibility to detrimental frost action.

h. Settlement potential.

I. Expansion potential.

j. Drainage characteristics.

2. GRADE LINE. The soil type together with information on the drainage requirements, balancing cut and fill, flooding potential, depth to water table, depth to bedrock, and the compaction and strength characteristics should be considered in locating the grade line of the top of the subgrade. Generally, this grade line should be established to obtain the best possible subgrade material consistent with the proper utilization of available materials; however, economics of plans for construction must be given prime consideration.

3. SUBGRADE CBR. The strength of the subgrade may be expressed in terms of the CBR for flexible pavement design. The CBR test is described in CRD-C 654. It includes procedures for making tests on samples compacted to the design density in test molds and is soaked 4 days for making in-place CBR tests and for making tests on undisturbed samples. These tests are used to estimate the CBR that will develop in the pavement structure. However, a subgrade design CBR value above 20 is not permitted unless the subgrade meets the requirements for subbases. The CBR selected for the subgrade will be based on the predominant moisture conditions occurring during the life of the pavement. This moisture situation can be obtained from pavement evaluation reports and from soil tests under existing pavements. Where long duration soil moisture conditions cannot be determined with confidence, the soaked laboratory CBR will be selected for the subgrade soil.

a. Laboratory Tests. Tests results should include a full family of curves (Figure 6-1) as described in CRD-C 654. These curves show the three-way relationship of water content at the time of compaction, compacted density, and CBR after soaking. These curves should be studied in view of the actual water contents and densities that can be expected considering the natural scatter when specific control values are specified. The scatter that can be expected with normal control procedures will vary with the soil type. A spread of plus or minus 2 percent can be anticipated for soils with low optimum moisture contents (in the range of 10 percent), whereas a spread of plus or minus 4 percent can be anticipated for soils with high optimum moisture contents (in the range of 25 percent). Poor construction control may result in even greater scatter. A comparable scatter in the density can also be expected. After the range of moisture contents and densities that can be expected during actual construction is estimated, the range of CBR values that will result from these variations in moisture and density should be determined. The design CBR value for the specific soil tested should be selected near the lower part of the range. The following steps along with Figure 6-1 illustrate the selection of a design CBR value.

(1) Step A. Determine moisture/density relationship (CRD-C 653) at 12, 26, and 55 blows/layer. Plot density to which soil can be compacted in the field. For the clay of this example, use 95 percent of maximum density. Plot the desired moisture content range. For the clay of this example, use ±1-1/2 percent of optimum moisture content for approximately 13 and 16 percent. Shaded area represents compactive effort greater than 95 percent and within ±1-1/2 percent of optimum moisture content.

(2) Step B. Plot laboratory CBR (CRD-C 654) for 12, 26, and 55 blows/layer.

(3) Step C. Plot CBR versus dry density at constant moisture content. Plot attainable compaction limits of 1,770 and 1,840 kg/m^3 (110.6 and 115 lb/ft^3) for this example. The hatched area represents attainable CBR limits for desired compaction 1,770 and 1,840 kg/m^3 (110.6 to 115 lb/ft^3) and moisture content (13 to 16 percent). CBR varies from 11 (95 percent compaction and 13 percent moisture content) to 26 (15 percent moisture content and maximum compaction). For design purposes, a CBR at the low end of range is used. In the example, a CBR of 12 with a moisture content specified between 13 and 16 percent is selected.

b. In-place Tests and Tests on Undisturbed Samples. Where an existing pavement at the site has a subgrade constructed to the same standards as the job being designed, in-place tests or tests on undisturbed samples may be used in selecting the design CBR value. Also, where no compaction is anticipated, as in the layers below the zone of compaction, tests should be conducted on the natural material. The in-place CBR may be used where little increase in moisture is anticipated, such as coarse grained cohesionless soils, soils which are at least 80 percent saturated in the natural state, and soils under existing similar pavements which have reached the maximum water content expected, and thus no soaking is required. When in-place tests or tests on undisturbed soils are used, a statistical approach is recommended for selecting the design CBR. An illustration of selecting the design CBR is as follows: Given 20 CBR test values from a runway site.

(1) CBR = 4, 4, 4, 4, 5, 5, 5, 5, 5, 6, 6, 6, 6, 6, 7, 7, 8, 8, 10, and 11. This is a total of 20 separate tests.

(2) Percent of CBR values equal to or greater than each different value:

CBR	Number Equal to or Greater than Each Different Value	Percent Equal to or Greater than Each Different Value
4		
4		
4		
4	20	(20/20)100 = 100
5		
5		
5		
5		
5	16	(16/20)100 = 80
6		
6		
6		
6		
6	11	(11/20)100 = 55
7		
7	6	(6/20)100 = 30
8		
8	4	(4/20)100 = 20
10	2	(2/20)100 = 10
11	1	(1/20)100 = 5

(3) Plot CBR versus percent equal to or greater as shown in Figure 6-2.

(4) Enter Figure 6-2 at 85 percent. Continue to plotted curve then down to design CBR value of 4.7. If a sample from a test location has a value so low (indicating a weak area) that it is not representative of the other tests in the area, obtain additional samples to determine the extent of the area and whether special consideration is required. Where soil conditions vary substantially, a separate set of CBR determinations will be required for each distinct soil type.

4. SUBGRADE MODULUS OF SOIL REACTION. The strength of the subgrade is expressed in terms of the modulus of soil reaction (k) for rigid pavement design. The k value will be determined by the field plate bearing test as described in CRD-C 655.

a. Strength Test. The field plate bearing test will be performed on representative areas of the subgrade, taking into consideration such things as changes in material classification, fill or cut areas, and varying moisture (drainage) conditions which would affect the support value of the subgrade. While it is not practical to perform a sufficient number of field plate bearing tests to make a statistical analysis of the k value, a sufficient number must be performed to give confidence that the selected value will be representative of the in-place conditions. This means that at least two tests for each significantly different subgrade condition should be conducted. Considering the limited number of measured k values that can be obtained, maximum use of other pertinent soil data must be made to aid in the selection of the design k value. The pavement thickness is not affected appreciably by small changes in k values.

Therefore, the assignment of k values in increments of 2.71 MN/m^3 (10 pci) for values up to and including 68 MN/m^3 (250 pci) and in increments of 6.8 MN/m^3 (25 pci) for values exceeding 68 MN/m^3 (250 pci) should be sufficient. A maximum k value of 135 MN/m^3 (500 pci) will be used. Typical values of k for different soil types and moisture contents are shown in Table 6-1.

Table 6-1
Typical Values of Modulus of Soil Reaction

Soils	Typical Range (lb/in.^{2}/in.)	Suggested Default Pavement Design Values if No Test Data is Available (lb/in.^{2}/in.)
Organic Soils (OL, OH, Pt)	25 - 100	25
Silts and Clays of High Plasticity (CH, MH)	50 - 150	50
Silts and Clays of Low Plasticity (CL, ML)	50 - 200	100
Silty and Clayey Sands (SM, SC)	50 - 250	150
Well- and Poorly-Graded Sands (SW, SP)	150 - 400	200
Silty and Clayey Gravels (GC, GM)	200 - 500	250
Well- and Poorly-Graded Gravels (GW, GP)	300 - 500	350

Pavement design should be based on test data or at least historical data of past designs and evaluations at the same facility if at all possible. These default values are suggested for use for preliminary calculations or for small projects or projects where better data simply cannot be obtained. Inadequate testing or evaluation budgets are not an excuse to use these values for final design.

b. Special Conditions. Test Method CRD-C 655 requires a correction of the field plate bearing test results to account for saturation of the soil after the pavement has been constructed. Most fine-grained soils exhibit a marked reduction in the modulus of soil reaction with an increase in moisture content, and a saturation correction is applicable. However, in arid regions or regions where the water table is 3.0 meters (10 feet) or more below ground level throughout the year, the degree of saturation that may result after the pavement has been constructed may be less than that on which the saturation correction is based. If examination of existing pavements (highway or airfield) in the near vicinity indicates that the degree of saturation of the subgrade is less than 95 percent and if there is no indication of excessive loss of subgrade support at joints due to erosion or pumping, the correction for saturation may be deleted.

5. SUBGRADE COMPACTION FOR FLEXIBLE PAVEMENTS - NORMAL CASES. In general, compaction increases the strength of subgrade soils and the normal procedure is to specify compaction in accordance with the following requirements.

a. Subgrades with CBR values above 20.

(1) Army and Air Force. One hundred percent density from ASTM D 1557 except where it is known that a higher density can be obtained practically. Then, the higher density will be required.

(2) Navy and Marine Corps. Compact to 95 percent of ASTM D 1557 maximum density.

b. Subgrades with CBR values of 20 or less.

(1) Fills. Subgrades in fills shall have densities equal to or greater than the values determined from Tables 6-2 through 6-7. Cohesionless fill will not be placed at less than 95 percent nor cohesive fill at less than 90 percent of maximum density from ASTM D 1557. The top 6 inches of subgrade will be compacted to 95 percent of maximum density from ASTM D 1557.

(2) Cuts. Subgrades in cuts shall have natural densities equal to or greater than the values determined from Tables 6-1 through 6-6. When they do not, the subgrade shall be (a) compacted from the surface to meet the densities required, (b) removed and replaced (then the requirements given above for fills apply), or (c) covered with sufficient select material, subbase, and base so that the uncompacted subgrade will be at a depth where the in-place densities are satisfactory. The top 152 millimeters (6 inches) of subgrade will be compacted to 95 percent of maximum density from ASTM D 1557.

c. Natural Densities. The natural densities occurring in the subgrade should be compared with the compaction requirements to determine if densification at the deeper depths under design traffic is a problem. If such densification is likely to occur, means must be provided for compacting these layers, or the flexible pavement structure must be established so that these layers are deep enough that they will not be affected by aircraft traffic.

d. Compaction Levels and Moisture Content. Compaction of soils and aggregates accomplishes two specific purposes: (1) it achieves sufficient density in each layer of material such that future traffic will not cause additional densification and consequent rutting and (2) it achieves the designer's desired engineering properties, normally strength used for the pavement design. The requirements for density in Tables 6-2 through 6.6 coupled with proof rolling (paragraph 9 of Chapter 8) accomplish the first objective. The interaction between specified compaction levels and moisture contents and design strength is described in paragraph 3 of this chapter and Figure 6-1. Controlling field compaction of soils and aggregates using a specified percent of a laboratory compaction value and a specific range of allowable compaction moisture contents based on the laboratory optimum has proven simple and effective in practice for over a half century. Compaction curves of actual rollers in the field conform to the general shape and characteristics of the laboratory compaction curves but will deviate slightly from the actual laboratory curve. This deviation is not generally significant. Failure to control compaction moisture is probably one of the most common causes of failure to achieve specified density in the field. The contractor must thoroughly mix and disperse the moisture in the soils and aggregates and must allow for evaporation which can be significant on clear or windy days in many soils. Some soils such as silts have very steep compaction curves requiring fairly close control of the moisture to achieve compaction. Truly cohesionless soils compact best saturated but a relatively small increase in fines in such materials can make them spongy and uncompactable at saturation. Experience and field evaluation of each soil's behavior under compaction is usually needed to meet the stringent compaction standards used in military airfield construction. It is important to meet both the minimum specified density and to accomplish the compaction within the specified ranges of moisture content.

Table 6-2
Compaction Requirements for Cohesive Subgrades and Select Materials Under Flexible Pavements - Air Force Pavements (LL > 25; PI > 5)

	Depth of Compaction Below the Pavement Surface, inches															
	85 percent				90 percent				95 percent				100 percent			
Airfield Type	A	B	C	D or Overruns	A	B	C	D or Overruns	A	B	C	D or Overruns	A	B	C	D or Overruns
Light	34	32	28	16	27	25	22	12.5	20	19	16	9.5	13	12	10	4
Medium	62	60	50	33	46	45	36	24	31	30	24	16	17	16	13	9
Heavy	69	68	57	36	53	52	41	27	34	34	28	19	21	20	17	11
Modified heavy	68	66	55	35	51	49	40	26	35	33	26	17	21	19	15	10
Shortfield	42	--	--	21	31	--	--	16	22	--	--	12	12	--	--	6
Auxiliary	14	13	11	8	11	10	9	6	8	7	6	4	4	4	3	3

Conversion Factor: Millimeters = 25.4 × inches.

Table 6-3
Compaction Requirements for Cohesionless Subgrades and Select Materials Under Flexible Pavements - Air Force Pavements (LL < 25; PI < 5)

	Depth of Compaction Below the Pavement Surface, inches															
	85 percent				90 percent				95 percent				100 percent			
Airfield Type	A	B	C	D or Overruns	A	B	C	D or Overruns	A	B	C	D or Overruns	A	B	C	D or Overruns
Light	64	60	52	27	50	44	37	21	33	31	26	15	20	19	16	10
Medium	109	106	91	65	85	82	70	48	58	56	47	31	31	30	24	16
Heavy	149	145	105	73	95	94	79	55	65	64	55	34	35	34	28	19
Modified heavy	123	119	102	70	96	93	78	52	65	62	51	33	35	33	26	17
Shortfield	79	--	--	39	59	--	--	29	39	--	--	--	22	--	--	11
Auxiliary	24	23	20	11	19	18	15	9	14	13	11	6	8	7	6	3

Conversion Factor: Millimeters = 25.4 × inches.

Table 6-4
Compaction Requirements for Cohesive Subgrades and Select Materials Under Flexible Pavements - Army Pavements (LL ≤ 25; PI ≤ 5)

	Depth of Compaction Below the Pavement Surface, inches											
	85 percent			90 percent			95 percent			100 percent		
Airfield Type	A	B	C	A	B	C	A	B	C	A	B	C
Class I												
Heliport	--	14	--	--	11	--	--	8	--	--	5	--
Helipad	--	13	--	--	10	--	--	7	--	--	5	--
Class II												
VFR Heliport	--	24	--	--	19	--	--	13	--	--	7	--
VFR Heliport	--	22	--	--	17	--	--	12	--	--	7	--
IFR Heliport	--	25	--	--	20	--	--	14	--	--	8	--
IFR Heliport	--	23	--	--	18	--	--	12	--	--	7	--
Class III												
Runway ≤ 4,000 feet	17	16	13	13	12	10	10	9	7	6	5	4
Runway > 4,000 feet	13	12	11	10	10	8	6	6	5	3	3	2
Class IV												
Runway ≤ 5,000 feet	40	38	32	30	28	24	21	20	16	11	11	8
Runway > 5,000 feet and Runway ≤ 9,000 feet	57	55	46	43	41	33	29	27	22	16	16	12
Runway > 9,000 feet	59	57	47	44	42	34	29	28	23	17	16	13
Class V												
Heliport or Helipad	--	20	--	--	16	--	--	11	--	--	6	--

Conversion Factor: Millimeters = 25.4 × inches.
Meters = 0.3048 × feet.

Table 6-5
Compaction Requirements for Cohesionless Subgrades and Select Materials Under Flexible Pavements - Army Pavements (LL > 25; PI > 5)

	Depth of Compaction Below the Pavement Surface, inches											
	85 percent			90 percent			95 percent			100 percent		
Airfield Type	A	B	C	A	B	C	A	B	C	A	B	C
Class I												
Heliport	--	25	--	--	19	--	--	14	--	--	9	--
Helipad	--	22	--	--	17	--	--	13	--	--	8	--
Class II												
VFR Heliport	--	41	--	--	32	--	--	23	--	--	13	--
VFR Heliport	--	38	--	--	29	--	--	21	--	--	12	--
IFR Heliport	--	44	--	--	35	--	--	25	--	--	14	--
IFR Heliport	--	40	--	--	31	--	--	22	--	--	12	--
Class III												
Runway ≤ 4,000 feet	27	26	23	21	20	18	15	15	13	9	9	7
Runway > 4,000 feet	24	23	19	18	17	14	12	12	10	6	6	5
Class IV												
Runway ≤ 5,000 feet	76	72	61	57	54	45	38	36	30	21	20	16
Runway > 5,000 feet and Runway ≤ 9,000 feet	104	100	85	79	77	65	54	52	43	29	28	22
Runway > 9,000 feet	106	103	87	81	79	66	56	54	44	30	28	23
Class V												
Heliport or Helipad	--	30	--	--	27	--	--	19	--	--	11	--

Conversion Factor: Millimeters = 25.4 × inches.
Meters = 0.3048 × feet.

Table 6-6
Compaction Requirements for Navy and Marine Corps Flexible Pavements

Aircraft	Depth of Compaction Below the Pavement Surface, inches											
	85 percent			90 percent			95 percent			100 percent		
	Primary	Secondary	Supporting	Primary	Secondary	Supporting	Primary	Secondary	Supporting	Primary	Secondary	Supporting
	Cohesive Soils											
Single wheel	39	37	14	31	29	11	23	22	8	15	14	5
P-3	45	43	18	35	34	14	25	24	10	15	14	6
C-130	41	39	18	31	30	14	22	21	10	12	11	5
C-141	57	54	26	42	40	19	28	27	13	16	15	10
C-5A	57	56	32	39	38	23	25	24	15	14	13	9
	Cohesionless Soils											
Single wheel	65	62	23	51	49	18	37	35	13	23	22	8
P-3	78	75	34	61	58	25	43	41	17	25	24	10
C-130	79	75	34	59	56	26	39	37	18	22	21	10
C-141	102	98	69	79	76	38	54	52	24	28	27	17
C-5A	125	124	74	88	87	51	53	52	30	25	24	15

Conversion Factor: Millimeters = 25.4 × inches.

6. SUBGRADE COMPACTION FOR RIGID PAVEMENTS - NORMAL CASES. Compaction improves soil strength and ensures that densification with resulting voids under the concrete slab does not occur. Subgrade soils that gain strength when remolded and compacted will be prepared in accordance with the following criteria.

Table 6-7
Compaction Requirements for Shoulders

Percent Compaction	[1]Depth of Compaction in inches for Cohesive Subgrades and Select Materials (LL ≤ 25; PI ≤ 5)	[1]Depth of Compaction in inches for Cohesionless Subgrades and Select Materials (LL > 25; PI > 25)
85	17	29
90	14	23
95	10	16
100	6	10

[1] Depth is measured from pavement surface.
Conversion Factor: Millimeters = 25.4 × inches.

a. Compacting Fill Sections. Fills composed of soil having a plasticity index (PI) greater than 5 or a liquid limit (LL) greater than 25 will be compacted to not less than 90 percent of ASTM D 1557 maximum density. Fills composed of soil having a PI equal to or less than 5 and an LL equal to or less than 25 will be compacted as follows: the top 152 millimeters (6 inches) will be 100 percent of ASTM D 1557 maximum density; the remaining depth of fill will be 95 percent of ASTM D 1557 maximum density. Large fills on natural soil should be analyzed for bearing capacity and settlement using conventional soil mechanics.

b. Compacting Cut Sections. The top 152 millimeters (6 inches) of subgrades composed of soil having a PI greater than 5 or an LL greater than 25 will be compacted to not less than 90 percent of ASTM D 1557 maximum density. If the natural subgrade exhibits densities equal to or greater than 90 percent of other ASTM D 1557 maximum density, no compaction is necessary other than that required to provide a smooth surface. Soils having a PI equal to or less than 5 and an LL equal to or less than 25 will be compacted as follows: the top 152 millimeters (6 inches) will be 100 percent of ASTM D 1557 maximum density; the 455 millimeters (18 inches) below the top 152 millimeters (6 inches) will be 95 percent of ASTM D 1557 maximum density. Again, if the natural subgrade exhibits densities equal to or in excess of the specified densities, no compaction will be necessary other than that required to provide a smooth surface; in most cases, these densities can be obtained by surface rolling only.

c. Permissible Variations in Field Density. The above criteria should be considered as minimal values. Also, it is emphasized that it is often difficult to correlate field densities with those obtained by practical compaction procedures in the field. Higher densities should result in higher foundation strengths and thus thinner pavements which may offset the added cost of compaction. Experience has shown that the highest densities for all but the special cases (that is, soils that lose strength when

remolded, become “quick” when remolded, or have expansive characteristics) result in lower permanent deformations, less susceptibility to pumping, and improved overall performance.

7. TREATMENT OF PROBLEM SOILS. Although compaction increases the strength of most soils, some soils decrease in stability when scarified, worked, and rolled. There are also some soils that shrink excessively during dry periods and expand excessively when allowed to absorb moisture. When these soils are encountered, special treatment is required. General descriptions of the soils in which these conditions may occur and suggested methods of treatment are outlined as follows:

a. Clays that Lose Strength When Remolded. These types of clays have a high strength in the undisturbed state. Scarifying, reworking, and rolling these soils in cut areas may produce a lower bearing value than that of the undisturbed soils. When such clay soils are encountered, bearing values should be obtained for both the undisturbed soil and the soil remolded and compacted to the design density at the design moisture content and adjusted to the future moisture content conditions. If the undisturbed value is the higher, no compaction should be attempted, and construction operations should be conducted to produce the least possible disturbance of the soil. Since compaction cannot be effected in these cases, the total thickness design above the subgrade may be governed by the required depth of compaction rather than the CBR requirements.

b. Soils that Become “Quick” When Molded. It is difficult to obtain the desired densities in these silts and very fine sands at moisture contents greater than optimum. Also, during compaction of the base, the water from a wet, spongy silt subgrade will often enter the subbase and base with detrimental effects. The bearing value of these silts and very fine sand is reasonably good if they can be compacted at the proper moisture content. Drying is not difficult if the source of water can be removed, since the soils are usually friable and can be scarified readily. If the soils can be dried, normal compaction requirements should be applied. However, removing the source of water is often very difficult and in some cases impossible in the allotted construction period. In cases of high water table, drying is usually not satisfactory until the water table is lowered, as recompacting operations will again cause water to be pumped to the surface. Local areas of this nature are usually treated satisfactorily by replacing the soil with subbase and base materials or with a dry soil that is not critical to water. There are cases where drainage is not feasible and a high water table cannot be lowered, or cases where such soils become saturated from sources other than high water table and cannot be dried out (as in necessary construction during wet seasons). In such cases, the subgrade should not be disturbed, and additional thickness of base and pavement should be used to ensure that the subgrade will not be overstressed or compacted during subsequent traffic by aircraft.

c. Soils with Expansive Characteristics. Soils with expansive characteristics, if highly compacted, will swell and produce uplift pressures of considerable intensity if the moisture content of the soil increases after compaction. This action may result in intolerable differential heaving of flexible pavements. Where the amount of swell is less than about 3 percent (as determined from soaked CBR test), special consideration will not normally be needed. However, where an airfield subgrade includes interspersed patches of soil with different swell characteristics, even amounts of swell less than 3 percent may require special consideration.

(1) Proper moisture content and density. A common method of treating a subgrade with expansive characteristics is to compact it at a moisture content and to a unit weight that will minimize expansion. The proper moisture content and unit weight for compaction control of a soil with marked expansion characteristics are seldom the optimum moisture content and unit weight determined by the compaction test. These factors may be determined from a study of the relations between moisture content, unit weight, percentage of swell, and CBR for a given soil. A combination of moisture, density,

CBR, and swell that will give the greatest CBR and density consistent with a tolerable amount of swell must be selected. The CBR and density values so selected are those that must be considered in the design of overlying layer thickness. Field control of the moisture content must be carefully exercised because if the soil is too dry when compacted, the expansion will increase; and if it is too wet, low unit weight will be obtained and the soil will shrink during a dry period and then expand during a wet period. This method requires detailed testing and extensive field control of compaction.

(2) Overburden load. In order to limit swell of expansive soils, it may be desirable to provide overburden if expansion cannot be limited by other procedures to acceptable amounts. Special swell tests normally will be needed to determine the amount of weight (overburden) necessary to restrict the swell to tolerable magnitudes. These tests can be variations of the standard soaked CBR test described in CRD-C 656, or they can be specially designed tests using a consolidometer apparatus.

(3) Special solutions. Special solutions to the problem of swelling soils are sometimes possible and should not be overlooked where pertinent. For instance, where climate is suitable, it may be possible to place a permeable layer (aquifer) over a swelling soil to maintain the swelling soil in a saturated condition. Moisture buildup in this layer maintains the soil in a stable, swelled condition. Designs must, of course, be based on the swelled CBR and density values of such a material when so treated. Other possible solutions are treatment with lime (TM 5-822-14/AFJMAN 32-1019), replacement of the swelling soil, or working the soil to make it more uniform.

d. Design Considerations for Special Cases. Whenever subgrades are given special treatments that cause their resulting strength or their resulting density to be less than when normally treated, these lesser values must be considered in design of the overlying layers. When a low CBR results, sufficient thickness of overlying structure must be provided to protect a subgrade of such low strength. When a low density results, the thickness of overlying material must be such that the density versus depth requirements of the specifications are met.

8. STABILIZED SUBGRADES. Subgrades can be stabilized by the addition of lime, cement, or a combination of these materials with flyash. Design of pavements using stabilized soils is discussed in Chapter 9 of this document and in TM 5-822-14/AFJMAN 32-1019. Lime should not be used with soils containing sulfates.

9. SUBGRADES IN FROST AREAS. In areas where frost susceptible subgrade soils will be subjected to cycles of freeze-thaw, pavements must be designed in accordance with the requirements of Chapter 20.

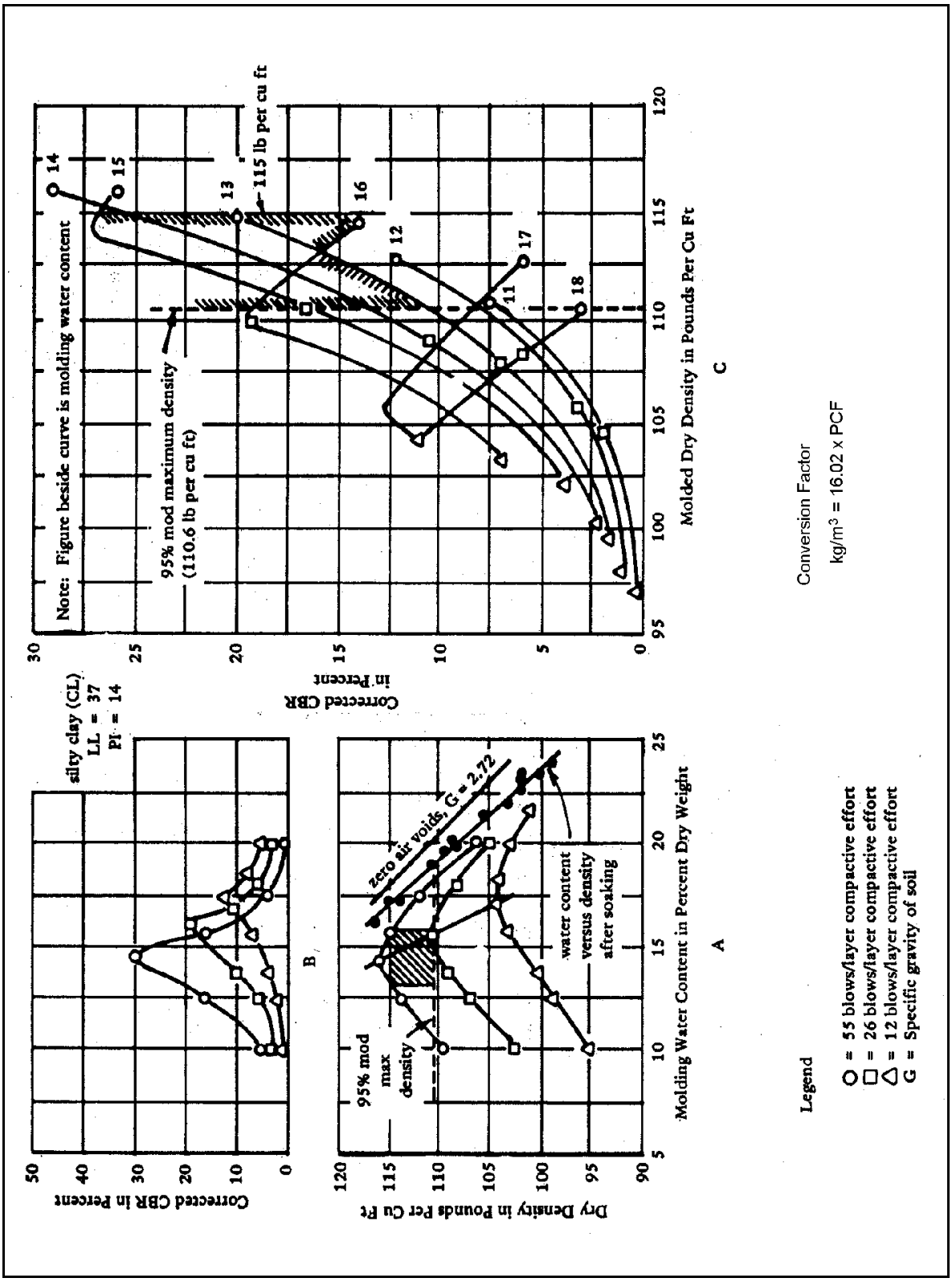


Figure 6-1. Procedure for determining laboratory CBR of subgrade soils

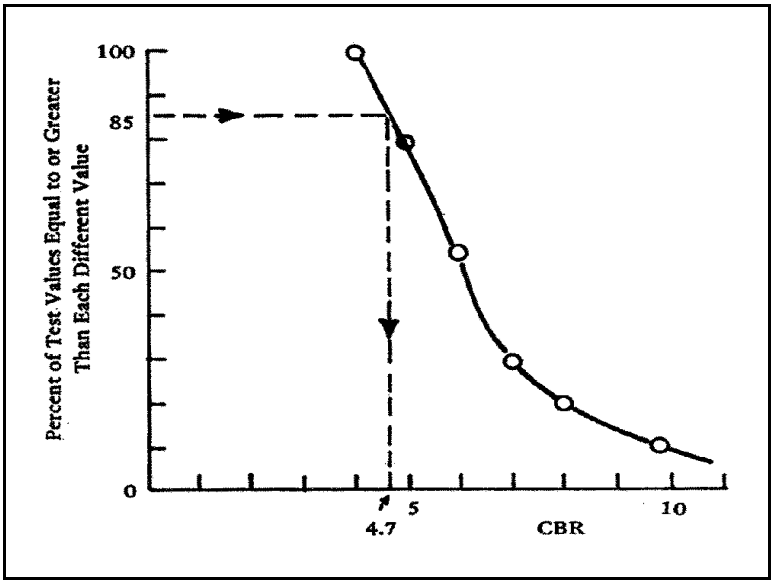


Figure 6-2. Selection of design subgrade CBR using in-place tests

CHAPTER 7

SELECT MATERIALS AND SUBBASE COURSES FOR FLEXIBLE PAVEMENTS

1.GEN ERAL. It is common practice in flexible pavement design to use locally available or other readily available materials between the subgrade and base course for economy. The Navy and Marine Corps designate these layers as subbases and require a minimum CBR of 30. The Army and Air Force refer to these layers as subbases when the design CBR is above 20 and as select materials subbase when the CBR is 20 or less. Minimum thicknesses of pavement and base have been established to eliminate the need for subbases with design CBR values above 50. Guide specifications have been prepared for select materials and subbases. Where the design CBR value of the subgrade without processing is in the range of 20 to 50, select materials and subbases may not be needed. However, the subgrade cannot be assigned design CBR values above 20 unless it meets the gradation and plasticity requirements for subbases. In some cases, where subgrade materials meet plasticity requirements but are deficient in grading requirements, it may be possible to treat an existing subgrade by blending in stone, limerock, sand, etc., to produce an acceptable subbase. However, “blending in” cohesionless materials to lower the plasticity index will not be allowed.

2.MAT ERIALS. The investigations described in Chapter 5 will be used to determine the location and characteristics of suitable soils for select material and subbase construction. Limerock, coral, shell, blast furnace slags (steel slag is not suitable), cinders, caliche, recycled concrete and asphalt, and other such materials in addition to gravels and rock should be considered when they are economical and when they meet the requirements of paragraph 4 entitled SELECTION OF DESIGN CBR. Do not use material which has a swell of 3 percent or greater, as determined from the CBR mold, for subbase. These materials will meet the LA Abrasion requirements of not more than 50 percent..

a. Select Materials. Select materials will normally be locally available coarse-grained soils. Recommended gradation and plasticity requirements for select materials are listed in paragraph 4 entitled SELECTION OF DESIGN CBR.

b. Subbase Materials. Subbase materials may consist of naturally occurring coarse-grained soils or blended and processed soils. Gradation and plasticity requirements for subbases are listed in paragraph 4 entitled SELECTION OF DESIGN CBR. The existing subgrade may meet the requirements for a subbase course or it may be possible to treat the existing subgrade to produce a subbase. Also, admixing native or processed materials will be done only when the unmixed subgrade meets the liquid limit and plasticity index requirements for subbases because it has been found that “cutting” plasticity in this way is not satisfactory. However, it may be permissible to decrease the plasticity of some materials by using lime or portland cement in sufficient amounts to meet the plasticity requirements of subbases. In order to be considered stabilized for thickness design purposes, the soil must meet the minimum strength requirements as shown in Table 7-1.

3.CO MPACTION REQUIREMENTS. Subbases will be compacted to 100 percent of maximum density as determined by ASTM D 1557. Select materials will be compacted to the densities shown in Tables 6-2 to 6-7, except that cohesionless select materials will be placed at no less than 95 percent and cohesive select materials at no less than 90 percent of ASTM D 1557 maximum density.

4.SELECT ION OF DESIGN CBR. The select material or subbase will generally be uniform, and the problem of selecting a limiting condition, as described for the subgrade, does not ordinarily exist. Tests

Table 7-1
Minimum Unconfined Compressive Strength for Cement, Lime, Lime-Cement, and Lime-Cement-Fly Ash Stabilized Soils

	Minimum Unconfined Compressive Strength, psi[1]	
Stabilized Soil Layer	Flexible Pavement	Rigid Pavement
Base course	750	500
Subbase course, select material or subgrade	250	200

[1] Unconfined compressive strength determined at 7 days for cement stabilization and 28 days for lime, lime fly ash, or lime-cement-fly ash stabilization.

are usually made on soaked remolded samples; however, where existing similar construction is available, CBR tests should be made in-place on material when it has attained its maximum expected water content or on undisturbed soaked samples. The procedures for selecting test values described for subgrades apply to select materials and subbases. Experience has shown that CBR tests on gravelly materials in the laboratory have tended to give CBR values higher than those obtained in tests in the field. The difference is attributed to the processing necessary to test the sample in the 152-millimeter (6-inch) mold, and to the confining effect of the mold. Therefore, the CBR test is supplemented by gradation and Atterberg limits requirements for subbases, as shown in Table 7-2. Suggested limits for select materials are also indicated. In addition to these requirements, the laboratory CBR must be equal to or higher than the CBR assigned to the material for design purposes.

Table 7-2
Gradation and Atterberg Limit Requirements for Subbases and Select Materials

			Maximum Permissible Value[1]			
	Maximum[1]		Gradation Requirements Percent Passing			
Material	Design CBR	Size, mm (in.)	2.0 mm (No. 10)	.075 mm (No. 200)	LL	PI
Subbase	50	75 (3)	50	15	25	5
Subbase	40	75 (3)	80	15	25	5
Subbase	30	75 (3)	100	15	25	5
Select material	20	75 (3)[2]	--	25[2]	35[2]	12[2]

Note: LL signifies liquid limit; PI signifies plasticity index.
[1] EI 02C202/AFJMAN 32-1016 contains maximum values for open graded and rapid draining materials.
[2] Suggested limits.

a.Navy Minimum Subbase CBR. On Navy airfield pavements, material with a minimum CBR of 30 should be used in the upper 152 millimeters (6 inches) of the subbase.

b.Exceptions to Gradation Requirements. Cases may occur in which certain natural materials that do not meet the gradation requirements may develop satisfactory CBR values in the field. Exceptions to the gradation requirements are permissible when supported by adequate in-place CBR tests on construction that has been in service for several years.

c.Example. As an example of the selection of a design CBR for subbases or select materials, consider the following material.

Soaked laboratory CBR = 40
Maximum size, millimeters (inches) = 50 (2.0)
Percent passing 2.0 millimeters (No. 10) = 85
Percent passing 0.075 millimeters (No. 200) = 14
Liquid limit = 12
Plasticity index = 3

The design CBR for this material would be 30 rather than the measured value of 40 because 80 percent passing the 2.0 millimeters (No. 10) sieve is the maximum permitted for higher CBR values and this material has 85 percent passing.

5.SEPARATION LAYERS. The gradation requirements shown in paragraph 4 are the maximum allowable limits. The designers can and should include additional gradation requirements to ensure that this material will meet the requirements for a separation layer as described in EI02C202/AFJMAN 32-1016. These additional gradations are dependent on the base course or drainage layer gradations and the gradations of the existing subgrade material; therefore, the designer should tailor these changes for each project.

6.STABILIZED SELECT MATERIALS AND SUBBASES. The design of pavements using stabilized soils is discussed in Chapter 9 of this document and in TM 5-822-14/AFJMAN 32-1019.

7.DESIGN FOR SEASONAL FROST CONDITIONS. In areas where the pavement will be subject to cycles of freezing and thawing, Army and Air Force pavements will be designed in accordance with the requirements in Chapter 9.

8.DRAINAGE LAYERS. The requirements for drainage layers used for subbase are presented in EI 02C202/AFJMAN 32-1016 and NAVFAC DM 21.06. For pavements in nonfrost areas and having a subgrade with a permeability greater than 20 feet/day, one can assume that the vertical drainage will be sufficient such that no drainage layer is required. Also, flexible pavements in nonfrost areas with a total thickness of 8 inches or less are not required to have a drainage layer. For pavements requiring drainage layers, the design of the drainage layer shall be based on the premise that the capacity of the drainage layer should be greater than the volume of water entering the pavement and that the drainage layer, if saturated, should reach a degree of drainage of 0.85 within 1 day after the inflow of water stops. The degree of drainage for the drainage layer is defined as the volume of water that has drained from the layer over a specified time period divided by the total volume of water in the layer that can be drained by gravity.

CHAPTER 8

AGGREGATE BASE COURSES

1. USE OF AGGREGATE BASE COURSES. Aggregate base courses may be required for one or more of the following reasons: distribution of load, provide drainage, protect from frost, provide uniform bearing surface for the pavement surfacing, replace unsuitable soils, provide working platform, increase strength of pavement system or prevent pumping.

2. MATERIALS FOR AGGREGATE BASE COURSES IN FLEXIBLE PAVEMENTS. Aggregate base-course materials for flexible pavement must be of high quality and conform to agency guide specifications. Since natural cementation of the materials listed in subparagraphs c, d, e, f, and g occurs progressively in place, there is a potential that the strength of these materials will increase with time, resulting in higher CBR values than laboratory tests indicate. Special requirements for aggregate base courses in frost areas are discussed in Chapter 20. Aggregate base courses used as drainage layers must meet the requirements of EI 02C202/AFJMAN 32-1016. Those materials generally used as aggregate base-course materials are listed below:

a. Graded Crushed Aggregate Base Course--100 CBR. Stone is quarried from formations of granite, traprock, and limestone. Gravel is quarried from deposits of river or glacial origin. The stone and gravel are crushed and screened to produce a dense-graded crushed aggregate material meeting requirements of guide specifications. The percentage of loss shall not exceed 40 when tested in accordance with ASTM C-131. The material shall also meet the requirements listed in CEGS 02722 for flat and elongated particles, liquid limit and plasticity index, and magnesium sulfate soundness when tested in accordance with ASTM C 88. Gradation requirements for graded crushed aggregates are as follows:

Table 8-1
Gradation Requirements for Graded Crushed Aggregates, Base Courses, and Aggregate Base Courses

	Percentage by Weight Passing Square-Mesh Sieve		
Sieve Designation	No. 1	No. 2	No. 3
50-mm (2-in.)	100	--	--
37.5-mm (1-1/2-in.)	70-100	100	--
25-mm (1-in.)	45-80	60-100	100
12.5-mm (1/2-in.)	30-60	30-65	40-70
4.75-mm (No. 4)	20-50	20-50	20-50
2.0-mm (No. 10)	15-40	15-40	15-40
0.425-mm (No. 40)	5-25	5-25	5-25
0.075-mm (No. 200)	0-8	0-8	0-8

b. Aggregate Base Course--80 CBR. This material is a blend of crushed and natural materials processed to provide a dense graded mix (often referred to as mechanically stabilized base course).

The percentage of loss shall not exceed 50 when tested in accordance with ASTM C-131. The material shall also meet the requirements listed in CEGS 02722 for flat and elongated particles, liquid limit and plasticity index, and magnesium sulfate soundness when tested in accordance with ASTM C 88. The gradation requirements are the same as for the 100 CBR material, but fractured faces relaxed to 50 percent.

c. Blast Furnace Slag. Slag is a by-product of steel manufacturing. It is air cooled, crushed, and graded to produce a dense mix. Fines from other sources may be used for blending. Requirements for a graded crushed aggregate apply. Only blast furnace slag will be used. Minimum required unit weight of slag is 1,200 kg/m^3 (75 lb/ft^3).

d. Shell Sand. Shell sand consists of oyster and clam shells that have been crushed, screened, and blended with sand filler. Ratio of the blend shall be not less than 67 percent shell to 33 percent sand. Refer to local specifications where available.

e. Coral. Coral consists of hard cemented deposits of skeletal origin. Coralline limestone quarried from inland deposits and designated quarry coral is the most structurally sound of the various coral materials available. Other types useful for base materials are reef coral and bank run coral. Quarry coral is crushed and graded to a dense mix. The following gradation is recommended:

Sieve Designation	**Percent Passing**
50-mm (2-in.)	100
37.5-mm (1-1/2-in.)	70-100
19-mm (3/4-in.)	40-90
4.75-mm (No. 4)	25-60
0.425-mm (No. 40)	5-20
0.075-mm (No. 200)	0-10

The percentage of wear (ASTM C-131) is not to exceed 50.

f. Limerock. Limerock is a fossiliferous limestone of the oolitic type generally located in Florida.

g. Shell Rock. Shell rock or marine limestone are deposits of hard cemented shells located in North Carolina and South Carolina. Refer to local guide specifications where available. Percentage of loss should not exceed 50 when tested in accordance with ASTM C-131..

h. Stabilized Materials. Stabilized materials consist of granular materials that have been improved by the addition of cement, lime, bitumen, or a combination of those additives with flyash. See Chapter 9 for a discussion of stabilization.

i. Crushed Recycled Concrete. Crushed recycled concrete shall consist of previously hardened portland cement concrete or other concrete containing pozzolanic binder material. The recycled material shall be free of all reinforcing steel, bituminous concrete surfacing, and any other foreign material and shall be crushed and processed to meet the required gradations for coarse aggregate. Crushed

recycled concrete shall meet all other applicable requirements specified below. Recycled concrete to be exposed to sulfates in the ground or water must be checked for sulfate resistance. Contact MAJCOM for guidance.

3. AGGREGATE BASE COURSES FOR ARMY AND AIR FORCE RIGID PAVEMENT.

a. General. Drainage layers generally serve as aggregate base courses under rigid pavements and must meet the requirements of EI 02C202/AFJMAN 32-1016. A minimum aggregate base-course thickness of 102 millimeters (4 inches) will be required over subgrades that are classified as CH, CL, MH, ML, and OL (ASTM D 2487) for protection against pumping except in arid climates where experience has shown that there is no need for the aggregate base course to prevent pumping. In certain cases of adverse moisture conditions (high water table or poor drainage), SM and SC soils may also require aggregate base courses to prevent pumping. Engineering judgment must be exercised in the design of aggregate base-course drainage to ensure that water is not trapped directly beneath the pavement, which invites the pumping condition that the base course is intended to prevent. In addition, aggregate base courses in inlay sections should be constructed to drain toward the outside edge. Daylighting of the aggregate base course may also be required. Care must also be exercised when selecting aggregate base-course materials to be used with slipform construction of the pavement. Generally, slipform pavers will operate satisfactorily on materials meeting aggregate base-course requirements. However, cohesionless sands, rounded aggregates, etc., may not provide sufficient stability for slipform operation and should be avoided if slipform paving is to be a construction option. The designer should consider extending the aggregate base course 1.5 to 3.0 meters (5 to 10 feet) outside the edge of the pavement to provide a working platform for construction equipment.

b. Material Requirements. A complete investigation will be made to determine the source, quantity, and characteristics of available materials. The aggregate base course may consist of natural materials or processed materials, as discussed for flexible pavements. In general, the unbound aggregate base material will be a well-graded, high-stability material. All aggregate base courses to be placed beneath airfield rigid pavements will conform to the following requirements in addition to those requirements in base course guide specifications (sieve designations are in accordance with American Society for Testing and Materials (ASTM E 11):

- Well-graded, coarse to fine.
- Not more than 85 percent passing the 2.0-millimeter (No. 10) sieve.
- Not more than 15 percent passing the 0.075-millimeter (No. 200) sieve.
- PI not more than 8 percent.

However, when it is necessary for the base course to provide drainage, the requirements set forth in EI 02C202/AFJMAN 32-1016 will be followed.

4. AGGREGATE BASE COURSES FOR NAVY AND MARINE CORPS RIGID PAVEMENTS.

a. General. The main structural support element in a rigid pavement is the portland cement concrete slab. The most important function of the aggregate base-course material in a rigid pavement is to provide uniform long-term support to the slab with adequate drainage to prevent pumping and loss of support. The aggregate base course must be constructed of quality material and properly designed to ensure a good foundation. If pumping and loss of support occur, the performance of the concrete slab will be reduced.

b. Material Requirements. Suitable materials for aggregate base courses include natural, processed, manufactured, and stabilized materials which meet ASTM D 2940. These are the most common types of base course materials. Select local materials if possible, and consider local experience and practices when selecting a base material.

c. Gradation. To provide adequate drainage, the base course must contain little or no fines (material that passes the 0.075-millimeter (No. 200) sieve). Gradation requirements assure adequate stability and drainage by the base course under repeated loads. Crushed aggregates have greater stability than round-grained materials.

d. Wear Resistance. Aggregates suitable for base-course material must have the ability to withstand abrasion and/or crushing. Do not use soft aggregates for base course material because they may break down into fines which will inhibit drainage. Use the Los Angeles abrasion test (ASTM C 131) for determining aggregate abrasion resistance. Aggregates suitable for base course shall have a percentage loss in the Los Angeles abrasion test less than or equal to 40 percent.

e. Lean Concrete Bases. Lean concrete mixtures may be used as base material to provide increased support and reduce pumping. They may also be more economical than stabilized bases. Lean concrete refers to a mixture composed of low-cost, locally available aggregates that may not meet specifications for normal concrete mixtures and an amount of portland cement that is usually less than for normal concrete mixtures. Local aggregates, substandard aggregates, and recycled materials may all be used in lean concrete mixtures for base materials. When properly designed, these materials can provide a strong and erosion-resistant base.

(1) Material specifications and gradation requirements for aggregates used in lean concrete mixtures are not as restrictive as those for aggregates used in normal concrete. Aggregate gradations should conform to one of the gradations given in Table 8-2. The aggregate materials should be free from any elongated or soft pieces and dirt. Mix design for lean concrete bases is discussed in Chapter 11.

Table 8-2
Gradations for Lean Concrete Base Materials

Sieve Size (square opening) mm (in.)	Percentage by Weight Passing Sieve		
	A	B	C
50 (2)	100	--	--
37.5 (1.5)	--	100	--
25 (1.0)	55-85	70-95	100
19 (0.75)	50-80	55-85	70-100
4.75 (No. 4)	30-60	30-60	35-65
0.425 (No. 40)	10-30	10-30	15-30
0.075 (No. 200)	0-15	0-15	0-15

(2) Any bond between the lean concrete base and the concrete slab to be placed on top must be prevented to retard reflective cracking. A bond breaking material such as a wax-based curing compound should be placed on top of all lean concrete base courses.

f. Recycled Concrete Bases. Recycled portland cement concrete can serve as an aggregate for use in a granular base course or in recycled concrete base. The concrete must be properly crushed and sized to meet gradation requirements.

g. Geotextile Fabrics. Geotextile fabrics may be considered for reinforcement of the subgrade to provide a working platform for base course construction and to separate the subgrade and base course to maintain the original base course gradation. See NAVFAC DM 7.01 and NAVFAC DM 21.06 for design criteria on geotextile fabrics. The use of geotextile fabric is encouraged to prevent loss of fines from the surrounding soil through subsurface utility lines.

5. STRENGTH OF AGGREGATE BASE COURSES FOR RIGID PAVEMENTS. The modulus of soil reaction k of the unbound base courses will be determined by field plate bearing tests performed on the surface of the compacted base course or by tests on the subgrade and from Figure 8-1. If both methods are used, the lower value obtained by the two methods will be used for the pavement design. A sufficient number of field plate bearing tests must be performed on the top of a finished base course to determine a realistic design K value. Consideration should be given to the variations in base-course thickness, types of materials, and the variation in subgrade strengths. Figure 8-1 yields an effective k value at the surface of the base course as a function of the subgrade k value and base-course thickness. These relationships have been generated by field testing. If the design k value is selected from Figure 8-1, it should be verified in the field. The maximum value for the modulus of soil reaction to be used in design is 135 KPa/mm (500 pci).

6. STRENGTH OF AGGREGATE BASE COURSES FOR FLEXIBLE PAVEMENTS. Because of the effects of processing samples for the laboratory CBR tests and because of the effects of the test mold, the laboratory CBR test will not be used in determining CBR values of base courses. Instead, selected CBR ratings will be assigned as shown in the following tabulation. These ratings have been based on service behavior records and, where pertinent, on in-place tests made on materials that have been subjected to traffic. It is imperative that the materials conform to the quality requirements given in the guide specifications so that they will develop the needed strengths.

Aggregate Base Course	**Design CBR**
Graded Crushed Aggregate	100[1]
Aggregate[2]	80
Limerock	80
Shell Sand	80
Coral	80
Shell Rock	80

Note: See Chapter 6 for open-graded and rapid-draining material requirements
[1] Limited to 80 CBR for Navy and Marine Corps.
[2] Formerly mechanically stabilized aggregate.

7. MINIMUM THICKNESS REQUIREMENTS FOR FLEXIBLE PAVEMENTS. The minimum allowable thicknesses for aggregate base courses in flexible pavements are listed in Table 8-3 for Army airfields, Table 8-4 for Navy and Marine Corps airfields, and Table 8-5 for Air Force airfields. These thicknesses have been established so that the required subbase CBR will always be 50 or less.

Table 8-3
Minimum Surface and Aggregate Base-Course Thickness Requirements for Army Flexible Pavement Airfields, Inches

		100 CBR Base			80 CBR Base[1]		
Airfield Heliport Class	Traffic Area	Surface	Base	Total	Surface	Base	Total
I	B	2	6	8	2	6	8
II	B	2	6	8	3	6	9
III	A	2	6	8	2	6	8
	B	2	6	8	2	6	8
	C	2	6	8	2	6	8
IV	A	4	6	10	5	6	11
(Runway ≤ 5,000 feet)	B	4	6	10	5	6	11
	C	3	6	9	4	6	10
IV	A	4	6	10	5	6	11
(Runway > 5,000 feet)	B	4	6	10	5	6	11
	C	3	6	9	4	6	10
IV	A	4	6	10	5	6	11
(Runway ≥ 9,000 feet)	B	4	6	10	5	6	11
	C	3	6	9	4	6	10
V	B	2	6	8	3	6	9

[1] Florida limerock and graded crushed aggregate (80 CBR) permitted.
Conversion Factor: Millimeters = 25.4 × inches

Table 8-4
Minimum Flexible Pavement Surface and Aggregate Base-Course Thickness Requirements for Navy and Marine Corps Flexible Pavement Airfields

Aircraft Gross Weight kg (kips)	Tire Pressure MPa (psi)	Minimum Thicknesses, mm (in.)		
		Surface	Base[1]	Total
< 5,440 (<12)	All pressures	50 (2)	152 (6)	203 (8)
5,440 to 13,600 (12 to 30)	<1.38 (200)	76 (3)	152 (6)	228 (9)
5,440 to 13,600 (12 to 30)	1.38 (200) or greater	102 (4)	203 (8)	305 (12)
>13,600 (>30)	All pressures	102 (4)	203 (8)	305 (12)

[1] Unbound or stabilized.

Table 8-5
Minimum Surface and Aggregate Base-Course Thickness Requirements for Air Force Flexible Pavement Airfields, Inches

		100 CBR Base			80 CBR Base[1,2,3]		
Airfield Type	Traffic Area	Surface	Base	Total	Surface	Base	Total
Light load	A	4	6	10	5	6	11
	B	4	6	10	5	6	11
	C	3	6	9	4	6	10
	Shoulders	2	6	8	2	6	8
Medium load	A	4	6	10	5	6	11
	B	4	6	10	5	6	11
	C	3	6	9	4	6	10
	D	3	6	9	3	6	9
	Shoulders	2	6	8	2	6	8
Heavy load	A	5	10	15	6	9	15
	B	5	9	14	6	8	14
	C	4	9	13	5	8	13
	D	3	6	9	3	6	9
	Shoulders	2	6	8	2	6	8
Modified heavy load	A	5	8	13	6	8	14
	B	5	8	13	6	8	14
	C	4	8	12	5	8	13
	D	3	6	9	3	6	9
	Shoulders	2	6	8	2	6	8
Shortfield	A	4	6	10	5	6	11
Auxiliary	A	3	6	9	3	6	9
	B	3	6	9	3	6	9
	C	3	6	9	3	6	9
	Shoulders	2	6	8	2	6	8

Note: When the underlying subbase has a design CBR of 80, the minimum base-course thickness will be 6 inches.

[1] Restricted to Florida limerock for heavy load pavements and modified heavy load pavements except that graded crushed aggregate (80 CBR) or cement modified or bituminous modified aggregate will be permitted in type D traffic areas.

[2] Florida limerock or graded crushed aggregate (80 CBR) cement modified or bituminous modified aggregates permitted in type B, C, and D traffic areas for medium load pavements.

[3] Florida limerock or graded crushed aggregate (80 CBR), cement modified or bituminous modified permitted for light load, shortfield, and auxiliary pavements.

Conversion Factor: Millimeters = 25.4 × inches.

8. MINIMUM THICKNESS REQUIREMENTS FOR RIGID PAVEMENTS.

a. Army and Air Force. The minimum thickness of aggregate base course under rigid pavements will be 100 millimeters (4.0 inches) over CH, CL, MH, ML, and OH subgrades or that required to meet minimum thicknesses for drainage layers as shown in EI 02C202/AFJMAN 32-1016.

b. Navy and Marine Corps. The minimum thickness requirements for aggregate base courses are listed in Table 8-6. The minimum thickness for granular materials is set for construction purposes. The additional base thickness required over clays and silts is to aid in preventing pumping. Consider experience with local aggregates and materials when selecting the base course thickness.

Table 8-6
Aggregate Base-Course Minimum Thickness Requirements for Navy and Marine Corps Rigid Pavements

Base Material	Minimum Thickness
Granular Material	152 mm (6 in.)
Cement Stabilized	152 mm (6 in.)
Asphalt Stabilized	152 mm (6 in.)
Asphalt Concrete	102 mm (4 in.)
Lean Concrete Mixture	102 mm (4 in.)

Note: For subgrades classified as CH, CL, MH, ML, or OL, the minimum granular base-course thickness shall be 203 mm (8 in.).

9. COMPACTION AND PROOF ROLLING REQUIREMENTS FOR FLEXIBLE PAVEMENTS. The aggregate base course will be compacted to 100 percent of ASTM D 1557 maximum density. In addition to compacting the base course to the required density, proof rolling shall be performed on the surface of completed aggregate base courses as designated below. Open-graded and rapid-draining layers will not be proof rolled. The layer immediately under lying the open-graded or rapid-draining layer shall be proof rolled instead. The proof roller will consist of a heavy rubber-tired roller having four tires, each loaded to 13,608 kilograms (30,000 pounds) and inflated to 720 kPa (125 psi). Repetitions of the proof roller are expressed as coverages where a coverage is the application of one tire print over each point on the surface of the designated area. TM 5-820-2/AFJMAN 32-1016 presents special proof rolling and compaction requirements for drainage layers.

a. Air Force Bases. Proof roll top of subbase and each layer of base course of type A traffic areas and the center 23 meters (75 feet) of heavy, modified heavy, and medium load runways with 30 coverages.

b. Navy and Marine Corps Airfields. Proof roll top of completed aggregate base course on center 12 meters (40 feet) of taxiways and on center 30.5 meters (100 feet) of runways with eight coverages. To all other paved areas exclusive of runway overrun and blast protection areas, apply four coverages.

c. Army Airfields. On Class IV airfields with runways greater than 1,525 meters (5,000 feet), proof roll top of subbase and each layer of crushed aggregate base course in type A traffic areas and center 23 meters (75 feet) of runways with 30 coverages.

10. COMPACTION REQUIREMENTS FOR ARMY AND AIR FORCE RIGID PAVEMENT AGGREGATE BASE COURSES. High densities are essential to keep future consolidation to a minimum, but thin aggregate base courses placed on yielding subgrades are difficult to compact to high densities. Therefore, the design density in the aggregate base-course materials should be the maximum that can be obtained by practical compaction procedures in the field but not less than:

a. 95 percent of ASTM D 1557 maximum density for aggregate base courses less than 254 millimeters (10 inches) thick.

b. 100 percent of ASTM D 1557 maximum density in the top 152 millimeters (6 inches) and 95 percent of ASTM D 1557 maximum density for the remaining thickness for aggregate base courses 254 millimeters (10 inches) or more in thickness.

11. COMPACTION REQUIREMENTS FOR NAVY AND MARINE CORPS RIGID PAVEMENT AGGREGATE BASE COURSES. Compact granular and cement-treated base courses to 100 percent of maximum density according to ASTM D 1557 and D 558, respectively. Compact asphaltic concrete base courses to 97 percent of the maximum density as determined from the Marshall mix design method.

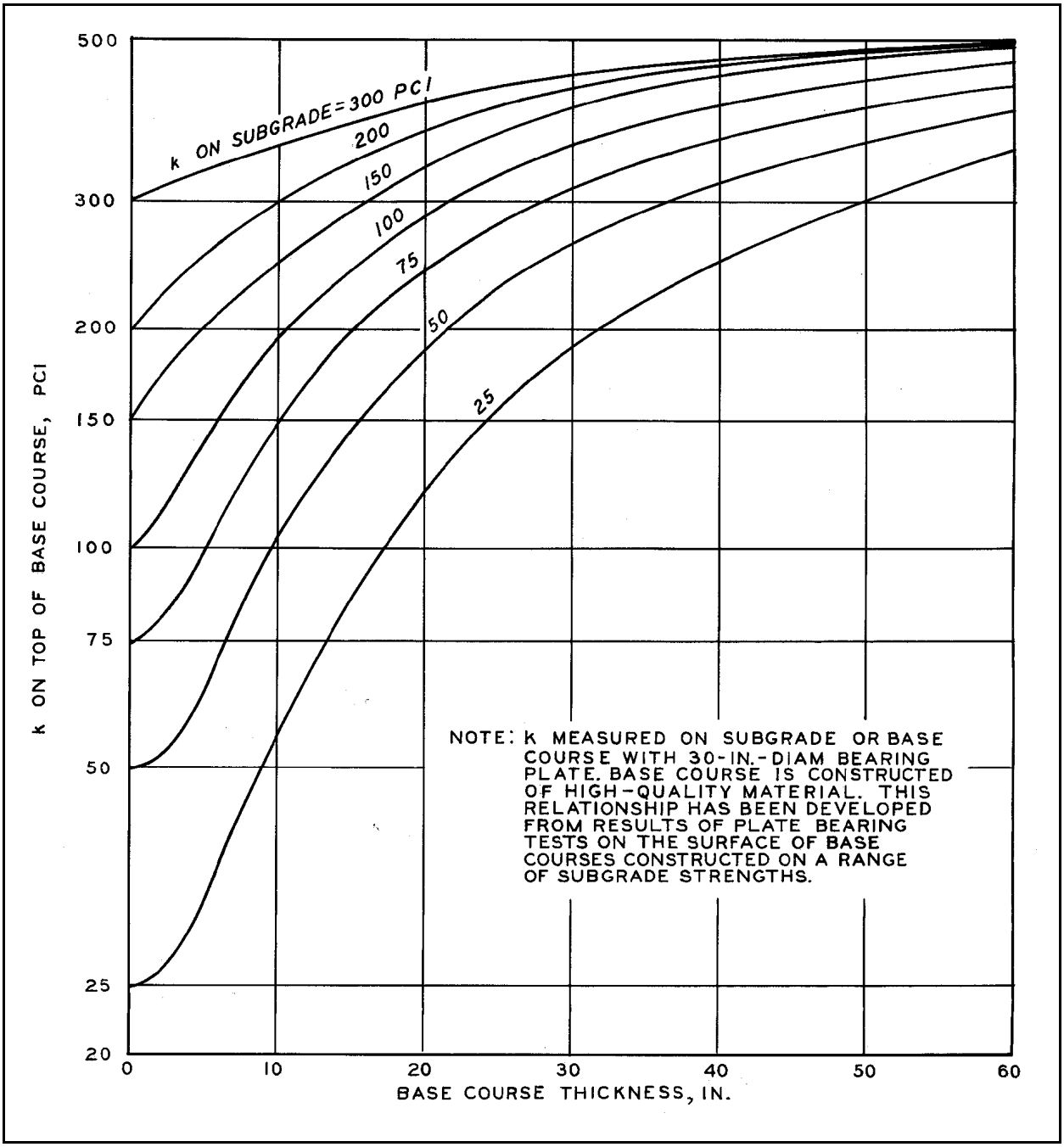


Figure 8-1. Effect of base-course thickness on modulus of soil reaction for nonfrost conditions

CHAPTER 9

PAVEMENT MATERIALS

1. GENERAL. This chapter provides the designer an overview of pavement materials that might be used in military airfield pavements. This overview will include soil and aggregate stabilization, asphaltic concrete, portland cement concrete, and recycled materials. More comprehensive and detailed descriptions, policy, and guidance on uses and limitations, testing requirements, suitable materials, mixture proportioning, and construction can be found in TM 5-822-14/AFJMAN 32-1019 for stabilization, TI 822-08/AFMAN 32-1131 V8(1)/DM 21.11 for asphalt concrete, and TM 5-822-7/AFM 88-6, Chapter 8, for portland cement concrete. In addition, each service also maintains recommended guide specifications for these materials that the engineer can edit for specific jobs. Materials technology evolves constantly, and new guidance on pavements materials is available from HQUSACE (CEMP-ET), appropriate Air Force MAJCOM pavements engineer, or Naval Facilities Engineering Service Center as changes develop. This chapter is a short overview to aid the designer during the design process, and the more comprehensive guidance documents noted above should be consulted concerning each service's specific limitations and requirements for these materials and for preparing individual project specifications.

2. STABILIZATION. . Existing soils or aggregates may not be suitable for use in airfield construction (e.g., poor grading, low strength, or excessive plasticity) or may have other undesirable characteristics (e.g., tendency to shrink or swell with moisture content changes). By stabilizing such materials with appropriate additives, their engineering and construction properties can be improved. Lime, portland cement, and asphalt are the most common stabilizers, but pozzolans (notably fly ash), ground granulated blast furnace slag, and a wide variety of proprietary materials are also available. TM 5-822-14/AFJMAN 32-1019 provides official guidance on use of lime, portland cement, lime-fly ash, and bituminous materials for stabilization. HQUSACE (CEMP-ET), appropriate Air Force MAJCOM pavements engineer, or Naval Facilities Engineering Service Center should be consulted for assistance on use of other stabilizers and conditions not covered in the existing guidance.

a. Purpose. Stabilization is most commonly associated with achieving strength to reduce pavement thickness requirements. However, other equally important and perhaps even more important uses of stabilization include improvement in soil workability, prevention of pumping in rigid pavements, mitigation of adverse volume changes in expansive soils, providing a construction platform to ease and speed construction operations, reduction of effects of adverse weather during construction, and allowing use of an economical local material that fails conventional specifications in lieu of importing more expensive materials from elsewhere.

b. Requirements. Subsequent chapters in this manual provide detailed guidance on how to incorporate stabilized materials in each of the different thickness design methods for flexible and rigid pavements. To qualify for a reduced thickness in these design methods, the stabilized material must achieve a compressive strength of not less than 5.17 MPa (750 psi) for base courses in flexible pavements, 3.45 MPa (500) psi for base courses in rigid pavements, and 1.72 MPa (250) psi for flexible pavement subbases for the Army and Air Force or 1.03 MPa (150 psi) for subbases for the Navy. These strengths are determined after 7 days of curing at 22.8 °C (73 °F) for portland cement and after 28 days of curing at 22.8 °C (73 °F) for lime, slag, and combinations with pozzolanic materials (e.g., lime-fly ash mixtures). In addition to strength, there are specific requirements for durability and material properties that must also be met. Even if a material fails to qualify for the reduced pavement thickness requirements, stabilization may prove desirable for some of the other reasons noted above. If

stabilization results in granular layers sandwiched between relatively impervious layers (e.g., granular base course between an asphalt concrete surface and a stabilized subbase), then this pervious intermediate layer should be positively drained. Because of the potential for poor performance of such geometries, such designs must be approved before use by HQUSACE (CEMP-ET), appropriate Air Force MAJCOM pavements engineer, or Naval Facilities Engineering Service Center.

c. Terminology. The term "*stabilization*" as used in this chapter will encompass the addition of any materials to a soil or aggregate to improve its strength or physical characteristics for use as pavement subgrade, fill, subbase, or base course. As employed here, the term will include combinations with common additives such as lime and portland cement or lime-portland cement-fly ash as well as those materials often referred to as *soil-cement, lean concrete base*, *econocrete*, etc. TM 5-822-14/AFJMAN 32-1019 differentiates between soil stabilization and soil modification where the later only results in an improvement in some property but does not by design cause a significant increase in strength. This level of differentiation is not needed for the generalized discussion of the topic in this chapter, so *stabilization* is used here as an all-inclusive term.

d. Seasonal Frost Areas. Use of stabilized materials in areas subject to seasonal frost must address two extra concerns. First, the stabilized material must be durable for its intended purpose under the freezing and thawing exposure to which it will be exposed. Secondly, many stabilizers (e.g., portland cement or lime) must cure to gain strength, and the necessary chemical reactions to gain strength are greatly retarded and may cease altogether at low temperatures. Consequently, some stabilized materials placed late in the fall may not be able to gain adequate strength prior to the onset of freezing weather. Consequently, local climatic conditions will determine a cutoff date well in advance of anticipated freezing conditions after which date it is not prudent to place stabilized materials. Additional assistance on problems with stabilized materials under seasonal frost exposure is available from the Army Cold Regions Research and Engineering Laboratory, 72 Lyme Road, Hanover, NH 03755.

e. Combinations of Stabilizers. Under some circumstances, it may be desirable to use combinations of stabilizers to take advantage of each stabilizer's characteristics (e.g., use of a combination of lime and then portland cement relying on the lime to improve a plastic clay's workability and the portland cement for more rapid strength gain than available from the slower pozzolanic reactions of lime alone).

f. Mixing. The stabilizer and soil or aggregate to be stabilized may be mixed in situ or mixed at a central plant and then transported to the construction site and placed according to the project specifications. Proper mixing is crucial to stabilizers achieving their desired purpose. Central plants provide the best and most consistent product. In situ mixing may vary from repeated working with a grader to highly sophisticated mixers specifically designed for the task. It is harder to achieve good distribution and mixing of the stabilizer with in situ mixing techniques than with plant mixing. Consequently, stabilizer contents are sometimes increased ½ to 1 percent over the laboratory determined design stabilizer content to account for uncertainties of in situ mixing.

g. Compaction. Stabilized materials must be adequately compacted to achieve their desired purpose. Stabilization is not a substitute for compaction, and poorly compacted stabilized layers are prone to premature failure. Essentially, the compaction equipment and procedures and the quality-control techniques used with conventional earthwork are adequate for stabilized materials. Compaction equipment of sufficient size is needed, and lift thicknesses should be restricted to a maximum of 150 millimeters (6 inches) unless the contractor can demonstrate in the field that project specified density levels are achieved throughout the lift for thicker placements. To check the latter, the density must be measured in the bottom of the lift and not just at the surface or as an average through the entire

lift. Generally, stabilized layers used in subbase and base courses of military airfields should be compacted to 100 percent of the laboratory modified compaction-energy density. TM 5-822-14/AFJMAN 32-1019 provides more comprehensive guidance on requirements for laboratory compaction and testing procedures to be used with different stabilized materials. Addition of the stabilizer changes the laboratory compaction characteristics of the soil or aggregates, and the trends are not always predictable. For example, increasing the percent of portland cement used to stabilize a soil may either shift the laboratory compaction curve up and to the left (i.e., increase maximum density and decrease optimum moisture content) or down and to the right (i.e., decrease maximum density and increase optimum moisture content). On the other hand, increasing lime contents decrease the laboratory maximum density and increase the optimum moisture content for compaction. If field stabilizer contents are increased for in situ mixing as noted in the previous paragraph, this may affect the laboratory maximum density value that the contractor is required to meet in the field, and assessment of the contractor's field compaction must take this into account. For instance, if the lime content is increased in the field over that used in the laboratory, the contractor may encounter problems achieving the specified density because the actual laboratory target density was decreased by the additional lime. When these complex soil-stabilizer interactions are combined with field variation from distribution and mixing of the stabilizer, fairly assessing the contractor's compaction efforts may become difficult. In circumstances where stabilizer contents are being increased in the field, supplemental one-point compaction tests of the in situ stabilized materials may prove helpful for assessing compaction compliance. HQUSACE (CEMP-ET), appropriate Air Force MAJCOM pavements engineer, or the Naval Facilities Engineering Service Center may be consulted for assistance with difficult cases.

h. Curing. In the subsequent sections, curing requirements are identified for many stabilizers. It is crucial that this curing take place adequately for the stabilizer to achieve the desired results. Generally, this means that temperatures must be high enough for the desired chemical reactions to occur, and moisture must be maintained within the material and evaporation stopped or at least severely retarded. Inadequate curing can negate the benefits of stabilization.

i. Testing. Tight financial restraints on military construction today often discourage adequate testing. However, when working with stabilized materials, it is important to verify in the laboratory that the proposed stabilization scheme will achieve the desired results. For instance, it is not sufficient to simply select a suggested lime content for stabilizing a clay because the soils/clay mineralogy or the presence of organic or some iron compounds in the soil may totally change or inhibit the chemical reactions that occur. It is always prudent to perform sufficient laboratory work to verify that the percentages of stabilizer, stabilizer type, and actual soil or aggregate will achieve the desired results when they are mixed, compacted, and cured.

j. Lime Stabilization. Hydrated lime ($Ca(OH)_2$), quick lime (CaO), or the dolomitic variants of these limes are suitable for lime stabilization of soils. Requirements for the limes for soil stabilization are contained in ASTM C 977. Calcium carbonate ($CaCO_3$) is often sold under names such as agricultural lime and is not suitable for soil stabilization.

(1) Mechanisms. Several things happen when lime is added to a soil. As the lime hydrates, it dries the soil. Anhydrous quicklime is particularly effective for this. Some fine clay-sized soil particles agglomerate when lime is added to the soil which results in a decrease in the measured number of clay-sized soil particles. Essentially, a clayey soil fabric becomes siltier, and the soil is easier to work, dry, etc. Also, cation exchange occurs, and the calcium from the lime replaces sodium and potassium in clay minerals. This results in a reduction in plasticity of the soil. The above reactions (drying, particle agglomeration, and cation exchange) occur rapidly after the lime is added to the soil. With time, some, but not all, clays may undergo a further pozzolanic reaction with the lime and develop additional strength

from the resulting calcium silicate and calcium aluminate hydrate compounds. Soil compressive strength gain, after 28-day cures at 22.8 °C (73 °F) from the pozzolanic reaction between lime and some clay minerals may range from negligible to 10.34 MPa (1,500 psi). Typically, a well-compacted, reactive lime-stabilized soil will achieve compressive strengths in the range of 100 to 500 psi.

(2) Uses. Lime added to soil can rapidly dry the soil; it coarsens the particle texture which often makes the soil easier to work; and it reduces the soil's plasticity, making it more workable, generally reducing the soil's strength loss when it is wetted, and often reducing adverse shrinking and swelling behavior. The pozzolanic strength gain, which is typically assessed after 28 days of curing at 22.8 °C (73 °F), can significantly improve soil strength of subgrades and can often meet the strength requirements for a stabilized subbase for flexible pavements. The requirements for stabilized bases are harder to meet with lime alone, and the addition of cement with the lime may be needed to gain the required strength. Many characteristics of lime stabilization make it very useful as a construction expedient and soil improvement additive for difficult plastic clay soils (e.g., drying, coarser texture, reduced plasticity and water susceptibility, construction platform, reduced shrink-swell behavior) rather than for structural strength alone.

(3) Durability. Lime stabilization should provide sufficient durability to accomplish the required objectives under the anticipated exposure conditions.

(a) Moisture. Lime-stabilized soils generally retain over two-thirds of their strength when exposed to water and have performed well in structures exposed to water (e.g., levees, canals, and dams and as expedient (lime-stabilized clay surface) military airfields in Latin America). However, a few clays have shown poor strength retention when soaked in the laboratory. Consequently, some soaked strength tests or the optional wet-dry test (ASTM D 560) limits in TM 5-822-14/AFJMAN 32-1019 may be checked if strength when exposed to soaking or wetting and drying is a critical design parameter.

(b) Seasonal frost exposure. Lime-stabilized materials generally expand and lose strength when exposed to freezing and thawing. As cycles of freezing and thawing increase there is a progressive decrease in the strength of the lime-stabilized material. Generally, the first winter is the critical exposure as extended curing in subsequent seasons will provide additional strength, and there are data to suggest these materials may heal autogenously under favorable curing temperatures. TM 5-822-14/AFJMAN 32-1019 has specific testing criteria and limits based on ASTM D 560 that must be met if the lime-stabilized material is to be exposed to freezing and thawing. Because of the relatively slow rate of pozzolanic strength gain in lime stabilization, adequate time for curing must be allowed prior to the stabilized layer's being exposed to freezing. Consequently, the lime-stabilized material must be in place well in advance (e.g., perhaps 30 days) prior to the onset of freezing weather which shortens the construction season for some areas. Alternatively, it must be protected from freezing (e.g., by placement of overlying pavement layers), and the temperature maintained high enough to allow pozzolanic reactions to occur. Additional assistance on problems with lime-stabilized materials under seasonal frost exposure is available from the Army Cold Regions Research and Engineering Laboratory, 72 Lyme Road, Hanover, NH 03755.

(c) Leaching. There is some limited evidence that soils stabilized with low levels of lime may have the benefits of lime stabilization reduced by leaching over time. The problem appears to be relatively rare and generally associated with low levels of lime stabilization (e.g., 3 percent and less). In general, this should not be an issue for lime stabilization levels for airfield pavements as their strength and durability requirements would normally require lime contents above those where leaching has been a reported problem.

(d) Carbonation. Atmospheric carbon dioxide can react with lime to form calcium carbonate which can adversely affect lime-stabilization reactions. Proper and prompt mixing, storage, compaction, and curing procedures that minimize the exposure of the lime-stabilized soil to atmospheric carbon dioxide avoids the problem. Reported problems have been with highly weathered materials in Africa that were poorly compacted and cured.

(e) Sulfate attack. Lime-stabilized materials are susceptible to sulfate attack if sulfates are present in the soil or water in contact with the stabilized material or if they are present in materials that are being stabilized. The sulfate attack reactions are expansive and highly disruptive. Technical guidance on this problem is incomplete. If lime stabilization is contemplated where sulfates are present, the HQUSACE (CEMP-ET), appropriate Air Force MAJCOM pavements engineer, or Naval Facilities Engineering Service Center should be consulted for up-to-date guidance on this difficult issue.

(4) Suitable Soils. Clayey soils with a plasticity index of 12 or more are generally best suited for lime stabilization. Organic soils and clays containing some iron compounds do not respond well to lime stabilization, and some highly weathered soils may require a larger than expected dosage of lime stabilizer to be effective.

k. Portland-Cement Stabilization. Type I portland cement and, more rarely, Types II, I/II, and III meeting the requirements of ASTM C 150 may be mixed with soils or aggregates to provide a cohesive cemented material often referred to *as soil-cement, econocrete, lean concrete base*, etc.

(1) Mechanisms. When mixed with water, portland cement develops cementing compounds that bind the soil and aggregate particles together. Unlike lime, there is no necessary chemical reaction with the soil particles themselves. Portland cement contains free lime as one of its constituents so the same cation exchange and pozzolanic reactions with clayey soils will occur with portland cement, but these are minor effects compared with the dominant formation of the conventional portland-cement hydration compounds that serve to bind the particles together.

(2) Uses. Portland-cement stabilization can provide a material with compressive strengths from a few MPa (few hundred) to well over ten MPa (several thousand psi), depending on amount of stabilizer and soil properties. These higher-strength stabilized materials are often referred to as *econocrete, lean concrete*, etc. with cement contents in the range of 134 to 223 kg/m^3 (225 to 375 lb/yd^3). Such high cement content and high-quality stabilized mixes are usually proportioned and placed with the same techniques as conventional concrete. In general, cement stabilization of fine-grained soils provides a lower strength than cement stabilization of coarse-grained soils. The reactions of portland cement are faster than pozzolanic stabilizers such as lime. A major drawback for cement stabilization is the formation of shrinkage cracks which can reflect up through surfacing layers. This is usually a severe problem with cement-stabilized bases under asphaltic concrete surfaces, but it has also occurred with concrete surfaces placed directly on high-strength cement-stabilized layers. To minimize problems with reflective cracking, the Air Force limits the allowable content of portland cement in stabilized bases in flexible pavements to a 4-percent maximum. A double application of curing compound is often sprayed on cement-stabilized bases to reduce the chance of reflective cracking in overlying portland-cement concrete surfaces in rigid pavements. Portland-cement stabilization is most often used for a relatively high-strength layer that may provide a construction platform, an all-weather construction surface, or a significant structural layer within the pavement. It is also probably the most expensive of the common soil stabilizers. Materials stabilized with portland cement should be placed and compacted within 2 hours of the mix water coming into contact with the cement.

(3) Durability.

(a) Seasonal frost exposure. Cycles of freezing and thawing can damage cement-stabilized materials so TM 5-822-14/AFJMAN 32-1019 has specific testing criteria and limits based on ASTM D 560 that must be met if the cement-stabilized material is to be exposed to freezing and thawing. Adequate curing time in the field must also be available prior to the onset of freezing. Additional assistance on problems with cement-stabilized materials under seasonal frost exposure is available from the Army Cold Regions Research and Engineering Laboratory, 72 Lyme Road, Hanover, NH 03755.

(b) Carbonation. As with lime, atmospheric carbon dioxide can react with portland cement to form calcium carbonate which can adversely affect portland cement-stabilization reaction products. Proper and prompt mixing, compaction, and curing procedures that minimize the exposure of the stabilized soil to atmospheric carbon dioxide avoid the problem. Reported problems have been with highly weathered materials in Africa that were poorly compacted and cured.

(c) Sulfate attack. Cement-stabilized materials are susceptible to sulfate attack if sulfates are present in the soil or water in contact with the stabilized material or if sulfates are present in materials that are being stabilized. The sulfate attack reactions are expansive and highly disruptive. If the soils or aggregates being stabilized contain clay minerals, sulfate resistant cements (Type II and V) will not prevent sulfate attack. If cement-stabilization is contemplated where sulfates are present, the HQUSACE (CEMP-ET), appropriate Air Force MAJCOM pavements engineer, or Naval Facilities Engineering Service Center should be consulted for up-to-date guidance on this issue.

(4) Suitable Soils. The most economical materials for cement stabilization will generally be well-graded sandy gravels or gravelly sands with a spectrum of particle sizes. Fine materials, coarse materials, or poorly-graded materials will often require uneconomically high cement contents to achieve adequate stabilization. Sticky materials such as CH clays may be difficult or impossible to mix adequately with the cement stabilizer. Organic soils and some acidic sands respond poorly to cement stabilization.

l. Pozzolan and Slag Stabilization. ASTM C 618 classifies pozzolans as Type N (natural pozzolans), Type C (high-lime-content fly ash, a byproduct of burning lignite or subbituminous coal), or Type F (low-lime-content fly ash, a by product of burning bituminous or anthracite coal). These materials are not normally cementitious by themselves, but when combined with calcium hydroxide (lime), they will form cementitious, pozzolanic bonds. Granulated blast furnace slag is a by-product of iron production which can be ground to produce a slag cement. ASTM C 989 provides requirements and grade classifications for this material. Neither material has been used extensively as a stabilizer by the military, but their use is expanding in the construction industry. TM 5-822-14/AFJMAN 32-1019 provides guidance on fly ash (the most commonly available pozzolan) stabilization. Slag is not addressed in the manual, and HQUSACE (CEMP-ET), appropriate Air Force MAJCOM pavements engineer, or Naval Facilities Engineering Service Center should be consulted for current guidance on use of this material in military construction.

(1) Mechanisms. Pozzolans and ground granulated blast furnace (GGBF) slag react with hydroxides to form cementitious bonds. Lime or occasionally portland cement are mixed with these materials to provide the hydroxide activator. Some Class C fly ashes contain sufficient free lime (calcium hydroxide) to be self-cementing, but the military has no experience at present using these materials as a stabilizer without the addition of lime or portland cement. Properly cured lime-fly ash mixes often have compressive strengths of 3.45 to 6.89 MPa (500 to 1,000 psi) with appreciably higher long-term

strengths. If more rapid strength gain is needed, addition of 0.5 to 1.5 percent portland cement can be used as an activator for the fly ash and as contributor to early-age strength.

(2) Uses. Pozzolans and slags gain strength more slowly than portland cement, but are more economical, have less shrinkage and shrinkage cracking, and longer working times than portland cement. Typical fly ash-stabilized mixes will use 2-1/2 to 4 percent lime with 10 to 30 percent fly ash. Coarser soils and aggregates require less stabilizer than fine-grained soils. Some slag mixes used overseas have 8 to 20 percent GGBF slag mixed with 1 percent lime.

(3) Durability. Because of the slower strength gain of these materials, it is crucial that sufficient time be allowed between their placement and the onset of freezing weather. These chemical reactions almost cease below 4.4 °C (40 °F) so this curing period must include moderate temperatures to assure adequate curing of these materials. They can be vulnerable to freezing and thawing damage, so TM 5-822-14/AFJMAN 32-1019 requires laboratory freeze-thaw testing after 28 days curing. Additional assistance on problems with lime-pozzolan or slag-stabilized materials under seasonal frost exposure is available from the Army Cold Regions Research and Engineering Laboratory, 72 Lyme Road, Hanover, NH 03755.

(4) Suitable Soils. Granular materials are effectively stabilized with these materials. Because of their relative economy compared to portland cement, they are particularly effective with poorly graded materials where they can effectively function as a filler more efficiently than the more expensive portland cement. Many clays are naturally pozzolanic so there is little value in adding another pozzolanic material like fly ash. These are usually best handled with lime alone. However, for clays that do not develop pozzolanic reactions with lime or for silty materials that do not contain sufficient clay minerals to react with lime, pozzolanic and slag stabilizers offer an economical and effective alternative to portland cement.

m. Bituminous Stabilization. Asphalt cement (AASHTO PP6, ASTM D 3381, or ASTM D 946), emulsified asphalt (asphalt emulsified with water, ASTM D 977 and D2397), or cutback asphalt (asphalt dissolved in a solvent, D 2026, 2027, and 2028) may be mixed with a soil or aggregate to provide a water resistant, cohesive stabilized material. The mix design for bituminous stabilized materials in a military airfield subbase or base course will be done using a conventional Marshall mix design. Binder contents for subgrade stabilization are often estimated on the basis of empirical equations and then adjusted during construction in the field to achieve the desired results. TM 5-822-14/AFJMAN 32-1019 provides detailed guidance on bituminous stabilization requirements and procedures.

(1) Mechanisms. Asphalt coats the soil and aggregate particles being stabilized and binds it into a water-resistant, cohesive material. Both strength and waterproofing are provided. No chemical reactions are involved. Asphalt-cement stabilization requires no curing other than cooling. Liquid asphalts require different amounts of curing depending on the emulsifying agent or solvent used and the atmospheric conditions. The emulsion must break and the water must either evaporate or drain off for the emulsified asphalt to be effective. Similarly, the solvent in cutback asphalts must evaporate. Premature compaction of liquid-asphalt stabilized materials before adequate water or solvent evaporation may cause very slow curing and leave the stabilized material too soft. The asphalt droplets in an emulsified asphalt may have either a negative electric charge (anionic emulsion) or a positive electric charge (cationic emulsion) that can be matched to the aggregate charge (e.g., an anionic emulsion (negatively charged droplets) used with limestone aggregate (positive charge)).

(2) Uses. Asphalt stabilization provides cohesion to bind individual particles into a mass and can provide significant waterproofing. Asphalt cements are generally mixed with a higher quality

aggregate at an asphalt plant to produce a structural quality subbase or base course stabilized material. The liquid asphalts (emulsified and cutback asphalts) may be plant mixed but are often in situ mixed for less severe loading such as in the subgrade or the subbase or for lighter load applications. As a general rule, the local paving grade asphalt cement will be appropriate for the binder for asphalt-cement stabilization. For liquid asphalts, the highest possible viscosity liquid asphalt that can be handled in the field and mixed with the soil or aggregate being stabilized should be used.

(3) Durability. Water may displace asphalt particles on a soil or aggregate particle in a process known as stripping. Some aggregates have a strong affinity for water and tend to be particularly difficult to coat with asphalt. They are prone to stripping and may prove impossible to coat with liquid asphalt. Additions of lime or liquid antistrip agents or changing the charge of an emulsified asphalt may help combat these problems. Potential moisture problems and effective countermeasures should be a fundamental part of a bituminous stabilization laboratory evaluation and mix design.

(4) Suitable Soils. Bituminous stabilization is most effective with granular materials as excess fines or plastic fines may make it impossible to properly mix the materials and require high binder contents. As the plasticity index increases past 6 and the fines (percent passing the No. 200 sieve) increases above 12 percent, problems with bituminous stabilization increase. In general, the plasticity index should be below 10 and the fines should be less than 30 percent. As the plasticity and percent fines increase, liquid asphalt become better stabilizing agents than asphalt cement. The plasticity of a material to be stabilized can be reduced by adding lime.

n. Nontraditional Stabilizers. A wide variety of special, and often proprietary, stabilizers are actively marketed. These materials have seen very little use or testing by the military, and no guidance is currently available. Many, but not all, proprietary stabilizers that have been evaluated by the military have not lived up to the manufacturer's claims, and no proprietary stabilizer should be used on a military airfield without first evaluating it in the laboratory and in independent field trials. HQUSACE (CEMP-ET), appropriate Air Force MAJCOM pavements engineer, or Naval Facilities Engineering Service Center should be consulted prior to using any of these nontraditional stabilizers.

(1) Types: Nontraditional stabilizers include a wide variety of acids, salts, electrolytes (often a sulfonated oil), polymers, enzymes, natural resins, cation exchange agents, lignins, and polymers among others. Claimed benefits include strength gain, reduced water susceptibility, improved compaction, reduced dusting, reduced plasticity, and better soil texture.

(2) Evaluation. The claimed benefit of any stabilizer should be evaluated quantitatively so that the cost-effectiveness of including the material on a specific project can be determined. It is important to identify what soil property is being changed by the stabilizer and develop a quantitative scheme for evaluating this property. For example, electrolytes reduce a clay mineral's ability to hold water so they have a potential role in dealing with expansive soils. A swelling test with and without the stabilizer is appropriate to evaluate this stabilizer's effectiveness, whereas a strength test would provide no information on the electrolyte's effectiveness. Experience with some of these materials has found that often the amount of the stabilizer needed is higher than the manufacturer's suggested dosage.

3. PORTLAND-CEMENT CONCRETE. Portland-cement concrete is the surfacing for rigid pavement. It carries load through bending and is the major structural component for supporting load. Unreinforced concrete is generally the most serviceable and cost-effective surfacing for military airfields and will be used in most circumstances.

a. Reinforcing. Reinforcement may be added to concrete pavement to accomplish specific purposes, but reinforcing is the exception rather than the rule for military airfield pavements. Reinforcing concrete pavements usually adds cost and complicates construction so it is used only where its added value balances these negative factors. Conventional reinforcing steel is added to keep cracks tightly closed and to slow deterioration of the cracks. Therefore, it is useful wherever cracking cannot be avoided (e.g., odd-shaped slabs, extra-large slabs, etc.). Because reinforcing slows the deterioration of cracks, a relatively small empirical reduction in pavement design thickness is allowed by the material for reinforcing up to 0.5 percent. Continuously reinforced concrete pavements use much more steel (0.6 percent and more) which added to resist deterioration in cracks developed from environmental stresses. The steel is continuous, and the pavement has no joints. It provides a joint-free, smooth pavement, but repairs to these pavements are often difficult. Fiber reinforcing products are actively marketed. Steel fibers can significantly reduce the required pavement thickness, but there are concerns that the fibers pose a foreign object damage (FOD) on military airfields with current finishing techniques. Plastic fibers are of no particular value for military airfields. Their primary advantage for conventional concrete appears at present to be resistance to plastic shrinkage cracking, but proper construction and curing should handle this concern without adding plastic fibers at additional expense to the military. As noted later, these fibers have been found useful in concrete exposed to exhaust from vertical and short take off aircraft like the Harrier. Prestressed pavements are very efficient and produce the most structural capacity for any given cross section of concrete pavement. The design and construction of prestressed pavement is more sophisticated than conventional pavements, but prestressing construction technology has been evolving and is more cost-effective today than in past years. More details on these various reinforced pavements and their design is provided in subsequent chapters.

b. Constituents. Portland-cement concrete is composed of portland cement, aggregates, water, and various additives. Portland cement must meet the requirements of ASTM C 150, and the various types of portland cement are described in Table 9.1. Type I cement will be the most common cement, although Type II, Type I/II, and more seldom Type V may be used in areas with sulfate exposures. Type III cement might be encountered where its rapid strength gain is necessary or in cold weather concreting where its higher heat of hydration is useful. Cements may be specified to be low alkali when problems with alkali-aggregate reactions are anticipated, but such cements may not always be readily available and may be expensive. Addition of fly ash is very common in modern concretes, and the addition of ground granulated blast furnace slags is beginning to be used more often. Both may be used as economical partial replacements for portland cement in the concrete mixture and can be used to provide other desirable characteristics such as enhanced workability, lower permeability, sulfate resistance, protection against alkali-aggregate reaction, etc. Aggregate quality requirements in TM 5-822-7/AFM 88-6, Chapter 8, for military airfield pavements are appreciably tighter than those used in ASTM C 33 which is the most commonly specified concrete aggregate requirement for the concrete industry. The tighter requirements reflect the military's concern over potential FOD hazards to aircraft on airfield pavements. These tighter restriction were adopted by the military in the 1950's after severe problems with popouts developed on new airfield pavements at Selfridge AFB. Air entrainment is crucial for protecting the concrete matrix against damage from freezing and thawing and will be used in all military airfield pavements unless clearance not to do so is first obtained from the HQUSACE (CEMP-ET), appropriate Air Force MAJCOM pavements engineer, or Naval Facilities Engineering Service Center. Air entrainment causes some loss in strength, but it also enhances workability. Therefore, proper mixture proportioning can use this enhanced workability to reduce the water-cement ratio and thereby negate the strength loss from air entrainment. The proper dosage of air-entraining admixture to achieve the targeted air content is affected by factors such as the amount of carbon (measured as loss on ignition) in fly ash or the temperature. Therefore, all air entrainment for military airfield concrete will be provided by liquid admixtures added at the plant. This allows the dosage to be adjusted to reflect specific mixture characteristics and environmental fluctuations at the project site. Air entraining

admixtures that are interground with the cement and designated as Type IA, Type IIA, etc. are not suitable for this use as they do not provide the flexibility of adjusting admixture dosage to reflect changing mixture and site conditions. A number of other admixtures besides those for air-entrainment are available to accomplish specific tasks (primarily retarders, accelerators, and those for enhanced workability at a given water-cement ratio). Use of these is generally at the discretion of the engineer doing the mixture proportioning for a specific project or of the contractor who must deal with a specific site problem. The engineer responsible for the mixture proportioning is responsible for selection of admixtures and concrete materials that are compatible and cause no adverse interactions. If the contractor elects to use an admixture (e.g., a retarder because of lengthy haul times), then he or she is responsible for selecting an admixture compatible with the concrete mixture and which has no adverse effect on the fresh or hardened concrete mixture.

c. Special Air Force Requirement. During the 1980s and 1990s, newly placed concrete airfield pavement on Air Force bases had widespread problems with excessive spalling derived primarily from construction related problems, part of which sprung from the common use of concrete mixtures with poor workability. To partially address these problems, the Air Force now requires a well-graded concrete aggregate be used for all their airfield pavements with specific limitations depending on anticipated placement methods (i.e., slipform, with form-riding equipment, or by hand). Specific requirements and details are contained in the Air Force Concrete Mix Design Handbook and will be conformed to for all Air Force pavements unless a waiver is obtained from the Air Force MAJCOM pavements engineer.

d. Durability. Properly proportioned and placed, portland-cement concrete is a highly durable material. Protection against freezing and thawing is achieved by ensuring adequate strength gain before the concrete is first allowed to freeze (crucial issue in cold-weather concreting), using aggregates that are resistant to freezing effects (avoiding aggregates that are prone to produce popouts and D-cracking), and providing adequate air entrainment to protect the concrete matrix. Special precautions are needed when concrete will be exposed to sulfates or if the concrete mixture contains certain aggregates susceptible to reactions between the portland cement alkalis and some aggregate minerals (most commonly certain specific forms of silica and more rarely certain dolomitic materials). Details on these durability issues and guidelines on selecting appropriate levels of air entrainment are provided in TM 5-822-7/AFM 88-6, Chapter 8. The water-cement ratio in military airfield paving mixtures is limited to a maximum of 0.45. This requirement enhances durability by keeping the concrete permeability low as well as improves strength when compared to using higher water to cement ratios in the concrete mixture.

e. Design Strength.

(1) Test Method. Military airfield pavements are designed on the basis of the third point, flexural beam test (ASTM C 78). Thickness design is based on fatigue relationships from full-scale field tests that characterized the test pavement with the flexural test determined in this manner. Other test methods (e.g., center-point flexural beam or splitting tensile test) give numerically different values from this test and are therefore not suitable substitutes. Pavement thickness design is based on classical fatigue analysis, and the results are very sensitive to the specific value of flexural strength used in the design. Consequently, it is important that military airfield pavement design define the concrete strength consistently with the fatigue relationship used in the design procedure. Consequently, all military airfield design will be based on the ASTM C 78 flexural strength.

(2) Correlations. There are no unique relationships between different concrete strength tests (third-point flexural beam, center-point flexural beam, compressive, splitting tensile, etc.), and all such tests are indices of strength rather than an inherent material property. There are many published relationships that allow estimation of one strength test result as a function of another test (e.g., estimate

third-point flexural strength from the concrete compressive strength). However, the variation of the data upon which such relations are based is quite large and the results too inaccurate to allow the use of such relations reliably for military airfield pavement design. The different tests respond differently to changes in the concrete mixture. For example, flexural tests are much more sensitive to inclusion of crushed aggregates in the mixture than are compressive strength tests. It is possible to develop very good correlations between the different tests if the correlation is based on tests on the specific concrete mixture and the same materials are used in the laboratory as will be used in the field mixture. However, simply changing an aggregate source can change the correlation. Correlations are allowed for quality control testing of military concrete pavements during construction, but the correlations must be developed for the specific concrete mixture being used on the project, and the mixture constituents used during construction must be the same as used to develop the correlation in the laboratory.

(3) Selection of Design Strength. The designer should base the pavement thickness design on a strength that is readily achievable with local materials. Design strengths on past projects at the base or discussions with local producers should allow selection of a design strength that is readily achievable with local materials. If no such information is available, some trial laboratory mixtures should be prepared to evaluate local aggregate sources. Traditionally, pavement thickness design for military airfields is based on the 90-day strength of laboratory-cured specimens. This lengthy cure time takes maximum advantage of the long-term gradual strength gain characteristic of conventional portland-cement concrete. On many rehabilitation projects today, pavements are returned to the user after much shorter periods. Consequently, design strengths are often specified based on these shorter periods when the pavement is returned to the user. Fly ash and GGBF slag gain strength more slowly than portland cement, so the designer must be aware that strength tests at early ages for concrete mixtures containing these materials may not reflect the ultimate long-term strength well at all. Specifying very high strengths, particularly at early ages, usually requires very rich mixtures with liberal use of admixtures. This may introduce workability and construction problems, excessive shrinkage, or other undesirable characteristics that negate the economies of higher strength. In general, design ASTM C 78 flexural strengths of 414 to 448 MPa (600 to 650 psi) are readily achievable with most local materials, and the designer should use higher design strengths only with caution.

f. Special Airfield Exposure Conditions. Properly proportioned, placed, and cured portland-cement concrete requires no surface sealers, coatings, or treatments to withstand normal military aircraft operations such as startup, warmup, taxiing, takeoff, and landing.

(1) Heat Effects on Portland-Cement Concrete. Rapid heating of moist concrete can vaporize water in the concrete capillaries and cause explosive spalling. As the concrete temperature begins to rise above about 149 °C (300 °F), the progressive cement paste dehydration, thermal incompatibilities between paste and aggregate, and aggregate deterioration lead to irreversible damage and progressive loss of strength that is more pronounced as the temperature rises. Aggregates have a major impact on the thermal behavior of concrete and in decreasing order of desirability for thermal resistance they are lightweight aggregates (e.g., expanded slags, clays, and shales or natural pumice or scoria), fine-grained igneous rocks such as basalt or diabase, calcareous aggregates, and siliceous aggregates. Including slag cements in the concrete mixture also seems to enhance thermal resistance. Heat resistant conventional concrete can be achieved by proper mixture proportioning, use of appropriate aggregates, inclusion of slag cement, and high-quality concrete placement, finishing, and curing. However, if the concrete temperature will reach 204 °C (400 °F), conventional concrete probably will not be sufficient, and thermal cycling at lower temperatures can cause damage. HQUSACE (CEMP-ET), appropriate Air Force MAJCOM pavements engineer, or Naval Facilities Engineering Service Center should be consulted for guidance for concrete that will be exposed to high temperatures or that will be exposed to repeated cycles of high thermal exposure. Concrete is a moderately good insulator so there

is a significant lag between exposure to an elevated temperature and heating of the concrete to that temperature. Normal military aircraft operations do not heat concrete pavements to temperatures that cause damage.

(2) Power Check Pads and Similar Facilities. If the jet engine exhaust plume is allowed to impinge directly on the concrete surface, severe erosion can occur. This is a potential problem for facilities such as power check pads where engines have to be operated for extended periods and where the configuration of some aircraft will project the engine exhaust plume into contact the pavement surface. For this reason, these facilities are often specifically designed to have larger slopes than normal to keep the exhaust plume from directly impinging on the pavement surface. Pavement damage can arise when parking ramps, old taxiways, etc. are converted to use as power check pads, and the conventional slopes on these facilities allow the exhaust to come into direct contact with the pavement surface.

(3) Pavements Exposed to Vertical/Short Take-Off and Landing Aircraft Exhaust. The introduction of the Harrier aircraft exposed pavements to new higher levels of heat and blast than conventional aircraft. This trend is likely to continue with development of new aircraft like the joint strike fighter currently scheduled for deployment in about 2008. The Naval Facilities Engineering Services Center has conducted extensive research in support of deployment of the Harrier in the Marine Corps. They found that reinforced conventional concrete made with diabase aggregate has provided good performance in the field for up to 15 years. Recent studies have also found that improved performance could be achieved with portland-cement concrete with lightweight aggregate and nylon fibers, a proprietary blended cement with lightweight aggregate, and nylon fibers, and a proprietary magnesium phosphate cement with lightweight aggregate. The Naval Facilities Engineering Service Center, 1100 23rd Avenue, Port Hueneme, CA 93043-4370, should be contacted for current guidance and research results in this area.

(4) Pavements Exposed to Auxiliary Power Unit (APU) Exhaust. The APU on the B-1, FA-18, and certain models of aircraft currently under development are mounted so that the exhaust is directed downward and into contact with the pavement surface. With extended operation of these units, the surface of the concrete may be heated to temperatures approaching 177°C (350°F). This leads to scaling and spalling in the limited area around the exhaust impingement area. Studies by the Naval Engineering Service Center, Air Force Wright Laboratories, and the U.S. Army Engineer Research and Development Center have identified two mechanisms contributing to this damage. Repeated heating and cooling lead to thermal fatigue and surface failure. At these elevated temperatures, fluids high in esters such as fuel, lubricants, and hydraulic fluids can chemically react with portland-cement concrete and lead to scaling of the pavement. In parking areas for these aircraft, the APU exhaust impinges on the concrete where there is significant collection of these fluids that have leaked from the aircraft in normal maintenance and operation. At present there is no technical solution to this problem. Ad-hoc solutions and trials in the field have included bolting steel plates to the pavement in the area where the exhaust contacts the pavement, various coatings, refractory concretes, and specialty concretes with generally mixed or unsatisfactory results. HQUSACE (CEMP-ET), appropriate Air Force MAJCOM pavements engineer, or Naval Facilities Engineering Service Center should be contacted for guidance when designing parking areas for these aircraft.

g. Specification and Construction. It is crucial that proper material and construction specifications be developed to accompany the thickness design and geometric design and detailing. There have been numerous problems with military concrete airfield pavements in recent decades as the result of improper construction techniques, poor finishing, inadequate curing, late saw-cutting of joints, use of aggregates susceptible to alkali-aggregate reactions without proper countermeasures, inclusion of deleterious

materials, and inadequate durability when exposed to freezing and thawing or sulfates. The result has been unsatisfactory performance, increased maintenance, and dissatisfied users in some cases. The designer should be certain to consult current versions of each service's guide specification and TM 5-822-7/AFM 88-6, Chapter 8, for assistance in preparing project specifications.

4. ASPHALTIC CONCRETE. Asphaltic Concrete is the normal surfacing for flexible pavements. Unlike portland cement concrete, it normally functions as a relatively thin wearing surface and is not the major structural element of the pavement. Asphaltic concrete on airfields is exposed to much more severe loads than on highways and is quite different from highway asphaltic concrete mixes. Substitution of asphaltic concrete highway mixes for asphaltic concrete airfield mixes is not acceptable and is a major engineering blunder. The requirements of TM 822-08/AFMAN 32-1131 V8(1)/DM 21.11 will provide an asphaltic concrete that will stand up to the loads of modern military aircraft in all environmental conditions.

a. Constituents. Asphaltic concrete is composed of well-graded aggregates (approximately 95 percent by weight) and an asphalt cement binder (approximately 5 percent by weight).

(1) Binder. Asphalt cement from the distillation of petroleum is the most common binder in asphaltic concrete. Liquid asphalts from emulsifying asphalt cement with water or dissolving the asphalt cement in a solvent have many applications in pavements but are not normally used as a binder for high-quality airfield pavements. Tars from the distillation of coal are seldom used as binder in airfield pavements today. There are also natural asphalts that occasionally are used as binder material for asphaltic concrete.

(a) Characteristics. Asphalt is a complex hydrocarbon product whose composition and properties vary depending on the petroleum source and distillation process. Asphalt is probably the most viscoelastic material used by civil engineers in routine construction. Its stiffness increases as its temperature drops or as the speed of loading increases, and in reverse the stiffness drops as temperature increases or as the speed of loading is slowed. Asphalt cement functions as a cohesive binder for the aggregate and helps provide a nominally waterproof surface.

(b) Specification. The asphalt binder should be specified in accordance with the new Strategic Highway Research Program (SHRP) pavement grading (PG) system (AASHTO PP6). This new system matches specific characteristics of the asphalt cement with environmental exposure conditions. This improved matching of binder properties and project environmental conditions should extend the effective life of asphaltic concrete pavements. TM 822-08/AFMAN 32-1131 V8(1)/DM 21.11 provides guidance on selecting PG grades of asphalt cement for different project locations. SHRP PG grading is not used universally worldwide, therefore alternate specification methods based on viscosity (ASTM D 3381) and penetration (ASTM D 946) can be substituted depending on the local market practice. Polymer additives are increasingly being used with asphalt binders and have been particularly effective for enhancing cold-weather properties. This is an evolving area so TM 822-08/AFMAN 32-1131 V8(1)/DM 21.11 and HQUSACE (CEMP-ET), appropriate Air Force MAJCOM pavements engineer, or Naval Facilities Engineering Service Center should be consulted for up-to-date guidance.

(2) Aggregates. The deformation resistance of asphalt concrete exposed to military aircraft traffic is primarily a function of the aggregate, and the binder's contribution is secondary in comparison. The aggregate gradation, particle shape, and control of these parameters during production are crucial in providing an asphalt concrete that will resist the high tire pressure of modern military aircraft. Limiting the natural sand that has rounded particles to no more than 15 percent of the total aggregate by weight is an important requirement in the military requirements for asphalt concrete for military airfields. At

higher natural sand contents, there have been repeated problems with rutting under military aircraft. TM 822-08/AFMAN 32-1131 V8(1)/DM 21.11 provides detailed guidance on aggregate requirements.

b. Mix Design. Mix design of asphalt concrete requires balancing durability, load resistance, and economics. Relatively lean mixes tend to have high load resistance but suffer environmental aging more quickly than richer mixes. Rich mixes tend to be unstable but are more resistant to environmental aging.

(1) Military Requirements. Asphalt concrete for military airfields will be designed based on the 75-blow Marshall mix design method. Details are provided in TM 822-08/AFMAN 32-1131 V8(1)/DM 21.11 and the Asphalt Institute MS-2 procedures.

(2) SHRP Mix Design. The SHRP produced an asphalt concrete mix design procedure and recommended aggregate gradations that are being widely used by state Departments of Transportation. These gradations and mix design procedures were developed for highway use and have not been evaluated for airfield use. These SHRP mix design procedures and aggregate gradations are not approved for military airfields until testing and trials demonstrate their adequacy for airfield loads and conditions. Approval from HQUSACE (CEMP-ET), appropriate Air Force MAJCOM pavements engineer, or Naval Facilities Engineering Service Center is needed before these new guidelines are used on military airfields.

c. Special Asphalt Mixes. Porous friction courses are relatively thin (~ 25 to 38 mm (~1 to 1-1/2 in.)) surface layers of a special open-graded asphalt concrete with clearly visible voids. This mix provides high skid resistance and combats aircraft hydroplaning, but its open texture allows more rapid environmental aging of the asphalt binder and makes it very vulnerable to fuel spills. These mixes were widely used by the Air Force in the 1970s and 1980s, but their use has declined as improved grooving of conventional asphalt concrete mixes provides similar skid resistance without the disadvantages of the porous friction courses. Stone mastic asphalt (SMA), sometimes also called stone matrix asphalt, has a coarse aggregate gradation that provides stone-to-stone contact with the voids between aggregate particles filled with a relatively rich mastic of asphalt cement, sand, and fibers. The stone-to-stone contact of the coarse aggregate provides a stiff rut-resistant mineral skeleton, while the rich mastic provides improved environmental resistance. Two trial applications of SMA by the Air Force for airfield pavements in the United Kingdom and Italy have performed well to date. Thin applications of fuel resistant sealers to asphalt concrete pavements provide limited resistance to fuel spills. The fuel-resistant sealers economically available in the United States are usually coal tar based and are prone to environmental induced cracking that limits their effectiveness. This cracking often occurs at early ages. Polymer modification of some of these products has helped but not solved the cracking problem. Slurry seals are thin applications of emulsified asphalt and sand to oxidized asphalt concrete surfaces to try to extend the pavement life. They have problems with low skid resistance and are prone to localized failures that generate FOD. Slurry seals are not allowed on military airfield pavements. Highly polymerized proprietary systems known as *microtexturing* that use thin surface applications of a binder and aggregate to oxidized asphalt concrete surfaces have shown promise but are still in the evaluation stage. Rejuvenators are composed of lighter-end hydrocarbons that, when sprayed on an oxidized asphaltic concrete surface, soften the binder and counter some of the aging effect. These materials have given mixed results in practice and invariably lower the skid resistance of the pavement. Consequently, they are not allowed to be used on military airfields. The military has used an open-graded asphalt concrete mix with its voids filled with a proprietary modified hydraulic cement grout to provide a surface more abrasion and fuel resistant than conventional asphalt concrete. This system is referred to as resin-modified pavement, and several successful pavements have been built with this material. HQUSACE (CEMP-ET), appropriate Air Force MAJCOM pavements engineer, or Naval

Facilities Engineering Service Center should be consulted for up-to-date guidance on these and other specialty asphalt mixes.

d. Durability.

(1) Aging and Oxidation. Asphalt oxidizes and stiffens over time which leads to a loss of cohesion and flexibility. This eventually leads to cracking and raveling. Asphalt cements from different sources oxidize and age differently. Research suggests that additives to the asphalt cement may slow oxidation, but firm conclusions and guidance are not available yet.

(2) Cold Weather Cracking. As the temperature drops, asphalt cement becomes stiffer and more brittle. With repeated exposure to cold temperatures and in conjunction with other stiffening and aging mechanisms, the asphalt concrete will develop cracking. The SHRP PG grading system of rating asphalt binders that has been adopted by the military specifically tries to select binder characteristics to resist this cracking based on the exposure at the project location.

(3) Fuel Spillage. Fuels, oils, hydraulic fluids, and similar liquids are solvents for the asphalt binder. Hence asphalt concrete should not be used where it will be exposed to such materials. Resin-modified pavement may be used as a surfacing over conventional asphalt concrete to obtain fuel resistance. Coal-tar based fuel resistant sealers have only a temporary life expectancy before cracking reduces their effectiveness.

(4) Stripping. Several mechanisms contribute to moisture damage to asphalt concrete and are generally referred to as stripping. These mechanisms include displacement of the asphalt film coating the aggregate by water, emulsion of the asphalt cement, and pore pressure development. Stripping seems to require water, stripping susceptible aggregates (e.g., siliceous aggregates), and repeated loads. Lime and proprietary liquid antistrip agents can combat the problem. Also, proper aggregate selection, and drainage to reduce the asphalt concrete's exposure to water can help mitigate the dangers of stripping. Fortunately, stripping seems to be relatively uncommon in military airfield pavements. Stripping potential and the need for countermeasures should be addressed in the mix design process.

e. Construction. Production and placement of high-quality asphaltic concrete suitable for military airfields is a demanding and skillful operation. Proper mixing and delivery of the asphaltic concrete, proper placement procedures that prevent segregation, skillful construction of the longitudinal joints, and compaction with equipment of adequate size and at appropriate temperatures are all required to achieve a suitable final product.

5. RECYCLED MATERIALS. Today, portland-cement concrete and asphaltic concrete are routinely recycled as aggregate for subbase and base course material, drainage layers, fill, and as aggregate in new asphaltic and portland-cement concrete. In all recycling operations, maintaining consistency in the recycled product is a challenge. If the recycled product all comes from a single project with consistent properties and constituents, the recycled product will probably have consistent properties and can be incorporated into construction without difficulty. If recycled materials from different projects are intermingled, the recycled product properties are likely to be highly variable, and meeting stringent airfield pavement material requirements with such mixed-source materials is highly problematic. Including debris from building demolition in the recycled product to be used in the airfield pavement structure is not allowed as contamination with undesirable material such as brick or gypsum board is likely and the recycled material from such sources tends to be highly variable. Recently, major problems developed on a project that used recycled portland-cement concrete as fill and as base course in an

environment with abundant sulfates in the soils and water. The recycled concrete suffered from sulfate attack causing heaving of the overlying surfaces. This occurred even though the recycled concrete came from nearby airfield pavements that were built to be sulfate resistant and had existed in the same environment for 30 years without problem. Reliable guidance on use of recycled concrete to be exposed to sulfate exposure is not available, and HQUSACE (CEMP-ET), appropriate Air Force MAJCOM pavements engineer, or Naval Facilities Engineering Service Center should be consulted for guidance if recycled concrete is to be exposed to sulfates. As a general policy, the military encourages use of recycled materials in airfield pavements, but this should not be done at the expense of quality or performance of the final pavement. More extensive guidance and specific limitations used by each service can be found in TM 5-822-14/AFJMAN 32-1019, TI 822-08/AFMAN 32-1131 V8(1)/DM 21.11, and TM 5-822-7/AFM 88-6, Chapter 8, and each service's guide specifications.

Table 9-1
Types of Portland Cement

Type of Cement	Characteristics
I	Ordinary
II	Moderate sulfate resistant
I/II	Meets ASTM C 150 for both Type I and II cements
III	High, early strength
IV	Low heat of hydration
V	Sulfate resistant for more severe sulfate exposure conditions

CHAPTER 10

FLEXIBLE PAVEMENT DESIGN - CBR METHOD

1.REQ UIREMENTS. Flexible pavement designs must provide sufficient compaction of the subgrade and each layer during construction to prevent objectionable settlement under traffic; provide adequate thickness above the subgrade and above each layer together with adequate quality of base and subbase materials to prevent detrimental shear deformation under traffic; provide adequate subsurface drainage control or reduce to acceptable limits the effects of frost heave or permafrost degradation where frost conditions are a factor; and provide a stable, weather-resistant, wear-resistant, waterproof pavement. Attention must also be given to providing adequate friction characteristics.

2.BASI S FOR DESIGN. The thickness design procedures included herein for conventional flexible pavement construction are based on CBR design methods. Design procedures for pavements that include stabilized layers are based on modifications of the conventional procedures utilizing thickness equivalencies developed from research and field experience. Design of flexible pavements using the elastic layer method is covered in Chapter 11.

3.T HICKNESS DESIGN CURVES. Figures 10-1 through 10-32 are design curves for use in determining the required pavement thickness for Army, Navy, Marine Corps, and Air Force airfield pavements. The individual curves indicate the total thickness of pavement required above a soil layer of given strength for a given gross aircraft weight and aircraft passes.

4.THICKNESS DESIGN. The thickness design procedure consists of determining the CBR of the material to be used in a given layer and applying this CBR to design curves (Figures 10-1 through 10-32) to determine the thickness required above the layer to prevent detrimental shear deformation in that layer during traffic. The specific steps to follow are:

a.Det ermine design CBR of subgrade.

b.Det ermine total thickness above subgrade.

(1) For Army and Navy design and Air Force design for a specific aircraft, enter appropriate design curve with subgrade design CBR and follow it downward to the intersection with design gross weight curve, then horizontally to design aircraft passes curve then downward to the required total thickness above the subgrade.

(2) For Air Force standard designs, enter the appropriate design curve with the design subgrade and read the thickness required above the subgrade for a given traffic area.

c.Det ermine design CBR of subbase.

d.Det ermine thickness of material required above the subbase by entering the appropriate design curve with the design subbase CBR and using above procedures to read the required thickness.

e.Det ermine the minimum thickness of surface and base course from Tables 8-3, 8-4, or 8-5. When the minimum thickness of surface and base is less than the thickness of surface and base required above the subbase, the minimum thicknesses would be increased to the actual thickness required.

f.Subtract thickness of the surface and base from total thickness required above subgrade to obtain the required thickness of subbase. If thickness of subbase is less than 150 millimeters (6 inches), consider increasing thickness of base course.

5.ADDITIONAL CONSIDERATIONS FOR THICKNESS DESIGN.

a.CBR Values less than 3. Normally, sites which include large areas of the natural subgrade with CBR values of less than 3 are not considered adequate for airfield construction. However, CBR values of less than 3 are included on the flexible pavement design curves so that thickness requirements for occasional isolated weak areas can be determined.

b.F rost Areas. Pavement sections in frost areas must be designed and constructed with nonfrost-susceptible materials of such depth to prevent destructive frost penetration into underlying susceptible materials. Design for frost areas in accordance with Chapter 20.

c.T he thickness of the rapid-draining or open-graded material is determined from AFJMAN 32-1016 and is substituted for an equivalent thickness of base or subbase according to design requirements.

d.Expansive Subgrade. Ensure that moisture condition of expansive subgrade is controlled and that adequate overburden is provided.

e.Limit ed Subgrade Compaction. Where subgrade compaction must be limited for special conditions, pavement thickness must be increased in conformance with reduced density and CBR of the prepared subgrade.

f.Rainfall and Water Table. In regions where the annual precipitation is less than 380 millimeters (15 inches) and the water table (including perched water table) will be at least 4.6 meters (15 feet) below the finished pavement surface, the potential for subgrade saturation is reduced. Where in-place tests on similar construction in these regions indicate that the water content of the subgrade will not increase above the optimum, the total pavement thickness, as determined by CBR tests on soaked samples, may be reduced by as much as 20 percent. The reduction will be effected in the subbase course having the lowest CBR value. When only limited rainfall records are available, or the annual precipitation is close to the 380-millimeter (15-inch) criterion, careful consideration will be given to the sensitivity of the subgrade to small increases in moisture content before any reduction in thickness is made. For assistance in interpolating limited rainfall data, the USAF Environmental Technical Applications Center, USAFETAC/ECE Scott AFB, IL 62225-5000, may be contacted.

6.DESIGN EXAMPLES.

a. Example 1.

(1) Design an Air Force heavy-load pavement type B traffic area. Design CBR of the lean clay subgrade is 13; the natural in-place density of the clay is 87 percent extending to 3 meters (10 feet). The analysis that follows assumes that subgrade does not require special treatment and frost penetration is not a problem.

(2) Enter Figure 10-19 at a CBR equal to 13, move down to type B traffic area curve, then move horizontally to the required total thickness of pavement above the subgrade, 735 millimeters (29 inches).

(3) The design CBR of the subbase material has been determined to be 30. Enter Figure 10-19 at a CBR equal to 30 and find that the required thickness of base and surface is 405 millimeters (16 inches) for the design aircraft. From Table 8-5, the required minimum thickness of the surface course is 127 millimeters (5 inches) and of the 100 CBR base, 228 millimeters (9 inches). Use a 127-millimeter (5-inch) asphalt concrete (AC) surface and 280 millimeters (11 inches) of 100 CBR base to provide the 405 millimeters (16 inches) required above the 30 CBR subbase.

(4) The required thickness of subbase is 330 millimeters (13 inches), 735 minus 405 millimeters (29 less 16 inches).

(5) From Table 6-2, it is determined that for cohesive subgrade soils, 95 percent compaction is required for 864 millimeters (34 inches) below pavement surface and 90 percent compaction for a 1,320-millimeter (52-inch) depth.

(6) The design section for type B traffic area is illustrated below:

127-mm (5-in.) AC surface

280-mm (11-in.) 100 CBR Base[1]

330-mm (13-in.) 30 CBR Subbase[1]

Top of Subgrade

203-mm (8-in.) 95 percent compaction

457-mm (18-in.) 90 percent compaction

[1] Base and subbase compacted to 100 percent.

(7) Design for drainage layers is illustrated in TM 5-820-2/AFJMAN 32-1016.

b.Example 2.

(1) Design an Army Class III airfield apron (type B traffic area) for a single-wheel tricycle gear aircraft with a gross weight of 11,200 kilograms (24.6 kips) for 50,000 passes plus 10,000 passes of a CH-47 with a gross weight of 22,680 kilograms (50,000 pounds). The runway length is less than 1,220 meters (4,000 feet). Subgrade is a poorly graded sand with a design CBR of 16; in-place density of the subgrade is 90 percent to a depth of 3 meters (10 feet).

(2) From Figure 10-3, the total pavement section required is 240 millimeters (9.5 inches).

(3) From Table 8-3, the minimum required surface and base thicknesses are 50 and 152 millimeters (2 and 6 inches), respectively, for a total of 203 millimeters (8 inches).

(4) Use a 240-millimeter (9.5-inch) pavement section consisting of 50 millimeters (2 inches) of AC surface and 190 millimeters (7.5 inches) of 100 CBR base on subgrade to provide the 241 millimeters (9.5 inches) required above the subgrade.

(5) Determine the compaction requirements from Table 6-5.

(6) The design section is as follows:

50-mm (2-in.) AC surface
190-mm (7.5-in.) Base[1]
Top of Subgrade
152-mm (6.0-in.) 95 percent compaction

[1] Base is compacted to 100 percent.

Since the existing subgrade has an in-place density of 90 percent, the compaction of the 152-millimeter (6.0-inch) upper layer of the subgrade may be achieved by moistening and compacting in place.

c.Example 3.

(1) Design a secondary traffic area pavement for a Navy single-wheel aircraft with a gross weight of 31,750 kilograms (70 kips) and 2.75-MPa (400-psi) tire pressure for 300,000 passes. The subgrade consists of a silty sand (SM) with a design CBR of 6 and an in-place density of 86 percent. Subbase is a sand-shell mixture with a CBR rating of 30. Base is also a sand-shell mixture with a CBR of 80.

(2) From Figure 10-8 (2.75-MPa (400-psi) tire pressure) for a design subgrade CBR of 6 and a gross weight of 31,750 kilograms (70 kips) and 300,000 passes, the pavement section required is 635 millimeters (25 inches). The thickness of base and surface required above the 30 CBR subbase is 228 millimeters (9 inches).

(3) From Table 8-4, the minimum thickness requirements are 102 millimeters (4 inches) of bituminous surface and 203 millimeters (8 inches) of base. Use 330-millimeter (13-inch) subbase.

(4) Determine the compaction requirements from Table 6-6. This table would require the top 102 millimeters (4 inches) of the subgrade to be compacted to 90 percent of maximum density. However, there is an overriding requirement that the top 152 millimeters (6 inches) of the subgrade be compacted to 95 percent of maximum density.

(5) The design section is as follows:

102-mm (4-in.) AC surface
203-mm (8-in.) Base[1]
330-mm (13-in.) Subbase[1]
Top of Subgrade
152-mm (6-in.) 95 percent compaction
In situ density of 86 percent is satisfactory

[1] Base and subbase compacted to 100 percent maximum density.

d.Example 4.

(1) The design curves may be used to design an airfield pavement for a mix of aircraft traffic. This example will demonstrate the procedure for an Air Force airfield using the aircraft, gross weights, and pass levels shown in Table 10-1. The subgrade has a CBR of 6 and the traffic area is type B.

(2) The procedure is demonstrated as follows using Table 10-1 as an example.

(a) Column 1. List aircraft to be considered in design.

(b) Column 2. List pavement design curve figure no. for respective aircraft.

(c) Column 3. List gross weight of aircraft at which they will operate on pavement.

(d) Column 4. List number of passes anticipated at indicated gross weight.

(e) Column 5. Select the thickness required for each aircraft at the pass level and gross weight shown from the appropriate design curve (Figures 10-1 to 10-32).

(f) Column 6. Determine the pass level permissible for each aircraft for the greatest thickness in column 4. The C-141 and the F-15 both require 635 millimeters (25 inches) of total thickness. In this case, the larger aircraft would normally be selected for comparisons, although it may be necessary to check design in terms of both aircraft. The C-141 is therefore selected for comparisons. The design curves are entered with the subgrade CBR of 6, then move downward to intersection with the aircraft gross weight curve, then horizontally to intersection with the 635-millimeter (25-inch) thickness line. The pass level occurring at this intersection should be recorded in column 6.

(g) Column 7. Divide the passes in column 6 by the passes permissible at 635 millimeters (25 inches) for the C-141 (1,000) and enter in column 7. Column 7 gives the equivalent passes on a 635-millimeter (25-inch) pavement by each aircraft in terms of one pass of the C-141. That is, one pass of the C-141 is equivalent to 1.2 passes of the B-52 or is equivalent to 7.5 passes of the P-3.

(h) Column 8. Divide the number of passes in column 4 by the equivalencies in column 6 to determine the design passes in terms of the C-141 and record in column 8. The total equivalent passes of all aircraft in terms of the C-141 is 2,910. Figure 12-31 is entered with the subgrade CBR of 6, the C-141 gross weight of 145,150 kilograms (320 kips,) and the equivalent pass level of 2,910 to select the required thickness of pavement of 711 millimeters (28 inches). The thickness of the individual layers will then be determined in the conventional manner using the minimum thicknesses of pavement and base for the C-141.

7.ST ABILIZED PAVEMENT SECTIONS. Stabilized layers may be incorporated in the pavement sections to make use of locally available materials which cannot otherwise meet the criteria for base course or subbase course. The major factor in deciding whether or not to use a stabilized layer is usually economic. Additional factors include moderate reduction of the overall pavement section and increased design options. The strength and durability of the stabilized courses must be in accordance with requirements of Chapter 9. For Air Force and Army, see requirements in TM 5-822-14/ AFJMAN 32-1019. For Air Force design, stabilized subbase may not be used without a stabilized base unless the base course has adequate drainage. (Approval from Air Force major command is required when use of stabilized components is contemplated.)

Table 10-1
Example Design Using Mixed Traffic

(1) Aircraft	(2) Figure No.	(3) Gross Weight kg (kips)	(4) Aircraft Passes	(5) Preliminary Thickness in.	(6) Allowable Passes at 25 in.	(7) Column 6 Divided by 1,000	(8) Column 4 Divided by Column 7
B-52	10-32	136,080 (300)	300	21.5	1,200	1.20	250
C-141	10-28	145,150 (320)	1,000	25.0	1,000	1.0	1,000
P-3	10-9	64,410 (142)	5,000	24.0	7,500	7.5	660
F-15	10-26	31,750 (70)	200,000	25.0	200,000	200	1,000
OV-1	10-3	6,800 (15)	1,000,000	12.5	Unlimited	--	—
						Total passes on basis of C-141 aircraft = 2,910	

Conversion Factor: Millimeters = 25.4 × inches; kilograms = 453.6 × kips

a.Navy and Marine Corps Design.

(1) Thickness reduction factors. Stabilized base course and subbase course materials meeting the requirements for strength and durability in Chapter 8 may be substituted for unstabilized materials. Procedures for pavement design with stabilized layers are as follows:

(a) Design a conventional pavement section as previously described.

(b) Convert the base or subbase courses into equivalent thicknesses of stabilized materials by use of the equivalency factors shown in Chapter 9.

(c) Adjust the thicknesses of stabilized base and subbase courses so that the minimum base course thickness requirements are met.

(2) Design examples. Design a primary traffic area pavement section for a C-5A aircraft with a gross weight of 385,560 kilograms (850 kips) at 100,000 passes. Design CBR of subgrade is 5; CBR of unstabilized subbase is 20; CBR of unstabilized base is 100.

(a) Alternative design 1, Conventional Section. From Figure 10-18 the required conventional pavement section is 1,093 millimeters (43 inches) for a subgrade CBR of 5, and the required cover over the subbase is 355 millimeters (14 inches). The required minimum thickness of base and surface from Table 8-3 is 203 millimeters (8 inches) of aggregate base course and 102 millimeters (4 inches) of AC surface. The conventional section is as follows:

Conventional Flexible Pavement Section, mm (in.)	Layer Description
102 (4)	Bituminous surface
254 (10)	Aggregate base course
737 (29)	Aggregate subbase course
1,093 (43) Total thickness	

(b) Alternative design 2. A 102-millimeter (4-inch) surface over cement stabilized base with unbound aggregate subbase is required.

Conventional Thickness, mm (in.)		Stabilized Section Thickness, mm (in.)	
Surface	102 (4)	Surface	102 (4)
Base	254 (10)	CT base 254/1.5 (10/1.5) =	169 (6.7)
Subbase	737 (29)	Subbase	737 (29)
Total	1,093 (43)	Total	1,008 (39.7)
CT = Cement treated			

(c) Alternative design 3. A 102-millimeter (4-inch) surface over unbound aggregate base with lime stabilized subbase is required.

Conventional Thickness, mm (in.)		Tentative Stabilized Section Thickness, mm (in.)	
Surface	102 (4)	Surface	102 (4)
Base	254 (10)	Base	254 (10)
Subbase	737 (29)	Lime stabilized subbase 737/1.2 (29/1.2) =	614 (24)
Total	1,093 (43)	Total	990 (38)

(d) Alternative design 4. Bituminous base and lime-stabilized subbase are required.

Conventional Thickness, mm (in.)		Tentative Stabilized Section Thickness, mm (in.)	
Surface	102 (4)	Surface	102 (4)
Base	254 (10)	Bituminous base 254/1.5 (10/1.5) =	169 (6.7)
Subbase	737 (29)	Lime stabilized subbase 737/1.2 (29/1.2) =	614 (24)
Total	1,093 (43)	Total	882 (34.7)

b.Army and Air Force Design.

(1) Equivalency factors. The use of stabilized soil layers within a flexible pavement provides the opportunity to reduce the overall thickness of pavement structure required to support a given load. An equivalency factor represents the number of millimeters (inches) of conventional base or subbase that can be replaced by 25 millimeters (1 inch) of stabilized material. Equivalency factors will be determined for Army and Air Force designs from Table 10-2 and for Navy and Marine Corps designs from Table 10-3.

(2) Design. The design of a pavement having stabilized soil layers is accomplished through the application of the equivalency factors to the individual unbound soil of a pavement. A conventional flexible pavement is first designed, and then the base and subbase are converted to an equivalent thickness of stabilized soil. This conversion is made by dividing the thickness of unbound material by the equivalency factor for Army and Air Force airfields. For example, assume that a conventional pavement has been designed consisting of 102 millimeters (4 inches) of AC, 254 millimeters (10 inches) of base, and 381 millimeters (15 inches) of subbase for a total thickness above the subgrade of 737 millimeters (29 inches). It is desired to replace the base and subbase with cement-stabilized GW material having an unconfined compressive strength of 6.27 MPa (910 psi). The equivalency factor from Table 9-1 for the base-course layer is 1.15; therefore, the thickness of stabilized GW to replace 254 millimeters (10 inches) of base course is 254/1.15 (10/1.15) or 220 millimeters (8.7 inches). The equivalency factor for the subbase layer is 2.3, and the thickness of stabilized GW to replace the 381-millimeter (15-inch) subbase is 381/2.3 (15/2.3) or 165 millimeters (6.5 inches). The thickness of stabilized GW needed to replace the base and subbase would be 406 millimeters (16 inches).

c.All-Bituminous Pavement Section. Alternate procedures have been developed for design of Army and Air Force airfield pavements composed entirely of AC. These procedures are based on layered elastic theory and incorporate the concept of limiting tensile strain in the AC and vertical compressive strain in the subgrade. The procedures are applicable for trial optional designs with the approval of TSMCX, for Army airfields and the appropriate Major Command for Air Force airfields. These design procedures are contained in Chapter 11.

8.SPECIAL AREAS. Areas such as overrun areas, airfield and heliport shoulders, blast areas, and reduced load areas require special treatment as described in the following text for the various services.

a.Air Force Bases.

(1) Overrun areas. Overrun areas will be paved for the full width of the runway exclusive of shoulders, and for a length of 305 meters (1,000 feet) on each end of heavy, modified heavy, medium, light, and auxiliary runways and for 90 meters (300 feet) on each end of an assault landing zone runway. Surface the overrun areas with double-bituminous surface treatment except for the first 45 meters (150 feet) abutting the runway pavement end which will have a wearing surface of 51 millimeters (2 inches) of dense graded AC. That portion of the overrun used to certify barriers or that must support snow removal equipment may also be surfaced with dense graded AC. Design the pavement thickness in accordance with Figures 10-17 to 10-32 herein, except that the minimum base-course thickness will be 152 millimeters (6 inches). The strength of the assault overrun shall be equal to the strength of the runway. Minimum base-course CBR values are as follows:

Table 10-2
Equivalency Factors for Army and Air Force Pavements

Material	Equivalency Factors	
	Base	Subbase
Asphalt-Stabilized		
All-Bituminous Concrete	1.15	2.30
GW, GP, GM, GC	1.00	2.00
SW, SP, SM, SC	--[1]	1.50
Cement-Stabilized		
GW, GP, SW, SP	1.15[2]	2.30
GC, GM	1.00[2]	2.00
ML, MH, CL, CH	--[1]	1.70
SC, SM	--[1]	1.50
Lime-Stabilized		
ML, MH, CL, CH	--[1]	1.00
SC, SM, GC, GM	--[1]	1.10
Lime, Cement, Fly Ash Stabilized		
ML, MH, CL, CH	--[1]	1.30
SC, SM, GC, GM	--[1]	1.40
Unbound Crushed Stone	1.00	2.00
Unbound Aggregate	–[1]	1.00

[1] Not used as base course.
[2] For Air Force Bases, cement is limited to 4 percent by weight or less.

Table 10-3
Equivalency Factors for Navy and Marine Corps Pavements

Stabilized Material	Equivalency Factors
1 mm (in.) of lime-stabilized subbase	1.2 mm (in.) of unstabilized subbase course
1 mm (in.) of cement-stabilized subbase	1.2 mm (in.) of unstabilized subbase course
1 mm (in.) of cement-stabilized base	1.5 mm (in.) of unstabilized base course
1 mm (in.) of bituminous base	1.5 mm (in.) of unstabilized base course

Design Loading	Minimum Base-Course CBR for Overruns
Heavy-load pavement	80
Modified heavy-load pavement	80
Medium-load pavement	80
Light-load pavement	50
Assault landing zone pavement	50
Auxiliary pavement	50

(2) Paved shoulders. Paved shoulders will be provided adjacent to runways, taxiways, aprons, and pads where authorized by AFM 86-2. The remaining shoulder width will be constructed of existing soils, select soils, or stabilized soils with a turf cover. Design the paved shoulders in accordance with Table 3-1, Table 8-4, and Figure 10-27.

b.Army Airfields.

(1) Paved shoulders. Paved shoulders should be provided for airfields and heliport/helipad facilities as designated in EI 02C013/AFJMAN 32-1013/NAVFAC P-971. Design paved shoulders in accordance with Chapters 2 and Figure 10-27. Use a 50-millimeter (2-inch) dense graded AC wearing surface on a minimum 150-millimeter (6-inch) base consisting of 50 CBR material or better. The remaining shoulder width will be constructed of existing compacted soils, select soils, or stabilized soils with a vegetative cover or liquid palliative to provide dust and erosion control against jet blast and rotor wash.

(2) Paved overruns. Paved overruns should be provided for runways and landing lanes in accordance with EI 02C013/AFMAN 32-1013/NAVFAC P-971. Design the pave portion of overruns for 75 percent of the gross weight of the design aircraft and 1 percent of the design pass levels. The paved overrun should also be checked for adequacy of supporting crash rescue vehicles. Use a 50-millimeter (2-inch) dense graded AC wearing surface on a minimum 150-millimeter (6-inch) base consisting of 50 CBR material or better. The remaining overrun area will be constructed of double-bituminous surface treatment on a 100-millimeter (4-inch) base course of 40 CBR material or better.

c.Navy and Marine Corps Airfields.

(1) Overrun areas. Pave the overrun areas for a width of 61 meters (200 feet) or the width of the runway if less than 61 meters (200 feet), centered on the runway centerline and for a length of 305 meters (1,000 feet), where feasible. Surface the overrun areas with an AC surface course. Design the pavement thickness for 75 percent of the gross weight of the design aircraft at 200 passes, except that a minimum 152-millimeter (6-inch) base course of 80 CBR or better will be provided.

(2) Blast protection areas. Design the pavement thickness of the blast protection areas for 200 passes at 75 percent of the gross weight of the design aircraft. Normally, these areas are constructed of portland cement concrete for Navy and Marine Corps airfields; where operational experience has shown asphalt surfacing to be satisfactory, use a minimum 76-millimeter (3-inch)

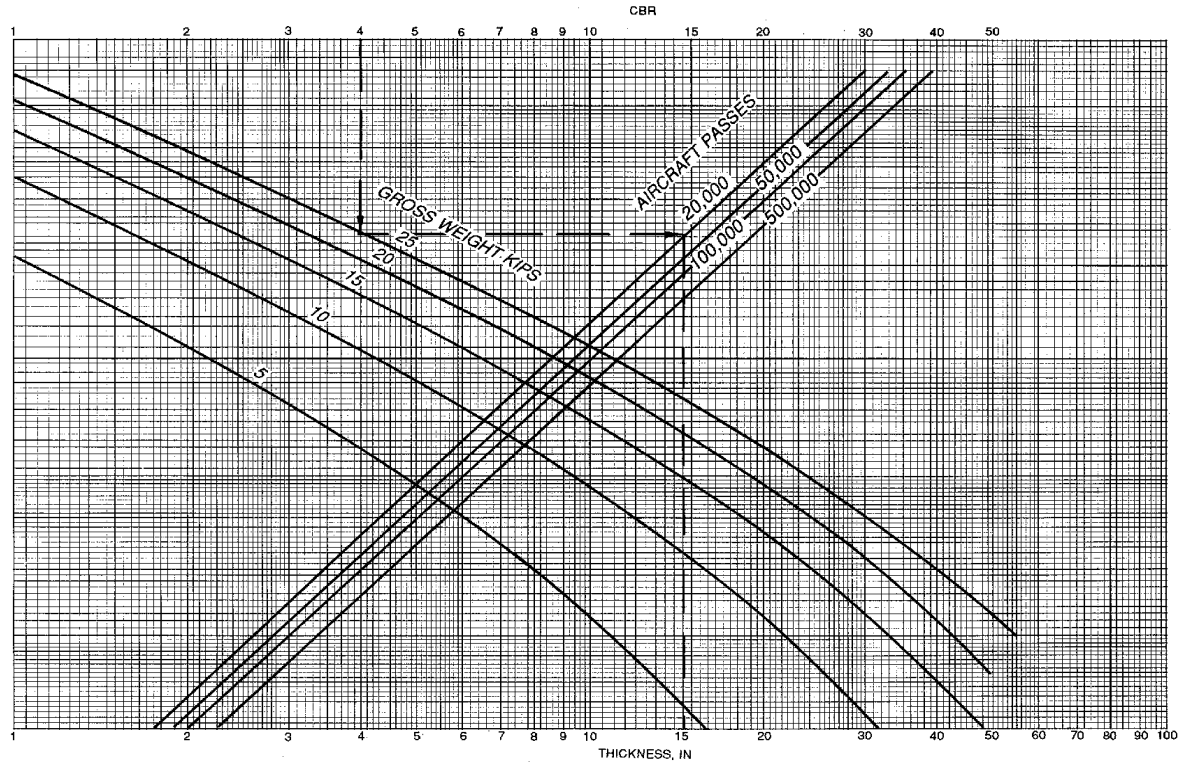
AC surface over 152 millimeters (6 inches) of 80 CBR base. Blast protection pavement design should be checked for adequacy for crash rescue vehicles.

(3) Shoulders.

(a) Fixed-wing aircraft. Pave the first 3 meters (10 feet) of runway shoulders. Design the pavement thickness for 75 percent of the gross weight of the design aircraft at 200 passes. Surface with 50 millimeters (2 inches) of AC on a minimum 152-millimeter (6-inch) base of 80 CBR. Provide the outer 43 meters (140 feet) of runway shoulders and all taxiway shoulders with dust and erosion control using vegetative cover, liquid palliative, such as asphalt, or a combination of methods.

(b) Rotary-wing aircraft. Pave the first 7.5 meters (25 feet) of shoulder adjacent to helicopter pads, runways, and taxiways with 50 millimeters (2 inches) of AC on a minimum 152-millimeter (6-inch) base course of 60 CBR. Provide the outer 15 meters (50 feet) of shoulder with a liquid palliative or vegetative cover, or a combination of methods.

9. JUNCTURE BETWEEN RIGID AND FLEXIBLE PAVEMENTS. (See paragraph 12.j of Chapter 12.)

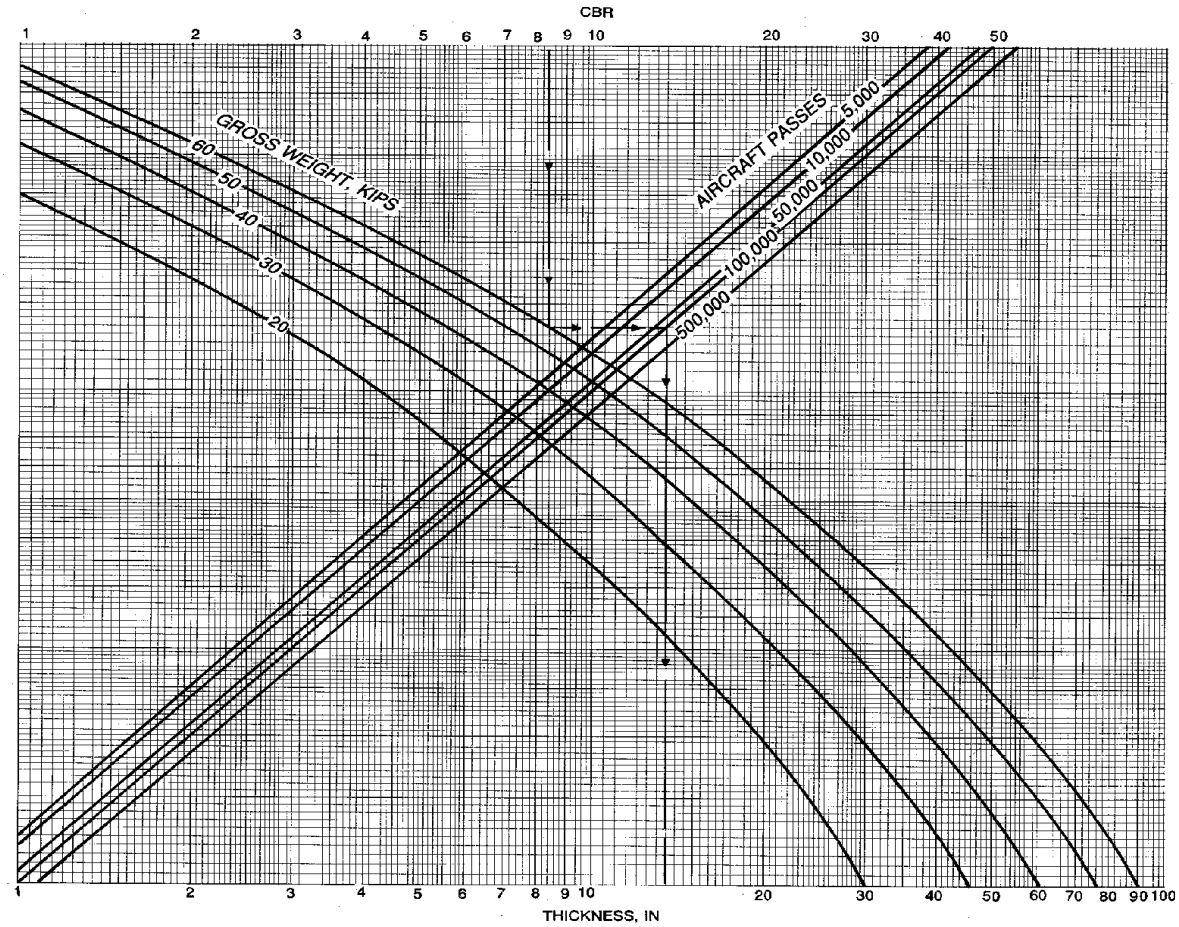


FLEXIBLE PAVEMENT DESIGN CURVE FOR ARMY CLASS I HELIPORTS AND HELIPADS.

CONVERSION FACTORS

MILLIMETERS = 25.4 x INCHES
KILOGRAMS = 0.4535 x POUNDS

Figure 10-1. Flexible pavement design curves for Army Class I heliports and helipads



FLEXIBLE PAVEMENT DESIGN CURVE FOR ARMY CLASS II AND V HELIPORTS AND HELIPADS.

CONVERSION FACTORS

MILLIMETERS = 25.4 x INCHES
KILOGRAMS = 0.4535 x POUNDS

Figure 10-2. Flexible pavement design curves for Army Class II and V heliports and helipads

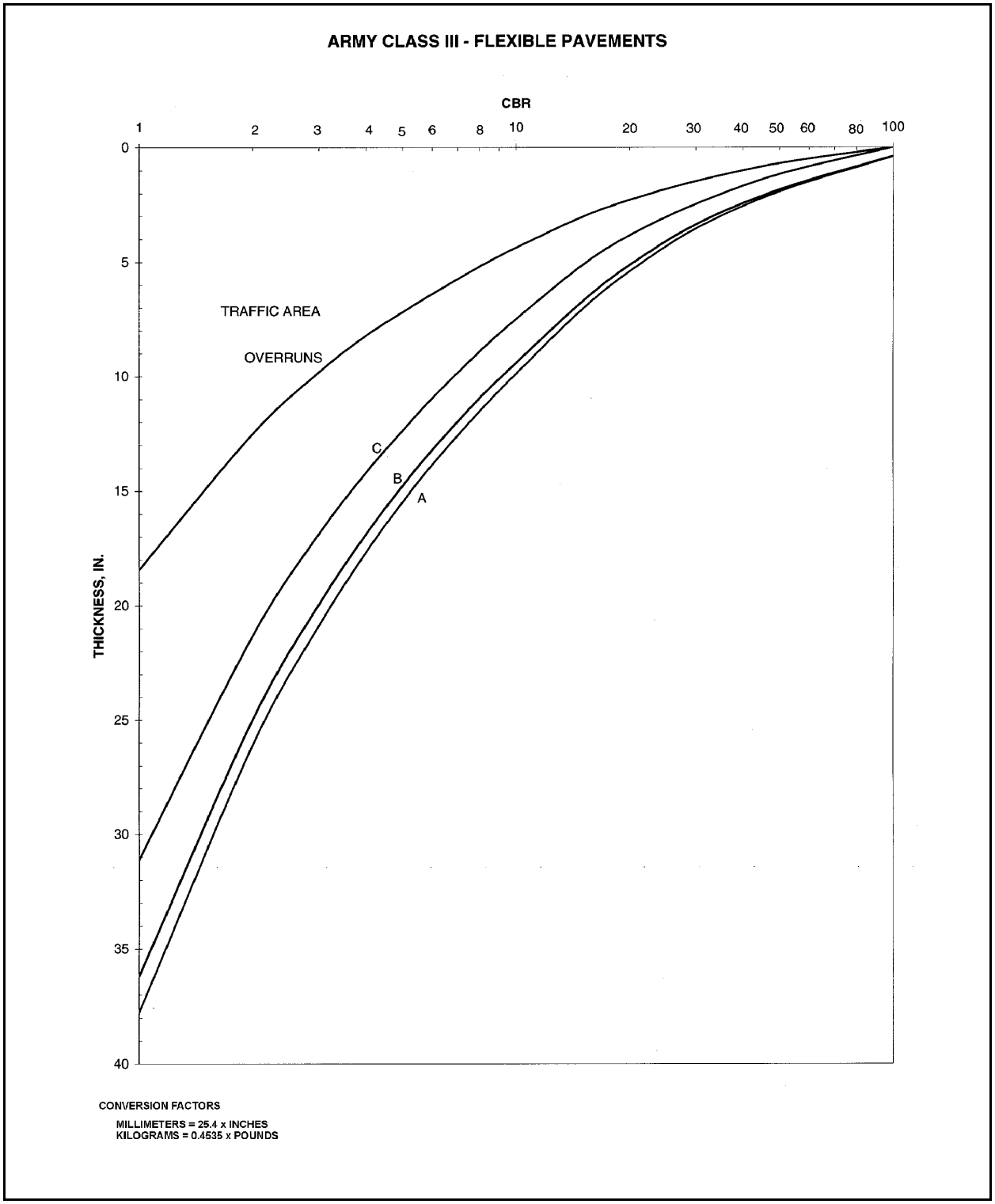


Figure 10-3. Flexible pavement design curves for Army Class III airfields as defined in paragraph 4.c of Chapter 2

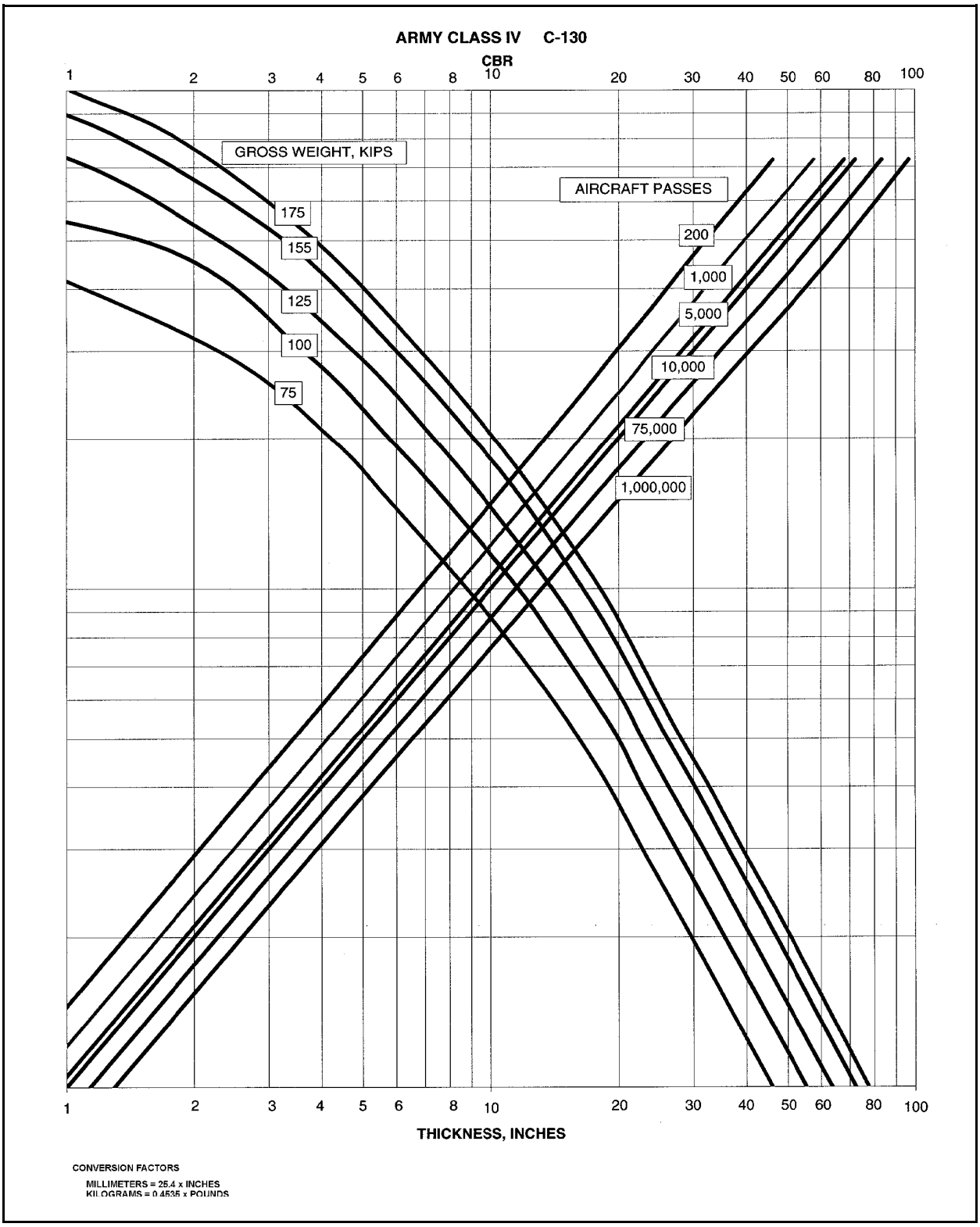


Figure 10-4.F lexible pavement design curves for Army Class IV airfields (C-130 aircraft) with runway ≤ 1,525 meters (≤ 5,000 feet), type A traffic areas

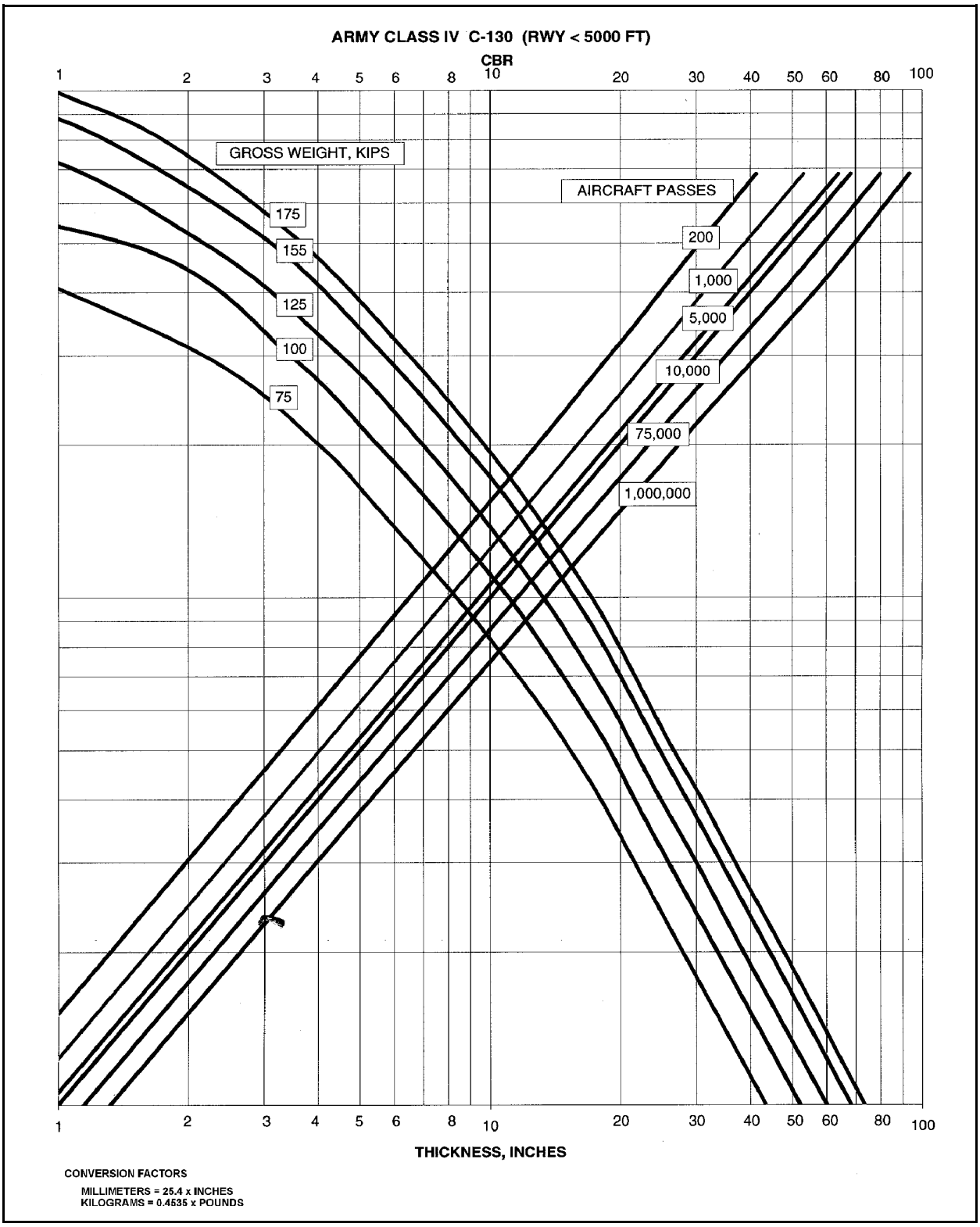


Figure 10-5.F lexible pavement design curves for Army Class IV airfields (C-130 aircraft) with runway ≤ 1,525 meters (≤ 5,000 feet), types B and C traffic areas

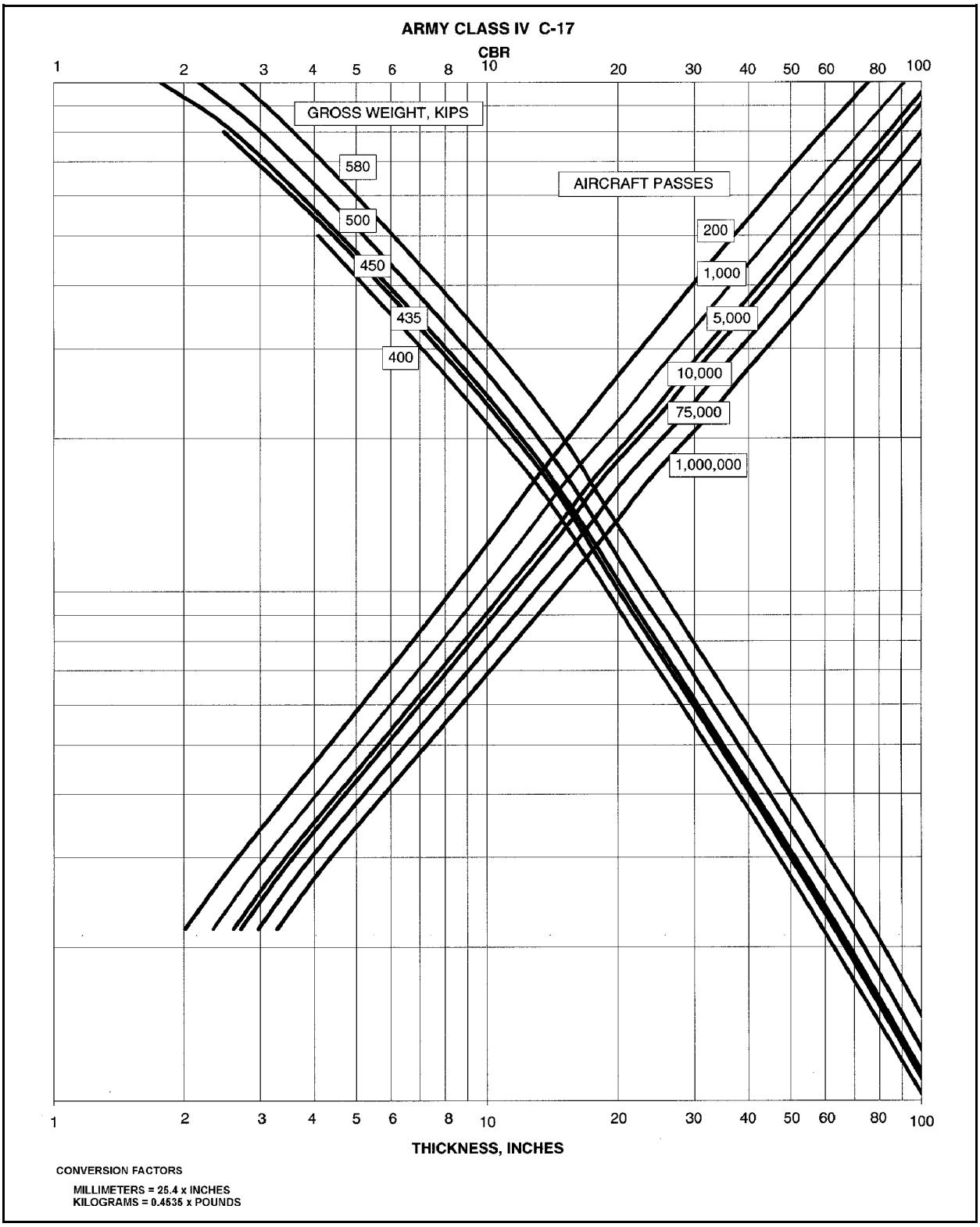


Figure 10-6.F lexible pavement design curves for Army Class IV airfields (C-17 aircraft) with runway > 1,525 meters (> 5,000 feet), type A traffic areas

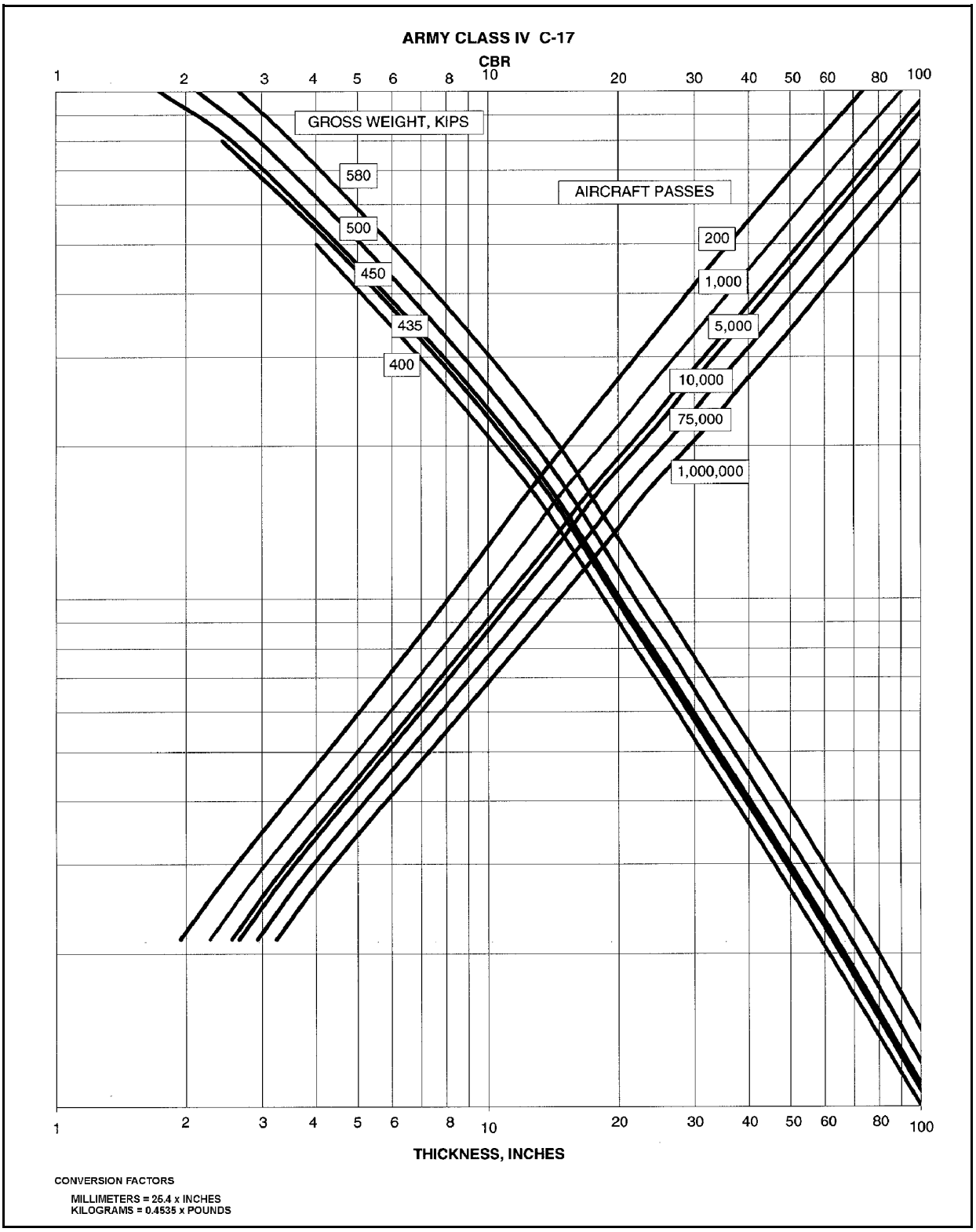


Figure 10-7.F lexible pavement design curves for Army Class IV airfields (C-17 aircraft) with runway > 1,525 meters (> 5,000 feet), types B and C traffic areas

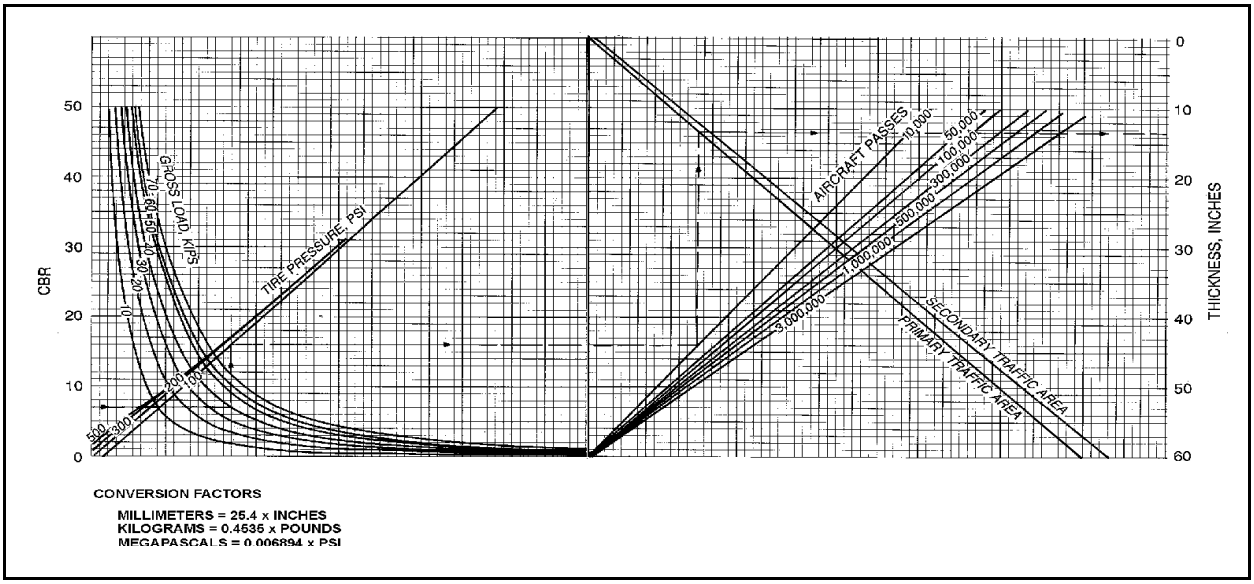


Figure 10-8. Flexible pavement design curves for Navy and Marine Corps single-wheel aircraft, primary and secondary traffic areas

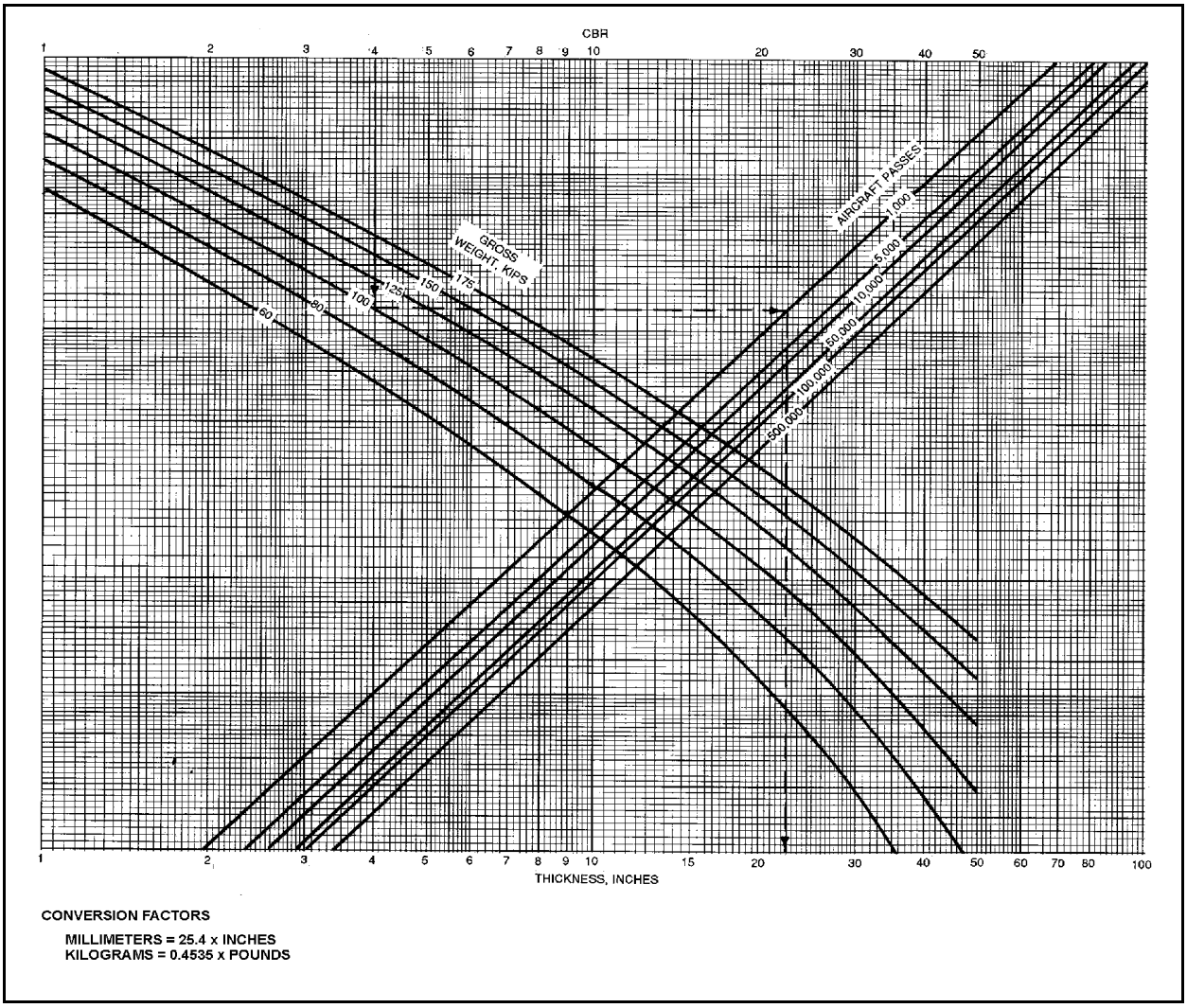


Figure 10-9. Flexible pavement design curve for Navy and Marine Corps dual-wheel aircraft, primary traffic areas

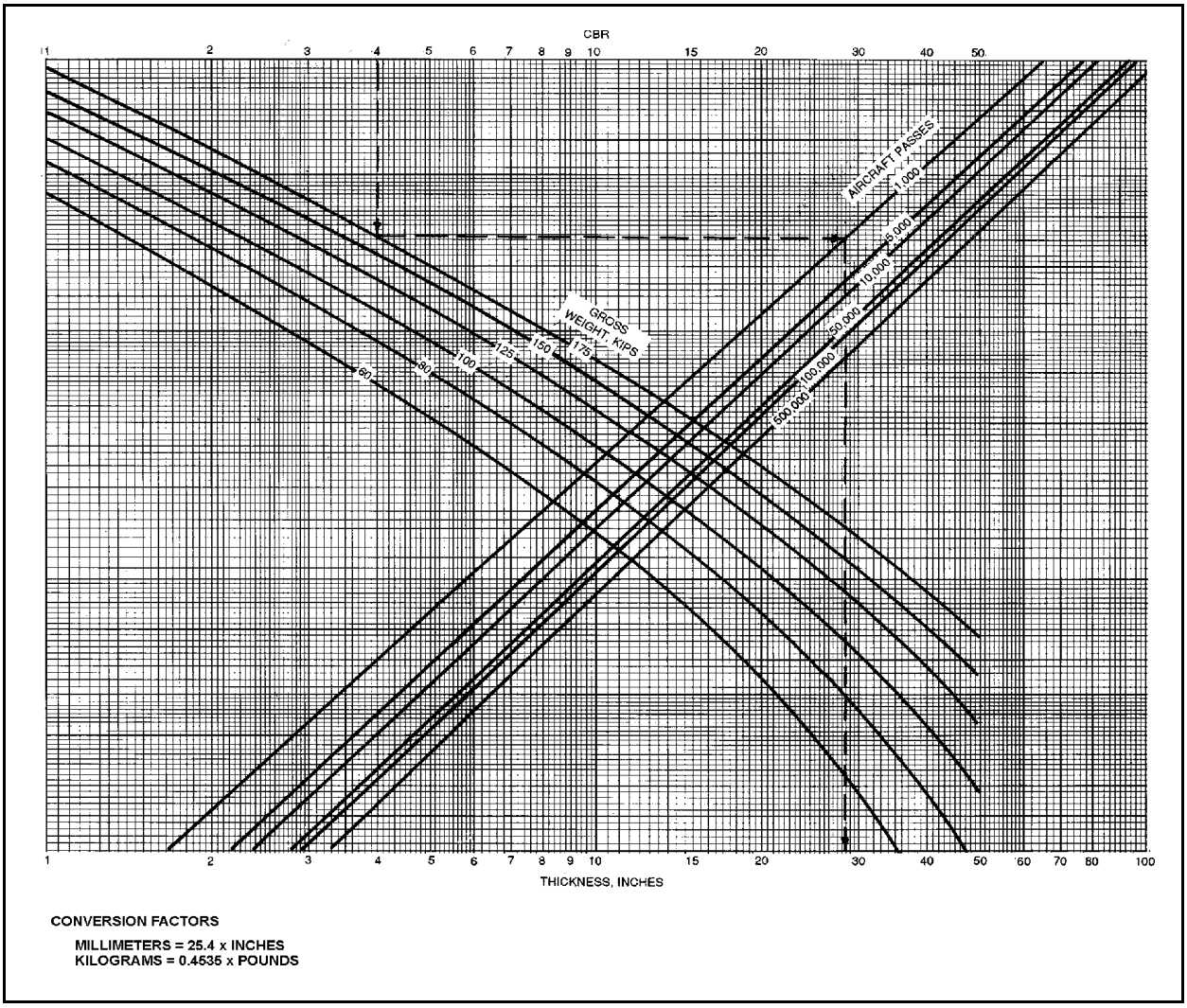


Figure 10-10. Flexible pavement design curve for Navy and Marine Corps dual-wheel aircraft, secondary traffic areas

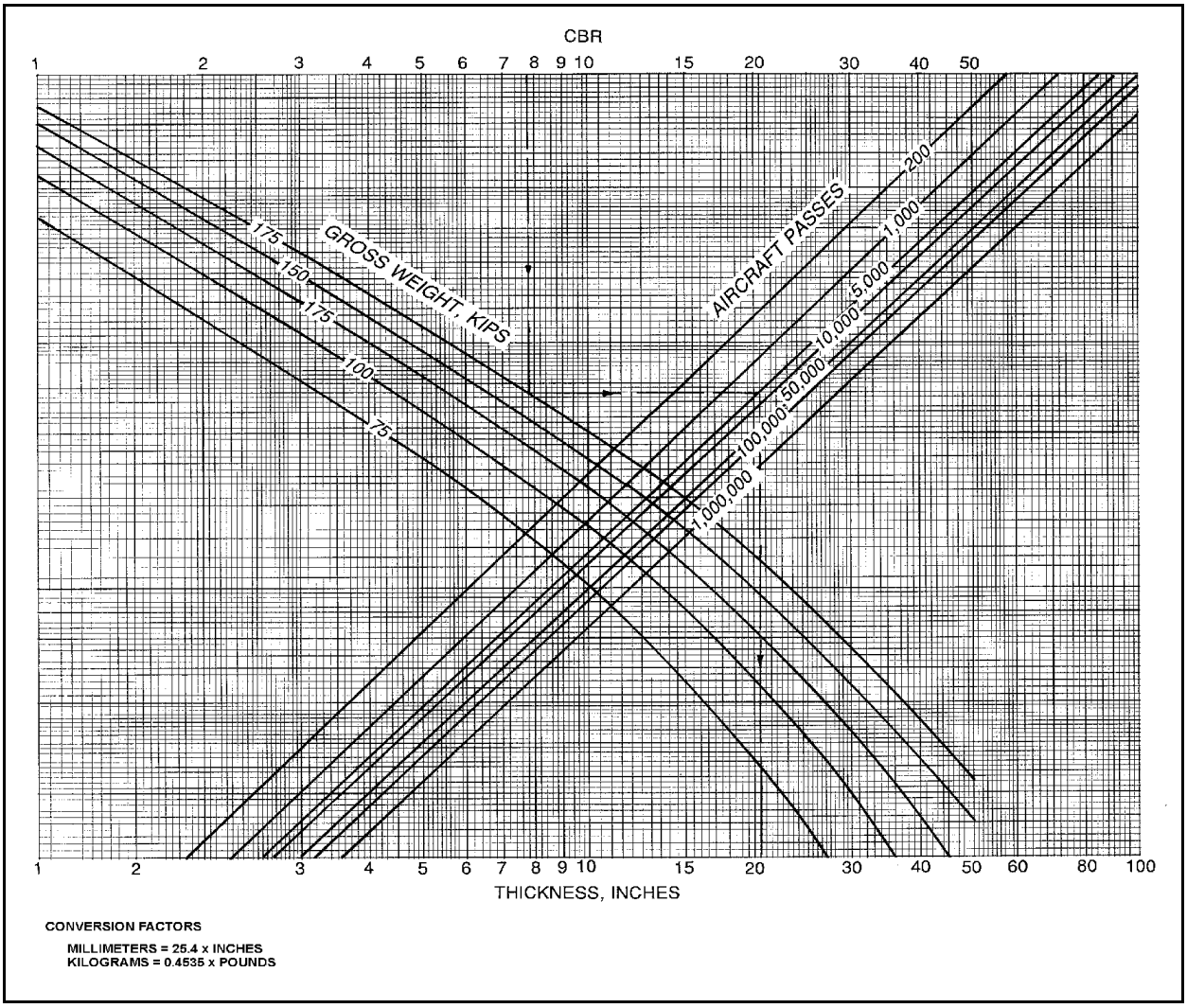


Figure 10-11. Flexible pavement design curve for Navy and Marine Corps C-130, primary traffic areas

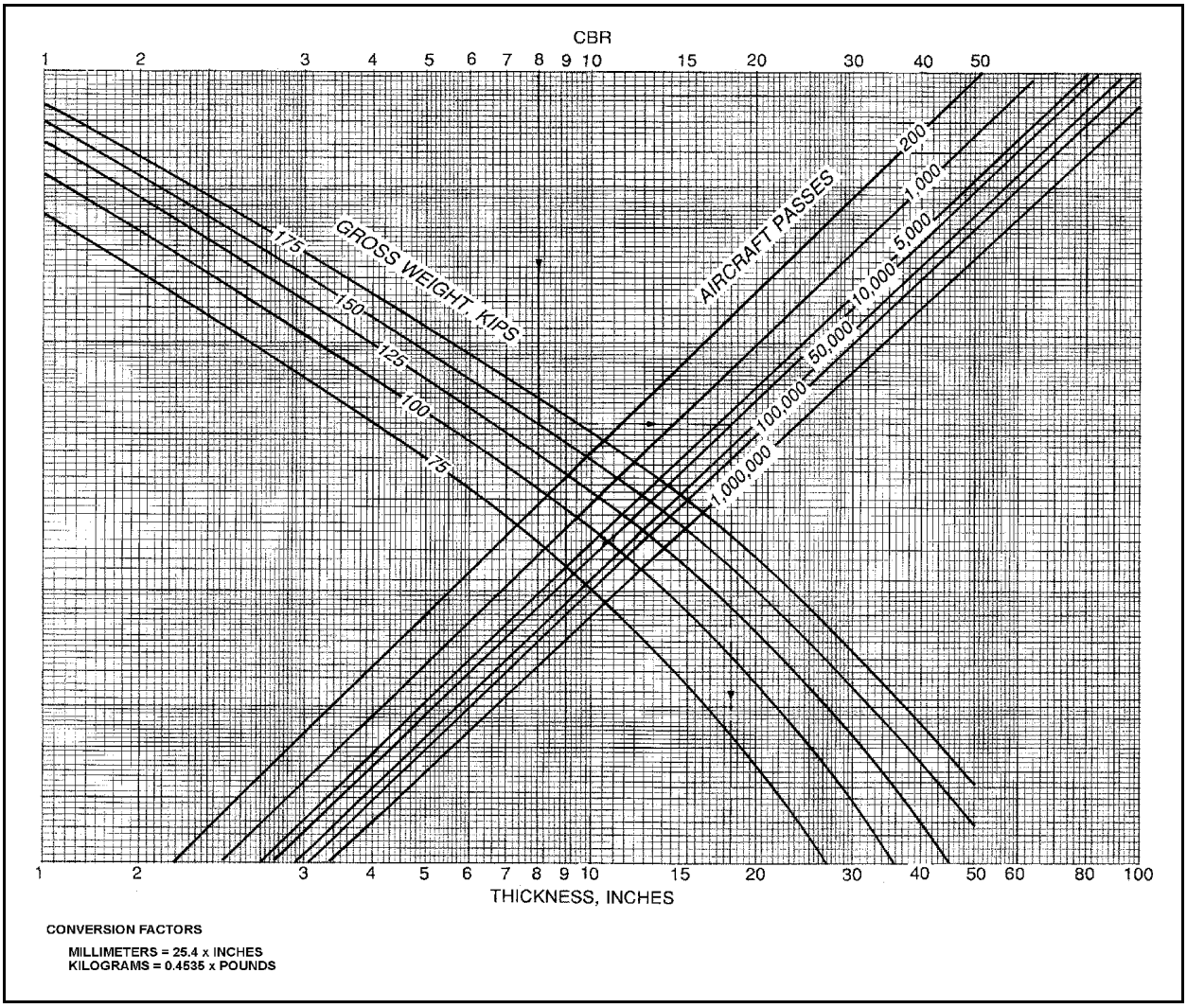


Figure 10-12. Flexible pavement design curve for Navy and Marine Corps C-130, secondary traffic areas

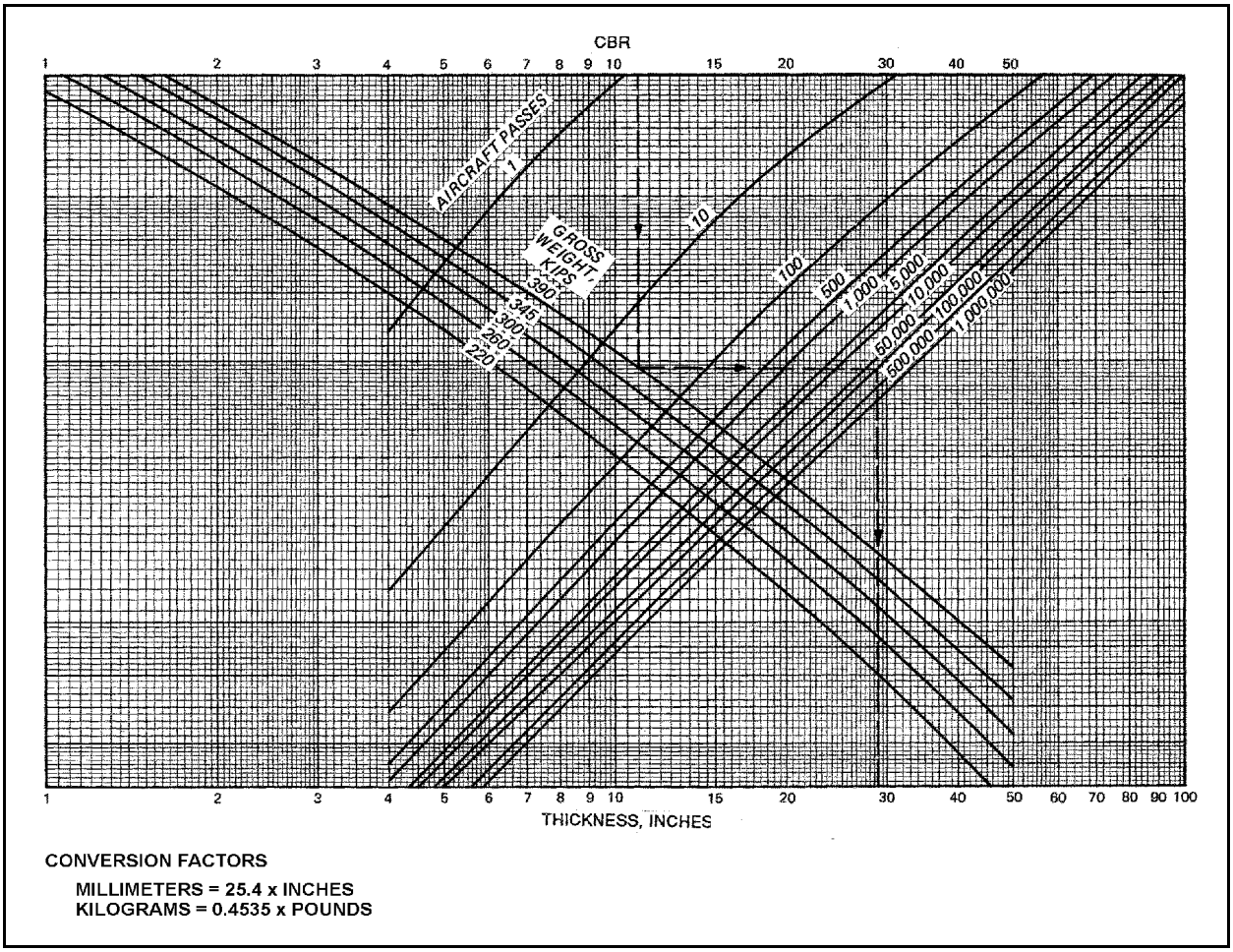


Figure 10-13. Flexible pavement design curve for Navy and Marine Corps C-141, primary traffic areas

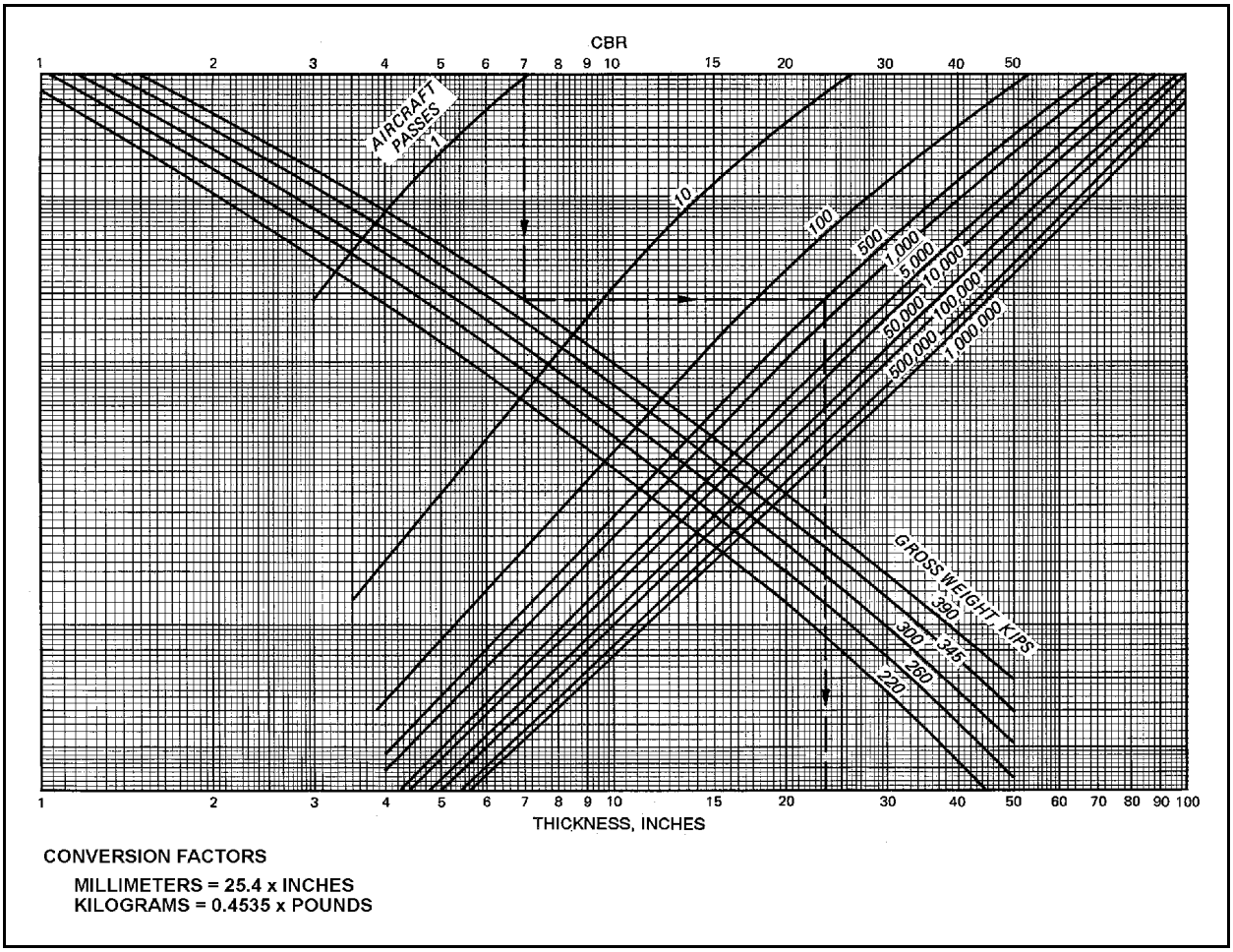
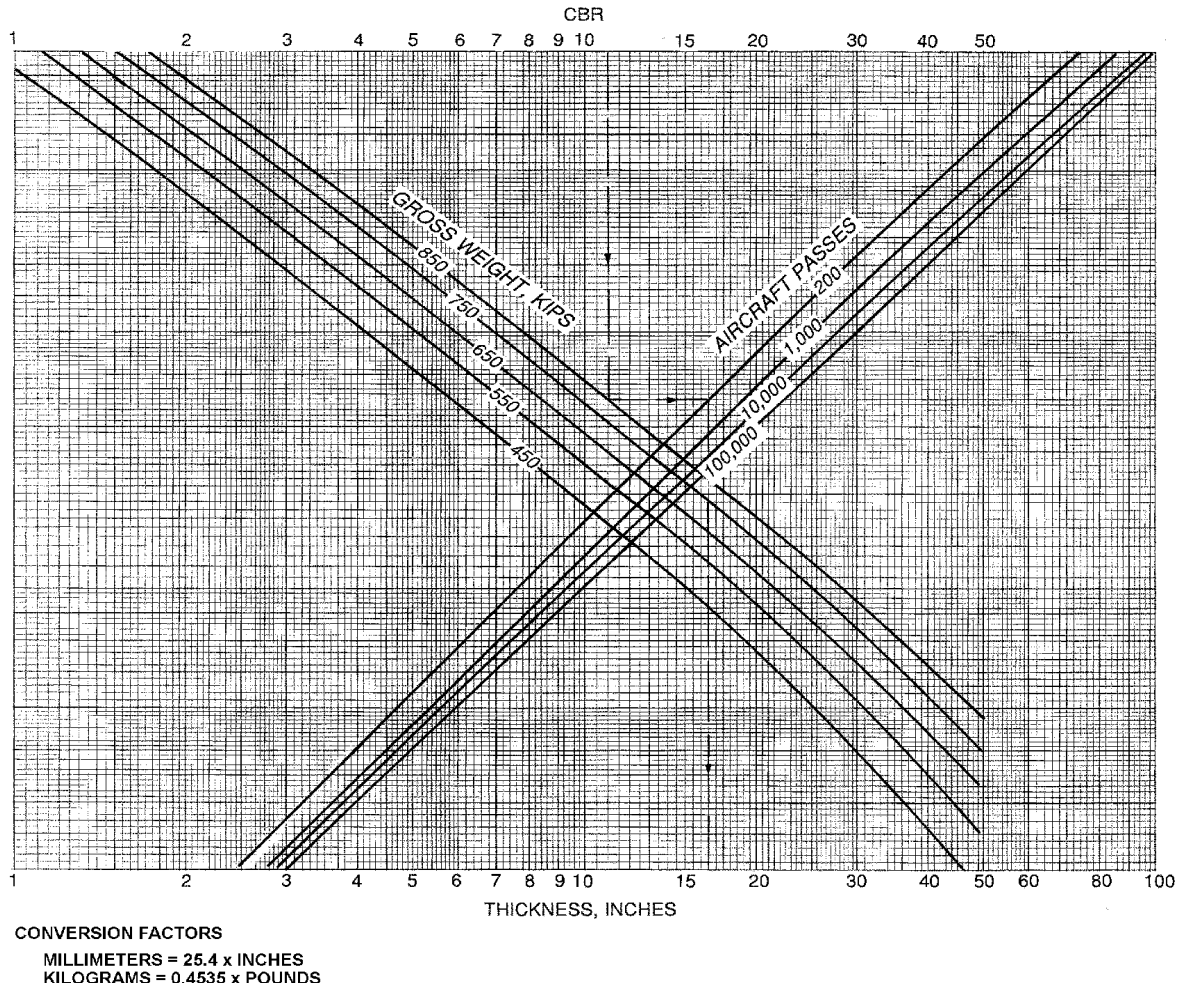


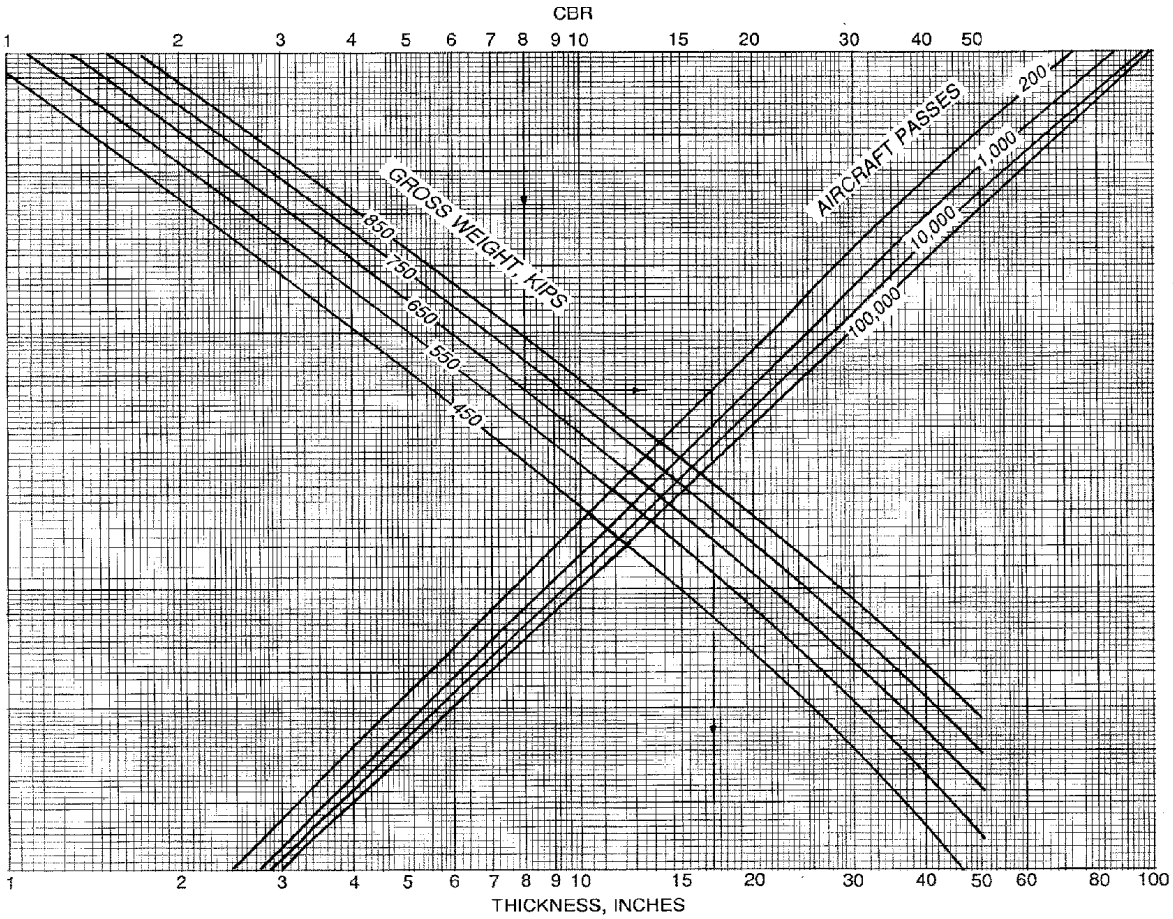
Figure 10-14. Flexible pavement design curve for Navy and Marine Corps C-141, secondary traffic areas



CONVERSION FACTORS

MILLIMETERS = 25.4 x INCHES
KILOGRAMS = 0.4535 x POUNDS

Figure 10-15. Flexible pavement design curve for Navy and Marine Corps C-5A, primary traffic areas



CONVERSION FACTORS

MILLIMETERS = 25.4 x INCHES
KILOGRAMS = 0.4535 x POUNDS

Figure 10-16. Flexible pavement design curve for Navy and Marine Corps C-5A, secondary traffic areas

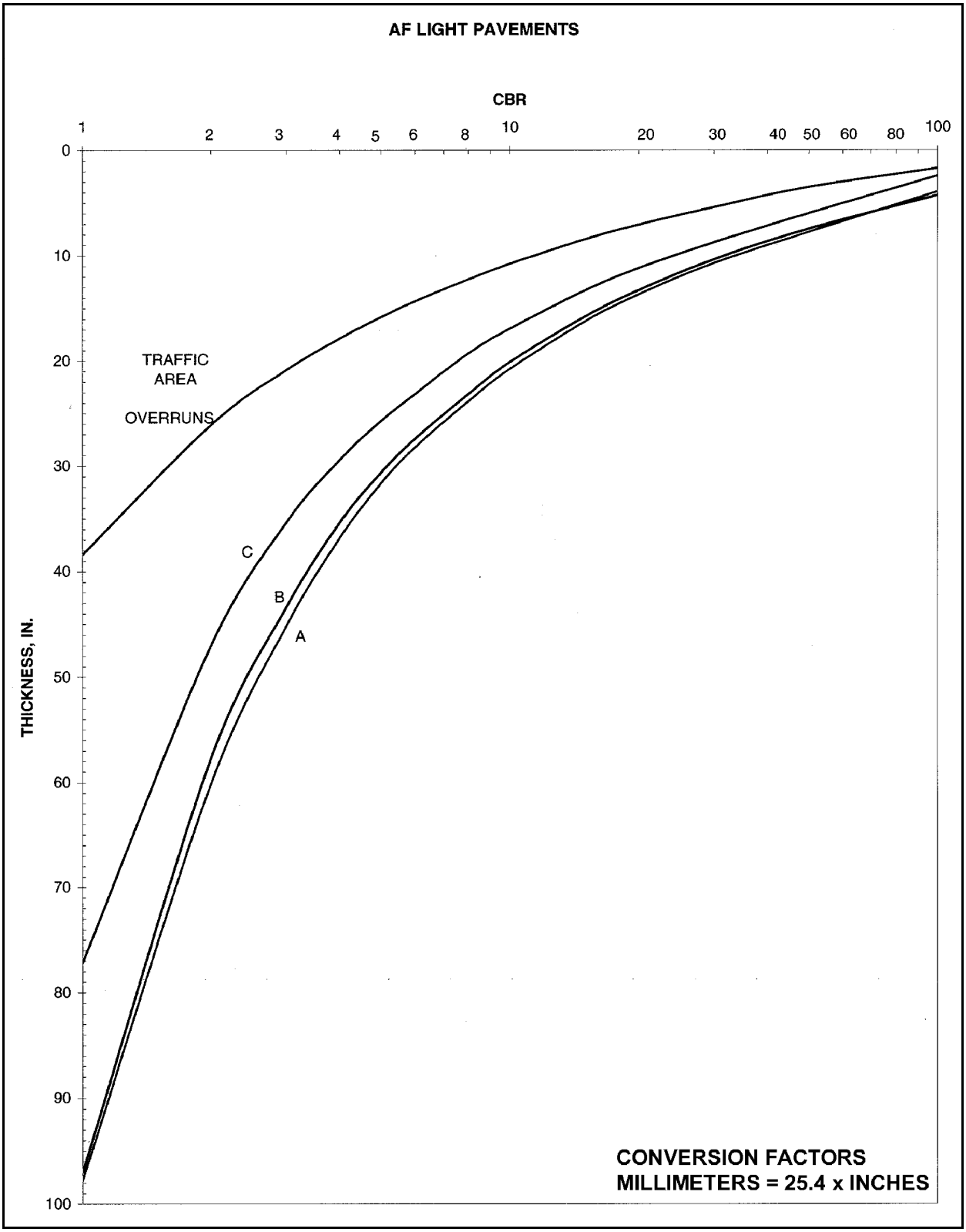


Figure 10-17.F lexible pavement design curve for Air Force light-load airfield

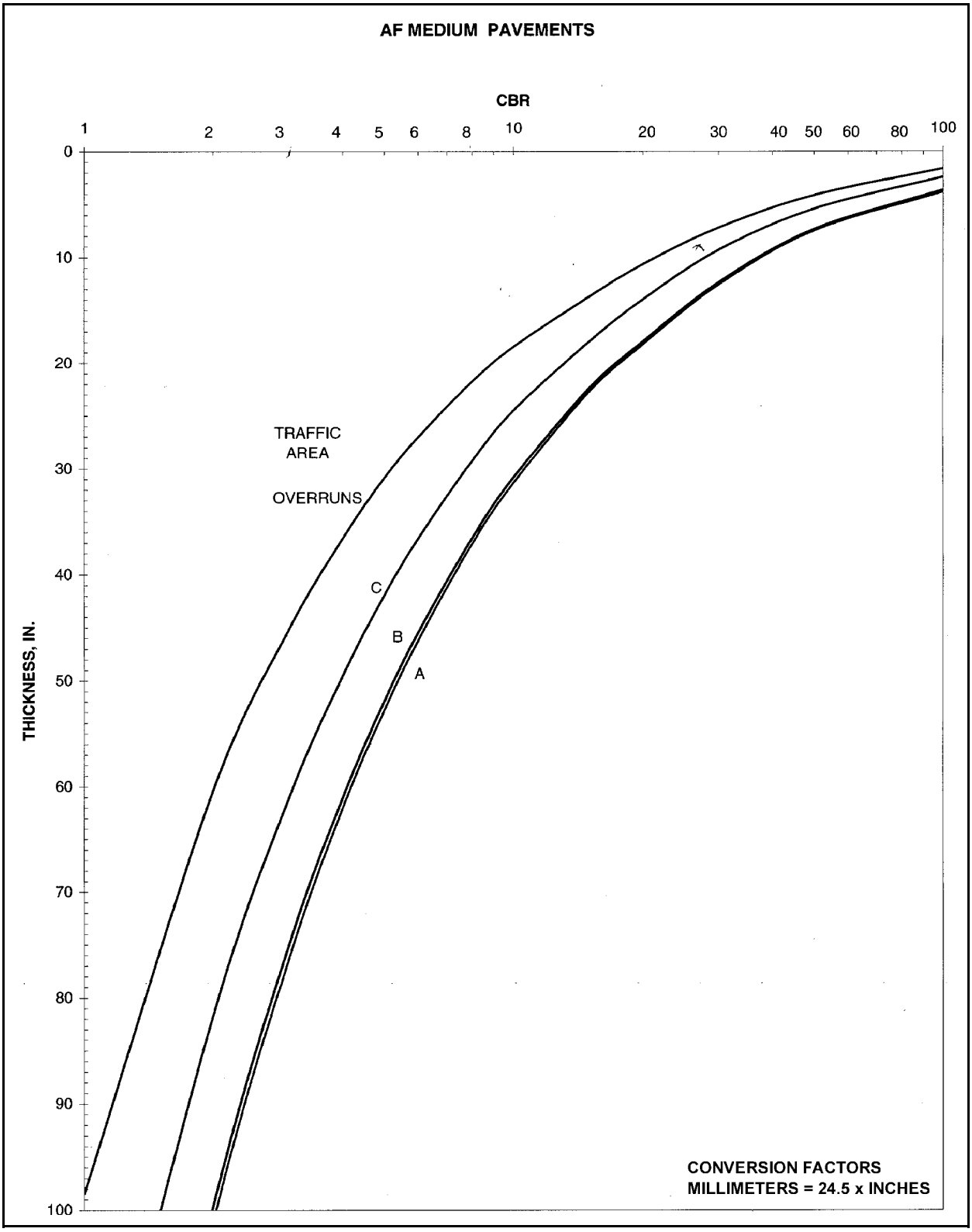


Figure 10-18.F lexible pavement design curve for Air Force medium-load airfield

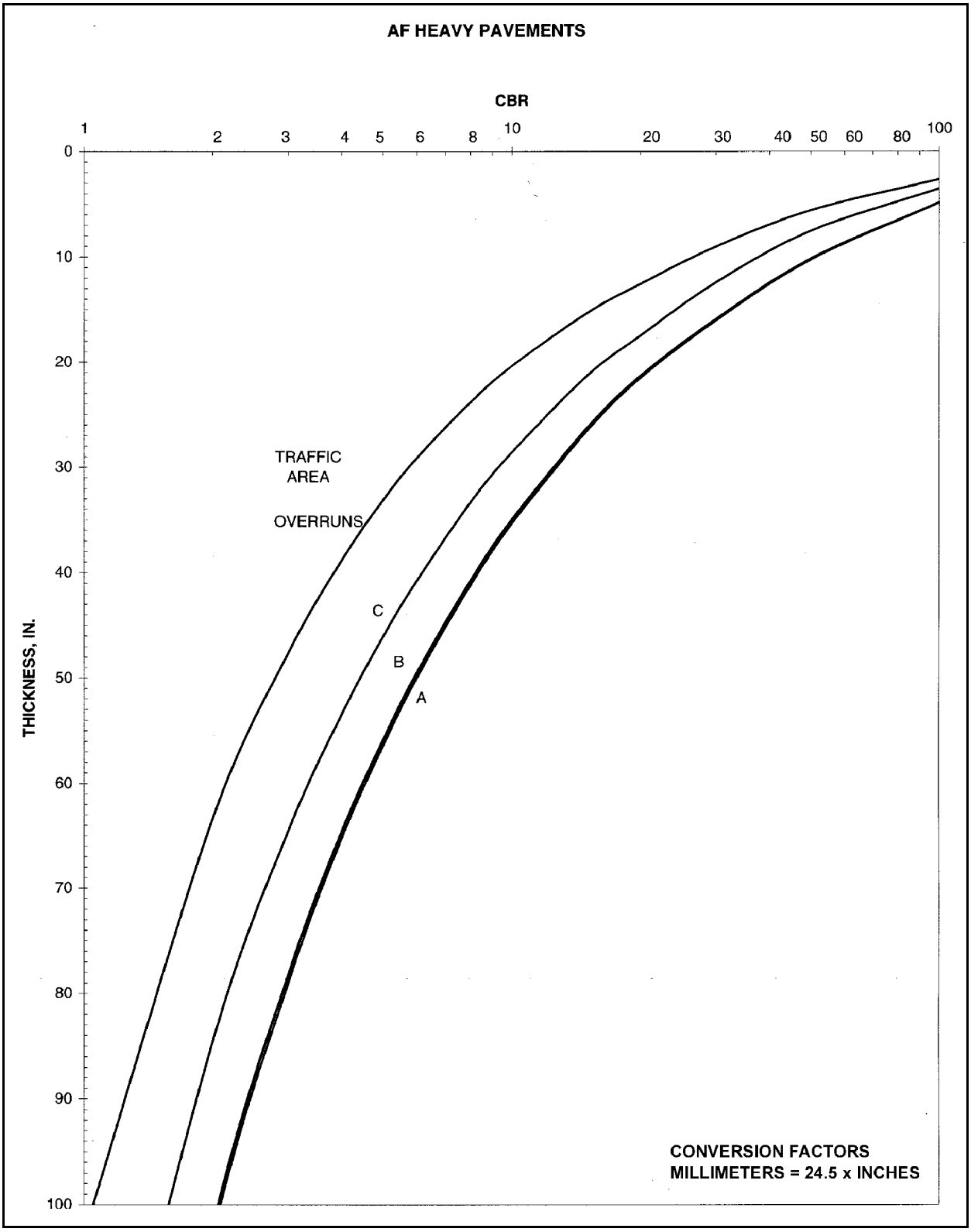


Figure 10-19.F lexible pavement design curve for Air Force heavy-load pavement

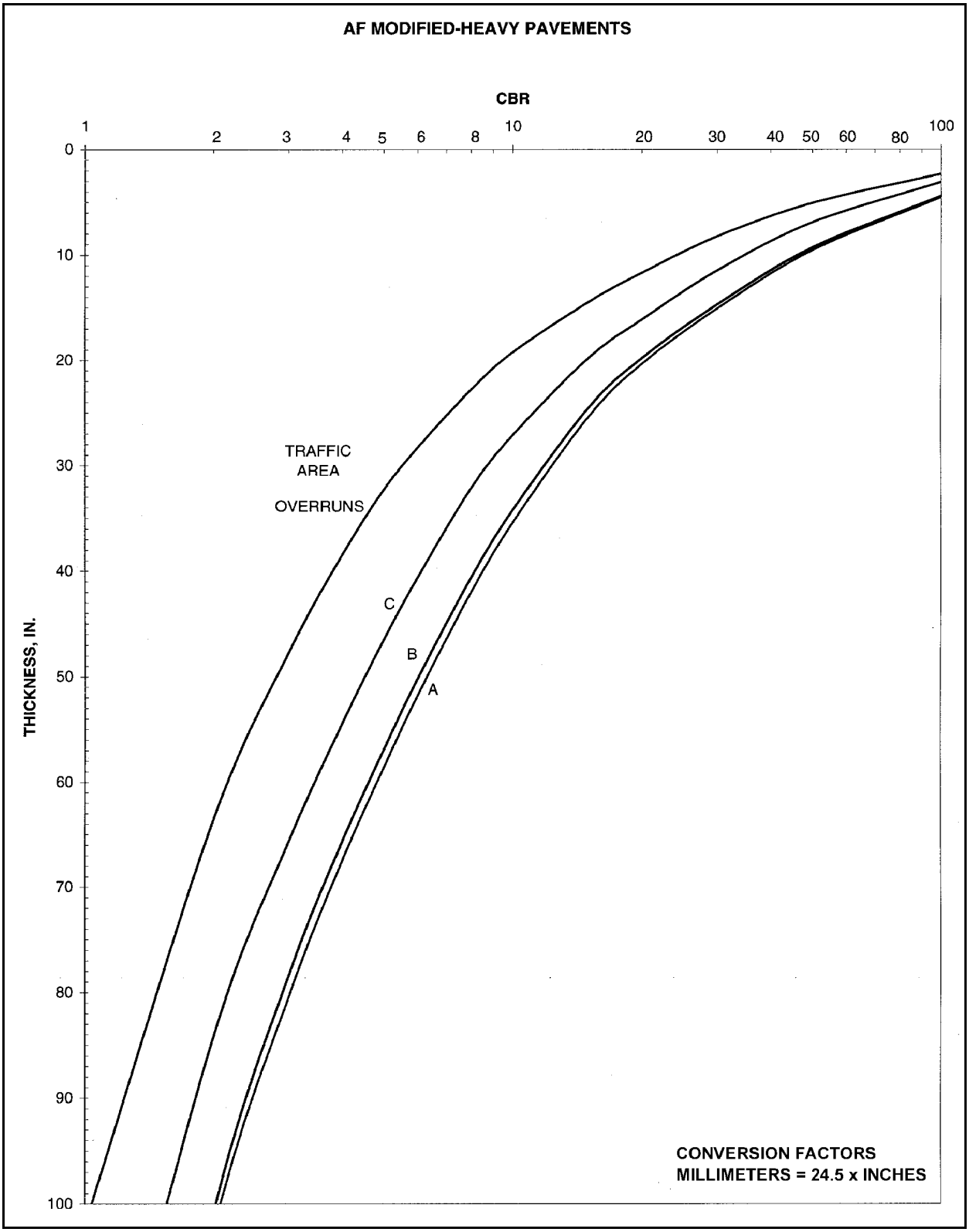


Figure 10-20.F lexible pavement design curve for Air Force modified heavy-load pavement

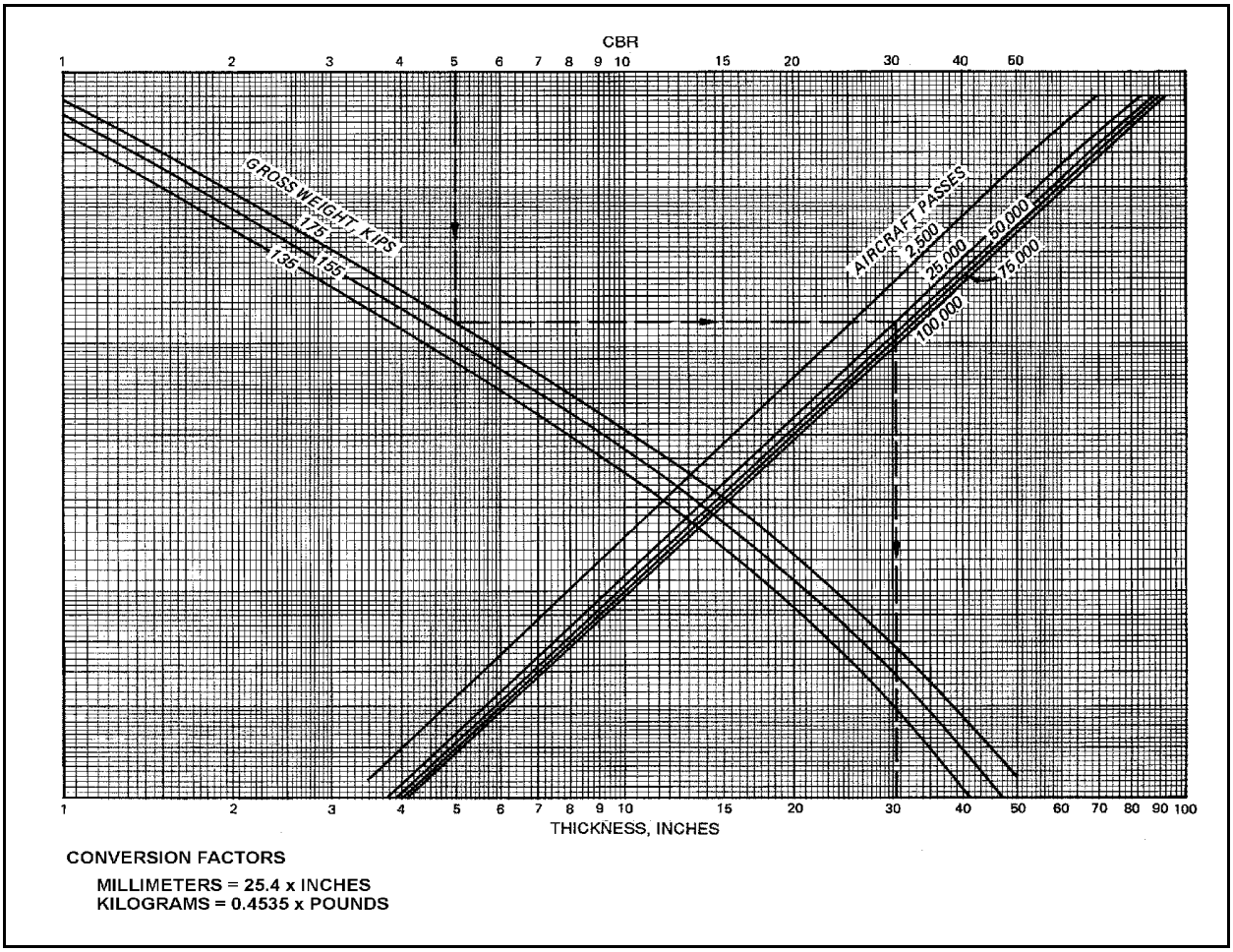


Figure 10-21a. Flexible pavement design curve for Air Force C-130 assault landing zone airfield

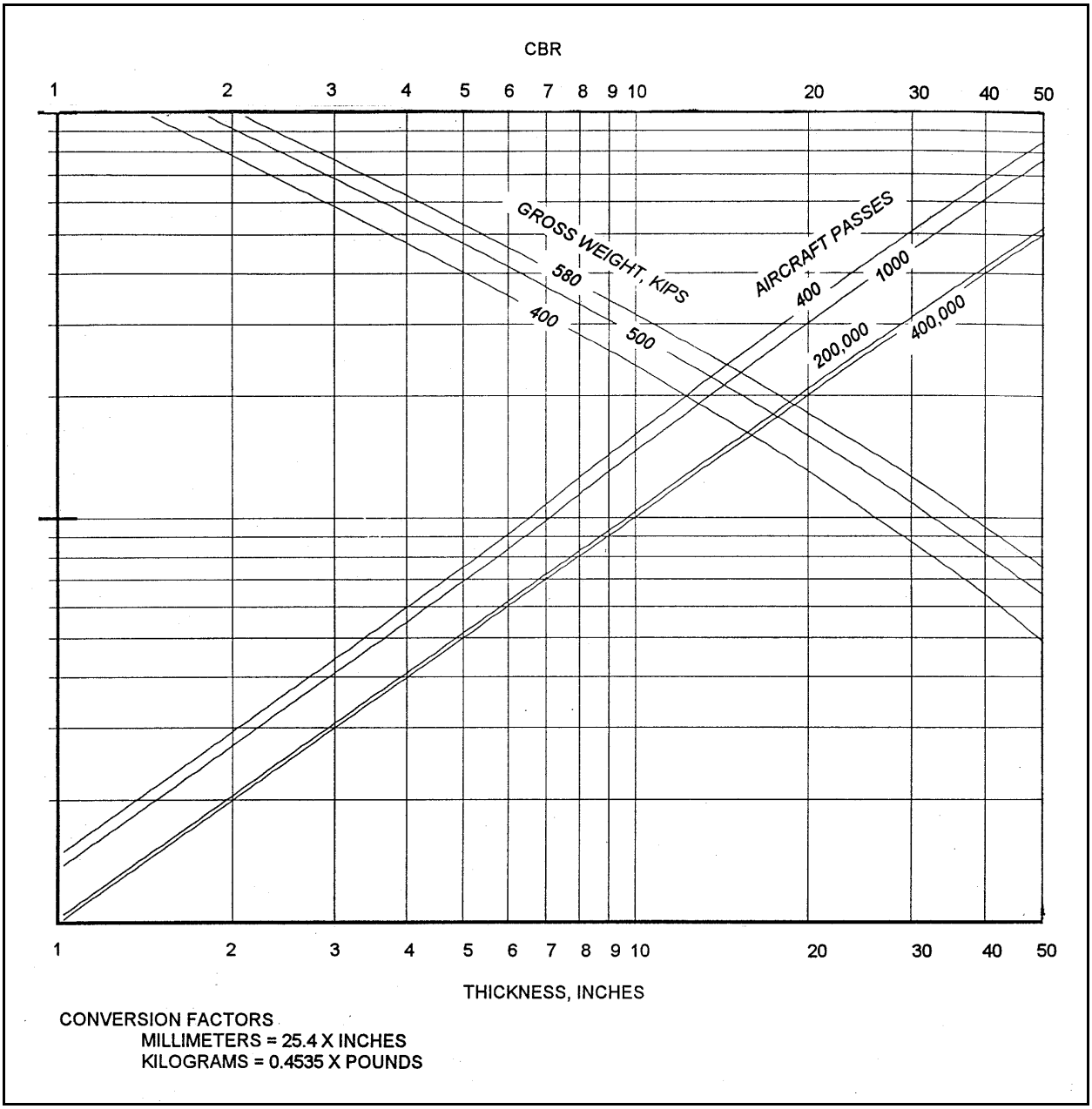
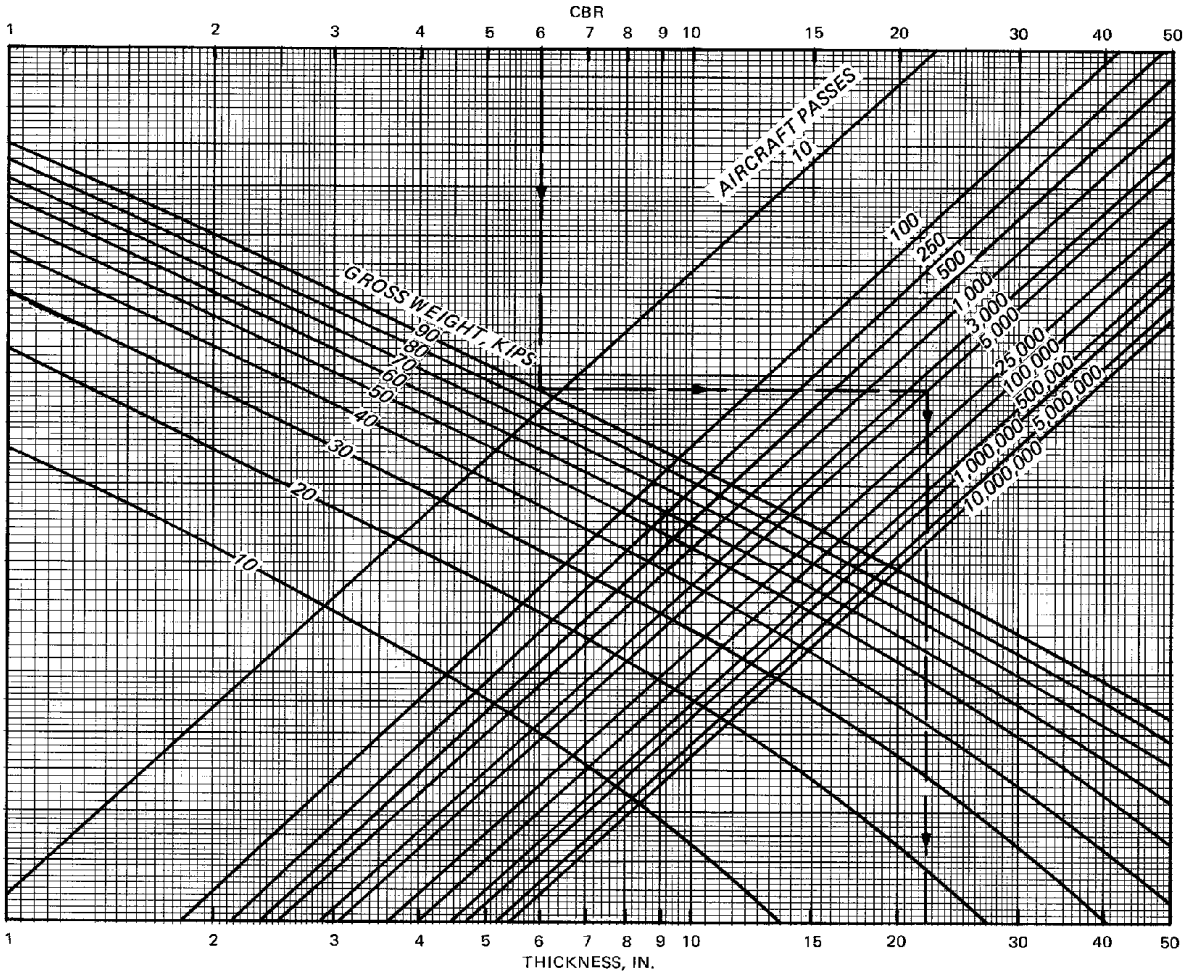


Figure 10-21b. Flexible pavement design curve for Air Force C-17 assault landing zone airfield

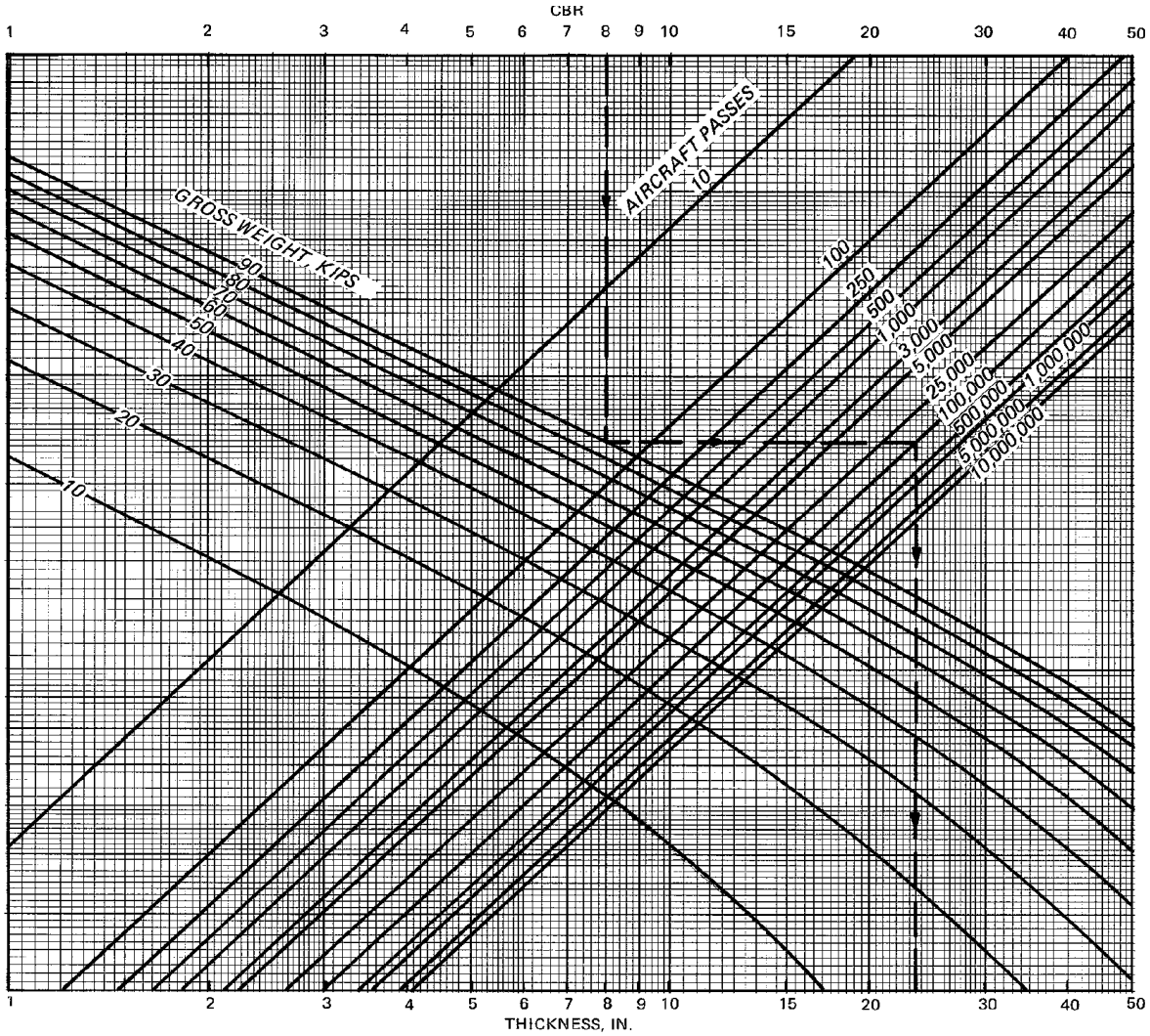


FLEXIBLE PAVEMENT DESIGN CURVE FOR F-15, TYPE A TRAFFIC AREA

CONVERSION FACTORS

MILLIMETERS = 25.4 x INCHES
KILOGRAMS = 0.4535 x POUNDS

Figure 10-22. Flexible pavement design curve for Air Force auxiliary airfield, type A traffic areas



FLEXIBLE PAVEMENT DESIGN CURVE FOR F-15, TYPE B AND C TRAFFIC AREAS

CONVERSION FACTORS

MILLIMETERS = 25.4 x INCHES
KILOGRAMS = 0.4535 x POUNDS

Figure 10-23. Flexible pavement design curve for Air Force auxiliary airfield, types B and C traffic areas

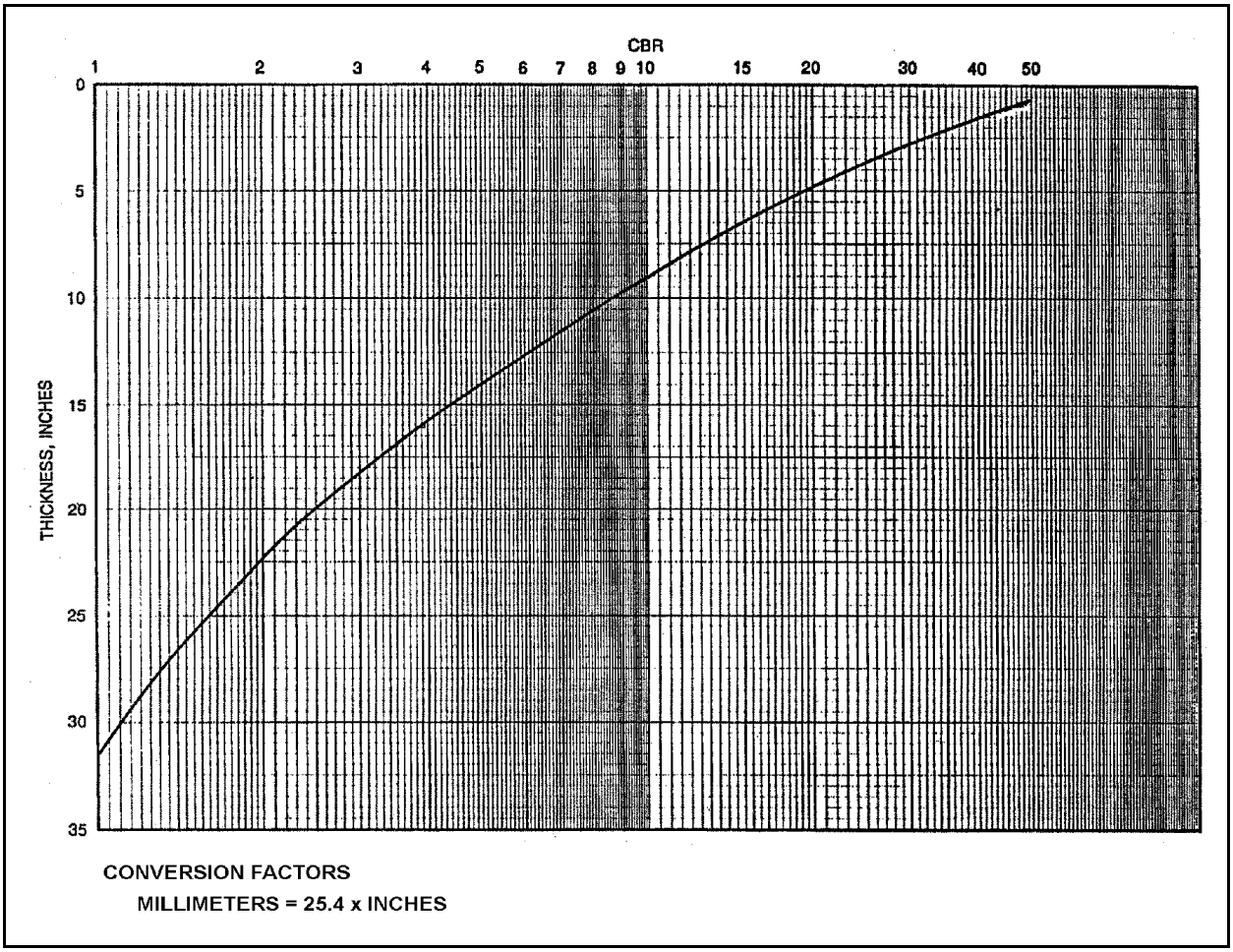


Figure 10-24. Flexible pavement design curve for shoulders on Army and Air Force pavements

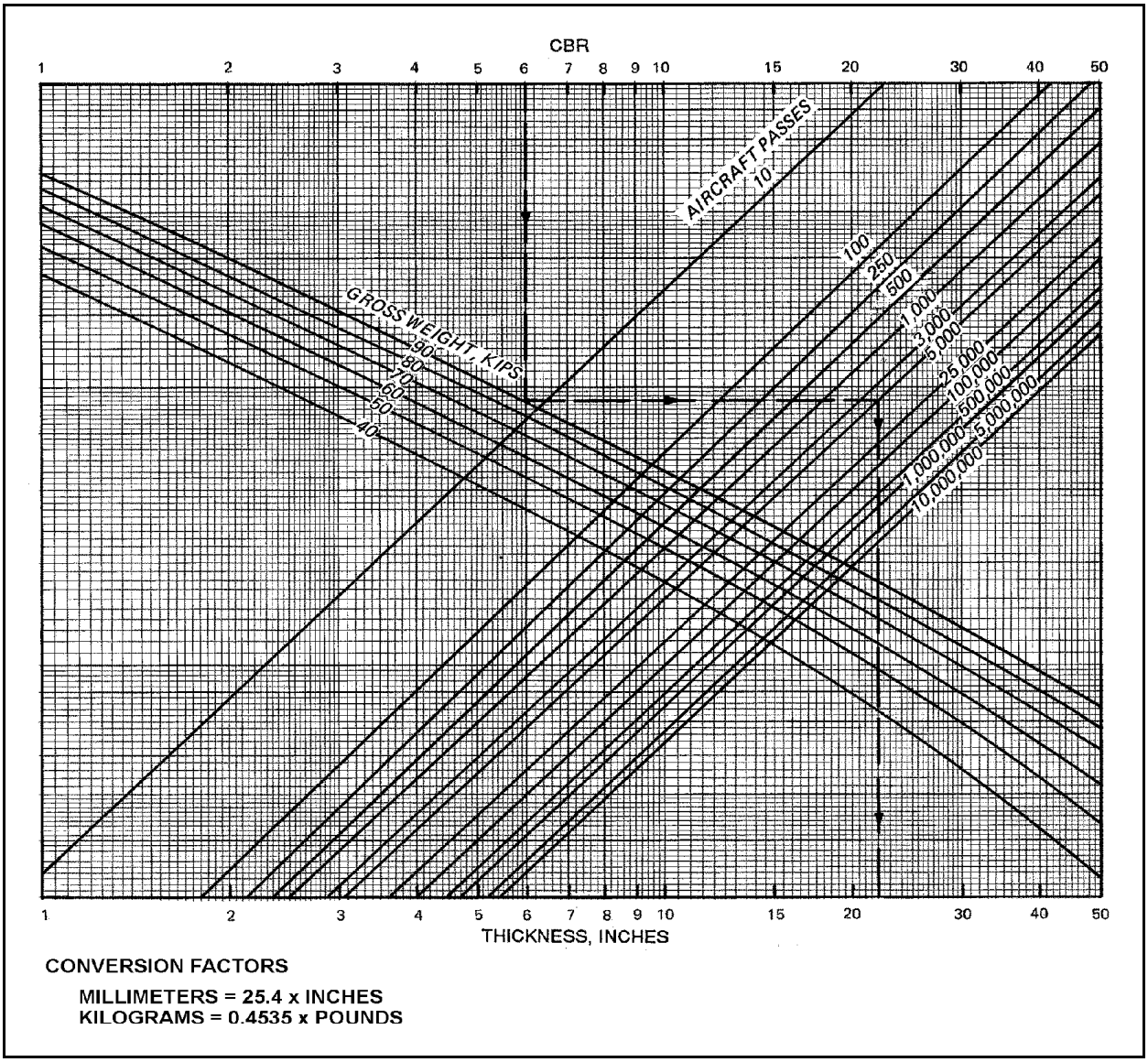


Figure 10-25. Air Force flexible pavement design curve for F-15, type A traffic areas

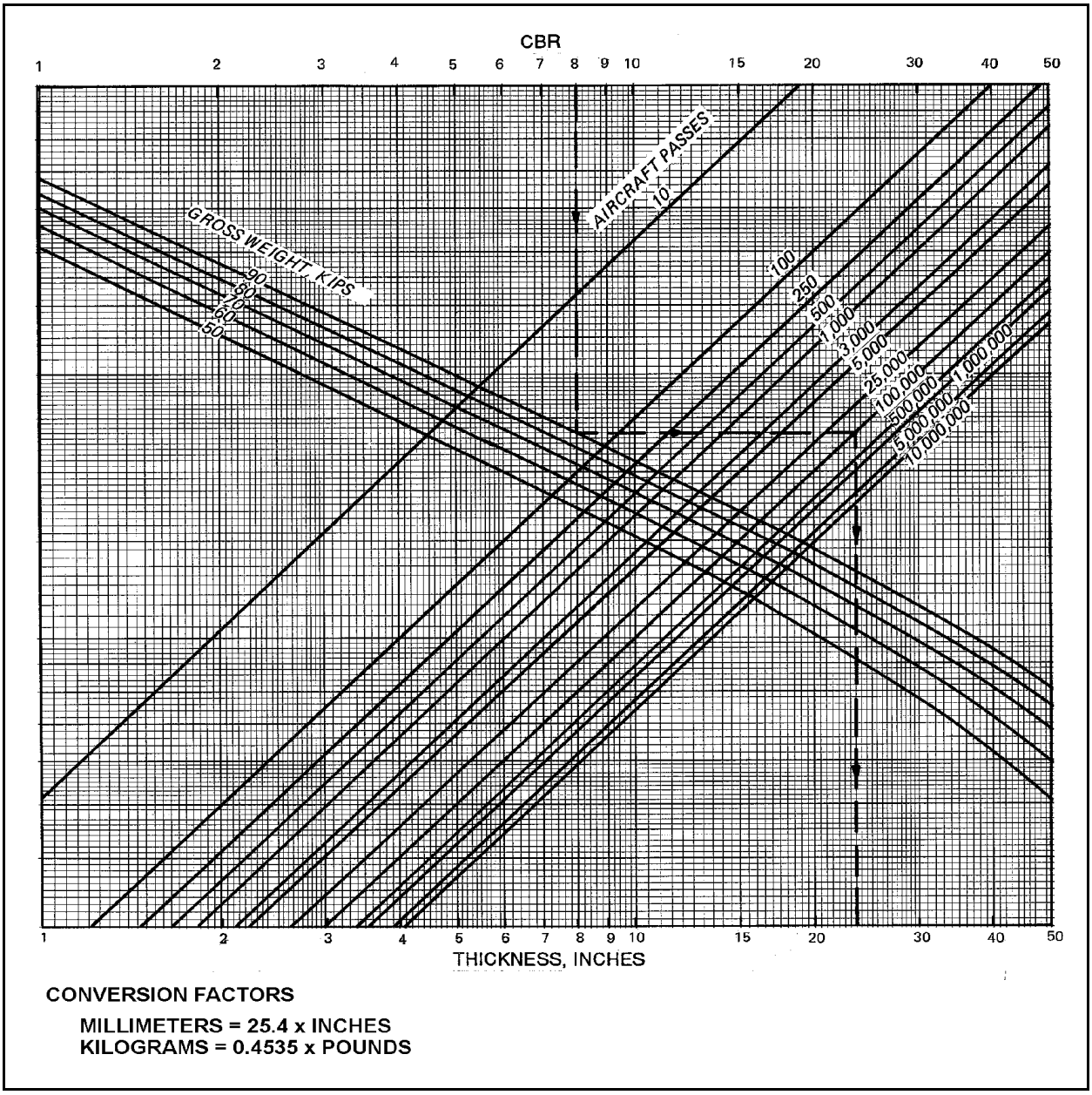


Figure 10-26. Air Force flexible pavement design curve for F-15, types B and C traffic areas

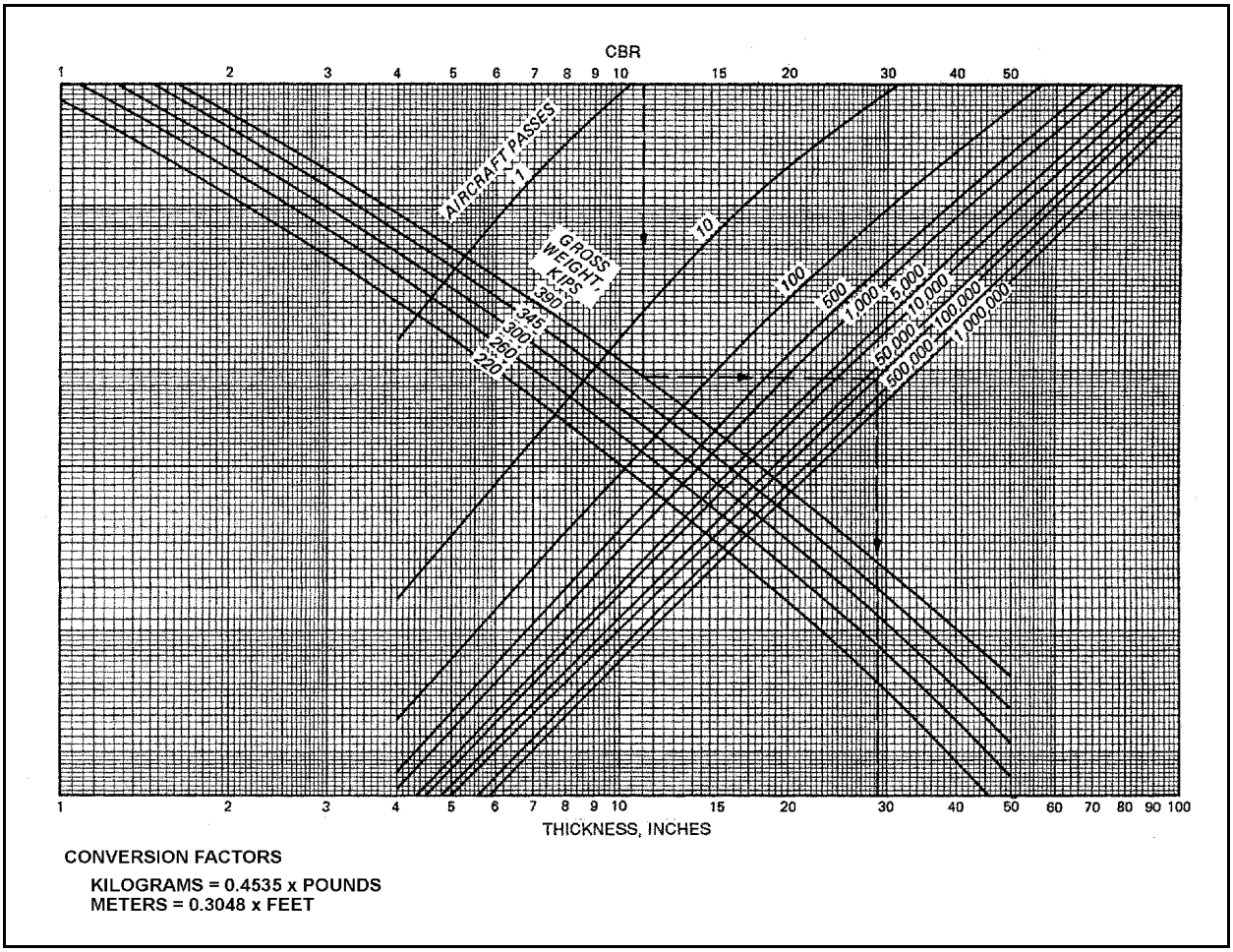


Figure 10-27. Air Force flexible pavement design curve for C-141, type A traffic areas

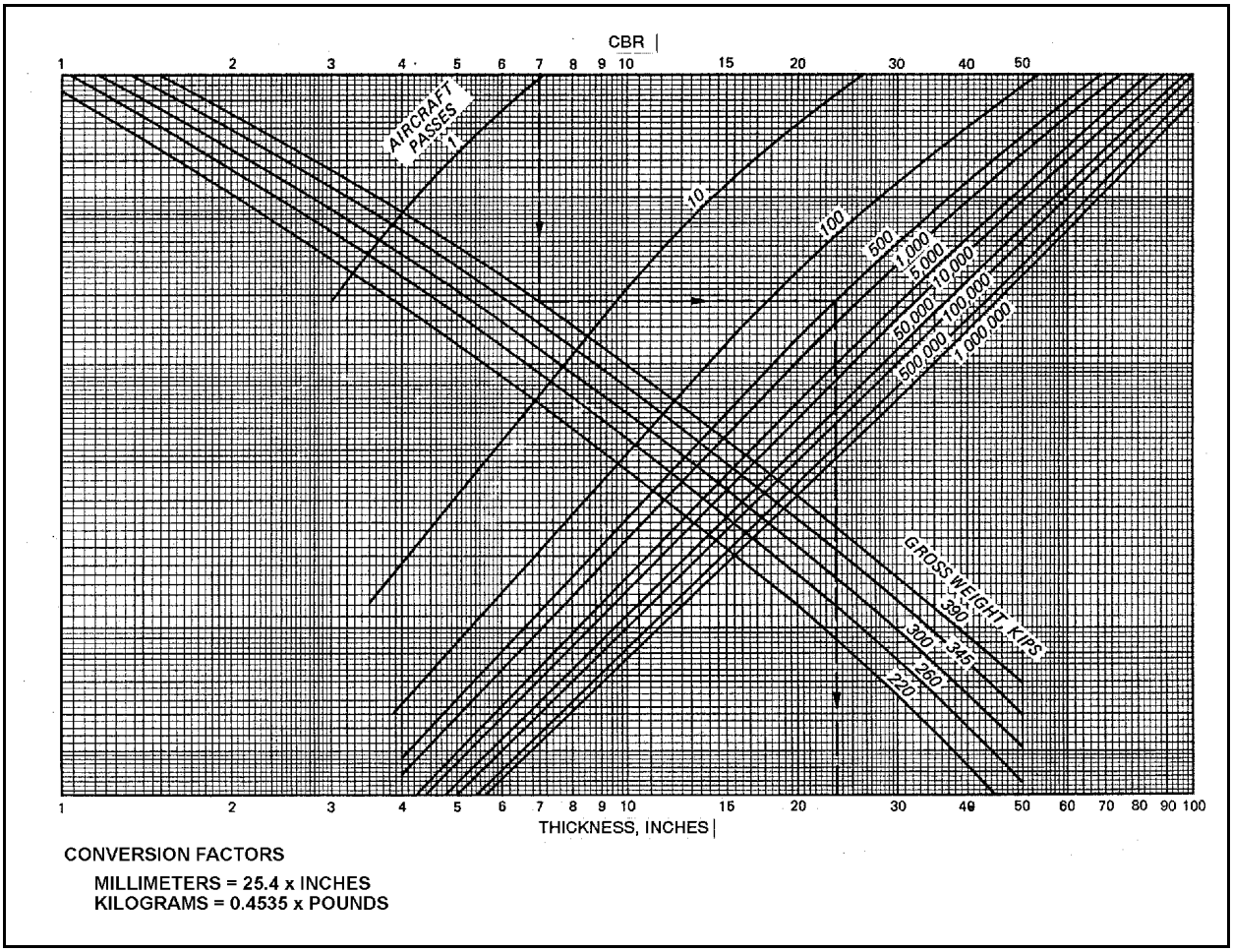


Figure 10-28. Air Force flexible pavement design curve for C-141, types B, C, and D traffic areas

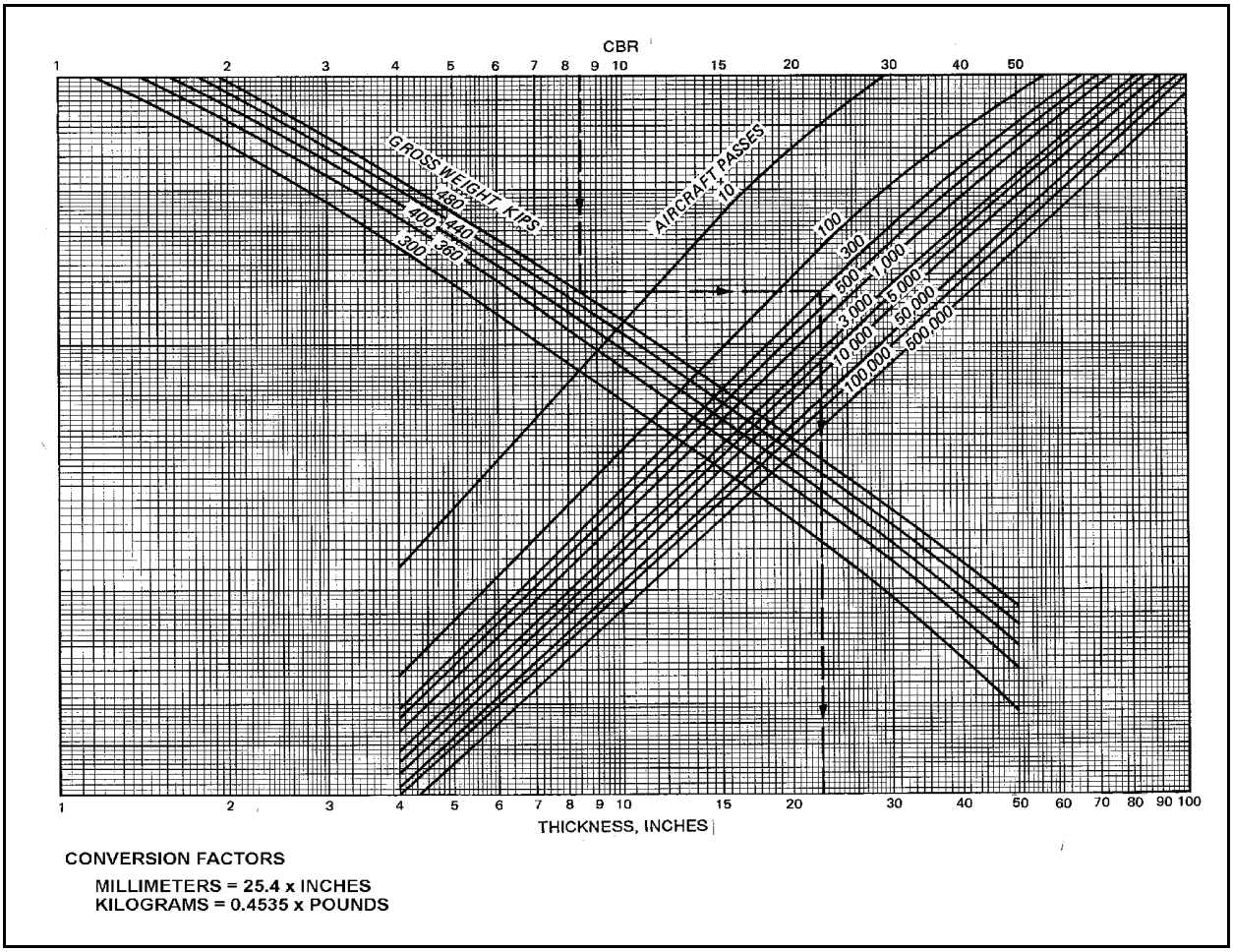


Figure 10-29. Air Force flexible pavement design curves for B-1, type A traffic areas

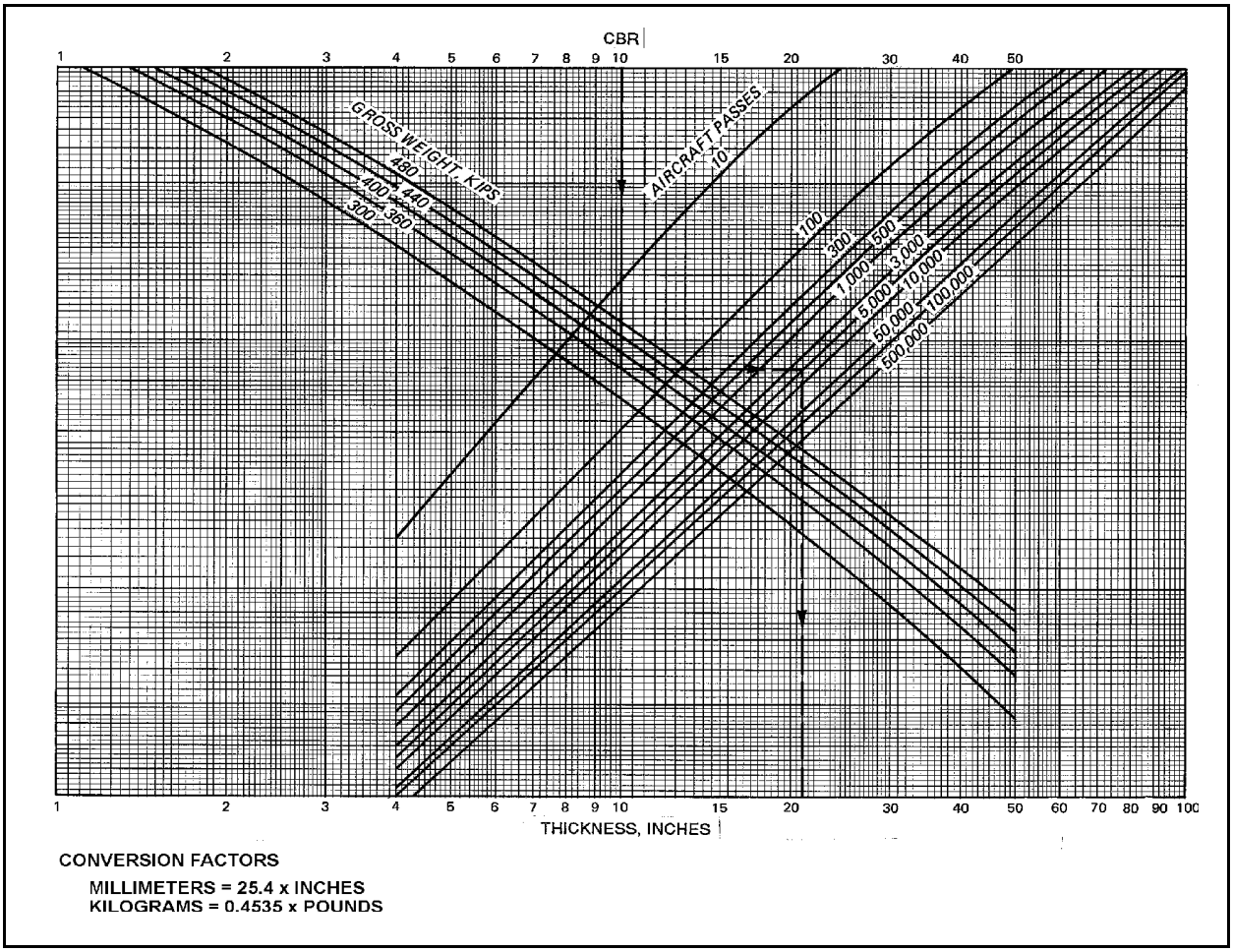


Figure 10-30. Air Force flexible pavement design curve for B-1, types B, C, and D traffic areas

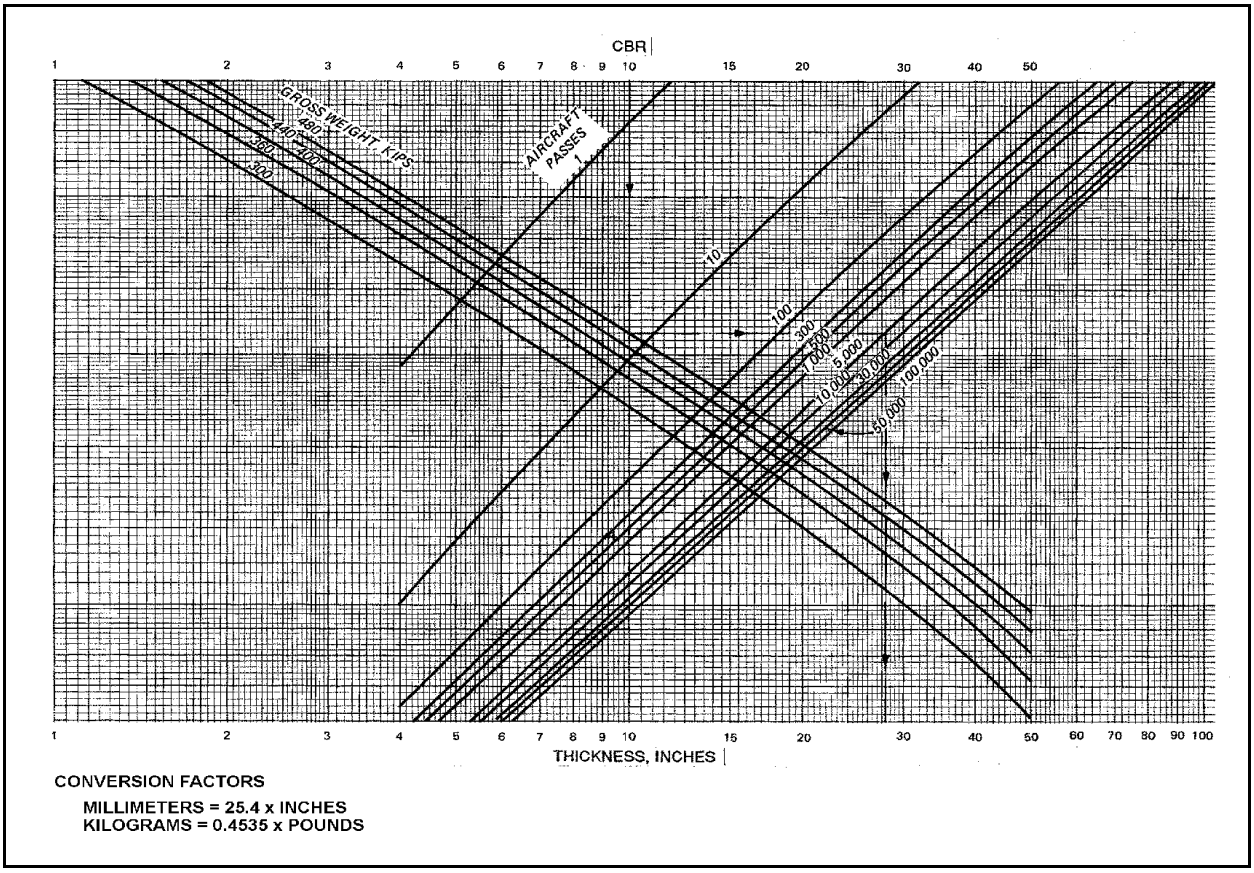


Figure 10-31. Air Force flexible pavement design curve for B-52, type A traffic areas

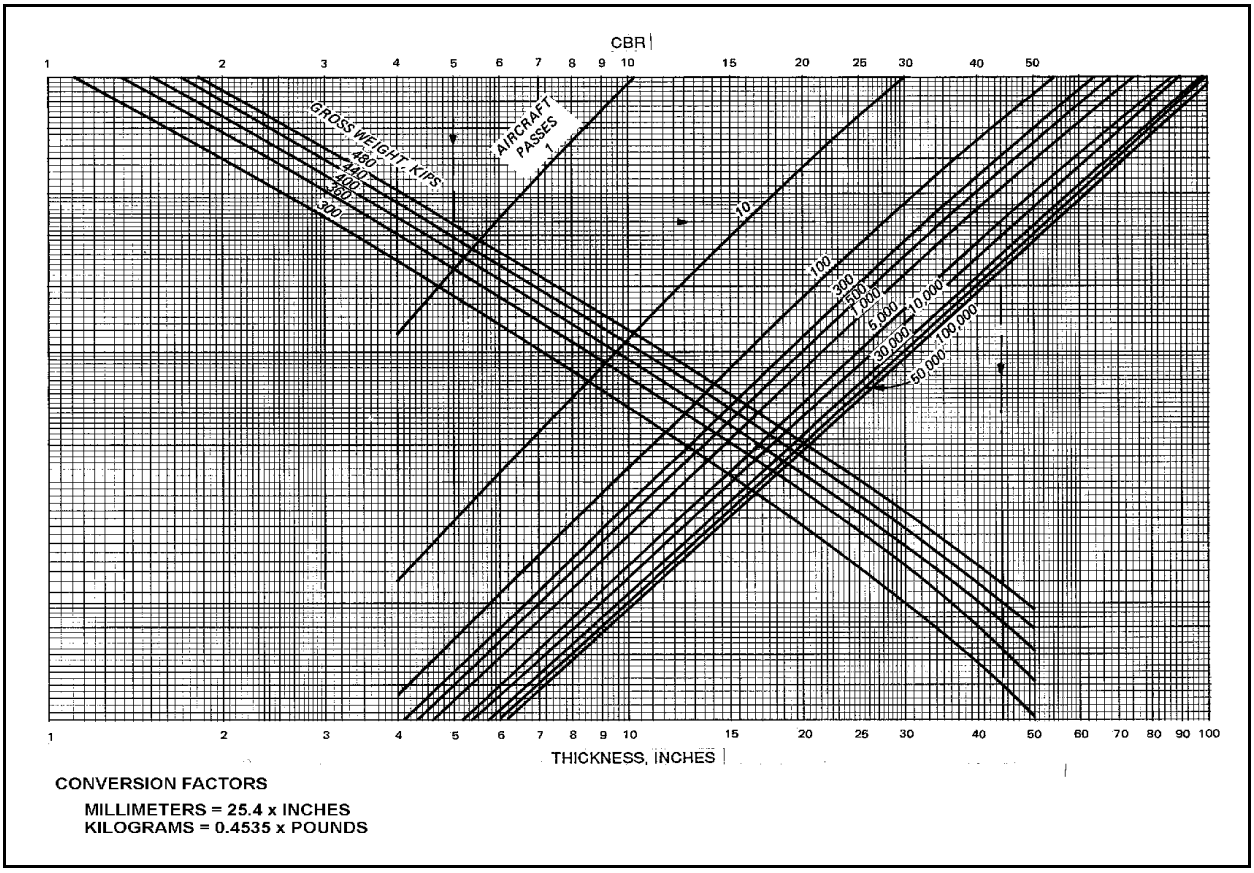


Figure 10-32. Air Force flexible pavement design curve for B-52, types B, C, and D traffic areas

CHAPTER 11

LAYER ELASTIC DESIGN OF FLEXIBLE PAVEMENTS

1. DESIGN PRINCIPLES. The structural deterioration of a flexible pavement caused by traffic is normally evidenced by cracking of the bituminous surface course and development of ruts in the wheel paths. The design procedure handles these two modes of structural deterioration through limiting values of the strain at the bottom of the bituminous concrete and at the top of the subgrade. Use of a cumulative damage concept permits the rational handling of variations in the bituminous concrete properties and subgrade strength caused by cyclic climatic conditions. The strains used for entering the criteria are computed by the use of Burmister's solution for multilayered elastic continua. The solution of Burmister's equations for most pavement systems will require the use of computer programs and the characterization of the pavement materials by the elastic constants of the modulus of elasticity and Poisson's ratio.

2. FLEXIBLE PAVEMENT RESPONSE MODEL. The computer code recommended for computing the pavement response is the JULEA code. When the code is used, the following assumptions are made.

a. The pavement is a multilayered structure, and each layer is represented by a modulus of elasticity and Poisson's ratio.

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

c. The bottom layer is of infinite thickness.

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

3. DESIGN DATA.

a. Climatic Factors. In the design system, two climatic factors, temperature and moisture, are considered to influence the structural behavior of the pavement. Temperature influences the stiffness and fatigue of bituminous material and is the major factor in frost penetration. Moisture conditions influence the stiffness and strength of the base course, subbase course, and subgrade.

(1) Pavement temperature. The design procedure requires the determination of a design pavement temperature for consideration of vertical compressive strain at the top of subgrade and horizontal tensile strain at the bottom of cement- or lime-stabilized layers and a different design pavement temperature for consideration of the fatigue damage of the bituminous concrete surface. In either case, a design air temperature from Figure 11-1 is used to determine the design pavement temperature. Temperature data for computing the design air temperatures are available from the National Oceanic and Atmospheric Administration (NOAA) "Local Climatological Data Annual Summary with Comparative Data." These data may be obtained by requesting it from personnel at NOAA's web site: *http://www.noaa.gov/.* With respect to subgrade strain and fatigue of cement- and lime-stabilized base or subbase courses, the design air temperature is the average of the average daily mean temperature and the average daily maximum temperature during the traffic period. For consideration of the fatigue damage of bituminous materials, the design air temperature is the average daily mean temperature. Thus, for each traffic period, two design air temperatures are determined. Normally,

monthly traffic periods should be adequate. For design purposes, it is best to use the long-term averages such as the 30-year averages given in the annual summary. The determination of the design pavement temperatures for 254-millimeter (10-inch) bituminous pavement can be demonstrated by considering the climatological data for Jackson, MS. For the month of August, the average daily mean temperature is 27.5 degrees Celsius (81.5 degrees Fahrenheit) and the average daily maximum is 33.6 degrees Celsius (92.5 degrees Fahrenheit); therefore, the design air temperature for consideration of the subgrade strain is 30.5 degrees Celsius (87 degrees Fahrenheit), and the design pavement temperature (determined from Figure 11-1) would be approximately 37.8 degrees Celsius (100 degrees Fahrenheit). For consideration of bituminous fatigue, the design air temperature for August in Jackson is 27.5 degrees Celsius (81.5 degrees Fahrenheit), resulting in a design pavement temperature of approximately 33.3 degrees Celsius (92 degrees Fahrenheit). These design pavement temperatures are determined for each of the traffic periods. Temperature data for Jackson (from "Local Climatological Data Annual Summary with Comparative Data, Jackson, Mississippi") are shown in Table 11-1.

Table 11-1
Temperature Data for Jackson, Mississippi

	Temperature, degrees C (degrees F)	
Month	Average Daily Maximum	Average Daily Mean
January	14.7 (58.4)	8.4 (47.1)
February	16.5 (61.7)	9.9 (49.8)
March	20.4 (68.7)	13.4 (56.1)
April	25.7 (78.2)	18.7 (65.7)
May	29.4 (85.0)	22.6 (72.7)
June	32.8 (91.0)	26.3 (79.4)
July	33.7 (92.7)	27.6 (81.7)
August	33.6 (92.5)	27.5 (81.5)
September	31.1 (88.0)	24.4 (76.0)
October	26.7 (80.1)	18.8 (65.8)
November	20.3 (68.5)	12.9 (55.3)
December	15.8 (60.5)	9.4 (48.9)

(2) Thaw periods. The effects of temperature on subgrade materials are considered only with regard to frost penetration. The basic requirement of frost protection is given in Chapter 20. If the pavement is to be designed for a weakened subgrade condition, the design must consider a period of time during which the subgrade will be in a weakened condition.

(3) Subgrade moisture content for material characterization. In most design situations, pavement design will be predicated on the assumption that the moisture content of the subgrade will approach saturation. If sufficient data are available that indicate the subgrade will not reach saturation, then the design may be based on a lower moisture content. Sufficient data for basing the design on a

moisture content lower than saturation would normally consist of field moisture content measurements under similar pavements located in the area. These measurements should be made during the most critical period of the year when the water table is at its highest elevation. Extreme caution should be exercised when the design is based on other than the saturated condition.

b. Traffic Data. The traffic parameters to be considered are the type of design aircraft, aircraft loading, traffic volume, and traffic area.

(1) Traffic volume. The design traffic volume is expressed in terms of total operations of the design aircraft expected during the life of the pavement. This traffic volume must be converted to a number of expected strain repetitions. In converting operations to strain repetitions, the concept of effective gear print is introduced. The effective gear print is the width of pavement that sustains an effective strain repetition at a given depth in the pavement. The effective gear print is a function of the number of tires in a transverse line, the transverse spacing, the width of the contact area, and the effective thickness of pavement above the location of strain. The effective thickness of the pavement is the sum of the thickness of unbound material plus twice the thickness of bound material where a bound material is an asphalt concrete or stabilized layer. Thus, for a pavement having 76 millimeters (3 inches) of asphalt and 381 millimeters (15 inches) of unbound gravel, the effective thickness with reference to the strain at the top of the subgrade would be 381+(2×76) (15+(2×3)), or 533 millimeters (21 inches), and with respect to the strain at the bottom of the asphalt, the effective thickness would be 2×76 (2×3), or 152 millimeters (6 inches). With the determination of the effective thickness, the gear print is computed as illustrated in Figures 11-2 and 11-3. If the gear is composed of tracking tires such as tandem gear, then the number of strain repetitions may be somewhat greater than if the gear were not tandem. When the tracking tires are located far enough apart, two distinct strain pulses will occur and the multiplication factor for the tandem gear is 2. When the tires are sufficiently close, the strain pulses merge into a single pulse and the multiplication factor is 1. The computation of F is shown in Figure 11-4. In the figure, B is the spacing between tandem tires in the gear; t_e is the effective pavement thickness; and T_w is the length of the ellipse that is formed by the tire imprint. When t_e is less than $B - T_w$, F is 2. When t_e is greater than twice the difference between B and T_w, F is 1. For values of t_e between the two conditions, F is computed based on the equation:

$$F = \frac{3 \cdot (B - T_w) - t_e}{B - T_w} \qquad (11\text{-}1)$$

(a) The concept for conversion of aircraft operations to effective strain repetitions involves assuming that traffic distribution on the pavement can be represented by a normal distribution. For traffic on taxiways and runway ends (first 305 meters (1,000 feet)), the distribution has a wander width of approximately 178 millimeters (70 inches), and traffic on runway interiors has a wander width of approximately 355 millimeters (140 inches). (Note that wander width is defined as the width that contains 75 percent of the applied traffic.) From the normal distribution, the fraction of traffic for which the effective gear print will encompass a given point in the pavement can be computed. This fraction times F gives the number or fraction of the effective strain repetitions at a point in the pavement for each aircraft operation.

(b) The number of effective strain repetitions the pavement sustains at a point for every aircraft operation is the pass-to-strain conversion percentage. For an effective thickness of 0.00 millimeters (0 inches), the percentage is the inverse of the pass-to-coverage ratio multiplied by 100.

The procedure for computing the pass-to-strain conversion percentage has been computerized, and the factors can easily be computed for single, twin, single-tandem, twin-twin, twin-tandem, or other gears.

(c) The distribution of the pass-to-strain conversion percentages as a function of point location and effective thicknesses is given in Appendix E. These pass-to-strain conversion percentages can be used to convert, for any point location, the number of aircraft operations to effective strain repetitions.

(2) Aircraft loading. The aircraft loading and gear characteristics are used in the response model for computing the magnitude of strain. The information needed includes the number of tires, tire spacing, load per tire, and contact pressure. The radius of the loaded area is computed based on the assumption of a uniformly loaded circular area, i.e.,

$$r = \sqrt{\frac{L}{\pi p}} \tag{11-2}$$

where

r = radius of loaded area, millimeters (inches)

L = load per tire, Newtons (pounds)

p = contact pressure, MPa (psi)

Note: units should be consistent with units of the section parameters.

In principle, all main tires should be used in computing the strain, but usually only the tires on one landing gear need to be used. The distance between gears for common aircraft is sufficiently great to prevent interaction between gears. Within a main gear, some searching for the maximum strain may be needed. For most cases the maximum strain will occur under one of the tires, but for closely spaced tires or strains at a great depth, the maximum may move toward the center of the tire group.

(3) Traffic grouping. The traffic is grouped so that within each group each individual pass of an aircraft will cause damage similar a pass of any other aircraft in the group. That is, the pattern of strain of every pass of the group would be almost the same; then the value of the allowable number of passes (N) would be the same. For this to be true, the loading characteristics for aircraft within a group must be similar, and the single set of material properties must be applicable for all passes within the group. Grouping reduces considerably the design effort, and it is advantageous to reduce traffic to as few groups as possible. Grouping of the aircraft by similar pass-to-strain conversion percents has already been accomplished in Appendix E. Additional subgrouping would be necessary to account for other differences, such as load magnitude and tire pressure. Also, other groupings may be necessary to account for changes in material properties such as changes in subgrade modulus caused by thaw and changes in asphalt modulus caused by temperature. For pavements that are relatively unaffected by changes in temperature and are designed based on a single critical aircraft, it may be possible to reduce the aircraft operations to a single group. In this case, the design procedure simplifies to determining allowable strains for the design aircraft and to adjusting the pavement thicknesses to obtain the

allowable strain. Where the grouping cannot be reduced to a single group, then the concept of the cumulative damage must be used in the design process.

4. MATERIAL CHARACTERIZATION. Characterization of the pavement materials requires the quantification of the material stiffness as defined by the resilient modulus of elasticity and Poisson's ratio and, for selected pavement components, a fatigue strength as defined by a failure criterion. Inasmuch as possible, repeated load laboratory tests designed to simulate aircraft loading are used to determine the resilient stiffness of the materials. For some materials, such as unbound granular bases and subbases, an empirically based procedure was judged a better approach for obtaining usable material parameters. Failure criteria have been provided; thus, fatigue testing will not be necessary. In general, the use of layered elastic design procedures does not negate the material requirements set forth in Chapters 7, 8, and 9. In particular, the gradation, strength, and durability requirements as stated must be maintained.

a. Modulus of Elasticity.

(1) Bituminous mixtures. The term "bituminous mixtures" refers to a compacted mixture of bitumen and aggregate designed in accordance with standard practice. The modulus for these materials is determined by use of the repetitive triaxial test. The procedure for preparation of the sample is given in Appendix F with the procedure for the conduct of the repetitive triaxial test given in Appendix G.

(a) The stiffness of the bituminous mixtures will be greatly affected by both the rate of loading and by temperature. For runway design, a loading rate of 10 hertz is recommended. For taxiway and apron design, a loading rate of 2 hertz is suggested. These loading rates are appropriate for aircraft speeds of over 45 meters/second (100 miles/hour) on runways and less than 9 meters/second (20 miles/hour) on taxiways and aprons. Specimens should be tested at temperatures of 44, 21, and 38 degrees Celsius (40, 70, and 100 degrees Fahrenheit) so that a modulus-temperature relationship can be established. If temperature data indicate greater extremes than 4.4 and 38 degrees Celsius (40 and 100 degrees Fahrenheit), tests should be conducted at these extreme ranges if possible. The modulus value to be used for each strain computation would be the value applicable for the specific pavement temperature determined from the climatic data.

(b) An indirect method of obtaining an estimated modulus value for bituminous concrete is presented in detail in Appendix H. Use of this method requires that the ring-and-ball softening point and the penetration of the bitumen as well as the volume concentration of the aggregate and percent air voids of the compacted mixture be determined.

(2) Unbound granular base- and subbase-course materials. The terms "unbound granular base-course material" and "unbound granular subbase-course material" as used herein refer to materials meeting grading requirements and other requirements for base and subbase for airfield pavements, respectively. These materials are characterized by use of a chart in which the modulus is a function of the underlying layer and the layer thickness. The chart and the procedure for use of the chart are given in Appendix I.

(3) Stabilized material. The term "stabilized material" as used herein refers to soil treated with such agents as bitumen, portland cement, slaked or hydrated lime, and fly ash or a combination of such agents to obtain a substantial increase in the strength of the material. Stabilization with portland cement, lime, fly ash, or other agent that causes a chemical cementation to occur shall be referred to as chemical stabilization. Chemically treated soils having unconfined compressive strengths greater than the minimum strength specified for subbases are considered to be stabilized materials and should be tested

in accordance with the methods specified for stabilized materials. Chemically treated soils having unconfined compressive strengths less than that specified for subbases are considered to be modified subgrade soils and should be tested under the provisions for subgrade soils. Most likely this will result in using the maximum allowable subgrade modulus. Bituminous-stabilized materials should be characterized in the same manner as bituminous concrete. Stabilized materials other than bituminous-stabilized should be characterized using flexural beam tests or cracked-section criteria. Flexural modulus values determined directly from laboratory tests can be used when the effect of cracking is not significant and the computed strain based on this modulus does not exceed the allowable strain for the material being used.

(a) The general approach in the flexural beam test is to subject the specimen to repeated loadings at third points, measure the maximum deflection at the center of the beam (i.e., at the midpoint of the neutral axis), and calculate the values for the flexural modulus based on the theory of a simply supported beam. A correlation factor for stress is applied.

(b) Procedures for preparing specimens of and conducting flexural beam tests on chemically stabilized soils are presented in detail in Appendix J.

(c) The stabilized material for the base and subbase must meet the strength and durability requirement of TM 5-822-14/AFJMAN 32-1019. The strength requirements are as summarized in Chapter 9.

(4) Subgrade soils. The modulus of the subgrade is determined through the use of the repetitive triaxial test. For most subgrade soils, the modulus is greatly affected by changes in moisture content and state of stress. As a result of normal moisture migration, water table fluctuation, and other factors, the moisture content of the subgrade soil can increase and approach saturation with only a slight change in density. Since the strength and stiffness of fine-grained materials are particularly affected by such an increase in moisture content, these soils should be tested in the near-saturation state. Two methods are available to obtain a specimen with this moisture content: the soil can be molded at optimum moisture content and subsequently saturated or molded at the higher moisture content using static compaction methods. Evidence exists that the resilient properties of both specimen types are similar. It is not apparent whether this concept is valid for materials compacted at the higher densities; therefore, for the test procedures presented herein, back-pressure saturation of samples compacted at optimum is recommended for developing high moisture contents in test specimens.

(a) For cohesive subgrades, the resilient modulus of the subgrade will normally decrease with an increase in deviator stress, and therefore, the modulus is determined as a function of deviator stress. The modulus of granular subgrades will be a function of the first invariant. Procedures for specimen preparation, testing, and interpretation of test results for cohesive and granular subgrades are presented in Appendix K. For the layered elastic theory design procedure, however, the maximum allowable modulus for a subgrade soil should be restricted to 207 MPa (30,000 psi).

(b) In areas where the subgrade is to be subjected to freeze-thaw cycles, the subgrade modulus must be determined during the thaw-weakened state. Testing soils subject to freeze-thaw requires specialized test apparatus and procedures. Where commercial laboratories are not available, the U.S. Army Cold Regions Research and Engineering Laboratory (CRREL), 72 Lyme Road, Hanover, NH 03755, can conduct tests to characterize subgrade soils subjected to freeze-thaw.

(c) For some design situations, estimating the resilient modulus of the subgrade (M_R) based on available information may be necessary when conducting the repetitive load triaxial tests. An

estimate of the resilient modulus in megapascals (pounds per square inch) can be made from the relationship of M_R = 10.3×CBR (M_R = 1,500×CBR). The relationship does provide a method for checking the reasonableness of the laboratory results.

b. Poisson's Ratio. Because of the complexity of laboratory procedures involved in the direct determination of Poisson's ratio for pavement materials and because of the relatively minor influence on pavement design of this parameter when compared with other parameters, use of values commonly recognized as acceptable is recommended. These values for the four classes of pavement materials considered herein are presented in Table 11-2.

Table 11-2
Typical Poisson's Ratios for Four Classes of Pavement Materials

Pavement Materials	Poisson's Ratio v
Bituminous concrete	0.5 for E < 3,450 MPa (500,000 psi) 0.3 for E > 3,450 MPa (500,000 psi)
Unbound granular base- or subbase-course	0.3
Chemically stabilized base- or subbase-course	0.2
Subgrade	
Cohesive subgrade	0.4
Cohesionless subgrade	0.3

Note: E = elastic modulus of bituminous concrete (psi)

5. SUBGRADE EVALUATION. Chapter 6 provides for the evaluation of the subgrade for design by the CBR design procedure and also provides the background for evaluation of the subgrade modulus. After the establishment of the grade line, the pavement will be grouped as to soil type, strength, expected moisture content, compaction requirements, and other characteristics. For each soil group, a minimum of six resilient modulus tests should be conducted and the design modulus determined according to procedures given in Appendix K. The design modulus would be the average of the moduli obtained from the testing.

6. DESIGN CRITERIA. The damage factor (DF) is defined as $DF = \frac{n}{N}$, where n is the number of effective strain repetitions and N is the number of allowable strain repetitions. The cumulative damage factor is the sum of the damage factors for all aircraft. The value of n is determined from the number of aircraft operations. The value of N must be determined from the computed strain and the appropriate criteria. Basically, there are three criteria to determine N . These are the allowable number of repetitions as a function of the vertical strain at the top of the subgrade, the allowable number of repetitions as a function of the horizontal strain at the bottom of the bituminous concrete, and the allowable number of repetitions as a function of the horizontal strain at the bottom of a chemically stabilized base or chemically stabilized subbase. It should be noted that there is no strain criterion for unbound base. In the development of the procedure, it has been assumed that an unbound base and subbase that meets the specifications for quality will perform satisfactorily.

a. Subgrade Strain Criteria. The subgrade strain criteria were developed from the analysis of field test data and present the allowable number of strain repetitions as a function of strain magnitude. The data analysis indicated that the relationship between allowable repetitions and strain magnitude is slightly different for subgrades having different resilient moduli. The criteria are presented in graphic form in Figure 11-5 and can be approximated using the following equation:

$$\textit{allowable repetitions} = 10{,}000 \cdot \left(\frac{A}{S_s}\right)^B \qquad (11\text{-}3)$$

where

$A = 0.000247 + 0.000245 \log M_R$

M_R = resilient modulus of the subgrade, psi

S_s = vertical strain at the top of the subgrade (in./in.)

$B = 0.0658\ M_R^{0.559}$

b. Asphalt Strain Criteria.

(1) The primary means recommended for determining values of limiting horizontal tensile strain for bituminous concrete is the use of the repetitive load flexural beam tests on laboratory-prepared specimens. Procedures for the tests are presented in detail in Appendix L. Several tests are run at different stress levels and different sample temperatures such that the number of load repetitions to fracture can be represented as a function of temperature and initial stress. The initial stress is converted to initial strain to yield criteria based on the tensile strain of the bituminous concrete.

(2) An alternate method for determining values of limiting tensile strain for bituminous concrete is the use of the provisional laboratory fatigue data employed by Heukelom and Klomp. These data are presented in Appendix L in the form of a relationship between stress, strain, load repetitions, and elastic moduli of bituminous concrete. The allowable strain repetitions may be approximated by the equation

$$\textit{Allowable strain repetitions} = 10^X \qquad (11\text{-}4)$$

where

$X = 2.68 - 5.0 \log S_A - 2.665 \log E$

S_A = tensile strain of asphalt (in/in)

E = elastic modulus of the bituminous concrete (psi)

c. Chemically Stabilized Layers. For cement- and lime-stabilized materials, the criteria are to be developed using test procedures outlined in Appendix B. When flexural fatigue tests are not possible, then a preestablished relationship as shown in Figure 11-6 should be used.

d. Computer Programs for Computing Cumulative Damage Factor. Two computer codes are used for computing the subgrade and asphalt damage factors based on Equations 11-3 and 11-4. Both programs require material strains obtained by the running of the layered elastic computer programs. The listings of the programs contain an explanation of the input and instructions on the use of the programs. An example illustrating the use of the programs is given in this chapter in the paragraph entitled Example Design for Conventional Flexible Pavement.

7. CONVENTIONAL FLEXIBLE PAVEMENT DESIGN. Conventional flexible pavements consist of relatively thick aggregate layers with a 75- to 125-millimeter (3- to 5-inch) wearing course of bituminous concrete. In this type of pavement, the bituminous concrete layer is a minor structural element of the pavement, and thus, the temperature effects on the stiffness properties of the bituminous concrete may be neglected. Also, it must be assumed that if the minimum thickness of bituminous concrete is used as specified in Tables 8-2 through 8-5, then fatigue cracking will not be considered. Thus, for a conventional pavement, the design problem is one of determining the thickness of pavement required to protect the subgrade. The steps for determining the required thickness for nonfrost areas are:

a. The subgrade resilient modulus is determined based on the soil exploration, climatic conditions, and laboratory testing. The resilient modulus of the bituminous concrete is assumed to be 1,380 MPa (200,000 psi).

b. The traffic data determine the design loadings and repetitions of strain.

c. An initial pavement section is determined from the minimum thickness requirements as determined using Chapter 10 or by estimation. The resilient modulus of the base and of the subbase is determined based on the chart and the initial thickness.

d. The vertical strain at the top of the subgrade is computed for each aircraft being considered in the design.

e. The number of allowable strain repetitions for each computed strain is determined from the subgrade strain criteria.

f. The value of n/N is computed for each aircraft and summed to obtain the cumulative damage factor.

g. The assumed thicknesses are adjusted to make the value of the cumulative damage factor approach 1.0. This may be accomplished by first making the computations for three thicknesses and developing a plot of thickness versus damage factor. From this plot the thickness that gives a damage factor of 1.0 may be selected.

8. FROST CONDITIONS. Where frost conditions exist and the design is to be based on a base and subbase thickness less than the thickness required for complete frost protection, the design must be based on two traffic periods as described previously. In some cases, it may be possible to replace part of the subgrade with material not affected by cycles of freeze-thaw but which will not meet the specifications for a base or subbase. In this case, the material must be treated as a subgrade and characterized by the procedures given for subgrade characterization.

9. ASPHALT CONCRETE PAVEMENTS. The asphalt concrete pavement differs from the conventional flexible pavement in that the asphalt concrete is sufficiently thick to contribute significantly to the strength of the pavement. In this case, the variation in the stiffness of the asphalt concrete caused

by yearly climatic variations must be taken into account by dividing the traffic into increments during which variation of the resilient modulus of the asphalt concrete is at a minimum. One procedure is to determine the resilient modulus of the asphalt concrete for each month, then group the months when the asphalt concrete has a similar resilient moduli. Thus, it may be possible to reduce the traffic to three or four groups. Also, since the asphalt concrete is a major structural element, the failure of this element due to fatigue cracking must be checked. The flow diagram for design of the asphalt concrete pavements is given in Figure 11-7.

10. PAVEMENTS WITH A STABILIZED BASE COURSE. For a pavement having a chemically stabilized base course and an unbound aggregate subbase course, damage must be accumulated for subgrade strain, for horizontal tensile strain at the bottom of the bituminous concrete surfacing, and for horizontal tensile strain at the bottom of the chemically stabilized layer. Normally in this type of pavement, the base-course resilient modulus is sufficiently high (≥ 690 MPa (100,000 psi)) to prevent fatigue cracking of the bituminous concrete surface course (where the bituminous concrete surface course has a thickness equal to or greater than the minimum required in Tables 8-2 through 8-5), and thus this mode of failure is only a minor consideration. For most cases, a very conservative approach can be taken in checking for this mode of failure; i.e., all the traffic can be grouped into the most critical time period and the computed bituminous concrete strain compared with the allowable strain. If the conservative approach indicates that the surface course is unsatisfactory, then the damage should be accumulated in the same manner used for conventional flexible pavement. For the pavement having a stabilized base or subbase, checking the subgrade strain criteria becomes more complicated than for conventional flexible or bituminous concrete pavements. Two cases in particular should be considered. In the first case, the stabilized layer is considered to be continuous, with cracking due only to curing and temperature. In the second case, the stabilized layer is considered cracked because of load. The first step in evaluating the stabilized layer is to compute the horizontal tensile strain at the bottom of the stabilized layer and the vertical compressive strain at the top of the subgrade under assumptions that the stabilized layer is continuous and has a modulus value as determined by the flexural resilient modulus test. To account for the increase in stress due to loadings near shrinkage cracks, the computed strains should be multiplied by 1.5 for comparison with the allowable strains. If the analysis shows that the stabilized base will not crack under load, then it will be necessary to compare the adjusted value of subgrade strain with the allowable subgrade strain. If this analysis indicates that the adjusted strain is not less than or equal to the allowable strain, then the thickness should be increased and the process repeated, or the section should be checked under the assumption that the base course will crack and behave as a granular material. The cracked stabilized base course is represented by a reduced resilient modulus value, which is determined from the relationship between resilient modulus and unconfined compressive strength shown in Figure 11-8. When the cracked base concept is used, only the subgrade criteria need to be satisfied. The section obtained should not differ greatly from the section obtained by use of the equivalency factors in Table 9-1 or 9-2. A flow diagram for the design of this type of pavement is shown in Figure 11-9.

11. PAVEMENTS WITH STABILIZED BASE AND STABILIZED SUBBASE. This type of pavement is handled almost identically to a pavement with a stabilized base. If the base is a bituminous-stabilized material, then the cumulative damage procedure must be employed to determine if the subbase will crack. If the analysis indicates that the subbase will crack due to loading, an equivalent cracked-section modulus is determined from Figure 11-8, and the pavement is treated as a bituminous concrete pavement. If both the base and subbase courses are chemically stabilized, then both layers must be checked for cracking. A conservative approach is taken by checking for cracking of one layer by considering the other stabilized layer as cracked and having a reduced modulus. The vertical compressive strain at the top of the subgrade is computed by use of the flexural modulus or the reduced modulus, as appropriate. If either of the two layers is considered uncracked, then the computed

subgrade strain is multiplied by 1.5 to account for the shrinkage cracks that will exist. The basic flow diagram for this type of pavement is shown in Figure 11-10.

12. EXAMPLE DESIGN FOR CONVENTIONAL FLEXIBLE PAVEMENT. To illustrate the application of the design procedure, consider a design for an Army Class IV airfield. The subgrade is a lean clay classified as CL. The design is to be for 200,000 passes of the C-130 aircraft. The design loading for the C-130 on the taxiway is 70,310 kilograms (155,000 pounds) with a tire contact area of 0.258 square meters (400 square inches). For the runway interior the loading is 75 percent of the taxiway loading. The reduction is accomplished by reducing the contact area, giving a contact area of 0.194 square meters (300 square inches). The design process may best be illustrated in steps. The basic steps are material investigation, determination of trial pavement sections, computation of critical strains, determination of applied strain repetitions, and computation of damage factors.

a. Step 1 - Material Investigation.

(1) The evaluation of the subgrade is to be accomplished by field and laboratory studies. The subgrade is to be classified according to different material types and material processing. For this example, it is assumed that the subgrade is fairly uniform and consists of a compacted lean clay placed according to existing compaction requirements. The subgrade evaluation involves conducting a series of resilient modulus tests according to the procedures given in Appendix L. For a location such as Shreveport, LA, it must be assumed that the subgrade would become saturated and thus the resilient modulus tests are conducted on saturated samples. A minimum of six samples should be tested and a design modulus determined for each sample. For determination of a design modulus, the data from the laboratory tests are plotted on a log-log plot of M_R versus σ_d and overlaid on the design curves as shown in Figure 11-11. For the design example, the design modulus obtained using this process is assumed to be 62 MPa (9,000 psi) for both taxiway and runway designs. Base and subbase materials must be obtained that meet the requirements of Chapters 7 and 8. The modulus values for the base and subbase are to be determined by the procedures given in Appendix J. Because the modulus of these materials depends on layer thicknesses, the modulus cannot be obtained until the trial sections are determined.

(2) The bituminous surfacing must meet the requirements of Chapter 9 as to minimum thickness and composition. The modulus-temperature relationship is determined according to the test procedures given in Appendix H or by the provisional procedure given in Appendix I. Assume for the example problem that the relationship as shown in Figure 11-12 is obtained from laboratory test data. (For simplicity, these data will be used for both taxiway and runway.) From the climatic data, the design air temperature is obtained and the design modulus values are determined as shown in Tables 11-3 and 11-4. To reduce the number of computations, the 12 groups are reduced to four groups as shown in Table 11-5. The Poisson's ratio for all materials is selected from Table 11-2.

b. Step 2 - Determination of Initial Section.

(1) From Chapter 10 the total thickness of pavement required for a gross aircraft load of 70,310 kilograms (155,000 pounds), 200,000 passes, and a subgrade CBR of 6 is determined to be 710 millimeters (28 inches). For the runway interior design, the thickness would be based on a gross aircraft load of 52,730 kilograms (116,250 pounds) and would result in an estimated thickness of 610 millimeters (24 inches). For taxiway design, subgrade damage factors will be computed for pavement thicknesses of 680, 760, and 840 millimeters (27, 30, and 33 inches) in an attempt to bracket the final required pavement thickness. The total thickness of pavement is made up of the asphalt

Table 11-3
Bituminous Concrete Moduli for Each Month for Conventional Flexible Pavement Design Based on Subgrade Strain

Month (1)	Average Daily Mean Air Temperature,[1] degrees F (2)	Average Daily Maximum Air Temperature,[2] degrees F (3)	Design Air Temperature,[2] degrees F (4)	Design Pavement Temperature,[3] degrees F (5)	Dynamic Modulus[4] |E*| 10^3 psi (6)
Jan	47.5	56.4	52	60	1,270
Feb	50.7	60.1	55	64	1,060
Mar	58.0	68.0	63	72	700
Apr	66.1	76.0	71	81	420
May	73.3	83.2	78	90	250
Jun	80.5	90.4	85	97	160
Jul	83.1	92.9	88	100	130
Aug	82.7	92.8	88	100	130
Sep	77.3	87.4	82	94	190
Oct	67.2	78.1	73	83	380
Nov	56.2	66.4	61	71	720
Dec	49.3	58.3	54	61	1,200

[1] Determined from local climatological data for Shreveport, LA.
[2] Average of values from columns 2 and 3.
[3] Estimated from 5-inch bituminous concrete thickness curve in Figure 6-1. (Figure 6-1 is entered with the appropriate design air temperature.)
[4] Determined by laboratory testing of bituminous concrete.
Conversion Factors: degrees C = degrees F - 32/1.8, megapascals = 0.006894 × psi

thickness, base thickness, and subbase thickness. The section for a thickness of 760 millimeters (30 inches) is shown in Figure 11-13 as an example.

(2) For runway design, thicknesses of 510, 610, and 660 millimeters (20, 24, and 26 inches) are assumed for the initial sections for computing the subgrade damage factor. The section for 610 millimeters (24 inches) is shown in Figure 11-14. In the initial section, a 13-millimeter (5-inch) asphalt layer is assumed for the taxiway design, and a 10-millimeter (4-inch) asphalt layer is assumed for the runway design. After determining the total thickness required for these asphalt thicknesses, the design can be refined for other asphalt thicknesses.

c. Step 3 - Computation of Strains.

Table 11-4
Bituminous Concrete Moduli for Each Month for Conventional Flexible Pavement Design Based on Bituminous Concrete Strain

Month	Average Daily Mean Air Temperature,[1] degrees F	Design Pavement Temperature,[2] degrees F	Dynamic Modulus |E*| 10^3 psi
Jan	47.5	56	1,500
Feb	50.7	60	1,270
Mar	58.0	67	920
Apr	66.1	76	570
May	73.3	84	360
Jun	80.5	92	220
Jul	83.1	95	180
Aug	82.7	95	180
Sep	77.3	89	260
Oct	67.2	77	540
Nov	56.2	65	1,000
Dec	49.3	57	1,400

[1] Determined from local climatological data for Shreveport, LA.
[2] Estimated from 5-inch bituminous concrete thickness curve in Figure 6-1. (In design for bituminous concrete strain, the average daily mean air temperature is used as the design air temperature for entering Figure 6-1.)
Conversion Factors: degrees C = degrees F - 32/1.8, millimeters = 25.4 × inches

(1) The horizontal strain at the bottom of the asphalt layer and the vertical strain at the top of the subgrade are computed for each traffic grouping shown in Table 11-5. The data needed for input into the JULEA computer program for the computation of asphalt and subgrade strains for the 760-millimeter (30-inch) pavement structure for a taxiway design are given in Table 11-6. Note that the input contains data for one run, but four runs would be required to compute the subgrade data, i.e., one run for each grouping to account for variation in asphalt modulus. The strain is computed considering only two of the four main tires; the transverse spacing of the tires is sufficiently large to prevent an overlapping effect for the other two tires. The individual tire loading is computed by considering 90 percent of the gross load on the main gear, equally distributed between the four tires of the main gear, resulting in a weight on each tire of 15,820 kilograms (34,875 pounds). The radius of the loaded area is computed as a circle having an area equal to the tire contact area. A contact area of 0.258 square meters (400 square inches) results in a radius of the contact area of 28.6 millimeters (11.28 inches). The pavement system is a five-layer system having full friction between layers. For a

Table 11-5
Grouping Traffic into Traffic Groups According to Similar Asphalt Moduli

		Modulus Values, kips per square inch				
		For Computation of Asphalt Damage		For Computation of Subgrade Damage		
Group	Month	Monthly Values	Group Average	Monthly Values	Group Average	Percent of Total Traffic
1	Jan	1,500	1,390	1,270	1,180	25.0
	Dec	1,400		1,200		
	Feb	1,270		1,060		
2	Nov	1,000	960	720	710	16.7
	Mar	920		700		
3	Apr	570	490	420	400	25.0
	Oct	540		380		
	May	360		250		
4	Sep	260	210	190	150	33.3
	Jun	220		160		
	Jul	180		130		
	Aug	180		130		

Conversion Factors: megapascals = 6.894 × kips per square inch

flexible pavement system, the rough computational procedure is sufficiently accurate. The subgrade vertical strain is computed at the top of the subgrade layer and under the center of one of the tires and midway between the tires. The maximum strain is found to occur under the tire. Results of computer runs for the example problem are shown in Table 11-7.

(2) A similar set of runs is made for the computation of the horizontal strain at the bottom of the asphalt layer. This set of runs will use the asphalt moduli determined for consideration of asphalt strains and given in Table 11-5. For computing the strains for the runway design, the load and contact area are reduced by 75 percent. The resulting tire loading is 11,850 kilograms (26,125 pounds) applied over a circular contact area having a radius of 248 millimeters (9.77 inches).

d. Step 4 - Determination of Applied Repetitions.

(1) The design is for 200,000 passes of the aircraft over the life of the pavement. The pavement life has been divided into four periods as shown in Table 11-5. Considering that the traffic is to be equally distributed throughout the year would result in 25, 16.7, 25, and 33.3 percent of the traffic to be applied in the first, second, third, and fourth periods, respectively.

(2) The 200,000 passes will result in a total number of effective strain repetitions that will be a function of transverse location on the pavement and on the depth at which the strain is being considered.

Table 11-6
Structure Data File for Input into the JULEA Computer Program

STRUCTURE Data File
Job Title
TM EXAMPLE 1
Number of Pavements
1
Number Thickness and Moduli Variations
1 1
Pavement Description
Flexible Pavement
Slab Flexural Strength (only for rigid pavements)
.00000000

No. of Layers
6

Layer Number	Thicknesses (in.)	Modulus of Elasticity (psi)	Poisson's Ratio	Interface Condition	Layer Code
1	5.00	1,180,000.00	0.300	0.00	0
2	6.00	58,000.00	0.300	0.00	0
3	7.00	32,000.00	0.300	0.00	0
4	6.00	25,000.00	0.300	0.00	0
5	6.00	17,000.00	0.300	0.00	0
6		9,000.00	0.400		0

No. of Depths
2

Depth No.	Depth (in.)
1	-5.00000000
2	30.00000000

(Continued)

Table 11-6 (Concluded)

LOAD Data File
Job Title
TM EXAMPLE
No. of Aircraft
1
Aircraft Identification Number 1
C-130
Gross Load
39750.00
Fraction of Gross Load on the Gear to be analyzed
1.000

No. of Tires
2

Tire No.	Radius (in.)	Cont. Area (sq in.)	Cont. Press. (psi)	Tire Load (pounds)	X-Coord. (in.)	Y-Coord. (in.)
1	11.28	400.00	87.19	34,875.00	-30.00	.00
2	11.28	400.00	87.19	34,875.00	30.00	.00

No. of Evaluation Points (X, Y Sets)
5

Point No.	X-Coord. (in.)	Y-Coord. (in.)
1	0.00	0.00
2	7.50	0.00
3	15.00	0.00
4	22.50	0.00
5	30.00	0.00

Table 11-7
Results of Computer Runs for the Example Problem. (Horizontal Strain for the Asphalt and Vertical Strains for the Subgrade)

Traffic Group or Season	Percent Traffic	Strains, in./in.					
		33-in. Pavement, 4-in. AC		30-in. Pavement, 5-in. AC		27-in. Pavement, 6-in. AC	
		Asphalt	Subgrade	Asphalt	Subgrade	Asphalt	Subgrade
1	25.0	0.000217	0.000654	0.000218	0.000733	0.000200	0.000831
2	16.7	0.000228	0.000698	0.000234	0.000789	0.000227	0.000908
3	25.0	0.000247	0.000741	0.000267	0.000844	0.000270	0.000980
4	33.3	0.000219	0.000806	0.000263	0.000927	0.000295	0.001080

From plots in Appendix E showing the conversion from passes to strain repetitions for the taxiway and runway, the conversion percentages are determined. For the taxiway and depth to the top of subgrade of 760 millimeters (30 inches), the maximum conversion percentage for converting passes to effective strain repetitions from Figure E-10 is approximately 100. This maximum occurs at a distance of 26 meters (86 inches) from the centerline of the taxiway. Thus, the effective number of subgrade repetitions would be 200,000. For consideration of the asphalt strain at a depth of 130 millimeters (5 inches), the conversion percentage is approximately 50, resulting in 100,000 strain repetitions. From Figure E-9, the conversion percentages for the runway are 60 and 30 for consideration of subgrade strain and asphalt strain, respectively.

(3) The effective number of strain repetitions for a traffic group then is determined by multiplying the total strain repetitions by the factor of traffic occurring in a group.

e. Step 5 - Computation of Damage Factors.

(1) The damage factor for one traffic group is defined as n/N where n represents the effective strain repetitions for that group and N equals the allowable numbers of strain repetitions as computed from Equations 11-3 and 11-4. The damage factors for the different periods are summed to obtain the cumulative damage factor.

(2) The computations were performed by use of the computer programs SUBGRADE for the subgrade damage and ASPHALT for the asphalt damage. The data file, SDATA1, required by the program SUBGRADE for computing the subgrade damage factor for 840-, 760-, and 685-millimeter (33-, 30-, and 27-inch) pavements is given in Table 11-8 and the output is given in Table 11-9. The data file ADATA1 required by the program ASPHALT for computing the asphalt damage factor for the asphalt is given in Table 11-10 and for the output in Table 11-11.

(3) To speed the design procedure, the subgrade damage factor was computed for several pavement thicknesses, and the results for the taxiway pavement were plotted as shown in Figure 11-15. For pavements having an asphalt concrete thickness of 130 millimeters (5 inches), the subgrade damage factor was computed for thicknesses of 685, 760, and 840 millimeters (27, 30, and 33 inches). From the plot of damage factor versus pavement thickness, it is determined that the required thickness for the taxiway pavement would be 735 millimeters (29 inches). Using the 735-millimeter (29-inch) overall thickness as a constant thickness, the subgrade damage factor can be computed for varying thickness of asphalt concrete. Lines can then be constructed that will provide the total thickness for each asphalt concrete thickness. Alternate designs, rounded to the nearest inch, might be 152-millimeter (6-inch) asphalt concrete with 710 millimeters (28 inches) in total thickness or 102-millimeter (4-inch) asphalt concrete with 762 millimeters (30 inches) in total thickness. The relationship between pavement thickness and subgrade damage factors for the runway is given in Figure 11-16.

(4) For these designs, the asphalt damage must be computed. Plots of asphalt damage versus asphalt thickness for both the taxiway and runway are given in Figure 11-17. The asphalt damage factor would not control the asphalt thickness since the damage factor for this case is always less than 1.0. The minimum thickness of the asphalt layer would be dictated by the minimum thickness criterion in the basic manual.

Table 11-8
Data File for Computing Subgrade Damage for Pavement Thicknesses of 840, 760, and 685 millimeters (33, 30, and 27 inches)

```
List SDATA1
100     Taxiway design subgrade damage thickness = 33 inches
110     4 200000
120     .25  .16666667  .25  .333333
130     9000.  9000.  9000.  9000.
140     .000654  .000698  .000741  .00806
150     Taxiway design subgrade damage thickness = 30 inches
160     4  200000
170     .25  .1666667  .25  .333333
180     9000.  9000.  9000.  9000.
190     .000733,  .000789  .000844  .000927
200     Taxiway design subgrade damage thickness = 27 inches
210     4  200000
220     .25  .166667  .25  .33333
230     9000.  9000.  9000.  9000.
240     .000831  .000908  .000980  .001080
250     End of data
260     0  0
```

13. EXAMPLE DESIGN FOR ALL BITUMINOUS CONCRETE (ABC) PAVEMENT.

a. The thickness of the ABC pavement required for the taxiway design is estimated by considering the thickness of conventional pavement, i.e., 130 millimeters (5 inches) of asphaltic concrete and 610 millimeters (24 inches) of granular base and subbase. For this conventional pavement the effective thickness would be 865 millimeters (34 inches) which when converted to an ABC pavement would give an estimated thickness of 430 millimeters (17 inches) (computed by using the equivalence of 2 for bound materials). For computation of the fatigue damage and subgrade damage, monthly time periods are used as shown in Tables 11-12 and 11-13, respectively. Normally for ABC designs, the subgrade damage will be the controlling criteria and thus the thickness for satisfying the subgrade criteria is first determined. The subgrade strains are computed for six time periods so as to produce a plot as shown in Figure 11-18. From this plot, the subgrade strains for each time period are determined and are given in Tables 11-14 and 11-15. The data shown in Table 11-14 are input into the computer program SUBGRADE to compute the subgrade damage factor. It is noted that an equivalent thickness of 865 millimeters (34 inches) is used to determine the applied strain repetitions, resulting in the same number of strain repetitions as was used for the design of the conventional pavement. Damage factors were computed for pavement thicknesses of 405, 430, 480, and 535 millimeters (16, 17, 19, and 21 inches) from which the plot of damage factor versus pavement thickness (Figure 11-19) was

Table 11-9
Program Output for Subgrade Damage for Pavement Thicknesses of 840, 760, and 685 millimeters (33, 30, and 27 inches)

* Run SUBGRAD

00 Taxiway Design Subgrade Damage Thickness = 33 inches

Damage based on subgrade strain criteria and on 200,000 total strain repetitions

Sub Modulus	Subg Strain	Allow Reps	Applied Reps	Damage
9000.	0.000654	7523438.	5000.	0.665-02
9000.	0.000698	3758888.	3333.	0.895-02
9000.	0.000741	1981678.	50000.	0.258-01
9000.	0.000806	807168.	66667.	0.835-01

Total Damage = 0.1235E+00

50 Taxiway Design Subgrade Damage Thickness = 30 inches

Damage based on subgrade strain criteria and on 200,000 total strain repetitions

Sub Modulus	Subg Strain	Allow Reps	Applied Reps	Damage
9000.	0.000733	8885301.	50000.	0.825-01
9000.	0.000789	1018577.	33333.	0.338-01
9000.	0.000844	493454.	50000.	0.10E+00
9000.	0.000927	181174.	66667.	0.37E+00

Total Damage = 0.5256E+00

20 Taxiway Design Subgrade Damage Thickness = 27 inches

Damage based on subgrade strain criteria and on 200,000 total strain repetitions

Sub Modulus	Subg Strain	Allow Reps	Applied Reps	Damage
9000.	0.000831	582449.	50000.	0.86E-01
9000.	0.000908	231418.	33333.	0.14E+00
9000.	0.000980	100039.	50000.	0.50E+00
9000.	0.001080	32112.	66666.	0.21E+00

Total Damage = 0.281E+01

Table 11-10
Data File for Computing Asphalt Damage

```
List ADATA1
    100     Taxiway Design; A5.  Thickness = 4 inches
    110     4 100000
    120     .25  .16667  .25  .33333
    130     1390000.  960000.  490000.  210000.
    140     .000217  .000228  .000247  .000219
    150     Taxiway Design; A5.  Thickness = 5 inches
    160     4  100000
    170     .25  .16667  .25  .33333
    180     1390000.  960000.  490000.  210000.
    190     .000218  .000234  .000267  .000263
    200     Taxiway Design; A5.  Thickness = 6 inches
    210     4  100000
    220     .25  .166667  .25  .3333
    230     1390000.  960000.  490000.  210000.
    240     .000200  .000227  .000270  .000295
    250     End of data
    260     0  0
```

developed. From Figure 11-19, the taxiway thickness for a damage factor of 1.0 is determined to be 430 millimeters (16.9 inches). The fatigue damage factor based on the asphalt criteria is then computed for a pavement thickness of 430 millimeters (16.9 inches). Also, from Figure 11-19 the runway thickness for a damage factor of 1.0 is determined to be 345 millimeters (13.6 inches).

b. The plot of asphalt strain versus asphalt modulus is shown in Figure 11-20. The asphalt strain for each time period is given in Table 11-14. Using the computer program ASPHALT, the fatigue damage factor is computed to be 0.15, which is considerably less than 1.0. Thus, a pavement thickness of 430 millimeters (16.9 inches) meets both the subgrade criteria and the asphalt fatigue criteria.

c. The runway design is accomplished in the same manner as the taxiway design. The conventional runway section of 102 millimeters (4 inches) of asphaltic concrete and 635 millimeters (25 inches) of granular base and subbase converted to a 840-millimeter (33-inch) effective thickness. An ABC pavement of 370 millimeters (14.5 inches) would be required to give the same effective thickness. Based on the estimated thickness, the subgrade damage factor was computed for pavement thicknesses of 330, 355, and 405 millimeters (13, 14, and 16 inches). The aircraft wheel load and the number of load repetitions for the computations were the same as used in the design for the conventional section. The subgrade strains and the asphalt strains as a function of pavement thickness are given in Figures 11-21 and 11-22, respectively. The data for computing the damage factors are

Table 11-11
Program Output for Asphalt Damage

* Run ASPHALT

00 Taxiway Design; A5. Thickness = 4 inches

Damage based on asphalt strain criteria and on 100,000 total strain repetitions

Asp Modulus	Asph Strain	Allow Reps	Applied Reps	Damage
1390000.	0.000217	42328.	25000.	0.59E+00
960000.	0.000228	88688.	16687.	0.19E+00
490000.	0.000247	356568.	25000.	0.70E-01
210000.	0.000219	6333934.	33333.	0.54E-03

Total Damage = 0.854E+00

50 Taxiway Design; A5. Thickness = 5 inches

Damage based on asphalt strain criteria and on 100,000 total strain repetitions

Asp Modulus	Asph Strain	Allow Reps	Applied Reps	Damage
1390000.	0.000218	47554.	25000.	0.52E+00
960000.	0.000234	77834.	16667.	0.21E+00
490000.	0.000267	241586.	25000.	0.10E+00
210000.	0.000263	2491763.	33333.	0.13E-01

Total Damage = 0.857E+00

20 Taxiway Design; A5. Thickness = 6 inches

Damage based on asphalt strain criteria and on 100,000 total strain repetitions

Asp Modulus	Asph Strain	Allow Reps	Applied Reps	Damage
1390000.	0.000200	63638.	25000.	0.39E+00
960000.	0.000227	90598.	16667.	0.18E+00
490000.	0.000270	228459.	85000.	0.11E+00
210000.	0.000295	1403381.	33333.	0.84E-00

Total Damage = 0.710E+00

Table 11-12
Bituminous Concrete Moduli for Each Month for ABC Pavement Design Based on Bituminous Concrete Strain

Month	Average Daily Mean Air Temperature degrees F	Design Pavement Temperature degrees F	Dynamic Modulus $\lvert E^* \rvert$ 10^3 psi
Jan	47.5	54	1,600
Feb	50.7	57	1,400
Mar	58.0	64	1,060
Apr	66.1	72	700
May	73.3	80	460
Jun	80.5	88	280
Jul	83.1	91	230
Aug	82.7	91	230
Sep	77.3	85	340
Oct	67.2	73	670
Nov	56.2	61	1,200
Dec	49.3	56	1,500

Conversion Factors: Degrees C = degrees F - 32/1.8, Megapascals = 0.006894 × PSI

given in Table 11-15. The plot of the subgrade damage factor versus thickness is given in Figure 11-20. From the plot, it is determined that a 345-millimeter (13.5-inch) ABC pavement would satisfy the subgrade criteria. The asphalt fatigue damage factor for a 330-millimeter (13-inch) pavement was computed to be 0.24, thus determining that the 345-millimeter (13.5-inch) pavement satisfies both criteria.

Table 11-13
Bituminous Concrete Moduli for Each Month for ABC Pavement Design Based on Subgrade Strain

Month	Average Daily Mean Air Temperature, degrees F	Average Daily Maximum Air Temperature, degrees F	Design Air Temperature, degrees F	Design Pavement Temperature, degrees F	Dynamic Modulus |E*| 10^3 psi
Jan	47.5	56.4	52	57	1,400
Feb	50.7	60.1	55	62	1,150
Mar	58.0	68.0	63	70	790
Apr	66.1	76.0	71	77	540
May	73.3	83.2	78	86	320
Jun	80.5	90.4	85	95	180
Jul	83.1	92.9	88	97	160
Aug	82.1	92.8	88	97	160
Sep	77.3	87.4	82	91	230
Oct	67.2	78.1	73	82	400
Nov	56.2	66.4	61	69	830
Dec	49.3	58.3	54	61	1,200

Conversion Factors: degrees C = degrees F - 32/1.8, megapascals = 0.006894 × psi

Table 11-14
Data for Computing Damage Factors for Taxiway Design

Month	Strain Repetitions	Subgrade Modulus, psi	Subgrade Strain, inches/inch × 10^{-5}				Asphalt Modulus kips per square inch	Asphalt Strain inches/inch × 10^{-5}
			t = 16 inches	t = 17 inches	t = 19 inches	t = 21 inches		
Jan	16,666	9,000	35	31	26	23	1,600	87
Feb	16,666	9,000	40	35	30	26	1,400	96
Mar	16,666	9,000	50	44	37	32	1,060	118
Apr	16,666	9,000	64	56	47	41	700	161
May	16,666	9,000	78	68	59	50	460	194
Jun	16,666	9,000	112	94	82	74	280	275
Jul	16,666	9,000	120	104	89	78	230	315
Aug	16,666	9,000	120	104	89	78	230	315
Sep	16,666	9,000	96	84	72	62	340	240
Oct	16,666	9,000	68	60	53	48	670	167
Nov	16,666	9,000	49	43	37	31	1,200	108
Dec	16,666	9,000	39	34	29	25	1,500	91

Conversion Factors:
Megapascals = 0.006894 × psi
Megapascals = 6.894 × kips per square inch
Millimeters = 25.4 × inches

Table 11-15
Data for Computing Damage Factors for Runway Design

Month	Strain Repetitions	Subgrade Modulus	Subgrade Strain, inches/inch × 10^{-5}			Asphalt Modulus, kips psi	Asphalt Strain, inches/inch × 10^{-5}	
			t = 13 inches	t = 14 inches	t = 16 inches		t = 13 inches	t = 14 inches
Jan	10,000	9,000	36	32	26	1,600	99	91
Feb	10,000	9,000	40	36	30	1,400	110	100
Mar	10,000	9,000	51	46	38	1,060	134	123
Apr	10,000	9,000	64	58	48	700	186	167
May	10,000	9,000	79	71	59	460	220	200
Jun	10,000	9,000	113	100	84	280	310	285
Jul	10,000	9,000	121	106	91	230	357	325
Aug	10,000	9,000	121	106	91	230	357	325
Sep	10,000	9,000	97	86	73	340	372	248
Oct	10,000	9,000	69	62	51	670	190	172
Nov	10,000	9,000	49	45	37	1,200	122	112
Dec	10,000	9,000	39	35	29	1,500	105	95

Conversion Factors:
Megapascals = 0.006894 × psi
Millimeters = 25.4 × inches

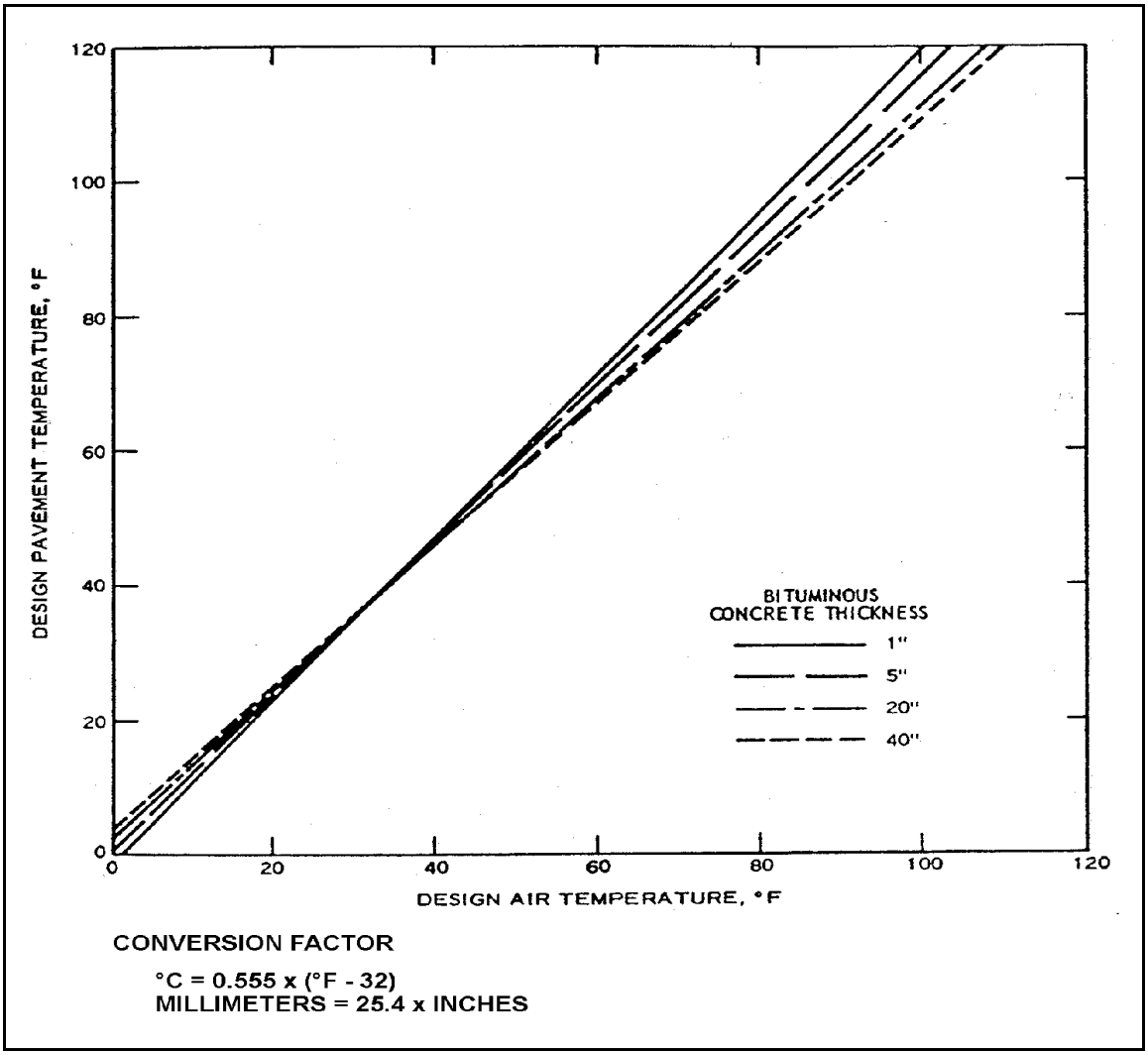


Figure 11-1. Temperature relationships for selected bituminous concrete thickness

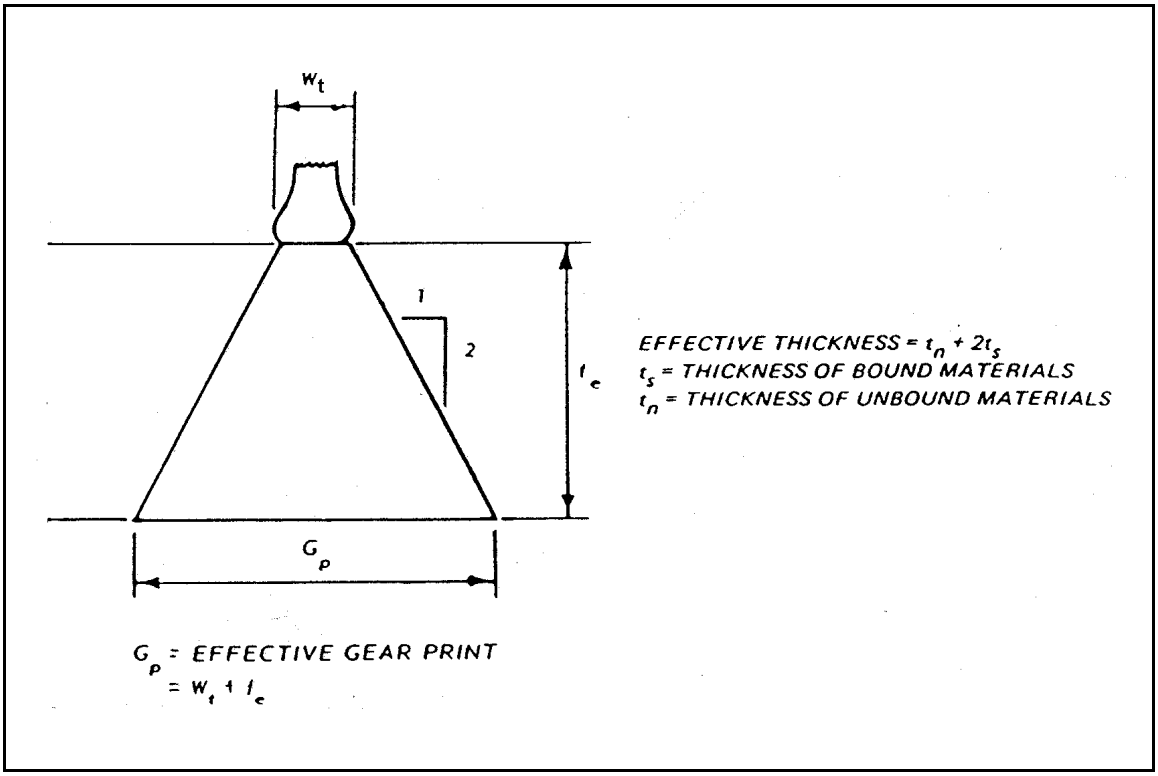


Figure 11-2. Computation of effective gear print for single gear

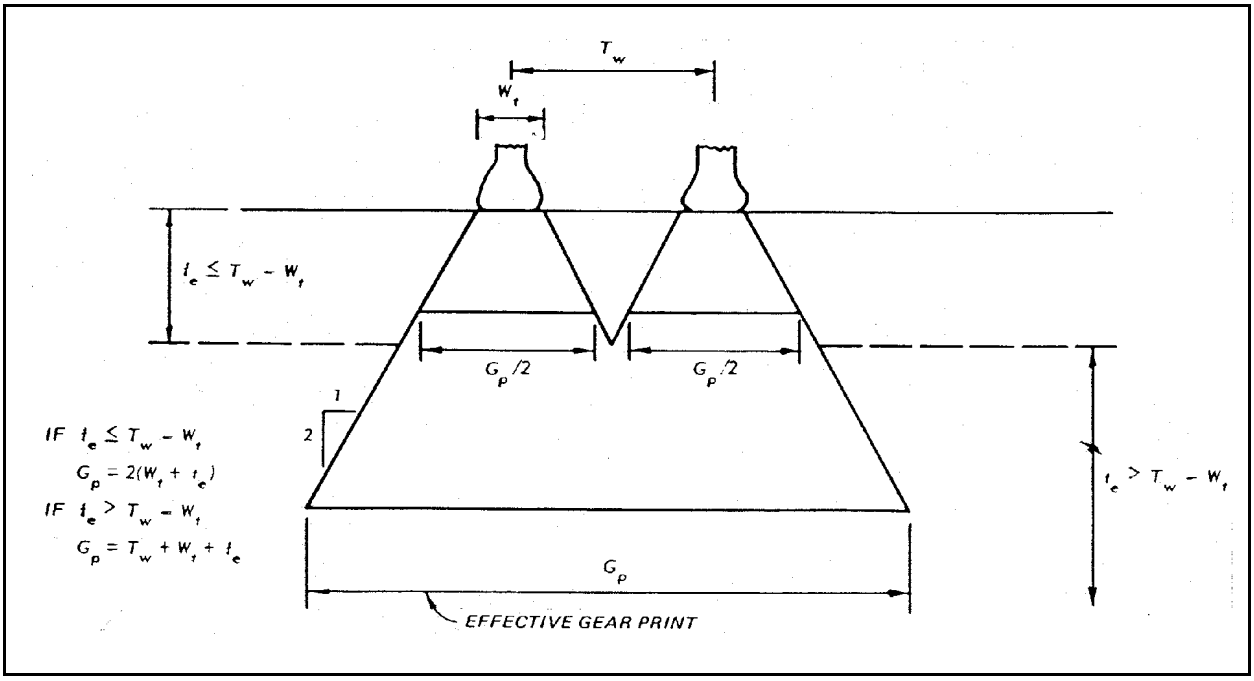


Figure 11-3. Computation of effective gear print for twin gear

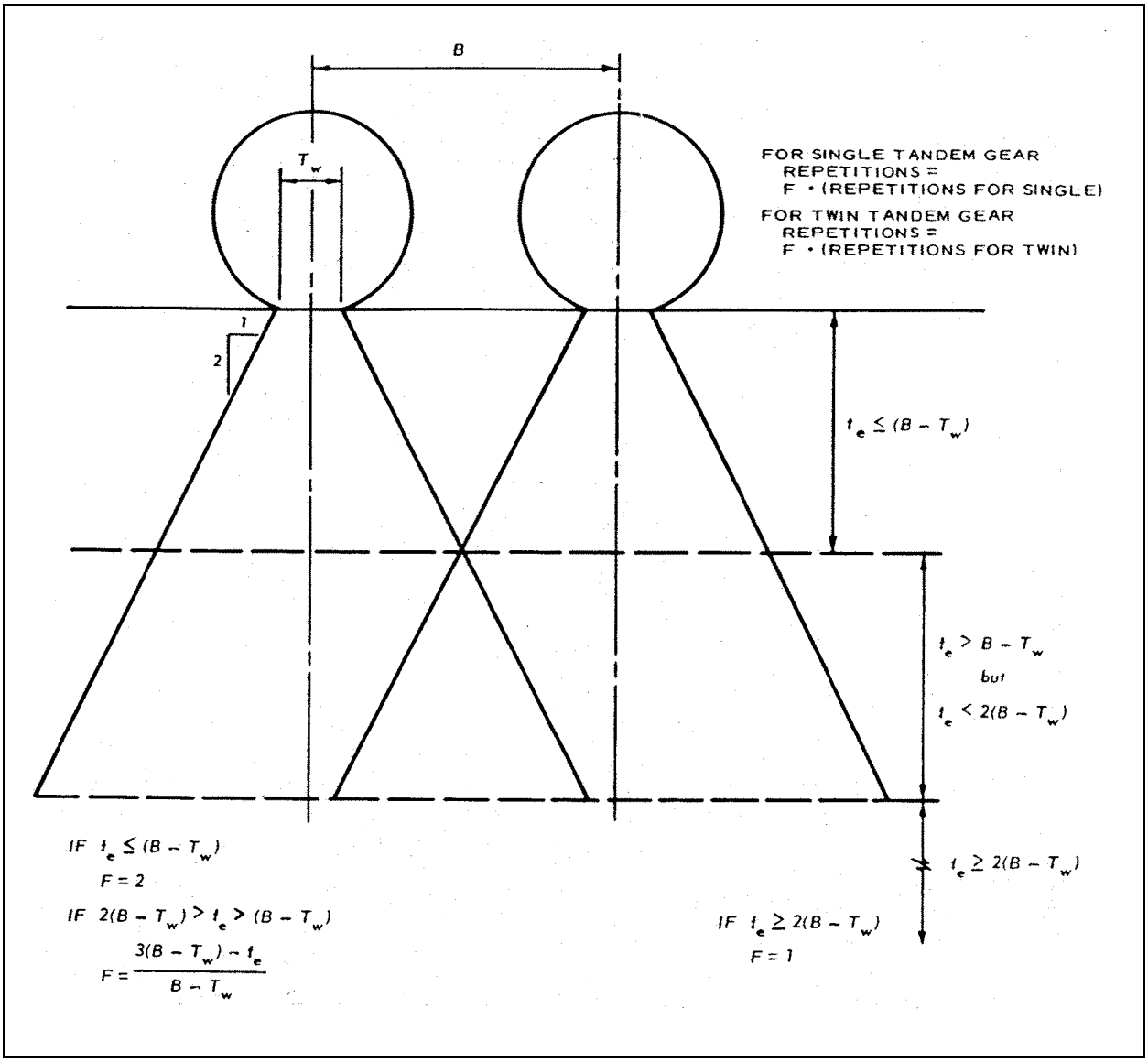


Figure 11-4. Computation of repetition factor for tandem gear

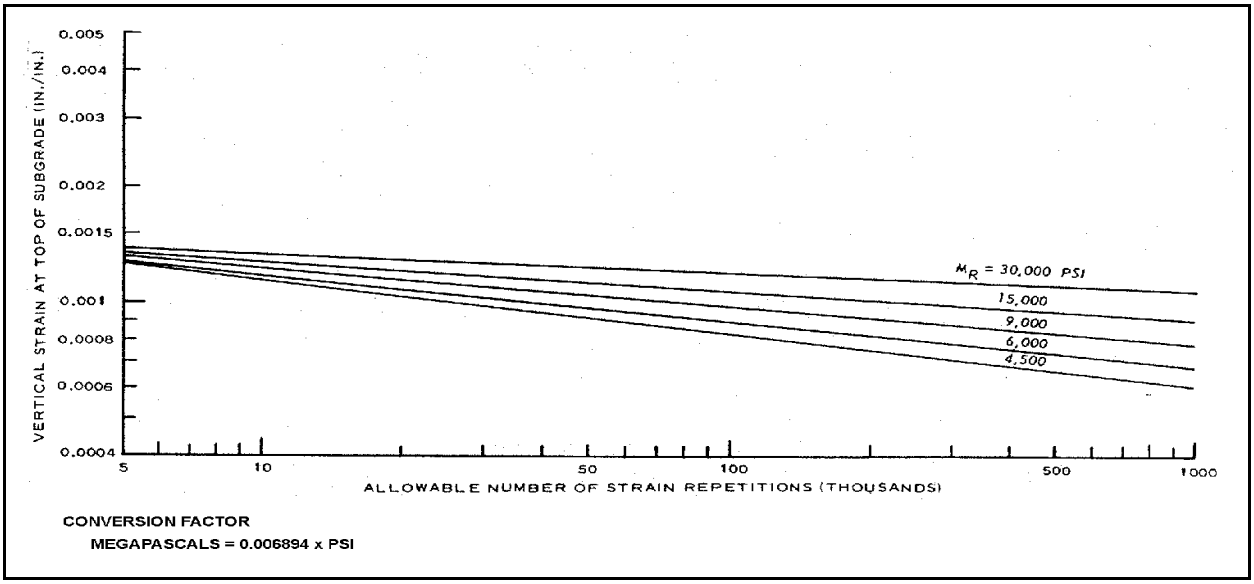


Figure 11-5. Design criteria based on subgrade strain

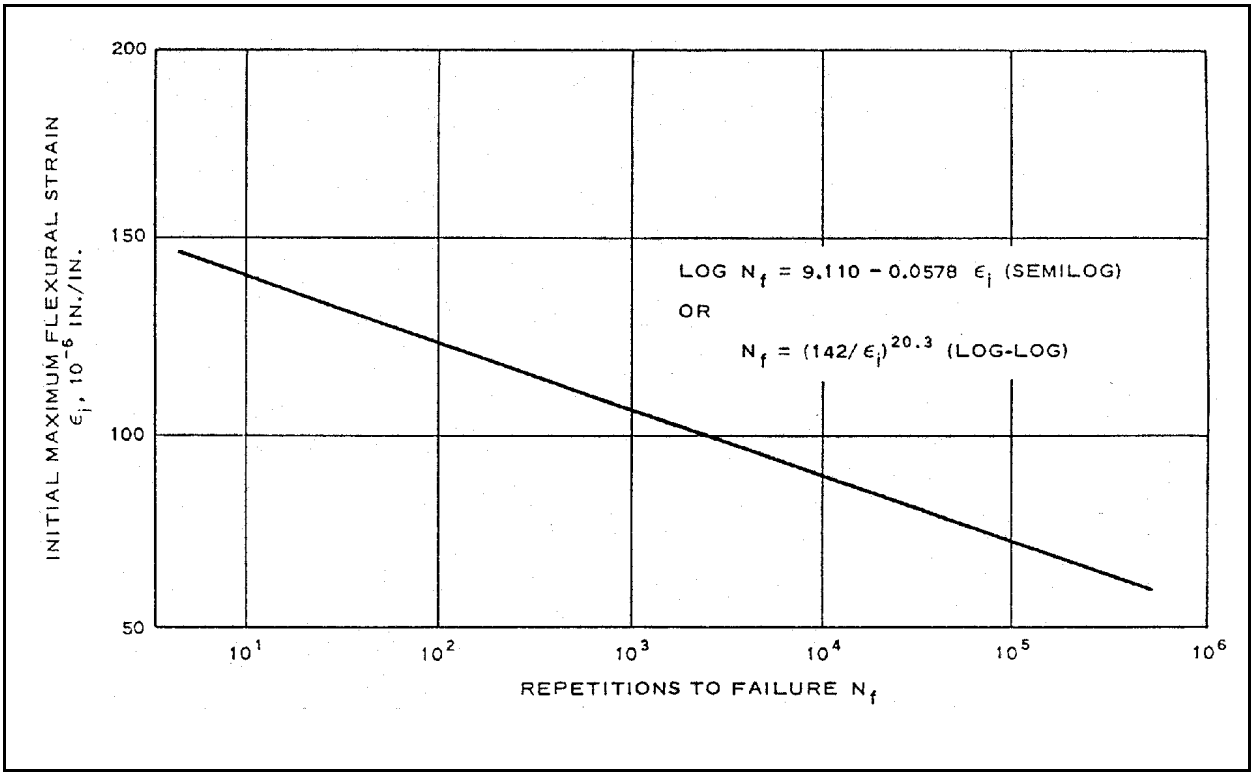


Figure 11-6. Fatigue life of flexural specimens

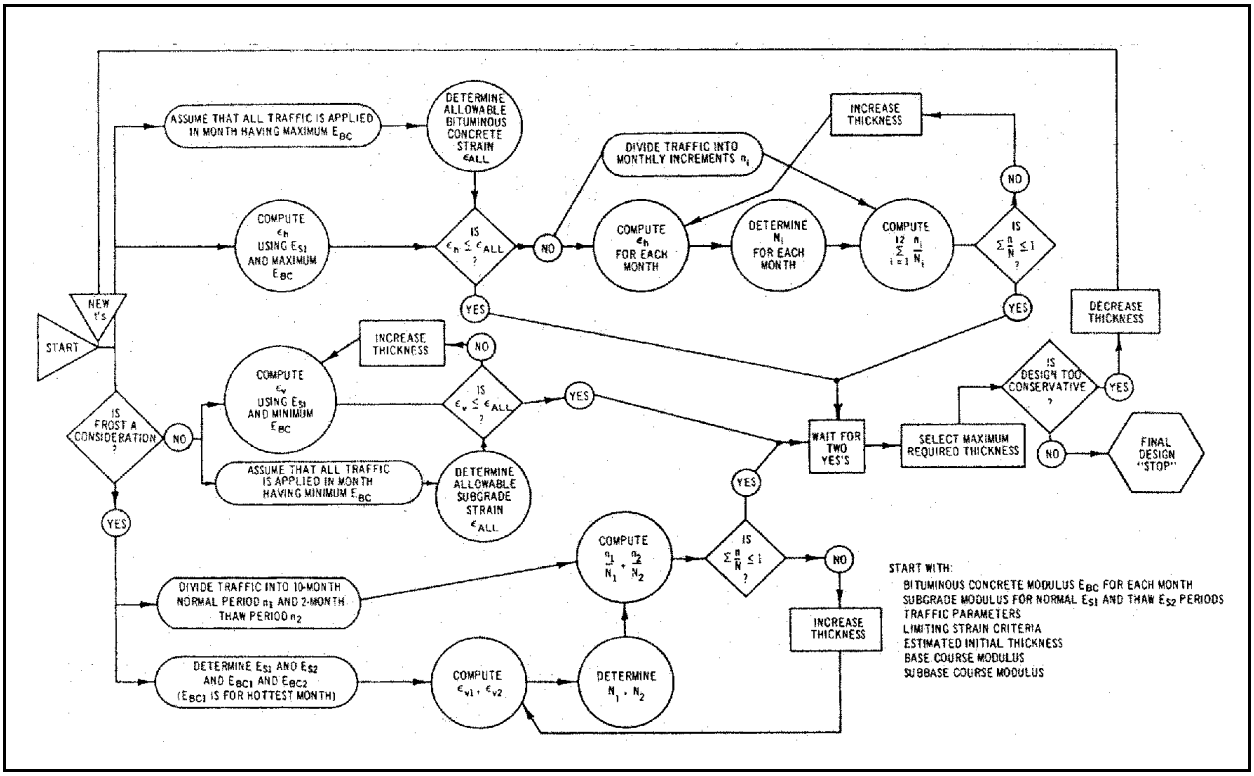


Figure 11-7. Flow diagram of important steps in design of bituminous concrete pavement

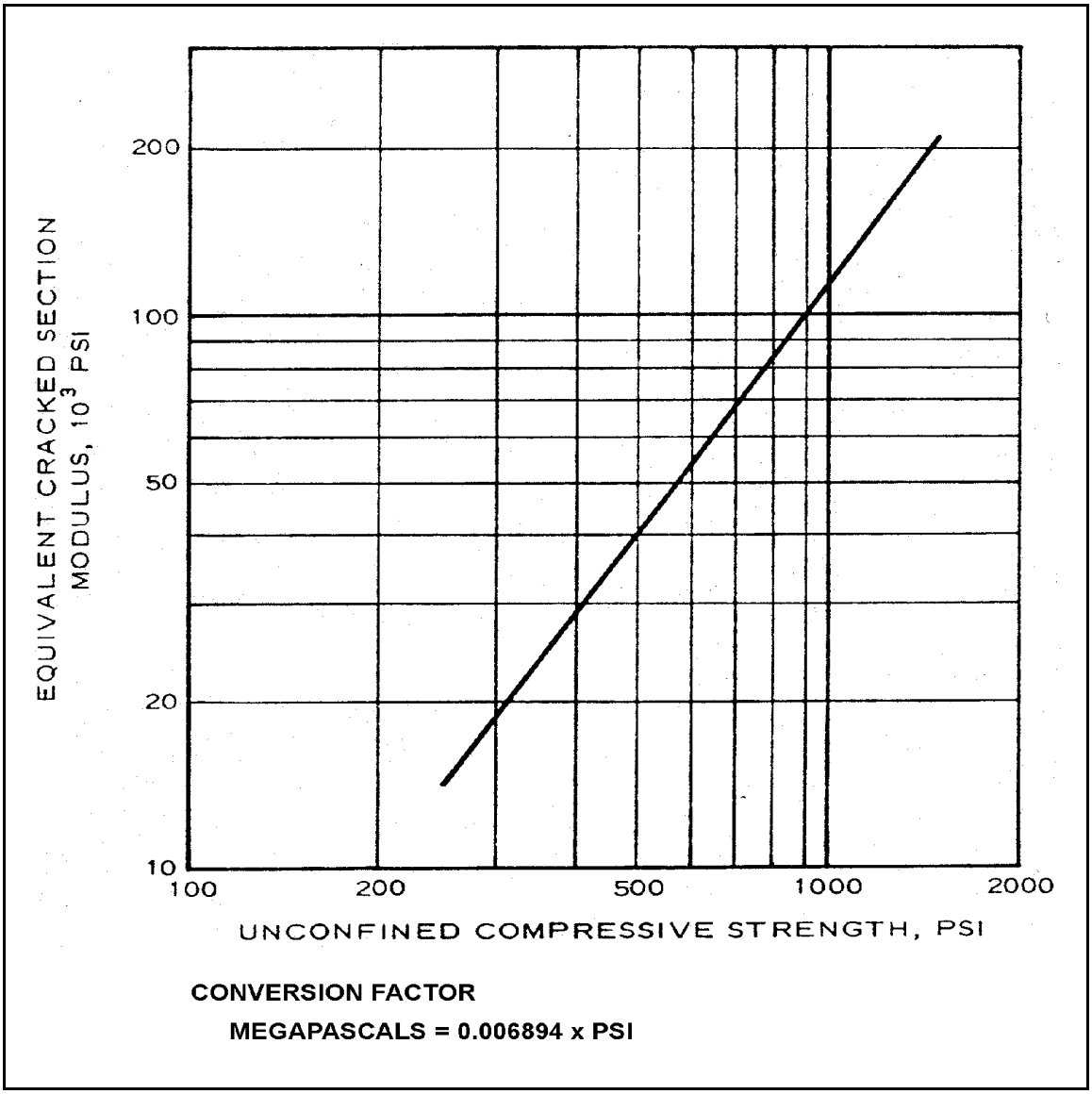


Figure 11-8. Relationship between cracked section modulus and unconfined compressive strength

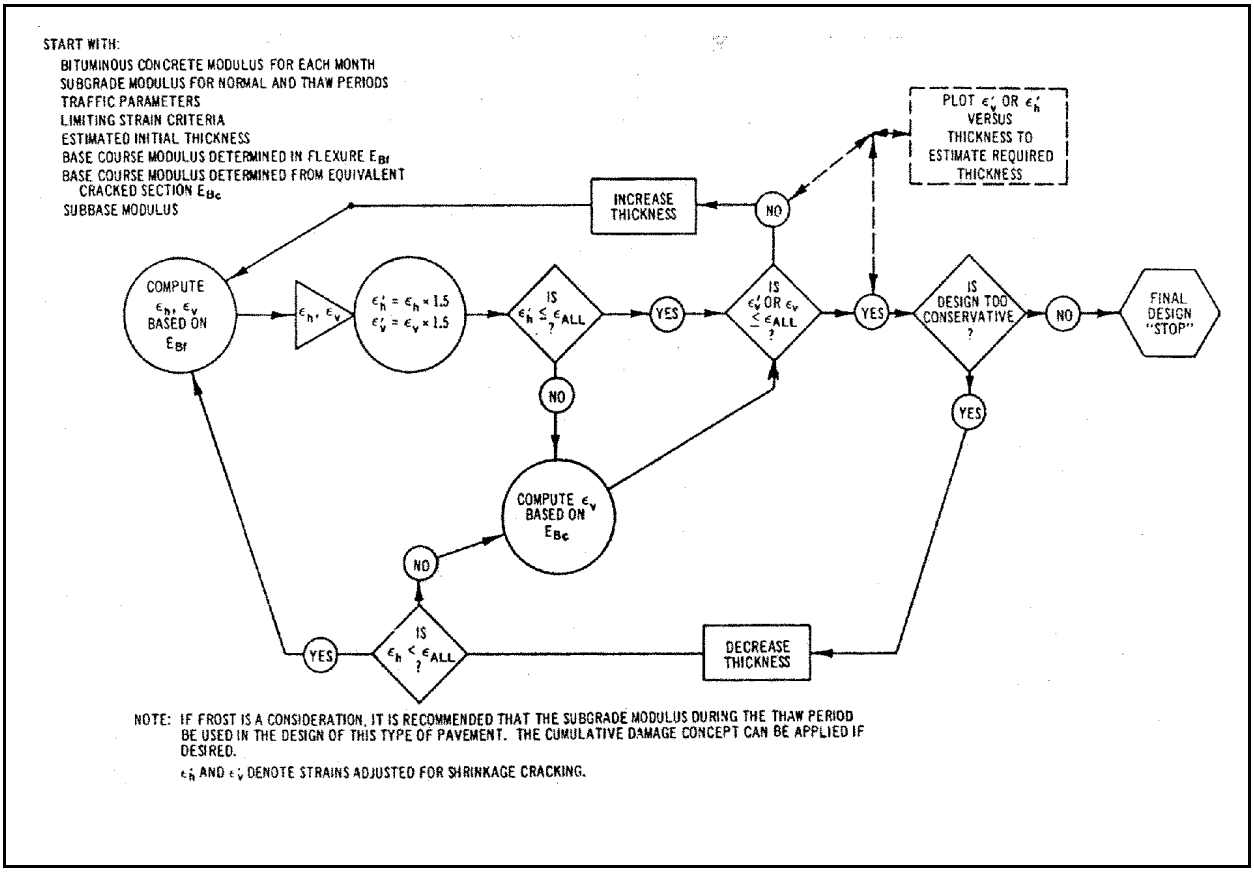


Figure 11-9. Flow diagram of important steps in design of pavements having chemically stabilized base course and unstabilized subbase course

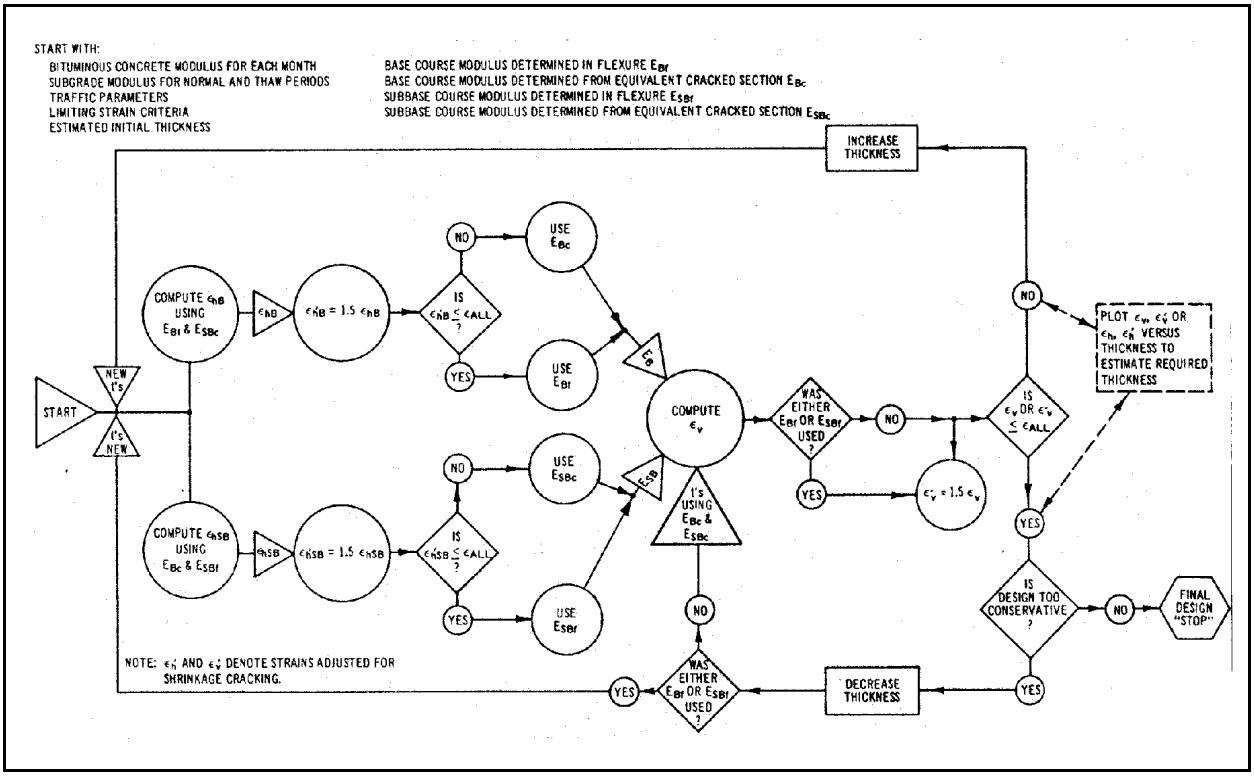


Figure 11-10. Flow diagram of important steps in design of pavements having stabilized base and chemically stabilized subbase courses

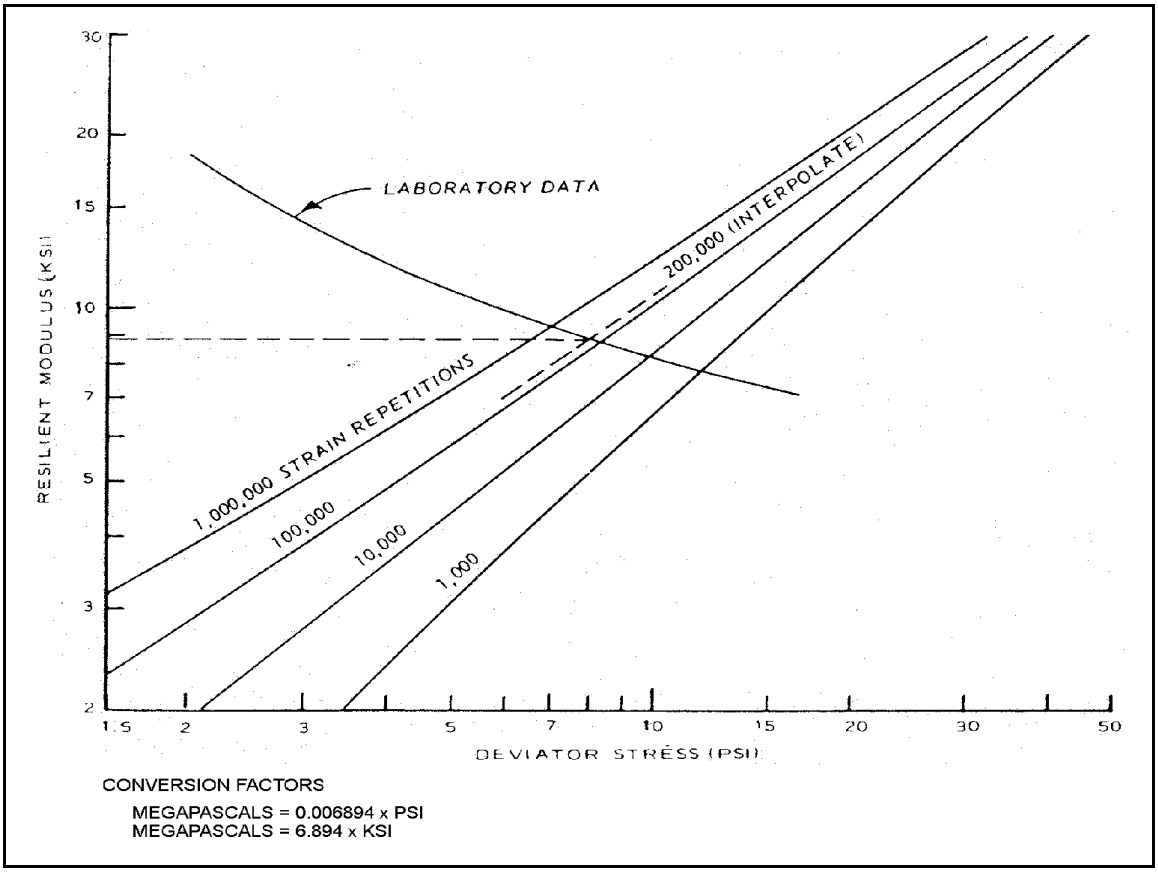


Figure 11-11. Estimation of resilient modulus M_R

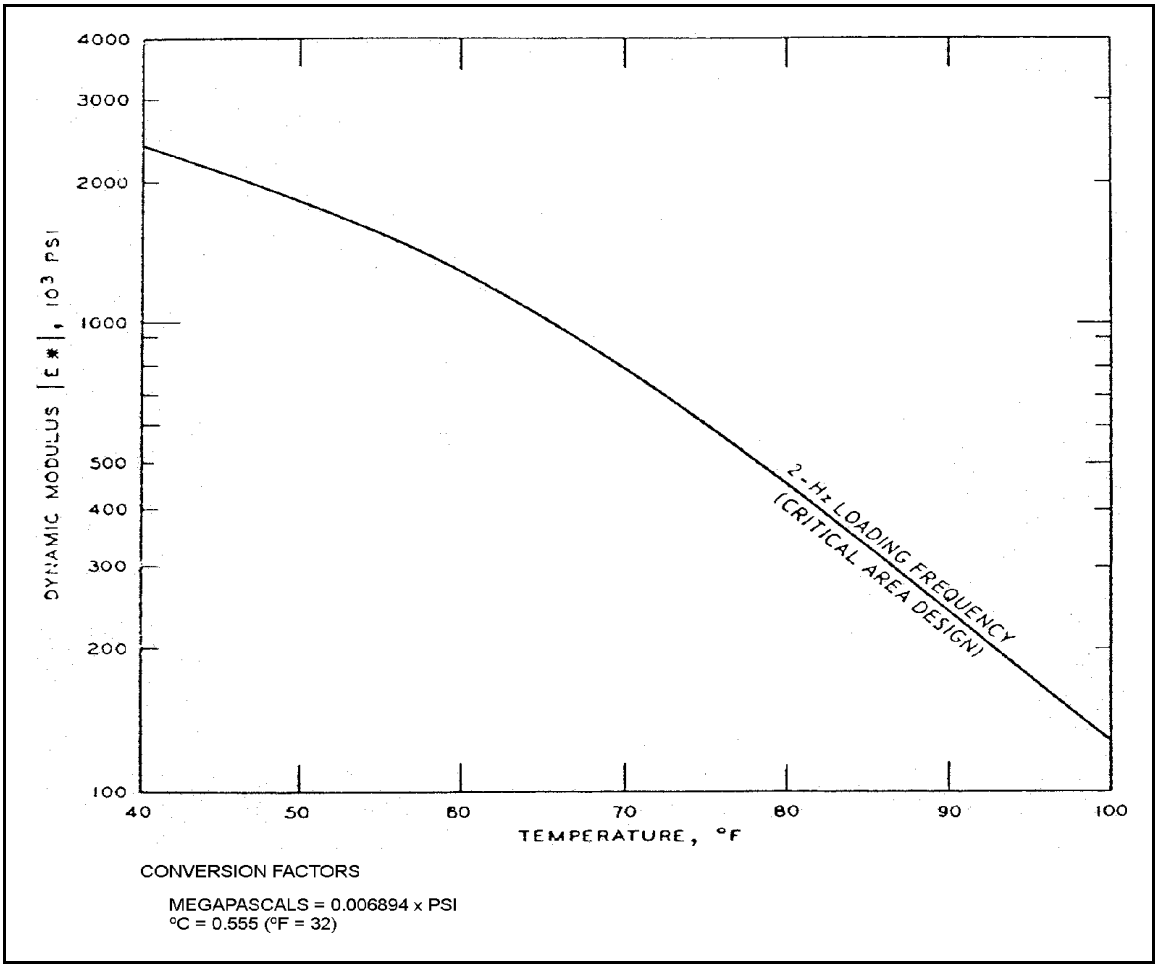


Figure 11-12. Results of laboratory tests for dynamic modulus of bituminous concrete

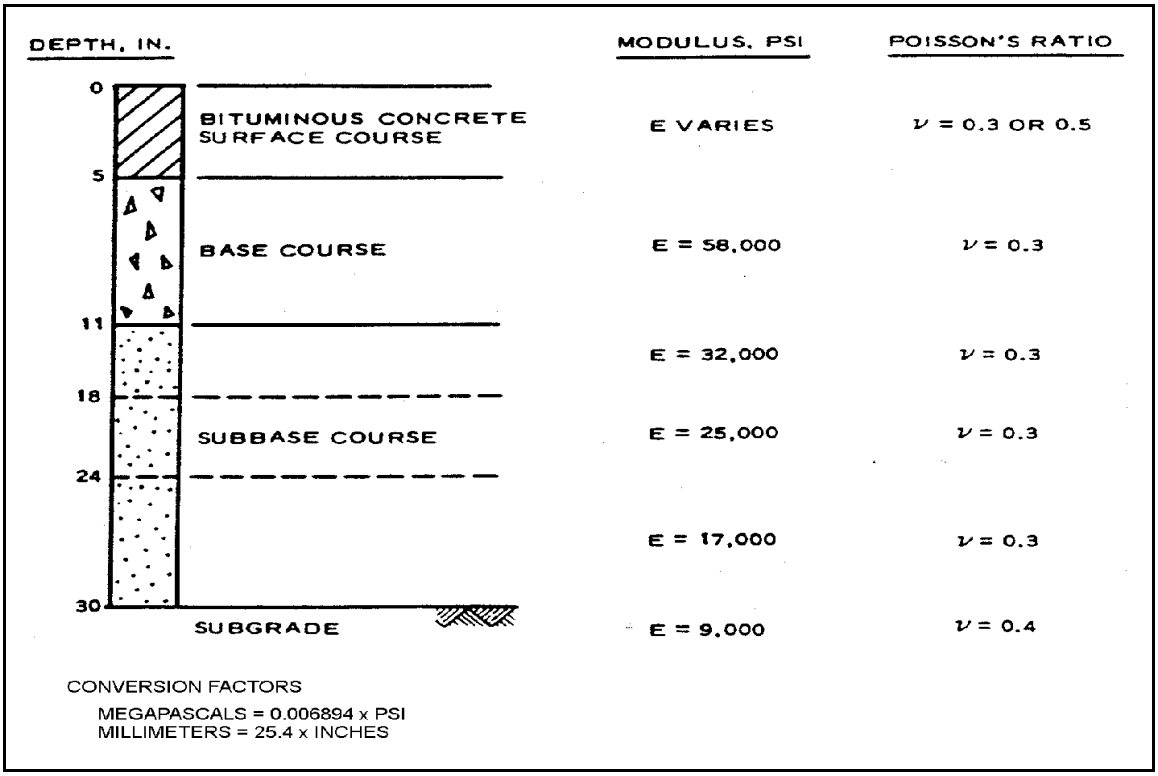


Figure 11-13. Section for pavement thickness of 760 millimeters (30 inches) for initial taxiway design

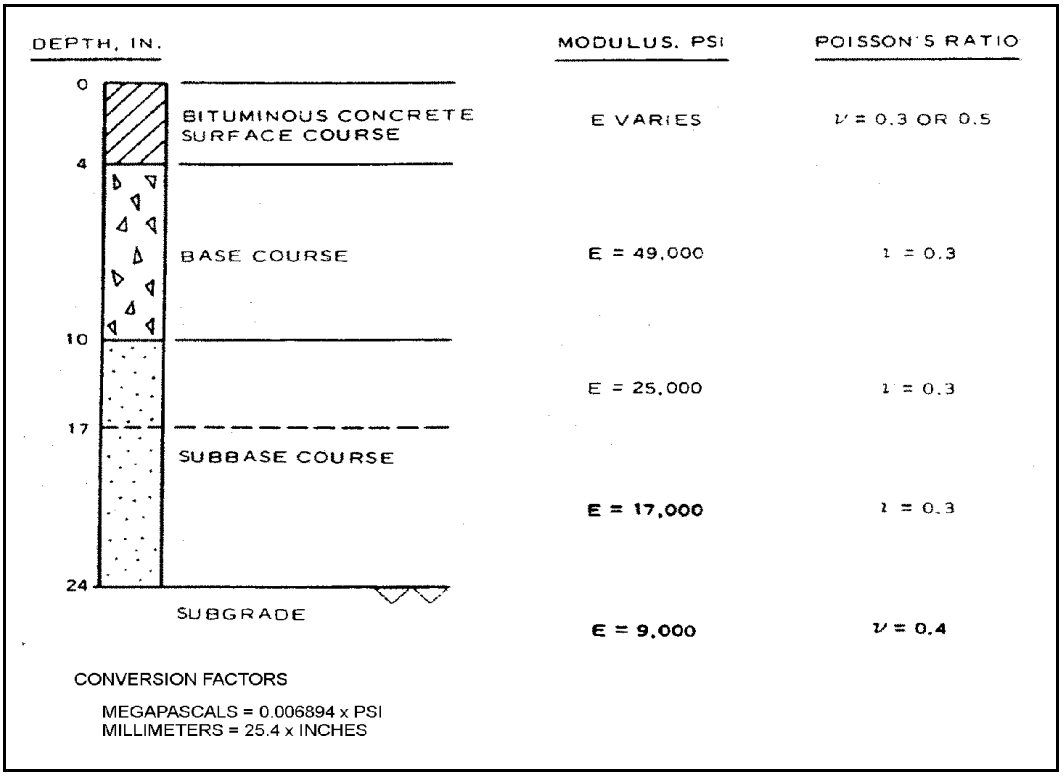


Figure 11-14. Section for pavement thickness of 610 millimeters (24 inches) for initial runway design

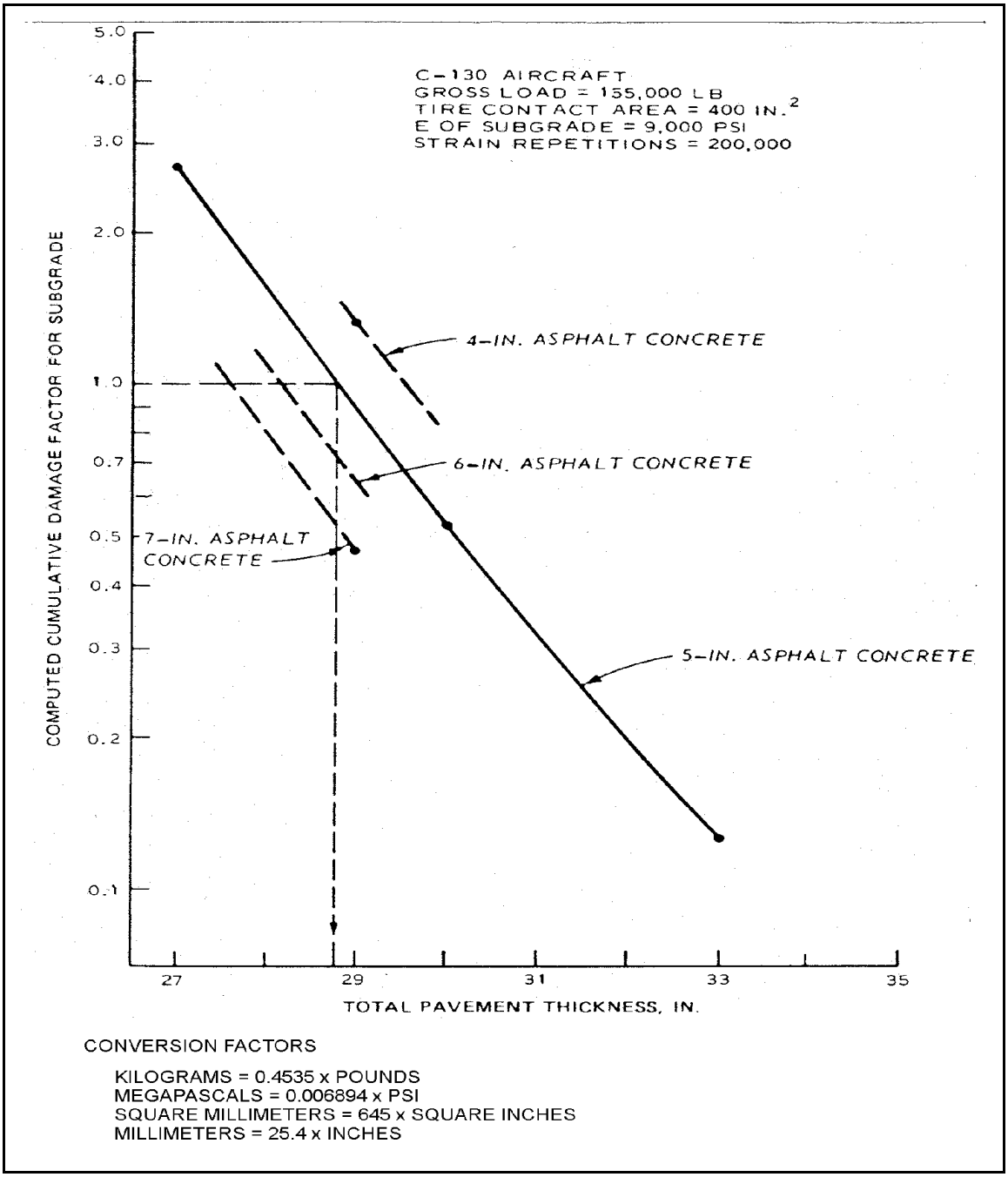


Figure 11-15. Pavement design for taxiways

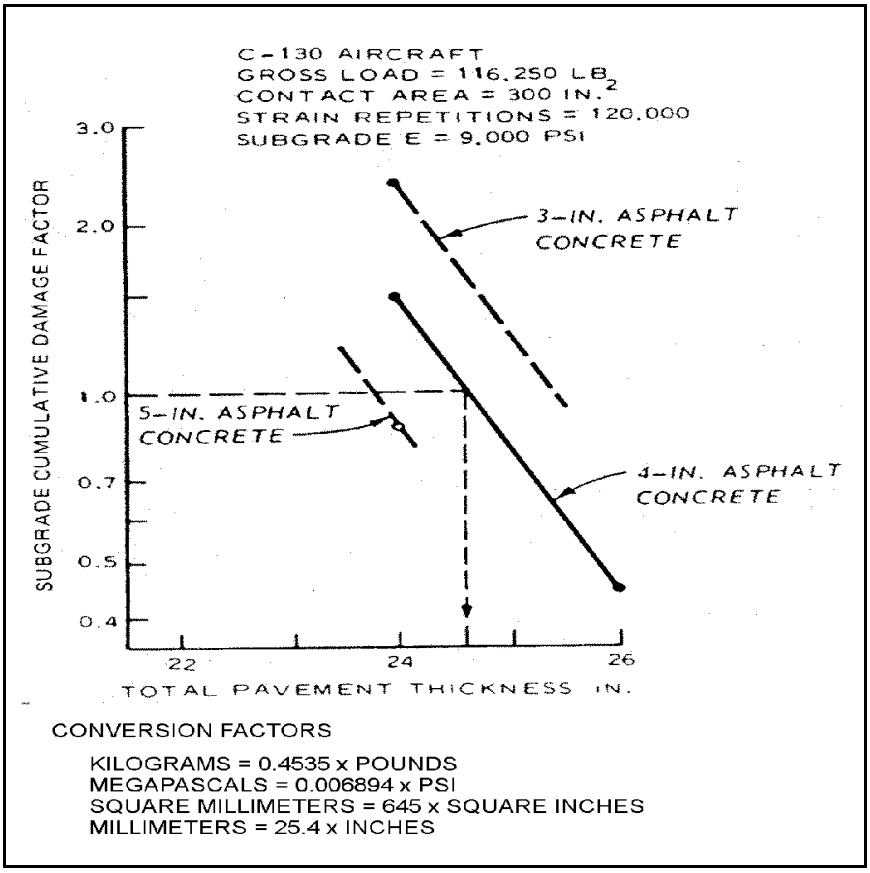
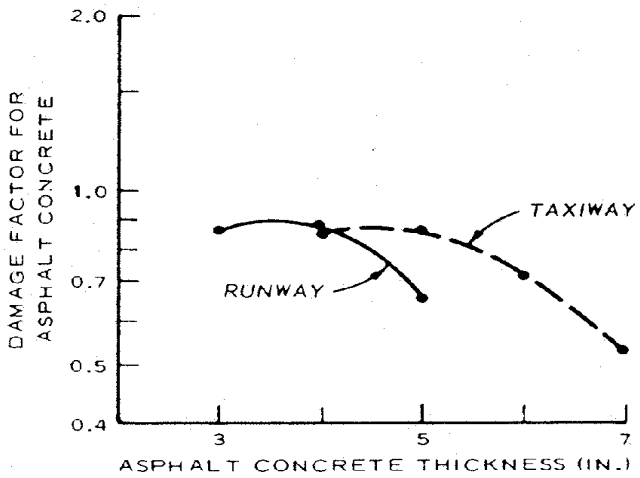


Figure 11-16. Design for runways

C-130 AIRCRAFT

	TAXIWAY	RUNWAY
GROSS LOAD, LB	155,000	116,250
CONTACT AREA, IN.^{2}	400	300
STRAIN REPETITIONS	100,000	60,000



CONVERSION FACTORS

KILOGRAMS = 0.4535 x POUNDS
SQUARE MILLIMETERS = 645 x SQUARE INCHES
MILLIMETERS = 25.4 x INCHES

Figure 11-17. Design for asphalt concrete surface

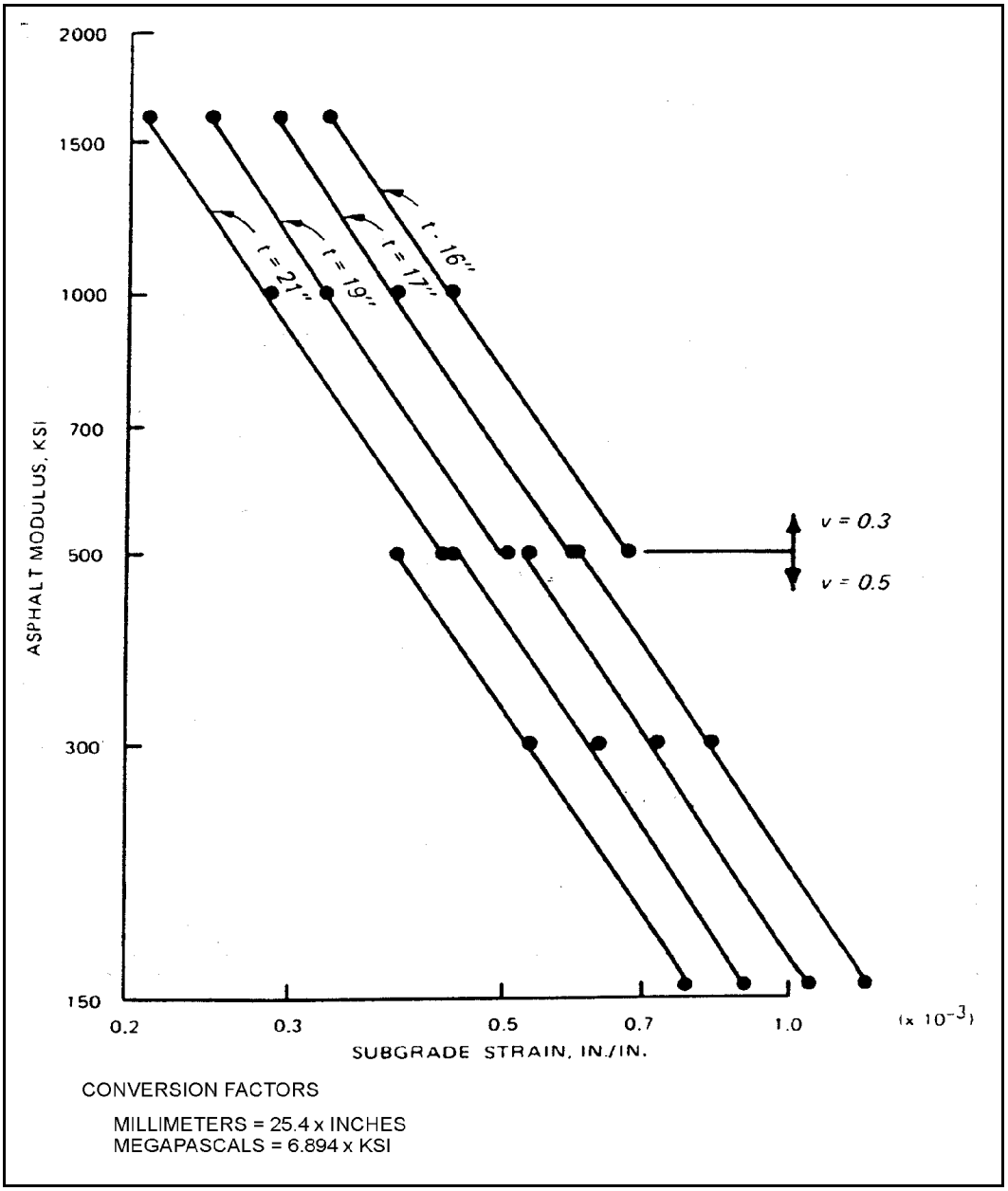


Figure 11-18. Computed strain at the top of the subgrade for taxiway design

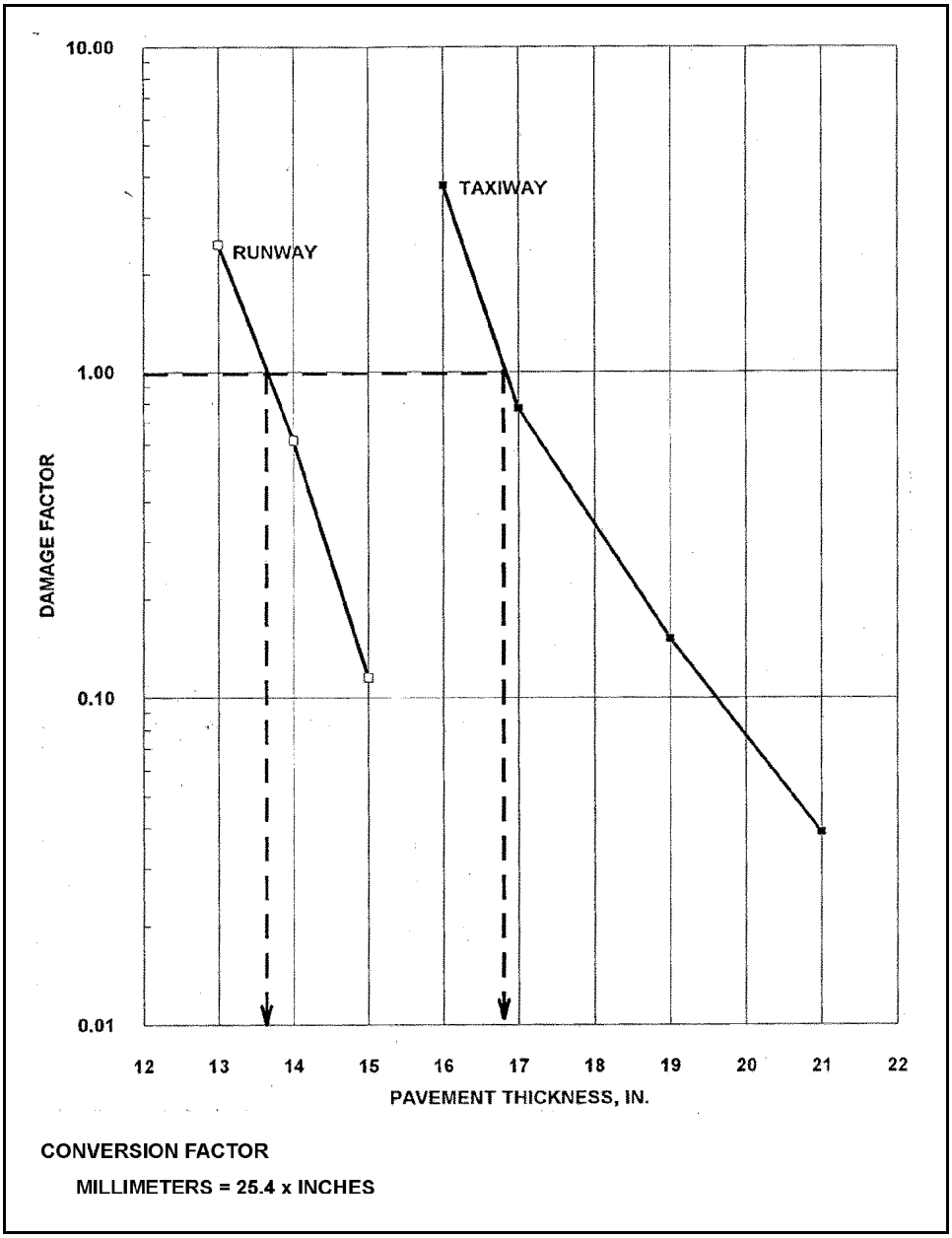


Figure 11-19. Damage factor versus pavement thickness

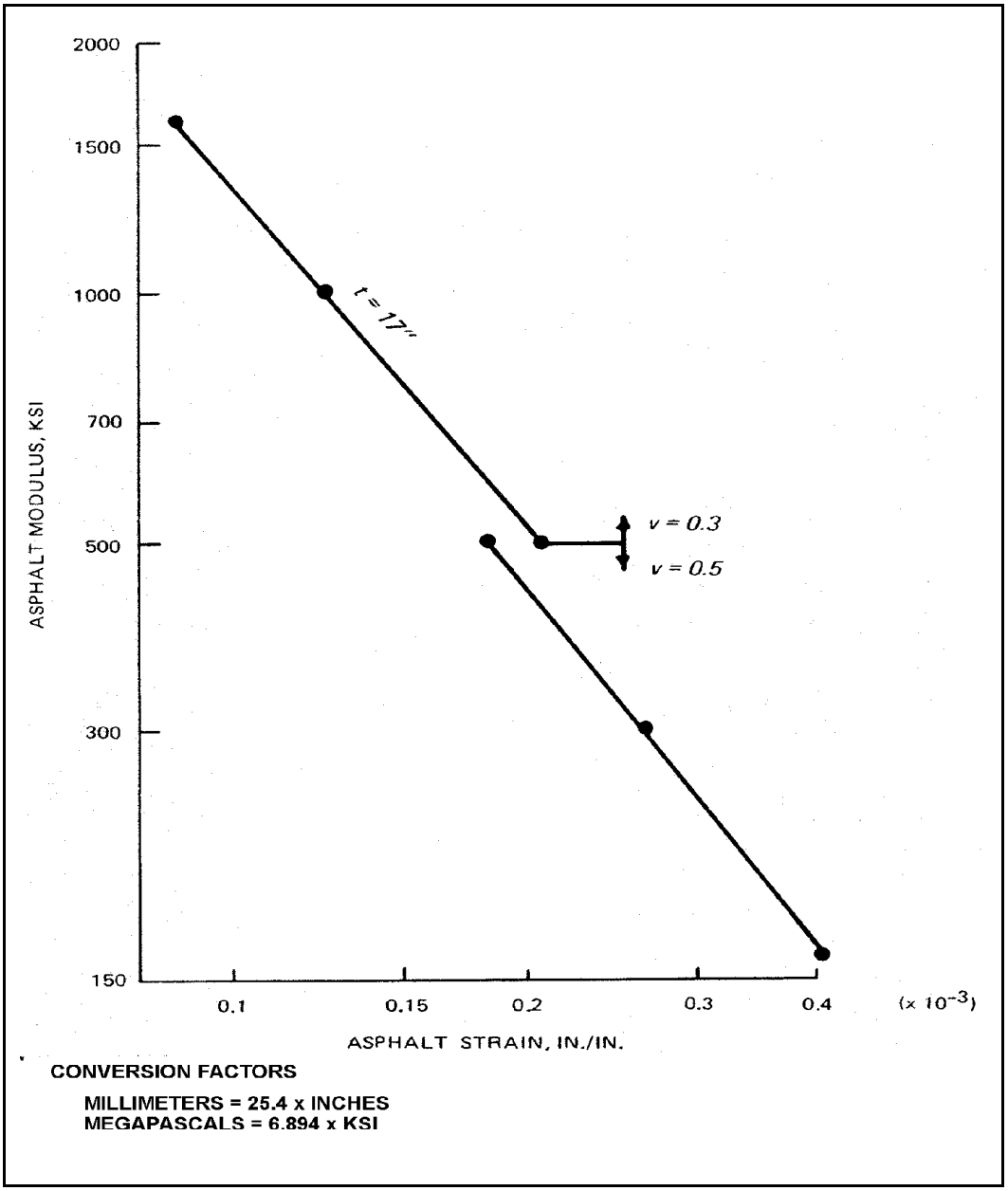


Figure 11-20. Computed strain at the bottom of the asphalt for taxiway design

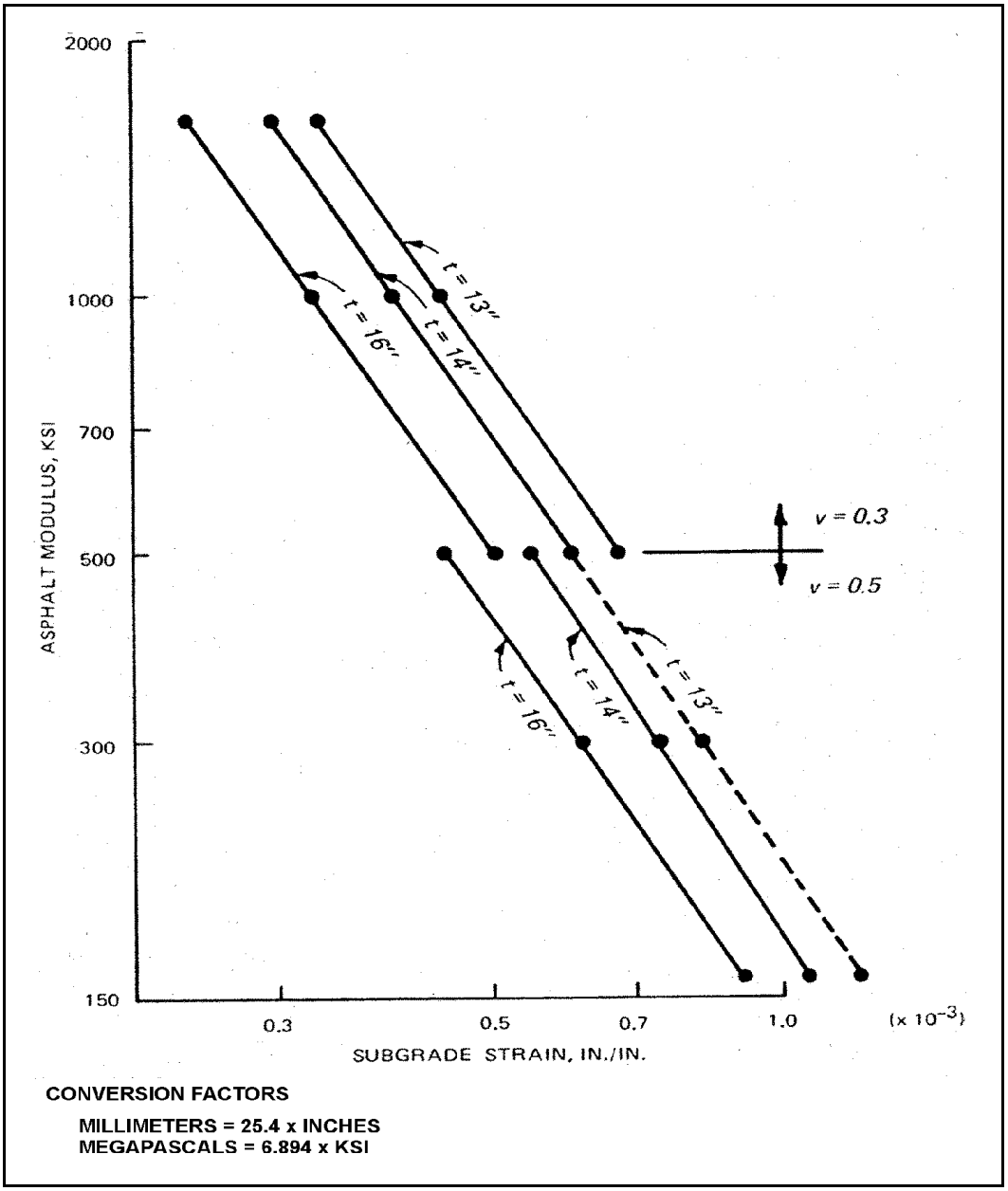


Figure 11-21. Computed strain at the top of the subgrade for runway design

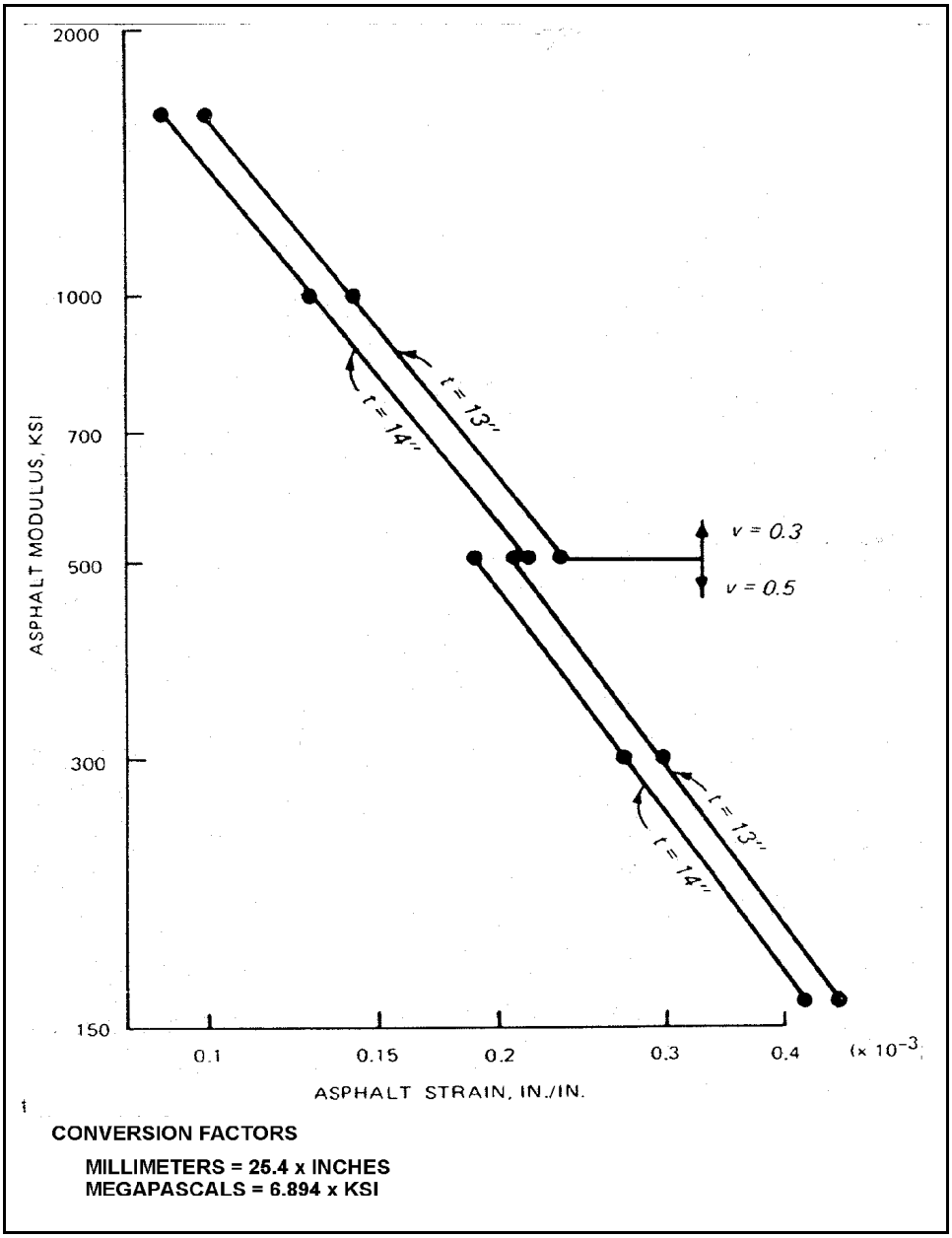


Figure 11-22. Computed strain at the bottom of the asphalt for runway design

CHAPTER 12

PLAIN CONCRETE PAVEMENTS

1. BASIS OF DESIGN – ARMY AND AIR FORCE. The pavement thickness requirement is calculated using a mechanistic fatigue analysis. Stresses under design aircraft are calculated using the Westergaard edge-loaded model. These calculated edge stresses are related to the concrete flexural strength and repetitions of traffic through a field fatigue curve based on full-scale accelerated traffic test of aircraft loads. A wide variety of model tests, theoretical analyses, and field measurements over the years have demonstrated that part of the load applied to the edge of a pavement slab is transferred to and carried by the adjacent slab through dowels, aggregate interlock, etc. For design, a load transfer value of 25 percent is routinely used as a reasonable approximation of the load transfer measured over time on the types of joints approved for use in Army and Air Force airfields. The actual load transfer at a joint will vary depending on joint type, quality of construction, slab length, number of load repetitions, temperature conditions, etc. The design charts in this chapter were developed based on a 25 percent load transfer value. If adequate load transfer is not provided at the joints of trafficked slabs, the pavement should be designed for no load transfer using the PCASE pavement design program that allows direct input of the load transfer value or the gross load used in the design charts in this chapter should be increased by 1/3 to remove the load transfer effect. Alternatively, a thickened edge detail can be used at joints without adequate load transfer. This design method also includes a thickness reduction for high-strength subgrades (modulus of subgrade reaction, k, > 54 kPa/mm (200 pci) in recognition that after the initial flexural fatigue crack forms (classical design failure condition for this design method) the continued slab deterioration through additional cracking and spalling proceeds more slowly on high-strength subgrades than on low-strength subgrades.

2. BASIS OF DESIGN – NAVY. The pavement thickness requirement is calculated using a mechanistic fatigue analysis. Stresses under design aircraft are calculated using the Westergaard interior load model for light traffic or training base commands. For medium to heavy loaded pavements, the Army and Air Force design procedure described above should be considered. The Navy recognizes edge stress design as a way to reduce pavement life cycle costs for bases with medium to heavy traffic missions. These calculated interior stresses are related to the concrete flexural strength and repetitions of traffic through a fatigue curve based on a conservative laboratory beam test relation originally developed by the Portland Cement Association. Adequate quality joints at short joint spacing are required to provide load transfer with the Navy assumptions. Alternate thickness design methods are allowed for Navy pavements if the method is approved by the NAVFAC.

3. USES FOR PLAIN CONCRETE. Military airfield experience has found that plain, unreinforced concrete is generally the most economical concrete airfield surface to build and maintain. Unreinforced concrete will be used for concrete military airfield pavements unless special circumstances exist. The most common exception will be for cases requiring conventional reinforcing as noted in Chapter 1, paragraph 7 and Chapter 13. Other reinforcing for which design techniques are provided in this manual are for special circumstances and their use must be approved by TSMCX, AF MAJCOM pavements engineer, or NAVFAC as appropriate.

4. THICKNESS DESIGN - ARMY AND AIR FORCE PAVEMENTS.

a. General. Figures 12-1 through 12-22 are design curves to be used in designing plain concrete pavements as defined in Chapters 2, 3, and 4. Figures 12-1 through 12-5 are for Army Class I to IV airfields, and Figures 12-6 to 12-17 are for Air Force (Figures 12-6 to 12-11 are for standard mixed traffic

designs.). Figure 12-17 is a design curve for shoulders and is applicable to all airfields requiring concrete shoulders. Figures 12-6 to 12-12 are design curves for the six Air Force standard airfield types, and Figures 12-13 to 12-16 are individual design curves for various aircraft to be used in designing pavements for conditions other than the basic airfield types. Thicknesses may also be determined using the computer programs referenced in Chapter 1.

b. Plain Concrete Pavements on Nonstabilized or Modified Soil Foundations. For plain concrete pavements that will be placed directly on nonstabilized or modified base courses or subgrade, the thickness requirement will be determined from the appropriate design curve using the design parameters of concrete flexural strength, R; modulus of soil reaction, k; gross weight of aircraft; aircraft pass level; and pavement traffic area type (except for shoulder design). The design gross aircraft weight and pass level may vary depending upon the type of traffic area or pavement facility. When using English units and the thickness from the design curve indicates a fractional value, it will be rounded up to the nearest full- or half-inch thickness. The minimum thickness of plain concrete pavement will be 150 millimeters (6.0 inches). When it is necessary to change from one thickness to another within a pavement facility, such as from one traffic area to another, the transition will be accomplished in one full paving lane width or slab length. SI thickness values will be rounded up to the nearest 10 millimeters.

c. Plain Concrete Pavements on Stabilized Base and/or Subgrade. Stabilized base and/or subgrade layers meeting the strength requirements of Chapter 9 and lean concrete base will be treated as low-strength base pavements, and the plain concrete pavement will be considered an overlay with a thickness determined using the following modified, partially bonded rigid overlay pavement design equation:

$$h_o = \sqrt[1.4]{h_d^{1.4} - \left[\left(\sqrt[3]{\frac{E_b}{E_c}}\right) h_b\right]^{1.4}} \qquad (12\text{-}1)$$

where

h_o = thickness of plain concrete overlay, millimeters (inches)

h_d = design thickness of equivalent single slab placed directly on foundation, millimeters (inches)

E_b = modulus of elasticity of base MPa (psi)

E_c = modulus of elasticity of concrete, usually taken as 27,575 MPa (4×10^6 psi)

h_b = thickness of stabilized layer or lean concrete base, millimeters (inches)

5. EXAMPLES OF PLAIN CONCRETE PAVEMENT DESIGN FOR ARMY AND AIR FORCE.

a. General. It is required that an airfield be designed as a medium-load pavement. Types A and B traffic areas are designed for the F-15 at 36,740 kilograms (81,000 pounds), the C-17 at 263,100 kilograms (580,000 pounds), and the B-52 at 181,400 kilograms (400,000 pounds). Types C and D traffic areas and overruns are designed for the F-15 at 27,555 kilograms (60,750 pounds), the C-17 at 197,100 kilograms (435,000 pounds), and the B-52 at 136,080 kilograms (300,000 pounds). Types A, B, and C traffic areas are designed for 100,000 passes of the F-15, 400,000 passes of the C-17, and 400 passes of the B-52. Type D traffic areas and overruns are designed for 1,000 passes of

the F-15, 4,000 passes of the C-17, and 4 pass of the B-52. (Since the B-52 is included in the design, the runway must be 61 meters (200 feet) wide.) On-site and laboratory investigations have yielded the following data required for design: (1) the subgrade material is classified as a silty sand (SM); (2) the modulus of soil reaction, k, of the subgrade is 54 kPa/mm (200 pci); (3) a nearby source of crushed gravel meets the requirements for base course; (4) frost does not enter subgrade material; and (5) 90-day concrete flexural strength, R, is 4.8 MPa (700 psi).

b. Example Design, Slab on Grade. Figure 12-8 is entered with the subgrade k = 200 pci, concrete design flexural strength R = 700 psi, and the pavement thickness is determined for the various traffic areas and overruns as follows:

Traffic Area	**Thickness, mm (in.)**[1]
A	406 (16.0)[1]
B	406 (16.0)
C	302 (12.0)
D and overruns	229 (9.0)

[1] Fractional values would be rounded up to the nearest full- or half-inch for design.

c. Example Design, Slab on Unbound Base. For comparison purposes, designs are developed below for three base-course thicknesses. Field plate bearing tests conducted in a test section to establish the modulus of soil reaction for three thicknesses of base course give k values of 68 kPa/mm (250 pci) for a 152-millimeter (6-inch) base, 81 kPa/mm (300 pci) for a 305-millimeter (12-inch), and 95 kPa/mm (350 pci) for 460-millimeter (18-inch) base. These values are supported by Figure 8-1 and are thus selected for design. Figure 12-8 is entered with the design flexural strength, modulus of soil reaction, and traffic areas to determine the required concrete pavement thicknesses. Thicknesses for shoulders were determined from Figure 12-17. The thicknesses for this example are summarized as follows:

Foundation Condition (1)	**Modulus of Soil Reaction kPa/mm (pci)** (2)	**Thickness, mm (in.)**				
		A (3)	**B** (4)	**C** (5)	**D & Overruns** (6)	**Shoulders**[1] (7)
150-mm (6-in.) base	68 (250)	370 (14.5)	370 (14.5)	290 (11.5)	215 (8.5)	150 (6)
300-mm (12-in.) base	81 (300)	340 (13.5)	340 (13.5)	260 (10.5)	200 (8.0)	150 (6)
460-mm (18-in.) base	95 (350)	330 (13)	320 (12.5)	250 (10)	200 (8.0)	150 (6)

Note: Final thickness should be rounded values.
[1] Use minimum thickness of 150 millimeters (6 inches) for shoulders.
The final selection of concrete pavement thickness must be based upon a study of the cost of importing and placing base course versus savings in concrete pavements.

d. Example Design, Slab on Stabilized Base. Assume that a cement-stabilized base course will be used. Laboratory tests on base-course material have shown that a cement content of 7 percent by weight will yield a 7-day compressive strength of 6.89 MPa (1,000 psi) and a flexural modulus of elasticity E_b of 3,450 MPa (500,000 psi) at an age of 90 days. According to TM 5-822-14/ AFJMAN 32-1019, the compressive strength of 6.89 MPa (1,000 psi) qualifies as a stabilized layer (that is, permits a thickness reduction), and the design is made using Equation 12-1. The single slab thickness h_d of plain concrete is determined from Figure 12-7 using R = 4.8 MPa (700 psi) and k = 54 kPa/mm (200 pci) for the design load and pass level for each type traffic area. The thicknesses of plain concrete overlay determined with Equation 12-1 for several thicknesses of stabilized layer are shown in the following tabulation:

Type Traffic Area	Thickness of Stabilized Layer h_b, mm (in.)	Thickness of Slab on Grade h_d, mm (in.)	Overlay Thickness h_o, mm (in.)
A	150 (6)	406 (16.0)	380 (15.0)
	300 (12)	406 (16.0)	330 (13.0)
	450 (18)	406 (16.0)	267 (10.5)
B	150 (6)	406 (16.0)	370 (14.5)
	300 (12)	406 (16.0)	330 (13.0)
	450 (18)	406 (16.0)	254 (10.0)
C	150 (6)	300 (12.0)	280 (11.0)
	300 (12)	300 (12.0)	215 (8.5)
	450 (18)	300 (12.0)	150 (6.0)
D & Overrun	150 (6)	229 (9.0)	200 (8.0)
	300(12)	229 (9.0)	150 (6.0)[1]
	450 (18)	229 (9.0)	150 (6.0)[1]

Note: Final design overlay thicknesses should be rounded in accordance with paragraph 4b.
[1] Minimum thickness of plain concrete pavement.

The final selection of plain concrete pavement and stabilized base thicknesses will be based upon the economics involved.

e. Design Example for Mixed Traffic.

(1) General. The design of rigid airfield pavements has been based on a standard definition of aircraft mixture, load, and pass levels. However, pavements may be designed for a mixture of aircraft type, loadings, and repetitions other than the standard. This design example presents a procedure for the design of pavements which will be subjected to a mixture of traffic types and loadings based upon equivalent aircraft loadings.

(2) Procedure. The design of a concrete pavement to accommodate a mixture of aircraft traffic is accomplished using the following steps:

(a) Determine the aircraft traffic that is anticipated to use the pavements during the life of the pavements. Arrange this traffic in accordance with aircraft type, gross weight, and number of passes.

(b) Select the pavement thickness required for each aircraft at the design gross weight, pass level, and pavement characteristics.

(c) Select the controlling aircraft as the one requiring the maximum thickness.

(d) Evaluate the controlling thickness in terms of allowable passes for each aircraft in the design mix using the appropriate design curves from Figures 12-1 to 12-17. Those curves are entered from the left with the flexural strength, modulus of subgrade reaction, and load and from the right with controlling thickness and traffic. The intersection point of these two lines will estimate the allowable number of passes of an aircraft. An example of this operation is shown in Figure 12-18.

(e) Determine the number of each aircraft equivalent to one pass of the controlling aircraft by dividing the allowable passes of each aircraft by the allowable pass level of the controlling aircraft.

(f) The number of design passes for each aircraft is then divided by the equivalent passes to determine the total number of equivalent passes of the controlling aircraft to be considered for final design.

(3) Example problem solution.

(a) Determine the thickness of pavement required for a taxiway having the mixture of aircraft, gross weights, and number of passes as shown in columns 1-3 in Table 12-1. The concrete design flexural strength R is 4.48 MPa (650 psi), and the modulus of soil reaction k is 54 kPa/mm (200 pci).

(b) The pavement thickness required for each aircraft is shown in column 4 as determined from appropriate design curves. (Figures 12-3, 12-4, 12-13 through 12-15)

(c) Determine the allowable number of passes of each aircraft for the controlling thickness in column 4 of 363 millimeters (14.3 inches) for the C-141. These allowable passes are determined from the aircraft respective design curve and listed in column 5.

(d) Divide the allowable number of passes (column 5) by the allowable number of passes for the C-141 (10,000). This gives the number of equivalent passes of each aircraft in terms of one pass of the C-141 and is shown in column 6. For example, one pass of the C-141 is equivalent to 780 passes of the F-15 at the design weights.

(e) Divide the number of design passes in column 3 by the number of equivalent passes in column 6 to determine the total number of equivalent C-141 passes for design. These values are shown in column 7.

(f) Determine the total number of equivalent C-141 passes by totaling the values in column 7. Enter the C-141 design curve (Figure 12-14) with the total number of equivalent passes (20,129), the design load of 156,490 kilograms (345 kips), R of 4.48 MPa (650 psi), k of 54 kPa/mm (200 pci), and traffic area A to determine the final design thickness of 381 millimeters (15.0 inches). These values will be rounded to 380 millimeters (15.0 inches).

Table 12-1
Example of Mixed Traffic Design

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Aircraft	Gross Weight, kg (kips)	Aircraft Passes	Preliminary Thickness, in.	Allowable Passes at 14.3 in.	Column 5 Divided by 10,000	Column 3 Divided by Column 6
B-52	181,400 (400)	300	14.2	336	0.03	10,000
C-141	156,490 (345)	10,000	14.3	10,000	1.00	10,000
C-130	70,310 (155)	5,000	9.3	53,000,000	5,300	1
F-15	30,840 (68)	100,000	12.2	7,800,000	780	128
OV-1	8,160 (18)	1,000,000	6.0	Unlimited	--	--
					Total Passes on Basis of C-141 Aircraft	20,129

Conversion Factors:
Millimeters = 25.4 × inches

6. THICKNESS DESIGN - NAVY AND MARINE CORPS PAVEMENTS.

a. General. Figures 12-19 to 12-23 are design curves for various aircraft to be used in determining thickness requirements for individual aircraft. Figures 12-24 to 12-28 are design curves to be used for mixed aircraft traffic in determining thickness requirements. Thicknesses may also be determined using the computer program DESIGN OF RIGID AIRFIELD PAVEMENTS. See paragraph 8 in Chapter 4 for design policy.

b. Fatigue Damage. Repeated aircraft loading results in fatigue damage in the concrete slabs which results in microcracks at the bottom of the slab. These cracks work their way to the surface of the slab, eventually dividing the slab into two or more pieces. In addition, if pumping and loss of support occur at slab corners, the critical stress could increase until a corner break develops. As the proportion of cracked slabs increases, the airfield pavement requires increasing maintenance and repair.

c. Structural Characterization. The slab and foundation are characterized using the Westergaard theory of a slab loaded at the interior resting on a uniformly supported foundation (as modeled using the k value). Stresses may be computed using the computer program RPDESIGN. A major design assumption is that adequate load transfer is provided at the joints so that the load stresses that occur at the joints are not significantly higher than the stresses at the interior of the slab. Adequate load transfer

must be provided by a stabilized base, keyways, mechanical load transfer devices or aggregate interlock.

d. Structural Slab Cracking from Aircraft Loadings. The cracking of a nonreinforced jointed concrete slab with relatively short joint spacing is controlled by:

(1) The magnitude of flexural stress caused by aircraft traffic.

(2) The flexural strength of the concrete.

(3) The number of stress applications.

The number of allowable stress applications to crack the concrete slab is controlled by the ratio of critical stress to flexural strength of the concrete. The relationship used in this design procedure to relate stress/flexural strength ratio to the number of stress applications to cracking was developed by the PCA and is shown in Table 12-2. The lower the ratio of critical stress to flexural strength, the larger the number of load applications that the slab can carry before cracking occurs.

e. Structural Slab Cracking and Mixed Aircraft Loading. When two or more aircraft will utilize a given pavement, each may cause a certain amount of fatigue damage in the concrete slab. The effect of mixed traffic can be provided for in the pavement design by using Miner's cumulative fatigue damage procedure. Fatigue damage is defined as the ratio of the number of loading cycles actually applied (at a given stress level) to the number of allowable load applications to cracking failure (at the same stress level). The resulting fraction represents the proportion of the useful life of the concrete that is consumed by repeated loading.

$$\textit{Cumulative Fatigue Damage} = \sum \frac{n_i}{N_i} \qquad (12\text{-}1)$$

where

n_i = number of applied loads (coverages) at a given stress level (as denoted by i)

N_i = number of allowable loads (coverages) at the same stress level to cracking of the concrete

The fatigue damage can be accumulated over any number of stress levels (or different aircraft loadings) as indicated by the summation sign.

f. Thickness Design Inputs. Five key design inputs are needed to determine the required slab thickness.

(1) Design concrete flexural strength. The 28-day third-point loading flexural strength is used for pavement design. The design flexural strength should be as high as practicable and economical but not less than 4.48 MPa (650 psi). The actual mean flexural strength in the field will be greater than the design flexural strength.

(2) Value of k at top of base. The k value on the subgrade and at the top of the base layers will be determined using the procedure presented in Chapter 5. The value used for design is that obtained at the top of the base. The combined base and subgrade should have a minimum design k value of

54 kPa/mm (200 pci) to prevent excessive permanent deformation of the subgrade due to slab corner deflections. A base course of sufficient thickness and quality should be used to achieve this modulus. However, in no case should design be based on a k value greater than 135 kPa/mm (500 pci). A stabilized base or lean concrete base may be used as a substitute for a granular base course on a 1:1.5 thickness replacement ratio. However, the k value used for design remains the same as that determined at the top of the granular base. The design k value is not increased due to the use of a stabilized or lean concrete base. An unbonded stabilized or lean concrete base does not increase the effective k value greatly due to slippage between the slab and base.

Table 12-2
Stress-Strength Ratios and Allowable Coverages

Stress-Strength[1] Ratio	Allowable Coverages	Stress-Strength[1] Ratio	Allowable Coverages
0.45	2,300,000	0.63	14,000
0.46	1,700,000	0.64	11,000
0.47	1,300,000	0.65	8,000
0.48	1,000,000	0.66	6,000
0.49	720,000	0.67	4,500
0.50	540,000	0.68	3,500
0.51	400,000	0.69	2,500
0.52	300,000	0.70	2,000
0.53	240,000	0.71	1,500
0.54	180,000	0.72	1,100
0.55	130,000	0.73	850
0.56	100,000	0.74	650
0.57	75,000	0.75	480
0.58	57,000	0.76	370
0.59	42,000	0.77	280
0.60	32,000	0.78	210
0.61	24,000	0.79	160
0.62	18,000	0.80	120

[1] Interior or edge stress and design flexural strength.

(3) Type and design gear load of aircraft using facility. Pavement thickness design can be determined for a single design aircraft or for a mix of aircraft traffic. Determine the design gear load for a given aircraft by first selecting the design gross aircraft weight. This is normally the maximum gross aircraft weight at departure. Then estimate the design gear load by assuming that 95 percent of the gross weight is carried by the main gears. Design values are given in Chapter 4.

(4) Number of Aircraft Passes. Forecast the total number of passes (not coverages) of each aircraft that is expected to use the pavement feature over its design life. The “number of passes” is normally the number of departures. The exception to this is in touchdown areas on runways where the impact due to aircraft performing touch-and-go operations will cause pavement damage. On pavements that are to be used for touch-and-go operations, add the expected number of touch-and-go operations over the design life to the number of departures to arrive at the design traffic passes. Minimum pass levels for design are given in Chapter 4.

(5) Primary or Secondary Traffic Area. Guidance on determining if the pavement feature is a primary or secondary traffic area is given in Chapter 4.

g. Thickness design procedure for a single design aircraft. Use Figures 12-19 to 12-23 to determine the concrete slab thickness for single-design aircraft. This procedure will provide the required slab thickness for a specified type of aircraft when the flexural strength, k value, gear load, tire pressure for single-wheel gear aircraft, number of passes, and type of design traffic area are specified. The design chart for aircraft with single wheel gear is shown in Figure 12-19 and is used by entering the design flexural strength and the tire load and projecting as shown by the dashed example lines until the required slab thickness is obtained. The design charts shown in Figures 12-20 through 12-23 are also entered with the design concrete flexural strength and projecting as shown by the dashed example lines until the required slab thickness is obtained. The calculated slab thickness is then rounded to obtain the design thickness.

h. Thickness Design Procedure for Mixed Traffic. When an airfield pavement will be loaded by two or more aircraft, the combined damage caused by the aircraft mix must be used in the design. The required slab thickness may be determined for a mix of aircraft types using Miner’s damage hypothesis and data on forecasted operations of different aircraft types operating at the facility. The slab thickness design for mixed traffic is an iterative procedure in which the designer selects a trial slab thickness that is normally the thickness required for the most critical aircraft using the feature plus 25 millimeters (1 inch). The designer then computes the proportion of the fatigue life of the pavement consumed as the sum of the individual damage contributions of the forecasted volume of each aircraft type, and subsequently varies the slab thickness until less than 100 percent of the fatigue life of the pavement is consumed by the forecasted mix of traffic. This procedure is described in the following sections and is facilitated by the use of a table for computations as shown in Table 12-3.

(1) Required Inputs. The specific aircraft types and their design gear load (typically 95 percent of the maximum gross departure gear load) are entered in columns 1 and 2 of Table 12-3. The projected number of passes (departures) over the selected design period are entered in column 3 of Table 12-3. Divide the projected passes by the appropriate pass-coverage ratio from Table 12-4 to obtain projected coverages for each aircraft. If the forecasted number of passes is not available, use the minimum pass levels given in Chapter 4. Use the pass-coverage ratios given for primary (channelized) traffic areas when designing for runway ends, primary taxiways, and aprons. Use the pass-coverage ratios given for secondary (unchannelized) traffic areas when designing for other areas. Enter the pass-coverage ratio selected for each aircraft in column 4 of Table 12-3, and the number of coverages computed in column 5. The other required inputs are the concrete flexural strength, the effective k value on top of the base, and the tire pressure for each single wheel aircraft, which should be recorded in the spaces provided at the top of Table 12-3.

(2) Determination of interior flexural stresses. Select a trial slab thickness and record it in the space provided for the iteration being performed. For the initial trial, use the required thickness for the expected critical aircraft (determined from Figures 12-19 through 12-23) plus 25 millimeters (1 inch).

Table 12-3
Fatigue Damage Summary Sheet for Mixed Traffic

PAVEMENT IDENTIFICATION _____ TRAFFIC AREA _____
SLAB THICKNESS: _____ SINGLE WHEEL AIRCRAFT TIRE PRESSURE, psi
BASE K: _____ 1. _____ 1. _____
FLEXURAL STRENGTH _____ 2. _____ 2. _____

① AIRCRAFT	② DESIGN GEAR LOAD	③ PROJECTED PASSES	④ P/C	⑤ PROJECTED COVERAGES (n)	⑥ INTERIOR STRESS	⑦ STRESS/F S	⑧ ALLOWABLE COVERAGES (N)	⑨ FATIGUE LIFE CONSUMED (n/N)
							$\sum$ n/N =	

Table 12-4
Pass-to-Coverage Ratios

	Rigid Pavements		Flexible Pavements	
Aircraft	Traffic Area A	Traffic Area B	Traffic Area A	Traffic Area B
B-1	3.41	5.65	1.71	2.82
B-52	1.58	2.15	1.58	2.15
B-727	3.32	5.87	3.32	5.87
C-5A	1.66	2.11	0.83	1.05
C-9	3.73	6.89	3.73	6.89
C-12	7.07	13.89	7.07	13.89
C-17	2.74	3.80	1.37	1.90
C-130	4.40	8.54	2.20	4.27
C-141	3.49	6.23	1.75	3.12
CH-46E	8.01	15.22	8.01	15.22
CH-47	4.38	7.64	4.38	7.64
CH-53E	5.23	9.53	5.23	9.53
CH-54	4.31	8.51	4.31	8.51
DC-10-10	3.64	5.80	1.82	2.87
DC-10-30	3.77	5.59	1.88	2.80
E-2C	8.58	17.00	4.29	8.50
E-4	3.62	5.12	1.81	2.56
F-4C	8.77	17.37	8.77	17.37
F-14	7.78	15.34	7.78	15.34
F-15 C&D	9.30	15.34	9.30	15.34
F-15E	8.10	13.36	8.10	13.36
F/A-18	9.57	17.04	9.57	17.04
F-111	5.63	9.77	5.63	9.77
KC-135	3.48	6.14	1.74	3.07
L-1011	3.58	5.44	1.79	2.72
ORBITER	3.60	6.49	3.60	6.49
OV-1	10.36	17.28	10.36	17.28
P-3	3.58	6.66	3.58	6.66
S-3A	10.43	20.87	10.43	20.87
UH-60	11.94	19.49	11.94	19.49

Determine the flexural stress at the bottom of the slab caused by each particular aircraft gear for the interior loading position, using Figures 12-24 through 12-20. For each single wheel gear aircraft, enter Figure 12-24 with the trial slab thickness, tire load, and tire pressure, move either up or down to the base effective k value and continue horizontally to the flexural stress. For each of the multiwheel gear aircraft types listed, enter the appropriate Figures 12-25 through 12-28 with the trial slab thickness, project a horizontal line left to the effective k value, move either up or down to the design gear load, and continue horizontally to the flexural stress. Record the stress values in column 6 of Table 12-3.

(3) Fatigue life consumption. The stress-strength ratio recorded for each aircraft in column 7 of Table 12-3 is the flexural stress in column 6 divided by the design concrete flexural strength. Select from Table 12-2 the allowable number of coverages corresponding to the stress-strength ratio computed for each aircraft type, and record the allowable number of coverages in column 8 of Table 12-3. For each aircraft type, divide the projected number of coverages in column 5 by the allowable number of coverages in column 8 to determine the portion of fatigue life consumed by the forecasted volume of each aircraft type and record in column 9. The sum of the values in column 9 is the total damage, the proportion of total fatigue life of the slab consumed by the forecasted volumes of the aircraft types listed. If this number is considerably less than 1.00 (100 percent), indicating that the slab has considerable remaining fatigue life at the end of the design period not consumed by the forecasted mix of traffic, then the trial slab thickness may be reduced in the next iteration. If the total damage is greater than 1.00 (100 percent), indicating that the fatigue life of the slab will be consumed by lower traffic volumes than those projected over the design period, then the trial slab thickness must be increased in the next iteration. The process of selecting a slab thickness, determining the flexural stress, and calculating the fatigue life consumption is repeated until the slab thickness which corresponds to an acceptable value for damage (less than 1.00 or 100 percent) is determined.

i. Minimum Thickness. The minimum allowable new concrete pavement thickness is 200 millimeters (8 inches) in primary and secondary traffic areas and 100 millimeters (4 inches) in blast protective areas not subject to aircraft loading. For helicopter and basic training fields the minimum thickness in primary and secondary traffic areas is 150 millimeters (6 inches).

7. DESIGN EXAMPLES FOR NAVY AND MARINE CORPS PLAIN CONCRETE PAVEMENTS.

a. Thickness Design for a Single Aircraft. It is desired to design a plain rigid pavement for the following conditions.

Aircraft = C-141
Design gear load = 70,300 kilograms (155,000 pounds)
Design flexural strength = 4.48 MPa (650 psi)
Effective k value at top of base course = 54 kPa/mm (200 pci)
Total departures over 20-year design life = 25,000
Traffic area = primary taxiway (channelized traffic)

Using Figure 12-22, the required slab thickness is 340 millimeters (13.4 inches). This thickness would then be rounded upward to 350 millimeters (14.0 inches).

b. Thickness Design for Mixed Traffic. A new runway is to be designed to serve frequent operations of C-141, C-130, C-17, and C-5A aircraft. In addition to these aircraft, the new facility will be used by F-14 and P-3 aircraft. The runway is located in a warm climatic region where frost penetration does not need to be considered in the design process. Use the following general design procedure when designing the rigid pavement for this runway.

(1) Subgrade evaluation and testing. A subgrade investigation was performed to evaluate the support of the subgrade soil. Prior to the actual field survey, previous soils investigations, soils maps, climatic data, etc., were collected to provide background information on the soil conditions. Soil borings were then obtained to aid in evaluating the physical properties of the soil. Soil borings were taken at 60-meter (200-foot) intervals along the location of the proposed runway. Soil tests show that the subgrade soil can be classified as CL according to the Unified Soil Classification System. Test results show that there is no significant soil variation in the area for the new runway. Swelling soils are not a problem at the site. Plate load-bearing tests were performed according to ASTM D 1196 to determine the modulus of subgrade reaction (k value). Because of the uniform soils throughout the area, only three plate load tests were taken. The results are summarized below:

Test Number	k Value, kPa/mm (pci)
1	27 (100)
2	41 (150)
3	35 (130)
Average = 34 kPa/mm (127 pci)	

Because of the uniform soil conditions throughout the site, a design k value of the average of the three tests, or 34 kPa/mm (127 pci), is used.

(2) Base course design. Results of a field survey and soil tests indicate that the subgrade soil has a high degree of saturation and low permeability. Thus, very little bottom drainage is likely. Therefore, a base material that is resistant to the detrimental effects of moisture should be used. A free-draining granular base course may be used to increase the subgrade k value to the minimum acceptable k value of 54 kPa/mm (200 pci) on top of the base course. According to Figure 8-1, a 203-millimeter (8-inch) granular base course will raise the k value on top of the base to 54 kPa/mm (200 pci). To prevent intrusion of subgrade fines into the base course, a filter course is included in the design.

(3) Traffic projections. The following tabulations summarize the projected traffic for the new runway over a 20-year design period and the design gear loads.

	Passes	Pass-Coverage Ratio		Coverages, 20 Years	
Aircraft	20 Years	Channelized	Nonchannelized	Channelized	Nonchannelized
C-141	12,500	3.49	6.23	3,582	2,006
C-130	50,000	4.40	8.54	11,364	5,855
C-5	25,000	1.66	2.11	15,060	11,848
C-17	12,500	1.37	1.90	9,124	6,579
P-3	100,000	3.58	6.66	27,933	15,015
F-14	100,000	7.78	15.34	12,854	6,519

Aircraft	Design Gear Load, kg (lb)
C-141	70,300 (155,000)
C-130	38,100 (84,000)
C-5	86,180 (190,000)
C-17	1,179,360 (260,000)
P-3	30,845 (68,000)
F-14[1]	13,600 (30,000)

[1] The design tire pressure for the F-14 is 1.65 MPa (240 psi).

(4) Slab thickness and joints. The k value of 54 kPa/mm (200 pci) as determined above and a design flexural strength of 4.48 MPa (650 psi) are used to determine the required slab thickness. Results of the mixed traffic analysis for the channelized traffic areas are summarized in Table 12-5. Results for the unchannelized traffic areas are summarized in Table 12-6. This shows that a 345-millimeter (13.6-inch) concrete slab is required in areas of channelized traffic to serve the projected aircraft over a design life of 20 years. This is rounded up to a recommended thickness of 350 millimeters (14.0 inches). A 330-millimeter (13.0-inch) concrete slab is required in areas with unchannelized traffic.

8. JOINT USES. Joints are provided to permit contraction and expansion of the concrete resulting from temperature and moisture changes, to relieve warping and curling stresses due to temperature and moisture differentials, to prevent unsightly irregular breaking of the pavement, and to act as a construction expedient to separate sections or strips of concrete placed at different times. The three general types of joints are contraction, construction, and expansion. A typical jointing layout of the three types is illustrated in Figure 12-29.

9. SELECTION OF JOINT TYPES. Joints are either construction or contraction joints. Construction joints are used because there is a physical limit on the concrete placement such as the beginning or end of a placement lane (transverse construction joint) or at the edges of the placement lane (longitudinal construction joint). Concrete is a dynamic material that changes volume throughout its life as chemical reactions occur and as temperature and moisture fluctuations occur. Either joints must be provided to accommodate these natural volume changes in concrete or the concrete will crack. Such joints are contraction joints and are formed by sawing partial depth into the concrete at early ages before cracking can occur. This sawing must be done as soon as the concrete has hardened sufficiently to allow saw cutting without raveling or damage to the concrete. The exact timing of the saw cutting depends on the characteristics of the concrete mixture and the environmental conditions. This cutting occurs on the same day as placing except under very unusual circumstances. Waiting overnight to cut these joints generally will result in uncontrolled cracking. Contraction joints made by inserts forced into the plastic concrete or by manually grooving the plastic concrete surface are unacceptable for military airfields. The most common contraction joints are the regularly spaced transverse joints (transverse contraction joints) placed down the length of the concrete placement lane. The maximum spacing between joints is a function of the slab thickness and allowable limits are provided in paragraph 10. When the concrete placement lane width exceeds these allowable limits between joints, a longitudinal contraction joint will be placed to bring the joint spacing within the maximum limits. The resulting slabs should be square. If the ratio of length to width falls outside of the range of 0.75 to 1.25 or if the geometry of the pavement dictates an irregular shaped slab (e.g., filet slabs), the slabs will have to be reinforced as required in Chapters 1 and 13. Expansion joints are special construction joints that are used to isolate structures from the concrete pavement movement (e.g., isolate hangar from an apron) or to separate two intersecting pavements (e.g., a taxiway intersecting a runway at right angles). Expansion joints often are

Table 12-5
Design Example for Primary (Channelized) Traffic Areas

PAVEMENT IDENTIFICATION New E-W Runway
SLAB THICKNESS: 13.0
BASE K: 200 pci
FLEXURAL STRENGTH 650 psi

TRAFFIC AREA Channelized

SINGLE WHEEL AIRCRAFT	TIRE PRESSURE, psi
1. F-14	1. 240
2.	2.

① AIRCRAFT	② DESIGN GEAR LOAD	③ PROJECTED PASSES	④ P/C	⑤ PROJECTED COVERAGES (n)	⑥ INTERIOR STRESS	⑦ STRESS/FS 650	⑧ ALLOWABLE COVERAGES (N)	⑨ FATIGUE LIFE CONSUMED (n/N)
C-141	155,000	12,500	3.49	3,582	430	0.66	6,000	0.60
C-130	84,000	50,000	4.40	11,364	330	0.51	400,000	0.03
C-5	190,000	25,000	1.66	15,060	360	0.55	130,000	0.12
P-3	68,000	100,000	3.58	27,934	370	0.57	75,000	0.37
F-14	30,000	100,000	7.78	12,853	260	0.40	--	--
C-17	260,000	12,500	1.37	9,124	375	0.58	56,700	0.16
	Conversion Factors:							
	kilograms = 0.453 × pounds							
	megapascals = 0.006894 × psi							
							$\sum$ n/N =	1.28

Table 12-5 (Continued)

PAVEMENT IDENTIFICATION New E-W Runway
SLAB THICKNESS: 13.8
BASE K: 200 pci
FLEXURAL STRENGTH 650 psi

TRAFFIC AREA Channelized

SINGLE WHEEL AIRCRAFT	TIRE PRESSURE, psi
1 F-14	1 240
2	2

① AIRCRAFT	② DESIGN GEAR LOAD	③ PROJECTED PASSES	④ P/C	⑤ PROJECTED COVERAGES (n)	⑥ INTERIOR STRESS	⑦ STRESS/FS 650	⑧ ALLOWABLE COVERAGES (N)	⑨ FATIGUE LIFE CONSUMED (n/N)
C-141	155,000	12,500	3.49	3,582	400	0.62	18,000	0.20
C-130	84,000	50,000	4.40	11,364	300	0.46	1,700,000	0.01
C-5	190,000	25,000	1.66	15,060	340	0.52	300,000	0.05
P-3	68,000	100,000	3.58	27,934	330	0.51	400,000	0.07
F-14	30,000	100,000	7.78	12,853	230	0.35	Unlimited	--
C-17	260,000	12,500	1.37	9,124	346	0.53	230,000	0.04
	Conversion Factors:							
	kilograms = 0.453 × pounds							
	megapascals = 0.006894 × psi							
							$\sum$ n/N =	0.37

Table 12-6
Design Example for Secondary (Unchannelized) Traffic Areas

PAVEMENT IDENTIFICATION New E-W Runway
SLAB THICKNESS: 12.5 inches
BASE K: 200 pci
FLEXURAL STRENGTH 650 psi

TRAFFIC AREA Unchannelized

SINGLE WHEEL AIRCRAFT	TIRE PRESSURE, psi
1 F-14	1 240
2	2

① AIRCRAFT	② DESIGN GEAR LOAD	③ PROJECTED PASSES	④ P/C	⑤ PROJECTED COVERAGES (n)	⑥ INTERIOR STRESS	⑦ STRESS/F S	⑧ ALLOWABLE COVERAGES (N)	⑨ FATIGUE LIFE CONSUMED (n/N)
C-141	155,000	12,500	6.23	2,006	455	0.70	2,000	1.00
C-130	84,000	50,000	8.54	5,855	350	0.54	180,000	0.03
C-5	190,000	25,000	2.11	11,848	370	0.57	75,000	0.16
P-3	68,000	100,000	6.66	15,015	390	0.60	32,000	0.47
F-14	30,000	100,000	15.34	6,519	270	0.42	Unlimited	--
C-17	260,000	12,500	1.90	6,579	395	0.61	24,000	0.27
	Conversion Factors:							
	kilograms = 0.453 × pounds							
	megapascals = 0.006894 × psi							
							$\sum$ n/N =	1.93

Table 12-6 (Continued)

PAVEMENT IDENTIFICATION New E-W Runway
SLAB THICKNESS: 13.0
BASE K: 200 pci
FLEXURAL STRENGTH 650 psi

TRAFFIC AREA Unchannelized
SINGLE WHEEL AIRCRAFT 1. F-14 2.
TIRE PRESSURE, psi 1. 240 2.

① AIRCRAFT	② DESIGN GEAR LOAD	③ PROJECTED PASSES	④ P/C	⑤ PROJECTED COVERAGES (n)	⑥ INTERIOR STRESS	⑦ STRESS/F S	⑧ ALLOWABLE COVERAGES (N)	⑨ FATIGUE LIFE CONSUMED (n/N)
C-141	155,000	12,500	6.23	2,006	430	0.66	6,000	0.33
C-130	84,000	50,000	8.54	5,855	330	0.51	400,000	0.01
C-5	190,000	25,000	2.11	11,848	360	0.55	130,000	0.09
P-3	68,000	100,000	6.66	15,015	370	0.57	75,000	0.20
F-14	30,000	100,000	15.34	6,519	260	0.40	Unlimited	--
C-17	260,000	12,500	1.90	6,579	375	0.58	56,700	0.12
	Conversion Factors:							
	kilograms = 0.453 × pounds							
	megapascals = 0.006894 × psi							
							$\sum$ n/N =	0.75

the source of maintenance headaches so they are used only when concrete movement has to be isolated. The old practice of automatically placing expansion joints at prescribed intervals down a pavement feature is unnecessary and has been discontinued since the 1950s. Doweled construction joints and saw-cut contraction joints without dowels will be the default joints used for military airfield pavement construction. Other joints will be used for special circumstances if needed or with the specific approval of the AF MAJCOM pavements engineer, TSMCX, or NAVFAC as appropriate. These other special application joints include:

a. Thickened-edge expansion or doweled expansion joints where isolation from concrete movement is required.

b. Thickened-edge construction joint where load transfer cannot be provided by dowels and aircraft traffic will cross or be adjacent to the joint.

c. Doweled contraction joint where load transfer from aggregate interlock might be lost due to slab movement (e.g., last three contraction joints on a runway are commonly doweled because of possible joint opening from accumulated slab movements or on long reinforced slabs where environmental changes may result in excessive joint opening).

d. Butt longitudinal construction joints but this requires special design for no load transfer for Army and Air Force airfield pavements.

e. Tied joints (Navy only) where relative movement and separation between slabs must be restricted. Such situations are rare on airfield pavements.

f. Doweled construction joints will normally be used at the intersection of new and old concrete, or alternatively the new concrete may have a thickened edge. Note that this latter situation will leave the old concrete slab without load transfer and its premature failure should be anticipated and planned for. Special junctures that require undercutting and placing concrete below the old slab require approval from the AF MAJCOM pavements engineer, TSMCX, or NAVFAC as appropriate before use.

10. JOINTS FOR ARMY AND AIR FORCE PAVEMENTS.

a. Contraction Joints.

(1) General. Weakened-plane contraction joints are provided to control cracking in the concrete and to limit curling or warping stresses resulting from drying shrinkage and contraction and from temperature and moisture gradients in the pavement. Shrinkage and contraction of the concrete causes slight cracking and separation of the pavement at the weakened planes, which will provide some relief from tensile forces resulting from foundation restraint and compressive forces cause by subsequent expansion. Contraction joints will be required transversely and may be required longitudinally depending upon pavement thickness and spacing of construction joints. A typical contraction joint is shown in Figure 12-30. Instructions regarding the use of saw cuts to form the weakened plane are contained in TM 5-822-7/AFM 88-6, Chapter 8.

(2) Width and Depth of Weakened-Plane Groove. The width of the weakened-plane groove will be a +3 millimeters (+1/8 inch) or greater. The depth of the weakened plane groove must be great enough to cause the concrete to crack under the tensile stresses resulting from the shrinkage and contraction of the concrete as it cures. Experience, supported by analyses, indicates that this depth should be at least one-fourth of the slab thickness for pavements less than 300 millimeters (12 inches),

75 millimeters (3 inches) for pavements 300 to 450 millimeters (12 to 18 inches) in thickness, and one-sixth of the slab thickness for pavements greater than 450 millimeters (18 inches) in thickness. In no case will the depth of the groove be less than the maximum nominal size of aggregate used. Concrete placement conditions may influence the fracturing of the concrete and dictate the depth of groove required. For example, concrete placed early in the day, when the air temperature is rising, may experience expansion rather than contraction during the early life of the concrete with subsequent contraction occurring several hours later as the air temperature drops. The concrete may have attained sufficient strength before the contraction occurs so that each successive weakened plane does not result in fracturing of the concrete. As a result, excessive opening may result where fracturing does occur. If this situation occurs, increase the depth of the initial groove by 25 percent to assure the fracturing and proper functioning of each of the scheduled joints.

(3) Width and depth of sealant reservoir. The width and depth of the sealant reservoir for the weakened plane groove will conform to dimensions shown in Figure 12-31. The dimensions of the sealant reservoir are critical to satisfactory performance of the joint sealing materials.

(4) Spacing of transverse contraction joints. Transverse contraction joints will be constructed across each paving lane, perpendicular to the centerline, at intervals of not less than 3.8 meters (12.5 feet) and generally not more than 6 meters (20 feet) for the Navy and 6 meters (20 feet) for the Army and Air Force. The joint spacing will be uniform throughout any major paved area, and each joint will be straight and continuous from edge to edge of the paving lane and across all paving lanes for the full width of the paved area. Staggering of joints in adjacent paving lanes can lead to sympathetic cracking and will not be permitted unless reinforcement is used. The maximum spacing of transverse joints that will effectively control cracking will vary appreciably depending on pavement thickness, thermal coefficient and other characteristics of the aggregate and concrete, climatic conditions, and foundation restraint. It is impractical to establish limits on joint spacing that are suitable for all conditions without making them unduly restrictive. The joint spacings in Table 12-7 have given satisfactory control of transverse cracking in most instances and may be used as a guide, subject to modification based on available information regarding the performance of existing pavements in the vicinity or unusual properties of the concrete. For the best pavement performance, the number of joints should be kept to a minimum by using the greatest allowable joint spacing that will control cracking. Experience has shown, however, that oblong slabs, especially in thin pavements, tend to crack into smaller slabs of nearly equal dimensions under traffic. Therefore, it is desirable, insofar as practicable, to keep the length and width dimensions as nearly equal as possible. In no case should either dimension exceed the other dimension by more than 25 percent. Under certain climatic conditions, joint spacings different from those in Table 12-7 may be satisfactory. Where it is desired to change the joint spacing, a request will be submitted to the Transportation System Mandatory Center of Expertise, TSMCX (CENWO-ED-TX) for Army projects, or the appropriate Air Force Major Command for Air Force projects, regardless of who performs the design.

Table 12-7
Recommended Spacing of Transverse Contraction Joints

Pavement Thickness, millimeters (inches)	Spacing, meters (feet)
Less than 230 (9)	3.8 to 4.6 (12.5 to 15)
230 to 305 (9 to 12)	4.6 to 6 (15 to 20)
Over 305 (12)[1]	6 (20 max)

[1] 6-meter (20-foot) maximum spacing for Army and Air Force pavements.

(5) Spacing of longitudinal contraction joints. Contraction joints will be placed along the centerline of paving lanes that have a width greater than the determined maximum spacing of transverse contraction joints in Table 12-7. Contraction joints may also be required in the longitudinal direction of overlays, regardless of overlay thickness, to match joints existing in the base pavement unless a bond-breaking medium is used between the overlay and base pavement or the overlay pavement is reinforced.

(6) Doweled contraction joints. Dowels will be required in the last three transverse contraction joints back from the ends of all runways to provide positive load transfer in case of excessive joint opening due to cumulative shrinkage of the pavement. Similar dowel requirements may be included in the transverse contraction joints at the end of other long paved areas, such as taxiways or aprons where local experience indicates that excessive joint opening may occur. In rigid overlays in Air Force and Army Type A traffic areas, longitudinal contraction joints that would coincide with an expansion joint in the base pavement will be doweled. Dowel size and spacing will be as specified in Table 12-8.

(7) Aggregate Interlock. Aggregate interlock can provide adequate load transfer across joints when the pavement is originally constructed during hot weather. However, as joint movements due to temperature variation and load applications increase and the joint begins to open, aggregate interlock is lost and load transfer is greatly reduced. The effectiveness of aggregate interlock may be improved by increasing base strength and the angularity of coarse aggregate and shorter spacing of joints.

b. Construction Joints. Centerline longitudinal construction joints should be used on runways and taxiways.

(1) General. Construction joints may be required in both the longitudinal and transverse direction. Longitudinal construction joints (generally spaced 6 meters (20 feet) apart but may be more than one lane wide depending on construction equipment capability) will be required to separate successively placed paving lanes. Transverse construction joints will be installed when it is necessary to stop concrete placement within a paving lane for a length of time that will allow the concrete to start to set. All transverse construction joints will be located in place of other regularly spaced transverse joints (contraction or expansion types) and will normally be doweled butt joints. There are several types of construction joints available for use as shown in Figure 12-32 and as described below. The selection of the type of construction joint will depend on such factors as the concrete placement procedure (formed or slipformed), airfield type, adjacent existing pavement, and foundation conditions.

Table 12-8
Dowel Size and Spacing for Construction, Contraction, and Expansion Joints

Pavement Thickness mm (in.)	Minimum Dowel Length mm (in.)	Maximum Dowel Spacing mm (in.)	Dowel Diameter and Type
Less than 203 (8)	406 (16)	305 (12)	20-mm (3/4-in.) bar
203-292 (8-11.5)	406 (16)	305 (12)	25-mm (1-in.) bar
305-394 (12-15.5)	508 (20)	381 (15)	25- to 30-mm (1- to 1-1/4-in.) bar or 25-mm (1-in.) extra-strength pipe
406-521 (16 - 20.5)	508 (20)	457 (18)	25- to 40-mm (1- to 1-1/2-in.) bar or 25- to 60-mm (1- to 1-1/2-in.) extra-strength pipe
533-648 (21 - 25.5)	610 (24)	457 (18)	50-mm (2-in.) bar or 50-mm (2-in.) extra-strength pipe
660 (26) or more	762 (30)	457 (18)	75-mm (3-in) bar or 75-mm (3-in.) extra-strength pipe

(2) Doweled butt joint. The doweled butt joint is considered to be the best joint for providing load transfer and maintaining slab alignment. Therefore, it is the desirable joint for the most adverse conditions, such as heavy loading, high traffic intensity, and lower strength foundations. However, because the alignment and placement of the dowel bars are critical to satisfactory performance, the dowels must be carefully aligned, especially for slipformed concrete. The doweled butt joint is required for all transverse construction joints.

(3) Thickened-edge joint. Thickened-edge-type joints may be used in lieu of other types of joints employing load-transfer devices. The thickened-edge joint is constructed by increasing the thickness of the concrete at the edge to 125 percent of the design thickness. The thickness is then reduced by tapering from the free-edge thickness to the design thickness at a distance 1.5 meters (5 feet) from the longitudinal edge. The thickened-edge butt joint is considered adequate for the load-induced concrete stresses. The thickened-edge joint may be used at free edges of paved areas to accommodate future expansion of the facility or where aircraft wheel loadings may track the edge of the pavement.

c. Expansion Joints.

(1) General. Expansion joints will be used at all intersections of pavements with structures and may be required within the pavement features. A special expansion joint required at pavement intersections is the slip joint. The types of expansion joints are the thickened-edge, the thickened-edge slip joint, and the doweled type (Figures 12-33 and 12-34). Filler material for the thickened-edge and doweled-type expansion joint will be a nonextruding type. Bituminous filler material will not be used when the sealer is non-bituminous. The type and thickness of filler material and the manner of its installation will depend upon the particular case. Usually a preformed material of 19-millimeter (3/4-inch) thickness will be adequate, but in some instances a greater thickness of filler material may be required. Filler material for slip joints will be either a heavy coating of bituminous material not less than

6 millimeters (1/4 inch) in thickness when joints match or normal nonextruding-type material not less than 6.3 millimeters (1/4 inch) in thickness when joints do not match. Where large expansions may have a detrimental effect on adjoining structures, such as at the juncture of rigid and flexible pavements, expansion joints in successive transverse joints back from the juncture should be considered. The depth, length, and position of each expansion joint will be sufficient to form a complete and uniform separation between the pavements and between the pavement and the structure concerned and, unless doweled, must be completely straight from end to end so translation can occur. The designer should dowel expansion joints only under special conditions. (Use thickened edge expansion joints.) Expansion joint filler must cover the full depth of the joint surface so there is no point-to-point contact of concrete.

(2) Between pavement and structures. Expansion joints will be installed to surround, or to separate from the pavement, any structures that project through, into, or against the pavements, such as at the approaches to buildings or around drainage inlets and hydrant refueling outlets. The thickened-edge-type expansion joint will normally be best suited for these places.

(3) Within pavements.

(a) Expansion joints within pavements must be carefully constructed. Except for protecting abutting structures and taxiways intersecting at an angle, their use will be kept to the absolute minimum necessary to prevent excessive stresses in the pavement from expansion of the concrete or to avoid distortion of a pavement feature through the expansion or translation of an adjoining pavement. The determination of the need for and spacing of expansion joints will be based upon pavement thickness, thermal properties of the concrete, prevailing temperatures in the area, temperatures during the construction period, and the experience with concrete pavements in the area.

(b) Longitudinal expansion joints within pavements will be of the thickened-edge type (Figure 12-33). Dowels are not recommended in longitudinal or most transverse expansion joints because differential expansion and contraction and subgrade movement parallel with the joints may develop undesirable localized strains and possibly failure of the concrete, especially near the corners of slabs at transverse joints.

(c) Transverse expansion joints within pavements will often be the doweled type (Figure 12-33). There may be instances when it will be desirable to allow some slippage in the transverse joints, such as at the angular intersection of pavements to prevent the expansion of one pavement from distorting the other. In some of these instances, instead of a transverse expansion joint, a thickened-edge slip joint may be used (Figure 12-34). When a thickened-edge joint (slip joint) is used at a free edge not perpendicular to a paving lane, a doweled transverse expansion joint will be provided as shown in Figure 12-32.

d. Dowels. The important functions of dowels or any other load-transfer device in concrete pavements are to help maintain the alignment of adjoining slabs and to transmit loads across the joint. Different sizes of dowels will be specified for different thicknesses of pavements (Table 12-8). When extra-strength pipe is used for dowels, the pipe will be filled with either a stiff mixture of sand-asphalt or portland cement mortar, or the ends of the pipe will be plugged. If the ends of the pipe are plugged, the plug must fit inside the pipe and be cut off flush with the end of the pipe so that there will be no protruding material to bond with the concrete and prevent free movement of the dowel. Figures 12-30, 12-32, and 12-33 show the dowel placement. All dowels will be straight, smooth, and free from burrs at the ends. One end of the dowel will be painted and oiled to prevent bonding with the concrete. Dowels

used at expansion joints will be capped at one end, in addition to painting and oiling, to permit further penetration of the dowels into the concrete when the joints close.

e. Special Provisions of Slipforming Paving.

(1) Provisions must be made for slipform pavers when there is a change in longitudinal joint configuration. The thickness may be varied without stopping the paving train, but the joint configuration cannot be varied without modifying the side forms, which will normally require stopping the paver and installing a header. The requirements discussed as follows shall apply.

(2) The header may be set on either end of the transition slab with the transverse construction joint doweled as required. As an example, for the transition between the type A and type D areas on a medium-load pavement, the header could be set at the end of either type pavement. The dowel size and location in the transverse construction joint should be commensurate with the thickness of the pavement at the header.

f. Joint Sealing. All joints will be sealed with a suitable sealant to prevent infiltration of surface water and solid substances. The Army and Air Force do not require all joints to be sealed with preformed compression seals. Jet-fuel-resistant (JFR) sealants will be used in the joints of aprons, warm-up holding pads, hardstands, washracks, and other paved areas where fuel may be spilled during the operation, parking, maintenance, and servicing of aircraft. In addition, heat-resistant JFR joint sealant materials will be used for runway ends and other areas where the sealant material may be subject to prolonged heat and blast of aircraft engines. Non-JFR sealants will be used in the joints of all other airfield pavements. JFR sealants will conform to Federal Specification SS-S-200 or ASTM D 3569 and D 3581. Non-JFR sealants will conform to ASTM D 3405, D 3406, D 1190, and CRD-C-525. Silicone sealants meeting ASTM D 5893 may also be used in both JFR and non-JFR areas. When heat- and blast-resistant JFR sealants are required, they will conform to Federal Specification SS-S-200. An optimal sealant, meeting both the heat- and blast-resistant JFR and non-JFR sealant requirements, is a preformed compression seal conforming to ASTM D 2628 and D 2835. As a general rule, compression-type preformed sealants must have an uncompressed width of not less than twice the width of the joint reservoir. However, the maximum and minimum dimensions for the seal width should be based on the joint opening and expected movement. The selection of a pourable or preformed sealant should be based upon the economics involved and the service life desired. Compression seals will remain effective five to seven times as long as liquid sealants.

g. Special Joints and Junctures. Situations will develop where special joints or variations of the more standard-type joints will be needed to accommodate movements that will occur and to provide a satisfactory operational surface. Some of these special joints or junctures as shown in Figure 13-2 are discussed in the following paragraphs and in particular, paragraph 11.

11. JOINTS FOR NAVY AND MARINE CORPS PAVEMENT.

a. Expansion Joints. Expansion joints allow for the expansion of the pavement and the reduction of high compressive stresses at critical locations in the concrete pavement in hot weather. Expansion joints are placed the full depth of the slab. Expansion joints should be used at all intersections of pavements with fixed structures, at nonperpendicular pavement intersections, and between existing and new concrete pavements when the joints in the adjacent slabs are not aligned. Expansion joints are not otherwise required within the nonreinforced concrete pavement. See Figure 12-34 for expansion joint details.

b. Contraction (Weakened Plane) Joints. Contraction joints should be used to control cracking in the pavement due to volume changes resulting from a temperature decrease or a moisture decrease and to limit curling and warping stresses from temperature and moisture gradients in the pavement. Contraction joints are formed in concrete by partial depth sawing or by installing sawable inserts. The saw cut joint or formed groove provides a weakened plane which will crack through the full slab depth during shrinkage and contraction of the concrete as it cures. Contraction joints are required in the transverse direction and also in the longitudinal direction depending upon slab thickness and spacing of the construction joints. See Figure 12-30 for contraction joint details.

c. Construction Joints. Construction joints are used between paving lanes or when abutting slabs are placed at different times. Longitudinal and transverse construction joints may be required. Transverse construction joints will be required when it is necessary to stop concrete placement for a length of time sufficient to allow the concrete to begin to set. Longitudinal construction joints are generally spaced 6 meters (20 feet) apart but may be multiple lane width, depending on the construction equipment.

(1) Transverse construction joints. When possible, locate all transverse construction joints at the same location as regularly spaced transverse joints. Provide for load transfer or a thickened edge.

(2) Longitudinal construction joints. Construct longitudinal construction joints as shown in Figure 12-32 and indicated below.

(a) Keyed joint. Keyways have been used extensively to provide load transfer along longitudinal joints. However, there has been a substantial amount of keyway failure under heavy aircraft loading on thinner slabs. Keyed joints may only be used on slabs 225 mm (9 in.) thick or greater.

(b) Butt joint. A butt joint may be used for longitudinal construction joints on pavements less than 229 mm (9 in.) thick constructed with a stabilized base.

(c) Thickened-edge joint. A thickened-edge joint may be used for longitudinal construction joints. The thickened-edge joint may be used for any pavement thickness and base type.

d. Joint Spacing. The standard slab size for pavements is 3.8 by 4.6 meters (12.5 by 15 feet). Transverse joint spacing is 4.6 meters (15.0 feet) and longitudinal joint spacing is 3.8 meters (12.5 feet). For slabs having a thickness greater than 300 millimeters (12 inches), joint spacing can be increased to a maximum of 6.1 meters (20 feet). The transverse joint spacing shall not vary from the longitudinal joint spacing by more than 25 percent. Figure 12-29 shows standard joint spacings.

e. Load Transfer Design. A properly designed joint must provide adequate load-transfer across the joint. Load transfer efficiency is normally defined as the ratio of deflection of the unloaded side to the deflection of the loaded side of the joint. Good load transfer will aid in preventing deterioration such as corner breaks, transverse and longitudinal cracking, faulting, pumping, and spalling. Different amounts of load transfer can be obtained through the use of aggregate interlock, dowel bars, keyways, a stabilized base, or a combination of these.

(1) Aggregate interlock. Aggregate interlock can provide adequate load transfer across joints when the pavement is originally constructed or during hot weather. However, as joint movements due to temperature variation and load applications increase and the joint begins to open, aggregate interlock is lost and load transfer is greatly reduced. The effectiveness of aggregate interlock may be improved by increasing base strength and the angularity of coarse aggregate and shorter spacing of joints.

(2) Dowel bars. Dowel bars are used to provide load transfer and prevent excessive vertical displacements of adjacent slabs. There are some situations where the use of dowels is appropriate, such as for creating load transfer where tying in to existing pavements.

(3) Stabilized base. A stabilized base can be used to improve load transfer effectiveness by reducing joint deflections through increased support across a joint. Use a stabilized base for all pavements less than 225 millimeters (9 inches) thick to provide improved load transfer and lower deflections and stresses. A stabilized base may also be used for pavements greater than 225 millimeters (9 inches) thick to provide additional load transfer. Where thickened-edge joints are used, the stabilized base is not required.

f. Joint Sealants. Joint sealants are used to provide a seal to reduce infiltration of water and incompressibles. An effective joint seal will help retard and reduce distress related to free water and incompressibles, such as pumping, spalling, faulting, and corrosion of mechanical load transfer devices. Several pavement areas require fuel-resistant or blast-resistant joint sealants. Use jet fuel-resistant sealants for all aprons. Use blast-resistant sealants for the first 305 meters (1,000 feet) of runways and exits at runway ends. Use sealing compounds meeting ASTM D 1190, D 3405, or D 3406 for taxiways and runway interiors.

(1) Types of sealant materials. The three major types of sealant materials are (a) field poured, hot applied; (b) field poured, cold applied; and (c) preformed compression seals. These materials may be jet fuel resistant (tar-based) or nonjet fuel resistant (typically asphalt based).

(a) Field poured, hot applied. This group of sealants includes rubberized asphalt sealant and rubberized tar sealant. Rubberized asphalt joint sealants must meet ASTM D 1190, D 3405, or D 3406. Rubberized tar sealants must meet ASTM D 3569 or D 3581.

(b) Field poured, cold applied. These are two-component, polymer-based, cold-applied heat and jet fuel-resistant joint sealants. These sealants must meet Federal Specification SS-S-200E. The Air Force and Navy recommends the use of silicone sealants that conform to NFGS 02522, 02562, and ASTM 5893 in lieu of sealants that meed Federal Specification SS-S-200E.

(c) Preformed compression seals. The most common type of preformed compression seal is the neoprene compression seal. Neoprene compression seals must satisfy ASTM D 2628. Preformed compression seals may be used in the areas designated in NFGS-02522. Preformed compression seals are designed to be in compression for their entire life. There is little bond between the compression seal and the sidewalls of the joint to sustain tension.

(2) Joint reservoir design. The joint reservoir must be properly designed so that the joint sealant can withstand compressive and tensile strains.

(a) Field poured sealants. The shape factor, which is defined as the ratio of the depth of the sealant to the width of the joint, should be between 1.0 and 1.5. Dimensions of the joint sealant and reservoir are shown in Figure 12-30. A backer rod or bond breaking tape must be used to help obtain a proper shape factor and to prevent the joint sealant from bonding to the bottom of the joint reservoir. Most field poured liquid joint sealants can withstand strains of approximately 25 percent of their original width. Joint reservoir and sealant dimensions shown in Figure 12-30 are based on a slab size of 3.8 by 4.5 meters (12.5 by 15.0 feet).

(b) Preformed compression seals. The reservoir width for preformed compression seals must be designed to keep the sealant in compression at all times. The depth of the reservoir must exceed the depth of the seal but is not related directly to the width of the joint. The width of the compression seal should be approximately twice the width of the joint. The limits on the compression seal are normally 20 percent minimum and 50 percent maximum compression strain of the original sealant width. For example, the working range of a 25-millimeter (1-inch) wide neoprene compression seal is from 13 to 20 millimeters (0.5 to 0.8 inches). If the seal is subjected to compression greater than the 50 percent level for extended periods of time, the seal may take a compression set, and the webs may bond to each other. If this happens, the seal will not open as the joint opens, and the seal will no longer be effective. The joint dimensions for the standard size slab are shown in Figure 12-30. Design sealant dimensions based on the actual joint spacing. Choose preformed neoprene compression seal dimensions so that the working range of the joint is within the working range of the sealant.

12. JOINTING PATTERN FOR RIGID AIRFIELD PAVEMENTS. Proper jointing pattern for rigid airfield pavement is a critical item of design and construction for all services. Not only is it important for a quality product, but it can and should promote efficiency for the construction contractor, and thus cost savings. Criteria for type of joints, their location, and maximum allowable spacing have been given in the previous paragraphs. This paragraph focuses on appropriate and efficient layout of the jointing pattern. Laying out a good jointing pattern depends on experience working at it and is more of an art than a science. The designer must learn to play with it and try various combinations until the optimum layout is reached. Every productive hour spent on this produces appreciable cost savings.

a. General. All project joint layout drawings should have a prominent note on them saying "No changes in the jointing pattern shall be made without the written approval of the design engineer." The design engineer must make every effort to provide an efficient layout for construction, consistent with the limits of criteria. However, once the joint layout is finalized, no change whatever should be made by field personnel unless examined and approved in writing by the designer to be sure that it does not compromise his plan or violate criteria.

b. Layout. Joint layouts should be as simple and as uniform as possible and meet all criteria of the preceding paragraphs. Except for unusual circumstances, all joints should have straight lines with the longitudinal and transverse joints at right angles. Careful study must always be made to ensure that the paving lanes (longitudinal construction joints vs transverse joints) are laid out in the right direction for the contractor's efficient work--particularly where the area has irregular boundaries.

c. Spacing. Longitudinal construction joints should be spaced such that the widths of pioneer (pilot) lanes are all equal and any variability in total distance is taken care of in a few fill-in lanes, where setting the paver width is not such a problem. Except where impractical, the jointing pattern should not require slabs that have one side exceeding the other by more than 25 percent; if any slab exceeds this, it must be reinforced--an extra expense.

d. Longitudinal Construction Joints. Never should longitudinal construction joints be spaced by simply dividing the overall distance into a whole number of lanes of equal width, unless that width comes out to an easily used value for the paving operations. If practical, pioneer lanes should have widths in multiples of 6 inches, or, if metric is used for the project, multiples of 250 millimeters. Extensions to the paver are easily made in these intervals. Other, odd intervals can be used, but they are more expensive for the contractor to adjust. Fill-in lane widths should be reasonably close to the pioneer lanes, and all fill-in lanes can be made the same width as necessary to accommodate the total distance. However, if the take-up distance is small, it is usually better to provide it in just one or two lanes and make the rest of the fill-in lanes uniform in width--simply to reduce the chance of measurement error during construction.

e. Transverse Contraction Joints. For transverse contraction joints, the spacing should be the same as the longitudinal construction joints, or close to the same. Again, it is usually not appropriate to design the transverse joints all with the same spacing, unless this comes out with easily measured spacing. Otherwise, make spacing an easily remembered and an easily measured distance, with any take-up distance provided in one or two spaces. One main objective is to provide spacings that are easy for the joint saw crew (usually working at night) to follow and not get confused (no fractional inches, or odd metric units, and as little variation as possible).

f. Replacements and Additions. Much of the present airfield paving work consists of replacement areas and additions to existing pavement. This often results in odd-shaped areas with irregular boundaries, proving difficult to provide a really good jointing pattern. As much as possible, the guidelines in the previous subparagraphs should be followed, modified as absolutely necessary. Care should be taken to, as much as possible, prevent small slabs and odd-shaped slabs requiring reinforcement. When working with areas having irregular boundaries, it becomes a process of trial and error to provide the best fit to the area, following criteria and minimizing as much as possible the need for odd-shaped reinforced slabs--an expense to be avoided. When abutting existing PCC pavement, an attempt should be made to match the existing joint pattern, where possible. Older pavements will often have 7.6-meters (25-foot) joint spacing, when now we are usually allowed a maximum of 6 meters (20 feet). For jobs of moderate size, if it is possible to match the existing joint pattern, the new joint spacing can be made 7.6 meters (25 feet), provided the existing has shown no distress because of the 7.6-meter (25-foot) spacing. Otherwise, use 3.8-meter (12.5-foot) spacing. Either is acceptable, but the Using Service should be contacted to get their preference--some like one and some the other.

g. Expansion Joints and Slip Joints.

(1) New PCC to New PCC. Where pavements abut buildings and other fixed objects, an expansion joint should be provided. Where two new PCC pavements meet a an angle, an expansion joint is necessary. If they meet at a 90-degree angle, the intersection should be a, thickened-edge expansion joint. If they meet at other than a 90-degree angle, it should be a thickened-edge joint, either expansion joint or slip joint. If the joints on new-to-new construction do not match and no expansion or slip joint is used, 900-millimeter (3-foot) wide strips of reinforcing should be installed along each side of the joint to prevent sympathetic cracks from forming in line with mismatched joints. Normally, expansion joints of any kind should not be doweled if load transfer can be provided in another way. There have been projects where doweled expansion joints have been successfully used, but this should be used only where no translation movements or stresses are expected.

(2) New PCC to Old PCC. Where new PCC pavement meets old (existing) PCC pavement at an angle, an attempt should be made to provide load transfer. At a 90-degree intersection, an ordinary thickened edge (one side) expansion joint can be used if no load transfer is necessary (existing pavement so understrength that it will not match the new pavement). At a 90-degree intersection and at an intersection other than 90 degrees, it usually will be best to put in a doweled construction joint at the intersection, and then install a thickened-edge expansion joint far enough back on the new pavement to totally clear any fillets and give the shortest unobstructed (straight) line across the pavement. .

(3) Slip Joints. Slip joints, 6-millimeter (1/4-inch) minimum thickness, can be used in lieu of expansion joints in places where only translation is expected, and no movement perpendicular to the joint is expected. At 6-millimeter (1/4-inch) thickness, they are sufficient to prevent sympathetic cracking across the joint, and thus eliminate the need for the 900-millimeter (3-foot) strip of reinforcing on each side of new-to-new construction.

h. Special Joint. A "special joint", as shown in Figure 12-32 (Sheet 3 of 3), can be used to provide load transfer on the existing side of a new PCC to old PCC joint. This can be used under the conditions listed below. Although somewhat expensive, this is an excellent joint when constructed properly, but requires close supervision in the field to ensure that the constructor builds it properly. Note that considerable handwork is required in grading the undercut and placing concrete and reinforcement. (Never should the contractor be allowed to attempt to fill the undercut with concrete spread by the paver.) Special joint (undercut) between new and existing pavements. A special joint (undercut) (Figure 12-32 (Sheet 2 of 3)) may be used at the juncture of new and existing pavements for the following conditions:

(1) When load-transfer devices (keyways or dowels) or a thickened edge was not provided at the free edge of the existing pavement.

(2) When load-transfer devices or a thickened edge was provided at the free edge of the existing pavement, but neither met the design requirements for the new pavement.

(3) For any joints, when removing and replacing slabs in an existing pavement if the existing load-transfer devices are damaged during the pavement removal, and if other types of joints are suitable.

The special joint design need not be required if a new pavement joins an existing pavement that is grossly inadequate to carry the design load of the new pavement or if the existing pavement is in poor structural condition. If the existing pavement can only carry a load that is 50 percent or less of the new pavement design load, special efforts to provide edge support for the existing pavement may be omitted. However, if the provisions for edge support are omitted, accelerated failures in the existing pavement may be experienced. Any load-transfer devices in the existing pavement should be used at the juncture to provide as much support as possible to the existing pavement. The new pavement will simply be designed with a thickened edge at the juncture. Drilling and grouting dowels in the existing pavement for edge support may be considered, if structurally suitable, as an alternative to the special joint, but a thickened-edge design will be used for the new pavement at the juncture.

i. Tied Joints (Navy Only). Tied joints are seldom used for airfield pavement. However, two instances occur:

(1) As required and shown in Figure 12-29, "Typical Jointing". (The situation must be evaluated and existing service experience observed to prevent tying two slabs that have conditions (dimensions or aggregate properties) which may cause a crack to form between the tied joint and the next adjacent joint.)

(2) Where half a slab is removed across a paving lane halfway between transverse joints (at least 3 meters (10 feet) must be removed and not less than 3 meters (10 feet) remain). In this instance, the new construction joint of new to existing, at mid-slab, must be tied (with drilled and epoxied reinforcing bars). No joint reservoir should be sawed, or sealant applied.

j. Portland Cement Concrete to Asphalt Concrete Intersections. Figures 12-35, 12-36, 12-37, and 12-38 show various types of joints to use for the juncture of PCC and AC pavements.

(1) Figure 12-35. This joint is to be used for most transverse joints that will receive aircraft traffic at Army installations and for all transverse joints that will receive aircraft traffic at Air Force installations.

(2) Figure 12-36. This detail can be used for transverse joints in areas where high-speed aircraft traffic is expected. It is a more conservative joint, but also more expensive. The Using Service should be contacted to determine which joint they prefer. This joint should no be used for Air Force jobs.

(3) Figures 12-37 and 12-38 show joints that can be used where no appreciable aircraft traffic is expected to cross. (Such as longitudinal joints on the outer edges of PCC keel sections in an AC pavement and similar locations.)

(4) Normally, the joint between PCC pavement and AC shoulder pavement should be a plain butt joint. Depending on local experience, it may be well to saw a reservoir in this joint and apply joint sealer.

k. Sample Joint Layouts. Figures 12-39 through 12-43 are samples of various typical jointing patterns. An explanation of the significance and details of each is in the following subparagraphs.

(1) Figure 12-39. This shows a perfect jointing pattern for a rectangular pavement with easily divided boundary dimensions. Unfortunately such regular dimensions and shapes do not often occur--particularly in all the replacement and repair work being required now.

(2) Figure 12-40. Metric.

(a) This figure shows the same 30.4 meters (100 feet) by 42.7 meters (140 feet) pavement as Figure 12-39, but everything is in metric (SI). It can be seen at the bottom of the page that the longitudinal construction joints have been evenly spaced across the 30.4 meters (100 feet.) This may look nice on paper, but it requires the contractor to set the width of his paver for an odd width--more expensive. If there were a large number of longitudinal lanes, it could be appropriate to make them all the same width, even if this were an odd dimension for all the lanes, since this would require only one odd setting of the paver width. At the top of the page is a layout showing four lanes at 6.0-meter (19.7-foot) width and a single fill-in lane at an odd width. (The width of the fill-in lanes is not so critical.)

(b) At the right-hand side of the figure is shown a spacing for transverse contraction joints--an odd spacing obtained by dividing the total distance into a series of equal width spacing. This is, of course, feasible to construct a series of very odd cumulative spacings. This makes it more likely that the joint sawing crew (usually working at night) may make a mistake in adding the cumulative distance, and thus get a joint out of line. The spacing shown on the left side of the page, with six spaces at 6.0 meters (19.7 feet) and one takeup space of 6.56 meters (21.5 feet), is much easier for the joint sawing crew to work with, and thus much less likely to get out of line. Always make joint layouts as simple as possible, within criteria.

(3) Figure 12-41.

(a) This figure, for a 54-meter (180-foot) wide pavement, shows nice, easy spacing of longitudinal and transverse joints, if everything is in the inch-pound system. See the spacing of 6 meters (20 feet) by 6 meters (20 feet), at the top and right side of the drawing. However, if the same overall width has to be designed in metric, it gets more complicated. Still, there is a variety of spacing that can be feasiblely used for transverse contraction joints.

(b) Longitudinal construction joints are a problem, however. At the bottom of the figure are shown three possible solutions. The top one of the three is very pretty and easy to design, but it requires the contractor to adjust his paver to an odd width for the pioneer lanes--an extra expense. The middle

line of the three shows a good jointing pattern with nine lanes at 6.0 meters (20 feet) and two fill-in lanes at 6.36 meters (20.9 feet)--well within criteria for shape. The bottom one of the three is also a good jointing pattern with five pioneer lanes at 6.0 meters (20 feet) and four fill-in lanes at 6.18 meters (20.3 feet). Neither of these two last systems requires the contractor to adjust his paver to anything other than an even width or to make any changes in adjustment.

(4) Figure 12-42. This is simply a further explanation of Figure 13-2b, with more details. See also subparagraph 12g.

(a) This figure shows a new PCC pavement intersecting an existing PCC pavement at an angle (90 degrees). Such an intersection requires a joint that can tolerate movement, both at right angles to the joint and along the joint, as well as providing load transfer across the joint.

(b) One approach would be to drill and grout dowels in the existing PCC and put in a doweled expansion joint at the intersection. This is not desirable, because it locks the two pavements together and does not permit any translation movement along the joint. This is particularly significant if the angle of intersection is other than 90 degrees.

(c) Another approach would be to put in a thickened-edge expansion joint at the intersection. But often the existing pavement will not have a thickened edge--and thus no true load transfer across the joint can take place.

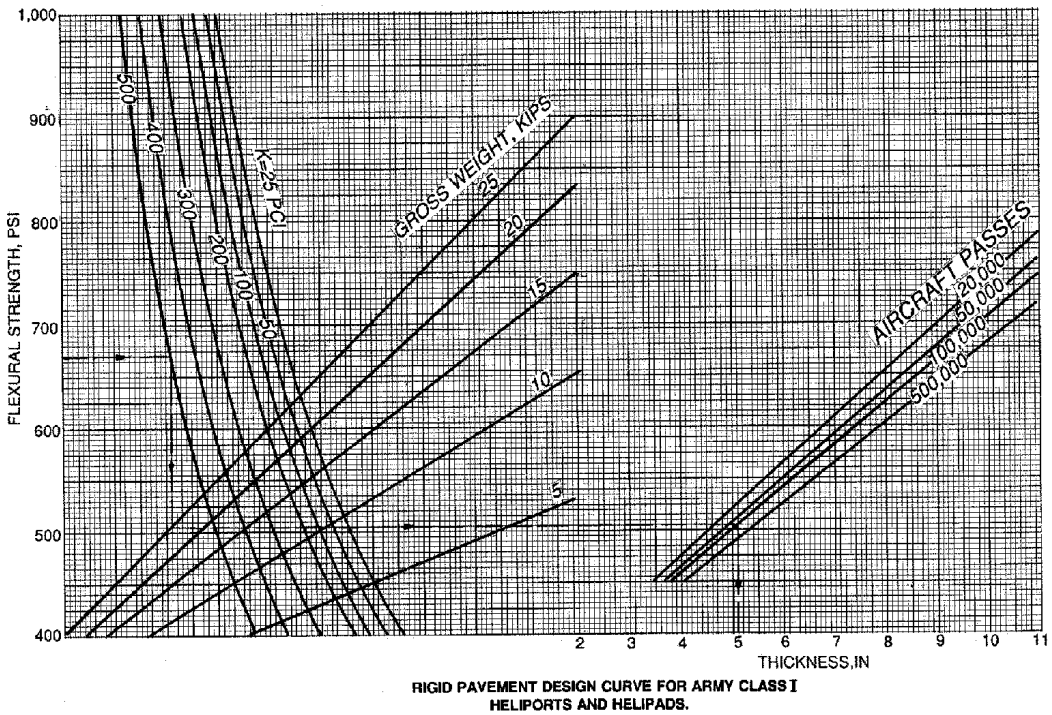
(d) The usual approach is to provide joints as shown. A doweled construction joint is installed at the intersection of the two pavements--dowels drilled and grouted into the existing PCC. This provides load transfer but no chance for translation movement. Opportunity for movement is provided by installing a thickened-edge expansion joint at a transverse joint in the new pavement. This should be just far enough back to provide a straight joint from edge to edge of the pavement (primarily to get past the end of the fillet). Note that transverse joints within the fillet area are not straight lines and would prohibit any movement along the joint.

(e) Note that the existing joints and the joints in the new area between the intersection and the expansion joint are at the same spacing. This prevents the need for any other action to prevent sympathetic cracking from any mismatched joints at the intersection. (Not always is it feasible to line up these joints, but an attempt to should be made.) At the expansion joint it is not necessary to line up joints on both sides. This permits making an easy change from the existing joint spacing to a different spacing in the new pavement.

(f) Also note that the 900-millimeter (3-foot) ends of joints intersecting curved fillets must be angled to be perpendicular to the curve at their intersection.

(5) Figure 12-43. This figure illustrates what can happen when a good jointing pattern is messed up by the joint sawing crew. This occurred on a big PCC apron at a military base. What happened was that the crew sawing transverse contraction joints (at night of course) spaced the sawed transverse joints as intended in about 85 percent of the longitudinal paving lanes, with 37 uniformly spaced joints at the left end of the apron, and 3 lesser spaced joints at the right end. But, on the other 15 percent of the longitudinal lanes, they measured the transverse joints backward, with uniform spacing at the right end and lesser spacing at the left end. Outside of the fact that there will be sympathetic cracking at the mismatched joints, structurally the pavement is excellent with a good surface finish. However, the appearance is shocking because of the mismatched lanes, and every commander that sees it will ask "Who built this queer thing."

(6) To reiterate, regardless of the shape and dimensions of the pavement to be constructed, the simplest jointing pattern, conforming to criteria, will be best.



RIGID PAVEMENT DESIGN CURVE FOR ARMY CLASS I HELIPORTS AND HELIPADS.

CONVERSION FACTORS

KILOPASCALS PER MILLIMETER = 0.271 x PCI
KILOGRAMS = 0.453 x POUNDS
MILLIMETERS = 25.4 x INCHES
MEGAPASCALS = 0.006894 x PSI

Figure 12-1. Plain concrete design curves for Army Helipads, Class I

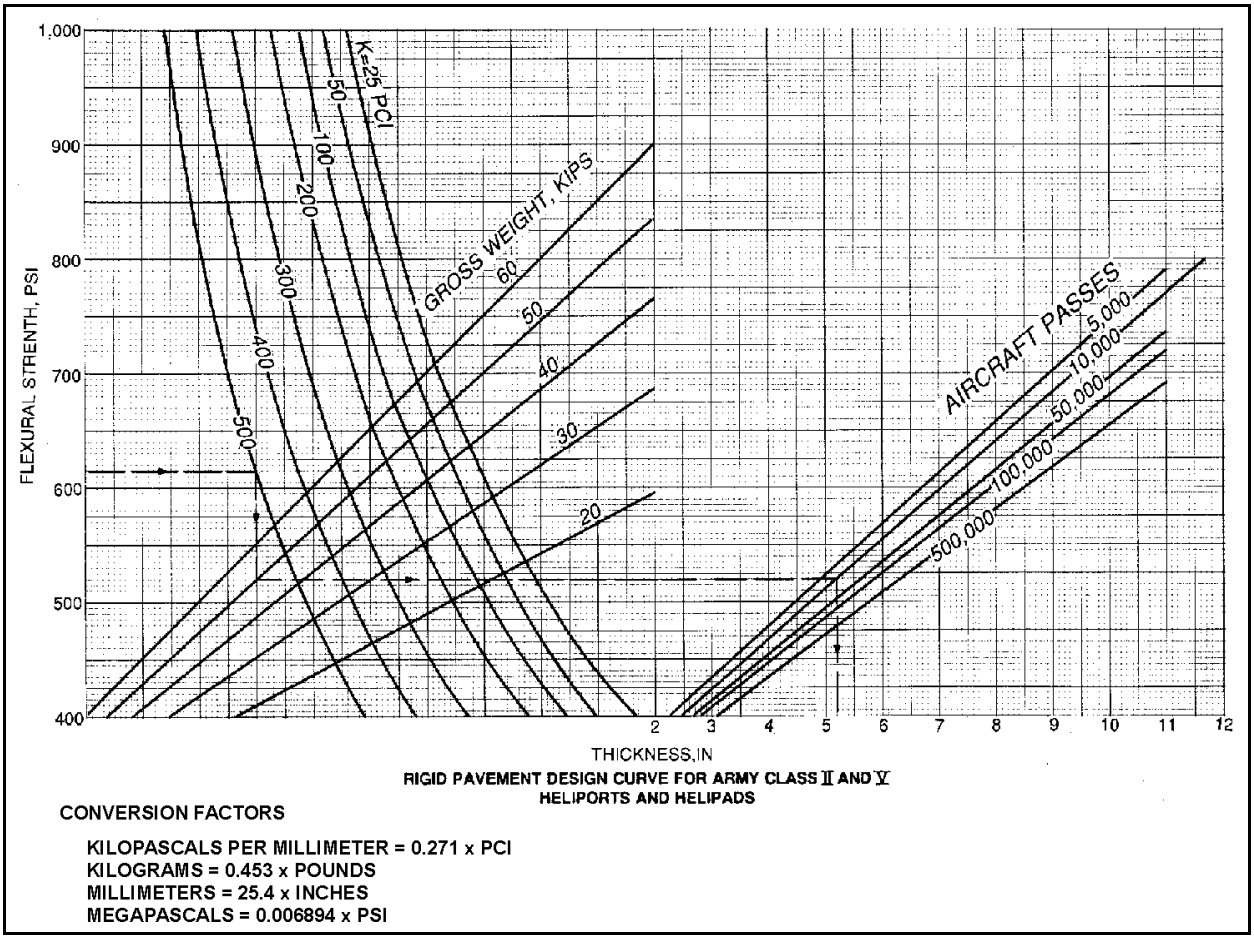


Figure 12-2. Plain concrete design curves for Army Class II airfields

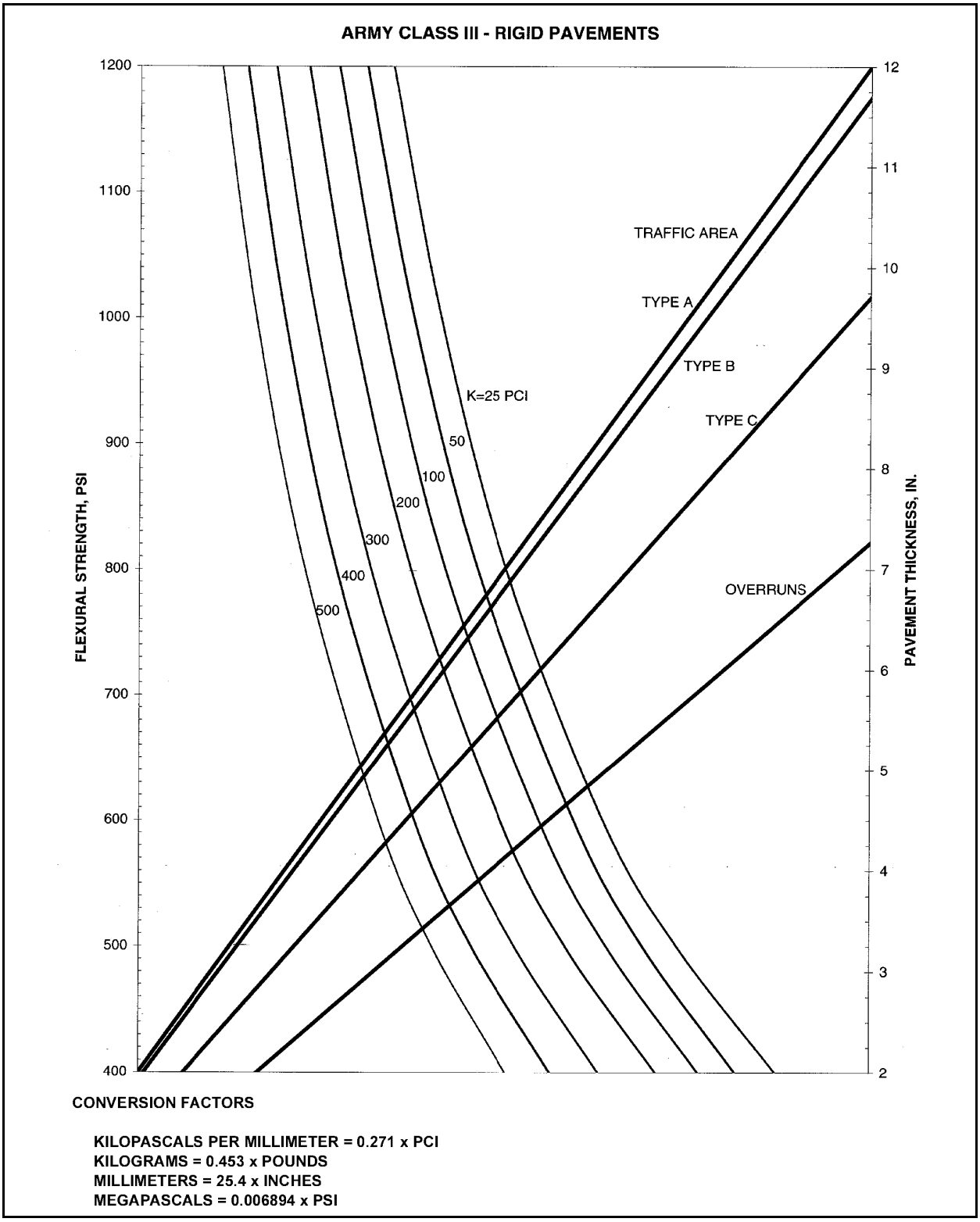


Figure 12-3. Plain concrete design curves for Army Class III airfields as defined in paragraph 4.c of Chapter 2

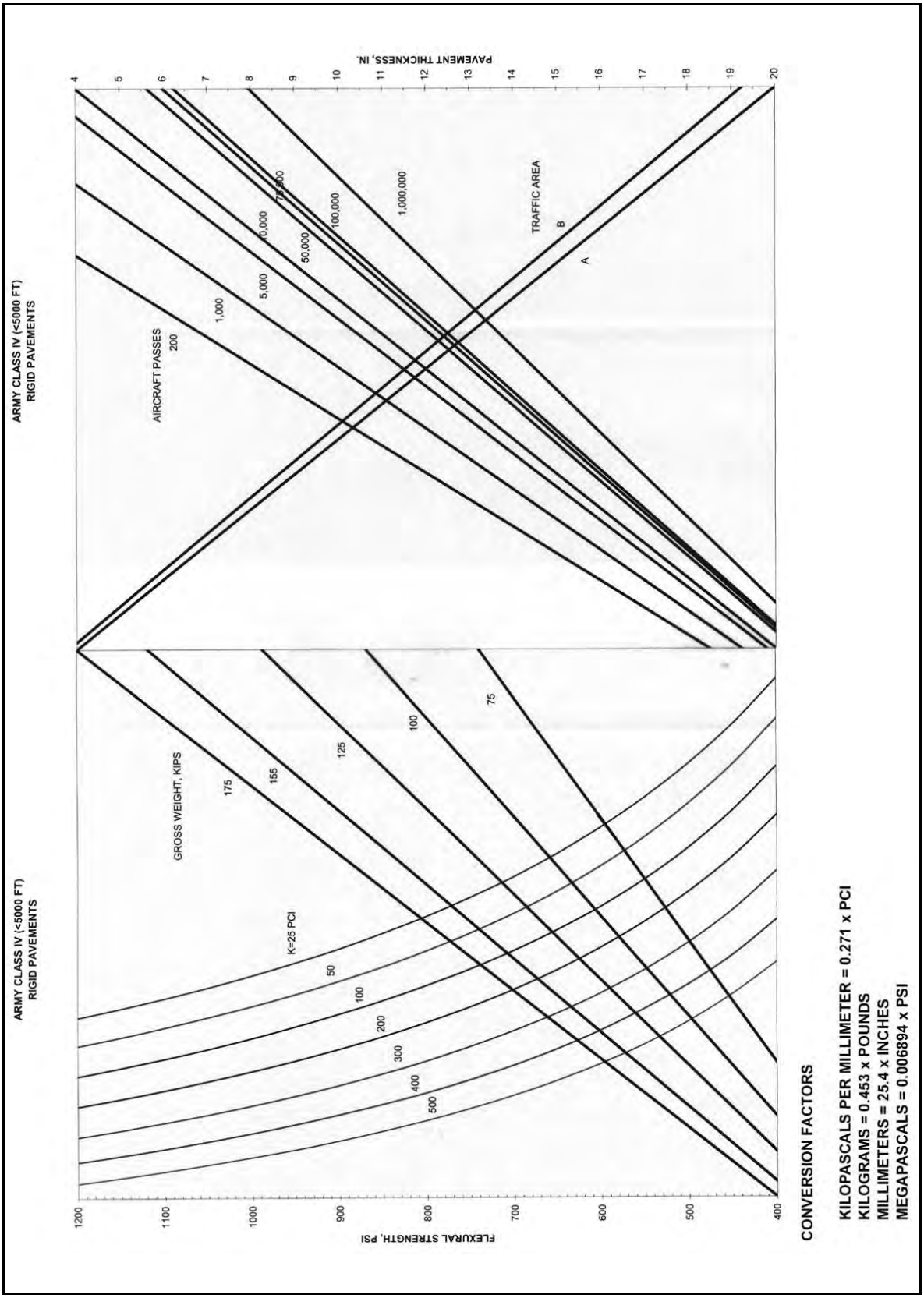


Figure 12-4. Plain concrete design curves for Army Class IV airfields (C-130 aircraft) with runway ≤ 1,525 meters (5,000 feet)

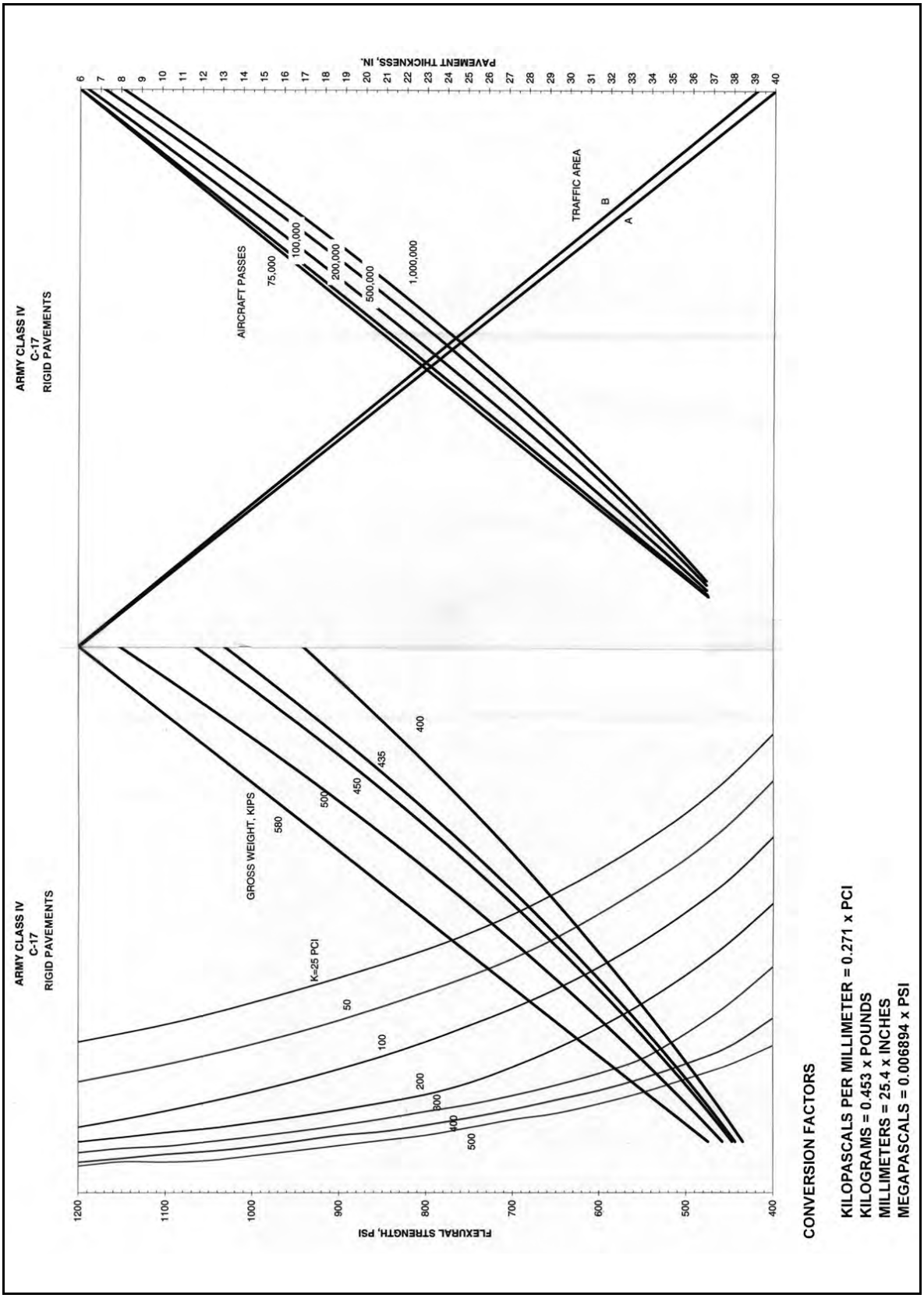


Figure 12-5. Plain concrete design curves for army Class IV airfields (C-17 aircraft) with runway > 2,745 meters (9,000 feet)

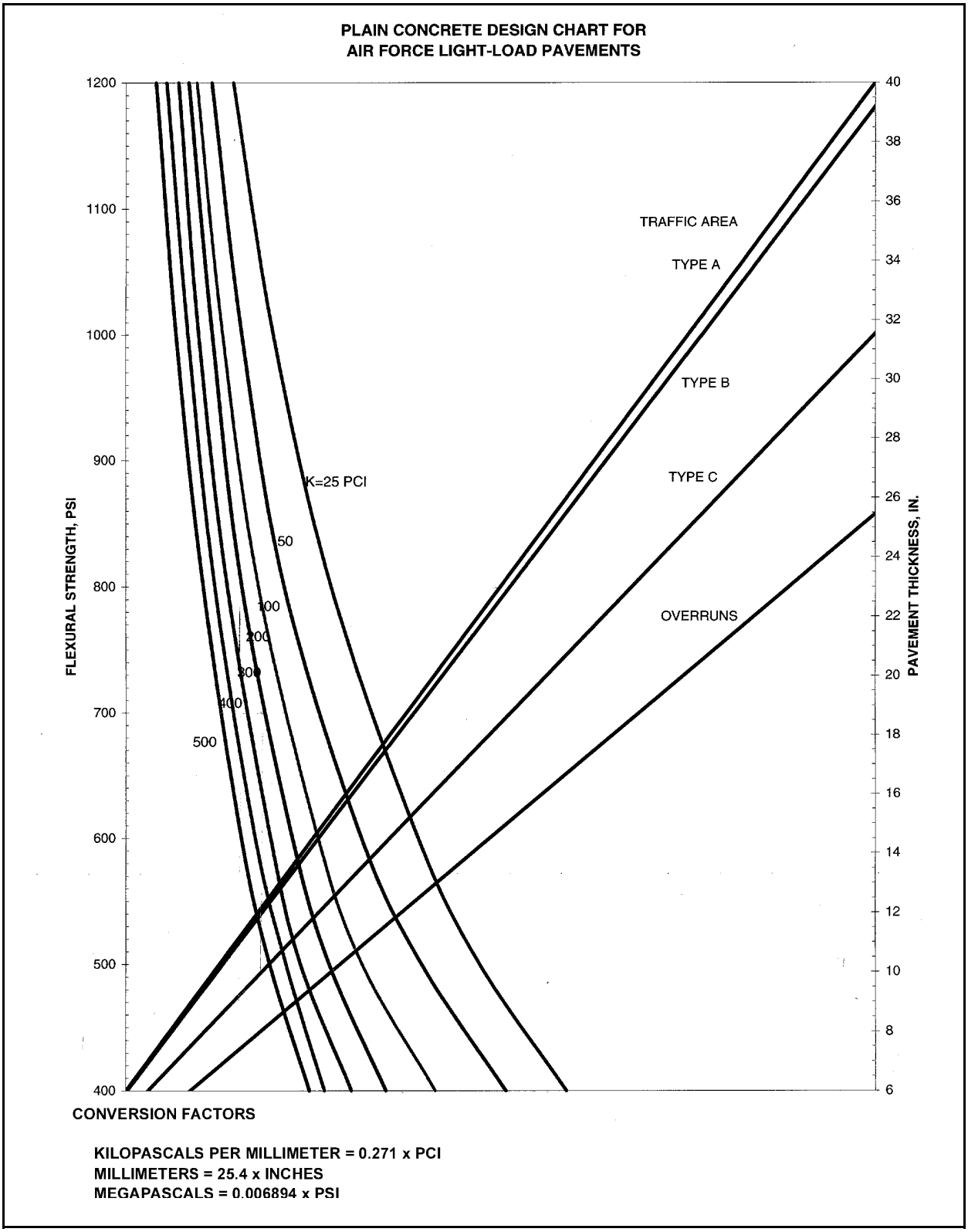


Figure 12-6. Plain concrete design curves for Air Force light-load pavements

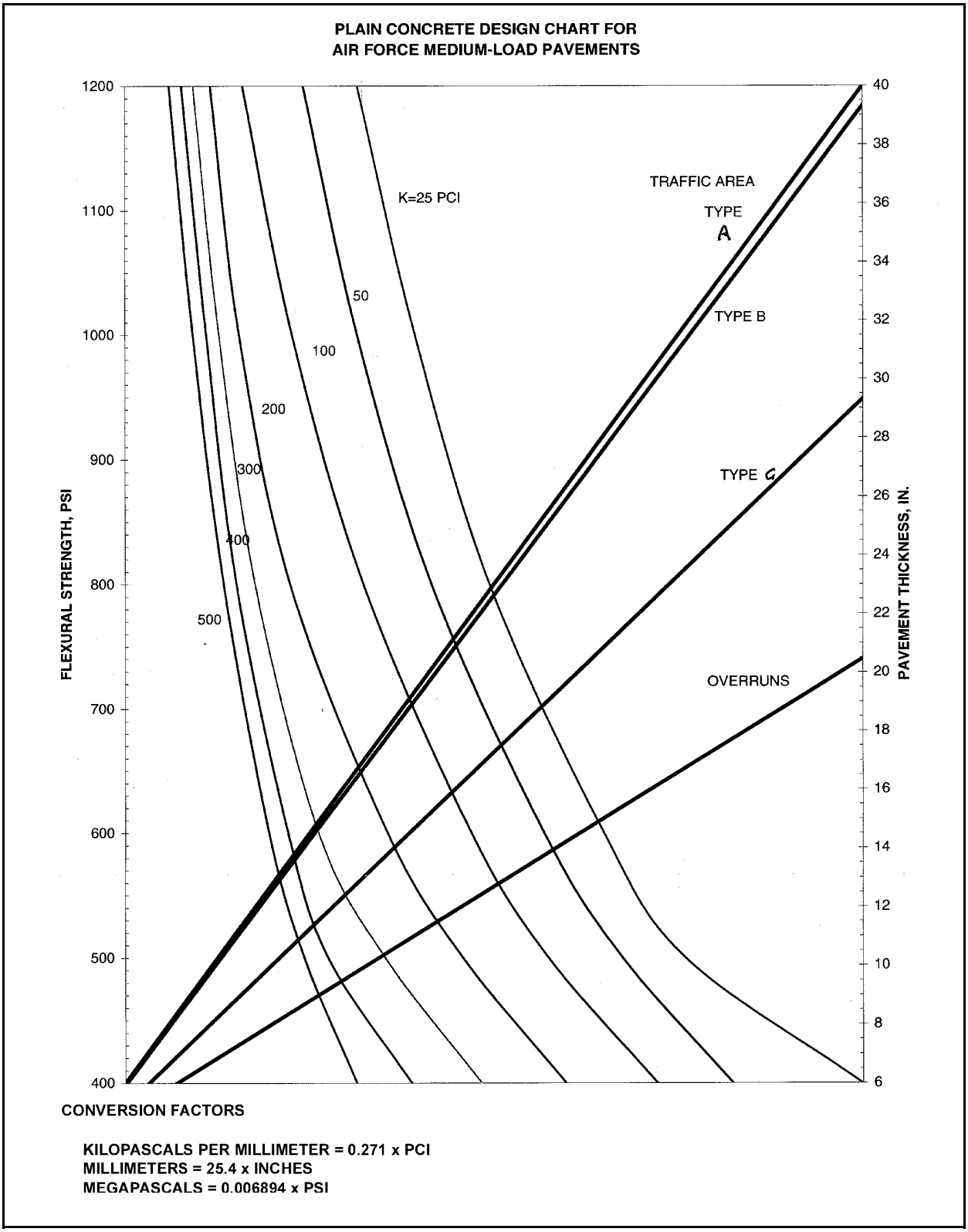


Figure 12-7. Plain concrete design curves for Air Force medium-load pavements

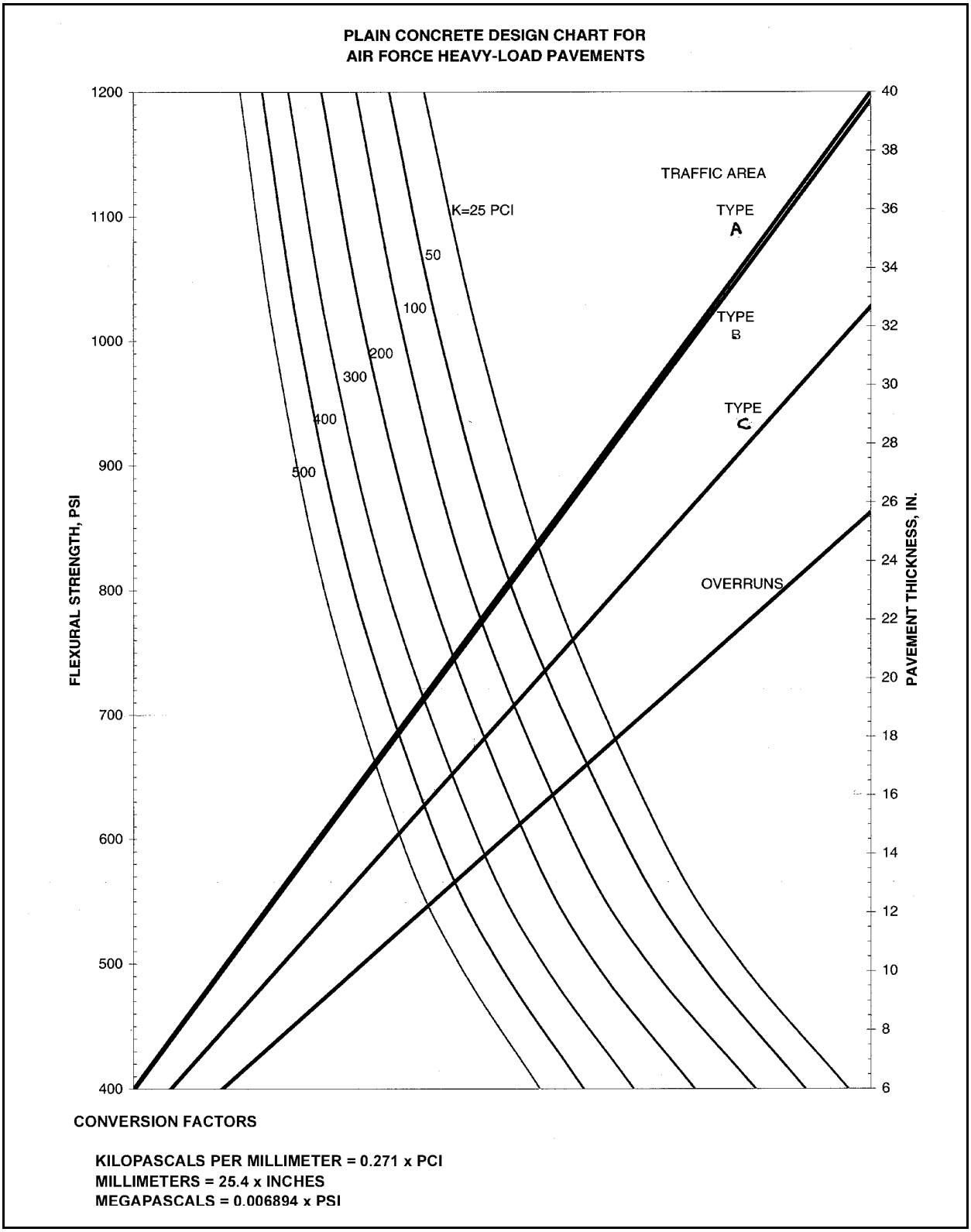


Figure 12-8. Plain concrete design curves for Air Force heavy-load pavements

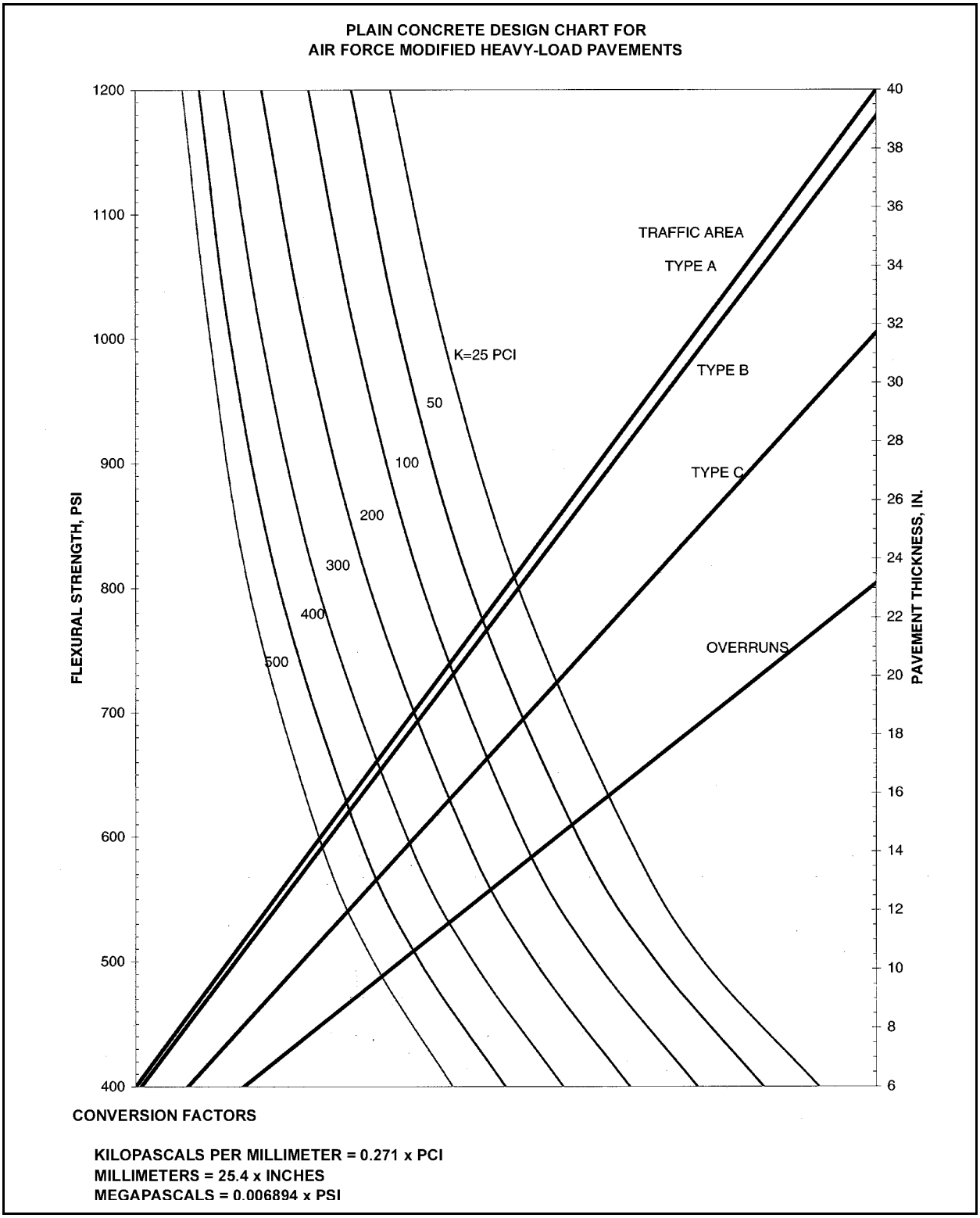


Figure 12-9. Plain concrete design curves for Air Force modified heavy-load pavements

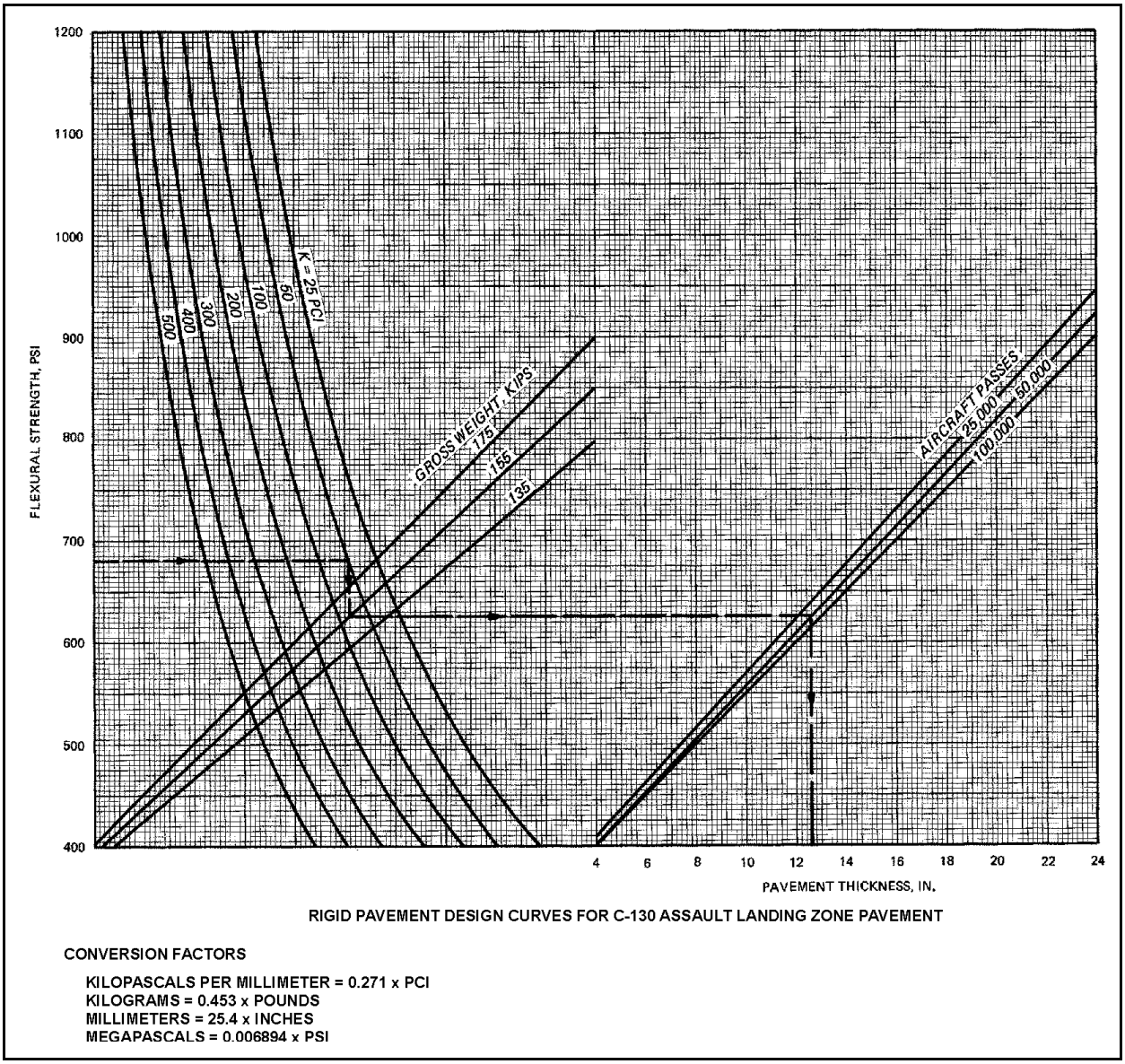


Figure 12-10. Plain concrete design curves for Air Force C-130 assault landing zone pavements

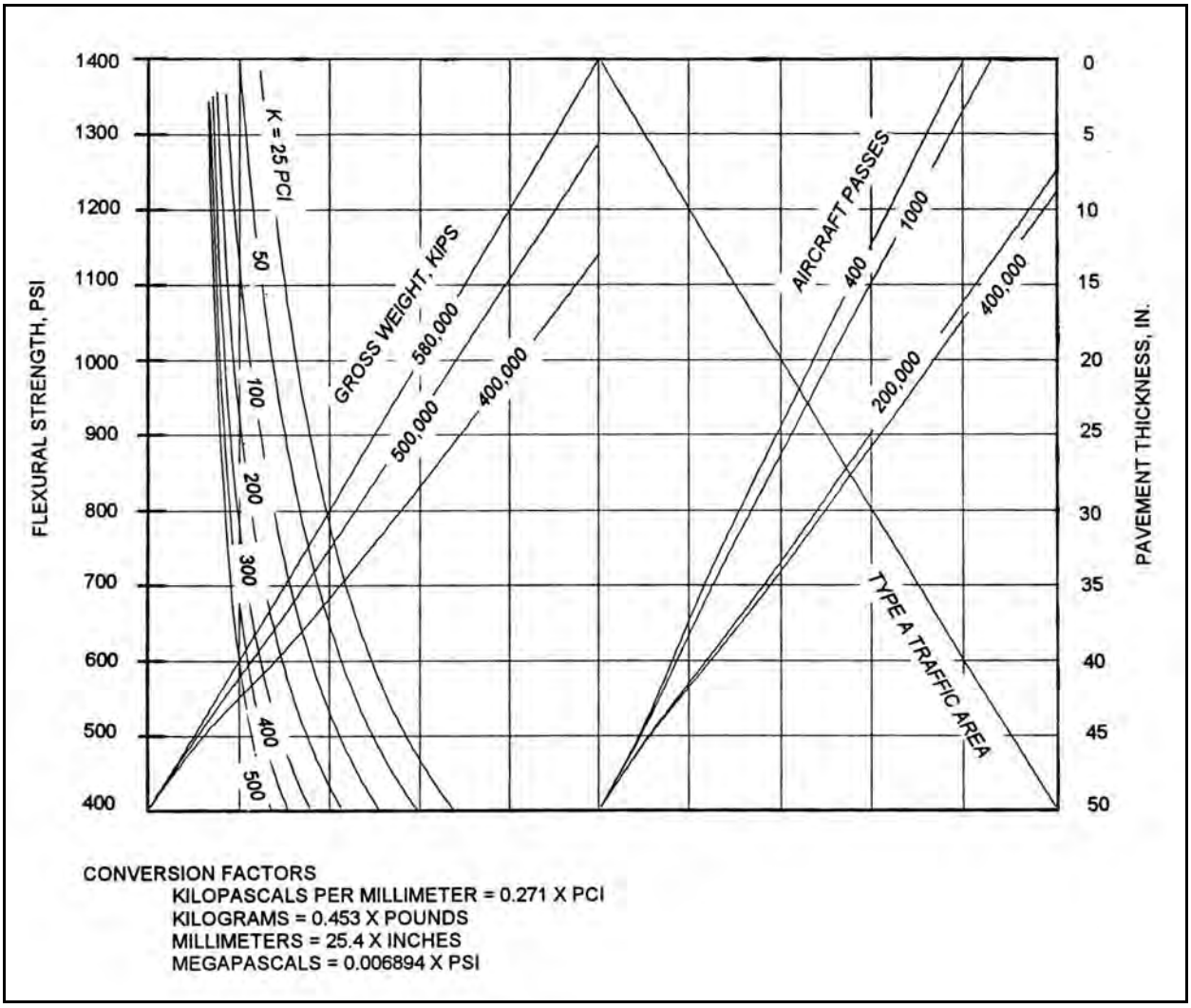
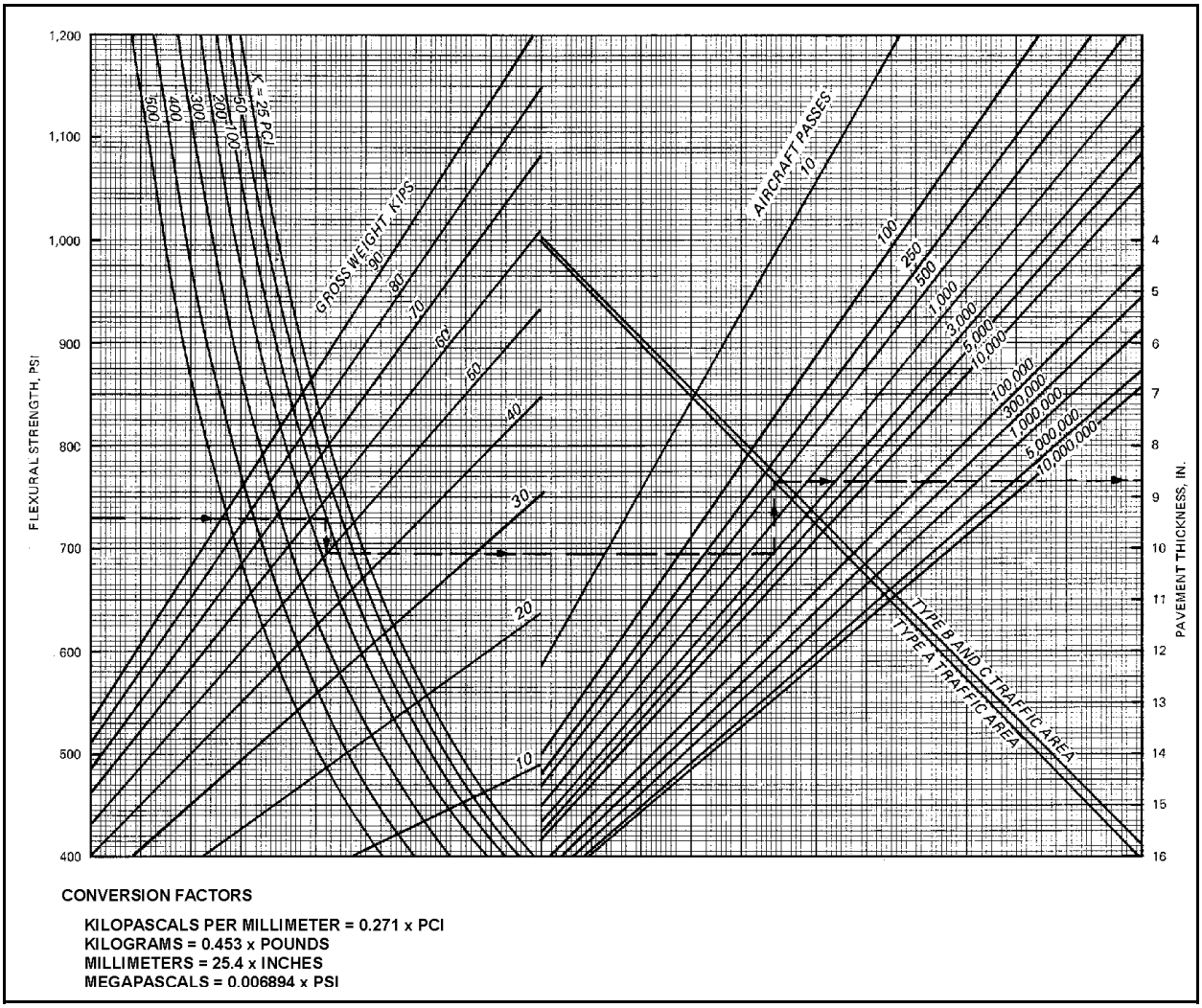


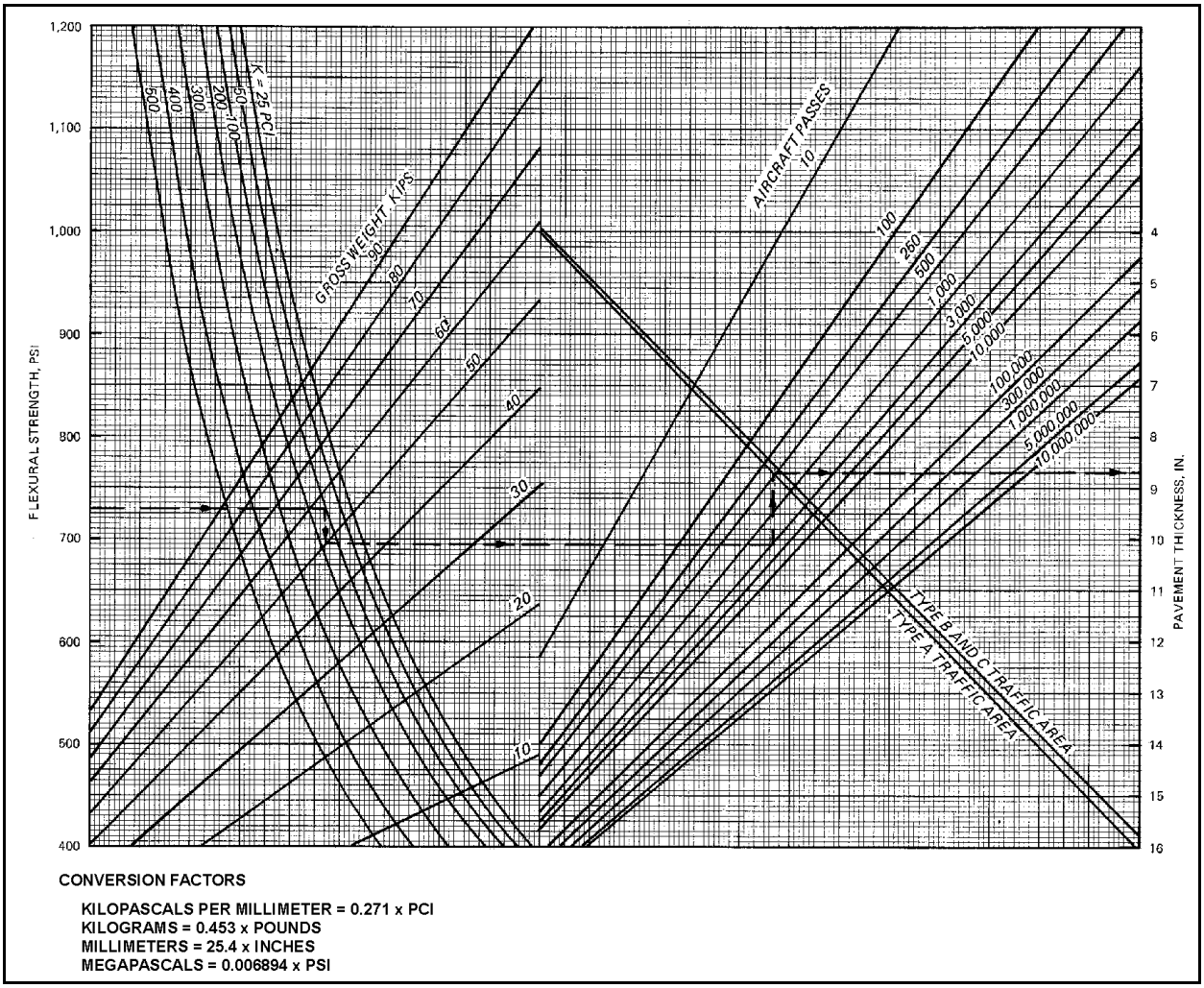
Figure 12-11. Plain concrete design curves for Air Force C-17 assault landing zone



CONVERSION FACTORS

KILOPASCALS PER MILLIMETER = 0.271 x PCI
KILOGRAMS = 0.453 x POUNDS
MILLIMETERS = 25.4 x INCHES
MEGAPASCALS = 0.006894 x PSI

Figure 12-12. Plain concrete design curves for Air Force auxiliary pavements



CONVERSION FACTORS

KILOPASCALS PER MILLIMETER = 0.271 x PCI
KILOGRAMS = 0.453 x POUNDS
MILLIMETERS = 25.4 x INCHES
MEGAPASCALS = 0.006894 x PSI

Figure 12-13. Plain concrete design curves for F-15 aircraft

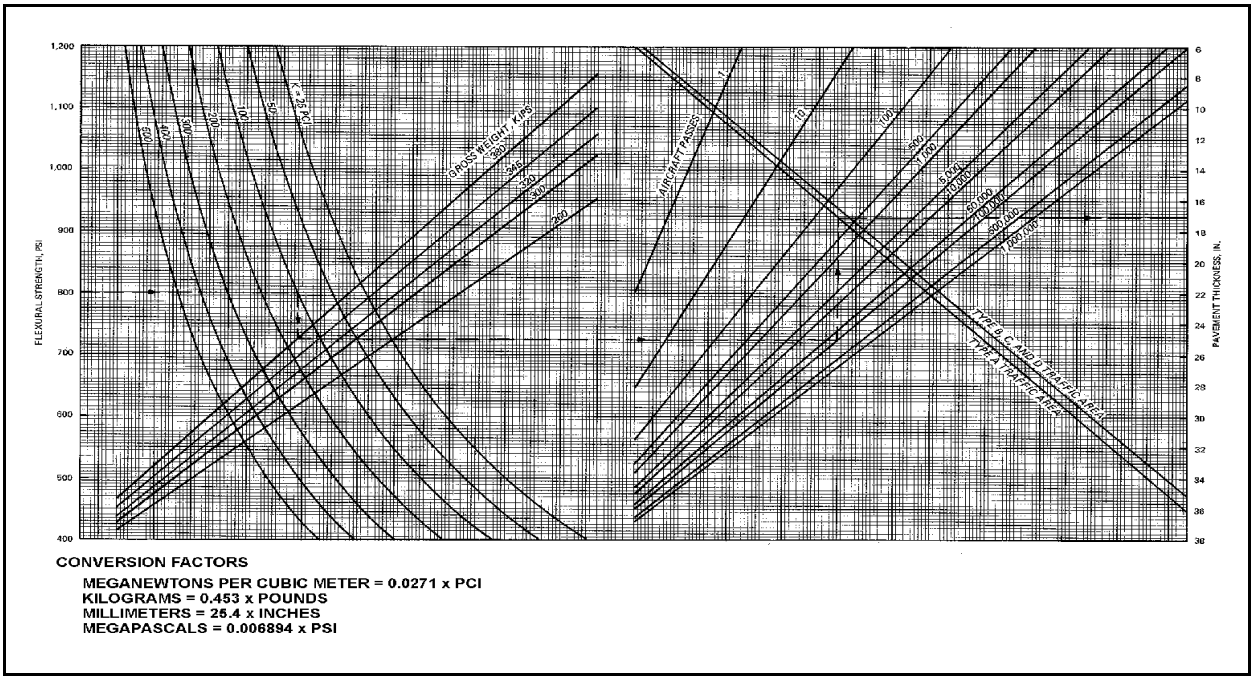


Figure 12-14. Plain concrete design curves for C-141 aircraft

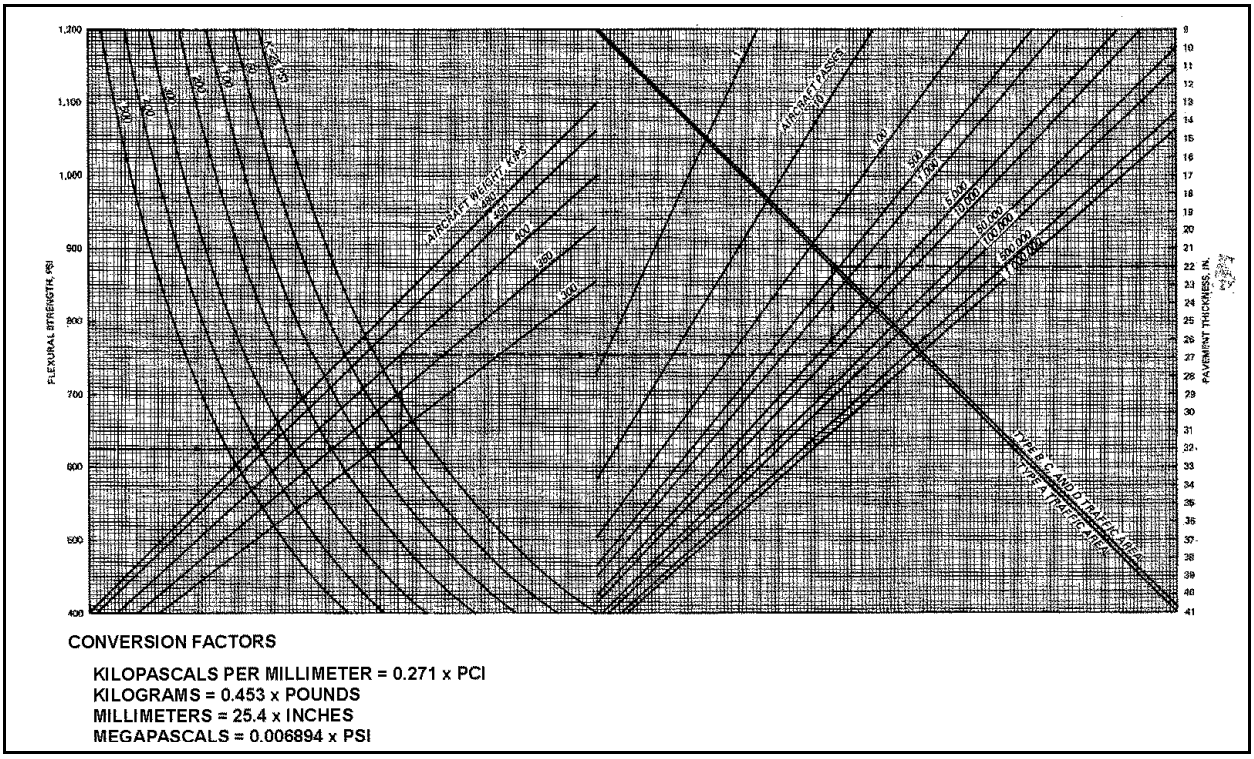
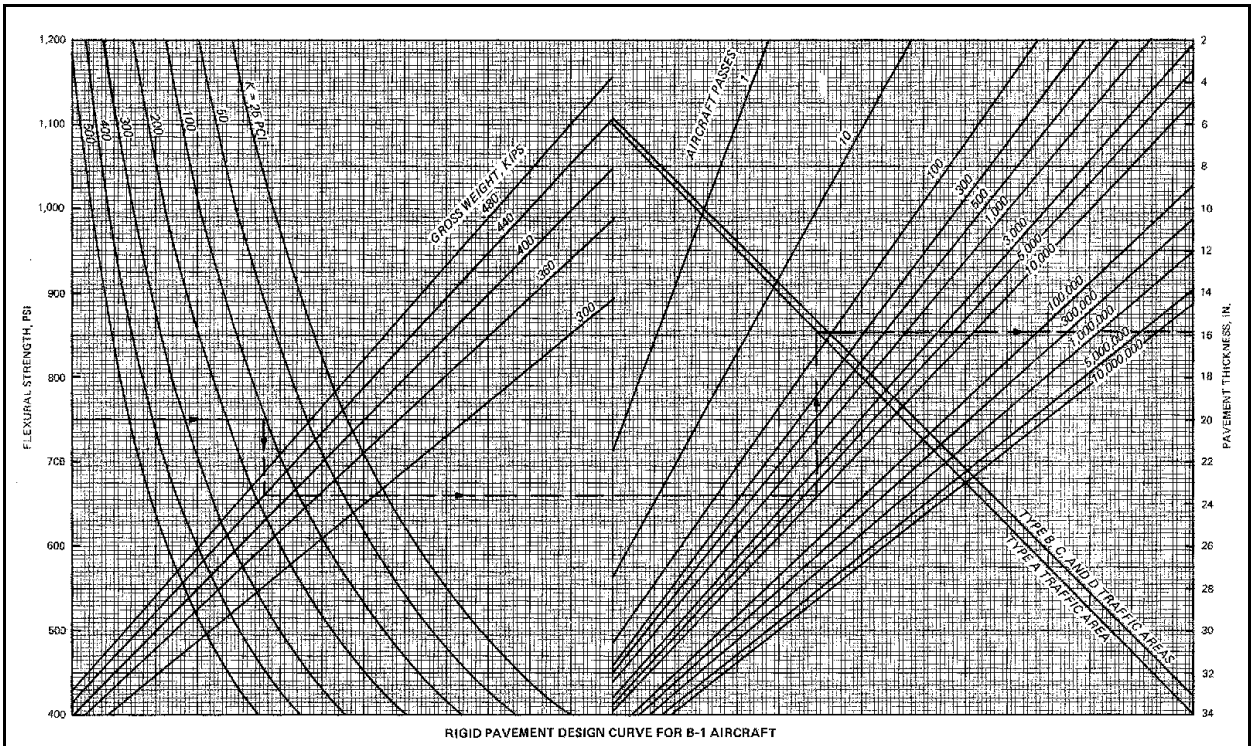


Figure 12-15. Plain concrete design curves for B-52 aircraft



CONVERSION FACTORS

KILOPASCALS PER MILLIMETER = 0.271 x PCI
KILOGRAMS = 0.453 x POUNDS
MILLIMETERS = 25.4 x INCHES
MEGAPASCALS = 0.006894 x PSI

Figure 12-16. Plain concrete design curves for B-1 aircraft

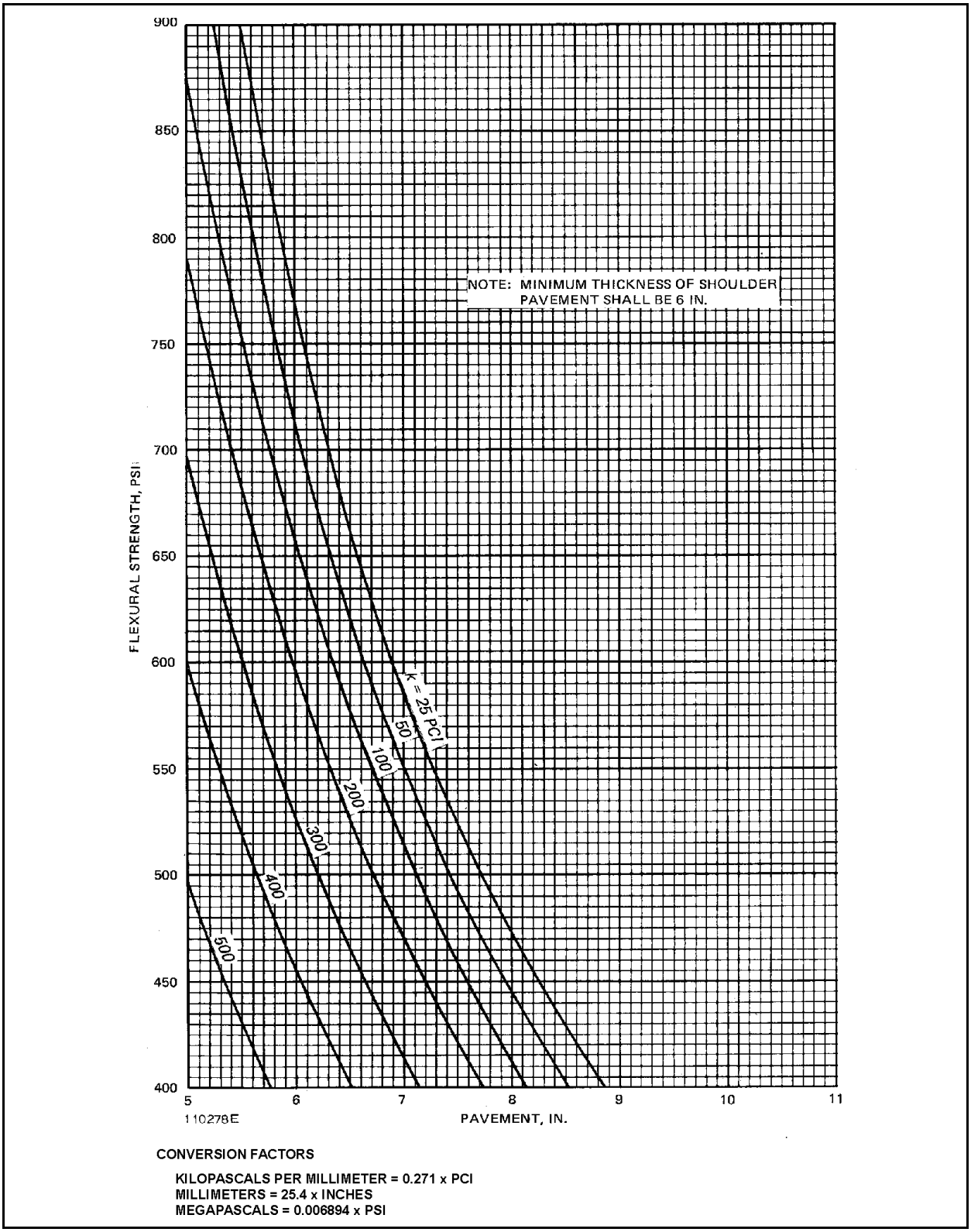


Figure 12-17. Plain concrete design curves for shoulders

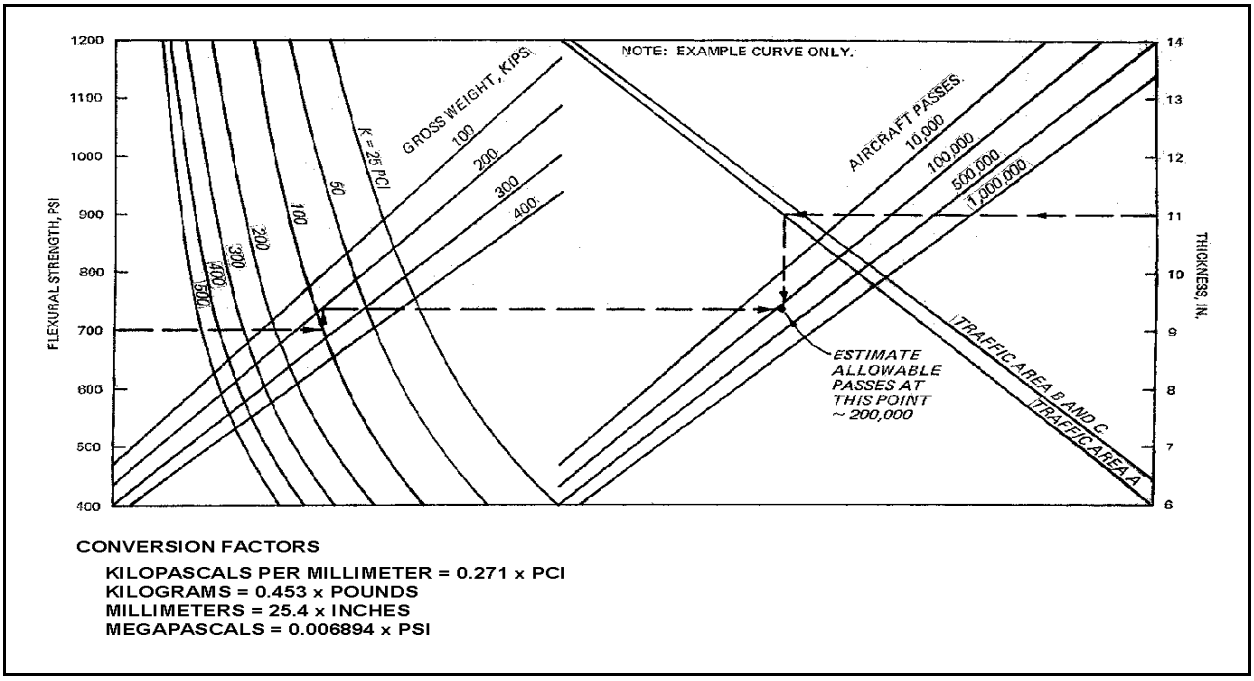


Figure 12-18. Example of allowable passes determination

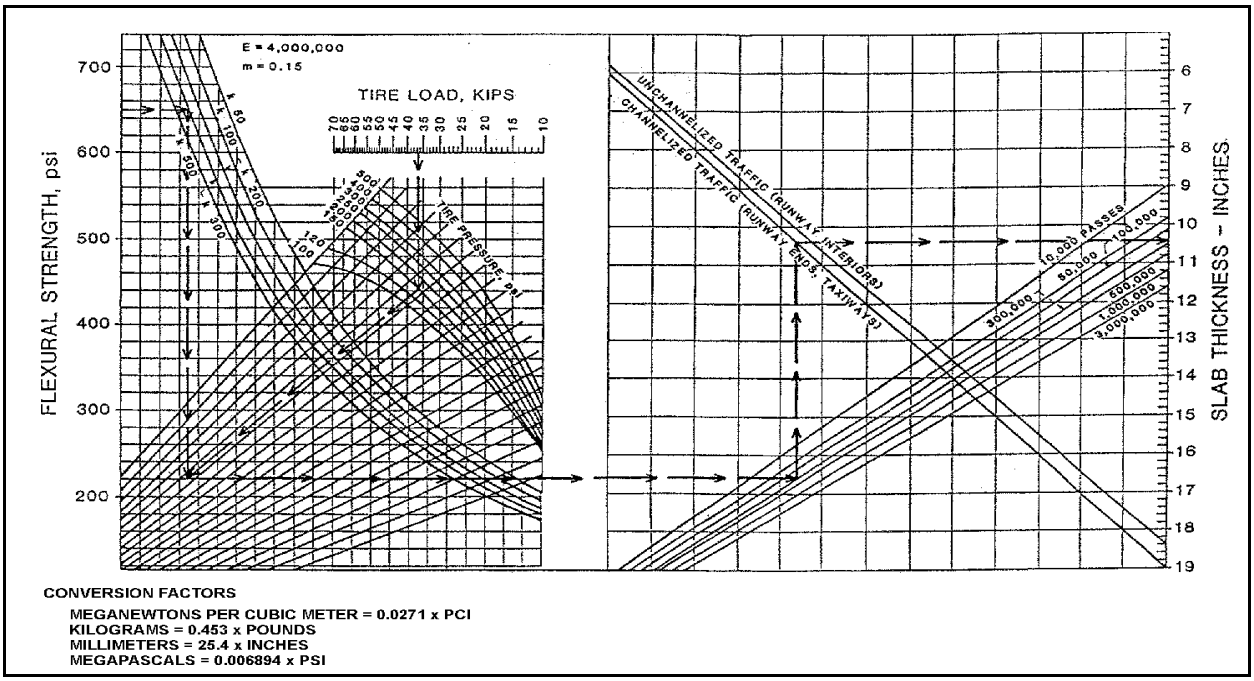


Figure 12-19. Rigid pavement thickness design chart for single-wheel load (Navy)

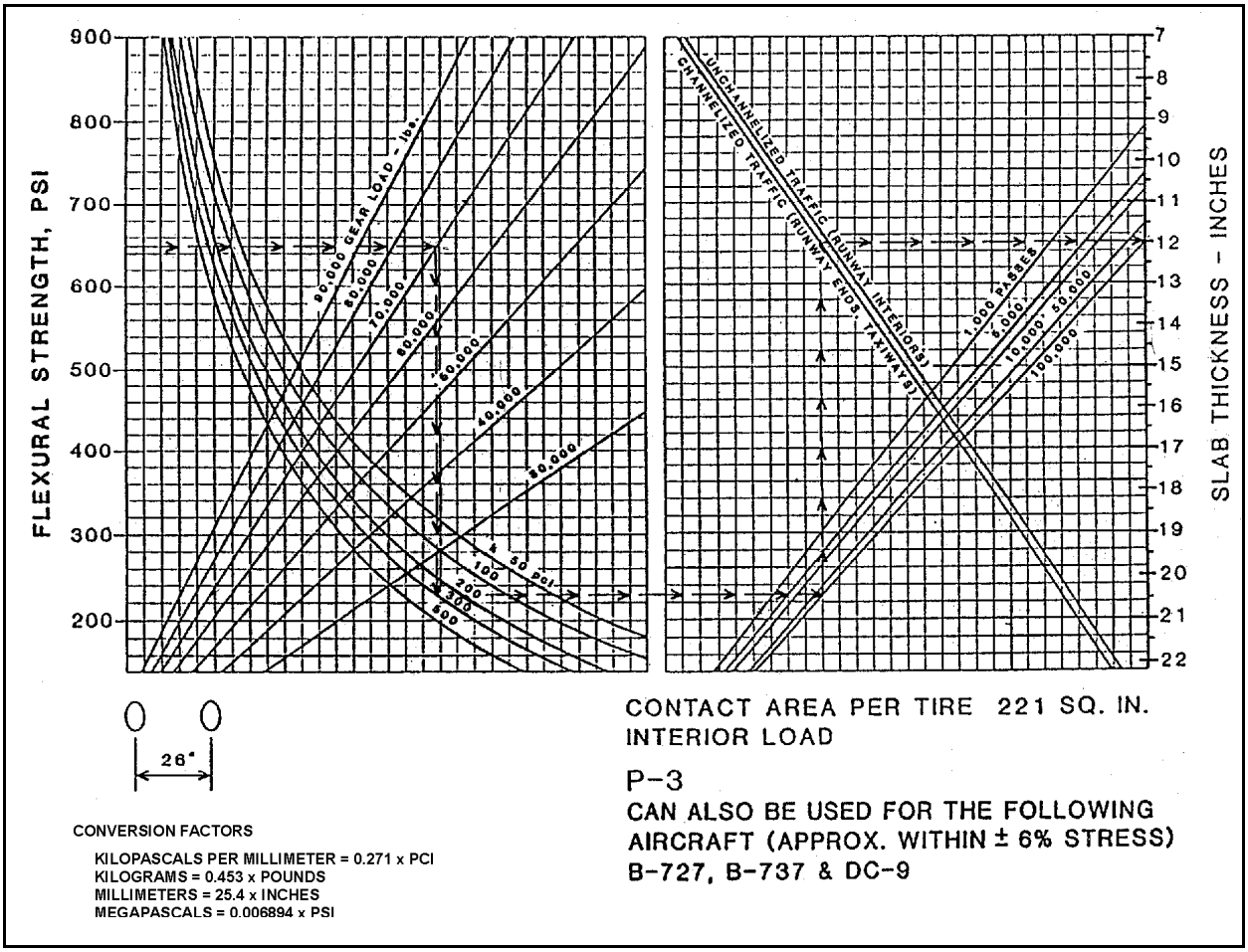


Figure 12-20. Rigid pavement thickness design chart for P-3 aircraft (Navy)

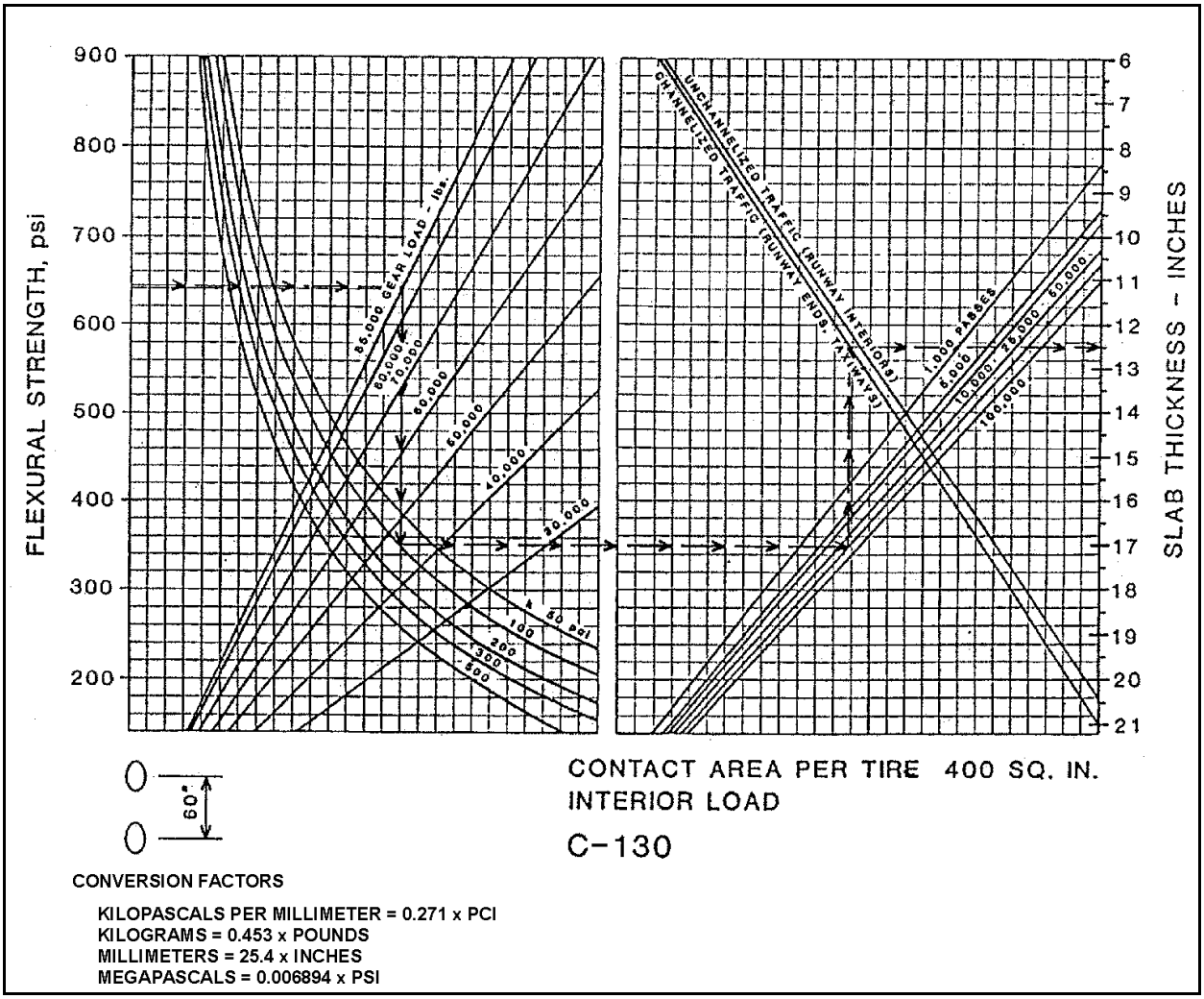


Figure 12-21. Rigid pavement thickness design chart for C-130 aircraft (Navy)

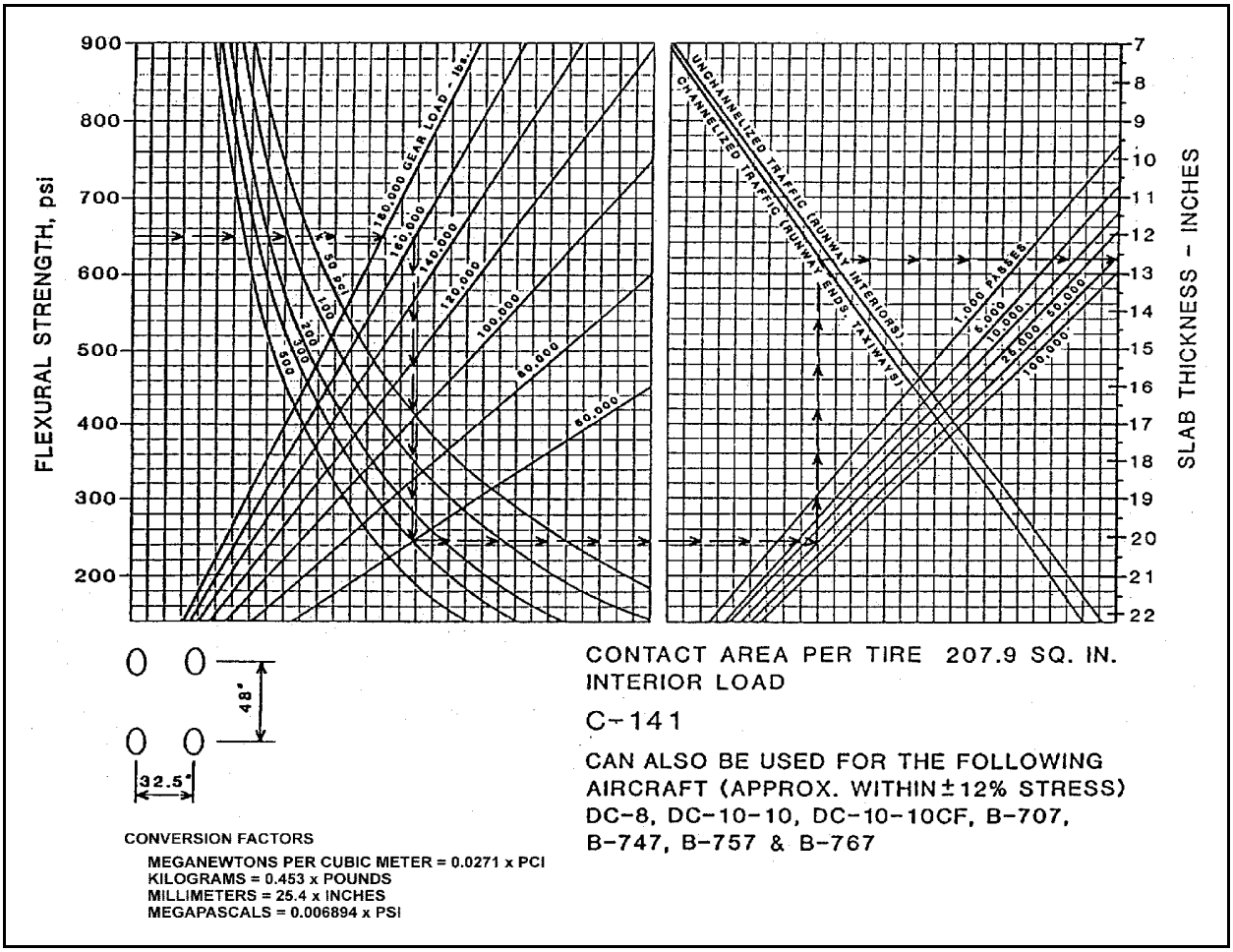


Figure 12-22. Rigid pavement thickness design chart for C-141 aircraft (Navy)

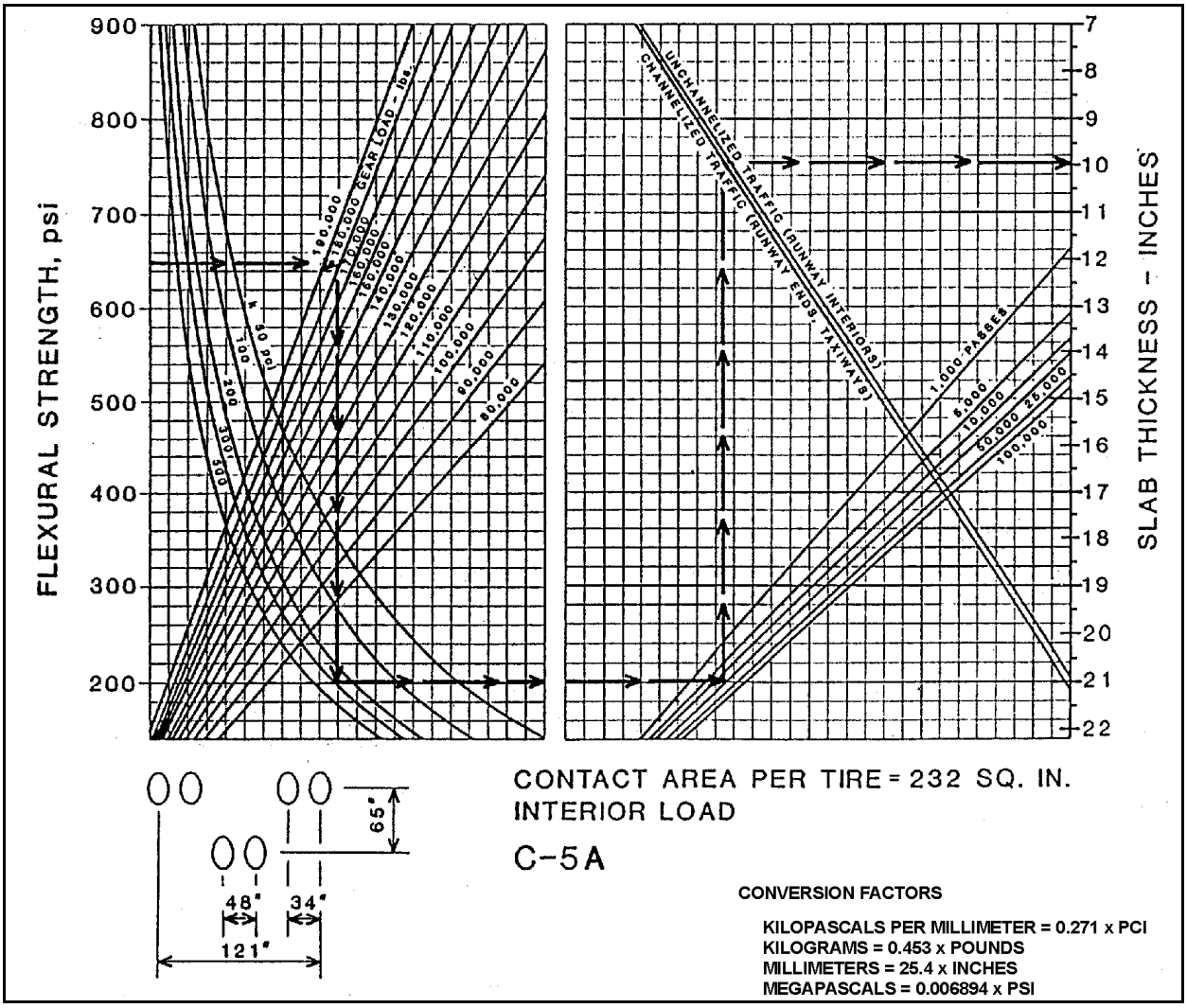


Figure 12-23. Rigid pavement thickness design chart for C-5A aircraft (Navy)

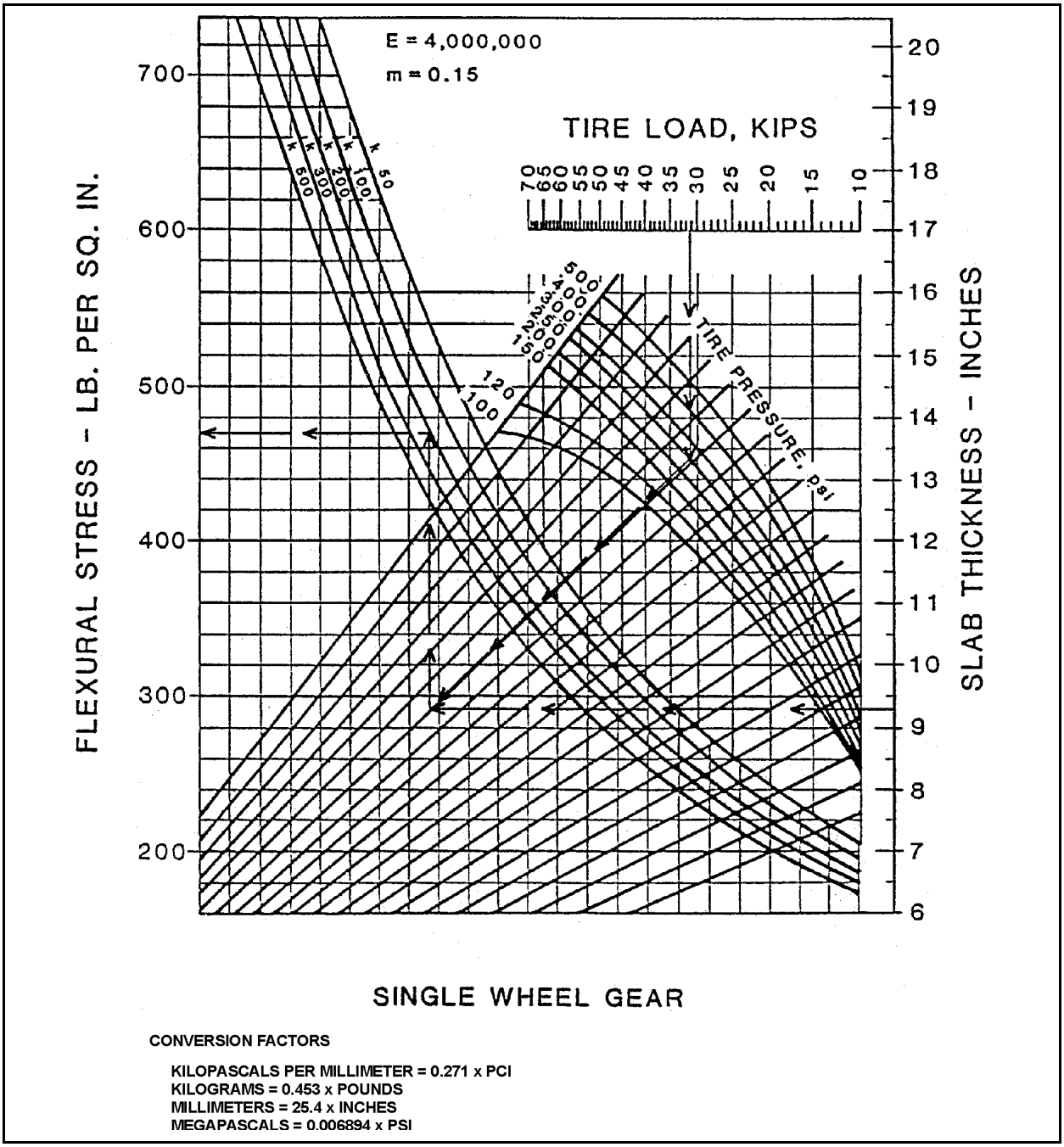


Figure 12-24. Chart for determining flexural stress for single-wheel gear (Navy)

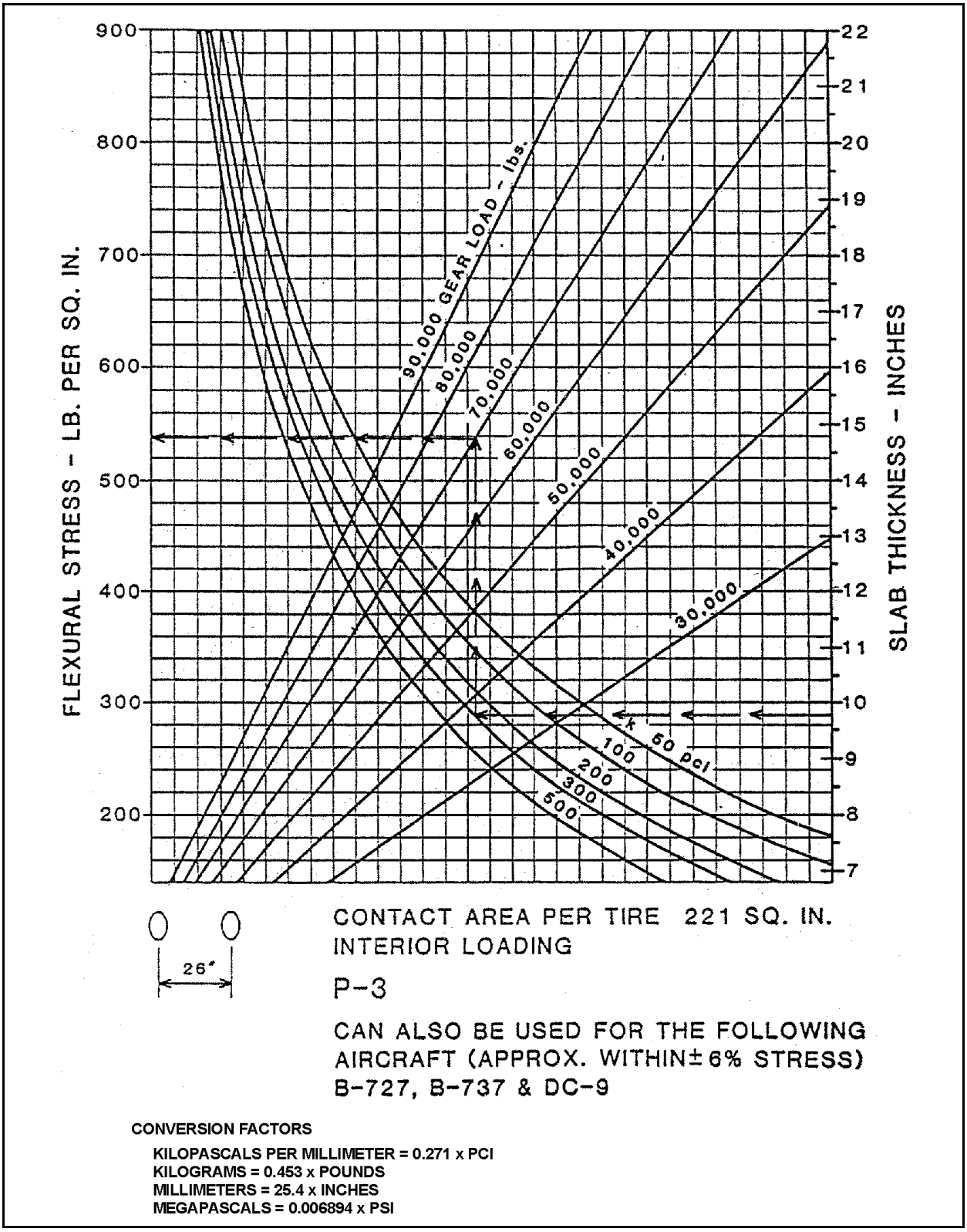


Figure 12-25. Chart for determining flexural stress for P-3 aircraft (Navy)

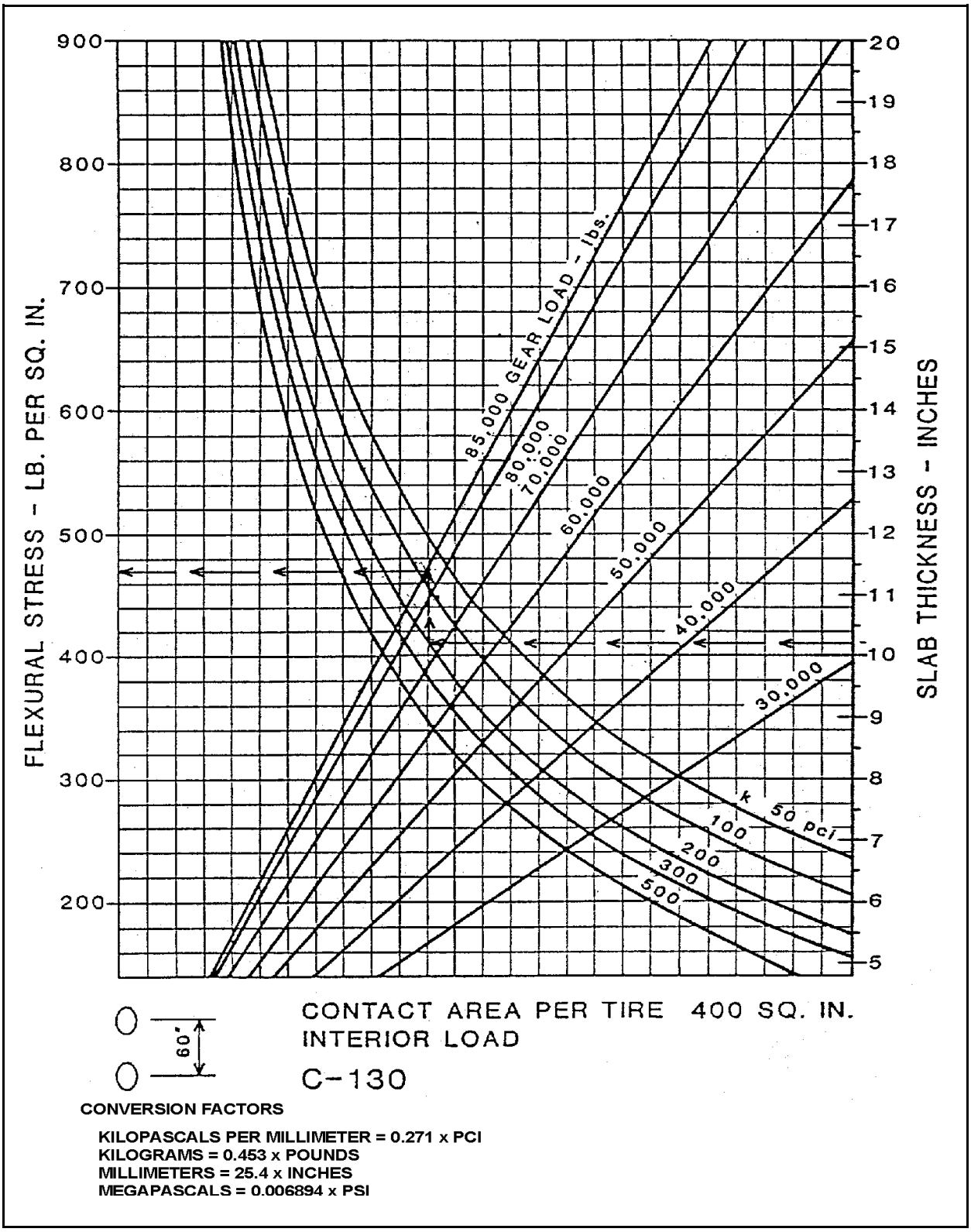
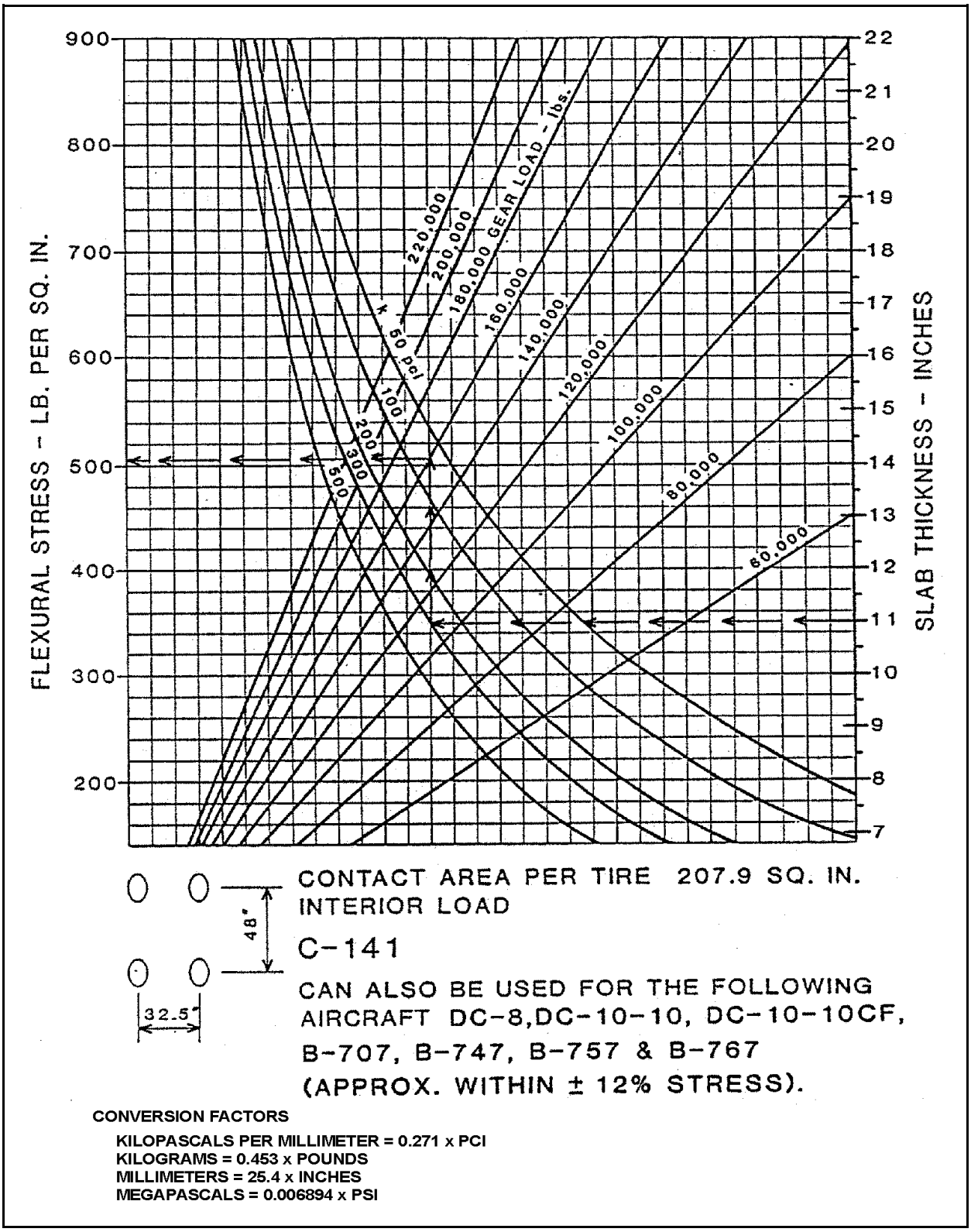


Figure 12-26. Chart for determining flexural stress for C-130 aircraft (Navy)



CONVERSION FACTORS

KILOPASCALS PER MILLIMETER = 0.271 x PCI
KILOGRAMS = 0.453 x POUNDS
MILLIMETERS = 25.4 x INCHES
MEGAPASCALS = 0.006894 x PSI

Figure 12-27. Chart for determining flexural stress for C-141 aircraft (Navy)

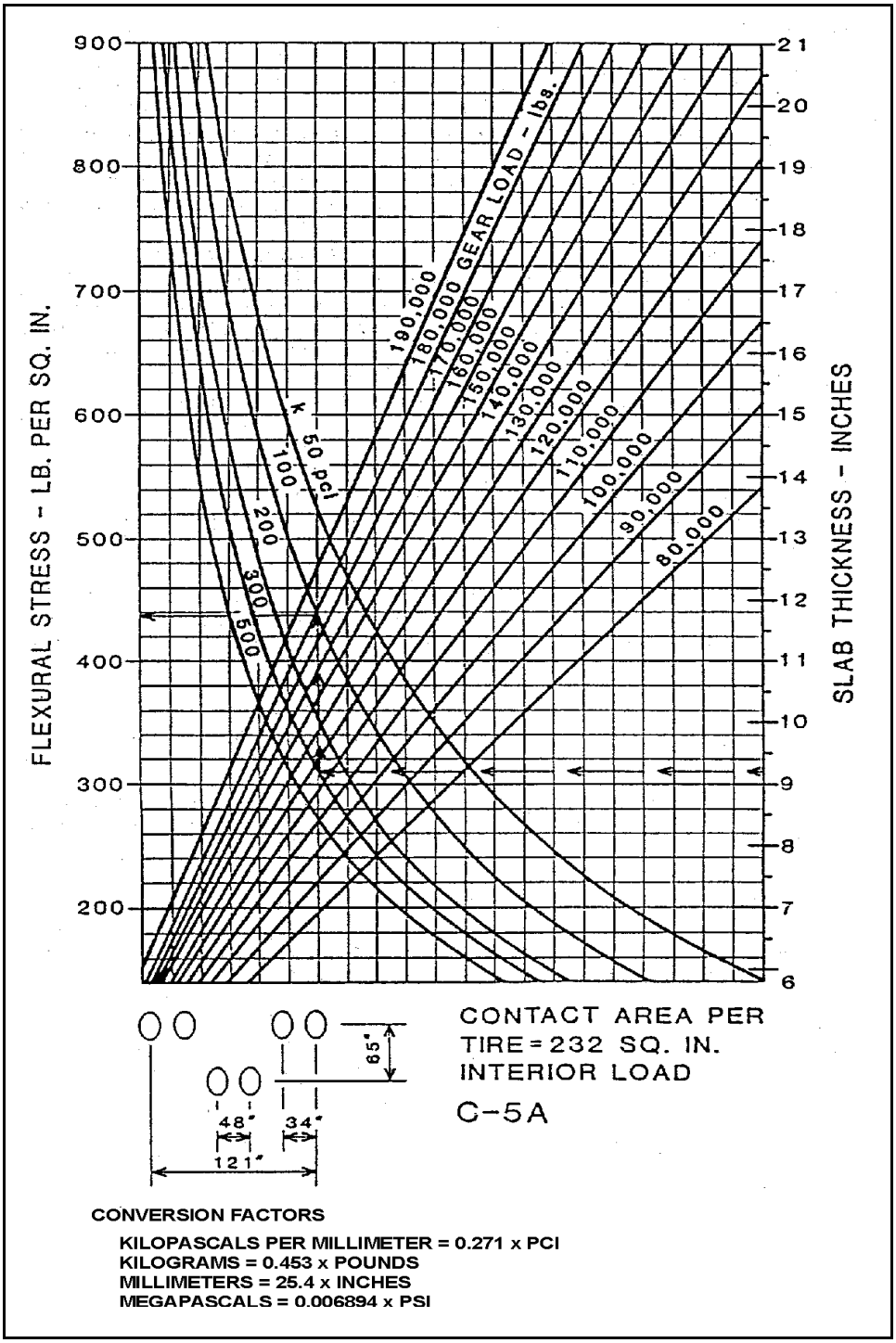


Figure 12-28. Chart for determining flexural stress for C-5A aircraft (Navy)

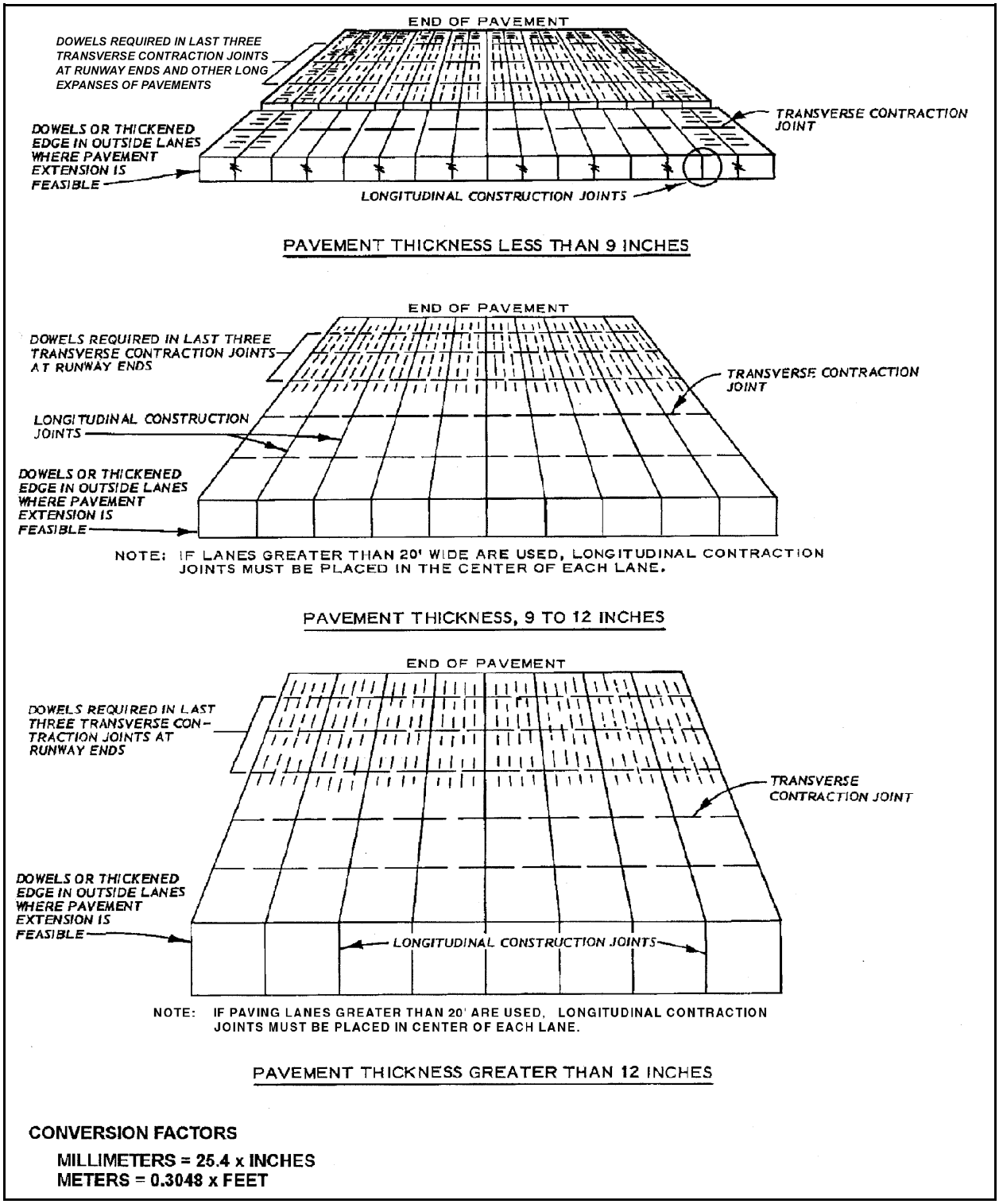


Figure 12-29. Typical jointing

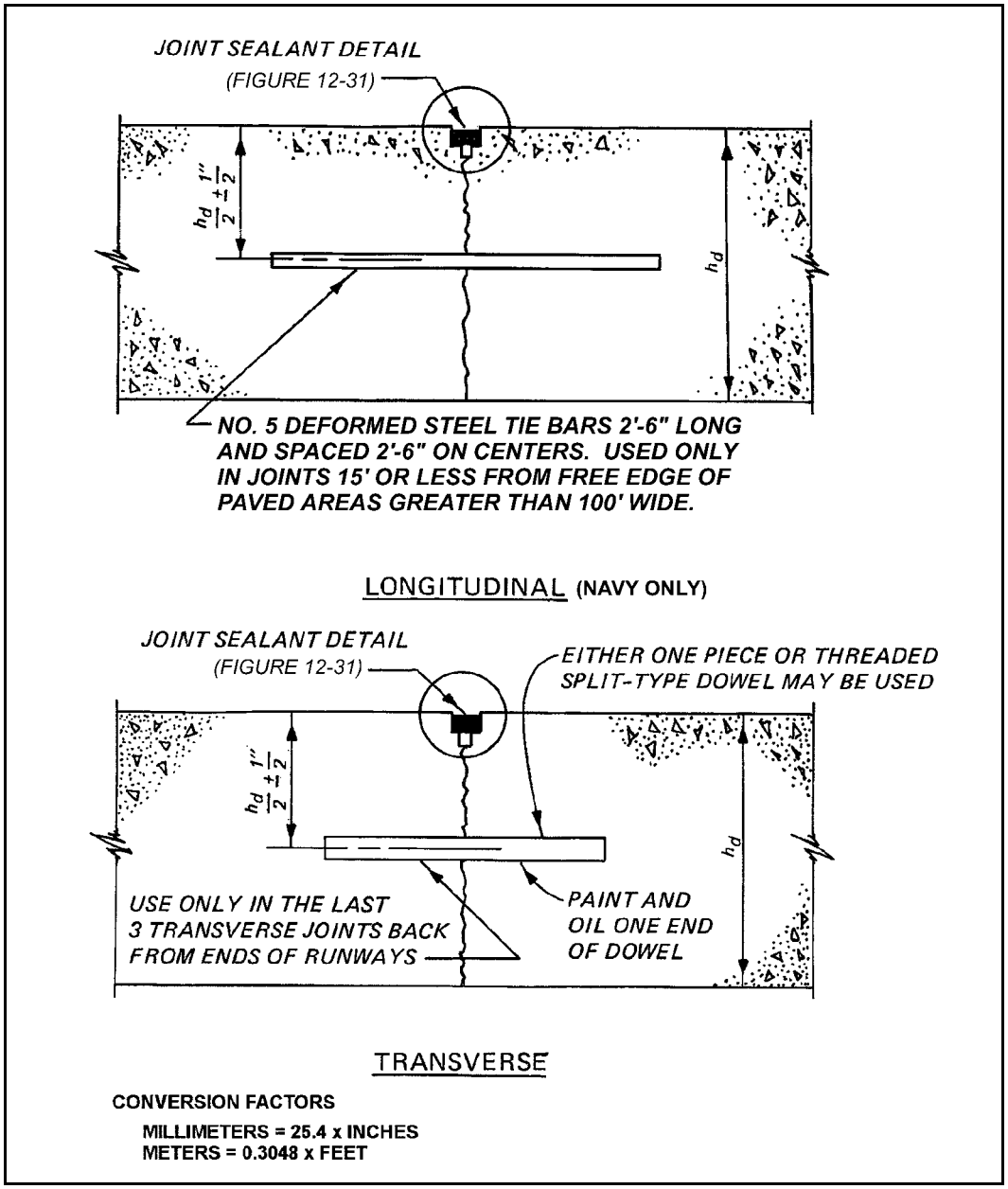
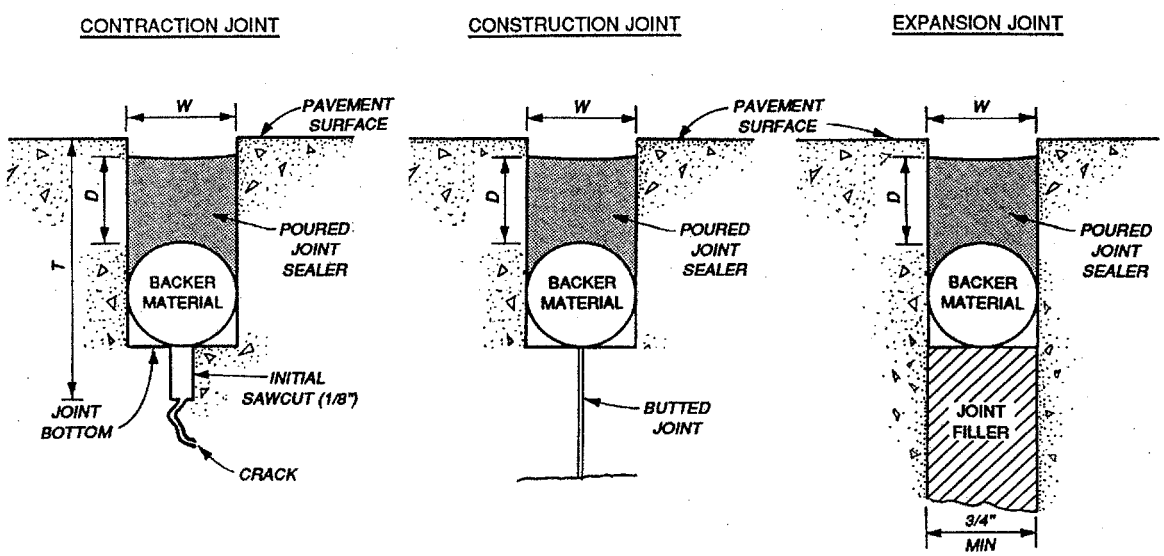


Figure 12-30. Contraction joints for plain concrete pavements



W = WIDTH OF SEALANT RESERVOIR (SEE TABLE)
D = DEPTH OF SEALANT (1.0 TO 1.5 x W) FOR POURED SEALANT; 0.5 W FOR SILICONE SEALANT
T = DEPTH OF INITIAL SAWCUT TYPE JOINT FORMER (CONTRACTION JOINT)

a. 1/4 SLAB THICKNESS ± 1/2" FOR PAVEMENTS LESS THAN 12 INCHES
b. 3 INCHES FOR PAVEMENTS 12-18 INCHES*
c. 1/6 SLAB THICKNESS FOR PAVEMENTS MORE THAN 18 INCHES*
d. DIAMETER OF BACKER MATERIAL = W + 1/8"

TABLE

JOINT SPACING FT	WIDTH, IN.	
	MIN	MAX
20	1/2	5/8
20 (NAVY)	3/8	--

*DESIGNER MAY WANT TO CONSIDER REQUIRING 1/4 SLAB THICKNESS

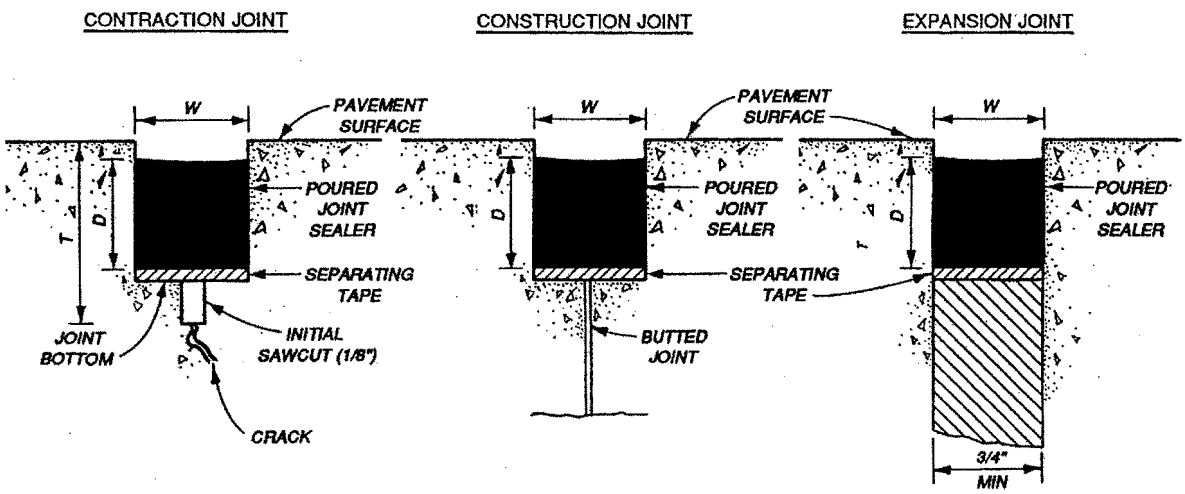
NOTE: TOP OF SEALANT SHALL BE 1/4-IN. TO 1/16 IN. BELOW TOP OF PAVEMENT. IN AREAS TO BE GROOVED, THE JOINT SEAL SHALL BE RECESSED BELOW THE DEPTH OF THE GROOVES.

CONVERSION FACTORS

METERS = 0.3048 X FEET
MILLIMETERS = 25.4 X INCHES

POURED JOINT SEALANT DETAILS (BACKER MATERIAL)

Figure 12-31. Joint sealant details for plain concrete pavements (Sheet 1 of 3)



W = WIDTH OF SEALANT RESERVOIR (SEE TABLE)
D = DEPTH OF SEALANT (1.0 TO 1.5 x W) FOR POURED SEALANT; 0.5 W FOR SILICONE SEALANT
T = DEPTH OF INITIAL SAWCUT TYPE JOINT FORMER (CONTRACTION JOINT)

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c. 1/6 SLAB THICKNESS FOR PAVEMENTS MORE THAN 18 INCHES*

*DESIGNER MAY WANT TO CONSIDER REQUIRING 1/4 SLAB THICKNESS

TABLE

JOINT SPACING FT	WIDTH, IN.	
	MIN	MAX
20	1/2	5/8
20 (NAVY)	3/8	--

NOTE: TOP OF SEALANT SHALL BE 1/4-IN. TO 1/16 IN. BELOW TOP OF PAVEMENT. IN AREAS TO BE GROOVED, THE JOINT SEAL SHALL BE RECESSED BELOW THE DEPTH OF THE GROOVES.

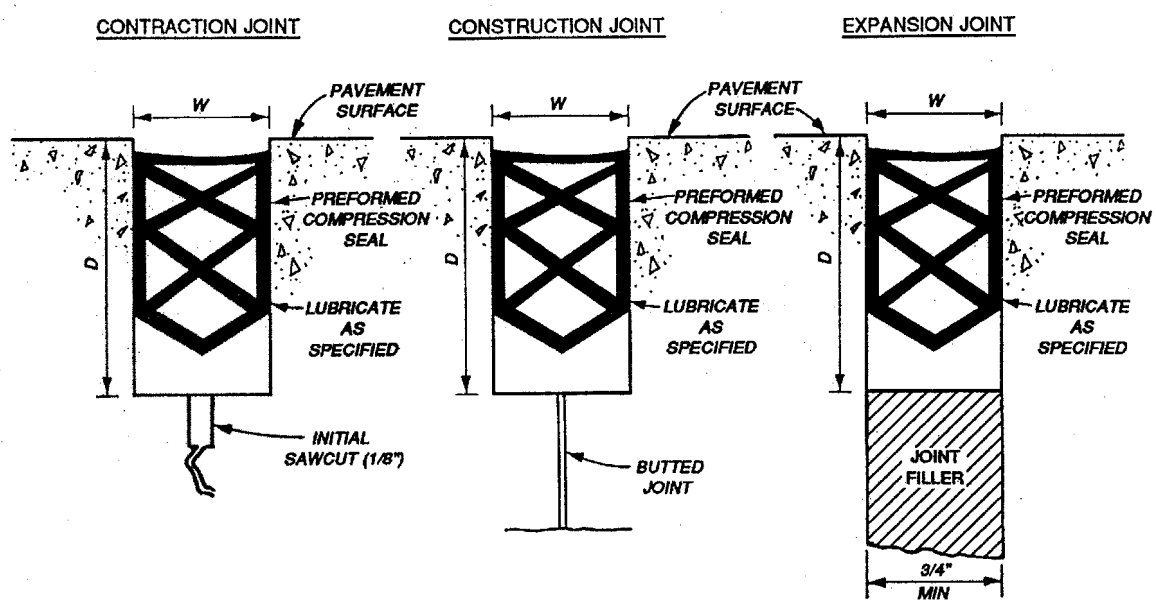
CONVERSION FACTORS

METERS = 0.3048 X FEET
MILLIMETERS = 25.4 X INCHES

POURED JOINT SEALANT DETAILS FOR RIGID PAVEMENTS (SEPARATING TAPE)

Figure 12-31. (Sheet 2 of 3)

PREFORMED COMPRESSION SEAL



NOTE: DEPTH AND WIDTH OF UNCOMPRESSED SEAL SHALL BE AS RECOMMENDED BY MANUFACTURER, EXCEPT THE DEPTH SHALL NOT BE LESS THAN 1.5 INCHES.

TOP OF PREFORMED SEAL SHALL BE 1/8 TO 1/4 INCH BELOW THE PAVEMENT SURFACE. IN AREAS TO BE GROOVED, THE JOINT SEAL SHALL BE RECESSED BELOW THE DEPTH OF THE GROOVES.

COMPRESSION SEALS MUST BE IN COMPRESSION AT ALL TIMES.

JOINT SPACING FT	WIDTH, IN.	
	MIN	MAX
20	1/2	5/8
20 (NAVY)	3/8	--

CONVERSION FACTORS

METERS = 0.3048 X FEET
MILLIMETERS = 25.4 X INCHES

JOINT SEALANT DETAILS (PREFORMED COMPRESSION SEAL) FOR RIGID PAVEMENT

Figure 12-31. (Sheet 3 of 3)

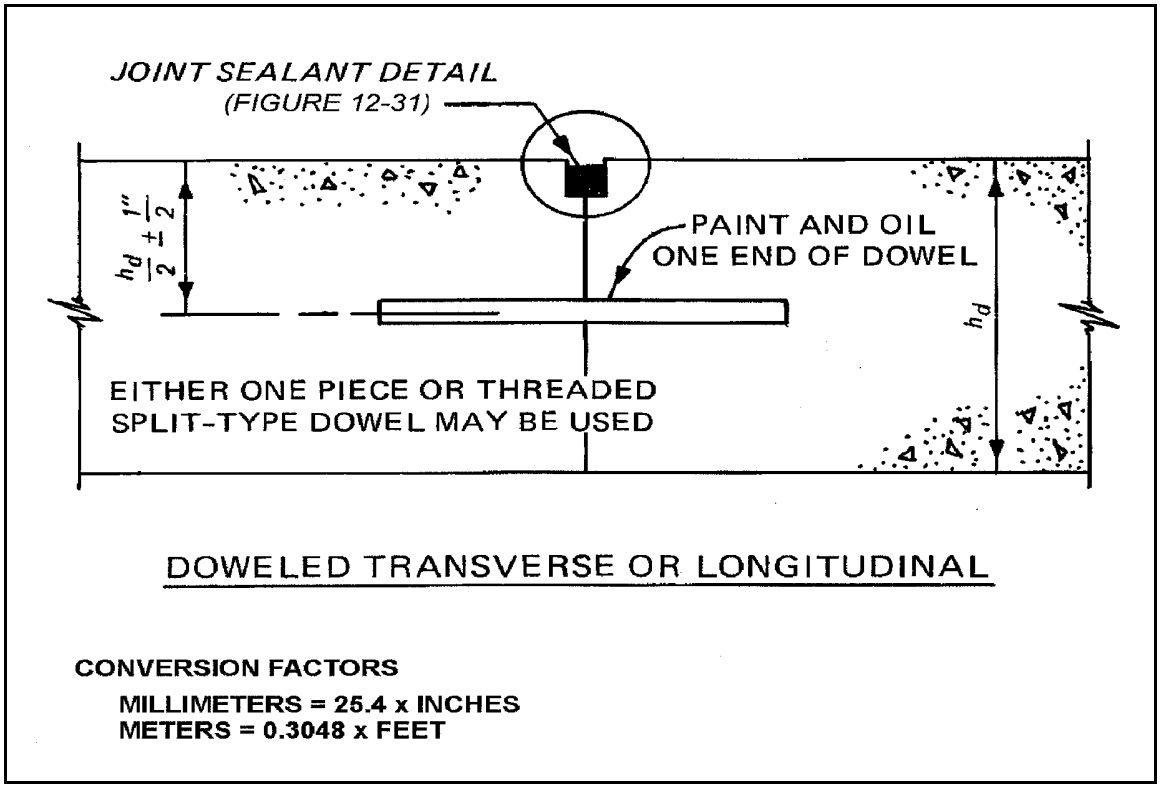
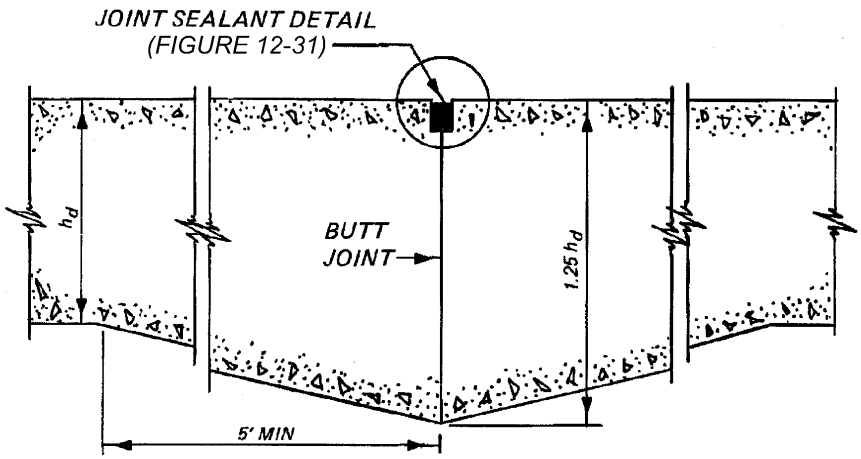
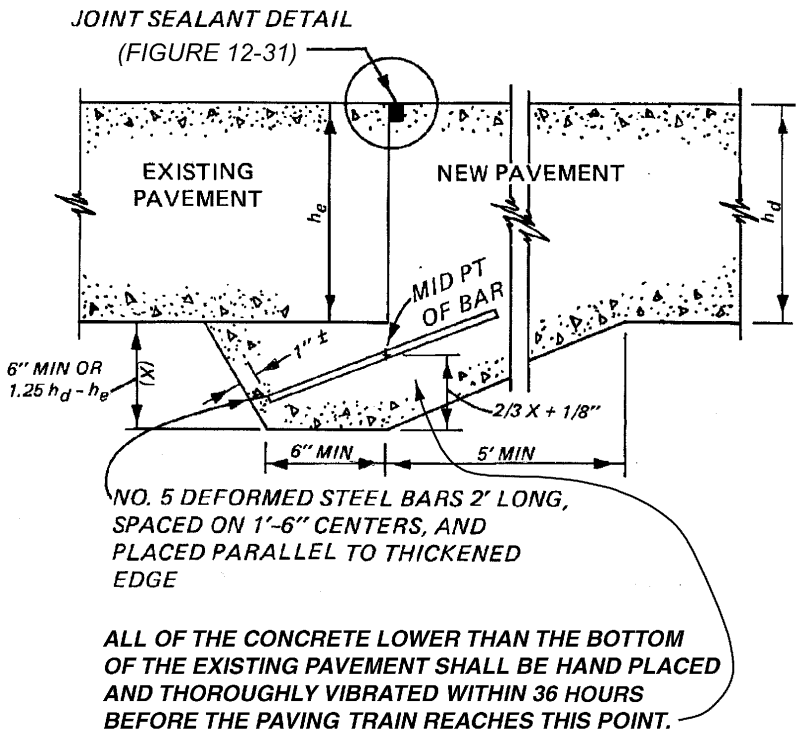


Figure 12-32. Construction joints for plain concrete pavements (Sheet 1 of 3)



THICKENED EDGE LONGITUDINAL

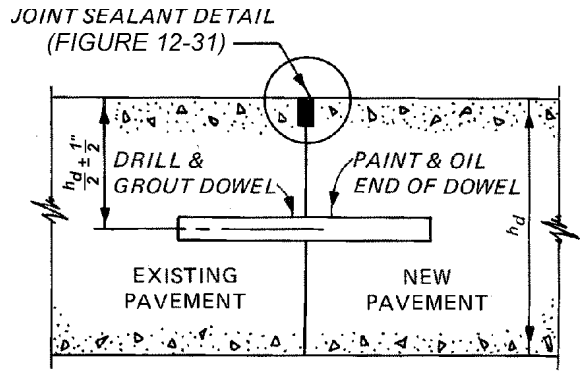


SPECIAL JOINT BETWEEN NEW AND EXISTING PAVEMENT
TRANSVERSE OR LONGITUDINAL

CONVERSION FACTORS

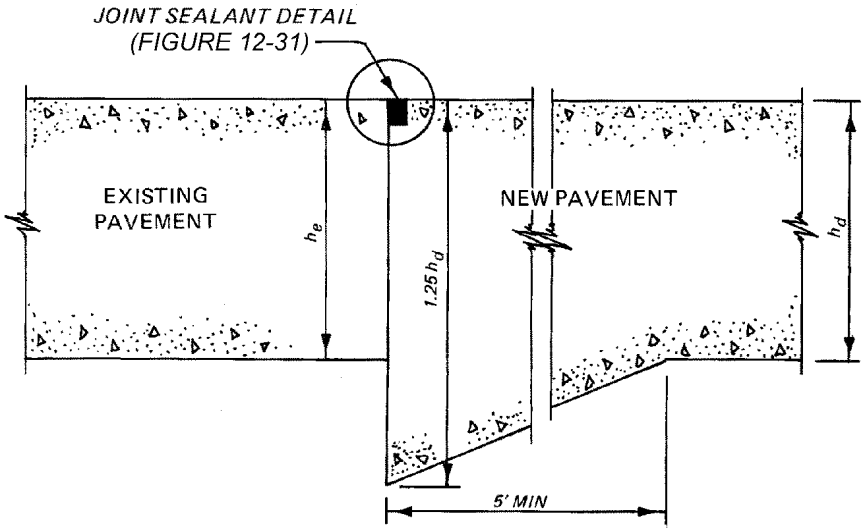
MILLIMETERS = 25.4 x INCHES
METERS = 0.3048 x FEET

Figure 12-32. (Sheet 2 of 3)



NOTE: EITHER ONE PIECE OR THREADED SPLIT-TYPE DOWEL MAY BE USED

DOWELED JOINT BETWEEN
NEW AND EXISTING PAVEMENT



NOTE: THIS TYPE JOINT SHOULD BE USED ONLY WHEN EXISTING PAVEMENT IS TO BE REPLACED IN A SHORT PERIOD OF TIME, SINCE WITHOUT LOAD TRANSFER IT WILL DETERIORATE QUICKLY.

THICKENED EDGE JOINT BETWEEN
NEW AND EXISTING PAVEMENT

CONVERSION FACTORS

MILLIMETERS = 25.4 x INCHES
METERS = 0.3048 x FEET

Figure 12-32. (Sheet 3 of 3)

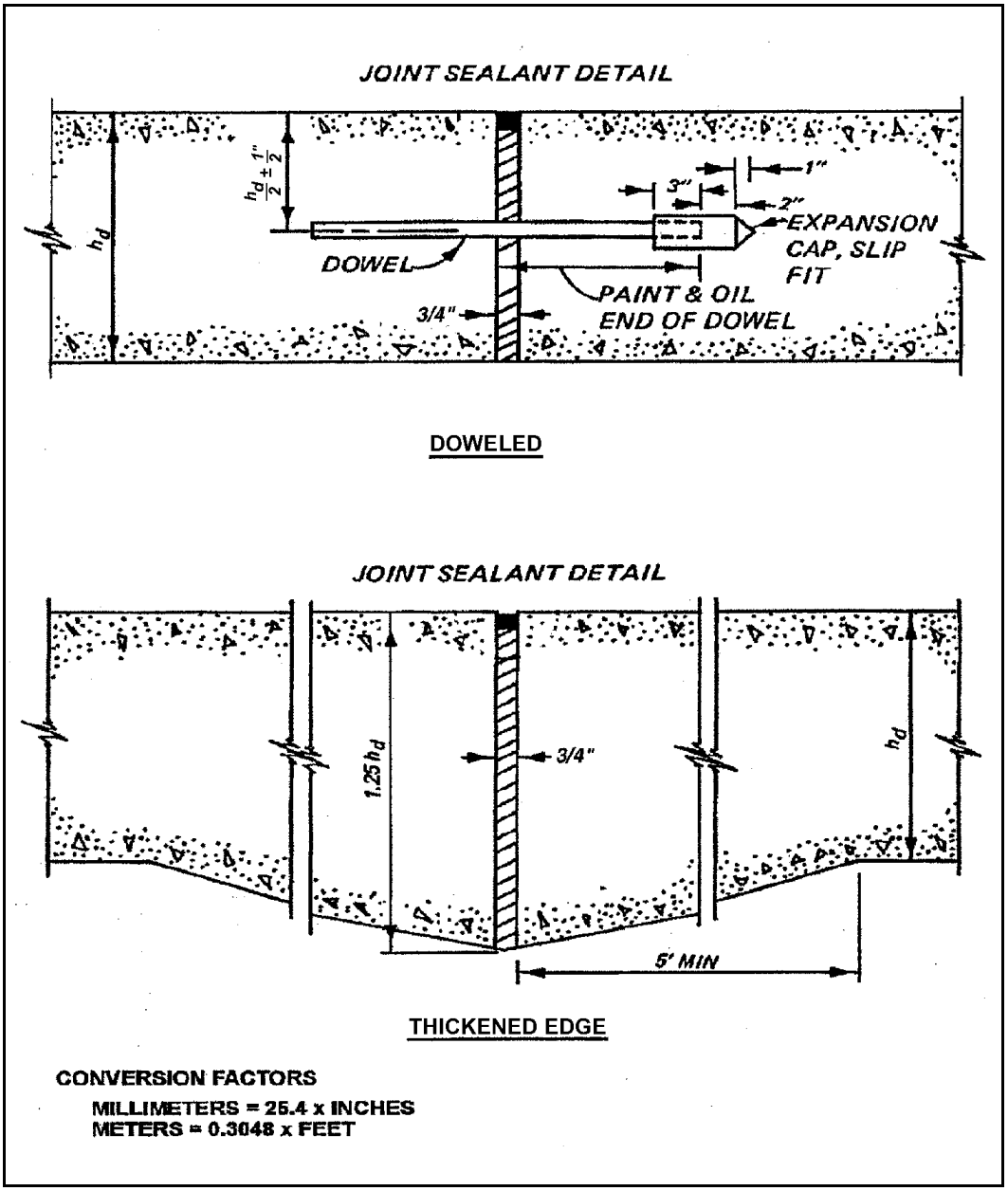


Figure 12-33. Expansion joints for plain concrete pavements

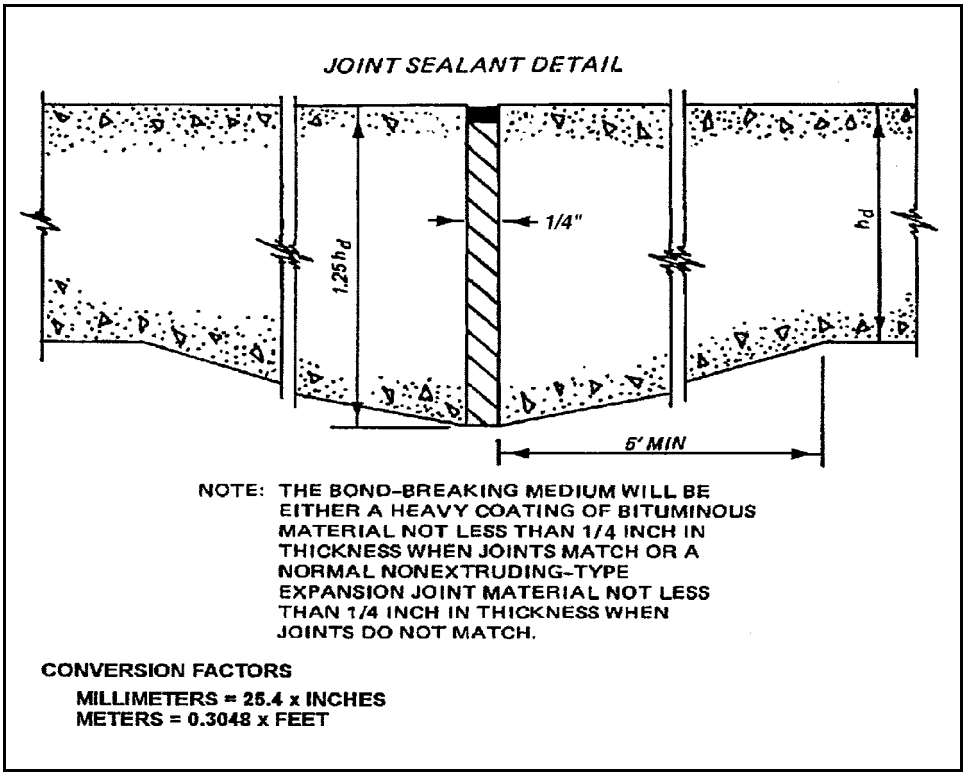


Figure 12-34. Slip joints for plain concrete pavements

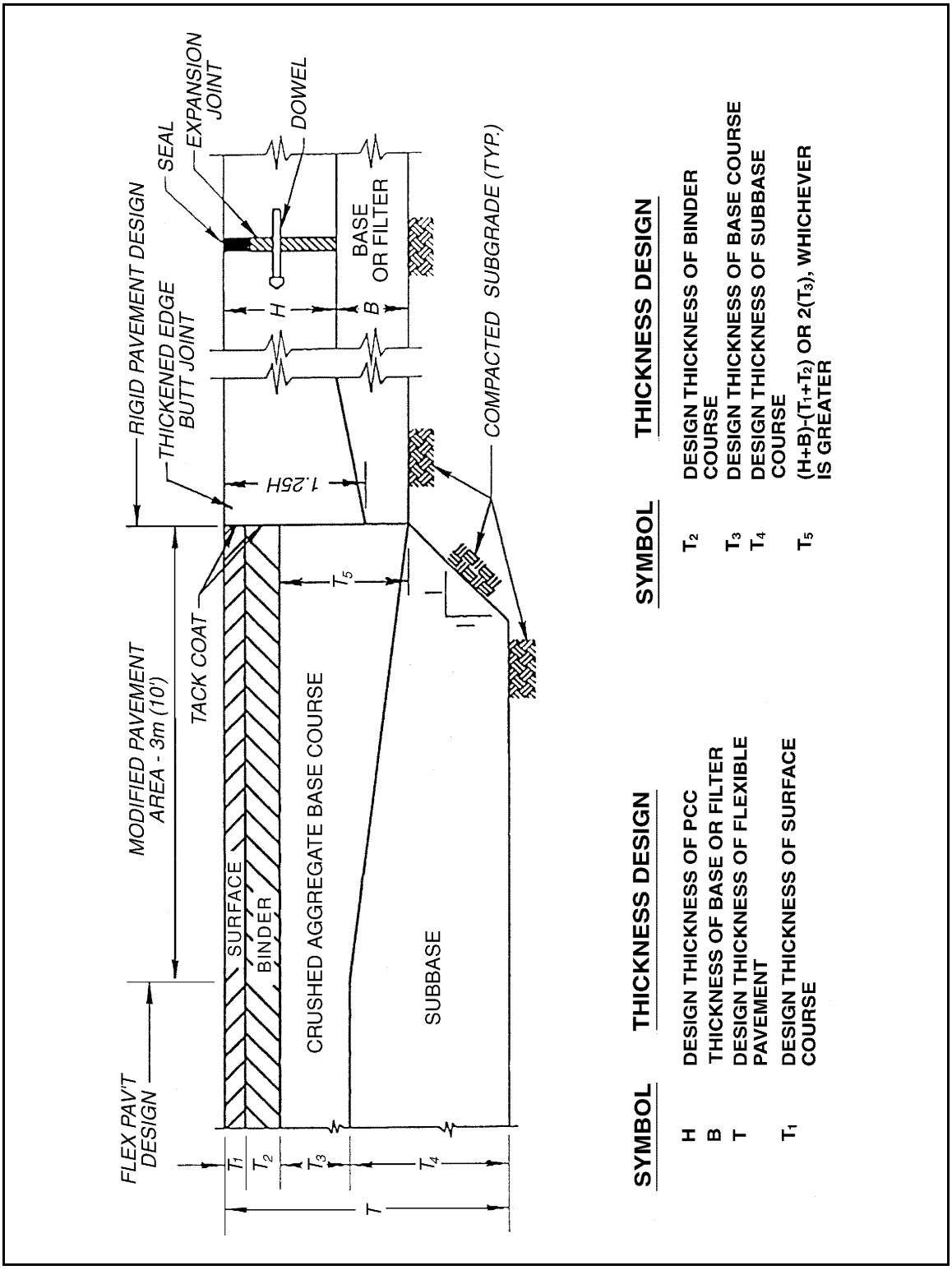
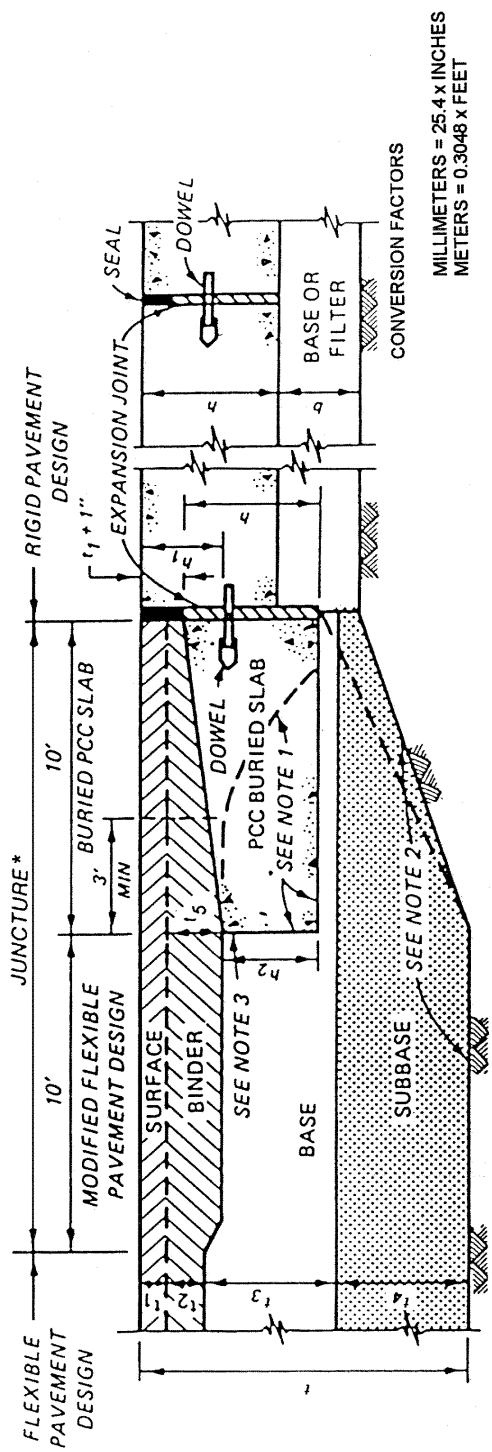


Figure 12-35. Rigid-flexible pavement junction (Army or Air Force)



CONVERSION FACTORS

MILLIMETERS = 25.4 x INCHES
METERS = 0.3048 x FEET

THICKNESS DESIGN

SYMBOL	
h	DESIGN THICKNESS OF PCC
h_1	$h + \frac{t_1 + 1''}{2}$
h_2	$h - (t_1 + 1'')$ BUT NOT LESS THAN 6"
b	THICKNESS OF BASE OR FILTER
t	DESIGN THICKNESS OF FLEXIBLE PAVEMENT
t_1	DESIGN THICKNESS OF SURFACE COURSE
t_2	DESIGN THICKNESS OF BINDER COURSE
t_3	DESIGN THICKNESS OF BASE COURSE
t_4	DESIGN THICKNESS OF SUBBASE COURSE
t_5	$h - h_2$ BUT NOT LESS THAN t_2

* INTENDED FOR CRITICAL TRAFFIC AREAS OR AREAS WHERE SLIGHT DEVIATIONS FROM THE DESIGN GRADE IS OBJECTIONABLE. JUNCTURES SHOULD BE INCORPORATED WHERE PCC JOINS FLEXIBLE PAVEMENT AT ALL TRANSVERSE JUNCTURES IN RUNWAYS AND TAXIWAYS

NOTES:

1. COMPACTED BASE TO DASHED LINE. CUT OUT TO SOLID LINE NOT DISTURBING THE MATERIALS OUTSIDE LIMITS OF BURIED SLAB.
2. EXCAVATE AND COMPACT SUBGRADE TO DASHED LINE IF A BASE OR FILTER IS NOT USED BENEATH PCC. EXCAVATE AND COMPACT SUBGRADE TO SOLID LINE WHEN BASE OR FILTER COURSE IS USED BENEATH PCC.
3. PLACE PCC BURIED SLAB DIRECTLY AGAINST CUT BACK BASE COURSE. NO FORM WILL BE USED.
4. TOP LIFT OF BINDER COURSE TO BE PLACED AND ROLLED TRANSVERSELY. SURFACE COURSE PLACED AND ROLLED LONGITUDINALLY STOPPING ROLLERS ON RIGID PAVEMENT.
5. FOR HEAVY LOAD DESIGN h_2 NOT LESS THAN 6 IN. FOR LIGHT LOAD DESIGN h_2 NOT LESS THAN 4 IN.
6. JOINT THE BURIED SLAB THE SAME AS THE ADJACENT RIGID PAVEMENT.

Figure 12-36. Rigid-flexible pavement junction

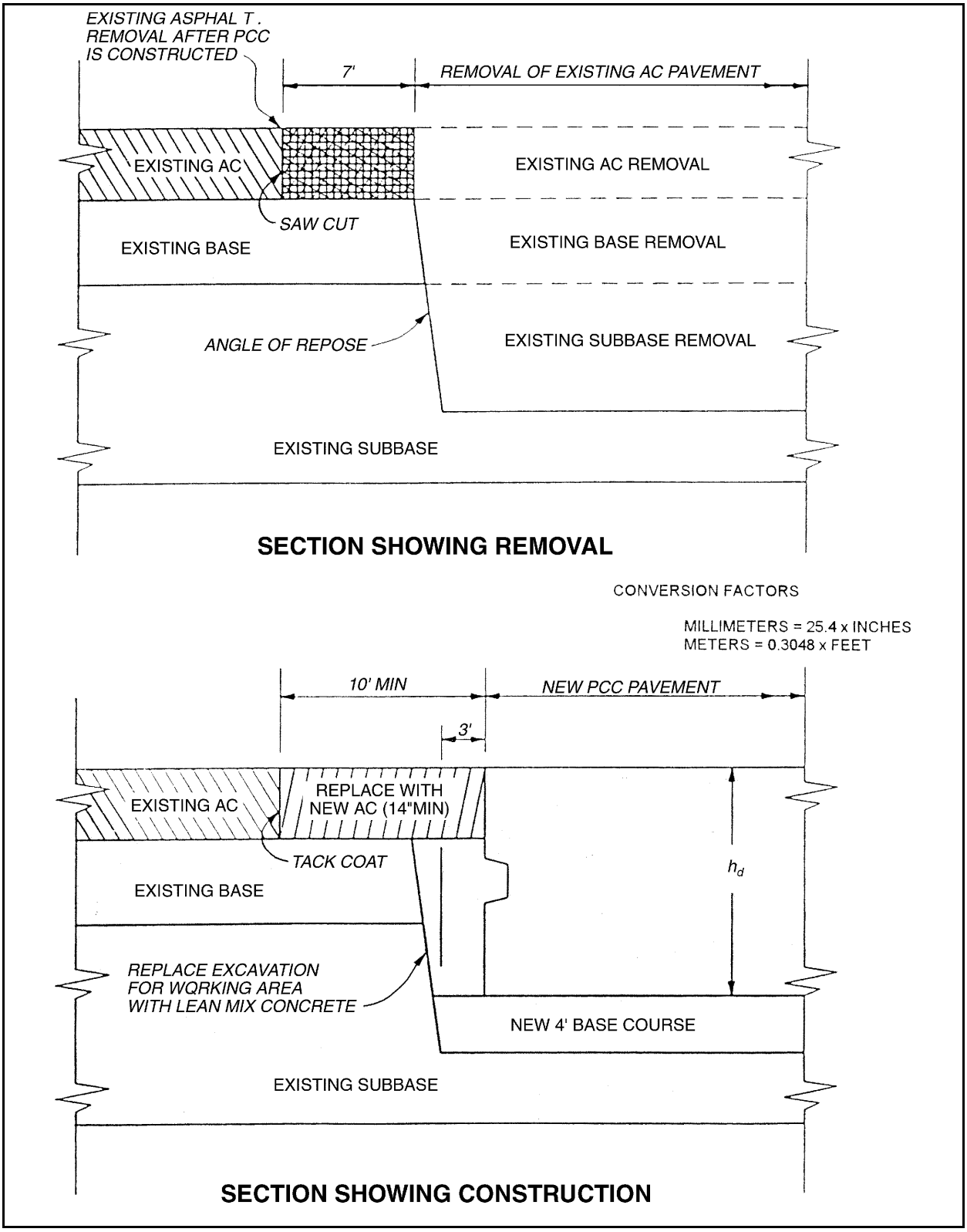


Figure 12-37. PCC to AC joint detail (removal and construction)

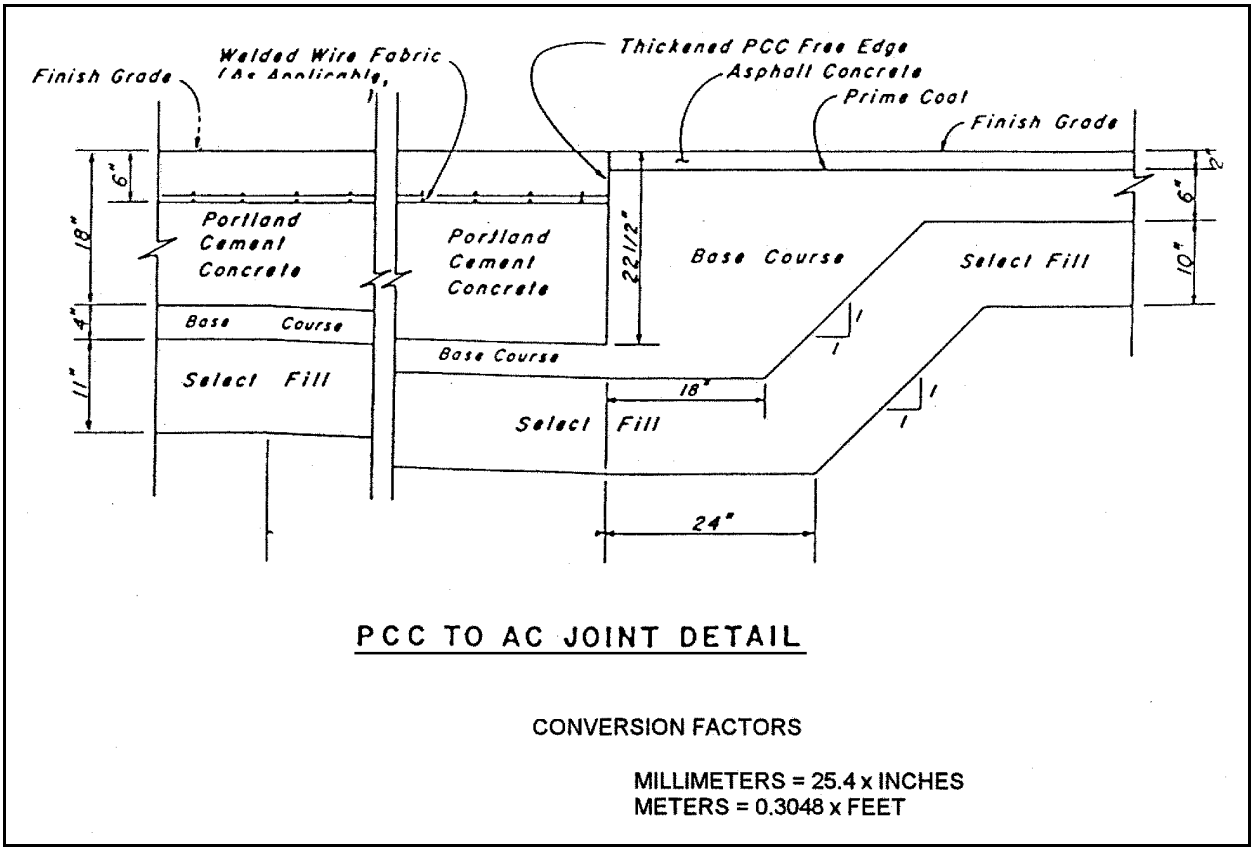


Figure 12-38. PCC to AC joint detail (very little traffic expected)

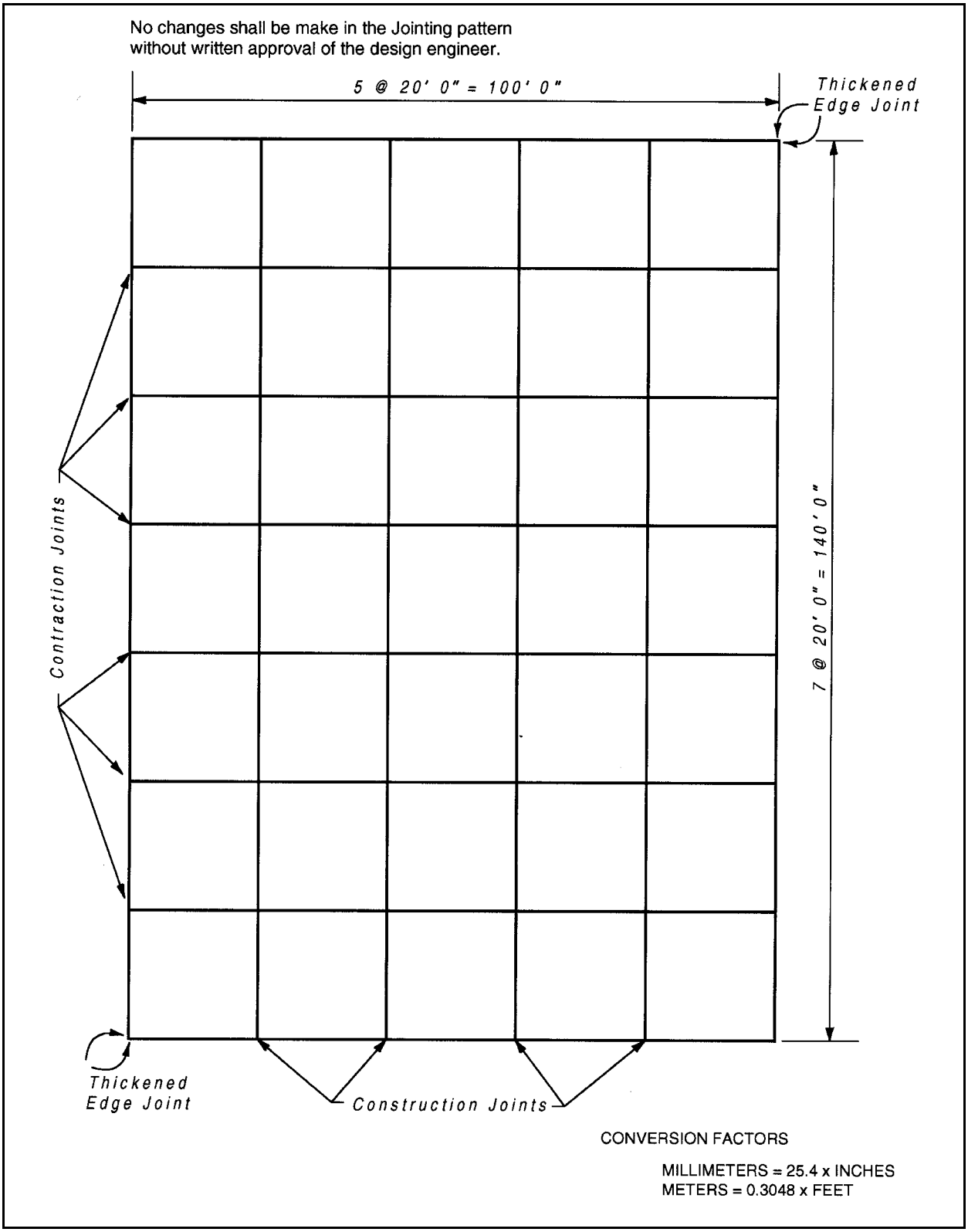


Figure 12-39. Sample jointing pattern (SI units)

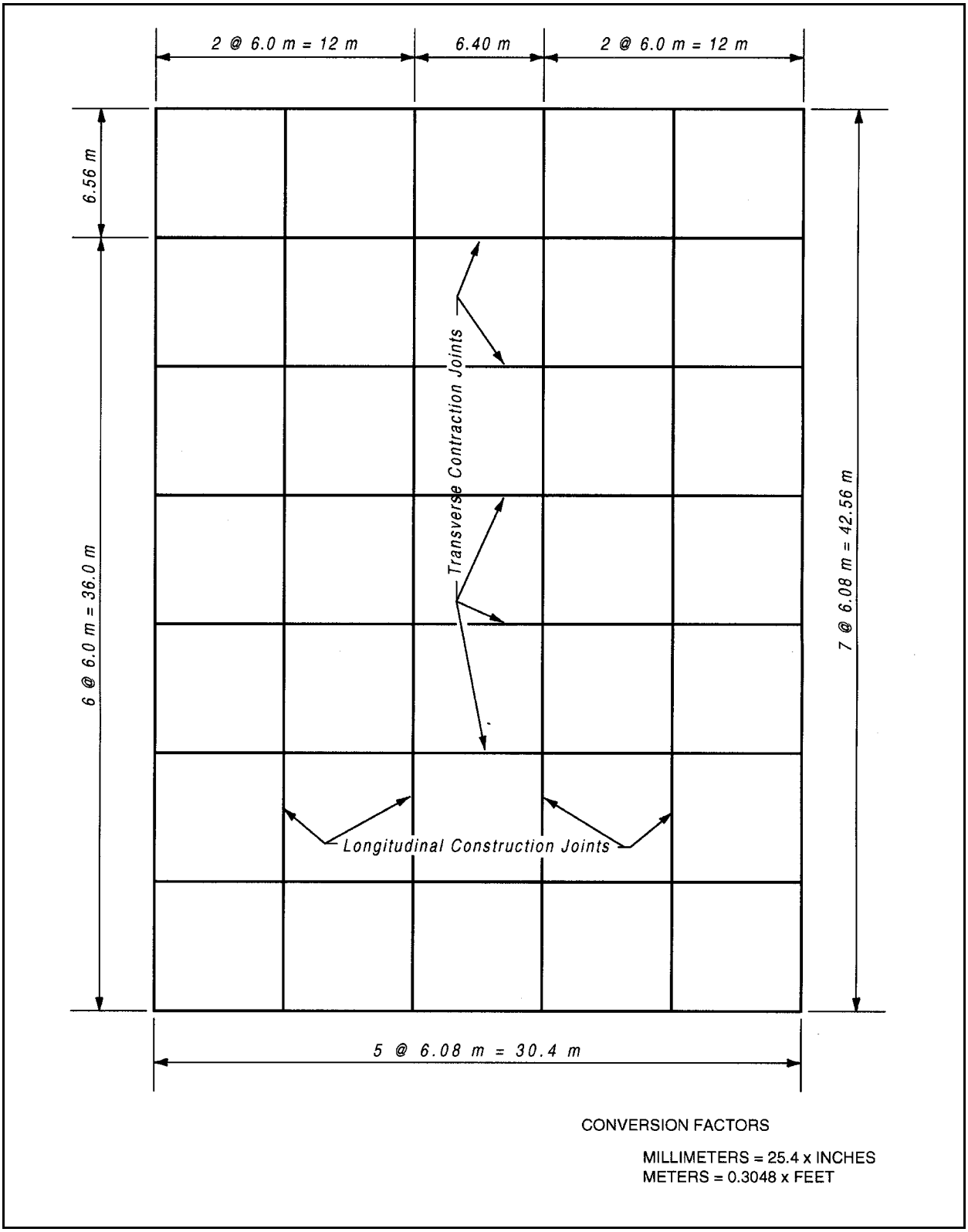


Figure 12-40. Sample jointing pattern (metric units)

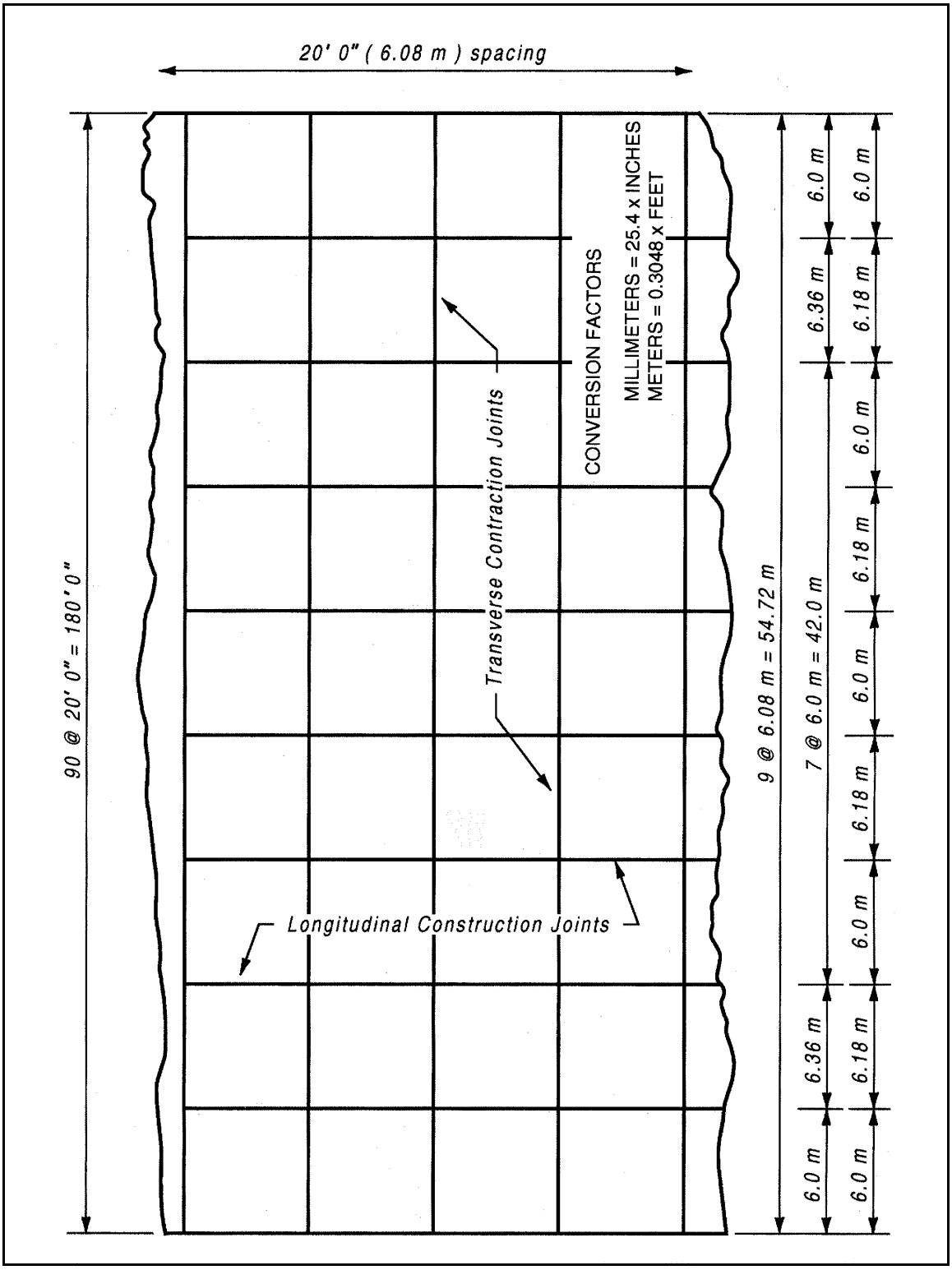


Figure 12-41. Sample jointing pattern for 180-ft.-wide lanes

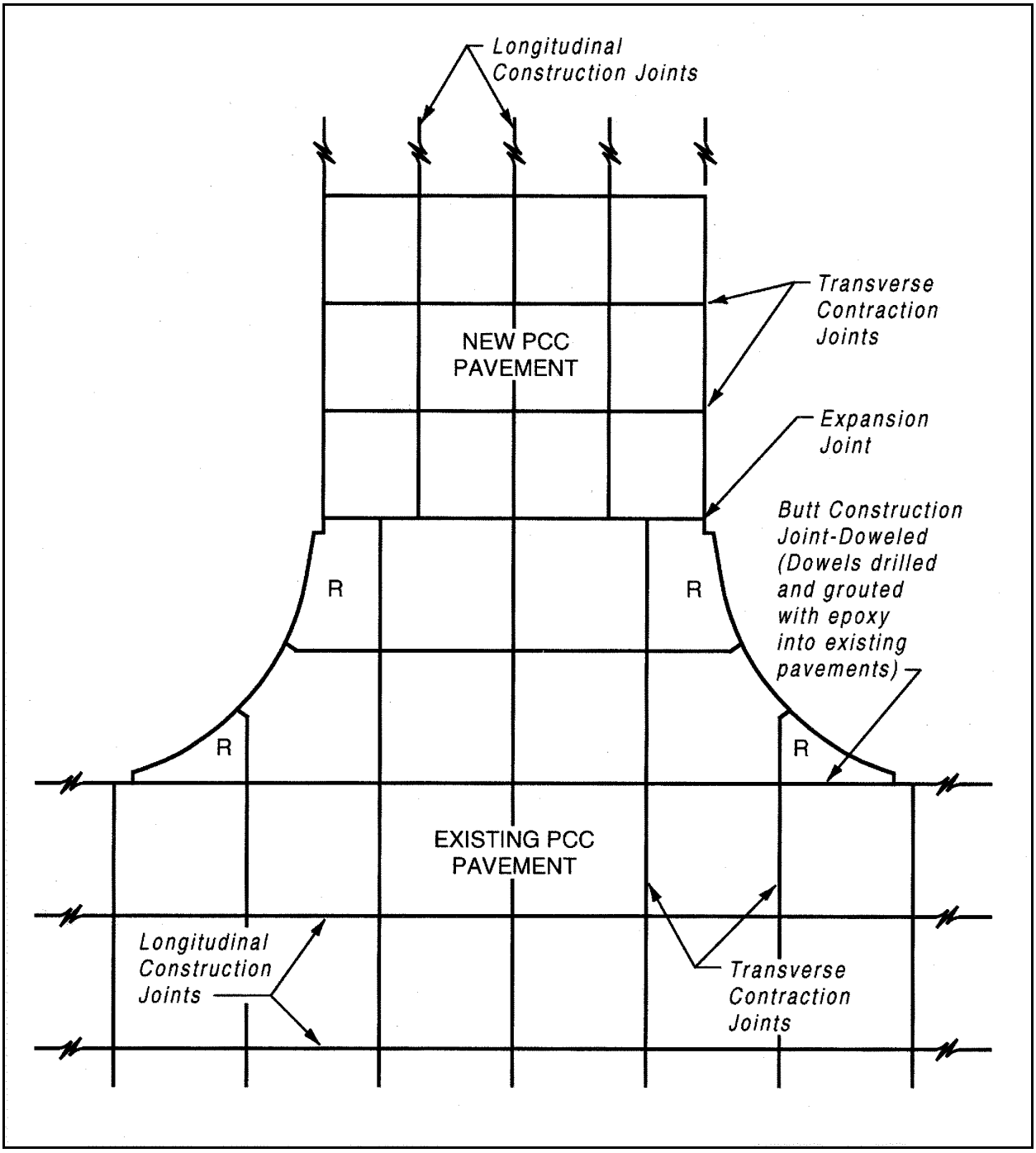


Figure 12-42. Sample jointing pattern at an intersection

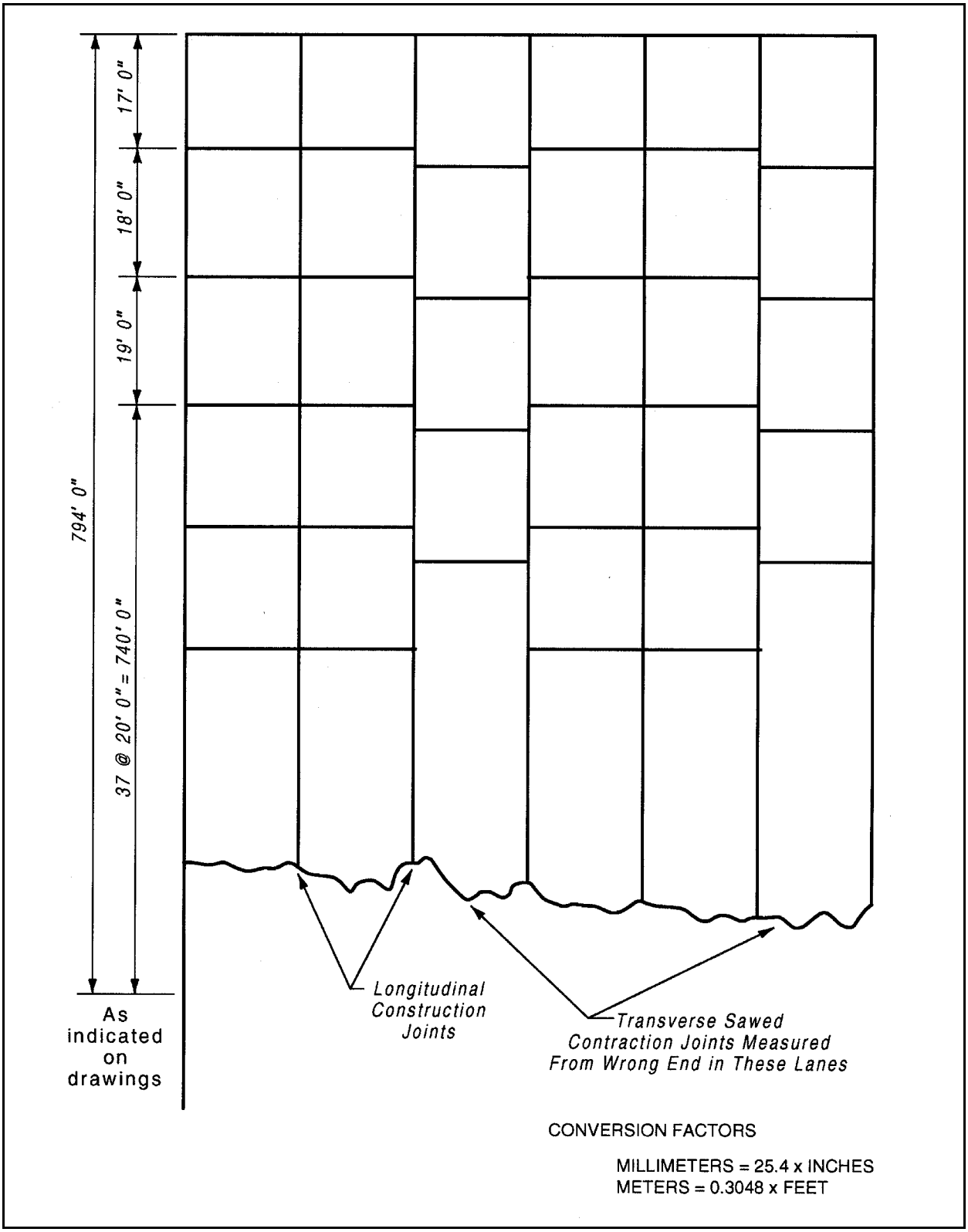


Figure 12-43. Effects of confusion in sawing joints

CHAPTER 13

REINFORCED CONCRETE PAVEMENT DESIGN

1. GENERAL. These designs are applicable to Army and Air Force pavements but will not normally be used for Navy and Marine Corps projects. However, reinforced concrete may be considered for special or unusual design conditions on a case-by-case basis and must be approved by the Naval Facilities Engineering Command. The exception to this is in odd-shaped slabs and mismatched joints where reinforcing is required.

2. BASIS FOR DESIGN - NAVY AND MARINE CORPS. Reinforced concrete pavements employ longer joint spacings than plain concrete pavements. The cracks that develop from shrinkage, warping, curling, and traffic load stresses are held together by reinforcement. Steel reinforcing is used to slow the deterioration of cracks that develop in the concrete slab by holding these cracks tightly together to maintain aggregate interlock. When approved for use, design procedures for Navy and Marine Corps reinforced concrete pavements will be the same as for Army and Air Force reinforced pavements.

a. Thickness. The thickness design for reinforced concrete pavement is similar to plain concrete pavement design, modified by the results of accelerated traffic tests. These tests demonstrate that the required pavement thickness may be less than the required thickness of a plain concrete pavement that provides equal performance. However, as thickness is reduced substantially, premature distress may occur. Therefore, because of inconsistent performance of thin reinforced pavements, for new construction, the thickness shall not be reduced from that determined for plain concrete.

b. Reinforcement. Reinforcing steel is usually required in both the transverse and longitudinal directions. The steel may be deformed bars or welded wire fabric. Typical amounts of reinforcing range from 0.05 to 0.25 percent area.

c. Joints. The maximum slab size for reinforced concrete pavements is a function of the slab thickness, yield strength of the reinforcing steel, and the percent of reinforcement. Slab size is commonly 7.6 meters (25 feet) square. All joints in reinforced concrete pavements, with the exception of keyways and thickened-edge joints, are doweled. Dowels are effective in providing load transfer. Alignment of the dowel bars and adequate consolidation around the dowel basket are critical factors.

3. BASIS FOR DESIGN - ARMY AND AIR FORCE. Steel reinforcement in the concrete provides improved continuity across the cracks that develop because of environmental factors or induced loads. The improved crack continuity results in better performance under traffic and less maintenance than an equal thickness of plain concrete pavement. Thus, for equal performance, the thickness of reinforced concrete pavement can be less than the thickness of plain concrete pavements. The design procedure presented herein yields the thickness of reinforced concrete pavement and the percentage of steel reinforcement required to provide the same performance as a predetermined thickness of plain concrete pavement constructed on the same foundation condition. The procedure has been developed from full-scale accelerated traffic testing. Failure is considered to be severe spalling of the concrete along the cracks that develop during traffic.

4. USES FOR REINFORCED CONCRETE. Reinforced concrete pavement may be used as slabs on grade or as overlay pavements for any traffic area of the airfield. Reinforcement may be used to reduce the required thickness and permit greater spacing between joints. Its selection should be based upon

the economics involved. In certain instances, reinforcement will be required to control cracking that may occur in plain concrete pavements without any reduction in thickness requirements.

5. REDUCED THICKNESS DESIGN - ARMY AND AIR FORCE.

a. General. The greatest use of reinforcement to reduce the required plain concrete pavement thickness will probably be to provide a uniform thickness for the various types of traffic areas as different structural conditions of the base pavement. Since these changes in thickness cannot be made at the surface, reinforcement can be used to reduce the required thickness and thereby avoid the necessity for removal and replacement of pavements or overdesigns. There are other instances in which reinforcement to reduce the pavement thickness may be warranted and must be considered, but the economic feasibility for the use of reinforcement must also be considered. The design procedure consists of determining the percentage of steel required, the thickness of the reinforced concrete pavement, and the maximum allowable length of slabs. In addition, a computer program discussed in Chapter 1 may be used for the design of reinforced concrete pavement.

b. Determination of Required Percent Steel and Required Thickness of Reinforced Concrete Pavement. It is first necessary to determine the required thickness of plain concrete pavement using the design loading and physical properties of the pavement and foundation. When the reinforced concrete pavement is to be placed on stabilized or nonstabilized bases or subgrades, the procedure outline in Chapter 12 will be used to determine the thickness of plain concrete. The thickness of plain concrete is then used to enter Figure 13-1 to determine the required percent steel and the required thickness of reinforced concrete pavement. Since the thickness of reinforced concrete and percent steel are interrelated, it will be necessary to establish a desired value of one and determine the other. The resulting values of reinforced concrete thickness and percent steel will represent a reinforced concrete pavement that will provide the same performance as the required thickness of plain concrete pavement. In all cases, when the required thickness of plain concrete pavement is reduced by the addition of reinforcing steel, the design percentage of steel will be placed in each of two directions (transverse and longitudinal) in the slab. For construction purposes, the required thickness of reinforced concrete must be rounded to the nearest full- and half-inch increment. When the indicated thickness is midway between full- and half-inch, the thickness will be rounded upward.

c. Determination of Maximum Reinforced Concrete Pavement Slab Size. The maximum length or width of the reinforced concrete pavement slabs is dependent largely upon the resistance to movement of the slab on the underlying material and the yield strength of the reinforcing steel. The latter factor can be easily determined, but very little reliable information is available regarding the sliding resistance of concrete on the various foundation materials. For this design procedure, the sliding resistance has been assumed to be constant for a reinforced concrete pavement cast directly on the subgrade, on a stabilized or nonstabilized base course, or on an existing flexible pavement. The maximum allowable width W or length L of reinforced concrete pavement slabs will be determined from the following:

$$W \text{ or } L = 0.2224 \sqrt[3]{h_d (y_s S)^2} \quad \text{for SI Units}$$

$$W \text{ or } L = 0.0777 \sqrt[3]{h_d (y_s S)^2} \quad \text{for English Units} \qquad (13\text{-}1)$$

where

h_d = design thickness of reinforced concrete, millimeters (inches)

y_s = yield strength of reinforcing steel, normally 413.7 MPa (60,000 psi)

S = percent reinforcing steel

The formula above has been expressed on the nomograph (Figure 13-1) for a steel yield strength y_s of 413.7 MPa (60,000 psi), and the maximum length or width can be obtained from the intersection of a straight line drawn between the values of design thickness and percent steel that will be used for the reinforced concrete pavement. The width of reinforced concrete pavement will generally be controlled by the concrete paving equipment and will normally be 7.6-12.1 meters (25-40 feet), unless smaller widths are necessary to meet dimensional requirements.

d. Limitations to Reinforced Concrete Pavement Design Procedure. The design procedure for reinforced concrete pavements presented herein has been developed from a limited amount of investigational and performance data. Consequently, the following limitations are imposed:

(1) No reduction in the required thickness of plain concrete will be allowed for percentages of steel reinforcement less than 0.05.

(2) No further reduction in the required thickness of plain concrete pavement will be allowed over that indicated for 0.5 percent steel reinforcement in Figure 13-1 regardless of the percent steel used.

(3) No single dimension of reinforced concrete pavement slabs will exceed 30.5 meters (100 feet) regardless of the percent steel used or slab thickness.

(4) The minimum thickness of a reinforced concrete pavement or overlay will be 152 millimeters (6 inches).

6. REINFORCEMENT TO CONTROL PAVEMENT CRACKING.

a. General. Reinforcement is mandatory in certain pavement areas to control or minimize the effects of cracking. The reinforcing steel holds cracks tightly closed, thereby preventing spalling at the edges of the cracks and progression of the cracks into adjacent slabs. For each of the following conditions, the slabs or portions of the slabs will be reinforced with 0.05 percent steel in two directions normal to each other unless otherwise specified. No reduction in thickness will be allowed for this steel.

b. Odd-shaped Slabs. It is often necessary in the design of pavement facilities to resort to odd-shaped slabs. Unless reinforced, these odd-shaped slabs often crack and eventually spall along the cracks, producing debris that is objectionable from operational and maintenance viewpoints. In addition, the cracks may migrate across joints into adjacent slabs. In general, a slab is considered to be odd-shaped if the longer dimension exceeds the shorter one by more than 25 percent or if the joint pattern does not result in essentially a square or rectangular slab. Figure 13-2 presents typical examples of odd-shaped slabs requiring reinforcement. Where practicable, the number of odd-shaped slabs can be minimized by using a sawtooth fillet and not reinforcing.

c. Mismatched Joints. Steel reinforcement in the slabs is mandatory to prevent migration of cracks into adjacent pavements for the following two conditions of mismatched joints:

(1) Where joint patterns of abutting pavement facilities do not match, partial reinforcement of slabs may be necessary. In such a condition, the mismatch of joints can cause a crack to form in the adjacent pavement unless there is sufficient width of bond-breaking medium installed in the joint. The determination relative to using reinforcement at mismatched joints in such junctures is based upon the type of joint between the two pavement sections. A partial reinforcement of the slab, as described below, is required when the joint between the abutting pavement is one of the following: (a) doweled construction joint, (b) keyed construction joint, (c) thickened-edge butt joint without a bond-breaking medium, (d) doweled expansion joint, and (e) thickened-edge slip joint with less than 6.4-millimeter (1/4-inch) bond-breaking medium. Reinforcement is not required if the joint between the abutting pavement facilities is either a thickened-edge expansion joint or a thickened-edge slip joint with 6.4 millimeters (1/4 inch) or more of bond-breaking medium, except for a mismatch of joints in the center 23-meter (75-foot) width of runway where reinforcement of the slabs of mismatched joints will be required regardless of the type of joint between the facilities. When reinforcement at mismatched joints is required, the slab in the pavement facility directly opposite the mismatched joint will be reinforced with the minimum 0.05 percent steel. The reinforcing steel will be placed in two rectangular directions for a distance 915 millimeters (3 feet) back from the juncture and for the full width or length of the slab in a direction normal to the mismatched joint. When a new pavement is being constructed abutting an existing pavement, the new slabs opposite mismatched joints will be reinforced in the manner described above. When two abutting facilities are being constructed concurrently, the slabs on both sides of the juncture opposite mismatched joints will be reinforced in the manner described above. For this condition shown in Figure 13-2, the slip joint bond-breaking medium can be specified to be a full 6.4 millimeters (1/4 inch) thick, and the reinforcing may be omitted.

(2) The second condition of mismatched joints where reinforcement is required occurs in the construction of a plain concrete overlay on an existing rigid pavement. Joints in the overlay should coincide with joints in the base pavement. Sometimes this is impracticable due to an unusual jointing pattern in the existing pavement. When necessary to mismatch the joints in the overlay and the existing pavement, the overlay pavement will be reinforced with the minimum 0.05 percent steel. The steel will be placed in two rectangular directions for a distance of at least 915 millimeters (3 feet) on each side of the mismatched joint in the existing pavement. The steel will, however, not be carried through any joint in the overlay except as permitted or required to meet joint requirements. If the joint pattern in the existing pavement is highly irregular or runs at an angle to the desired pattern in the overlay, the entire overlay will be reinforced in both the longitudinal and transverse directions. When a bond-breaker course (see Chapter 17) is placed between the existing pavement and overlay, reinforcement of the overlay over mismatched joints is not required, except for mismatched expansion joints.

d. Reinforcement of Pavements Incorporating Heating Pipes. Plain concrete pavements, such as hangar floors that incorporate radiant heating systems within the concrete, are subject to extreme temperature changes. These temperature changes cause thermal gradients in the concrete that result in stresses of sufficient magnitude to cause surface cracking. To control such cracking, these pavement slabs will be reinforced with the minimum 0.05 percent steel placed in the transverse and longitudinal directions.

e. Reinforcement of Slabs Containing Utility Blockouts. The minimum 0.05 percent steel reinforcement is required in plain concrete pavement slabs containing utility blockouts, such as for hydrant refueling outlets, storm drain inlets, and certain types of flush lighting fixtures. The entire slab or slabs containing the blockouts will be reinforced in two rectangular directions.

7. REINFORCED CONCRETE PAVEMENTS IN FROST AREAS. Normally, plain concrete pavements in frost areas will be designed in accordance with Chapter 22, and reinforcement will be unnecessary. There may, however, be special instances when it will be directed that the pavement thickness be less than required by frost design criteria. Two such instances are: the design of new pavements to the strength of existing pavement when the existing pavement does not meet the frost design requirements, and the design of an inlay section of adequate strength pavement in the center portion of an existing runway when the existing pavement does not meet the frost design requirements. In such instances, the new pavements will be reinforced with a minimum of 0.15 percent steel. The minimum 0.15 percent steel will be placed in each of two directions (transverse and longitudinal) in the slab. The reinforcing steel is required primarily to control cracking that may develop because of differential heaving. The pavement thickness may be reduced, and the maximum slab length, consistent with the percent steel, may be used. Longer slabs will help reduce roughness that may result from frost action. Greater percentages of steel reinforcement may be used when it is desired to reduce the pavement thickness more than is allowable for the required minimum percentage of steel.

8. REINFORCING STEEL.

a. Type or Reinforcing Steel. The reinforcing steel may be either deformed bars or welded wire fabric. Deformed bars should conform to the requirements of ASTM A 615, A 616, or A 617. In general, grade 60 deformed bars should be specified, but other grades may be used if warranted. Fabricated steel bar mats should conform to ASTM A 184. Cold drawn wire for fabric reinforcement should conform to the requirements of ASTM A 82, and welded steel wire fabric to ASTM A 185.

b. Placement of Reinforcing Steel. The reinforcing steel will be placed at a depth of $h_d/4$ + 25 millimeters ($h_d/4$ + 1 inch) from the surface of the reinforced slab. This will place the steel above the neutral axis of the slab and will allow clearance for dowel bars. The wire or bar sizes and spacing should be selected to give, as nearly as possible, the required percentage of steel per foot of pavement width or length. In no case should the percent steel used be less than that required by Figure 13-1. Two layers of wire fabric or bar mat, one placed directly on top of the other, may be used to obtain the required percent of steel; however, this should only be done when it is impracticable to provide the required steel in one layer. If two layers of steel are used, the layers must be fastened together (either wired or clipped) to prevent excessive separation during concrete placement. When the reinforcement is installed and concrete is to be placed through the mat or fabric, the minimum clear spacing between bars or wires will be one and one-half times the maximum size of aggregate. If the strike-off method is used to place the reinforcement (layer of concrete placed and struck off at the desired depth, the reinforcement placed on the plastic concrete, and the remaining concrete placed on top of the reinforcement), the minimum spacing of wires or bars will not be less than the maximum size of aggregate. Maximum bar or wire spacing shall not exceed 305 millimeters (12 inches) nor the slab thickness. Figure 13-3 shows the typical details of slab reinforcement with wire fabric or bar mats. The bar mat or wire fabric will be securely anchored to prevent forward creep of the steel mats during concrete placement and finishing operations. The reinforcement shall be fabricated and placed in such a manner that the spacing between the longitudinal wire or bar and the longitudinal joint, or between the transverse wire or bar and the transverse joint, will not exceed 76 millimeters (3 inches) or one-half of the wire or bar spacing in the fabric or mat (Figure 13-3). The wires or bars will be lapped as follows.

(1) Deformed steel bars will be overlapped for a distance of at least 24 bar diameters, measured from the tip of one bar to the tip of the other bar. The lapped bars will be wired or otherwise securely fastened to prevent separation during concrete placement.

(2) Wire fabric will be overlapped for a distance equal to at least one spacing of the wire in the fabric or 32 wire diameters, whichever is greater. The length of lap is measured from the tip of one wire to the tip of the other wire normal to the lap. The wires in the lap will be wired or otherwise securely fastened to prevent separation during concrete placement.

9. JOINTING.

a. Requirements. Figures 13-4 through 13-6 present details of joints in reinforced concrete pavements. Joint requirements and types will be the same as for plain concrete except for the following:

(1) All joints will be doweled with the exception of thickened-edge-type joints and longitudinal construction joints. One end of the dowel will be painted and oiled to permit movement at the joint.

(2) Thickened-edge-type joints (expansion, butt, or slip) will not be doweled. The edge will be thickened to $1.25h_d$.

(3) When a transverse construction joint is required within a reinforced slab unit, the reinforcing steel will be carried through the joint. In addition, dowels meeting the size and spacing requirements of Table 12-8 or the design thickness h_d will be used in the joint.

b. Joint Sealing. Joint sealing for reinforced concrete pavements will be the same as for plain concrete pavements.

10. EXAMPLES OF REINFORCED CONCRETE PAVEMENT DESIGN.

a. A reinforced concrete pavement is to be used for an Air Force heavy-load airfield. Field and laboratory test programs have yielded design values of 4.8 MPa (700 psi) for the concrete flexural strength R and 54 MN/m^3 (200 pci) for the modulus of soil reaction k for the foundation.

b. Assuming that stabilization will not be used, it is first necessary to determine the required thicknesses of plain concrete pavement. By entering Figure 12-8 with the design values of R and k, the required thicknesses of plain concrete are as shown in column 2 of Table 13-1. At this point, it is necessary to decide whether to preselect the percentage of reinforcing steel and determine the required thickness of reinforced pavements, or to select a thickness of reinforced concrete and determine the percent steel. First, let it be assumed that a S = 0.20 percent will be used and that it is desired to determine the required thickness. Figure 13-1 is entered with S = 0.20 percent and the thickness of plain concrete for each traffic area, and values of reinforced concrete pavement thickness determined as shown in column 3 of Table 13-1. These thicknesses are rounded to the nearest 10-millimeter (½-inch) increment for construction (column 4). After the thicknesses are rounded, it is then necessary to reenter Figure 13-1 to determine the percent steel commensurate with the rounded thickness values (column 5). Next, let it be assumed that types A, B, and C traffic areas are to be constructed to the same thickness of 405 millimeters (16 inches) of reinforced concrete pavement, and type D traffic areas are to be 255 millimeters (10 inches). Figure 13-1 is entered with the thickness of plain concrete and selected values of reinforced concrete thickness to determine the required percent steel (column 7). The maximum length or width of a reinforced concrete pavement slab is a function of the yield strength of the steel, thickness of the slab, and percent steel and can be determined either from Figure 13-1 or by Equation 13-1. Columns 6 and 8 of Table 13-1 present the maximum allowable lengths or widths for the examples using a steel with a yield strength of 413 MPa (60,000 psi).

c. Assume that a 152-millimeter (6-inch) lean concrete base course will be used. The compressive strength of the lean concrete is 20.6 MPa (3,000 psi), and the flexural modulus of elasticity is 13,788 MPa (2×10^6 psi). As with the previous example, the required thicknesses of plain concrete pavement on both the nonstabilized and on the lean concrete base are determined and are shown in columns 2 and 3 of Table 13-2. A value may then be selected for the required thickness of reinforced concrete or the percentage of reinforcing steel and determine the other using Figure 13-1. If a percent steel value of 0.20 is selected, the values of reinforced concrete from Figure 13-1 would be shown in column 4 of Table 13-2. These values rounded for construction are listed in column 5. Then, reenter Figure 13-1 with the rounded values of reinforced concrete to obtain the required percent steel shown in column 6. The allowable slab lengths are determined from Equation 13-1 or Figure 13-1 using a reinforcing steel with a yield strength of 413 MPa (60,000 psi) (column 7). If a reinforced concrete thickness of 380 millimeters (15 inches) is selected for the type A, B, and C traffic areas and a thickness of 255 millimeters (10 inches) is selected for the type D traffic area, then the required percent steel determined from Figure 13-1 would be as shown in column 8. Column 9 presents the allowable lengths or widths of slab for the reinforced concrete pavement.

Table 13-1
Reinforced Concrete Pavement Design Example

Traffic Area (1)	Thickness of Plain Concrete, in. (2)	Initial Thickness of Reinforced Concrete, in. (3)	Design Thickness of Reinforced Concrete, in. (4)	Percent Steel (5)	Length or Width of Slab, ft (6)	Design Example Preselecting Thickness of Reinforced Concrete	
						Percent Steel (7)	Length or Width of Slab, ft (8)
A	21.7	17.4	17.5	0.190	100[1]	0.356	100[1]
B	21.5	17.3	17.5	0.177	98	0.308	100[1]
C	17.5	14.1	14.5	0.156	85	0.080	56
D	13.5	10.8	11.0	0.178	84	0.309	100[1]

[1] Maximum length or width allowed.
Conversion Factors: Millimeters = 25.4 × inches, Meters = 0.3048 × feet

Table 13-2
Reinforced Concrete Pavement Design Example on a Lean Concrete Base Course

Traffic Area (1)	Thickness of Plain Concrete in. (2)	Plain[1] Concrete Overlay Thickness, in. (3)	Initial Reinforced Concrete Overlay Thickness, in. (4)	Design Thickness of Reinforced Concrete, in. (5)	Percent Steel (6)	Length or Width of Slab, ft (7)	Design Example Preselecting Thickness of Reinforced Concrete	
							Percent Steel (8)	Length or Width of Slab, ft (9)
A	21.7	19.8	16.0	16.0	0.200	100[2]	0.320	100[2]
B	21.5	19.6	15.7	16.0	0.180	100[2]	0.256	100[2]
C	17.5	15.4	12.4	12.5	0.184	95	0.052	42
D	13.5	11.2	9.0	9.0	0.200	85	0.093	53

[1] Thickness of plain concrete overlay determined using Equation 12-1.
[2] Maximum length or width allowed.
Conversion Factors: Millimeters = 25.4 × inches, Meters = 0.3048 × feet

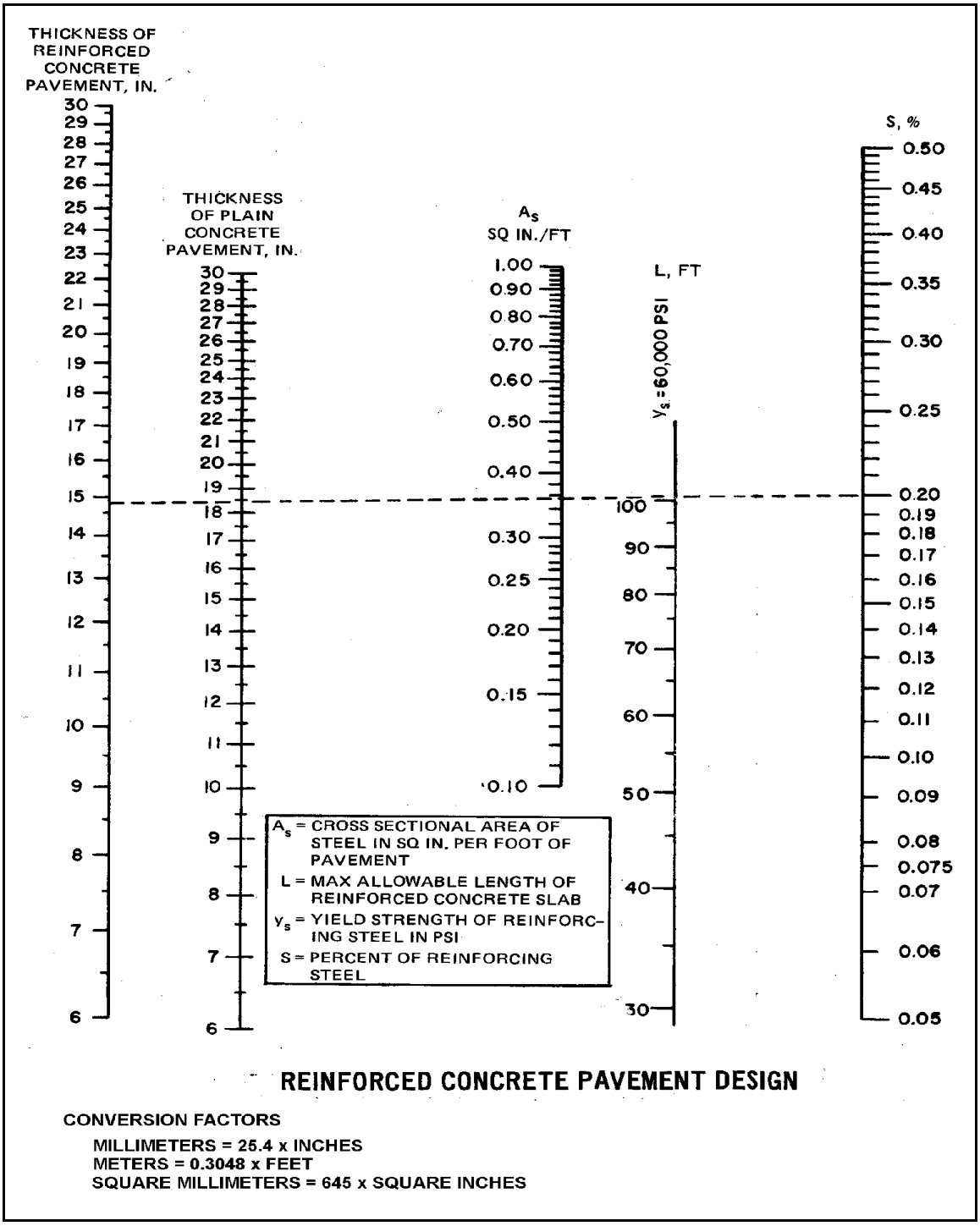


Figure 13-1. Reinforced concrete pavement design

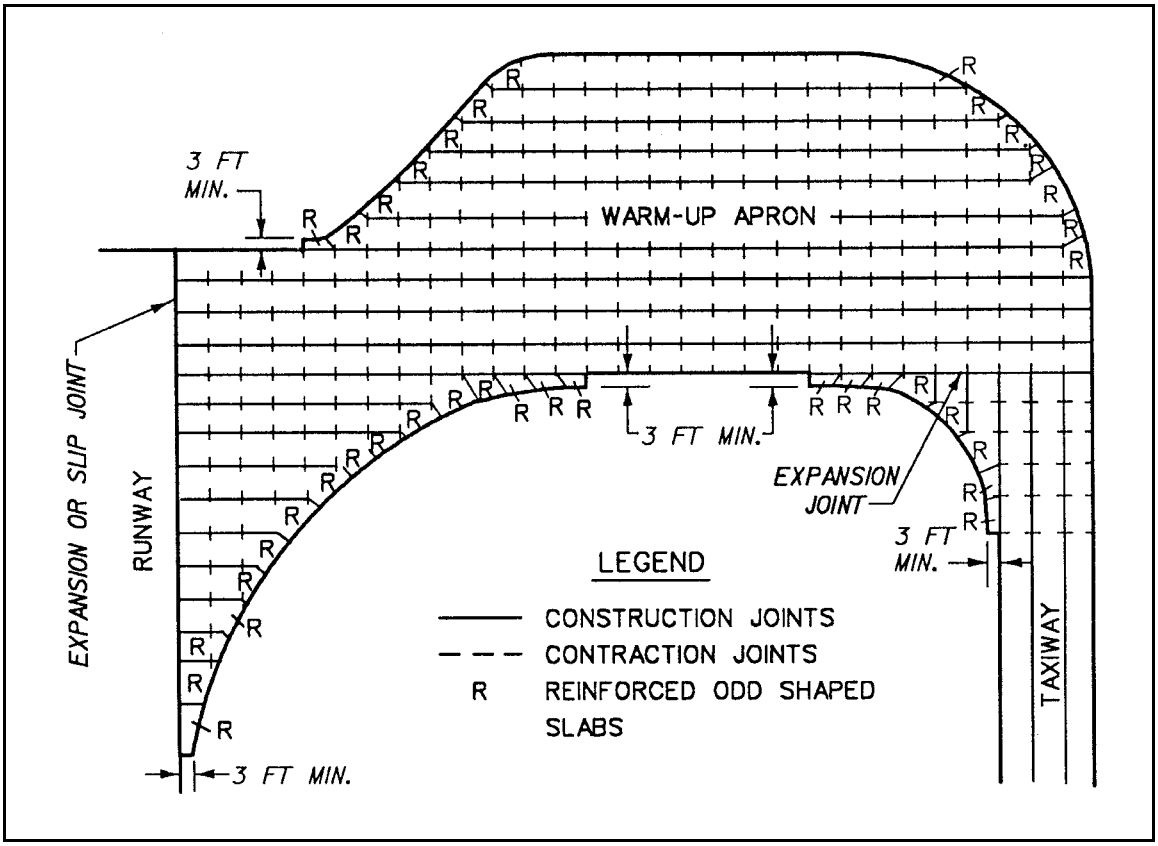
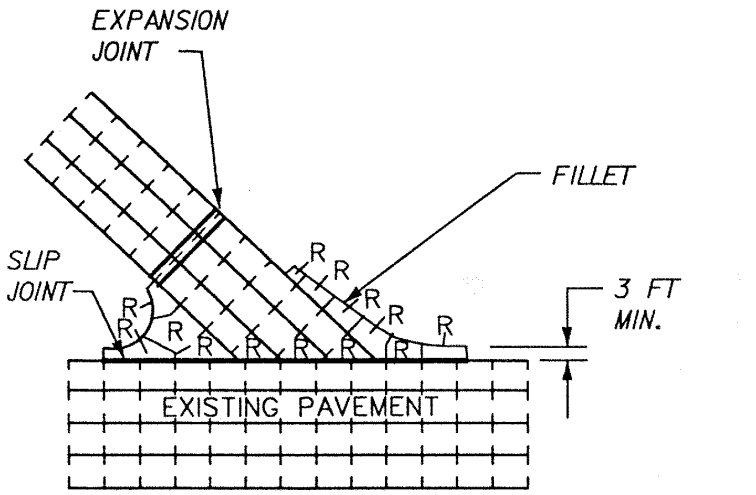
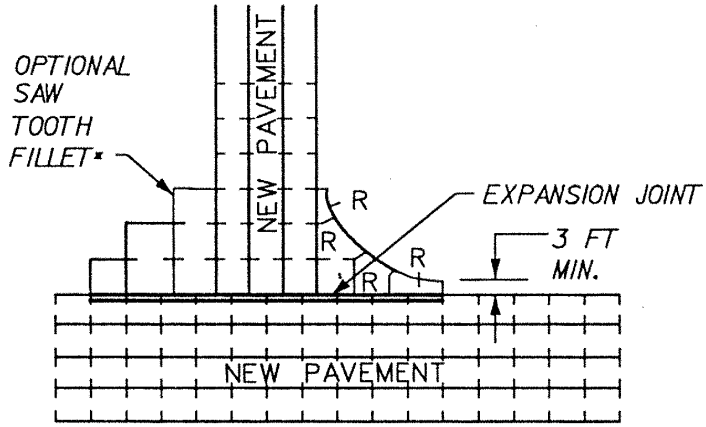


Figure 13-2. Typical layouts showing reinforcement of odd-shaped slabs and mismatched joints (Continued)





* A SAWTOOTH FILLET IS ACCEPTABLE IF LESS EXPENSIVE THAN A CIRCULAR FILLET.

LEGEND

———	CONSTRUCTION JOINTS
— — —	CONTRACTION JOINTS
═══	EXPANSION JOINT
━━━	SLIP JOINT
R	REINFORCED ODD SHAPED SLABS

Figure 13-2. (Concluded)

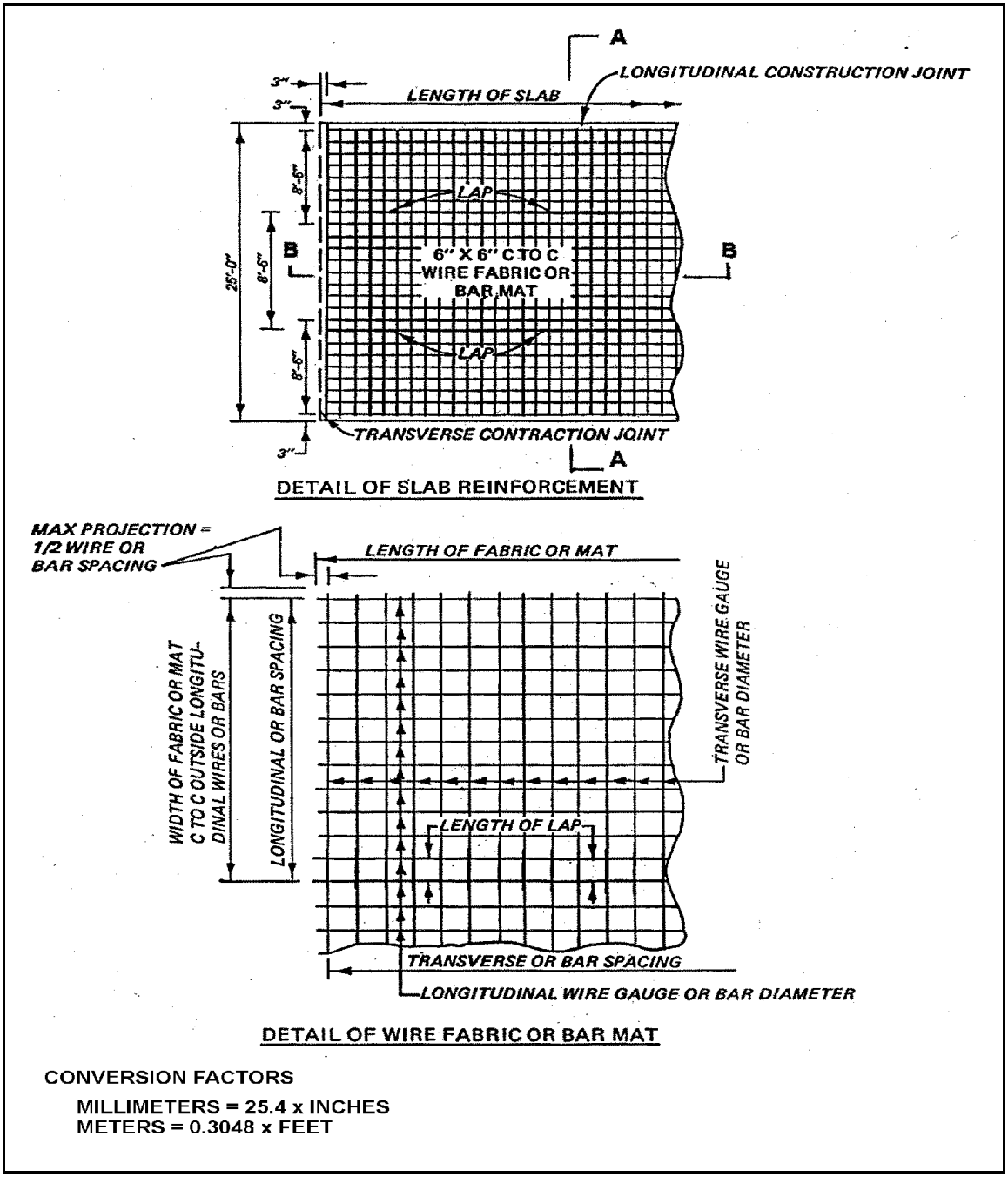
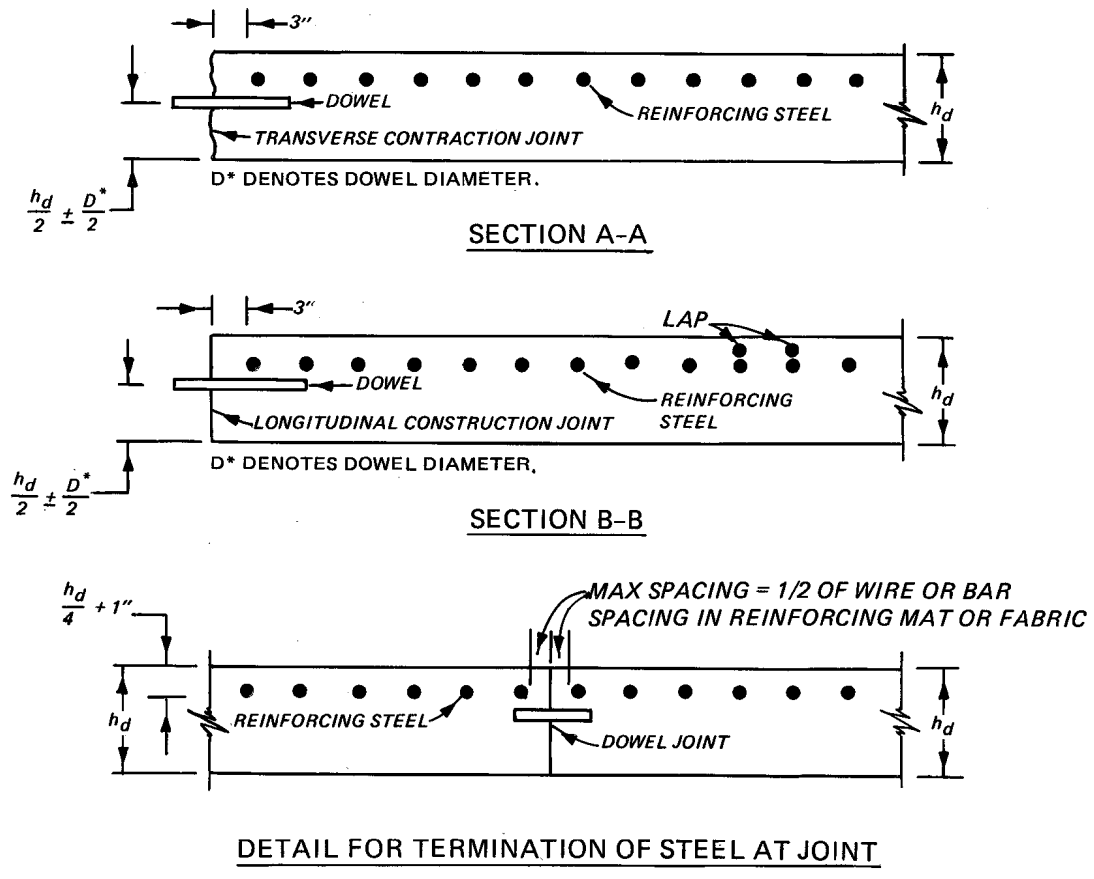


Figure 13-3. Reinforcing steel details (Continued)



NOTES: 1. h_d DENOTES REINFORCED CONCRETE PAVEMENT DESIGN THICKNESS
2. REINFORCING STEEL NOT CARRIED THROUGH JOINT
3. ALL JOINTS IN REINFORCED CONCRETE PAVEMENTS DOWELED EXCEPT THICKENED-EDGE AND TRANSVERSE CONSTRUCTION JOINTS

CONVERSION FACTORS

MILLIMETERS = 25.4 x INCHES

Figure 13-3. (Concluded)

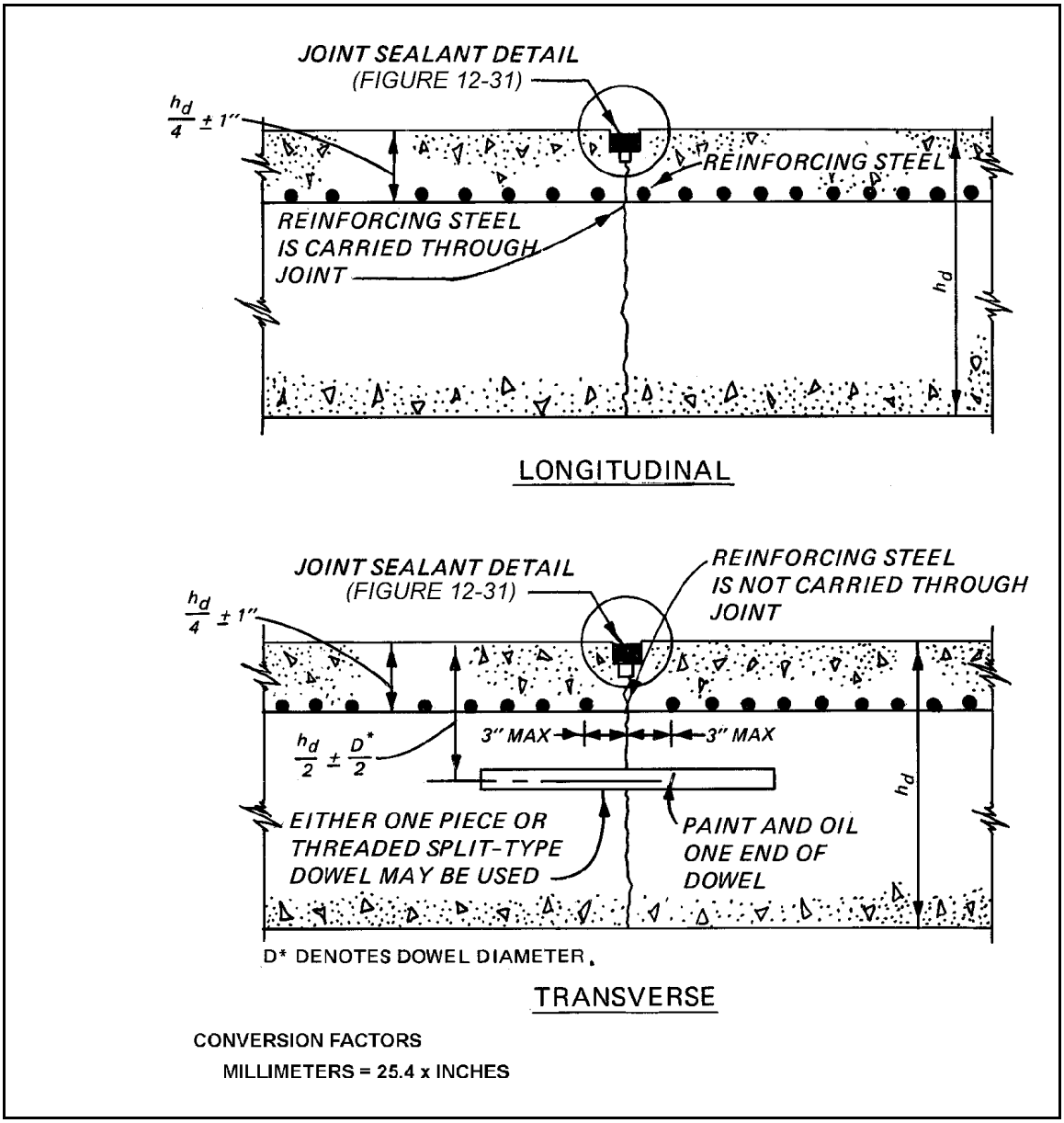


Figure 13-4. Contraction joints for reinforced concrete pavements

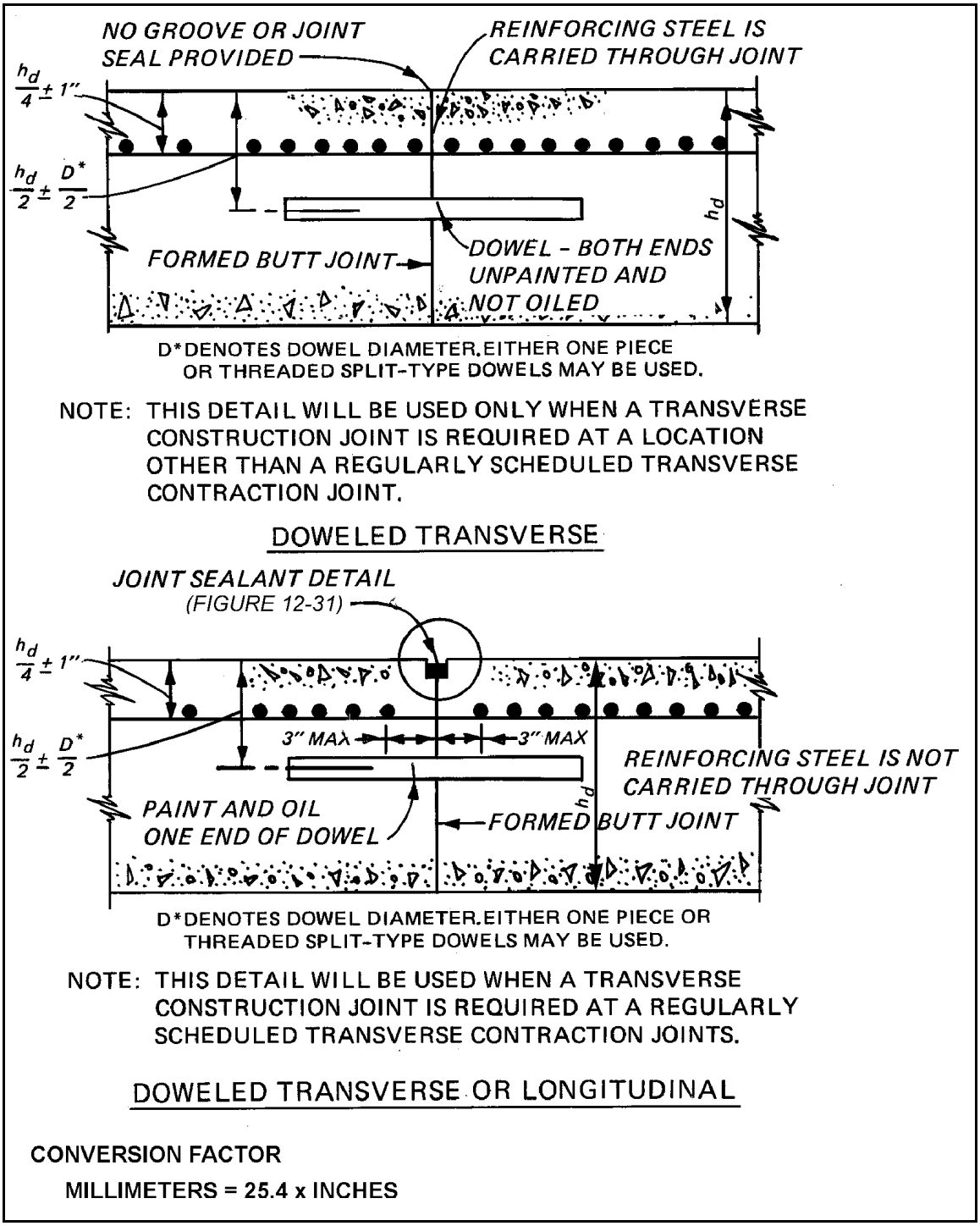
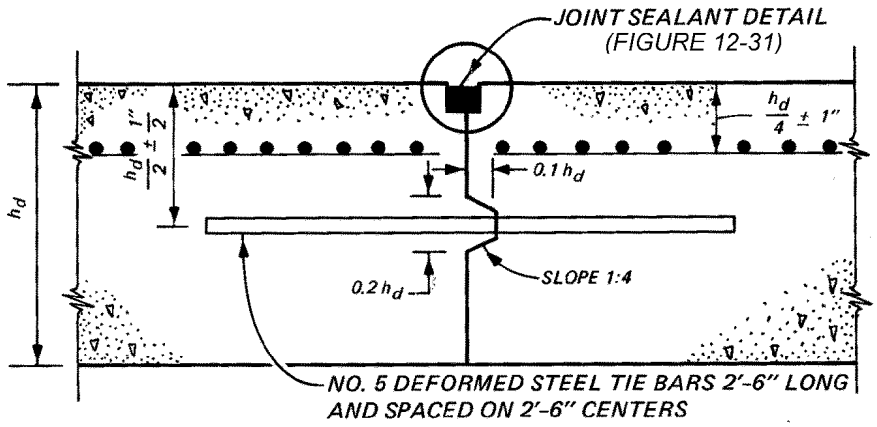
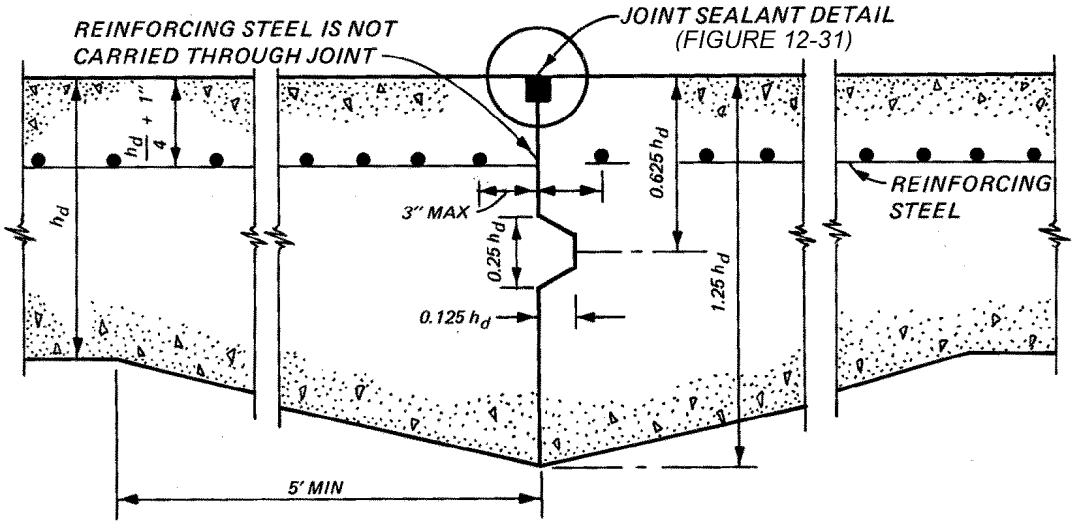


Figure 13-5. Construction joints for reinforced concrete pavements (Sheet 1 of 4)



KEYED AND TIED LONGITUDINAL



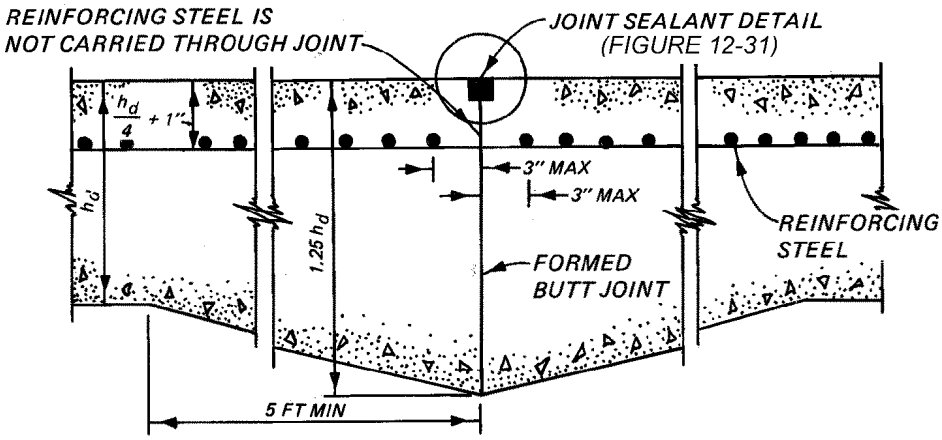
NOTE: A TOLERANCE OF ±1/16 INCH MAY BE ALLOWED
FOR KEY DIMENSIONS AND LOCATION

KEYED THICKENED EDGE LONGITUDINAL

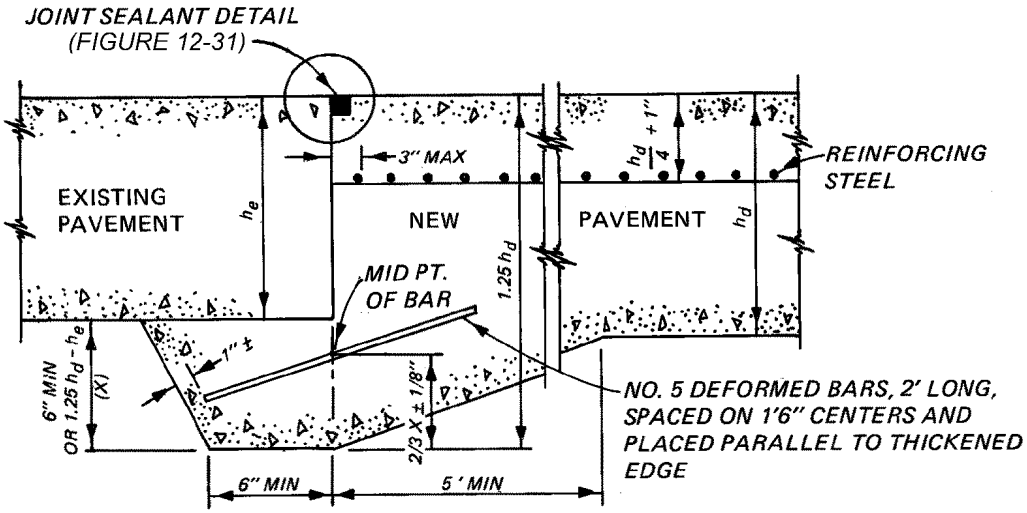
CONVERSION FACTORS

MILLIMETERS = 25.4 x INCHES

Figure 13-5. (Sheet 2 of 4)



THICKENED EDGE LONGITUDINAL

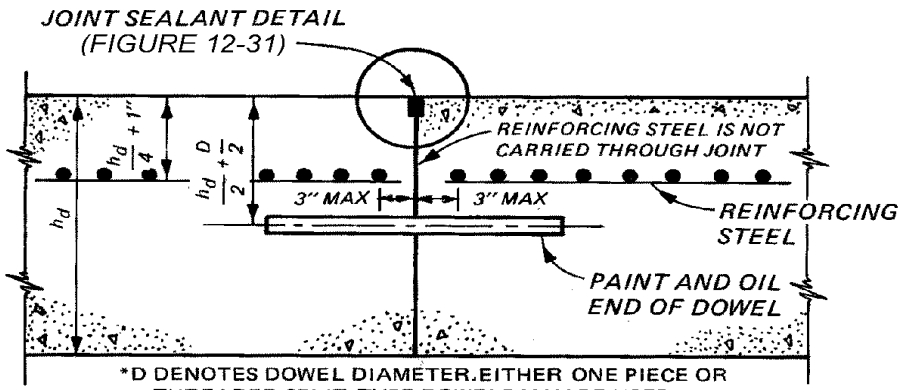


SPECIAL JOINT BETWEEN NEW
AND EXISTING PAVEMENTS

CONVERSION FACTORS

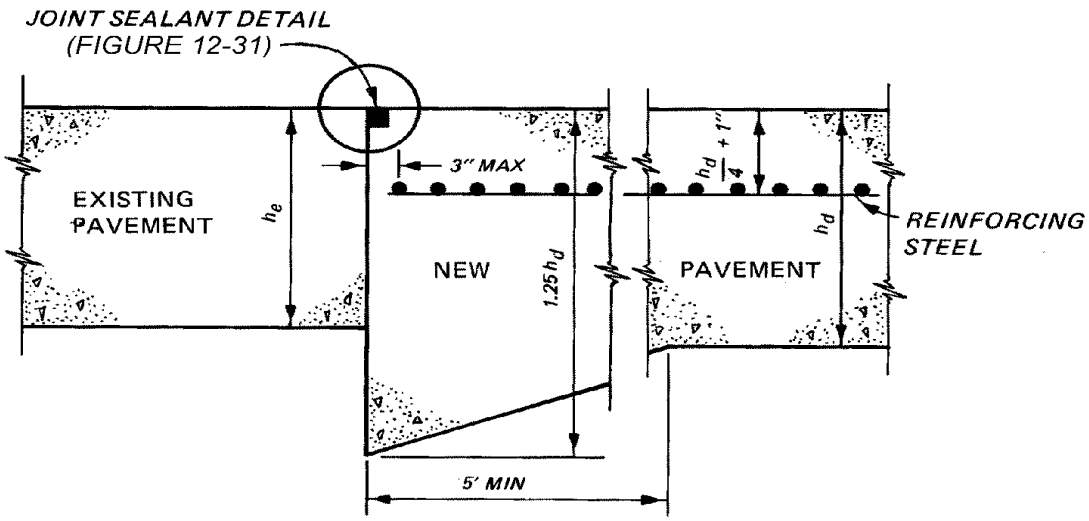
MILLIMETERS = 25.4 x INCHES
METERS = 0.3048 x FEET

Figure 13-5. (Sheet 3 of 4)



*D DENOTES DOWEL DIAMETER. EITHER ONE PIECE OR THREADED SPLIT-TYPE DOWELS MAY BE USED.

DOWELED JOINT BETWEEN
NEW AND EXISTING PAVEMENT



NOTE: THIS TYPE JOINT SHOULD BE USED ONLY WHEN EXISTING PAVEMENT IS TO BE REPLACED IN A SHORT PERIOD OF TIME, SINCE WITHOUT LOAD TRANSFER, IT WILL DETERIORATE QUICKLY.

THICKENED EDGE JOINT BETWEEN
NEW AND EXISTING PAVEMENTS

CONVERSION FACTORS

MILLIMETERS = 25.4 x INCHES
METERS = 0.3048 x FEET

Figure 13-5. (Sheet 4 of 4)

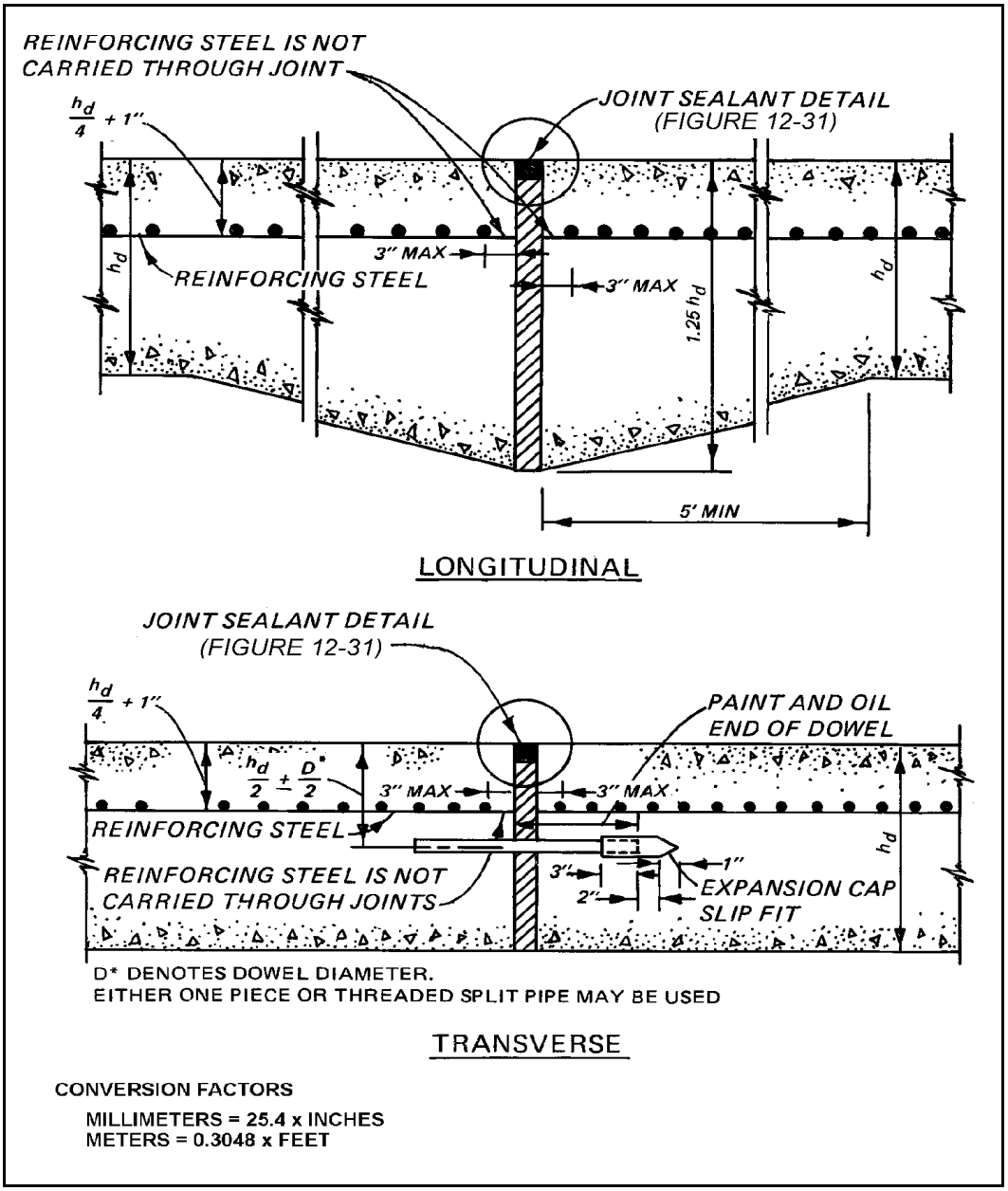


Figure 13-6. Expansion joints for reinforced concrete pavements

CHAPTER 14

FIBROUS CONCRETE PAVEMENT DESIGN

1. BASIS OF DESIGN. The design of fibrous concrete pavement is based upon limiting the ratio of the concrete flexural strength and the maximum tensile stress at the joint, with the load either parallel or normal to the edge of the slab, to a value found to give satisfactory performance in full-scale accelerated test tracks. Because of the increased flexural strength of the fibrous concrete and the bridging of fibers across cracks that develop in the concrete, the thickness can be significantly reduced; however, this results in a more flexible structure, which causes an increase in vertical deflections and potential for densification and/or shear failures in the foundation, pumping of the subgrade material, and joint deterioration. To protect against these latter factors, a limiting vertical deflection criterion has been applied to the thickness developed from the tensile stress criteria.

2. USES FOR FIBROUS CONCRETE. Although several types of fiber have been studied for concrete reinforcement, most of the experience has been with steel fibers, and the design criteria presented herein are limited to steel fibrous concrete. Fibrous concrete is a relatively new material for pavement construction and lacks a long-time performance history. Experience indicates that with time and number of passes logged on a fibrous concrete pavement, the fiber becomes exposed at the wearing surface and becomes an FOD problem. Because of this, its use will require approval of the Headquarters, U.S. Army Corps of Engineers (HQUSACE) (CEMP), HQ Air Force Command, or the Naval Facilities Engineering Command. The major uses to date have been for thin resurfacing or strengthening overlays where grade problems restrict the thickness of overlay that can be used. The use of fibrous concrete pavement should be based upon the economics involved. Fibrous concrete will not be used in Navy pavements.

3. MIX PROPORTIONING CONSIDERATIONS.

a. The design mix proportioning of fibrous concrete will be determined by a laboratory study. Typical mix proportions are shown on Table 14-1. The following are offered as guides and to establish limits where necessary for the use of the design criteria included herein. Additional details may be found in TM 5-822-7/AFM 88-6, Chapter 8.

Table 14-1
Range of Proportions for Normal-Weight Fibrous Concrete[1]

	9.5-mm (3/8-in.) Maximum Sized Aggregate	19-mm (3/4-in.) Maximum Sized Aggregate
Cement kg/m^3 (lb/yd^3)	355-590 (600-1,000)	295-535 (500-900)
Water-cement ratio	0.35-0.45	0.40-0.50
Percent of fine to coarse aggregate	45-60	45-55
Entrained air content (percent)	4-7	4-6
Fiber content (volume percent)		
Deformed steel fiber	0.4-0.9	0.3-0.8
Smooth steel fiber	0.9-1.8	0.8-1.6

[1] From ACI 544.1R-82, used with permission of the American Concrete Institute.

b. The criteria contained herein are based upon fibrous concrete containing 1 to 2 percent by volume 45 to 113 kilograms (100 to 250 pounds) of steel fibers per cubic yard of concrete, and fiber contents within this range are recommended.

c. Most experience to date has been with fibers 25 to 38 millimeters (1 to 1-½ inches) long, and for use of the criteria contained herein, fiber lengths within this range are recommended.

d. For proper mixing, the maximum aspect ratio (length to diameter or equivalent diameter) of the fibers should be about 100.

e. The large surface-area-to-volume ratio of the steel fibers requires an increase in the paste necessary to ensure that the fibers and aggregates are coated. To accomplish this, cement contents of 445 to 535 kg/m^3 (750 to 900 lb/yd^3) of concrete are common. The cement content may be all portland cement or a combination of portland cement and up to 25 percent by volume of fly ash or other pozzolans.

f. Maximum size coarse aggregates should fall between 9.5 and 19 millimeters (3/8 and 3/4 inches). The percent of fine to coarse aggregate has been between 45 and 60 percent on typical projects using fibrous concrete.

4. THICKNESS DETERMINATION. The required thickness of fibrous concrete will be a function of the design concrete flexural strength, the modulus of soil reaction, the thickness and flexural modulus of elasticity of stabilized material if used, the aircraft gross weight, the volume of traffic, the type of traffic area, and the allowable vertical deflection. When stabilized material is not used, the required thickness of fibrous concrete is determined directly from the appropriate chart (Figures 14-1 through 14-9). If the base or subgrade is stabilized and meets the minimum strength requirements of TM 5-822-14/ AFJMAN 32-1019, the stabilized layer will be treated as a low-strength base and the design will be made using Equation 12-1. The resulting thickness must then be checked for allowable deflection. The minimum thickness for fibrous concrete pavements will be 102 millimeters (4 inches).

5. ALLOWABLE DEFLECTION FOR FIBROUS CONCRETE PAVEMENT. The elastic deflection that fibrous concrete pavements experience must be limited to prevent overstressing of the foundation material and thus premature failure of the pavement. Curves are provided (Figures 14-10 through 14-18) for the determination of the vertical elastic deflection that a pavement will experience when loaded and must be checked for all design aircraft. Use of the curves requires three different inputs: slab thickness, subgrade modulus, and gross weight of the design aircraft. The modulus value to use for stabilized layers is determined from Figure 9-1. The slab thickness is that which is determined from Figures 14-1 to 14-19. The computed vertical elastic deflection is then compared with appropriate allowable deflections determined from Figure 14-19 or, in the case of shoulder design, with an allowable deflection value of 0.15 millimeters (0.06 inches). If the computed deflection is less than the allowable deflection, the thickness meets allowable deflection criteria and is acceptable. If the computed deflection is larger than the allowable deflection, the thickness must be increased or a new design initiated with a modified value for either concrete flexural strength or subgrade modulus. The process must be repeated until a thickness based upon the limiting stress criterion will also have a computed deflection equal to or less than the allowable value. Should the vertical deflection criteria indicate the need for a thickness increase greater than that required by the limiting stress criteria, the thickness increase should be limited to that thickness required for plain concrete with a flexural strength of 6.2 MPa (900 psi).

6. JOINTING. The jointing types and designs discussed for plain concrete pavements generally apply to fibrous concrete pavement. For the mix proportioning in Table 14-1, the maximum spacing of

contraction joints will be the same as for plain concrete, except that for thicknesses of 102 to 152 millimeters (4 to 6 inches), the maximum spacing will be 3.8 meters (12.5 feet). Joints in pavements 152 millimeters (6 inches) or greater in thickness will be cut one-third of the depth of the pavement and joints less than 152 millimeters (6 inches) long will be cut one-half the depth of the pavement. Longitudinal construction joints may be either doweled, keyed, keyed and tied, or thickened-edge with a key, in which case the key dimensions will be based upon the thickened-edge thickness. The keyed and tied construction joint will be limited to a width of 30.5 meters (100 feet). For widths greater than 30.5 meters (100 feet), combinations of keyed and tied, doweled, or thickened-edge-type joints may be used. Sealing of joints in fibrous concrete will follow the criteria presented in Chapter 12.

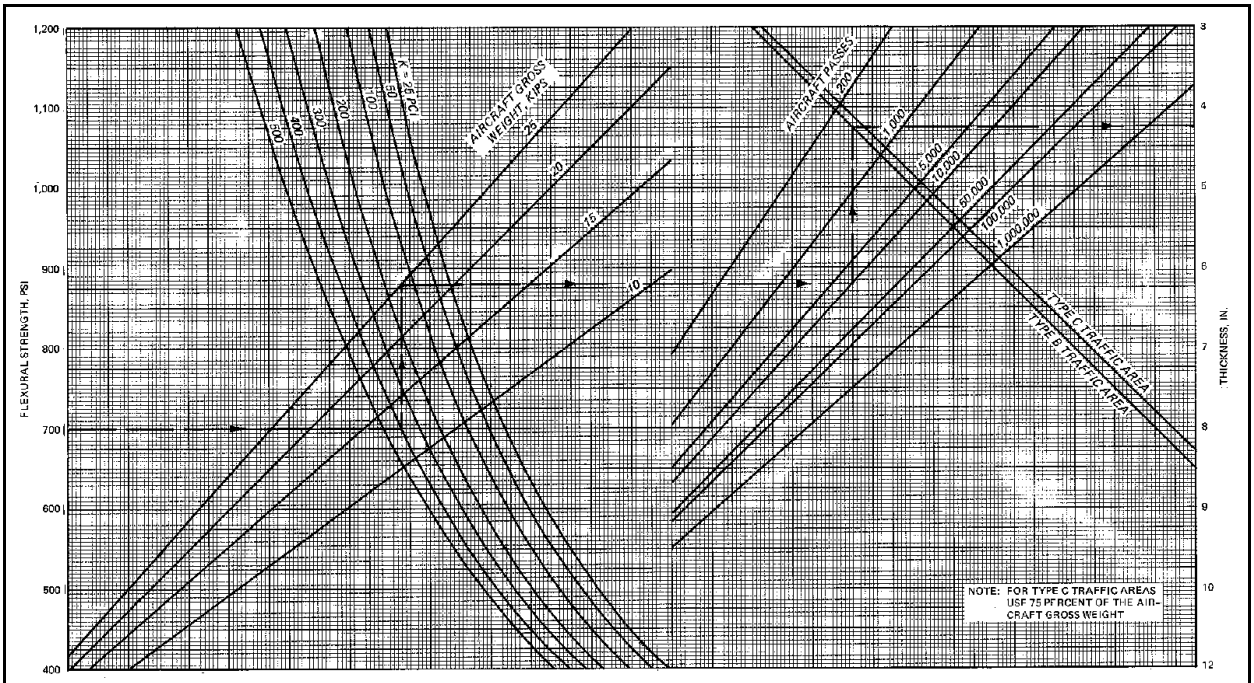
7. EXAMPLE OF FIBROUS CONCRETE PAVEMENT DESIGN.

a. General. An Air Force medium-load airfield is to be designed using fibrous concrete. On-site and laboratory investigations have yielded the following data required for design: (a) subgrade material is a silty sand; (b) modulus of subgrade reaction is 54 kPa/mm (200 pci); (c) an available source of crushed gravel meets the base course requirements; (d) frost does not enter subgrade; and (e) 90-day flexural strength is 6.9 MPa (1,000 psi) with 0.15 percent steel fibers.

b. Example Design—Slab On Grade. Figure 14-5 is entered with the subgrade k, concrete flexural strength, and the pavement thickness determined for the various traffic areas as follows:

Traffic Area	Thickness mm (in.)	Computed Deflection mm (in.)	Allowable Deflection mm (in.)
A	265 (10.5)	0.13 (0.050)	0.13 (0.050)
B	265 (10.5)	0.13 (0.050)	0.14 (0.053)
C	215 (8.5)	0.12 (0.045)	0.14 (0.053)
D	152 (6.0)	0.16 (0.062)	0.29 (0.114)

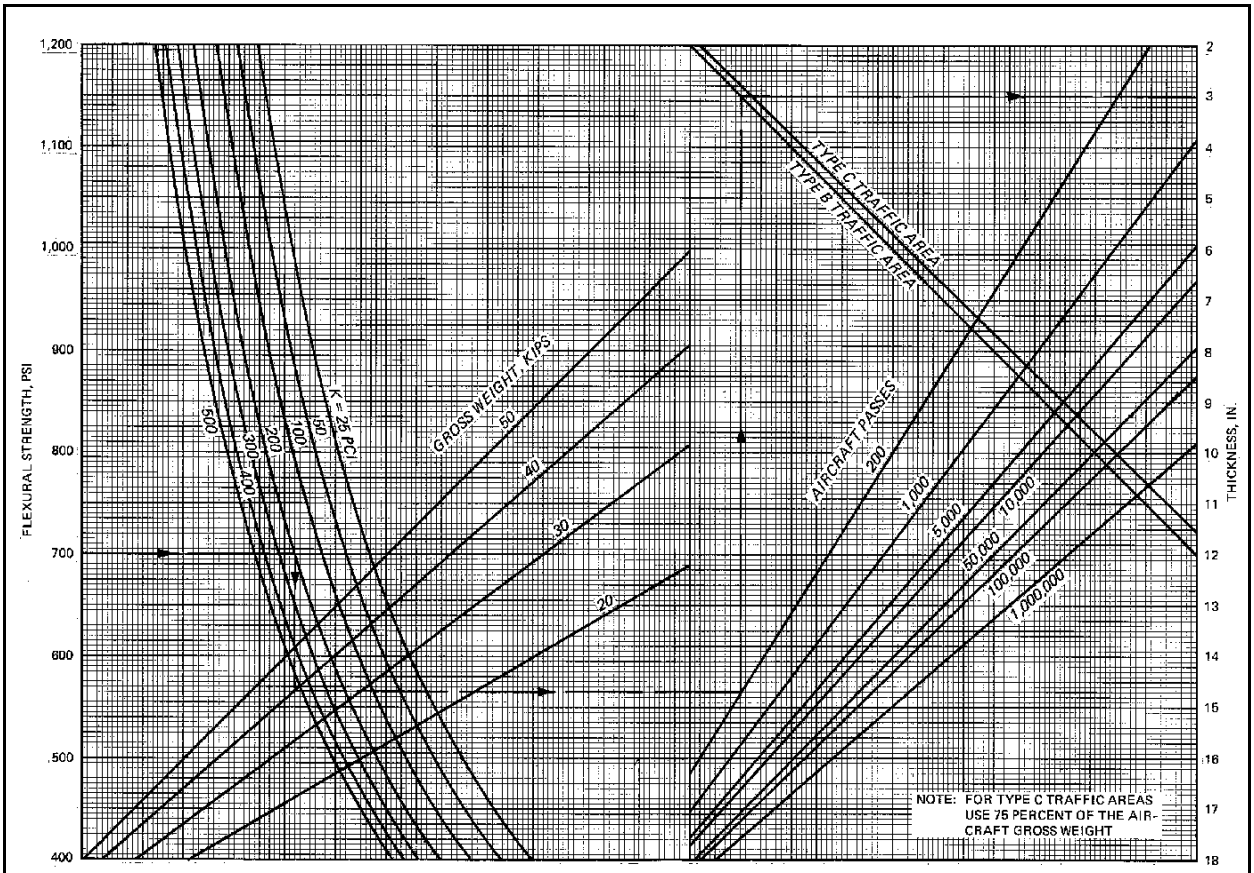
Since the medium-load pavement is designed for the F-15, C-141, and B-52, deflections must be determined for each aircraft. Therefore, by entering Figures 14-13, 14-14, and 14-15 with these thicknesses, the computed deflections for all aircraft may be determined and the controlling value is shown in the tabulation. It should be noted that a comparison of the computed deflections with the allowable deflections from Figure 14-19 reveals that the thicknesses determined by the allowable stress criterion are satisfactory, since the allowable deflections are equal to or greater than the computed deflections for all traffic areas.



CONVERSION FACTORS

MEGAPASCALS = 0.006894 x PSI
KILOPASCALS PER MILLIMETER = 0.271 x PCI
KILOGRAMS = 0.453 x POUNDS
MILLIMETERS = 25.4 x INCHES

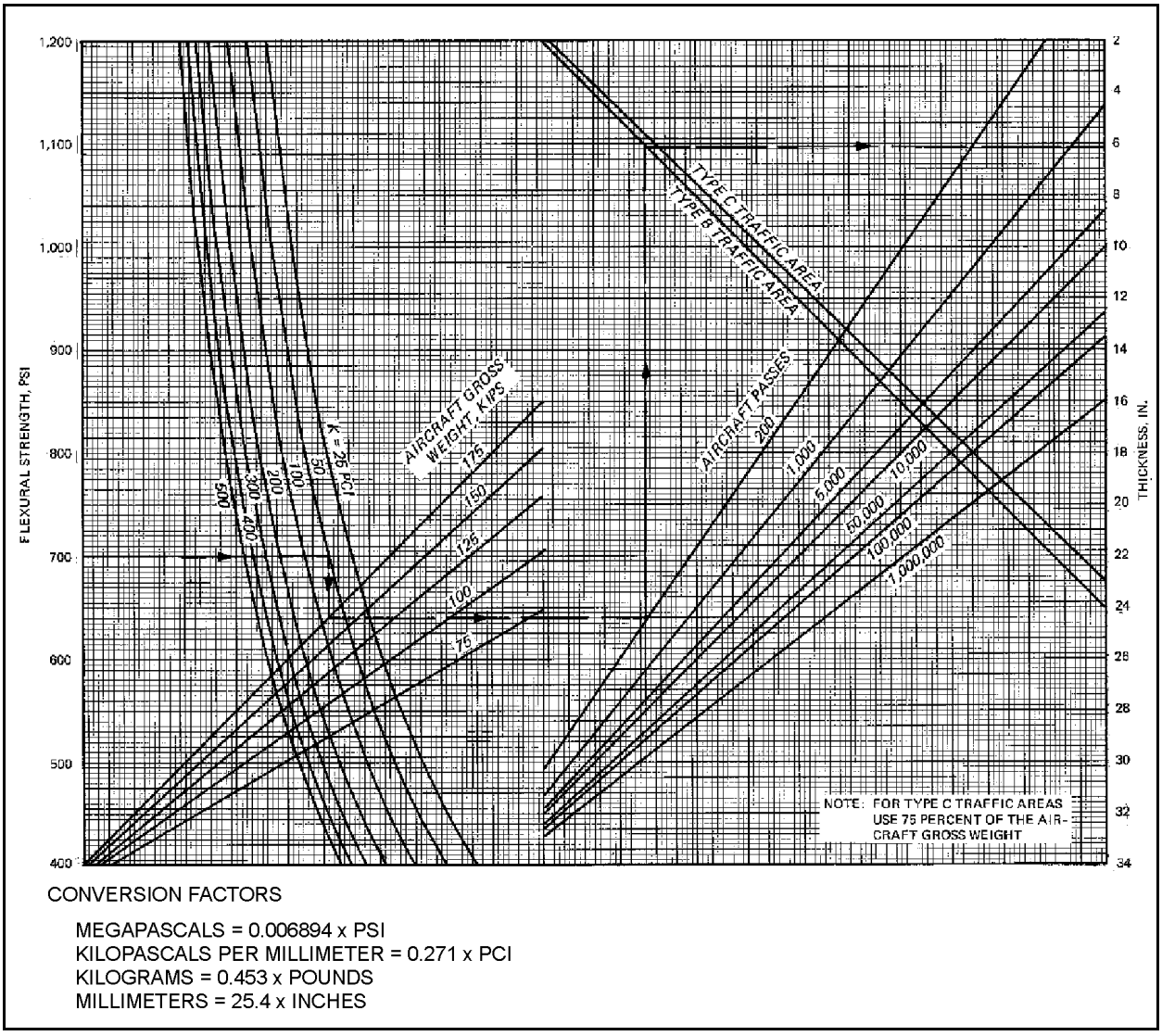
Figure 14-1. Fibrous concrete pavement design curves for UH-60



CONVERSION FACTORS

MEGAPASCALS = 0.006894 x PSI
KILOPASCALS PER MILLIMETER = 0.271 x PCI
KILOGRAMS = 0.453 x POUNDS
MILLIMETERS = 25.4 INCHES

Figure 14-2. Fibrous concrete pavement design curves for CH-47



CONVERSION FACTORS

MEGAPASCALS = 0.006894 x PSI
KILOPASCALS PER MILLIMETER = 0.271 x PCI
KILOGRAMS = 0.453 x POUNDS
MILLIMETERS = 25.4 x INCHES

Figure 14-3. Fibrous concrete pavement design curves for C-130

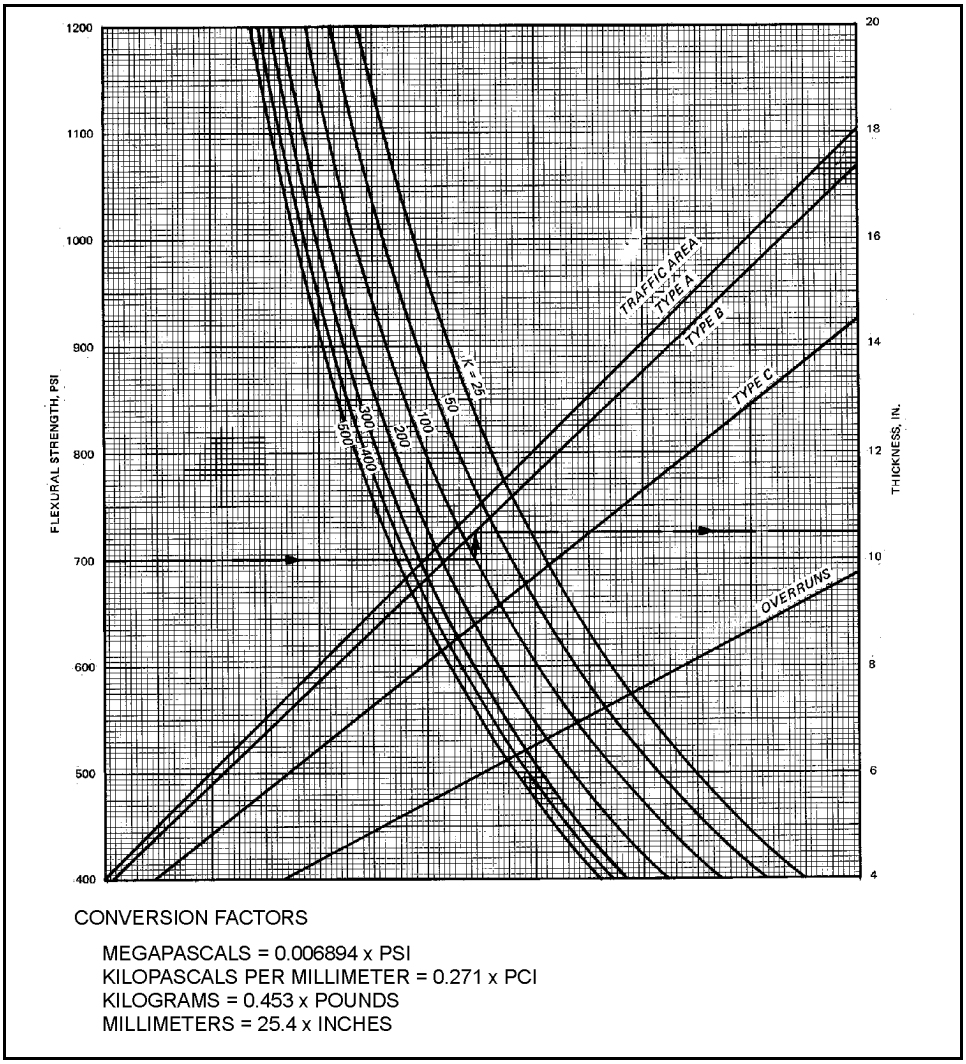
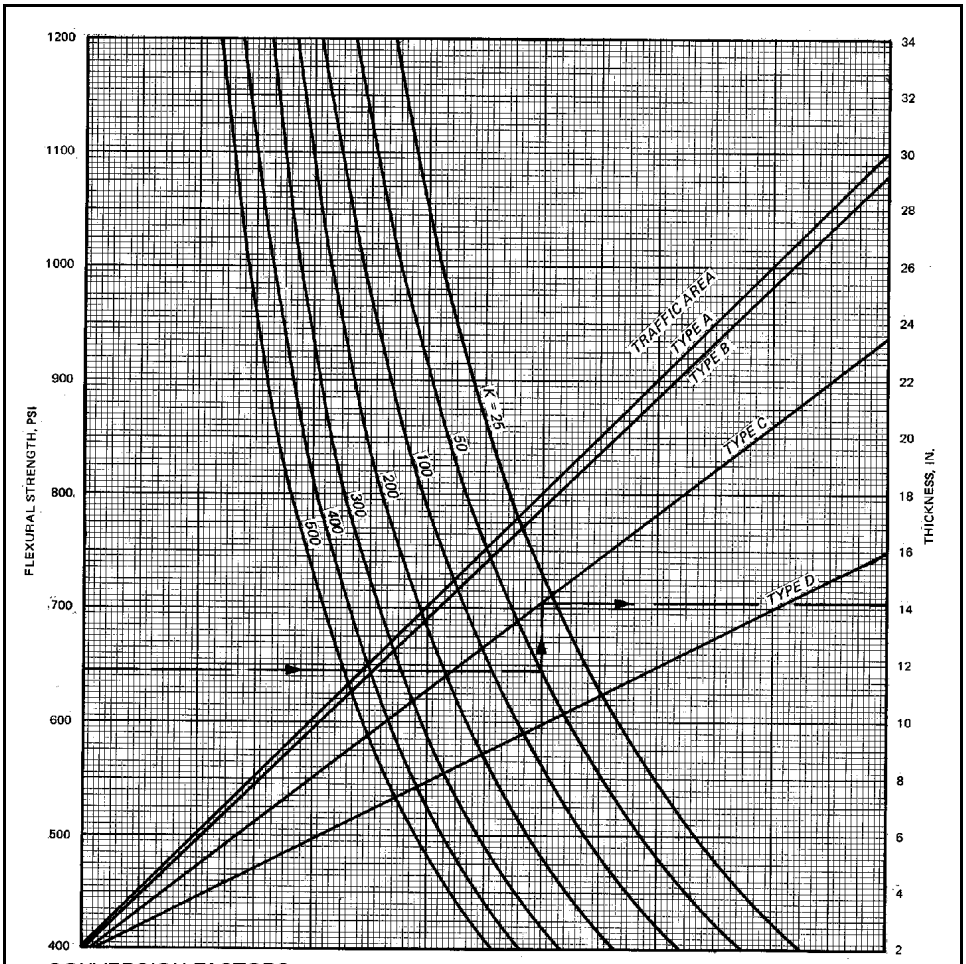


Figure 14-4. Fibrous concrete pavement design curves for Air Force light-load airfields



CONVERSION FACTORS

MEGAPASCALS = 0.006894 x PSI
KILOPASCALS PER MILLIMETER = 0.271 x PCI
KILOGRAMS = 0.453 x POUNDS
MILLIMETERS = 25.4 x INCHES

Figure 14-5. Fibrous concrete pavement design curves for Air Force medium-load airfields

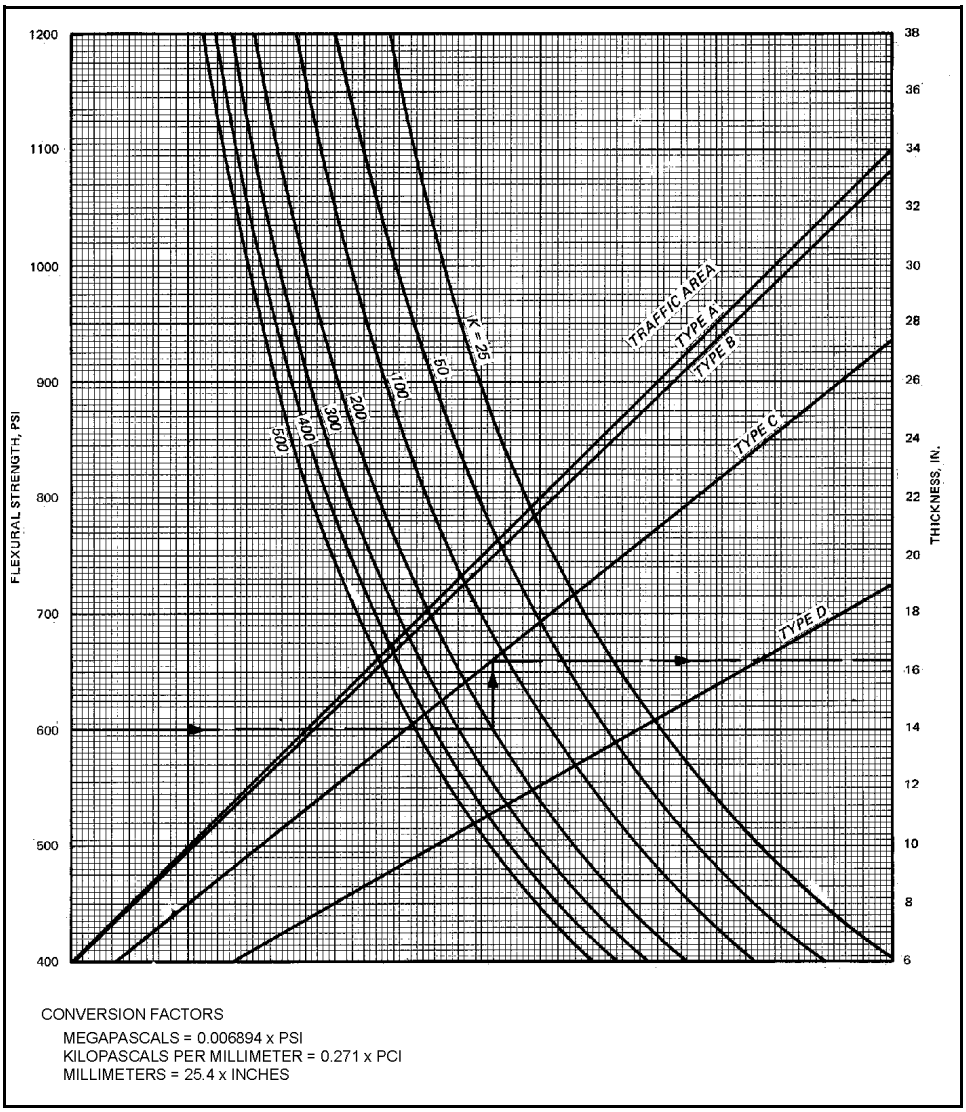


Figure 14-6. Fibrous concrete pavement design curves for Air Force heavy-load airfields

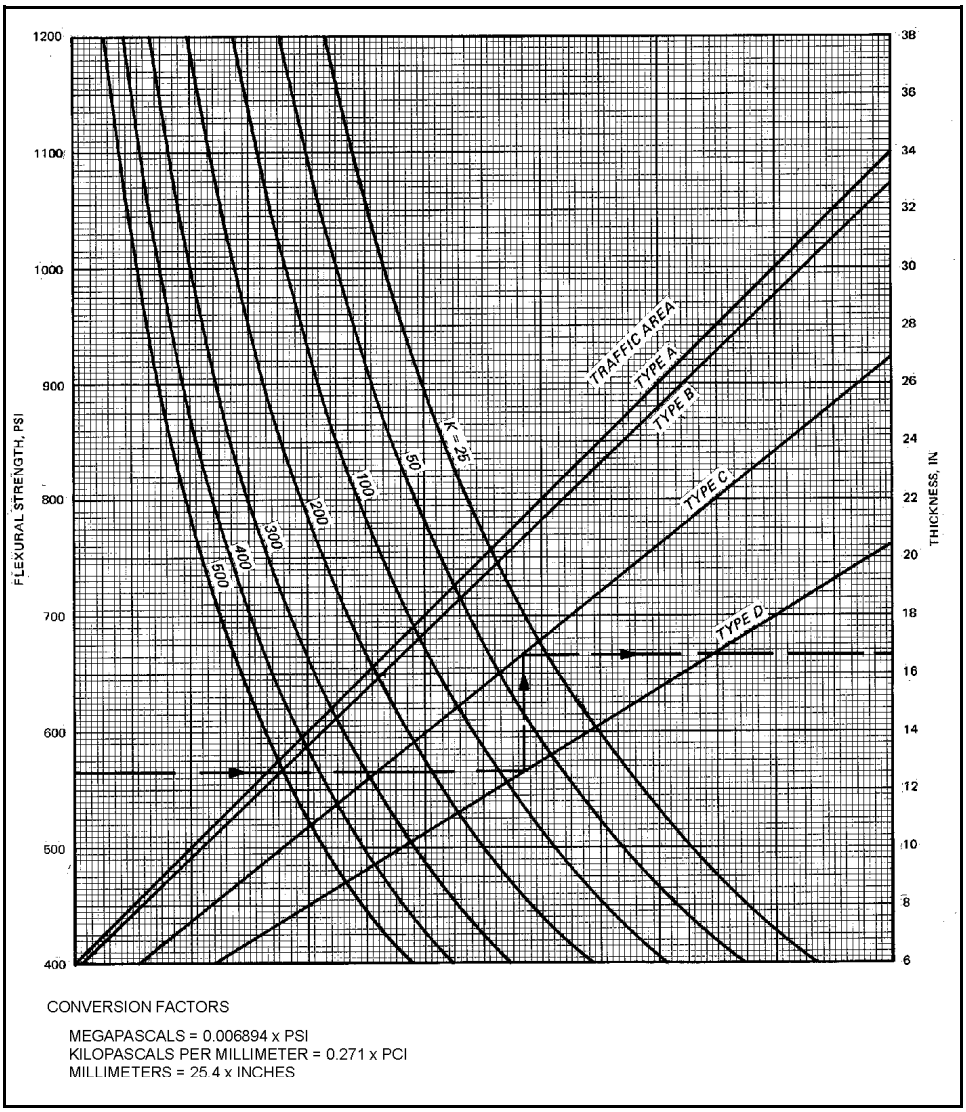
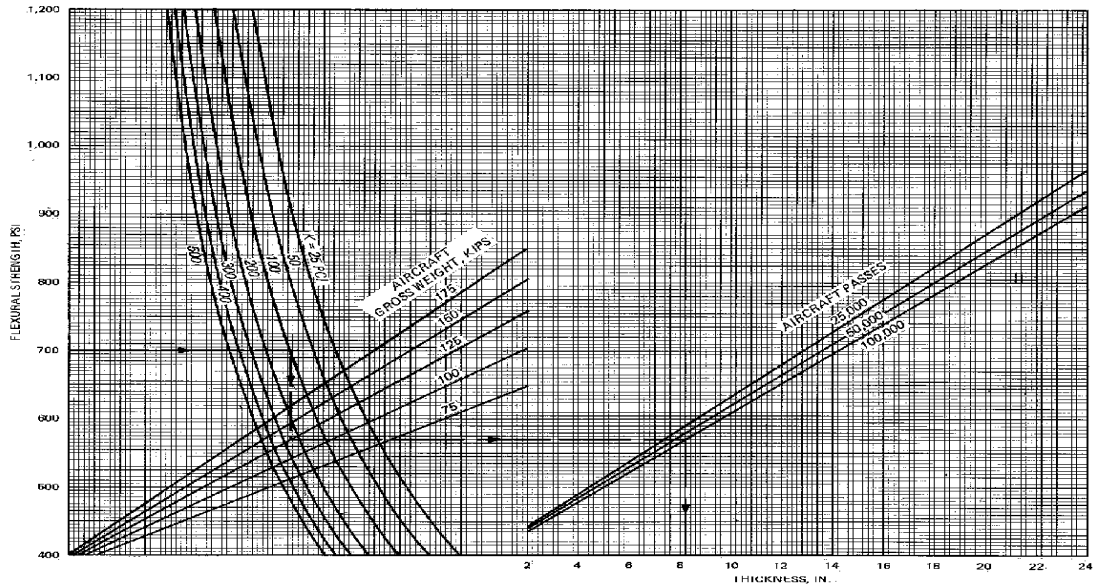


Figure 14-7. Fibrous concrete pavement design curves for Air Force modified heavy-load airfields



CONVERSION FACTORS

MEGAPASCALS = 0.006894 x PSI
KILOPASCALS PER MILLIMETER = 0.271 x PCI
KILOGRAMS = 0.453 x POUNDS
MILLIMETERS = 25.4 x INCHES

Figure 14-8. Fibrous concrete pavement design curves for Air Force shortfield airfields

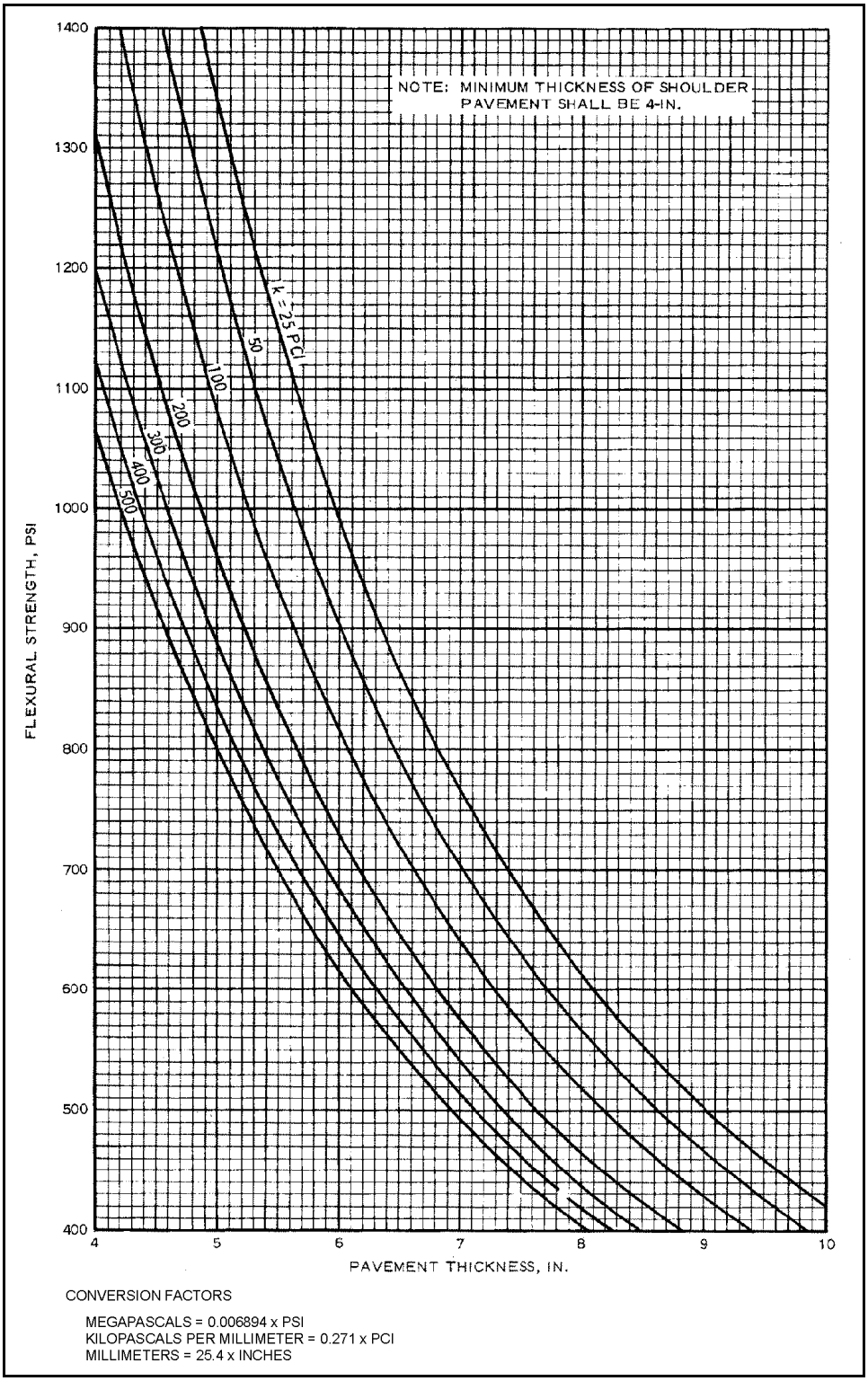
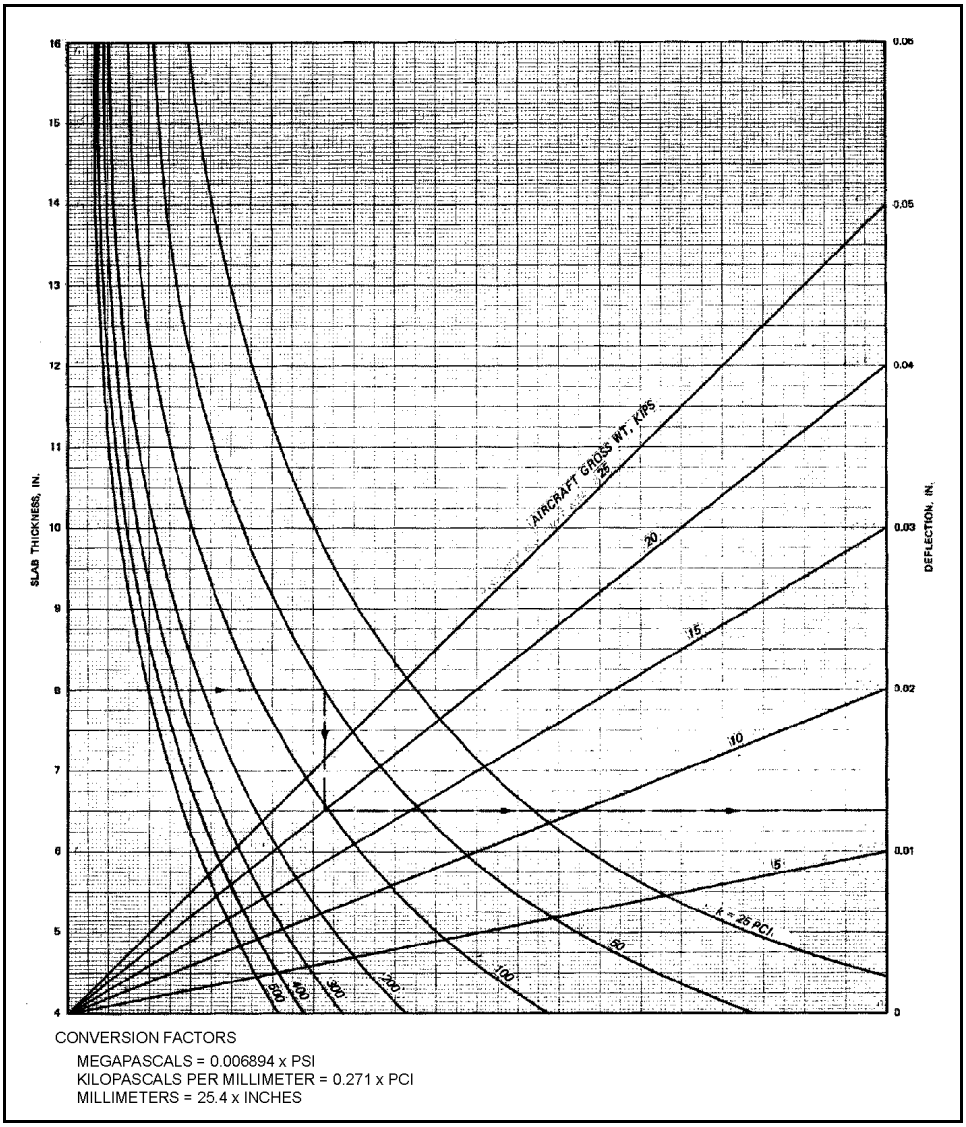


Figure 14-9. Fibrous concrete pavement design curves for shoulders



CONVERSION FACTORS

MEGAPASCALS = 0.006894 x PSI
KILOPASCALS PER MILLIMETER = 0.271 x PCI
MILLIMETERS = 25.4 x INCHES

Figure 14-10. Deflection curves for UH-60

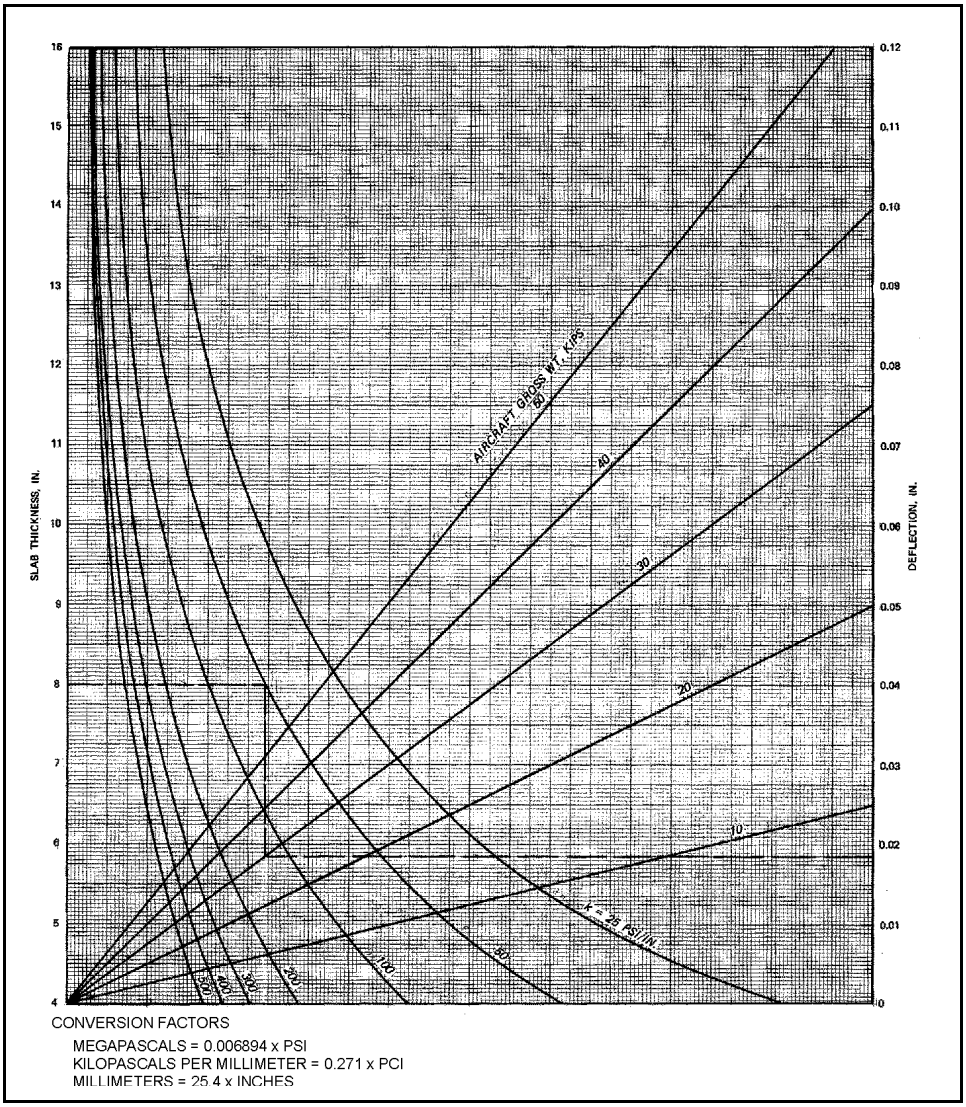


Figure 14-11. Deflection curves for CH-47

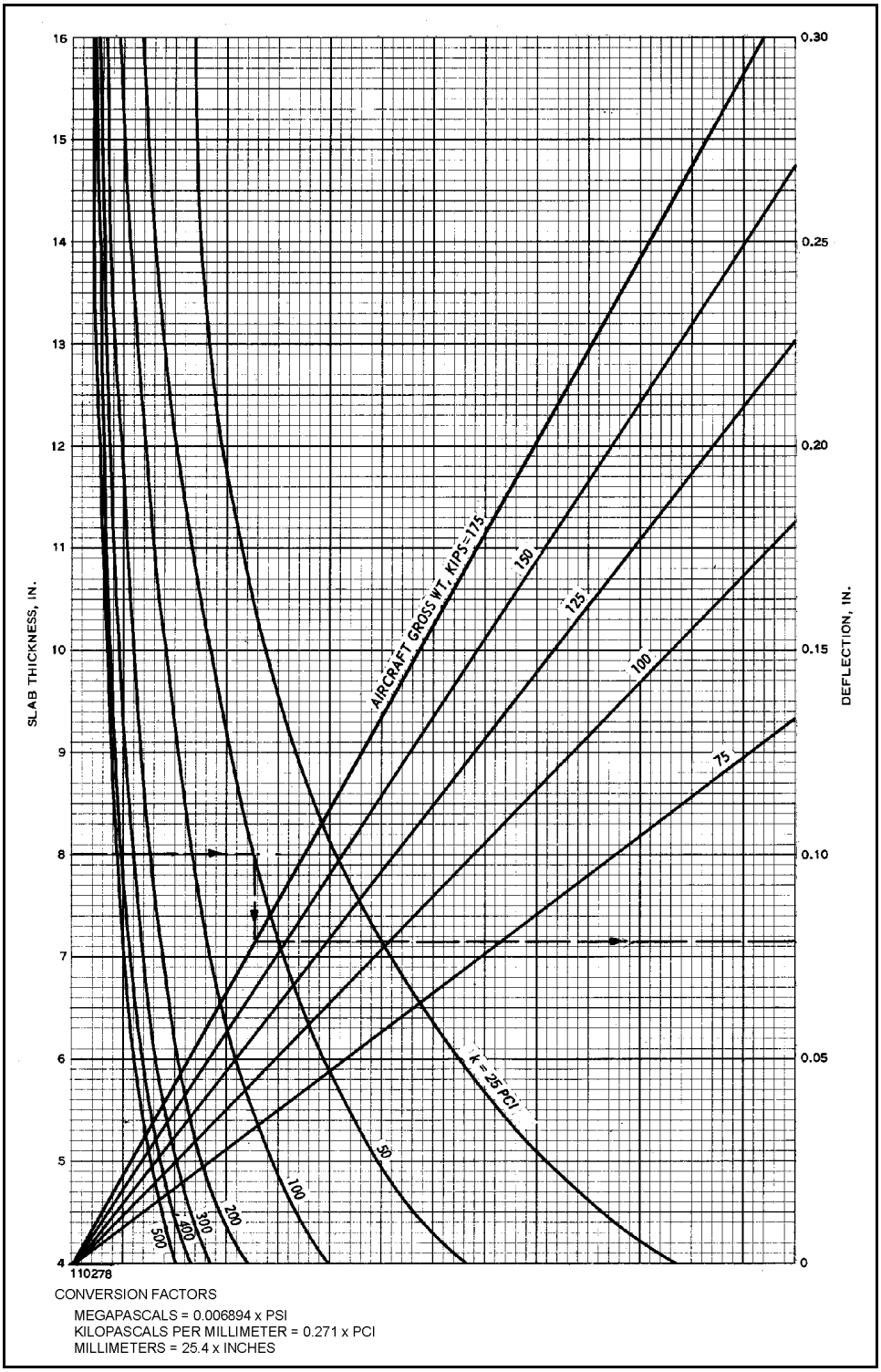


Figure 14-12. Deflection curves for C-130

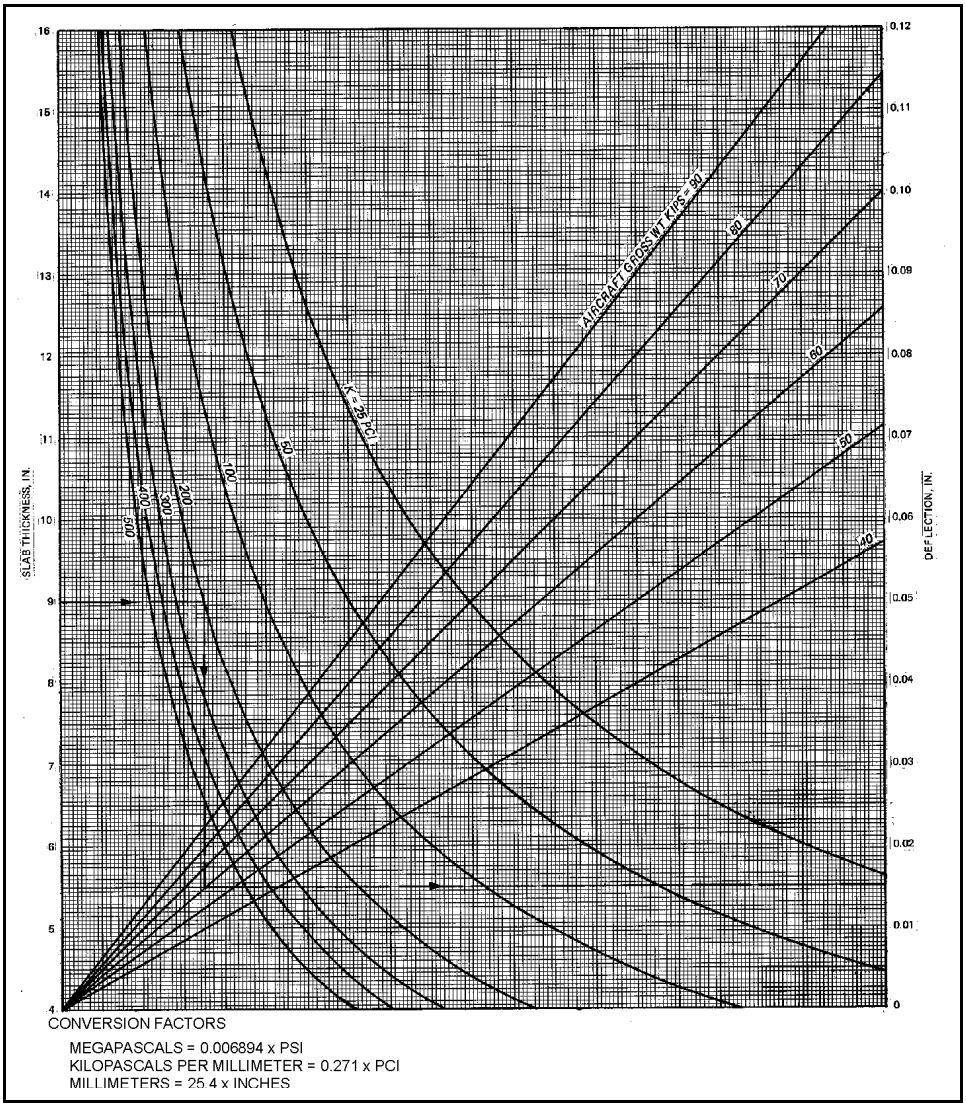


Figure 14-13. Deflection curves for Air Force light-load pavements

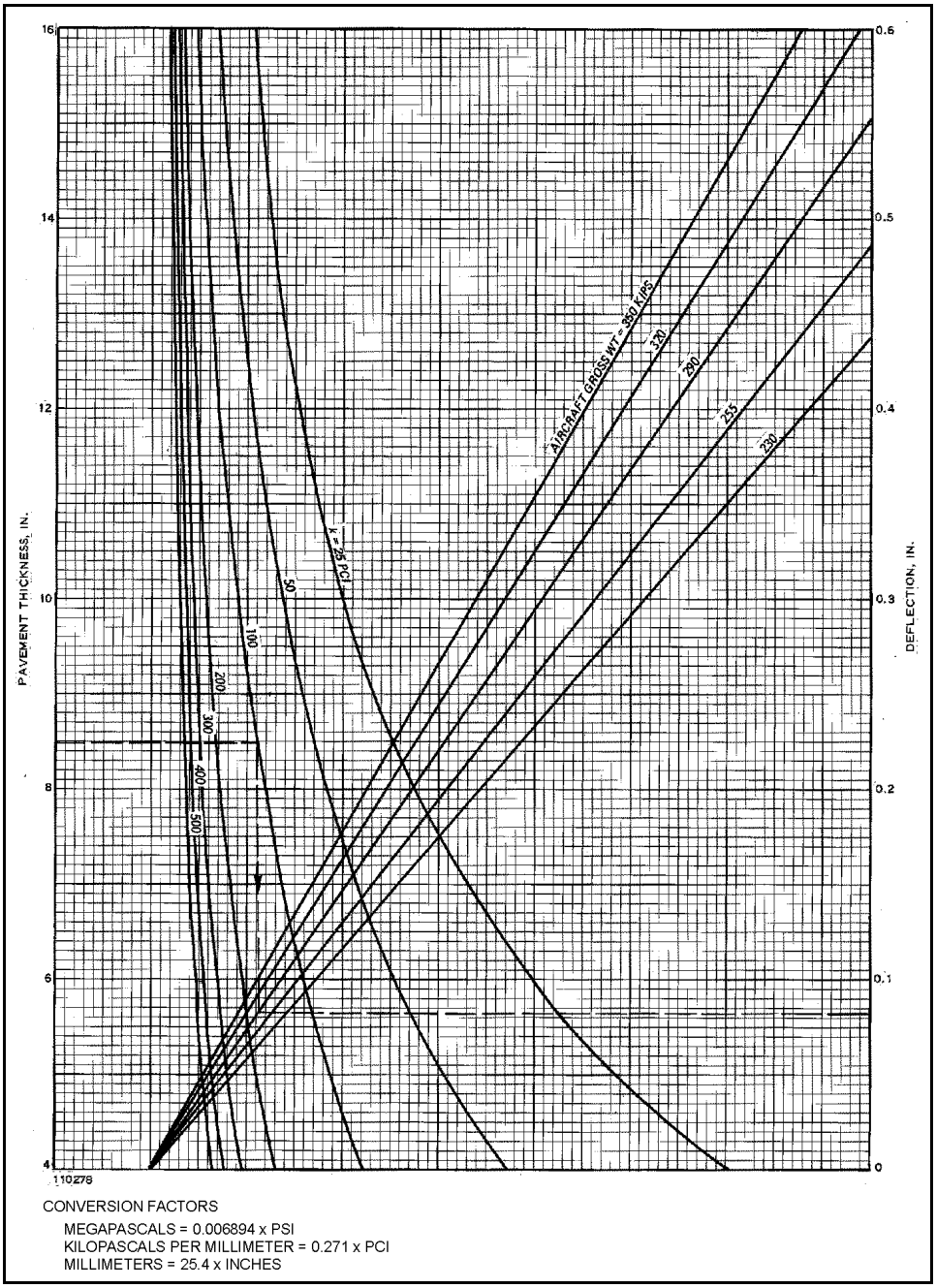


Figure 14-14. Deflection curves for Air Force medium-load pavements

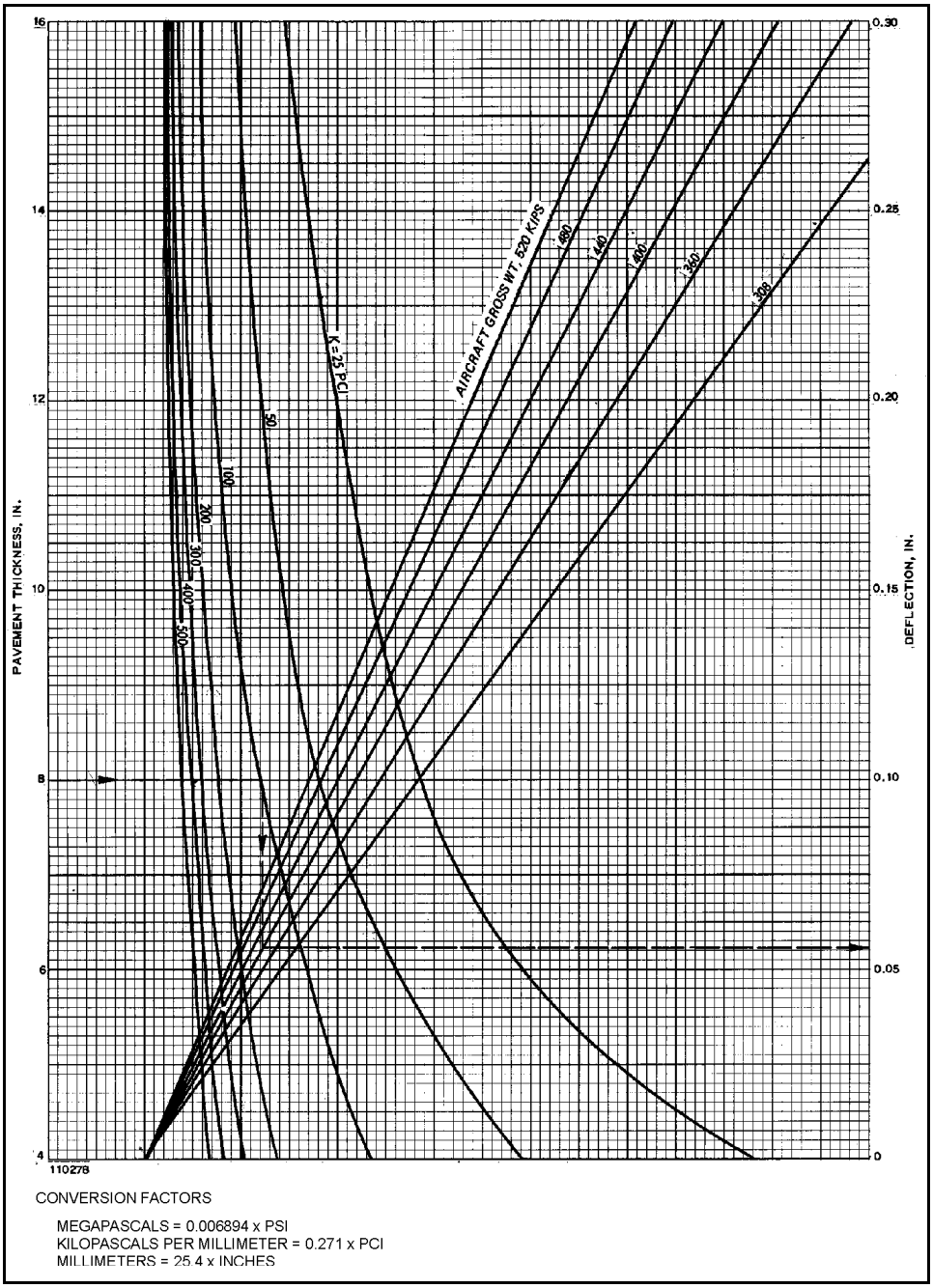


Figure 14-15. Deflection curves for Air Force heavy-load pavements

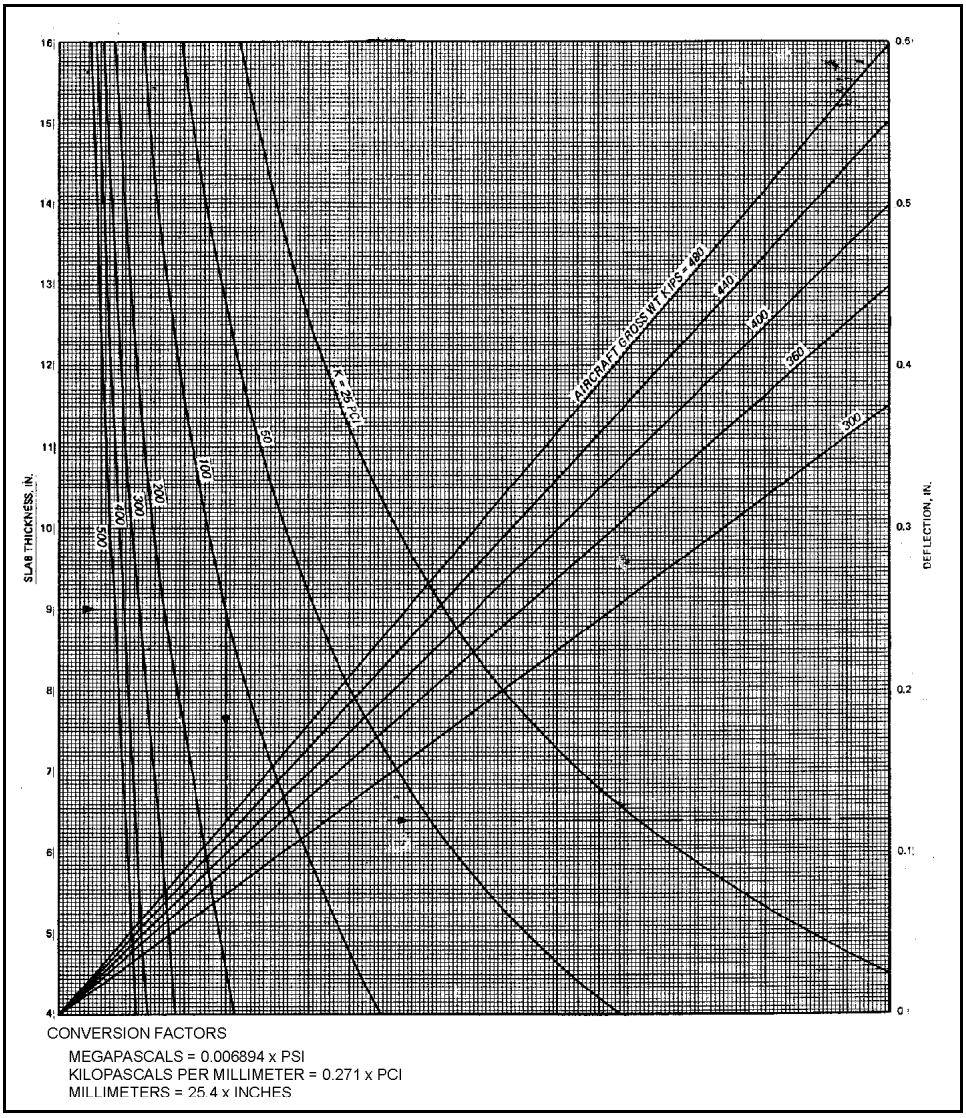


Figure 14-16. Deflection curves for Air Force modified heavy-load pavements

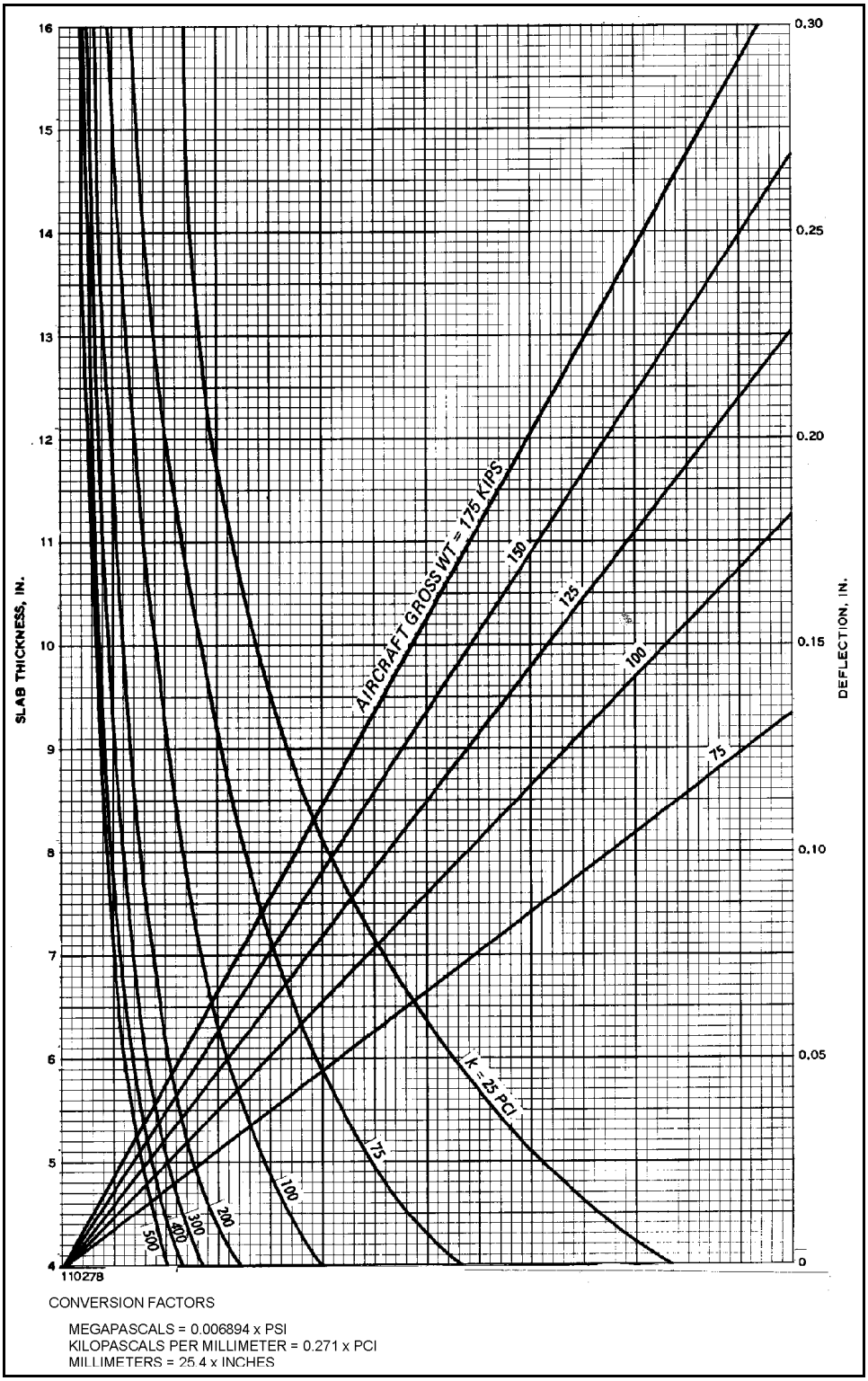


Figure 14-17. Deflection curves for Air Force shortfield pavements

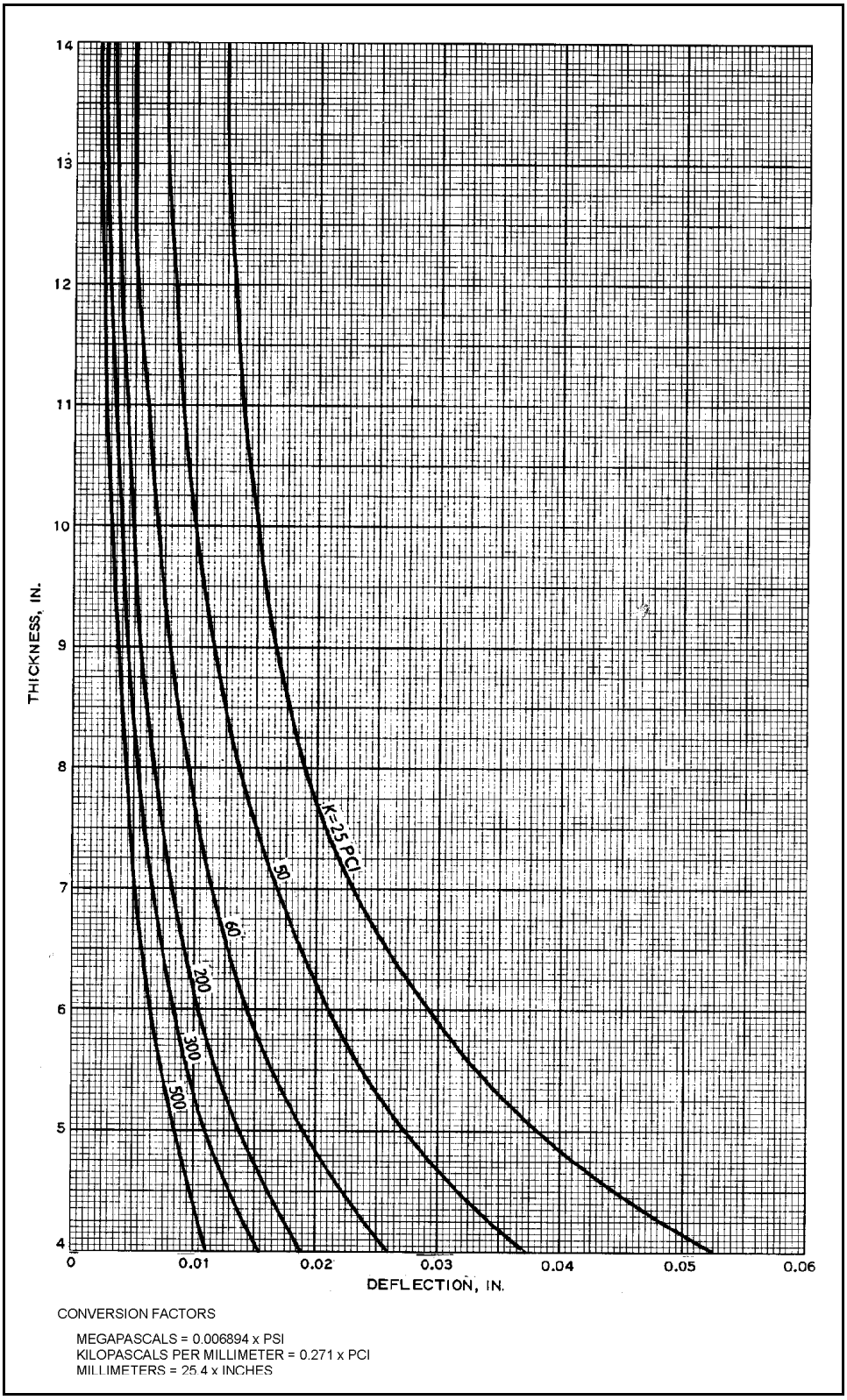


Figure 14-18. Deflection curves for shoulder pavements

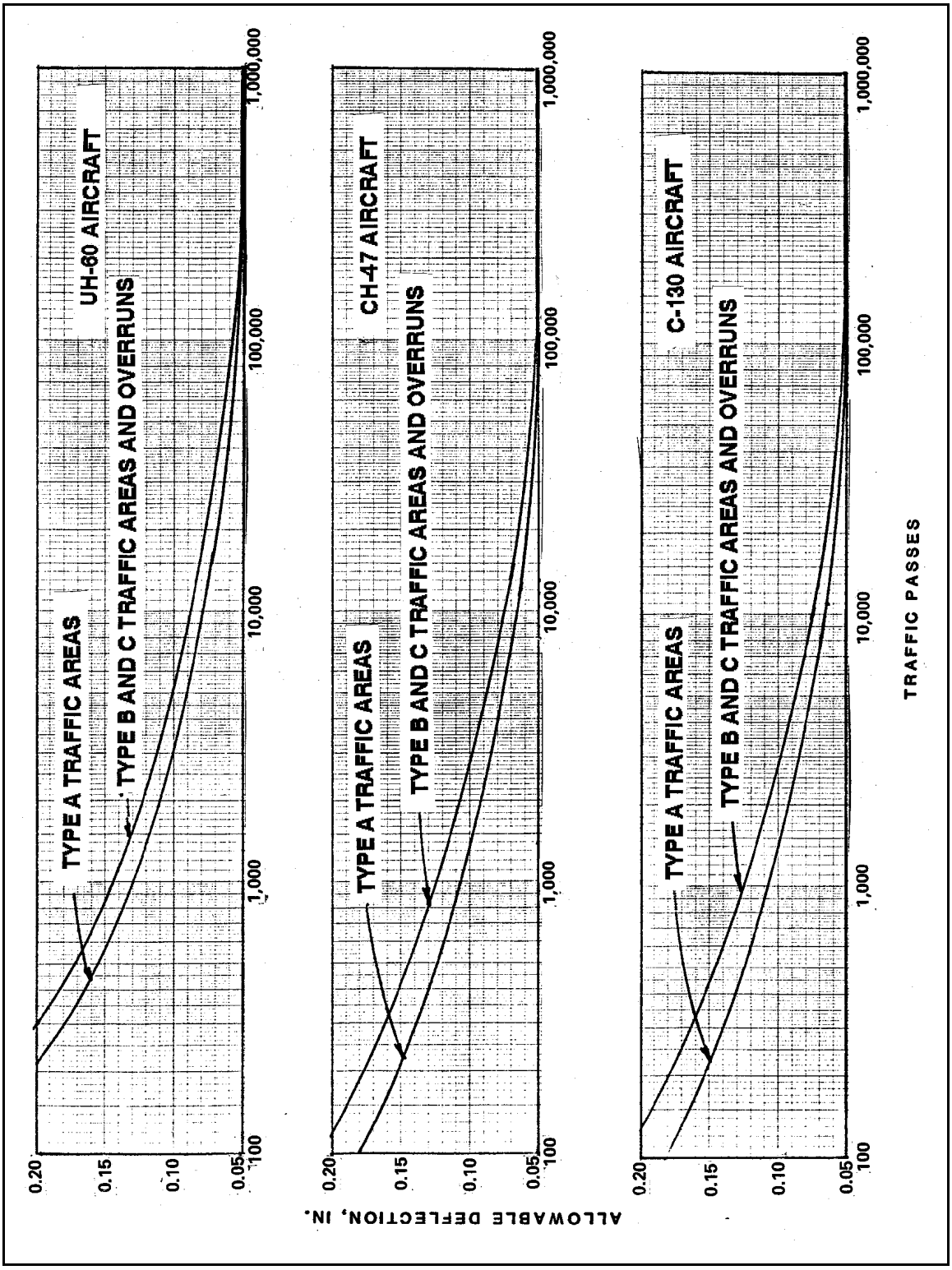


Figure 14-19. Allowable deflection curves for fibrous concrete pavements (Continued)

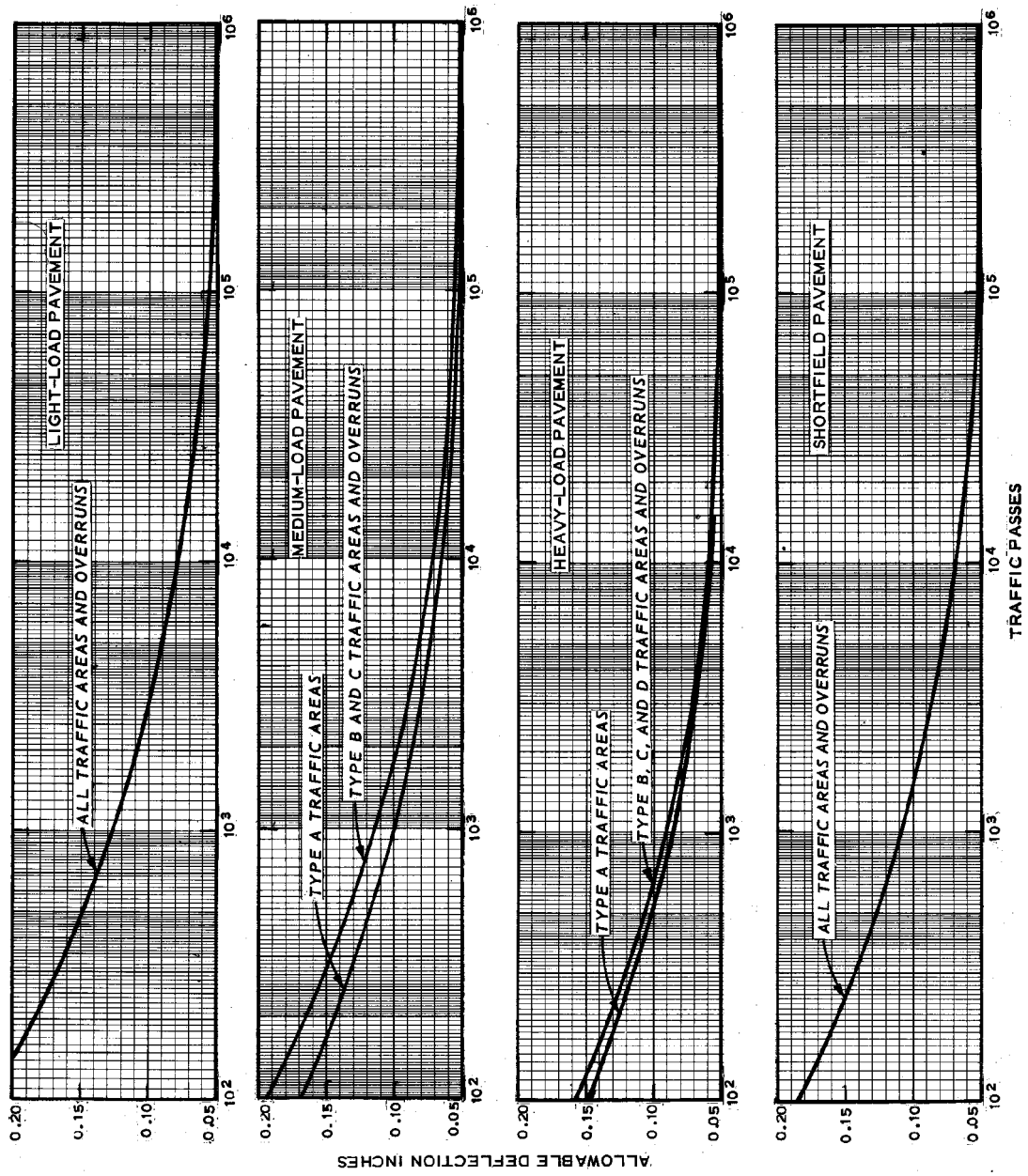


Figure 14-19. (Concluded)

CHAPTER 15

CONTINUOUSLY REINFORCED CONCRETE PAVEMENT DESIGN

1.BASI S OF DESIGN. A continuously reinforced concrete pavement is one in which the reinforcing steel is carried continuously, in both the longitudinal (direction of paving) and transverse (normal to direction of paving) directions, between terminal points. The terminal points may be either the longitudinal construction joints or ends of the pavement, junctures with other pavements or structures, etc. No joints are required between the terminal points; instead, the pavement is permitted to crack. The crack spacing will vary and be dependent upon the percent of reinforcing steel used, interface conditions between the pavement and foundation, and environmental conditions during the early life of the pavement. A transverse crack spacing ranging from 1.5 to 2.5 meters (5 to 8 feet) is desirable; however, experience has shown that even for the most carefully designed system, the crack spacing will vary from as little as 0.6 meters (2 feet) to as much as 3.5 meters (12 feet). The reinforcing steel provides continuity across the nonload-induced cracks, holding them tightly closed and providing good transfer of load. Considerable trouble has been encountered from underdesigned continuously reinforced concrete highway pavements. Consequently, the current trend and the approach adopted here is to make continuously reinforced concrete pavements the same thickness as plain concrete. The steel is assumed to only handle nonload-related stresses and any structural contribution to resisting loads is ignored. When properly designed and constructed, continuously reinforced concrete pavements provide very smooth, low-maintenance pavements. Experience has shown that continuously reinforced concrete pavements perform satisfactorily until the level of cracking reaches the point where punchout of the concrete between the reinforcing steel bars is imminent. The design procedure has been developed primarily from the results of continuously reinforced concrete pavement performance on highways since there has been only limited experience with airfield pavements.

2.USE FOR CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS. Continuously reinforced concrete pavements are applicable for any airfield pavement, but they have received very limited usage for airfield pavement construction. Therefore, long-time performance history is minimal. Because of this, its use will require approval of the Commander, U.S. Army Corps of Engineers (CEMP-ET), the appropriate Air Force Major Command, or the Naval Facilities Engineering Command. The use of continuously reinforced concrete pavement should be based upon the economics involved.

3.F OUNDATION REQUIREMENTS AND EVALUATION. Subgrade compaction and evaluation for a continuously reinforced concrete pavement shall be as described for plain concrete pavements. If economically feasible, the subgrade and/or base course may be modified or stabilized. Stabilized materials must achieve the strength and durability requirements specified in TM 5-822-14/AFJMAN 32-1019.

4.T HICKNESS DESIGN. The required thickness of a continuously reinforced concrete pavement is determined using the same procedures as for plain concrete pavement and will be the same thickness as plain concrete pavement. Although continuously reinforced concrete pavement contains steel in addition to being the same thickness as plain concrete pavement, the advantage of using it is that contraction joints are eliminated.

5.R EINFORCING STEEL DESIGN.

a. Longitudinal Direction. The percent of reinforcing steel required in the longitudinal direction for continuously reinforced concrete pavements will be the maximum calculated by the following three equations with the minimum percent steel being 0.43 percent.

$$P_s = (1.3 - 0.2F)\frac{f_t}{f_s} \times 100 \qquad (15\text{-}1)$$

$$P_s = \frac{100 f_t}{2(f_s - \Delta T \epsilon_c E_s)} \qquad (15\text{-}2)$$

$$P_s = \frac{f_t}{f_s} \times 100 \qquad (15\text{-}3)$$

where

P_s = percent of reinforcing steel required in the longitudinal direction

F = friction factor; suggested values are 1.0 for unbound fine-grained soils, 1.5 for unbound coarse-grained soils, and 1.8 for stabilized soils

f_t = 7-day tensile strength of the concrete in MPa (psi) determined using the splitting tensile test (Figure 5-1 may be used to convert 7-day flexural strength into tensile strength.)

f_s = working stress in the steel, MPa (psi) (75 percent of yield tensile strength of steel). This produces a safety factor of 1.33.

ΔT = seasonal temperature differential in degrees Celsius (Fahrenheit)

ϵ_c = thermal coefficient of expansion of concrete in millimeters per millimeter per degree Celsius (inches per inch per degree Fahrenheit)

E_s = modulus of elasticity of the reinforcing steel in tension, MPa (psi)

b. Transverse Direction. Transverse reinforcement is required for all continuously reinforced concrete airfield pavements to control any longitudinal cracking that may develop from load repetitions. The percent steel required in the transverse direction will be determined as follows:

$$P_s = \frac{W_s F}{2 f_s} \times 100 \qquad (15\text{-}4)$$

where

W_s = width of slab, m (ft)

c. Type of Reinforcing Steel. The reinforcing steel may be either deformed bars conforming to ASTM A 615 or welded deformed steel wire fabric conforming to ASTM A 497. Generally, longitudinal reinforcement is provided by deformed billet bars with 413-MPa (60,000-psi) minimum yield strength; however, other grades may be used. A grade 40 deformed bar should be used for the transverse reinforcement or for tie bars if bending is anticipated during construction.

d. Placement of Reinforcing Steel. When the slab thickness is 203 millimeters (8 inches) or less, the longitudinal reinforcement should be placed at the middepth of the slab. For thickness in excess of 200 millimeters (8 inches), the longitudinal steel should be placed slightly above the middepth, but a minimum cover of 75 millimeters (3 inches) of concrete shall be maintained in all cases. Transverse reinforcement is normally placed below and used to support the longitudinal steel; however, it may be placed on top of the longitudinal steel if the minimum of 75 millimeters (3 inches) of concrete cover is maintained. Proper lapping of the longitudinal reinforcement is important from the standpoint of load development and is essential for true continuity in the steel. The deformed bars or welded deformed wire fabric shall be lapped in accordance with Chapter 15. It is particularly important to stagger the laps in the reinforcing steel. Generally, not more than one-third to one-half of the longitudinal steel should be spliced in a single transverse plane across a paving lane. The width of this plane should be 610 millimeters (24 inches) if the one-third figure is used, and 1,220 millimeters (48 inches) if the one-half requirement is used. The latter case shall be interpreted to read that not more than one-half of the longitudinal reinforcing members may be spliced in any 1,220-millimeter (48-inch) length of pavement. The stagger of laps with deformed bars may be on a continuous basis rather than the one-third or one-half detail described above.

6.T ERMINAL DESIGN. When appreciable lengths of continuously reinforced concrete pavement are used, the ends experience large movements if unrestrained and will exert large forces if restrained. To protect abutting pavements or structures from damage, the ends of continuously reinforced concrete pavements must be either isolated or restrained. Experience has shown that it is practically impossible to completely restrain or completely isolate the pavement ends, and a combination of these schemes (that is, partial restrain and limited available expansion space) has proven practical. End anchorage and/or expansion joints must be provided when continuously reinforced concrete pavement is not continuous through intersections or when it abuts a structure. Although numerous terminal treatment systems have been attempted, especially on highway pavements, the most successful system appears to be the wide-flange beam joint. Typical drawings of this terminal system are shown in Figure 15-1. For runways, the continuously reinforced concrete pavement should extend to the runway end, where the wide-flange beam joint would be placed as a part of the overrun area.

7.JO INTING. Continuously reinforced concrete pavements will normally use the same type of joints as used for plain concrete pavements except that contraction joints are not normally required. Longitudinal construction joints will be required with the spacing dictated by the paving equipment. The longitudinal construction joints will be butt joints as shown in Figure 12-32. Transverse construction joints, which are required for construction expediency, will be designed to provide slab continuity by continuing the normal longitudinal steel through the joint. The normal reinforcement will be supplemented by additional steel bars, 1.5 meters (5 feet) long (0.75 meters (2.5 feet) on each side of the joint) and the same diameter as the longitudinal reinforcement. The additional steel will be placed between the normal reinforcement and at the same depth in the slab. Thickened-edge slip joints will be used at intersections of pavements where slippage will occur. Otherwise, doweled expansion joints will be used. Expansion joint design will be in accordance with Chapter 12. It will be necessary to provide for expansion at all barriers located in or adjacent to continuously reinforced concrete pavement.

8.JO INT SEALING. The only joints requiring sealing in continuously reinforced concrete pavements will be longitudinal construction joints and expansion joints. Transverse construction joints need not be sealed since they will behave as conventional volume-change cracks that are present elsewhere in the pavement. Joint sealing membranes will be as specified for plain concrete pavements.

9.EXAMPLE OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENT DESIGN. It is required that a pavement be designed as an Air Force medium-load airfield. Types A and B traffic areas are designed for the F-15 at 23,130 kilograms (81,000 pounds), the C-17 at 263,000 kilograms (580,000 pounds), and the B-52 at 181,440 kilograms (400,000 pounds). Types C and D traffic areas and overruns are designed for the F-15 at 27,555 kilograms (60,750 pounds), the C-17 at 197,280 kilograms (435,000 pounds), and the B-52 at 136,080 kilograms (300,000 pounds). Types A, B, and C traffic areas are designed for 100,000 passes of the F-15, 400,000 passes of the C-17, and 400 passes of the B-52. Type D traffic areas and overruns are designed for 1,000 passes of the F-15, 4,000 passes of the C-17, and 4 passes of the B-52. On-site and laboratory investigations have yielded the following data required for design:

- Subgrade = silty sand (SM)
- Modulus of subgrade reaction = 54 kPa/mm (200 lb/in.^{3})
- Flexural strength = 4.83 MPa (700 psi)

The thickness of the continuously reinforced concrete pavement will be the same as required for plain concrete according to the procedures set forth in Chapter 12. The required thicknesses are therefore as follows:

Traffic Area	Calculated Thickness, mm (in.)	Design Thickness, mm (in.)
A	396 (15.6)	405 (16.0)
B	388 (15.3)	394 (15.5)
C	297 (11.7)	305 (12.0)
D and Overruns	238 (9.4)	241 (9.5)

Additional data required for determining the percent longitudinal steel are as follows:

- Tensile strength of concrete (from Figure 15-2) = 3.45 MPa (500 psi)
- Yield strength of steel = 414 MPa (60,000 psi)
- Coefficient of thermal expansion of concrete = 7.2×10^{-6} millimeters per millimeter per degree Celsius (4×10^{-6}a inches per inch per degree Fahrenheit)
- Modulus of elasticity of steel = 206 × 820 MPa (30×10^6 psi)
- Seasonal temperature differential of pavement = 72 degrees Celsius (130 degrees Fahrenheit)

– Friction factor for fine-grained soils = 1.0

The required percentage of longitudinal reinforcement steel is the maximum from Equations 15-1, 15-2, or 15-3.

$$P_s = (1.3 - 0.2F)\frac{f_t}{f_s} \times 100$$

$$= (1.3 - 0.2F)\frac{3.45}{310} \times 100 = 1.22 \text{ in SI units} \qquad (15\text{-}5)$$

$$= [1.3 - 0.2(1.0)]\frac{500}{45{,}000} \times 100 = 1.22 \text{ in English units}$$

$$P_s = \frac{100f_t}{2(f_s - \Delta T \epsilon_c E_s)}$$

$$= \frac{100(3.45)}{2(310.2 - 72.2 \times .0000072 \times 206820)} = 0.85 \text{ in SI units} \qquad (15\text{-}6)$$

$$= \frac{100(500)}{2[45{,}000 - 130(4 \times 10^{-6})(30 \times 10^{6})]}$$

$$= 0.850 \text{ in English units}$$

$$P_s = \frac{f_t}{f_s} \times 100$$

$$= \frac{3.45}{310.2} \times 100 = 1.11 \text{ in SI units} \qquad (15\text{-}7)$$

$$= \frac{500}{45{,}000} \times 100 = 1.11 \text{ in English units}$$

The design percent of longitudinal steel is therefore 1.222. The cross-sectional area of steel A_s required for the Type A traffic area is:

$$A_s = \frac{P_s \times A_p}{100}$$

$$= \frac{1.22 \times 405 \times 1{,}000}{100} = \begin{array}{l} 4{,}941 \text{ mm}^2 \text{ per meter} \\ \text{of pavement (SI units)} \end{array} \qquad (15\text{-}8)$$

$$= \frac{1.22 \times 16.0 \times 12}{100}$$

$$= 2.342 \text{ square inches per foot of pavement (English units)}$$

where

A_p = the cross-sectional area of 1 meter (1 foot) of pavement, square millimeters (square inches)

In determining the percent of steel required in the transverse direction, it is assumed that 6-meter (20-foot) paving lanes will be used along with the following equation:

$$P_s = \frac{W_s F}{2 f_s} \times 100 = \frac{20 \times 1.0}{2(45{,}000)} \times 100 = 0.022 \qquad (15\text{-}9)$$

The design percent steel in the transverse direction is therefore 0.022. The cross-sectional area of steel required per 300 millimeters (12 inches) of pavement for the 405-millimeter (16.0-inch) pavement is therefore

$$A_s = \frac{P_s \times A_p}{100} = \frac{0.022 \times 405 \times 300}{100} = 26.7 \text{ mm}^2 \ (0.0414 \text{ in.}^2) \qquad (15\text{-}10)$$

The percent steel for other traffic areas would be computed in the same manner.

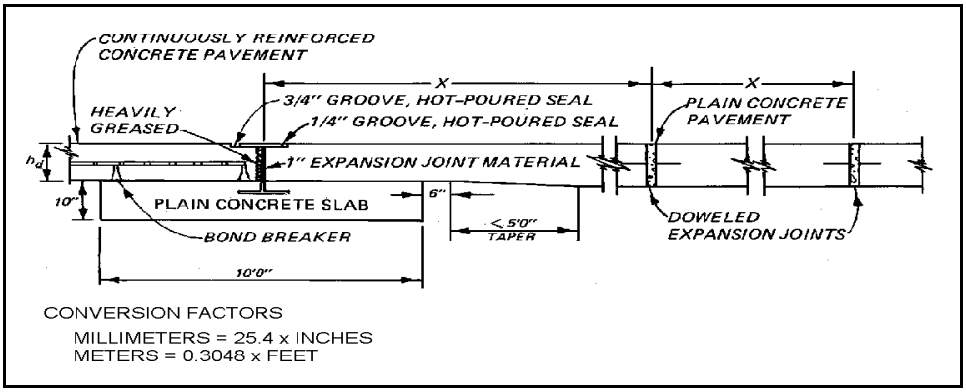


Figure 15-1. Details of a wide-flange beam joint

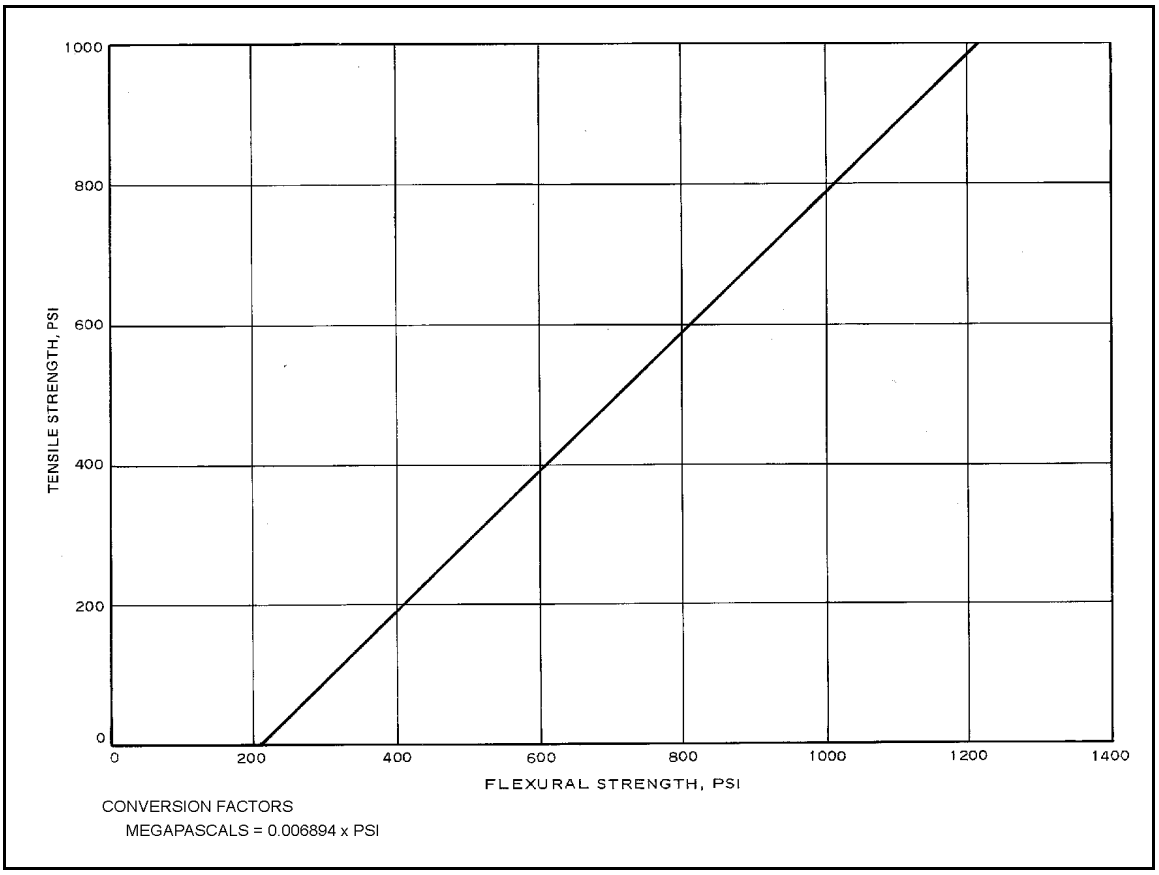


Figure 15-2. Relationship between flexural strength and tensile strength of concrete

CHAPTER 16

PRESTRESSED CONCRETE PAVEMENT DESIGN

1. BASIS OF DESIGN. A prestressed concrete pavement is one in which a significant compressive stress has been induced in both the longitudinal and transverse directions prior to the application of a live load. The induced compressive stress offsets the damaging effects of tensile stresses resulting from applied live loads and permits the formation of momentary, or partial, plastic hinges under passage of wheel loads that change the failure mode from tensile cracking at the bottom of the pavement to tensile cracking in the upper surface of the pavement due to negative moments. These two factors permit the prestressed concrete pavement to carry substantially greater loadings than equal thickness of plain concrete or reinforced concrete pavement and still provide a functionally adequate pavement.

2. UNITS. The design equations and criteria in this chapter are controlled by English units. Therefore, the equations have not been converted to SI units.

3. USES FOR PRESTRESSED CONCRETE PAVEMENT. Although prestressed concrete pavements have been used in Europe, a long-time performance history of prestressed concrete pavements in the United States is not extensive. Therefore, its use will require the approval of HQUSACE (CEMP), the approval Air Force Major Command, or Naval Facilities Engineering Command. Several test or demonstration sections in the United States have shown good performance, but problems have been experienced with joints between long prestressed sections where large movements are experienced. For this reason, complex joints and extreme care are required during construction. The selection of prestressed concrete pavements should be based upon the economics involved.

4. FOUNDATION REQUIREMENTS.

a. Subgrade and base. In general, the subgrade for a prestressed concrete pavement will be treated and evaluated in the same manner as for other types of rigid pavements. The reduced thickness of prestressed concrete pavement will result in a more flexible system and higher vertical stresses in the foundation than for plain concrete pavements. For this reason, the quality and strength of the foundation becomes more important. The foundation should be strengthened through the use of a high-quality (stabilized or nonstabilized) base course and/or stabilized or modified subgrade to provide a minimum modulus of soil reaction or composite modulus of soil reaction of 54 kPa/mm (200 pcl). In addition, because the amount of design prestress is a function of the foundation restraint, the surface of the foundation should be finished as smooth and as free of undulations, holes, etc., as possible.

b. Friction-Reduction Layer. A friction-reducing layer shall be used between the prestressed concrete pavement and the foundation. A satisfactory friction-reducing layer may consist of two polyethylene sheets over a thin 6- to 13-millimeter (1/4- to 1/2-inch) uniform size sand layer. The sand layer is used primarily to smooth out the surface irregularities of the foundation. Other types of friction-reducing material may be considered.

5. METHOD OF PRESTRESSING. Pavements may be prestressed using pretensioning or posttensioning. The method most commonly used for pavements is posttensioning, in which tendons are installed before concrete placement and stressed after concrete placement. The tendons either are placed in conduits or are plastic-encased to prevent bonding with the concrete. The tendons are threaded through bearing plates cast into the face of the concrete at the ends or sides of the concrete slabs. After the concrete has gained sufficient strength, the tendons are stressed, using the bearing

plates and concrete slab as a reaction, to the required total stress level and locked. The total stress level in the tendons is the sum of the stress needed to provide the design prestress level in the concrete plus the stress necessary to offset the various losses that will occur. To help reduce cracking in the concrete during the cure period, a preliminary level of prestress is normally applied at a very early age, and the final level of prestress applied after several days of curing. Both longitudinal and lateral prestressing is needed to obtain the desired structural capacity in the pavement.

6. DESIGN PROCEDURE.

a. General. In the design of prestressed pavements, both thickness and level of prestress will be unknowns; therefore, their determination, in both the longitudinal and transverse directions, becomes an iterative process (that is, one is selected and other computed). A normal practice is to compute the thickness requirements for a range of prestress levels, after which the final selection is made based upon an economic analysis. A maximum value of design prestress of 400 psi is recommended; and based upon experience, a design prestress level falling between 100 and 400 psi has been most economical. The minimum thickness of prestress concrete pavement will be 150 millimeters (6 in.).

b. Design Equation. The design prestress for a given thickness of pavement will be determined as follows:

$$d_s = \frac{6PNB}{wh_p^2} - R + r_s + t_s \qquad (16\text{-}1)$$

where

d_s = design prestress required in concrete, psi

P = aircraft gear load, pounds

N = load-repetition factor

B = load-moment factor

w = ratio of multiple-wheel gear load to single-wheel gear load

h_p = design thickness of prestressed concrete pavement, inches

R = design flexural strength of concrete, psi

r_s = foundation restraint stress, psi

t_s = temperature warping stress, psi

Since both d_s and h_p will be unknown, it is necessary to select values of h_p and compute d_s. For guidance, experience has shown that d_s levels between 100 and 400 psi are generally economical, and at these levels h_p will be about one-third of the required thickness of plain concrete pavement. The design gear load P will depend upon the aircraft for which the pavement is being designed. The load-repetition factor N is a function of the type of design aircraft and the traffic area type. The design aircraft

pass level is divided by the aircraft pass per coverage factor to determine the design number of stress repetitions, which are in turn used in Figure 16-1 to obtain N. The load-moment factor B and ratio of multiple-wheel load to single-wheel load w are determined from Figures 16-2 and 16-3, respectively, by entering with a value of A/ℓ^2 (note that for the light-load and Class I airfields, w is 1.0 for all values of A/ℓ^2). A is the contact area in square inches of a tire in the main gear of the design aircraft, and ℓ is computed by

$$\ell = \left[\frac{Eh_p^3}{12(1 - \mu^2)k}\right]^{1/4} \qquad (16\text{-}2)$$

where

ℓ = radius of relative stiffness, inches

E = the modulus of elasticity of concrete (a value of 4,000,000 psi is normally used)

h_p = design thickness of prestressed concrete pavement, inches

μ = Poisson's ratio

k = modulus of subgrade reaction, pci

c. Foundation Restraint Stress. The subgrade restraint stress r_s is a function of the coefficient of sliding friction between the pavement and underlying foundation and the length or width of the prestressed concrete slab and is determined by

$$r_s = \frac{C_f L \rho}{2(144)} \quad or \quad r_s = \frac{C_f W \rho}{2(144)} \qquad (16\text{-}3)$$

where

r_s = foundation restraint stress, psi

C_f = coefficient of sliding friction

L = length of prestressed concrete slab, feet

W = width of prestressed concrete slab, feet

ρ = density of concrete, lb/ft^3

Experience has shown that for a prestressed concrete pavement constructed with sand and polyethylene sheet bond-breaking medium on the surface of the prepared foundation, a value of C_f of 0.60 is representative. This value can be reduced, with a subsequent reduction in the design prestress level, through the selection of materials with lower coefficients of friction and through careful preparation of the foundation layer.

d. Temperature Warping Stress. The temperature warping stress results from the development of a temperature gradient through the prestressed concrete pavement thickness and can be determined by:

$$t_s = \frac{ET\epsilon_c}{2(1 - v)} \tag{16-4}$$

where

t_s = temperature warping stress, psi

T = difference in temperature in degrees Fahrenheit between the top and bottom of the prestressed concrete pavement

ϵ_c = coefficient of thermal expansion, inches/inch

Values of T should be determined by a test on a pavement in the vicinity of the proposed prestressed concrete pavement; however, without other data, a value of 1 to 3 degrees per inch of pavement has been found to be fairly representative of the maximum temperature gradient.

7. PRESTRESSING TENDON DESIGN.

a. General. The size and spacing of prestressing tendons required will be a function of the required prestress level and the various losses that will occur in the steel tendons during and following construction.

b. Size and Spacing on Tendons. The tendon stress losses occur as a result of elastic shortening and creep of the concrete, concrete shrinkage, tendon relaxation, and slippage in the anchorage system. The determination of these tendon losses is complex because of the many variables, some of which are unknown without extensive field testing. From the experience gained in the few test and demonstration sections and actual pavement sections, the tendon losses can be approximated as 20 percent of the tendon stress needed to achieve the design prestress level in the concrete. With this approximation, the total area of tendon steel required to accomplish the prestress level in the concrete after allowance for tendon losses can be determined by

$$A_s = \frac{1.2 d_s A_c}{0.7 f_\mu} \tag{16-5}$$

where

A_c = cross-sectional area of concrete being prestressed, square inches

f_μ = ultimate strength of the tendon steel, psi

The equation above is applicable to the determination of A_s based upon a recommended maximum anchorage stress equal to seven-tenths of the ultimate strength of the tendon steel. If the steel is anchored at a stress other than seven-tenths of the ultimate strength, the equation above must be modified accordingly. With the total required A_s determined, the number and size of prestressing

tendons can be selected. Spacings of two to four times the prestressed concrete pavement thickness are recommended for the longitudinal tendons, and spacings of three to six times the prestressed concrete pavement thickness are recommended for the transverse tendons.

c. Prestressing Steel Tendons. The tendons used for prestressed concrete pavement will consist of either high-strength wires, strands, or bars.

(1) Wires will conform to the requirements of ASTM A 421.

(2) Seven-wire strands will conform to the requirements of ASTM A 416.

(3) High-strength bars will conform to the requirements of section 405(f) of ACI 318.

d. Prestressing Conduits. Conduits used for enclosing the steel tendons should be either rigid or flexible metal tubing. However, the tendons may be plastic-encased.

(1) Metal conduits must be strong enough to resist damage in transit or during handling. The metal may be bright or galvanized.

(2) When tendons are plastic-encased, the tendons should be permanently protected from rust or corrosion.

e. Placement of Tendons and Conduits. The transverse conduits will be placed on metal chairs at the desired depth and used to support the longitudinal conduits or tendons. Conduits and tendons will be tied firmly in place to maintain proper alignment during placement of the concrete. A preliminary stress applied to the tendons may help maintain the alignment. The inside diameter of metal conduits will be at least 6 millimeters (0.25 inch) larger than the diameter of the stressing tendons. The minimum cover of the conduits will be 75 millimeters (3 inches) at the pavement surface and 50 millimeters (2 inches) at the bottom of the pavement.

f. Tendon Stressing. The prestressed tendons must be stressed to provide a stress in the concrete equal to 1.2 times the design prestress d_s plus sufficient stress to overcome the frictional resistance between the tendon and conduit. After concrete placement and prior to beginning the prestressing operation, any preliminary tension in the tendons must be released. If the tendons are conduit-encased, they should be pulled back and forth several times to reduce and to measure the tendon stress due to friction. This need not be done for plastic-encased tendons. The measured tendon-friction stress must be added to the tendon stress required to produce $1.2d_s$ in the concrete. If the tendons were sized as described in b above, the required tendon stress will be the selected anchorage stress ($0.7f_{\mu}$ or other value if used to size the tendon), plus the stress required to overcome friction. After the maximum tendon stress is reached, it will be held for several minutes and then released to the selected anchorage stress. The longitudinal tendon stressing will be applied in three stages with the amount of prestress at each successive stage being 25, 50, and 100 percent of the anchorage stress. The prestressing will be applied as soon as possible to prevent or minimize the occurrence of contraction cracking in the concrete.

g. Grouting. When the stressing tendons are placed in conduits, the space between the tendons and conduits will be grouted after the final prestressing load is reached. The grout will be made from either cement and water or cement, fine sand, and water. Admixtures to obtain high early strength or to increase workability may be used if they will have no injurious effects on the stressing tendons or conduits. Grouting vents will be provided at each end of the conduits and along the conduits at intervals

not to exceed 45 meters (150 feet). A grouting pumping will be used to inject the grout. The grouting will commence at an end vent and continue until grout is forced out of the first interior vent along the conduit. The end vent will then be sealed, and grout will be injected through the first interior vent until it is extruded from the second interior vent. This procedure will be continued until the entire length of conduit has been grouted.

8. JOINTING.

a. Joint Spacing. Experience has shown that from a practical standpoint, the maximum length of prestressed concrete slabs should be 150 meters (500 feet), although lengths of 180 and 215 meters (600 and 700 feet) have been constructed. The width of the slab will vary depending upon the capability of the construction equipment but will generally be a minimum of 7.6 meters (25 feet).

b. Joint Types.

(1) Longitudinal joint. Runway and taxiway pavements will be prestressed for their full width, and the longitudinal joints will be the butt type with the prestressed tendons carried through the joint. The transverse prestressing operation will be carried out after all paving lanes have been completed. For areas wider than 150 meters (500 feet) (such as aprons), the pavement must be constructed in widths not to exceed 150 meters (500 feet); therefore, longitudinal fill-in lanes will be required to permit access for applying the transverse prestressing.

(2) Transverse joint. Because of the length of prestressed slabs and the low subgrade restraint, large movements will occur at the transverse joints. The transverse joint must be designed to accommodate these movements that are a function of the temperature change, slab length, and moisture conditions. The anticipated movements can be determined by

$$\Delta_{LT} = 12L\epsilon_c\Delta T \tag{16-6}$$

and

$$\Delta_{LM} = 12L\epsilon_M \tag{16-7}$$

where

Δ_{LT} = change in length of slab due to temperature change ΔT, inches

L = slab length, feet

ΔT = change in temperature in degrees (either daily or seasonally)

Δ_{LM} = maximum change in length of slab due to seasonable moisture change

ϵ_M = coefficient of moisture expansion of concrete (assumed to be 1×10^{-4} inch per inch seasonally)

The transverse joint must be capable of withstanding the sum of the temperature and moisture change in length. Figure 16-4 shows typical sections of two general methods of construction of the transverse joints. Type A consists of having the transition slab rest directly on the subbase. The transition slab will be constructed to the thickness requirements of either plain or reinforced concrete pavements and connected to the prestressed slabs with dowel bars to provide load transfer through the joint. The size and spacing of the dowel bars will be determined from Chapter 12 based upon the plain or reinforced concrete thickness requirements. Type B consists of a grade slab underlying the ends of the prestressed concrete pavement and transition slab. The transition slab will be reinforced concrete of the same thickness as the prestressed concrete pavement. The grade slab will also be reinforced concrete. The thickness of the grade slab and the percent of reinforcing steel in both the transition slab and grade slab will be determined in accordance with overlay design procedures if the transition slab is a reinforced concrete overlay of the reinforced grade slab.

c. Joint Seals. Longitudinal joints in prestressed concrete pavements, except where longitudinal transition lanes will be required to permit prestressing operations of wide paved areas, need not be sealed since they will be held tightly closed by the prestressing. However, if these joints are sealed, materials meeting the requirements for plain concrete pavements should be used. When longitudinal transition lanes are required, the longitudinal joint should be treated in the same manner as a transverse joint. Several types of sealants have been used for the transverse joints, but no standardized seals have been established. Poured-in-place materials have not been satisfactory to accommodate the large movements that occur. Preformed and mechanical seals, such as shown in Figure 16-5 are recommended. The final selection of a sealant will be a matter of engineering judgment that must be approved by HQUSACE (CEMP-ET), the appropriate Air Force Major Command, or the Naval Facilities Engineering Command.

9. EXAMPLES OF PRESTRESSED CONCRETE PAVEMENT DESIGN.

a. General. A 75-foot-wide by 10,000-foot-long taxiway pavement is to be designed for 100,000 passes of the C-141 aircraft at 320,000 pounds gross weight using prestressed concrete. Laboratory and field test programs have yielded the following pertinent physical property data for the foundation and concrete: modulus of soil reaction, k = 200 pci; 90-day flexural strength of concrete, R = 700 psi; density of concrete = 150 lb/ft^3; modulus of elasticity in flexure of concrete, E = 4 × 10^6 psi; Poisson's ratio of concrete, v = 0.15; and coefficient of thermal expansion of concrete, ϵ_c = 4 × 10^{-6} inch per inch per degree Fahrenheit.

b. Determination of Design Prestress Level. Prestress loads will be determined for preselected thicknesses h_p of 6, 7, and 8 inches. Following the procedures described in paragraph 5, the load-repetition factor N is 2.46 (Figure 16-1) and the load-moment factor B is 0.0523, 0.0544, and 0.0565 for thicknesses of 6, 7, and 8 inches, respectively. The ratio of multiple-wheel gear load to single-wheel gear load, w is 2.22, 2.23, and 2.335 for thickness of 6, 7, and 8 inches, respectively. A polyethylene sheet bond-breaking medium will be used between the foundation and prestressed slab, and the coefficient of sliding friction C_f will be 0.60. A slab length L of 400 feet will be used; therefore, the subgrade restraint stress in the longitudinal direction will be

$$r_s = \frac{C_f L \gamma}{2(144)} = \frac{0.60 \times 400 \times 150}{2(144)} = 125 \text{ psi} \qquad (16\text{-}8)$$

In the transverse direction, the subgrade restraint stress will be

$$r_s = \frac{C_f W \gamma}{2(144)} = \frac{0.60 \times 75 \times 150}{2(144)} = 23.4 \text{ psi} \tag{16-9}$$

The maximum difference in temperature between the top and bottom of the prestressed concrete pavement is estimated to be about 6, 7, and 8 degrees for the 6-, 7-, and 8-inch pavements, respectively, with resulting temperature warping stresses of 46, 65, and 75 psi, respectively. The design prestressing required in the concrete is then determined by the following equation:

$$d_s = \frac{6PNB}{wh_p^2} - R + r_s + t_s \tag{16-10}$$

For h_p values of 6, 7, and 8 inches, the design prestress d_s in the longitudinal direction will be 853, 492, and 253 psi, respectively, and in the transverse direction the values of d_s will be 761, 391, and 151 psi, respectively. Plotting these values, as shown in Figure 16-6, permits the selection of various thicknesses and prestressing levels that will support the design loading condition. Experience has shown that d_s levels between 100 and 400 psi are most practicable; therefore, from Figure 16-6, a 7.5-inch pavement with longitudinal prestress of 360 psi and transverse prestress of 250 psi would provide a satisfactory pavement. With a slab length of 400 feet, 25 slabs and thus 24 joints will be required for the 10,000-foot-long taxiway. In actual design, several combinations of k, h_p, slab length, etc., should be considered, and the final selection should be based on an economic study considering all aspects of material and construction costs.

c. Prestressed Tendon Design. Plastic-encased stranded wire having an ultimate strength f_μ of 240,000 psi is selected for the prestressed tendons. The stranded wire tendon will be finally anchored at a stress not to exceed $0.7f_\mu$ or 168,000 psi. The required area of steel in the longitudinal and transverse directions to achieve the design prestressing level in the concrete and allowing for the various tendon stress losses will be

Longitudinal Direction

$$A_s = \frac{1.2 \times 360 \times 7.5 \times 75 \times 12}{0.7 \times 240{,}000} = 17.4 \text{ square inches} \tag{16-11}$$

Transverse Direction

$$A_s = \frac{1.2 \times 250 \times 7.5 \times 400 \times 12}{0.7 \times 240{,}000} = 64.3 \text{ square inches} \tag{16-12}$$

Several combinations of wire diameter and spacing will yield the required cross-sectional area of steel for the stressing tendons. For example, if in the longitudinal direction, a spacing of four times the prestressed concrete pavement thickness (30 inches) is selected, then 30 tendons will be required, each having a cross-sectional area of 0.58 square inch and diameter of 0.86 inch. Therefore, a 7/8-inch-diameter tendon could be selected. Selection of a tendon that is greater or less than that required may require the final anchor stress to be revised. If, in the transverse direction, a spacing of five times the prestressed concrete pavement thickness (37.5 inches) is selected, then 128 tendons

would be needed and the required cross-sectional area of the tendons would be 0.50 square inch. Therefore, a 13/16-inch-diameter tendon would provide the required prestressing.

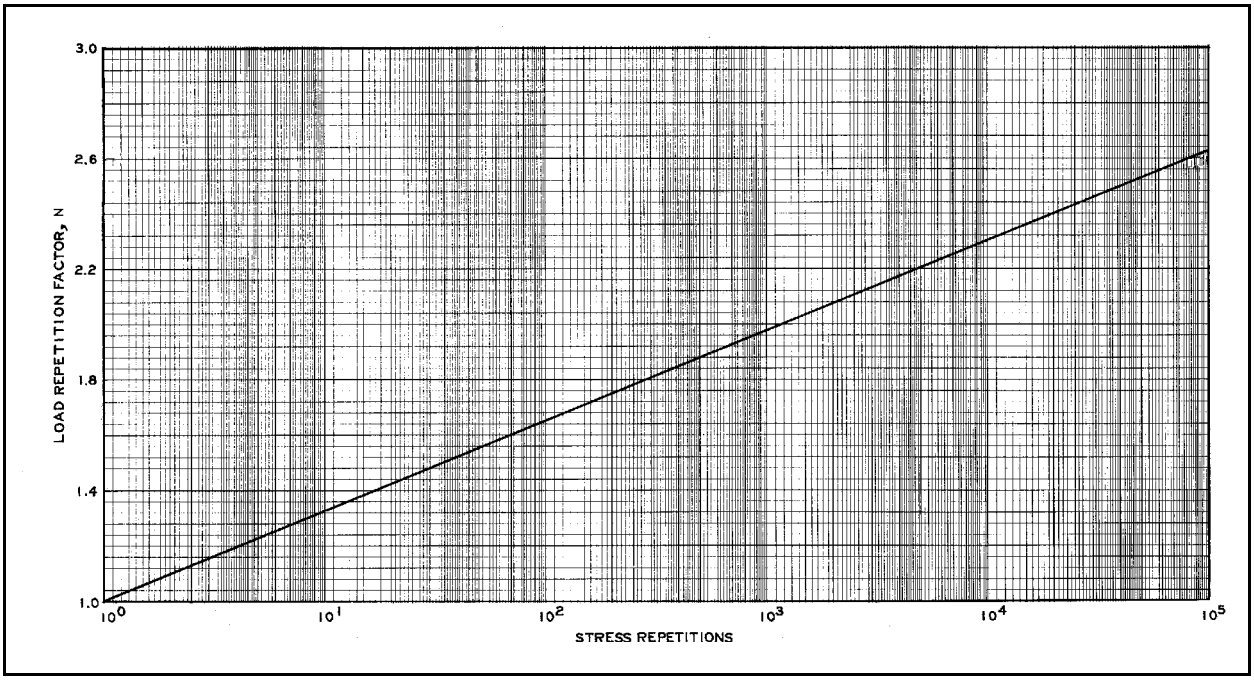


Figure 16-1. Stress repetitions versus load repetition factor

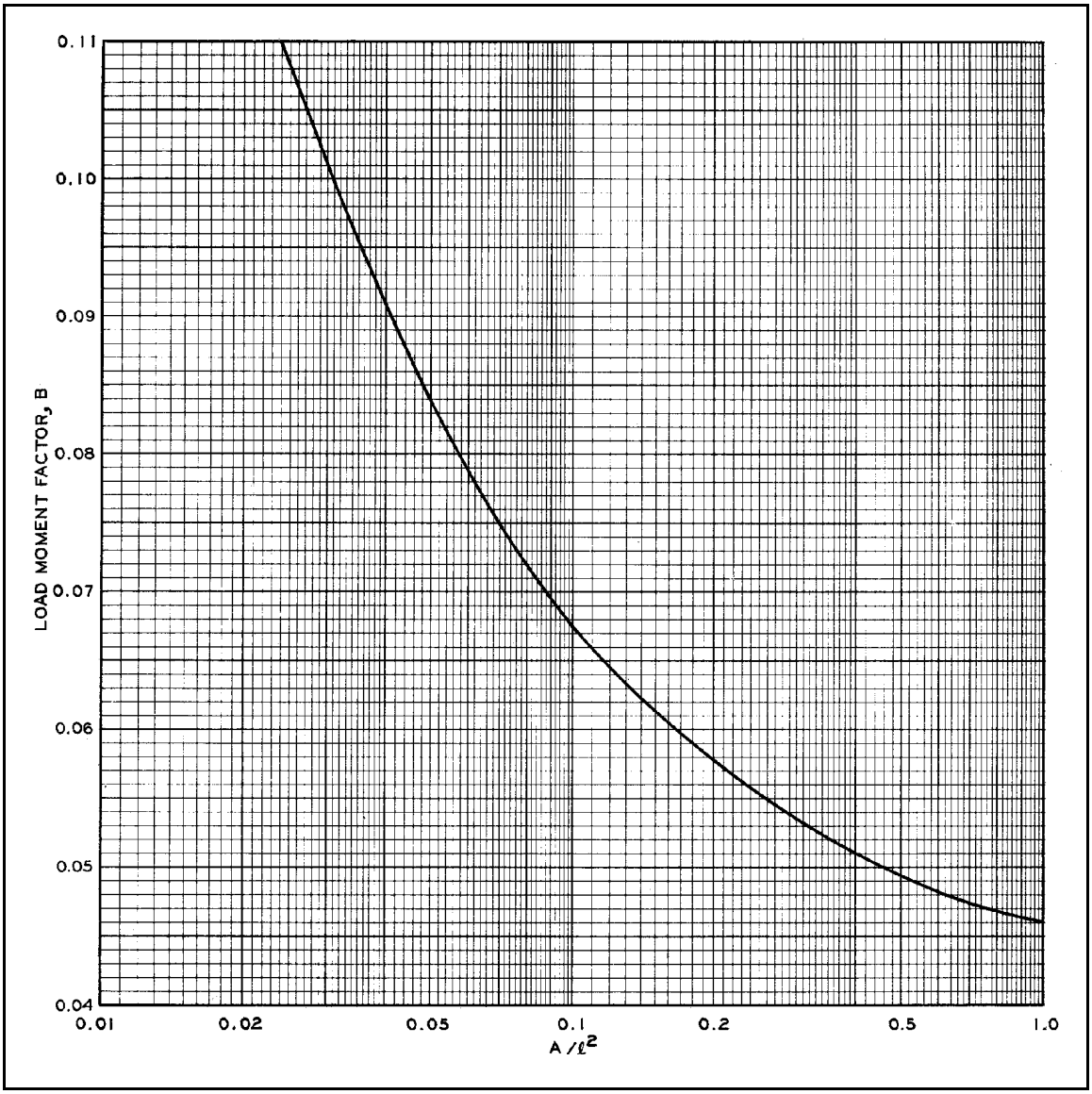


Figure 16-2. A/ℓ^2 versus load-moment factor

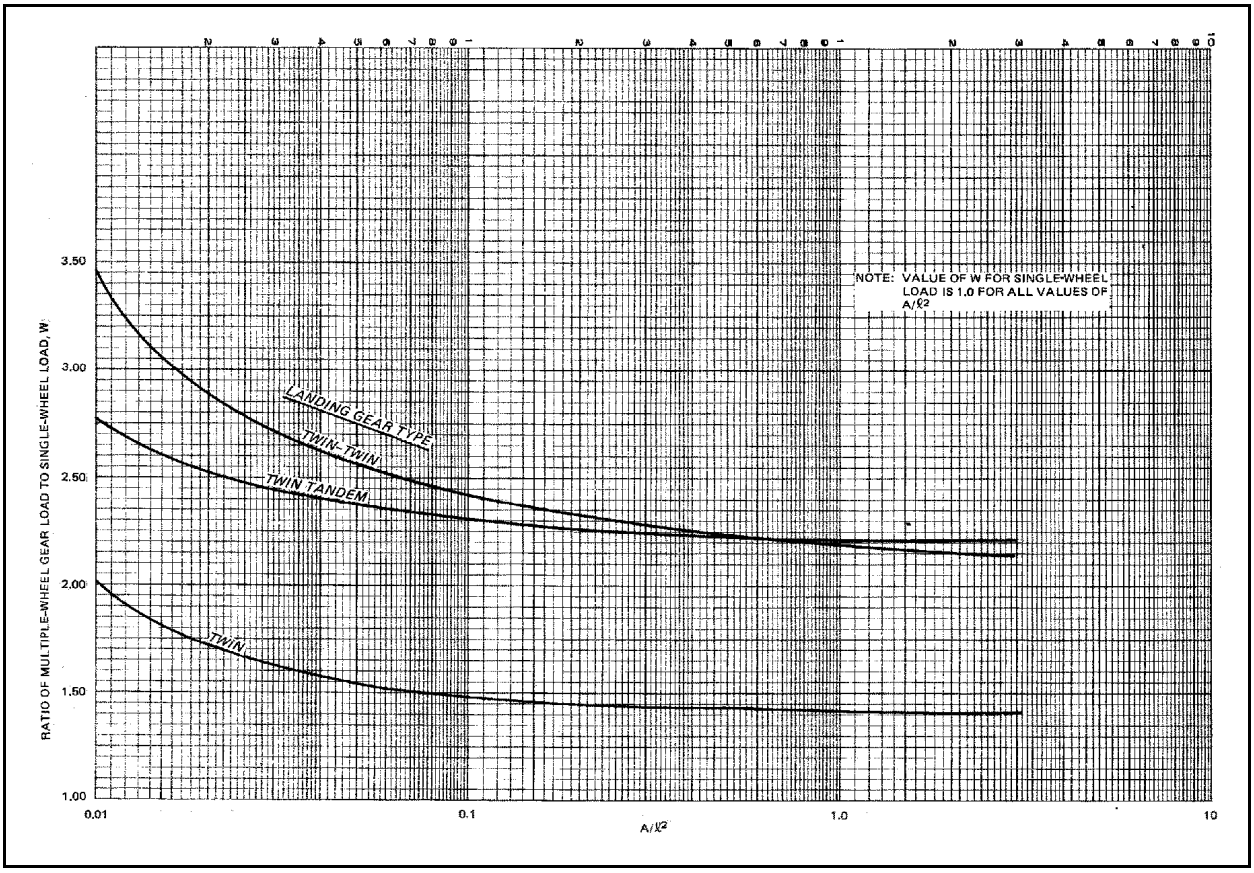


Figure 16-3. Ratio of multiple wheel gear to single-wheel gear load versus A/ℓ^2

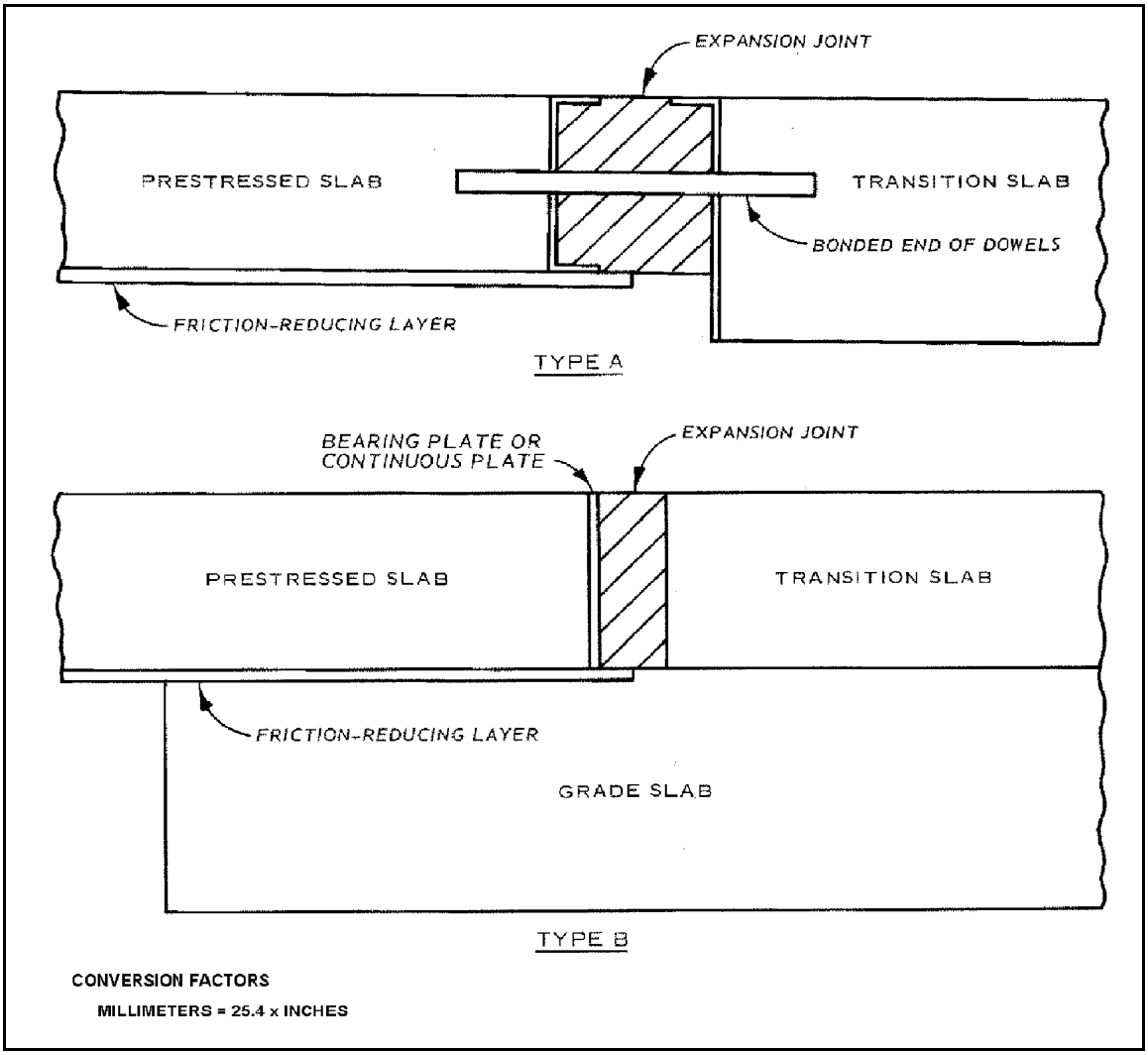


Figure 16-4. Typical section of transverse joints

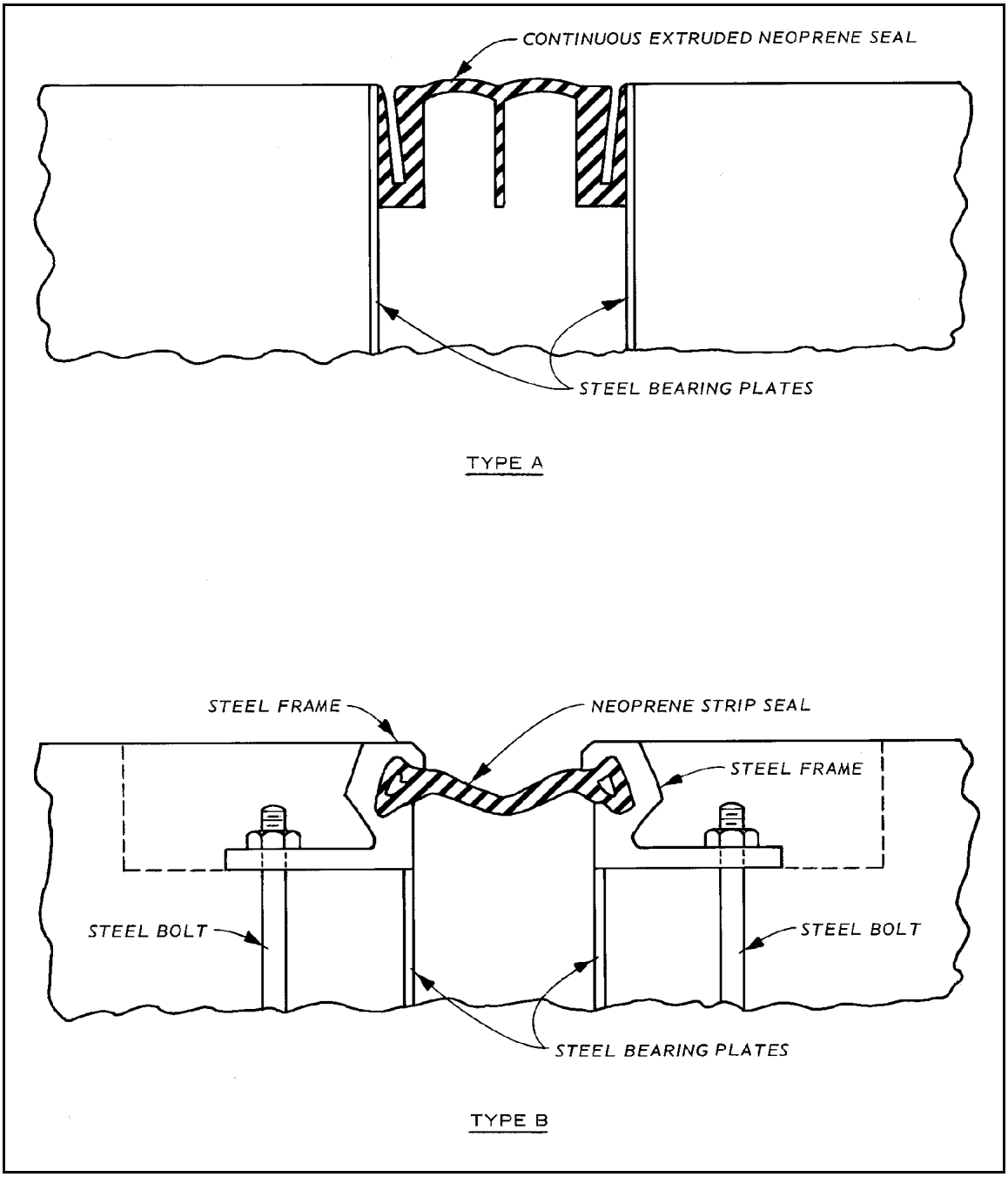


Figure 16-5. Typical transverse joint seals (Sheet 1 of 3)

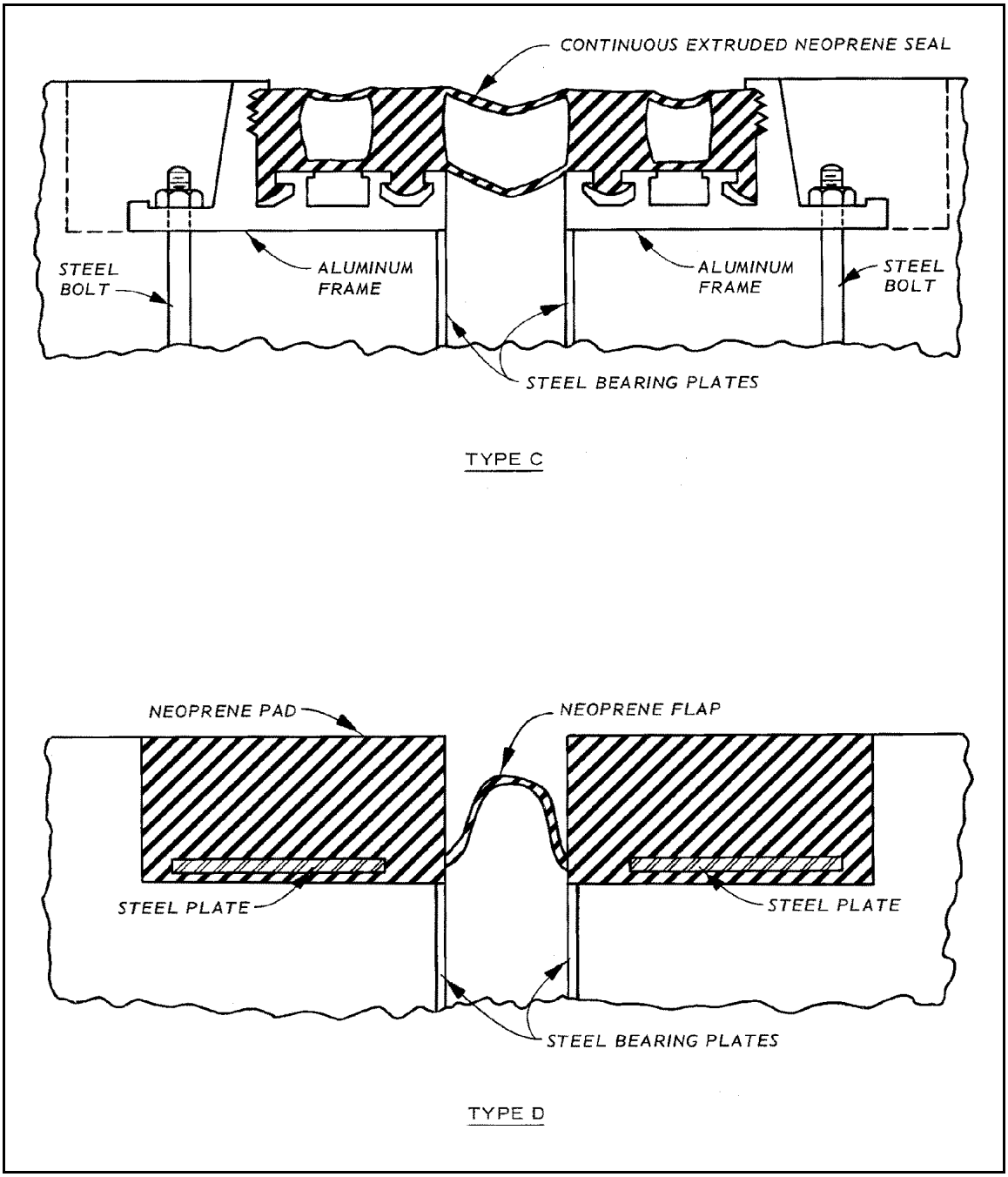
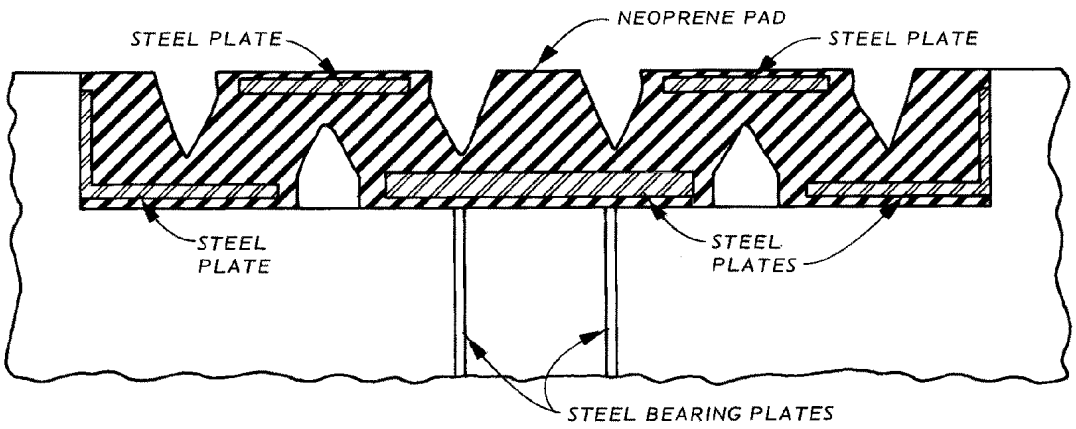
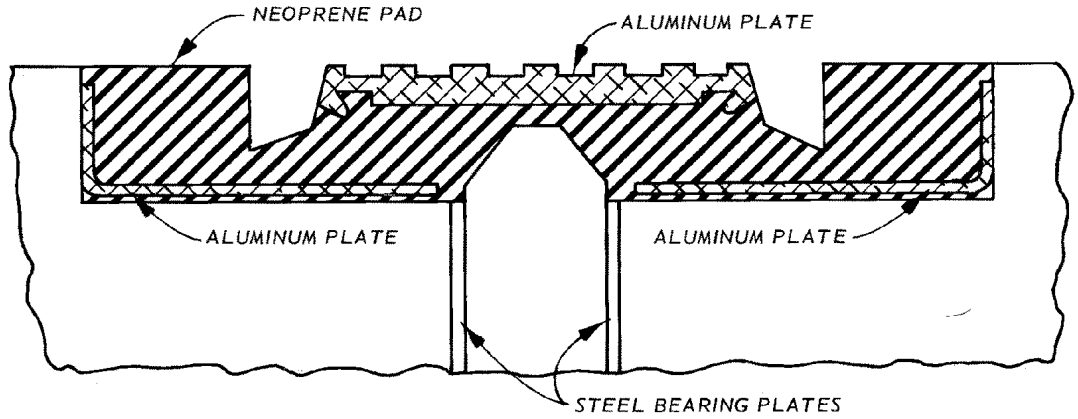


Figure 16-5. (Sheet 2 of 3)



TYPE E



TYPE F

Figure 16-5. (Sheet 3 of 3)

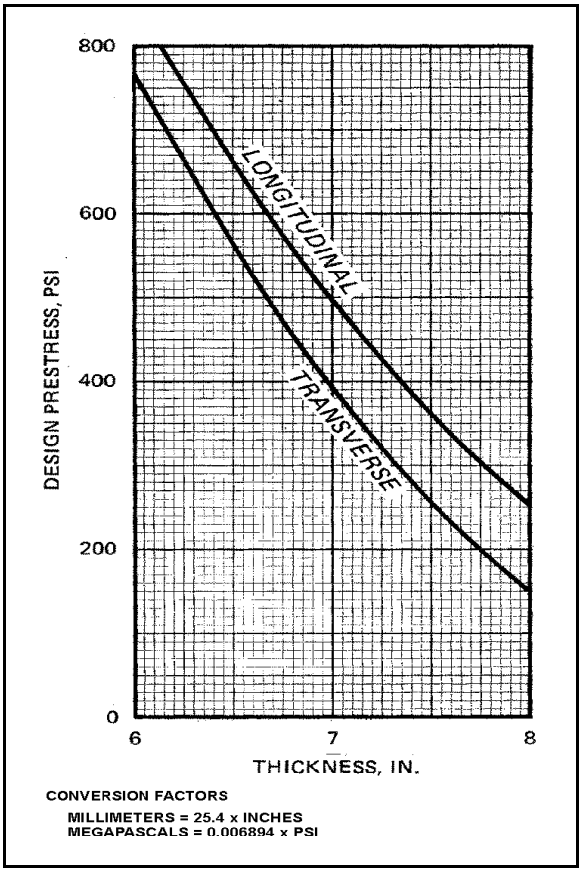


Figure 16-6. Thickness versus design prestress

CHAPTER 17

OVERLAY PAVEMENT DESIGN

1. GENERAL. Overlay pavements are designed to increase the load-carrying capacity (strength) of the existing pavement. The basis for design is to provide a layer or layers of material on the existing pavement that will result in a layered system which will yield the predicted performance of a new rigid pavement if constructed on the same foundation as the existing pavement. Two general types of overlay pavement are considered: rigid and nonrigid. Procedures are presented for the design of plain concrete, reinforced concrete, continuously reinforced concrete, fibrous concrete, prestressed concrete, and nonrigid overlays. Nonrigid overlays include both flexible (nonstabilized base and bituminous concrete wearing course) and all-bituminous concrete for strengthening existing plain concrete, reinforced concrete, and flexible pavements. Continuously reinforced, fibrous, and prestressed concrete overlays will not be permitted unless it is technically and economically justified and approved by HQUSACE (CEMP), Air Force Major Command, or Naval Facilities Engineering Command. Overlays will be used when the nonstabilized aggregate base course can be positively drained. When the overlay includes a nonstabilized aggregate base course layer, the unbound base course must be positively drained.

2. CONVENTIONAL OVERLAY DESIGN EQUATION BACKGROUND AND LIMITATIONS. The overlay design equations for rigid and flexible overlays of rigid pavements presented in this chapter are based on full-scale accelerated traffic tests conducted in the 1950's modified with experience and performance observations in succeeding years. The equations were developed to support a program of strengthening Air Force airfield pavements to accommodate the introduction of the large B-47 and B-52 aircraft into the inventory. Because of theoretical limitations of the time, the overlay equations are empirical. They have the advantage of simplicity for use, but their empirical basis means that they are valid only for conditions consistent with their original development. To use these equations effectively, one must be aware of their limitations and their proper application as discussed in this chapter. For more complex situations, a more comprehensive overlay analysis as presented in the layered elastic design chapter may be necessary.

a. The overlay equations for rigid and flexible overlays of rigid pavements recognize four basic conditions:

(1) Fully bonded overlay where the rigid overlay and rigid base pavement are fully bonded and behave monolithically. Because of problems with providing load transfer, these overlays are generally limited to correcting surface deficiencies of a structurally adequate pavement in good condition other than the surface problems.

(2) Partially bonded overlay where no particular attempt is made to achieve or prevent bond between the rigid overlay and the base pavement. This equation is a best fit to empirical data and therefore can give either conservative or nonconservative thicknesses. Partially bonded overlays are particularly well suited for structurally upgrading an essentially sound pavement to accommodate larger loads as might happen when a mission change brings new heavier aircraft to a base.

(3) Unbonded overlay where a thin separation layer asphalt concrete or other material is interposed between the rigid overlay and the base pavement to avoid direct bonding between the

two. This equation gives generally conservative results. Unbonded overlays are best suited for restoring a deteriorated pavement to structural and functional capacity.

(4) Flexible overlay where an asphaltic concrete is placed directly on a rigid base pavement to restore surface and structural quality. For very thick overlays, a combination of granular base and asphalt concrete surface can be used provided the granular base is positively drained so that no water can be trapped in the overlay. When compared to more powerful layered elastic based overlay analysis, the flexible overlay equation tends to be somewhat unconservative for thin overlay thicknesses and conservative for relatively thick overlay designs. Because of reflective cracking problems, flexible overlays are probably best suited as an interim rehabilitation technique that postpones more comprehensive restoration of a deteriorated pavement.

b. Because of concerns over FOD damage to jet aircraft engines, the empirical rigid and flexible overlay equations were developed for entirely different failure conditions in the accelerated traffic field tests upon which they were based. The rigid overlay sections were considered failed when initial structural cracks appeared, since such cracking was considered the precursor of spalling and potential FOD problems. Failure for the flexible overlays was taken to be when the underlying slab was shattered into 35 or more pieces and the subgrade was on the verge of failing. Because these equations represent two vastly different pavement conditions at the end of the pavement design life, it is not appropriate to try to make comparative cost comparisons between flexible overlays and rigid overlays designed using these equations. Also, this extreme terminal design condition for the flexible overlay equation is empirical and can give anomalous results such as negative numbers. This simply means the design case is outside the valid conditions, and the minimum thickness flexible overlay of 100 millimeters (4 inches) should be used.

3. SITE INVESTIGATIONS. Explorations and tests of the existing pavement will be made to determine the structural condition of the existing pavement prior to overlay, assess the required physical properties of the existing pavement and foundation materials, and locate and analyze all existing areas of defective pavement and subgrade that will require special treatment. The determination of the structural condition and required physical properties of the existing pavement will depend upon the type of overlay used as described in subsequent paragraphs. An investigation will be conducted to determine whether there are voids under the existing rigid pavement. This investigation is especially important if there has been, or is, any evidence of pumping or bleeding of water at cracks, joints, or edges of the existing rigid pavement. Nondestructive pavement test equipment has application for this type of investigation. If voids are found under the existing rigid pavements, fill the voids with grout before the overlay is placed. The results of the investigation, especially the nondestructive tests, may show rather large variations in the strength of the existing pavement and may lead to a requirement for more extensive testing to determine the cause of the variation. It will then be necessary to determine the feasibility and economics of using a variable thickness overlay, basing the design on the lower-strength pavement section, or removing and replacing the low-strength pavement areas.

4. PREPARATION OF EXISTING PAVEMENT.

a. General. The preparation of the existing pavement prior to overlay will vary, depending upon whether the overlay is rigid or nonrigid.

b. Rigid Overlay. Overlay thickness criteria are presented for three conditions of bond between the rigid overlay and existing rigid pavement: fully bonded, partially bonded, and nonbonded. The fully bonded condition is obtained when the concrete is cast directly on concrete

and special efforts are made to obtain bond. The partially bonded condition is obtained when the concrete is cast directly on concrete with no special efforts to achieve or destroy bond. The nonbonded condition is obtained when the bond is prevented by an intervening layer of material. When a fully bonded or partially bonded rigid overlay is to be used, the existing rigid pavement will be cleaned of all foreign matter (such as oil and paint), spalled concrete, extruded joint seal, bituminous patches, or anything else that would act as a bond-breaker between the overlay and existing rigid pavement.

(1) In addition, fully bonded overlays use careful surface preparation to ensure the overlay and underlying base slab are fully bonded and behave monolithically. To reliably achieve this full bond, the base slab is cold milled or shotblasted to remove all deteriorated or defective concrete and all surface contamination. This roughened surface must be thoroughly cleaned by sandblasting followed by airblasting, waterblasting, or both. Achieving and maintaining the surface cleanliness concrete placement is critical for achieving good bond. A portland-cement grout is then pneumatically applied immediately ahead of the concrete placement to help achieve a high degree of bond between the new and old concrete. This grout must not dry prior to placement of the concrete so usually it is only applied about 3 to 4 m ahead of the concrete placement. If the grout dries out prior to the concrete placement, the grout should be removed by sandblasting or other similarly reliable method and reapplied prior to continuing concrete placement. Older requirements for acid etching the base concrete surface are unnecessary and are not environmentally sound. Portland-cement grouts have proven adequate, and more expensive epoxy or polymer grouts are not normally needed. Some bonded overlays have reportedly been successfully placed with no bonding grout, but the military has no experience with such at present. For military airfield work where debonding poses such a serious FOD hazard, the intense surface preparation, surface cleaning, and use of a portland-cement grout are considered to be the minimum allowable effort for fully bonded overlays.

Past tests and studies have failed to identify adequate methods of providing satisfactory load transfer in fully bonded overlays. Consequently, fully bonded overlays will only be used on military airfields to correct surface deficiencies, and they are not suitable for structural upgrades unless the pavement is redesigned assuming no load transfer exists. The minimum thickness for a fully bonded overlay is 50 millimeters (2 inches), and most military airfield bonded overlays have been 75 to 125 millimeters (3 to 5 inches) thick. Typical past uses have included correction of surface smoothness or skid resistance problems, providing a sound operational surface over underlying pavements that are scaling, posing an FOD hazard from popouts or spalling and raveling, or to cover pavement surfaces that pose an FOD hazard from D-cracking, excess surface grout, or alkali-aggregate reaction deterioration.

All joints and cracks in the base pavement will reflect through a fully bonded overlay. Therefore, the overlay joints must match the base slab joints. Cracked slabs in the pavement to be overlaid should be removed and replaced, or the bonded overlay slab above the cracked slab should be reinforced.

(2) When a nonbonded rigid overlay is being used, the existing rigid pavement will be cleaned of all loose particles and covered with a leveling or bond-breaking course of bituminous concrete, sand-asphalt, heavy building paper, polyethylene, or other similar stable material. The bond-breaking medium generally should not exceed a thickness of about 25 millimeters (1 inch), except in the case of leveling courses where greater thicknesses may be necessary. When a rigid overlay is being applied to an existing flexible pavement, the surface of the existing pavement will be cleaned of loose materials and any potholing or unevenness, exceeding about 25 millimeters

(1 inch), will be repaired by cold planing, localized patching, or the application of a leveling course using bituminous concrete, sand-asphalt, or a similar material.

c. Nonrigid Overlay. When a flexible overlay is used, no special treatment of the surface of the existing pavement will be required other than the removal of loose material. When an all-bituminous concrete overlay is used, the surface of the existing pavement will be cleaned of all foreign matter. Spalled concrete, fat spots in bituminous patches, and extruded soft or spongy joint seal material on rigid pavements will be removed. Joints or cracks less than 25 millimeters (1 inch) wide in an existing rigid pavement will be filled with joint sealant. Joints or cracks that are 25 millimeters (1 inch) or greater in width will be cleaned and filled with an acceptable bituminous mixture (such as sand-asphalt) which is compatible with the overlay. Leveling courses of bituminous concrete will be used to bring the existing pavement to the proper grade when required. Prior to placing the all-bituminous concrete overlay, a tack coat will be applied to the surface of the existing pavement.

5. CONDITION OF EXISTING CONCRETE PAVEMENT.

a. General. The support that the existing rigid pavement will provide to an overlay is a function of its structural condition just prior to the overlay. In the overlay design equations, the structural condition of the existing concrete pavement is assessed by a condition factor, C. The value of C should be selected based upon a condition survey (ASTM D 5340) of the existing rigid pavement. Interpolation of C values between those shown below may be used if it is considered necessary to more accurately define the existing structural condition. As an alternative, Figure 17-1 may be used to select the C value for plain concrete or nonrigid overlays. This figure relates a structural condition index (SCI) and C. The SCI is that part of the pavement condition index (PCI) related to structural distress types as deduct values. To determine SCI values, a condition survey is conducted according to ASTM D 5340. However, rather than calculating the PCI, an SCI is calculated by subtracting the deduct values for corner breaks, longitudinal, transverse and diagonal cracking, shattered slabs, spalling along joints, and spalling corners from 100.

b. Rigid Overlay. The following values of C are assigned for the following conditions of plain and reinforced concrete pavements.

(1) Condition of existing plain concrete pavement:

C = 1.00 - Pavements in the trafficked areas are in good condition with little or no structural cracking because of load

C = 0.75 - Pavements in the trafficked areas exhibit initial cracking because of load but no progressive cracking or faulting of joints or cracks

C = 0.35 - Pavements in the trafficked areas exhibit progressive cracking because of load accompanied by spalling, raveling, or faulting of cracks and joints

(2) Condition of existing reinforced concrete pavement.

C = 1.00 - Pavements in the trafficked areas are in good condition with little or no short-spaced transverse (305- to 610-millimeter (1- to 2-foot)) cracks, no longitudinal cracking, and little spalling or raveling along cracks

C = 0.75 - Pavements in the trafficked areas exhibit short-spaced transverse cracking but little or no interconnecting longitudinal cracking because of load and only moderate spalling or raveling along cracks

C = 0.35 - Pavements in the trafficked areas exhibit severe short-spaced transverse cracking and interconnecting longitudinal cracking because of load, severe spalling along cracks, and initial punchout-type failures

c. Nonrigid Overlay. The following values of C are assigned for the following conditions of plain and reinforced concrete pavement.

(1) Condition of existing plain concrete pavements.

C = 1.00 - Pavements in the trafficked areas are in good condition with some cracking because of load but little or no progressive-type cracking

C = 0.75 - Pavements in the trafficked areas exhibit progressive cracking because of load and spalling, raveling, and minor faulting at joints and cracks

C = 0.50 - Pavements in the trafficked areas exhibit multiple cracking along with raveling, spalling, and faulting at joints and cracks

(2) Condition of existing reinforced concrete pavement.

C = 1.00 - Pavements in the trafficked areas are in good condition but exhibit some closely spaced load-induced transverse cracking, initial interconnecting longitudinal cracks, and moderate spalling or raveling of joints and cracks

C = 0.75 - Pavements in trafficked areas exhibit numerous closely spaced load-induced transverse and longitudinal cracks, rather severe spalling or raveling or initial evidence of punchout failures

6. RIGID OVERLAY OF EXISTING RIGID PAVEMENT.

a. General. There are three basic equations for the design of rigid overlays which depend upon the degree of bond that develops between the overlay and existing pavement: fully bonded, partially bonded, and nonbonded. The fully bonded overlay equation is used when special care is taken to provide bond between the overlay and the existing pavement. The partially bonded equation will be used when the rigid overlay is to be placed directly on the existing pavement and no special care is taken to provide bond. A bond-breaking medium and the nonbonded equation will be used when (a) a plain concrete overlay is used to overlay an existing reinforced concrete pavement, (b) when a continuously reinforced or prestressed concrete overlay is used to overlay an existing plain concrete or reinforced concrete pavement, (c) when a plain concrete overlay is being used to overlay an existing plain concrete pavement that has a condition factor $C \leq 0.35$, and

(d) when matching joints in a plain concrete overlay with those in the existing plain concrete pavement cause undue construction difficulties or result in odd-shaped slabs.

b. Plain Concrete Overlay.

(1) Thickness Determination. The required thickness h_o of plain concrete overlay will be determined from the following applicable equations:

Fully bonded

$$h_o = h_d - h_E \tag{17-1}$$

Partially bonded

$$h_o = \sqrt[1.4]{h_d^{1.4} - C\left(\frac{h_d}{h_e} \times h_E\right)^{1.4}} \tag{17-2}$$

Nonbonded

$$h_o = \sqrt{h_d^2 - C\left(\frac{h_d}{h_e} \times h_E\right)^2} \tag{17-3}$$

where

h_E = existing plain concrete pavement thickness

h_d and h_e = design thicknesses of rigid pavement determined using the design flexural strength of the overlay and measured flexural strength of the existing rigid pavement, respectively; the modulus of soil reaction k of the existing rigid pavement foundation; and the design loading, traffic area, and pass level needed for overlay design.

Use of fully bonded overlay is limited to existing pavements having a condition index of 1.0, and to overlay thickness of 50 to 120 millimeters (2.0 to 5.0 inches). The fully bonded overlay is used only to correct a surface problem such as scaling rather than as a structural upgrade. The factor h_E represents the thickness of the existing plain concrete pavement or the equivalent thickness of plain concrete pavement having the same load-carrying capacity as the existing pavement. If the existing pavement is reinforced concrete, h_E is determined from Figure 13-1 using the percent reinforcing steel S and design thickness h_e. The minimum thickness of plain concrete overlay will be 50 millimeters (2 inches) for a fully bonded overlay and 150 millimeters (6 inches) for a partially bonded or nonbonded overlay. The required thickness of overlay must be rounded to the nearest full- or half-inch increment. When the indicated thickness falls midway between a full and half-inch, the thickness will be rounded upward.

(2) Jointing. For all partially bonded and fully bonded plain concrete overlays, joints will be provided in the overlay to coincide with all joints in the existing rigid pavement. It is not necessary for joints in the overlay to be of the same type as joints in the existing pavement. When it is

impractical to match the joints in the overlay to joints in the existing rigid pavement, either a bond-breaking medium will be used and the overlay designed as a nonbonded overlay, or the overlay will be reinforced over the mismatched joints. Should the mismatch of joints become severe, a reinforced concrete overlay design should be considered as an economic alternative to the use of nonbonded plain concrete overlay. For nonbonded plain concrete overlays, the design and spacing of transverse contraction joints will be in accordance with requirements for plain concrete pavements on grade. For both partially bonded and nonbonded plain concrete overlays, the longitudinal construction joints will be doweled using the dowel size and spacing given in Table 12-8. Any contraction joint in the overlay that coincides with an expansion joint in the existing rigid pavement within the prescribed limits of a type A traffic area will be doweled. Dowels and load-transfer devices will not be used in fully bonded overlays. Joint sealing for plain concrete overlays will conform to the requirements for plain concrete pavements on grade.

(3) Example of Plain Concrete Overlay Design. An existing plain concrete pavement will be strengthened to serve as a type A traffic area for an Air Force medium-load pavement using a plain concrete overlay. The pertinent physical properties of the existing rigid pavement are: h_E = 200 millimeters (8 inches), R = 4.83 MPa (700 psi), and k = 27 MN/m^3 (100 pci). The design (90-day) flexural strength of the concrete for the overlay is 5.17 MPa (750 psi).

(a) The existing pavement is showing some initial cracking due to load so that the condition factor C is 0.75. The condition of the existing pavement is such that there is no reason to use a leveling course or other bond-breaking medium. The required thickness h_o of the plain concrete overlay is then determined using the partially bonded overlay equation (Equation 17-2). The design thickness h_d of plain concrete pavement, using the design flexural strength of 5.17 MPa (750 psi) for the overlay concrete and k = 27 MN/m^3 (100 pci) for the existing foundation, from Figure 12-7 (medium-load design curve) and type A traffic area, is 457 millimeters (18.0 inches). The design thickness h_e of plain concrete pavement, using the 4.82 MPa (700 psi) flexural strength of the existing pavement, a k value of 27 MN/m^3 (100 pci), and Figure 12-7 is 488 millimeters (19.2 inches). Since the existing rigid pavement is plain concrete, h_E = 200 millimeters (8 inches). Substituting these values in Equation 17-2,

$$h_o = \sqrt[1.4]{457^{1.4} - 0.75\left(\frac{457}{488} \times 200\right)^{1.4}}$$

$$= 384 \text{ mm (SI units)}$$

$$h_o = \sqrt[1.4]{18.0^{1.4} - 0.75\left(\frac{18.0}{19.2} \times 8\right)^{1.4}}$$

$$= 15.1 \text{ inches (use 15.0 inches) (English units)}$$

(b) The existing rigid pavement is 200 millimeters (8 inches) of reinforced concrete with 0.15 percent of reinforcing steel S and a condition factor C of 0.75. All properties of the existing pavement and proposed plain concrete overlay are the same as above. Since the existing pavement is reinforced concrete, it will be necessary to use a bond-breaking medium and determine the required thickness of plain concrete overlay using the nonbonded overlay equation

(Equation 17-3). The design thickness h_d of plain concrete is 457 millimeters (18.0 inches), and the design thickness h_e is 488 millimeters (19.2 inches). The value of h_E, the thickness of plain concrete pavement equivalent to the existing thickness of reinforced concrete pavement, determined from Figure 13-1 using the existing thickness of reinforced concrete pavement of 200 millimeters (8 inches) and S = 0.15 percent, is 241 millimeters (9.5 inches). Substituting these values in the equation above,

$$h_o = \sqrt{457^2 - 0.75\left(\frac{457}{488} \times 241\right)^2}$$

$$= 413 \text{ millimeters (SI units)}$$

$$h_o = \sqrt{18.0^2 - 0.75\left(\frac{18.0}{19.2} \times 9.5\right)^2}$$

$$= 16.3 \text{ inches (use 16.5 inches) (English units)}$$

c. Reinforced Concrete Overlay. A reinforced concrete overlay may be used to strengthen either an existing plain concrete or reinforced concrete pavement. Generally, the overlay will be designed as a partially bonded overlay. The nonbonded overlay design will be used only when a leveling course is required over the existing pavement. The reinforcement steel for reinforced concrete overlays will be designed and placed in accordance with reinforced concrete slabs on grade.

(1) Thickness determination. The required thickness of reinforced concrete overlay will be determined using Figure 13-1 after the thickness of plain concrete overlay has been determined using the appropriate overlay equation. Then, using the value for the thickness of plain concrete overlay, either the thickness of reinforced concrete overlay can be selected and the required percent steel determined, or the percent steel can be selected and the thickness of reinforced concrete overlay determined from Figure 13-1. The minimum thickness of reinforced concrete overlay will be 152 millimeters (6 inches).

(2) Jointing. Whenever possible, the longitudinal construction joints in the overlay should match the longitudinal joints in the existing pavement. All longitudinal joints will be of the butt-doweled type with dowel size and spacing designated in accordance with Chapter 12 using the thickness of reinforced concrete overlay. It is not necessary for transverse joints in the overlay to match joints in the existing pavement; however, when practical, the joints should be matched. The maximum spacing of transverse contraction joints will be determined in accordance with Figure 13-1, but it will not exceed 30 meters (100 feet) regardless of the thickness of the pavement or the percent steel used. Joint sealing for reinforced concrete pavements will conform to the requirements for plain concrete pavements.

(3) Example of reinforced concrete overlay design. An existing rigid pavement will be strengthened to serve as a type B traffic area for a heavy-load pavement using a reinforced concrete overlay. The pertinent physical properties of the existing plain concrete pavement are: h_E = 250 millimeters (10 inches), R = 4.48 MPa (650 psi), and k = 54 MN/m^3 (200 pci). The design (90-day) flexural strength of the overlay is 5.17 MPa (750 psi).

(a) The existing rigid pavement is plain concrete with a structural condition C of 0.35; however, there is no significant faulting of the slabs and a leveling course is not needed. The required thickness of plain concrete overlay is determined using the partially bonded overlay equation (Equation 17-2). The required thickness h_d of plain concrete pavement for the overlay design flexural strength of 5.17 MPa (750 psi) and the k value of 54 MN/m^3 (200 pci) for the foundation under the existing pavement determined from Figure 12-8 (heavy-load design curve) type B traffic area is 521 millimeters (20.5 inches). The design thickness h_e of plain concrete pavement for the flexural strength of 4.48 MPa (650 psi) of the existing pavement and the k value of 54 MN/m^3 (200 pci) from Figure 12-8 is 574 millimeters (22.6 inches). Since the existing pavement is plain concrete, the equivalent thickness h_E is equal to the 250-millimeter (10-inch) thickness of the existing slab. Substituting these values in Equation 17-2,

$$h_o = \sqrt[1.4]{521^{1.4} - 0.35\left(\frac{521}{574} \times 250\right)^{1.4}}$$

$$= 478 \text{ millimeters (SI units)}$$

$$h_o = \sqrt[1.4]{20.5^{1.4} - 0.35\left(\frac{20.5}{22.6} \times 10\right)^{1.4}}$$

$$= 18.8 \text{ inches (English units) (Use 19.0 inches)}$$

This is the thickness of the plain concrete overlay required to strengthen the existing plain concrete pavement for the design loading condition. The thickness of reinforced concrete overlay is then dependent upon the percent of reinforcing steel S that will be used. Let it be assumed that because of grade problems, the overlay thickness must be limited to 380 millimeters (15 inches). Then, the value of S required, determined from Figure 13-1 using the plain concrete overlay thickness of 478 millimeters (19.0 inches) and the reinforced concrete overlay thickness of 380 millimeters (15 inches), is 0.25 percent. It is also noted from Figure 13-1 that a maximum joint spacing of 30 meters (100 feet) may be used with a reinforcing steel having a yield strength y_s of 413 MPa (60,000 psi).

(b) The existing pavement in the example above consists of 250 millimeters (10 inches) of reinforced concrete with 0.10 percent of reinforcing steel and all other properties and design requirements remain the same. The thickness of plain concrete pavement h_E equivalent to the 250 millimeters (10 inches) of existing reinforced concrete pavement, determined from Figure 13-1 using the existing thickness of 250 millimeters (10 inches) and S = 0.10, is 287 millimeters (11.3 inches). Substituting these values in the partially bonded overlay equation yields a required overlay thickness h_o of plain concrete equal to:

$$h_o = \sqrt[1.4]{521^{1.4} - 0.35\left(\frac{521}{574} \times 287\right)^{1.4}}$$

$$= 470 \text{ millimeters (SI units)}$$

$$h_o = \sqrt[1.4]{20.5^{1.4} - 0.35\left(\frac{20.5}{22.6} \times 11.0\right)^{1.4}}$$

$$= 18.5 \text{ inches (English units)}$$

From Figure 13-1, the thickness of reinforced concrete overlay using the thickness of plain concrete of 470 millimeters (18.5 inches) and a percent steel of 0.20 is 380 millimeters (15 inches).

d. Continuously Reinforced Concrete Overlay. A continuously reinforced concrete overlay may be used to strengthen either an existing plain concrete or reinforced concrete pavement. For both conditions, a bond-breaking medium is required between the overlay and the existing pavement. The required thickness of a continuously reinforced concrete pavement is determined in the same manner and will be equal in thickness to a plain concrete overlay. Jointing and sealing of joints in a continuously reinforced concrete pavement will be the same as for continuously reinforced concrete pavements on grade.

e. Fibrous Concrete Overlay. A fibrous concrete overlay may be used to strengthen either an existing plain or reinforced concrete pavement. The mix proportioning of the fibrous concrete overlay will follow the considerations presented for fibrous concrete pavements on grade.

(1) Thickness determination. The required thickness of fibrous concrete overlay will be determined using the partially bonded or nonbonded overlay equations. Normally, the partially bonded equation will be used, but in cases of extremely faulted or uneven existing pavement surfaces, a leveling course may be required and the design of the overlay will be made using the nonbonded overlay equation. If the existing rigid pavement is plain concrete, then the equivalent thickness is equal to the existing slab thickness. If the existing rigid pavement is reinforced concrete, however, then the equivalent thickness must be determined from Figure 13-1 using the thickness of the existing slab and the percent of reinforcing steel. The minimum thickness of fibrous concrete overlay will be 100 millimeters (4 inches).

(2) Jointing. In general, the joint types, spacing, and designs discussed for plain concrete pavements apply to fibrous concrete overlays, except that for thicknesses from 100 millimeters (4 inches) to 150 millimeters (6 inches), the maximum spacing will be 3.8 meters (12.5 feet). Joints in the fibrous overlay should coincide with joints in the existing rigid pavement. Longitudinal construction joints will be the butt-doweled type, and dowels will be required in transverse contraction joints exceeding 15-meter (50-foot) spacings. For pavement thickness less than 150 millimeters (6 inches), it will be necessary to obtain guidance on joint construction from HQUSACE (CEMP-ET), the appropriate Air Force Major Command, or the Naval Facilities Engineering Command. Sealing of joints in fibrous overlays will be in accordance with sealing of joints in fibrous concrete pavements on grade.

(3) Example of fibrous concrete overlay design. An existing rigid pavement will be strengthened to serve as a type B traffic area for an Air Force light-load pavement using a fibrous concrete overlay. The pertinent physical properties of the existing rigid pavement are: existing thickness is 150 millimeters (6 inches), R = 4.8 MPa (700 psi), and k = 27 MN/m^3 (100 pci). The design (90-day) flexural strength of the fibrous concrete overlay is 6.2 MPa (900 psi). The existing rigid pavement is plain concrete with a structural condition, C, of 1.0. A leveling course will not be required; therefore, the required thickness of fibrous concrete overlay will be determined using the

partially bonded overlay equation (Equation 17-2). Use of this equation requires that h_d be the thickness of fibrous concrete from the appropriate fibrous concrete design curve. The design thickness of fibrous concrete pavement is determined from Figure 14-4 to be 228 millimeters (9 inches) using the design flexural strength of the fibrous concrete overlay (6.2 MPa (900 psi)) and k value of (27 MN/m^3 (100 pci)) for the existing rigid pavement foundation. The design thickness of plain concrete, using the flexural strength of the existing pavement (4.8 MPa (700 psi)) and k of 27 MN/m^3 (100 pci) for the existing foundation strength, is 310 millimeters (12.2 inches). Since the existing rigid pavement is plain concrete, h_E = 150 millimeters (6 inches); substituting these values in the partially bonded overlay equation yields a required thickness of fibrous concrete overlay of:

$$h_o = \sqrt[1.4]{228^{1.4} - 1.0\left(\frac{228}{310} \times 150\right)^{1.4}}$$

$$= 165 \text{ millimeters (SI units)}$$

$$h_o = \sqrt[1.4]{8.99^{1.4} - 1.0\left(\frac{9.00}{12.2} \times 6\right)^{1.4}}$$

$$= 6.5 \text{ inches (English units)}$$

7. PRESTRESSED CONCRETE OVERLAY OF RIGID PAVEMENT. A prestressed concrete overlay may be used above any rigid pavement. The procedure for designing the prestressed concrete overlay is to consider the base pavement to have a k value of 135 MN/m^3 (500 pci) and design the overlay as a prestressed concrete pavement on grade.

8. RIGID OVERLAY OF EXISTING FLEXIBLE OR COMPOSITE PAVEMENT. Any type of rigid overlay may be used to strengthen an existing flexible or composite pavement. The existing pavement is considered to be a composite pavement when it is composed of a rigid base pavement that has been strengthened with 100 millimeters (4 inches) or more of nonrigid (flexible or all-bituminous) overlay. If the nonrigid overlay is less than 100 millimeters (4 inches), the rigid overlay is designed using the nonbonded overlay equation. The design of the rigid overlay will follow the procedures outlined in Chapters 12 through 16 of this document. The strength afforded by the existing pavement will be characterized by the modulus of soil reaction k determined using the plate bearing test, or Figure 8-1. The following modifications or limitations apply: (a) The plate bearing test will be performed when the pavement temperature equals or exceeds the maximum ambient temperature for the hottest period of the year, and (b) in no case will a k value greater than 135 MN/m^3 (500 pci) be used for design. When Figure 8-1 is used to estimate the k value at the surface of the existing flexible pavement, the bituminous concrete portion will be assumed to be unbound base material since its performance will be similar to a base course.

9. NONRIGID OVERLAY OF EXISTING RIGID PAVEMENT.

a. General. Two types of nonrigid overlay, all-bituminous concrete overlay, and flexible overlay, may be used with certain reservations to strengthen an existing rigid pavement.

b. All-Bituminous Overlay. The all-bituminous overlay will be composed of hot-mix bituminous concrete meeting the requirements of TM 5-822-8/AFM 88-6, Chapter 9. A tack coat is required between the existing rigid pavement and the overlay. The all-bituminous overlay is the preferred nonrigid type overlay to lessen the danger of entrapped moisture in the overlay.

c. Flexible Overlay. The flexible overlay will be composed of hot-mix bituminous concrete and high-quality crushed aggregate base with a CBR of 100, provided positive drainage of the base course is achieved. The bituminous concrete will meet the requirements of TM 5-822-8/AFM 88-6, Chapter 9 and the minimum thickness requirements of Chapter 8. If the design thickness of nonrigid overlay is less than that required by the minimum thickness of bituminous concrete and base course, the overlay will be designed as an all-bituminous overlay.

d. Thickness Determination. Regardless of the type of nonrigid overlay, the required thickness t_o will be determined by

$$t_o = 3.0(Fh_d - Ch_E) \tag{17-4}$$

where

h_d = design thickness of plain concrete pavement using the flexural strength R of the concrete in the existing rigid pavement and the modulus of soil reaction k of the existing pavement.

The factor h_E represents the thickness of plain concrete pavement equivalent in load-carrying ability to the thickness of existing rigid pavement. If the existing rigid pavement is plain concrete, then the equivalent thickness equals the existing thickness; however, if the existing rigid pavement is reinforced concrete, the equivalent thickness must be determined from Figure 13-1. F is a factor, determined from Figure 17-2, that projects the cracking expected to occur in the base pavement during the design life of the overlay. Use of Figure 17-2 requires converting passes to coverages using values shown in Table 17-1. C is a coefficient based upon the structural condition of the existing rigid pavement. The minimum thickness of overlay used for strengthening purposes will be 50 millimeters (2 inches) for Air Force type D traffic areas and all overruns, 75 millimeters (3 inches) for Army Class I, II, and III pavements, 75 millimeters (3 inches) for Air Force types B and C traffic areas on light-load pavements, 75 millimeters (3 inches) for Navy and Marine Corps secondary pavements designed for fighter aircraft, and 100 millimeters (4 inches) for all other Army, Air Force, Navy and Marine Corps pavements. In certain instances, the nonrigid overlay design equation will indicate thickness requirements less (sometimes negative values) than the minimum values. In such cases the minimum thickness requirement will be used. When strengthening existing rigid pavements that exhibit flexural strength less than 3.5 MPa (500 psi) or that are constructed on foundations with k values exceeding 54 MN/m^3 (200 pci), it may be found that the flexible pavement design procedure in Chapter 10 or 11 may indicate a lesser required overlay thickness than the overlay design formula. For these conditions, the overlay thickness will be determined by both methods, and the lesser thickness used for design. For the flexible pavement design procedure, the existing rigid pavement will be considered to be either an equivalent thickness of high-quality crushed aggregate base with a CBR = 100 or an equivalent thickness of all-bituminous concrete (equivalency factor of 1.15 for base and 2.3 for subbase), and the total pavement thickness determined based upon the subgrade CBR. Any existing base or subbase layers will be considered as corresponding layers in the flexible pavement. The thickness of required overlay will then be the difference between the required flexible pavement thickness

and the combined thicknesses of existing rigid pavement and any base or subbase layers above the subgrade.

e. Reflective Cracking. If a flexible overlay is placed over a rigid pavement, the underlying joints will reflect through the overlay, and these cracks will progressively deteriorate by raveling. This reflective cracking is primarily caused by seasonal and diurnal environmental changes occurring in the overlaid rigid pavement, and reflective cracking will often appear during the first winter after the placement of the overlay. At present there is no completely reliable method of preventing reflective cracking. Consequently, in many cases, the designer should probably consider a flexible overlay as a maintenance tool to upgrade the serviceability and to a more limited extent, the structural capacity of a rigid pavement for a relatively limited time while more comprehensive rehabilitation is postponed to the future. Some methods of ameliorating the adverse effects of reflective cracking include:

(1) Overlay Thickness: The thicker the overlay, the longer the cracking will be postponed and the slower it will deteriorate. Hence, abiding by minimum flexible overlay thicknesses is an important issue.

(2) Saw and Seal: Since there is no way to reliably avoid reflective cracking, another approach is to saw the flexible overlay directly above the rigid pavement joints and seal this with an appropriate sealer. These sealed cuts are then more easily and effectively maintained than the reflective cracks would be. The Air Force has found this to be an effective approach, and it is generally their preferred approach to dealing with flexible overlays over rigid pavements.

(3) Geotextiles: Geotextiles have shown a limited ability to slow the development and severity of reflective cracking in warm climates. Field trials found that in Area I of Figure 17-3, geotextiles were usually helpful, in Area II they gave mixed results, and in Area III, they were ineffective in dealing with reflective cracking. The minumum overlay thickness is 100 millimeters (4 inches).

(4) Crack & Seat and Rubblizing: An alternative approach is to break the existing rigid pavement slabs into smaller individual segments (crack and seat) or to pulverize them into shattered small fragments (essentially rubblize to aggregate) before overlaying . Concetually, the shattered slabs or rubblized concrete fragments are then too small to develop movements to generate reflective cracks. This technique has proven successful on highways, but experience on thicker pavements such as found on airfields is very limited at present.

(5) Bond Breakers: Open graded materials, aggregate bases as part of the flexible overlay, and specially designed stress/strain absorbing membranes have all been tried to provide a layer capable of absorbing the movement of the underlying rigid pavement without transmitting it to the asphalt concrete overlay surface. These have given mixed results, and some systems are proprietary.

(6) Reinforcing: Besides geotextiles, other proprietary reinforcing systems using steel wire and fiberglass grids to combat reflective cracking are available. These have not been evaluated by the military.

Military experience has found thicker overlays and in some warm climates geotextiles may help mitigate but not prevent reflective cracking in flexible overlays. Sawing and sealing above the rigid pavement joints has also been found to be a pragmatic way of minimizing the problems with

Table 17-1
Pass per Coverage Ratios

Aircraft Type	Pass per Coverage Ratios for Traffic Areas	
	Traffic Area A	Traffic Areas B, C, D, and Overruns
B-1	3.41	5.65
B-52	1.58	2.15
B-727	3.32	5.87
C-5A	1.66	2.11
C-9	3.73	6.89
C-12	7.07	13.89
C-17	1.37	1.90
C-130	4.40	8.54
C-141	3.49	6.23
CH-46E	8.01	15.22
CH-47	4.38	7.64
CH-53E	5.23	9.53
CH-54	4.31	8.51
DC-10-10	3.64	5.80
DC-10-30	3.77	5.59
E-4	3.62	5.12
F-4C	8.77	17.37
F-14	7.78	15.34
F-15 C&D	9.30	15.34
F-15E	8.10	13.36
F/A-18	9.57	17.04
F-111	5.63	9.77
KC-135	3.48	6.14
L-1011	3.58	5.44
ORBITER	3.60	6.49
OV-1	10.36	17.28
P-3	3.58	6.68
UH-60	11.94	19.49

reflective cracking. The other techniques discussed above have given mixed results or have not been independently evaluated by the military.

f. Example of Nonrigid Overlay Design. An existing rigid pavement will be strengthened to support 75,000 operations of the C-130 aircraft using a nonrigid overlay. It is a type B traffic area. The existing rigid pavement is 229 millimeters (9 inches) of plain concrete on a 152-millimeter (6-inch) crushed aggregate base and has the following properties: R = 4.8 MPa (700 psi), k of subgrade = 41 MN/m^3 (150 pci), and C = 0.75. The k value on top of the base course is determined to be 54 MN/m^3 (200 pci) from Figure 8-1 using the subgrade k of 41 MN/m^3 (150 pci) and 152 millimeters (6 inches) of base course. The required thickness of nonrigid overlay is determined by

$$t_o = 3.0\ (Fh_d - Ch_E) \qquad (17\text{-}5)$$

To determine F, convert passes of the C-130 into coverages using the pass per coverages ratio of 8.54 from Table 17-1. The 75,000 passes convert to 1,171 coverages. Therefore, for a k of 54 MN/m^3 (200 pci) and 1,171 coverages, the F factor from Figure 19-2 is 0.78. Values of h_d, determined from Figure 12-5 with the design gross aircraft weight of 79,380 kilograms (155 kips), flexural strength of 4.8 MPa (700 psi), and k value of 54 MN/m^3 (200 pci) is 256 millimeters (10 inches). Since the existing pavement is concrete, then the equivalent thickness h_E is equal to the existing thickness of 229 millimeters (9.0 inches). Therefore, the required overlay thickness t_o is:

$$(3)(0.78 \times 256 - 0.75 \times 229) = 84 \text{ mm (SI units)}$$

$$(3)(0.78 \times 10 - 0.75 \times 9) = 3.15 \text{ in. (English units)}$$

Use the minimum thickness of 102 millimeters (4 inches).

10. NONRIGID OVERLAY ON FLEXIBLE PAVEMENT. After a determination has been made that strengthening of a flexible pavement is required, design the overlay thickness as follows:

a. Determine the total thickness of the section, and the thickness of the base and surface courses from the criteria in Chapter 12 or 13 for the design aircraft. Compare the new design requirements with the existing section to determine the thickness of overlay required.

b. Where the in-place density of the existing material is less than required, the overlay thickness should be increased or the low-density material recompacted. In some instances this is possible by using heavy rollers on the surface to compact the underlying layers. However, if the moisture content of these layers, particularly if cohesive, is above optimum, their shear strength may be decreased by heavy rolling. Heavy rolling, also, will frequently damage the surface layer if brittle. The decision to excavate and recompact low density layers or to increase the overlay thickness should be examined very carefully in each case. Factors to be considered in this examination are depth of water table, subgrade soil properties, and the performance of the existing pavement.

c. Overlaid asphalt courses must meet the quality requirements for their position in the strengthened pavement.

d. As an example of the overlay design procedure, it is assumed that an existing pavement (type B traffic area) is to be upgraded to an Air Force medium-load pavement. The existing pavement consists of 75 millimeters (3 inches) of AC, 150 millimeters (6 inches) of base (100 CBR), and 533 millimeters (21 inches) of subbase (50 CBR). The subgrade is a lean clay with a CBR of 6, and has a density of 95 percent in the top 150 millimeters (6 inches) and 90 percent below 150 millimeters (6 inches). From Figure 10-18, the total thickness of new pavement required is 1,143 millimeters (45 inches) and the thickness of base and surface required over the 50 CBR subbase is 178 millimeters (7 inches). However, from Table 8-5, the minimum surface and aggregate base course required for medium load and traffic area B is 25 millimeters (10 inches). From Table 6-2, it is noted that the in-place subgrade density is adequate. Based on above information, the following analysis is made:

(1) New design criteria requires a 1,143-millimeter (45-inch) pavement section above the subgrade and, from Table 8-5, 254 millimeters (10 inches) above the subbase.

(2) Existing pavement section is 758 millimeters (30 inches) with 225 millimeters (9 inches) above the subbase.

(3) In this example, the existing thickness would require an additional inch of pavement to meet the minimum thickness asphalt and the thickness required above the subbase. A 1-inch overlay however is not sufficient to protect the subgrade which requires 1,143 millimeters (45 inches) (or equivalent) of pavement above it. Any thickness of asphalt exceeding the minimum of 100 millimeters (4 inches) can be converted to an equivalent thickness of subbase. This excessive thickness of asphalt is equal to the thickness of overlay plus any existing thickness of asphalt minus the minimum thickness. The required thickness of overlay (t_o) is then determined as follows:

$$t_m + (t_o + t_e - t_m)E_f + t_b + t_{Sb} = T_p \qquad (17\text{-}6)$$

where

t_m = minimum thickness of asphalt

t_o = thickness of overlay

t_e = thickness of existing asphalt

E_f = equivalency factor for converting asphalt to an equivalent thickness of subbase

t_{Sb} = thickness of existing base course

t_b = thickness of existing subbase

T_p = total thickness of pavement required above the subgrade

For this example, the equation is

$$4 + (t_o + 3 - 4)2.3 + 6 + 21 = 45 \text{ (English units)}$$

The thickness of overlay to use would then be 178 millimeters (7 inches).

11. OVERLAYS IN FROST REGIONS. Whenever the subgrade is subject to frost action, the design will meet the requirements for frost action stated in Chapter 22. The design will conform to frost requirements for rigid pavements. If subgrade conditions will produce detrimental nonuniform frost heaving, overlay pavement design will not be considered unless the combined thickness of overlay and existing pavement is sufficient to prevent substantial freezing of the subgrade.

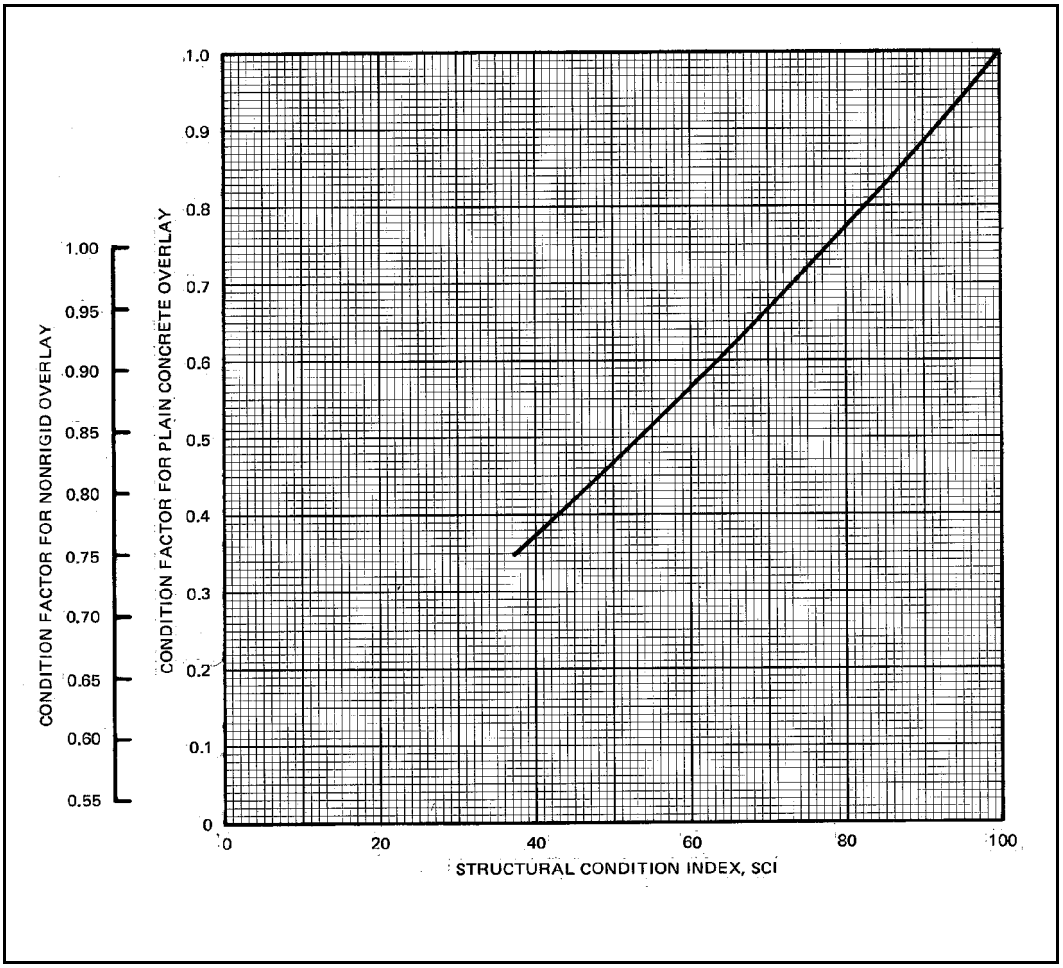


Figure 17-1. Structural condition index versus condition factor

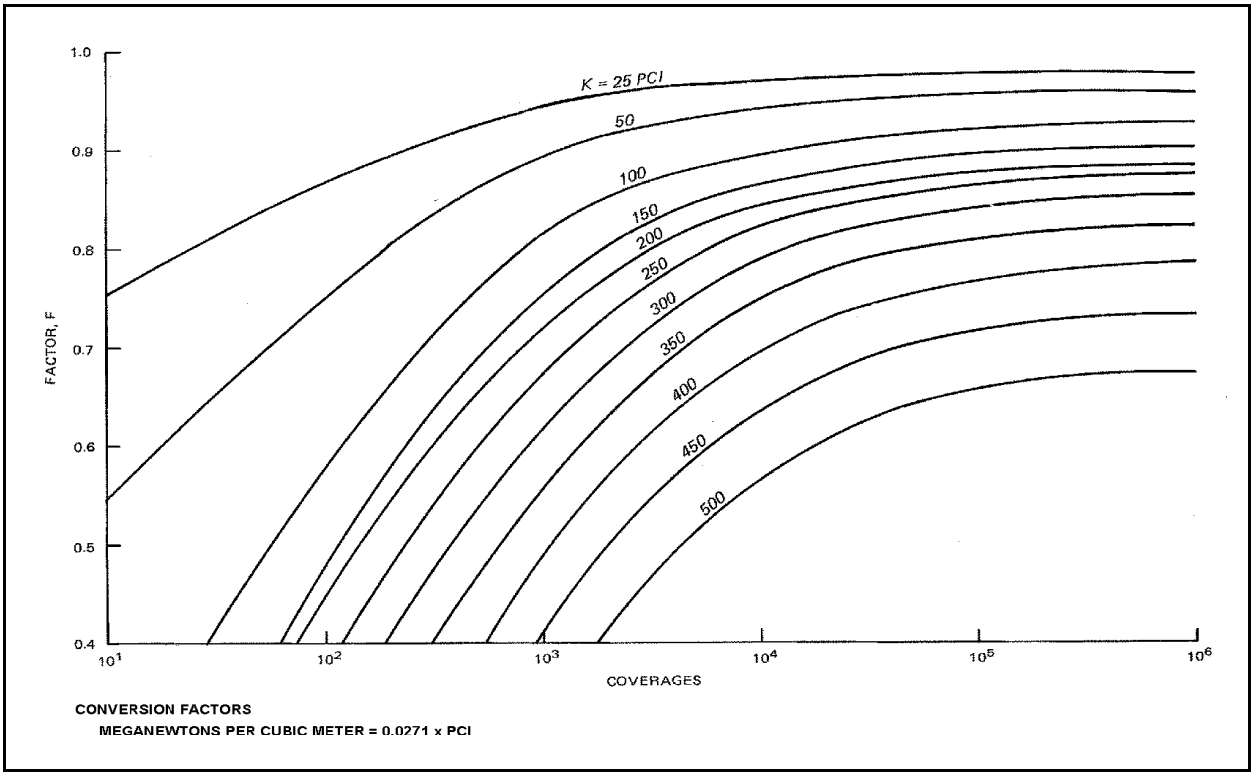


Figure 17-2. Factor for projecting cracking in a flexible pavement

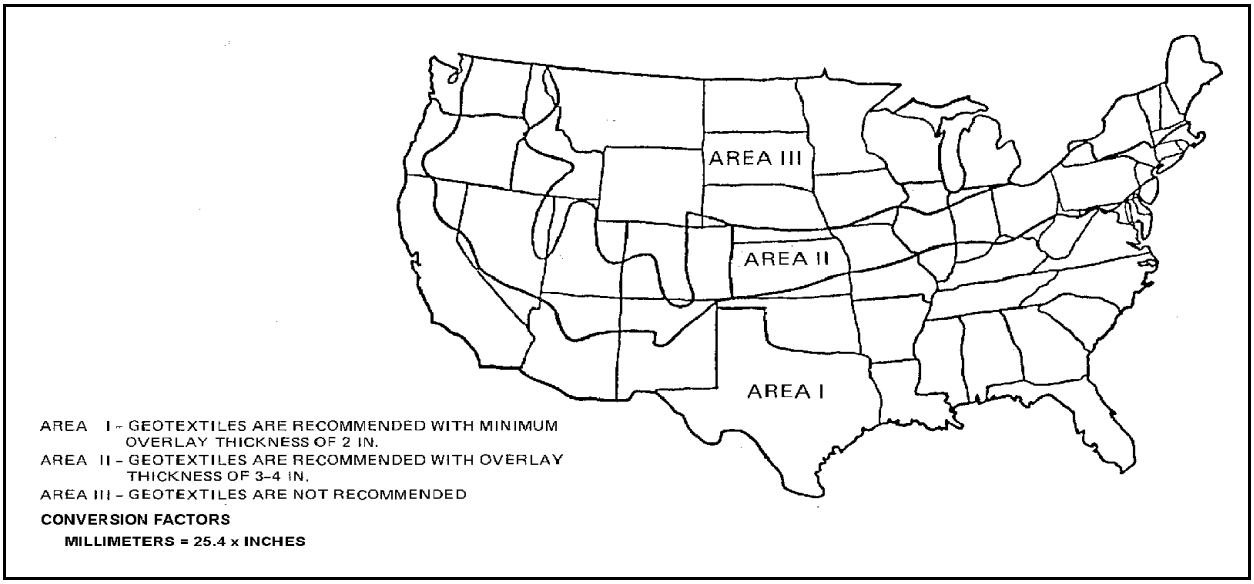


Figure 17-3. Location guide for the use of geotextiles in retarding reflective cracking

CHAPTER 18

RIGID PAVEMENT INLAY DESIGN

1. GENERAL. Many existing airfield pavement facilities have developed severe distress because the design life or the load-carrying capacity of the facilities has been exceeded. The distress normally occurs first in the center lanes of the runways and taxiways because of the concentration of traffic. A method commonly used to rehabilitate these distressed facilities is to construct an adequately designed rigid pavement inlay section in the center of the facility. These inlays are generally 15 meters (50 feet) wide for taxiways and 23 meters (75 feet) wide for runways; however, the widths will be influenced by the lateral traffic distribution and, in existing rigid pavements, by the joint configuration. The inlay pavement may consist of plain concrete or reinforced concrete. The thickness design of the rigid inlay will be the same as outlined in Chapters 12 through 16 or 19, except for the special requirements presented herein.

2. RIGID INLAYS IN EXISTING FLEXIBLE PAVEMENT.

a. Figure 18-1 shows a section of a typical rigid pavement inlay in an existing flexible pavement.

b. Removal of the existing flexible pavement will be held to the absolute minimum. The depth of the excavation will not exceed the design thickness of rigid inlay pavement. The width of excavation of the existing pavement will not exceed the required width of the inlay section plus the minimum necessary, approximately 1 meter (3 feet), for forming or slipforming the edges of the concrete pavement (Figure 18-1).

c. Subdrains and drainage layers will be considered only when they are essential to the construction of the inlay section or necessary for proper drainage. When required, the subdrains will be placed outside of the edge of the rigid inlay and at least 100 millimeters (4 inches) below the bottom of the inlay pavement to permit construction of the stabilized layer required in the following paragraph.

d. Unless the material in the bottom of the excavation is granular and free-draining or the airfield is located in a arid climate, the bottom full width of the excavation will be scarified to a minimum depth of 150 millimeters (6 inches), and recompacted to the density requirements for the top 150 millimeters (6 inches) of base course or subgrade as specified previously. This type of overlay may trap water, and satisfactory drainage must be provided. Reference should be made to TM 5-822-14/AFJMAN 32-1019 for selection of stabilizing agent and minimum strength requirements.

e. The modulus of soil reaction k used for the design of the rigid pavement inlay will be determined on the surface of the material at the bottom of the excavation prior to stabilization. If stabilization is used and if the strength of the stabilized material does not meet the requirements in TM 5-822-14/AFJMAN 32-1019 for pavement thickness reduction, no structural credit will be given to the stabilized material in the design of the rigid pavement inlay. If the strength of the stabilized layer meets the minimum strength requirement for pavement thickness reduction in TM 5-822-14/AFJMAN 32-1019, the rigid pavement inlay will be designed in accordance with applicable sections of Chapters 12 through 16 pertaining to the use of stabilized soil layers.

f. If the existing pavement is not composed of nonfrost-susceptible materials sufficient to eliminate substantial frost penetration into an underlying frost-susceptible material, an appropriate reduction in the k value will be made in accordance with Chapter 20.

g. After the construction of the rigid pavement inlay, the working areas used for forming or slipforming the sides of the concrete will be backfilled to within 100 millimeters (4 inches) of the pavement surface with either lean-mix concrete or normal paving concrete.

h. The existing bituminous concrete will be sawed parallel to and at a distance of 3 meters (10 feet) from each edge of the inlay. The bituminous concrete surface and binder courses and, if necessary, the base course will be removed to provide a depth of 100 millimeters (4 inches). The exposed surface of the base course will be recompacted, and a 3-meter (10-foot) wide paving lane of bituminous concrete, 100 millimeters (4 inches) thick, will be used to fill the gap (Figure 18-1). The bituminous concrete mix will be designed in accordance with Chapter 9.

I. In cases where the 3-meter (10-foot) width of new bituminous concrete at either side of the inlay section does not permit a reasonably smooth transition from the inlay to the existing pavement, additional leveling work outside of the 3-meter (10-foot) lane will be accomplished by removal and replacement, planer operation, or both.

3. RIGID INLAYS IN EXISTING RIGID PAVEMENT.

a. Figure 18-2 shows a section of a typical rigid pavement inlay in an existing rigid pavement.

b. The existing rigid pavement will be removed to the nearest longitudinal joints that will provide the design width of the rigid pavement inlay. Care will be exercised in the removal of the existing rigid pavement to preserve the load-transfer device (key, keyway, or dowel) in the longitudinal joint at the edge of the new inlay pavement. If the existing load-transfer devices can be kept intact, they will be used to provide load transfer between the rigid pavement inlay and the existing pavement except that a male key will be removed. If the load-transfer devices are damaged or destroyed, a thickened-edge joint shall be used to protect against edge loading of the existing pavement or the face shall be sawed vertically and dowels installed. In addition to the removal of the existing pavement, the existing base and/or subgrade will be removed to the depth required for the design thickness of the rigid pavement inlay.

c. The criteria for subdrains, stabilization, soil strength and frost also pertain to rigid pavement inlays in existing rigid pavements.

d. The design of the rigid pavement inlay, including joint types and spacing, will be in accordance with the chapter pertaining to the type of rigid pavement selected.

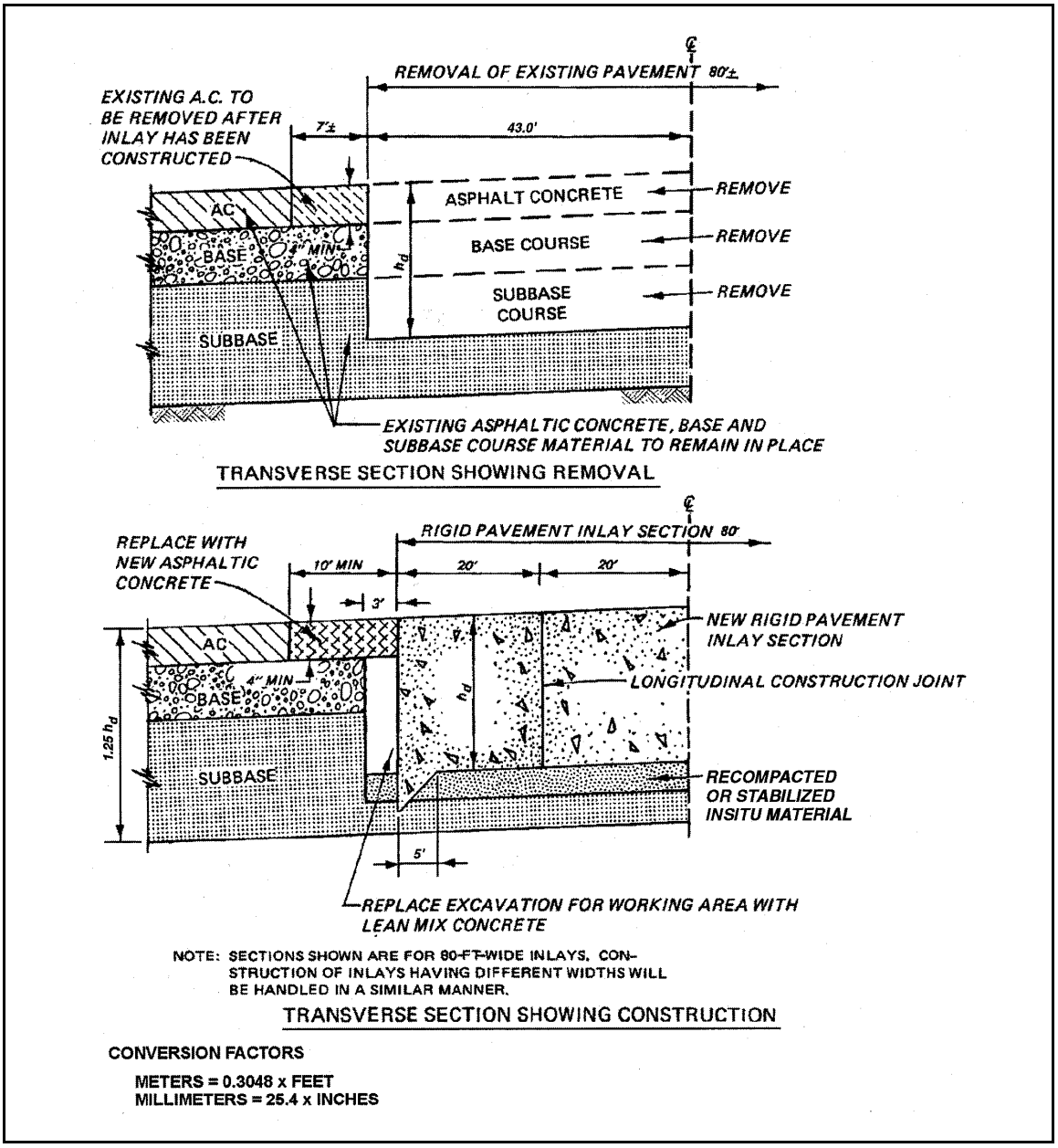


Figure 18-1. Typical rigid pavement inlay in existing flexible pavement

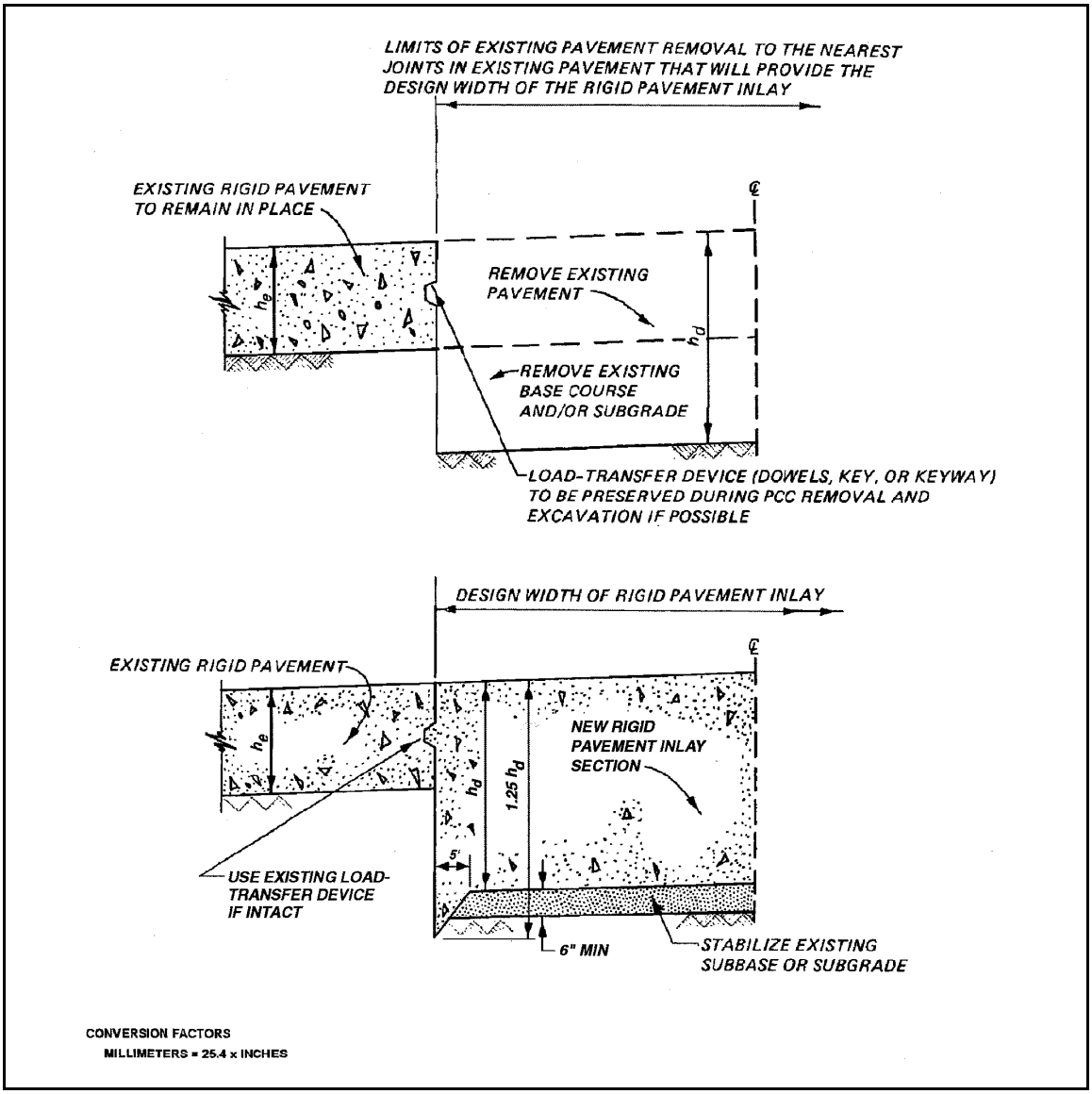


Figure 18-2. Typical rigid pavement inlay in existing rigid pavement

CHAPTER 19

LAYER ELASTIC DESIGN OF RIGID PAVEMENTS

1.RIGID PAVEMENT DESIGN PRINCIPLES. The basic design principle for this design procedure is to limit the tensile stresses in the Portland Cement Concrete (PCC) to levels that are sufficiently below the flexural strength of the concrete such that failure occurs only after the pavement has sustained a number of load repetitions. The tensile stress is modeled by the use of Burmister's solution for elastic multilayered continua and calculated using the JULEA computer program. The computed tensile stress divided by the concrete strength is the design parameter and is referred to as the design factor. This parameter has been related to pavement performance through a study of test section data. To account for mixed traffic, i.e., traffic producing stresses of varying magnitudes, the cumulative damage concept based on Miner's hypothesis is employed. This procedure may be used as an option to the empirical procedure for the design of new Navy pavements. The design procedure is illustrated in Figure 19-1 and summarized as follows:

a. Select three or four concrete slab thicknesses and compute the maximum tensile stresses in the slabs under the design aircraft load.

b. Based on the computed stresses, determine the allowable coverages N_i ($N_i = C_o$ for initial cracking criteria or $N_i = C_f$ for complete failure criteria) using Equations 19-1, 19-2, and 19-3 for each thickness design.

c. Compute the damage for each design which is equal to the ratio of the design coverage n_i to the allowable coverage N_i, where i varies from 1 to the number of aircraft.

d. Select the proper slab thickness at a damage value of 1.0 from the relationship between damage and slab thicknesses.

e. The selection of an unbound granular base or a stabilized base under the concrete slab is a matter of engineering judgment depending on many factors such as cost, material availability, frost penetration requirement, and subgrade swell potential. Subgrade soil may be stabilized to gain strength or modified to increase its workability and reduce swell potential.

2.R IGID PAVEMENT RESPONSE MODEL. The pavement is assumed to be a multilayered continuum with each layer being elastic, isotropic, and homogeneous. Each layer is to extend to infinity in the horizontal direction and to have, except for the bottom layer, a finite thickness. The applied loads to the pavement are considered as static circular and uniform over the contact area. The program chosen for the analysis is JULEA computer code. This program was chosen because it provided accurate computations and provisions for different degrees of bond between interfaces. Investigations into modeling rigid pavements with this code have resulted in the performance criteria being developed with the assumptions that the interface between the PCC slab and the supporting subgrade is considered smooth with no bond; i.e., there is no frictional resistance at the interface and all other interfaces are considered to be completely bonded. At a depth of 6 meters (20 feet), a very stiff bottom layer is used to mitigate the assumption that the bottom layer extends to infinity. Figure 21-1 presents a diagram for the design of pavements using the layered elastic analysis.

3.DESI GN PROCEDURE. Design of rigid pavements using the elastic layered procedure is initiated by assuming a pavement section. The assumptions are the number of layers, type of materials, and

layer thicknesses. For each material in the assumed section, the modulus of elasticity (E) and Poisson's ratio (μ) are determined. The design flexural strength (R) of the concrete is also determined. The aircraft parameters are defined beginning with the first aircraft (AC_1) in the list of aircraft. The parameters required for the response model are tire contact area, tire loading, number of tires, and tire spacing. Traffic volume is expressed in terms of coverages. The elastic parameters for the materials, the layer thicknesses, and the aircraft parameters for the first aircraft are input into the response model (JULEA computer code) to calculate the tensile stress (σ_1) in the concrete resulting from loading the first aircraft. The computed stress is used along with the concrete design strength to compute a design factor for the first aircraft (DF_1). The design factor is input into the performance model to determine the allowable traffic (N_1) in terms of coverages for the first aircraft on the assumed pavement section. The damage caused by the first aircraft is computed by dividing the applied traffic by the allowable traffic, i.e., n_1/N_1 . The damage caused by the first aircraft is then added to the damage caused by subsequent aircraft. After computing the damage for the first aircraft, the procedure is repeated for other aircraft. After completing the damage computations for all aircraft, the computed cumulative damage is compared with unity. If the assumed section gives a computed cumulative damage substantially different from unity, then a new section is assumed and the procedure repeated for all aircraft. After computing the damage for two sections, a plot of log damage as a function of pavement thickness can be used to estimate the required thickness and used as the assumed section for the next iteration. By updating the plot, the thickness yielding a cumulative damage approximately equal to unity can quickly be established.

4.MAT ERIAL CHARACTERIZATION.

a. Portland-Cement Concrete (PCC).

(1) General. The effects of repeated load on PCC modulus of elasticity are not considered because of the complexity of the relationship between modulus of elasticity and repeated loads and the apparently small magnitude of change caused by traffic. There may be some decrease in modulus because of repeated loads or exposure, but conversely, there should be some increase because of the effects of long-term hydration. The net result is that the computation of the modulus of elasticity from the stress-strain relationship obtained from the initial loading of a PCC specimen is considered adequate for characterizing the material for the life of a pavement.

(a) Poisson's ratio for PCC normally receives very little attention. The range of statically determined Poisson's ratio is only about 0.11 to 0.21, and the average of dynamically determined values was about 0.24. Added factors are the difficulty of measurement and relatively small influence that varying Poisson's ratio within a reasonable range has on the computed response. No procedures are recommended for determining Poisson's ratio for PCC. It is recommended that a value of 0.15 be used for all PCC.

(b) The magnitude of stress that can be sustained by PCC before cracking is a function of the number of repetitions of the stress. This stress magnitude decreases as the number of stress repetitions increases. The number of stress repetitions of a given magnitude that a material can sustain is dependent on numerous factors, such as age, mix proportions, type of aggregate, rate of loading, range of loading, etc. The most important, however, is the static strength of the material. The stress in the slabs is due primarily to bending, and a flexural test is considered the most appropriate for characterizing PCC.

(2) Modulus of elasticity and flexural strength. The modulus of elasticity E_f and flexural strength R of PCC will be determined from static flexural tests of beams having a cross-sectional area

of 152 by 152 millimeters (6 by 6 inches) with a length long enough to permit testing over a span of 457 millimeters (18 inches). The recommended procedures are widely accepted and extensively used for determining the properties of PCC. The test procedure for determining flexural strength and modulus of elasticity will be determined in accordance with ASTM C 78 and CRD-C 21, respectively. When aggregate larger than the 51-millimeter (2-inch) nominal size is used in the concrete, the mix will be wet-screened over a 51-millimeter (2-inch) square mesh sieve before it is used for casting the beam specimen.

(3) Mix proportioning and control. Proportioning of the concrete mix and control of the concrete for pavement construction will be in accordance with TM 5-822-7/AFM 88-6, Chapter 8. Normally, a design flexural strength at 90-days will be used for pavement thickness determination. Should it be necessary to use the pavements at an earlier age, consideration should be given to the use of a design flexural strength at the earlier age or to the use of high early-strength cement, whichever is more economical.

b. Bound Bases (Subbases).

(1) General. Chemically stabilized materials (portland cement, lime, fly ash, etc.) and bituminous-stabilized materials need to be discussed separately, even though the conclusions regarding inclusion of effects of repeated loading are the same. Due to the viscous and temperature-dependent behavior of the bituminous binder, bituminous-stabilized materials are affected by temperature and rate of loading to a much greater extent than any other component in a pavement structure.

(2) Requirements. Bituminous base materials are designed in accordance with TM 5-822-8/ AFM 88-6, Chapter 9. The design for frost consideration will be in accordance with Chapter 20, herein. Chemically stabilized materials should meet requirements set forth in TM 5-822-14/AFM 32-1019. Among these are requirements for durability and the requirement that strength increase with age. These requirements are intended to ensure that the materials continue to function with age and that no adverse chemical reactions occur. However, in terms of ensuring that the material functions as a bound material (sustains flexural loading), it is required that the material attain an unconfined compressive strength of 1.7 MPa (250 psi) at 28 days. This requirement should be used in lieu of strength requirements in the above references. Chemically treated soils in which no substantial increase in strength is considered are modified soils and should be characterized using the methods presented herein for unbound base, subbase, and subgrade materials. Chemically treated soils having unconfined compressive strengths greater than 1.7 MPa (250 psi) should be tested in accordance with the methods specified for stabilized materials. Pavement designs that result in a nonstabilized (pervious) layer sandwiched between a stabilized or modified soil (impervious) layer and the pavement present the danger of entrapped water with subsequent instability in the nonstabilized layer. These designs will not be used unless the nonstabilized layer is positively drained, and its use on Air Force bases will require the approval of the appropriate Air Force Major Command.

(3) Modulus and Poisson's ratio. The modulus of elasticity E_f of bound base material will be determined from cyclic flexural tests of beams. The recommended test procedures have not been standardized but are described in Appendix J. There are differences in the procedures for chemically stabilized materials and those stabilized with bituminous binders. These differences are necessary because of the sensitivity of bituminous-stabilized bases to rates of loading and temperature.

(a) A simply supported unconfined beam loaded at the third point with essentially point loads will be used for bound bases (subbases). For chemically stabilized bound bases, the ultimate load is first determined. Loads of 0.4, 0.6, and 0.8 times the ultimate load are applied repetitively, and the

modulus is computed from the load-deflection curves. The modulus used should be the average obtained for the three loadings. For bituminous-stabilized materials, the definition of an ultimate load will be dependent on the rate of application of load and the temperature. Several loads should be selected that will result in stresses in the outer fibers of the beam, which are less than the values shown in Table 19-1. One test should be conducted at about 0.34 MPa (50 psi).

Table 19-1
Recommended Maximum Stress Levels to Test Bituminous-Stabilized Materials

Temperature Range, °C(°F)	Maximum Stress Level in Extreme Fibers, MPa (psi)
4.4-15.5 (40-60)	3.1 (450)
15.5-27 (60-80)	2.1 (300)
27-38 (80-100)	1.4 (200)

(b) An indirect method of obtaining an estimated modulus value for bituminous concrete is presented in detail in Appendix C. The use of this method requires that the ring-and-ball softening point and the penetration of the bitumen as well as the volume concentration of the aggregate and percent air voids of the compacted mixture be determined.

(c) No procedures are provided for determining Poisson's ratio of bound base material. It is recommended that the values in Table 19-2 be used.

Table 19-2
Poisson's Ratio Values for Bound Base Material

Material	Poisson's Ratio
Bituminous-stabilized	0.5 for E < 3,447 MPa (500,000 psi)
	0.3 for E > 3,447 MPa (500,000 psi)
Chemically stabilized	0.2

c. Unbound (Granular) Bases (Subbases).

(1) General. Unbound granular materials are extremely difficult to characterize. The state of stress, particularly the confining stress, is the dominating factor in determining load-deformation properties. Repeated loadings also affect the modulus of granular materials. The general pattern noted was that repeated loadings increased the stiffness provided shear failure was not progressing. This implies that the modulus of elasticity is increased.

(2) Material requirement. A complete investigation will be made to determine the source, quantity, and characteristics of available materials. A study should be made to determine the most economical thickness of material for a base course that will meet requirements. The base course may consist of natural materials or processed materials, well-graded and high-stability, as referred to in Chapter 8. All base courses to be placed beneath airfield rigid pavements will conform to the following requirements:

(a) Well-graded course to fine.

(b) Not more than 85 percent passing the 2-millimeter (No. 10) sieve.

(c) Not more than 15 percent passing the 0.075-millimeter (No. 200) sieve.

(d) PI not more than 8 percent.

However, when it is necessary that the base course provide drainage, the requirements set forth in TM 5-820-2/AFM 88-5, Chapter 2, will be followed. When frost penetration is a factor, the requirements set forth in Chapter 20, herein, will be followed.

(3) Compaction requirements. High densities are essential to keep future consolidation to a minimum; however, thin base courses placed on yielding subgrades are difficult to compact to high densities. Therefore, the design density in the base course materials should be as required in Chapters 7 and 8.

(4) Modulus and Poisson's ratio. The modulus values of unbound granular bases (subbases) will be determined from cyclic triaxial tests on prepared samples. The recommended test procedure is outlined in Appendix O. The outputs from the test procedure are measures of modulus of elasticity and Poisson's ratio. Triaxial compression tests should be conducted at confining pressures of 13.8, 34.5, 41.4, and 68.9 KPa (2, 5, 6, and 10 psi). Axial stresses should be applied that result in ratios with confining stresses (σ_1/σ_3) of 13.8, 20.7, 27.6, and 34.5 KPa (2, 3, 4, and 5 psi). Plots of resilient modulus versus first stress invariant ($\sigma_1 + \sigma_2 + \sigma_3$ or $\sigma_x + \sigma_y + \sigma_z$) should be prepared and an average relationship established. From this relationship, a value of resilient modulus at a first-stress invariant of 68.9 KPa (10 psi) should be selected. No well-defined relationships exist for Poisson's ratio. However, plots of Poisson's ratio versus ratio of axial to confining stress (σ_1/σ_3) may be made and representative values selected. The modulus value of granular material may also be estimated from the relationship in a chart in which the modulus is a function of the underlying layer and the layer thickness. The chart and the procedure for use of the chart are given in Appendix I. However, it is recommended that the chart be used in conjunction with test results to determine a representative modulus rather than as the sole method. A Poisson's ratio of 0.3 will be used unless there is a reason to believe that it is significantly different for the material in question.

d. Subgrade Soils.

(1) General. Subgrades may be divided into the general classes of cohesive and cohesionless soils. Repeated loadings affect both cohesive and cohesionless soils. Cohesionless sands, gravels, or sand-gravel combinations will respond much like granular bases or subbase. Cohesive soils are more sensitive to repeated loadings. The resilient modulus of cohesive subgrades generally increases with load repetitions provided the level of stress is lower than that required to initiate shear failure. However, the number of stress repetitions required before a stable condition is reached may be greater than for bound bases, granular bases, or cohesionless subgrades.

(2) Exploration. In all instances, field and laboratory tests will be conducted to determine the classification, moisture-density relations, expansion characteristics, and strength of the subgrade. If stabilization of the subgrade is to be considered, other tests as required by TM 5-822-14/AFM 32-1019, will be made, as well as chemical analysis and clay mineralogy determination. When a subgrade soil that has a chemical stabilizing agent added but does not meet the 1.72-MPa (250-psi) compressive strength requirement, the soil should be characterized with procedures for subgrades and be considered simply as part of the subgrade. The engineer is cautioned that although the elastic layered method requires only the modulus of elasticity and Poisson's ratio of the subgrade, such factors as groundwater, surface water infiltration, soil capillarity, topography, drainage, rainfall, and frost conditions may affect the future support rendered by the prepared subgrade or base course. Experience has shown that the subgrade will reach near saturation, even in semiarid and arid regions, after a pavement has been constructed. If conditions exist that will cause the subgrade soil to be affected adversely by frost action, the subgrade will be treated in accordance with the requirements in Chapter 20. Subgrades and base courses are grouped into three types with respect to behavior during saturation: low plastic soils exhibiting little or no swell, swelling soils, and cohesionless sands and gravels. Special cases of subgrade soil are discussed in Chapter 6.

(3) Modulus and Poisson's ratio. The modulus of elasticity and Poisson's ratio of subgrade soils will be determined from repetitive triaxial tests on undisturbed samples when possible or on samples prepared as close as possible to field conditions when fill is involved. The samples considered should represent the worst anticipated condition in the field. The recommended test procedures are outlined in Appendix K. The procedures are similar to those used for granular base (subbase) materials. There are differences in details of the test procedures and presentation of results for cohesive and cohesionless materials. These differences are necessary because of the sensitivity of cohesive soils to moisture and the differences in the behavior as a function of the state of stress.

(a) For characterizing cohesive materials, the triaxial tests should be conducted at a range of stress conditions. Tests should be conducted at confining stresses of 13.8, 27.6, and 41.4 KPa (2, 4, and 6 psi), and at axial stresses applied that will result in a range of deviator stress from about 13.8 to 110 KPa (2 to 16 psi). From the composite curve, the resilient modulus used to represent the material should be selected at a deviator stress of 34.5 KPa (5 psi). No well-defined relationships exist for Poisson's ratio, but similar plots may be made and a representative value selected.

(b) For cohesionless soils, the confining stress in the triaxial tests should approximate conditions in the subgrade. The minor principal stress in the subgrade is a measure of the confinement. For cohesionless subgrade soils, it is considered appropriate to select properties at minimum values of the first stress invariant and confining stress, since the general trends are applicable for cohesionless subgrade soils, i.e., as the confining stress and the first stress invariant decreases, the resilient modulus decreases.

(c) Basically, the same stresses should be used in the triaxial tests for characterizing cohesionless material as are used for granular bases. Confining pressures of 13.8, 27.6, 41.4, and 68.9 KPa (2, 4, 6, and 10 psi) and axial stresses that result in principal stress ratios (σ_1/σ_3) of 2, 3, 4, and 5 should be applied. From the average relationship of resilient modulus versus first stress invariant, a representative modulus value should be selected at a first stress invariant of 68.9 KPa (10 psi). A representative value of Poisson's ratio should be selected from a composite plot of Poisson's ratio versus principal stress ratio. If test results prove unreliable or are not available, the values of 0.4 for cohesive and 0.3 for cohesionless materials may be used.

(4) Modulus of soil reaction. In Westergaard-type solutions, the modulus of soil reaction k characterizes the foundation support under a rigid pavement. Consequently, the modulus of soil reaction k has been used extensively to define the supporting value of all unbound subgrade and base-course materials and all soils that have been additive-modified (TM 5-822-14/AFM 32-1019). The k value has been determined by the field plate bearing test as described in CRD-C 655. When elastic-layered procedures are used for pavements in which only information on modulus of soil reaction k is available, a correlation between the modulus of elasticity E and modulus of soil reaction k may be employed. Figure 19-2 shows such a correlation for subgrade soils. Figure 19-2 should be used with caution as the correlation was developed based on very limited data.

5.DESI GN CRITERIA.

a. The limiting stress (fatigue) criteria form the backbone of the design of rigid airfield pavements. The criteria provide for a prediction of pavement deterioration in terms of a structural condition index (SCI). The SCI is derived from a pavement condition index (PCI) as presented in ASTM D 5340. The SCI is defined as

$$\text{SCI} = \text{PCI} - \text{All nonload-related deducts}$$

The SCI prediction is based on a relationship between design factor and stress repetitions for initial cracking (SCI = 100) and for complete failure (SCI = 0). It is assumed to be linearly related to the logarithm of coverages between initial cracking and complete failure, which results in the relationship illustrated in Figure 19-3.

b. The thickness of the PCC is so selected that the maximum tensile stress at the bottom of the slab does not exceed the allowable value. The criteria are presented as a relationship between design factor, the SCI, and the logarithm (to the base ten) of coverages by the equations:

$$DF = 0.5234 + 0.3920 \log_{10} (C_o) \tag{19-1}$$

and

$$DF = 0.2967 + 0.3881 \log_{10} (C_f) \tag{19-2}$$

and the design factor is defined as

$$DF = R/\sigma \tag{19-3}$$

where

DF = design factor computed with elastic-layered method

R = concrete slab flexural strength, MPa (psi)

σ = maximum computed tensile stress with elastic-layered model such as JULEA computer program, MPa (psi)

C_o = coverage level at which the SCI begins to decrease from 100

C_f = coverage level at which the SCI becomes 0

SCI = the structural condition index desired at the end of the pavement design life

c. When aircraft passes are given, then the pass-per-coverage ratio for the particular design aircraft will be used to convert passes to coverages. The engineer is cautioned that Equations 19-1 and 19-2 were formulated based on accelerated traffic tests with volumes less than 10,000 coverages. The use of the relationship to design for traffic volume greater than 10,000 coverages, which will frequently be the case for current traffic volumes, will require extrapolation of the linear relationship. The pass-per-coverage ratios for some aircraft are shown in Table 17-1.

6.F ROST CONSIDERATION. Two methods have been developed for determining the thickness design of a pavement in frost areas. One method is to limit subgrade frost penetration and the other is to design the pavement for reduced subgrade strength. The first method is directed specifically to the control of pavement distortion caused by frost heave. It requires a sufficient thickness of pavement, base, and subbase to limit the penetration of frost into the frost-susceptible subgrade to an acceptable amount. Complete frost penetration prevention is nearly always uneconomical and unnecessary except in regions with a low design freezing index or where the pavement is designed for heavy-load aircraft. When the rigid airfield pavement is designed by the reduced subgrade strength method, a minimum thickness of 102 millimeters (4 inches) of granular unbound base will be used. A mechanistic procedure for seasonal frost is being developed. Until it is available, the method in Chapter 20 should be used.

7.ALT ERNATE OVERLAY DESIGN PROCEDURE. A methodology for the design of rigid overlays of rigid pavements has been developed that predicts pavement structural deterioration from load induced stresses. The performance of the pavement is expressed in terms of an SCI which relates the type, degree, and severity of pavement cracking and spalling on a scale from 0 to 100. The design methodology for rigid overlays uses the layered-elastic analytical model and the analysis of fatigue cracking in the base slab to predict rigid overlay deterioration in terms of an SCI. Because the methodology predicts performance, an accurate characterization of the materials, structural pavement condition, and fatigue are required. The steps for designing rigid overlays of rigid pavements are illustrated in Figure 19-1 and are implemented in the LEDRRO group of programs.

a. Material Properties. Each layer of the pavement must be described by a modulus of elasticity and a Poisson's ratio.

(1) The modulus value for the concrete can be determined in the laboratory or conservatively estimated as 27,576 MPa (4,000,000 psi).

(2) Modulus values for subgrade soils are often estimated from correlations with existing tests.

(3) Flexural strength of concrete overlays should be determined as part of the mixture proportioning studies. The flexural strength of the base slab may be determined from historical data, flexural beams cut from the base pavement, or approximate correlations between flexural strength and tests run on cores taken from the base pavement.

(4) The interface condition between layers also needs to be determined. The condition of the base slab at the time of the overlay determines the bonding condition used for the overlay. In general, the interface between concrete and other materials is considered to be frictionless. A frictionless interface may be attained by providing a bond breaker course between the overlay and the base pavement. If special effort is taken to prepare the surface for complete bonding, then the interface is considered to be fully bonded.

b. Base Slab Pavement Fatigue and Structural Condition. Traffic applied on the base slab before the overlay is placed consumes some of its fatigue life. If it has begun to deteriorate from traffic, an SCI can be determined from a pavement condition survey. The ratio between the effective modulus of elasticity (E_e) and the initial undamaged modulus of elasticity (E_i) is determined by the relationship:

$$R_E = \frac{E_e}{E_i} = 0.02 + 0.0064 * SCI + (0.00584 * SCI)^2 \qquad (19\text{-}4)$$

This equation is used to account for the deterioration of the base pavement with the application of traffic. If the SCI of the base pavement is equal to 100, the amount of past traffic must be determined to estimate the remaining fatigue life of the base slab.

c. Selection of Trial Thickness. The rigid overlay design procedure is an iterative process. A trial overlay thickness is assumed, and its condition assessed in terms of the overlay life predicted for the design SCI. If the predicted life is unacceptably low, then a thicker overlay thickness is assumed. If the initial trial overlay thickness predicts a pavement life that is too high, then a thinner overlay is tried.

d. Base Slab Performance. The base pavement performance curve is determined by calculating the damage rate at the time of initial cracking (DR_o) and the damage rate at the time of complete failure (DR_f). Equations 19-1, 19-2, and 19-3 in conjunction with the following equations are used to compute the damage rates.

$$DR_o = \sum_{i=1}^{nac} \frac{(C_r)_i}{C_o} \quad and \quad DR_f = \sum_{i=1}^{nac} \frac{(C_r)_i}{C_f} \qquad (19\text{-}5)$$

$$t_o = \frac{B_o}{DR_o} \quad and \quad t_f = \frac{B_f}{DR_f} \qquad (19\text{-}6)$$

where

C_r = design traffic rate, coverages per year

C_o = allowable coverage level at the time of initial cracking (SCI begins to decrease from 100)

C_f = allowable coverage level at the time of complete failure (SCI = 0)

nac = number of aircraft

t_o, t_f = time to initial cracking and time to complete failure, respectively

B_o, B_f = remaining life of base pavement to initial cracking and complete failure, respectively. The remaining life may be estimated from PCI surveys or by computing the damage caused by applied traffic before overlay. ($B_o = 1 - \sum C_i/C_o$ and $B_f = 1 - \sum C_i/C_f$)
C_i = applied past traffic, coverages

e. Time Periods. The base pavement performance curve (with the overlay in place) is divided into time periods so that the variation of the base slab support with time can be determined. The first time period is up to the base slab t_o. The last time period is the time past the t_f. If some traffic has been applied before overlay, the fatigue life consumed must be subtracted from t_o and t_f because this damage has already occurred. To calculate the stresses in the overlay, Equation 19-4 is used to determine the varying base slab support for each of the time periods. If the base slab has begun to deteriorate before the overlay is placed (SCI is less than 100), the base SCI value at the time of the overlay determines the initial support condition. If the time to initial cracking computed exceeds t_o, the time to initial cracking can be set to t_o. Doing so is equivalent to assuming that the base pavement will start to deteriorate with the first coverage of traffic on the overlay. Figure 19-4 illustrates the performance curve for the base slab subdivided into five time periods.

f. Overlay Performance Curve. Once the base pavement performance curve is established, the damage is computed and accumulated for each time period. The damage for a time period is computed as:

$$(d_o)_j = \Delta T_j * \sum_{i=1}^{nac} \frac{C_{ij}}{(C_o)_j} \quad \text{and} \quad (d_f)_j = \Delta T_j * \sum_{i=1}^{nac} \frac{C_{ij}}{(C_f)_j} \tag{19-7}$$

where

$(d_o)_j$ = damage to initial cracking for time period j

$(d_f)_j$ = damage to complete failure for time period j

$(C_o)_j$ and $(C_f)_j$ = a function of the changing modulus of elasticity of the base slab in each time period whereas $j\Delta T_j$ is the magnitude of the time interval in years.

By plotting the cumulative damage versus time in years, the time to initial cracking and complete failure for the overlay can be established. These times correspond to the times when the cumulative damage reaches a value of one. From these time values, a plot of SCI versus logarithm of time (performance curve) then indicates how long the trial thickness will last for the selected design aircraft, traffic rate, and design SCI at the end of the composite overlay pavement design life. Figure 19-5 illustrates the composite overlay performance. If the life of the overlay for the trial overlay thickness is not adequate, a new overlay thickness is assumed and the process is repeated. If several overlay thicknesses are assumed, then a plot of thickness versus logarithm of time, like the one shown in Figure 19-6, can be generated for the selected design SCI, and the design overlay thickness can be chosen.

8.REI NFORCED CONCRETE. Limited full-scale accelerated traffic test data are available for the design of reinforced concrete pavements. The test tracks contained reinforced test sections of varying thickness and percentages of reinforcement. Comparisons were made between the performance of plain and reinforced pavements. The improvements in performance were related to the amount of steel in the concrete slabs. The basis for the comparison was the thickness of unreinforced pavement. The

established criteria for the design of reinforced pavements is shown in Figure 19-7. Assuming that the proposed elastic layer design procedure can result in adequate thicknesses of unreinforced pavement, application of the criterion illustrated in Figure 19-7 will result in adequate thicknesses of reinforced pavements.

9.DESI GN EXAMPLES. Design examples are given illustrating various layer elastic design procedures. The first example illustrates the procedure for selecting a concrete thickness for an airfield designed for a single aircraft. This design example considers the cases of unreinforced concrete slabs. The second example is for an airfield subject to mixed traffic. Overlay designs are given in the last example. The designed concrete pavements are for a type A or primary traffic area. The steps in designing a rigid pavement using the elastic layered method are to establish input data, compute critical stresses, and complete final design.

a. Input Data Required for the Design.

(1) Modulus values and Poisson's ratios of the PCC, bonded and nonbonded granular materials, and subgrade soil. For the purpose of this design example, the following values are assumed in the computation.

E_{PCC} and μ_{PCC} = 27,580 MPa (4,000,000 psi) and 0.2, respectively

E_{bound} and μ_{bound} = 1,034 MPa (150,000 psi) and 0.2, respectively (stabilized base)

$E_{unbound}$ and $\mu_{unbound}$ = 207 MPa (30,000 psi) and 0.3, respectively (granular base)

$E_{subgrade}$ and $\mu_{subgrade}$ = 42 MPa (6,000 psi) and 0.4, respectively

(2) Flexural strength of the PCC. A value of 4.48 MPa (650 psi) is assumed.

(3) Aircraft parameters. The characteristics of the design aircraft and other traffic data required in design are wheel load, number and spacing of wheels in an assembly, tire contact pressure, design life, design traffic, design coverage level, and pass-to-coverage ratio for the particular aircraft.

(4) Limiting stress criteria. Equation 19-1 and 19-2 are used to determine the allowable coverages based on the computed critical stresses induced by the design aircraft.

b. Computation of Critical Stresses. The critical tensile stress in the trial concrete section is computed using the JULEA elastic layered model based on the design aircraft loading and the material properties of each component layer. The interface conditions between layers are such that frictional constraints do not exist between the PCC slab and the base layer and that frictional constraint is developed between the base layer and subgrade soil. Several concrete trial sections are needed for each design. A nearly optimum concrete slab thickness should first be selected, and concrete thicknesses less and greater than the optimum value are then selected. Computations should also be made for different thickness of base-course materials.

c. Final Design. The accumulated damage for each trial concrete section is computed based on the design and the allowable coverages. The final concrete slab thickness is selected as that thickness having an accumulated damage of 1.

10.DESI GN EXAMPLE 1, SINGLE AIRCRAFT.

a. Plain Concrete. This design example is for an airfield taxiway supporting the C-130 aircraft. The design loading for the C-130 on the taxiway is 70,300 kilograms (155,000 pounds). The design is for the C-130 aircraft having a single tandem gear with a tire spacing of 1.5 meters (60 inches) c-c, a tire load of 15,820 kilograms (34,875 pounds), a tire contact area of 0.258 m^2 (400 $in.^2$), a design traffic of 200,000 passes and a pass to coverage ratio of 4.40. For this example an SCI of 80 is desired at the end of the design life.

(1) Computations of critical stresses and damages. Several trial concrete slab thicknesses, i.e., 330, 356, 380, and 405 millimeters (13, 14, 15, and 16 inches) and two thicknesses of granular base and stabilized base, i.e., 15 and 457 millimeters (6 and 18 inches), were selected for design. The maximum tensile stresses in each concrete slab were computed using the elastic layered model JULEA. Equations 19-1, 19-2, and 19-3 were then used to calculate the allowable coverages based on the calculated stresses and the 4.48-MPa (650-psi) flexural strength of the PCC. The amount of damage is the ratio of the design passes to the allowable passes. The computed values, together with other pertinent pavement information, are presented in Tables 19-3 and 19-4 for different base materials. As an illustration, the determination of values shown in the first line of Table 19-3 is explained. For a pavement with 330-millimeter (13-inch) PCC and a 15-millimeter (6-inch) base, the maximum stress under the C-130 aircraft using the computer program JULEA is 2.36 MPa (343 psi). Since an SCI = 80 is desired at the end of the design life, the allowable pass level should be determined from the linear variation between initial cracking (C_o) and complete failure (C_f) (Figure 19-3). From Equation 19-1, the log C_o = 3.50, and from Equation 19-2, the log C_f = 4.12. Interpolating for an SCI = 80, a coverage level of 4,248 is obtained. The allowable pass level is computed as 4,248 * 4.40 = 18,691. The damage is calculated as the ratio of 200,000 and 18,691, i.e., 200,000/18,691 = 10.7.

(2) Selection of Concrete Thickness. The results between PCC thickness and damage presented in Table 19-3 for granular bases and in Table 19-4 for stabilized bases are plotted in Figure 19-8. The required PCC thicknesses are determined at a damage of 1. The required concrete thicknesses are 373 millimeters (14.7 inches) and 378 millimeters (14.9 inches) for granular bases of 457 millimeters (18 inches) and 152 millimeters (6 inches), respectively, and are 358 millimeters (14.1 inches) and 373 millimeters (14.7 inches) for stabilized bases of 457 millimeters (18 inches) and 152 millimeters (6 inches), respectively (thicknesses will be rounded to the nearest 10 millimeters (½ inches) for construction). Figure 19-8 shows that in the case of granular base, the increase of the base thickness from 152 to 457 millimeters (6 to 18 inches) reduces the PCC only 5 millimeters (2/10 inch). In the case of the stabilized base, the increase of the base thickness from 152 to 457 millimeters (6 to 18 inches) can reduce 13 millimeters (½ inch) of PCC. However, an economical comparison should be made between the 13-millimeter (½-inch) reduction in PCC and the 305-millimeter (12-inch) additional stabilized base to determine the final design.

b. Reinforced Concrete. For reinforced concrete pavements, the increase in effective slab thickness due to the presence of the steel in the pavement can be determined from the relationship shown in Figure 19-7. For example, if 0.10 percent reinforcing steel is used for the particular concrete thickness of 381 millimeters (15.0 inches), which was computed in the previous example (see Figure 19-8 for the case of a 152-millimeter (6-inch) base), the relationship shown in Figure 19-7 indicates that the slab thickness can be reduced to 381 millimeters × 0.9 = 343 millimeters (15 inches × 0.9 = 13.5 inches).

c. Frost Action. When frost action needs to be considered in the design, it should first be determined if the subgrade soil is frost susceptible. A description of frost susceptible soils is given in Chapter 20. The depth of frost penetration in the region shall be determined to check if the frost action is

deep enough to weaken the subgrade soil. When this is the case, the reduced subgrade strength method shall be used for design. The procedures to determine the PCC thickness using the elastic layered model are then applied in the same manner as in the first part of this example; the only input parameter change is the (reduced) subgrade elastic modulus. To check for a lesser thickness requirement with limited frost penetration procedures, the criteria in Chapter 20 should be used.

11. DESIGN EXAMPLE 2, MIXED AIRCRAFT TRAFFIC. This design example is for type A traffic areas. The airfield has traffic of 12,500 passes of C-141 (156,500 kilograms (345,000 pounds)), 100 passes of B-52 (181,450 kilograms (400,000 pounds)), and 25,000 passes of F-15 (30,850 kilograms (68,000 pounds)), and 12,500 passes of C-17 (1,179,400 kilograms (260,000 pounds)). The characteristics of the design aircraft are presented in Table 19-5. A 250-millimeter (10-inch) thick stabilized base layer is used in this example. An SCI = 80 is desired at the end of the design life.

a. Computations of Critical Stresses and Damage. A number of trial concrete slab thicknesses were selected for design. The maximum tensile stress in each concrete slab under each aircraft loading was computed using the elastic layered model. Equations 19-1, 19-2, and 19-3 were used to calculate the allowable coverages based on the calculated stresses and the flexural strength of 4.48 MPa (650 psi) for the PCC following the same procedure outlined in example 1. The amount of damage is the ratio of the design or applied passes to the allowable passes. The computed damage for different PCC

Table 19-3
Computation of Cumulative Damage for Selected Pavement Sections, Granular Bases, and C-130 Aircraft

PCC Thickness, in. (1)	Base Thickness, in. (2)	Maximum Stress, psi (3)	Design Passes (4)	Allowable Passes[1] (5)	Damage (6) = (4)/(5)
13	6	343	200,000	18,691	10.7
14	6	309	200,000	64,299	3.11
15	6	280	200,000	232,481	0.86
16	6	255	200,000	894,250	0.22
13	18	336	200,000	23,888	8.37
14	18	303	200,000	80,842	2.47
15	18	275	200,000	290,452	0.69
16	18	251	200,000	1,111,962	0.18

[1] Allowable passes are computed using equations 19-1, 19-2, and 19-3 based on the computed maximum stress shown in column (3), on a selected value of R, and the pass-to-coverage ratio for the C-130 aircraft.
Conversion Factors: Millimeters = 25.4 × inches, Megapascals = 0.006894 × psi

Table 19-4
Computation of Cumulative Damage for Selected Pavement Sections, Stabilized Bases, and C-130 Aircraft

PCC Thickness, in. (1)	Base Thickness, in. (2)	Maximum Stress, psi (3)	Design Passes (4)	Allowable Passes[1] (5)	Damage (6) = (4)/(5)
13	6	338	200,000	22,008	9.09
14	6	305	200,000	75,689	2.64
15	6	276	200,000	276,198	0.72
16	6	252	200,000	1,073,564	0.19
13	18	313	200,000	54,498	3.67
14	18	285	200,000	179,847	1.11
15	18	261	200,000	635,265	0.31
16	18	239	200,000	2,403,285	0.08

[1] Allowable passes are computed using equations 19-1, 19-2, and 19-3 based on the computed maximum stress shown in column (3), on a selected value of R, and the pass-to-coverage ratio for the C-130 aircraft.
Conversion Factors: Millimeters = 25.4 × inches, Megapascals = 0.006894 × psi

Table 19-5
Characteristics of Design Aircraft

Aircraft	Pass-to-Coverage Ratio	Gear Type	Wheel Spacing, m (in.)	Wheel Load, kg (lb)	Tire Contact Area, sq m (sq in.)
C-141	3.50	Twin-Tandem	0.83×1.22 (32.5×48)	17,605 (38,812)	0.134 (208)
B-52	1.58	Twin-Twin Bicycle	0.94×1.57×0.94 (37×62×37)	23,590 (52,000)	0.172 (267)
F-15	9.34	Single	N/A	13,880 (30,600)	0.06 (87)
C-17	1.37	Triple-Tandem	(43 x 43) x 97	19,700 (43,300)	0.242 (314)

thicknesses under each of the four different aircraft are presented in Table 19-6. The total damage induced by the mixed traffic for different PCC thicknesses are tabulated in Table 19-7. The total damage is the sum of the damage caused by all the design aircraft.

b. Selection of Concrete Thickness. The results between the PCC thickness and damage presented in Table 19-7 are plotted in Figure 19-9, and the required slab thickness corresponding to a damage of 1 is determined as 470 millimeters (18.5 inches). Results in Table 19-7 indicate that for

Table 19-6
Computation of Cumulative Damage for Mixed Traffic, 10-inch Stabilized Base Courses

Aircraft (1)	P/C Ratio (2)	PCC Thickness (in.) (3)	Maximum Tensile Stress, psi (4)	Design Passes (5)	Design Coverages (6)	Allowable Coverages C_o (SCI = 100) (7)	Allowable Coverages C_f (SCI = 0) (8)	Allowable Coverages (SCI = 80) (9)	Damage (10) = (6)/(9)
C-141	3.49	16	358.1	12500	3582	1974	8175	2622	1.37
		18	311.3	12500	3582	9803	41268	13068	0.27
		20	272.7	12500	3582	55633	238314	74422	0.05
		22	240.8	12500	3582	355513	1551544	477351	0.00
		24	217.0	12500	3582	2023491	8986671	2726470	0.00
B-52	1.58	16	480.2	100	63	131	529	173	0.37
		18	417.0	100	63	438	1786	580	0.11
		20	365.3	100	63	1599	6612	2124	0.03
		22	322.4	100	63	6427	26939	8560	0.01
		24	286.7	100	63	28081	119466	37513	0.00
F-15	9.30	16	175.5	25000	2688	1.30E+08	6.01E+10	1.76E+08	0.00
		18	143.0	25000	2688	1.82E+10	8.86E+10	2.50E+10	0.00
		20	118.6	25000	2688	4.43E+12	2.28E+13	6.14E+12	0.00
		22	99.9	25000	2688	1.83E+15	1.00E+16	2.57E+15	0.00
		24	85.2	25000	2688	1.34E+18	7.82E+18	1.91E+18	0.00
C-17	1.37	16	363.0	12500	9124	1709	7069	2270	4.02
		18	321.1	12500	9124	6742	28275	8980	1.02
		20	287.5	12500	9124	27060	115076	36145	0.25
		22	259.7	12500	9124	112128	483711	150206	0.06
		24	235.6	12500	9124	504476	2209407	677870	0.01

Note: Allowable passes are computed using equations 19-1, 19-2, and 19-3 based on the computed maximum stress shown in column 3 based on a selected value of k and the pass-to-coverage ratio for the specific aircraft.
Conversion Factors: Millimeters = 25.4 × inches, Megapascals = 0.006894 × psi

Table 19-7
Relationship Between Cumulative Damage and PCC Thickness for Mixed Traffic

PCC Thickness mm (in.)	Damage				
	C-141	B-52	F-15	C-17	Cumulative
406 (16)	1.37	0.37	0.00	4.02	5.76
457 (18)	0.27	0.11	0.00	1.02	1.40
508 (20)	0.05	0.03	0.00	0.25	0.33
558 (22)	0.00	0.01	0.00	0.06	0.07
610 (24)	0.00	0.00	0.00	0.01	0.01

this mix of traffic, most pavement damage is caused by the 12,500 passes of the C-17 aircraft. Although the B-52 is a very heavy bomber aircraft, the 100 passes used for design at a reduced load cause minor damage to the pavement. Because the F-15 aircraft is very light, its 25,000 passes cause practically no damage to the pavement.

12.D ESIGN EXAMPLE 3, OVERLAY DESIGN. The overlay design example presented is for the airfield pavement illustrated in design example 1. The airfield had a 380-millimeter (15-inch) PCC originally designed for the C-130 aircraft. After several years of service, the airfield is to be upgraded to the mixed traffic presented in example 2. Based on the results of subgrade evaluation and following the design procedures in design example 2, the required PCC thickness without a base layer is computed as 470 millimeters (18.5 inches). The existing pavement is in good condition structurally, and the C factor for Equations 17-2 and 17-3 is equal to 0.75. Also, the flexural strength of the concrete in the existing slab is very close to that of the overlay (therefore, $h_d = h_c$), and the required overlay thicknesses h_o can be computed from Equations 17-2 and 17-3 as follows:

a. Computation for Nonbonded Concrete:

$$h_o = \sqrt{470^2 - 0.75(380)^2} = 335 \text{ millimeters}$$

$$h_o = \sqrt{18.5^2 - 0.75(15)^2} = 13.2 \text{ inches}$$

b. Computation for Partially Bonded Concrete:

$$h_o = \sqrt[1.4]{470^{1.4} - 0.75(380)^{1.4}} = 262 \text{ millimeters in SI units}$$

$$h_o = \sqrt[1.4]{18.5^{1.4} - 0.75(15)^{1.4}} = 10.3 \text{ in English units}$$

13.DESI GN EXAMPLE 4, ALTERNATE OVERLAY DESIGN PROCEDURE. This example is based on predicting structural deterioration from load-induced stresses. A concrete overlay must be designed to support the traffic mix from example 2. The pavement consisted of a 381-millimeter (15-inch) PCC pavement originally designed for the C-130 aircraft. It was considered that the concrete flexural strength of the overlay will be approximately equal to that of the base slab which is 4.48 MPa (650 psi). Only minor cracking of the base slab was observed; therefore, a SCI of 100 was assumed. It was also estimated from pavement condition surveys that 90 percent of the pavement life to reach initial cracking has been consumed. A bond breaker layer will be placed between the base slab and the overlay (unbonded case). The overlay will be designed for a 20-year operating life and an SCI = 80 at the end of its life.

a. Establish Base Slab Performance Curve. The first step in designing concrete overlays using the alternate methodology is to determine the performance of the base slab under the new traffic when the overlay is placed. For this example, overlay trial thicknesses of 305, 356, and 406 mm (12, 14, and 16 in.) are used. For each trial overlay, the tensile stress caused by each aircraft is computed at the bottom of the base slab. From these computed stresses the coverages to initial cracking and complete failure are computed. Table 19-8 presents in detail the necessary data and equations to perform the calculations. Column 1 of Table 19-8 contains the overlay trial thicknesses, column 2 contains the design aircraft, and column 3 contains design traffic data for each aircraft. Since the design is for a 20-year life, the traffic rate for the B-52 would be 100 passes (from example 2) divided by 20 years, or 5 passes per year. Column 4 contains the factor to convert passes to coverages. Column 5 contains the calculated tensile stresses at the bottom of the base slab using the elastic layer computer program JULEA. The coverages for initial cracking and complete failure of the base slab were computed using Equations 19-1 and 19-2. The damage rate (DR_o) is then computed and accumulated in column 7. The sum of the damage rates is used to calculate the time for initial cracking (T_o) in column 8. Similar calculations are performed in columns 9, 10, and 11 to calculate the damage rate for complete failure (DR_f) and the time for complete failure (T_f) of the base slab. These two time values establish the base slab rate of deterioration between the SCI=100 and an SCI=0. Figure 19-10 shows the base slab performance curve for the 356-millimeter (14-inch) overlay. Similar plots are generated for the 305- and the 406-millimeter (12- and the 16-inch) overlay trials. Finally, the time from the placement of the overlay (m_o) to initial cracking of the base slab is computed in column 12. Since 90 percent of the base slab life has been consumed, the remaining life is then 10 percent (B_o = 0.10). For the 356-millimeter (14-inch) overlay, m_o = 0.10/0.0158 = 6.33 years.

b. Subdivide Base Slab Performance Curve. The next step is to subdivide the base slab performance curve for a trial overlay thickness into time intervals. For the 356-millimeter (14-inch) overlay (Figure 19-10), the curve is divided into six intervals. The first interval will be from the time when the overlay is placed (m_o) to initial cracking of the base slab (T_o). This first interval then corresponds to 6.33 years. Between T_o and T_f, the performance curve is then divided into time intervals to account for the deterioration of the base slab with time (the base slab modulus of elasticity decreases). The slope between T_o and T_f represents the rate of deterioration of the base slab when the SCI decreases from 100. To illustrate this, the time between T_o and T_f in Figure 19-10 is divided in four equal time intervals (on a logarithm scale) and the magnitude of each time interval recorded. The last time period corresponds to the time T_f and beyond (SCI=0). Table 19-9 summarizes this procedure and Figure 19-11 illustrates the actual base slab performance curve when the 356-millimeter (14-inch) overlay is in place. The magnitude of these time intervals will be used in the calculation of the cumulative damage in the overlay within a time interval.

Table 19-8
Data for Overlay Design, Example 4 (Base Slab Calculations)

Overlay Thickness (in.) (1)	Design Aircraft (2)	Traffic Rate (Passes/Year) (3)	P/C (4)	Tensile Stress (psi) (5)	C_o (Coverages) (6)	DR_o (Damage/Year) (7)	T_o (Years) (8)	C_f (Coverages) (9)	DR_f (Damage/Year) (10)	T_f (Years) (11)	m_o (Years) (12)
12.00	B-52	5	11.88	423.00	384.42	0.0011		1,566.41	0.0003		
	C-141	625	3.87	324.00	6,061.55	0.0266		25,393.50	0.0064		
	F-15	1,250	356.03	131.00	2.10E+11	0.0000		1.05E+12	0.0000		
	C-17	625	1.44	318.00	7,570.87	0.0573		31,787.40	0.0137		
					Sum DR_o =	0.0851	11.76	Sum DR_f =	0.0203	49.30	1.18
14.00	B-52	5	11.88	365.00	1,613.28	0.0003		6,669.16	0.0001		
	C-141	625	3.87	283.00	33,421.82	0.0048		142,435.96	0.0011		
	F-15	1,250	356.03	110.00	5.48E+13	0.0000		2.89E+14	0.0000		
	C-17	625	1.44	279.00	40,553.98	0.0107		173,167.81	0.0025		
					Sum DR_o =	0.0158	63.31	Sum DR_f =	0.0037	270.03	6.33
16.00	B-52	5	11.88	312.00	9,537.22	0.0000		40,136.42	0.0000		
	C-141	625	3.87	244.00	288,766.63	0.0006		1,257,614.71	0.0001		
	F-15	1,250	356.03	92.00	4.88E+16	0.0000		2.76E+17	0.0000		
	C-17	625	1.44	243.00	307,973.30	0.0014		1,342,130.33	0.0003		
					Sum DR_o =	0.0020	496.64	Sum DR_f =	0.0005	2,163.14	49.68

DEFINITIONS:

Traffic Rate = Number of design aircraft passes per year.
P/C = Pass to Coverage Ratio at offset where the maximum damage occurs.
C_o = Coverage level at which the SCI begins to decrease from 100 (initial cracking).
C_f = Coverage level at which the SCI becomes 0 (complete failure).
DR_o = Damage rate to initial cracking.
DR_f = Damage rate to complete failure.
T_o = Time in years to initial cracking.
T_f = Time in years to completed failure.
m_o = Time from the placement of overlay to initial cracking of the base slab.

B_o = Remaining life of base slab.
(5) = Computed tensile stress at the bottom of the base slab.
(6) = Coverages to initial cracking (C_o) calculated using stress in (5).
(7) = (3)/(6)/(4).
(8) = 1.0/(7) = 1.0/Sum DR_o .
(9) = Coverage to complete failure (C_f) calculated using stress in (5).
(10) = (3)/(9)/(4).
(11) = 1.0/(10) = 1.0/Sum DR_f .
(12) = Bo/(7) = Bo/Sum DR_o .
Conversion Factors:
Millimeters = 25.4 × inches
Megapascals = 0.006894 × psi

Table 19-9
Time Intervals for the 356-millimeter (14-inch) Overlay

Time Interval Number	Time at the Beginning of Each Interval, years	Time Between Intervals, years
1	0.000	
2	6.33	6.33
3	34.00	27.67
4	73.77	39.77
5	130.92	57.15
6	213.05	82.13

c. Compute the Cumulative Damage in the Overlay. Once the base slab performance curve is established, the damage in the trial overlay can be assessed. The procedure basically consists of computing the tensile stresses at the bottom of the overlay, calculating the number of coverages to initial cracking and complete failure, and calculating and cumulating the damage in the overlay for each time period. This process is demonstrated in Table 19-10. Columns 1 and 2 contain the interval number and the magnitude of the interval in years, respectively. Column 3 contains the average SCI's within each interval that is used to compute the reduced modulus of elasticity (Equation 19-4). Columns 4, 5, and 6 contain the design aircraft, traffic rate, and pass-to-coverage ratio. Column 7 contains the computed tensile stresses at the bottom of the overlay. These stresses are computed with the elastic layer computer program JULEA assuming the interface between the overlay and the slab is unbonded since a bond-breaker layer is used. Column 8 contains the number of coverages to initial cracking (Co) of the overlay for each aircraft. The damage (Do) is computed in column 9 and accumulated in column 10 (DAMo). In a similar fashion the damage to complete failure (DAMf) is calculated and accumulated in columns 11, 12, and 13. This process is repeated for each time interval as is shown in the table.

d. Determine Required Overlay Thickness. The cumulative damage for initial cracking (DAMo) and complete failure (DAMf) of the trial overlay for each time interval can now be plotted. Figure 19-12 shows the plot for the 356-millimeter (14-inch) overlay. From this plot, the years to initial cracking and complete failure of the overlay can be obtained by reading the years at which the DAMo and DAMf curves cross a cumulative damage of 1.0. For the 356-millimeter (14-inch) overlay shown in Figure 19-12, these values correspond approximately to 26 years to initial overlay cracking and 50 years to complete failure of the overlay. Similar curves can be generated for the 406-millimeter (16-inch) and 457-millimeter (18-inch) overlay trials. Figure 19-13 summarizes the analysis performed on the 305-, 356-, and 406-millimeter (12-, 14-, and 16-inch) overlay trials. The values obtained from Figure 19-12 are used to generate the composite overlay performance curve. From Figure 19.13, for the case of the 356-millimeter (14-inch) overlay, the overlay performs at an SCI of 100 for 4.0 years before it starts to deteriorate. It then deteriorates linearly with the logarithm of time until it reaches a complete failure condition (SCI=0.0) after 50 years. Finally from Figure 19-13, the life of each overlay trial can be obtained for the design overlay SCI of 80. These values are 4.2 years, 29.6 years, and 81.7 years for the 305-, 356-, 406-millimeter (12-, 14-, and 16-inch) overlays, respectively. To obtain the required thickness for the design life of 20 years, a plot of the overlay thicknesses versus the life of each overlay is generated as illustrated in Figure 19-14. From this figure, a 426-millimeter (16.8-inch) overlay would be required for a design life of 20 years and a SCI of 80.

Table 19-10
Computation of the Cumulative Damage for the 14-inch Overlay

Time Interval Number (1)	Time Interval Years (2)	Average SCI (3)	Design Aircraft P/C (4)	Traffic Rate (Passes/Year) (5)	P/C (6)	Tensile Stress (psi) (7)	C_o (8)	D_o (9)	DAM_o (10)	C_f (11)	D_f (12)	DAM_f (13)
1	6.33	100	B-52	5	11.88	389.00	846.10	0.0011		3,475.13	0.0003	
			C-141	625	3.87	286.00	29,011.46	0.0130		123,464.34	0.0030	
			F-15	1,250	356.03	137.00	58.64E+09	1.3956E-10		2.89E+11	2.8342E-11	
			C-17	625	1.44	292.00	22,051.45	0.0459		93,586.25	0.0108	
							Sum D_o =	0.0600	0.1630	Sum D_f =	0.0141	0.0384
2	27.67	87.5	B-52	5	11.88	416.00	447.47	0.0260		1,826.13	0.0064	
			C-141	625	3.87	306.00	12,123.56	0.3686		51,143.94	0.0874	
			F-15	1,250	356.03	147.00	8.81E+09	1.1032E-08		4.25E+10	2.2834E-09	
			C-17	625	1.44	311.00	9,919.98	1.2107		41,763.72	0.2876	
							Sum D_o =	1.6054	1.7684	Sum D_f =	0.3813	0.4198
3	39.77	62.5	B-52	5	11.88	479.00	133.82	0.1251		539.53	0.0310	
			C-141	625	3.87	352.00	2,374.03	2.7053		9,852.24	0.6519	
			F-15	1,250	356.03	171.00	2.30E+08	6.0714E-07		1.07E+09	1.3036E-07	
			C-17	625	1.44	354.00	2,232.92	7.7298		9,260.91	1.8638	
							Sum D_o =	2.8303	4.5987	Sum D_f =	2.5466	2.9664

DEFINITIONS:

D_o = Damage within a time interval when analyzing for initial cracking.
D_f = Damage within a time interval when analyzing for complete failure.
DAM_o = Cumulative damage up to a time interval when analyzing for initial cracking.
DAM_f = Cumulative damage up to a time interval when analyzing for complete failure.
(7) = Computed tensile stress at the bottom of the overlay layer.
(8) = Coverages to initial cracking (C_o) calculated using stress in (6).
(9) = (2)*(((5)/(8)/(6).
(10) = Cumulative D_o up to that interval.
(11) = Coverages to complete failure (C_f) calculated using stress in (6).
(12) = (2)*(((5)/(11)/(6).
(13) = Cumulative D_f up to that interval.
Conversion Factors:
Millimeters = 25.4 × inches
Megapascals = 0.006894 × psi

(Continued)

Table 19-10 (Concluded)

Time Interval Number (1)	Time Interval Years (2)	Average SCI (3)	Design Aircraft P/C (4)	Traffic Rate (Passes/Year) (5)	P/C (6)	Tensile Stress (psi) (7)	C_o (8)	D_o (9)	DAM_o (10)	C_f (11)	D_f (12)	DAM_f (13)
4	57.14	37.5	B-52	5	11.88	553.00	46.06	0.5223		183.71	0.1309	
			C-141	625	3.87	406.00	560.97	16.4527		2,294.53	4.0224	
			F-15	1,250	356.03	200.00	9.03E+09	2.2224E-05		4.07E+07	4.9294E-06	
			C-17	625	1.44	405.00	574.15	43.2018		2,348.98	10.5596	
							Sum D_o =	16.9750	21.5737	Sum D_f =	14.7130	17.6794
5	82.13	0.0	B-52	5	11.88	639.00	18.18	1.9007		71.86	0.4810	
			C-141	625	3.87	468.00	161.39	82.1834		651.92	20.3456	
			F-15	1,250	356.03	234.00	563,590.57	0.0005		2.47E+06	0.0001	
			C-17	625	1.44	464.00	173.15	205.8681		699.92	50.9293	
							Sum D_o =	84.0847	105.6584	Sum D_f =	71.7559	89.4353

DEFINITIONS:

D_o = Damage within a time interval when analyzing for initial cracking.
D_f = Damage within a time interval when analyzing for complete failure.
DAM_o = Cumulative damage up to a time interval when analyzing for initial cracking.
DAM_f = Cumulative damage up to a time interval when analyzing for complete failure.
(7) = Computed tensile stress at the bottom of the overlay layer.
(8) = Coverages to initial cracking (C_o) calculated using stress in (6).
(9) = (2)*(((5)/(8)/(6).
(10) = Cumulative D_o up to that interval.
(11) = Coverages to complete failure (C_f) calculated using stress in (6).
(12) = (2)*(((5)/(11)/(6).
(13) = Cumulative D_f up to that interval.
Conversion Factors:
Millimeters = 25.4 × inches
Megapascals = 0.006894 × psi

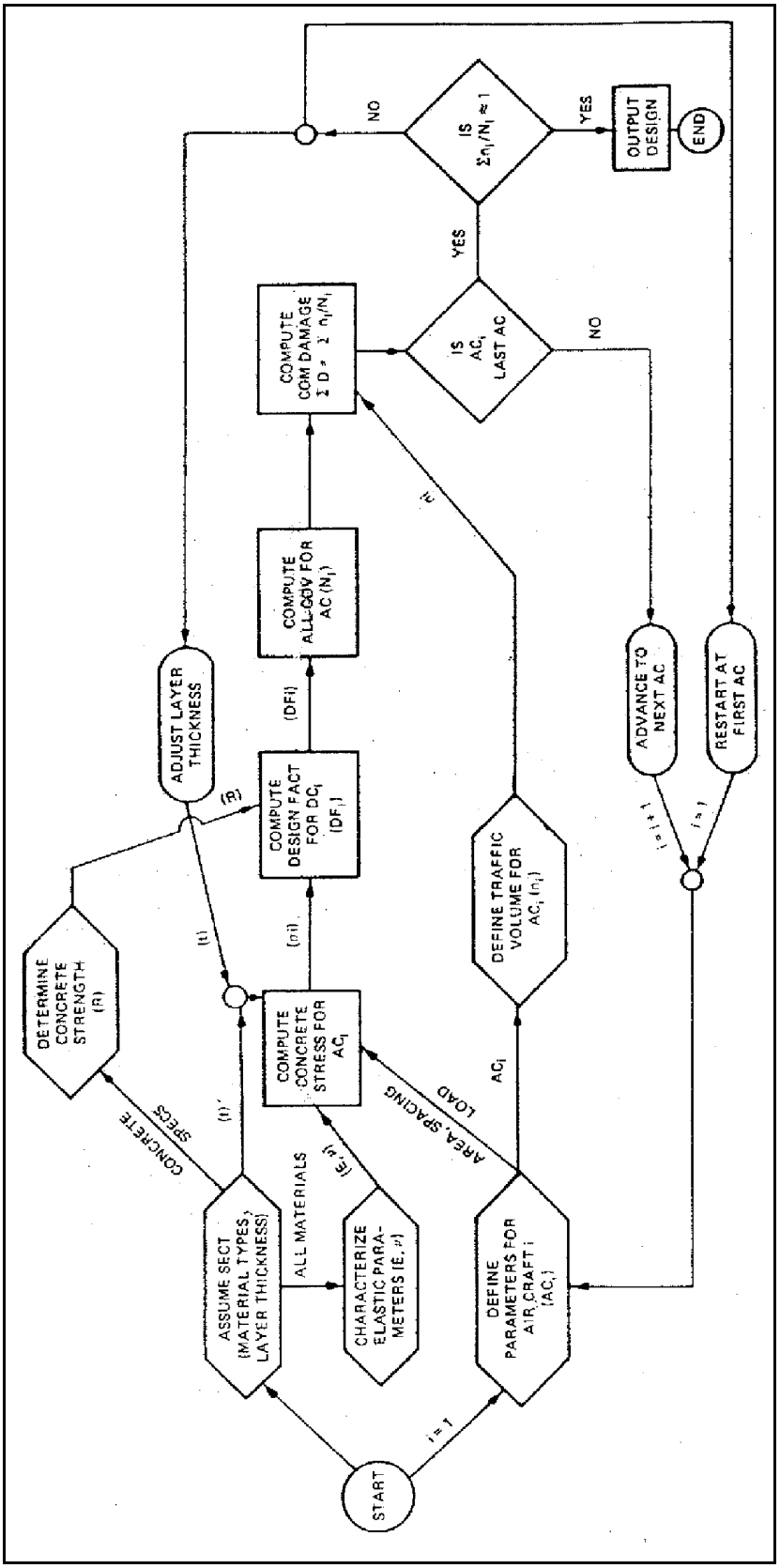


Figure 19-1. Diagram for design of airfield rigid pavements by layered elastic theory

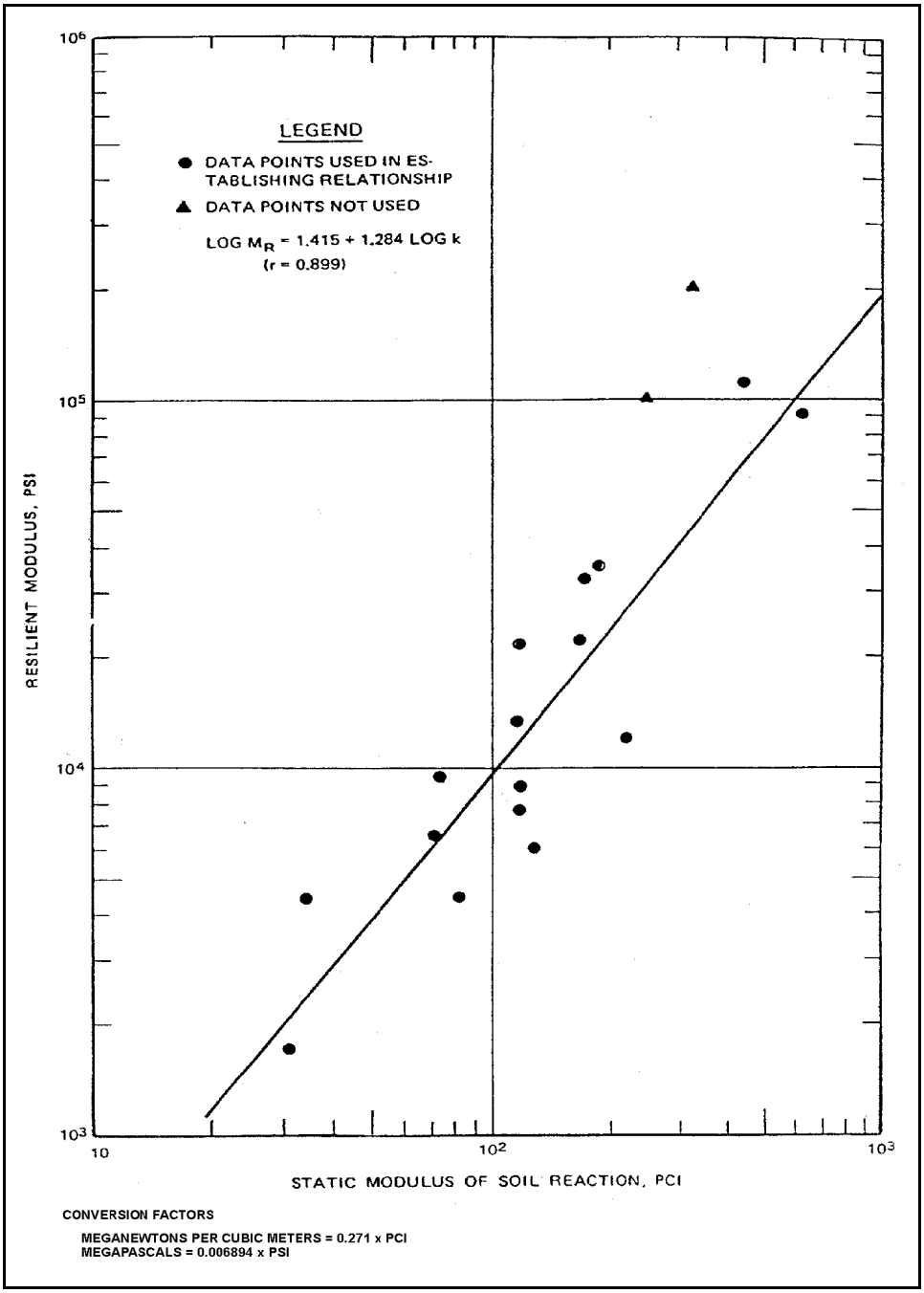


Figure 19-2. Correlation between resilient modulus of elasticity and static modulus of soil reaction

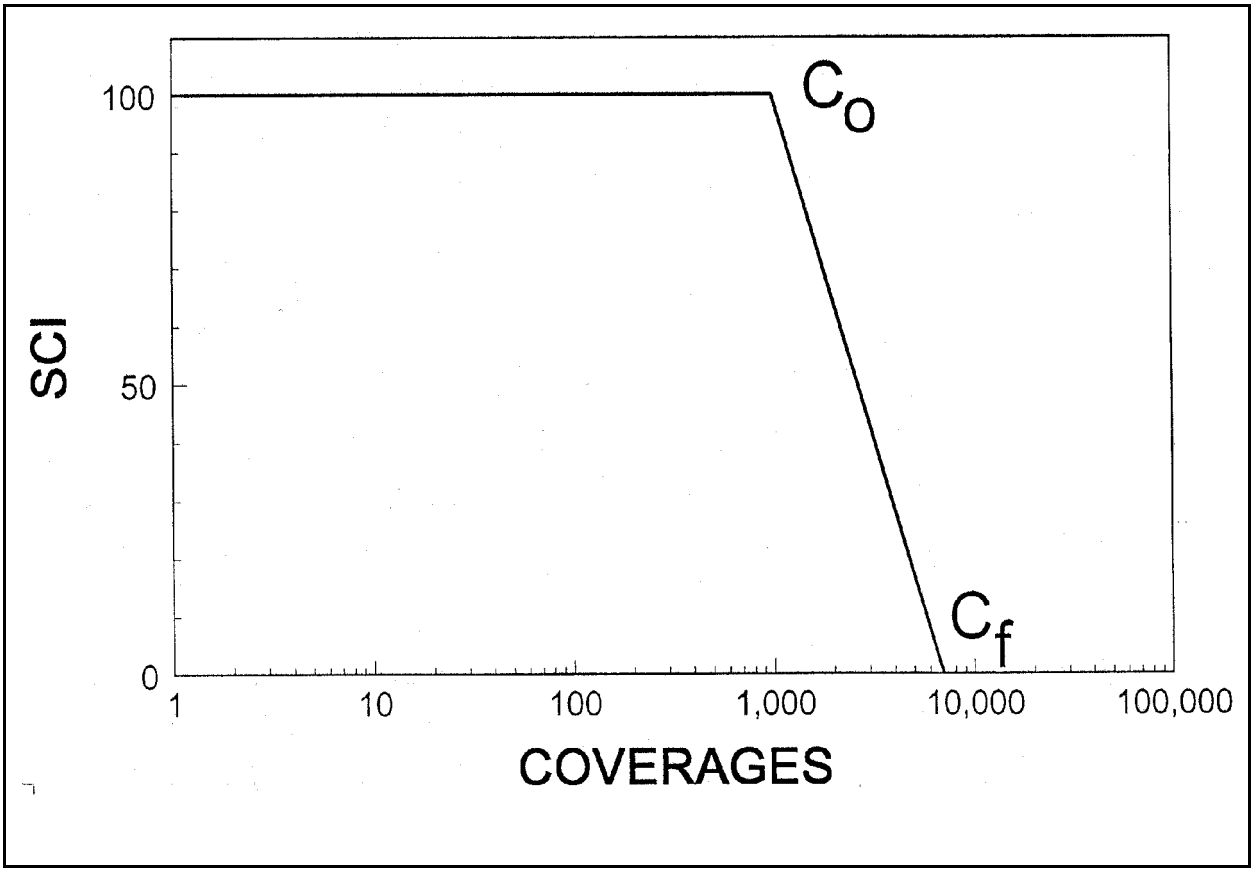


Figure 19-3. Relationship between SCI and coverages at initial cracking and complete failure

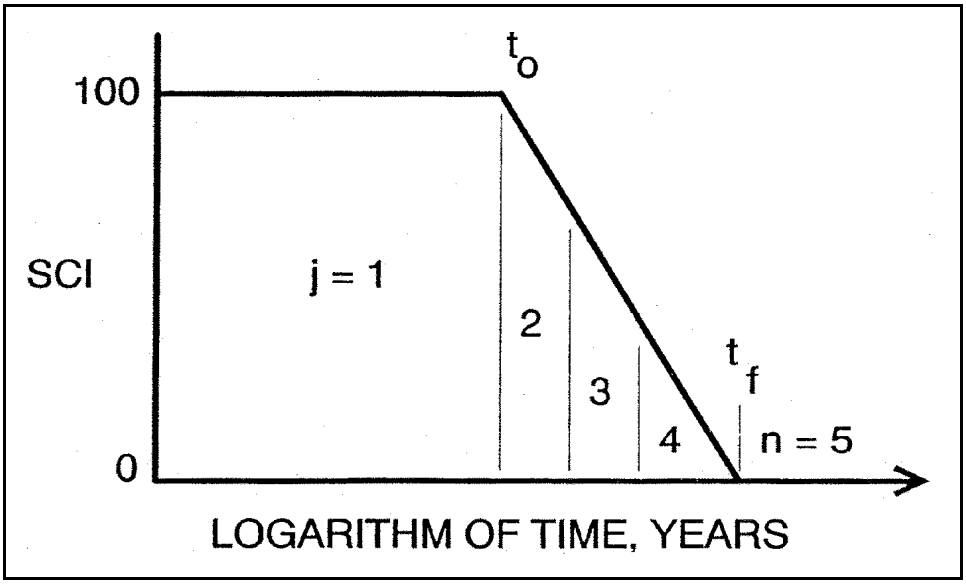


Figure 19-4. Base slab performance curve (SCI versus logarithm of time) subdivided into five time periods

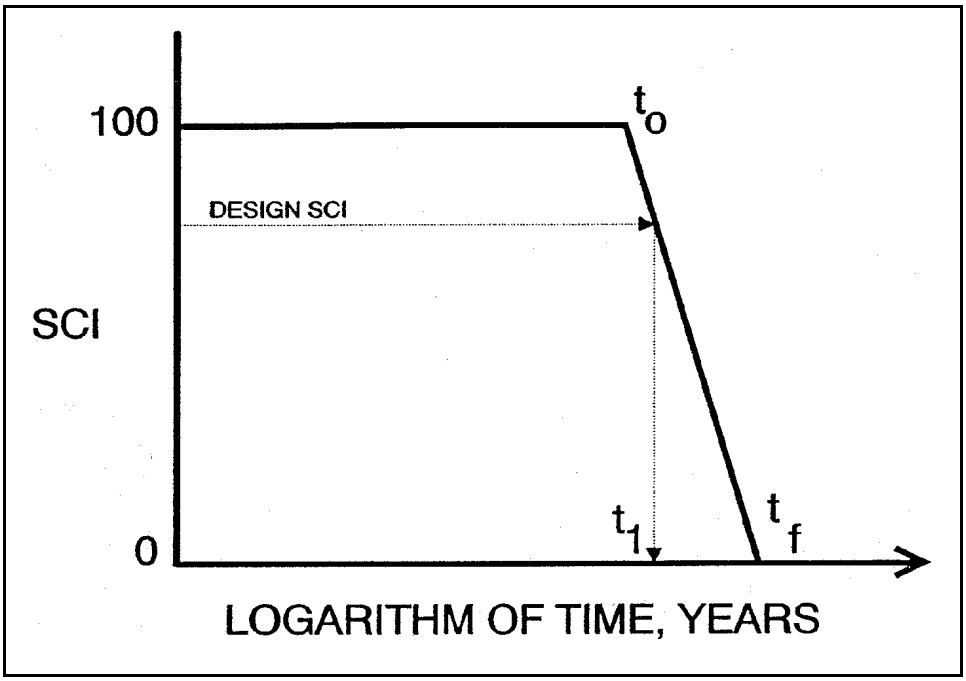


Figure 19-5. Composite overlay performance curve

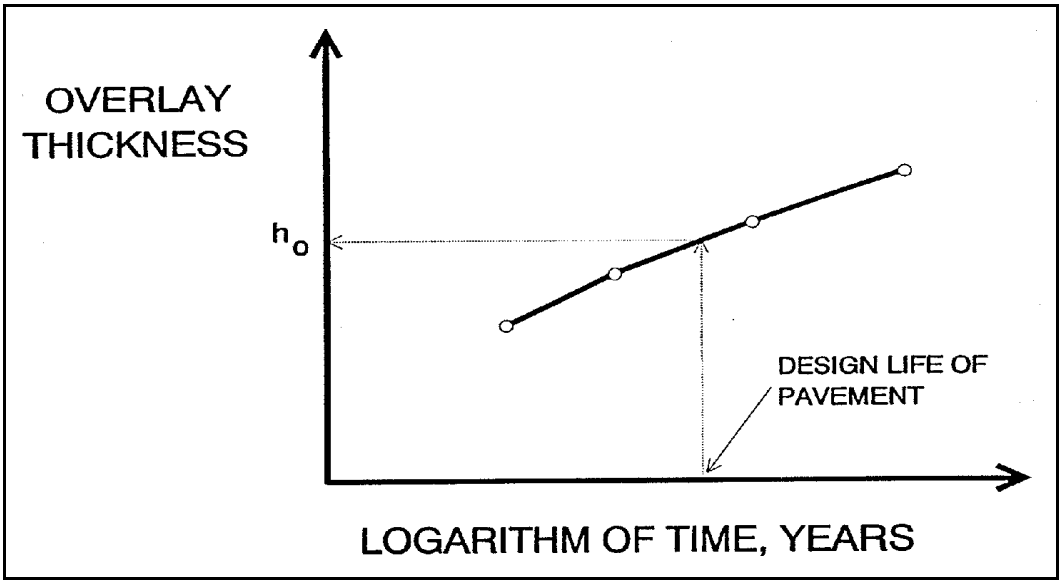


Figure 19-6. Overlay thickness versus logarithm of time

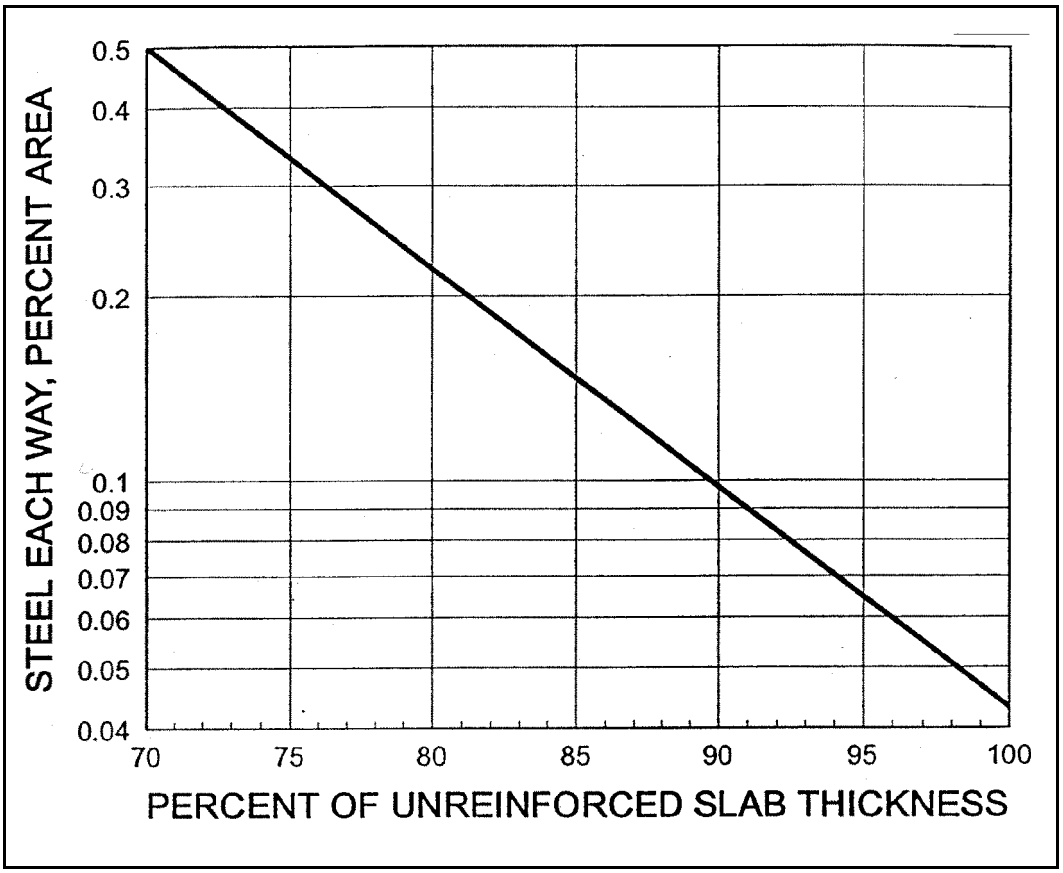


Figure 19-7. Effect of steel reinforcement on rigid pavements

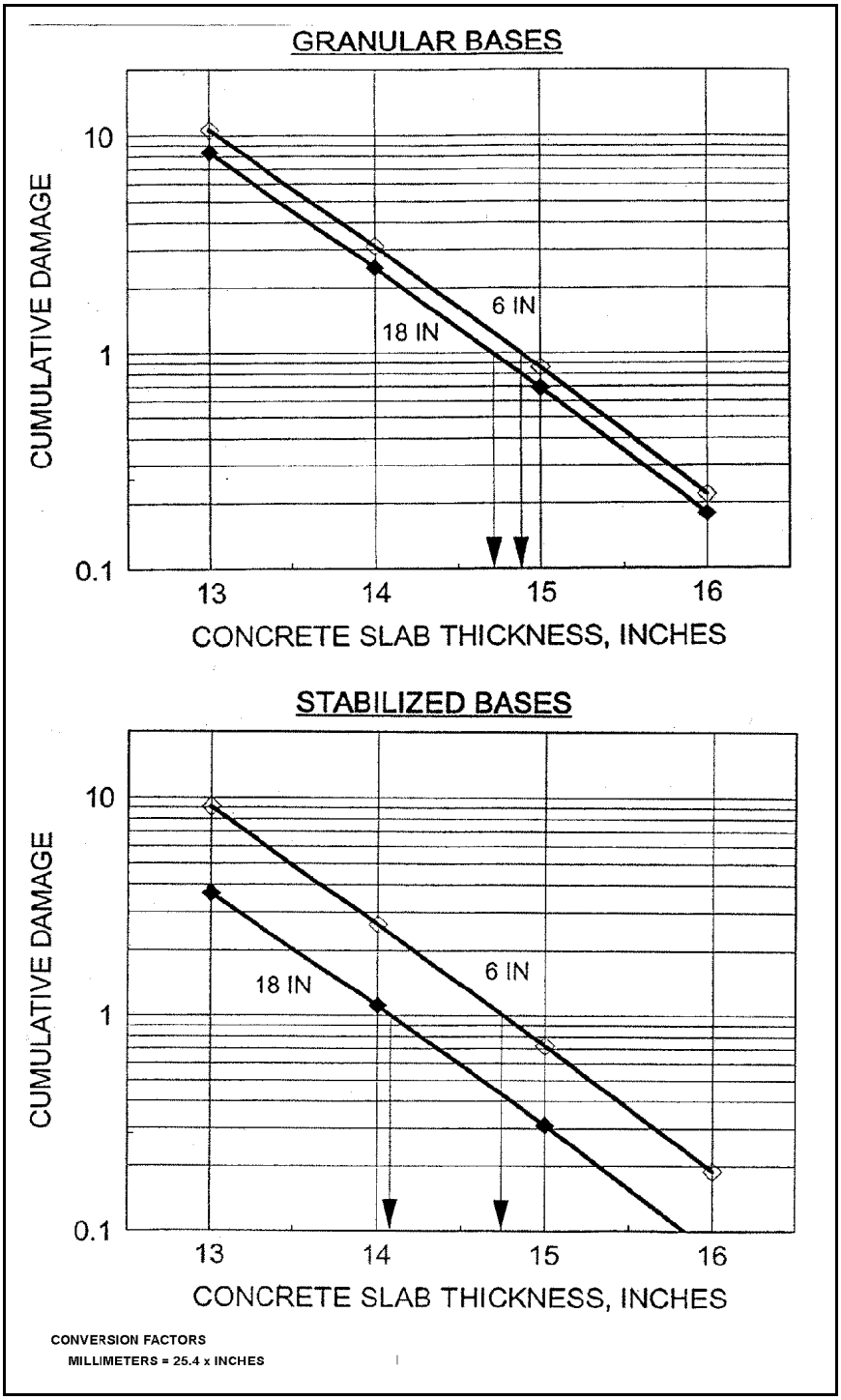


Figure 19-8. Relationship between cumulative damage and concrete slab thickness for granular and stabilized bases (see Design Example 1)

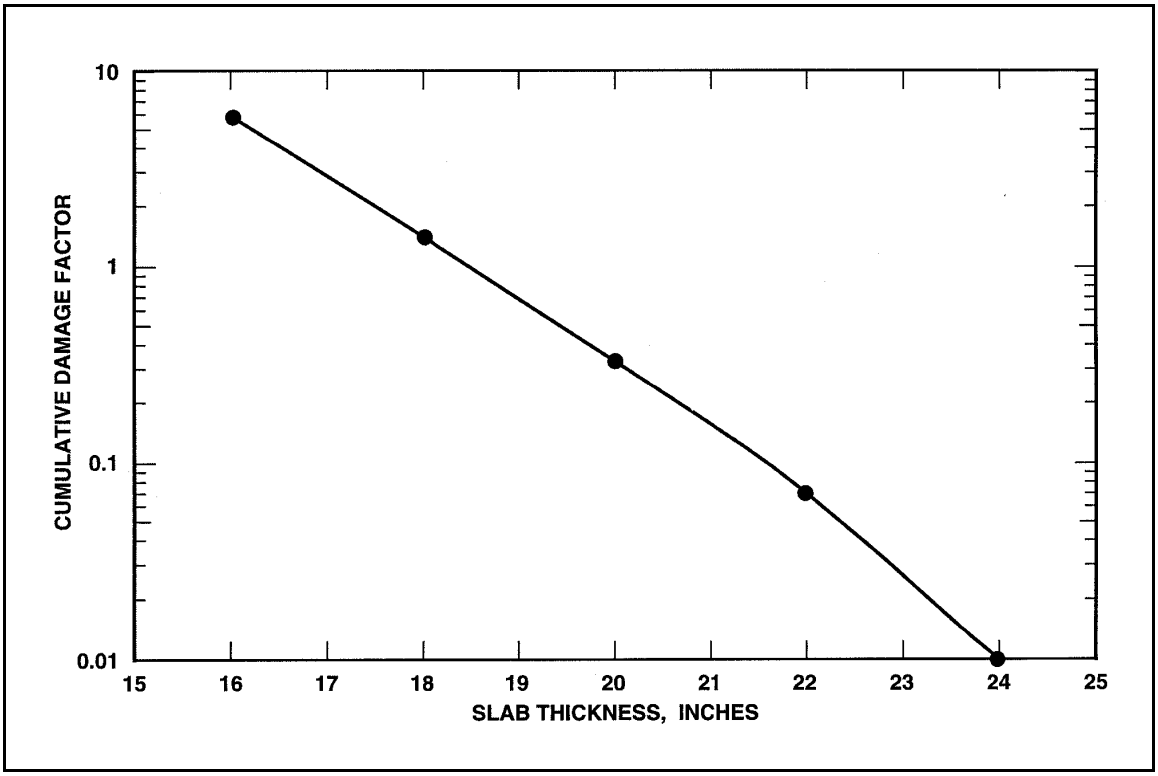


Figure 19-9. Relationship between cumulative damage and pavement thickness, mixed aircraft traffic (see Design Example 2)

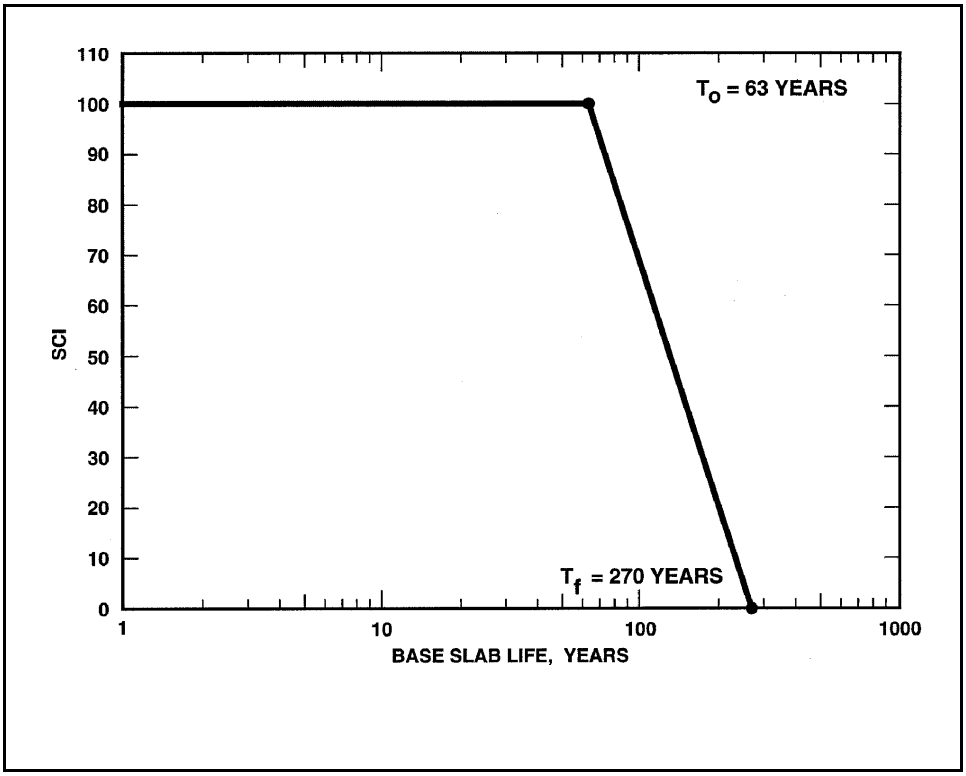


Figure 19-10. Base slab performance curve for the 356-millimeter (14-inch) overlay

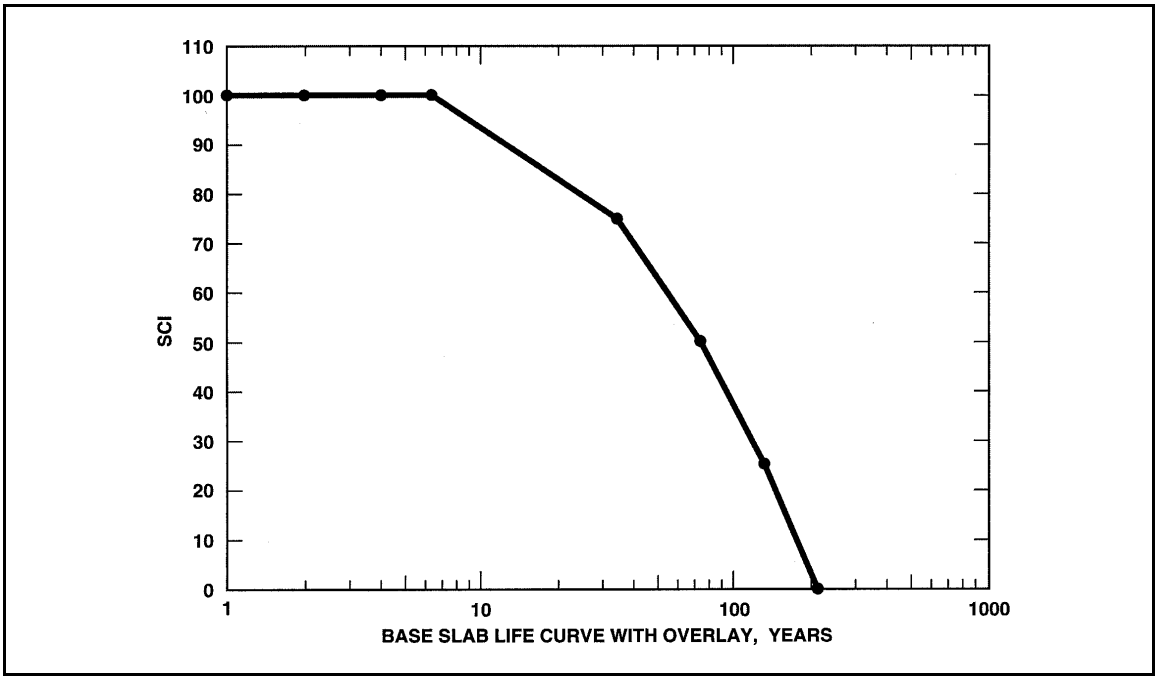


Figure 19-11. Actual performance curve with the 356-millimeter (14-inch) overlay in place

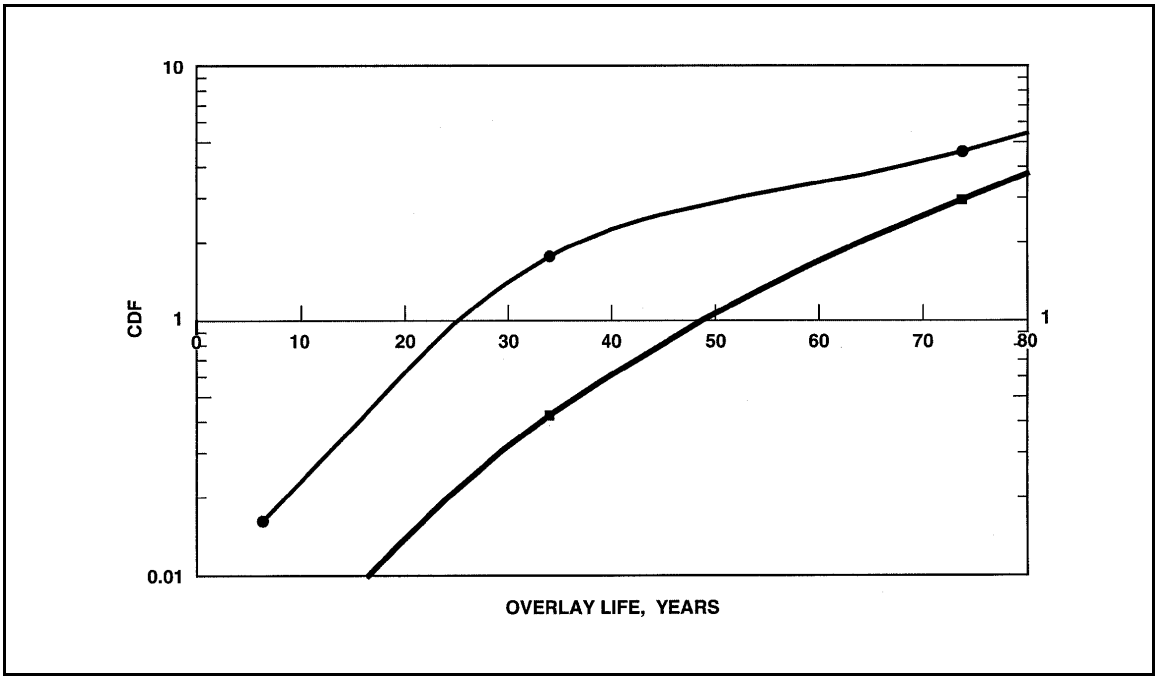


Figure 19-12. Cumulative damage plot for the 356-millimeter (14-inch) overlay

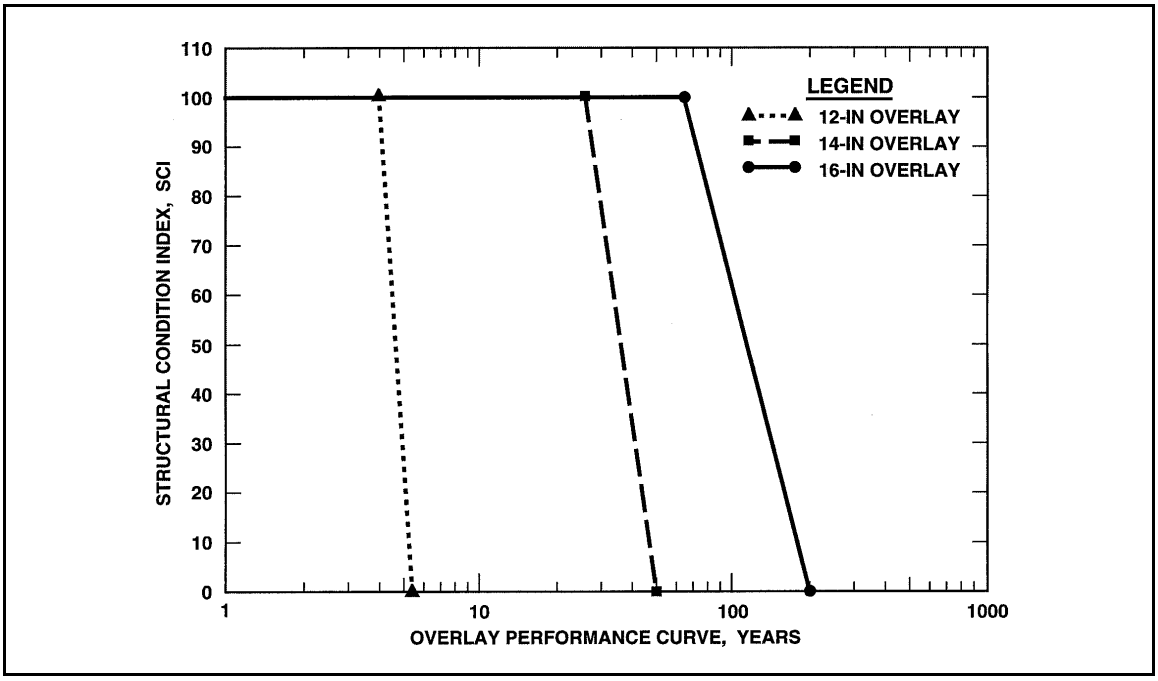


Figure 19-13. Results of the analysis performed on the 305-, 356-, and 406-millimeter (12-, 14-, and 16-inch) overlay trials

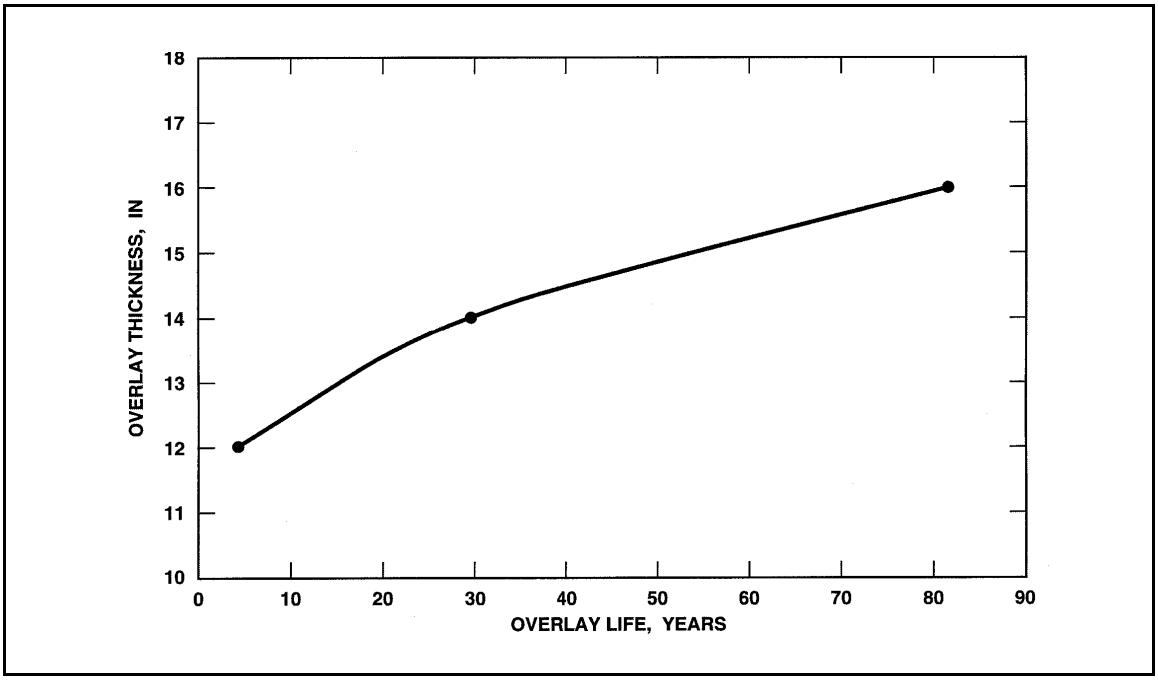


Figure 19-14. Plot showing the overlay thickness versus the overlay life

CHAPTER 20

SEASONAL FROST CONDITIONS

1. GENERAL. This chapter presents criteria and procedures required for the design and construction of pavements placed on subgrade materials subject to seasonal frost action. If frost does not penetrate into the subgrade using thicknesses necessary for nonfrost design, pavement design need not consider effects of frost action unless the base, subbase courses contain other than NFS, PFS, S1, or S2 materials. The designer must select subbase materials which do not allow pumping of subbase course or subgrade fines during periods of saturated or nearly saturated conditions. The detrimental effects of frost action in frost susceptible subsurface materials are manifested by nonuniform heave of pavements during the winter and/or loss of strength of affected soils during the ensuing thaw periods. Studies have shown that the modulus of subgrade reaction is reduced substantially during the thaw period. Application of load on thaw-weakened pavements can lead to premature failure. Other effects of frost on pavements are possible loss of compaction, pumping, increased pavement roughness, restriction of drainage by frozen layers, and cracking of asphalt concrete pavements. In extreme conditions, these problems can cause hazardous operating conditions, or Foreign Object Damage (FOD) to aircraft, and can lead to extensive maintenance of the pavement surface. Except in cases where other criteria are specifically established, pavements should be designed so that there will be no interruptions of traffic at any time due to differential heave or to reduction in load-supporting capacity. Pavements should also be designed so that the rate of deterioration during critical periods of thaw weakening and thermally induced cracking will not be so high that the useful life of the pavement is less than that assumed as the design objectives. For interior pavements which fall within a geographical area subject to subgrade frost action, the “reduced subgrade strength” or the “limited subgrade frost penetration” pavement design criteria should be used for all aircraft hangar pavements in heated or unheated areas.

2. FROST-SUSCEPTIBILITY CLASSIFICATION. For frost design purposes, soils are divided into eight groups as shown in Table 20-1. Soils are listed in approximate order of increasing frost susceptibility and decreasing bearing capacity during periods of thaw.

a. The frost susceptibility of the soils classified in Table 20-1, based on laboratory tests, are shown in Table 20-2. The NFS, S1, and S2 groups are negligible to very low frost susceptible soils. Based on laboratory tests, the heave rates range between 1 and 4 mm/day and the thawed CBR ranges between 12 to 20 percent. These soils are considered to be suitable as base and subbase course material. Soils categorized as F1, F2, F3, and F4 are unsuitable as base or subbase materials.

b. Under special conditions the frost group classifications adopted for design may be permitted to differ from that obtained by application of the above frost group definitions provided a written waiver is obtained and a valid justification is presented in the design analysis. Such justification may take into account special conditions of subgrade moisture or soil uniformity, in addition to soil gradation and plasticity, and should include data on performance of existing pavements near those proposed to be constructed. This will require the approval of HQUSACE (CEMP-ET), the appropriate Air Force Major Command or the Naval Facilities Engineering Command.

3. METHODS OF THICKNESS DESIGN. Three methods are prescribed for determining the thickness of a pavement that will have adequate resistance to distortion by frost heave, cracking from differential frost heave and distortion under traffic load as affected by seasonal variation of

Table 20-1
Frost Design Classification

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

[1] Nonfrost susceptible.
[2] Possibly frost susceptible, requires laboratory test to determine frost design soil classification.

supporting capacity, including severe weakening during frost melting periods. The three methods are (a) complete frost penetration method, (b) reduced subgrade strength method, and (c) limited subgrade frost penetration method.

a. Complete Frost Penetration Method. In the complete frost penetration method, frost is not allowed to penetrate into frost susceptible subgrade soils. This method completely prevents affects of frost action, i.e., frost heave and thaw weakening in the subgrade, subbase, or base course. The total pavement thickness from this method is seldom used in the final design since prevention of frost penetration into the subgrade is nearly always uneconomical and unnecessary.

Table 20-2
Frost Susceptibility Classification

Heave Rate (mm/day)	Thawed CBR	Frost Susceptibility Classification	Frost Group
<1	>20	Negligible	NFS, PFS
<2	>15	Very Low	S1, PFS
<4	>12	Very Low	S2, PFS
<6	>10	Low	F1
<8	>6	Medium	F2
<16	>3	High	F3
No Limit	<3	Very High	F4

It will not be used to design pavements to serve conventional traffic, except when approved by appropriate written waiver.

b. Reduced Subgrade Strength Method. The reduced subgrade strength method does not seek to limit the penetration of frost into the subgrade. It determines the thickness of pavement, base, and subbase that will adequately carry traffic. This approach relies on uniform subgrade conditions, adequate subgrade preparation, and transitions for adequate control of pavement roughness resulting from differential frost heave.

c. Limited Subgrade Frost Penetration Method. The limited subgrade frost penetration method requires a sufficient thickness of pavement, base, and subbase to limit the penetration of frost into the frost susceptible subgrade.

4. SELECTION OF DESIGN METHOD. In most cases the choice of the pavement design method will be made in favor of the one that gives the lower cost. The limited subgrade frost penetration method will be used, even at higher costs, in areas where the subgrade soils are extremely variable (e.g., in some glaciated areas) and the required subgrade preparation could not be expected to sufficiently restrict differential frost heave, and when special operational demands on the pavement might dictate unusually severe restrictions on pavement roughness, requiring that subgrade frost penetration be severely restricted or even prevented. If the use of limited subgrade frost penetration method is not required, tentative designs must be prepared using both methods for comparison of costs.

5. REDUCED SUBGRADE STRENGTH METHOD. The thickness design is based on the seasonally varying subgrade bearing capacity that includes sharply reduced values during frost melting periods. This design procedure usually requires less thickness of pavement and base than that needed for limited subgrade frost penetration. The method may be used for pavements where the subgrade is reasonably uniform or can be made reasonably uniform horizontally by the required subgrade preparation techniques discussed later in this chapter. This will prevent or minimize significant or objectionable differential heaving and resultant cracking of pavements. When the reduced

subgrade strength method is used with an F3 or F4 subgrade soil, rigorous control of subgrade preparation is required. In situations, based on previous experience, where use of the reduced subgrade strength procedure has resulted in pavement thicknesses allowing objectionable frost heave, but use of the greater base-course thickness obtained from the limited subgrade frost penetration method is considered over conservative, intermediate design base-course thickness may be used. However, these must be justified on the basis of frost heaving experience developed from existing pavements where climatic and soil conditions are comparable.

a. Thickness of Flexible Pavements. The thickness design procedure is identical to the thickness design for nonfrost conditions, with the exception that instead of using the subgrade CBR, Frost Areas Soil Support Index (FASSI) values are used. The flexible pavement design curves are used in connection with the reduced subgrade strength procedure. In place of the estimated or determined subgrade CBR, use the applicable FASSI values outlined in Table 20-3 with the design curves. The FASSI values for the F1 to F4 subgrade soils were backcalculated from performance data of in-service pavements, and are the weighted average CBR valued for an annual cycle. These values cannot be determined by CBR tests. The FASSI values for S1 and S2 materials meeting current specifications for base and subbase will be determined by conventional CBR procedures. The reduced subgrade strength design procedure is included in the design computer program PDSF discussed in Chapter 1.

Table 20-3
Frost Area Soil Support Indexes (FASSI) for Subgrade Soils

Frost Group of Subgrade Soil	FASSI Values
F1 and S1	9.0
F2 and S2	6.5
F3 and F4	3.5

Once the overall thickness of the pavement structure has been determined, criteria for nonfrost design should be used to determine the thickness of individual layers. It should also be ascertained whether it will be advantageous to incorporate bound base layer(s) in the system. Although the use of bound bases will reduce the thickness of the base and subbase layers, it is possible that deeper frost penetration may occur leading to increased frost heave. The base-course requirements set forth in this chapter must be followed rigorously.

(1) Design of overrun pavements. The runway overrun pavement thicknesses for providing adequate strength during frost melting periods are determined from the appropriate flexible pavement design curves and the applicable FASSI values outlined in Table 20-3. The thickness established by this procedure shall have the following limitations:

(a) It shall not be less than required for nonfrost condition design.

(b) It shall not exceed the thickness required under the limited subgrade frost penetration design method.

(c) It shall not be less than that required for normal operation of snowplows and other support vehicles.

The subgrade preparation techniques and transition details outlined in this chapter are required for overrun pavements.

(2) Control of surface roughness in overruns. For a frost group F3 and F4 subgrade, differential heave can generally be controlled to 75 millimeters (3 inches) in 15.2 meters (50 feet) by providing a thickness of base and subbase equal to 60 percent of the base-course thickness required by the limited subgrade frost penetration design method. For well drained F1 and F2 subgrade soils, the minimum thickness of pavement and base course in overruns should not be less than 40 percent of the total thickness required for limited subgrade frost penetration design.

(3) Design of shoulder pavements. When paved shoulders are required, the paved shoulder pavement, base, and subbase, shall have the combined thickness obtained from the flexible pavement design curve and the appropriate FASSI value in Table 20-3. The subgrade preparation techniques and transition details outlined in this chapter are required. If the subgrade is highly susceptible to frost heave, local experience may indicate a need for a shoulder section that incorporates an insulating layer or an additional granular unbound material to moderate the frost heave. The base-course requirements set forth in this chapter must be followed.

(4) Control of differential frost heave at small structures located within shoulder pavements. To prevent objectionable heave of small structures inserted in shoulder pavements, such as drain inlets and bases for airfield lights, the shoulder base and subbase courses extending at least 1.5 meters (5 feet) radially from the structures should be designed and constructed entirely with nonfrost susceptible material to a depth to prevent subgrade freezing. Gradual transitions are required. Alternatively, synthetic insulation could be placed below a base of the minimum prescribed thickness to prevent the advance of freezing temperatures into the subgrade; suitable transitions to the adjoining uninsulated pavement would be needed.

(5) Drainage. Subsurface drainage must be provided in flexible pavements in accordance with EI 02C202/AFJMAN 32-1016.

b. Rigid Pavement Thickness Design. The thickness design procedure is identical to the thickness design for nonfrost conditions, with the exception that instead of using the modulus of subgrade reaction k, Frost Area Index of Reaction (FAIR) values are used. The design curves for plain concrete and for fibrous concrete are used in connection with the reduced subgrade strength procedure. In place of the estimated or determined subgrade k in the design curves, use the applicable FAIR values from Figure 20-1. The FAIR values can also be estimated from the following equations:

S1 or F1 material:

$$\text{FAIR (pci)} = 6.7 + 10.7 \times \text{base-course thickness (in.)}$$

or (20-1)

$$\text{FAIR}\left(\frac{\text{MN}}{\text{m}^3}\right) = 1.8 + 114 \times \text{base-course thickness (m)}$$

S2 or F2 material:

$$\text{FAIR (pci)} = 4.5 + 8.0 \times \text{base-course thickness (in.)}$$

or (20-2)

$$\text{FAIR}\left(\frac{\text{MN}}{\text{m}^3}\right) = 1.2 + 83.8 \times \text{base-course thickness (m)}$$

F3 or F4 material:

$$\text{FAIR (pci)} = 5.4 + 5.7 \times \text{base-course thickness (in.)}$$

or (20-3)

$$\text{FAIR}\left(\frac{\text{MN}}{\text{m}^3}\right) = 1.5 + 60.8 \times \text{base-course thickness (m)}$$

The FAIR values for the S1, and F1 to F4 subgrade soils were determined from field measurements and are the weighted average k values for an annual cycle. These values cannot be determined from plate bearing tests.

(1) It is good practice to use a combined base thickness equal in thickness to the slab. The design procedure is as follows:

(a) Determine frost group soil classification of subgrade, Table 20-1.

(b) Assume three combined base thicknesses, enter Figure 20-1 or use appropriate equations, determine the FAIR value for each thickness.

(c) Use the FAIR values with appropriate design curves to determine pavement thickness.

(d) Plot combined base thickness and pavement thickness. From the figure, pick out base-course and pavement thickness of similar values.

(e) If unable to converge to a solution, repeat steps b to d with new base-course thickness.

(f) A minimum of 203 millimeters (8 inches) of combined base (100-millimeter (4-inch) drainage layer plus 100-millimeter (4-inch) separation layer) is required for rigid pavements in frost areas.

(2) The combined base must meet the drainage and filter requirements outlined in EI 02C202/AFJMAN 32-1016. A 100-millimeter (4-inch) separation layer meeting the filter requirements must be placed between the subgrade and base or subbase course. A geotextile separator can also be used in lieu of the granular filter. No structural advantage will be attained in the design when a geotextile is used. Guidance for selection of geotextile fabric materials

proposed for a specific project is provided in EI 02C202/AFJMAN 32-1016 and TM 5-818-8/ AFJMAN 32-1030.

(3) Bound base also has significant structural value and is considered to be a low-strength concrete for design purposes. A minimum 200-millimeter (8-inch) drainage plus separation layers must be placed between the bound base and the subgrade.

(4) If sufficient high-quality base material is not locally available, the nonfrost design base layer thickness can be used. The appropriate FAIR value will be used for the base to determine the PCC thickness.

(5) The subgrade preparation techniques and transition details outlined in this chapter are also required for the design of overrun pavements.

(6) The control of differential frost heave at small structures is located within shoulder pavements. To prevent objectionable heave of small structures inserted in shoulder pavements, such as drain inlets and bases for airfield lights, the shoulder base and subbase courses extending at least 1.5 meters (5 feet) radially from the structures should be designed and constructed entirely with nonfrost susceptible material to a depth to prevent subgrade freezing. Gradual transitions are required. Alternatively, synthetic insulation could be placed below a base to prevent the advance of freezing temperatures into the subgrade; suitable transitions to the adjoining uninsulated pavement would be needed.

6. LIMITED SUBGRADE FROST PENETRATION METHOD. This design method permits a small amount of frost penetration into frost susceptible subgrades. The procedure uses a design freezing index (DFI) as illustrated in Figure 20-2. Typical DFI values are shown in Figures 20-3 and 20-4. The procedure is described in the following subparagraphs. A computer program (PDSF) for providing the limited subgrade frost penetration design thickness is discussed in Chapter 1.

a. Step One. Determine frost penetration depths. The maximum frost penetration depths with respect to the design freezing index shown in Figure 20-5 are calculated from the Modified Berggren formula and computational procedures outlined in TM 5-852-6/AFM 88-19, Chap. 6. Frost penetration depths presented in Figure 20-5 are measured from the pavement surface. The pavement is considered free of snow and ice. Computations also assume that all soil beneath the pavement within the depth of frost penetration are granular and nonfrost susceptible. It was assumed in the computations that all soil moisture freezes at 0 degrees Celsius (32 degrees Fahrenheit). Use straight-line interpolation where necessary. The frost penetration depth a in meters for SI units and inches for English units can also be estimated from the following equations:

(1) For γ = 2,160 kg/m^3 (135 lb/ft^3) and ω = 3 percent,

$$a = 0.157 + 9E\text{-}5(DFI) - 4E\text{-}10(DFI)^2 \text{ in SI units} \tag{20-4}$$

$$a = 6.183 + 0.047(DFI) - 2.91E\text{-}6(DFI)^2 \text{ in English units} \tag{20-5}$$

(2) For γ = 2,160 kg/m^3 (135 lb/ft^3) and ω = 7 percent,

$$a = 0.1852 + 8E\text{-}5(DFI) - 4E\text{-}10(DFI)^2 \text{ in SI units} \quad (20\text{-}6)$$

$$a = 7.291 + 0.044(DFI) - 2.58E\text{-}6(DFI)^2 \text{ in English units} \quad (20\text{-}7)$$

(3) For γ = 2,400 kg/m^3 (150 lb/ft^3) and ω = 3 percent,

$$a = 0.1725 + 0.0001(DFI) - 5E\text{-}10(DFI)^2 \text{ in SI units} \quad (20\text{-}8)$$

$$a = 6.793 + 0.055(DFI) - 3.41E\text{-}6(DFI)^2 \text{ in English units} \quad (20\text{-}9)$$

(4) For γ = 2,400 kg/m^3 (150 lb/ft^3) and ω = 7 percent,

$$a = 0.1583 + 9E\text{-}5(DFI) - 4E\text{-}10(DFI)^2 \text{ in SI units} \quad (20\text{-}10)$$

$$a = 6.231 + 0.049(DFI) - 2.98E\text{-}6(DFI)^2 \text{ in English units} \quad (20\text{-}11)$$

where

DFI = °C-hours in SI units or °F-days in English units.

γ = soil density

ω = soil moisture content

In Figure 20-5, the frost penetration curves for γ = 2,160 kg/m^3 (135 lb/ft^3) and ω = 3 and 7 percent are combined because the curves were very close together. Also, note that these densities and moisture contents represent an approximation of a weighted average value of combined base.

b. Step Two. Estimate the moisture content and dry density of the nonfrost susceptible base-course material. For a conservative design, the 3 percent moisture content, 2,400 kg/m^3 (150 lb/ft^3) base material should be selected. Determine frost penetration depth for complete frost penetration from Figure 20-5.

c. Step Three. Compute thickness of combined base (combined thickness of base, subbase, drainage layer and separation layer) required for zero frost penetration into the subgrade (Figure 20-6) as follows:

$$c = a - p \quad (20\text{-}12)$$

where

c = thickness of unbound base, millimeters (inches)

a = thickness for complete frost protection, millimeters (inches)

p = thickness of asphalt or concrete for nonfrost design

d. Step Four. For limited frost penetration into the subgrade, determine the average moisture content of the subgrade prior to freezing. Compute water content ratio r.

$$r = \frac{\text{moisture content of subgrade}}{\text{moisture content of base}} \tag{20-13}$$

where

moisture content of the base = same as that assumed for nonfrost base material in step 2.

If the computed r exceeds 2.0, use 2.0 for types A, B, and primary traffic areas. If r exceeds 3.0, use 3.0 for all pavements other than those in types A, B, or primary traffic areas.

e. Step Five. Enter Figure 20-6, with c (from step 3) as the abscissa and, at the applicable value of r, find the design combined base thickness b on the left scale and the allowable frost penetration into the subgrade s on the right scale or use Equations 20-10 and 20-11. This procedure will result in a sufficient thickness of material between the frost susceptible subgrade and the pavement so that for average field conditions subgrade frost penetration of the amount s should not cause excessive differential heave of the pavement surface during the design freezing index year.

$$b = c \times f \tag{20-14}$$

$$s = c \times g \tag{20-15}$$

where

b = design combined base thickness

c = combined base thickness for zero penetration

s = limited subgrade frost penetration depth

f and g = factors from the following tabulation

Water Content Ratio (*r*)	*f*	*g*
0.6	0.881	0.216
0.8	0.850	0.209
1.0	0.806	0.200
1.2	0.781	0.197
1.4	0.756	0.188
1.6	0.725	0.181
1.8	0.706	0.178
2.0	0.644	0.175
2.5	0.613	0.156
3.0	0.550	0.144

g. Step Six. When the maximum combined thickness of pavement layers required by this design procedure exceeds 1.5 meters (60 inches), a total combined thickness to 1.5 meters (60 inches) will be used. Limiting the combined thickness of pavement and base to 1.5 meters (60 inches) may result in a greater surface roughness because of the greater subgrade frost penetration. To minimize pavement damage and roughness, steel reinforcements can be used in the concrete slabs to prevent large cracks. Smaller unreinforced slabs can also be considered. Alternatively, the design could incorporate subbase layers of uniform fine sand with a high moisture content to reduce frost penetration into the subgrade. These materials would be allowed only in the lower 500 millimeters (20 inches) of the subbase. When using this alternative the designer must be certain that materials of Frost Groups S2 or better are used as subbase layers. If either the high moisture retention subbase course or a combined thickness over 1.5 meters (60 inches) is selected for frost design purposes, specific approval of HQUSACE (CEMP-ET), the appropriate Air Force Major Command, or Naval Facilities Engineering Command shall be obtained.

h. Step Seven. The combined thickness of pavement layers required for limited subgrade frost penetration is then compared with that obtained for nonfrost conditions, and the thicker of the two cross sections will be adopted as the design thickness.

7. GRANULAR BASE- AND SUBBASE-COURSE REQUIREMENTS. The base-course material used in pavements in seasonal frost areas will meet the requirements for base course outlined in Chapter 8. In addition, the following requirements must be met:

(1) The top 50 percent of the combined base thickness must be nonfrost susceptible.

(2) The lower 50 percent thickness of combined base may be either nonfrost susceptible material, partially frost susceptible material, S1 or S2 material. If the separation layer meets the minimum S1 or S2 frost susceptibility criterion, then it can be considered to be part of the combined base. If not, then an additional 100-millimeter (4-inch) separation layer is required.

(3) Base- and subbase-course materials of borderline quality should be tested frequently after compaction to ensure that the compacted material meets requirement (1). For material expected to exhibit serious degradation during placement and compaction (>3 percent finer than 0.02 millimeters by weight), a test embankment may be needed to study the formation of fines by the proposed compaction method. If the test embankment shows serious degradation, the material gradation should be changed to account for the fines obtained during compaction. If experience indicates that the base- or subbase-course materials degrade rapidly under traffic loads or due to environmental effects, consideration should be given to stabilizing the material with asphalt or Portland cement.

(4) Mixing of base or subbase course material with frost susceptible subgrade soils should be avoided. The subgrade should be properly graded and compacted prior to the placement of the base or subbase course. Separation layer requirements must be met.

8. DRAINAGE LAYER REQUIREMENTS. A minimum 100-millimeter (4-inch) thick nonfrost susceptible drainage layer must be placed at the bottom of the asphalt concrete layer, the PCC layer or below the bound base for all pavements constructed in frost areas. The rapid draining nonfrost susceptible material must meet the gradation requirements shown in EI 02C202/ AFJMAN 32-1016. The drainage layer will be designed in accordance with the requirements in EI 02C202/AFJMAN 32-1016. The layer is considered as a structural component of the pavement and will serve as part of the base course. In seasonal frost areas, as frost penetrates into the frost susceptible subgrades, water is drawn to the cold front and ice lenses form. During the frost melting period, the ice lenses will melt and the water will have to be removed. In extremely wet conditions or with F3 and F4 subgrade soils, a drainage layer should be considered between the subbase and the subgrade in lieu of a drainage layer under the surfacing as illustrated in Figure 20-7.

9. SEPARATION LAYER. If subgrade freezing will occur, a minimum of a 100-millimeter (4-inch) granular separation layer as specified in EI 02C202/AFJMAN 32-1016 will be placed between the subgrade and the overlying base course. Over weak subgrades, a 152-millimeter (6-inch) or greater thickness may be necessary to support construction equipment and to provide a working platform for placement and compaction of the base course. This layer is not intended to be a drainage layer. The gradation of this separation layer should meet the requirements in EI 02C202/ AFJMAN 32-1016. An additional requirement is that the separation layers must be nonfrost susceptible or of frost group S1 or S2. Alternatively, where stable foundation already exists, geotextile fabrics meeting the requirements of EI 02C202/AFJMAN 32-1016 can be used in lieu of a granular material as a separation layer. No structural advantage will be attained in the design when a geotextile fabric is used. The fabrics must meet the requirements of EI 02C202/ AFJMAN 32-1016.

10. SUBGRADE REQUIREMENTS. In addition to the requirement outlined for subgrades in nonfrost areas in Chapter 6, the following additional requirements shall be required for subgrades in frost areas. It is a basic requirement for all pavements constructed in frost areas that subgrades in which freezing will occur be as uniform as possible. This will be done by mixing nonhomogeneous soils, eliminating isolated pockets of soil of higher or lower frost susceptibility, and blending the various types of soils into a single, relatively homogeneous mass. This attempts to produce a subgrade of uniform frost susceptibility and thus create conditions tending to make both surface heave and subgrade thaw weakening as uniform as possible over the paved area. To achieve uniformity in some cases, it will be necessary to remove high frost-susceptible soils or soils of low frost susceptibility. In that case, the pockets of soil to be removed should be excavated to the full

depth of frost penetration and replaced with material similar to the material left in place. This replacement should be completed before any required mixing and blending of the subgrade. This will minimize the potential for large variations in frost heave and subgrade support. In fill sections, the least frost susceptible soils shall be placed in the upper portion of the subgrade by temporarily stockpiling, cross hauling, and selective grading. If the upper layers of fill contain frost susceptible soils, the completed fill section shall be subjected to the subgrade preparation procedures, outlined below for cut sections. In cut sections, no matter the type of frost susceptible subgrade soil, the subgrade shall be scarified and excavated to a prescribed depth, and the excavated material windrowed and bladed successively until thoroughly blended, and relaid and compacted. Alternatively, a soil mixing and pulverizing machine may be used to blend the material in place. Multiple passes of the machine will be required for proper blending.

a. Depth of Subgrade Preparation. The depth of subgrade preparation is applicable for limited subgrade penetration and reduced subgrade strength design. The depth of subgrade preparation measured downward from the top of the subgrade shall be lesser of:

(1) 0.6 meter (24 inches).

(2) Two-thirds of the frost penetration less the actual combined thickness of pavement, base course, drainage layers, and subbase course under types A, B, or primary traffic areas.

(3) Under type C, D, and secondary traffic areas and under overruns and shoulder pavements, it will be one-half the frost penetration less the actual combined thickness of pavement, base course, drainage layers, and subbase course.

(4) 1.8 meters (72 inches) less the actual combined thickness of pavement, base course, drainage layers, and subbase course.

The prepared subgrade must meet the designated compaction requirements for nonfrost areas discussed in Chapter 6. The construction inspection personnel should be alert to verify that the processing of the subgrade will yield uniform soil conditions throughout the section.

b. Exceptional Conditions. An exception to the basic requirements for subgrade preparation are subgrades that are nonfrost susceptible or of very low frost susceptibility (NFS, S1, S2) to the depth prescribed for subgrade preparation. These subgrades contain no frost susceptible layers or lenses, as demonstrated and verified by extensive and thorough subsurface investigations and by the performance of nearby existing pavements. Also, fine-grained subgrades containing moisture well in excess of the optimum for compaction, with no feasible means of drainage nor of otherwise reducing the moisture content, and which consequently are not feasible to scarify and recompact, are also exceptions. If a wet fine-grained subgrade exists at the site, it will be necessary to prevent frost penetration with fill material. This may be done by raising the grade by an amount equal to the depth of subgrade preparation that otherwise would be prescribed, or by undercutting and replacing the wet fine-grained subgrade to that same depth. In either case, the fill or backfill material may be nonfrost susceptible or frost susceptible material. If the fill or backfill is frost susceptible, it should be subjected to the same subgrade preparation procedures prescribed above.

c. Cobbles or Boulders. A critical condition requiring the attention of designers and inspection personnel is the presence of cobbles or boulders in the subgrade. All stones larger than about 150 millimeters (6 inches) in diameter should be removed from fill materials for the full depth of frost penetration, either at the source or as the material is spread in the embankment. Any such

large stones exposed during subgrade preparation work must also be removed, down to the full depth to which subgrade preparation is required. Failure to remove stones or large roots can result in increasingly severe pavement roughness as the stones or roots are heaved gradually upward toward the pavement surface. They eventually break through the surface in extreme cases, necessitating complete reconstruction.

d. Soil Conditions. Abrupt changes in soil conditions must not be permitted. Where the subgrade changes from a cut to a fill section, a wedge of subgrade soil in the cut section with the dimensions shown in Figure 20-8 should be removed and replaced with fill material. Discontinuities in subgrade conditions require the most careful attention of designers and construction inspection personnel, as failure to enforce strict compliance with the requirements for transitions may result in serious pavement distress.

e. Rock Excavation. In areas where rock excavation is required, the character of the rock and seepage conditions should be considered. In any case, the excavation should be made so that positive transverse drainage is provided and no pockets are left on the rock surface that will permit ponding of water within the depth of freezing. The irregular ground water availability created by such conditions may result in markedly irregular heaving under freezing conditions. It may be necessary to fill drainage pockets with lean concrete. At intersections of fills with rock cuts, the tapered transitions illustrated in Figure 20-8 are essential. Rock subgrades where large quantities of seepage are involved should be blanketed with a highly pervious material to permit the escape of water. Frequently, the fractures and joints in the rock contain frost susceptible soils. These materials should be cleaned out of the joints to the depth of frost penetration and replaced with nonfrost susceptible material. If this is impractical, it may be necessary to remove the rock to the full depth of frost penetration. An alternative method of treating rock subgrades, in-place fragmentation, has been used effectively in airfield construction. Blast holes 0.9 to 1.8 meters (3 to 6 feet) deep are commonly used. They are spaced suitable for achieving thorough fragmentation of the rock to permit effective drainage of water through the shattered rock and out of the zone of freezing in the subgrade. A tapered transition should be provided between the shattered rock cut and the adjacent fill. Underdrains are essential to quickly remove excess water.

11. CONTROL OF DIFFERENTIAL HEAVE AT DRAINS, CULVERTS, DUCTS, INLETS, HYDRANTS, AND LIGHTS.

a. Design Details and Transitions for Drains, Culverts, and Ducts. Drains, culverts, or utility ducts placed under pavements on frost-susceptible subgrades frequently experience differential heaving. Wherever possible, the placing of such facilities beneath pavements should be avoided. Where this cannot be avoided, construction of drains should be in accordance with the"correct" method indicated in Figure 20-9, while treatment of culverts and large ducts should conform with Figure 20-10. All drains of similar features should be placed first and the base and subbase course materials carried across them without bread so as to obtain maximum uniformity of pavement support. The practice of constructing the base and subbase course and then excavating back through them to lay drains, pipes, etc. is unsatisfactory as a marked discontinuity in support will result. It is almost impossible to compact material in a trench to the same degree as the surrounding base and subbase course materials. Also, the amount of fines in the excavated and backfilled material may be increased by incorporation of subgrade soil during the trench excavation or by manufacture of fines by the added handling. The poor experience record of combination drains—those intercepting both surface and subsurface water—indicates that the filter material should never be carried to the surface as illustrated in the "incorrect" column in Figure 20-9. Under winter conditions, this detail may allow thaw water accumulating at the edge of the pavement to

feed into the base course. This detail is also undesirable because the filter is a poor surface and is subject to clogging, and the drain is located too close to the pavement to permit easy repair. Recommended practice is shown in the "correct" column in Figure 20-9.

b. Frost Protection and Transitions for Inlets, Hydrants, and Lights. Experience has shown that drain inlets, fueling hydrants, and pavement lighting systems, which have different thermal properties that the pavements in which they are inserted, are likely to be locations of abrupt differential heave. Usually, the roughness results from progressive movement of the inserted items. To prevent these damaging movements, the pavement section beneath the inserts and extending at least 1.5 meters (5 feet) radially from them should be designed to prevent freezing of frost-susceptible materials by sue of an adequate thickness of nonfrost-susceptible base course, and by use of insulation. Consideration should also be given to anchoring footings with spread bases at appropriate depths. Gradual transitions are required to surrounding pavements that are subject to frost heave.

12. PAVEMENT THICKNESS TRANSITIONS.

a. Longitudinal Transitions. Where interruptions in pavement uniformity cannot be avoided, differential frost heaving should be controlled by use of gradual transitions. Length of longitudinal transitions should vary directly with the speed of traffic and the amount of heave differential. Transition sections should begin and end directly under the pavement joints, and should in no case be shorter than one slab length. As an example, at an airfield where differentials of heave of 25 millimeters (1 inch) may be expected at changes from one subgrade soil condition to another, gradual changes in base thicknesses should be effected over distances of 61 meters (200 feet) for the runway area, 30.5 meters (100 feet) for taxiways, and 15.25 meters (50 feet) for aprons. The transition in each case should be located in the section having the lesser total thickness of pavement and base. Pavements designed to lower standards of frost heave control, such as shoulders and overruns, have less stringent requirements, but may nevertheless need transition sections.

b. Transverse Transitions. A need for transitions in the transverse direction arises at changes in total thickness of pavement and base, and at longitudinal drains and culverts. Any transverse transition beneath pavements that carry the principal wheel assemblies of aircraft traveling at moderate to high speed should meet the same requirements applicable to longitudinal transitions. Transverse transitions between the traffic areas C and D should be located entirely within the limits of traffic area D and should be sloped no steeper than 10 horizontal to 1 vertical. Transverse transitions between pavements carrying aircraft traffic and adjacent shoulder pavements should be located in the shoulder and should not be sloped steeper than 4 horizontal to 1 vertical.

13. OTHER MEASURES TO REDUCE FROST HEAVE. Another measure to reduce the effects of heave is the use of insulation to control depth of frost penetration. Insulation can only be used in shoulders and overruns. The use of synthetic insulating materials within a pavement cross section must have the approval of HQUSACE (CEMP-ET) or the appropriate Air Force Major Command. When synthetic insulating materials are used, transitions between cut and fill, changes in character and stratification of subgrade soils, subgrade preparation, and boulder removal should also receive special attention in field construction control.

14. REPLACEMENT OR RECONSTRUCTION OF EXISTING PAVEMENTS. Objectionable differential heave has been noticed where existing airfield pavements have been partially reconstructed or new segments added. These discontinuities in elevation can result in problems of snow removal,

ponding of water, surface icing, and loss of control of aircraft or unnecessary stresses to the aircraft or vehicles using the pavement. This objectionable and abrupt differential movement is caused by the use of different material in the base and subbase and/or the use of different thicknesses than existing material. Longitudinal abrupt differences have been noted where the keel section has been replaced on airfields. Transverse abrupt differences have been noticed in newly added taxiways where the total thickness of pavement, base, and subbase has been different from that previously used. The differences are most pronounced when the pavement type is changed from PCC to asphalt concrete. PCC pavements generally require smaller base and subbase thicknesses than asphalt concrete pavements resulting in deeper frost penetration and potentially greater frost heave. To minimize these abrupt differences in pavement elevation, pavement surface elevation surveys should be conducted in the summer and again in the winter when frost penetration is near its maximum depth. Both surveys should be completed before the new facility is designed. The difference in the two surveys will indicate the potential for abrupt differences in pavement surface elevation resulting from differing designs. The abrupt differences can be eliminated or substantially reduced by using proper transitions, or by using the same materials previously used. However, care must be taken if consideration is being given to the use of similar materials that resulted in the initial distress. Materials which are frost susceptible and placed too near the pavement surface can result in premature failure.

15. COMPACTION. Subgrade, subbase, and base-course materials must meet the applicable compaction requirements for nonfrost materials.

16. FLEXIBLE PAVEMENT DESIGN EXAMPLES FOR SEASONAL FROST CONDITIONS.

a. Example 1. Design an Air Force heavy-load pavement type B traffic area. The design freezing index at the site is 9,331-degree Celsius hours (700-degree Fahrenheit days). The highest elevation of ground water is about 915 millimeters (3 feet) below the surface of the subgrade. The subgrade is a lean clay (CL), with a plasticity index of 18. The average moisture content of the subgrade is 18 percent. The nonfrost design CBR of the lean clay subgrade is 13. A high quality crushed base-course material with a normal period CBR of 100 is to be used.

(1) Reduced subgrade strength design.

(a) The subgrade is classified as an F3 frost susceptible soil from Table 20-1. From Table 20-3, the FASSI value for an F3 soil is 3.5.

(b) Use the FASSI value with Figure 10-19 as though it were a CBR. Locate the value of 3.5 and move down to type B curve; the combined thickness of pavement required is 1.78 meters (70 inches).

(c) From Table 8-5, the minimum thickness of the surface course is 127 millimeters (5 inches). Therefore, the required base and subbase course thickness is 1.65 meters (65 inches).

(d) Compare pavement thickness with the limited subgrade frost penetration design.

(2) Limited subgrade frost penetration design.

(a) The moisture content of the base course is 3 percent, and the density of the base course is 2,400 kg/m^3 (150 lb/ft^3). From Table 8-5, a minimum thickness of a 127-millimeter

(5-inch) asphalt concrete layer is required. The frost penetration a from Figure 20-5 is 1,143 millimeters (45 inches).

(b) The required base thickness c for zero frost penetration from Equation 20-8 is:

$$c = a - p = 1{,}145\text{-}127 = 1{,}016 \text{ millimeters (40 inches)}$$

$$p = \text{asphalt concrete thickness of 127 millimeters (5 inches)}$$

(c) Compute water content ratio r from Equation 20-13. $r = 18/3 = 6$. Since $r > 2.0$ for type B traffic area, use $r = 2.0$ with Figure 20-6. From Figure 20-6, the allowable subgrade frost penetration is approximately 178 millimeters (7 inches). Again, from Figure 20-6, the design base thickness b as determined by the limited subgrade frost penetration method is 685 millimeters (27 inches).

(d) The base thickness of 685 millimeters (27 inches) is less than the thickness of 1,651 millimeters (65 inches) from the reduced subgrade design. In this case, the thickness from limiting subgrade frost penetration design is more economical than from the reduced subgrade design. Also, the thickness from the limited subgrade frost penetration is greater than that obtained from the nonfrost design (see Paragraph 5a, Example 1, Chapter 12). Therefore, the combined thickness (combined asphalt plus base and subbase material) of 813 millimeters (32 inches) will be used as the design thickness.

(e) The pavement structure could be made up of 127 millimeters (5 inches) of surface course, 102 millimeters (4 inches) of a NFS drainage layer beneath the surface course, 254 millimeters (10 inches) of NFS base course, 228 millimeters (9 inches) of S1 or S2 subbase, 102 millimeters (4 inches) of a NFS drainage layer. A geotextile fabric separation layer shall be placed between the subgrade and the drainage layer.

(f) No subgrade preparation is required because the 813-millimeter (32-inch) combined thickness of pavement and base exceeds two-thirds of the design frost penetration depth of 762 millimeters (30 inches).

b. Example 2. Design a type A traffic area on an Army Class III airfield as defined in Paragraph 4c, Chapter 2. The design freezing index of the area is 39,990-degree Celsius hours (3,000-degree Fahrenheit days). The subgrade is a mixture of poorly graded gravely sand and silty sand with a fine content of about 9 percent. The average moisture content of the subgrade is 9 percent. The nonfrost design CBR of the subgrade is 16.

(1) Reduced subgrade strength design.

(a) The subgrade can be classified as SP-SM soil. It also classifies as an F2 frost susceptible soil from Table 20-1. From Table 20-3, the FASSI value for an F2 soil is 6.5.

(b) From Figure 10-3 with a FASSI (CBR) value of 6.5, the combined thickness of pavement required is 330 millimeters (13.0 inches).

(c) From Table 8-3, the minimum thickness of the surface course is 51 millimeters (2 inches). Therefore, the required base and subbase course thickness is 279 millimeters (11.0 inches).

(d) Compare pavement thickness with the limited subgrade frost penetration design.

(2) Limited subgrade frost penetration design.

(a) The moisture content of the base course is assumed to be 3 percent. The density of the base course is assumed to be 2,403 kg/m^3 (150 lb/ft^3). From Table 8-3, a minimum thickness of a 51-millimeter (2-inch) asphalt concrete layer is required. The frost penetration a from Figure 20-5 is 3.6 meters (142 inches).

(b) The required base thickness c for zero frost penetration from Equation 20-8 is:

$$c = a - p = 142 - 2 = 140 \text{ inches in English units}$$

$$= 3{,}600 - 51 = 3{,}549 \text{ millimeters in SI units}$$

(c) In this case the base thickness of 3,549 millimeters (140 inches) is more than the thickness of 339 millimeters (13 inches) from the reduced subgrade design. Therefore, the thickness from reduced subgrade design is more economical than from the limiting subgrade frost penetration design. Also, the thickness from the reduced subgrade design is greater than that obtained from the nonfrost design. Therefore, the combined thickness of 330 millimeters (13.0 inches) will be used as the design thickness. The pavement structure could be made up of 51 millimeters (2 inches) of surface course, 100 millimeters (4 inches) of an NFS drainage layer beneath the surface course, and 178 millimeters (7.0 inches) of NFS base over a separation layer. A geotextile fabric could be placed between the subgrade and base course as a separation layer. Subgrade preparation is required to a depth of 610 millimeters (24 inches) based on the subgrade preparation criteria described in Paragraph 10a.

17. RIGID PAVEMENT DESIGN EXAMPLES FOR SEASONAL FROST CONDITIONS.

a. Example 1. Design an Air Force medium-load pavement. The design air freezing index at the site is 9,330-degree Celsius hours (700-degree Fahrenheit days). The highest elevation of groundwater is about 914 millimeters (3 feet) below the surface of the subgrade. The subgrade is a silty sand with 20 percent finer than 0.02 mm by weight. The average moisture content of the subgrade is 15 percent. The nonfrost design modulus of soil reaction k is 54 MN/m^3 (200 lb/in.^{3}). The 90-day concrete flexural strength R is 4.8 MPa (700 psi).

(1) Reduced subgrade strength design.

(a) From Table 20-1, the subgrade is classified as a F3 frost susceptible soil.

(b) Select several combined base thicknesses and obtain FAIR values from Figure 20-5 or from Equation 20-1. For example:

Combined Base Thickness, mm (in.)	FAIR Values, MN/m^3 (lb/in.^{3})
100 (4)	7.6 (28)
150 (6)	10.6 (40)
200 (8)	13.7 (51)
300 (12)	19.7 (74)
460 (18)	29.5 (108)
610 (24)	38.6 (142)

(c) Use the FAIR value with Figure 12-7 as though it were a k value and determine the thickness of PCC pavement.

Combined Base Thickness, mm (in.)	FAIR Value MN/m^3 (lb/in.^{3})	Traffic Area PCC Thickness, mm (in.)			
		A	B	C	D
100 (4)	7.6 (28)	627 (24.7)	620 (24.4)	500 (19.7)	386 (15.2)
150 (6)	10.6 (40)	594 (23.4)	584 (23.0)	465 (18.3)	353 (13.9)
200 (8)	13.7 (51)	566 (22.3)	556 (21.9)	439 (17.3)	330 (13.0)
300 (12)	19.7 (74)	523 (20.6)	516 (20.3)	399 (15.7)	292 (11.5)
460 (18)	29.5 (108)	475 (18.7)	467 (18.4)	356 (14.0)	254 (10.0)
610 (24)	38.6 (142)	439 (17.3)	432 (17.0)	320 (12.6)	244 (9.6)

(d) Plot combined base versus PCC thickness for the different traffic areas as shown in Figure 20-11. Locate equal or nearly equal thickness of PCC and base course as shown in the figure. From Figure 20-11, the minimum thickness for the combined base course and PCC layer are as follows:

Traffic Area	Combined Base Thickness, mm (in.)	PCC Pavement Thickness, mm (in.)
A	483 (19.0)	483 (19.0)
B	470 (18.5)	470 (18.5)
C	381 (15.0)	381 (15.0)
D	292 (11.5)	292 (11.5)
Shoulder[1]	152 (6.0)	152 (6.0)

[1] The thickness of the shoulder was determined in a similar fashion as for the other traffic areas. Use Figure 12-17 for determining pavement thickness.

(e) Compare these reduced subgrade strength pavement thicknesses with those required for the Limited Subgrade Frost Penetration Design procedure.

(2) Limited subgrade frost penetration design.

(a) From Figure 12-7, the minimum thickness of PCC concrete layer required in nonfrost areas are:

Traffic Area	PCC Thickness, mm (in.)
A	406 (16.0)
B	394 (15.5)
C	305 (12.0)
D	241 (9.5)

The average moisture content and density of the combined base is assumed to be 3 percent and 2,403 kg/m^3 (150 lb/ft^3), respectively. The frost penetration a is obtained from Figure 20-5 or from Equation 20-9. For pavement thickness exceeding 305 millimeters (12 inches), deduct 133-degree Celsius hours (10-degree Fahrenheit days) from the design freezing index for each inch in excess of 305 millimeters (12 inches).

Traffic Area	PCC Thickness mm (inches)	Design Freezing Index °C-hr (°F-days)	Frost Penetration, a, mm (in.)[1]
A	406 (16.0)	8,800 (660)	1,067 (42)
B	394 (15.5)	8,865 (665)	1,067 (42)
C	305 (12.0)	9,331 (700)	1,118 (44)
D	241 (9.5)	9,331 (700)	1,118 (44)

[1] Obtained from Equation 20-9.

(b) The required combined base thickness c for complete frost penetration into the subgrade from Equation 20-12 is:

$$c = a - p \qquad p = \text{PCC thickness}$$

Traffic Area	Frost Penetration, a, mm (in.)	PCC Thickness, p, mm (in.)	Combined Base Thickness c, mm (in.)
A	1,067 (42)	406 (16.0)	661 (26.0)
B	1,067 (42)	394 (15.5)	673 (26.5)
C	1,118 (44)	305 (12.0)	813 (32.0)
D	1,118 (44)	241 (9.5)	877 (34.5)

(c) Compute water content ratio r from Equation 20-13.

r = water content of subgrade/water content of base

= 15/3 = 5

For r ≥ 2.0, for types A and B traffic areas, use r = 2.0 with Figure 20-6. For r ≥ 3.0, for types C and D traffic areas use r = 3.0 with Figure 20-6. Determine the design combined base-course thickness (b) and amount of subgrade frost penetration (s) for the combined base thickness.

Traffic Area	Design Combined Base Thickness b mm (in.)	Subgrade Frost Penetration Depths mm (in.)
A	432 (17)	114 (4.5)
B	432 (17)	127 (5.0)
C	457 (18)	127 (5.0)
D and Overruns	508 (20)	127 (5.0)

(d) Compare these design pavement thicknesses with those obtained with the reduced subgrade strength design procedures.

In this case, the thicknesses required for traffic areas A, B, and C using the limited subgrade frost penetration design are more economical than from the reduced subgrade strength design. For traffic area D, even though the limited subgrade frost penetration design requires the greatest thickness of pavement and base, it may still be the most economical design as it requires only 241 millimeters (9.5 inches) of PCC versus the 292 millimeters (11.5 inches) required by the reduced subgrade strength design procedure. The designer must make a decision based upon a comparison of costs between the PCC and the base material. The design pavement thickness selection is shown below. The final thickness of PCC for the same base-course thickness for nonfrost conditions is also shown. The thicker value of the two will be used.

	Reduced Subgrade Strength Method mm (in.)			Limited Subgrade Frost Penetration mm (in.)		
Traffic Area	PCC	Combined Base	Total	PCC	Combined Base	Total
A	483 (19.0)	483 (19.0)	966 (38.0)	406 (16.0)	432 (17.0)	838 (33.0)
B	470 (18.5)	470 (18.5)	940 (37.0)	394 (15.5)	432 (17.0)	826 (32.5)
C	381 (15.0)	381 (15.0)	762 (30.0)	305 (12.0)	457 (18.0)	762 (30.0)
D	292 (11.5)	292 (11.5)	584 (23.0)	241 (9.5)	508 (20.0)	749 (29.5)

		Frost Design Thickness, mm (in.)		Nonfrost Design Thickness, mm (in.)	
Traffic Area	**Design Method**	**PCC**	**Combined Base**	**PCC**	**Combined Base**
A	LSFP	406 (16.0)	432 (17.0)	318 (12.5)	432 (17.0)
B	LSFP	394 (15.5)	432 (17.0)	318 (12.5)	432 (17.0)
C	LSFP	305 (12.0)	457 (18.0)	254 (10.0)	457 (18.0)
D	RSS or LSFP	292 (11.5) 241 (9.5)	292 (11.5) 508 (20.0)	203 (8.0)	292 (11.5)

(e) The combined base course can be divided into several layers having thicknesses as given in the following table. With F3 soils, in lieu of drainage layer under the PCC pavement, a drainage layer between the subbase and the separation layer should be considered. The divisions shown are one of many possibilities. Judgment must be used when layer thicknesses are selected.

Traffic Area	Combined Base Thickness mm (in.)	NFS Base Layer mm (in.)	S1 or S2 Subbase Layer mm (in.)	Drainage Layer mm (in.)	Separator Layer mm (in.)
A	432 (17.0)	229 (9.0)	--	102 (4)	102 (4)
B	432 (17.0)	229 (9.0)	--	102 (4)	102 (4)
C	457 (18.0)	254 (10.0)	--	102 (4)	102 (4)
D	292 (11.5)	89 (3.5)	--	102 (4)	102 (4)

(f) Compute the required depth of subgrade preparation.

Traffic Area	Total Pavement Thickness, mm (in.)	Frost Penetration, mm (in.)	Depth of Subgrade Preparation[1] mm (in.)
A	965 (38.0)	1,067 (42.0)	-102 (-4.0)
B	940 (37.0)	1,067 (42.0)	-127 (-5.0)
C	762 (30.0)	1,118 (44.0)	-356 (-14.0)
D	584 (23.0)	1,118 (44.0)	-533 (-21.0)

[1] No subgrade preparation required.

b. Example 2. Design an Air Force heavy-load pavement airfield. The design air freezing index at the site is 26,660-degree Celsius hours (2,000-degree Fahrenheit days). The highest

elevation of groundwater is about 1.5 meters (5 feet) below the surface of the subgrade. The subgrade is a very fine silty sand with 20 percent finer than 0.02 mm by weight. The average moisture content of the subgrade is 21 percent. The nonfrost design modulus of soil reaction is 27.1 MN/m^3 (100 lb/in.^{3}). The 90-day concrete flexural strength R is 4.83 MPa (700 psi).

(1) Reduced subgrade design.

(a) From Table 20-1, the subgrade is classified as F4 frost susceptible soil.

(b) Select 6 combined base-course thicknesses and obtain FAIR values from Figure 20-1 or from Equation 20-3.

Combined Base Thickness, mm (in.)	FAIR Values, MN/m^3 (lb/in.^{3})
102 (4)	7.6 (28)
152 (6)	10.6 (40)
203 (8)	13.7 (51)
305 (12)	19.7 (74)
457 (18)	29.5 (108)
610 (24)	38.6 (142)

(c) Use FAIR values with Figure 20-7 as though they were k values and determine the thickness of PCC pavement.

Combined Base Thickness, mm (in.)	FAIR Value MN/m^3 (lb/in.^{3})	Traffic Area PCC Thickness, mm (in.)			
		A	B	C	D
102 (4)	7.6 (28)	693 (27.3)	688 (27.1)	572 (22.5)	450 (17.7)
152 (6)	10.6 (40)	671 (26.4)	666 (26.2)	546 (21.5)	432 (17.0)
203 (8)	13.7 (51)	653 (25.7)	648 (25.5)	531 (20.9)	419 (16.5)
305 (12)	19.7 (74)	622 (24.5)	617 (24.3)	508 (20.0)	399 (15.7)
457 (18)	29.5 (108)	594 (23.4)	589 (23.2)	485 (19.1)	379 (14.9)
610 (24)	38.6 (142)	574 (22.6)	572 (22.5)	467 (18.4)	363 (14.3)

(d) Plot base course versus PCC thickness for the different traffic areas as shown in Figure 20-12. Locate equal or nearly equal thickness of PCC and combined base course as shown in the figure. The minimum thicknesses for the base and PCC layers are as follows:

Traffic Area	Combined Base Thickness, mm (in.)	PCC Pavement Thickness, mm (in.)
A	579 (22.8)	579 (22.8)
B	577 (22.7)	577 (22.7)
C	483 (19.0)	483 (19.0)
D	386 (15.2)	386 (15.2)
Shoulder[1]	152 (6)	152 (6)

[1] The thickness of the shoulder was determined in a similar fashion as for the other traffic areas. Use Figure 12-16 to determine minimum pavement thickness.

(e) Compare these reduced subgrade strength pavement thicknesses with those required for the limited subgrade frost penetration design.

(2) Limited subgrade frost penetration design.

(a) From Figure 20-7, the minimum PCC thickness for nonfrost conditions is as follows:

Traffic Area	PCC Thickness, mm (in.)
A	599 (23.6)
B	597 (23.5)
C	490 (19.3)
D	384 (15.1)

For pavement thicknesses greater than 305 millimeters (12 inches), deduct 133-degree Celsius hours (10-degree Fahrenheit days) from the design freezing index for each 25 millimeters (1 inch) in excess of 508 millimeters (12 inches).

(b) Assuming the average moisture content and density of the combined base course is 7 percent and 2,403 kg/m^3 (150 lb/ft^3), respectively, the maximum frost penetration a is obtained from Figure 20-5 or from Equation 20-10.

Traffic Area	PCC Thickness mm (in.)	Design Freezing Index °C hr (°F-days)	Frost Penetration mm (in.)
A	592 (23.6)	25,114 (1,884)	2,235 (88)
B	597 (23.5)	25,127 (1,885)	2,235 (88)
C	490 (19.3)	25,687 (1,927)	2,286 (90)
D	384 (15.1)	26,246 (1,969)	2,311 (91)

(c) The required base thickness c for zero frost penetration into the subgrade from Equation 20-12 is as shown in the following tabulation:

Traffic Area	Frost Penetration, a mm (in.)	PCC Thickness, p mm (in.)	Combined Base Thickness, c mm (in.)
A	2,235 (88)	599 (23.6)	1,636 (64.4)
B	2,235 (88)	557 (23.5)	1,638 (64.5)
C	2,286 (90)	490 (19.3)	1,796 (70.7)
D	2,311 (91)	384 (15.1)	1,928 (75.9)

(d) Compute water content ratio r.

r = water content of subgrade/water content of base

= 21/7 = 3

For r ≥ 2.0, for type A and B traffic areas, use r = 2.0 with Figure 20-6. For r ≥ 3.0, for type C and D traffic areas use r = 3.0 with Figure 20-6. Determine the design combined base-course thickness b and amount of subgrade frost penetration s for the combined base thickness.

Traffic Area	Design Base Thickness b, mm (in.)	Subgrade Frost Penetration s, mm (in.)
A	1,054 (41.5)	287 (11.3)
B	1,054 (41.5)	287 (11.3)
C	988 (38.9)	259 (10.2)
D	1,059 (41.7)	277 (10.9)

(e) Compare these design pavement thicknesses with those obtained with the reduced subgrade strength design procedure.

	Reduced Subgrade Strength Method			Limited Subgrade Frost Penetration		
Traffic Area	PCC mm (in.)	Combined Base mm (in.)	Total mm (in.)	PCC mm (in.)	Combined Base mm (in.)	Total mm (in.)
A	579 (22.8)	579 (22.8)	1,158 (45.6)	599 (23.6)	1,054 (41.5)	1,654 (65.1)
B	577 (22.7)	577 (22.7)	1,153 (45.4)	597 (23.5)	1,054 (41.5)	1,651 (65.0)
C	483 (19.0)	483 (19.0)	965 (38.0)	490 (19.3)	988 (38.9)	1,478 (58.2)
D	386 (15.2)	386 (15.2)	772 (30.4)	384 (15.1)	1,059 (41.7)	1,443 (56.8)

The reduced subgrade strength method produced the more economical design for all traffic areas. A comparison must be made with the nonfrost pavement thickness. With F4 subgrade soils, in lieu of drainage layer under the PCC pavement, a drainage layer between the subbase and the separation layer should be considered. The divisions shown are one of many possibilities. Judgment must be used when layer thicknesses are selected.

Traffic Area	Total "Combined Base" Thickness mm (in.)	NFS Base Layer mm (in.)	S1 or S2 Subbase Layer mm (in.)	Drainage Layer mm (in.)	Separator Layer mm (in.)
A	1,168 (46.0)	610 (24.0)	356 (14)	102 (4)	102 (4)
B	1,156 (45.5)	622 (24.5)	330 (13)	102 (4)	102 (4)
C	965 (38.0)	508 (20.0)	250 (10)	102 (4)	102 (4)
D	775 (30.5)	419 (16.5)	152 (6)	102 (4)	102 (4)

(3) Subgrade preparation. The depth of subgrade preparation D will be the lesser of the following:

(a) D = 610 millimeters (24.0 inches).

(b) D = [2/3 · (maximum frost penetration)] - (Combined thickness of PCC, base, subbase and drainage layers).

Traffic Area	Frost Penetration mm (in.)	Combined Pavement Thickness, mm (in.)	Subgrade Preparation Depth, mm (in.)
A	2,235 (88)	1,168 (46.0)	330 (13.0)
B	2,235 (88)	1,156 (45.5)	330 (13.0)
C[1]	2,285 (90)	965 (38.0)	178 (7.0)
D[1]	2,311 (91)	775 (30.5)	381 (15.0)

[1] Use one-half rather than two-thirds frost penetration for traffic areas C and D.

(c) D = 1,829 millimeters (72 inches) - (Combined Pavement Thickness).

Traffic Area	Combined Pavement Thickness, mm (in.)	D, mm (in.)
A	1,168 (46.0)	660 (26.0)
B	1,156 (45.5)	673 (26.5)
C	965 (38.0)	864 (34.0)
D and Overruns	775 (30.5)	1,054 (41.5)

The Final Depth of Subgrade Preparation D will be:

Traffic Area	D, mm (in.)
A	330 (13.0)
B	330 (13.0)
C	179 (7.0)
D	381 (15.0)

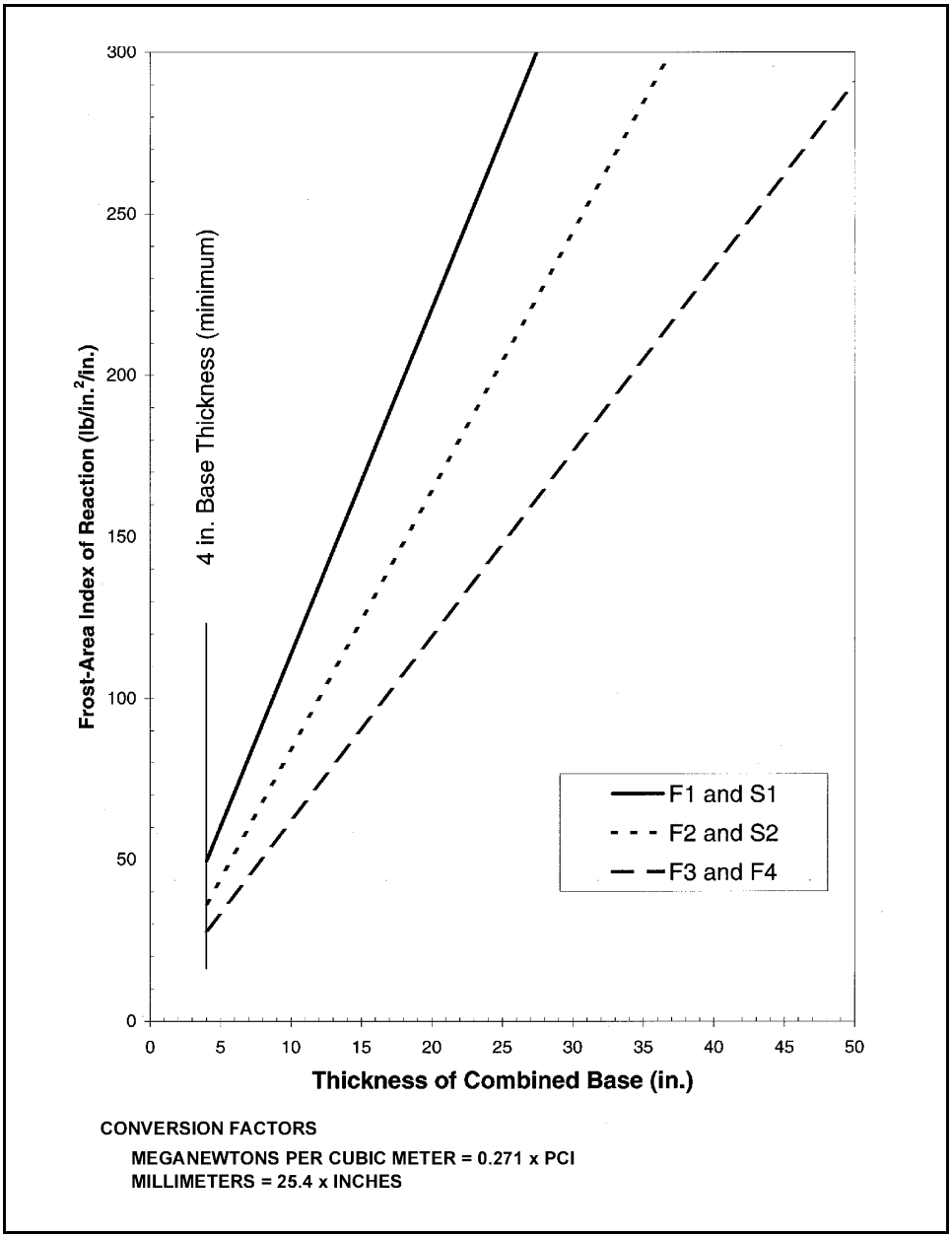


Figure 20-1. Frost area index of reaction (FAIR) for design of rigid pavements

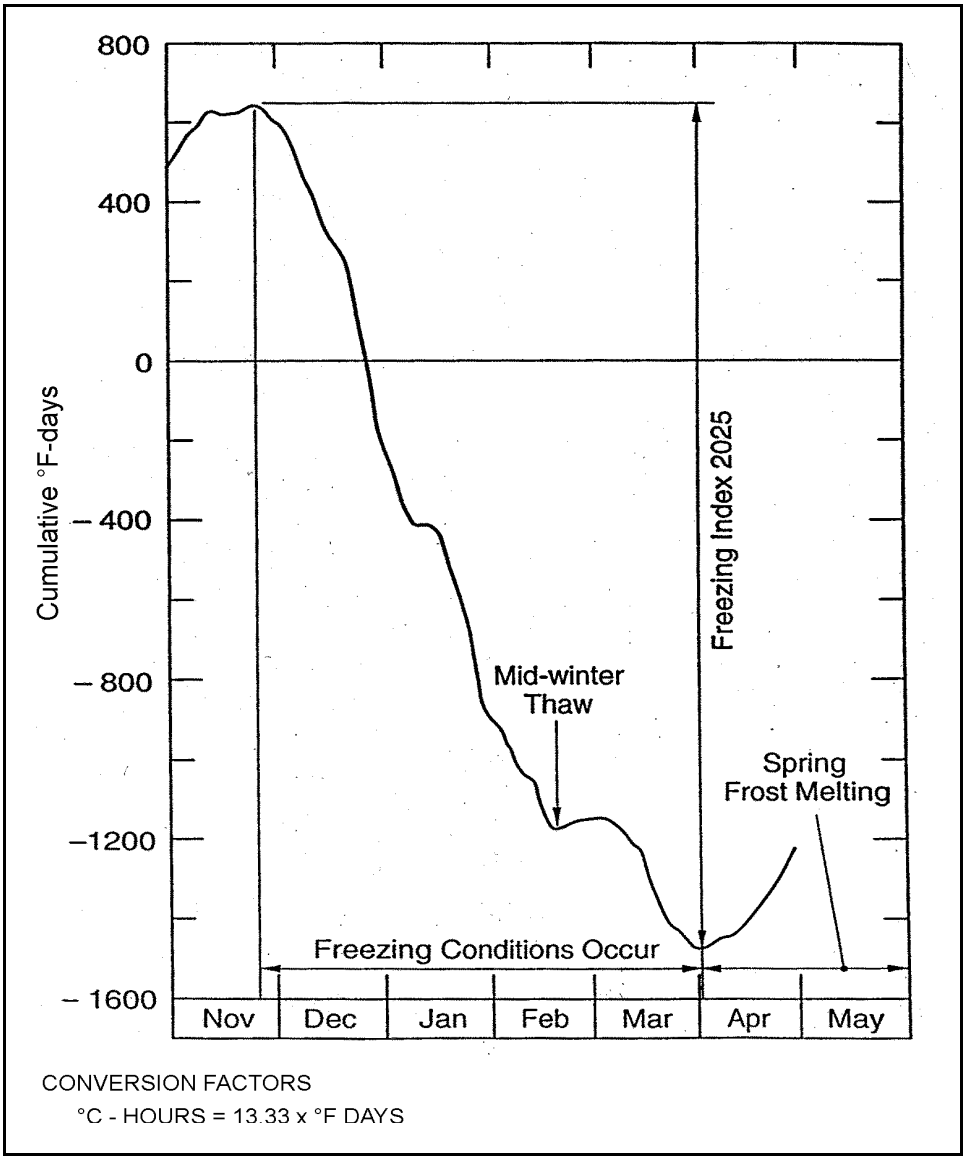


Figure 20-2. Determination of freezing index

CONVERSION FACTORS
°C - HOURS = 13.33 x °F DAYS

Figure 20-3. Distribution of design freezing indexes in North America

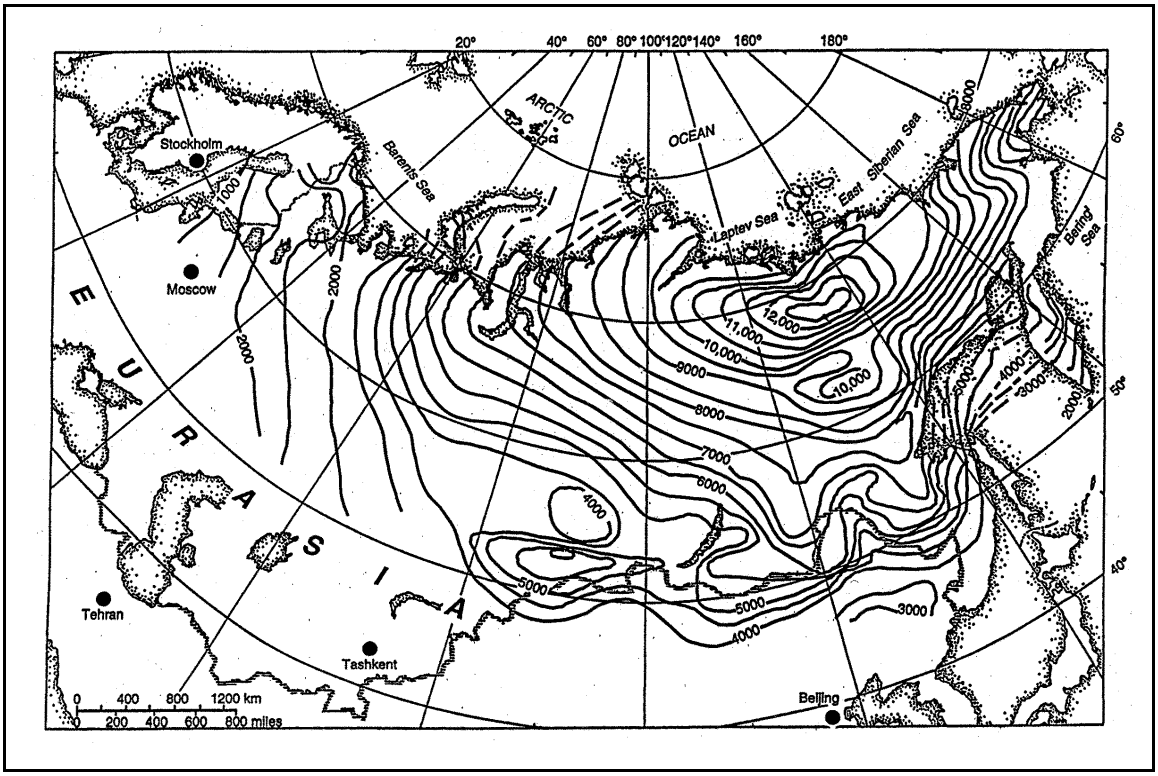


Figure 20-4. Distribution of mean freezing indexes in Northern Eurasia

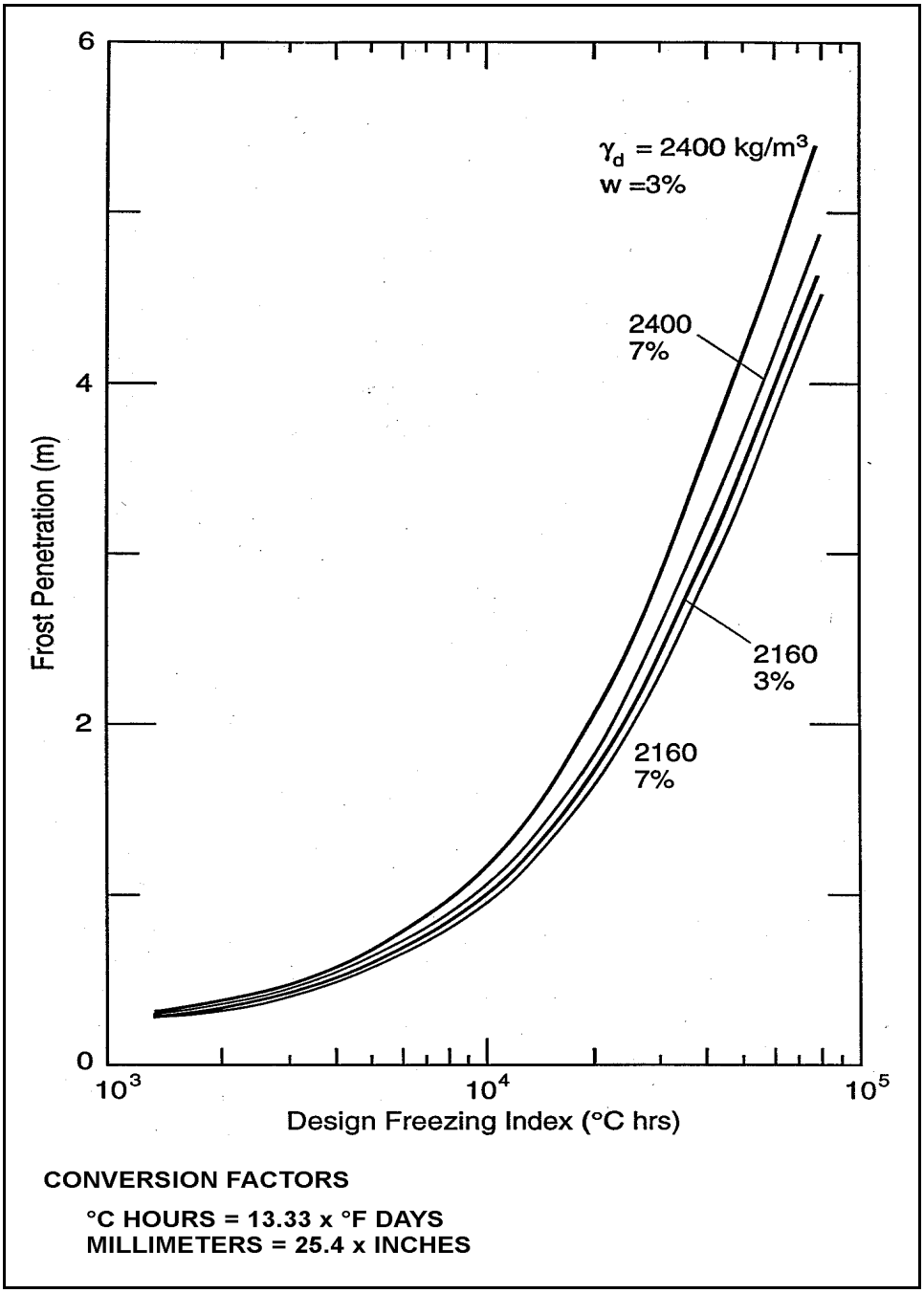


Figure 20-5. Frost penetration beneath pavements

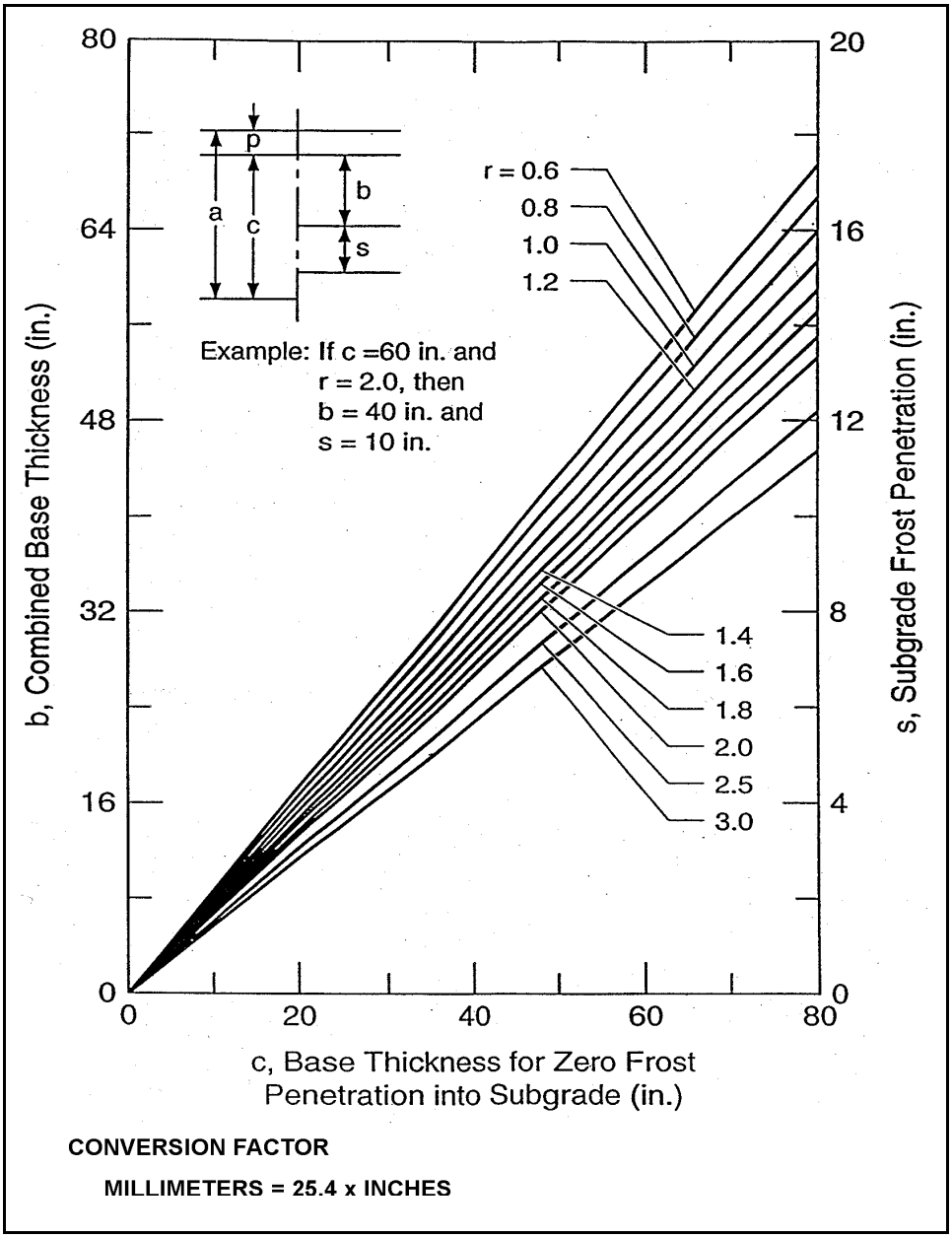


Figure 20-6. Design of combined base thickness for limited subgrade frost penetration

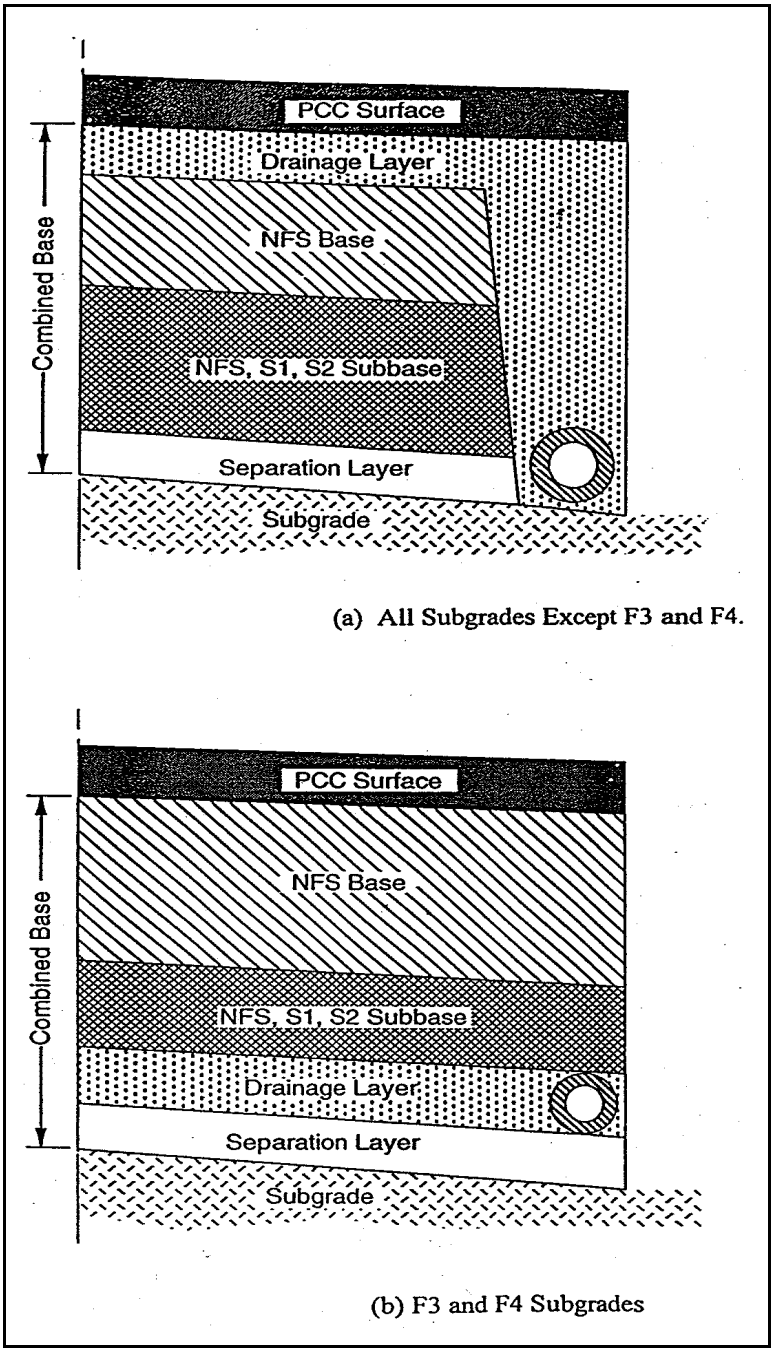


Figure 20-7. Placement of drainage layer in frost areas

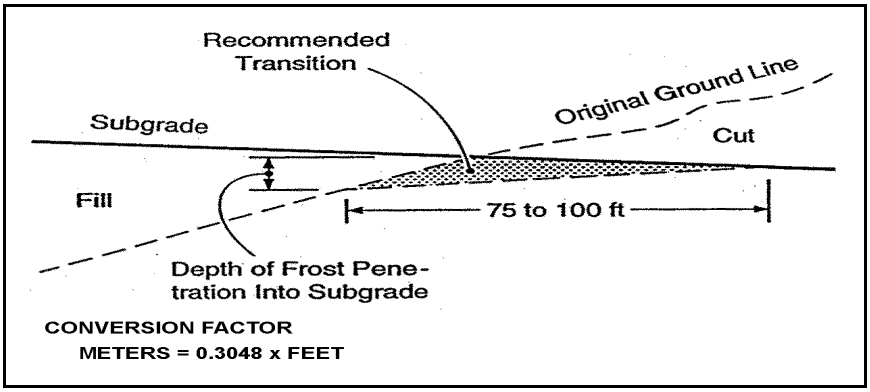
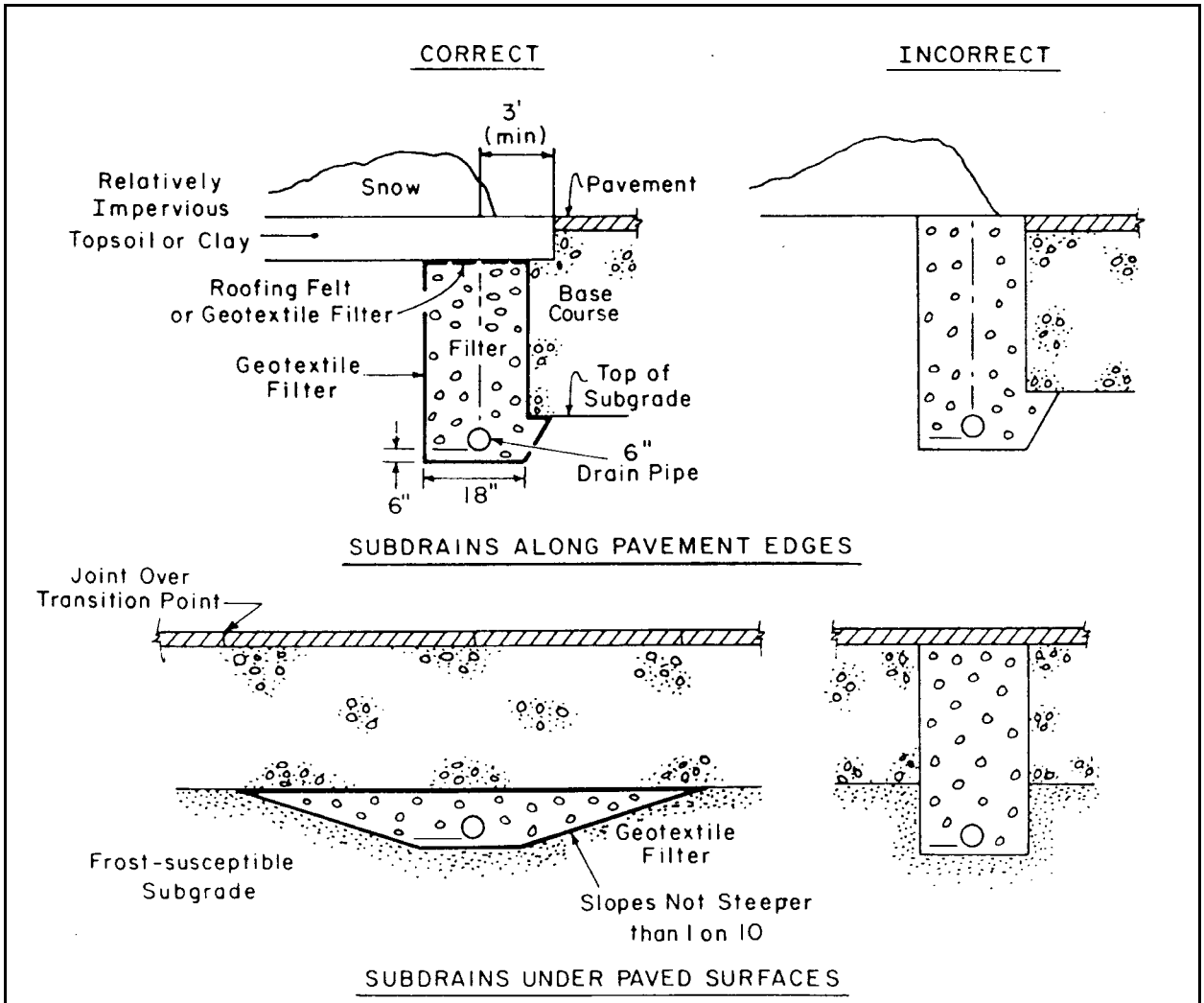


Figure 20-8. Tapered transition used where embankment material differs from natural subgrade in cut



NOTES

1. For additional details on design and depth of subdrains and filter courses see TM 5-820-2 (AFM 88-5, Chap. 2).

2. Granular or geotextile fabric filter may be necessary between base course and subgrade (para. 5-4).

3. Upper 4 inches of base course must have free-draining characteristics (para. 5-1).

Figure 20-9. Subgrade details for cold regions

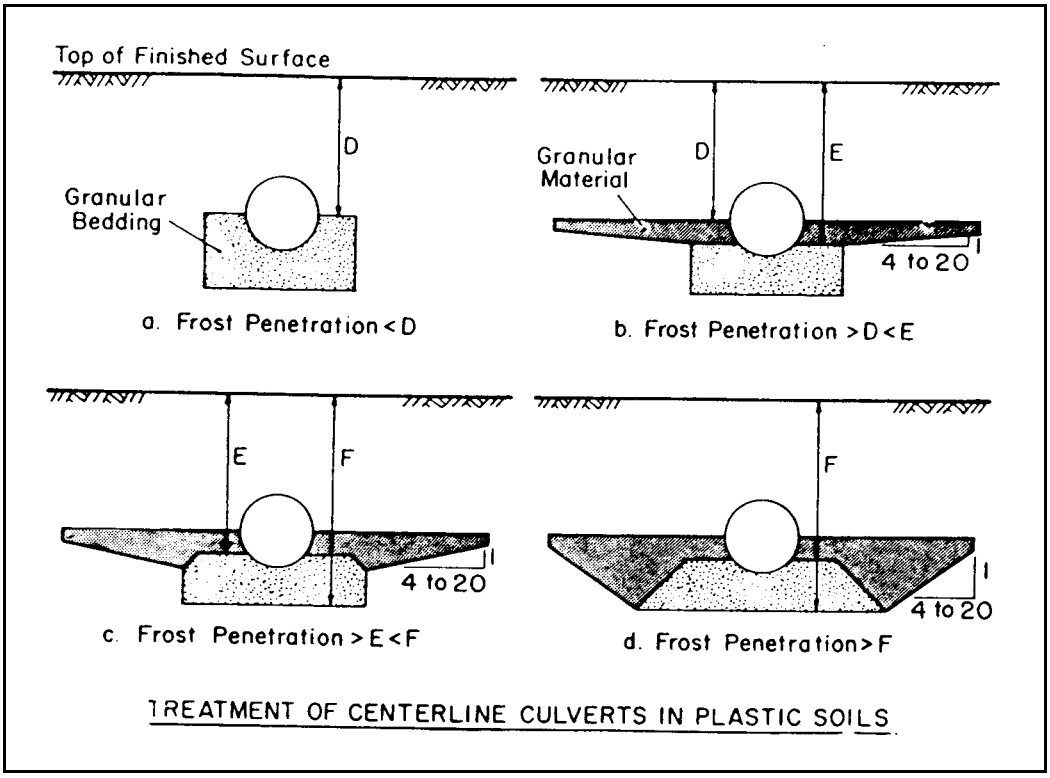
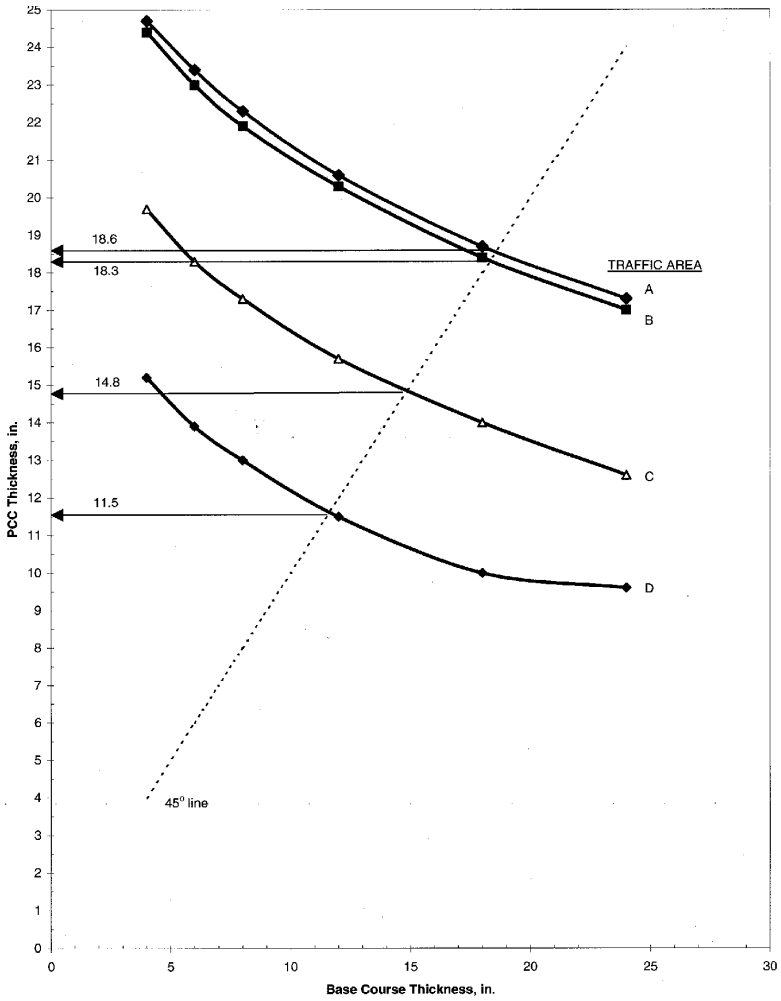
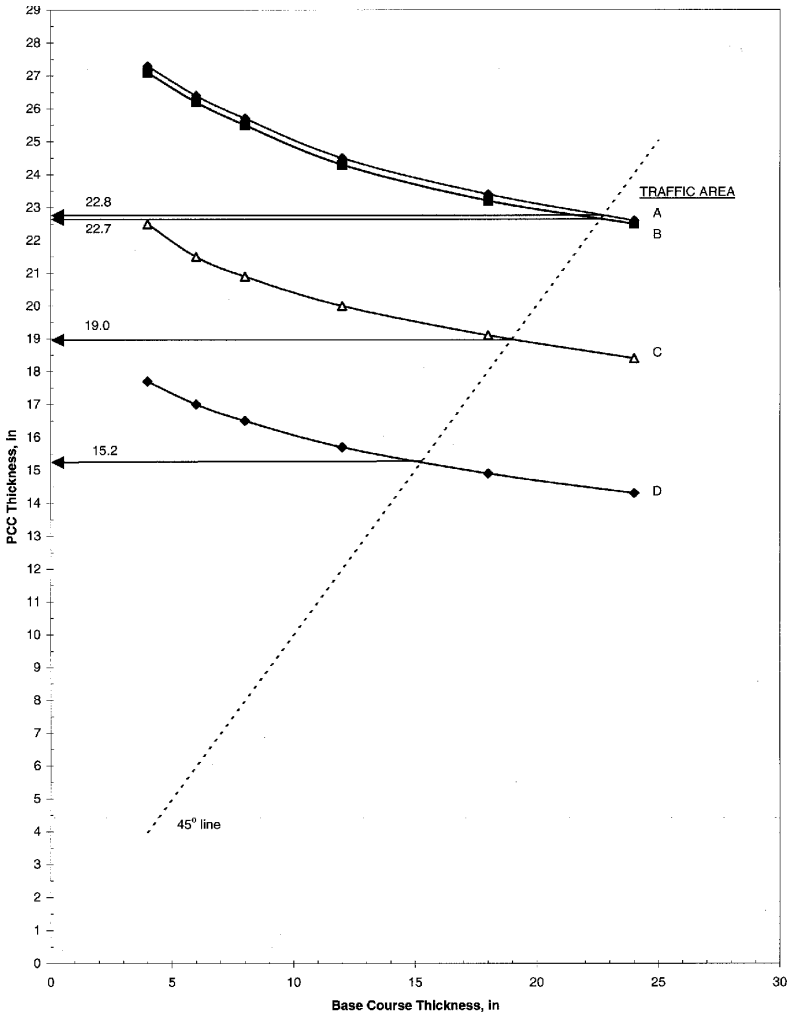


Figure 20-10. Transitions for culverts beneath pavements



CONVERSION FACTOR
MILLIMETERS = 25.4 x INCHES

Figure 20-11. Relationship between base course thickness and PCC thickness for example 1



CONVERSION FACTOR
MILLIMETERS = 25.4 x INCHES

Figure 20-12. Relationship between base course thickness and PCC thickness for example 2

CHAPTER 21

IMPROVING SKID RESISTANCE/REDUCING HYDROPLANING POTENTIAL OF RUNWAYS

1. GENERAL. This chapter presents procedures for improving skid resistance and reducing hydroplaning tendency of runways. It applies to the Army and Air Force. Navy guidance is contained in NAVFAC Criteria Office Memorandum dated 24 March 1999, Subject: INTERIM TECHNICAL GUIDANCE (ITG)-SKID RESISTANCE CRITERIA FOR AIRFIELD PAVEMENTS.

a. Skid resistance is the resistance to sliding by aircraft tires on a pavement surface. Skid resistance is related to the frictional resistance of the pavements. A high coefficient of friction is indicative of high skid resistance. Low friction resistance may result from polishing of the surface aggregate, rubber buildup, improper seal coating, or poor drainage.

b. Hydroplaning occurs when a tire looses contact with the surface as a result of the buildup of water pressure in the tire-ground contact area. The potential for hydroplaning is a function of speed, water depth, pavement texture, tire inflation pressure, and tread design.

c. Procedures for conducting friction testing and an approved equipment list are contained in Federal Aviation Administration Advisory Circular, AC 150/5320-12C, "Measurement, Construction, and Maintenance of Skid-Resistant Airport Pavement Surfaces".

2. IMPROVING RUNWAY FRACTION CHARACTERISTICS. New, reconstructed, or resurfaced runways must be grooved except when resurfaced with a Porous Friction Surface (PFS). The grooving is required to provide an acceptable surface for safe operation of aircraft. Friction characteristics of existing runways need to be improved when tests indicate the surface has a potential for hydroplaning. Considerations for improving the friction characteristics include grooving, Porous Friction Surfaces, retexturing, improving runway slopes, and rubber removal. Table 21-1, developed by the National Aeronautics and Space Administration, provides guidance on friction ratings for friction measuring equipment. Improving friction characteristics of existing runways should be considered when friction ratings are less than Good.

Table 21-1
Nominal Test Speed, 65 km/h (40 mph)

	Ground Vehicle Readings									
Braking Action Level	RCR	Grip-Tester	James Brake Index	MU-Meter	Surface Friction Tester	Runway Friction Tester	BV-11 Skiddo-Meter	Decel Meters	Locked Wheel Devices	ICAO INDE
Good	>17	>0.49	.0.58	>0.50	>0.55	>0.51	>0.59	>0.53	>0.51	5
Fair	12-17	0.34-0.49	0.40-0.58	0.35-0.50	0.38-0.54	0.35-0.51	0.42-0.59	0.37-0.53	0.37-0.51	3-4
Poor	6-11	0.16-0.33	0.20-0.39	0.15-0.34	0.18-0.37	0.18-0.34	0.21-0.41	0.17-0.36	0.18-0.36	2-3
NL	≤5	≤0.14	≤0.17	≤0.14	≤0.16	≤0.15	≤0.19	≤0.16	≤0.15	

a. Sawcut grooving is a proven way of reducing the hydroplaning potential of runways. Grooves drain water laterally, permit water to escape under tires, prevent buildup of surface water, and increase the texture of the pavement.

(1) Pavement condition. Grooves should only be applied to structurally adequate pavement free from defects. Pavements requiring corrective action should be overlaid or rehabilitated prior to grooving. Porous Friction Surfaces should not be grooved.

(2) Grooving flexible pavements. Studies indicate that grooving of flexible pavements does not cause any appreciable deterioration of the pavement nor has maintenance effort been increased. No problems have occurred from ice and snow removal. Minor distortion and creeping of grooves have been observed, but these conditions have not required maintenance or adversely affected pavement performance.

(3) Groove pattern. Grooves will be continuous for the entire length of the usable runway and perpendicular to the centerline. Grooves should terminate within 1.5 to 3 meters (5 to 10 feet) of pavement edge to allow for operation of grooving equipment. The standard groove configuration is 6 millimeters (¼ inch) ± 2 millimeters (± 1/16 inch) in depth by 6 millimeters (¼ inch) + 2 millimeters – 0 millimeters (+1/16 inch, -0 inch) in width by 38 millimeters (1 1/2 inch) – 3 millimeters + 0 millimeters (-1/8 inch, + 0 inch) center-to-center spacing. The recommended groove detail for airfield pavements is shown in Figure 21-1.

(4) Limitations. Do not groove within 6 inches (± 3 inches) of the runway centerline. Do not groove within 152 millimeters (6 inches) of transverse joints or working cracks, through compression seals, in-runway lighting fixtures or similar items, or the first 3 meters (10 feet) either side of an arresting barrier cable which requires hook engagement for operation. There is no need for grooving on either side of barrier cables that are placed on overruns.

b. Porous Friction Surfaces. A porous friction course is an open-graded, free draining asphalt mixture that can be placed on an existing pavement to minimize hydroplaning and to improve skid resistance. A PFS is placed in a layer varying from 19 to 25 millimeters (3/4 to 1 inch) thick. It has a coarse surface texture and is sufficiently porous to permit internal drainage as well as along the surface. Existing pavements should be in good condition before placing the mix. Concerns with PFS include rubber buildup that might prevent internal drainage, possible freezing of water trapped in voids, and loss of expertise in designing and constructing these surfaces. PFS should not be placed within 3 meters (10 feet) of an arresting gear cable.

c. Retexturing. Retexturing of runways has been successfully accomplished using several types of equipment. Contact the MAJCOM Pavements Engineers for guidance on Air Force projects and the TSMCX on Army projects.

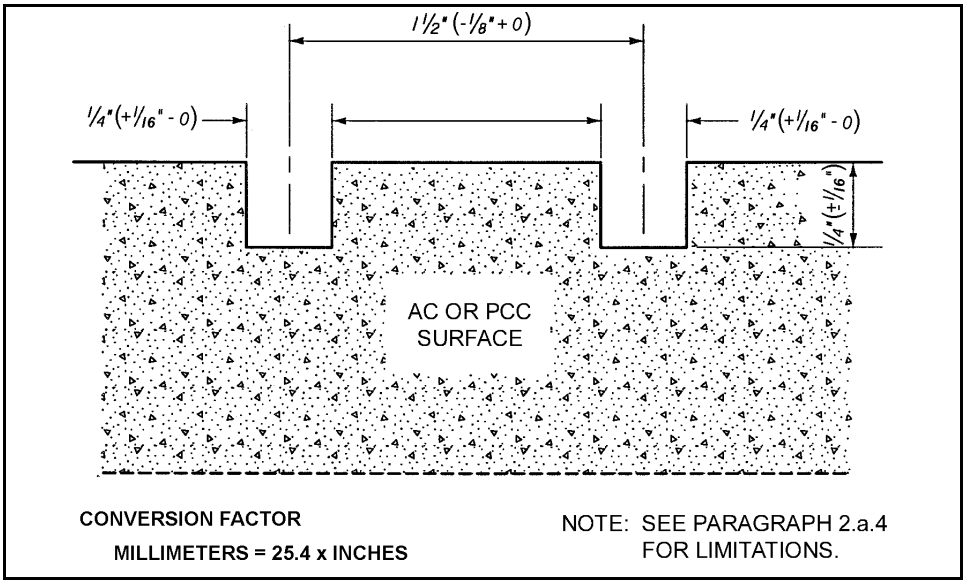


Figure 21-1. Groove configuration for airfields

APPENDIX A

REFERENCES

GOVERNMENT PUBLICATIONS

United Facility Criteria

Efforts were begun in FY00 to unify all Army, Navy, and Air Force design and construction technical criteria. The United Facility Criteria (UFC) number, old corresponding document number, and title for those documents referenced in this publication that now have UFC numbers are as follows:

UFC 3-260-01 (TM 5-803-7/ AFMAN 32-1123(I)/NAVFAC P-971)	Airfield and Heliport Planning and Design
UFC 3-260-03 (TI 826-01/ AFMAN 32-1121V1(I)/ NAVFAC DM 21.7)	Airfield Pavement Evaluation

Departments of the Army, Navy, and the Air Force

TM 5-803-4	Planning of Army Aviation Facilities
TM 5-820-1/AFM 88-5, Chap. 1	Surface Drainage Facilities for Airfields and Heliports
TM 5-820-2/AFM 88-5, Chap. 2	Subsurface Drainage Facilities for Airfields
TM 5-818-1/AFM 88-3, Chap. 7	Soils and Geology: Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures)
TM 5-818-7	Foundations in Expansive Soils
TM 5-818-8/AFJMAN 32-1030	Engineering Use of Geotextiles
TM 5-820-1/AFM 88-5, Chap. 1	Surface Drainage Facilities for Airfields and Heliports
TM 820-3/AFM 880-5, Chap. 3	Drainage and Erosion Control Structures for Airfields and Heliports
TM 5-822-12	Design of Aggregate Surfaced Roads and Airfields
TM 5-822-14/AFJMAN 32-1019	Soil Stabilization for Pavements
TM 5-822-7/AFM 88-6, Chap. 8	Standard Practice for Concrete Pavements

TM 5-822-8/AFM 88-6, Chap. 9	Bituminous Pavements Standard Practice
TM 5-825-1/AFMAN 32-8008, Vol 1	General Provisions for Airfield/Heliport Pavement Design, Appendix D, Operations Plan for Runway Friction Characteristics Testing
TM 5-826-1/AFM 88-24, Chap. 1	Army Airfield Pavement Evaluation Concepts
TM 5-826-2/AFM 88-24, Chap. 2	Airfield Flexible Pavement Evaluation
TM 5-826-3/AFM 88-24, Chap. 3	Airfield Rigid Pavement Evaluation
TM 5-852-6/AFM 88-19, Chap. 6	Arctic and Subarctic Construction, Calculation Methods for Determination of Depth of Freeze and Thaw
EI 02C013/AFJMAN 32-1013/ NAVFAC P971	Airfield and Heliport Planning Criteria
EI 02C202/AFJMAN 32-1016	Subsurface Drainage for Pavements
EI 02C029/AFJMAN 32-1029	Asphalt Concrete Pavements Standard Practice
ETL 1110-3-475	Roller Compacted Concrete Pavement Design and Construction
TI 822-08/AFMAN 32-1131 V8(I)/DM 21.11	Standard Practice Manual for Flexible Pavements
FM 5-430-00-2/AFJPAM 32-8013, Vol II	Planning and Design of Roads, Airfields and Heliports in the Theater of Operations
AFM 86-2	Standard Facility Requirements
AFR 86-5	Planning Criteria and Waivers for Airfield Support Facilities
AFR 86-14	Airfield and Heliport Planning Criteria
AFR 93-5	Airfield Pavement Evaluation Program
AF ETL 98-5	C-130 and C-17 Contingency and Training Airfield Dimensional Criteria
MIL-HDBK-1021/2	General Concepts for Airfield Pavement Design
NAVFAC DM 21.06	Airifield Pavement Design for Frost Conditions and Subsurface Drainage
NAVFAC DM 21.09	Skid Resistant Runway Surfaces

NAVFAC DM 5.04	Civil Engineering - Pavements
NAVFAC DM 7.01	Soil Mechanics
NAVFAC NFGS 02522	Joints, Reinforcement, and Mooring Eyes in Concrete Pavement
NAVFAC NFSC02562	Resealing of Joints in Rigid Pavements
NAVFAC P-272	Design Definitives for Navy and Marine Corps Shore Facilities
NAVFAC P-971	Airfield and Heliport Planning and Design

General Services Administration

Fed. Spec. SS-S-200E	Sealing Compounds, Two-Component, Elastomeric, Polymer Type, Jet-Fuel-Resistant, Cold-Applied
Federal Aviation Administration Advisory Circular AC 150/5320-12C	Measurement, Construction, and Maintenance of Skid-Resistance Airport Pavement Surfaces
Corps of Engineers CEGS 02721	Subbase Courses

U.S. Army Engineers, Waterways Experiment Station, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199

CRD-C 21	Method of Test of Modulus of Elasticity of Concrete in Flexure
CRD-C 525	Corps of Engineers Test Method 4, Evaluation of Hot-applied Joint Sealants for Bubbling due to Heating
CRD-C 653	Standard Test Method for Determination of Moisture Density Relations of Soils
CRD-C 654	Standard Test Method for Determining the California Bearing Ratio of Soils
CRD-655	Standard Test Method for Determining the Modulus of Soil Reaction
CRD-656	Standard Test Method for Determining the California Bearing Ratio and for Sampling Pavement by the Small Aperture Method

Nongovernment Publications

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

A 82	Cold-Drawn Wire for Concrete Reinforcement
A 184	Fabricated Deformed Steel Bar Mats for Concrete Reinforcement
A 185	Welded Steel Wire Fabric for Concrete Reinforcement
A 416	Uncoated Seven-Wire Stress-Relieved Strand for Prestressed Concrete
A421	Uncoated Stress-Relieved Wire for Prestressed Concrete
A497	Welded Deformed Steel Wire Fabric for Concrete Reinforcement
A 615	Deformed and Plain Billet-Steel Bars for Concrete Reinforcement
A 616	Rail-Steel Deformed and Plain Bars for Concrete Reinforcement
A617	Specification for Axle-Steel Deformed and Plain Bars For Concrete Reinforcement
C 29	Test for Unit Weight and Voids in Aggregates
C 33	Specification for Concrete Aggregates
C 78	Flexural Strength of Concrete (using Single-Beam with Third Point Loading)
C 88	Test for Soundness of Aggregate by use of Sodium Sulfate or Magnesium Sulfate
C 127-88	Test for Specific Gravity and Absorption of Coarse Aggregate
C 128-88	Test for Specific Gravity and Absorption of Fine Aggregate
C 131-89	Test for Resistance to Abrasion of Small Size Coarse Aggregate by Use of the Los Angeles Machine

C 150	Portland Cement
C 289	Test Method for Potential Reactivity of Aggregates (Chemical Method)
C-294	Descriptive Nomenclature of Constituents of Natural Mineral Aggregates
C 617-85	Capping Cylindrical Concrete Specimens
C 618	Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete
C 977	Test method for Sulfide Resistance of Ceramic Decorations on Glass
C 989	Specification for Ground Iron Blast-Furnace Slag for Use in Concrete and Mortars
D 5-86	Penetration of Bituminous Materials
D 36-86	Softening Paint of Bitumen
D 75-87	Sampling Aggregates
D 242-85	Specifications for Mineral Filler for Bituminous Paving Mixtures
D 422-63	Particle Size Analysis of Soils
D 560-89	Freezing and Thawing Tests of Compacted Soil Cement Mixtures
D 558	Test Method for Moisture-Density Relations of Soil Cement Mixtures
D 946	Penetration-Graded Asphalt Cement for Use in Pavement Construction
D 977	Emulsified Asphalt
D 1140-54	Amount of Material in Soils Finer than the No. 200 Sieve
D 1190-97	Specifications for Concrete Joint Sealer, Hot-Applied Elastic Type

D 1196	Nonrepetative Static Plates Load Tests of Soils and Flexible Pavement Components for Use in Evaluation and Design of Airport and Highway Pavements
D 1452-80	Soil Investigations and Sampling by Auger Borings
D 1556-90	Density and Unit Weight of Soil in Place by the Sand Cone Method
D 1557-78	Moisture Density Relations of Soils and Soil Aggregate Mixtures
D 1559-89	Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus
D 1560-81	Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus
D 1561-81	Preparation of Bituminous Mixture Test Specimens by Means of California Kneading Compactor
D 1586-84	Penetration Test and Split-Barrell Sampling of Soils
D 1587-83	Thin-Walled Tube Sampling of Soils
D1632-87	Making and Curing Soil-Cement Compression and Flexure Test Specimens in the Laboratory
D 1633-84	Compressive Strength of Molded Soil-Cement Cylinders
D 1883-87	California Bearing Ratio of Laboratory Compacted Soils
D 2026	Cutback Asphalt (Slow-Curing Type)
D 2027	Cutback Asphalt (Medium-Curing Type)
D 2028	Cutback Asphalt (Rapid-Curing Type)
D 2397	Specifications for Cationic Emulsified Asphalt
D 2628-98	Specification for Preformed Polychloroprene Elastomeric Joint Seals for Concrete Pavements

D 2835-89	Lubricant for Installation of Preformed Compression Seals in Concrete Pavements
D 2922-81	Density of Soil and Soil-Aggregate In-Place by Nuclear Methods
D 2937-83	Density of Soil In-Place by the Drive-Cylinder Method
D 2940	Specification for Graded Aggregate Material for Bases or Subbases for Highways or Airports
D 3017-78	Moisture Content of Soil and Soil Aggregate In-Place by Nuclear Methods
D 3202	Recommended Practice for Preparation of Bituminous Mixture Beam Specimens by Means of the California Kneading Compactor
D 3405-96	Specifications for Joint Sealants, Hot-Applied, for Concrete and Asphalt Pavements
D 3406-95	Specifications for Joint Sealants, Hot-Applied, Elastomeric-Type, for Portland Cement Concrete Pavements
D 3381-83	Viscosity-Graded Asphalt Cement for Use in Pavement Construction
D 3515	Specification for Hot-Mixed, Hot-Laid Bituminous Paving Mixtures
D 3569-95	Specifications for Joint Sealants, Hot-Applied, Elastomeric, Jet-Fuel-Resistant-Type for Portland Cement Concrete Pavements
D 3581-96	Specifications for Joint Sealants, Hot-Applied, Elastomeric, Jet-Fuel-Resistant Type for Portland Cement and Tar-Concrete Pavements
D 5893-96	Specifications for Cold-Applied, Single Component Chemically Curing Silicon Joint Sealant for Portland Cement Concrete Pavement
D 4318-84	Liquid Limit, Plastic Limit, and Plasticity Index of Soils
D 4429-84	Bearing Ratio of Soils in Place

D 5340-94	Airport Pavement Condition Index Surveys
E 11-87	Wire Cloth Sieves for Testing Purposes

McLeod, N. W. "Using Paving Asphalt Rheology to Impair or Improve Asphalt Pavement Design and Performance," *Asphalt Rheology: Relationship to Mixture*, ASTM STP 941, O. E. Briscoe, Ed., American Society for Testing and Materials, Philadelphia, 1987.

McLeod, N. W. "A 4-Year Survey of Low-Temperature Transverse Pavement Cracking on Three Ontario Test Roads," *Proceedings, Association of Asphalt Paving Technologists*, Vol 41, 1972.

Asphalt Institute, Asphalt Institute Building, College Park, MD 20740

MS-2	Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types

American Concrete Institute, P.O. Box 19150, Redford Station, Detroit, MI 48219

ACI 318	Building Code Requirements for Reinforced Concrete
ACI 544.IR-82	State of the Art Report on Fiber Reinforced Concrete

American Association of State Highway and Transportation Officials (AASHTO), 444 North Capitol Street, N.W., Suite 249, Washington, DC 20001

PP6	Grading or Verifying the Performance Grade of an Asphalt Binder

APPENDIX B

AIRFIELD/HELIPORT DESIGN ANALYSIS OUTLINE

B-1. INTRODUCTION.

a. Purpose of Report. To describe the project design in sufficient detail for review, evaluation, and documentation of the design.

b. Scope of Report.

(1) State the design phase that the report covers.

(2) List topics discussed in report.

c. Project Description.

(1) Extent of proposed construction (new construction; runway extension; apron expansion; overlay; rehabilitation and repair; upgrade lighting; drainage, security, and navigational aids improvements; etc.)

(2) Purpose of proposed construction or improvements.

(3) Types and amount of construction activities (demolition, excavation and embankment, grading, paving, patching, marking, fencing, seeding, etc.)

d. Project Authorization (reference authorization letter, directive, or other pertinent items, with dates).

e. Design Criteria. (Reference the key criteria and directives used in the design, with dates. Since criteria are constantly being revised and updated, the key criteria should be documented so that the basis of the design can become a historical record.)

(1) Correspondence and Directives.

(2) Engineering Technical Letters (ETLs).

(3) Technical Manuals (TM's and AFMs).

(4) Engineering Circulars (ECs).

(5) Pavement Evaluations/Condition Surveys.

(6) Computer Programs.

(7) Other special design criteria.

B-2. SITE DESCRIPTION.

a. Location (location map with graphical scale).

(1) Existing airfield/heliport facilities (layout, type, etc.)

(2) Location of proposed project with respect to existing facilities, utilities, or improvements.

(3) Extent of proposed construction (size, dimensions, etc.).

b. Topography/Drainage of Site.

(1) Topography (hilly, rolling, flat, terrace, floodplain, etc.).

(2) Surface drainage (characteristics and direction).

(3) Subsurface drainage (characteristics, groundwater conditions and elevations, including seasonal variations).

(4) Existing surface and subsurface drainage facilities (type, location, capacity, condition, etc.).

c. Climate (use National Oceanographic and Atmospheric Administration or Military installation's weather service center for climatological data where available).

(1) Temperatures (especially with reference to frost condition and design air freezing index).

(2) Rainfall (particularly with respect to its effect on construction operations).

(3) Seasonal variations.

d. Vegetation (wooded, open, brush, cultivated fields).

e. Geology.

(1) Sequence and character of surface and near-surface deposits. Soil overburden (glacial, stream, loess deposits, etc.).

(2) Rock outcroppings.

B-3. FIELD INVESTIGATIONS.

a. Subgrade explorations (type of investigations, number, locations, depth, samples obtained).

b. Borrow explorations for fill (type of investigations, number, locations, depth, samples obtained).

c. Availability of construction materials (type of material, location; name and description of pits, quarries, or other sources; samples obtained).

(1) Sand and gravel deposits.

(2) Aggregates (base-course, concrete, and bituminous mixtures).

(3) Cementitious materials (portland cement, fly ash, and asphalt; type; class; grade).

(4) Water.

d. Evaluations of Existing Pavements (describe all evaluations conducted).

(1) Destructive.

(2) Nondestructive.

B-4. TESTING.

a. Laboratory (describe lab testing conducted).

b. Field (describe field testing conducted).

B-5. RESULTS OF INVESTIGATIONS AND TESTING.

a. Material Characterization.

(1) Subgrade characteristics (soil classifications, unit weights, moisture-density relationships, gradations, Atterberg limits, CBR and/or modulus of subgrade reaction, permeability, etc.).

(2) Characteristics of borrow (same as above).

(3) Characteristics of base and subbase material (same as above).

(4) Characteristics of pavement surfacing materials.

b. Groundwater and Subsurface Drainage Conditions.

c. Frost Conditions (where applicable).

(1) Frost susceptibility of materials (based on gradation and frost classification, laboratory freeze tests, heave measurements, observations or ice lense formations in test pits, etc.).

(2) Frost penetration (based on field observations or design air-freezing index and modified Berggen equation).

(3) Moisture availability.

(4) Mean annual temperature.

(5) Duration of freezing season.

(6) Number of freeze-thaw cycles.

d. Existing Pavement Evaluation/Characterization.

e. Summarize Adopted Design Parameters.

B-6. PAVEMENT THICKNESS DESIGN CRITERIA.

a. Load (include copy of Airfield/Heliport Mission List).

(1) Airfield/heliport/helipad class or type.

(2) Design aircraft or aircraft mix.

(3) Pass levels.

(4) Mission operational weights.

(5) Traffic areas.

B-7. PAVEMENT THICKNESS DESIGN.

a. Flexible Pavement Design (for each pavement feature).

(1) Design of curves or computer programs used.

(2) Layers (thicknesses, type, design CBR-values).

(3) Compaction requirements.

(4) Proof rolling requirements.

(5) Bituminous mixture requirements (gradation, stability).

(6) Selection of AC grade.

(7) Tack and prime coat requirements (type, grade).

(8) Grooving requirements.

b. Rigid Pavement Design (for each pavement feature).

(1) Design curves or computer programs used.

(2) Flexural strength.

(3) Layers (thicknesses, type, subgrade modulus values).

(4) Compaction requirements.

(5) Joint design (spacing, type).

(6) Joint sealant (type).

(7) Grooving requirements.

c. Overlay design (for each pavement feature).

(1) Type of design (flexible, rigid, bonded, unbonded).

(2) Existing paving system characteristics.

(3) Design curves or computer programs used.

(4) Overlay layers (thicknesses, type, etc.).

(5) Surface preparation requirements.

d. Frost Design (for each pavement feature).

(1) Design methodology limited subgrade frost penetration (LSFP) or reduced subgrade strength (RSS).

(2) Design air-freezing index (for LSFP method).

(3) FASSI or FAIR value (for RSS method).

(4) Design curves or computer program used.

(5) Layers (number, thickness, type).

(6) Special subgrade, subbase, and base course preparation for frost design.

B-8. DRAINAGE DESIGN.

a. General Criteria.

b. Hydrology.

c. Surface Drainage (including drainage plans and profiles).

d. Subsurface Drainage.

B-9. PROPOSED GRADES.

a. Longitudinal (for each pavement feature).

b. Transverse (for each pavement feature).

B-10. AIRFIELD LIGHTING AND NAVAIDS IMPROVEMENTS.

B-11. CONSTRUCTION MATERIALS.

a. Rigid Pavement.

(1) Coarse aggregate (type, gradation, deleterious limits, wear, particle shape).

(2) Fine aggregate (type, gradation, deleterious limits).

(3) Cement (type).

(4) Fly ash (class).

(5) Admixtures (type).

(6) Curing compound (type).

(7) Dowels (size, type).

(8) Reinforcing (size, type).

(9) Joint filler.

(10) Joint seals (type).

b. Flexible Pavement.

(1) Aggregates (type, gradation, percent fractured faces, wear).

(2) Mineral filler.

(3) Asphalt cement (grade).

(4) Prime coat material (type, grade).

(5) Tack coat material (type, grade).

c. Base Courses.

(1) Graded crushed-aggregate base course (gradation, percent fractured faces, wear).

(2) Rapid draining base course (RDM or OGM gradation, percent fractured faces, wear).

(3) Separation layer (gradation, design CBR-value).

(4) Subbase course (gradation, design CBR-value).

d. Borrow Material.

e. Surface and Subsurface Drainage System.

(1) Pipe (size, type).

(2) Structure construction.

(3) Bedding material.

(4) Filter material.

(5) Manhole construction.

f. Pavement Marking Materials.

B-12. LIST OF REQUIRED WAIVERS.

a. Reference regulation document (title, page, para.).

b. State the regulation in violation.

c. State the reason the waiver is required.

B-13. COST ESTIMATES.

a. Capital costs.

b. Life-cycle costs.

APPENDIX C

RECOMMENDED CONTRACT DRAWING OUTLINE

FOR AIRFIELD/HELIPORT PAVEMENTS

The list of drawings that follows should be used as a guide. All drawings may not be needed for all jobs.

C-1. TITLE SHEET.

a. Project Title.

b. Location.

c. Year

d. Volume Number.

C-2. INDEX SHEET.

a. Listing of Sheet Names.

b. Assigned Sheet Numbers (in sequential order).

C-3. COMBINED TITLE/INDEX SHEETS.

C-4. LEGEND.

a. Civil.

b. Electrical.

c. Mechanical.

d. Architectural.

C-5. LOCATION/SITE PLAN.

a. Base Map with State (Vicinity) Map.

b. Project Location.

c. Contractor Access Routes.

d. Location of Base Gates and any Restrictions.

e. Borrow/Waste Areas.

f. Batch Plant Area.

g. Contractor's Staging and/or Storage Area.

h. Utility Hookup Locations.

I. General or Special Notes.

j. Concurrent Construction (Not in Contract).

C-6. PHASING PLAN AND DETAILS.

a. Location and Sequencing of Work Areas.

b. Scheduling for each Phase of Project.

c. General Listing of Tasks to be Performed under each Phase.

d. Concurrent Construction that may Affect each Phase.

e. Location and Type of Area Control (Security) Measures.

(1) Temporary Barricades and Fencing.

(2) Obstruction Lighting.

(3) Temporary Pavement Markings (Closure Markings).

f. Traffic Circulation (Aircraft and Vehicular).

g. Special Notes.

(1) Security Measures.

(2) Contractor's Housekeeping Measures.

(3) Controls on Contractor's Traffic.

C-7. HORIZONTAL AND VERTICAL CONTROLS.

a. Layout.

b. Bench Marks (USGS Datum) with only one Master Bench Mark.

c. Control Stationing.

d. Horizontal Control (Coordinates).

C-8. GEOMETRIC LAYOUT PLAN (OPTIONAL).

a. Curve Data.

b. Control Stationing.

c. Geometric Layout.

C-9. BORING LOCATION PLAN AND BORING LOG DATA.

C-10. PAVEMENT REMOVAL PLAN.

a. Pavement Removal Limits (Dimensions, Stationing, etc.).

b. Type and Thickness of Pavement Removed.

c. Utilities and Structures Affected by the Removal.

(1) Manholes.

(2) Barrier Arresting Cables.

(3) Blast Deflectors.

(4) Runway/Taxiway Lighting.

(5) Communication Cables.

(6) Water/Sewer Lines.

(7) In Ground Aircraft Support Systems.

d. Special Notes Regarding Removals.

e. Location of Removal Sections.

C-11. REMOVAL SECTIONS AND DETAILS. Sections should be specific, not general or typical. Show several sections. Show new sections for changes in pavement type, thickness, or any other condition that has an impact on pavement construction. Sections should be complete both laterally and vertically for the entire pavement structure including subgrade preparation.

a. Removal Limits (Lateral Dimensions, Depth).

b. Show Make-Up of the Existing Pavement.

(1) Pavement Type and Thickness.

(2) Joint Type (Doweled, Tied, Contraction, etc.).

(3) Existing Reinforcing (if any).

c. Special Notes.

(1) Equipment Type/Size.

(2) Procedures.

(3) Housekeeping.

(4) Other.

C-12. EXISTING UTILITIES PLAN.

a. Show Existing Utility Locations and Type.

b. Show Pavement Penetrations.

C-13. PAVING PLAN.

a. Thickness.

b. Type.

c. Location.

d. Location of Section Cuts.

e. Stationing.

f. Dimensions.

C-14. PAVING SECTIONS. Make the sections specific. Do not overuse "Typical Sections." Cut a section wherever there is a change from one pavement section to another in any direction and on all pavement edges. The same section may be referenced numerous places on the plan sheets, but each location must be marked and properly annotated. Remember, only by including everything in the plans can the design be built as envisioned. One hour spent by the designer will save several hours work by the field engineer.

a. Include the entire paving section from surface through subgrade.

(1) Thickness of Surface.

(2) Prime Coat Requirements.

(3) Thickness of Bases and Subbases.

(4) Thickness of Drainage Layer.

(5) Depth and Type of Subgrade Preparation.

b. Jointing Locations and Type.

c. Surface Grades/Slope.

d. Subsurface Drainage/Subdrain Provisions.

C-15. PLAN AND PROFILE SHEETS.

a. Plan.

(1) Outline of Pavement.

(2) Utilities.

(3) Stationing.

(4) Geometrics.

b. Profile.

(1) Stationing.

(2) Elevations (new and existing).

(3) Vertical Curve Data.

(4) Utility Depth and Location.

C-16. GRADING AND DRAINAGE PLANS.

a. Contours (new and existing).

b. Surface and Subsurface Drainage System Layouts, Structure Locations, Types, and Sizes.

c. Ditch Alignment.

C-17. GRADING SECTIONS.

a. Cut/Fill Requirements.

b. Topsoil Requirements.

C-18. PAVEMENT SURFACE ELEVATIONS.

a. Spot Elevation Plan (joint intersections or grid pattern).

b. Spot Elevation Schedule.

C-19. PAVEMENT JOINTING PLANS.

a. Legend with Joint Types.

b. Joint Location.

C-20. JOINT AND JOINT SEALANT DETAILS.

C-21. REINFORCING DETAILS.

a. Dowels.

b. Reinforcement.

c. Tie Bars.

d. Complete Pavement Joint Details.

C-22. SURFACE AND SUBSURFACE DRAINAGE SYSTEMS.

a. Profiles.

b. Schedules.

c. Details.

C-23. AIRFIELD REPAIR PLAN AND DETAILS.

C-24. PAVEMENT MARKING.

a. Plan.

b. Details.

C-25. AIRCRAFT MOORING AND GROUNDING POINTS.

a. Plan.

b. Details.

C-26. GROOVING PLAN AND DETAILS.

C-27. RUNWAY/TAXIWAY LIGHTING.

a. Plan.

b. Schedule.

c. Details.

C-28. MECHANICAL (FUEL).

a. Plans.

b. Profiles.

c. Schedules.

d. Details.

APPENDIX D

WAIVER PROCESSING PROCEDURES

D.1. Army:

D.1.1. Waiver Procedures:

D.1.1.1. Installation. The installation's design agent, aviation representative (Safety Officer, Operations Officer, and/or Air Traffic and Airspace AT&A Officer) and DEH Master Planner will:

D.1.1.1.1. Jointly prepare/initiate waiver requests.

D.1.1.1.2. Submit requests through the installation to the Major Command (MACOM).

D.1.1.1.3. Maintain a complete record of all waivers requested and their disposition (approved or disapproved). A list of waivers to be requested and those approved for a project should also be included in the project design analysis prepared by the design agent, aviation representative, or DEH Master Planner.

D.1.1.2. The MACOM will:

D.1.1.2.1. Ensure that all required coordination has been accomplished.

D.1.1.2.2. Ensure that the type of waiver requested is clearly identified as either "Temporary" or "Permanent." "Permanent Waivers" are required where no further mitigative actions are intended or necessary.

D.1.1.2.2.1. "Temporary Waivers" are for a specified period during which additional actions to mitigate the situation must be initiated to fully comply with criteria or to obtain a permanent waiver. Followup inspections will be necessary to ensure that mitigative actions proposed for each Temporary Waiver granted have been accomplished.

D.1.1.2.3. Review waiver requests and forward all viable requests to U. S. Army Aeronautical Service Agency (USAASA) for action. To expedite the waiver process, MACOMs are urged to simultaneously forward copies of the request to:

D.1.1.2.3.1. Director, U. S. Army Aeronautical Services Agency (USAASA), ATTN: ATAS-AI, 9325 Gunston Road, Suite N319, Fort Belvior, VA 22060-5582.

D.1.1.2.3.2. Commander, U.S. Army Safety Center (USASC), ATTN: CSSC-SPC, Bldg. 4905, 5th Ave., Fort Rucker, AL 36362-5363.

D.1.1.2.3.3. Director, U. S. Army Aviation Center (USAAVNC), ATTN: ATZQ-ATC-AT, Fort Rucker, AL 36362-5265.

D.1.1.2.3.4. Director, USACE Transportation Systems Center (TSMCX), ATTN: CENWO-ED-TX, 215 N 17th St., Omaha, NE 68102.

D.1.1.3. USAASA. USAASA is responsible for coordinating the following reviews for the waiver request:

D.1.1.3.1. Air traffic control assessment by USATCA.

D.1.1.3.2. Safety and risk assessment by USASC.

D.1.1.3.3. Technical engineering review by TSMCX.

D.1.1.3.4. From these reviews, USAASA formulates a consolidated position and makes the final determination on all waiver requests and is responsible for all waiver actions for Army operational airfield/airspace criteria.

D.1.2. Contents of Waiver Requests. Each request must contain the following information:

D.1.2.1. Reference to the specific standard and/or criterion to be waived by publication, paragraph, and page.

D.1.2.2. Complete justification for noncompliance with the airfield/airspace criteria and/or design standards. Demonstrate that noncompliance will provide an acceptable level of safety, economics, durability and quality for meeting the Army mission. This would include reference to special studies made to support the decision. Specific justification for waivers to criteria and allowances must be included as follows:

D.1.2.2.1. When specific site conditions (physical and functional constraints) make compliance with existing criteria impractical and/or unsafe; for example: the need to provide hangar space for all aircraft because of recurring adverse weather conditions; the need to expand hangar space closer to and within the runway clearances due to lack of land; maintaining fixed-wing Class A clearances when support of Class B fixed-wing aircraft operations are over 10% of the airfield operations.

D.1.2.2.2. When deviation(s) from criteria fall within a reasonable margin of safety and do not impair construction of long range facility requirements; for example, locating security fencing around and within established clearance areas.

D.1.2.2.3. When construction that does not conform to criteria is the only alternative to meet mission requirements. Evidence of analysis and efforts taken to follow criteria and standards must be documented and referenced.

D.1.2.3. The rationale for the waiver request, including specific impacts upon assigned mission, safety, and/or environment.

D.1.3. Additional Requirements:

D.1.3.1. Operational Factors. Include information on the following existing and/or proposed operational factors used in the assessment:

D.1.3.1.1. Mission urgency.

D.1.3.1.2. All aircraft by type and operational characteristics.

D.1.3.1.3. Density of aircraft operations at each air operational facility.

D.1.3.1.4. Facility capability (VFR or IFR).

D.1.3.1.5. Use of self-powered parking versus manual parking.

D.1.3.1.6. Safety of operations (risk management).

D.1.3.1.7. Existing NAVAIDS.

D.1.3.2. Documentation. Record all alternatives considered, their consequences, necessary mitigative efforts, and evidence of coordination.

D.2. Air Force:

D.2.1. Waivers to Criteria and Standards. When obstructions violate airfield imaginary surfaces or safe clearance criteria established in this manual, they must be analyzed to determine impact to aircraft operations. Facilities listed as permissible deviations (see attachment 14) do not require waiver if sited properly. Facilities constructed under previous standards should be documented as exemptions and programmed for replacement away from the airfield environment at the end of their normal life cycle, or when mission needs dictate earlier replacement. When documenting waiverable items, consider grouping adjacent supporting items with a controlling obstruction, or grouping related items such as a series of drainage structures, as one waiver. ***Example:*** The base operations building violates the 7H:1V Transitional Surface and apron clearance criteria. There are also four utility poles, a 36-inch tall fire hydrant, and numerous trees and shrubs located on the side of the building that is farthest away from the apron. These items are essential to provide architectural enhancement and utilities for this structure, but they also violate apron clearance criteria. Because these items are isolated from aircraft operations by the base operations building, they would not become a hazard to aircraft operations until the base operations building is relocated. Therefore, the base operations building is the controlling obstruction. Document

the base operations building as an exemption (constructed under previous standards) and develop one waiver request for all supporting structures to analyze impact to aircraft operations.

D.2.1.1. Temporary Waivers (One Year or Less). Establish temporary waivers for obstructions caused by construction activities by documenting the deviations and establishing a plan (including the issuance of NOTAMs or airfield advisories) that will allow safe operations during the temporary period. Coordinate the plan with airfield management, flying safety, and flight operations before asking the Wing Commander for approval.

D.2.1.2. Permanent Waivers. Use a permanent waiver when:

D.2.1.2.1. Natural geographical features violate criteria, and it is not economical or practical to remove them.

D.2.1.2.2. Existing facilities deviate from criteria but removal is not feasible.

D.2.1.2.3. Installation, construction, or erection of a required facility or equipment item according to criteria in this manual is not practical.

D.2.1.2.4. Removal of the cause of the violation of criteria is not economical or practical.

D.2.2. Waiver Authority. Major Commands (MAJCOM) may waive deviation from airfield and airspace criteria in this manual. The responsible MAJCOM Civil Engineer approves the waiver after coordination with all appropriate staff offices and concurrence by the MAJCOM Directors of Operations and Safety. The appropriate staff office for the Air National Guard (ANG) is ANGRC/CEPD. This authority is not delegated below MAJCOM level unless published as a MAJCOM policy. The following are exceptions:

D.2.2.1. Permissible deviations to airfield and airspace criteria, which do not require waivers, are listed in Attachment 14 to this manual.

D.2.2.2. Permanent waivers may require approval or coordination from various field operating agencies when AFI 32-1042, *Standards for Marking Airfields* or AFI 32-1076, *Visual Air Navigation Facilitie*s, standards apply.

D.2.2.3. Waiver approval is required according to AFMAN 11-230, *Instrument Procedure*s, when deviations from criteria in AFMAN 32-1076 would constitute deviations from the instrument procedure criteria or obstructions to air navigational criteria in AFMAN 11-230 or AFJMAN 11-226, *United States Standard for Terminal Instrument Procedures (TERPS*).

D.2.2.4. Authority is delegated to the Wing Commander when temporary waivers for construction activities are involved.

D.2.3. Deviations From Criteria for Land Not Under Air Force Jurisdiction. Refer waivers to airfield and airspace criteria on land not under Air Force jurisdiction to the next level of command for ultimate resolution.

D.2.4. Effective Length of Waiver. Waivers will be reviewed annually.

D.2.5. Responsibilities:

D.2.5.1. HQ AFCESA/CESC:

D.2.5.1.1. Recommends policy on waivers and provides technical assistance on the waiver program.

D.2.5.2. HQ AFFSA/XA:

D.2.5.2.1. Reviews all requests for waivers (operational requirements) to sighting criteria and airspace requirements.

D.2.5.2.2. Approves all requests for waivers to instrument procedure criteria in AFMAN 11- 230 or AFJMAN 11-226.

D.2.5.2.3. Processes requests for waivers according to AFMAN 11-230.

D.2.5.3. MAJCOM/CE:

D.2.5.3.1. Coordinates with flight operations and flight safety offices to grant waivers.

D.2.5.3.2. Sets and enforces reasonable safety precautions.

D.2.5.3.3. Monitors actions to correct temporarily waived items within specified periods.

D.2.5.3.4. Establishes procedures to ensure an annual review of all waived items.
D.2.5.3.5. Establishes the administrative procedures for processing waivers.
D.2.5.3.6. Maintains (for record) one copy of all pertinent documents relative to each waiver, including a record of staff coordination on actions at base and command levels.

D.2.5.4. Base Civil Engineer:

D.2.5.4.1. Coordinates with base flight safety, airfield management, and flight operations offices to request waivers.

D.2.5.4.2. Following Airfield Management, Flight Safety, and Civil Engineer analysis and recommendation about a waiverable condition, annotates proposed waiver location on appropriate E series map for MAJCOM evaluation.

D.2.5.4.3. Establishes maps of approved waivered items in accordance with AFI 32-7062, Base Comprehensive Planning, and maintains this information on the appropriate E-series map (see AFI 32-7062, Attachment 7). Also see AFJMAN 11-226 US Standard for Terminal Instrument Procedures (TERPS), and AFMAN 11-230, Instrument Procedures.

D.2.5.4.4. Develops a Military Construction Program or other project to systematically correct non-permanent waivers.

D.2.5.4.5. Presents a summary of waived items to the Facility Board each year for information and action.

D.2.5.4.6. Establishes a procedure for recording, reviewing, and acting on waivers. Maintains records similar to those required at the MAJCOM.

D.2.5.4.7. Requests a temporary waiver from the facility commander for any construction projects which violate any airfield clearance criteria during or after the completion of the construction project. The base must request a temporary waiver at least 45 days before the scheduled construction start date, or an emergency temporary waiver when 45 days are not possible. **NOTE:** Quick reaction or emergency maintenance and repair requirements are exempt from this requirement; however, the Base Civil Engineer will coordinate with base flight safety and flight operations offices to ensure implementation of safety measures.

D.2.5.4.8. Advises the MAJCOM of any canceled waivers.

D.2.5.5. ANGRC/CEP (for ANG facilities):

D.2.5.5.1. Develops policy on waivers and manages the ANG waiver program.

D.2.5.5.2. Processes and coordinates inquiries and actions for deviations to criteria and standards.

D.3. Navy and Marine Corps:

D.3.1. Applicability:

D.3.1.1. Use of Criteria. The criteria in this manual apply to Navy and Marine Corps aviation facilities located in the United States, its territories, trusts, and possessions. Where a Navy or Marine Corps aviation facility is a tenant on a civil airport, use these criteria to the extent practicable; otherwise, FAA criteria apply. Where a Navy or Marine Corps aviation facility is host to a civilian airport, these criteria will apply. Apply these standards to the extent practical at overseas locations where the Navy and Marine Corps have vested base rights. While the criteria in this manual are not intended for use in a theater-of-operations situation, they may be used as a guideline where prolonged use is anticipated and no other standard has been designated.

D.3.1.2. Criteria at Existing Facilities. The criteria will be used for planning new aviation facilities and new airfield pavements at existing aviation facilities (exception: primary surface width for Class B runway). Existing aviation facilities have been developed using previous standards which may not conform to the criteria herein. Safety clearances at existing aviation facilities need not be upgraded solely for the purpose of conforming to this criteria. However, at existing aviation

facilities where few structures have been constructed in accordance with previous safety clearances, it may be feasible to apply the revised standards herein.

D.3.2. Approval. Approval from Headquarters NAVFACENGCOM must be obtained prior to revising safety clearances at existing airfield pavements to conform with new standards herein. NAVFACENGCOM will coordinate the approval with the Naval Air Systems Command and CNO/CMC as required.

D.3.3. Obtaining Waiver. Once safety clearances have been established for an aviation facility, there may be occasions where it is not feasible to meet the designated standards. In these cases a waiver must be obtained from the Naval Air Systems Command. The waiver and its relation to the site approval process is defined in NAVFACINST 1010.44, *Shore Facilities Planning Manua*l.

D.3.4. Exemptions From Waiver. Certain navigational and operational aids normally are sited in violation of airspace safety clearances in order to operate effectively. The following aids are within this group and require no waiver from NAVAIR, provided they are sited in accordance with NAVFAC Definitive Designs (P-272) and/or the NAVFAC Design Manuals (DM Series):

D.3.4.1. Approach lighting systems.

D.3.4.2. Visual Approach Slope Indicator (VASI) systems and Precision Approach Path Indicator (PAPI).

D.3.4.3. Permanent Optical Lighting System (OLS), portable OLS and Fresnel lens equipment.

D.3.4.4. Runway distance markers.

D.3.4.5. Arresting Gear systems including signs.

D.3.4.6. Taxiway guidance, holding, and orientation signs.

D.3.4.7. All beacons and obstruction lights.

D.3.4.8. Arming and de-arming pad.

APPENDIX E

DETERMINATION OF FLEXURAL STRENGTH AND MODULUS OF ELASTICITY OF BITUMINOUS CONCRETE

E-1. SCOPE. These procedures describe preparation and testing of bituminous concrete to determine flexural strength and modulus of elasticity. The procedures are an adaptation from tests conducted on portland cement concrete (PCC) specimens.

E-2. APPLICABLE STANDARDS. The standard applicable to this procedure is ASTM C 78.

E-3. APPARATUS. The following apparatus are required:

a. A testing machine capable of applying repetitive loadings for compaction of beam specimens 152 by 152 by 533 millimeters (6 by 6 by 21 inches) to the design density (an Instron electromechanical testing machine meets this requirement).

b. A steel mold, suitably reinforced to withstand compaction of specimens without distortion.

c. Two linear variable differential transformers (LVDTs).

d. A 22,240-Newton (5,000-pound) load cell.

e. An X-Y recorder.

f. A testing machine for load applications conforming to ASTM C 78 (a Baldwin or Tinius Olsen hydraulic testing machine is suitable for this purpose).

E-4. MATERIALS. Sufficient aggregate and bitumen meeting applicable specifications to produce six 152- by 152- by 533-millimeter (6- by 6- by 21-inch) test specimens are required. In the event the proportioning of aggregate and bitumen, bitumen content, and density of compacted specimens are not known, additional materials will be required to conduct conventional Marshall tests to develop the needed mix design data.

E-5. SAMPLE PREPARATION.

a. Prepare in a laboratory mixer four portions of paving mixture for one 152- by 152- by 533-millimeter (6- by 6- by 21-inch) beam test specimen consisting of aggregate and bitumen in the proportions indicated for optimum bitumen content. The total quantity of paving mixture should be such that when compacted to a uniform 152- by 152-millimeter (6- by 6-inch) cross section, the density of the beam will be as specified from previous laboratory mix design tests or other sources. The temperature of the paving mixture at the time of mixing should be such that subsequent compaction can be accomplished at 121 ± 2.8 degrees Celsius (250 ± 5 degrees Fahrenheit). Place two of the four portions in the 152- by 152- by 533-millimeter (6- by 6- by 21-inch) reinforced steel mold and compact to a 76-millimeter (3-inch) thickness with a 152- by 152-millimeter (6- by 6-inch) foot attached to the repetitive loading machine. Shift the mold between load applications to distribute the compaction effort uniformly. Add the remaining two portions and continue compaction until the paving mixture is compacted to exactly a 152- by

152-millimeter (6- by 6-inch) cross section. After compaction, place a 152- by 533-millimeter (6- by 21-inch) steel plate on the surface of the paving mixture and apply a leveling load of 8,896 Newtons (2,000 pounds) to the plate. Prepare six beam test specimens in the manner described.

b. After cooling, remove the beams from the molds and rotate 90 degrees so that the smooth, parallel sides will become the top and bottom. Cement an L-shaped metal tab with quick-setting epoxy glue to each 152- by 533-millimeter (6- by 21-inch) side of the beams on the beams' neutral axes at midspan. The taps should be drilled for attachment of the LVDTS. Cure the beams at 10 ± 1.7 degrees Celsius (50 ± 3 degrees Fahrenheit) for 4 days prior to testing.

E-6. TEST PROCEDURES.

a. Condition three specimens each at 10 and 24 ± 1.7 degrees Celsius (50 and 75 ± 3 degrees Fahrenheit) for at least 12 hours prior to testing. If testing occurs immediately after curing the specimens at 10 ± 1.7 degrees Celsius (50 ± 3 degrees Fahrenheit) for 4 days, no additional conditioning is required for the specimens tested at this temperature.

b. Place the specimen in the test machine as described in ASTM C 78. Place thin Teflon strips at the point of contact between the test specimens and the load-applying and load-support blocks. While the beams are being prepared for testing, place an additional support block at midspan to prevent premature sagging of the beams. Remove this support block immediately prior to the initiation of load application. Mount the LVDTs on laboratory stands on each side of the beams, and attach the LVDTs to the L-shaped tabs on the sides of the beams. Connect the LVDTs and load cell to the X-Y recorder. Make final adjustments and checks on specimens and test equipment. Apply loading in accordance with ASTM C 78, omitting the initial 4,448-Newton (1,000-pound) load.

E-7. CALCULATIONS

a. The modulus of rupture R is calculated from the following equation (from ASTM C 78):

$$R = \frac{PL}{bd^2} \qquad \text{(E-1)}$$

where

R = modulus of rupture, MPa (psi)

P = maximum applied load, Newtons (pounds)

L = span length, millimeters (inches) (457 millimeters (18 inches))

b = average width of beam, millimeters (inches)

d = average depth (height) of beam, millimeters (inches)

b. The modulus of elasticity E is calculated from the following equation:

$$E = \frac{23PL^3}{1296\Delta I} k \qquad \text{(E-2)}$$

where

E = static Young's modulus of elasticity, MPa (psi)

P = applied load, Newtons (pounds)

L = span length, millimeters (inches) (457 millimeters (18 inches))

Δ = deflection of neutral axis, millimeters (inches), under load, P

I = moment of inertia, millimeter^4 (inch^4) (=bd^3/12)

b = average width of beam, millimeters (inches)

d = average depth (height) of beam, millimeters (inches)

k = Pickett's correction for shear (third-point loading). (Values of E for bituminous beams should be calculated without using Pickett's correction K for shear).

E-8. REPORT. The report shall include the following:

a. Gradation of Aggregate.

b. Type and Properties of Bituminous Cement.

c. Bituminous Concrete Mix Design Properties.

d. Bituminous Concrete Beam Properties.

e. Modulus of Rupture.

f. Modulus of Elasticity.

APPENDIX F

CURVES FOR DETERMINING EFFECTIVE STRAIN REPETITIONS

F-1. GENERAL. This appendix contains plots (Figures F-1 through F-22) for converting aircraft operations to effective repetitions of strain when given the type of aircraft, the effective thickness of the pavement, and the offset from the center of the runway or taxiway.

F-2. COMPUTER PLOTS. A computer program was developed by the U.S. Army Engineer Research and Development Center (ERDC) for producing the plots for effective strain repetitions. Should the plots not be adequate, the computer program could be used to determine the conversion factors for any design situation. Information for accessing the computer programs is described in Chapter 1.

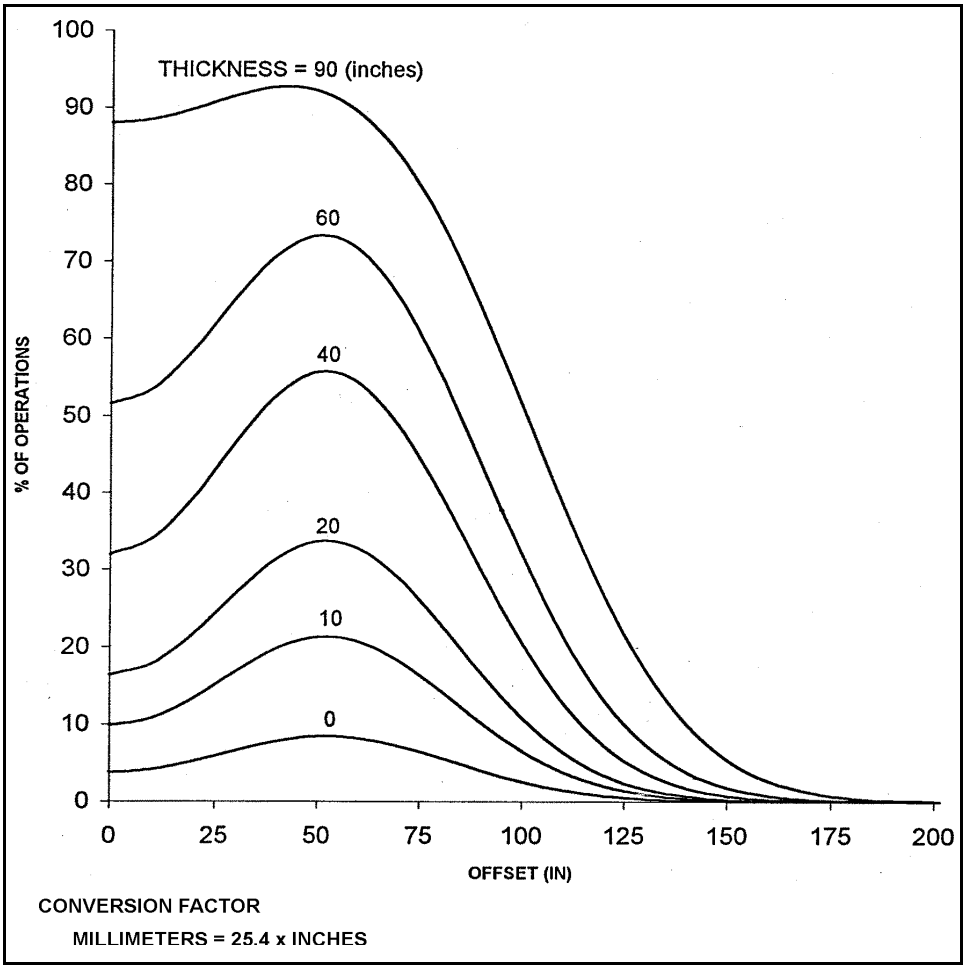


Figure F-1. Effective repetitions of the strain for UH-60 aircraft, types B, C, and secondary traffic areas.

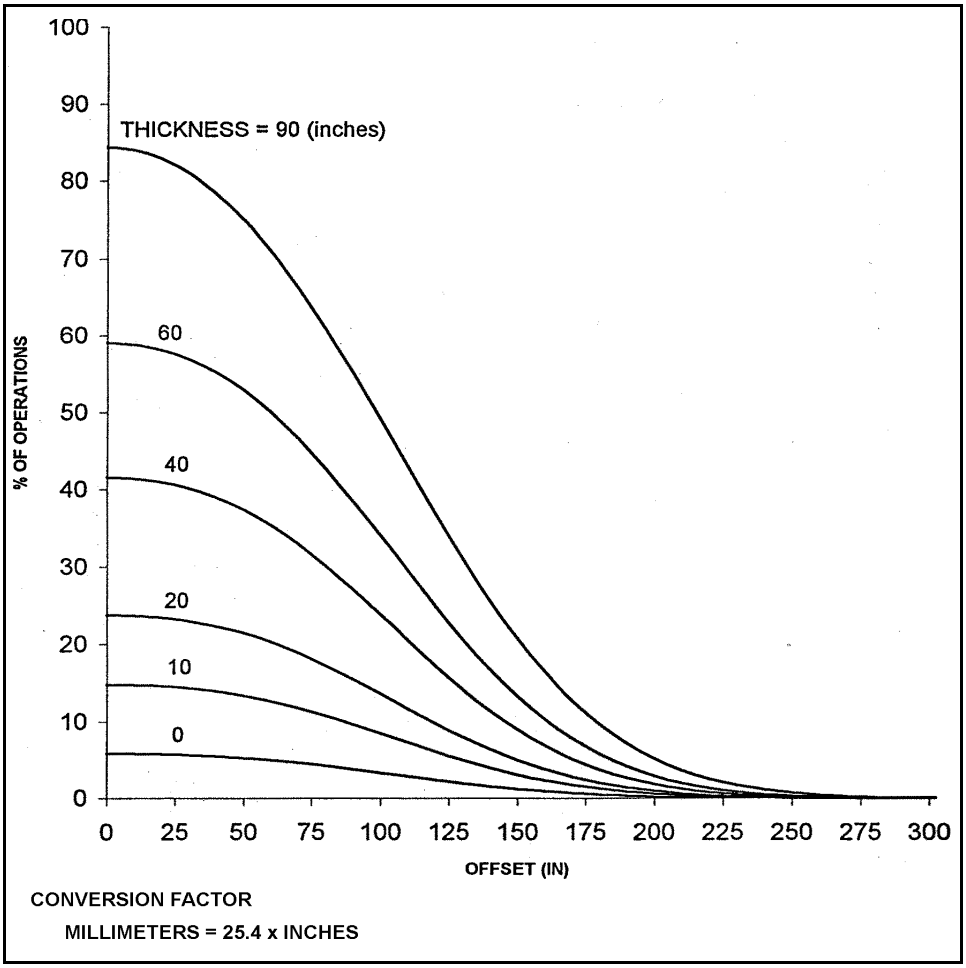


Figure F-2. Effective repetitions of strain for UH-60 aircraft, type A or primary traffic areas

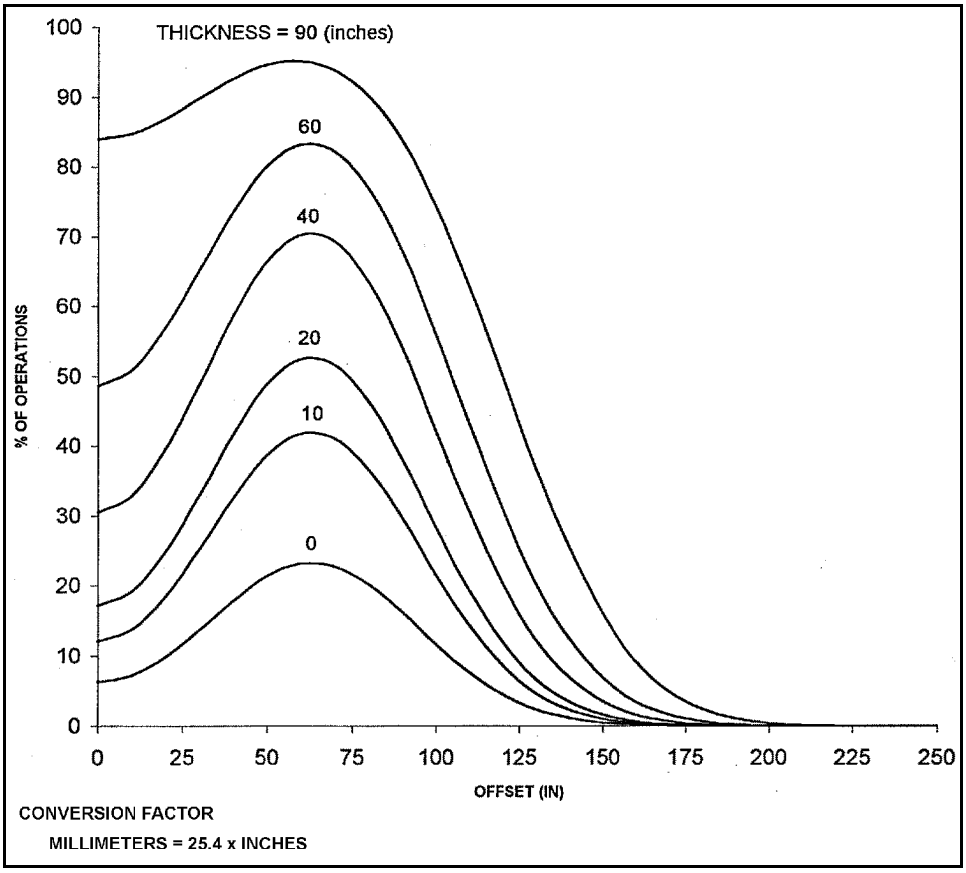


Figure F-3. Effective repetitions of strain for CH-47 aircraft, types B, C, or secondary traffic areas

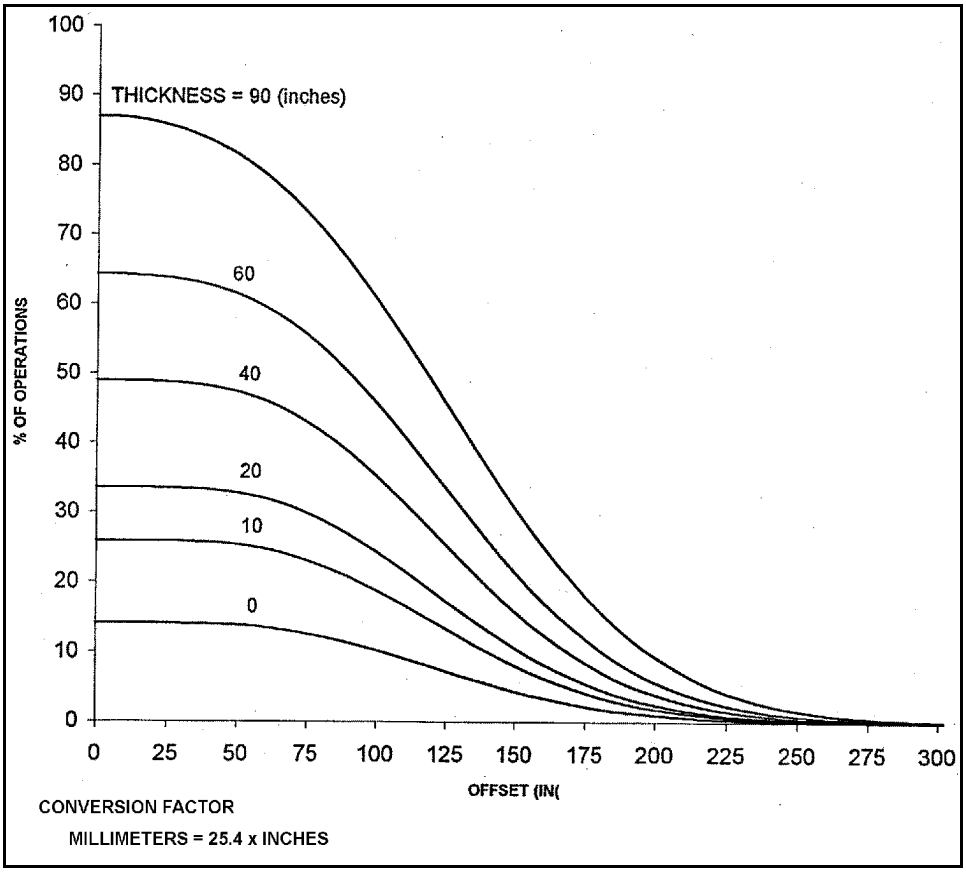


Figure F-4. Effective repetitions of strain for CH-47 aircraft, type A or primary traffic areas

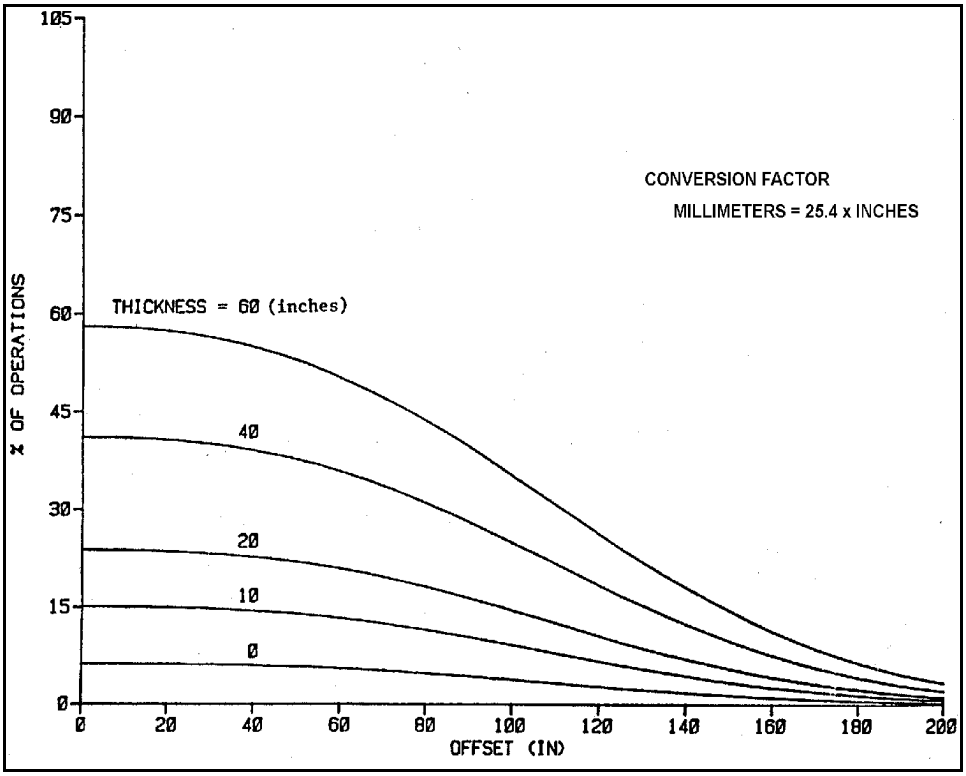


Figure F-5. Effective repetitions of strain for OV-1 aircraft, types B, C, or secondary traffic areas

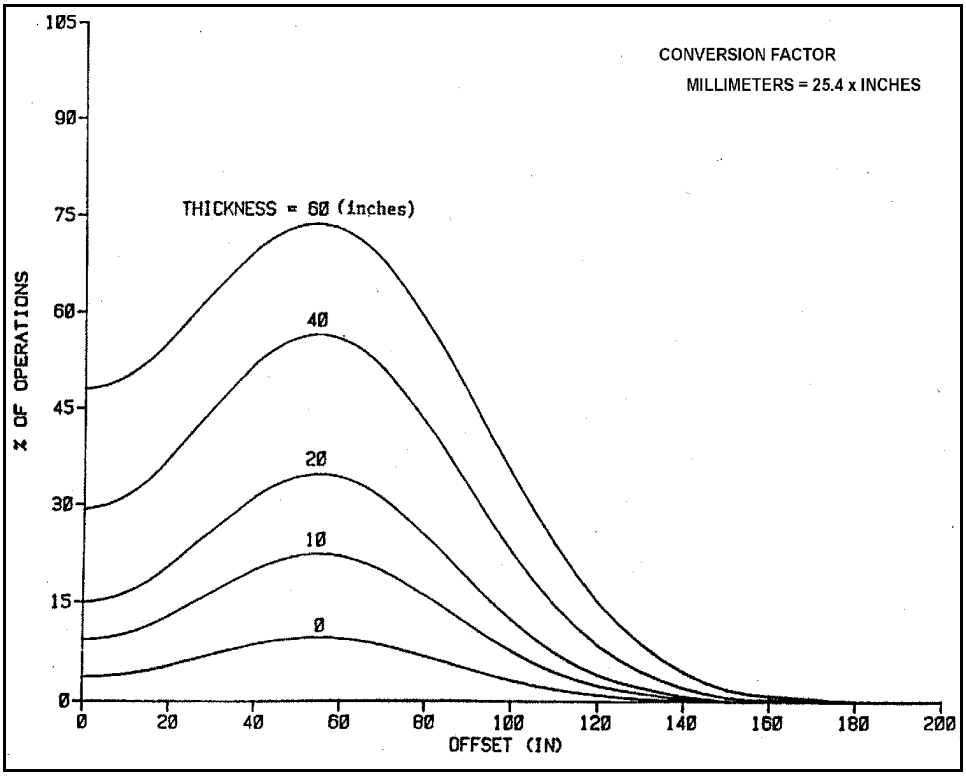


Figure F-6. Effective repetitions of strain for OV-1 aircraft, type A or primary traffic areas

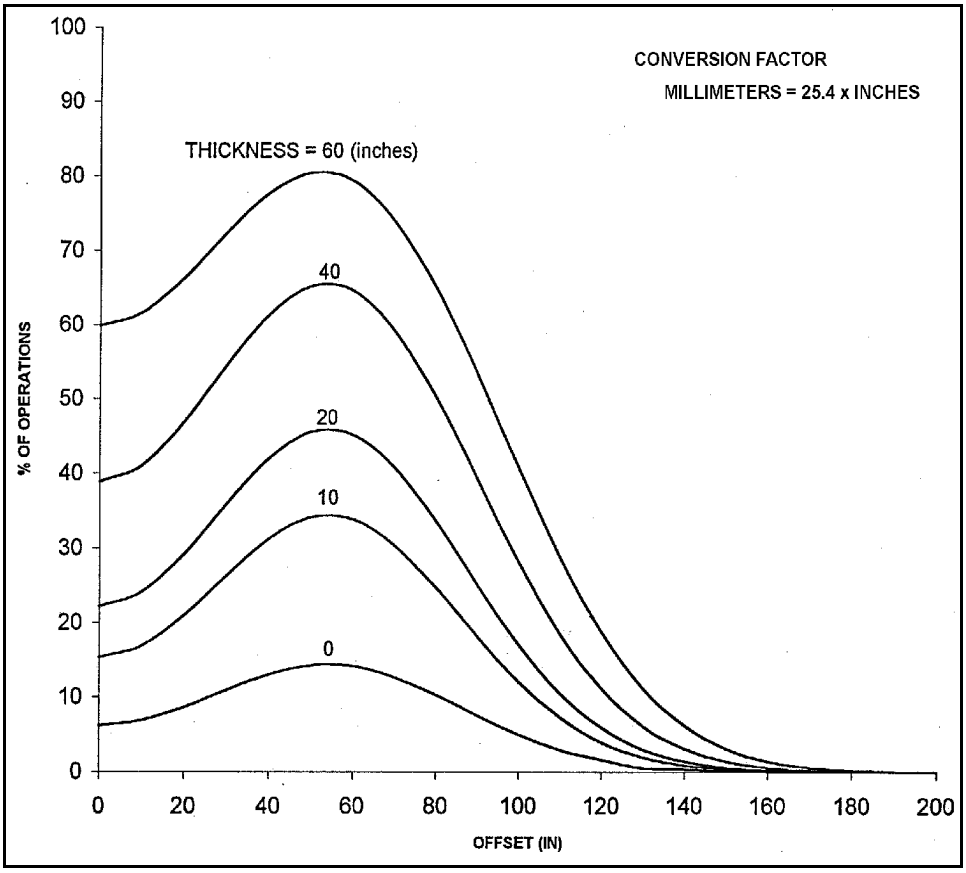


Figure F-7. Effective repetitions of strain for C-12 aircraft, types B, C, or secondary traffic areas

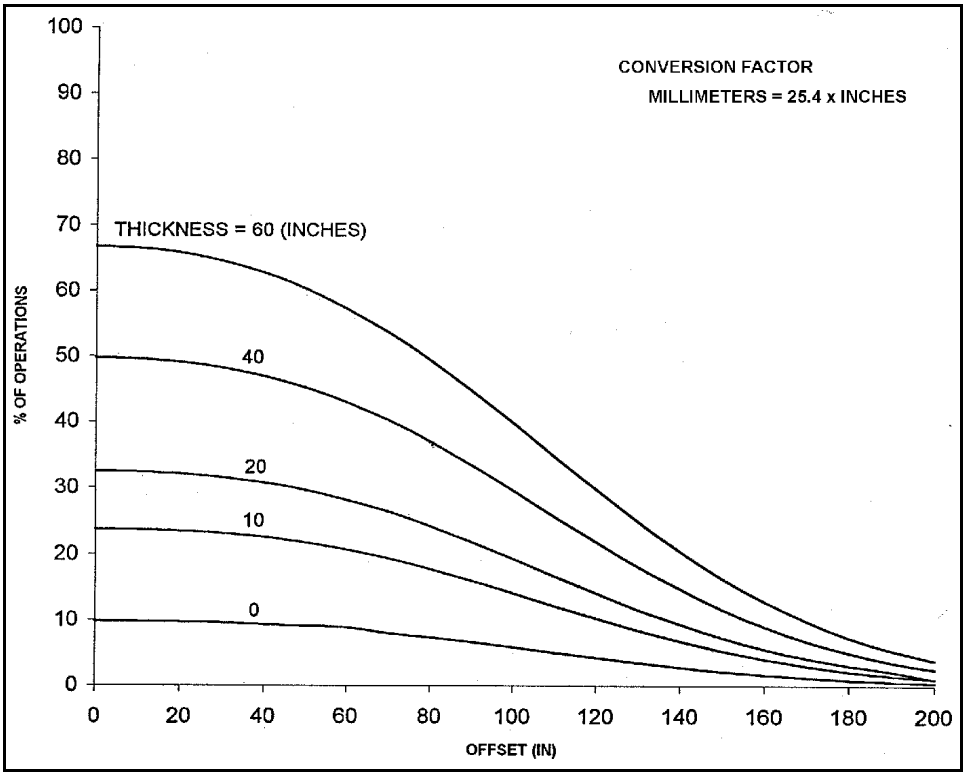


Figure F-8. Effective repetitions of strain for C-12 aircraft, type A or primary traffic areas

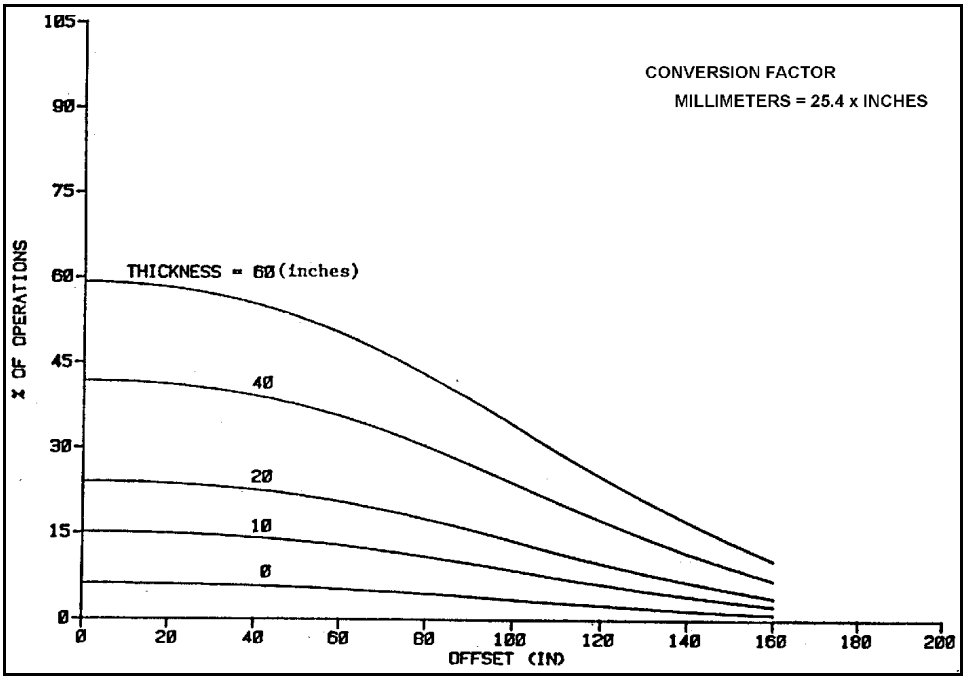


Figure F-9. Effective repetitions of strain for C-130 aircraft, types B, C, or secondary traffic areas

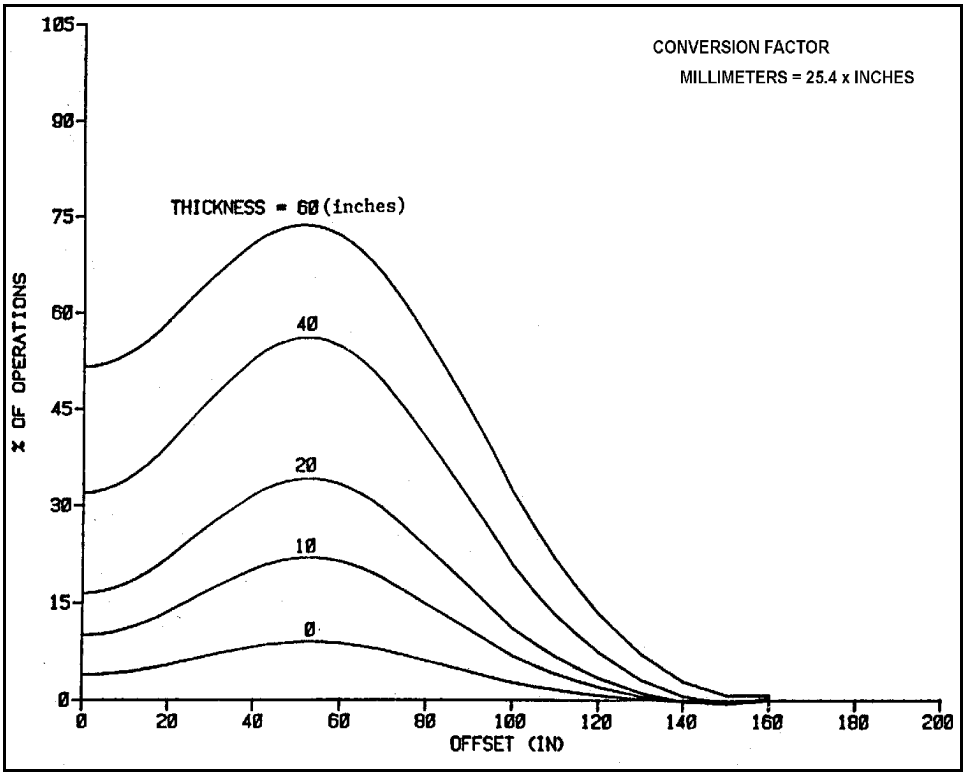


Figure F-10. Effective repetitions of strain for C-130 aircraft, type A or primary traffic areas

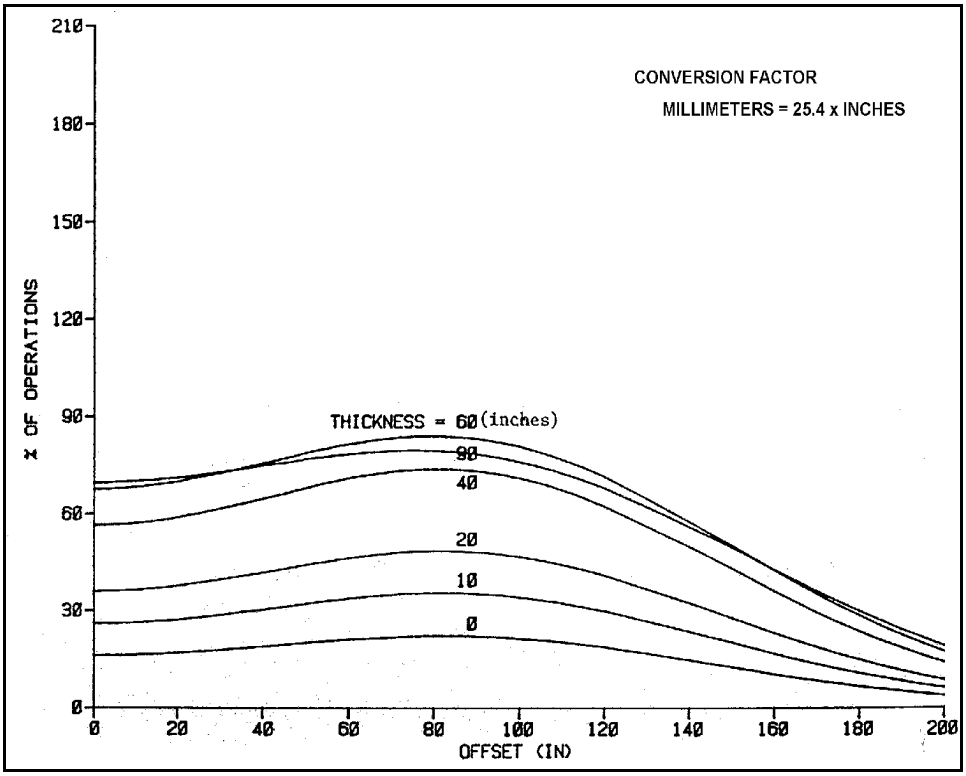


Figure F-11. Effective repetitions of strain for F-15 aircraft, Air Force types B and C traffic areas

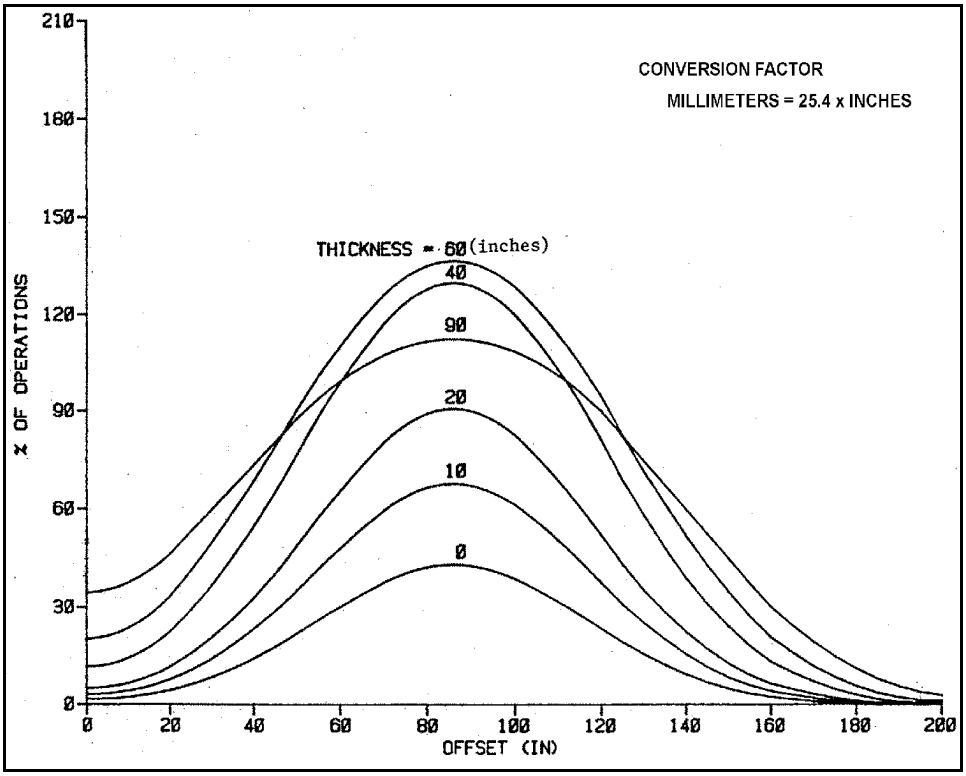


Figure F-12. Effective repetitions of strain for F-15 aircraft, Air Force type A traffic areas

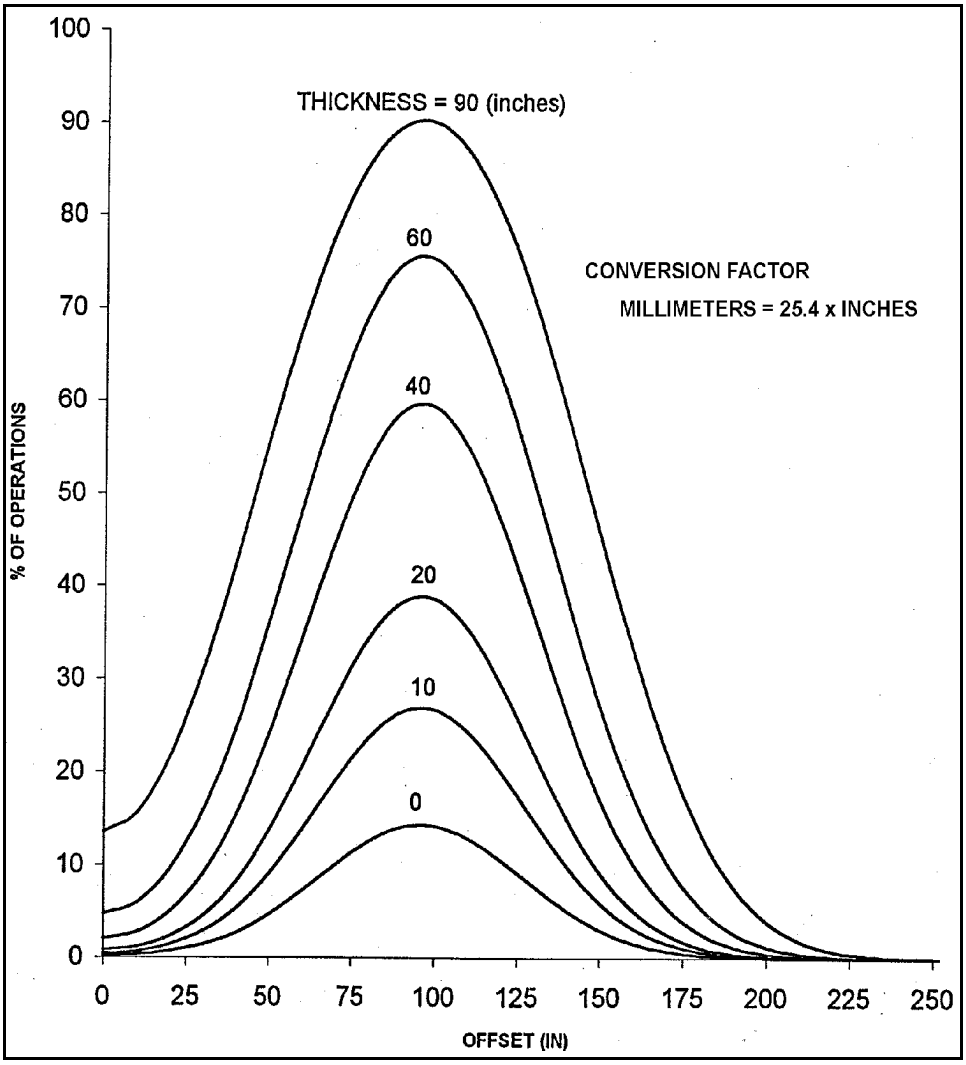


Figure F-13. Effective repetitions of strain for F-14 aircraft, types B, C and secondary traffic areas

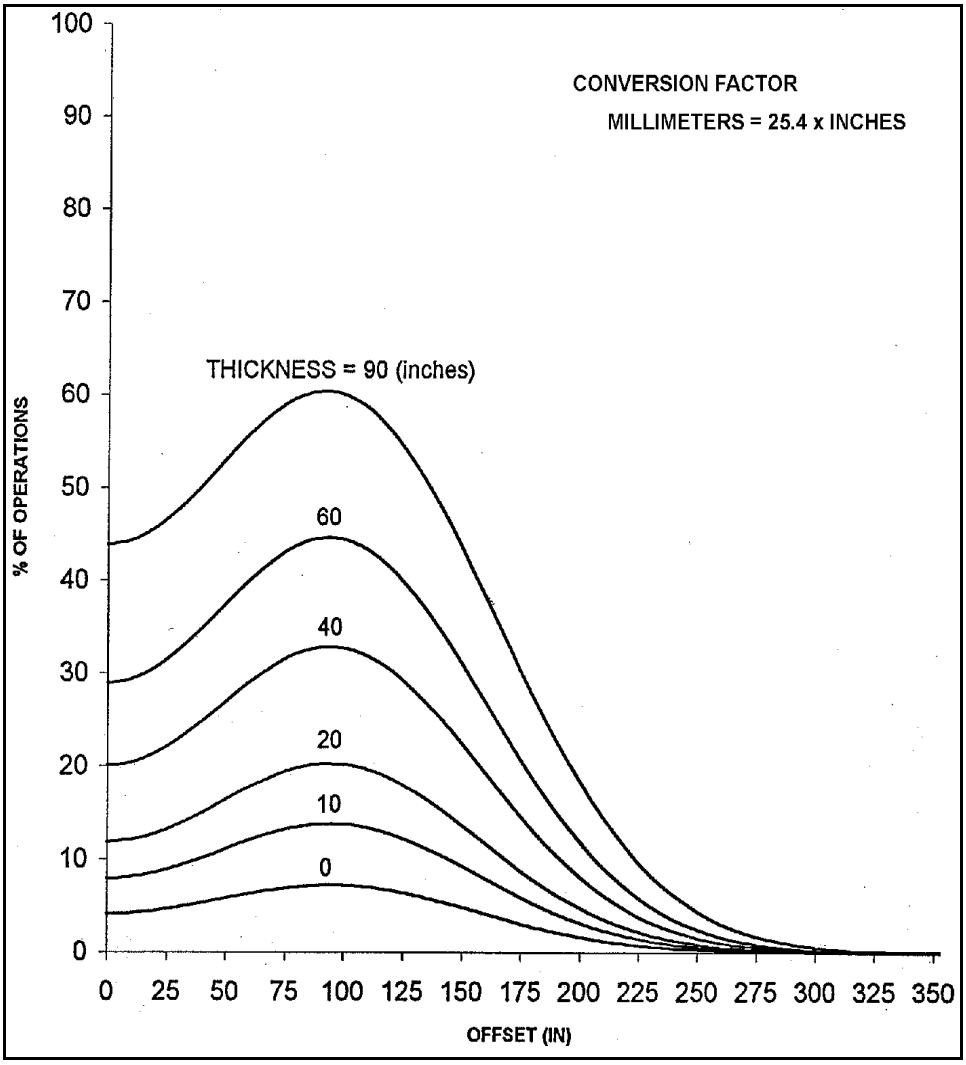


Figure F-14. Effective repetitions of strain for F-14 aircraft, type A or primary traffic areas

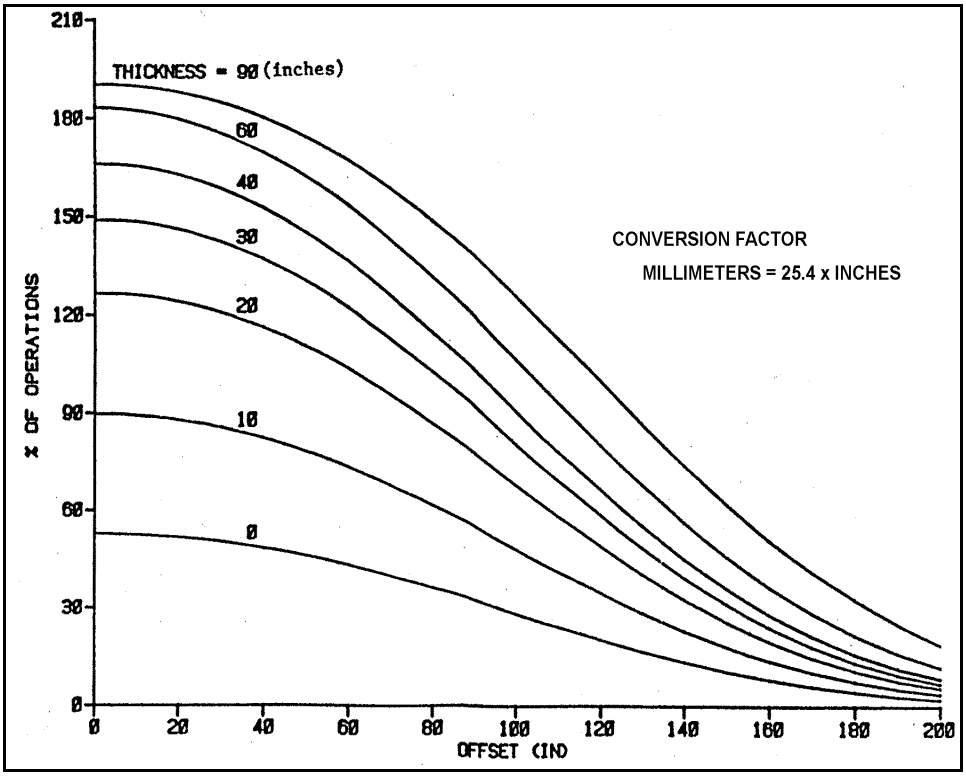


Figure F-15. Effective repetitions of strain for B-52 aircraft, types B, C or secondary traffic areas

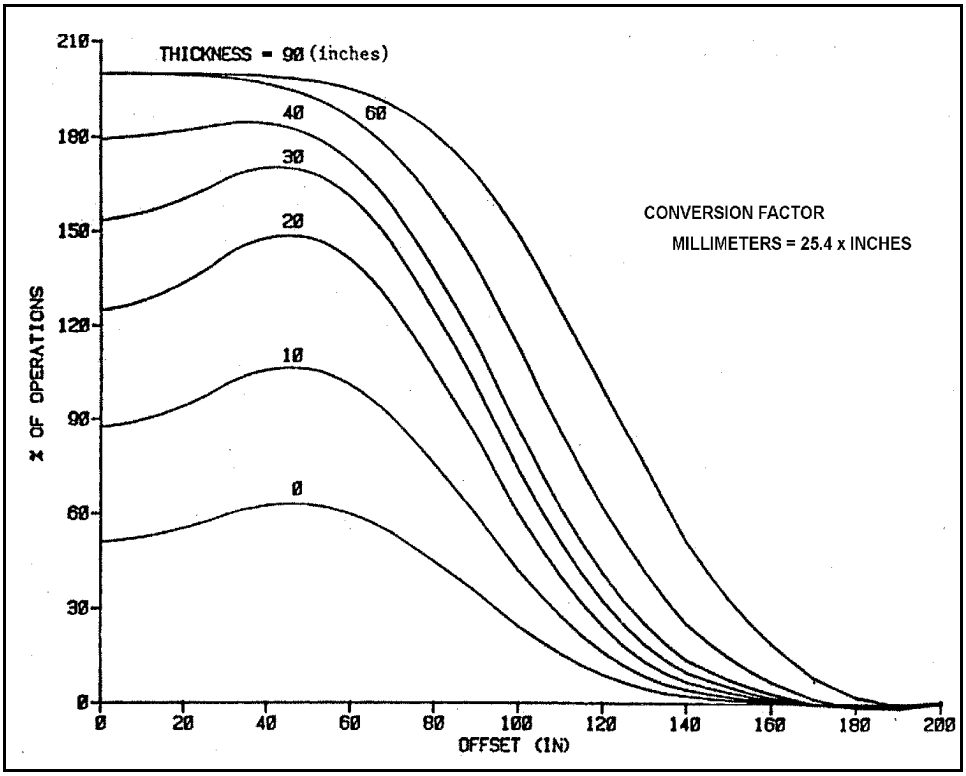


Figure F-16. Effective repetitions of strain for B-52 aircraft, type A or primary traffic areas

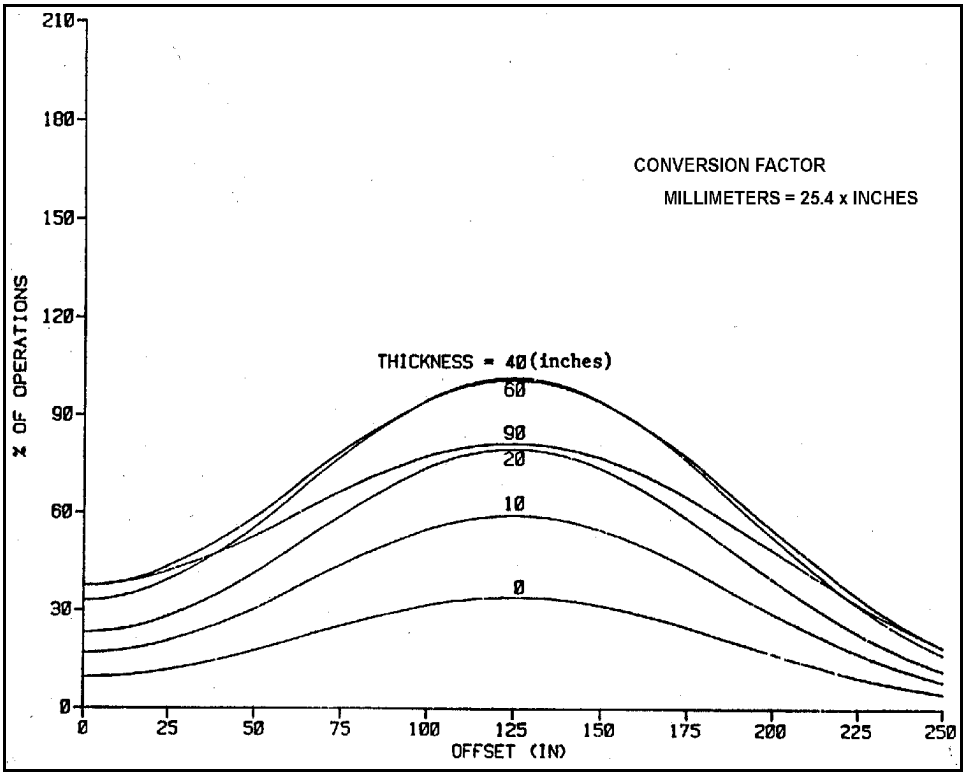


Figure F-17. Effective repetitions of strain for B-1 and C-141 aircraft, types B, C, or secondary traffic areas

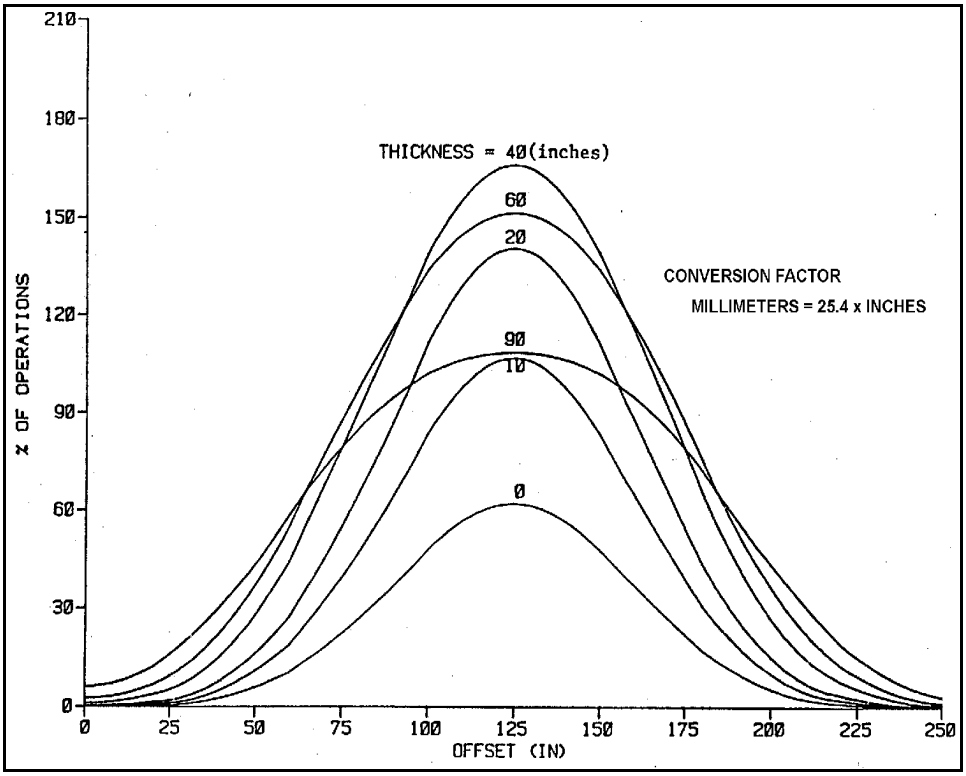


Figure F-18. Effective repetitions of strain for B-1 and C-141 aircraft, type A or primary traffic areas

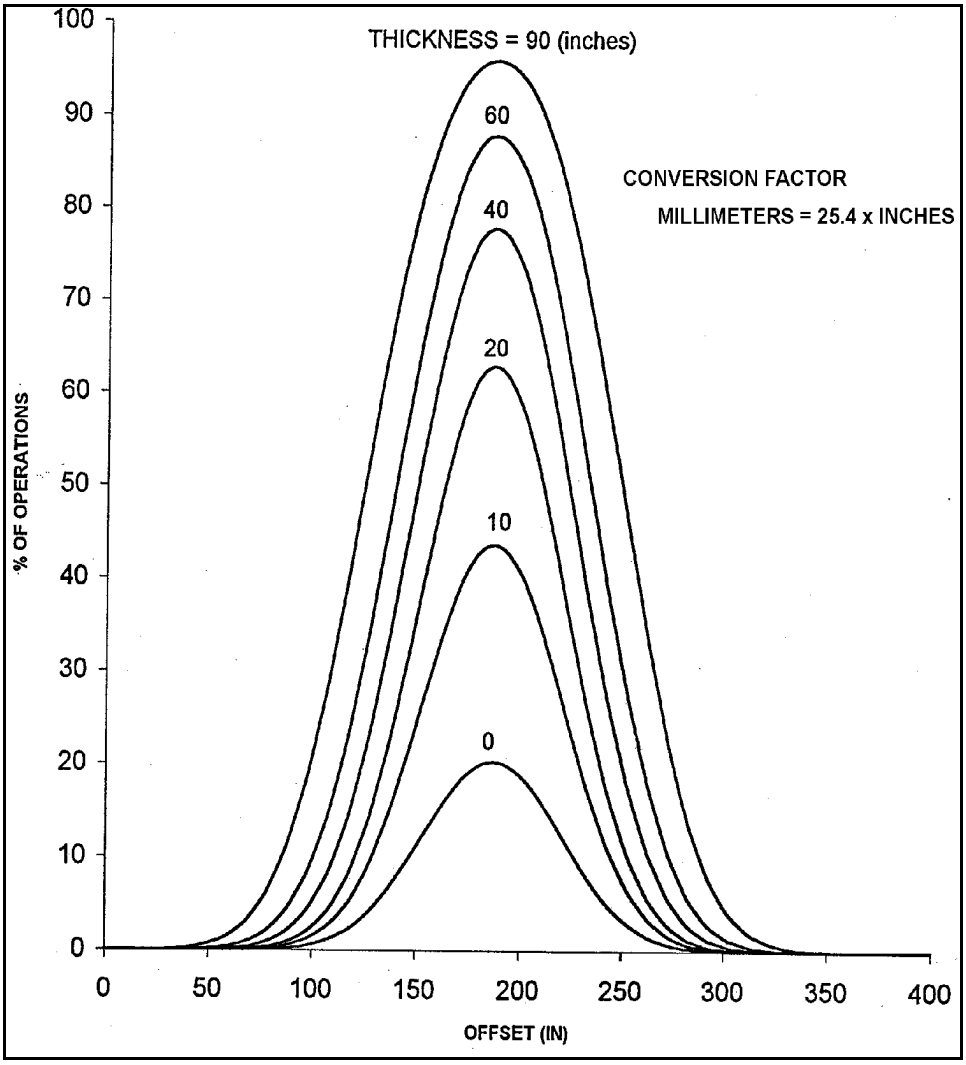


Figure F-19. Effective repetitions of strain for P-3 aircraft, types B, C, or secondary traffic areas

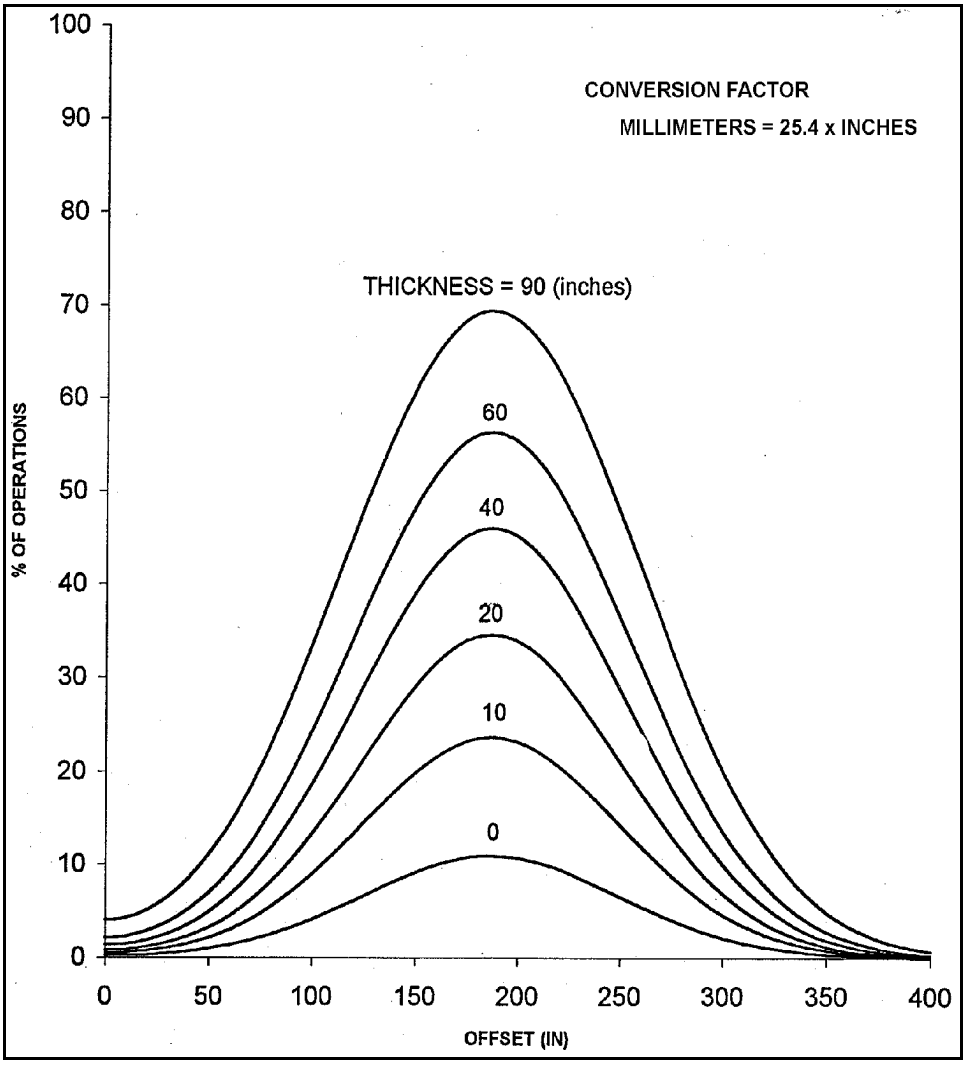


Figure F-20. Effective repetitions of strain for P-3 aircraft, type A or primary traffic areas

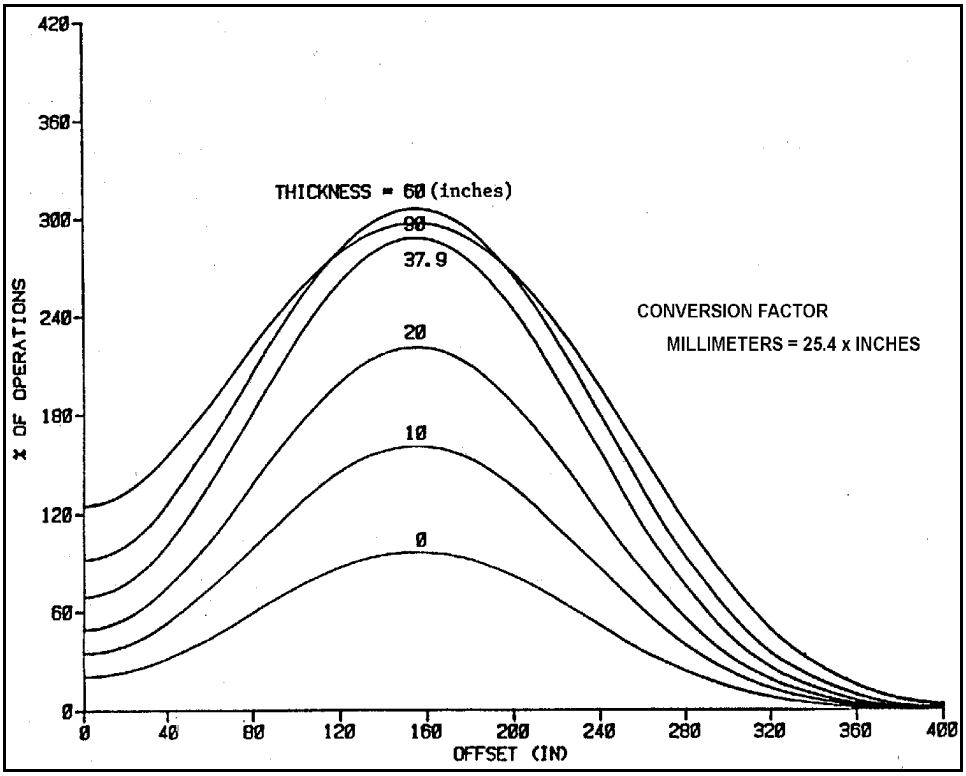


Figure F-21. Effective repetitions of strain for C-5 aircraft, types B, C, or secondary traffic areas

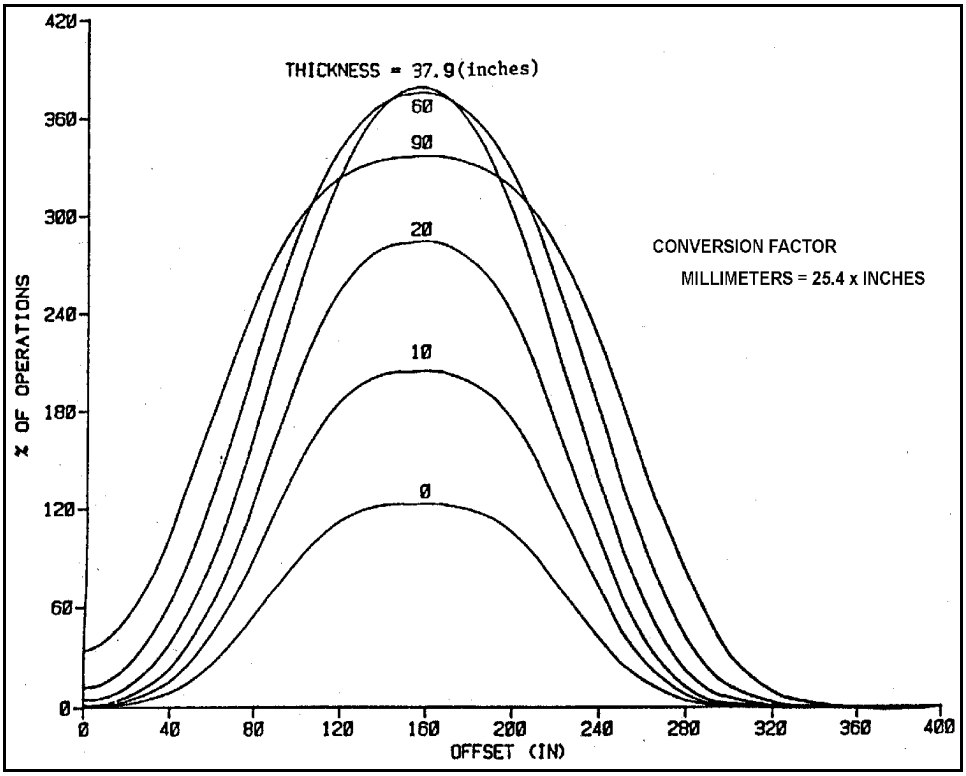


Figure F-22. Effective repetitions of strain for C-5 aircraft, type A or primary traffic areas

APPENDIX G

PROCEDURE FOR PREPARATION OF BITUMINOUS CYLINDRICAL SPECIMENS

G-1. SCOPE. This procedure describes the preparation of cylindrical specimens of bituminous paving mixture suitable for dynamic modulus testing. The procedure is intended for densegraded bituminous concrete mixture containing up to 25-millimeter (1-inch) maximum-size aggregate.

G-2. APPLICABLE STANDARDS. The following ASTM publications are applicable to this procedure: D 1559, D 1560, and D 1561.

G-3. SPECIMENS. Approximately 4,000 grams of bituminous mixture should be prepared as specified by ASTM D 1560. Cylindrical specimens should be 102 millimeters (4 inches) in diameter by 203 millimeters (8 inches) in height.

G-4. APPARATUS.

a. Testing Apparatus. The apparatus used in preparing the specimens should be as specified by ASTM D 1561, except that steel molding cylinders with 6.3-millimeter (1/4-inch) wall thickness having an inside diameter of 102 millimeters (4 inches) and height of 254 millimeters (10 inches) should be used.

b. Measurement System. The measurement system should consist of a two-channel recorder, stress and strain measuring devices, and suitable signal amplification and excitation equipment. The measurement system should have the capability for determining loading up to 13,344 Newtons (3,000 pounds) from a recording with a minimum sensitivity of 2 percent of the test load per millimeter of chart paper. This system should also be capable of use in determining strains over a range of full-scale recorder outputs from 300 to 5,000 microunits of strain. At the highest sensitivity setting, the system should be able to display 4 microunits of strain or less per millimeter on the recorder chart.

c. Recorder Amplitude. The recorder amplitude should be independent of frequency for tests conducted up to 20 hertz.

d. Measurement of Axial Strain. The values of axial strain should be measured by bonding two wire strain gauges at midheight opposite each other on the specimens. (The Baldwin Lima Hamilton SR-4 Type A-1S 13 strain gauge has been found satisfactory for this purpose). The gauges are wired in a wheatstone bridge circuit with two active gauges on the test specimen exposed to the same environment as the test specimen. The temperature-compensating gauges should be at the same position on the specimen as the active gauges. The sensitivity and type of measurement device should be selected to provide the strain readout required above.

e. Load Measurements. Loads should be measured with an electronic load cell meeting requirements for load and stress measurements above.

G-5. PROCEDURE.

a. Procedure. The compaction temperature for the bituminous mixture should be as specified by ASTM D 1561. For the first step in molding specimens, heat the compaction mold to the same temperature as the mix. Next, place the compaction mold in position in the mold holder and insert a paper disk 102 millimeters (4 inches) in diameter to cover the baseplate of the mold holder. Weigh out one-half of the required amount of bituminous mixture for one specimen at the specified temperature and place uniformly in the insulated feeder trough, which has been preheated to the compaction temperature for the mixture. By means of the variable transformer controlling the heater, maintain the compactor foot sufficiently hot to prevent the mixture from adhering to it. By means of a paddle of suitable dimensions to fit the cross section of the trough, push 30 approximately equal portions of the mixture continuously and uniformly into the mold while 30 tamping blows at a pressure of 1.7 Mpa (250 psi) are applied. Immediately place the remaining one-half of the mixture uniformly in the feeder trough. Push 30 approximately equal portions of the mixture into the mold in a continuous and uniform manner while applying tamping blows at a pressure of 1.7 MPa (250 psi). If sandy or unstable material is involved and there is undue movement of the mixture under the compactor foot, reduce the compaction temperature and compactor foot pressure until kneading compaction can be accomplished.

b. Immediately after compaction with the California kneading compactor, apply a static load to the specimen using a compression testing machine. Apply the load by the double-plunger method in which metal followers are employed as free-fitting plungers on the top and bottom of the specimen. Apply the load on the specimen at a rate of 13 millimeters (0.5 inches) per minute until an applied pressure of 6.9 MPa (1,000 psi) is reached. Release the load immediately. After the compacted specimen has cooled sufficiently so that it will not deform on handling, remove it from the mold. Place the specimen on a smooth flat surface and allow to cool to room temperature. Cylindrical specimens will have approximately the same bulk specific gravity as specimens prepared as specified by ASTM D 1559 and ASTM D 1561.

APPENDIX H

PROCEDURE FOR DETERMINING THE DYNAMIC MODULUS OF BITUMINOUS CONCRETE MIXTURES

H-1. GENERAL. The purpose of this procedure is to determine dynamic modulus values of bituminous concrete mixtures. The procedure described covers a range of both temperature and loading frequency. The minimum recommended test series consists of testing at 4.5, 21, and 37.8 degrees Celsius (40, 70, and 100 degrees Fahrenheit) at loading frequencies of 2 and 10 hertz for each temperature. The method is applicable to bituminous paving mixtures similar to the 25.4, 19, 12.7, and 9.5 millimeter (1-, 3/4-, 1/2-, and 3/8-inch), and No. 4 mixes as defined by Table 3 of ASTM D 3515.

H-2. APPLICABLE STANDARDS. The following ASTM standards are applicable to this procedure: C 617, D 1559, D 1561, and D 3515.

H-3. SUMMARY PROCEDURE. The dynamic modulus test is run by applying a sinusoidal (haversine) axial compressive stress to a specimen of bituminous concrete at a given temperature and loading frequency. The resulting recoverable axial strain response of the specimen is measured and used to calculate the dynamic modulus.

H-4. DEFINITIONS. The following terms are used in this procedure:

a. Dynamic Modulus. The absolute value of the complex modulus which defines the elastic properties of a linear viscoelastic material subjected to a sinusoidal loading.

b. Complex Modulus. A complex number which defines the relationship between stress and strain for a linear viscoelastic material.

c. Linear material. A material whose stress-to-strain ratio is independent of the loading stress applied.

H-5. APPARATUS. An electrohydraulic testing machine with a frequency generator capable of producing a haversine wave form has proven to be most suitable for use in dynamic modulus testing. The testing machine should have the capability of applying loads over a range of frequencies from 1 to 20 hertz and stress levels up to 0.69 MPa (100 psi). The temperature control system should be capable of a temperature range of 0.0 to 49 degrees Celsius (32 to 120 degrees Fahrenheit). The temperature chamber should be large enough to hold six specimens. A hardened steel disk with a diameter equal to that of the test specimen should be used to transfer the load from the testing machine to the specimen.

H-6. SPECIMENS. The laboratory-molded specimens should be prepared according to Appendix H. A minimum of three specimens is required for testing. The molding procedure is as follows: Cap all specimens with a sulfur mortar meeting ASTM C 617 requirements prior to testing. Bond the strain gauges with epoxy cement to the sides of the specimen near midheight in position to measure axial strains. (Baldwin Lima Hamilton EPY 150 Epoxy Cement has been found satisfactory for this purpose. On specimens with large-size aggregate, care must be taken so that the gauges

are attached over areas between the aggregate faces). Wire the strain gauges as required in paragraph G-5 and attach suitable lead wires and connectors.

H-7. PROCEDURE. The following testing procedure is recommended:

a. Place test specimens in a controlled temperature cabinet, and bring them to the specified test temperature. A dummy specimen with a thermocouple in the center can be used to determine when the desired test temperature is reached.

b. Place a specimen in the loading apparatus, and connect the strain gage wires to the measurement system. Put the hardened steel disk on top of the specimen and center both under the loading apparatus. Adjust and balance the electronic measuring system as necessary.

c. Apply the haversine loading to the specimen without impact and with loads varying between (0 and 35 psi) for each load application for a minimum of 30 seconds and not exceeding 45 seconds at temperature of 4.5, 21, and 37.8 degrees Celsius (40, 70, and 100 degrees Fahrenheit) and at loading frequencies of 2 hertz for taxiway design and 10 hertz for runway design. If excessive deformation (greater than 2,500 microunits of strain) occurs, reduce the maximum loading stress level to 0.12 MPa (17.5 psi).

d. Test three specimens at each temperature and frequency condition twice. Start at the lowest temperature and repeat the test at the next highest temperature. Bring the specimens to the specified test temperature before each test is commenced.

e. Monitor both the loading stress and the axial strain during the test. Increase the recorder chart speed so that one cycle covers 25 to 50 millimeters of chart paper for five to ten repetitions before the end of the test.

f. Complete the loading for each test within 2 minutes from the time specimens are removed from the temperature control cabinet. The 2-minute testing time limit is waived if loading is conducted within a temperature control cabinet meeting requirements in paragraph H-5.

H-8. CALCULATIONS. Measure the average amplitude of the load and the strain over the last three loading cycles to the nearest 1/2 millimeter. Calculate the loading stress σ_o using the equation

$$\sigma_o = \frac{H_1 L}{H_2 A} \qquad \text{(H-1)}$$

where

H_1 = measured height of load, millimeters (inches)

H_2 = measured chart height, millimeters (inches)

L = full-scale load amplitude determined by settings on the recording equipment, Newtons (pounds)

A = cross-section area of the test specimen, square millimeters (square inches)

Calculate the recoverable axial strain ϵ_o using the equation

$$\epsilon_o = \frac{H_3 S}{H_4}$$

where

H_3 = measured height of recoverable strain, millimeters (inches)

H_4 = measured chart height, millimeters (inches)

S = full-scale strain amplitude determined by settings on the recording equipment

Calculate the dynamic modulus $|E^*|$ using the equation

$$|E^*| = \frac{\sigma_o}{\epsilon_o} \qquad \text{(H-3)}$$

where

σ_o = axial loading stress, MPa (psi)

ϵ_o = recoverable axial strain, millimeters per millimeter (inches per inch)

Report the average dynamic modulus at temperatures of 4.5, 21, and 37.8 degrees Celsius (40, 70, and 100 degrees Fahrenheit) for each loading frequency at each temperature.

APPENDIX I

PROCEDURE FOR ESTIMATING THE MODULUS OF ELASTICITY OF BITUMINOUS CONCRETE

I-1. GENERAL. The procedure for estimating the modulus of elasticity of bituminous concrete presented here is based on relationships developed by Shell.[1] Parameters needed for input into this method are:

a. Ring-and-ball softening point in degrees Celsius (degrees Fahrenheit) of the bituminous material used in the mix in accordance with ASTM D 36.

b. Penetration of the bituminous material, in 1/10 millimeters in accordance with ASTM D 5.

c. Volume concentration of the aggregate C_v used in the mix defined by

$$C_v = \frac{\textit{aggregate volume}}{\textit{aggregate volume} + \textit{bitumen volume}} \qquad \text{(I-1)}$$

I-2. STEPS OF PROCEDURE. The steps in using this method are as follows:

a. Penetration Index. With known values of penetration and ring-and-ball softening point, enter Figure I-1 and determine the penetration index PI.

b. Stiffness Modulus. The next step involves the use of the nomograph presented in Figure H-2. In addition to the PI, two other values are required: the temperature of the bituminous concrete mix for which the modulus value is desired and the estimated loading frequency or time of loading to which the prototype pavement will be subjected. Use of a loading frequency of 2 hertz is recommended for taxiway design and 10 hertz for runway design. With values for the loading frequency and the difference in temperature between the bituminous concrete and the ring-and-ball softening point, a stiffness value for the bitumen S_{bit} can be determined from the appropriate PI line at the top of the nomograph. The value of S_{bit} is then used to determine the modulus of the mix S_{mix}.

c. Determining Modulus of Mix S_{mix}. A value for S_{mix} may be determined by

$$s_{mix} = s_{bit}\left[1 + \left(\frac{2.5}{n}\right)\left(\frac{c_v}{1 - c_v}\right)\right]^n \qquad \text{(I-2)}$$

[1] Heukelom, W., and Klomp, A. J. G. (1964). "Road Design and Dynamic Loading." *Proceedings, Association of Asphalt Paving Technologists*. Vol 33, 92-125.

where

$$n = 0.83 \log \left(\frac{400{,}000}{s_{bit}} \right) \tag{I-3}$$

The value thus determined for S_{mix} is in units of kilograms per square centimeter.

d. Corrected Aggregate Volume Concentration. This expression should be used for aggregate volume concentrations of 0.7 to 0.9 and air void contents of 3 percent or less. For larger air void contents, use a corrected aggregate volume concentration (C'_v)

$$C'_v = \frac{c_v}{1 + \Delta air\ void\ content} \tag{I-4}$$

where Δair void content is the actual air void content (expressed in decimal form) minus 0.03. Equation I-4 is valid only when

$$c_B \geq \frac{2}{3} (1 - c'_v) \tag{I-5}$$

where

$$c_B = \frac{bitumen\ volume}{aggregate\ volume + bitumen\ volume} \tag{I-6}$$

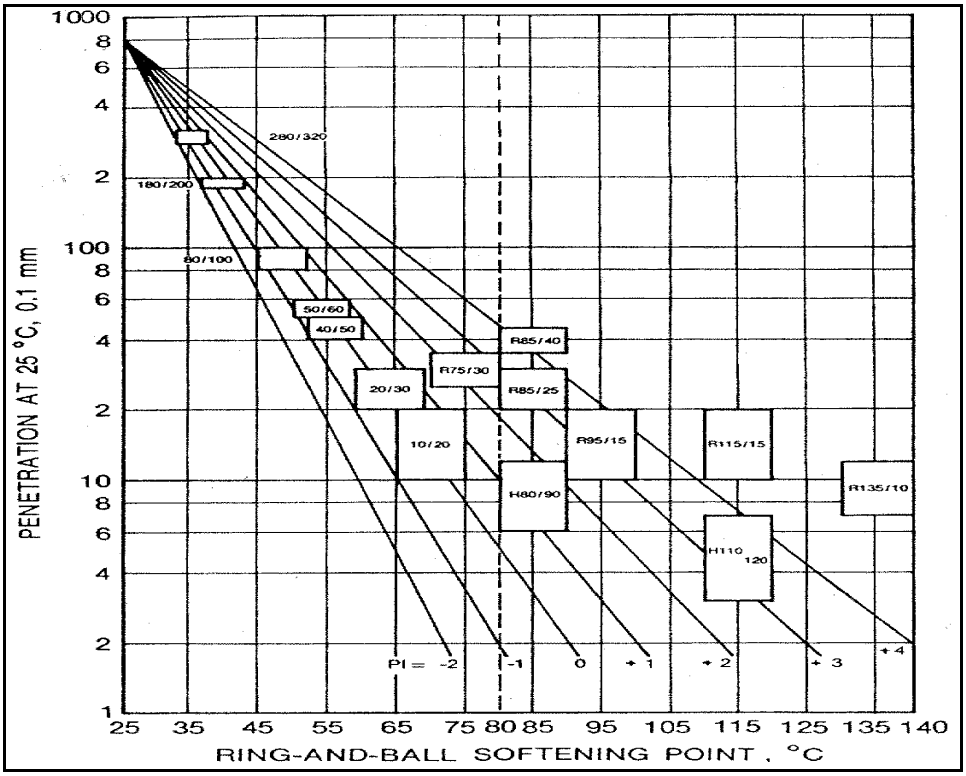


Figure I-1. Relationship between penetration at 25 degrees C and ring-and-ball softening point for bitumens with different PI's

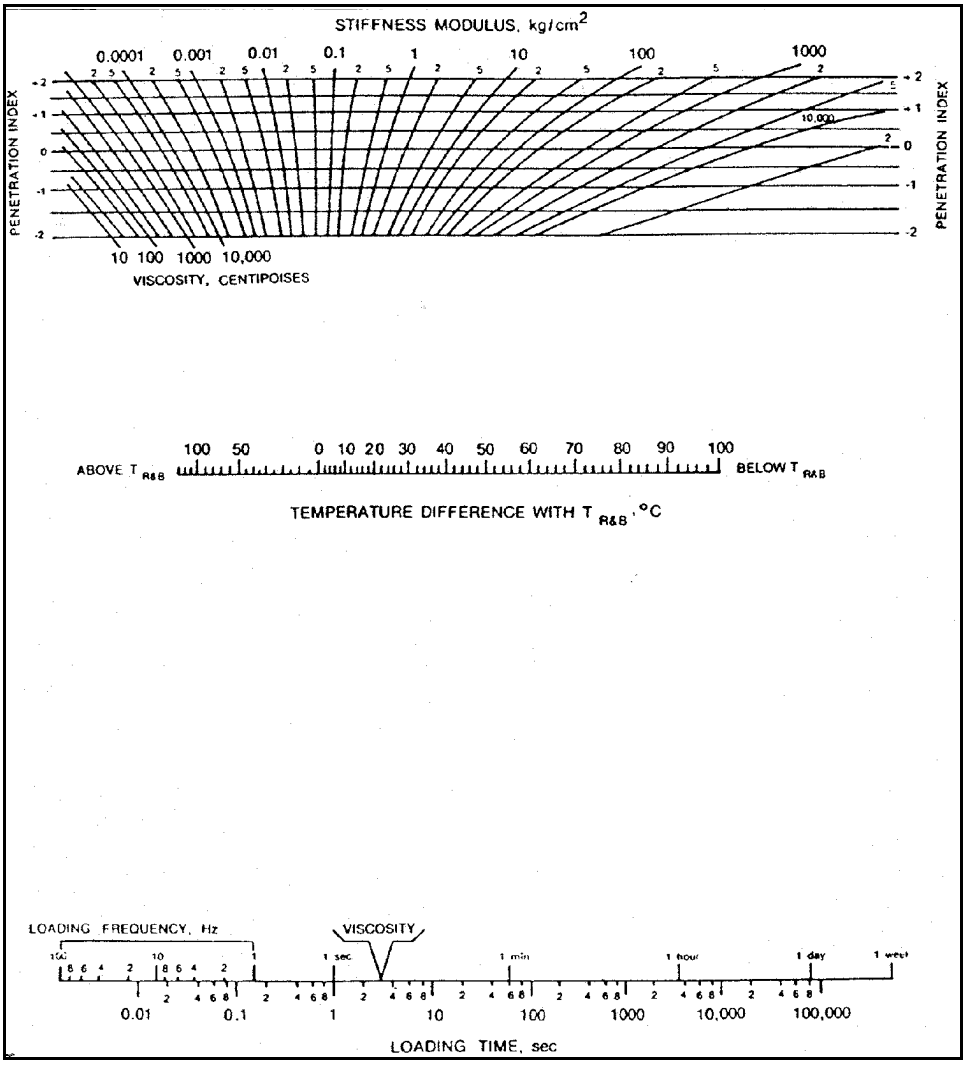


Figure I-2. Nomograph for determining the stiffness modulus of bitumens

APPENDIX J

PROCEDURE FOR DETERMINING THE MODULUS OF ELASTICITY OF UNBOUND GRANULAR BASE AND SUBBASE COURSE MATERIALS

J-1. PROCEDURE.

a. Relationships. The procedure is based on relationships developed for the resilient modulus of unbound granular layers as a function of the thickness of the layer and type of material. The modulus relationships are shown in Figure J-1. Modulus values for layer n (the upper layer) are indicated on the ordinate, and those for layer n + 1 (the lower layer) are indicated on the abscissa. Essentially linear relationships are indicated for various thicknesses of base- and subbase-course materials. For subbase courses, relationships are shown for thicknesses of 102, 127, 152, 178, and 203 millimeters (4, 5, 6, 7, and 8 inches). For subbase courses having a design thickness of 203 millimeters (8 inches) or less, the applicable curve or appropriate interpolation can be used directly. For a design subbase-course thickness in excess of 203 millimeters (8 inches), the layer should be divided into sublayers of approximately equal thickness and the modulus of each sublayer determined individually. For base courses, relationships are shown for thicknesses of 102, 152, and 254 millimeters (4, 6, and 10 inches). These relationships can be used directly or by interpolation for design base course thicknesses up to 254 millimeters (10 inches). For design thicknesses in excess of 254 millimeters (10 inches), the layer should also be divided into sublayers of approximately equal thickness and the modulus of each sublayer determined individually.

b. Modulus Values. To determine modulus values from this procedure, Figure J-1 is entered along the abscissa using modulus values of the subgrade or underlying layer (modulus of layer n + 1). At the intersection of the curve applicable to this value with the appropriate thickness relationship, the value of the modulus of the overlying layer is read from the ordinate (modulus of layer n). This procedure is repeated using the modulus value just determined as the modulus of layer n + 1 to determine the modulus value of the next overlying layer.

J-2. EXAMPLES.

a. Thickness. Assume a pavement having a base-course thickness of 102 millimeters (4 inches) and a subbase-course thickness of 203 millimeters (8 inches) over a subgrade having a modulus of 69 MPa (10,000 psi). Initially, the subgrade is assumed to be layer n + 1 and the subbase course to be layer n. Entering Figure I-1 with a modulus of layer n + 1 of 69 MPa (10,000 psi) and using the 203-millimeter (8-inch) subbase course curve, the modulus of the subbase (layer n) is 127.5 MPa (18,500 psi). In order to determine the modulus value of the base course, the subbase course is now assumed to be layer n + 1 and the base course to be layer n. Entering Figure J-1 with a modulus value of layer n + 1 of 127.5 MPa (18,500 psi) and using the 102 millimeter (4-inch) base-course relationship, the modulus of the base course is 248 MPa (36,000 psi). Modulus values determined for each layer are indicated in Figure J-2.

b. Design Thickness. If, in the first example, the design thickness of the subbase course had been 305 millimeters (12 inches), it would have been necessary to divide this layer into two 152-millimeter- (6-inch-) thick sublayers. Then, using the procedure described above for the second example, the modulus values determined for the lower and upper sublayers of the subbase course

and for the base course are 121, 176, and 303 MPa (17,500, 25,500, and 44,000 psi), respectively. These values are shown in Figure J-3.

c. Relationships. The relationships indicated in Figure I-1 can be expressed as

$$E_n = E_{n+1} (1 + 10.52 \log t - 2.10 \log E_{n+1} \log t) \quad \text{(J-1)}$$

where

n = a layer in the pavement system

E_n = resilient modulus (in psi) of layer n

E_{n+1} = the resilient modulus (in psi) of the layer beneath layer n

t = the thickness (in inches) of layer n for base-course materials and as

$$E_n = E_{n+1} (1 + 7.18 \log t - 1.56 \log E_{n+1} \log t) \quad \text{(J-2)}$$

for subbase-course materials. Use of these equation for direct computation of modulus values for the examples given above yields the value indicated in parentheses in Figures I-2 and I-3. It can be seen that comparable values are obtained with either graphical or computational determination of the modulus value for either material.

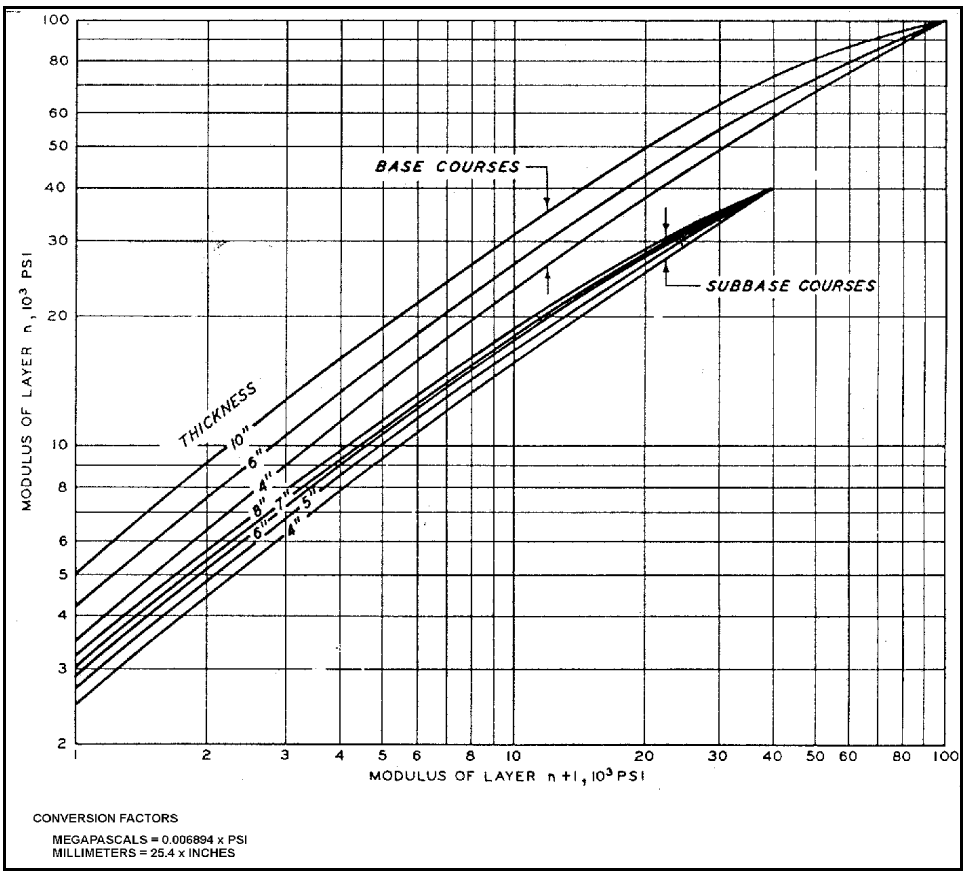


Figure J-1. Relationships between modulus of layer n and modulus of layer n + 1 for various thicknesses of unbound base course and subbase course

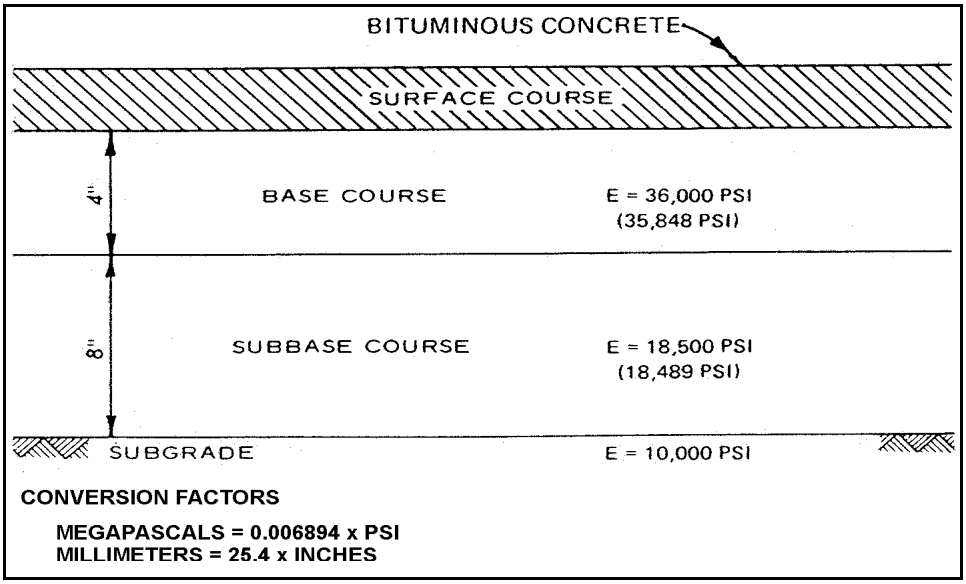


Figure J-2. Modulus values determined for first example

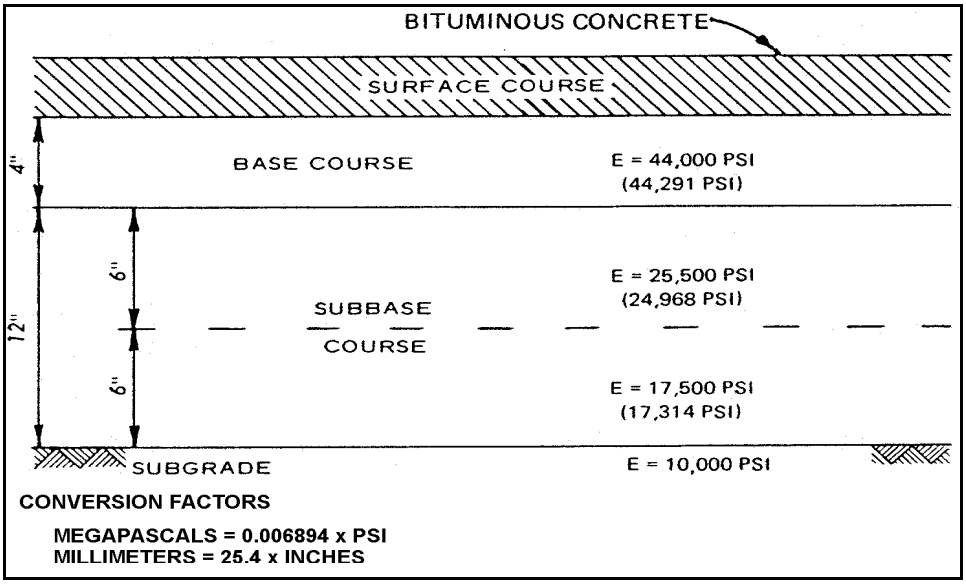


Figure J-3. Modulus values determined for second example

APPENDIX K

PROCEDURE FOR DETERMINING THE FLEXURAL MODULUS AND FATIGUE CHARACTERISTICS OF STABILIZED SOILS

K-1. LABORATORY PROCEDURE.

a. General. The procedure involves application of a repetitive loading to a laboratory-prepared beam specimen under controlled stress conditions. Applied load and deflection along the neutral axis and at the lower surface are monitored, and the results are used to determine the flexural modulus and fatigue characteristics.

b. Specimen Preparation. Beam specimens should be prepared following the general procedures indicated in ASTM D 1632. This method describes procedures for molding 76- by 76- by 286-millimeter (3- by 3- by 11-1/4-inch) specimens; however, any size mold may be used for the test. For soils containing aggregate particles larger than 19 millimeters (3/4 inch), it is recommended that molds on the order of 102 by 102 to 152 by 152 millimeters (4 by 4 to 6 by 6 inches) be used. In general, specimens should have an approximately square crosssectional configuration and a length adequate to accommodate an effective test span equal to three times the height or width. Specimens should be molded to the stabilizer treatment level, moisture content, and density expected in the field structures. Cement-treated materials should be moist-cured for 7 days. Lime-treated materials should be cured for 28 days at 23 degrees Celsius (73 degrees Fahrenheit).

c. Equipment. The following equipment is required:

(1) Loading frame capable of receiving specimen for third-point loading test.

(2) Electrohydraulic testing machine. This machine must be capable of applying static and haversine loads.

(3) Load cell (approximately 907-kilogram (2,000-pound) capacity).

(4) Two LVDT's and one SR-4 type strain gauge.

(5) Recording equipment for monitoring deflection, strain, and load.

(6) Miscellaneous pins and yokes, as described in the equipment setup below for mounting the LVDT'S.

d. Equipment Setup. Details of the equipment setup are shown in Figures K-1 to K-3. The beam should be positioned so that the molding laminations are horizontal. The three yokes are positioned over the top of the beam and held in place by threaded pins, positioned along the neutral axis. The end pins, pins A and C, are positioned directly over the end reaction points, and the middle pin, pin B, is positioned at the center of the beam. A metal bar rests on top of the pin. At the A position, the bar is equipped with a lower vertical tab having a hole that slips loosely over the pin. A nut is placed on the end of the pin to prevent the bar from slipping. At the center or B position, the bar is equipped with a vertical tap onto which an LVDT is cemented in a vertical

position. At this point on the bar, there is a hole through which the LVDT core pin falls to rest on the B pin. This pin must be fabricated with flat sides on the shaft to provide a horizontal surface on which the LVDT core pin rests. At the C position, the end of the bar simply rests on the unthreaded portion of the C pin. A nut is placed on the end of the C pin to prevent excessive side movement of the bar end. This type of bar, pin, and LVDT arrangement is provided on both sides of the beam. Although no dimensions are provided in Figures K-1 to K-3, this type of equipment can easily be dimensioned and fabricated to fit any size beam. Either steel or aluminum may be used. The beam should be positioned and arranged to accommodate third point loading as indicated in Figure K-2. As the beam bends under loading, defection at the center is measured by determining the movement of the LVDT stems from their original positions. The LVDTs are connected to the monitoring system to give an average deflection reading. Since it is also desired to determine the maximum tensile strain of the beam under loading, an SR-4 strain gauge should be attached to the lower beam surface with epoxy or some other suitable cement and should also be connected to the monitoring system. If it is not possible to determine strain directly, a strain value may be found using Equation K-2.

e. Test Procedure. The flexural beam test is a stress-controlled test. Therefore, an initial specimen should be statically loaded to failure, and the stress level for the initial repetitive load tests should be set at 50 percent of the maximum rupture load. The repetitive load test should be conducted using a haversine wave form, a loading duration of 0.5 second, and a frequency of about 1 hertz. To develop a strain repetition pattern, it is recommended that tests be conducted at 40, 50, 60, and 70 percent of the maximum rupture value; however, stress levels can be varied to higher or lower levels. Data to be monitored include load, deflection along the neutral axis, strain at the lower surface of the specimen, and number of repetitions.

f. Reporting of Test Results.

(1) Flexural Modulus. The flexural modulus should be determined at 100, 1,000, and 10,000 load repetitions or at failure. This value may be determined from load deflection data monitored at these repetition levels using the expression

$$E_f = \frac{23PL^3}{1296dI}\left(1 + 2.11\,\frac{h}{(L)}\right)^2 \qquad \text{(K-1)}$$

where

E_f = flexural modulus, MPa (psi)

P = maximum load amplitude, kilograms (pounds)

L = specimen length, millimeters (inches)

d = deflection at the neutral axis, millimeters (inches)

I = moment of inertia, millimeters^4 (inch^4)

h = specimen height, millimeters (inch)

The value to be used for E_f in the performance model is the arithmetic mean of all values obtained during the test.

(2) Fatigue characteristics. Fatigue characteristics are presented as a plot of strain indicated at the bottom surface of the specimen versus load repetitions at failure. Generally, the value of the strain obtained during the first few load repetitions is the value to be plotted. If no direct means of measuring strain is available, a strain value ϵ may be computed using the expression

$$\epsilon = \frac{PLh}{6E_f I} \qquad \text{(K-2)}$$

K-2. GRAPHICAL DETERMINATION OF FLEXURAL MODULUS FOR CHEMICALLY STABILIZED SOILS (CRACKED SECTION). The procedure for determining a flexural modulus value for chemically stabilized soils based on the cracked section concept involves the use of a relationship between unconfined compressive strength and flexural modulus determined analytically. This relationship is shown in Figure K-2. To use this relationship, specimens of the stabilized material should be molded and tested following procedures indicated in ASTM D 1633. Values obtained from the unconfined compression test can then be used to determine the values of the equivalent cracked section modulus using Figure K-2.

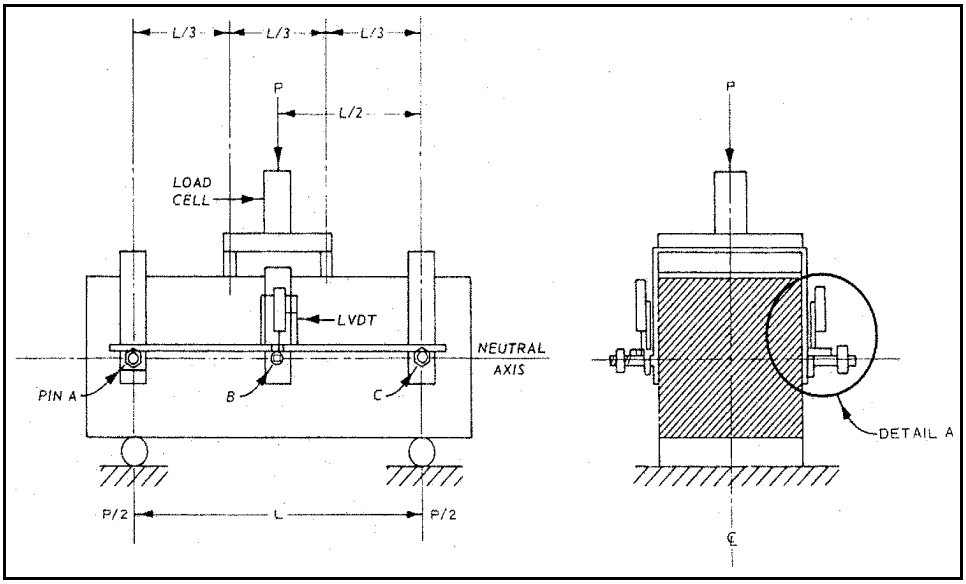


Figure K-1. General view of equipment setup

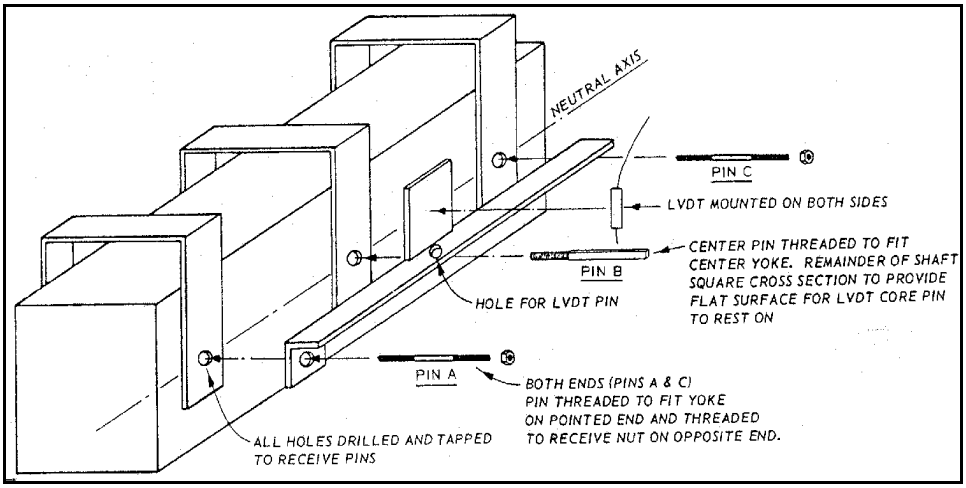


Figure K-2. Details of equipment setup

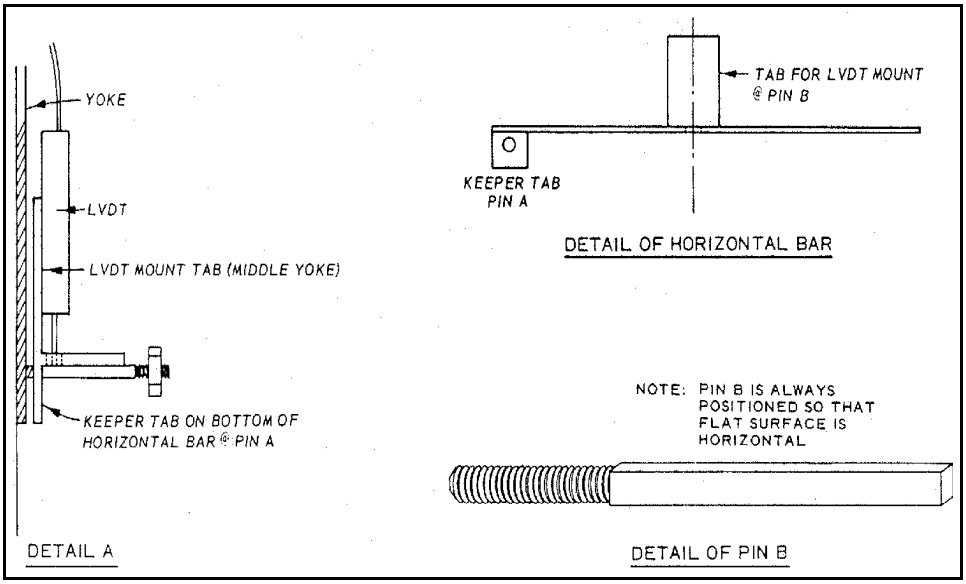


Figure K-3. Miscellaneous details

APPENDIX L

PROCEDURE FOR DETERMINING RESILIENT MODULUS OF SUBGRADE MATERIAL

L-1. GENERAL. The objective of this test procedure is to determine a modulus value for subgrade soils by means of resilient triaxial techniques. The test is similar to a standard triaxial compression test, the primary exception being that the deviator stress is applied repetitively and at several stress levels. This procedure allows testing of soil specimens in a repetitive stress state similar to that encountered by a soil in a pavement under a moving wheel load.

L-2. DEFINITIONS. The following symbols and terms are used in the description of this procedure:

σ_1 = total axial stress

σ_3 = total radial stress; i.e., confining pressure in the triaxial test chamber

$\sigma_d = \sigma_1 - \sigma_3$ = deviator strain; i.e, the repeated axial stress in this procedure

ϵ_1 = total axial strain due to σ_d

ϵ_R = resilient or recoverable axial strain due to σ_d

ϵ_{R1} = resilient or recoverable axial strain due to σ_d in the direction perpendicular to ϵ_R

$M_R = \sigma_d/\epsilon_{Rl}$ = resilient modulus

$\theta = \sigma_1 + 2\sigma_3 = \sigma_d + 3\sigma_3$ = sum of the principal stresses in the triaxial state of stress

σ_1/σ_3 = principal stress ratio

Load duration = time interval over which the specimen is subject to a deviator stress

Cycle duration = time interval between successive applications of a deviator stress

L-3. SPECIMENS. Various diameter soil specimens may be used in this test with the specimen height at least twice the diameter. Undisturbed or laboratory molded specimens can be used. Methods for laboratory preparation of molded specimens and for backpressure saturation of specimens are given in the following paragraphs.

L-4. PREPARATION OF SPECIMENS. Specimens shall have an initial height of not less than 2.1 times the initial diameter, though the minimum initial height of a specimen must be 2.25 times the diameter if the soil contains particles retained on the No. 4 sieve. The maximum particle size permitted in any specimen shall be no greater than one-sixth of the specimen diameter. Triaxial specimens 35.5, 71, 102, 152, 305, and 381 millimeters (1.4, 2.8, 4, 6, 12, and 15 inches) in diameter are most commonly used.

a. Cohesive Soils Containing Negligible Amounts of Gravel. Specimens 35.5 millimeters (1.4 inches) in diameter are generally satisfactory for testing cohesive soils containing a negligible amount of gravel, while specimens of larger diameter may be advisable for undisturbed soils having marked stratification, fissures, or other discontinuities. Depending on the type of sample, specimens shall be prepared by either of the following procedures:

(1) Trimming Specimens of Cohesive Soil. A sample that is uniform in character and sufficient in amount to provide a minimum of three specimens is required. For undisturbed soils, samples about 127 millimeters (5 inches) in diameter are preferred for triaxial tests using 35.5-millimeter-(1.4-inch-) diameter specimens. Specimens shall be prepared in a humid room and tested as soon as possible thereafter to prevent evaporation of moisture. Extreme care shall be taken in preparing the specimens to preclude the least possible disturbance to the structure of the soil. The specimens shall be prepared as follows:

(a) Cut a section of suitable length from the sample. As a rule, the specimens should be cut with the long axes parallel to the long axis of the sample; any influence of stratification is commonly disregarded. However, comparative tests can be made, if necessary, to determine the effects of stratification. When a 127-millimeter- (5-inch-) diameter undisturbed sample is to be used for 35.5-millimeter- (1.4-inch-) diameter specimens, cut the sample axially into quadrants using a wire saw or other convenient cutting tool. Use three of the quadrants for specimens; seal the fourth quadrant in wax and preserve it for a possible check test.

(b) Carefully trim each specimen to the required diameter, using a trimming frame or similar equipment. Use one side of the trimming frame for preliminary cutting and the other side for final trimming. Ordinarily, the specimen is trimmed by pressing the wire saw or trimming knife against the edges of the frame and cutting from top to bottom. In trimming stiff or varved clays, move the wire saw from the top and bottom toward the middle of the specimen to prevent breaking off pieces at the ends. Remove any small shells or pebbles encountered during the trimming operations. Carefully fill voids on the surface of the specimen with remolded soil obtained from the trimming. Cut specimen to the required length (usually 76 to 89 millimeters (3 to 3-1/2 inches) for 35.5-millimeter- (1.4-inch-) diameter specimens and 152 to 178 millimeters (6 to 7 inches) for 71-millimeter- (2.8-inch-) diameter specimens) using a miter box.

(c) From the soil trimmings, obtain 200 grams of material for specific gravity and water content determination.

(d) Weigh the specimen to an accuracy of ± 0.01 gram for 35.5-millimeter- (1.4-inch-) diameter specimens and ± 0.1 grams for 71-millimeter- (2.8-inch-) diameter specimens.

(e) Measure the height and diameter of the specimen to an accuracy of ± 0.25 millimeters (0.01 inch). Specimen dimensions based on measurements of the trimming frame guides and miter box length are not sufficiently accurate. The average height H_o of the specimen should be determined from at least four measurements, while the average diameter should be determined from measurements at the top, center, and bottom of the specimen, as follows:

$$D_o = \frac{D_t + 2D_c + D_b}{4} \qquad \text{(L-1)}$$

where

D_o = average diameter

D_t = diameter at top

D_C = diameter at center

D_b = diameter at bottom

(2) Compacting Specimens of Cohesive Soil. Specimens of compacted soil may be trimmed, as described above, from samples formed in a compaction mold (a 102-millimeter- (4-inch-) diameter sample is satisfactory for 35.5-millimeter- (1.4-inch-) diameter specimens), though it is preferable to compact individual specimens in a split mold having inside dimensions equal to the dimensions of the desired specimen. The method of compacting the soil into the mold should duplicate as closely as possible the method that will be used in the field. In general, the standard impact type of compaction will not produce the same soil structure and stress-deformation characteristics as the kneading action of the field compaction equipment. Therefore, the soil should preferably be compacted into the mold (whether a specimen-size or a standard compaction mold) in at least six layers, using a pressing or kneading action of a tamper having an area in contact with the soil of less than one-sixth the area of the mold, and thoroughly scarifying the surface of each layer before placing the next. The sample shall be prepared, thoroughly mixed with sufficient water to produce the desired water content, and then stored in an airtight container for at least 16 hours. The desired density may be produced by either kneading or tamping each layer until accumulative weight of soil placed in the mold is compacted to a known volume, or adjusting the number of layers, the number of tamps per layer, and the force per tamp. For the latter method of control, special constant-force tampers (such as the Harvard miniature compactor for 35.5-millimeter- (1.4-inch-) diameter specimens or similar compactors for 71-millimeter- (2.8-inch-) diameter and larger specimens) are necessary. After each specimen compacted to finished dimensions has been removed from the mold, proceed in accordance with steps c through e of (1) above.

b. Cohesionless Soils Containing Negligible Amounts of Gravel. Soils which possess little or no cohesion are difficult if not impossible to trim into a specimen. If undisturbed samples of such materials are available in sampling tubes, satisfactory specimens can usually be obtained by freezing the sample to permit cutting out suitable specimens. Samples should be drained before freezing. The frozen specimens are placed in the triaxial chamber, allowed to thaw after application of the chamber pressure, and then tested as desired. Some slight disturbance probably occurs as a result of the freezing, but the natural stratification and structure of the material are retained. In most cases, however, it is permissible to test cohesionless soils in the remolded state by forming the specimen at the desired density or at a series of densities which will permit interpolation to the desired density. Specimens prepared in this manner should generally be 71 millimeters (8 inches) in diameter or larger, depending on the maximum particle size. The procedure for forming the test specimen shall consist of the following steps:

(1) Oven-dry and weigh an amount of material sufficient to provide somewhat more than the desired volume of specimen.

(2) Place the forming jacket, with the membrane inside, over the specimen base of the triaxial compression device.

(3) Evacuate the air between the membrane and the inside face of the forming jacket.

(4) After mixing the dried material to avoid segregation, place the specimen, by means of a funnel or special spoon, inside the forming jacket in equal layers. For 71-millimeter- (8-inch-) diameter specimens, 10 layers of equal thickness are adequate. Starting with the bottom layer, compact each layer by blows with a tamping hammer, increasing the number of blows per layer linearly with the height of the layer above the bottom layer. The total number of blows required for a specimen of a given material will depend on the density desired. Considerable experience is usually required to establish the proper procedure for compacting a material to a desired uniform density by this method. A specimen formed properly in the above-specified manner, when confined and axially loaded, will deform symmetrically with respect to its midheight, indicating that a uniform density has been obtained along the height of the specimens.

(5) As an alternate procedure, the entire specimen may be placed in a loose condition by means of a funnel or special spoon. The desired density may then be achieved by vibrating the specimen in the forming jacket to obtain a specimen of predetermined height and corresponding density. A specimen formed properly in this manner, when confined and axially loaded, will deform symmetrically with respect to its mid height.

(6) Subtract weight of unused material from original weight of the sample to obtain weight of material in the specimen.

(7) After the forming jacket is filled to the desired height, place the specimen cap on the top of the specimen, roll the ends of the membrane over the specimen cap and base, and fasten the ends with rubber bands or o-rings. Apply a low vacuum to the specimen through the base and remove the forming jacket.

(8) Measure height and diameter as specified in paragraph L-4.

c. Soils Containing Gravel. The size of specimens containing appreciable amounts of gravel is governed by the requirements of this paragraph. If the material to be tested is in an undisturbed state, the specimens shall be prepared according to the applicable requirements of a and b above, with the size of specimen based on an estimate of the largest particle size. In testing compacted soils, the largest particle size is usually known, and the entire sample should be tested, whenever possible, without removing any of the coarser particles. However, it may be necessary to remove the particles larger than a certain size to comply with the requirements for specimen size, though such practice will result in lower measured values of the shear strength and should be avoided if possible. Oversize particles should be removed and, if comprising more than 10 percent by weight of the sample, be replaced by an equal percentage by weight of material retained on the No. 4 sieve and passing the maximum allowable sieve size. The percentage of material finer than the No. 4 sieve thus remains constant. It will generally be necessary to prepare compacted samples of material containing gravel inside a forming jacket placed on the specimen base. If the material is cohesionless, it should be oven-dried and compacted in layers inside the membrane and forming jacket using the procedure in b above as a guide. When specimens of very high density are required, the samples should be compacted preferably by vibration to avoid rupturing the membrane. The use of two membranes will provide additional insurance against possible leakage during the test as a result of membrane rupture. If the sample contains a significant amount of fine-grained material, the soil usually must posses the proper water content before it can be compacted to the desired density. Then, a special split compaction mold is used for forming the specimen. The inside dimensions of the mold are equal to the dimensions of the triaxial specimen

desired. No membrane is used inside the mold, as the membrane can be readily placed over the compacted specimen after it is removed from the split mold. The specimen should be compacted to the desired density in accordance with paragraph L-4.

L-5. Q TEST WITH BACK-PRESSURE SATURATION.

a. Equipment Setup. For the Q test with back-pressure saturation, the apparatus should be set up similar to that shown in Figure L-1. Filter strips should not be used and as little volume changes as possible should be permitted during the test. Complete the steps outlined in paragraph L-4 and the following steps:

(1) Record all identifying information for the sample project number or name, boring number, and other pertinent data on a sheet.

(2) Place one of the prepared specimens on the base.

(3) Place a rubber membrane in the membrane stretcher, turn both ends of the membrane over the ends of the stretcher, and apply a vacuum to the stretcher. Carefully lower the stretcher and membrane over the specimen. Place the specimen cap on the top of the specimen and release the vacuum on the membrane stretcher. Turn the ends of the membrane down around the base and up around the specimen cap and fasten the ends with o-rings or rubber bands. With a 35.5-millimeter- (1.4-inch-) diameter specimen of relatively insensitive soils, it is easier to roll the membrane over the specimen.

(4) Assemble the triaxial chamber and place it in position in the loading device. Connect the tube from the pressure reservoir to the base of the triaxial chamber. With valve C on the pressure reservoir closed and valves A and B open, increase the pressure inside the reservoir and allow the pressure fluid to fill the triaxial chamber. Allow a few drops of the pressure fluid to escape through the vent valve (valve B) to ensure complete filling of the chamber with fluid. Close valve A and the vent valve.

b. Back-Pressure Procedure. Then apply a 0.02-MPa (3-psi) chamber pressure to the specimen with all drainage valves closed. Allow a minimum of 30 minutes for stabilization of the specimen pore water pressure, measure the change of deformation ΔH, and begin back-pressure procedures as follows:

(1) Estimate the magnitude of the required back pressure by theoretical relations. Specimens should be completely saturated before any appreciable consolidation is permitted for ease and uniformity of saturation as well as to allow volume changes during consolidation to be measured with the burette; therefore, the difference between the chamber pressure and the back pressure should not exceed 0.034 MPa (5 psi) during the saturation phase. To ensure that a specimen is not prestressed during the saturation phase, the back pressure must be applied in small increments, with adequate time between increments to permit equalization of pore water pressure throughout the specimen.

(2) With all valves closed, adjust the pressure regulators to a chamber pressure of about 0.048 MPa (7 psi) and a back pressure of about 0.013 MPa (2 psi). Record these pressures on a data sheet. Next, open valve A to apply the back pressure through the specimen cap. Immediately, open valve G and read and record the pore pressure at the specimen base. When the measured pore pressure becomes essentially constant, close valves F and G and record the burette

reading. (If an electrical pressure transducer is used to measure the pore pressure, valve G may be safely left open during the entire saturation procedure).

(3) Using the technique described above, increase the chamber pressure and the back pressure in increments, maintaining the back pressure at about 0.034 MPa (5 psi) less than the chamber pressure. The size of each increment might be 0.034, 0.069, or even 0.138 MPa (5, 10, or even 20 psi) depending on the compressibility of the soil specimen and the magnitude of the desired consolidation pressure. Open valve G and measure the pore pressure at the base immediately upon application of each increment of back pressure and observe the pore pressure until it becomes essentially constant. The time required for stabilization of the pore pressure may range from a few minutes to several hours depending on the permeability of the soil. Continue adding increments of chamber pressure and back pressure until, under any increment, the pore pressure reading equals the applied back pressure immediately upon opening valve G.

(4) Verify the completeness of saturation by closing valve F and increasing the chamber pressure by about 0.034 MPa (5 psi). The specimen shall not be considered completely saturated unless the increase in pore pressure immediately equals the increase in chamber pressure.

c. Chamber Pressure. After verification of saturation, and remeasurement of ΔH, close all drainage lines leading to the back pressure and pore water measurement apparatus. Holding the maximum applied back pressure constant, increase the chamber pressure until the difference between the chamber pressure and the back pressure equals the desired effective confining pressure as follows. With valves A and C closed, adjust the pressure regulator to preset the desired chamber pressure. The range of chamber pressures for the three specimens will depend on the loadings expected in the field. The maximum confining pressure should be at least equal to the maximum normal load expected in the field so that the shear strength data need not be extrapolated for use in design analysis. Record the chamber pressure on data sheets. Now open valve A and apply the preset pressure to the chamber. Application of the chamber pressure will force the piston upward into contact with the ram of the loading device. This upward force is equal to the chamber pressure acting on the cross-sectional area of the piston minus the weight of the piston minus piston friction.

d. Operation.

(1) Start the test with the piston approximately 2.5 millimeters (0.1 inch) above the specimen cap. This allows compensation for the effects of piston friction, exclusive of that which may later develop as a result of lateral forces. Set the load indicator to zero when the piston comes into contact with the specimen cap. In this manner, the upward thrust of the chamber pressure on the piston is also eliminated from further consideration. Contact of the piston with the specimen cap is indicated by a slight movement of the load indicator. Set the strain indicator and record on the data sheet the initial dial reading at contact. Axially strain the specimen at a rate of about 1 percent per minute for plastic materials or about 0.3 percent per minute for brittle materials that achieve maximum deviator stress at about 3 to 6 percent strain; at these rates, the elapsed time to reach maximum deviator stress would be about 15 to 20 minutes.

(2) Observe and record the resulting load at every 0.3 percent strain for about the first 3 percent and, thereafter, at every 1 percent or, for large strains, at every 2 percent strain; sufficient readings should be taken to completely define the shape of the stress-strain curve so frequent readings may be necessary as failure is approached. Continue the test until an axial strain of 15 percent has been reached; however, when the deviator stress decreases after attaining a

maximum value and is continuing to decrease at 15 percent strain, the test shall be continued to 20 percent.

(3) For brittle soils (i.e., those in which maximum deviator stress is reached at 6 percent axial strain or less), tests should be performed at rates of strain sufficient to produce times to failure as set forth above; however, when the maximum deviator stress has been clearly defined, the rate may be increased such that the remainder of the test is completed in the same length of time as that taken to reach maximum deviator stress. However, for each group of tests about 20 percent of the samples should be tested at the rates set forth above.

(4) Upon completion of axial loading, release the chamber pressure by shutting off the air supply with the regulator and opening valve C. Open valve B and draw the pressure fluid back into the pressure reservoir by applying a low vacuum at valve C. Dismantle the triaxial chamber. Make a sketch of the specimen, showing the mode of failure.

(5) Remove the membrane from the specimen. For 35.5-millimeter- (1.4-inch-) diameter specimens, carefully blot any excess moisture from the surface of the specimen and determine the water content of the whole specimen. For 71-millimeter- (2.8-inch-) diameter or larger specimens, it is permissible to use a representative portion of the specimen for the water content determination. It is essential that the final water content be determined accurately, and weighings should be verified, preferably by a different technician.

(6) Repeat the test on the two remaining specimens at different chamber pressures, though using the same rate of strain.

L-6. EQUIPMENT.

a. Triaxial Test Cell.

(1) A triaxial cell suitable for use in resilience testing of soils is shown in Figure L-2. This equipment is similar to most standard cells, except that it is somewhat larger so that it can facilitate the internally mounted load and deformation measuring equipment and the equipment has additional outlets for the electrical leads from the measuring devices. For the type of equipment shown, air or nitrogen is used as the cell fluid.

(2) The external loading source may be any device capable of providing a variable load of fixed cycle and load duration, ranging from a simple cam-and-switch control of static weights or air pistons to a closed-loop electrohydraulic system. A load duration of 0.2 seconds and a cycle duration of 3 seconds have been satisfactory for most applications. A square-wave load form is recommended.

b. Deformation-Measuring Equipment.

(1) The deformation-measuring equipment consists of LVDTs attached to the soil specimen by a pair of clamps. Two LVDTs are used for the measurement of axial deformation. The clamps and LVDTs are shown in position on a soil specimen in Figure L-2. Details of the clamps are shown in Figure L-3. Load is measured by placing a load cell between the specimen cap and the loading piston as shown in Figure L-2.

(2) Use of the type of measuring equipment described above offers several advantages:

(a) It is not necessary to reference deformations to the equipment, which deforms during loading.

(b) The effect of end-cap restrain on soil response is virtually eliminated.

(c) Any effects of piston friction are eliminated by measuring loads inside the triaxial cell.

(3) In addition to the measuring devices it is also necessary to maintain suitable recording equipment. Simultaneous recording of load and deformation is desirable. The number of recording channels can be reduced by wiring the leads from the LVDTs so that only the average signal from each pair is recorded. The introduction of switching and balancing units permits use of a single-chamber recorder.

c. Additional Equipment. In addition to the equipment described above, the following items are also used:

(1) A 9- to 27-metric ton- (10- to 30-short ton-) capacity loading machine.

(2) Calipers, a micrometer gauge, and a steel rule (calibrated to 0.25 millimeter (0.01 inch)).

(3) Rubber membranes, 0.25 to 0.635 millimeter (0.01 to 0.025 inch) thick.

(4) Rubber O-rings.

(5) A vacuum source with a bubble chamber and regulator.

(6) A back-pressure chamber with pressure transducers.

(7) A membrane stretcher.

(8) Porous stones.

L-7. PREPARATION OF SPECIMENS AND PLACEMENT IN TRIAXIAL CELL. The following steps should be followed in preparing and placing specimens:

a. In accordance with procedures specified in paragraph L-5, prepare the specimen and place it on the baseplate complete with porous stones, cap, and base and equipped with a rubber membrane secured with o-rings. Check for leakage. If back-pressure saturation is anticipated for cohesive soils, procedures indicated in paragraph L-5a should be followed. For purely noncohesive soils, it will be necessary to maintain the vacuum during placement of the LVDTs. The specimen is now ready to receive the LVDTs.

b. Extend the lower LVDT clamp and slide it carefully down over the specimen to approximately the lower third point of the specimen.

c. Repeat this step for the upper clamp, placing it at the upper third point. Ensure that both clamps lie in horizontal planes.

d. Connect the LVDTs to the recording unit, and balance the recording bridges. This step will require recorder adjustments and adjustment of the LVDT stems. When a recording bridge balance has been obtained, determine (to the nearest 0.25 millimeter (0.01 inch)) the vertical spacing between the LVDT clamps and record this value.

e. Place the triaxial chamber in position. Set the load cell in place on the specimen.

f. Place the cover plate on the chamber. Insert the loading piston and obtain a firm connection with the load cell.

g. Tighten the tie rods firmly.

h. Slide the assembled apparatus into position under the axial loading device. Bring the loading device to a position in which it nearly contacts the loading piston.

I. If the specimen is to be back-pressure saturated, proceed in accordance with paragraph L-5.

j. After saturation has been completed, rebalance the recorder bridge to the load cell and LVDTs.

L-8. RESILIENCE TESTING OF COHESIVE SOILS.

a. General. The resilient properties of cohesive soils are only slightly affected by the magnitude of the confining pressure σ_3. For most applications, this effect can be disregarded. When back-pressure saturation is not used, the confining pressure used should approximate the expected in situ horizontal stresses. These will generally be on the order of 0.0069 to 0.034 MPa (1 to 5 psi). A chamber pressure of 0.021 MPa (3 psi) is a reasonable value for most testing. If back-pressure saturation is used, the chamber pressure will depend on the required saturation pressure.

b. Resilient Properties. Resilient properties are highly dependent on the magnitude of the deviator stress σ_d. It is therefore necessary to conduct the tests for a range in deviator stress values. The following procedure should be followed:

(1) If back-pressure saturation is not used, connect the chamber pressure supply line and apply the confining pressure (equal to the chamber pressure). If back-pressure saturation is used, the chamber pressure will already have been established.

(2) Rebalance the recording bridges for the LVDTs and balance the load cell recording bridge.

(3) Begin the test by applying 500 to 1,000 repetitions of a deviator stress of not more than one-half the unconfined compressive strength.

(4) Decrease the deviator load to the lowest value to be used. Apply 200 repetitions of load, recording the recovered vertical deformation at or near the last repetition.

(5) Increase the deviator load, recording deformations as in step 4. Repeat over the range of deviator stresses to be used.

(6) At the completion of the loading, reduce the chamber pressure to zero. Remove the chamber LVDTs and load cell. Use the entire specimen for the purpose of determining the moisture content.

c. The results of the resilience tests can be presented in the form of a summary table, such as Table L-1, and graphically as is shown in Figure L-4 for the resilient modulus.

L-9. RESILIENCE TESTING OF COHESIONLESS SOILS.

a. General. The resilient modulus of cohesionless soils M_R is dependent upon the magnitude of the confining pressure σ_3 and is nearly independent of the magnitude of the repeated axial stress. Therefore, it is necessary to test cohesionless materials over a range of confining and axial stresses. (The confining pressure is equal to the chamber pressure less the back pressure for saturated specimens). The following procedure should be used for this type of test:

(1) Use confining pressures of 0.034, 0.069, 0.103, and 0.138 MPa (5, 10, 15, and 20 psi) at each confining pressure, and test at five values of the principal stress difference corresponding to multiples (1, 2, 3, 4) of the cell pressure.

(2) Before beginning to record deformations, apply a series of conditioning stresses to the material to eliminate initial loading effects. The greatest amount of volume change occurs during the application of the conditioning stresses. Simulation of field conditions suggest that drainage of saturated specimens should be permitted during the application of these loads but that the test loading (beginning in step 6 below) should be conducted in an undrained state.

(3) Set the axial load generator to apply a deviator stress of 0.069 MPa (10 psi) (i.e., a stress ratio equal to 3). Activate the load generator and apply 200 repetitions of this load. Stop the loading.

(4) Set the axial load generator to apply a deviator stress of 0.138 MPa (20 psi) (i.e., a stress ratio equal to 5). Activate the load generator and apply 200 repetitions of this load. Stop the loading.

(5) Repeat as in step 4 above maintaining a stress ratio equal to 6 and using the following order and magnitude of confining pressures: 0.069, 0.138, 0.069, 0.034, 0.021, and 0.0069 MPa (10, 20, 10, 5, 3, and 1 psi).

(6) Begin the record test using a confining pressure of 0.0069 MPA (1 psi) and an equal value of deviator stress. Record the resilient deformation after 200 repetitions. Increase the deviator stress to twice the confining pressure and record the resilient deformation after 200 repetitions. Repeat until a deviator stress of four times the confining pressure is reached (stress ratio of 5).

(7) Repeat as in step 6 above for each value of confining pressure.

(8) When the test is completed, decrease the back pressure to zero, reduce the chamber pressure to zero, and dismantle the cell. Remove the LVDT clamps, etc. Remove the soil specimen, and use the entire amount of soil to determine the moisture content.

b. Calculations. Calculations can be performed using a similar tabular arrangement as was shown in Table L-1. Test results should be presented in the form of a plot of log M_R versus log of the sum of the principal stresses as shown in Figure L-5.

L-10. INTERPRETATION OF TEST RESULTS.

a. Cohesive Soils. As previously indicated, test results for cohesive soils are presented in the form of a plot of resilient modulus M_R versus deviator stress σ_d. Normally for cohesive soils, the test results will indicate that the resilient modulus decreases rapidly with increases in deviator stress. Thus, selection of a resilient modulus from the laboratory tests results requires an estimate of the deviator stress at the top of the subgrade with respect to the design aircraft. For a properly designed pavement, the deviator stress at the top of the subgrade will primarily be a function of the subgrade modulus and the design traffic level. Shown in Figure L-6 are relationships between deviator stress at the top of the subgrade and applicable subgrade modulus values determined from an analysis of the pavement sections. The relationships shown in Figure L-6 were determined using a layered elastic pavement model with the modulus values as input parameters and the deviator stress values as computed responses. Thus, these relationships are essentially limiting criteria. Relationships are shown for 1,000, 10,000, 100,000, and 1,000,000 repetitions of strain. To determine the appropriate modulus value to use in the performance model, the test results from the resilient modulus tests on the laboratory specimens are superimposed on the appropriate relationship from Figure L-6, and the design modulus value is taken from the intersection of the plotted functions.

b. Example on Cohesive Soils. Assume a design problem involving 100,000 repetitions of strain. Figure L-7 shows a plot of relationships taken from Figure L-6 superimposed on test results from a laboratory resilient modulus test. For this particular design, a subgrade modulus value of 62 MPa (9,000 psi) would be used.

c. Cohesionless Soils. For cohesionless soils, laboratory test results are presented in the form of a plot of resilient modulus versus the first stress invariant, i.e., sum of the principal stress θ. For cohesionless soils, this relationship is generally linear in form on a log-log plot, with the resilient modulus being directly proportional to the sum of the principal stresses. Selection of a specific resilient modulus value for use in the design model requires an estimate of the sum of the principal stresses at the top of the subgrade. Since a cohesionless material is involved, the influence of both applied stresses and estimated overburden stresses from the pavement structure must be considered. In Figure L-8, a relationship is shown between the pavement thickness and the sum of the principal stresses at the top of the subgrade due to overburden. In Figure L-9, relationships are shown between the subgrade modulus and limiting values of the sum of the principal stresses due to applied force. For each figure, relationships are shown for 1,000, 10,000, 100,000, and 1,000,000 repetitions of stress. Using the value of the estimated pavement thickness, that part of the total sum of the principal stresses due to overburden can be obtained from Figure L-8. The applicable relationship from Figure L-9 is then selected and adjusted to include the influence of overburden by increasing all values of the principal stress sum by the value obtained from Figure L-8. Thus, a new limiting relationship is obtained and replotted. The results of the laboratory modulus test are superimposed on the plot, and the design subgrade modulus values are taken at the intersection of these relationships.

d. Example on Cohesionless Soils. Assume a design problem involving a pavement having an estimated initial thickness of 762 millimeters (30 inches). The design aircraft has a dual-wheel main gear assembly, and the design life is for 100,000 repetitions of strain. From Figure L-8, the

value of the sum of the principal stresses due to overburden is 0.045 MPa (6.5 psi). Using the 100,000 strain repetition curve from Figure L-9, the value obtained from Figure L-8 is added to all values of the sum of the principal stresses indicated in the relationship and the adjusted curve is replotted (Figure L-10). The result of adjusting the original relationship is to shift it to the right of its original position. In Figure G-10, the results of laboratory resilient modulus tests on specimens of the subgrade soil are also shown. From the intersection of these two relationships, a design modulus M_R of 103 MPa (15,000 psi) is determined.

e. Special Considerations. In some situations, the laboratory curve may not converge with the limiting stress-modulus relationship within the range of values indicated. Obviously, two possibilities are involved in this situation: the laboratory relationships could plot above or below the limiting criteria curve. In the former case, since all values of the sum of the principal stresses indicated by the laboratory curve would exceed the stress criteria within the region under consideration, the value of 207 MPa (30,000 psi) should be used for the subgrade modulus. In the latter case, the initial design thickness value should be increased and the limiting criteria curve readjusted until convergence with the laboratory relationships is obtained.

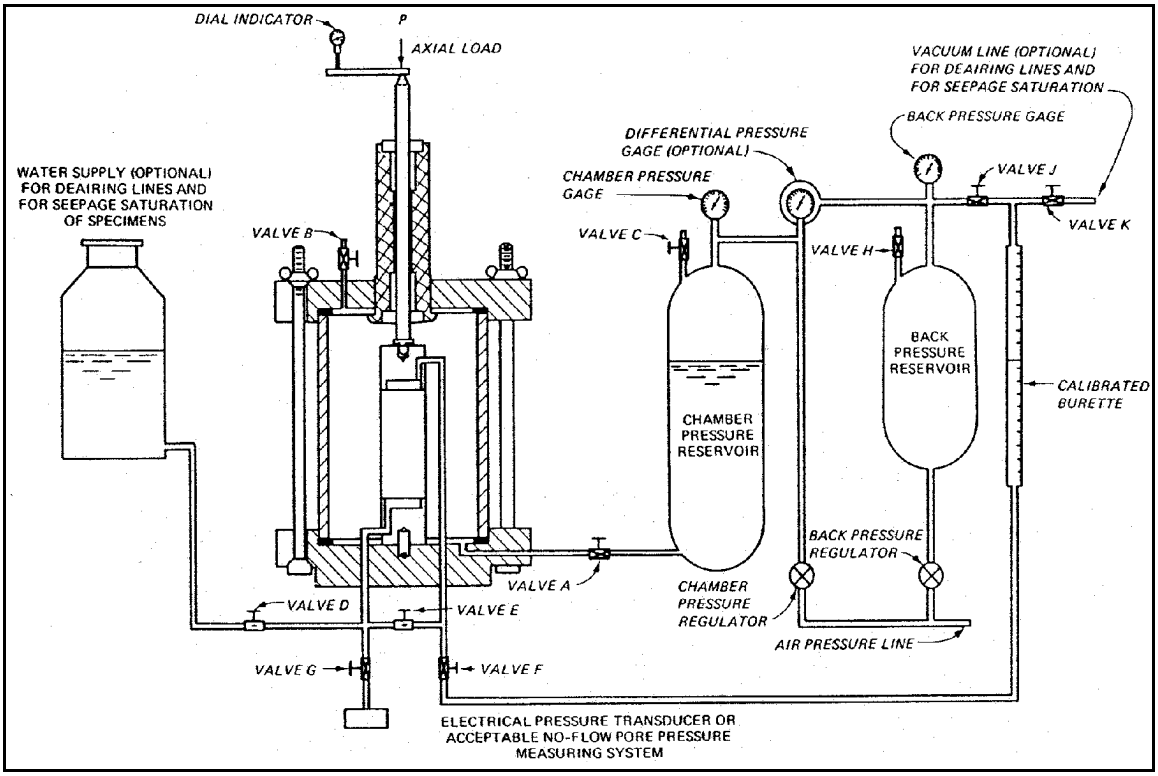


Figure L-1. Schematic diagram of typical triaxial compression apparatus

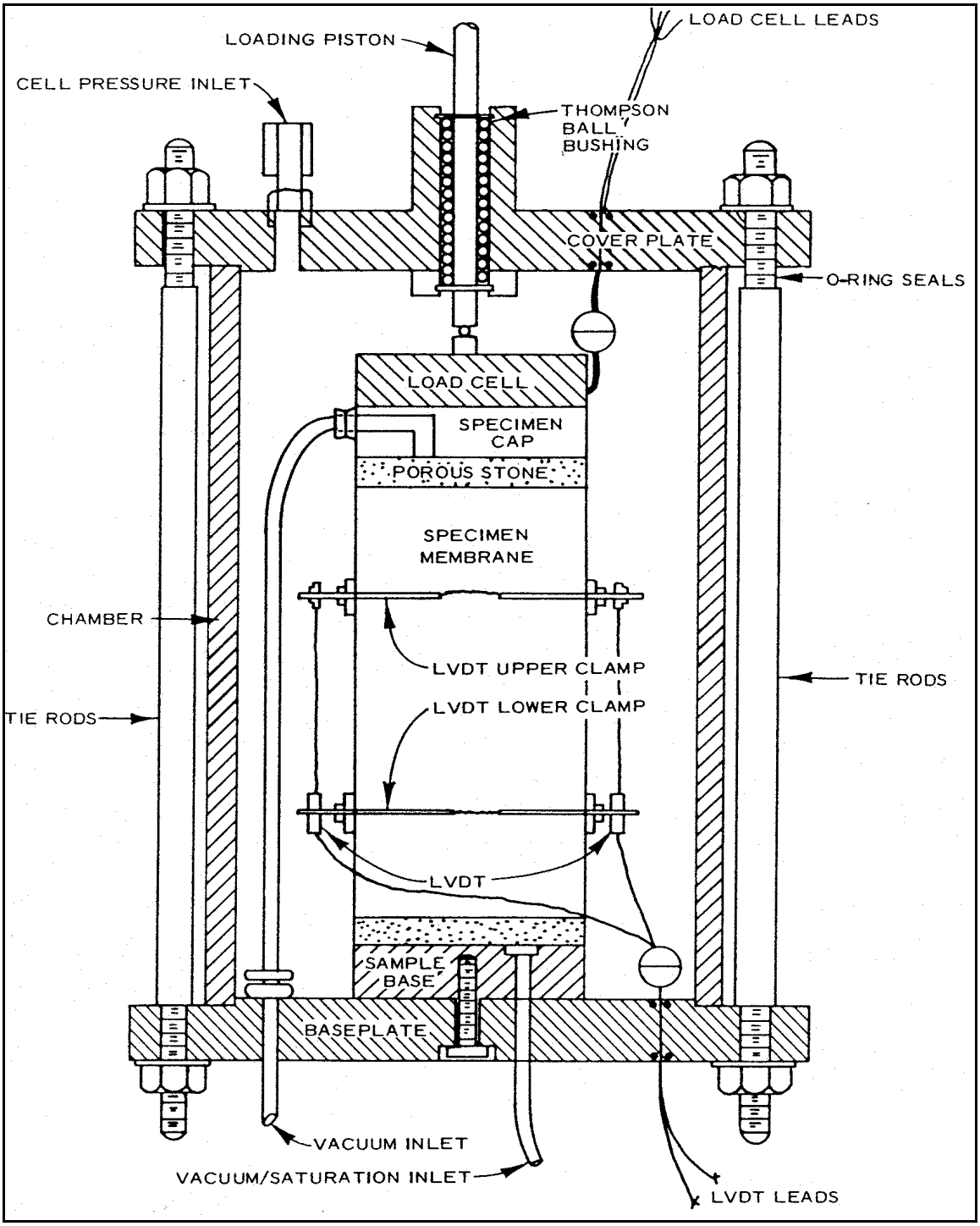


Figure L-2. Triaxial cell

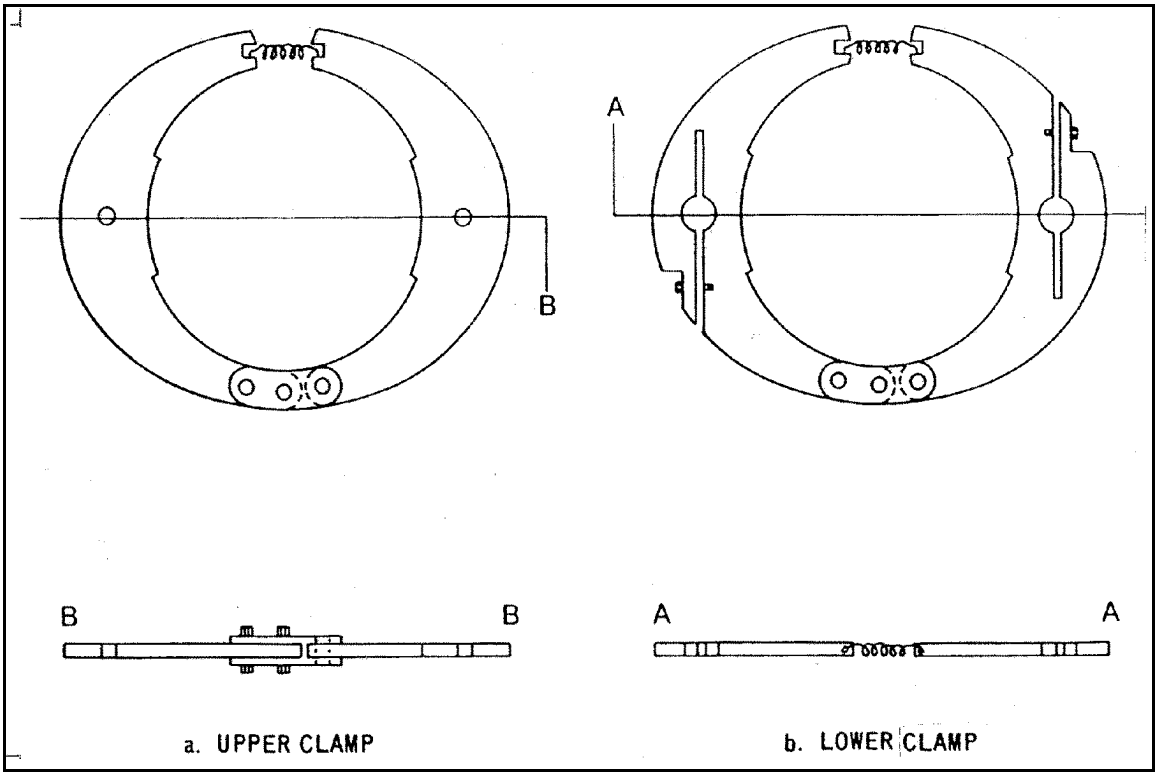


Figure L-3. LVDT clamps

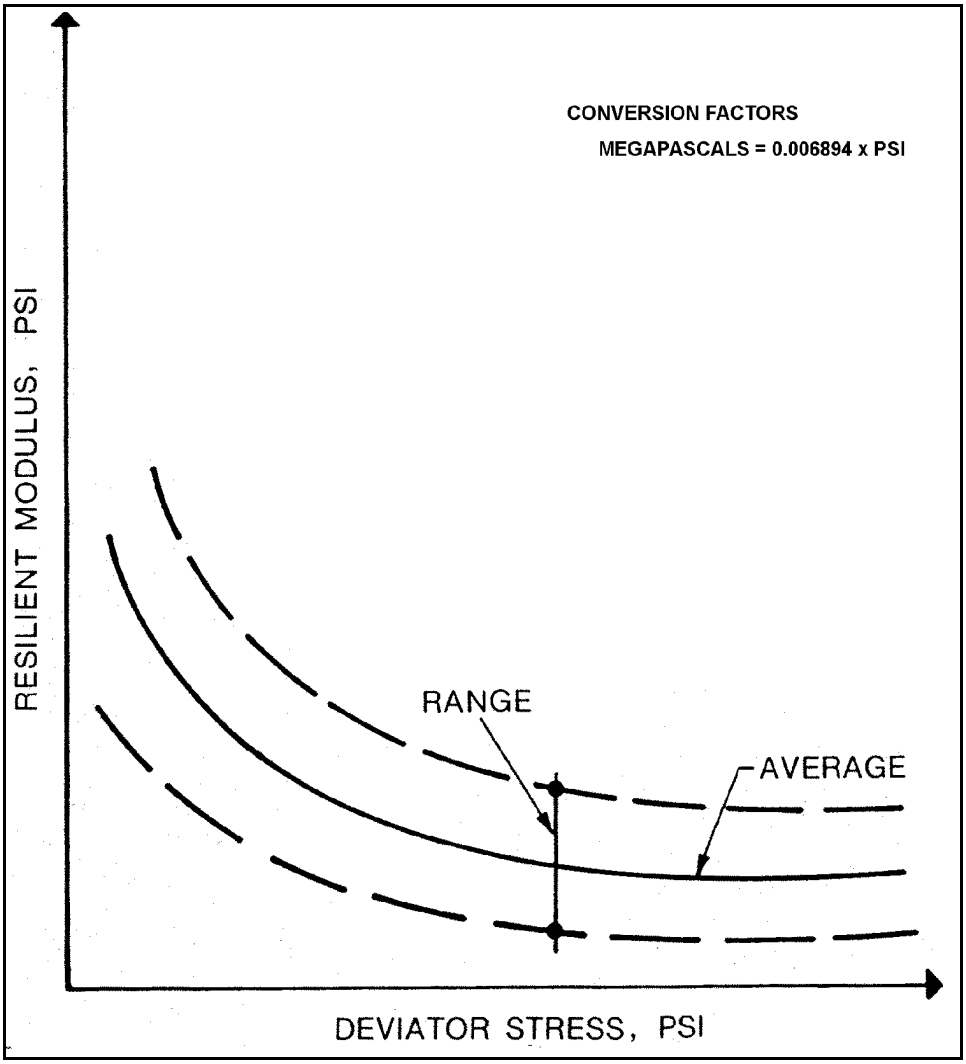


Figure L-4. Presentation of results of resilience tests on cohesive soils

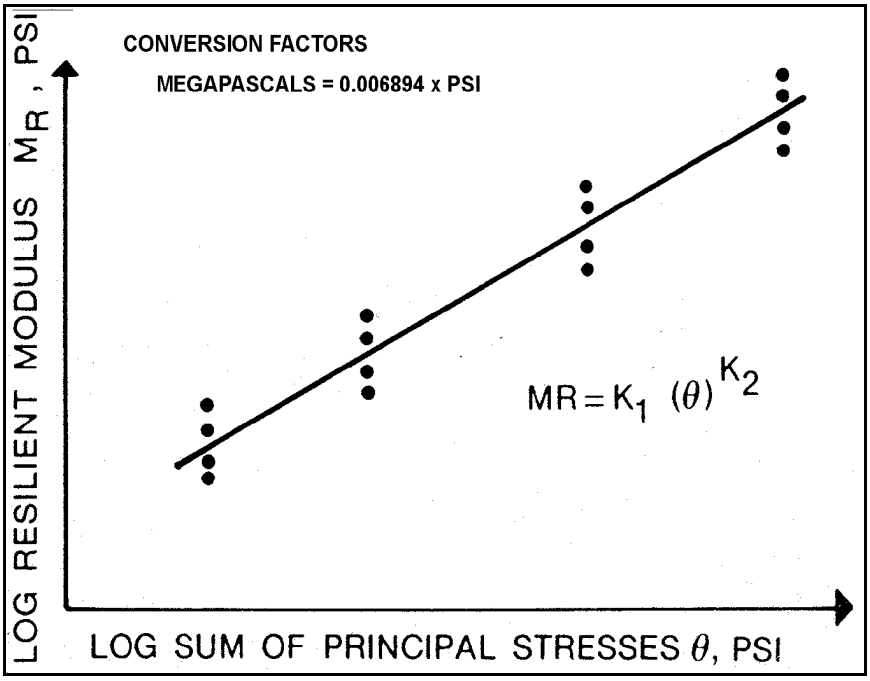


Figure L-5. Presentation of results of resilience tests on cohesionless soils

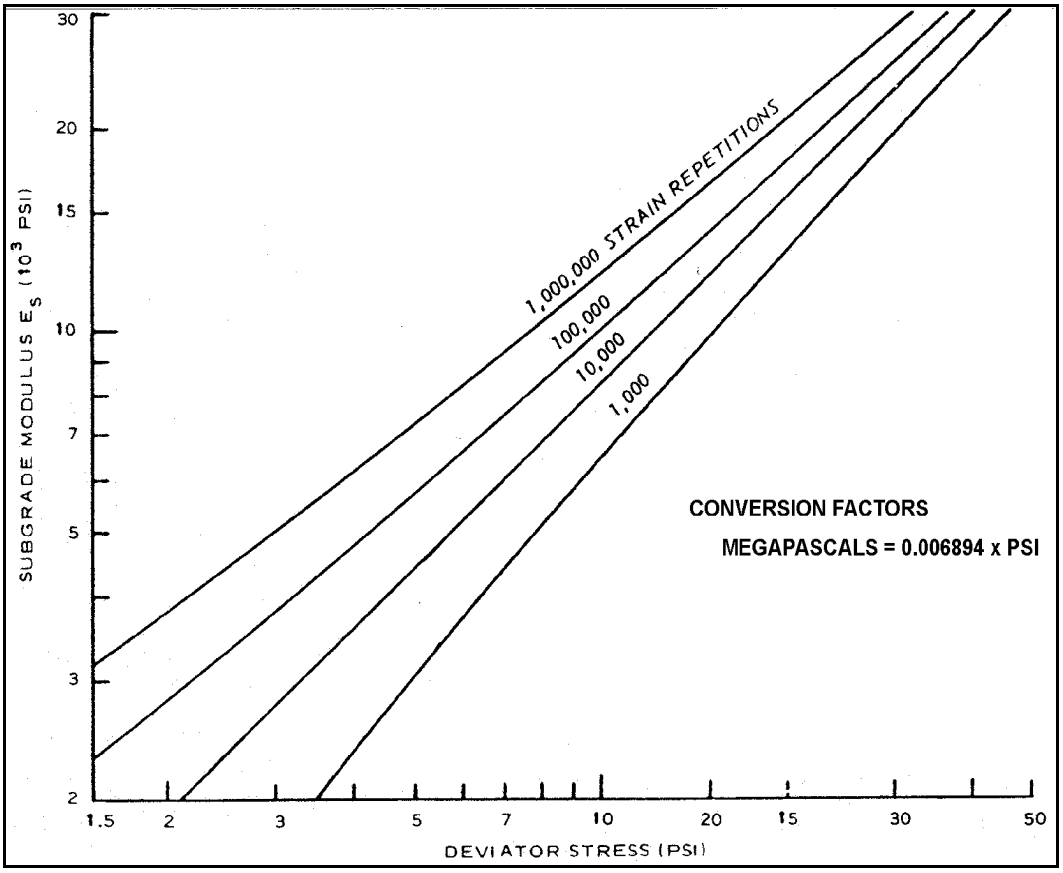


Figure L-6. Estimated deviator stress at top of subgrade

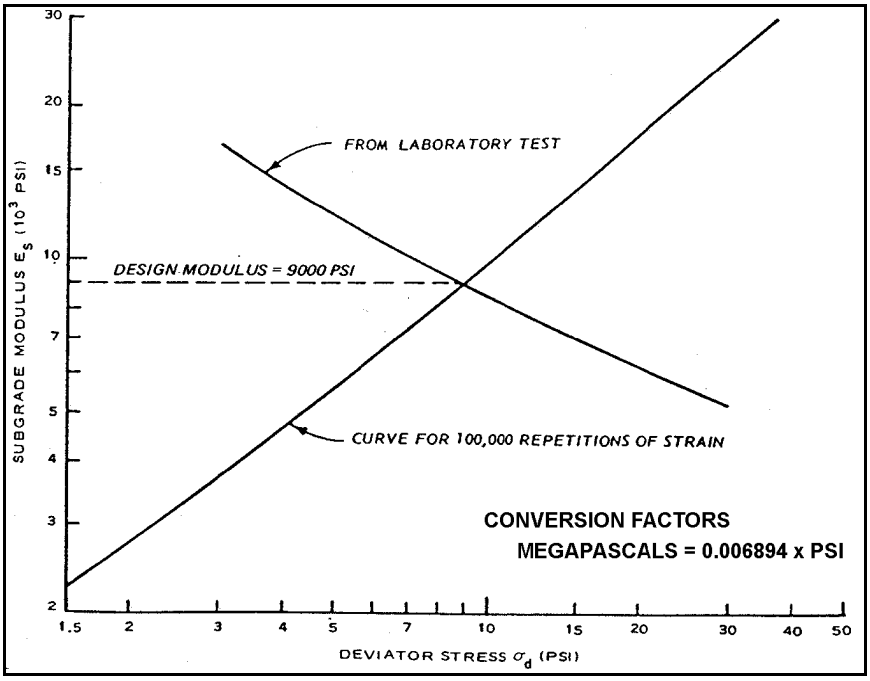


Figure L-7. Determination of subgrade modulus for cohesive soils

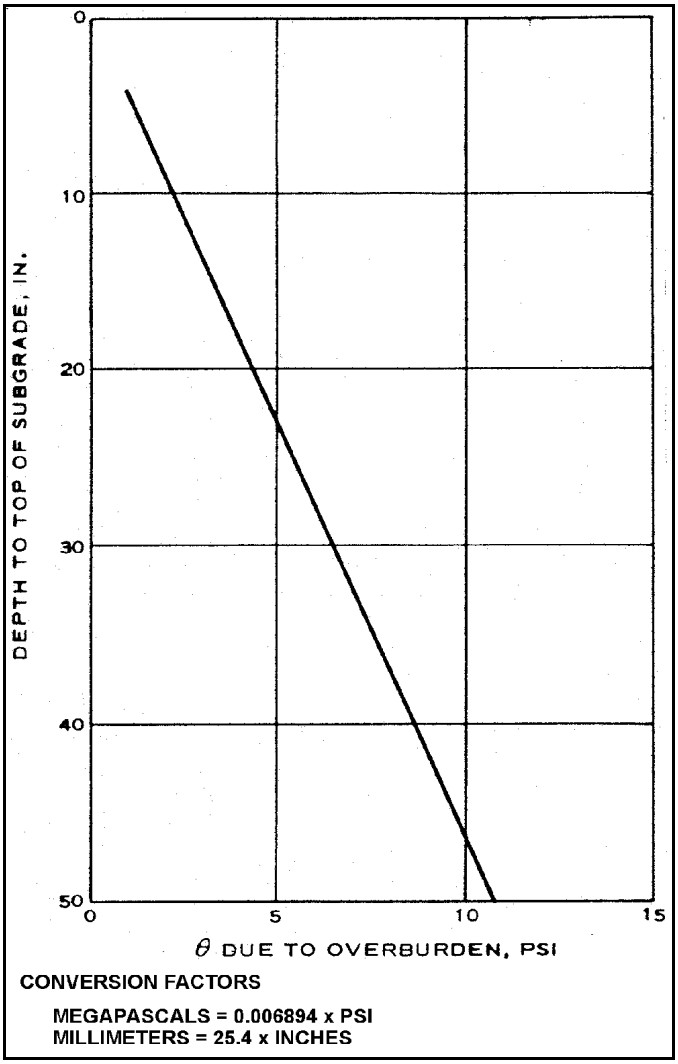


Figure L-8. Relationship for estimating θ due to overburden

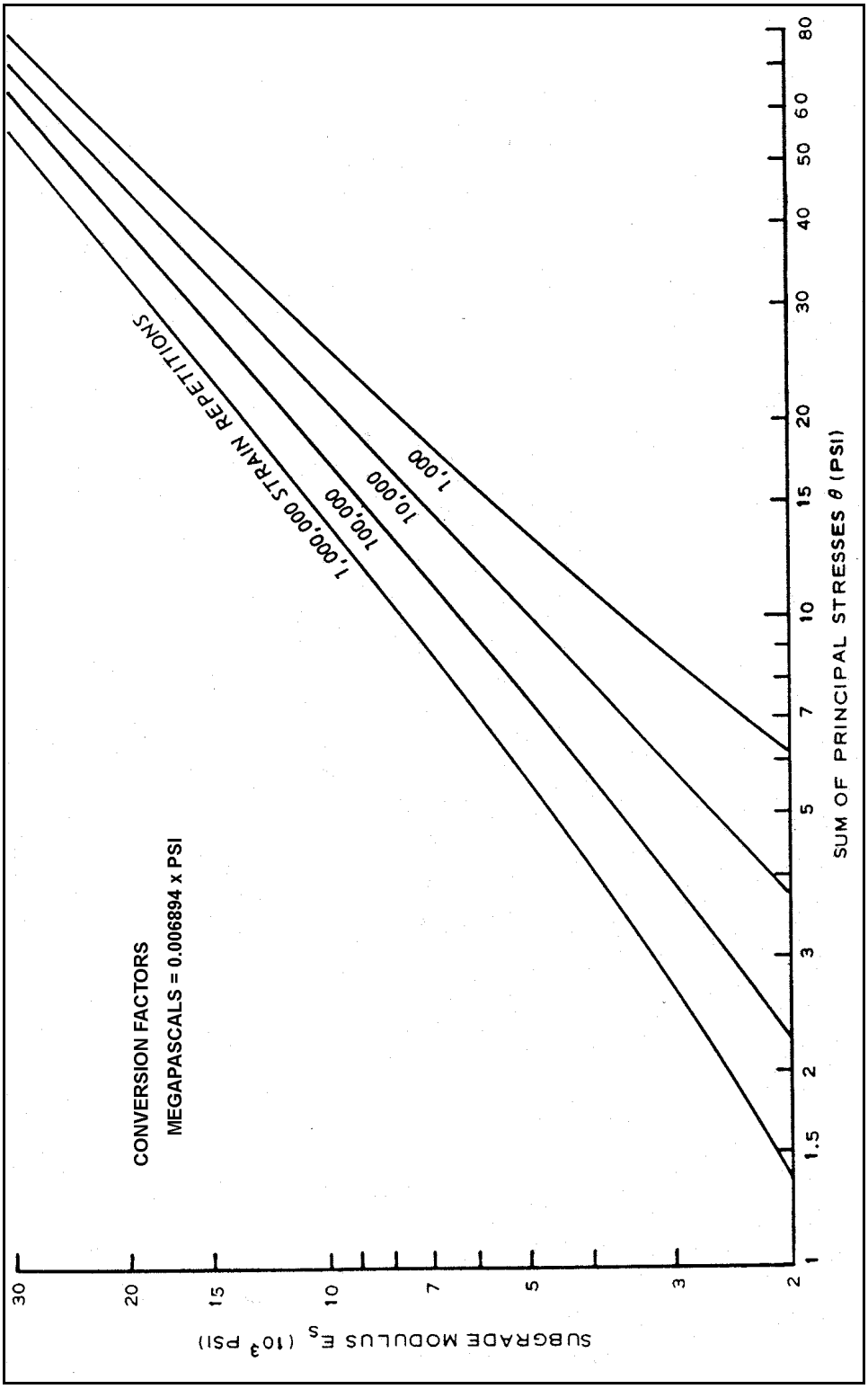


Figure L-9. Estimated θ at top of subgrade

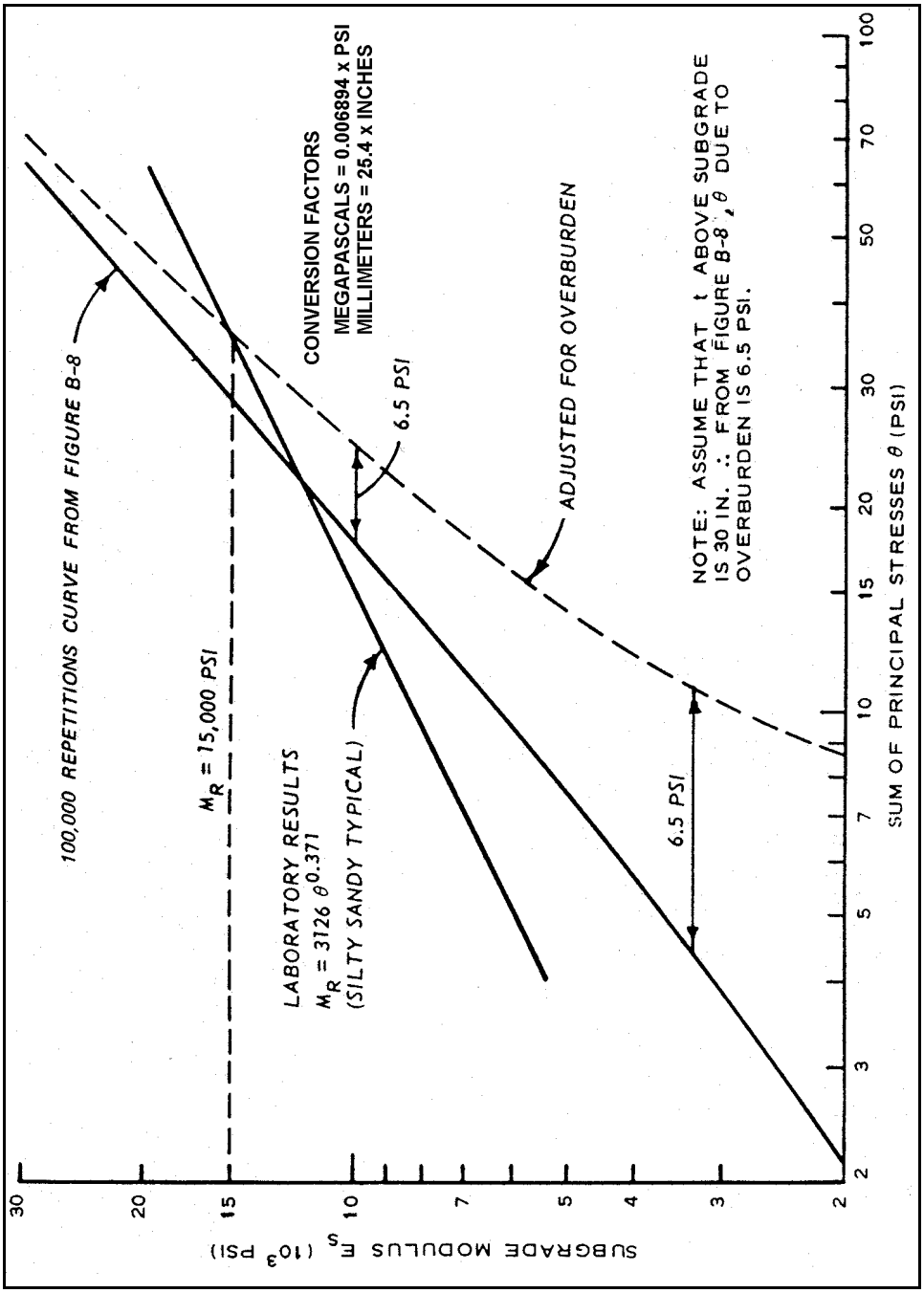


Figure L-10. Selection of M_R for silty-sand subgrade with estimated thickness of 762 millimeters (30 inches) for 100,000 repetitions of strain

APPENDIX M

PROCEDURES FOR DETERMINING THE FATIGUE LIFE OF BITUMINOUS CONCRETE

M-1. LABORATORY TEST METHOD.

a. General. A laboratory procedure for determining the fatigue life of bituminous concrete paving mixtures containing aggregate with maximum sizes up to 31.8 millimeters (1-1/2 inches) is described in this chapter. The fatigue life of a simply supported beam specimen subjected to third-point loading applied during controlled stress-mode flexural fatigue tests is determined.

b. Definitions. The following symbols are used in the description of this procedure:

(1) ϵ = initial extreme fiber strain (tensile and compressive, inches per inch)

(2) N_f = fatigue life of the specimen, number of load repetitions to fracture.

Extreme fiber strain of simply supported beam specimens subjected to third-point loadings, which produces uniaxial bending stresses, is calculated from

$$\epsilon = \frac{12td}{(3L^2 - 4a^2)} \qquad \text{(M-1)}$$

where

t = specimen depth, millimeters (inches)

d = dynamic deflection of beam center, millimeters (inches)

L = reaction span length, millimeters (inches)

a = L/3, millimeters (inches)

c. Test Equipment.

(1) The repeated flexure apparatus is shown in Figure M-1. It accommodates beam specimens 381 millimeters (15 inches) long with widths and depths not exceeding 76 millimeters (3 inches). A 1,361-kilogram- (3,000-pound-) capacity electrohydraulic testing machine capable of applying repeated tension-compression loads in the form of haversine waves for 0.1-second durations with 0.4-second rest periods is used for flexural fatigue tests. Any dynamic testing machine or pneumatic pressure system with similar loading capabilities is also suitable. Third-point loading, i.e., loads applied at distances of L/3 from the reaction points, produces an approximately constant bending moment over the center 102 millimeters (4 inches) of a 381-millimeter- (15-inch-) long beam specimen with widths and depths not exceeding 76 millimeters (3 inches). A sufficient load, approximately 10 percent of the load deflecting the beam upward, is applied in the opposite

direction, forcing the beam to return to its original horizontal position and holding it at that position during the rest period. Adjustable stop nuts installed on the flexure apparatus loading rod prevent the beam from bending below the initial horizontal position during the rest period.

(2) The dynamic deflection of the beam's center is measured with an LVDT. An LVDT suitable for this purpose is the Sheavitz type 100 M-L. The LVDT core is attached to a nut bonded with epoxy cement to the center of the specimen. Outputs of the LVDT and the electrohydraulic testing machine's load cell, through which loads are applied and controlled, can be fed to any suitable recorder. The repeated flexure apparatus is enclosed in a controlled-temperature cabinet capable of controlling temperatures within ±0.28 degrees Celsius (±½ degree Fahrenheit). A Missimer's model 100 by 500 carbon dioxide plug-in temperature conditioner has been found to provide suitable temperature control.

d. Specimen Preparation. Beam specimens 380 millimeters (15 inches) long with 59-millimeter (3-1/2-inch) depths and 83 millimeter (3-1/4-inch) widths are prepared according to ASTM D 3202. If there is undue movement of the mixture under the compactor foot during beam compaction, the temperature, foot pressure, and number of tamping blows should be reduced. Similar modifications to compaction procedures should be made if specimens with less density are desired. A diamond-blade masonry saw is used to cut 76-millimeter (3-inch) or slightly less deep by 76 millimeters (3-inch) or slightly less wide test specimens from the 380-millimeter- (15-inch-) long beams. Specimens with suitable dimensions can also be cut from pavement samples. The widths and depths of the specimens are measured to the nearest 0.25 millimeter (0.01 inch) at the center and at 51 millimeters (2 inches) from both sides of the center. Mean values are determined and used for subsequent calculations.

e. Test Procedures.

(1) Repeated flexure apparatus loading clamps are adjusted to the same level as the reaction clamps. The specimen is clamped in the fixture using a jig to position the centers of the two loading clamps 51 millimeters (2 inches) from the beam center and to position the centers of the two reactions clamps 165 millimeters (6-1/2 inches) from the beam center. Double layers of Teflon sheets are placed between the specimen and the loading clamps to reduce friction and longitudinal restraint caused by the clamps.

(2) After the beam has reached the desired test temperature, repeated loads are applied. Duration of a load repetition is 0.1 second with 0.4-second rest periods between loads. The applied load should be that which produces an extreme fiber stress level suitable for flexural fatigue tests. For fatigue tests on typical bituminous concrete paving mixtures, the following ranges of extreme fiber stress levels are suggested:

Temperatures, degrees Celsius (degrees Fahrenheit)	**Stress Level Range MPa (psi)**
13 (55)	1.03 to 3.1 (150 to 450)
21 (70)	0.52 to 2.1 (75 to 300)
30 (85)	0.24 to 1.38 (35 to 200)

The beam center point deflection and applied dynamic load are measured immediately after 200 load repetitions for calculation of extreme fiber strain ϵ. The test is continued at the constant stress level until the specimen fractures. The apparatus and procedures described have been found suitable for flexural fatigue tests at temperatures ranging from 4.4 to 38 degrees Celsius (40 to 100 degrees Fahrenheit) and for extreme fiber stress levels up to 3.1 MPa (450 psi). Extreme fiber stress levels for flexural fatigue tests at any temperature should not exceed that which causes specimen fracture before at least 1,000 load repetitions are applied.

(3) A set of 8 to 12 fatigue tests should be run for each temperature to adequately describe the relationship between extreme fiber strain and the number of load repetitions to fracture. The extreme fiber stress should be varied such that the resulting number of load repetitions to fracture ranges from 1,000 to 1,000,000.

f. Report and Presentation of Results. The report of flexural fatigue test results should include the following:

(1) Density of test specimens.

(2) Number of load repetitions to fracture, N_f.

(3) Specimen temperature.

(4) Extreme fiber stress, σ.

The flexural fatigue relationship is plotted in Figure M-2.

M-2. PROVISIONAL FATIGUE DATA FOR BITUMINOUS CONCRETE. Use of the graph shown in Figure M-3 to determine a limiting strain value for bituminous concrete involves first determining a value for the elastic modulus of the bituminous concrete. Using this value and the design pavement service life in terms of load repetitions the limiting tensile strain in the bituminous concrete can be read from the ordinate of the graph.

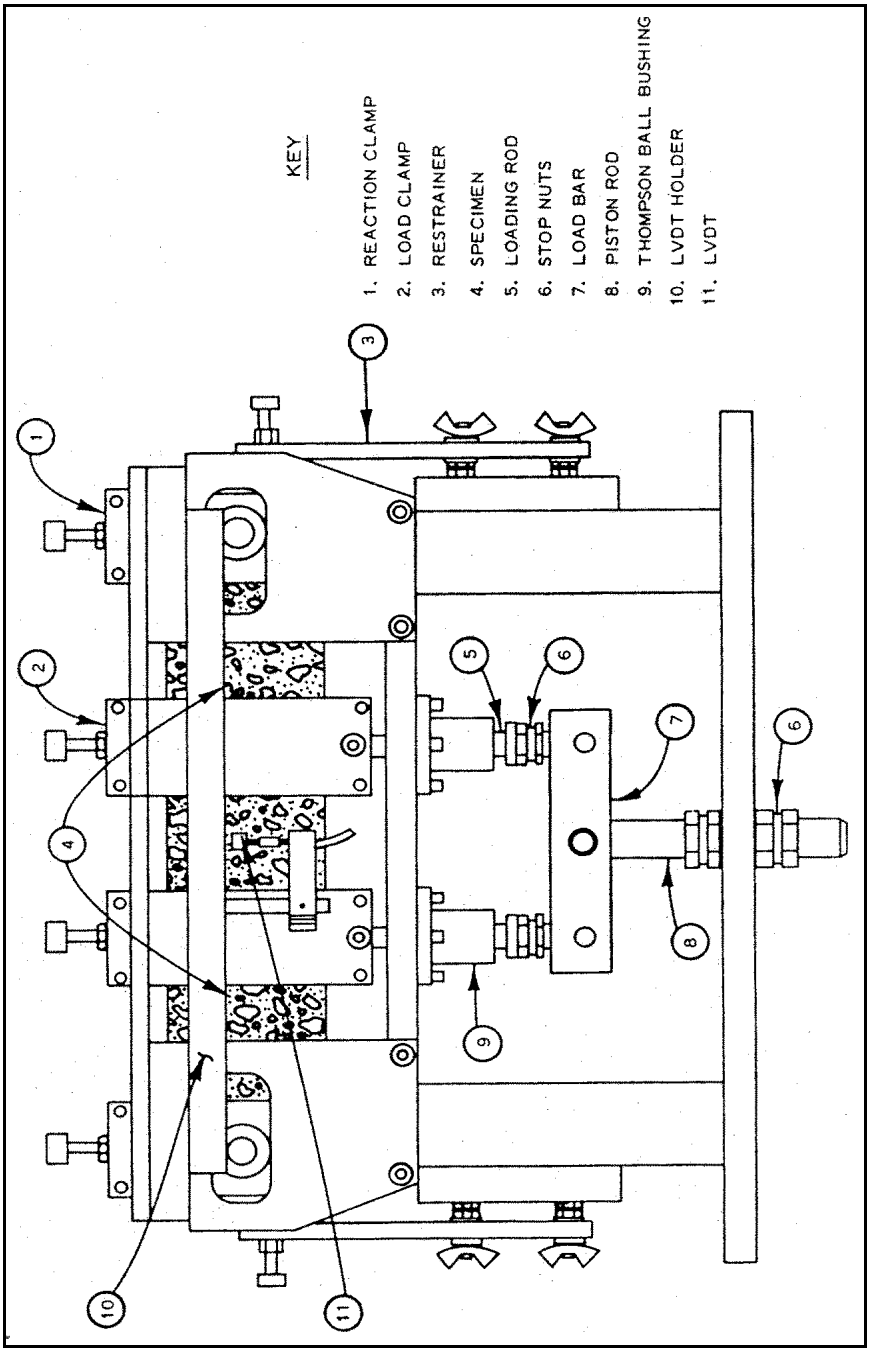


Figure M-1. Repeated flexure apparatus

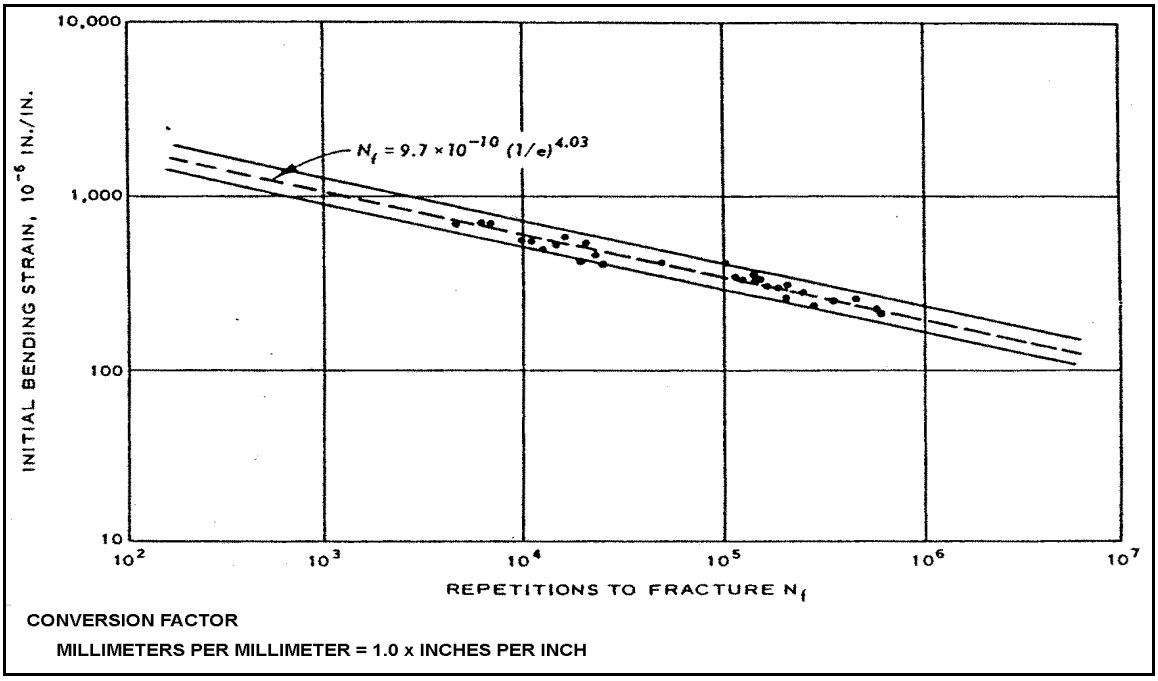


Figure M-2. Initial mixture bending strain versus repetitions to fracture in controlled stress tests

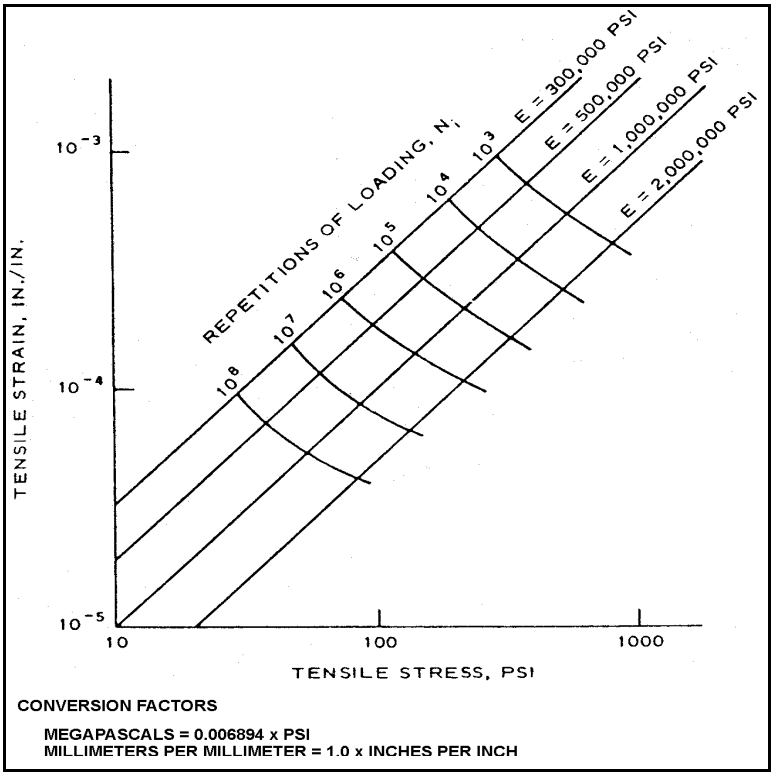


Figure M-3. Provisional fatigue data for bituminous base-course materials

APPENDIX N

PROCEDURE FOR DETERMINING THE RESILIENT MODULUS OF GRANULAR BASE MATERIAL

N-1. PROCEDURE. This procedure is designed to determine resilient properties of granular base (subbase) materials. The test is similar to a standard triaxial compression test, the primary exception being that the deviator stress is applied repetitively at several stress levels. The procedure allows testing under a repetitive stress state similar to that encountered in a base (subbase) course layer in a pavement under a moving wheel load.

N-2. DEFINITIONS. The following symbols and terms are used in the description of this procedure:

a. σ_1 = total axial stress.

b. σ_3 = total radial stress, i.e., confining pressure in the triaxial test.

c. σ_d = deviator stress (σ_1 - σ_3), i.e., the repeated axial stress in this procedure.

d. ϵ_1 = total axial strain due to σ_d.

e. ϵ_R = resilient axial strain due to σ_d.

f. ϵ_ℓ = resilient lateral strain due to σ_d.

g. M_R = the resilient modulus = σ_d/ϵ_R.

h. v_R = the resilient Poisson's ratio = ϵ_ℓ/ϵ_R.

i θ = sum of the principal stresses in the triaxial state of stress ($\sigma_1 + 2\sigma_3 = \sigma_d + 3\sigma_3$).

j. σ_1/σ_3 = principal stress ratio.

k. Load duration = time interval during which the sample is subjected to a stress deviator.

l. Cycle duration = time interval between successive applications of the deviator stress.

N-3. SPECIMENS. For base-course materials, 152-millimeter- (6-inch-) diameter specimens are generally required with the maximum particle size being limited to 25 millimeters (1 inch). The specimen height should be at least twice the diameter.

N-4. EQUIPMENT.

a. Triaxial Test Cell. The triaxial cell shown schematically in Figure N-1 is suitable for use in resilient testing of soils. The equipment is similar to most standard cells. However, there are a few specialized criteria that must be met to provide acceptable test results. Generally, the equipment is slightly larger than most standard cells to accommodate the 152-millimeter- (6-inch-) diameter specimens and the internally mounted load and deformation measuring equipment.

Additional outlets for the electrical leads from these measuring devices are required. Cell pressures of 80 psi are generally sufficient to duplicate the maximum confining pressures under aircraft loadings. Compressed air is generally used as the confining fluid to avoid detrimental effects of water on the internally mounted electronic measuring equipment.

b. End Platens. End platens should be "frictionless," as "barreling" caused by end restraint jeopardizes resilient Poisson's ratio values by causing lateral deformations to be concentrated in the middle of the specimen. Furthermore, nonuniform displacements can create problems with axial strain measurements due to realignment of the LVDT clamps. Whereas "frictionless" platens (Figure N-2) may not be entirely frictionless under short-term repetitive loadings, they constitute an improvement over conventional end platens. The essential features of "frictionless" end platens are hard polished end plates, coated by high-vacuum silicone grease, and covered by a thin rubber sheet. If externally mounted axial deformation measuring devices such as an LVDT or potentiometer mounted on the loading piston, or devices measuring the total specimen displacements are used, the use of frictionless caps and bases with grease invalidates any measurements. In this case, the deformation due to the grease and rubber sheet or Teflon probably exceeds the actual deformation of the specimen. Hence, frictionless caps and bases are restricted to use with internally mounted deformation sensors.

c. Repetitive Loading Equipment. The external loading source may be any device capable of providing a variable load of fixed cycle and load duration, ranging from simple switch control of static weights or air pistons to a close-loop electrohydraulic system. A load duration of 0.1 to 0.2 second and a cycle duration of 3 seconds have been found satisfactory for most applications. A haversine wave form is recommended; however, a rectangular wave form can be used.

d. Deformation and Load Measuring Equipment. The deformation measuring equipment consists of four LVDTs attached to the soil specimen with a pair of clamps, as shown in Figure N-1. Two LVDTs are used to measure axial deformations, and two are used to measure lateral deformations. Figures N-3 and N-4 show the details of the clamps for attaching the LVDTs to the soil specimens. Only alternating current transducers that have a minimum sensitivity of 0.2 millivolt per 0.025 millimeter (0.001 inch) per volt should be used. Load is measured with an internally mounted load cell that is sufficiently lightweight so as not to provide any significant inertia forces. It should have a capacity no greater than two to three times that of the maximum applied load and a minimum sensitivity of 2 millivolts per volt.

e. Additional Equipment. In addition to the equipment described above, the following items are also used:

(1) Calipers, a micrometer gauge, and a steel rule (calibrated to 0.25 millimeter (0.01 inch)).

(2) Rubber membranes (0.03 to 0.06 millimeter (0.012 to 0.025 inch) thick) and a membrane stretcher.

(3) Rubber O-rings.

(4) Guide rods for positioning LVDT clamps.

(5) Epoxy for cementing clamps to membrane.

(6) A vacuum source with a bubble chamber (optional) and regulator.

(7) Specimen forming jacket.

f. Recommendations. It is also necessary to have a fast recording system for accurate testing. It is recommended, for analog recording equipment, that the resolution of the parameter being controlled be better than 1.5 percent of the maximum value of the parameter being measured and that any variable amplitude signals be changed from high to low resolution as required during the test. If multichannel recorders are not available, by introducing switching and balancing units, a single-channel recorder can be used.

N-5. PREPARATION OF SPECIMENS AND PLACEMENT IN TRIAXIAL CELL. The following procedures describe a step-by-step account for preparing remolded specimens. Generally, for base-course materials, 152-millimeter- (6-inch-) diameter specimens are required with the maximum particle size being limited to 25 millimeters (1 inch) in diameter.

a. Material Preparation. The material should be air-dried and subsequently sufficient water added to bring the material to the desired compaction water content (usually field condition). Sealing the material in a container for 24 hours prior to compaction will allow the moisture to equilibrate. For well-graded materials, it may be necessary to break the material down into several sieve sizes and recombine for each layer to prevent serious segregation of material in the specimen. If the compaction effort required to duplicate the desired testing water content and density is not known, sufficient material for several specimens may have to be prepared. The compaction effort required will then be established on a trial-and-error basis.

b. Specimen Compaction. Generally, base-course materials are compacted on the triaxial cell baseplate using a split mold. If the particles are angular, two membranes may be required: one used during compaction and the second placed after compaction to seal any holes punctured in the membrane. A successful procedure has been to use a Teflon-lined mold and a thin sheet of wrapping paper instead of a membrane. Often the density is sufficiently high and the water content such that effective cohesion will permit a free-standing specimen to be prepared. In this case, the wrapping paper is carefully removed and a membrane substituted. In most cases, impact or kneading compaction is used. Vibratory compaction is only permitted on uniform materials where segregation is not a problem. The specimens should be compacted in layers, the height of which exceeds the maximum particle size.

(1) It may be necessary to place a thin layer of fine sand in the bottom layer to provide a smooth bearing surface. Likewise, after compacting and trimming the topmost layer (it may be necessary to remove large particles from this layer), fine sand can be sieved on the surface to fill in the voids and provide a smooth bearing surface for the top cap.

(2) The top cap should be centered and lightly tapped to level and ensure a good smooth contact of the cap on the specimen. A level placed on top of the cap is used to check leveling. The forming mold is then removed, the membrane placed using a membrane stretcher and sealed with O-rings or a hose clamp, and a vacuum applied. Leakage should be checked by using a bubble chamber or closing the vacuum line and observing if a vacuum is maintained in the specimen. Specimen dimensions should be measured to determine density conditions. A π-tape has been found most useful for diametrical measurements.

c. Placement of LVDT Measurement Clamps.

(1) Measure the diameter as accurately as possible at the location of the LVDT clamps for calculation of radial strains. Place the lower LVDT clamp in the specimen at approximately the lower third point of the specimen. A "jig" or gauge rods have been used successfully to assist in placing the clamps. The lower LVDT clamp generally holds the LVDT body. Repeat the procedure for the upper clamp being careful to align the clamps so that the LVDT core matches the LVDT body. It is essential that the clamps lie in a horizontal plane and their spacing be precisely known for calculating the axial strain. Again, gauge rods or a "jig" in conjunction with a small level have been used successfully for this operation. With the clamps in a position and secured by the springs, a small amount of epoxy (a "5-minute" epoxy has been used; rubber cement was found unacceptable) is placed on top of the four contact points and allowed to dry.

(2) Install the LVDTs and connect the recording unit. Generally, ±0.10-millimeter (0.040-inch) LVDTs are used for radial deformation, and ± 0.25-millimeter (0.100-inch) LVDTs are used for axial deformations. Balance the vertical spacing between LVDT clamps or check gauge rods for secure contact, and record LVDT readings and spacing. Remove gauge rods and assemble triaxial chamber. Any shifting of LVDT clamps during chamber assembly will be noted by LVDT reading changes and can be accounted for.

d. Resilient Testing. The resilient properties of granular materials are dependent primarily upon confining pressure and to a lesser extent upon cyclic deviator stress. Therefore, it is necessary to conduct the tests for a range of confining pressures and deviator stress values. Generally, chamber pressure values of 0.014, 0.027, 0.041, and 0.069 MPa (2, 4, 6, and 10 psi) are suitable. Ratios of σ_1/σ_3 of 2, 3, 4, and 5 are typically used for the cyclic deviator stress. Tests should be conducted in an undrained condition with excess pressures relieved after application of each stress state. The testing procedure is as follows:

(1) Balance the recorders and recording bridges and record calibration steps.

(2) Apply about 0.014 MPa (2 psi) axial load σ_d as a seating load simulating the weight of the pavement and ensuring contact is maintained between the loading piston and top cap during testing.

(3) Condition the specimen by applying 500 to 1,000 load repetitions with drainage lines open. This conditioning stress should be the maximum stress expected to be applied to the specimen in the field by traffic. If this is unknown, a chamber pressure of 0.034 to 0.069 MPa (5 to 10 psi) and a deviator stress (σ_1 - σ_3) twice the chamber pressure can be used.

(4) Decrease the chamber pressure to the lowest value to be used. Apply 200 load repetitions of the smallest deviator stress under undrained conditions, recording the resilient deformations and load at or near the 200th repetition. After 200 load repetitions, relieve any pore pressures, increase the deviator stress to the next highest value, and repeat procedure over the range of deviator stresses to be used.

(5) After completing the stress states for the initial confining pressure, repeat for each succeedingly higher chamber pressure.

(6) After completion of the loading, remove the axial load, apply a vacuum to the specimen, release the confining pressure, and disassemble the triaxial chamber.

(7) Check the calibration of the LVDTs and load cell.

(8) Dry the entire specimen for determination of the water content.

N-6. COMPUTATIONS AND PRESENTATION OF RESULTS.

a. Computation. The computations consist of the following:

(1) From the measured dimensions and weights, compute and record the initial dry density, degree of saturation, and water content.

(2) The resilient modulus is computed and recorded for each stress state using the following formulas:

(a) Resilient axial strain $\epsilon_R = \Delta H_r/H_i$.

(b) Resilient lateral strain $\epsilon_L = \Delta D_r/D_i$.

(c) Deviator stress $\sigma_d = \Delta P/A_O$.

(d) Resilient modulus $M_R = \sigma_d/\epsilon_R$.

(e) Resilient Poisson's ratio $v_R = \epsilon_L/\epsilon_R$.

where

ΔH_r = resilient change in gauge height (distance between LVDT clamps) after specified number of load repetitions.

H_i = instantaneous gauge height after specified number of load repetitions. Can be calculated from $H_O - \Delta H$. If ΔH is small, H_o can be used.

H_O = initial gauge height or distance between LVDTs less adjustment occurring during triaxial chamber assembly.

ΔH = permanent change in gauge height.

ΔP = change in axial load, maximum axial load minus surcharge load.

A_O = original cross-sectional area of specimen.

ΔD_r = resilient change in diameter after specified number of load repetitions.

D_i = instantaneous diameter after specified number of load repetitions. Can be calculated from $D_O + \Delta D$.

D_o = initial specimen diameter.

ΔD = permanent change in specimen diameter.

b. Presentation of Results. Test results should be presented in the form of plots of log M_R versus log of the sum of the principal stresses and v_r versus the principal stress ratio (Figure N-5). The equation of the line for resilient modulus is $M_R = K_1\theta^{K_2}$ where K_1 is the intercept when θ = 1 psi and K_2 is the slope of the line.

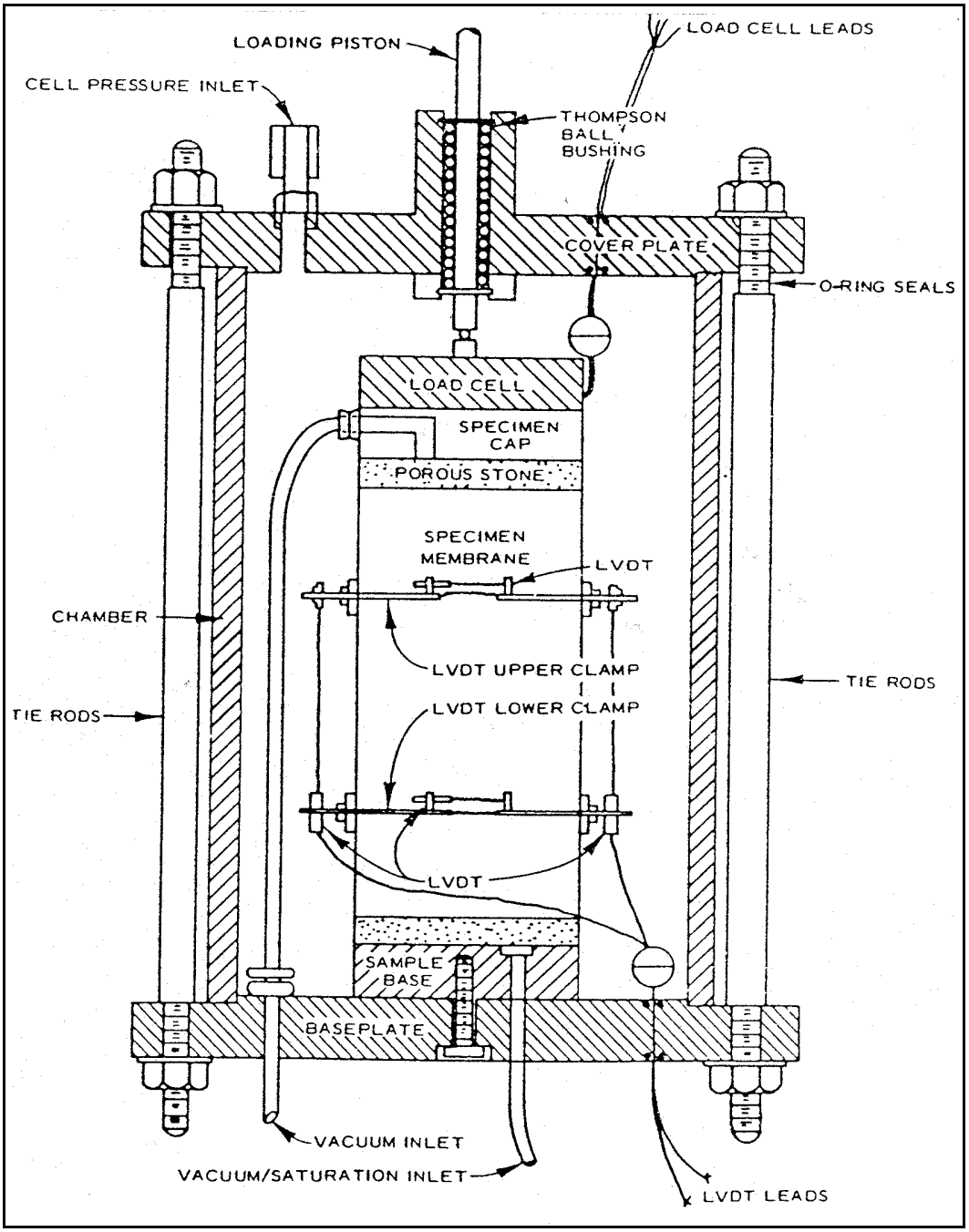


Figure N-1. Triaxial cell used in resilience testing of granular base material

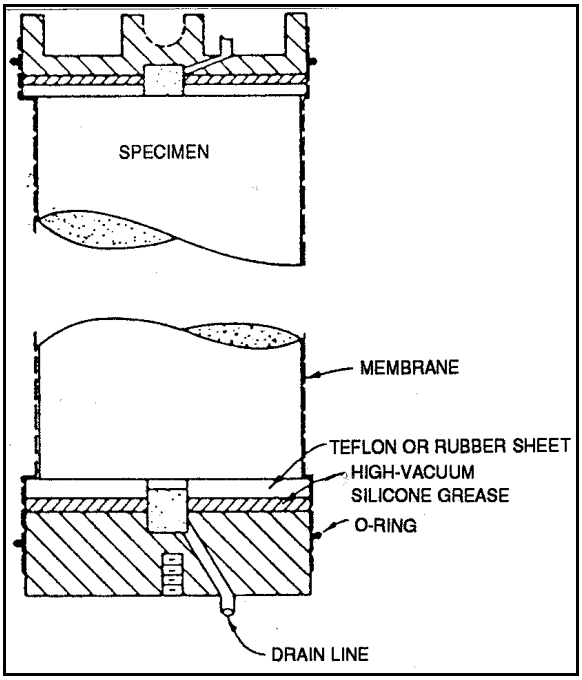


Figure N-2. Schematic of frictionless cap and base

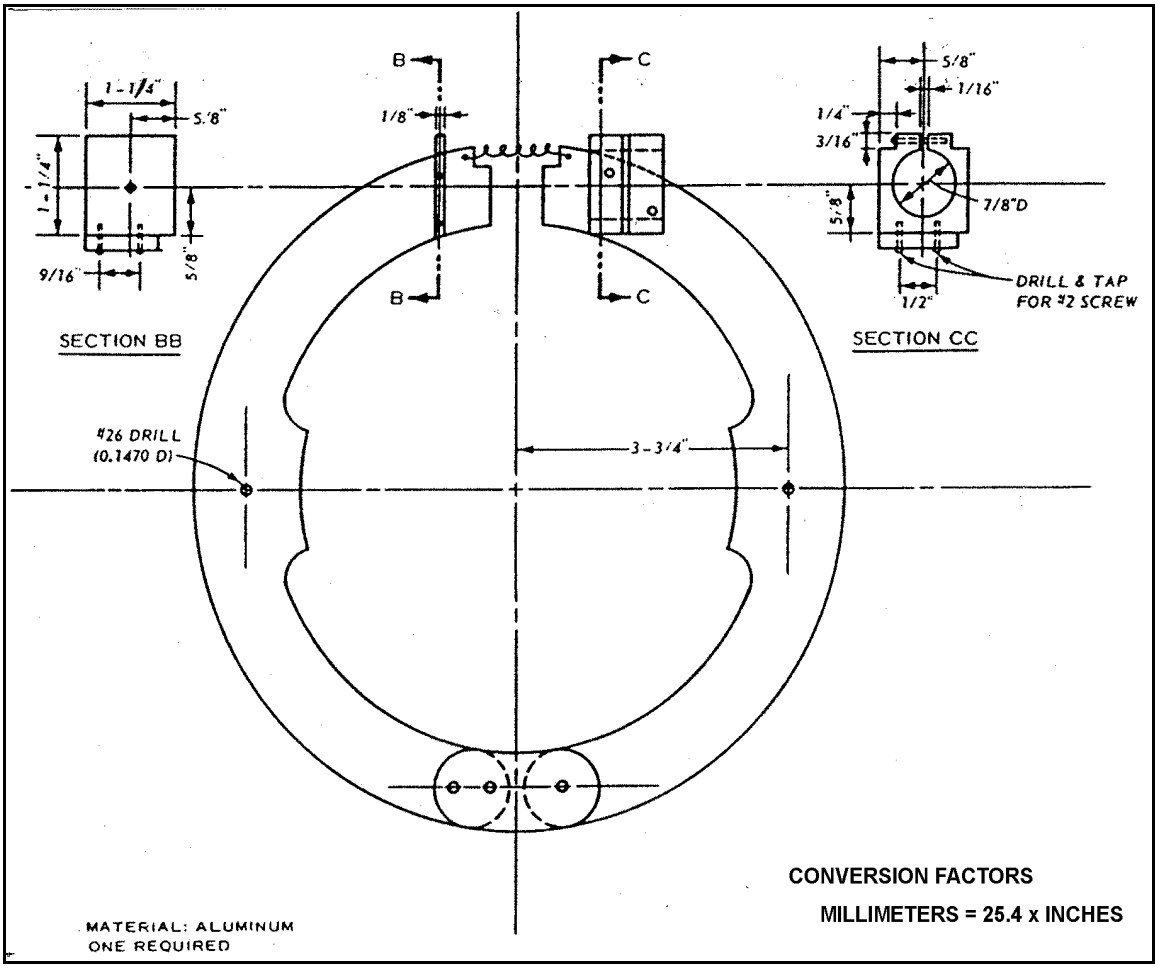
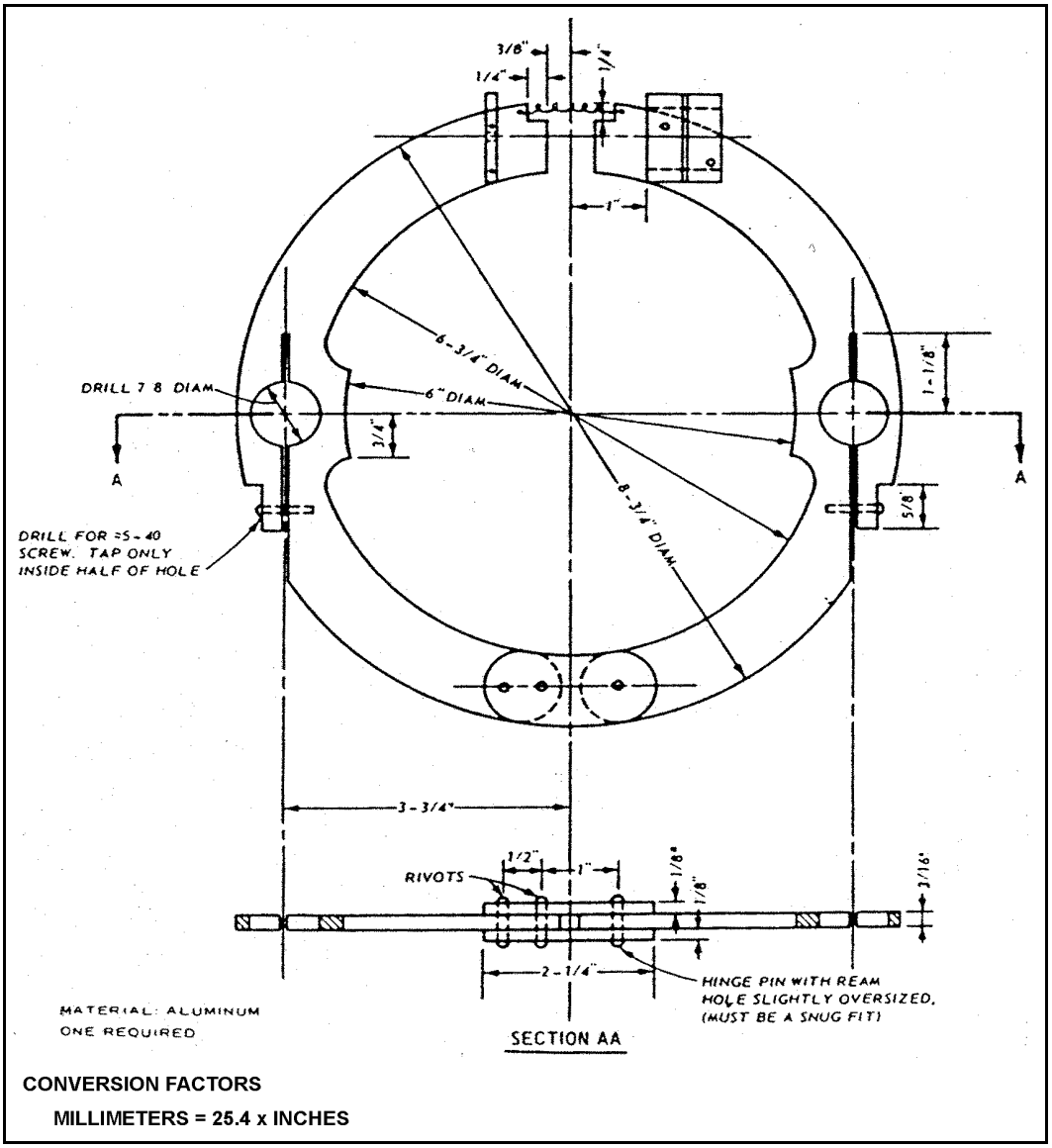


Figure N-3. Details of Top LVDT ring clamp



CONVERSION FACTORS

MILLIMETERS = 25.4 x INCHES

Figure N-4. Details of bottom LVDT ring clamp

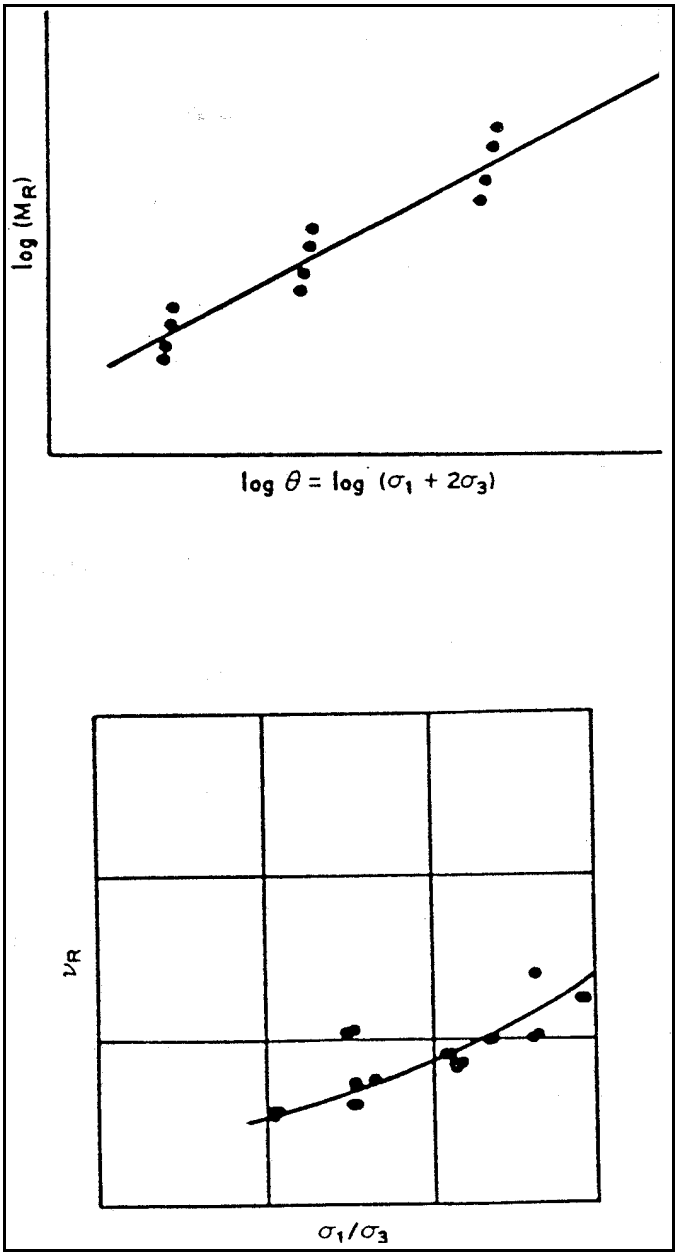


Figure N-5. Representation of results of resilience test on cohesionless soils