UNIFIED FACILITIES CRITERIA (UFC)

DRAFT
PAVEMENT DESIGN FOR AIRFIELDS

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UNIFIED FACILITIES CRITERIA (UFC)

DRAFT 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 UFC 3-260-02, dated 30 June 2001.

The format of this document does not conform to UFC 1-300-01.

FOREWORD

The Unified Facilities Criteria (UFC) system is prescribed by MIL-STD 3007 and provides planning, design, construction, sustainment, restoration, and modernization criteria, and applies to the Military Departments, the Defense Agencies, and the DOD Field Activities in accordance with USD(AT&L) Memorandum dated 29 May 2002. UFC will be used for all DOD projects and work for other customers where appropriate. All construction outside of the United States is also governed by Status of Forces Agreements (SOFA), Host Nation Funded Construction Agreements (HNFA), and in some instances, Bilateral Infrastructure Agreements (BIA.) Therefore, the acquisition team must ensure compliance with the more stringent of the UFC, the SOFA, the HNFA, and the BIA, as applicable.

UFC are living documents and will be periodically reviewed, updated, and made available to users as part of the Services' responsibility for providing technical criteria for military construction. Headquarters, U.S. Army Corps of Engineers (HQUSACE), Naval Facilities Engineering Command (NAVFAC), and Air Force Center for Engineering and the Environment (AFCEE) are responsible for administration of the UFC system. Defense agencies should contact the preparing service for document interpretation and improvements. Technical content of UFC is the responsibility of the cognizant DOD working group. Recommended changes with supporting rationale should be sent to the respective service proponent office by the following electronic form: Criteria Change Request (CCR). The form is also accessible from the Internet sites listed below.

UFC are effective upon issuance and are distributed only in electronic media from the following source:

- Whole Building Design Guide web site http://dod.wbdg.org/.

Hard copies of UFC printed from electronic media should be checked against the current electronic version prior to use to ensure that they are current.

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UNIFIED FACILITIES CRITERIA (UFC)
REVISION SUMMARY SHEET

Document: UFC 3-260-02
Superseding: UFC 3-260-02, dated 30 June 2001

Description of Changes: This update to UFC 3-260-02 incorporates new pavement design requirements for the Navy and Marine Corps and updates the design traffic for the Army and Air Force. Design figures were revised accordingly. Chapters on aggregate-surfaced runways and heliports and pavement subdrainage were added. Finally, the pavement design examples were revised to follow PCASE procedures and were consolidated into Appendix B, Section 14, of the manual.

Reasons for Changes:

- The Navy and Marine Corps revised their design procedure.
- The added chapters were the result of consolidation of criteria into one manual.
- The revised design figures were the result of aircraft traffic changes and minor changes in the computer programs.
- The consolidation of the design examples makes for a better flow of information.

Impact: There are negligible cost impacts; however, these benefits should be realized:

- The consolidation of all aircraft pavement design information into one manual will reduce the number of publications.
- The Navy and Marine Corps change in design methodology brings all three service components in agreement on pavement design methodology, which allows the use of one design program (PCASE).
- The revised design figures will result in pavements being designed to the proper level, thus giving the appropriate design life.

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CHAPTER 1

INTRODUCTION

1-1 PURPOSE

This document establishes general concepts and criteria for the design of airfield pavements for the United States (U.S.) Army, Air Force, Navy, and Marine Corps.

1-2 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 layered elastic analysis for flexible pavements, and for the Westergaard analysis and layered elastic analysis for rigid pavements. The layered elastic analysis for rigid pavements covers only plain concrete, reinforced concrete, and concrete overlay pavements.

1-3 REFERENCES

Appendix A contains a list of references used in this Unified Facilities Criteria (UFC).

1-4 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 parentheses following a comparable SI value or the IP values may be shown without a corresponding SI value.

1-5 PAVEMENT

As used in this document, a pavement 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.

1-5.1 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. Figure 1-2 and Figure 1-3 provide examples of an all-bituminous concrete (ABC) pavement and a flexible pavement using stabilized layers, respectively. Not all layers shown in the figures are required in every pavement.

Figure 1-1. Typical Flexible Pavement Structure

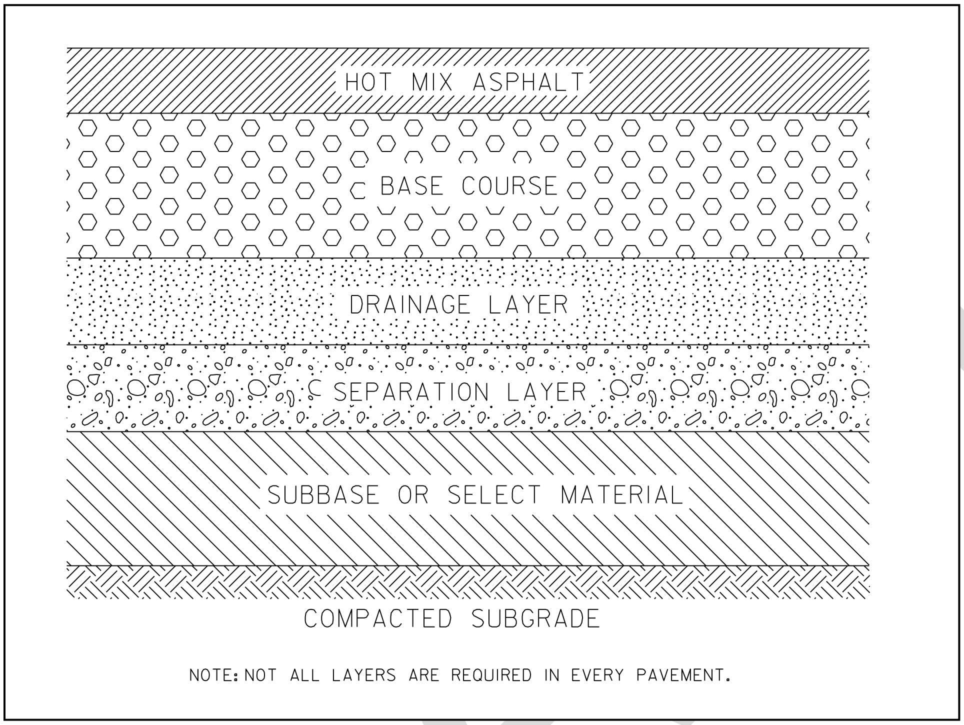


Figure 1-2. Typical All-Bituminous Concrete Pavement, Army and Air Force

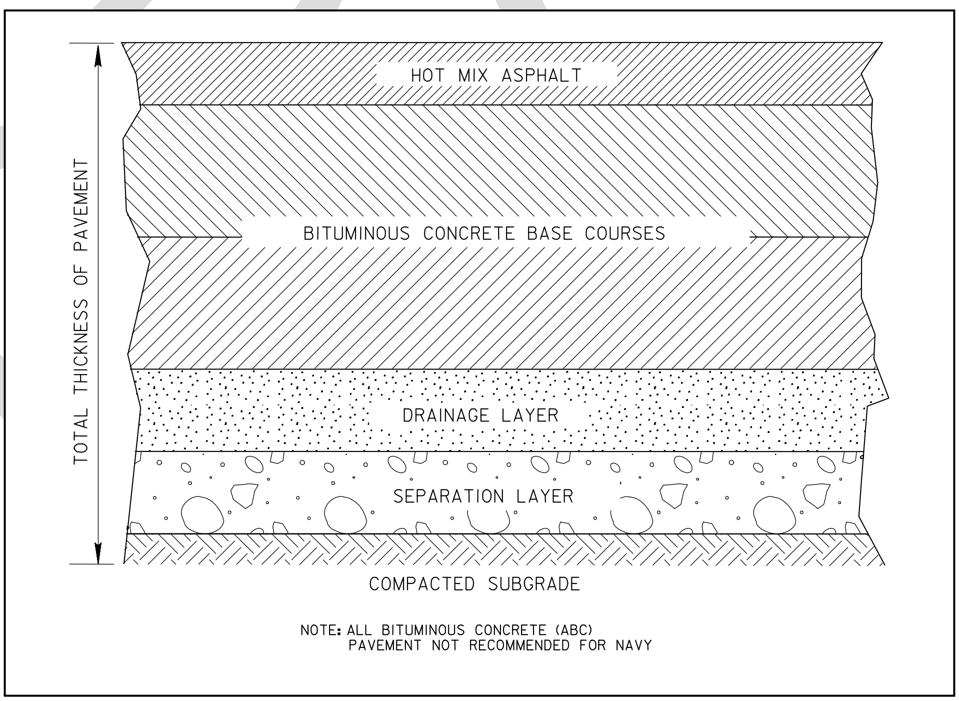
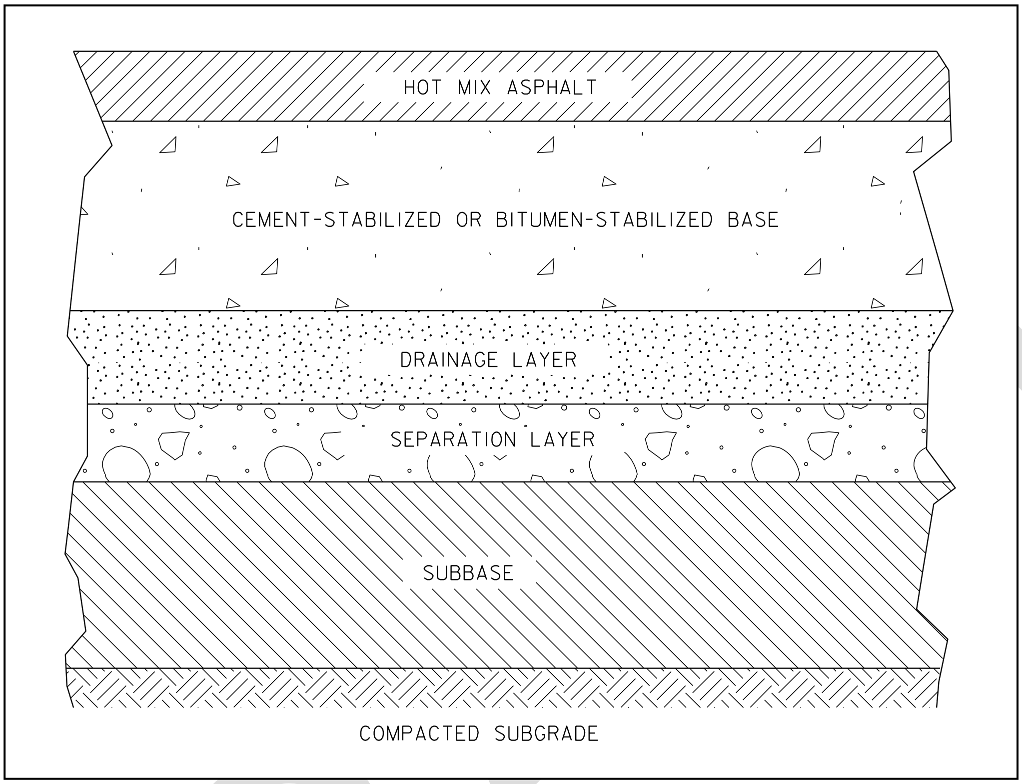


Figure 1-3. Typical Flexible Pavement with Stabilized Base



1-5.2 Rigid Pavement

A rigid pavement is any pavement system that contains portland cement concrete (PCC) 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. These pavements are considered rigid pavements:

- Plain concrete pavement is a nonreinforced jointed rigid pavement.
- Reinforced concrete pavement is a jointed rigid pavement that has been strengthened with deformed bars or welded wire fabric.
- 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.

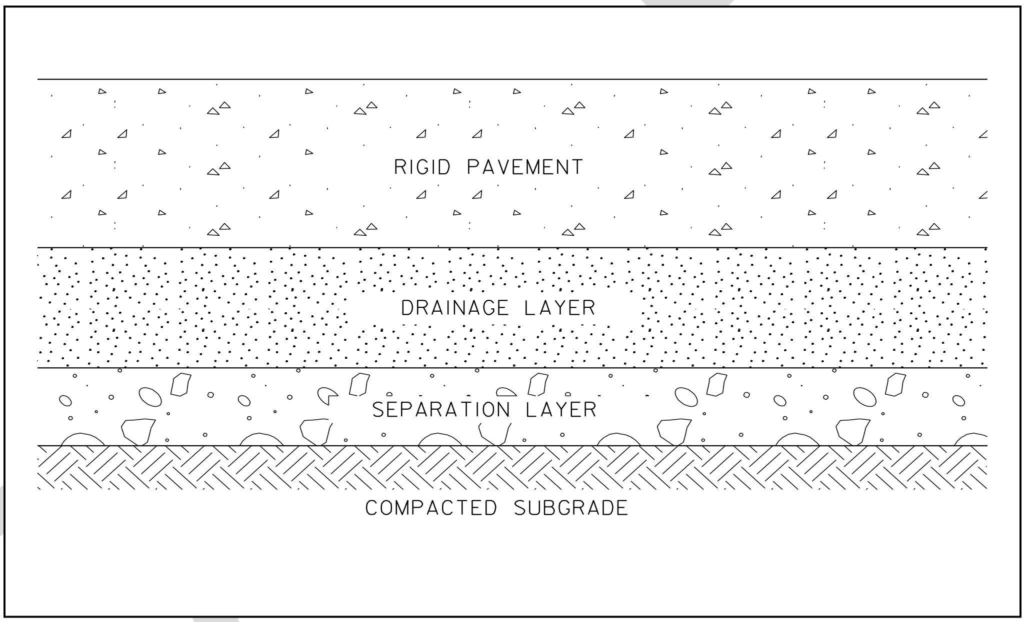
- Fibrous concrete pavement is a rigid pavement that has been strengthened by the introduction of randomly mixed, short, small-diameter steel fibers. Nonsteel fibers have been used in PCC to control shrinkage cracking, but their use is not covered in this document.

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

- Rigid overlay pavement is a rigid pavement used to strengthen an existing flexible or rigid pavement.

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

Figure 1-4. Typical Rigid Pavement Structure



1-6 USE OF FLEXIBLE PAVEMENTS

The use of flexible pavements on airfields must be limited to those pavement areas not subjected to the detrimental effects of fuel spillage, severe jet blast, or parked aircraft. 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. 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. Usually hot-mix asphaltic concretes (AC) will resist erosion at temperatures up to 150 degrees Celsius (C) (300 degrees Fahrenheit [F]). Temperatures of this magnitude are produced only when aircraft are standing and are operated for an extended time or with afterburners operating. Flexible pavements are generally satisfactory for runway interiors, secondary taxiways, shoulders, paved portions of overruns, and other areas not specifically required to have a rigid pavement surfacing.

1-7 USE OF RIGID PAVEMENTS

These pavements will be rigid pavement:

- All paved areas on which aircraft or helicopters are regularly parked, maintained, serviced, or preflight checked
- Hangar floors and hangar access aprons
- Helipads
- Runway ends (305 meters (m) (1,000 feet [ft]) of a Class B runways as defined in UFC 3-260-01
- Areas that may be used from the runway end to 60 m (200 ft) past the barrier to control hook skip
- Primary taxiways for Class B runways
- Hazardous cargo, power check, compass calibration, warm-up, alert, arm/disarm, holding, and wash rack pads
- 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
- Pavement intersections where aircraft or vehicles have a history of distorting flexible pavements and where sustained operations of aircraft or vehicles with tire pressures in excess of 2.06 megapascals (MPa) (300 pounds per square inch [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 (for example, joint sealers, asphalt pavement) in those areas. In general, the type of pavement to be used on all other paved areas will be selected on the basis of life cycle costs.

The 2 m (6.56 ft) of pavement on both the approach and departure sides of the arresting gear pendent shall be PCC for the Navy and Marine Corps. Navy aircraft arresting gear pavement protection designs drawings will be provided by the Naval Facilities Engineering Command (NAVFAC).

1-8 SOIL STABILIZATION

Soils used in pavements may be stabilized or modified through the addition of chemicals or bitumens. A stabilized soil is one that has improved load-carrying and durability characteristics because of the addition of admixtures. There are several principal benefits of stabilization:

- Reduces pavement thickness
- Provides a construction platform
- Decreases swell potential
- Reduces susceptibility to pumping as well as 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 that has improved construction characteristics because of 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 are contained in UFC 3-250-11.

1-9 DESIGN ANALYSIS

The outlines in Appendix B, Section 1, will be used to prepare design analyses for all projects. Appendix B, Section 2, provides a recommended contract drawing outline for airfield paving projects. Include all pertinent items and computational details to show how design results were obtained.

1-10 WAIVERS TO CRITERIA

Each Department of Defense (DOD) service component is responsible for setting administrative procedures necessary to process and grant formal waivers. Waivers to the criteria contained in this UFC will be processed in accordance with Appendix B, Section 3.

1-11 COMPUTER PROGRAMS

The Pavement-Transportation Computer Assisted Structural Engineering (PCASE) computer program has been developed for the design of pavements. PCASE and other computer programs may be obtained electronically from this Internet site:

World Wide Web address: http://www.pcase.com

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

1-12 ADDITIONAL GUIDANCE

The standard practice for flexible pavements is defined in UFC 3-250-03. The standard practice for rigid pavements is defined in UFC 3-250-04. For the Air Force and Army, the requirements for preparing airfield design RFP documents are contained in UFC 3-260-11FA.

1-13 MAJOR COMMAND (MAJCOM) PAVEMENT ENGINEER PREFERENCES

For all Air Force projects, the pavement designer must obtain the MAJCOM pavements engineer’s preferences for items such as joint sealant type, joint types, paving materials, and permissible use of recycled material.

CHAPTER 2

ARMY AIRFIELD AND HELIPORT REQUIREMENTS

2-1 ARMY AIRFIELD AND HELIPORT CLASSES

Army airfields are divided into six classes referred to as Class I (heliports-helipads with aircraft 11,340 kilograms (kg) (25,000 pounds [lb]) or less), Class II (heliports-helipads with aircraft over 11,340 kg [25,000 lb]), Class III (airfields with Class A runways per UFC 3-260-01), Class IV (airfields with Class B runways per UFC 3-260-01), 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.

2-2 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 Air Force Engineering Technical Letter (ETL) 09-6. Class VI airfields used for Army contingency training missions shall be designed in accordance with ETLs 09-6 and 97-9. Any deviations from these criteria must be submitted through the installation major command (MACOM) to the U.S. Army Aeronautical Services Agency (USAASA) for waiver approval.

2-3 TRAFFIC AREAS FOR ARMY AIRFIELD AND HELIPORT 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.

2-3.1 Class I and II Heliports

These heliport classes have only one traffic area: Type B.

2-3.2 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 m (500 ft) 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-m (500-ft) end sections, secondary (ladder) taxiways, hangar floors, wash racks, 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.

2-3.3 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 m (1,000 ft) 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-m (1,000-ft) end sections, secondary (ladder) taxiways (between runway and primary taxiway), hangar floors, hangar access aprons, and wash racks. A typical layout of Army airfield traffic areas for Class IV airfields is shown in Figure 2-1.

2-3.4 Class V Heliports

These heliports have only one traffic area: Type B.

2-3.5 Class VI Airfields

These airfields have only one traffic area: Type A.

2-3.6 Exceptions

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

2-4 ARMY AIRCRAFT DESIGN LOADS AND PASS LEVELS

Army airfield pavements will be designed according to the 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 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 but shall not be less than the traffic described in the subparagraphs below or contained in Table 2-1. Aircraft hangar floors or apron pavements shall not be designed for jacking loads as long as the footprint 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 these Army airfield and heliport classes:

2-4.1 Class I

Class I heliports and helipads accommodate aircraft maximum operational weights equal to or less than 11,340 kg (25,000 lb). Base the design of heliports and helipads on the number of equivalent passes of the UH-60 aircraft at a 7,395-kg (16,300-lb) operational weight. The projected equivalent passes generated for the airfield mission traffic shall be at least 50,000 passes for a heliport or at least 20,000 passes for a helipad.

2-4.2 Class II

Class II consists of heliports and helipads that support aircraft with maximum operational weights over 11,340 kg (25,000 lb). Base the design on the number of equivalent passes of the CH-47 aircraft at a 22,680-kg (50,000-lb) operational weight. The projected equivalent passes will be generated for the airfield mission traffic but shall not be less than:

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

- 20,000 passes for VFR helipads.
- 100,000 passes for instrument flight rules (IFR) heliports.
- 30,000 passes for IFR helipads.

2-4.3 **Class III**
Class III consists of airfields that primarily support fixed-wing aircraft requiring a Class A runway as defined in UFC 3-260-01, Chapter 3. Base the design on the projected number of aircraft operations of at least 50,000 passes of a C-23 aircraft at an 11,200-kg (24,600-lb) operational weight plus at least 100,000 passes of a CH-47 aircraft at an operational weight of 22,680 kg (50,000 lb).

2-4.4 **Class IV**
Class IV consists of airfields supporting aircraft requiring a Class B runway as defined in UFC 3-260-01.

2-4.4.1 The design for an airfield with its longest runway extending less than or equal to 1,525 m (5,000 ft) will be based on the number of projected equivalent passes of the C-130 aircraft at a 70,310-kg (155,000-lb) operational weight or the C-17 aircraft at 263,100-kg (580,000-lb) operational weight. The projected equivalent passes will be generated for the airfield mission traffic but shall not be less than 20,000 passes for the C-130 or 20,000 passes for the C-17.

2-4.4.2 The design for an airfield with its longest runway extending over 1,525 m (5,000 ft) but less than or equal to 2,745 m (9,000 ft) will be based on the number of projected equivalent passes of the C-17 aircraft at a 263,100-kg (580,000-lb) operational weight. The projected equivalent passes will be generated for the airfield mission traffic but shall not be less than 30,000 passes.

2-4.4.3 The design for an airfield with its longest runway extending over 2,745 m (9,000 ft) will be based on the number of projected equivalent passes of the C-17 aircraft at a 263,100-kg (580,000-lb) operational weight. The projected equivalent passes will be generated for the airfield mission traffic but shall not be less than 50,000 passes. At installations with a mobilization mission, increase the C-17 pass level to 100,000 passes.

2-4.5 **Class V**
Class V consists of 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-kg (50,000-lb) 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 structurally designed in accordance with the criteria in this document but provided with a bituminous surface or a military landing mat.

2-4.6 Class VI

Class VI consists of 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-kg (155,000-lb) operational weight or the C-17 aircraft at a 263,100-kg (580,000-lb) 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 UFC 3-250-11, Chapter 22 of this manual, and ETL 09-6.

2-5 ROLLER-COMPACTED CONCRETE PAVEMENT (RCCP)

RCCP shall not be used for Army airfield or heliport pavements.

2-6 RESIN MODIFIED PAVEMENT (RMP)

RMP shall not be used for Army airfield or heliport pavements.

2-7 PAVED SHOULDERS

2-7.1 Location

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

2-7.2 Structural Requirements

As a minimum, paved shoulders shall be designed to support 5,000 coverages of a load of 4,535 kg (10,000 lb) 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 (for example, snow removal equipment, fire trucks, fuel trucks), the shoulder should be designed accordingly for whichever governs.

2-8 SURFACE DRAINAGE

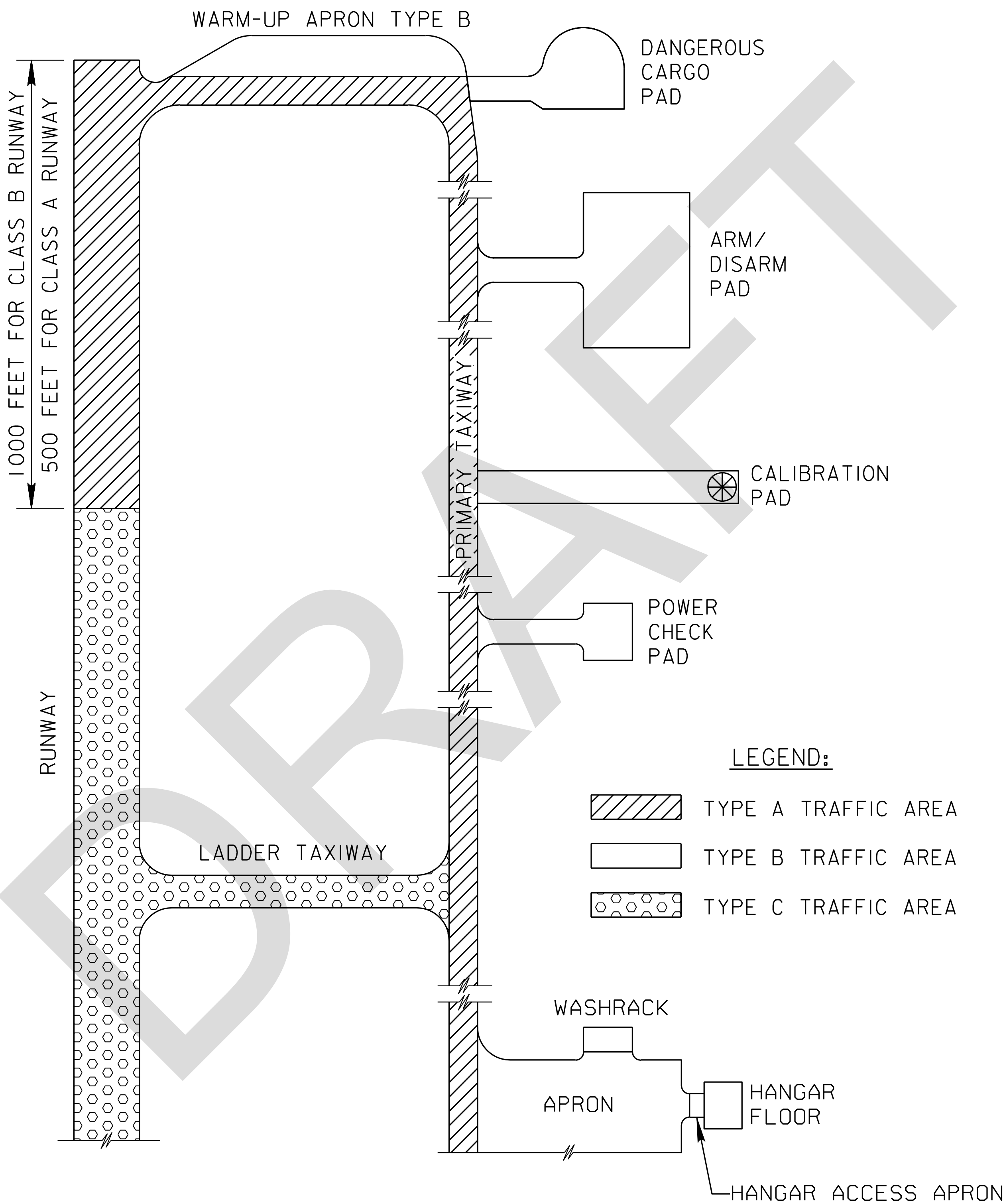
Design of surface drainage shall be in accordance with Federal Aviation Administration (FAA) Advisory Circular (AC) 150/5320-5C.

Table 2-1. Design Gross Weights and Pass Levels for Army Airfield and Heliport Pavement Design

Airfield Class		Design Aircraft	A Traffic Area		B Traffic Area		C Traffic Area		Overruns		Shoulders
			Weight	Passes*	Weight	Passes*	Weight	Passes*	Weight	Passes	
Class I		UH60	N/A	N/A	7,395 kg (16,300 lb)	20,000 for helipads 50,000 for heliports	N/A	N/A	Same as Shoulder Pavements	Same as Shoulder Pavements	5,000 coverages of a 4,536 kg (10,000 lb) single-wheel load having a tire pressure of 690 kPa (100 psi)
Class II	VFR	CH47	N/A	N/A	22,680 kg (50,000 lb)	20,000 for helipads 50,000 for heliports	N/A	N/A	Same as Shoulder Pavements	Same as Shoulder Pavements	
	IFR	CH47	N/A	N/A	22,680 kg (50,000 lb)	30,000 for helipads 100,000 for heliports	N/A	N/A	Same as Shoulder Pavements	Same as Shoulder Pavements	
Class III		C-23	11,200 kg (24,600 lb)	50,000	11,200 kg (24,600 lb)	50,000	8,460 kg (18,450 lb)	50,000	Same as Shoulder Pavements	Same as Shoulder Pavements	
		CH47	22,680 kg (50,000 lb)	100,000	22,680 kg (50,000 lb)	100,000	17,010 kg (37,500 lb)	100,000			
Class IV	Runway Length <1525 m (5000 ft)	C-130	70,310 kg (155,000 lb)	20,000	70,310 kg (155,000 lb)	20,000	52,730 kg (116,250 lb)	20,000	52,730 kg (116,250 lb)	200	
	Runway Length <1525 m (5000 ft)	C-17	263,100 kg (585,000 lb)	20,000	263,100 kg (585,000 lb)	20,000	199,014 kg (438,750 lb)	20,000	199,014 kg (438,750 lb)	200	
	Runway Length >1525 m (5000 ft) and <2745 m (9000 ft)	C-17	263,100 kg (585,000 lb)	30,000	263,100 kg (585,000 lb)	30,000	199,014 kg (438,750 lb)	30,000	199,014 kg (438,750 lb)	300	
	Runway Length >2745 m (9000 ft)	C-17	263,100 kg (585,000 lb)	50,000*	263,100 kg (585,000 lb)	50,000	199,014 kg (438,750 lb)	50,000	199,014 kg (438,750 lb)	500***	
Class V		CH47	N/A	N/A	22,680 kg (50,000 lb)	5,000	N/A	N/A	Same as Shoulder Pavements	Same as Shoulder Pavements	
Class VI	Paved Landing Zones**	C-130	70,310 kg (155,000 lb)	10,000	N/A	N/A	N/A	N/A	Same as Runway Pavements	Same as Runway Pavements	Same as Runway Pavements
		C-17	265,325 kg (585,000 lb)	10,000	N/A	N/A	N/A	N/A			
	Semiprepared Landing Zone	C-130	70,300 kg (155,000 lb)	500	N/A	N/A	N/A	N/A			
		C-17	220,445 kg (486,000 lb)	500	N/A	N/A	N/A	N/A			

* At installations with mobilization missions, increase the C-17 pass level to 100,000 passes.
** For paved landing zones less than 1325 m (5,000 ft) reduce the aircraft weight to 502,000 lb.
*** 1000 at installations with mobilization missions.

Figure 2-1. Typical Layout of Traffic Areas for Army Class III and IV Airfields



CHAPTER 3

AIR FORCE AIRFIELD AND HELIPORT REQUIREMENTS

3-1 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 of which airfield type to design for will be made by the appropriate MAJCOM. Designs should be based on medium-load criteria with these exceptions:

- Airfield pavements at Air Education and Training Command bases will be designed for the load and pass level selected by the MAJCOM.
- For bases where B-52s are the critical mission, use heavy-load criteria.
- For bases where the B-1 or KC-10s are the critical mission, use modified heavy-load criteria.
- Landing zone criteria should be used to design runways for C-130 or C-17 training. See ETL 09-6.
- 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.
- Heliport and helipad pavements will be designed in accordance with Chapter 2, “Army Airfield and Heliport Requirements.”
- In lieu of the above criteria, MAJCOMs have the option to design for specific aircraft and projected pass levels.

3-2 TRAFFIC AREAS FOR AIR FORCE AIRFIELDS

On normal operational airfields, the pavements can be grouped into four traffic areas designated as Type A, Type B, Type C, or Type D, which are defined in paragraphs 3-2.1 through 3-2.4 and shown in Figures 3-1 and 3-2. A layout of the assault landing zone is not shown since all areas are Type A traffic areas. Heavy-load and 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.

3-2.1 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 the aircraft. Runways should be designed for Type A traffic their full length if the facility does not have a parallel taxiway that can support large aircraft. These traffic areas require a greater pavement thickness than those areas where the traffic is more evenly distributed. Paragraphs 3-2.1.1 through 3-2.1.4 describe the pavement features considered Type A traffic areas on each airfield type.

3-2.1.1 Heavy-Load Airfield

3-2.1.1.1 Portions of long straight sections of primary taxiways will be Type A traffic areas. Traffic channelization is limited to the center of the primary taxiway for B-52 aircraft; therefore, the center 7.5 m (25 ft) (minimum) of long straight sections will be designed as Type A. 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.

3-2.1.1.2 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.

3-2.1.1.3 Through taxilanes or portions of through taxiways on aprons (7.5 m [25 ft] minimum) will be designed as Type A traffic areas.

3-2.1.1.4 Portions of the first 305 m (1,000 ft) of runway ends will be Type A traffic areas. On these pavements, the effects of channelized traffic are usually confined to the center 23-m (75-ft) 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 m (1,000 ft) of runway ends is to provide a uniform thickness (Type A traffic area) for the full width of the pavement. This is required when using drainage layers. 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. In seasonal frost areas, it is often desirable to use a constant transverse section to preclude differential frost heave.

3-2.1.2 Medium-Load and Modified Heavy-Load Airfield

3-2.1.2.1 Primary taxiways will be designed as Type A traffic areas. The effects of channelized traffic are well defined on long straight sections. The channelization is not as confined as for a heavy-load pavement, however, and it is not practical to construct primary taxiways of alternating variable thicknesses as indicated by traffic requirements. Consequently, 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 the pavement facility. All areas of the primary taxiway, including straight sections, turns, and intersections, will be designated as Type A traffic areas.

3-2.1.2.2 Through taxilanes and portions of through taxiways on aprons (11-m [35-ft] minimum) will be designed as Type A traffic areas.

3-2.1.2.3 Portions of the first 305 m (1,000 ft) of runway ends will be designed as Type A traffic areas. On these pavements, the effects of channelized traffic are usually confined to the center 23-m (75-ft) 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 m (1,000 ft) of runway ends would be to provide a uniform thickness (Type A traffic area) for the full width of the pavement facility. This is required when using drainage layers. In frost areas, using a uniform thickness to preclude differential frost heave is often desirable.

3-2.1.3 **Light-Load and Auxiliary Airfields**
Primary taxiways and the first 305 m (1,000 ft) 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 practical to construct primary taxiways and runway ends of alternating variable thicknesses for light-load and auxiliary airfields as indicated by traffic requirements. Consequently, the primary taxiways and the first 305 m (1,000 ft) of runway ends for light-load and auxiliary airfields will normally be constructed to provide a uniform thickness for the full width of the pavement facility. All areas of the primary taxiway, including straight sections, turns, and intersections, will be designated as Type A traffic areas.

3-2.1.4 **Landing Zone Airfield**
The type of aircraft operations conducted on these pavements will require the entire runway, the 91-m (300-ft) overruns, and the short access taxiways to be designed as Type A traffic areas.

3-2.2 **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 that receive the full design weight of the aircraft during traffic operations. Since the traffic is better distributed 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. These pavement facilities are considered Type B traffic areas on each airfield type:

3-2.2.1 **Heavy-Load Airfield**
All aprons (except hangar access aprons), pads, and traffic lanes adjacent to the center lane on long straight sections of primary taxiways are designed as Type B traffic areas.

3-2.2.2 **Medium-Load and Modified Heavy-Load Airfields**
All aprons (except hangar access aprons) and pads are Type B traffic areas.

3-2.2.3 **Light-Load and Auxiliary Airfields**
All aprons (except hangar access aprons) and pads are Type B traffic areas.

3-2.2.4 **Landing Zone**
There are no Type B traffic areas on landing zones (all traffic area A).

3-2.3 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 usually 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 considerably reduce the stresses applied to the pavements; thus, the pavement thickness can be reduced in these portions of the runways. 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-m (75-ft) width of runway interior. In seasonal frost areas, however, it may be necessary to use a uniform thickness for the entire width of the runway to preclude frost heave. These pavement facilities at all airfields are considered Type C traffic areas:

3-2.3.1 Heavy-Load Airfields

- Secondary (ladder) taxiways.
- The center 23-m (75-ft) width of runway interior between the 305-m (1,000-ft) runway ends, and at runway edges adjacent to intersections with ladder taxiways.
- Hangar access aprons, hangar floors, and wash rack pavements shall be designed as heavy-load Type C traffic areas for the main gear width plus 3 m (10 ft) on each side. The remainder of the pavement in these areas shall be designed as light-load Type C traffic areas.

3-2.3.2 Medium-Load and Modified Heavy-Load Airfields

- Secondary (ladder) taxiways.
- The center 23-m (75-ft) width of runway interior between the 305-m (1,000-ft) runway ends, and at runway edges adjacent to intersections with ladder taxiways.
- Hangar access aprons, hangar floors, and wash rack pavements shall be designed as heavy-load Type C traffic areas for the main gear width plus 3 m (10 ft) on each side. The remainder of the pavement in these areas shall be designed as light-load Type C traffic areas.

3-2.3.3 Light-Load and Auxiliary Airfields

- The full width of the runway interior between the 305-m (1,000-ft) runway ends, and secondary (ladder) taxiways.
- Hangar access aprons and floors.
- Wash rack pavements.

3-2.3.4 Landing Zone
There are no Type C traffic areas on landing zones (all traffic area A).

3-2.4 Type D Traffic Areas
Type D traffic areas are those in which the traffic volume is extremely low 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-m (75-ft) 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. These pavement facilities are considered Type D traffic areas:

3-2.4.1 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.

3-2.4.2 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.

3-2.4.3 Light-Load and Auxiliary Airfields
There are no Type D traffic areas on light-load or auxiliary pavements.

3-2.4.4 Landing Zone
There are no Type D traffic areas on landing zones (all traffic area A).

3-3 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 and apron pavements shall not be designed for jacking loads as long as the footprint of the jack is equal to or greater than the contact area of the combined tires on the aircraft gear being elevated.

3-4 DESIGN PASS LEVELS FOR AIR FORCE PAVEMENTS
Aircraft traffic data reports indicating the 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 MAJCOM.

3-5 RESIN MODIFIED PAVEMENT (RMP)
RMP shall not be used for Air Force airfield pavements.

3-6 ROLLER-COMPACTED PAVEMENT (RCCP)

RCCP shall not be used for Air Force airfield pavements.

3-7 PAVED SHOULDERS

3-7.1 Location

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

3-7.2 Structural Requirements

As a minimum, paved shoulders shall be designed to support 5000 coverages of a load of 4,535 kg (10,000 lb) imposed by a single wheel with a tire pressure of 0.69 MPa (100 psi). When shoulder pavements will be used by support vehicles (e.g., snow removal equipment, fire trucks, fuel trucks), the shoulders should be designed to accommodate the most demanding load.

3-8 SURFACE DRAINAGE

Design of surface drainage shall be in accordance with FAA AC 150/5320-5C.

3-9 MAJCOM DESIGN PREFERENCES

MAJCOM preferences for pavement design shall be obtained from the appropriate MAJCOM. These preferences must be included in the request for proposal (RFP) documents for design build projects.

Figure 3-1. Typical Layout of Traffic Areas for Air Force Light-Load and Auxiliary Airfield Pavements

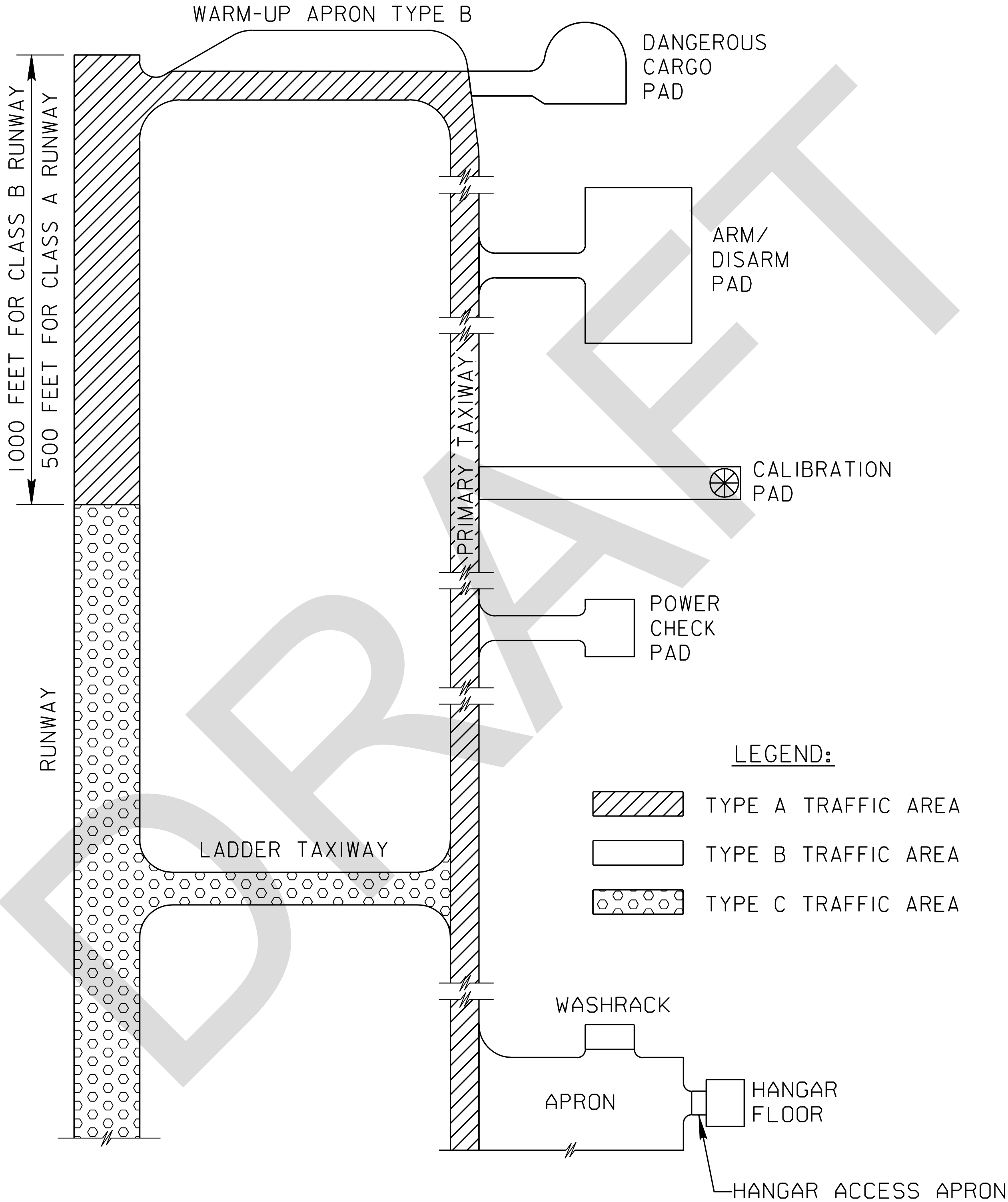


Figure 3-2. Typical Layout of Traffic Areas for Air Force Medium-, Heavy-, and Modified Heavy-Load Airfield Pavements

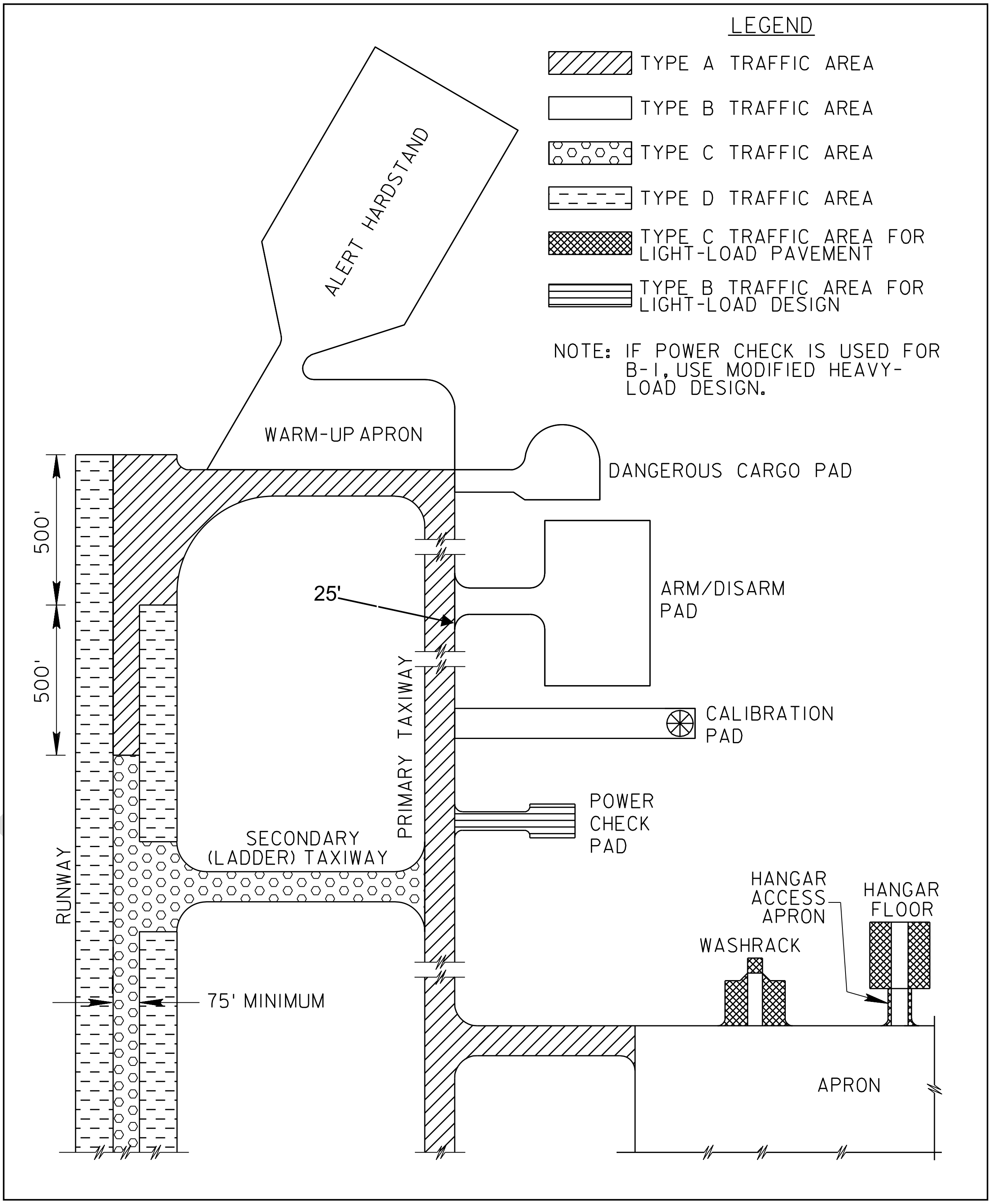


Table 3-1. Design Gross Weights and Pass Levels for Airfield Pavements

Airfield Type	Design Aircraft	A Traffic Area		B Traffic Area		C Traffic Area[1]		D Traffic Area[1]		Overruns[1]		Shoulder
		Weight Pounds	Passes	Weight Pounds	Passes	Weight Pounds	Passes	Weight Pounds	Passes	Weight Pounds	Passes	
Light	F-15 C/D C-17	68,000 585,000	400,000 400	68,000 585,000	400,000 400	51,000 438,750	400,000 400	NA	NA	51,000 435,000	4,000 4	
Medium	F-15 E C-17 B-52[2]	81,000 585,000 400,000	100,000 400,000 400	81,000 585,000 400,000	100,000 400,000 400	60,750 438,750 300,000	100,000 400,000 400	60,750 438,750 300,000	1,000 4,000 4	60,750 438,750 300,000	1,000 4,000 4	
Heavy	F-15 E C-17 B-52	81,000 585,000 480,000	100,000 200,000 120,000	81,000 585,000 480,000	100,000 200,000 120,000	60,750 438,750 360,000	100,000 200,000 120,000	60,750 438,750 360,000	1,000 2,000 1,200	60,750 438,750 360,000	1,000 2,000 1,200	Shoulders are designed to support 5,000 coverages of a 10,000-lb single-wheel load having a tire pressure of 100 psi.
Modified Heavy	F-15 E C-17 B-1	81,000 585,000 480,000	100,000 200,000 120,000	81,000 585,000 480,000	100,000 200,000 120,000	60,750 438,750 360,000	100,000 200,000 120,000	60,750 438,750 360,000	1,000 2,000 1,200	60,750 438,750 360,000	1,000 2,000 1,200	
Landing Zone	C-130 C-17	175,000 502,000	50,000 per squadron 100,000	NA NA	NA NA	NA NA	NA NA	NA NA	NA NA	175,000 502,000	50,000 per squadron 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 for Type A traffic area. 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-ft-wide runways.

Conversion Factors
Kilograms = 0.453 × lb
Megapascals = 0.006894 × psi
Meters = 0.3048 × ft

CHAPTER 4

NAVY AND MARINE CORPS AIRFIELD REQUIREMENTS

4-1 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:

- Aircraft types (gear configurations)
- The maximum gross weight of each aircraft type
- The lateral wander associated with each aircraft type
- The predicted number of operations of each aircraft type over the design life of the pavement

4-2 TRAFFIC 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 Air Force A and B traffic areas, 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.

4-2.1 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:

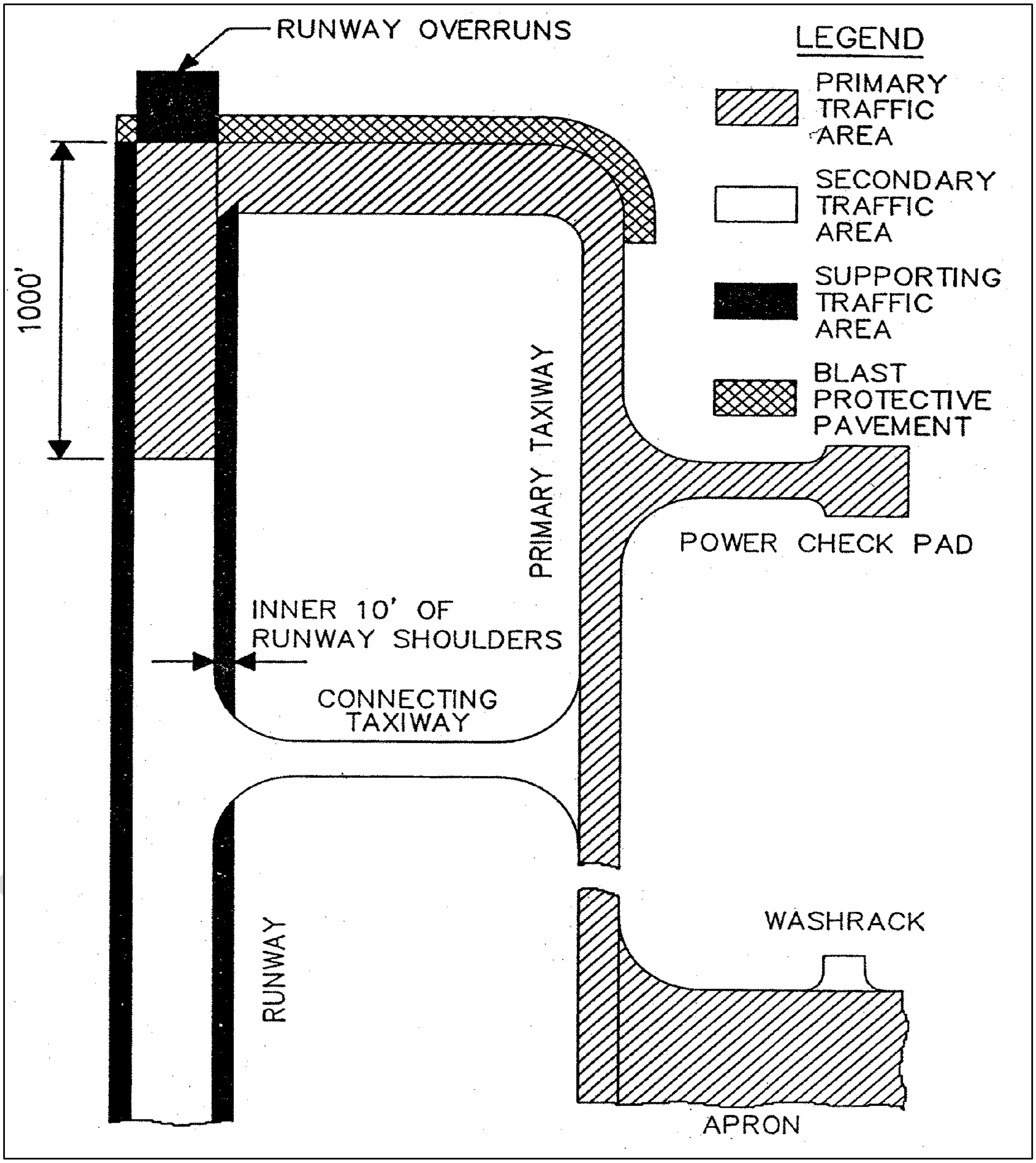
- The first 305 m (1,000 ft) of runways
- Primary taxiways
- Holding areas
- Aprons

4-2.2 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:

- Runway interiors
- Intermediate taxiway turnoffs

Figure 4-1. Primary, Secondary, and Supporting Traffic Areas for Navy and Marine Corps Airfield Pavements



4-2.3 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:

- The inner 3 m (10 ft) of runway shoulders
- Stabilized portions of runway overruns
- Blast protective pavement

4-3 AIRCRAFT LOADINGS

Factors that 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.

4-3.1 Aircraft Types

A landing gear assembly shall consist of a single wheel for smaller aircraft or dual and dual tandem wheels for larger aircraft. Figures 4-2a and 4-2b illustrate the various multiwheel landing gear assemblies and list typical aircraft for each assembly.

4-3.2 Design Weight

The maximum static gear loads are used for pavement thickness design. Tables 4-1a and 4-1b present the design gear loads and other characteristics for Navy and Marine Corps aircraft. Table 4-1b includes the maximum war-time aircraft weights and corresponding gear loads, which could lead to conservative designs for non-strategic airfields. Table 4-1a includes the maximum peace-time aircraft weights and gear loads, which may be more appropriate for many airfields. The design gear loads provided in Tables 4-1a and 4-1b represent the maximum static gear loads expected to be applied to a pavement.

4-3.3 Use of Other Gear Loads in Design

Gear loads other than those listed in Tables 4-1a and 4-1b may be used for design when required. Since certain areas of an airfield (for example, runway shoulders, runway overruns) do not normally carry fully loaded aircraft, they do not need to be designed for the maximum gross weight.

4-3.4 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 and apron pavements shall not be designed for jacking loads as long as the footprint of the jack is equal to or greater than the contact area of the combined tires on the aircraft gear being elevated.

4-3.5 Standard Design Aircraft

One aircraft in each gear assembly group has been designated as the representative aircraft for that group. Table 4-1a identifies the standard aircraft types that are to be used as default values in the design of rigid and flexible pavements only when site-specific aircraft loadings are not available (use in conjunction with Table 4-2).

4-4 TRAFFIC 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. To determine the design traffic for 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. 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 and the pavement is for Navy use only, the traffic pass levels listed in Table 4-2 are the minimum pass levels to be used in design (based on the station type). For bi- or tri-service use (joint use), establish the minimum pass level for each service and use the highest one. For the aircraft shown in Table 4-2, using peace-time take-off weights (Table 4-1a) is recommended. Note that Table 4-2 indicates recommended pass levels for the runway; other features will typically require lower pass levels depending on expected use.

4-5 ROLLER-COMPACTED CONCRETE PAVEMENT (RCCP)

RCCP shall not be used for Navy or Marine airfield or heliport pavements.

4-6 RESIN MODIFIED PAVEMENT (RMP)

RMP shall not be used for Navy or Marine airfield or heliport pavements.

4-7 PAVED SHOULDERS

4-7.1 Location

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

4-7.2 Structural Requirements

See Chapter 10 (Special Areas).

4-8 PAVEMENT DESIGN POLICY

The Navy recognizes the PCASE rigid and flexible pavement design program. For concrete slabs, this implies the use of edge loading. 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 (in) or less. Designers shall complete a sensitivity analysis using the

PCASE program and review the analysis with the senior airfield designer in their geographic area of responsibility.

Figure 4-2a. Landing Gear Configurations for Common Military Aircraft

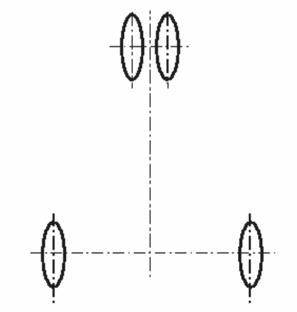
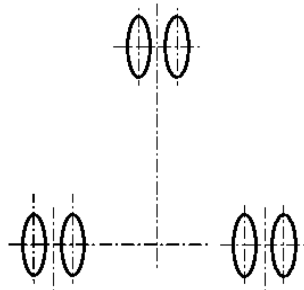
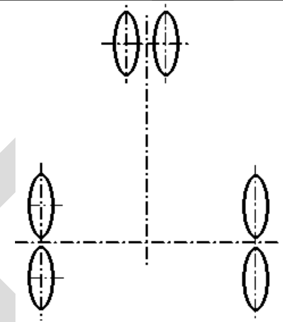
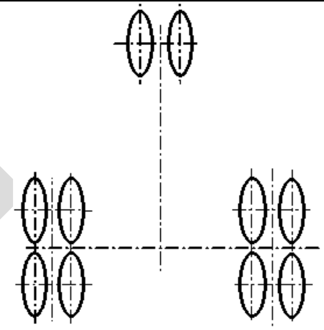
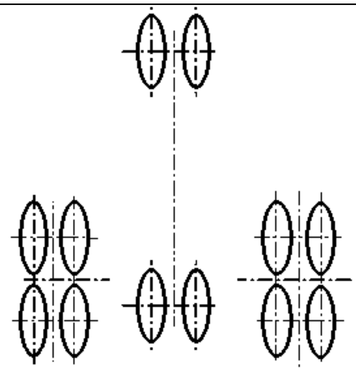
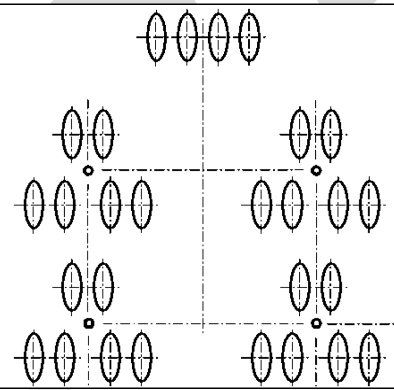
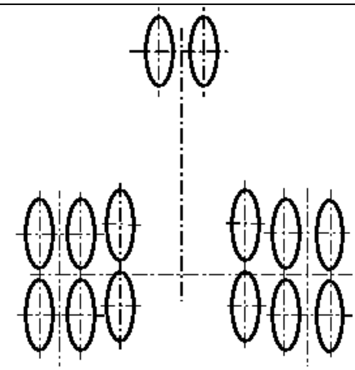
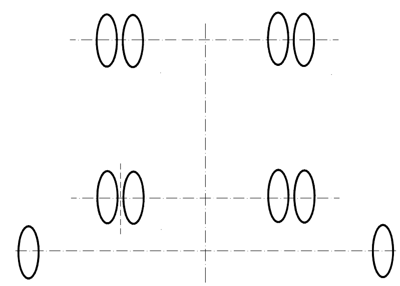
			
S – Single Wheel (S or D nose) F-4, F-5, F-10, F-14, F-15, F-16, F/A-18, F-100, F-106, F-111, T-28, T-33, T-34, T-37, T-38, T-39, T-45, A-7, A-10, A-37, P-2, S-3, E-2, C-12, C-20, C-21, C-23, OV-1, OV-10, UH-60	D – Dual Wheel (S or D nose) DC-9, CH-54, B-727, B-737, T-43, C-7, C-9, C-140, C-22, P-3, CH-47, CH-53, UH-46, C-118	4-9 **2S – Two Single Wheels in Tandem** Lockheed C-130	4-10 **2D – Two Dual Wheels in Tandem** C-141, KC-135, DC-8, DC-10-10, DC-10-10CV, B-1, B-2, B-707, B-757, B-767, E-3, VC-137, A-300, EC-18, E-6
			
2D/D1 – Dual Wheels in Tandem Main Gear / Dual Wheel Body Gear 1 McDonnell Douglas DC-10-30, KC-10, Lockheed L-1011	C5 – Dual Wheel and Quadruple Wheel Combination with Quadruple Wheel Nose Gear Lockheed C-5 Galaxy	2T – Two Triple wheels in Tandem Boeing C-17	D2 - Dual Wheel Gear Two Struts per Side Main Gear with No Separate Nose Gear (single wheel outriggers are ignored) Boeing B-52 Bomber

Figure 4-2b. Special Landing Gear Configurations

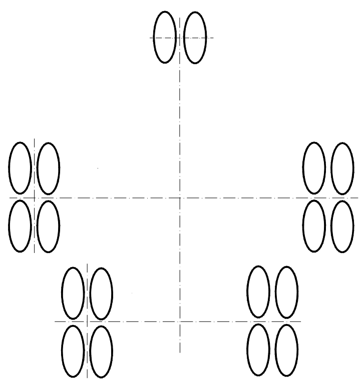
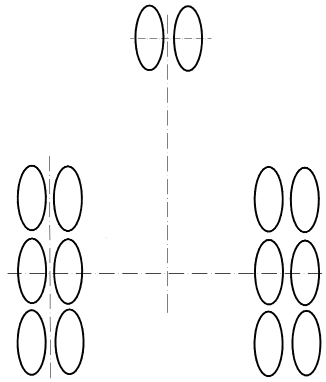
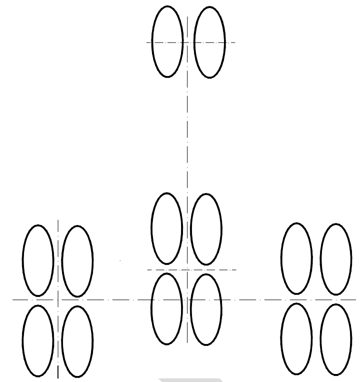
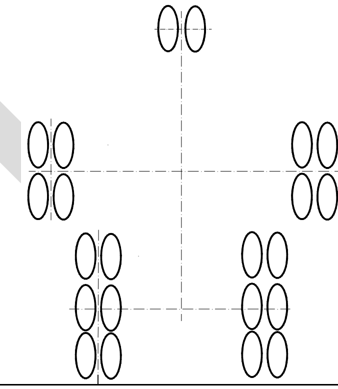
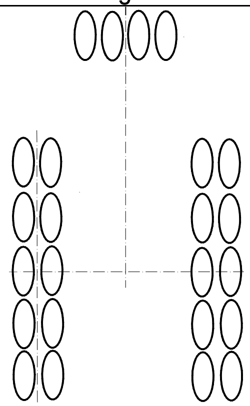
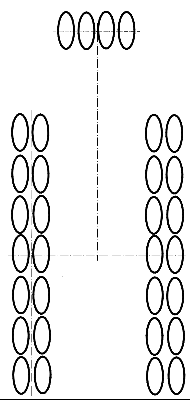
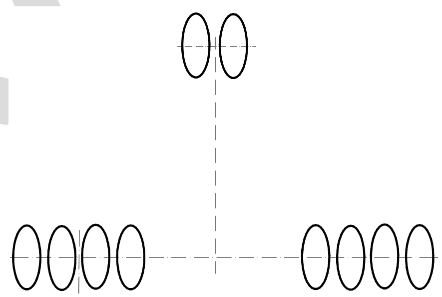
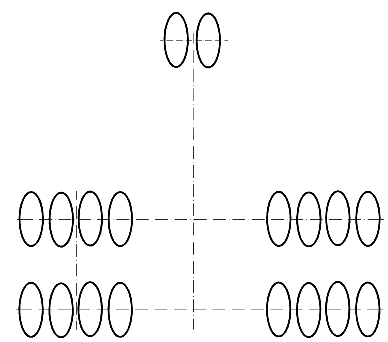
			
<u>2D/2D2 - Two Dual Wheels in Tandem Main Gear / Two Dual Wheels in Tandem Body Gear and Dual Wheel Nose Gear</u> Boeing B-747	<u>3D - Three Dual Wheels in Tandem Main Gear with Dual Wheel Nose Gear</u> Boeing B-777	<u>2D/2D1 Two Dual Wheels in Tandem Main Gear/Two Dual Wheels in Tandem Body Gear with Dual Wheel Nose Gear</u> Airbus A340-600	<u>2D/3D2 - Two Dual Wheels in Tandem Main Gear / Three Dual Wheels in Tandem Body Gear, Dual wheel Nose Gear</u> Airbus A380
			
<u>5D - Five Dual Wheels in Tandem Main Gear, Quadruple Nose Gear</u> Antonov AN-124	<u>7D - Seven Dual Wheels in Tandem Main Gear with Quadruple Nose Gear</u> Antonov AN-225	<u>Q - Quadruple Wheel Main Gear with Dual Wheel Nose Gear</u> Hawker Siddeley HS-121 Trident	<u>Q2 - Quadruple Wheels Two Struts per Side with Quadruple Nose Gear</u> Ilyushin IL-76

Table 4-1a. Standard Design Aircraft Types (at Peace-Time Take-off Weights)

Landing Gear Assembly	Representative Aircraft	Tire Pressure MPa (psi)	Take-off Weight kg (lb)	Design Gear Load kg (lb)
S (Single Wheel)	F-15	2.1 (305)	36,741 (81,000)	15,982 (35,235)
D (Dual Wheel)	P-3	1.31 (190)	61,236 (135,000)	29,087 (64,125)
2T (C-17)	C-17	0.98 (142)	265,352 (585,000)	122,062 (269,100)
2D/D1 (KC-10, DC-10-30)	KC-10	1.14 (165)	267,624 (590,000)	99,021 (218,300)

Table 4-1b. 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	208.8	203.8			--	--
	C-40	T					126,000	171,000	117.4	110.3				
	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	55,000	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	--

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 4-1b. (Concluded)

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]
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	--
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	--	--

Table 4-2. Default Design Passes for 20-Year Design Life

Type of Station	Traffic Group	F-14	P-3	C-17	KC-10
NAEC,NAF,NAS, NAWS, NS, NSA, MCAS, MCAF, MCB	I	300,000	150,000	15,000	10,000
NALF, PMRF	II	50,000	40,000	1,000	--
NOLF	III	100,000	1,000	150	--
Joint Use*	Other	--	--	--	--
*Establish the minimum pass level for each service present and use the highest one.					

CHAPTER 5

SITE INVESTIGATIONS

5-1 OVERVIEW

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 American Society for Testing and Materials (ASTM) 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.

Table 5-1. Soil Sampling and Testing Standards

Category	Description	ASTM
Exploratory borings	Auger samples	D1452
	Split barrel sampling	D1586
	Thin walled sampling	D1587
Identification and classification tests	Liquid limit	D4318
	Plastic limit	D4318
	Sieve analysis	D422
	Finer than No. 200 Sieve	D1140
	Classification (Unified Soil Classification System)	D2487
Laboratory tests	Moisture-density relations	D1557
	Remolded CBR	D1883
	Moisture content	D2216
	Unconfined compression	D2166
	Permeability test	D2434
	Consolidation test	D2435
In-place tests	Density and moisture content:	
	Sand cone	D1556
	Drive cylinder	D2937
	Rubber balloon	D2167
	Nuclear method (density and moisture content)	D6938
	In-place CBR	D4429
	Dynamic cone penetrometer (DCP)	D6951
	Modulus of soil reaction	D1196

5-2 SUBGRADE INVESTIGATIONS

5-2.1 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.

5-2.2 Spacing of Preliminary Borings

The subgrade conditions in the area to be used for airfield pavement construction should be determined by exploratory borings. The maximum spacing of borings is shown in Table 5-2, and should be supplemented with additional borings whenever variations in soil conditions or unusual features are encountered.

Table 5-2. Recommended Maximum Spacing of Borings

Item	Spacing of Borings
Runway and taxiways ≤ 60 m (200 ft) wide	One boring every 60 to 150 m (200 to 500 ft) longitudinally on alternating sides of the pavement centerline
Runways > 60 m (200 ft) wide	Two borings every 60 to 150 m (200 to 500 ft) longitudinally (one boring on each side of the centerline)
Parking aprons and pads	One boring per 2,325-square meter (m^2) (25,000-square foot [ft^2]) area

5-2.3 Depth of Borings

In cut sections, borings should extend to a minimum depth of 3 m (10 ft) below the finished grade or to rock. In shallow fill sections, borings should extend to a minimum depth of 3 m (10 ft) 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 m (6 ft) or less). In high-fill sections, borings should extend to a minimum depth of 15 m (50 ft) below the surface of the natural subgrade or to rock. The results of borings will be used to develop boring logs as illustrated in Figure 5-1.

5-2.4 Soil 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 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.

Figure 5-1. Typical Boring Log

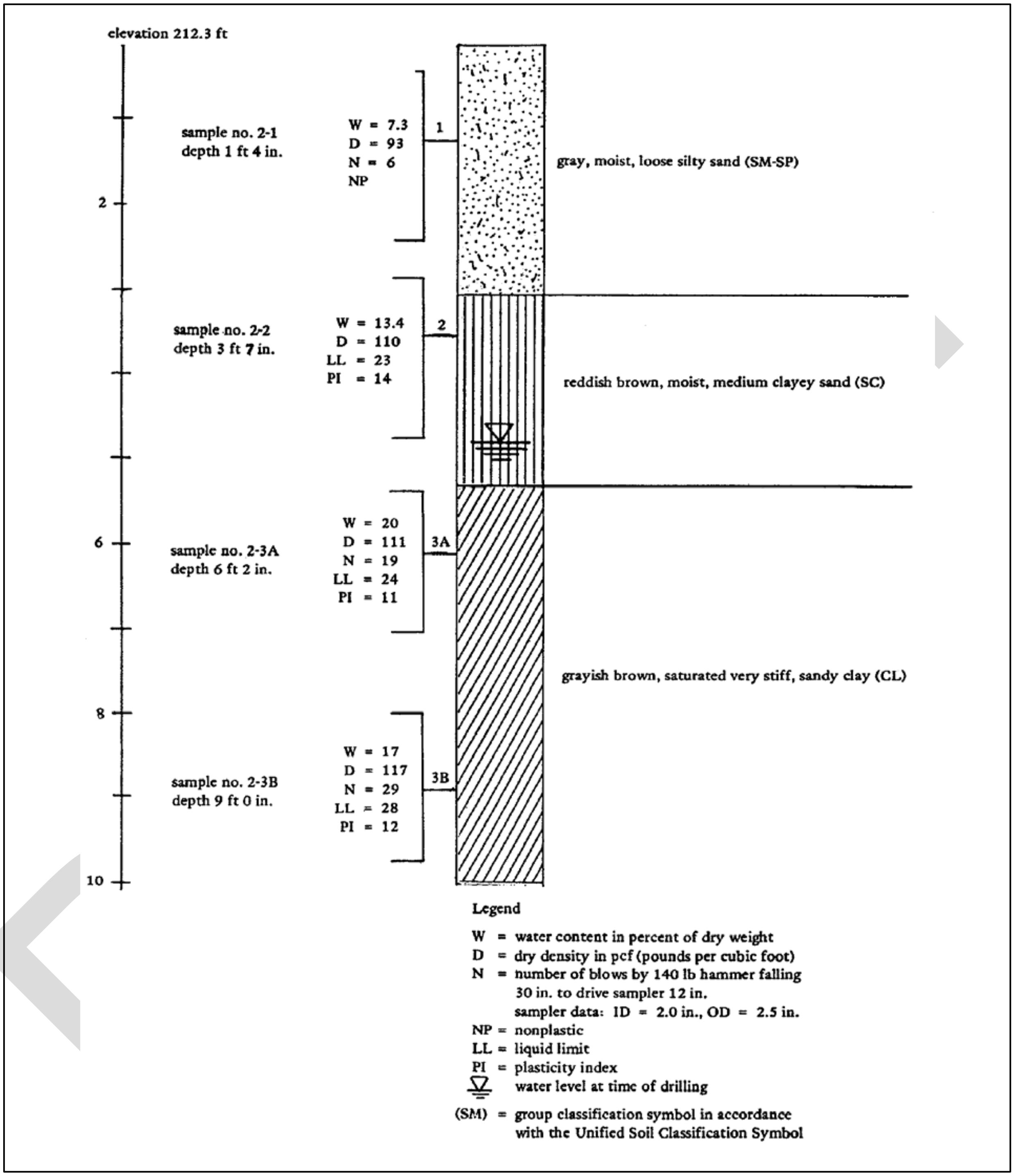
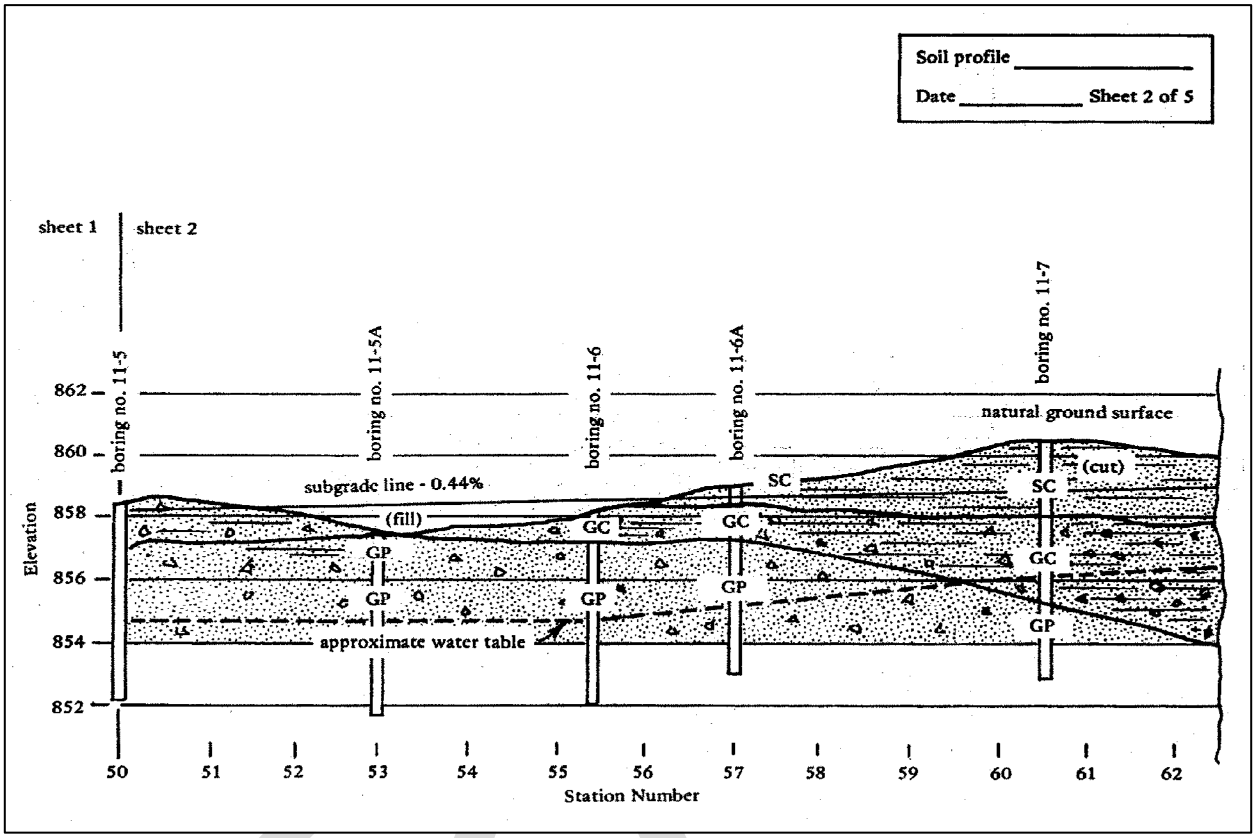


Figure 5-2. Typical Soil Profile



5-2.5 Borrow Areas

Where material is to be borrowed, borings should be made to a depth of 0.6 to 1.2 m (2 to 4 ft) below the anticipated depth of borrow. One boring should be made for each 930 m^2 (10,000 ft^2), with a minimum of three borings per borrow area. Samples from the borings should be classified and tested for water content, density, and strength.

5-2.6 Environmental Hazards

When conducting subsurface investigations, hazardous or toxic waste material may be located, and appropriate environmental actions must 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. In some areas, buried materials may also require some action.

5-3 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.

5-4 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.

5-5 OTHER CONSTRUCTION MATERIALS

Determine the availability and quality of bituminous materials and portland cement. Determining 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.

5-6 SOIL CLASSIFICATION

All soils will be classified in accordance with the Unified Soil Classification System (USCS) as specified in ASTM D2487. 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 necessary. When classifying soils, be alert to the presence of problem soils such as those described in paragraphs 5-6.1 through 5-6.3.

5-6.1 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.

5-6.2 Soils that Become Quick When Molded

The deposits of some soils 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 most of the void space.

5-6.3 Soils with Expansive Characteristics

Expansive soils are generally those with a liquid limit (LL) more than 40 and a plasticity index (PI) more than 15. Soils with expansive characteristics give the most trouble when significant changes occur in the moisture content of the subgrade during different seasons of the year. UFC 3-220-07 may be helpful in identifying expansive soils.

5-7 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 a higher laboratory density. In some cases, constructing field test sections to establish compaction characteristics may be necessary.

5-8 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 and modulus of soil reaction test, along with guidance in determining soil strength values, are presented in Chapters 6 through 8.

5-9 IN-PLACE SOIL STRENGTH TESTS

Test pits for in-place soil strength tests and associated moisture-density tests should be located at approximately 305-m (1,000-ft) intervals for runways and taxiways. For parking aprons and pads, one test pit should be located for each 16,720 m^2 (20,000 square yards [yd^2]). The number and spacing of test pits may be modified whenever variations in soil conditions or unusual features are encountered.

CHAPTER 6

SUBGRADE

6-1 SUITABILITY OF SUBGRADE

The information obtained from the explorations and tests described in Chapter 5 should be adequate to enable full consideration of all factors affecting the suitability of the subgrade and subsoil. There are ten primary factors:

- The general characteristics of the subgrade soils
- Depth to bedrock
- Depth to water table (including perched water table)
- The compaction that can be attained in the subgrade and the adequacy of the existing density in the layers beneath the zone of compaction
- The strength that the compacted subgrade, uncompacted subgrade, and subsoil will have under local environmental conditions
- The presence of weak or soft layers in the subsoil
- Susceptibility to detrimental frost action
- Settlement potential
- Expansion potential
- Drainage characteristics

6-2 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, the economics of plans for construction must be given prime consideration.

6-3 SUBGRADE CBR

Traditional flexible pavement design subgrade strength is expressed in terms of CBR. Several test methods are used to obtain the appropriate range of design strengths. Paragraphs 6-3.1 through 6-3.3 discuss the test methods. 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 object of the testing program is to determine the long-term CBR value for the subgrade, taking into account various densities and moisture conditions to be experienced during the life of the pavement.

6-3.1 Applicable ASTM Standards

6-3.1.1 ASTM D1883 is used to determine the CBR of remolded material. This test method allows for the remolding of the material to a range of densities and moisture contents, a 4-day soaking period meant to mimic long-term moisture gain, and presents the data in a plot of the range of CBR versus density and molding moisture content. When using ASTM D1883, the designer should always select the option in the test method for a range of densities and moisture contents. Paragraph 6-3.2 provides guidance on interpretation of the test data and selection of a design CBR.

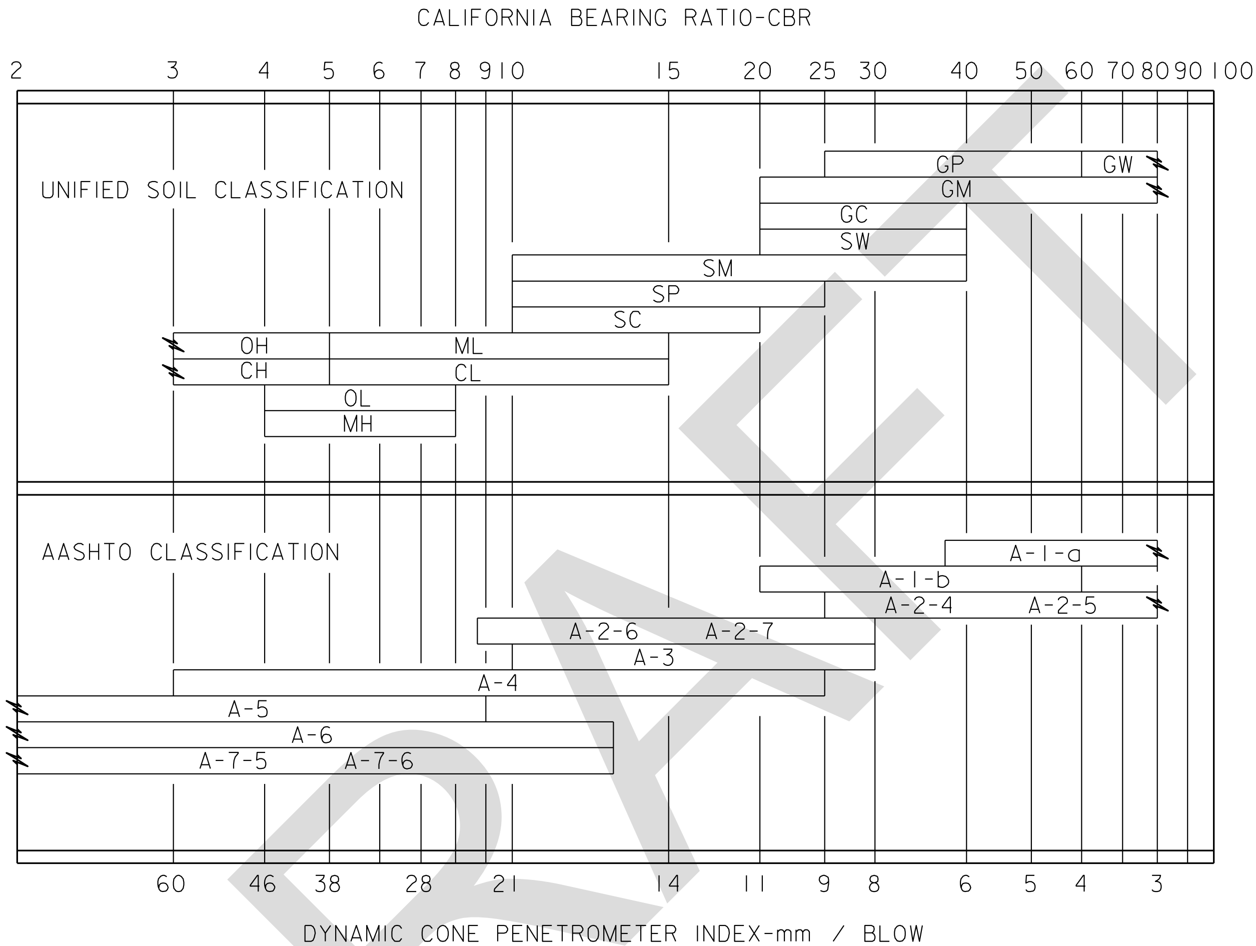
6-3.1.2 ASTM D4429 allows for CBR testing of in-place materials. This method does give a value of CBR that represents the in-situ material but does not allow the designer to estimate possible variations in the CBR value with respect to changes in the density and moisture content. This method can be used to estimate the CBR strength under an existing pavement by making adjustments to the test value for small aperture testing described in paragraph 6-3.3.2.

6-3.1.3 ASTM D6951 allows index testing for the CBR using the dynamic cone penetrometer (DCP). This method is extremely fast and can be conducted through small holes in the paving surface. Paragraph 6-3.3.3 addresses this test method. Figure 6-1 shows typical CBR values based on soil classifications and the DCP index. Figure 6-2 provides a comparison of CBR to modulus of subgrade reaction (*k*) values for various soil classifications.

6-3.2 Laboratory Tests

CBR test results should include a full family of curves as shown in Figure 6-3 and described in ASTM D1883. The test procedures for highly swelling soils are the same as those for cohesive soils; however, the objectives of the testing program are not exactly the same. Tests shall be performed on soils having expansive characteristics to determine a moisture content and a density that will minimize expansion. The 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 estimating the range of moisture contents and densities that can be expected during actual construction, determine the range of CBR values that will result from these variations in moisture and density. Select the design CBR value for the specific soil tested near the lower part of the range. Paragraphs 6-3.2.1 through 6-3.2.3 outline the steps in the selection of a design CBR value, and Figures 6-3 and 6-4 illustrate these steps.

Figure 6-1. CBR Values versus Soil Types and Dynamic Cone Penetrometer Index



NOTE: CORRELATIONS NOT TO BE USED FOR DESIGN. AFTER PORTLAND CEMENT ASSOCIATION "THICKNESS DESIGN FOR CONCRETE HIGHWAY AND STREET PAVEMENTS" PACKARD, ROBERT G., 1984. MODIFIED FOR THIS PUBLICATION.

Figure 6-2. CBR versus Modulus of Soil Reaction for Varying Soil Types

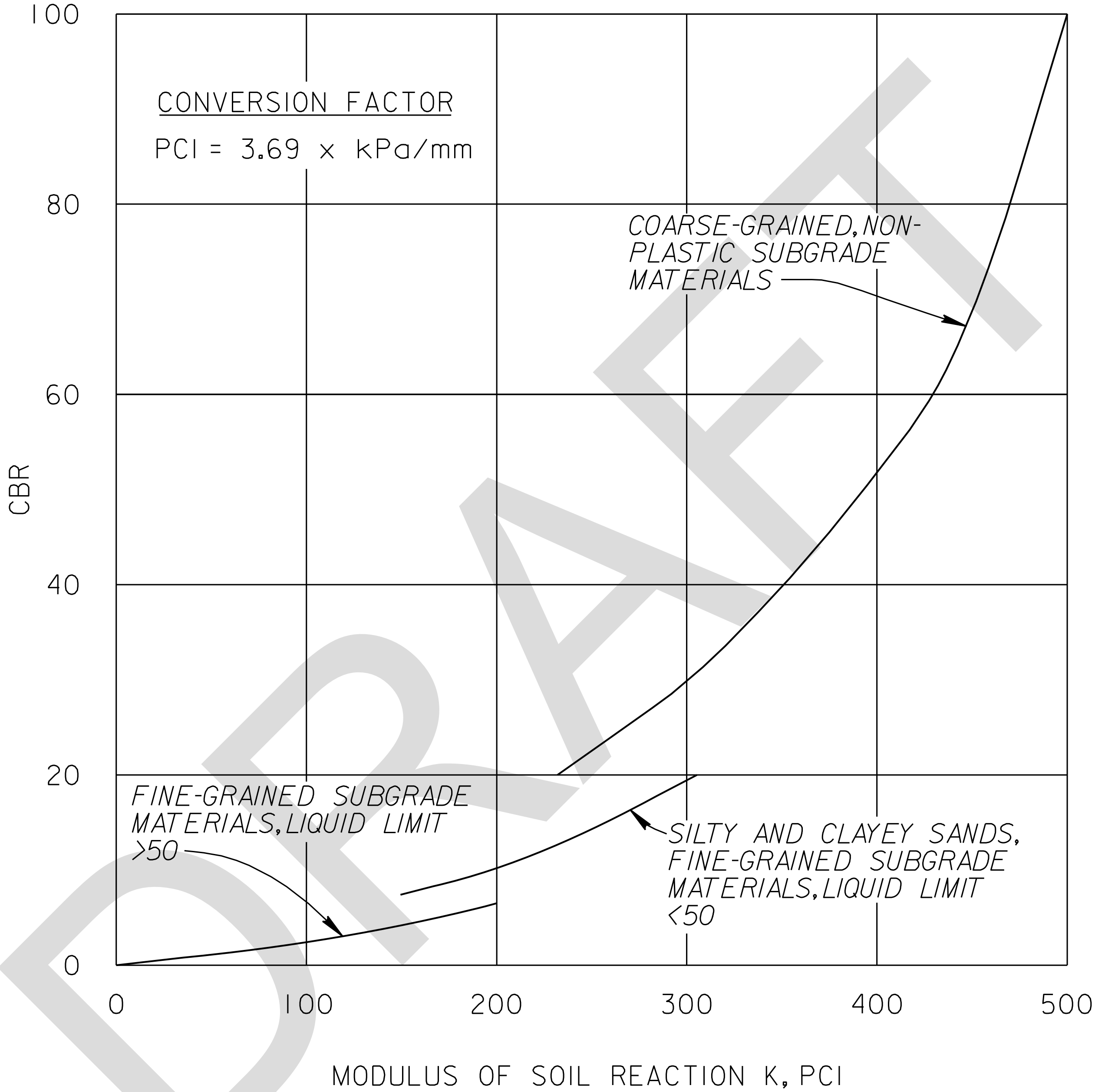
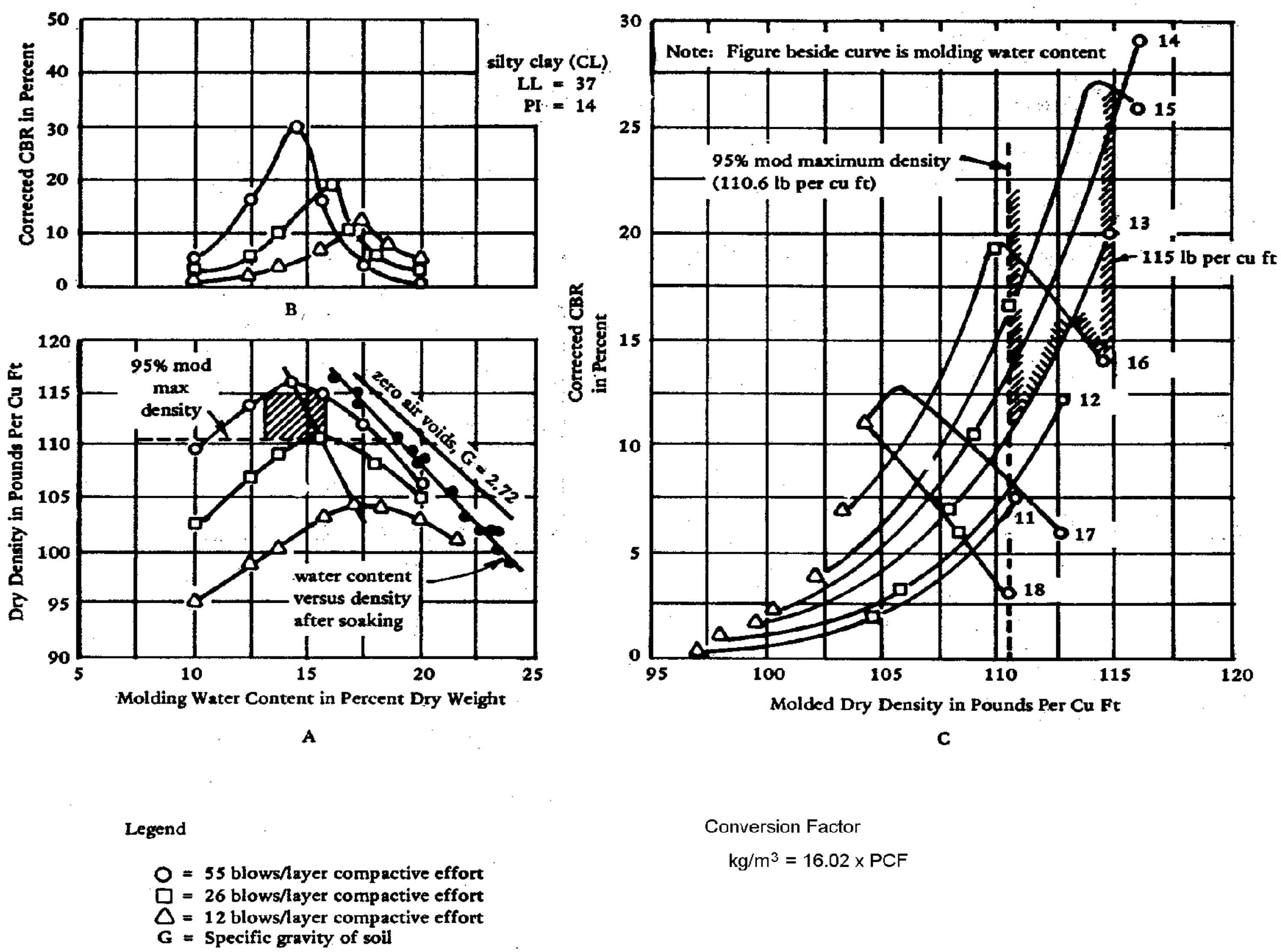


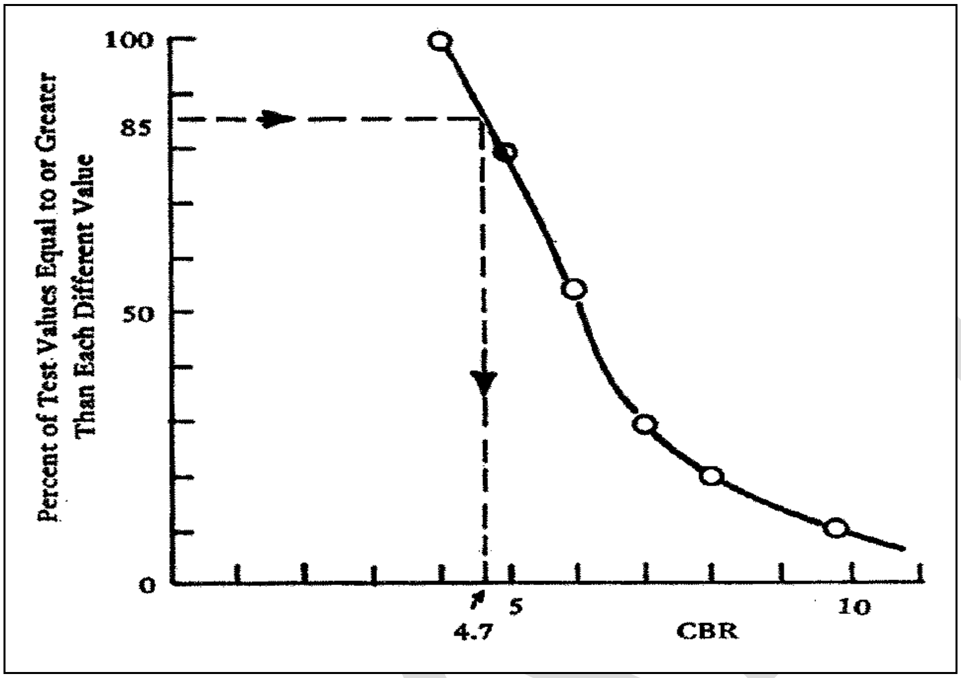
Figure 6-3. Procedure for Determining Laboratory CBR of Subgrade Soils



Conversion Factor

kg/m^3 = 16.02 x PCF

Figure 6-4. Selection of Design Subgrade CBR using In-place Tests



6-3.2.1 **Step A**

Determine the moisture/density relationship (ASTM D1883) at 10, 25, and 56 blows per layer. Plot the density to which the 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 ±1.5 percent of optimum moisture content for approximately 13 and 16 percent. The shaded area represents compactive effort greater than 95 percent and within ±1.5 percent of optimum moisture content.

6-3.2.2 **Step B**

Plot the laboratory CBR (ASTM D1883) for 10, 25, and 56 blows per layer.

6-3.2.3 **Step C**

Plot the CBR versus dry density at a constant moisture content. Plot attainable compaction limits of 1,770 and 1,840 kilograms per cubic meter (kg/m^3) (110.6 and 115 lb per cubic foot [lb/ft^3]) for this example. The hatched area represents attainable CBR limits for the desired compaction of 1,770 and 1,840 kg/m^3 (110.6 to 115 lb/ft^3) and moisture content (13 to 16 percent). The 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 the range is used. In the example, a CBR of 12 with a moisture content specified between 13 and 16 percent is selected.

6-3.3 **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 that are at least 80 percent saturated in the natural state, and soils under existing similar pavements that have reached the maximum water content expected and thus require no soaking. When in-place tests or tests on undisturbed soils are used, a statistical approach is recommended for selecting the design CBR.

6-3.3.1 **Example of Statistical Approach to Selecting the Design CBR**

(1) Given these 20 CBR test values from a runway site: 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) Table 6-1 shows the percent of CBR values equal to or greater than each different value.

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

(4) Enter Figure 6-4 at 85 percent. Continue to the plotted curve, then down to the 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.

Table 6-1. Statistical Comparison of CBR Values

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		

CBR	Number Equal to or Greater than Each Different Value	Percent Equal to or Greater than Each Different Value
4		
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

6-3.3.2 ASTM D4429

This test method uses the small aperture technique to estimate the CBR values of in-situ base courses, subbases, and subgrades below rigid or flexible pavements. For this application, access to the underlying courses is provided through a 6-in-diameter (152-millimeter [mm]-diameter) core hole in the pavement. Surcharge weights are not required because the underlying courses are confined by the existing paving system. A correction to the field-measured CBR value is required in accordance with Figure 6-5.

6-3.3.3 ASTM D6951

This test method can be used to evaluate the in-place CBR values. The data should be adjusted for variations in moisture or density that might occur in the tested material.

Note: These test methods give the designer various tools to test and evaluate the CBR for the given site; however, engineering judgment should be used to establish the appropriate design value to represent the over-the-life-of-the-pavement CBR.

6-4 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. Determine the *k* value by the field plate bearing test using a 30-in-diameter (760-mm-diameter) plate, as described in ASTM D1196.

6-4.1 Strength Test

The field plate bearing test will be performed on representative areas of the subgrade, taking into consideration such factors as changes in material classification, fill or cut areas, and varying moisture (drainage) conditions that would affect the support value of the subgrade. Though 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 meganewtons per cubic meter (MN/m^3) (10 lb per cubic inch [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 6-2.

Figure 6-5. CBR Adjustment Curve

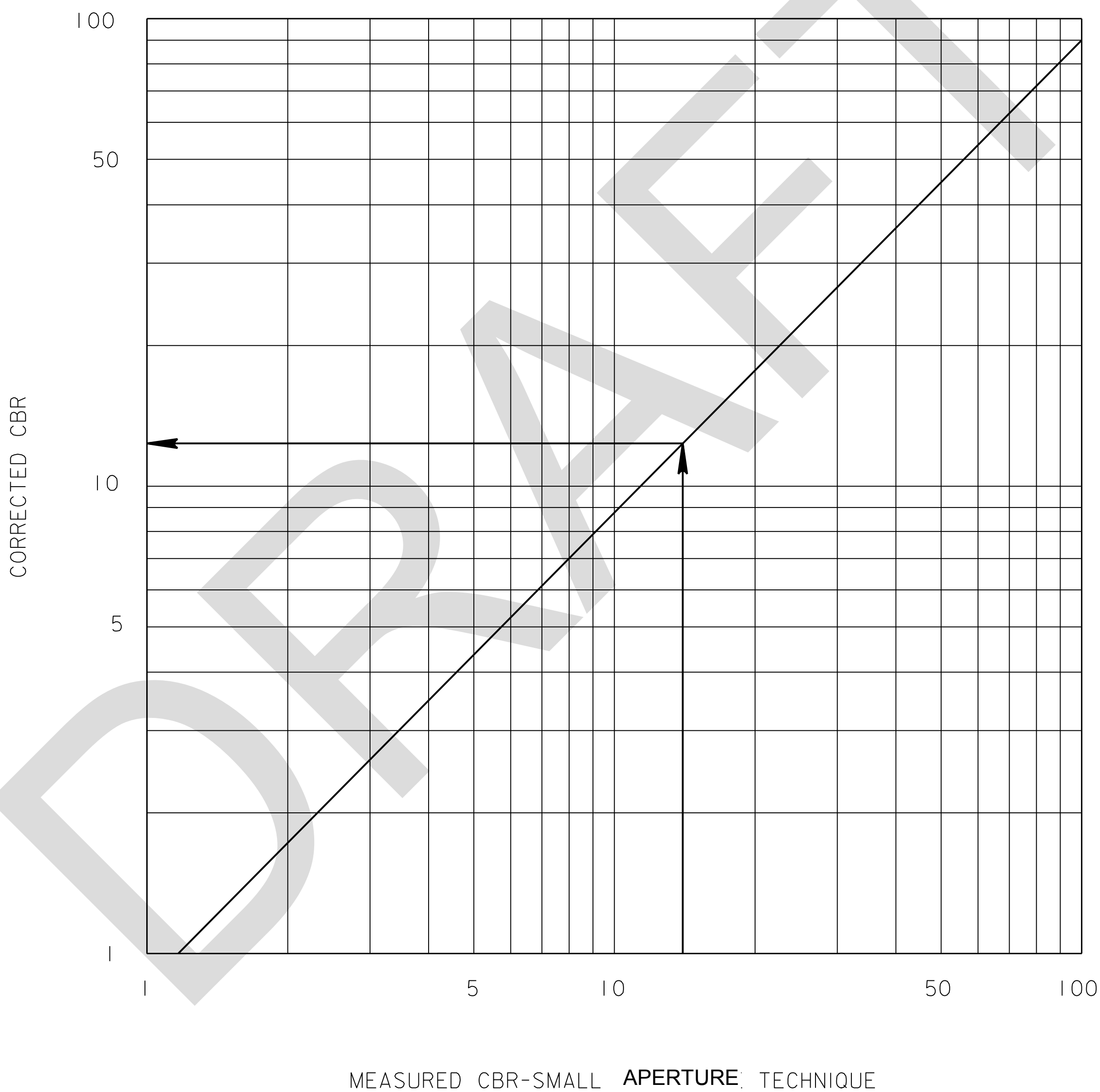


Table 6-2. Typical Values of Modulus of Soil Reaction

Soils	Typical Range kPa/mm (psi/in)	Suggested Default Pavement Design Values if No Test Data is Available kPa/mm (psi/in)
Organic Soils (OL, OH, Pt)	6.77–27.10 (25–100)	6.77 (25)
Silts and Clays of High Plasticity (CH, MH)	13.55–40.65 (50–150)	13.55 (50)
Silts and Clays of Low Plasticity (CL, ML)	13.55–54.20 (50–200)	27.10 (100)
Silty and Clayey Sands (SM, SC)	13.55–67.75 (50–250)	40.65 (150)
Well- and Poorly-Graded Sands (SW, SP)	40.65–108.40 (150–400)	54.20 (200)
Silty and Clayey Gravels (GC, GM)	54.20–135.50 (200–500)	67.75 (250)
Well- and Poorly-Graded Gravels (GW, GP)	81.30–135.50 (300–500)	98.85 (350)

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

6-4.2 Special Conditions

The field plate bearing test results require a saturation correction 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. The saturation correction factor is the ratio of the deformation of the consolidation specimen at the natural moisture content to the deformation in a saturated specimen under a 2.7 kilopascals per mm (kPa/mm) (10-psi) loading. Two specimens of the undisturbed material are placed in a consolidometer. One specimen will be tested at the in-situ moisture content, and the other specimen will be saturated after the seating load has been applied. Each specimen is then subjected to the same seating load (0.27 or 0.55 kPa/mm [1 or 2 psi]) that was used for the field test. The seating load is allowed to remain on the in-situ moisture content specimen until all deformation occurs, at which time a “zero” reading is taken on the vertical deformation dial. Without releasing the seating load, an additional 2.7 kPa/mm (10-psi) load is applied to the specimen and allowed to remain until all deformation has occurred. A final reading is then taken on the vertical deformation dial. The other specimen is allowed to soak in the consolidometer under the seating load 0.27 or 0.55 kPa/mm (1 or 2 psi). After the specimen is saturated, a “zero” dial reading is obtained. Then, without releasing the seating load, an additional 2.7 kPa/mm (10-psi) load is applied. This load is allowed to remain on the specimen until all vertical deformation has occurred, after which a final reading on the dial is obtained. The correction for saturation will be applied in proportion to the deformation of the two specimens under a unit load of 2.7 kPa/mm (10 psi) as in this equation:

$$k = k_u[d/d_s+\{b/75\ (1-d/d_s)\}] \quad (6\text{-}1)$$

where

k = corrected modulus of soil reaction, psi/in

k_u = modulus of soil reaction uncorrected for saturation, psi/in

d = deformation of a consolidometer specimen at in-situ moisture content under a unit load of psi

In arid regions, however, or regions where the water table is 3 m (10 ft) 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.

6-5 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 these requirements:

6-5.1 Subgrades with CBR Values above 20

6-5.1.1 Army and Air Force

Compact to 100 percent density from ASTM D1557, except where a higher density is known to be obtainable practically. Then, the higher density will be required.

6-5.1.2 Navy and Marine Corps

Compact to 95 percent of ASTM D1557 maximum density.

6-5.2 Subgrades with CBR Values of 20 or Less

6-5.2.1 Fills

Subgrades in fills shall have densities equal to or greater than the values determined from Tables 6-3 through 6-8. Cohesionless fill will not be placed at less than 95 percent or cohesive fill at less than 90 percent of maximum density from ASTM D1557. The top 152 mm (6 in) of subgrade will be compacted to 95 percent of maximum density from ASTM D1557.

6-5.2.2 **Cuts**

Subgrades in cuts shall have natural densities equal to or greater than the values determined from Tables 6-3 through 6-8. 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 in paragraph 6-5.2.1 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 mm (6 in) of subgrade will be compacted to 95 percent of maximum density from ASTM D1557.

6-5.3 **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.

6-5.4 **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 the strength used for the flexible pavement design. The requirements for density in Tables 6-3 through 6-7 coupled with proof rolling (section 8-9) accomplish the first objective. The interaction between specified compaction levels and moisture contents and design strength is described in section 6-3 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 half a 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 curves. 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 both meet the minimum specified density and to accomplish the compaction within the specified ranges of moisture content.

Table 6-3. Compaction Requirements for Cohesive Subgrades and Select Materials under Flexible Pavements: Air Force Pavements (LL ≥ 25, PI ≥ 5)

Airfield Type	Depth of Compaction below the Pavement Surface, inches															
	85 percent				90 percent				95 percent				100 percent			
	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
Short Field	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 6-4. Compaction Requirements for Cohesionless Subgrades and Select Materials under Flexible Pavements: Air Force Pavements (LL < 25, PI < 5)

Airfield Type	Depth of Compaction below the Pavement Surface, Inches															
	85 percent				90 percent				95 percent				100 percent			
	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
Short Field	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 6-5. Compaction Requirements for Cohesive Subgrades and Select Materials under Flexible Pavements: Army Pavements (LL ≥ 25, PI ≥ 5)

Airfield Type	Depth of Compaction below the Pavement Surface, inches											
	85 percent			90 percent			95 percent			100 percent		
	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	17	16	13	13	12	10	10	9	7	6	5	4
Class IV												
Runway ≤ 5,000 ft	40	38	32	30	28	24	21	20	16	11	11	8
Runway > 5,000 ft and Runway ≤ 9,000 ft	57	55	46	43	41	33	29	27	22	16	16	12
Runway > 9,000 ft	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 6-6. Compaction Requirements for Cohesionless Subgrades and Select Materials under Flexible Pavements: Army Pavements (LL < 25, PI < 5)

Airfield Type	Depth of Compaction below the Pavement Surface, inches											
	85 percent			90 percent			95 percent			100 percent		
	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	27	26	23	21	20	18	15	15	13	9	9	7
Class IV												
Runway ≤ 5,000 ft	76	72	61	57	54	45	38	36	30	21	20	16
Runway > 5,000 ft and Runway ≤ 9,000 ft	104	100	85	79	77	65	54	52	43	29	28	22
Runway > 9,000 ft	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 6-7. 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-17	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-17	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												

Table 6-8. Compaction Requirements for Shoulders

Percent Compaction	[1]Depth of Compaction in inches for Cohesive Subgrades and Select Materials (LL $\geq$ 25, PI $\geq$ 5)	[1]Depth of Compaction in inches for Cohesionless Subgrades and Select Materials (LL < 25, PI < 5)
85	17	29
90	14	23
95	10	16
100	6	10
[1] Depth is measured from the pavement surface. Conversion Factor: Millimeters = 25.4 $\times$ inches		

6-6 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 paragraphs 6-6.1 through 6-6.3.

6-6.1 Compacting Fill Sections

Fills composed of soil having a PI greater than 5 or a LL greater than 25 will be compacted to not less than 90 percent of ASTM D1557 maximum density. Fills composed of soil having a PI equal to or less than 5 and a LL equal to or less than 25 will be compacted to these specifications: the top 152 mm (6 in) will be 100 percent of ASTM D1557 maximum density, and the remaining depth of fill will be 95 percent of ASTM D1557 maximum density. Large fills on natural soil should be analyzed for bearing capacity and settlement using conventional soil mechanics.

6-6.2 Compacting Cut Sections

The top 152 mm (6 in) of subgrades composed of soil having a PI greater than 5 or a LL greater than 25 will be compacted to not less than 90 percent of ASTM D1557 maximum density. If the natural subgrade exhibits densities equal to or greater than 90 percent of ASTM D1557 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 a LL equal to or less than 25 will be compacted to these specifications: the top 152 mm (6 in) will be 100 percent of ASTM D1557 maximum density, and the 455 mm (18 in) below the top 152 mm (6 in) will be 95 percent of ASTM D1557 maximum density. 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.

6-6.3 Permissible Variations in Field Density

The criteria in paragraphs 6-6.1 and 6-6.2 should be considered as minimal values. Also, note that it is often difficult to correlate lab 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.

6-7 TREATMENT OF PROBLEM SOILS

Although compaction increases the strength of most soils, some soils decrease in stability when scarified, worked, and rolled. Some soils also shrink excessively during dry periods and expand excessively when allowed to absorb moisture. All of these soils require special treatment. General descriptions of the soils in which these conditions may occur and suggested methods of treatment are outlined in paragraphs 6-7.1 through 6-7.4.

6-7.1 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. For such clay soils, 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.

6-7.2 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 sands 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 a high water table, drying is usually not satisfactory until the water table is lowered because recompacting operations will cause water to be pumped to the surface again. 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. In some cases, drainage is not feasible and a high water table cannot be lowered; in others, such soils become saturated from sources other than a high water table and cannot be dried (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.

6-7.3 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 approximately 3 percent (as determined from a 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.

6-7.3.1 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 exercised carefully because if the soil is too dry when compacted, the expansion will increase, and if the soil 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.

6-7.3.2 Overburden Load

To limit the swell of expansive soils, it may be desirable to provide overburden if expansion cannot be limited to acceptable amounts by other procedures. Normally special swell tests will be needed to determine the weight (overburden) necessary to restrict the swell to tolerable magnitudes. These tests can be variations of the standard soaked CBR test described in Concrete Research Division (CRD) C 656, or they can be specially designed tests using a consolidometer apparatus.

6-7.3.3 Special Solutions

Special solutions to the problem of swelling soils are sometimes possible and should not be overlooked where viable. For instance, where the 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 (UFC 3-250-11), replacement of the swelling soil, or working the soil to make it more uniform.

6-7.4 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 the overlying structure must be provided to protect the low strength subgrade. When a low density results, the thickness of the overlying material must be such that the density versus depth requirements of the specifications are met.

6-8 STABILIZED SUBGRADES

Subgrades can be stabilized by the addition of lime, cement, or a combination of these materials with fly ash. The design of pavements using stabilized soils is discussed in Chapter 9 of this document and in UFC 3-250-11. Lime should not be used with soils containing sulfates.

6-9 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.

CHAPTER 7

SELECT MATERIALS AND SUBBASE COURSES FOR FLEXIBLE PAVEMENTS

7-1 OVERVIEW

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 subbases 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 such materials as stone, limerock, or sand to produce an acceptable subbase; however, "blending in" cohesionless materials to lower the PI will not be allowed.

7-2 MATERIALS

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 7-4, Selection of Design CBR, and meet the Los Angeles abrasion test (ASTM C131) requirement of not more than 50 percent. Do not use material that has a swell of 3 percent or greater, as determined from the CBR mold, for subbase.

7-2.1 Select Materials

Select materials will usually be locally available coarse-grained soils. Recommended gradation and plasticity requirements for select materials are listed in Table 7-2.

7-2.2 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 7-4. The existing subgrade may meet the requirements for a subbase course, or treating the existing subgrade to produce a subbase may be possible. Also, admixing native or processed materials will be done only when the unmixed subgrade meets the LL and PI requirements for subbases because experience has shown that "cutting" plasticity in this way is not satisfactory. It may be permissible, however, to decrease the plasticity of some materials by using lime or portland cement in sufficient amounts to meet the plasticity requirements of subbases. To be considered stabilized for thickness design purposes, the soil must meet the minimum strength requirements in Table 7-1.

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

Stabilized Soil Layer	Minimum Unconfined Compressive Strength[1] KPa/mm (psi)	
	Flexible Pavement	Rigid Pavement
Base course	5.17 (750)	3.45 (500)
Subbase course, select material, or subgrade	1.72 (250)	1.38 (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.

7-3 COMPACTION REQUIREMENTS

Subbases will be compacted to 100 percent of maximum density as determined by ASTM D1557. 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 not less than 95 percent and cohesive select materials at not less than 90 percent of ASTM D1557 maximum density.

7-4 SELECTION OF DESIGN CBR

The select materials or subbase will generally be uniform, and ordinarily the problem of selecting a limiting condition, as described for the subgrade, will not exist. Tests 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-mm (6-in) 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.

7-4.1 Navy Minimum Subbase CBR

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

7-4.2 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.

Table 7-2. Maximum Permissible Values for CBR, Gradation, and Atterberg Limit Requirements[1]

Material	Maximum CBR	Maximum Size	Maximum Percent Passing		Maximum Liquid Limit[3]	Maximum Plasticity Index[3]
			#10	#200[2]		
Subbase	50	76 mm (3 in)	50	15	25	5
Subbase	40	76 mm (3 in)	80	15	25	5
Subbase	30	76 mm (3 in)	100	15	25	5
Select Material	20	76 mm (3 in)[4]	--	25[4]	35[4]	12[4]

[1] Chapter 23 in this UFC contains maximum values for open-graded and rapid-draining materials.
[2] This limit shall be 8 percent or less if the material is used over a drainage layer.
[3] ASTM D4318
[4] Suggested limits

7-4.3 **Example**

As an example of the selection of a design CBR for subbases or select materials, consider this material:

- Soaked laboratory CBR = 40
- Maximum size, mm (in) = 50 (2.0)
- Percent passing 2.0 mm (No. 10) = 85
- Percent passing 0.075 mm (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 mm (No. 10) sieve is the maximum permitted for higher CBR values and this material has 85 percent passing.

7-5 SEPARATION LAYERS

The gradation requirements in Table 7-2 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 Chapter 23.

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.

7-6 STABILIZED SELECT MATERIALS AND SUBBASES

The design of pavements using stabilized soils is discussed in Chapter 9 of this document and in UFC 3-250-11.

7-7 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 20.

7-8 DRAINAGE LAYERS

The requirements for drainage layers used for subbase are presented in Chapter 23. For pavements in nonfrost areas and having a subgrade with permeability greater than 6 m/day (20 ft/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 203 mm (8 in) 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

8-1 USE OF AGGREGATE BASE COURSES

Aggregate base courses may be required for one or more of these reasons: to distribute load, provide drainage, protect from frost, provide a uniform bearing surface for the pavement surfacing, replace unsuitable soils, provide a working platform, increase the strength of the pavement system, and to prevent pumping.

8-2 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 paragraphs 8-2.3 through 8-2.7 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 Chapter 14. Those materials generally used as aggregate base course materials are listed in paragraphs 8-2.1 through 8-2.10.

8-2.1 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 the requirements of the guide specifications. The percentage of loss shall not exceed 40 when tested in accordance with ASTM C131. The material shall also meet the requirements listed in the guide specification for flat and elongated particles, LL and PI, and magnesium sulfate soundness when tested in accordance with ASTM C88. The gradation requirements for graded crushed aggregates are shown in Table 8-1.

8-2.2 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 C131. The material shall also meet the requirements listed in Unified Facilities Guide Specification (UFGS) 32 11 24 for flat and elongated particles, LL and PI, and magnesium sulfate soundness when tested in accordance with ASTM C88. The gradation requirements are the same as for the 100 CBR material, but with fractured faces relaxed to 50 percent.

8-2.3 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. The requirements for a graded crushed aggregate apply. Only blast-furnace slag will be used. The minimum required unit weight of slag is 1,200 kg/m^3 (75 lb/ft^3).

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

Sieve Designation	Percentage by Weight Passing Square-Mesh Sieve		
	No. 1	No. 2	No. 3
50-mm (2-in)	100	--	--
37.5-mm (1.5-in)	70–100	100	--
25-mm (1-in)	45–80	60–100	100
12.5-mm (0.5-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

8-2.4 **Shell Sand**
Shell sand consists of oyster and clam shells that have been crushed, screened, and blended with sand filler. The ratio of the blend shall be not less than 67 percent shell to 33 percent sand. Refer to local specifications where available.

8-2.5 **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. Table 8-2 shows the recommended gradations.

8-2.6 **Limerock**
Limerock is a fossiliferous limestone of the oolitic type generally located in Florida.

8-2.7 **Shell Rock**
Deposits of hard cemented shells located in North Carolina and South Carolina are known as shell rock, or marine limestone. Refer to local guide specifications where available. The percentage of loss should not exceed 50 when tested in accordance with ASTM C131.

8-2.8 **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 fly ash. See Chapter 9 for a discussion of stabilization.

Table 8-2. Recommended Gradations for Coral

Sieve Designation	Percent Passing
50-mm (2-in)	100
37.5-mm (1.5-in)	70–100
19-mm (0.75-in)	40–90
4.75-mm (No. 4)	25–60
0.425-mm (No. 40)	5–20
0.075-mm (No. 200)	0–10

Note: The percentage of wear (ASTM C131) is not to exceed 50.

8-2.9 Recycled Concrete Aggregate

Recycled concrete aggregate (RCA) shall consist of previously hardened PCC 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 in UFC 3-250-07. The RCA should be tested for alkali-aggregate reactivity (ASR) in accordance with ASTM C1260. If the resulting expansion exceeds 0.08 percent, the RCA should not be used as an aggregate base course. Recycled concrete to be exposed to sulfates in the ground or water must be checked for sulfate resistance. Contact the MAJCOM for guidance.

8-2.10 Recycled Bituminous Concrete

Cold recycled bituminous concrete shall not be used as a base course under flexible pavements. Use of recycled AC under rigid pavements will require approval of the Transportation Systems Center (USACE-TSC) for Army projects, the MAJCOM pavements engineer for Air Force projects, and the Naval Facilities Engineering Service Center for Navy projects.

8-3 AGGREGATE BASE COURSES FOR ARMY AND AIR FORCE RIGID PAVEMENT

8-3.1 Overview

Drainage layers generally serve as aggregate base courses under rigid pavements and must meet the requirements of Chapter 23. A minimum aggregate base course thickness of 102 mm (4 in) is recommend for all subgrades but is required over subgrades that are classified as CH, CL, MH, ML, or OL (ASTM D2487) and 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 (where LL is greater than 40) 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. Exercise care 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, materials such as cohesionless sands and rounded aggregates 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 to 3 m (3 to 10 ft) outside the edge of the pavement to provide a working platform for construction equipment.

8-3.2 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. When the base course must provide drainage, follow the requirements set forth in Chapter 23. Otherwise, all aggregate base courses to be placed beneath airfield rigid pavements will conform to these requirements from UFGS 32 11 16:

- Well-graded, coarse to fine
- Not more than 85 percent passing the 2.0-mm (No. 10) sieve
- Not more than 8 percent passing the 0.075-mm (No. 200) sieve
- Not more than 5 percent passing the 0.075-mm (No. 200) sieve above a drainage layer
- PI not more than 8 percent

Note: Sieve designations are in accordance with ASTM E11.

8-4 AGGREGATE BASE COURSES FOR NAVY AND MARINE CORPS RIGID PAVEMENTS

The main structural support element in a rigid pavement is the PCC 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.

8-4.1 Material Requirements

Suitable materials for aggregate base courses include natural, processed, manufactured, and stabilized materials that meet ASTM D2940. 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.

8-4.2 Gradation

To provide adequate drainage, the base course must contain little or no fines (material that passes the 0.075-mm [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.

8-4.3 Wear Resistance

Aggregates suitable for base course material must have the ability to withstand abrasion and 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 C131) for determining aggregate abrasion resistance. Aggregates suitable for base course use shall have a percentage loss that is less than or equal to 40 percent in the Los Angeles abrasion test.

8-4.4 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.

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 listed in Table 8-3. The aggregate materials should be free from any elongated or soft pieces and dirt. Mix design for lean concrete bases is discussed in Chapter 9.

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 double coat of a wax-based curing compound should be placed on top of all lean concrete base courses.

8-4.5 Recycled Concrete Bases

Recycled PCC can serve as an aggregate for use in a granular base course or in a recycled concrete base. The concrete must be properly crushed and meet the requirements of UFC 3-250-07 and all the requirements of the applicable guide specification.

Table 8-3. Gradations for Lean Concrete Base Materials

Sieve Size (square opening)	Percentage by Weight Passing Sieve		
	A	B	C
50 mm (2 in)	100	--	--
37.5 mm (1.5 in)	--	100	--
25 mm (1 in)	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

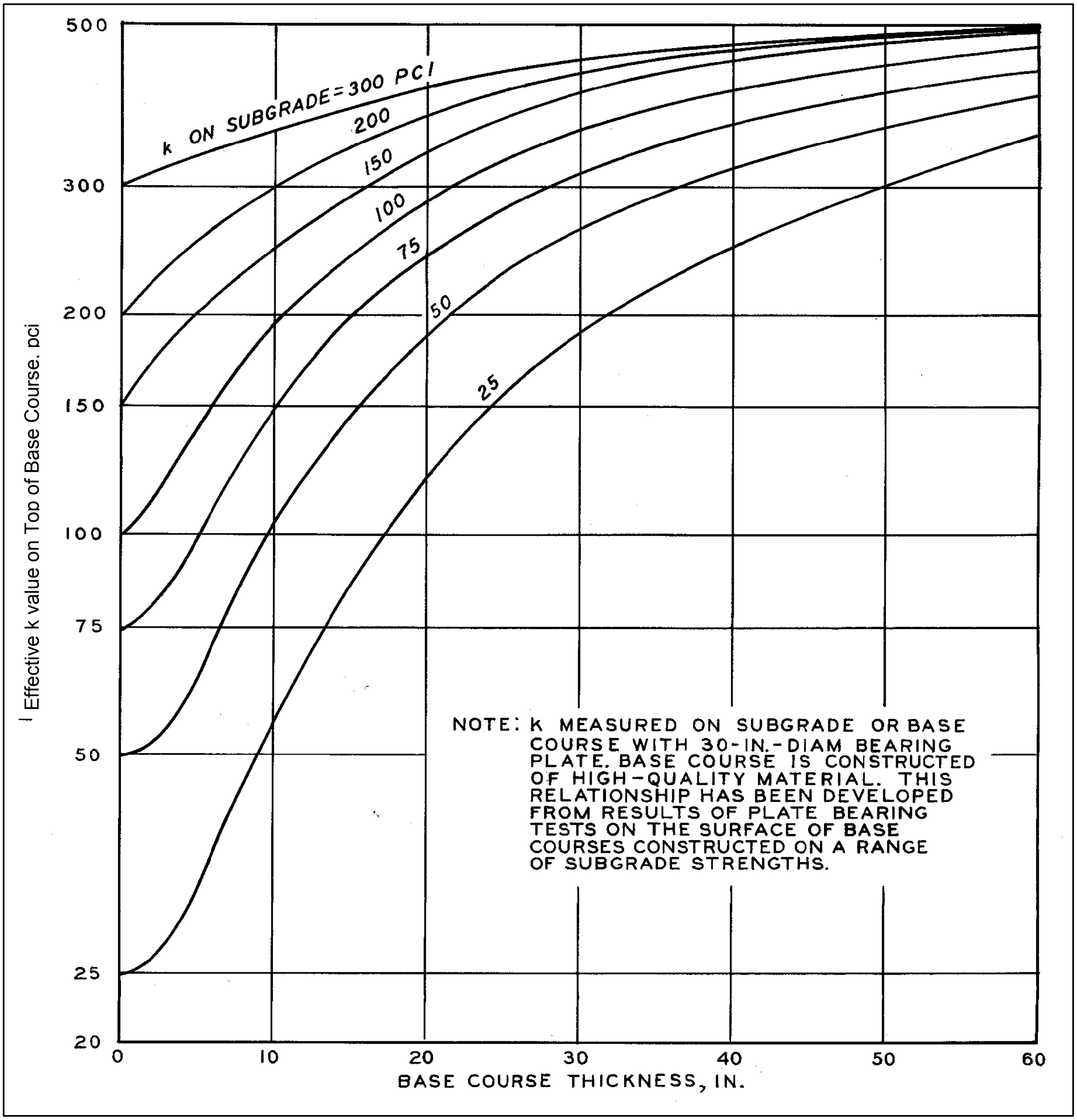
8-4.6 Geotextile Fabrics

Geotextile fabrics may be considered for reinforcing 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 UFC 3-220-01N for design criteria on geotextile fabrics. The use of geotextile fabrics is encouraged to prevent loss of fines from the surrounding soil through subsurface utility lines.

8-5 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. Consider the variations in base course thickness, the 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. The maximum value for the modulus of soil reaction to be used in design is 135 kPa/mm (500 pci).

Figure 8-1. Effect of Base Course Thickness on Modulus of Soil Reaction for Nonfrost Conditions



8-6 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 Table 8-4. These ratings are 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.

Table 8-4. Design CBR Values of Aggregate Base Courses for Flexible Pavements

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

8-7 MINIMUM THICKNESS REQUIREMENTS FOR FLEXIBLE PAVEMENTS

The minimum allowable thicknesses for aggregate base courses in flexible pavements are listed in Table 8-5 for Army airfields, Table 8-6 for Navy and Marine Corps airfields, and Table 8-7 for Air Force airfields. These thicknesses have been established so that the required subbase CBR will always be 50 or less.

Table 8-5. Minimum Surface and Aggregate Base Course Thickness Requirements for Army Flexible Pavement Airfields

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

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

Table 8-6. 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 8-7. Minimum Surface and Aggregate Base Course Thickness Requirements for Air Force Flexible Pavement Airfields

Airfield Type	Traffic Areas	100 CBR Base, in			80 CBR Base, in [1,2,3,4]		
		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
Short Field	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 in.

[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 aggregates are permitted in Type D traffic areas.

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

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

Conversion Factor: 25.4 Millimeters = 1 inch.

[4]See paragraph 10-7.1.1 for overrun surfacing requirements.

8-8 MINIMUM THICKNESS REQUIREMENTS FOR RIGID PAVEMENTS

8-8.1 Army and Air Force

The minimum thickness of aggregate base courses under rigid pavements will be 102 mm (4.0 in) over CH, CL, MH, ML, or OH subgrades or that required to meet minimum thicknesses for drainage layers as shown in Chapter 23.

8-8.2 Navy and Marine Corps

The minimum thickness requirements for aggregate base courses are listed in Table 8-8. 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 8-8. 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).	

8-9 COMPACTION AND PROOF ROLLING REQUIREMENTS FOR FLEXIBLE PAVEMENTS

The aggregate base course will be compacted to 100 percent of ASTM D1557 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 in this paragraph and its subparagraphs. Open-graded and rapid-draining layers will not be proof rolled. The layer immediately underlying 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 kg (30,000 lb) and inflated to 862 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. Chapter 23 presents special proof rolling and compaction requirements for drainage layers.

8-9.1 Air Force Bases

Proof roll the top of the subbase and each layer of the base course of Type A traffic areas and the center 23 m minimum (75 ft minimum) of heavy, modified heavy, and medium-load runways with 30 coverages.

8-9.2 Navy and Marine Corps Airfields

Proof roll the top of the completed aggregate base course on the center 12 m (40 ft) of taxiways and on the center 30.5 m (100 ft) of runways with eight coverages. To all other paved areas exclusive of runway overrun and blast protection areas, apply four coverages.

8-9.3 Army Airfields

On Class IV airfields with runways greater than 1,525 m (5,000 ft), proof roll the top of the subbase and each layer of crushed aggregate base course in Type A traffic areas and the center 23 m (75 ft) of runways with 30 coverages.

8-10 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:

- 95 percent of ASTM D1557 maximum density for aggregate base courses less than 254 mm (10 in) thick
- 100 percent of ASTM D1557 maximum density in the top 152 mm (6 in) and 95 percent of ASTM D1557 maximum density for the remaining thickness for aggregate base courses 254 mm (10 in) or more in thickness

8-11 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 D1557 and D558, respectively. Compact AC base courses to 97 percent of the maximum density as determined from the Marshall mix design method.

CHAPTER 9

PAVEMENT MATERIALS

9-1 OVERVIEW

This chapter provides the designer with an overview of pavement materials that might be used in military airfield pavements. This overview includes soil and aggregate stabilization, AC, PCC, and recycled materials. More comprehensive and detailed descriptions, policy, and guidance on uses and limitations, testing requirements, suitable materials, mixture proportioning, and construction are located in UFC 3-250-11 for stabilization, UFC 3-250-03 for AC, and UFC 3-250-04 for PCC. 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 USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or the 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 in this paragraph should be consulted concerning each service’s specific limitations and requirements for these materials and for preparing individual project specifications.

9-2 STABILIZATION

For various reasons, existing soils or aggregates may not be suitable for use in airfield construction (for example, because of poor grading, low strength, excessive plasticity, or the 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. UFC 3-250-11 provides official guidance on the use of lime, portland cement, lime-fly ash, and bituminous materials for stabilization. Consult USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or the Naval Facilities Engineering Service Center for assistance on use of other stabilizers and conditions not covered in the existing guidance.

9-2.1 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, creation of a construction platform to ease and speed construction operations, and reduction of effects of adverse weather during construction. In addition, stabilization allows use of an economical local material that fails conventional specifications in lieu of importing more expensive materials from elsewhere.

9-2.2 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 degrees C (73 degrees F) for portland cement and after 28 days of curing at 22.8 degrees C (73 degrees F) for lime, slag, and combinations with pozzolanic materials (for example, lime-fly ash mixtures). In addition to requirements for strength, specific requirements for durability and material properties must be met, and the layer must have a minimum thickness of 152 mm (6 in). Even if a material fails to qualify for the reduced pavement thickness requirements, stabilization may prove desirable for some of the other reasons noted in paragraph 9-2.1. If stabilization results in granular layers sandwiched between relatively impervious layers (for example, a granular base course between an AC 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 USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or the Naval Facilities Engineering Service Center.

9-2.3 Terminology

The term “stabilization” as used in this chapter will include 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 by terms such as “soil-cement,” “lean concrete base,” and “econocrete.” UFC 3-250-11 differentiates between soil stabilization and soil modification where the latter results only 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.

9-2.4 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. Second, many stabilizers (for example, 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. As a result, 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 USACE Cold Regions Research and Engineering Laboratory (CRREL), 72 Lyme Road, Hanover, NH 03755-1290, http://www.crrel.usace.army.mil/.

9-2.5 Combinations of Stabilizers

Under some circumstances, it may be desirable to use combinations of stabilizers to take advantage of each stabilizer’s characteristics (for example, using a combination of lime and then portland cement because the lime will improve a plastic clay's workability and the portland cement provides for more rapid strength gain than from the slower pozzolanic reactions of lime alone).

9-2.6 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 for 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 more difficult to achieve good distribution and mixing of the stabilizer with in situ mixing techniques than with plant mixing. Consequently, stabilizer contents are sometimes increased 2 to 1 percent over the laboratory determined design stabilizer content to account for the uncertainties of in situ mixing.

9-2.7 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 for total layer thickness exceeding 152 mm (6 in), individual lift thicknesses should be restricted to a maximum of 152 mm (6 in) 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. UFC 3-250-11 provides more comprehensive guidance on the requirements for laboratory compaction and the 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 (that is, increase maximum density and decrease optimum moisture content) or down and to the right (that is, decrease maximum density and increase optimum moisture content). On the other hand, increasing lime content decreases the laboratory maximum density and increases the optimum moisture content for compaction.

If field stabilizer contents are increased for in situ mixing as noted in paragraph 9-2.6, 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 variations from distribution and mixing of the stabilizer, fairly assessing the contractor's compaction efforts may become difficult. In circumstances in which 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. Consult USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or the Naval Facilities Engineering Service Center for assistance with difficult cases.

9-2.8 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.

9-2.9 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 or clay mineralogy or the presence of organic or some iron compounds in the soil may totally change or inhibit the chemical reactions. 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.

9-2.10 Lime Stabilization

Hydrated lime ($Ca\,(OH)_2$), quicklime (CaO), or the dolomitic variants of these limes are suitable for lime stabilization of soils. The requirements for the limes for soil stabilization are contained in ASTM C977. Calcium carbonate ($CaCO_3$) is often sold under names such as agricultural lime and is not suitable for soil stabilization.

9-2.10.1 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, and manipulate in other ways. 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. These 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. After 28-day cures at 22.8 degrees C (73 degrees F), soil compressive strength gain 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 0.69 to 3.45 MPa (100 to 500 psi).

9-2.10.2 **Uses**

Lime added to soil can rapidly dry the soil. Lime coarsens the particle texture, which often makes the soil easier to work. In addition, lime reduces the soil's plasticity, making the soil 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 degrees C (73 degrees F), can significantly improve the 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 lime very useful as a construction expedient and soil improvement additive for difficult plastic clay soils (for example, its drying characteristics, coarser texture, reduced plasticity and water susceptibility, creation of a construction platform, and reduced shrink-swell behavior) rather than as an additive for structural strength alone.

9-2.10.3 **Durability**

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

9-2.10.3.1 **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—for example, 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 D560) limits in UFC 3-250-11 may be checked if strength when exposed to soaking or wetting and drying is a critical design parameter.

9-2.10.3.2 **Seasonal Frost Exposure**

Lime-stabilized materials generally expand and lose strength when exposed to freezing and thawing. As cycles of freezing and thawing increase, the strength of the lime-stabilized material decreases progressively. Generally, the first winter is the critical exposure because extended curing in subsequent seasons will provide additional strength, and some data suggest that these materials may heal autogenously under favorable curing temperatures. UFC 3-250-11 contains specific testing criteria and limits based on ASTM D560 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 (for example, perhaps 30 days) prior to the onset of freezing weather, which shortens the construction season for some areas. Alternatively, lime-stabilized material must be protected from freezing (for example, by placement of overlying pavement layers) and the temperature maintained high enough to allow pozzolanic reactions to occur. Assistance on problems with lime-stabilized materials under seasonal frost exposure is available from USACE CRREL, http://www.crrel.usace.army.mil/.

9-2.10.3.3 **Leaching**
Limited evidence suggests that the benefits of lime stabilization may be reduced by leaching over time in soils stabilized with low levels of lime. The problem appears to be relatively rare and is generally associated with low levels of lime stabilization (for example, 3 percent and less). In general, this should not be an issue for lime stabilization levels for airfield pavements because their strength and durability requirements would normally require lime contents above those where leaching has been reported as a problem.

9-2.10.3.4 **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 avoid the problem. Reported problems have been with highly weathered materials in Africa that were poorly compacted and cured.

9-2.10.3.5 **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, consult USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or the Naval Facilities Engineering Service Center for up-to-date guidance on this difficult issue.

9-2.10.4 **Suitable Soils**
Clayey soils with a PI 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.

9-2.11 **Portland Cement Stabilization**
Type I portland cement and, more rarely, Types II, I/II, and III meeting the requirements of ASTM C150 may be mixed with soils or aggregates to provide a cohesive cemented material often referred to by terms such as “soil-cement,” “econocrete,” and “lean concrete base.”

9-2.11.1 **Mechanisms**
When mixed with water, portland cement develops cementing compounds that bind the soil and aggregate particles together. Unlike with 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 effects are minor compared with the dominant formation of the conventional portland cement hydration compounds that serve to bind the particles together.

9-2.11.2 Uses

Portland cement stabilization can provide a material with compressive strengths from a few megapascals (a few hundred pounds per square inch) to well over ten megapascals (several thousand pounds per square inch), depending on the amount of stabilizer and the soil properties. These higher-strength stabilized materials are often referred to by names such as "econocrete" and "lean concrete," with cement contents in the range of 134 to 223 kg/m^3 (225 to 375 pounds per cubic yard [lb/yd^3]). Such high cement content and high-quality stabilized mixes are usually proportioned and placed with the same techniques as those for 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 that can reflect up through surfacing layers. This is usually a severe problem with cement-stabilized bases under AC 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. Often a double application of curing compound is sprayed on cement-stabilized bases to reduce the chance of reflective cracking in overlying PCC surfaces in rigid pavements. Portland cement stabilization is used most often 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. Portland cement is 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.

9-2.11.3 Durability

9-2.11.3.1 Seasonal Frost Exposure

Because cycles of freezing and thawing can damage cement-stabilized materials, UFC 3-250-11 contains specific testing criteria and limits based on ASTM D560 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. Assistance on problems with cement-stabilized materials under seasonal frost exposure is available from USACE CRREL, http://www.crrel.usace.army.mil/.

9-2.11.3.2 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.

9-2.11.3.3 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 (Types II and V) will not prevent sulfate attack. If cement stabilization is contemplated where sulfates are present, consult USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or the Naval Facilities Engineering Service Center for up-to-date guidance on this issue.

9-2.11.3.4 **Suitable Soils**

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

9-2.12 **Pozzolan and Slag Stabilization**

ASTM C618 classifies pozzolans as Type N (natural pozzolans), Type C (high-lime-content fly ash, a by-product 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 that can be ground to produce a slag cement. ASTM C989 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. UFC 3-250-11 provides guidance on fly ash (the most commonly available pozzolan) stabilization. Slag is not addressed in the manual, and USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or the Naval Facilities Engineering Service Center should be consulted for current guidance on use of this material in military construction.

9-2.12.1 **Mechanisms**

Pozzolans and ground granulated blast-furnace (GGBF) slag react with hydroxides to form cementitious bonds. Lime, or occasionally portland cement, is 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, the addition of 0.5 to 1.5 percent portland cement can be used as an activator for the fly ash and as a contributor to early-age strength.

9-2.12.2 **Uses**

Pozzolans and slags gain strength more slowly than portland cement but are more economical, have less shrinkage and shrinkage cracking, and have longer working times than portland cement. Typical fly ash-stabilized mixes will use 2.5 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 contain 8 to 20 percent GGBF slag mixed with 1 percent lime.

9-2.12.3 **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 degrees C (40 degrees 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 UFC 3-250-11 requires laboratory freeze-thaw testing after 28 days of curing. Assistance on problems with lime-pozzolan-stabilized or slag-stabilized materials under seasonal frost exposure is available from USACE CRREL, http://www.crrel.usace.army.mil/.

9-2.12.4 **Suitable Soils**

Granular materials are effectively stabilized with pozzolanic and slag stabilizers. Because of their relative economy compared to portland cement, they are particularly effective with poorly graded materials with which they can function as a filler more economically than the more expensive portland cement. Many clays are naturally pozzolanic, so there is little value in adding another pozzolanic material such as fly ash. These clays are usually handled best 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.

9-2.13 **Bituminous Stabilization**

Asphalt cement (American Association of State Highway and Transportation Officials [AASHTO] M 320, ASTM D3381, or ASTM D946), emulsified asphalt (asphalt emulsified with water, ASTM D977 and D2397), or cutback asphalt (asphalt dissolved in a solvent, ASTM D2026, D2027, and D2028) may be mixed with a soil or aggregate to provide a water-resistant, cohesive stabilized material. 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. UFC 3-250-11 provides detailed guidance on bituminous stabilization requirements and procedures. The mix design for bituminous-stabilized materials in a military airfield subbase or base course will be a conventional Marshall mix design in accordance with UFC 3-250-03.

9-2.13.1 **Mechanisms**

Asphalt coats the soil and aggregate particles being stabilized and binds them 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 (for example, an anionic emulsion [negatively charged droplets] used with limestone aggregate [positive charge]).

9-2.13.2 **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, use the highest possible viscosity liquid asphalt that can be handled in the field and mixed with the soil or aggregate being stabilized.

9-2.13.3 **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. Adding 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.

9-2.13.4 **Suitable Soils**

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

9-2.14 **Nontraditional Stabilizers**

A wide variety of special, and often proprietary, stabilizers are marketed actively. 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 performed to the manufacturer's claims, and no proprietary stabilizer should be used on a military airfield without first evaluating the stabilizer in the laboratory and in independent field trials. Consult USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or the Naval Facilities Engineering Service Center prior to using any of these nontraditional stabilizers.

9-2.14.1 **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.

9-2.14.2 **Evaluation**

The claimed benefit of any stabilizer should be evaluated quantitatively to determine the cost-effectiveness of including the material on a specific project. It is important to identify what soil property is being changed by the stabilizer and to 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 electrolytes’ effectiveness. Experience with some of these materials has found that often the necessary amount of the stabilizer is higher than the manufacturer’s suggested dosage.

9-3 **PCC**

PCC 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.

9-3.1 **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 reinforcement is used only where its added value balances these negative factors. Details on various reinforced pavements and their design are provided in paragraphs 9-3.1.1 through 9-3.1.4 and in subsequent chapters.

9-3.1.1 **Conventional Reinforcing Steel**

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 (for example, odd-shaped slabs, extra-large slabs). 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 is added to resist deterioration in cracks developed from environmental stresses. The steel is continuous and the pavement has no joints. Continuously reinforced concrete pavements provide a joint-free, smooth pavement, but repairs to these pavements are often difficult.

9-3.1.2 **Fiber Reinforcing Products**

Fiber reinforcing products are actively marketed. Steel fibers can reduce the required pavement thickness significantly, but there are concerns that with current finishing techniques, the fibers pose a foreign object damage (FOD) threat on military airfields.

9-3.1.3 **Plastic Fibers**

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 in paragraph 9-3.6.3, these fibers have been found useful in concrete exposed to exhaust from vertical and short take-off aircraft like the Harrier.

9-3.1.4 **Prestressed Pavements**

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 are more sophisticated than the design and construction of conventional pavement, and prestressing construction technology has been evolving and is more cost-effective today than in past years.

9-3.2 **Constituents**

PCC is composed of portland cement, aggregates, water, and various additives. Portland cement must meet the requirements of ASTM C150, and the various types of portland cement are described in Table 9-1. Type I cement is 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 be readily available and may be expensive. The addition of fly ash is very common in modern concretes, and GGBF slags are beginning to be used more often as additives. Both additives 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, and protection against alkali-aggregate reaction.

9-3.2.1 **Aggregate Quality Requirements**

Aggregate quality requirements in UFC 3-250-04 for military airfield pavements are appreciably tighter than those used in ASTM C33, 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 restrictions were adopted by the military in the 1950s after severe problems with popouts developed on new airfield pavements at Selfridge Air Force Base.

Table 9-1. Types of Portland Cement

Type of Cement	Characteristics
I	Ordinary
II	Moderate sulfate resistant
I/II	Meets ASTM C150 for both Types I and II cements
III	High, early strength
IV	Low heat of hydration
V	Sulfate resistant for more severe sulfate exposure conditions

9-3.2.2 **Air Entrainment**

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 a waiver is first obtained from USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or the Naval Facilities Engineering Service Center. Air entrainment causes some loss in strength, but it also enhances workability. Proper mixture proportioning can use this enhanced workability to reduce the water-cement ratio and thereby negate the strength loss from air entrainment. Because 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, 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 by names such as Type IA and Type IIA are not suitable for this use because they do not provide the flexibility of adjusting admixture dosage to reflect changing mixture and site conditions.

9-3.2.3 **Other Admixtures**

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 admixtures is generally at the discretion of the engineer proportioning the mixture for a specific project or the contractor who must deal with a specific site problem. The engineer responsible for the mixture proportioning is responsible for selecting admixtures and concrete materials that are compatible and cause no adverse interactions. If the contractor elects to use an admixture (for example, a retarder because of lengthy haul times), then the contractor is responsible for selecting an admixture that is compatible with the concrete mixture and that has no adverse effect on the fresh or hardened concrete mixture.

9-3.3 **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 use of a well-graded concrete aggregate for all airfield pavements, with specific limitations depending on anticipated placement methods (that is, slipform, with form-riding equipment, or by hand). Specific requirements and details are contained in Air Force ETL 97-5 and will be conformed to for all Air Force pavements unless a waiver is obtained from the Air Force MAJCOM pavements engineer.

9-3.4 **Durability**

Properly proportioned and placed, PCC 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 (a 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 UFC 3-250-04. 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 and improves strength when compared to using higher water-to-cement ratios in the concrete mixture.

9-3.5 Design Strength

9-3.5.1 Test Method

Military airfield pavements are designed on the basis of the third-point, flexural beam test (ASTM C78). 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 (for example, 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. Therefore, all military airfield design will be based on the ASTM C78 flexural strength.

9-3.5.2 Correlations

There are no unique relationships between different concrete strength tests (for example, third-point flexural beam, center-point flexural beam, compressive, splitting tensile), and all such tests are indices of strength rather than of inherent material property. Many published relationships allow estimation of one strength test result as a function of another test (for example, estimating third-point flexural strength from the concrete compressive strength); however, the variation of the data on which such relations are based is quite large and the results too inaccurate to allow the use of such relations 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.

9-3.5.3 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 PCC. 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 accurately. 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 C78 flexural strengths of 4.14 to 4.48 MPa (600 to 650 psi) are readily achievable with most local materials, and the designer should not use higher than 4.83 MPa (700 psi) unless approved by USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or the Naval Facilities Engineering Service Center.

9-3.6 **Special Airfield Exposure Conditions**

Properly proportioned, placed, and cured PCC requires no surface sealers, coatings, or treatments to withstand normal military aircraft operations such as start-up, warm-up, taxiing, takeoff, and landing.

9-3.6.1 **Heat Effects on PCC**

Rapid heating of moist concrete can vaporize water in the concrete capillaries and cause explosive spalling. As the concrete temperature begins to rise above approximately 149 degrees C (300 degrees 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 (for example, 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. If, however, the concrete temperature will reach 204 degrees C (400 degrees F), conventional concrete probably will not be sufficient, and thermal cycling at lower temperatures can cause damage. Consult USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or the Naval Facilities Engineering Service Center 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.

9-3.6.2 **Power Check Pads and Similar Facilities**
If a 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 with 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 areas such as parking ramps and old taxiways 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.

9-3.6.3 **Pavements Exposed to Vertical or 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 the joint strike fighter. The Naval Facilities Engineering Service Center has conducted extensive research in support of deployment of the Harrier in the Marine Corps. The Naval Facilities Engineering Service Center found that reinforced conventional concrete made with diabase aggregate has provided good performance in the field for up to 15 years. Recent studies have found that improved performance could be achieved with normal portland or specialty cement concrete with lightweight aggregate and polypropylene and nylon fibers. Contact the Naval Facilities Engineering Service Center, 1100 23rd St, Port Hueneme, CA 93043-4370, https://portal.navfac.navy.mil/, for current guidance and research results in this area.

9-3.6.4 **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 degrees C (350 degrees F). This leads to scaling and spalling in the limited area around the exhaust impingement area. Studies by the Naval Facilities Engineering Service Center, the Air Force Research Laboratory, and the U.S. Army Engineer Research and Development Center have identified two mechanisms contributing to this damage. Repeated heating and cooling leads 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 PCC 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. Air Force ETL 02-7 summarizes the different strategies used to prevent APU-caused pavement damage and notes that the most successful mitigation method has been the use of a thin magnesium phosphate cement (MPC) overlay. Contact USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or the Naval Facilities Engineering Service Center for guidance when designing parking areas for these aircraft.

9-3.7 Specification and Construction

It is crucial that proper material and construction specifications be developed to accompany the thickness design and geometric design and detailing. Numerous problems with military concrete airfield pavements in recent decades have resulted from 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 to sulfates. The result has been unsatisfactory performance, increased maintenance, and in some cases, dissatisfied users. The designer should be certain to consult current versions of each service's guide specification and UFC 3-250-04 for assistance in preparing project specifications.

9-4 AC

AC is the normal surfacing for flexible pavements. Unlike PCC, it normally functions as a relatively thin wearing surface and is not the major structural element of the pavement. AC on airfields is exposed to much more severe loads than on highways and is quite different from highway AC mixes. Substitution of AC highway aggregates and mixes for AC airfield mixes is not acceptable and is a major engineering blunder. The requirements of UFC 3-250-03 will provide an AC that will stand up to the loads of modern military aircraft in all environmental conditions.

9-4.1 Constituents

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

9-4.1.1 Binder

Asphalt cement from the distillation of petroleum is the most common binder in AC. Liquid asphalts created by 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 also seldom used as binder in airfield pavements. Some natural asphalts are used occasionally as binder material for AC.

9-4.1.1.1 Characteristics

Asphalt is a complex hydrocarbon product with composition and properties that 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 its 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.

9-4.1.1.2 Specification

The asphalt binder should be specified in accordance with the Strategic Highway Research Program (SHRP) performance grading system (AASHTO M 320). 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 AC pavements.

UFC 3-250-03 provides guidance on selecting performance grade (PG) asphalt cement for different project locations. SHRP PG grading is not used universally worldwide; therefore, alternate specification methods based on viscosity (ASTM D3381) and penetration (ASTM D946) 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 consult USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or the Naval Facilities Engineering Service Center for up-to-date guidance.

9-4.1.2 Aggregates

The deformation resistance of AC 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 AC that will resist the high tire pressure of modern military aircraft. Limiting natural sand with rounded particles to no more than 15 percent of the total aggregate by weight is an important requirement for AC for military airfields. In AC with higher natural sand contents, rutting under military aircraft has been a repeated problem. UFC 3-250-03 provides detailed guidance on aggregate requirements.

9-4.2 Mix Design

The mix design of AC 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.

9-4.2.1 Military Requirements

AC for military airfields will be designed based on the 75-blow Marshall mix design method. Details are provided in UFC 3-250-03 and the Asphalt Institute's MS-2 procedures using a handheld compaction hammer.

9-4.2.2 SHRP Mix Design

The SHRP produced an AC mix design procedure and recommended aggregate gradations that are being used widely by state departments of transportation. These gradations and mix design procedures were developed for highway use and have not been evaluated for airfield use; therefore, 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 USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or the Naval Facilities Engineering Service Center is needed before these new guidelines are used on military airfields.

9-4.3 Special Asphalt Mixes

Porous friction courses are relatively thin (approximately 25- to 38-mm [1- to 1.5-in]) surface layers of a special open-graded AC 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 used widely by the Air Force in the 1970s and 1980s, but their use has declined as improved grooving of conventional AC mixes provides similar skid resistance without the disadvantages of the porous friction courses.

9-4.3.1 Stone Mastic Asphalt (SMA)

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.

9-4.3.2 Fuel-Resistant Sealers

Thin applications of fuel-resistant sealers to AC 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 environmentally induced cracking that limits their effectiveness. Often this cracking occurs at early ages. Polymer modification of some of these products has helped but not solved the cracking problem.

9-4.3.3 Slurry Seals

Slurry seals are thin applications of emulsified asphalt and sand to oxidized AC 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.

9-4.3.4 Microtexturing

Highly polymerized proprietary systems known as “microtexturing” that use thin surface applications of a binder and aggregate to oxidized AC surfaces have shown promise but are still in the evaluation stage.

9-4.3.5 Rejuvenators

Rejuvenators are composed of lighter-end hydrocarbons that, when sprayed on an oxidized AC surface, soften the binder and counter some of the aging effect. These materials have provided mixed results in practice and invariably lower the skid resistance of the pavement. Consequently, they are not allowed on military airfields.

9-4.4 Durability

9-4.4.1 Aging and Oxidation

Asphalt oxidizes and stiffens over time, which leads to a loss of cohesion and flexibility. This loss 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 yet available.

9-4.4.2 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 AC will develop cracking. The SHRP performance 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.

9-4.4.3 **Fuel Spillage**

Fuels, oils, hydraulic fluids, and similar petroleum-based liquids are solvents for the asphalt binder; therefore, AC should not be used where it will be exposed to such materials. Coal tar-based fuel-resistant sealers have only a temporary life expectancy before cracking reduces their effectiveness.

9-4.4.4 **Stripping**

Several mechanisms contribute to moisture damage to AC, and they 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 (for example, siliceous aggregates), and repeated loads. Lime and proprietary liquid antistrip agents can combat the problem. Also, selecting proper aggregates and ensuring drainage to reduce the AC’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.

9-4.5 **Construction**

Production and placement of high-quality AC suitable for military airfields is a demanding and skillful operation. Proper mixing and delivery of the AC, 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.

9-5 **RECYCLED MATERIALS**

As a general policy, the military encourages use of recycled materials in airfield pavements, but using recycled materials should not be done at the expense of the quality or performance of the final pavement. Reclaimed hot-mix asphalt (HMA) millings may be used as a hot-mix aggregate (in recycled asphalt pavement [RAP] mixtures), as a flexible pavement subbase, or as a rigid pavement base. Asphalt millings should not be used in a HMA surface course or as a flexible pavement base course. Recycled concrete aggregate (RCA) may be used as a flexible pavement base course, subbase, or rigid pavement base course provided that the subgrade soil is free of sulfates and the RCA is not alkali-silica reactive. More extensive guidance and specific limitations for each service are located in UFCs 3-250-11, 3-250-03, 3-250-04, 3-250-07, and each service’s guide specifications.

CHAPTER 10

FLEXIBLE PAVEMENT DESIGN: CBR METHOD

10-1 REQUIREMENTS

Flexible pavement designs must provide these characteristics:

- Sufficient compaction of the subgrade and each layer during construction to prevent objectionable settlement under traffic
- 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
- Adequate subsurface drainage control to reduce to acceptable limits the effects of frost heave or permafrost degradation where frost conditions are a factor
- A stable, weather-resistant, wear-resistant, waterproof pavement

Attention must also be given to providing adequate friction characteristics.

10-2 BASIS FOR DESIGN

The thickness design procedures included in this chapter for conventional flexible pavement construction are based on CBR design methods with a failure criterion of a 25-mm (1-in) rut. 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 layered elastic method is covered in Chapter 11.

10-3 THICKNESS DESIGN CURVES

Figures 10-1 through 10-24 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 a given strength for a given gross aircraft weight and aircraft passes.

10-4 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 10-24) to determine the thickness required above the layer to prevent detrimental shear deformation in that layer during traffic. These are the specific steps of the procedure:

(1) Determine the design CBR of the subgrade.

(2) Determine the total thickness above the subgrade.

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

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

(3) Determine the design CBR of the subbase.

(4) Determine the thickness of material required above the subbase by entering the appropriate design curve with the design subbase CBR and using these procedures to read the required thickness.

(5) Determine 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, increase the minimum thicknesses to the actual thickness required.

(6) Subtract the thickness of the surface and base from the total thickness required above the subgrade to obtain the required thickness of the subbase. If the thickness of the subbase is less than 152 mm (6 in), consider increasing the thickness of the base course.

10-5 ADDITIONAL CONSIDERATIONS FOR THICKNESS DESIGN

10-5.1 CBR Values Less than Three

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

10-5.2 Frost Areas

Pavement sections in frost areas must be designed and constructed with non-frost-susceptible (NFS) materials of such depth to prevent destructive frost penetration into underlying susceptible materials. Design for frost areas in accordance with Chapter 20.

10-5.3 Rapid-Draining or Open-Graded Material

The thickness of rapid-draining or open-graded material is determined from Chapter 23 and is substituted for an equivalent thickness of base or subbase according to design requirements.

10-5.4 Expansive Subgrade

Ensure that the moisture condition of expansive subgrade is controlled and that adequate overburden is provided.

10-5.5 Limited Subgrade Compaction

Where subgrade compaction must be limited for special conditions, pavement thickness must be determined based on the reduced density and CBR of the prepared subgrade.

10-6 STABILIZED PAVEMENT SECTIONS

Stabilized layers may be incorporated in the pavement sections to make use of locally available materials that 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 stabilized courses must be in accordance with the requirements of Chapter 9. For the Air Force and Army, see the requirements in UFC 3-250-11. For Air Force design, a stabilized subbase may not be used without a stabilized base unless the base course has adequate drainage. Approval from the appropriate Air Force MAJCOM is required to use stabilized components.

10-6.1 Army and Air Force Design: Equivalency Factors

The use of stabilized soil layers within a flexible pavement provides the opportunity to reduce the overall thickness of the 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 mm (1 in) of stabilized material. Equivalency factors will be determined for Army and Air Force designs from Table 10-1. Equivalency factors cannot be applied to layers less than the minimum required. For example, the computer design indicates that a surface course of AC should be 76 mm (3 in) and the minimum is 102 mm (4 in). One cannot apply the equivalency factor to the extra inch of thickness not required by the design but required by the minimum requirements.

For an example of how to use equivalency factors, consider that a given pavement design requires 4 inches of HMA surface course on 6 inches of 100 CBR base course and 16 inches of subbase course. The contractor may choose to replace the subbase course material with 100 CBR base material using an equivalency factor of 2 as specified in Table 10-1 for unbound crushed stone (100 CBR Material). The pavement section would then be reduced to 4 inches of HMA surface course on 6 inches of 100 CBR base course on 8 more inches of 100 CBR base (16 divided by 2). Using the equivalency factor of 2 in this case reduced the paving section by 8 inches. These equivalency factors may affect the assumptions used for frost design, and any reductions in thickness should take the frost design requirements into consideration.

10-6.2 Navy and Marine Corps Design: Thickness Reduction Factors

Stabilized base course and subbase course materials meeting the requirements for strength and durability in Chapter 9 may be substituted for unstabilized materials. These are the procedures for pavement design with stabilized layers:

(1) Design a conventional pavement section using the guidelines in this chapter.

(2) Convert the base or subbase courses into equivalent thicknesses of stabilized materials by using the equivalency factors shown in Table 10-2.

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

Table 10-1. 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[2,3]		
Material stabilized to 5.17 mPa (750 psi)	1.15[2,3]	2.30[2,3]
Material Stabilized to 1.72 mPa (250 psi)	--[1]	1.70[2,3]
Material Stabilized to <1.04 mPa (250 psi)	--[1]	--[4]
Lime-Stabilized[3]		
Material stabilized to 5.17 mPa (750 psi)	1.15[3]	2.30[3]
Material Stabilized to 1.72 mPa (250 psi)	--[1]	1.70[3]
Material Stabilized to <1.04 mPa (250 psi)	--[1]	--[4]
Lime-Cement-Fly Ash Stabilized[2,3]		
Material stabilized to 5.17 mPa (750 psi)	1.15[2,3]	2.30[2,3]
Material Stabilized to 1.72 mPa (250 psi)	--[1]	1.70[2,3]
Material Stabilized to <1.04 mPa (250 psi)	--[1]	--[4]
Unbound Crushed Stone Aggregates		
Graded Crushed Aggregate Base Course (100 CBR Material)	1.00	2.00
Aggregate Base Course (80 CBR Material)	1.00	1.00
Unbound Aggregate Subbase	–[1]	1.00

[1] Not used as base course.

[2] For Air Force bases and Army installations, cement is limited to 4 percent by weight or less

[3] Materials must meet the strength, gradation, and other requirements in UFC 3-250-11.

[4] To be used for subgrade only.

10-6.3 **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 USACE-TSC for Army airfields and the appropriate MAJCOM for Air Force airfields. These design procedures are explained in Chapter 11.

Table 10-2. Equivalency Factors for Navy and Marine Corps Pavements

Stabilized Material	Equivalency Factors
1 unit of lime-stabilized subbase	1.2 units of unstabilized subbase course
1 unit of cement-stabilized subbase	1.2 units of unstabilized subbase course
1 unit of cement-stabilized base	1.5 units of unstabilized base course
1 unit of bituminous base	1.5 units of unstabilized base course

10-7 SPECIAL AREAS

Areas such as overrun areas, airfield and heliport shoulders, blast areas, and reduced load areas require special treatment. This section details the requirements for each service: Air Force (section 10-7.1), Army (section 10-7.1.2), and Navy and Marine Corps (section 10-7.3).

10-7.1 Air Force Bases

10-7.1.1 Overrun Areas

Overrun areas will be paved for the full width of the runway, exclusive of shoulders, for a length of 305 m (1,000 ft) on each end of heavy, modified heavy, medium, light, and auxiliary runways, and for 90 m (300 ft) on each end of assault landing zone runways. Surface the overrun areas with double-bituminous surface treatment except for the first 45 m (150 ft) abutting the runway pavement end, which will have a wearing surface of 50 mm (2 in) of dense-graded AC. That portion of the overrun used to certify barriers and support snow removal equipment must be surfaced with dense-graded AC. Design the pavement thickness in accordance with the appropriate figures in this chapter, except that the minimum base course thickness will be 152 mm (6 in). The strength of the assault landing zone overrun shall be equal to the strength of the runway. Minimum base course CBR values are shown in Table 10-3.

Table 10-3. Minimum Base Course CBR Values

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

10-7.1.2 Paved Shoulders

Paved shoulders will be provided adjacent to runways, taxiways, aprons, and pads where authorized by Air Force handbook (AFH) 32-1084. Design the paved shoulders in accordance with Table 3-1 and Table 8-4. The remaining shoulder width will be constructed of existing soils, select soils, or stabilized soils with a turf cover.

10-7.2 Army Airfields

10-7.2.1 Paved Shoulders

Paved shoulders should be provided for airfields and heliport or helipad facilities as designated in UFC 3-260-01. Design paved shoulders in accordance with Table 2-1. The remaining unpaved 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.

10-7.2.2 Paved Overruns

Paved overruns should be provided for runways and landing lanes in accordance with UFC 3-260-01. Design the paved 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 to make sure that it is adequate for supporting crash rescue vehicles. Use a 50-mm (2-in) dense-graded AC wearing surface on a minimum 152-mm (6-in) base consisting of 80 CBR material or better. The remaining overrun area will be constructed of double-bituminous surface treatment on a 102-mm (4-in) base course of 80 CBR material or better.

10-7.3 Navy and Marine Corps Airfields

10-7.3.1 Overrun Areas

Pave the overrun areas for a width of 60 m (200 ft) or the width of the runway if less than 60 m (200 ft), centered on the runway centerline and for a length of 305 m (1,000 ft), 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-mm (6-in) base course of 80 CBR or better will be provided.

10-7.3.2 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 PCC for Navy and Marine Corps airfields; where operational experience has shown asphalt surfacing to be satisfactory, use a minimum 76-mm (3-in) AC surface over 152 mm (6 in) of 80 CBR base. Blast protection pavement design should be checked to make sure that it is adequate for supporting crash rescue vehicles.

10-7.3.3 Shoulders

10-7.3.3.1 Fixed-Wing Aircraft

Pave the first 3 m (10 ft) of runway shoulders. Design the pavement thickness for 75 percent of the gross weight of the design aircraft at 200 passes. Surface with 50 mm

(2 in) of AC on a minimum 152-mm (6-in) base of 80 CBR. Provide the outer 43 m (140 ft) 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.

10-7.3.3.2 Rotary-Wing Aircraft

Pave the first 7.5 m (25 ft) of shoulder adjacent to helicopter pads, runways, and taxiways with 50 mm (2 in) of AC on a minimum 152-mm (6-in) base course of 60 CBR. Provide the outer 15 m (50 ft) of shoulder with a liquid palliative or vegetative cover, or a combination of methods.

10-8 JUNCTURE BETWEEN RIGID AND FLEXIBLE PAVEMENTS

See paragraph 12-8.10.

Figure 10-1. Flexible Pavement Design Curves for Army Class I Helipads

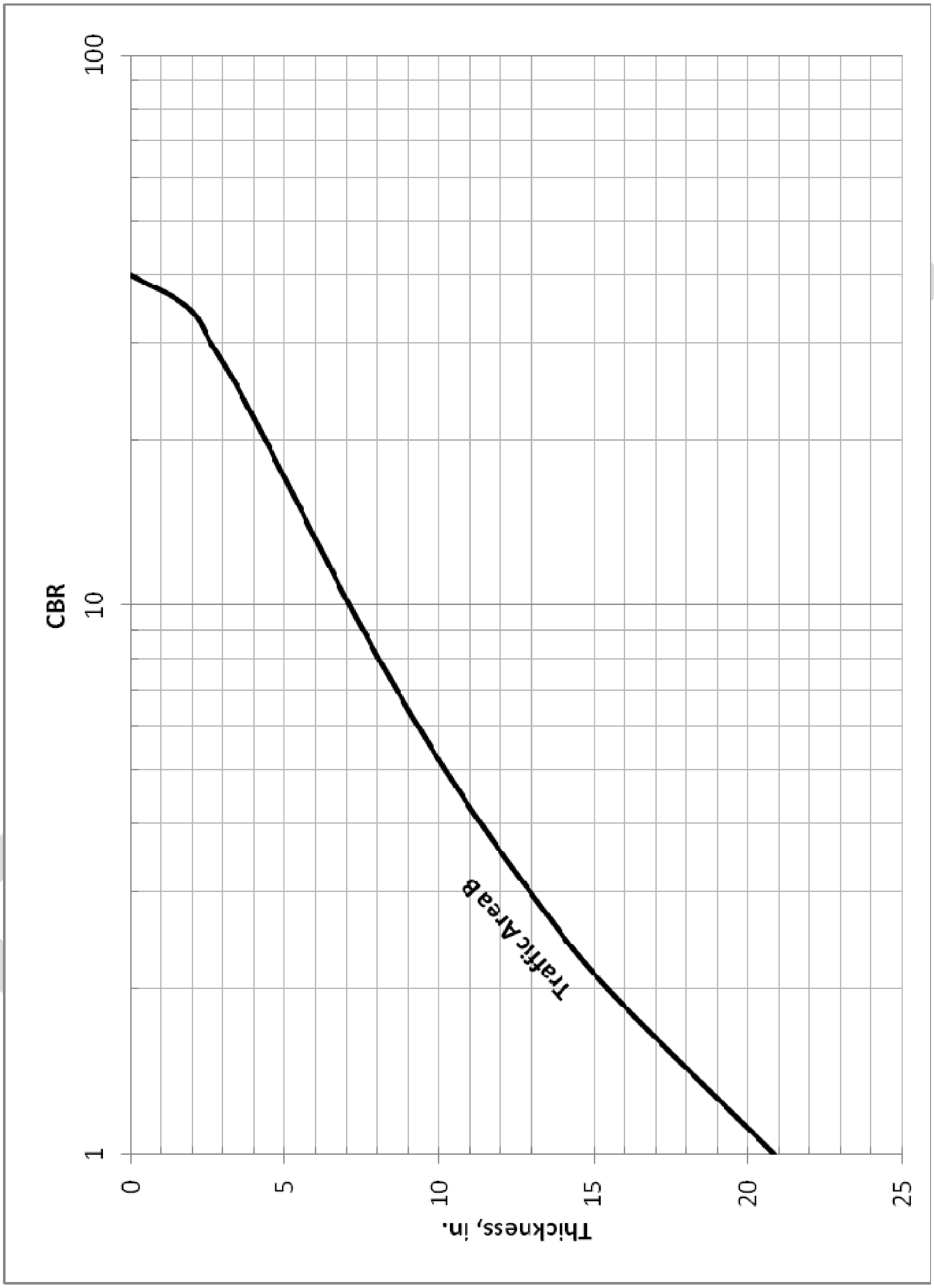


Figure 10-2. Flexible Pavement Design Curves for Army Class I Heliports

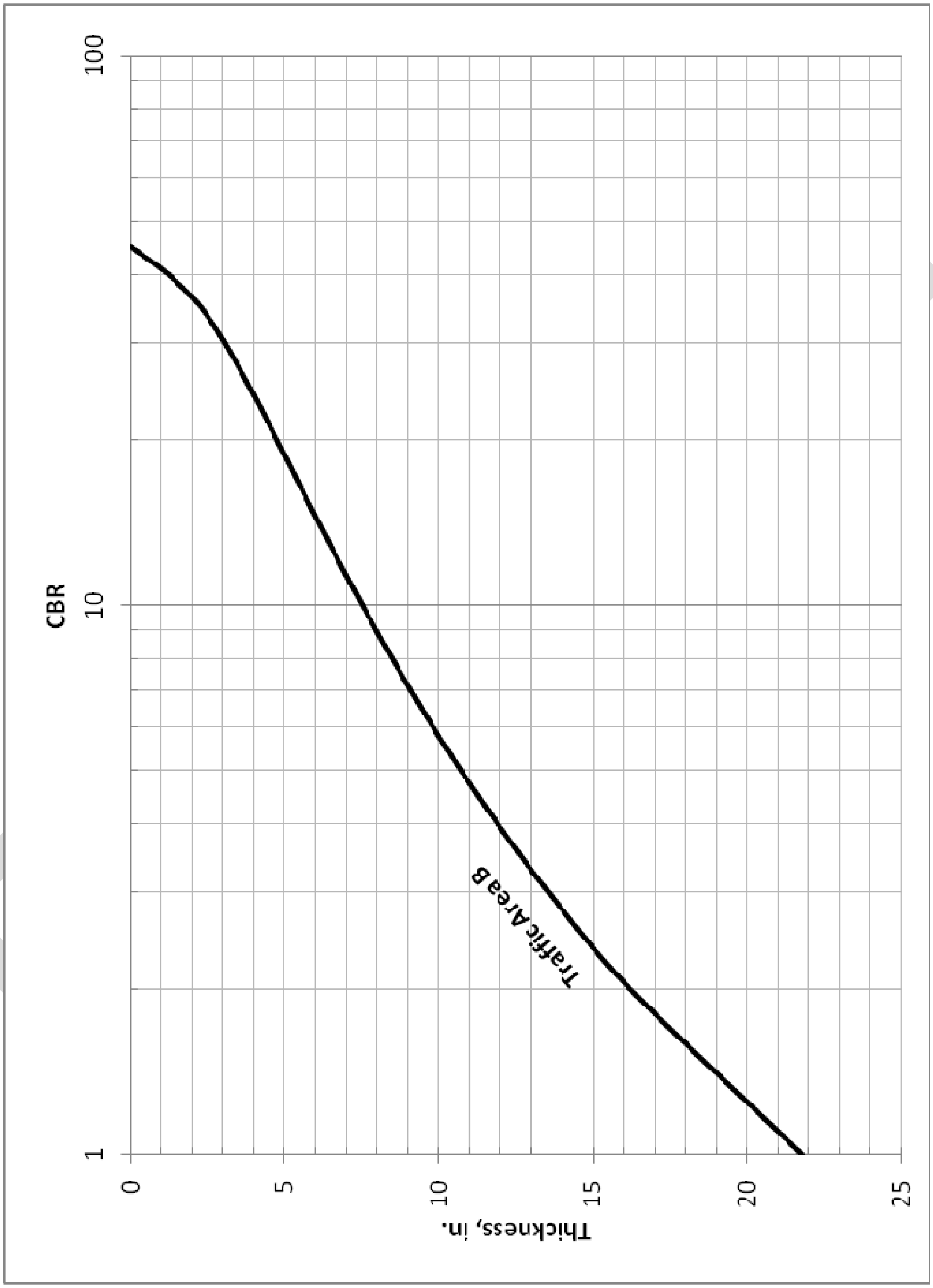


Figure 10-3. Flexible Pavement Design Curves for Army Class II VFR Helipad

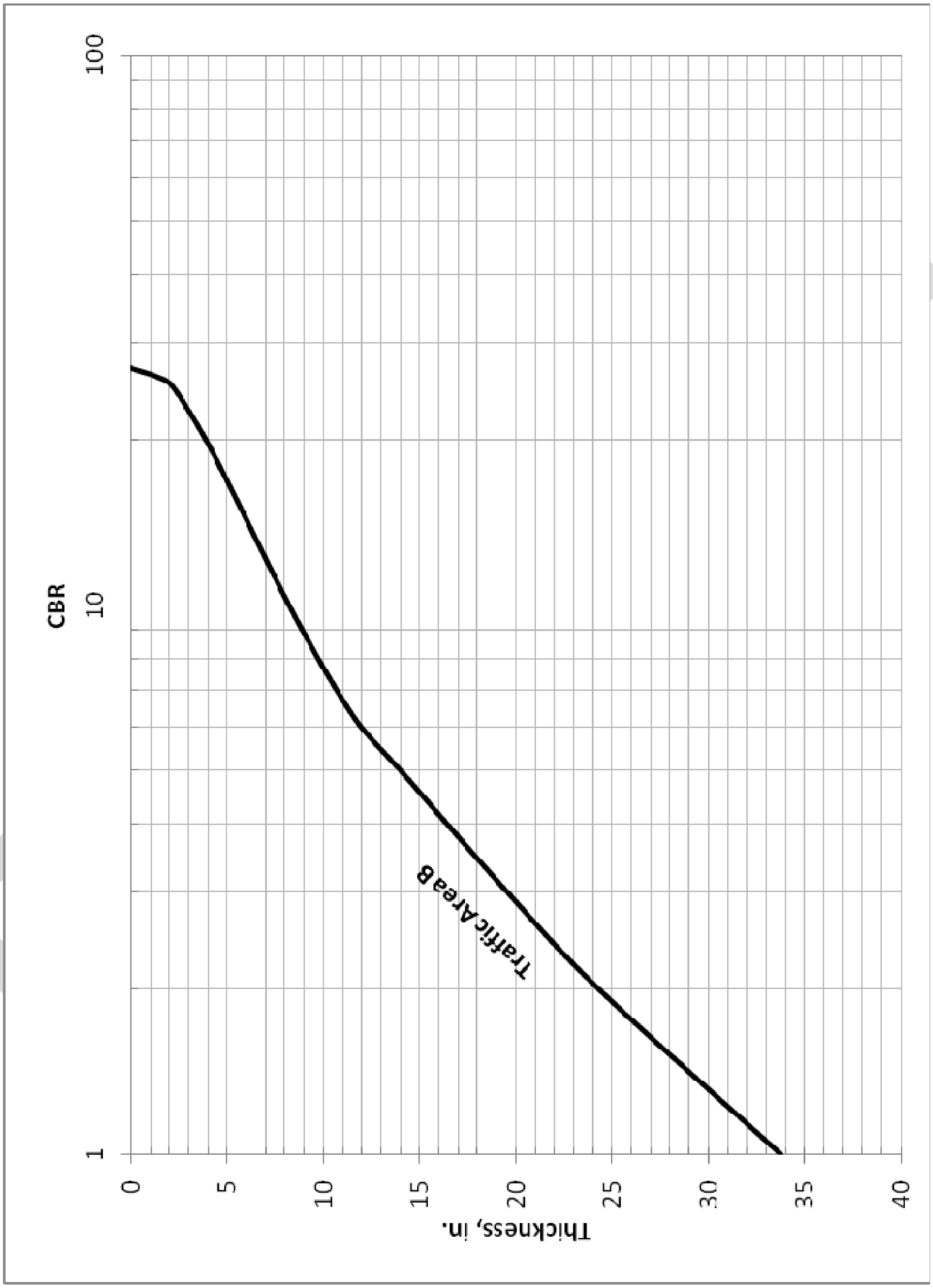


Figure 10-4. Flexible Pavement Design Curves for Class II VFR Heliport

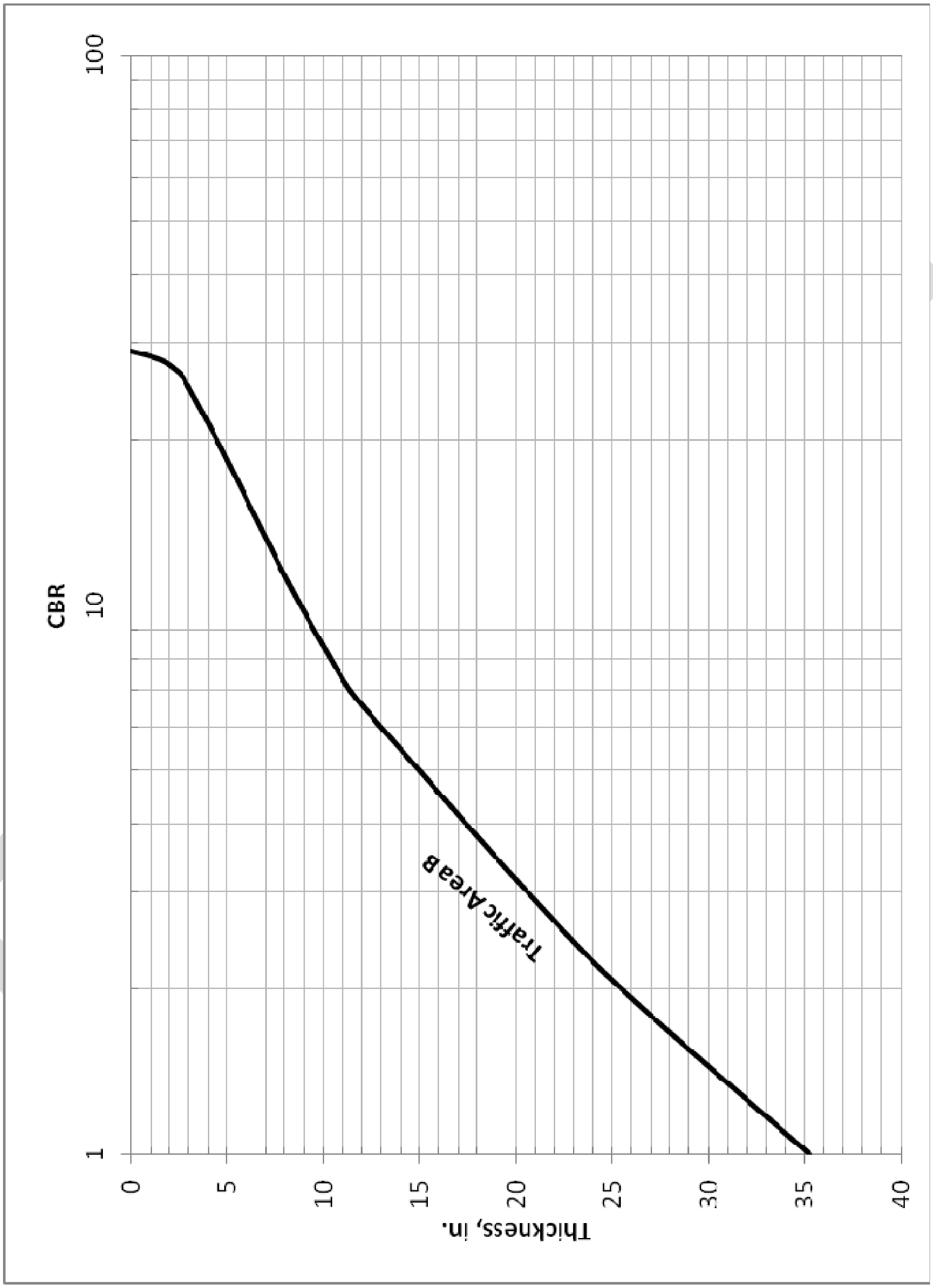


Figure 10-5. Flexible Pavement Design Curves for Army Class II - IFR - Helipad

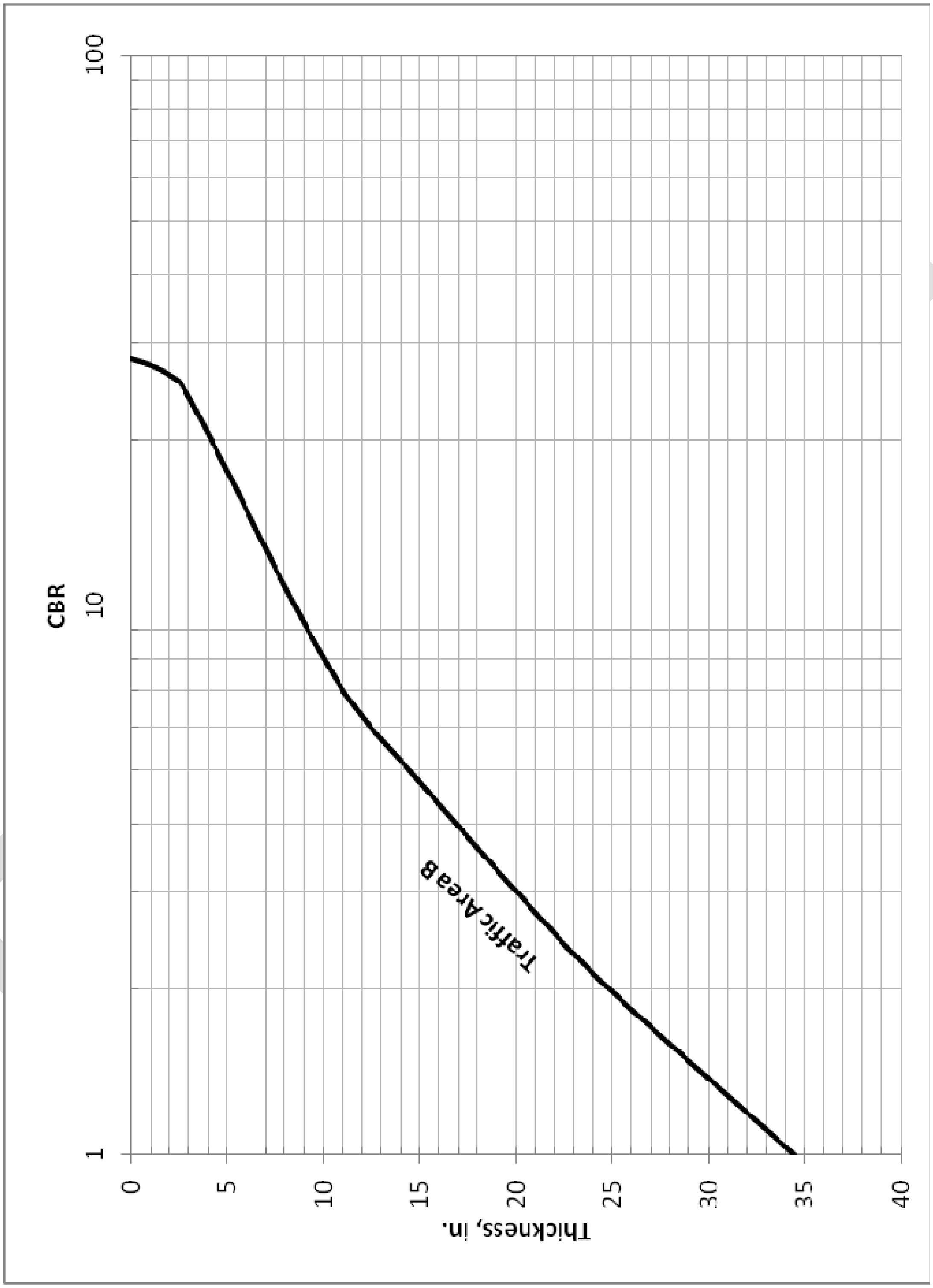


Figure 10-6. Flexible Pavement Design Curves for Army Class II - IFR - Heliport

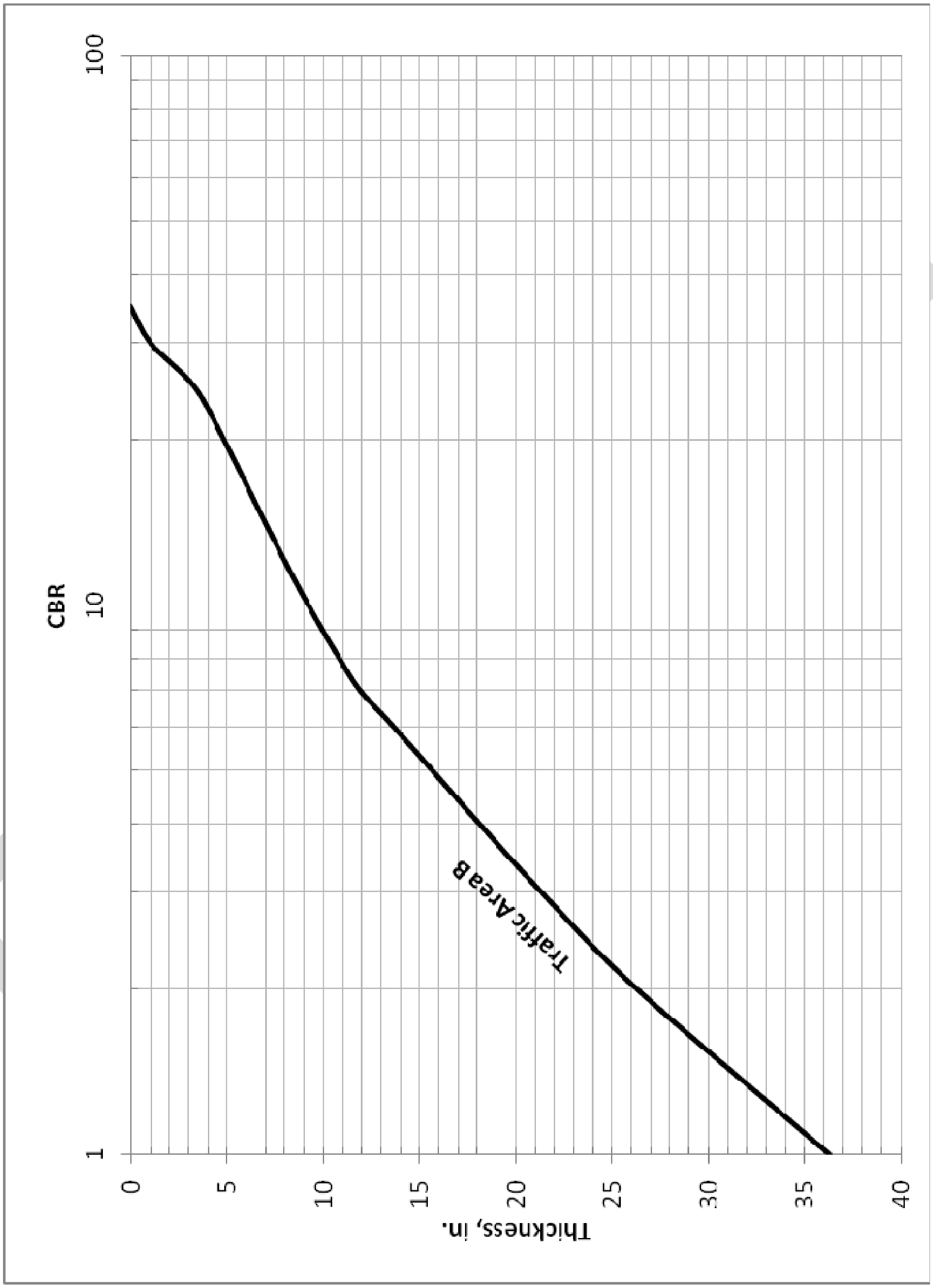


Figure 10-7. Flexible Pavement Design Curves for Army Class III

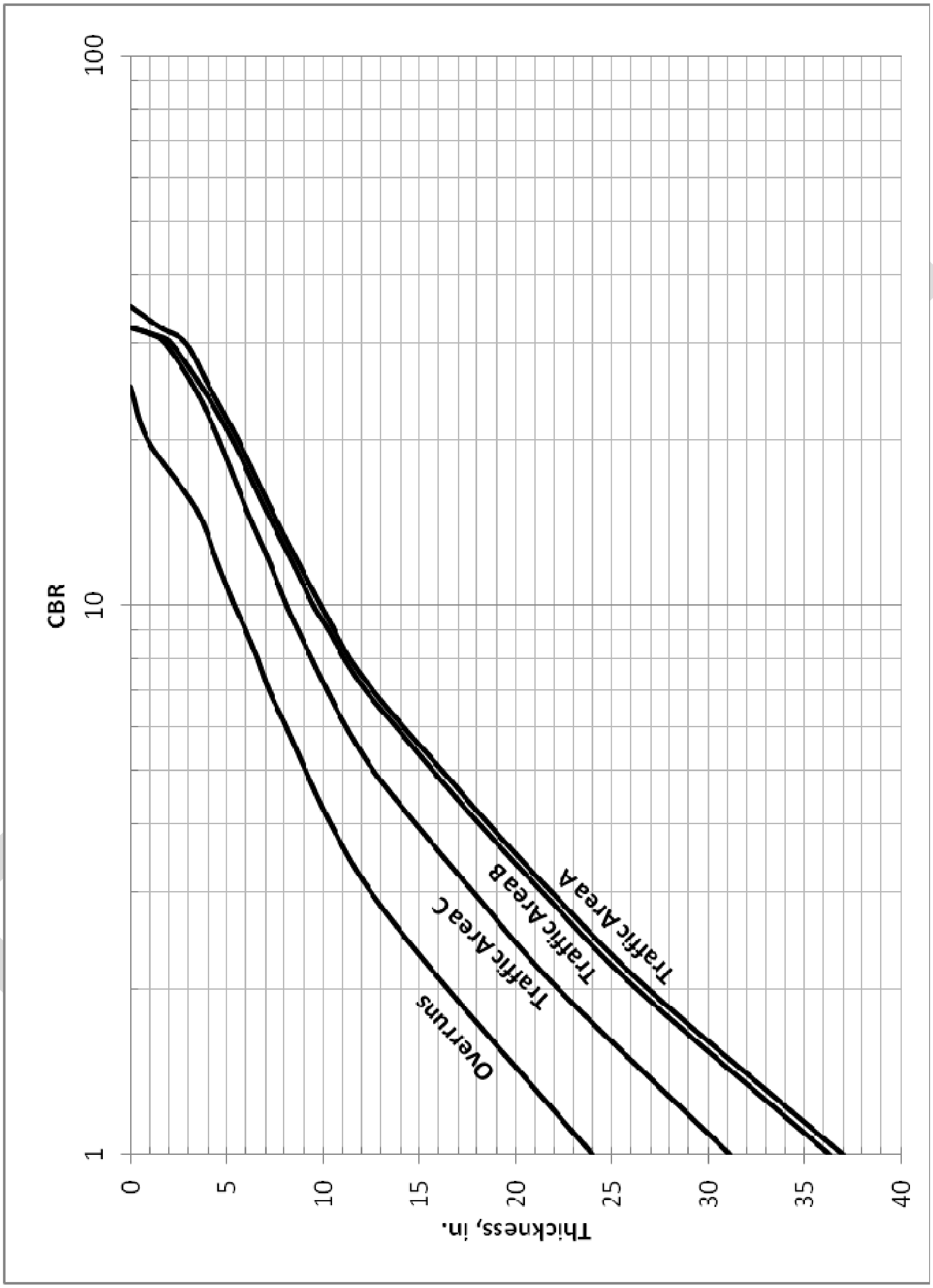


Figure 10-8. Flexible Pavement Design Curves for Army Class IV Runway Length < 5,000 ft - C-130

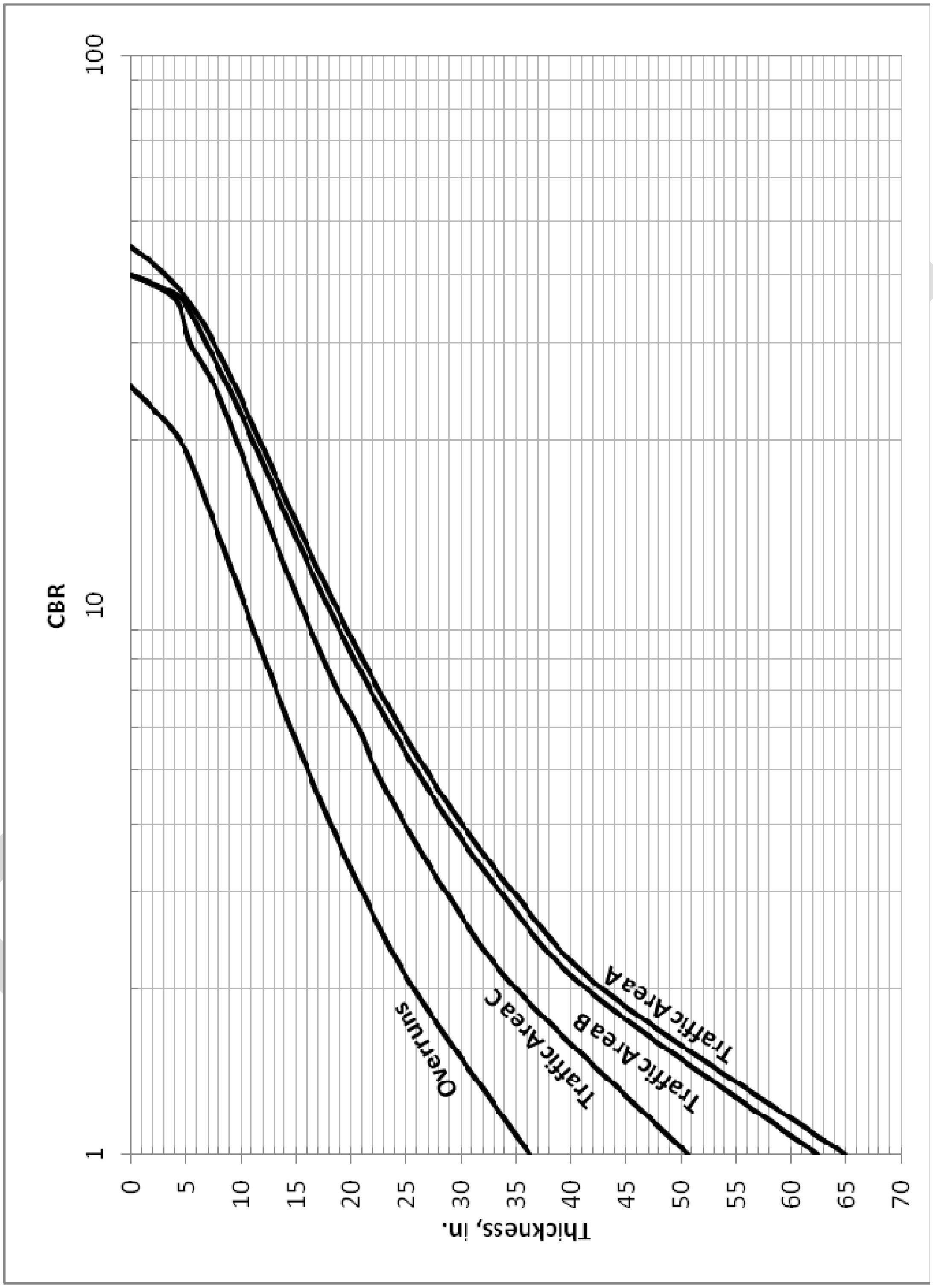


Figure 10-9. Flexible Pavement Design Curves Army Class IV Runway Length < 5,000 ft - C-17

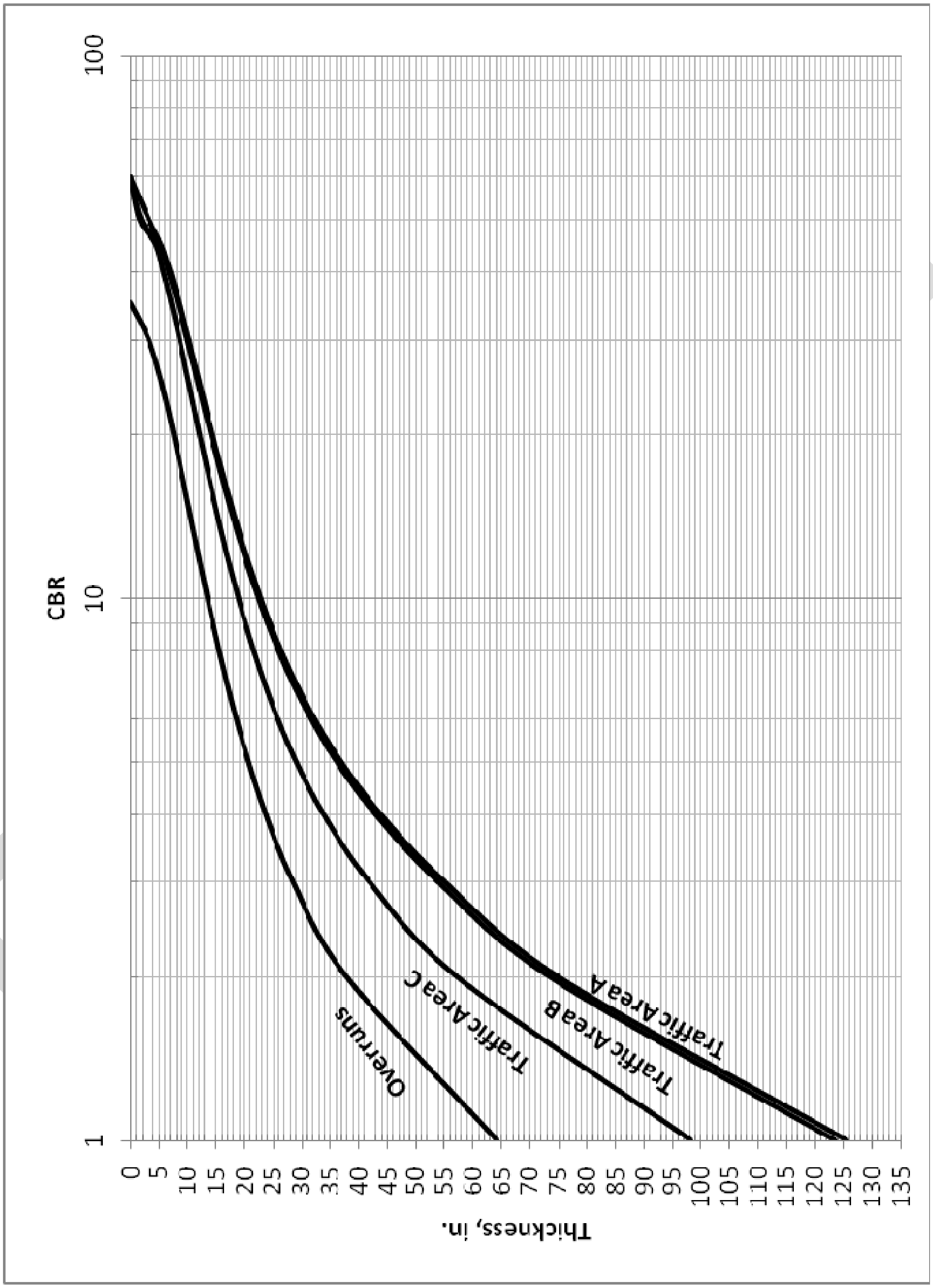


Figure 10-10. Flexible Pavement Design Curves for Army Class IV - 5,000 ft < Runway Length < 9,000 ft - C-17

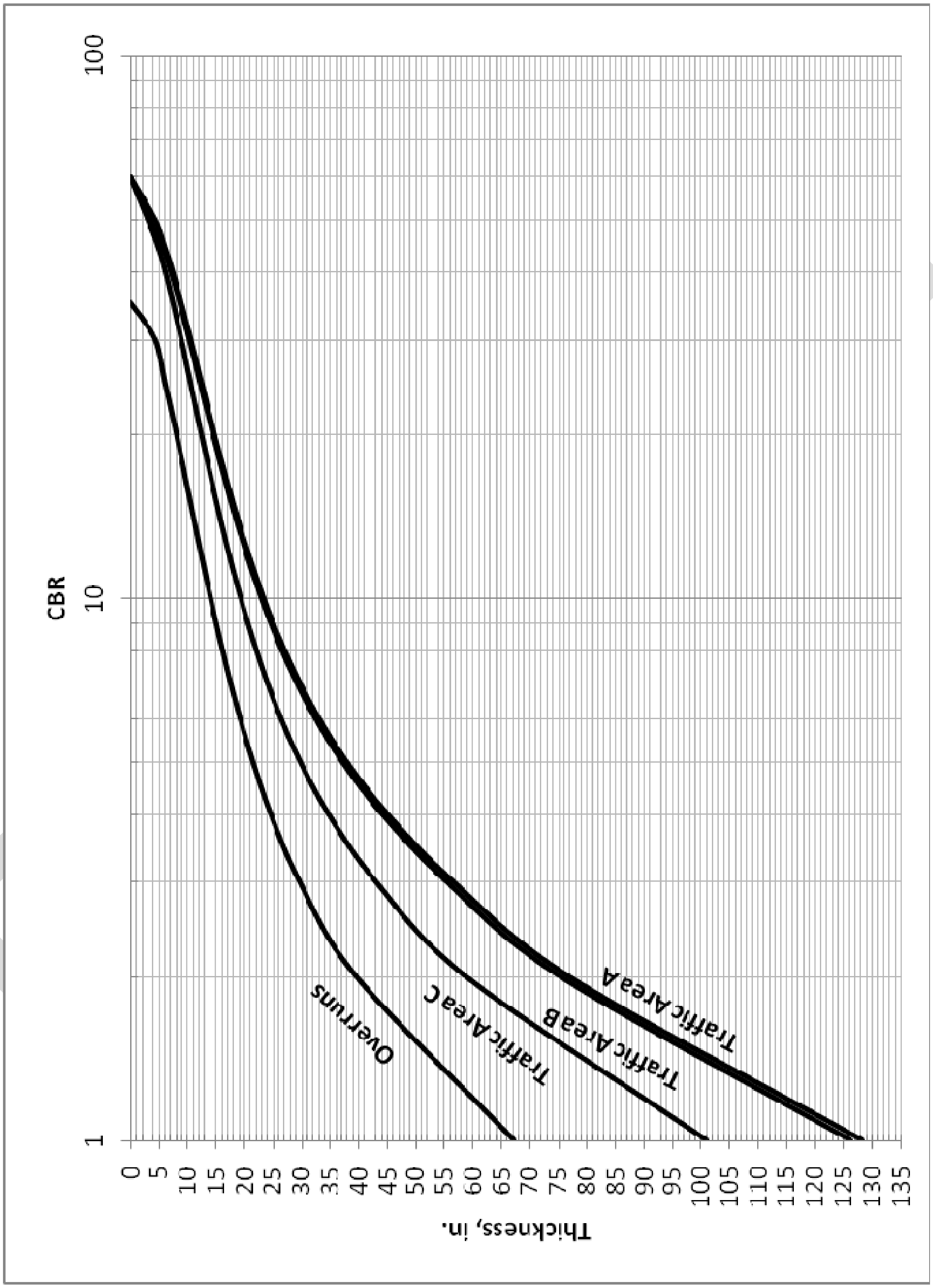


Figure 10-11. Flexible Pavement Design Curves for Army Class IV Runway Length > 9,000 ft - C-17

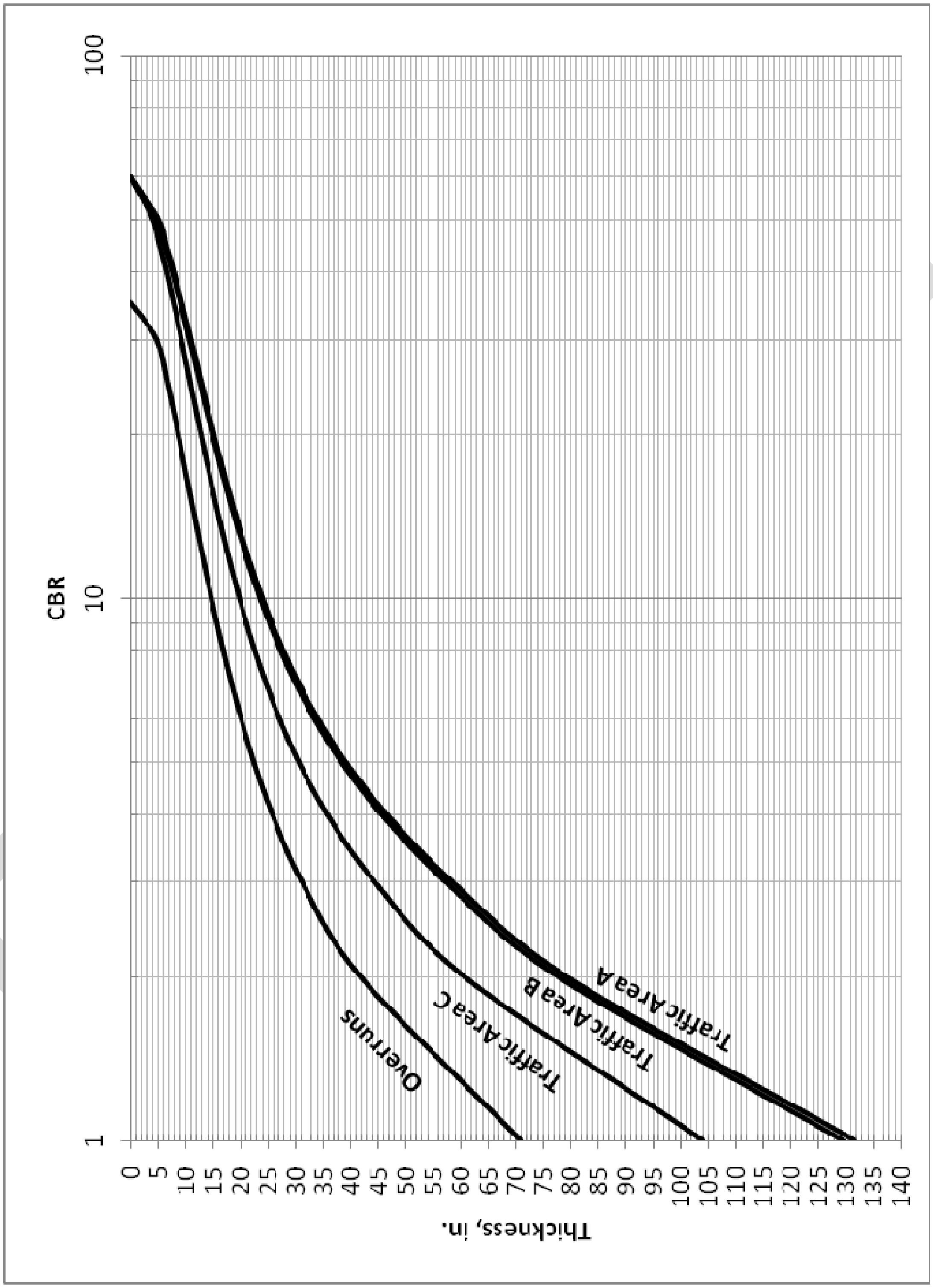


Figure 10-12. Flexible Pavement Design Curves for Army Class IV Runway Length > 9,000 ft with mobilization mission - C-17

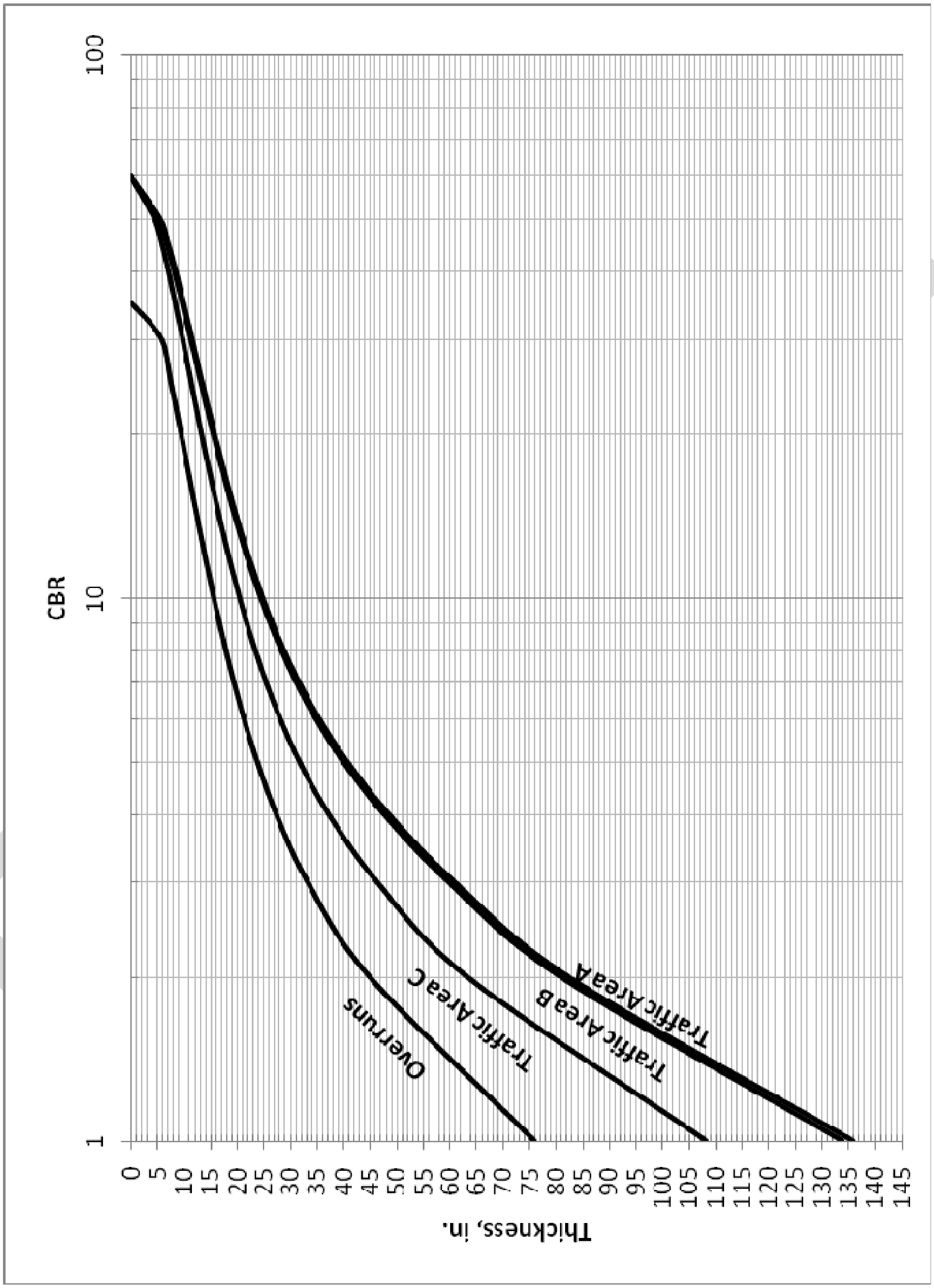


Figure 10-13. Flexible Pavement Design Curves for Army Class V

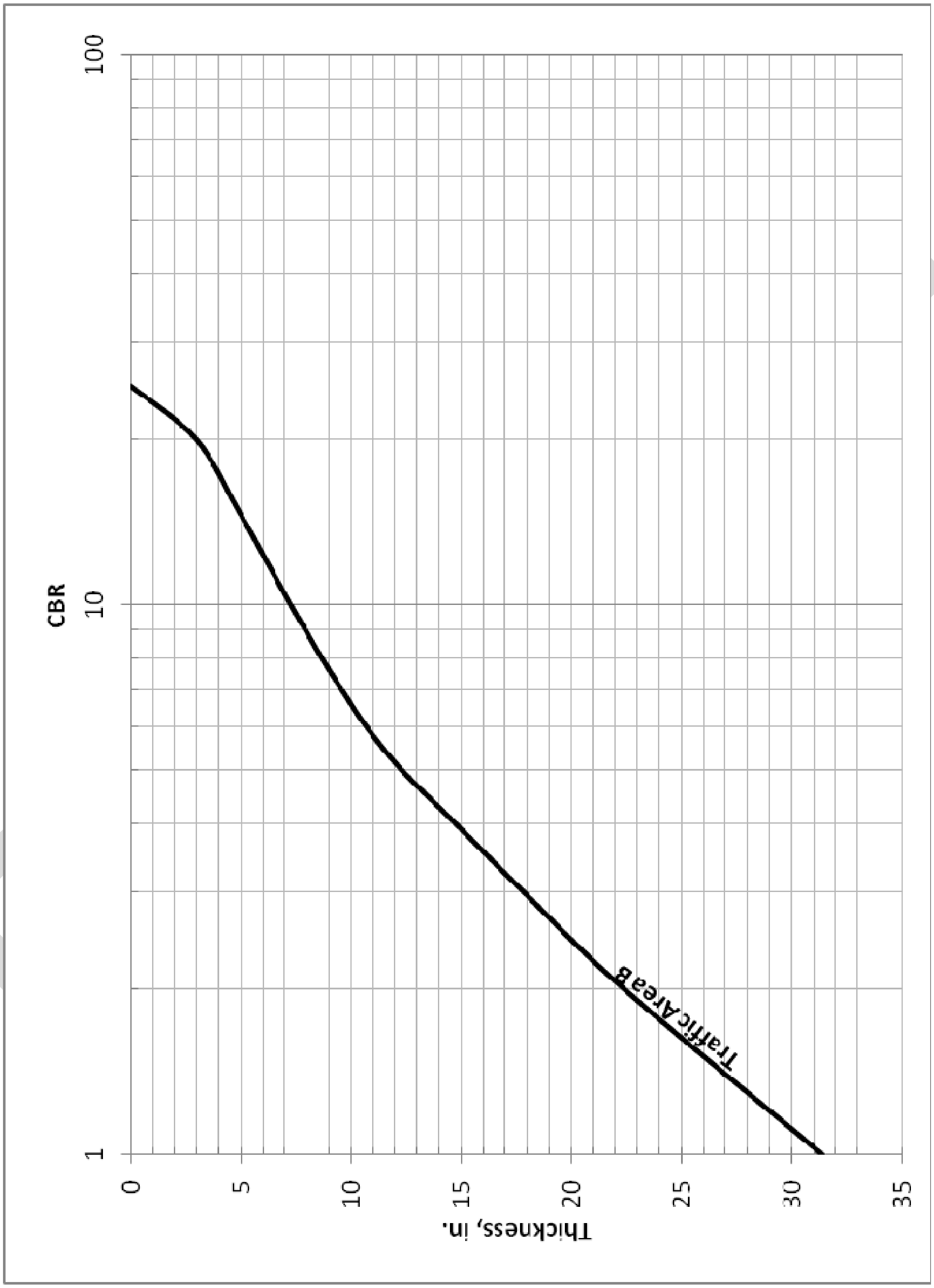


Figure 10-14. Flexible Pavement Design Curves for Army Class VI - Paved Landing Zone C-130

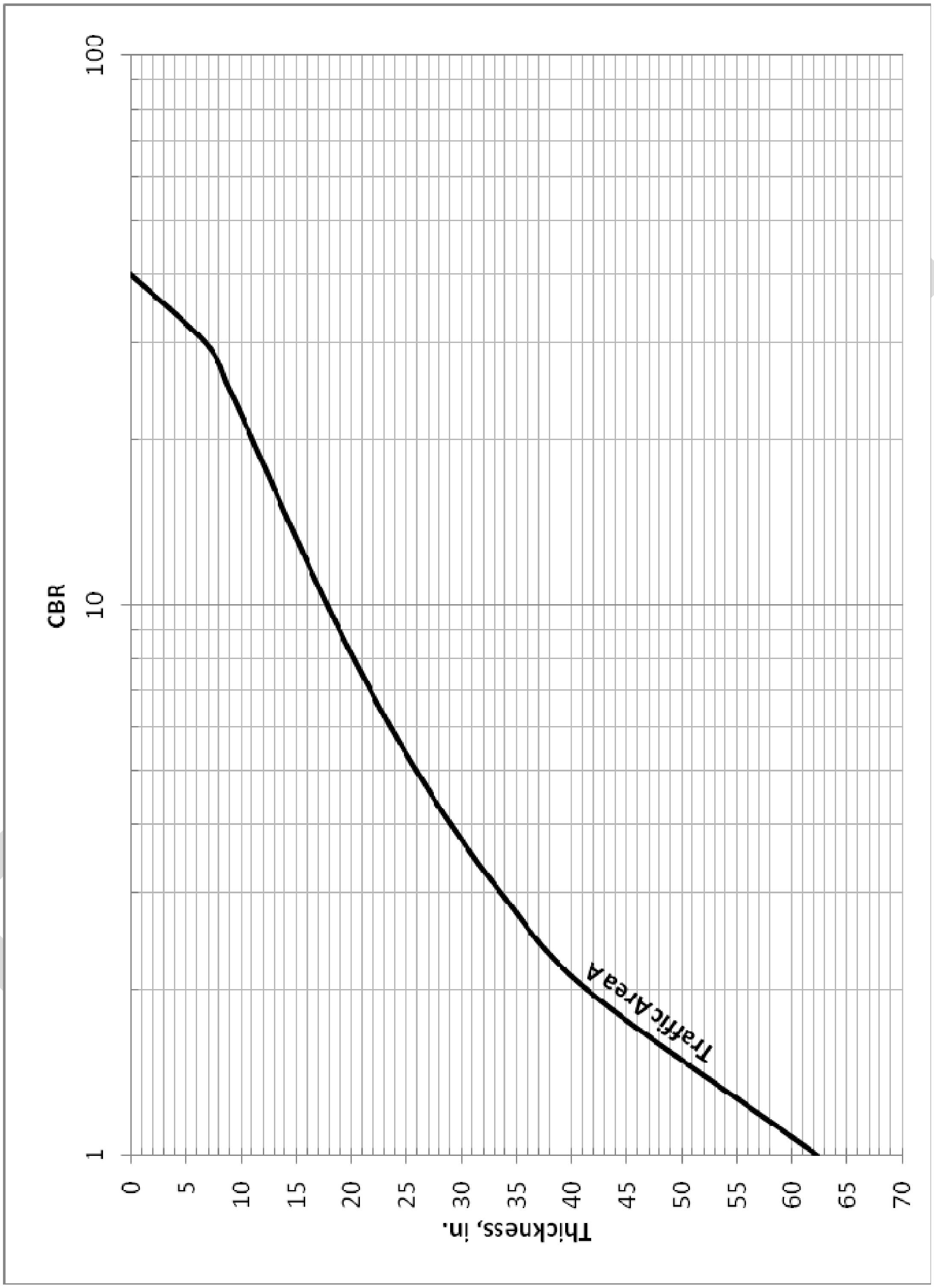


Figure 10-15. Flexible Pavement Design Curves for Army Class VI - Paved Landing Zone < 5000 ft - C-17

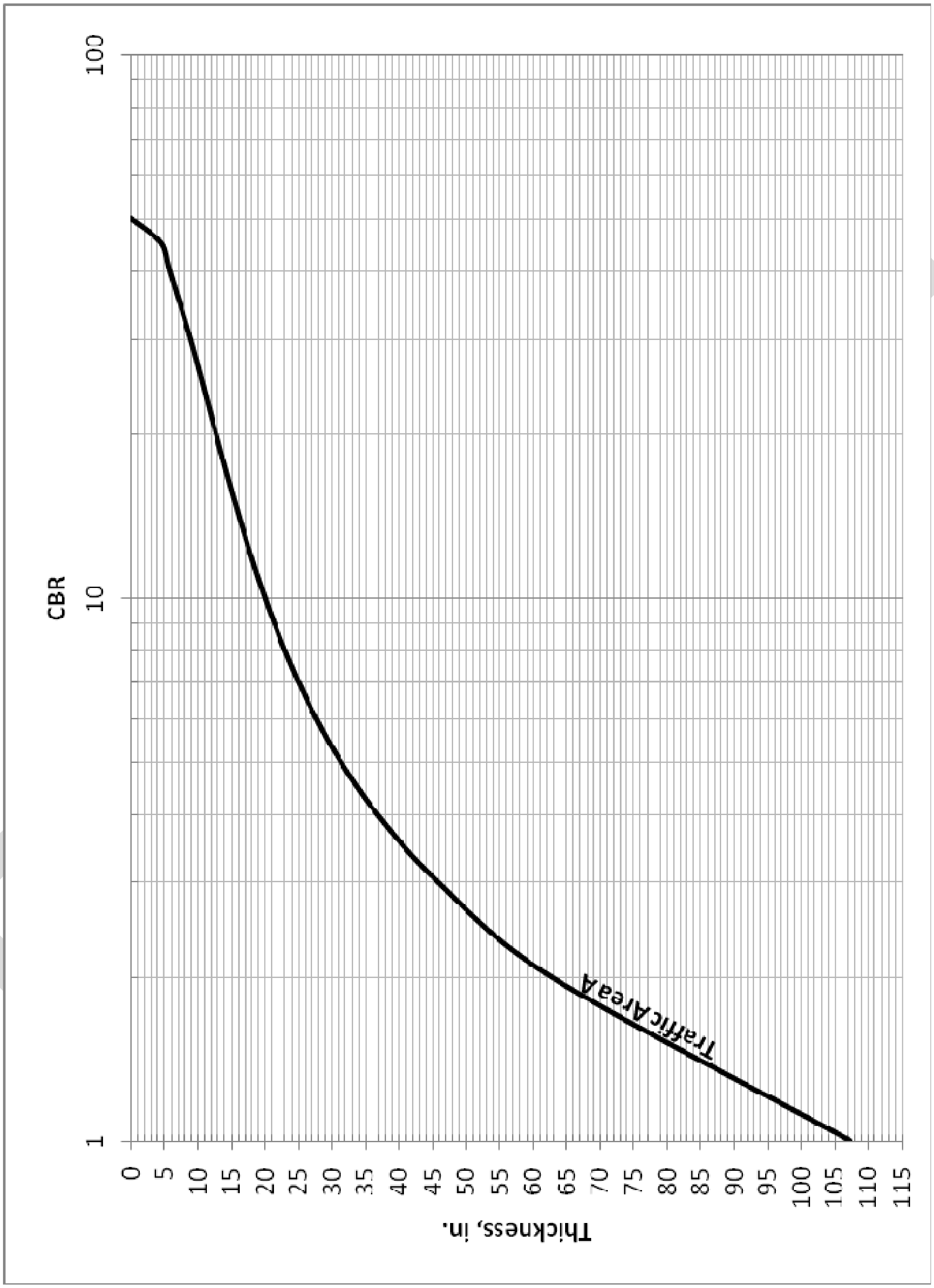


Figure 10-16. Flexible Pavement Design Curves for Army Class VI - Paved Landing Zone ≥ 5000 ft - C-17

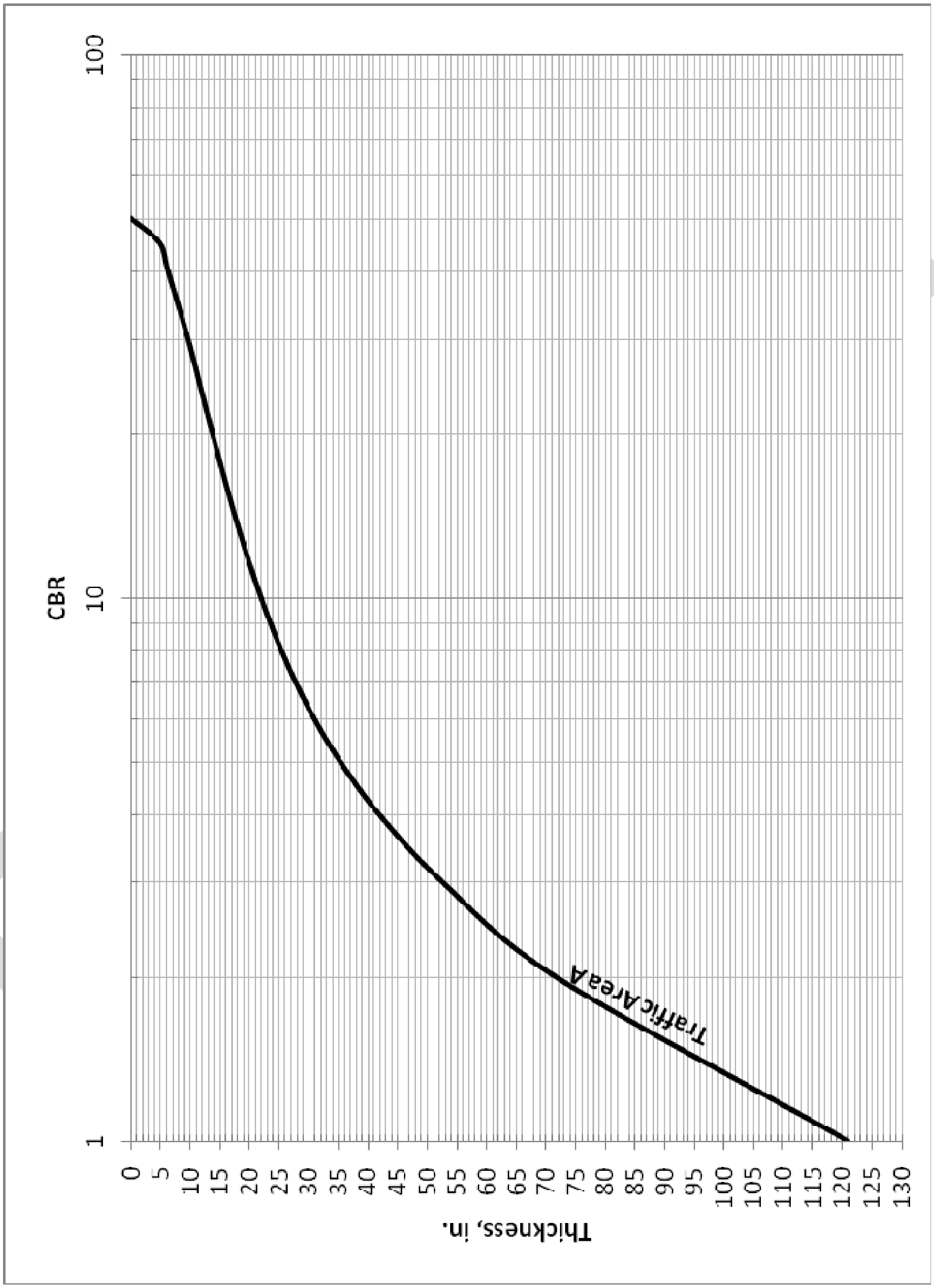


Figure 10-17. Flexible Pavement Design Curves for Air Force Light

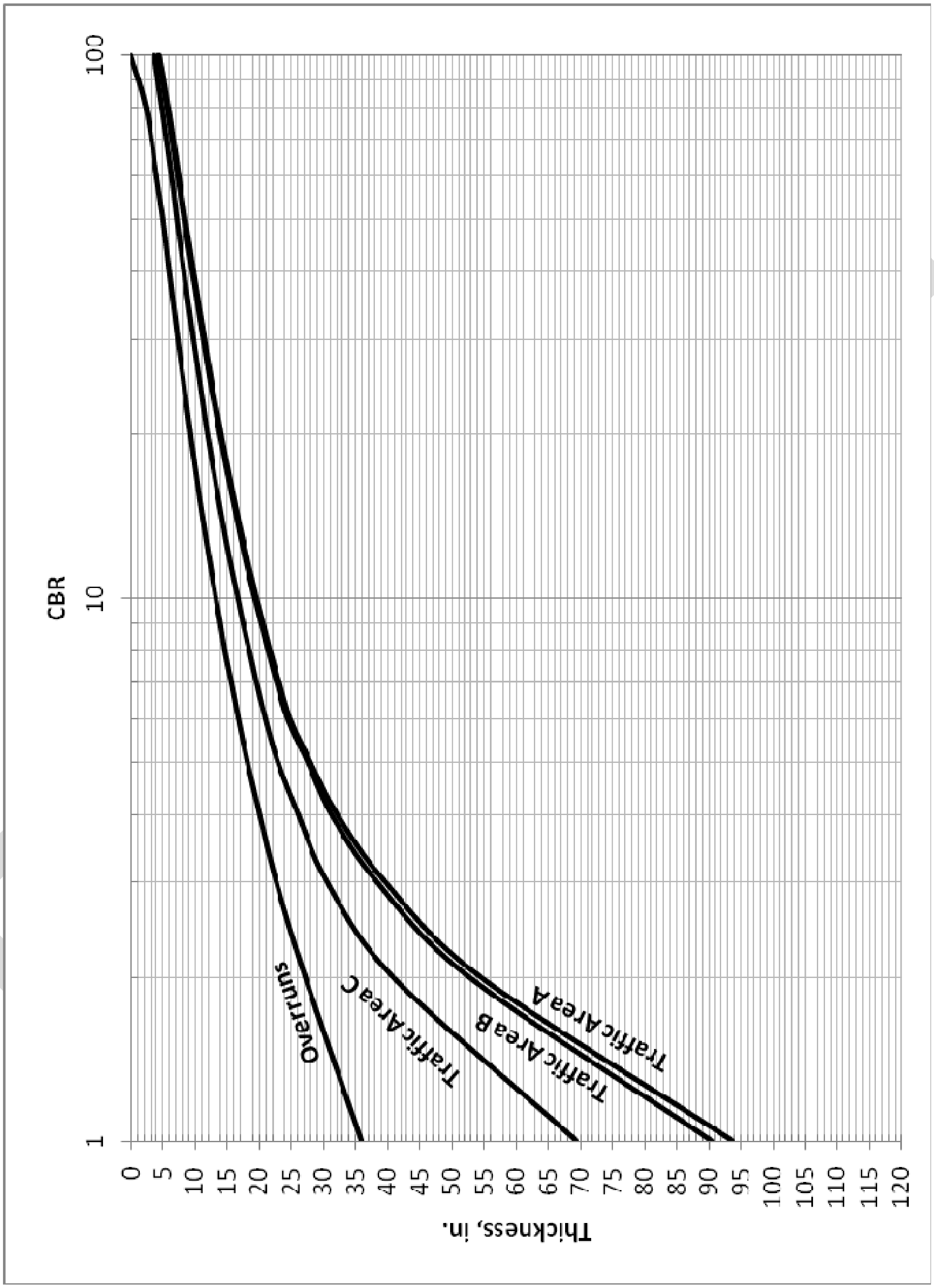


Figure 10-18. Flexible Pavement Design Curves for Air Force Medium - Runway Width ≥ 200 ft

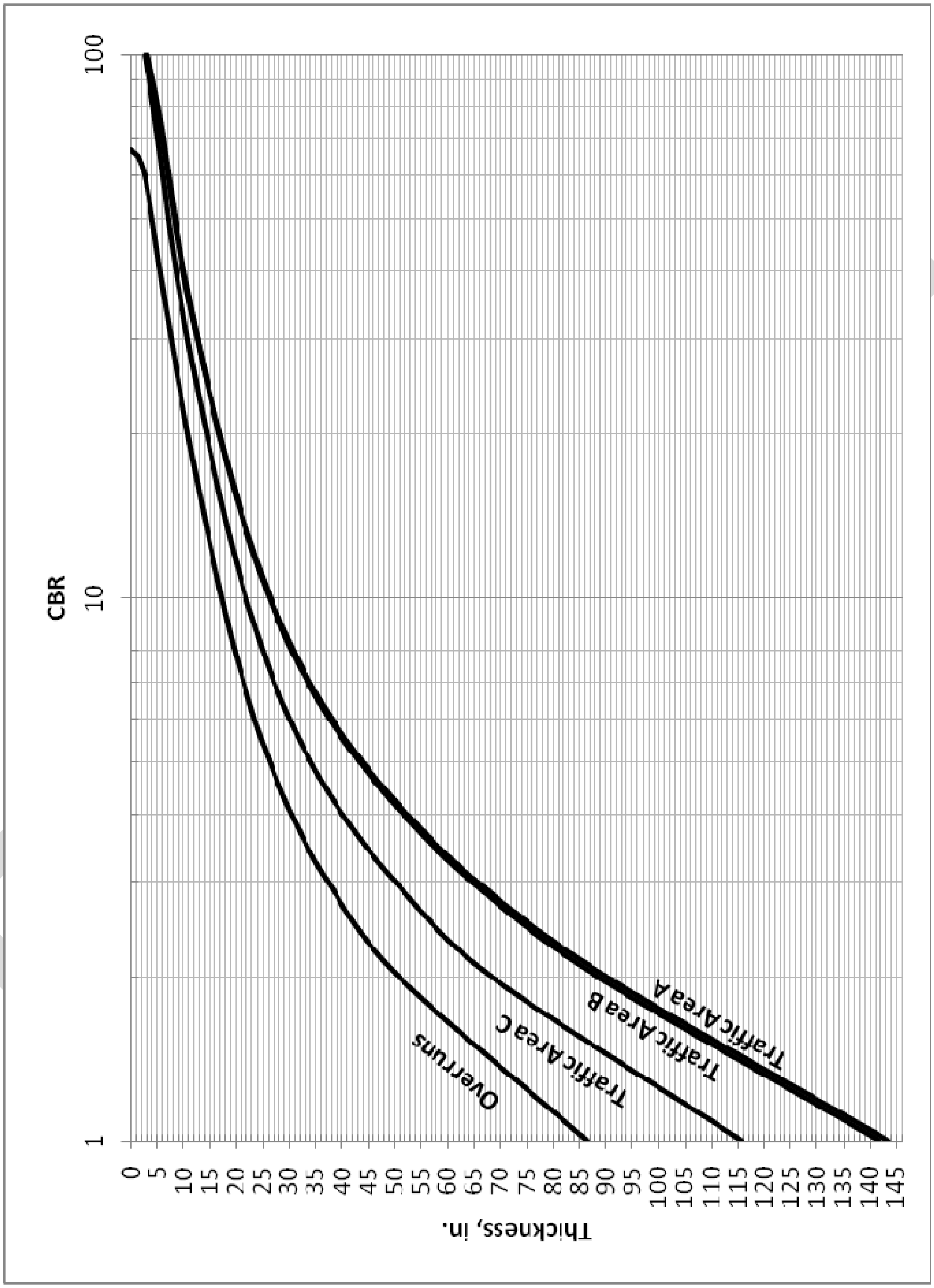


Figure 10-19. Flexible Pavement Design Curves for Air Force Medium - Runway Width < 200 ft

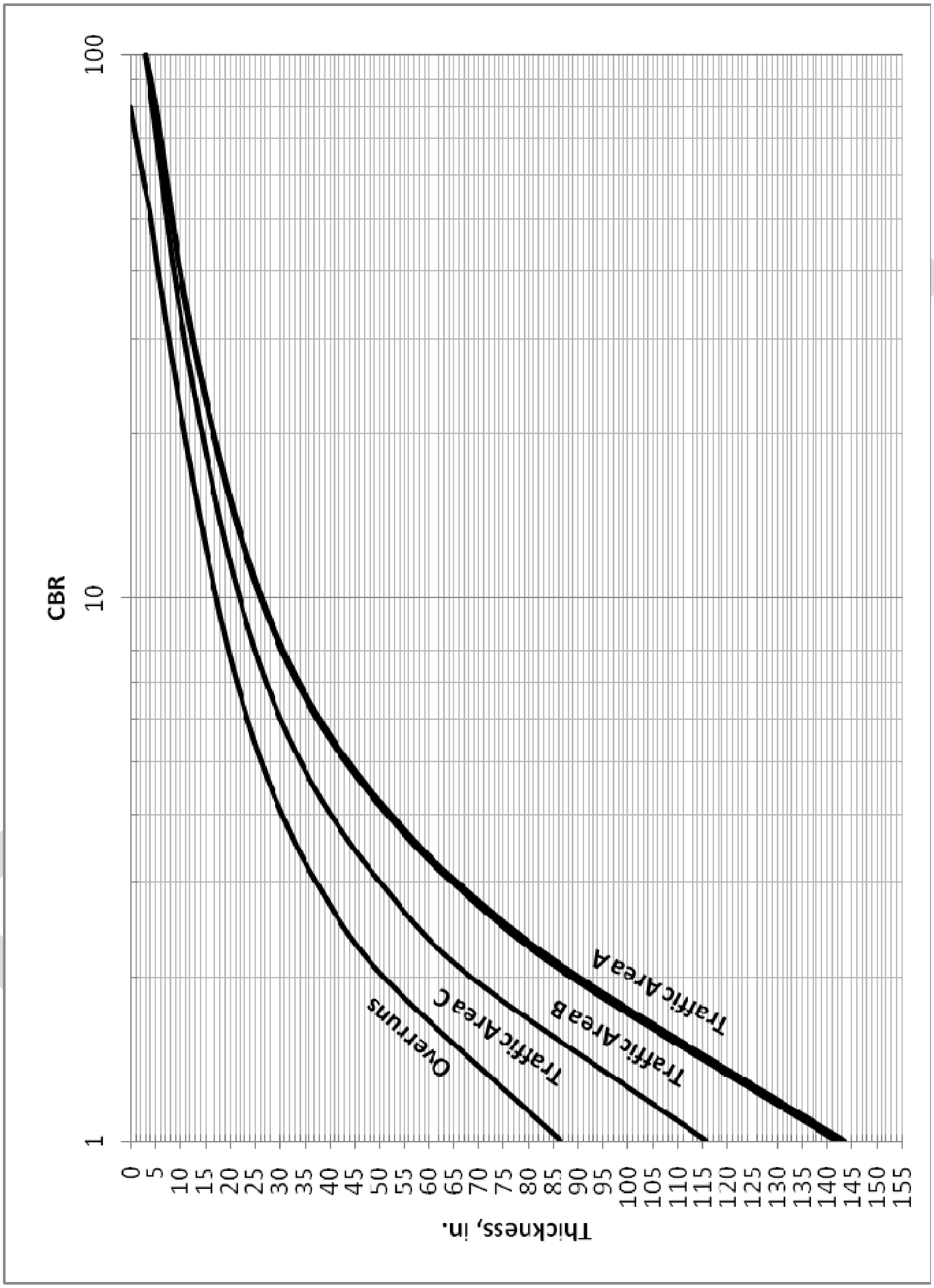


Figure 10-20. Flexible Pavement Design Curves for Air Force Heavy

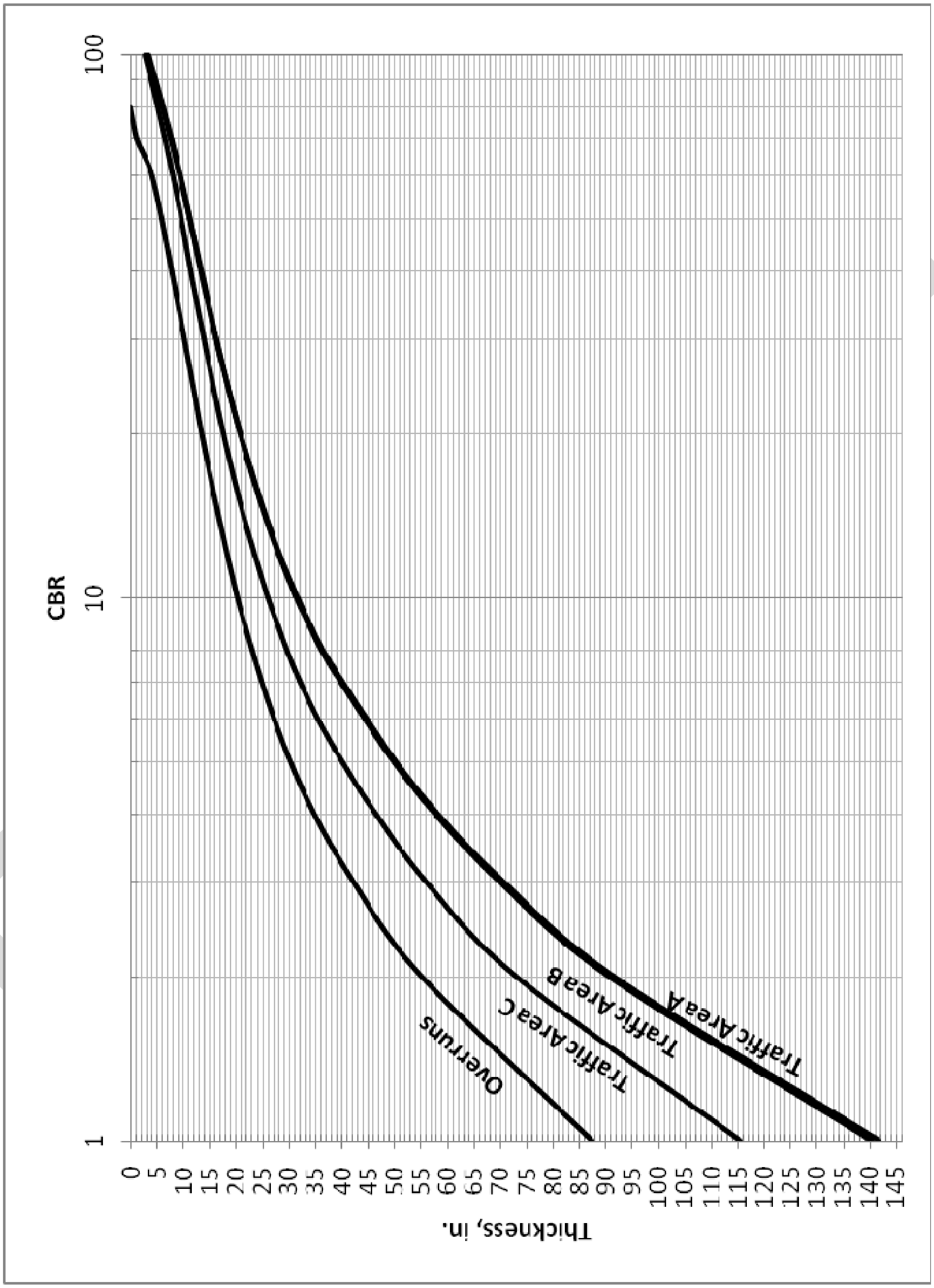


Figure 10-21. Flexible Pavement Design Curves for Air Force Modified Heavy

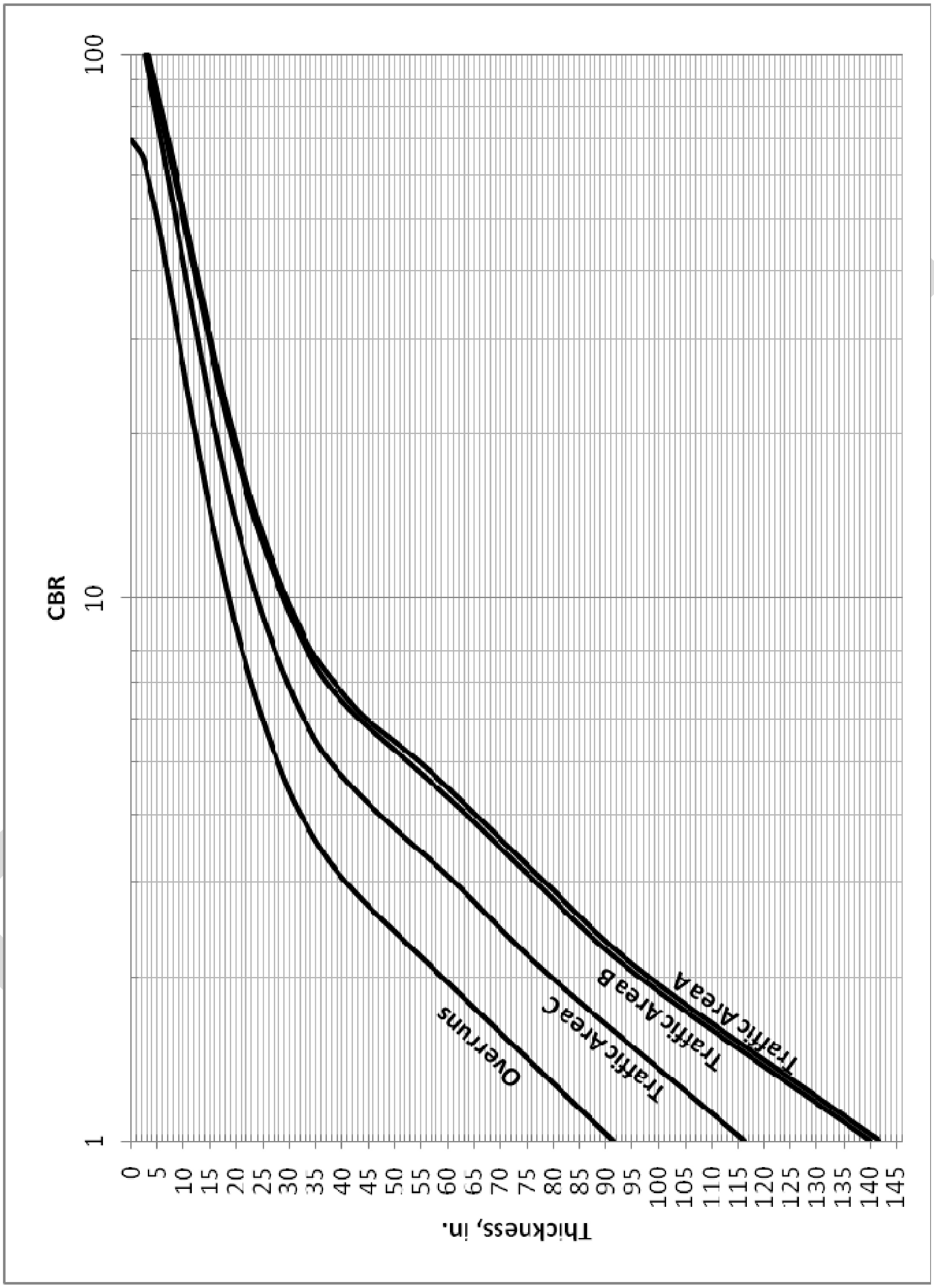


Figure 10-22. Flexible Pavement Design Curves for Air Force Landing Zone - C-130

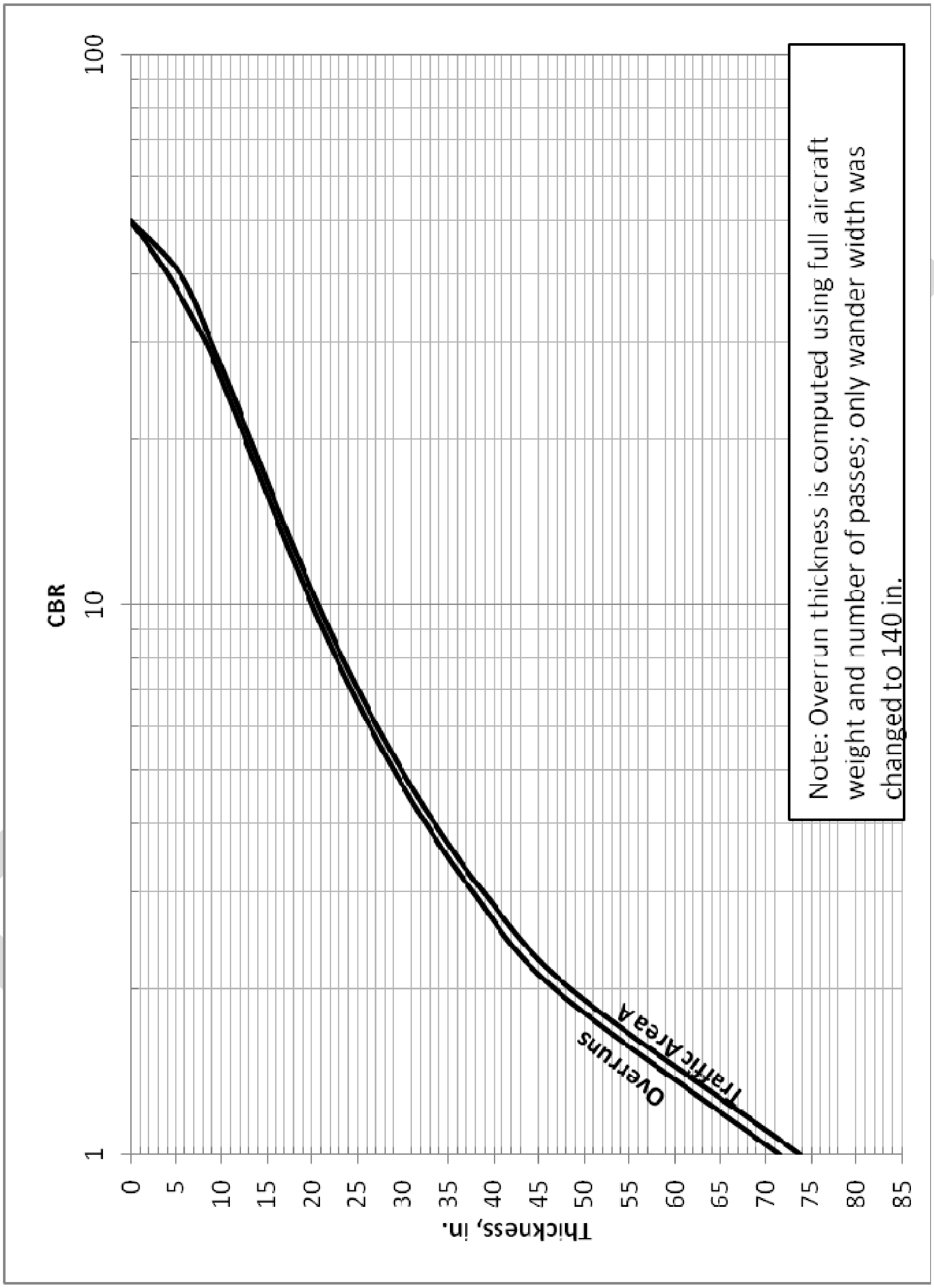


Figure 10-23. Flexible Pavement Design Curves for Air Force Landing Zone - C-17

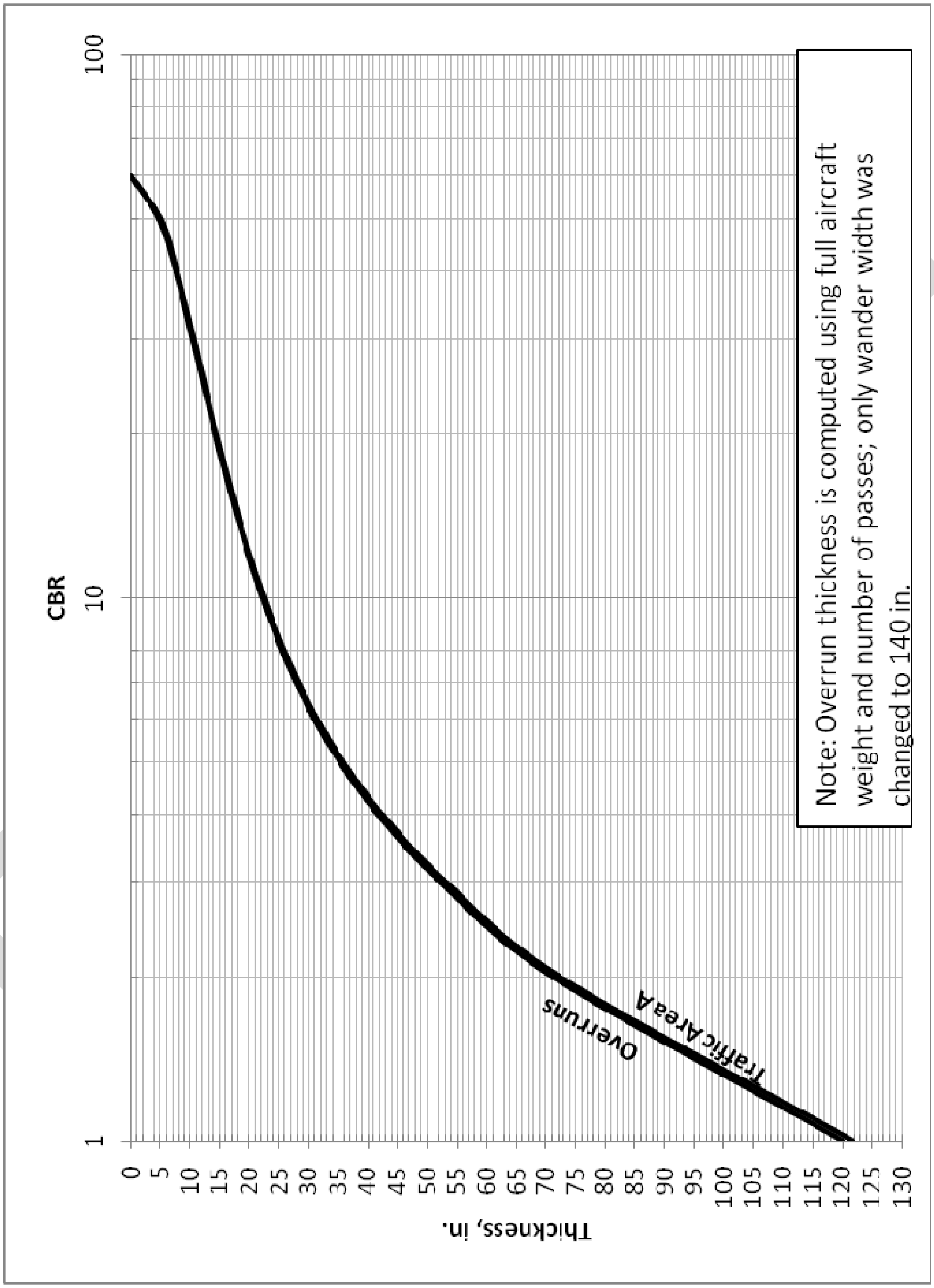


Figure 10-24. Flexible Pavement Design Curve for Navy Design Traffic Group I

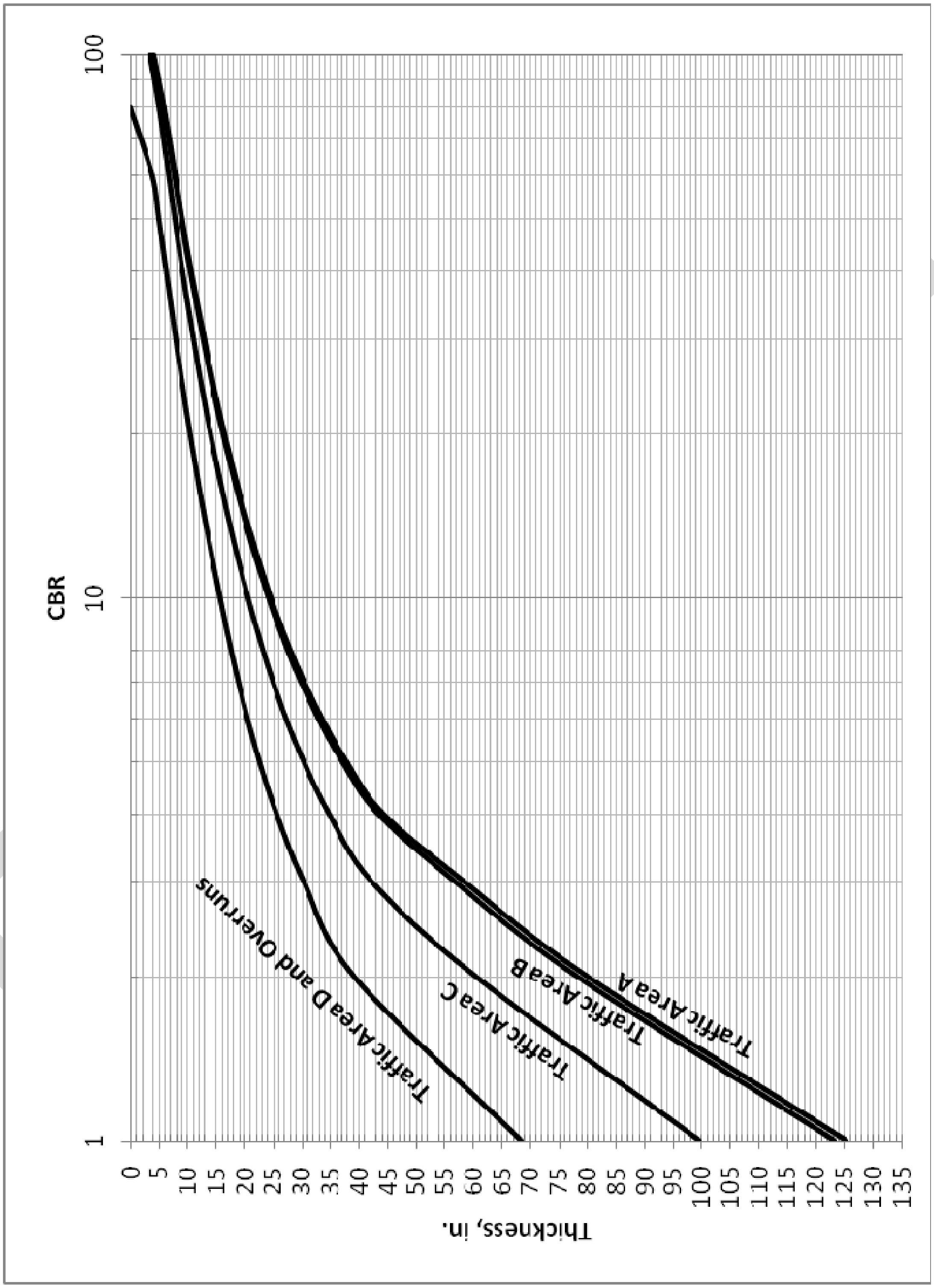


Figure 10-25. Flexible Pavement Design Curve for Navy Design Traffic Group II

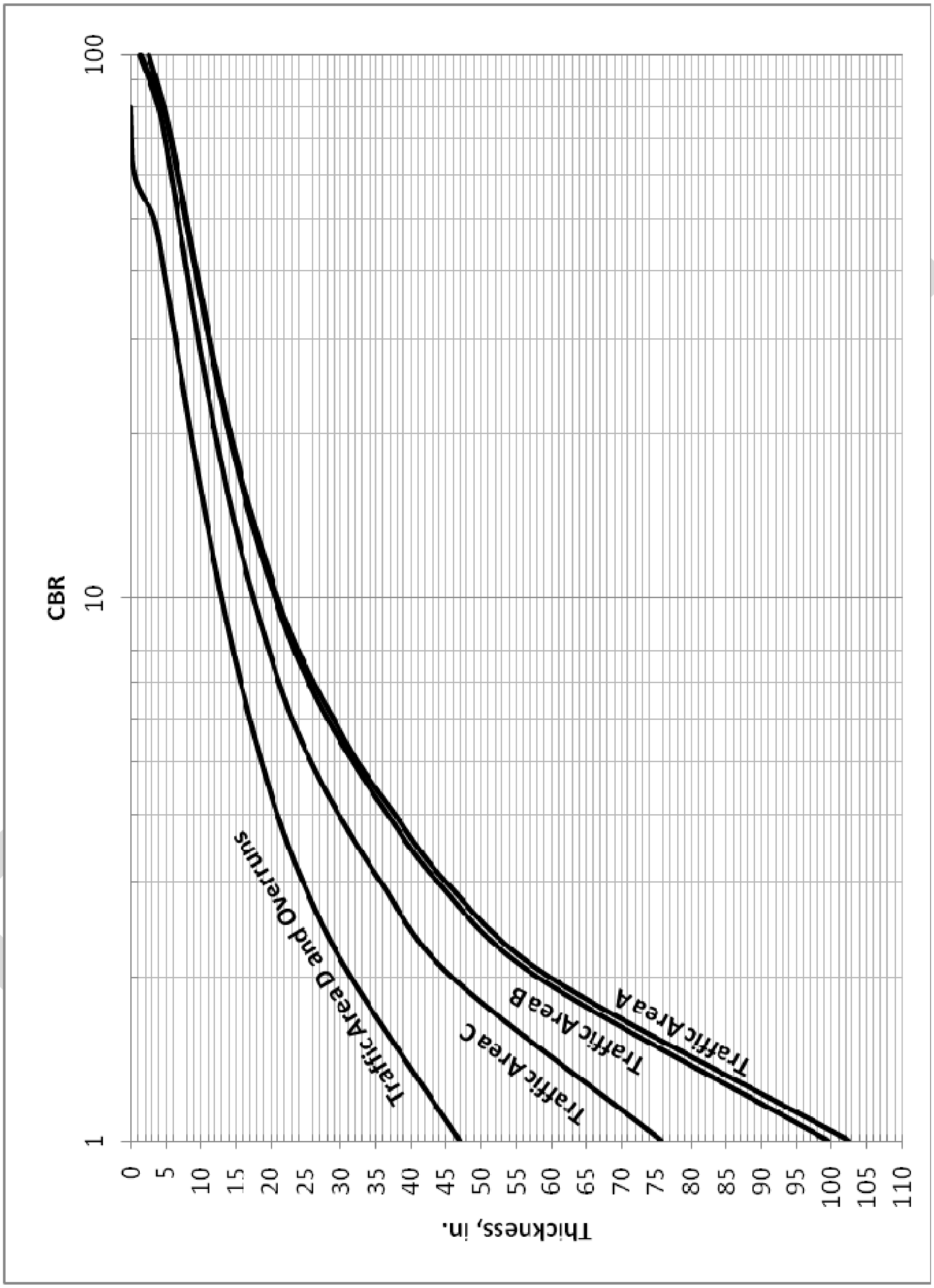


Figure 10-26. Flexible Pavement Design Curve for Navy Design Traffic Group III

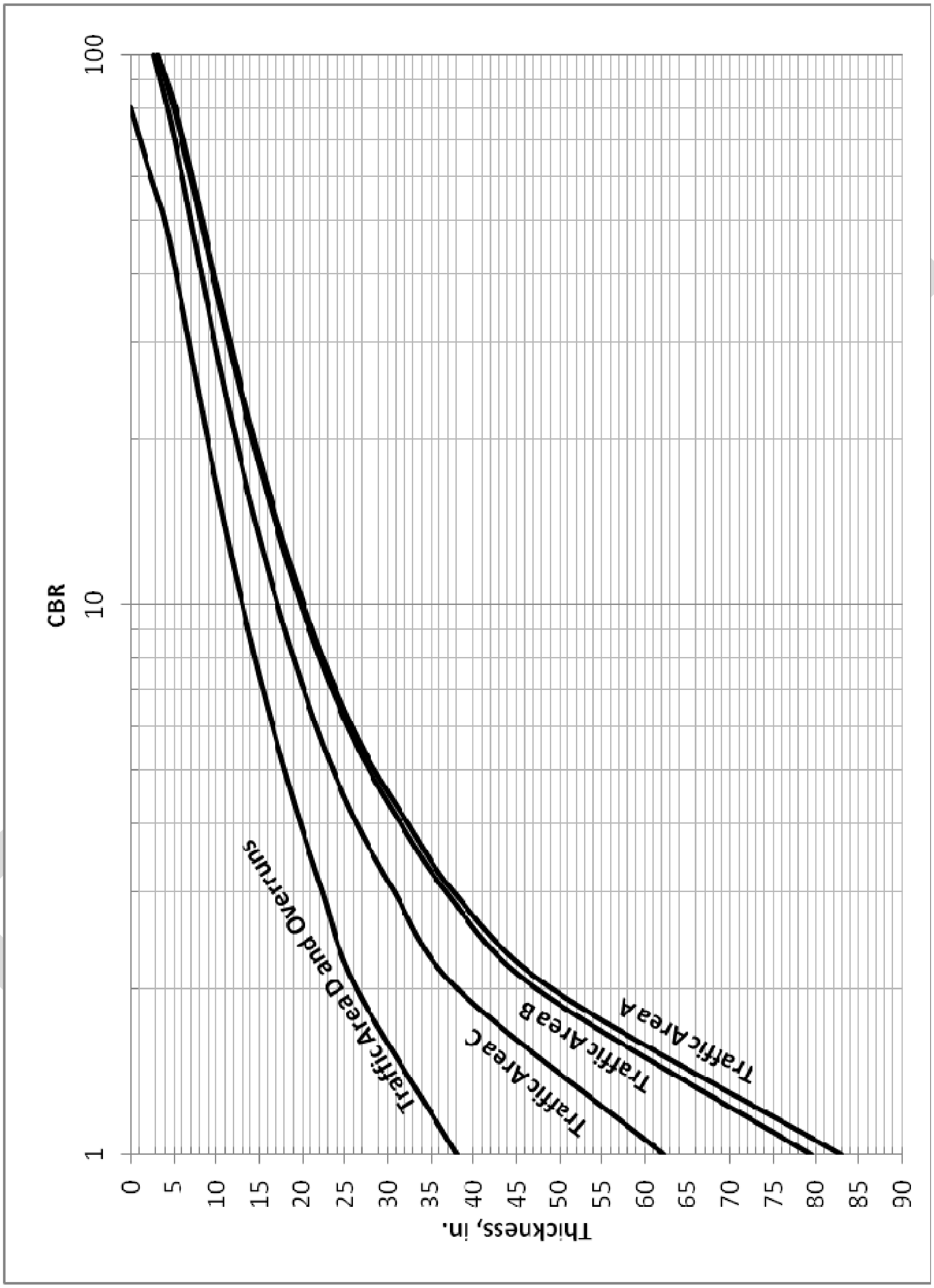
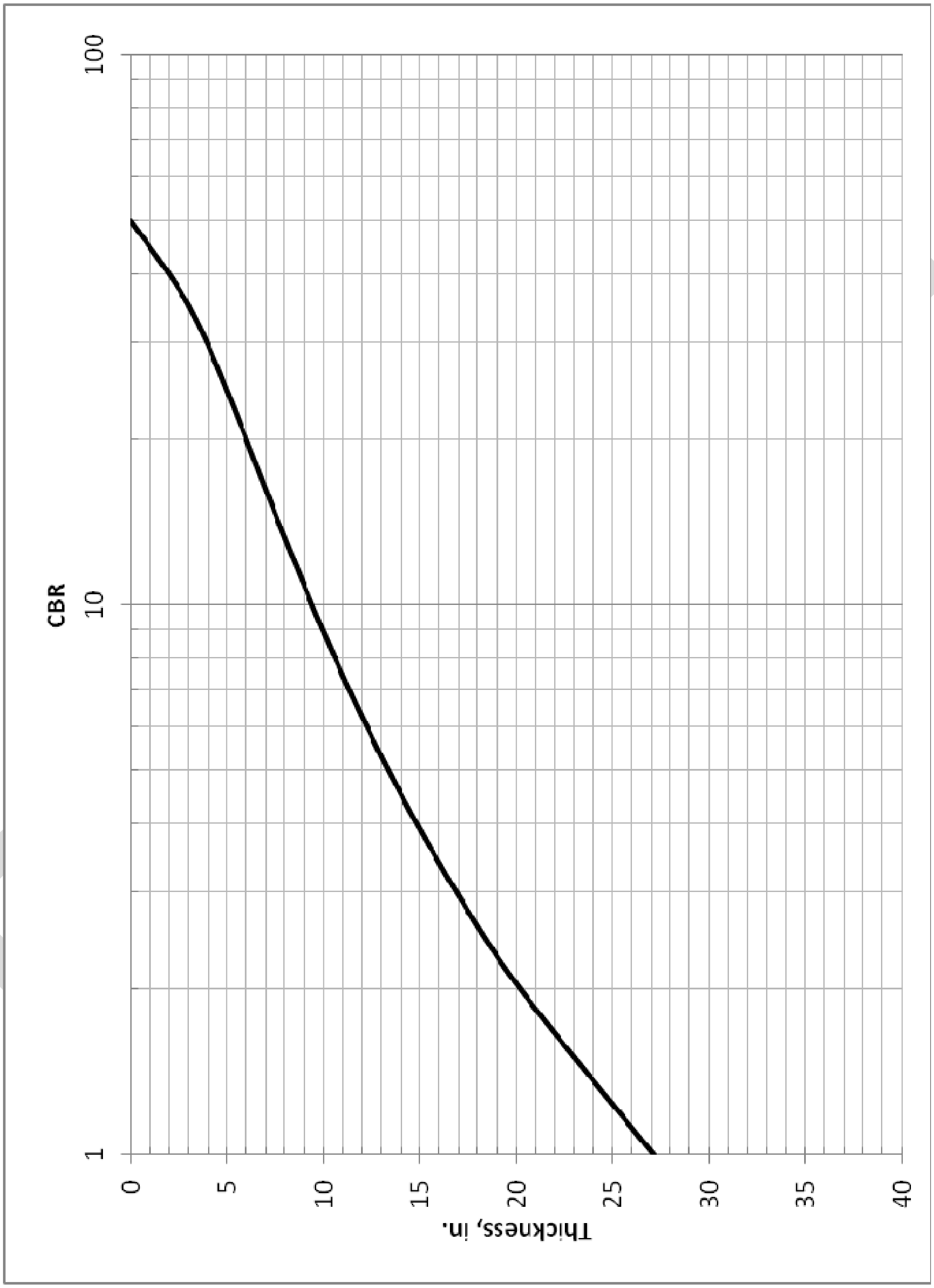


Figure 10-27. Flexible Pavement Design Curve for Flexible Shoulders



CHAPTER 11

LAYERED ELASTIC DESIGN OF FLEXIBLE PAVEMENTS

11-1 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 accounts for these two modes of structural deterioration through limiting values of 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 using computer programs and characterizing the pavement materials by the elastic constants of the modulus of elasticity and Poisson's ratio.

11-2 FLEXIBLE PAVEMENT RESPONSE MODEL

The computer program recommended for computing the pavement response is PCASE. When PCASE is used, these assumptions are made:

- The pavement is a multilayered structure, and each layer is represented by a modulus of elasticity and Poisson's ratio.
- The interface between layers is continuous; that is, the friction resistance between layers is greater than the developed shear force.
- The bottom layer is of infinite thickness.
- All loads are static, circular, and uniform over the contact area.

11-3 DESIGN DATA

11-3.1 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.

11-3.1.1 Pavement Temperature

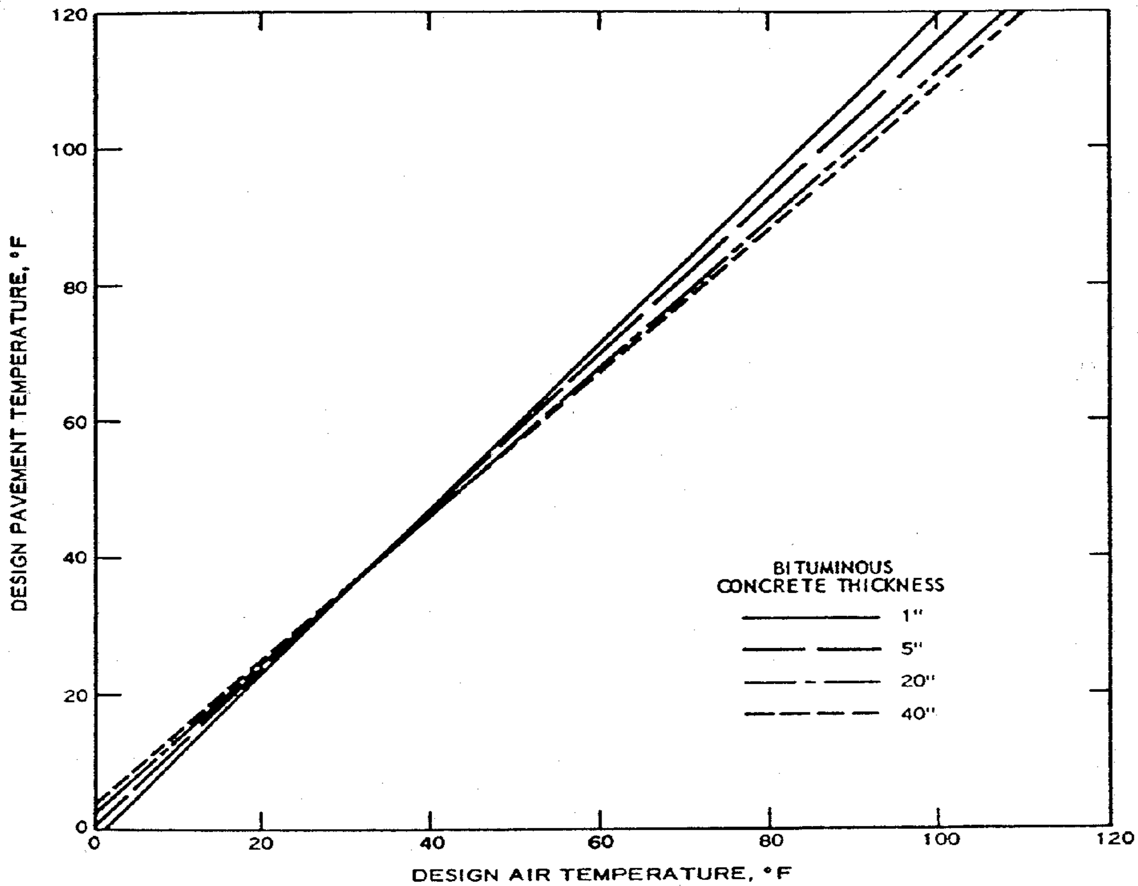
The design procedure requires determining a design pavement temperature for consideration of vertical compressive strain at the top of the subgrade and horizontal tensile strain at the bottom of cement- or lime-stabilized layers, and a different design pavement temperature for consideration of the fatigue 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 request from NOAA at 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 provided in the annual summary.

The determination of the design pavement temperatures for 254-mm (10-in) bituminous pavement can be demonstrated by considering the climatological data for Jackson, Mississippi. For the month of August, the average daily mean temperature is 27.5 degrees C (81.5 degrees F) and the average daily maximum is 33.6 degrees C (92.5 degrees F); therefore, the design air temperature for consideration of the subgrade strain is 30.5 degrees C (87 degrees F), and the design pavement temperature (determined from Figure 11-1) would be approximately 37.8 degrees C (100 degrees F). For consideration of bituminous fatigue, the design air temperature for August in Jackson is 27.5 degrees C (81.5 degrees F), resulting in a design pavement temperature of approximately 33.3 degrees C (92 degrees F). These design pavement temperatures are determined for each of the traffic periods. Temperature data for Jackson, Mississippi (from *Local Climatological Data, Annual Summary with Comparative Data*), are shown in Table 11-1.

Table 11-1. Temperature Data for Jackson, Mississippi

Month	Temperature, degrees C (degrees F)	
	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)

Figure 11-1. Temperature Relationships for Selected Bituminous Concrete Thickness



CONVERSION FACTOR

°C = 0.555 x (°F - 32)
MILLIMETERS = 25.4 x INCHES

11-3.1.2 Thaw Periods

The effects of temperature on subgrade materials are considered only with regard to frost penetration. The basic requirement of frost protection is provided in section 20-10. 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.

11-3.1.3 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 that the subgrade will not reach saturation, 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.

11-3.2 Traffic Data

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

11-3.2.1 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. To convert operations to strain repetitions, use the concept of effective gear print. 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 the 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 the bound material is an AC or stabilized layer. Thus, for a pavement having 76 mm (3 in) of asphalt and 381 mm (15 in) 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 mm (21 in), and with respect to the strain at the bottom of the asphalt, the effective thickness would be 2×76 (2×3), or 152 mm (6 in). With the determination of the effective thickness, the gear print is computed as illustrated in Figure 11-2 and Figure 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 Equation 11-1:

$$F = \frac{3 \bullet (B - T_w) - t_e}{B - T_w} \tag{11-1}$$

11-3.2.1.1 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 m [1,000 ft]), the distribution has a wander width of approximately 178 mm (7 in), and traffic on runway interiors has a wander width of approximately 355 mm (14 in). (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 multiplied by *F* gives the number or fraction of the effective strain repetitions at a point in the pavement for each aircraft operation.

11-3.2.1.2 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 mm (0 in), 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 be computed easily for single, twin, single-tandem, twin-twin, twin-tandem, or other gears.

11-3.2.1.3 The distribution of the pass-to-strain conversion percentages as a function of point location and effective thicknesses is provided in Appendix B, Section 4. These pass-to-strain conversion percentages can be used to convert, for any point location, the number of aircraft operations to effective strain repetitions.

Figure 11-2. Computation of Effective Gear Print for Single Gear

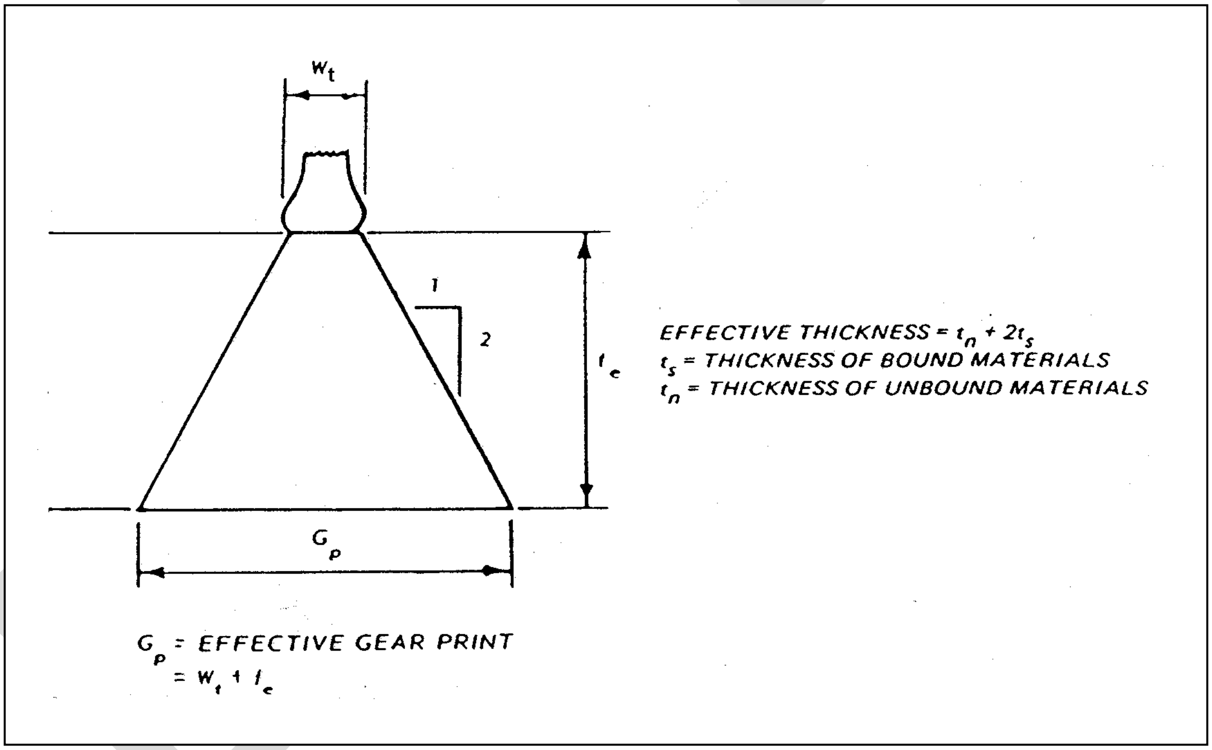
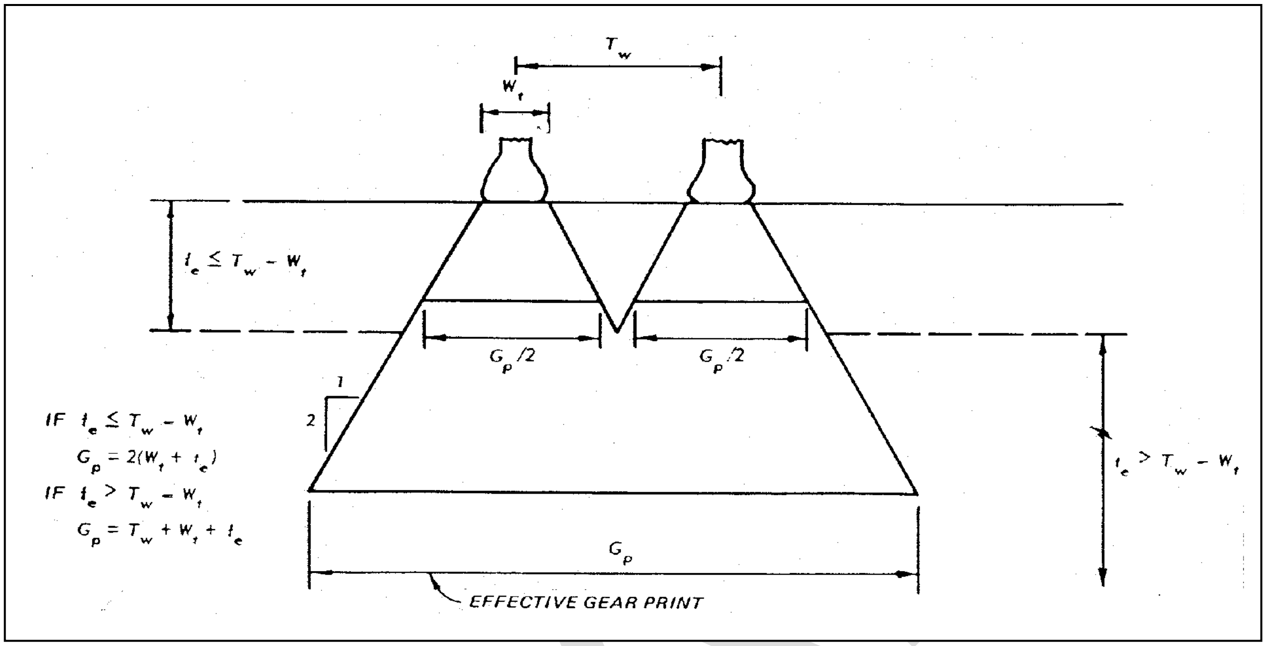


Figure 11-3. Computation of Effective Gear Print for Twin Gear



11-3.2.2 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, that is,

$$r = \sqrt{\frac{L}{\pi p}} \qquad (11\text{-}2)$$

where

r = radius of loaded area, mm (in)

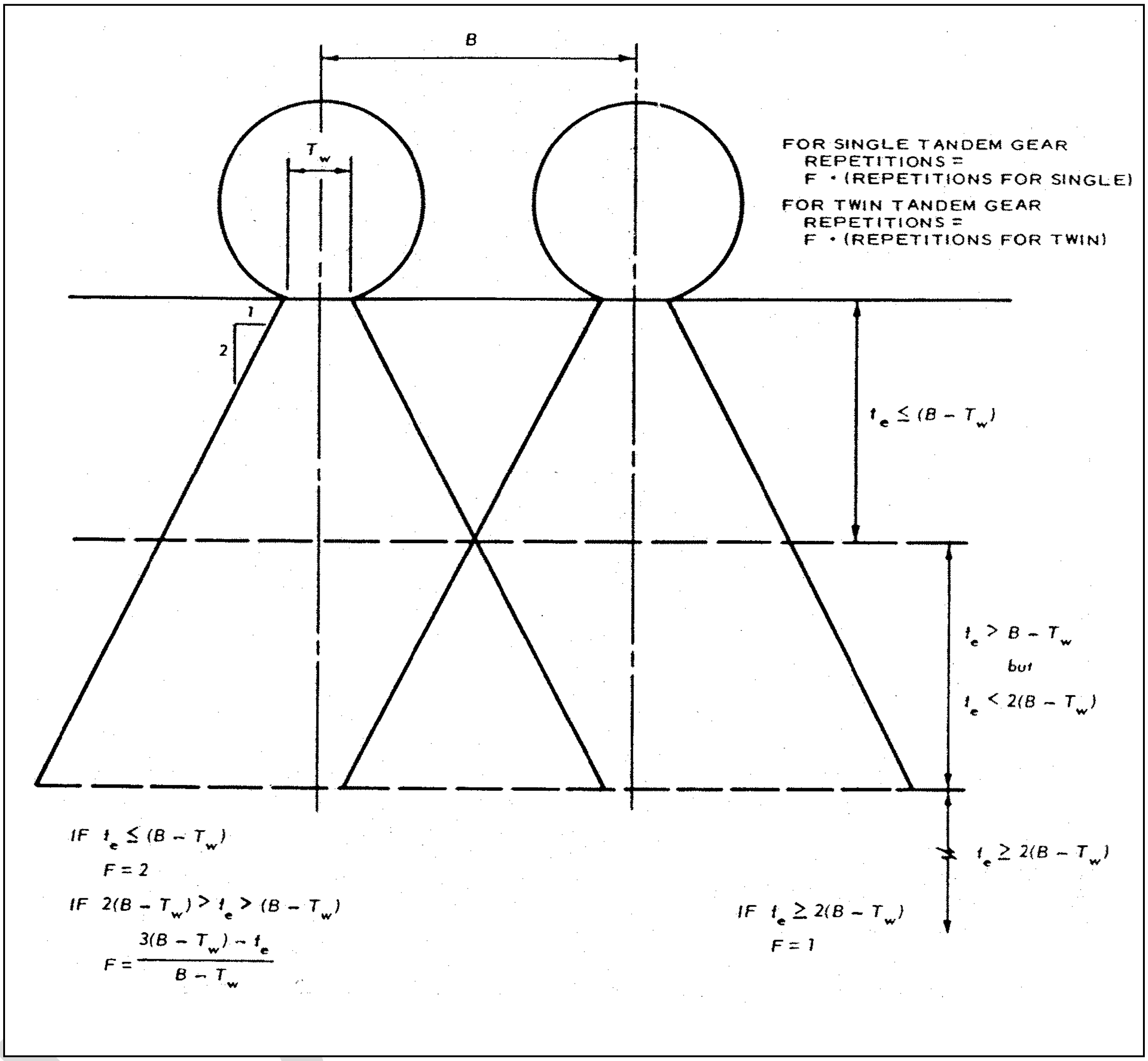
L = load per tire, newtons (N) (lb)

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 using only the tires on one main landing gear will usually suffice. 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.

Figure 11-4. Computation of Repetition Factor for Tandem Gear



11-3.2.3 **Traffic Grouping**

The traffic is grouped so that within each group, each individual pass of an aircraft will cause damage similar to 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 the design effort considerably, and reducing traffic to as few groups as possible is advantageous. Grouping of the aircraft by similar pass-to-strain conversion percents has already been accomplished in Appendix B, Section 5. 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, reducing the aircraft operations to a single group may be possible. 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, the concept of the cumulative damage factor must be used in the design process.

11-4 MATERIAL CHARACTERIZATION

Characterization of the pavement materials requires quantifying 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. 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.

11-4.1 Modulus of Elasticity

11-4.1.1 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 preparing the sample is provided in Appendix B, Section 6, and the procedure for conducting the repetitive triaxial test is provided in Appendix B, Section 7.

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

An indirect method of obtaining an estimated modulus value for bituminous concrete is presented in detail in Appendix B, Section 8. Use of this method requires determining 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.

11-4.1.2 **Unbound Granular Base and Subbase Course Materials**
The terms "unbound granular base course material" and "unbound granular subbase course material" as used in this UFC refer to materials meeting the 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 provided in Appendix B, Section 9.

11-4.1.3 **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 any 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 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 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 materials 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.

11-4.1.3.1 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 (that is, 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.

11-4.1.3.2 Procedures for preparing specimens of, and conducting flexural beam tests on, chemically stabilized soils are presented in detail in Appendix B, Section 10.

11-4.1.3.3 The stabilized material for the base and subbase must meet the strength and durability requirements of UFC 3-250-11. The strength requirements are as summarized in Chapter 9.

11-4.1.4 **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 the soil can be 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 in this UFC, back-pressure saturation of samples compacted at optimum is recommended for developing high moisture contents in test specimens.

11-4.1.4.1 For cohesive subgrades, the resilient modulus of the subgrade will usually decrease with an increase in deviator stress; 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 B, Section 10. 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).

11-4.1.4.2 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, USACE CRREL (http://www.crrel.usace.army.mil/) can conduct tests to characterize subgrade soils subjected to freeze-thaw.

11-4.1.4.3 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.

11-4.2 **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 in this section 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

Pavement Materials	Poisson's Ratio v
Subgrade Cohesive subgrade Cohesionless subgrade	 0.4 0.3
Note: E = elastic modulus of bituminous concrete, MPa (psi)	

11-5 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 the procedures in Appendix B, Section 10. The design modulus would be the average of the moduli obtained from the testing.

11-6 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. Note 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 meet the specifications for quality will perform satisfactorily.

11-6.1 Subgrade Strain Criteria

The subgrade strain criteria were developed from the analysis of field test data. These criteria 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 Equation 11-3:

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

where

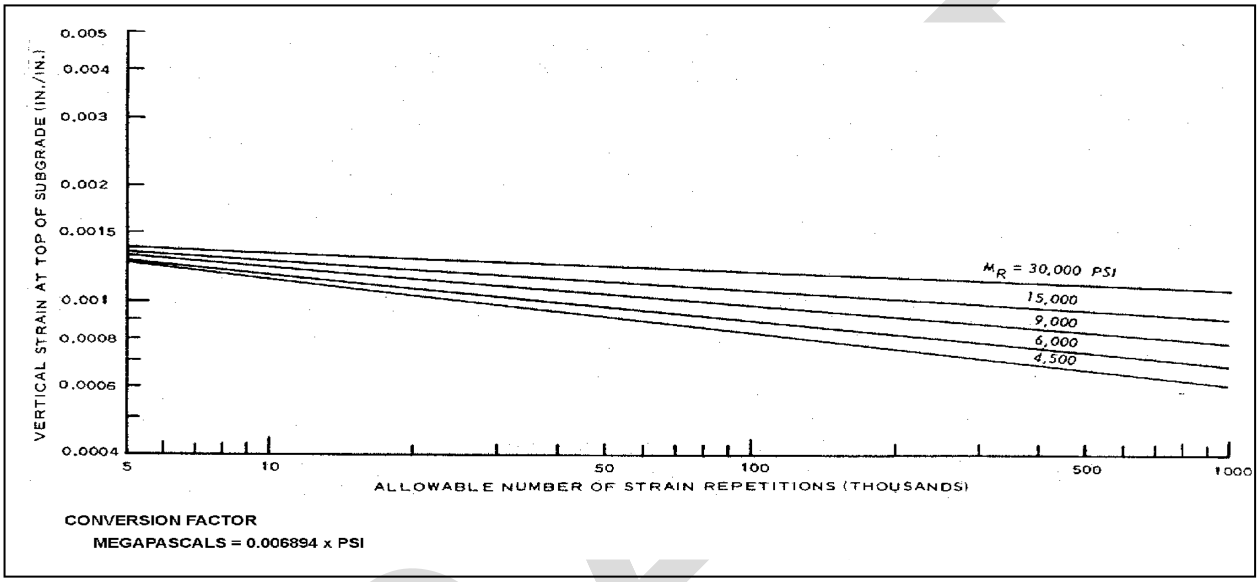
A = 0.000247+0.000245 log M_R

M_R = resilient modulus of the subgrade, MPa (psi)

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

$B = 0.0658\ M_R^{0.559}$

Figure 11-5. Design Criteria Based on Subgrade Strain



11-6.2 Asphalt Strain Criteria

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 B, Section 12. Several tests are conducted 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.

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 B, Section 12, 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 Equation 11-4:

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

where

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

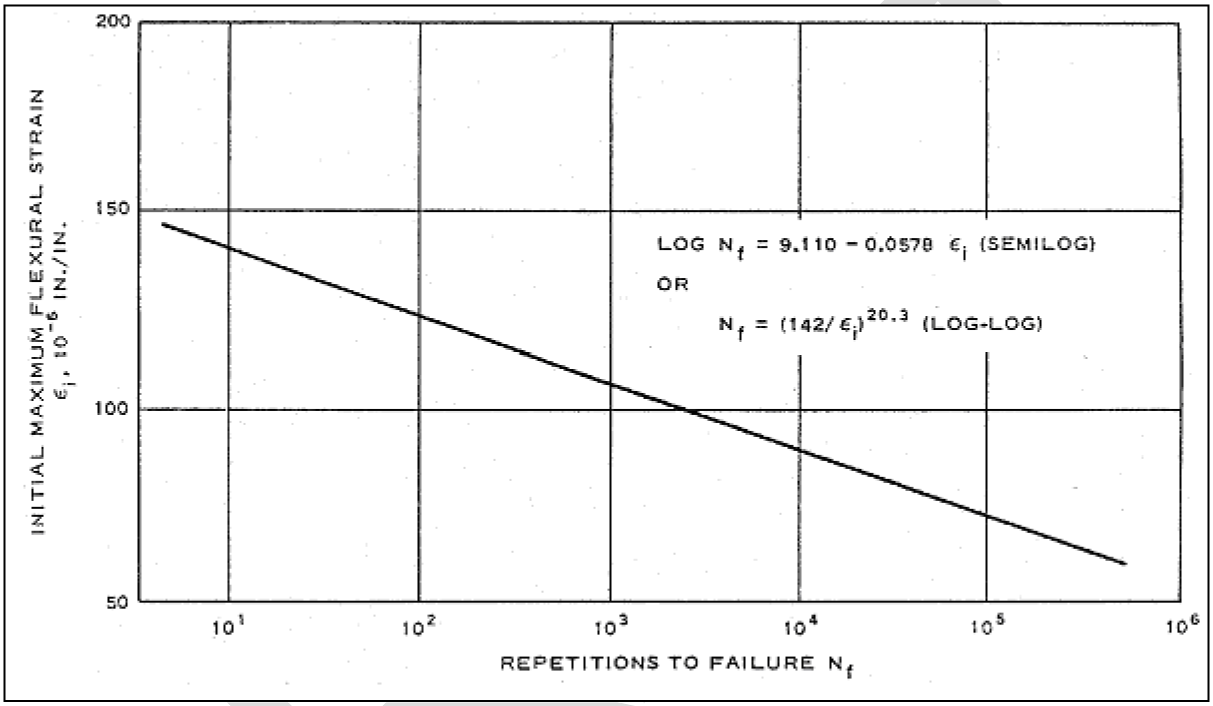
S_A = tensile strain of asphalt, mm/mm (in/in)

E = elastic modulus of the bituminous concrete, MPa (psi)

11-6.3 Chemically Stabilized Layers

For cement- and lime-stabilized materials, the criteria are to be developed using the test procedures outlined in Appendix B, Section 10. When flexural fatigue tests are not possible, then a pre-established relationship as shown in Figure 11-6 should be used.

Figure 11-6. Fatigue Life of Flexural Specimens



11-7 CONVENTIONAL FLEXIBLE PAVEMENT DESIGN

Conventional flexible pavements consist of relatively thick aggregate layers with a 76- to 127-mm (3- to 5-in) 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 to determine the thickness of pavement required to protect the subgrade. These are the steps for determining the required thickness for nonfrost areas:

(1) Determine the subgrade resilient modulus 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).

(2) Use the traffic data to determine the design loadings and repetitions of strain.

(3) Determine an initial pavement section from the minimum thickness requirements using Chapter 10 or by estimation. Determine the resilient modulus of the base and the resilient modulus of the subbase in accordance with Appendix B, Section 11.

(4) Compute the vertical strain at the top of the subgrade for each aircraft being considered in the design.

(5) Determine the number of allowable strain repetitions for each computed strain from the subgrade strain criteria.

(6) Compute the value of n/N for each aircraft and sum the values to obtain the cumulative damage factor.

(7) Adjust the assumed thicknesses 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, select the thickness that gives a damage factor of 1.0.

11-8 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 in section 11-3. In some cases, it may be possible to replace part of the subgrade with material not affected by cycles of freeze-thaw but that 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.

11-9 AC PAVEMENTS

AC pavement differs from conventional flexible pavement in that the AC is sufficiently thick to contribute significantly to the strength of the pavement. In this case, the variation in the stiffness of the AC 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 AC is at a minimum. One procedure is to determine the resilient modulus of the AC for each month, then group the months when the AC has similar resilient moduli. Thus, it may be possible to reduce the traffic to three or four groups. Also, since the AC is a major structural element, its failure due to fatigue cracking must be checked. Figure 11-7 shows the flow diagram for the design of AC pavements.

11-10 PAVEMENTS WITH A STABILIZED BASE COURSE

For a pavement with 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; that is, 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, the damage should be accumulated in the same manner as for conventional flexible pavement.

For pavements with 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 and compared 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 10-1 or Table 10-2. A flow diagram for the design of this type of pavement is shown in Figure 11-9.

11-11 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, 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, 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.

Figure 11-7. Flow Diagram of Important Steps in Design of Bituminous Concrete Pavement

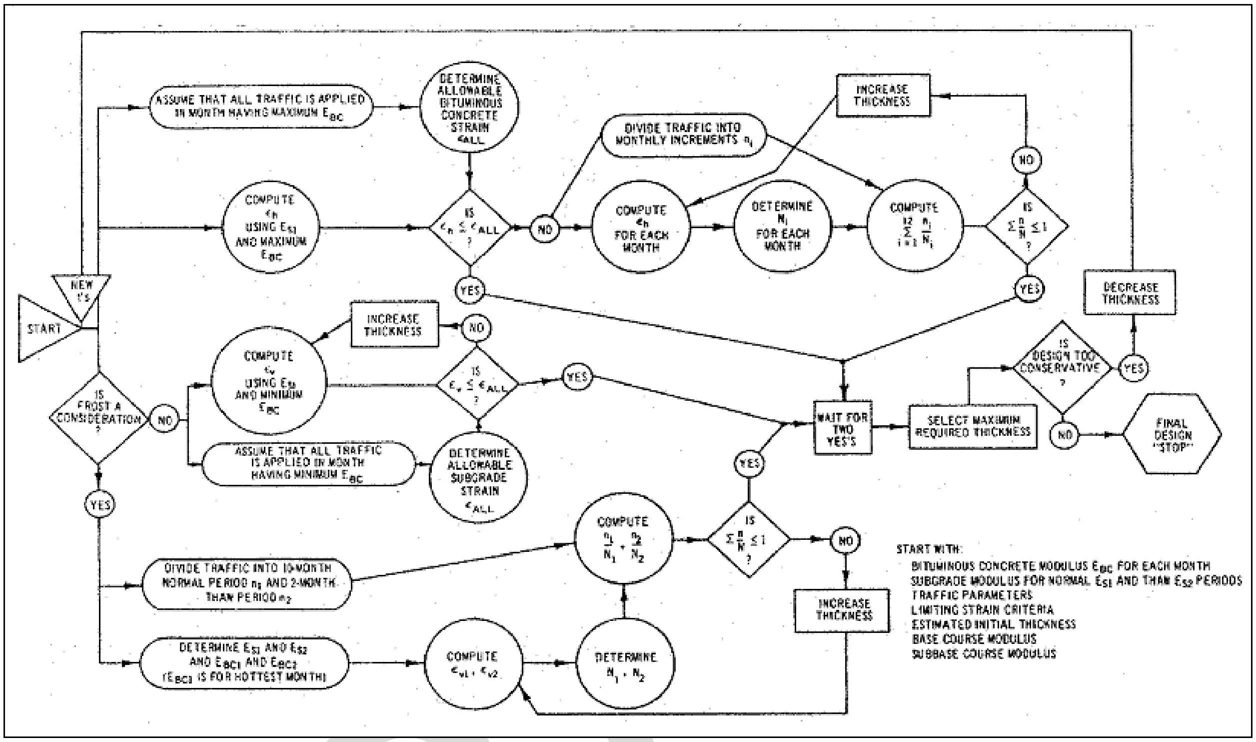
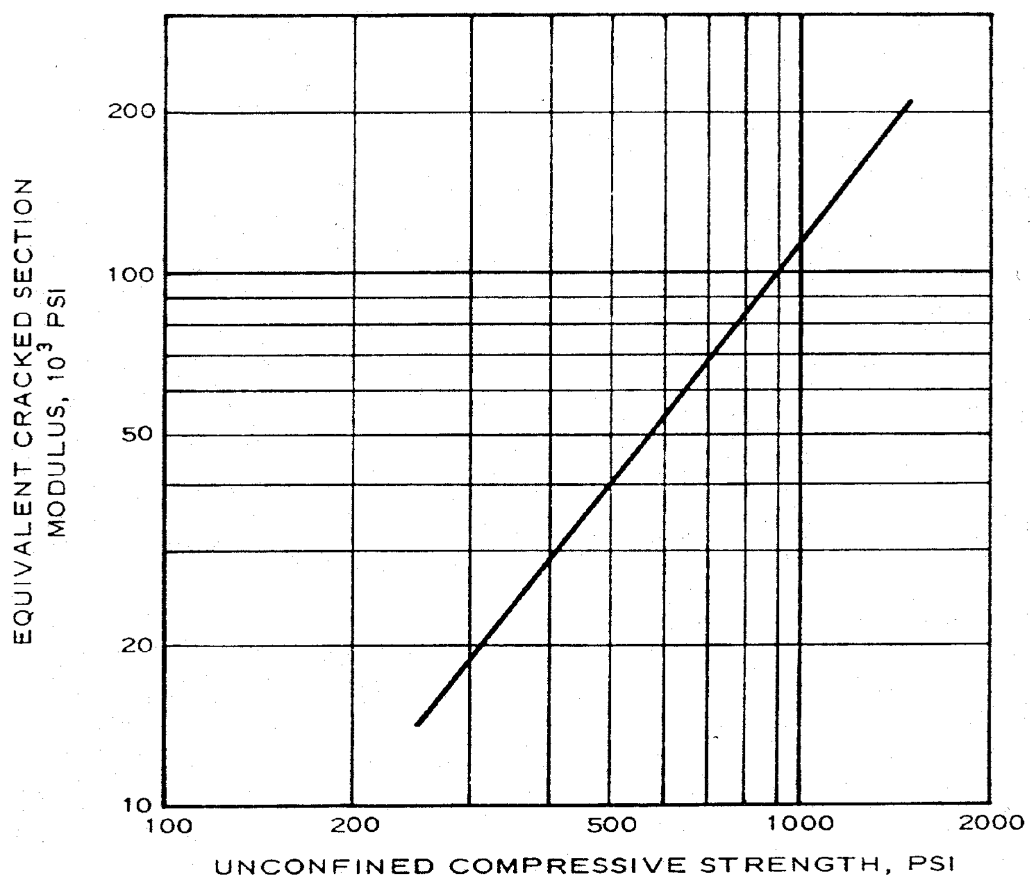


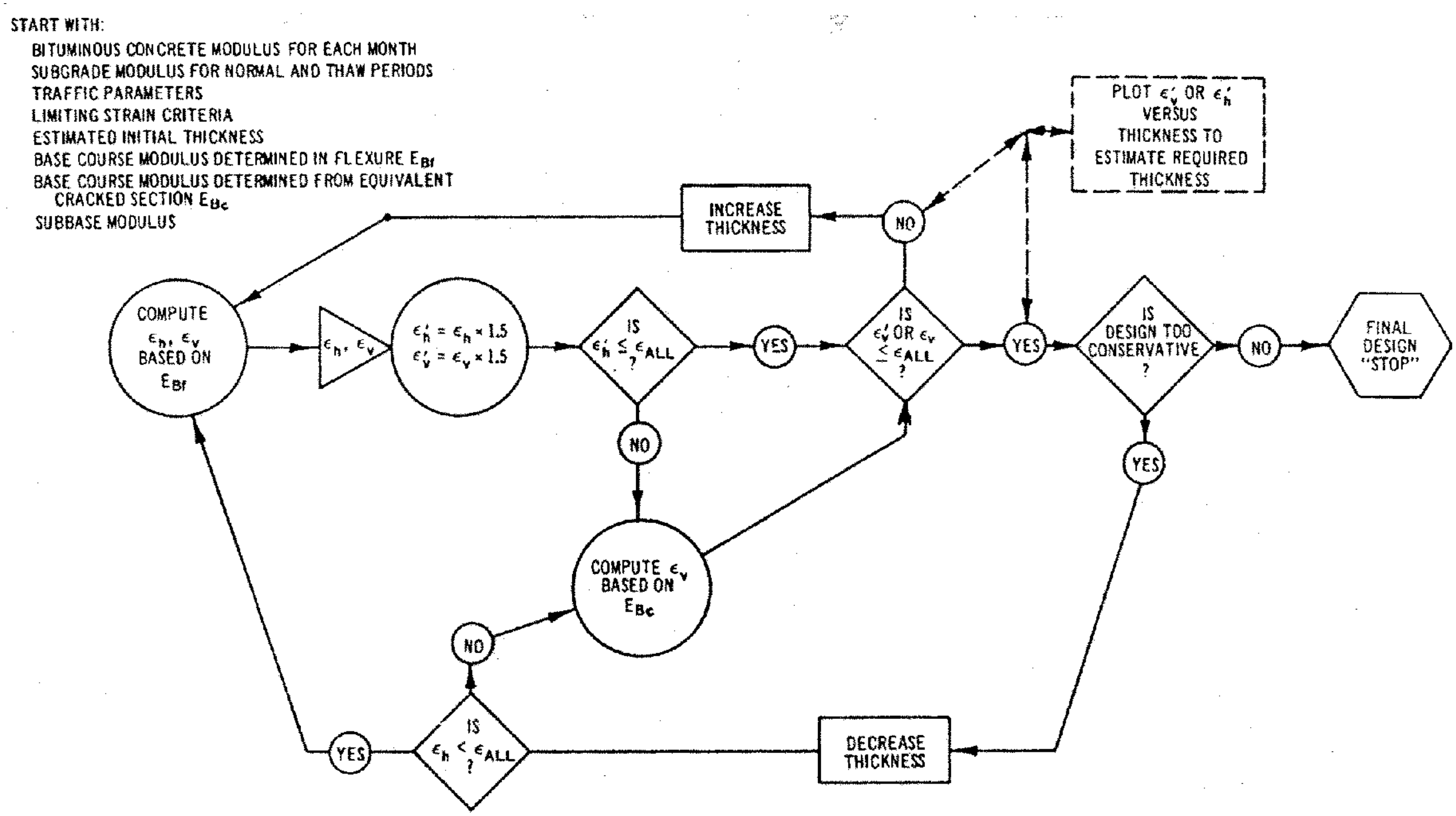
Figure 11-8. Relationship between Cracked Section Modulus and Unconfined Compressive Strength



CONVERSION FACTOR

MEGAPASCALS = 0.006894 x PSI

Figure 11-9. Flow Diagram of Important Steps in Design of Pavements with a Chemically Stabilized Base Course and an Unstabilized Subbase Course



NOTE: IF FROST IS A CONSIDERATION, IT IS RECOMMENDED THAT THE SUBGRADE MODULUS DURING THE THAW PERIOD BE USED IN THE DESIGN OF THIS TYPE OF PAVEMENT. THE CUMULATIVE DAMAGE CONCEPT CAN BE APPLIED IF DESIRED.

ϵ'_h AND ϵ'_v DENOTE STRAINS ADJUSTED FOR SHRINKAGE CRACKING.

Figure 11-10. Flow Diagram of Important Steps in Design of Pavements with Stabilized Base and Chemically Stabilized Subbase Courses

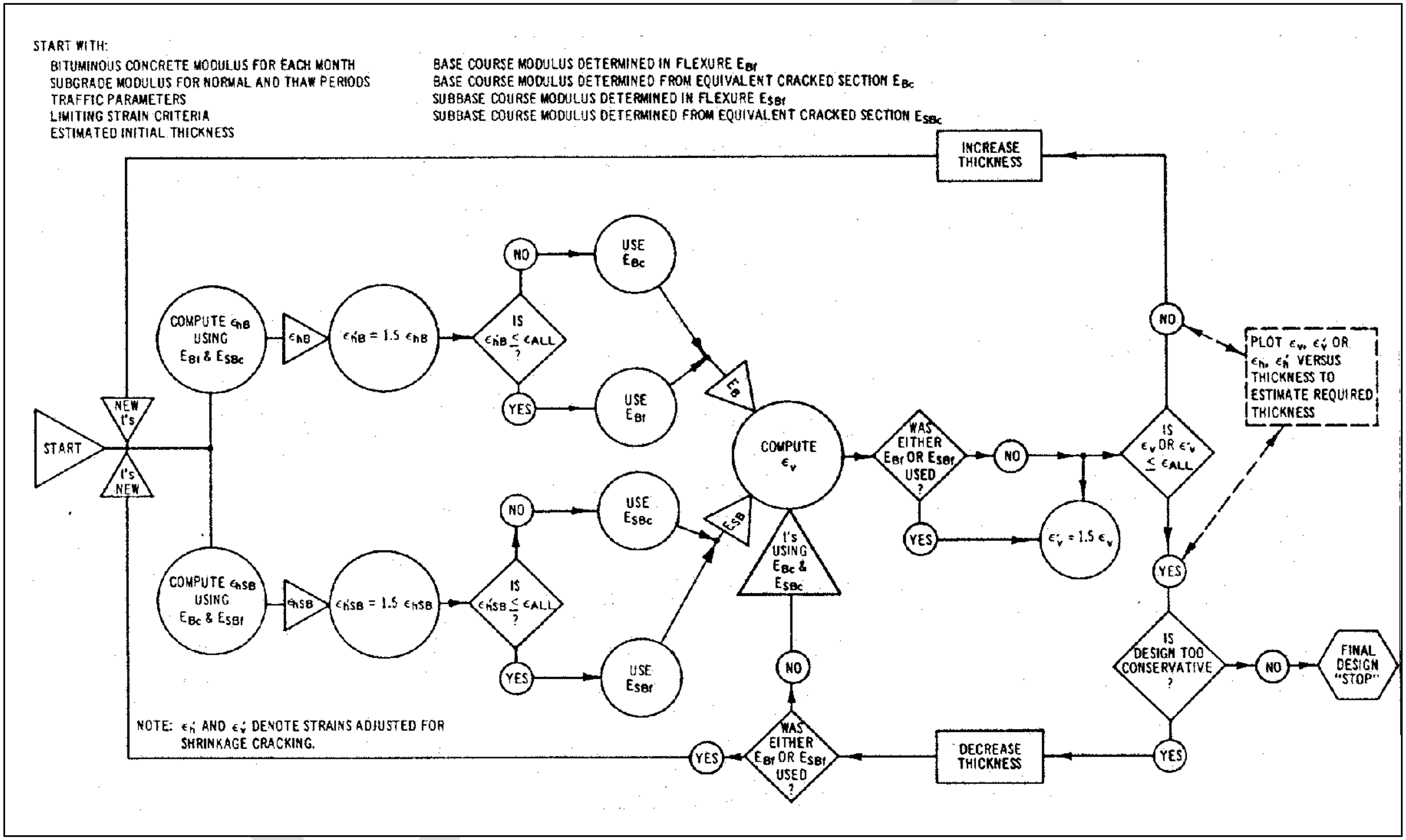
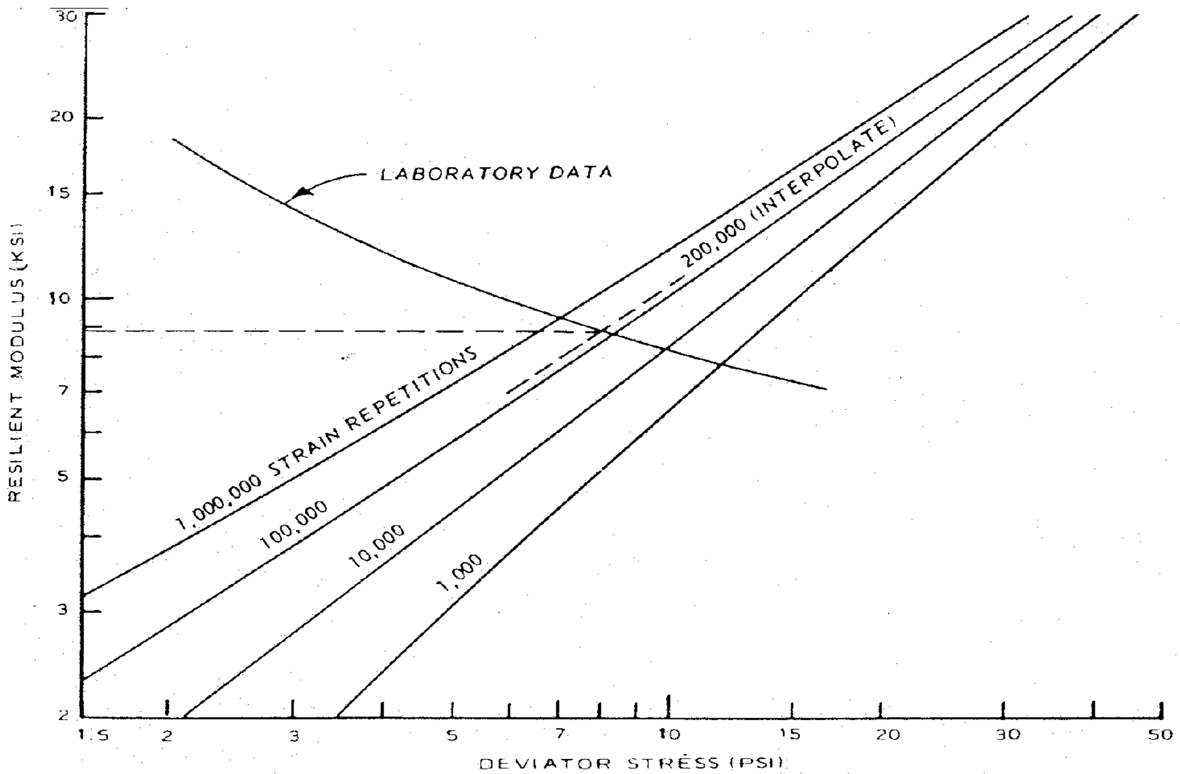


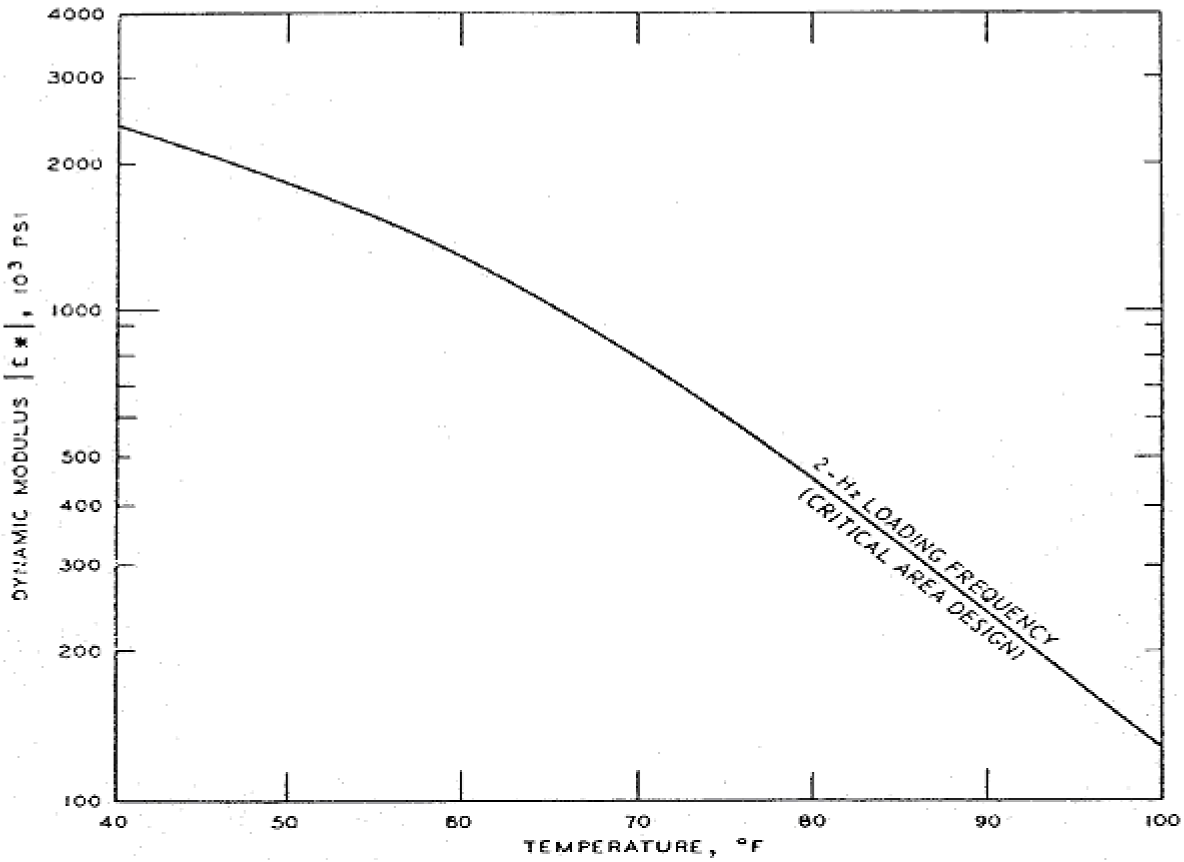
Figure 11-11. Estimation of Resilient Modulus, M_R



CONVERSION FACTORS

MEGAPASCALS = 0.006894 x PSI
MEGAPASCALS = 6.894 x KSI

Figure 11-12. Results of Laboratory Tests for Dynamic Modulus of Bituminous Concrete



CONVERSION FACTORS

MEGAPASCALS = 0.006894 x PSI
°C = 0.555 (°F − 32)

Figure 11-13. Section for Pavement Thickness of 760 Millimeters (30 Inches) for Initial Taxiway Design

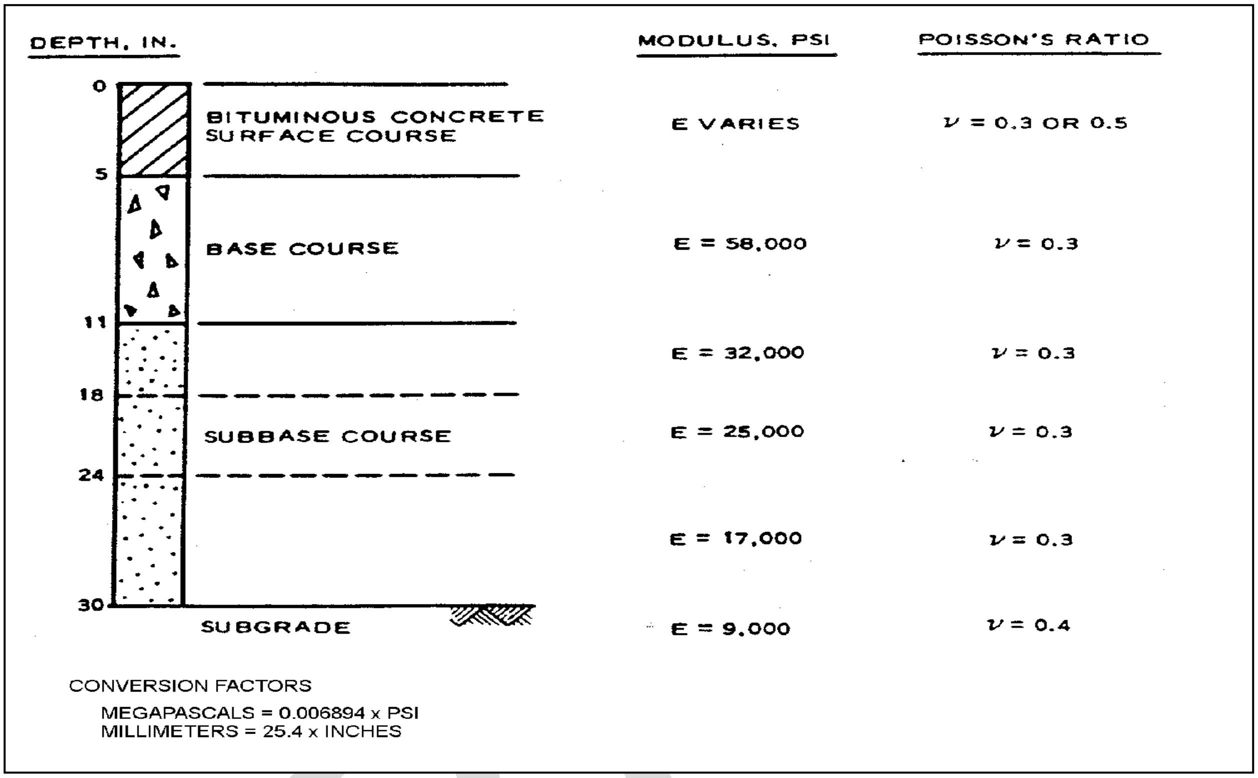
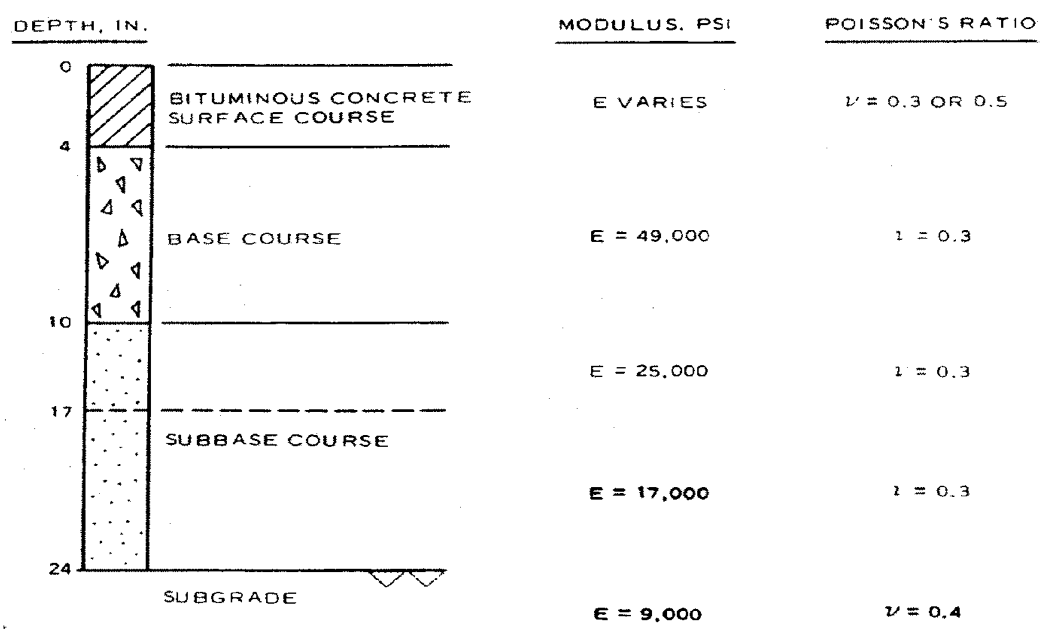


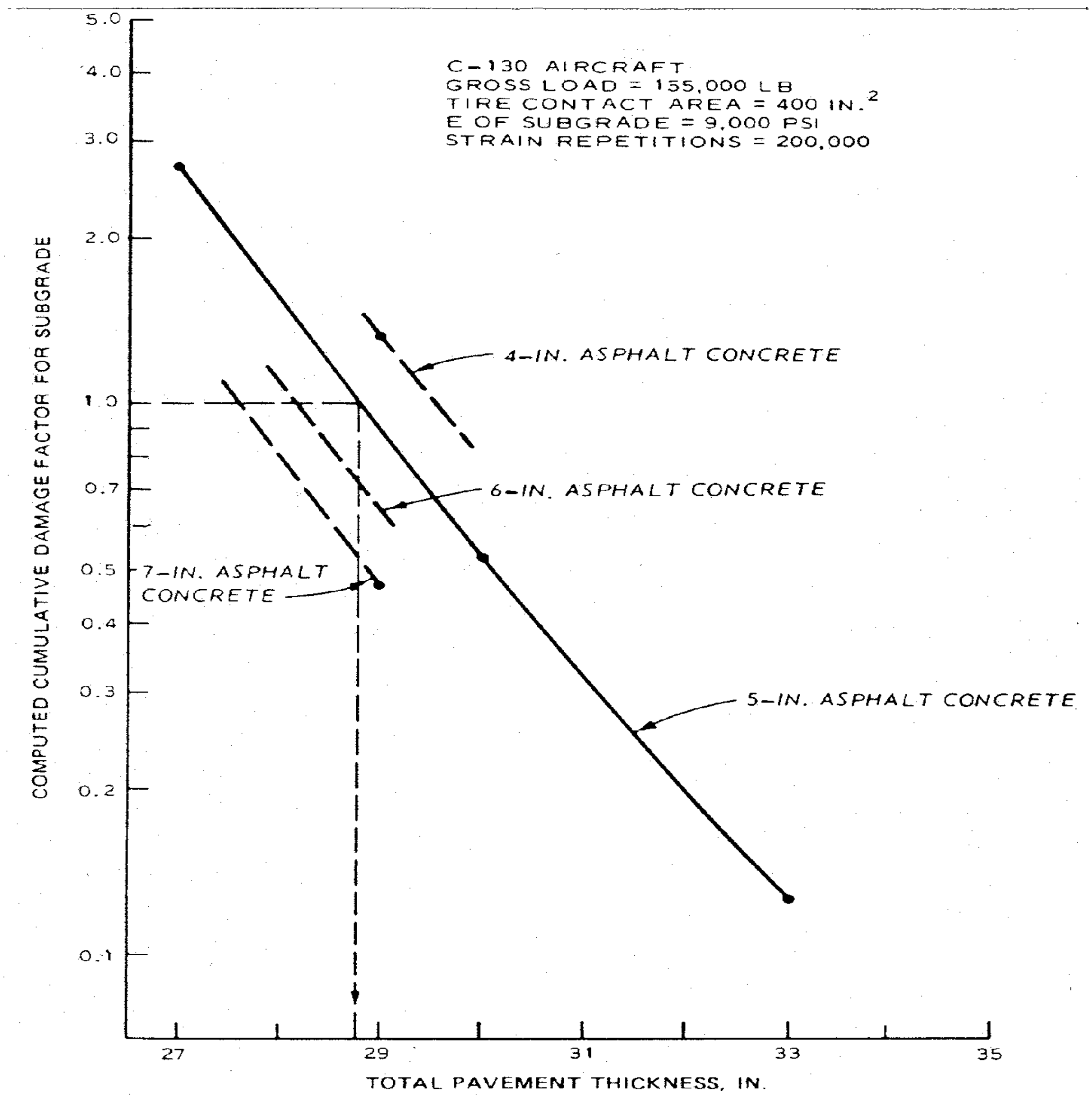
Figure 11-14. Section for Pavement Thickness of 610 Millimeters (24 Inches) for Initial Runway Design



CONVERSION FACTORS

MEGAPASCALS = 0.006894 x PSI
MILLIMETERS = 25.4 x INCHES

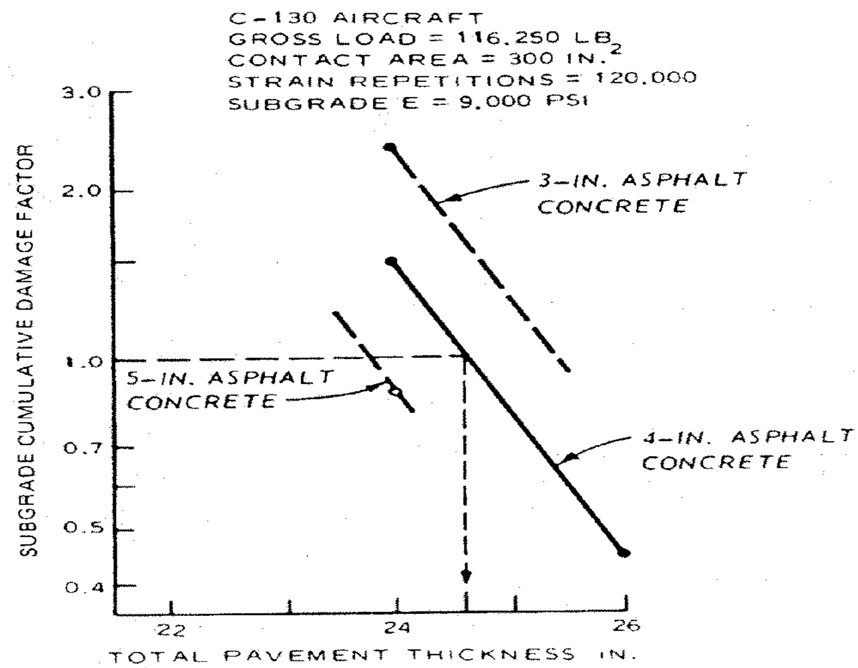
Figure 11-15. Pavement Design for Taxiways



CONVERSION FACTORS

KILOGRAMS = 0.4535 x POUNDS
MEGAPASCALS = 0.006894 x PSI
SQUARE MILLIMETERS = 645 x SQUARE INCHES
MILLIMETERS = 25.4 x INCHES

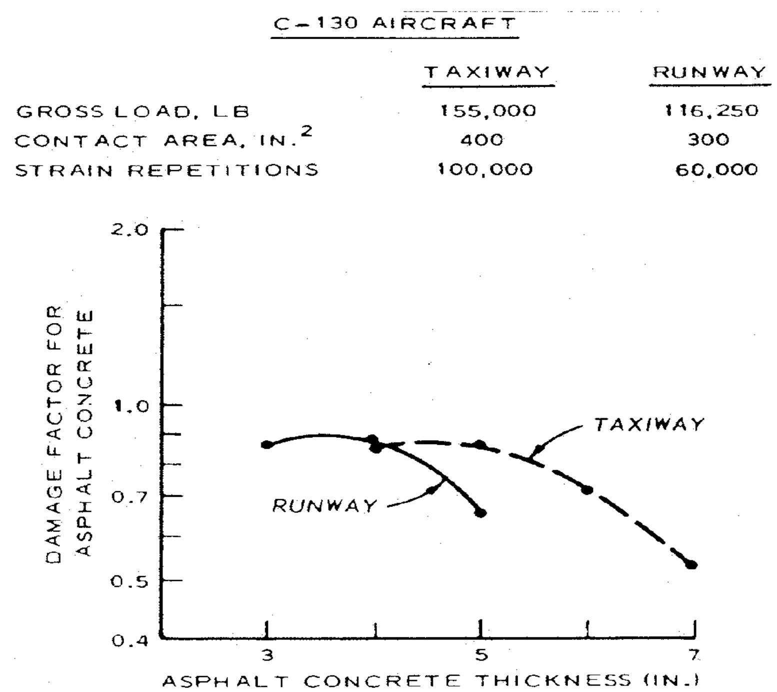
Figure 11-16. Design for Runways



CONVERSION FACTORS

KILOGRAMS = 0.4535 x POUNDS
MEGAPASCALS = 0.006894 x PSI
SQUARE MILLIMETERS = 645 x SQUARE INCHES
MILLIMETERS = 25.4 x INCHES

Figure 11-17. Design for Asphalt Concrete Surface



CONVERSION FACTORS

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

Figure 11-18. Computed Strain at the Top of the Subgrade for Taxiway Design

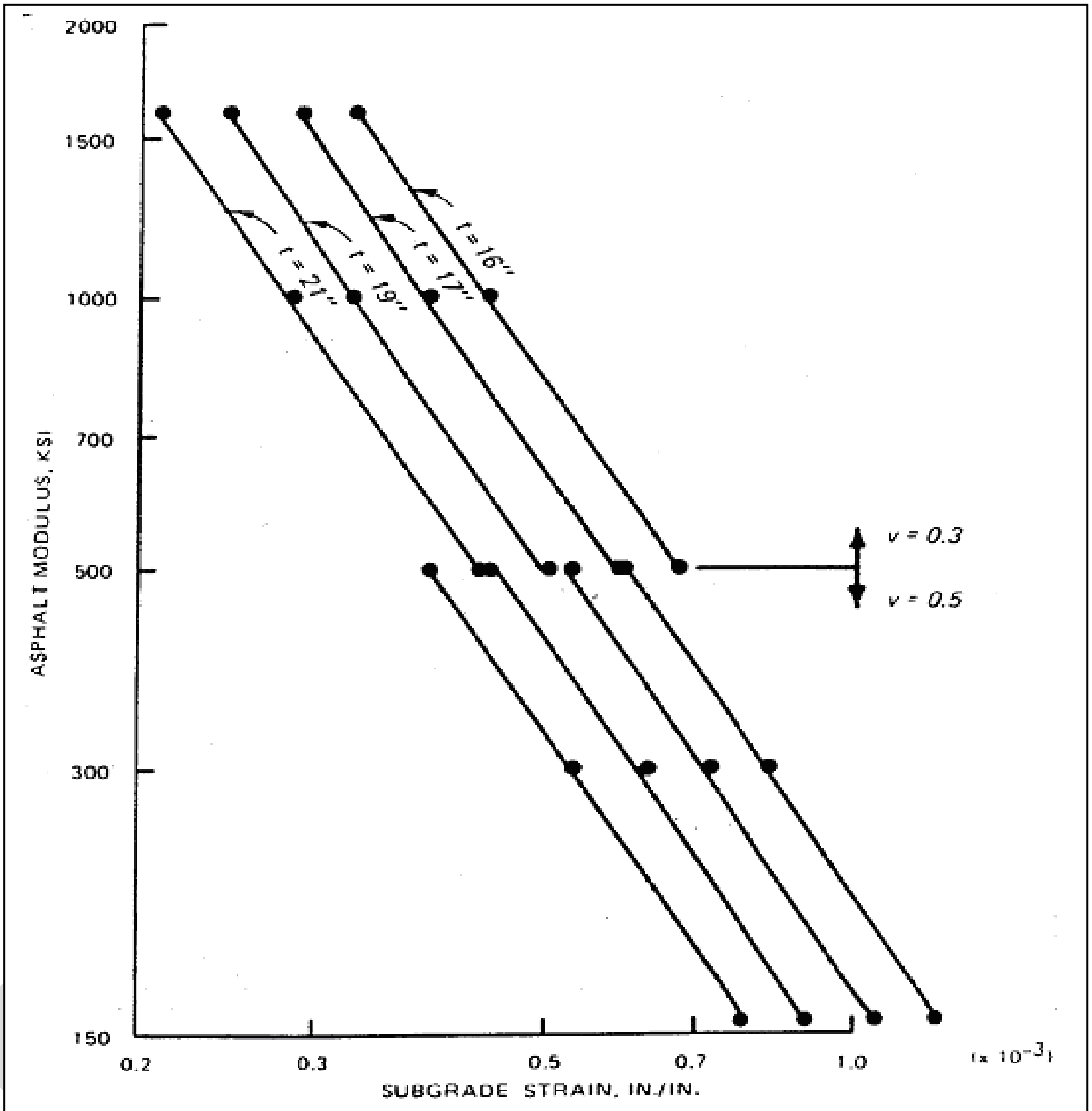
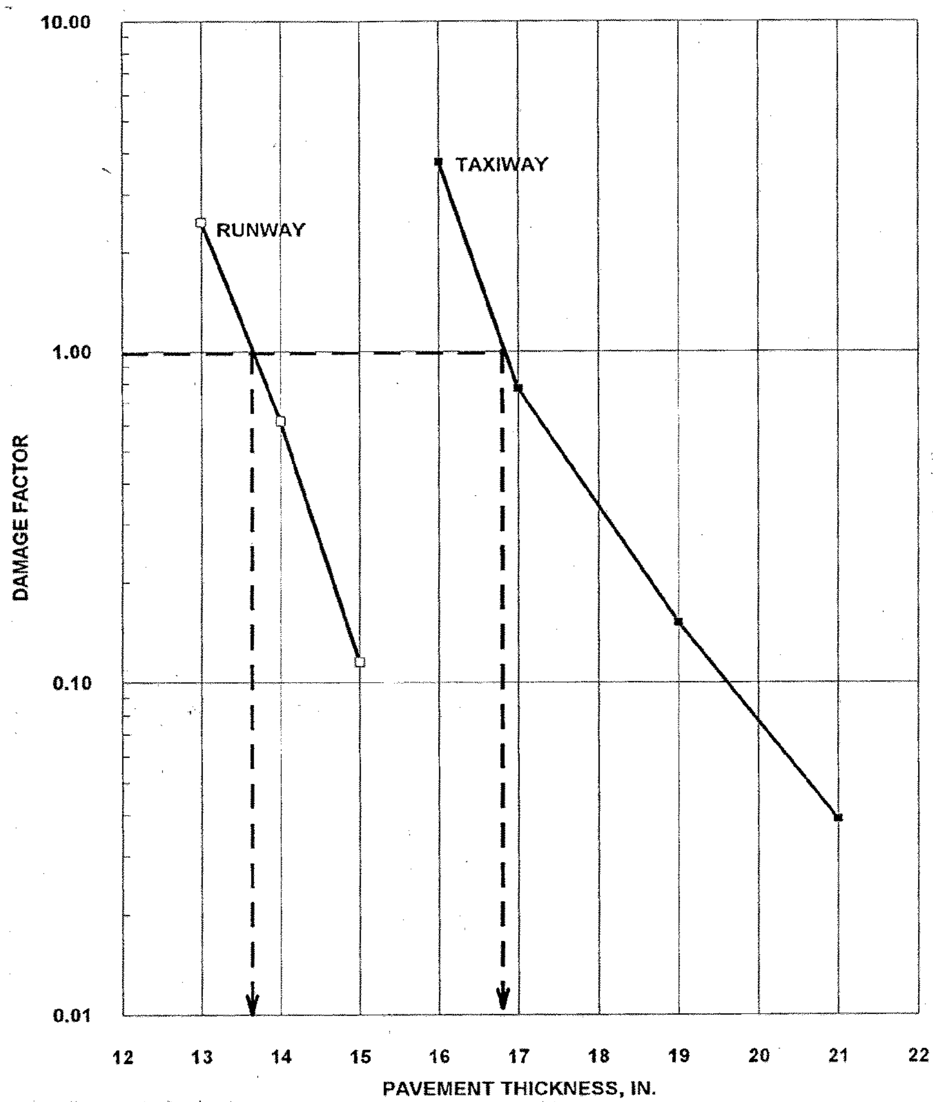


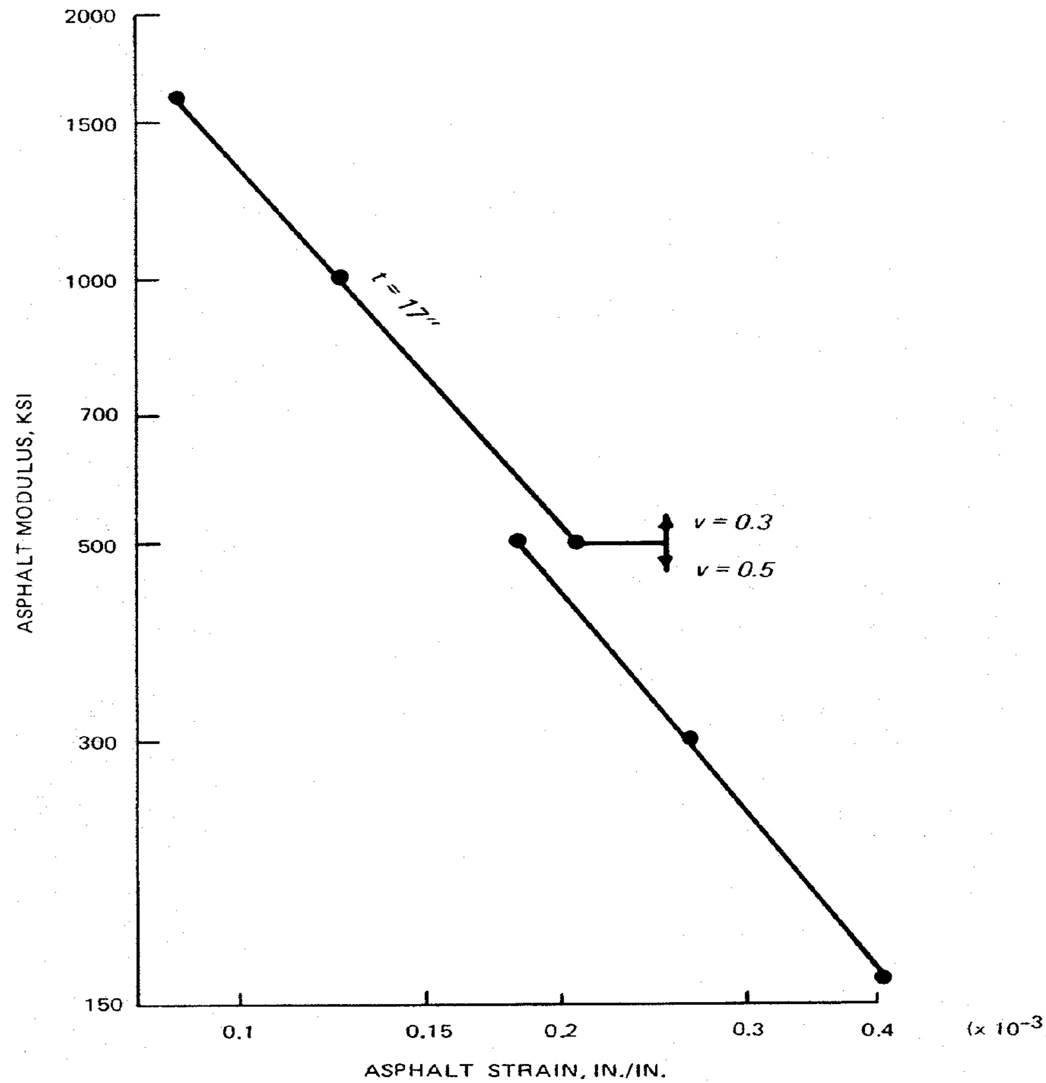
Figure 11-19. Damage Factor versus Pavement Thickness



CONVERSION FACTOR

MILLIMETERS = 25.4 x INCHES

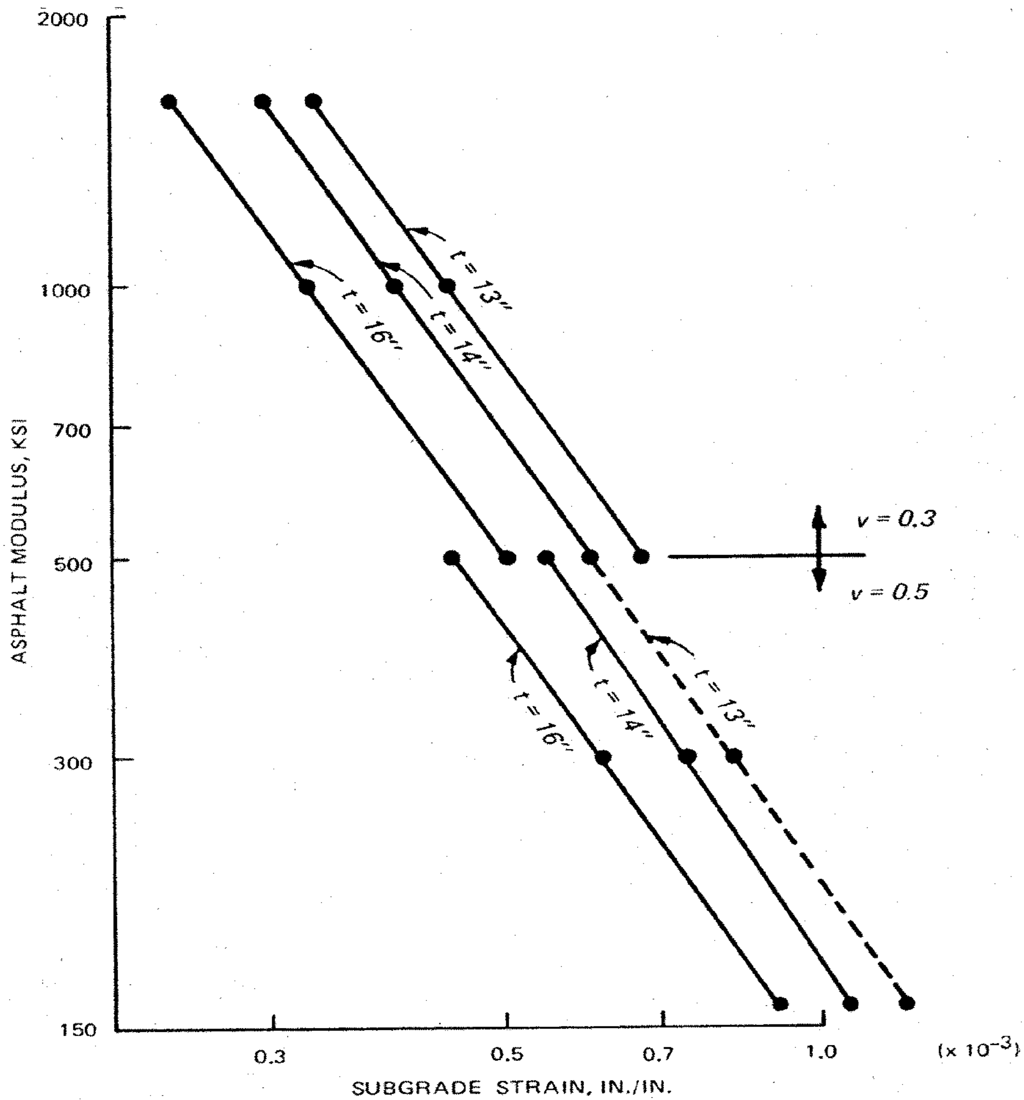
Figure 11-20. Computed Strain at the Bottom of the Asphalt for Taxiway Design



CONVERSION FACTORS

MILLIMETERS = 25.4 x INCHES
MEGAPASCALS = 6.894 x KSI

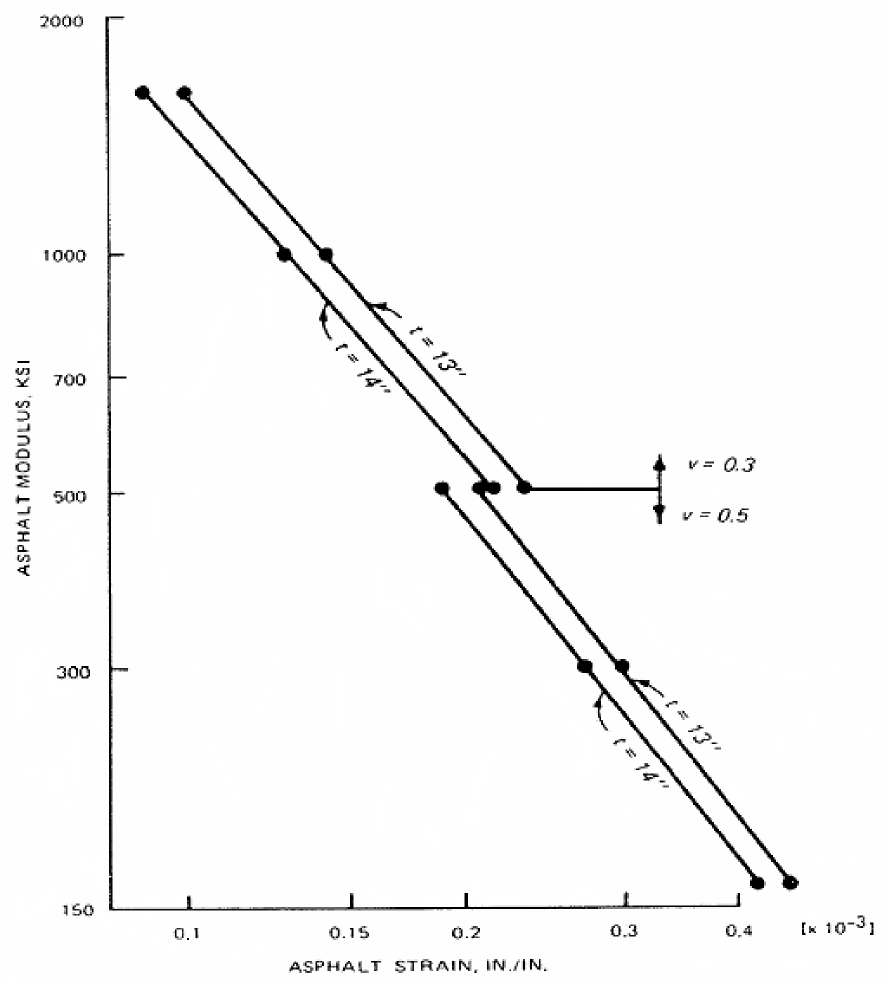
Figure 11-21. Computed Strain at the Top of the Subgrade for Runway Design



CONVERSION FACTORS

MILLIMETERS = 25.4 x INCHES
MEGAPASCALS = 6.894 x KSI

Figure 11-22. Computed Strain at the Bottom of the Asphalt for Runway Design



CONVERSION FACTORS

MILLIMETERS = 25.4 x INCHES
MEGAPASCALS = 6.894 x KSI

CHAPTER 12

PLAIN CONCRETE PAVEMENTS

12-1 BASIS OF DESIGN: ARMY, NAVY, 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 tests 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 load transfer methods such as dowels and aggregate interlock. For design, a load transfer value of 25 percent is used routinely as a reasonable approximation of the load transfer measured over time on the types of joints approved for use in Army, Navy, and Air Force airfields. The actual load transfer at a joint will vary depending on factors such as joint type, quality of construction, slab length, number of load repetitions, and temperature conditions. 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* is greater than 54 kPa/mm [200 pci]) in recognition that after the initial flexural fatigue crack forms (the 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.

Army pavements are designed using aircraft weights and passes assigned to each airfield and heliport class (see section 2-4). Air Force pavements are designed using aircraft weights and passes assigned to each airfield type (see Table 3-1). Navy pavements are designed using projected traffic, which is derived from previous air operations (available from previous structural evaluations) and new requirements. If such data do not exist, use the default traffic shown in Chapter 4.

12-2 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 paragraph 1-7 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 USACE-TSC, the Air Force MAJCOM pavements engineer, or Naval Facilities Engineering Service Center.

12-3 THICKNESS DESIGN

12-3.1 Design Curves

Figures 12-1 through 12-30 are design curves to be used in designing plain concrete pavements as defined in Chapters 2, 3, and 4. Thicknesses may also be determined using the PCASE computer program referenced in Chapter 1.

Figure 12-1. Plain Concrete Pavement Design Curves for Army Class I Helipads

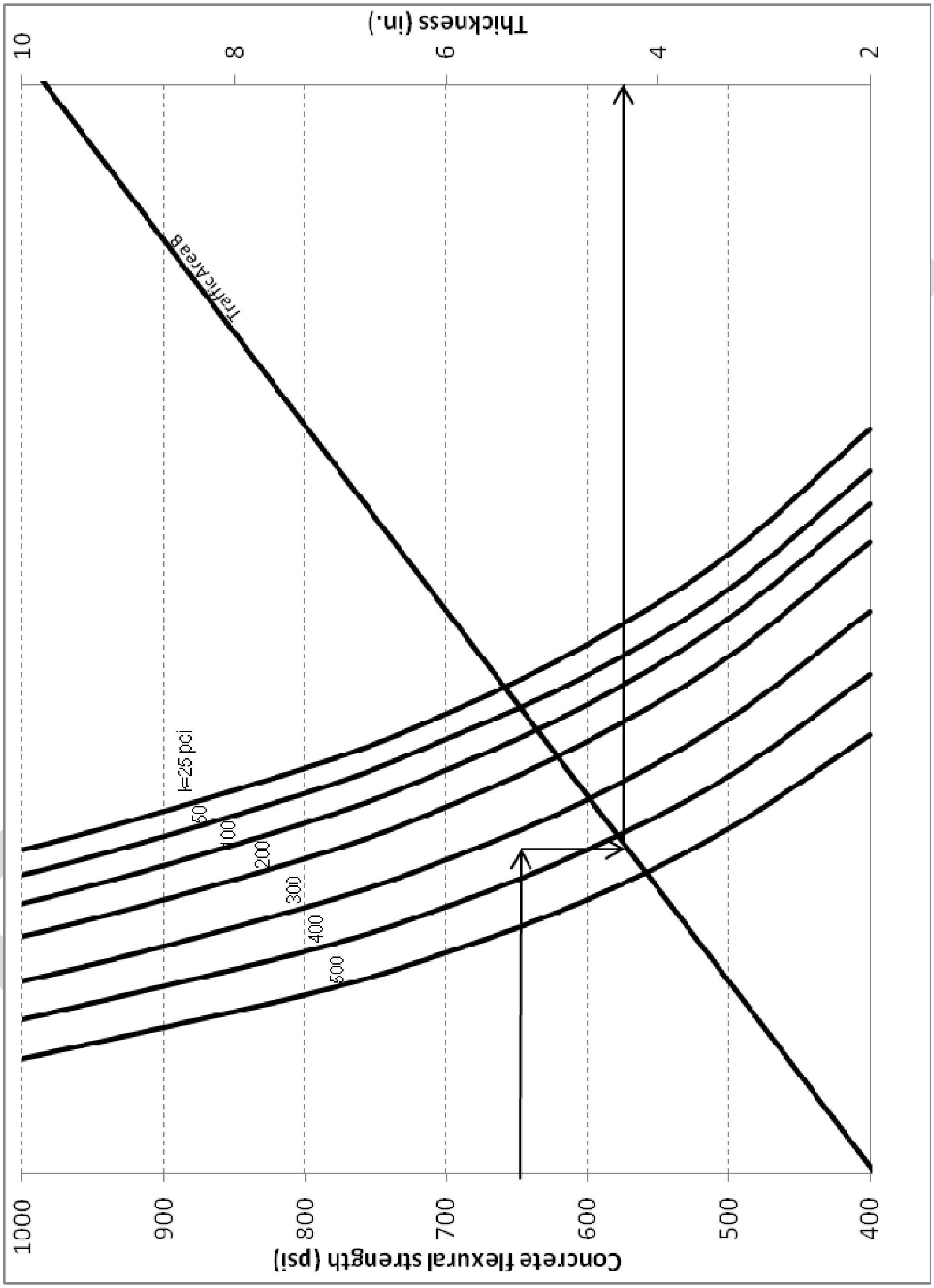


Figure 12-2. Plain Concrete Pavement Design Curves for Army Class I Heliports

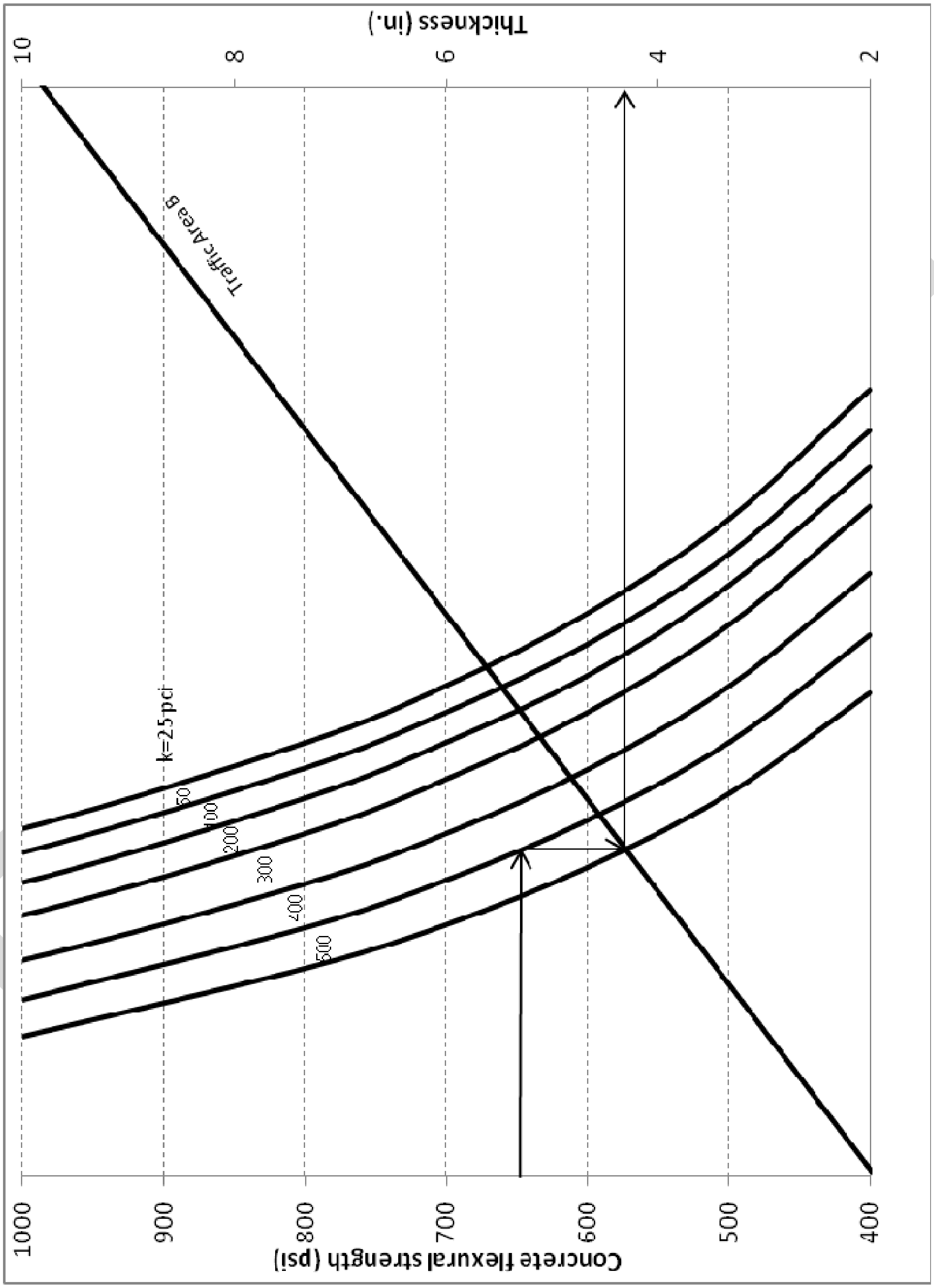


Figure 12-3. Plain Concrete Pavement Design Curves for Army Class II - VFR - Helipad

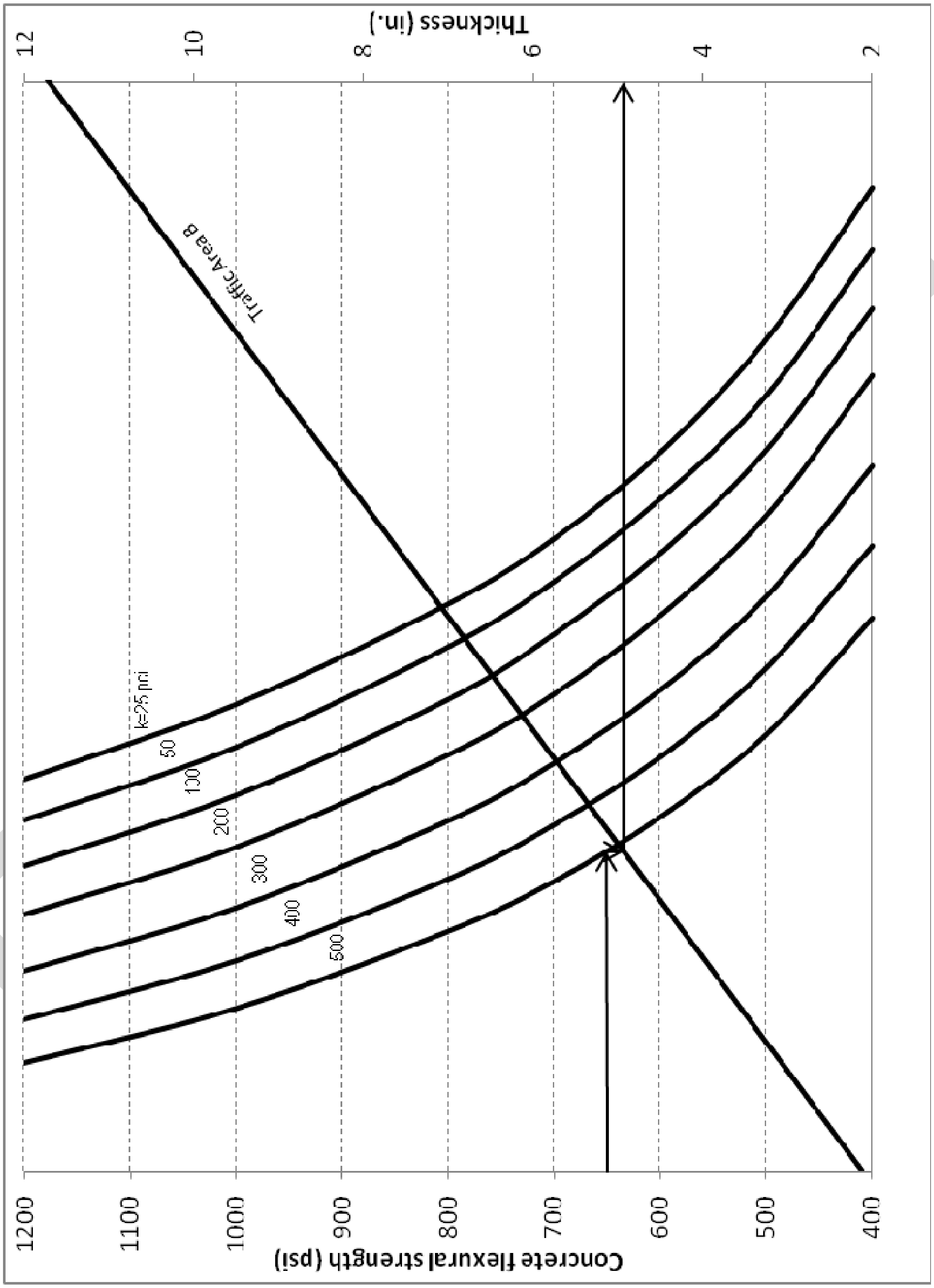


Figure 12-4. Plain Concrete Pavement Design Curves for Army Class II - VFR - Heliport

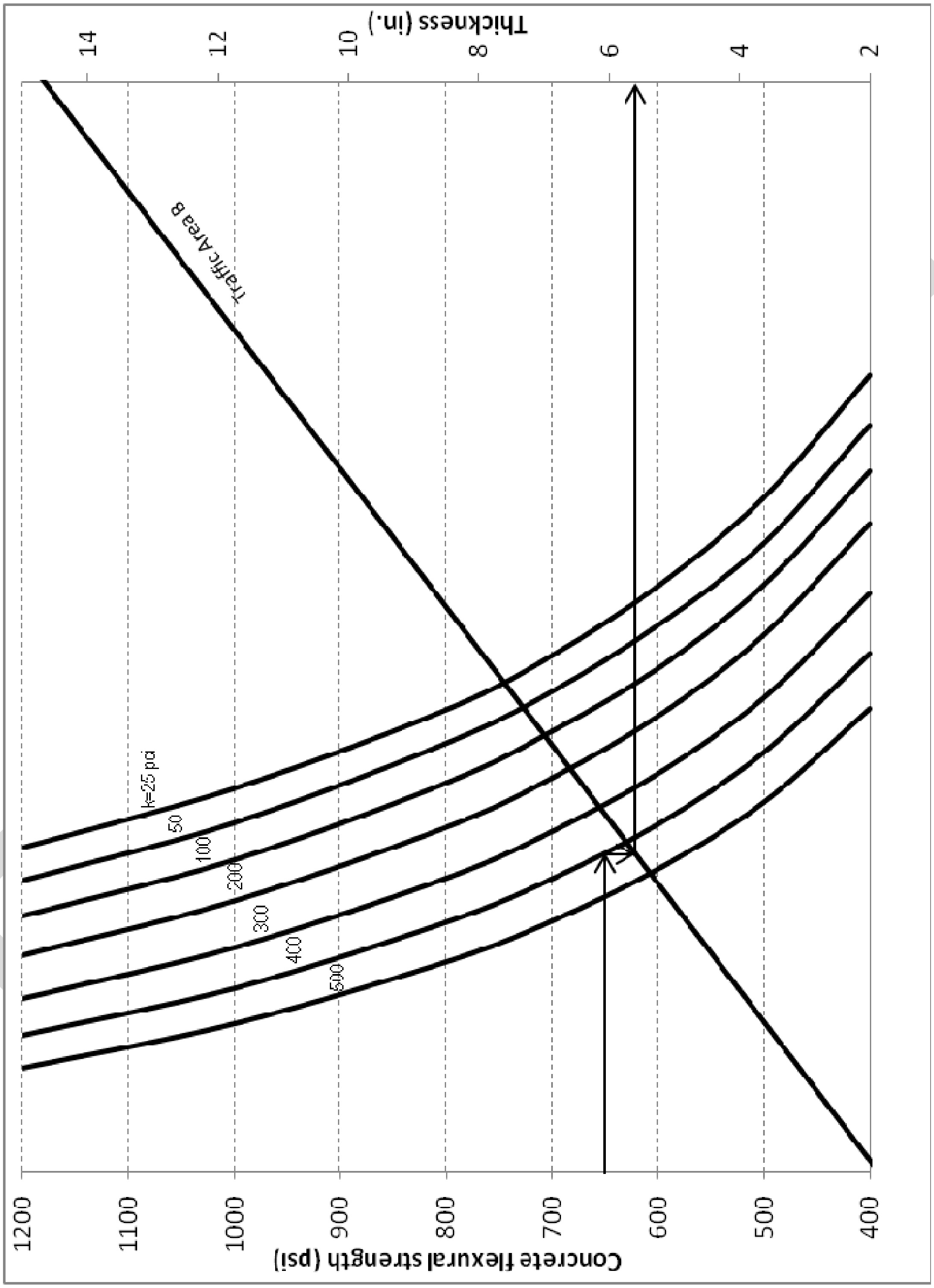


Figure 12-5. Plain Concrete Pavement Design Curves for Army Class II - IFR - Helipad

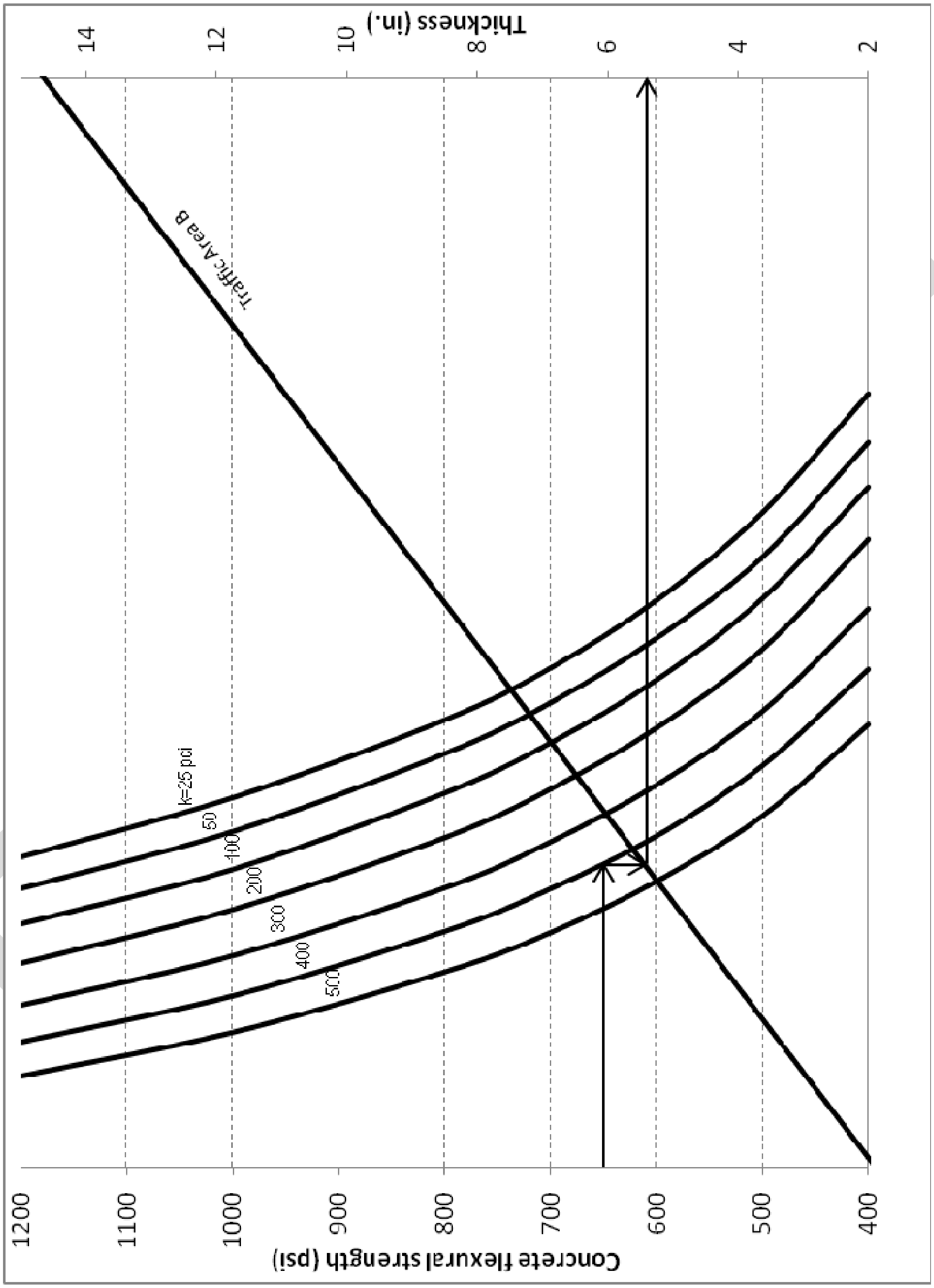


Figure 12-6. Plain Concrete Pavement Design Curves for Army Class II - IFR - Heliport

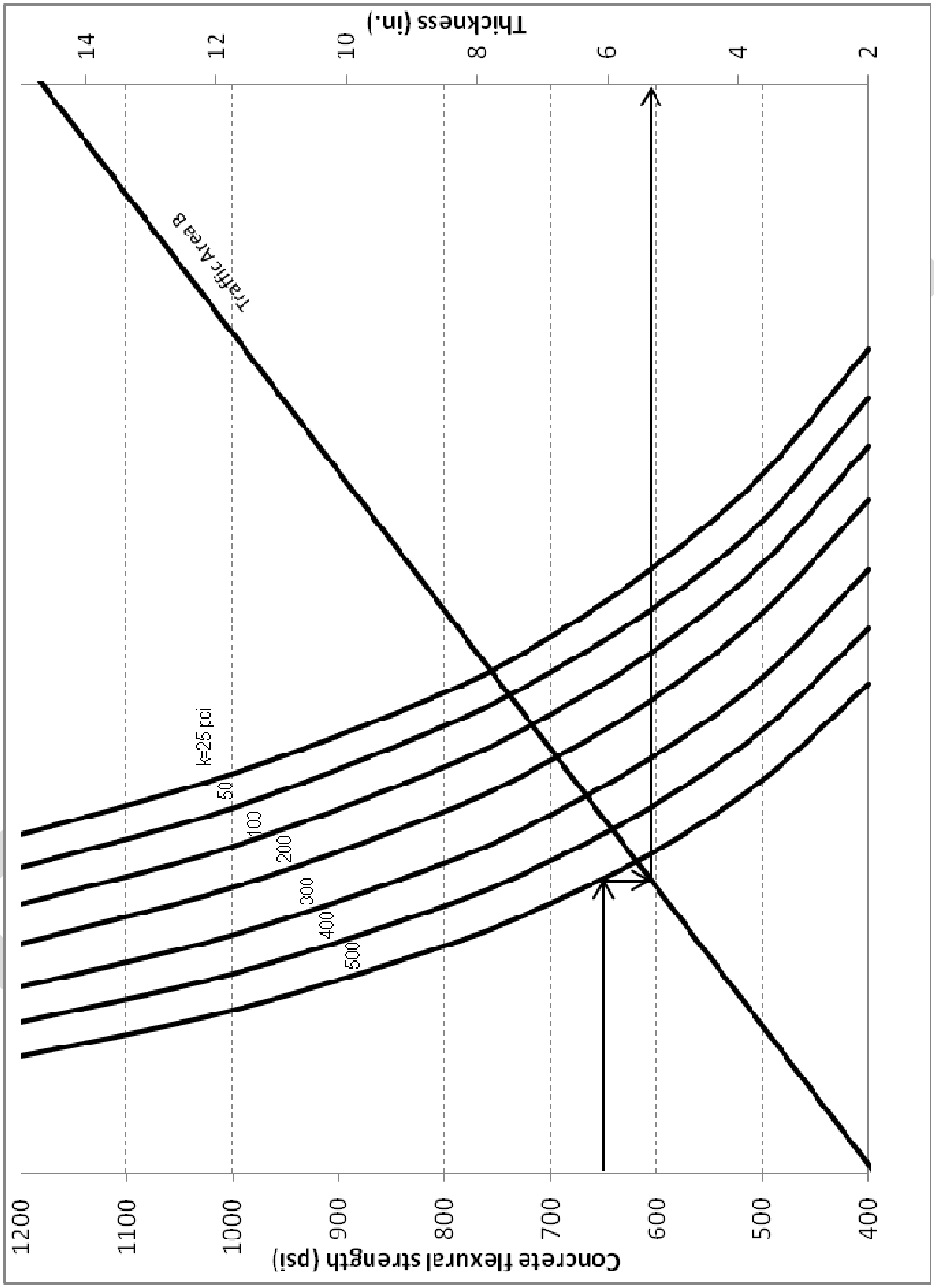


Figure 12-7. Plain Concrete Pavement Design Curves for Army Class III Airfield

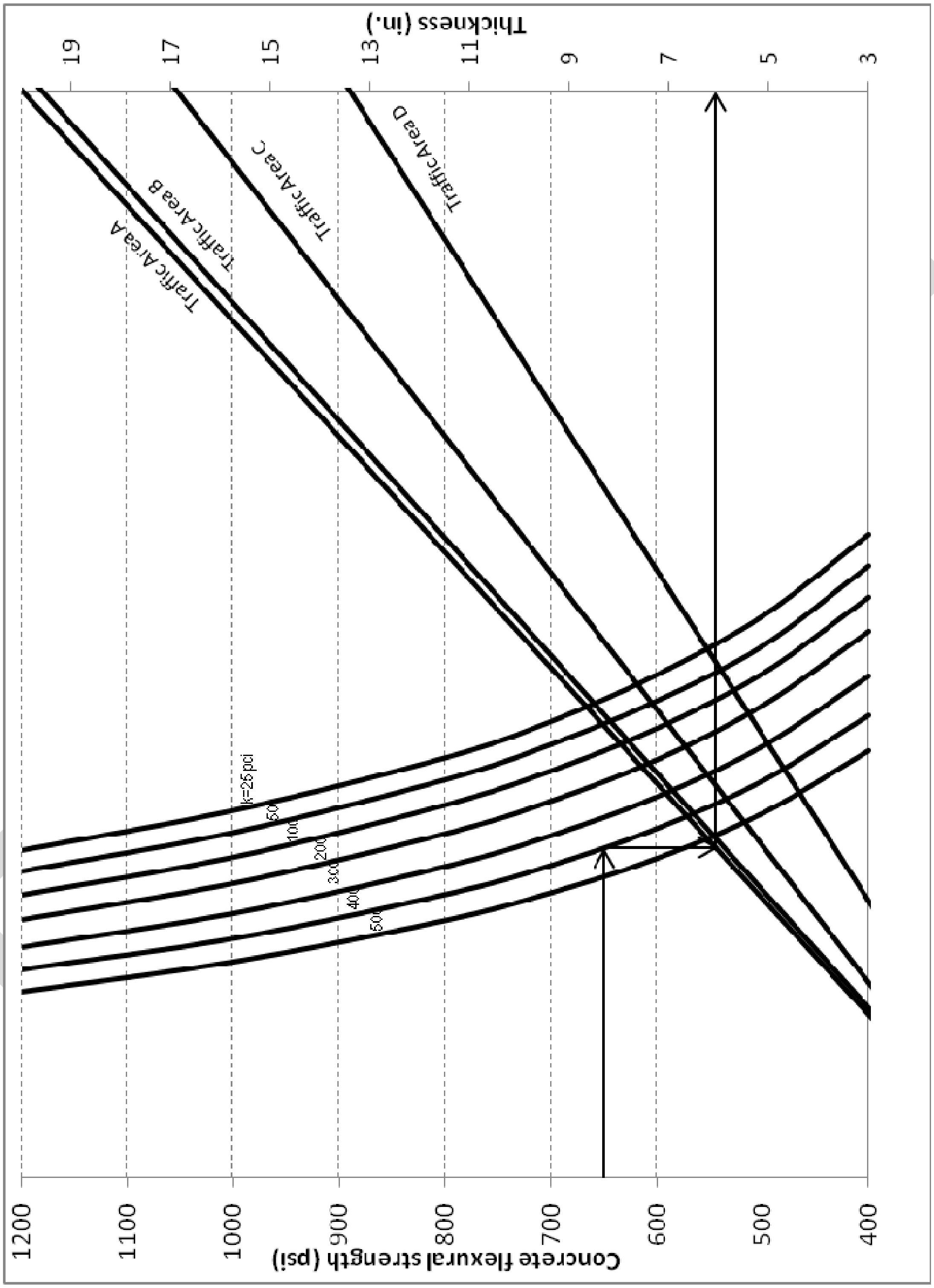


Figure 12-8. Plain Concrete Pavement Design Curves for Army Class IV - Runway Length < 5,000 ft - C-130

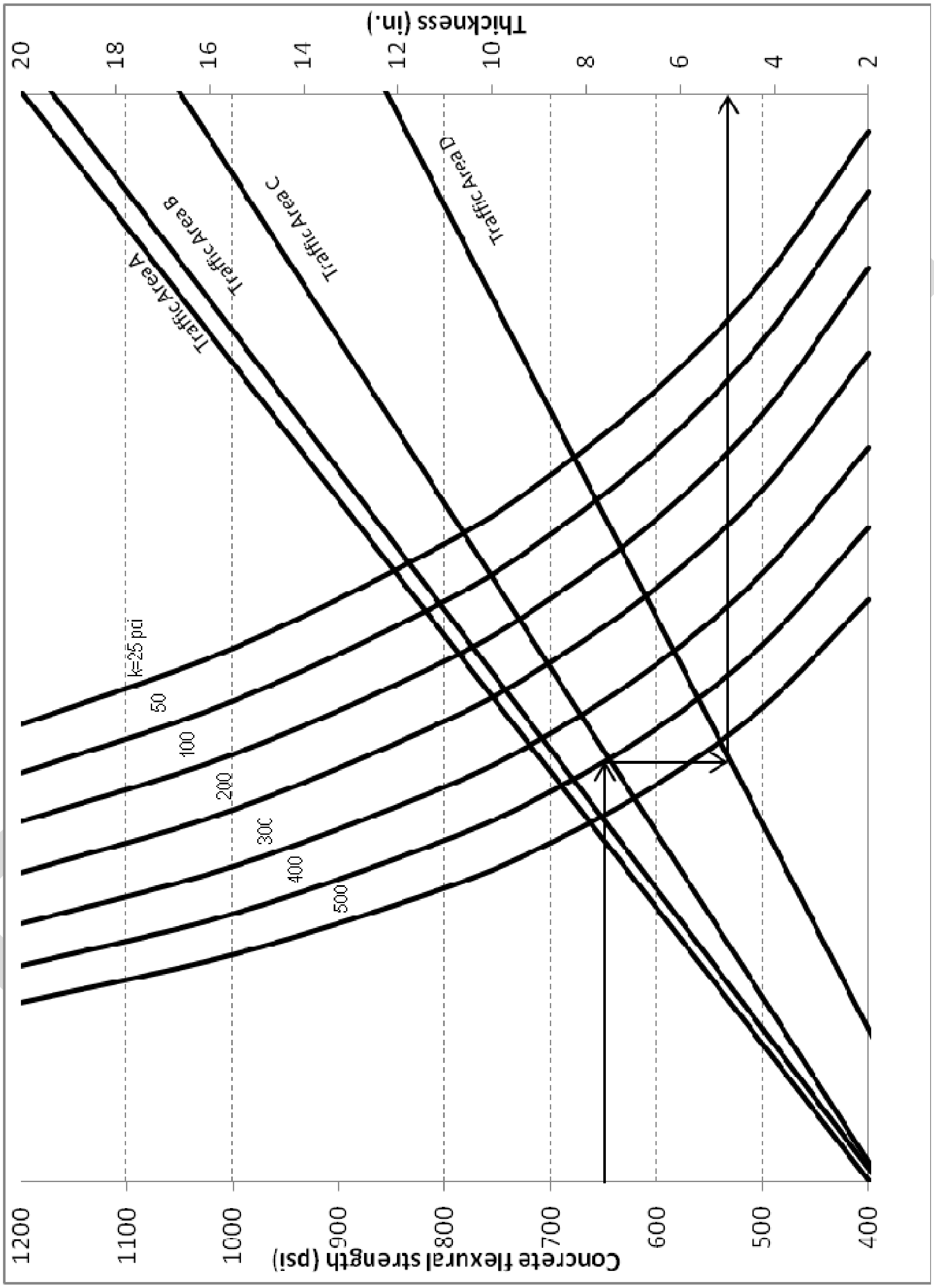


Figure 12-9. Plain Concrete Pavement Design Curves for Army Class IV - Runway Length < 5,000 ft - C-17

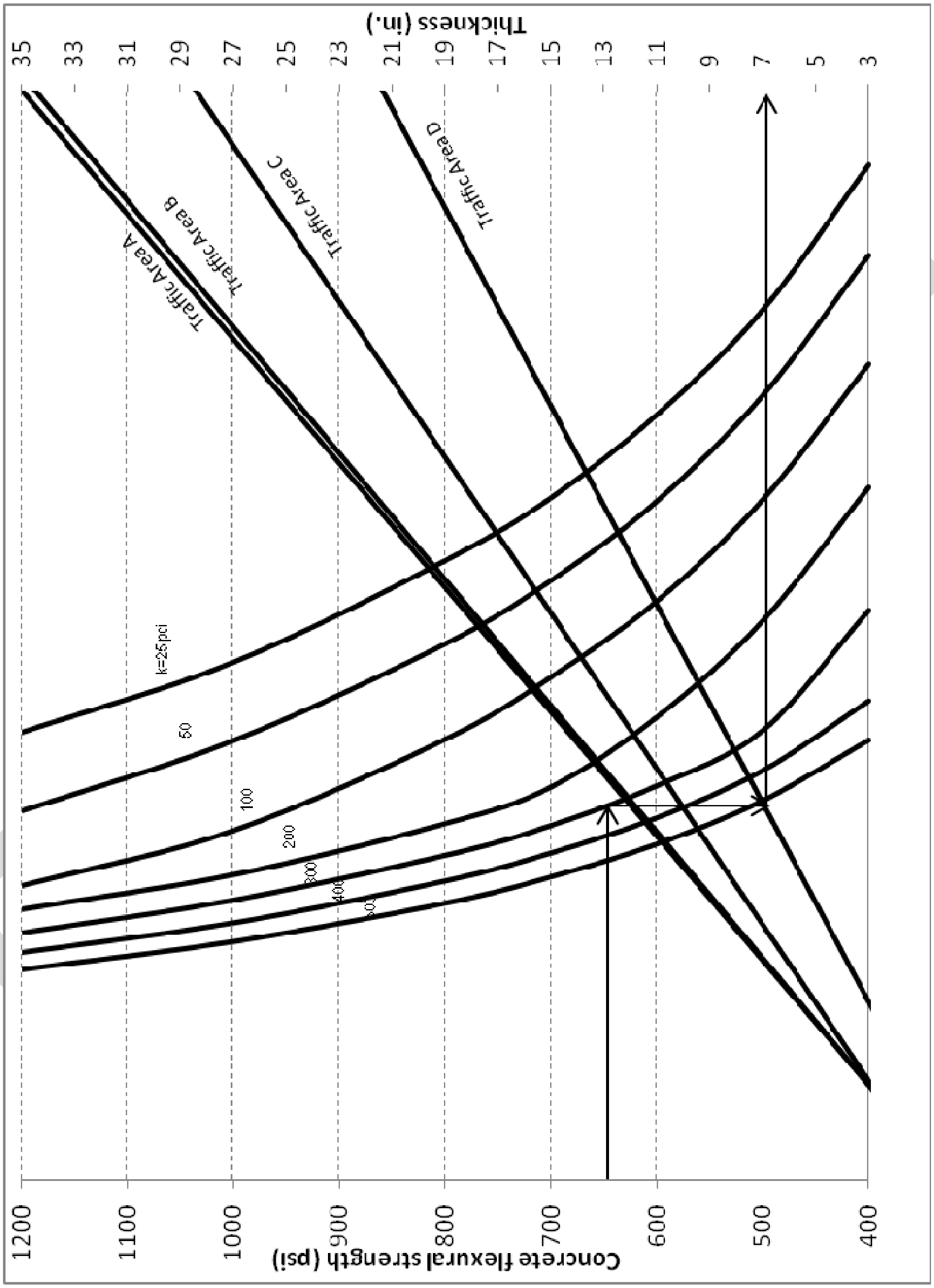


Figure 12-10. Plain Concrete Pavement Design Curves for Army Class IV - 5,000 ft < Runway Length < 9,000 ft - C-17

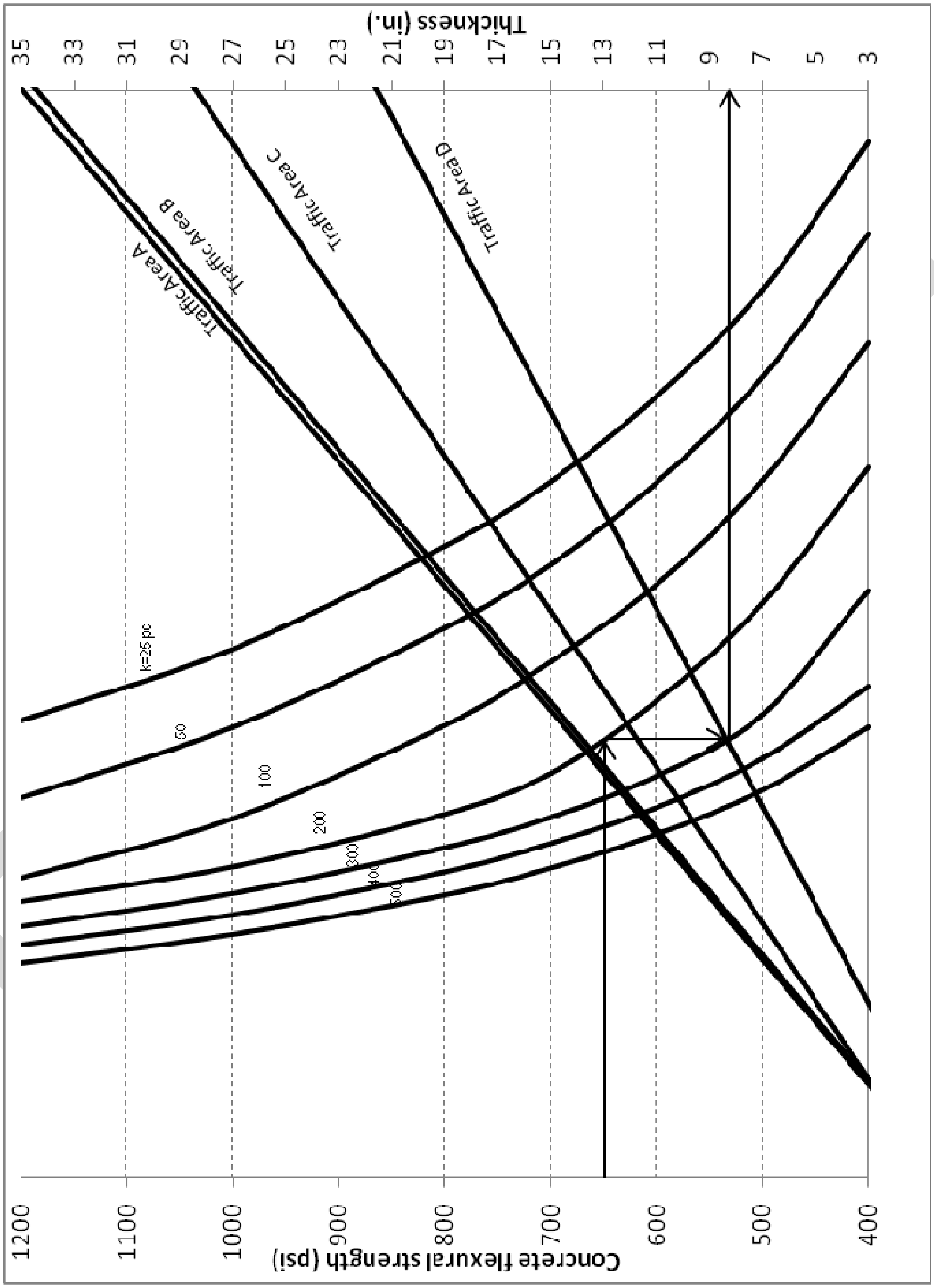


Figure 12-11. Plain Concrete Pavement Design Curves for Army Class IV - Runway Length > 9,000 ft - C-17

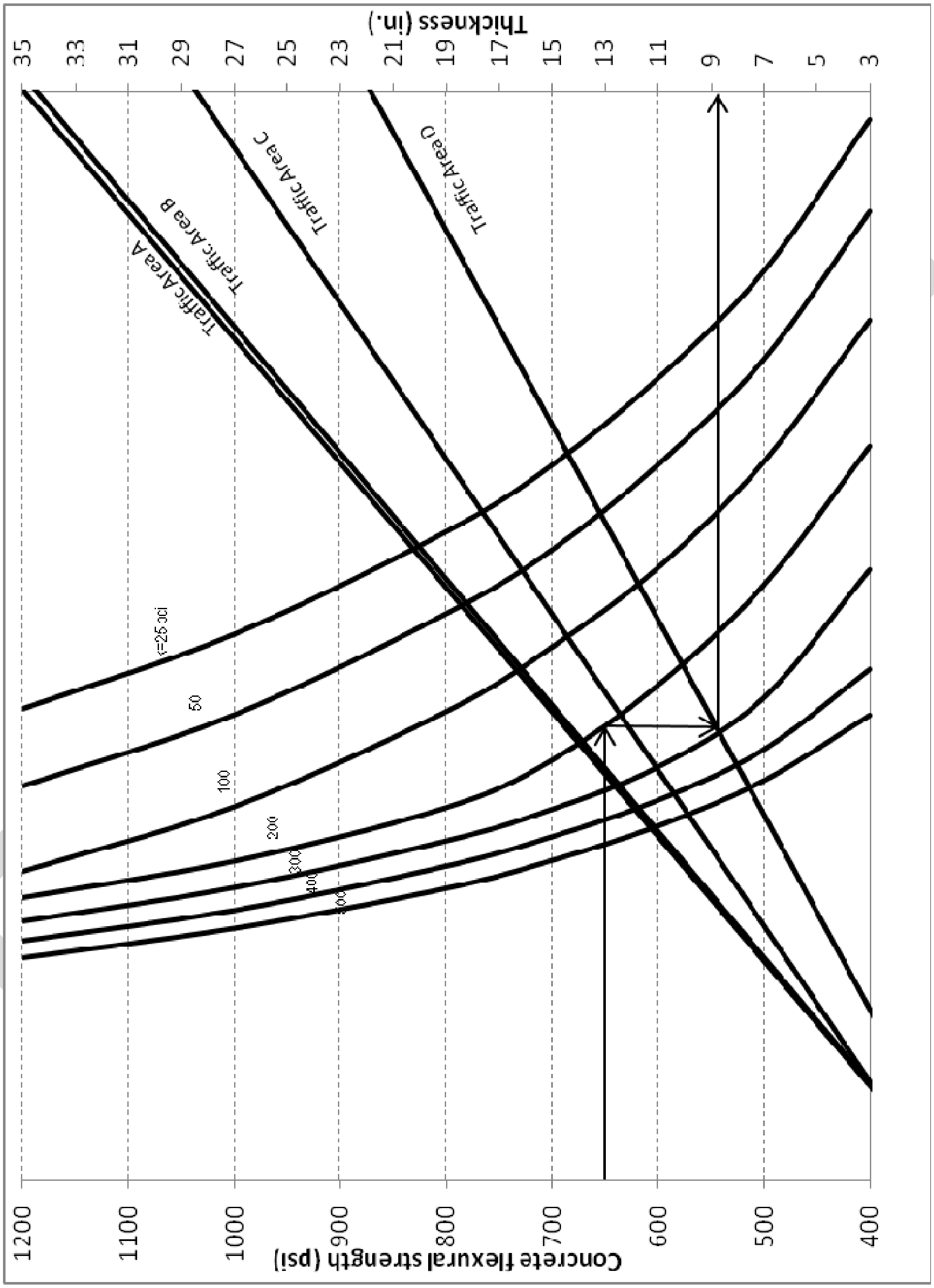


Figure 12-12. Plain Concrete Pavement Design Curves for Army Class IV - Runway Length > 9,000 ft with Mobilization Mission C-17

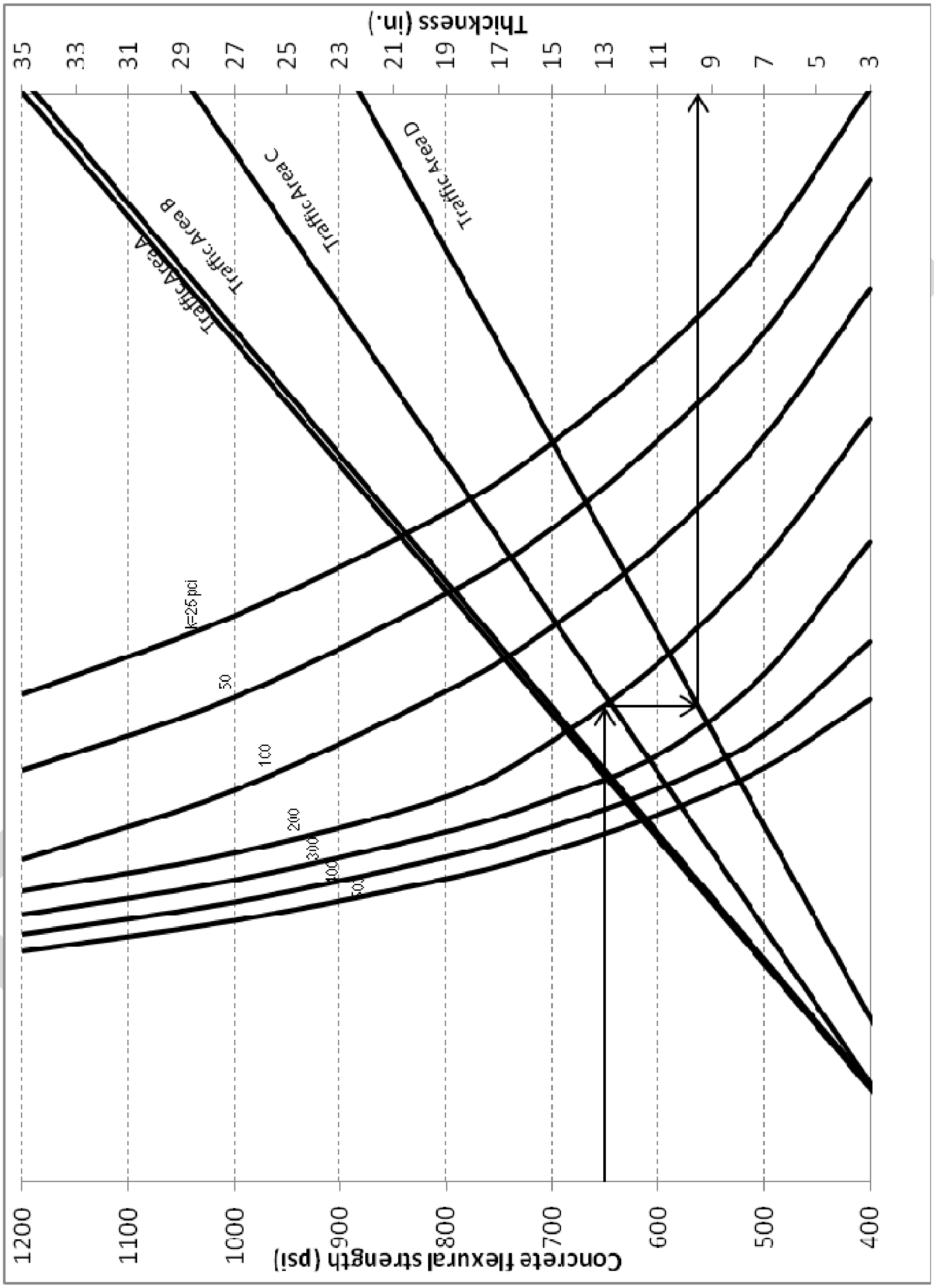


Figure 12-13. Plain Concrete Pavement Design Curves for Army Class V

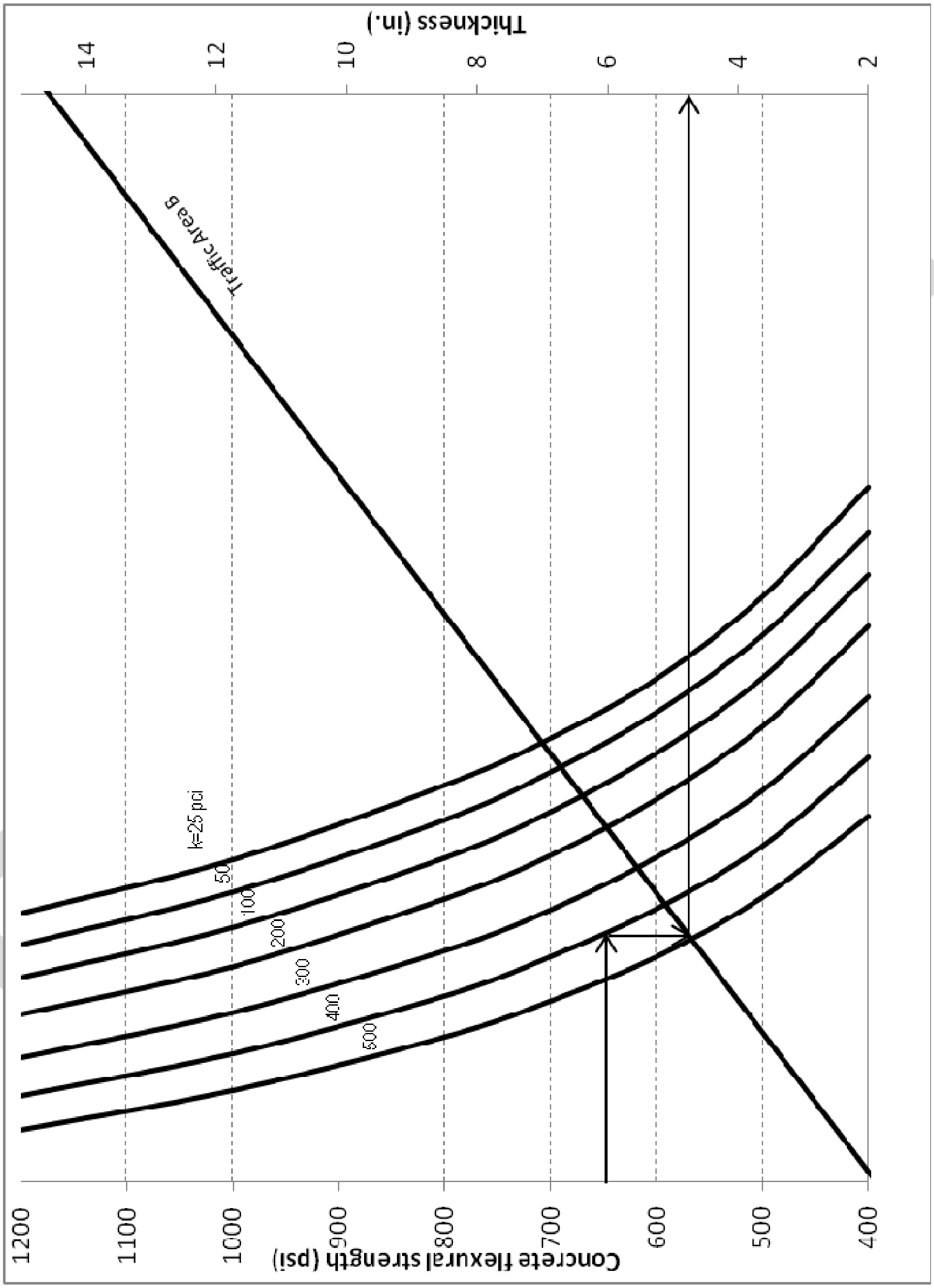


Figure 12-14. Plain Concrete Pavement Design Curves for Army Class VI Paved C-130

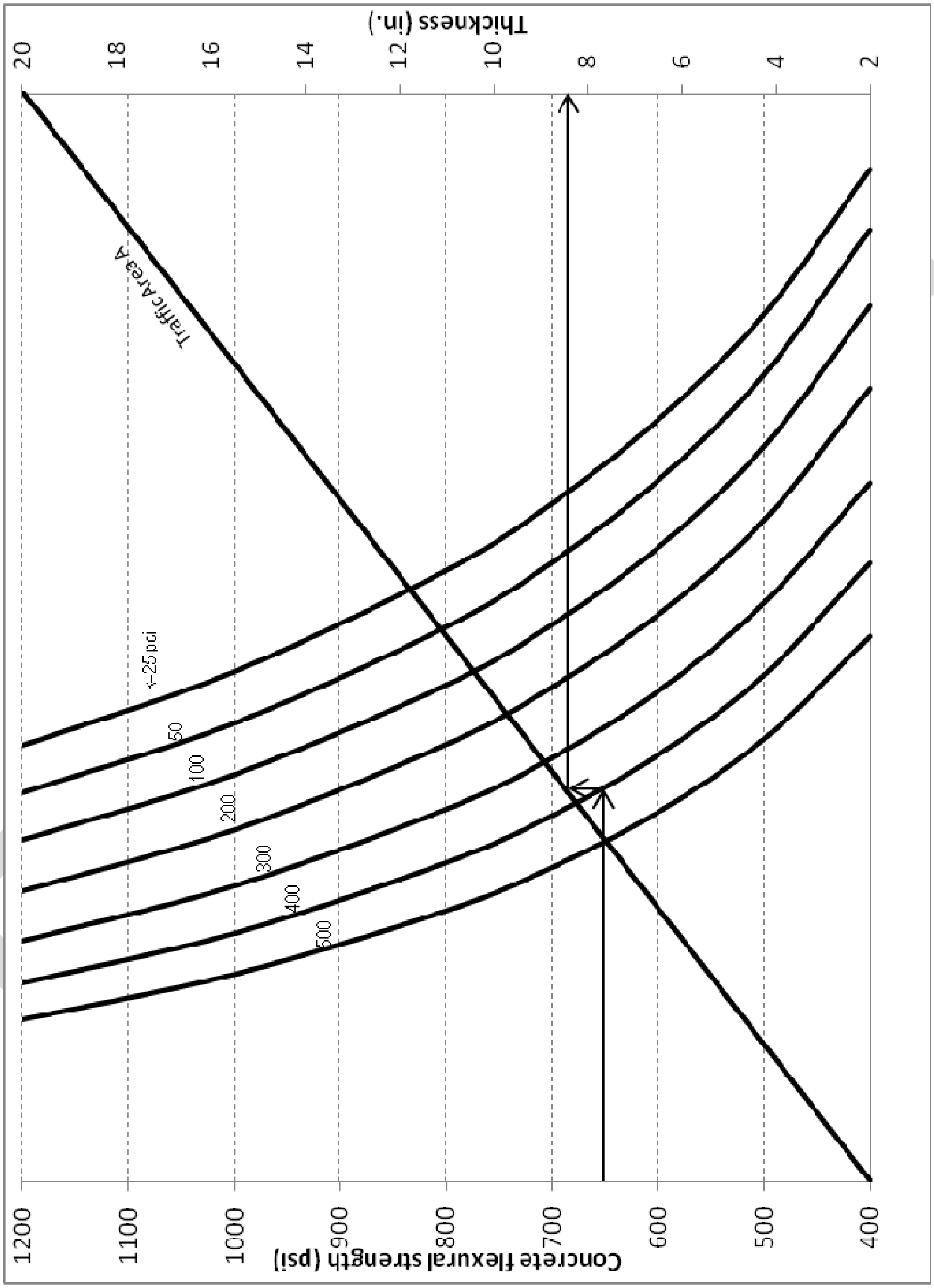


Figure 12-15. Plain Concrete Pavement Design Curves for Army Class VI Runway Length < 5,000 ft paved C-17

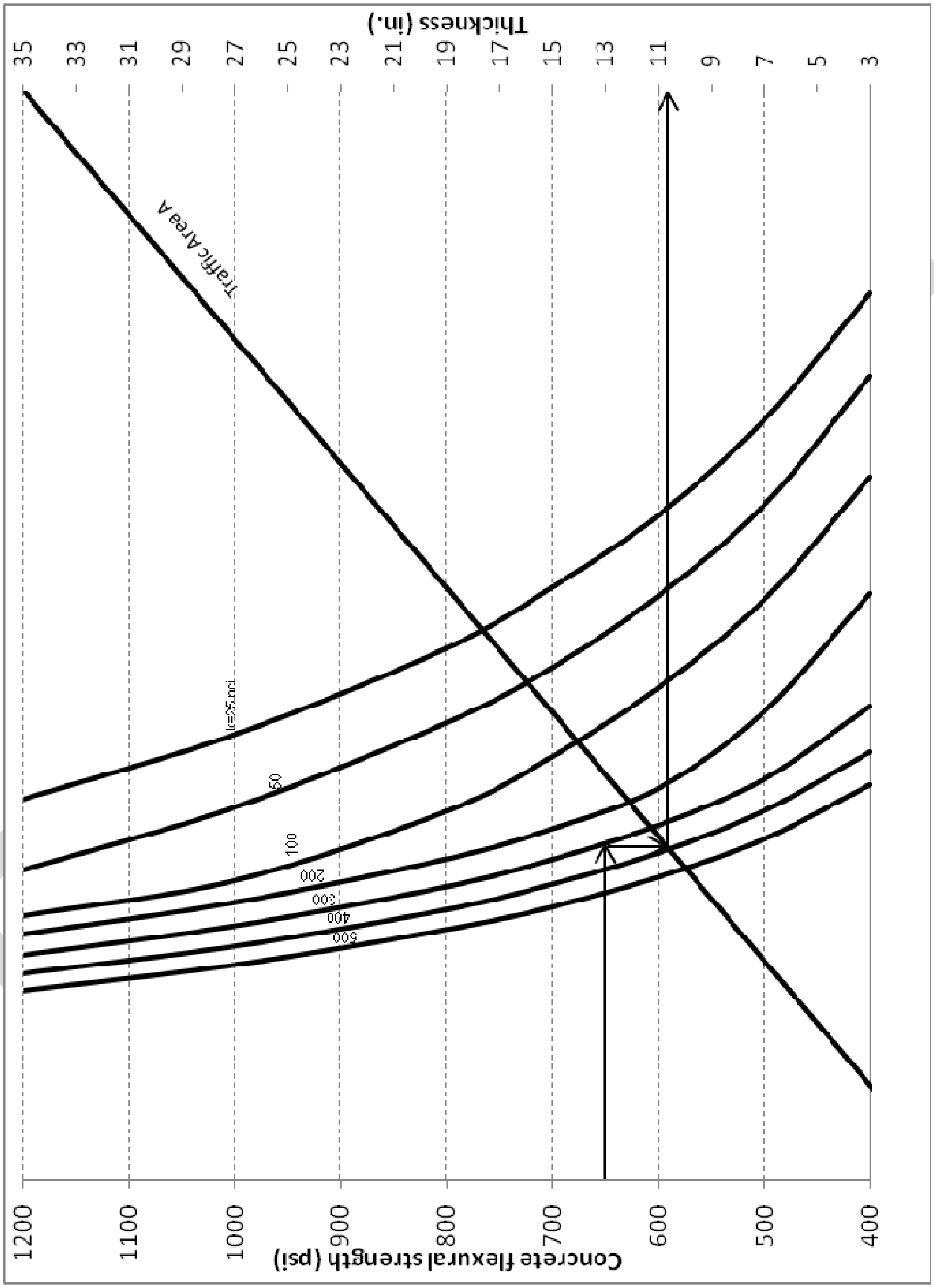


Figure 12-16. Plain Concrete Pavement Design Curves for Army Class VI RW Length ≥ 5,000 ft paved C-17

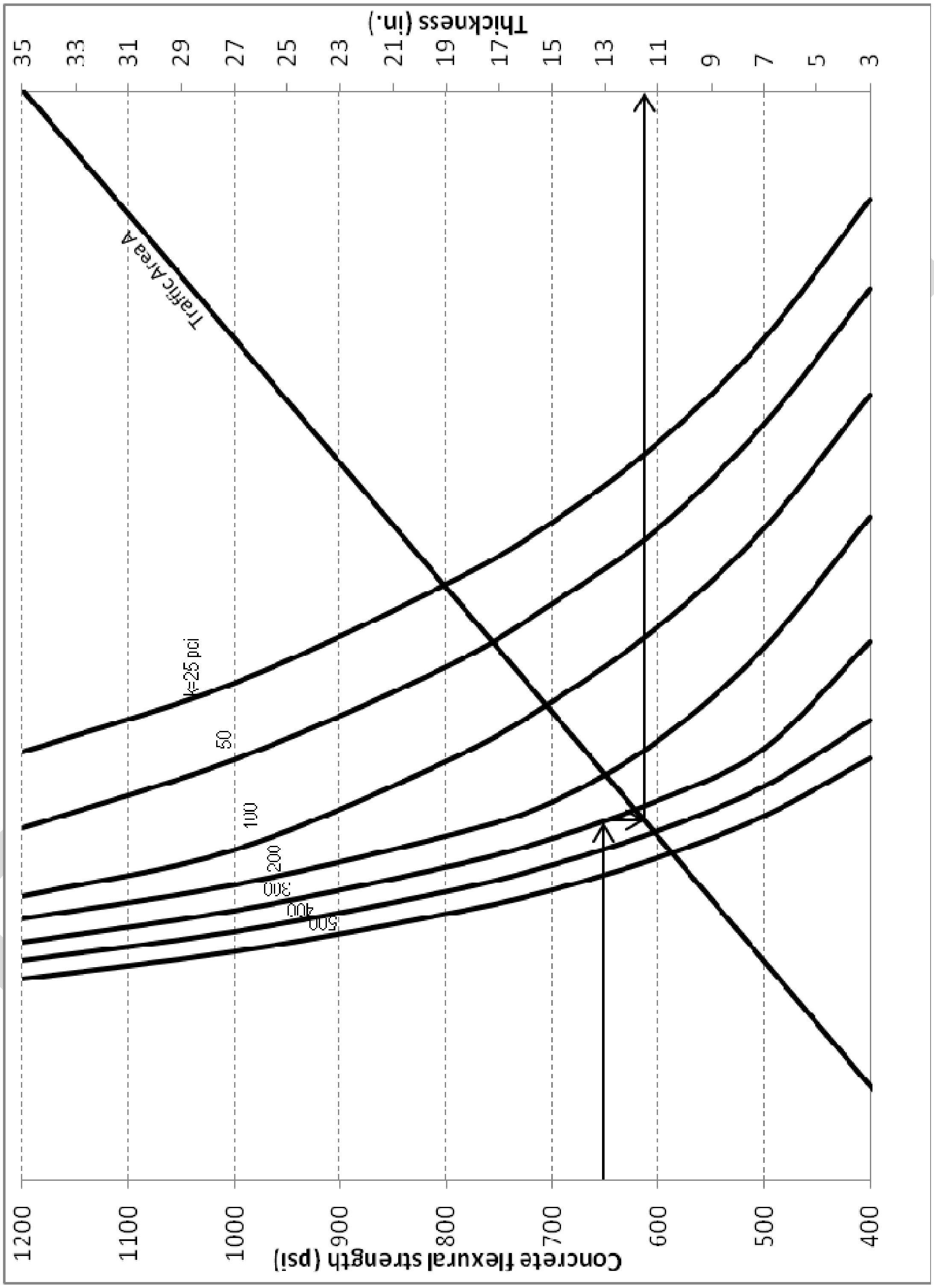


Figure 12-17. Plain Concrete Pavement Design Curves for Air Force Light Load

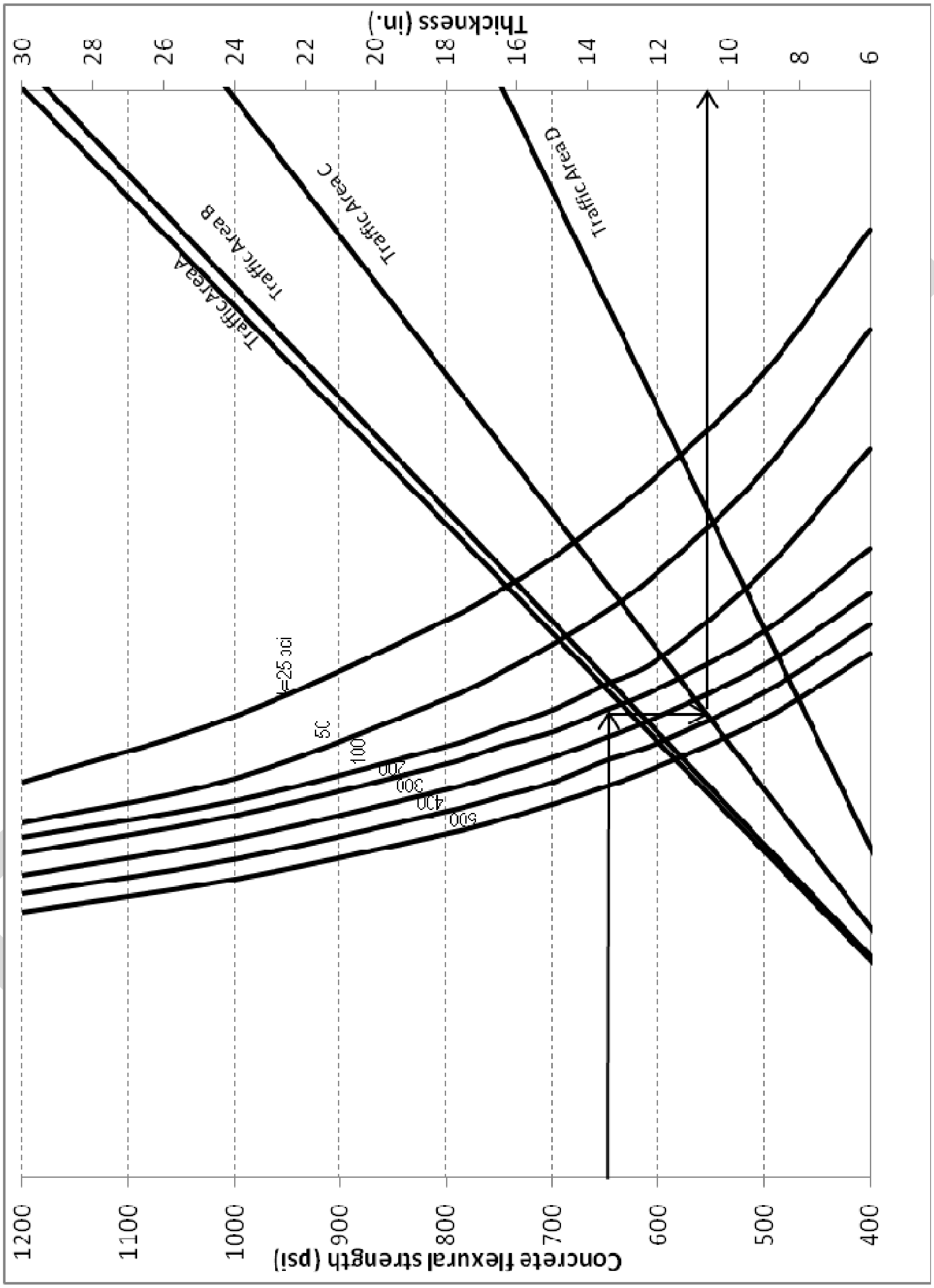


Figure 12-18. Plain Concrete Pavement Design Curves for Air Force Medium Runway Width ≥ 200 ft

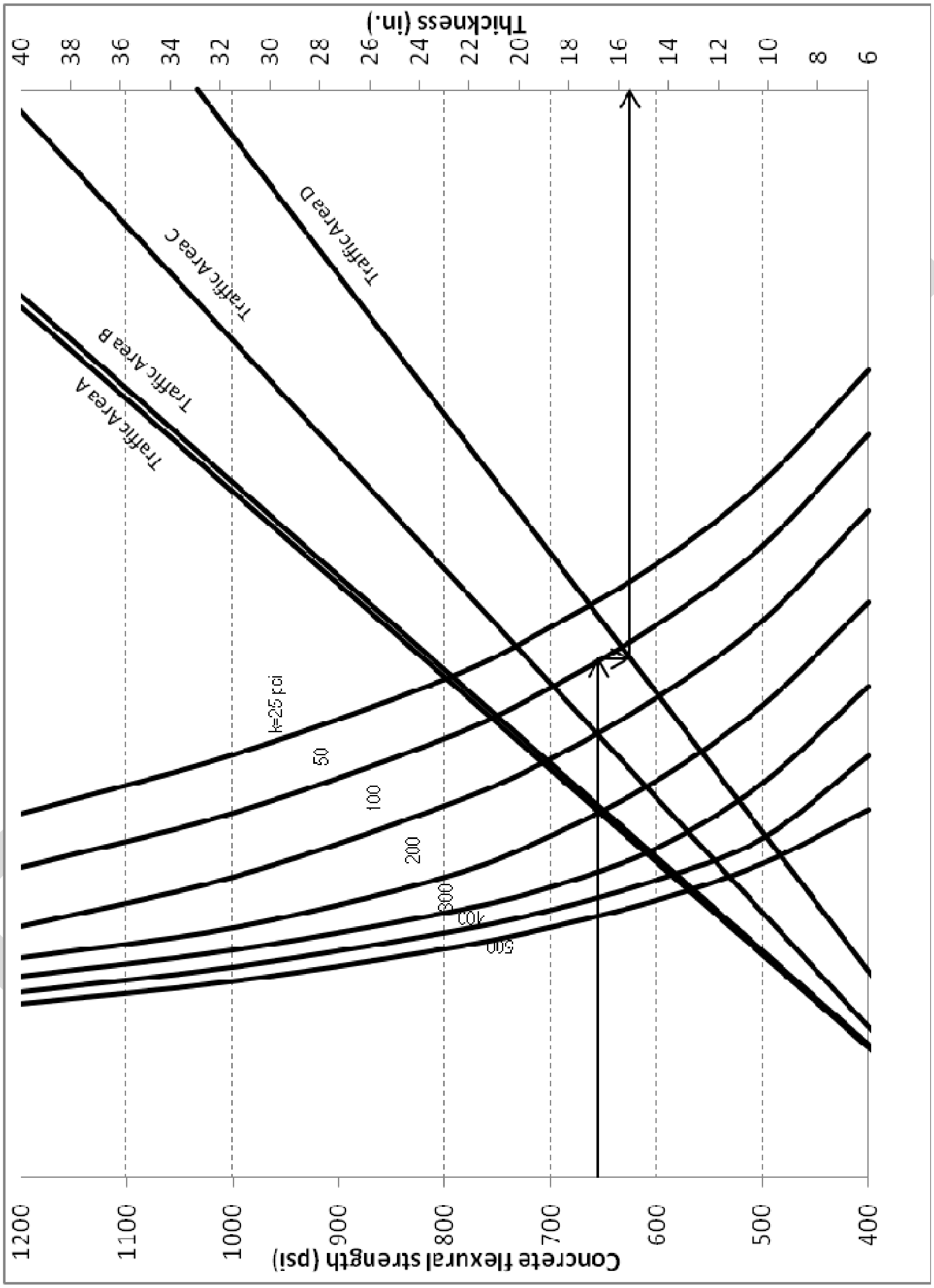


Figure 12-19. Plain Concrete Pavement Design Curves for Air Force Medium Runway Width < 200 ft

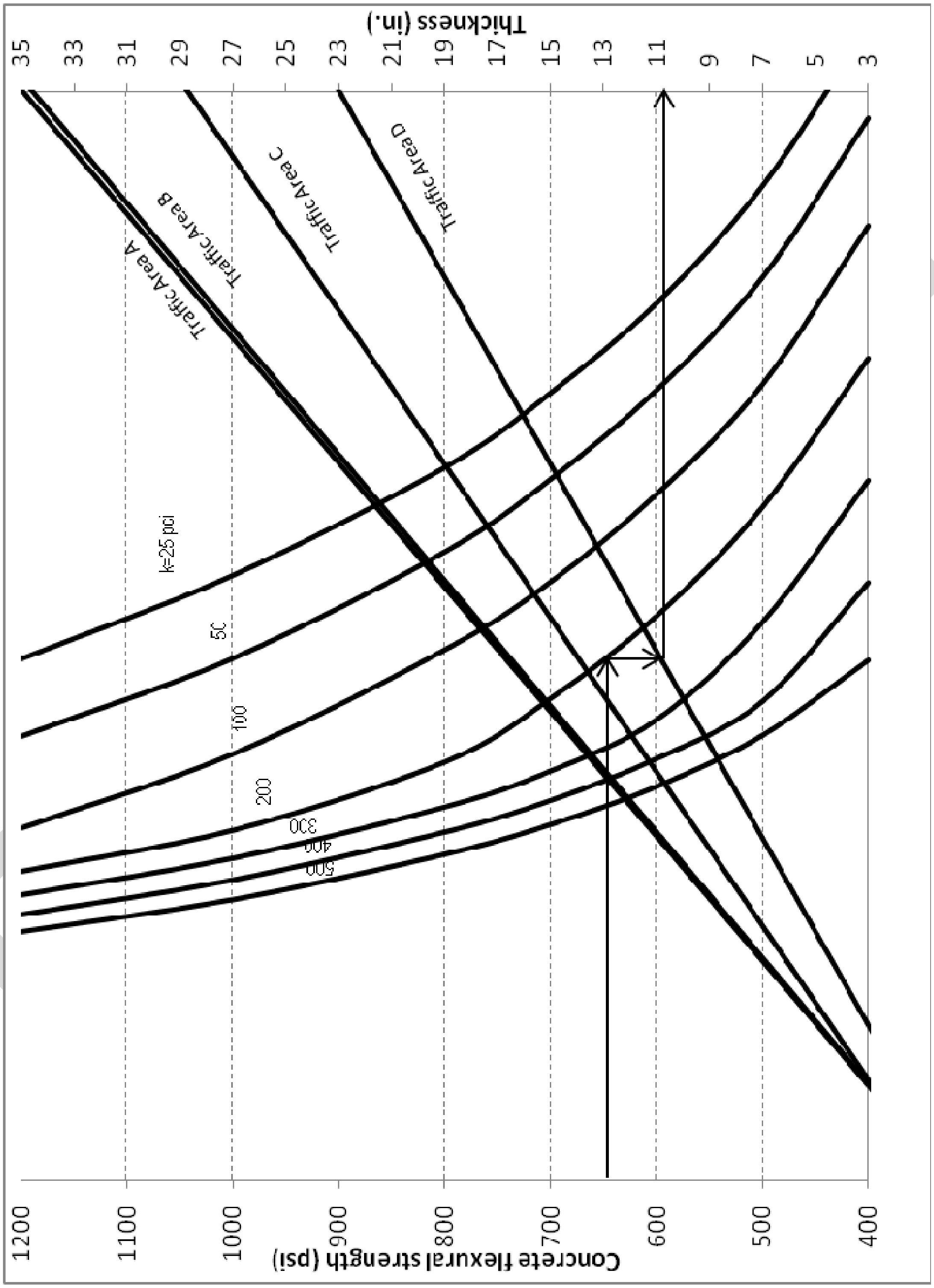


Figure 12-20. Plain Concrete Pavement Design Curves for Air Force Heavy Load

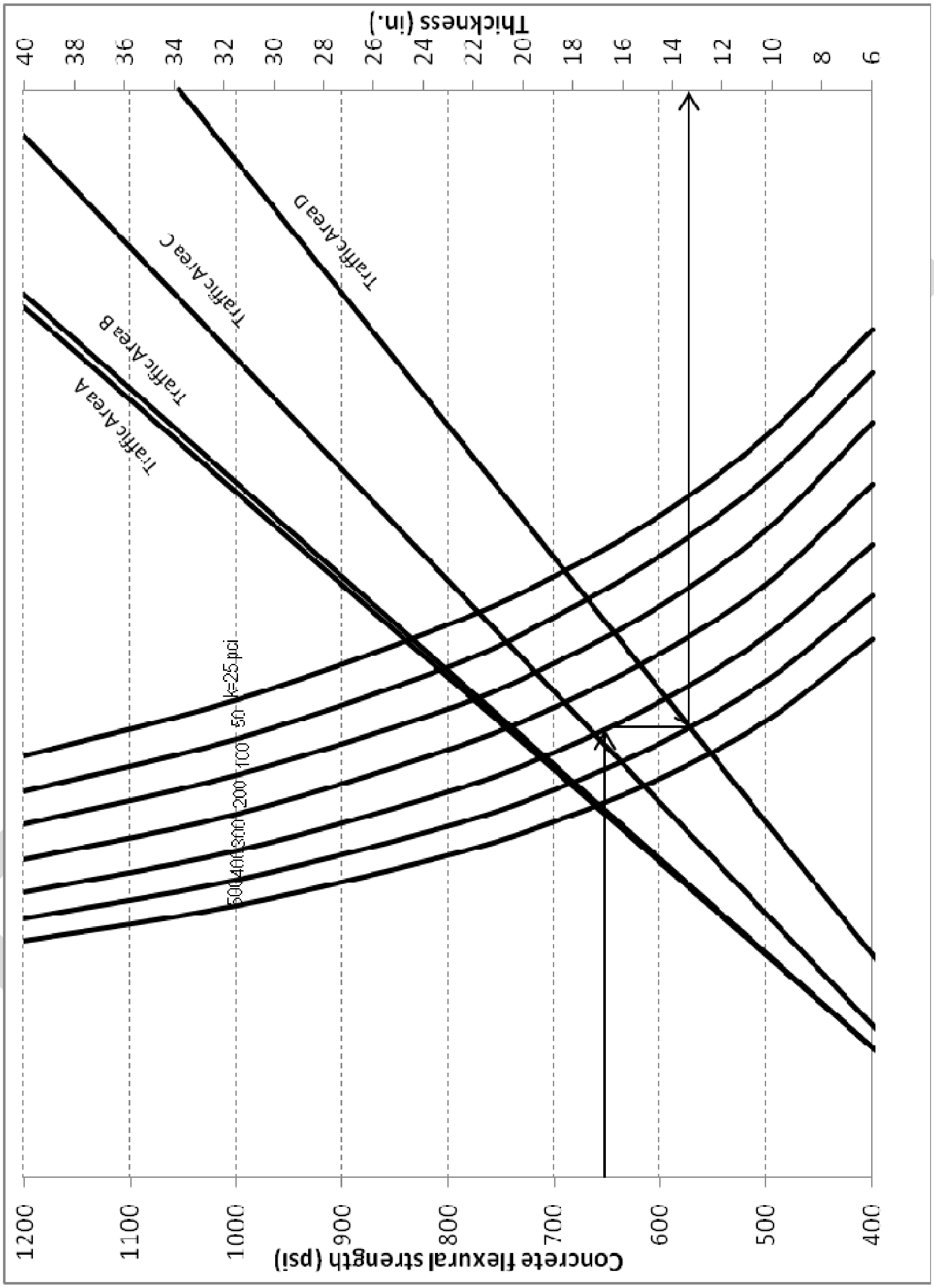


Figure 12-21. Plain Concrete Pavement Design Curves for Air Force Modified Heavy Load

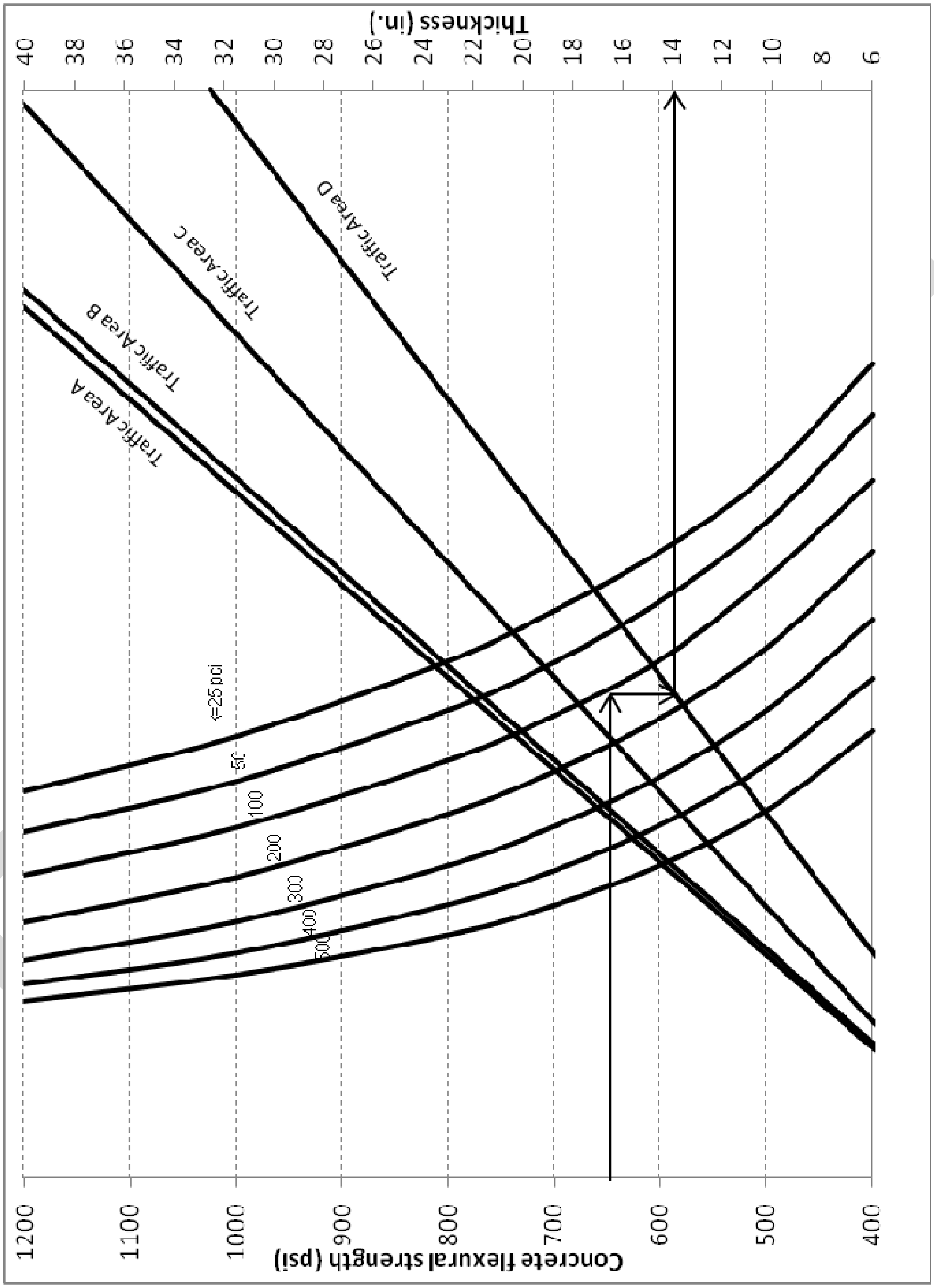


Figure 12-22. Plain Concrete Pavement Design Curves for Air Force Landing Zone - C-130

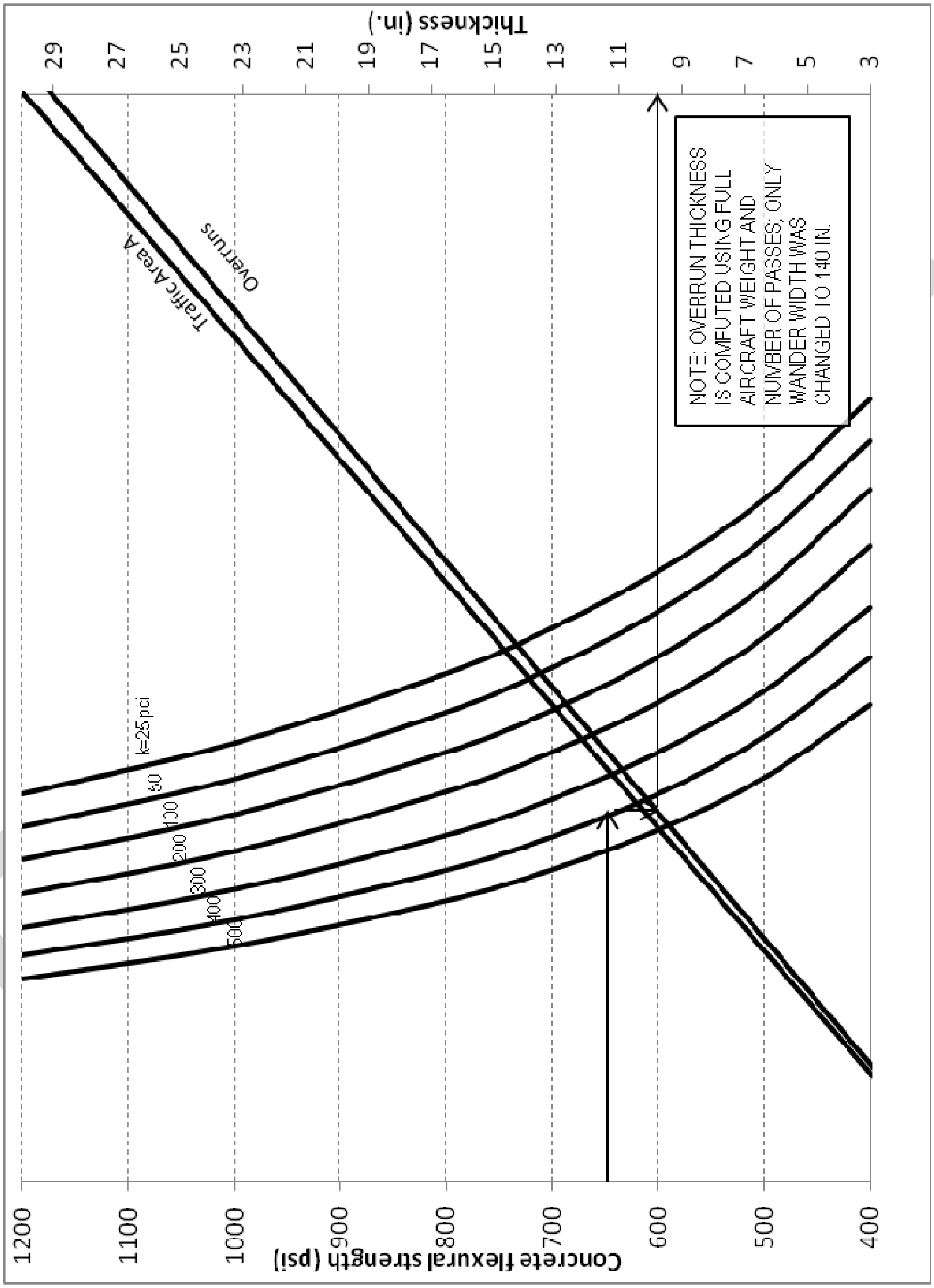


Figure 12-23. Plain Concrete Pavement Design Curves for Air Force Landing Zone - C-17

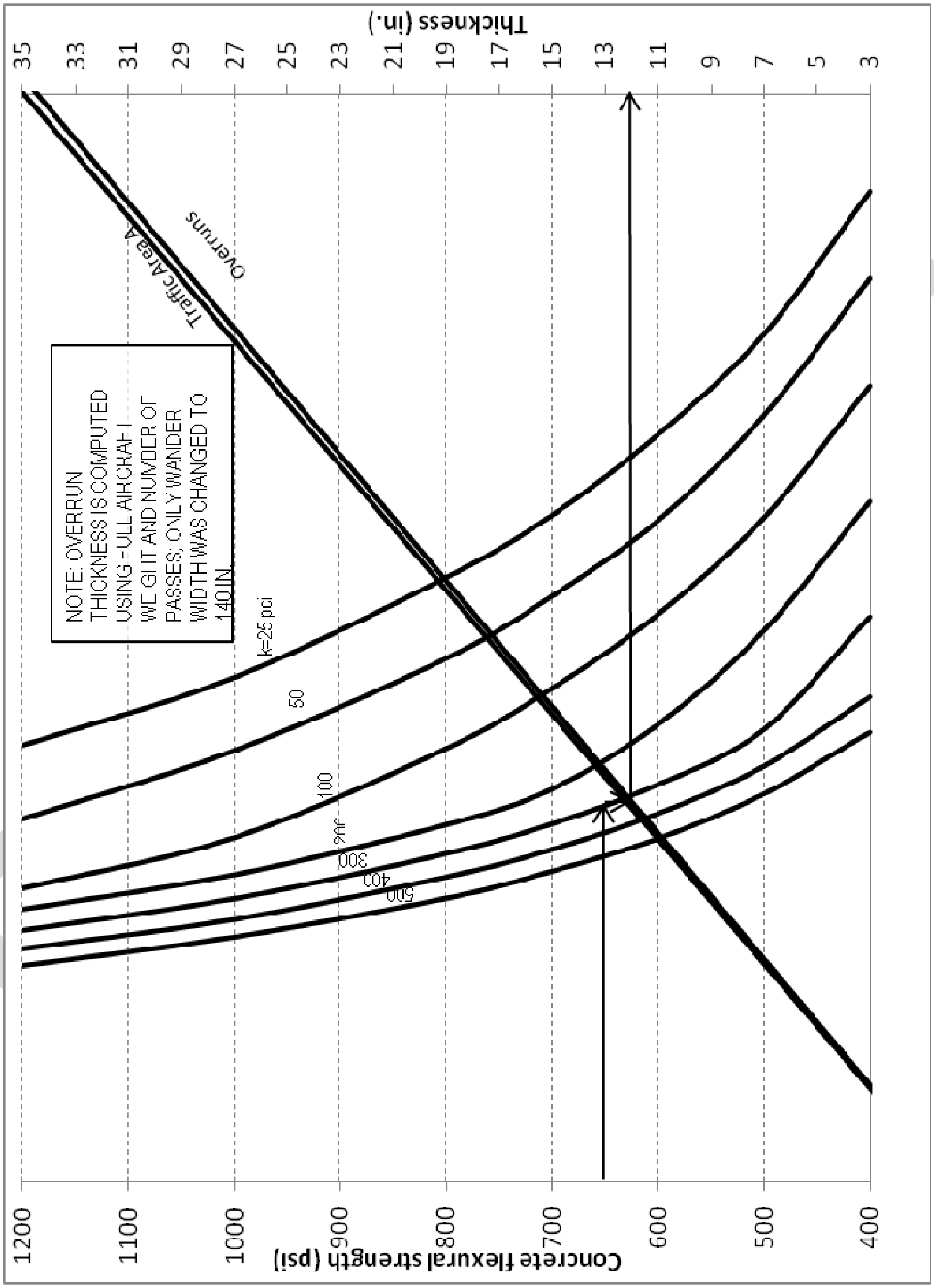


Figure 12-24. Plain Concrete Pavement Design Curves for Navy Design Traffic Group I

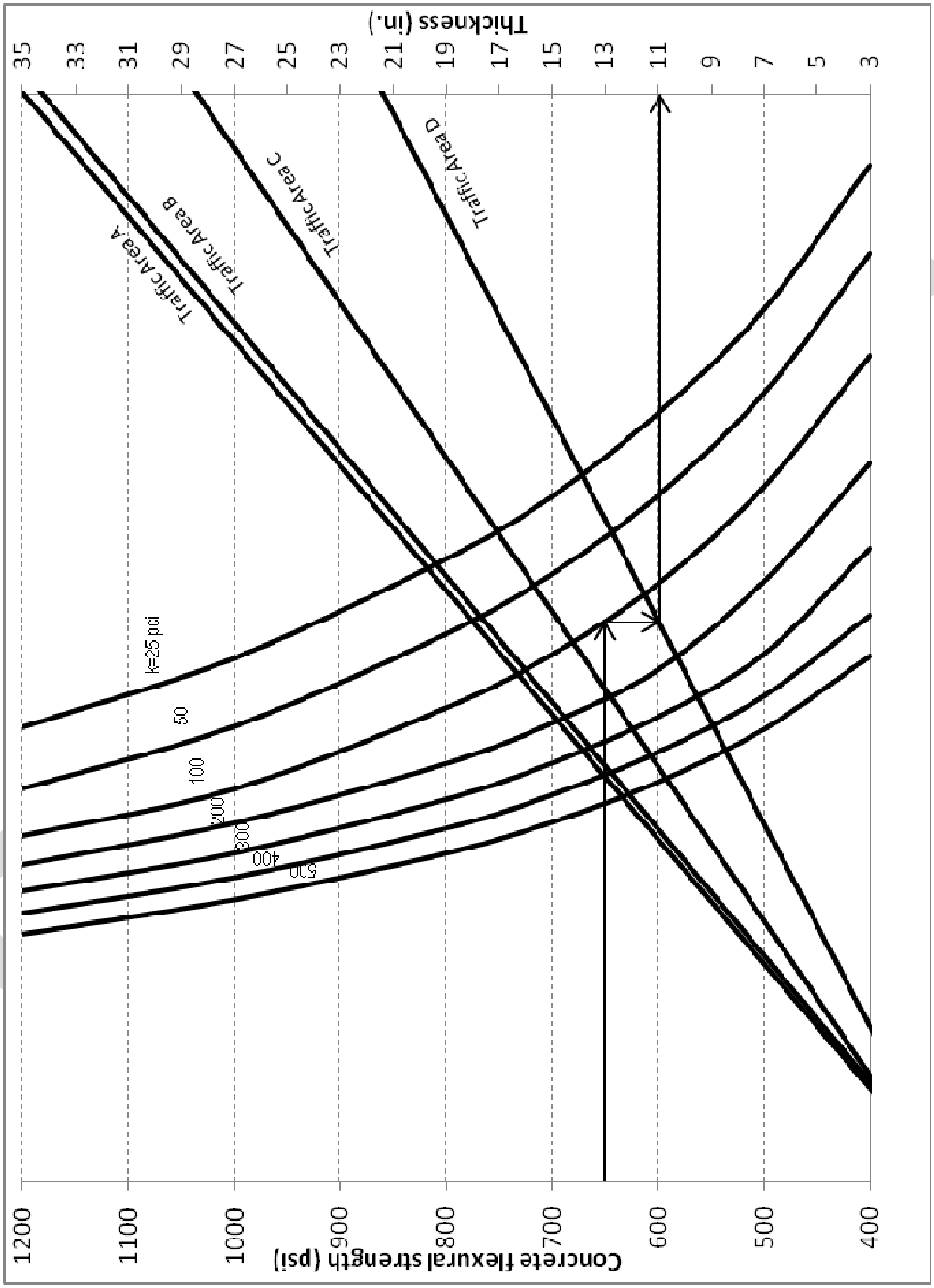


Figure 12-25. Plain Concrete Pavement Design Curves for Navy Design Traffic Group II

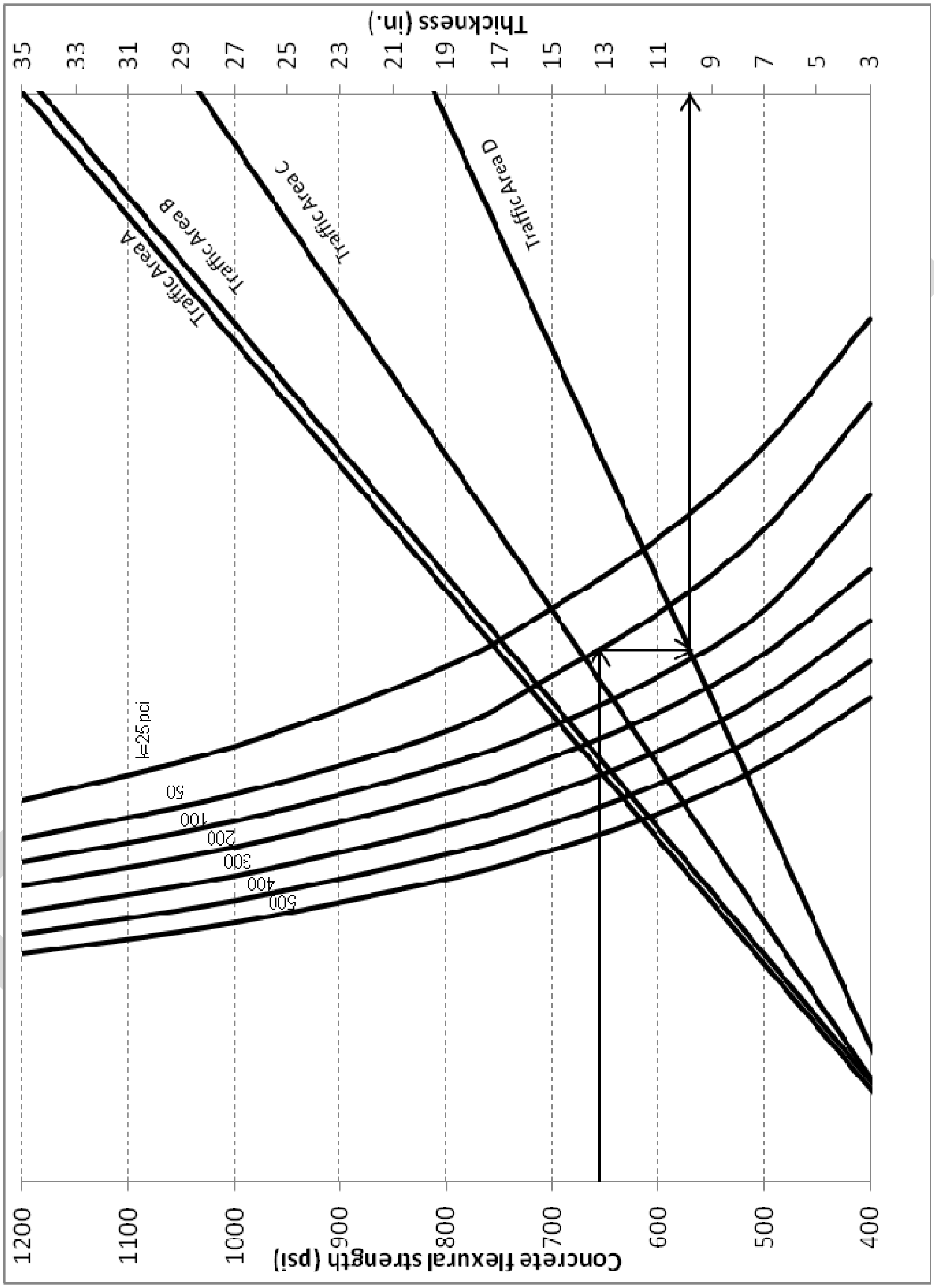


Figure 12-26. Plain Concrete Pavement Design Curves for Navy Design Traffic Group III

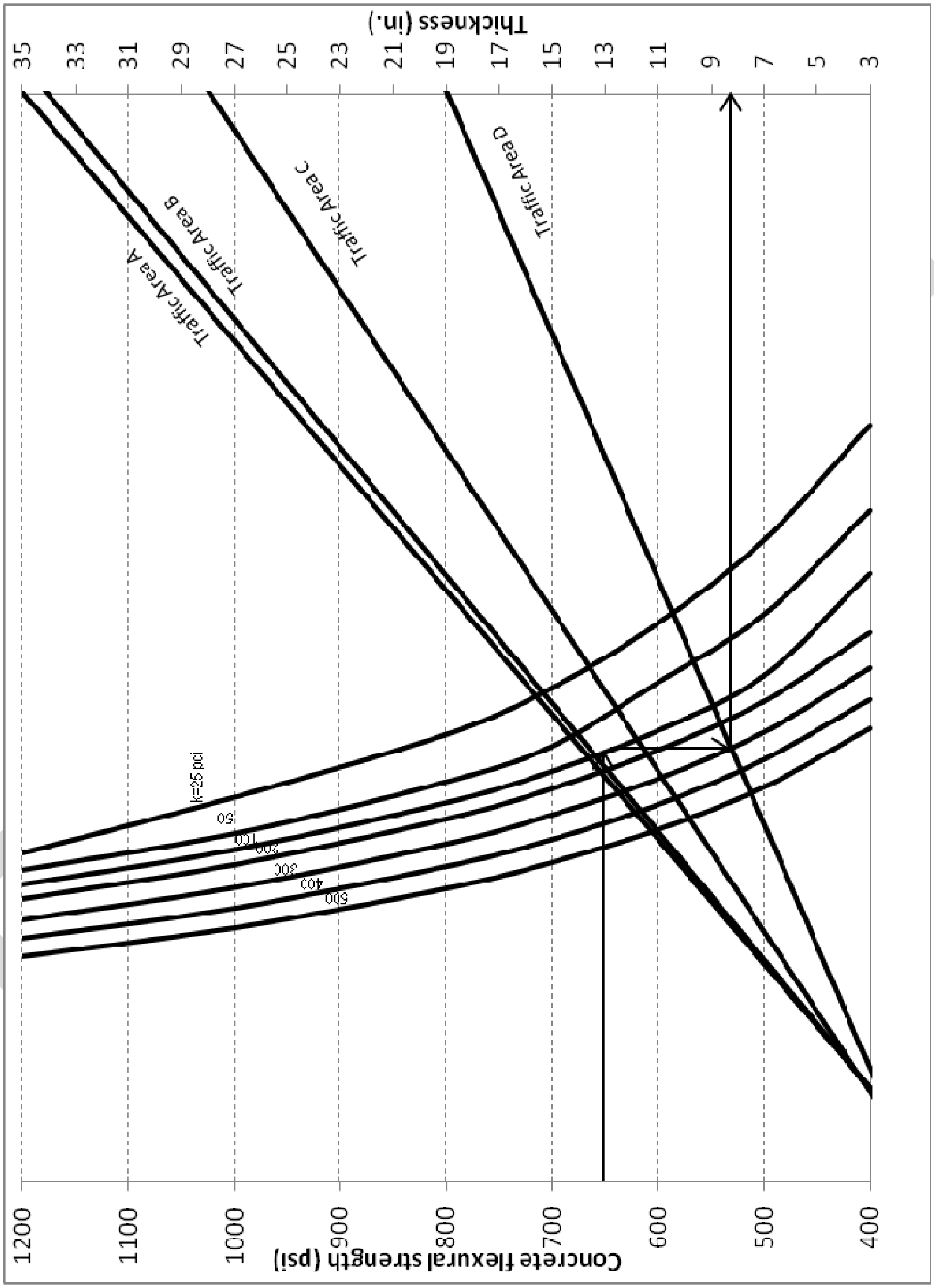
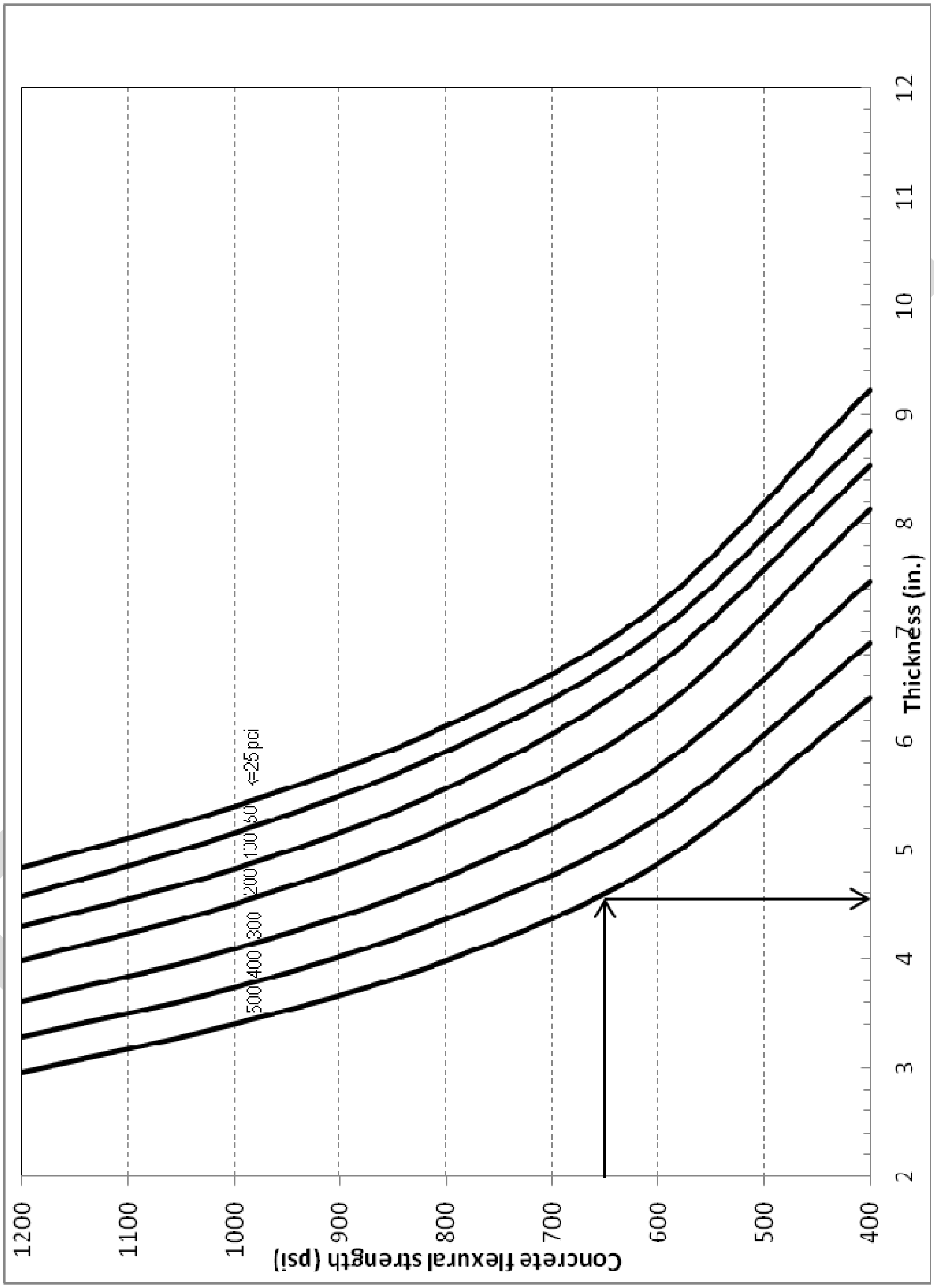


Figure 12-27. Plain Concrete Pavement Design Curves for Rigid Shoulders



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, determine the thickness requirement 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 on the type of traffic area or pavement facility. When using IP 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 152 mm (6 in). For new construction 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. Round SI thickness values up to the nearest 10 mm.

12-3.2 Plain Concrete Pavements on Stabilized Base or Subgrade

Stabilized base 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:

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

where

h_o = thickness of plain concrete overlay, mm (in)

h_d = design thickness of equivalent single slab placed directly on foundation, mm (in)

E_b = modulus of elasticity of base, MPa (psi)

E_c = modulus of elasticity of concrete, usually taken as 27,575 MPa ($4{\times}10^6$ psi)

h_b = thickness of stabilized layer or lean concrete base, mm (in)

Examples of plain concrete pavement design for the Army, Navy, and Air Force are contained in Appendix B, Section 14.

12-4 PAVEMENT JOINT USES

Joints are provided in concrete pavement to permit contraction and expansion of the concrete resulting from temperature and moisture changes; relieve warping and curling stresses due to temperature and moisture differentials; prevent unsightly irregular breaking of the pavement; and act as a construction expedient to separate sections or strips of concrete placed at different times. The two general types of joints are construction and contraction. A typical jointing layout is illustrated in Figure 12-31.

12-5 SELECTION OF PAVEMENT JOINT TYPES

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 a joint 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 Table 12-1. 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 (for example, filet slabs), the slabs must be reinforced as required in Chapters 1 and 13. Reinforcing steel shall never be carried through contraction joints. Expansion joints are special construction joints that are used to isolate structures from the concrete pavement movement (for example, isolate a hangar from an apron) or to separate two intersecting pavements (for example, a taxiway intersecting a runway at right angles). Expansion joints often are the source of maintenance headaches so they are used only when concrete movement must 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 may be used for special circumstances only with specific approval of the Air Force MAJCOM pavements engineer, USACE-TSC, or Naval Facilities Engineering Service Center, as appropriate. These other special application joints include:

- Thickened-edge expansion or doweled expansion joints where isolation from concrete movement is required

- Thickened-edge construction joint where load transfer cannot be provided by dowels and aircraft traffic will cross or be adjacent to the joint

- Doweled contraction joint where load transfer from aggregate interlock might be lost due to slab movement (For example, the 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.)

- Butt longitudinal construction joints. This requires special design for no load transfer for Army and Air Force airfield pavements.

- Tied joints (Navy only) where relative movement and separation between slabs must be restricted. Such situations are rare on airfield pavements.

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 before use. Contact the Air Force MAJCOM pavements engineer, USACE-TSC, or Naval Facilities Engineering Service Center, as appropriate.

12-6 JOINTS FOR ARMY AND AIR FORCE PAVEMENTS

12-6.1 Contraction Joints

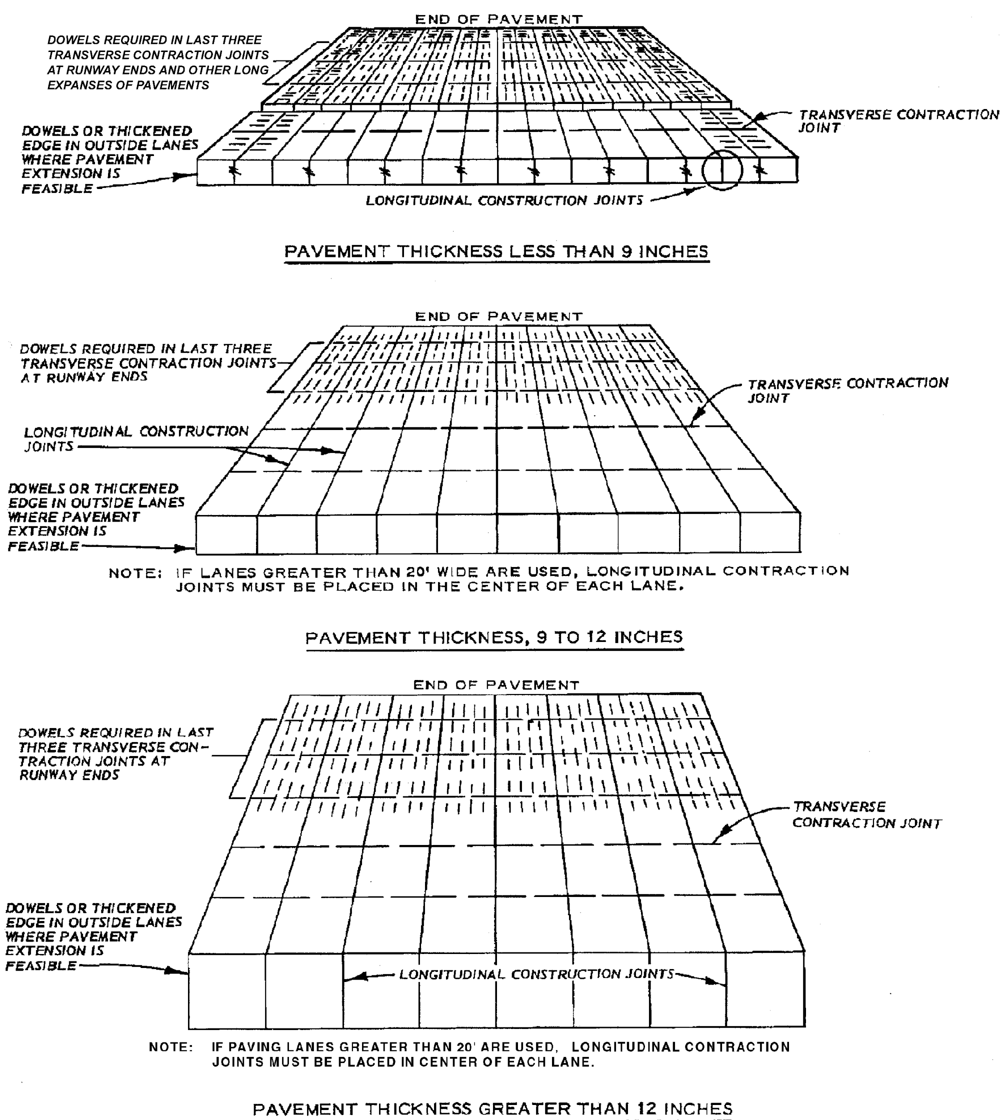
Weakened-plane contraction joints are provided to control cracking in the concrete resulting from drying shrinkage and contraction, and to limit curling or warping stresses from temperature and moisture gradients in the pavement. Contraction joints are formed in concrete by partial-depth sawing or by installing sawable inserts. The saw-cut joint or formed groove provides a weakened plane that will crack through the full slab depth during shrinkage and contraction of the concrete as it cures. This will provide some relief from tensile forces resulting from foundation restraint and compressive forces caused by subsequent expansion. Contraction joints will be required transversely and may be required longitudinally depending on pavement thickness and spacing of construction joints. A typical contraction joint is shown in Figure 12-32. Instructions regarding the use of saw cuts to form the weakened plane are contained in UFC 3-250-04. Guidance for sealing joints in rigid pavements for the Air Force and Army is contained in UFC 3-250-08FA.

12-6.1.1 Width and Depth of Weakened-Plane Saw Cut

The width of the weakened-plane saw cut will be 3 mm (1/8 in) or greater. The depth of the weakened plane saw cut 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 305 mm (12 in), 76 mm (3 in) for pavements 305 to 457 mm (12 to 18 in) in thickness, and one-sixth of the slab thickness for pavements greater than 457 mm (18 in) in thickness. In no case will the depth of the saw cut 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 saw cut 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 saw cut by 25 percent to assure the fracturing and proper functioning of each of the scheduled joints.

Figure 12-31. Typical Jointing

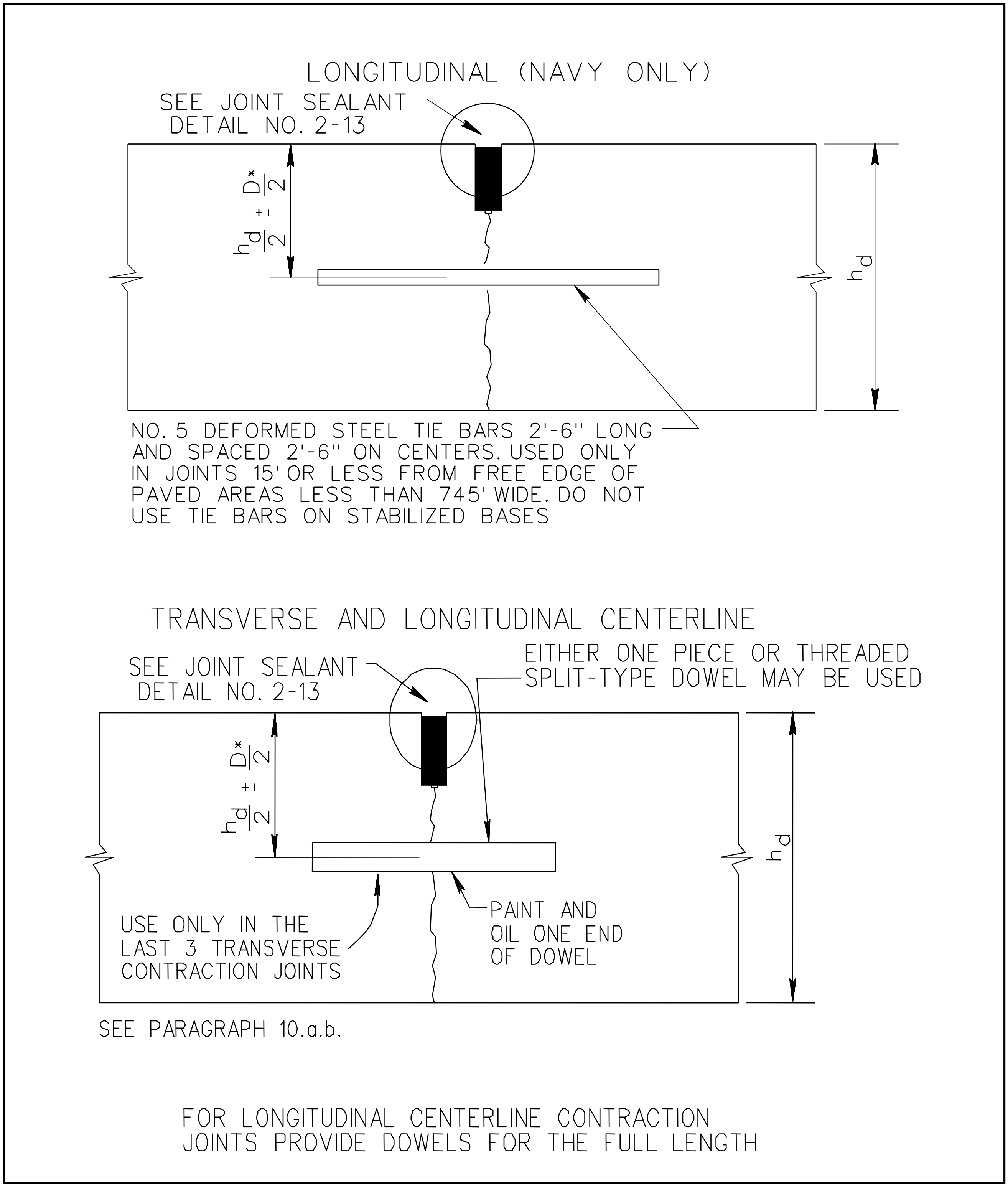


CONVERSION FACTORS

MILLIMETERS = 25.4 x INCHES
METERS = 0.3048 x FEET

All construction joints for Air Force and Army shall be doweled.

Figure 12-32. Contraction Joints for Plain Concrete Pavements



12-6.1.2 **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 12-33. The dimensions of the sealant reservoir are critical to satisfactory performance of the joint sealing materials.

12-6.1.3 **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 m (12.5 ft) and generally not more than 6 m (20 ft). 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 12-1 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 it is 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 12-1 may be satisfactory. Where it is desired to exceed the maximum allowable slab size dimension in any direction, the slabs shall be reinforced in accordance with the odd-shaped slab reinforcement requirements in Chapter 13, or approval must be obtained from the USACE-TSC for Army projects, or the appropriate Air Force MAJCOM for Air Force projects, regardless of who performs the design.

12-6.1.4 **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 12-1. 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. Reinforcing steel shall never be carried through contraction joints.

Table 12-1. Maximum Allowable Slab Size

Pavement Thickness, mm (in)	Spacing, m (ft)
Less than 230 (9)	3.8 to 4.6 (12.5 to 15)
230 and Over (9)	6 (20 max)

12-6.1.5 **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 shall 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.

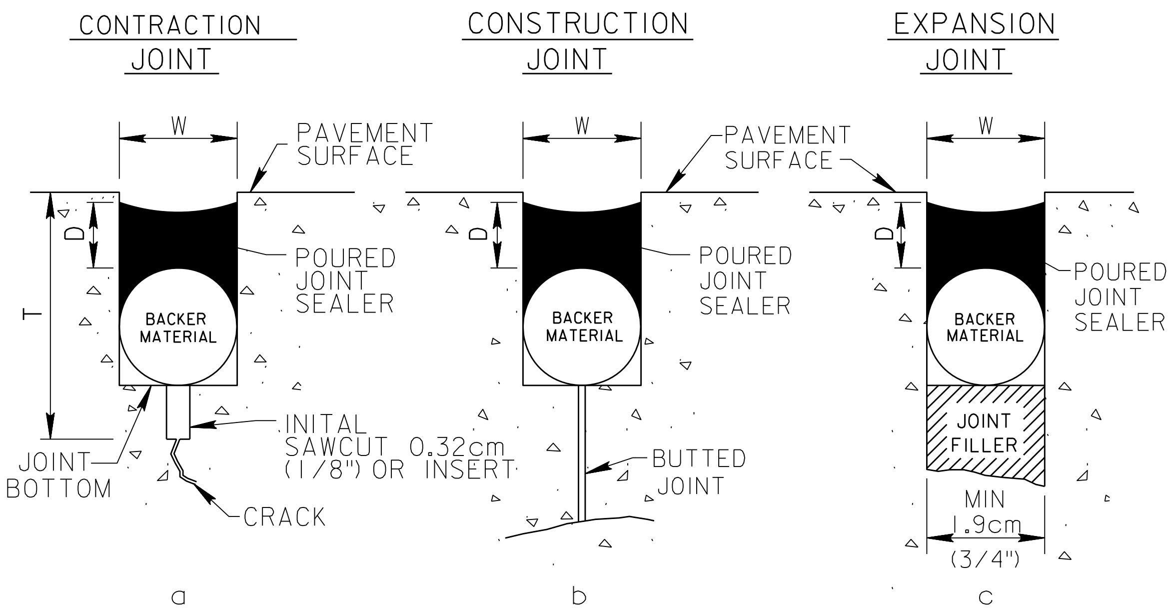
In rigid overlays in Air Force and Army Type A traffic areas, longitudinal contraction joints that would coincide with an expansion joint in the base pavement will be doweled. For longitudinal contraction joints at runway or taxiway centerlines, the joint shall be dowelled. Dowel size and spacing will be as specified in Table 12-2.

12-6.1.6 **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 the base strength and the angularity of coarse aggregate and shorter spacing of joints.

Table 12-2. 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 (.75-in) bar
203-292 (8-12)	406 (16)	305 (12)	25-mm (1-in) bar
305-394 (12.5-15.5)	508 (20)	381 (15)	25- to 30-mm (1- to 1.25-in) bar
406-521 (16-20.5)	508 (20)	457 (18)	30- to 40-mm (1.25- to 1.5-in) bar
533-648 (21 -25.5)	610 (24)	457 (18)	50-mm (2-in) bar
660 (26) or more	762 (30)	457 (18)	76-mm (3-in) bar

Figure 12-33a. Joint Sealant Details for Plain Concrete Pavements (Backer Material)

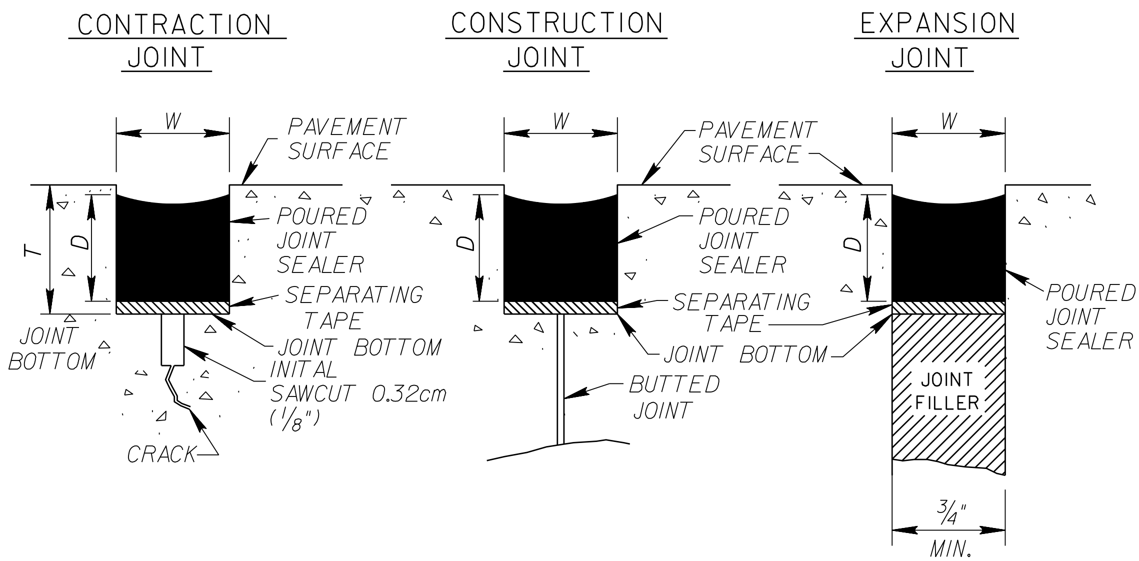


W = WIDTH OF SEALANT RESERVOIR (SEE TABLE)
D = DEPTH OF SEALANT (1.0 TO 1.5 x W)
FOR SILICONE USE (0.5 TO 1.0x w)
T = DEPTH OF INITIAL SAWCUT OR INSERT TYPE JOINT FORMER (CONTRACTION JOINT)
a. 0.64cm (1/4") SLAB THICKNESS FOR PAVEMENTS LESS THAN 30.5cm (12")
b. 7.62cm (3") FOR PAVEMENTS 30.5cm to 45.7cm (12"-18") *
c. 0.42cm (1/6") SLAB THICKNESS FOR PAVEMENTS MORE THAN 45.7cm (18") *
d. DIAMETER OF BACKER MATERIAL = W+0.32CM (1/8")
*DESIGNER MAY WANT TO CONSIDER REQUIRING 0.64cm (1/4") SLAB THICKNESS

JOINT SPACING FT	WIDTH IN.	
	MIN	MAX
< 25	1/2	5/8
25-50	3/4	7/8
> 50	1.0	1-1/8

NOTE: TOP OF SEALANT WILL BE 0.32CM (1/8") TO 0.64CM (1/4") BELOW TOP OF PAV'T. IN AREAS TO BE GROOBED, THE JOINT SEAL SHALL BE RECESSED BELOW THE DEPTH OF THE GROOVES

Figure 12-33b. Joint Sealant Details for Plain Concrete Pavements (Separating Tape)



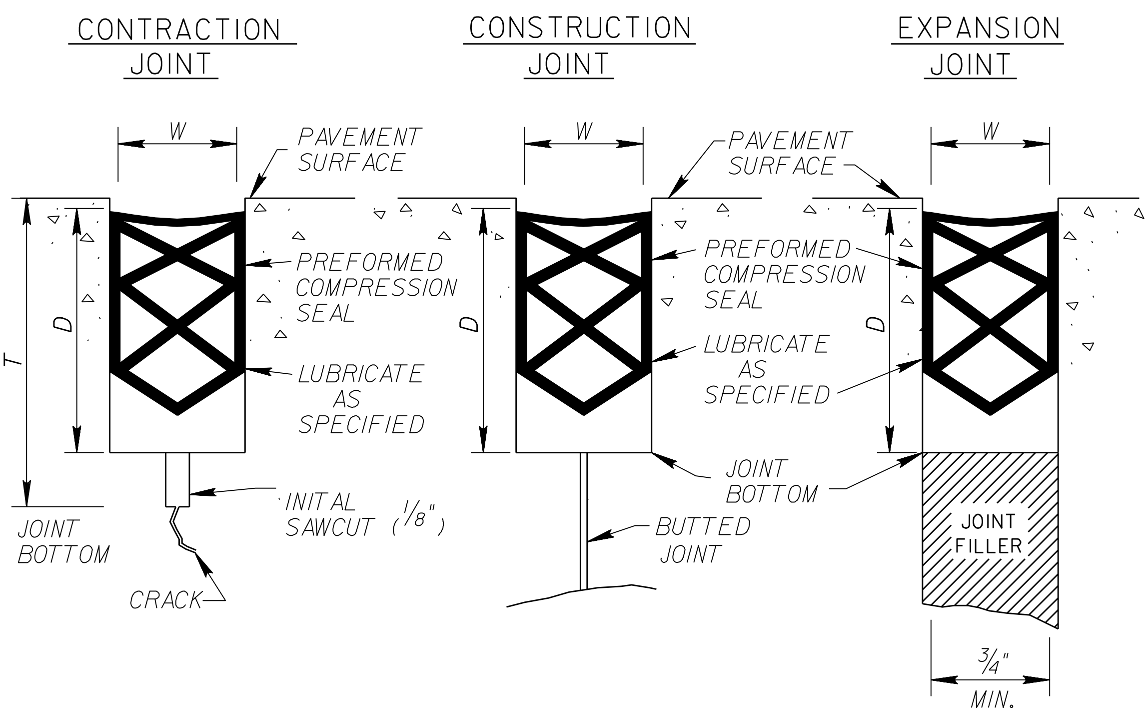
W = WIDTH OF SEALANT RESERVOIR (SEE TABLE)
D = DEPTH OF SEALANT (1.0 TO 1.5 x W)
T = DEPTH OF INITIAL SAWCUT
a. 1/4 SLAB THICKNESS FOR PAVEMENTS LESS THAN 30.5cm (12")
b. 7.62cm (3") FOR PAVEMENTS 30.5cm TO 45.7cm (12"-18") *
c. 1/6 SLAB THICKNESS FOR PAVEMENTS MORE THAN 45.7cm (18") *
* DESIGNER MAY WANT TO CONSIDER REQUIRING 1/4 SLAB THICKNESS

TABLE

JOINT SPACING FT	WIDTH, IN.	
	MIN	MAX
< 25	1/2	5/8
25-50	3/4	7/8
> 50	1.0	1-1/8

NOTE: TOP OF SEALANT WILL BE 1/4" (6.4mm) ± 1/8" (3.2mm) BELOW TOP OF PAV'T. IN AREAS TO BE GROOVED, THE JOINT SEAL SHALL BE RECESSED BELOW THE DEPTH OF THE GROOVES.

Figure 12-33c. Joint Sealant Details for (Preformed Compression Seals) Plain Concrete Pavements



T = DEPTH OF INITIAL SAWCUT

DEPTH & WIDTH OF UNCOMPRESSED SEAL SHALL BE AS RECOMMENEDED BY MANUFACTURER, EXCEPT THE DEPTH SHALL NOT BE LESS THAN 3.81 cm (1.5")

NOTE: TOP OF SEALANT TO BE 6.4mm (1/4") + 3.2mm (1/8") BELOW TOP OF PAV'T. IN AREAS TO BE GROOVED, THE JOINT SEAL SHALL BE RECESSED BELOW THE DEPTH OF THE GROOVES

COMPRESSION SEAL MUST BE IN COMPRESSION AT ALL TIMES.

TABLE

JOINT SPACING METERS (FT)	WIDTH, CM (IN)	
	MIN	MAX
6.1 (20)	1.27 (0.5)	1.6 (5/8)
6.1 (20) NAVY	0.95 (3/8)	----

12-6.2 **Construction Joints**
Doweled centerline longitudinal construction joints shall be used on runways and taxiways.

Construction joints may be required in both the longitudinal and transverse direction. Longitudinal construction joints will be required to separate successively placed paving lanes. These longitudinal construction joints generally will be spaced 6 m (20 ft) apart but may be more than one lane wide depending on construction equipment capability. Transverse construction joints will be installed when 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. Several types of construction joints are available for use as shown in Figure 12-34 and as described in paragraphs 12-6.2.1 and 12-6.2.2. 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.

Figure 12-34a. Construction Joints for Plain Concrete Pavements

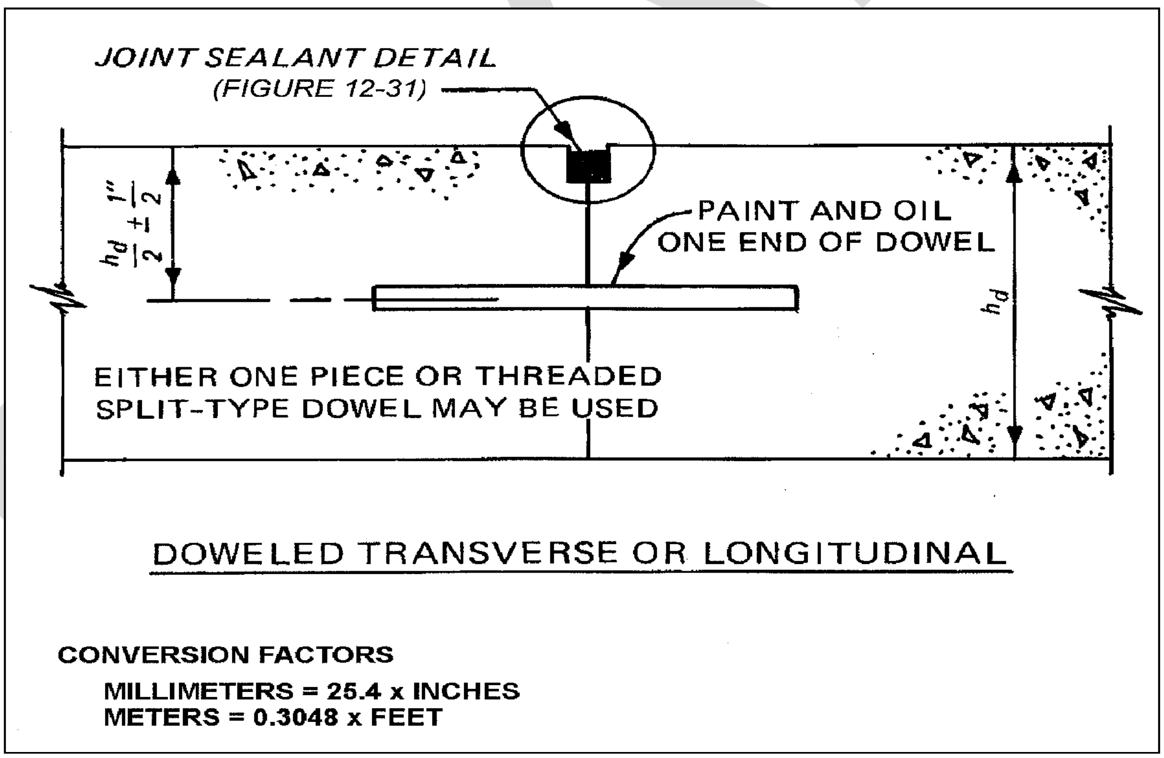


Figure 12-34b. Construction Joints for Plain Concrete Pavements

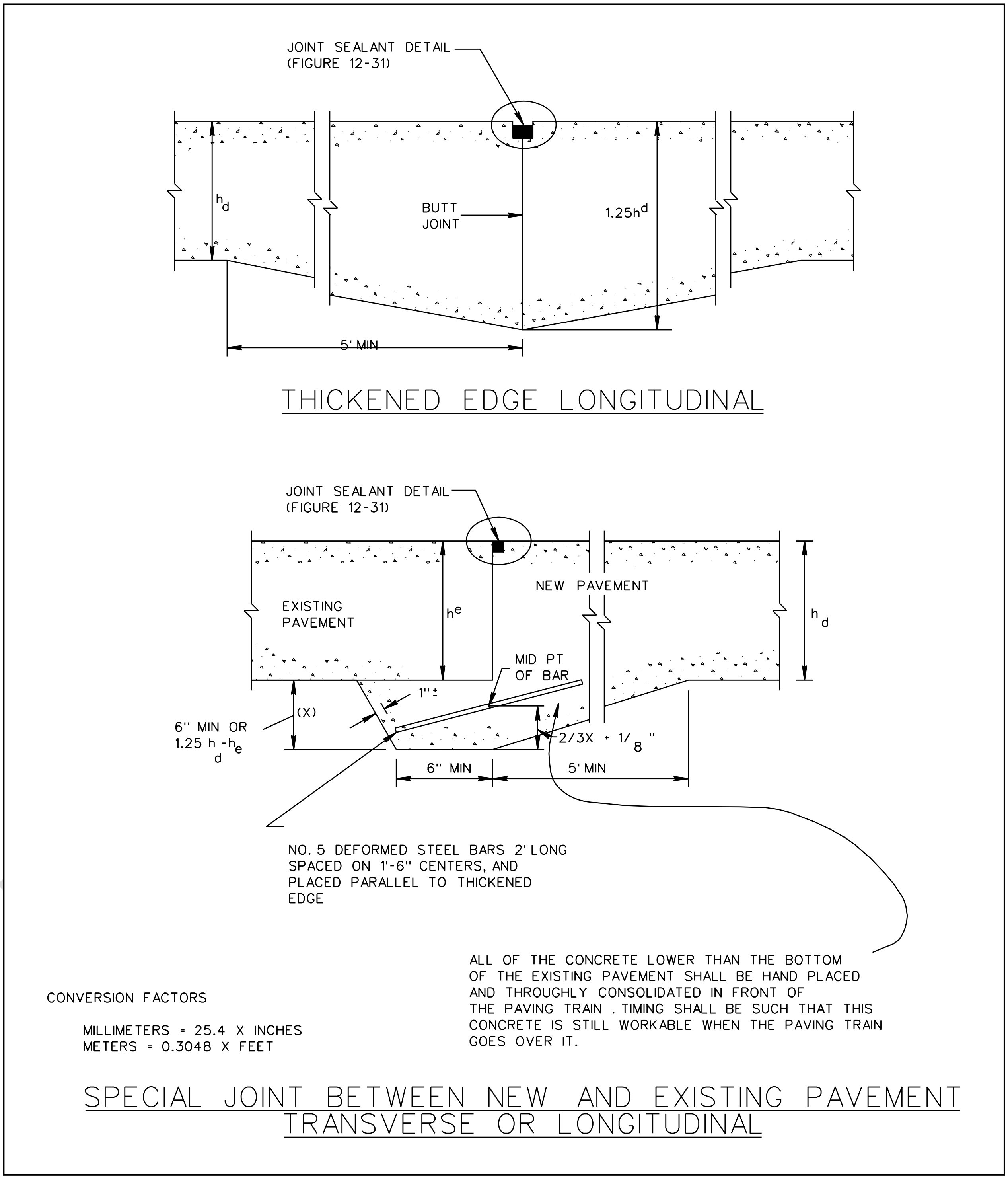
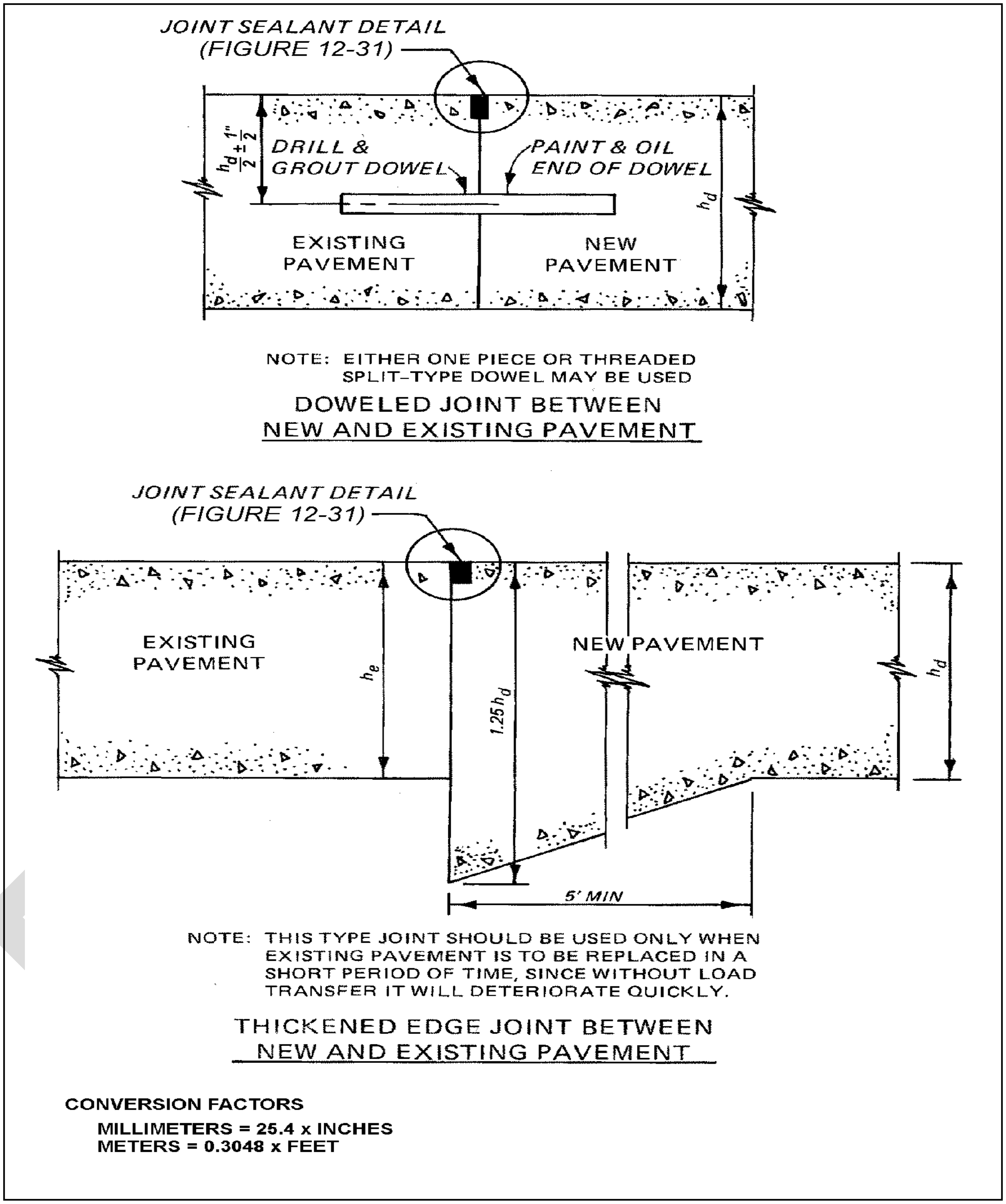


Figure 12-34c. Construction Joints for Plain Concrete Pavements



12-6.2.1 **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. Because the alignment and placement of the dowel bars are critical to satisfactory performance, however, the dowels must be carefully aligned, especially for slipformed concrete. Butt joints used for transverse construction joints must be doweled.

12-6.2.2 **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 m (5 ft) from the longitudinal edge. The thickened-edge butt joint is considered adequate for 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.

12-6.3 **Expansion Joints**

12-6.3.1 **Use of Expansion Joints**
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 12-35 and 12-36).

12-6.3.2 **Filler Material**
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 nonbituminous. The type and thickness of filler material and the manner of its installation will depend on the particular case. Usually a preformed material of 19-mm (0.75-in) 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 mm (0.25 in) in thickness when joints match or normal nonextruding-type material not less than 6.3 mm (0.25 in) in thickness when joints do not match.

12-6.3.3 **Expansion Joints and Adjoining Structures**
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.

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, bollards, and hydrant refueling outlets. The thickened-edge-type expansion joint will normally be best suited for these places.

Figure 12-35. Expansion Joints for Plain Concrete Pavements

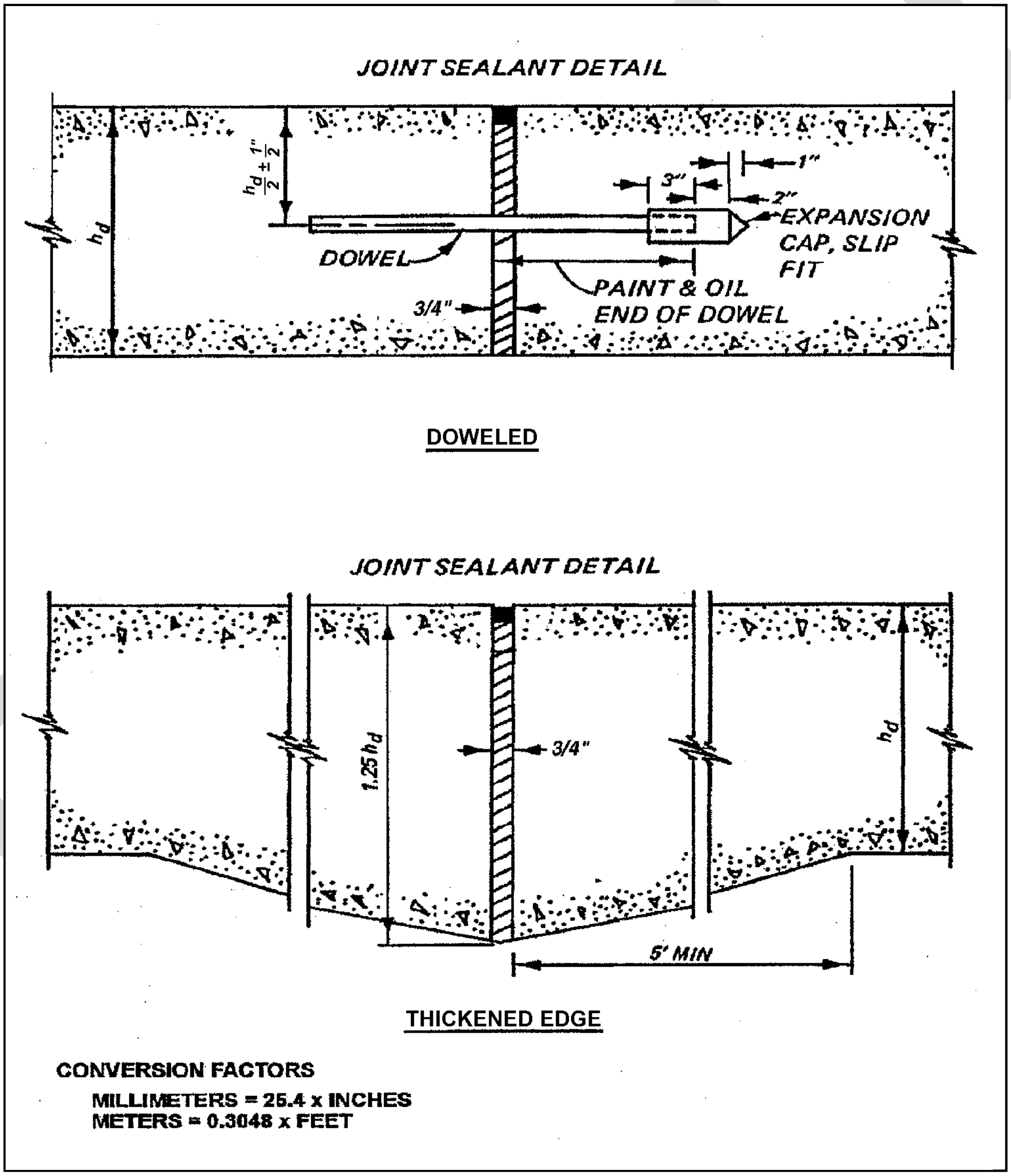
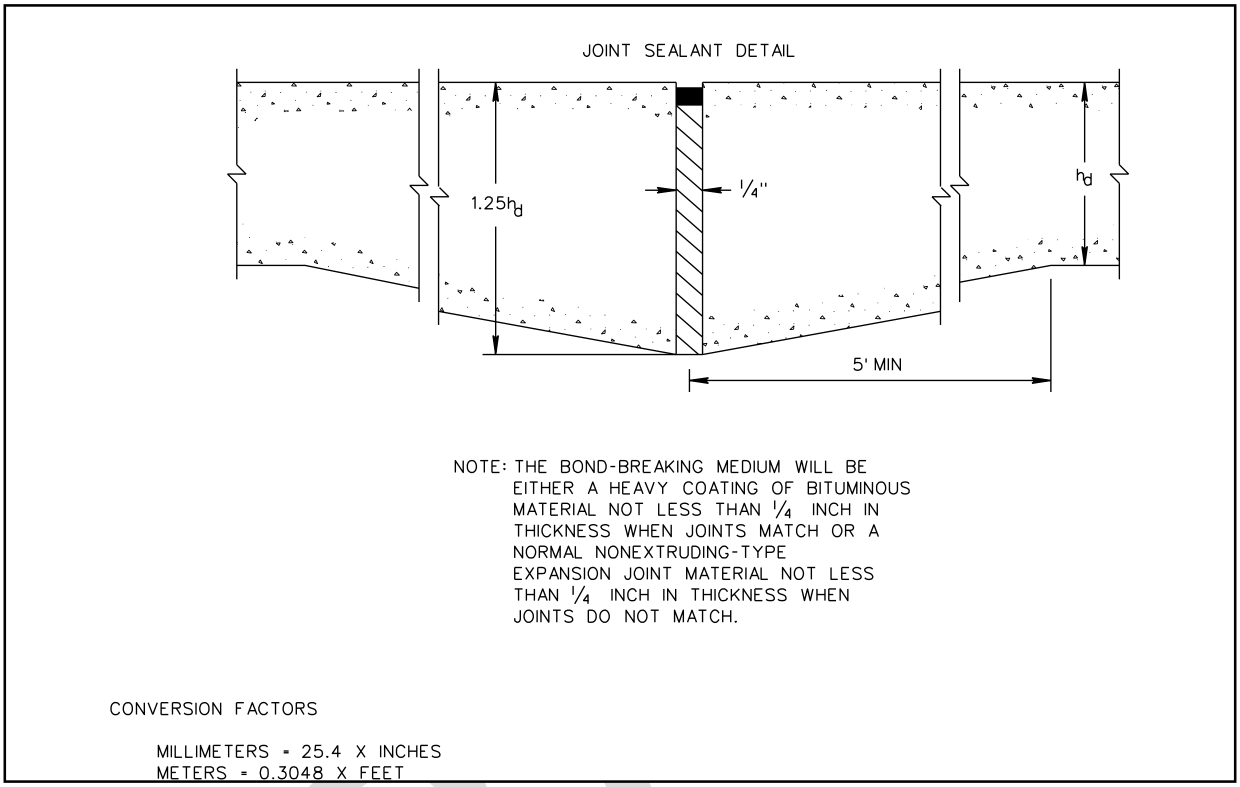


Figure 12-36. Slip Joints for Plain Concrete Pavements



12-6.3.4 Expansion Joints within Pavements

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 on 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.

12-6.3.4.1 Longitudinal Expansion Joints

Longitudinal expansion joints within pavements will be of the thickened-edge type (Figure 12-35). 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.

12-6.3.4.2 Transverse Expansion Joints

Transverse expansion joints within pavements will often be the doweled type (Figure 12-35). In certain instances, it may 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 12-36). 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 the first transverse joint past the fillet as shown in Figure 13-2b.

12-6.4 **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 12-2). 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 12-32, 12-34, and 12-35 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.

12-6.5 **Special Provisions of Slipform Paving**

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. These requirements apply:

- 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 shall be commensurate with the thickness of the pavement at the header.

12-6.6 **Joint Sealing**

All joints will be sealed with a suitable sealant to prevent infiltration of surface water and solid substances. The Air Force and Army require all joints in pavement to be sealed with preformed compression seals unless waived by the MAJCOM or USACE-TSC. Jet fuel-resistant (JFR) sealants will be used in the joints of aprons, warm-up holding pads, hardstands, wash racks, 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-200E or ASTM D7116. Non-JFR sealants will conform to ASTM D3406, D6690, and USACE CRD-C 525. Silicone sealants meeting ASTM D5893 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-200E. An optimal sealant that meets both the heat- and blast-resistant JFR and non-JFR sealant requirements is a preformed compression seal conforming to ASTM D2628 and the lubricant-adhesive with ASTM D2835. 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. Compression seals will remain effective five to seven times as long as liquid sealants. A field-molded sealant is generally required where new pavement abuts existing pavement.

12-6.7 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 joint layouts are shown in Figures 13-2a and 13-2b and are discussed in section 12-8 of this chapter.

12-7 JOINTS FOR NAVY AND MARINE CORPS PAVEMENTS

12-7.1 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 12-35 for expansion joint details.

12-7.2 Contraction (Weakened Plane) Joints

Contraction joints should be used to control cracking in the pavement due to volume changes resulting from a temperature 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. The saw-cut joint or formed groove provides a weakened plane that 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 on slab thickness and spacing of the construction joints. See Figure 12-32 for contraction joint details.

12-7.3 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 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 m (20 ft) apart but may be multiple lane width depending on the construction equipment.

12-7.3.1 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.

12-7.3.2 Longitudinal Construction Joints

Construct longitudinal construction joints as shown in Figure 12-34 and indicated in paragraphs 12-7.3.2.1 to 12-7.3.2.3.

12-7.3.2.1 Keyed Joint

Keyways were used extensively in the past to provide load transfer along longitudinal construction joints; however, a substantial amount of keyway failure has been experienced over the years. For this reason, keyed joints will no longer be used for any new or replacement pavements for Navy or Marine Corps airfield pavements. Doweled construction joints shall be use in place of the keyed joints.

12-7.3.2.2 Butt Joint

A butt joint may be used for longitudinal construction joints on pavements less than 228 mm (9 in) thick constructed with a stabilized base.

12-7.3.2.3 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.

12-7.4 Joint Spacing

The standard slab size for pavements is 3.8 by 4.6 m (12.5 by 15 ft). Transverse joint spacing is 4.6 m (15 ft), and longitudinal joint spacing is 3.8 m (12.5 ft). For slabs having a thickness greater than 305 mm (12 in), joint spacing can be increased to a maximum of 6.1 m (20 ft). The transverse joint spacing shall not vary from the longitudinal joint spacing by more than 25 percent.

12-7.5 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, a stabilized base, or a combination of these.

12-7.5.1 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.

12-7.5.2 **Dowel Bars**

Dowel bars are used to provide load transfer and prevent excessive vertical displacements of adjacent slabs. In some situations, the use of dowels is appropriate, such as for creating load transfer where tying in to existing pavements.

12-7.5.3 **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 228 mm (9 in) thick to provide improved load transfer and lower deflections and stresses. A stabilized base may also be used for pavements greater than 228 mm (9 in) thick to provide additional load transfer. Where thickened-edge joints are used, the stabilized base is not required.

12-7.6 **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 JFR sealants for all aprons. Use blast-resistant sealants for the first 305 m (1,000 ft) of runways and exits at runway ends. Use sealing compounds meeting ASTM D6690 or D3406 for taxiways and runway interiors.

12-7.6.1 **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 JFR (tar based) or non-JFR (typically asphalt based).

12-7.6.1.1 **Field Poured, Hot Applied**

This group of sealants includes rubberized asphalt sealant and rubberized tar sealant. Rubberized asphalt joint sealants must meet ASTM D6690 or D3406. Rubberized tar sealants must meet ASTM D7116.

12-7.6.1.2 **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 Navy recommends the use of silicone sealants that conform to NFGS 02522, 02562, and ASTM D5893 in lieu of sealants that meet Federal Specification SS-S-200E.

12-7.6.1.3 **Preformed Compression Seals**

The most common type of preformed compression seal is the neoprene compression seal. Neoprene compression seals must satisfy ASTM D2628. 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.

12-7.6.2 **Joint Reservoir Design**

The joint reservoir must be properly designed so that the joint sealant can withstand compressive and tensile strains.

12-7.6.2.1 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. For silicones, the shape factor should be 2.0. The dimensions of the joint sealant and reservoir are shown in Figure 12-33. 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. The joint reservoir and sealant dimensions shown in Figure 12-33 are based on a slab size of 6 by 6 m (20 by 20 ft).

12-7.6.2.2 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-mm-wide (1-in-wide) neoprene compression seal is from 13 to 20 mm (0.5 to 0.8 in). 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 12-33. 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.

12-8 JOINTING PATTERN FOR RIGID AIRFIELD PAVEMENTS

A proper jointing pattern for rigid airfield pavements is a critical item of design and construction for all military services. Not only is it important for a quality product, but it can and should promote efficiency for the construction contractor, and thus result in cost savings. Criteria for the type of joints, their location, and the maximum allowable spacing have been provided in previous paragraphs. This section focuses on appropriate and efficient layout of the jointing pattern. Laying out a good jointing pattern depends on experience and is more of an art than a science. The designer must learn to try various combinations until the optimum layout is achieved. Every productive hour spent on this task produces appreciable cost savings.

12-8.1 Changes to the Layout

All project joint layout drawings should display this prominent note: “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 the criteria. Once the joint layout is finalized, no change whatsoever should be made by field personnel unless the change is examined and approved in writing by the designer to be sure that it does not compromise the plan or violate the criteria.

12-8.2 Layout

Joint layouts should be as simple and as uniform as possible and meet all the 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 versus transverse joints) are laid out in the right direction for the contractor's efficient work—particularly where the area has irregular boundaries.

12-8.3 Spacing

Longitudinal construction joints should be spaced such that the widths of pilot lanes are all equal and any variability in total distance is accounted for 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.

12-8.4 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, pilot lanes should have widths in multiples of 6 in, or, if metric is used for the project, multiples of 150 mm. Extensions to the paver are made easily 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 those of the pilot 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 to reduce the chance of measurement error during construction.

12-8.5 Transverse Contraction Joints

For transverse contraction joints, the spacing should be the same as for 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 doing so results in easily measured spacing. Otherwise, make spacing an easily remembered and 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 (i.e., no fractional inches or odd metric units, and as little variation as possible).

12-8.6 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 and modified only as absolutely necessary. As much as possible, care should be taken to 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 while following criteria and minimizing as much as possible the need for odd-shaped reinforced slabs—which is an expense to be avoided. When abutting existing PCC pavement, attempt to match the existing joint pattern where possible.

Older pavements will often have 7.6-m (25-ft) joint spacing, whereas now the maximum allowed is usually 6 m (20 ft). For jobs of moderate size, if matching the existing joint pattern is possible, the new joint spacing can be made 7.6 m (25 ft) provided the existing pavement has shown no distress because of the 7.6-m (25-ft) spacing. Otherwise, use 3.8-m (12.5-ft) spacing. Either is acceptable, but the using service should be contacted since the choice of either spacing is a matter of preference.

12-8.7 Expansion Joints and Slip Joints

12-8.7.1 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 at 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, the intersection should be a thickened-edge joint, either an 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-mm-wide (3-ft-wide) strips of reinforcing should be installed along each side of the joint to prevent sympathetic cracks from forming in line with the mismatched joints. Normally, expansion joints of any kind should not be doweled if load transfer can be provided in another way. On some projects, doweled expansion joints have been used successfully, but expansion joints should be doweled only where no translation movements or stresses are expected.

12-8.7.2 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 under strength 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.

12-8.7.3 Slip Joints

Slip joints, 6-mm (0.25-in) 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-mm (0.25-in) thickness, slip joints are sufficient to prevent sympathetic cracking across the joint and thus eliminate the need for a 900-mm (3-ft) strip of reinforcing on each side of new-to-new construction.

12-8.8 Special Joint

A “special joint,” as shown in Figure 12-34b, can be used to provide load transfer on the existing side of a new PCC to old PCC joint. This special joint can be used under the conditions listed below. Although somewhat expensive, this is an excellent joint when constructed properly, but close supervision in the field is required 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.)

A special joint (undercut) (Figure 12-34b) may be used at the juncture of new and existing pavements for these conditions:

- When load-transfer devices (dowels) or a thickened edge was not provided at the free edge of the existing pavement
- 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
- 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 unsuitable

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.

12-8.9 **Tied Joints (Navy Only)**

Tied joints are construction or contraction joints held together by deformed steel reinforcing bars. Tied joints are seldom used for airfield pavement; however, two instances occur:

(1) As required and shown in Figure 12-31, “Typical Jointing,” the dowel bars in the last three joints might be replaced by tie bars The situation must be evaluated and existing service experience observed to prevent tying two slabs that have conditions (dimensions or aggregate properties) that 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 m (10 ft) must be removed and not less than 3 m (10 ft) 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.

12-8.10 **PCC to AC Intersections**

Figures 12-37 through 12-40 show various types of joints to use for the juncture of PCC and AC pavements:

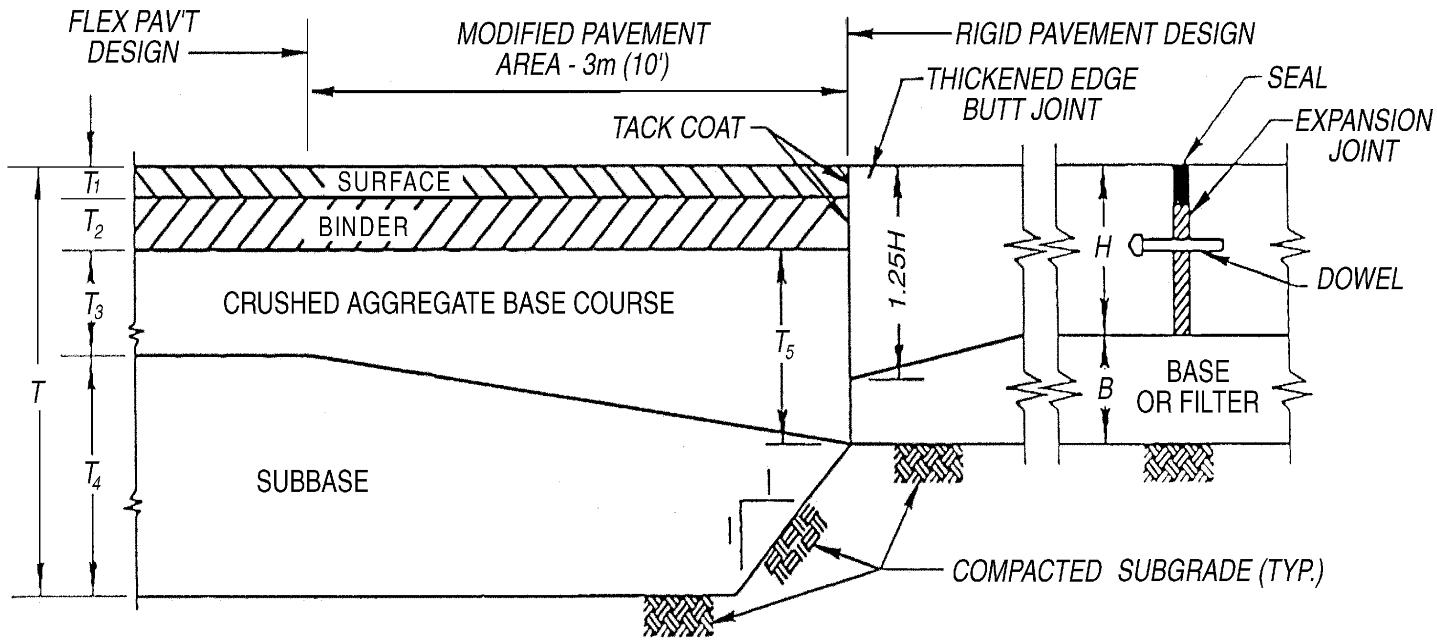
- Figure 12-37: This joint is to be used for most transverse joints that will receive aircraft traffic at Army installations and for all transverse joints that will receive aircraft traffic at Air Force installations.

- Figure 12-38: 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 that service prefers. This joint should not be used for Air Force pavements.

- Figures 12-39 and 12-40 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.

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.

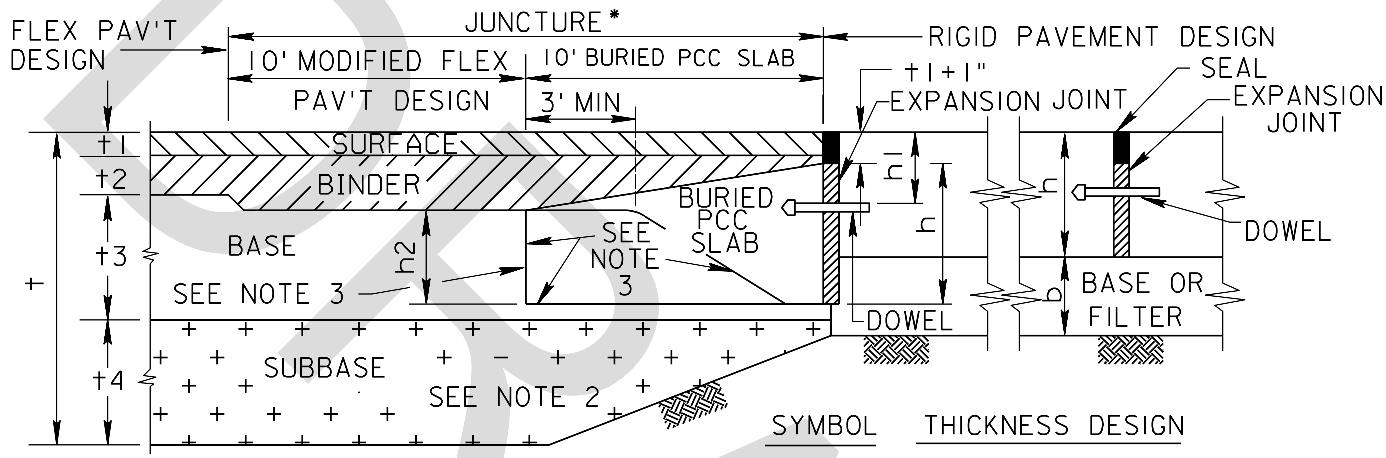
Figure 12-37. Rigid-Flexible Pavement Junction (Army or Air Force)



SYMBOL	THICKNESS DESIGN
H	DESIGN THICKNESS OF PCC
B	THICKNESS OF BASE OR FILTER
T	DESIGN THICKNESS OF FLEXIBLE PAVEMENT
T_1	DESIGN THICKNESS OF SURFACE COURSE

SYMBOL	THICKNESS DESIGN
T_2	DESIGN THICKNESS OF BINDER COURSE
T_3	DESIGN THICKNESS OF BASE COURSE
T_4	DESIGN THICKNESS OF SUBBASE COURSE
T_5	$(H+B)-(T_1+T_2)$ OR $2(T_3)$, WHICHEVER IS GREATER

Figure 12-38. Rigid-Flexible Pavement Junction (Not for Air Force)



COMPACTED SUBGRADE

SYMBOL	THICKNESS DESIGN
h	DESIGN THICKNESS OF PCC
h1	$\frac{h + t1 + 1"}{2}$
h2	h-(t1+1") BUT NOT LESS THAN 6"
b	THICKNESS OF BASE OR FILTER
t	DESIGN THICKNESS OF FLEXIBLE PAVEMENT
t1	DESIGN THICKNESS OF SURFACE COURSE
t2	DESIGN THICKNESS OF BINDER COURSE
t3	DESIGN THICKNESS OF BASE COURSE
t4	DESIGN THICKNESS OF SUBBASE COURSE
t5	h-h2 BUT NOT LESS THAN t2

*SEE NOTES

NOTES:

1. COMPACTED BASE TO DASHED LINE. CUT OUT TO SOLID LINE NOT DISTURBING THE MATERIALS OUTSIDE LIMITS OF BURIED SLAB.
2. MATCH BOTTOM OF BURIED SLAB OR TOP OF ADJACENT SUBGRADE, WHICHEVER IS DEEPER
3. PLACE PCC BURIED SLAB DIRECTLY AGAINST CUT BACK BASE COURSE. NO FORM WILL BE USED.
4. TOP LIFT OF BINDER COURSE TO BE PLACEDAND ROLLED TRANSVERSELY. SURFACE COURSE PLACED AND ROLLED LONGITUDINALLY STOPPING ROLLERS ON RIGID PAVEMENT.
5. FOR HEAVY LOAD DESIGN h2 NOT LESS THAN 6 IN. FOR LIGHT LOAD DESIGN, h2 NOT LESS THAN 4 IN.
6. JOINT THE BURIED SLAB THE SAME AS THE ADJACENT RIGID PAVEMENT

REF: DM 21.3/TM 5-825-2/AFM 88-6, CHAPTER 2, FLEXIBLE PAVEMENT DESIGN FOR AIRFIELDS

*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.

Figure 12-39. PCC to AC Joint Detail (Removal and Construction)

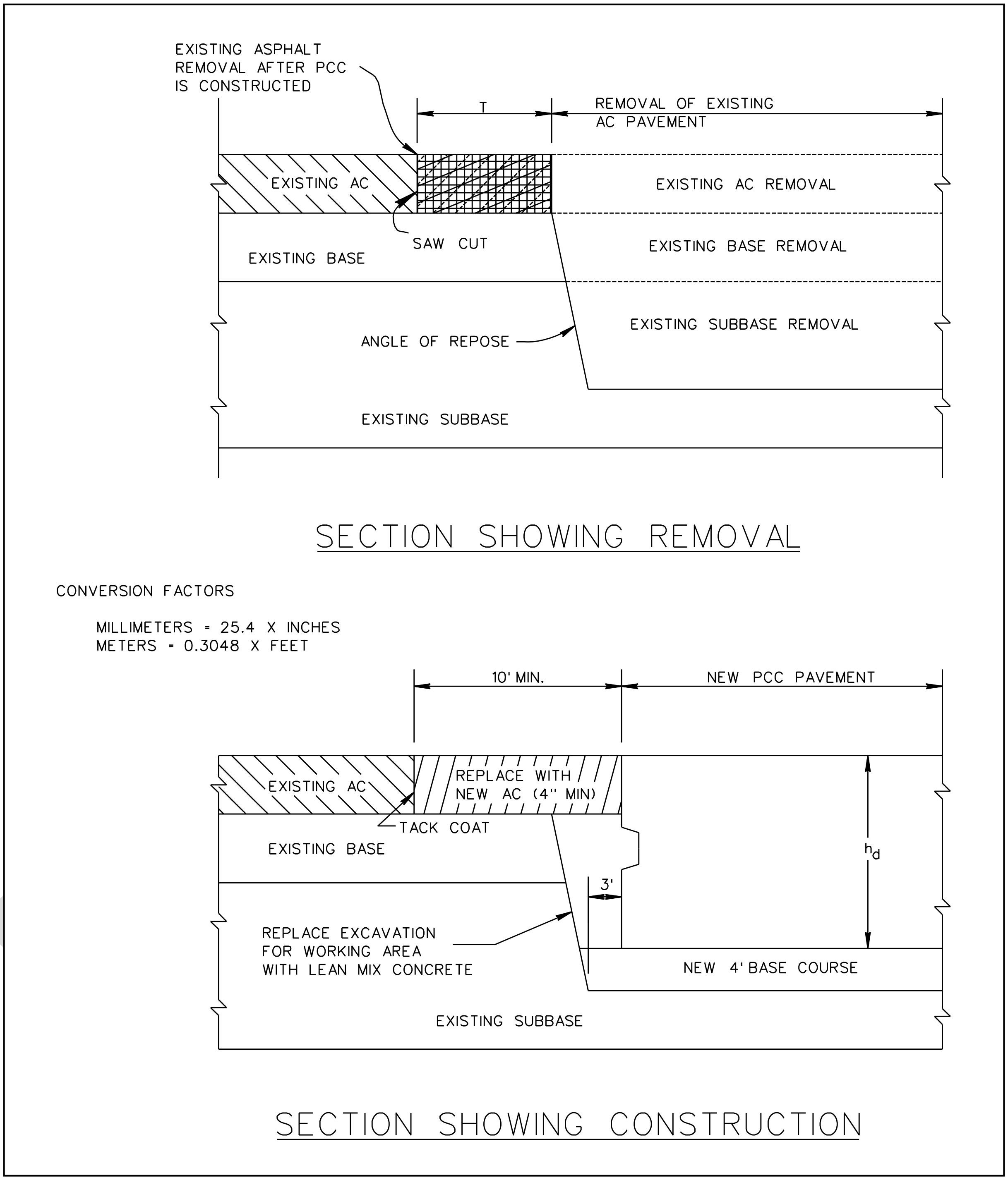
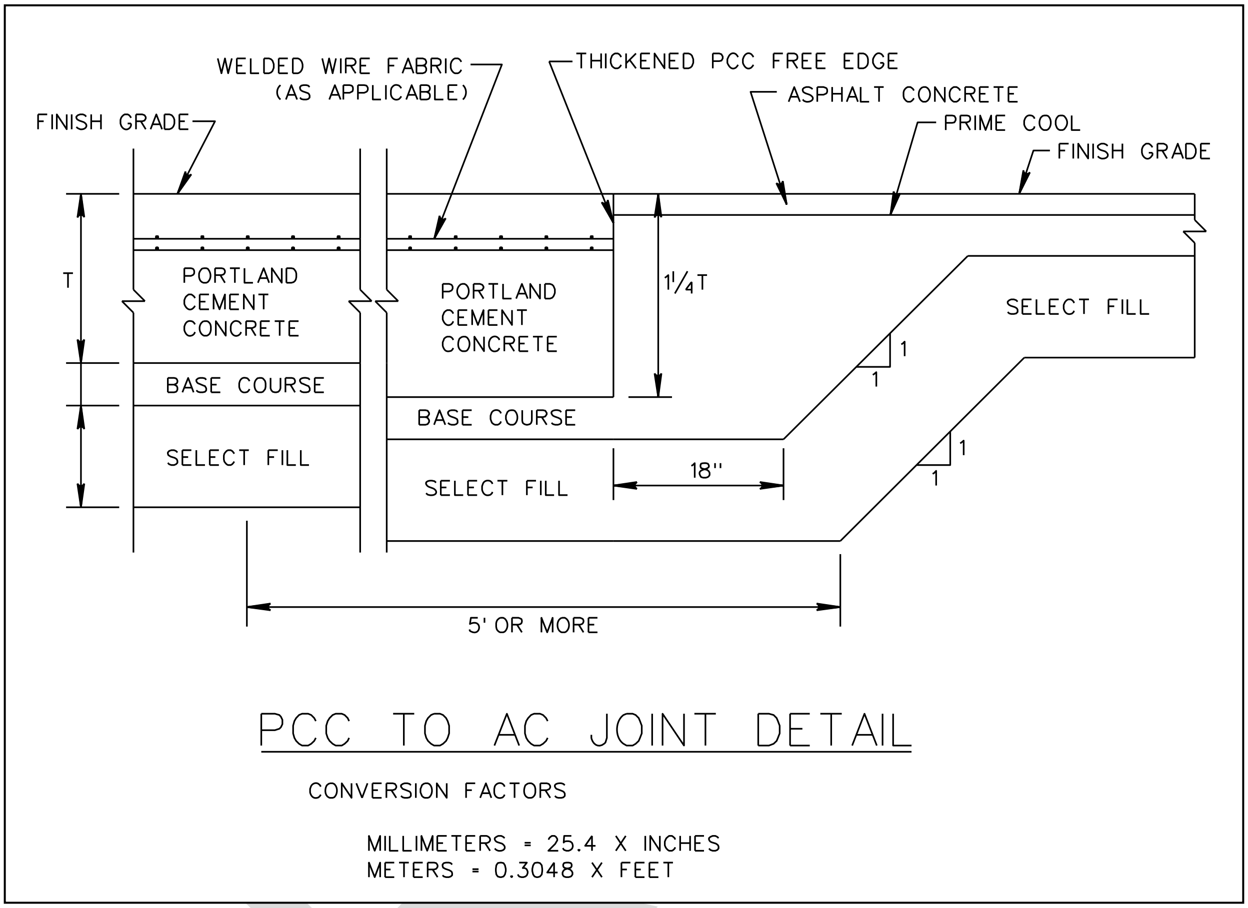


Figure 12-40. PCC to AC Joint Detail (Very Little Traffic Expected)



12-8.11 Sample Joint Layouts

Figures 12-41 through 12-45 are samples of various typical jointing patterns. The following subparagraphs provide an explanation of the significance and details of each figure.

12-8.11.1 Figure 12-41

This figure shows a perfect jointing pattern for a rectangular pavement with easily divided boundary dimensions. Unfortunately, such regular dimensions and shapes do not occur often—particularly in all the replacement and repair work that is currently required.

12-8.11.2 Figure 12-42

The dimensions in this figure are shown as metric. This figure shows the same 30.4 m (100 ft) by 42.7 m (140 ft) pavement as does Figure 12-28, with lane width changes to match even increments of metric measure. Note that the longitudinal construction joints (at the bottom of the page) have been evenly spaced across the 30.4 m (100 ft). This may look nice on paper, but it requires the contractor to set the width of the paver for an odd width, which is 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 width 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-m (19.7-ft) width and a single fill-in lane at an odd width. (The width of the fill-in lanes is not as critical.)

The right side of the figure shows 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 the joint sawing crew (usually working at night) more likely to 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 m (19.7 ft) and one take-up space of 6.56 m (21.5 ft), is much easier for the joint sawing crew to work with and therefore they are much less likely to get a joint out of line. Always make joint layouts as simple as possible within the criteria.

12-8.11.3 **Figure 12-43**

This figure, for a 54-m-wide (180-ft-wide) pavement, shows nice, easy spacing of longitudinal and transverse joints if all the dimensions are in the IP system. Note the spacing of 6 m (20 ft) by 6 m (20 ft) at the top and right side of the drawing. If the same overall width has to be designed in metric, however, it gets more complicated. Still, a variety of spacing can be used feasibly for transverse contraction joints.

Longitudinal construction joints are a problem, however. The bottom of the figure shows three possible solutions. The top solution is very pretty and easy to design, but it requires the contractor to adjust the paver to an odd width for the pilot lanes—an extra expense. The middle solution shows a good jointing pattern, with nine lanes at 6 m (20 ft) and two fill-in lanes at 6.36 m (20.9 ft)—well within the criteria for shape. The bottom solution is also a good jointing pattern, with five pilot lanes at 6.0 m (20 ft) and four fill-in lanes at 6.18 m (20.3 ft). Neither of the last two options requires the contractor to adjust the paver to anything other than an even width or to make any changes in adjustment.

12-8.11.4 **Figure 12-44**

This figure shows a new PCC pavement intersecting an existing PCC pavement at a 90-degree angle. Such an intersection requires a joint that can tolerate movement, both at right angles to the joint and along the joint, as well as provide load transfer across the joint. Several approaches are possible:

(a) One approach would be to drill and grout dowels in the existing PCC and put in a doweled expansion joint at the intersection. This approach 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.

(b) 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 therefore no true load transfer across the joint can take place.

(c) The usual approach is to provide joints as shown in the figure. A doweled construction joint is installed at the intersection of the two pavements, with the 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.

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. (Lining up these joints is not always feasible, but an attempt should be made.) Lining up joints on both sides at the expansion joint is not necessary. This permits making an easy change from the existing joint spacing to a different spacing in the new pavement.

Also note that the 900-mm (3-ft) ends of joints intersecting curved fillets must be angled to be perpendicular to the curve at their intersection.

12-8.11.5 **Figure 12-45**

This figure illustrates what can happen to a good jointing pattern when a joint sawing crew makes a mistake. This situation occurred on a large PCC apron at a military base. The crew sawing the 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; however, on the other 15 percent of the longitudinal lanes, the crew measured the transverse joints backward, with uniform spacing at the right end and lesser spacing at the left end. Outside of the fact that sympathetic cracking will occur at the mismatched joints, structurally the pavement is excellent, with a good surface finish.

Note: Regardless of the shape and dimensions of the pavement to be constructed, the simplest jointing pattern that conforms to the criteria will be best.

Figure 12-41. Sample Jointing Pattern

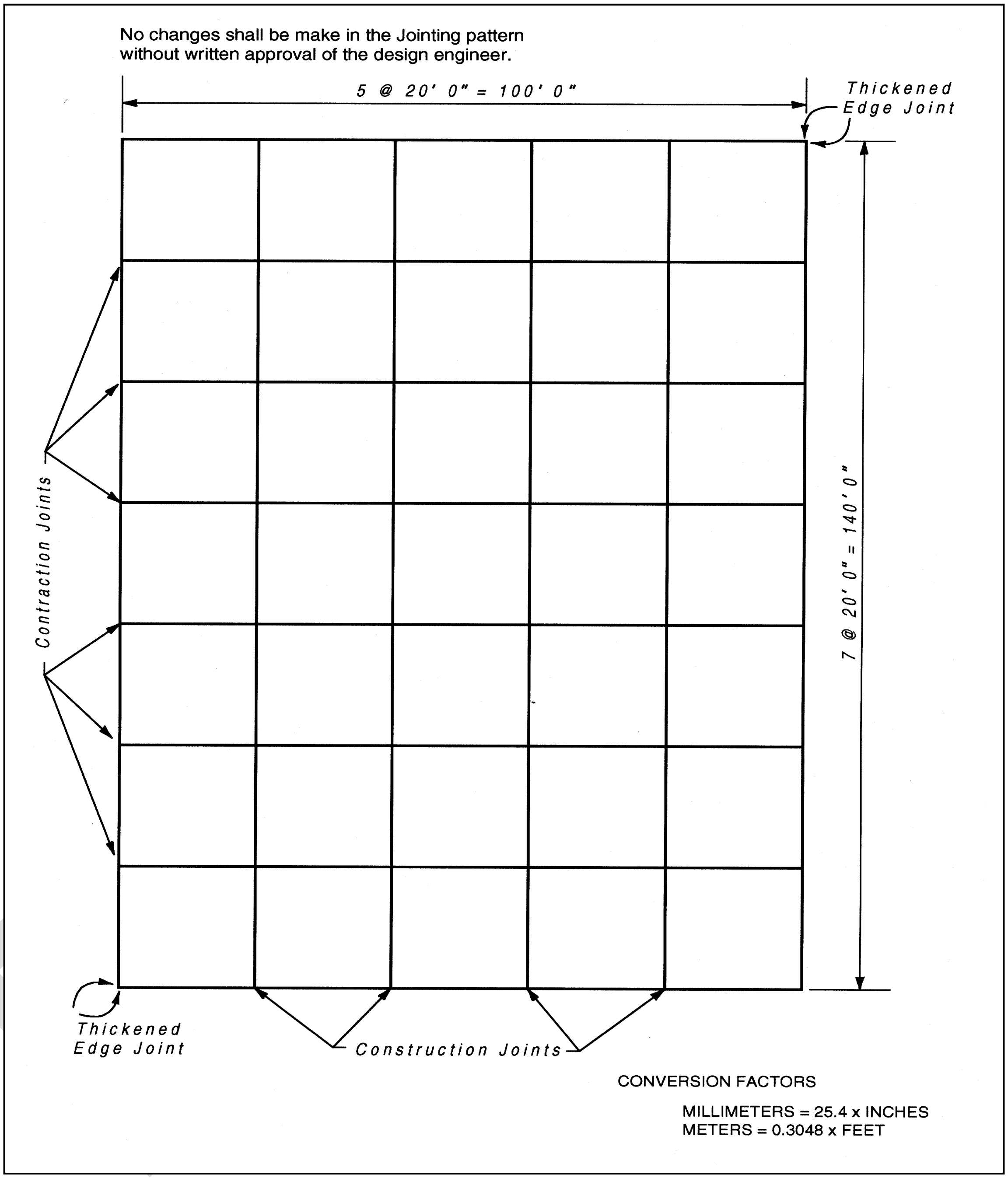


Figure 12-42. Sample Jointing Pattern (SI Units)

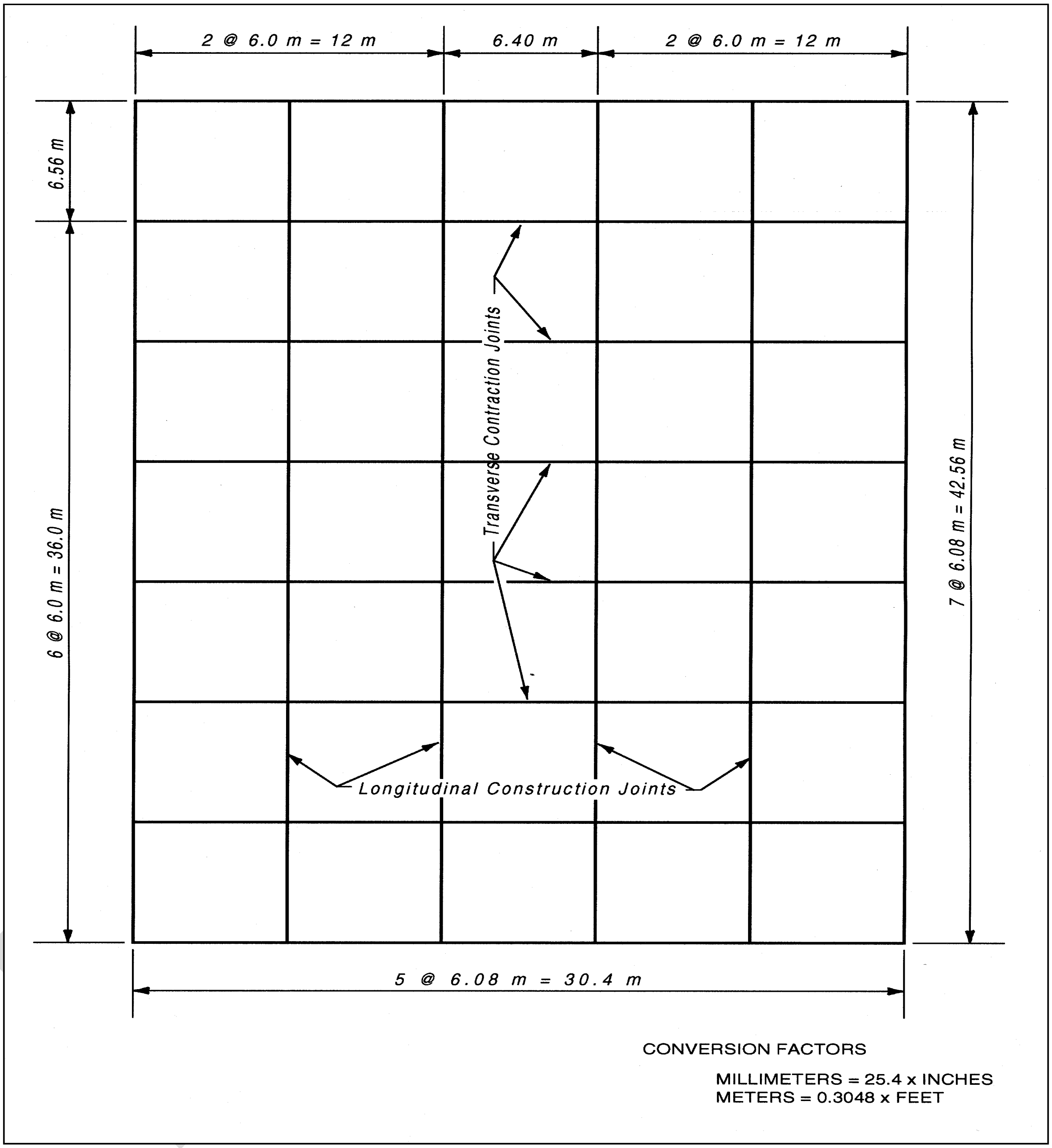


Figure 12-43. Sample Jointing Pattern for 180-ft-wide Pavement

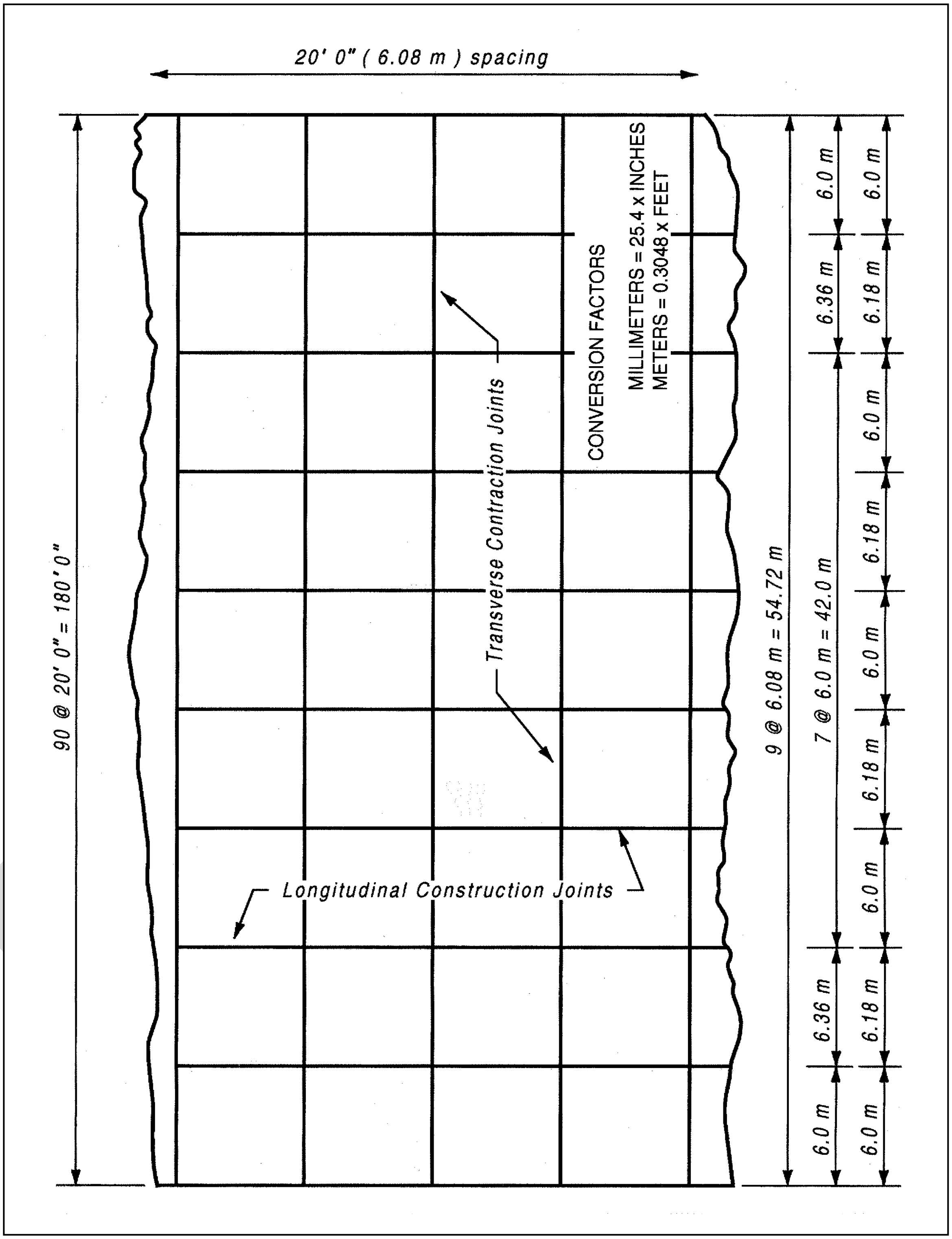


Figure 12-44. Sample Jointing Pattern at an Intersection

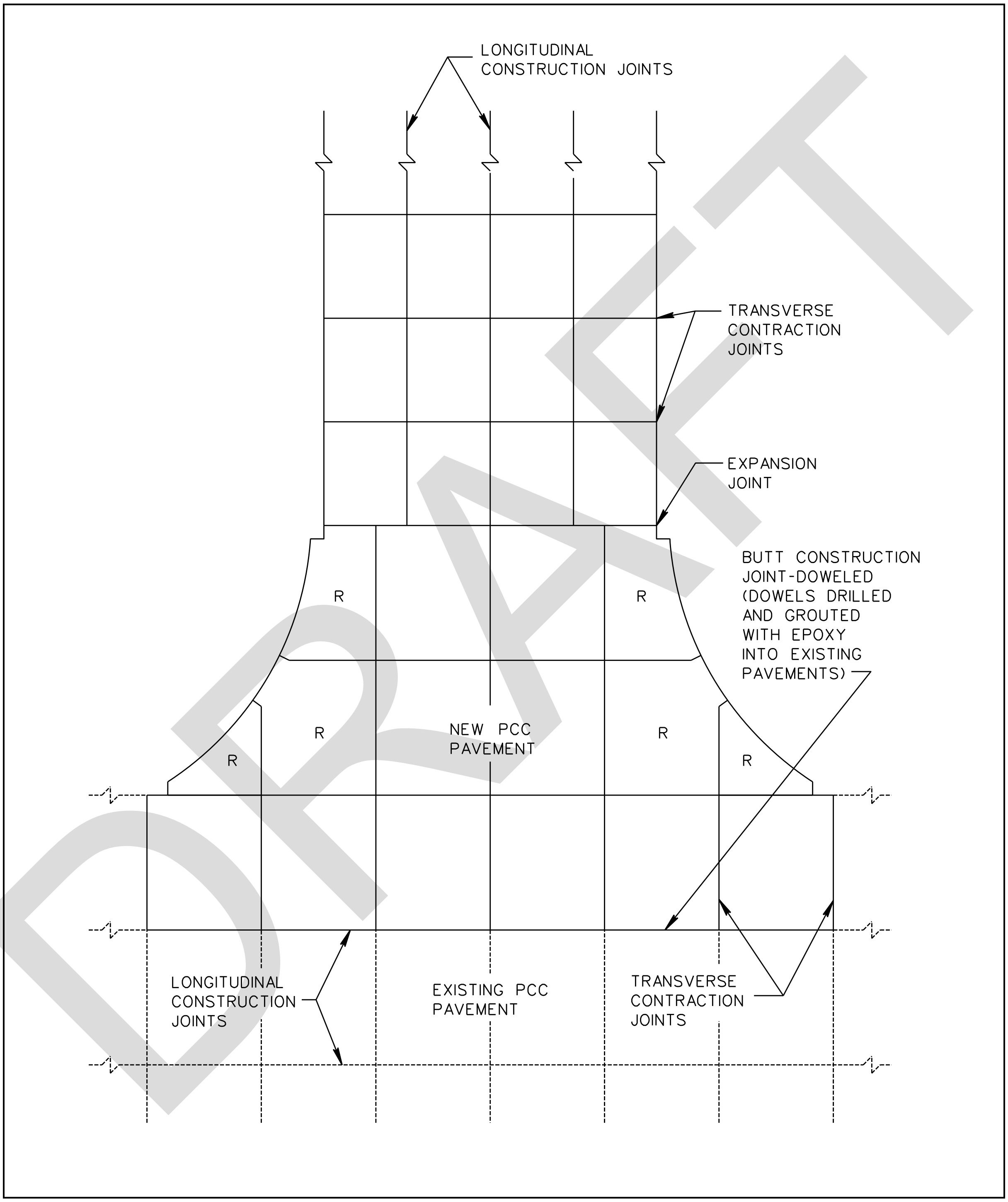
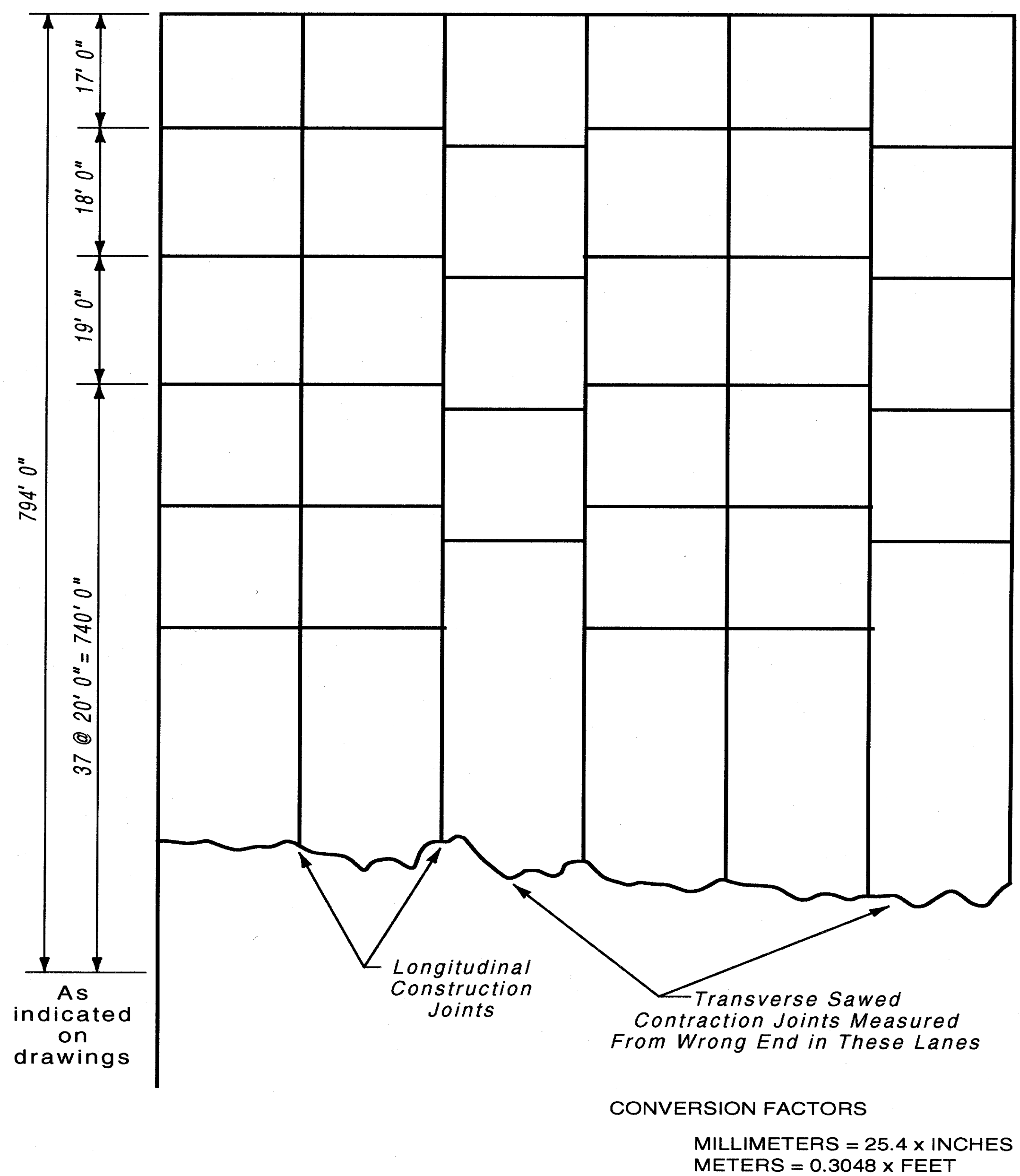


Figure 12-45. Effects of Confusion in Sawing Joints



CHAPTER 13

REINFORCED CONCRETE PAVEMENT DESIGN

13-1 OVERVIEW

These designs are applicable to Army and Air Force pavements but will not normally be used for DOD projects; however, reinforced concrete may be considered for special or unusual design conditions on a case-by-case basis and must be approved by USACE-TSC for the Army, the MAJCOM pavements engineer for the Air Force, or the Naval Facilities Engineering Service Center for the Navy and Marine Corps. The exception to this is in odd-shaped slabs and mismatched joints where reinforcing is required.

13-2 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 reinforced concrete pavements are approved for use, the design procedures for these pavements will be the same for the Navy and Marine Corps as for the Army and Air Force.

13-2.1 Thickness

The thickness design for reinforced concrete pavement is similar to plain concrete pavement design but 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 the inconsistent performance of thin reinforced pavements, for new construction, the thickness shall not be reduced from that of plain concrete.

13-2.2 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 reinforcement range from 0.05 to 0.25 percent of the area.

13-2.3 Joints

The maximum slab size for reinforced concrete pavements is a function of the slab thickness, the yield strength of the reinforcing steel, and the percent of reinforcement. The slab size is commonly 7.6 m (25 ft) square. All joints in reinforced concrete pavements, with the exception of 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.

13-3 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 pavement. The design procedure presented in this chapter 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.

13-4 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 shall not be based solely on the economics involved. In certain situations, such as temperature differentials, moisture changes, or frost, excessive cracking may develop and reinforcement will be required to control this cracking without any reduction in pavement thickness requirements.

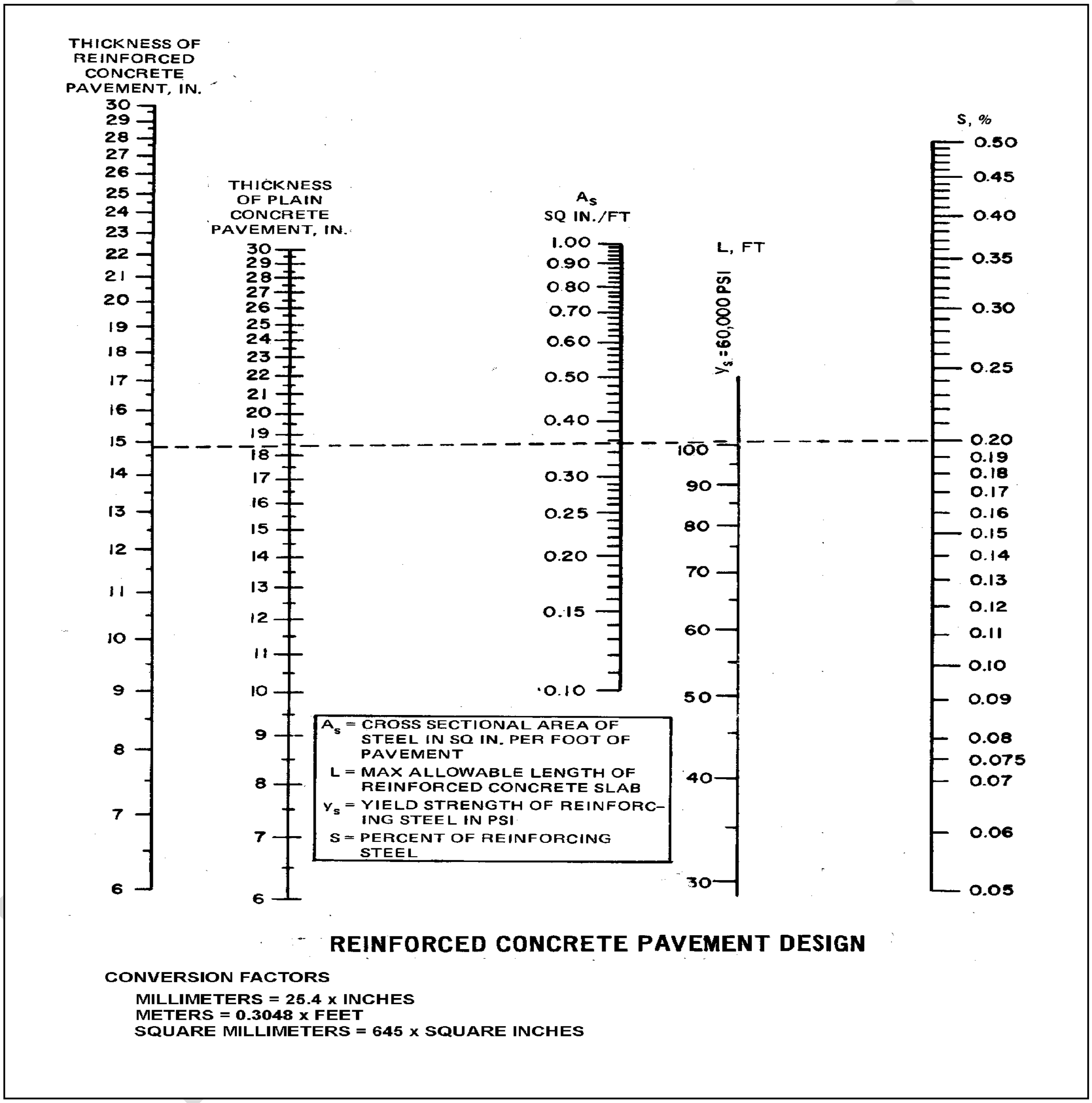
13-5 REDUCED THICKNESS DESIGN: ARMY AND AIR FORCE

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 and to meet surface grade requirements. Since these changes in thickness cannot be made at the surface, reinforcement can be used to reduce the required thickness and thereby avoid removal and replacement of pavements or overdesigns. In other instances, 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 the slabs.

13-5.1 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 outlined in Chapter 12 will be used to determine the thickness of plain concrete. This thickness 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 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 a full and half inch, the thickness will be rounded upward.

Figure 13-1. Reinforced Concrete Pavement Design



13-5.2 **Determination of Maximum Reinforced Concrete Pavement Slab Size**
The maximum length or width of the reinforced concrete pavement slabs depends largely on the resistance to movement of the slab on the underlying material and the yield strength of the reinforcing steel. The latter factor can be determined easily, 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 this equation:

$$
\begin{aligned}
W \text{ or } L &= 0.2224\sqrt[3]{h_d\,(y_s S)^2} \quad \text{for SI Units} \\
W \text{ or } L &= 0.0777\sqrt[3]{h_d\,(y_s S)^2} \quad \text{for English Units}
\end{aligned}
\qquad (13\text{-}1)
$$

where

h_d = design thickness of reinforced concrete, mm (in)

y_s = yield strength of reinforcing steel, normally 413.7 MPa (60,000 psi)

S = percent reinforcing steel

This formula 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 to 12.1 m (25 to 40 ft) unless smaller widths are necessary to meet dimensional requirements.

13-5.3 **Limitations to Reinforced Concrete Pavement Design Procedure**
Because the design procedure for reinforced concrete pavements presented in this chapter has been developed from a limited amount of investigational and performance data, these limitations are imposed:

- No reduction in the required thickness of plain concrete will be allowed for percentages of steel reinforcement less than 0.05.
- 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.
- No single dimension of reinforced concrete pavement slabs will exceed 30.5 m (100 ft) regardless of the percent steel used or the slab thickness.
- The minimum thickness of a reinforced concrete pavement or overlay will be 152 mm (6 in).

13-6 REINFORCEMENT TO CONTROL PAVEMENT CRACKING

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 conditions in paragraphs 13-6.1 through 13-6.4, 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.

13-6.1 Odd-Shaped Slabs

In the design of pavement facilities, resorting to odd-shaped slabs is often necessary. 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 that require reinforcement. Where practicable, the number of odd-shaped slabs can be minimized by using a sawtooth fillet and not reinforcing.

13-6.2 Mismatched Joints

Steel reinforcement in the slabs is mandatory to prevent migration of cracks into adjacent pavements for two conditions of mismatched joints:

13-6.2.1 Joint Patterns of Abutting Pavement Facilities

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 a sufficient width of bond-breaking medium is installed in the joint. The determination relative to using reinforcement at mismatched joints in such junctures is based on the type of joint between the two pavement sections. A partial reinforcement of the slab, as described in this paragraph, is required when the joint between the abutting pavement is one of these joints: (a) doweled construction joint, (b) keyed construction joint, (c) thickened-edge butt joint without a bond-breaking medium, (d) doweled expansion joint, or (e) thickened-edge slip joint with less than 6.4-mm (0.25-in) 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 mm (0.25 in) or more of bond-breaking medium, except for a mismatch of joints in the center 23-m (75-ft) 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 mm (3 ft) 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 this manner. When two abutting facilities are being constructed concurrently, the slabs on both sides of the juncture opposite mismatched joints will also be reinforced in this manner. For this condition (shown in Figure 13-2b), the slip joint bond-breaking medium can be specified to be a full 6.4 mm (0.25 in) thick, and the reinforcing may be omitted.

13-6.2.2 Plain Concrete Overlay on an Existing Rigid Pavement

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 mismatching the joints in the overlay and the existing pavement is necessary, 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 mm (3 ft) on each side of the mismatched joint in the existing pavement. The steel will not be carried through any joint in the overlay, however, 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 the overlay, reinforcement of the overlay over mismatched joints is not required, except for mismatched expansion joints.

13-6.3 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.

13-6.4 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 for slabs containing the blockouts will be reinforced in both directions. Slabs that have utility penetrations that are installed as an integral part of the slab (such as observation risers, in-pavement light cans, etc) shall have a reinforcing cage installed around the penetration. For slabs that contain more than one integral utility penetration, the entire slab shall also be reinforced with the 0.05 percent steel reinforcement in both directions.

Figure 13-2a. Typical Layouts Showing Reinforcement of Odd-Shaped Slabs

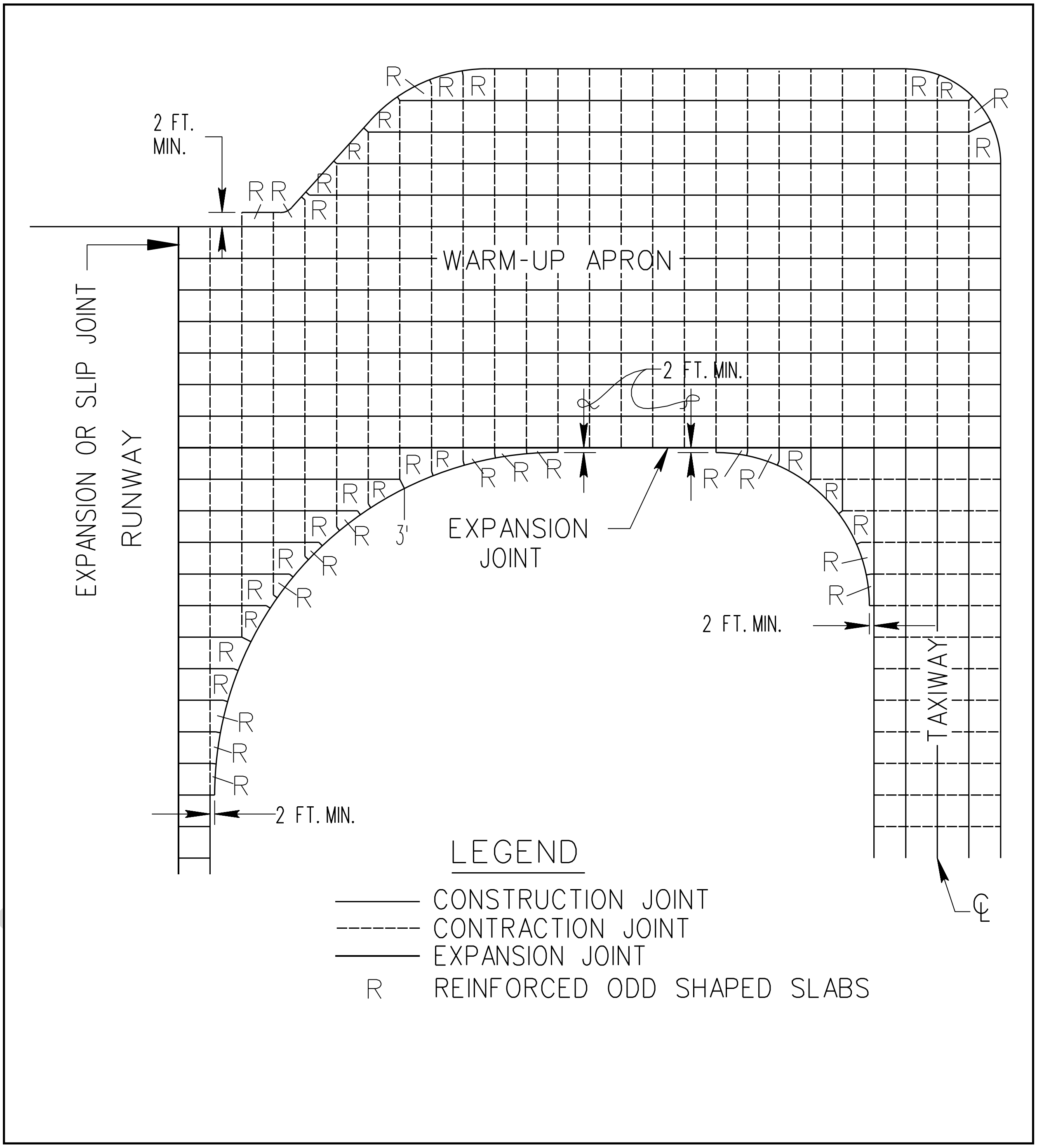
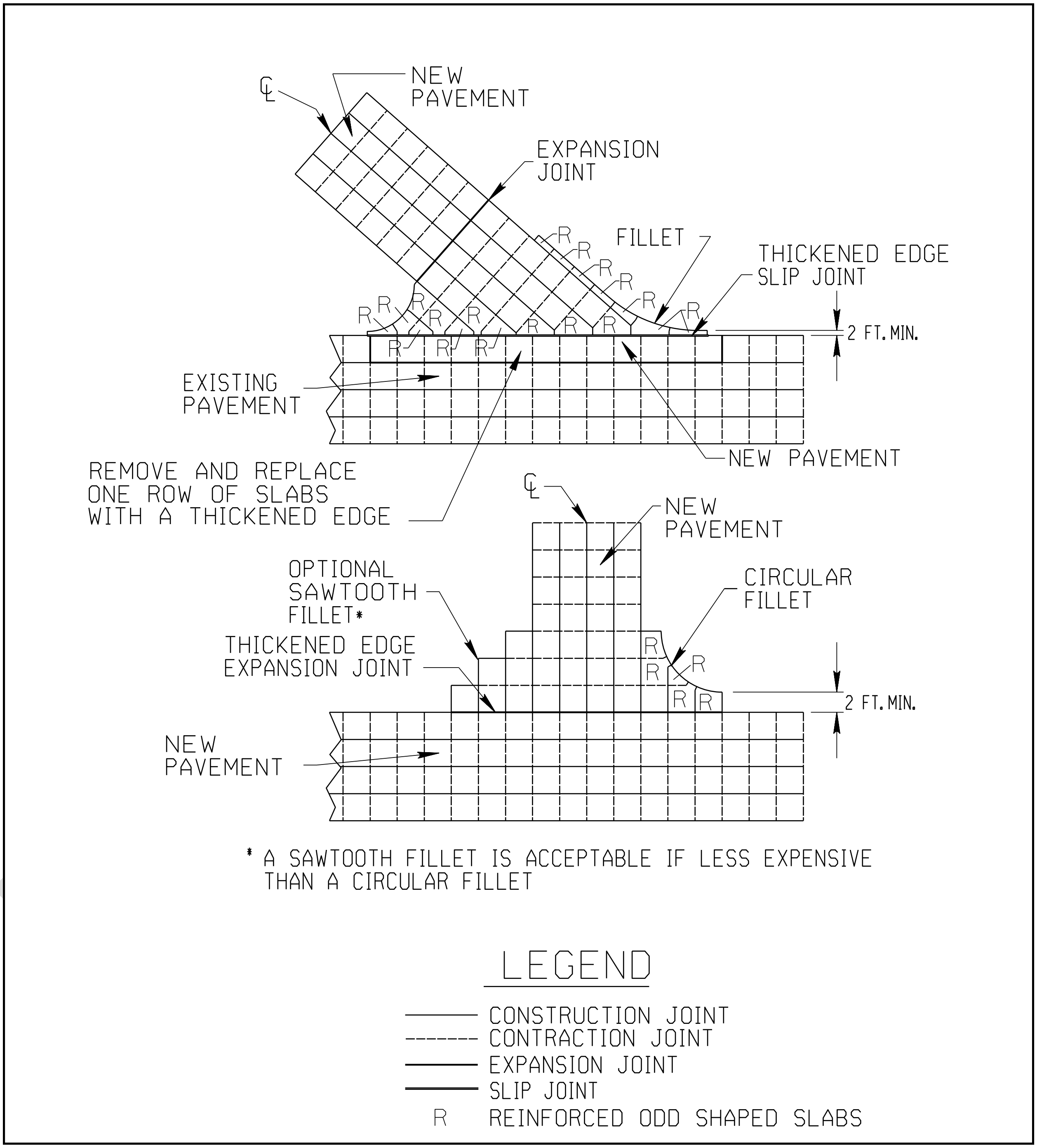


Figure 13-2b. Typical Layouts Showing Reinforcement of Odd-Shaped Slabs and Mismatched Joints



13-7 REINFORCED CONCRETE PAVEMENTS IN FROST AREAS

Normally, plain concrete pavements in frost areas will be designed in accordance with Chapter 12, and reinforcement will be unnecessary. In special instances, however, the pavement thickness will be required to be less than that 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 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 necessary to reduce the pavement thickness more than is allowable for the required minimum percentage of steel.

13-8 REINFORCING STEEL

13-8.1 Types of Reinforcing Steel

The reinforcing steel may be either deformed bars or welded wire fabric. Deformed bars should conform to the requirements of ASTM A615 or ASTM A996. 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 A184. Cold-drawn wire for fabric reinforcement should conform to the requirements of ASTM A82, and welded steel wire fabric to ASTM A185.

13-8.2 Placement of Reinforcing Steel

The reinforcing steel will be placed at a depth to the top of the reinforcement of h_d /4+25 mm (h_d /4+1 in) from the pavement surface to the center of the reinforcing grid. 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 be done only when providing the required steel in one layer is impracticable. 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 the aggregate. If the strike-off method is used to place the reinforcement (a 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 the aggregate. Maximum bar or wire spacing shall not exceed 305 mm (12 in) or 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 such 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 mm (3 in) or one-half of the wire or bar spacing in the fabric or mat (Figure 13-3a). The wires or bars will be lapped in accordance with these requirements:

- 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.

- 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.

13-9 JOINTING

13-9.1 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 in these cases:

- All joints will be doweled or thickened-edge-type joints except for longitudinal contraction joints where the joint spacing is greater than 6 m (20 ft); in those instances, the dowels may be omitted. One end of the dowel will be painted and oiled to permit movement at the joint.

- Thickened-edge-type joints (expansion, butt, or slip) will not be doweled. The edge will be thickened to $1.25h_d$.

- When a transverse construction joint is required at an unplanned location, the reinforcing steel will be carried through the joint. In addition, dowels meeting the size and spacing requirements of Table 12-2 for the design thickness h_d will be used in the joint, and will not be oiled or painted.

- Reinforcement will not be carried through the joints except for unplanned longitudinal contraction joints.

13-9.2 Joint Sealing

Joint sealing for reinforced concrete pavements will be the same as for plain concrete pavements.

Figure 13-3a. Reinforcing Steel Details

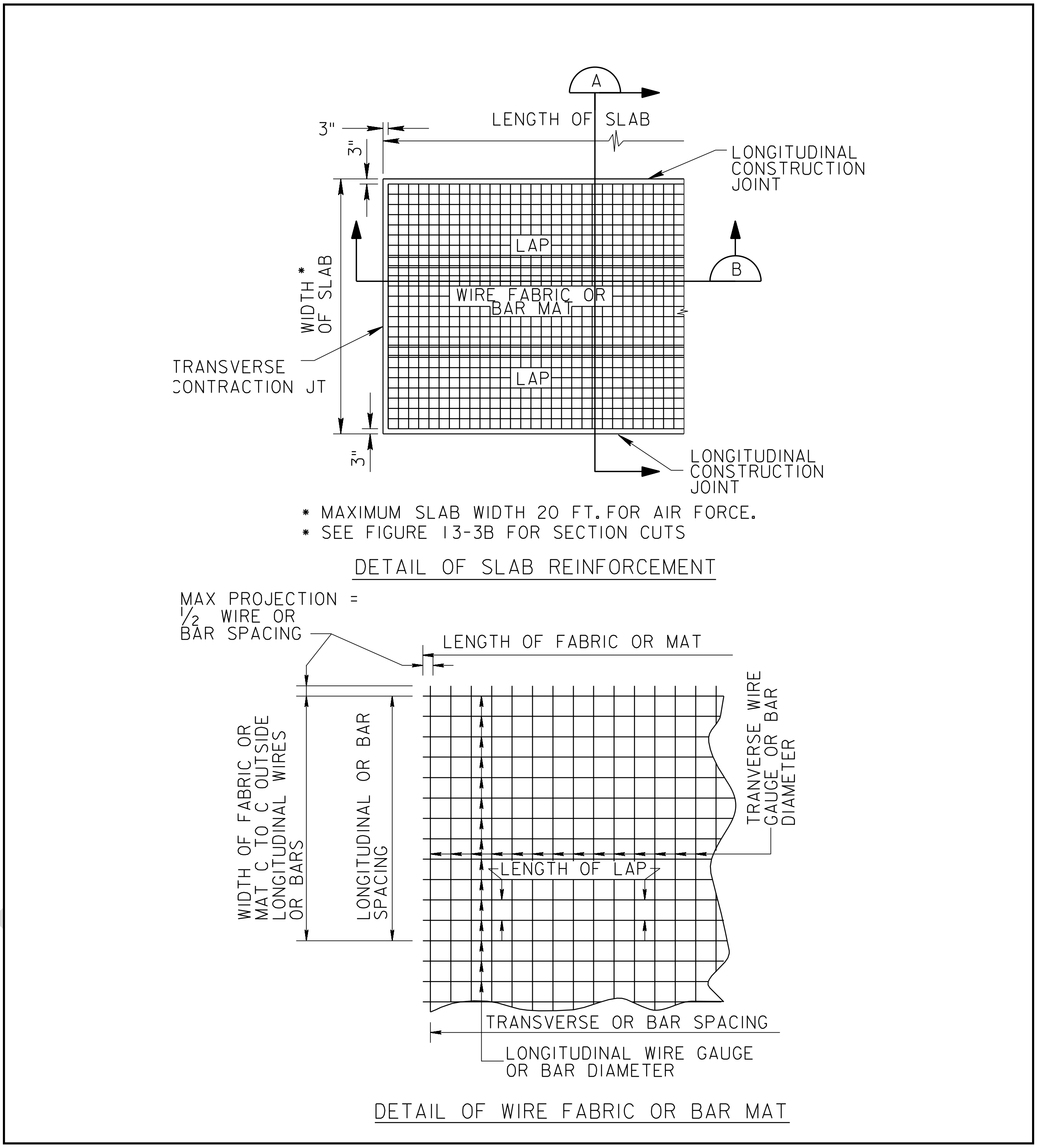
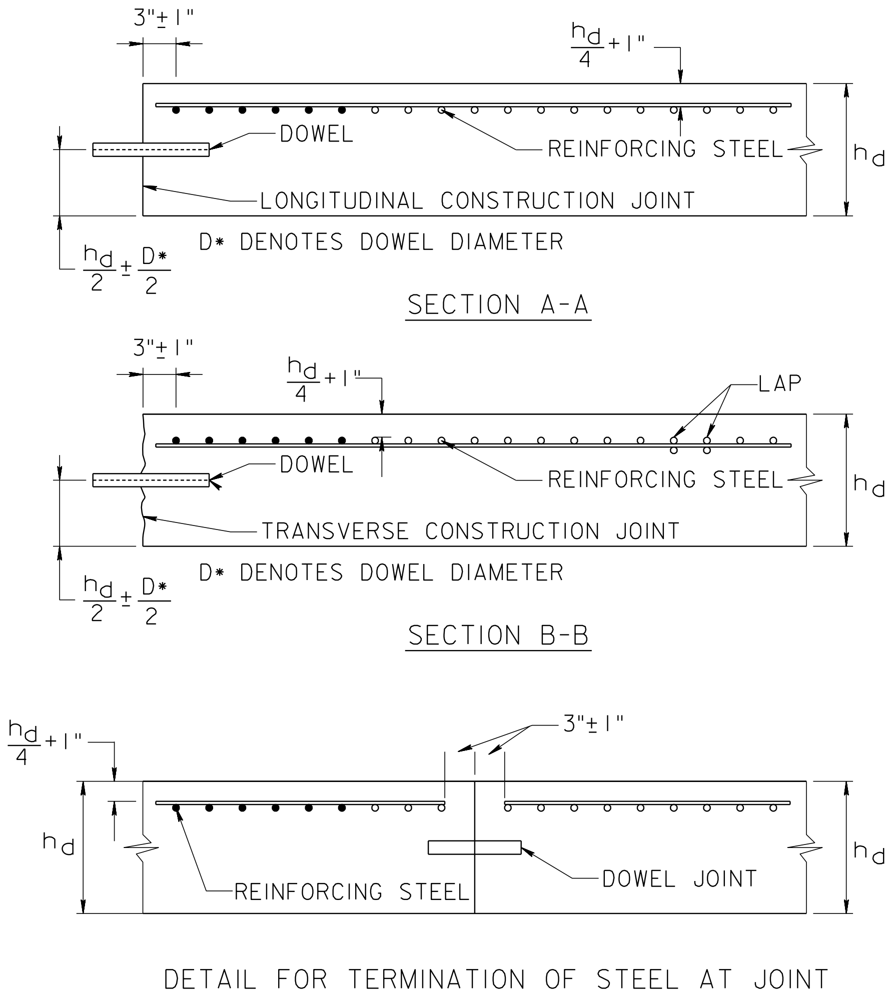


Figure 13-3b. Reinforcing Steel Cross Section Details



- h_d DENOTES REINFORCED CONCRETE PAVEMENT DESIGN THICKNESS
- REINFORCING STEEL NOT CARRIED THROUGH JOINT
- ALL JOINTS IN REINFORCED CONCRETE PAVEMENTS DOWELED EXCEPT THICKENED-EDGE

CONVERSION FACTORS
25.4 MILLIMETERS = 1 INCH

Figure 13-4. Contraction Joints for Reinforced Pavements

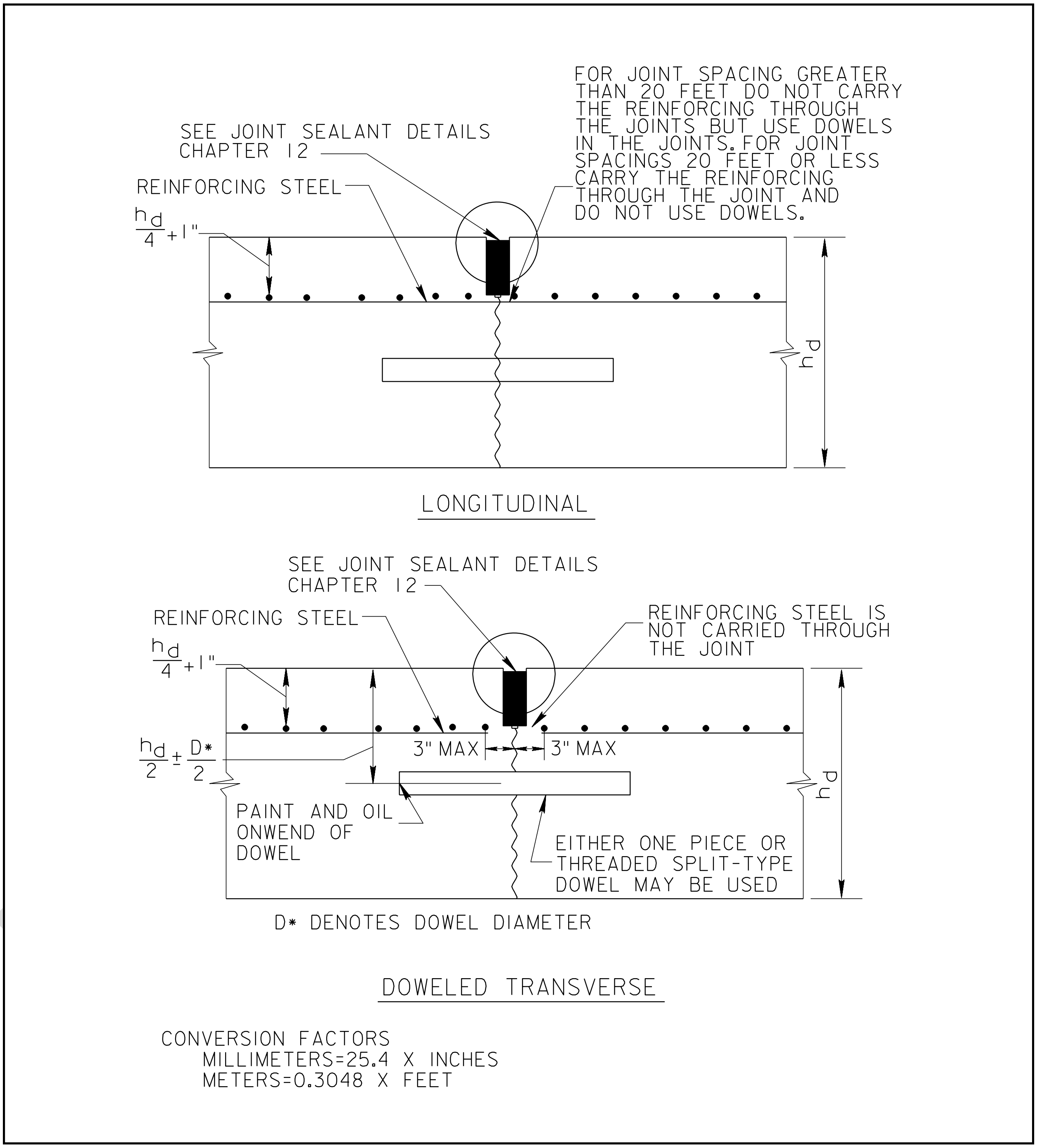
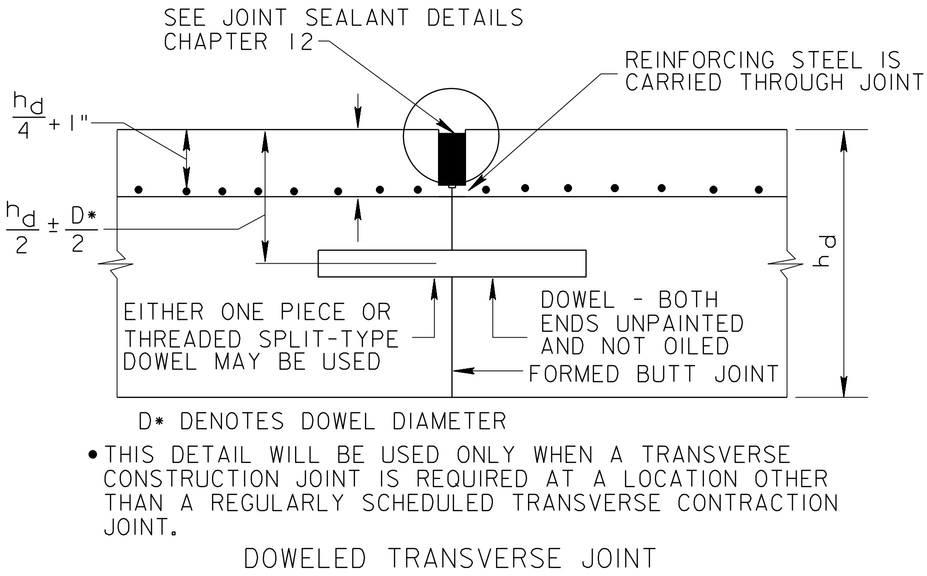
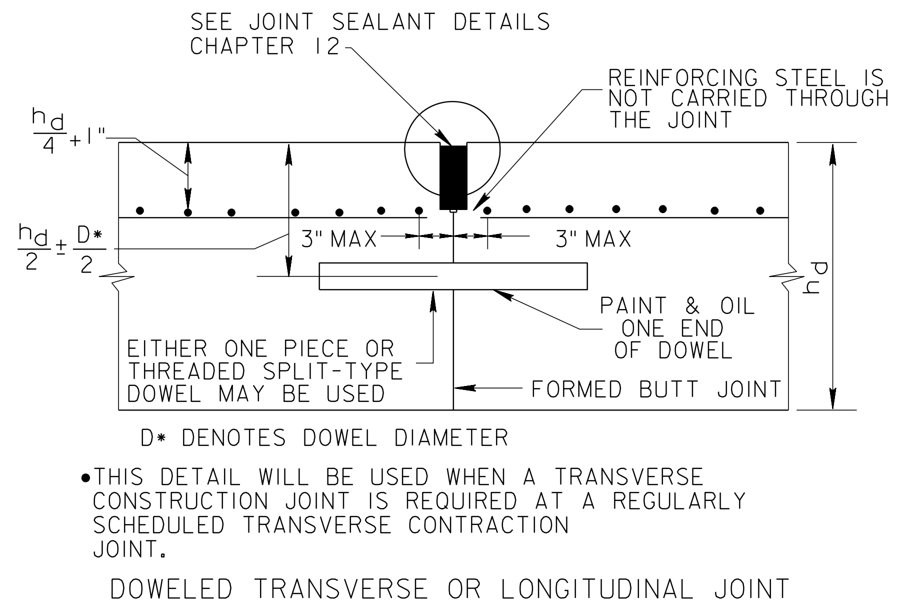


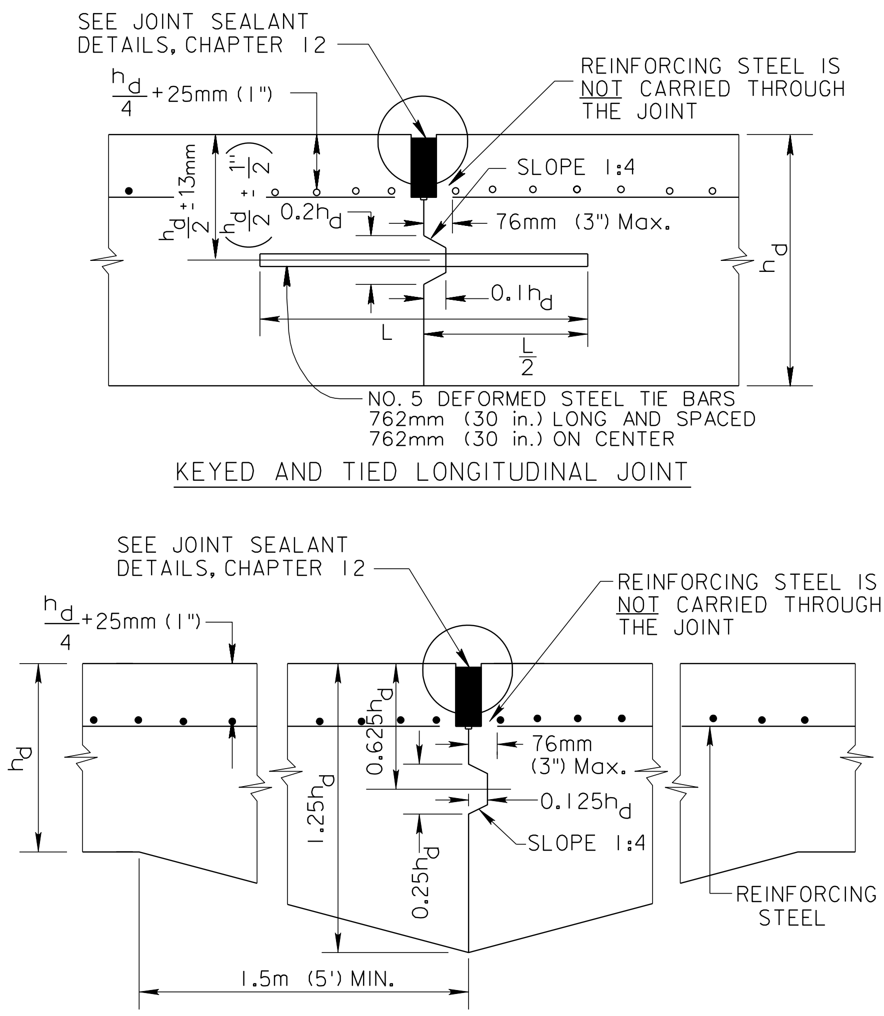
Figure 13-5a. Construction Joints for Reinforced Pavements





CONVERSION FACTOR
MILLIMETERS = 25.4 X INCHES

Figure 13-5b. Construction Joints for Reinforced Pavements



NOTE: A TOLERANCE OF 1.5mm (1/16 IN.) MAY BE ALLOWED FOR KEY DIMENSIONS AND LOCATION

KEYED THICKENED EDGE LONGITUDINAL JOINT

- REQUIRED FOR KEYED JOINTS IN PAVEMENT LESS THAN 9" THICK
- NAVY ONLY

CONVERSION FACTORS
MILLIMETERS = 25.4 INCHES

Figure 13-5c. Construction Joints for Reinforced Pavements

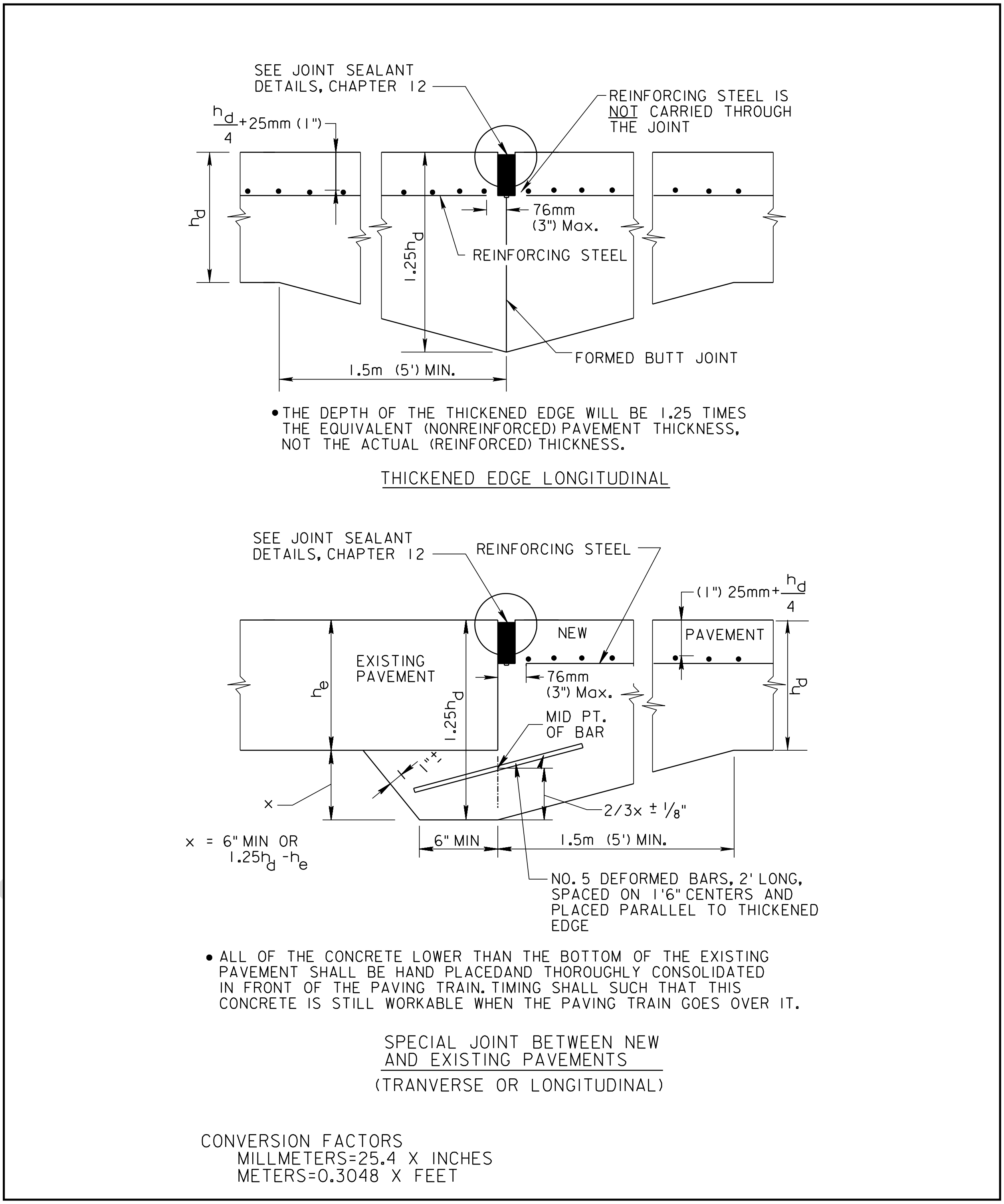


Figure 13-5d. Construction Joints for Reinforced Pavements

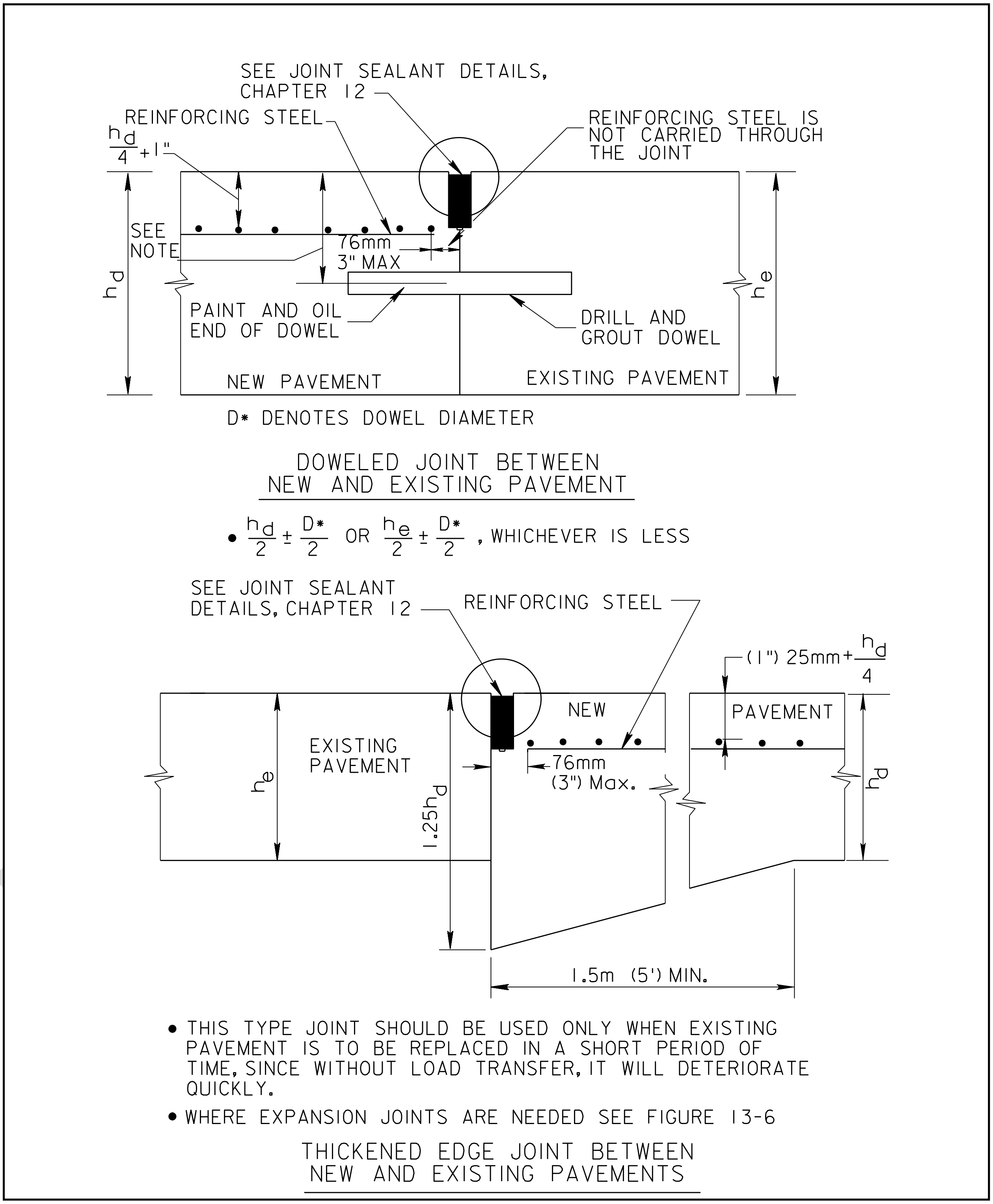
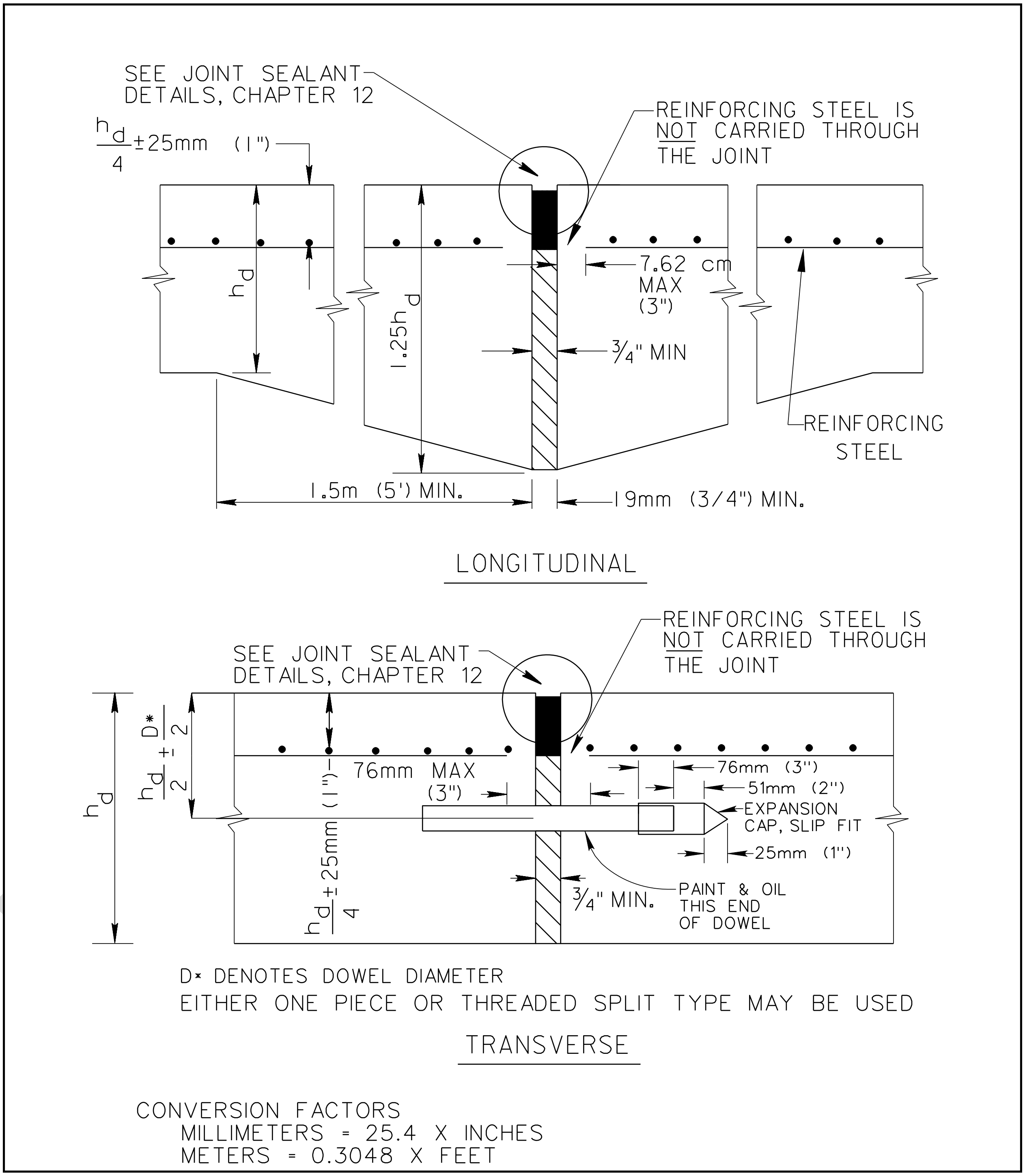


Figure 13-6. Expansion Joints for Reinforced Concrete Pavements



CHAPTER 14

(Reserved for Future Use)

CHAPTER 15

CONTINUOUSLY REINFORCED CONCRETE PAVEMENT DESIGN

15-1 BASIS 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 the direction of paving) directions, between terminal points. The terminal points may be either the longitudinal construction joints, the ends of the pavement, junctures with other pavements, or structures. No joints are required between the terminal points; instead, the pavement is permitted to crack. The crack spacing will vary and be dependent on 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 m (5 to 8 ft) 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 m (2 ft) to as much as 3.5 m (12 ft). The reinforcing steel provides continuity across the non-load-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 handle only non-load-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 at which 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. PCASE does not cover this design procedure. Design examples are shown in section 15-9 of this chapter.

15-2 USE FOR CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS

Continuously reinforced concrete pavements are applicable for any airfield pavement, but they have received very limited use for airfield pavement construction; consequently, long-term performance history is minimal. Because of this, use of continuously reinforced concrete pavements will require the approval of USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or Naval Facilities Engineering Service Center. The use of continuously reinforced concrete pavement shall not be based solely on economics.

15-3 FOUNDATION 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 or base course may be modified or stabilized. Stabilized materials must achieve the strength and durability requirements specified in UFC 3-250-11.

15-4 THICKNESS 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.

15-5 REINFORCING STEEL DESIGN

15-5.1 Longitudinal Direction

The percent of reinforcing steel required in the longitudinal direction for continuously reinforced concrete pavements will be the maximum calculated by these three equations with the minimum percent steel being 0.60 percent:

$$P_s = (1.3 - 0.2F)\frac{f_t}{f_s} \times 100 \tag{15-1}$$

$$P_s = \frac{100\, f_t}{2\,(f_s - \Delta T \varepsilon_c E_s)} \tag{15-2}$$

$$P_s = \frac{f_t}{f_s} \times 100 \tag{15-3}$$

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 15-1 may be used to convert 7-day flexural strength into tensile strength.)

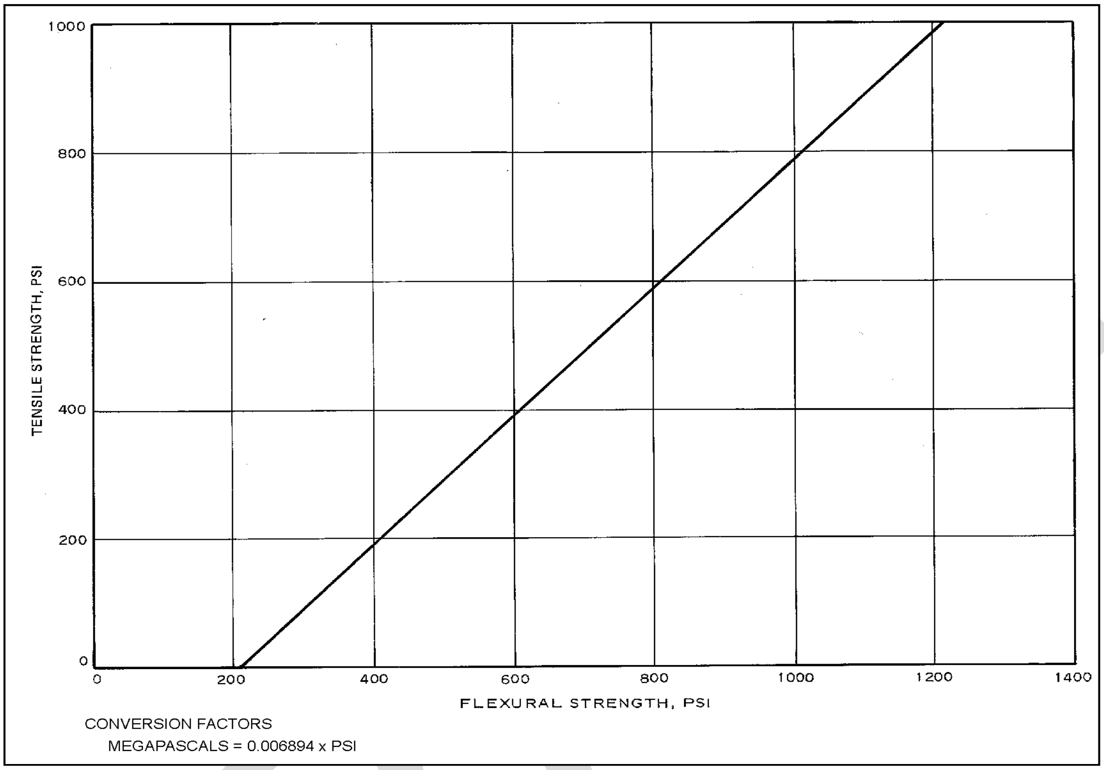
f_s = working stress in the steel, MPa (psi) (75 percent of yield tensile strength of steel). This produces a safety factor of 1.33.

T = seasonal temperature differential in degrees C (degrees F)

ε_c = thermal coefficient of expansion of concrete in mm per mm per degree C (in per in per degree F)

E_s = modulus of elasticity of the reinforcing steel in tension, MPa (psi)

Figure 15-1. Relationship between Flexural Strength and Tensile Strength of Concrete



15-5.2 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 using Equation 15-4:

$$P_s = \frac{W_s F}{2f_s} \times 100 \qquad (15\text{-}4)$$

where

W_s = width of slab, m (ft)

15-5.3 Type of Reinforcing Steel

The reinforcing steel may be either deformed bars conforming to ASTM A615 or welded deformed steel wire fabric conforming to ASTM A497. 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.

15-5.4 Placement of Reinforcing Steel

When the slab thickness is 203 mm (8 in) or less, the longitudinal reinforcement should be placed at the middepth of the slab. For thickness in excess of 203 mm (8 in), the longitudinal steel should be placed slightly above the middepth, but a minimum cover of 76 mm (3 in) of concrete shall be maintained in all cases. Transverse reinforcement is normally placed below and used to support the longitudinal steel; however, transverse reinforcement may be placed on top of the longitudinal steel if the minimum of 76 mm (3 in) 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 the requirements in this chapter. 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 mm (24 in) if the one-third figure is used, and 1,220 mm (48 in) if the not more than one-half requirement is used. The stagger of laps with deformed bars may be on a continuous basis rather than the one-third or one-half detail described in this paragraph.

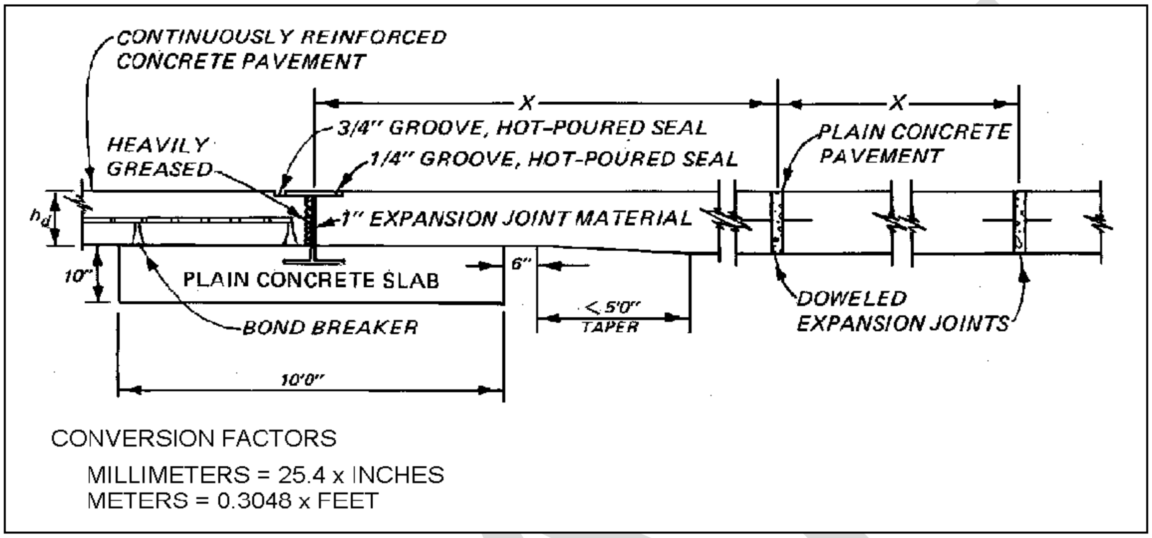
15-6 TERMINAL DESIGN

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 completely restraining or completely isolating the pavement ends is practically impossible, and a combination of these schemes (that is, partial restrain and limited available expansion space) has proven practical. End anchorage 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 15-2. 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.

15-7 JOINTING

Continuously reinforced concrete pavements will normally use the same type of joints as are 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 12-21. 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 m (5 ft) long (0.75 m [2.5 ft] 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.

Figure 15-2. Details of a Wide-Flange Beam Joint



15-8 JOINT 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.

15-9 EXAMPLE OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENT DESIGN

In this example, a pavement must be designed as an Air Force medium-load airfield. Types A and B traffic areas are designed for the F-15 at 23,130 kg (81,000 lb), the C-17 at 263,000 kg (580,000 lb), and the B-52 at 181,440 kg (400,000 lb). Types C and D traffic areas and overruns are designed for the F-15 at 27,555 kg (60,750 lb), the C-17 at 197,280 kg (435,000 lb), and the B-52 at 136,080 kg (300,000 lb). 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 specific 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 in Chapter 12. The required thicknesses are listed in Table 15-1.

Table 15-1. Required Thicknesses for Design Example

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)

15-9.1 **Percent of Longitudinal Steel**
These are the additional data required for determining the percent of longitudinal steel:

f_t – Tensile strength of concrete (from Figure 15-1) = 3.45 MPa (500 psi)

f_s – Yield strength of steel = 414 MPa (60,000 psi)

ε_c – Coefficient of thermal expansion of concrete = 7.2×10^{-6} mm per mm per degree C (4×10^{-6} in per in per degree F)

E_s – Modulus of elasticity of steel = 206×820 MPa (30×10^{6} psi)

ΔT – Seasonal temperature differential of pavement = 72 degrees C (130 degrees F)

F – Friction factor for fine-grained soils = 1.0

The required percentage of longitudinal reinforcement steel is the largest value obtained from Equations 15-1, 15-2, or 15-3.

$$P_s = (1.3 - 0.2F)\frac{f_t}{f_s}\times 100$$

$$= (1.3 - 0.2F)\frac{3.45}{310}\times 100 = 1.22 \text{ in SI units} \tag{15-5}$$

$$= \left[1.3 - 0.2(1.0)\right]\frac{500}{45{,}000}\times 100 = 1.22 \text{ in English units}$$

$$P_s = \frac{100f_t}{2(f_s - \Delta T \varepsilon_s E_s)}$$

$$= \frac{100(3.45)}{2(310.2 - 72.2 \times .0000072 \times 206820)} = 0.85 \text{ in SI units} \quad (15\text{-}6)$$

$$= \frac{100(500)}{2[45{,}000 - 130(4 \times 10^{-6})(30 \times 10^{6}]}$$

$$= 0.850 \text{ in English units}$$

$$P_s = \frac{f_t}{F_s} \times 100$$

$$= \frac{3.45}{310.2} \times 100 = 1.11 \text{ in SI units} \quad (15\text{-}7)$$

$$= \frac{500}{45{,}000} \times 100 = 1.11 \text{ in English units}$$

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:

$$A_s = \frac{P_s \times A_p}{100}$$

$$= \frac{1.22 \times 405 \times 1{,}000}{100} = 4{,}941 \text{ mm}^2 \text{ per meter of pavement (SI units)} \quad (15\text{-}8)$$

$$= \frac{1.22 \times 16.0 \times 12}{100}$$

$$= 2.342 \text{ square inches per foot of pavement (English units)}$$

where

A_p= the cross-sectional area of 1 m (1 ft) of pavement, mm^2 (in^2)

15-9.2 Percent of Steel in the Transverse Direction

In determining the percent of steel required in the transverse direction, it is assumed that 6-m (20-ft) paving lanes will be used with Equation 15-9:

$$P_s = \frac{W_s F}{2f_s} \times 100 = \frac{20 \times 1.0}{2(45{,}000)} \times 100 = 0.022 \quad (15\text{-}9)$$

The design percent steel in the transverse direction is therefore 0.022. The cross-sectional area of steel required per 305 mm (12 in) of pavement for the 405-mm (16-in) pavement is therefore:

$$A_s = \frac{P_s \times A_p}{100} = \frac{0.022 \times 405 \times 300}{100} \times 26.7\ \text{mm}^2\ (0.0414\ \text{in}^2) \qquad (15\text{-}10)$$

The percent steel for other traffic areas would be computed in the same manner.

CHAPTER 16

PRESTRESSED CONCRETE PAVEMENT DESIGN

16-1 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 an equal thickness of plain concrete or reinforced concrete pavement and still provide a functionally adequate pavement. PCASE does not cover this design procedure. Design examples are shown in section 16-9 of this chapter.

16-2 UNITS OF MEASURE

The design equations and criteria in this chapter are controlled by English units; therefore, the equations have not been converted to SI units.

16-3 USES FOR PRESTRESSED CONCRETE PAVEMENT

Although prestressed concrete pavements have been used in Europe, the performance history of these pavements in the United States is not extensive. Therefore, the use of prestressed concrete pavement will require the approval of USACE-TSC, the Air Force MAJCOM pavements engineer, or Naval Facilities Engineering Service Center. Several test or demonstration sections in the United States have shown good performance, but problems have occurred with joints between long prestressed sections where large movements are experienced. For this reason, complex joints and extreme care are required during construction.

16-4 FOUNDATION REQUIREMENTS

16-4.1 Subgrade and Base

In general, the subgrade for a prestressed concrete pavement will be treated and evaluated in the same manner as a subgrade 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 or stabilized or modified subgrade to provide a minimum modulus of soil reaction or composite modulus of soil reaction of 54 kPa/mm (200 pci). 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 as possible of irregularities such as undulations and holes.

16-4.2 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-mm (0.25- to 0.50-in) 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.

16-5 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 are either plastic-encased or are placed in conduits 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 usually applied at a very early stage, and the final level of prestress applied after several days of curing. Both longitudinal and lateral prestressing are needed to obtain the desired structural capacity in the pavement.

16-6 DESIGN PROCEDURE

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 the 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 on an economic analysis. A maximum value of design prestress of 2.76 MPa (400 psi) is recommended, and based on experience, a design prestress level between 0.69 MPa and 2.76 MPa (100 and 400 psi) has been most economical. The minimum thickness of prestressed concrete pavement will be 152 mm (6 in).

16-6.1 Design Equation

The design prestress for a given thickness of pavement will be determined by Equation 16-1:

$$d_s = \frac{6PNB}{wh_p^2} - R + r_s + t_s \qquad (16\text{-}1)$$

where

d_s = design prestress required in concrete, psi

P = aircraft gear load, lb

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, in

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 0.69 MPa and 2.76 MPa (100 and 400 psi) are generally economical, and at these levels h_p will be approximately one-third of the required thickness of plain concrete pavement. The design gear load P will depend on 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 is in turn used in Figure 16-1 to obtain N. The load-moment factor B and ratio of multiple-wheel gear load to single-wheel gear load w are determined from Figure 16-2 and Figure 16-3, respectively, by entering with a value of A/ℓ^2. (Note that for 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 P is computed by Equation 16-2:

$$\ell = \left[\frac{E h_p^3}{12(1-\mu^2)k} \right]^{1/4} \tag{16-2}$$

where

ℓ = radius of relative stiffness, in

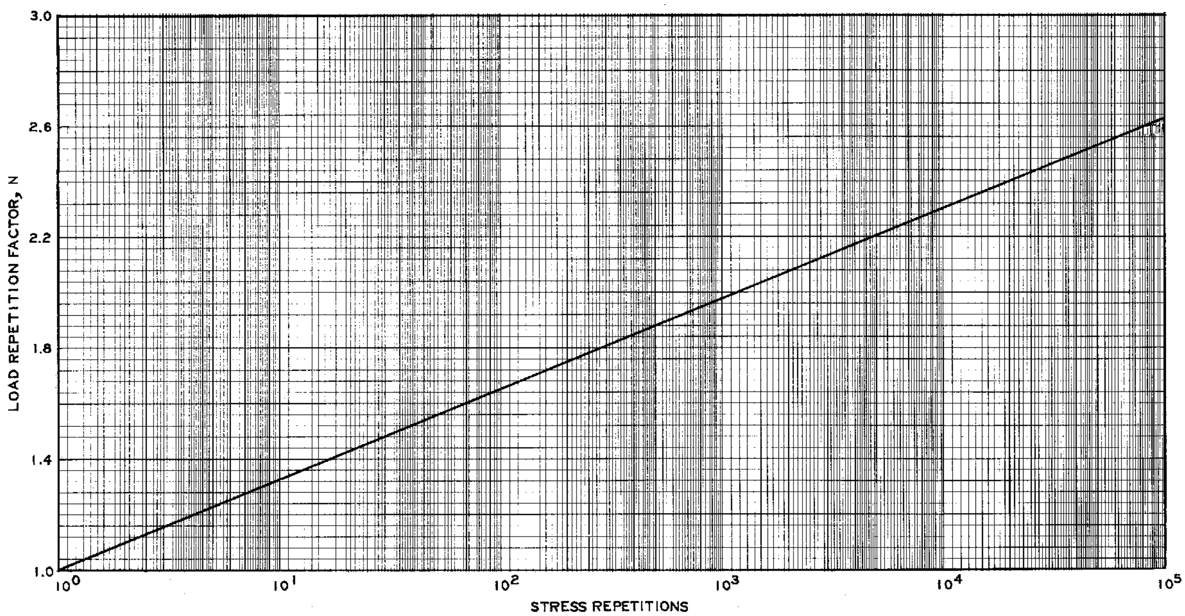
E = the modulus of elasticity of concrete (a value of 4,000,000 psi is normally used)

h_p = design thickness of prestressed concrete pavement, in

μ = Poisson's ratio

k = modulus of subgrade reaction, pci

Figure 16-1. Stress Repetitions versus Load Repetition Factor



16-6.2 **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 Equation 16-3:

$$r_s = \frac{C_f L \rho}{2(144)} \quad or \quad r_s \frac{C_f W \rho}{2(144)} \tag{16-3}$$

where

r_s = foundation restraint stress, psi

C_f = coefficient of sliding friction

L = length of prestressed concrete slab, ft

W = width of prestressed concrete slab, ft

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.

Figure 16-2. A/ℓ^2 versus Load-Moment Factor

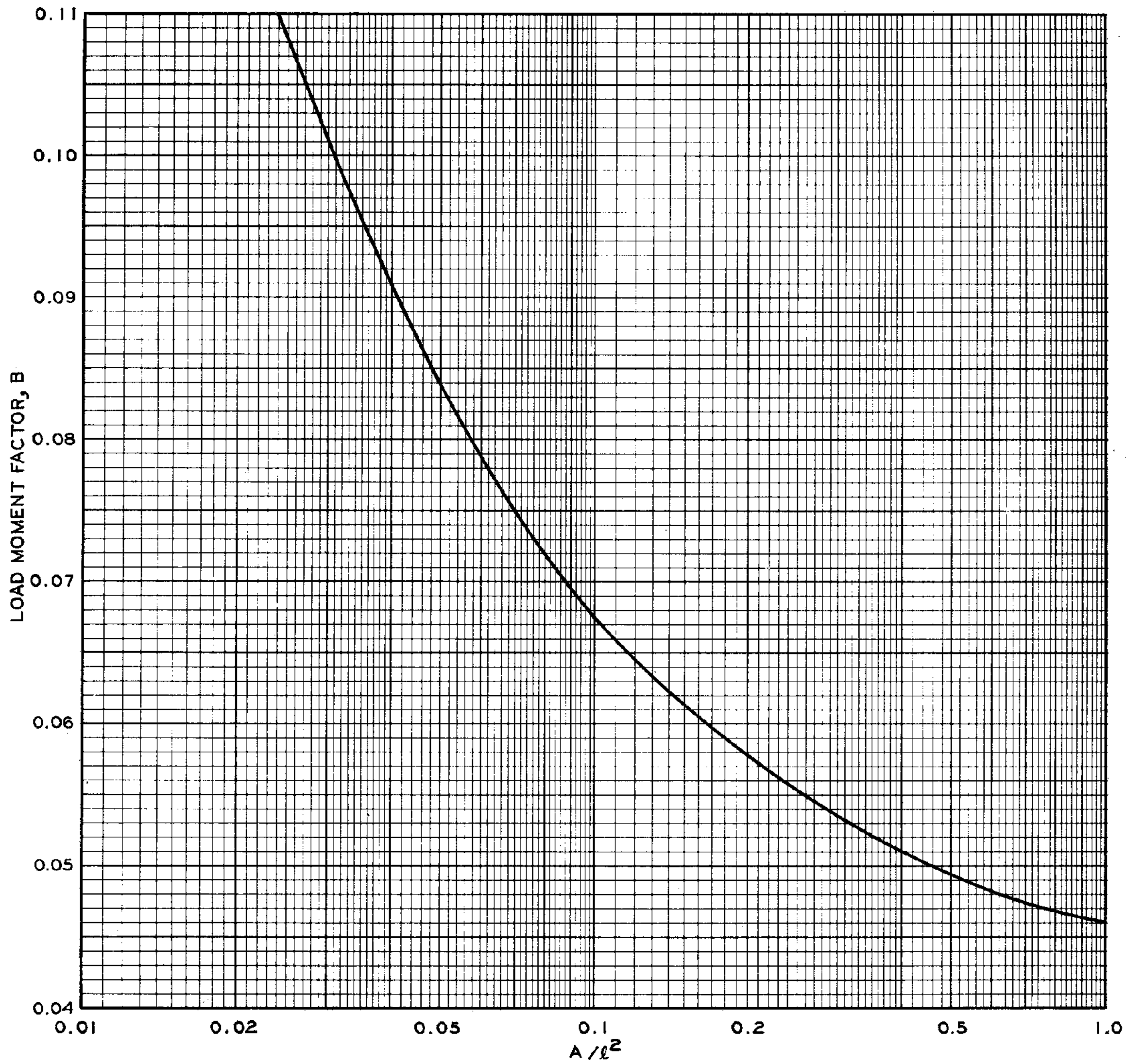
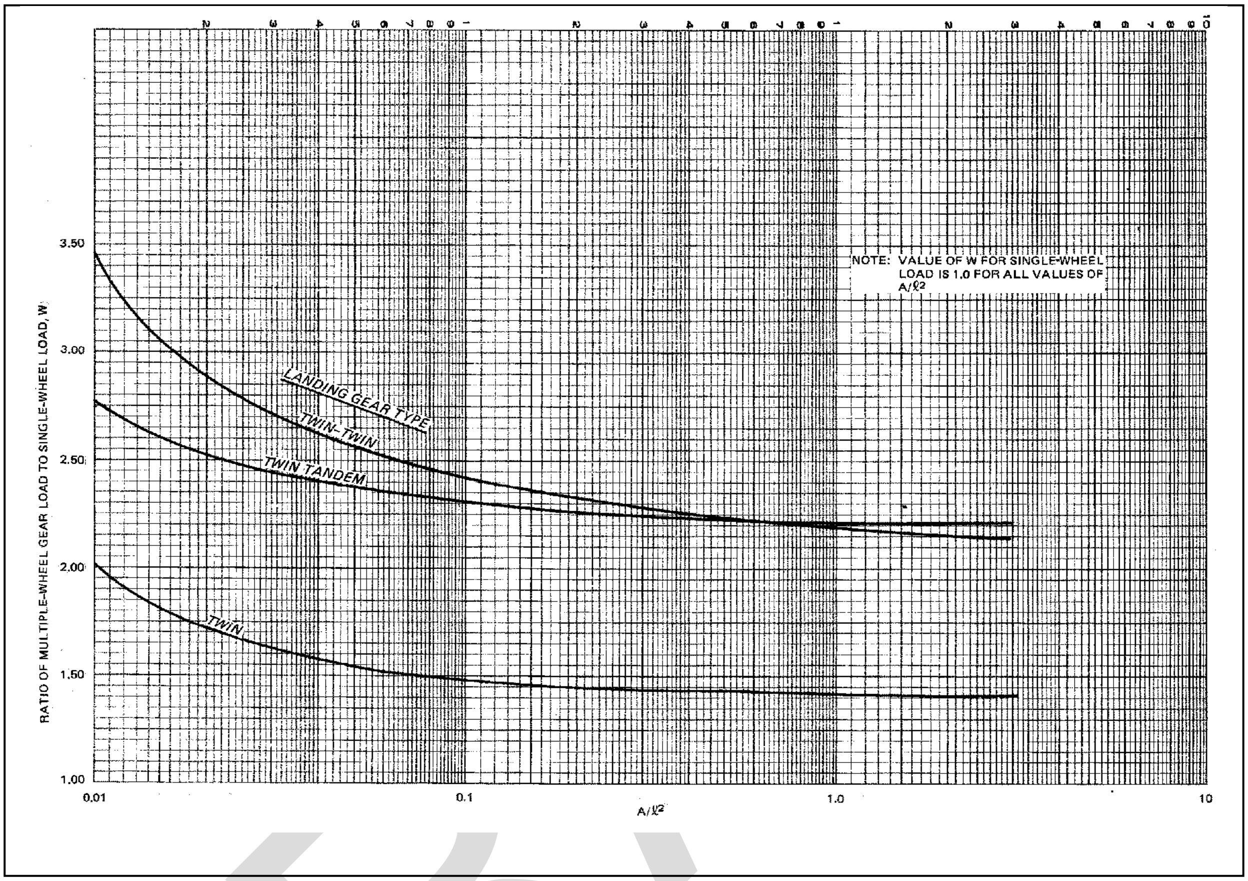


Figure 16-3. Ratio of Multiple-Wheel Gear to Single-Wheel Gear Load versus A/ℓ²



16-6.3 **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 Equation 16-4:

$$t_s = \frac{ET \in_c}{2(1 - v)} \qquad (16\text{-}4)$$

where

t_s = temperature warping stress, psi

T = difference in temperature in degrees Fahrenheit between the top and bottom of the prestressed concrete pavement

$\in_c$ = coefficient of thermal expansion, in/in

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.

16-7 PRESTRESSING TENDON DESIGN

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.

16-7.1 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 Equation 16-5:

$$A_s = \frac{1.2 d_s A_c}{0.7 f_\mu} \tag{16-5}$$

where

A_c = cross-sectional area of concrete being prestressed, in^2

f_μ = ultimate strength of the tendon steel, psi

Equation 16-5 is applicable to the determination of A_s based on a recommended maximum anchorage stress equal to 7/10 of the ultimate strength of the tendon steel. If the steel is anchored at a stress other than 7/10 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.

16-7.2 Prestressing Steel Tendons

The tendons used for prestressed concrete pavement will consist of high-strength wires, strands, or bars. Follow these requirements:

(a) Wires will conform to the requirements of ASTM A421.

(b) Seven-wire strands will conform to the requirements of ASTM A416.

(c) High-strength bars will conform to the requirements of section 405(f) of American Concrete Institute (ACI) 318.

16-7.3 **Prestressing Conduits**
Conduits used for enclosing the steel tendons should be either rigid or flexible metal tubing; however, the tendons may be plastic encased. Follow these requirements:

(a) Metal conduits must be strong enough to resist damage in transit or during handling. The metal may be bright or galvanized.

(b) When tendons are plastic-encased, the tendons should be permanently protected from rust or corrosion.

16-7.4 **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 mm (0.25 in) larger than the diameter of the stressing tendons. The minimum cover of the conduits will be 76 mm (3 in) at the pavement surface and 50 mm (2 in) at the bottom of the pavement.

16-7.5 **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 paragraph 16-7.1, 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. Apply the prestressing as soon as possible to prevent or minimize the occurrence of contraction cracking in the concrete.

16-7.6 **Grouting**
When the stressing tendons are placed in conduits, grout the space between the tendons and conduits after the final prestressing load is reached. Make the grout 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. Provide grouting vents at each end of the conduits and along the conduits at intervals not to exceed 45 m (150 ft). Use a grouting pump to inject the grout. Start the grouting at an end vent and continue until grout is

forced out of the first interior vent along the conduit. Seal the end vent then, and inject grout through the first interior vent until it is extruded from the second interior vent. Continue this procedure until the entire length of conduit has been grouted.

16-8 JOINTING

16-8.1 Joint Spacing

Experience has shown that from a practical standpoint, the maximum length of prestressed concrete slabs should be 150 m (500 ft), although lengths of 180 and 215 m (600 and 700 ft) have been constructed. The width of the slab will vary depending on the capability of the construction equipment but will generally be a minimum of 7.6 m (25 ft).

16-8.2 Joint Types

16-8.2.1 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 m (500 ft) (such as aprons), the pavement must be constructed in widths not to exceed 150 m (500 ft); therefore, longitudinal fill-in lanes will be required to permit access for applying the transverse prestressing.

16-8.2.2 Transverse Joint

Because of the length of prestressed slabs and the low subgrade restraint, large movements will occur at the transverse joints. The transverse joints must be designed to accommodate these movements that are a function of temperature change, slab length, and moisture conditions. The anticipated movements can be determined by these equations:

$$\Delta_{LT} = 12L \in_c \Delta T \tag{16-6}$$

and

$$\Delta_{LM} = 12L \in_M \tag{16-7}$$

where

Δ_{LT} = change in length of slab due to temperature change ΔT, in

L = slab length, ft

ΔT = change in temperature in degrees (either daily or seasonally)

Δ_{LM} = maximum change in length of slab due to seasonable moisture change

$\in_M$ = 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 joint. 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 on 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.

16-8.3 **Joint Seals**

Except where longitudinal transition lanes will be required to permit prestressing operations of wide paved areas, longitudinal joints in prestressed concrete pavements need not be sealed because these joints will be held tightly closed by the prestressing. If these joints are sealed, however, 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 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 USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or Naval Facilities Engineering Service Center.

Figure 16-4. Typical Section of Transverse Joints

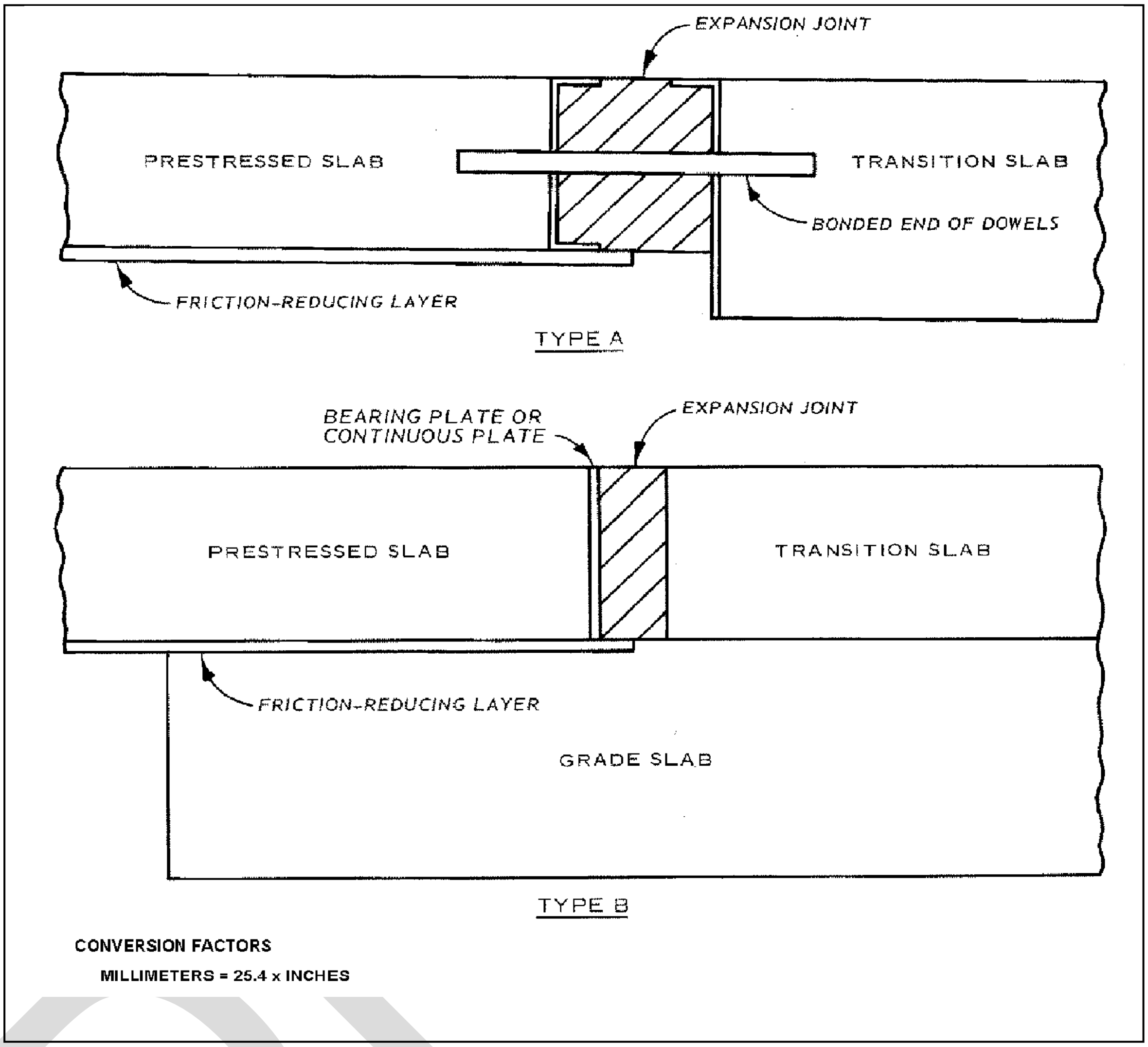


Figure 16-5. Typical Transverse Joint Seals (Sheet 1 of 3)

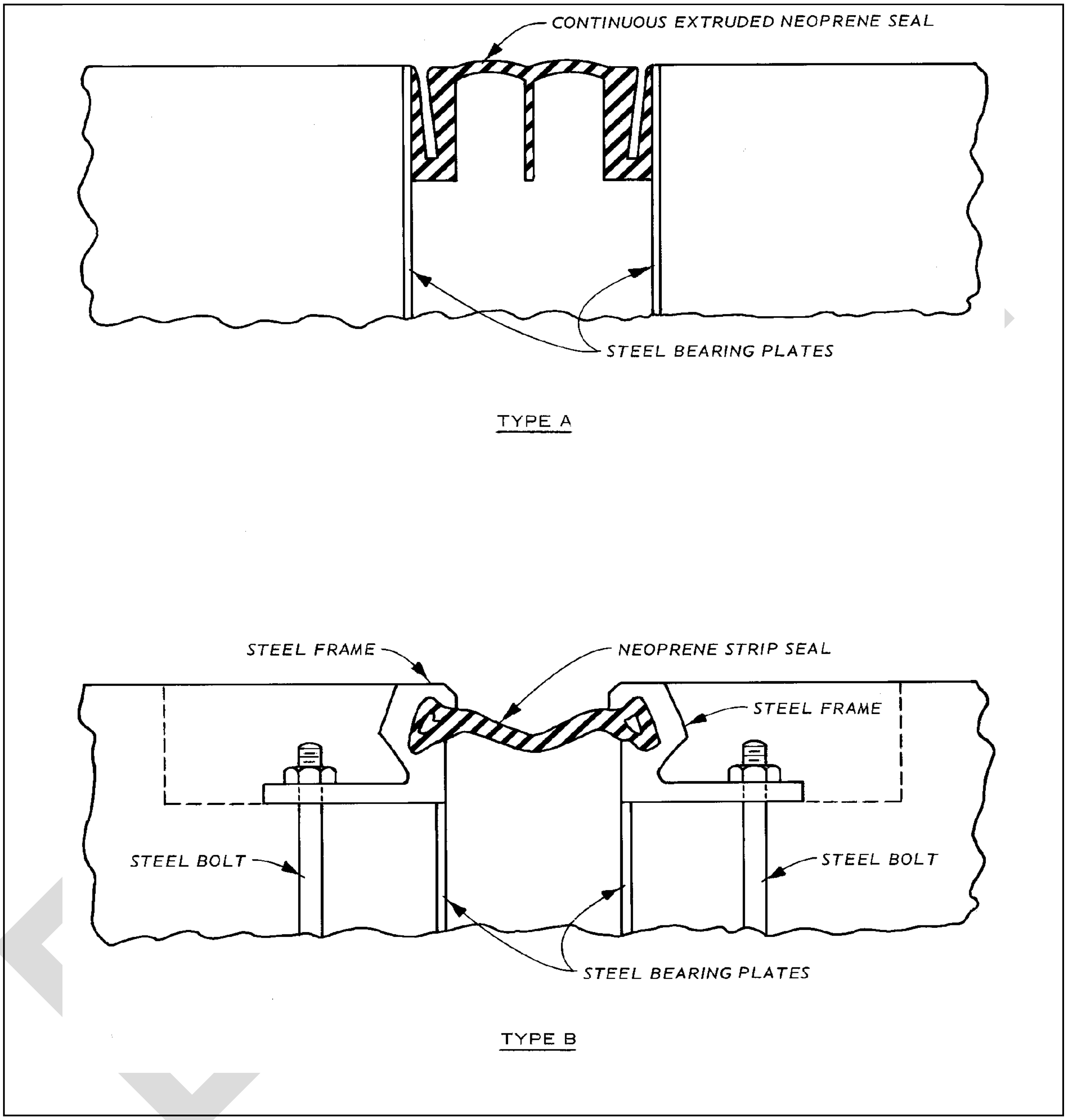
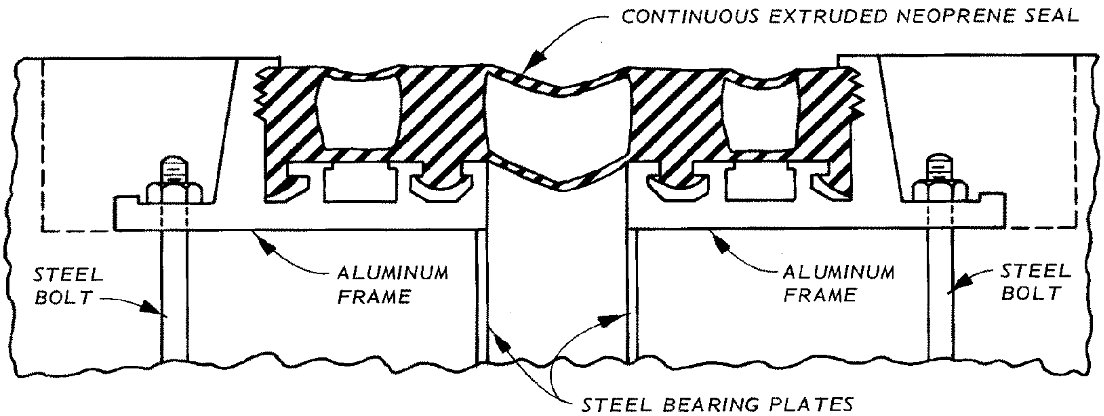
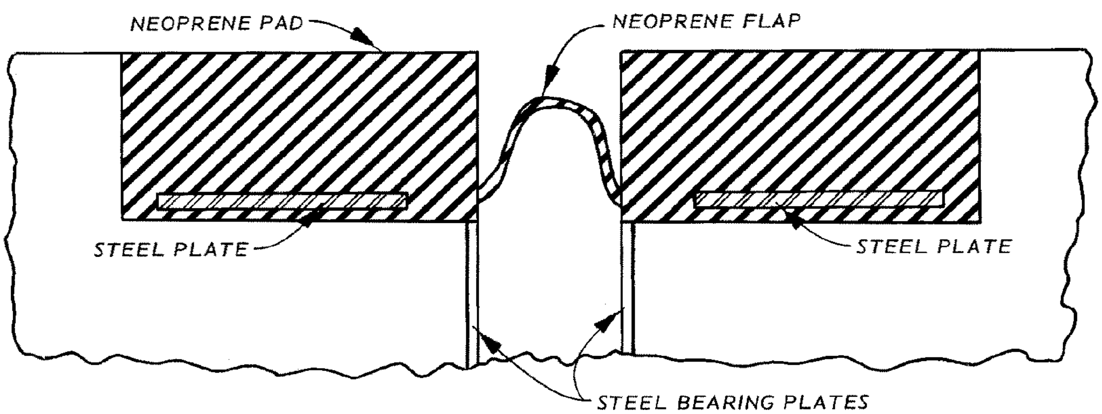


Figure 16-5. Typical Transverse Joint Seals (Sheet 2 of 3)

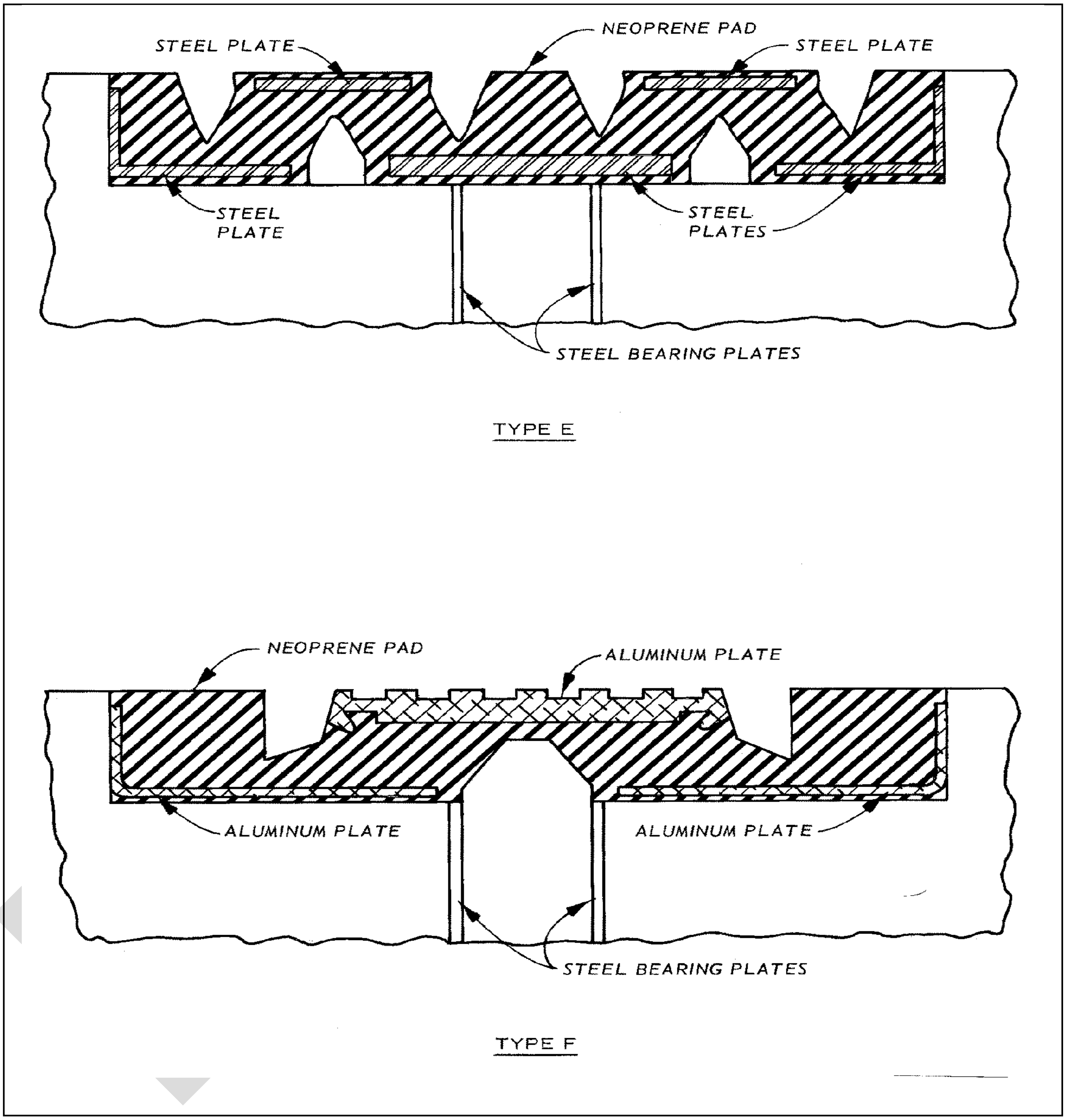


TYPE C



TYPE D

Figure 16-5. Typical Transverse Joint Seals (Sheet 3 of 3)



16-9 EXAMPLES OF PRESTRESSED CONCRETE PAVEMENT DESIGN

16-9.1 Overview

A 75-ft-wide by 10,000-ft-long taxiway pavement is to be designed for 100,000 passes of the C-141 aircraft at 320,000 lb 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 $E = 4\times10^6$ psi; Poisson's ratio of concrete μ = 0.15; and coefficient of thermal expansion of concrete $\varepsilon_c = 4\times10^{-6}$ in/in per degree Fahrenheit.

16-9.2 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 16-6, 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 in, respectively. The ratio of multiple-wheel gear load to single-wheel gear load w is 2.22, 2.23, and 2.335 for thicknesses of 6, 7, and 8 in, 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 ft 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:

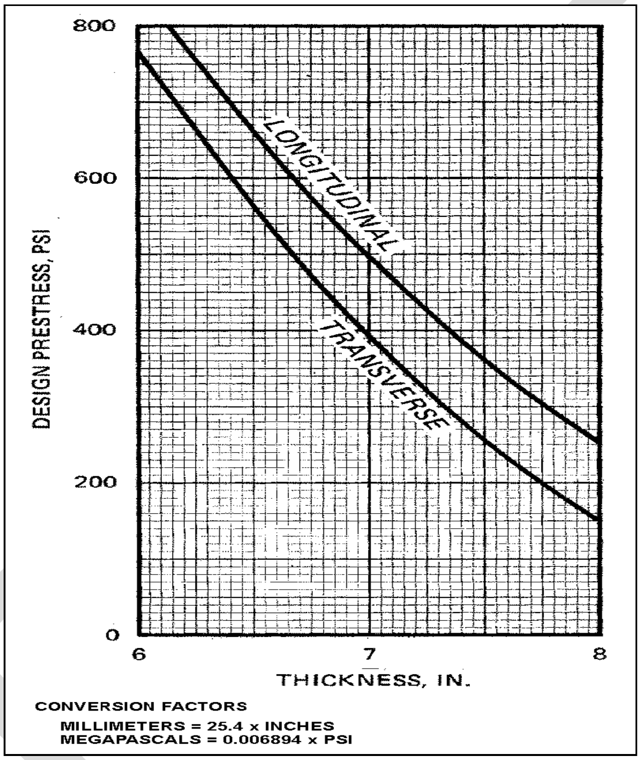
$$r_s = \frac{C_f W \gamma}{2(144)} = \frac{0.60 \times 75 \times 150}{2(144)} = 23.4 \text{ psi} \qquad (16\text{-}9)$$

16-9.2.1 The maximum difference in temperature between the top and bottom of the prestressed concrete pavement is estimated to be approximately 6, 7, and 8 degrees for the 6-, 7-, and 8-in 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 Equation 16-10:

$$d_s = \frac{6PNB}{wh_p^2} - R + r_s + t_s \qquad (16\text{-}10)$$

16-9.2.2 For h_p values of 6, 7, and 8 in, 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-in pavement with longitudinal prestress of 360 psi and transverse prestress of 250 psi would provide a satisfactory pavement. With a slab length of 400 ft, 25 slabs and thus 24 joints will be required for the 10,000-ft-long taxiway. In actual design, several combinations of variables such as k, h_p, and slab length should be considered, and the final selection should be based on an economic study considering all aspects of material and construction costs.

Figure 16-6. Thickness versus Design Prestress



16-9.3 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 allow for the various tendon stress losses will be:

Longitudinal Direction

$$A_s = \frac{1.2 \times 360 \times 7.5 \times 7.5 \times 12}{0.7 \times 240{,}000} = 1.74 \textit{ square inches} \tag{16-11}$$

Transverse Direction

$$A_s = \frac{1.2 \times 250 \times 7.5 \times 4.00 \times 12}{0.7 \times 240{,}000} = 0.643 \textit{ square inches} \tag{16-12}$$

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 in) is selected, then 30 tendons will be required, each having a cross-sectional area of 0.58 in^2 and a diameter of 0.86 in. Therefore, a 7/8-in-diameter tendon could be selected. Selecting a tendon that is greater or less than required may require revising the final anchor stress. If, in the transverse direction, a spacing of five times the prestressed concrete pavement thickness (37.5 in) is selected, then 128 tendons would be needed and the required cross-sectional area of the tendons would be 0.50 in^2. Therefore, a 13/16-in-diameter tendon would provide the required prestressing.

CHAPTER 17

OVERLAY PAVEMENT DESIGN

17-1 OVERVIEW

Overlay pavements are designed to increase or restore the load-carrying capacity (strength) of the existing pavement or correct surface deficiencies. The basis for design is to provide a layer or layers of material on the existing pavement that will result in a layered system that will yield the predicted performance of a new rigid pavement if constructed on the same foundation as the existing pavement. This chapter considers two general types of overlay pavement, rigid and nonrigid, and presents procedures for the design of plain concrete, reinforced concrete, continuously reinforced concrete, prestressed concrete, and nonrigid overlays. Nonrigid overlays include both flexible (nonstabilized base and bituminous concrete wearing course) and ABC for strengthening existing plain concrete, reinforced concrete, and flexible pavements. Continuously reinforced concrete overlays and prestressed concrete overlays will not be permitted unless approved by USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or Naval Facilities Engineering Service Center. The use of overlays shall not be based solely on economics. When the overlay includes a nonstabilized aggregate base course layer, the unbound base course must be positively drained in accordance with Chapter 23.

17-2 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 1950s modified with experience and performance observations in the 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.

17-2.1 Basic Conditions

The overlay equations for rigid and flexible overlays of rigid pavements recognize four basic conditions:

17-2.1.1 Fully Bonded Overlay

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.

17-2.1.2 Partially Bonded Overlay

For a partially bonded overlay, 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.

17-2.1.3 Unbonded Overlay

For an unbonded overlay, a thin separation layer of AC 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.

17-2.1.4 Flexible Overlay

For a flexible overlay, an AC is placed directly on a rigid base pavement to restore surface and structural quality. For very thick overlays, a combination of granular base and AC 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.

17-2.2 Failure Conditions

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 on which they were based. The rigid overlay sections were considered failed when initial structural cracks appeared because such cracking was considered the precursor of spalling and potential FOD problems. The flexible overlays were considered failed 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 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 that the design case is outside the valid conditions, and the minimum thickness flexible overlay of 102 mm (4 in) should be used.

17-3 SITE INVESTIGATIONS

Explorations and tests of the existing pavement will be made to determine the structural condition of the pavement prior to overlay, to assess the required physical properties of the existing pavement and foundation materials, and to 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 on the type of overlay used as described in subsequent paragraphs. An investigation will be conducted to determine whether voids exist 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 variations. 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.

17-4 PREPARATION OF EXISTING PAVEMENT

The preparation of the existing pavement prior to overlay will vary depending on whether the overlay is rigid or nonrigid.

17-4.1 Rigid Overlay

Overlay thickness criteria are presented for three conditions of bond between the rigid overlay and the 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.

17-4.1.1 Fully Bonded Overlay

Fully bonded overlays use careful surface preparation to ensure that the overlay and underlying base slab are fully bonded and behave monolithically. To achieve this full bond reliably, the base slab is diamond ground or shot-blasted to remove all deteriorated or defective concrete and all surface contamination. This roughened surface must be cleaned thoroughly by sandblasting followed by air blasting, water blasting, or both. Achieving and maintaining the surface cleanliness is critical for achieving a 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 applied only 3 to 4 m (10 to 13 ft) ahead of the concrete placement. If the grout dries out prior to the concrete placement, the grout should be removed by sandblasting or another 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 placed successfully with no bonding grout, but the military has no experience with this at present. For military airfield work where debonding poses a serious FOD hazard, the intense surface preparation, surface cleaning, and use of a portland cement grout are considered 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 be used on military airfields only to correct surface deficiencies. Fully bonded overlays 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 mm (2 in), and most military airfield bonded overlays have been 76 to 127 mm (3 to 5 in) thick. Typical past uses have included correcting surface smoothness or skid resistance problems; providing a sound operational surface over underlying pavements that are scaling, posing a FOD hazard from popouts or spalling and raveling; and covering pavement surfaces that pose a 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.

17-4.1.2 **Nonbonded Rigid Overlay**

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 approximately 25 mm (1 in) 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 approximately 25 mm (1 in) will be repaired by cold planing, localized patching, or the application of a leveling course using bituminous concrete, sand-asphalt, or a similar material.

17-4.2 **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 ABC 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 mm (1 in) wide in an existing rigid pavement will be filled with joint sealant. Joints or cracks that are 25 mm (1 in) or greater in width will be cleaned and filled with an acceptable bituminous mixture (such as sand-asphalt) that is compatible with the overlay. When required, leveling courses of bituminous concrete will be used to bring the existing pavement to the proper grade. Prior to placing the ABC overlay, a tack coat will be applied to the surface of the existing pavement.

17-5 CONDITION OF EXISTING CONCRETE PAVEMENT

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 on a condition survey (ASTM D5340) of the existing rigid pavement. Interpolation of *C* values between those shown in paragraph 17-5.1 may be used if it is 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 D5340; however, rather than calculating the PCI, calculate a SCI by subtracting from 100 the deduct values for corner breaks; longitudinal, transverse and diagonal cracking; shattered slabs; spalling along joints; and spalling corners.

17-5.1 Rigid Overlay

Values of *C* are assigned based on the condition of plain and reinforced concrete pavements:

(a) 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.

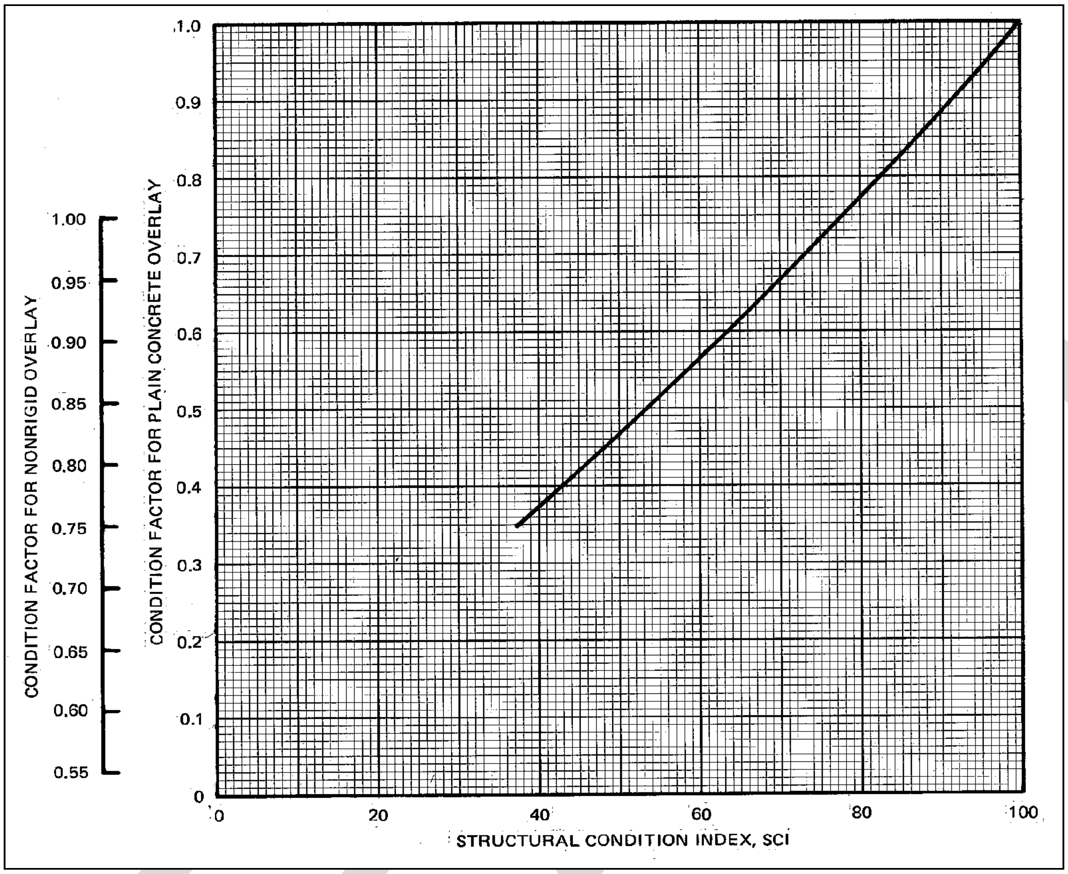
(b) Condition of existing reinforced concrete pavement:

C = 1.00 Pavements in the trafficked areas are in good condition with little or no short-spaced transverse (305- to 610-mm [1- to 2-ft]) 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.

Figure 17-1. Structural Condition Index versus Condition Factor



17-5.2 **Nonrigid Overlay**

Values of *C* are assigned based on the condition of plain and reinforced concrete pavements:

(a) 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.

(b) 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 the trafficked areas exhibit numerous closely spaced, load-induced transverse and longitudinal cracks, rather severe spalling or raveling, or initial evidence of punchout failures.

17-6 RIGID OVERLAY OF EXISTING RIGID PAVEMENT

17-6.1 Design Equations

The three basic equations for the design of rigid overlays are based on the degree of bond that develops between the overlay and the 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 with no special care taken to provide bond. A bond-breaking medium and the nonbonded equation will be used (a) when 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 used to overlay an existing plain concrete pavement that has a condition factor C of 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.

17-6.2 Plain Concrete Overlay

17-6.2.1 Thickness Determination

The required thickness h_o of plain concrete overlay will be determined from these 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 thicknesses of 50 to 120 mm (2.0 to 5.0 in). 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 mm (2 in) for a fully bonded overlay and 152 mm (6 in) 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.

17-6.2.2 **Jointing**

For all partially bonded and fully bonded plain concrete overlays, provide joints 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 use a bond-breaking medium and design the overlay as a nonbonded overlay or reinforce the overlay over the mismatched joints. Should the mismatch of joints become severe, consider a reinforced concrete overlay design 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 in Table 12-7. 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. Do not use dowels and load-transfer devices in fully bonded overlays. Joint sealing for plain concrete overlays will conform to the requirements for plain concrete pavements on grade.

17-6.2.3 Example

Appendix B, Section 14, provides an example of plain concrete overlay design.

17-6.3 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. Use the nonbonded overlay design only when a leveling course is required over the existing pavement. Design and place the reinforcement steel for reinforced concrete overlays in accordance with the requirements for reinforced concrete slabs on grade.

17-6.3.1 Thickness Determination

Determine the required thickness of reinforced concrete overlay using Figure 13-1 after determining the thickness of plain concrete overlay using the appropriate overlay equation. Then, using the value for the thickness of plain concrete overlay, select either the thickness of reinforced concrete overlay and determine the required percent steel, or select the percent steel and determine the thickness of reinforced concrete overlay from Figure 13-1. The minimum thickness of reinforced concrete overlay will be 152 mm (6 in).

17-6.3.2 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 the spacing will not exceed 30 m (100 ft) 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.

17-6.3.3 Example

Appendix B, Section 14, provides an example of reinforced concrete overlay design.

17-6.4 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 as, 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.

17-7 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.

17-8 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 if it is composed of a rigid base pavement that has been strengthened with 102 mm (4 in) or more of nonrigid (flexible or all-bituminous) overlay. If the nonrigid overlay is less than 102 mm (4 in), 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. These 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.

17-9 NONRIGID OVERLAY OF EXISTING RIGID PAVEMENT

With certain reservations, two types of nonrigid overlay, ABC overlay and flexible overlay, may be used to strengthen an existing rigid pavement.

17-9.1 All-Bituminous Overlay

The all-bituminous overlay will be composed of hot-mix bituminous concrete meeting the requirements of UFC 3-250-03. 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.

17-9.2 Flexible Overlay

The flexible overlay will be composed of a 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 UFC 3-250-03 and the minimum thickness requirements of Chapter 8. If the design thickness of the 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.

17-9.3 Thickness Determination

Regardless of the type of nonrigid overlay, the required thickness t_o will be determined by:

$$t_o = 3.0(Fh_d - Ch_E) \qquad (17\text{-}4)$$

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 the existing rigid pavement. If the existing rigid pavement is plain concrete, 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. The factor *F*, determined from Figure 17-2, 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 the values in Table 17-1. The factor *C* is a coefficient based on the structural condition of the existing rigid pavement. The minimum thickness of overlay used for strengthening purposes will be 50 mm (2 in) for Air Force Type D traffic areas and all overruns, 76 mm (3 in) for Army Class I, II, and III pavements, 76 mm (3 in) for Air Force Types B and C traffic areas on light-load pavements, 76 mm (3 in) for Navy and Marine Corps secondary pavements designed for fighter aircraft, and 102 mm (4 in) 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 either an equivalent thickness of high-quality crushed aggregate base with a CBR equal to 100 or an equivalent thickness of ABC (equivalency factor of 1.15 for base and 2.3 for subbase) and the total pavement thickness determined based on 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.

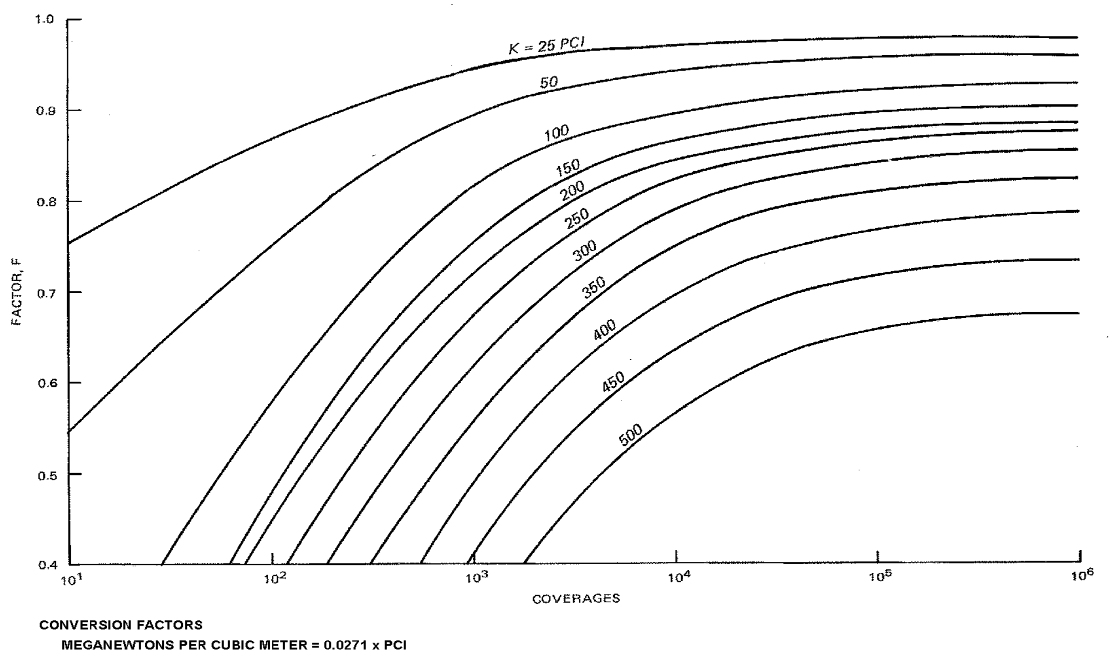
17-9.4 **Reflective Cracking**

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 caused primarily 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 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 limited time while more comprehensive rehabilitation is postponed to the future. Some methods of ameliorating the adverse effects of reflective cracking are outlined in paragraphs 17-9.4.1 through 17-9.4.6.

Table 17-1. Pass-to-Coverage Ratios

Aircraft	Rigid Pavements		Flexible Pavements	
	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 and 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

Figure 17-2. Factor for Projecting Cracking in a Flexible Pavement



17-9.4.1 **Overlay Thickness**

The thicker the overlay, the longer the cracking will be postponed and the more slowly it will deteriorate; hence, abiding by minimum flexible overlay thicknesses is important.

17-9.4.2 **Saw and Seal**

Since there is no reliable way to avoid reflective cracking, another approach is to saw the flexible overlay directly above the rigid pavement joints and seal these cuts with an appropriate sealer. The 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 the Air Force's preferred approach to dealing with flexible overlays over rigid pavements.

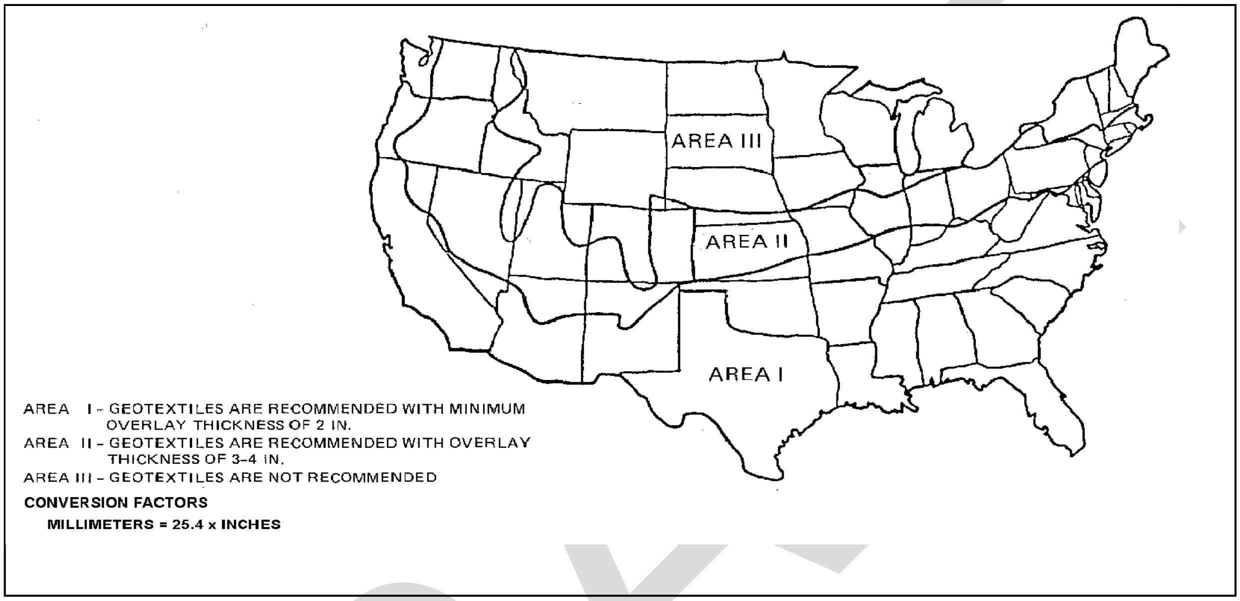
17-9.4.3 **Geotextiles**

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 minimum overlay thickness is 102 mm (4 in).

17-9.4.4 **Crack and 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. Conceptually, the shattered slabs or rubblized concrete fragments are then too small to develop movements that generate reflective cracks. This technique has proven successful on highways, but experience is very limited at present on thicker pavements such as airfield pavements.

Figure 17-3. Location Guide for the Use of Geotextiles in Retarding Reflective Cracking



17-9.4.5 Bond Breakers

Many materials have been tried for providing a layer capable of absorbing the movement of the underlying rigid pavement without transmitting the movement to the AC overlay surface. These materials include open-graded materials, aggregate bases as part of the flexible overlay, and specially designed stress- or strain-absorbing membranes. These materials have given mixed results, and some systems are proprietary.

17-9.4.6 Reinforcing

Besides geotextiles, other proprietary reinforcing systems are available. These systems use steel wire and fiberglass grids to combat reflective cracking, and these systems have not been evaluated by the military.

Military experience has found that 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 reflective cracking. The other techniques discussed here have given mixed results or have not been evaluated by the military.

17-9.5 Example

An example of nonrigid overlay design is provided in Appendix B, Section 14.

17-10 NONRIGID OVERLAY ON FLEXIBLE PAVEMENT

When strengthening of a flexible pavement is required, follow these steps to design the overlay thickness:

(1) 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.

(2) Where the in-place density of the existing material is less than required, increase the overlay thickness or recompact the low-density material. 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. Also, heavy rolling will frequently damage the surface layer if it is brittle. In each case, examine very carefully the decision to excavate and recompact low-density layers or to increase the overlay thickness. Factors to be considered in this examination are the depth of the water table, the subgrade soil properties, and the performance of the existing pavement.

Note that overlaid asphalt courses must meet the quality requirements for their position in the strengthened pavement. An example of a flexible overlay over a flexible pavement is contained in Appendix B, Section 14.

17-11 OVERLAYS IN FROST REGIONS

Whenever the subgrade is subject to frost action, the design will meet the requirements for frost action in Chapter 20. 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.

CHAPTER 18

RIGID PAVEMENT INLAY DESIGN

18-1 OVERVIEW

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 usually 15 m wide (50 ft minimum) for taxiways and 23 m wide (75 ft minimum) 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. Except for the special requirements presented in this chapter, the thickness design of the rigid inlay will be the same as outlined in Chapters 12 through 16 or Chapter 19.

18-2 RIGID INLAYS IN EXISTING FLEXIBLE PAVEMENT

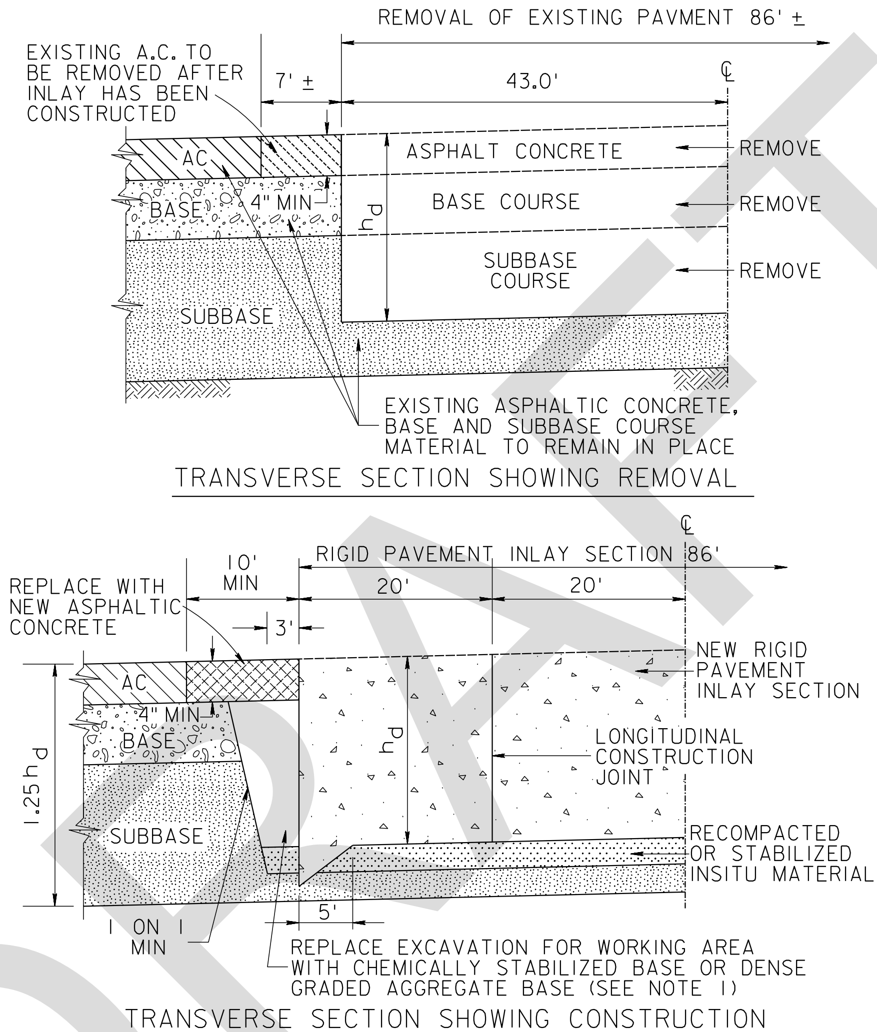
Figure 18-1 shows a section of a typical rigid pavement inlay in an existing flexible pavement. Paragraphs 18-2.1 through 18-2.8 detail the requirements for rigid inlays in existing flexible pavement.

18-2.1 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 the 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 m (3 ft), for forming or slipforming the edges of the concrete pavement (Figure 18-1).

18-2.2 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 102 mm (4 in) below the bottom of the inlay pavement to permit construction of the stabilized layer required in paragraph 18-2.3.

18-2.3 Unless the material in the bottom of the excavation is granular and free-draining or the airfield is located in an arid climate, the bottom full width of the excavation will be scarified to a minimum depth of 152 mm (6 in) and recompacted to the density requirements for the top 152 mm (6 in) of base course or subgrade. This type of overlay may trap water, and satisfactory drainage must be provided. Refer to UFC 3-250-11 for selection of a stabilizing agent and for minimum strength requirements.

Figure 18-1. Typical Rigid Pavement Inlay in Existing Flexible Pavement



NOTES:
1. AT LOCATIONS WHERE AIRCRAFT CROSS PERPENDICULAR TO THIS CROSS SECTION USE DETAILS 12-35 OR 12-36 IN CHAPTER 12.
2. SECTIONS SHOWN ON ARE FOR 86-FT-WIDE INLAYS, CONSTRUCTION OF INLAYS HAVING DIFFERENT WIDTHS WILL BE HANDLED IN A SIMILAR MANNER.

CONVERSION FACTORS
METERS = 0.3048 x FEET
MILLIMETERS = 25.4 x INCHES

18-2.4 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 UFC 3-250-11 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 UFC 3-250-11, 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.

18-2.5 If the existing pavement is not composed of NFS 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.

18-2.6 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 102 mm (4 in) of the pavement surface with either lean-mix concrete or normal paving concrete.

18-2.7 The existing bituminous concrete will be sawed parallel to and at a distance of 3 m (10 ft) 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 102 mm (4 in). The exposed surface of the base course will be recompacted, and a 3-m-wide (10-ft-wide) paving lane of bituminous concrete, 102 mm (4 in) thick, will be used to fill the gap (Figure 18-1). The bituminous concrete mix will be designed in accordance with Chapter 9.

18-2.8 In cases where the 3-m (10-ft) 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-m (10-ft) lane will be accomplished by removal and replacement, planer operation, or both.

18-3 RIGID INLAYS IN EXISTING RIGID PAVEMENT

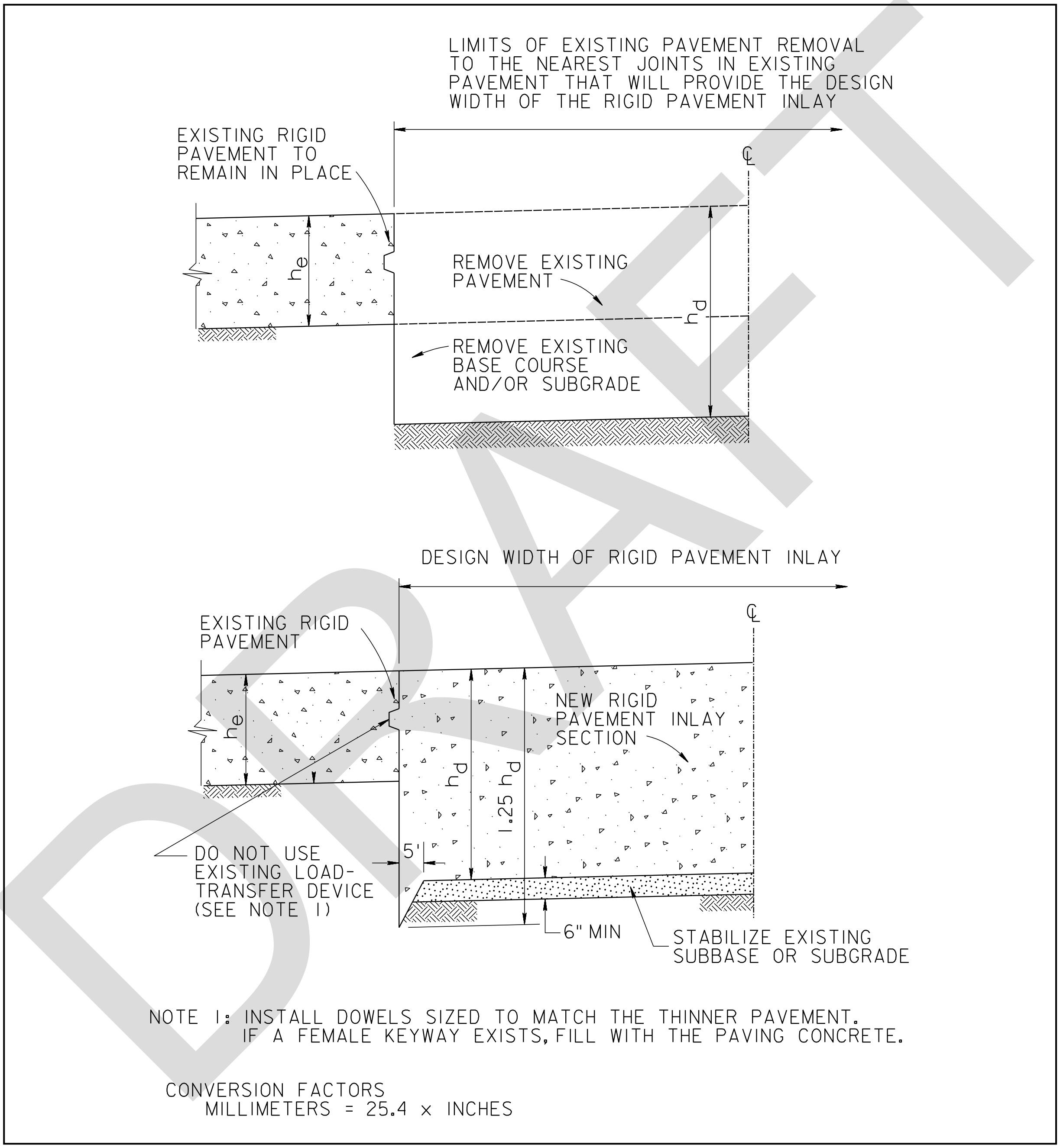
Figure 18-2 shows a section of a typical rigid pavement inlay in an existing rigid pavement. Paragraphs 18-3.1 through 18-3.3 detail the requirements for rigid inlays in existing rigid pavement.

18-3.1 The existing rigid pavement will be removed to the nearest longitudinal joint 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 pavement to remain. Where load transfer is required, it shall be provided by dowels or a thickened edge. If a female keyway exists, it shall be filled with hand-packed PCC prior to pavement placement. In addition to the removal of the existing pavement, the existing base or subgrade will be removed to the depth required for the design thickness of the rigid pavement inlay.

18-3.2 The criteria for subdrains, stabilization, soil strength, and frost also pertain to rigid pavement inlays in existing rigid pavements.

18-3.3 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 18-2. Typical Rigid Pavement Inlay in Existing Rigid Pavement



CHAPTER 19

LAYERED ELASTIC DESIGN OF RIGID PAVEMENTS

19-1 RIGID PAVEMENT DESIGN PRINCIPLES

The basic design principle for this design procedure is to limit the tensile stresses in the 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 PCASE 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 in these steps:

(1) Select three or four concrete slab thicknesses and compute the maximum tensile stresses in the slabs under the design aircraft load.

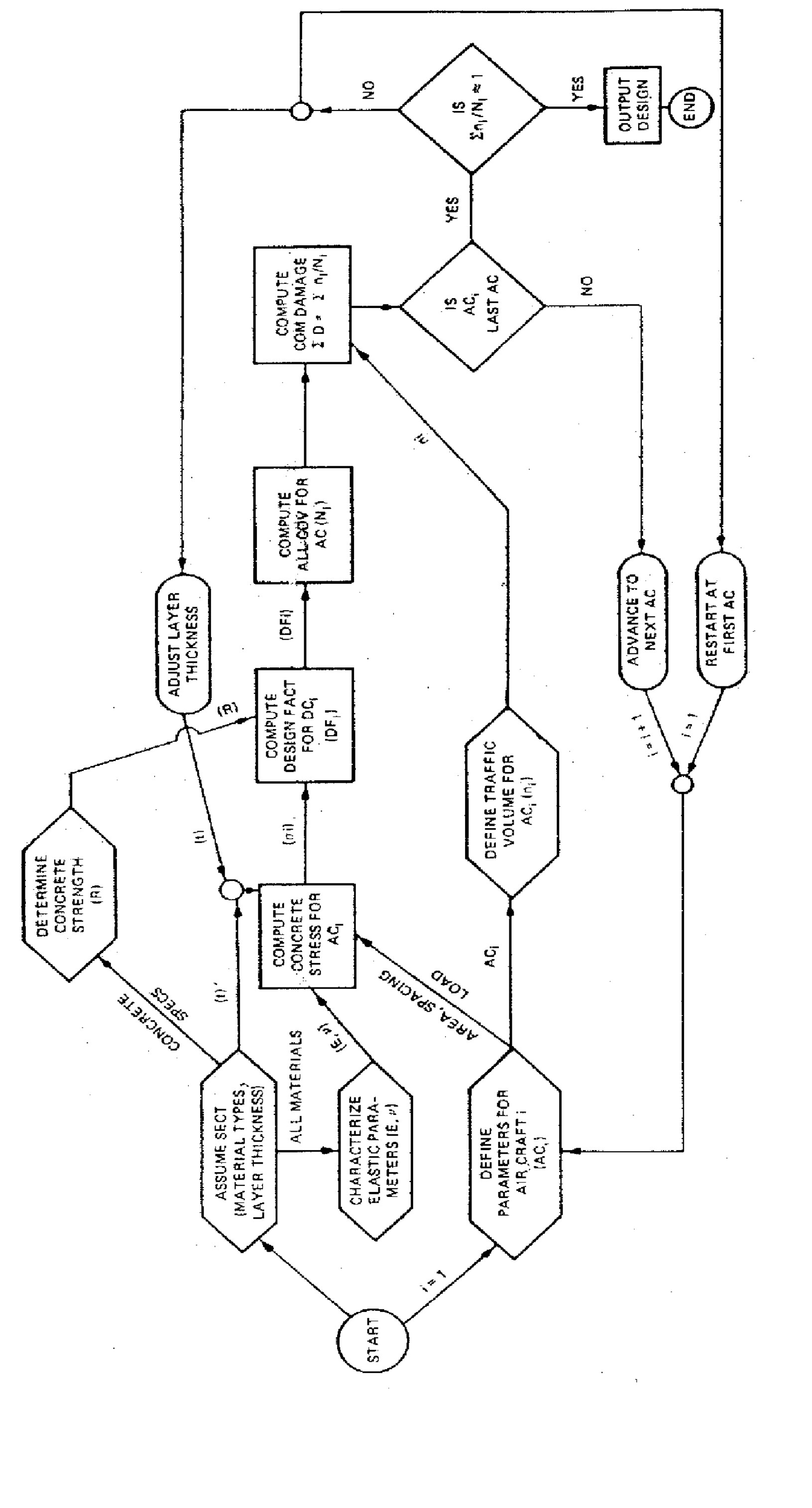
(2) Based on the computed stresses, determine the allowable coverages N_i ($N_i = C_o$ for initial cracking criteria, or $N_i = C_f$ for complete failure criteria) using Equations 19-1, 19-2, and 19-3 for each thickness design.

(3) Compute the damage for each design that 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.

(4) Select the proper slab thickness at a damage value of 1.0 from the relationship between damage and slab thicknesses.

(5) 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.

Figure 19-1. Diagram for Design of Airfield Rigid Pavements by Layered Elastic Theory



19-2 RIGID 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 the PCASE computer program. This program was chosen because it provides accurate computations and provisions for different degrees of bond between interfaces. Investigations into modeling rigid pavements with this program have resulted in the development of the performance criteria with the assumption 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 m (20 ft), a very stiff bottom layer is used to mitigate the assumption that the bottom layer extends to infinity. Figure 19-1 presents a diagram for the design of pavements using the layered elastic analysis.

19-3 DESIGN PROCEDURE

Design of rigid pavements using the layered elastic procedure is initiated by assuming a pavement section. The assumptions are the number of layers, types 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 (PCASE) to calculate the tensile stress F_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 the 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 be established quickly.

19-4 MATERIAL CHARACTERIZATION

19-4.1 PCC

The effects of repeated load on the 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.

Poisson's ratio for PCC usually receives very little attention. The range of statically determined Poisson's ratio is only approximately 0.11 to 0.21, and the average of dynamically determined values is approximately 0.24. Added factors are the difficulty of measurement and the 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. A value of 0.15 is recommended to be used for all PCC.

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, and range of loading. 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.

19-4.1.1 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 mm (6 by 6 in) with a length long enough to permit testing over a span of 457 mm (18 in). The recommended procedures are widely accepted and extensively used for determining the properties of PCC. The test procedures for determining flexural strength and modulus of elasticity will be determined in accordance with ASTM C78. When aggregate larger than the 51-mm (2-in) nominal size is used in the concrete, the mix will be wet-screened over a 51-mm (2-in) square mesh sieve before being used for casting the beam specimen.

19-4.1.2 Mix Proportioning and Control

Proportioning of the concrete mix and control of the concrete for pavement construction will be in accordance with UFC 3-250-04. Normally, a design flexural strength at 90 days will be used for pavement thickness determination. Should using the pavements at an earlier age be necessary, consider the use of a design flexural strength at the earlier age or the use of high early-strength cement, whichever is more economical.

19-4.2 Bound Bases (Subbases)

Chemically stabilized materials (for example, portland cement, lime, fly ash) and bituminous-stabilized materials must be discussed separately. 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.

19-4.2.1 Requirements

Bituminous base materials are designed in accordance with UFC 3-250-03. The design for frost consideration will be in accordance with Chapter 20 herein. Chemically stabilized materials should meet the requirements set forth in UFC 3-250-11. 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), the material must attain an unconfined compressive strength of 1.7 MPa (250 psi) at 28 days. This requirement should be used in lieu of the strength requirements in UFC 3-250-03, UFC 3-250-11, and Chapter 20. Chemically treated soils in which no substantial increase in strength is considered are deemed modified soils and should be characterized using the methods presented in this chapter 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 MAJCOM pavements engineer.

19-4.2.2 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 B, Section 10. 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.

19-4.2.2.1 A simply supported, unconfined beam loaded at the third point with what are essentially point loads will be used for bound bases (subbases). For chemically stabilized bound bases, the ultimate load is determined first. 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 that are less than the values shown in Table 19-1. One test should be conducted at approximately 0.34 MPa (50 psi).

19-4.2.2.2 An indirect method of obtaining an estimated modulus value for bituminous concrete is presented in detail in Appendix B, Section 4. This method requires determining 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.

19-4.2.2.3 No procedures are provided for determining Poisson's ratio of bound base material. The values in Table 19-2 are the recommended values to be used.

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)

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

19-4.3 **Unbound (Granular) Bases (Subbases)**

Unbound granular materials are extremely difficult to characterize. The state of stress, particularly the confining stress, is the dominant factor in determining load-deformation properties. Repeated loadings also affect the modulus of granular materials. The general pattern is that repeated loadings increase the stiffness if shear failure is not progressing. This implies that the modulus of elasticity is increased.

19-4.3.1 **Material Requirement**

Make a complete investigation to determine the source, quantity, and characteristics of available materials. In addition, make a study to determine the most economical thickness of material for a base course that will meet the requirements. The base course may consist of natural materials or processed materials, well graded and with high stability as specified in Chapter 8. All base courses to be placed beneath airfield rigid pavements will conform to these requirements:

(a) Well-graded course to fine

(b) Not more than 85 percent passing the 2-mm (No. 10) sieve

(c) Not more than 15 percent passing the 0.075-mm (No. 200) sieve

(d) PI not more than 8 percent

However, when it is necessary that the base course provide drainage, follow the requirements in Chapter 23. When frost penetration is a factor, follow the requirements in Chapter 20.

19-4.3.2 **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.

19-4.3.3 **Modulus and Poisson's Ratio**

Determine the modulus values of unbound granular bases (subbases) from cyclic triaxial tests on prepared samples. The recommended test procedure is outlined in Appendix B, Section 9. 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 provided in Appendix B, Section 8; however, the chart should be used in conjunction with test results to determine a representative modulus rather than as the sole method. Use a Poisson's ratio of 0.3 unless there is a reason to believe that this ratio is significantly different for the material in question.

19-4.4 **Subgrade Soils**

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 subbases. 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.

19-4.4.1 **Exploration**

In all instances, conduct field and laboratory tests to determine the classification, moisture-density relations, expansion characteristics, and strength of the subgrade. If stabilization of the subgrade is a possibility, perform other tests as required by UFC 3-250-11 in addition to determining chemical analysis and clay mineralogy. When a subgrade soil 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 only as part of the subgrade.

Note: Although the layered elastic method requires only the modulus of elasticity and Poisson's ratio of the subgrade, factors such 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.

19-4.4.2 **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 should represent the worst anticipated condition in the field. The recommended test procedures are outlined in Appendix B, Section 11. The procedures are similar to those used for granular base (subbase) materials. There are differences in the details of the test procedures and the presentation of the 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.

19-4.4.2.1 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 approximately 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.

19-4.4.2.2 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 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; in other words, as the confining stress and the first stress invariant decrease, the resilient modulus decreases.

19-4.4.2.3 Basically, the same stresses should be used in the triaxial tests for characterizing cohesionless material as are used for granular bases. Apply 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. From the average relationship of resilient modulus versus first stress invariant, select a representative modulus value at a first stress invariant of 68.9 kPa (10 psi). Select a representative value of Poisson's ratio 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.

19-4.4.3 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 (UFC 3-250-11). The *k* value has been determined by the field plate bearing test as described in CRD-C 655. When layered elastic procedures are used for pavements in which only information on the 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.

Note: Figure 19-2 should be used with caution because the correlation was developed based on very limited data.

19-5 DESIGN CRITERIA

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 SCI. The SCI is derived from a PCI as presented in ASTM D5340. The SCI is defined as:

$$SCI = PCI - All\ nonload-related\ deducts$$

The SCI prediction is based on a relationship between the design factor and stress repetitions for initial cracking (SCI = 100) and for complete failure (SCI = 0). The SCI 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.

19-5.1 Thickness

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 the design factor, the SCI, and the logarithm (to the base 10) of coverages by the equations:

$$DF = 0.5234 + 0.3920 \log_{10}(C_o) \tag{19-1}$$

and

$$DF = 0.2967 + 0.3881 \log_{10}(C_f) \tag{19-2}$$

The design factor is defined as:

$$DF = R / \sigma \tag{19-3}$$

where

DF = design factor computed with layered elastic method

R = concrete slab flexural strength, MPa (psi)

σ = maximum computed tensile stress with layered elastic model such as the PCASE 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 = SCI desired at the end of the pavement design life

19-5.2 Traffic Volumes

When aircraft passes are given, the pass-per-coverage ratio for the particular design aircraft will be used to convert passes to coverages.

Note: Exercise caution when using Equations 19-1 and 19-2. These equations 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.

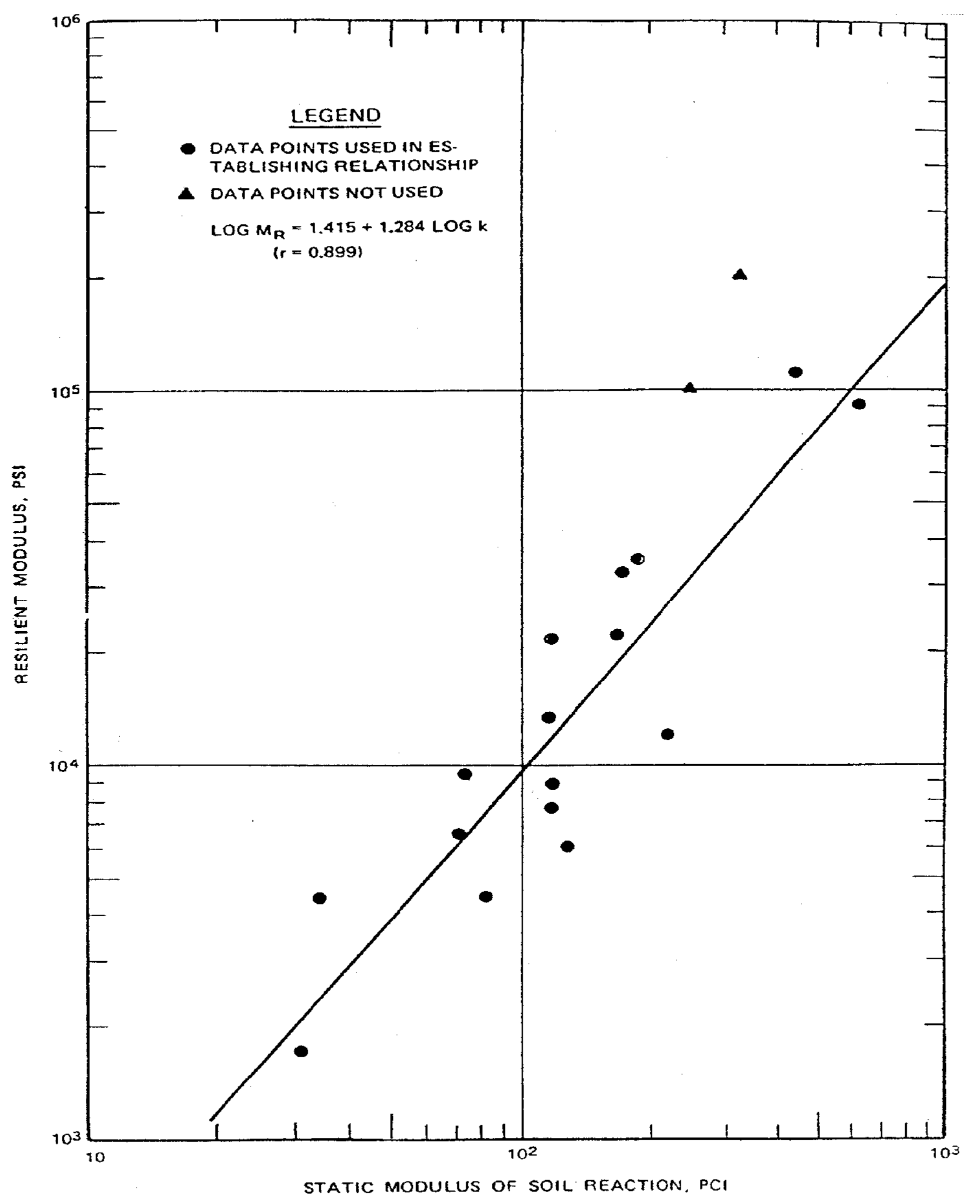
19-6 FROST CONSIDERATION

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 (DFI) or where the pavement is designed for heavy-load aircraft. When rigid airfield pavement is designed by the reduced subgrade strength method, use a minimum thickness of 102 mm (4 in) of granular unbound base. A mechanistic procedure for seasonal frost is being developed. Until it is available, use the method in Chapter 20.

19-7 ALTERNATE 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 a SCI that 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 a SCI. Because the methodology predicts performance, an accurate characterization of the materials, structural pavement condition, and fatigue is required. The steps for designing rigid overlays of rigid pavements are illustrated in Figure 19-1.

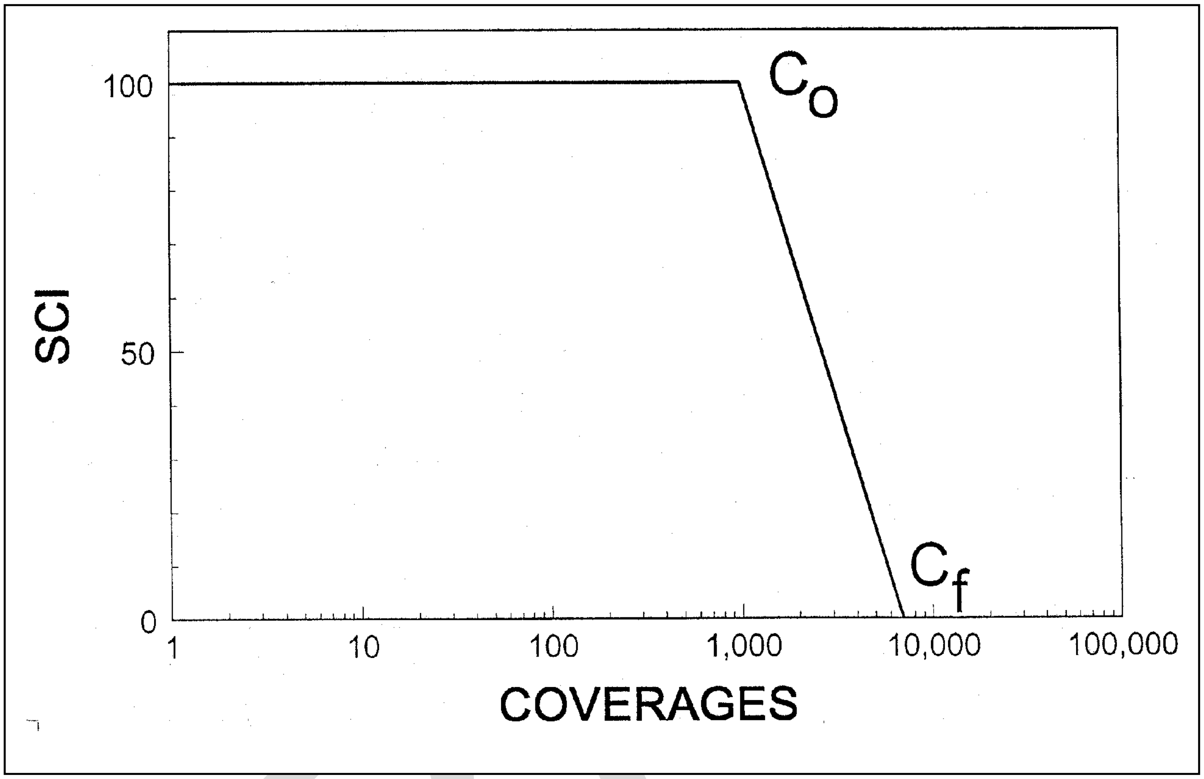
Figure 19-2. Correlation between Resilient Modulus of Elasticity and Static Modulus of Soil Reaction



CONVERSION FACTORS

MEGANEWTONS PER CUBIC METERS = 0.271 x PCI
MEGAPASCALS = 0.006894 x PSI

Figure 19-3. Relationship between SCI and Coverages at Initial Cracking and Complete Failure



19-7.1 **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) The 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 should 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 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 fully bonded.

19-7.2 Base Slab Pavement Fatigue and Structural Condition

Traffic applied on the base slab before the overlay is placed consumes some of the base slab's fatigue life. If the base slab has begun to deteriorate from traffic, a 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 this relationship:

$$R_E = \frac{E_e}{E_i} = 0.02 + 0.0064 * SCI + (0.00584 * SCI)^2 \quad (19\text{-}4)$$

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.

19-7.3 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.

19-7.4 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 are used in conjunction with Equations 19-5 and 19-6 to compute the damage rates:

$$DR_o = \sum_{i=1}^{nac} \frac{(C_r)_i}{C_o} \text{ and } DR_f = \sum_{i=1}^{nac} \frac{(C_r)_i}{C_f} \quad (19\text{-}5)$$

$$t_o = \frac{B_o}{DR_o} \text{ and } t_f = \frac{B_f}{DR_f} \quad (19\text{-}6)$$

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. ($B_o = 1 - \Sigma C_i / C_o$ and $B_f = 1 - \Sigma C_i / C_f$) C_i = applied past traffic, coverages

19-7.5 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.

19-7.6 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:

$$(d_o)_j = \Delta T_j * \sum_{i=1}^{nac} \frac{C_{ij}}{(C_o)_j} \quad and \quad (d_f)_j = \Delta T_j * \sum_{i=1}^{nac} \frac{C_{ij}}{(C_f)_j} \qquad (19\text{-}7)$$

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 1. 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.

19-8 REINFORCED 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 thicknesses 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 are shown in Figure 19-7. Assuming that the proposed elastic layer design procedure can result in adequate thicknesses of unreinforced pavement, application of the criteria illustrated in Figure 19-7 will result in adequate thicknesses of reinforced pavements.

19-9 DESIGN EXAMPLES

Design examples are provided in Appendix B, Section 14.

Figure 19-4. Base Slab Performance Curve (SCI versus Logarithm of Time) Subdivided into Five Time Periods

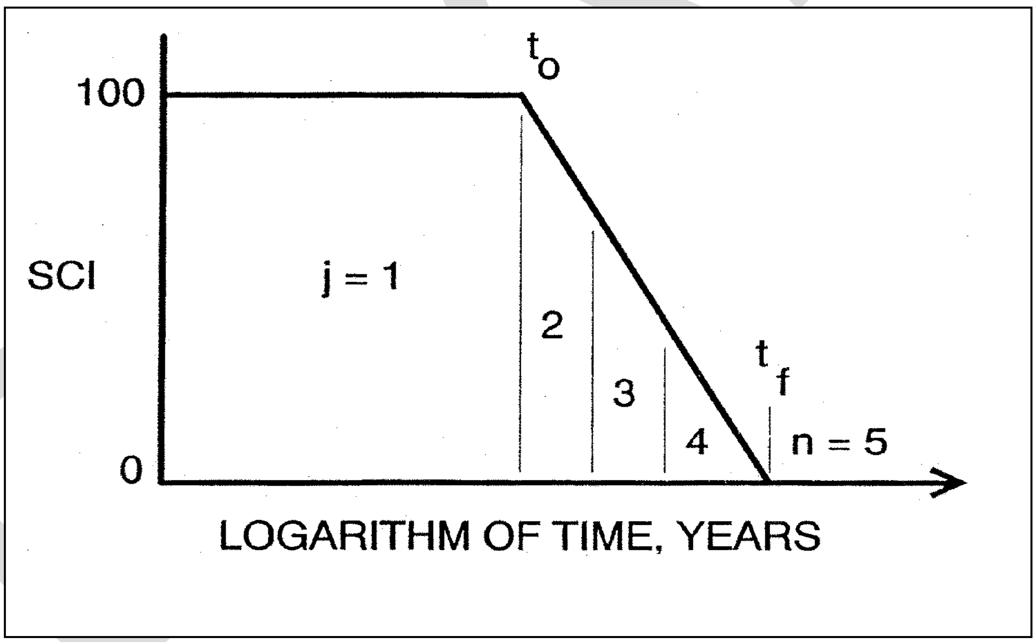


Figure 19-5. Composite Overlay Performance Curve

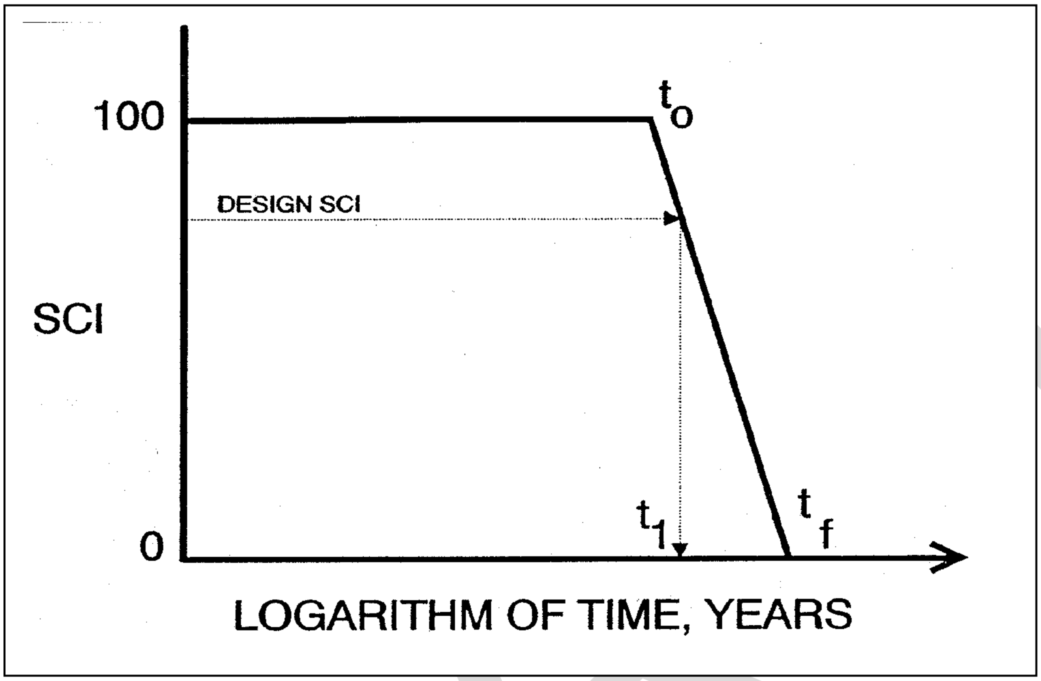


Figure 19-6. Overlay Thickness versus Logarithm of Time

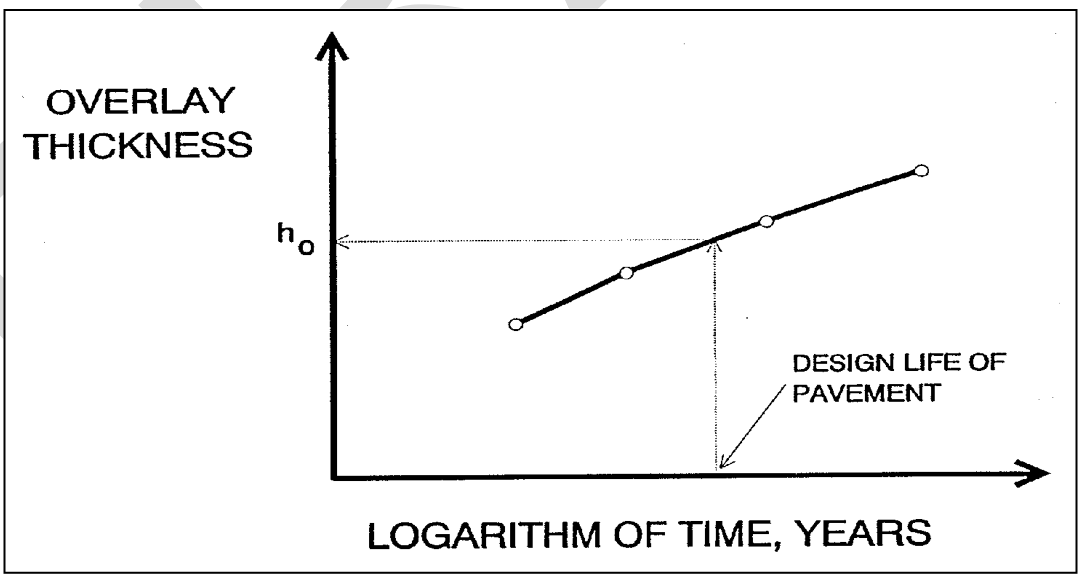
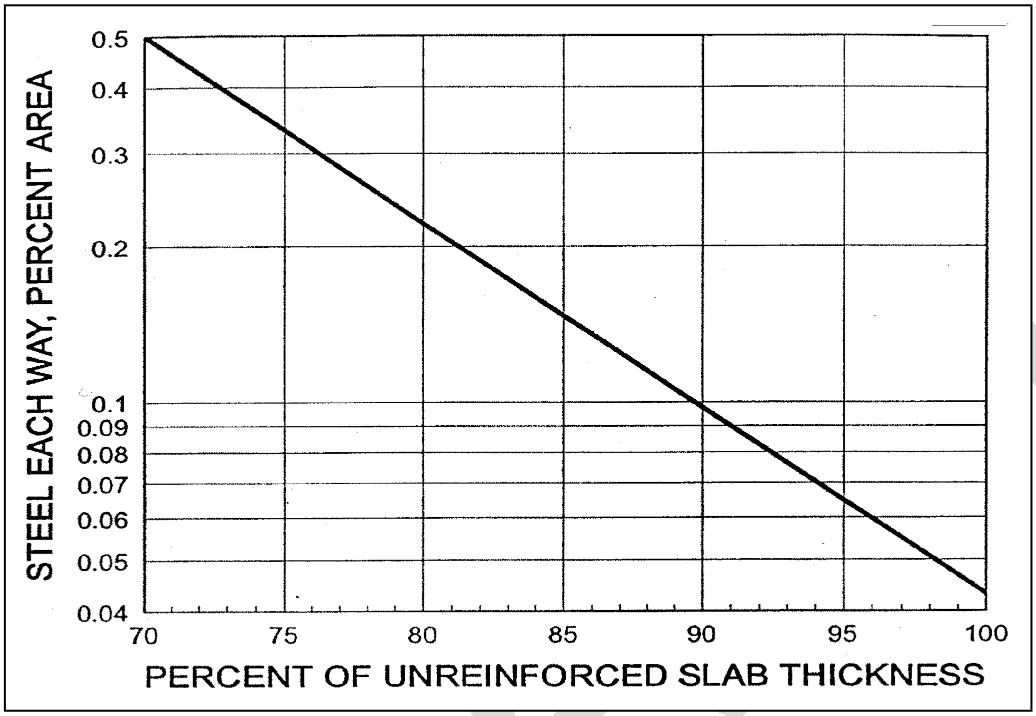


Figure 19-7. Effect of Steel Reinforcement on Rigid Pavements



CHAPTER 20

SEASONAL FROST CONDITIONS

20-1 OVERVIEW

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 non-frost design, pavement design need not consider the effects of frost action unless the base and subbase courses contain other than NFS, PFS, S1, or S2 materials (see Table 20-1). The designer must select subbase materials that 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 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 AC pavements. In extreme conditions, these problems can cause hazardous operating conditions or 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 that 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.

20-2 FROST SUSCEPTIBILITY

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.

20-2.1 Classification

The frost susceptibility of the soils classified in Table 20-1, based on laboratory tests, is 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 per day (mm/day), and the thawed CBR ranges between 12 and 20 percent. These soils are suitable as base and subbase course materials. Soils categorized as F1, F2, F3, and F4 are unsuitable as base or subbase materials.

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 (b) Sands	0-1.5 0-3	GW, GP SW, SP
PFS[2]	(a) Gravel Crushed Stone Crushed Rock (b) Sands	1.5-3 3-10	GW-GP 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 (b) Sands	10-20 6-15	GM, GW-GM, GP-GM SM, SW-SM, SP-SM
F3	(a) Gravely Soils (b) Sands, except very fine silty sands (c) Clays, PI>12	Over 20 Over 15 --	GM, GC SM, SC CL, CH
F4	(a) Silts (b) Very fine silty sands (c) Clays, PI<12 (d) Varved clays and other fine grained, banded sediments	-- Over 15 -- --	ML, MH SM CL, CL-ML CL, ML, CL-ML, CL, ML, and SM, CL, CH, and ML, CL, CH, ML, and SM

[1] Non-frost-susceptible
[2] Possibly frost susceptible; requires laboratory test to determine frost design soil classification.

20-2.2 Waivers

Under special conditions, the frost group classification adopted for design may differ from that obtained by application of the frost group definitions in paragraph 20-2.1, but only if 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 the performance of existing pavements near those proposed to be constructed. This will require the approval of the Naval Facilities Engineering Service Center, the appropriate Air Force MAJCOM pavements engineer, or USACE-TSC.

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

20-3 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 supporting capacity, including severe weakening during frost melting periods. The three methods are (a) the complete frost penetration method, (b) the reduced subgrade strength method, and (c) the limited subgrade frost penetration method.

20-3.1 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 the effects 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. This method will not be used to design pavements to serve conventional traffic except when approved with the appropriate written waiver.

20-3.2 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.

20-3.3 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.

20-4 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 (for example, in some glaciated areas) and the required subgrade preparation could not be expected to sufficiently restrict differential frost heave. Additionally, it will be used 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 use of the limited subgrade frost penetration method is not required, preliminary designs must be prepared using both methods for comparison of costs.

20-5 REDUCED SUBGRADE STRENGTH METHOD

The thickness design procedure 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. This 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 in which the 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 overly conservative, intermediate design base course thickness may be used; however, this must be justified on the basis of frost heaving experience developed from existing pavements where climatic and soil conditions are comparable.

20-5.1 Thickness of Flexible Pavements

The thickness design procedure is identical to the thickness design for non-frost conditions with the exception that instead of using the subgrade CBR, this procedure uses Frost Area Soil Support Index (FASSI) values. 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 back-calculated from performance data of in-service pavements and are the weighted average CBR values 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.

Once the overall thickness of the pavement structure has been determined, use the criteria for non-frost design to determine the thickness of individual layers. In addition, ascertain whether it will be advantageous to incorporate one or more bound base layers in the system. Although the use of bound bases will reduce the thickness of the base and subbase layers, deeper frost penetration may occur, leading to increased frost heave. The base course requirements set forth in this chapter must be followed rigorously.

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

20-5.1.1 **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 these limitations:

- It shall not be less than required for non-frost conditions design.
- It shall not exceed the thickness required under the limited subgrade frost penetration design method.
- It shall not be less than the thickness 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.

20-5.1.2 **Control of Surface Roughness in Overruns**
For frost groups F3 or F4 subgrades, differential heave can usually be controlled to 76 mm (3 in) in 15.2 m (50 ft) 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.

20-5.1.3 **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.

20-5.1.4 **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 m (5 ft) radially from the structures should be designed and constructed entirely with NFS 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, though suitable transitions to the adjoining uninsulated pavement would be needed.

20-5.1.5 **Drainage**

Subsurface drainage must be provided in flexible pavements in accordance with Chapter 23.

20-5.2 **Rigid Pavement Thickness Design**

The thickness design procedure is identical to the thickness design for non-frost conditions, with the exception that instead of using the modulus of subgrade reaction *k*, the thickness design procedure uses Frost Area Index of Reaction (FAIR) values. 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 these equations:

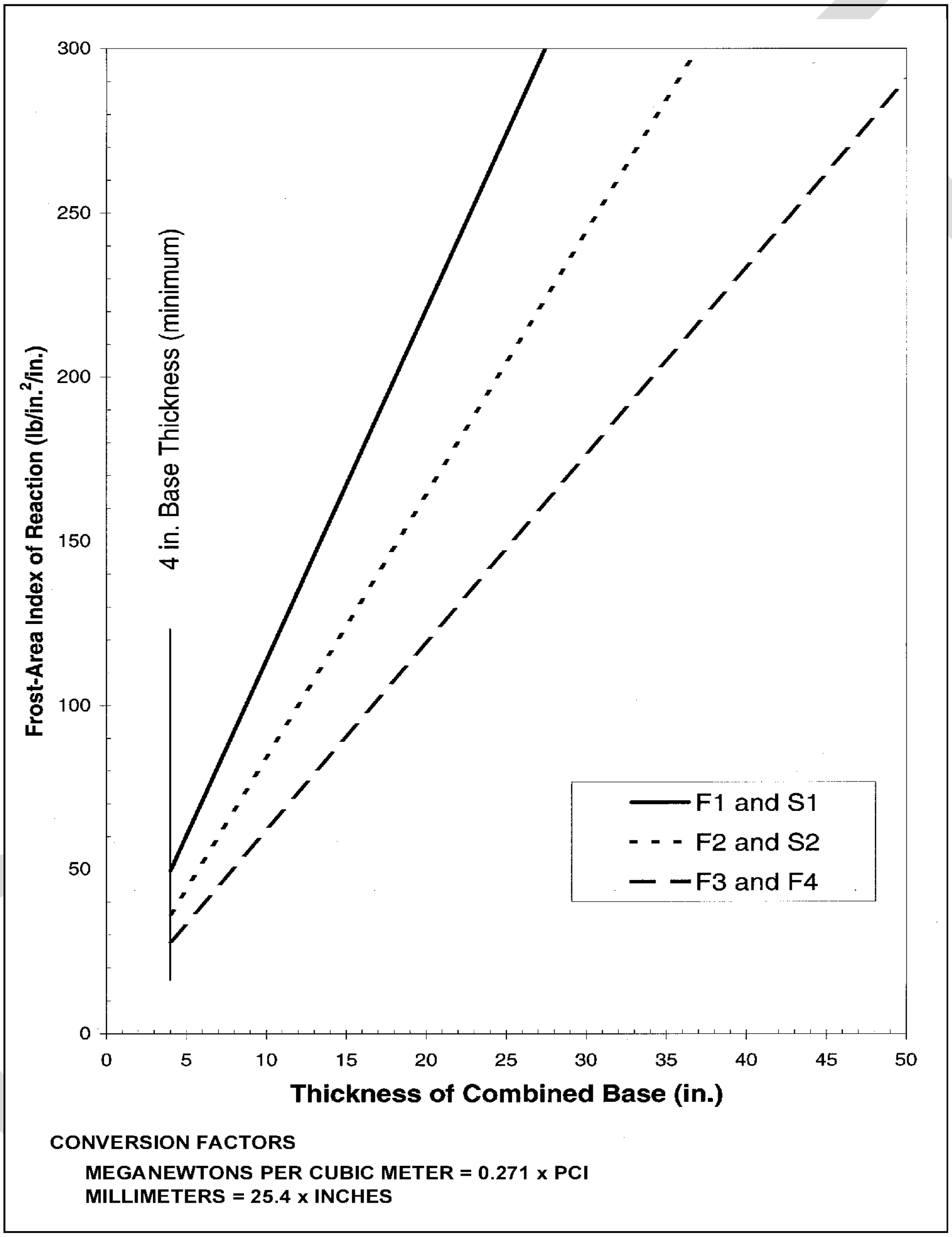
$$\begin{aligned}&\text{S1 or F1 material:}\\&\text{FAIR (pci)} = 6.7 + 10.7 \times \text{base - course thickness (in)}\\&\text{or}\\&\text{FAIR}\left(\frac{\text{MN}}{\text{m}^3}\right) = 1.8 + 114 \times \text{base - course thickness (m)}\end{aligned} \tag{20-1}$$

$$\begin{aligned}&\text{S2 or F2 material:}\\&\text{FAIR (pci)} = 4.5 + 8.0 \times \text{base} - \text{course thickness (in)}\\&\text{or}\\&\text{FAIR}\left(\frac{\text{MN}}{\text{m}^3}\right) = 1.2 + 83.8 \times \text{base} - \text{course thickness (m)}\end{aligned} \tag{20-2}$$

$$\begin{aligned}&\text{F3 or F4 material:}\\&\text{FAIR (pci)} = 5.4 + 5.7 \times \text{base} - \text{course thickness (in)}\\&\text{or}\\&\text{FAIR}\left(\frac{\text{MN}}{\text{m}^3}\right) = 1.5 + 60.8 \times \text{base} - \text{course thickness (m)}\end{aligned} \tag{20-3}$$

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.

Figure 20-1. Frost Area Index of Reaction (FAIR) for Design of Rigid Pavements



20-5.2.1 It is a good practice to use a combined base thickness equal in thickness to the slab. This is the design procedure:

(1) Determine the frost group soil classification of the subgrade using Table 20-1.

(2) Assume three combined base thicknesses; enter Figure 20-1 or use the appropriate equations; and determine the FAIR value for each thickness.

(3) Use the FAIR values with the appropriate design curves to determine pavement thickness.

(4) Plot the combined base thickness and pavement thickness. From the figure, pick out a base course and pavement thickness of similar values.

(5) If unable to converge to a solution, repeat steps two to four with a new base course thickness.

(6) A minimum of 203 mm (8 in) of combined base (100-mm [4-in] drainage layer plus 100-mm [4-in] separation layer) is required for rigid pavements in frost areas.

20-5.2.2 The combined base must meet the drainage and filter requirements outlined in Chapter 23. A 100-mm (4-in) 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 Chapter 23.

20-5.2.3 Bound base also has significant structural value and is considered a low-strength concrete for design purposes. A minimum 203-mm (8-in) drainage plus separation layer must be placed between the bound base and the subgrade.

20-5.2.4 If sufficient high-quality base material is not locally available, the non-frost design base layer thickness can be used. The appropriate FAIR value will be used for the base to determine the PCC thickness.

20-5.2.5 The subgrade preparation techniques and transition details outlined in this chapter are also required for the design of overrun pavements.

20-5.2.6 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 m (5 ft) radially from the structures should be designed and constructed entirely with NFS 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, though suitable transitions to the adjoining uninsulated pavement would be needed.

20-6 LIMITED SUBGRADE FROST PENETRATION METHOD

This design method permits a small amount of frost penetration into frost-susceptible subgrades. The procedure uses a 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.

Figure 20-2. Determination of Freezing Index

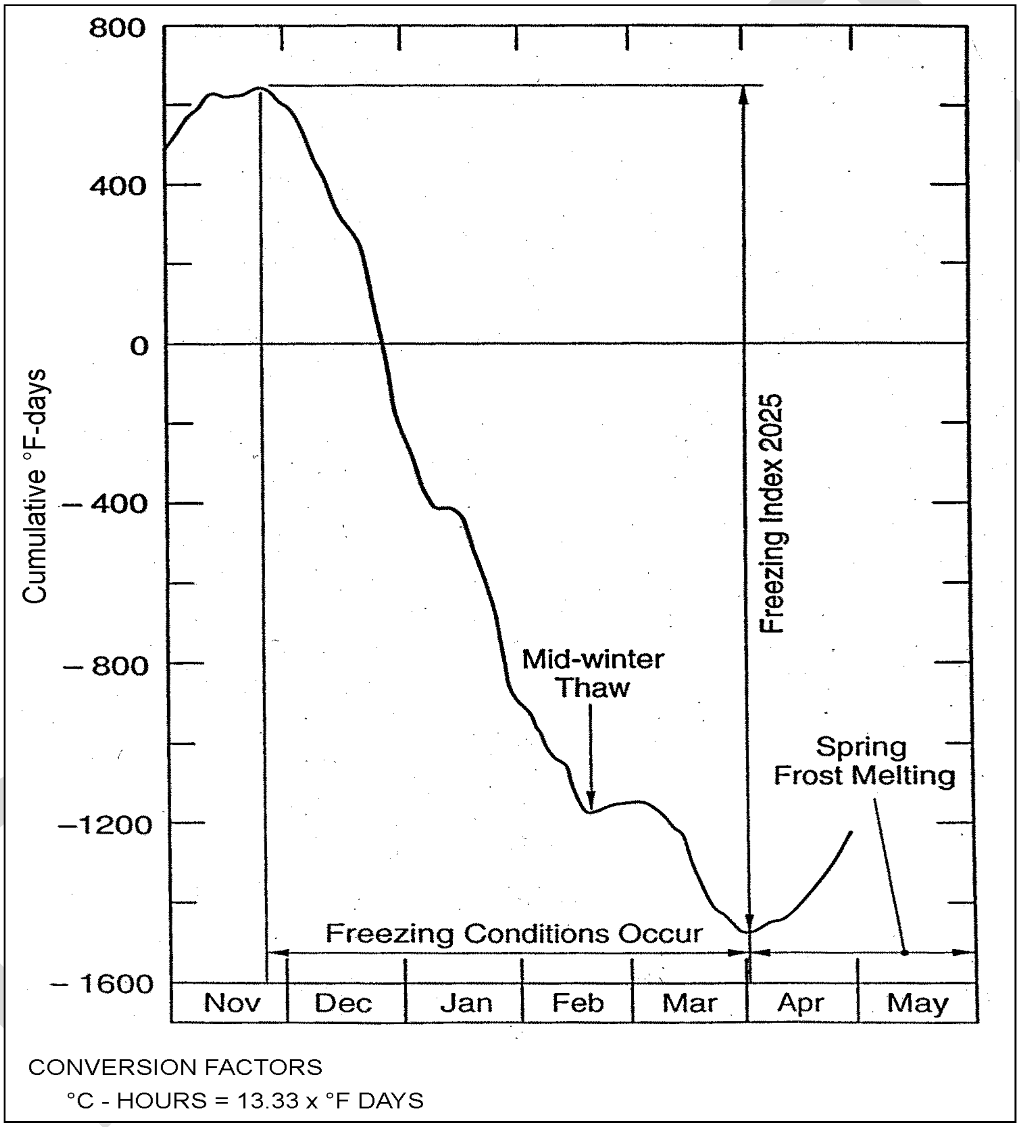
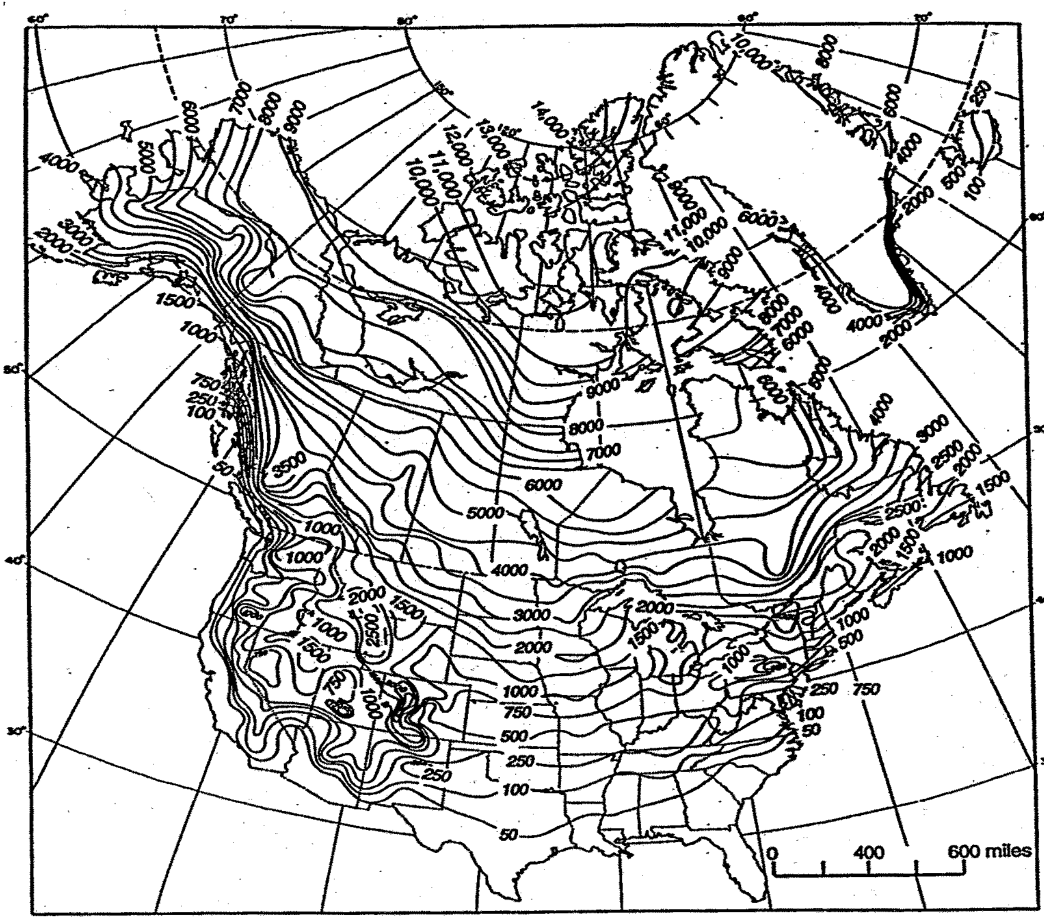
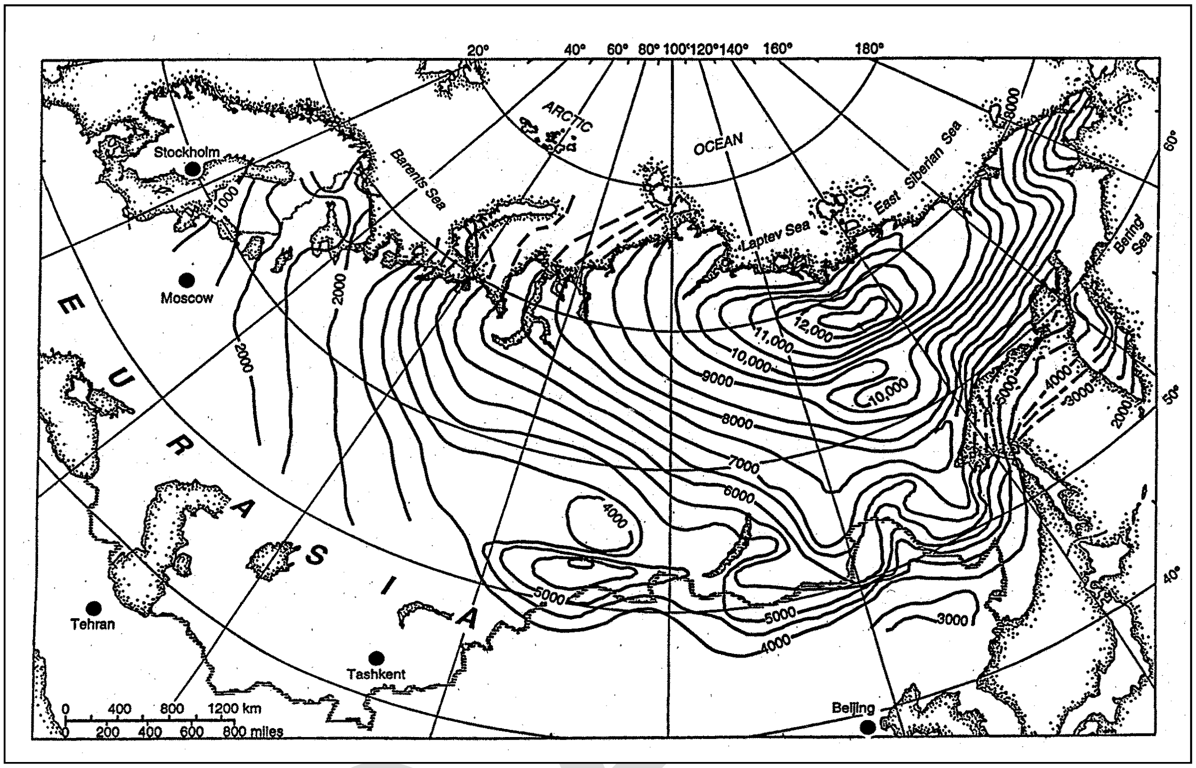


Figure 20-3. Distribution of Design Freezing Indexes in North America



CONVERSION FACTORS
°C - HOURS = 13.33 x °F DAYS

Figure 20-4. Distribution of Design Freezing Indexes in North Eurasia



20-6.1 **Limited Subgrade Frost Penetration Method: Step One**
Determine the frost penetration depths. The maximum frost penetration depths with respect to the DFI shown in Figure 20-5 are calculated from the modified Berggren formula and the computational procedures outlined in UFC 3-130-06. 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 soils beneath the pavement within the depth of frost penetration are granular and NFS, and that all soil moisture freezes at 0 degrees C (32 degrees F). Use straight-line interpolation where necessary. The frost penetration depth (in meters for SI units and inches for English units) can also be estimated from these equations:

(1) For γ = 2,160 kg/m^3 (135 lb/ft^3) and ω = 3 percent,

$$a = 0.157 + 9E - 5(DFI) - 4E - 10(DFI)^2 \text{ in SI units} \quad (20\text{-}4)$$

$$a = 6.183 + 0.047(DFI) - 2.91E - 6(DFI)^2 \text{ in English units} \quad (20\text{-}5)$$

(2) For γ = 2,160 kg/m^3 (135 lb/ft^3) and ω = 7 percent,

$$a = 0.1852 + 8E - 5(DFI) - 4E - 10(DFI)^2 \text{ in SI units} \quad (20\text{-}6)$$

$$a = 7.291 + 0.044(DFI) - 2.58E - 6(DFI)^2 \text{ in English units} \quad (20\text{-}7)$$

(3) For γ = 2,400 kg/m^3 (150 lb/ft^3) and ω = 3 percent,

$$a = 0.1725 + 0.0001(DFI) - 5E - 10(DFI)^2 \text{ in SI units} \qquad (20\text{-}8)$$

$$a = 6.793 + 0.055(DFI) - 3.41E - 6(DFI)^2 \text{ in English units} \qquad (20\text{-}9)$$

(4) For γ = 2,400 kg/m^3 (150 lb/ft^3) and ω = 7 percent,

$$a = 0.1583 + 9E - 5(DFI) - 4E - 10(DFI)^2 \text{ in SI units} \qquad (20\text{-}10)$$

$$a = 6.231 + 0.049(DFI) - 2.98E - 6(DFI)^2 \text{ in English units} \qquad (20\text{-}11)$$

where

DFI = degrees Celsius-hours in SI units or Fahrenheit degree days in English units

γ = soil density

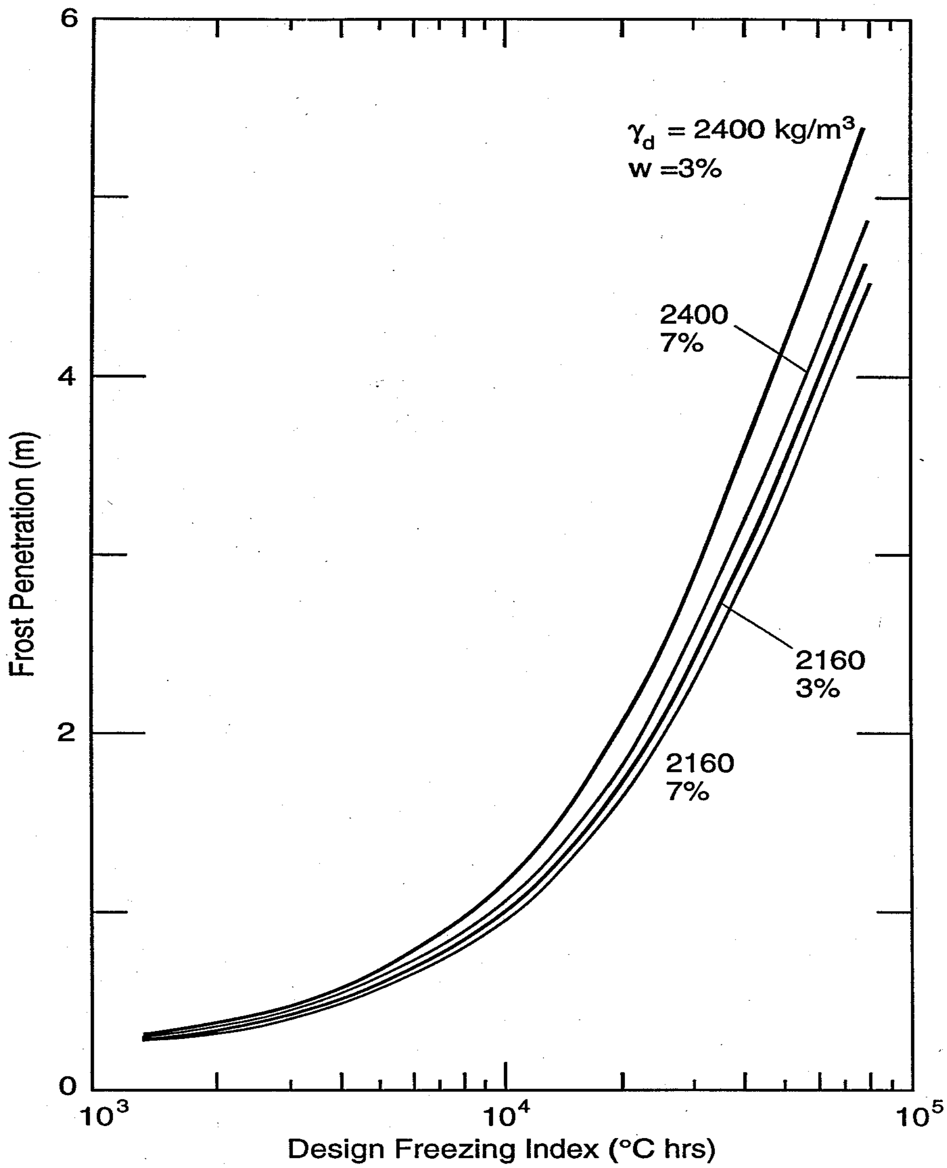
ω = soil moisture content/water content

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.

20-6.2 Limited Subgrade Frost Penetration Method: Step Two

Estimate the moisture content and dry density of the NFS 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 the frost penetration depth for complete frost penetration from Figure 20-5.

Figure 20-5. Frost Penetration beneath Pavements



CONVERSION FACTORS

°C HOURS = 13.33 x °F DAYS
MILLIMETERS = 25.4 x INCHES

20-6.3 Limited Subgrade Frost Penetration Method: Step Three

Compute the thickness of the combined base (combined thickness of base, subbase, drainage layer, and separation layer) required for zero frost penetration into the subgrade (Figure 20-6) using Equation 20-12:

$$c = a - p \qquad (20\text{-}12)$$

where

c = thickness of unbound base, mm (in)

a = thickness for complete frost protection, mm (in)

p = thickness of asphalt or concrete for non-frost design

20-6.4 Limited Subgrade Frost Penetration Method: Step Four

For limited frost penetration into the subgrade, determine the average moisture content of the subgrade prior to freezing. Compute the water content ratio *r* using Equation 20-13:

$$r = \frac{\text{moisture content of subgrade}}{\text{moisture content of base}} \qquad (20\text{-}13)$$

where

moisture content of the base = same as that assumed for non-frost 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.

20-6.4 Limited Subgrade Frost Penetration Method: 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 20-14 and 20-15. 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 DFI year.

$$b = c \times f \qquad (20\text{-}14)$$

$$s = c \times g \qquad (20\text{-}15)$$

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 in Table 20-4

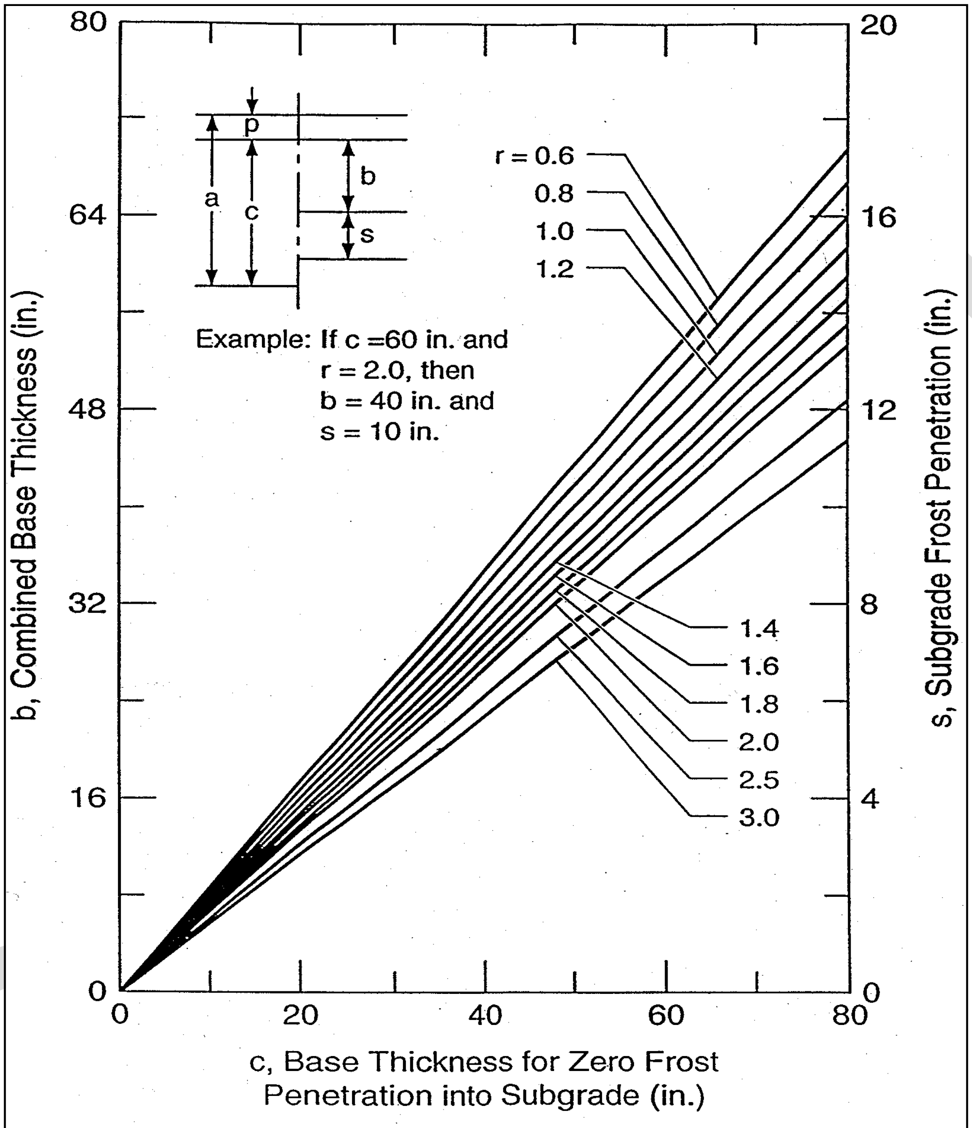
Table 20-4. Factors *f* and *g*

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

20-6.5 **Limited Subgrade Frost Penetration Method: Step Six**
When the maximum combined thickness of pavement layers required by this design procedure exceeds 1.5 m (60 in), use a total combined thickness to 1.5 m (60 in). Limiting the combined thickness of pavement and base to 1.5 m (60 in) 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 508 mm (20 in) of the subbase. When using this alternative, be certain to use materials from frost groups S2 or better as subbase layers. If selecting either the high moisture retention subbase course or a combined thickness over 1.5 m (60 in) for frost design purposes, obtain the specific approval of USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or the Naval Facilities Engineering Service Center.

20-6.6 **Limited Subgrade Frost Penetration Method: Step Seven**
Compare the combined thickness of pavement layers required for limited subgrade frost penetration with that obtained for non-frost conditions, and adopt the thicker of the two cross sections as the design thickness.

Figure 20-6. Design of Combined Base Thickness for Limited Subgrade Frost Penetration



20-7 GRANULAR BASE AND SUBBASE COURSE REQUIREMENTS

The base course material used in pavements in seasonal frost areas will meet the requirements in Chapter 8 for base course. In addition, these requirements must be met:

(a) The top 50 percent of the combined base and subbase thickness must be NFS.

(b) The lower 50 percent thickness of combined base may be either NFS material, possibly 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 part of the combined base. If not, an additional 100-mm (4-in) separation layer is required.

(c) Base and subbase course materials of borderline quality should be tested frequently after compaction to ensure that the compacted material meets requirement (a). For material expected to exhibit serious degradation during placement and compaction (greater than 3 percent finer than 0.02 mm 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, change the material gradation 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, consider stabilizing the material with asphalt or portland cement.

(d) 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.

20-8 DRAINAGE LAYER REQUIREMENTS

A minimum 100-mm-thick (4-in-thick) NFS drainage layer is required and will be placed in accordance with Chapter 23. The layer is considered a structural component of the pavement.

20-9 SEPARATION LAYER

If subgrade freezing will occur, a minimum of a 100-mm (4-in) granular separation layer is required and will be placed in accordance with Chapter 23. Over weak subgrades, a 152-mm (6-in) 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 Chapter 23. An additional requirement is that the separation layers must be NFS or of frost group S1 or S2. Alternatively, where stable foundation already exists, geotextile fabrics meeting the requirements of Chapter 23 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 Chapter 23.

20-10 SUBGRADE REQUIREMENTS

In addition to the requirements outlined in Chapter 6 for subgrades in non-frost areas, these additional requirements shall apply for subgrades in frost areas. For all pavements constructed in frost areas, a basic requirement is that subgrades in which freezing will occur must be as uniform as possible. This will be accomplished 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 practice 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. In some cases, to achieve uniformity, removing high frost-susceptible soils or soils of low frost susceptibility will be necessary. In those cases, 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 here 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. Then the material will be re-laid 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.

20-10.1 **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 the lesser of:

(a) 0.6 m (24 in)

(b) 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

(c) One-half of the frost penetration less the actual combined thickness of pavement, base course, drainage layers, and subbase course under Types C, D, and secondary traffic areas, and under overruns and shoulder pavements

(d) 1.8 m (72 in) less the actual combined thickness of pavement, base course, drainage layers, and subbase course

The prepared subgrade must meet the designated compaction requirements for non-frost 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.

20-10.2 **Exceptional Conditions**

Subgrades that are NFS or of very low frost susceptibility (NFS, S1, S2) to the depth prescribed for subgrade preparation are an exception to the basic requirements 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 or of otherwise reducing the moisture content and which consequently cannot feasibly be scarified and recompacted are also exceptions. If a wet, fine-grained subgrade exists at the site, preventing frost penetration with fill material will be necessary. 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 NFS or frost-susceptible material. If the fill or backfill is frost susceptible, it should be subjected to the same subgrade preparation procedures prescribed in this section.

20-10.3 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 approximately 152 mm (6 in) 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. In extreme cases, they eventually break through the surface, necessitating complete reconstruction.

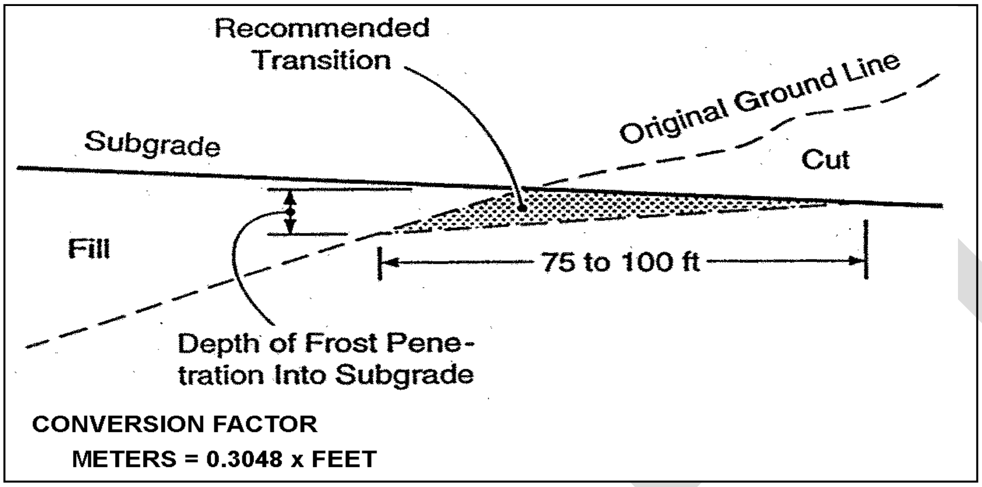
20-10.4 Soil Conditions

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 20-7 should be removed and replaced with fill material. Discontinuities in subgrade conditions require the most careful attention of designers and construction inspection personnel because failure to enforce strict compliance with the requirements for transitions may result in serious pavement distress.

20-10.5 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. Filling drainage pockets with lean concrete may be necessary. At intersections of fills with rock cuts, the tapered transitions illustrated in Figure 20-7 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 NFS 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 m (3 to 6 ft) 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 remove excess water quickly.

Figure 20-7. Tapered Transition used where Embankment Material Differs from Natural Subgrade in Cut



20-11 CONTROL OF DIFFERENTIAL HEAVE AT DRAINS, CULVERTS, DUCTS, INLETS, HYDRANTS, AND LIGHTS

20-11.1 Design Details and Transitions for Drains, Culverts, and Ducts

Drains, culverts, and utility ducts placed under pavements on frost-susceptible subgrades frequently experience differential heaving. Wherever possible, avoid placing such facilities beneath pavements. Where this cannot be avoided, construction of drains should be in accordance with the “correct” method indicated in Figure 20-8, and treatment of culverts and large ducts should conform to Figure 20-9. All drains of similar features should be placed first and the base and subbase course materials carried across them without break to obtain maximum uniformity of pavement support. The practice of constructing the base and subbase course and then excavating back through them for operations such as to install drains and pipes is undesirable because a marked discontinuity in support will result. Transitions shall be provided in accordance with section 20-12. 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 20-8. 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. The recommended practice is shown in the “correct” column in Figure 20-8.

Figure 20-8. Subdrain Details for Cold Regions

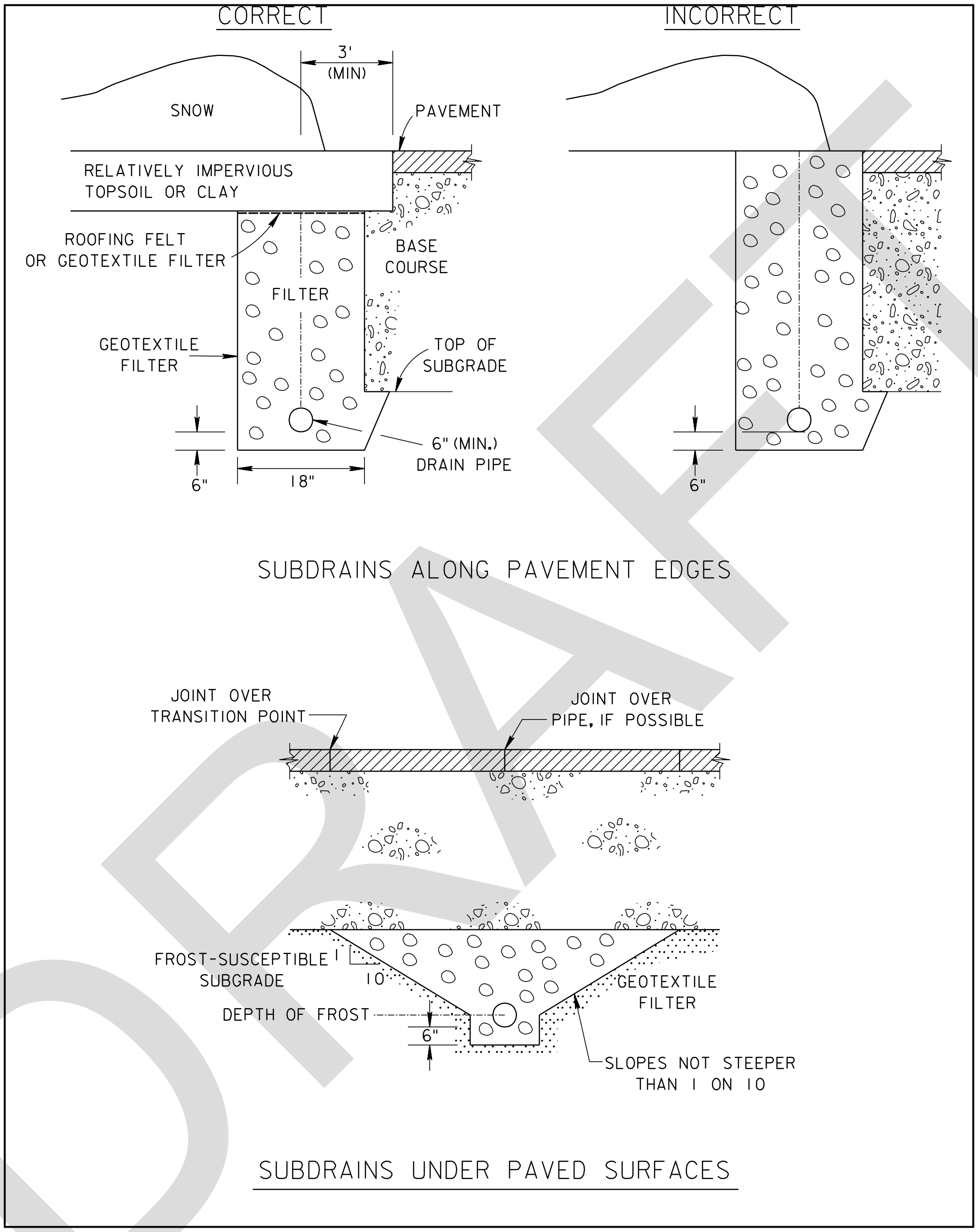
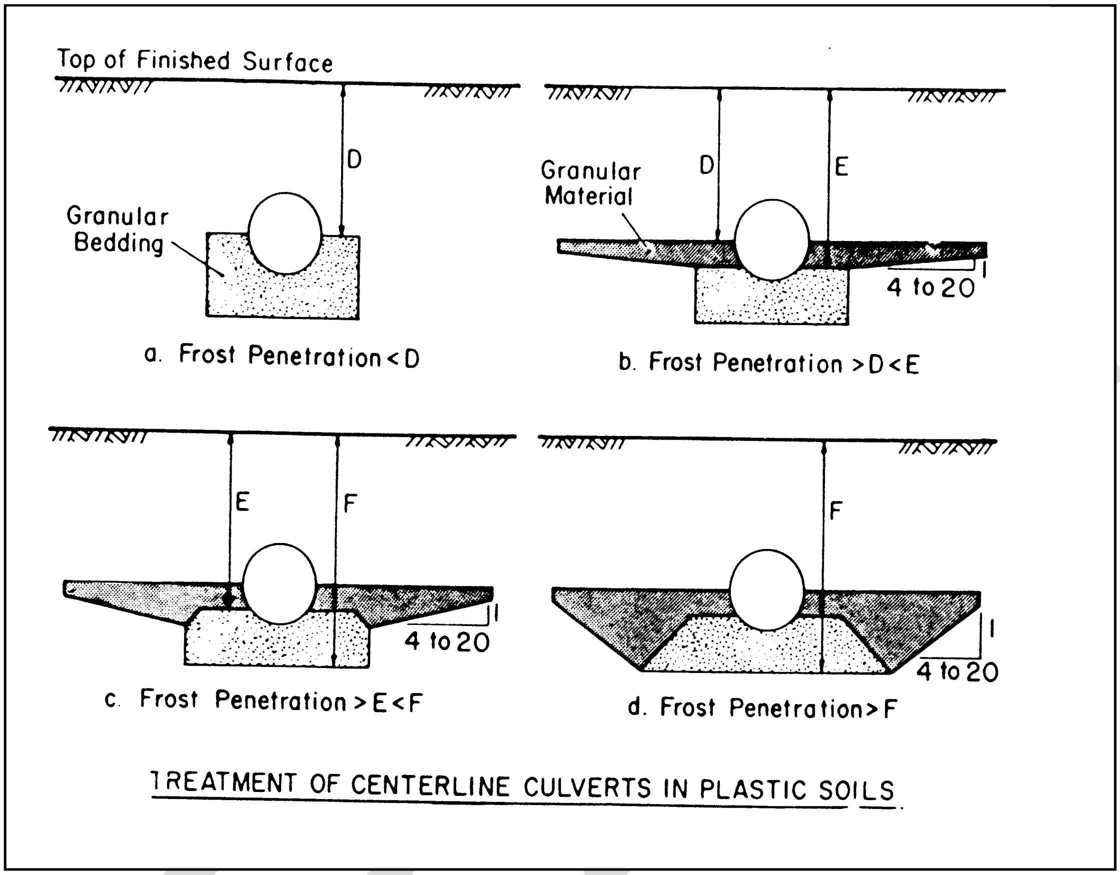


Figure 20-9. Transitions for Culverts beneath Pavements



20-11.2 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 than 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, design the pavement section beneath the inserts and extending at least 1.5 m (5 ft) radially from them to prevent freezing of frost-susceptible materials by use of an adequate thickness of NFS base course and by use of insulation. Also, consider anchoring footings with spread bases at appropriate depths. Gradual transitions are required to surrounding pavements that are subject to frost heave.

20-12 PAVEMENT THICKNESS TRANSITIONS

20-12.1 Longitudinal Transitions

Where interruptions in pavement uniformity cannot be avoided, differential frost heaving should be controlled by use of gradual transitions. The 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 mm (1 in) may be expected at changes from one subgrade soil condition to another, gradual changes in base thicknesses should be effected over distances of 61 m (200 ft) for the runway area, 30.5 m (100 ft) for taxiways, and 15.25 m (50 ft) for aprons. In each case, the transition 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.

20-12.2 **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.

20-13 **OTHER MEASURES TO REDUCE FROST HEAVE**

Another measure to reduce the effects of heave is the use of insulation to control the depth of frost penetration. Insulation can be used only in shoulders and overruns. The use of synthetic insulating materials within a pavement cross section must have the approval of USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or the Naval Facilities Engineering Service Center. 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.

20-14 **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 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 noted 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 AC. PCC pavements generally require smaller base and subbase thicknesses than AC 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 kind of materials previously used; however, be careful using materials similar to those that resulted in the initial distress. Materials that are frost susceptible and placed too near the pavement surface can result in premature failure.

20-15 COMPACTION

Subgrade, subbase, and base course materials must meet the applicable compaction requirements for non-frost materials.

20-16 DESIGN EXAMPLES

Design examples for seasonal frost are contained in Appendix B, Section 14.

CHAPTER 21

IMPROVING SKID RESISTANCE/REDUCING HYDROPLANING POTENTIAL OF RUNWAYS

21-1 OVERVIEW

This chapter presents procedures for improving skid resistance and reducing the hydroplaning tendency of runways. It applies to the Army and Air Force. Navy guidance on this topic is contained in NAVFAC Interim Technical Guidance (ITG) FY99-02, *Skid Resistance Criteria for Airfield Pavements*.

21-1.1 Skid Resistance

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.

21-1.2 Hydroplaning

Hydroplaning occurs when a tire loses contact with the pavement 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.

21-1.3 Friction Testing and Equipment

Procedures for conducting friction testing and an approved equipment list are contained in FAA AC 150/5320-12C.

21-2 IMPROVING RUNWAY FRICTION CHARACTERISTICS

New, reconstructed, or resurfaced runways must be grooved except when resurfaced with a porous friction surface (PFS). Grooving is required to provide an acceptable surface for the safe operation of aircraft. Friction characteristics of existing runways should be improved when tests indicate that the surface has a potential for hydroplaning. Improving the friction characteristics can be accomplished in several ways: grooving, resurfacing with a PFS, retexturing, improving runway slopes, and removing rubber. Table 21-1, developed by the National Aeronautics and Space Administration (NASA), provides guidance on friction ratings for friction measuring equipment. Consider improving the friction characteristics of existing runways when friction ratings are less than Good (Table 21-1). Do not groove helicopter runways.

21-2.1 Sawcut Grooving

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.

Table 21-1. Friction Ratings for Friction Measuring Equipment

Ground Vehicle Readings[1]										
Braking Action Level	**RCR**	**GripTester®**	**James Brake Index**	**MU-Meter**	**Surface Friction Tester**	**Runway Friction Tester**	**BV-11 Skiddo-Meter**	**Decel Meters**	**Locked Wheel Devices**	**ICAO[2] INDEX**
Good	>17	>0.49	.0.58	>0.50	>0.55	>0.51	>0.59	>0.53	>0.51	
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
N/L	≤5	≤0.14	≤0.17	≤0.14	≤0.16	≤0.15	≤0.19	≤0.16	≤0.15	

[1]Nominal Test Speed, 65 km/h (40 mph)
[2]International Civil Aviation Organization

21-2.1.1 **Pavement Condition**

Grooves should be applied only to structurally adequate pavement free from defects. Pavements requiring corrective action should be overlaid or rehabilitated prior to grooving. PFS should not be grooved.

21-2.1.2 **Grooving Flexible Pavements**

Studies indicate that grooving flexible pavements does not cause any appreciable deterioration of the pavement or increase maintenance efforts. In addition, 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.

21-2.1.3 **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 m (5 to 10 ft) of pavement edges to allow for the operation of the grooving equipment.

The standard groove configuration is 6 mm (¼ in)±2 mm (±1/16 in) in depth by 6 mm (¼ in)+2 mm–0 mm (+1/16 in–0 in) in width by 38 mm (1.5 in)–3 mm+0 mm (–1/8 in+0 in) center-to-center spacing. The recommended groove detail for airfield pavements is shown in Figure 21-1.

21-2.1.4 **Limitations**

Do not groove within 152 mm (6 in) of:

- Centerline joints
- Transverse joints or working cracks
- Through compression seals
- In-runway lighting fixtures or similar items,
- The first 3 m (10 ft) on either side of an arresting barrier cable that requires hook engagement for operation

There is no need for grooving on either side of barrier cables that are placed on overruns. Do not groove pavements until pavements are a minimum of 30 days old.

21-2.2 **PFS**

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 mm (0.75 to 1 in) thick. It has a coarse surface texture and is sufficiently porous to permit internal drainage as well as drainage along the surface. Ensure that existing pavements are in good condition before placing the mix.

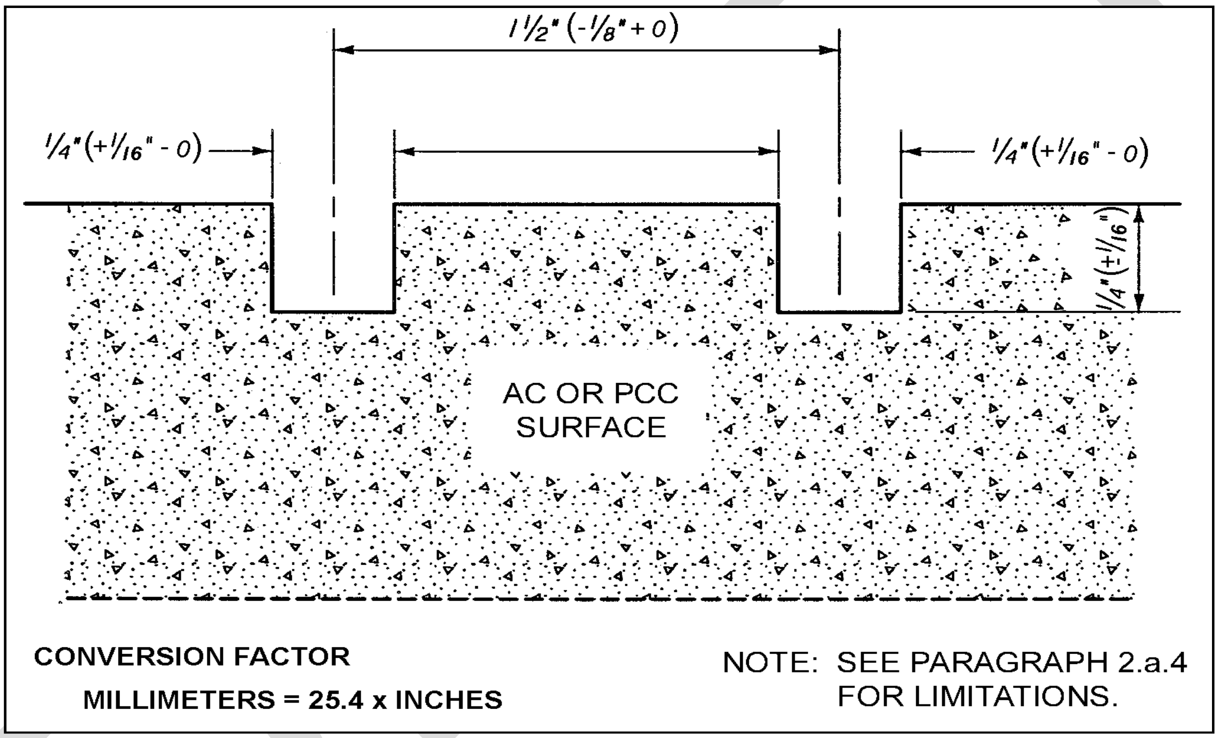
Concerns with PFS include rubber buildup that might prevent internal drainage, possible freezing of water trapped in voids, and loss of expertise in designing and constructing these surfaces.

Note: PFS should not be placed within 3 m (10 ft) of an arresting gear cable.

21-2.3 Retexturing

Retexturing of runways has been successfully accomplished using several types of equipment. Contact the MAJCOM pavements engineer for guidance on Air Force projects and USACE-TSC for Army projects.

Figure 21-1. Groove Configuration for Airfields



CHAPTER 22

DESIGN OF UNSURFACED AIRFIELDS, HELIPORTS, AND HELICOPTER SLIDE AREAS

22-1 OVERVIEW

This chapter presents the procedures for the design of aggregate-surfaced airfields, heliports, and helicopter slide areas.

22-2 SCOPE

This chapter presents criteria for determining the thickness, material, and compaction requirements for aggregate-surfaced heliports and helicopter slide areas at Air Force installations and Class I, II, and III airfields at Army installations. Use of the term “airfields” includes heliports, runways, taxiways, and parking aprons. This chapter presents design requirements for both frost and non-frost areas. Geometric criteria for these facilities are presented in UFC 3-260-01.

22-3 ARMY THICKNESS DESIGN OF AGGREGATE-SURFACED AIRFIELDS

The thickness design of aggregate-surfaced airfields is similar to the design of flexible pavement airfields. This procedure involves assigning a class to the airfield based on the aircraft controlling the design. Once the class of airfield has been selected, the design is accomplished using Figures 22-1 through 22-3.

22-3.1 Classes of Airfields

Six classes of Army airfields are defined in Chapter 2. These are Classes I through VI, although only Classes I through III and Class VI are considered candidates for aggregate surfacing. Each class of airfield is designed for a standard loading condition and pass level. Where necessary, airfields may be designed for loads and pass levels other than the standard, and the criteria in this chapter provide thicknesses for varying pass and load levels.

22-3.2 Traffic Areas

As described in Chapter 2, Army airfields are divided into traffic areas A through C for design purposes; however, for unsurfaced design, all areas are considered Type A traffic areas.

22-3.3 Thickness Criteria (Non-Frost Areas)

Thickness requirements for aggregate-surfaced airfields are determined from Figures 22-1 through 22-3 for Type A traffic areas. The minimum thickness requirement for all cases will be 152 mm (6 in). Enter the figure for the appropriate airfield class with the subgrade CBR to determine the thickness required for a given load and pass level. This thickness may be constructed of compacted granular fill for the total depth over the natural subgrade or in a layered system of granular fill and compacted subgrade for the same total depth. Check the layered section to ensure that the material is thick enough to protect the underlying layer based on the CBR of the underlying layer. The granular fill may consist of base and subbase material provided the top 152 mm (6 in) meet the gradation requirements of Table 22-1.

Figure 22-1. Army Design Curve for Army Class VI – Semi-prepared Landing Zone - C-130

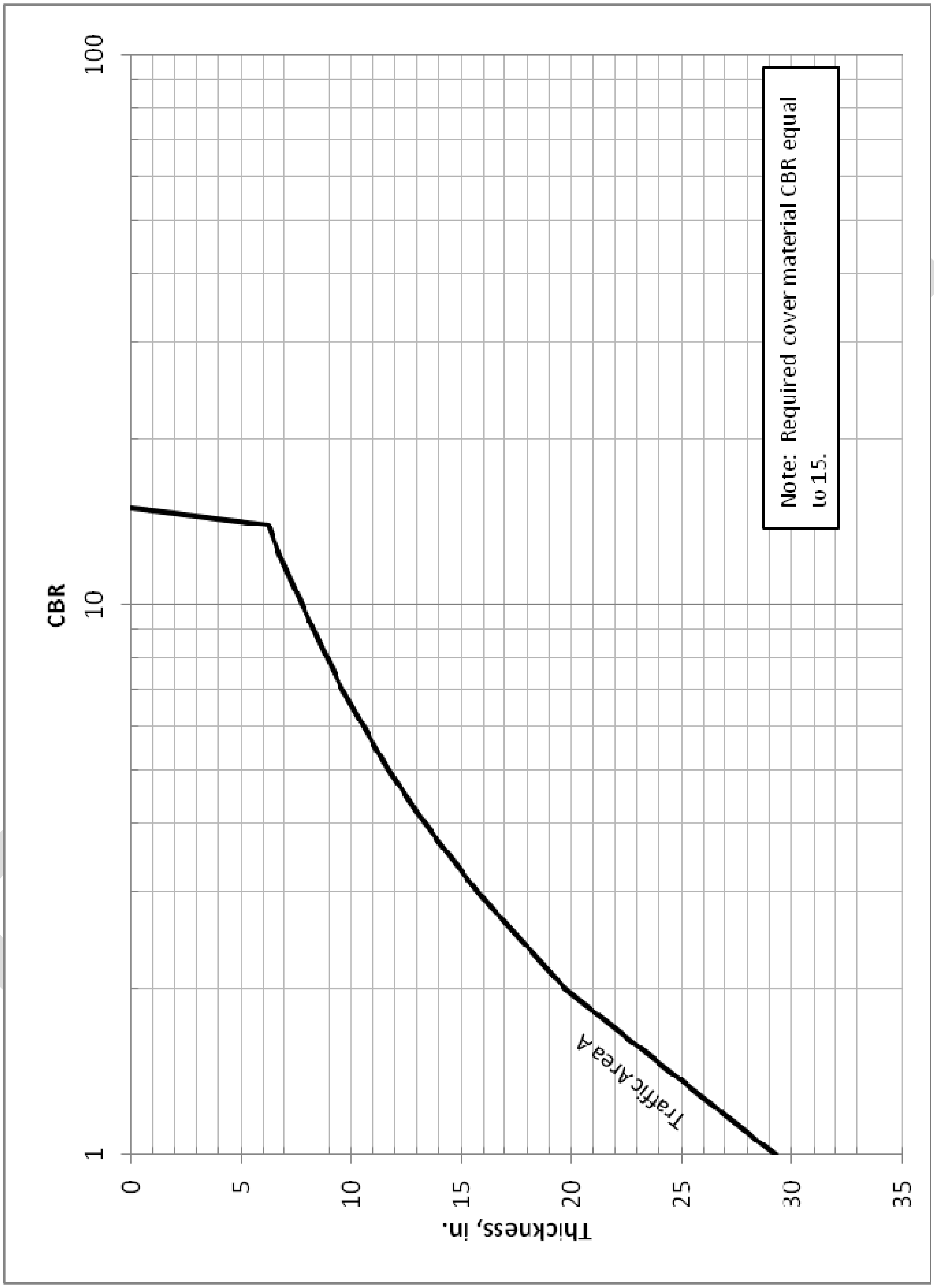


Figure 22-2. Army Design Curve for Army Class VI – Semi-prepared Landing Zone - C-17

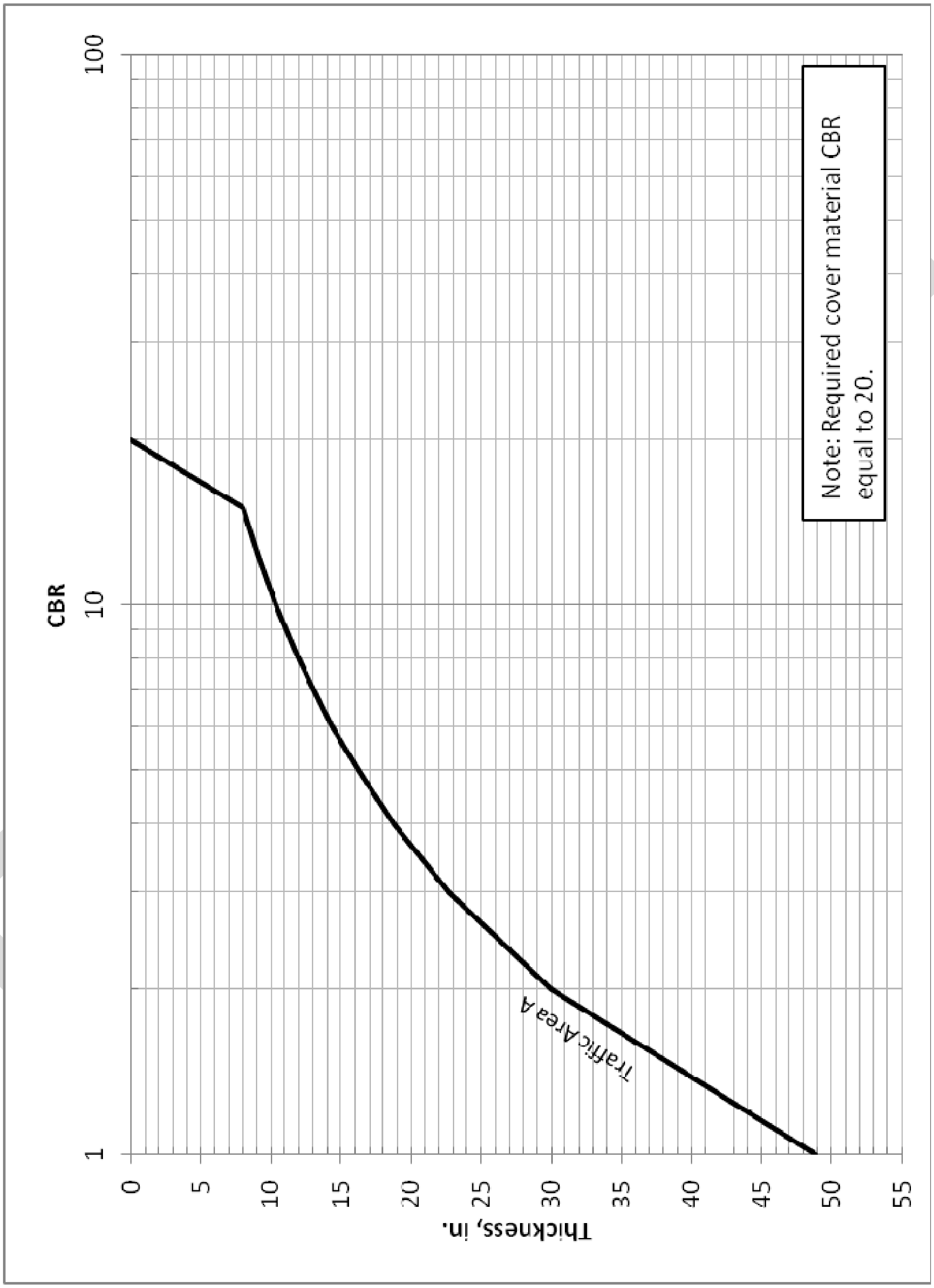


Figure 22-3. Army Design Curve for Air Force Landing Zone Unsurfaced - C-130

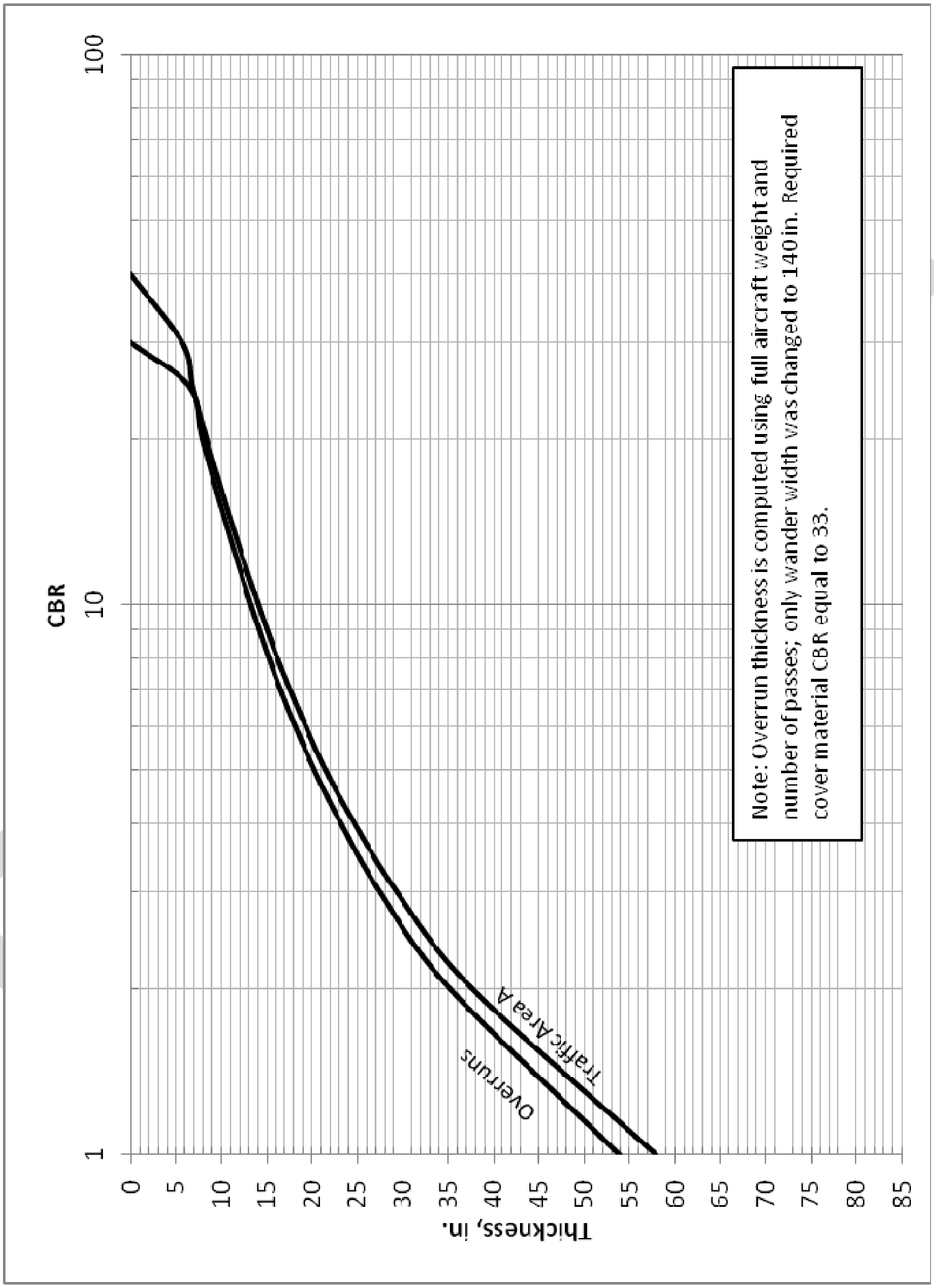
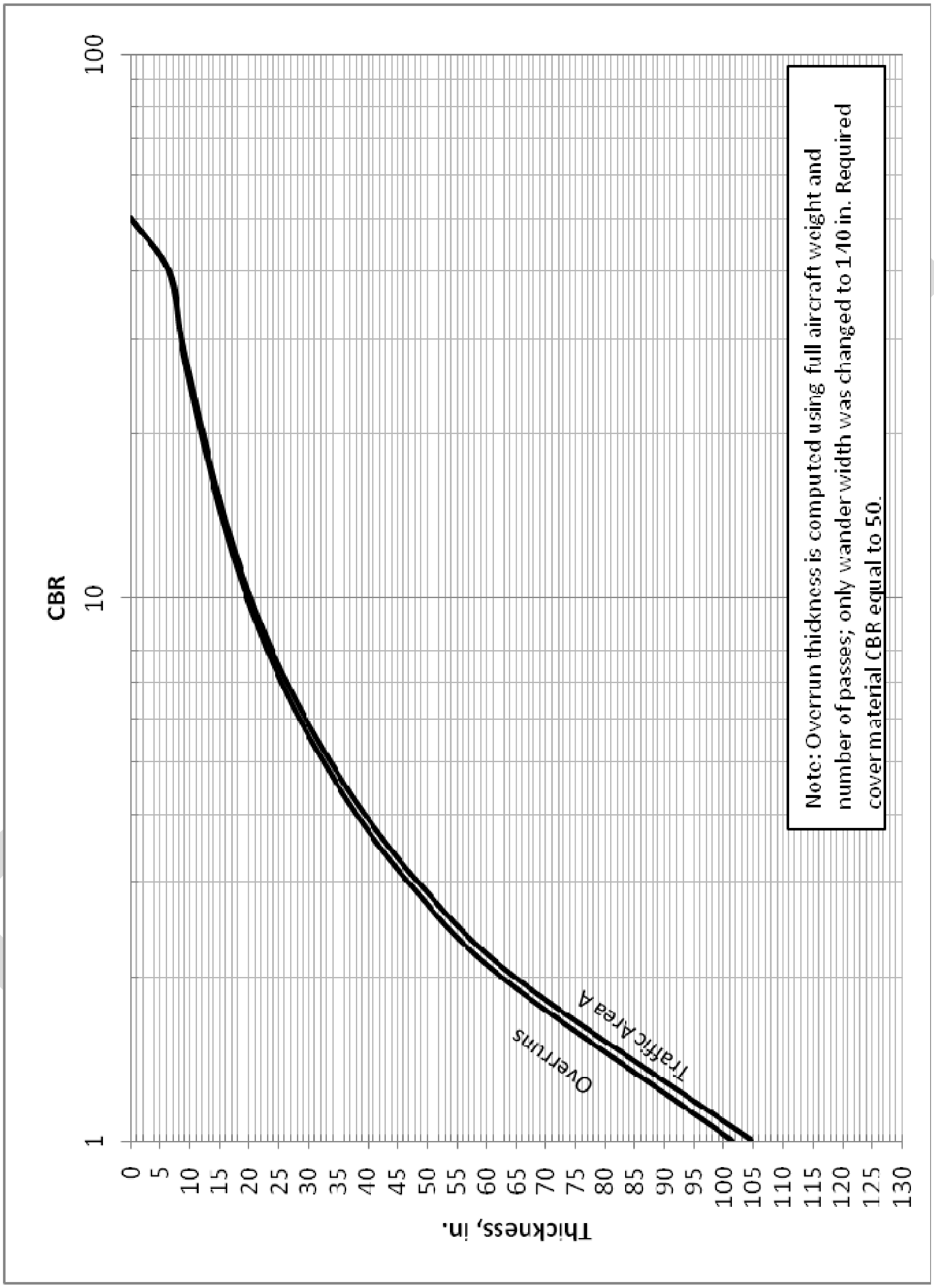


Figure 22-4. Army Design Curve for Air Force Landing Zone Unsurfaced - C-17



AIR FORCE THICKNESS DESIGN OF AGGREGATE-SURFACED HELIPORTS AND HELICOPTER SLIDE AREAS (NON-FROST AREAS)

Factors that determine thickness are the CBR of the subgrade, helicopter weight, and passes. The minimum required thickness is 152 mm (6 in). Use Figure 22-1 for the design of aggregate surface thickness for heliports. Enter Figure 22-1 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 that sufficient material protects the underlying layer based on the CBR of the underlying layer. The top 152 mm (6 in) must meet the gradation requirements of Table 22-1.

22-4 DESIGN CBR FOR SELECT MATERIALS AND SUBBASES

Select design CBR values for select materials and subbases in accordance with Chapter 7 except as modified in Table 22-2.

22-5 THICKNESS IN 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 22-3. Only the NFS group is suitable for base course. NFS, S1, or S2 soils may be used for subbase course, and any of the eight groups may be subgrade soils. Soils are listed in approximate order of decreasing bearing capability during periods of thaw.

22-5.1 Required Thickness

Where there are frost-susceptible subgrades, determine section thickness according to the reduced subgrade strength method. The reduced subgrade strength method uses the FASSI values in Table 22-4. Use FASSI values in the same way as CBR values. (The term “CBR” is not applied because FASSI values are weighted average values for an annual cycle and the values cannot be determined by CBR tests.) Enter Figure 22-1 through 22-3, as applicable, with the soil support indexes (not CBR values) to determine the required section thickness.

Table 22-1. Gradation for Aggregate Surface Courses (Percent Passing)

Sieve Designation	No. 1	No. 2	No. 3
25.0 mm (1 in)	100	100	100
9.5 mm (3/8 in)	60–100	100	100
4.76 mm (No. 4)	50–85	55–100	70–100
2.00 mm (No. 10)	40–70	40–100	55–100
0.42 mm (No. 40)	24–45	20–50	30–70
0.037 mm (No. 200)	8–15	8–15	8–15

Note: These gradations are not suitable for surfaces to support C-17 aircraft operations.

Table 22-2. Maximum Permissible Values for CBR and Gradation Requirements

Material	Maximum CBR	Maximum Size	Maximum Percent Passing		Maximum Liquid Limit*	Maximum Plasticity Index*
			#10	#200		
Subbase	50	50 mm (2 in)	50	15	25	5
Subbase	40	50 mm (2 in)	80	15	25	5
Subbase	30	50 mm (2 in)	100	15	25	5
Select Material	20	76 mm (3 in)	--	25	35	12
*ASTM D4318						

Table 22-3. Frost Design Soil Classification

Frost Group	Type Soil	Percentage Finer Than 0.02 mm by Weight	Unified Soil Classification Soil Types***
NFS*	(a) Gravels Crushed Stone Crushed rock (b) Sands	0–1.5 0–3	GW, GP SW, SP
PFS*	(a) Gravels Crushed Stone Crushed rock (b) Sands	1.5-3 3–10	GW, GP 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 (b) Sands	10–20 6–15	GM, GW-GM, GP-GM SM, SW-SM, SP-SM
F3	(a) Gravelly soils (b) Sands, except very fine silty sands (c) Clays, PI 12	over 20 over 15 --	GM, GC SM, SC CL, CH
F4	(a) Gravelly soils (b) Sands, except very fine silty sands (c) Clays, PI 12 (d) Verved clays and other fine grained banded sediments	-- over 15 -- --	ML, MH SM CL, CL-ML CL, ML, SM and CH

* Non-frost-susceptible.
** Possible frost-susceptible, but requires laboratory test to determine frost design soil classification.
*** Defined in AFJMAN 32-1034.

Table 22-4. FASSI of Subgrade Soils

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

22-5.2 Pavement Section Layers

When frost is a consideration, the recommended practice is to create the pavement section using layers that will ensure the stability of the system, particularly during thaw periods. The layered system may consist of a 152-mm-thick (6-in-thick) minimum wearing surface of fine crushed stone, a coarse-graded base course, and a well-graded subbase of sand or gravelly 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 m (1 ft) on each side.

22-5.3 Wearing Surface

The wearing surface contains fines (material passing the No. 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.

22-5.4 Base Course

The coarse-graded base course is important in providing drainage of the granular fill. The base course should be NFS to retain strength during spring thaw periods.

22-5.5 Subbase

A well-graded subbase provides additional bearing capacity over the frost-susceptible subgrade. Such a subbase also provides a filter layer between the coarse-graded 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 NFS or of low frost susceptibility (S1 or S2). The filter layer may or may not be necessary depending on 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 a geotextile is being used, the sand subbase or filter layer may be omitted because the fabric will be placed directly on the subgrade and will act as a filter.

22-5.6 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 FASSI; 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.

22-5.7 **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.

22-5.8 **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, the 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.

22-6 **SURFACE COURSE**

Materials requirements for construction of aggregate-surfaced airfields depend on whether frost is a factor in the design.

22-6.1 **Non-Frost Areas**

Material used for airfields should be sufficiently cohesive to resist abrasive action. This material should have a LL no greater than 35 and a PI between 4 and 9. The material should also 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 22-1. 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.

22-6.2 **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 2 and 3 (Table 22-1) with caution because they may be unstable in a freeze-thaw environment.

22-7 **COMPACTION REQUIREMENTS**

Compaction requirements for the subgrade and granular layers are expressed as a percent of maximum CE 55 density as determined by using CRD-C 653. For granular layers, compact the material to 100 percent of maximum CE 55 density. Select materials and subgrades in fills must have densities equal to or greater than the values in Table 22-5, 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 in Table 22-5. Subgrades occurring in cut sections will be either compacted from the surface to meet the densities shown in Table 22-5, 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 22-5 are measured from the surface of the aggregate, not the surface of the subgrade.

Table 22-5. Compaction Requirements for Army Airfields and Air Force Heliports and Helicopter Slide Areas

Airfield Type	Traffic Area	Depth Below Pavement Surface, millimeters (inches)									
		Cohesive Soils, percent					Cohesionless Soils, percent				
		100	95	90	85	80	105	100	95	90	85
Army Class I	A	102 (4)	152 (6)	203 (8)	254 (10)	305 (12)	50 (2)	152 (6)	254 (10)	330 (13)	406 (16)
Army Class II	A	127 (5)	203 (8)	305 (12)	381 (15)	483 (19)	50 (2)	203 (8)	356 (14)	533 (21)	660 (26)
Army Class III	A	225 (9)	381 (15)	533 (21)	686 (27)	864 (34)	102 (4)	381 (15)	635 (25)	940 (37)	1219 (48)
Air Force	N/A	102 (4)	152 (6)	203 (8)	254 (10)	305 (12)	N/A	152 (6)	254 (10)	330 (13)	406 (16)

22-8 DRAINAGE

Drainage is a critical factor in aggregate-surfaced airfield design, construction, and maintenance. Drainage should be considered prior to construction and, when necessary, should serve as a basis for site selection. Design of surface drainage shall be in accordance with Chapter 23 of this UFC.

22-9.1 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 that 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.

22-9.2 The surface geometry of an airfield should be designed so that drainage is provided at all points. Depending on the surrounding terrain, surface drainage can be achieved by a continual cross slope or by a series of two or more interconnecting cross slopes.

22-9.3 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 necessary to prevent water from other areas from flowing across the airfield.

22-9 MAINTENANCE

Aggregate-surfaced areas that require frequent maintenance suffer from two primary causes of deterioration: the environment and traffic. Rain or water flow will wash fines from the aggregate surface, and traffic action causes erosion of surface materials. Maintenance should be performed at least every 6 months and more frequently if required. The 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 the original thickness, and the wearing surface recompacted to the specified density.

22-10 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 the 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 mm (0.5 in) will usually destroy any thin layer or shallow-depth penetration dust palliative treatment. A wide selection of dust control materials is available. Several materials have been recommended for use and are discussed in UFC 3-250-11 and UFC 3-260-17.

22-11 DESIGN EXAMPLES

Design examples are contained in Appendix B, Section 14.

CHAPTER 23

DESIGN OF SUBSURFACE PAVEMENT DRAINAGE SYSTEMS

23-1 INTRODUCTION

23-1.1 Purpose
This chapter provides guidance for the design and construction of subsurface drainage facilities for airfield runways, taxiways, and aprons.

23-1.2 Scope
The criteria within this chapter apply to paved runways, taxiways, and aprons of Air Force, Army, and Navy airfields. The criteria is limited to situations in which the water can be drained from the pavement structure by gravity flow and is mainly concerned with elimination of water that enters the pavement through the surface.

23-1.3 Definitions
Several terms in this chapter have a unique usage within the chapter or may not be in common usage. Paragraphs 23-1.3.1 through 23-1.3.16 define these terms.

23-1.3.1 Apparent Opening Size (AOS)
The AOS is a measure of the opening size of a geotextile. AOS is the sieve number corresponding to the sieve size at which 95 percent of the single-size glass beads pass the geotextile O_{95} when tested in accordance with ASTM D4751.

23-1.3.2 Coefficient of Permeability *k*
The coefficient of permeability is a measure of the rate at which water passes through a unit area of material in a given amount of time under a unit hydraulic gradient.

23-1.3.3 Choke Stone
A choke stone is a small-size stone used to stabilize the surface of an open-graded material (OGM). For a choke stone to be effective, the ratio of d_{15} of the coarse aggregate to the d_{15} of the choke stone must be less than 5, and the ratio of the d_{50} of the coarse aggregate to the d_{50} of the choke stone must be greater than 2.

23-1.3.4 Drainage Layer
A drainage layer is a layer in the pavement structure that is specifically designed to allow rapid horizontal drainage of water from the pavement structure. The drainage layer is also considered to be a structural component of the pavement and may serve as part of the base or subbase.

23-1.3.5 Effective Porosity
The effective porosity is defined as the ratio of the volume of voids that will drain under the influence of gravity to the total volume of a unit of aggregate. The difference between the porosity and the effective porosity is the amount of water that will be held by the aggregate. For materials such as rapid-draining material (RDM) and OGM, the water held by the aggregate will be small; thus, the difference between the porosity and

effective porosity will be small (less than 10 percent). The effective porosity may be estimated by computing the porosity from the unit dry weight of the aggregate and the specific gravity of the solids, which then should be reduced by 5 percent to allow for water retention in the aggregate.

23-1.3.6 **Geocomposite Edge Drain**
A geocomposite edge drain is a manufactured product using geotextiles, geogrids, geonets, or geomembranes in laminated or composite form, which can be used as an edge drain in place of trench-pipe construction.

23-1.3.7 **Geotextile**
A geotextile is a permeable textile used in geotechnical projects. For this UFC, geotextile will refer to a nonwoven needle punch fabric that meets the requirements of the AOS, grab strength, and puncture strength specified for the particular application.

23-1.3.8 **Hazen's Effective Particle Diameter**
The Hazen's effective particle diameter is the particle size, in millimeters, that corresponds to 10 passing on the grain-size distribution curve. This parameter is one of the major parameters in determining the permeability of a soil.

23-1.3.9 **OGM**
An OGM is a granular material with a very high permeability (greater than 1,500 m/day [5,000 ft/day]) that may be used for a drainage layer. Such a material will normally require stabilization for construction stability or for structural strength to serve as a base in a flexible pavement.

23-1.3.10 **Pavement Structure**
Pavement structure is the combination of subbase, base, and surface layers constructed on a subgrade.

23-1.3.11 **Permeable Base**
A permeable base is an open-graded, granular material with most of the fines removed (for example, less than 10 percent passing the No. 16 sieve) to provide high permeability (305 m/day [1,000 ft/day] or more) for use in a drainage layer.

23-1.3.12 **Porosity**
Porosity refers to the volume of voids in a material and is expressed as the ratio of the volume of voids to the total volume.

23-1.3.13 **RDM**
A granular material having a sufficiently high permeability (300 to 1,500 m/day [1,000 to 5,000 ft/day]) to serve as a drainage layer and also having the stability to support construction equipment and the structural strength to serve as a base or a subbase or both.

23-1.3.14 **Separation Layer**

A separation layer is a layer provided directly beneath the drainage layer to prevent fines from infiltration or pumping into the drainage layer and to provide a working platform for construction and compaction of the drainage layer.

23-1.3.15 **Stabilization**

Stabilization refers to either mechanically or chemically stabilizing the drainage layer to increase the stability and strength to withstand construction traffic and design traffic. Mechanical stabilization is accomplished by the use of a choke stone and compaction. Chemical stabilization is accomplished by the use of either portland cement or asphalt.

23-1.3.16 **Subsurface Drainage**

Subsurface drainage refers to the process of collecting and removing water from the pavement structure. Subsurface drainage systems are categorized by function: those that drain surface infiltration water and those that control groundwater.

23-1.4 **Bibliography**

In recent years, subsurface drainage has received increasing attention, particularly in the area of highway design. A number of studies have been conducted by state highway agencies and by the Federal Highway Administration (FHWA) that have resulted in a large number of publications on the subject of subsurface drainage. Appendix A provides a list of publications that contain information pertaining to the design of subsurface drainage for pavements.

23-1.5 **Effects of Subsurface Water**

Water has a detrimental effect on pavement performance, primarily by either weakening subsurface materials or eroding material by free water movement. For flexible pavements, the weakening of the base, subbase, or subgrade when saturated with water is one of the main causes of pavement failures. In rigid pavement, free water, trapped between the concrete surface and an impermeable layer directly beneath the concrete, moves due to pressure caused by loadings. This movement of water, which is referred to as pumping, erodes the subsurface material, creating voids under the concrete surface. In frost areas, subsurface water will contribute to frost damage by heaving during freezing and loss of subgrade support during thawing. Poor subsurface drainage can also contribute to secondary damage such as “D” cracking or swelling of subsurface materials.

23-1.6 **Traffic Effects**

The type, speed, and volume of traffic will influence the criteria used in the design of pavement drainage systems. For rigid pavements, pumping is greatly increased as the volume and speed of the traffic increases. For flexible pavements, the buildup of pore pressures as a result of high-volume, high-speed traffic is a primary cause of the weakening of the pavement structure. For these reasons, the criteria for a subsurface under airfield runways and taxiways will be more stringent than for airfield parking aprons or other pavements that have low-volume and low-speed traffic.

23-1.7 **Sources of Water**

The two types of water to be considered are water from infiltration and subterranean water. Infiltration is the most important source of water and is the source of most concern in this document. Subterranean water is important in frost areas and areas of very high water table or areas of artesian water. In many areas, perched water may develop under pavements due to a reduced rate of evaporation of the water from the surface. In frost areas, free water collects under the surface by freeze-thaw action.

23-1.7.1 **Infiltration**

Infiltration is surface water that enters the pavement from the surface through cracks or joints in the pavement, through the joint between the pavement and shoulder, through pores in the pavement, and through shoulders and adjacent areas. Since surface infiltration is the principal source of water, it is the source requiring greatest control measures. Groundwater tables rise and fall depending on the relation between infiltration, absorption, evaporation, and groundwater flow. Seasonal fluctuations are normal because of differences in the amount of precipitation and may be relatively large in some localities. Prolonged drought or wet periods will cause large fluctuations in the groundwater level.

23-1.7.2 **Subterranean Water**

Subterranean water can be a source of water from a high water table, capillary forces, artesian pressure, and freeze-thaw action. This source of water is particularly important in areas of frost action, where large volumes of water can be drawn into the pavement structure during the formation of ice lenses. For large paved areas, the evaporation from the surface is greatly reduced, which causes saturation of the pavement structure by capillary forces. Also, if impervious layers exist beneath the pavement, perched water can be present or develop from water entering the pavement through infiltration. This perched water then becomes a subterranean source of water. In general, the presence of near surface subterranean water must be identified during soil exploration, and drainage facilities must be designed to mitigate the influence of such water.

23-1.7.3 **Freeze-Thaw**

Freeze-thaw action can result in large amounts of water being drawn into the pavement structure. In freeze-thaw conditions, water flows to the freeze front by capillary action. Repeated cycles of freeze-thaw result in the growth of ice lenses that can cause heave in the pavement structure. It is not uncommon to note heaves in soils as great as 60 percent; under laboratory conditions, heaves of as much as 300 percent have been recorded. The formation of ice lenses in the pavement structure has two very detrimental effects on the pavement. One effect is that the formation of ice lenses causes a loss of density of the pavement materials, resulting in strength loss. A second effect is that thawing of the ice results in a large volume of free water that must be drained from the pavement. Because thawing usually occurs simultaneously from both the top and bottom of the pavement structure, the free water can be trapped within the pavement structure. Providing adequate drainage will minimize pumping and promote the restoration of pavement strength. In the design of subdrain systems in frost areas, free water in both the upper and lower sections of the pavement must be considered.

23-1.7.4 **Classification of Subdrain Facilities**

Subdrain facilities can be categorized into two functional categories: those that control infiltration and those that control groundwater. An infiltration control system is designed to intercept and remove water that enters the pavement from precipitation or surface flow. An important function of this system is to keep water from being trapped between impermeable layers. A groundwater control system is designed to reduce water movement into subgrades and pavement sections by controlling the flow of groundwater or by lowering the water table. Often, subdrains are required to perform both functions, and the two subdrain functions can be combined into a single subdrain system. Figures 23-1 and 23-2 illustrate examples of infiltration and groundwater control systems, respectively.

23-1.8 **Subsurface Drainage Requirements**

Determining the subsurface soil properties and water conditions is a prerequisite for the satisfactory design of a subsurface drainage system. Field explorations and borings made in connection with the project design should include certain investigations pertinent to subsurface drainage. A topographic map of the proposed area and the surrounding vicinity should be prepared, and the map should indicate all streams, ditches, wells, and natural reservoirs. Analyzing aerial photographs of the areas selected for construction may furnish valuable information on general soil and groundwater conditions. An aerial photograph presents a graphic record of the extent, boundaries, and surface features of soil patterns occurring at the surface of the ground. The presence of vegetation, the slopes of a valley, the colorless monotony of sand plains, the farming patterns, the drainage pattern, gullies, eroded lands, and evidences of human works are revealed in detail by aerial photographs. The use of aerial photographs may supplement both the detail and knowledge gained in topographic survey and ground explorations. The sampling and exploratory work can be made more rapid and effective after an analysis of aerial photographs has developed the general soil features.

The location and depth of permanent and perched groundwater tables may be sufficiently shallow to influence the design. The season of the year and rainfall cycle will measurably affect the depth to the water table. In many locations, information may be obtained from residents of the surrounding areas regarding the behavior of wells and springs and other evidences of subsurface water. The soil properties investigated for other purposes in connection with the design will supply information that can be used for the design of the drainage system. It may be necessary to supplement these explorations at locations of subsurface drainage structures and in areas where soil information is incomplete for designing of the drainage system.

Figure 23-1. Collector Drain to Remove Infiltration Water

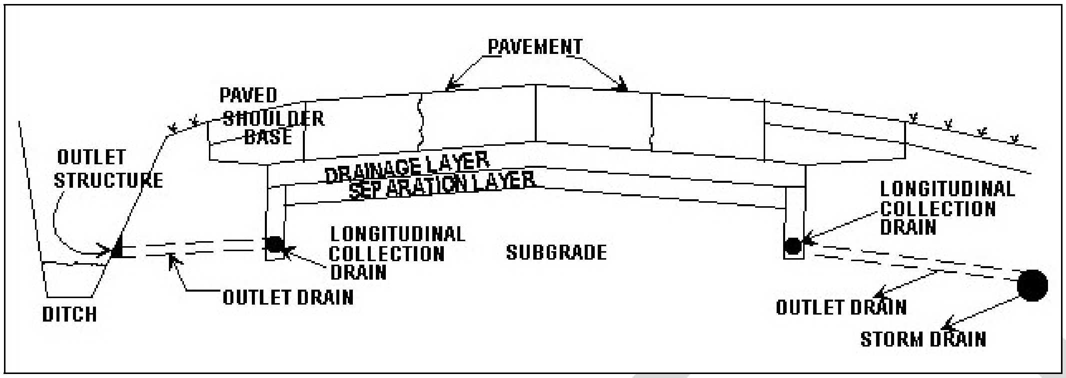
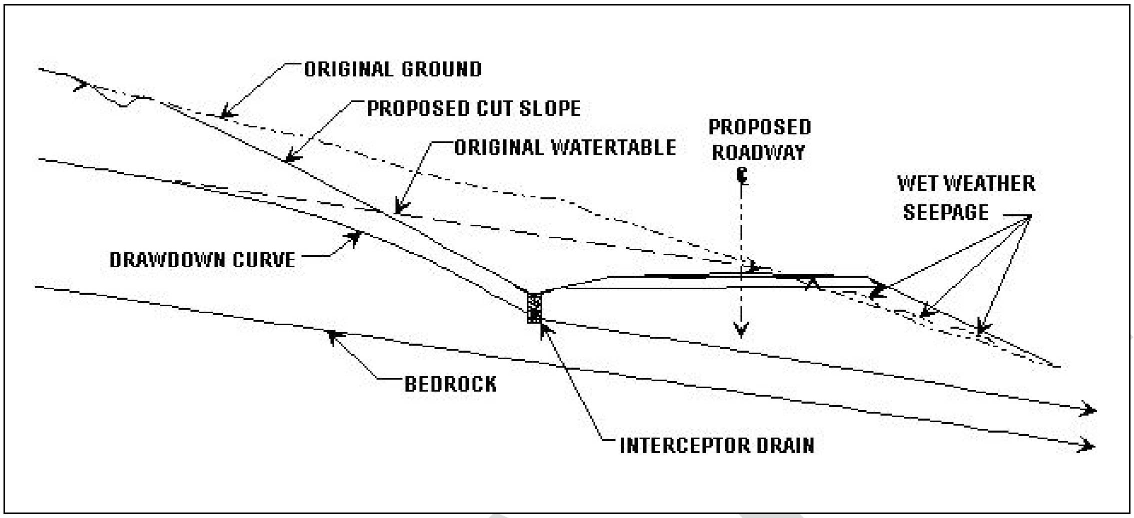


Figure 23-2. Collector Drain to Intercept Seepage and Lower the Groundwater Table



23-1.9 **Laboratory Tests**

The design of subsurface drainage structures requires knowledge of these soil properties: strength, compressibility, swell and dispersion characteristics, the in situ and compacted unit dry weights, the coefficient of permeability, the in situ water content, specific gravity, grain-size distribution, and the effective void ratio. These soil properties may be satisfactorily determined by experienced soil technicians through laboratory tests. The final selected soil properties for design purposes may be expressed as a range, one extreme representing a maximum value and the other a minimum value. The true value should be between these two extremes, but it may approach or equal one or the other depending on the variation within a soil stratum.

23-1.10 **Drainage of Water from Soil**

The quantity of water removed by a drain will vary depending on the type of soil and location of the drain with respect to the groundwater table. All of the water contained in a given specimen cannot be removed by gravity flow because water retained as thin films adhering to the soil particles and held in the voids by capillarity will not drain. Consequently, to determine the volume of water that can be removed from a soil in a given time, the effective porosity as well as the permeability must be known. Limited effective porosity test data for well-graded base course materials, such as bank-run sands and gravels, indicate a value for effective porosity of not more than 0.15. Uniformly graded soils such as medium coarse sands may have an effective porosity of not more than 0.25. Open-graded aggregate used for drainage layers will have an effective porosity of between 0.25 and 0.35.

23-2 PRINCIPLES OF PAVEMENT DRAINAGE

23-2.1 Flow of Water through Soils

The flow of water through soils is expressed by Darcy's empirical law, which states that the velocity of flow *v* is directly proportional to the hydraulic gradient *i*. This law can be expressed as:

$$v = k \cdot i \tag{23-1}$$

where *k* is the coefficient of proportionality known as the coefficient of permeability. Equation 23-1 can be expanded to obtain the rate of flow through an area of soil *A*. The equation for the rate of flow *Q* is:

$$Q = k \cdot i \cdot A \tag{23-2}$$

According to Darcy's law, the velocity of flow and the quantity of discharge through a porous medium are directly proportional to the hydraulic gradient. For this condition to be true, flow must be laminar or non-turbulent. Investigations have indicated that Darcy's law is valid for a wide range of soils and hydraulic gradients; however, in developing criteria for subsurface drainage, liberal margins have been applied to allow for turbulent flow. The criteria and uncertainty depend heavily on the permeability of the soils in the pavement structure. It is therefore useful to examine the influence of various factors on the permeability of soils. In examining the permeability of soils in regard to pavement drainage, the materials of most concern are base and subbase aggregate and aggregate used as drainage layers.

23-2.2 Factors Affecting Permeability

23-2.2.1 Coefficient of Permeability

The value of permeability depends primarily on the characteristics of the permeable materials, but it is also a function of the properties of the fluid. An equation (after Taylor) demonstrating the influence of the soil and pore fluid properties on permeability was developed based on flow through porous media similar to flow through a bundle of capillary tubes. This equation is given here as Equation 23-3:

$$k = D_s^2 \cdot C \cdot \left(\frac{\gamma \cdot e^3}{\mu \cdot (1-e)} \right) \tag{23-3}$$

where

k = the coefficient of permeability

D_s = Hazen's effective particle diameter

C = shape factor

γ = unit weight of pore fluid

μ = viscosity of pore fluid

e = void ratio

23-2.2.2 **Effect of Pore Fluid and Temperature**

In the design of subsurface drainage systems for pavements, the primary pore fluid of concern is water; therefore, when permeability is mentioned in this chapter, water is assumed to be the pore fluid. Equation 23-3 indicates that the permeability is directly proportional to the unit weight of water and inversely proportional to the viscosity. The unit weight of water is essentially constant, but the viscosity of water will vary with temperature. Over the widest range of temperatures ordinarily encountered in seepage problems, viscosity varies about 100 percent. Although this variation seems large, it can be insignificant when considered in the context of the variations that can occur with changes in material properties.

23-2.2.3 **Effect of Grain Size and Void Ratio**

It is logical that the smaller the grain size, the smaller the voids that constitute the flow channels, and hence, the lower the permeability. Equation 23-3 suggests that permeability varies with the square of the effective particle diameter and the cube of the void ratio. Since for the most part the void ratio is a function of the material gradation, the influence of effective particle diameter will be magnified. Consider that according to Equation 23-3, when the effective particle size increases from 0.075 mm (No. 200) to 1.18 mm (No. 16), the permeability would increase by a factor of approximately 250. Assuming the increase in effective particle size would result in an increase in the void ratio by a minimum of 2 times, the permeability due to the increase in void ratio would be by a factor of 8. Thus the total increase in permeability due to the increase in the effective particle size and increase in void ratio would be by a factor of approximately 2000.

23-2.2.3.1 Also, the shape of the void spaces has a marked influence on the permeability. As a consequence, the relationships between grain size, void ratio, and permeability are complex. Intuition and experimental test data suggest that the finer particles in a soil have the most influence on permeability. The coefficient of permeability of sand and gravel materials, graded between limits usually specified for pavement bases and subbases, depends principally on the percentage by weight of particles passing the 0.075 mm (No. 200) sieve. Table 23-1 provides estimates of the permeability for these materials for various amounts of material finer than the 0.075 mm (No. 200) sieve.

23-2.2.3.2 Figure 23-3 presents the permeability for different soils as a function of the void ratio. The amount of water that can be contained in a soil will directly relate to the void ratio. Not all water contained in a soil can be drained by gravity flow because water retained as thin films adhering to the soil particles and held by capillarity will not drain. Consequently, to determine the volume of water that can be removed from a soil, the effective porosity n_e must be known. The effective porosity is defined as the ratio of the volume of the voids that can be drained under gravity flow to the total volume of soil, and can be expressed mathematically as:

$$n_e = 1 - \frac{\gamma_d}{G_S \cdot \gamma_W}\left(1 + G_S \cdot W_e\right) \quad (23\text{-}4)$$

where

γ_d = dry density of the soil

G_s = specific gravity of solids

γ_w = unit weight of water

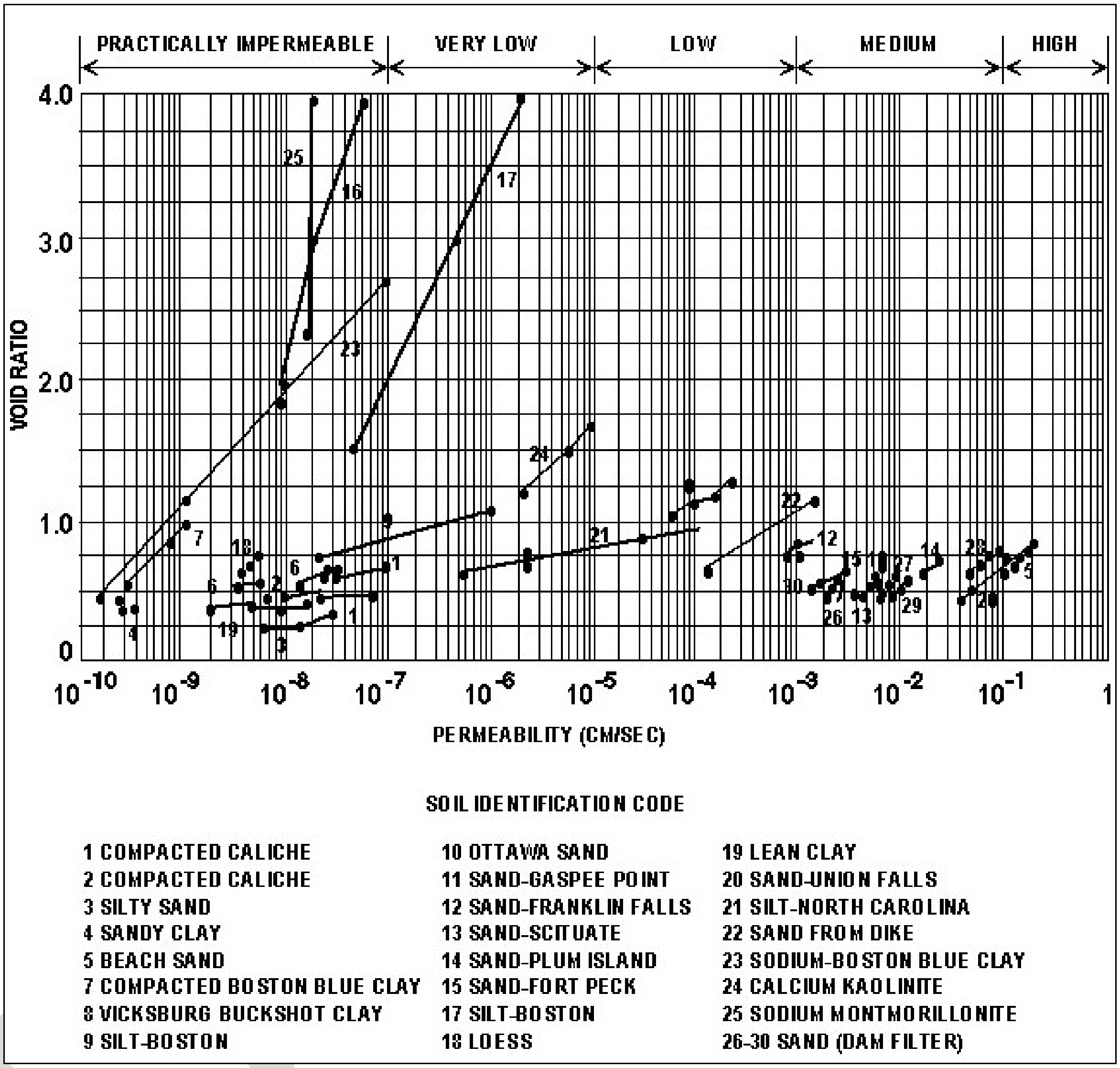
W_e = effective water content (after the soil has drained) expressed as a decimal fraction relative to dry weight

23-2.2.3.3 Limited effective porosity test data for well-graded base course materials, such as bank-run sands and gravels indicate a value for effective porosity of not more than 0.15. Uniformly graded medium or coarse sands may have an effective porosity of not more than 0.25, while for a uniformly graded aggregate such as would be used in a drainage layer, the effective porosity may be above 0.25.

Table 23-1. Coefficient of Permeability for Sand and Gravel Materials (Coefficient of 55)

Percent by Weight Passing 0.075 mm (No. 200) Sieve	Permeability for Remolded Samples	
	mm/sec	ft/min
3	5×10^{-1}	10^{-1}
5	5×10^{-2}	10^{-2}
10	5×10^{-3}	10^{-3}
15	5×10^{-4}	10^{-4}
20	5×10^{-5}	10^{-5}

Figure 23-3. Permeability Test Data (from Lambe and Whitman, with permission)



23-2.2.4 Effect of Structure and Stratification

Generally, in situ soils show a certain amount of stratification or a heterogeneous structure. Water-deposited soils usually exhibit a series of horizontal layers that vary in grain size distribution and permeability, and generally these deposits are more permeable in the horizontal than in the vertical direction. In pavement construction, the subgrade, subbase, and base materials are placed and compacted in horizontal layers, which results in having a different permeability in the vertical direction than in the horizontal direction. The vertical drainage of water from a pavement can be disrupted by a single relatively impermeable layer. For most pavements, the subgrades have a very low permeability compared to the base and subbase materials; therefore, water in the pavement structure can be removed best by horizontal flow. For a layered pavement system, the effective horizontal permeability is obtained from a weighted average of the layer permeability by the formula:

$$k = \frac{(k_1 \cdot d_1 + k_{2.} \cdot d_2 + k_3 \cdot d_3 + ...)}{(d_1 + d_2 + d_3 + ...)} \quad (23\text{-}5)$$

where

k = the effective horizontal permeability

$k_1, k_2, k_3...$ = the coefficients of horizontal permeability of individual layers

$d_1, d_2, d_3...$ = the thicknesses of the individual layers

When a drainage layer is employed in the pavement section, the permeability of the drainage material will likely be several orders of magnitude greater than that of the other materials in the section. Since water flow is proportional to permeability, the flow of water from the pavement section can be computed based only on the characteristics of the drainage layer.

23-2.3 **Quantity and Rate of Subsurface Flow**
Water flowing from the pavement section may come from infiltration through the pavement surface and groundwater. Normally, groundwater flows into collector drains from the subgrade and will be an insignificant flow compared to the flow coming from infiltration. The computation of groundwater flow is beyond the scope of this manual; therefore, should it be necessary to compute the groundwater flow, consult a textbook on groundwater flow. The volume of infiltration water flow from the pavement will depend on factors such as the type and condition of the surface, the length and intensity of rainfall, the properties of the drainage layer, the hydraulic gradient, the time allowed for drainage, and the drained area. In the design of the subsurface drainage system, all of these factors must be considered.

23-2.3.1 **Effects of Pavement Surface**
The type and condition of the pavement surface will have considerable influence on the volume of water entering the pavement structure. In the design of surface drainage facilities, all rain falling on paved surfaces is assumed to be runoff. For new well designed and constructed pavements, the assumption of 100 percent runoff is probably a good, conservative assumption for the design of surface drainage facilities. For design of the subsurface drainage facilities, the design should be based on the infiltration rate for a deteriorated pavement. Studies have shown that for badly deteriorated pavements, well over 50 percent of the rainfall can flow through the pavement surface. For well maintained pavements, the infiltration rate will be greatly reduced such that the runoff will approach 100 percent.

23-2.3.2 **Effects of Rainfall**
It is only logical that the volume of water entering the pavement will be directly proportional to the intensity and length of the rainfall. Relatively low-intensity rainfalls

can be used for designing the subsurface drainage facilities because high-intensity rainfalls do not greatly increase the adverse effect of water on pavement performance. The excess rainfall would, once the base and subbase were saturated, run off as surface drainage. For this reason, a seemingly nonconservative design rainfall can be selected.

23-2.3.3 **Capacity of Drainage Layers**

If water enters the pavement structure at a greater rate than the discharge rate, the pavement structure becomes saturated. The design of horizontal drainage layers for the pavement structure is based, in part, on the drainage layer serving as a reservoir for the excess water entering the pavement. The capacity of the drainage layer as a reservoir is a function of the storage capacity of the drainage layer plus the amount of water that drains from the layer during a rain event. The storage capacity of the drainage layer will be a function of the effective porosity of the drainage material and the thickness of the drainage layer. The storage capacity of the drainage layer q_s in terms of depth of water per unit area is computed by Equation 23-6:

$$q_s = n_e \cdot h \qquad (23\text{-}6)$$

where

n_e = the effective porosity

h = the thickness of the drainage layer

In the equation, the dimensions of q_s will be the same as the dimensions of h. If it is assumed that not all the water will be drained from the drainage layer, then the storage capacity will be reduced by the amount of water in the layer at the start of the rain event. The criterion for design of the drainage layer calls for 85 percent of the water to be drained from the drainage layer within 24 hours; therefore, it is conservatively assumed that only 85 percent of the storage volume will be available at the beginning of a rain event.

To account for the possibility of water in the layer at the beginning of a rain event, Equation 23-6 is modified to be:

$$q_s = 0.85 \cdot n_e \cdot h \qquad (23\text{-}7)$$

23-2.3.3.1 The amount of water q_d that will drain from the drainage layer during the rain event may be estimated using Equation 23-8:

$$q_d = \frac{t \cdot k \cdot i \cdot h}{2 \cdot L} \qquad (23\text{-}8)$$

where

t = duration of the rain event

L = length of the drain path

k = permeability of the drainage layer

i = slope of the drainage layer

h = thickness of the drainage layer

23-2.3.3.2 In these equations, the dimensions of q_s, q_d, t, k, h, and L should be consistent. The total capacity q of the drainage layer will be the sum of q_s and q_d, resulting in this equation for the capacity:

$$q = (0.85 \cdot n_e \cdot h) + \left(\frac{t \cdot k \cdot i \cdot h}{2 \cdot L}\right) \quad (23\text{-}9)$$

23-2.3.3.3 If the amount of water entering the pavement is known, Equation 23-9 can be used to estimate the thickness of the drainage layer such that the drainage layer will have the capacity for a given design rain event. For most situations, the amount of water draining from the drainage layer will be small compared to the storage capacity; therefore, in most cases, Equation 23-7 can be used in estimating the thickness required for the drainage layer.

23-2.3.4 Time for Drainage

The water should be drained from the base and subbase layers as rapidly as possible. The time for drainage of these layers is a function of the effective porosity, the length of the drainage path, the thickness of the layers, the slope of the drainage path, and the permeability of the layers. Past criterion has specified that the base and subbase obtain 50 percent drainage within 10 days. The equation for computing the time for 50 percent drainage is:

$$T_{50} = \frac{\left(n_e \cdot D^2\right)}{\left(2 \cdot k \cdot H_o\right)} \quad (23\text{-}10)$$

where

T_{50} = time for 50 percent drainage

n_e = effective porosity of the soil

k = coefficient of permeability

D, H_o, and H = base and subbase geometry dimensions (illustrated in Figure 23-4)

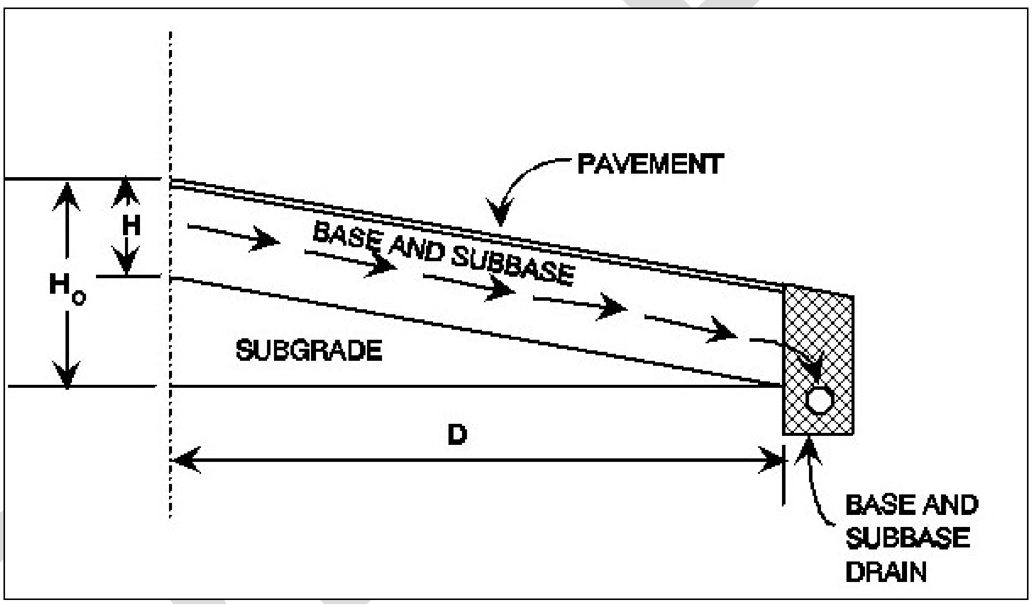
The dimensions of time k, H_o, H, and D must be consistent. If in Figure 23-4 the thickness of the drainage layer is small compared to the length of the drainage path, the slope of the drainage path i can represent the value of $\left(\frac{H_o}{D}\right)$ and Equation 23-10 can be written as:

$$T_{50} = \frac{n_e \cdot D}{2 \cdot i \cdot k} \tag{23-11}$$

Experience has shown that base and subbase materials, when compacted to densities required in pavement construction, seldom have sufficient permeability to meet the 10-day drainage criterion. In such pavements, the base and subbase materials become saturated, causing a reduced pavement life. When a drainage layer is incorporated into the pavement structure to improve pavement drainage, the criterion for design of the drainage layer shall be that the drainage layer shall reach a degree of drainage of 85 percent within 24 hours. The time for 85 percent drainage is approximately twice the time for 50 percent drainage. The time for 85 percent drainage T_{85} is computed by:

$$T_{85} = \frac{n_e \cdot D}{i \cdot k} \tag{23-12}$$

Figure 23-4. Pavement Geometry for Computation of Time for Drainage



23-2.3.5 Length and Slope of the Drainage Path

As can be seen in Equation 23-10, the time for drainage is a function of the square of the length of the drainage path. For this reason and the fact that for most pavement designs the length of the drainage path can be controlled, the drainage path length is an important parameter in the design of the drainage system. The length of the drainage path L may be computed from Equation 23-13:

$$L = \frac{L_t \cdot \sqrt{i_t^2 + i_e^2}}{i_t} \tag{23-13}$$

where

L_t = the length of the transverse slope of the drainage layer

i_t = the transverse slope of the drainage layer

i_e = the longitudinal slope of the drainage layer

The slope of the drainage path i is a function of the transverse slope and the longitudinal slope of the drainage layer and is computed by Equation 23-14:

$$i = \sqrt{i_t^2 + i_e^2} \qquad (23\text{-}14)$$

23-2.3.6 **Rate of Flow**
The edge drains for pavements having drainage layers shall be designed to handle the maximum rate of flow from the drainage layer. This maximum rate of flow will be obtained when the drainage layer is flowing full and may be estimated using Equation 23-2.

23-2.4 **Use of Drainage Layers**

23-2.4.1 **Purpose of Drainage Layers**
Special drainage layers may be used to promote horizontal drainage of water from pavements, prevent the buildup of hydrostatic water pressure, and facilitate the drainage of water generated by cycles of freeze-thaw.

23-2.4.2 **Placement of Drainage Layers**
In rigid pavements, the drainage layer will generally be placed directly beneath the concrete slab. In this location, the drainage layer will intercept water entering through cracks and joints and permit rapid drainage of the water away from the bottom of the concrete slab. In flexible pavements, the drainage layer will normally be placed beneath the dense-graded aggregate (DGA) base. Placing the drainage layer beneath the base will reduce the stresses on the drainage layer to an acceptable level and drainage will be provided for the base course.

23-2.4.3 **Permeability Requirements for the Drainage Layer**
The material for drainage layers in pavements must be of sufficient permeability to provide rapid drainage and to rapidly dissipate water pressure in addition to providing sufficient strength and stability to withstand load-induced stresses. There is a trade-off between strength or stability and permeability; therefore, the material for the drainage layers should have the minimum permeability for the required drainage application. For most applications, a material with a permeability of 300 m/day (1,000 ft/day) will provide sufficient drainage.

23-2.5 **Use of Filters**

23-2.5.1 **Purpose of Filters in Pavement Structures**
The purpose of filters in pavement structures is to prevent the movement of soil (piping) yet allow the flow of water from one material to another. The need for a filter is dictated

by the existence of water flow from a fine-grain material to a coarse-grain material, which generates a potential for piping of the fine-grain material. The principal location in the pavement structure for a flow from a fine-grain material into a coarse-grain material is where water flows from the base, subbase, or subgrade into the coarse aggregate surrounding the drain pipe. Thus, the principal use of a filter in a pavement system will be in preventing piping into the drain pipe. Although rare, the possibility exists for hydrostatic head forcing a flow of water upward from the subbase or subgrade into the pavement drainage layer. For such a condition, it would be necessary to design a filter to separate the drainage layer from the finer material.

23-2.5.2 Piping Criteria

The criteria for preventing movement of particles from the soil or granular material to be drained into the drainage material are:

$$\frac{\text{15 percent size of drainage or filter material}}{\text{85 percent size of material to be drained}} \leq 5$$

and

$$\frac{\text{50 percent size of drainage or filter material}}{\text{50 percent size of material to be drained}} \leq 25$$

These criteria will be used when protecting all soils except clays without sand or silt particles. For these soils, the 15 percent size of drainage or filter material may be as great as 0.4 mm and the d_{50} criteria may be disregarded.

23-2.5.3 Permeability Requirements

To ensure that the filter material is sufficiently permeable to permit passage of water without hydrostatic pressure buildup, this requirement should be met:

$$\frac{\text{15 percent size of filter material}}{\text{15 percent size of material to be drained}} \geq 5$$

23-2.6 Use of Separation Layers

23-2.6.1 Purpose of Separation Layers

When drainage layers are used in pavement systems, the drainage layers must be separated from fine-grain subgrade materials to prevent penetration of the drainage material into the subgrade or pumping of fines from the subgrade into the drainage layer. The separation layer is different from a filter in that except during frost thaw, there is no requirement to protect against water flowing from the subgrade through the layer into the drainage layer.

23-2.6.2 Requirements for Separation Layers

The main requirements of the separation layer are that the material for the separation layer have sufficient strength to prevent the coarse aggregate of the drainage layer from being pushed into the fine material of the subgrade and that the material have sufficient permeability to prevent buildup of hydrostatic pressure in the subgrade. To satisfy the strength requirements, the material of the separation layer should have a minimum CBR of 50. To allow for release of hydrostatic pressure in the subgrade, the separation layer should have a permeability greater than that of the subgrade. This would not normally be a problem because the permeability of subgrades are orders of magnitude less than the permeability of a 50 CBR material, but to ensure sufficient permeability, the permeability requirements of a filter would apply.

23-2.7 **Use of Geotextiles**

23-2.7.1 **Purpose of Geotextiles**

Geotextiles (engineering fabrics) may be used to replace either the filter or the separation layer. The principal use of geotextiles is for the filter around the pipe for the edge drain. Although geotextiles can be used as a replacement for the separation layer, a geotextile adds no structure strength to the pavement; therefore, this practice is not recommended.

23-2.7.2 **Requirements of Geotextiles for Filters**

When geotextiles are to serve as a filter lining the edge drain trench, the most important function of the filter is to keep fines from entering the edge drain system. For pavement systems having drainage layers, there is little requirement for water flow through the fabric; therefore, for most applications, it is better to have a heavier fabric than would normally be used as a filter. Since drainage layers have a very high permeability, geotextile fabric should never be placed between the drainage layer and the edge drain. The permeability of geotextiles is governed by the size of the openings in the fabric, which is specified in terms of the AOS in millimeters. For use as a filter for the trench of the edge drain, the geotextile should always have an AOS that is equal to or less than 0.212 mm. For geotextiles used as filters with drains installed to intercept groundwater flow in subsurface aquifers, the geotextile should be selected based on criteria similar to the criteria used to design a granular filter.

23-2.7.3 **Requirements for Geotextiles Used for Separation**

Geotextiles used as separation layers beneath drainage layers should be selected based primarily on survivability of the geotextiles, with slightly less emphasis placed on the AOS. When a geotextile is used as a separation layer, the geotextile’s survivability should be rated very high by the rating scheme in AASHTO M 288. This would ensure survival of the geotextile under the stress of traffic during the life of the pavement. To ensure that fines will not pump into the drainage layer yet allow water flow to prevent hydrostatic pressure, the AOS of the geotextile must be equal to or less than 0.212 mm and also equal to or greater than 0.125 mm.

23-3 **DESIGN OF THE PAVEMENT SUBSURFACE DRAINAGE SYSTEM**

The design methodology contained in this chapter is for the design of a pavement subsurface drainage system for the rapid removal of surface infiltration water and water generated by freeze-thaw action. Although the primary emphasis will be on removing water from under the pavement, on occasion the system will also serve as an interceptor drain for groundwater.

23-3.1 Methods

For most pavement structures, water is to be removed by a special drainage layer that allows the rapid horizontal drainage of water. The drainage layer must be designed to handle surface infiltration from a design storm and withstand the stress of traffic. A separation layer must be provided to prevent intrusion of fines from the subgrade or subbase into the drainage layer and facilitate construction of the drainage layer. The drainage layer should feed into a collection system consisting of trenches with a drain pipe, backfill, and filter. The collection system must be designed to maintain progressively greater outflow capabilities in the direction of flow. The outlet for the subsurface drains should be properly located or protected to prevent backflow from the surface drainage system. Some pavements may not require a drainage system because the subgrade may have sufficient permeability for the water to drain vertically into the subgrade. In addition, some pavements designed for very light traffic may not justify the expense of a subsurface drainage system. Even for pavements designed for very light traffic, care must be taken to ensure that base and subbase material are free draining and that water will be not trapped in the pavement structure. For pavement without collection systems, the base and subbase must daylight at the shoulders.

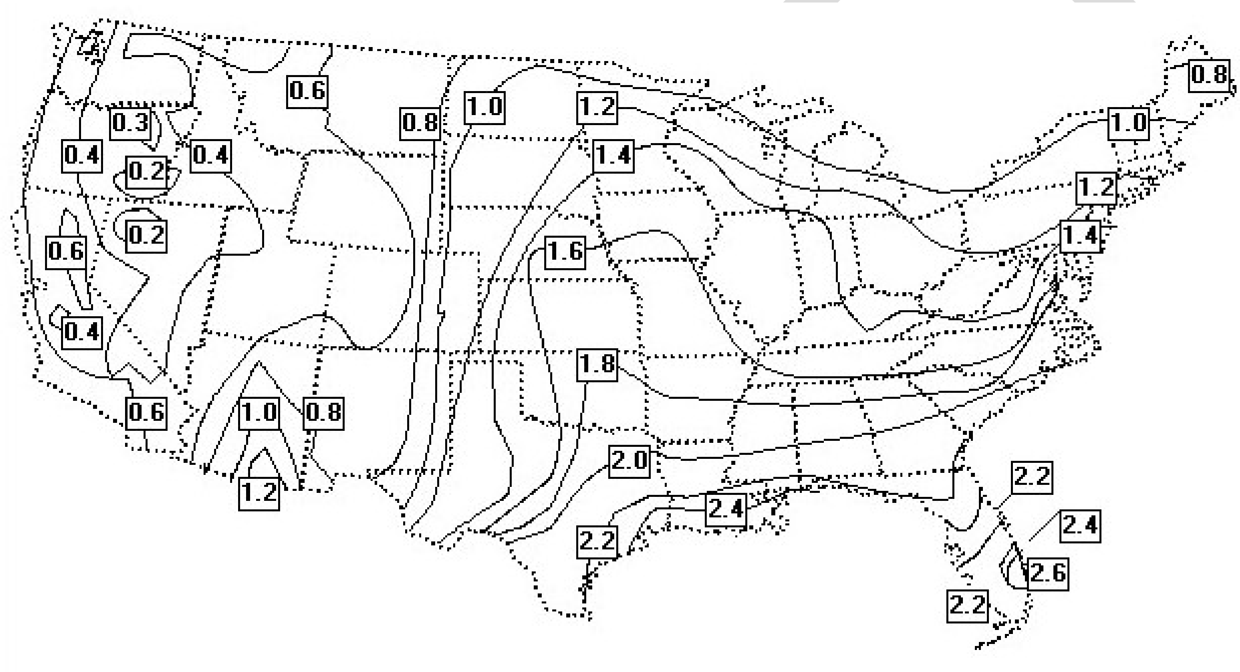
23-3.2 Design Prerequisites

For the satisfactory design of a subsurface drainage system, the designer must have an understanding of environmental conditions, subsurface soil properties, and groundwater conditions.

23-3.2.1 **Environmental Conditions**

Temperature and rainfall data applicable to the local area should be obtained and studied. The depth of frost penetration is an important factor in the design of a subsurface drainage system. For most areas, the approximate depth of frost penetration can be determined by referring to Chapter 22 of this UFC or by using the PCASE computer program. Rainfall data are used to determine the volume of water to be handled by the subsurface drainage system. The data can be obtained from local weather stations or by using Figure 23-5.

Figure 23-5. Design Storm Index, 1-Hour Rainfall Intensity-Frequency Data for the Continental United States Excluding Alaska



23-3.2.2 **Subsurface Soil Properties**
In most cases, the soil properties investigated for other purposes in connection with the pavement design will supply information that can be used for the design of the subsurface drainage system. The two properties of most interest are the coefficient of permeability and the frost susceptibility of the pavement materials.

23-3.2.3 **Coefficient of Permeability**
Knowing the coefficient of permeability of the existing subsurface soils is essential for determining if special horizontal drainage layers are necessary in the pavement. For pavements having subgrades with a high coefficient of permeability, the water entering the pavement will drain vertically and therefore horizontal drainage layers will not be required. For pavements having subgrades with a low coefficient of permeability, the water entering the pavement must be drained horizontally to the collector system or to edge drains.

23-3.2.4 **Frost-Susceptible Soils**
Soils susceptible to frost action are those that have the potential of ice formation when the soil is subjected to freezing conditions with water available. Ice formation takes place at successive levels as freezing temperatures penetrate into the ground. Soils possessing a high capillary rate and low cohesive nature act as a wick in feeding water to ice lenses. Soils are categorized according to their degree of frost susceptibility as shown in Table 23-2. Because a large volume of free water is generated during the thaw of ice lenses, horizontal drainage layers are required to permit the escape of the water from the pavement structure and thus facilitate restoring the pavement strength.

Table 23-2. Frost-Susceptible Soils

Typical Soil			
Frost Group	**Type of Soil**	**Percent Finer than 0.02 mm by Weight**	**Types Under Unified Soil Classification System**
F1	Gravely soils	6–10	GW-GM, GP-GM, GW-GC, GP-GC
F2	(a) Gravely soils (b) Sands	23–20 6–15	GM, GC, GM-GC SM, SC, SW-SM, SP-SM, SW-SC, SP-SC, SM-SC
F3	(a) Gravely soils (b) Sands, except very fine silty sands (c) Clays (PI > 12)	> 20 > 15 --	GM, GC, GM-GC SM, SC, SM-SC CL, CH, ML-CL

Typical Soil			
Frost Group	Type of Soil	Percent Finer than 0.02 mm by Weight	Types Under Unified Soil Classification System
F4	(a) Silts (b) Very fine sands (c) Clays (PI < 12) (d) Varved clays and other fine grained, with banded sediments	-- > 15 -- --	ML, MH, ML-CL SM, SC, SM-SC CL, ML-CL CL or CH layered ML, MH, SM, SC SM-SC or ML-CL

23-3.2.5 **Sources for Data**

From the field explorations made in connection with the project design, include a topographic map of the proposed pavement facility and surrounding vicinity indicating all streams, ditches, wells, and natural reservoirs. Analyze aerial photographs for information on general soil and groundwater conditions. Borings taken during the soil exploration should provide depth to water tables and subgrade soil types. Obtain typical values of permeability for subgrade soils from Figure 23-3. Although the value of permeability determined from Figure 23-3 must be considered as an estimate only, the value should be sufficiently accurate to determine if subsurface drainage is required for the pavement. For the permeability of granular materials, determine estimates of the permeability from these equations:

$$k = \frac{217.5 \cdot (D_{10})^{1.478} \cdot (n)^{6.654}}{(P_{200})^{0.597}} \text{ in mm/sec} \qquad (23\text{-}15)$$

or

$$k = \frac{(6.214 \times 10^5) \cdot (D_{10})^{1.478} \cdot (n)^{6.654}}{(P_{200})^{0.597}} \text{ in ft/day} \qquad (23\text{-}16)$$

where

n = porosity $= 1 - \dfrac{\gamma_d}{\gamma_w \cdot G}$

G = specific gravity of solids (assumed 2.7)

γ_d = dry density of material

γ_w = density of water

D_{10} = effective grain size at 10 percent passing, mm

P_{200} = percent passing 0.075 mm (No. 200) sieve

For the most part, the permeability values needed for design of the drainage layer will be assigned based on the gradation of the drainage material. In some cases, laboratory permeability tests may be necessary; however, use caution and be aware that the permeability of very open granular materials is very sensitive to test methods, methods of compaction, and gradation of the sample. Because of this, use conservative drainage layer permeability values for design.

23-3.3 Criteria for Subsurface Drainage Systems

23-3.3.1 Criteria for Requiring a Subsurface Drainage System

Not all pavements will require a subsurface drainage system, either because the subgrade is sufficiently permeable to allow water to drain vertically into the subgrade or because the pavement structure does not justify the expense of a subsurface drainage system. For pavements in nonfrost areas and having a subgrade with permeability greater than 6 m/day (20 ft/day), one can assume that the vertical drainage will be sufficient such that no drainage system is required. In addition to this exemption for the requirement for drainage systems, flexible pavements that are in nonfrost areas and that have a total thickness of structure above the subgrade of 203 mm (8 in) or less are not required to have a drainage system. All pavements not meeting these criteria are required to have a subsurface drainage system. Even if a pavement meets the exemption requirements, conduct a drainage analysis for possible benefits for including the drainage system. For rigid pavements in particular, take care to ensure that water is drained rapidly from the bottom of the slab and that the material directly beneath the concrete slab is not susceptible to pumping.

23-3.3.2 Design Water Inflow

Design the subsurface drainage of the pavement to handle infiltrated water from a design storm of 1-hour duration at an expected return frequency of 2 years. Figure 23-5 shows the design storm index for the continental United States. The inflow is determined by multiplying the design storm index R times an infiltration coefficient F. The infiltration coefficient will vary over the life of the pavement depending on the type of pavement, surface drainage, pavement maintenance, and the structural condition of the pavement. Since determining a precise value of the infiltration coefficient for a particular pavement is very difficult, a value of 0.5 may be assumed for design.

23-3.3.3 Length and Slope of the Drainage Path

The length of the drainage path is measured along the slope of the drainage layer from the crest of the slope to where the water will exit the drainage layer. In simple terms, the length of the drainage path is the maximum distance water will travel in the drainage layer. The length of the drainage path L in meters (feet) may be computed using Equation 23-13, and the slope i of the drainage path may be computed using Equation 23-14.

23-3.3.4 Thickness of the Drainage Layer

The thickness of the drainage layer is computed such that the capacity of the drainage layer will be equal to or greater than the infiltration from the design storm. When the length of the drainage path L is in meters (feet), the design storm index R is in meters/hour (feet/hour), the permeability of the drainage layer *k* is in meters/hour (feet/hour), and the length of the design storm *t* is in hours, the equation for computing the thickness *H* in meters (feet) is

$$H = \frac{2 \cdot F \cdot R \cdot L \cdot t}{(1.7 \cdot n_e \cdot L) + (k \cdot i \cdot t)} \tag{23-17}$$

The effective porosity n_e, the infiltration coefficient *F*, and the slope of the drainage path *i* are non-dimensional. If the term $(k \cdot i \cdot t)$ is small compared to the term $(1.7 \cdot n_e \cdot L)$, which would be the case for long drainage paths, i.e., for drainage paths longer than approximately 6 m (20 ft), then the required thickness of the drainage layer can be estimated by deleting the term $(k \cdot i \cdot t)$ from Equation 23-17, or:

$$H = \frac{F \cdot R}{0.85 \cdot n_e} \tag{23-18}$$

where the units are the same as in Equation 23-17.

23-3.3.5 **Drainage Criteria**

The subsurface drainage criteria for airfield runways and taxiways require that should the drainage layer become saturated, it should be capable of attaining 85 percent drainage within 24 hours. For airfield parking aprons and other pavement areas receiving only low-volume, low-speed traffic, the time for 85 percent drainage is 10 days. The time for 85 percent drainage is computed by the equation

$$T_{85} = \frac{n_e \cdot L}{i \cdot k} \tag{23-19}$$

where the dimensions of T_{85} will be in days when *L* is in meters (feet) and *k* is in meters/day (feet/day). The time of drainage may be adjusted by changing the drainage material, the length of the drainage path, or the slope of the drainage path. Changing the drainage material will change both the effective porosity and the permeability, but the effective porosity will change, at the most, by a factor of 3, whereas the permeability may change by several orders of magnitude. Thus, providing a more open drainage material would decrease the time for drainage, but more open materials are less stable and more susceptible to rutting. It is therefore desirable to keep the drainage material as dense as possible.

The drainage layer of a pavement is usually placed parallel to the surface; therefore, in most cases, the slope of the drainage path is governed by the geometry of the pavement surface. For large paved areas such as airfield apron areas, the time for drainage is best controlled by designing the collection system to minimize the length of the drainage path. For edge drains along airfield taxiways and runways, it may be difficult to reduce the length of the drainage path without resorting to placing drains under the pavement. Pavements having long longitudinal slopes may require transverse collector drains to prevent long drainage paths. Thus, designing the subsurface drainage system to meet the criteria for time of drainage involves matching the type of drainage material with the drainage path length and slope.

23-3.4 Placement of Subsurface Drainage Systems

23-3.4.1 Rigid Pavements

In the case of rigid pavements, the drainage layer, if required, shall be placed directly beneath the concrete slab. In the structural design of the concrete slab, the drainage layer along with any granular separation layer shall be considered a base layer, and structural benefit may be realized from the layers.

23-3.4.2 Flexible Pavements

In the case of flexible pavements, the drainage layer should be placed either directly beneath the surface layer or beneath a graded crushed aggregate base course. If the required thickness of the granular subbase is equal to or greater than the thickness of the drainage layer plus the thickness of the separation layer, the drainage layer is placed beneath the graded crushed aggregate base. Where the total thickness of the pavement structure is less than 305 mm (12 in), the drainage layer may be placed directly beneath the surface layer and the drainage layer used as a base. When the drainage layer is placed beneath an unbound aggregate base, take care to limit the material passing the 0.075 mm (No. 200) sieve in the aggregate base to 8 percent or less.

23-3.4.3 Separation Layer

The drainage layer must be protected from contamination of fines from the underlying layers by a separation layer placed directly beneath the drainage layer. In most cases, the separation layer should be a graded aggregate material meeting the requirements of a 50 CBR subbase and can, in fact, be considered as part of the subbase. For design situations where a firm foundation already exists and thickness of the separation layer is not needed in the structure for protection of the subgrade, a filter fabric may be substituted for the granular separation layer. In frost areas, the separation layer should be NFS, and some materials used as non-susceptible fill may qualify as a separation layer.

23-3.5 Material Properties

23-3.5.1 Material Properties for Drainage Layers

The material for a drainage layer should be a hard, durable crushed aggregate to withstand degradation under construction traffic as well as in-service traffic. The gradation of the material should be such that the material has sufficient stability for the operation of construction equipment. While well-graded aggregate is desirable for strength and stability, the permeability of the material must be maintained. For most drainage layers, the drainage materials should have a minimum permeability of 300 m/day (1,000 ft/day). Two materials, a RDM and an OGM, have been identified for use in drainage layers. The RDM is a material that has a sufficiently high permeability (300 m/day [1,000 ft/day] to 1,500 m/day [5,000 ft/day]) to serve as a drainage layer and that also has the stability to support construction equipment and the structural strength to serve as a base or subbase. The OGM is a material that has a very high permeability (greater than 1,500 m/day [5,000 ft/day]) and can be used for a drainage layer. The OGM will normally require stabilization for construction stability or for structural strength to serve as a base in a flexible pavement. Gradation limits for the two materials are provided in Table 23-3, and the design properties are provided in Table 23-4. The gradations given in Table 23-3 provide very wide bands, and it is possible to produce gradations within these bands that may not be sufficiently stable for construction without the use of chemical stabilization. Table 23-5 provides the gradation specifications for three aggregate materials, each of which will meet the criteria for stability. These gradations were developed to produce the maximum density given maximum aggregate sizes of 1.5, 1, and 0.75 in, and a maximum of 4 percent passing the number 16 sieve. For drainage layer thicknesses less than 152 mm (6 in), gradations number 1 or 2 may be used. For drainage layers 152 mm (6 in) or more in thickness, any of the three gradations may be used, but the gradations with larger aggregates will produce the more stable aggregate. Each of the gradations would produce a drainage layer with a permeability of approximately 300 m/day (1000 ft/day).

Table 23-3. Gradations of Materials for Drainage Layers and Choke Stone

Drainage Layer Material			
Sieve Designation (mm)	**Rapid Draining Material**	**Open-Graded Material**	**Choke Stone**
38.0 (1-1/2 in)	100	100	100
25.0 (1 in)	70–100	95–100	100
19.0 (3/4 in)	55–100	--	100
12.5 (1/2 in)	40–80	25–80	100
9.5 (3/8 in)	30–65	--	80–100
4.75 (No. 4)	23–50	0–10	23–100
2.4 (No. 8)	0–25	0–5	5–40
1.2 (No. 16)	0–5	--	0–10

Table 23-4. Properties of Materials for Drainage Layers

Property	Rapid Draining Material	Open-Graded Material
Permeability in m/day (ft/day)	300–1,500 (1,000–5,000)	> 1,500 (> 5,000)
Effective Porosity	0.25	0.32
Percent Fractured Faces (Corps of Engineers method)	90 percent for 80 CBR 75 percent for 50 CBR	90 percent for 80 CBR 75 percent for 50 CBR
C_v	> 3.5	--
LA Abrasion	< 40	< 40

Note: C_v is the uniformity coefficient = D60/D10

Table 23-5. Material Gradations for Drainage Layers

Sieve Size	Gradation #1 ¾ inch max.		Gradation #2 1 inch max.		Gradation #3 1½ inch max	
	Percent Passing	Tolerance	Percent Passing	Tolerance	Percent Passing	Tolerance
1.5 in (37.0 mm)					100	–5
1 in (25 mm)			100	–5	79	±8
0.75 in (19 mm)	100	–5	85	±8	66	±8
0.5 in (12.5 mm)	78	±8	65	±8	52	±8
3/8 in (9.5 mm)	63	±8	53	±8	42	±8
No. 4 (4.75 mm)	38	±8	32	±6	25	±6
No. 8 (2.36 mm)	19	±6	16	±6	12	±4
No. 16 (1.18 mm)	2	±2	2	±2	2	±2

23-3.5.2 **Aggregate for Separation Layer**
The separation layer serves to prevent fines from infiltrating or pumping into the drainage layer and to provide a working platform for construction and compaction of the drainage layer. The material for the separation layer should be a graded aggregate meeting the requirements of a 50 CBR subbase as specified in Chapter 7 of this UFC except that the maximum aggregate size should not be greater than 0.25 times the thickness of the separation layer. The permeability of the separation layer should be greater than the permeability of the subgrade, but the material should not be so open as to permit pumping of fines into the separation layer. To prevent pumping of fines, the ratio of d_{15} of the separation layer to d_{85} of the subgrade must be equal to or less than 5. The material property requirements for the separation layer are provided in Table 23-6.

Table 23-6. Criteria for Granular Separation Layers

Maximum Aggregate Size	Lesser of 50 mm (2 in) or 0.25 of layer thickness
Maximum CBR	50
Maximum Percent Passing 2.00 mm (No. 10)	50
Maximum Percent Passing 0.075 mm (No. 200)	15
Maximum Liquid Limit	25
Maximum Plasticity Index	5

d_{15} of Separation Layer to d_{85} of Subgrade	≤ 5

23-3.5.3 Filter Fabric for Separation Layer

Although filter fabric provides protection against pumping, it does not provide extra stability for compaction of the drainage layer; therefore, fabric should be selected only when the subgrade provides adequate support for compaction of the drainage layer. The important characteristics of the fabric are strength for surviving construction and traffic loads and AOS to prevent pumping of fines into the drainage layer. Filter fabric for separation shall be a nonwoven needle punch fabric having a minimum grab strength in accordance with ASTM D4632 of 0.80 kilonewton (kN) (180 lb) at 50 percent elongation and a minimum puncture strength in accordance with ASTM D4833 of 0.35 kN (80 lb). The AOS for the filter fabric shall be determined from Table 23-7.

Table 23-7. Criteria for Filter Fabric to be used as a Separation Layer

Soil Type	Criteria	ASTM Test Method
Soil with 50% or Less Passing No. 200 Sieve	AOS (mm) < 0.6 mm Greater than No. 30 sieve	D4751
Soil with Greater Than 50% Passing No. 200 Sieve	AOS (mm) < 0.297 Greater than No. 50 sieve	D4751

23-4 STABILIZATION OF DRAINAGE LAYER

Stabilization of OGM is normally required for stability and strength and for preventing degradation of the aggregate in handling and compaction. Stabilization may also be used when high-quality crushed aggregate is not available and on occasions when stabilization of RDM is necessary. Stabilization may be accomplished mechanically by use of a choke stone or by the use of a binder such as asphalt or portland cement.

23-4.1 Choke Stone Stabilization

A choke stone is a small-size stone used to stabilize the surface of an OGM. The choke stone should be a hard, durable, crushed aggregate having 90 percent fractured faces. The ratio of d_{15} of the coarse aggregate to the d_{15} of the choke stone must be less than 5, and the ratio of the d_{50} of the coarse aggregate to the d_{50} of the choke stone must be greater than 2. The gradation range for acceptable choke stone is shown in Table 23-3. Normally, ASTM No. 8 or No. 9 stone will meet the requirements of a choke stone for the OGM.

23-4.2 Asphalt Stabilization

Stabilization of the drainage material with asphalt is accomplished by using only enough asphalt as is required to coat the aggregate. Take care so that the voids are not filled by excess asphalt. The asphalt grade used for stabilization should be AC-20 or higher. For stabilization of OGM, 2 to 2.5 percent asphalt by weight should be sufficient to coat the aggregate. Higher rates of application may be necessary when stabilization of less open aggregate such as RDM is necessary.

23-4.3 Cement Stabilization

As with asphalt stabilization, portland cement stabilization is accomplished by using only enough cement paste to coat the aggregate, and care should be taken so that the voids are not filled by excess paste. The amount of portland cement required should be approximately 170 kg/m^3 (2 bags per cubic yard) depending on the gradation of the aggregate. The water-cement ratio should be just sufficient to provide a paste that will adequately coat the aggregate.

23-5 CONSTRUCTION OF THE DRAINAGE LAYER

23-5.1 Experience

Construction of drainage layers can present problems in handling, placement, and compaction. If the drainage material does not have adequate stability, major problems can develop in the placement of the surface layer above the drainage layer. Experience with highly permeable bases (drainage layers) both by USACE and various state departments of transportation indicates that pavements containing such layers can be constructed without undue difficulties if necessary precautions are taken. The key to successful construction of the drainage layer is the training and experience of the construction personnel. Prior to the start of construction, the construction personnel should be taught how to handle and place the drainage material. Placing test strips is recommended for training construction personnel.

23-5.2 Placement of the Drainage Layer

The material for the drainage layer must be placed to prevent segregation and to obtain a layer of uniform thickness. The materials for the drainage layer will require extra care in stockpiling and handling. Placement of the RDM and OGM is best accomplished using an AC paver. To ensure good compaction, the maximum lift thickness should be no greater than 152 mm (6 in). If choke stone is used to stabilize the surface of the OGM, place the choke stone after compaction of the final lift of OGM. Spread the choke stone in a thin layer no thicker than 12.7 mm (0.5 in) using a spreader box or paver. Work the choke stone into the surface of the OGM by using a vibratory roller and by wetting. The choke stone remaining on the surface should not migrate into the OGM by the action of water or traffic.

23-5.3 Compaction

Compaction is a key element in the successful construction of the drainage layer. Compaction control normally used in pavement construction is not appropriate for materials such as RDM and OGM. It is therefore necessary to specify compaction techniques and level of effort instead of the properties of the end product. It will be important to place the drainage material in relatively thin lifts of 152 mm (6 in) or less and to have a good, firm foundation beneath the drainage material. The recommended method of determining the required compaction effort is to construct a test section and closely monitor the aggregate during compaction to determine when crushing of the aggregate appears excessive. Experience has indicated that sufficient compaction can be obtained by 6 passes or fewer of a vibratory roller loaded at approximately 9 metric tons (10 short tons). Material not being stabilized with asphalt or cement should be kept moist during compaction. Asphalt-stabilized material for drainage layers must be compacted at a slightly lower temperature than a dense-graded asphalt material. In most cases, it will be necessary to allow an asphalt-stabilized material to cool to less than 93 degrees C (200 degrees F) before beginning compaction.

23-5.4 Protection after Compaction

After compaction, protect the drainage layer from contamination by fines from construction traffic and from the flow of surface water. The surface layer should be placed as soon as possible after placement of the drainage layer. Also, take precautions to protect the drainage layer from disturbance by construction equipment. Only tracked asphalt pavers should be allowed for paving over any RDM or OGM that has not been stabilized. Drivers should avoid rapid acceleration, hard braking, or sharp turning on the completed drainage layer. Although curing of cement-stabilized drainage layers is not critical, efforts should be made at curing until the surface layer is placed.

23-5.5 Proof Rolling

For Army Class IV airfields with runways over 1,524 m (5,000 ft) and Air Force heavy-, modified-heavy, and medium-load flexible airfield pavements, proof rolling as per Chapter 8 of this UFC is required on the graded crushed aggregate base even when the base is used over a drainage layer. Proof rolling the separation layer prior to placing the drainage layer for other airfield pavements is recommended. For other Air Force flexible airfield pavements and Army Class IV flexible airfield pavements with runways less than 1,524 m (5,000 ft), it is recommended that the proof rolling be accomplished using a rubber-tired roller load to provide a minimum tire force of 89 kN (20,000 lb) and inflated to at least 620 kPa (90 lb/in^2). A minimum of six coverages should be applied, where a coverage is the application of one tire print over each point in the surface of the designated area. For rigid pavements and flexible pavements for roads, streets, parking areas, and Class I, II, and III Army airfields, proof rolling of the separation layer may be accomplished using the rubber-tired roller described in this paragraph or by using a truck having tandem axles with either dual tires or super single tires. The truck should be loaded to provide 89 kN (20,000 lb) per axle. During proof rolling, action of the separation layer must be monitored for any sign of excessive movement or pumping that would indicate soft spots in the separation layer or the subgrade. Since the successful placement of the drainage layer depends on the stability of the separation layer, all weak spots must be removed and replaced with stable material. All replaced material must be proof rolled as specified in this paragraph.

23-6 COLLECTOR DRAINS

23-6.1 Design Flow

Provide collector drains to collect and transport water from under the pavement. For pavements having drainage layers, collector drains are mandatory. The collector system should have the capacity to handle the water from the drainage layer plus water from other sources. The amount of water entering the collector system from the drainage layer is computed assuming that the drainage layer is flowing full. Thus, the volume of water Q in cubic millimeters per second per meter (cubic feet per day per foot) of length of collector pipe (assuming the drainage layer is on only one side of the collector) would be:

$$Q = 1000 \cdot H \cdot i \cdot k \text{ in cubic mm per second per meter} \tag{23-20}$$

or

$$Q = H \cdot i \cdot k \text{ in cubic ft per day per foot} \tag{23-21}$$

where

H = thickness of the drainage layer, mm (ft)

i = slope of the drainage layer

k = permeability of the material in the drainage layer, mm/sec (ft/day)

If the collector system has water entering from both sides, the volume of water entering the collector would be twice that given by Equation 23-20 or Equation 23-21.

23-6.2 Design of Collector Drains

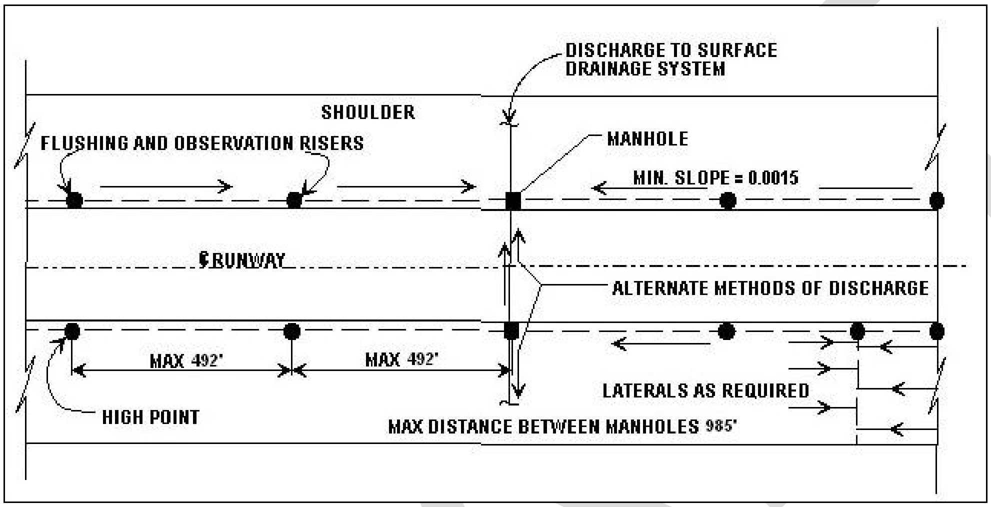
23-6.2.1 Drainage System Layout

The collector drains are normally placed along the shoulder of the pavement as illustrated in Figure 23-6. The system will consist of the drain pipe, flushing and observation risers, manholes, discharge laterals, filter fabric, and trench backfill. Since placing subsurface drains under pavements may result in differential settlement or heave, avoid this when possible. The drainage system for large areas of pavement may require placement of subsurface drains under the pavement. For these cases, place the subsurface drains to avoid high traffic areas. In areas of extreme cold temperatures and heavy snow buildup, place laterals to reduce the probability that they will become clogged with ice or snow. Also, in areas of extreme cold temperatures, placing the collector drains below the depth of frost penetration may not be possible; therefore, the collector pipe may be filled with ice while thawing is occurring near the surface. For this case, make provisions to drain the upper portion of the pavement either by daylighting the drainage layer or providing special laterals to drain the drainage layer.

23-6.2.2 Collector Pipe

The collector pipe may be perforated flexible acrylonitrile butadiene styrene (ABS), corrugated polyethylene (CPE), or smooth rigid polyvinyl chloride pipe (PVC). Pipe should conform to the appropriate AASHTO specification. Most state highway agencies use either CPE or PVC. For CPE pipe, AASHTO M 252 is suggested, while for PVC pipe, AASHTO M 278 is recommended. Though asphalt-stabilized material is not recommended as backfill around pipe, if asphalt-stabilized material is to be used, the pipe should be PVC 90 degrees C electrical plastic conduit EPC-40 or EPC-80 conforming to the requirements of National Electrical Manufacturers Association (NEMA) Specification TC-2. Geocomposite edge drains (strip drains) may be used in special situations but only with the approval of USACE-TSC, the appropriate Air Force MAJCOM pavements engineer, or the Naval Facilities Engineering Service Center. Geocomposite edge drains should be considered only for pavements without a drainage layer.

Figure 23-6. Plan View of Subsurface Drainage System



23-6.2.3 **Pipe Size and Slopes**

The pipe must be sized according to Equation 23-22 or 23-23 to have a capacity sufficient to collect the peak flow from under the pavement. Equations 23-22 and 23-23 are Manning equations for computing the capacity of a full-flowing circular drain. The equation for flow Q in cubic feet per second is:

$$Q = \frac{1.486}{n} \cdot (A) \cdot \left[\frac{d}{4}\right]^{2/3} \cdot \left(s^{1/2}\right) \tag{23-22}$$

where

n = coefficient of roughness for the pipe

A = area of the pipe, ft^2

d = pipe diameter, ft

s = slope of the pipe invert

For metric units, the equation for flow in cubic meters per second is:

$$Q = \frac{1.0}{n} \cdot (A) \cdot \left[\frac{d}{4}\right]^{2/3} \cdot \left(s^{1/2}\right) \qquad (23\text{-}23)$$

where

n and s are as defined in Equation 23-22

A = pipe area, m^2

d = pipe diameter, m

The coefficient of roughness for different pipe types can be obtained from Table 23-8. Except for long intercepting lines and extremely severe groundwater conditions, 152-mm-diameter (6-in-diameter) drains should be satisfactory for most subsurface drainage installations. The minimum size pipe recommended for all collector drains is 152 mm (6 in) diameter. The recommended minimum slope for subdrains is 0.15 percent.

Table 23-8. Coefficient of Roughness for Different Types of Pipe

Type of Pipe	Coefficient of Roughness, *n*
Clay, concrete, smooth-wall plastic, and asbestos-cement	0.013
Bituminous-coated, non-coated corrugated metal pipe or corrugated metal pipe	0.024

23-6.3 **Placement of the Drainage Layer and Collector Drains**
In general, the drainage layer is placed below the concrete surface for a rigid pavement and below the base course for a flexible pavement. Typical designs details for placement of the drainage layer and the collector drains in non-frost areas are provided in Figures 23-7a, 23-8a, 23-9a, and 23-10a. In most cases, the trench for the collector drains should be wide enough to provide 152 mm (6 in) of clearance on each side of the pipe. The depth of the trench must be sufficient to provide a minimum 305 mm (12 in) from the top of the pavement subgrade to the center of the pipe, plus 76 mm (3 in) of clearance beneath the pipe. In frost areas, use extra care in placing subsurface drains. The typical design details for placement of the drainage layer and the collector drains for frost areas are provided in Figures 23-7b, 23-7c, 23-8b, 23-9b, 23-9c, and 23-10b. For F3 and F4 subgrades, always place a collector pipe such that there will be positive drainage for the drainage layer and any NFS fill. If possible, place the drains below the depth of frost penetration. For many locations, placing the drains below the depth of frost penetration will not be economically feasible and therefore the drains and backfill will be subject to freezing. In areas where the depth of frost penetration is greater than 1.2 m (4 ft) below the bottom of the drainage layer, the pipe need not be located deeper than 1.2 m (4 ft) from the bottom of the drainage layer. Because differential frost heave will cause pavement problems in frost areas, the sides of the trench shall be sloped not

steeper than 1 vertical on 10 horizontal for the depth of frost penetration. At the edge of the pavement where the pavement will not be subject to traffic, the sides of the trench may be sloped at a slope of 1 vertical on 4 horizontal. The sloping of the trench sides is not required for the parts of the trench in NFS materials or for F1 or S1 soils unless the pavement over the trench is subjected to high-speed traffic.

The placement of collector drains under the interior portion of a pavement in frost areas is a special case in which the collector drain is not directly connected to the drainage layer by an OGM or a RDM. This case is illustrated in Figures 23-7b, 23-7c, 23-9b, and 23-9c. The interior designs are based on the premise that NFS fill will have sufficient permeability to allow vertical drainage of the drainage layer into the collector pipes. Another premise is that the filter fabric will have sufficient area as not to impede the flow of water from the NFS fill to the collector pipe. The exception to the minimum requirement for the depth of the collector pipe below the surface of the subgrade is the interior case in a frost area for an F3 or F4 subgrade when the collector pipe is above the depth of frost penetration. For this case, keep the depth of the pipe below the surface of the subgrade to a minimum.

Figure 23-7a. Typical Interior Subdrain Detail for Rigid Pavement (Non-Frost Areas)

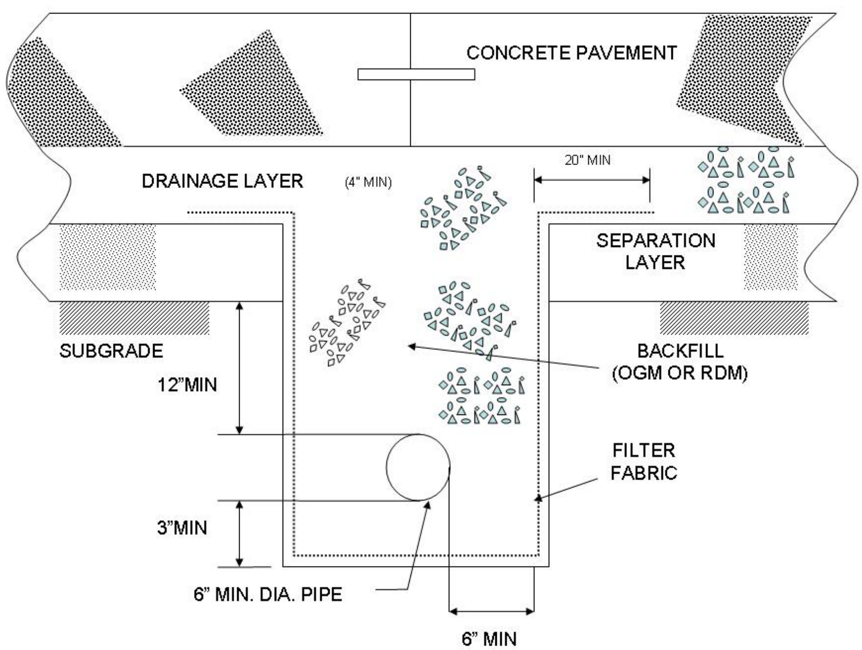


Figure 23-7b. Typical Interior Subdrain for Rigid Pavement (Frost Areas, Depth of Frost > Depth to Pipe)

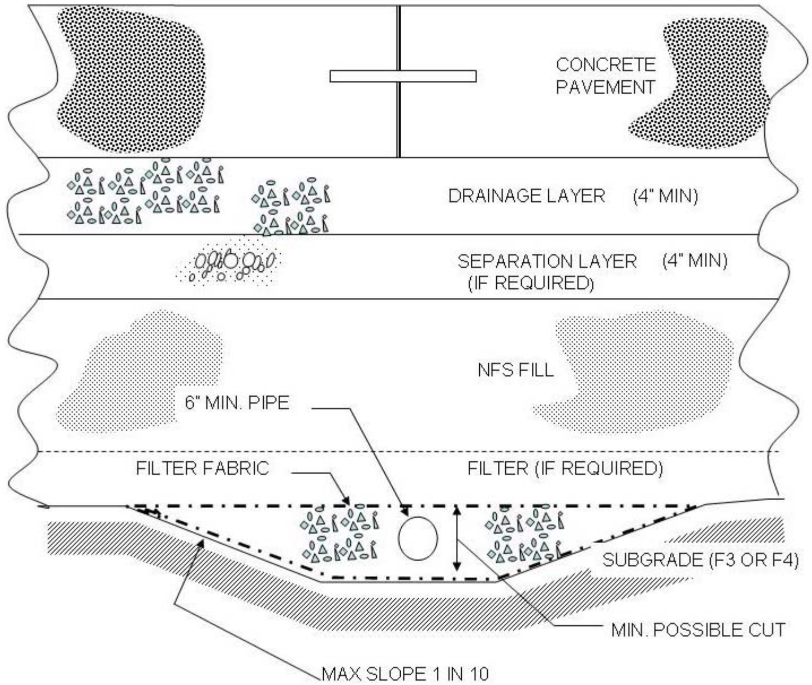


Figure 23-7c. Typical Interior Subdrain for Rigid Pavement (Frost Areas, Depth of Frost < Depth to Pipe)

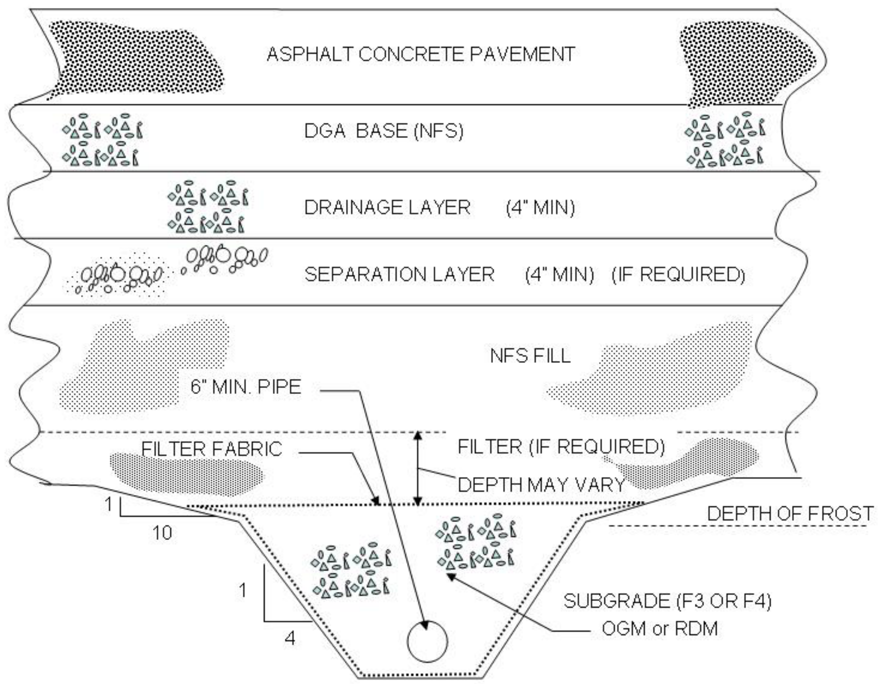


Figure 23-8a. Typical Edge Subdrain Detail for Rigid Pavement (Non-Frost Areas)

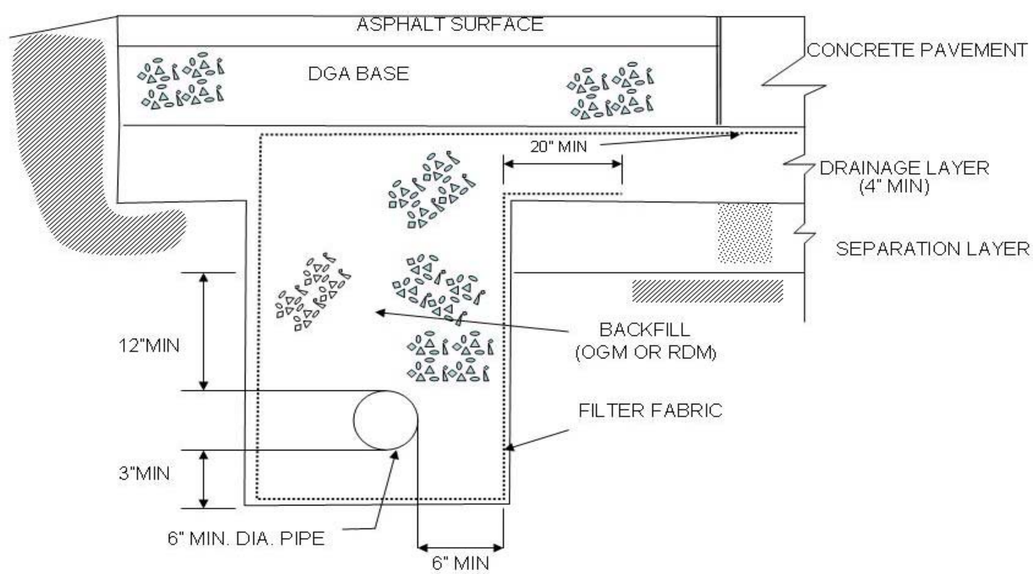


Figure 23-8b. Typical Edge Subdrain Detail for Rigid Pavement (Frost Areas)

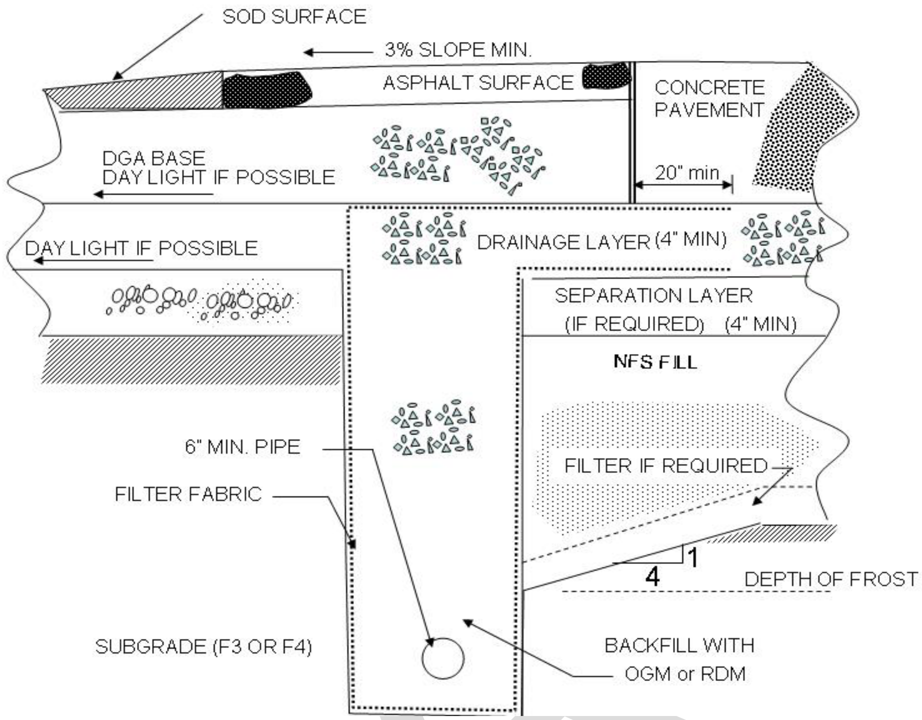


Figure 23-9a. Typical Interior Subdrain Detail for Flexible Pavement (Non-Frost Areas)

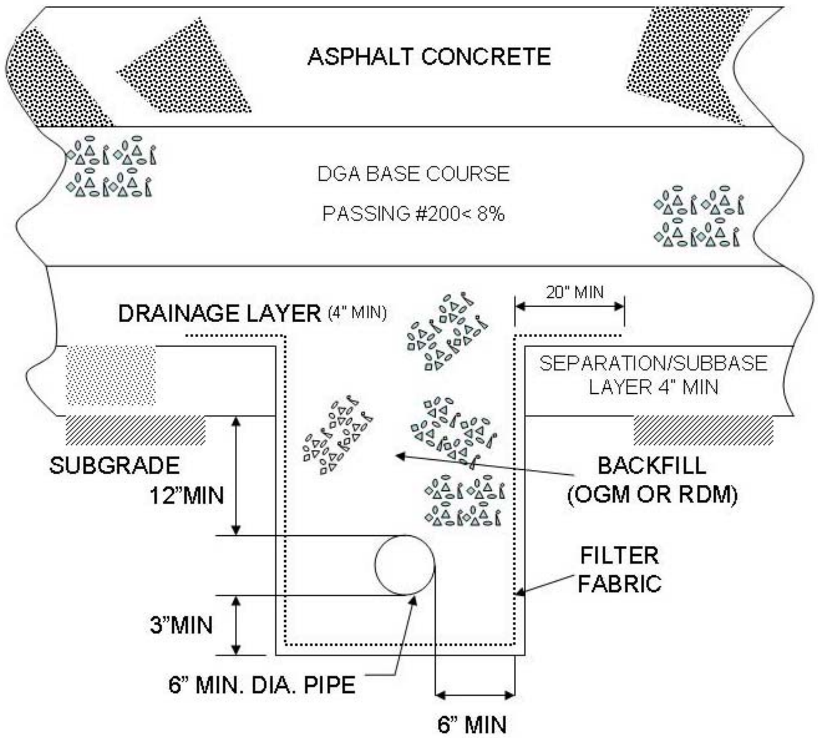


Figure 23-9b. Typical Interior Subdrain Detail for Flexible Pavement (Frost Areas, Depth of Frost > Depth of Pipe)

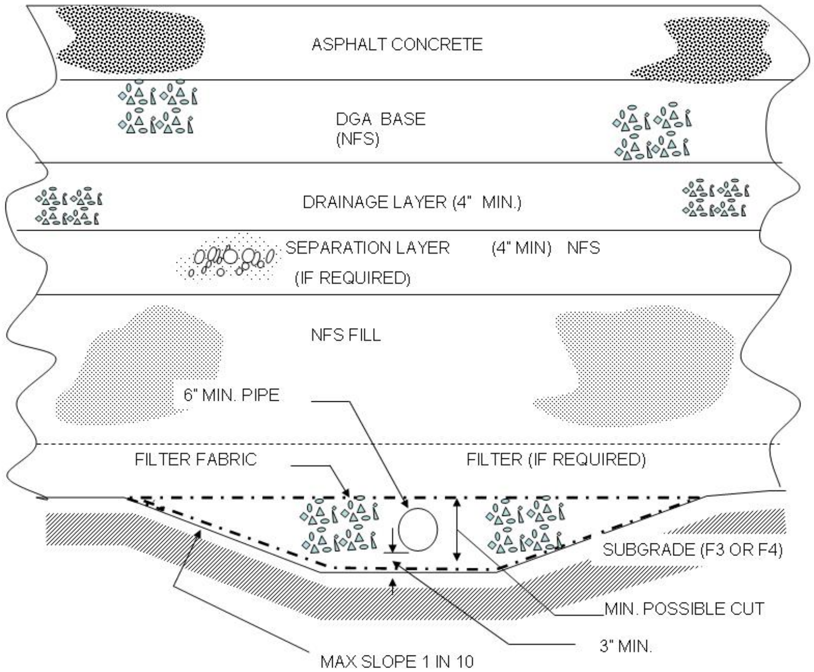


Figure 23-9c. Typical Interior Subdrain Detail for Flexible Pavement (Frost Areas, Depth of Frost < Depth of Pipe)

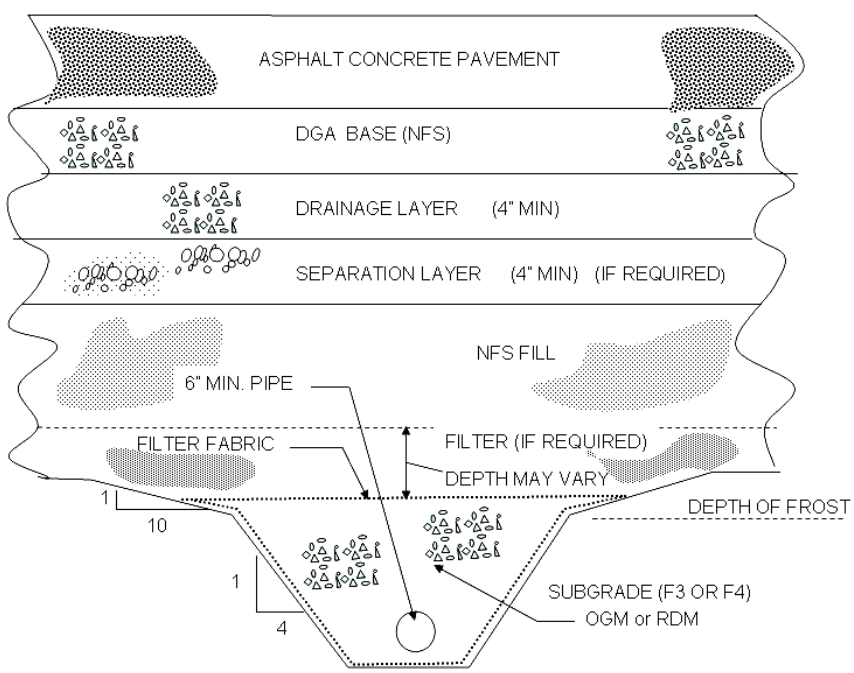


Figure 23-10a. Typical Edge Subdrain Detail for Flexible Pavement (Non-Frost Areas)

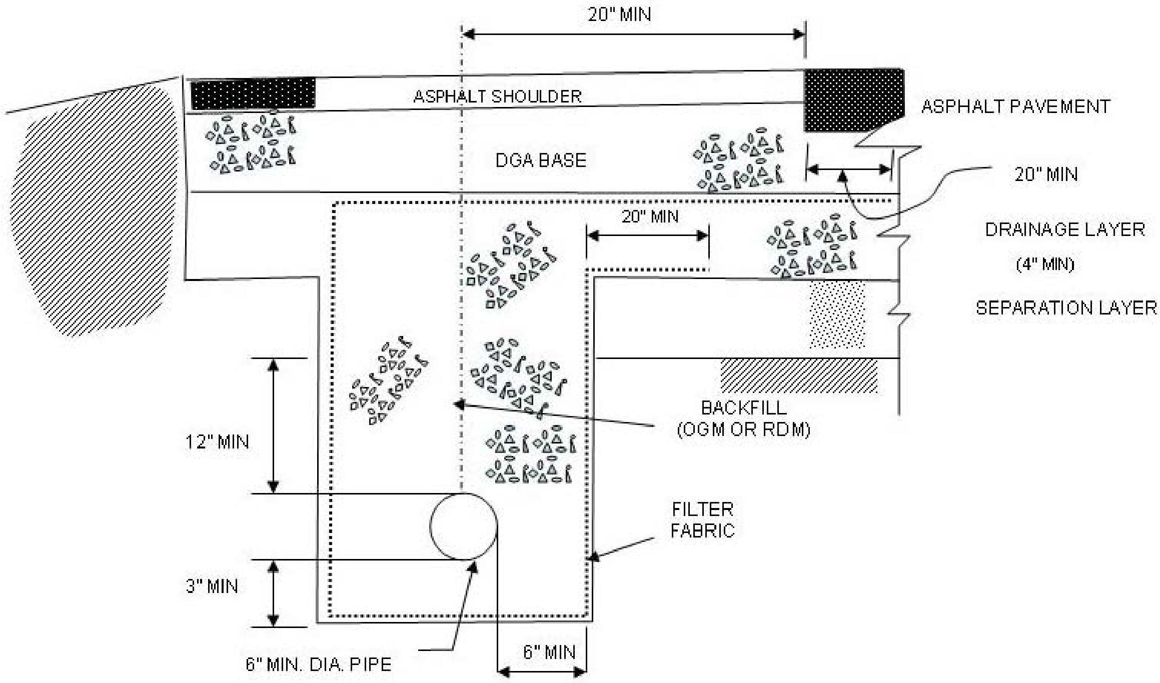
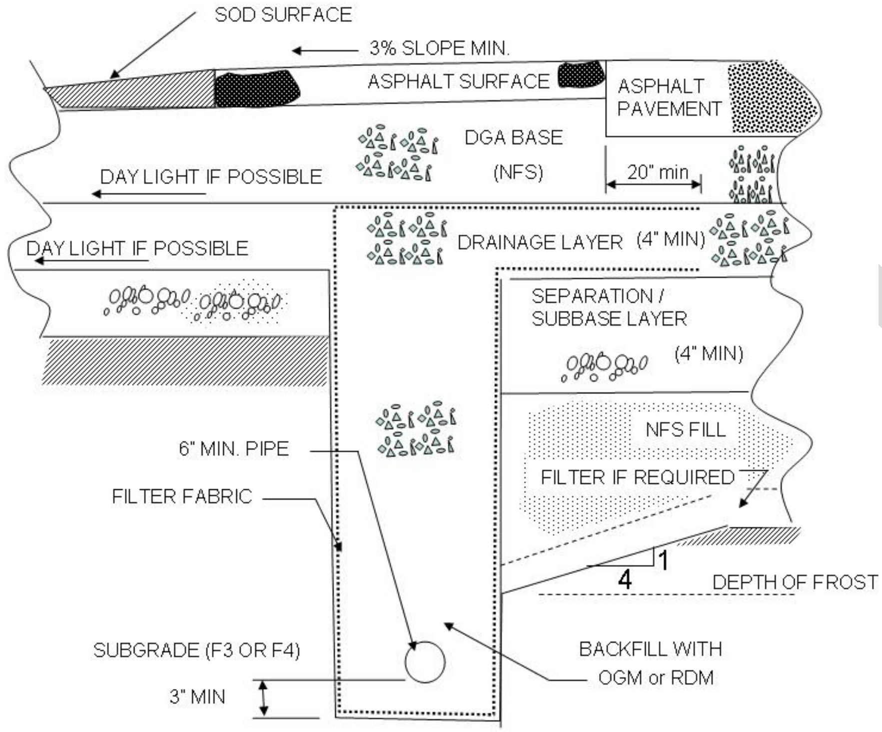


Figure 23-10b. Typical Edge Subdrain Detail for Flexible Pavement (Frost Areas)



23-6.3.1 Backfill

The trench should be backfilled with a permeable material to rapidly convey water to the drainage pipe. The backfill material may be an OGM, RDM, or other uniformly graded aggregate. A minimum of 76 mm (3 in) of aggregate should be placed beneath the drainage pipe. Proper compaction or chemical stabilization of the backfill is necessary to prevent settlement of the fill. In placing the backfill, compact it in lifts not exceeding 152 mm (6 in). When using geocomposites in place of pipe, placing the geocomposites against the material to be drained should keep the backfill from conveying water. For this reason, the backfill for the geocomposites will not require the high permeability required for the backfill around the pipe drains; however, since the backfill for the geocomposites will be against the side of the trench, the backfill should meet the requirements of a granular filter.

23-6.3.2 Geotextiles in the Trench

Line the trench with a geotextile filter fabric as shown in Figures 23-7 through 23-10, which provide the typical details. The filter fabric should be placed to separate the permeable backfill of the trench from the subgrade or subbase materials, but it must not impede the flow of water from the drainage layer to the drain pipe. The filter fabric must also protect from the infiltration of fines from any surface layers. This is particularly important for drains placed outside the pavement area where surface water can enter the drain through a soil surface. The filter fabric for the trench shall be a nonwoven needle punch fabric meeting the criteria in Table 23-9.

Table 23-9. Criteria for Fabrics Used in Trench Construction

Soil or Fabric Characteristic	ASTM Test Method	Criteria
Soil with 50% or Less Passing No. 200 Sieve	D4751	AOS < 0.6 mm (Sieve No. 30)
Soil with Greater Than 50% Passing No. 200 Sieve	D4751	AOS < 0.297 mm (Sieve No. 50)
Minimum Grab Strength in kN (lb) at 50% Elongation	D4632	0.6 (130)
Minimum Puncture Strength in kN (lb)	D4833	0.25 (55)

23-6.3.3 Trench Cap

Edge drains placed outside of a paved area should be capped with a layer of low permeability material, such as an asphalt-stabilized surface, to reduce the infiltration of surface water into the subsurface drainage system. If the area above the edge drain is to be sod surfaced, a filter layer will be required between the drain layer and sod.

23-6.4 Lateral Outlet Pipe

23-6.4.1 Design

The lateral outlet pipe provides a means of getting water out of the edge drains and of cleaning and inspecting the system. Edge drains should be provided with lateral outlet pipes spaced at intervals (90 to 150 m) (300 to 500 ft) along the edge drains and at the low point of all vertical curves. To facilitate drain cleanout, the outlet pipes should be placed at approximately a 45 degree angle from the direction of flow in the collector drain. The lateral pipe should be a metal or rigid solid-walled pipe and should be equipped with an outlet structure. A 3 percent slope from the edge drain to the outlet structure is recommended. Where possible, outlet pipes should be connected to existing storm drains or inlets to reduce outlet maintenance. For a lateral pipe flowing to a ditch, the invert of the outlet pipe should be a minimum of 152 mm (6 in) above the 2-year design flow in the ditch. To prevent piping, the trench for the outlet pipes must be backfilled with a material of low permeability or provided with a cutoff wall or diaphragm. Dual outlets are recommended for maintenance considerations, as shown in Figure 23-11. The dual outlet system allows sections of collector drains to be flushed to clear any debris material blocking the free flow of water. Note these additional recommended design details for drainage outlets:

(a) Provide dual outlets with large-radius bends as shown in Figure 23-12.

(b) Use rigid walls, not perforated pipes. For pipe drains, use the same diameter pipe as the collector drains. For prefabricated, geocomposite drains, 102- to

152-mm-diameter (4- to 6-in-diameter) pipe should provide adequate hydraulic capacity. The flow capacity of the outlets must be greater than that of the collector drains. In general, because of the greater slope provided for outlet pipes, the hydraulic capacity is not a problem.

(c) Place the discharge end of the outlet pipe at least 152 mm (6 in) above the 23-year design flow in the drainage ditch (Figure 23-13). This requirement applies even if the outlet is discharging into storm drain inlets.

(d) In frost areas, give special attention to the placement of the outlet pipes so they do not become clogged with ice or snow.

23-6.4.2 **Outfall for Outlet Pipe**

The outfall for the outlet pipe should be provided with a headwall to protect the outlet pipe from damage, prevent slope erosion, and facilitate the location of outlet pipes. Headwalls should be placed flush with the slope so that mowing operations are not impaired. Easily removable rodent screens should be installed at the pipe outlet. The headwall may be precast or cast in place. Figure 23-14 is an example of a design for a headwall.

Figure 23-11. Schematic of Dual Outlet System Layout (Baumgardner, 1998)

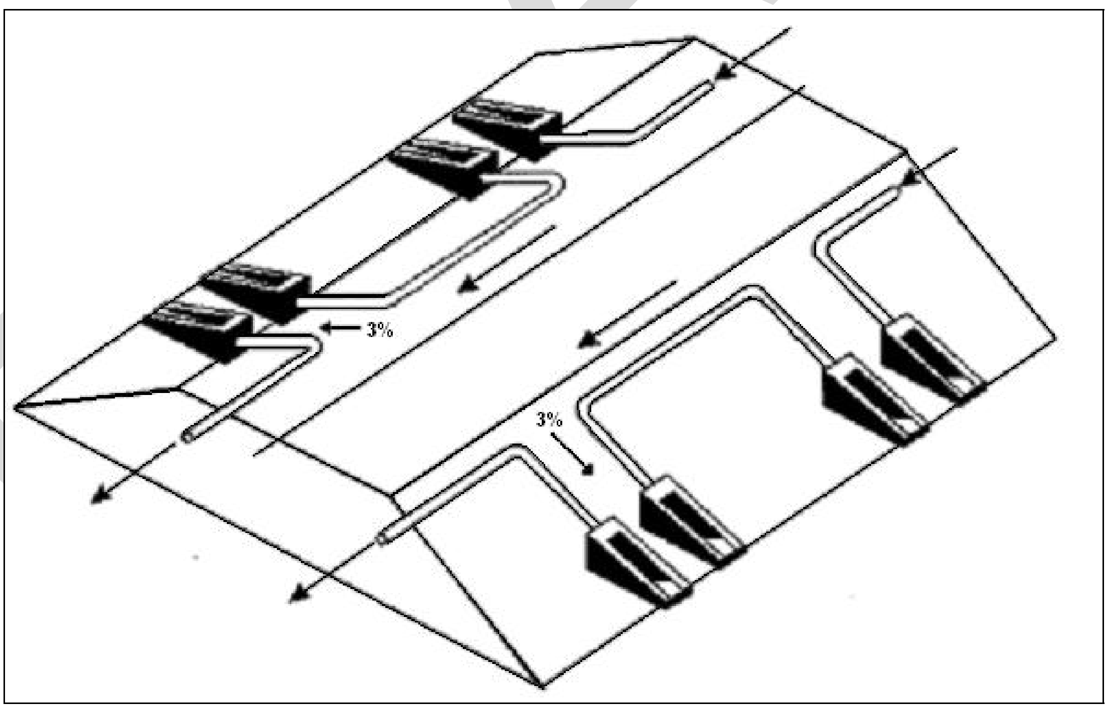


Figure 23-12. Illustration of Large-Radius Bends Recommended for Drainage Outlet

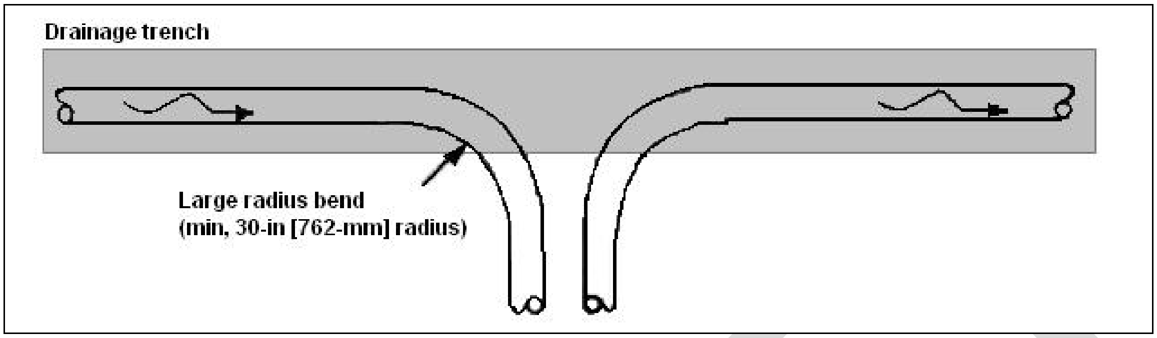
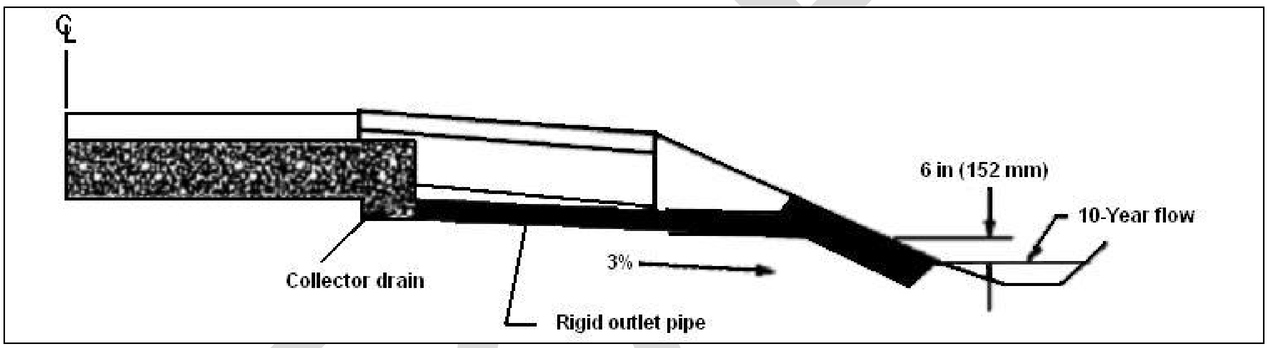


Figure 23-13. Recommended Outlet Design Detail



23-6.4.3 Reference Markers

Although not a requirement, reference markers are recommended for the outlets to facilitate maintenance and observation. A simple flexible marker post or marking on the shoulder will suffice to mark the outlet.

23-6.5 Cross Drains

Cross drains may be required at locations where flow in the drainage layer is blocked, for steep longitudinal grades, or at the bottom of vertical curves. For example, cross drains may be required where pavements abut building foundations, at bridge approach slabs, or where drainage layers abut impermeable bases.

23-6.6 Manholes, Observation Basins, and Risers

Manholes, observation basins, and risers are installed on subsurface drainage systems for access to the system to observe its operation and to flush or rod the pipe for cleaning. When required, manholes on subgrade pipe drains should be located at intervals of not over 300 m (1,000 ft) with one flushing riser located between manholes and at dead ends. Manholes should be provided at principal junction points of several drains. Typical details of construction are provided in Chapter 4.

23-7 MAINTENANCE OF SUBSURFACE DRAINAGE SYSTEMS

Commitment to maintenance is as important as providing subsurface drainage systems. In fact, an improperly maintained drainage system can cause more damage to the pavement structure than if no drainage were provided at all. Poor maintenance leads to clogged or silted outlets and edge-drain pipes, missing rodent screens, excessive growth of vegetation blocking outlet pipes and openings on daylighted bases, and growth of vegetation in side ditches. These problems can potentially cause backing up of water within the pavement system, thereby defeating the purpose of providing the drainage system; therefore, inspections and maintenance of subsurface drainage systems should be made an integral part of the policy of any agency installing these systems. The inspection process is comprised of two parts: (a) visual inspection, and (b) video inspection.

23-7.1 Visual Inspection

The visual inspection process includes these items:

23-7.1.1 Evaluation of external drainage-related features, including measuring ditch depths and checking for crushed outlets, excessive vegetative growth, clogged and debris-filled daylighted openings, condition of headwalls, presence of erosion, and missing rodent screens. This operation should be performed at least once a year.

23-7.1.2 Pavement condition evaluation to check for moisture-related pavement distresses such as pumping, faulting, and D-cracking in PCC pavements and fatigue cracking and AC stripping in AC pavements. This operation could be either a full-scale PCI survey or a brief overview survey, depending on agency needs. The recommended frequency for this activity is once every 2 years.

23-7.2 Video Inspection

Video inspections play a vital role in monitoring in-service drainage systems. The video inspection process can be used to check for clogged drains due to silting and intrusion of surrounding soil as well as for any problems with the drainage system such as ruptured pipes and broken connections. Video inspections should be carried out on an as-needed basis whenever there is evidence of drainage-related problems. Table 23-10 provides a detailed list of equipment used in a FHWA study (Daleiden, 1998). A video inspection system typically consists of a camera head, a long flexible probe mounted on a frame for inserting the camera head into the pipe, and a data acquisition unit fitted with a video screen and a video recorder. This system can be used to detect and correct any construction problems before a project is accepted. The construction-related problems that are easily detected using video equipment include crushed or ruptured drainage pipes, improper connections between drainage pipes, and problems with the connection between the outlet pipe and headwall.

Figure 23-14. Example Design for a Headwall

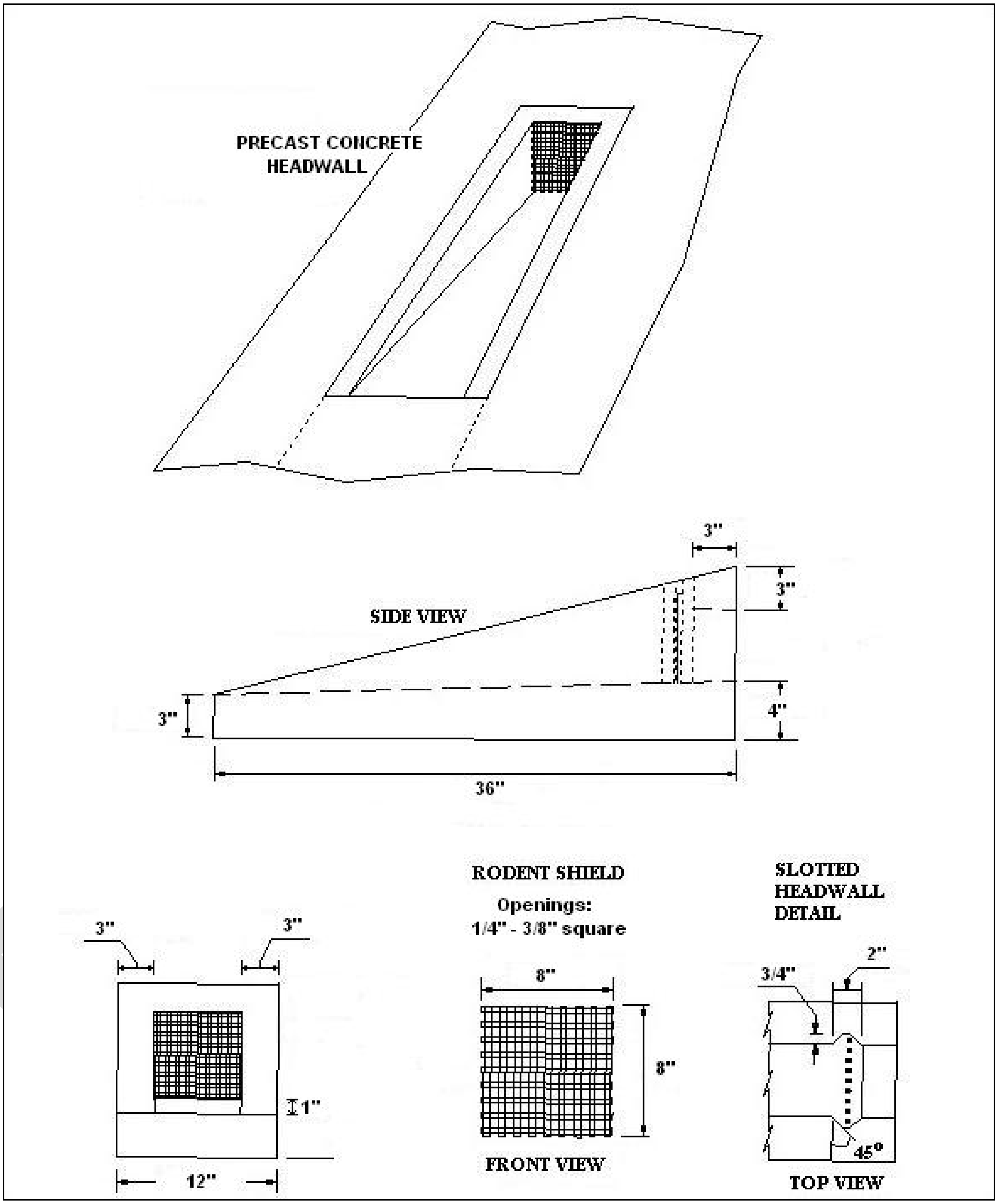


Table 23-10. Equipment Description of FHWA Video Inspection Study (Daleiden, 1998)

Equipment	Characteristics
Camera	Pearpoint flexiprobe high-resolution, high-sensitivity, waterproof color video camera engineered to inspect pipes 76 to 152 mm (3 to 6 in) in diameter. The flexiprobe light head and camera has a physical size of 71 mm (2.8 in) and is capable of negotiating 102- by 102-mm (4- by 4-in) plastic tees. The light head incorporates 6 high-intensity lights. This lighting provides the ability to obtain a “true” color picture of the entire surface periphery of a pipe. The camera includes a detachable hard plastic ball that centers the camera during pipe inspections.
Camera Control Unit	The portable color control unit includes a built-in 203-mm (8-in) color monitor and controls including remote iris, focus, video input/output, audio in with built-in speaker, and light level intensity control. Two VCR input/output jacks are provided for video recording as well as tape playback verification through the built-in monitor.
Metal Coiler and Push Rod With Counter	The portable coiler contains 152 mm (6 in) of integrated semi-rigid push rod, gold and rhodium slip rings, electro-mechanical cable counter, and electrical cable. The integrated push rod/electrical cable consists of a special epoxy glass reinforced rod with polypropylene sheathing material, which will allow for lengthy inspections due to the semi-rigid nature of this system.
Video Cassette Recorder	The video cassette recorder is a high-quality four-head industrial grade VHS recorder with audio dubbing, still frame, and slow speed capabilities.
Generator	A compact portable generator capable of providing 650 watts at 115 volts to power the inspection equipment.

Equipment	Characteristics
Molded Transportation Case	A molded transportation case, specifically built for air transportation, encases the control unit, camera, and videocassette recorder.
Color Video Printer	A video printer is incorporated into the system to allow the technician to obtain color prints of pipe anomalies or areas of interest.

23-7.3 **Maintenance Guidelines**

23-7.3.1 **Collector Drains and Outlets**
The collector drains and outlets should be flushed periodically with high-pressure water jets to loosen and remove any sediment that has built up within the system. The key to this operation is having the appropriate outlet details that facilitate the process, such as the dual headwall system shown in Figure 23-11. The area around the outlet pipes should be kept mowed to prevent any buildup of water. Missing rodent screens and outlet markers and damaged pipes and headwalls need to be either repaired or replaced.

23-7.3.2 **Daylighted Systems**
Routine removal of roadside debris and vegetation clogging the daylighted openings of a permeable or dense-graded base is very important for maintaining the functionality of these systems.

23-7.3.3 **Drainage Ditches**
Drainage ditches should be kept mowed to prevent excessive vegetative growth. Debris and silt deposited at the bottom of the ditch should be cleaned periodically to maintain the ditch line and to prevent water from backing up into the pavement system.

GLOSSARY

Abbreviations and Acronyms

AASHTO—American Association of State Highway and Transportation Officials

ABC—all bituminous concrete

ABS—acrylonitrile butadiene styrene

AC—Advisory Circular

AC—asphalt concrete

ACI—American Concrete Institute

AFB—Air Force Base

AFH—Air Force handbook

AFJPAM—Air Force Joint Pamphlet

AFM—Air Force manual

AOS—apparent opening size

APU—auxiliary power unit

ASR—alkali-silica reaction

ASTM—American Society for Testing and Materials

AT&A—Air Traffic and Airspace

bags/yard^3—bags per cubic yard

C—Celsius

CEGS—Corps of Engineers Guide Specifications

CBR—California Bearing Ratio

CNO/CMC—Chief of Naval Operations/Command Master Chief

CPE—corrugated polyethylene

CRD—Concrete Research Division

CRREL—Cold Regions Research and Engineering Laboratory (Army)

DCP—dynamic cone penetrometer

DEH—Directorate of Engineering and Housing

DFI—design freezing index

DGA—dense-graded aggregate

DM—Design Manual

DME—distance measuring equipment

DOD—Department of Defense

EC—Engineering Circular

EPC—electrical plastic conduit

ERDC—U.S. Army Engineer Research and Development Center

ETL—Engineering Technical Letter

F—Fahrenheit

FAA—Federal Aviation Administration

FAIR—Frost Area Index of Reaction

FASSI—Frost Areas Soil Support Index

FHWA—Federal Highway Administration

FM—Field Manual

FOD—foreign object damage

ft^2—square foot

ft/day—feet per day

ft/min—feet per minute

g—gram

GGBF—ground granulated blast-furnace

HMA—hot-mixed asphalt

Hz—hertz

ICAO—International Civil Aviation Organization

IFR—instrument flight rules

in—inch

lb—pound

lb/ft^3—pounds per cubic foot

lb/in^3—pounds per cubic inch

IP—inch-pound

JFR—Jet-fuel-resistant

kg—kilogram

kg/m^3—kilograms per cubic meter

kip—kilopound

kN—kilonewton

kPa/mm—kilopascals per millimeter

ksi—kips per square inch

LL—liquid limit

LOX—liquid oxygen

LSFP—limited subgrade frost penetration

LVDT—linear variable differential transformer

m^2—square meter

MACOM—major command (Army)

MAJCOM—major command (Air Force)

m/day—meters per day

miles/hour—miles per hour

mm—millimeter

mm/day—millimeters per day

mm/sec—millimeters per second

MN/m^3—meganewton per cubic meter

MPa—megapascal

MPC—magnesium phosphate cement

m/second—meters per second

MTI—moving target indicator

N—newton

nac—number of aircraft

NASA—National Aeronautics and Space Administration

NAVAIDS—air navigation aids

NAVAIR—Naval Air Systems Command

NAVFAC—Naval Facilities Engineering Command

NDB—non-directional beacon

NEMA—National Electrical Manufacturers Association

NFS—non-frost-susceptible

No.—number

NOAA—National Oceanic and Atmospheric Administration

OGM—open-graded material

OLS—optical lighting system

PAPI—Precision Approach Path Indicator

PCASE—Pavement-Transportation Computer Assisted Structural Engineering

PCC—portland cement concrete

pcf—pounds per cubic foot

PCI—pavement condition index

pci—pounds per cubic inch

PFS—porous friction surface

PG—pavement grading

PI—penetration index

PI—plasticity index

psi—pounds per square inch

psi/in—psi per inch (or, pounds per cubic inch)

PVC—polyvinyl chloride pipe

RAP—recycled asphalt pavement

RCA—recycled concrete aggregate

RCCP—roller-compacted concrete pavement

RDM—rapid-draining material

RFP—request for proposal

RMP—resin modified pavement

RSS—reduced subgrade strength

RVR—runway visual range

SCI—structural condition index

SHRP—Strategic Highway Research Program

SI—International System of Units

SMA—stone mastic asphalt

TACAN—Tactical Air Navigation

TRR—tracking reference reflector

UFC—Unified Facilities Criteria

UFGS—Unified Facilities Guide Specifications

U.S.—United States

USAASA—U.S. Army Aeronautical Services Agency

USAAVNC—U.S. Army Aviation Center

USACE—U.S. Army Corps of Engineers

USACE-TSC—U.S. Army Corps of Engineers Transportation Systems Center

USASC—U.S. Army Safety Center

USACRC—U.S. Army Combat Readiness Center

USCS—Unified Soil Classification System

USGS—United States Geological Survey

VASI—Visual Approach Slope Indicator

VFR—visual flight rules

VMC—visual meteorological conditions

VOR—Very High Frequency Omnidirectional Range

VORTAC—co-located VOR and TACAN

WWW—World Wide Web

yd^2—square yard

APPENDIX A

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APPENDIX B BEST PRACTICES

SECTION 1: AIRFIELD/HELIPORT DESIGN ANALYSIS OUTLINE

B1-1 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 (e.g., new construction; runway extension; apron expansion; overlay; rehabilitation and repair; upgrade lighting; drainage, security, and navigational aids improvements)

(2) Purpose of proposed construction or improvements

(3) Types and amount of construction activities (e.g., demolition, excavation and embankment, grading, paving, patching, marking, fencing, seeding)

d. Project Authorization. Reference the 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. Key criteria and directives may include:

(1) Correspondence and Directives

(2) Engineering Technical Letters (ETLs)

(3) Technical Manuals (TMs and AFMs)

(4) Unified Facilities Criteria (UFC)

(5) Engineering Circulars (ECs)

(6) Pavement Evaluations/Condition Surveys

(7) Computer Programs

(8) Other Special Design Criteria

B1-2 SITE DESCRIPTION

a. Location (location map with graphical scale)

(1) Existing airfield/heliport facilities (e.g., layout, type)

(2) Location of proposed project with respect to existing facilities, utilities, or improvements

(3) Extent of proposed construction (e.g., size, dimensions)

b. Topography/Drainage of Site

(1) Topography (e.g., hilly, rolling, flat, terrace, floodplain)

(2) Surface drainage (characteristics and direction)

(3) Subsurface drainage (characteristics, groundwater conditions and elevations, including seasonal variations)

(4) Existing surface and subsurface drainage facilities (e.g., type, location, capacity, condition)

c. Climate. Use the National Oceanographic and Atmospheric Administration (NOAA) or the 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 (e.g., glacial, stream, loess deposits)

(2) Rock outcroppings

B1-3 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

B1-4 TESTING

a. Laboratory. Describe lab testing conducted.

b. Field. Describe all field testing conducted.

B1-5 RESULTS OF INVESTIGATIONS AND TESTING

a. Material Characterization

(1) Subgrade characteristics (e.g., soil classifications, unit weights, moisture-density relationships, gradations, Atterberg limits, CBR or modulus of subgrade reaction or both, permeability)

(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 (e.g., based on gradation and frost classification, laboratory freeze tests, heave measurements, observations or ice lense formations in test pits)

(2) Frost penetration (based on field observations or design air freezing index and modified Berggren 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. Design Parameters. Summarize the adopted design parameters.

B1-6 **PAVEMENT THICKNESS DESIGN CRITERIA**

a. Load. Include a copy of the 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

B1-7 **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 (e.g., thicknesses, types)

(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 programs used

(5) Layers (number, thickness, type)

(6) Special subgrade, subbase, and base course preparation for frost design

B1-8 **DRAINAGE DESIGN**

a. General Criteria

b. Hydrology

c. Surface Drainage (including drainage plans and profiles)

d. Subsurface Drainage

e. Location of Fueling Aprons and Required Drainage Facilities

f. Location of Deicing Aprons and Required Drainage Facilities

g. Location and Description of Future Airfield Use within Project Drainage Area

h. Location of any Temporary Ponding

i. Aircraft Wheel Loads for Inlet/Manhole Design

j. Drainage Areas Contiguous to Project Area that Contribute Storm Flow to Project

B1-9 PROPOSED GRADES

a. Longitudinal (for each pavement feature)

b. Transverse (for each pavement feature)

B1-10 AIR NAVIGATION AIDS (NAVAIDS)

a. Airfield Operational Category. Visual and electronic NAVAID requirements are based on the runway operational category. Designers shall specify one of the following operational categories for the project. If the airfield has more than one runway, the operational category shall be specified for each runway that is included in the project.

(1) Night visual meteorological conditions (VMC)

(2) Non-Precision

(3) Category I

(4) Category II

(5) Category III

b. Visual NAVAIDS

(1) Required Visual NAVAIDS. Airfield lighting, signage, and marker systems shall include all required facilities in conformance with UFC 3-535-01 Table 2-1A, Table 2-1B, and Table 2-2 for the project airfield operational category. Designers shall list each individual type, location, and extent of the required visual NAVAIDS (Approach Aids, Runway Aids, Taxiway Aids and Miscellaneous Aids), as detailed in the UFC 3-535-01 tables.

(2) Optional Visual NAVAIDS. Optional visual NAVAIDS listed in UFC 3-535-01 Table 2-1A, Table 2-1B, and Table 2-2 may be included in the project. Designers shall clearly identify which optional items are required in this project.

(3) Designers shall ensure that these additional items are addressed in the design report:

(a) Precision Approach Path Indicator (PAPI). If a PAPI is included in the project, identify the height group of aircraft for design.

(b) Airfield Signs and Markers. All signs must be frangible. Signs shall be lighted if nighttime operations are to be conducted. Identify sign sizes and sign lighting requirements.

c. Electronic NAVAIDS. Military electronic NAVAID equipment within the United States must meet FAA standards. Overseas facilities generally meet FAA requirements unless superseded by Base Rights Agreements (see UFC 3-535-01, paragraph 1-6). Designers shall list all electronic NAVAIDS, including all major components that are included in the project together with the appropriate siting and installation references. Additional information about several of the most common electronic NAVAIDs follows.

(1) Instrument Landing System (ILS)

(a) ILS Components: Localizer antenna, equipment and shelter; glide slope antenna, equipment and shelter, critical area grading. Markers and monitors are required as indicated in this table:

MARKER OR MONITOR	OPERATIONAL CATEGORY		
	CAT I	CAT II	CAT III
Inner Marker		R**	R
Middle Marker		R	R
Outer Marker	R*	R	R
Near Field Monitor		R	R
Far Field Monitor		R	R

R indicates required. *Distance measuring equipment (DME) or tactical air navigation (TACAN) can be used for a final approach fix in lieu of the outer marker. **Required for operations below runway visual range (RVR) 1600 ft

(b) ILS Siting. ILS siting shall conform to FAA Order 6750.16D. Marker beacons shall conform to FAA Order 6750.24D, paragraphs 7d and 7c.

(2) Precision Approach Radar (PAR)

(a) PAR Components: Equipment shelter and antenna, tracking reference reflector (TRR) and moving target indicator (MTI) (each runway serviced by the PAR should have TRR and MTI reflectors), power source and standby generator

(b) PAR Siting. PAR siting shall conform to FAA Order 7031.2C.

(3) Non-Directional Beacon (NDB)

(a) NDB Components: Equipment shelter and transmitter, monitor, antenna coupler and antenna, power source and standby generator

(b) NDB Siting. NDB siting shall conform to FAA Order 7031.2C.

(4) VORTAC (co-located VOR and TACAN)

(a) VORTAC Components:

- Very High Frequency Omnidirectional Range (VOR): Equipment shelter, transmitter, monitor and conical shaped antenna

- Tactical Air Navigation (TACAN): Equipment shelter, transmitter, receiver, monitor and rotating antenna

(b) VORTAC Siting. VORTAC siting shall conform to FAA Order 6820.10.

d. Power Supply

(1) Source: Specify air base system or commercial utility source.

(2) Voltage: Specify primary voltage and appropriate transformation.

(3) Standby Power: Specify standby power as required for electronic NAVAIDS.

e. NAVAID Controls. Specify operational controls, brightness controls, status indication, and alarm for each visual NAVAID as required for electronic NAVAIDS.

f. Airfield Electrical Systems

(1) Constant Current Series Circuits. Identify the lighting systems connected to each circuit.

(2) Constant Current Regulators. Identify all lighting and NAVAIDS to be served by new regulators. List the location of new regulators. Include calculations for all regulator sizing. (See UFC 3-535-01, Table 15-4.)

(3) Transformers. Identify all transformers. Include calculations for all transformer sizing (excluding light isolation transformers).

(4) Cabling and Ductwork. Include calculations for cable sizing. Identify the location, size, and number of spare ducts required in all pavement crossings.

B1-11 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 (rapid-draining material (RDM) or open-graded material (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

g. Recycling

(1) List any proposed use of recycled materials.

(2) List percentages of recycled materials in any pavement mix.

B1-12 **LIST OF REQUIRED WAIVERS**

a. Reference regulation document (title, page, paragraph).

b. State the regulation in violation.

c. State the reason the waiver is required.

B1-13 **COST ESTIMATES**

a. Capital costs

b. Life-Cycle costs

B1-14 **LIST OF GOVERNMENT-FURNISHED EQUIPMENT**

a. Aircraft Arresting Gear

b. Electronic NAVAIDS

c. Other

B1-15 **APPLICABLE OBSTRUCTION SURVEYS, PROFILES**

a. Light Plane Profiles for ALSF-1, ALSF-2, MALSR

b. PAPI Clearance Plane and Approach Plane

c. Clear Zone Obstruction Profiles

d. Part 77 Obstruction Surveys

SECTION 2: RECOMMENDED CONTRACT DRAWING OUTLINE FOR AIRFIELD/HELIPORT PAVEMENTS

Note: This list of drawings should be used as a guide. All drawings may not be needed for all jobs.

B2-1 **TITLE SHEET**

a. Project Title

b. Location

c. Year

d. Volume Number

B2-2 **INDEX SHEET**

a. Listing of Sheet Names

b. Assigned Sheet Numbers (in sequential order)

B2-3 **COMBINED TITLE/INDEX SHEETS**

B2-4 **LEGEND**

a. Civil

b. Electrical

c. Mechanical

d. Architectural

B2-5 **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)

B2-6 **PHASING PLAN AND DETAILS**

a. Location and Sequencing of Work Areas

b. Scheduling for Each Phase of the 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

B2-7 **HORIZONTAL AND VERTICAL CONTROLS**

a. Layout

b. Bench Marks (United States Geological Survey (USGS) Datum) with Only One Master Bench Mark

c. Control Stationing

d. Horizontal Control (Coordinates)

B2-8 **GEOMETRIC LAYOUT PLAN (OPTIONAL)**

a. Curve Data

b. Control Stationing

c. Geometric Layout

B2-9 **BORING LOCATION PLAN AND BORING LOG DATA**

B2-10 **PAVEMENT REMOVAL PLAN**

a. Pavement Removal Limits (e.g., Dimensions, Stationing)

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

B2-11 **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 Composition of the Existing Pavement

(1) Pavement Type and Thickness

(2) Joint Type (e.g., Doweled, Tied, Contraction)

(3) Existing Reinforcing (if any)

c. Special Notes

(1) Equipment Type/Size

(2) Procedures

(3) Housekeeping

(4) Other

B2-12 EXISTING UTILITIES PLAN

a. Show Existing Utility Locations and Type

b. Show Pavement Penetrations

B2-13 PAVING PLAN

a. Thickness

b. Type

c. Location

d. Location of Section Cuts

e. Stationing

f. Dimensions

B2-14 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 of work by the field engineer.

a. Paving Section. 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

B2-15 **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

B2-16 **GRADING AND DRAINAGE PLANS**

a. Contours (new and existing)

b. Surface and Subsurface Drainage System Layouts, Structure Locations, Types, and Sizes

c. Ditch Alignment

B2-17 **GRADING SECTIONS**

a. Cut/Fill Requirements

b. Topsoil Requirements

B2-18 **PAVEMENT SURFACE ELEVATIONS**

a. Spot Elevation Plan (Joint Intersections or Grid Pattern)

b. Spot Elevation Schedule

B2-19 **PAVEMENT JOINTING PLANS**

a. Legend with Joint Types

b. Joint Location

B2-20 **JOINT AND JOINT SEALANT DETAILS**

B2-21 **REINFORCING DETAILS**

a. Dowels

b. Reinforcement

c. Tie Bars

d. Complete Pavement Joint Details

B2-22 **SURFACE AND SUBSURFACE DRAINAGE SYSTEMS**

a. Profiles

b. Schedules

c. Details

B2-23 **AIRFIELD REPAIR PLAN AND DETAILS**

B2-24 **PAVEMENT MARKING**

a. Plan

b. Details

B2-25 **AIRCRAFT MOORING AND GROUNDING POINTS**

a. Plan

b. Details

B2-26 **GROOVING PLAN AND DETAILS**

B2-27 **RUNWAY/TAXIWAY LIGHTING**

a. Plan

b. Schedule

c. Details

B2-28 **MECHANICAL (FUEL)**

a. Plans

b. Profiles

c. Schedules

d. Details

SECTION 3: WAIVER PROCESSING PROCEDURES

B3-1 ARMY

B3-1.1 Waiver Procedures

B3-1.1.1 **Installation.** The installation's design agent, aviation representative (Safety Officer, Operations Officer, and/or Air Traffic and Airspace [AT&A] Officer) and Directorate of Engineering and Housing (DEH) Master Planner will:

B3-1.1.1.1 Jointly prepare/initiate waiver requests.

B3-1.1.1.2 Submit requests through the installation to the major command (MACOM).

B3-1.1.1.3 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.

B3-1.1.2 **MACOM.** The MACOM will:

B3-1.1.2.1 Ensure that all required coordination has been accomplished.

B3-1.1.2.2 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. 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. Follow-up inspections will be necessary to ensure that mitigative actions proposed for each Temporary Waiver granted have been accomplished.

B3-1.1.2.3 Review waiver requests and forward all viable requests to the U. S. Army Aeronautical Services Agency (USAASA) for action. To expedite the waiver process, MACOMs are urged to simultaneously forward copies of the request to:

- Commander, U. S. Army Aeronautical Services Agency (USAASA), ATTN: ATAS-AI, 9325 Gunston Road, Suite N319, Fort Belvior, VA 22060-5582.
- Commander, U.S. Army Safety Center (USASC), ATTN: CSSC-SPC, Bldg. 4905, 5th Ave., Fort Rucker, AL 36362-5363.
- Commander, U. S. Army Aviation Center (USAAVNC), ATTN: ATZQ-ATC-AT, Fort Rucker, AL 36362-5265.
- Director, USACE Transportation Systems Center (USACE-TSC), ATTN: CENWO-ED-TX, 215 N 17th St., Omaha, NE 68102.

B3-1.1.3 **USAASA.** USAASA is responsible for coordinating the following reviews for the waiver request:

B3-1.1.3.1 Safety and risk assessment by U.S. Army Combat Readiness Center (USACRC).

B3-1.1.3.2 Technical engineering review by USACE-TSC.

B3-1.1.3.3 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.

B3-1.2 **Contents of Waiver Requests.** Each request must contain this information:

B3-1.2.1 Reference to the specific standard and/or criterion to be waived by publication, paragraph, and page.

B3-1.2.2 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. This specific justification for waivers to criteria and allowances must be included:

B3-1.2.2.1 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 percent of the airfield operations.

B3-1.2.2.2 When deviations 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.

B3-1.2.2.3 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.

B3-1.2.3 The rationale for the waiver request, including specific impacts on the assigned mission, safety, and/or the environment.

B3-1.3 **Additional Requirements**

B3-1.3.1 **Operational Factors.** Include information on all of these existing and/or proposed operational factors that were used in the assessment:

B3-1.3.1.1 Mission urgency

B3-1.3.1.2 All aircraft by type and operational characteristics

B3-1.3.1.3 Density of aircraft operations at each air operational facility

B3-1.3.1.4 Facility capability (VFR or IFR)

B3-1.3.1.5 Use of self-powered parking versus manual parking

B3-1.3.1.6 Safety of operations (risk management)

B3-1.3.1.7 Existing NAVAIDS

B3-1.3.2 **Documentation.** Record all alternatives considered, their consequences, necessary mitigative efforts, and evidence of coordination.

B3-2 AIR FORCE

B3-2.1 **Waivers to Criteria and Standards.** Waivers to criteria and standards in this publication must be approved by the MAJCOM pavements engineer.

B3-2.1.1 **Waiver Procedure.** The design agent or, if designed by the Air Force, the base pavements engineer, prepares a request for waiver for each project. The request must contain a complete listing of all deviations from criteria and standards, including justification. If the base civil engineer concurs, the request is forwarded to the MAJCOM pavements engineer for consideration.

B3-3 NAVY AND MARINE CORPS

B3-3.1 Applicability

B3-3.1.1 **Use of Criteria.** The criteria in this UFC 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 UFC 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.

B3-3.1.2 **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 that may not conform to the criteria in this UFC. Safety clearances at existing aviation facilities need not be upgraded solely for the purpose of conforming to these criteria; however, at existing aviation facilities where few structures have been constructed in accordance with previous safety clearances, it may be feasible to apply these revised standards.

B3-3.2 **Approval.** Approval from HQ NAVFAC must be obtained prior to revising safety clearances at existing airfield pavements to conform to the new standards in this

UFC. NAVFAC will coordinate the approval with the Naval Air Systems Command (NAVAIR) and Chief of Naval Operations or Command Master Chief (CNO/CMC) as required.

B3-3.3 **Obtaining a Waiver.** Once safety clearances have been established for an aviation facility, on occasions it may not be feasible to meet the designated standards. In these cases, a waiver must be obtained from NAVAIR. The waiver and its relation to the site approval process are defined in NAVFACINST 11010.45.

B3-3.4 **Exemptions from Waiver.** Certain navigational and operational aids normally are sited in violation of airspace safety clearances to operate effectively. The aids in paragraphs B3-3.4.1 through B3-3.4.8 are within this group and require no waiver from NAVAIR, provided they are sited in accordance with UFC 3-535-01.

B3-3.4.1 Approach lighting systems

B3-3.4.2 Visual Approach Slope Indicator (VASI) and PAPI systems

B3-3.4.3 Permanent optical lighting system (OLS), portable OLS, and Fresnel lens equipment

B3-3.4.4 Runway distance markers

B3-3.4.5 Arresting gear systems, including signs

B3-3.4.6 Taxiway guidance, holding, and orientation signs

B3-3.4.7 All beacons and obstruction lights

B3-3.4.8 Arming and de-arming pad

SECTION 4: DETERMINATION OF FLEXURAL STRENGTH AND MODULUS OF ELASTICITY OF BITUMINOUS CONCRETE

B4-1 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 PCC specimens.

B4-2 APPLICABLE STANDARDS

The standard applicable to this procedure is ASTM C78.

B4-3 APPARATUS

These apparatus are required:

- A testing machine capable of applying repetitive loadings for compaction of beam specimens 152 by 152 by 533 mm (6 by 6 by 21 in) to the design density. (An Instron® electromechanical testing machine meets this requirement.)
- A steel mold, suitably reinforced to withstand compaction of specimens without distortion
- Two linear variable differential transformers (LVDT)
- A 22,240-N (5,000-lb) load cell
- An X-Y recorder
- A testing machine for load applications conforming to ASTM C78. (A Baldwin or Tinius Olsen hydraulic testing machine is suitable for this purpose.)

B4-4 MATERIALS

Sufficient aggregate and bitumen meeting applicable specifications to produce six 152- by 152- by 533-mm (6- by 6- by 21-in) test specimens are required. If 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.

B4-5 SAMPLE PREPARATION

Follow these steps to prepare a test sample:

(1) Prepare in a laboratory mixer four portions of paving mixture for one 152- by 152- by 533-mm (6- by 6- by 21-in) 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-mm (6- by 6-in) 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 C (250±5 degrees F).

(2) Place two of the four portions in the 152- by 152- by 533-mm (6- by 6- by 21-in) reinforced steel mold and compact to a 76-mm (3-in) thickness with a 152- by 152-mm (6- by 6-in) foot attached to the repetitive loading machine. Shift the mold between load applications to distribute the compaction effort uniformly.

(3) Add the remaining two portions and continue compaction until the paving mixture is compacted to exactly a 152- by 152-mm (6- by 6-in) cross section.

(4) After compaction, place a 152- by 533-mm (6- by 21-in) steel plate on the surface of the paving mixture and apply a leveling load of 8,896 N (2,000 lb) to the plate. Prepare six beam test specimens in the manner described.

(5) After the specimens have cooled, remove the beams from the molds and rotate them 90 degrees so that the smooth, parallel sides will become the top and bottom.

(6) Cement an L-shaped metal tab with quick-setting epoxy glue to each 152- by 533-mm (6- by 21-in) side of the beams on the beams' neutral axes at midspan. The taps should be drilled for attachment of the LVDTs.

(7) Cure the beams at 10±1.7 degrees C (50±3 degrees F) for 4 days prior to testing.

B4-6 TEST PROCEDURES

Follow these steps to test the samples:

(1) Condition three specimens each at 10 and 24±1.7 degrees C (50 and 75±3 degrees F) for at least 12 hours prior to testing. If testing occurs immediately after curing the specimens at 10±1.7 degrees C (50±3 degrees F) for 4 days, no additional conditioning is required for the specimens tested at this temperature.

(2) Place the specimen in the test machine as described in ASTM C78.

(3) Place thin Teflon® strips at the point of contact between the test specimens and the load-applying and load-support blocks.

(4) 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.

(5) 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.

(6) Connect the LVDTs and load cell to the X-Y recorder.

(7) Make final adjustments and checks on the specimens and test equipment.

(8) Apply loading in accordance with ASTM C78, omitting the initial 4,448-N (1,000-lb) load.

B4-7 CALCULATIONS

B4-7.1 **Modulus of Rupture.** The modulus of rupture *R* is calculated from this equation (from ASTM C78):

$$R = \frac{PL}{bd^2} \tag{B4-1}$$

where

R = modulus of rupture, MPa (psi)

P = maximum applied load, N (lb)

L = span length, mm (in) (457 mm [18 in])

b = average width of beam, mm (in)

d = average depth (height) of beam, mm (in)

B4-7.2 **Modulus of Elasticity.** The modulus of elasticity *E* is calculated from this equation:

$$E = \frac{23PL^3}{1296\Delta I}\ k \tag{B4-2}$$

where

E = static Young's modulus of elasticity, MPa (psi)

P = applied load, N (lb)

L = span length, mm (in) (457 mm [18 in])

Δ = deflection of neutral axis, mm (in), under load, P

I = moment of inertia, mm^4 ($inch^4$) ($=bd^3/12$)

b = average width of beam, mm (in)

d = average depth (height) of beam, mm (in)

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.)

B4-8 **REPORT**

The report shall include this information:

- Gradation of aggregate
- Type and properties of bituminous cement
- Bituminous concrete mix design properties
- Bituminous concrete beam properties
- Modulus of rupture
- Modulus of elasticity

SECTION 5: CURVES FOR DETERMINING EFFECTIVE STRAIN REPETITIONS

B5-1 OVERVIEW

This appendix contains plots (Figures F-1 through F-22) for converting aircraft operations to effective repetitions of strain using the type of aircraft, the effective thickness of the pavement, and the offset from the center of the runway or taxiway.

B5-2 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. If the plots are not adequate, the computer program can be used to determine the conversion factors for any design situation. Chapter 1 provides information for accessing the computer program.

Figure B5-1. Effective Repetitions of the Strain for UH-60 Aircraft, Types B, C, and Secondary Traffic Areas

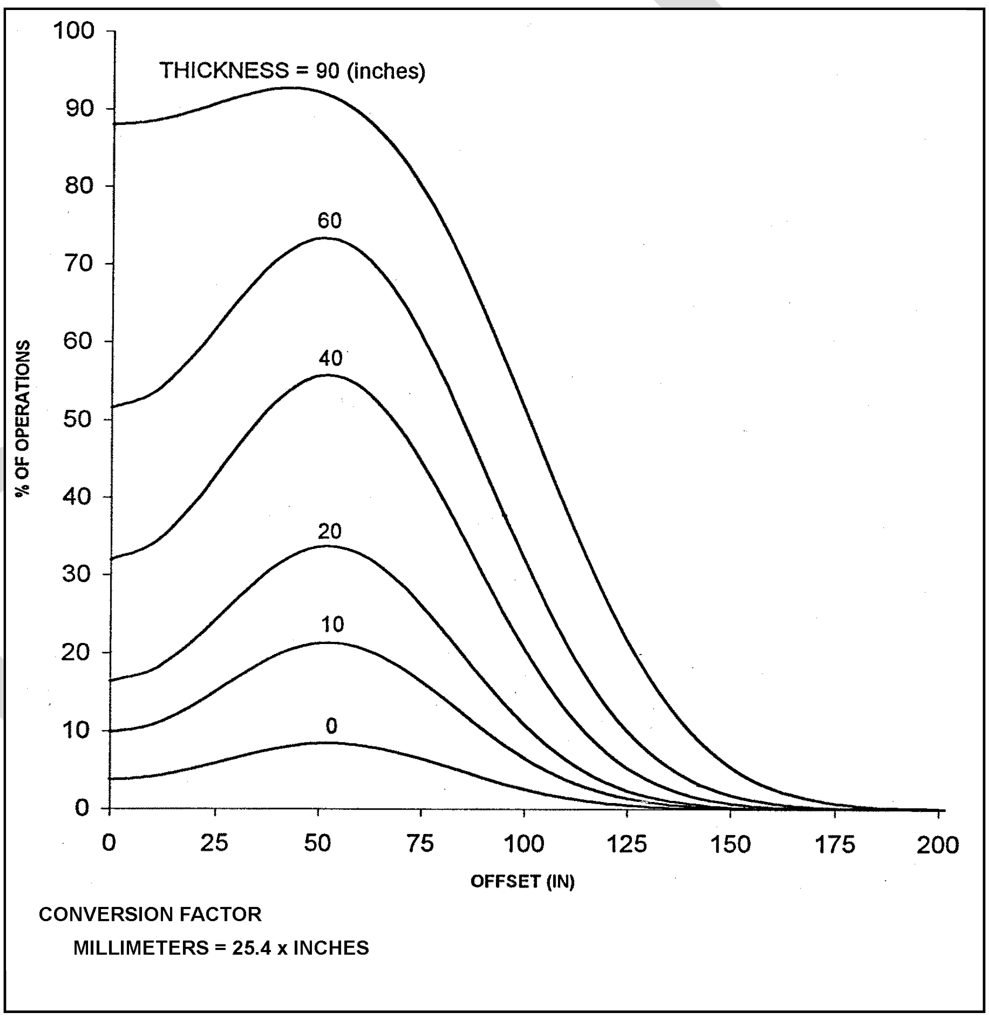
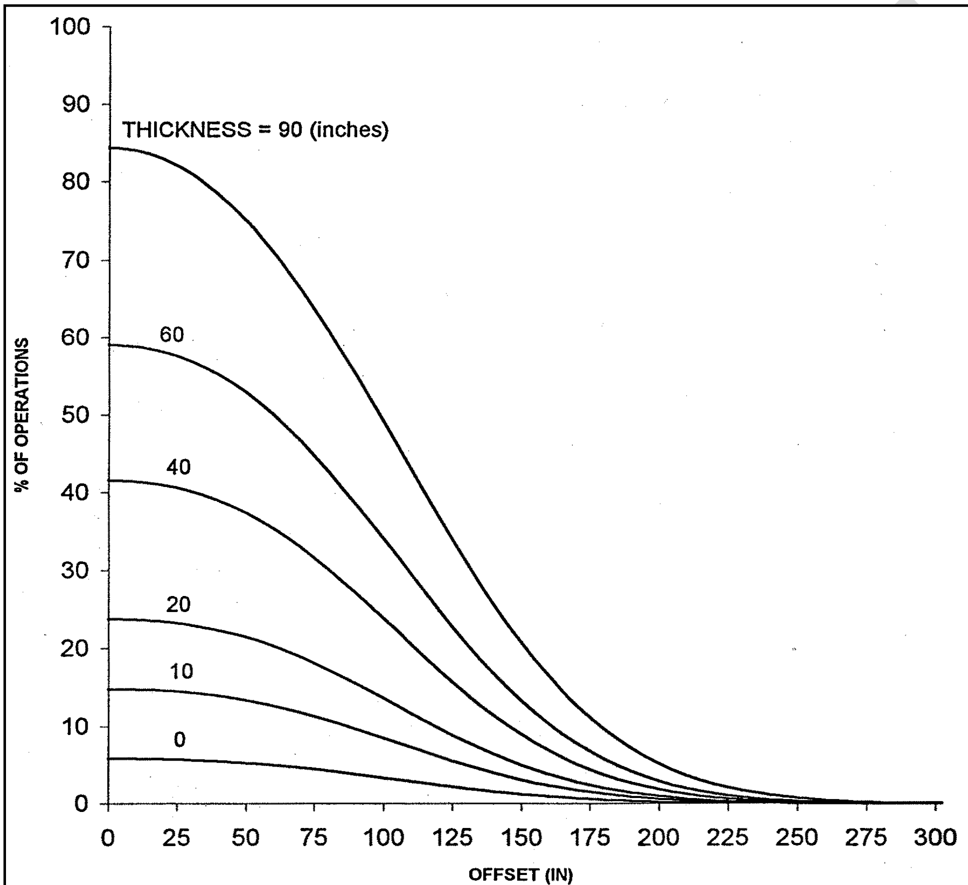


Figure B5-2. Effective Repetitions of Strain for UH-60 Aircraft, Type A or Primary Traffic Areas



CONVERSION FACTOR

MILLIMETERS = 25.4 x INCHES

Figure B5-3. Effective Repetitions of Strain for CH-47 Aircraft, Types B, C, or Secondary Traffic Areas

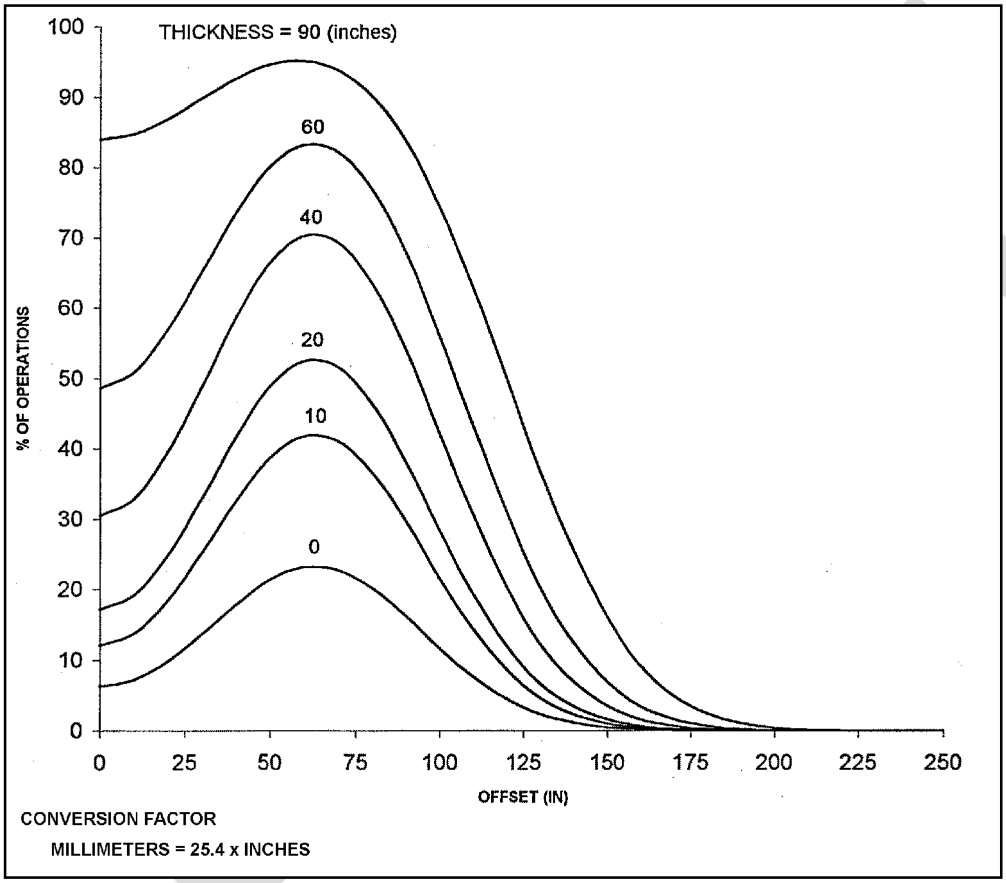


Figure B5-4. Effective Repetitions of Strain for CH-47 Aircraft, Type A or Primary Traffic Areas

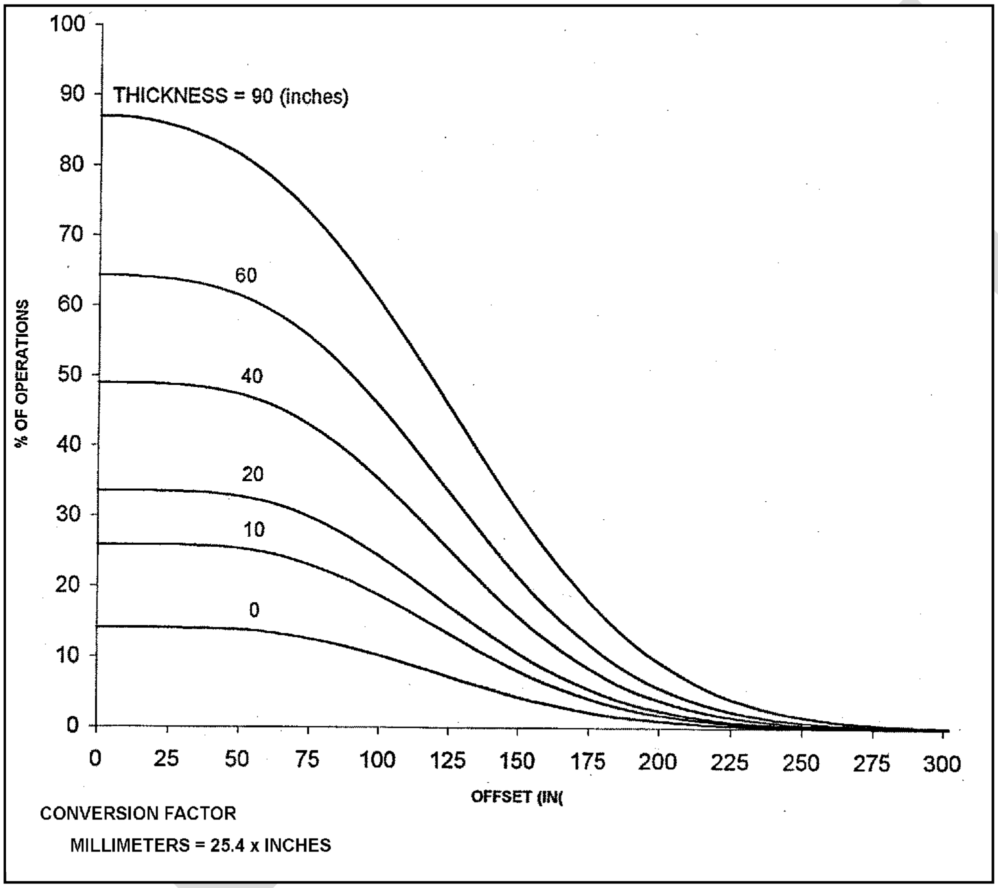


Figure B5-5. Effective Repetitions of Strain for OV-1 Aircraft, Types B, C, or Secondary Traffic Areas

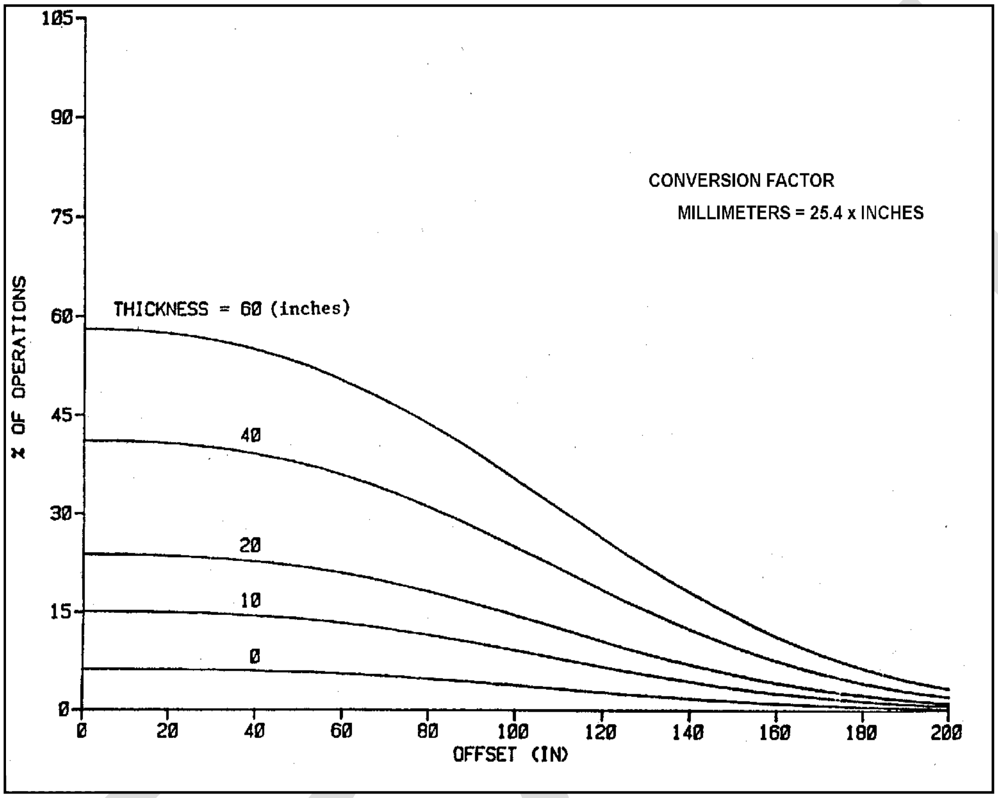


Figure B5-6. Effective Repetitions of Strain for OV-1 Aircraft, Type A or Primary Traffic Areas

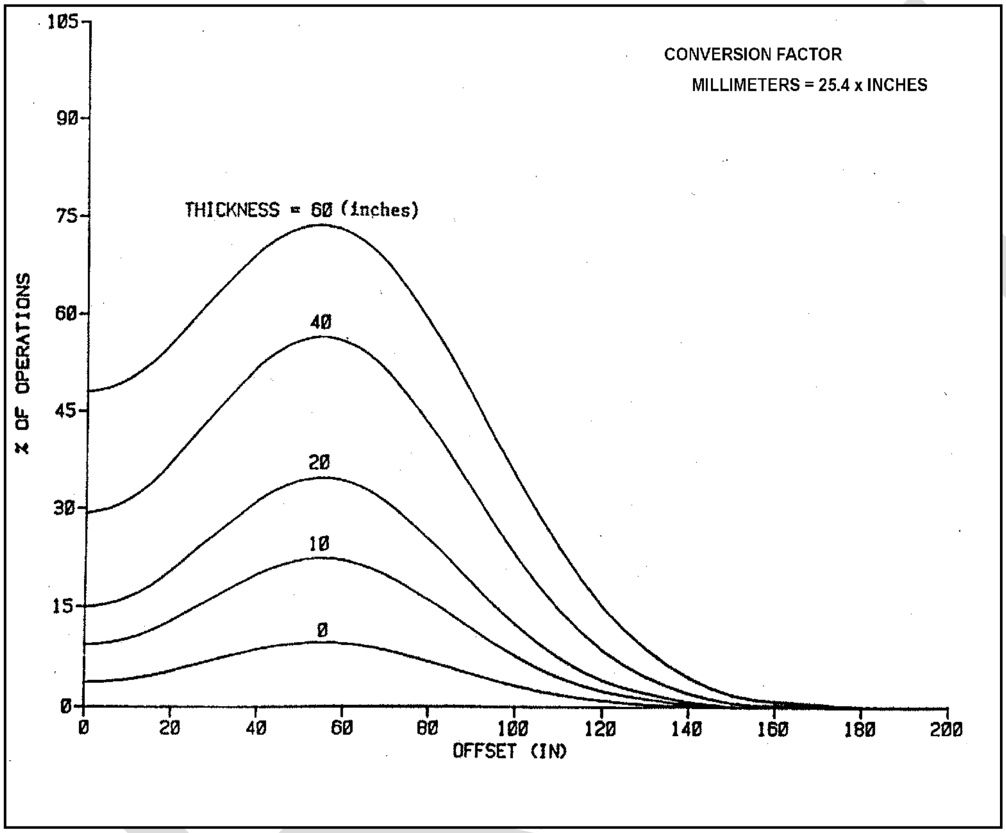


Figure B5-7. Effective Repetitions of Strain for C-12 Aircraft, Types B, C, or Secondary Traffic Areas

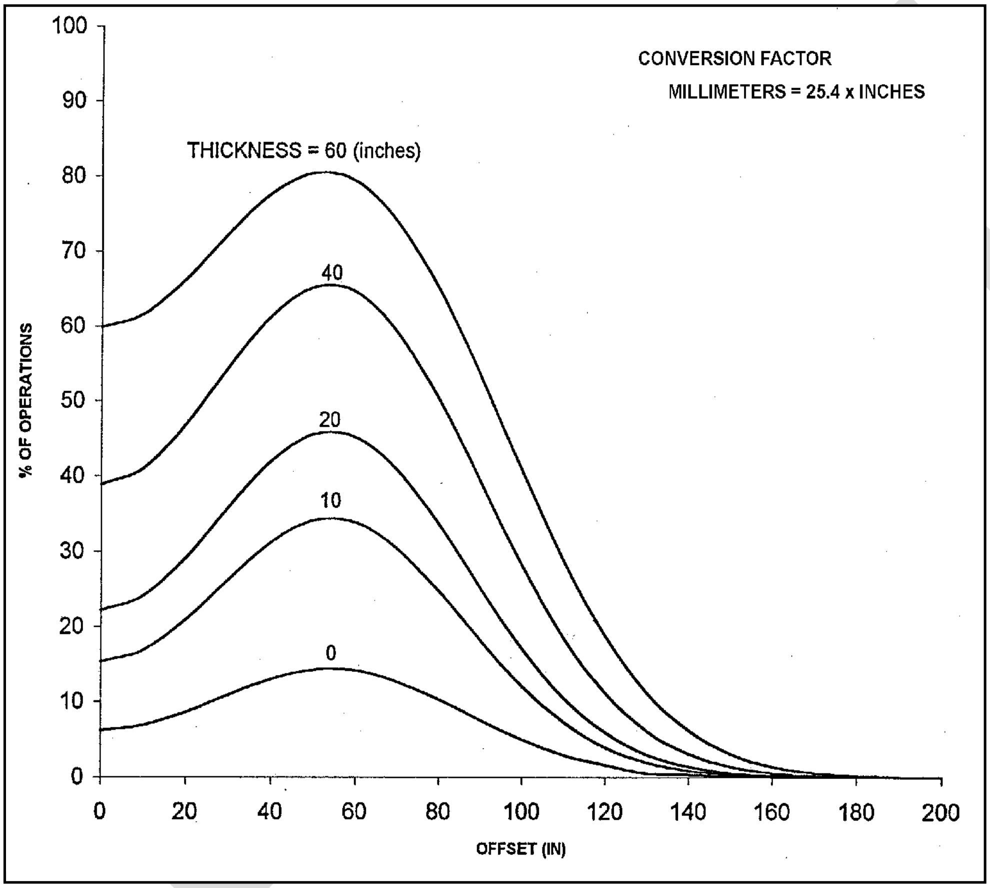


Figure B5-8. Effective Repetitions of Strain for C-12 Aircraft, Type A or Primary Traffic Areas

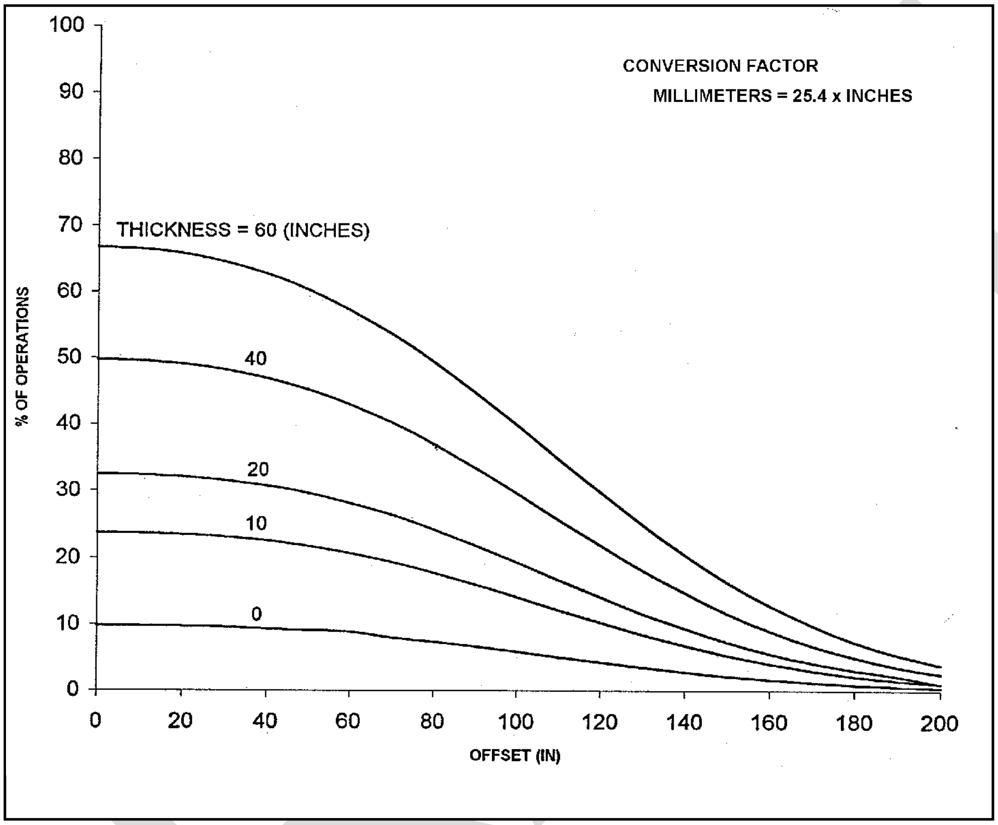


Figure B5-9. Effective Repetitions of Strain for C-130 Aircraft, Types B, C, or Secondary Traffic Areas

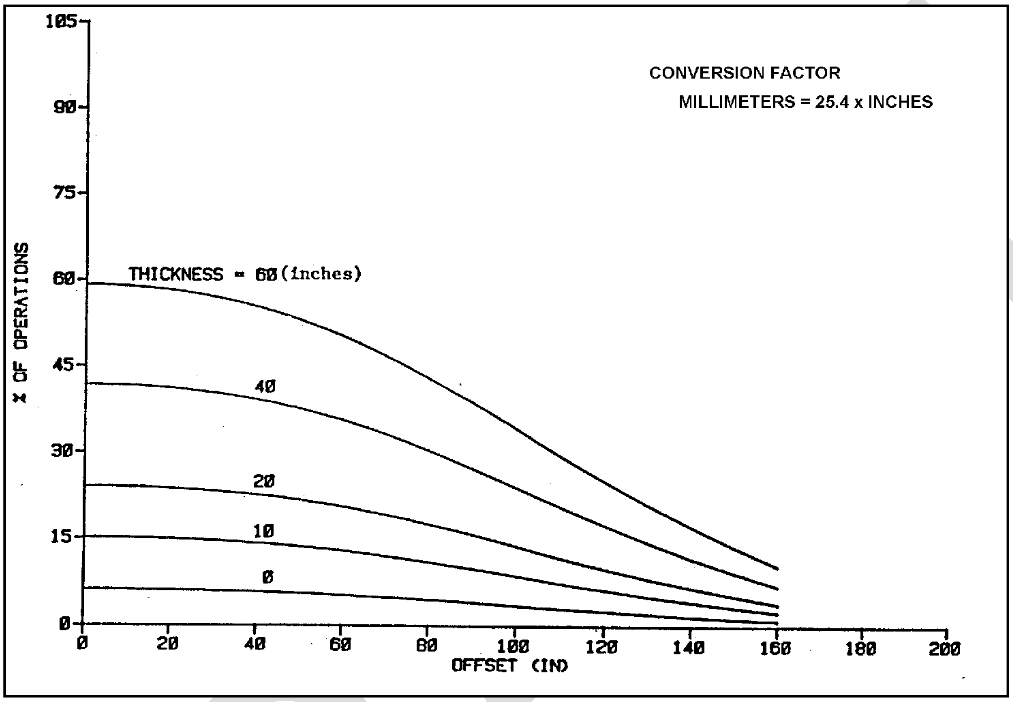


Figure B5-10. Effective Repetitions of Strain for C-130 Aircraft, Type A or Primary Traffic Areas

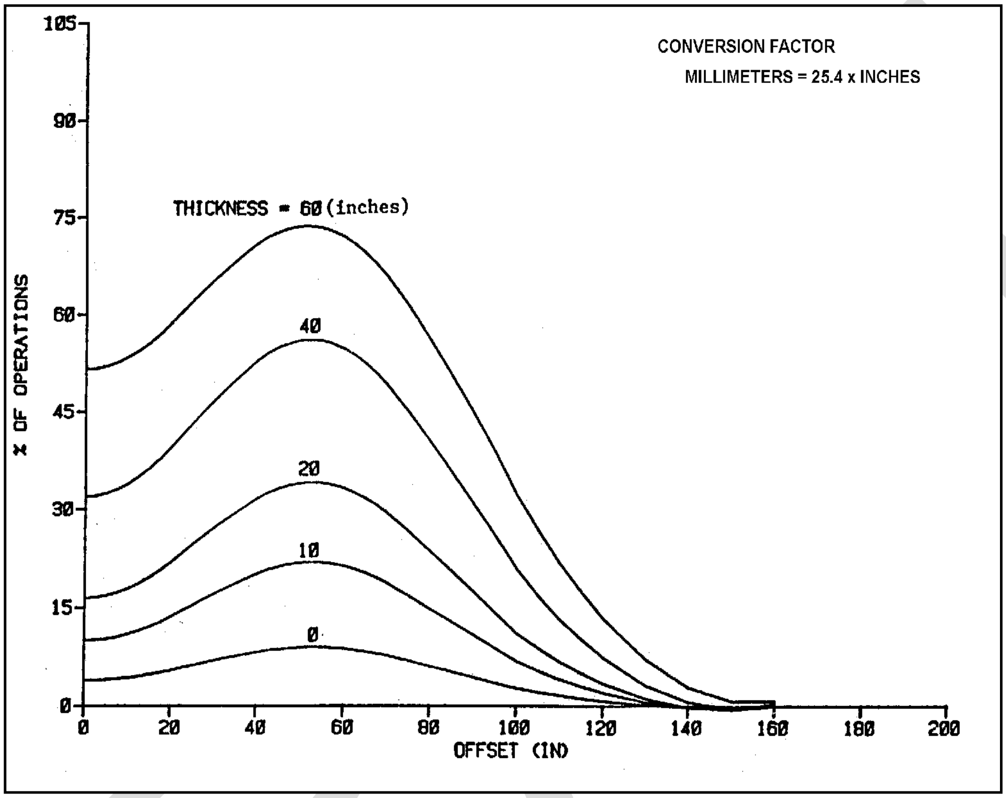


Figure B5-11. Effective Repetitions of Strain for F-15 Aircraft, Air Force Types B and C Traffic Areas

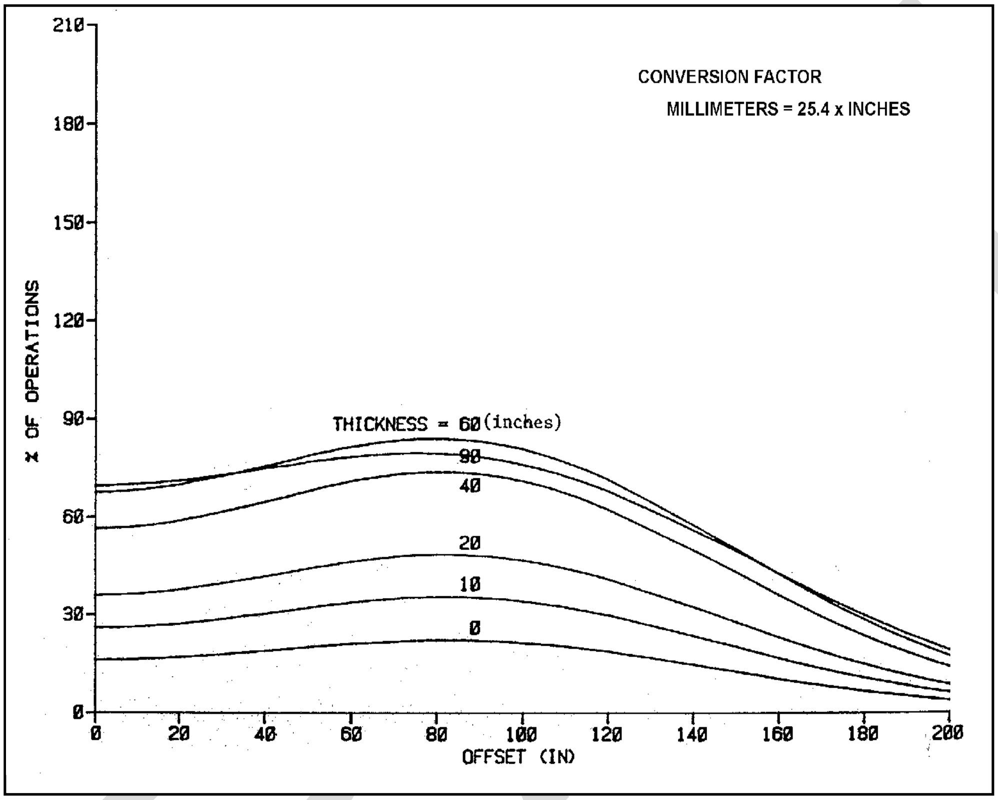


Figure B5-12. Effective Repetitions of Strain for F-15 Aircraft, Air Force Type A Traffic Areas

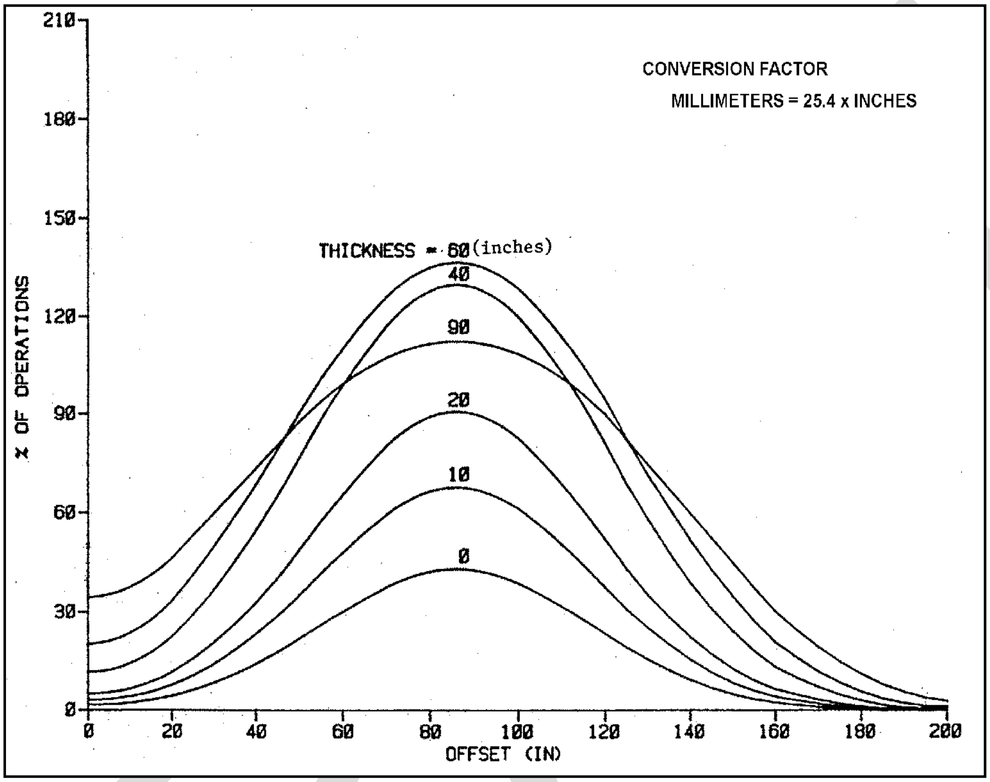


Figure B5-13. Effective Repetitions of Strain for F-14 Aircraft, Types B, C, and Secondary Traffic Areas

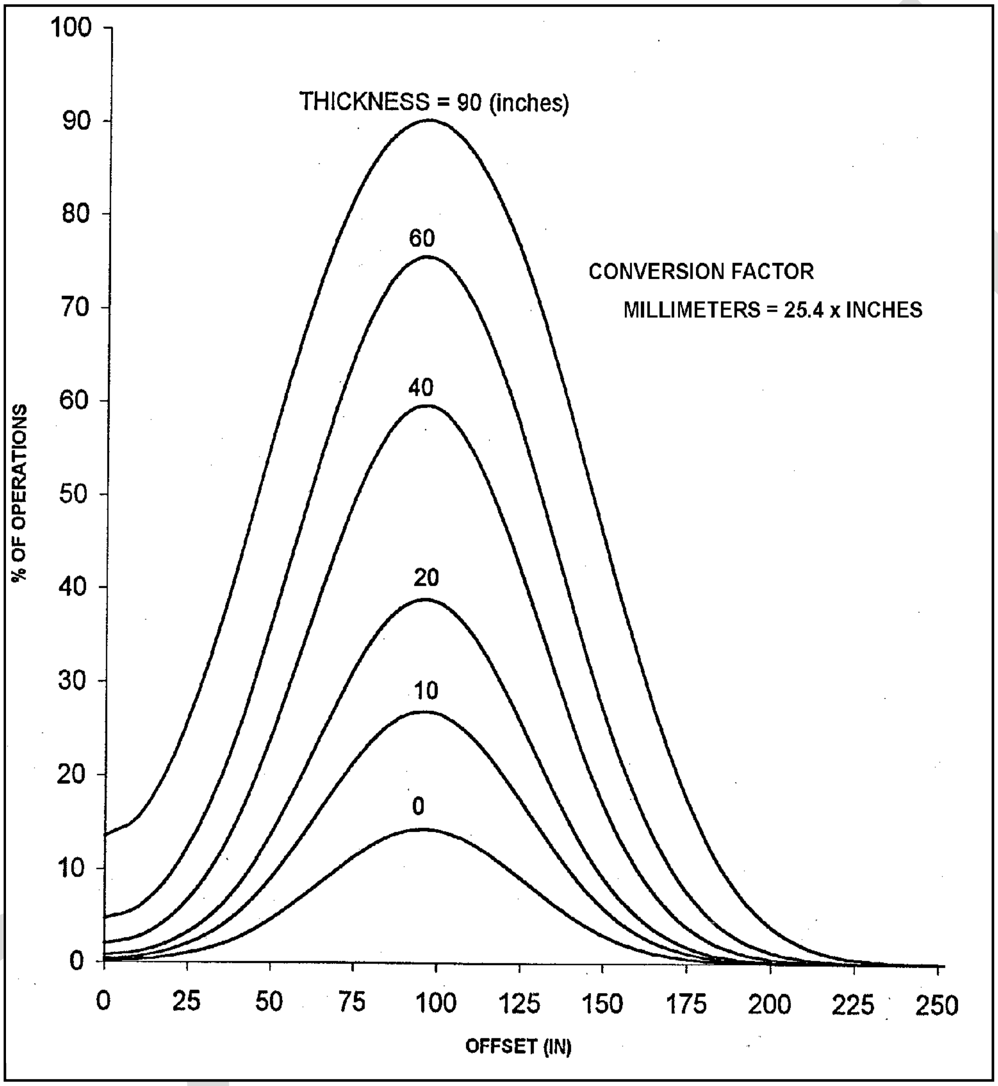


Figure B5-14. Effective Repetitions of Strain for F-14 Aircraft, Type A or Primary Traffic Areas

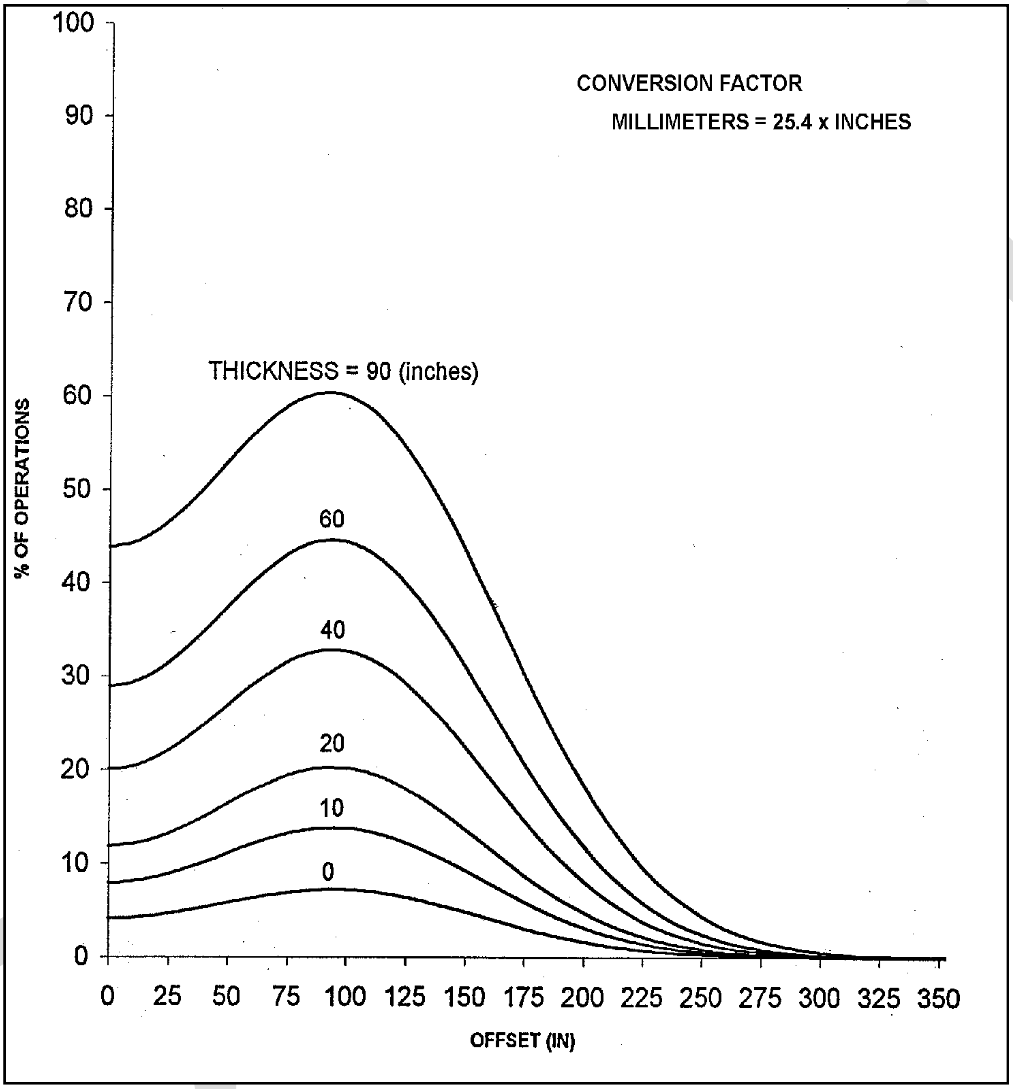


Figure B5-15. Effective Repetitions of Strain for B-52 Aircraft, Types B, C, or Secondary Traffic Areas

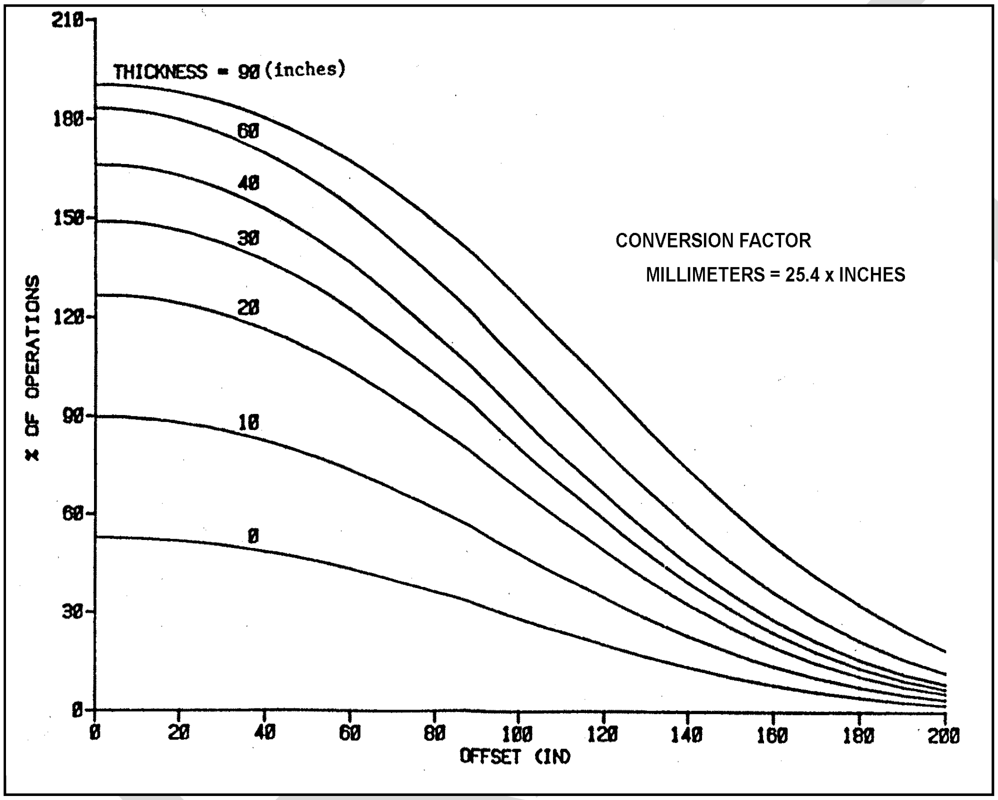


Figure B5-16. Effective Repetitions of Strain for B-52 Aircraft, Type A or Primary Traffic Areas

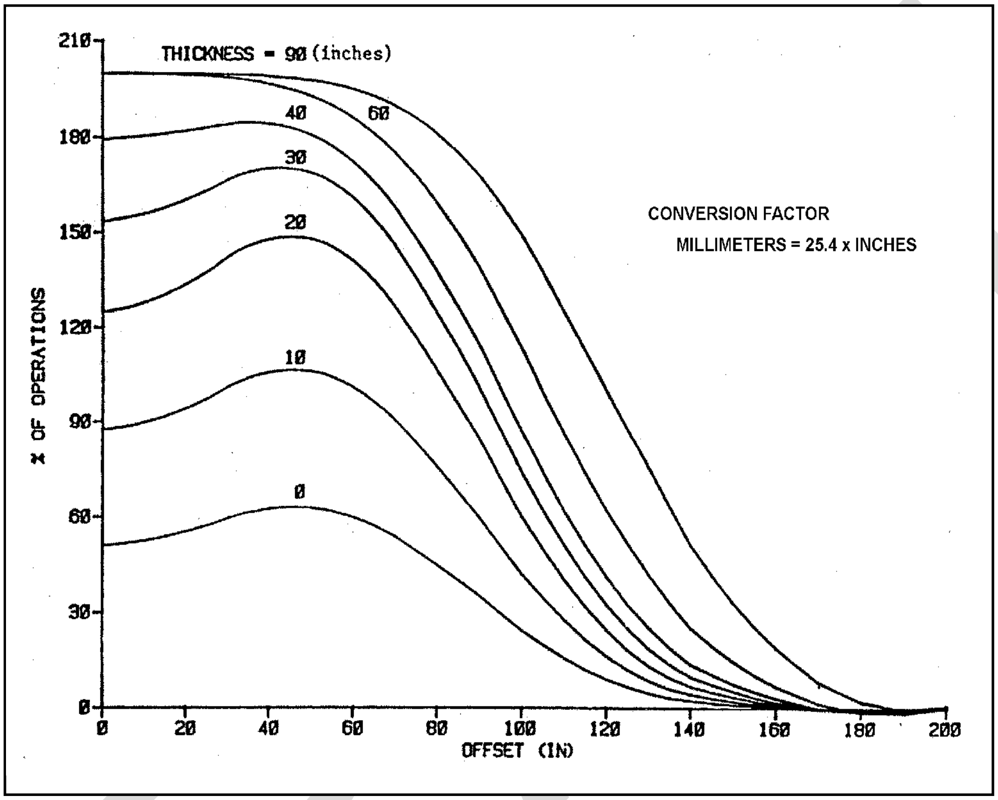


Figure B5-17. Effective Repetitions of strain for B-1 and C-141 Aircraft, Types B, C, or Secondary Traffic Areas

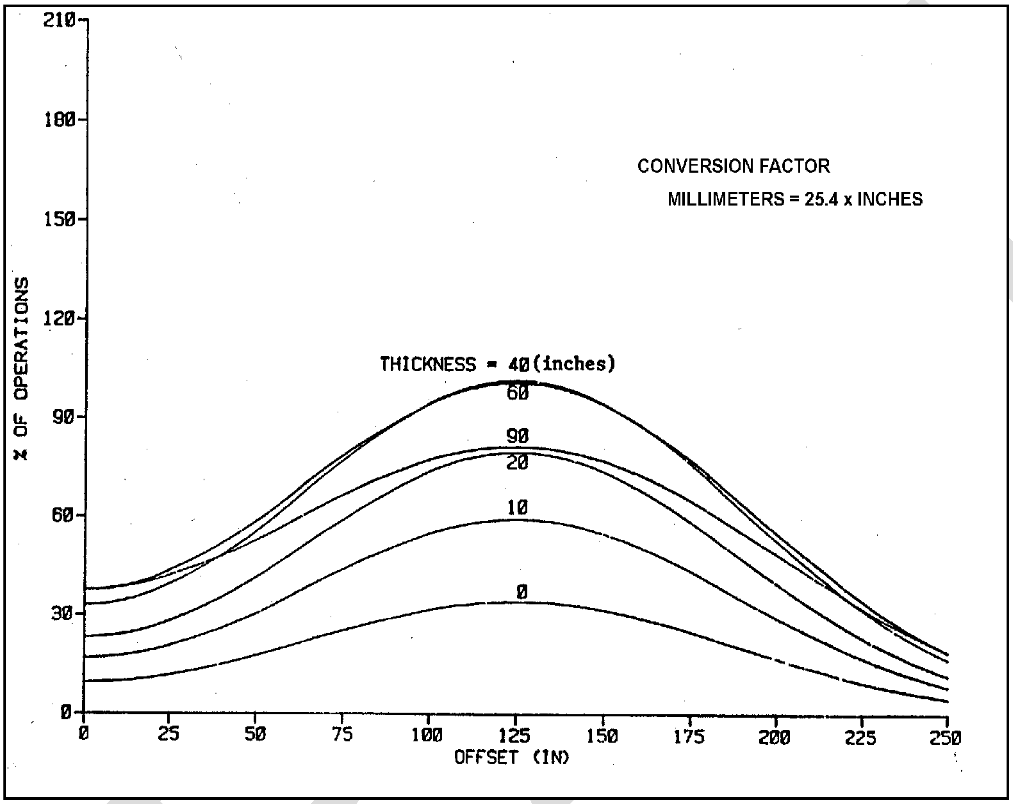


Figure B5-18. Effective Repetitions of Strain for B-1 and C-141 Aircraft, Type A or Primary Traffic Areas

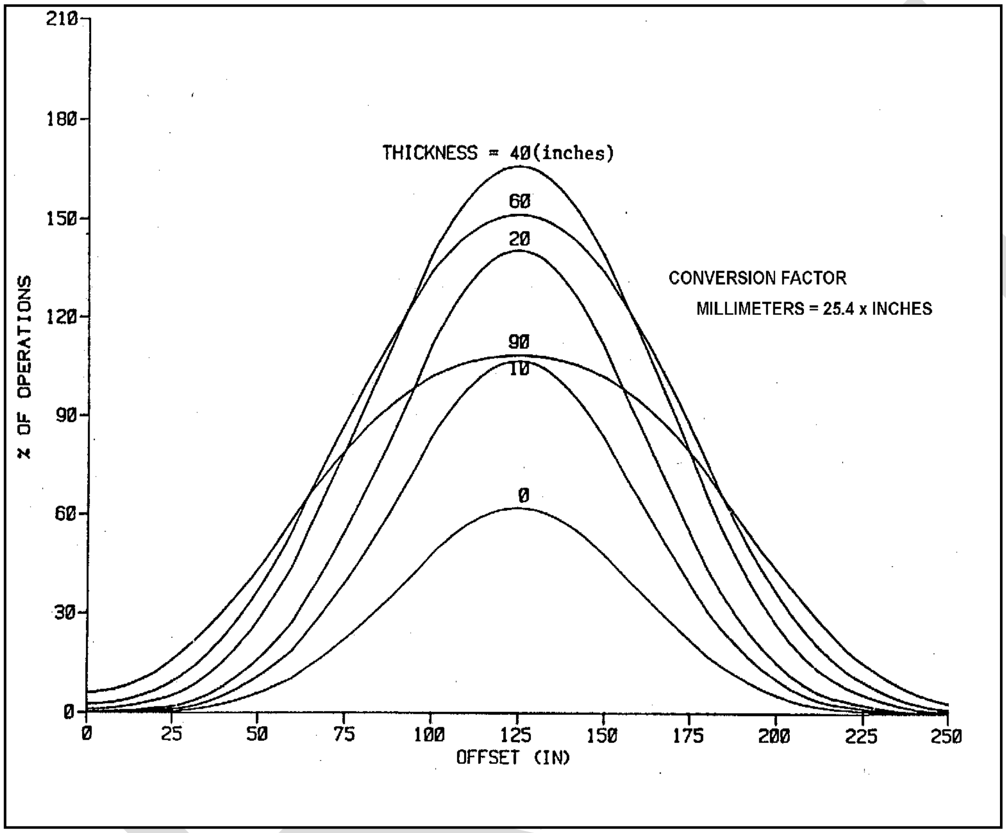


Figure B5-19. Effective Repetitions of Strain for P-3 Aircraft, Types B, C, or Secondary Traffic Areas

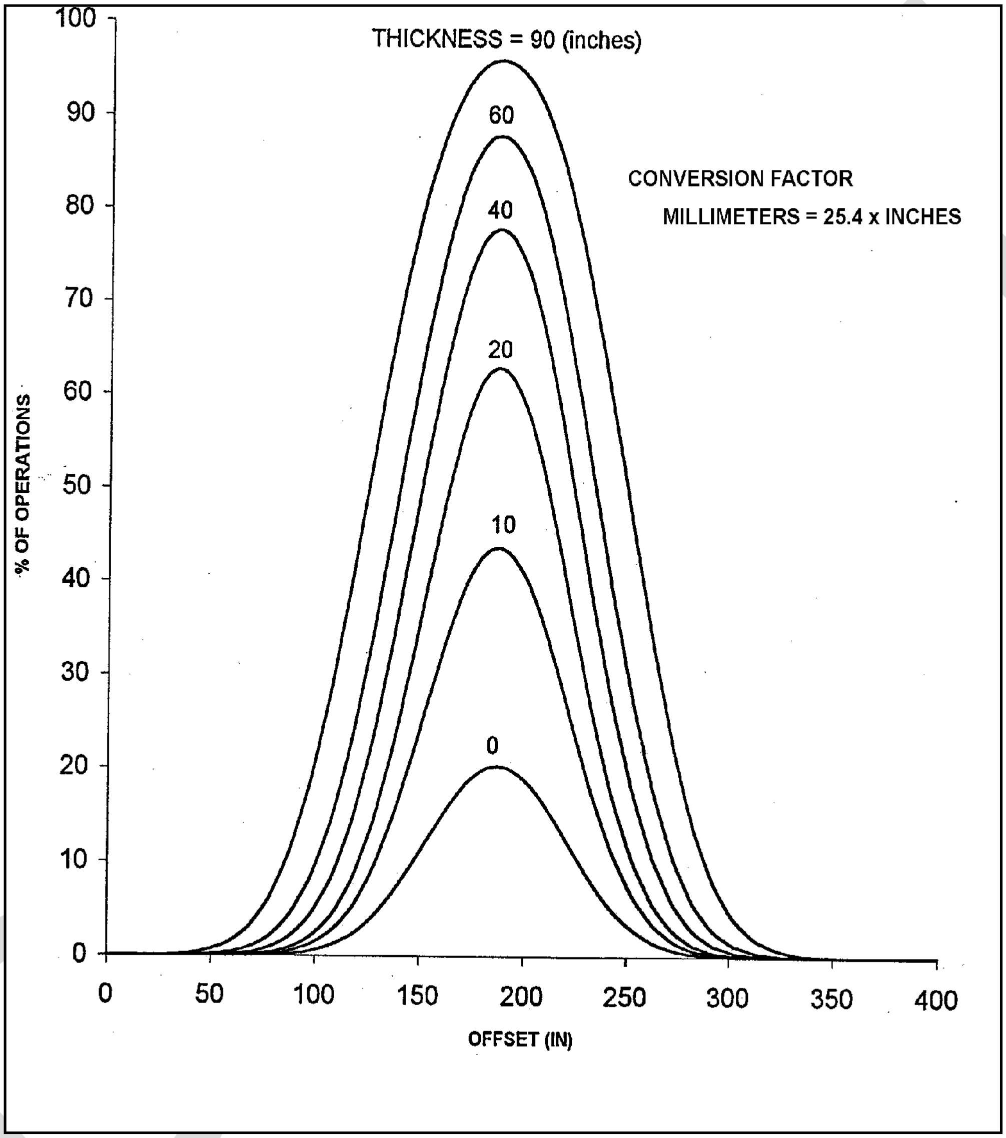


Figure B5-20. Effective Repetitions of Strain for P-3 Aircraft, Type A or Primary Traffic Areas

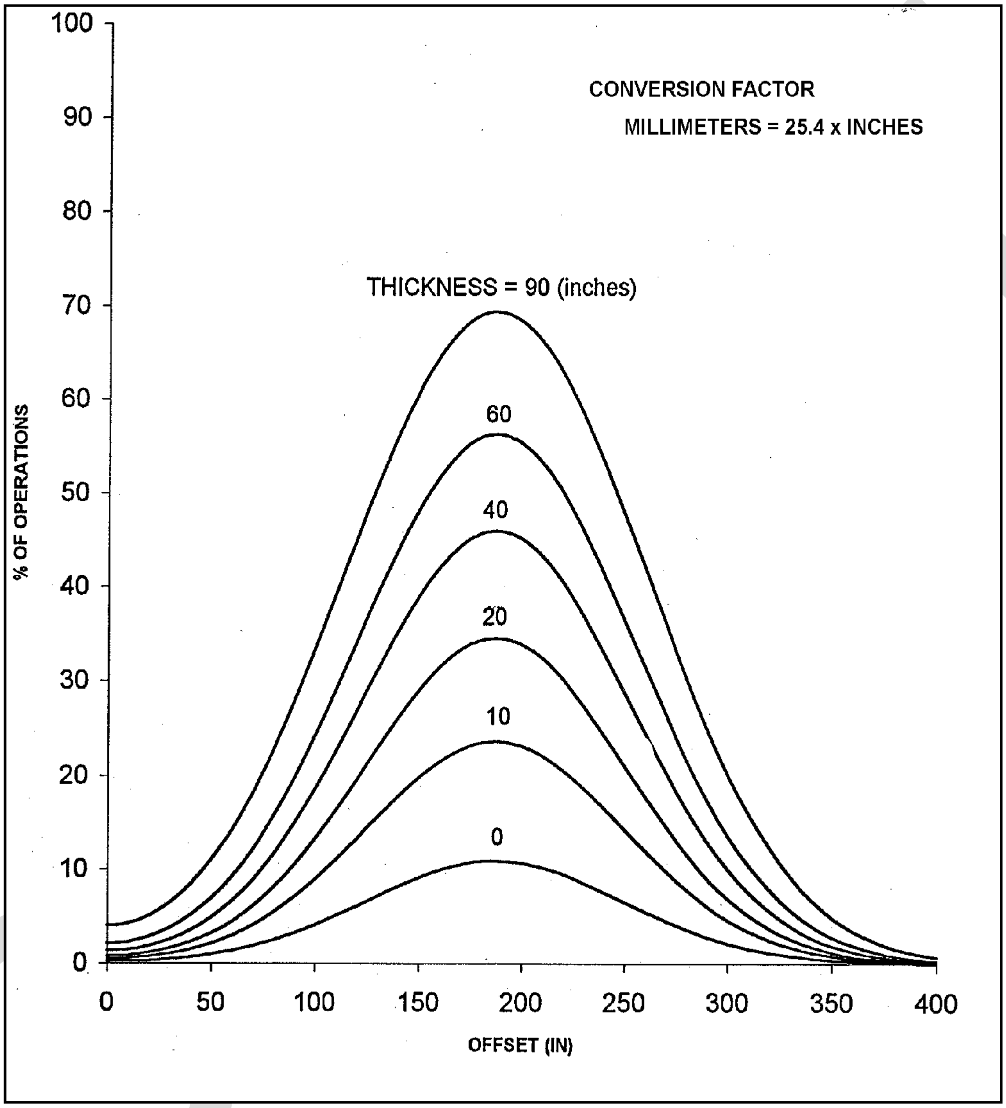


Figure B5-21. Effective Repetitions of Strain for C-5 Aircraft, Types B, C, or Secondary Traffic Areas

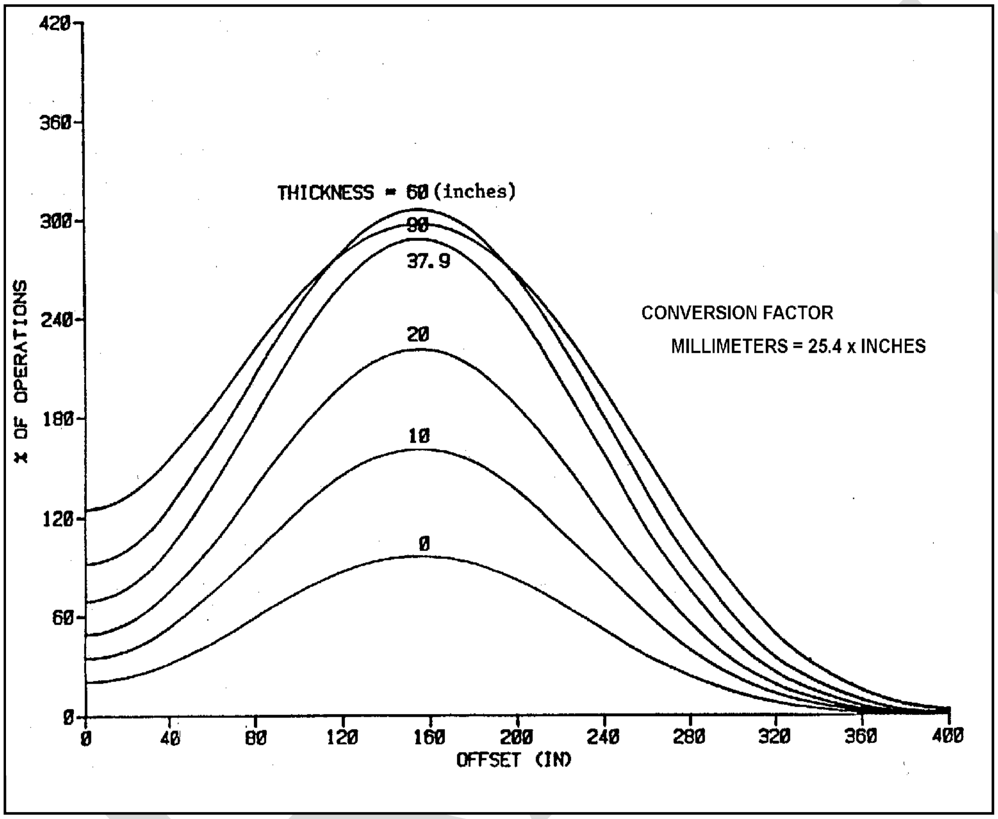
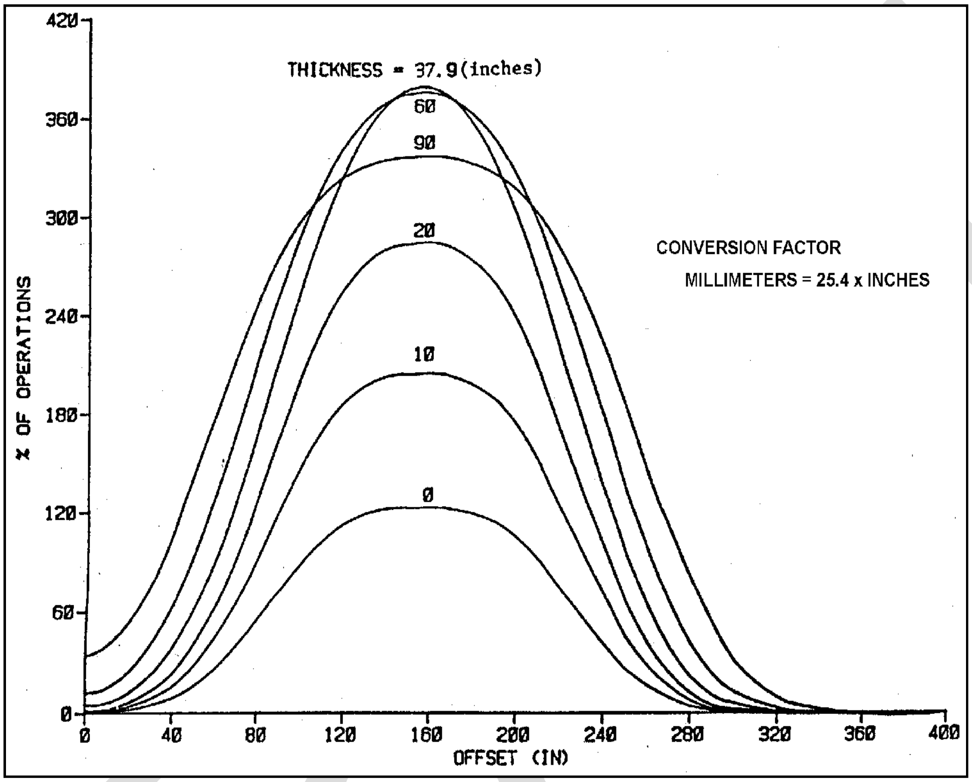


Figure B5-22. Effective Repetitions of Strain for C-5 Aircraft, Type A or Primary Traffic Areas



SECTION 6: PROCEDURE FOR PREPARATION OF BITUMINOUS CYLINDRICAL SPECIMENS

B6-1 SCOPE

This procedure describes the preparation of cylindrical specimens of bituminous paving mixture suitable for dynamic modulus testing. The procedure is intended for dense-graded bituminous concrete mixture containing up to 25-mm (1-in) maximum-size aggregate.

B6-2 APPLICABLE STANDARDS

These ASTM standards are applicable to this procedure: ASTM D1560 and D1561.

B6-3 SPECIMENS

Approximately 4,000 grams (g) of bituminous mixture should be prepared as specified by ASTM D1560. Cylindrical specimens should be 102 mm (4 in) in diameter by 203 mm (8 in) in height.

B6-4 APPARATUS

B6-4.1 **Testing Apparatus.** The apparatus used in preparing the specimens should be as specified by ASTM D1561, except that steel molding cylinders with 6.3-mm (0.25-in) wall thickness having an inside diameter of 102 mm (4 in) and height of 254 mm (10 in) should be used.

B6-4.2 **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 of determining loading up to 13,344 N (3,000 lb) 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.

B6-4.3 **Recorder Amplitude.** The recorder amplitude should be independent of frequency for tests conducted up to 20 Hertz (Hz).

B6-4.4 **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 in paragraph B6-4.2.

B6-4.5 **Load Measurements.** Loads should be measured with an electronic load cell meeting the requirements for load and stress measurements in this procedure.

B6-5 PROCEDURE

Follow these steps to prepare the specimens:

(1) The compaction temperature for the bituminous mixture should be as specified by ASTM D1561. For the first step in molding specimens, heat the compaction mold to the same temperature as the mix.

(2) Next, place the compaction mold in position in the mold holder and insert a paper disk 102 mm (4 in) in diameter to cover the base plate of the mold holder.

(3) Weigh out one-half of the required amount of bituminous mixture for one specimen at the specified temperature and place it uniformly in the insulated feeder trough, which has been preheated to the compaction temperature for the mixture.

(4) With the variable transformer controlling the heater, maintain the compactor foot sufficiently hot to prevent the mixture from adhering to it.

(5) Using 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 applying 30 tamping blows at a pressure of 1.7 Mpa (250 psi).

(6) Immediately place the remaining one-half of the mixture uniformly in the feeder trough.

(7) Again 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.

(8) 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 mm (0.50 in) per minute until reaching an applied pressure of 6.9 MPa (1,000 psi).

(9) Release the load immediately.

(10) After the compacted specimen has cooled sufficiently so that it will not deform on handling, remove it from the mold.

(11) Place the specimen on a smooth, flat surface and allow it to cool to room temperature. Cylindrical specimens will have approximately the same bulk specific gravity as specimens prepared as specified by ASTM D1560 and ASTM D1561.

SECTION 7: PROCEDURE FOR DETERMINING THE DYNAMIC MODULUS OF BITUMINOUS CONCRETE MIXTURES

B7-1 GENERAL

The purpose of this procedure is to determine dynamic modulus values of bituminous concrete mixtures. The procedure 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 C (40, 70, and 100 degrees F) at loading frequencies of 2 and 10 Hz for each temperature. The method is applicable to bituminous paving mixtures similar to the 25.4-, 19-, 12.7-, and 9.5-mm (1-, 0.75-, 0.50-, and 3/8-in) and No. 4 mixes as defined by Table 3 of ASTM D3515.

B7-2 APPLICABLE STANDARDS

These ASTM standards are applicable to this procedure: ASTM C617 and D1561.

B7-3 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.

B7-4 DEFINITIONS

These terms are used in this procedure:

- Dynamic modulus: The absolute value of the complex modulus that defines the elastic properties of a linear viscoelastic material subjected to a sinusoidal loading.

- Complex modulus: A complex number that defines the relationship between stress and strain for a linear viscoelastic material.

- Linear material: A material with a stress-to-strain ratio that is independent of the loading stress applied.

B7-5 APPARATUS

An electrohydraulic testing machine with a frequency generator capable of producing a haversine wave form has proven most suitable for use in dynamic modulus testing. The testing machine should be capable of applying loads over a range of frequencies from 1 to 20 Hz 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 C (32 to 120 degrees F). 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.

B7-6 SPECIMENS

The laboratory molded specimens should be prepared according to Appendix B, Section 6. A minimum of three specimens is required for testing. These steps outline the molding procedure:

(1) Cap all specimens with a sulfur mortar meeting ASTM C617 requirements prior to testing.

(2) 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.)

Note: On specimens with large-size aggregate, take care so that the gauges are attached over areas between the aggregate faces.

(3) Wire the strain gauges as required in paragraph B6-4.4 and attach suitable lead wires and connectors.

B7-7 PROCEDURE

This testing procedure is recommended:

(1) Place the 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.

(2) Place a specimen in the loading apparatus, and connect the strain gauge 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.

(3) Apply the haversine loading to the specimen without impact and with loads varying between 0 and 0.24 MPa (0 and 35 psi) for each load application for a minimum of 30 seconds and not exceeding 45 seconds at temperatures of 4.5, 21, and 37.8 degrees C (40, 70, and 100 degrees F) and at loading frequencies of 2 Hz for taxiway design and 10 Hz 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).

(4) 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 starting each test.

(5) 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 mm (1 to 2 in) of chart paper for 5 to 10 repetitions before the end of the test.

(6) Complete the loading for each test within 2 minutes from the time the specimens are removed from the temperature-controlled cabinet. The 2-minute testing

time limit is waived if loading is conducted within a temperature control cabinet meeting the requirements in paragraph B7-5.

B7-8 CALCULATIONS

Calculations are performed as follows:

(1) Measure the average amplitude of the load and the strain over the last three loading cycles to the nearest 0.5 mm.

(2) Calculate the loading stress σ_o using the equation:

$$\sigma_o = \frac{H_1 L}{H_2 A} \tag{B7-1}$$

where

H_1 = measured height of load, mm (in)

H_2 = measured chart height, mm (in)

L = full-scale load amplitude determined by settings on the recording equipment, N (lb)

A = cross-section area of the test specimen, mm^2 (in^2)

(3) Calculate the recoverable axial strain $\in_o$ using the equation:

$$\in_o = \frac{H_3 S}{H_4} \tag{B7-2}$$

where

H_3 = measured height of recoverable strain, mm (in)

H_4 = measured chart height, mm (in)

S = full-scale strain amplitude determined by settings on the recording equipment

(4) Calculate the dynamic modulus $|E^*|$ using the equation:

$$|E^*| = \frac{\sigma_o}{\in_o} \tag{B7-3}$$

where

σ_o = axial loading stress, MPa (psi)

$\in_o$ = recoverable axial strain, mm per mm (in per in)

(5) Report the average dynamic modulus at temperatures of 4.5, 21, and 37.8 degrees C (40, 70, and 100 degrees F) for each loading frequency at each temperature.

SECTION 8: PROCEDURE FOR ESTIMATING THE MODULUS OF ELASTICITY OF BITUMINOUS CONCRETE

B8-1 OVERVIEW

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:

- Ring-and-ball softening point in degrees C (degrees F) of the bituminous material used in the mix in accordance with ASTM D36.
- Penetration of the bituminous material in 1/10 mm in accordance with ASTM D5.
- Volume concentration of the aggregate C_v used in the mix defined by:

$$C_v = \frac{\text{aggregate volume}}{\text{aggregate volume} + \text{bitumen volume}} \qquad \text{(B8-1)}$$

B8-2 STEPS OF THE PROCEDURE

These are the steps of this method:

(1) Penetration Index. With known values of penetration and ring-and-ball softening point, enter Figure B8-1 and determine the PI.

(2) Stiffness Modulus. The next step involves using the nomograph presented in Figure B8-2. 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 Hz is recommended for taxiway design and 10 Hz 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}.

(3) Determining Modulus of Mix S_{mix}. A value for S_{mix} may be determined by Equation B8-2:

$$S_{mix} = S_{bit}\left[1+\left(\frac{2.5}{n}\right)\left(\frac{c_v}{1-c_v}\right)\right]^n \qquad \text{(B8-2)}$$

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[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

$$n = 0.83 \log\left(\frac{400{,}000}{S_{bit}}\right) \qquad \text{(B8-3)}$$

The value thus determined for S_{mix} is in units of kilograms per square centimeter.

(4) 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:

$$C'_v = \frac{c_v}{1 + \Delta \text{ air void content}} \qquad \text{(B8-4)}$$

where Δ air void content is the actual air void content (expressed in decimal form) minus 0.03. Equation B8-4 is valid only where:

$$c_B \geq \frac{2}{3}\left(1 - c'_v\right) \qquad \text{(B8-5)}$$

where

$$c_B = \frac{\text{bitumen volume}}{\text{aggregate volume} + \text{bitumen volume}} \qquad \text{(B8-6)}$$

Figure B8-1. Relationship between Penetration at 25 Degrees C and Ring-and-Ball Softening Point for Bitumens with Different PIs

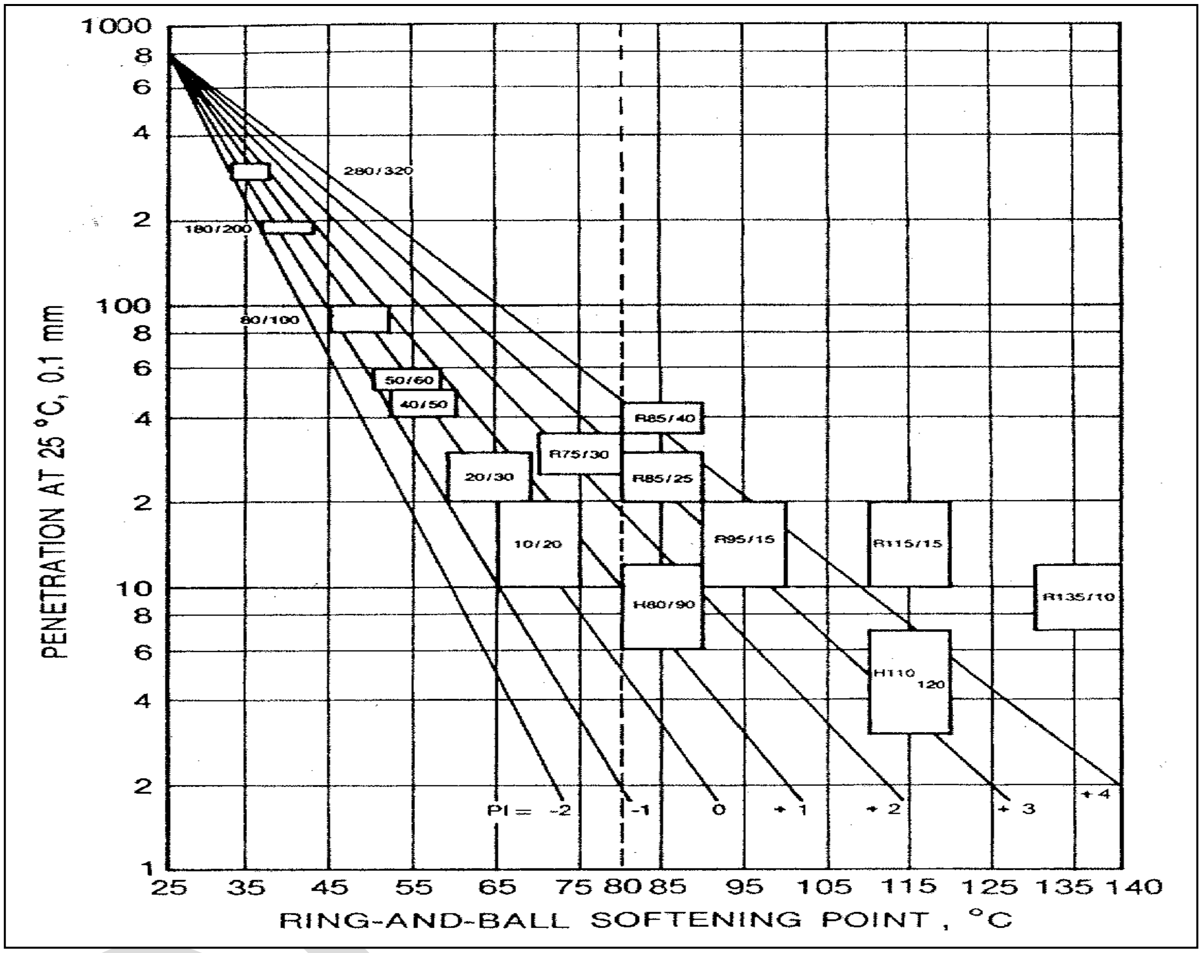
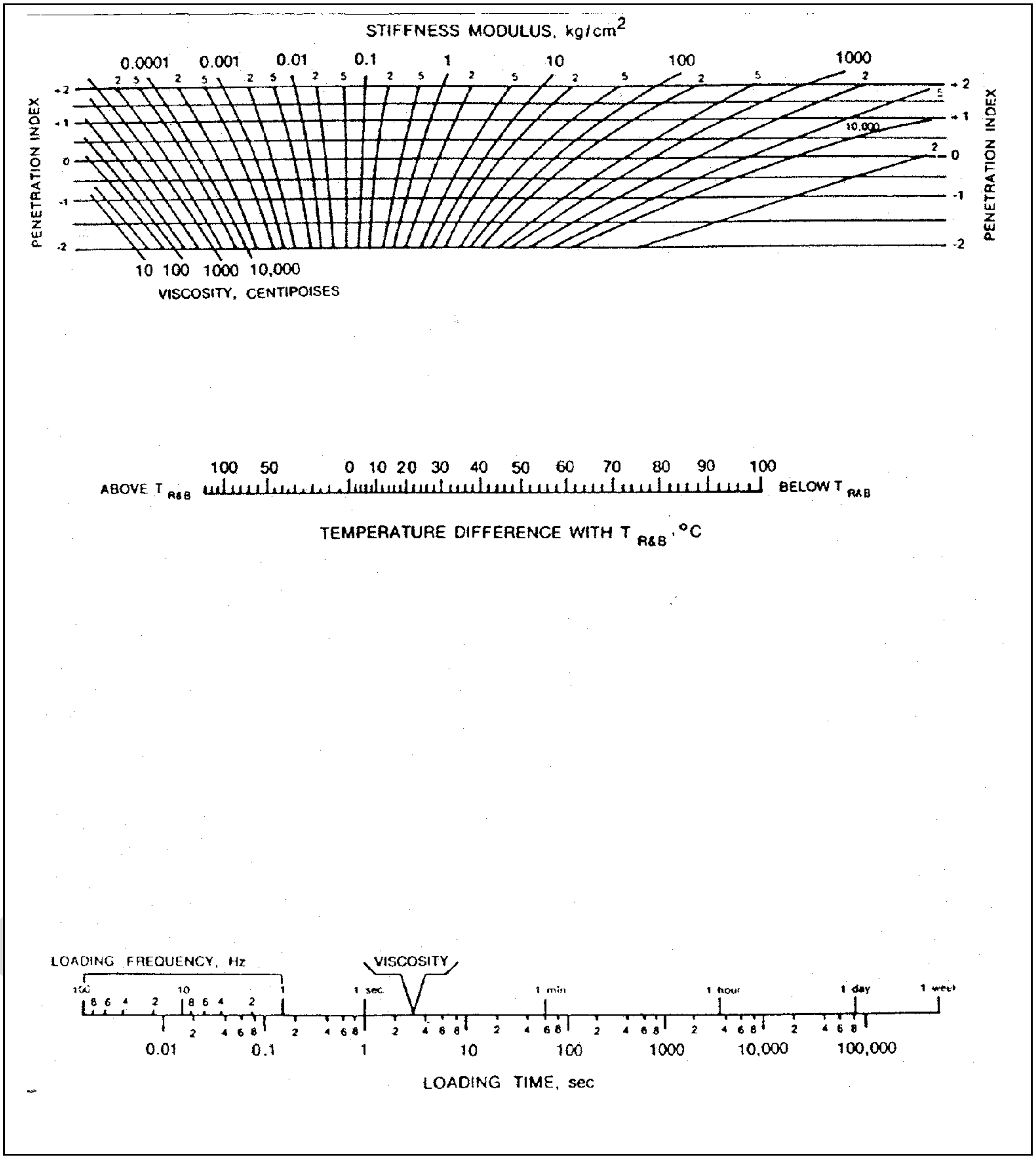


Figure B8-2. Nomograph for Determining the Stiffness Modulus of Bitumens



SECTION 9: PROCEDURE FOR DETERMINING THE MODULUS OF ELASTICITY OF UNBOUND GRANULAR BASE AND SUBBASE COURSE MATERIALS

B9-1 PROCEDURE

B9-1.1 **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 B9-1. 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 mm (4, 5, 6, 7, and 8 in). For subbase courses having a design thickness of 203 mm (8 in) or less, the applicable curve or appropriate interpolation can be used directly. For a design subbase course thickness in excess of 203 mm (8 in), 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 mm (4, 6, and 10 in). These relationships can be used directly or by interpolation for design base course thicknesses up to 254 mm (10 in). For design thicknesses in excess of 254 mm (10 in), the layer should also be divided into sublayers of approximately equal thickness and the modulus of each sublayer determined individually.

B9-1.2 **Modulus Values.** To determine modulus values from this procedure, Figure B9-1 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.

B9-2 EXAMPLES

B9-2.1 **Thickness.** Assume a pavement having a base course thickness of 102 mm (4 in) and a subbase course thickness of 203 mm (8 in) 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 B9-1 with a modulus of layer *n*+1 of 69 MPa (10,000 psi) and using the 203-mm (8-in) subbase course curve, the modulus of the subbase (layer *n*) is 127.5 MPa (18,500 psi). 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 B9-1 with a modulus value of layer *n*+1 of 127.5 MPa (18,500 psi) and using the 102-mm (4-in) 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 B9-2.

B9-2.2 **Design Thickness.** If, in the first example, the design thickness of the subbase course had been 305 mm (12 in), it would have been necessary to divide this layer into two 152-mm-thick (6-in-thick) sublayers. Then, using the procedure described in paragraph B9-2.1 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 B9-3.

B9-2.3 **Relationships.** The relationships indicated in Figure B9-1 can be expressed as:

$$E_n = E_{n+1}(1 + 10.52 \log t - 2.10 \log E_{n+1} \log t) \quad \text{(B9-1)}$$

where

n = a layer in the pavement system

E_n = resilient modulus (in psi) of layer n

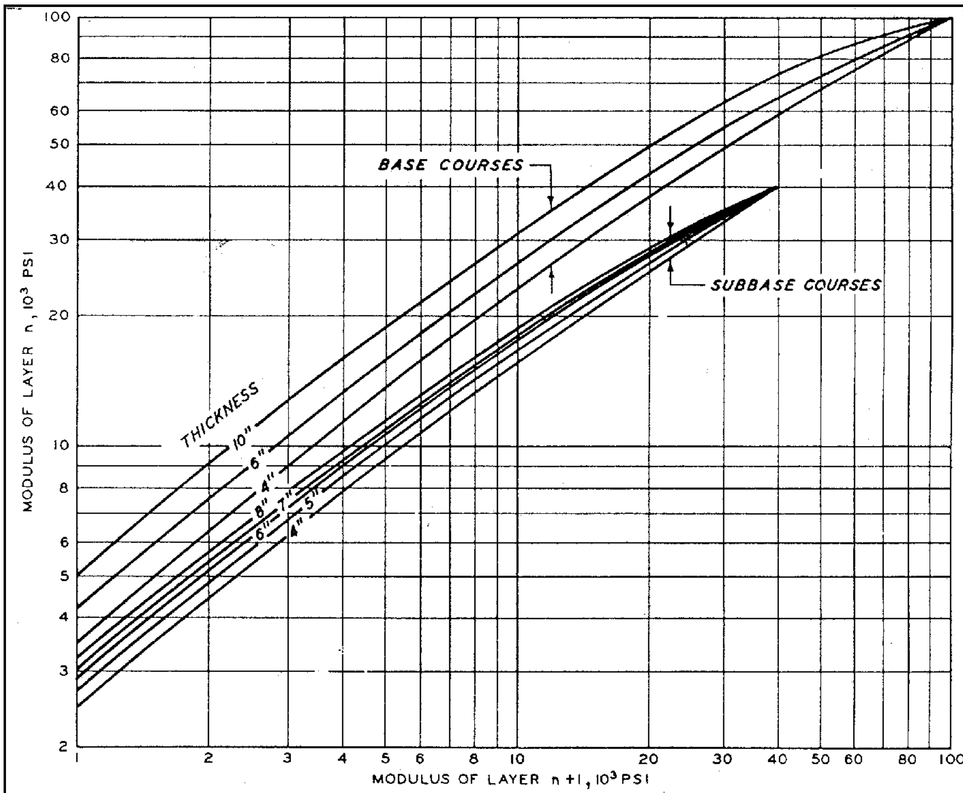
E_{n+1} = the resilient modulus (in psi) of the layer beneath layer n

t = the thickness (in) of layer n for base course materials and as:

$$E_n = E_{n+1}(1 + 7.18 \log t - 1.56 \log E_{n+1} \log t) \quad \text{(B9-2)}$$

for subbase course materials. Use of these equations for direct computation of modulus values for the examples given above yields the values indicated in parentheses in Figures B9-2 and B9-3. It can be seen that comparable values are obtained with either graphical or computational determination of the modulus value for either material.

Figure B9-1. Relationships between Modulus of Layer *n* and Modulus of Layer *n*+1 for Various Thicknesses of Unbound Base Course and Subbase Course



CONVERSION FACTORS

MEGAPASCALS = 0.006894 x PSI
MILLIMETERS = 25.4 x INCHES

Figure B9-2. Modulus Values Determined for First Example

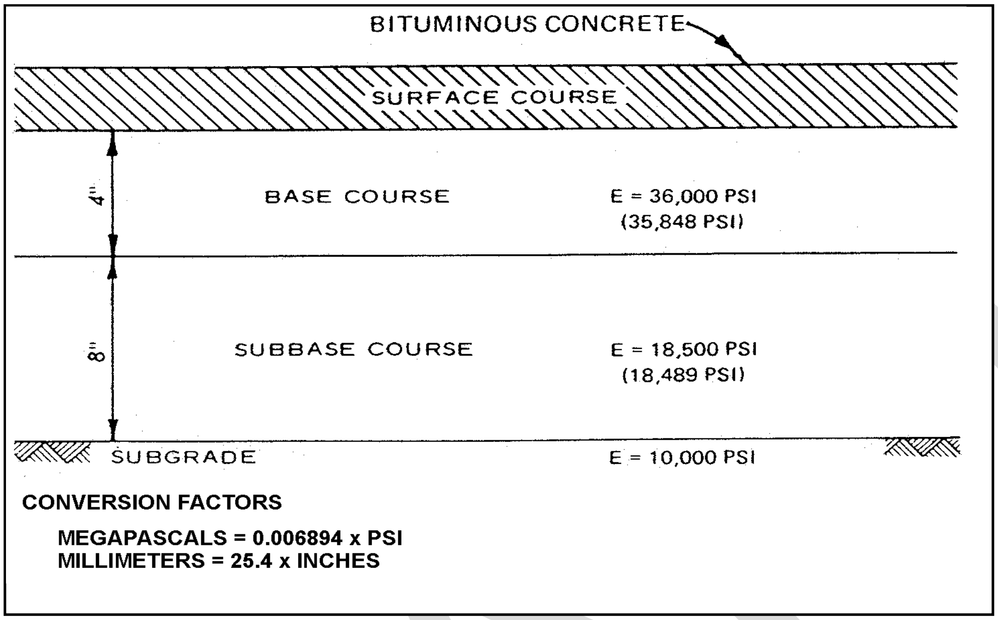
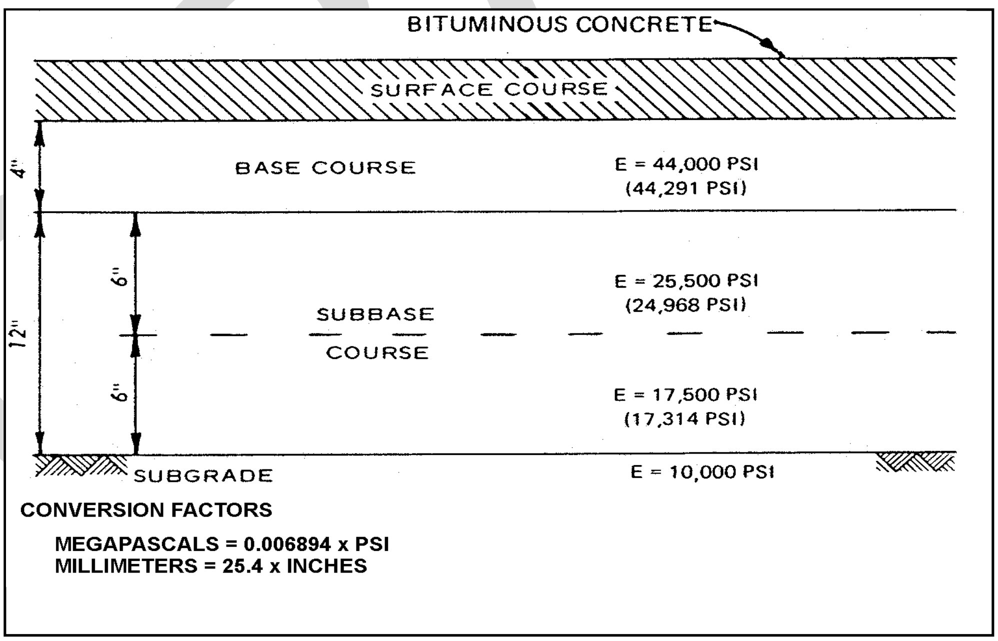


Figure B9-3. Modulus Values Determined for Second Example



SECTION 10: PROCEDURE FOR DETERMINING THE FLEXURAL MODULUS AND FATIGUE CHARACTERISTICS OF STABILIZED SOILS

B10-1 LABORATORY PROCEDURE

B10-1.1 **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.

B10-1.2 **Specimen Preparation.** Beam specimens should be prepared following the general procedures indicated in ASTM D1632. This method describes procedures for molding 76- by 76- by 286-mm (3- by 3- by 11.25-in) specimens; however, any size mold may be used for the test. For soils containing aggregate particles larger than 19 mm (0.75 in), it is recommended that molds on the order of 102 by 102 to 152 by 152 mm (4 by 4 to 6 by 6 in) be used. In general, specimens should have an approximately square cross-sectional 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 C (73 degrees F).

B10-1.3 **Equipment.** This equipment is required:

- Loading frame capable of receiving the specimen for third-point loading test
- Electrohydraulic testing machine. This machine must be capable of applying static and haversine loads.
- Load cell (approximately 907-kg [2,000-lb] capacity)
- Two LVDTs and one SR-4-type strain gauge
- Recording equipment for monitoring deflection, strain, and load
- Miscellaneous pins and yokes as described in the equipment setup below for mounting the LVDTs

B10-1.4 **Equipment Setup.** Details of the equipment setup are shown in Figures B10-1 to B10-3. 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 tab 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 B10-1 to B10-3, 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 B10-2. As the beam bends under loading, deflection 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 necessary 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 B10-2.

B10-1.5 **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 approximately 1 Hz. To develop a strain repetition pattern, tests should 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.

B10-1.6 **Reporting of Test Results**

B10-1.6.1 **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:

$$E_f = \frac{23PL^3}{1296DI}\left(1 + 2.11\frac{h}{(L)}\right)^2 \qquad \text{(B10-1)}$$

where

E_f = flexural modulus, MPa (psi)

P = maximum load amplitude, kg (lb)

L = specimen length, mm (in)

D = deflection at the neutral axis, mm (in)

I = moment of inertia, mm^4 (in^4)

h = specimen height, mm (in)

The value to be used for E_f in the performance model is the arithmetic mean of all values obtained during the test.

B10-1.6.2 **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 $\in$ may be computed using the expression:

$$\in = \frac{PLh}{6E_f I} \qquad \text{(B10-2)}$$

B10-2 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 B10-2. To use this relationship, specimens of the stabilized material should be molded and tested following the procedures indicated in ASTM D1633. Values obtained from the unconfined compression test can then be used to determine the values of the equivalent cracked section modulus using Figure B10-2.

Figure B10-1. General View of Equipment Setup

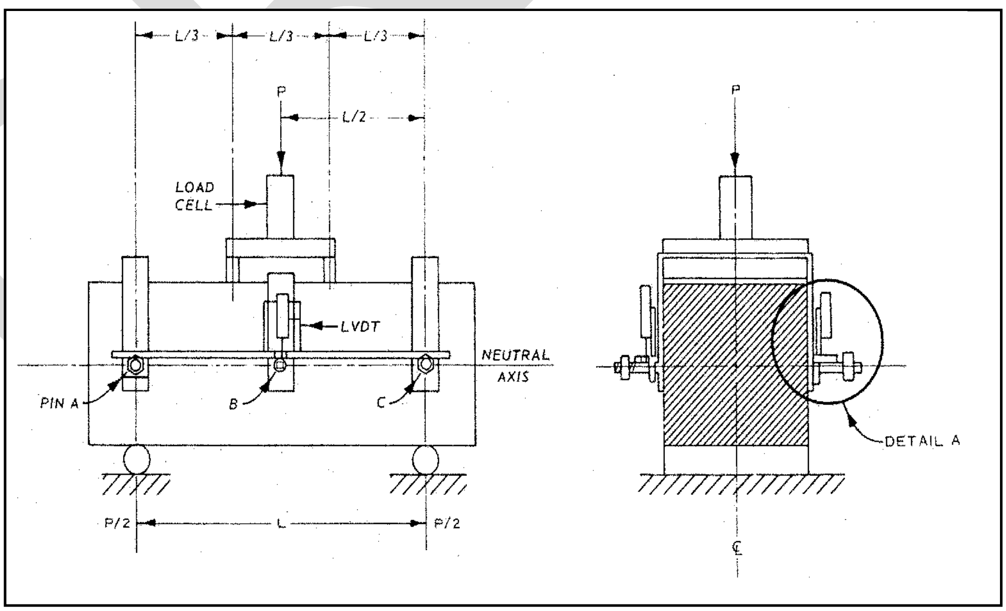


Figure B10-2. Details of Equipment Setup

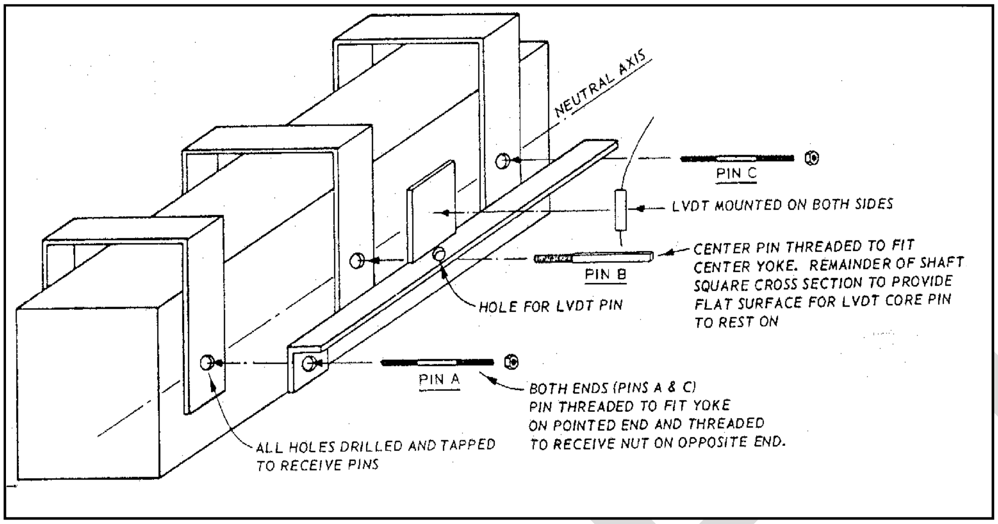
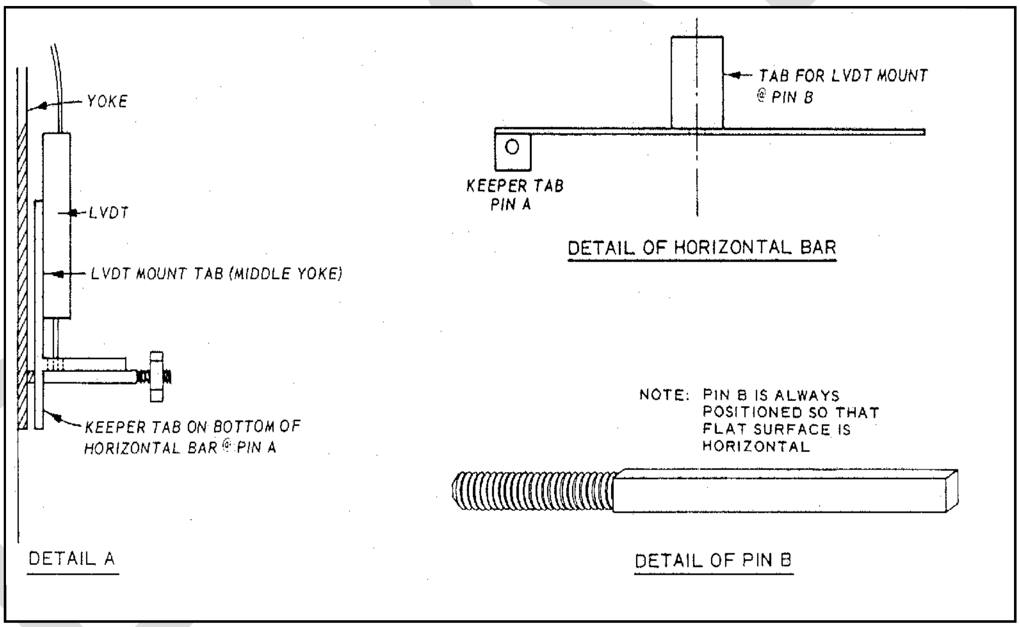


Figure B10-3. Miscellaneous Details



SECTION 11: PROCEDURE FOR DETERMINING RESILIENT MODULUS OF SUBGRADE MATERIAL

B11-1 **OVERVIEW**

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.

B11-2 **DEFINITIONS**

These 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

σ_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 = σ_d / ϵ_{RI} = resilient modulus

θ = $\sigma_1 + 2\sigma_3 = \sigma_d + 3\sigma_3$ = 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

B11-3 **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 paragraphs B11-4 through B11-5.

B11-4 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 mm (1.4, 2.8, 4, 6, 12, and 15 in) in diameter are used most commonly.

B11-4.1 **Cohesive Soils Containing Negligible Amounts of Gravel.** Specimens 35.5 mm (1.4 in) 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 the procedure in paragraph B11-4.1.1 or B11-4.1.2.

B11-4.1.1 **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 approximately 127 mm (5 in) in diameter are preferred for triaxial tests using 35.5-mm-diameter (1.4-in-diameter) specimens. Specimens shall be prepared in a humid room and tested as soon as possible thereafter to prevent evaporation of moisture. In preparing the specimens, take extreme care to prevent any disturbance to the structure of the soil. The specimens shall be prepared following these steps:

(1) 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-mm-diameter (5-in-diameter) undisturbed sample is to be used for 35.5-mm-diameter (1.4-in-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.

(2) 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 the specimen to the required length (usually 76 to 89 mm [3 to 3.5 in] for 35.5-mm-diameter [1.4-in-diameter] specimens and 152 to 178 mm [6 to 7 in] for 71-mm-diameter [2.8-in-diameter] specimens) using a miter box.

(3) From the soil trimmings, obtain 200 g of material to use for specific gravity and water content determination.

(4) Weigh the specimen to an accuracy of ±0.01 g for 35.5-mm-diameter (1.4-in-diameter) specimens and ±0.1 g for 71-millimeter-diameter (2.8-in-diameter) specimens.

(5) Measure the height and diameter of the specimen to an accuracy of ±0.25 mm (0.01 in). 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 in Equation B11-1:

$$D_o = \frac{D_t + 2D_c + D_b}{4} \tag{B11-1}$$

where

D_o = average diameter

D_t = diameter at top

D_C = diameter at center

D_b = diameter at bottom

B11-4.1.2 **Compacting Specimens of Cohesive Soil.** Specimens of compacted soil may be trimmed, as described in paragraph B11-4.1.1, from samples formed in a compaction mold (a 102-mm-diameter [4-in-diameter] sample is satisfactory for 35.5-mm-diameter [1.4-in-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 the accumulative weight of the 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-mm-diameter [1.4-in-diameter] specimens or similar compactors for 71-mm-diameter [2.8-in-diameter] and larger specimens) are necessary. After each specimen compacted to finished dimensions has been removed from the mold, proceed in accordance with steps 3 through 5 of paragraph B11-4.1.1.

B11-4.2 **Cohesionless Soils Containing Negligible Amounts of Gravel.** Soils that 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, usually satisfactory specimens can 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 necessary. 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 that will permit interpolation to the desired density. Generally, specimens prepared in this manner should be 71 mm (8 in) in diameter or larger, depending on the maximum particle size. The procedure for forming the test specimen shall consist of these steps:

(1) Oven-dry and weigh an amount of material sufficient to provide slightly 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-mm-diameter (8-in-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 desired density. 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 this 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 midheight.

(6) Subtract the weight of the unused material from the original weight of the sample to obtain the 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 the height and diameter as specified in paragraph B11-4.

B11-4.3 **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 paragraphs B11-4.1 and B11-4.2, with the size of the specimen based on an estimate of the largest particle size. In testing compacted soils, the largest particle size is usually known, and whenever possible the entire sample should be tested 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, should 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 paragraph B11-4.1 as a guide. When specimens of very high density are required, the samples should be compacted 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, usually the soil must possess 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 because the membrane can be placed over the compacted specimen readily after the specimen is removed from the split mold. The specimen should be compacted to the desired density in accordance with paragraph B11-4.

B11-5 Q TEST WITH BACK-PRESSURE SATURATION

B11-5.1 **Equipment Setup.** For the Q test with back-pressure saturation, the apparatus should be set up similar to that shown in Figure B11-1. Do not use filter strips and permit as little volume change as possible during the test. Complete the steps outlined in paragraph B11-4 and these 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-mm-diameter (1.4-in-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.

B11-5.2 **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 the back-pressure procedure as specified in these steps:

(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 approximately 0.048 MPa (7 psi) and a back pressure of approximately 0.013 MPa (2 psi). Record these pressures on a data sheet.

(3) 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.

(4) 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.)

(5) Using the technique described in the steps above, increase the chamber pressure and the back pressure in increments, maintaining the back pressure at approximately 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.

(6) 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.

(7) Verify the completeness of saturation by closing valve F and increasing the chamber pressure by approximately 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.

B11-5.3 **Chamber Pressure.** After verifying saturation, and re-measuring 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.

B11-5.4 **Operation**

(1) Start the test with the piston approximately 2.5 mm (0.1 in) above the specimen cap. This allows compensation for the effects of piston friction, exclusive of that which may develop later 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 approximately 0.3 percent per minute for brittle materials that achieve maximum deviator stress at approximately 3 to 6 percent strain; at these rates, the elapsed time to reach maximum deviator stress would be approximately 15 to 20 minutes.

(2) Observe and record the resulting load at every 0.3 percent strain for approximately 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 in this procedure; 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. For each group of tests, however, approximately 20 percent of the samples should be tested at the rates set forth in this procedure.

(4) On 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-mm-diameter (1.4-in-diameter) specimens, carefully blot any excess moisture from the surface of the specimen and determine the water content of the whole specimen. For 71-mm-diameter (2.8-in-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 but using the same rate of strain.

B11-6 EQUIPMENT

B11-6.1 Triaxial Test Cell

A triaxial cell suitable for use in resilience testing of soils is shown in Figure B11-2. This equipment is similar to most standard cells, except that it is slightly larger so it can facilitate the internally mounted load and deformation measuring equipment and 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.

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.

B11-6.2 **Deformation-Measuring Equipment.** 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 B11-2. Details of the clamps are shown in Figure B11-3. Load is measured by placing a load cell between the specimen cap and the loading piston as shown in Figure B11-2. Use of this type of measuring equipment offers several advantages:

(a) It is not necessary to reference deformations to the equipment, which deforms during loading.

(b) The effect of end-cap restraint on soil response is virtually eliminated.

(c) Any effects of piston friction are eliminated by measuring loads inside the triaxial cell.

In addition to having the proper 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.

B11-6.3 **Additional Equipment.** In addition to the equipment described in paragraph B11-6.2, these items are also used:

(a) A 9- to 27-metric ton capacity (10- to 30-short ton capacity) loading machine

(b) Calipers, a micrometer gauge, and a steel rule (calibrated to 0.25 mm [0.01 in])

(c) Rubber membranes, 0.25 to 0.635 mm (0.01 to 0.025 in) thick

(d) Rubber o-rings

(e) A vacuum source with a bubble chamber and regulator

(f) A back-pressure chamber with pressure transducers

(g) A membrane stretcher

(h) Porous stones

B11-7 PREPARATION OF SPECIMENS AND PLACEMENT IN TRIAXIAL CELL

These steps should be followed in preparing and placing specimens:

(1) In accordance with the procedures specified in paragraph B11-5, prepare the specimen and place it on the base plate 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, follow the procedures

indicated in paragraph B11-5.1. 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.

(2) Extend the lower LVDT clamp and slide it carefully down over the specimen to approximately the lower third point of the specimen.

(3) Repeat this step for the upper clamp, placing it at the upper third point. Ensure that both clamps lie in horizontal planes.

(4) 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 mm [0.01 in]) the vertical spacing between the LVDT clamps and record this value.

(5) Place the triaxial chamber in position. Set the load cell in place on the specimen.

(6) Place the cover plate on the chamber. Insert the loading piston and obtain a firm connection with the load cell.

(7) Tighten the tie rods firmly.

(8) 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.

(9) If the specimen is to be back-pressure saturated, proceed in accordance with paragraph B11-5.

(10) After saturation has been completed, rebalance the recorder bridge to the load cell and LVDTs.

B11-8 RESILIENCE TESTING OF COHESIVE SOILS

B11-8.1 **Confining Pressure.** The resilient properties of cohesive soils are only slightly affected by the magnitude of the confining pressure F_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 in the range 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.

B11-8.2 **Deviator Stress.** Resilient properties are highly dependent on the magnitude of the deviator stress F_d. It is therefore necessary to conduct the tests for a range in deviator stress values. Follow this procedure:

(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.

B11-8.3 **Test Results.** The results of the resilience tests can be presented in the form of a summary table and as a graphic as shown in Figure B11-4 for the resilient modulus.

B11-9 RESILIENCE TESTING OF COHESIONLESS SOILS

B11-9.1 **Overview.** The resilient modulus of cohesionless soils M_R is dependent on the magnitude of the confining pressure F_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 procedure outlined in these eight steps 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 suggests that drainage of saturated specimens should be permitted during the application of these loads but that the test loading (beginning in step 6) should be conducted with the specimens 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, maintaining a stress ratio equal to 6 and using this 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 reaching a deviator stress of four times the confining pressure (stress ratio of 5).

(7) Repeat as in step 6 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 items such as the LVDT clamps. Remove the soil specimen and use the entire amount of soil to determine the moisture content.

B11-9.2 **Calculations.** Calculations performed should be shown in a table format. 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 B11-5.

B11-10 INTERPRETATION OF TEST RESULTS

B11-10.1 **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 B11-6 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 B11-6 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 B11-6, and the design modulus value is taken from the intersection of the plotted functions.

B11-10.2 **Example on Cohesive Soils.** Assume a design problem involving 100,000 repetitions of strain. Figure B11-7 shows a plot of relationships taken from

Figure B11-6 and 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.

B11-10.3 **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. Figure B11-8 shows a relationship between the pavement thickness and the sum of the principal stresses at the top of the subgrade due to overburden. Figure B11-9 shows relationships 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 B11-8. The applicable relationship from Figure B11-9 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 B11-8. 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.

B11-10.4 **Example on Cohesionless Soils.** Assume a design problem involving a pavement having an estimated initial thickness of 762 mm (30 in). The design aircraft has a dual-wheel main gear assembly, and the design life is for 100,000 repetitions of strain. From Figure B11-8, 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 B11-9, the value obtained from Figure B11-8 is added to all values of the sum of the principal stresses indicated in the relationship and the adjusted curve is replotted (Figure B11-10). The result of adjusting the original relationship is to shift it to the right of its original position. Figure B11-10 also shows the results of laboratory resilient modulus tests on specimens of the subgrade soil. From the intersection of these two relationships, a design modulus M_R of 103 MPa (15,000 psi) is determined.

B11-10.5 **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 it converges with the laboratory relationships.

Figure B11-1. Schematic Diagram of Typical Triaxial Compression Apparatus

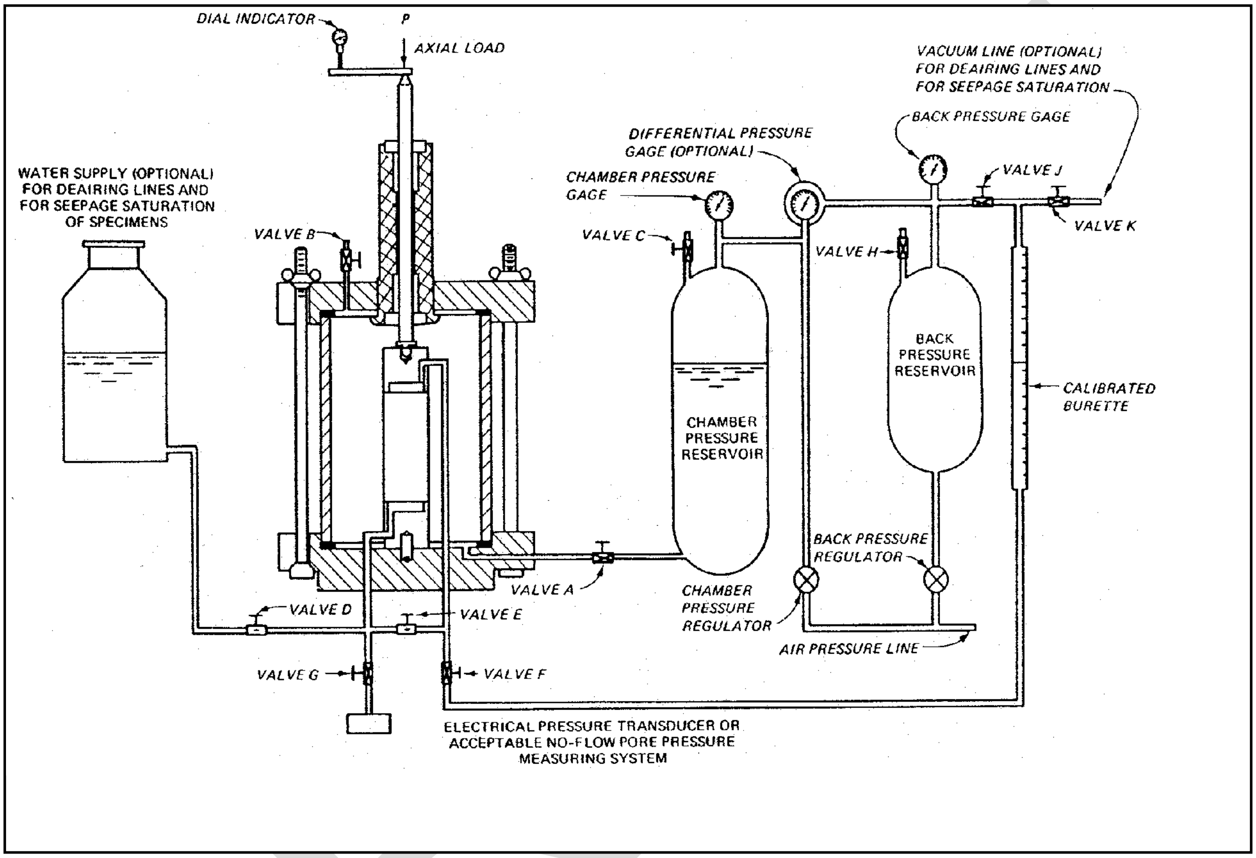


Figure B11-2. Triaxial Cell

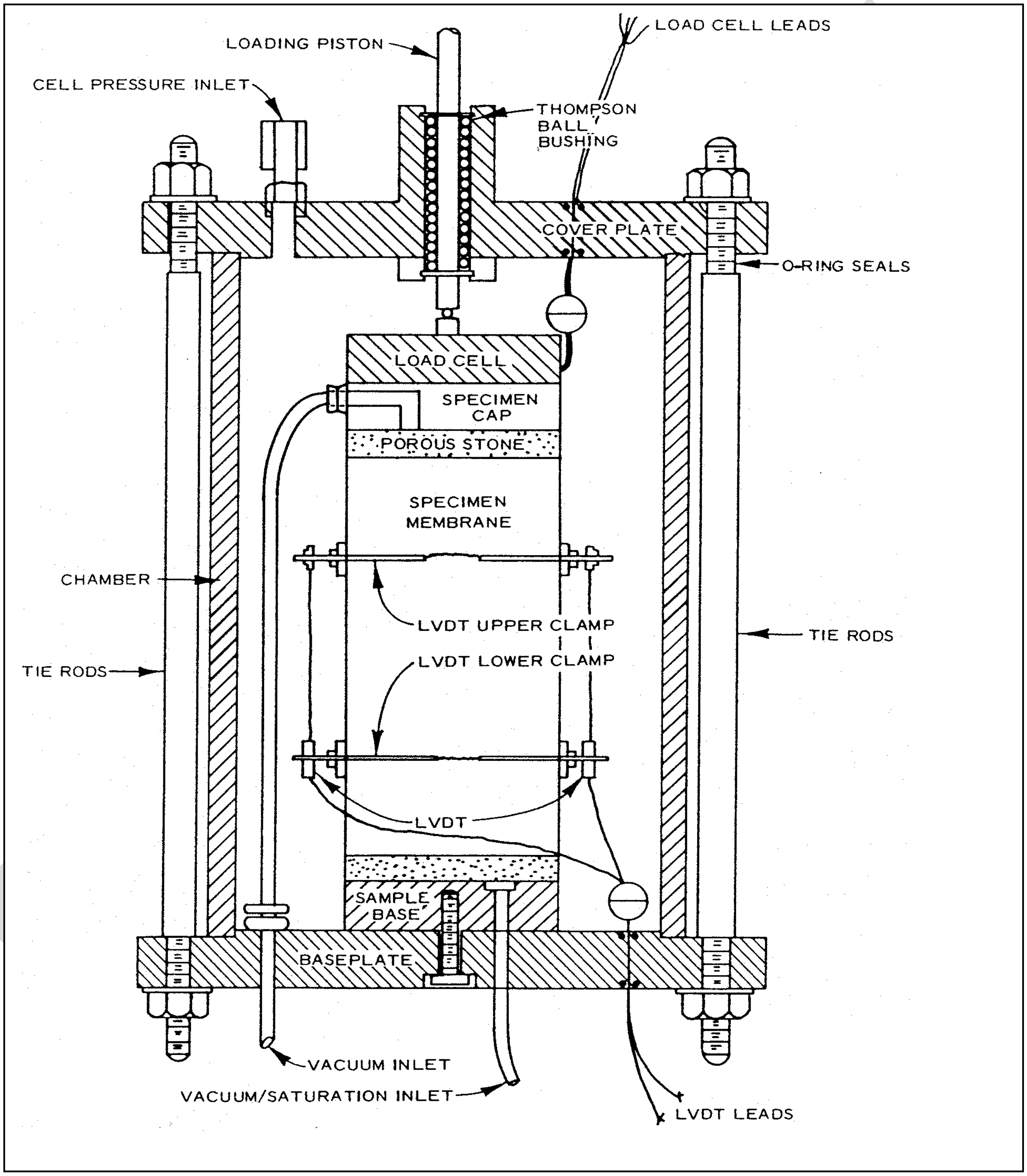


Figure B11-3. LVDT Clamps

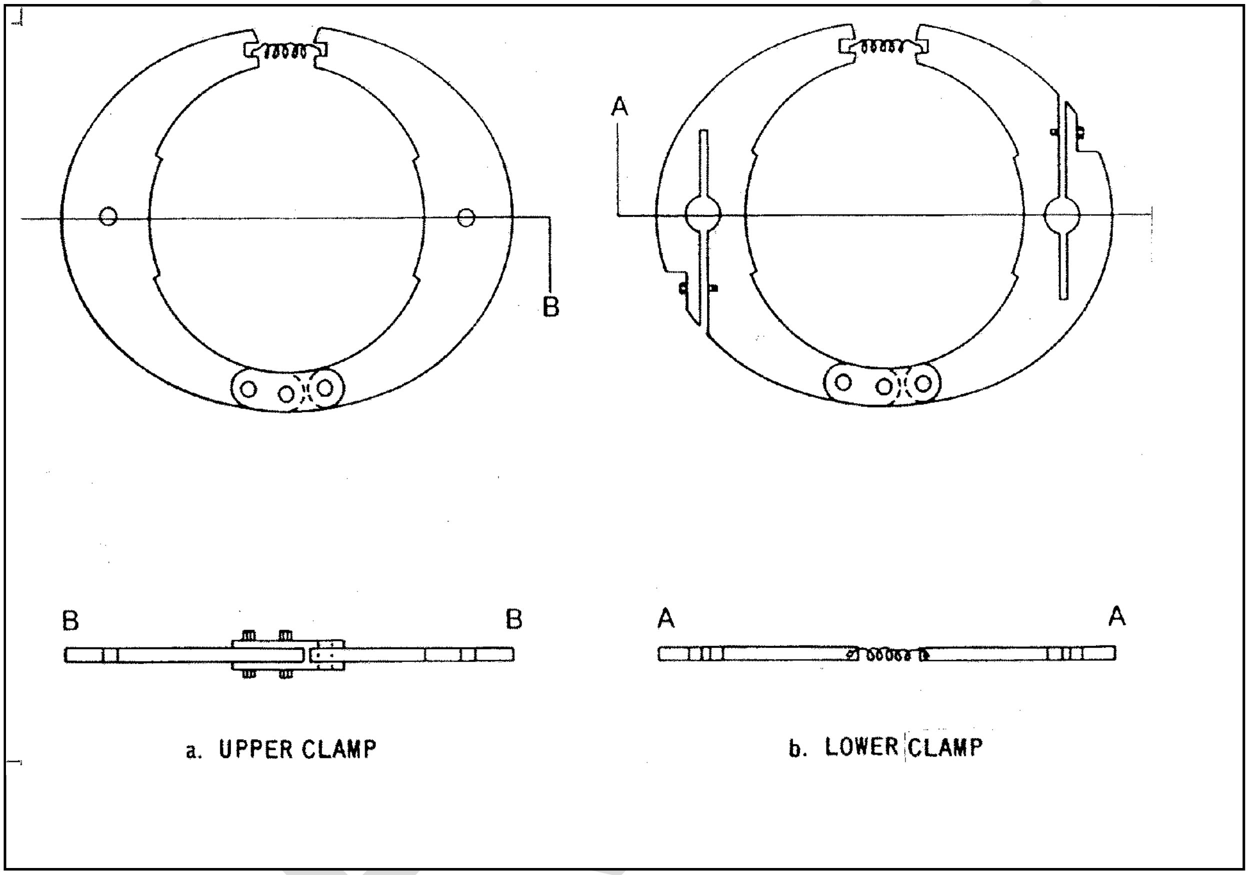


Figure B11-4. Presentation of Results of Resilience Tests on Cohesive Soils

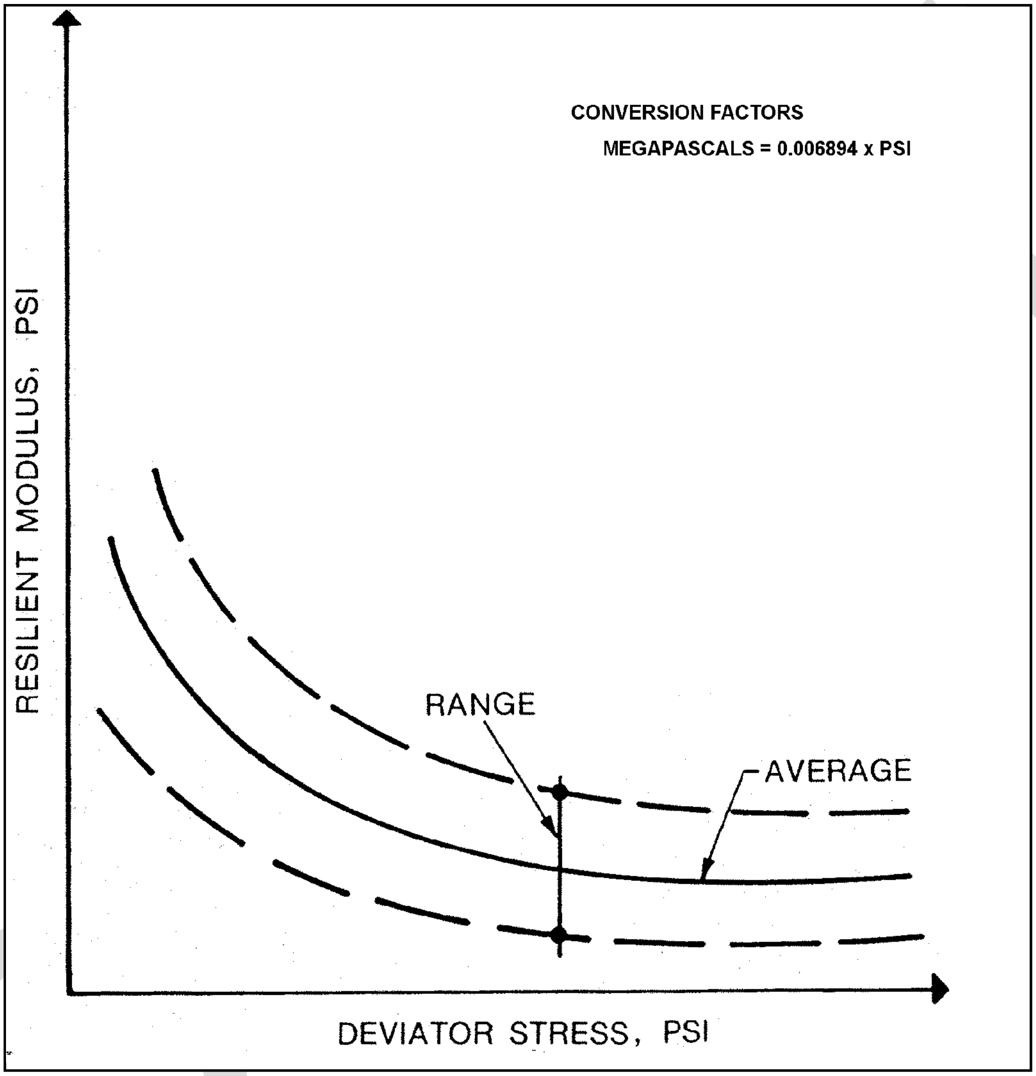


Figure B11-5. Presentation of Results of Resilience Tests on Cohesionless Soils

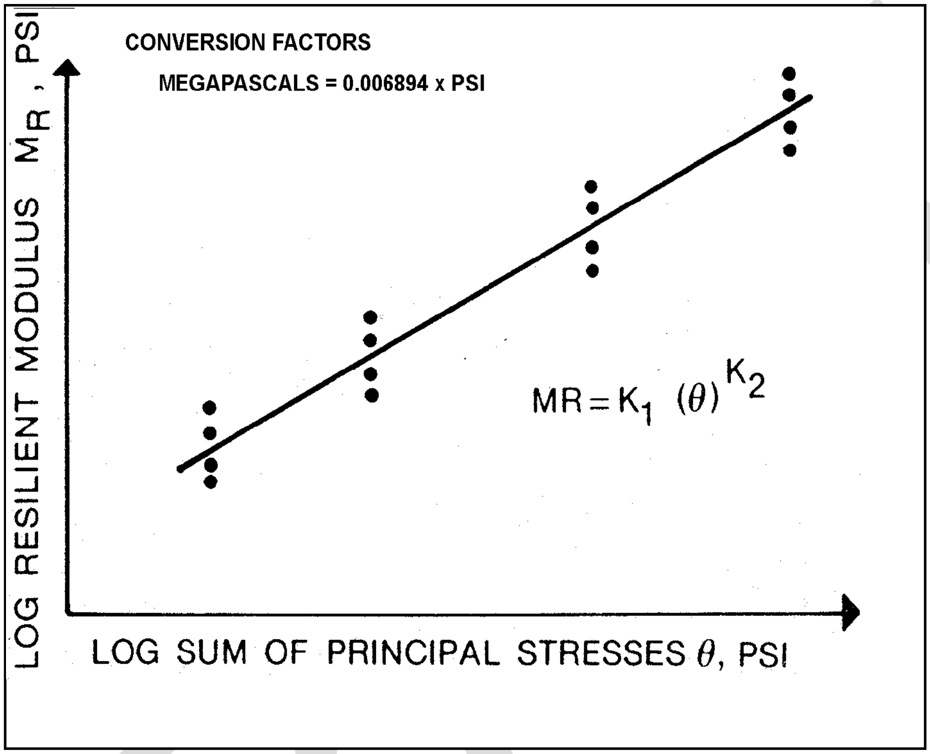


Figure B11-6. Estimated Deviator Stress at Top of Subgrade

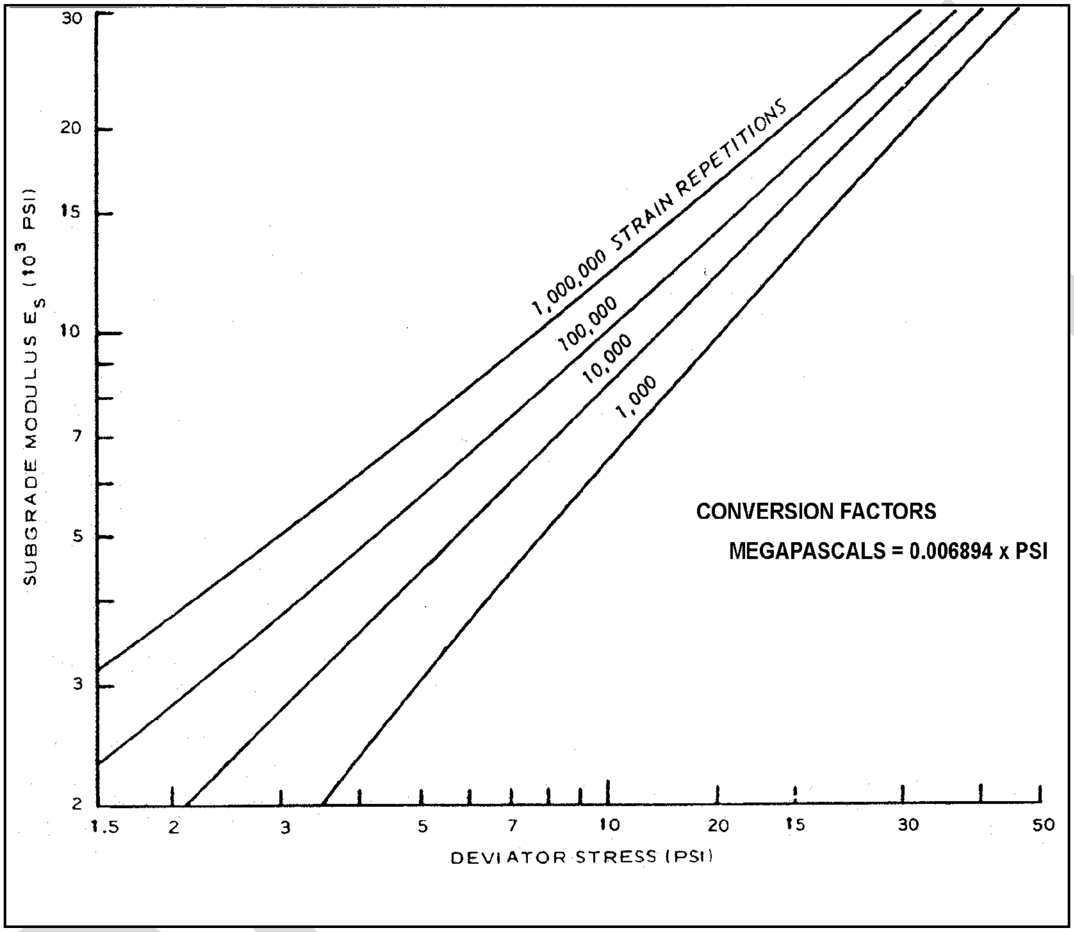


Figure B11-7. Determination of Subgrade Modulus for Cohesive Soils

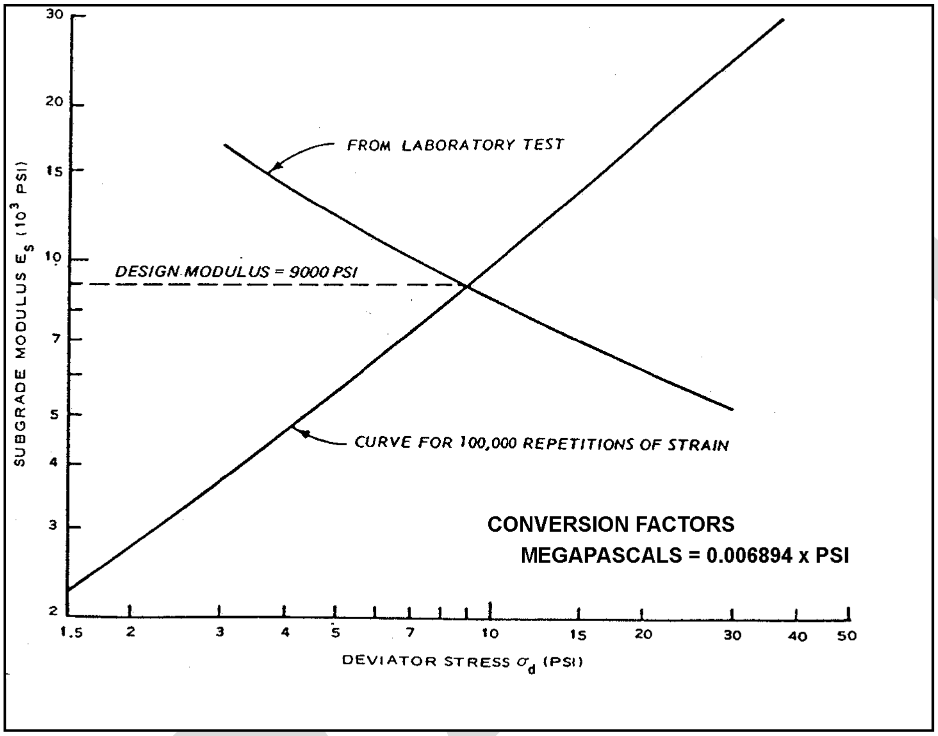
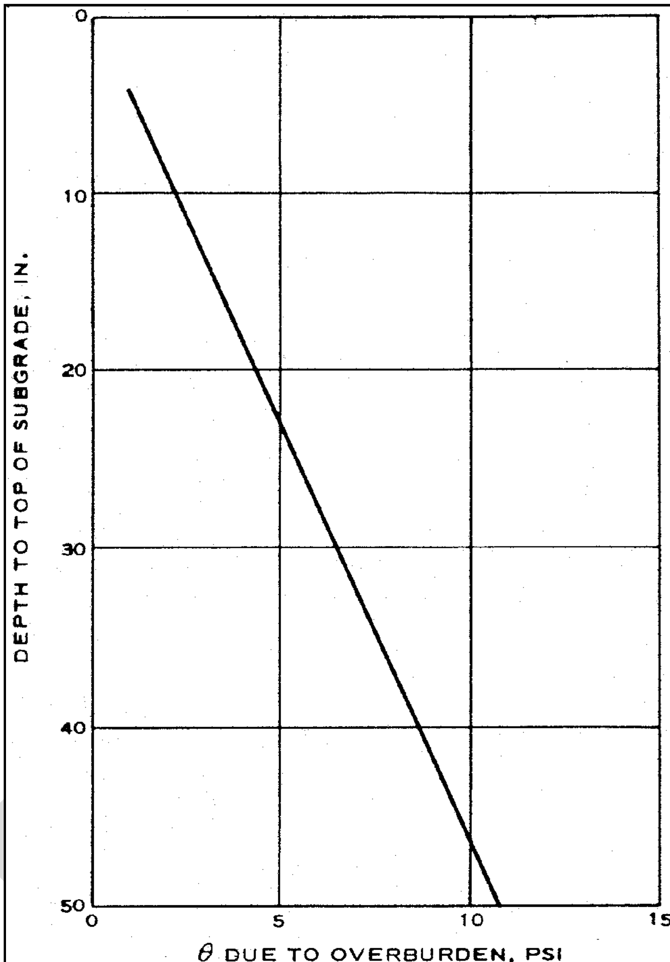


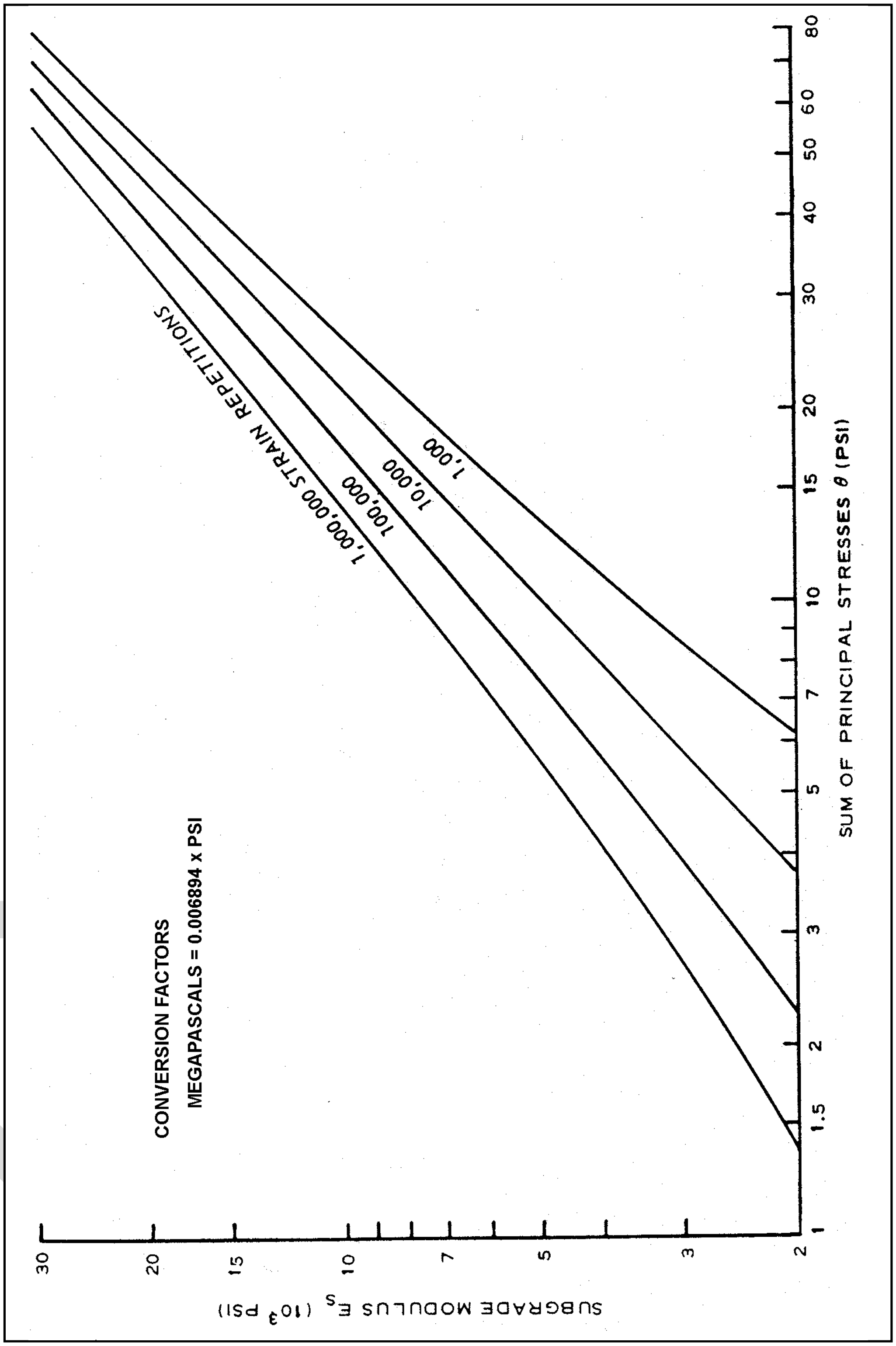
Figure B11-8. Relationship for Estimating θ Due to Overburden



CONVERSION FACTORS

MEGAPASCALS = 0.006894 x PSI
MILLIMETERS = 25.4 x INCHES

Figure B11-9. Estimated θ at Top of Subgrade



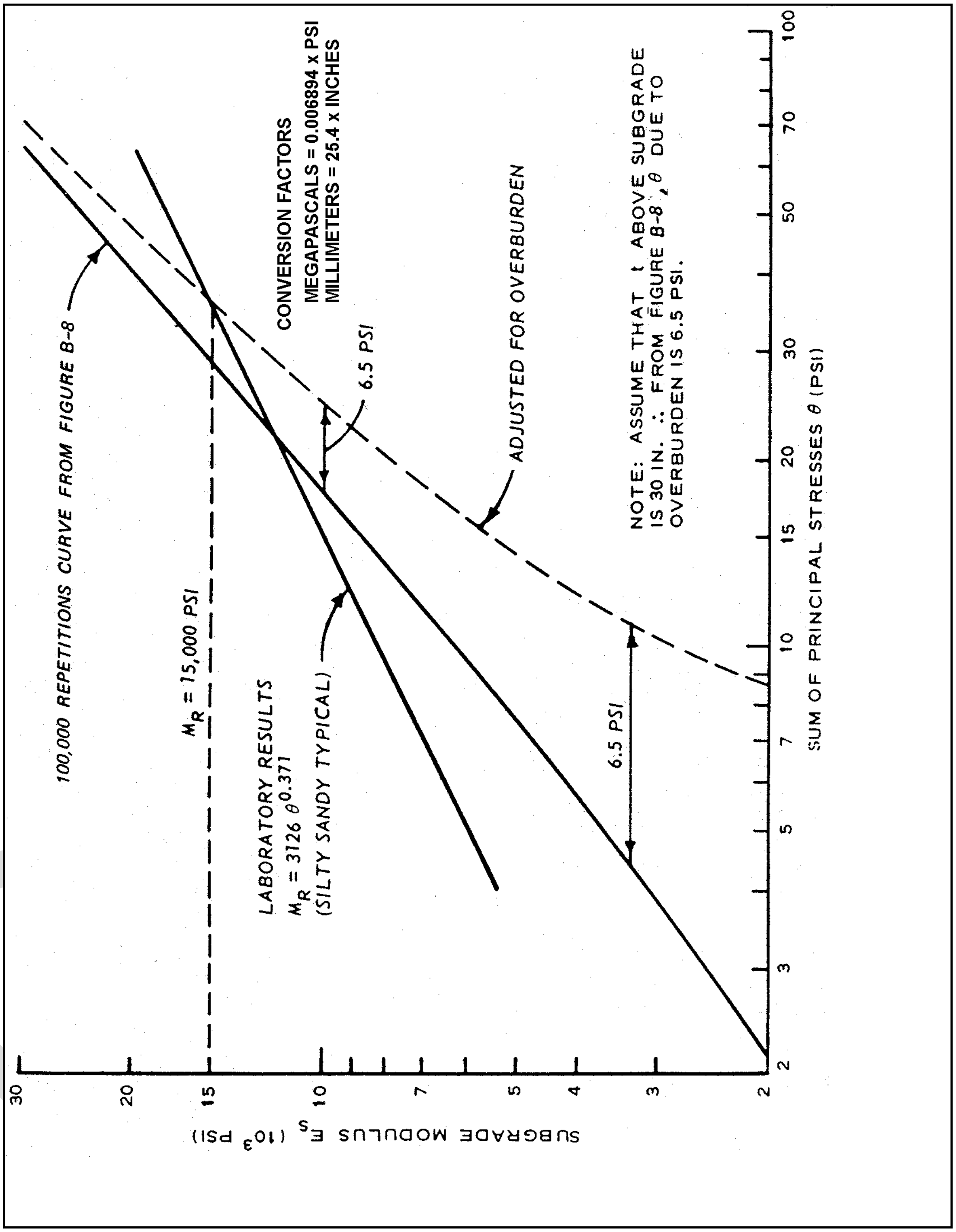


Figure B11-10. Selection of M_R for Silty Sand Subgrade with Estimated Thickness of 762 mm (30 in) for 100,000 Repetitions of Strain

SECTION 12: PROCEDURES FOR DETERMINING THE FATIGUE LIFE OF BITUMINOUS CONCRETE

B12-1 LABORATORY TEST METHOD

This chapter describes a laboratory procedure for determining the fatigue life of bituminous concrete paving mixtures containing aggregate with maximum sizes up to 31.8 mm (1.5 in). The fatigue life of a simply supported beam specimen subjected to third-point loading applied during controlled stress-mode flexural fatigue tests is determined.

B12-1.1 **Definitions.** These symbols are used in the description of this procedure:

$\in$ = initial extreme fiber strain (tensile and compressive, in per in)

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:

$$\in = \frac{12td}{(3L^2 - 4a^2)} \qquad \text{(B12-1)}$$

where

t = specimen depth, mm (in)

d = dynamic deflection of beam center, mm (in)

L = reaction span length, mm (in)

a = $L/3$, mm (in)

B12-1.2 Test Equipment

B12-1.2.1 **Repeated Flexure Apparatus.** The repeated flexure apparatus is shown in Figure B12-1. It accommodates beam specimens 381 mm (15 in) long with widths and depths not exceeding 76 mm (3 in). A 1,361-kg-capacity (3,000-lb-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 mm (4 in) of a 381-mm-long (15-in-long) beam specimen with widths and depths not exceeding 76 mm (3 in). 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.

B12-1.2.2 **LVDT.** The dynamic deflection of the beam's center is measured with an LVDT. An LVDT suitable for this purpose is the Schaevitz™-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 C (±0.5 degree F). A Missimer's model 100 x 500 carbon dioxide plug-in temperature conditioner has been found to provide suitable temperature control.

B12-1.3 **Specimen Preparation.** Beam specimens 380 mm (15 in) long with 59-mm (3.5-in) depths and 83-mm (3.25-in) widths are prepared according to ASTM D3202. 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-mm (3-in) or slightly less deep by 76 mm (3 in) or slightly less wide test specimens from the 380-mm-long (15-in-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 mm (0.01 in) at the center and at 51 mm (2 in) from both sides of the center. Mean values are determined and used for subsequent calculations.

B12-1.4 **Test Procedures.** Follow these procedures:

(1) Adjust repeated flexure apparatus loading clamps to the same level as the reaction clamps. Clamp the specimen in the fixture using a jig to position the centers of the two loading clamps 51 mm (2 in) from the beam center and to position the centers of the two reaction clamps 165 mm (6.5 in) from the beam center. Place double layers of Teflon sheets 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, apply repeated loads. The 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 suggested ranges of extreme fiber stress levels are listed in Table B12-1.

(3) Measure the beam center point deflection and applied dynamic load immediately after 200 load repetitions for calculation of extreme fiber strain $\in$. Continue the test 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 C (40 to 100 degrees F) 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.

Table B12-1. Suggested Ranges of Extreme Fiber Stress Levels

Temperature, degrees Celsius (degrees Fahrenheit)	Stress Level Range MPa (psi)
13 (55)	1.03 to 3.1 (150–450)
21 (70)	0.52 to 2.1 (75–300)
30 (85)	0.24 to 1.38 (35–200)

(4) 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.

B12-1.5 **Report and Presentation of Results.** The report of flexural fatigue test results should include these elements:

(a) Density of test specimens

(b) Number of load repetitions to fracture N_f

(c) Specimen temperature

(d) Extreme fiber stress σ

The flexural fatigue relationship is plotted in Figure B12-2.

B12-2 PROVISIONAL FATIGUE DATA FOR BITUMINOUS CONCRETE

Use of the graph shown in Figure B12-3 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 B12-1. Repeated Flexure Apparatus

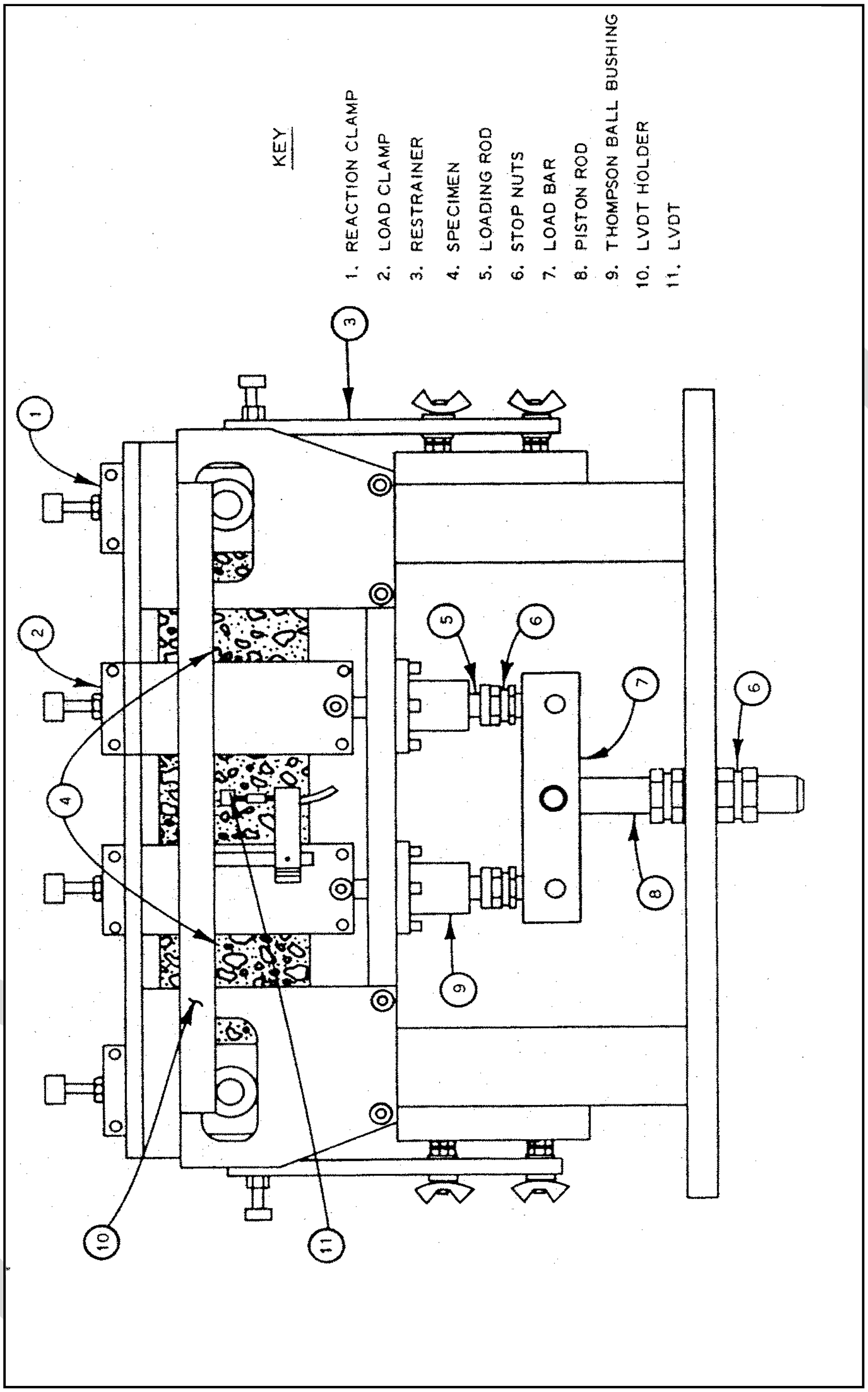
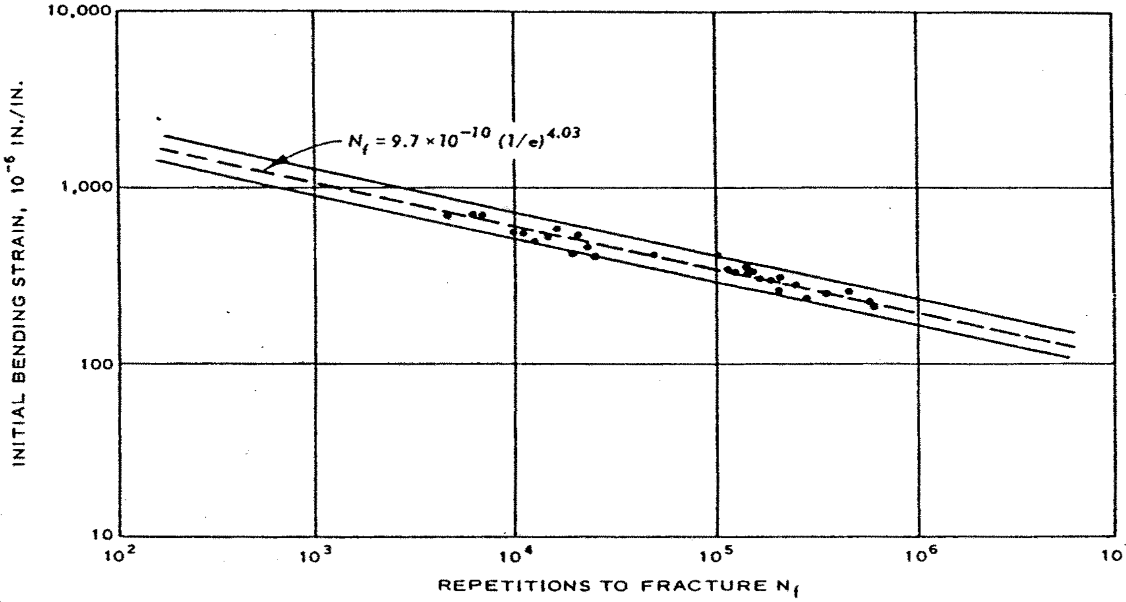


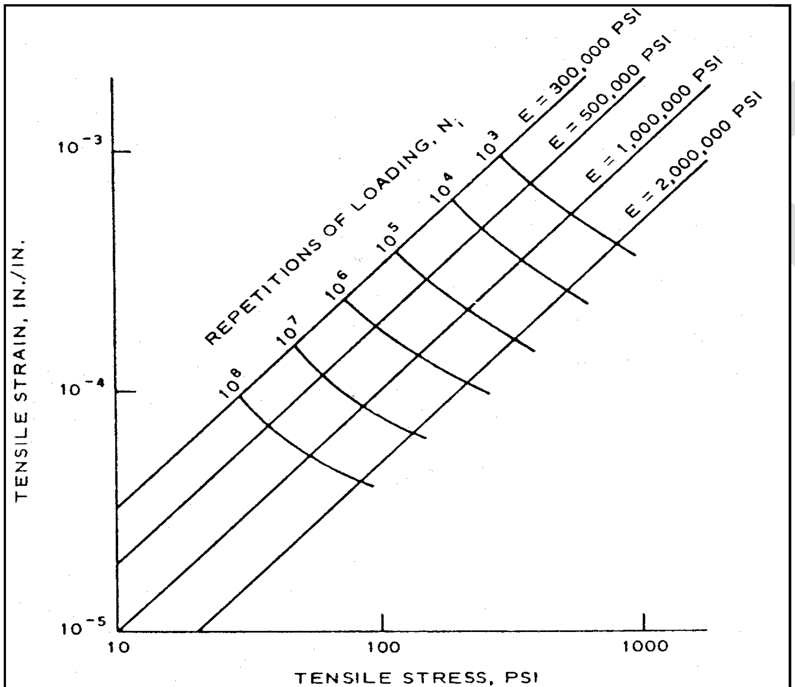
Figure B12-2. Initial Mixture Bending Strain versus Repetitions to Fracture in Controlled Stress Tests



CONVERSION FACTOR

MILLIMETERS PER MILLIMETER = 1.0 x INCHES PER INCH

Figure B12-3. Provisional Fatigue Data for Bituminous Base Course Materials



CONVERSION FACTORS

MEGAPASCALS = 0.006894 x PSI
MILLIMETERS PER MILLIMETER = 1.0 x INCHES PER INCH

SECTION 13: PROCEDURE FOR DETERMINING THE RESILIENT MODULUS OF GRANULAR BASE MATERIAL

B13-1 PROCEDURE

This procedure is designed to determine resilient properties of granular base (subbase) materials. The test is similar to a standard triaxial compression test, with the primary exception 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.

B13-2 DEFINITIONS

These 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

σ_d = deviator stress ($\sigma_1 – \sigma_3$), i.e., the repeated axial stress in this procedure

$\in_1$ = total axial strain due to σ_d

$\in_R$ = resilient axial strain due to σ_d

$\in_P$ = resilient lateral strain due to σ_d

M_R = the resilient modulus = $\sigma_d/\in_R$

v_R = the resilient Poisson's ratio = $\in_P/\in_R$

θ = sum of the principal stresses in the triaxial state of stress ($\sigma_1+2\sigma_3 = \sigma_d+3\sigma_3$).

σ_1/σ_3 = principal stress ratio

Load duration = time interval during which the sample is subjected to a stress deviator

Cycle duration = time interval between successive applications of the deviator stress

B13-3 SPECIMENS

For base course materials, 152-millimeter-diameter (6-in-diameter) specimens are generally required, with the maximum particle size being limited to 25 mm (1 in). The specimen height should be at least twice the diameter.

B13-4 EQUIPMENT

B13-4.1 **Triaxial Test Cell.** The triaxial cell shown schematically in Figure B13-1 is suitable for use in resilient testing of soils. The equipment is similar to most standard cells; however, a few specialized criteria must be met to provide acceptable test results. Generally, the equipment is slightly larger than most standard cells to accommodate the 152-mm-diameter (6-in-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 usually used as the confining fluid to avoid the detrimental effects of water on the internally mounted electronic measuring equipment.

B13-4.2 **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 B13-2) 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.

B13-4.3 **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 closed-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.

B13-4.4 **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 B13-1. Two LVDTs are used to measure axial deformations, and two are used to measure lateral deformations. Figures B13-3 and B13-4 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 mm (0.001 in) 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.

Figure B13-1. Triaxial Cell Used in Resilience Testing of Granular Base Material

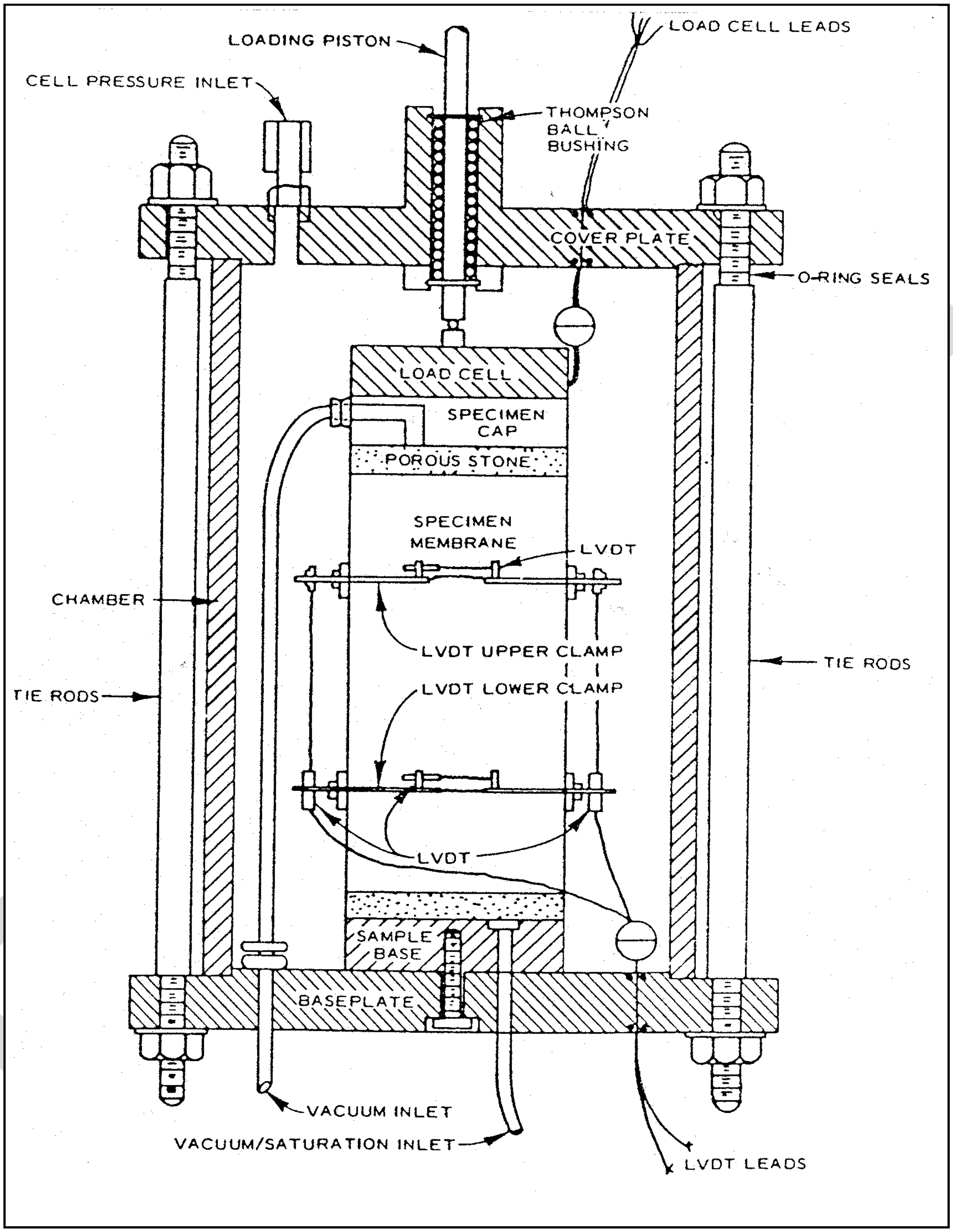


Figure B13-2. Schematic of Frictionless Cap and Base

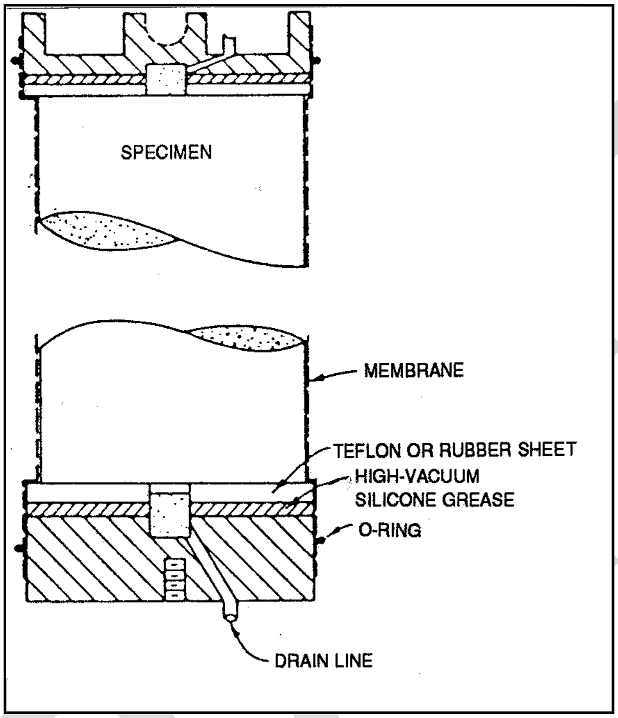


Figure B13-3. Details of Top LVDT Ring Clamp

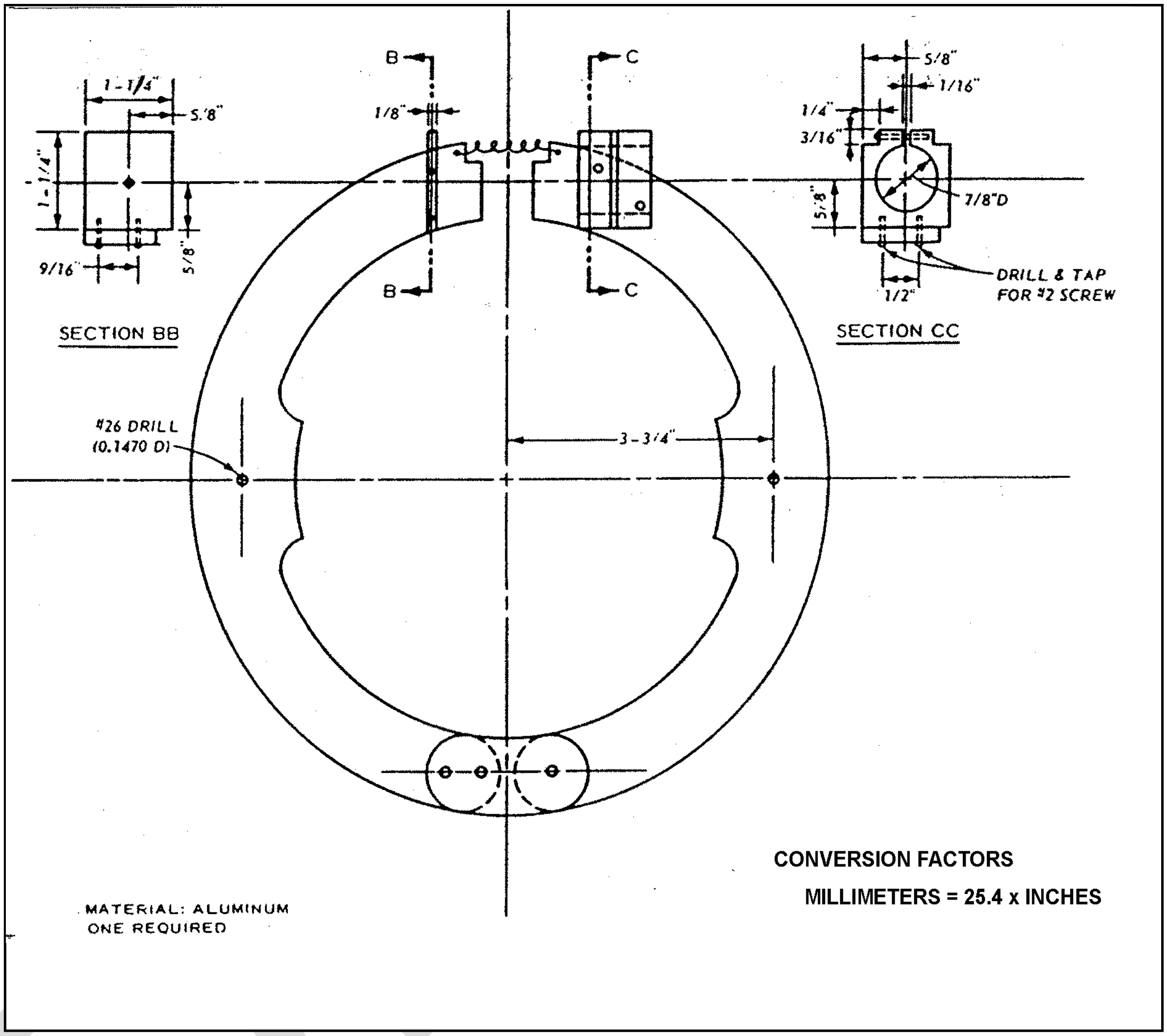
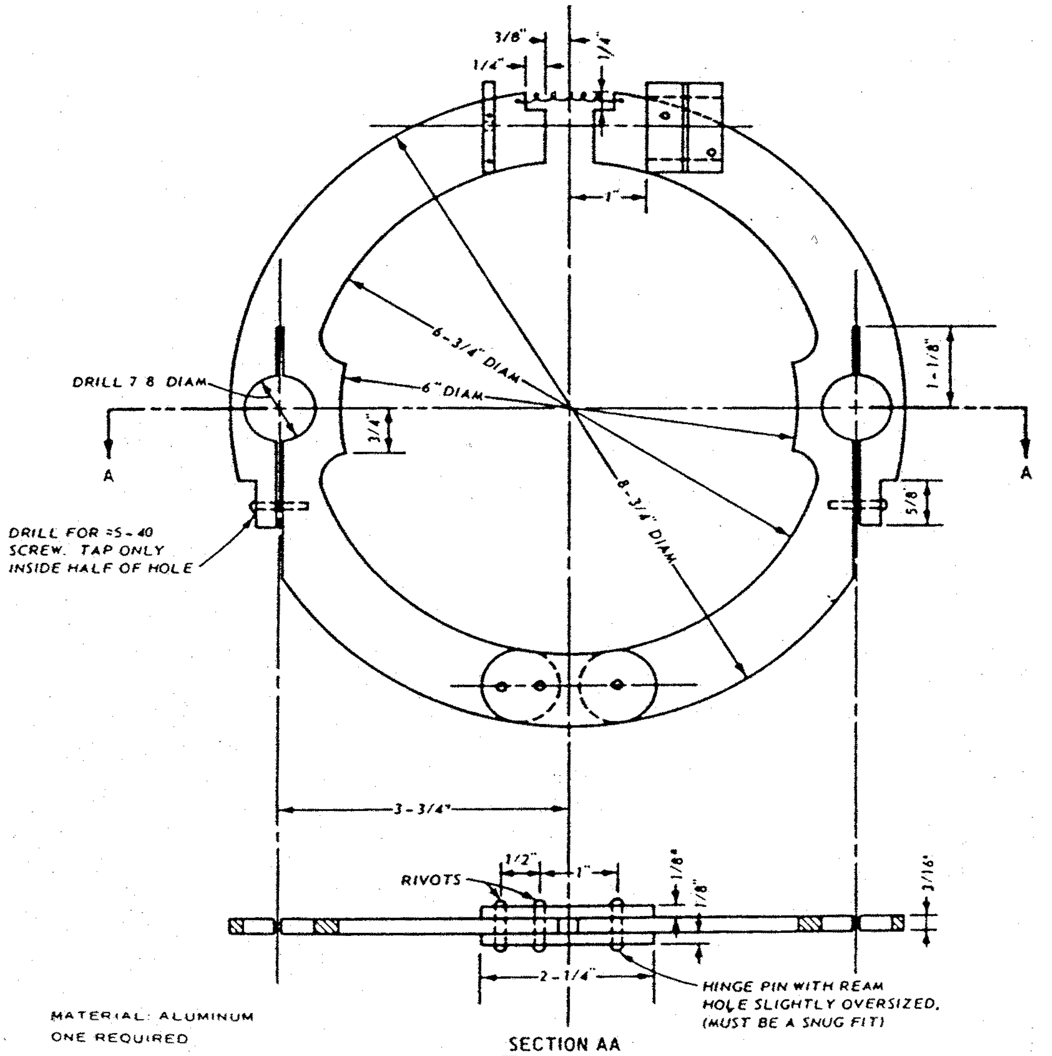


Figure B13-4. Details of Bottom LVDT Ring Clamp



CONVERSION FACTORS

MILLIMETERS = 25.4 x INCHES

B13-4.5 **Additional Equipment.** In addition to the equipment described in paragraphs B13-4.1 to B13-4.4, these items are also used:

- Calipers, a micrometer gauge, and a steel rule (calibrated to 0.25 mm [0.01 in])
- Rubber membranes (0.03 to 0.06 mm [0.012 to 0.025 in] thick) and a membrane stretcher
- Rubber o-rings
- Guide rods for positioning LVDT clamps
- Epoxy for cementing clamps to membrane
- A vacuum source with a bubble chamber (optional) and regulator
- A specimen forming jacket

B13-4.6 **Recommendations.** It is also necessary to have a fast recording system for accurate testing. It is recommended that for analog recording equipment, 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, a single-channel recorder can be used by introducing switching and balancing units.

B13-5 **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-mm-diameter (6-in-diameter) specimens are required with the maximum particle size being limited to 25 mm (1 in) in diameter.

B13-5.1 **Material Preparation.** The material should be air-dried and subsequently sufficient water should be 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.

B13-5.2 **Specimen Compaction.** Generally, base course materials are compacted on the triaxial cell base plate 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 permitted only 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 the specimen 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.

B13-5.3 **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 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, ±10-mm (0.040-in) LVDTs are used for radial deformation, and ±0.25-mm (0.100-in) 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 the gauge rods and assemble the triaxial chamber. Any shifting of LVDT clamps during chamber assembly will be noted by LVDT reading changes and can be accounted for.

B13-5.4 **Resilient Testing.** The resilient properties of granular materials are dependent primarily on confining pressure and to a lesser extent on 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. These are the steps of the testing procedure:

(1) Balance the recorders and recording bridges and record calibration steps.

(2) Apply approximately 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 the procedure over the range of deviator stresses to be used.

(5) After completing the stress states for the initial confining pressure, repeat for each successively 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.

B13-6 COMPUTATIONS AND PRESENTATION OF RESULTS

B13-6.1 **Computation.** These are the computations:

(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 these formulas:

(a) Resilient axial strain $\in_R = \Delta H_r / H_i$.

(b) Resilient lateral strain $\in_L = \Delta D_r / D_i$.

(c) Deviator stress $\sigma_d = \Delta P / A_O$.

(d) Resilient modulus $M_R = \sigma_d / \in_R$.

(e) Resilient Poisson's ratio $v_R = \in_L / \in_R$.

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

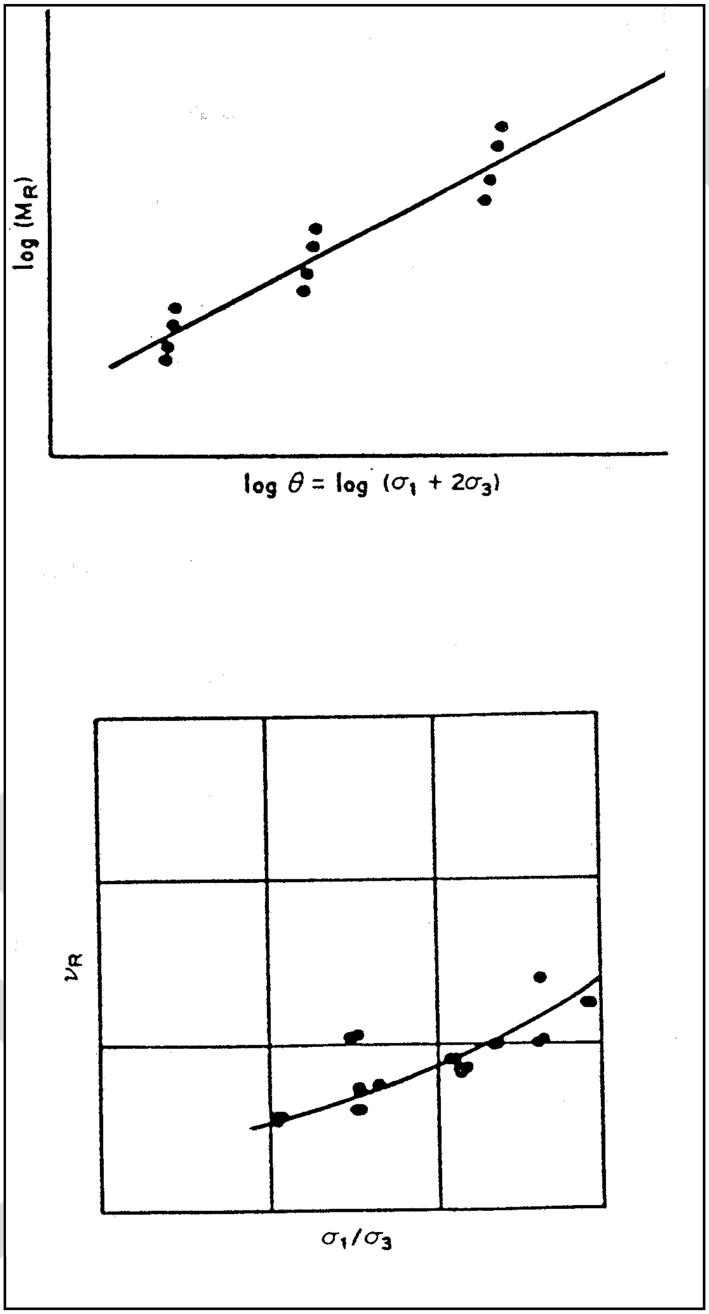
D_i = instantaneous diameter after specified number of load repetitions. Can be calculated from $D_o + \Delta D$.

D_o = initial specimen diameter

ΔD = permanent change in specimen diameter

B13-6.2 **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 B13-5). 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 B13-5. Representation of Results of Resilience Test on Cohesionless Soils



SECTION 14: EXAMPLE PROBLEMS USING PCASE FOR AIRFIELD/HELIPORT PAVEMENTS

B14-1 GENERAL INSTRUCTIONS FOR USING PCASE

The Pavement-Transportation Computer Assisted Structural Engineering (PCASE) computer program was created for use in the design and evaluation of transportation systems.

B14-1.1 Downloading PCASE

To download the PCASE software, open an Internet browser, enter the Web site address www.pcase.com, and follow these steps:

(1) At the top of the website page click on the **Software** button.

(2) Click on link **Download PCASE Desktop**.

(3) As soon as you click the link, a **File Download** dialog box will display. Click the **Save** button and store the file named PCASE209.zip in a directory on your hard drive. This does not install PCASE; this only downloads the installation file and instructions to your computer.

B14-1.2 Installing PCASE

Once you have downloaded the file, go to **My Computer** or **Windows Explorer** and browse to the folder that contains the file. Open the zip file. If you have a previous version of PCASE on your computer it is important to follow the installation instructions included in the zip file. If you do not have previous versions of PCASE double-click the file **pcasesetup209.exe** to start the installation process. Most users can simply select the **Next** button on all installation windows without making any changes. The installation creates an icon on your desktop called PCASE 2.09. Double-click this icon to start using PCASE. If you do not have write permissions to your computer you will need to contact your Information Technology department to install the software.

B14-1.3 Starting a Project File

Once you open the PCASE software, a Welcome to PCASE window displays. The tab headings are Create New Project, Open Existing Project, and Recent Projects.

B14-1.3.1 Creating a New Project

To create a new project, double-click the red Create New Project icon or click the **Open** button on the bottom right of the Open window. In the Enter New File Name window, enter your project file name. You don't need to add the **.pvr** extension; it will be added to the file name automatically. This creates a new database with a clean inventory, ready for data entry.

B14-1.3.2 Opening an Existing Project

To open an existing project, click the Open Existing Project tab. Scroll to the project file (the .pvr file, not the folder of the same name) and double-click the file name or highlight the file name and click the **Open** button.

B14-1.3.3 **Opening a Recently Created File**
To open a recently created file, click the Recent Projects tab. Scroll to the project file and double-click the file name or highlight the file name and click the **Open** button.

Note: In the Open window, you can access the Help System by clicking the **Getting Started** button; it takes you directly to the PCASE tutorials in the Help System.

B14-1.4 **Traffic Patterns**
All pavement designs are based on a planned set of traffic. Before beginning a pavement design, you must first build a traffic pattern.

B14-1.5 **Traffic Module**

B14-1.5.1 **Opening the Traffic Module**
The traffic module allows you to build a traffic model that will be used on the airfield that you are designing. To open the traffic module, click the **Traffic** button on the toolbar. Once the Traffic Module window opens, you have two choices: you can create a traffic pattern or choose a standard pattern.

B14-1.5.2 **Creating a Traffic Pattern**
When you click the **Create Pattern** button, the initial window for building the pattern will open. Fill in the pattern name, choose the design type, and specify whether the traffic will be entered for the entire design life or for multiple design periods (see descriptions below).

B14-1.5.2.1 **Pattern Name**
The pattern name is a user-defined name given to the traffic pattern.

B14-1.5.2.2 **Design Type**
Set the design type to either Airfield or Roads depending on whether this pattern will be for an airfield or road pavement analysis.

B14-1.5.2.3 **Passes Entered for Entire Design Life versus Multiple Design Periods**
If you select **Passes entered for entire design life** (the default), this tells the software that you are entering passes for the entire life of the pavement. Typically in design, the passes are entered for the entire life of the pavement. For example, if the you are designing a runway that will support 2,000 passes per year of a C-17 and the design life is for 20 years, then you will select **Passes entered for entire design life** and on a later window (after you add your vehicle to the pattern) you will enter 40,000 passes. For this same example, if you select **Multiple Design Periods**, then you would enter 20 for the number of design periods and then enter 2,000 for pass levels. If you select multiple design periods, there is the option of assigning various pass levels for various design periods and applying growth factors. The **Modify Vehicle** button brings up the window for applying these options.

B14-1.5.3 **Adding Vehicles to a Traffic Pattern**
After you click the **Ok** button on the Traffic Pattern window, you can start adding vehicles by clicking the **Add Vehicle** button. The **Air** button on the top of the window limits the display to aircraft only. The **Ground** button limits the display to ground

vehicles only. The **Both** button displays all vehicles, aircraft and ground. To select a vehicle for a traffic pattern, you must click the check box next to the vehicle name. Be sure to click inside the box or double-click the vehicle name. You can use the scrollbar on the right side to scroll down to see more vehicles. Once you have selected all the vehicles for your design, click the **Add** button at the bottom of the window. The **Cancel** button cancels the action of adding vehicles to the traffic pattern and returns you to the Traffic Module window.

B14-1.5.4 **Changing Loads and Pass Levels for Airfield Traffic**
After you select vehicles to add to the traffic pattern and click **Add**, the Add Vehicle window closes and your selected list of vehicles and their default loads and pass levels are imported to the Traffic Module window. Edit the weights and pass levels as needed. (See the field descriptions below.) Click the **Apply** button to send the traffic information to the other modules.

- **Vehicle:** List of vehicles selected for the pattern.
- **Traffic Area Weight Column - Areas A, B:** Displays the full aircraft weight. The weight can be changed by clicking the cell displaying the weight and entering the new weight.
- **Traffic Area Weight Column - Areas C, D:** Displays the aircraft weight reduced by 25 percent. If the full weight was changed in the Areas A, B column, the weight in Areas C, D will automatically be reduced by 25 percent.
- **Traffic Area Pass Levels - Areas A, B, C:** Displays the pass level for the design life of the pavement. The default is 100 passes, but you must enter the number of passes (by clicking the cell displaying the pass level and typing the new number of passes) for each aircraft for the life of the pavement. For example, enter 40,000 passes for a 20-year design life at 2,000 passes per year.
- **Traffic Area Pass Levels - Area D:** Displays 1 percent of the passes entered for the design life (1 percent of Areas A, B, C). The passes in Area D will automatically be reduced based on passes entered in Areas A, B, C.

B14-1.5.5 **Analysis Type: Mixed Traffic vs. Individual Traffic**
By setting the analysis type, you can select how to use your traffic pattern. Choose from these analysis types:

- **Mixed:** Mixed analysis takes all of the vehicles in your traffic pattern and reduces the mix to one controlling vehicle and equivalent pass level. Only the controlling vehicle will be used for the design. For pavement designs using the empirical (CBR/K) model, the mixed traffic analysis is used. Note: in the Design module if you are using the empirical model for design it will automatically use a mixed traffic analysis.

- **Individual:** Individual analysis will analyze each vehicle in the traffic pattern. This type of pattern should be used only for layered elastic design. Note: in the Design module if you are using the layered elastic model for design it will automatically use an individual traffic analysis.

- **Cumulative Group or Cumulative Mixed:** Cumulative group and mixed analysis are used for Navy evaluations only.

B14-1.5.6 **Using a Standard Pattern**

Instead of creating a pattern, you have the option of using a pattern that has been built for you according to criteria. These include such patterns as the Air Force Medium, Navy Design Group II, or Army Class IV. To select a standard pattern, click the **Choose Standard Pattern** button from the Traffic Module window. To select a standard pattern, you must select (click) the check box next to the pattern name. Be sure to click inside the square or double-click the pattern name. You can use the scrollbar on the right side to scroll down for more patterns. Once you have selected your pattern, click **Ok**. This will make a copy of the standard pattern. Any changes made to the pattern after it is selected will not be made permanently to the standard pattern list. Click the **Apply** button to send the traffic information to the other modules.

B14-1.5.7 **Other Traffic Buttons and Options**

- **Delete Pattern:** Deletes the current traffic pattern from the database.

- **Modify Pattern:** Allows you to rename the traffic pattern.

- **Copy Pattern:** Makes a duplicate copy of the current traffic pattern.

- **Import Patterns:** Allows you to import a traffic pattern from another project.

- **Edit Standard Patterns:** Allows you to edit the standard patterns. The existing standard patterns are based on criteria, and it is strongly recommended that you ***do not edit the standard patterns***.

- **English/Metric:** Allows you to toggle between metric and English units.

B14-1.6 **Design Module**

B14-1.6.1 **Starting the Design Module**
Now that you have built a traffic pattern, you are ready to open the design module to create a pavement design. Click the **Design** button on the toolbar to open the PCASE Design Module window.

B14-1.6.2 **Building a New Design**
With the PCASE Design Module window open, click the **Add** button under the top grid to create a new pavement design. The Add Design window will display requiring the information described below:

- **Design Name:** Enter a user-defined name for the design.
- **Design Type:** Select either Airfield or Roads.
- **Pavement Type:** Select the pavement type you are designing: flexible, rigid, unsurfaced, mat, flexible shoulder, or rigid shoulder.
- **Analysis Type:** Select the design model: empirical (CBR) or layered elastic (LED).
- **Traffic Area:** For airfield designs, select the traffic area for your design.
- **Traffic Pattern:** Select the traffic pattern you created in the traffic module from the drop-down list. You can view the vehicles in a traffic pattern by clicking the **View Traffic Pattern** button.

Once you enter the appropriate fields, click the **Ok** button at the bottom of the window to continue. A series of windows will display to help you build a layer structure. A separate window will display for each required layer in the structure.

B14-1.6.3 **Modifying a Design**
After you have created a design, you may modify it by selecting it and clicking the **Modify** button under the top grid. You may change the file name, traffic area (for airfields only), or traffic pattern. The pavement type, design type (airfield or roads), and analysis type cannot be changed. The file name, traffic area (for airfields only), and traffic pattern parameters can also be modified on the main window by clicking the cell of the value to be changed and typing the new value or selecting an item from a drop-down list. After you modify your design, be sure to click **Save and Calculate** to recalculate and save your design.

B14-1.6.4 **Copying a Design**
Once you have created a design, you may copy it by selecting it and clicking the **Copy** button under the top grid. You must then assign a new design name to your copy. You may then modify the copy for a comparison of similar designs. After you modify your design, be sure to click **Save and Calculate** to recalculate and save your design.

B14-1.6.5 **Deleting a Design**

After you have created a design, you may delete it by selecting it and clicking the **Delete** button under the top grid. You will be asked to confirm the deletion.

B14-1.6.6 **Creating a Flexible Pavement Design: CBR Criteria**
On the Add Design window, if you choose to design a flexible pavement using the CBR criteria, a series of windows will display requiring the input described in paragraphs

B14-1.6.6.1 through B14-1.6.6.4. The first window is for the asphalt layer, followed by windows for the base course, drainage layer (if drainage is required, there will be a window for the separation layer), and subgrade. After you have entered the required information on each window, click the **Ok** button to continue. After you have entered all the layer information, click **Save and Calculate** to calculate your design.

B14-1.6.6.1 **Analysis**
You can manually input the thickness of your asphalt layer or have the software compute the thickness. Leaving this option on the default of Compute instructs the PCASE software to calculate the thickness required.

B14-1.6.6.2 **CBR**
The California Bearing Ratio indicates the strength of the layer. On the base course window, you can select a CBR of 100 (crushed graded aggregate) or 80 (unbound aggregate). On the subgrade window, you must manually enter the CBR of your subgrade.

B14-1.6.6.3 **Drainage**
In PCASE, the drainage layer is optional. If a drainage layer is not required, click the **Not Required** button and the software will bypass the drainage requirements. If the drainage layer is required, click the **Required** button and a second window will display with a **Compute Drainage Layer** button (refer to paragraph B14-1.6.10, "Using the Drainage Layer Worksheet") or you can manually input the thickness of the drainage layer. If you create a drainage layer, you will also be prompted for information on a separation layer.

B14-1.6.6.4 **Thickness**
In flexible design, you will input a thickness only for layers for which you select manual analysis; otherwise, all thicknesses will be computed for you.

B14-1.6.7 **Adding Layers**
Once you have completed the initial design, you may want to add layers that were not included in the step-by-step design builder (e.g., add a subbase to your flexible structure or add an overlay). To add a layer, click the **Add** button at the bottom of the window under the layer grid. The Add Layer window displays with list boxes for Layer Type and Material Type. (See the descriptions of layers that can be added below.) Select the layer type to be added and the material type (as applicable) and then click **Ok**. A series of windows prompts you for information about the layer. Enter the appropriate information and click **Ok**. After the last of these windows closes, click **Save and Calculate** on the PCASE Design Module window to recalculate your design.

These layers can be added to a flexible design:

- Asphalt Overlay: No additional information is required.
- Stabilized Base: Select the material type.
- Drainage/Separation: Refer to "Using the Drainage Layer Worksheet" (paragraph B14-1.6.10).
- Drainage/Geotextile: Refer to "Using the Drainage Layer Worksheet" (paragraph B14-1.6.10).
- Stabilized Subbase: Select the material type.
- Subbase: Input the CBR of the material.
- Select Fill. Select the material type and input the thickness and the CBR of the material.
- Modified/Stabilized Subgrade: Select the material type and input the thickness and the CBR of the material.
- Natural Subgrade: Select the material type and input the CBR of the material.

B14-1.6.8 **Changing a Layer**
Once you have completed the initial design, you may want to change a layer (e.g., use a stabilized base course instead of an aggregate base course, or use a geotextile instead of a separation layer). To change a layer, click the **Add** button at the bottom of the window, under the layer grid. The Add Layer window displays with a list box of layer options. Select the layer to be added and then click **Ok**. You will then be prompted for information about the layer. Fill in the appropriate information and click **Ok**. You will then be returned to the PCASE Design Module window. The software will automatically replace the appropriate layer. For example, if you are replacing the separation layer with a geotextile, you only need to add the geotextile; the software will automatically take out the separation layer and replace it with the geotextile. Once the layers are in place, click **Save and Calculate** to recalculate your design.

B14-1.6.9 **Modifying Layers**
You may change layer data in several ways. You may change values for a specific layer directly on the PCASE Design Module window by typing the new value in the appropriate cell or by selecting the new value from a drop-down list. Alternatively, you may select a layer and click the **Edit** button at the bottom of the window and change the values on the window for the specific layer. The final method is to click the **Layer Details** button at the bottom of the PCASE Design Module window and change the values on the Layers window, which displays values for all layers in your design. After making any change, be sure to click **Save and Calculate** to recalculate and save your design.

B14-1.6.10 **Using the Drainage Layer Worksheet**
When you click **Compute Drainage Layer** on the Drainage window, the Drainage Worksheet window displays. To navigate through the Input Parameters, press the TAB key to go to the next parameter or use your mouse and click the next text box. The worksheet will update the calculations only after you proceed to the next parameter. After you have entered a value in the last text box, click on the **Calculate Drainage** button for the Output Parameters at the bottom of the window. Input parameter descriptions are provided in paragraphs (a) through (m) below. Once you have entered the required information, click the **Ok** button. Your required subsurface drainage thickness will be transferred to the design module.

(a) **Design Storm Index**
Double-click inside this box to open the Precipitation window (precipitation database). You can select the design storm index for a given location by selecting the state or country (countries are listed after the states) from the Regions list box on the left. Selecting a state or country will populate the Stations list box on the right with the weather stations located in your selected state or country. Select the closest weather station for your design. Storm drain indices for various years will display at the bottom of the window. Clicking **Ok** on the Precipitation window will close the window and import the 2-year storm data to the Drainage Worksheet window. You can overwrite the value by clicking in the cell and typing in a value.

(b) **Length of Drainage Path**
Total distance (longitudinal and transverse) that water must travel to exit the pavement. This value can be entered manually or it can be calculated by clicking the **Enable Drainage Path Calculations** check box.

(c) **Length of Transverse Slope of Drainage Layer**
Horizontal distance from the crown to the exit point of the pavement structure.

(d) **Transverse Slope of Drainage Layer**
Slope of the pavement from the crown to the exit point of the pavement structure.

(e) **Longitudinal Slope of Drainage Layer**
Slope of the pavement along the length of the pavement from the highest to lowest point.

(f) **Permeability of Drainage Material**
A measure of the rate at which water passes through a unit area of material in a given amount of time under a unit hydraulic gradient. Double-click the cell for typical values.

(g) **Effective Porosity**
The effective porosity is defined as the ratio of the volume of voids that will drain under the influence of gravity to the total volume of a unit of aggregate. Double-click the cell for typical values.

(h) **Slope of Drainage Path**
Slope (longitudinal and transverse) that water must travel to exit the pavement. This value can be entered manually or it can be calculated by clicking the **Enable Drainage Path Calculations** check box.

(i) **Infiltration Coefficient**
Ratio of infiltration to rainfall. The infiltration coefficient will vary over the life of the pavement depending on the type of pavement, surface drainage, pavement maintenance, and structural condition of the pavement. Since the determination of a precise value of the infiltration coefficient for a particular pavement is very difficult, a value of 0.5 is assumed for design.

(j) **Required Thickness**
Thickness required to meet the drainage criteria. In comparing calculated versus minimum thickness, the required thickness will be the greater of the two.

(k) **Minimum Thickness**
Minimum thickness required to meet the drainage criteria.

(l) **Calculated Thickness**
Thickness of the drainage layer based on input.

(m) **Time for 85% Drainage (T85)**
The amount of time it will take 85 percent of the water to exit the pavement. Criteria require that T85 be less than 1 day unless designing for a parking apron or other areas of low volume and slow moving traffic, for which T85 is required to be less than 10 days. If T85 is greater than required, adjust the parameters by increasing the permeability or shortening the drainage path length.

B14-1.6.11 **Frost Design**

B14-1.6.11.1 **Calculating a Depth of Frost**
In the design module, the button for opening the depth of frost calculator is located in the top grid in the Depth of Frost column. To open the calculator, click the **…** (dot-dot-dot) button in the Depth of Frost column on the row of your design. **Note:** This button does not appear for layered elastic designs; seasonal variations are calculated in another manner.

In the design module, a standard non-frost design must be calculated in order to calculate a depth of frost. This gives the depth of frost calculator a set of existing thicknesses to use for the frost thickness determination. When you click the **…** button in the Depth of Frost column, the Depth of Frost Penetration Calculator window (worksheet) will display. Paragraphs (a) through (g) below provide descriptions of the Depth of Frost Penetration Calculator worksheet and the required input. Once you have entered the required information, click the **Calculate** button and then the **Apply and Close** button. Your depth of frost calculation will be complete and will be transferred to the design module. You can overwrite the value by clicking in the cell and typing in the value.

(a) **Select a State or Country**
Select the state or country (countries are listed after the states) for which the pavement design or evaluation will be performed. Selecting a state or country from the list box on the left will populate the list box on the right with the weather stations located in your selected state or country.

(b) **Select a Station**
Select the weather station closest to the area for which the pavement design or evaluation will be performed.

(c) **Layer Type**
The non-frost layer structures you built in the design process are imported into the form. The layers in the Depth of Frost Penetration Calculator correspond to the layers in your design. For example, AC refers to the asphalt layer, Course Grained refers to the base course, and Fine Grained refers to the natural subgrade.

(d) **Dry Unit Weight**
Enter the dry unit weight of each material in your structure.

e) **Moisture Content**
Enter the moisture content of each material in your structure.

(f) **Calculate**
Once you have entered the dry unit weights and moisture contents, click the Calculate button to perform the depth of frost calculation.

(g) **Apply & Close**
Clicking this button will complete the depth of frost calculation by closing the window and transferring the value to the PCASE Design Module window.

B14-1.6.11.2 **Assigning Frost Codes to Layers**
Once you have calculated or entered the depth of frost, a new column labeled **Frost Code** appears in the layer grid for the current design. You can then assign a frost code to the subgrade by clicking the cell for a drop-down list of available frost codes. If the subgrade is not frost susceptible, then a frost design is not necessary and will not be calculated. After you have entered the frost codes, click **Save and Calculate** to finish the frost design calculations.

B14-1.6.12 **Creating a Flexible Shoulder Design: CBR Criteria**
For shoulder design, you do not need to have a traffic pattern; you can go directly to the design module. *However, if you just opened a new file and have not applied a traffic pattern, you will need to do so. Any traffic pattern (standard or created) will work. The software is set up so that a traffic pattern is necessary for the design module to open.* The design module includes a pavement type that meets the shoulder criteria of 5,000 coverages of a 4,535-kg (10,000-lb) single wheel load with a tire pressure of 0.69 Mpa (100 psi). To use the shoulder criteria, go directly to the design module by clicking the **Design** button to open the PCASE Design Module window. Then follow these steps:

(1) Click the **Add** button to add a design.

(2) On the Add Design window, enter a design name in the Design Name text box.

(3) From the Design Type list box, select Airfield.

(4) From the Pavement Type list box, select Flex Shoulder.

(5) Click **Ok**.

(6) After you enter the required information on each window, click **Ok** to continue.

(7) After you enter all the layer information and you add any additional layers required, click **Save and Calculate** to calculate your design.

(8) If the pavement will be used by support vehicles (e.g., snow removal equipment, fire trucks, fuel trucks), the shoulder should be checked by creating a road design. You will need to create a traffic pattern that includes the support vehicles using the shoulder and create a design using Roads as your design type. Compare the results; the governing design will be the one that produces the thickest section.

B14-1.6.13 **Creating a Flexible Pavement Design: LE Criteria**
On the Add Design window, if you choose to design a flexible pavement using the LED criteria, a series of windows will display requiring input as described in paragraphs (a) through (f) below. The first window is for the asphalt layer, followed by windows for the base course, drainage layer (if drainage is required, there will be a window for the separation layer), and subgrade. In PCASE, the primary difference between an empirical design and a layered elastic design is the use of modulus values for layer strengths instead of CBR values. After you enter the required information on each window, click **Ok** to continue. After you enter all the layer information, click **Save and Calculate** to calculate your design. The Damage curves are available for layered elastic designs (only) by clicking the **Damage** button located at the bottom the design window. For flexible pavements the damage to the surface and subgrade can be viewed. The damage per season can also be displayed by selecting the Seasons radio button. The curves are not cumulative; the damage (per vehicle or season) is determined by the

area under the curve. Click on the **Print** button to open up the curve in Excel for printing or saving. The **Exit** button will close the curves and return you to the main design window. The Damage to Surface and Subgrade indicates the cumulative damage factor. The goal is to have 100% of the life (CDF=1.0) used up for the surface and subgrade plus or minus 1%.

(a) **Analysis**
You can manually input the thickness of your asphalt layer or have the software compute the thickness. Leaving this option on the default of Compute instructs the PCASE software to calculate the thickness required.

(b) **Modulus**
Modulus of elasticity defaults are set at 2,413 MPa (350,000 psi) for asphalt, 421 MPa (61,000 psi) for base course and drainage layers, and 103 MPa (15,000 psi) for the subgrade. The modulus will vary with temperature and moisture.

(c) **Poisson's Ratio**
Defaults are set at 0.35 for the asphalt, base, and drainage layers, and 0.40 for the subgrade. 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, use of the defaults is recommended.

(d) Slip
Slip refers to the interface value between layers. Slip should be set at 0 for complete adhesion between the layers or 1,000 for almost frictionless slip between the layers. Values between 0 and 1,000 may be inputted to simulate varying degrees of friction. Almost frictionless slip is usually assumed at the bottom of a PCC layer and full adhesion is generally assumed for most other pavement materials. In PCASE, Slip defaults to 1,000 for PCC and 0 for all other material types.

(e) **Drainage**
In PCASE, the drainage layer is optional. If a drainage layer is not required, click the **Not Required** button and the software will bypass the drainage requirements. If the drainage layer is required, click the **Required** button and a second window will display with a **Compute Drainage Layer** button (refer to paragraph B14-1.6.10, "Using the Drainage Layer Worksheet") or you can manually input a thickness of the drainage layer. If you create a drainage layer, you will be prompted for information on a separation layer.

(f) **Thickness**

In flexible design, you will input a thickness only for layers for which you select manual analysis; otherwise, all thicknesses will be computed for you.

B14-1.6.14 **Creating Season Sets for Layered Elastic Designs**
One of the most powerful features of the layered elastic criteria is the ability to model your design to account for changing conditions—to assign different modulus values to different seasons of the year.

When you design a pavement using layered elastic criteria, PCASE defaults to one annual season. To build a custom season (or assign the percentage of the year per season), follow these steps:

(1) Go to the Seasons form by clicking the **Edit Seasons** button located on the right side of the design window under the design grid. A dialog box will display indicating that your values (e.g., modulus, Poisson's ration) will be reset to the defaults.

(2) Click **Ok**; you will be entering new values for each season anyway.

(3) On the Seasons window, click the **Add** button to build a new season set.

(4) Enter a name to identify your season set (e.g., 3 seasons, 4 seasons, my seasons); then click **Ok**. You will then divide the year into various seasons. The easiest way to create the seasons is to click the **end date** of each season (e.g., Feb 28, May 31, Aug 15, Dec 31). PCASE will then assign a percentage of the year to each season.

(5) When you have completed building a season set, click **Exit** to return to the PCASE Design Module window.

(6) On the PCASE Design Module window, click the **…** (dot-dot-dot) button in the Seasons column of the design grid to assign the newly created season set to your design.

(7) When the Seasons window displays, select the appropriate season set and then click **Ok**. The Seasons window closes and the season you created is applied to the appropriate design on the PCASE Design Module window.

(8) You must now enter modulus values, Poisson's ratios, and slip for each of the seasons. Enter the values by clicking on the cell and typing in the value for each layer. Above the layer grid, on the right side, is a scrollbar indicating the season. To go to the next season, click the **>** (right arrow) button. Enter the modulus values, Poisson's ratios, and slip for each layer. Continue scrolling to the next season and entering modulus values until you have changed the values for all seasons.

(9) After you enter all seasons of modulus values, click **Save and Calculate**.

B14-1.6.15 **Creating a Rigid Pavement Design: CBR Criteria**
On the Add Design window, if you choose to design a rigid pavement using the K criteria, a series of windows will display requiring the input described in paragraphs (a) through (g) below. The first window is for the rigid layer, followed by windows for the drainage layer (if drainage is required, there will be a window for the separation layer) and subgrade. After you enter the required information on each window, click **Ok** to continue. After you enter all the layer information, click **Save and Calculate** on the PCASE Design Module window to calculate your design.

(a) **Flexural Strength**
Enter the flexural strength of the new concrete to be placed.

(b) **Percent Steel**
If the new concrete will be reinforced, enter the percent steel to be used for reinforcement.

(c) **% Joint Load Transfer**
For airfields, the design charts were developed based on a 25 percent load transfer value, which is the default in PCASE. A load transfer value of 25 percent is typically used as a reasonable approximation of the load transfer measured over time on the types of joints approved for use on airfields. You can override the default by entering a new value.

(d) **Modulus**
This refers to the modulus of elasticity of concrete. The default is 27,579 MPa (4,000,000 psi).

(e) **Poisson's Ratio**
The default for the Poisson's ratio of the concrete material is 0.15.

(f) **Drainage**
In PCASE, the drainage layer is optional. If a drainage layer is not required, click the **Not Required** button and the software will bypass the drainage requirements. If the drainage layer is required, click the **Required** button and a second window will display with a **Compute Drainage Layer** button (refer to paragraph B14-1.6.10, "Using the Drainage Layer Worksheet"), or you can manually input a thickness of the drainage layer. If you create a drainage layer, you will also be prompted for information on a separation layer.

(g) **Thickness**
In rigid design, you will input a thickness only for layers for which you select manual analysis; otherwise, all thicknesses will be computed for you.

B14-1.6.16 **Adding Layers**

After you have completed the initial design, you may want to add layers that were not included in the step-by-step design builder (e.g., add a base course to your rigid structure, add an overlay). To add a layer, follow these steps:

(1) Click the **Add** button at the bottom of the window, under the layer grid. The Add Layer window will display (see the descriptions of layers that can be added below).

(2) From the Layer Type list box, select the layer to be added; click **Ok**.

(3) A layer window displays to prompt you for information about the layer. Fill in the appropriate information and then click **Ok**.

(4) The layer window closes. On the PCASE Design Module window, click **Save and Calculate** to recalculate your design.

These layers can be added to a rigid design:

- Base. Input thickness and a K value for the base if K was measured on top of the base course, otherwise leave K at 0 (default) and the software will calculate and use the effective K, which is based on the K of the subgrade and thickness of base course material.

- Stabilized Base. Select the material type and input the modulus of the material.

- Asphalt Overlay. No additional information is required.

- PCC Fully Bonded Overlay. Input the flexural strength of the new overlay (4.48 MPa [650 psi] default), percent steel (0 default), percent joint load transfer (25 percent default), modulus (27,579 MPa [4,000,000 psi] default), and Poisson's ratio (0.15 default).

- PCC Unbonded Overlay. Input the flexural strength of the new overlay (4.48 MPa [650 psi] default), percent steel (0 default), condition factor of the existing pavement (use the default of 1.0 for pavements in good condition with little or no structural cracking due to load; 0.75 for pavements exhibiting initial cracking due to load but no progressive cracking or faulting of joints or cracks; 0.35 for pavements exhibiting progressive cracking due to load accompanied by spalling, raveling, or faulting of cracks and joints; or a value in between to indicate the existing condition of the pavement), percent joint load transfer (25 percent default), modulus (27,579 MPa [4,000,000 psi] default), and Poisson's ratio (0.15 default).

- PCC Partially Bonded Overlay. Input the flexural strength of the new overlay (4.48 MPa [650 psi] default), percent steel (0 default), condition factor of the existing pavement (use the default of 1.0 for pavements in good condition with little or no structural cracking due to load; 0.75 for pavements exhibiting initial cracking due to load but no progressive cracking or faulting of joints or cracks; 0.35 for pavements exhibiting progressive cracking due to load accompanied by spalling, raveling, or faulting of cracks and joints; or a value in between to indicate the existing condition of the pavement), percent joint load transfer (25 percent default), modulus (27,579 MPa [4,000,000 psi] default), and Poisson's ratio (0.15 default).

- Drainage/Separation. Refer to paragraph B14-1.6.10, "Using the Drainage Layer Worksheet."
- Drainage/Geotextile. Refer to paragraph B14-1.6.10, "Using the Drainage Layer Worksheet."
- Natural Subgrade. Select the material type and input the K of the material.

B14-1.6.17 **Creating a Rigid Shoulder Design: CBR Criteria**
For shoulder design, you do not need to have a traffic pattern; you can go directly to the design module. *However, if you just opened a new file and have not applied a traffic pattern, you will need to do so. Any traffic pattern (standard or created) will work. The software is set up so that a traffic pattern is necessary for the design module to open.* The design module provides a pavement type that meets the shoulder criteria of 5,000 coverages of a 4,535-kg (10,000-lb) singe wheel load with a tire pressure of 0.69 Mpa (100 psi). To use the shoulder criteria, follow these steps:

(1) Open the design module by clicking the **Design** button.

(2) On the PCASE Design Module window, select the **Add** button under the design grid.

(3) On the Add Design window, enter a design name in the Design Name text box.

(4) From the Design Type list box, select Airfields.

(5) From the Pavement Type list box, select Rigid Shoulder.

(6) Click **Ok**.

(7) After you enter the required information on each window, click the **Ok** button to continue. After you enter all the layer information and you add any additional layers, click **Save and Calculate** to calculate your design. If pavements will be used by support vehicles (e.g., snow removal equipment, fire trucks, fuel trucks), the shoulder should be checked by creating a road design. You will need to create a traffic pattern that includes the support vehicles using the shoulder and create a design

using Roads as your design type. Compare the results; the governing design will be the one that produced the thickest section.

B14-1.6.18 **Creating a Rigid Pavement Design: LE Criteria**
On the Add Design window, if you choose to design a rigid pavement using the LED criteria, a series of windows displays requiring the input described in paragraphs (a) through (h) below. The first window is for the rigid layer, followed by windows for the drainage layer (if drainage is required, there will be a window for the separation layer) and subgrade. In PCASE, the primary difference between an empirical design and a layered elastic design is the use of modulus values for layer strengths instead of K values. After you enter the required information on each window, click the **Ok** button to continue. After you enter all the layer information, click **Save and Calculate** to calculate your design. To model your design to account for changing conditions refer to paragraph B14-1.6.14, Creating Season Sets for Layered Elastic Designs. The Damage curves are available for layered elastic designs (only) by clicking the **Damage** button located at the bottom the design window. For rigid pavements the damage to the surface is displayed. The damage per season can also be displayed by selecting the Seasons radio button. The curves are not cumulative; the damage (per vehicle or season) is determined by the area under the curve. Click on the **Print** button to open up the curve in Excel for printing or saving. The **Exit** button will close the curves and return you to the main design window. The Damage to Surface indicates the cumulative damage factor. The goal is to have 100% of the life (CDF=1.0) used up for the surface plus or minus 1%.

(a) **Flexural Strength**
Enter the flexural strength of the new concrete to be placed.

(b) **Percent Steel**
If the new concrete will be reinforced, enter the percent steel to be used for reinforcement.

(c) **% Joint Load Transfer**
For airfields, the design charts were developed based on a 25 percent load transfer value, which is the default in PCASE. A load transfer value of 25 percent is typically used as a reasonable approximation of the load transfer measured over time on the types of joints approved for use on airfields. You can override the default by entering a new value.

(d) **Modulus**
The modulus of elasticity of concrete default is 27,579 MPa (4,000,000 psi).

(e) **Poisson's Ratio**
Defaults are 0.15 for the concrete layer, 0.25 for the base, 0.35 for the drainage layer, and 0.40 for the subgrade. 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, use of the defaults is recommended.

(f) Slip
Slip refers to the interface value between layers. Slip should be set at 0 for complete adhesion between the layers or 1,000 for almost frictionless slip between the layers. Values between 0 and 1,000 may be inputted to simulate varying degrees of friction. Almost frictionless slip is usually assumed at the bottom of a PCC layer and full adhesion is generally assumed for most other pavement materials. In PCASE, Slip defaults to 1,000 for PCC and 0 for all other material types.

(g) **Drainage**
In PCASE, the drainage layer is optional. If a drainage layer is not required, click the **Not Required** button and the software will bypass the drainage requirements. If the drainage layer is required, click the **Required** button and a second window displays with a **Compute Drainage Layer** button (refer to paragraph B14-1.6.10, “Using the Drainage Layer Worksheet”), or you can manually input a thickness of the drainage layer. If you have a drainage layer, you will also be prompted for information on a separation layer.

(h) **Thickness**
In rigid design, you will input a thickness only for layers for which you select manual analysis; all other thicknesses will be computed for you.

B14-1.6.19 **Other Window Options**

- **Multiple Design Builder.** Clicking on the **Multiple Design Builder** button allows you to build numerous design scenarios to be calculated simultaneously. On the first screen click on the **Add** button to setup a multiple design; enter a 3 character ID for your design set; click **OK**; and click **OK** on the Multiple Design Setup screen. The next screen will be for selecting a location for calculating depth of frost penetration. Select the state or country (countries are listed after the states) for which the pavement design will be performed. Selecting a state or country from the list box on the left will populate the list box on the right with the weather stations located in your selected state or country. If frost is not a consideration you can click on the **Cancel** button. On the Multiple Design Selection screen choose the Design Type (Airfield or Roads), Pavement Type (Flexible and/or Rigid), Analysis Type (CBR and/or LED), and Traffic Area (airfield only); click **Ok**. On the Traffic Patterns screen select the patterns you would like to use for the different design scenarios by clicking in the box or click **Select all**; click **OK**. On the Multiple Designs screen enter the layer information for each layer you wish to include in your design scenarios; click **Create designs**. The results will be displayed in the Design Module under the name you assigned to your Multiple Design Set.

- **Materials Cost.** Clicking on the **Materials Cost** button allows you to input unit costs for materials used in your designs. The unit costs will be used to calculate unit price for each design which is displayed in View Alternatives (See Description below).

- **Viewing Alternatives:** Clicking the **View Alternatives** button in the lower left corner of the PCASE Design Module window displays a graphical representation of all the designs you have created. By scrolling vertically or horizontally, you can compare the designs drawn to scale. Moving the mouse across the various layers will display the layer thickness and material type at the bottom of the form.

- **Layer Details:** Clicking the **Layer Details** button in the lower left corner of the main window displays the Layers window, which contains additional layer information not displayed on the main window. The information in this window is dependent upon the design model and surface type.

- **Sensitivity Curves:** Sensitivity curves are available for flexible and rigid empirical designs by clicking the **Sensitivity** button in the lower left corner of the main window. By choosing from the list of variables on the Sensitivity window (flexural strength, modulus, base thickness, K, and percent steel for PCC pavements; base and subgrade CBR for flexible pavements), you can view the effect the variables have on the pavement thickness. As the mouse moves along the curve, the relationship between the variable and the pavement thickness is displayed at the bottom of the curve.

- **English / Metric:** This button gives the user the option of displaying the design in either SI (metric) or U.S. Customary (English) units.

- **Reports:** To view reports, click the **Reports** button in the lower right corner of the PCASE Design Module window (next to the **Save and Calculate** button). The data from the design and traffic modules are provided in a Microsoft® Office Excel® spreadsheet. Templates have been created for you providing a summary of the data that went into your design and the results. More information is available by clicking the Data and Drainage tabs at the bottom of the spreadsheet. **NOTE:** If you want to save the spreadsheet, it is strongly recommended that you select **Save As** from the **File** menu and rename the file; otherwise, you will overwrite the template.

B14-2 EXAMPLE PROBLEMS USING PCASE

B14-2.1 Example 1: Flexible CBR Design (Including Compaction Requirements) using an Army Standard Pattern

Design an Army Class III airfield apron (Type B traffic area). The subgrade is poorly graded sand with a design CBR of 9; the in-place density of the subgrade is 90 percent to a depth of 3 m (10 ft); and the CBR of the base is 80. The analysis in this example assumes that the subgrade does not require special treatment because drainage and frost penetration are not problems.

B14-2.1.1 **Part 1: Traffic Module Steps**
In PCASE, open the Traffic Module window by clicking the **Traffic** button and then follow these steps:

(1) Click **Choose Standard Pattern**.

(2) Select Army Class III Airfield by clicking the check box or by double-clicking the label.

(3) Click **Ok**.

(3) On the Traffic Module window, click **Apply**.

(4) Select **Exit** to exit the traffic module or leave the module open.

B14-2.1.2 **Part 2: Design Module Steps**
In PCASE, open the PCASE Design Module window by clicking the **Design** button and then follow these steps:

(1) Click **Add** under the design grid (middle of the window).

(2) On the Add Design window, enter a design name of your choice in the Design Name text box.

(3) From the Design Type list box, select Airfield.

(4) From the Pavement Type list box, select Flexible.

(5) Select CBR.

(6) From the Traffic Area list box, select Area B.

(7) From the Traffic Patterns drop-down list, select Army Class III Airfield Copy-1.

(8) Click **Ok**. A series of windows prompts you for the following information:

(a) Asphalt Layer. On the Asphalt window, leave Analysis as Compute; click **Ok**.

(b) Base Layer. On the Base window, leave Analysis as Compute; select 80 CBR from the pull down menu; click **Ok**.

(c) Drainage. On the Drainage window, click **Not Required**.

(d) Natural Subgrade. On the Natural Subgrade window, enter 9 in the CBR text box and then click **Ok.**

(e) Click **Save and Calculate.** Your design is complete, and the layer structure should give the results in Table B14-1.

Table B14-1. Example 1 Results

Layer	Thickness, mm (in)	CBR
Asphalt	76 (3)	--
Base	192 (7.56) round to 203 (8)	80
Natural Subgrade	--	9

B14-2.1.3 **Part: 3 Compaction Steps**

To determine the compaction requirements, follow these steps:

(1) Click the **Compaction** button on the lower right side of the PCASE Design Module window. The Compaction Requirements window will populate the compaction requirements from Tables 6-3 through 6-7 of this manual.

(2) Select the appropriate table at the top of the window by highlighting the airfield and subgrade type. For this example, use the table for an Army pavement with cohesionless soils.

(3) Once you select the appropriate table, read the compaction requirements values for B traffic areas on Class III airfields. The values, in mm (inches) from the top of the surface, are: 660 mm (26 in) for 85 percent compaction; 508 mm (20 in) for 90 percent compaction; 381 mm (15 in) for 95 percent compaction; and 229 mm (9 in) for 100 percent compaction. The total pavement thickness is 279 mm (11 in); therefore, the base will need to be compacted to 100 percent (also as required in Paragraph 8-9); and from the top of the subgrade, 102 mm (4 in) (381 mm minus 279 mm [15 in minus 11 in]) will need to be compacted to 95 percent. However, according to Paragraphs 6-5.2.1 and 6-5.2.2 the top 152 mm (6 in) of subgrade will be compacted to 95 percent of maximum density from ASTM D1557. The natural in-place density is at 90 percent, so the 90 percent and 85 percent compaction requirements are not of concern.

(4) Click the **Ok** button. Table B14-2 provides the final design section:

Table B14-2. Example 1 Final Design Section

Layer	Thickness, mm (in)	CBR	Compaction, %
Asphalt	76 (3)	--	--
Base	203 (8)	80	100
Top of Subgrade			

Compacted Subgrade	152 (6)	--	95
Natural Subgrade	--	9	90

B14-2.2 **Example 2: Flexible CBR Design (Including Compaction Requirements) using an Navy Standard Pattern**

Design a Navy primary pavement (Type B traffic area) to support Navy Design Traffic Group III. The design CBR of the lean clay subgrade is 13, and the natural in-place density of the clay is 87 percent extending to 3.05 m (10 ft). The CBR of the base and subbase are 100 and 30, respectively. The analysis in this example assumes that the subgrade does not require special treatment because drainage and frost penetration are not problems.

B14-2.2.1 **Part 1: Traffic Module Steps**

In PCASE, open the Traffic Module window by clicking the **Traffic** button and then follow these steps:

(1) Click **Choose Standard Pattern**.

(2) On the Standard Patterns window, select Navy Design Traffic Group III by clicking the check box or by double-clicking the label; click **Ok**.

(3) On the Traffic Module window, click **Apply**.

B14-2.2.2 **Part 2: Design Section Steps**

In PCASE, open the PCASE Design Module window by clicking the **Design** button and then follow these steps:

(1) Click **Add** under the design grid (middle of the window).

(2) On the Add Design window, enter a design name of your choice in the Design Name text box.

(3) From the Design Type list box, select Airfield.

(4) From the Pavement Type list box, select Flexible.

(5) Select CBR.

(6) From the Traffic Area list box, select Area B.

(7) From the Traffic Patterns drop-down list, select Navy Design Traffic Group III Copy-1.

(7) Click **Ok**. A series of windows prompts you for this information:

(a) Asphalt Layer. On the Asphalt window, leave Analysis as Compute; click **Ok**.

(b) Base Layer. On the Base window, leave Analysis as Compute; leave CBR at 100; click **Ok**.

(c) Drainage. On the Drainage window, click **Not Required**.

(d) Natural Subgrade. On the Natural Subgrade window, enter 13 in the CBR text box; click **Ok**.

(8) Now you need to add a subbase layer. Under the layer grid, click the **Add** button.

(9) On the Add Layer window, select Subbase from the Layer Type list box; click **Ok**.

(10) On the Subbase window, leave Analysis as Compute; enter a CBR of 30 in the CBR text box; click **Ok**.

(11) On the PCASE Design Module window, click **Save and Calculate**. Your design is complete, and the layer structure should give these results:

- Asphalt: 102 mm (4 in)
- Base: 173 mm (6.81 in), 100 CBR
- Subbase: 154 mm (6.05 in), 30 CBR
- Natural Subgrade: 13 CBR

In reality, it would be difficult to construct a 173-mm (6.81-in) base thickness. To make your section more constructible, round up the base thickness and calculate a new subbase thickness. To do this, click the cell for the base layer under the Analyze column and change Compute to Manual; then in the thickness column, enter 178 mm (7 in); then click **Save and Calculate**. The results for your new section should match those in Table B14-3.

Table B14-3. Example 2 Results

Layer	Thickness, mm (in)	CBR
Asphalt	102 (4)	--
Base	178 (7)	100
Subbase	144 (5.67) round to 152 (6)	30
Natural Subgrade	--	13

B14-2.2.3 **Part 3: Compaction Steps**

To determine the compaction requirements, follow these steps:

(1) Click the **Compaction** button on the lower right side of the PCASE Design Module window. This will populate the compaction requirements from Tables 6-3 through 6-7 of this manual.

(2) Select the appropriate table at the top of the window by highlighting the airfield type. For this example, use the table for Navy and Marine Corp pavement.

(3) Once you select the appropriate table, and using the compaction requirements for a cohesive soil (clay), read across for a Single Wheel pavement under the Primary traffic area column for each of the compaction requirements. *The Single Wheel is chosen because the controlling vehicle is the F-15E which is shown by clicking the … button in the Traffic column. According to Figure 4-2a the Navy lists the F-15 as a Single Wheel. Primary traffic is the same as Air Force and Army traffic areas A and B.* The values, in inches from the top of the surface, are: 990 mm (39 in) for 85 percent compaction, 787 mm (31 in) for 90 percent compaction, 584 mm (23 in) for 95 percent compaction, and 381 mm (15 in) for 100 percent compaction. The total pavement thickness is 432 mm (17 in); therefore, the base and subbase will need to be compacted to 100 percent (also as required in Paragraphs 8-9 (base course) and 7-3 (subbase course); and from the top of the subgrade, 152 mm (6 in) (584 mm minus 432 mm [23 in minus 17 in]) will need to be compacted to 95 percent; 203 mm (8 in) (787 mm minus 584 mm [31 in minus 23 in]) compacted to 90 percent; and 406 mm (8 in) (990 mm minus 787 mm [39 in minus 31 in]) compacted to 85 percent. However, the natural in-place density is at 87 percent, so the 85 percent compaction requirement is not of concern. Table B14-4 provides the final design section.

Table B14-4. Example 2 Final Design Section

Layer	Thickness, mm (in)	CBR	Compaction, %
Asphalt	102 (4)	--	--
Base	178 (7)	100	100
Subbase	152 (6)	30	100
Top of Subgrade			
Compacted Subgrade	152 (6)	--	95
Compacted Subgrade	203 (8)	--	90
Natural Subgrade	--	13	87

B14-2.3 **Example 3: Flexible CBR Design with Stabilized Base**
Assume that you want to replace the base in Example 2's design with cement-stabilized material.

B14-2.3.1 To keep your original design, follow these steps:

(1) Click **Copy** under the design grid.

(2) In the Copy Design window, enter a name for your new design; click **Ok**.

B14-2.3.2 To replace the base with stabilized material:

(1) Click **Add** under the layer grid.

(2) On the Add Layer window, select Stabilized Base for the Layer Type.

(3) In the Material Type list box, select PCC Stab -GW, GP, SW, SP.

(4) Click **Ok**.

(5) On the Stabilized Base window leave the defaults and click **Ok**.

(6) Click **Save and Calculate**. Your new section should give these results:

Table B14-5. Example 3 Results

Layer	Thickness, mm (in)	CBR
Asphalt	102 (4)	--
Stabilized Base (PCC-stabilized GW, GP, SW, SP)	152 (6) *	100
Subbase (Unbound Aggregate)	149 (5.87) round up to 152 (6)	100
Natural Subgrade	--	13

* Paragraph 9-2.2 also requires stabilized layers to be a minimum of 152mm (6 in).

B14-2.4 Example 4: Flexible CBR Design using Mixed Traffic with Subsurface Drainage

Design a flexible primary taxiway (traffic area A) in Kuwait for the mix of aircraft traffic in Table B14-6. The taxiway is 38-m-wide (125-ft-wide) (including shoulders) with the crown in the center and has a transverse slope of 1.75 percent and a longitudinal slope of 1 percent. The analysis in this example assumes that the subgrade does not require special treatment but that drainage is a problem and is to be considered for this design. The drainage layer is a rapid drainage material with a permeability of 305 m/day (1000 ft/day) and is made up of uniform-graded medium sands (0.25 effective porosity). The design CBRs of the base and subgrade are 80 and 10, respectively.

Table B14-6. Example 4 Traffic Characteristics

Aircraft	Gross Weight, kg (lb)	Aircraft Passes
B-52H	181,437 (400,000)	300
C-130H	70,307 (155,000)	1,000

Aircraft	Gross Weight, kg (lb)	Aircraft Passes
F-15E	34,019 (75,000)	200,000
OV-1	6,804 (15,000)	1,000,000
P-3C	58,967 (130,000)	5,000

B14-2.4.1 **Part 1: Traffic Module Steps**
In PCASE, open the Traffic Module window by clicking the **Traffic** button and then follow these steps:

(1) Click the **Create Pattern** button.

(2) On the Traffic Pattern window, enter a pattern name and click **Ok**.

(3) On the Traffic Module window, click **Add Vehicle**.

(4) On the Choose Vehicles window, select the boxes for the aircraft in Table B14-6 by clicking the check box or by double-clicking the label; then click the **Add** button.

(5) Once you are returned to the Traffic Module window, change the weight and passes for each of the aircraft according to Table B14-6. To change the weight, click the cell under the Traffic Area Weight column for Areas A and B and enter the new weight for each aircraft. To change the pass level, click the cell under the Traffic Area Pass Levels column for Areas A, B, and C, and enter the new number of passes for each aircraft.

(6) Click **Apply** to send your traffic pattern to the design module.

B14-2.4.2 **Part 2: Design Section Steps**
In PCASE, open the PCASE Design Module window by clicking the **Design** button; then follow these steps:

(1) Click **Add** under the design grid (middle of the window).

(2) On the Add Design window, enter a design name of your choice in the Design Name text box.

(3) From the Design Type list box, select Airfield.

(4) From the Pavement Type list box, select Flexible.

(5) Select CBR.

(6) From the Traffic Area list box, select Area A.

(7) Choose the pattern you created in Step 1 and click **Ok**. A series of windows prompts you for this information:

(a) Asphalt Layer. On the Asphalt window, leave Analysis as Compute; click **Ok**.

(b) Base Layer. On the Base window, leave Analysis as Compute; select 80 from the CBR drop-down list; click **Ok**.

(c) Drainage. On the Drainage window, click **Required**; then click **Compute Drainage Layer**. On the Drainage Worksheet, complete these actions:

(i) Double-click the Storm Drain Index box.

(ii) On the Precipitation window, scroll down to Kuwait (countries are listed after the United States) and click it. Only one station displays in the Stations list box for Kuwait, so a selection in this box is not necessary. Click **Ok**.

(iii) On the Drainage Worksheet, check the box for **Enable Drainage Path Calculations** and enter the transverse length (half of the width of the taxiway since the section is crowned), transverse slope, and longitudinal slope.

(iv) Click the **Calculate** button.

(v) Enter the permeability and effective porosity and click **Calculate Drainage** to complete the calculations.

(vi) Check that the Time for 85% drainage is equal to or less than 1 (one) day. If not, vary the length and slope of the drainage path or permeability of the material. If T85 is equal to or less than 1 (one) day, click **Ok**. The Drainage Worksheet closes and the appropriate value is imported into the Required Thickness text box on the Drainage window.

(vii) On the Drainage window, leave the defaults for selecting the separation layer and CBR of 50, and click **Ok**.

(d) Separation Layer. On the Separation window, leave at the defaults of 102 mm (4 in) and CBR of 50; click **Ok**.

(e) Natural Subgrade. On the Natural Subgrade window, enter 10 in the CBR text box and then click **Ok**.

(f) On the PCASE Design Module window, click **Save and Calculate**. Your design is complete, and the layer structure should give these results:

- Asphalt: 139-mm (5.47-in)
- Base: 152-mm (6-in), 80 CBR

- Drainage layer: 138-mm (5.44-in), 50 CBR
- Separation layer: 102-mm (4-in), 50 CBR
- Natural Subgrade: 10 CBR

In reality, it would be difficult to construct a 139-mm (5.47-in) asphalt layer and 138-mm (5.44-in) drainage layer. To make your section more constructible, round up the layer thicknesses and recalculate. To do this, on the PCASE Design Module window, for the asphalt layer change Analyze to Manual and enter 140-mm (5.5-in) and for the drainage layer enter 152 mm (6 in) in the cell for the drainage layer thickness and then click **Save and Calculate**. Your new section should give the results in Table B14-7.

Table B14-7. Example 4 Results

Layer	Thickness, mm (in)	CBR
Asphalt	140 (5.5)	--
Base	152 (6)	80
Drainage layer	152 (6)	50
Separation layer	102 (4)	50
Natural Subgrade	--	10

Instead of using a separation layer, you could use a geotextile. To do this, follow these steps:

(1) Under the layer grid, click the **Add** button.

(2) On the Add Layer window, select Geotextile as the Layer Type; then click **Ok**.

(3) On the PCASE Design Module window, click **Save and Calculate**. Your design is complete, and the layer structure should give the results in Table B14-8:

Table B14-8. Example 4 Results with Geotextile instead of Separation Layer

Layer	Thickness - mm (in)	CBR
Asphalt	140 (5.5)	--
Base	212 (8.35) (round up to 229 [9])	80
Drainage layer	152 (6)	50
Geotextile	--	--
Natural Subgrade	--	10

B14-2.5 **Example 5: Flexible CBR Design for Mixed Traffic in Frost Conditions**
Design a new heavy-load runway interior pavement (Type C traffic area) at McGrath, Alaska, for the material conditions given in Table B14-9. Design for frost conditions; therefore, a drainage layer is recommended. Past experience indicates that a 102-mm (4-in) drainage layer and 102-mm (4-in) separation layer will perform satisfactorily for meeting drainage requirements.

Table B14-9. Example 5 Physical Properties

Layer	Soil Classification	Frost Code	Design CBR	Dry Unit Weight kg/m^3 (pcf)	Percent Moisture
Wearing Surface	Asphalt Cement (AC)	F0	--	2,323 (145)	0
Base Course	Gravel-Sand (GW)	F0	80	2,162 (135)	5
Drainage Layer	Bank Run Sands	F0	50	2,082 (130)	10
Separation Layer	Gravel Sand (GP)	F0	50	2,082 (130)	10
Natural Subgrade	Clay (CL), PI>12	F3	10	1,762 (110)	20

B14-2.5.1 **Part 1: Traffic Module Steps**
In PCASE, open the Traffic Module window by clicking the **Traffic** button and then follow these steps:

(1) Click **Choose Standard Pattern**.

(2) On the Standard Patterns window, select Air Force Heavy by clicking the check box or by double-clicking the label; click **Ok**.

(3) On the Traffic Module window, click **Apply**.

B14-2.5.2 **Part 2: Design Section Steps**
In PCASE, open the PCASE Design Module window by clicking the **Design** button and then follow these steps:

(1) Click **Add** under the design grid (middle of the window).

(2) On the Add Design window, enter a design name of your choice.

(3) From the Design Type list box, select Airfield.

(4) From the Pavement Type list box, select Flexible.

(5) Select CBR.

(6) From the Traffic Area list box, select Area C.

(7) From the Traffic Patterns drop-down list, select Air Force Heavy Copy-1.

(8) Click **Ok**. A series of windows prompts you for information on these characteristics:

(a) Asphalt Layer. On the Asphalt window, leave Analysis as Compute; click **Ok**.

(b) Base Layer. On the Base window, leave Analysis as Compute; select 80 from the CBR pull down menu; click **Ok**.

(c) Drainage. On the Drainage window, click **Required**. Since the thickness of the drainage layer is known from previous experience, you do not need to compute the drainage layer. Leave the defaults for selecting the separation layer, thickness of 102 mm (4 in) and CBR of 50, and then click **Ok**.

(d) Separation. On the Separation window, leave the default of 102 mm (4 in) and the CBR of 50; then click **Ok**.

(e) Natural Subgrade. On the Natural Subgrade window, enter 10 in the CBR text box; click **Ok**.

(f) Click **Save and Calculate**. Your non-frost design is complete and the layer structure should give these results:

- Asphalt: 132 mm (5.19 in)
- Base: 325 mm (12.81 in), 80 CBR
- Drainage layer: 102 mm (4 in), 50 CBR
- Separation layer: 102 mm (4 in), 50 CBR
- Natural Subgrade: 10 CBR

In reality, it would be difficult to construct a 132-mm (5.19-in) asphalt layer. To make your section more constructible, round up the layer thickness and recalculate. To do this, on the PCASE Design Module window, for the asphalt layer change Analyze to Manual and enter 140-mm (5.5-in) and then click **Save and Calculate**. Your new section should give the following results:

- Asphalt: 140 mm (5.5 in)
- Base: 307 mm (12.09 in), 80 CBR
- Drainage layer: 102 mm (4 in), 50 CBR
- Separation layer: 102 mm (4 in), 50 CBR
- Natural Subgrade: 10 CBR

B14-2.5.2.1 **Calculating for Frost Conditions**
Now you need to calculate for frost conditions. To do this, locate the button for opening the depth of frost calculator, which is in the top grid of the PCASE Design Module window in the Depth of Frost column, and then follow these steps:

(1) To open the calculator, click the **...** (dot-dot-dot) button in the Depth of Frost column on the row of your design.

(2) On the Depth of Frost Penetration Calculator window, select Alaska for the state and McGrath WSO Airport for the station. The layer structure you built in the design is imported into the form, and the layers in the form correspond to the layers in your design. For this example, AC refers to the asphalt layer, Course Grained refers to the base, drainage and separation layers, and Fine Grained refers to the natural subgrade.

(3) Change the default dry unit weight and moisture content of each material in the structure as needed to match the conditions given in Table B14-9.

(4) Click the **Calculate** button. The depth of frost penetration for each layer is displayed in the far right column.

(5) Click **Apply and Close**, which will take you back to the PCASE Design Module window and import the frost penetration depth.

(6) Once you have calculated the depth of frost, a new column, Frost Code, appears in the layer grid for the current design. You can now assign a frost code to your subgrade by clicking the cell for a drop-down list of available frost codes. Select F3/F4.

(7) After entering the frost code, click **Save and Calculate**. Your results should be the same as those in Table B14-10 for Non-Frost Design, Reduced Subgrade Strength (RSS), and Limited Subgrade Frost Penetration (LSFP).

Table B14-10. Example 5 Frost Condition Results

Layer	Thickness, mm (in)		
	Non-Frost Design	Reduced Subgrade Strength (RSS)	Limited Subgrade Frost Penetration (LSFP)
Asphalt	140 (5.5)	140 (5.5)	140 (5.5)
Base course	305 (12)	940 (37)	1,194 (47)
Drainage layer	102 (4)	102 (4)	102 (4)
Separation layer	102 (4)	102 (4)	102 (4)

Layer	Thickness, mm (in)		
	Non-Frost Design	Reduced Subgrade Strength (RSS)	Limited Subgrade Frost Penetration (LSFP)
Natural Subgrade (F3)	--	--	--
NOTE: Final thicknesses are rounded values.			

For this example, RSS is the controlling design. (The controlling design on the PCASE window is in the white column, not the dimmed column).

B14-2.6 **Example 6: Flexible Airfield Paved Shoulders**
Design flexible airfield shoulders for a runway using an 80 CBR base course and a 10 CBR subgrade. Assume that frost and drainage are not issues.

Note: The traffic criteria are built into the design module for airfield paved shoulders; therefore, there is no need to create a pattern or choose a standard pattern; *however, if you just opened a new file and have not applied a traffic pattern, you will need to do so. Any traffic pattern (standard or created) will work. The software is set up so that a traffic pattern is necessary for the design module to open.*

On the PCASE Design Module window, click **Add** under the design grid and then follow these steps:

(1) On the Add Design window, enter a design name of your choice in the Design Name text box.

(2) In the Design Type list box, select Airfield.

(3) In the Pavement Type list box, select Flex Shoulder.

(4) CBR is the only Analysis Type for flexible should design; click **Ok**. A series of windows prompts you for the following information:

(a) Asphalt Layer. On the Asphalt window, leave Analysis as Compute; click **Ok**.

(b) Base Layer. On the Base window, leave Analysis as Compute; select 80 from the CBR pull down menu; click **Ok**.

(c) Drainage. On the Drainage window, click **Not Required**.

(d) Natural Subgrade. On the Natural Subgrade window, enter a CBR of 10; click **Ok**.

(e) Click **Save and Calculate**. Your design is complete, and the layer structure should give the results in Table B14-11:

Table B14-11. Example 6 Results

Layer	Thickness, mm (in)*	CBR
Asphalt	51 (2)	--
Base	183 (7.20) (round up to 203 [8])	80
Natural Subgrade	--	10
*If pavements are to be used by support vehicles (e.g., snow removal equipment, fire trucks, fuel trucks), the shoulder should be checked by creating a road design. Create a traffic pattern inputting the support vehicles using the shoulder and create a design using Roads as your design type. Compare the results of the Shoulder and Roads designs; the governing design will be the one that produces the thickest section.		

B14-2.7 **Example 7: Flexible Layered Elastic Design using an Army Standard Pattern – One Season**

Design a runway interior (Traffic Area C) for an Army Class VI (C-130) Paved Landing Zone. The subgrade is lean clay classified as CL, and laboratory results determined a design modulus of 62.1 MPa (9,000 psi). For this design, the recommended modulus of elasticity of the unbound granular base and subbase course material are 421 MPa (61,000 psi) and 165 MPa (24,000 psi), respectively. The base course will meet specifications to be used as a drainage layer. Based on temperature and moisture in the area, the modulus of elasticity of the asphalt layer was determined to be 6,895 MPa (1,000,000 psi). The Poisson's ratio has little effect on the pavement thickness; therefore, use the defaults built into the software.

B14-2.7.1 **Part 1: Traffic Module Steps**

Open the Traffic Module window by clicking the **Traffic** button and then follow these steps:

(1) Click **Choose Standard Pattern**.

(2) Select Army Class VI (C-130) Paved Landing Zone by clicking the check box or by double-clicking the label.

(3) Click **Apply**.

B14-2.7.2 **Part 2: Design Section Steps**

Open the Design Module window by clicking the **Design** button and then follow these steps:

(1) Click **Add** under the design grid (middle of the window).

(2) On the Add Design window, enter a design name of your choice in the Design Name text box.

(3) In the Design Type list box, select Airfield.

(4) In the Pavement Type list box, select Flexible.

(5) Select LED.

(6) In the Traffic Area list box, select Area C.

(7) From the Traffic Patterns drop-down list, select Army Class VI (C130) Paved Landing Zone Copy-1.

(8) Click **Ok**. A series of windows prompts you for this information:

(a) Asphalt Layer. On the Asphalt window, leave Analysis as Compute.

(b) In the Modulus text box, change the value to 1,000,000.

(c) Use the defaults in the Poisson's Ratio (0.35) and Slip (0) text boxes.

(d) Click **Ok**. **Note:** When there is more than one season, you can use the Seasonal Values scrollbar to scroll through each season and input the modulus, Poisson's ratio, and slip.

(e) Base Layer. On the Base window, use the defaults for Modulus (421 MPa [61,000 psi]), Poisson's Ratio (0.35) and Slip (0); and then click **Ok**.

(f) Drainage. On the Drainage window, click **Not Required**.

(g) Natural Subgrade. On the Natural Subgrade window, change the value in the Modulus box to 62.1 MPa (9,000 psi); use the defaults in the Poisson's Ratio (0.40) and Slip (0) text boxes; and then click **Ok**.

(h) To add a subbase layer, click the **Add** button under the layer grid and follow these steps:

(i) On the Add Layer window, select Subbase from the Layer Type list box; click **Ok**.

(ii) On the Subbase window, leave Analysis as Compute.

(iii) Use the default for Modulus (165 MPa [24,000 psi]), Poisson's Ratio (0.35) and Slip (0); and then click **Ok**.

(v) Click **Save and Calculate**. Your design is complete, and the layer structure should give the results in Table B14-12:

Table B14-12. Example 7 Results

Layer	Thickness, mm (in)*	Modulus of Elasticity, MPa (psi)
Asphalt	76 (3)	6,894 (1,000,000)
Base	152 (6)	421 (61,000)
Subbase	250 (9.85) (round up to 254 [10])	165 (24,000)
Natural Subgrade	--	62 (9,000)

B14-2.8 **Example 8: Flexible Layered Elastic Design using an Army Standard Pattern – Multiple Seasons**

Design the pavement in Example 7 to account for changing conditions by assigning different modulus values to different seasons of the year. Use Table B14-13 to establish the seasons and corresponding modulus values.

Table B14-13. Example 8 Season and Modulus Value Information

Season Information			Modulus Values, MPa (psi)			
Season	**Start Date**	**End Date**	**Asphalt**	**Base**	**Subbase**	**Subgrade**
1	Nov 1	Feb 28	7,583 (1,100,000)	421 (61,000)	103 (35,000)	83 (12,000)
2	Mar 1	May 31	5,170 (750,000)	421 (61,000)	124 (18,000)	34 (5,000)
3	Jun 1	Sep 15	1,379 (200,000)	421 (61,000)	165 (24,000)	62 (9,000)
4	Sep 16	Oct 31	5,860 (850,000)	421 (61,000)	145 (21,000)	48 (7,000)

To use the same layer structure as in Example 7, click the **Copy** button under the design grid and give a name to your new design.

To create seasons, follow these steps:

(1) Click the **Edit Seasons** button located on the right side of the PCASE Design Module window under the design grid. A dialog box will display indicating that your values (such as modulus, Poisson's ratio, and slip) will be reset to the defaults.

(2) Click **Ok**. You will be entering new values for each season anyway.

(3) On the Seasons window, click the **Add** button to build a new season set and then enter a name to identify the set (e.g., 3 seasons, 4 seasons, my seasons) on the **Add set of seasons** window. Click **Ok**.

(4) To divide the year into various seasons, click the **end date** of each season (i.e., Feb 28, May 31, Sep 15, and Oct 31) as indicated in Table 14-13.

(5) When you have completed building a season set, click the **Exit** button to return to the PCASE Design Module window.

(6) On the PCASE Design Module window, click the **...** (dot-dot-dot) button in the Seasons column of the design grid to assign the newly created season set to your design.

(7) On the Seasons window, select the appropriate season set and click **Ok**.

(8) You must now enter modulus values for each of the seasons. Enter the modulus values for each season by typing the modulus values in the modulus column of the layer grid. Above the layer grid, on the right side, is a scrollbar indicating the season. To go to the next season, click the **>** (right arrow) button. Continue scrolling to the next season and entering modulus values until you have changed the values for all the seasons. After you have entered modulus values for all the seasons, click **Save and Calculate**. Your design is complete, and the layer structure should give the results in Table B14-14.

Table B14-14. Example 8 Results

Layer	Thickness, mm (in)*	Modulus of Elasticity, MPa (psi)
Asphalt	76 (3)	varies
Base	152 (6)	varies
Subbase	381 (15)	varies
Natural Subgrade	--	varies

B14-2.9 Example 9: Rigid Empirical Design using an Air Force Standard Pattern

Design Air Force medium-load rigid pavements for all traffic areas (A, B, C, and D). The design will include (a) plain slab on grade; (b) reinforced slab on grade using 0.20 percent steel; (c) plain slab on unbound base; and (d) plain slab on stabilized base. On-site and laboratory investigations have yielded the data required for the 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 the subgrade material; and (5) the 90-day concrete flexural strength *R* is 4.82 MPa (700 psi).

B14-2.9.1 **Part 1: Traffic Module Steps**
Open the Traffic Module window by clicking the **Traffic** button and then follow these steps:

(1) Click **Choose Standard Pattern**.

(2) On the Standard Patterns window, select Air Force Medium by clicking the check box or by double-clicking the label.

(3) Click the **Ok** button.

(3) On the Traffic Module window, click **Apply**.

B14-2.9.2 **Part 2: Design Module Steps**

B14-2.9.2.1 **Design a Plain Slab on Grade**
Open the PCASE Design Module window by clicking the **Design** button and then follow these steps:

(1) Click **Add** under the design grid (middle of the window).

(2) On the Add Design window, enter a design name of your choice in the Design Name text box.

(3) In the Design Type list box, select Airfield.

(4) In the Pavement Type list box, select Rigid.

5) Select K.

(5) In the Traffic Area list box, select Area A.

(6) From the Traffic Patterns drop-down list, select Air Force Medium Copy-1.

(7) Click **Ok**. A series of windows prompts you for this information:

(a) PCC layer. On the PCC window, change the flexural strength to 4.82 MPa (700 psi); leave percent steel (0), percent joint load transfer (25), modulus 27,579 MPa (4,000,000 psi), and Poisson's ratio (0.15) set to the defaults; and click **Ok**.

(b) Drainage. On the Drainage window, click **Not Required**.

(c) Natural Subgrade. On the Natural Subgrade window, enter 54 kPa/mm (200 pci) in the **K (pci)** text box; click **Ok**.

(d) Click **Save and Calculate**. Your design is complete, and the layer structure should give these results for Traffic Area A:

- PCC: 396 mm (15.61 in) (round up to 16 in)
- Natural Subgrade: 54 kPa/mm (200 pci)

(8) Instead of creating new designs for Traffic Areas B, C, and D, you can change the traffic area in the design grid and recalculate. **Note:** You may want to copy your design first and use a design name such as “PCC Traffic Area B.” Locate the row for your design and click the cell under the Traffic Area column. Click the drop-down list button, select Traffic Area B, and then click **Save and Calculate**. Your design is complete, and the layer structure should give these results for Traffic Area B:

- PCC: 389 mm (15.32 in) (round off to 15.5 in)
- Natural Subgrade: 54 kPa/mm (200 pci)

(9) For Traffic Area C, locate the row for your design and click the cell under the Traffic Area column. Click the drop-down list button, select Traffic Area C, and then click **Save and Calculate**. Your design is complete, and the layer structure should give these results for Traffic Area C:

- PCC: 319 mm (12.57 in) (round off to 12.5 in)
- Natural Subgrade: 54 kPa/mm (200 pci)

(10) For Traffic Area D, locate the row for your design and click the cell under the Traffic Area column. Click the drop-down list button, select Traffic Area D, and then click **Save and Calculate**. Your design is complete, and the layer structure should give these results for Traffic Area D:

- PCC: 255 mm (10.02 in) (round off to 10 in)
- Natural Subgrade: 54 kPa/mm (200 pci)

B14-2.9.2.2 **Design a Reinforced Slab on Grade using 0.20 Percent Steel**
Open the PCASE Design Module window and then follow these steps:

(1) Click **Add** under the design grid (middle of the window).

(2) On the Add Design window, enter a design name of your choice in the Design Name text box.

(3) In the Design Type list box, select Airfield.

(4) In the Pavement Type list box, select Rigid.

(5) Select K.

(6) In the Traffic Area list box, select Area A.

(7) From the Traffic Patterns drop-down list, select Air Force Medium Copy-1.

(8) Click **Ok**. A series of windows prompts you for this information:

(a) PCC layer. On the PCC window, change flexural strength to 4.82 MPa (700 psi); change the percent steel to 0.20; leave the percent joint load transfer (25), modulus 27,579 MPa (4,000,000 psi), and Poisson's ratio (0.15) set to the defaults; and click **Ok**.

(b) Drainage. On the Drainage window, click **Not Required**.

(c) Natural Subgrade. On the Natural Subgrade window, enter 54 kPa/mm (200 pci) in the **K (pci)** text box; click **Ok**.

(d) Click **Save and Calculate**. Your design is complete, and the layer structure should give these results for Traffic Area A:

- PCC: 318-mm (12.51 in) (round off to 12.5 in)
- Natural Subgrade: 54 kPa/mm (200 pci)

(9) Instead of creating new designs for Traffic Areas B, C, and D, you can change the traffic area in the design grid and recalculate. **Note:** You may want to copy your design first and use a design name such as "PCC Reinforced Area B." Locate the row for your design and click the cell under the Traffic Area column. Click the drop-down list button, select Traffic Area B, and then click **Save and Calculate**. Your design is complete, and the layer structure should give these results for Traffic Area B:

- PCC reinforced with 0.20 steel: 312 mm (12.28 in) (round up to 12.5 in)
- Natural Subgrade: 54 kPa/mm (200 pci)

(10) For Traffic Area C, locate the row for your design and click the cell under the Traffic Area column. Click the drop-down list button, select Traffic Area C, and then click **Save and Calculate**. Your design is complete, and the layer structure should give these results for Traffic Area C:

- PCC reinforced with 0.20 steel: 256 mm (10.07 in) (round off to 10 in)
- Natural Subgrade: 54 kPa/mm (200 pci)

(11) For Traffic Area D, locate the row for your design and click the cell under the Traffic Area column. Click the drop-down list button, select Traffic Area D, and then click **Save and Calculate**. Your design is complete, and the layer structure should give these results for Traffic Area D:

- PCC reinforced with 0.20 steel: 204 mm (8.03 in) (round off to 8 in)
- Natural Subgrade: 54 kPa/mm (200 pci)

Table B14-15. Slab on Grade Final Results

Layer	Thickness, mm (in)			
	Traffic Area A	Traffic Area B	Traffic Area C	Traffic Area D
PCC Thickness (with 0.00 reinforcement steel)	406 (16)	394 (15.5)	318 (12.5)	254 (10)
PCC Thickness (with 0.20 reinforcement steel)	318 (12.5)	318 (12.5)	254 (10)	203 (8)

B14-2.9.2.3 **Design a Plain Slab on Unbound Base using Three Base Course Thicknesses (152-mm [6-in], 305-mm [12-in], and 457-mm [18-in]) for Comparison** Open the PCASE Design Module window and then follow these steps:

(1) Click **Add** under the design grid (middle of the window).

(2) On the Add Design window, enter a design name of your choice in the Design Name text box.

(3) In the Design Type list box, select Airfield.

(4) In the Pavement Type list box, select Rigid.

(5) Select K.

(6) In the Traffic Area list box, select Area A.

(7) From the Traffic Patterns drop-down list, select Air Force Medium Copy-1.

(8) Click **Ok**. A series of windows prompts you for this information:

(a) PCC layer. On the PCC window, change flexural strength to 4.8 MPa (700 psi); leave the percent steel (0), percent joint load transfer (25), modulus (25,759 MPa [4,000,000 psi]), and Poisson's ratio (0.15) set to the defaults; click **Ok**.

(b) Drainage. On the Drainage window, click **Not Required**.

(c) Natural Subgrade. On the Natural Subgrade window, enter 54 kPa/mm (200 pci) in the **K (psi)** text box; click **Ok**.

(9) Now you need to add a base layer. Under the layer grid, click the **Add** button.

(10) On the Add Layer window, choose Base from the Layer Type list box and choose Unbound Aggregate for the material type; click **Ok**.

(11) On the Base window, change the base design thickness to 152 mm (6 in); leave K at 0; then click **Ok**.

(12) On the PCASE Design Module window, click **Save and Calculate**. Your design is complete. Table B14-16 lists thicknesses for all traffic areas and trial thicknesses.

To change the thickness of the base course, click the cell for base course under the Non-Frost Design Thickness column and enter the new thickness; click **Save and Calculate** to recalculate.

Sensitivity curves are another option for calculating slab thicknesses based on varying base course thicknesses. Sensitivity curves are available for flexible and rigid empirical designs by clicking the **Sensitivity** button in the lower left corner of the PCASE Design Module window. By choosing from the list of variables on the Sensitivity window (flexural strength, modulus, K, base thickness, and percent steel for PCC pavements; and base and subgrade CBR for flexible pavements), you can view the effect of the variables on the pavement thickness. As the mouse moves along the curve, the relationship between the variable and the pavement thickness is displayed at the bottom of the curve.

Table B14-16. Air Force Plain Slab on Unbound Base Results

Base Course Thickness, mm (in)	Effective Modulus of Soil Reaction, kPA/mm (pci)	PCC Thickness, mm (in)			
		A	B	C	D
152 (6)	70 (260)	354 (13.93)	348 (13.70)	300 (11.83)	241 (9.48)
305 (12)	85 (313)	336 (13.23)	331 (13.05)	289 (11.38)	229 (9.03)
457 (18)	97 (357)	324 (12.77)	320 (12.60)	262 (11.07)	221 (8.71)
Note: Final thicknesses should be rounded values.					

B14-2.9.2.4 **Design a Plain Slab on Stabilized Base using Three Stabilized Base Course Thicknesses (152-mm [6-in], 305-mm [12-in], and 457-mm [18-in]) for Comparison**

Assume that a well-graded 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 5.17 MPa (750 psi) and a flexural modulus of elasticity of 5,515 MPa (800,000 psi) at an age of 90 days.

Open the PCASE Design Module window and then follow these steps:

(1) Click **Add** under the design grid (middle of the window).

(2) On the Add Design window, enter a design name of your choice in the Design Name text box.

(3) In the Design Type list box, select Airfield.

(4) In the Pavement Type list box, select Rigid.

(5) Select K.

(6) In the Traffic Area list box, select Area A.

(7) From the Traffic Patterns drop-down list, select Air Force Medium Copy-1.

(8) Click **Ok**. A series of windows prompts you for this information:

(a) PCC layer. On the PCC window, change flexural strength to 4.82 MPa (700 psi); leave the percent steel (0), percent joint load transfer (25), modulus (27,579 MPa [4,000,000 psi]), and Poisson’s ratio (0.15) set to the defaults; and click **Ok**.

(b) Drainage. On the Drainage window, click **Not Required**.

(c) Natural Subgrade. On the Natural Subgrade window, enter 54 kPa/mm (200 pci) in the **K (psi)** text box; click **Ok**.

(9) Now you need to add a stabilized base layer. Under the layer grid, click the **Add** button.

(10) On the Add Layer window, select Stabilized Base from the Layer Type list box; click **Ok**.

(11) From the Material Type list box, select PCC Stab - GW, GP, SW, SP; then click **Ok**.

(12) On the Stabilized Base window, change the modulus value to 5,515 MPa (800,000 psi); click **Ok**.

(13) On the PCASE Design Module window, in the Analyze column for the PCC layer, click the drop-down list button and select Compute. The Stabilized Base layer will automatically change to Manual; change the stabilized base thickness to 152 mm (6 in).

(14) Click **Save and Calculate**. Your design is complete. Table B14-17 lists thicknesses for all traffic areas and trial thicknesses.

To change the thickness of the stabilized base course, click the cell for the stabilized base course under the Non-Frost Design Thickness column and enter the new thickness; click **Save and Calculate** to recalculate.

Table B14-17. Plain Slab on Stabilized Base Results

Base Course Thickness, mm (in)	Thickness, mm (in)			
	A	B	C	D
152 (6)	361 (14.20)	353 (13.90)	281 (11.03)	211 (8.31)
305 (12)	299 (11.77)	291 (11.44)	210 (8.28)	152 (6.0)*
457 (18)	215 (8.46)	205 (8.08)	152 (6.0)*	152 (6.0)*

Note: Final thickness should be rounded values.
*Minimum thickness (calculated thickness can be viewed by clicking the **Layer Details** button and scrolling to the right to the Required PCC Thickness column.

B14-2.10 **Example 10: Rigid Empirical Design using the Navy Standard Pattern**
Design a new rigid apron (Traffic Area B) for the Navy at an undisclosed location. The only information available about the site reveals that it has a temperate climate and that the subgrade material falls in the subgrade strength category C range. For subgrade

category C, the strength range is from 27.10 to 54.20 kPa/mm (100 to 200 pci); use 40.65 kPa/mm (150 pci). There is no previous information on 90-day concrete flexural strengths; therefore, use the default of 4.48 MPa (650 psi). Since traffic information is not available, use the Navy Design Traffic Group II standard pattern.

B14-2.10.1 **Part 1: Traffic Module Steps**
In PCASE, open the Traffic Module window by clicking the **Traffic** button and then follow these steps:

(1) Click **Choose Standard Pattern**.

(2) On the Standard Patterns window, select Navy Design Traffic Group II from the list box by clicking the check box or by double-clicking the label; then click **Ok**.

(3) On the Traffic Module window, click **Apply**.

B14-2.10.2 **Part 2: Design Module Steps**

B14-2.10.2.1 **Plain Slab on Grade**
Open the PCASE Design Module window and then follow these steps:

(1) Click **Add** under the design grid (middle of the window).

(2) On the Add Design window, enter a design name of your choice in the Design Name text box.

(3) From the Design Type list box, select Airfield.

(4) From the Pavement Type list box, select Rigid.

(5) Select K.

(6) From the Traffic Area list box, select Area B.

(7) Choose the Traffic Patterns drop-down list; select Navy Design Traffic Group II Copy-1.

(8) Click **Ok**. A series of windows prompts you for this information:

(a) PCC layer. On the PCC window, leave the defaults for flexural strength at 4.48 MPa (650 psi), percent steel (0), percent joint load transfer (25), modulus 27,579 MPa (4,000,000 psi), and Poisson's ratio (0.15); click **Ok**.

(b) Drainage. On the Drainage window, click Not Required.

(c) Natural Subgrade. On the Natural Subgrade window, enter 40.65 kPa/mm (150 pci) in the **K (pci)** text box; click **Ok**.

(d) On the PCASE Design Module window, click **Save and Calculate**. Your design is complete, and the layer structure should give these results:

- PCC: 364 mm (14.33 in) (round up to 14.5 in)
- Natural Subgrade: 40.65 kPa/mm (150 pci)

B14-2.10.2.2 **Plain Slab on Unbound Base using Three Base Course Thicknesses (152-mm [6-in], 305-mm [12-in], and 457-mm [18-in]) for Comparison**
Open the PCASE Design Module window and then follow these steps:

(1) Click **Add** under the design grid (middle of the window).

(2) On the Add Design window, enter a design name of your choice in the Design Name text box.

(3) From the Design Type list box, select Airfield.

(4) From the Pavement Type list box, select Rigid.

(5) Select K.

(6) From the Traffic Area list box, select Area B.

(7) Choose the Traffic Patterns drop-down list; select Navy Design Traffic Group II Copy-1.

(8) Click **Ok**. A series of windows prompts you for this information:

(a) PCC Layer. On the PCC window, leave the defaults for flexural strength at 4.48 MPa (650 psi), percent steel (0), percent joint load transfer (25), modulus 27,579 MPa (4,000,000 psi), and Poisson's ratio (0.15); click **Ok**.

(b) Drainage. On the Drainage window, click **Not Required**.

(c) Natural Subgrade. On the Natural Subgrade window, enter 40.65 kPa/mm (150 pci) in the **K (pci)** text box.

(d) Click **Ok**.

(9) Now you need to add a base layer. Under the layer grid, click **Add**.

(10) On the Add Layer window, select Base from the Layer Type list box and Unbound Aggregate for the material type; then click **Ok**.

(11) On the Base window, change the base design thickness to 152 mm (6 in); leave K at 0; then click **Ok**.

(12) On the PCASE Design Module window, click **Save and Calculate**. Your design is complete. Thicknesses for all trial thicknesses are listed in Table B14-18.

To change the thickness of base course, click the cell for the base course under the Non-Frost Design Thickness column and enter the new thickness; click **Save and Calculate** to recalculate.

Sensitivity curves are another option for calculating slab thicknesses based on varying base course thicknesses. Sensitivity curves are available for flexible and rigid empirical designs by clicking the **Sensitivity** button in the lower left corner of the PCASE Design Module window. By choosing from the list of variables on the Sensitivity window (flexural strength, modulus, K, base thickness, and percent steel for PCC pavements; and base and subgrade CBR for flexible pavements), you can view the effect of the variables on the pavement thickness. As the mouse moves along the curve, the relationship between the variable and the pavement thickness is displayed at the bottom of the curve.

Table B14-18. Navy Plain Slab on Unbound Base Results

Base Course Thickness, mm (in)	Effective Modulus of Soil Reaction, kPA/mm (pci)	PCC Thickness, mm (in)
152 (6)	56 (208)	352 (13.84)
305 (12)	71 (262)	336 (13.22)
457 (18)	85 (314)	321 (12.62)

B14-2.11 **Example 11: Rigid Empirical Design for Mixed Traffic in Frost Conditions**

Design new rigid pavement runway ends (Traffic Area A) for Grand Forks, North Dakota, using the traffic mix and material conditions in Tables B14-19a and B14-19b, respectively. Design for frost conditions; therefore, a drainage layer is recommended. The runway is 91-m (300-ft) wide (including shoulders) with the crown in the center and has a transverse slope of 1.5 percent and a longitudinal slope of 1 percent.

Table B14-19a. Example 11 Traffic Mix

Aircraft	Gross Weight, kg (lb)	Aircraft Passes
B-52	181,440 (400,000)	300
C-17	265,352 (585,000)	10,000
C-130H	70,300 (155,000)	5,000
F-15C	30,840 (68,000)	100,000
OV-1	8,170 (18,000)	1,000,000

Table B14-19b. Example 11 Material Conditions

Layer	Soil Classification	Frost Code	Flexural Strength MPa (psi)	Design K kPa/mm (pci)	Dry Unit Weight kg/m^3 (pcf)	% Moisture
Wearing Surface	Portland Cement Concrete (PCC)	F0	4.48 (650)	--	2,323 (145)	0
Drainage Layer	Permeability 2000 ft/day Eff. Porosity 0.20	F0	-	-	2,002 (125)	10
Separation Layer	Gravel sand (GP)	F0	-	-	1,922 (120)	10
Natural Subgrade	Clay (CL)	F3	-	60 (223)*	1,842 (115)	15

*Field information provided in Airfield Pavement Evaluation, Grand Forks, North Dakota, March 1994, by Air Force Civil Engineering Services Agency

B14-2.11.1 **Part 1: Traffic Module Steps**

Open the Traffic Module window by clicking the **Traffic** button and then follow these steps:

(1) Click **Create Pattern**.

(2) In the Traffic Pattern window, enter a name in the Pattern Name text box.

(3) Click **Ok**.

(4) On the Traffic Module window, click **Add Vehicle**.

(5) On the Choose Vehicles window, select the aircraft listed in Table B14-19a by clicking the check boxes or by double-clicking the labels for those aircraft.

(6) Click **Add**.

(7) On the Traffic Module window, change the weight and passes for each of the aircraft according to Table B14-19a. To change the weight, click the cell under the Traffic Area Weight column for Areas A and B, and enter the new weight for each aircraft. To change the pass level, click the cell under the Traffic Area Pass Levels column for Areas A, B, and C, and enter the new number of passes for each aircraft.

(8) Click **Apply** to send your traffic pattern to the design module.

B14-2.11.2 **Part 2: Design Module Steps**

B14-2.11.2.1 Open the PCASE Design Module window by clicking **Design** and then follow these steps:

(1) Click **Add** under the design grid (middle of the window).

(2) On the Add Design window, enter a design name of your choice in the Design Name text box.

(3) From the Design Type list box, select Airfield.

(4) From the Pavement Type list box, select Rigid.

(5) Select K.

(6) From the Traffic Area list box, select Area A.

(7) From the Traffic Patterns drop-down list, select the pattern you created in Part 1 and then click **Ok**. A series of windows prompts you for this information:

(a) PCC Layer. On the PCC window, leave flexural strength (4.48 MPa [650 psi]), percent steel (0), percent joint load transfer (25), modulus (25,759 MPa [4,000,000 psi]), and Poisson's ratio (0.15) set to the defaults; click **Ok**.

(b) Drainage. On the Drainage window, click **Required** and then click **Compute Drainage Layer**.

(c) On the Drainage Worksheet, follow these steps:

(i) Double-click the Storm Drain Index box. The Precipitation window displays.

(ii) On the Precipitation window, scroll down in the Regions list box to North Dakota and select it; select Grand Forks Univ from the Stations list box; and then click **Ok**.

(iii) On the Drainage Worksheet, select the Enable Drainage Path Calculations check box and enter the transverse length (half of the width of the runway since the section is crowned), transverse slope, and longitudinal slope, and click **Calculate**.

(iv) Enter the permeability and effective porosity values from Table B14-19b and click on Calculate Drainage to complete the calculations.

(v) Click **Ok**. The Drainage Worksheet will close and the appropriate value will be imported into the Required Thickness text box on the Drainage window.

(vi) On the Drainage window, leave the defaults for the drainage layer and click **Ok**.

(d) Separation Layer. On the Separation window, leave the defaults and click **Ok**.

(e) Natural Subgrade. On the Natural Subgrade window, enter 60 kPa/mm (223 pci) in the **K (pci)** text box and then click **Ok**.

(f) On the PCASE Design Module window, click **Save and Calculate**. Your design is complete, and the layer structure should give these results:

- PCC: 328 mm (12.90 in)
- Drainage layer: 143 mm (5.62 in)
- Separation layer: 102 mm (4 in)
- Natural Subgrade: 60 kPa/mm (223 pci)

In reality, it would be difficult to construct a 143-mm (5.62-in) drainage layer. To make your section more constructible, round up the drainage layer thickness and recalculate. In the Non-Frost Design Thickness column for the drainage layer, enter 152 mm (6-in) and then click **Save and Calculate**. Your non-frost design is complete, and the layer structure should give these results:

- PCC: 326 mm (12.85 in) (round up to 13 in)
- Drainage layer: 152 mm (6 in)
- Separation layer: 102 mm (4 in)
- Natural Subgrade: 60 kPa/mm (223 pci)

You can run trials by adding thickness to the separation layer to come up with the most economical section as shown in Table B14-20.

Table B14-20. Example 11 Trial Layer Structures

Layer	Thickness, mm (in)		
	Trial 1	Trial 2	Trial 3
PCC	326 (12.85)	315 (12.40)	305 (12.00)
Drainage layer	152 (6)	152 (6)	152 (6)
Separation layer	102 (4)	203 (8)	305 (12)
Natural Subgrade	--	--	--
Note: Final thicknesses should be rounded values.			

B14-2.11.2.2 Now to you need to account for frost conditions. On the PCASE Design Module window, the button for opening the Depth of Frost Penetration Calculator is located in the top grid in the Depth of Frost column. To open the calculator, click the **...** (dot-dot-dot) button in the Depth of Frost column on the row of your design. Then follow these steps:

(1) On the Depth of Frost Penetration Calculator worksheet, select North Dakota from the state list box and Grand Forks Univ NWs from the station list box. The layer structure you built in the design is imported into the form, and the layers in the form correspond to the layers in your design. For the example, PCC refers to the concrete layer, Course Grained refers to the drainage and separation layers, and Fine Grained refers to the natural subgrade.

(2) Enter the dry unit weight and moisture content of each material in the structure as given in Table B14-19b.

(3) Click **Calculate**.

(4) Click **Apply and Close**, which will take you back to the PCASE Design Module window and import the frost penetration depth.

(5) After you have calculated the depth of frost, a new column labeled “Frost Code” appears in the layer grid for the current design. You can then assign a frost code to your subgrade by clicking the cell for a drop-down list of available frost codes. Select F3/F4.

(6) After entering the frost code, click **Save and Calculate**. Your results should be the same as those in Table B14-21 for Non-Frost Design, Reduced Subgrade Strength (RSS), and Limited Subgrade Frost Penetration (LSFP). The separation layer could be increased to decrease the PCC required.

Table B14-21. Example 11 Frost Design Results

Layer	Thickness, mm (in)		
	Non-Frost Design	Reduced Subgrade Strength (RSS)	Limited Subgrade Frost Penetration (LSFP)
PCC	326 (12.85)	442 (17.42)	326 (12.85)
Drainage layer	152 (6)	152 (6)	152 (6)
Separation layer	102 (4)	102 (4)	978 (38.51)
Natural Subgrade	--	--	--
Note: Final thickness should be rounded values.			

For this example, RSS is the controlling design. (The controlling design on the PCASE window is in the white column, not the dimmed column.)

B14-2.12 **Example 12: Overlays**

B14-2.12.1 **Overlay of Existing PCC Pavement**

An existing plain concrete pavement requires strengthening to serve as a Type A traffic area for an Air Force medium-load pavement. Provide thicknesses for the various types of overlays for an existing rigid pavement with these physical properties: PCC thickness is 305 mm (12 in), flexural strength is 4.83 MPa (700 psi), and modulus of soil reaction *k* is 41 kPa/mm (150 pci). The existing pavement is in Fair condition (PCI is 48), with 55 percent of the distresses due to load. The design flexural strength of the concrete for the overlay is 5.17 MPa (750 psi).

B14-2.12.1.1 **Part 1: Traffic Module Steps**

In Example 9, the Air Force Medium traffic pattern was selected and applied, so it may already be available for use in the design module. If it is not available, refer to Example 9.

B14-2.12.1.2 **Part 2: Condition Factors**

To calculate the condition factors, open the Help/Utilities module by clicking the **Help/Utilities** button and then follow these steps:

(1) On the PCASE Utilities window, under Utilities, click ConditionFactorsCbCr.

(2) On the Overlay Condition Factors window, enter 48 in the PCI text box.

(3) In the % Load Distress text box, enter 55.

(4) Click **Calc**. The condition factors Cb (for flexible overlays) and Cr (for rigid overlays) are 0.89 and 0.75, respectively. Note: These values are not stored in memory or transferred to other modules. It is best to write them down for future reference.

(5) Click **Done**.

(6) Click **Exit** to close the PCASE Utilities window.

B14-2.12.1.3 **Part 3: Design Module Steps**

In PCASE, you must build your existing structure first and then add and calculate the new overlay thickness. Open the PCASE Design Module window by clicking **Design** and then follow these steps:

(1) Click **Add** under the design grid (middle of the window).

(2) On the Add Design window, enter a design name of your choice in the Design Name text box.

(3) From the Design Type list box, select Airfield.

(4) From the Pavement Type list box, select Rigid.

(5) Select K.

(6) From the Traffic Area list box, select Area A.

(7) From the Traffic Patterns drop-down list, choose Air Force Medium Copy-1.

(8) Click **Ok**. A series of windows prompts you for this information:

(a) PCC Layer. On the PCC window, change the flexural strength to 4.83 MPa (700 psi); leave the percent steel (0), percent joint load transfer (25), modulus 25,759 MPa (4,000,000 psi), and Poisson's ratio (0.15) set to the defaults; click **Ok**.

(b) Drainage. On the Drainage window, click **Not Required**.

(c) Natural Subgrade. On the Natural Subgrade window, enter 41 kPa/mm (150 pci) in the **K (pci**) text box; then click **Ok**.

(9) Now you need to add an overlay layer. On the PCASE Design Module window, click the **Add** button under the layer grid.

(10) From the Layer Type list box on the Add Layer window, select PCC Unbonded Ov; then click **Ok**.

(11) On the PCC Unbonded Ov window, change the flexural strength to 5.17 MPa (750 psi).

(12) Change the condition factor to 0.75.

(13) Leave the percent steel (0), percent joint load transfer (25), modulus 25,759 MPa (4,000,000 psi), and Poisson's ratio (0.15) set to the defaults; then click **Ok**.

(14) On the PCASE Design Module window, change the Non-Frost Design Thickness column value for the PCC layer to 305 mm (12 in).

(15) Click **Save and Calculate**. Your design is complete, and the PCC unbonded overlay thickness is 330 mm (13 in).

The same process can be used for calculating a thickness for a partially bonded overlay and asphalt overlay (use 0.89 for the condition factor for an asphalt overlay) with the results in Table B14-22. Changing overlay types requires deleting and adding layers. You may want to copy your design first and use a different design name. The bonded overlay could not be calculated for this example because the condition factor is less than one.

Table B14-22. Overlay Thicknesses

Overlay Type	Thickness, mm (in)
AC	381 (15.0)
PCC – partially bonded	267 (10.5)
PCC – unbonded	330 (13.0)

B14-2.12.2 **Overlay of Existing Asphalt Pavement**
An existing asphalt pavement requires strengthening to serve as a Type A traffic area for an Air Force medium-load pavement. Provide the thickness required for an asphalt overlay over an existing asphalt pavement with these physical properties:

- AC: 102 mm (4 in)
- Base: 304 mm (12 in), 80 CBR
- Natural Subgrade: 7 CBR

B14-2.12.2.1 **Part 1: Traffic Module Steps**
In Example 9, the Air Force Medium was selected and applied so it is already available for use in the design module.

B14-2.12.2.2 **Part 2: Design Module Steps**
In PCASE, you must build your existing structure first and then add and calculate the new overlay. Open the PCASE Design Module window and then follow these steps:

(1) Click **Add** under the design grid (middle of the window).

(2) On the Add Design window, enter a design name of your choice in the Design Name text box.

(3) From the Design Type list box, select Airfield.

(4) From the Pavement Type list box, select Flexible.

(5) Select CBR.

(6) From the Traffic Area list box, select Area A.

(7) From the Traffic Patterns drop-down list, choose Air Force Medium Copy-1.

(8) Click **Ok**. A series of windows prompts you for this information:

(a) Asphalt Layer. On the Asphalt window, select Manual from the Analysis drop-down list; enter 4 in the Design Thickness text box; click **Ok**.

(b) Base Layer. On the Base window, leave the Analysis method as Compute (must change it after the overlay is added); select 80 from the CBR drop-down list; click **Ok**.

(c) Drainage. On the Drainage window, click **Not Required**.

(d) Natural Subgrade. On the Natural Subgrade window, enter 7 in the CBR text box; click **Ok**.

(9) Now you need to add an overlay layer. Under the layer grid on the PCASE Design Module window, click the **Add** button.

(10) On the Add Layer window, select Asphalt Overlay from the Layer Type list box; click **Ok**.

(11) On the Asphalt Overlay window, click **Ok**.

(12) On the PCASE Design Module window, change the thickness of the base layer in the Non-Frost Design Thickness column to 305 mm (12 in).

(13) Click **Save and Calculate**. Your design is complete, and the asphalt overlay thickness is 216 mm (8.51 in) (round off to 8.5 in).

B14-2.13 **Example 13: Rigid Airfield Paved Shoulders**
Design rigid airfield shoulders for a runway with a PCC flexural strength of 4.83 MPa (700 psi) and a modulus of soil reaction *k* of 41 kPa/mm (150 pci). Assume that frost and drainage are not issues.

The traffic criteria are built into the design module for airfield paved shoulders; therefore, there is no need to create a traffic pattern or choose a standard traffic pattern. *If you just opened a new file, however, and have not yet applied a traffic pattern, you will need to do so. Any traffic pattern (standard or created) will work. The software requires a traffic pattern for the design module to open.* Open the PCASE Design Module window and then follow these steps:

(1) Click **Add** under the design grid.

(2) On the Add Design window, enter a design name of your choice in the Design Name text box.

(3) From the Design Type list box, select Airfield.

(4) From the Pavement Type list box, select Rigid Shoulder.

(5) From the Analysis Type list box, select K, then click **Ok**. A series of windows prompts you for this information:

(a) PCC Layer. On the PCC window, change the flexural strength to 4.83 MPa (700 psi); leave the percent steel (0), percent joint load transfer (25), modulus (25,759 MPa [4,000,000 psi]), and Poisson's ratio (0.15) set to the defaults; then click **Ok**.

(b) Drainage. On the Drainage window, click **Not Required**.

(c) Natural Subgrade. On the Natural Subgrade window, enter 41 kPa/mm (150 pci) in the **K** text box; then click **Ok.**

(d) On the PCASE Design Module window, click **Save and Calculate**. Your design is complete, and the layer structure should give these results:

Table B14-23. Example 13 Results

Layer	Thickness, mm (in)*	K, kPa/mm (pci)
PCC	152 (6)	--
Natural Subgrade	--	41 (150)

*If pavements are to be used by support vehicles (e.g., snow removal equipment, fire trucks, fuel trucks), the shoulder should be checked by creating a Road design. You will need to create a traffic pattern that includes the support vehicles that use the shoulder and create a design using Roads as your design type. Compare the results of the Shoulder and Road designs; the governing design will be the one that produces the thickest section.

B14-2.14 **Example 14: Rigid Layered Elastic Design (LED) using Current Mission Traffic – One Season**

Design a new PCC pavement section to replace the existing AC runway ends (Traffic Area A) using the current mission traffic in Table B14-24. Use the LED model for design using the defaults in the software for the flexural strength, modulus values, Poisson's Ratios, and slip. A base course thickness of 152 mm (6 in) will be used and will meet the requirements of a drainage layer. Frost does not penetrate the subgrade.

Table B14-24. Example 14 Mission Traffic

Aircraft	Weight, kg (lb)	365-day Period	20-year Traffic
B-727-200	95,500 (210,000)	8	160
C-9A/C	50,000 (110,000)	104	2080
C-12J	7,500 (16,600)	2,200	44,000
C-17A	263,000 (580,000)	30	600
C-130H	70,000 (175,000)	48	960

B14-2.14.1 **Part 1: Traffic Module Steps**
Open the Traffic Module window by clicking the **Traffic** button and then follow these steps:

(1) Click **Create Pattern**.

(2) On the Traffic Pattern window, enter a pattern name in the Pattern Name text box.

(3) Click **Ok**.

(4) In previous versions of PCASE it was necessary in design to set the Analysis Type to Individual for layered elastic design. In PCASE2.09.02 it is automatically calculated as individual; therefore it is not necessary to change the analysis type to Individual.

(5) Click **Add Vehicle**.

(6) On the Choose Vehicles window, select the aircraft listed in Table B14-24 by clicking the check box or by double-clicking the label for each aircraft; click **Add**. The Choose Vehicles window closes.

(7) On the Traffic Module window, change the weight and passes for each of the aircraft to the values in Table B14-24. To change the weight, click the cell under the Traffic Area Weight column for Areas A and B, and enter the new weight for each aircraft. To change the pass level, click the cell under the Traffic Area Pass Levels column for Areas A, B, and C, and enter the new number of passes for each aircraft.

(8) Click **Apply** to send your traffic pattern to the design module.

B14-2.14.2 **Part 2: Design Module Steps**
Open the PCASE Design Module window by clicking **Design** and then follow these steps:

(1) Click **Add** under the design grid (middle of the window).

(2) On the Add Design window, enter a design name of your choice in the Design Name text box.

(3) From the Design Type list box, select Airfield.

(4) From the Pavement Type list box, select Rigid.

(5) Select LED.

(6) From the Traffic Area list box, select Area A.

(7) From the Traffic Patterns drop-down list, select the pattern you created in Step 1 and then click **Ok**.

A series of windows prompts you for this information:

(a) PCC Layer. On the PCC window, leave the Analysis method as Compute; use the defaults for Flexural Strength (4.48 MPa [650 psi]), % Joint Load Transfer (25), Modulus (25,759 MPa [4,000,000 psi]), Poisson's Ratio (0.15), and Slip (1000); click **Ok**. When there is more than one season, you can use the Seasonal Values scrollbar to scroll through each season and input the modulus, Poisson's ratio, and slip.

(b) Drainage. On the Drainage window, click **Not Required**.

(c) Natural Subgrade. On the Natural Subgrade window, change the Modulus to 103 MPa (15,000 psi) and use the defaults for Poisson's Ratio (0.40) and Slip (0); click **Ok**.

(d) Add a base layer by clicking the **Add** button under the layer grid; on the Add Layer window, select Base from the Layer Type list box; click **Ok**. On the Base window, change the value in the Design Thickness text box to 152 mm (6 in); use the default of 420 MPa (61,000 psi), and use the defaults for Poisson's Ratio (0.35) and Slip (0); click **Ok**.

(e) On the PCASE Design Module window, click **Save and Calculate**. Your design is complete, and the layer structure should match the results in Table B14-25.

Table B14-25. Example 14 Results

Layer	Thickness, mm (in)*	Modulus of Elasticity, MPa (psi)
PCC	320 (12.60) (round up to 330 [13])	27,576 (4,000,000)
Base	152 (6)	420 (61,000)
Natural Subgrade	--	103 (15,000)

B14-2.15 Example 15: Rigid Layered Elastic Design (LED) using Current Mission Traffic – Multiple Seasons

In reality, modulus values will vary with changing environmental conditions. Design the pavement in Example 14 to account for changing conditions by assigning different modulus values to different seasons of the year. Use Table B14-26 for establishing the seasons and their corresponding modulus values.

Table B14-26. Example 15 Seasons and Modulus Values

Season Information			Modulus Values, MPa (psi)		
Season	Start Date	End Date	PCC	Base	Subgrade
1	Dec 1	Feb 28	27,576 (4,000,000)	420 (61,000)	103 (15,000)
2	Mar 1	May 31	27,576 (4,000,000)	420 (61,000)	34 (5,000)
3	Jun 1	Nov 30	27,576 (4,000,000)	420 (61,000)	69 (10,000)

B14-2.15.1 **Creating the Layer Structure**
To use the same layer structure as in Example 14, click **Copy** under the design grid. On the Copy Design window, enter a name for your new design in the Design Name text box; then click **Ok**.

B14-2.15.2 **Creating the Seasons**
To create seasons, follow these steps:

(1) Click the **Edit Seasons** button on the right side of the design window under the design grid.

(2) A dialog box displays indicating that seasonal values will be re-set to their defaults; click **Ok.**

(3) On the Seasons window, click **Add** to build a new season set.

(4) On the Add Set of Seasons window, enter a name for your season set (e.g., 3 seasons, 4 seasons, my seasons) in the Set Name text box; click **Ok**.

(5) To break the year up into various seasons, click the **end date** of each season (e.g., Feb 28, May 31, Nov 30).

(6) Once you have completed building a season set, click **Exit** to return to the PCASE Design Module window.

(7) On the PCASE Design Module window, click the **...** (dot-dot-dot) button in the Seasons column of the design grid to assign the newly created season set to your design.

(8) On the Seasons window, select the appropriate season set from the list box and then click **Ok**.

(9) You must now enter modulus values for each of the seasons. Enter the modulus values for each season by entering the modulus values in the

modulus column of the layer grid. Above the layer grid, on the right side, a scrollbar indicates the season. To go to the next season, click the **>** (right arrow) button. Continue scrolling to the next season and entering modulus values until you have changed the values for all seasons.

(10) After you have entered all the modulus values for the seasons, click **Save and Calculate**. Your design is complete, and the layer structure should give these results:

Table B14-27. Example 15 Results

Layer	Thickness, mm (in)*	Modulus of Elasticity, MPa (psi)
PCC	369 (14.52) (round off to 338 [14.5])	27,576 (4,000,000)
Base	152 (6)	207 (30,000)
Natural Subgrade	--	varies

SECTION 15: LIST OF SYMBOLS

Symbol	Description	Units, SI (IP)
A	Regression constant for use in Equation 11-3. Used to compute resilient modulus in IP units. $A = 0.000247 + 0.000245 \log M_R$	Dimensionless, but Equation 11-3 is only for IP units.
a	Thickness for complete frost protection	mm (in)
a	*L*/3 (Equation M-1)	mm (in)
A_c	Cross-sectional area of concrete being prestressed	mm^2 (in^2)
A_p	Cross-sectional area of 1 m (1 ft) of pavement	mm^2 (in^2)
A_s	Cross-sectional area of steel	mm^2 (in^2)
B	Spacing between tandem tires in the gear (Equation 11-1)	mm (in)
B	Regression constant computed for use in Equation 11-3. Used to compute resilient modulus in IP units. $B = 0.0658\ M_R^{0.559}$	Dimensionless, but Equation 11-3 is only for IP units.
B	Load-moment factor	Dimensionless
B_f	Remaining life of base pavement to complete failure	Cycles
B_o	Remaining life of base pavement to initial cracking and complete failure	Cycles
C	Condition factor	Dimensionless
c	Thickness of unbound base	mm (in)
$CaCO_3$	Calcium carbonate	--
CaO	Quick lime	--
$Ca(OH)_2$	Hydrated lime	--
C_f	Coefficient of sliding friction	--
C_f	Allowable coverage level at the time of complete failure (SCI = 0)	
CH	Unified Soil Classification System group symbol for clay of high plasticity, fat clay	--
CL	Unified Soil Classification System group symbol for clay of low plasticity.	--
C_o	Coverage level at which the SCI begins to decrease from 100	

Symbol	Description	Units, SI (IP)
C_o	Allowable coverage level at the time of initial cracking (SCI begins to decrease from 100)	
$(C_o)_j$ and $(C_f)_j$	A function of the changing modulus of elasticity of the base slab in each time period (Equation 19-7)	
C_r	Design traffic rate, coverages per year	--
D	Dry density (Figure 5-1)	kg/m^3 (pcf)
d	Dynamic deflection of beam center	mm (in)
DF	Damage factor (in $DF = \frac{n}{N}$; paragraph 11-6)	--
DF	Design factor computed with layered elastic model	--
$(d_f)_j$	Damage to complete failure for time period *j*	
$(d_o)_j$	Damage to initial cracking for time period *j*	
DR_f	Damage rate at the time of complete failure	
DR_o	Damage rate at the time of initial cracking	
d_s	Design prestress	
E	Modulus of elasticity	MPa (psi)
E_e	Effective modulus of elasticity	MPa (psi)
E_i	Initial undamaged modulus of elasticity	MPa (psi)
E_s	Modulus of elasticity of the reinforcing steel in tension	MPa (psi)
F	Infiltration coefficient	
F	Multiplication factor for tandem gear (Equation 11-1)	--
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_s	Working stress in the steel, MPa (psi) (75 percent of yield tensile strength of steel). This produces a safety factor of 1.33.	MPa (psi)
f_s	Yield strength of steel = 414 MPa (60,000 psi)	MPa (psi)
f_t	7-day tensile strength of the concrete determined using the splitting tensile test	MPa (psi)
f_t	Tensile strength of concrete	MPa (psi)
f_μ	Ultimate strength of the tendon steel	MPa (psi)
G	Specific gravity of soil (Figure 6-3)	Dimensionless

Symbol	Description	Units, SI (IP)
GC	Unified Soil Classification System group symbol for clayey gravel	--
GM	Unified Soil Classification System group symbol for silty gravel	--
GP	Unified Soil Classification System group symbol for poorly graded gravel	--
GW	Unified Soil Classification System group symbol for well graded gravel, fine to coarse gravel	--
H	Thickness of the drainage layer	m (ft)
h_d	Design thickness of reinforced concrete	mm (in)
h_d, 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.	mm (in)
h_E	Existing plain concrete pavement thickness	mm (in)
h_e	Design thickness	mm (in)
h_p	Design thickness of prestressed concrete pavement	mm (in)
i	Slope of the drainage path	Percent
ID	Inside diameter (Figure 5-1)	mm (in)
$j\Delta T_j$	Magnitude of the time interval in years (Equation 19-7)	Time
k	Permeability of the drainage layer	Dimensionless
k	Modulus of soil reaction	MPa/mm (psi/in)
k	Modulus of subgrade reaction	MPa/mm (psi/in)
L	Length of the drainage path	m (ft)
L	Load per tire (Equation 11-2)	kg (lb)
L	Reaction span length	mm (in)
L	Length of prestressed concrete slab	m (ft)
L	Slab length	m (ft)
LL	Liquid limit	--
MH	Unified Soil Classification System group symbol for silt of high plasticity, elastic silt	--
ML	Unified Soil Classification System group symbol for silt	--

Symbol	Description	Units, SI (IP)
$M_R = \sigma_d/\in_{RI}$	Resilient modulus	MPa (psi)
M_R	Resilient modulus of the subgrade	MPa (psi)
N	Load-repetition factor	--
N	Number of blows (Figure 5-1)	--
N	Allowable number of passes	--
N	Number of allowable strain repetitions (in $DF = \frac{n}{N}$)	--
n	Number of effective strain repetitions (in $DF = \frac{n}{N}$)	--
n	Applied traffic in terms of passes or coverages.	--
nac	Number of aircraft	--
n_e	Effective porosity	Dimensionless
N_f	Fatigue life of the specimen, number of load repetitions to fracture	Dimensionless
NP	Nonplastic (Figure 5-1)	--
OD	Outside diameter (Figure 5-1)	
OH	Unified Soil Classification System group symbol for organic clay, organic silt	--
OL	Unified Soil Classification System group symbol for organic silt, organic clay	--
P	Aircraft gear load	kg (lb)
p	Contact pressure (Equation 11-2)	MPa (psi)
p	Thickness of asphalt or concrete for nonfrost design	mm (in)
PI	Plasticity index	--
P_s	Percent of reinforcing steel required in the longitudinal direction	--
Pt	Unified Soil Classification System group symbol for peat	--
Q	Volume of water	m^3 (ft^3)
R	Concrete slab flexural strength	MPa (psi)
R	Design storm index	--
r	Radius of loaded area (Equation 11-2)	mm (in)
r_s	Foundation restraint stress	MPa (psi)
S	Percent reinforcing steel	--
S_A	Tensile strain of asphalt (Equation 11-4)	mm/mm (in/in)

Symbol	Description	Units, SI (IP)
SC	Unified Soil Classification System group symbol for clayey sand	--
SM	Unified Soil Classification System group symbol for silty sand	--
SP	Unified Soil Classification System group symbol for poorly-graded sand	--
S_s	Vertical strain at the top of the subgrade (Equation 11-3)	mm/mm (in/in)
SW	Unified Soil Classification System group symbol for well graded sand, fine to coarse sand	--
T	Difference in temperature between the top and bottom of the prestressed concrete pavement	degrees C (degrees F)
T	Seasonal temperature differential	degrees C (degrees F)
t	Length of the design storm	Time
t	Specimen depth	mm (in)
t_e	Effective pavement thickness (Equation 11-1)	mm (in)
t_f	Time to complete failure	Time
t_o	Time to initial cracking	Time
t_s	Temperature warping stress	MPa (psi)
T_w	Length of the ellipse that is formed by the tire imprint (Equation 11-1)	mm (in)
T_{85}	Time it takes for a drainage layer to achieve 85 percent drainage	Time
W	Width of prestressed concrete slab	m (ft)
W	Water content in percent of dry weight (Figure 5-1)	percent
w	Ratio of multiple-wheel gear load to single-wheel gear load	Dimensionless
W_s	Width of slab	m (ft)
X	2.68 - 5.0 log S_A - 2.665 log E (Equation 11-4)	Dimensionless
y_s	Yield strength of reinforcing steel, normally 413.7 MPa (60,000 psi)	MPa (psi)
Δ_{LT}	Change in length of slab due to temperature change ΔT	mm (in)
Δ_{LM}	Maximum change in length of slab due to seasonable moisture change	mm (in)

Symbol	Description	Units, SI (IP)
ΔT	Change in temperature (either daily or seasonally)	degrees C (degrees F)
$\in$	Initial extreme fiber strain (tensile and compressive)	mm/mm (in/in)
ε_c	Coefficient of thermal expansion	mm/mm per degree C (in/in per degree F)
$\in_M$	Coefficient of moisture expansion of concrete (assumed to be 1×10^{-4} inch per inch seasonally)	mm/mm, (in/in)
$\in_R$	Resilient or recoverable axial strain due to σ_d	MPa (psi)
$\in_{R1}$	Resilient or recoverable axial strain due to σ_d in the direction perpendicular to ε_R	MPa (psi)
$\in_1$	Total axial strain due to σ_d	MPa (psi)
$\theta = \sigma_1+2\sigma_3 = \sigma_d+3\sigma_3$	Sum of the principal stresses in the triaxial state of stress	MPa (psi)
μ	Poisson's ratio	Dimensionless
ρ	Density of concrete	kg/m^3 (lb/ft^3)
σ	Maximum computed tensile stress with layered elastic model such as PCASE computer program	MPa (psi)
σ_o	Axial loading stress	MPa (psi)
σ_1	Total axial stress	MPa (psi)
σ_3	Total radial stress; i.e., confining pressure in the triaxial test chamber	MPa (psi)
$\sigma_d = \sigma_1-\sigma_3$	Deviator strain	MPa (psi)
σ_1/σ_3	Principal stress ratio	MPa (psi)
γ	Soil density	kg/m^3 (lb/ft^3)
Φ	Diameter	--
$<$	Less than	--
$\leq$	Equal to or less than	--
$>$	Greater than	--
$\geq$	Equal to or greater than	--
$=$	Equals	--
%	Percent	--
^{o}	Degree	--
P	Radius of relative stiffness	mm (in)
ω	Soil moisture content	Percent