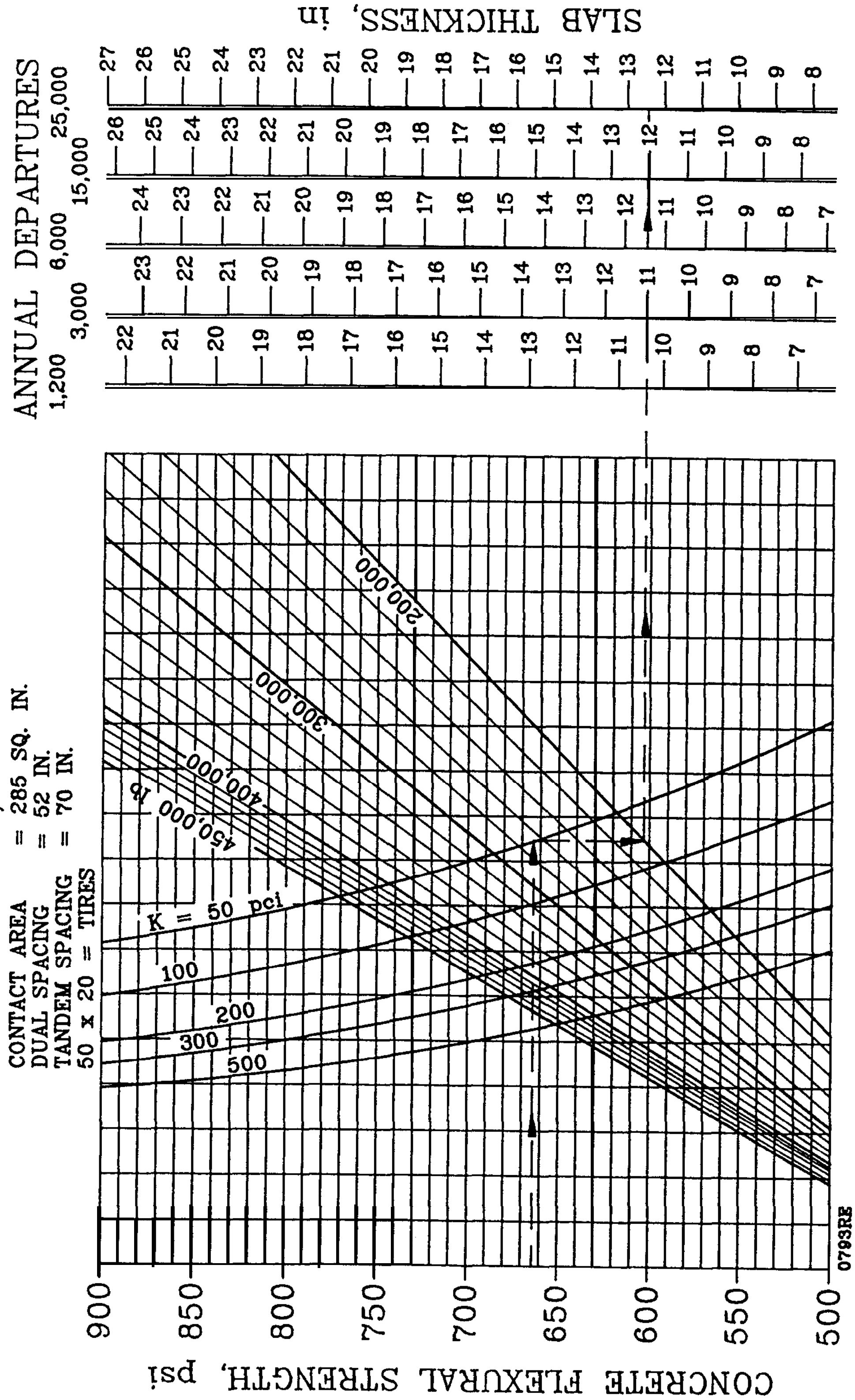
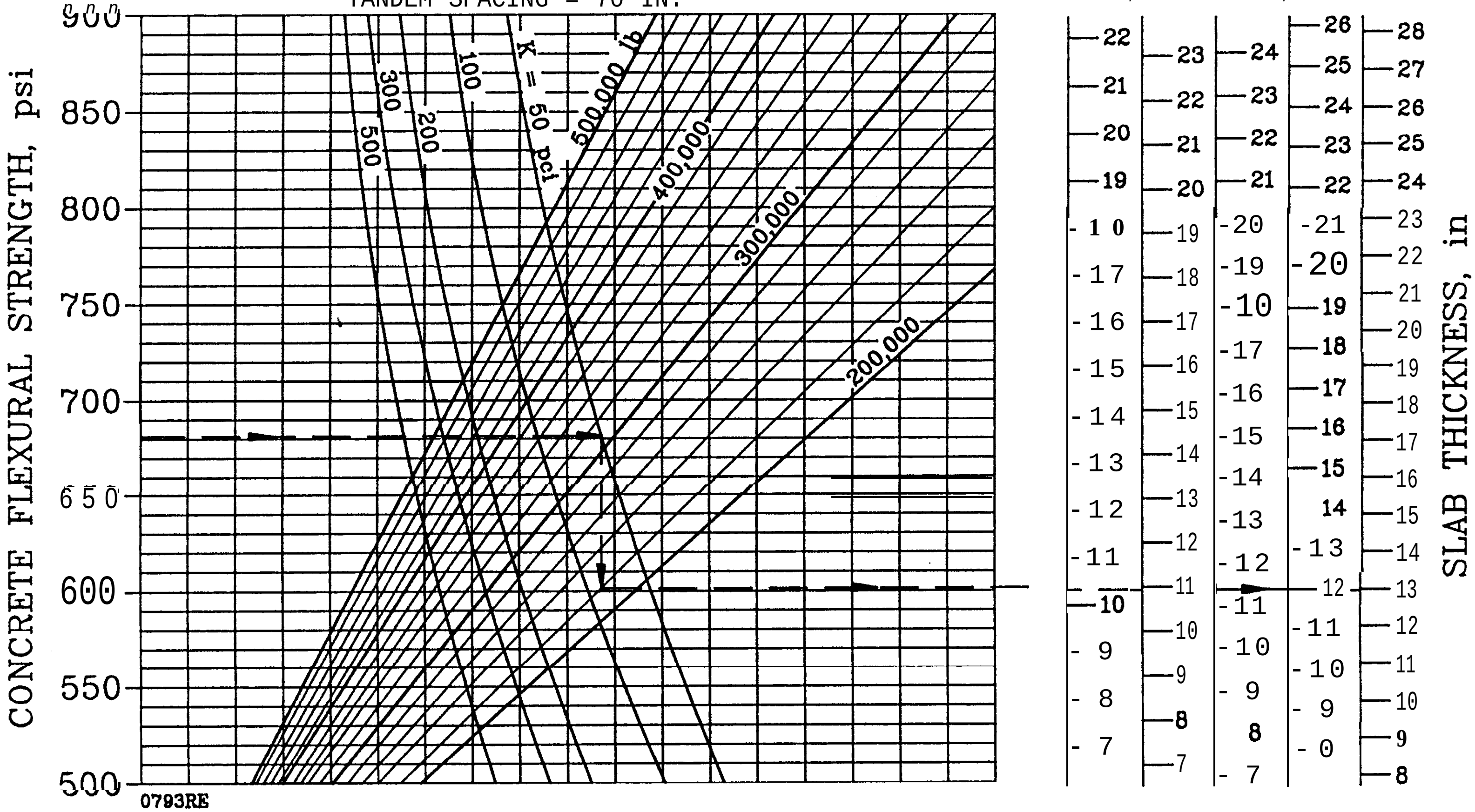


NOTE:
1 inch = 25.4 mm 1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg 1 pci = 0.272 MN/m^3

FIGURE 3-40. OPTIONAL RIGID PAVEMENT DESIGN CURVES, L-1011-1, 100

L-1011-100, 200

CONTACT AREA = 337 SQ. IN.
DUAL SPACING = 52 IN.
TANDEM SPACING = 70 IN.



NOTE:
1 inch = 25.4 mm   1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg   1 pci = 0.272 MN/m^3

FIGURE. 3-41. OPTIONAL PAVEMENT DESIGN CURVES, L-1011-100,200

334. DESIGN EXAMPLE. As an example of the use of the design curves, assume that a rigid pavement is to be designed for dual tandem aircraft having a gross weight of 350,000 pounds (160 000 kg) and for 6,000 annual equivalent departures of the design aircraft. The equivalent annual departures of 6,000 include 1,200 annual departures of B-747 aircraft weighing 780,000 pounds (350 000 kg) gross weight. The subgrade modulus of 100 PCI (25 MN/m^3) with poor drainage and frost penetration is 18 inches (460 mm). The feature to be designed is a primary runway and requires 100 percent frost protection. The subgrade soil is CL. Concrete mix designs indicate a flexural strength of 650 PSI (4.5 MN/m*) can be readily produced with locally available aggregates. The gross weight of the design aircraft dictates the use of a stabilized subbase. Several thicknesses of stabilized subbases should be tried to determine the most economical section. Assume a stabilized subbase of P-304 will be used. Try a subbase thickness of 6 inches (150 mm). Using Figure 3-16, a 6-inch (150 mm) thickness of P-304 would likely increase the foundation modulus from 100 PCI (25 MN/m^3) to 210 PCI (57 MN/m^3). Using Figure 3-19, dual tandem design curve, with the assumed design data, yields a concrete pavement thickness of 16.6 inches (422 mm). This thickness would be rounded off to 17 inches (430 mm). Since the frost penetration is only 18 inches (460 mm) and the combined thickness of concrete pavement and stabilized subbase is 23 inches (585 mm), no further frost protection is needed. Even though the wide body aircraft did not control the thickness of the slab, the wide bodies would have to be considered in the establishment of jointing requirements and design of drainage structures. Other stabilized subbase thicknesses should be tried to determine the most economical section.

335. FROST EFFECTS. As with flexible pavements, frost protection should be provided for rigid pavements in areas where conditions conducive to detrimental frost action exist. Frost protection considerations for rigid pavements are similar to those for flexible pavements. The determination of the depth of frost protection required is given in paragraph 308.b. Local experience may be used to refine the calculations.

a. Example. Assume the above design example is for a primary runway and requires complete frost protection. The subgrade soil is CL, weighing 115 lbs/cu ft (184 kg/cu m). The design freezing index is 500 degree days. Referring to Figure 2-6 shows the depth of frost penetration to be 34 inches (865 mm). The structural considerations yield a 23 inch (585 mm) thickness of non-frost susceptible material. Since the frost penetration is only 18 inches (460 mm) and the combined thickness of concrete pavement and stabilized subbase is 23 inches (585 mm), no further frost protection is needed. Even though the wide body aircraft did not control the thickness of the slab, the wide bodies would have to be considered in the establishment of jointing requirements and design of drainage structures. Other stabilized subbase thicknesses should be tried to determine the most economical section.

(1) Complete Frost Protection. The complete frost protection method applies only to FG-3 and FG-4 soils which are extremely variable in horizontal extent. These soil deposits are characterized by very large, frequent, and abrupt changes in frost heave potential. The variability is such that the use of transition sections is not practical.

(2) Limited Subgrade Frost Penetration. This design method should be used for FG-4 soils except where the conditions require complete protection, see (1) above. The method also applies to soils in frost groups FG-1, FG-2, and FG-3 when the functional requirements of the pavement permit a minor amount of frost heave. Consideration should be given to using transition sections where horizontal variability of frost heave potential permits.

(3) Reduced Subgrade Strength. The reduced subgrade strength method is recommended for FG-1, FG-2, and FG-3 subgrades which are uniform in horizontal extent or where the functional requirements of the pavement will permit some degree of frost heave. the method may also be used for variable FG-1 through FG-3 subgrades for less sensitive pavements which are subject to slow speed traffic and heave can be tolerated.

336. HIGH TRAFFIC VOLUMES. There are a number of airports which experience traffic intensities in excess of those indicated on the design curves. Pavement maintenance is difficult and costly at high activity airports due to traffic intensity and the potential for aircraft delays. Performance of airport pavements under high traffic intensities has been reported in FAA-PM-84/14 (see Appendix 4). Rigid pavement designed to serve in situations where traffic intensity is high should reflect the following considerations.

a. Foundation. The foundation for the pavement provides the ultimate support to the structure. Every effort should be made to provide a stable foundation as problems arising later from an inadequate foundation cannot be practicably corrected after the pavement is constructed. The use of stabilized subbase will aid greatly in providing a uniform, stable foundation.

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

c. Panel Size. Slab panels should be constructed to minimize joint movement. Panel sizes given in paragraph 336 should be selected conservatively. Small joint movement tends to provide for better load transfer across joints and reduces the elongation the joint sealant materials must accommodate when the slabs expand and contract. High quality joint sealants should be specified to provide the best possible performance.

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

a. Joint Categories. Pavement joints are categorized according to the function which the joint is intended to perform. The categories are expansion, contraction, and construction joints. All joints, regardless of type, should be finished in a manner which permits the joint to be sealed. Pavement joint details are shown in Figure 3-42 and are summarized in Table 3-10. These various joints are described as follows:

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

(i) Type A. Type A is used when load transfer across the joint is required. This joint contains a 3/4-inch (19 mm) nonextruding compressible material and is provided with dowel bars for load transfer.

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

(2) Contraction Joints. The function of contraction joints is to provide controlled cracking of the pavement when the pavement contracts due to decrease in moisture content, or a temperature drop. Contraction joints also decrease stresses caused by slab warping. Details for contraction joints are shown as Types F, G, and H, in Figure 3-42.

(3) Construction Joints. Construction joints are required when two abutting slabs are placed at different times such as at the end of a day's placement, or between paving lanes. Details for construction joints are shown as Types C, D, and E in Figure 3-42.

b. Joint Spacing.

(1) Without Stabilized Subbase, A rule-of-thumb for joint spacing given by the Portland Cement Association is applicable for rigid pavements without stabilized subbase: "As a rough guide, the joint spacing (in feet) should not greatly exceed twice the slab thickness (in inches)." Table 3-l 1 shows the recommended maximum joint spacings. Shorter spacings may be more convenient in some instances. The ratio of slab length to slab width should not exceed 1.25 in unreinforced pavements.

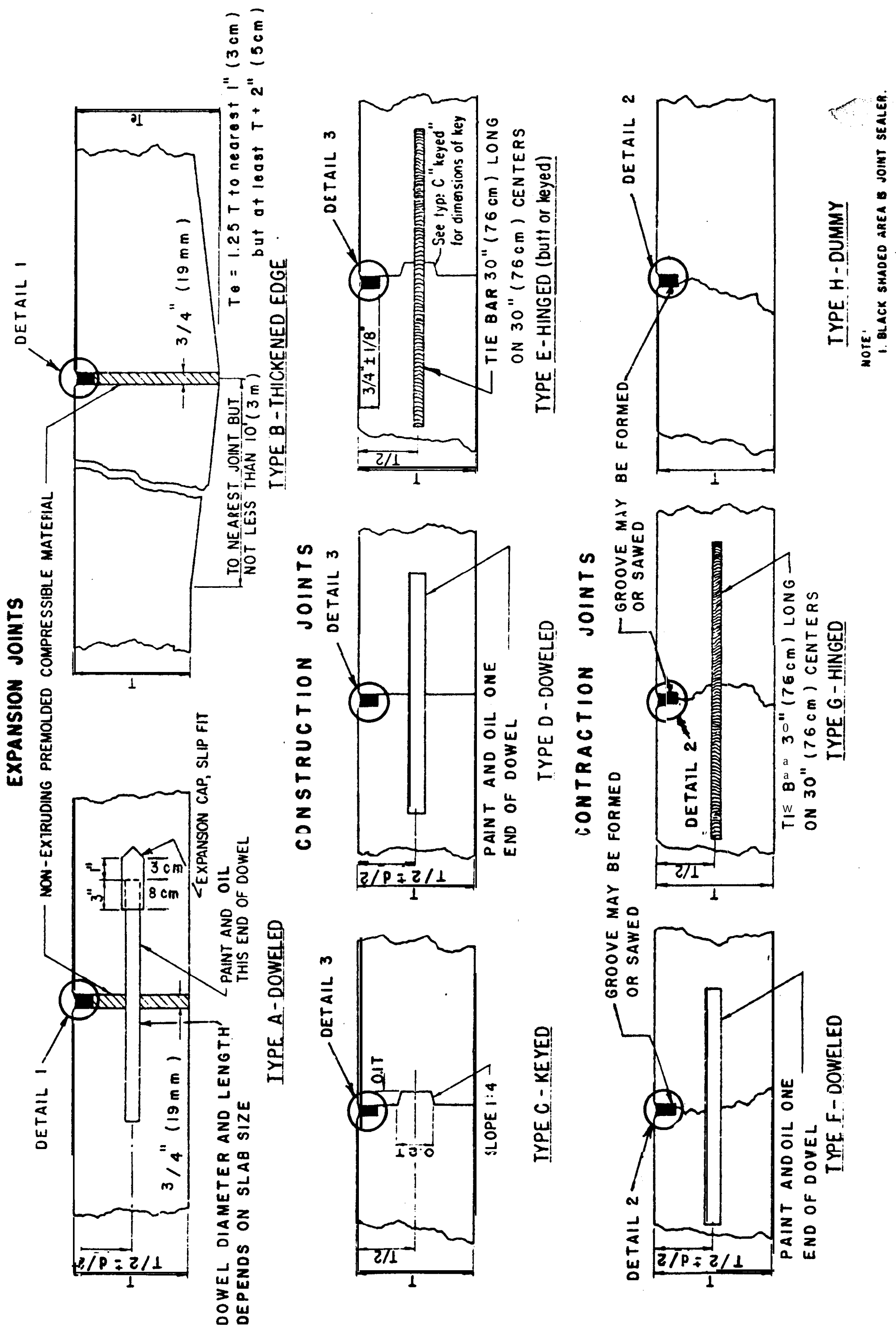
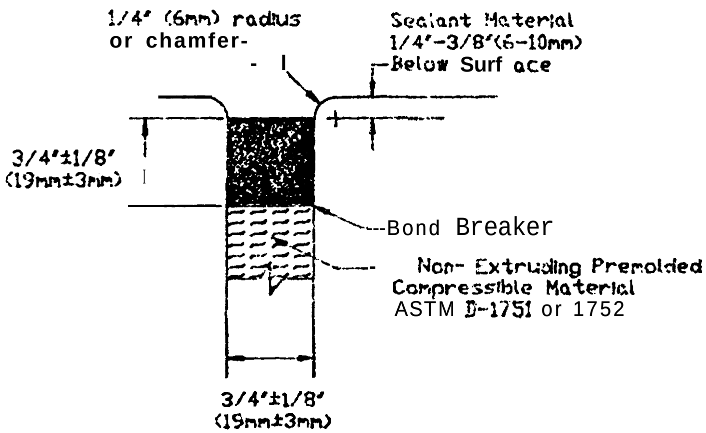
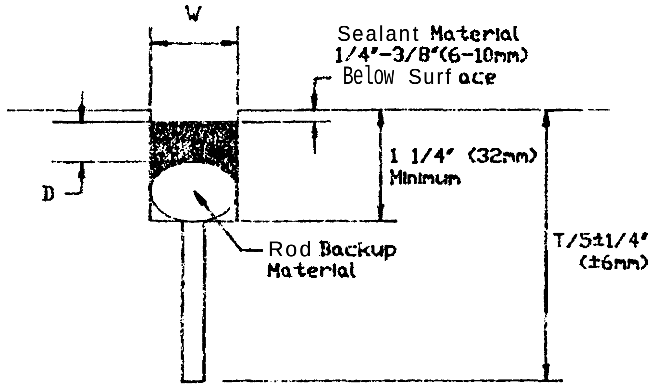


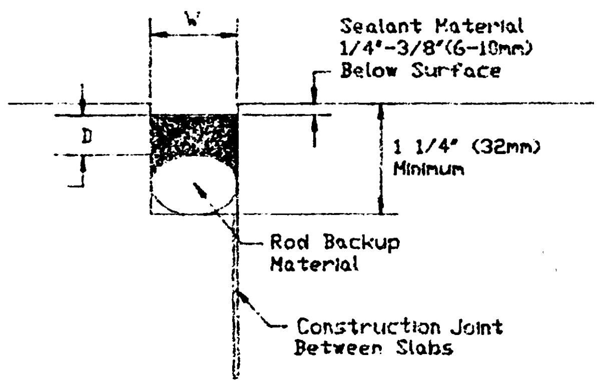
FIGURE 3-42. RIGID PAVEMENT JOINT TYPES AND DETAILS



DETAIL 1
EXPANSION JOINT



DETAIL 2
CONTRACTION JOINT



DETAIL 3
CONSTRUCTION JOINT

Notes:
1. Sealant Reservoir Sized to Provide Proper Shape Factor, W/D, Field Poured and Prefporned Sealants Require Dif ferent Shape factors for Optimum Perfernonce.
2. Rod Bock-Up Material Kust Be Compatible With the Type of Liquid Sealant Used and Sized to Provide the Desired Shape Factor.

FIGURE 3-42. RIGID PAVEMENT JOINT TYPES AND DETAILS

With Stabilized Subbase. Rigid pavements supported on stabilized subbase are subject to higher warping and curling stresses than those supported on unstabilized foundations. When designing a rigid pavement supported on a stabilized subbase a different procedure is recommended to determine joint spacing. Joint spacing should be a function of the radius of relative stiffness of the slab. The joint spacing should be selected such that the ratio of the joint spacing to the radius of relative stiffness is between 4 and 6. The radius of relative stiffness is defined by Westergaard as the stiffness of the slab relative to the stiffness of the foundation. It is determined by the following formula:

$$l = \left(\frac{Eh^3}{12(1-u^2)k}\right)^{\frac{1}{4}}$$

Where:
E = modulus of elasticity of the concrete, usually 4 million psi
h = slab thickness, in.
u = Poisson's ratio for concrete, usually 0.15
k = modulus of subgrade reaction, pci

The radius of relative stiffness has the dimension of length and when calculated in accordance with the above, the units of l are inches.

TABLE 3-11. RECOMMENDED MAXIMUM JOINT SPACINGS RIGID PAVEMENT WITHOUT STABILIZED SUBBASE

Slab Thickness		Transverse		Longitudinal	
Inches	Millimeters	Feet	Meters	Feet	Meters
6	150	12.5	3.8	12.5	3.8
7-9	175-230	15	4.6	15	4.6
9-12	230-305	20	6.1	20	6.1
> 12	> 305	25	7.6	25	7.6

Note: The joint spacings shown in this table are recommended maximum values. Smaller joint spacings should be used if indicated by past experience. Pavements subject to extreme seasonal temperature differentials or extreme temperature differentials during placement may require smaller joint spacings. See also Chapter 5 for light load rigid pavement jointing.

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

a. Keyed Joints. Keyed construction joints should not be used for slabs less than 9 inches (230 mm) in thickness. Keyed joints in slabs of lesser thickness result in very small keys and key-ways with limited strength.

b. **Jointing Systems for Wide Body Jet Aircraft.** Experience indicates poor performance may result from keyed longitudinal construction joints supported on low-strength foundations when wide body aircraft loadings are encountered. Special jointing recommendations are discussed below.

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

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

(3) High Strength Foundations. For foundation moduli of 400 PCI (109 MN/m^3) or greater conventional keyed joints, Type C, may be used regardless of traffic usage. The designer is reminded, however, that the prohibition against keyed joints in pavements less than 9 inches (230 mm) thick shall still remain in effect.

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

339. JOINTING STEEL.

a. Tie Bars. Tie bars are used across certain longitudinal contraction joints and keyed construction joints to hold the slab faces in close contact. The tie bars themselves do not act as load transfer devices. By preventing wide opening of the joint, load transfer is provided by the keyed joint or by aggregate interlock in the crack below the groove-type joint. Tie bars should be deformed bars conforming to the specifications given in Item P-501. The bars should be 5/8 inch (16 mm) in diameter and 30 inches (760 mm) long and spaced 30 inches (760 mm) on center.

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

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

(2) **Size Length and Spacing.** Dowels should be sized such that they will resist the shearing and bending stresses produced by the loads on the pavement. They should be of such length and spacing that the bearing stresses exerted on the concrete will not cause failure of the concrete slab. Table 3-12 indicates the dowel dimensions and spacing for various pavement thicknesses.

TABLE 3-12. DIMENSIONS AND SPACING OF STEEL DOWELS

Thickness of Slab	Diameter	Length	Spacing
6-7 in	3/4 in	18 in	12in
(150-180 mm)	(20 mm)	(460 mm)	(305 mm)
8-12 in	1 in	19 in	12in
(210-305 mm)	(25 mm)	(480 mm)	(305 mm)
13-16 in	1 1/4 in[1]	20 in	15 in
(330-405 mm)	(30 mm)	(510 mm)	(380 mm)
17-20 in	1 1/2 in[1]	20 in	18 in
(430-5 10 mm)	(40 mm)	(510 mm)	(460 mm)
21-24 in	2 in[1]	24 in	18 in
(535-610 mm)	(50 mm)	(610 mm)	(460 mm)

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

(3) **Dowel Positioning.** The alignment and elevation of dowels is extremely important in obtaining a satisfactory joint. Transverse dowels will require the use of a fixture, usually a wire cage or basket firmly anchored to the subbase, to hold the dowels in position. During the concrete placement operations, it is advisable to place plastic concrete directly on the dowel assembly immediately prior to passage of the paver to prevent displacement of the assembly by the paving equipment. Some paving machines have a dowel placer which can also be used to accurately position dowels.

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

341. JOINT LAYOUT. Pavement joint layout is a matter of selecting the proper joint types and locations so that the joints can perform their intended function. Construction considerations are also vitally important in determining the joint layout pattern. Paving lane widths will often dictate how the pavement should be jointed. Generally speaking it is more economical to keep the number of passes of the paving train to a minimum while maintaining proper joint function. Figure 3-43 shows a typical jointing plan for a runway end, parallel taxiway and connector. It is impossible to illustrate all of the variations which can occur at pavement intersections. Reference 8 in Appendix 4 contains further information on jointing patterns. Two important considerations in designing joint layouts for intersections are isolation joints and odd-shaped shapes. More discussion on these follows:

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

b. Odd-Shaped Slabs. Cracks tend to form in odd-shaped slabs; therefore, it is normally good practice to maintain sections which are nearly square or rectangular in shape. Pavement intersection which involve fillets are difficult to design without a few odd-shaped slabs. In instances where odd-shaped slabs cannot be avoided, steel reinforcement is recommended. Steel reinforcement should consist of 0.050% steel in both directions in slabs where the length-to-width ratio exceeds 1.25 or in slabs which are not rectangular in shape. The steel reinforcement should be placed in accordance with the recommendations given in Paragraph 34 1, Reinforced Concrete Pavement. Fillets may also be defined by constructing slabs to the normal, full dimensions and painting out the unused portion of the slab with bitumen.

342. REINFORCED CONCRETE PAVEMENT. The main benefit of steel reinforcing is that, although it does not prevent cracking, it keeps the cracks that form tightly closed so that the interlock of the irregular faces provides structural integrity and usually maintains pavement performance. By holding the cracks tightly closed, the steel minimizes the infiltration of debris into the cracks. The thickness requirements for reinforced concrete pavements are the same as plain concrete and are determined from the appropriate design curves, Figures 3-17 through 3-41. Steel reinforcement allows longer joint spacings, **thus the** cost benefits associated with fewer joints must be considered in the decision to use plain or reinforced concrete pavement.

343. TYPE AND SPACING OF REINFORCEMENT. Reinforcement may be either welded wire fabric or bar mats installed with end and side laps to provide complete reinforcement throughout the slab panel. End laps should be a minimum of 12 inches (305 mm) but **not** less than 30 times the diameter of the longitudinal wire or bar. Side laps should be a minimum of 6 inches (150 mm) but not less than 20 times the diameter of the transverse wire or bar. End and side clearances should be a maximum of 6 inches (150 mm) and a minimum of 2 inches (50 mm) to allow for nearly complete reinforcement and yet achieve adequate concrete cover. Longitudinal members should be spaced **not** less than 4 inches (100 mm) nor more than 12 inches (305 mm) apart; transverse members should be spaced not less than 4 inches (100 mm) nor more than 24 inches (610 mm) apart.

344. AMOUNT OF REINFORCEMENT.

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

$$A_s = 3.7 \frac{L^2 t}{f_s}$$

where:

A_s = area of steel per foot of width or length, square inches
L = length or width of slab, feet
t = thickness of slab, inches
f_s = allowable tensile stress in steel, psi

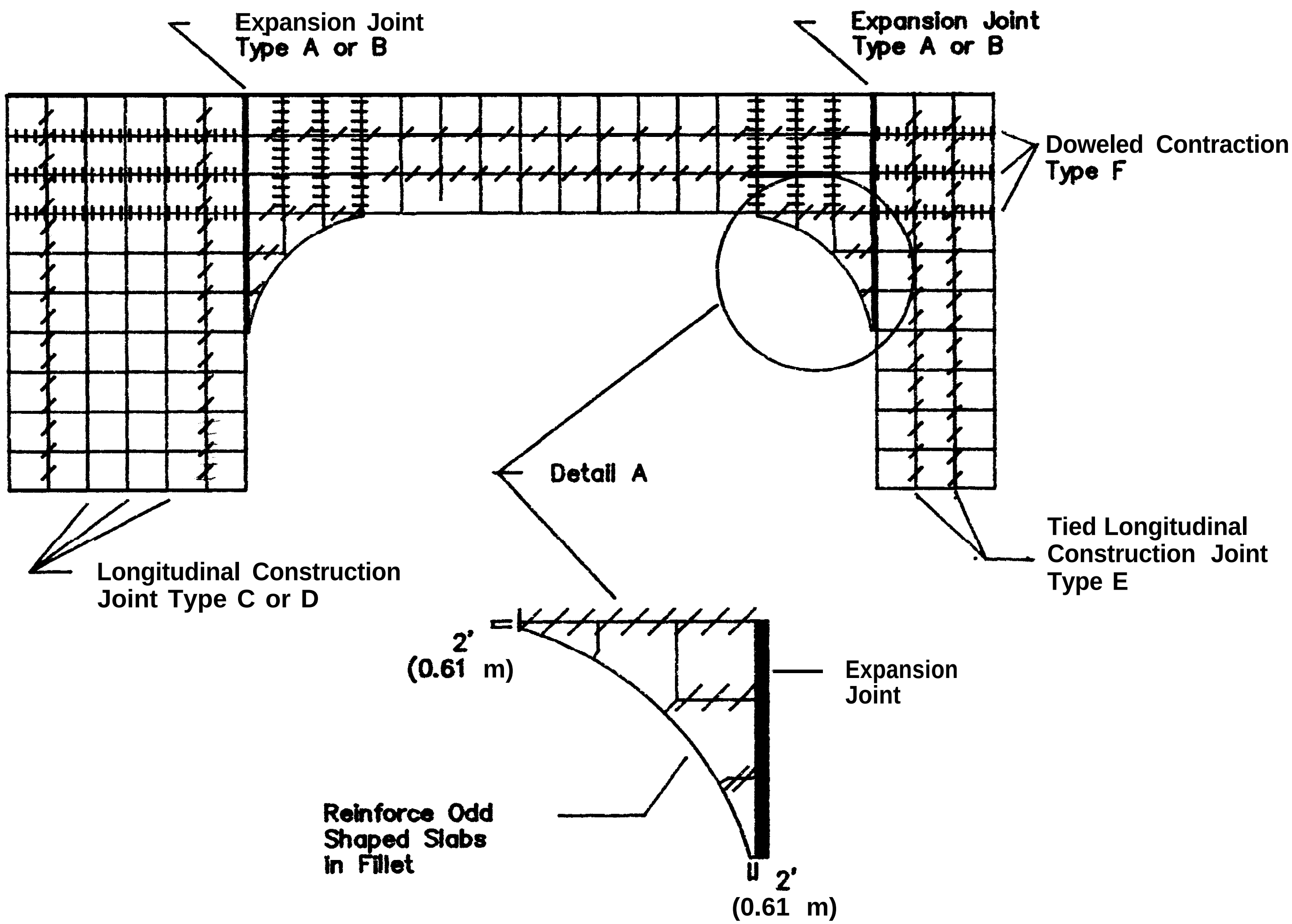


FIGURE 3-43. TYPICAL JOINT LAYOUT PATTERN FOR RUNWAY, PARALLEL; TAXIWAY AND CONNECTOR

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

L should be expressed in meters
t should be expressed in millimeters
f_s should be expressed in Mega newtons per square meter
The constant 3.7 should be changed to 0.64.
A, will then be in terms of square centimeters per meter.

b. In this formula the slab weight is assumed to be 12.5 pounds per square foot, per inch of thickness (23.6 MN/m*). The allowable tensile stress in steel will vary with the type and grade of steel. It is recommended that allowable tensile stress be taken as two-thirds of the yield strength of the steel. Based on current specifications the yield strengths and corresponding design stresses (fs) are as listed in Table 3-13.

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

TABLE 3-13. YIELD STRENGTHS OF VARIOUS GRADES OF REINFORCING STEEL

ASTM Designation	Type & Grade of Steel	Yield Strength psi (MN/m’)	FS psi (MN/n?)
A 615	Deformed Billet Steel, Grade 40	40,000 (300)	27,000 (200)
A 616	Deformed Rail Steel, Grade 50	50,000 (370)	33,000 (240)
A 616	Deformed Rail Steel, Grade 60	60,000 (440)	40,000 (300)
A 615	Deformed Billet Steel, Grade 60	60,000 (440)	40,000 (300)
A 185	Cold Drawn Welded Steel Wire Fabric	65,000 (480)	43,000 (320)
A 497	Cold Drawn Welded Deformed Steel Wire	70,000 (520)	47,000 (350)

TABLE 3-14. DIMENSIONS AND UNIT WEIGHTS OF DEFORMED STEEL REINFORCING BARS

NOMINAL DIMENSIONS

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

345. **DIMENSIONS AND WEIGHTS OF REINFORCEMENT.** Dimensions and unit weights of standard deformed reinforcing bars are given in Table 3-14, and wire size number, diameters, areas, and weights of wires used in welded wire fabric are given in Table 3-15.

346. **WELDED WIRE FABRIC.** The use of welded wire fabric requires some special design considerations to achieve the most economical design. The use of smooth welded wire fabric or deformed welded wire fabric is the option of the designer. The choice should be based on the difference in allowable design stresses, the availability of the desired sizes (smooth wire fabric is available in a wider range of sizes), and the costs associated with each style of fabric. It is recommended that the minimum size of longitudinal wire by W5 or D5. The minimum transverse wire should be no smaller than W4 or D4. In addition, should calculated area of longitudinal steel be less than 0.05 percent of the

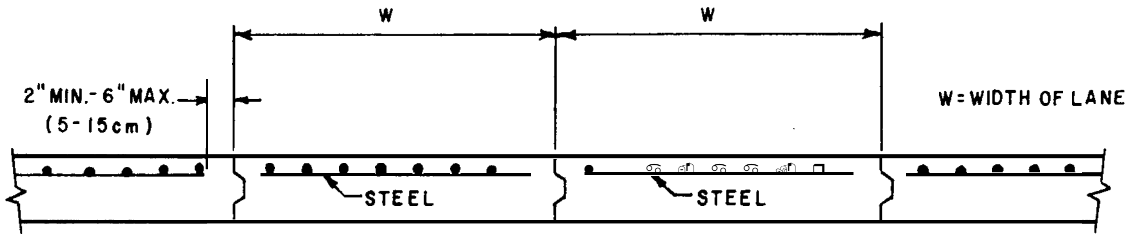
cross-sectional area of slab, the size and spacing of the steel members (bars or wire) should be determined on the premise that the minimum area should not be less than 0.05 percent. This percentage applies in the case of steel having a yield strength of 65,000 PSI (480 MN/m'). If lower grades are used, the percentage should be revised proportionately upward. For example, Table 3-15 shows that W10 wires, spaced 10 inches (255 mm) apart, furnish an area of 0.12 square inches (77 mm^2) which satisfies the requirement for pavements up to 20 inches (510 mm) thick. Sizing of individual sheets of welded wire fabric is also important in providing an economical design. Not all fabricators supply all wire sizes in all spacings. While nearly any fabric style can be produced on special order, it is generally more economical to specify a standard production configuration. Sheet and roll widths in excess of 8 feet (2.5 m) can result in higher shipping costs.

TABLE 3-15. SECTIONAL AREAS OF WELDED FABRIC

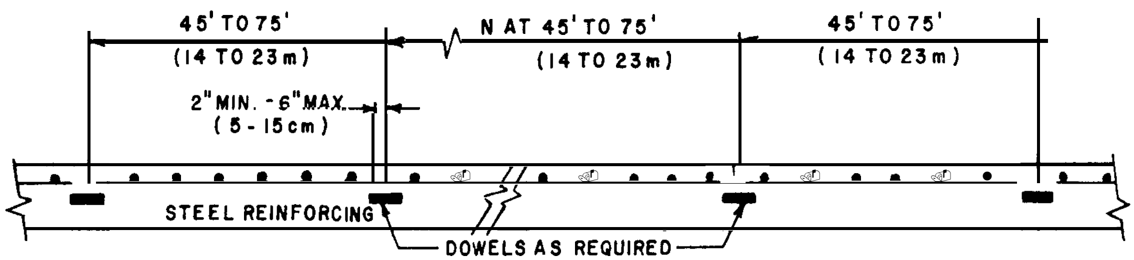
Wire Size Smooth	Number Deformed	Nominal Diameter Inches	Nominal Weight lbs./lin.ft.	Center-to-Center Spacing 4"	6"	8"	10"	12"
W31	D31	0.628	1.054	.93	.62	.465	.372	.31
W30	D30	0.618	1.020	.90	.60	.45	.36	.30
W28	D28	0.597	.952	.84	.56	.42	.336	.28
W26	D26	0.575	.934	.78	.52	.39	.312	.26
W24	D24	0.553	.816	.72	.48	.36	.288	.24
w22	D22	0.529	.748	.66	.44	.33	.264	.22
W20	D20	0.504	.680	.60	.40	.30	.24	.20
W18	D18	0.478	.612	.54	.36	.27	.216	.18
W16	D16	0.451	.544	.48	.32	.24	.192	.16
W14	D14	0.422	.476	.42	.28	.21	.168	.14
w12	D12	0.390	.408	.36	.24	.18	.144	.12
W11	D11	0.374	.374	.33	.22	.165	.132	.11
W10.5		0.366	.357	.315	.21	.157	.126	.105
W10	D10	0.356	.340	.30	.20	.15	.12	.10
w9.5		0.348	.323	.285	.19	.142	.114	.095
W9	D9	0.338	.306	.27	.18	.135	.108	.09
W8.5		0.329	.289	.255	.17	.127	.102	.085
W8	D8	0.319	.272	.24	.16	.12	.096	.08
w7.5		0.309	.255	.225	.15	.112	.09	.075
w 7	D7	0.298	.238	.21	.14	.105	.084	.07
W6.5		0.288	.221	.195	.13	.097	.078	.065
W6	D6	0.276	.204	.18	.12	.09	.072	.06
w5.5		0.264	.187	.165	.11	.082	.066	.055
W5	D5	0.252	.170	.15	.10	.075	.06	.05
w4.5		0.240	.153	.135	.09	.067	.054	.045
w 4	D4	0.225	.136	.12	.08	.06	.048	.04

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

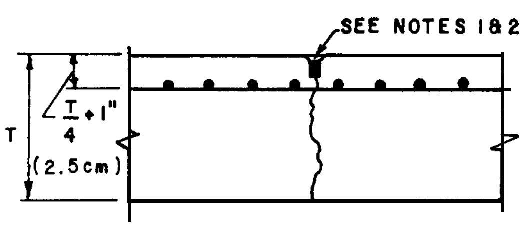
347. **JOINTING OF REINFORCED PAVEMENTS.** Contraction joints in reinforced pavements may be spaced up to 75 feet (23 m) apart, and all joints should be provided with load transfer devices as shown in Figure 3-44. Also, this figure presents other reinforcement details such as clearance at joints and edges of pavement and depth below the surface. The longer joint spacing allowed with reinforced pavements will result in larger joint openings. The joints must be sealed carefully to accommodate the larger movements at the joints.



TRANSVERSE CROSS SECTION OF PAVING LANES



LONGITUDINAL CROSS SECTION



LONGITUDINAL CONTRACTION JOINT
(SEE NOTE 3)

NOTES:

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

FIGURE 3-44 JOINTING OF REINFORCED RIGID PAVEMENTS

348. CONTINUOUSLY REINFORCED CONCRETE PAVEMENT. A continuously reinforce concrete pavement (CRCP) is a portland cement concrete pavement with continuous longitudinal steel reinforcement and no intermediate transverse expansion or contraction joints. Continuously reinforced concrete pavements normally contain from 0.5 to 1.0 percent longitudinal steel reinforcement. The main advantage of continuously reinforced concrete pavement is the elimination of transverse joints which are costly to construct, require periodic resealing, and are often a source of maintenance problems. Continuously reinforced concrete pavements usually provide a very smooth riding surface. A properly designed CRCP will develop random transverse cracks at 2 to 10 feet (0.6 to 3 m) intervals. The resultant pavement is composed of a series of articulated short slabs held tightly together by the longitudinal reinforcing steel. A high degree of shear transfer across the cracks can be achieved because the cracks are held tightly closed.

a. Foundation Support. The reinforcing steel in a CRCP provides continuity of load transfer however good uniform foundation support must still be provided for satisfactory performance. The embankment and subbase requirements given earlier in this Chapter for plain concrete pavements also apply to CRCP.

b. Thickness Design. The thickness requirements for CRCP are the same as plain concrete and are determined from the appropriate design curves, Figures 3-17 through 3-41. Design inputs are the same for concrete strength, foundation strength, aircraft weight and departure level.

c. Longitudinal Steel Design. The design of steel reinforcement for CRCP is critical to providing a satisfactory pavement. The steel percentage must be properly selected to provide optimum crack spacing and crack width. Crack widths must be small to provide a high degree of shear transfer across the crack and to prevent the ingress of water through the crack. The design of longitudinal steel reinforcement must satisfy three conditions. The maximum steel percentage determined by any of the three following requirements should be selected as the design value. In no case should the longitudinal steel percentage be less than 0.5 percent.

(1) Steel to Resist Subgrade Restraint. The longitudinal steel reinforcement required to resist the forces generated by the frictional restraint between the CRCP and the subbase should be determined by using the nomograph shown on Figure 3-45. Use of the nomograph requires three parameters: allowable working stress for steel, tensile strength of concrete and a friction factor for the subbase. The recommended working stress for steel is 75 percent of the specified minimum yield strength. The tensile strength of concrete may be estimated as 67 percent of the flexural strength. The recommended friction factor for stabilized subbase is 1.8. While not recommended as subbase for CRCP, friction factors for unbound fine-grained soils and coarse-grained soils are usually assumed to be 1.0 and 1.5 respectively.

(2) Steel to Resist Temperature Effects. The longitudinal steel reinforcement must be capable of withstanding the forces generated by the expansion and contraction of the pavement due to temperature changes. The following formula is used to compute the temperature reinforcement requirements.

$$P_s = \frac{50 f_t}{f_s - 195T}$$

where:

P_s = steel reinforcement in percent
f_t = tensile strength of concrete
f_s = working stress for steel usually taken as 75% of specified minimum yield strength
T = maximum seasonal temperature differential for pavement in degrees Fahrenheit

Reinforcing steel should be specified on the basis of minimum yield strength. All deformed reinforcing steel bars should conform to ASTM A 615, A 616 or A 617. Deformed welded wire fabric should conform to ASTM A 497.

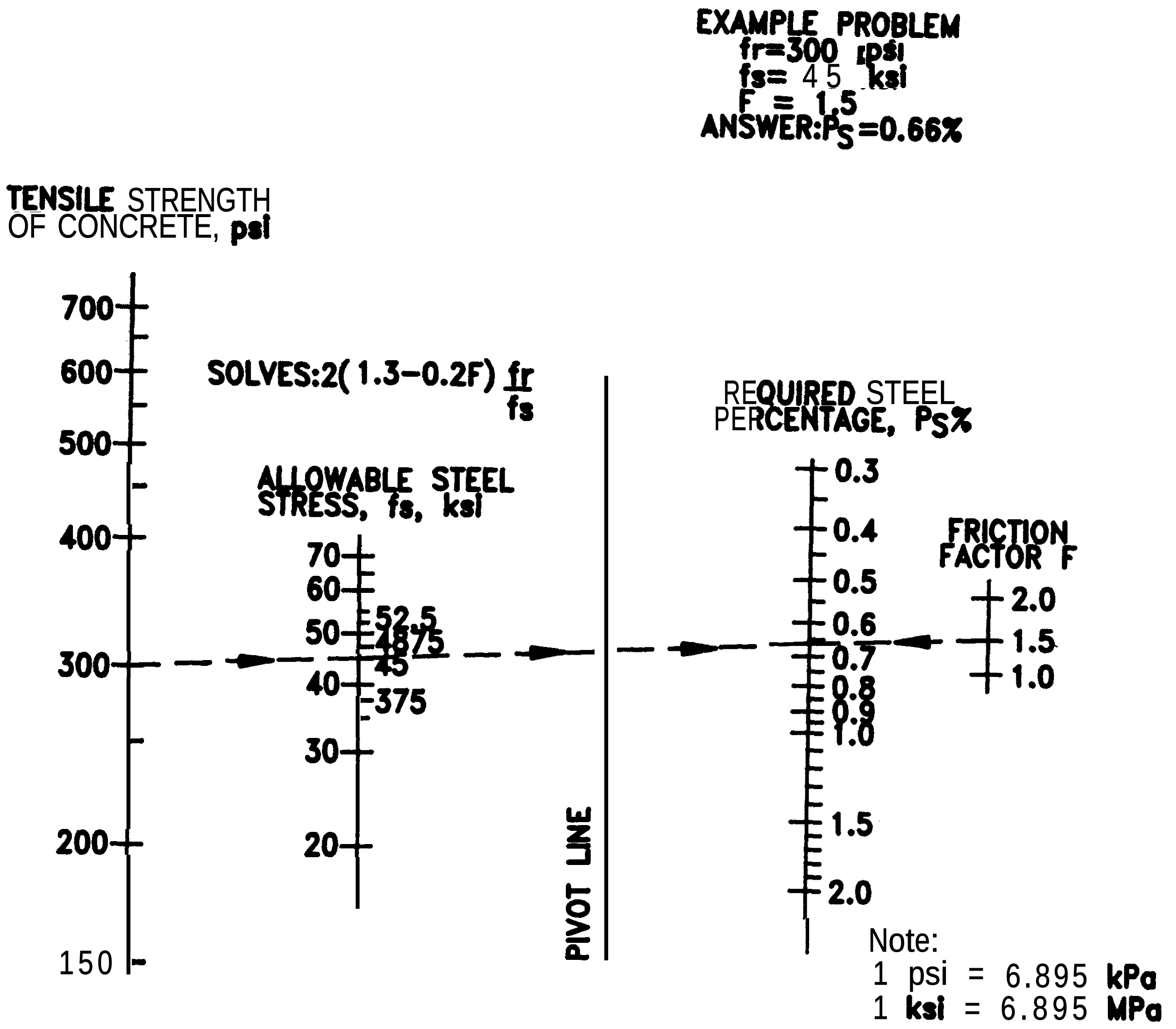


FIGURE 3-45. CONTINUOUSLY REINFORCING CONCRETE PAVEMENT - LONGITUDINAL STEEL REINFORCEMENT

(3) **Concrete to Steel Strength Ratio.** The third consideration in selecting the amount of longitudinal steel reinforcement is the ratio of concrete tensile strength to the specified minimum yield strength of steel. The steel percentage is obtained by multiplying the ratio of the concrete strength to the yield strength of steel by 100.

$$P_s = \frac{100 f_t}{f_Y}$$

where:
P_s = steel reinforcement in percent
f_t = tensile strength of concrete
f_y = minimum yield strength of steel

d. **Transverse Steel Design.** Transverse steel reinforcement is recommended for CRC airport pavements to control "chance" longitudinal cracks which sometimes form. It is also aids in construction by supporting and maintaining longitudinal steel reinforcement spacing. A nomograph for determining transverse steel requirements is shown in Figure 3-46.

e. **Steel Detailing.** Longitudinal steel reinforcement should be located at mid depth of the slab or slightly above. Transverse steel may be located either above or below the longitudinal steel. A minimum concrete cover of 3 inches (75 mm) should be maintained over all steel reinforcement, Longitudinal steel spacing should be 6 to 12 inches (150 to 310 mm). Transverse steel should be spaced at 12 inches (310 mm) or greater. The recommended overlap for splicing of reinforcing bars is 25 diameters of 16 inches (405 mm), whichever is greater. The recommended overlap for splicing deformed welded wire fabric is 32 diameters or 16 inches (405 mm), whichever is greater. When splicing longitudinal steel bar reinforcing it is recommended that the lap splices be made on a 60 degree skew from centerline or staggered such that not more than 1/3 of the bars are spliced on the same transverse plane.

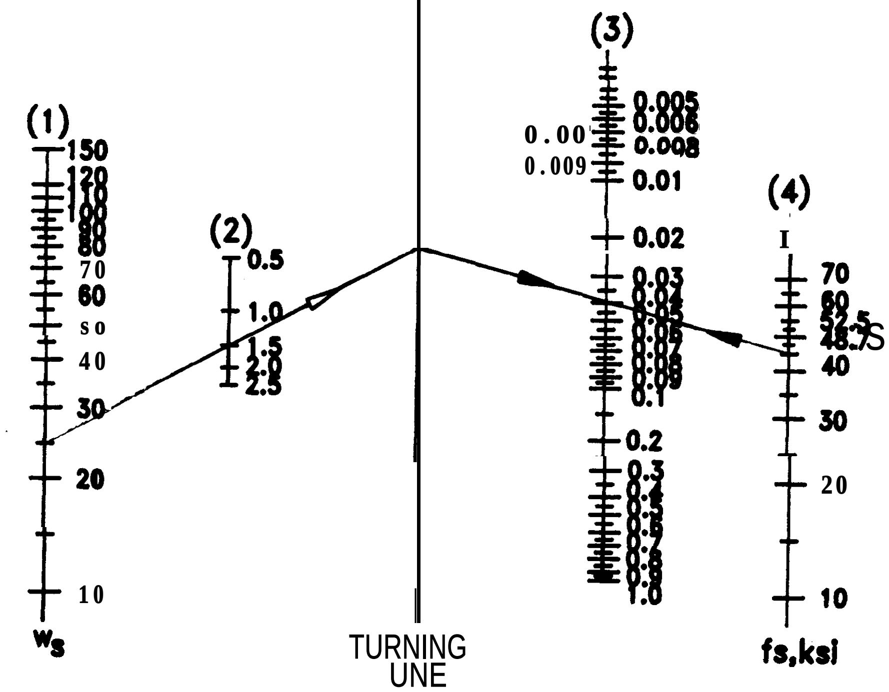
349. **CRCP JOINTING.** Even though transverse contraction joints can be eliminated with CRCP, some joints will be needed to accommodate construction and to control warping stresses. The two types of joints are discussed below:

a. **Construction Joints.** Two types of construction joints are necessary for CRCP. Because pavements are constructed in multiple lanes, a longitudinal construction joint is required between lanes. A transverse construction joint must be provided where paving ends and begins, such as at the finish of a day's paving and the start of the next day's paving. Typical construction joint details are shown in Figure 3-47.

b. **Warping Joints.** Warping joints or hinged joints are needed when paving lane width exceeds the recommended maximum longitudinal joint spacings shown in Table 3-l 1. Transverse steel is carried through the joint to provide continuity and positive aggregate interlock across the joint. Since carrying the steel through the joint eliminates any expansion or contraction capacity, the maximum width of tied pavement should not exceed 75 feet (23 m), see paragraph 337.b.(2). Typical warping joint details are shown in Figure 3-47.

350. **CRCP TERMINAL TREATMENT.** Since long slabs of CRCP are constructed with no transverse joints, provisions must be made to either restrain or accommodate end movements wherever the CRCP abuts other pavements or structures. Rather large end movements, up to 2 inches (50 mm), are experienced with CRCP due to thermal expansion and contraction. End movement is normally not a problem except where CRCP abuts another pavement or structure. Experience with highway CRCP shows that attempts to restrain end movement have not been too successful. More favorable results are achieved where end movement is accommodated rather than restrained. Joints designed to accommodate large movements are required where CRCP intersects other pavements or abuts another structures. Failure to do so may result in damage to the CRCP, pavement or other structure. Wide flange beam type joints or "finger" type expansion joints can accommodate the movements. The wide flange beam type joint is recommended due to its relatively lower costs. A sketch of the wide flange beam joint is shown on Figure 3-48.

SOLVES: $P = \frac{W_s F}{2 f_s} \times 100$



EXAMPLE PROBLEM
$w_s = 25$ FT
$F = 1.5$
$f_s = 45{,}000$ psi
ANSWER: $P = 0.04\%$

Note:
1 psi = 6.895 kPa
1 ksi = 6.895 MPa
1 ft = 0.3048 m

WHERE:
P_s = REQUIRED STEEL PERCENTAGE, %
W_s = WIDTH OF SLAB, FEET
F = FRICTION FACTOR OF SUBGRADE, SUBBASE, OR STRESS-RELEIVING LAYER
f_s = ALLOWABLE WORKING STRESS IN STEEL, PSI (0.75 OF YIELD STRENGTH RECOMMENDED, THE EQUIVALENT OF SAFETY FACTOR OF 1.33)

FIGURE 3-46. CONTINUOUSLY REINFORCING CONCRETE PAVEMENT - TRANSVERSE STEEL REINFORCEMENT

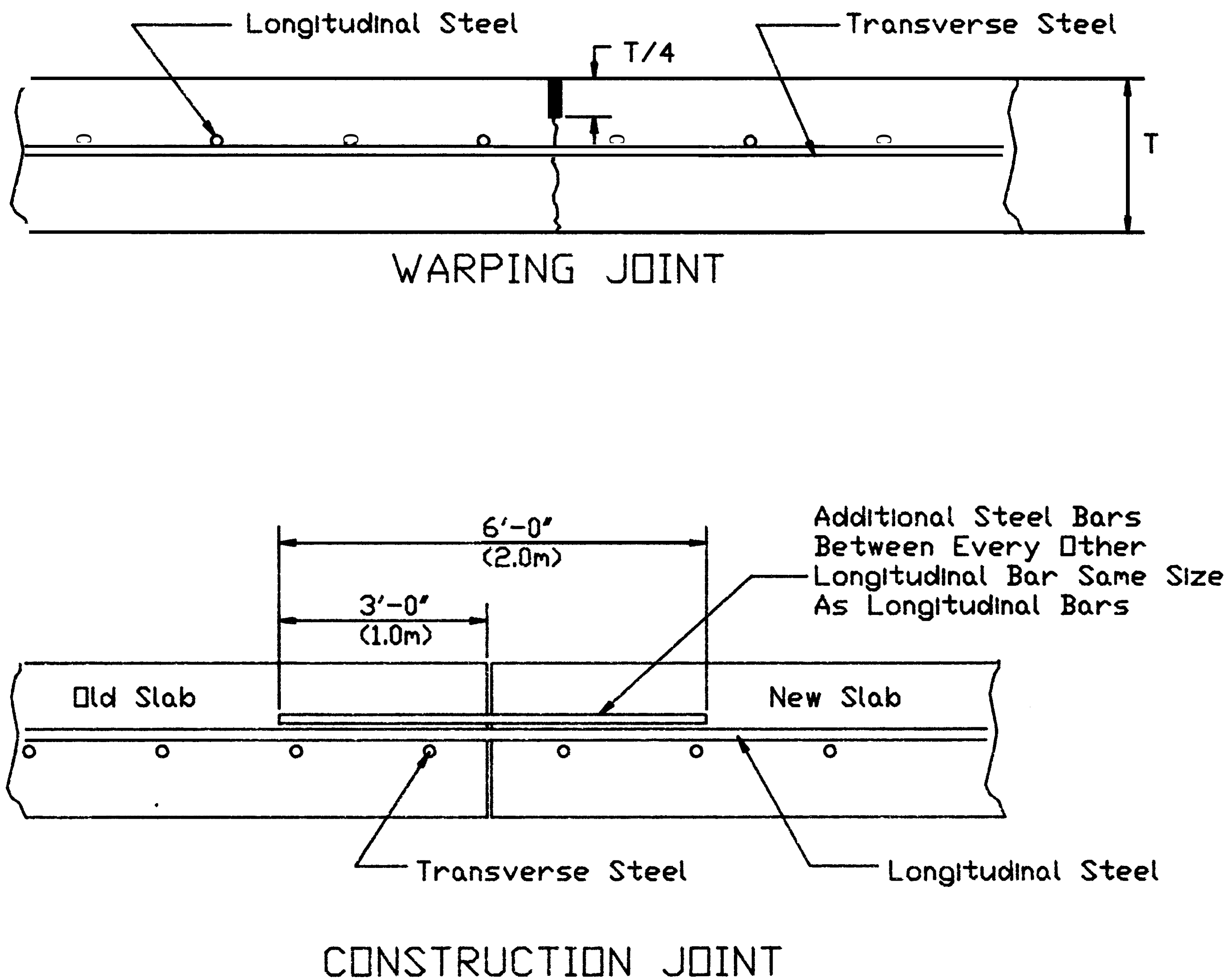
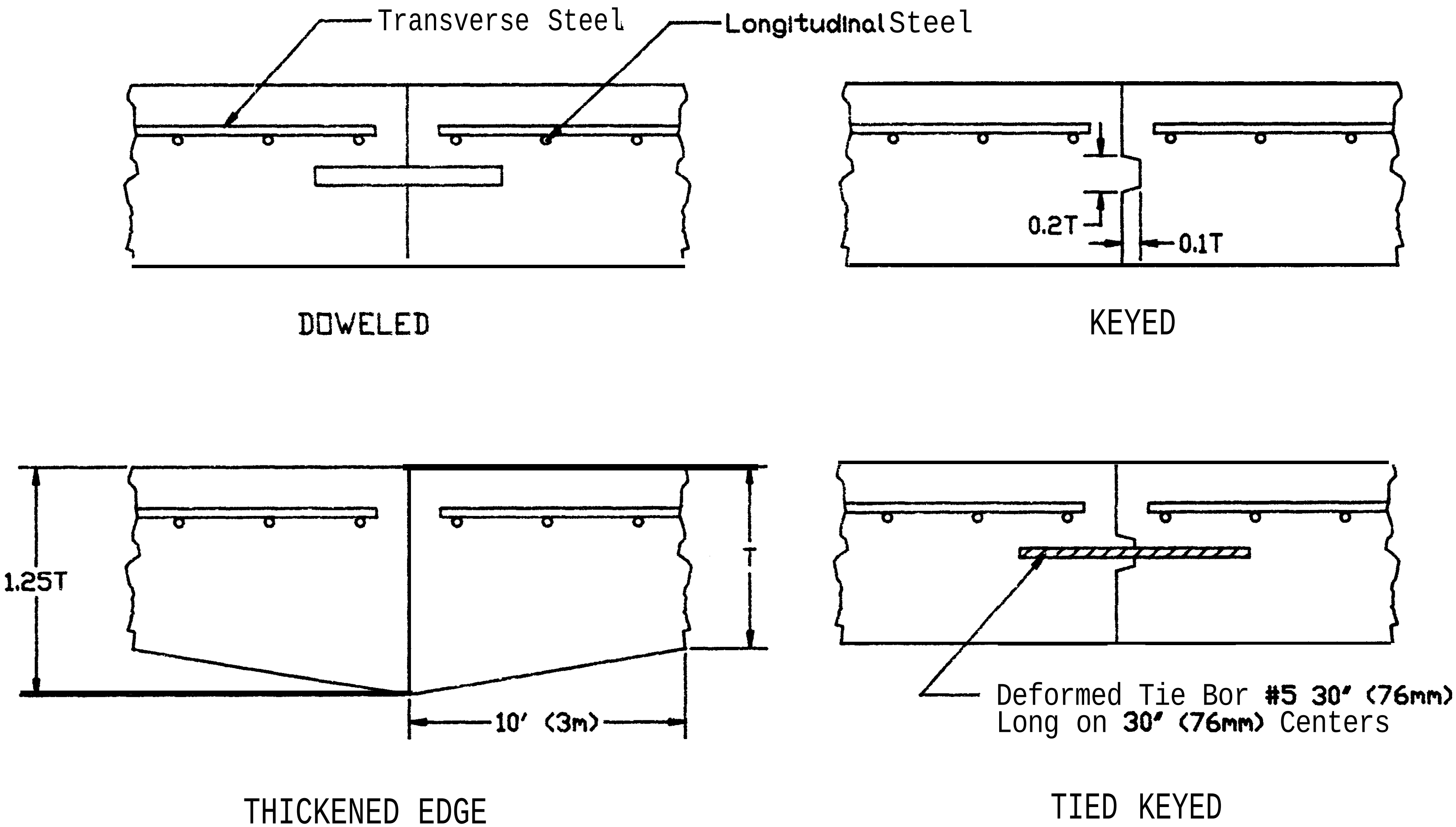
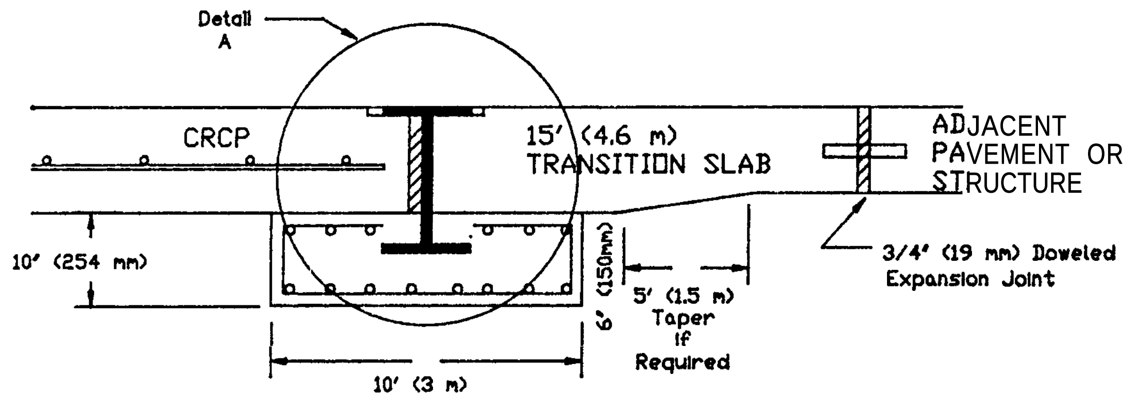


FIGURE 3-47. CONTINUOUSLY REINFORCED CONCRETE PAVEMENT - JOINTING DETAILS



LONGITUDINAL CONSTRUCTION JOINTS IN CRCP

FIGURE 3-47. CONTINUOUSLY REINFORCED CONCRETE PAVEMENT - JOINTING DETAILS



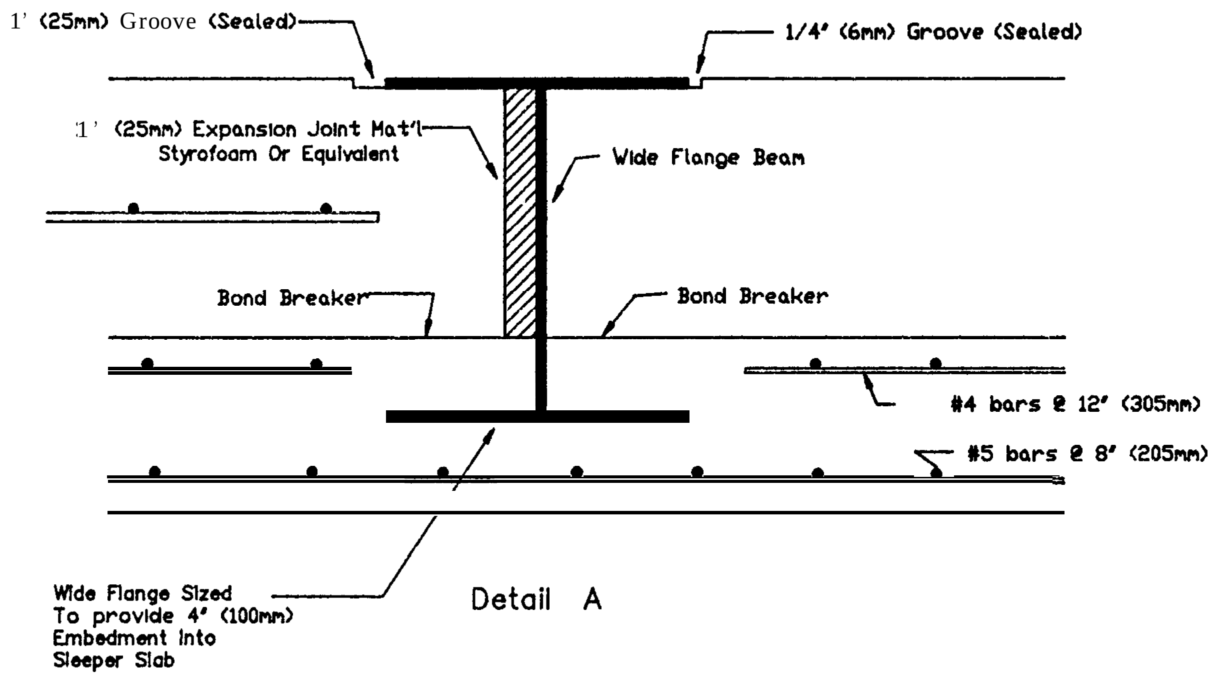


FIGURE 3-48. CONTINUOUSLY REINFORCED CONCRETE PAVEMENT - WIDE FLANGE BEAM TERMINAL JOINT

351. CRCP DESIGN EXAMPLE. An example design for CRCP is given below. Assume a CRCP is to be designed to serve the following conditions:

a. **Design Aircraft -** DC-lo-10 with a gross weight of 400,000 lbs (182,000 kg)

b. **Foundation Modulus - 400** pci (109 MN/m^3)

c. **Concrete Flexural Strength -** 600 psi (4.2 MPa)

d. **Annual Departures - 3000**

e. **Minimum Specified Yield Strength of Steel -** 60,000 psi (414 MPa) (Longitudinal and Transverse)

f. **Paving Lane Width - 25** feet (7.6 m)

g. **Cement Stabilized Subbase**

h. **Seasonal Temperature Differential -** 100°F (38°C)

(1) **Slab Thickness.** Enter the design curve for DC-1010 aircraft, Figure 3-33, with the parameters assumed above and read a pavement thickness of 12.2 inches (310 mm). This thickness would be rounded down to the next full inch or 12.0 inches (305 mm).

(2) **Steel Design.** The longitudinal reinforcing steel would be determined as described in paragraph c above:

(i) **Subgrade Restraint.** Using the nomograph in Figure 3-45 the longitudinal steel required to with the forces generated by subgrade restraint is found to be 0.83 percent. With the following inputs:
Working stress = 75% x 60,000 = 45,000 psi (310 MPa)
Friction factor = 1.8
Tensile strength of concrete = 67% of 600 = 400 psi (2.8 MPa)

(ii) **Temperature Effects.** The steel required to withstand the forces generated by seasonal temperature changes is computed using the formula given in paragraph 348.c.(2).

$$Ps = \frac{50\times}{45{,}000-195\times} = 0.78\%$$

(iiij **Concrete to Steel Strength Ratio.** The strength ratio between the concrete and steel is computed by the procedure given in paragraph 348.c.(3).

$$Ps = \frac{400\times}{60{,}000} = 0.67\%$$

(iv) **Transverse Steel.** The transverse reinforcing steel percentage would be determined using Figure 3-46. This will yield a transverse steel requirement of 0.055%

(v) **Final Design.** The final design would be a 12 inch (305 mm) thick concrete slab. Since the steel percentage necessary to satisfy the subgrade restraint condition is the largest steel percentage for longitudinal reinforcement, the value of 0.83 percent would be selected for design. The transverse steel requirement is 0.055%. The longitudinal steel requirement can be satisfied by using #7 reinforcing bars spaced at 6 inches (150 mm). The transverse steel requirement can be met by using #4 bars on 30 inch (760 mm) centers.

352. **PRESTRESSED CONCRETE PAVEMENT.** Prestressed concrete pavements have been used in airport applications in Europe and to a limited extent in the United States. Prestressed concrete airport pavements are usually post-tensioned with high strength steel strands. These pavements are usually considerably thinner than plain, jointed reinforced, or continuously reinforced concrete pavements yet provide high load carrying capacity. Slab lengths on the order of 400 to 500 feet (120 to 150 m) are generally used. A design procedure for prestressed airport pavements was developed under an FAA research effort and is reported in Research Report Number FAA-RD-74-34, Volume II. Use of prestressed concrete airport pavements on Federally assisted projects will require FAA approval on a case by case basis.

CHAPTER 4. AIRPORT PAVEMENT OVERLAYS AND RECONSTRUCTION

400. **GENERAL.** Airport pavement overlays or reconstruction may be required for a variety of reasons. A pavement may require an overlay or reconstruction because the original pavement has served its design life and it is simply "worn out." A pavement may have been damaged by overloading in such a way that it cannot be economically maintained at a serviceable level. Similarly, a pavement in good condition may require strengthening to serve heavier aircraft than those for which the pavement was originally designed. Generally, airport pavement overlays consist of either portland cement concrete or hot mix asphalt concrete. Techniques and equipment are now available to recycle old pavement materials into reconstructed sections. Pavements which are severely distressed in the center portions can sometimes be economically reconstructed by building a keel section using recycled materials. Use of this method of reconstruction is essentially the same as building a new pavement.

401. **CONDITION OF EXISTING PAVEMENT.** Assessment of the condition of the existing pavement is one of the most important and difficult steps in design of a reconstruction or overlay project. Determination of the properties of the existing pavement should include the thickness, condition and strength of each layer, the subgrade soil classification, and some estimate of foundation strength (CBR or subgrade modulus). An assessment of the structural integrity of the existing pavement is necessary. Failed areas in the existing pavement should be carefully studied to determine the probable cause of failure. Subsurface drainage conditions should be assessed carefully and corrected if found to be deficient. In some instances subsurface drainage corrections are best performed through reconstruction. Overlaying an existing pavement without correcting poor subsurface drainage will usually result in poor overlay performance. A valuable technique for assessing the condition of the existing pavement is nondestructive pavement testing (NDT). See Appendix 3. NDT can be used to estimate foundation strength, measure joint condition, and possibly detect voids in existing pavements.

402. **MATERIAL SELECTION CONSIDERATIONS.** Criteria are presented in this circular for both hot mix asphalt and concrete reconstruction or overlays. The selection of the material type should be made after careful consideration of many factors. The designer should consider the total life cycle cost of the reconstructed or overlay pavement. (see DOT/FAA/RD-8 l/78, Appendix 4). Life cycle costs should include initial construction and maintenance costs over the design life of the pavement. Other considerations such as allowable down time of the pavement and availability of alternate pavements to use during construction will have a significant impact on the material selected.

403. **OVERLAY DESIGN.** The remainder of this chapter is devoted to the design of overlay pavements. As previously mentioned, the design of reconstructed pavements is essentially the same as for new construction.

a. **Typical Overlay Cross Sections and Definitions.** Typical overlay pavement cross sections are shown in Figure 4-l. Definitions applicable to overlay pavements are as follows:

(1) **Overlay Pavement.** Pavement which is constructed on top of an existing pavement.
(2) **Hot Mix Asphalt Overlay.** Hot mix asphalt pavement placed on an existing pavement.
(3) **Concrete Overlay.** Portland cement concrete pavement placed on an existing pavement.
(4) **Sandwich Pavement.** Overlay pavement sections containing granular separation courses between the old and new impervious surfaces are called sandwich pavements.

b. **Sandwich Pavements.** Regardless of the type of overlay, FAA criteria does not permit the construction of sandwich overlay pavements. They are not allowed because the granular separation course usually becomes saturated with water and provides poor or, at best, unpredictable performance. Saturation of the separation course can be caused by the infiltration of surface water, ingress of ground or capillary water, or the condensation of water from the atmosphere. In any event, the water in the separation course usually cannot beadequatelydrained.Thetrapped water drastically reduces the stability of the overlay.

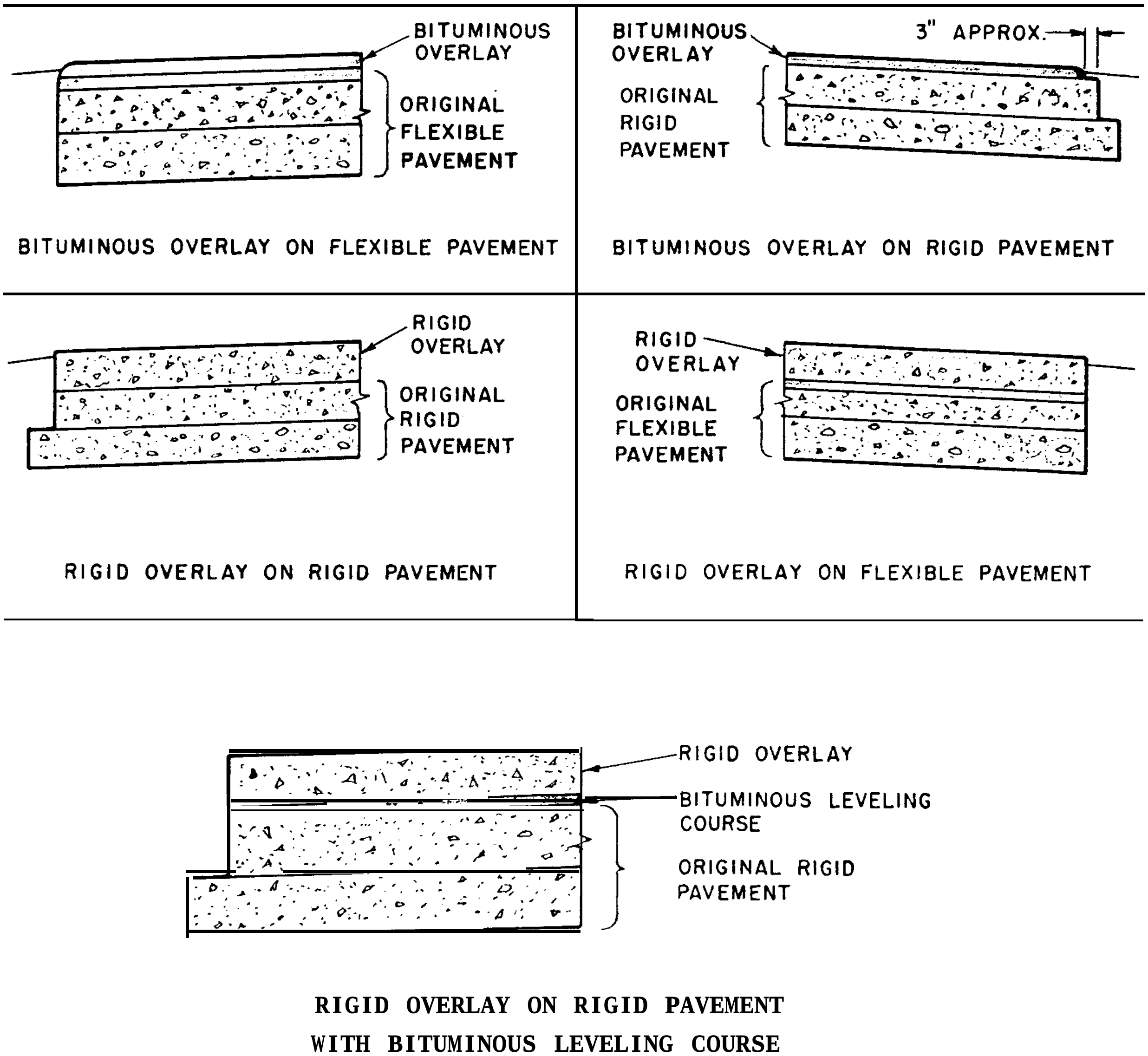


FIGURE 4-1. TYPICAL OVERLAY PAVEMENTS

404. DESIGN OF STRUCTURAL HOT MIX ASPHALT OVERLAYS. Structural hot mix asphalt overlays can be applied to either flexible or rigid pavements. Certain criteria and design assumptions are different for hot mix asphalt overlays of flexible and rigid pavements. The design for procedures are presented separately.

405. HOT MIX ASPHALT OVERLAYS ON EXISTING FLEXIBLE PAVEMENT. The design of structural hot mix asphalt overlays of existing flexible pavements is based on a thickness deficiency approach. That is, the existing pavement is compared to what is needed for a new pavement and any deficiency is made up in the overlay.

a. Calculate New Pavement Requirements. Using the appropriate flexible pavement design curves (Figures 3-2 through 3-15) calculate the thickness requirements for a flexible pavement for the desired load and number of equivalent design departures. A CBR value is required for the subgrade material and subbase. Thicknesses of all pavement layers must be determined.

b. Compare New Pavement Requirements With Existing Pavement. The thickness requirements for a new pavement are compared with the existing pavement to determine the overlay requirements. Adjustments to the various layers of the existing pavement may be necessary to complete the design. This is particularly difficult when overlaying old pavement. Hot mix asphalt surfacing may have to be converted to base, and/or base converted to subbase. Note that a high quality material may be converted to a lower quality material, such as: surfacing to base, or base to subbase. A lesser quality material may not be converted to a higher quality material. For example, excess subbase cannot be converted to base. The equivalency factors shown in Tables 3-6 through 3-8 may be used as guidance in the conversion of layers. It must be recognized that the values shown in Tables 3-6 through 3-8 are for new materials and the assignment of factors for existing pavements must be based on judgment and experience. Surface cracking, high degree of oxidation, evidence of low stability, etc., are a few of the considerations which would tend to reduce the equivalency factor. Any hot mix asphalt layer located between granular courses in the existing pavement should be evaluated inch for inch as granular base or subbase course.

c. Example. To illustrate the procedure of designing a hot mix asphalt overlay, assume an existing taxiway pavement composed of the following section. The subgrade CBR is 7, the hot mix asphalt surface course is 4 inches (100 mm) thick, the base course is 6 inches (150 mm) thick, the subbase is 10 inches (250 mm) thick, and the subbase CBR is 15. Frost action is negligible. Assume the existing pavement is to be strengthened to accommodate **a** dual wheel aircraft weighing 100,000 pounds (45 000 kg) and an annual departure level of 3,000. The flexible pavement required (referring to Figure 3-3) for these conditions is:

Hot mix asphalt Surface	4 inches (100 mm)
Base	9 inches (230 mm)
Subbase	10 inches (250 mm)
Total pavement thickness	23 inches (585 mm)

The total pavement thickness must be 23 inches (585 mm) in order to protect the CBR 7 subgrade. The combined thicknesses of surfacing and base must be 13 inches (330 mm) to protect the CBR 15 subbase. The existing pavement is 3 inches (75 mm) deficient in total pavement thickness. All of the thickness deficiency is in the base course. For the sake of illustration, assume the existing hot mix asphalt surface is in such a condition that surfacing can be substituted for base at an equivalency ratio of 1.3 to 1. Converting 2.5 inches (64 mm) of surfacing to base yields a base course thickness of 9.2 inches (234 mm) leaving 1.5 inches (40 mm) of unconverted surfacing. A 2.5 inch (64 mm) overlay would be required to achieve a 4 inch (100 mm) thick surface.

d. Summary. Structurally, a 2.5 inch thick overlay should satisfy the design conditions. The overlay thickness calculated from structural considerations should be compared with that required to satisfy geometric requirements. Geometric requirements include, for example, provision of drainage, correcting crown and grade, meeting grade of other adjacent pavements and structures, etc. The most difficult part of designing hot mix asphalt overlays for flexible pavements is the determination of the properties of the existing pavement. Subgrade and subbase CBR values can be determined by conducting field inplace CBR tests. Field CBR tests should be performed in accordance with the procedures given in Manual Series No. 10 (MS-IO by The Asphalt Institute. See Appendix 4. The subgrade and subbase must be at the equilibrium moisture content when field CBR tests are conducted. Normally a pavement which has been in place for at least 3 years will be in equilibrium. Procedures for calculating CBR values from NDT tests are also available. Layer conversions, i.e., converting base to subbase, etc., are largely a matter of engineering judgment. When performing the conversions, it is recommended that any converted thicknesses not be rounded off.

406. **HOT MIX ASPHALT OVERLAY ON EXISTING RIGID PAVEMENT.** The design of a hot mix asphalt overlay on an existing rigid pavement is also based on a thickness deficiency approach. However, new pavement thickness requirements for rigid pavements are used to compare with the existing rigid pavement. The formula for computing overlay thickness is as follows:

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

where:

t = thickness of hot mix asphalt overlay, inches (mm).

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

h_d = thickness of new rigid pavement required for design conditions, inches (mm). Use the exact value for h_d; do not round off. In calculating h_d use the k value of the existing foundation and the flexural strength of the existing concrete as design parameters.

C_b = a condition factor which indicates the structural integrity of the existing rigid pavement. Value ranges from 1 .O to 0.75.

h_e = thickness of existing rigid pavement, inches (mm).

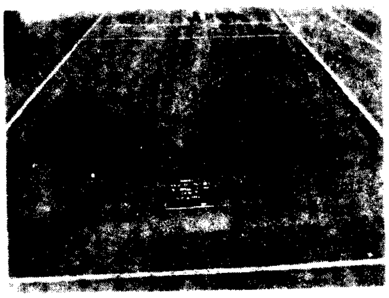
a. F Factor. The "F" factor is an empirical method of controlling the amount of cracking which will occur in the rigid pavement beneath the hot mix asphalt overlay. It is a function of the amount of traffic and the foundation strength. The assumed failure mode for a hot mix asphalt overlay on an existing rigid pavement is that the underlying rigid pavement cracks progressively under traffic until the average size of the slab pieces reaches a critical value. Further traffic beyond this point results in shear failures within the foundation producing a drastic increase in deflections. Since high strength foundations can better resist deflection and shear failure, the F factor is a function of subgrade strength as well as traffic volume. Photographs of various overlay and base pavements shown in Figure 4-2 illustrate the meaning of the "F" factor. Figures 4-2 a, b, and c show how the overlay and base pavements fail as more traffic is applied to a hot mix asphalt overlay on an existing rigid pavement. Normally an F factor of 1 .00 is recommended unless the existing pavement is in quite good condition, see paragraph 406b.(1) below. Figure 4-3 is a graph which should be used to determine the appropriate F factor for pavements in good condition.

b. C_b Factor. The condition factor "C_b" applies to the existing rigid pavement. The "C_b" factor is an assessment of the structural integrity of the existing pavement.

(1) Selection of C_b Factor. The overlay formula is rather sensitive to the "C_b" value. A great deal of care and judgment are necessary to establish the appropriate "C_b". NDT can be a valuable tool in determining an proper value. A "C_b" value of 1.0 should be used when the existing slabs contain nominal structural cracking and 0.75 when the slabs contain structural cracking. The designer is cautioned that the range of "C_b" values used in hot mix asphalt overlay designs is different from the "C_r" values used in rigid overlay pavement design. A comparison of "C_b" and "C_r" and the recommended "F" factor to be used for design is shown below:

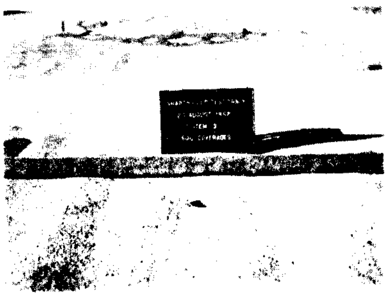
C_b	C_r	Recommended F factor
0.35 to 0.50	0.75 to 0.80	1.00
0.5 1 to 0.75	0.81 to 0.90	1 .00
0.76 to 0.85	0.91 to 0.95	1 .00
0.86 to 1.00	0.96 to 1.00	use Figure 4-3

The minimum "C_b" value is 0.75. A single "C_b" should be established for an entire area. The "C_b" value should not be varied along a pavement feature. Figures 4-4 and 4-5 illustrate "C_b" values of 1.0 and 0.75, respectively.

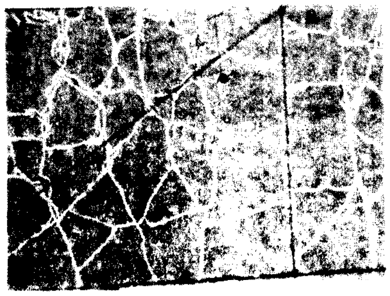


SURFACE OF OVERLAY

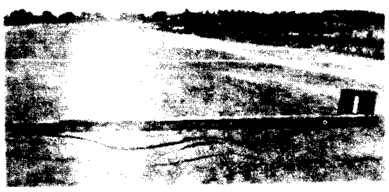
BASE PAVEMENT



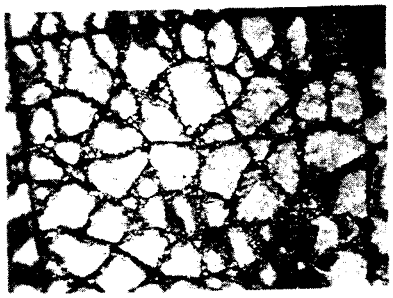
SURFACE OF OVERLAY



BASE PAVEMENT



SURFACE OF OVERLAY



BASE PAVEMENT

FIGURE 4-2. ILLUSTRATION OF VARIOUS "F" FACTORS FOR HOT MIX ASPHALT OVERLAY

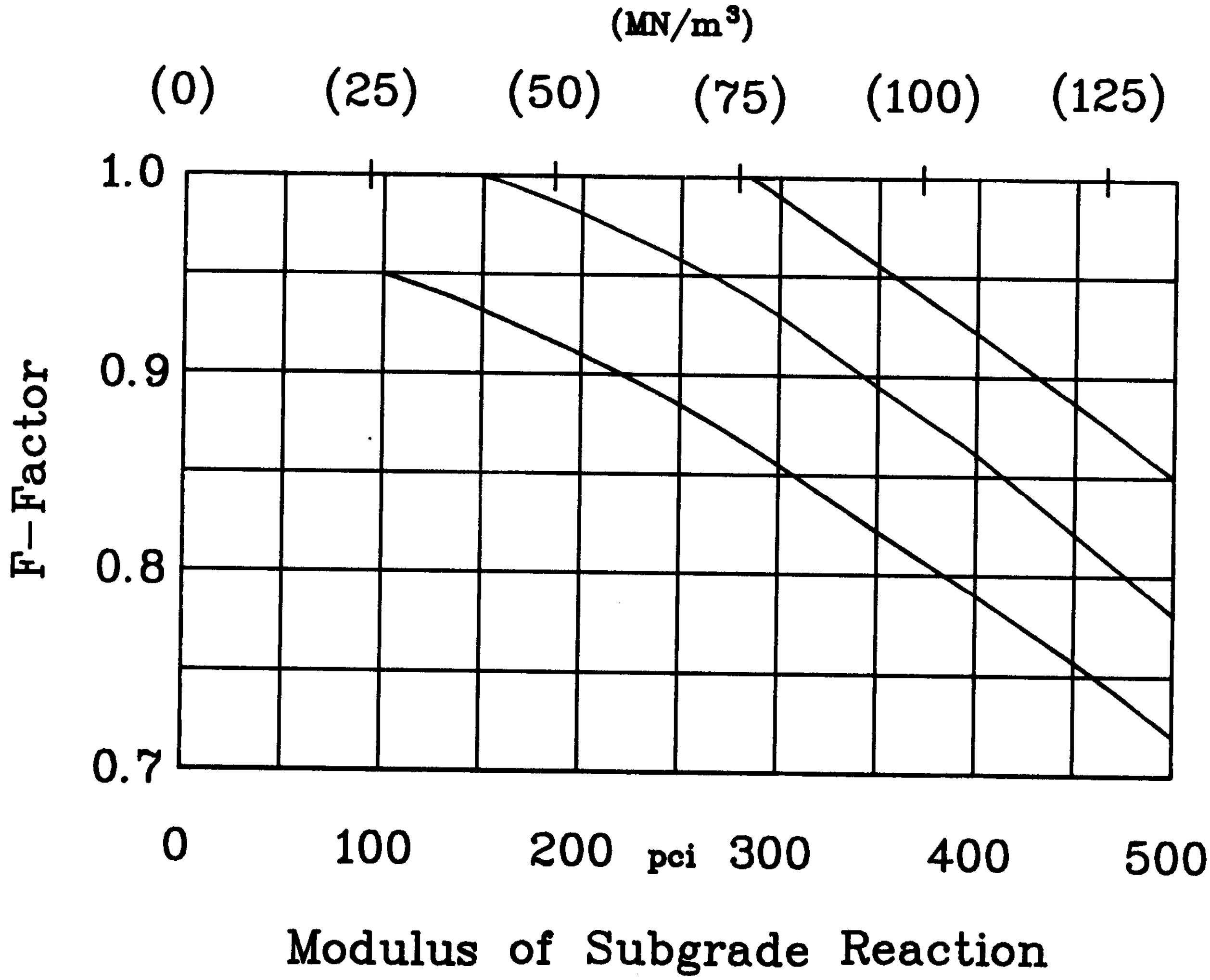
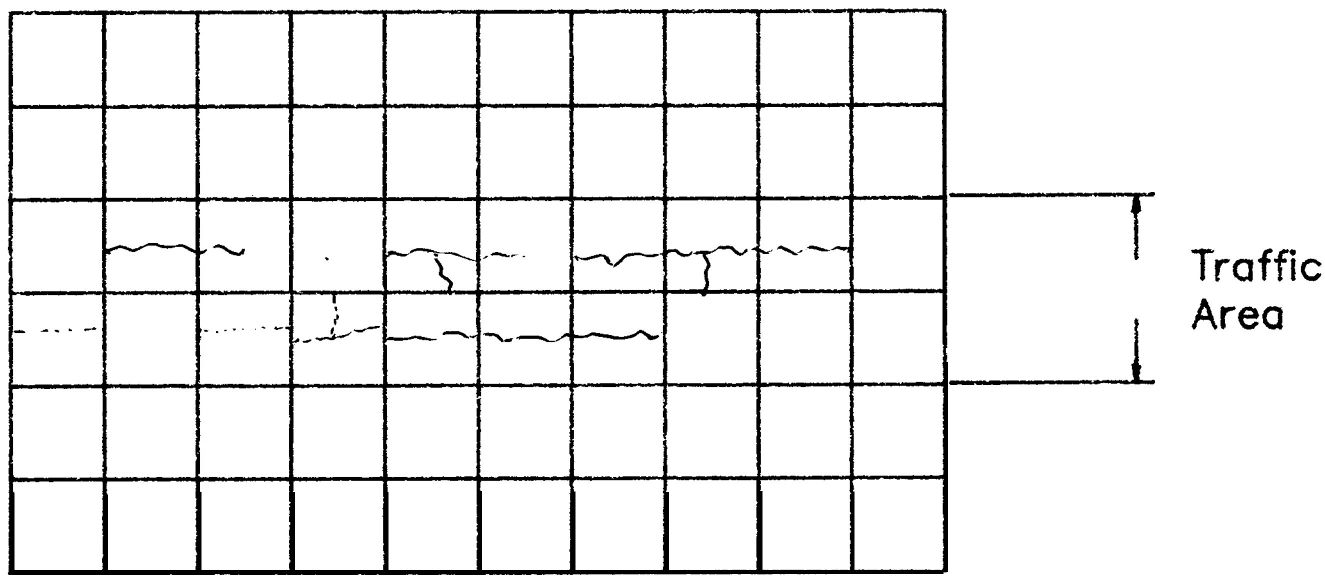


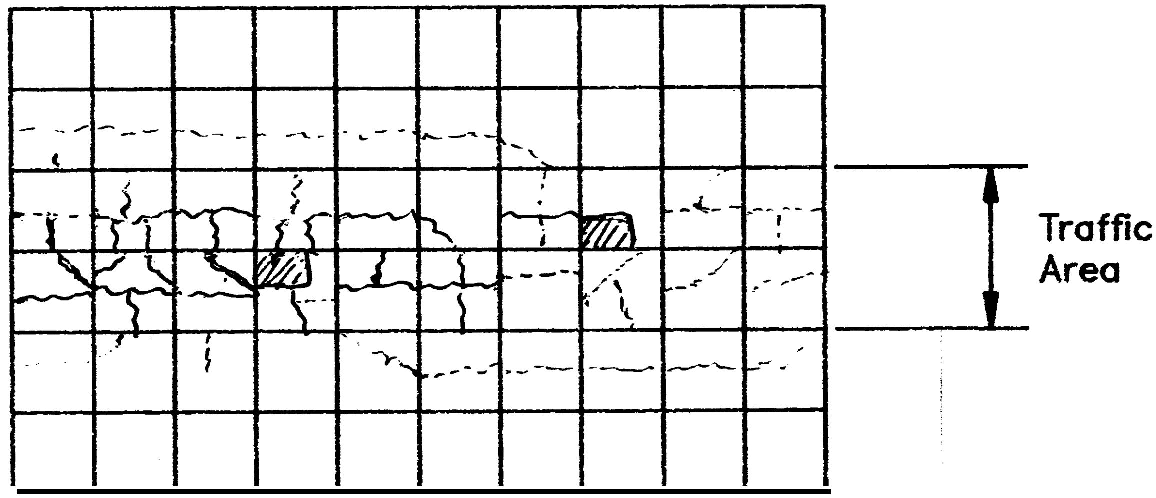
FIGURE 4.3 GRAPH OF "F" FACTOR VS. MODULUS OF **SUBGRADE** REACTION FOR DIFFERENT TRAFFIC LEVELS



Legend:
Crack Width
- - - - Less Than 1/4 inch (6mm)
——— Greater Than 1/4 inch (6mm)

Note:
50% of Slabs Within Traffic Area Broken Into 2 to 3 Pieces. No Working Cracks, Corner Breaks, or Faulted Joints.

FIGURE 4.4 ILLUSTRATION OF A "C_b" FACTOR OF 1.0 FOR HOT MIX ASPHALT OVERLAY DESIGN



Legend:

Crack Width

- - - -Less Than 1/4 inch 6mm)

———Greater Than 14 inch (6mm)

Patch

Note:

Within Traffic Area, 50% of Slabs Broken Into 6 Pieces, 50% of Slabs Broken Into 2 or More Pieces. Corner Breaks, Working Cracks, Spalling, and/or Faulted Joints and Patching Are Evident

FIGURE 4-5. ILLUSTRATION OF A "C_b" FACTOR OF 0.75 FOR HOT MIX ASPHALT OVERLAY DESIGN