PRESTRESSED PAVEMENT
VOL. 1, JOINT DESIGN

U.S. Department
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Research, Development,
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**Final Report
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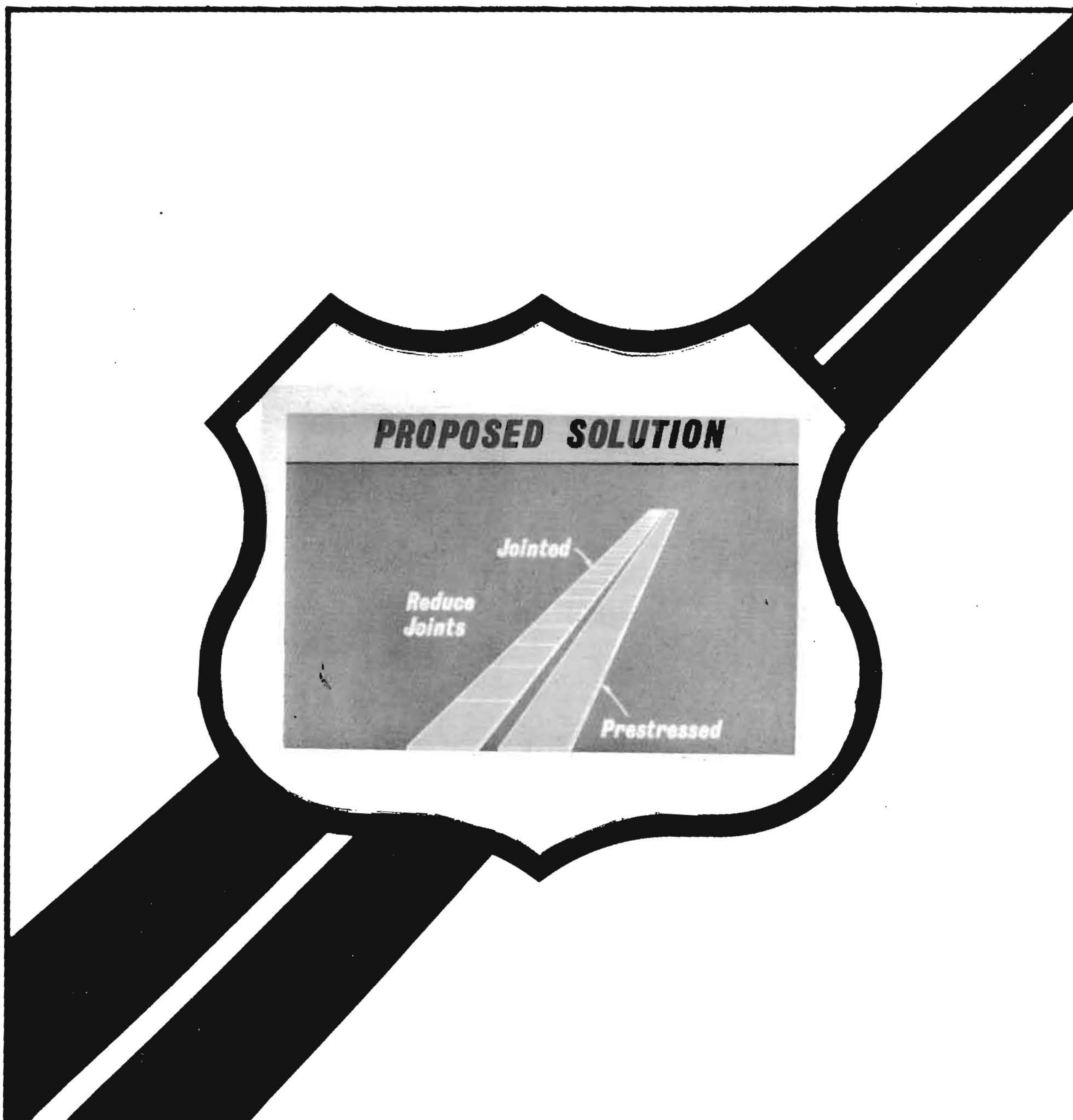

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FOREWORD

Highway administrators and engineers must select among alternative pavement investment and maintenance strategies. These decisions should be based upon economic analyses of the impacts expected for each pavement management strategy. Prestressed pavements offer an alternative type of strategy because of thinner slabs, fewer joints, and an expected reduction in maintenance costs.

In Volume 1, four types of transverse joints in prestressed pavements are presented. The other volumes are:

- FHWA/RD-82/091, Volume 2, "Thickness Design"
- FHWA/RD-82/092, Volume 3, "Construction Manual"
- FHWA/RD-82/115, Volume 4, "Prestressed Pavement Accelerated Testing Program" (available only from NTIS)
- FHWA/RD-82/114, Volume 5, "Evaluation of Innovative Concepts Relating to Prestressed Concrete Pavements" (available only from NTIS).

Richard E. Hay, Director
Office of Engineering
and Highway Operations
Research and Development

NOTICE

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16. Abstract

This report presents four transverse joint designs for Prestressed Concrete Pavements. Designs I, II, and III are for an 8-in (203 mm) thick pavement with main slab lengths of 350 ft (107 m). A short prestressed gap slab and a single active joint is used between main slabs.

Design IV is for a 7-in (178 mm) thick pavement with main slab lengths of 250 ft (76 m). A tied concrete shoulder is used. Gap slabs are 10 in (250 mm) thick and are conventionally reinforced. An active joint is provided at each end of the gap slab.

Design calculations are presented in Appendix A and results of laboratory tests are presented in Appendix B.

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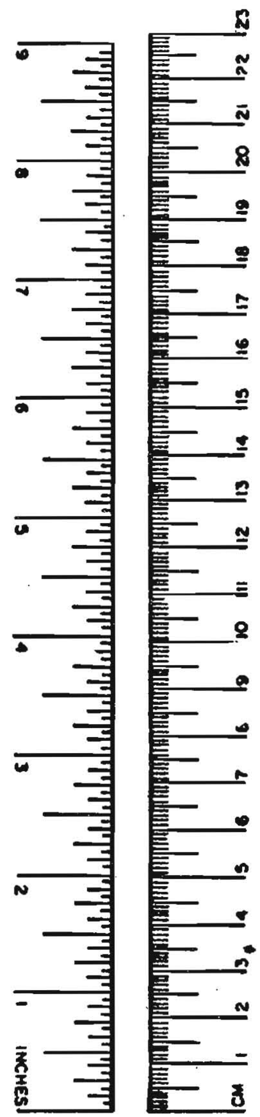
METRIC CONVERSION FACTORS

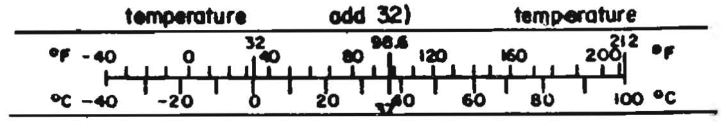
APPROXIMATE CONVERSIONS FROM METRIC MEASURES

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
in	inches	2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.6	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C



APPROXIMATE CONVERSIONS FROM METRIC MEASURES

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	8.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	36	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



PREFACE

This report has been prepared as part of a contract between FHWA and the Construction Technology Laboratories, Division of the Portland Cement Association.

It is the first of a five-volume series concerning design of prestressed concrete pavements. The series consists of the following reports.

1. Prestressed Pavement Joint Designs
2. Prestressed Pavement Thickness Design
3. Prestressed Pavement Construction Manual
4. Prestressed Pavement Accelerated Testing Program
5. Evaluation of Innovative Concepts Relating to Prestressed Concrete Pavements

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INTRODUCTION

The objective of Federal Highway Administration's Research Project 5E, Premium Pavements for "Zero Maintenance" is to exploit modern materials and technology in developing "Zero Maintenance" pavements for warranted use.

As a portion of this research project, an investigation has been conducted by the Construction Technology Laboratories, a Division of the Portland Cement Association, to develop design and construction techniques for prestressed concrete pavements. This work is presented in reports covering the following:

(1) Transverse joint design
(2) Thickness design procedure
(3) Construction techniques
(4) Accelerated testing program
(5) Laboratory studies

Prestressed pavement design includes determination of required pavement thickness and joint hardware selection based on anticipated slab movement and length. Figure 1 shows basic steps involved in prestressed pavement design.

As shown in Figure 1, prestressed pavement design process is iterative and involves interaction of many factors. The process starts with selection of an initial slab thickness. Then, under joint design, trial main slab length and prestress tendon size, spacing, and force are selected. Effective mid-slab prestress is computed. A minimum of about 50 psi mid-slab prestress should be obtained. If not obtained, slab length, tendon size, spacing or force is varied until the desired mid-slab prestress is obtained.

Once the mid-slab prestress criteria is met, anticipated maximum joint movement is computed. The selection of an infiltration prevention device such as a strip seal, compression seal, or steel cover plate depends on the magnitude of joint movement. Slab length is varied until the computed joint movement can be accommodated by the device selected. At this time

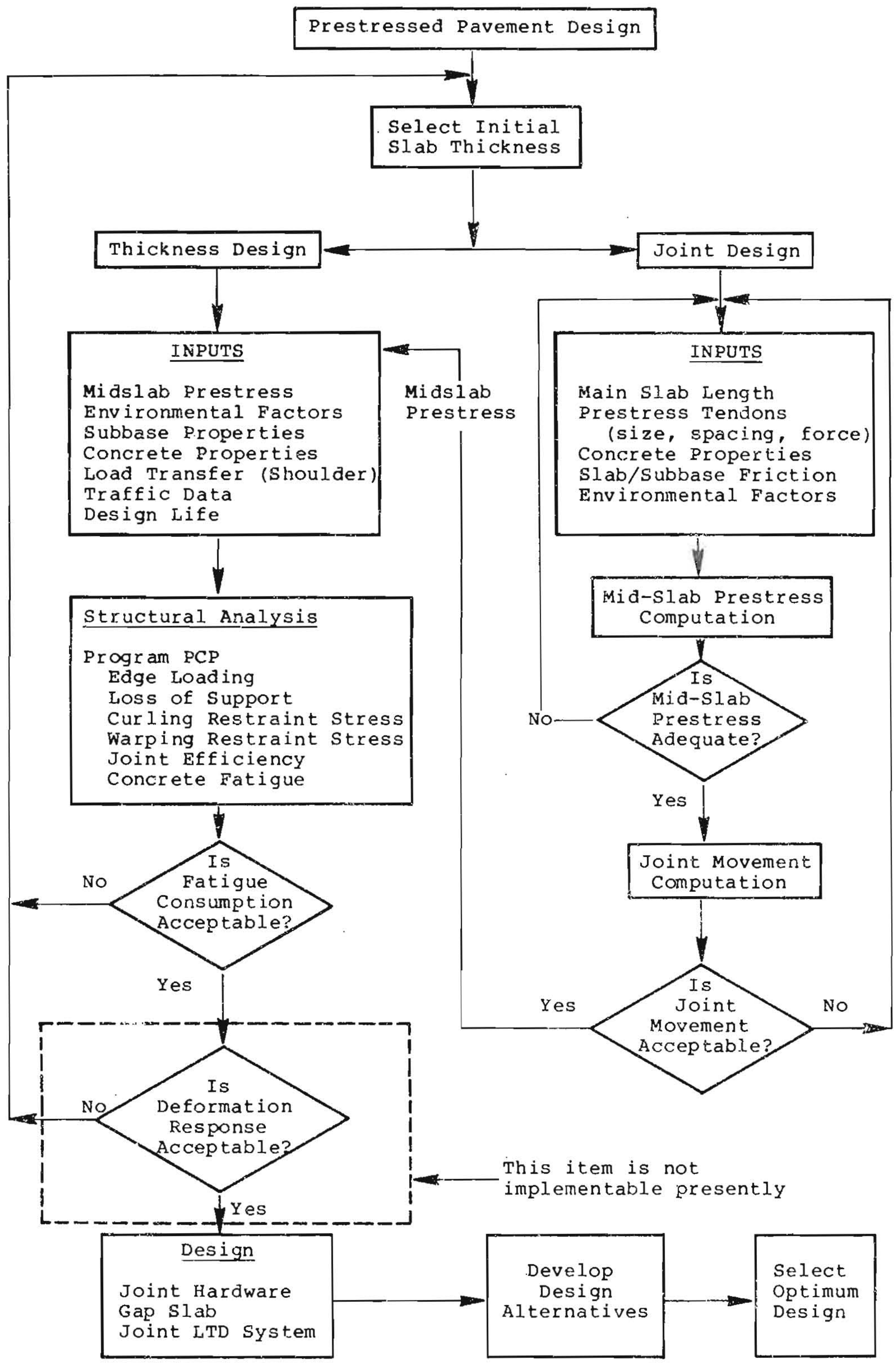


Figure 1 - Prestressed Pavement Design Process

a decision is made regarding use of a single active joint or two active joints between adjacent main slabs.

After an acceptable slab length, mid-slab prestress level, and joint hardware arrangement are established, a structural analysis is performed. The analysis requires the value of effective mid-slab prestress as an input. Alternatively, a minimum level of mid-slab prestress value may be assumed in the structural analysis. The structural analysis computes fatigue consumption due to edge stresses at mid-slab. If fatigue consumption is more than 100 percent, the design process is repeated using a larger slab thickness.

This report presents designs for transverse joints in prestressed pavements.

JOINT DESIGNS

Joint design for prestressed pavements involves the following steps:

(1) Determination of required pavement thickness

(2) Selection of slab length

(3) Determination of prestressing tendon size, spacing, and force

(4) Detailing of joint hardware

Proper attention must be given to joint hardware design because a large number of items such as anchors, strands, load transfer devices, infiltration prevention devices, reinforcement, and positioning bars are located within a few inches of the joint. Detailing of joint hardware is, therefore, a critical element of design. Detailing requirements also put a limit on the minimum slab thickness that can be practically used for construction.

Thickness design is based on edge loading with consideration given to stresses due to temperature, moisture, edge support loss, and the presence or absence of a tied concrete shoulder. On this basis thickness requirements were 7- and 8-in (178 and 203 mm) for "zero-maintenance" pavement with and without a tied

TABLE 1 - JOINT DESIGN PARAMETERS

Main Slab Thickness, in	8	7
Main Slab Length, ft	350	250
Gap Slab Thickness, in	8	10
Gap Slab Length, in	59	59
Design Condition	free edge	tied shoulder
No. of Active Joints	1	2
Tendon Diameter, in	0.6	0.6
Tendon Spacing, in	18	24
Tendon Load, kips	44	41
End of Slab Prestress, psi	306	244
Midslab Prestress, psi	57	75
Movement at Each Joint, in	3.3	1.2

1 in = 25.4 mm
1 ft = 0.30 m
1 kip = 4.448 kN
1 psi = 6.894 kPa

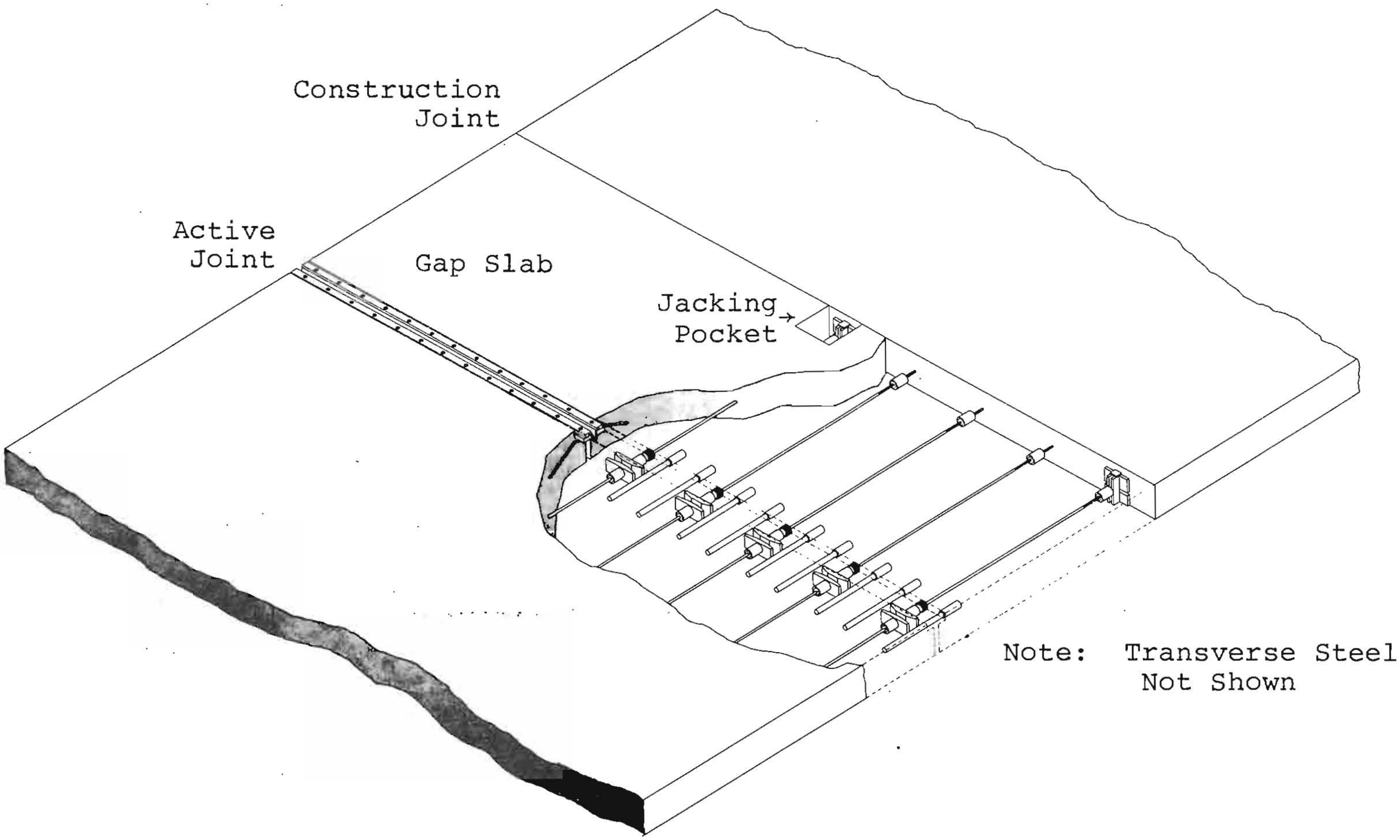


Figure 2 - Design I, Overall View

joint chuck and advances the tendon into the anchorage device. The anchorage device automatically grips the strand as it is advanced. A more descriptive account of each step in the process, together with details concerning jacking equipment and procedures, is given in a separate report entitled "Prestressed Pavement Construction Manual."

A plan view of Design I is shown in Figure 3. Additional details of joint hardware locations and dimensioning are shown in Figures 4 through 7 for Sections A-A through D-D. Circled numbers on drawings are used to reference parts that are listed with dimensions in Table 2.

Tendon anchors, load transfer devices, and methods of preventing infiltration are major joint hardware. These items are discussed separately.

<u>Anchors</u>

Two types of permanent anchors were designed for use at active joints. One type, located in the main slab, combines the anchor body and chuck into one casting. Bearing is provided close to the joint face. Tendons and wedges are protected from corrosion by a stainless steel cap threaded into the anchor after completion of tensioning. A sectional view of this anchor is shown in Figure 5. Details are given in Figure 8.

The second type of permanent anchor is located at the active joint face of the gap slab. The anchor body is threaded internally to accept an externally threaded chuck. Anchors are provided with springs to prevent wedges from moving toward the joint face during gap slab prestressing. Sufficient space is provided within the anchor body to permit strand extension during prestressing operations. These springs also provide positive seating of wedges. A sectional view of this anchor is shown in Figure 5. Details are presented in Figure 9.

Conventional anchors are used in jacking pockets located at the construction joint face. Location of anchor and bearing plates placed during prestress operations are shown in Figure 6.

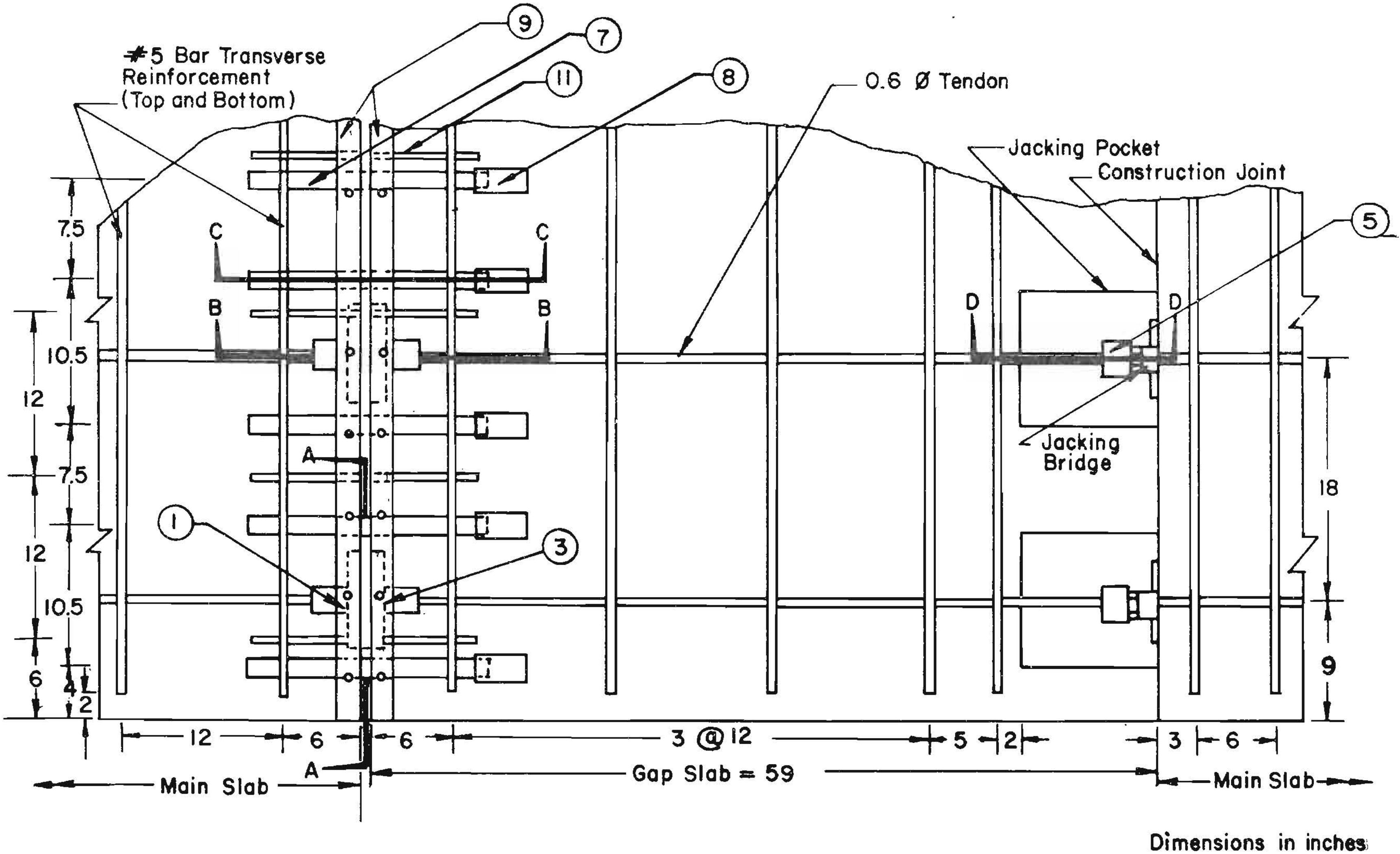


Figure 3 - Plan, Design I

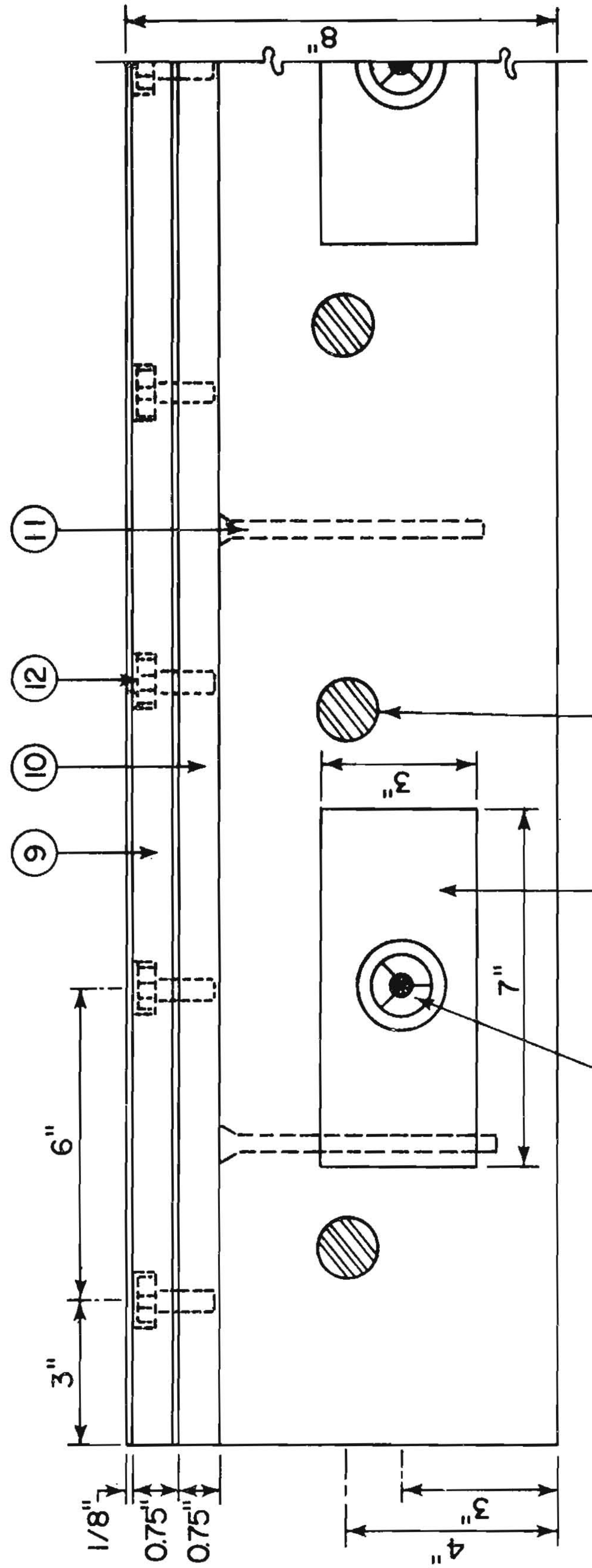


Figure 4 - Section A-A

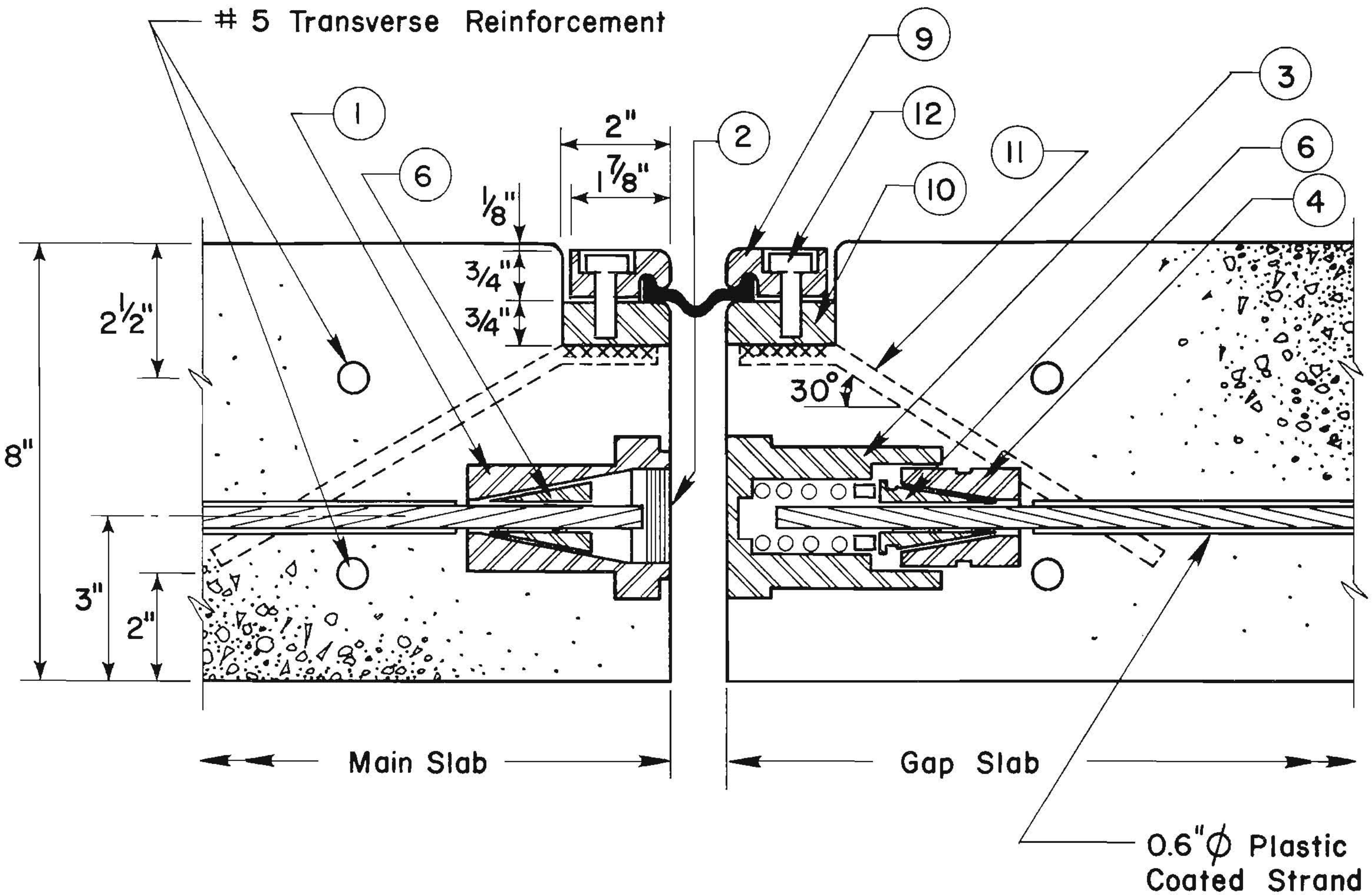


Figure 5 - Section B-B

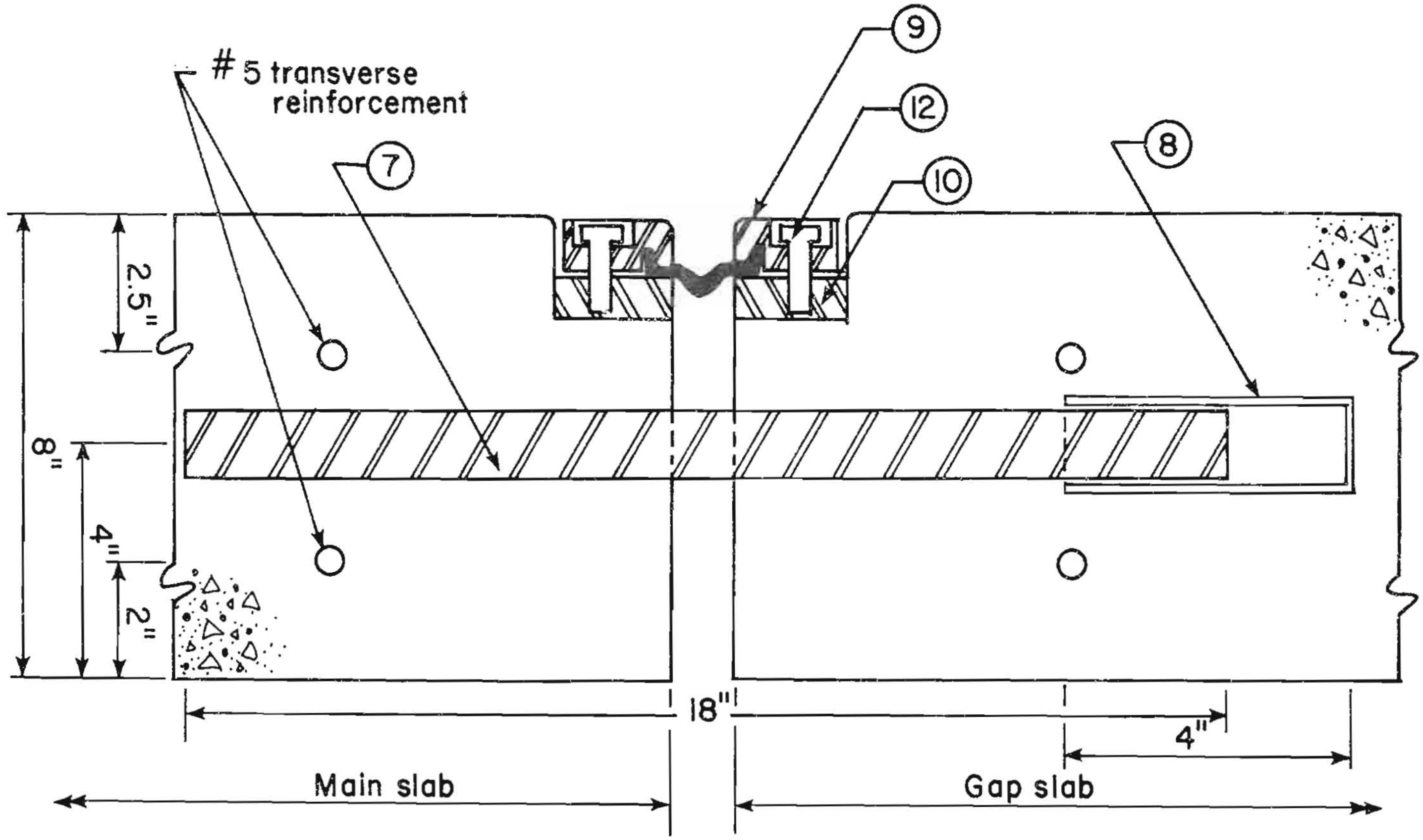
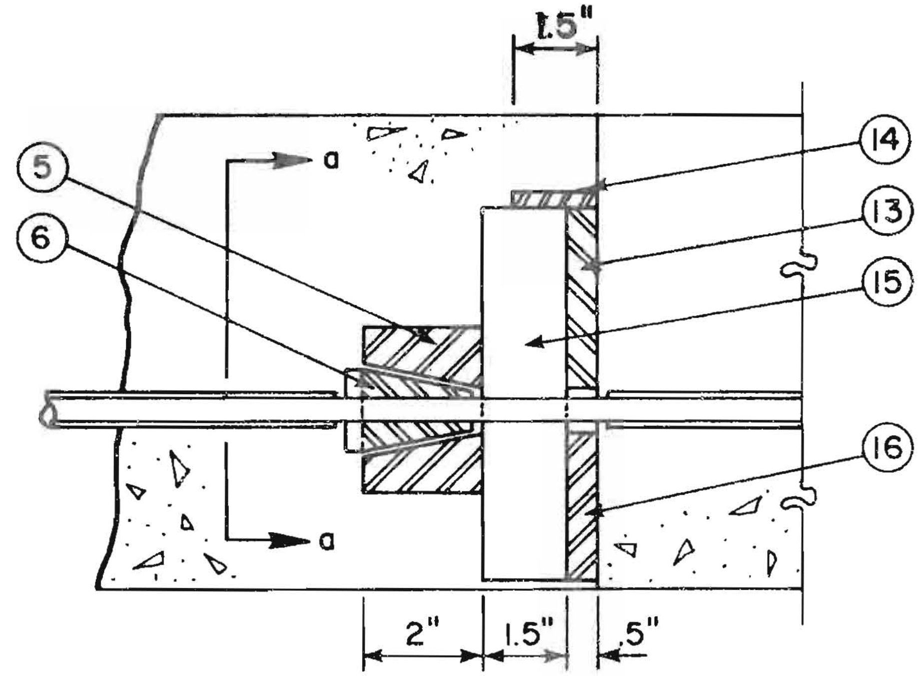
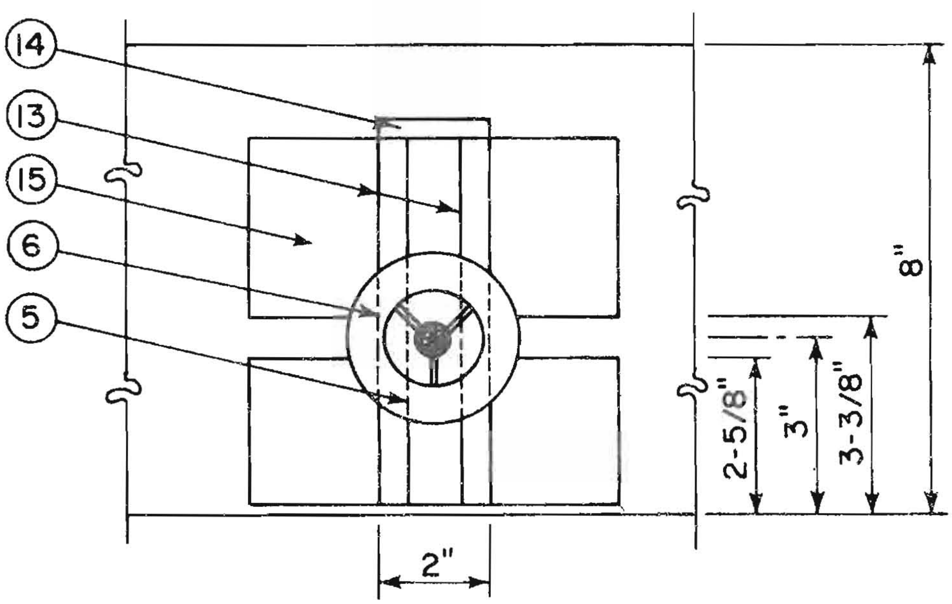


Figure 6 - Section C-C



Section D-D



View a-a

Figure 7 - Section D-D

TABLE 2 - JOINT HARDWARE QUANTITIES FOR STRAND EXTENSION THROUGH GAP SLAB

Item	Quantity	Description	Dimensions
1	16	Combined chuck anchor bodies (Figure 8)	for 0.6" strand
2	16	Stainless steel caps to fit anchor body	--
3	16	Automatic seating anchors (Figure 9)	for 0.6" strand
4	16	Externally threaded chucks	for 0.6" strand
5	16	Barrel chucks for temporary prestressing and stress transfer (Figure 7)	2-3/4" x 2" long
6	48	Three-jawed wedges	for 0.6" strand
7	32	Stainless steel dowels	1-1/4", 18" long
8	32	Dowel Caps (Figure 6)	1-5/16" I.D. X 4"
9	8	Extrusions for upper portion of seal holder with holes for bolts (Figures 4 and 6)	6' long, 3/4" x 1-7/8"
10	18	Extrusions for lower portion of seal holder drilled and tapped (Figures 4 and 5)	6' long, 3/4" X 2"
11	48	#3 deformed bars welded to lower seal holder extrusion (Figures 4 and 5)	9" long (bent as shown in Figure 6)
12	96	Bolts (Figures 4 and 6)	1/2"
13	32	Steel plates (Figure 7)	6-1/4" x 2-3/4" x 1/2"
14	16	Steel plates (Figure 7)	2" x 1-1/2" x 1/4"
15	16	Steel plates (Figure 7)	6" x 3" x 1/2"
16	16	Steel plates (Figure 7)	6" x 2-1/2" x 1/2"

1 in. = 25.4 mm

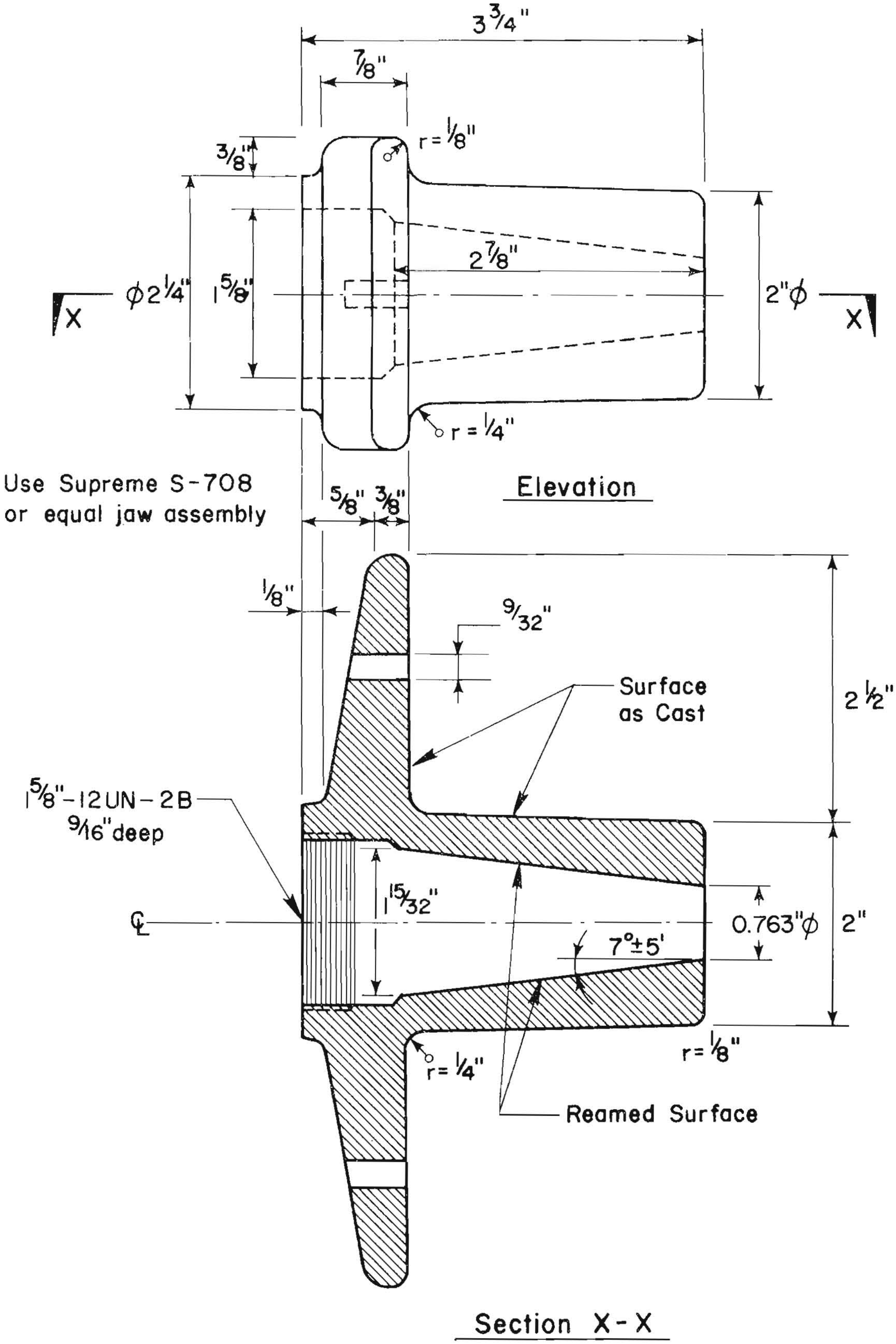


Figure 8 - Permanent Anchor

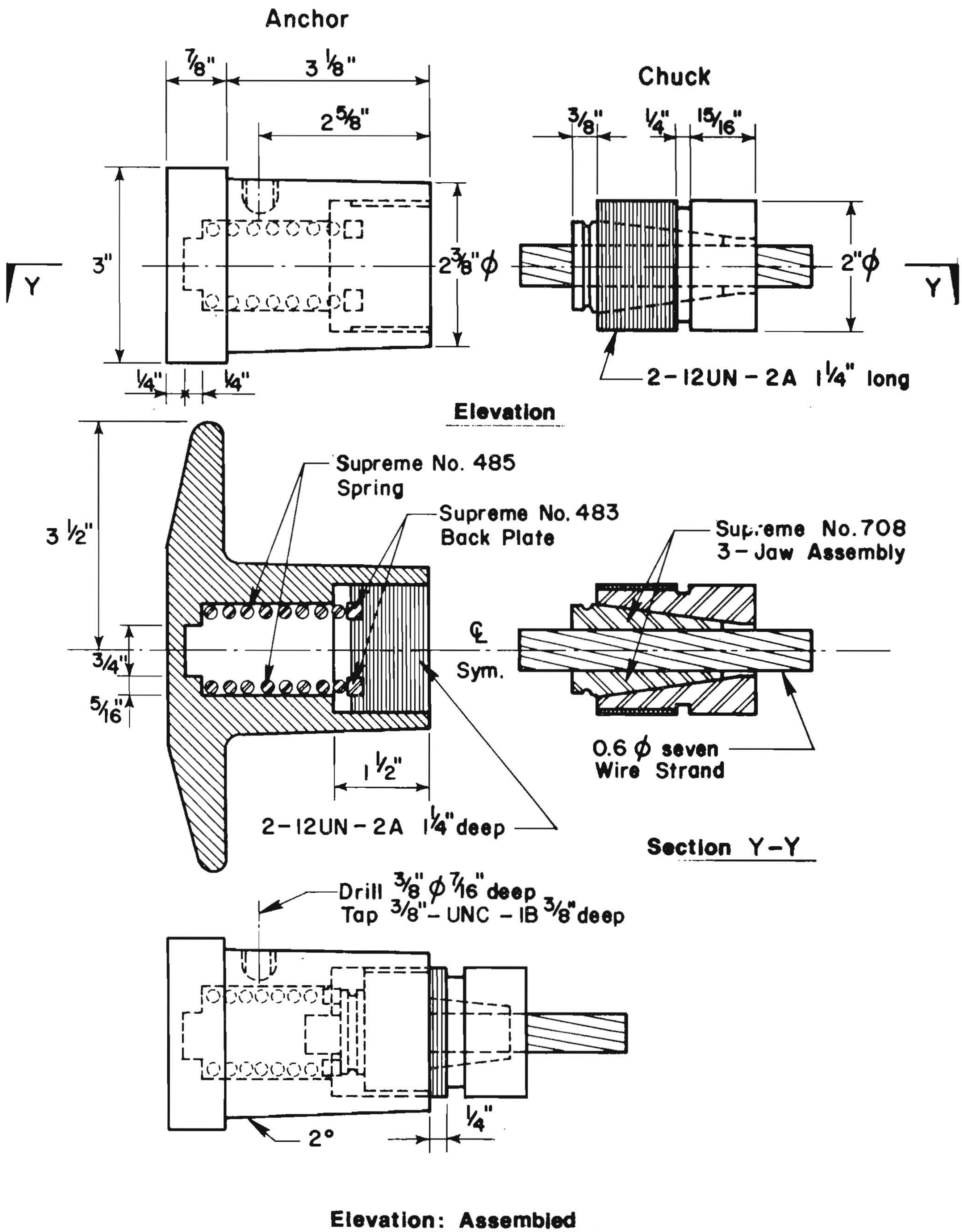


Figure 9 - Automatic Seating Anchor

Tendons and anchors are positioned below slab mid-depth to reduce upward warping of slab ends. Tendon centers are located 5 in (127 mm) below the slab surface.

Load Transfer

Stainless steel dowels, 1-1/4 in (32 mm) in diameter, are located on each side of anchors. Transverse spacing is shown in Figures 3 and 4. Dowels are embedded in the concrete of main slab at the active joint ends. Portions of dowels extending into gap slabs are greased and caps are placed over dowel ends to assure unrestrained slab end movements.

Infiltration Prevention

Premolded nylon reinforced neoprene strip seals are used to prevent joint infiltration. Strip seal holders consist of upper and lower steel extrusions as shown in Figures 5 and 6. Dimensions of top elements are 1-7/8 by 3/4 in (48 by 19 mm). One side is shaped to hold the longitudinal edge of strip seals. Tops of extrusions are set 1/8 in (3 mm) below the concrete surface for protection from snowplow blades.

Top extrusions are drilled and countersunk to recess 0.5-in (12.7 mm) bolts used for clamping upper seal holder extrusion to the lower extrusion. Bolt spacing is 6 in (152 mm) on centers. The lower extrusion is drilled and tapped at spacings to match those in the upper extrusion. The lower extrusion is anchored in the slab by No. 4 deformed bars.

Several different diaphragms or strip seals can be used with this joint design. Generally diaphragm or strip seals are pre-molded extruded neoprene or natural rubber folds extending the length of the joint. They are mechanically anchored to each side of the joint. Normally, 1/8- to 3/8-in (3.2 to 9.5 mm) thick neoprene is used. The neoprene is usually reinforced with fibers to provide tensile strength and resist puncturing.

Seals are held in place by steel sections fixed on each side of the joint. Two additional types of seals with different holddowns are shown in Figures 10a and 10b. These seals have

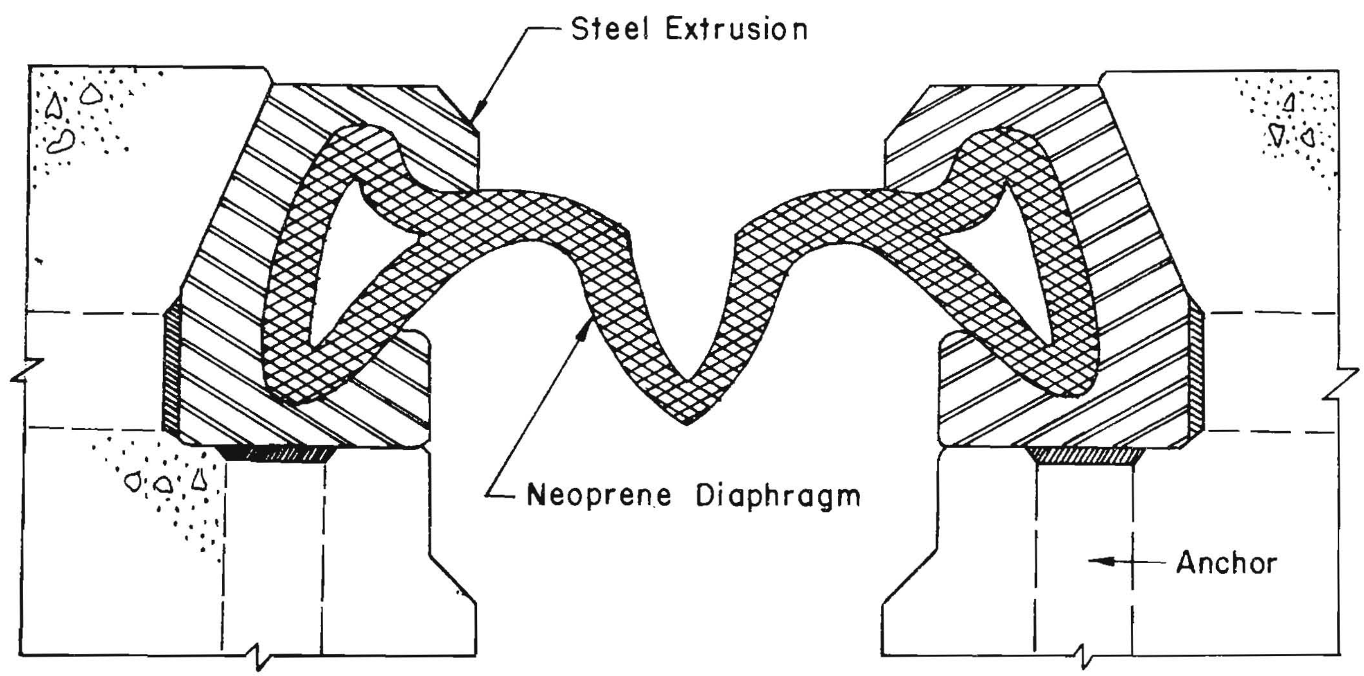


Figure 10a - Wabo-Maurer Strip Seal

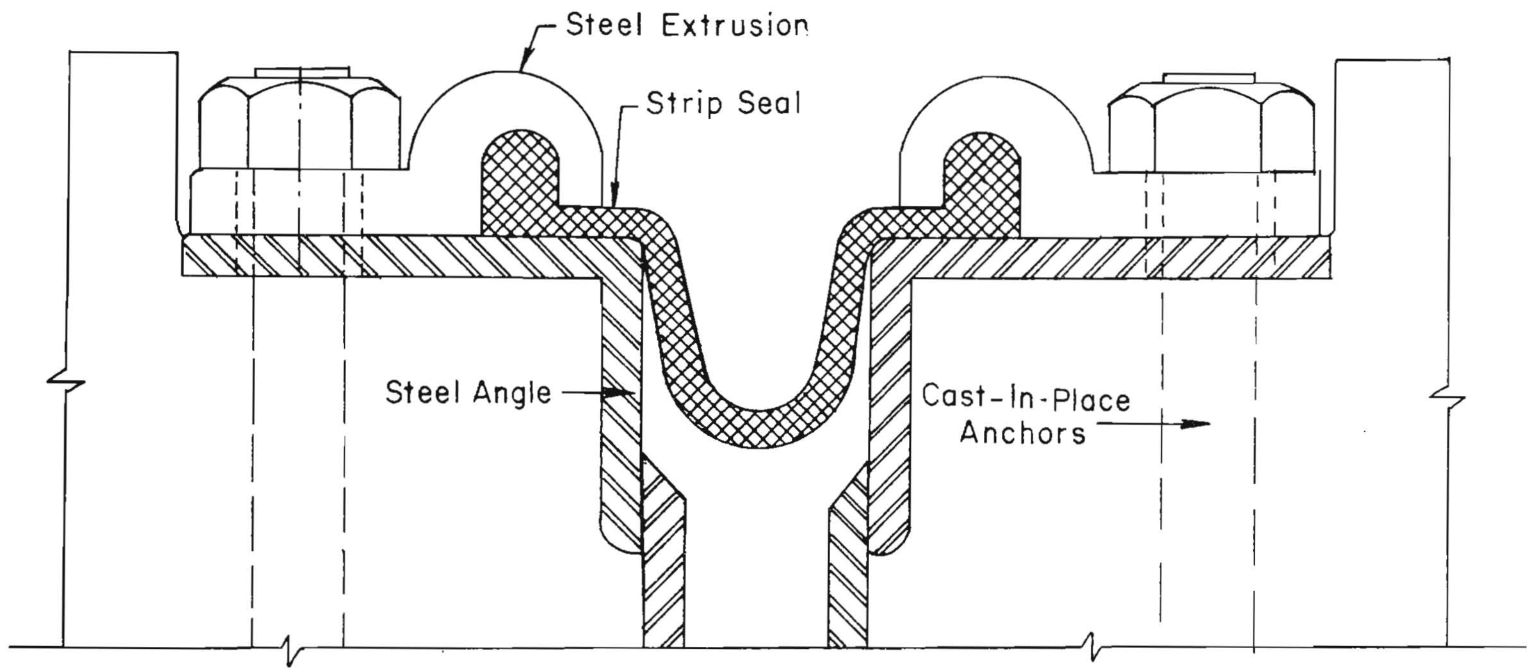


Figure 10b - Pro-Span Strip Seal

been used in bridge joints, where they have been proven satisfactory for movements of the magnitude required between prestressed pavement slab ends.

The Wabo-Maurer strip seal shown in Figure 10a is forced into the anchoring recess after construction. It can withstand a tension of about 1 kip per foot (14.6 kN/m) and permits the neoprene sealer to stretch 1 in (25.4 mm) or more beyond normal extended width.

The Pro-Span seal shown in Figure 10b is molded from doubly reinforced neoprene. The reinforcement provides high strength, but permits only limited stretch. Holders for the strip seals, with pre-attached anchors are vibrated into place during concrete placement.

Joint Hardware Quantities

Joint hardware types and quantities are listed in Table 2. Quantities are for a two-lane, 24-ft (7.3 m) wide highway pavement.

Design II - Rod Prestressed Gap Slab

An overall view of Design II is shown in Figure 11. This design is for an 8-in (203 mm) thick pavement with main slab lengths of 350 ft (101 m). Rods and nuts are used to prestress the gap slab from jacking pockets. Splice chunks located in these pockets join tendons and rods. Jacking in the pocket advances the rod into the anchor. The rod nut located at the gap slab active joint face, is tightened to maintain prestress. This system eliminated the need for the temporary jacking bridge used at the construction joint face of the main slab in Design I. In addition, specially designed anchors used at active joints in Design I are not required. Dowels are used at the active joint to transfer load.

A plan view of Design II is shown in Figure 12. Additional details of joint hardware locations and dimensioning are shown in Figures 13 through 15 for Sections A-A, C-C and D-D.

Conventional "off-the-shelf" anchors are used in this design. Anchors, located at the main slab active joint face,

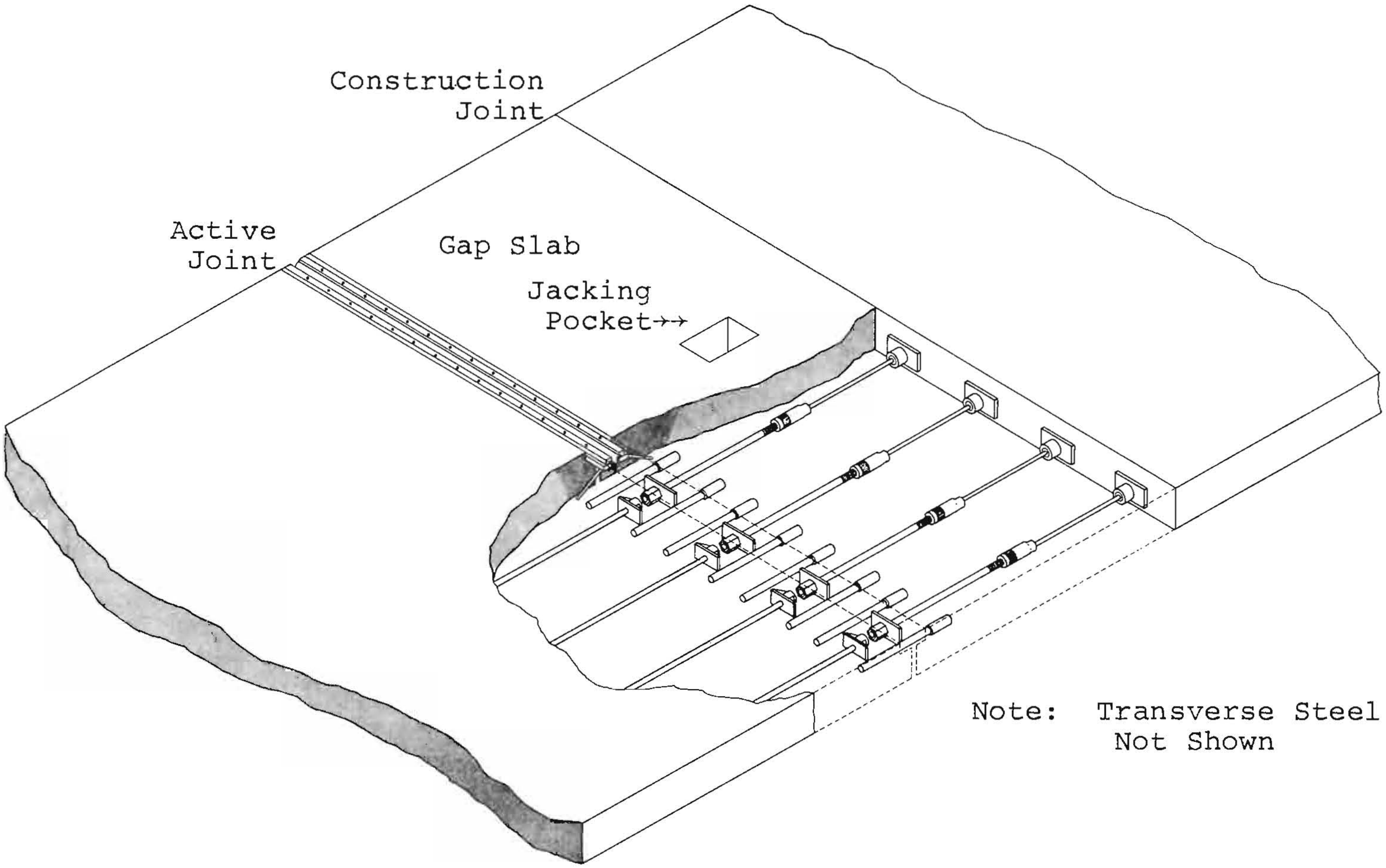
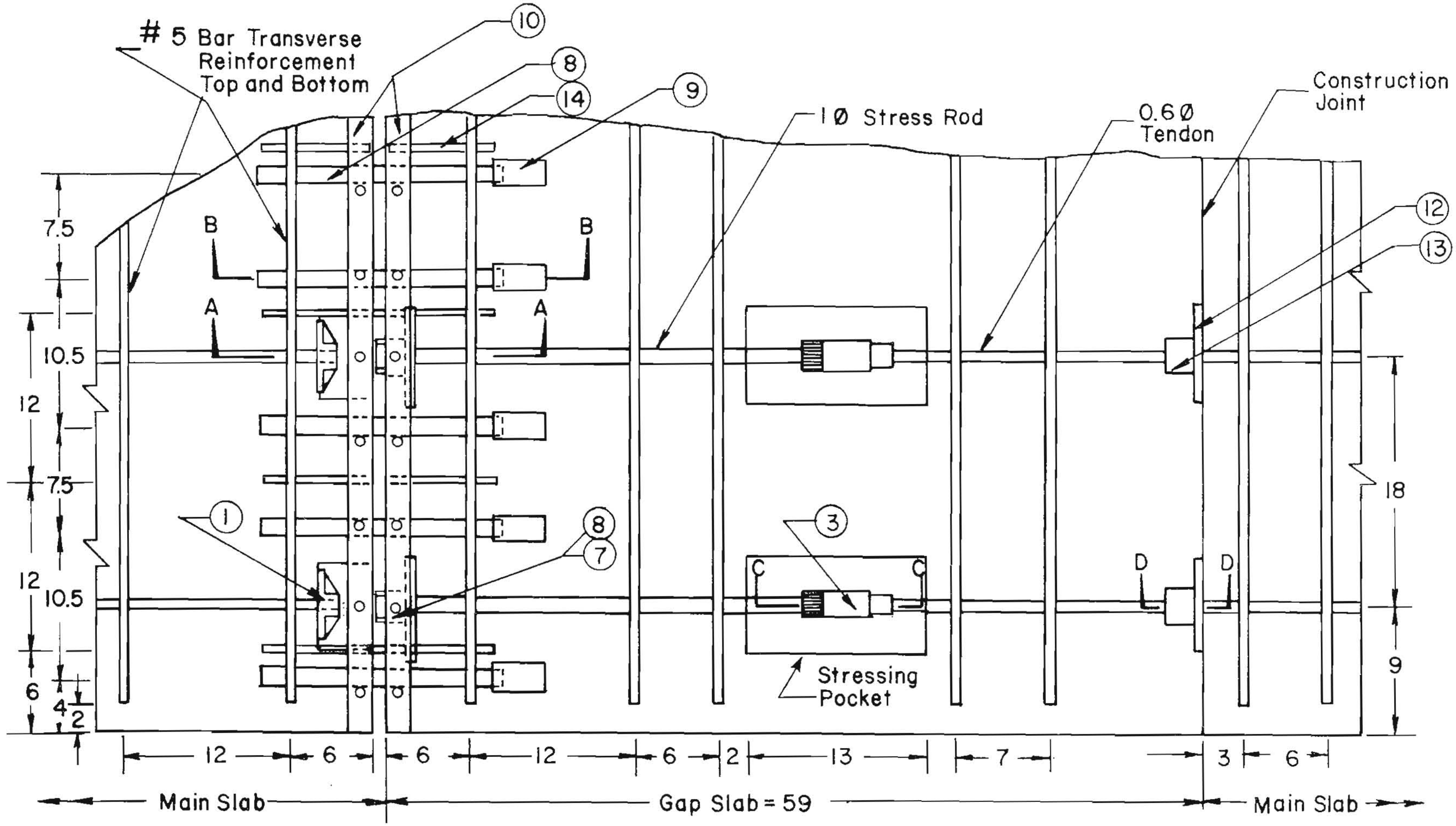


Figure 11 - Design II, Overall View



Dimensions in inches.

Section B-B same as
Section C-C, Figure 5

Figure 12 - Plan, Design II

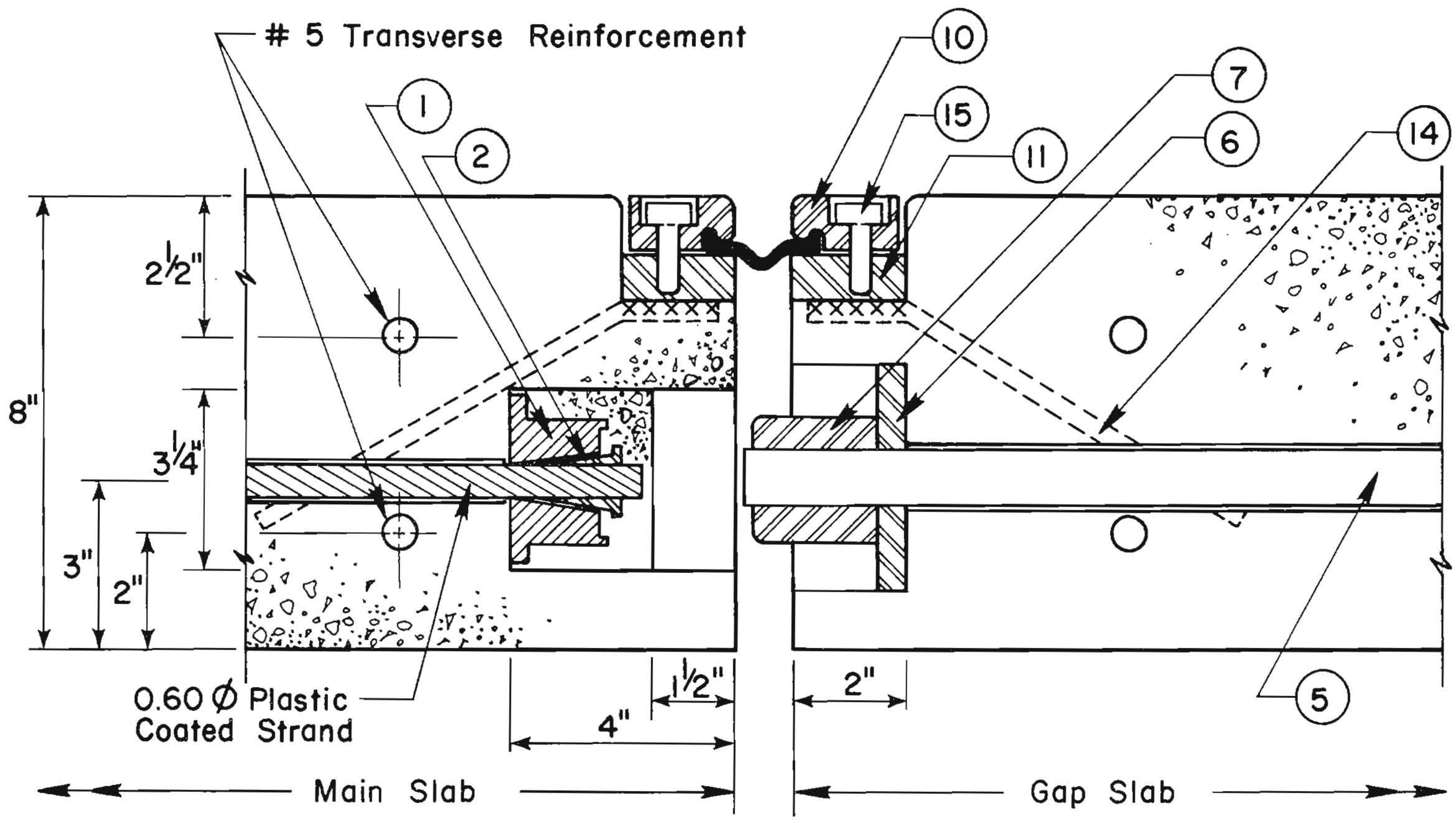


Figure 13 - Section A-A

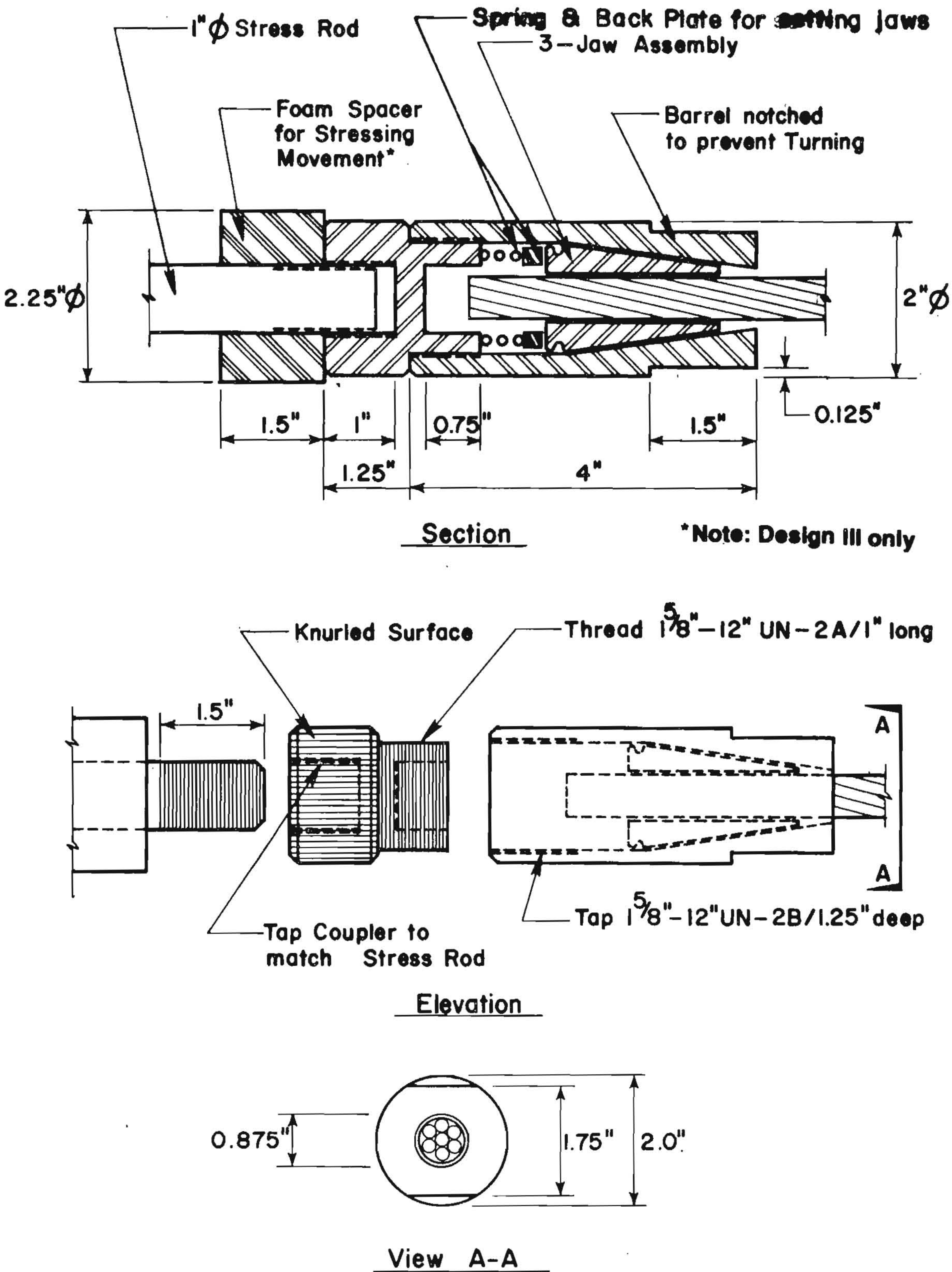


Figure 14 - Section C-C

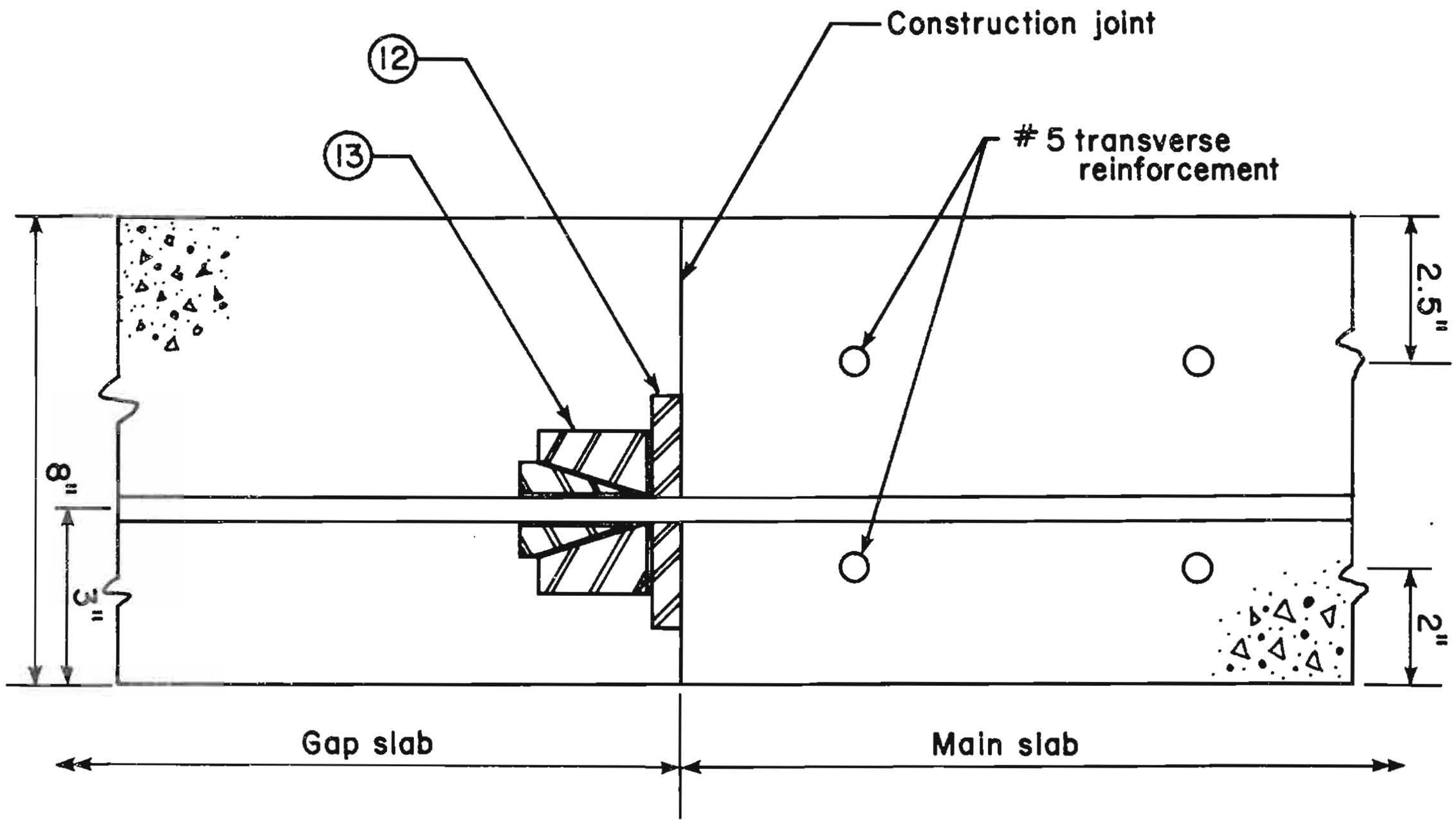


Figure 15 - Section D-D

are recessed 4 in (102 mm) to provide space for grout corrosion protection and also space that grip nuts can occupy during times of decreased joint opening.

Bearing plates, provide reaction for grip nuts located at the gap slab active joint face. These plates are recessed 2 in (50.8 mm) to provide sufficient threaded rod length for the grip nut.

Conventional barrel chucks, three-jawed wedges, and bearing plates are used for anchorage during post-tensioning of main slabs.

Splice chuck strand to rod connections are located at jacking pockets in the gap slab. Splice chuck details are shown in Figure 14. Chuck bodies include conical shaped holders for three-jawed tendon wedges at one end. The other end is threaded internally to accept the threaded rod end.

Jacking forces are applied to the construction joint side of the splice chuck. This assures positive seating of strands and three-jawed wedges and will advance the rods into the active joint space. Grip nuts are tightened for seating on bearing plates.

Jacking pocket dimensions are 7 in by 13 in (178 by 330 mm). Dimensions may be varied to accommodate jacking equipment and splice chuck movements. However, dimensions should be kept as small as practical. Jacking force does not need to exceed 20 kip (89 kN) per strand. This will put a compressive stress of about 130 psi (896 kPa) at the construction joint thus providing sufficient load transfer.

Joint hardware types and quantities are listed in Table 3. Quantities are for a two-lane 24-ft (7.3 m) wide highway pavement.

Design III - Cover Plate Joint

An overall view of Design III is shown in Figure 16. This design, for an 8-in thick pavement with main slab lengths of 350 ft (107 m), also uses a rod and nut system to prestress the gap slab. However, post-tensioning is accomplished by manual

TABLE 3 - JOINT HARDWARE QUANTITIES FOR STRESS RODS TROUGH GAP SLAB

Item	Quantity	Description	Dimensions
1	16	Standard anchor bodies (Figure 12)	3" x 5.25" for 0.6" strand
2	32	Three-jawed wedges	for 0.6" strand
3	16	Splice chucks including couplers, three-jawed edges and springs (Figure 14)	for 0.6" strand
4	16	Barrel Chucks for temporary prestressing and/or stress transfer (Figure 18)	for 0.6" strand
5	16	Plastic-coated stress rods, threaded both ends with thread length of about 2 in	1", 31.5" long (compatible with length of gap slab)
6	16	Bearing plates (Figure 13)	7" x 4" x 1/2"
7	16	Nuts (Figure 13)	2-1/4"
8	32	Stainless steel dowels (Figure 6)	1-1/4" , 18" long
9	32	Caps for dowels (Figure 5)	1-5/16" I.D. x 4"
10	8	Extrusion for upper seal holder (Figure 13)	6' long, 1-1/4" x 1-1/2"
11	8	Extrusion for lower seal holder (Figure 13)	(Same as Design I)
12	16	Bearing plates with center hole to slip over strand, used at construction joint (Figure 12)	7" x 4" x 1/2"
13	16	Barrel chucks for prestressing at construction joint with 3-jaw wedges (Figure 12)	2-3/4" x 2" long
14	48	#4 deformed bars (Figure 13)	10" length bent as shown and welded to seal holder extrusions
15	96	Bolts (Figure 13)	1/2"

1 in = 25.4 mm

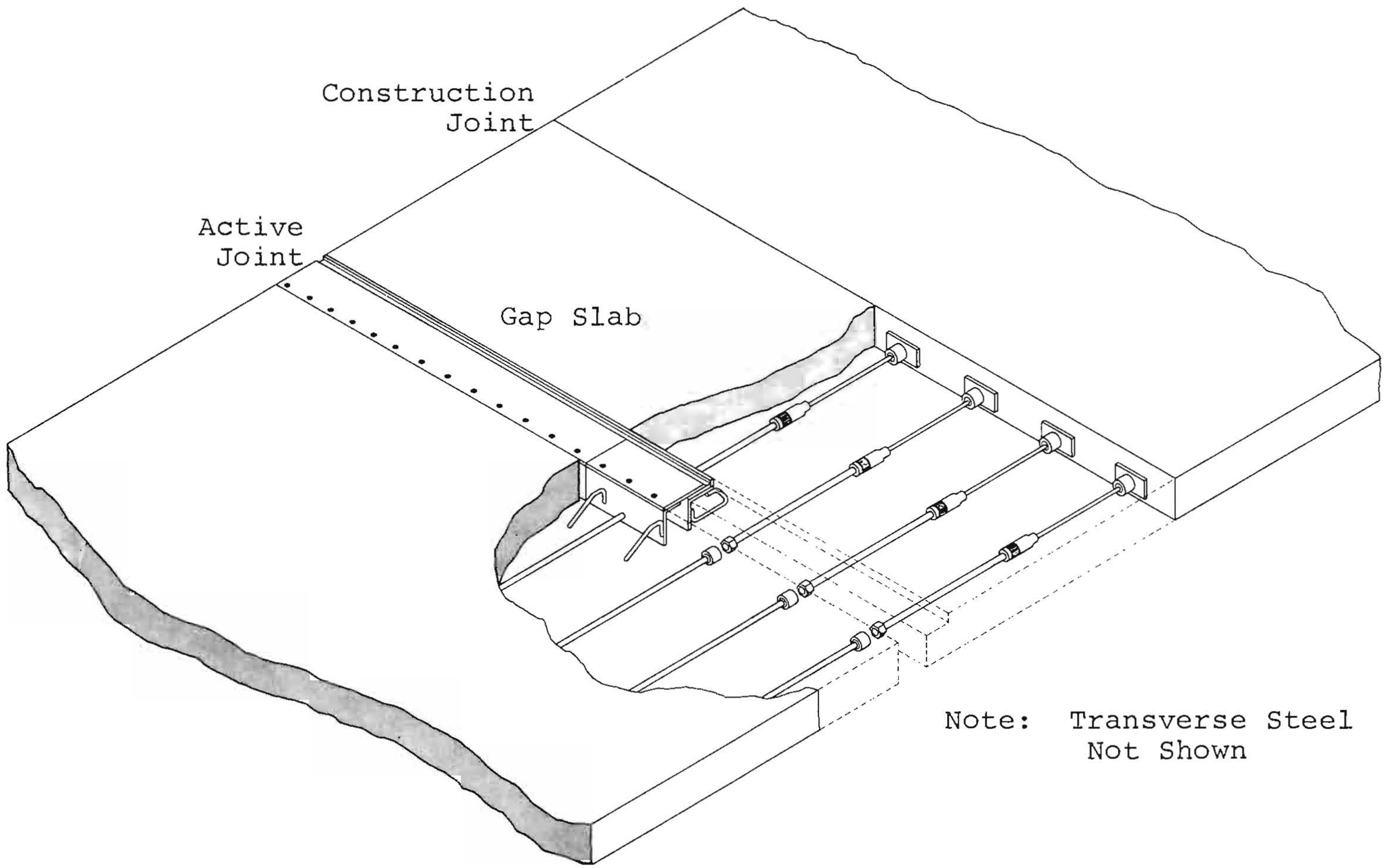


Figure 16 - Design III, Overall View

torquing of the nut located at the active joint face. Thus, jacking pockets are not required. The steel cover plate joint serves as bearing for tensioning tendons and nuts at the active joint. In addition, cover plates contribute to load transfer.

Splice chucks are the same as shown in Figure 14. For post-tensioning, rods are plastic coated and a 1-1/2-in (37 mm) wide foamed insert is placed at the active joint end of the splice chuck. This insert permits the rod and tendon to extend in the direction of movement as the nut is torqued.

Details of the steel cover plate joint assembly are shown in Figures 17 and 18. As shown in Figure 18, one angle of the cover plate joint assembly is fixed to the main prestressed slab. If tendons are placed continuously during construction, the angle can be notched to permit placement. If tendons are positioned prior to concreting, drilled holes are sufficient for passing tendons into the joint space. The vertical leg of the angle provides bearing for the tendon anchor. Barrel chuck anchors are protected from corrosion by epoxy filled caps placed after final prestressing.

A 6x6x3/8-in (152x152x9.5 mm) steel angle is used at the gap slab side of the active joint. Deformed bar loops welded to the angle anchor it to the gap slab. A nylon bearing pad is placed between the horizontal leg of the steel angle and the cover plate. It is provided to prevent joint chatter that may occur if the cover plate rested directly on the steel angle. To prevent excessive temperature restraint stresses, transverse lengths of cover plates and steel angles should not exceed 6 ft (1.8 m).

Dowels are not used with the steel cover plate joint. Minimum distance between joint faces is about 4 in (101 mm) making use of dowels ineffective. Sleeper slabs, lean concrete subbase, or a subbase of equivalent supporting strength is recommended for use with this design.

Joint hardware types and quantities for steel cover plate active joints are listed in Table 4. Quantities are for a two-lane 24-ft (7.3 m) wide pavement.

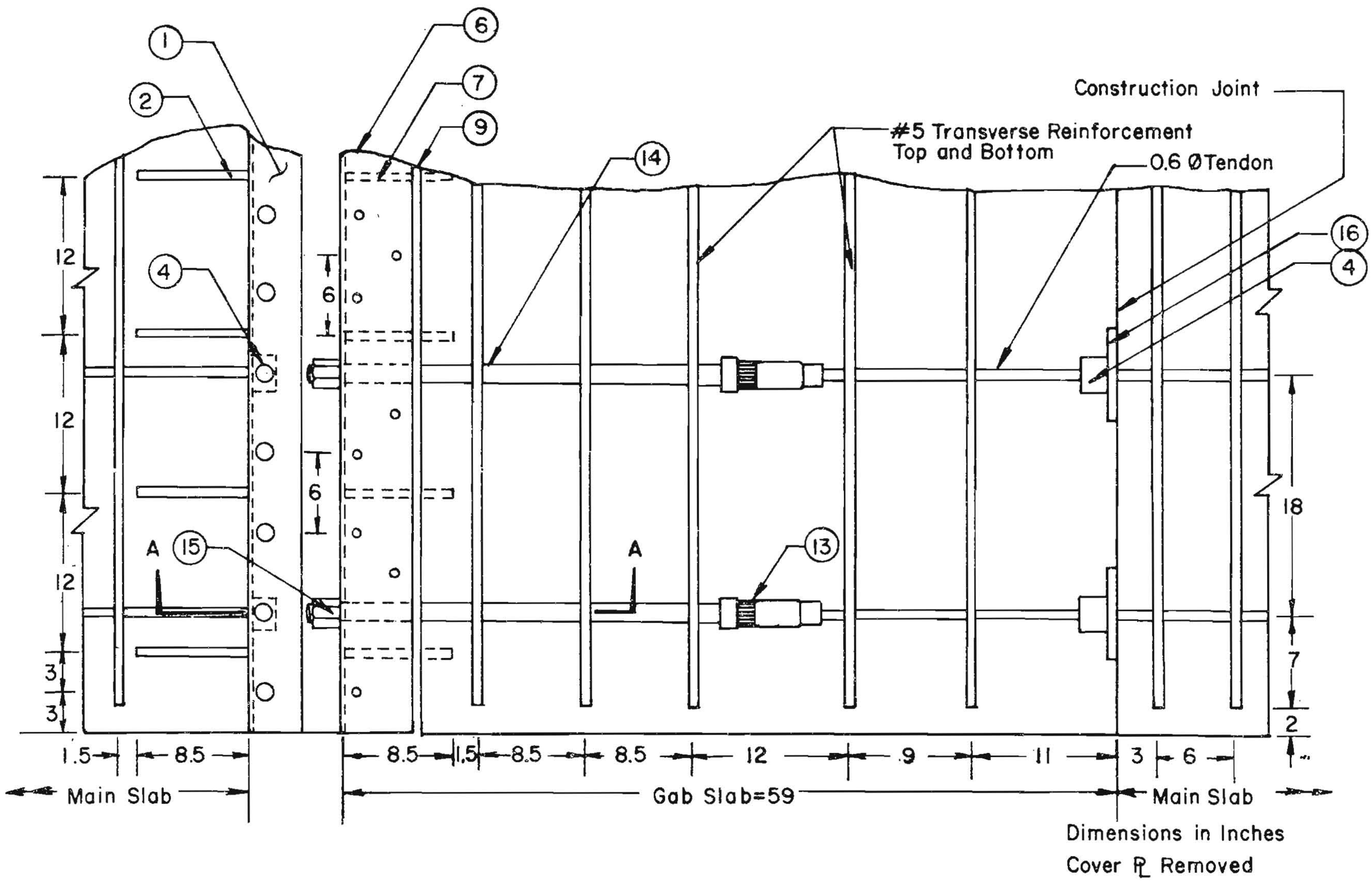


Figure 17 - Plan, Design III

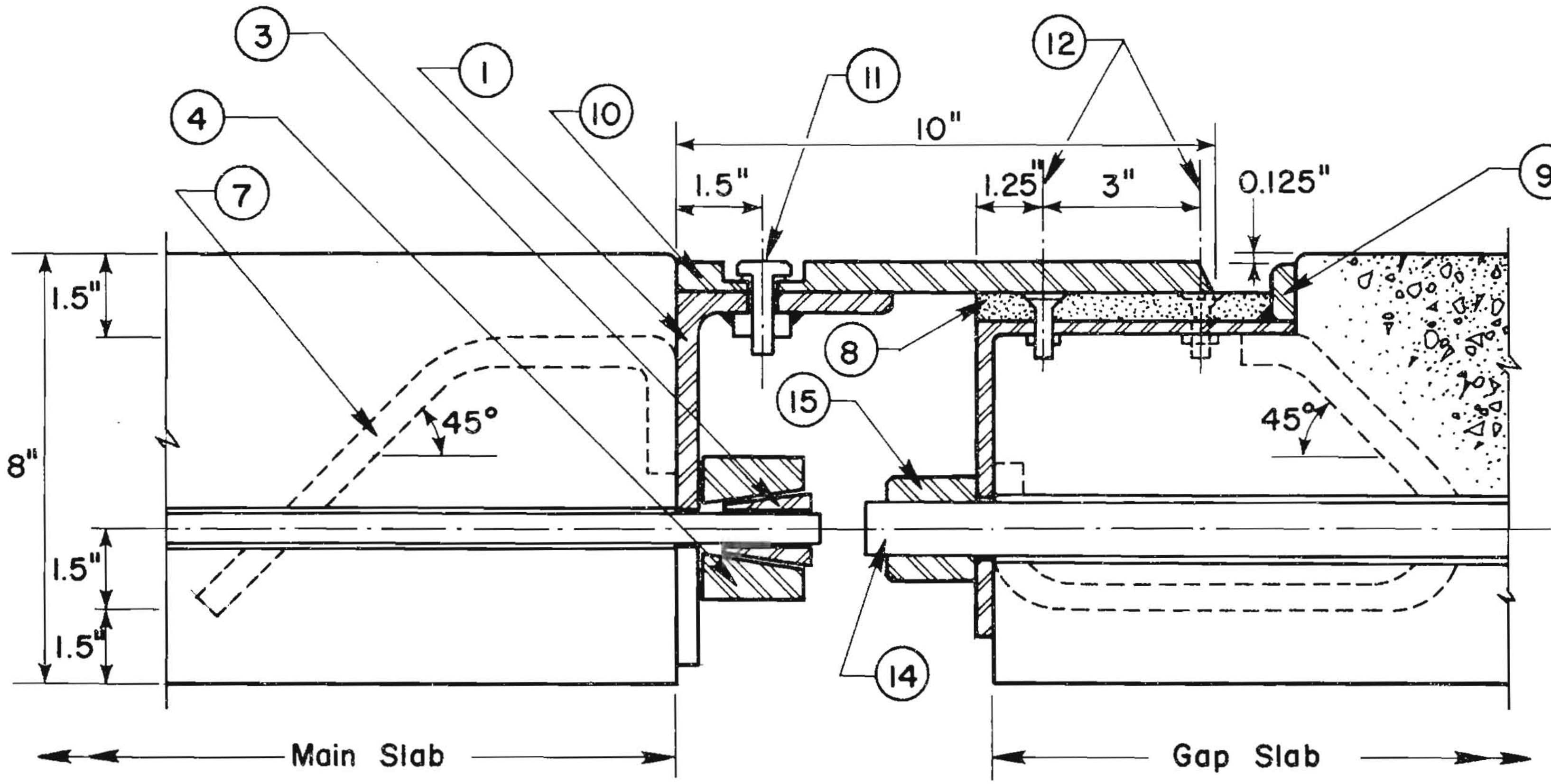


Figure 18 - Section A-A

TABLE 4 - JOINT HARDWARE QUANTITIES FOR
COVER PLATE EXPANSION JOINTS WITH
STRESS RODS THROUGH GAP SLAB

Item	Quantity	Description	Dimensions
1	4	Steel angles (Figure 17)	6' long angles 7" x 4" x 3/8"
2	48	#4 deformed bars welded to 7" x 4" x 3/8 angle (Figure 17)	12" long
3	32	Three-jawed wedges (Figure 18)	for 0.6" strand
4	16	Barrel chucks (Figure 18)	for 0.6" strand
5	16	Plastic caps for protecting strand and anchorage	
6	4	Steel angles (Figure 17)	6' long angles 6" x 6" x 3/8
7	48	#4 deformed bar loops (Figure 18)	
8	4	Neoprene pads (Figure 18)	6' long 1/2" x 5-1/2"
9	4	Steel strips (Figure 18)	6' long 1/2" x 1"
10	4	Steel Cover Plates (Figure 18)	6' long 1/2" x 10"
11	48	Bolts, nuts and lock washers to attach cover plate to angle (Figure 18)	1"
12	96	Machine bolts and nuts to attach neoprene pads (Figure 18)	1/2"
13	16	Splice chucks including couplers, three-jawed wedges, springs, and foam spacers (Figure 17)	for 0.6" strand

Continued on next page...

TABLE 4 - JOINT HARDWARE QUANTITIES FOR COVER PLATE EXPANSION JOINTS WITH STRESS RODS THROUGH GAP SLAB

(continued)

Item	Quantity	Description	Dimensions
14	16	Plastic-coated stress rods, threaded both ends with threaded length of about 2 in	(Compatible with length of gap slab)
15	16	Nuts (Figure 15)	1-7/8" x 1-5/8"
16	16	Bearing plates with center hole to slip over strand, used at construction joint (Figure 17)	6-1/4" x 1-1/2" x 1/2"

1 in = 25.4 mm
1 ft = 0.30 m

Design IV - Compression Seal Joint

An overall view of Design IV is shown in Figure 19. This design is for a 7-in (178 mm) thick pavement with main slab lengths of 250 ft (76 m). Main slab pavement thickness is reduced because a tied concrete shoulder is used. Prestressing is terminated at main slab ends. Gap slabs are 10-in (250 mm) thick and are conventionally reinforced. An active joint is provided at each end of the gap slab. Dowels provide load transfer at the active joints.

A plan view of Design IV is shown in Figure 20. Additional details of joint hardware and dimensioning are shown in Figures 21 and 22.

As shown in Figure 21, a compression seal is used at the active joint. Seals must accommodate anticipated joint opening of 1.2 in (30 mm). Concrete shoulders prevent downward seal movement. Stainless steel dowels spaced at 12-in (300 mm) centers are located 4 in (76 mm) below the slab surface. Tendon anchors are "off-the-shelf" models. Anchor faces are recessed to provide space for grout corrosion protection.

Joint hardware types and quantities are listed in Table 5. Quantities are for a two-lane 24-ft (7.3 m) wide highway pavement with a 10-ft (3 m) wide outside shoulder and a 2-ft (0.6 m) inside shoulder.

SUMMARY

Four joint designs for use with prestressed pavements have been presented. Details are provided for 7- and 8-in (178 and 203 mm) thick pavements. These thicknesses were computed to be satisfactory for zero-maintenance designs developed for heavily trafficked freeways. For less heavy traffic, smaller thicknesses may be satisfactory. Joint design details in this report can be easily adapted to other thicknesses by making the necessary dimensional changes.

APPENDIX A - DESIGN CALCULATIONS

Joint design calculations determine pavement thickness, prestress requirements, and joint movements. Examples are presented to illustrate calculation procedures. Materials coefficients and temperature data used in these calculations are for demonstration purposes only. For a specific project, coefficients should be determined by tests using proposed materials with consideration given to climatic conditions.

Pavement Thickness

Pavement thickness is determined using the computer program presented in, "Prestressed Pavement Thickness Design."[A1] This program considers the case of loads applied at or near the longitudinal edge. Bottom fiber stresses due to load, temperature differentials, moisture differentials, and mid-slab prestress are determined and summed for levels of traffic. Resultant stress for each level of loading consumes a portion of the pavements fatigue resistance. The design thickness is that for which fatigue consumption generally ranges between 60 to 100 percent. Using this procedure, a small compressive stress is maintained in the slab.

Two examples of the relative magnitude of these stresses and how they are combined are shown in Table A1. Examples are for a hypothetical case of unlimited applications of a 20 kip (89 kN) single axle load. Calculated edge load stresses assume a loss of subbase support for a distance 20 in (508 mm) inward from the pavement edge. In addition, the 7-in (178 mm) thick pavement includes a tied concrete shoulder. Load transfer efficiency at the shoulder joint is assumed to be 60 percent.

Prestress Requirements

End prestress must be sufficient to provide a minimum mid-slab prestress of about 50 psi (345 kPa) after subtracting losses due to tendon friction, concrete shrinkage, concrete creep, steel relaxation, and subbase friction restraint.

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SUMMARY

Four joint designs for use with prestressed pavements have been presented. Details are provided for 7- and 8-in (178 and 203 mm) thick pavements. These thicknesses were computed to be satisfactory for zero-maintenance designs developed for heavily trafficked freeways. For less heavy traffic, smaller thicknesses may be satisfactory. Joint design details in this report can be easily adapted to other thicknesses by making the necessary dimensional changes.

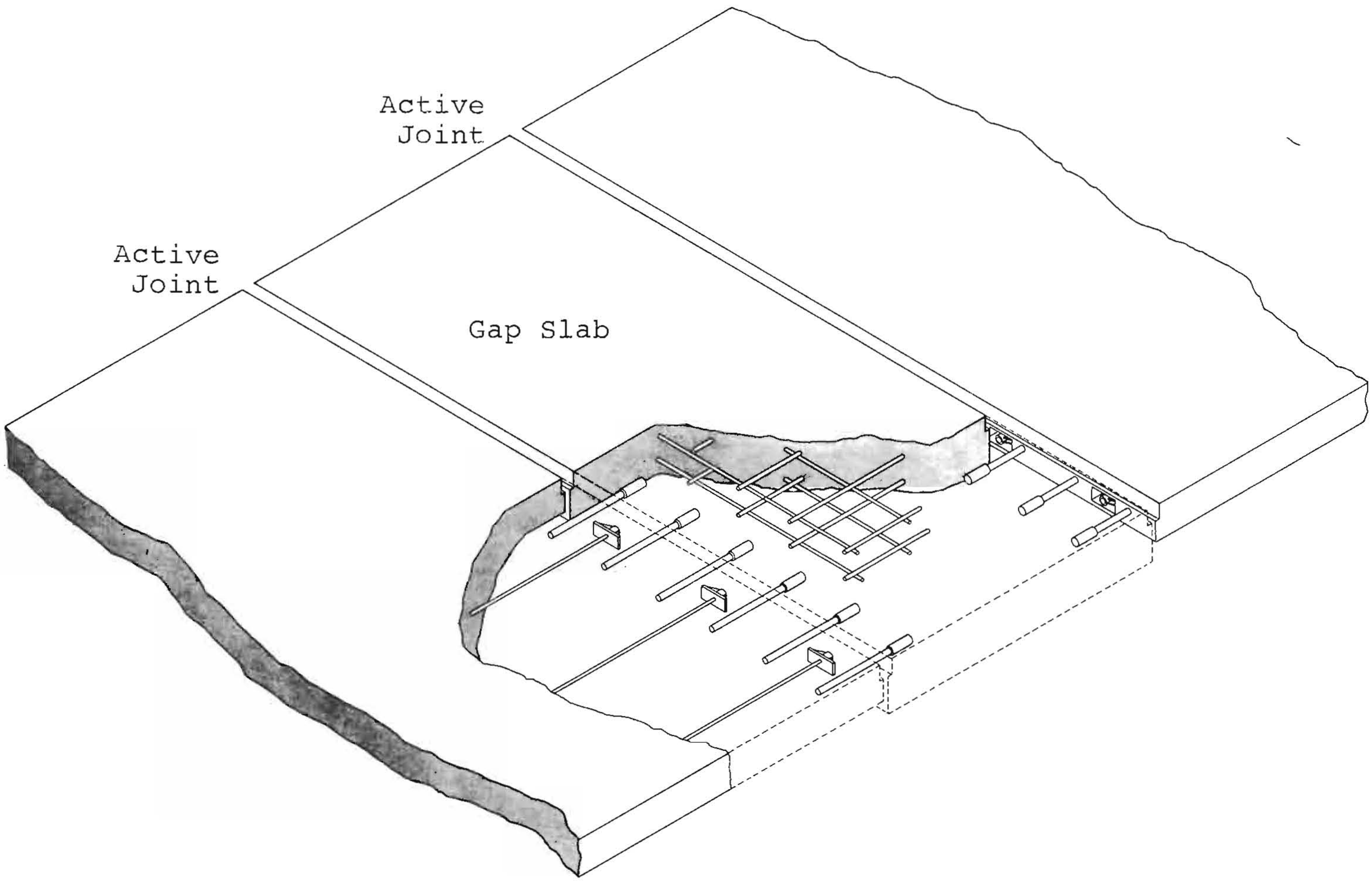

Figure 19 - Design IV, Overall View

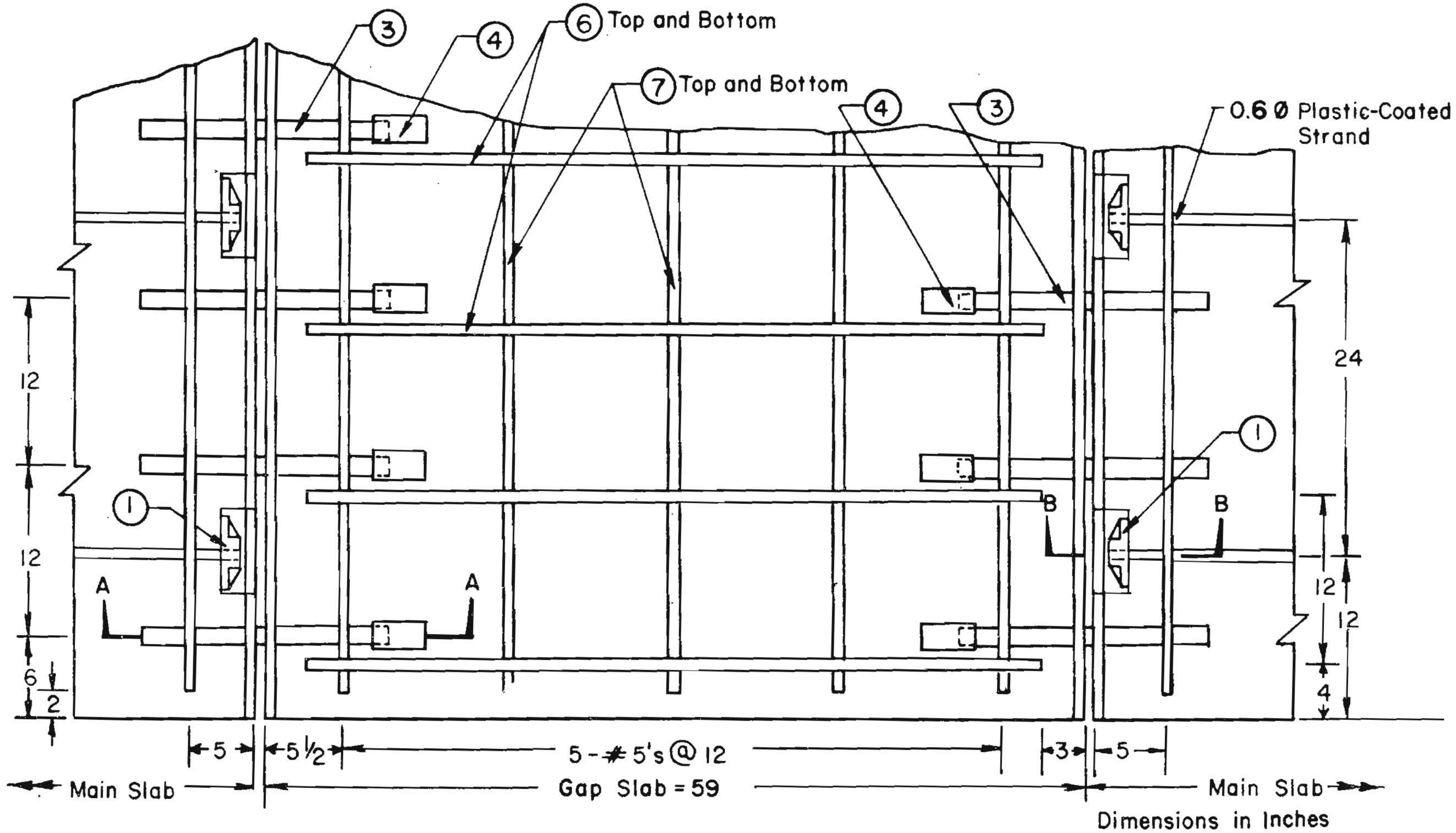

Figure 20 - Plan, Design IV

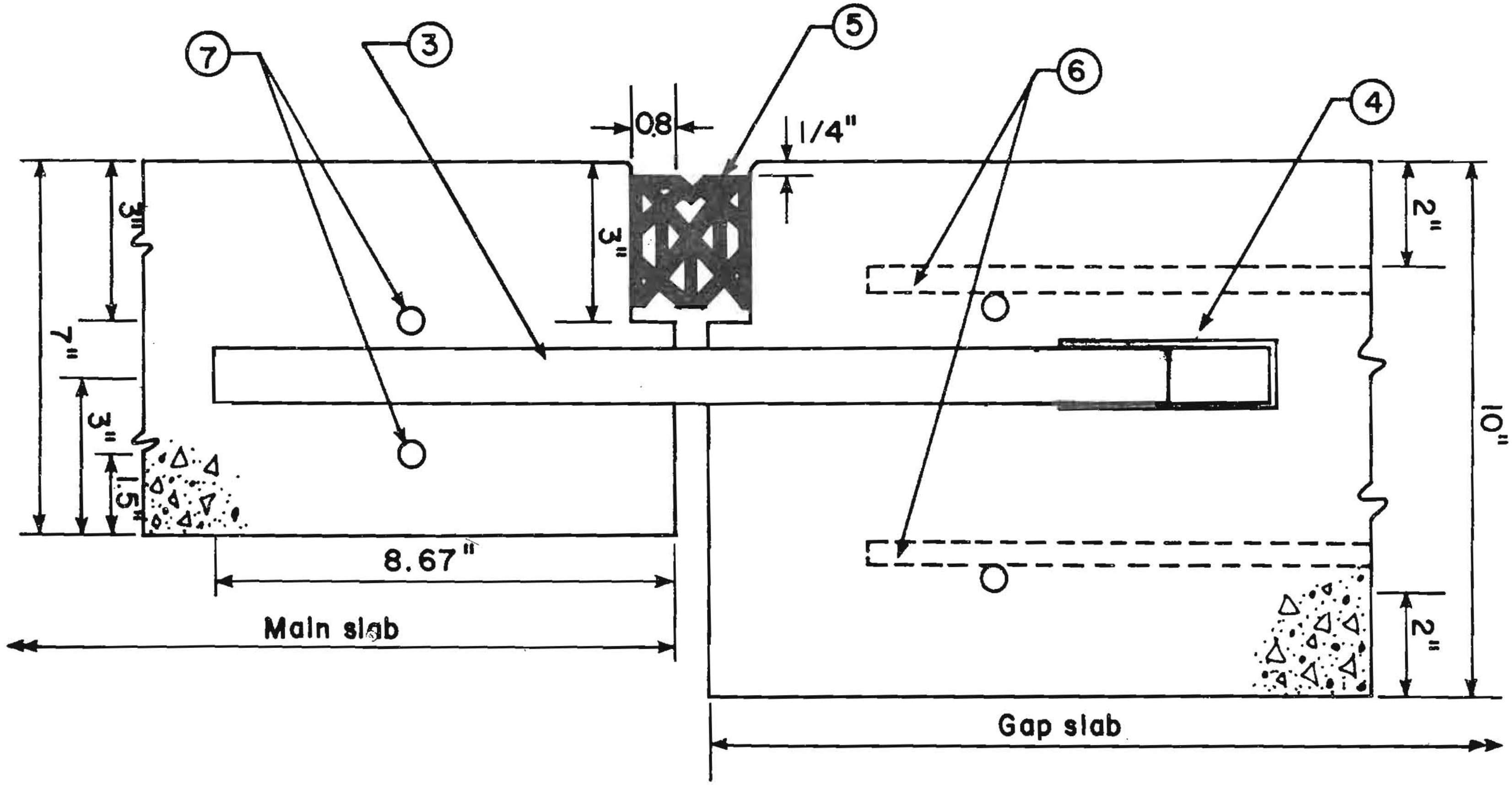

Figure 21 - Section A-A

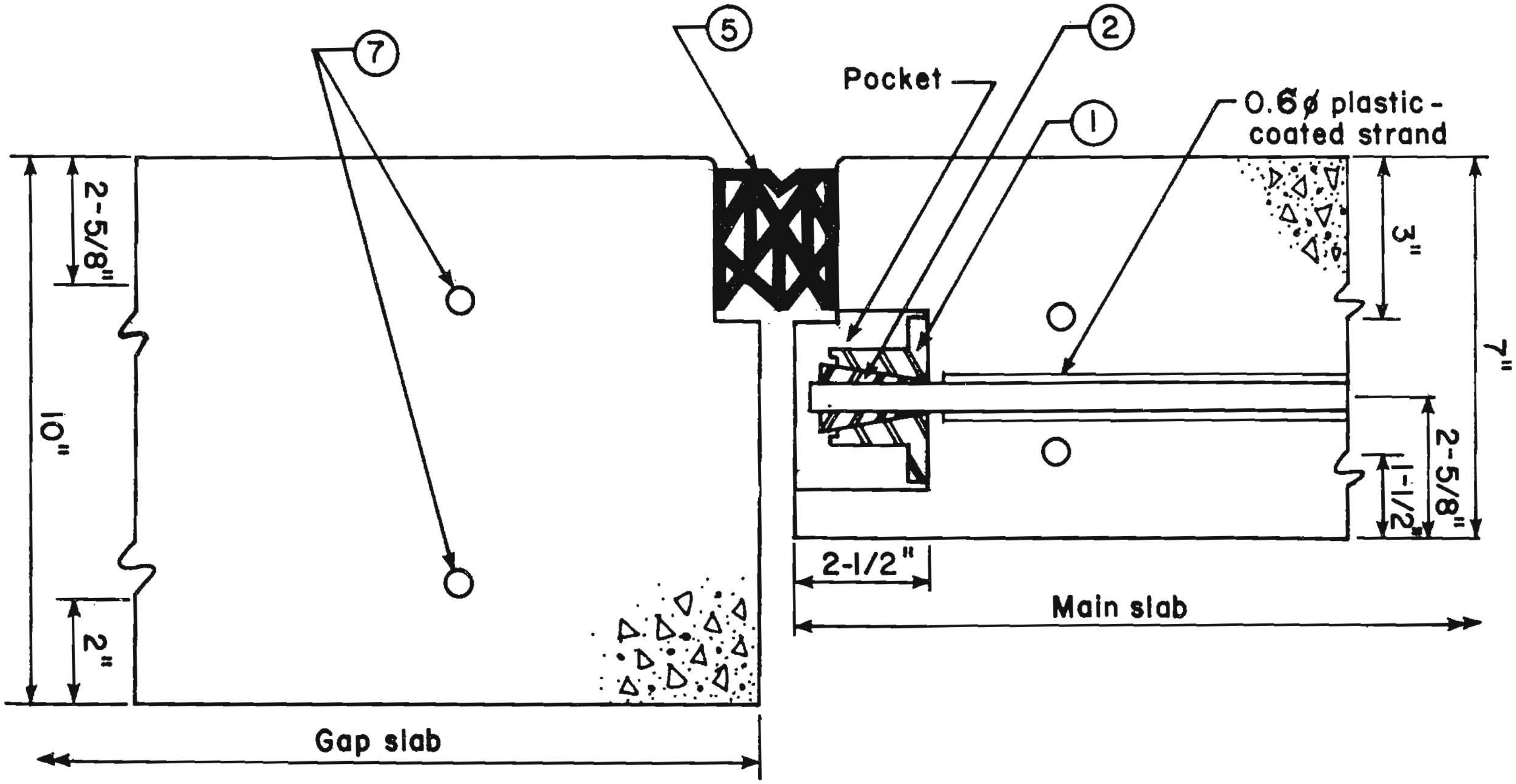

Figure 22 - Section B-B

TABLE 5 - JOINT HARDWARE QUANTITIES FOR COMPRESSION SEAL EXPANSION JOINTS

Item	Quantity	Description	Dimensions
1	36	Standard anchors (Figure 20 & 22)	3" x 5-1/4" for 0.6" strand
2	36	Three-jawed wedges (Figure 22)	for 0.6" strand
3	36	Stainless steel dowels (Figure 21)	18" long 1-1/8"
4	36	Dowel caps (Figure 21)	4" long, to fit over 1-1/8" dowel
5	1	Compression seal ACMASEAL (K-400) (Figure 21)	36'long, nominal uncompressed 2-1/2" high, 3.4" wide
6	36	#5 bar longitudinal reinforcement) (Figures 20 & 21)	54" long
7	9	#5 bar transverse reinforcement (Figure 20)	35' long

1 in = 25.4 mm
1 ft = 0.30 m

REFERENCES

1. Tayabji, S. D., Colley, B. E., and Nussbaum, P. J., "Prestressed Pavement Thickness Design," report prepared by Construction Technology Laboratories for Federal Highway Administration, December 1980.

2. Nussbaum, P. J., Friberg, B. F., Ciolko, A. T., and Tayabji, S. D., "Prestressed Pavement Construction Manual," report prepared by Construction Technology Laboratories for Federal Highway Administration, December 1980.

APPENDIX A - DESIGN CALCULATIONS

Joint design calculations determine pavement thickness, prestress requirements, and joint movements. Examples are presented to illustrate calculation procedures. Materials coefficients and temperature data used in these calculations are for demonstration purposes only. For a specific project, coefficients should be determined by tests using proposed materials with consideration given to climatic conditions.

Pavement Thickness

Pavement thickness is determined using the computer program presented in, "Prestressed Pavement Thickness Design."[(A1)] This program considers the case of loads applied at or near the longitudinal edge. Bottom fiber stresses due to load, temperature differentials, moisture differentials, and mid-slab prestress are determined and summed for levels of traffic. Resultant stress for each level of loading consumes a portion of the pavements fatigue resistance. The design thickness is that for which fatigue consumption generally ranges between 60 to 100 percent. Using this procedure, a small compressive stress is maintained in the slab.

Two examples of the relative magnitude of these stresses and how they are combined are shown in Table A1. Examples are for a hypothetical case of unlimited applications of a 20 kip (89 kN) single axle load. Calculated edge load stresses assume a loss of subbase support for a distance 20 in (508 mm) inward from the pavement edge. In addition, the 7-in (178 mm) thick pavement includes a tied concrete shoulder. Load transfer efficiency at the shoulder joint is assumed to be 60 percent.

Prestress Requirements

End prestress must be sufficient to provide a minimum mid-slab prestress of about 50 psi (345 kPa) after subtracting losses due to tendon friction, concrete shrinkage, concrete creep, steel relaxation, and subbase friction restraint.

TABLE A1 - THICKNESS DESIGN DATA

Slab Thickness, in	7	8
Slab Length, ft	250	350
Edge Load Transfer, %	60	0
Edge Load Stress, psi	-236	-319
Temperature Stress, psi	-263	-300
Warping Restraint Stress, psi	+190	+220
Mid-Slab Prestress, psi	+75	+57
Combined Edge Stress, psi	-234	-342
Allowable Stress, (Unlimited Applications) psi	-350	-350
Residual Compressive Stress, psi	+116	+ 8

\+ tensile stress
\- compressive stress
1 in = 25.4 mm, 1 kip = 4.448 kN, 1 psi = 6.894 kPa
$C = \frac{5}{9}(F-32)$

Equations for calculating losses follow.

Tendon Friction

Tendon friction results from curvature and wobble. Curvature is due to intentional and wobble to unintentional tendon profile variations. Tendon friction is determined from the following equation:[A2]

$$\sigma_t = \sigma_{pe}\left(1 - e^{-(ux + \frac{KL}{2})}\right) \qquad (1)$$

where: σ_{pe} = end prestress, psi,

u = curvature friction coefficient

x = angular change of tendon from jacking end to mid-slab, radians

K = wobble friction coefficient per foot

L = slab length, ft

For straight portions of pavements, intentional angular changes are negligible. Therefore, tendon friction can be obtained from the following equation:

$$\sigma_t = {}_{pe}\left(1 - e^{-\frac{KL}{2}}\right) \qquad (2)$$

Concrete Shrinkage

Prestress loss due to concrete shrinkage is given by the following equation:[A2]

$$\sigma_s = \varepsilon_s E_s \times \frac{A_s}{A_c} \qquad (3)$$

where: ε_s = concrete shrinkage strain

E_s = modulus of elasticity of tendon steel

A_s = area of tendon per unit width of slab

A_c = area of slab per unit width of slab

Concrete Creep

Prestress loss due to concrete creep is given by the following equations:[A2]

$$\sigma_c = C_u \frac{E_s}{E_c} \sigma_{pe} \times \frac{A_s}{A_c} \tag{4}$$

where: C_u = ultimate creep coefficient

E_c = modulus of elasticity of concrete

Steel Relaxation

Prestress loss due to steel relaxation is given by the following equation:(A2)

$$\sigma_r = \rho \sigma_{pe} \tag{5}$$

where: ρ = relaxation coefficient for appropriate stress level

Subbase Friction

Prestress loss due to subbase friction is given by the following equation:

$$\sigma_f = \frac{\mu_s \gamma L}{288} \quad \text{psi} \tag{6}$$

where: μ_s = slab to subbase friction factor

γ = concrete unit weight, lb/ft^3

L = slab length, ft

Example Calculations

Examination of the above equations show that some require use of an end prestress value for the calculation. This means an initial value is assumed and the final value is determined by an iterative process. One method of shortening this process is to make judgement assumptions regarding tendon diameter, spacing, and stress level. For example if 0.6-in (15 mm) diameter tendons are stressed to 70 percent of ultimate, allowable tendon force is 41,000 lb (182 kN). For tendons spaced at 18 in (457 mm) centers in an 8-in (203 mm) thick pavement, initial end prestress is 285 psi (1965 kPa). This procedure was used to compute losses for examples shown in Table A2. Coefficients and other values required for computations are listed in Table A3.

TABLE A2 - PRESTRESS CALCULATIONS

Slab Thickness, in	7	8
Slab Length, ft	250	350
Strand Diameter, in	0.6	0.6
Strand Force, kip	41	41
Strand Spacing, in	24	18
End of Slab Prestress, psi	244	285
Prestress Losses		
Shrinkage, psi	6	6
Creep, psi	4	6
Relaxation, psi	20	23
Strand Friction, psi	39	62
Subbase Friction, psi	100	140
Total Losses, psi	169	237
Mid-Slab Prestress, psi	75	48

1 in = 25.4 mm
1 ft = 0.30 m
1 kip = 4.448 kN
1 psi = 6.894 kPa

TABLE A3 - PRESTRESS LOSS
COMPUTATION COEFFICIENTS

Item	Magnitude
Tendon Ultimate Strength, psi	270,000
Area 0.6 in diameter tendon, in^2	0.217
Concrete Creep Coefficient	2.5
Concrete Shrinkage Strain, millionths	150
Strand Relaxation Coefficient 70 percent of Ultimate Stress 75 percent of Ultimate Stress	 0.08 0.10
Wobble Friction Coefficient per foot	0.0014
Subbase Friction Factor	0.8
Modulus of Elasticity of Steel, million psi	28
Modulus of Elasticity of Concrete, million psi	5

1 psi = 6.894 kPa
1 in = 25.4 mm
1 ft = 0.30 m

Computation results listed in Table A2 show that mid-slab prestress for the 8-in (203 mm) thick design is less than the desired minimum of 50 psi (345 kPa). In this case, recalculated losses for an assumed tendon stress of 75 percent of ultimate results in an end prestress of 306 psi (1361 kPa) and mid-slab prestress of 57 psi (393 kPa).

This solution is satisfactory. Therefore tendons in 8-in (203 mm) thick designs are stressed to 75 percent of ultimate. For 7-in (178 mm) thick designs tendons are stressed to 70 percent of ultimate.

Slab End Movements

Slab end movements result from daily and seasonal temperature variations and concrete drying shrinkage and creep. Subbase frictional restraint is applied only to movements due to daily temperature variations.

Temperature Associated Movements

Temperature associated movements are functions of the concrete coefficient of thermal expansion and local temperature variations. Movements are affected by seasonal as well as daily temperature effects. Seasonal movements take place over a long period of time and, therefore, it is assumed that slab to subbase friction does not restrict movement.[A3] However, daily movements due to temperature variation are affected by slab to subbase friction. Therefore, daily movements are corrected for subbase frictional restraint.

During winter, slab concrete is more moist than in the summer. This results in a concrete coefficient of thermal expansion about 15 percent lower than that for concrete in a drier state.[A4] This factor is considered for computation of daily temperature associated movements for winter months.

Seasonal slab movement is given by the following equation:

$$d_1 = \alpha(\Delta t)L \qquad (7)$$

where: d_1 = slab movement associated with seasonal temperature changes

α = coefficient of thermal expansion of concrete

Δt = seasonal variation in average concrete temperature

Maximum daily slab movement during summer months is given by the following equation:

$$d_2 = \alpha(\Delta t_s)\ L - d_f \qquad (8)$$

where: d_2 = slab movement associated with daily temperature variation during summer

Δt_s = summer maximum temperature less summer average temperature

d_f = slab movement restrained by subbase friction

$$d_f = \frac{\sigma_f}{2E_c}\ (L) \qquad (9)$$

where: σ_f = maximum subbase friction restraint stress (see Equation 6)

Maximum daily slab movement during winter months is given by the following equation:

$$d_3 = 0.85\ \alpha(\Delta t_w)\ L - d_f \qquad (10)$$

where: d_3 = slab movement associated with daily temperature variation during winter

Δt_w = winter average temperature less winter minimum temperature

Shrinkage

Shrinkage strain is influenced by amount of mixing water, water-cement ratio, aggregate type, and curing conditions. For concrete prisms drying from all faces long-term shrinkage strain varies from 100 to 500 millionths.[A5] For slabs drying only from the top a value of 250 millionths has been assumed. About 100 millionths of this strain takes place within the first month. Since gap slabs are not placed until about one month after main slabs are cast only 150 millionths strain needs to be considered in computing future slab shortening.

Slab shrinkage is given by the following equation:

$$d_4 = \varepsilon_s L \quad (11)$$

where: d_4 = slab shortening due to shrinkage

ε_s = concrete shrinkage strain

Concrete Creep

Creep is the long-term shortening of concrete subjected to sustained stress. The relationship between concrete creep strain, ε_k, and elastic strain is expressed by the equation: (A6)

$$\varepsilon_k = C_u \frac{fav}{E_c} \quad (12)$$

where: fav = average prestress along slab length, psi

C_u = ultimate creep coefficient

Slab shortening due to creep is then given by the following equation:

$$d_5 = \varepsilon_k L \quad (13)$$

Creep magnitude varies with gradation of concrete aggregates, particle shape, aggregate type, cement content, water-cement ratio, concrete density, curing, age at loading, load intensity, and concrete element size. A creep coefficient of 2.5 is suggested for computing creep associated shortening of prestressed pavements.

Two examples of slab end movement computations are presented. Coefficients, weather information, and other factors used in calculations are listed in Table A4.

Designs 1, 2, 3

Computations are for an 8-in (203 mm) thick pavement with 350 ft (106.7 m) long main slabs. One active joint is used at the gap slab.

Seasonal Temperature Effect

$$d_1 = 5.0 \times 10^{-6} \times 350 \times 12 \times 63$$
$$= 1.32 \text{ in} \quad (12.7 \text{ mm})$$

TABLE A4 - END MOVEMENT COMPUTATION COEFFICIENTS

Item	Magnitude
Modulus of Elasticity of Concrete, million psi	5
Concrete Creep Coefficient	2.5
Coefficient of Thermal Expansion, summer, in /in /F	0.0000050
Coefficient of Thermal Expansion, winter, in /in /F	0.0000043
Concrete Shrinkage Strain, millionths	150
Slab to Subbase Friction Factor	0.8
Summer Average Concrete Temperature, F	93
Winter Average Concrete Temperature, F	30
Seasonal Variation in Average Concrete Temperature, F	63
Summer Maximum Concrete Temperature, F	114
Winter Minimum Concrete Temperature, F	-8
Summer Maximum Temperature Excess of Average, F	21
Winter Minimum Temperature less than Average, F	38
Average Slab Prestress for 7 in Thick Pavement, psi	165
Average Slab Prestress for 8 in Thick Pavement, psi	160

1 psi = 6.894 kPa.

1 in = 25.4 mm

$C = \frac{5}{9}(F-32)$

Daily Temperature Effect

For daily temperature length changes, the influence of subbase frictional restraint is considered. It is assumed that restraint stress increases linearly from zero to a maximum value at mid-slab.

For 350 ft (106.7 m), frictional restraint stress at mid-slab is 140 psi for a slab to subbase friction coefficient of 0.8. Then, movement restrained by subbase friction is:

$$d_f = \frac{140 \times 350 \times 12}{2 \times 5{,}000{,}000} = 0.06 \text{ in}$$

a. Summer

Unrestrained Deformation $= 5.0 \times 10^{-6} \times 350 \times 12 \times 21$
$= 0.44$ in

Actual Deformation, d_2 = Unrestrained Deformation
- Deformation restrained by subbase friction

$= 0.44 - 0.06$
$= 0.38$ in (9.7 mm)

b. Winter

Unrestrained Deformation $= 4.25 \text{ X } 10^{-6} \text{ X } 350 \text{ X } 12 \text{ X } 38$
$= 0.68$ in

Actual Deformation, d_3 = Unrestrained Deformation
- Deformation restrained by subbase friction

$= 0.68 = 0.06$
$= 0.62$ in (15.7 mm)

Shrinkage

Shrinkage Strain $= 150 \times 10^{-6}$ in /in

$d_4 = 150 \times 10^{-6} \times 350 \times 12$
$= 0.64$ in (16.1 mm)

Total movement at active joint $= d_1 + d_2 + d_3 + d_4 + d_5$
$= 3.78$ in (83.3 mm)

Creep

Average slab prestress is 165 psi. Ultimate length change due to creep at end of design life is:

$$d_5 = 2.5 \times \frac{165 \times 350 \times 12}{5{,}000{,}000}$$

$$= 0.32 \text{ in} \quad (8.1 \text{ mm})$$

Design 4

Computations are made for 7-in (178 mm) thick pavement with 250-ft (76 m) long main slabs. Two active joints are used at the gap slab.

Seasonal Temperature Effect

$$d_1 = 5.0 \times 10^{-6} \times 125 \times 12 \times 63$$

$$= 0.47 \text{ in} \quad (7.4 \text{ mm})$$

Daily Temperature Effect

Deformation restrained due to subbase friction, d_f = 0.02 in.

a. Summer

Unrestrained Deformation = $5.0 \times 10^{-6} \times 125 \times 12 \times 21$
= 0.16 in

Actual Deformation, d_2 = Unrestrained Deformation
- Deformation restrained by subbase friction
= 0.16 - 0.02
= 0.14 in (4.1 mm)

b. Winter

Unrestrained Deformation = $4.25 \times 10^{-6} \times 125 \times 12 \times 38$
= 0.24 in

Actual Deformation, d_3 = Unrestrained Deformation
- Deformation restrained by subbase friction
= 0.24 - 0.02
= 0.22 in (5.6 mm)

Shrinkage

$$d_4 = 150 \times 10^{-6} \times 125 \times 12$$
$$= 0.22 \text{ in} \quad (5.7 \text{ mm})$$

Total movement at each active joint $= d_1 + d_2 + d_3 + d_4 + d_5$
$= 1.17$ in (29.7 mm)

Creep

$$d_5 = 2.5 \times \frac{160 \times 125 \times 12}{5{,}000{,}000}$$
$$= 0.12 \text{ in} \quad (3 \text{ mm})$$

Knowledge of slab end movements is essential to setting initial opening at active joints of gap slabs. It is also important in selecting size of compression seals, width of cover plates, and extensibility requirements of compressive and strip seals.

Average concrete temperature during placement of gap slabs is used to determine the initial width of active joints. From Design 1 calculations it is seen that a joint width set at 93F (34C) would be expected to close 0.38 in (9.7 mm) due to daily summer temperature variation. Extrapolation shows that for a placement temperature of 72F (22C), closure would be 0.76 in (19.4 mm). Thus, joint width set for the latter condition would be double that required for the first.

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A1. Tayabji, S.D., Colley, B.E., and Nussbaum, P.J., "Prestressed Pavement Thickness Design," report prepared by Construction Technology Laboratories for Federal Highway Administration, December 1980.

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APPENDIX B - LABORATORY TESTS

Laboratory tests were conducted to investigate factors that affect design details of prestressed concrete pavements. Tests were conducted to evaluate the following factors:

(1) Effectiveness of friction reducing mediums

(2) Prestress loss due to tendon friction

(3) Fatigue of prestressed concrete

(4) Durability of strand plastic encasements

(5) Effect of bearing plate condition on concrete cracking

(6) Prestress loss due to anchorage slippage

(7) Effect of tendon eccentricity on warping deformations

Test details, results and findings are presented.

EFFECTIVENESS FRICTION REDUCING MEDIUMS

Tests were conducted to measure coefficients of several friction reducing treatments. In addition, tests were conducted to evaluate the effect of repeated slab movements on the durability of a double polyethylene interlayer.

Specimen Preparation and Test Procedure

Econocrete base slabs were cast and finished using either a tube float or wooden screed. After placement of friction reducing treatments, top slabs were cast. Friction reducing treatments tested are listed in Table B1.

Test setup for evaluating effectiveness of friction reducing treatments is shown in Figure B1. A hydraulic jack was used to apply a thrusting force to the top slab. Horizontal movement was measured with a displacement transducer. Load was monitored with a calibrated load cell.

In one test, refrigeration coils embedded in the base slab, as shown in Figure B2, were used to reduce slab temperature to 40 F (4.4 C). Top slab was then raised and water was poured between the polyethylene layers. Base slab temperature was then lowered to permit water to freeze.

Test setup for evaluating durability of a double layer of polyethylene is shown in Figure B3. An electric motor with a connecting rod mounted eccentrically on a speed-reducer shaft was used to apply a push-pull thrusting force to the top slab. Load frequency was maintained at 0.09 cpm. Slab displacement in each loading cycle was 0.35-in (9 mm).

TABLE B1 - COEFFICIENT OF FRICTION FOR DIFFERENT FRICTION REDUCING MEDIUMS

Friction Reducing Medium*	Base Slab Finish**	Coefficient of Friction		
		1st Test	2nd Test	3rd Test
Single Polyethylene	a	0.91	0.64	0.50
Double Polyethylene	a	0.66	0.47	0.47
Double Polyethylene with a Teflon Interlayer	a	0.45	0.43	0.35
Double Polyethylene with a Graphite Interlayer	a	0.56	0.37	0.49
Double Polyethylene	b	0.53	0.36	0.33
Double Polyethylene on a Sand Layer	b	0.33	0.34	0.26
Double polyethylene with an Ice Interlayer	a	1.75	0.27	0.41

*All polyethylene layers were 0.004-in thick

**a = tube float

b = wood screed

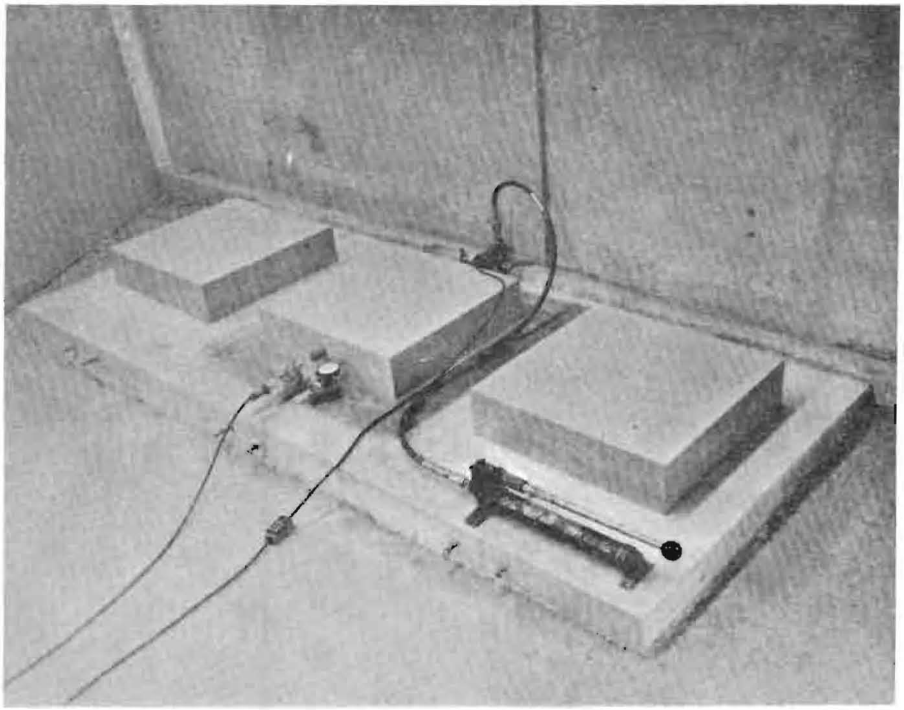


Figure B1 - Interlayer Friction

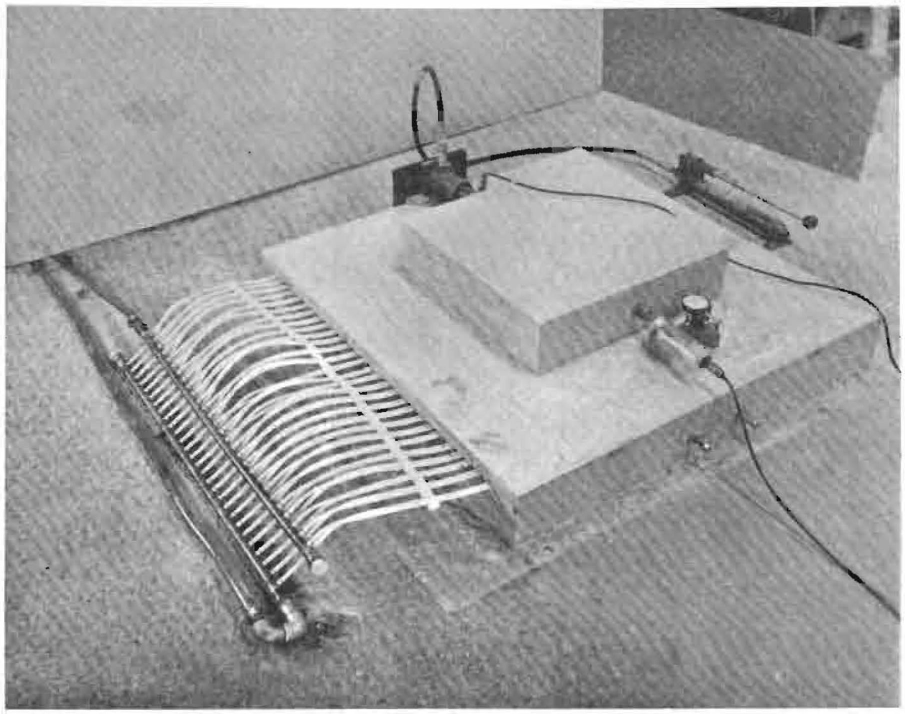


Figure B2 - Ice Formation

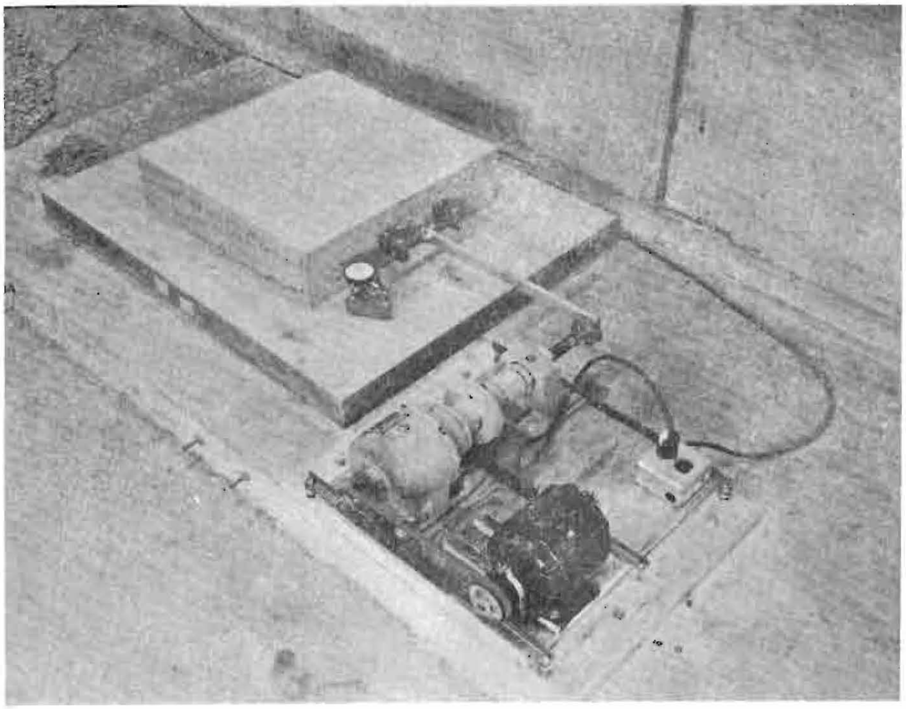


Figure B3 - Repeated Loading

Test Results and Findings

Measured coefficients of friction are listed in Table B1. Based on test results, the following conclusions are made:

(1) A double polyethylene layer provided an initial coefficient of friction ranging from 0.53 to 0.66 depending on subbase surface finish.

(2) Placement of fine sand between subbase and a double polyethylene layer reduced coefficient of friction from 0.53 to 0.33.

(3) A double polyethylene layer withstood over 7,700 displacement cycles without extensive deterioration. Displacement cycles increased coefficient of friction from 0.58 to 0.65.

(4) Ice formation between the two polyethylene layers increased the initial coefficient of friction from 0.66 to 1.75.

PRESTRESS LOSS DUE TO TENDON FRICTION

Prestress losses occur due to strand friction caused by wobble and curvature effects. Wobble friction occurs in straight tendons due to unintended misalignment. Curvature friction occurs due to an intended tendon alignment. Tests were conducted to determine prestress losses due to wobble and curvature effects for four 0.5-in (12.7 mm) diameter strands. These included the following types:

1. Polystrand CP supplied by PIC Inc., encased in a 0.020-in (0.51 mm) thick extruded polypropylene sheath.
2. CCS strand supplied by Concrete Construction Supply Inc., encased in a 0.025-in (0.64 mm) thick extruded polypropylene sheath.
3. Strand encased in a 0.75-in (19.1 mm) diameter rigid metal conduit.
4. Strand encased in a 0.75-in (19.1 mm) diameter flexible metal conduit.

Specimen Preparation and Test Procedure

Slabs were cast with tendons arranged as shown in Figure B4. Frictional losses were measured by load cells attached to both tendon ends. Difference in load cell readings constituted the frictional loss.

Test Results and Findings

Test data, listed in Table B2, indicate that coefficients of curvature friction and wobble depended on type of strand encasement. Values for curvature friction were in general agreement with those recommended by the Post-Tensioning Institute.

Coefficient of curvature friction for strands encased in a rigid metal conduit was 3.3 to 4.2 times that for strands encased in extruded polypropylene sheaths. Free wobble factor for strands encased in a flexible metal conduit was 78 times

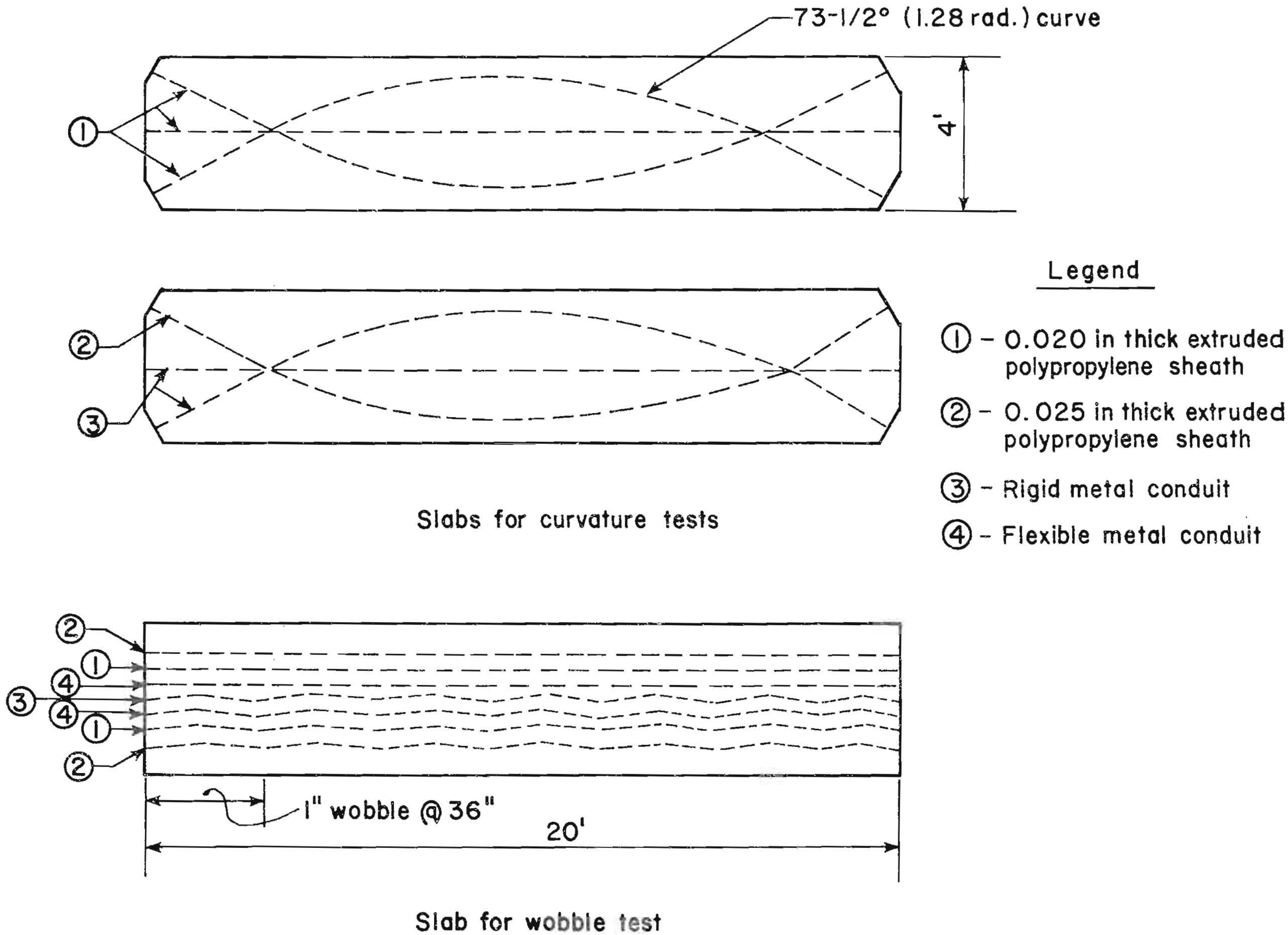


Figure B4 - Tendon Friction Slabs

TABLE B2 - TENDON FRICTION FACTORS

Strand Encasement Type	Curvature Friction	Free Wobble Factor, per foot	Induced Wobble Factor, K, per foot
0.020-in (0.51 mm) Thick Extruded Polyproplylene Sheath	0.036 (0.03-0.05)*	0.00086	0.00140
0.025-in (0.64 mm) Thick Extruded Polyproplylene Sheath	0.045 (0.03-0.05)*	0.00163	0.00102
0.75-in (19.1 mm) Diameter Rigid Metal Conduit	0.149 (0.16-0.30)*	0.00005	0.00212
0.75-in (19.1 mm) Diameter Flexible Metal Conduit	-	0.00389	0.00852

*Values recommended by Post-Tensioning Institute that for strands encased in a rigid metal conduit, and 2.4 to 4.5 times that for strands encased in extruded polypropylene sheaths.

From test results it can be concluded that strands encased in extruded polypropylene sheaths provide lower friction losses than those encased in metal conduits.

FATIGUE OF PRESTRESSED CONCRETE

Tests were conducted on prestressed concrete beams to determine effects of prestress load and loading cycles on concrete fatigue.

Specimen Preparation and Test Procedure

Twenty-eight, 6x6x30-in (15x15x762 mm), concrete beams were made for repeated load tests. Beams were cast with a 1/2-in (12.7 mm) diameter, 7-wire strand centrally positioned. In addition, plain concrete beams were made from each batch to determine flexural strength.

Following a curing period, beams were post-tensioned. Applied force was measured by load cells left in place throughout the test. Prestress force was selected to produce prestress levels of 50, 100, and 150 psi (345, 689, and 1,030 MPa).

Flexural strength was determined from tests on non-prestressed beams tested in third point loading. Repeated load tests were conducted on prestressed beams supported on an 18-in (457 mm) span and loaded at third points. Load was selected to produce a specified factor, F, ranging from 0.7 to 0.9 of beam cracking strength. Cracking strength was the sum of flexural strength, f_r, plus prestress, f_p. Thus, load, P, was calculated from the formula:

$$P = \frac{(f_r + f_p)\ b\ d^2}{L}\ F$$

where L designates test span and b and d designate beam width and depth, respectively.

Repeated load was applied with a hydraulic actuator, as shown in Figure B5. Loading frequency was maintained at 5 cps. A minimum load of 100 to 150 lb (445 to 667 N) was maintained in each load cycle to prevent impact.

Test was automatically terminated when first surface crack was detected. Cracking was detected using a 1/2-in (13 mm) wide strip of aluminum foil bonded to beam bottom surface. Breakage of foil due to crack formation caused equipment stoppage and test termination.

Test Results and Findings

Tests results are summarized in Table B3. Average modulus of rupture was 635 psi (4.38 MPa). Standard deviation was 49 psi (338 MPa) with a coefficient of variation of 7.6%.

Test data are presented in an S-N form in Figure B6. Stress level shown in this figure is the calculated tensile stress due to both prestress force and applied load.

Also shown in Figure B6 are results obtained from studies for plain concrete. These data indicate general agreement between test results and previous work.

Based on test results, the following conclusions are made:

1. The logarithm of number of load cycles sustained prior to failure of a prestressed beam in flexure is inversely proportional to concrete stress. This relationship is similar to that obtained for plain concrete.
2. Prestress level up to about 25% of flexural strength had no evident effect on fatigue life.

DURABILITY OF STRAND PLASTIC ENCASEMENT

Repeated load tests were conducted to evaluate the durability of plastic sheath used for encasing post-tensioning strands. Tests were made using 0.5-in (12.7 mm) diameter, 7-wire strands encased in two types of polypropylene sheath. These included the following:

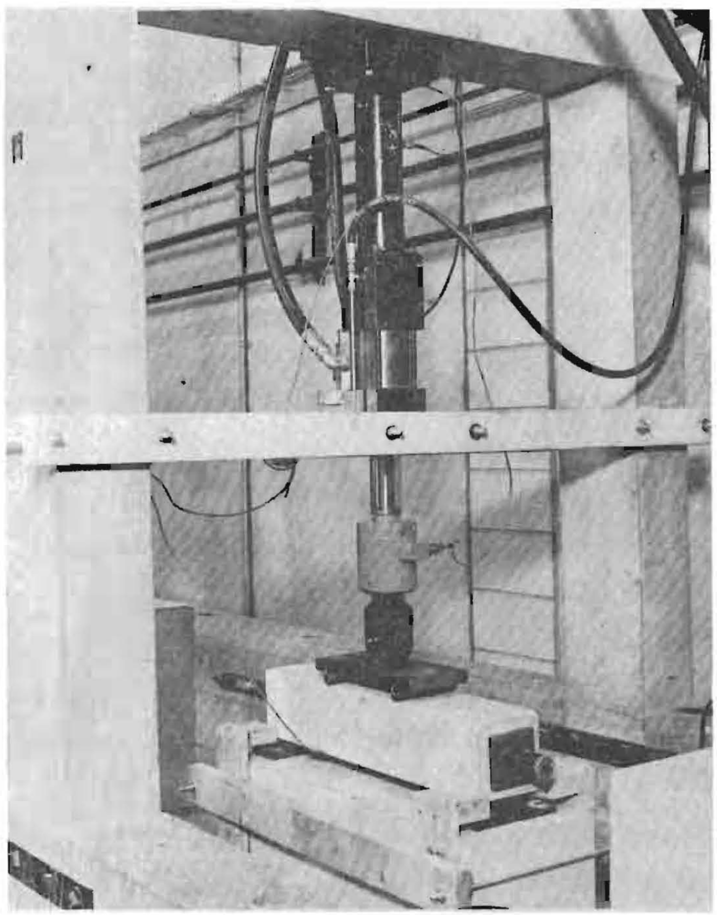
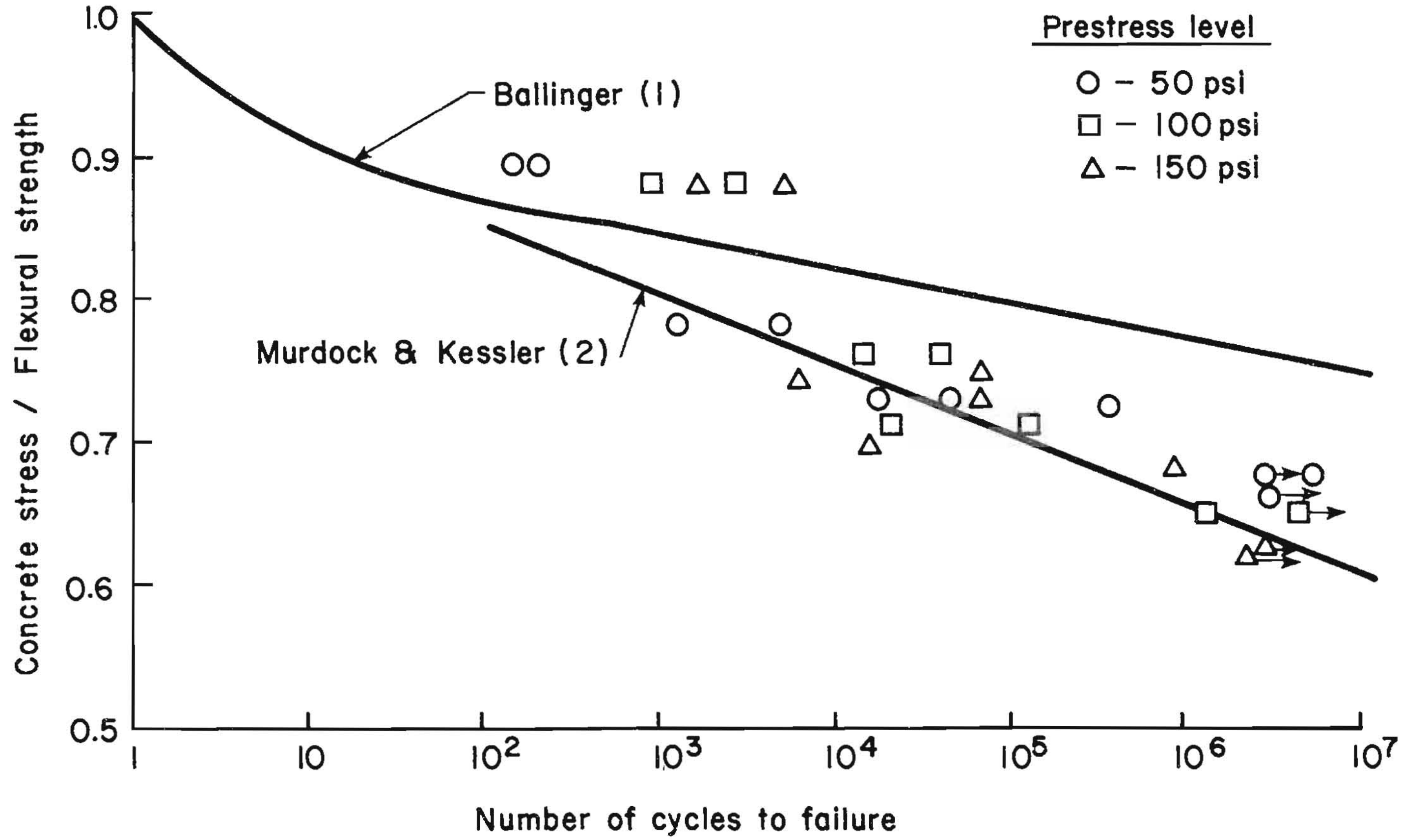


Figure B5 - Fatigue Test

TABLE B3 - FATIGUE TEST RESULTS

Modulus of Rupture, psi	Prestress Level, psi	Repeated Load Stress/Beam Cracking Strength, %	Concrete Stress/Modulus of Rupture, %	No. of Cycles to Failure
646	100	70	65	4,320,000*
653	100	80	76	39,800
590	100	70	65	1,140,000
590	100	80	76	13,300
571	100	90	88	2,300
571	100	90	88	870
695	100	75	71	113,000
692	100	75	71	19,800
550	150	70	62	2,200,000*
550	150	70	62	2,190,000*
625	150	80	75	6,340
567	150	75	68	876,000
661	150	80	75	67,900
672	150	90	88	1,650
672	150	90	88	5,050
717	150	75	70	16,800
717	150	75	70	56,200
609	50	70	68	5,190,000
636	50	80	78	4,230
636	50	80	78	1,050
675	50	90	89	170
675	50	90	89	180
590	50	70	67	2,950,000*
590	50	70	67	2,950,000*
610	50	75	73	17,100
610	50	75	73	57,100
668	50	75	73	42,800
668	50	75	73	337,000

*No failure occurred, test was terminated



(1) Balinger, C. A., "Cumulative Fatigue Damage Characteristics of Plain Concrete," Highway Research Record, No. 370, 1971, pp. 48-61

(2) Murdock, J. W. and Kesler, C. E., "Effect of Range of Stress on Fatigue Strength of Plain Concrete Beams," Journal of the American Concrete Institute, No. 2, Vol. 30, August 1958, pp. 221-233

Figure B6 - Fatigue Test Data

1. Polystrand CP supplied by PIC Inc., encased in a 0.020-in (0.51 mm) thick extruded polypropylene sheath, and
2. CCS strand supplied by Concrete Construction Supply Inc., encased in a 0.025-in (6.4 mm) thick extruded polypropylene sheath.

Specimen Preparation and Test Procedure

Thirteen, 6x6x30-in (152x152x762 mm) specimens were cast each with a prestressing strand centrally placed. Each beam was prepared with a crack inducer placed at mid-depth and extended between prestressing strand and beam bottom surface. Length change plugs were bonded to the beam bottom surface on each side of crack inducer. Strand in each beam was tensioned prior to testing.

Static tests were conducted on beams in third point loading to determine the deflection required to cause a 0.020 or 0.128-in (0.51 or 3.18 mm) crack opening at beam mid-length.

Repeated load tests were conducted on beams loaded in third points. Load was controlled to produce the desired deflection determined from static tests. Load was applied for 1,000 to 5,000,000 load cycles. Following test completion, concrete surrounding strands were carefully removed and plastic sheath was visually examined.

Test Results and Findings

Results of sheath examination after test completion are listed in Table B4. Test results indicated that for a 0.125-in. (3.18 mm) crack opening, cracks were observed on the Plystrand after 1,000 load cycles. However, no cracks were observed on the CCS strand after 11,800 load cycles. Cracks were detected after 23,200 load cycles. For a 0.020-in (0.51 mm) crack opening, no cracks were detected on either strand types after 1.8 million cycles. Cracks were detected on both strand types after 5 million load cycles.

Visual examination of sheath indicated that cracking of the CCS strand sheath was generally wider and longer than that for the Polystrand sheath. The Polystrand generally exhibited a larger number of shorter cracks.

From test results, it can be concluded that damage of strand plastic encasement may occur due to large slab deflections.

EFFECT OF BEARING PLATE DIMENSIONS ON CONCRETE CRACKING

Tests were conducted to determine effect of thickness and area of bearing plate on concrete failure in the anchorage zone during initial prestressing.

Specimen Preparation and Test Procedure

Six, 48x24x8-in (1,219x610x203 mm) specimens were cast. These included a slab cast with an embedded strand anchor at one end. Concrete compressive strength at time of test was determined from tests on 6x12 in (15x30.5 mm) cylinders cast with each slab. Strain gages were bonded to concrete surface to measure effect of applied load on concrete strain.

To perform tests, slab was placed vertically under a loading machine, as shown in Figure B7. A rectangular steel plate was placed on slab end. A steel cylinder, simulating an anchoring chuck, was placed on top of the bearing plate.

Load was applied in 2,000 lb (8.9 kN) increments to a maximum of 40,000 lb (177.9 kN). Strain readings were recorded at each load increment and rate of strain change was calculated. A rapid increase in rate of strain change indicated slab failure.

Tests were conducted using five rectangular steel plates. Plate sizes are listed in Table B5. In addition, a test was made on slab with an embedded single strand anchor type CONA 0.5 CM. Anchor's bearing area was 6-1/4x2-1/2 in (159x57 mm).

TABLE B4 - SHEATH CONDITION AFTER REPEATED LOAD TEST

No. of Cycles	Crack Opening, in	Strand Encasement Type*	Condition After Test Completion
1,000	0.125	a	Surface marring, 1/2" long longitudinal crack
11,800	0.125	b	Surface marring, no cracks
23,200	0.125	b	Surface marring, 3/8" long longitudinal crack
52,700	0.125	a	Surface marring, 1/2" long longitudinal crack
783,700	0.125	a	Surface marring, 1/2" diagonal crack
2,000,200	0.125	a	Surface marring, 1/4 to 1/2" long longitudinal cracks
2,100,000	0.125	b	Surface marring, 1/2 to 1" long longitudinal cracks
1,803,200	0.020	a	Surface marring, no cracks
1,886,800	0.020	b	Surface marring, no cracks
5,000,000	0.020	a	Surface marring, 1/2" long longitudinal cracks
5,000,000	0.020	b	Surface marring, 1-3/4" long longitudinal cracks
5,697,600	0.020	a	Surface marring, 1/4" long crack
5,753,000	0.020	b	Surface marring, 1/8" long transverse crack, 1-1/4" long longitudinal crack

*a = 0.020-in thick extruded polypropylene sheath
b = 0.025-in thick extruded polypropylene sheath

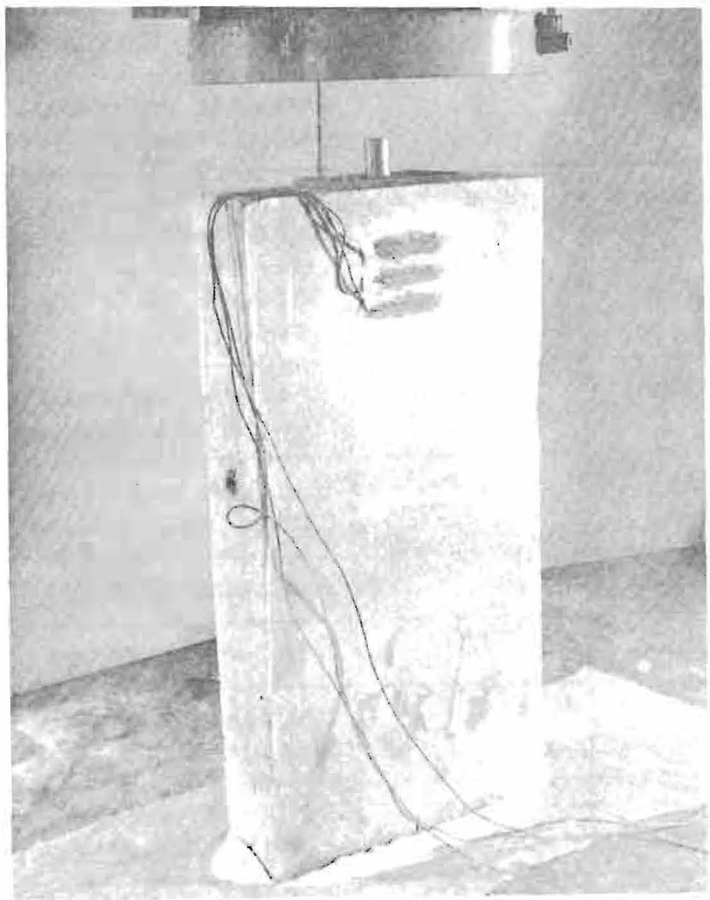


Figure B7 - Bearing Plate Test

Test Results and Findings

Results of compressive strength tests indicated an average compressive strength of 1,210 psi (8.34 MPa). Standard deviation was 178 psi (1.23 MPa) with a coefficient of variation of 14%.

Failure loads measured for the different bearing area conditions are listed in Table B5. Examination of test specimens indicated that slab failure was generally associated with longitudinal cracks occurring in a plane perpendicular to the wider slab side.

Test results shown in Table B5 indicated that failure load increased with an increased bearing plate thickness. For a 6x10-in (152x254 mm) plate, failure load increased by 22 and 36% as plate thickness was increased from 1/4 in (64 mm) to 3/8 and 1/2 (9.5 and 13 mm), respectively.

Also, failure load increased with an increasing plate bearing area. For a 3/8-in (9.5 mm) thick plate, failure load increased by 6 and 9% as plate dimensions were increased from 6x5 in (152x127 mm) to 6x10 and 6x15-in (152x254 and 152x 381 mm), respectively.

Failure load for an embedded single strand anchor type CONA 0.5 CM with a 6-1/2x2-1/2-in (159x57 mm) bearing area was 2% higher than that for a 6x10x3/8-in (152x254x9.5 mm) bearing plates.

From test results it can be concluded that plate thickness has a larger effect of bearing capacity than plate size. With a 6x10x1/4-in (152x254x6.4 mm) bearing plate, an initial post-tensioning of 10 kips (44.5 kN) can be applied when concrete compressive strength reaches 1,200 psi (8.27 MPa) with a safety factor of 1.48.

PRESTRESS LOSS DUE TO ANCHORAGE SLIPPAGE

Tests were conducted to determine prestress loss caused by anchorage slippage due to repeated loading. A nut-type and

TABLE B5 - EFFECT OF BEARING CONDITION ON FAILURE LOAD

Bearing Condition	Failure Load, lb
Plate, 6x10x1/2 in	20,200
Plate, 6x10x3/8 in	18,000
Plate, 6x10x1/4 in	14,800
Plate, 6x5x3/8 in	17,000
Plate, 6x15x3/8 in	18,600
Anchor, CONA® 0.5 CM (6-1/4x2-1/2 in)	18,300

wedge-type anchorage intended for use with smooth bar and strand, respectively, were tested.

Specimen Preparation

A slab was built with two prestressing tendons spaced 21 in (533 mm) apart. These were a 0.5-in (12.7 mm) diameter, Grade 270 (1.86 GPa), seven wire strand and a 0.75-in (19.1 mm) diameter smooth bar. Slab was supported on a 3-in (76 mm) thick neoprene rubber base.

Tendons were tensioned, 14 days after casting, to 30 kips (133 kN). Prestressing force was applied with hydraulic jacks and monitored with load cells that were attached to the tendons throughout the test period.

Concrete compressive and flexural strengths were determined from specimens tested 14 days after casting.

Test Procedure

Static tests were conducted to determine magnitude of load required to cause a 0.045-in (1.14 mm) deflection. Load was applied at one end through a 16-in (406-mm) wide, 1-in (25.4-mm) thick steel plate. A tie-down system was used to prevent slab uplift of the unloaded end.

A repeated load test was performed. Load magnitude was that determined from static tests. A total of 5 million load cycles were applied. Tendon prestress was determined during the test from load cell readings. Test set-up is shown in Figure B8.

Following repeated load test, tendons were extracted from slab and examined.

Test Results and Findings

Compressive and flexural strengths obtained from concrete specimens were 5,200 and 820 psi (35.9 and 5.7 MPa), respectively.

Static tests showed that load required to cause a 0.045-in (1.14 mm) deflection was 10 kips (44.5 kN).

Data obtained during repeated load test indicated that prestress losses occurred during 5 million load cycles were 3.3 and

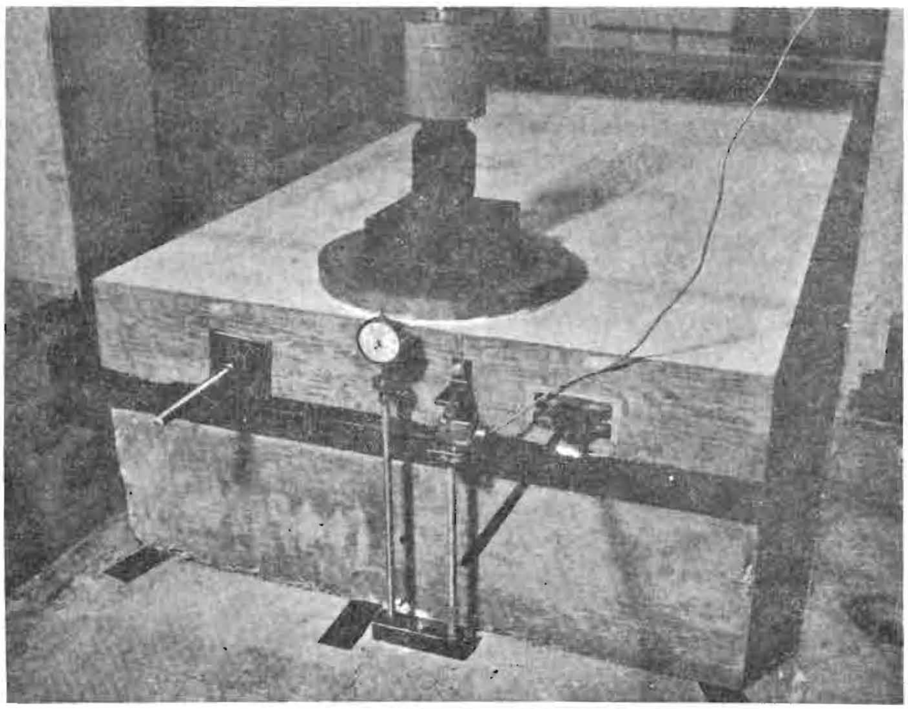


Figure B8 - Anchorage Slippage

2.4% for smooth bar and strand, respectively. This loss may be attributed to tendon relaxation during the 20-day test period.

Examination of tendons after test completion revealed no signs of distress on the smooth bar. However, a notch occurred on the strand in the anchorage area. To determine effect of this notch on strand strength, tensile strength tests were conducted on a notched and new strand. Test results showed that load-elongation curves were essentially identical for both strands.

From test results it can be concluded that prestress loss due to slippage of nut-type and wedge-type anchorages was insignificant after 5 million load applications.

EFFECT OF TENDON ECCENTRICITY ON WARPING DEFORMATION

Warping and curling deformations occur at slab ends due to temperature and moisture gradients. Tests were conducted to determine effect of tendon eccentricity on these deformations.

Test Specimens and Procedure

Two slabs were cast with two, 0.5-in (12.7 mm) diameter, seven wire plastic coated strands placed 1 in (25 mm) below slab mid-depth as shown in Figure B9. After casting, curing compound was applied to slab sides to permit drying from the top only.

Corner deflections were monitored with dial gages attached to reference rods that were buried into the subgrade.

To determine effect of an eccentrically-positioned prestressing on slab warping, one slab was post-tensioned in two stages. A 10 kip (44.5 kN) prestress force was applied 5 hours after casting. Force was increased to 30 kips (133 kN) 26 hours after casting and maintained until test termination 90 days after casting. No prestress was applied to the other slab.

During test duration, corner deflection of both nonprestressed and post-tensioned slab were measured. These data are shown in Figure B10.

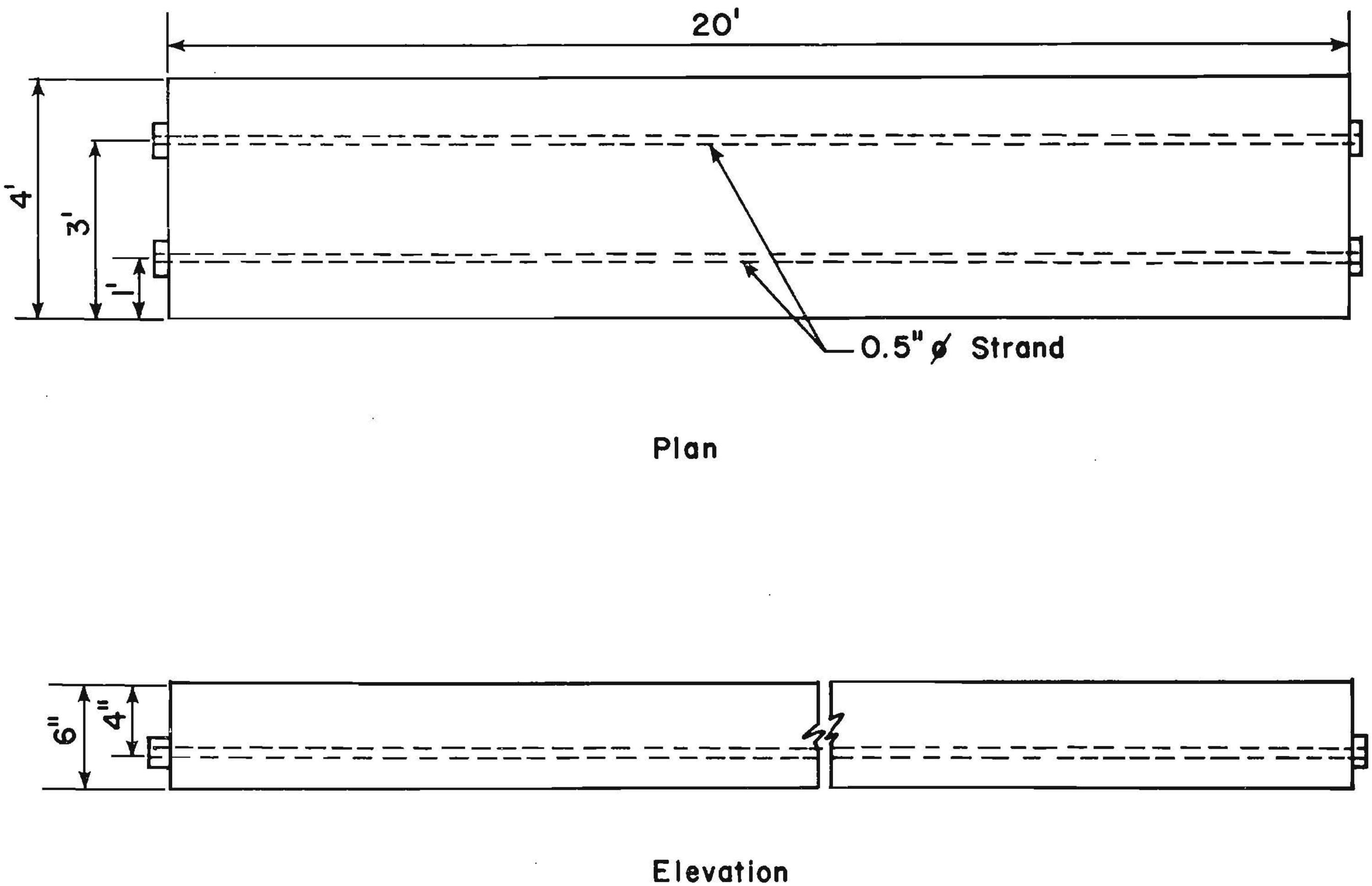


Figure B9 - Tendon Eccentricity Slab

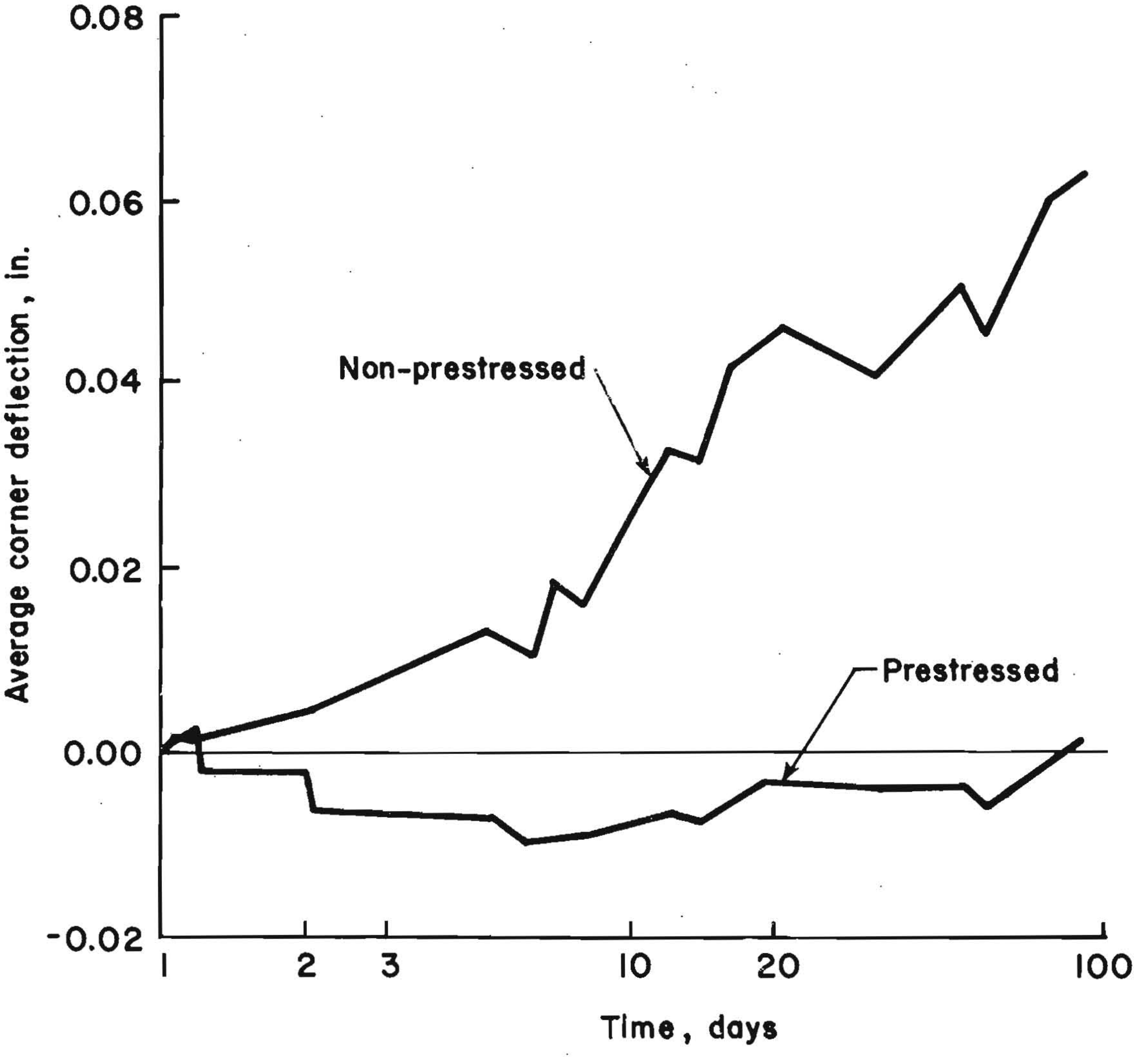


Figure B10 - Deflection Data

Test data indicate that application of prestress force below slab mid-depth reduced slab upward warping. Reduction in upward warping ranged from 0.04 in (1 mm) after 6 days to 0.09 in (2 mm) after 90 days. Therefore, it can be concluded that placement of prestressing tendons below slab mid-depth contributes to a reduction in slab upward warping.

FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

The Offices of Research and Development (R&D) of the Federal Highway Administration (FHWA) are responsible for a broad program of staff and contract research and development and a Federal-aid program, conducted by or through the State highway transportation agencies, that includes the Highway Planning and Research (HP&R) program and the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board. The FCP is a carefully selected group of projects that uses research and development resources to obtain timely solutions to urgent national highway engineering problems.*

The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion, and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements that affect the quality of the human environment. The goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge and technology of materials properties, using available natural materials, improving structural foundation materials, recycling highway materials, converting industrial wastes into useful highway products, developing extender or substitute materials for those in short supply, and developing more rapid and reliable testing procedures. The goals are lower highway construction costs and extended maintenance-free operation.

5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural and hydraulic designs, fabrication processes, and construction techniques to provide safe, efficient highways at reasonable costs.

6. Improved Technology for Highway Construction

This category is concerned with the research, development, and implementation of highway construction technology to increase productivity, reduce energy consumption, conserve dwindling resources, and reduce costs while improving the quality and methods of construction.

7. Improved Technology for Highway Maintenance

This category addresses problems in preserving the Nation's highways and includes activities in physical maintenance, traffic services, management, and equipment. The goal is to maximize operational efficiency and safety to the traveling public while conserving resources.

0. Other New Studies

This category, not included in the seven-volume official statement of the FCP, is concerned with HP&R and NCHRP studies not specifically related to FCP projects. These studies involve R&D support of other FHWA program office research.

* The complete seven-volume official statement of the FCP is available from the National Technical Information Service, Springfield, Va. 22161. Single copies of the introductory volume are available without charge from Program Analysis (HRD-3), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.