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16. Abstract

Although sections of prestressed concrete pavements were first built 30 years ago, the techniques of design and construction are still undergoing development. This report summarizes the principal developments in this field since the first prestressed pavement was constructed at the Paris International Airport at Orly in 1946.

The review covers papers, articles and reports published in the technical and society proceedings of many countries. The report describes the history of prestressed concrete pavements, and the features of prestressed concrete pavements constructed in various parts of the world, prestressing methods used on different projects, joint details used on various projects, particularly for those built in the U.S., information on performance, load tests and measurements on prestressed concrete pavements, methods used for designing prestressed concrete pavements, and properties of materials used for prestressed concrete pavements.

The report includes a glossary of terms related to prestressed concrete pavements and a comprehensive bibliography.

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METRIC CONVERSION FACTORS

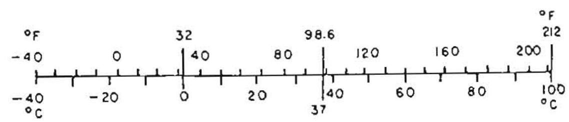
Approximate Conversions to Metric Measures

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	0.8	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

*1 in ≡ 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

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	0.03	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	35	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



INTRODUCTION

Although sections of prestressed concrete pavements were first built 30 years ago, the techniques of design and construction are still undergoing development. This report summarizes the principal developments in this field since the first prestressed pavement was constructed at the Paris International Airport at Orly in 1946.

The review covers papers, articles, and reports published in the technical press and society proceedings of many countries. Findings of the review are summarized in six sections.

Section 1 presents the history of prestressed concrete pavements, and the features of prestressed concrete pavements constructed in various parts of the world.

Section 2 outlines prestressing methods used on different projects.

Section 3 describes joint details used on various projects, particularly for those built in the U.S.

Section 4 presents information on performance, load tests, and measurements on prestressed concrete pavements.

Section 5 outlines some of the methods used for designing prestressed concrete pavements.

Section 6 identifies the properties of materials used for prestressed concrete pavementes including concrete, steel, and subbases.

The report includes a glossary of terms related to prestressed concrete pavements. In addition, a comprehensive bibliography is presented in two parts. Part A includes references pertinent to prestressed concrete pavements. Part B lists references pertinent to materials including concrete, steel, and subbases.

HISTORICAL REVIEW

The application of compressive stress to concrete by tensioning reinforcement appears in patent literature of

1888. In 1925, a U.S. patent mentions the use of high-strength steel bars, coated to prevent bond, and tensioned after hardening of the surrounding concrete.

In 1928 in France, E. Freyssinet began the use of high-strength cold-drawn steel wire for prestressing concrete. The wires were tensioned to near yield stress to provide a residual compressive stress in the concrete after shrinkage. The use of wedge anchors for prestressing steel, tensioned with hydraulic jacks, was reported in 1939. These early developments in prestressed concrete centered principally in France and Belgium were followed by widespread applications for bridge and building reconstruction after 1945.

Successful application of circular prestressing of round water tanks with threaded structural rods was begun by W.S. Hewett in 1918. The application was adequate as long as the tanks were filled with water. However, tanks left empty for some time lost their prestress due to shrinkage deformation and plastic flow in the concrete, and relaxation of the steel. This led to the use of wire for tank prestressing.

Prestressed concrete pavements were investigated in England in 1943. They were given their first field application at Orly Airport in France in 1946[(105)*]. This was followed by airport runway installations in Belgium 1947[(150)], England 1949[(94)], Holland 1951[(134)], and larger installations in France 1953[(72)], and in Algiers in 1954-55[(88)].

Prestressed concrete highway applications began with two short pavements in France 1946 and 1949. This was followed by British projects totaling 6,000 ft that were constructed during the period 1950 to 1952[(94)]. These projects included post-tensioned diagonal, longitudinal, and longitudinal and transverse prestressing. Fixed length prestressing between heavy abutments, first used on exploratory runway projects in France, at Orly with diagonal movable joints, was used in France on a highway at Bourg-Servas in 1953. These projects were poststressed with flat jacks.

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* Subscript numbers in parenthesis designate references in bibliography.

This was followed by an exploratory highway project in 1959 at Fontenay-Tresigny that used a number of prestressing methods. Also in 1959, fixed length pavements were constructed at Zwartberg-Meeuwen in Belgium.

A limited use of poststressing in highway construction occurred on some 10 miles (16 km) of pavements in Missouri, Michigan and Maryland between 1937 and 1941. Prestress to about 200 psi (1.4 MPa) was applied with pneumatic cells in narrow expansion joints. Stress was applied at the earliest possible age, with pressure increased with concrete strength. The prestress was removed after the concrete reached mature strength.

Other limited applications of prestress in concrete pavements were a 7-in. (178-mm) thick concrete test slab at Patuxent River NAS, Maryland, 1953[66] and a 4-in. (102-mm) overlay slab at San Antonio Airport, Texas, 1955[171]. The Corps of Engineers' 1955 tests at Sharonville, Ohio[122], were followed by taxi-ways at Biggs AFB, Texas[195], and Lemoore NAS, California in 1959.

The first known prestress application on highways in the U.S. was a short pavement slab in Delaware built in 1971[272]. This was followed the same year by a 3200-ft (975.4-m) demonstration and research project at Dulles International Airport[275]. In 1973 a 2.5-mile (4-km) demonstration project was constructed in Pennsylvania[288]. These projects had been preceeded by a construction and testing program on an experimental prestressed pavement constructed in 1956 at Pittsburgh, Pennsylvania[145].

Features of prestressed highway and airport projects completed in various parts of the world to 1960 were compiled by the Transportation Research Board[239]. These compilations are shown in Tables 1 and 2. Table 3 summarizes the features of the four demonstration projects built in the U.S. since 1971. The following is a general discussion of some of the principal features of those projects. The discussion is separated into the categories of highway and airport pavements.

TABLE 1 - PRESTRESSED CONCRETE HIGHWAY PAVEMENTS

Country	Date	Location	Pavement Dimension Length (ft)	Width (ft)	Thickness (in.)	Prestress (psi) Long.	Trans.	Prestressing System
Austria	1956	Vienna	427	24	8	228	107	Posttensioned both directions.
	1958	Strosshof	164	37	6	235	142	Poststressed longitudinally, posttensioned transversely.
	1958-9	Anif-Saltzburg	2,625 2,625	25 25	6.3 8	1,138 995	None do.	Poststressed with BBRV wedges. Poststressed with Leonhardt wedges.
Belgium	1959	Between Zwartberg and Meeuwen	11,484	23	3.2, 4, 4.7	284, 427, 569	107, 237	Poststressed longitudinally with flat jacks, posttensioned transversely with steel strands.
Gt. Britian	1950	Crawley, Sussex	404	24	6	212	23	Posttensioned diagonally with cables at 18 1/2^{o} to CL of pavement.
	1951	St. Leonards, Hampshire	1,200 (3 400' slabs)	24	6	280 (nominal)	13	Posttensioned both directions with cables.
	1951	Wexham Springs, Buckinghamshire	110 130 190	10 10 11 1/2	6 6 6	190 90 245	None None None	Posttensioned longitudinally with cables.
	1951	John Laing's, Ltd.	3,000 (15 180' slabs) (2 150' slabs)	 12 14 2/3	 10 6	 410 300	 None None	Posttensioned longitudinally with cables.
	1952	Basildon, Essex	660 (4 165' slabs)	18	6	280	None	Posttensioned longitudinally with cables.
	1952	Woolwich	3,350	18-24	6	250	28	Posttensioned diagonally with cables.
	1954	Port Talbot, S. Wales	1,500 (5 300' slabs)	22	6	220-320	0-35	Posttensioned longitudinally with cables, and one slab gap jacked.
	1954	South Benfleet, Essex	330	20	4	550	50	Poststressed longitudinally with flat jacks, posttensioned transversely with wires.

TABLE 1 - PRESTRESSED CONCRETE HIGHWAY PAVEMENTS (cont.)

			Pavement Dimension			Prestress (psi)		
Country	Date	Location	Length (ft)	Width (ft)	Thickness (in.)	Long.	Trans.	Prestressing System
France	1946	Luzancy	6, 81	19 2/3	6.3 edges 7.9 CL	224-300	242-300	Posttensioned diagonally with cables at 45^{o} to CL of pavement.
	1949	Esbly	160	19 2/3	5.9	228	228	Posttensioned diagonally with cables at 45^{o} to CL of pavement.
	1953	Bourg-Servas	984	23	4.7	585 Avg. (initial)	Variable	Poststressed longitudinally with flat jacks, posttensioned transversely with cables, reinforcing bars also used.
	1959	Fontenay-Tresigny	7,776	23, 24.5	4.7, 5.9 (thickened edges)	Variable	Variable	Poststressed and posttensioned longitudinal prestress applied by 11 different methods.
Germany	1953	Heidenheim	365 365 365	28 28 28	6 6 6 at CL 8 at edges	300 240 374	128 43 125	Posttensioned both directions with cables. Posttensioned diagonally with cables at 30^{o} to CL of pavement.
	1954	Speyer	558	20	8	85-341	None	Posttensioned longitudinally with Baur-Leonhardt cables.
	1954	Montabaur	460	25	3.35	460	460	Posttensioned both directions with steel bars.
	1954	Margelstetten	788	25	6	299	29-142	Posttensioned longitudinally with Baur-Leonhardt cables, and transversely with Bauer system.
			394	25	6	370	128	Posttensioned diagonally with Wayss and Freytag system.
	1957	Wolfsburg	5,906	30	6.3	356-498	185	Posttensioned both directions with cables.
	1959	Dietersheim	2,953 (6 492' slabs)	25	6.3	455-654	213	Posttensioned both directions with Held and Franke system.
Netherlands	1957	The Hague	3 slabs 312-377	24	4.7	230	None	Pretensioned longitudinally with wires, reinforced transversely with mild steel bars.
	1960	The Hague	328	28 1/2	4.7	384	None	Pretensioned longitudinally with strands.

TABLE 1 - PRESTRESSED CONCRETE HIGHWAY PAVEMENTS (cont.)

Country	Date	Location	Pavement Dimension Length (ft)	Width (ft)	Thickness (in.)	Prestress (psi) Long.	Trans.	Prestressing System
Italy	1957	Casena	1,641	24.5	3.2	355 at 20°C	355 at 20°C	Pretensioned both directions with steel cables.
Japan	1958	Osaka City	197	18	6	495	108	Posttensioned longitudinally with cables transversely with bars.
	1958	Osaka City	134	36	6	426	158	Posttensioned diagonally with cables at 30° to CL of pavement.
Switzerland	1955	Naz	1641, 1096	8.2	4.7	None at 0°C	--	Poststressed longitudinally with Freyssinet jacks, posttensioned transversely with steel strands.
	1957	Moricken-Brunegg	6,575 (200' - 400' 600' slabs)	18	5	840-1330	None	Poststressed longitudinally with transverse wedges.
	1960	Boudry	4,265	34.5	6	None at 10°C	None	Poststressed longitudinally with flat jacks.
United States	1956	Pittsburgh, Pa. (Jones and Laughlin Steel Co., for tests only)	400 100 30	12	5	450	None	Posttensioned longitudinally with wire strands by gap-jack method.

1 ft = 0.3048 m
1 in. = 25.4 mm
1 psi = 6.895 kPa

TABLE 2 - PRESTRESSED CONCRETE AIRPORT PAVEMENTS

Country	Date	Location	Pavement Dimension Length (ft)	 Width (ft)	 Thickness (in.)	Prestress (psi) Long.	 Trans.	Prestressing System
Algeria	1954-5	Maison Blanche Algiers	8,000 7,700	197 82	7.1 7.1	250 (net) 250 (net)	250 (net) 250 (net)	Poststressed longitudinally with flat jacks, posttensioned transversely with cables.
Austria	1954	Schwechat, Vienna	656	197	8	213	107	Posttensioned both directions with cables.
	1959	Schwechat, Vienna	3,280 3,609 1,425	147 1/2 74 172	6 6 6	227 by pretensioning. 427 by poststressing.(initial)	142 142 142	Pretensioned longitudinally with wire strands, then poststressed with flat jacks. Posttensioned transversely with wire strands.
Belgium	1947	Melsbroek, Brussels	243	164	6.3	--	--	Precast slab posttensioned both directions with cables.
	1958	Melsbroek, Brussels	1,148	75	4	620 (center area), 810,952(edge area)	470	Precast pretensioned slabs in forms of parallelograms, posttensioned transversely with cables.
	1959	Melsbroek, Brussels	10,820	148	7 (12 at active joints	--	--	Poststressed longitudinally with flat jacks, posttensioned transversely with cables.
England	1949	London	343	120	6.5	550	550	Precast square slabs divided into triangular pavement sections by diagonal joints; posttensioned transversely with cables longitudinally by reaction.
	1956	Finningley	200	200	6	250	250	Precast slabs 30'x 9', posttensioned both directions with cables.
	1958	Gatwick	290	230	5	300-350	250	Posttensioned both directions with cables.
	1959	Gatwick	150	132	6.5	100+(net)	100-(net)	Posttensioned both directions with steel bars.

TABLE 2 - PRESTRESSED CONCRETE AIRPORT PAVEMENTS (cont.)

Country	Date	Location	Pavement Dimension Length (ft)	Width (ft)	Thickness (in.)	Prestress (psi) Long.	Trans.	Prestressing System
France	1946-7	Orly, Paris	1,312	197	6.3	470	470	Precast square slabs divided into triangular pavement sections by diagonal joints; post-tensioned transversely with cables, longitudinally by reaction.
	1953	Orly, Paris	1,410	82	7.1	250 (net)	250 (net)	Poststressed longitudinally with flat jacks.
Germany	1956	Memmingen	1,135	98	5.5	--	--	Posttensioned both directions with cables.
	1959	Wunstorf	8,104	98	6	165	165	Posttensioned both directions with cables.
	1959	Diepholz	6,234	98	6	--	--	Posttensioned both directions with steel bars.
	1959	Hopsten	9,810	98	5.5	165	165	Posttensioned both directions with cables.
	1960	Nordholz	9,810	147	6	165	165	Posttensioned both directions with cables.
	1960	Wahn, Cologne	12,468 12,468	197 75	7.1 (interior) 7.9 (ends) 7.1	284 (slab ends) 214 (slab center)	--	Posttensioned both directions with cables.
Netherlands	1951	Schiphol	1,140 (5 92' slabs) (5 136' slabs)	136	5.5	510-570	510-570	Posttensioned both directions with cables.
New Zealand	1958	Woodbourne	450 (3 150' slabs)	150	6	305	305	Posttensioned both directions with cables.

TABLE 2 - PRESTRESSED CONCRETE AIRPORT PAVEMENTS (cont.)

Country	Date	Location	Pavement Dimension: Length (ft)	Pavement Dimension: Width (ft)	Pavement Dimension: Thickness (in.)	Prestress (psi): Long.	Prestress (psi): Trans.	Prestressing System
United States	1953	Patuxent Naval Air Station, Md. (Special tests only)	500	12	7	690	Only in areas	Posttensioned longitudinally with cables.
	1955	San Antonio, Texas	80	75	4	425	425	Posttensioned both directions with cables.
	1955	Sharonville, Ohio Corps of Eng. (Special tests only)	80 65	75 60	4 4	175 340	175 310	Pretensioned both directions with wires.
	1957	Sharonville, Ohio Corps of Eng. (Special tests only)	500	50	9	200-400	200-400	Posttenioned both directions with steel bars.
	1959	Biggs Air Force Base, Texas	1,500 (3 500' slabs)	75 (3 25' slabs)	9	350	175	Posttensioned both directions with cables.
	1959	Le moore Naval Air Station, Cal.	568	75	6 thickened to 9 at outside edges.	--	--	Posttensioned both directions with cables.

1 ft = 0.3048 m
1 in. = 25.4 mm
1 psi = 6.895 kPa

TABLE 3 - PRESTRESSED CONCRETE DEMONSTRATION PROJECTS IN THE U.S.

Date	Location	Pavement Dimensions Length (ft)	Width (ft)	Thickness (in.)	Prestress (psi) Long.	Trans.	Prestressing System
1971	Milford, Delaware	300	14	6	238	None	Post-tensioned longitudinally with 0.6 in. dia. Teflon coated 7-wire strands (transverse reinforcement
1971	Dulles International Airport, Virginia	400 500 600 760 500 400	24	6	200	None	Post-tensioned longitudinally with 1/2 in. dia. 7-wire steel strands (1/2 in. below mid-depth)
1973	Harrisburg, Pennsylvania	23 slabs approx. 600 ft (average)	24	6	331	None	Post-tensioned longitudinally with 0.6 in. dia. 7-wire strands
1973	Kutztown, Pennslyvania	500	24	6	331	None	Post-tensioned longitudinally with 0.6 in. dia. 7-wire strands in polypropylene conduit

1 ft = 0.3048 m
1 in. = 25.4 mm
1 psi = 6.895 kPa

Highway Pavements

As Table 1 indicates, pretensioned, post-tensioned, and poststressed pavements have been used in various parts of the world. However, as shown in Table 3, only post-tensioned pavements were used in highway projects in the U.S.

Among the 34 highway projects listed in Table 1, 22 pavements were post-tensioned, 4 poststressed, 5 post-stressed and post-tensioned, and 3 pretensioned. For discussion, these projects are categorized in terms of method of longitudinal prestressing.

Post-Tensioned

General features of post-tensioned highway pavement projects built to 1960 are:

(a) Post-tensioning was accomplished by longitudinal prestressing only, longitudinal and transverse prestressing, diagonal prestressing only, or longitudinal and diagonal prestressing.

(b) Slab length varied from 30 to 492 ft (9 to 150 m).

(c) Slab thickness varied from 3.35 to 10 in. (85 to 254 mm).

(d) Longitudinal prestress ranged from 90 to 654 psi (90 to 2448 kPa).

(e) Where transverse prestressing was used, it ranged from 13 to 355 psi (90 to 2448 kPa).

(f) Where diagonal prestressing was used, it varied from 212 to 374 psi (1.5 to 2.6 MPa).

(g) Cables, strands, or steel bars were used for prestressing.

The highway demonstration projects in the U.S. have the following features:

(a) Slab length varied from 300 to 760 ft (91 to 232 m).

(b) Slab thickness was 6 in. (152 mm) on all projects.

(c) Longitudinal prestress varied from 200 to 331 psi (1.4 to 2.3 MPa).

(d) No transverse or diagonal prestressing was used.

(e) Only 7-wire steel strands were used on all projects.

Pretensioned

General features of pretensioned highway pavement projects built in 1960 are:

(a) Pretensioning was accomplished by two different ways: longitudinal prestress only, or longitudinal and transverse prestressing.

(b) Slab length varied from 164 to 377 ft (50 to 115 m).

(c) Slab thickness varies from 3.2 to 4.7 in. (81.3 to 119.4 mm).

(d) Longitudinal prestress varied from 230 to 384 psi (1.6 to 2.6 MPa).

(e) Where transverse prestressing was used, it amounted to 355 psi (2.4 MPa).

(f) Wires or strands were used for prestressing.

Poststressed

General features of poststressed highway pavement projects built to 1960 are:

(a) Prestressing was accomplished by poststressing longitudinally only or by longitudinal poststressing and transverse post-tensioning.

(b) Slab length varied from 164 to 365 ft (50 to 111 m).

(c) Slab thickness varied from 3.2 to 6.0 in. (81.3 to 152.4 mm).

(d) Longitudinal prestress varied from 235 to 1138 psi (1.6 to 7.8 MPa).

(e) Where transverse post-tensioning was used, it varied from 50 to 273 psi (345 to 1882 kPa).

(f) Poststressing was accomplished with flat jacks or wedges.

Airport Pavements

Prestressed concrete pavements have been constructed at a number of airports in the U.S. and abroad. As in the case of prestressed highway pavements, most of the airfield installation have been post-tensioned.

Among the 26 projects listed in Table 2, 19 pavements were post-tensioned, 5 poststressed longitudinally and post-tensioned transversely, and 2 pretensioned longitudinally and post-tensioned transversely. For discussion, these projects are categorized in terms of method of longitudinal prestressing.

Post-Tensioned

General features of post-tensioned airport pavement projects built to 1960 are:

(a) In all projects except one at the Patuxent Naval Air Station in Maryland, prestressing was accomplished by longitudinal and transverse post-tensioning.

(b) Slab length varied from 80 to 623 ft (24 to 190 m).

(c) Slab thickness varied from 5.5 to 9.0 in. (140 to 229 mm).

(d) Longitudinal prestress ranged from 165 to 570 psi (1.1 to 3.9 MPa).

(e) Transverse prestress ranged from 107 to 570 psi (738 to 3930 kPa).

(f) Cables, strands, and steel bars were used for post-tensioning.

(g) In a number of projects, precast slabs were post-tensioned in place.

Pretensioned

General features of the two pretensioned airport pavement projects built to 1960 are:

(a) On the project at Sharonville, Ohio, pavement was pretensioned in both directions with wires.

(b) On the project at Schwechat, Vienna in Austria, prestressing was accomplished by pretensioning longitudinally with strands, then poststressing with flat jacks. Transverse post-tensioning with strands was used.

(c) Slab length varied from 65 to 412 ft (20 to 126 m).

(d) Slab thickness varied from 4 to 6 in. (102 to 152 mm).

(e) Longitudinal prestress ranged from 227 to 310 psi (1.6 to 2.1 MPa).

Poststressed

General features of poststressed airport pavement projects built to 1960 are:

(a) In all projects, longitudinal poststressing and transverse post-tensioning was used.

(b) In two projects, the pavement consisted of precast slabs that were prestressed together in place.

(c) Slab length varied from 343 to 1100 ft (105 to 335 m).

(d) Slab thickness varied from 6 to 7.1 in. (152.4 to 180.3 mm).

(e) Longtitudinal prestress ranged from 250 to 550 psi (1.7 to 3.8 MPa).

(f) Transverse prestress ranged from 250 to 550 psi (1.7 to 3.8 MPa).

PRESTRESSING METHODS AND TECHNIQUES

Pavements have been stressed longitudinally, longitudinally and transversally, or diagonally by the application of one or more of the systems of prestressing. The systems include pretensioning, post-tensioning, and poststressing. Pretensioning was used only on a few projects in the Netherlands, Italy, Austria, and Portugal. Poststressing was used on some of the early largest pavement projects in Europe. Post-tensioning has been most frequently used. All prestressed concrete pavement projects built in the U.S. were post-tensioned except that in Sharonville, Ohio.

Pretensioning

The pretensioning system of prestressing uses tendons consisting of wires, strands, or bars placed at mid-depth of a pavement. The tendons are attached to anchors or abutments and pulled by mechanical means until a predetermined stress has been attained in the steel. Next, the concrete is placed. After it has reached a specified compressive strength the tendons are released. The tensile stress in the tendons is then transferred as compressive stress to the concrete through bond.

Anchorages for pretensioning tendons have been provided by different means. In an experimental project at Leidschendam in the Netherlands[218], the wires were tensioned at the pavement ends. For this purpose, prestressed concrete beams were installed across the pavement with steel pipe reactive supports as shown in Fig. 1 installed longitudinally on each side.

In another project at Haarlemmermeer in the Netherlands[226], strands with lengths of 1100 ft (335 m) were tensioned between abutments in one operation. Each abutment consisted of a frame of concrete beams and a top slab as shown in Fig. 2. Concrete was placed continuously in one

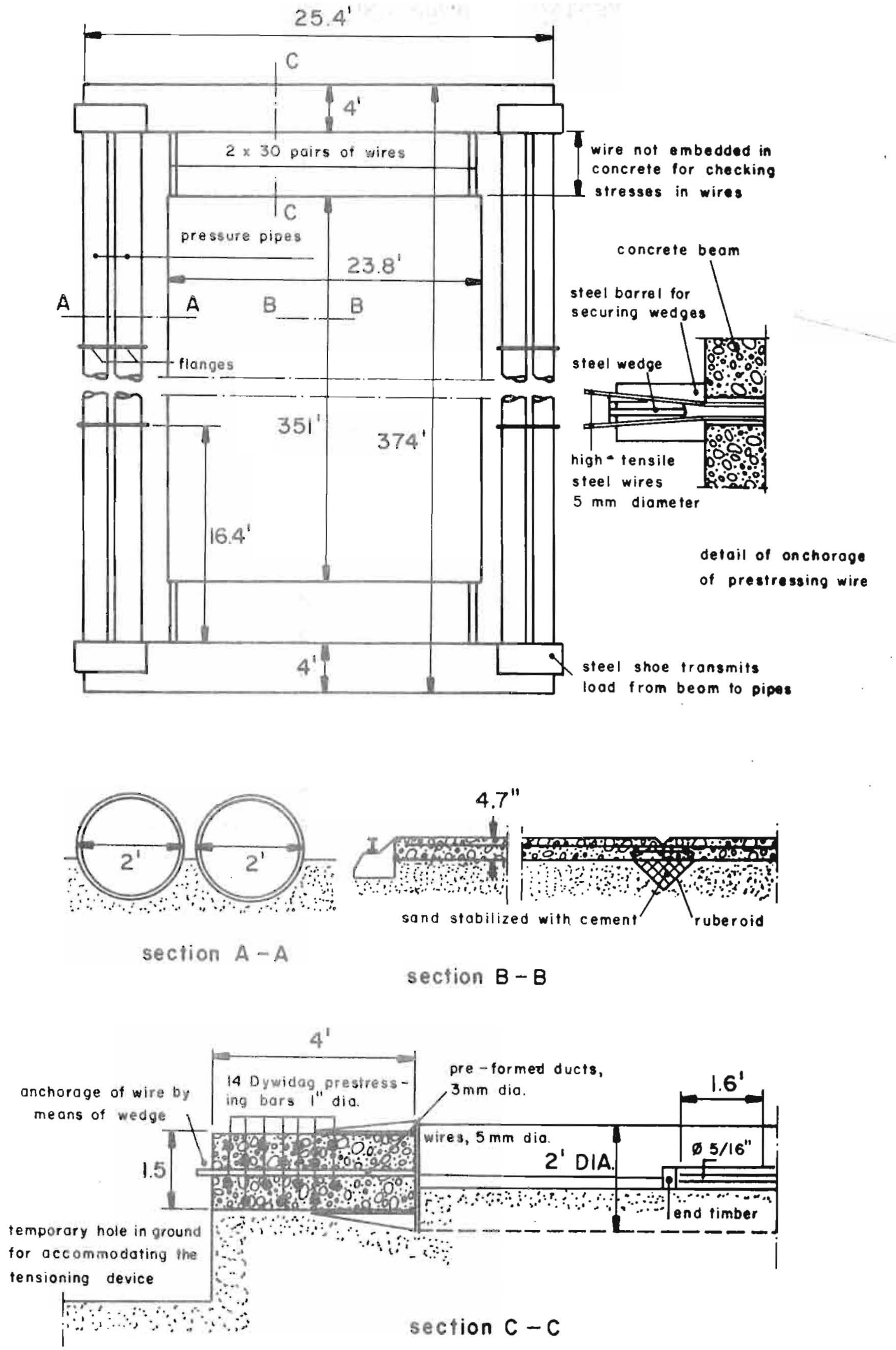


FIG. 1 - PRETENSIONING FRAME FOR PRESTRESSED PAVEMENT AT LEIDSCHENDAM, NETHERLANDS

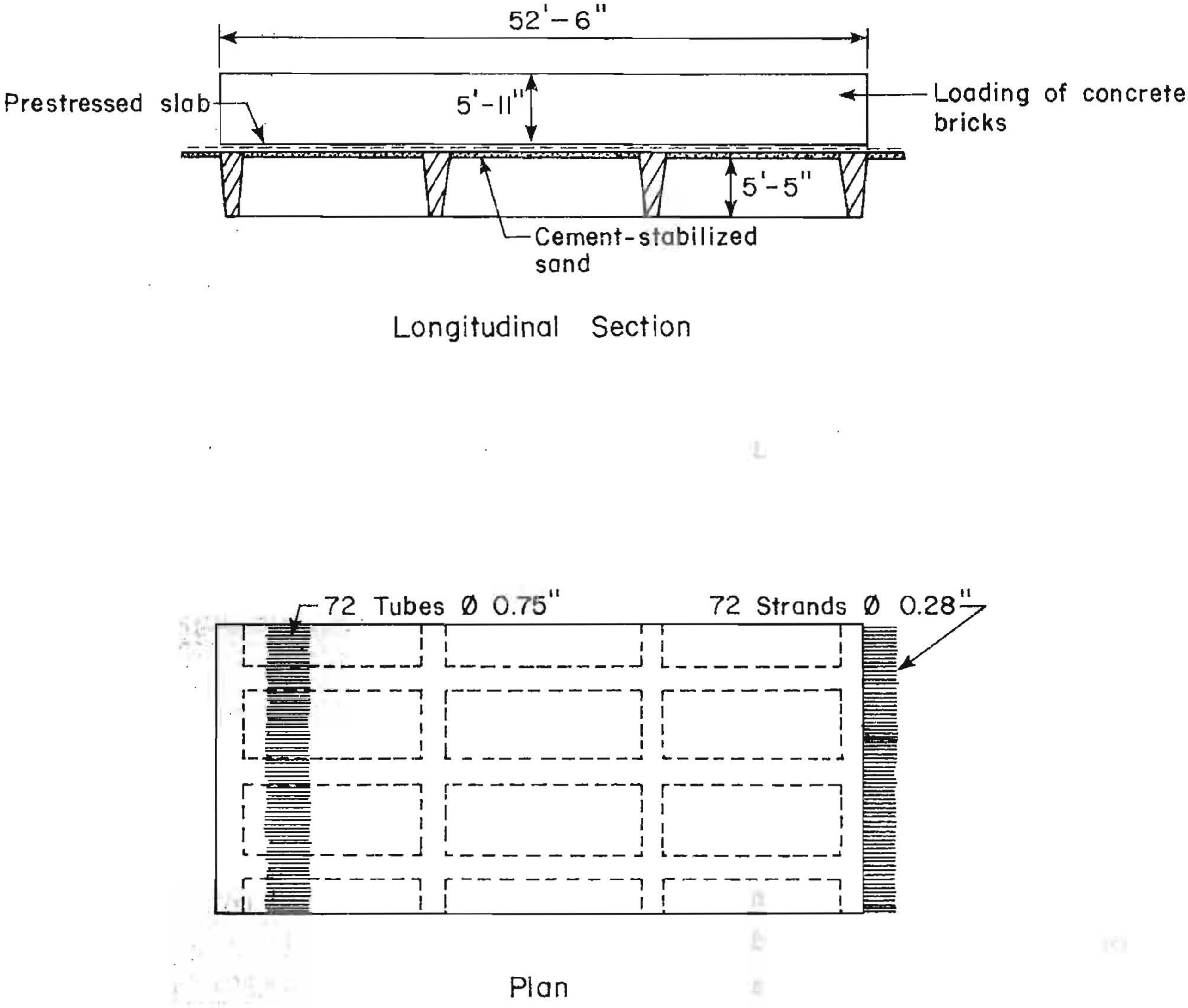


FIG. 2 - PRETENSIONING ABUTMENT FOR PRESTRESSED PAVEMENT AT HAARLEMMERMEER, NETHERLANDS

slab. After the concrete had hardened sufficiently to provide adequate bond to the steel, the joints were sawed and the prestress was allowed to develop in the separate slabs.

In an experimental section of a road on the Via Amila near Cesena, Italy(218), the tendons of a 1641 ft (500 m) long pavement were pretensioned between two abutments as shown in Fig. 3. Joints were spaced 152 ft (46 m) apart except at the center where the slab length was 305 ft (93 m).

An extension to the runway of the Vienna airport(208) was constructed as prestressed concrete pavement. In the longitudinal direction, pretensioned cables were initially anchored to intermediate abutments as shown in Fig. 4. The abutments were spaced approximately 820 ft (250 m) apart. The transverse joints between concrete sections were placed either above an abutment or at mid-distance between two abutments. In the latter case, a supporting slab was provided.

Pretensioned wire construction requires mechanical unreeling of wire coils on carriages to obtain reasonable production. Also, long-stroke jacking equipment is required for tensioning long tendons between abutments. In a project in the Netherlands(226), a mobile crane was used to unreel and tension the wires.

Post-Tensioning

Post-tensioning differs from pretensioning primarily in that unstressed tendons are placed in flexible or rigid conduits or are coated with a bond breaking material. Before concrete is placed, tendons usually are partially stressed for alignment only. After concrete has attained a specified strength, the tendons are stressed by jacking until the desired prestress is induced in the concrete. In a conduit system, grout is subsequently pumped into the space surrounding the tendons. This prevents corrosion and bonds the tendons to the pavement.

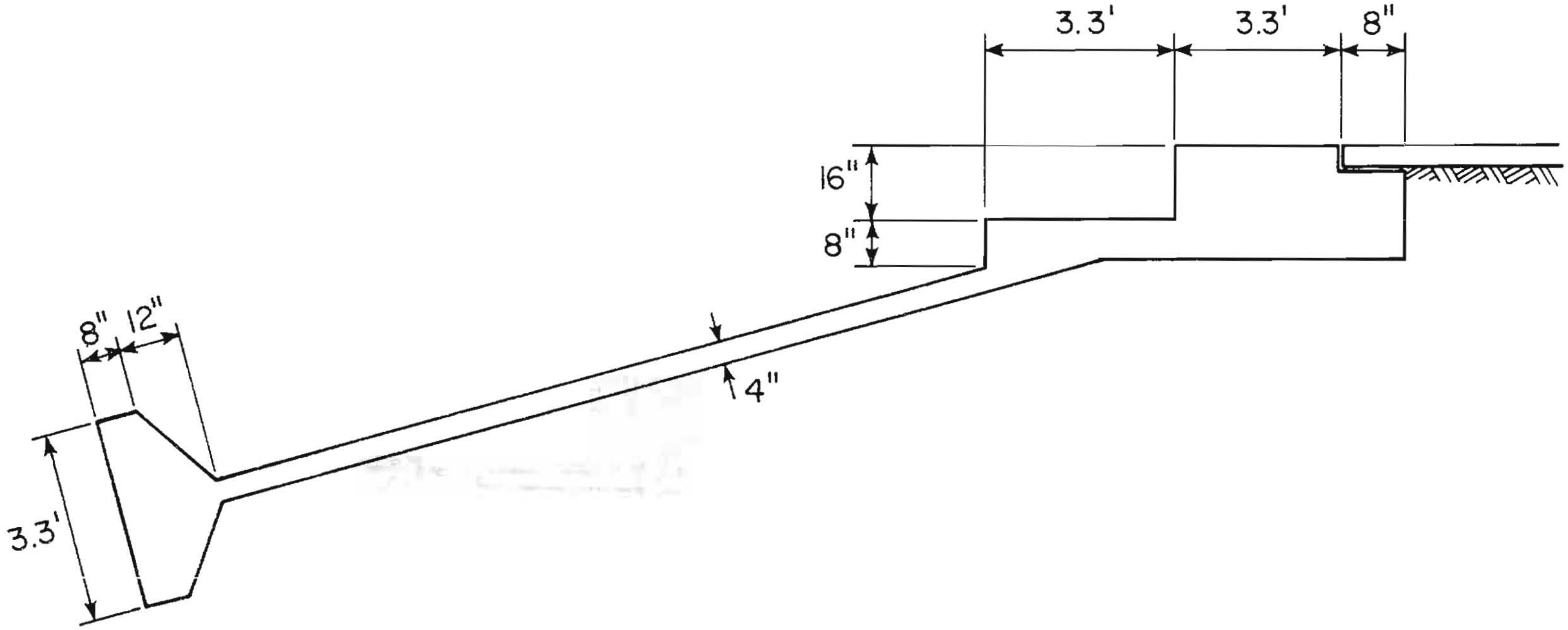


FIG. 3 - PRETENSIONING ABUTMENT FOR HIGHWAY
PAVEMENT AT CESENA, ITALY

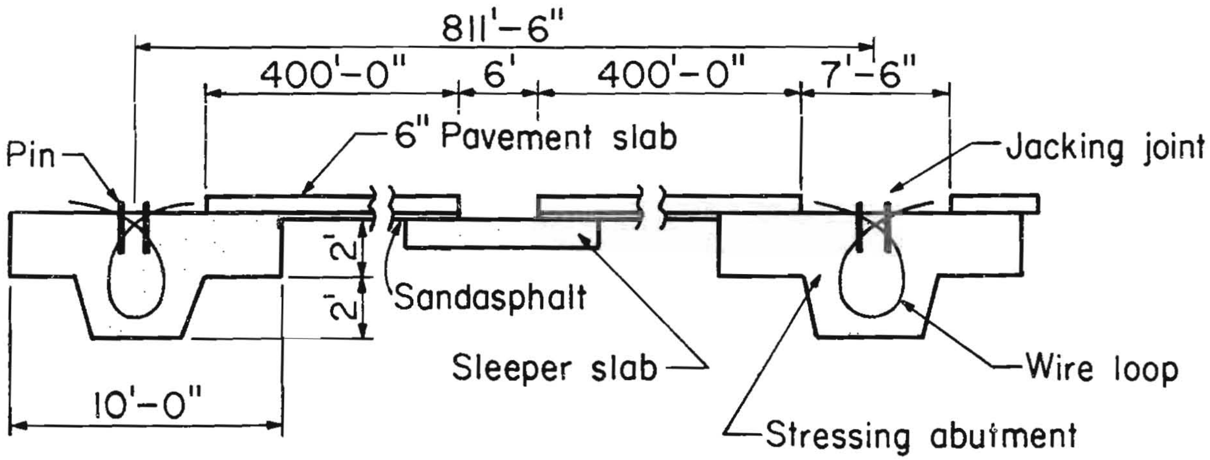


FIG. 4 - PRESTRESSED CONCRETE RUNWAY AT VIENNA, AUSTRIA

Post-tensioning can be provided by two different methods:

(a) by direct jacking at the boundary slab and anchoring in place after application of jacking force.

(b) by jacking in the gap between two segments of a slab. In this case, a tendon passing through the gap is anchored in opposite ends of the slab. After the concrete has reached a specified strength, the jacks are used to force the two segments of the slab apart, thus stressing the tendons and inducing compressive stress in the concrete. After completion, the gap is filled with concrete.

Predominantly, post-tensioning has been provided in the longitudinal direction. However, transverse post-tensioning at right angles has been provided on some projects. Also, diagonal post-tensioning was used in some of the early projects to provide for both longitudinal and transverse prestressing. In most cases, the diagonal prestressing was applied at 45° to the pavement centerline. However, in a project at Mergelstetten in Germany[242], post-tensioning at approximately 30° was used.

Examples of longitudinal post-tensioning only are illustrated by several projects including the four prestressed concrete pavement demonstration projects built in the U.S. at Milford, Delaware; at Kutztown, Pennsylvania; at Harrisburg, Pennsylvania; and at Dulles International Airport, Virginia.

Examples of longitudinal and transverse post-tensioning are projects built in Austria, England, Germany, and other countries.

Examples of diagonal post-tensioning are projects built in Germany, England, France, and other countries. Some of these are shown in Figs. 5, 6, 7, and 8. On all these projects, anchorage of tendons in slab ends was eliminated by curving the tendons towards the pavement edges.

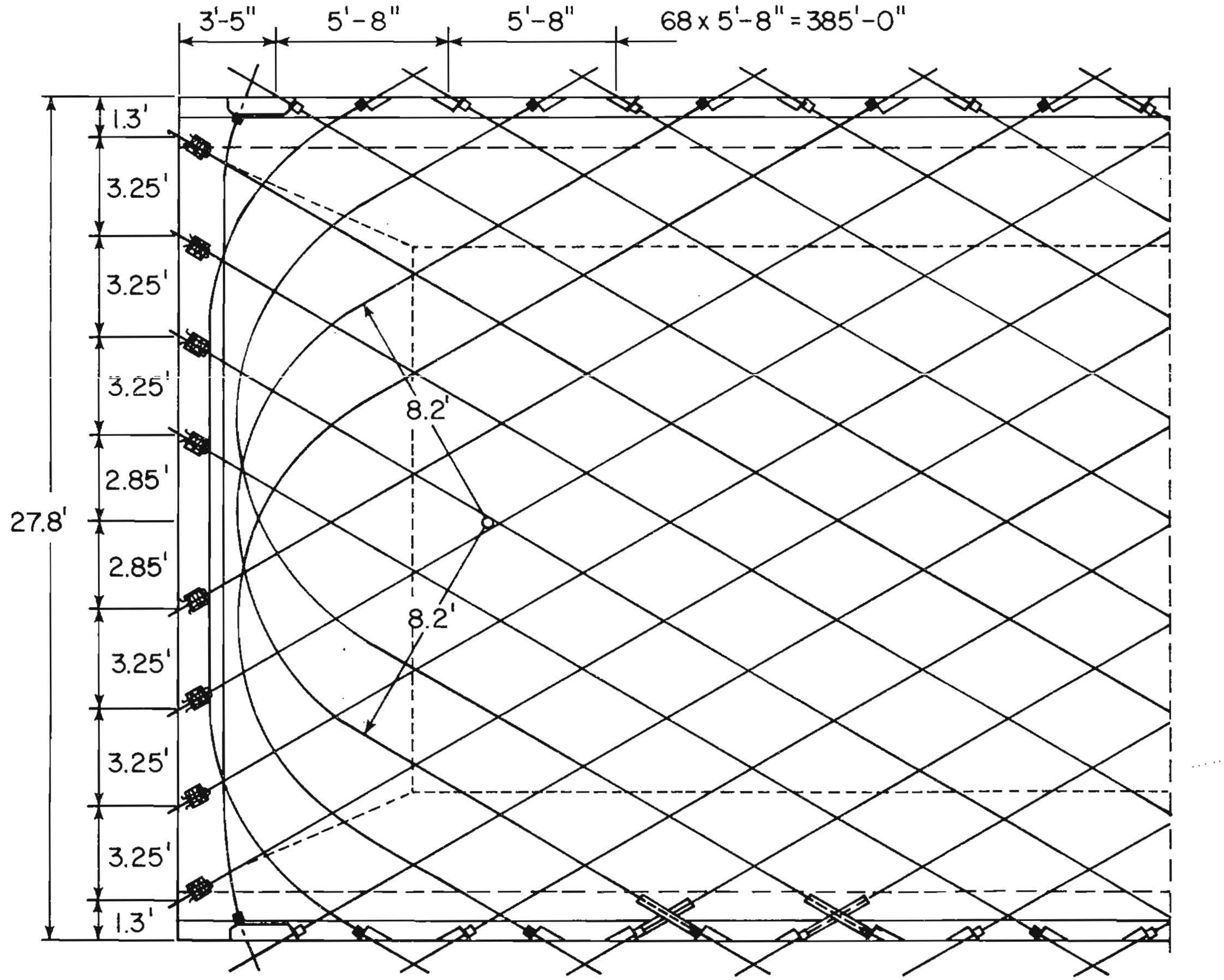


FIG. 5 - ARRANGEMENT OF TENDONS IN HIGHWAY PAVEMENT AT MERGELSTETTEN, GERMANY

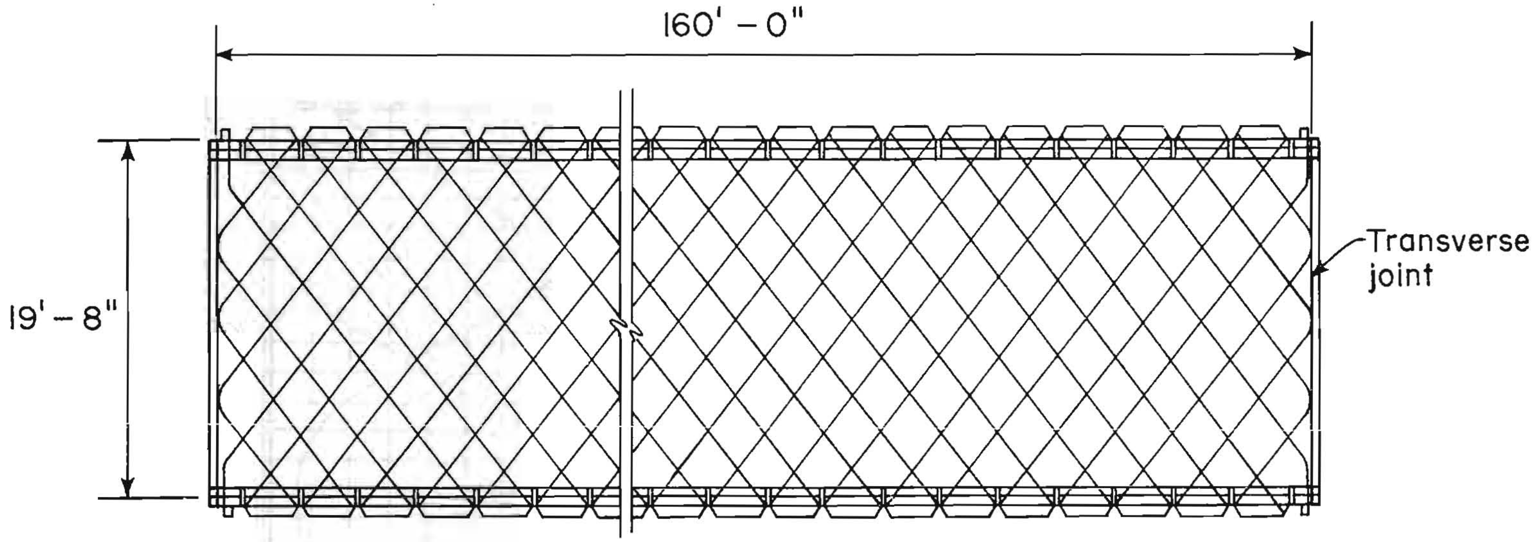


FIG. 6 - ARRANGEMENT OF TENDONS IN PRESTRESSED PAVEMENT AT ESBLY, FRANCE

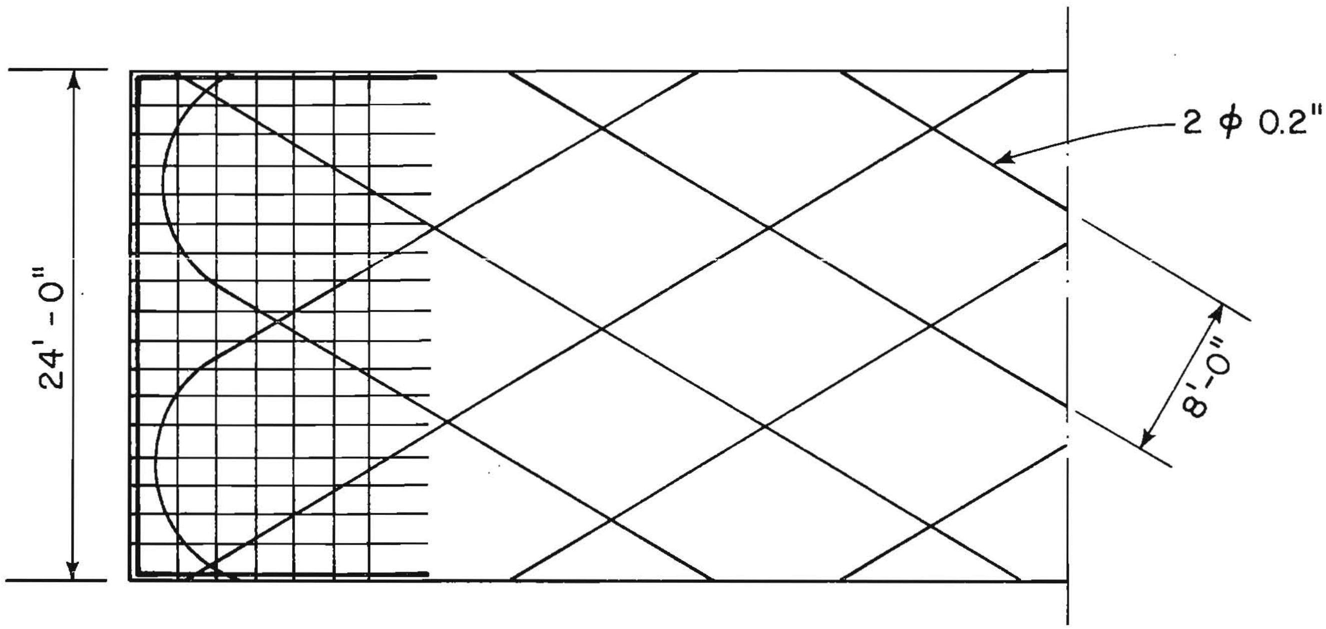


FIG. 7 - ARRANGEMENT OF TENDONS IN PRESTRESSED PAVEMENT AT CRAWLEY, ENGLAND

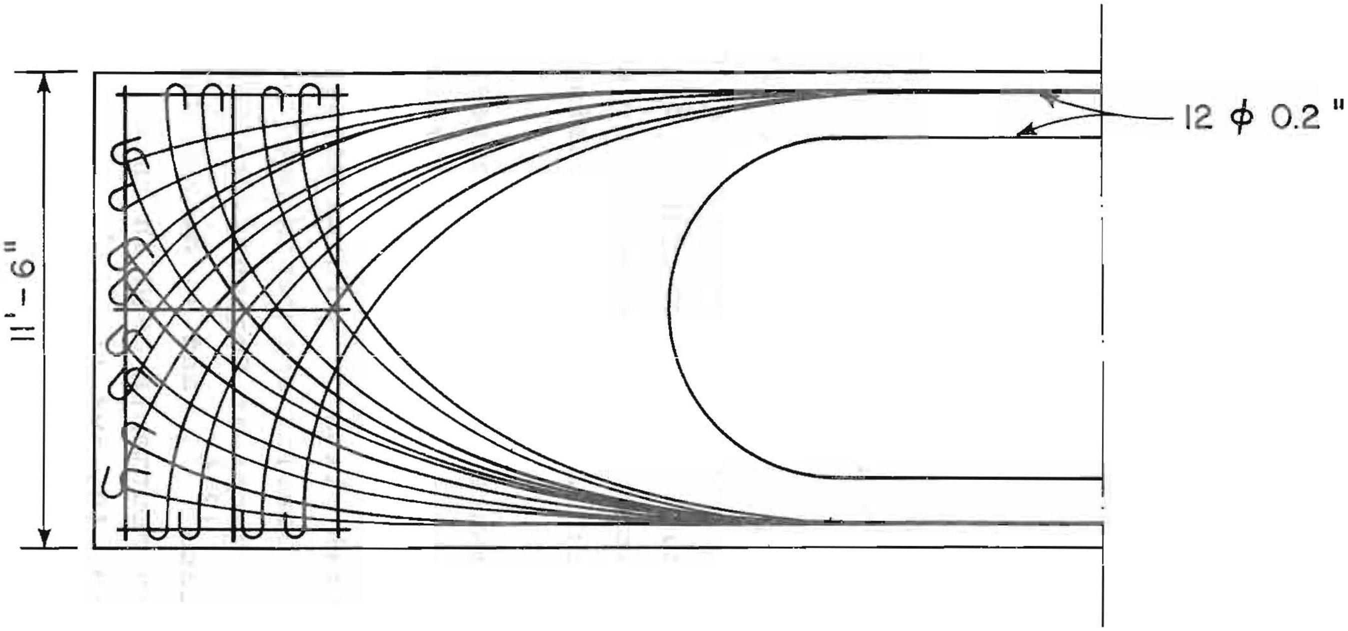


FIG. 8 - ARRANGEMENT OF TENDONS IN PRESTRESSED
PAVEMENT AT WEXHAM SPRINGS, ENGLAND

Poststressing

In this system, prestressing is accomplished without the use of tendons. Generally a series of slabs is placed between two end abutments. When the concrete has attained sufficient strength, a force is applied at the joints between the slabs until a specified compressive stress has been induced in the concrete. In some cases, abutments are designed to provide an elastic-type reaction to prevent the development of excessive compressive stresses, and to maintain the pavement in a prestressed condition during slab contraction. In other cases, abutments are designed to resist the forces caused by full restraint due to temperature increase, thus providing a fixed-length poststressed pavement.

Abutments for poststressed concrete pavements may be classified as gravity, compression, and tension types. In addition, lateral abutments may be required. These types are described and illustrated by examples from projects reported in the literature. Also, poststressing devices are discussed.

Gravity abutments are designed to resist thrust from pavement ends by their weight and soil friction. Not only the weight of the concrete, but also the weight of the soil above the potential plane of sliding contributes in mobilizing reactions. Gravity abutments have been used on postressed pavement projects at Fontenay-Tresigny in France[191], at Winthorpe, in England[256], and at Zwartberg-Meeuwen in Belgium[223]. Gravity abutments used in these projects are illustrated in Figs. 9, 10, and 11.

Compression abutments transfer compressive forces from the pavement slab to the foundation soils. The size and depth of compression abutments is governed by the soil and foundation conditions. Compression abutments have been used on poststressed pavement projects at Bourg-Servas in

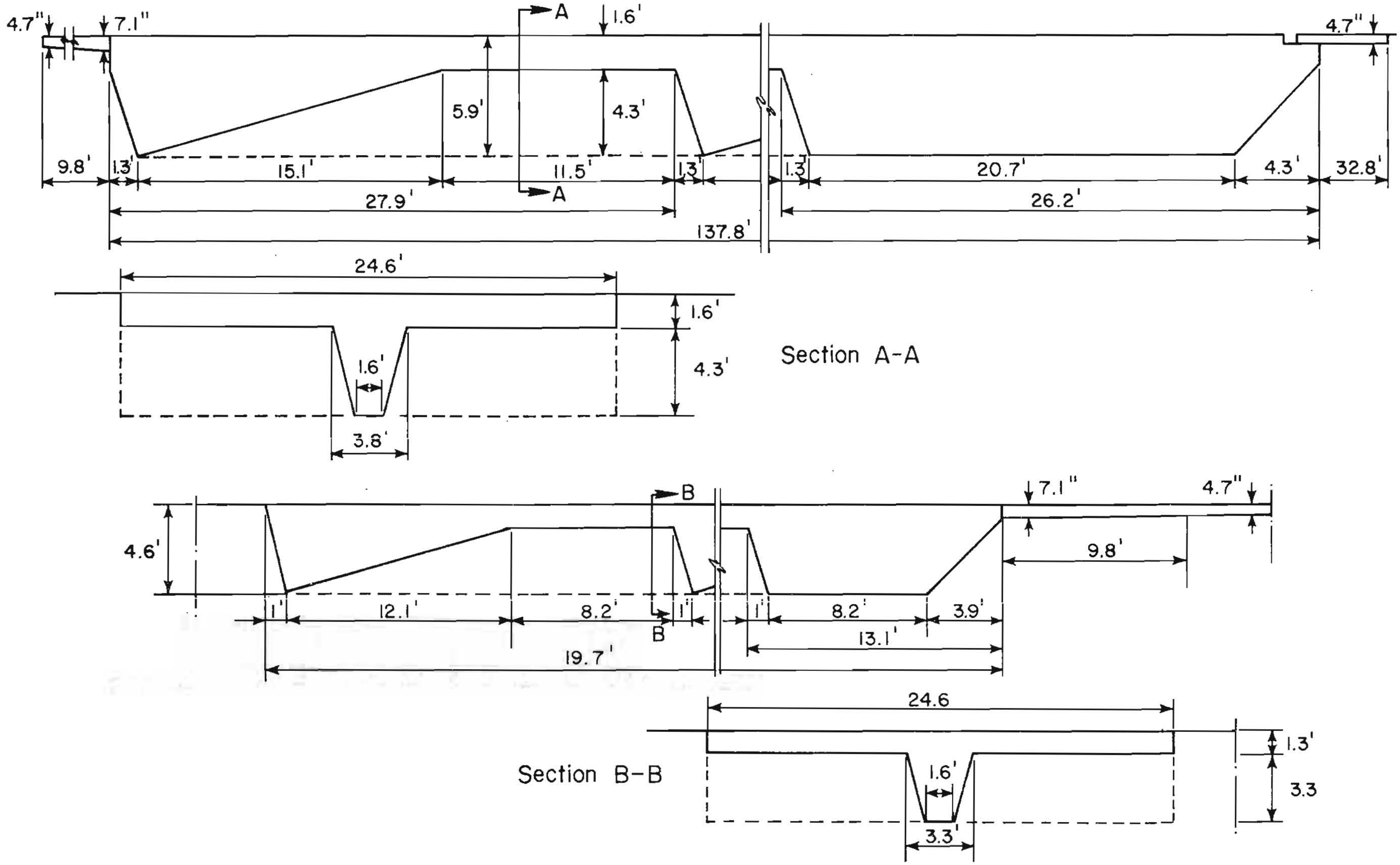


FIG. 9 - ABUTMENTS FOR TEST ROAD AT FONTENAY-TRESIGNY, FRANCE

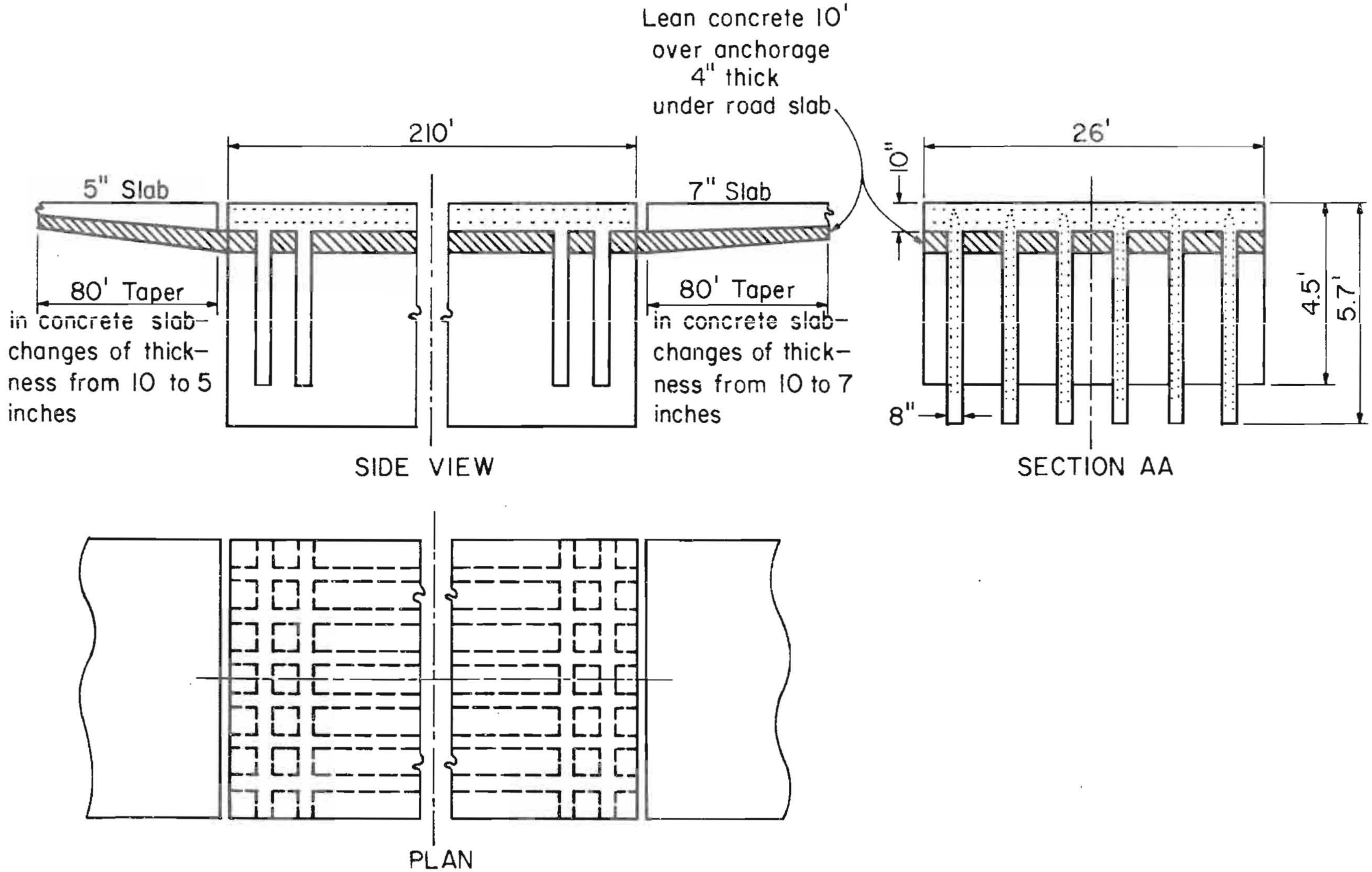


FIG. 10 - RIB SOIL ABUTMENT FOR PRESTRESSED PAVEMENT AT WINTHORPE, ENGLAND

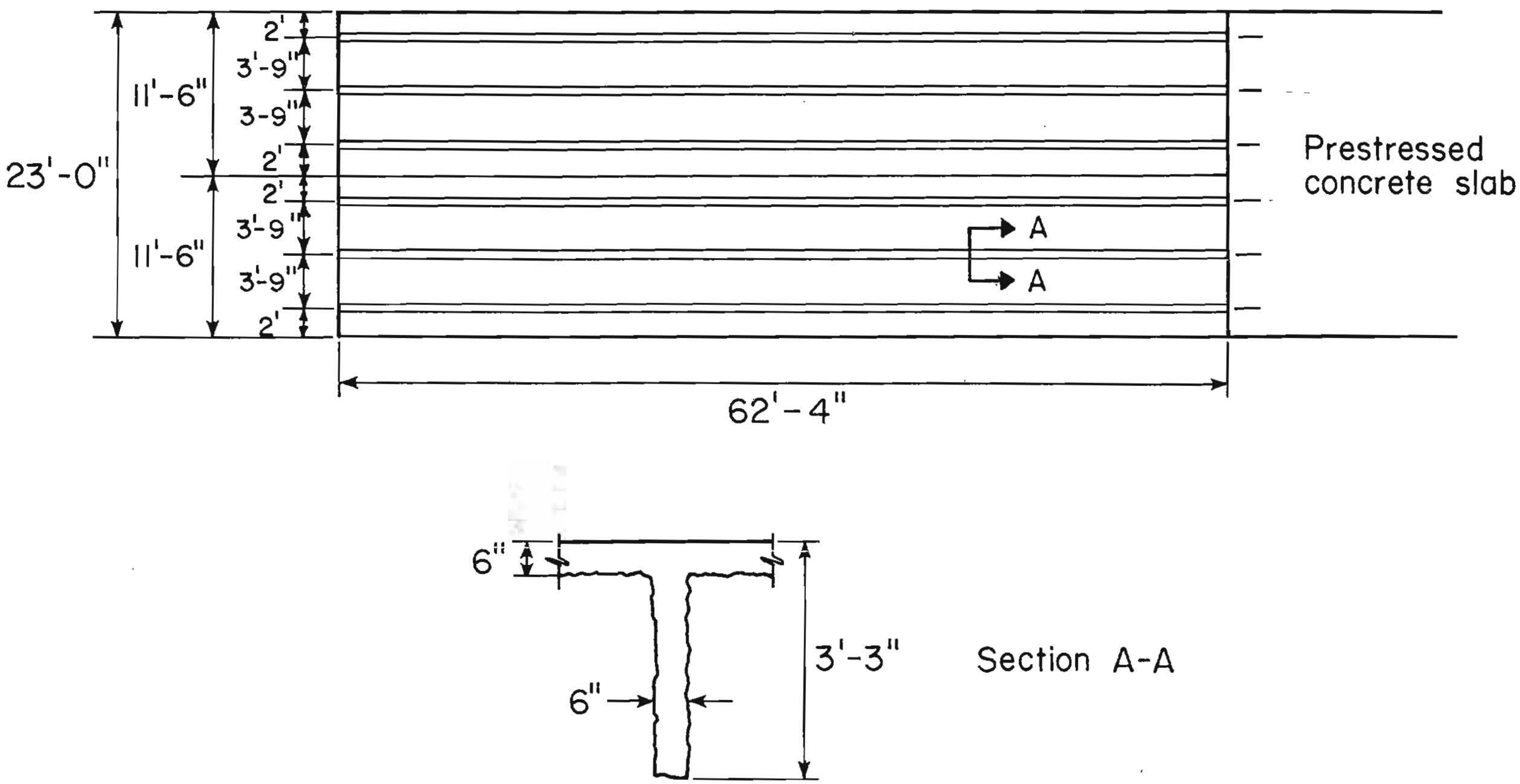


FIG. 11 - ABUTMENT FOR PRESTRESSED PAVEMENT AT ZWARTBERG-MEEUWEN, BELGIUM

France[58], at Zwartberg-Meeuwen in Belgium[223], and at Salzburg in Austria[193]. Compression abutments used in these projects are illustrated in Figs. 12, 13, and 14.

Tension abutments have been used on poststressed pavement projects at Bourg-Sevras in France[58], at Boudry in Switzerland[228], and Maison-Blanche Airport in Algeria[88]. Tension abutments used on these projects are illustrated in Figs. 15, 16, and 17.

Lateral abutments are needed on curves to resist lateral forces and prevent lateral slab movement. Lateral abutments have been used on Zwartberg-Meeuwen project in Belgium[223]. This abutment is illustrated in Fig. 18.

The need for end abutments and sleeper slabs are undesirable features of post-stressed construction. Temporary abutments are required for initial prestress applications during construction.

Different poststressing devices have been used. Freyssinet flat jacks have been used exclusively for fixed length pavements. These jacks can be used individually or in multiple assemblies as shown in Fig. 19. When used in multiple assemblies, the jacks are separated by cement mortar. The jacks are inflated with hydraulic oil, or cement grout if left in place.

A unique example of poststressed pavement where prestress was produced by flat jacks is an experimental section on the Zwartberg-Meeuwen road in Belgium[223]. A tunnel was constructed under each jacking point to permit subsequent adjustment of the jack without interruption of traffic. This system is shown in Fig. 20. The forces were taken by two abutments having the designs shown in Figs. 11 and 13. In addition, lateral abutments shown in Fig. 18 were provided on curves to prevent lateral slab movement.

Another prestressing method was used on the prestressed concrete test road at Fontenay-Tresigny, France. This project was built in 1959[191]. Joint designs consisting of

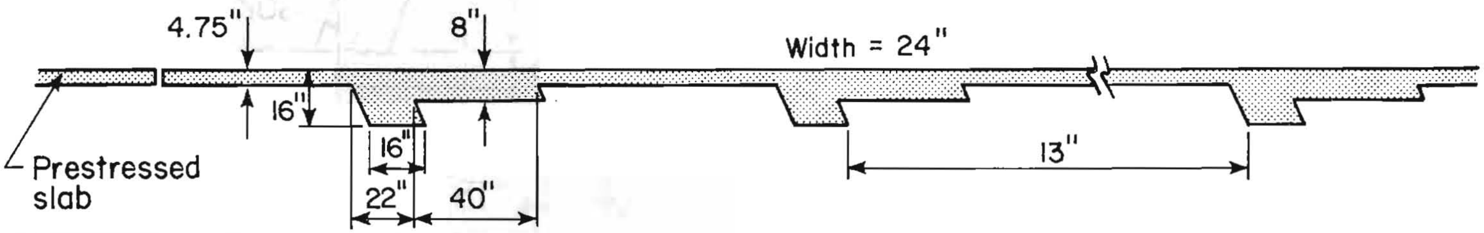


FIG. 12 - ABUTMENT FOR HIGHWAY PAVEMENT AT BOURG-SERVAS, FRANCE

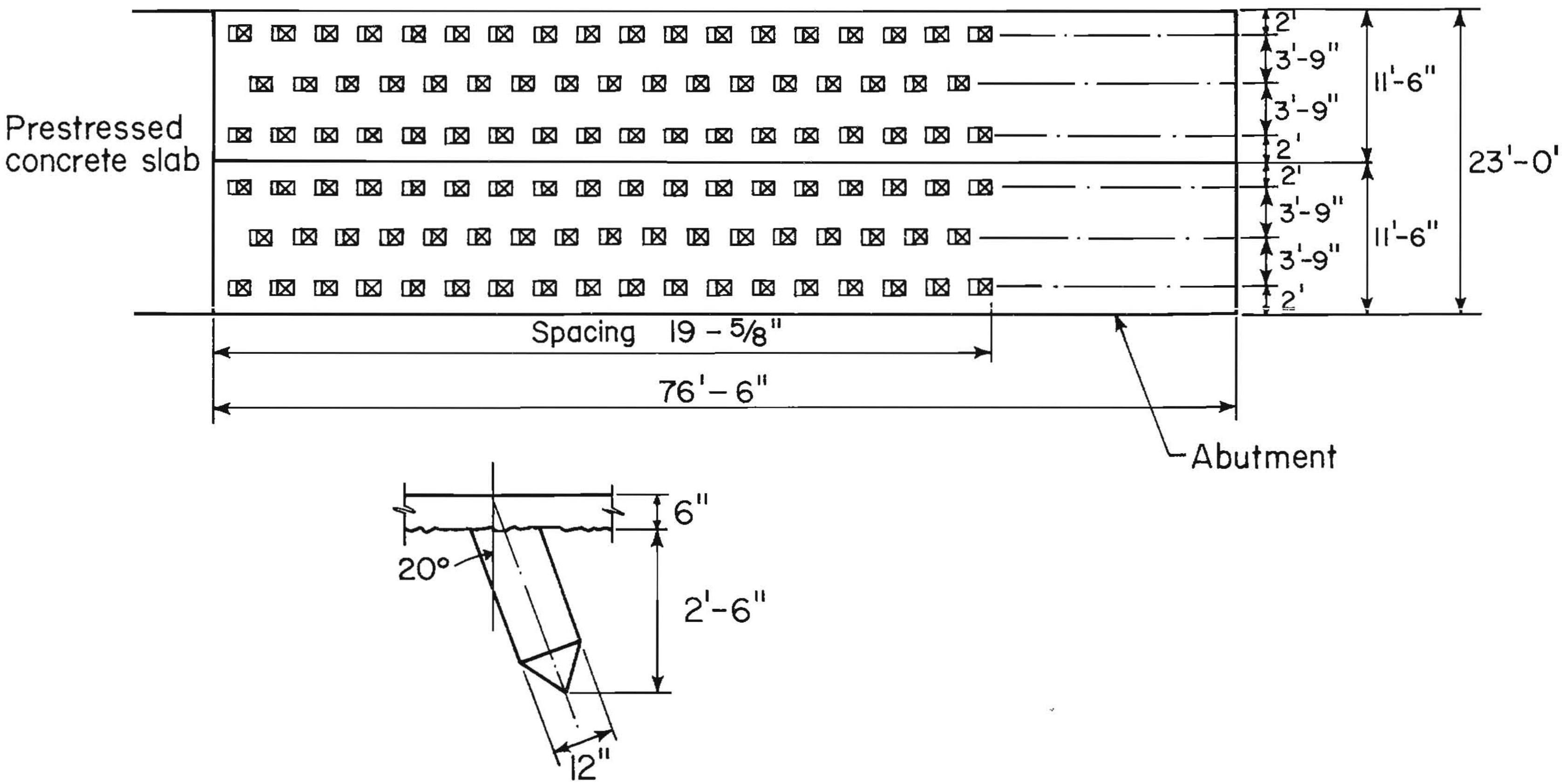


FIG. 13 - ABUTMENT ON PILES FOR PRESTRESSED PAVEMENT AT ZWARTBERG-MEEUWEN, BELGIUM

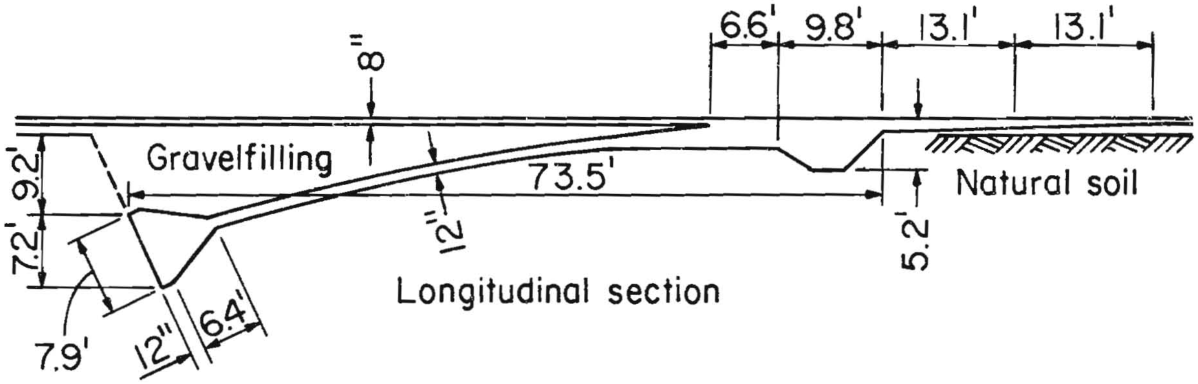


FIG. 14 - ABUTMENT FOR PRESTRESSED ROAD AT
SALZBURG, AUSTRIA

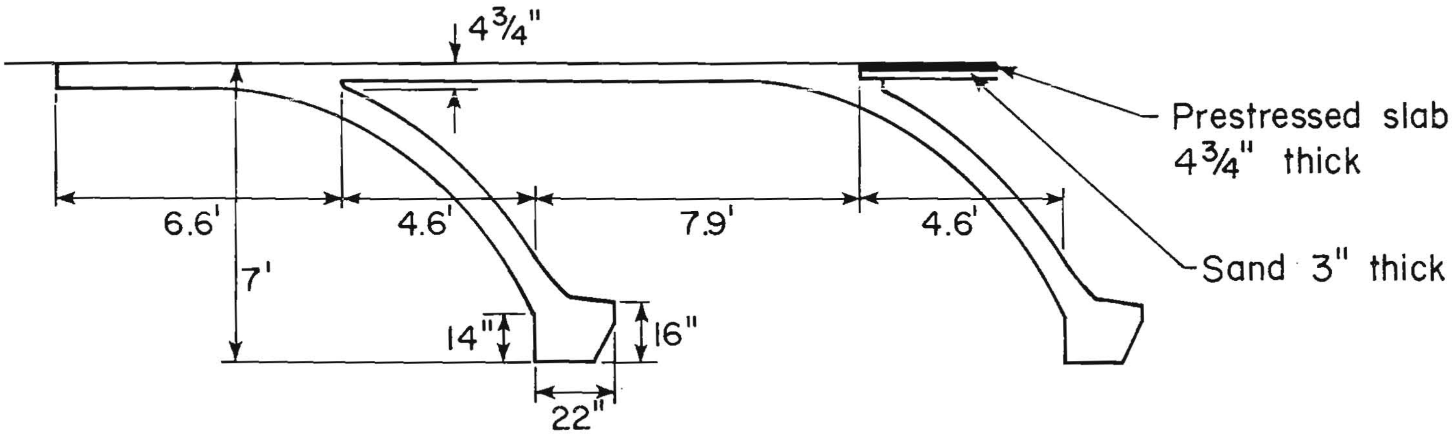


FIG. 15 - ABUTMENT FOR PRESTRESSED PAVEMENT AT BOURG-SERVAS, FRANCE

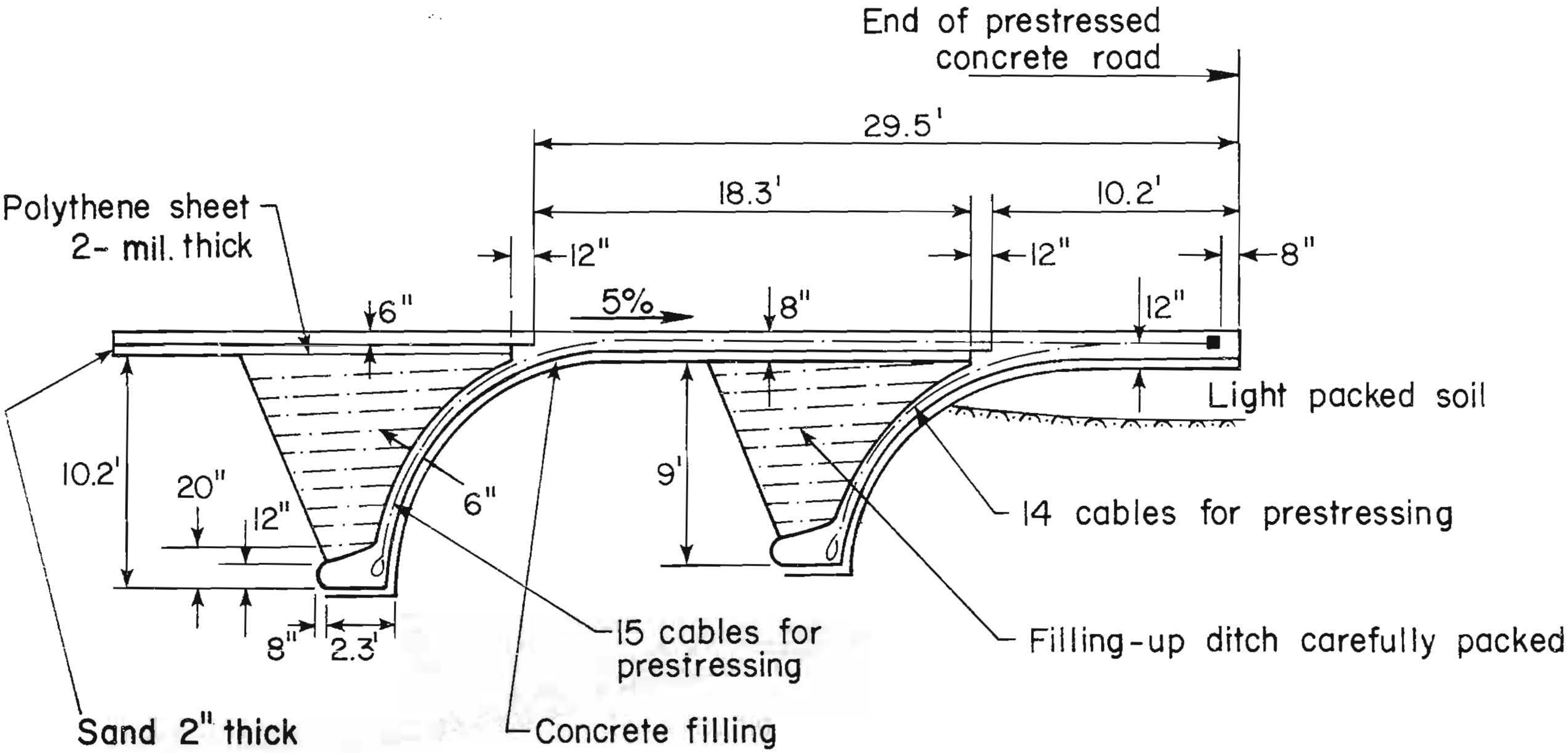


FIG. 16 - ABUTMENT FOR PRESTRESSED PAVEMENT AT BOUDRY, SWITZERLAND

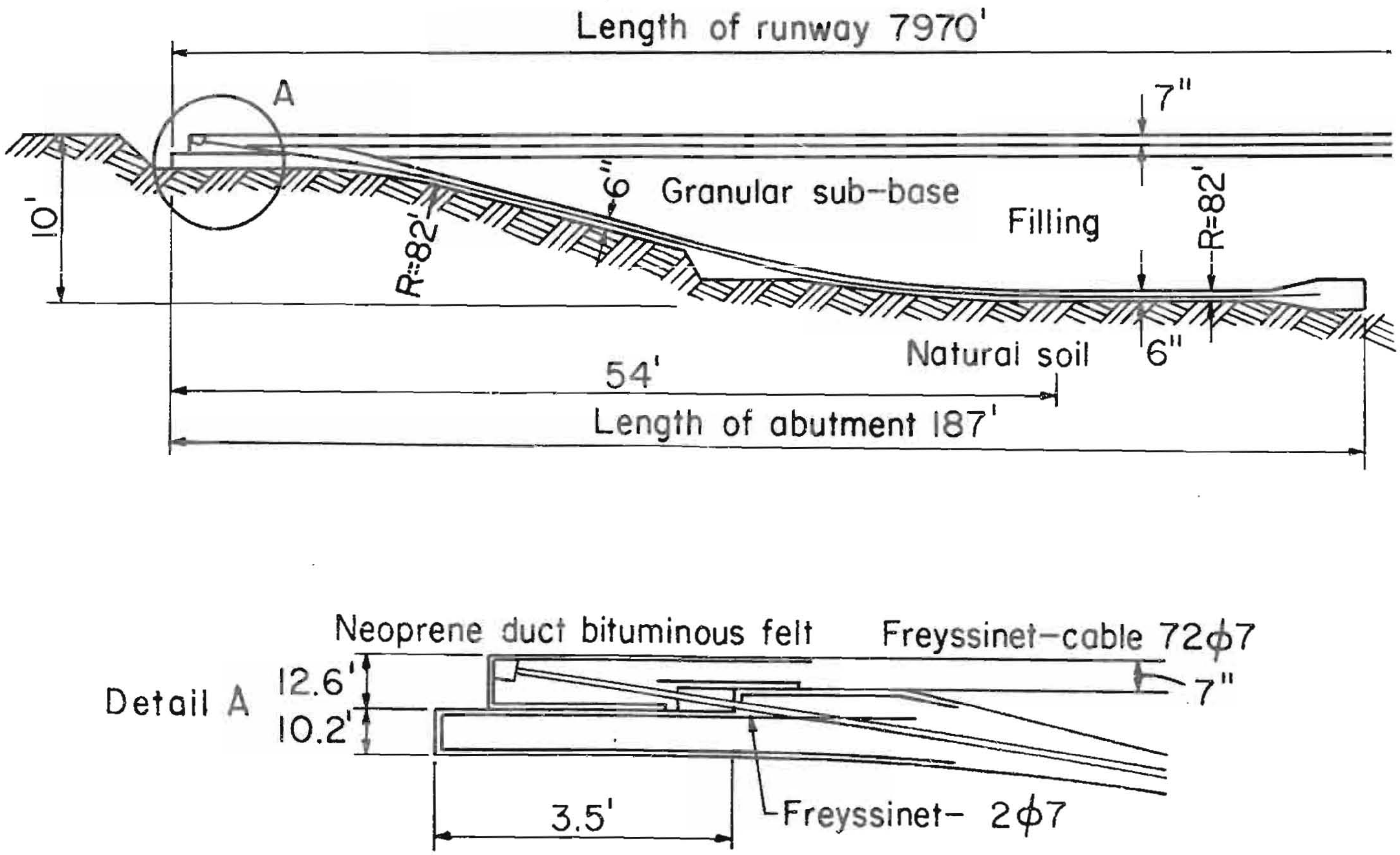


FIG. 17 - ABUTMENT FOR MAISON-BLANCHE AIRPORT, ALGERIA

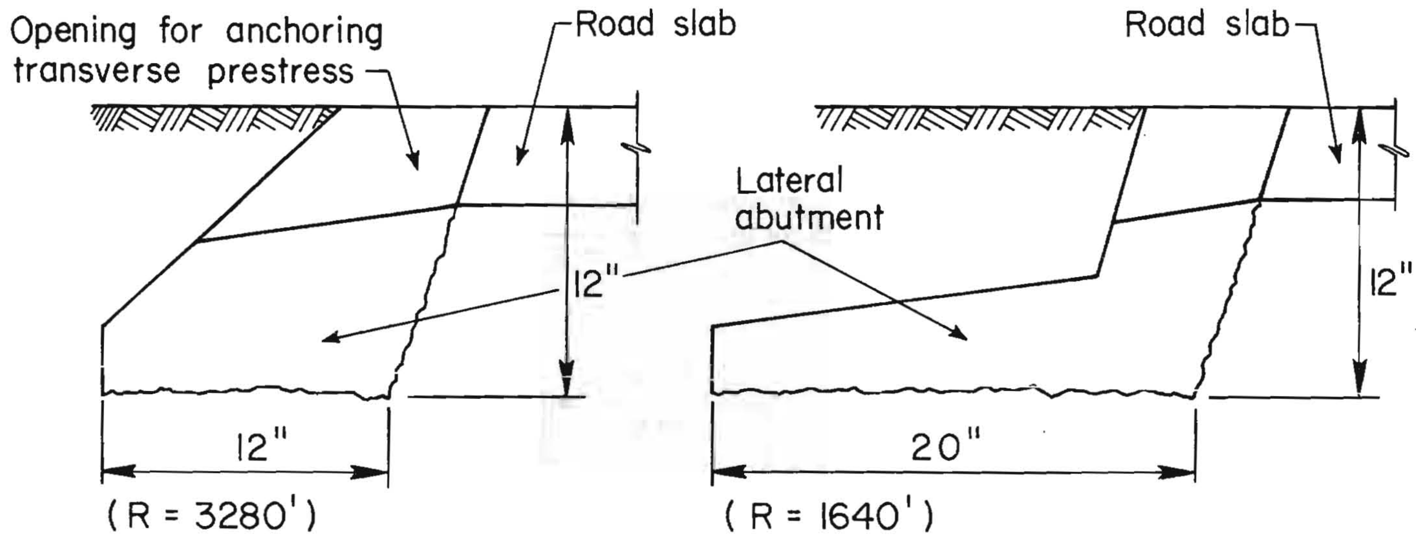


FIG. 18 - LATERAL ABUTMENTS ON CURVED SECTIONS OF HIGHWAY PAVEMENT AT ZWARTBERG-MEEUWEN, BELGIUM

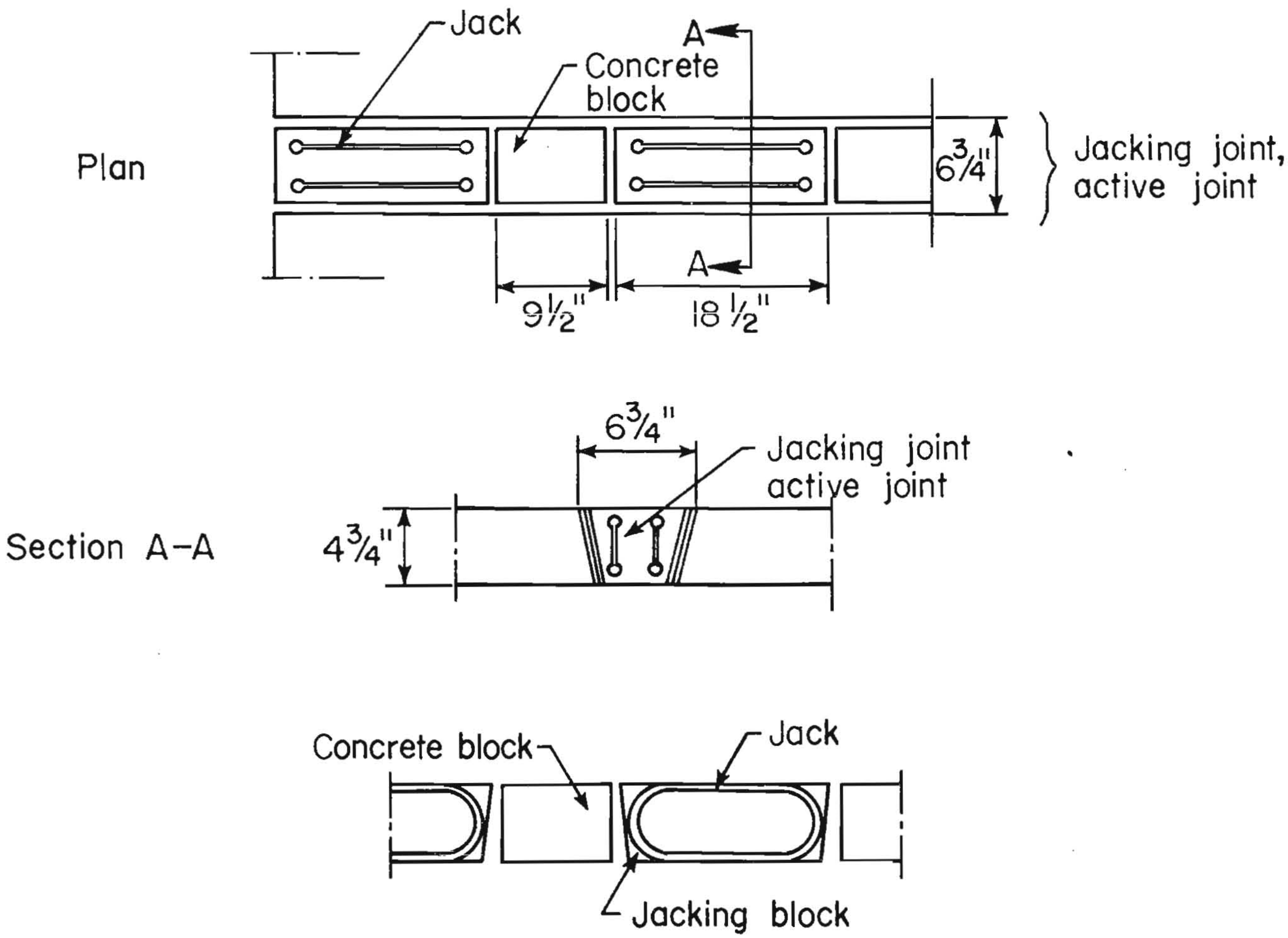


FIG. 19 - JACKING JOINT FOR TEST ROAD AT BOURG-SERVAS, FRANCE

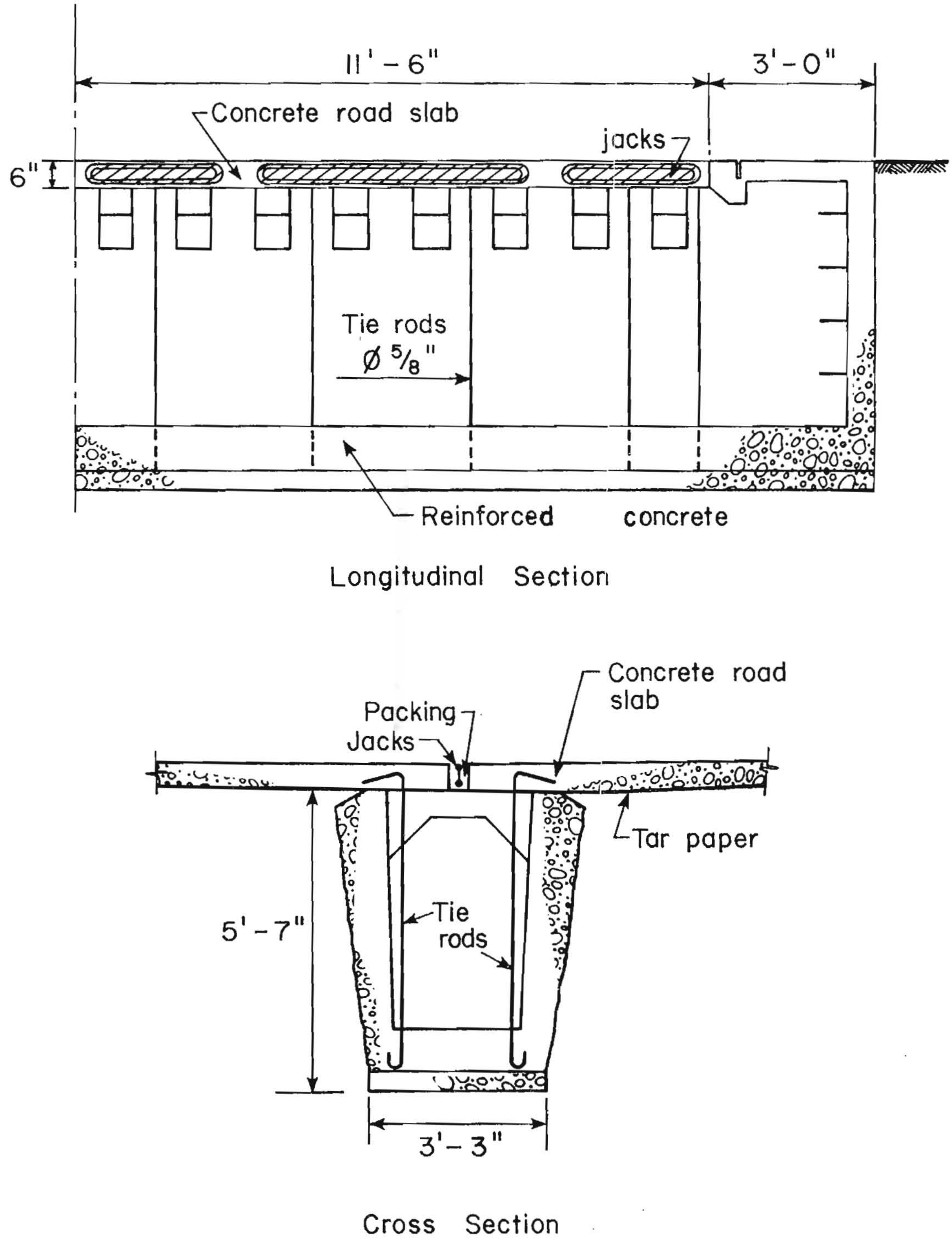


FIG. 20 - JACKING JOINT FOR HIGHWAY PAVEMENT AT ZWARTBERG-MEEUWEN, BELGIUM

a pneumatic cell as shown in Fig. 21 and a Butyl bladder as shown in Fig. 22 were used on this project. These designs permit adjustment of joint width to compensate for concrete creep and shrinkage.

TRANSVERSE JOINTS

In conventional concrete pavements, transverse joints control cracking that results from longitudinal restrained contraction and the combined effects of restrained warping and traffic loads. Joint spacing for conventional pavements generally varies from 15 to 80 ft (5 to 24 m) depending on whether the pavement is plain or conventionally reinforced.

For prestressed pavements, joint spacing generally varies from 200 to 600 ft (61 to 183 m). The spacing is chosen in an attempt to optimize two contrasting considerations. The first consideration is to reduce the number of joints by making the spacing as long as possible while still maintaining the minimum necessary longitudinal prestress at the center portion of the slab. The second consideration is to keep the spacing short enough so that the joint can accommodate the anticipated movements and transfer loads at a reasonable cost.

Reduction of the number of transverse joints in prestressed pavements is accompanied by design and construction problems. These include curling and warping deformations and stresses, large horizontal movements at the slab ends, compatibility of the joint details and the prestressing method, and economic aspects of joint spacing. To overcome these problems the ideal joint should meet the following requirements:

1. The joint should accommodate movement of the slab ends or be able to carry compressive forces from slab to slab.

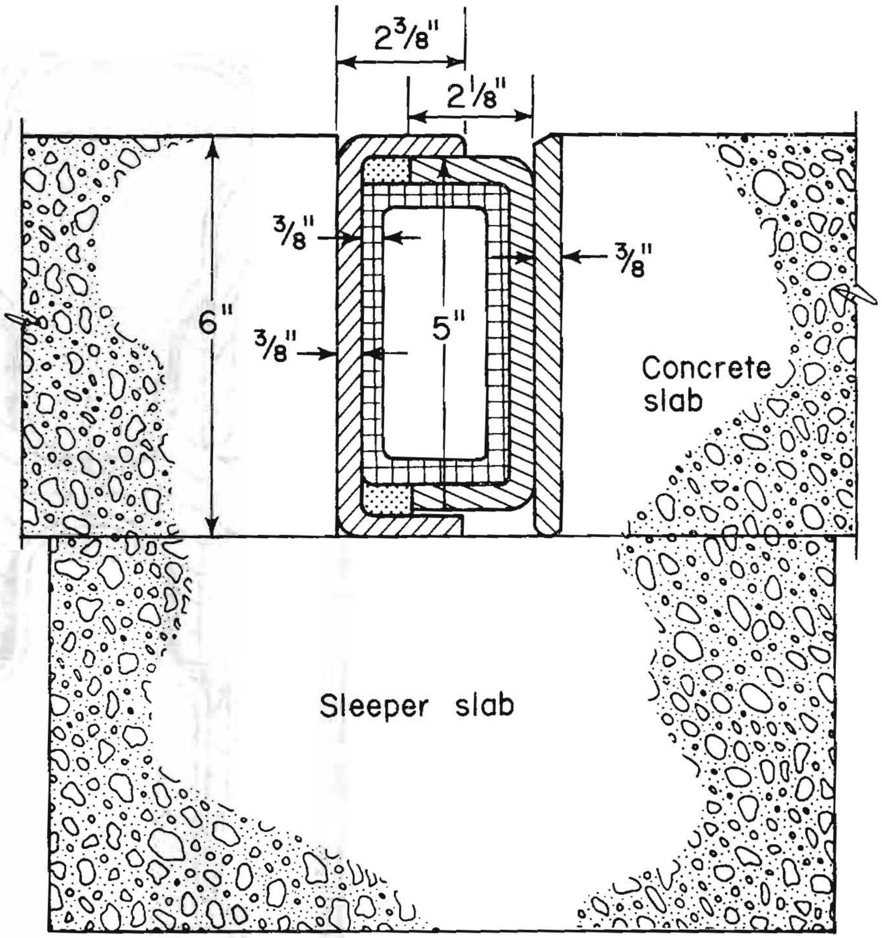
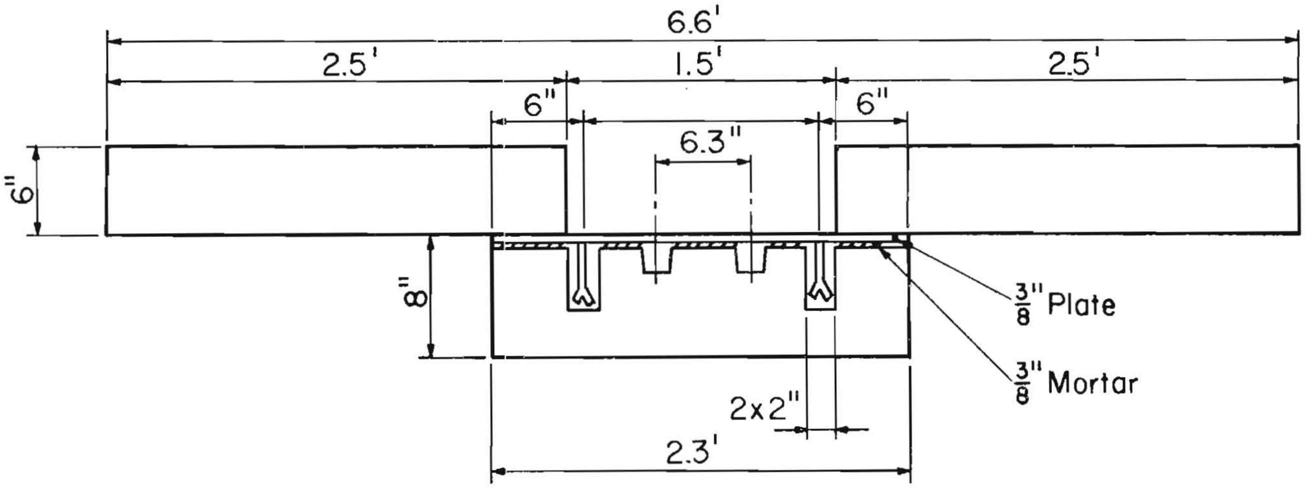


FIG. 21 - PNEUMATIC CELL JOINT DESIGN



CROSS SECTION

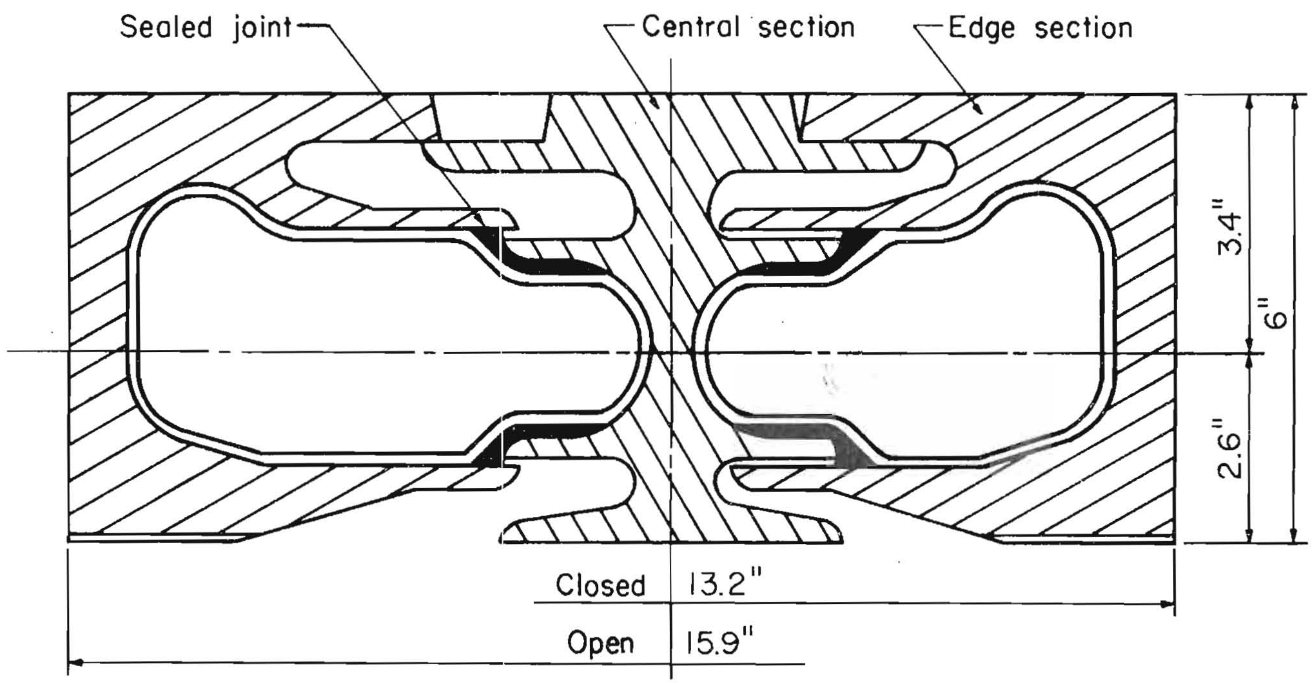


FIG. 22 - BUTYL BLADDER JOINT DESIGN

2. Traffic load must be carried by the joint without undesirable deflections or stresses at the joint or slab ends.
3. Joint material must be resistant to wear, fatigue, and corrosion caused by traffic, environment, or de-icer chemicals.
4. The joint should be sealed against the ingress of water and incompressibles, or should be drained and be self-cleaning, so that little maintenance is required.
5. Damaged components should be simple to remove and replace.
6. The slab and joint construction procedures must be compatible with the prestressing methods.
7. The costs should be low.

Transverse joints are classified as free, compression, and sliding. In the following sections, these types are discussed and illustrated by examples from projects described in the literature.

Free Joints

Free joints are designed to allow unrestrained movement of slab ends without transmitting compressive forces across the joint. For convenience of discussion, free joints are classified as open, covered, or closed. Open joints permit the ingress of water and debris from the pavement surface. Covered joints have plates over the top. Closed joints have an expandable laminated construction.

Open

An example of open joint geometry is illustrated in Fig. 23. In this joint proposed by Leonhardt[242], the top edge of the concrete is protected and the joint

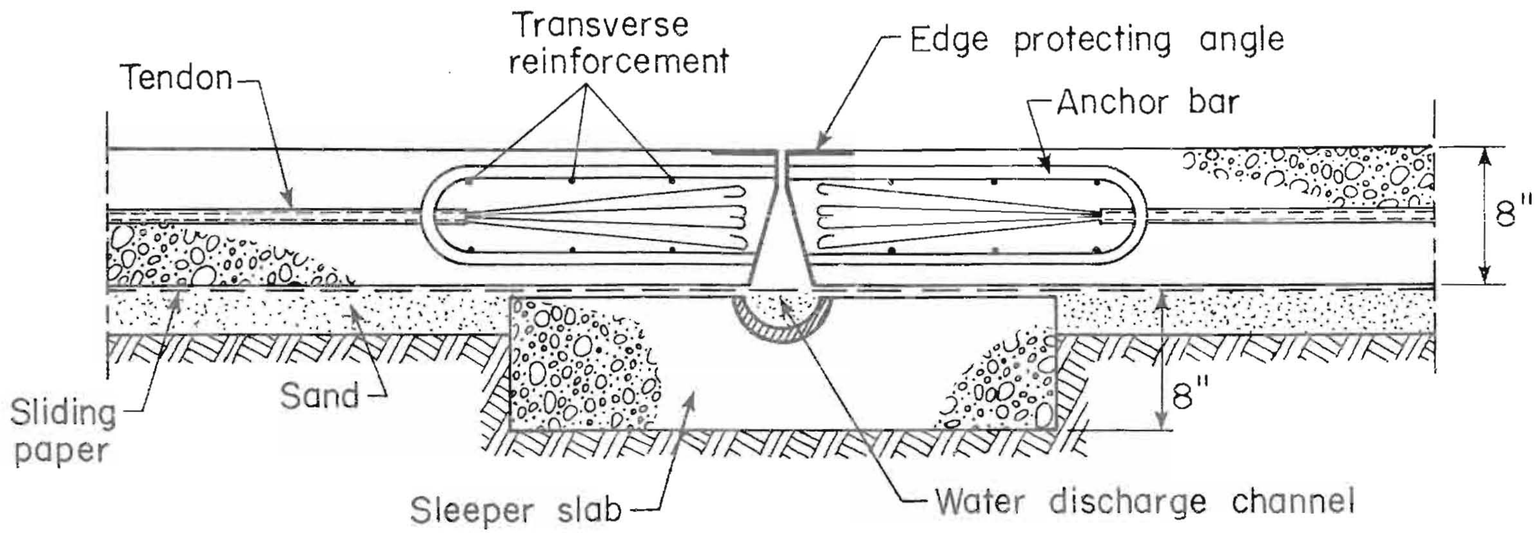


FIG. 23 - OPEN JOINT PROPOSED BY LEONHARDT

widens at the bottom. This widening is intended to assure that debris falling into the joint will not become lodged at the vertical joint faces. It was proposed that the joint be placed in combination with a sleeper slab provided with a water discharge channel. The channel could be cleaned with a water jet without interfering with roadway traffic.

For the joint in Fig. 23, prestressing would be done from a jacking joint. The jacking joint would be concreted upon completion of stressing the unbonded tendons. Thus, all movement would take place at the open joint. Unbonded tendons are anchored in stirrups extending a considerable distance into the slab end.

An open double joint was patented by Polensky and Zöllner[206] and installed between two adjacent 262 and 525-ft (80 and 162 m) long slabs. The double joint reduced movement. However, this was done at a cost of increasing the amount of joint hardware. Each face of the two joints was protected by steel angles for the pavement width. Jacking was done from joints located in the middle of the slabs. A sleeper slab was constructed below the open joint. Concrete between the double open joint was constructed after the stressing operation. An alternative method of construction would be to raise the sleeper beam to the same elevation as the adjacent roadway slabs.

An open finger joint, Fig. 24, similar to those used in bridge construction was installed in an experimental roadway near Montabaur, Germany in 1954[78]. The joint was assembled on the roadway prior to placing concrete. Joint fingers at the locations of the stressing tendons could be removed for access of inclined jacks. Stressing strands were curved upward at the slab ends to accommodate the direction of strand tensioning forces. This may be a disadvantage, as the upward slab deformation due to tendon positioning is additive to the upward curling deformation

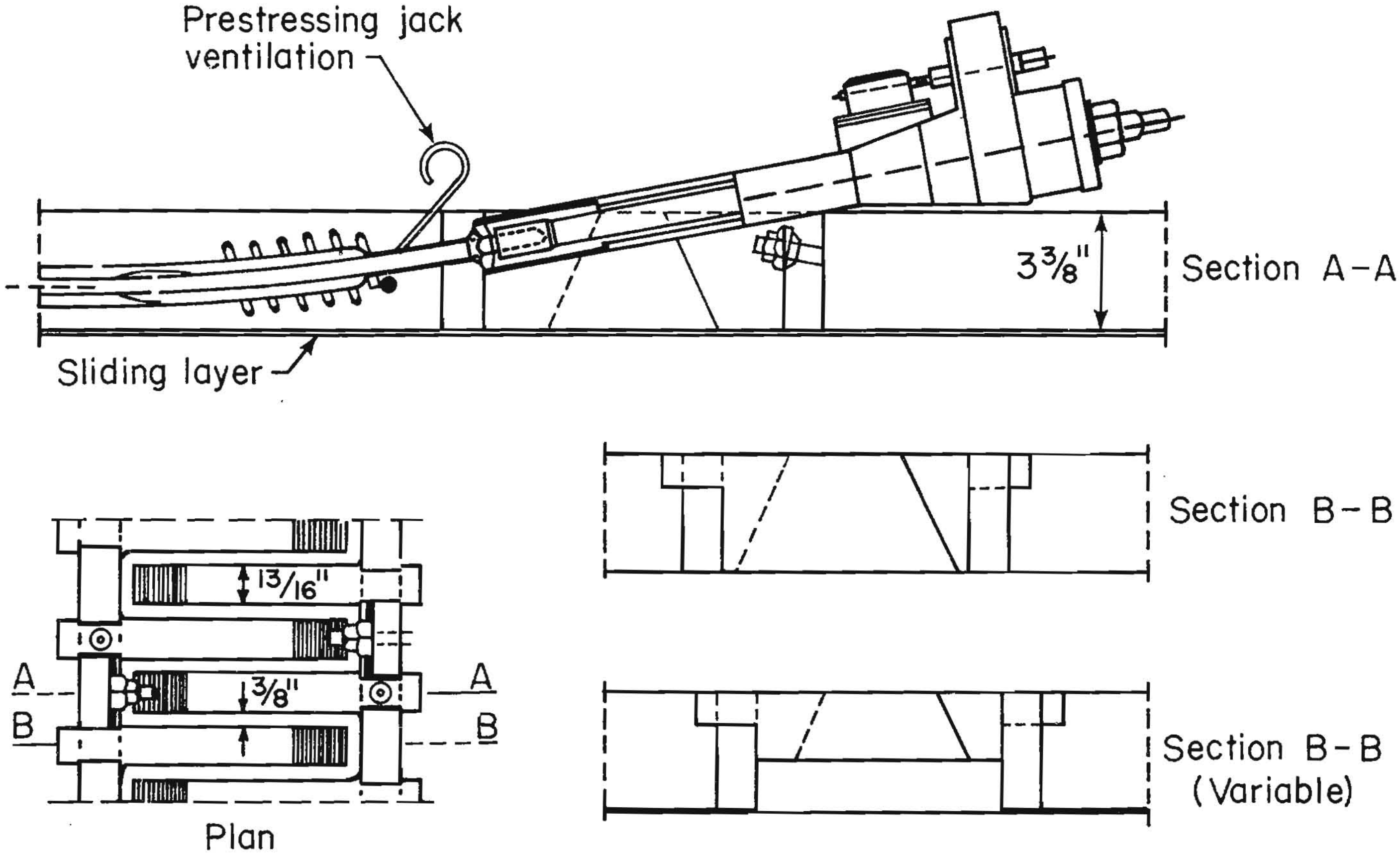


FIG. 24 - FINGER JOINT FOR ROAD AT MONTABAUR, GERMANY

commonly experienced at slab ends. However, with this joint, no subsequent concreting is necessary to fill a jacking gap.

Prestressing tendons are anchored in the vertical edge sections of the joint. Thus, the joint metal is held firmly to the ends of the slab by the prestressing forces. Repair of the finger joint can be accomplished by removal and replacement of sections of the toothed joint. This is done without releasing the prestress in the slab because of intermediate anchorages placed to the rear of individual finger joint sections.

Another joint designed to use the pretressing forces in fixing hardware to the slab end was installed on airfield pavements in Germany at Memmingen in 1956 (120), at Diepholz in 1958, and at Wunstorf in 1959 (168). The joint and the adjacent slab ends were placed on an approximately 4.6-ft (1.4-m) wide sleeper slab provided with a water discharge channel, as shown in Fig. 25. An approximately 30-in. (762-mm) wide joint gap was left open for stressing the tendons. Tendons were connected to turnbuckles upon completion of prestress application and grouting of tendon conduits. The other ends of the turnbuckles were welded to vertical edge strips forming the open movement joints faces. The two edge strips were temporarily connected with bolts until the turnbuckles were tightened and the concrete used to fill the gaps had hardened.

This joint was reported to be performing well in service, although collection of incompressible materials between the joint faces has not always been prevented. The design was refined for subsequent applications by the addition of a steel cover plate.

An 8-ft (2.4-m) joint gap is planned for the Mississippi prestressed pavement scheduled for construction in September of 1976. Two joints, one at each end of the gap slabs, accommodate slab movement. Gap concrete will not be prestressed, but will be reinforced in both directions

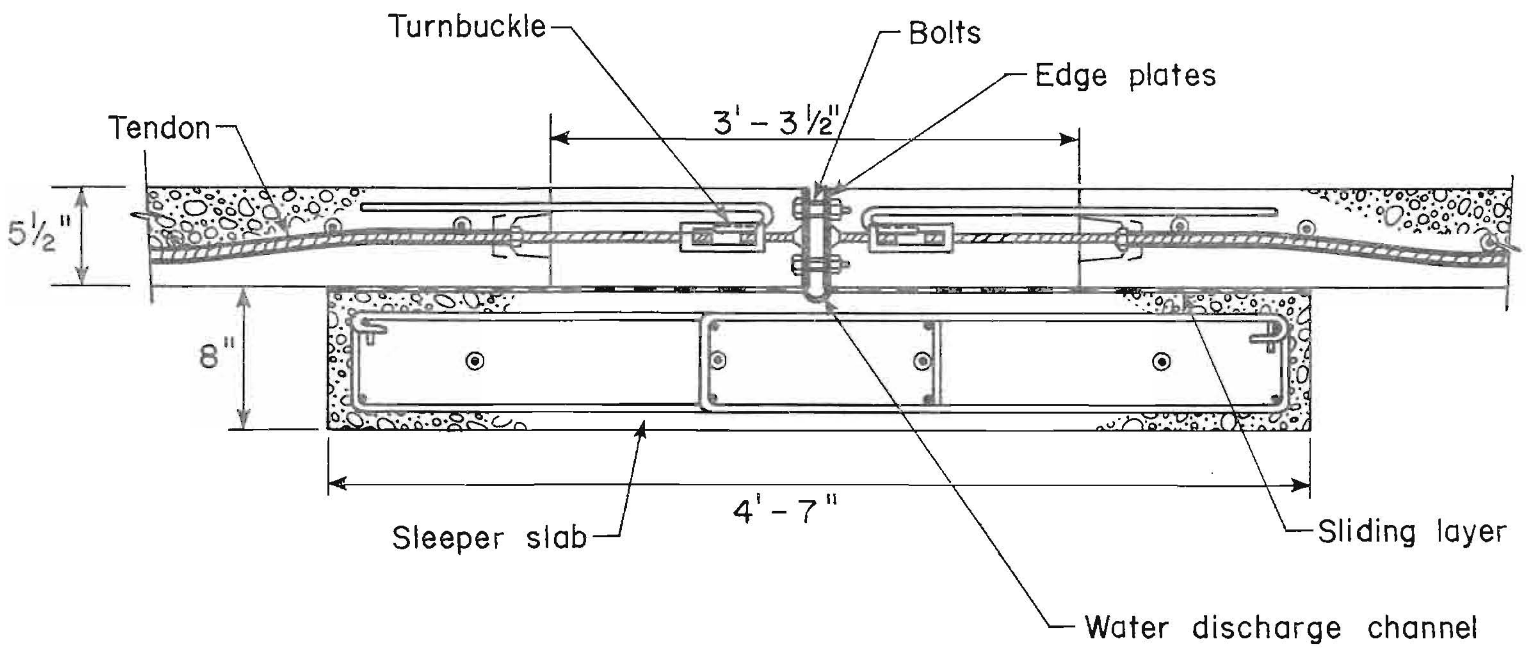


FIG. 25 - OPEN JOINT FOR AIRFIELD PAVEMENT AT MEMMINGEN, GERMANY

with two layers of deformed bars. Dowels will be used between the prestressed slab ends and the gap slabs.

Prestressing loads will be transferred to the vertical face of slab ends by 4-1/2x3-1/2-in. (114x89 mm) steel plates. No other steel plates or beams will be used to protect the concrete at the joints. Joint width between the ends of the prestressed slabs and the gap slabs will vary from 1/2 to 2 in. (13 to 51 mm). The lower 3-1/2 in. (89 mm) of the space will be filled with styrofoam. A 2-in. (51-mm) thickness of elastomeric joint material will be placed over the styrofoam.

Covered

Covered joints were included in the Fontenay-Tresigny bypass road demonstration project. A free covered joint was constructed with a toothed sliding plate. The plate was fixed to the steel edge strip. Offset opposing fingers on the opposite side of the joint were part of the vertical edge strip construction. The joint and slab ends were placed on a 72-ft (22-m) long sleeper slab of variable thickness. Sleeper slab ends were 13-in. (330-mm) thick and were reduced to 7.9 in. (201 mm) for a distance of about 33 ft (10 m) at the center. The prestressed roadway thickness was 4.7 in. (119 mm) However, the thickness increased to 9.8 in. (249 mm) at the slab ends located over the reduced thickness sleeper slab. Cables approximately 33-ft (10-m) long were used to post-tension the two slabs terminating at the joint. The 12 wire cables were anchored in the sleeper slab and were stressed from the joint gap. The unbonded wires acted as a spring in a manner similar to the Maison Blanche[113] prestressed pavement. In addition, the resultant forces counteract the direction of pavement curling thus assuring good contact between sleeper slabs and the ends of the prestressed slab.

A simple form of joint with a cover plate was used in two German airfield pavements in 1959 and 1960[199]. The cover plate rested on two angles with unequal legs as shown in Fig. 26. It was fixed to the narrower of the two legs. The concrete surface at the extremes of the wide angle leg was protected by a 1-in. (25-mm) wide piece of flat stock that was welded to the top of the angle.

The edge sections of the joint hardware were fixed to the slab by prestress. The joint consisted of a number of sections in the transverse direction. Each section was anchored by a prestressing strand. Placement of gap concrete was not necessary and the pavement could be placed with conventional construction equipment. Prestress was applied by inclined jacks prior to placing the cover plate as was done with the finger joint at Montabaur.

Another form of covered joints was developed in Germany for 2460-ft (750-mm) long pretensioned concrete pavements with 492-ft (150-m) joint spacing[264]. The cover plate, as shown in Fig. 27, was bolted to the top of one of the angles and slides on the leg of the angle forming the opposing joint face. Prestressing wires extended through the joints for the entire 2460-ft (750-m) long pavement section. After stressing, the tendons were clamped to the joint angles with anchor plates. The wires were cut between the joint faces after the concrete had hardened. However, care was taken to transfer forces slowly to the edge joint and slabs.

Anchor plates clamped to the pretensioned wire assured that the prestress was transferred to the joint hardware. The hardware was held tightly against the concrete and prevented separation between the steel angles and the concrete. Gradual release of the pretensioning forces into the joint face stressed the slab and prevented early shrinkage cracking. Sleeper slabs below each of the joints minimized deflections and stresses due to traffic loadings.

A prestressed pavement demonstration project consisting of 23 post-tensioned slabs with a total length of 13,232 ft (4 km) was built in 1973 in Cumberland County, near

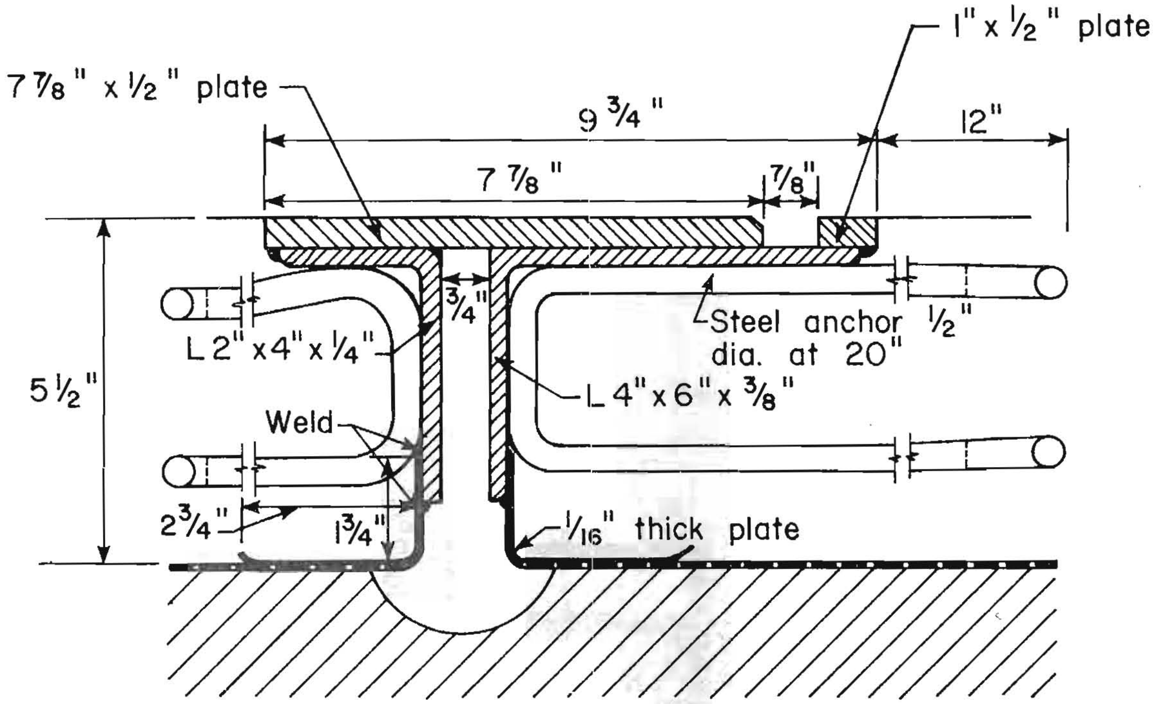


FIG. 26 - COVERED JOINT FOR AIRFIELD PAVEMENTS IN GERMANY

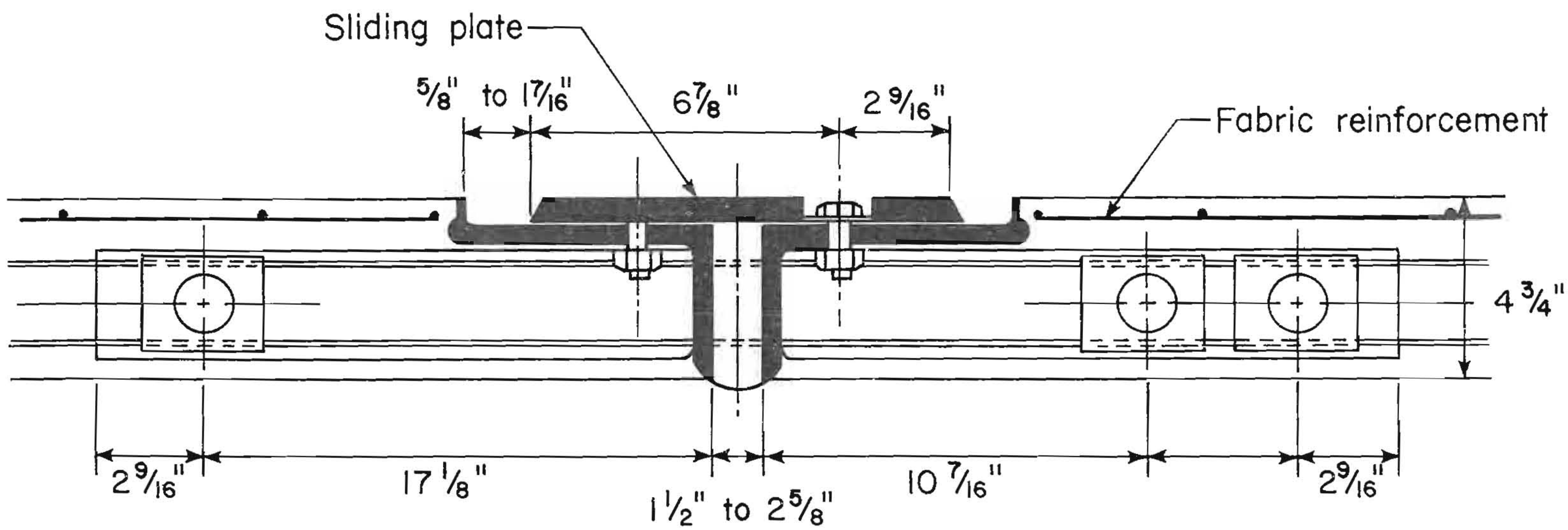


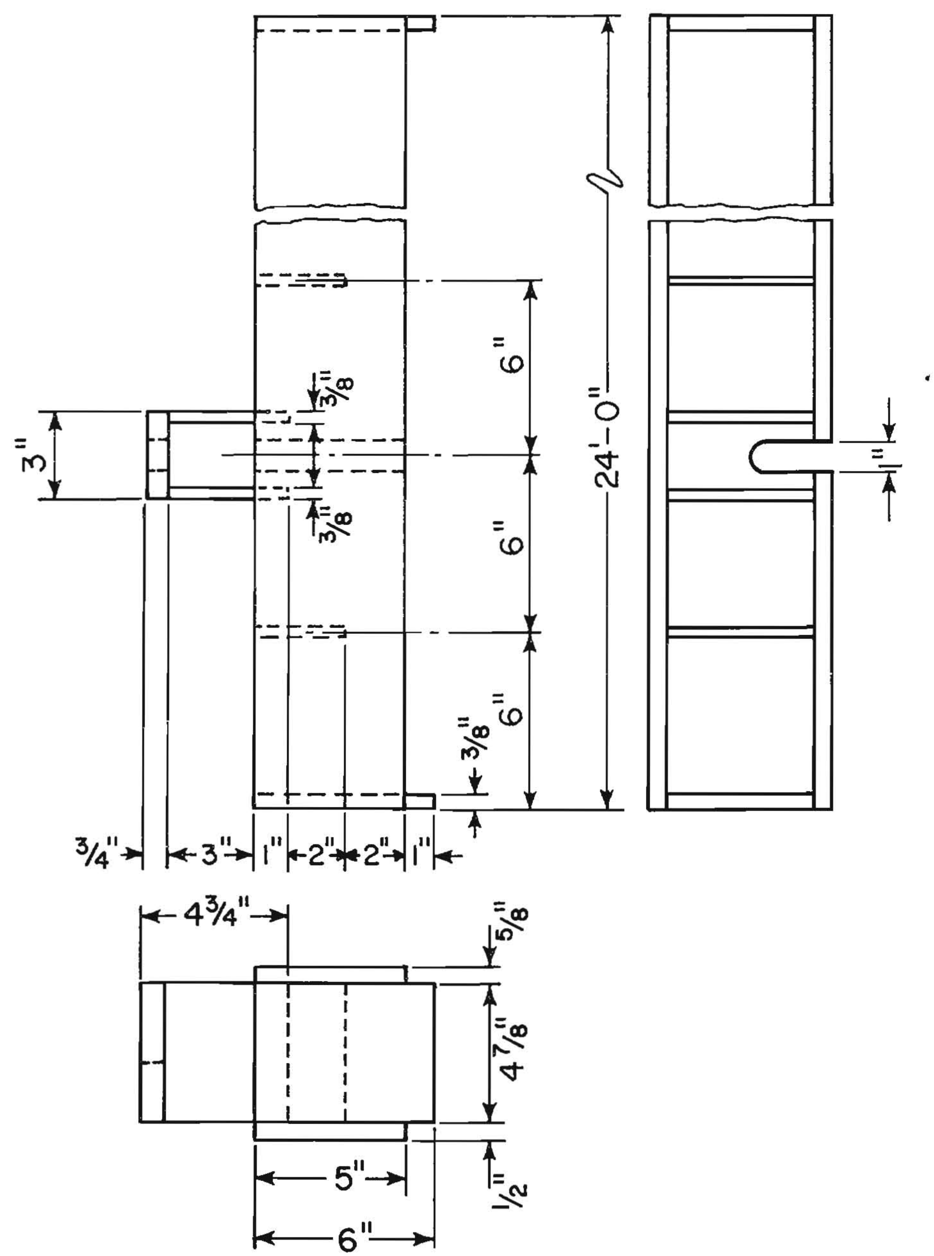
FIG. 27 - COVERED JOINT WITH SLIDING PLATE

Hogestown, Pennsylvania[288]. Each of the interior joints consisted of a female beam, a series of short male channels, and 1/4-in. (6-mm) spacers of expanded foam sheeting between the male beam and the web of the female beam. Details of the male channel and female beam are shown in Fig. 28.

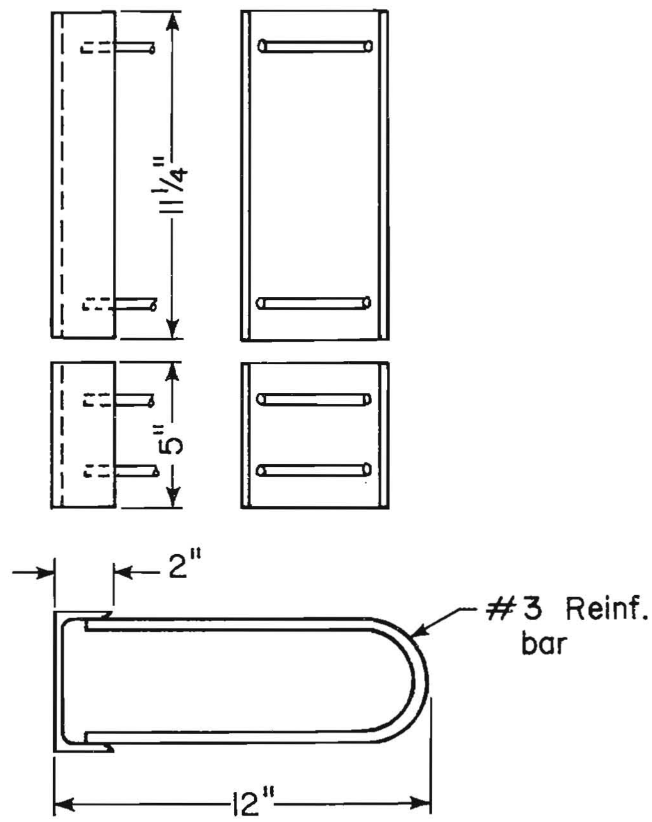
Anchor pockets in the female beam were used to secure the strands as shown in Fig. 29. Each male channel was secured to the gap concrete with a U-shaped reinforcing bar welded to the flanges of the channel. The top flange of the female beam slid over the top of the flange of the male channel. Male channels act as a load transfer device for the joint system. This eliminated the need for a sleeper slab to reduce stresses and deflections at slab ends.

Joint details and construction sequences for slip form paving method at the Harrisburg demonstration project were as follows:

1. The subbase was compacted and covered with a thin layer of sand to level minor subbase surface irregularities.
2. A double layer of polyethylene was fed from rolls onto the subbase.
3. Polypropylene sheathed wire strand was fed from reels carried on a truck bed onto the polyethylene covered subbase.
4. The slipform paver spread, consolidated, and finished the concrete. The covered prestressing strand was fed through tubes on the paver and positioned to the required elevation.
5. At joint locations, blockouts were used to assure continuity of strand and provide a space for joint construction. Blockouts are illustrated in Fig. 30. The female beam was placed at one end of the slab with the other end formed by a bulkhead. Both the female

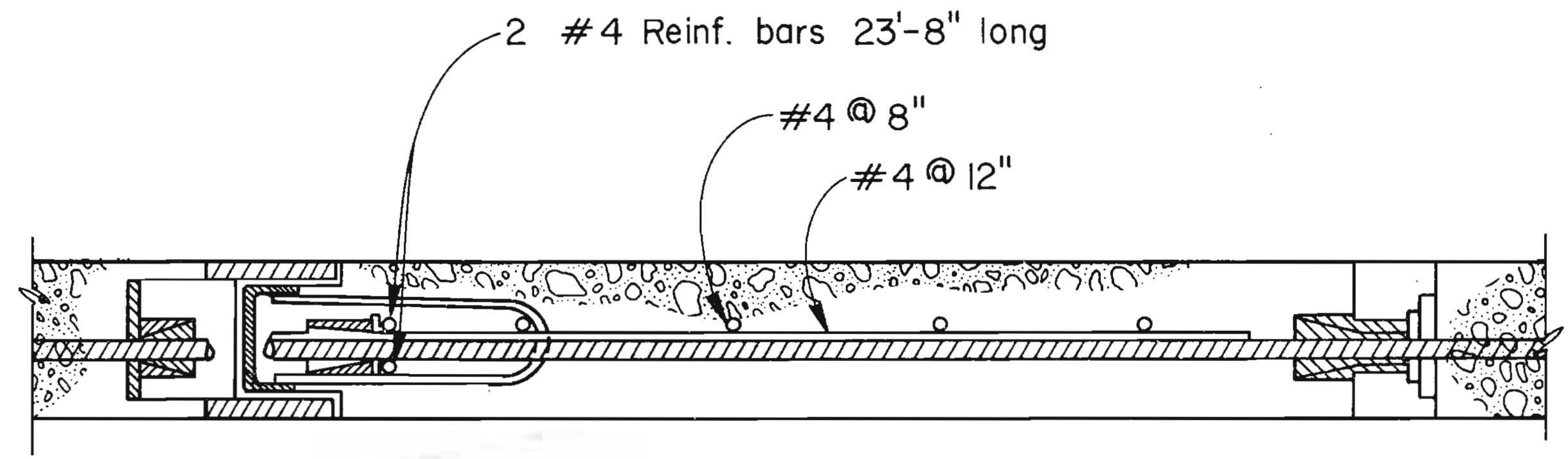


Female Beam

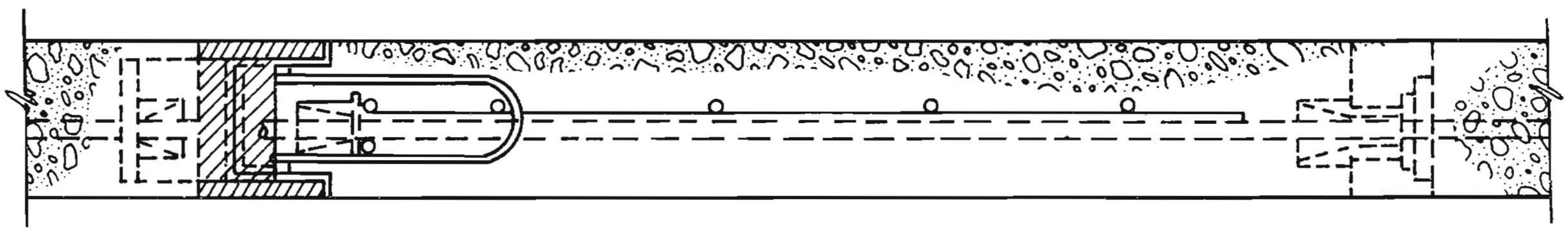


Male Face Channel

FIG. 28 - DETAILS OF FEMALE BEAM AND MALE CHANNEL FOR JOINTS ON PAVEMENT AT HARRISBURG, PENNSYLVANIA



Section Through Tendon Centerline at Joint



Section Through Vertical Member of the Female Beam

FIG. 29 - DETAIL OF ANCHOR POCKETS

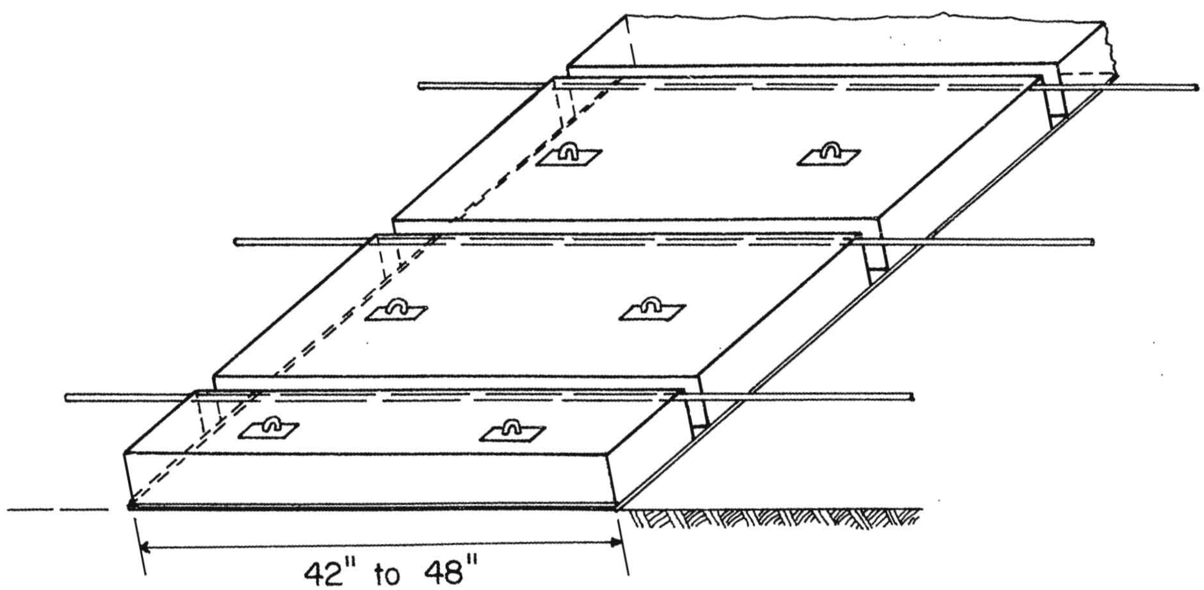


FIG. 30 - BLOCKOUT FOR JOINTS

beam and bulkhead had cutouts to permit them to be put into place above the strand.

6. The bulkhead was removed and a jacking bridge was placed at the vertical concrete face. Then the strand was cut and stressed.
7. The male beams were positioned and the U-shaped rebars were connected to the strand extending to the jacking space. Blockouts were placed around the jacking bridges.
8. The jacking space concrete was cast and cured. After hardening, the jacking bridge was cut. This prestressed the jacking gap concrete and tightly closed the construction joint between end and jacking gap concrete.

Closed

Closed joints are designed to prevent the intrusion of water and debris into the joint space. It is required that the joint be level with adjacent pavement in both the expanded and compressed condition. Movements are accommodated by laminated construction.

The B. F. Goodrich laminated closed joint was used for the Jones & Laughlin experimental pavement[145] in Pittsburgh, Pa. The joint was embedded in the slab faces with steel anchor bars. Prestress was produced from a jacking joint in the middle of the slab. Slab ends and the laminated neoprene joint were supported on a sleeper slab. Ends of the laminated, nine cavity construction were covered with a rubber slab and a bolted-on cover. Thus, water and debris were prevented from entering the joint space.

A similar laminated neoprene joint was used at the Lemoore Naval Air Station in California on a prestressed taxiway pavement[213]. This joint was observed to be in good condition ten years after installation.

A laminated neoprene joint was also used between six 492-ft (150 m) long prestressed slabs at the Dietersheim-

Bingen experimental roadway[192]. After stressing the slabs, the laminated joints were inserted in a joint gap between the prestressed slabs. Initial methods of inserting the laminated joint required an external I-beam framework to compress the joint to the width of the opening. Hydraulic presses then forced the joint into the gap.

A later method of joint insertion required a metal framework located on each side of the laminations for the length of the joint. The framework became part of the joint structure. Temporary bolts were used to compress the joint between the lateral steel framework so it would fit easily into the joint gap. The bolts were removed after installation and the joint then expanded against the slab ends.

The laminated joint was constructed, as shown in Fig. 31, with a series of 12 laminations. Each series consisted of two double cavity neoprene tube sections and steel plates. Upward vertical displacement was prevented by a series of steel guide pins fixed alternately to opposite sides of the joint. The ends of the slab and the laminated joint were supported on a sleeper slab. The steel plates of the laminated joint could slide on steel rails located in the top of the sleeper slab.

Joints closed with polyurethane foam were used in the prestressed pavement demonstration project at Dulles Airport[275]. An 8-ft (2.4-m) wide joint gap was provided between each of the six prestressed slabs. Gap concrete was not stressed but was reinforced with deformed bars. Two sets of joints, as shown in Fig. 32, were used between ends of prestressed slabs. This procedure reduced the magnitude of joint movement to be accommodated between slabs.

Each joint was formed with two I-beams as shown in Fig. 33. The beam fixed to the prestressed slab was used to anchor the prestressing wires and provide bearing against the fresh concrete during early prestressing. The other beam was fixed to the joint gap concrete. Load transfer across the joint was obtained with dowel bars. The top

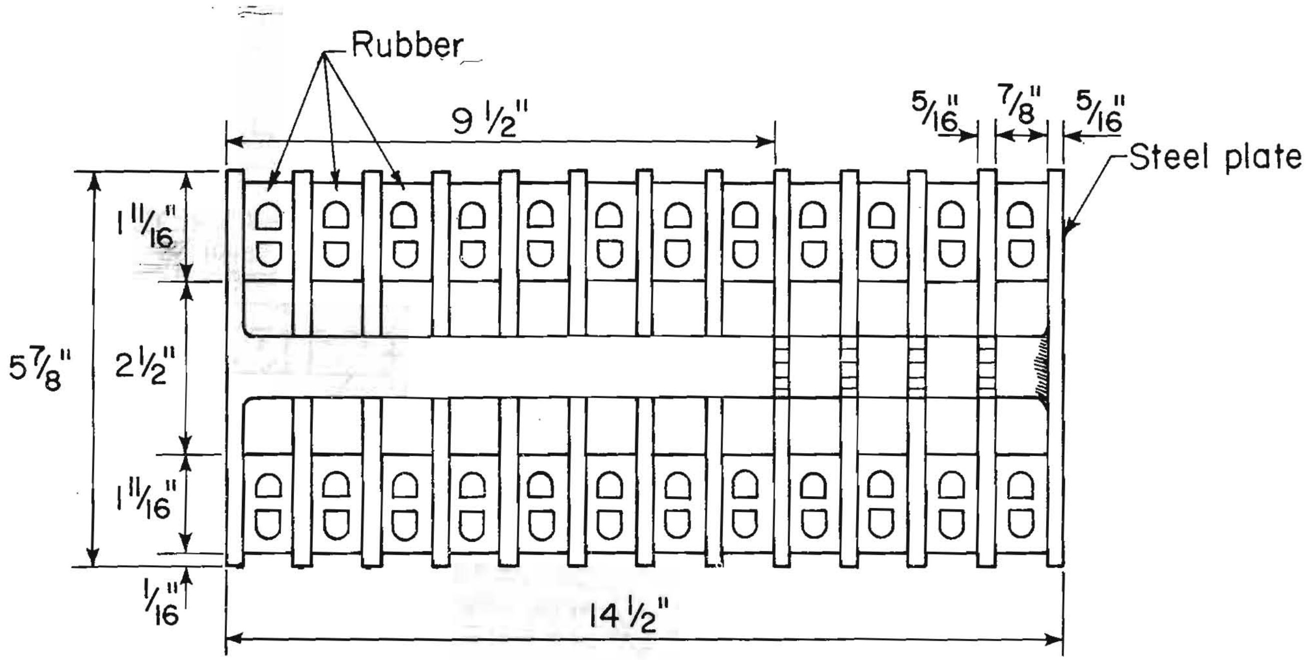


FIG. 31 - LAMINATED JOINT FOR ROAD AT
DIETERSHEIM/BINGEN, GERMANY

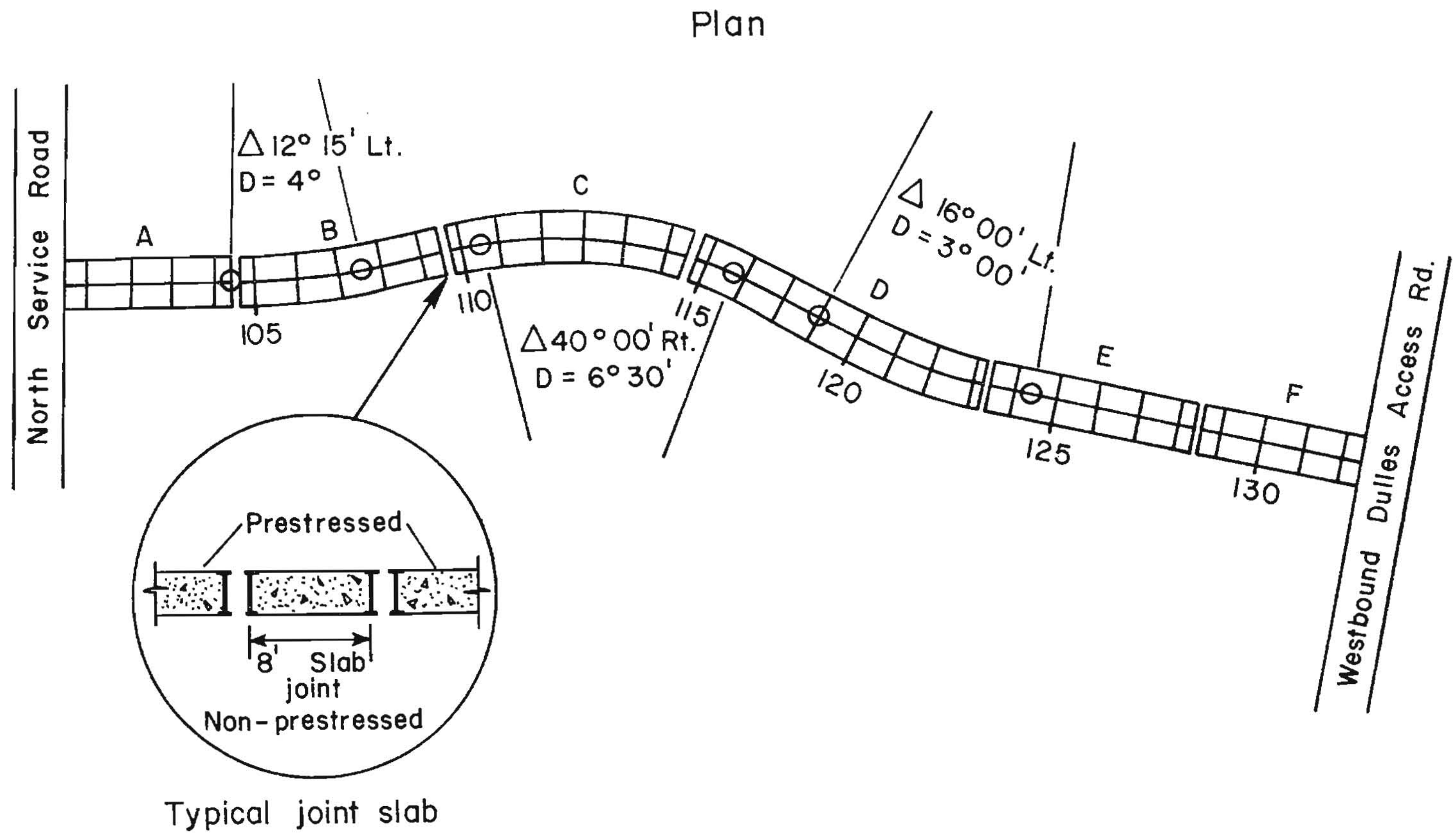


FIG. 32 - LAYOUT OF PRESTRESSED CONCRETE PAVEMENT AT DULLES INTERNATIONAL AIRPORT, VIRGINIA

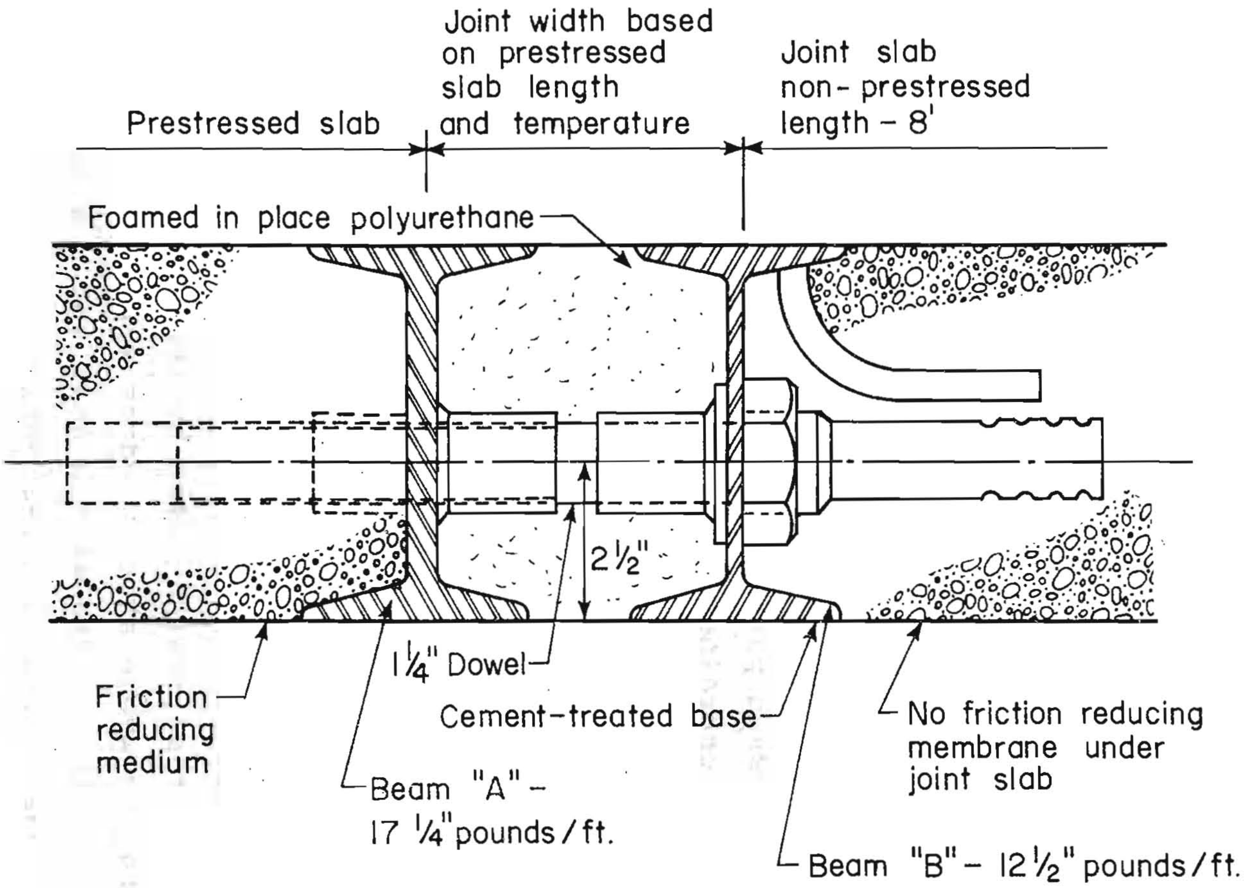


FIG. 33 - JOINT DETAIL FOR PRESTRESSED PAVEMENT AT DULLES INTERNATIONAL AIRPORT, VIRGINIA

flange of the I-beam was designed to carry compression in periods when temperature exceeds $100^{\circ}F$ ($43.3^{\circ}C$). However, it is not known if the joint closed to mobilize this design provision.

A steel drainage box and plastic drain were provided at the low side of the joint. This was done to prevent accumulations of water. However, the polyurethane foam was not able to adjust to the joint width changes and water and debris entered the joint. Therefore, the joint beams and attached reinforcing were torn away from the gap concrete. This separation may have been due to rusting and freezing of dowels.

Compression Joints

Compression joints are used in the construction of externally prestressed pavements. In this system, movements of the slabs are restrained by abutments or anchor systems. Jacking joints located at the slab center of internally prestressed pavements may also be considered compression joints. In this system, the joint is only a construction expediency and slab movements are accommodated at the free joints at the ends of the slabs.

Rigid

Transverse concrete wedges were used in Switzerland on the Moriken-Brunegg experimental prestressed roadway constructed in 1956[126]. The wedges, shown in Fig. 34, were forced into the jacking joint by tensioning of tendons oriented along the wedge axis. Wedges were heavily reinforced to prevent formation of cracks and spalling. A small space at the center was filled with removable material to facilitate later removal for additional prestressing. This was done to compensate for losses of stress due to creep and shrinkage. The prestressed slabs contained no internal reinforcement.

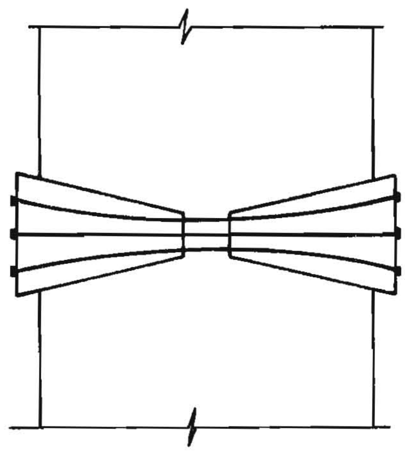


FIG. 34 - WEDGE PRESTRESSING METHOD

There was no movement in these slabs at the jacking joint and the other end was restrained by abutments.

Wedge shaped jacking joints were also used on a highway pavement near Salzburg, Austria[193]. The Salzburg pavement was about 2625-ft (800-m) long. End reaction to prevent pavement movement was provided by a transverse concrete lug, extending about 5 ft (1.5 m) into the subgrade, plus a concrete compression abutment. Six wedge shaped jacking joints were constructed at about 425 ft (130 m) on centers. The wedges, as shown in Fig. 35, tapered at 1 in 20 and were about 3-ft (1-m) wide at the outer edge.

Joint sides of the wedge and ends of the slabs were armoured with steel plate. A groove was provided in the steel plate attached to the joint face on the slab ends. This served as track for the guide bolts placed transversely through the jacking wedge.

The wedges were forced into position with six tendons that were removed upon completion of stressing operations. Friction forces across the joint faces locked the tapered wedge in place.

Leonhardt[193] reported that care should be taken in the design of the steel armour sheets to facilitate sliding along the joint faces. Insufficient thickness of the steel section led to installation problems. The reinforcing in back of the joint was also of concern. Cracks were observed to form during stressing. These were due to force concentrations associated with unequal bearing across the joint face.

Prefabricated jacking blocks were used in France on a number of installations. They were used on the Bourg-Servas road in 1953 and an experimental road near Naz, Switzerland in 1955[123]. Jacking blocks consisted of two or more Freyssinet flat jacks encased in a concrete block. The blocks were pumped to expand the concrete and crack the block. This process stressed the slab. After prestressing was completed, the flat jacks were filled with cement mortar.

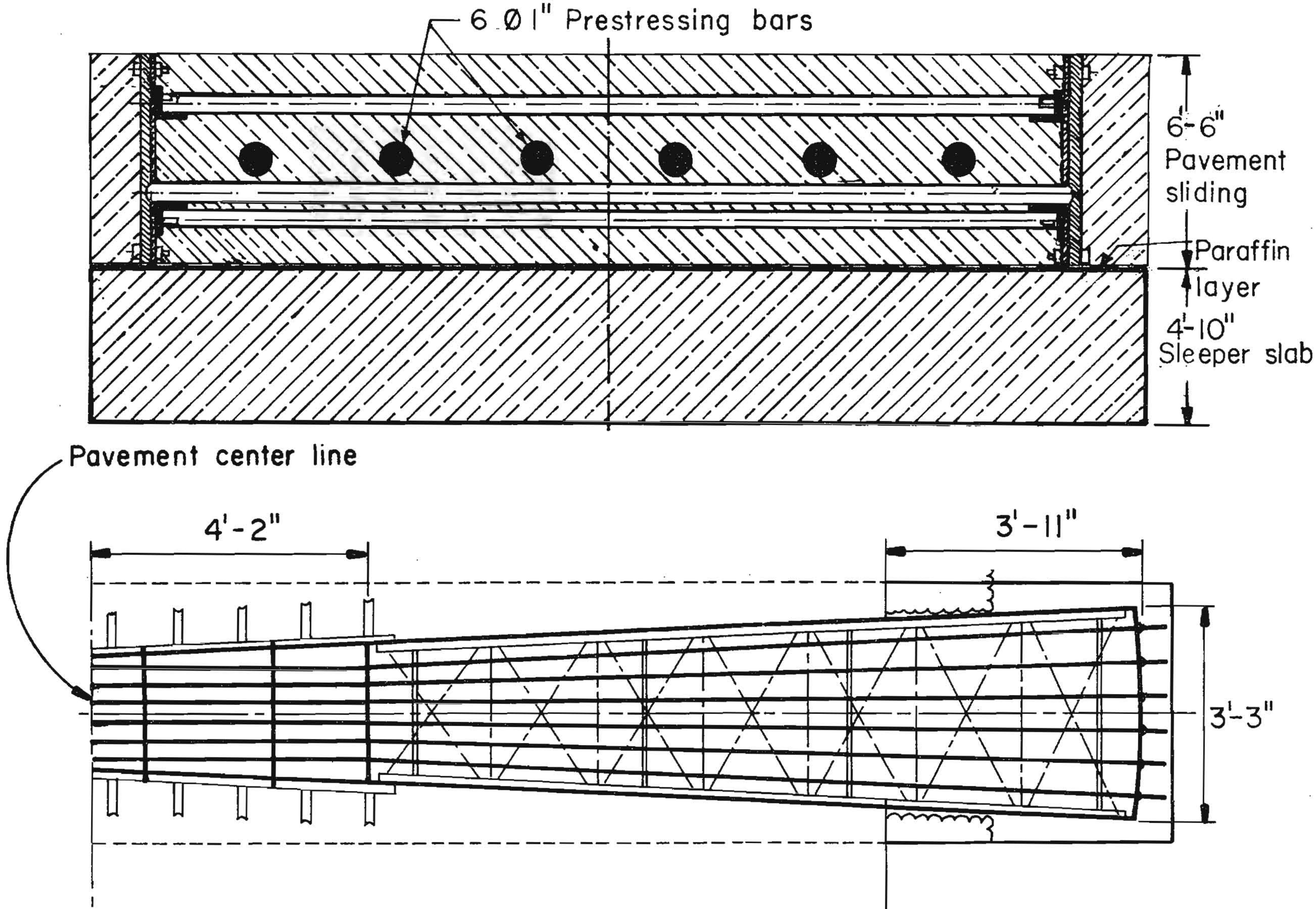


FIG. 35 - ARRANGEMENT OF TENDONS IN PRESTRESSING WEDGES

Then, the jacking blocks were covered with concrete to complete the joint. Problems have been encountered because of the small thickness of the concrete layer over the blocks[264].

Pavement blowups were prevented by an interlock system at the bottom of the slabs. Special anti-lifting devices were attached alternately to the two slab ends. They consisted of rigidly fixed teeth that slid under the opposite slab.

Flat jacks were connected with automatic hydraulic pumps on a second Swiss prestressed pavement built in 1960[228]. The automatic hydraulic system was used to apply uniform prestress with time.

At Fontenay-Tresigny[197], flat jacks were joined together and were placed transverse to the roadway ahead of concreting operations. At least three flat jacks were provided in the compression joint. The first was used for the initial pump stressing. The other two were planned for stressing to replace prestressing losses. The anti-blowup interlock system was replaced by using 1-3/4 in. (44.5 mm) diameter dowels.

Flat jacks, piston jacks, and screw jacks were used on the externally prestressed pavement built at Winthorpe, Nottinghamshire, England[256] during 1961. The flat jacks were not able to develop sufficient force for prestressing and leaks occurred in the brazed seams. Hydraulic piston jacks were subsequently developed by the Road Research Laboratory. About 40 of these jacks were used in one 24-ft (7-m) transverse jacking joint. Each jack developed a thrust of 75,000 lbs (33.4 kN). Screw jacks were used as replacements for the hydraulic piston jacks to maintain prestress while the blocking concrete gained strength. This procedure made the piston jacks available for use at other locations.

Mobile

The wedge type jacking joint used at Fontenay-Tresigny was different significantly from those used at

Salzburg. The single wedge was asymmetrical. Steel balls reduced the resistance to movement at the joint faces. A capping slab was used to anchor one end of the prestressing tendons.

The advantage of this system was the ability of the joint to slide without increasing joint width. This reduced the potential for buildup of high stresses with rising temperatures. Movement of the wedge was balanced by the tendon tension forces and the compression forces in the prestressed slab. The wedge with rollers provided less flexual rigidity in the joint area as compared to the rigid wedge system used near Salzburg.

Two types of pneumatic cells were used at Fontenay-Tresigny[197]. One consisted of a single rubber tube enclosed by two deep channel sections, as shown in Fig. 21. The channel sections were forced apart, as air pressure was increased in the tubes. This transmitted compressive forces into the roadway slab. The joint was able to move over a distance of about 1/2 in. (12.7 mm) and the working pressure range was 280 to 560 psi (1.9 to 3.9 MPa). Slab lengths were 230 ft (70 m).

A second pneumatic tube cell is shown in Fig. 22. It is a double pneumatic tube in combination with finger joint construction. The pressure range for this cell was 425 to 1000 psi (2.9 to 6.9 MPa). The cell could expand 3 in. (76 mm) Nitrogen was used in place of air for this cell.

The benefits of the pneumatic compression joint design are:

1. Initial prestress can be easily adjusted to prevent shrinkage cracking.
2. The pressure can be easily to adjusted compensate for losses.
3. Wheel loads are transmitted by the steel channels or finger joint construction to the sleeper slab used for both types of construction.

Sliding Joints

Sliding joints were used for construction of a 355-ft (108-m) long taxiway at London Airport[48] and a 1377-ft (420-m) long runway section at Orly, France[3]. The London pavement was constructed in 1949 and the Orly runway in 1946 to 1947. Unique to the construction of these runways was the use of precast elements that were set in place and post-tensioned with wire strands. The French precast elements were 3.28 sq ft (0.31 m^2) and the slabs used in London were 2.9 sq ft (0.27 m^2). All elements were 6.5-in. (165-mm) thick.

The 1377-ft (420-m) long and approximately 120-ft (37-m) wide runway at Orly was divided into six isoceles triangles with about 394 ft (120 m) at the hypotenuse. The 45 degree triangles were, separated by roller joints, as shown in Fig. 36. Steel rollers were placed in the gaps between joints. Transverse cables placed in the gaps were used to stress the pavement. Elongation in the longitudinal direction was prevented by abutments. The pavement elements were under continuous prestress. High pavement stresses were avoided by the lateral movement of the triangular sections, in the directions indicated in Fig. 37.

PERFORMANCE, LOAD TESTS, AND MEASUREMENTS

Although many prestressed pavements have been in-service for a number of years, only limited information has been reported on their performance. Load test information obtained during construction has been reported in the literature. However, in many cases the data are reported without discussing the procedures or details of the test. Because this knowledge is necessary for an independent analysis, no conclusions are made; although data conflicts are sometimes acknowledged.

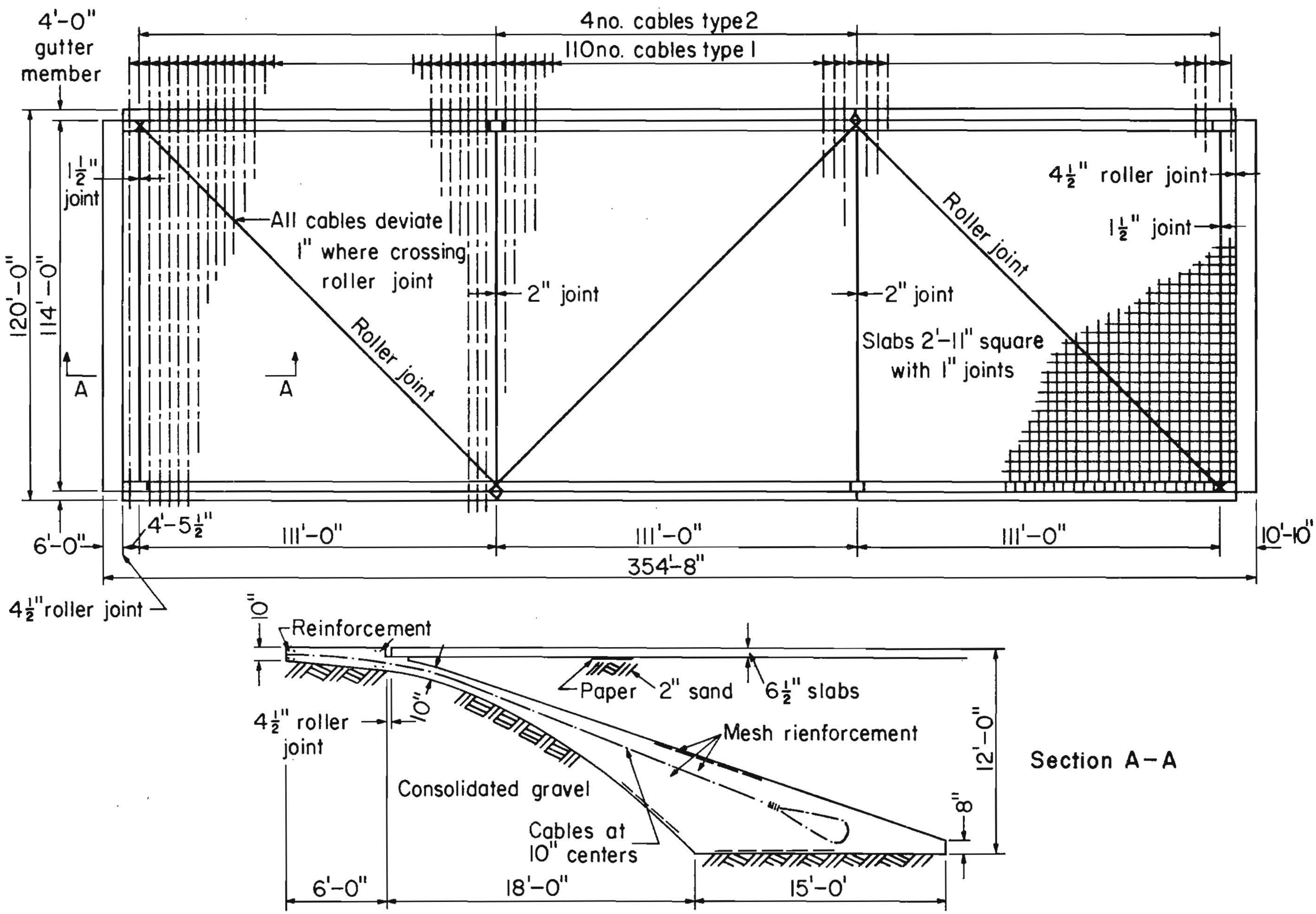
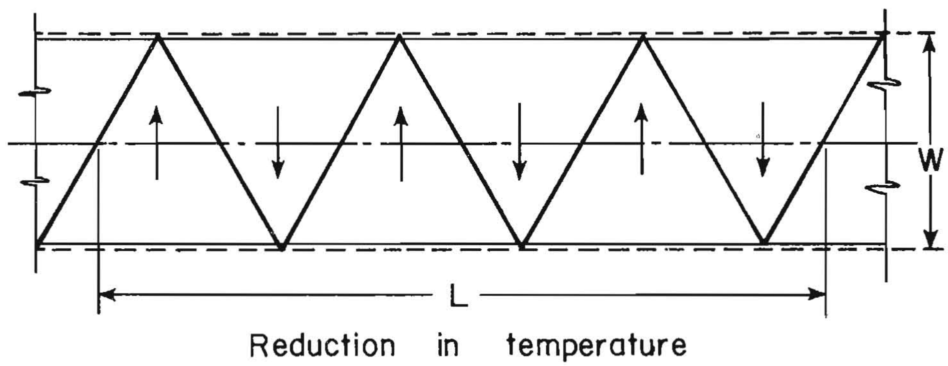


FIG. 36 - LAYOUT OF SLABS AT ORLY RUNWAY, FRANCE



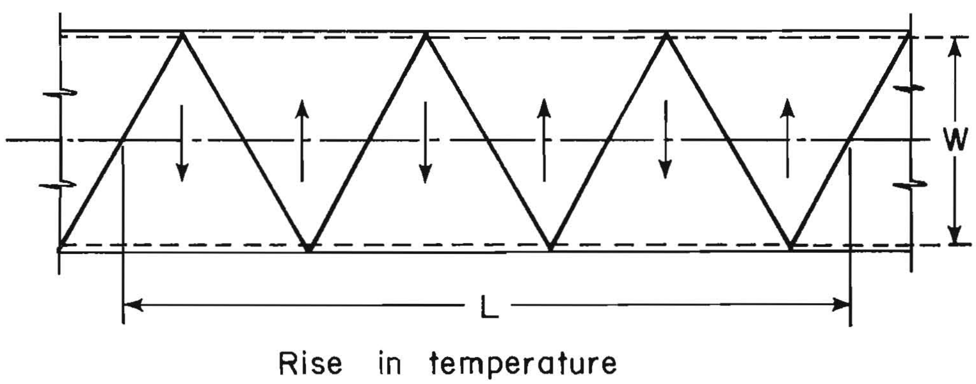


FIG. 37 - EFFECT OF TEMPERATURE CHANGE ON SLIDING JOINT

Performance Under Traffic

In-service performance is reported for four prestressed pavements located in the United States. The discussions are based on available literature plus prior knowledge and personal inspections of the projects.

Route 114 Near Harrisburg, Pennsylvania

Route 114 near Harrisburg, Pennsylvania is a four-lane divided highway, approximately 1,000 tractor-semitrailer vehicles use this route daily. The present serviceability rating was 3.9 in July, 1976. Present serviceability measurements made in 1975 with a Pennsylvania D.O.T. Mays Meter averaged 3.8.

An intermediate narrow transverse crack was observed in July, 1976, in two of the 23 pavement slabs. There was no spalling at either crack. Neither of the two cracks was observed at the time of construction and prestressing operations.

Slight spalling was observed in the concrete at the jacking gap. The spalls were less than 1-in. (25-mm) deep and not greater in area than about 15 sq. in. (9677 mm^2). They appeared to be due to insufficient clearance between the top flange of the female beam and the gap concrete.

Repairs have been made at two of the 19 joints. At these locations the female beam separated from the slab concrete for a distance of about 2 to 3 ft (0.6 to 0.9 m) inward from the edge of the outside lane. It may be that the male and female beam interlocked during pavement temperature contraction. It was reported that broken anchor pocket welds were noted during the pavement repair.

Repair at a second joint was due to gap concrete failure at a male beam that had interlocked with the female beam. Insufficient male to female beam clearance may have contributed to this joint failure.

Shoulder distress was observed along many of the slabs. Open joints with shoulder drop-offs in excess of 2 in. (51 mm) had occurred at a number of locations. The shoulder was constructed as an aggregate base with a double application of bituminous material. Evidence of subbase pumping was not noted during the July 1976 inspection and none has been reported.

Route 222 Bypass near Kutztown, Pennsylvania

The Route 222 bypass is a 500-ft (152-m) long prestressed slab. It was constructed in September, 1972. A transverse crack located about 300 ft (91 m) from the east end of the slab extended across both traffic lanes. This crack was reported to have occurred about one year after slab construction. The crack was about 0.05-in. (1.27-mm) wide at the slab surface. An oval shaped crack about 15-ft (5-m) long occurred in the outside traffic lane about 50 ft (15 m) from the west end of the slab. This crack was reported to be due to an overload during shoulder construction. No new cracking was observed during the July, 1967 inspection. Spalling of the concrete was not observed at either the transverse or the oval shaped crack.

Taxiway T-3 at Biggs Air Force Base, Texas

A 1500-ft (457-m) long post-tensioned segment of Taxiway, T-3, was constructed in 1959. During the July 1976 inspection no change in pavement condition was observed different from that reported upon completion of construction. Transverse cracks that occurred during construction in the slabs at the ends of the taxiway had been grooved to a depth of 1.5 in. (38 mm) and filled with an expoxy resin grout. No evidence of movement was observed at these cracks.

Serious joint performance problems were encountered. Initial construction required two 15-ft 4-in. (4.7-m) intermediate gap slabs between the 500-ft (152-m) prestressed slabs and two 9-ft 8-in. (2.9-m) joint slabs at the ends of the pavement. One, 1/4x1-1/2-in. (6x38-mm) contraction joint was provided in the center two joint slabs. Polyurethane foam filled, 1-1/2-in. (38-mm) wide expansion joints were constructed in the joint slabs at the ends of the section. The contraction joints at the intermediate gap slabs opened about 1 in. (25 mm). These joints were then filled with polyurethane foam in combination with a polysulfide rubber joint sealer. Because of continuing joint performance problems, the sealer was removed and the joint was filled with concrete.

Forces due to restrained temperature expansion caused spalling and crushing of the intermediate and end joint slabs. Removal and replacement of 8-ft (2 m) wide and 5-ft (1.5-m) wide sections of these slabs was initiated in 1960. Two expansion joints were provided in each of the intermediate slabs and one in each of the end slabs.

One, approximately 1-1/2-in. (38-mm) wide opening was observed in each of the end joint slabs when the pavement was inspected in March, 1962. No spalling or crushing of the gap slab was observed at that time, but distress was noted in the cork asphalt joint filler and sealant. During an October 1963 inspection, it was observed that none of the transverse joints had retained their sealant.

Premolded neoprene joint sealants were placed into the transverse joints during 1964. The uncompressed premolded seal width was about 1-1/2 in. (38 mm). The material was compressed to about 1 in. (25 mm) at the time of pavement inspection during July 1976. The top of the premolded sealant was squeezed from the joint and had been cut by aircraft traffic.

Dulles Airport Access Road, Virginia

Distress was experienced at the joints of the prestressed pavements located at an access road to Dulles

International Airport. Several of the joints were removed and replaced after the gap concrete separated from the I-beam. This was reported to be due to inadequacies in design details of the reinforcing of the gap concrete and to freezing of dowels.

Transverse cracks occurred in several of the prestressed slabs. The first crack was observed in the 760-ft (232-m) long slab approximately one month after construction. Formation of this crack occurred after a greater than 40°F (4.4°C) concrete temperature drop. New transverse cracking was noted during the January 1975 and 1976 pavement inspections. More than one crack was noted in some of the slabs. However, there was no significant surface spalling at the cracks.

Traffic on this road consists primarily of passenger cars. Truck use was limited to the early period of pavement life. The pavement was observed to be smooth and provided a comfortable ride at 40 mph (64 km/h). Only slight jarring was noted at the approximately 3-in. (76-mm) wide open joints.

Load Tests

Data are reported from load tests on full-scale pavements, small experimental slabs, and scale models. Load tests related to airfield pavements were generally made at an interior position while those concerned with highways were made at an edge or corner. Both static and moving load data are reported.

Airfield Static Load Tests

From its inception, prestressing appeared readily adaptable for use in airfield pavements. Numerous experimental installations were constructed and load tested to determine structural capacity. The results from several of these test programs are reported.

Orly Airfield, France: An experimental slab, 46 ftx41ftx6.3 in. (14x12.5x0.16 m) was constructed and post-tensioned at Orly Airport[105] in 1946. Part of the slab was built on a 33-in. (838-mm) thick subbase and another part on a 2-in. (51-mm) thick subbase. Prestress was adjustable up to 1,200 psi (8.3 MPa) in the slab and 125 ksi (862 MPa) in the steel.

The maximum available load of 314 kips (1397 kN) was applied at the slab interior on the portion of the pavement over the 2-in. (51-mm) thick subbase. The prestress was lowered gradually. When the slab prestress reached 140 psi (965 kPa), a top surface circular crack with a 6-ft (2-m) radius was observed. At this time the slab deflection was 0.59 in. (15 mm).

An 190 kip (845 kN) load was applied to the slab on the 33-in. (838-mm) subbase. For this loading no crack appeared until all prestress had been released. Deflection at cracking varied from 0.24 to 0.30 in. (6.1 to 7.6 mm).

Load tests were made also on the runway constructed at Orly[105] with precast square elements. A test load of 224 kips (996 kN) on a 32-in. (813-mm) diameter plate caused a deflection of 0.41 in. (10.4 mm) at the interior, and a deflection of 0.45 in. (11.4 mm) when the loading plate was located tangent to a 45 degree sliding joint.

Other load tests were made at Orly on a 1310-ft (400-m) long, 82-ft (25-m) wide, 7.1-in. (180-mm) thick prestressed pavement built in 1953. Prestress in the slab concrete was 250 psi (1.7 MPa). Deflections for interior load increments of 101, 134, and 224 kips (449, 596, and 996 kN) were 0.055, 0.075, and 0.13 in. (1.4, 1.9, and 3.3 mm), respectively. Permanent deflections after 1,000 applications of a 101 kip (449 kN) load and 2,000 additional applications of a 134 kip (596 kN) load were 0.024 and 0.042 in. (0.61

and 1.07 mm), respectively. An additional 700 applications of a 224 kip (996 kN) load increased the permanent deflection to 0.053 in. (1.35 mm).

Concrete compressive strains were 220 millionths after 1,000 applications of a 101 kip (449 kN) load, 340 millionths after an additional 2,000 applications of a 134 kip (596 kN) load, and 800 millionths after 700 applications of a 224 kip (996 kN) load. Top surface cracking was not reported.

Schiphol Airport, Holland: Load tests were made on 5.5-in. (140-mm) thick, 136-ft (41-m) long, by 136-ft (41-m) wide slabs built in 1951 at Schiphol Airport, Holland.(223) These slabs were prestressed to 500 psi (3.4 MPa). They rested on an 18- to 24-in. (450- to 600-mm) thick sand subbase. Deflections for 134 kip (596 kN) loads at the slab interior were 0.45 in. (11.4 mm). No top surface cracking was observed. Top surface cracking occurred for 45 kip (200 kN) corner and 89 kip (396 kN) edge loads. Deflections for these loads were 0.32 in. (8 mm) at the corner and 0.36 in. (9 mm) at the edge.

Patuxent Naval Air Station, Maryland: Load tests were made at the Patuxent NAS(96) on an instrumented 7-in. (178-mm) thick, 12-ft (4-m) wide and 500-ft (152-m) long prestressed runway. The longitudinal prestress was 690 psi (4.8 MPa), applied by 2 layers of 0.6-in. (15.2-mm) diameter 7-wire strands. The strands were located 13/16-in. (22-mm) above and below the mid-depth of the slab. Deflections for the prestressed slab were about 40 percent of those for the same slab without prestress. The load-deflection data for load tests on the interior of prestressed slabs are shown in Fig. 38.

In contrast, tests made at the Portland Cement Association(240) did not show any significant differences in slab deflections with reduced prestress. These tests were made on 5-in. (127-mm) slabs with three levels of prestress. Load-edge deflection data are shown in Fig. 39. Edge

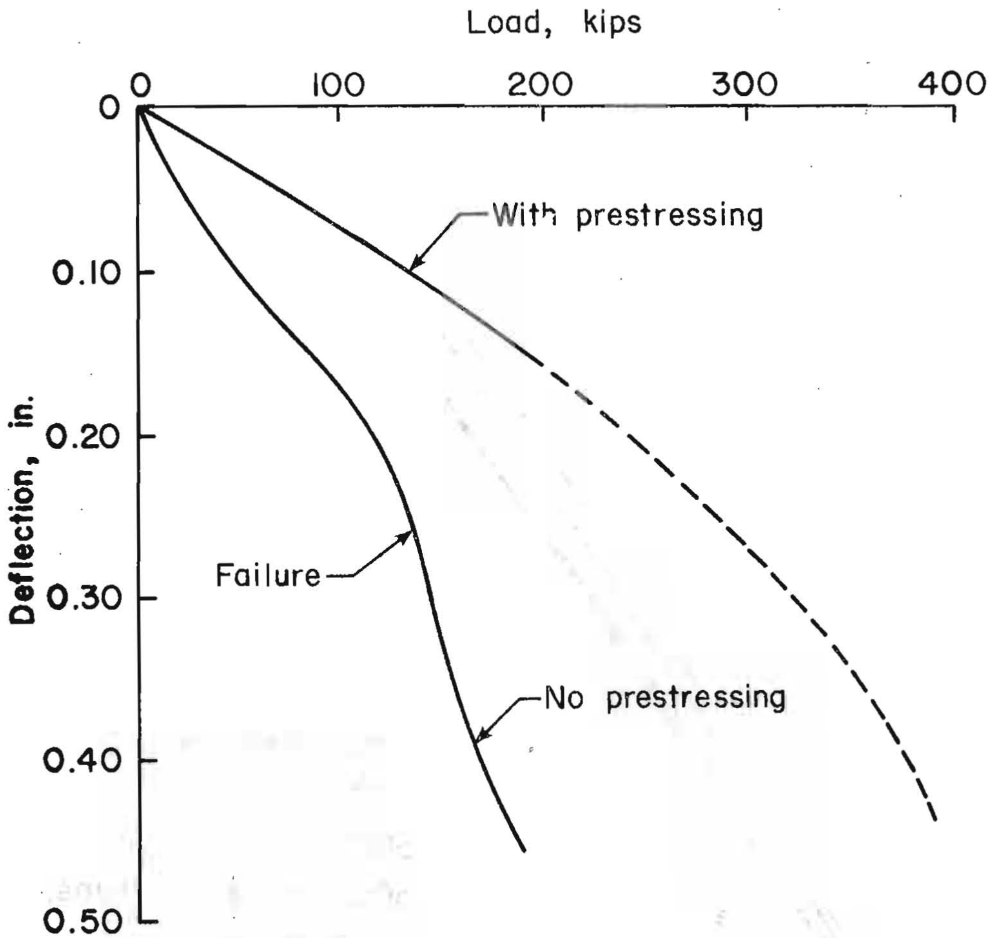


FIG. 38 - DEFLECTION VERSUS LOAD FOR SLABS WITH AND WITHOUT PRESTRESSING

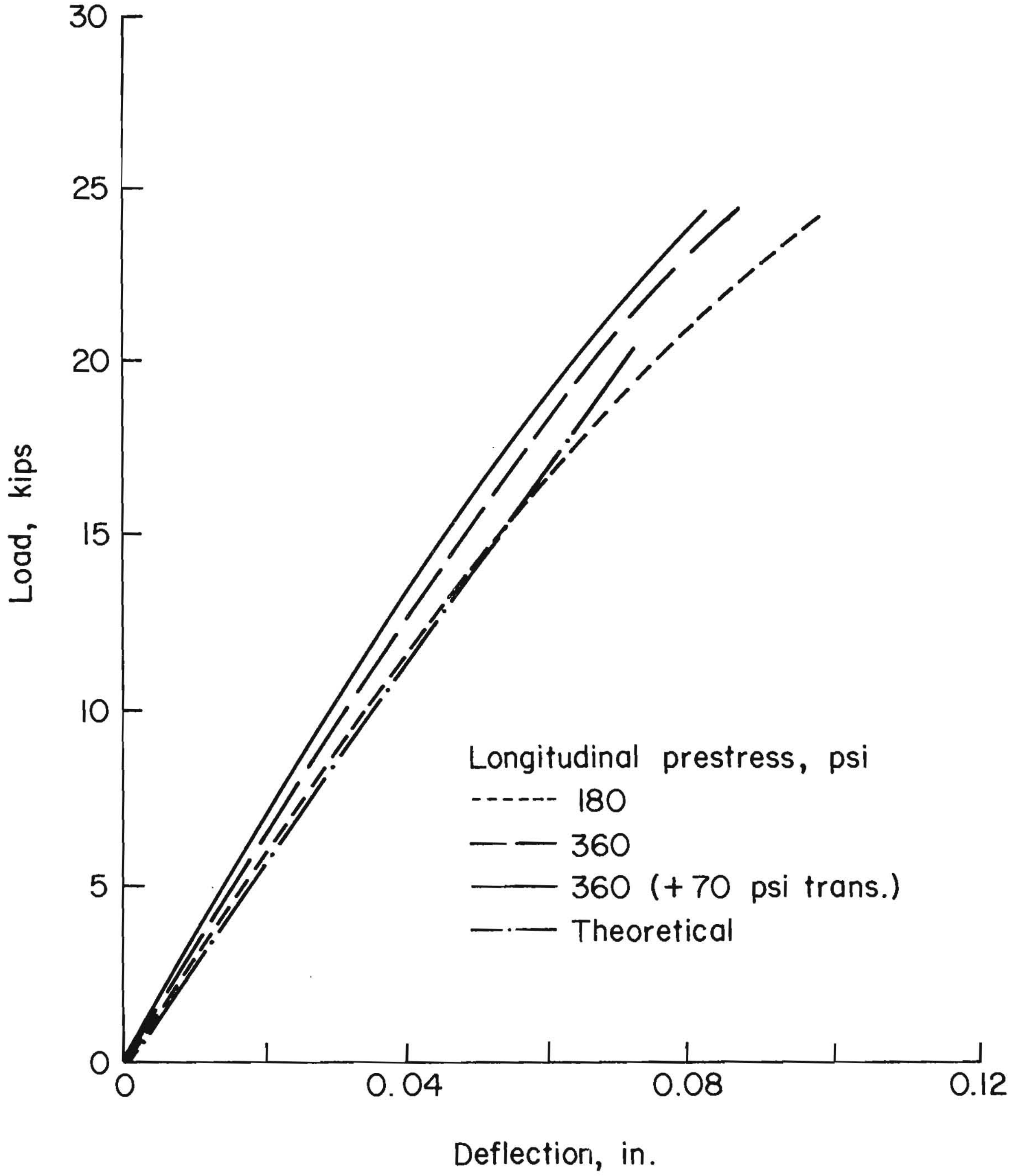


FIG. 39 - EDGE DEFLECTIONS FOR SLABS WITH DIFFERENT PRESTRESS LEVELS

deflections for the three slabs with different levels of prestress were in agreement with those predicted by elastic theory. Similar agreement was noted for the interior load tests.

Benefits of prestressed pavements are demonstrated by five tests at Patuxent made with the plate located tangent to one side of shrinkage cracks.[102] Good load transfer was observed for all tests with longitudinal prestress. However, significant differences in deflection between the loaded and unloaded side of the crack were observed when prestress was removed.

Strain measurements at Patuxent were reported for loads up to 100 kips (445 kN) applied through a 20-in. (508-mm) diameter plate. The data plot indicated linear strain distribution with equal magnitudes of compression and tensile strains. This indicated the absence of bottom surface cracking. Data from tests at Portland Cement Association[240] are shown in Fig. 40. These data indicated that the tensile strains at the slab bottom increased more than the top surface strains after bottom cracking.

Airfield Moving Load Tests

Static tests developed data on the load required to produce top surface cracking. They also showed the need for data on the effect of moving loads. These data would permit the fatigue aspects of the pavement and the foundation to be considered. Moving load test data are reported in this section.

Moving load tests made on reduced scale prestressed pavements at the Portland Cement Association[259] showed that strains and deflections increase with number of load repetitions. Stress and deflection data from the PCA tests are shown in Fig. 41. Total deflection increased with load application due to increase in both the permanent and elastic deflection component. The moving load tests were, for most

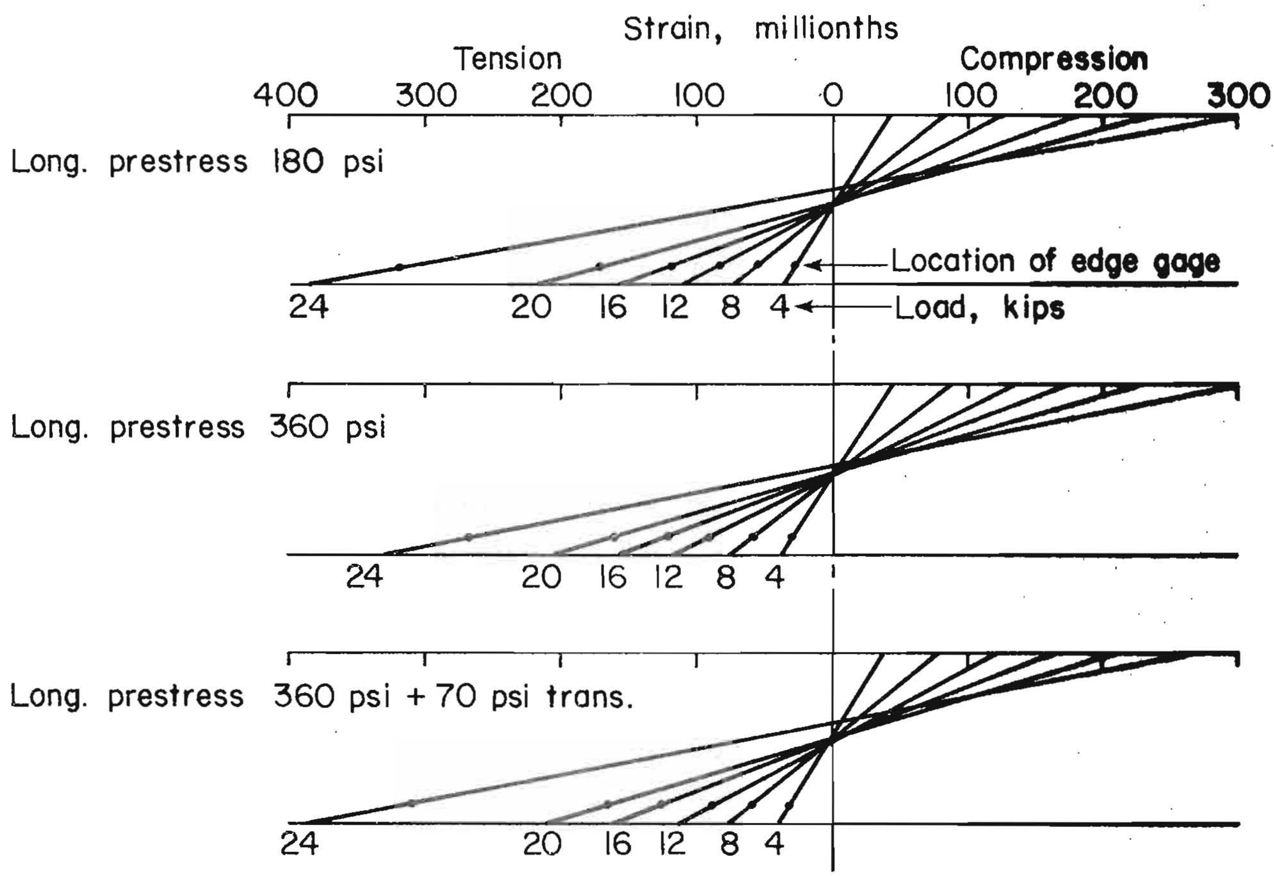


FIG. 40 - EDGE STRAIN DIAGRAMS FOR SLABS WITH DIFFERENT PRESTRESS LEVELS

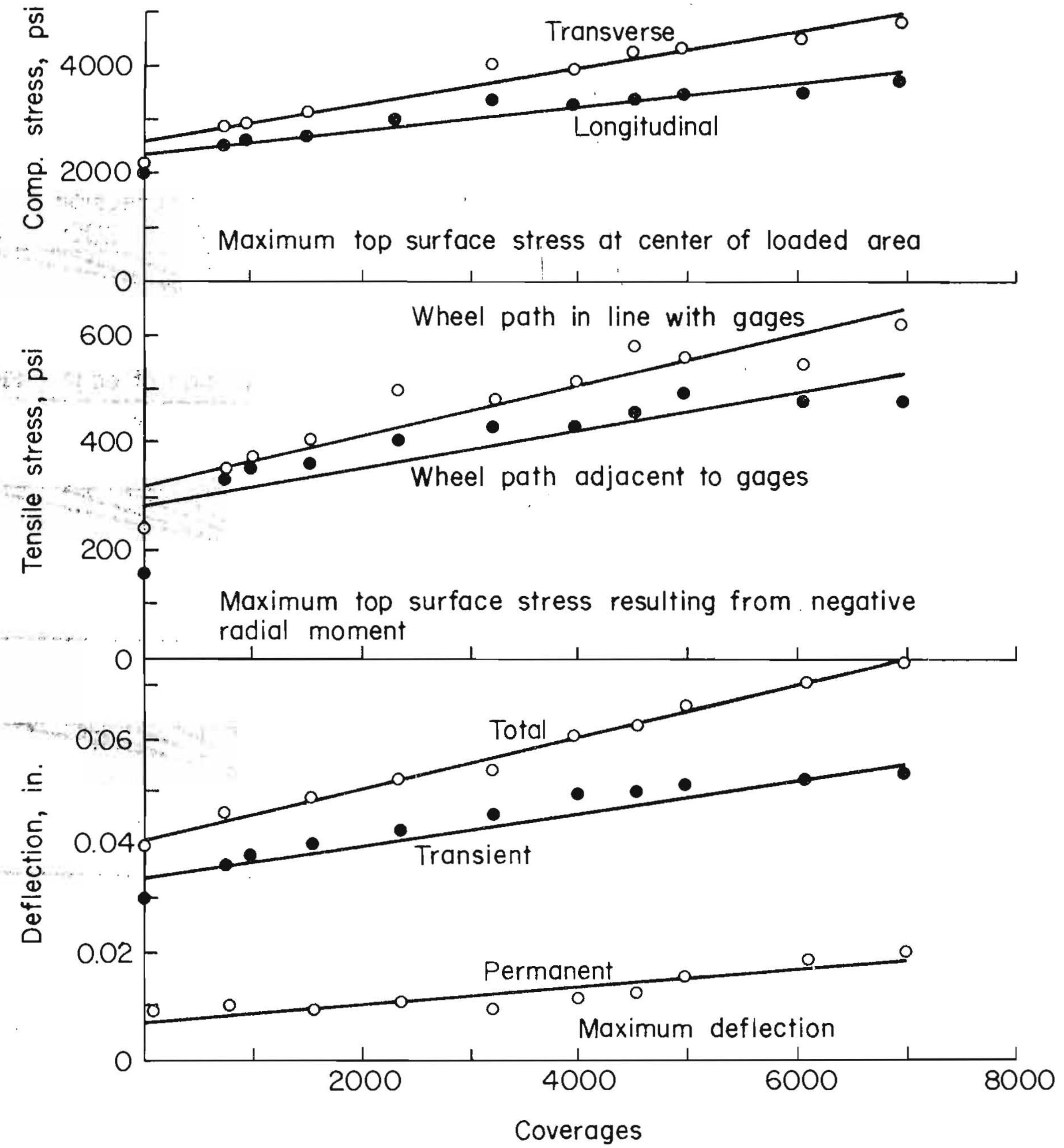


FIG. 41 - EFFECT OF LOAD COVERAGES ON STRESSES AND DEFLECTIONS

test slabs, continued until fatigue failure was observed. Initial distress leading to fatigue failure commenced with the appearance of cracks in the bottom of the slabs. The cracks extended upward to the top surface. At the same time deflections increased at a more rapid rate.

From tests of 1/8-scale prestressed slabs, McIntyre(224) reported failures similar to the type observed in the Portland Cement Association tests. The cracks became more closely spaced with increased numbers of moving load coverages. Failures in the model slabs were reported to be very similar to those of full-scale pavements.

Similar observations were reported by Carlton and Behrman.(235) The sequence of failure was:

1. formation of radial tensile cracks in the bottom surface of the model slab starting below the area of load applications;
2. appearance of circumferential top surface cracks away from the area of load application;
3. punchout destruction of the slab near the load area.

Repeated moving load tests were made at Sharonville, Ohio, on a 4-in. (102-mm) thick prestressed overlay and two full scale 9- and 9.4-in. (229- and 239-mm) thick prestressed pavements. Top surface failure of the pretensioned overlay slab occurred after 9,100, 6,200, and 2,068 coverages of a 60, 75, and 100, kip (267, 334, and 445 kN) loads applied through a gear assembly.(224) The tire sizes and spacings on the gear assembly were similar to those used on the landing gear of a B-52 aircraft.

The 9.0-in. (229-mm) thick Sharonville pavement was prestressed to 200 psi (1.4 MPa) in the longitudinal direction. The 9.4-in. (239-mm) thick pavement was to at 400 psi (2.8 MPa).(180) Transverse prestress was varied in both pavements from 200 to 400 psi (1.4 to 2.8 MPa). The load levels used with the 4-wheel twin-tandem gear assembly vehicle were 100, 175, 225, and 265 kips (445, 778, 1001, and 1179 kN). Dynamic deflections for loads moving at 2 mph (3.2 km/h), as shown in Fig. 42, were about 70 percent of the static load deflections. The predicted deflections in the elastic range were computed using the Westergaard theory. Deflections beyond the elastic range were predicted on basis of Ohio River Division Corps of Engineers model tests.[235] Tensile cracks occurred in the bottom of the 400 psi (2.8 MPa) Sharonville slab at the 175 kip (778 kN) load.

Moving load tests to failure were made at Sharonville with the 265 kip (1179 kN) load on the 400 psi (2.8 MPa) prestressed slab. First top surface circular tensile crack occurred after 440 coverages. The area where it occurred had no transverse prestress and was supported on 4 in. (102 mm) of silty sand subbase. The number of coverages to failure, load levels, deflections, subbase conditions, and flexural strengths are listed in Table 4 for the 400 and 200 psi (2.8 and 1.4 MPa) prestressed slabs. The shape and condition of the damaged areas were similar to those observed for reduced scale prestressed slabs.

At Sharonville, deflections increased with the number of coverages, as noted for the Gatwick,[233] Portland Cement Association,[259] and Ohio River Division tests.[235] The deflection versus coverage data are shown in Fig. 43. It may be seen that the total deflection and the permanent deflection component increased rapidly as failure was approached. However, the elastic deflection component did not show a similar trend. Rather, it increased at a constant rate.

Data from the Sharonville tests and the model tests by Carlton and Behrman were used for the design of the pavement at Biggs Air Force Base. The 9-in. (229-mm) thick T-3 taxiway at Biggs AFB was designed for 10,000 passes with a B-52 aircraft.[213] Load tests were made with a partially

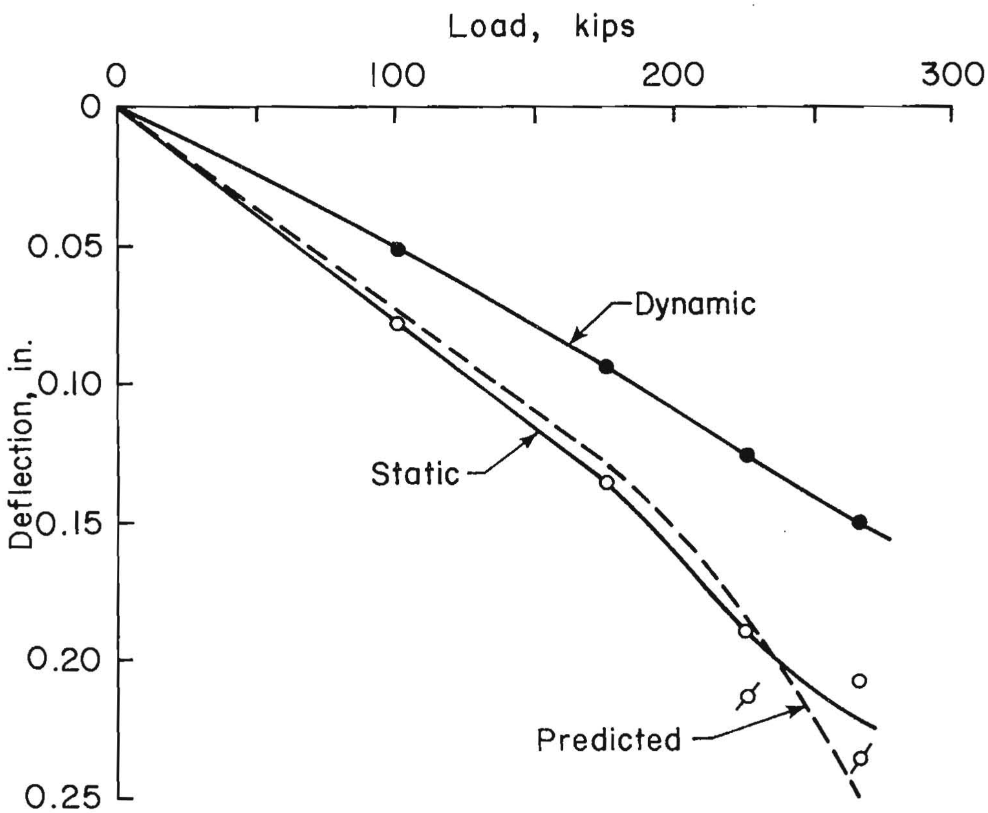


FIG. 42 - EFFECT OF STATIC AND DYNAMIC LOADS ON DEFLECTION

TABLE 4 - SHARONVILLE TRAFFIC TESTS

Subbase, in.	Load, kips	Flexural plus Prestress, psi		Coverages to Failure	Deflections, in.		
		Long.	Trans.		Dynamic	Permanent	Total
		Track P1 (prestress 400 psi)					
4	265	1150	750*	440	0.238	0.168	0.406
4	265	1240	1240	1920	0.200	0.223	0.423
18	265	1280	1080	1250	0.238	0.394	0.632
18	265	1080	1080	630	0.129	0.089	0.218
		Track P2 (prestress 200 psi)					
4	265	900	700*	220	0.368	0.435	0.803
4	265	1080	1280	1330	0.200	0.105	0.305
4	200	1010	1210	8400≠	0.144	0.267	0.411
18	200	1000	1000	4240≠	0.124	0.168	0.292
18	265	990	990	1420	0.224	0.471	0.695
18	265	1020	1220	2010	0.270	0.505	0.775
18	265	1070	1270	2680	-	-	-

* Deformed bars in lieu of transverse prestress. Flexural strength of the concrete was 700 psi.

≠ No failure.

1 kip = 4.448 kN
1 in. = 25.4 mm
1 psi = 6.895 kPa

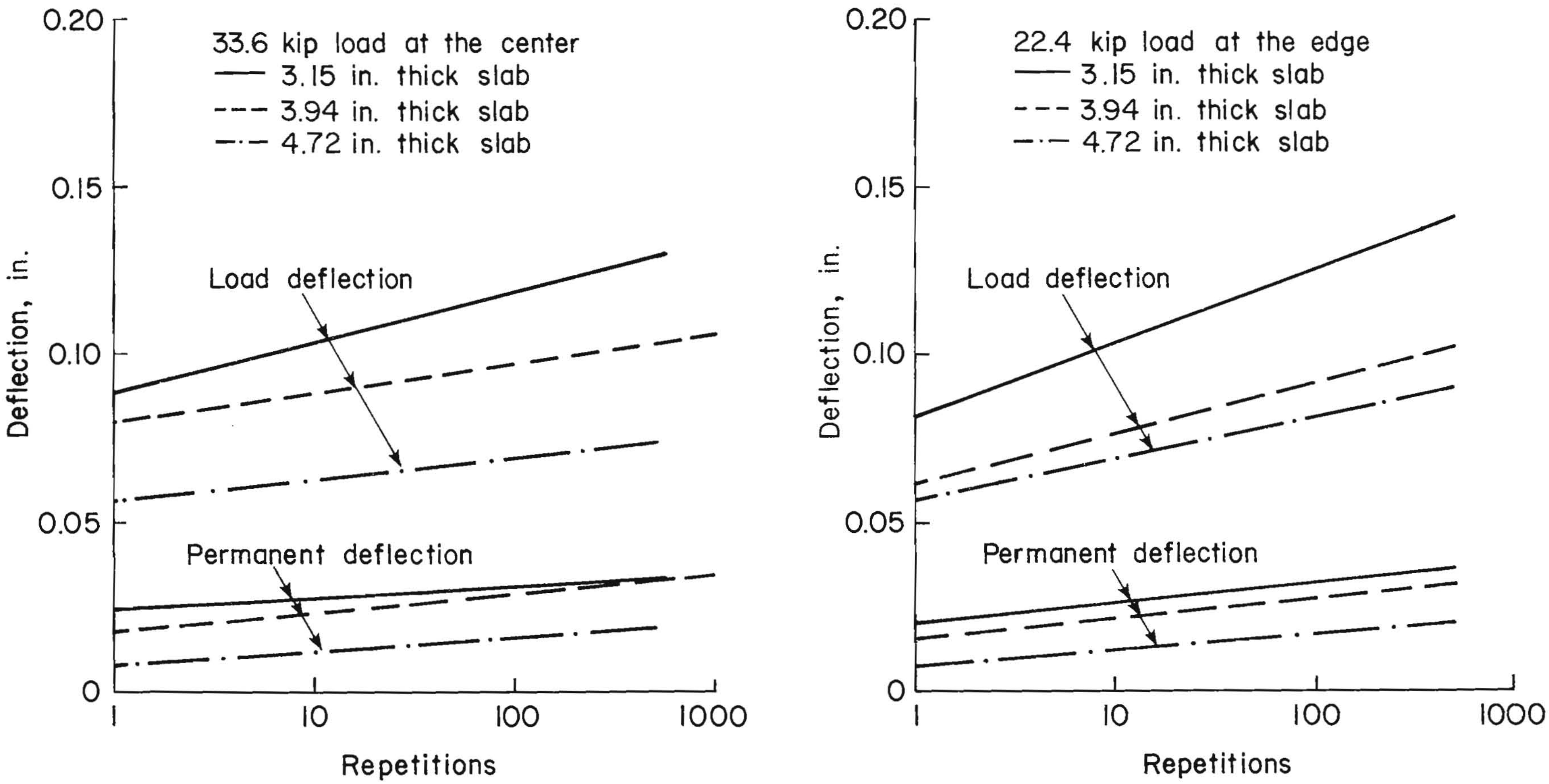


FIG. 44 - EFFECT OF LOAD REPETITIONS ON SLAB DEFLECTIONS

at Pittsburgh. Static loads were applied through one dual tire wheel. For a 16-kip (71-kN) wheel load the deflections were 0.032 in. (0.81 mm) at the interior and 0.04 in. (1.02 mm) at the edge.[146] Top surface fiber stresses were about 500 psi (3.4 MPa) for the interior and 750 psi (5.2 MPa) for the edge load position. Edge deflection for a 23 kip (102 kN) wheel load at creep speed was 0.030 in. (0.76 mm) during the day and 0.045 in. (1.14 mm) at night.

Loads were also placed on the closed neoprene laminated compression joint. Joint delfection was 0.04 in. for an interior load position and 0.05 in. for an edge load.

Moving load tests on the Jones and Laughlin pavement started on September 19, 1962. The loading rig consisted of two 14 ton (125 kN) axles at 20-ft (6-m) spacing. Dual tire truck wheels were spaced 6-ft (2-m) apart. About 580,000 axle load repetitions were applied from September 19, to December 1, 1962. Initial "run in" moving loads were applied in August 1962.

Longitudinal cracks occurred in the pavement surface during the August initial "run in" period prior to full-scale testing[245]. The cracks were located above the tendon locations. These longitudinal cracks were near one end of the 400-ft (122-m) slab. They progressed to within about 5 ft (1.5 m) of the joint face. This was almost to the edge of the underlying sleeper slab.

Longitudinal stresses measured during moving load testing for interior slab positions did not exceed +80 or -200 psi (+552 to -1379 kPa).[245] Stress data are shown in Fig. 45. Longitudinal stresses at the edge did not exceed +200 and -350 psi (+1.4 and -2.4 MPa). The highest longitudinal edge stresses were reported for the portion of the pavement with the change in slab support conditon, i.e., at transition from subbase to the end of the sleeper slab. The edge stress was approximately -425 psi (-2.9 MPa) at the start of loading. It decreased and did not exceed about -380 psi (-2.6 MPa) after 20,000 load coverages.

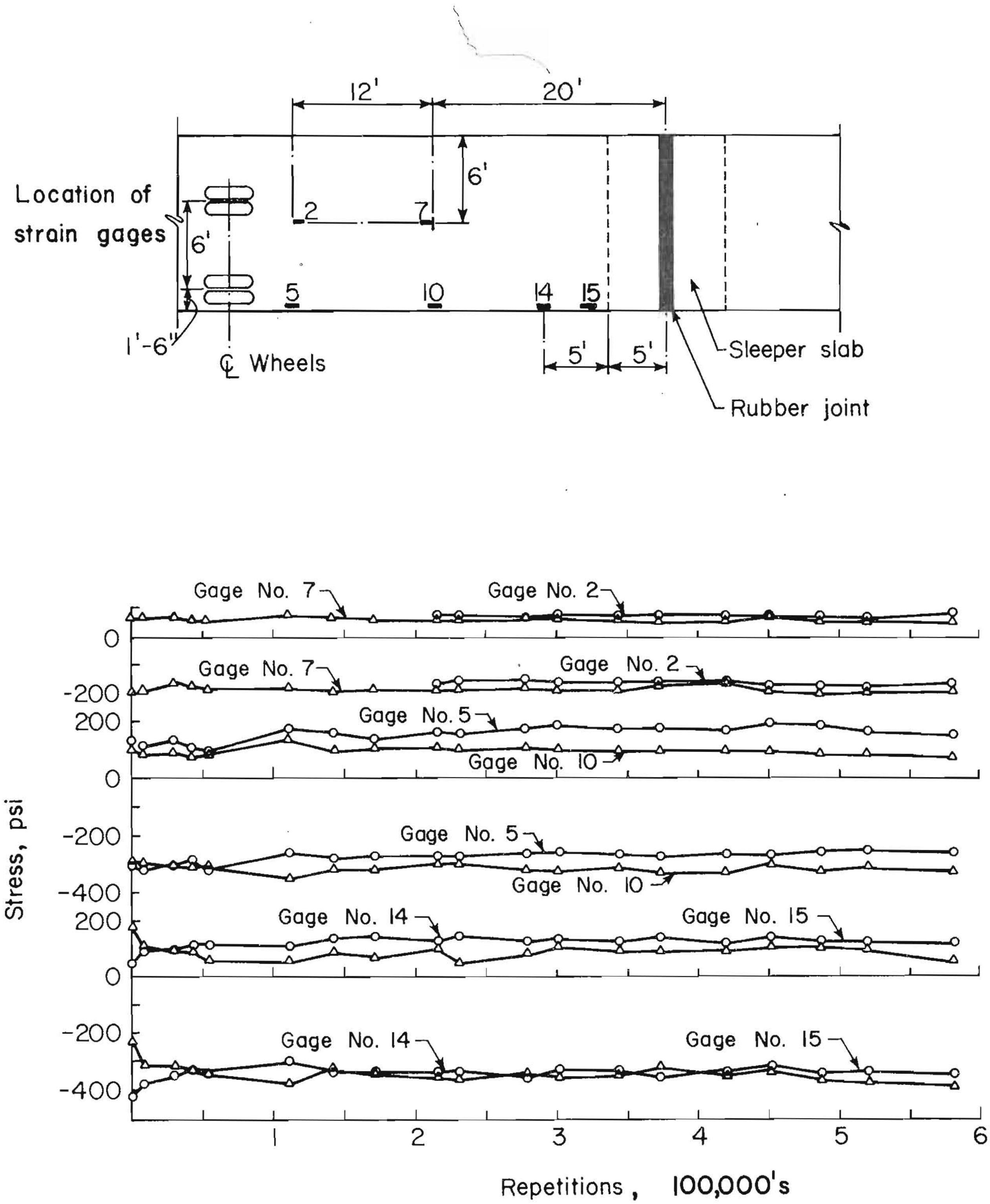


FIG. 45 - LONGITUDINAL STRESS UNDER MOVING LOAD

Edge deflections measured at the Jones and Laughlin project for moving loads did not exceed 0.03 in. (0.76 mm). However, considerable loss of slab support was noted at the sleeper slab. This was due to a separation between the 100-ft (30.5-m) prestressed pavement to the east side of the joint and the sleeper slab. This sleeper slab supported the east end of the 400-ft (122-m) prestressed slab and the west end of the 100-ft (30.5-m) prestressed slab. The sleeper slab also supported the laminated neoprene joint. Edge deflections of the 100-ft (30.5-m) pavement slab were 0.074 in. (1.88 mm) at the east end of the sleeper slab whereas the sleeper slab deflected no more than 0.04 in. (1.016 mm).

Severe edge pumping occurred at the ends and edges of the sleeper slab during and after heavy rains on September 27, November 9, 10, 17, and 21, 1962[245]. Total permanent change in pavement profile at stations along the pavement slab are shown in Fig. 46. The west edge of the sleeper slab is located at Station 7. Permanent settlement of the slab to the east of the joint was considerably greater than that of the west slab.

The structural integrity of the Jones and Laughlin prestressed pavement was preserved even though there was considerable loss of subbase support. After pumping had started, about 300,000 passes of the axle load were accomplished without cracking that would indicate structural failure. The longitudinal cracks over conduits were not considered structural failure. However, they demonstrated the need for adequate concrete cover over conduits and tendons.

Road at Naz, Switzerland: Load tests were made at seven locations of the prestressed 4.7-in. (119-mm) thick pavement constructed in 1955 near Naz, Switzerland.[123] Tensile stresses for a 15 kip (67 kN) load applied through an approximately 30-in. (762-mm) diameter plate varied from 140 to 327 psi (965 to 2255 kPa). Variations of stresses and deflections were reported as being due to variations in

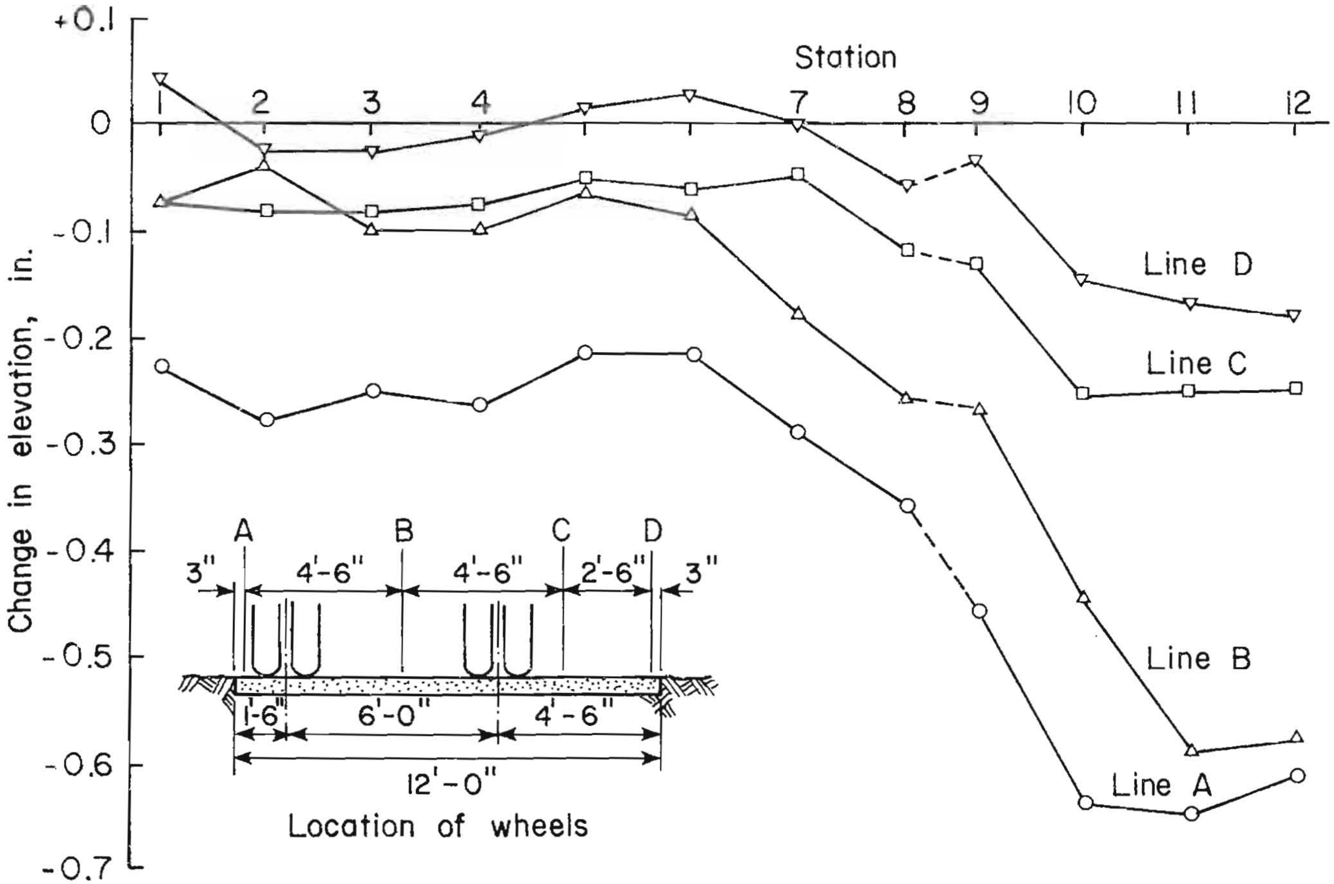


FIG. 46 - CHANGE IN PAVEMENT ELEVATION AFTER 580,000 LOAD APPLICATIONS

subgrade strength. The measured modulus of subgrade reaction for the pavement with the lower bending stress was about 2.5 times that for the pavement with the higher, 326 psi (3.3 MPa) bending stress. Maximum deflections for the two pavements were 0.15 and 0.06 in. (3.81 and 1.52 mm), respectively for weak and strong subgrade bearing values.

Stresses and deflections for the pavements at Naz were significantly greater when a 6.3-in. (160-mm) diameter plate was used to apply the 15 kip (67 kN) load. For example, the longitudinal tensile stresses for the slab on the weaker subgrade increased from 327 psi to 1,065 psi (2.3 to 7.3 MPa) and the deflections increased from 0.15 to 0.31 in. (3.81 to 7.87 mm). Similarly, the stresses and deflections for the prestressed slabs on the weaker subgrade increased by about 150 percent.

The data from Naz indicate the benefits derived from strong subbases in reducing prestressed pavement stresses and deflections. The 1.5 times increase in subgrade bearing modulus reduced stresses by approximately 70 percent and deflections by nearly 50 percent.

Moving load tests on reduced scale prestressed pavement slabs made at the Portland Cement Association[259] also demonstrated the advantages of high strength subbases under prestressed pavements. For example, the loads causing bottom surface cracking of the test slab on a cement-stabilized subbase were approximately one and one-half greater than those predicted by theory. Therefore, the behavior for the slab on a strong subbase was very conservative when compared to Westergaard's theoretical analysis.

Dulles Airport Access Road, Virginia: Load tests were made with a 20-kip (89-kN) axle truck on the 6-in. (152-mm) thick Dulles Airport access road prestressed pavements.[281] This pavement had been constructed on a soil-cement subbase. Benkelman beam deflections for loads away from the edge averaged about 0.006 in. (0.15 mm). Edge deflections at creep speed with the center of the nearest wheel at a distance of 20 in. (508 mm) from the edge were 0.016 in. (0.41 mm).

Maximum deflections at the joint for a standing 20-kip (89 kN) axle positioned 20 and 40 in. (508 and 1016 mm) away from edge were 0.016 and 0.032 in. (0.41 and 0.81 mm), respectively. For a 34-kip (151-kN) axle, the deflections were 0.020 and 0.032 in. (0.51 and 0.81 mm), respectively. For an axle positioned at the joint, the deflections were 0.044 and 0.046 in. (1.12 and 1.17 mm), respectively for the 20-kip (89-kN) and 34-kip (151-kN) axles. For 20-kip (89-kN) axle loads, edge strains as shown in Fig. 47, did not exceed 60 and 30 psi (414 and 207 kPa), respectively. The low values for stresses and deflections measured at the access road demonstrate the benefits of strong subbase support for prestressed pavement.

Measurements of Prestress Distribution

Variations of prestress distribution for externally and internally stressed systems are discussed together with the prestress response to environmental factors.

Zwartberg-Meeuwen Road, Belgium

This road was constructed during November 1959. Prestressing was started in November 1959 and completed in May 1960.[223] The intended prestress of the slabs at 59^{o}F (15^{o}C) was 280 psi (1.9 MPa) for the 3.15-in. (80-mm) thick slab, 430 psi (3.0 MPa) for the 3.95-in. (100-mm) slab, and 570 psi (3.9 MPa) for the 4.75-in. (120-mm) slab. The variation in prestress due to temperature changes was measured to be 30 to 40 psi per ^{o}F (115 to 153 kPa per ^{o}C) during the spring of 1960.

Poststressing of 260 to 342 psi (1.8 to 2.3 MPa) was applied six days after placing the concrete. Strains

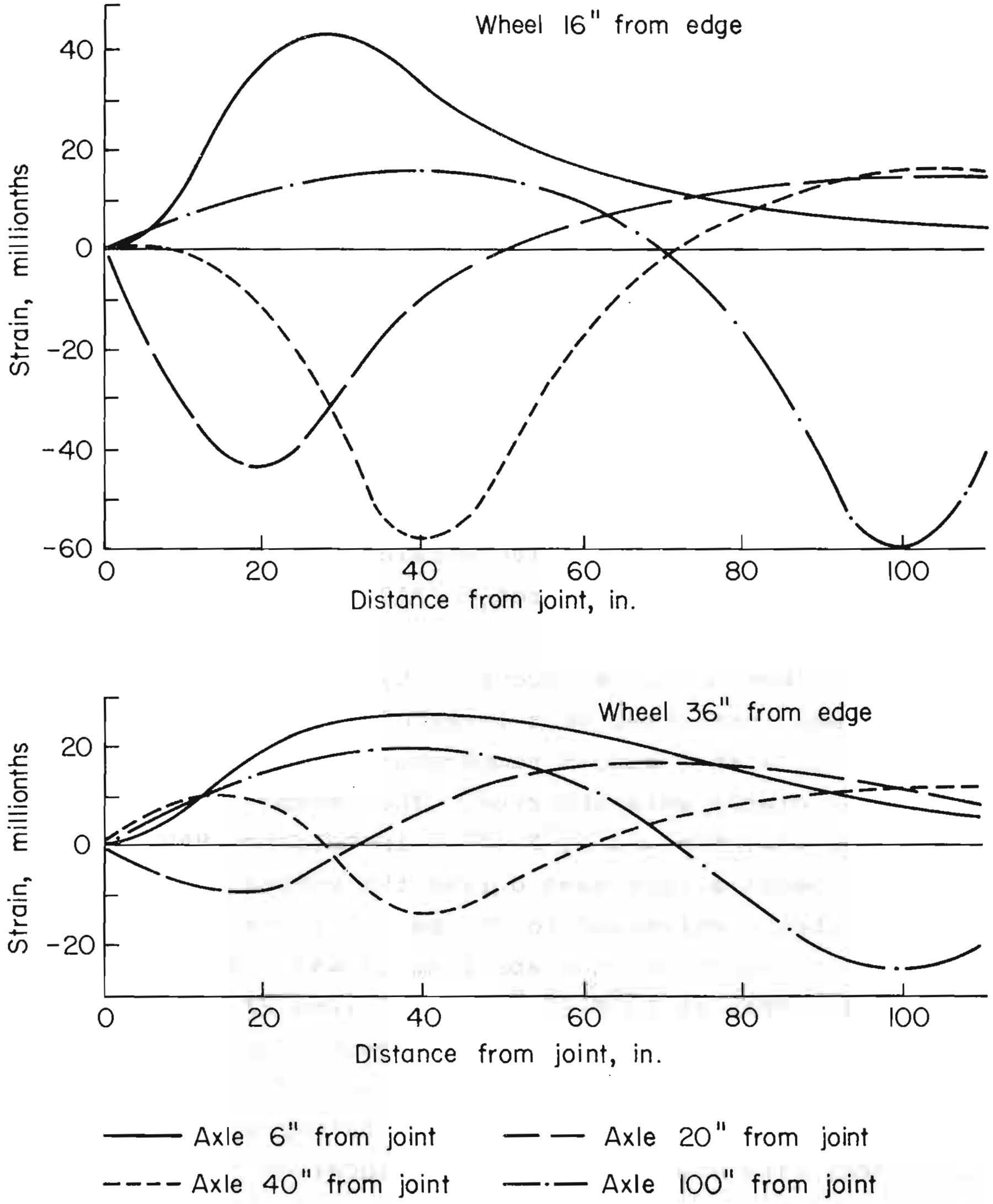


FIG. 47 - PAVEMENT EDGE STRAINS FOR 20-KIP AXLE LOAD

measured with vibrating wire gages near the slab ends indicated that the concrete modulus of elasticity at early pavement age was 3.1 million psi (21.4 GPa) in one slab and 2.5 million psi (17.2 GPa) in the other slab.(244) After eight days under stress, the sustained modulus of elasticity had decreased to 2.5 and 2.1 million psi (17.2 and 14.5 GPa), respectively. After 5 months, the sustained moduli were 2.0 and 1.5 million psi (13.8 and 10.3 GPa).

Strain measurements indicated that the concrete modulus of elasticity was 5 million psi (34.5 GPa) at 8 days age,(244) and 6.1 million psi (42.1 GPa) at 14 days age. These data were determined during restressing after the stress had been released for one hour. Restressing was done within 30 minutes.

Small tension strains were measured at a distance of 428 ft (131 m) from the jacking joint. After 14 days, strains at this distance were 20 millionths in compression. During April 1960, compressive strains at this location were 120 millionths. This compared to 210 millionths at the jacking joint.

Numerous cracks occurred by October 1960.(223) The road was restressed to a level of 480 psi (3.3 MPa) at 48°F (9°C). With a sudden temperature drop to 27°F (15°C) additional cracks were observed. The pavement was restressed to 540 psi (3.7 MPa) at 45°F (7°C) in October 1960.

Repairs were made during the spring of 1961. The road was then prestressed to 850 psi (5.9 MPa) at 41°F (5°C). The prestress in slabs 1 to 22 was 850 to 1,050 psi (5.9 to 7.2 MPa) at 50°F (10°C). In slabs 23 to 26 it was 640 psi (4.4 MPa) at 41°F (5°C). Blowups occurred in the 3.15-in. (80-mm) thick slabs 4 and 9 at 104°F (40°C). Prestress measured one day after the blowup was 2,800 psi (19.3 MPa) five slabs away from the location of failure. In the four slabs between the blowups, the prestress was about 1,200 psi (8.3 MPa) at 73°F (23°C). Old cracks reappeared when temperature dropped to 57°F (14°C). Slabs 1 to 13 were restressed to 700 psi (4.8 MPa) at 64°F (18°C).

The slabs were at zero prestress at 59°F (15°C) during December 1961. This was reported to have been due to creep. The slabs were repaired in December and restressed to 1,000 psi (6.9 MPa) at 41°F (5°C).[244] Strain gages were used for stress determination. A concrete modulus of elasticity of 5.7 million psi (39.3 GPa) was assumed. Joint widening was about 2.1 in. (53.3 mm). Thus, each 443-ft (135-m) long slab contracted 1.06 in. (27 mm) due to restressing. Restressing was done with screw jacks. The restressing was reported to be impossible when temperatures remained below 32°F (0°C). The screw jacks were intended to permit adjustment of prestress to avoid cracking at low temperatures and excessive pressures at high temperatures.

Prestress adjustments with screw jacks were initiated during April 1962. Joint openings were reduced to 1.6 in. (40.6 mm) from the 2.1 in. (53.3 mm) opening of December 1961. This corresponded to a decrease of 850 psi (5.9 MPa) in prestress.

During April 18, 1962 the temperature rose from 32°F (0°C) to 72°F (22°C). Failure occurred in Slab 24. This slab was still adjusted for winter conditions. A concrete stress of 2,700 psi (18.6 MPa) was measured 5 slabs away. The stress-temperature adjustment was 45 to 55 psi per °F (558 to 683 kPa per °C) as measured with Glotzl stress gages. Stress measurements with Glotzl gages were 20 percent higher than actual. This was determined from a correlation with strains measured in steel tubing of a joint.

During September 1962, prestress was adjusted to the winter condition. Prestress was increased to 1,280 psi (8.8 MPa) by widening the joint opening to 2.35 in. (60 mm). Additional cracks had formed since the April 18, 1962 pavement blowup. During the summer, prestress was lost at

49°F (9.4°C). Cracks also formed after a sharp temperature drop on September 15, 1962.

Joint faulting of 0.8 in. (20 mm) was observed between two slabs on January 11, 1963. Temperature had dropped to 14°F (-10°C). At several other active jacking joints, the slabs had lifted from grade for a distance of 20 ft (6 m) from the joint.

Failure in Slab 4 occurred 16 ft (5 m) away from the joint on March 8, 1963 at a prevailing temperature of 61°F (16°C). Prestress drop extended from Slab 7 to Slab 16. Prestress was lowered in the slabs to 780 psi (5.4 MPa) at 50°F (10°C). On May 8, failure occurred in Slab 24 at a distance of 16 ft (5 m) from the joint. The temperature was 95°F (35°C). On June 13, 1962 the prestress was zero in nearly all joints when the temperature was 52°F (11°C).

Melsbroek Airport Runway, Belgium

In June 1961, when blowups occurred at the Zwartberg-Meeuwen prestressed roadway, a prestress of 1,650 psi (11.4 MPa) was measured in the 7.1-in. (180-mm) thick runway at Melsbroek. Air and concrete temperature were 95°F and 90°F (35°C and 32°C), respectively.[223] Stress-temperature adjustment was 28 psi per °F (347 kPa per °C). During December of 1961, cracks appeared at 27°F (-2.8°C). Restressing was done to close the cracks.

Two compression failures were reported at active joints. One occurred in April, 1962 on a joint damaged in poststressing. The second occurred in June, 1963 close to active Joint 9. This compression failure extended across the entire runway width. The 1962 and 1963 compression failures occurred on days with no wind and intense sunlight. Temperatures rose from 59°F (15°C) to 86°F (30°C) and pavement surface temperature exceeded 120°F (40°C).[243] It was believed that steep temperature gradients caused additional prestress at the pavement surface. Problems were not observed in the slab interiors.

Industrial Road at Liege, Belgium

The 4.75-in. (120-mm) thick roadway at Liege was constructed in late July, 1964. Poststress application with screw jacks commenced at an age of 6 to 10 hr. Stress was increased gradually to 1,800 psi (12.4 MPa) at 10 days. After 50 days, the prestress was 1,560 psi (10.8 MPa) at 68°F (20°C). During the initial months of prestressing the jacking gaps widened by 6 to 12 in. (150 to 300 mm).

Prestress at 88 to 100°F (31 to 38°C) was about 1,800 psi (12.4 MPa) and less than 100 psi (690 kPa) at 32°F (0°C). Stress-temperature measurements for 24 hours in October indicated a stress-temperature variation of 22.2 psi per °F (276 kPa per °C) over a range from 49°F to 67°F (9°C to 19°C). An overall variation, from zero psi at 32°F (zero kPa at 0°C) to 1,800 psi at 95°F (12.4 MPa at 35°C), was 29 psi per °F (360 kPa per °C). At low temperatures and high moisture, the stress-temperature variation was 20 psi per °F (248 kPa per °C). Prestress variations over a four year period are shown in Fig. 48. Measured stresses varied between zero and 1,875 psi (12.9 MPa). Transverse cracks were not reported during the first 4 years of pavement age.

Autobahn Near Salzburg, Austria

The 7.9-in. (200-mm) thick externally prestressed pavement at Salzburg experienced a failure within one year after construction. At the time of the blowup, Spring 1960, the prestress in a normal section of the pavement was about 1,280 psi (8.8 MPa).

Pavement stresses during June 1959, a few months after construction, reached a maximum of about 1,135 psi (7.8 MPa). The corresponding air temperature was about 80°F

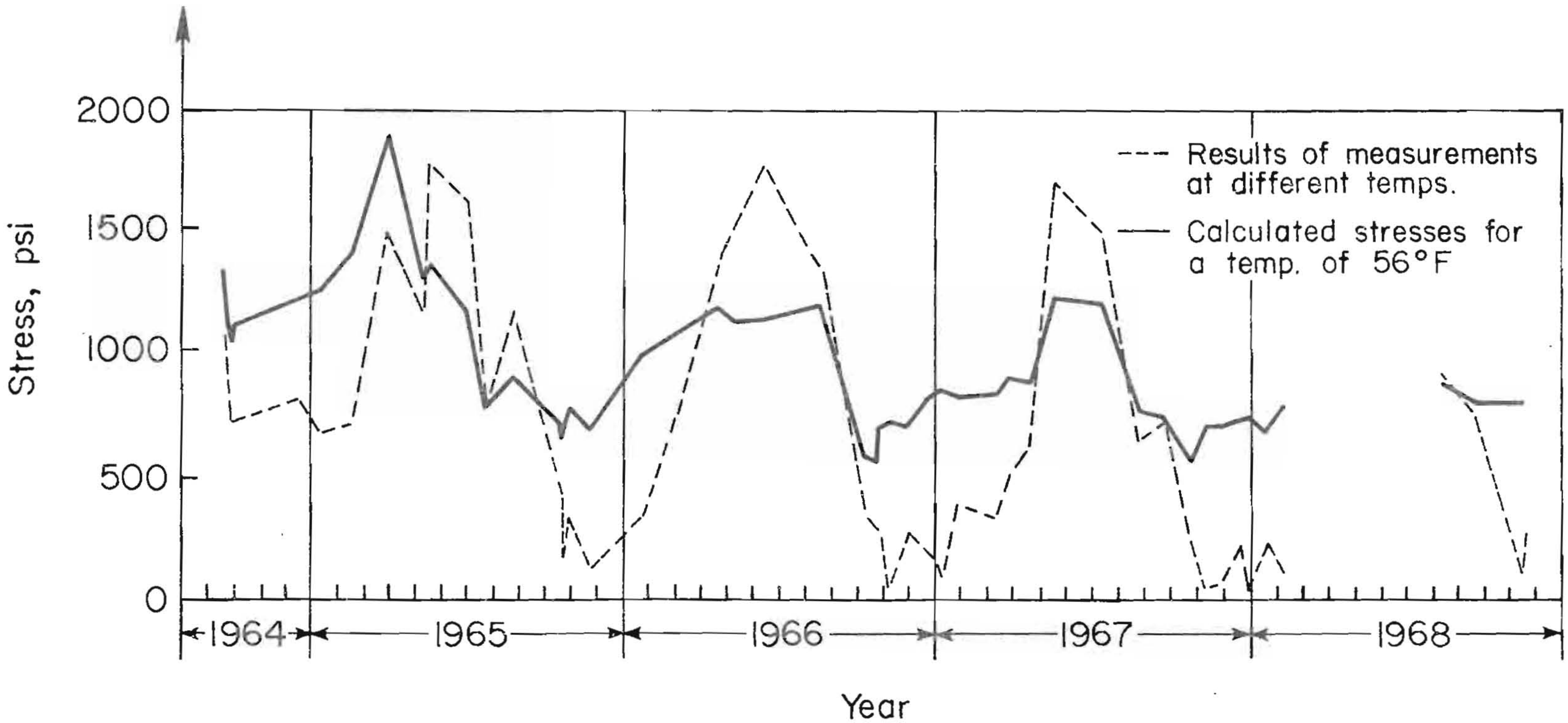


FIG. 48 - VARIATION OF PRESTRESS WITH TIME FOR ROAD AT LIEGE, BELGIUM

(26.7°C). Higher air temperatures were recorded in the following months, but prestress values were significantly lower.

Measurements of Length Changes, Prestress Loss, and Strand Friction

This discussion deals with observations and measurements on internally prestressed pavement slabs. Concrete length changes generally include effects of creep, shrinkage, temperature, and subgrade restraints to slab movement.

Dietersheim-Bingen Road, Germany

The internally prestressed pavement at Dietersheim-Bingen was stressed in two stages. Partial prestress of approximately 310 psi (2.1 MPa) was applied to the slab when compressive strength had reached a value of 2,558 psi (17.6 MPa).[192] Strength from concrete cylinder compression tests indicated that the two day strength varied from 2,401 psi (16.5 MPa) to 3,629 psi(25 MPa).

Strand elongation and concrete length changes are reported in Table 5 for initial prestress and final prestress[192]. Final stress in the concrete after about 1 month was 723 psi (5.0 MPa). The total end reaction was 13,264 kips (59 kN). Strand stresses at partial prestress and final prestress were 65.2 ksi (450 MPa) and 159 ksi (1096 MPa), respectively.

Each of the slabs was 492-ft (150-m) long. Slab contraction with an initial prestress of 310 psi (2.1 MPa) averaged about 0.25 in. (6 mm). After about a month without prestress adjustment the average slab contraction had increased to 0.65 in. (16.5 mm).

The final prestress of 753 psi (5.2 MPa) was applied about one month after casting. Slab contraction due to final prestress, i.e., using the length of slab immediately

TABLE 5 - STRAND ELONGATION AND CONCRETE SLAB LENGTH CHANGES

Slab	Date	Partial Prestress						Shrinkage & Creep Until Final Prestressing, in.			Date	Final Prestress					
		Strand Elongation, in.			Concrete Length Change, in.							Strand Elongation, in.			Concrete Length Change, in.		
		North	South	Total	North	South	Total	North	South	Total		North	South	Total	North	South	Total
1	10/3/58	8.82	2.72	11.54	0.14	0.32	0.17	0.38	0.43	0.81	10/29/58	1.57	18.03	19.60	0.23	0.21	0.44
2	10/11/58	8.19	2.87	10.67	0.15	0.15	0.30	0.42	0.41	0.83	10/30/58	2.48	17.32	19.49	0.20	0.24	0.44
3	10/19/58	9.45	-	-	0.16	0.12	0.28	-	-	-	-	-	-	-	-	-	-
4	10/25/58	9.57	2.52	12.09	0.09	0.13	0.22	0.26	0.31	0.57	11/18/58	1.18	18.03	19.21	0.15	0.17	0.32
5	11/17/58	2.20	8.43	10.63	0.11	0.126	0.23	0.26	0.26	0.52	11/28/58	16.30	2.36	18.66	0.19	0.25	0.44
6	11/8/58	2.68	8.54	11.22	0.15	0.146	0.29	0.25	0.277	0.53	11/27/58	17.20	1.417	18.62	0.14	0.15	0.29

1 in. = 25.4 mm

prior to final prestress as the datum, averaged about 0.39 in. (10 mm).

Longitudinal strands were, during intial pre-stressing, stressed first from the north end of the slab. The stress on the strand was 52 ksi (359 MPa). The jacks were then moved to the south end of the slab and a load was applied to produce a strand stress of 65.2 ksi (450 MPa). Total average strand elongation was 11.23 in. (285.2 mm).

Final prestressing after about one month imposed a 159 ksi (1096 MPa) strand stress. First one end and then the other end of the strand was stressed. The elongation due to final stressing, i.e, using conditions immediately before stressing as the datum, averaged about 19 in. (483 mm).

Japanese Experimental Slab

Two 467-ft (142 m) long 30-in. (760 mm) wide and 6-in. (150 mm) deep beams were cast using conventional concrete for one and conventional concrete plus an expansive component for a second slab. The amount of expansive component used was 89 lb per cu.yd. (53 kg/m^3) of concrete.[146]

Preliminary prestress at 2 days after casting was 70 psi (483 kPa). A second level of 370 psi (2.6 MPa) prestress was applied prior to final prestress of 730 psi (5.0 MPa) at 9 days after casting.

Observed slab end movements to 30 days were 0.9 in. (23 mm) and 0.6 in. (15 mm) for the slabs with conventional and expansive concrete, respectively. Slab contraction due to 730 psi (5.0 MPa) prestress was 0.25 in. (6 mm) and 0.4 in. (10 mm), respectively. Thus contraction due to time dependent and temperature changes were reported as 0.65 and 0.20 in. (16.5 and 5 mm).[146] Approximate temperature dependent contraction was 0.15 in. (3.8 mm) and 0.30 in. (7.6 mm). Contraction due to shrinkage and creep was thus 0.5 in. (12.7 mm) for the conventional concrete. After 30 days, a 0.1 in. (2.5 mm) expansion due to shrinkage and creep was retained in the slab with the expansive component. Less than 0.1 in. contraction was observed for the 30 to 100 day period.

Biggs, Lemoore, and Sharonville Airport Pavements

Variations of joint opening for the Grade Beam 2 location are reported in Table 6. This joint is located between two 500-ft (152 m) slabs, thus the movement represents the length change for a 500-ft long slab. The contractor completed casting of all pavements by May 6, 1959. Therefore, the data in Table 6 are for slabs at least 1 month old. A maximum joint opening of 2.816 in. (71.5 mm) was measured at 114^{o}F (45.6^{o}C). Thus the slab length change was 2.160 in. (54.86 mm) when the approximately 2 month to 7 month pavement age interval is considered. The strain for this period was 4.5 millionths per ^{o}F (8.1 miilionths per ^{o}C). A lower temperature contraction factor is obtained when the December 29, 1959 to May 10, 1960 period is considered. A slab length change of 1.369 in. (34.773 mm) was observed for a temperature range of 72^{o}F (40^{o}C). Thus, the strain was 3.16 millionths per ^{o}F (5.7 millionths per ^{o}C). The pavement age interval was 7 to 12 months. Movement from December to May was primarily pavement expansion, even though daily temperature change caused expansions and contractions. The effects due to creep and shrinkage were probably less than the 3.16 millionths per ^{o}F (5.7 millionths per ^{o}C) factor.

Daily length change on May 11, 1961, at age of one year, was 0.703 in. (17.856 mm). The temperature range was 28^{o}F (15.6^{o}C). Temperature length change factor for this day was 4.18 millionths per ^{o}F (7.5 millionths per ^{o}C).

Strand friction for the Lemoore, Sharonville, and Biggs pavements are summarized in Table 7. Tendon friction losses varied from 17 lb per ft (248 N/m) to 40 lb per ft

TABLE 6 - SLAB MOVEMENTS AT BIGGS AFB

Date	Average Joint Opening, in.		Movement Every 24 hrs, in.	Concrete Temperature, °F		
	Maximum	Minimum		Maximum	Minimum	Differential 24 hrs
6/25/59	1.145	0.656	0.489	114	80	34
6/30/59	1.218	0.708	0.510	102	79	23
7/8/59	1.085	0.671	0.414	112	88	24
7/13/59	1.241	0.713	0.528	115	78	37
7/23/59	1.181	0.781	0.401	102	84	18
7/27/59	1.244	0.744	0.500	114	86	28
9/10/59	1.213	0.885	0.328	116	87	29
9/17/59	1.353	1.921	0.432	115	82	33
10/5/59	2.062	1.499	0.563	88	62	26
11/12/59	2.395	1.890	0.505	74	52	22
12/29/59	2.816	2.723	0.093	48	34	14
3/10/60	2.457	2.436	0.021	88	56	32
5/11/60	2.150	1.447	0.703	106	78	28

1 in. = 25.4 mm

TABLE 7 - STRAND FRICTION LOSSES FOR PRESTRESSED AIRFIELD PAVEMENTS

	Sharonville	Lemoore	Biggs
Load on Strand at End, kips	70.4	109	114.8
Slab Length, ft	512	500	500
Strand Friction lb/ft	40	27	17

1 kip = 4.448 kN
1 ft = 0.3048 m
1 lb/ft = 14.59 N/m

(584 N/m). The tendons at Biggs Air Force Base were tensioned prior to paving. This was done to improve the alignment prior to concrete placement. The tendon stress to improve alignment was 10 ksi (68.9 MPa).

Losses in tendon force, to one year, were measured with stress links at the slab three quarter points. These cables were tensioned first and maintained ungrouted. Losses of 1 kip (4.4 kN) per cable were sustained during prestressing of the other cables. Losses in tendon force, to one year, were 16 kips (71 kN) on average. The concrete contracted in the same period by 160 millionths after post-tensioning. Total concrete contraction strain was 260 millionths. This corresponds to a cable stress loss of 7,300 psi (50.3 MPa) or 3 kip (13.3 kN) force.

Dulles, Harrisburg and Kuztown Pavements

Strand friction losses were reported for plastic coated and grouted tendons used on Dulles Airport Access Road, Virginia[281], and Harrisburg[288] and Kutztown[284] pavements in Pennsylvania.

The calculated tendon friction losses are summarized in Table 8. Friction loss in plastic coated tendons varied from 41.8 to 49.3 lb per ft (610 N/m to 719 N/m) with an average of 45.6 lb per ft (665 N/m). Friction loss for strands in rigid steel conduits was 13.9 and 17.4 lb per ft (202 and 254 N/m) for 500 and 600 ft (152 and 183 m) long slabs. However, for a 400 ft (122 m) long slab it amounted to 46.4 lb per ft (677 N/m).

PRESTRESSED PAVEMENT DESIGN

Conventional concrete pavement thickness design is accomplished in three steps. First, pavement stresses are computed for ranges and numbers of loadings anticipated for the facility. Second, the fatigue consumption of each stress level is computed for an assumed slab thickness.

TABLE 8 - STRAND FRICTION LOSSES

Pavement	Slab Length, ft	Strand Load, kips	Strand Stress, ksi	Strand Friction Loss, lb/ft	
				Plastic Coated Strand	Rigid Steel Conduit
Dulles	400	29	201		46.4*
	500	29	201		13.9*
	600	29	201		17.4*
	760	29	201	45.8*	
	500	29	201	41.8*	
	400	29	201	49.3*	
Kutztown	500	46.9	218	11.4**	
Harrisburg	600	46.9	218	46.7*	

* Calculated from strand elongation data

** Calculated from load cell data

1 kip = 4.448 kN
1 ft = 0.3048 m
1 lb/ft = 14.59 N/m
1 ksi = 6.895 MPa

Third, the design thickness is determined when the fatigue consumed is approximately equal to the flexural strength of the concrete.

If these principals are followed for the thickness design of prestressed pavements the following two modifications might be anticipated:

1. equations for determining the slab stress would be extended to loadings beyond the elastic range, and
2. the flexural strength of the concrete should be increased by the compressive prestress in the pavement.

In addition to load stresses, several investigators have included stresses due to temperature and moisture gradients and those due to subbase restraint.

Stress Equations

Elastic Range

Sargious[269,270] developed a design procedure for stresses in the elastic range based on the following relationship:

$$f_t + f_p > f_{\Delta t} + f_f + f_c$$

where f_t = the concrete flexural strength

f_p = compression due to prestress

$f_{\Delta t}$ = stresses due to temperature and moisture gradient

f_f = restraint stresses due to subbase friction

f_c = stress due to imposed load.

Rearrangement of the preceeding equation permits solutions for allowable loads, thickness, or level of pavement compressive stress. In this procedure the pavement thickness is designed on the basis of no cracking. Thus, the design considers pavement response only within the elastic range. This approach has been used by a number of designers.

A second approach to an elastic design was presented by Kellersmann.[282] His analysis of stress was not based on the Westergaard interior stress formula. He states his reason for not using the Westergaard analysis as follows:

> "To derive the tensile stresses from the bending moments (Westergaard), the stress distribution over the cross section of the slab is assumed to be mostly linear. For comparatively small stresses this may be true, but for increasing stress, near to rupture, the linear distribution will be lost."

Therefore, for calculation of tensile stresses, Kellersmann used a stress distribution that was linear in the compressive stress region and was a third degree equation in the tensile stress region. The stress diagram is shown in Fig. 49.

Sector analysis, a method of successive approximations, has been proposed by Friberg[87] for the appraisal of pavement load stresses. This method was initially developed for conventional concrete pavements but was also suggested for use in the design of prestressed concrete pavements. The design assumes an uncracked pavement.

For airfield pavements stresses are computed for interior loading because slab ends and joints are generally supported on sleeper slabs. In addition, because of the width of runways and taxiways, edge loads are seldom experienced.

Melville[144] in a review of French and British design procedures reported that airfield pavement thickness was computed on the basis of either Westergaard's or Boussinesq's analysis. Both methods assume a uniform homogeneous medium and that stresses in the pavement and subgrade are within the elastic range. Burmister's equations are used in the application of Boussinesq's theory to a multilayered system.

For example, Peltier[47] used Boussinesq's theory and developed the following equation for maximum bending moment for pavement edge loadings:

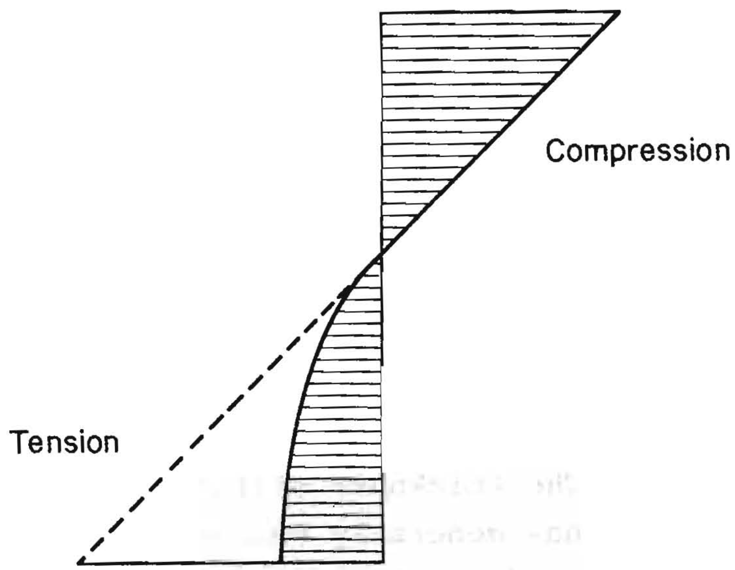


FIG. 49 - STRESS DISTRIBUTION IN LOADED PRE-
STRESSED PAVEMENT ACCORDING TO
KELLERSMANN

$$M = P(1-\mu)\frac{1}{1.7 + 4.1Z + 10.3Z^2}$$

where M = maximum bending moment

μ = Poisson's ratio

Z = radius of relative stiffness

P = load.

The above equation does not apply to the non-elastic state of stress, which occurs following bottom surface cracking.

Peltier suggested the following relationship to account for the effect of repeated load applications:

$$W_n = \rho\left(A + B \log \frac{T_n}{T_o}\right)$$

where W_n = vertical deformation of pavement

ρ = tire pressure

T_n = traffic in tons since construction

T_o = heaviest wheel load in tons since construction

A & B = soil constants determinable from bearing tests.

The thickness of internally prestressed highway pavements has generally been selected as the minimum necessary to provide sufficient cover for the prestressing steel. Balancing prestress compression and flexural strength has only been considered for airfield pavements.

Externally prestressed pavements generally have no tendons. Thus, cover requirements are not a consideration. Externally prestressed pavements of only 4-in. (100-mm) and 3.2-in. (80-mm) thickness were constructed in Holland. However, variation of prestress levels due to temperature change contributed to pavement blowups.

Only one of the preceding design procedures used the principal of fatigue consumption. The others applied a safety factor to the allowable concrete flexural stress.

Empirical Procedure

Designs restricted to the elastic pavement response predicted allowable loads that were considerably lower than those causing structural distress. To overcome the inadequacy of the elastic method an empirical equation was developed by Sale, Hutchinson, and Carlton(214). This equation was based on the results of reduced-scale and full-scale prestressed pavement load tests.

$$P_F = \frac{h^2 \, (R + R_p)}{6 \left(C \frac{M_o}{P_o} - \frac{M_r}{P_o} \right)}$$

where P_F = top surface cracking load

h = slab thickness

R = flexural strength of the concrete

R_p = compression in concrete due to prestress

P_o = bottom surface cracking load

M_o = maximum positive bending moment due to P_o

M_r = radial moment at circumference of top surface crack

C = radial moment correction factor

Values for M_o/P_o, C, and M_r/P_o were solved and presented in graphs.

Moving load tests on reduced-scale prestressed pavements were used to determine a fatigue factor. Full-scale pavements were tested at two different levels of longitudinal and transverse prestress to confirm the empirical formula. The full-scale tests were made on pavements built at Sharonville, Ohio. The empirically developed procedure was used for the design of the Biggs Air Force Base, Taxiway T-3.

Stress Equations for Non-Elastic Response

Cot and Becker[104] developed design methods that consider the state of stress beyond the elastic response. Cot and Becker assumed a number of radial cracks forming under the load on the bottom surface of the slab. These cracks are assumed to occur when the load for bottom surface cracking exceeded. The deformations are then mostly inelastic. Moments in the tangential direction, M_1, are computed from the following equation:

$$M_1 = T\,c\left(1 - \frac{2}{3}\,\frac{r + h}{c}\right) + \frac{R_1}{2\mu}\left(0.666 - \frac{2}{3}\,\frac{r + h}{c}\right) + \frac{R_o}{2\mu}\left(\frac{1}{2} - \frac{2}{3}\,\frac{r + h}{c}\right) + 4200$$

where

T = shear per unit length in the radial direction

R_o, R_1 = soil reactions at center and distance c from center of load

c = radius of area describing bottom surface radial cracks

h = slab thickness

μ = Poisson's ratio.

The soil reaction due to deflection, W, is related to the modulus of subgrade reaction, k, by the formula

$$R = k\pi\, c^2\, W$$

The equation for shear per unit length in the radial direction is

$$T = -\frac{P}{2\pi C} - R$$

The equation for moment in the radial direction is

$$M_2 = \frac{h^2}{6}\, n\left(3 - 2\lambda + \frac{f_t\,\lambda^2}{n}\right)$$

where

n = prestress

f_t = concrete tensile strength

λh = distance from surface to fiber subjected to a tensile stress of f_t

Effects of moving loads were also considered by Cot, Becker, and Lorin.[84] The measurements for repeated load tests at Orly were used to develop the following equation relating cumulative deformation to the deformation after one load application:

$$W_n = W_1 + a \log n$$

where

W_n = deformation after n cycles

W_1 = deformation after one cycle

a = constant

n = number of cycles.

As a result of design studies at Milan, Levi[64] used a derivation of Westergaard's equations for pavement response in the elastic range. He concluded however, that as far as prestressed pavements are concerned, the elastic range is limited. Thus, he proposed that design should also include the pavement response in the inelastic range. The primary considerations for inelastic phase response were described by Levi as follows:

1. increased deflections near the loaded area,
2. decreased bending moments in the radial direction,
3. increased negative bending moments and movement of moment maxima towards the center of the loaded area.

Levi assumed an initial circular crack at the bottom surface of the pavement. This crack would be caused by a load exceeding the available compression and flexural strength. Observations from tests by Carlton and Behrman[235] and by Christensen and Janes,[241] however, have indicated that the radial crack assumption by Cot and Becker[104] is more reasonable.

Osawa[227] developed a theoretical method for determining stresses outside the range where elastic theory is applicable. The method predicts the load required to produce top surface cracking in prestressed pavement. Solutions are limited to the case of a load applied at the interior of a slab on a dense liquid subgrade, and where the level of prestress is the same in both directions.

Solutions for the radial and tangential moment distributions, and deflections are shown in Figs. 50 and 51 respectively for increments of load between those causing bottom and top surface cracking.

MATERIALS FOR PRESTRESSED CONCRETE PAVEMENTS

Prestressed concrete pavements consist of concrete slabs prestressed with or without steel tendons. The slabs are generally placed on a subbase layer. To facilitate prestressing and to reduce subgrade restraint stresses, a friction reducing layer is usually provided between concrete slabs and subbase. Thus, materials for prestressed concrete pavements include conc ete, prestressing steel, subbases, and friction-reducing mediums. The properties of these materials as applicable to prestressed concrete pavements are reviewed in the following sections.

Concrete

Important properties of concrete include compressive strength, tensile strength, modulus of elasticity, and coefficient of expansion. Consideration is given to factors affecting these properties such as temperature and moisture variations, water-cement ratio, age, amount of entrained air, and curing conditions. In addition, discussions are presented on the strength properties of concrete at early age, creep, and shrinkage. A separate section is devoted to shrinkage-compensating cements.

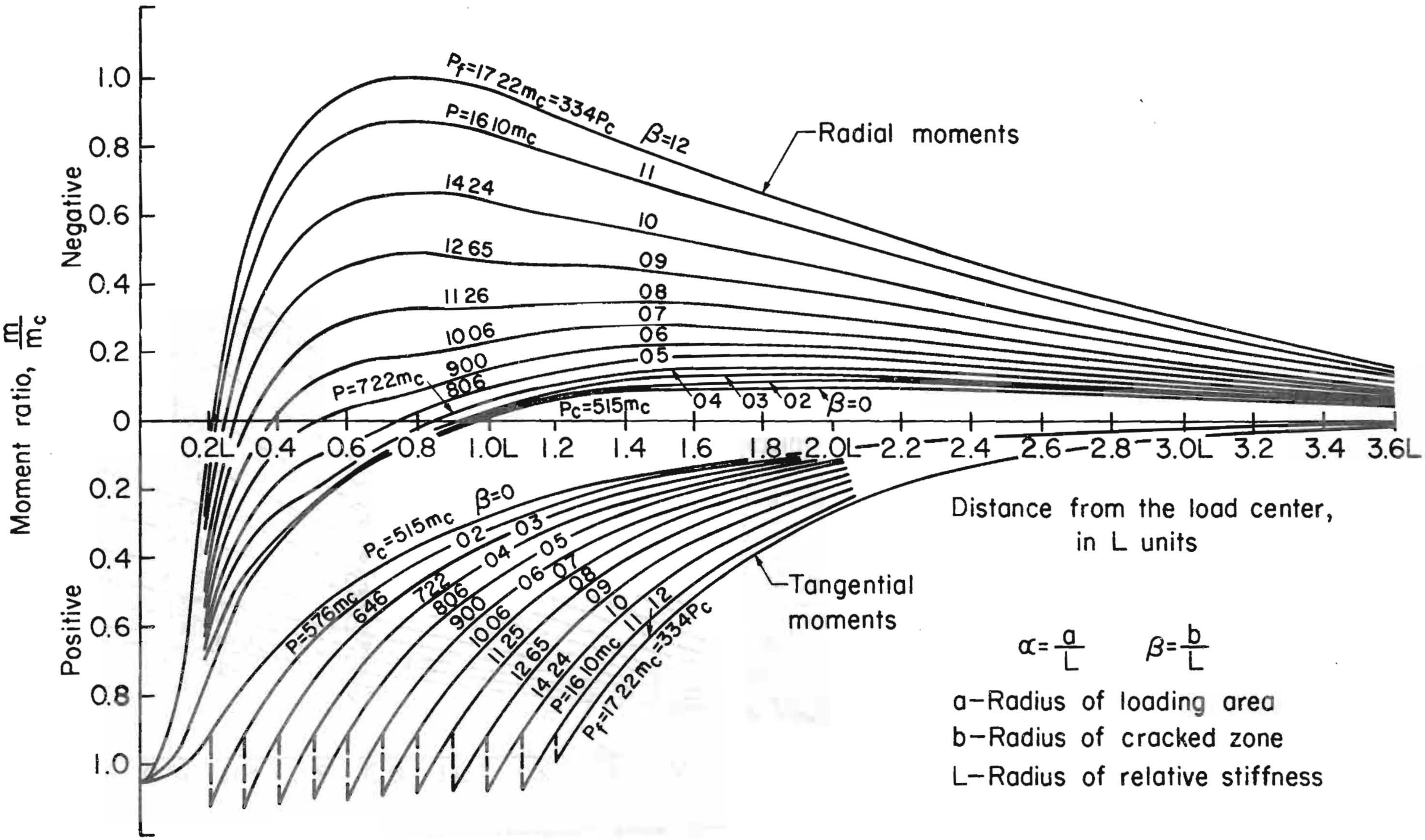


FIG. 50 - RADIAL AND TANGENTIAL MOMENT CURVES (α = 0.2)

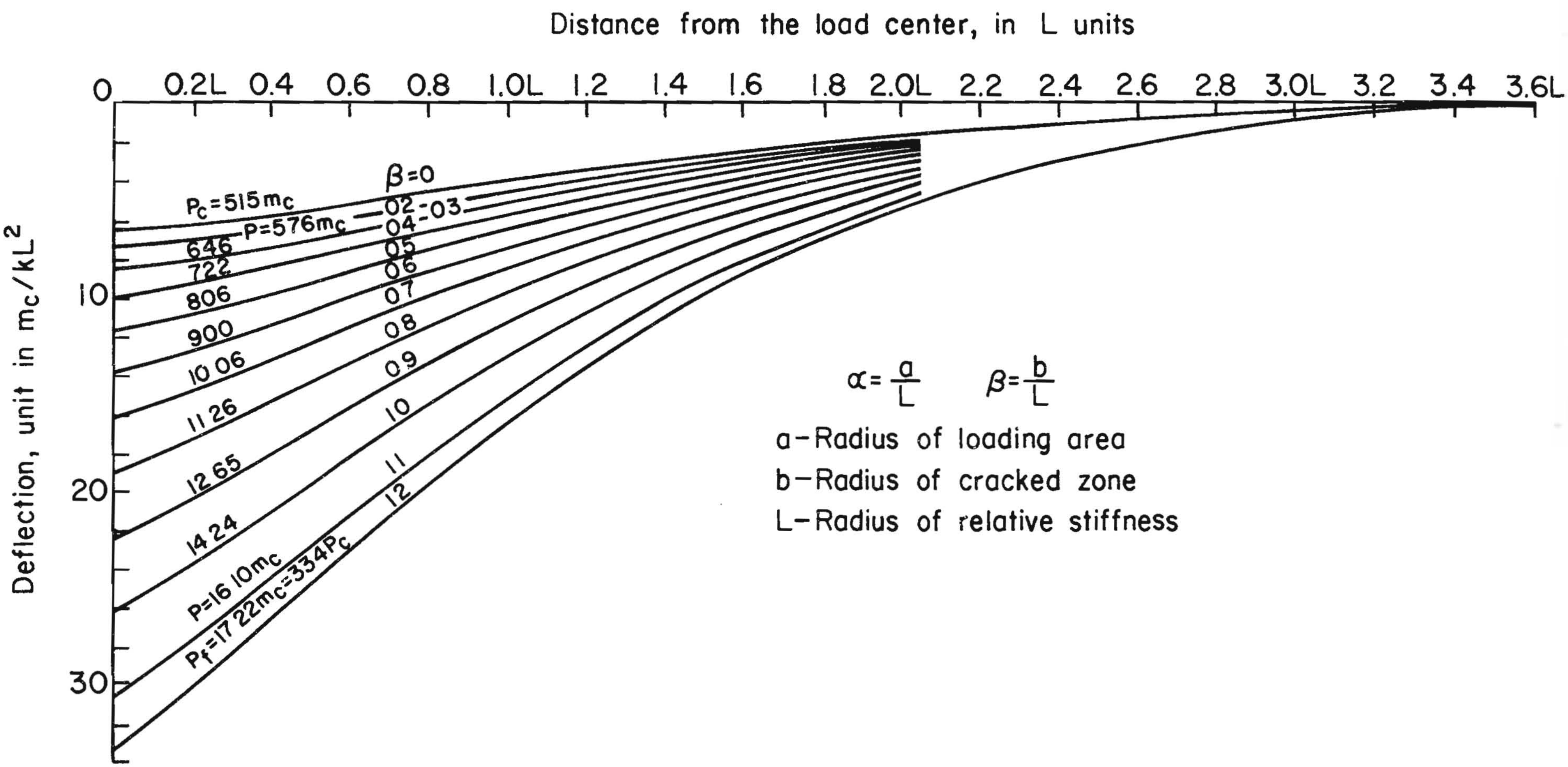


FIG. 51 - DEFLECTION CURVES (α = 0.2)

Compressive Strength

Nasser and Evans[369] investigated the effect of low temperatures on the strength of air-entrained concrete. The results showed practically no change in compressive strength for specimens cast and cured at 72°F (22°C) and subsequently lowered to a temperature of 32°F (0°C). In contrast, strength increased for specimens lowered to a temperature of -40°F (-40°C). The increase was 80 and 12.5 percent for wet-cured and oven-dried specimens, respectively. The larger strength increase obtained for the wet-cured specimens was attributed to ice fillets in the capillary pores and their binding effects.

In a study of Nasser[368], compressive strength decreased linearly by about 20 percent as the specimen temperature was increased from 70°F (21°C) to 160°F (71°C). The average decrease was 7.22 psi/°F (89.6 kPa/°C). These tests also showed that increased air content reduced both the compressive strength and the modulus of elasticity.

Klieger[298] investigated the effect of specimen age on compressive strength. The data showed that compressive strength increased rapidly during the first 28 days. At the end of one year, the compressive strength was 140 percent of the compressive strength at 28 days of age. There was little change in compressive strength of concrete after one year. For example, the compressive strength of concrete made with 564 lb of cement per cubic yard (334 lb/m^3) increased from 1000 psi (6.9 MPa) at 1 day to 6500 psi (44.8 MPa) at 28 days and 9000 psi (62.1 MPa) at 1 year. However, at three years the strength was only 9200 psi (63.4 MPa).

Investigations by Pihlajavaara[378] showed that compressive strengths of dry concrete specimens were 30 to 60 percent higher than those for wet specimens. The 30 and 60 percent strength increase occurred for specimens with water-cement ratios of 0.5 and 0.75 respectively.

Tensile and Flexural Strength

Monfore and Lentz[316] investigated the effect of temperature on the tensile strength of concrete. The results indicated that tensile strength did not change for temperatures above freezing. However, tensile strength increased for temperatures below freezing.

Klieger[298] studied the effect of specimen age on the flexural strength of concrete. He indicated that flexural strength increased rapidly in the first 28 days at which time it reached approximately 70 percent of its maximum. The maximum strength was attained at approximately 3 months to 1 year depending on the type of cement used. For example, the flexural strength of concrete made with 564 lbs (256 kg) of Type I cement was 250 psi (1.7 MPa) at 1 day, 700 psi (4.8 MPa) at 28 days, and 900 psi (6.2 MPa) at both 1 and 3 years.

Modulus of Elasticity

An investigation by Nasser[368] indicated that the modulus of elasticity decreased as temperature increased. The data showed that increasing the temperature from 70°F (21°C) to 115°F (46°C) decreased the modulus of elasticity by 16 percent. As indicated by numerous investigators, the modulus of elasticity decreased with increased air content.

Berwanger and Sarkar[362] investigated the effects of temperature, water-cement ratio, age, and curing conditions on the modulus of elasticity. Their major findings were:

1. Modulus of elasticity decreased linearly as water-cement ratio increased.
2. Modulus of elasticity increased with age.
3. Wet-cured specimens had higher modulus of elasticity than dry-cured specimens at the same age.
4. Modulus of elasticity decreased linearly as temperature increased above freezing. Also,

the modulus of elasticity increased linearly as temperature dropped below freezing. However, the rate of change of the modulus of elasticity with temperature was higher for temperatures below $32^{\circ}F$ ($0^{\circ}C$) than for temperatures above $32^{\circ}F$ ($0^{\circ}C$). The rate of change for temperatures above $32^{\circ}F$ ($0^{\circ}C$) was 1.6 ksi/$^{\circ}F$ (19.9 MPa/$^{\circ}C$) and for temperatures below $32^{\circ}F$ ($0^{\circ}C$) was 7.0 ksi/$^{\circ}F$ (86.9 MPa/$^{\circ}C$).

5. At temperatures above freezing, the rate of change of modulus of elasticity with temperature was not affected significantly by age for either dry or wet cured specimens.

A study by Berwanger(361) indicated that the rate of change of the modulus of elasticity with temperature was not affected by age after 200 days.

Thermal Coefficient of Expansion

Berwanger and Sarker(362) determined thermal coefficients of expansion of concrete for a temperature range of -100 to +150$^{\circ}F$ (-73.3 to 65.6$^{\circ}C$). As shown in Table 9, these data indicate the following:

1. The thermal coefficient of expansion of concrete was higher for temperatures above freezing than for temperatures below freezing.
2. The thermal coefficient of expansion increased with age for both wet and dry cured specimens.
3. The thermal coefficient of expansion increased as the water-cement ratio decreased.

Similar results were reported by Saemann and Washa(302). In their investigation, concrete specimens were moist-cured at normal temperature for 14 days, then stored in air at $70^{\circ}F$ ($21.1^{\circ}C$) and 50% relative humidity for 13 days. The coefficients of expansion varied from 3.9 millionths/$^{\circ}F$ to 4.4

TABLE 9 - EFFECT OF AGE ON THE COEFFICIENT OF THERMAL EXPANSION OF CONCRETE

Water-Cement Ratio	Curing Condition	Thermal Coefficient, Millionths/°F Age days 7	28	84
			Above Freezing (32 to 150°F)	
.45	dry	4.68	4.80	5.01
.45	wet	4.68	4.75	4.86
.49	dry	4.16	4.30	4.50
.49	wet	4.16	4.25	4.36
.58	dry	3.65	3.87	4.20
.58	wet	3.65	3.76	3.90
.67	dry	3.40	3.70	4.00
.67	wet	3.40	3.56	3.80
			Below Freezing (-100 to 32°F)	
.45	dry	3.60	3.90	4.30
.45	wet	3.60	3.75	3.95
.49	dry	3.27	3.70	4.10
.49	wet	3.27	3.45	3.75
.58	dry	3.10	3.35	3.80
.58	wet	3.10	3.25	3.60
.67	dry	2.94	3.20	3.52
.67	wet	2.94	3.16	3.38

millionth/°F = 1.8 millionth/°C

millionths/$^{\circ}$F (7.02 millionths/$^{\circ}$C to 7.92 millionths/$^{\circ}$C) for water-cement ratios varying from 0.84 to 0.48.

Tokuda and Ito(323) investigated the effect of the amount of aggregate in concrete on the thermal coefficient. The data showed that the coefficient varied from 4.5 to 6.6 millionths/$^{\circ}$F (8.1 to 11.9 millionths/$^{\circ}$C) for aggregate contents of 0.8 to 0.4 by absolute volume, respectively.

Properties of Concrete at Early Age

Friberg(222) investigated early-age strengths and associated deformations in compression and tension test specimens at different ambient temperatures. Earliest testing age varied from 3 to 24 hours depending on temperature. The investigation included 190 compression, 175 tensile splitting, and 165 flexural tests.

Compression strength, tensile splitting strength, and modulus of rupture for 100, 70 and 40°F (37.8, 21.1 and 4.4°C) concrete at ages up to 28 days are shown in Fig. 52. Moduli of elasticity in compression and tension for increasing age are shown in Fig. 53. The compression data were based on measurements at low stress levels. Tension date were based on beam strains at failure. Data are shown for concrete at both 100° and 70°F (37.8° and 21.1°C). Based on these data the following conclusions were stated:

1. Modulus of elasticity reached mature values early, and critical limits of extensibility were low at early age especially for the higher ambient temperature.
2. Under burlap curing insulation, the concrete strain variations due to daily temperature changes could be accommodated by the concrete without cracking.
3. Prestressing could be applied at early age, except at low construction temperatures.

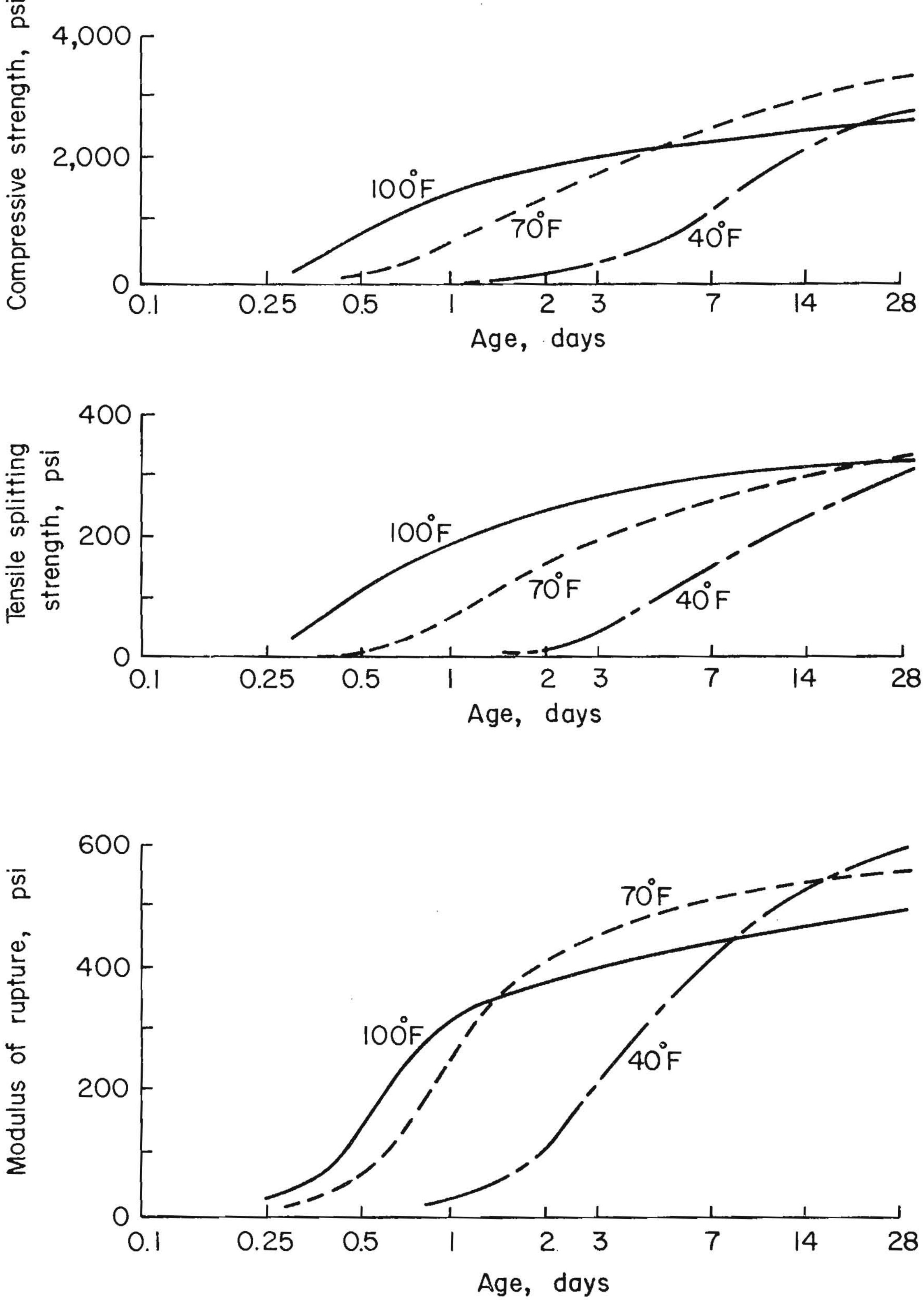


FIG. 52 - EFFECT OF AGE AND TEMPERATURE ON STRENGTH OF CONCRETE

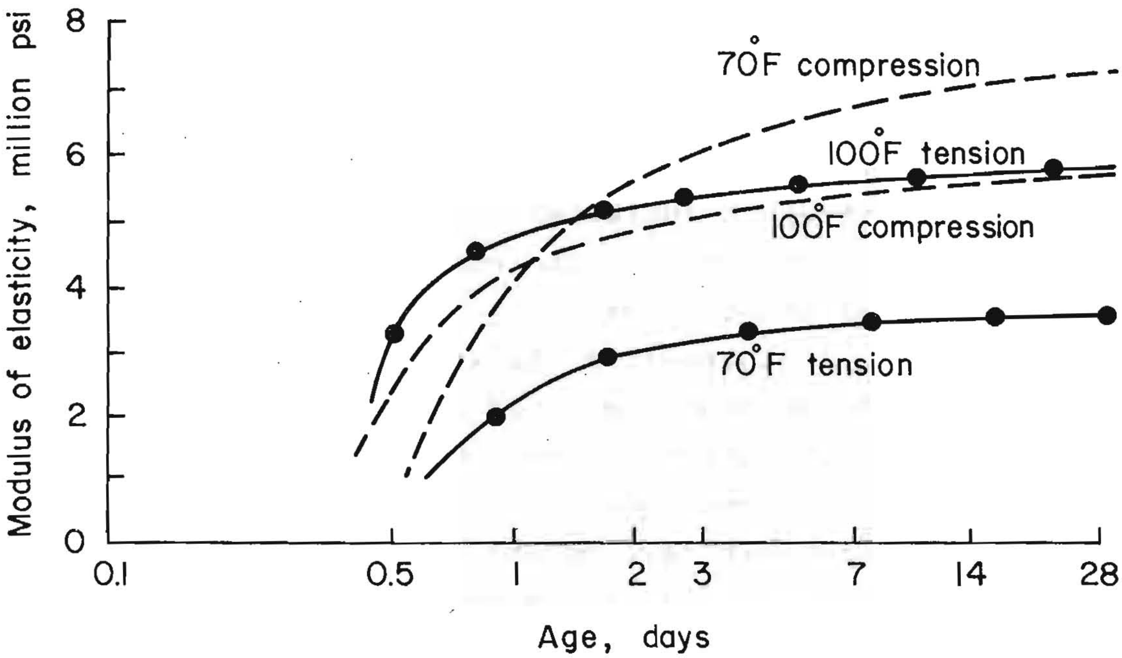


FIG. 53 - EFFECT OF AGE AND TEMPERATURE ON MODULUS OF ELASTICITY

4. The best possible curing and insulation should be used to protect the concrete at least up to the time of prestressing.

Klieger(307) investigated the strength of concrete made with different types of portland cement, and mixed, placed, and cured at various temperatures between 25°F and 120°F (-3.9°C and 48.9°C). The data indicated that there is a curing temperature during the early life that is considered optimum with regard to strength at later ages. Effect of cement temperature was unimportant, except as it affected concrete temperature after mixing. More air entraining agent was required to obtain a given air content as concrete temperature increased and slump decreased. In addition the following conclusions were stated:

1. At 1, 3, and 7 days, concrete strength increased with an increase in the initial and curing temperature of the concrete.
2. Increasing the initial and curing temperature resulted in significantly lower strengths at 3 months and 1 year.
3. Temperature influenced flexural strength development in much the same manner as it influenced compressive strength development.
4. Concrete strength did not correlate well with the index known as degree-days.
5. For concretes with calcium chloride added, strength increases due to the accelerator were proportionately greater at early ages and lower temperatures. The use of calcium chloride frequently resulted in flexural strengths at later ages somewhat lower than those for comparable concretes without calcium chloride. Maximum reductions were of the order of 10 percent.

Freeze-Thaw Durability

The resistance to freezing and thawing and de-icer scaling of concrete used in the fabrication of prestressed elements was studied by Klieger[311]. Variables were type of cement, accelerator, curing condition, air entrainment, and prestressing force. The following conclusions were stated.

1. All concretes required entrained air to provide a high degree of resistance to freezing and thawing and de-icer scaling.
2. Concretes made with Type I and Type III portland cements were equally durable.
3. The use of calcium chloride to accelerate strength gain resulted in significant reduction in the durability of moist-cured air-entrained concretes. In contrast, calcium chloride had little influence on durability of air-dried, air-entrained concretes. The possibility of corrosion of steel in the presence of chloride ions should be kept in mind when considering the use of an accelerator.
4. Curing at an elevated temperature to attain high early strength does not impair the durability of air-entrained concrete, provided some drying follows the elevated curing cycle prior to exposure. Additional curing of non-air-entrained concretes cured at 160°F (71.1°C) did not raise the level of durability to that for the air-entrained concretes.
5. A prestressing force providing a compressive stress of 350-400 psi (2.4-2.8 MPa) in the concrete had no significant influence on the resistance to freezing and thawing of either air-entrained or non-air-entrained concretes.
6. For the same water-cement ratio, the concretes made with Type IA and Type IIIA portland cements generally show similar drying shrinkage values.

7. For the same compressive strength at the time of loading, concretes made with Type IA and IIIA portland cements showed similar creep strain values.

8. Concretes cured at an elevated temperature in a 24-hr. cycle show less drying shrinkage and creep under sustained stress than those cured 28 days at normal temperature.

9. The low water-cement ratio, richer mixes showed lower drying shrinkage and creep strain values than the higher water-cement ratio, leaner concretes.

10. The different aggregates used in these tests had little influence on drying shrinkage and creep properties.

Cordon[330] made the following recommendations for obtaining good durability concrete.

1. Entrain 4 to 6 percent air in all exposed concrete that may be saturated in freezing weather.
2. Avoid concrete aggregates having high absorption.
3. Use the minimum amount of mixing water possible.
4. Avoid saturation of exposed concrete in freezing weather.
5. Be sure the hydration of portland cement is well advanced before concrete is subjected to freezing and thawing.
6. Prevent rapid drying of the surface of exposed concrete before the bleeding is complete.
7. Do not finish the surface of exposed concrete until bleed water has disappeared.
8. Avoid the use of salts for ice or snow removal.
9. After curing, allow exposed concrete to dry as much as possible, then seal the surface.
10. Provide adequate drainage for all exposed concrete surfaces.

Drying Shrinkage and Creep

To maintain the required degree of prestress during the service life of pavement, prestress losses that occur during and following construction should be considered in the design. Shrinkage and creep of concrete are two of the many factors contributing to loss of prestress. These two properties are discussed in the following.

Shrinkage: Drying shrinkage is shortening caused by withdrawal of water from concrete stored in unsaturated air. The shrinkage depends on the degree of drying that occurs and therefore upon the humidity, temperature and ventilation of the surroundings of the concrete. In humid cold air, concrete shrinks less than in dry warm air. The rate of shrinkage is therefore linked to the relative atmospheric humidity and temperature of the surroundings in which the concrete is placed.

Test results(344) have indicated that representative values for drying shrinkage of concrete specimens ranges from 400 to 800 millionths when stored at a relative humidity of 50 percent.

Shrinkage may continue for several years depending on the size and shape of the concrete mass. However, the rate of shrinkage decreases rapidly with time. The rate and amount of drying shrinkage for 4x4x40-in. (102x102x1016 mm) concrete specimens is shown in Fig. 54.(344) Specimens were initially moist-cured for 14 days at 70°F (21.1°C), then stored for 38 months in air at 70°F (21.1°C) and 50 percent relative humidity. Shrinkage recorded at an age of 38 months ranged from 600 to 790 millionths. An average of 34 percent of this shrinkage occurred within the first month. At the end of 11 months, an average of 90 percent of the 38-month shrinkage had taken place.

Drying shrinkage of concrete is greatly influenced by the aggregate which restrains the amount of shrinkage

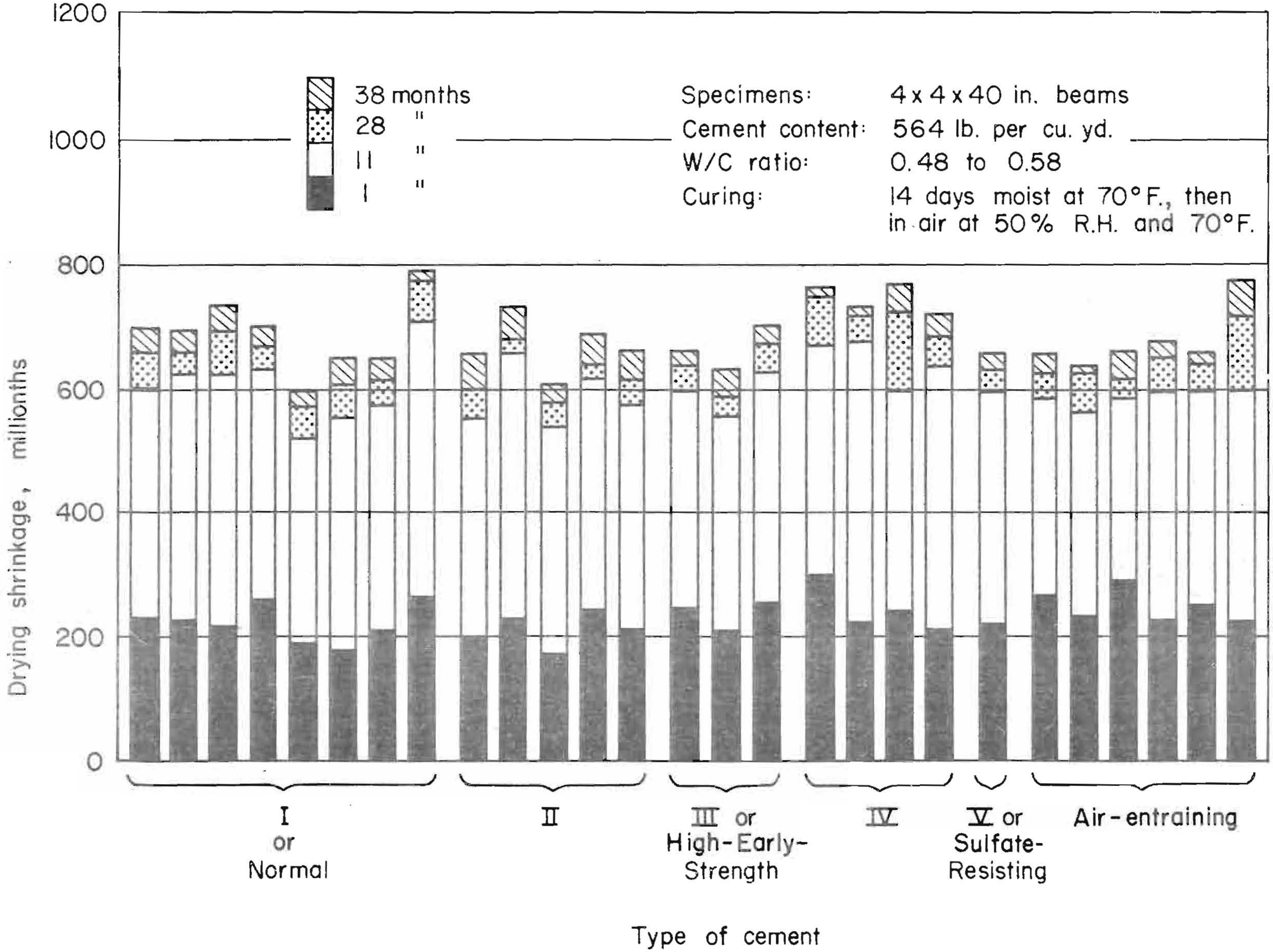


FIG. 54 - EFFECT OF CEMENT TYPE ON DRYING SHRINKAGE

that can occur. The size and grading of aggregate, do not influence the magnitude of shrinkage directly, but a larger aggregate permits the use of a leaner mix and hence results in lower shrinkage. The elastic properties of aggregate influence the degree of restraint. For example, expanded shale aggregate leads to higher shrinkage than normal weight aggregate.

The properties of cement have little influence on the shrinkage of concrete. For example, shrinkage of concrete made with aluminous cement is of the same magnitude, but it takes place much more rapidly than when portland cement is used.

The addition of calcium chloride was found to increase shrinkage by varying amounts, generally between 10 and 50 percent. In contrast, plastizing agents that allow a reduction in the water content of the mix have only negligible effect on shrinkage.

The most important controllable factor affecting shrinkage is the amount of water per unit volume of concrete. Shrinkage can be minimized by keeping the water content of the paste as low as possible and the total aggregate content of the concrete as high as possible. Use of low slumps and placing methods that minimize water requirements are thus major factors in controlling shrinkage. Any practice that increases the water requirements of the cement paste, such as the use of high slumps, excessively high concrete temperatures at placement, or smaller coarse aggregate, increases shrinkage.

The shrinkage of reinforced concrete is less than that for plain concrete, the difference depends on the amount of reinforcement. Steel reinforcement restrains but does not prevent drying shrinkage. In reinforced concrete structures, drying shrinkage is commonly assumed to be 200 to 300 millionths.[344]

Creep: When concrete is loaded, the deformation caused by the load may be divided into two parts. One is deformation that occurs immediately, the other time-dependent deformation that begins immediately but continues for years. The first deformation is known as elastic shortening, the latter is called creep. Thus, creep may be defined as the increase in strain under sustained stress.

In newly placed concrete, the change in volume or length due to creep is largely unrecoverable. However, creep that occurs in old or dry concrete is largely recoverable.

Creep is dependent upon (1) the magnitude of stress, (2) the age and strength of the concrete when the stress is applied, and (3) the length of time the concrete is stressed. It is also affected by other factors related to the quality of the concrete and conditions of exposure. Some of these factors are the kind, amount, and maximum size of aggregate; type of cement; amount of cement paste; size and shape of the concrete mass; amount of steel reinforcement; and curing conditions. It was reported that concrete which exhibits high shrinkage generally has high creep. However, not all factors affect creep and shrinkage in the same manner.

Under continuous load, creep continues for many years, but the rate decreases with time. Within the normal stress ranges, creep is proportional to stress. The ultimate magnitude of creep of plain concrete can range from 0.2 to 2.0 millionths per psi (0.03 to 0.29 millionths/kPa) and is ordinarily about 1 millionth per psi (0.145 millionths/kPa) or less.[344]

Fig. 55 illustrates that effect of compressive strength on the amount of creep during a period of one year. Creep tests from which these data are taken were made with 6x12-in. (152x305 mm) concrete cylinders. The specimens were loaded continuously in compression to a stress of 600 psi (4.1 MPa) after moist-curing for 7 days. The stress was maintained uniformly for one year while length changes were measured periodically. Total shortening due to creep at the

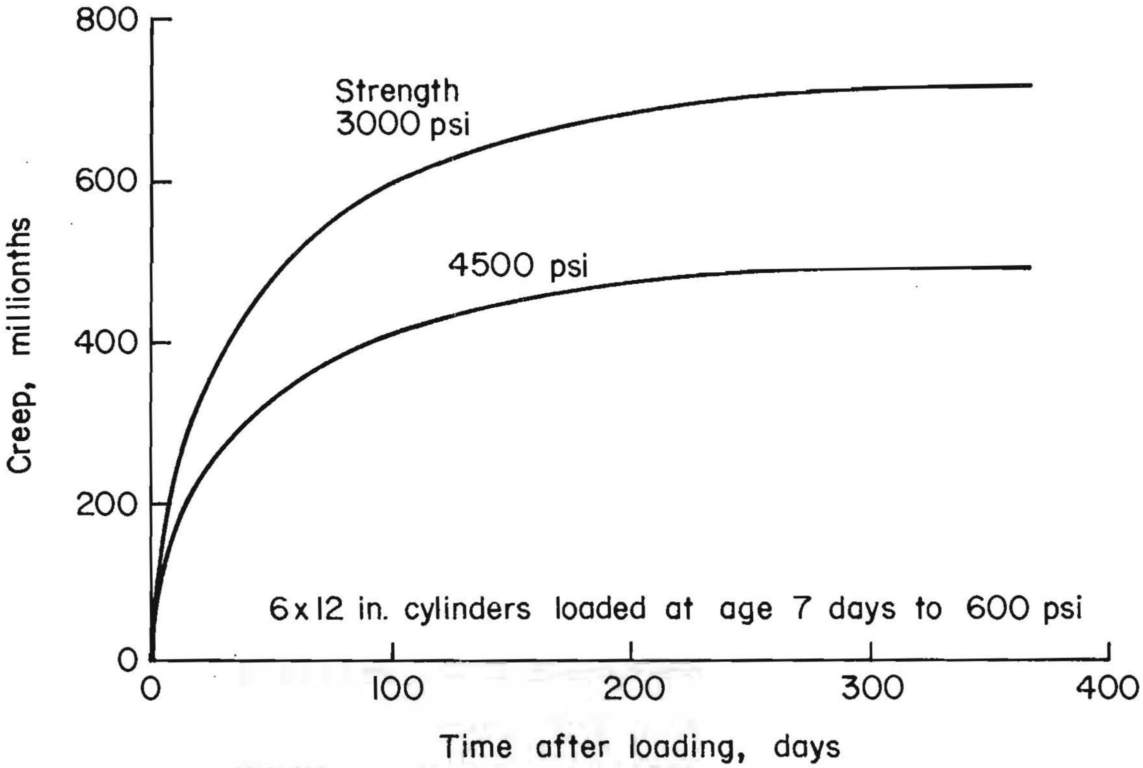


FIG. 55 - EFFECT OF COMPRESSIVE STRENGTH ON CREEP

end of the year was 730 millionths for the 3,000 psi (21 MPa) concrete and 490 millionths for the 4,500 psi (31 MPa) concrete. An increase in strength results in a significant reduction in creep as indicated by these data.

Concrete specimens of equal strength but of different age will have different creep characteristics. Those loaded at a late age will creep less than those loaded at an early age.

Numerous tests have shown that creep increases with a decrease in the relative humidity of the surrounding medium. For example, at a relative humidity of 50 percent, creep may be two to three times greater than at 100 percent. Therefore, proper curing of concrete at an early age is recommended not only to reduce drying shrinkage, but also to reduce creep.

Shrinkage-Compensating and Expansive Cements

Shrinkage-compensating cements have been developed during the past four decades primarily by investigators in France, the U.S.S.R., and the United States. These cements are designed to expand, during the curing period. The amount of expansion is intended to produce a compressive stress that will be partially or totally relieved during subsequent drying shrinkage, but that will prevent formation of high tensile stresses causing cracking of concrete. The properties of shrinkage-compensating concretes are in most respects similar to those of portland cement concretes. However, shrinkage-compensating concretes frequently exhibit a somewhat greater slump loss and tend to set more rapidly than portland cement concretes.[371] Properties of hardened expansive cement concretes were investigated and described in several summary reports[326, 327, 329,350].

Extensive studies were made in the 1940's in the Soviet Union[312], and in the 1950's and 1960's at the University of California, Berkley, where Type K cement was developed. Results of the Berkley studies were summarized in laboratory and research reports[329,328]. In the 1960's, the Portland Cement Association made extensive studies that resulted in development of Type S cement. A Type M cement was developed, also, in the 1960's by the Universal Atlas Cement Division of the U.S. Steel Corporation. The historical development of expansive cement concrete is covered in a paper by Kalousek.[359]

Factors influencing the characteristics of expansive cement concretes include cement type, cement content, type of aggregate and admixture, mixing time, type of curing, temperature during mixing and curing, and degree of restraint against expansion.

The properties of both plastic and hardened shrinkage-compensating cement concretes were determined to be essentially similar to those made with Type I portland cement.[363] However, to compensate for the slump loss of expansive cement concretes, the water-cement ratio for shrinkage-compensating concrete is increased by 0.05 to 0.10 above that required for a corresponding concrete mix made with Type I portland cement. Thus, similar workability level is obtained.

Studies on continuously reinforced concrete and prestressed concrete pavements made with expansive cement concrete[367] concluded that compressive stress in the concrete was as much as 300 psi (2.1 MPa) during the early curing period but decreased to 150 psi (1.0 MPa) after a longer period. The induced compressive stress reduced cracking in the continuously reinforced concrete pavement.

Another experimental self-stressing reinforced concrete pavement was constructed in September, 1968 in Glastonbury, England.[319] The 1500 ft (457 m) highway section contained 3 slabs, 24-ft (7 m) wide, 6-in. (152 mm) thick and approximately 490-ft (149 m) long. Difficulty in construction was caused by fast setting of the expansive cement concrete. Average slab expansion in the longitudinal direction was 3.12 in. (79.25 mm) and the maximum growth was reached about 24 to 48 hours after completion of paving operations.

Four years after construction, the Glastonbury pavement was inspected.[335] This inspection revealed two types of transverse cracks, those greater than 12-ft (3.7 m) long and those less than 12-ft (3.7 m) long. The cracks were opened but the transverse joints were closed. Also, spalling was observed at some full-width cracks. Based on the results of the experiments, two principal conclusions were stated regarding prestressed pavements made with shrinkage compensating cements. These were:

1. Some type of anchorage should be provided along the reinforcement to attain uniform longitudinal prestress.
2. Slab lengths should be less than 500 ft (152 m).

A rather successful application of expansive cement concrete was reported for two taxiway reconstruction projects at Love Field Airport, Dallas, Texas.[372] Field tests on expansive cement concrete projects showed similar performance to that obtained in the laboratory. Volume change in pavement apparently stabilized at 90 days. Three years of service on one project revealed little to no cracking between joints.

Prestressing Steel

Three types of high-strength steel are being used in the United States for prestressing. These are uncoated stress-relieved wires, stress-relieved strands, and high-tensile alloy bars. Others, such as straightened-as-drawn wire and oil-tempered wire are used in other parts of the world. In the following sections the three types of high-strength steel are described. This is followed by brief discussions of allowable stresses, relaxation of tendons, and fatigue of prestressing steel.

Stress-Relieved Wire

Commonly used in post-tensioned construction, cold-drawn stress-relieved wire is manufactured to conform to ASTM Designation: A421 "Standard Specifications for Uncoated Stress-Relieved Wire for Prestressed Concrete"[(380)]. Two types of wire are specified (BA and WA), depending on whether button or wedge-type anchorages are used. The specifications include the minimum elongation at rupture and diameter tolerances. The major strength requirements specified by ASTM A421 are summarized in Table 10.

Stress-Relieved Strand

Two forms of stress-relieved strand are available. These include seven wire strand, which conforms to ASTM Designation: A416 "Specifications for Uncoated Stress-Relieved Wire for Prestressed Concrete," and larger strands with diameters up to 1-11/16 in. (43 mm). Basic strength, area and weight requirements for seven-wire strands, specified by ASTM A416,[(379)] are given in Table 11 for two grades of steel.

Seven-wire strands are made by twisting six wires, on a pitch length of 12- to 16-wire diameters, around a slightly larger, straight central wire. The strands are stress-relieved after being stranded.

High-Tensile Strength Bars

Both smooth and deformed high-tensile-alloy steel bars are made to conform to ASTM Designation: A322 "Standard Specification for Hot Rolled Alloy Steel Bars."[(381)] The specification generally covers the chemical composition of many grade designations of alloy steel bars. Sizes and selected physical properties are summarized in Tables 12 and 13.

TABLE 10 - PROPERTIES OF STRESS-RELIEVED WIRE FOR PRESTRESSED CONCRETE REQUIRED BY ASTM A421

Nominal Diameter, in.	Min. Tensile Strength, psi		Min. Stress at 1% Extension, psi	
	Type BA	Type WA	Type BA	Type WA
0.192		250,000		200,000
0.196	240,000	250,000	192,000	200,000
0.250	240,000	240,000	192,000	192,000
0.276		235,000		188,000

1 in. = 25.4 mm
1 psi = 6.895 kPa

TABLE 11 - PROPERTIES OF UNCOATED SEVEN-WIRE, STRESS-RELIEVED STRAND FOR PRESTRESSED CONCRETE REQUIRED BY ASTM A416

Nominal Diameter of Strand, in.	Min. Breaking Strength, lb	Nominal Steel Area of Strand, sq. in.	Nominal Weight of Strand, lb/1000 ft	Minimum Load at 1% Extension, lb
		Grade 250		
1/4 (.255)	9,000	0.036	122	7,650
5/16 (.313)	14,500	0.058	197	12,300
3/8 (.375)	20,000	0.080	272	17,000
7/16 (.438)	27,000	0.108	367	23,000
1/2 (.500)	36,000	0.144	490	30,600
		Grade 270		
3/8 (.375)	23,000	0.085	290	19,550
7/16 (.438)	31,000	0.115	390	26,350
1/2 (.500)	41,300	0.153	520	35,100

1 in. = 25.4 mm
1 sq. in. = 645.2 mm^2
1 lb = 4.448 N
1 lb/1000ft = 14.59 N/km

TABLE 12 - PROPERTIES OF SMOOTH BARS

Nominal Bar Diameter, in.	Nominal Steel Area of Bar, Sq.in.	Min. Breaking Strength, kips		Min. Yield Strength at 0.7% Extension, kips	
		Grade 145	Grade 160	Grade 145	Grade 160
1/2	0.196	28	31	25	27
5/8	0.307	45	49	40	43
3/4	0.442	64	71	58	62
7/8	0.601	87	96	78	84
1	0.785	114	126	102	110
1-1/8	0.994	114	159	129	139
1-1/4	1.227	178	196	160	172
1-3/8	1.485	215	238	193	208

Min. Yield Strength (.20% offset): 0.85 f'_s

Approx. modulus of elasticity (based on nominal area): 30,000,000 psi

Elongation in twenty diameters after rupture, minimum: 4.0%

Reduction of area (from measured area) minimum: 20%

Permissible variation bar size is + 0.030 in., -0.01 in. from nominal specified diameter for smooth bars +3%, -2% in weight for deformed bars.

1 in. = 25.4 mm
1 sq. in. = 645.2 mm^2
1 kip = 4.448 kN

TABLE 13 - PROPERTIES OF DEFORMED BARS

Nominal Bar Diameter, in.	Effective Steel Area Sq.in.	Min. Yield Strength of Bar, kips		Min. Yield Strength of Bar at 0.7% Extension, kips		Weight lb/ft.
		Grade 150	Grade 160	Grade 150	Grade 160	
5/8	0.290	42	46	37	39	0.97
3/4	0.433	65	69	55	59	1.53
1	0.854	128	136	109	116	3.00
1-1/4	1.295	194	207	165	176	4.55

Min. Yield Strength (.20% offset): 0.85 f'_s

Approx. modulus of elasticity (based on nominal area): 30,000,000 psi

Elongation in twenty diameters after rupture, minimum: 4.0%

Reduction of area (from measured area), minimum: 20%
specified diameter
for smooth bars +3%, -2% in weight for deformed bars.

1 in. = 25.4 mm
1 sq. in. = 645.2 mm^2
1 kip = 4.448 kN
1 lb/ft = 1.488 kg/m

Allowable Steel Stresses

Stresses permitted in prestressing steel reported in the tenth edition of the AASHO Specification for highway bridges[347] are:

1. Temporary stress before losses due to creep and shrinkage $0.7f'_s$, where f'_s designates breaking stress. Overstressing to $0.80f'_s$ for short periods of time may be permitted provided the stress, after seating of the anchorage, does not exceed $0.70f'_s$.
2. Stress at design load (after losses) $0.60f'_s$ or $0.8f_{sy}$, where f_{sy} designates yield stress, whichever is smaller.

Steel stresses permitted by ACI 318-71[351] are:

1. Due to jacing force $0.8f'_s$ but not greater than the maximum value recommended by the manufacturer of the steel or of the anchorages.
2. Pretensioning tendons immediately after transfer, or posttensioning tendons immediately after anchoring, $0.70f'_s$.

Relaxation of Tendons

Relaxation is the loss of stress in stressed tendons held at constant length. Stussi[321] developed the following equation for determining stress relaxation:

$$f_s = \left[1 - \frac{\log t}{10}\left(\frac{f_{si}}{f_y} - 0.55\right)\right]\left[f_{si}\right]$$

for $f_{si} > 0.55\ f_y$

where f_s = remaining stress at anytime after prestressing,

f_{si} = initial stress,

f_y = 0.1% offset stress,

t = time after prestressing in hours.

For pretensioned specimens, the loss occurring before release should be subtracted from the total loss predicted for the effective stress at release. This can be accomplished by modifying the above equation in the following manner:

$$f_s = \left[1 - \left(\frac{f_{si}}{f_y} - 0.55\right)\left(\frac{\log t_n - \log t_r}{10}\right)\right]\left[f_{si}\right]$$

where t_n = time at which stress is to be estimated,

t_r = time at which tendon is released.

Relaxation loss of prestressing steel was investigated at the University of Illinois.[321] Variables in the relaxation tests were test duration, type of steel, initial stress, stress history (prestretching), and temperature.

The test results indicated that for initial stress ratios less than about one-half, relaxation losses were insignificant. The loss ratio increased at an increasing rate as the initial stress ratio increased. Loss ratios were different for different types of wire. The relaxation rate increased with the initial stress ratio approximately in direct proportion to the total loss expected.

The term prestretching is the operation in which the stress in the wire is increased to a level equal to or higher than the intended initial stress, held at that level for a short period of time, and then anchored at the initial stress. On the basis of test data, prestretching was not significant for post-tensioning when the prestretching period was limited to a matter of minutes. However, in pretensioning where the period of stretching was two days, the difference between the jacking stress and the stress at transfer was about 30,000 psi (207 MPa). In this case, the effect of prestretching is important as 30 percent of the loss might be expected to occur during the first two days.

Tests at Lehigh University[346] concluded that the relaxation loss depends strongly on the initial stress in the strand and that the rate of relaxation loss tends to decrease after approximately 100 days.

A report on stress relaxation of high-tensile steel by Mihajlav[342] showed that the relationship between relaxation and time consisted of two parts. One part was a short period during which the stress falls sharply at a rapidly decelerating rate. The second part was a period of unlimited duration during which the rate of relaxation was gradually damped or stabilized.

Fatigue Properties

Good performance of an internally prestressed pavement depends on maintaining the desired level of tensile stress in the prestressing steel. Prestressing strands are generally stressed to greater than 70 percent of yield strength. Thus, additional stresses in bending and stresses due to slab elongation may raise the stress to a critical level.

Fatigue properties of wire or bars are determined either from direct oscillating tension or from bending and vibration tests. The fatigue properties of 7/16-in. (11 mm) diameter seven-wire prestressing strand were studied by Warner and Hulsbos[324] in an experimental investigation involving static tests, constant cycle fatigue tests, and cumulative damage tests. The constant cycle fatigue tests were conducted with two minimum stress levels. The stress range was varied from 15 to 40 percent of breaking stress for a minimum stress level of 40 percent. However, the stress range was varied from 15 to 27 percent of breaking stress for a minimum stress level of 60 percent. For these stress combinations, fatigue lives varying between 50,000 and 5 million cycles were obtained.

To fabricate a test specimen, 20-ft (6 m) length of strand was tensioned to 70 percent of the breaking stress and the element of the gripping devices were assembled around it. The stress was then released to 45 percent of the breaking stress. A stiff sand cement-water grout was then packed around the strand and the transverse tension bolts. After the grout cured, the specimens were tested in fatigue.

Results from tests by Warner and Hulsbos[324] are shown in Figure 56. The data indicated that a maximum allowable stress range of 10 percent should be used if a minimum stress level of 60 percent of the breaking stress is maintained. In cases of lower minimum stress levels, the maximum allowable tendon stress is also reduced.

Lane and Ekberg[310] determined the fatigue strength for 7/16-in. (11-mm) and 3/8-in. (9.5-mm) diameter seven-wire uncoated stress-relieved prestressing strands. Data defining stress versus number of load cycles to fatigue failure are shown in Fig. 57. Curves are drawn through the data for tests on 7/16-in. (11 mm) strand tested at two minimum stress levels and varying stress ranges. Based on test results, stress ranges of 12.8 percent and 13.4 percent of breaking stress were recommended respectively for the minimum stress levels of 65.2 and 55.6 percent of breaking stress.

Laboratory tests[358] have shown that fatigue life of prestressing steel dpends on free length of strand, lateral pressure, abrasion, frequency of loading, and flaws in the steel. The reduction of fatigue life with strand length is shown in Fig. 58 for 1/2-in. (12.7-mm) diameter-seven wire prestressing strand. This was observed for the strand length tested in air and also for the strand length tested with lateral pressure. Lateral pressure was provided by concrete blocks cast on the strand. The concrete block extended over 1.5 in. (38 mm) of the wire.

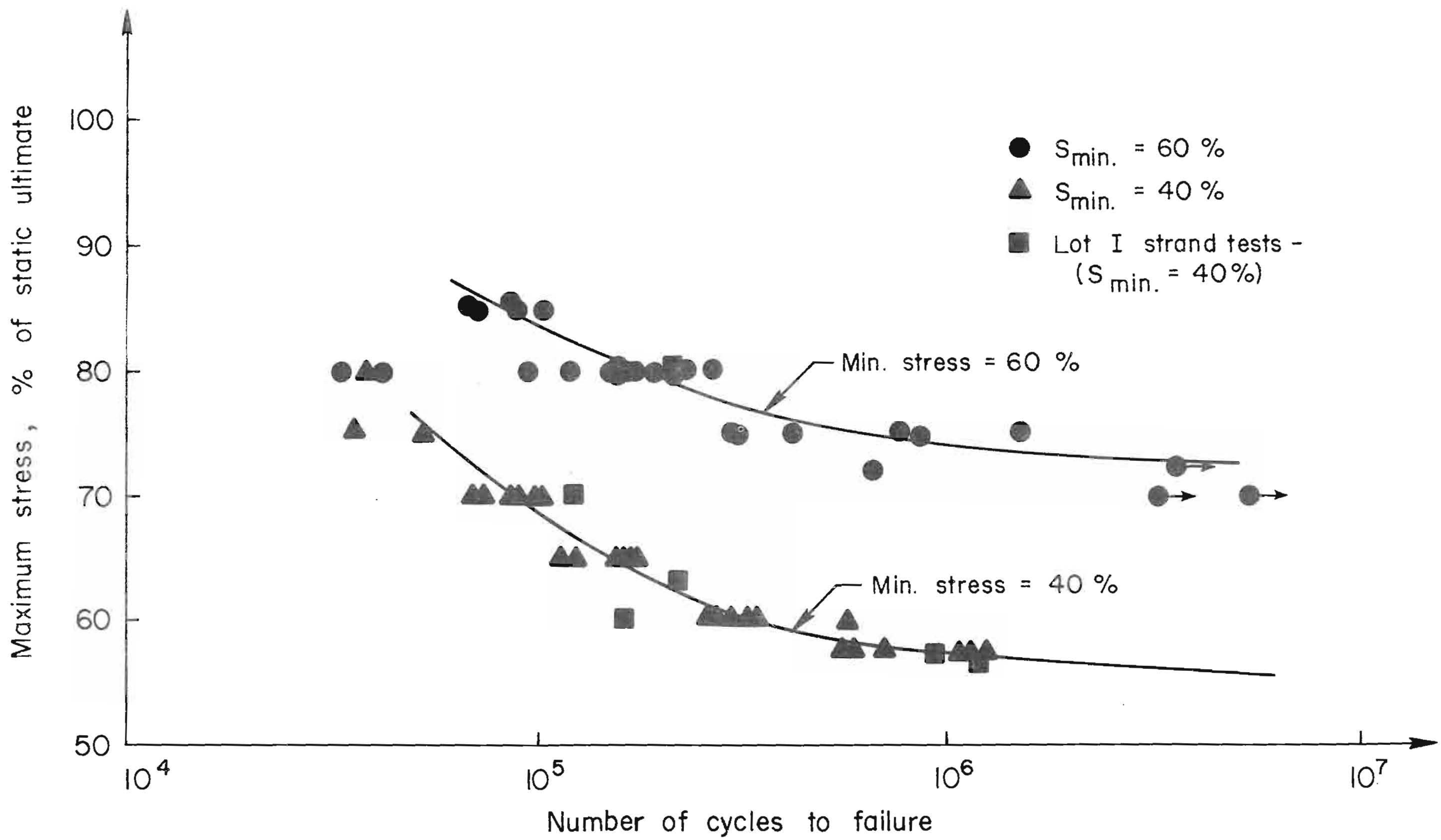


FIG. 56 - EFFECT OF STRESS LEVEL ON FATIGUE LIFE

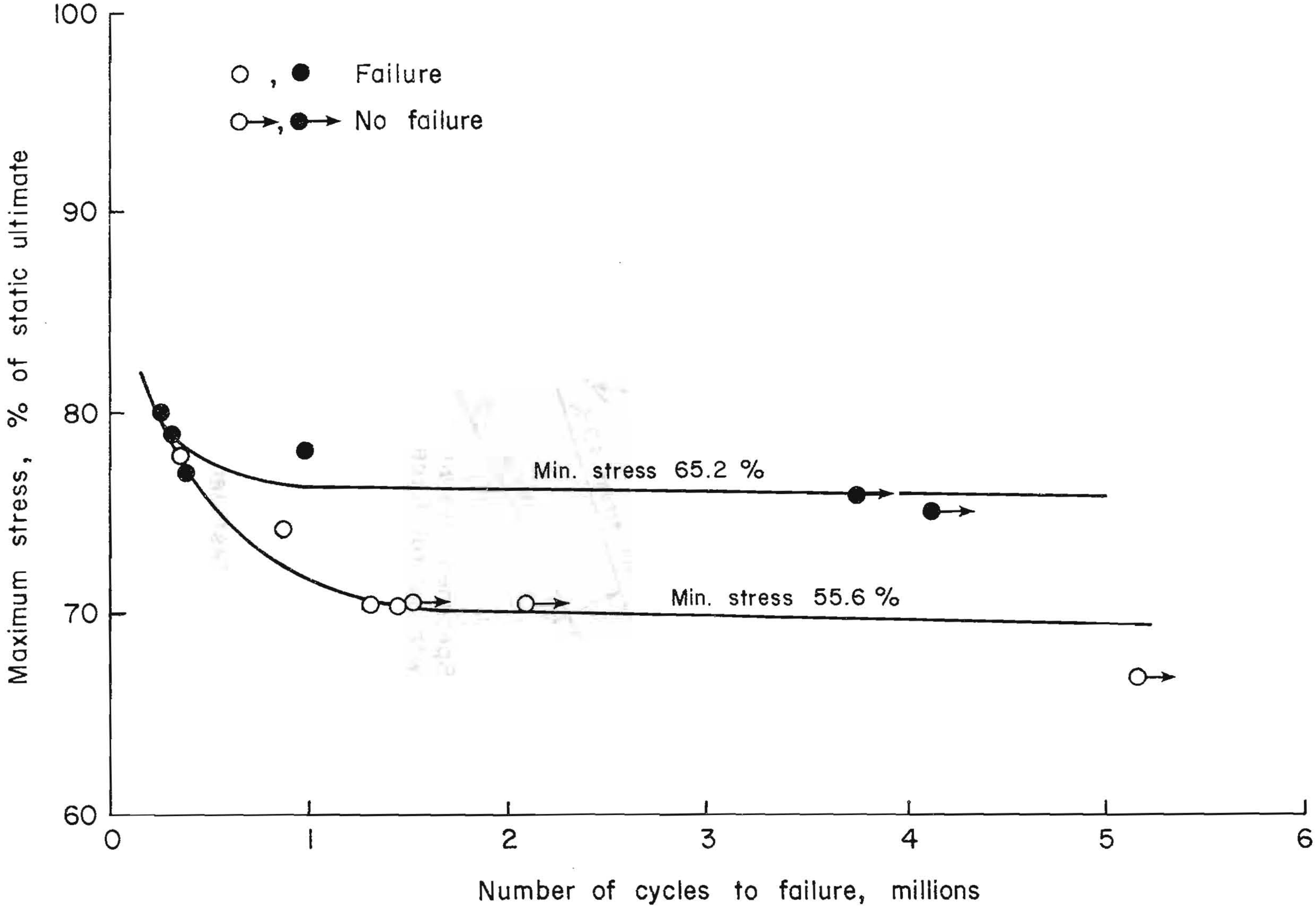


FIG. 57 - EFFECT OF STRESS LEVEL ON FATIGUE LIFE

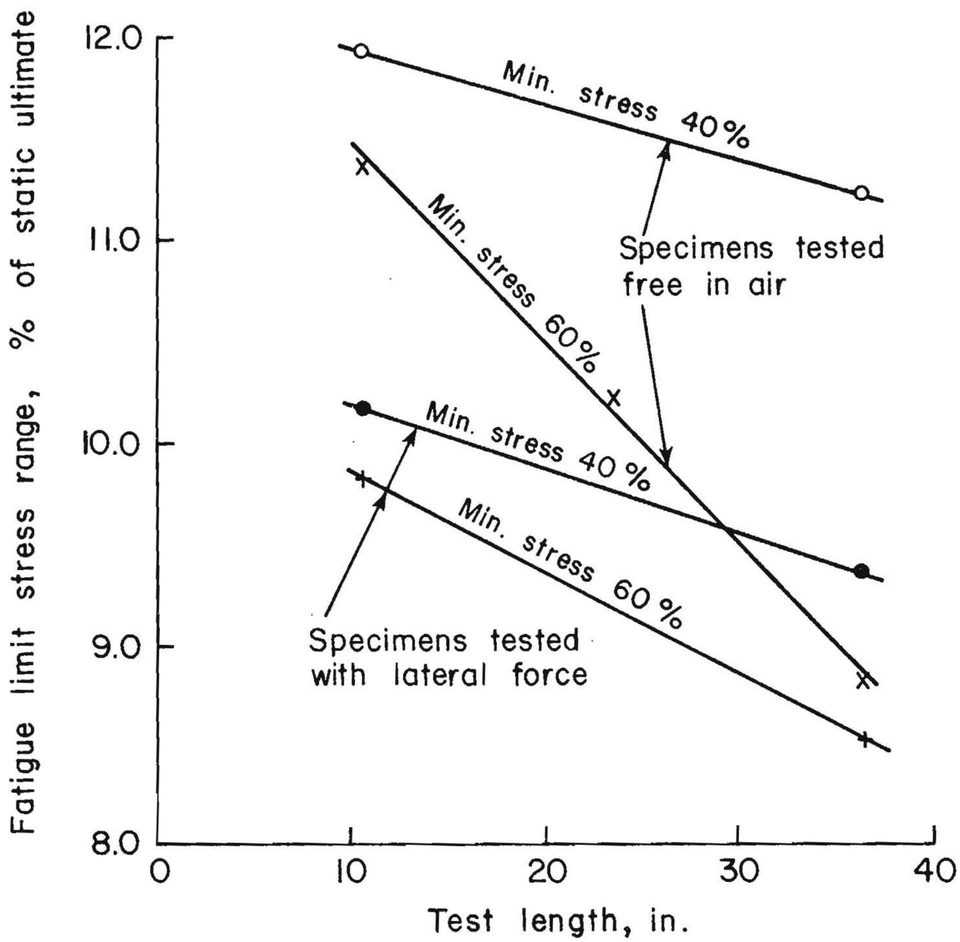


FIG. 58 - EFFECT OF STRAND LENGTH ON FATIGUE STRESS

Subbase Materials

Granular subbases are used under conventional concrete pavements to: (1) control mud pumping, (2) reduce frost action, (3) reduce shrink and swell of subgrade, and (4) expedite construction. Granular subbases also lend some structural capacity to the pavement.

Experience with conventional concrete pavements has shown that subbase pumping and the associated loss of foundation support increases as pavement thickness decreases. Therefore, prestressed concrete pavements with lesser thickness and greater deflection under traffic loads, will require subbases of higher quality than those needed for conventional pavements. These high quality subbases will also reduce pavement stresses thereby increasing the ability of the pavement to withstand repetitive loading.

Prestressed concrete pavements have been built in Europe and the United States on granular, bituminous-treated, and cement-treated subbases. Other materials with good strength and insulating properties have been used successfully in Europe. These materials are particularly suitable for prestressed concrete pavement subbases.

Untreated Granular Subbases

Granular subbases may consist of gravel, sand, or crushed stone. The choice of type of material normally depends on the economics of the area. For good performance, however, proper gradation and compaction is necessary. Granular materials are subjected to consolidation from the action of traffic after the pavements are placed in service. To prevent detrimental consolidation, subbases must be compacted to high densities. The results of laboratory tests[306] on subbases subjected to simulated truck loads are illustrated in Fig. 59. These data show that as few as 50,000 load repetitions can produce excessive consolidation

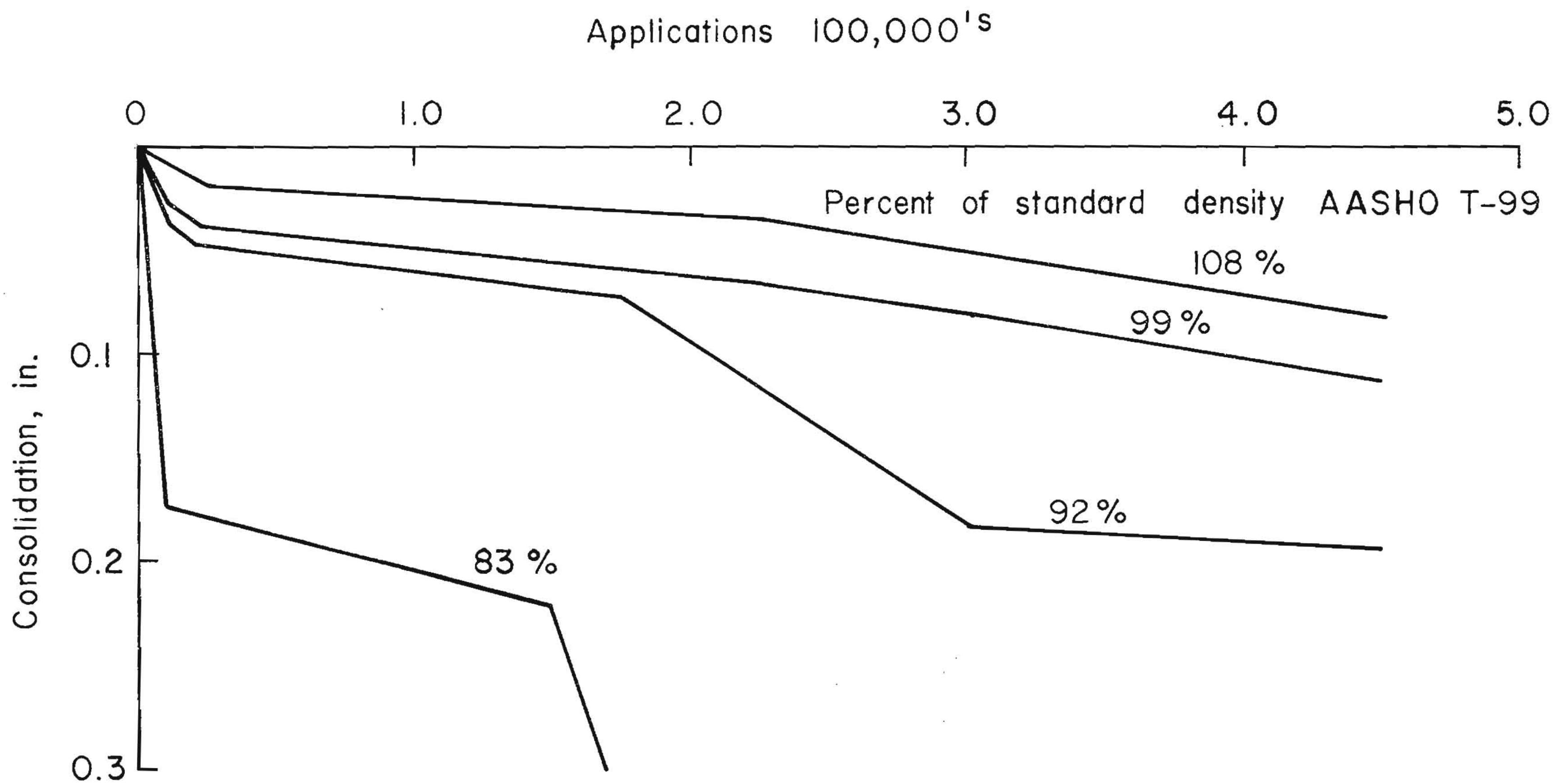


FIG. 59 - CONSOLIDATION OF SAND-GRAVEL SUBBASES UNDER REPETITIVE LOADING

when densities were low. The results also indicate that 100 percent of AASHO T99 density is the minimum density necessary to prevent detrimental consolidation.

Consolidation of granular subbases under traffic is a matter of concern in another respect. As the thickness of the subbase is increased, the consolidation from repetitive loads will produce even greater total amounts of settlement. Fig. 60 illustrates the results of repetitive load tests on 4-, 6-, and 12-in. (102-, 152-, and 305-mm) thick subbases placed on a clay-loam subgrade and compacted to 100 percent AASHO T99 density. The results show that after 450,000 load repetitions, the consolidation of the 12-in. (305-mm) thick subbase is twice as much that of the 4- or 6-in. (102- or 152-mm) subbase. The least amount of combined subgrade-subbase consolidation occurrs on the 4-in. (102-mm) subbase. Therefore, where subbase depths are increased beyond 4 to 6 in. (102 to 152 mm), there could be an increased risk of poor pavement performance due to subbase consolidation.

The effect of subbase on pavement strength can be expressed in terms of modulus of subgrade reaction "k". Test results on the effect of subbase thickness on the modulus of subgrade reaction are shown in Fig. 61. These results show that only small increases in the modulus of subgrade reaction can be obtained by using granular subbases.

Cement-Treated Subbases

Cement-treated subbase is a compacted mixture of pulverized soil, portland cement, and water. The factors that affect the physical properties of cement-treated subbases include soil type, quantity of cement, degree of mixing, time of curing, and dry density of the compacted mixture.

Unlike untreated granular subbase, the consolidation of cement treated subbases due to repeated traffic loads is very small. Laboratory tests[306] have shown that

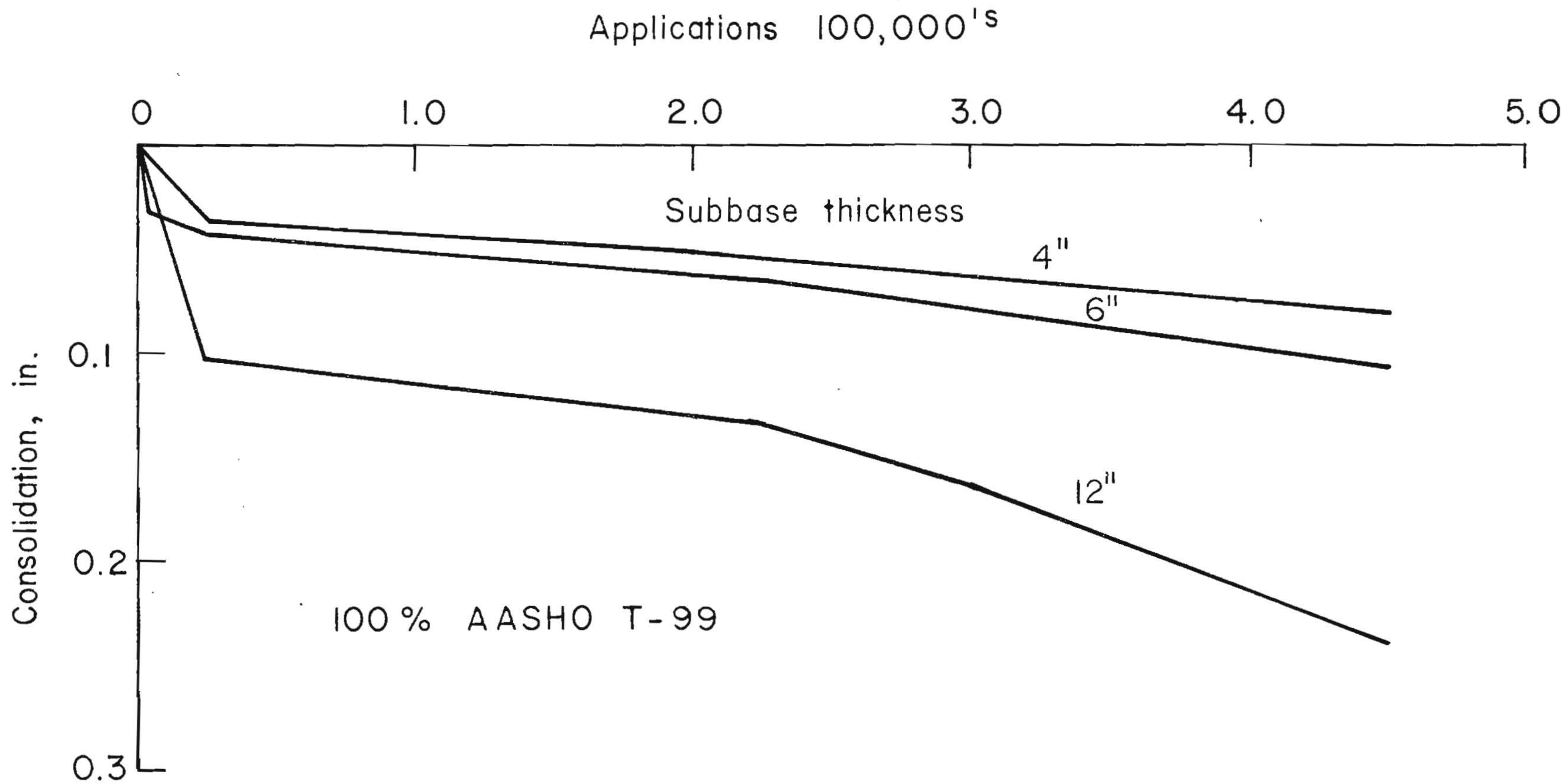


FIG. 60 - INFLUENCE OF SUBBASE THICKNESS ON CONSOLIDATION

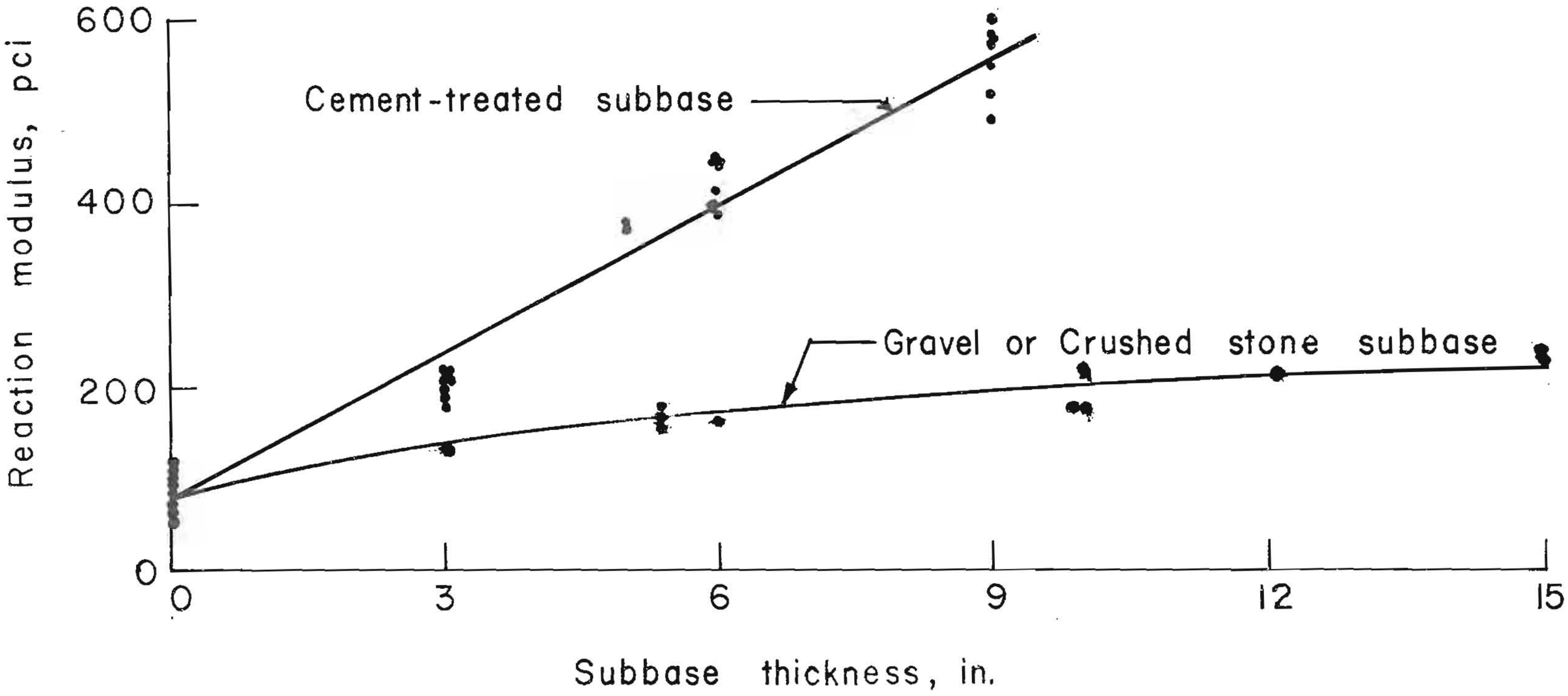


FIG. 61 - EFFECT OF SUBBASE THICKNESS AND MATERIAL ON k

no measurable consolidation of cement-treated subbase occurred after one million load repetitions.

Cement-treated subbases increase the supporting capacity of pavement. Test results on the effect of subbase thickness on the modulus of subgrade reaction are shown in Fig. 62. These results indicate that cement-treated subbases produce high "k" values, that result in reduced stresses in the concrete slabs. This was established from measurement on full-size slabs built on subgrades and subbases with known k values[334]. Fig. 62 shows both the strains measured in slabs under a 9,000 lb (40 kN) load and the "k" values computed from these data. The computed values were in close agreement with those determined from load-bearing tests.

The influence of subbase material on subgrade pressures was determined from laboratory tests[317]. These measurements were made for load placements at the interior, edge, and free corner. Fig. 63 shows test results for soil-cement and granular subbases. These results show that the pressures under soil-cement subbases are less than those under granular subbases. This difference is especially significant for loads near pavement edges and corners where pressures under soil-cement subbases were measured at approximately 46 percent of those under granular subbases of the same thickness.

Strength and elastic properties of soil-cement mixtures have been determined from laboratory tests[309]. The results showed that the modulus of rupture, compressive strength, and modulus of elasticity of soil-cement mixtures vary depending upon soil type, cement content, age, and type of curing. However, in all cases, the test values increased as cement content was increased, and as the time of moist curing was increased. The soil type, in particular, had a major influence on test values. For example, the modulus of rupture was 300 and 150 psi (2.1 and 1.0 MPa) respectively for a sandy soil and silt loam with the same cement content. Also, the modulus of elasticity was 3.5 million and 1.0

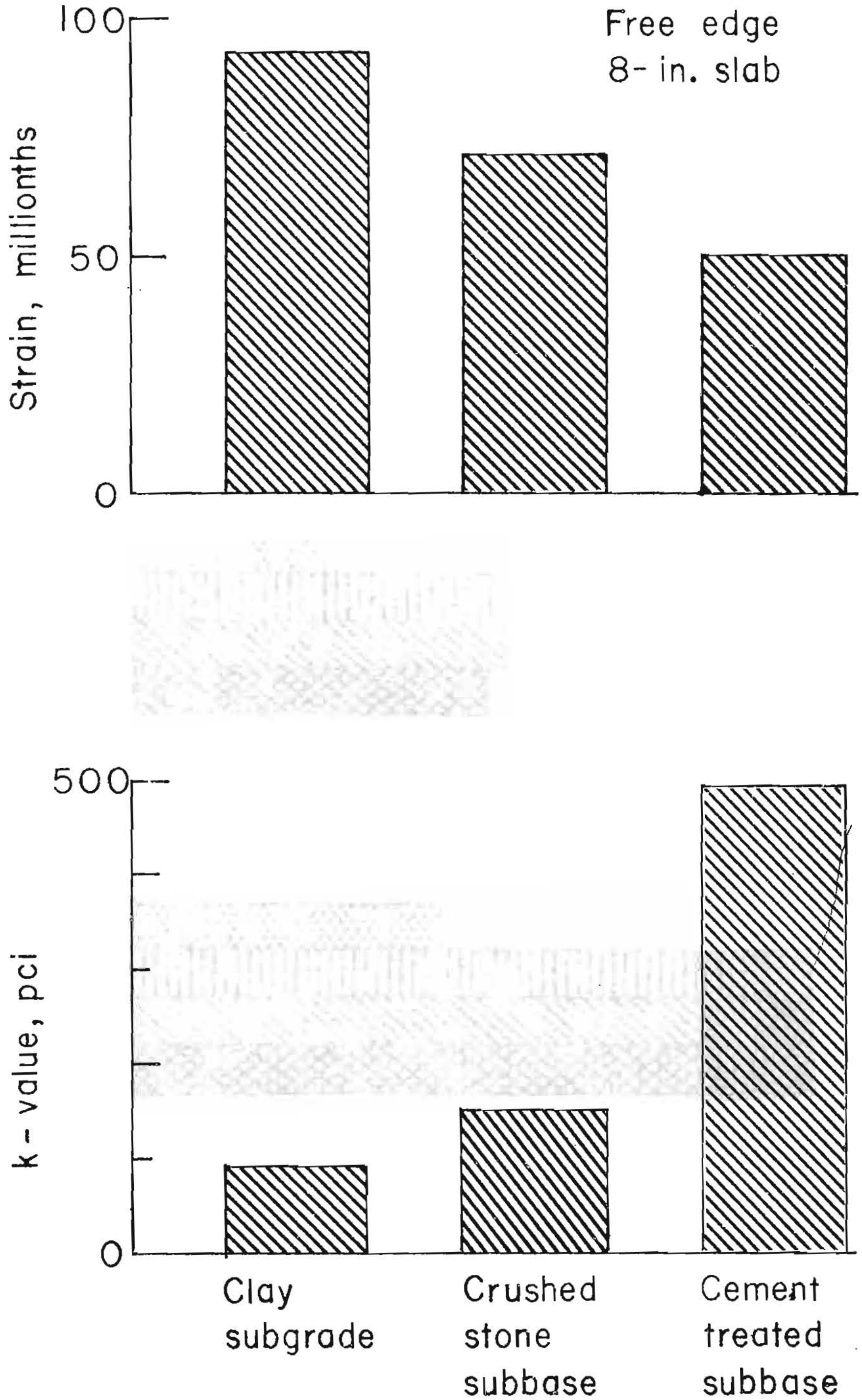


FIG. 62 - MEASURED STRAINS AND COMPUTED k VALUES FOR 9-KIP PLATE LOAD

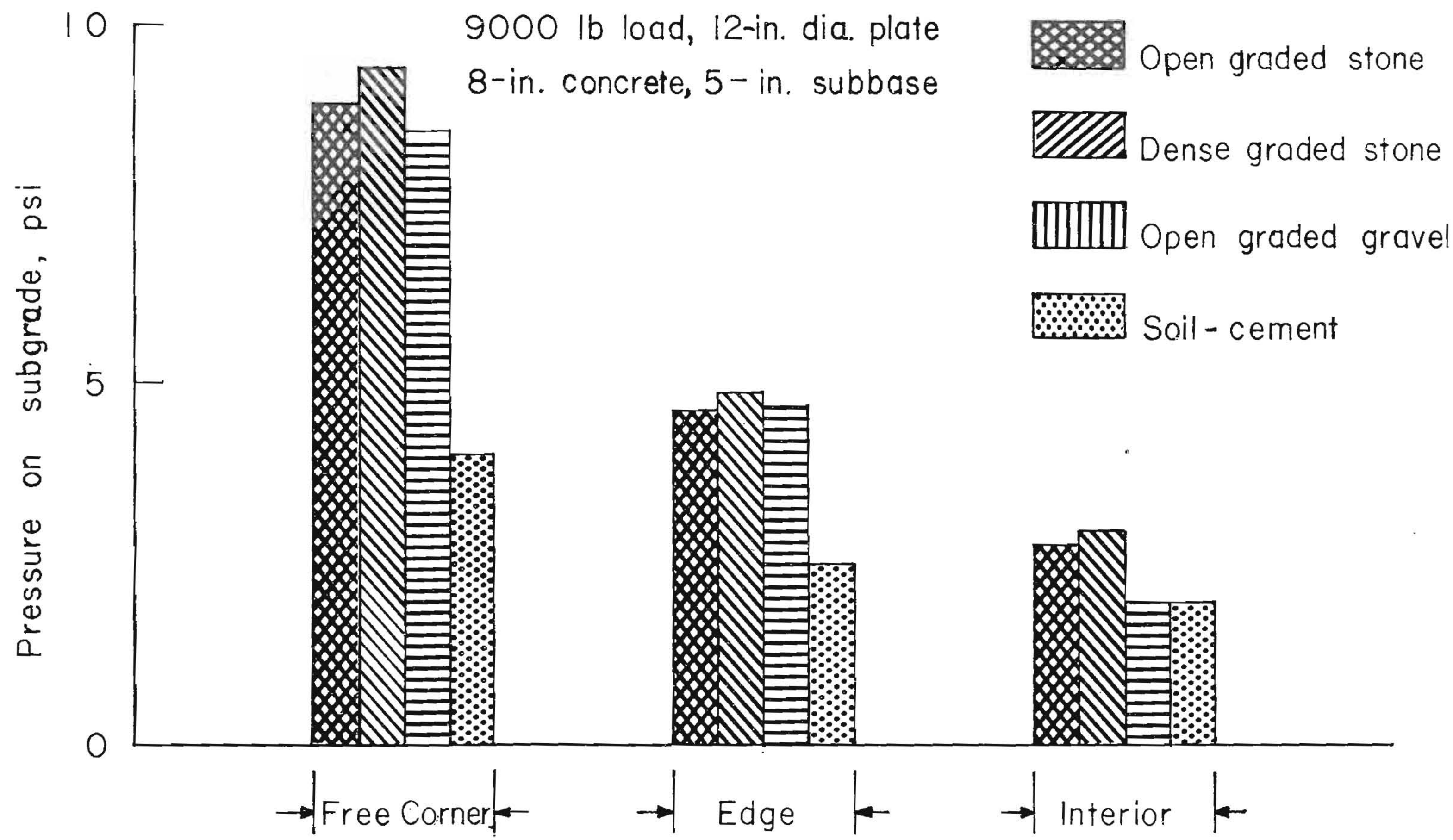


FIG. 63 - EFFECT OF SUBBASE MATERIAL AND LOAD LOCATION ON SUBGRADE PRESSURE

million psi (24 million and 6.9 million kPa), respectively for a sandy soil and a silty loam soil with the same cement content.

Asphalt-Treated Subbases

Stabilization of soil with bituminous materials is satisfactory for course-grained and granular soil, but its use for stabilizing plastic soils is limited. In contrast to cement-treated subbases, properties of asphalt-treated materials depend on temperature. For example, at higher temperatures deformation increases and permanent deformation occurs. On the other hand, risk of cracking increases at low or moderate temperatures.

The strength of asphalt-treated materials can be expressed in terms of bending tensile strength and the strain at break. These properties are influenced by temperature. Results of tests on gravel sand treated with asphalt[315] have shown the following data for dynamic modulus of elasticity, bending tensile strength, and breaking strain at different temperatures:

Temp. (°F)	Dynamic Modulus (psi)	Tensile Strength (psi)	Breaking Strain Millionths
26.5	1,770,000	1070	1100
32.0	1,210,000	1280	1400
37.5	710,000	1420	2700
43.1	326,000	1210	9000
48.6	142,000	920	13000

1 psi = 6.895 kPa

Other Materials

Expanded polystyrene concrete, a light weight concrete made from expended polystyrene beads, cement, sand, water, and admixtures has been used for pavement subbases with success in Europe[345]. The strength and elastic properties of this material have been investigated in the laboratory by the Portland Cement Association. The results have shown that the properties of expanded polystyrene concrete depend primarily on density. Test results have shown that the modulus of rupture of expanded polystyrene concrete ranges from 120 to 150 psi (827 to 1034 kPa). Similarly, the compressive strength ranges from 325 to 550 psi (2.2 to 3.8 MPa). The modulus of elasticity in flexure varies from 100,000 to 160,000 psi (689.5 to 1103.2 MPa).

Laboratory tests on the effect of repeated loads on the permanent deformation of expanded polystyrene concrete were also conducted by the Portland Cement Association. Test results have shown that the consolidation due to repeated loads is small. It amounted to 280 to 580 millionths strain after two million load cycles.

The effect of expanded polystyrene concrete subbase on the modulus of subgrade reaction was also investigated. Test results have shown that the "k" value increases as the thickness increases. However, the increase is less than that obtained by cement-treated subbases of the same thickness.

Temperature measurements in the subbase and subgrade of pavement test sections have shown that the use of expanded polystyrene concrete for pavement subbases provides insulation of subgrade. When proper depth is used, expanded polystyrene concrete protects the subgrade from freezing.

Performance of Subbases under Prestressed Pavements

Sufficient performance data are not available to evaluate the effect of subbase type on prestressed pavement designs. However, irrespective of the foundation used, drainage has been considered of prime importance[281].

Load tests have been conducted on many prestressed highway and airport pavements. In the following, some tests are described, and results pertinent to subbase behavior are outlined.

Static and creep speed load tests were conducted on the 5-in. (127-mm) thick post-tensioned slab constructed in Pittsburgh, Pa. in 1956[146]. Static edge deflections under a 16-kip (71.2-kN) wheel load were 0.03 in. (0.76 mm) with the load applied away from the edge and 0.04 in. (1.02 mm) with the load at the edge. The slab was tested with 580,000 applications of a 28-kip (125-kN) axle load with the nearest wheel centered 18 in. (457 mm) from the edge. For these tests, edge deflection was 0.03 in. (0.76 mm). At the transverse joint over a sleeper slab, deflection increased to 0.075 in. (1.91 mm) due to subsidence caused by pumping. This slab was placed on 6-in. (152-mm) of granular fill over 3-ft (0.9-m) of prepared subgrade compacted to 95 percent of maximum AASHO T-99 density. No drainage was provided at the joint.

Deflection data are available for a 6-in. (152-mm) post-tensioned slab built on 6-in. (152mm) CTB at Dulles International Airport, Va.[281]. Edge deflections under a 20-kip (89-kN) axle with the wheel centered 20-in. (508-mm) away were 0.016 in. (0.406 mm) at creep speed. Corner deflection at a doweled joint under a 34-kip (151-kN) axle, increased from 0.012 in. to 0.035 in. (0.305 to 0.889 mm) as wheel placement was changed from 36 in. (914 mm), normal tracking, to 16 in. (406 mm) away from edge. For a 34-kip (151-kN) axle stationary on one side of the joint, in normal tracking position, the rebound deflection at the corner was 0.03 in. (0.76 mm) and 0.04 in. (1.02 mm) at wheel location. These deflection tests were made before traffic had been allowed on the slab.

Three prestressed slabs placed on sand clay loam soil with 140-pci (38 MPa/m) modulus of subgrade reaction were tested at the Portland Cement Association Laboratories[240]. These were 5 in. (127 mm) thick with 180 and 360 psi (1.2 and 2.5 MPa) longitudinal prestress, and 70 psi (483 kPa) transverse prestress in one slab. Simulated dual-tire wheel loads were applied to determine cracking loads. Bottom cracking occurred at the edge under a 20-kip (89-kN) wheel load at 0.08-in. (2.03-mm) deflection. The first top crack occurred at some distance from the load at a 45 to 55 kip (200 to 245 kN) load. The deflection under this load was 0.3 in. (7.6 mm) with 0.1-in. (2.54-mm) permanent deflection.

A bottom crack under a wheel centered on the 12-ft (3.7-m) wide slab appeared at 35 kips (156 kN), and 0.07-in. (1.78-mm) deflection. Top surface cracks were evident at 70 kips (311 kN) load and 0.5-in. (12.7-mm) deflection in the two slabs without transverse prestress. No visible cracks were found after a load of 100 kips (445 kN) was applied twice on the slab with transverse prestress.

Load tests were also conducted on prestressed airport pavements. The 7-in. (180-mm) thick fixed-length prestressed pavement at Orly[105], placed on 8-in. (200-mm) compacted gravel or crushed stone, was load tested. Load repetitions included 1,000 cycles of 220 kip (979 kN) loads. Loads were applied through a 32-in. (810-mm) diameter plate. The deflections under the loads, away from an edge or crack, were reported to vary from 0.055 to 0.060 in. (1.4 to 1.5 mm) for 101 kip (449 kN), 0.075 to 0.087 in. (1.9 to 2.2 mm) for 134 kip (596 kN), and 0.13 to 0.19 in. (3.3 to 4.8 mm) for 224 kip (996 kN) loads. Permanent deflections amounted to 0.025 in. (0.635 mm), 0.042 in. (1.067 mm), and 0.053 in. (1.346 mm) after respectively 1,000, 3,000, and 3,700 load applications.

Tests were also conducted on the 7-in. (178-mm) prestressed pavement at Patuxent River NAS[96]. This pavement was placed on 18 in. (457 mm) of sand-clay base, compacted to over 95 percent of modified Proctor maximum density. The load was applied through circular plates. Tests were made at different ages ranging from one to 20 months. Edge load deflections for a 60 kip (267 kN) load on an 8-in. (203-mm) plate centered 6 and 15 in. (152 and 381 mm) from the edge amounted to 0.11 and 0.09 in. (2.79 and 2.29 mm), respectively. The deflections under an interior load on a 20-in. (508-mm) plate were 0.03, 0.06, and 0.15 in. (0.76, 1.52, and 3.81 mm) for 50, 100, and 200 kip (222, 445, and 890 kN) loads. Application of the 200 kip (890 kN) load 12 and 18 times during spring conditions, increased the deflection under load to 0.29 in. (7.37 mm). An interior load of 200 kips (890 kN) applied on a 20-in. (508-mm) diameter plate at a transverse crack deflected the slab 0.25 and 0.245 in. (6.35 and 6.223 mm) on both sides of the crack. Before prestressing the deflections for a 180 kip (801 kN) load amounted to 0.27 in. (6.86 mm) and 0.15 in. (3.81 mm) on the sides of the crack.

Load tests were conducted on the 9-in. (229-mm) thick taxiway at Biggs AFB[202]. This slab was placed on 6-in. (152-mm) of crushed stone compacted to 100% of maximum AASHO T-99 density, and well compacted subgrade. A 204 kip (907 kN) test load on four tires in line, spaced at 37, 62, and 37 in. (940, 1575, and 940 mm) deflected the slab 0.083 in. (2.108 mm) when centered on the 25-ft (7.6-m) wide slab. Centered over the construction joint with 102 kip (454 kN) on each slab, and the nearest wheel 31 in. (787 mm) away, the maximum deflection at the joint was 0.05 in. (1.27 mm). The modulus of subgrade reaction indicated by these tests was 100 pci (27.2 MPa/m).

Load tests have shown high load supporting capacities for prestressed pavements, without slab failures at high loads. However progressive deformations occurred in the foundations. For highway pavements, with load repetitions considerably larger than those included in the load tests, slab deflections and subbase deformation may be the limiting design criterion. This suggests the need for a quality subbase material under prestressed pavements so that pavement deformation can be reduced to acceptable limits.

French requirements for design of airport pavements specify a maximum corner deflection of 0.2 in. (5 mm) for the governing load. British requirements are based on a slab settlement of 0.2 in. (5 mm) for 10,000 passages of the same load. In view of the frequency of traffic on highway pavements, deflection of prestressed highway slabs under the design axles would have to be much smaller to prevent objectionable settlements during the life of the pavement.

Friction-Reducing Mediums

Friction exists at the interface between a concrete slab and the underlying subbase. The friction restrains movements of the slab. This restraint is detrimental to the performance and construction of a prestressed pavement. Performance is influenced because restraint during periods of drying shrinkage and temperature contraction of the slab induces tensile stresses in the concrete. These stresses contribute to cracking. During construction, subbase restraint prevents obtaining an even distribution of prestress throughout the length of the pavement. It has been estimated that a 50 percent reduction in friction could result in a 30 to 40 percent savings in prestress loss[246]. As the magnitude of prestress influences the cost of the pavement in terms of prestressing steel, labor, etc., an economical friction-reducing medium is important.

Numerous friction reducing materials have been tested both in the laboratory and in the field. These materials included sand building paper, plastic sheeting, bitumem, and others both single or in combination. Results showed that the coefficients of static friction were greater than the coefficients of sliding friction. The magnitudes of these coefficients were neither affected by the direction of movement nor by the slab size or weight[271]. However, they sometimes depended on the texture of the subbase surface[271]. Results also show that all materials require a certain amount of slab movement to overcome the initial restraint [215,246,271].

Several materials have provided friction coefficients less than one. Range of coefficient of friction of these materials is shown in Table 14. Each of these materials is discussed in more detail in following sections of the report. Table 15 summarizes friction coefficient data gathered in this study for different materials.

Sand Layer

The coefficients of static and kinetic friction of sand under a pavement ranged from approximately 0.49 to 1.03 [215,246,96,271,201]. In general, the greater the static coefficient, the larger the difference between the static and kinetic coefficients. For a static coefficient of 1.0 to 1.5, the difference was about 1.0. For a static coefficient below 1.0, the difference was about 30 percent[96,246]. The value of the coefficient was influenced by the following conditions:

1. Surface texture of slab and subbase[201].
2. Moisture content of the sand[201].
3. Thickness of the sand layer[271].
4. Construction conditions[96,201].

However, the coefficient was neither influenced by the rate of application of the thrusting forces nor by the rate of slab movement[215,246].

Several comments were made regarding the care that must exercised in placing concrete on a sand layer to avoid displacing the sand and obtaining a bond between the slab and subbase[215,198,94]. Also mentioned was the possibility of a pumping in a sand layer[246].

Polyethylene Sheets

The coefficient of friction when two polyethylene sheets were placed between the subbase and the pavement

TABLE 14 - COEFFICIENT OF FRICTION FOR SELECTED MATERIALS

Material	Coefficient of Friction
Sand	0.49 to 1.03
Polyethylene sheets	0.3 to 0.8
Polyethylene sheet over sand layer	0.55 to 0.8
Bitumen	Depends on Condition
Oil	0.33 to 0.49

TABLE 15 - SUMMARY OF FRICTION-REDUCING MEDIUMS

Conditions	Coefficient of Friction		Source
	Static	Kinetic	
Sand and aggregates-			
Sharp sand A	0.74	0.62	
Sharp sand B	-	0.69	
Dune sand	0.78	0.69	
Gravel	0.75	0.75	
Limestone chippings	-	0.56	
(The above results are from 1 in. thick layers covered with concreting paper before casting the concrete)			
Sharp sand B (from 1 in. thick layer covered with concreting paper)	-	0.64	
Sharp sand B (from 1 in. thick layer and concrete cast directly on the sand)	1.05	0.66	
Smooth mortar base	0.49	0.38	
Waterproof paper on smooth mortar base	0.90	0.65	(215)
Hessian-backed paper on smooth mortar base	0.74	0.60	
Polythene-			
Polythene on sand (slab placed, not cast, on to polythene. Sliding between concrete and plythene)	-	0.43	
Polythene on sand (slab cast on to polythene. Sliding between polythene and sand)	-	0.55	
Polythene on smooth mortar base*			
Paraffin wax on smooth mortar base	1.11	0.17 to 0.34	
High-pressure lubricating oil (cardium compound D) on smooth mortar base	0.37	0.33 to 0.49	
Asphalts-			
½ in. thick asphalt composed of 6% of Shelphalt by weight to Thames Valley sand	0.86	0.3 to 0.6	
Ditto, but with 10% of Shelphalt by weight	0.64	**	

* Impracticable due to rapid wear

** Similar results to previous asphalt

TABLE 15 - SUMMARY OF FRICTION-REDUCING MEDIUMS (CONTINUED)

Conditions	Coefficient of Friction		Source
	Static	Kinetic	
Sheet asphalt	3.2	-	
Emulsified asphalt	2.5	1.3	
Plastic soil	2.1	1.3	
Blend washed sand and gravel	2.0	1.3	(246)[a]
Granular subbase	1.7	0.90	
Sand layer	1.0	0.75	
Polyethylene sheeting	0.9	0.5	
Polyethylene sheeting under			
400-ft. slab	0.67		
500-ft. slab	0.53		
600-ft. slab	0.44		(281)[b]
400-ft. slab	0.66		
500-ft. slab	0.48		
600-ft. slab	0.46		(281)[c]
One-inch sand covered with one sheet of building paper	0.65	0.60	
Two sheets of building paper	0.94	0.52	(96)[e]
Two sheets of copper-clad Sisalkaft paper with soapstone in between	0.757	0.625	
Slab placed directly on prepared base	2.45	1.105	
Cement-stabilized subbase	1.4	1.1	
Sand	1.05	.93	
Tarred-gravel subbase	1.35	.95	(231)[b]
Cement-stabilized subbase	1.4	-	
Sand	-	-	
Tarred-gravel subbase	1.25	-	(231)[d]

(a) All under 5-in. thick slabs
(b) Field tests in February
(c) Field tests in March
(d) Field test in July
(e) Values are the average of 3 movement measurements

TABLE 15 - SUMMARY OF FRICTION-REDUCING MEDIUMS (CONTINUED)

Conditions: Friction-Reducer	Conditions: Subbase Texture	Coefficient of Friction: Static	Coefficient of Friction: Kinetic	Source
¼ in. sand + polyethylene	Medium	0.56	0.56	
¼ in. sand	Smooth	0.55	0.55	
	Medium	0.59	0.59	
	Rough	0.53	0.52	
1/16 in. sand layer + polyehtylene	Smooth	0.81	0.61	
	Medium	0.72	0.65	
	Rough	0.80	0.71	(271)
Double polyethylene	Smooth	0.65	0.62	
	Medium	0.72	0.66	
	Rough	0.79	0.71	
1/16 in. sand layer	Smooth	13.83	0.94	
	Medium	44.47	1.10	
	Rough	51.15	1.23	
No friction reducer (CTB coated with cu ing compound	Smooth	>13.5*		
	Medium	>44.0*		
	Rough	>51.0*		
No friction reducer (CTB uncoated)	Medium	> 8.0*		

CTB = bituminous-coated cement-treated subbase
* Values are based on the maximum load with the testing equipment.

ranged from 0.3 to 0.8[246,198,271,281]. The magnitude of the coefficient depended on the surface texture of the subbase[271]. With special additives in the polythene, the coefficient of friction was reduced to as low as 0.13[215].

Some concern was expressed that a single layer of plastic sheet might be ineffective in the field[215].

Polyethylene Sheet and Sand Layer

A polyethlene sheet placed over a 1/4-in. (6.4-mm) thick sand layer had a coefficient of friction ranging from 0.55 to 0.6 depending on the surface texture of the subbase[271]. For a 1/16 in. (1.6 mm) thick sand layer the range was 0.6 to 0.8[271].

Bitumen

The effectiveness of bitumen as a friction-reducing medium is primarily due to its viscous shear behavior[215], which depends on:

1. Grade of Bitumen - the higher the grade, the easier the slab movement.

2. Temperature of the bitumen - the restraint value increases as the temperature decreases, roughly doubling for each fall of 5.4°F (3°C). A 100-penetration bitumen can be used only when the temperature is about 41°F (5°C), or the movement of the slab end is greater than 0.1 in. (2.5 mm).

3. Thickness of the bitumen layer - the thicker the layer the easier to move the slab.

4. Rate of slab movement - the force of restraint is directly proportional to the rate of slab movement.

It was noted that in using bitumen, seals, usually consisting of thin metallic strips, must be provided at the slab edges.

Oil

Friction coefficients for a high-pressure lubrication oil on smooth mortar base ranged from 0.33 to 0.49[215]. Test results showed that oil was quickly squeezed out, leaving only a film.

APPENDIX A - BIBLIOGRAPHY

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APPENDIX B - GLOSSARY OF TERMS
RELATED TO PRESTRESSED CONCRETE PAVEMENTS

Abutments - External reactions used in pretensioning tendons and poststressing extenally prestressed pavements.

Anchorage - A mechanical device or means employed to attach the ends of the tendons to the concrete.

Blowup - A pavement failure due to high eccentric compressive stresses that force the slab upward from its at grade position. This is generally associated with rapid stress increase due to increase in concrete temperature.

Coefficient of Subgrade Reaction - See Modulus of Subgrade Reaction.

Conduit - A cover for encasing tendons in a post-tensioned concrete pavement. Generally flexible or rigid tubes, or pipes.

Consolidation (concrete) - A method of compacting plastic concrete by rodding or vibration.

Consolidation (soils) - The gradual reduction of a fine grained soil mass under sustained load due to squeezing out of water from the void spaces. The load is initially carried by the pore water and is gradually transferred to the soil structure.

Contraction of Prestressed Pavement - Shortening of the length of pavement slab due to drying shrinkage, temperature decrease, elastic shortening, or creep.

Creep - A contraction or shortening of the pavement under the sustained horizontal compressive stress.

Curling - Pavement profile change due to temperature gradient.

Dowel - A load transfer device embedded in a concrete pavement to provide shear or load transfer across a joint. Round Steel bars are generally used. One end of the bar is not bonded to allow slab end movements.

Drainage - The control of water accumulation in pavement subbases and subgrades.

Drying Shrinkage - Volumetric contraction of the concrete due to drying or loss of free water.

Eccentricity - Distance from the mid-depth of the slab to the center of the tendons.

Elastic Shortening - A reduction of prestressed concrete pavement length when compressive stress is applied.

Externally Prestressed Concrete Pavement - A concrete pavement that requires an external reaction system to preserve horizontal compressive stress in the pavement slab.

Final Prestress - The horizontal compressive stress imposed on the concrete after substantial strength gain. This is generally applied after a specified compressive strength has been attained.

Flexural Strength - See Modulus of Rupture.

Friction Reducing Layer - Material placed immediately below the prestressed pavement to reduce slab-subbase interface friction.

Frost Action - The action of freezing, thawing, and associated movement of water to the freezing surface.

Grouting - A method of filling the conduit void spaces with cement martar to provide bond between the post-tensioned tendon and the concrete.

Initial Prestress - The horizontal compressive stress imposed at an early age on the concrete pavement. This is generally applied to prevent early drying shrinkage cracking of the pavement.

Internally Prestressed Concrete Pavement - A concrete pavement that is prestressed with tendons and does not require permanent external reactions.

Jacks - Devices used to apply tension to tendons or compressive horizontal stresses to concrete pavements.

Joint - The junction of two adjacent slabs interrupting the continuity of the pavement.

Lay of Length - The length of a strand for one full turn of the spiral wire. It is generally measured as the distance parallel to the strand axis between successive turns of the wire.

Modulus of Elasticity - The ratio of normal stress to strain. For concrete the secant modulus may be used. The secant modulus of elasticity is the ratio of stress to strain defined by the chord between origin and a point on the stress-strain curve.

Modulus of Rupture - Concrete flexural strength determined from the breaking stress of a concrete beam tested in third point loading.

Modulus of Subgrade Reaction - A measure of the subgrade support equal to the unit load on a 30-in. diameter plate required to obtain a deflection of 0.05 in., divided by 0.05. in.

Moisture Gradient - The variation of moisture content in the vertical dimension of the pavement cross-section.

Poststressing - A method of imposing horizontal prestress in concrete pavements without use of tendons.

Post-tensioning - A method of prestressing wherein compressive stress is imposed after concrete has gained sufficient strength to withstand the applied forces.

Preliminary Stressing - See Initial Prestress.

Prestressed Concrete Pavement - A concrete pavement with a horizontal compressive stress intentionally imposed in the longitudinal direction. Compressive stresses may also be intentionally imposed in the transverse direction.

Prestress Loss - Reduction in the initially applied horizontal compressive stress due to creep, elastic shortening, shrinkage, relaxation, and friction losses.

Pretensioning - A method of prestressing wherein the tendons are stressed prior to casting concrete around the tendons. The concrete is then cured to gain the required bond strength prior to releasing tendons from reaction system.

Pumping - The ejection of water and/or soil from below pavements through joint, cracks, and/or edges due to moving loads.

Relaxation of Tendons - See Stress Relaxation.

Rigid Pavement - A Portland Cement concrete pavement.

Slab - A monolithic section of concrete pavement bounded by joints and edges.

Slab Length - The distance between transverse pavement joints.

Sleeper Slab - A concrete slab placed beneath joints and ends of adjoining slabs to provide support for slab ends and/or the joint hardware.

Stabilized Subbase - A compacted layer of granular soils, gravel or crushed stone mixed with Portland Cement, bitumen, or lime prior to compaction, placed on the subgrade below the pavement.

Stress Relaxation - The irreversible inelastic elongation of steel under tensile stress, that results in tendon lengthening and reduction of tendon stress.

Subbase - A compacted layer of high quality material with good bearing strength and/or permeability placed over the subgrade. Stabilized or untreated compacted subbases are generally used below concrete pavements.

Subbase Friction - The resistance to sliding movement of a concrete pavement over the subbase.

Swelling - Volumetric expansion of the concrete due to gain of moisture content.

Temperature Gradient - The variation of temperature in the vertical dimention of the pavement cross-section.

Tendons - Tensile members used to impose horizontal compressive forces into the concrete pavement. Tendons are generally embedded in the concrete pavement, and consist of steel wire, bar, or strand.

Tendon Friction Losses - The difference between the load applied on the tendon and the load on the tendon a distance away from the point oload appliatio due to friction between tendon and conduit. This is generally divided into wobble and curvature tendon losses.

Tiebar - A deformed steel reinforcing bar embedded in a concrete pavement at a joint to hold slabs together. Usually used in longitudinal joints separating paving lanes.

Warping - Pavement profile change due to moisture gradient.

Zero Maintenance Pavement - A pavement designed and constructed so that it performs for at least 20 years without structural maintenance at the required level of serviceability.

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