Experimental Prestressed Concrete Highway Project in Pittsburgh

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This pavement, 5 in. thick by 12 ft wide by 530 ft long, was constructed in Pittsburgh, Pa., not only to provide technical data on the structural action of prestressed concrete highway pavements but also to provide reliable information on the feasibility of construction and the relative economics involved. Almost a year was occupied in working out the engineering problems. Construction was started in September 1956 and the final prestressing operation was completed in February 1957.

Valuable experience was developed by the construction operation, which generally worked satisfactorily. Among some of the unanticipated problems was a failure resulting from prestressing before adequate concrete strengths were obtained. Other desirable changes in practice are a reduction in maximum size of aggregate and some refinement in the construction at the ends of the slab sections.

A unique solution to the age-old expansion joint problem was developed. The joint, which consists of laminations of rubber and steel resting on a long flexible sill, has performed satisfactorily to date.

An extensive test series was carried out which included both static testing and moving load tests, but because of the time and cost involved no traffic load testing was included. Results were considered excellent and the structural aspects of a 5 in. thick highway pavement with longitudinal prestressing only appear to be entirely adequate for modern highway loads.

• ROAD TESTS, as well as experience with existing roads, show that the failures associated with concrete pavements are almost invariably initiated at and propagated from the transverse joints.

Because transverse joints constitute points of weakness in the pavement, it is logical that methods should be studied to eliminate this source of weakness. It is natural that the thinking of engineers should turn to prestressed concrete for the solution. Considerable thought has been given to the possibility of prestressed concrete highway pavement by Friberg (*1*) in this country. Experimentation has been carried on in Europe as reported by Stott (*2*) and others. Although there has been some experimentation in prestressed airport pavement by the Navy and reported by Cholnoky (*3*) there has been no prototype experimental work in prestressed highways in America.

It was to test the concept of prestressed concrete highway pavement that Jones & Laughlin Steel Corporation decided to perform this experiment. Preliminary studies of the engineering feasibility of prestressed concrete highway construction by the corporation date back to 1954. Detailed engineering studies were begun late in 1955. The solution of the many detailed problems, the selection of the best methods and the completion of drawings and specifications occupied nearly a year. Construction started in September 1956 and the test program on the finished road was completed by September 1957.

A question which is raised by engineers is: "Prestressed concrete is a good idea for a straightaway pavement, but will it work on horizontal and vertical curves?" Study shows that the external forces at any section are in balance. Another way of describing the mechanics is to state that the transverse component of the prestress force resulting from the angle change in the prestressing tendon must be exactly balanced by the summation of unit forces due to the tendon pressing against the side of the duct through which it passes. The efficiency of prestressed concrete compression members depends on the same phenomenon. The construction is practical because the prestressing tendon inhibits the tendency to buckle.

The purposes of the experiment are as follows:

1. Develop practical construction techniques.
2. Provide basic design data on thin prestressed concrete pavement slabs.
3. Solve the special problem of the expansion joint.
4. Develop economic data.

The initial studies of possible design choices strongly indicated the possibility that prestressed concrete pavement could be made competitive with conventional reinforced concrete pavement, provided that the prestressing could be limited to longitudinal only and that construction methods could be readily adapted to mass production techniques. It was recognized that this had to be strictly a projection since accurate economic data could not be obtained without full-scale operation with experienced contractors.

Site selection was dependent on the suitability of the subgrade and also on convenience combined with proper isolation of the location to prevent interference by miscellaneous and uncontrolled traffic. The selected site is adjacent to the corporation's Graham Research Laboratory at Pittsburgh. In spite of the urban location the site is completely isolated on a hilltop.

Consulting engineers for the project were Richardson, Gordon and Associates. Construction was carried out by Allegheny Contracting Industries of Pittsburgh. The instrumentation was designed and fabricated by the Fritz Engineering Laboratory at Lehigh University. The detail of the instrumentation is the subject of a supplementary paper.

The load testing program started in May 1957 and extended to August 1957. The program included both static load and slowly moving truck loading tests. The scope was not intended to include traffic load testing because of the extremely high cost of such tests. It included the obtaining of stress and deflection data and evaluation of subgrade friction. The data were very extensive and interpretations were not simple. Advantages of the construction, as well as its limitations, had to be evaluated.

The experiment was basically a full-scale project. The primary unit was a slab 12 ft wide, 5 in. thick and 400 ft long. This unit was flanked on one end by a 100-ft long prestressed slab and on the other by a 30-ft long prestressed slab. These flanking slabs were intended to simulate continuous lengths of road which might consist of a series of units 400 ft or more in length. The construction of the subgrade, the granular subbase, and the details of the construction of the slab, such as the aggregate used, the mix employed, and the slump allowed, were all designed to follow standard state highway specifications. Only such minor modifications were made as were necessary to adapt the construction to the new prestressing concept. Figure 1 shows the general scope of the project.

REVIEW OF FOREIGN WORK

In order to facilitate the work and to eliminate repetition insofar as practical, a study of the foreign literature was made. By the spring of 1955, there had been at least 12 prestressed pavements

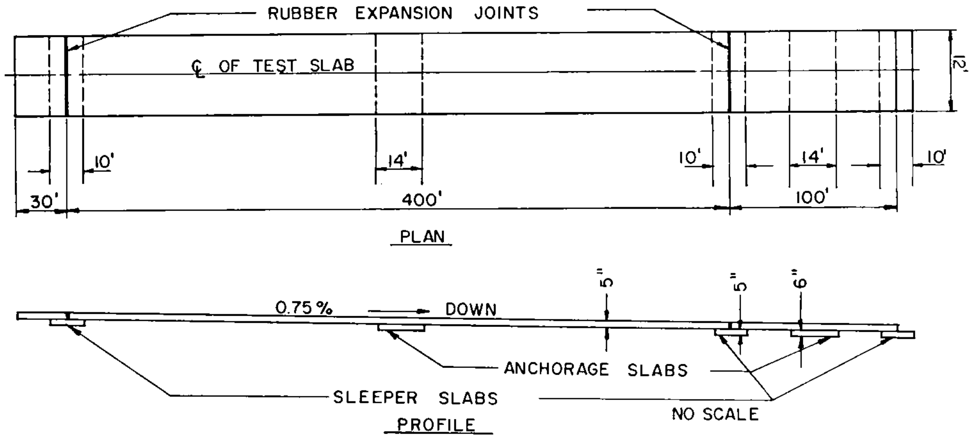

Figure 1. General scope of project.

constructed in Europe. The French had built pavements at Luzancy, Esbly, Bourg-Servas and Orly. The British had concentrated on highway slabs, constructing a number of pavements. Various schemes for prestressing had been employed. Some of the slabs had diagonal tendons for inducing transverse as well as longitudinal prestresses, while others were prestressed only in the longitudinal direction. These slabs varied in thickness from 4 to 10 in., and in length from 61 ft to 1,410 ft. The tendency for the more recently constructed slabs was to make them 6 in. thick, about 400 ft long with longitudinal prestress alone.

Since some of these pavements were built into actual highways subjected to heavy traffic conditions, it was necessary to provide various types of expansion joints in the pavements. One in particular at St. Leonards, Hampshire, utilized a steel toothed bridge type expansion joint. None of the joints tried were reported as satisfactory.

The general conclusions that the Europeans had drawn from their experience with prestressed pavements as reported by Stott (*2*) were as follows:

1. Prestressed pavements have good load carrying capacity.
2. The subgrade friction was the factor which limited the length of prestressed slabs.
3. Prestressed pavements required specially designed joints.
4. The cost of prestressed slabs appeared comparable to other forms of construction.

DESIGN

Early in the detailed engineering study it became evident that there were many gaps in the available data on prestressed pavements which could not be resolved by theoretical analysis.

An outline program of theoretical analysis and experimental investigation was developed. As this progressed, two conclusions became apparent: (a) Prestressing paving slabs greatly improves the load carrying capacity over conventional rigid pavements, and (b) the resulting crack free surfacing protects the subgrade against surface infiltration of water. The construction would eliminate most of the transverse joints; the exception being at the expansion point. If these widely spaced expansion joints could be adequately designed the points at which pavement failures normally occur would be eliminated. As a result, maintenance would be reduced, thus providing an economic incentive. Therefore, prestressed concrete pavement slabs appeared to be advantageous providing some practical means for their construction could be devised.

The study proceeded with the primary

objective of designing a prestressed pavement which could be practically and economically constructed as a highway. The main considerations were the technique of prestressing, the magnitude of the prestress required, the length and thickness of the slab, and the expansion joint.

The design was based on prestressing the slab in the longitudinal direction only. The work in England indicated that prestressing in one direction only would suffice and the simpler construction made it more desirable. The cost studies indicated that if two-way prestressing were required, prestressed highways would probably be uneconomical.

In the selection of the technique of prestressing on the project a great many factors had to be considered. Over-all cost as well as efficiency of action, had to be taken into account. It was agreed that a post-tensioning technique was likely to be most economic and feasible. It was difficult to conceive how in a pretensioning system the large forces involved could be held in the field during the setting period of the concrete. Moreover, horizontal and vertical curves would introduce such difficulties as almost certainly to restrict the application of the pretensioning method to straightaways.

Conventional post-tensioning techniques such as the Freysinnet, Magnel, or P. I. systems for wire, the Roebling method for large strand, or the high-strength steel bar system all posed serious problems.

First, the elongation involved was considerable. It is necessary to elongate the tendons at least 24 in. for a 400-ft unit length and proportionately more for longer lengths. Even though the jacking should be done at both ends the jack stroke would have to be very long or the operation would have to be set up as a multiple stage operation. Most of the conventional systems presented serious mechanical difficulties because of this factor.

A second consideration was the fact that the required end fittings occur at the most critical point of the construction. The slab ends, because of the relatively large thermal movements involved, require special treatment in any event. A short length of fill-in concrete would be required to cover the end fittings and to finish off against the expansion joint. It was feared that this short piece of non-prestressed concrete patch would be a point of weakness in the pavement.

A third point was involved with the mechanics of action of the prestressing operation. Whereas the steel tendons in the construction would be elongated about 24 in. the shortening of the concrete, under prestress, would be only about ½ in. total or ¼ in. at each end. Since the friction between the pavement and the subgrade resists the movement of the concrete, some portion of the prestress force would be dissipated. Using the technique of end stressing the prestress at the center would be less than at the ends and would possibly be completely dissipated before reaching the mid-length.

It was to overcome these difficulties that the gap jacking technique was selected. This technique involved the placement of flexible steel ducts containing the steel tendons for the full length of the 400-ft unit. At each end the tendons were to be anchored to the concrete by bond, supplemented by hooking or looping the individual units. At the center a gap was to be left. Jacks were to be placed in the gap and the two 200-ft half-sections were to be jacked apart, each moving 12 in. On the experiment a special frame was used to hold the gap open while the jacks were removed and the gap filled with concrete. Subsequent studies have developed the fact that it is feasible to jack open the gap from above the pavement, making the holding frame unnecessary and minimizing the gap dimension.

Using this technique the prestress in the slab increases from the slab end to the center. This is desirable because as the slab contracts due to seasonal temperature drop, the prestress is reduced from the slab ends toward its center by the restraint of the subgrade. Therefore, if a slab is constructed with an initial prestress higher at its center, the loss due to subgrade restraint will be of less consequence.

The exact nature of this restraint by the subgrade is not known. Experimentation in New Jersey (*4*) and work in England (*2*) indicated that it is partly frictional, partly elastic, and partly cohesive. Use of a friction reducing layer has been proven to have a beneficial effect. Because of its availability and relative ease of placement, a 1-in. layer of sand was chosen for this slab. The design calculations were based on the assumption that the subgrade restraint was a frictional effect. A coefficient of friction of 1.0 was used. Tests later, as the slab was being prestressed, gave a coefficient of about 0.7. There is no assurance that this factor is constant, or that the small, slow movements of the slab resulting from seasonal temperature changes are comparable to the friction during prestressing. Nevertheless, this was considered to be the most reasonable approach based upon available data.

To provide the best possible validity to the slab analysis and the subsequent test program, special consideration was given to the subgrade preparation. To determine the relative effectiveness of the prestressed slab itself, it was necessary to construct the roadway section supporting the slab according to the accepted specifications and methods used for conventional pavements. It was decided to attempt to satisfy only the minimum standards of the AASHO in order to create in the subgrade element its least favorable, yet acceptable, condition.

The analysis for determining how much prestress is required was based on the Westergaard (*5*) equations as modified by Kelley (*6*). It seemed reasonable to assume that as long as the pavement remained free of cracks it would behave elastically. As a matter of fact, a prestressed slab probably behaves more ideally in this respect than a conventional rigid pavement which is known to have cracking in many instances. Since special provisions were planned for the joints, which were to be spaced at 400 ft, the longitudinal tensile stress produced by edge loading at considerable distance from the joint by a 16,000-lb design wheel load was taken as the critical stress. A frictional restraint tensile stress and a critical warping stress were combined with this load stress. These computed stresses based on a subgrade K value of 150 were +966 psi load stress, −69 psi warping stress, and +200 psi restraint stress for the 400-ft long by 5-in. thick slab with a combined stress of +1,097 psi. To provide a prestress of this amount, enough to prevent the slab from cracking at all times, would make the cost prohibitive.

This brings out one of the inherent advantages of prestressed pavements. Cracked sections are not necessarily detrimental. If an edge loading is placed at a point where a shrinkage crack had occurred during construction, the concrete tensile value would be lost and thus the prestress would be all that remained for resisting tensile stresses. If the prestress value is exceeded the crack will open and the bending moment will be reduced at this section as it is transferred into the adjacent areas of slab. Since the negative moments away from the point of the load were calculated to be only about 20 percent of the positive moment at the loading point, this transfer utilizes the reserve capacity of the prestressed slab in the unloaded portion. Increased deflection will occur at the load point resulting in increasing the resistance offered by the subgrade, thus assisting in supporting the load. As the load passes, the pavement will rebound because of the prestress in the slab. This basic behavior of a prestressed pavement puts it in a category which can neither be described as rigid nor flexible, but lies between the two extremes and is actually elastic in nature. Therefore, a prestressed slab can be called an elastic pavement.

This reasoning led to the decision as to the amount of prestress required in the slab. The criteria selected required an initial prestress of sufficient magnitude to prevent the crack from opening under extreme conditions of temperature warping and frictional restraint stresses, with a moderate residual for the purpose of assisting in carrying load stresses. For the 5-in. thick by 400-ft long slab, the design prestress was chosen as 450 psi.

To obtain the selected value of prestress the proper design of the tendons had to be made. The steel design selected consisted of 24 stress-relieved $\frac{7}{16}$-in. diameter strands with a minimum ultimate of 250,000 psi. It was known that considerable losses due to friction between the tendon and the duct were to be expected. The design presupposed an initial unit stress at the center of the 400-ft section of 148,400 psi, or about 0.60 of the ultimate. Based on a loss of about 0.124 kips per ft of length the resulting stress at the ends would then be 124,000 psi which was sufficient to produce the desired minimum 450 psi prestress in the concrete. Since the friction loss between the strand and the conduit was not known with any degree of certainty it was decided to limit the maximum steel stress to the 0.60 of the ultimate. Creep and shrinkage losses would further reduce the minimum prestress obtained. However, since the friction losses were unknowns of more serious magnitude the creep and shrinkage losses were not considered. Moreover, it was believed that the prestress obtained would be adequate even though these additional losses occurred.

The 5-in. thickness was chosen because it was desirable to make the slab thin to utilize the prestressing steel to best advantage. With the amount of prestressing steel remaining constant the available unit prestress increases as the slab thickness decreases. Furthermore, assuming that the temperature gradient is the same, then the temperature warping stresses are proportionate to slab thickness. In addition there is the obvious advantage that the thinner the section, the cheaper it is. The final controlling factor in determining the slab thickness was cover over the tendons. Were it not for this limitation an even thinner slab could have been chosen.

The major factor which limited the length to the 400 ft selected was the magnitude of the tensile stress produced by the subgrade restraint. Provided the restraint loss is considered as a frictional effect, the loss in prestress is proportional to the length of the slab. The experience of the British, according to Stott (*2*), that 400 ft was the practical maximum length for prestressed concrete pavements as limited by the present knowledge of the nature of the subgrade restraint influenced the decision.

The design of the experimental slab may be summarized as follows:

1. The basic section was 12 ft wide by 5 in. thick by 400 ft long (see Fig. 2).
2. The tendons were 24 stress-relieved strands, $\frac{7}{16}$-in. diameter, placed within six $1\frac{1}{2}$-in. diameter flexible steel conduits.

EXPANSION JOINT

An important factor influencing the design, particularly the length of slab, was the expansion joint. It was recognized from the beginning of the studies that the joint between prestressed sections of road would be a major problem. Obviously the magnitude of movement involved precluded the possibility of using conventional mastic joints and load-transfer devices. For a 400-ft section the calculated range of movement under a 100 F temperature variation would be about 3 in.

Although reports by McIntosh and Mercer (*7*) indicated that the actual movements to be expected are much less than the calculated amount, they are still greatly in excess of acceptable amounts for conventional treatment. The explanation for this appears to be the compensating effects of moisture on dimension. Hot weather lengthens the slab through thermal expansion but the simultaneous drying out of the moisture produces shortening. In cold weather thermal shortening occurs but the concrete absorbs moisture which results in compensating lengthening. Table 1 shows measurements taken on the experimental slab with calculated thermal dimensional changes compared to measured changes.

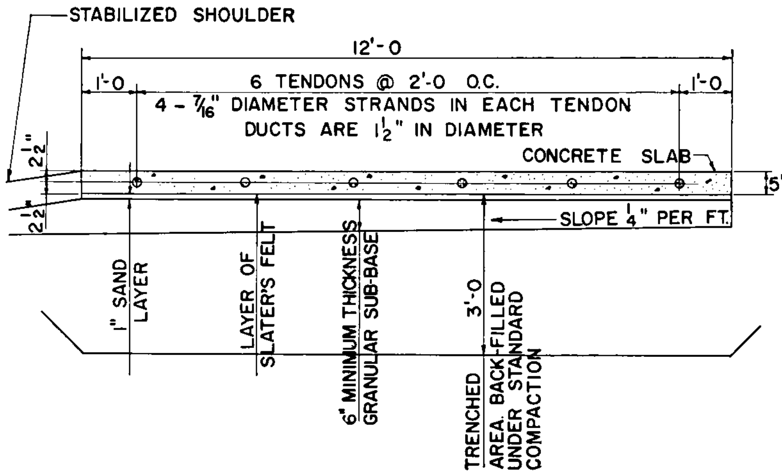

Figure 2. Experimental pavement cross-section.

The measurements confirm the reports of measurements taken on other projects. The measured changes are about 40 percent of the calculated thermal changes. For this reason it appears that the dimension changes which must be provided for are about 1¼ in. instead of the computed 3 in.

The initial thinking on the expansion joint problem was in terms of some mechanical joint which would simultaneously provide for appreciable movements, transfer shear, exclude dirt and moisture, and maintain a level surface which would resist wear and corrosion.

Experience with bridge expansion joints indicated that the hope for a successful and economical solution with mechanical devices was remote. Mechanical joints on bridges are frequently a source of trouble and seldom operate with the degree of perfection desired. It was evident that a new approach was needed to solve this problem.

TABLE 1

LENGTH CHANGES WITH TIME

Date	Temp., °F			Dim. Change, in.	
	Air	Slab	Diff.	Calc.[1]	Meas.
Aug. 7, 1957	89	95.3	—	—	—
Nov. 10, 1957	26	31.2	64.1	1.69	0.72
Jan. 3, 1958	14	17.8	77.5	2.05	0.81

[1] $K = 0.0000055$.

The shear transfer was abandoned in favor of load transfer through a subslab which has been termed the "sleeper" slab. The sleeper slab was intended to provide load transfer from the slab ends, and from the expansion device. The sleeper slab was conceived to provide a plane on which the slab ends could slide during the jacking operation and during subsequent thermal movements. It was deliberately made long and relatively thin, the same 5-in. thickness as the main slab, so that action on the subgrade would be similar. Unfavorable experiences in the past with sills were assumed to be due to the short and stiff character of the subslabs (sills) used. These short and stiff sills tended to rotate under load, permitting differential movement of the adjacent ends of the pavement.

The sleeper slab, 10 ft long and only 5 in. thick, had the required flexibility. For these reasons rotation would be avoided. The tests later substantiated the action of the sleeper slab. Deflections, under load, of the sleeper slab are given in the section on testing.

It was recognized that differential frost heave must be guarded against. The granular subbase under the sleeper slab would have to be gradually reduced in thickness and tapered into the granular subbase under the main slab.

The problem of load transfer being resolved, the designer's thoughts turned to the fundamental problem of providing for the slab movements. Requirements of a satisfactory joint were considered to be as follows:

1. The joint must adequately provide for the large anticipated movements between pavement units.
2. The top surface must remain level within reasonable tolerance limits regardless of the expansion or contraction of the pavement.
3. The joint must be capable of transmitting highway loads to the sleeper slab without excessive deflection.
4. The joint must be either sealed against dirt and moisture or must be self cleaning.
5. The material from which the joint is made must resist wear, attack from oil and grease, soil chemicals and alkalies which may be present from the cement, deterioration from sunlight, weathering or aging, and snow melting salts.
6. The joint must resist deterioration from frost action.
7. Maintenance must be a minimum factor. Furthermore, it must be possible to replace the joint in case of damage or wear.

A review of the requirements illustrates the fact that the problem is a complex one. It confirmed that conventional mastic joints or mechanical expansion joint devices would not suffice.

A new concept, a rubber expansion joint, was suggested. Loads could be carried by steel inserts. The rubber could be shaped with cored holes in such a manner that the rubber displaced by compression of the joint would fill the cores rather than produce a rise in the surface level. Modern rubber technology offers compounds capable of providing all of the durability qualities demanded of the joint.

The basic idea, together with sketches and specifications, was taken to the B. F. Goodrich Industrial Products Company. As a result, a joint meeting all of the requirements was designed, built, and tested.

The final design of the joint is shown in Figure 3. It is of laminar construction using steel plates to carry the load to the sleeper slab. The rubber component is a neoprene synthetic designed to resist wear and corrosion. The development of the joint required about 1½ years of engineering and testing. Three full-sized models of the joint were made and tested before the results were fully satisfactory.

CONSTRUCTION OF EXPERIMENTAL SLAB

Excavation and subgrade preparation began in September 1956. By October the subgrade work was complete and the granular subbase was placed.

Considerable care was taken in the preparation of the subgrade. The soil used in the subgrade is classified as an A-7-6 clay with a group index of 20. This is rated as poor subgrade material by AASHO and HRB. The minimum unit weight of embankment material accepted by AASHO is 90 pcf. The T-99 unit weight of the clay used in this project varied from 90 to 95 pcf, with average values of 93 pcf and an optimum moisture of 27 percent. Although the minimum compaction requirement for such a material is usually 100 percent of the T-99 maximum density, compactive effort was controlled during construction of the subgrade to within the range of 95 to 100 percent of the T-99 maximum at moisture contents at or above the optimum.

To provide uniform properties and characteristics throughout the paved section, the roadbed was undercut and backfilled to a depth of three feet. The A-7-6 clay was placed in 8-in. layers, and each layer was compacted to the limits given above. This combination of compaction requirements and embankment material produced a subgrade which satisfied only the minimum acceptable standards. This provided a pavement structure which could be tested under less than ideal conditions of subgrade support.

A natural sand-gravel mixture meeting the requirements of AASHO, Type 1, Grading B was used for the granular sub-

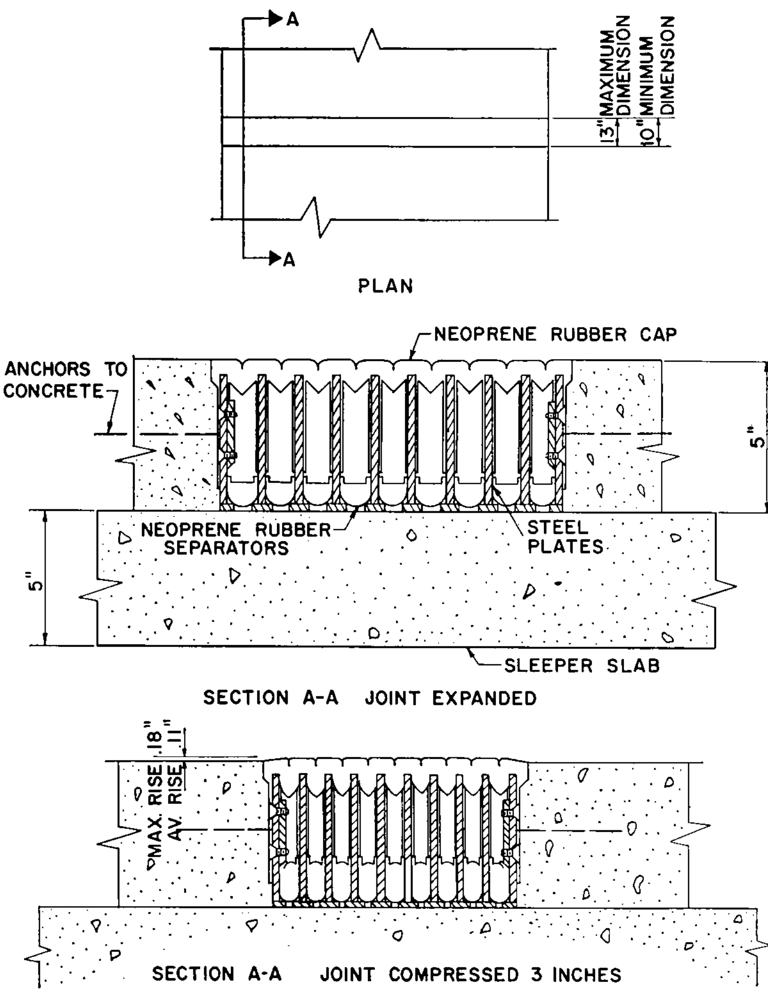

Figure 3. Neoprene rubber expansion joint.

base. The compacted thickness of this subbase was 6 in. covered by a 1-in. leveling layer of washed sand.

Figure 2 shows a cross-section of the construction, including the antifriction sand layer and the slater's felt, which was used to obtain a smooth underside on the slab. The location and makeup of the tendons are also shown. They were positioned by a device consisting of 6 split pipes secured to a "scratch board" which was pushed immediately ahead of the spreader. These split pipes were made long enough so that the ducts were held in position until the concrete had been spread around them (Fig. 4).

The four $\frac{7}{16}$-in. diameter strands used in the tendons were typical stress relieved prestressing strands. Physical properties are given in Table 2.

The east 100-ft section was placed on November 2 and prestressed on November 7 without incident. The main 400-ft center-section was placed on November 14 and the prestressing of this section was attempted on November 21. Shortly after starting the prestress operation, it became apparent that conditions were less than ideal. Early in the operation distress cracks began to appear in several of the jack bearing areas. After deliberation it was decided to continue with the

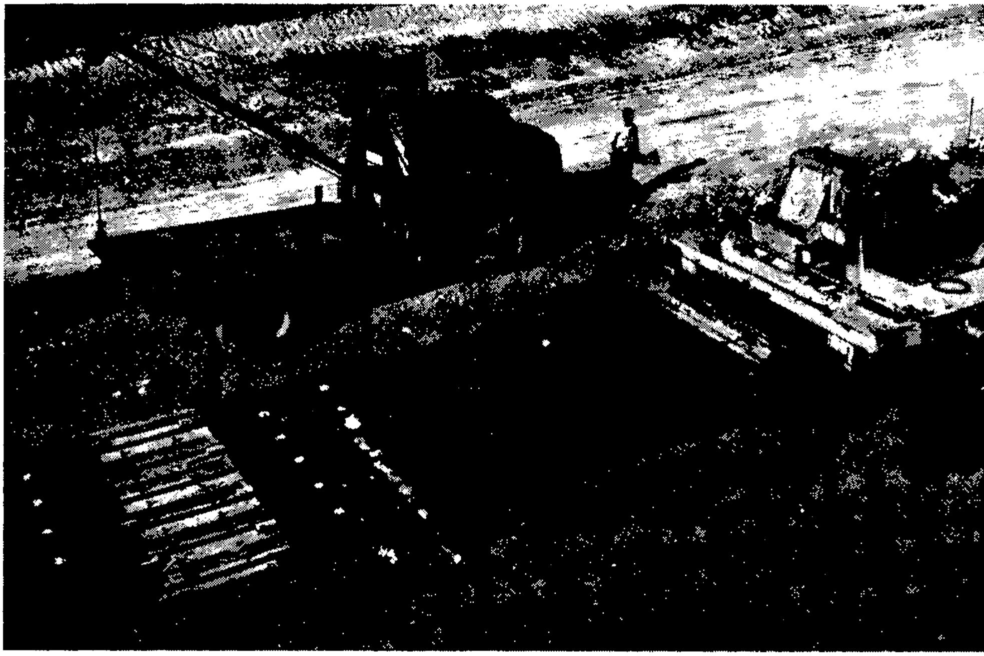

Figure 4. Tendon positioning device.

operation. At the maximum extension of the gap (nearly 18 in.) a failure of the west anchorage in the third tendon from the north side occurred. Finally the jack bearings on the east slab failed. The east slab slid up over the jacks completely dissipating the prestress.

The slab was repaired and was successfully prestressed on February 10, 1957. An investigation of the reasons for the failure was made. Obviously failures of this nature could not be tolerated on a construction project.

Before the start of construction, comprehensive laboratory tests were run to determine both the capacity of the anchorage and the ultimate bearing value under the jack bearing blocks. With test-cylinder strengths of about 2,700 psi (Table 3) anchorage failure in the 4-strand tendon group occurred at about 100,000 lb against a load during jacking of about 65,000 lb (Table 4). Ultimate bearing strength under the 3-in. by 3-in. by 15-in. bearing blocks was found to be 4,400 psi against a bearing value during jacking of 2,400 psi (Table 5). It was the apparent safety factor plus the previous successful prestressing of the 100-ft east section which influenced the decision to

TABLE 2

PROPERTIES OF $\frac{7}{16}$-IN. DIAMETER 7-WIRE STRAND

Reel	Metal Area, sq in.	Breaking Strength, lb	Ult. Unit Strength, psi	Elongation, %
1	0.1090	28,700	263,500	9.5
2	0.1090	29,800	273,500	10.5

TABLE 3

STRENGTH OF CONCRETE USED IN PRELIMINARY TESTS OF PULL-OUT AND BEARING SPECIMENS

Cylinder Test No.	Age, days	Cylinder Strength, psi		
		Spec.	Avg.	Corrected To ASTM Std.
2A	2	1,410		
2B	2	1,390		
2C	2	1,220	1,340	2,040
3A	3	1,270		
3B	3	1,500		
3C	3	1,760	1,510	2,290
4A	4	2,030		
4B	4	2,030	2,030	3,090
7A	7	2,800		
7B	7	2,920	2,860	4,350
8A	8	2,800		
8B	8	2,470	2,635	4,000

TABLE 4
RESULTS OF PRELIMINARY TESTS ON PULL-OUT SPECIMENS

Specimen Number	Age, days	Mode of Failure	Ultimate Capacity, lb	Unit Stress in Strands, psi	Factor of Safety [a]
1	4	Concrete split	98,000	225,000	1.51
2	7	Concrete split	95,800	220,000	1.48
3	7	Concrete split	105,000	241,000	1.62
4	9	Strand break	109,700	252,000	1.69
5	9	Strand break	116,500	268,000	1.80

[a] Compared to 65,000-lb maximum tendon load during jacking.

proceed in spite of the early signs of weakness during jacking.

The reason for the misjudgment was the fact that the test cylinders were capped and broken in different laboratories. Later tests indicated that the cylinders tested as 2,700 psi in the preliminary test specimens were actually equivalent to 3,500 psi when tested by the laboratory which capped and broke the cylinders for the 400-ft center-section. A contributing factor was the effect of the colder weather occurring at the time of placement of the 400-ft section. This resulted in a slower rate of gain of strength. A possible secondary factor was the fact that the bearing which failed was slightly out of square and had to be shimmed.

The tendon in which the anchorage failed was replaced and both anchorage ends of the section and both of the jacking ends were rebuilt.

Figure 5 shows the tendon anchorage design. Table 4 shows anchorage capacities measured in the laboratory. Since the cylinder strength tests for the concrete used were not made according to ASTM standards, the values of the concrete strengths used have been corrected to ASTM values and are shown in Table 3. Table 5 shows the results of the preliminary bearing tests.

The factors of safety noted are based on loads subsequently determined during the jacking operation. Although the field cylinder strength at which an adequate factor of safety is obtained was somewhat obscured by the non-standard cylinder strengths used during the pull-out and bearing-test program, it is evident that adequate safety factors are obtained at about 3,500 psi cylinder strengths when made according to ASTM standards.

As an expedient during the construction of the experimental slab, the tendons

TABLE 5
RESULTS OF PRELIMINARY TESTS ON BEARING SPECIMENS

Specimen Number	Age, days	Ultimate Capacity, lb	Unit Bearing Value, psi	Factor of Safety [a]
1	4	128,000	2,570	1.07
2	7	199,000	4,420	1.84
3	7	200,000	4,450	1.85

[a] Compared to 2,400-psi bearing value during jacking.

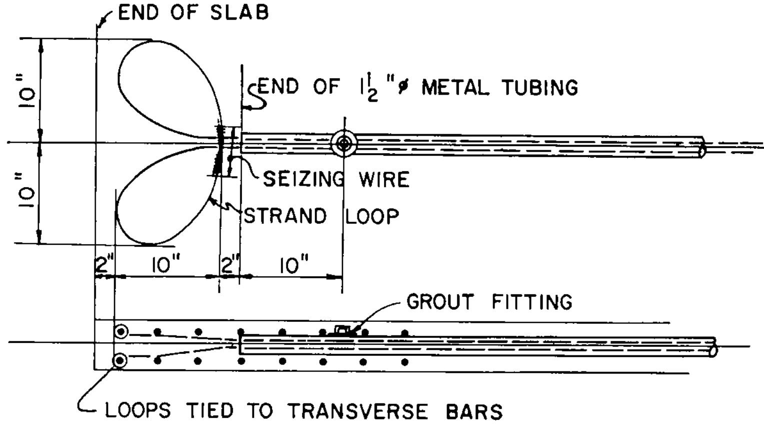

Figure 5. Detail of anchorage loop.

were manually pushed into the ducts and the end anchorages were made by hand. Whether strand or wire was to be used on a future full-scale construction project, it is expected that the strand or wire would be supplied to the contractor fabricated complete with duct and anchorage. The assembly would probably be supplied in 8- or 10-ft coils.

The mix used in the concrete for the slabs was Pennsylvania Department of Highways Class AA. The cement was air-entrained. Aggregate was stone with a 2½-in. maximum size. Maximum water was 5.3 gallons per sack. Table 6 shows cylinder strength, modulus of rupture, and slumps of the concrete used. Note the slow rate of gain of strength of the field specimens. This was the result of the low temperature prevailing at the time and further confirms the reasoning used to explain the causes of the failure.

The modulus of elasticity of the concrete was determined from a test curve. This was an important determination because it permitted translation of strain measurements to stress.

The jacking operation was performed using five pairs of 150-ton jacks placed back-to-back. The total available jacking effort was 750 tons. As recorded later, the actual maximum jacking effort required was 270 tons. However, the jacks used were commercially available.

The jack pairs were used for two reasons. The required total stroke of 24 in. was not available in a single jack. In addition it was necessary to control the movement of each 200-ft length to secure its proper location and to prevent one 200-ft length from doing all of the moving. The differential force was carried into an anchor slab to which the jacks were bolted (Fig. 6).

In addition to the 3-in. by 3-in. by 15-in. steel bearing blocks, ¼-in. hardwood plywood was used between the jack rams and the concrete.

To contain the prestress after jacking, a structural holding frame (Fig. 7) was inserted. The jacks were removed and the gap filled with concrete. After the concrete in the gap had set the holding frame was removed, thus transferring the prestress through the gap concrete. This operation was done at an age of 7 days.

After the jacking operation the ducts were grouted with neat cement grout, using nipples attached to the duct before placing the pavement concrete. No particular difficulty developed in the grouting operation. Blockages, which did occur, were at the right-angle nipple at the entrance to the duct. A Y-entrance fitting would correct this difficulty. It is not possible to report grouting pressure. The grouting equipment was not supplied with metering equipment to measure pressure.

The grout was forced through the duct under pressure until all air and water was excluded. At this point a valve at the exit end was closed. It would be preferable to shut off the flow at the entry end as soon as the grout flowed freely. Thus, any tendency to produce water hammer would be eliminated.

There is some indication that water hammer produced cracks in two short lengths over the ducts. These cracks were found immediately after the grouting operation. Inasmuch as they had not been observed previously during the many

TABLE 6

PROPERTIES OF CONCRETE USED ON 400-FT SLAB

Specimen Type	Curing	No. of Specimens	Test Age, days	Average, psi
Cylinder [a]	Lab.	12	3	1,716
		12	7	2,922
		12	14	3,763
		12	28	4,483
		12	50	4,803
		11	100	5,363
		12	225	5,680
			365	5,870
Cylinder [a]	Field	6	3	1,609
		6	4	2,015
		6	5	2,374
		6	6	2,784
		9	7	2,722
		6	8	3,003
		9	28	3,367
Beam [b]	Lab.	8	3	397
		8	7	552
		8	28	706
Beam [b]	Field	4	3	388
		4	7	551
		4	28	727

[a] 6-in. diameter by 12-in. height.
[b] Pennsylvania standard; 6 in. by 8 in. by 40 in.

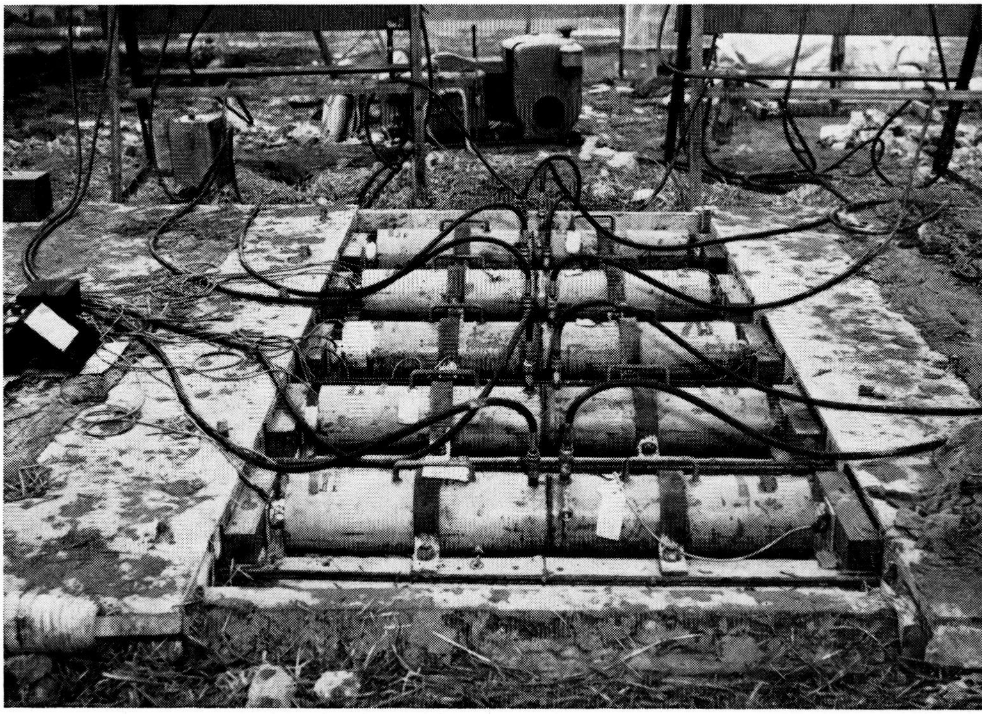

Figure 6. Jacks in position at gap ready for prestressing.

inspections of the slab, it is reasonable to assume that grouting contributed to their formation. The cracks appeared directly over the ducts at points where curling stresses would be expected to produce the greatest effect.

Although the length of the longitudinal cracks involved is relatively small, being only about 3 percent of the total length of tendon involved, they are recognized as a potential problem in one-way prestressing of pavement slabs.

The water hammer could be prevented by shutting off the flow at the entry end. Additional remedial measures suggested are to use a maximum aggregate size of 1½ in. in place of the 2½ in. used on the project in order to minimize the possibility of honeycombing, to use maximum curing precautions to prevent warping resulting from shrinkage of the top surface, and to reduce the duct size. In view of the small amount of actual longitudinal cracking obtained, it is believed that these measures should be sufficient to virtually eliminate the problem.

EXPERIMENTAL TESTING

The scope of the test program included soil tests, observations during the prestress operation, static load tests, slowly moving load tests, warping studies, and tests of the action of the expansion joint. All static and moving load testing was done under temperature warping conditions producing maximum combined stresses. The program did not include traffic load testing.

Because of the extremely complicated character of the instrumentation used, it will be described more fully in a supplementary paper. The instrumentation included strainometers for measuring concrete strains, Whittemore strain gages for strand measurements, sensitive deflectometers, temperature gages, clinometers, load cells, and pressure gages. Oscillograph recordings were made of the moving load tests. The instrumentation leaned heavily on the use of the SR 4 electric strain gage. All instrumentation except the Whittemore, the clinometer, and the pressure gage used the SR 4 as the transducer.

Soil Testing

Plate bearing tests performed on the completed subgrade indicated a modulus of subgrade reaction (*K* value) of slightly more than 200, with the moisture content very close to the plastic limit for the soil. This *K* value remained substantially constant during the 8-month testing period between October 1956 and June 1957.

The plate bearing tests also indicated that the rebound of the subgrade for the magnitude of deflections caused by dual wheel loads as high as 32,000 lb was 90 percent or greater. The rebound was reduced to approximately 50 percent under the maximum loads of 66 kips applied during the static load testing program.

Measurements During Jacking Operation

As the slab was prestressed by jacking the gap apart, slab movements, concrete strains, tendon strains, and jacking force were measured. Figure 8 shows the layout of the instrumentation used in this portion of the testing.

The design calculations had indicated that by jacking the gap open approximately 2 ft the required prestress would be obtained. The total strand strain would be practically this amount, because the concrete shortening would be only about ½ in. Each of the two 200-ft half-sections was moved over the subgrade approximately 1 ft. The jacking was done in 3-in. increments and a set of measurements was taken immediately after each incremental move.

Figure 6 shows the two banks of jacks during the prestressing operation. Figure 9 provides a convenient method for determining the static coefficient of friction for the two 200-ft slabs. Extending these curves until they intersect the abscissa

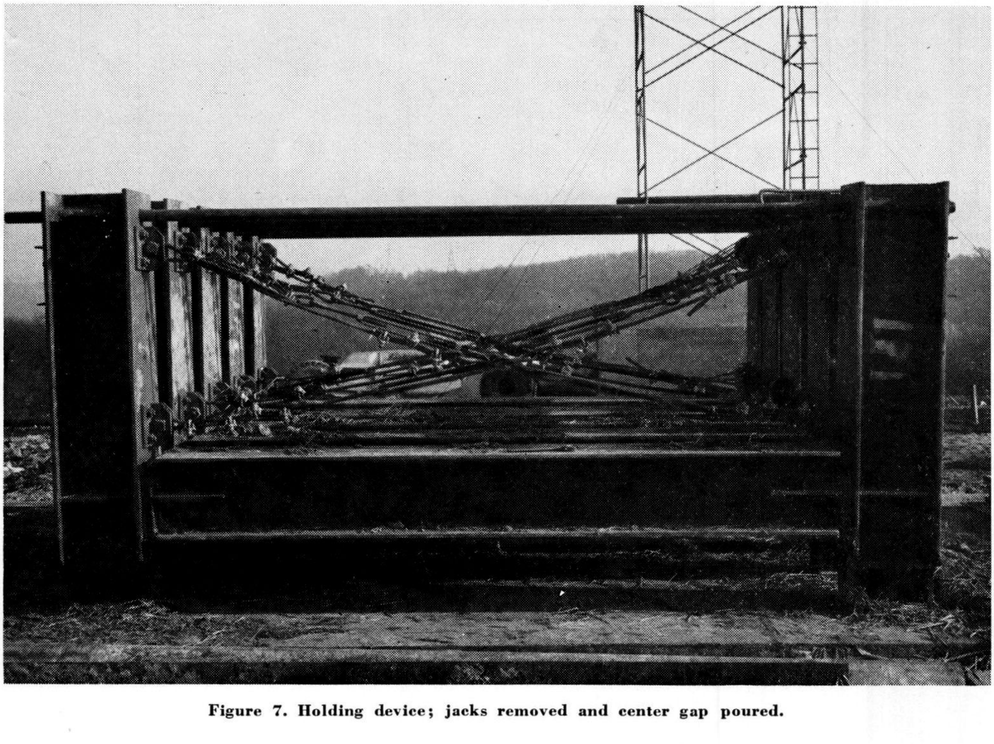

Figure 7. Holding device; jacks removed and center gap poured.

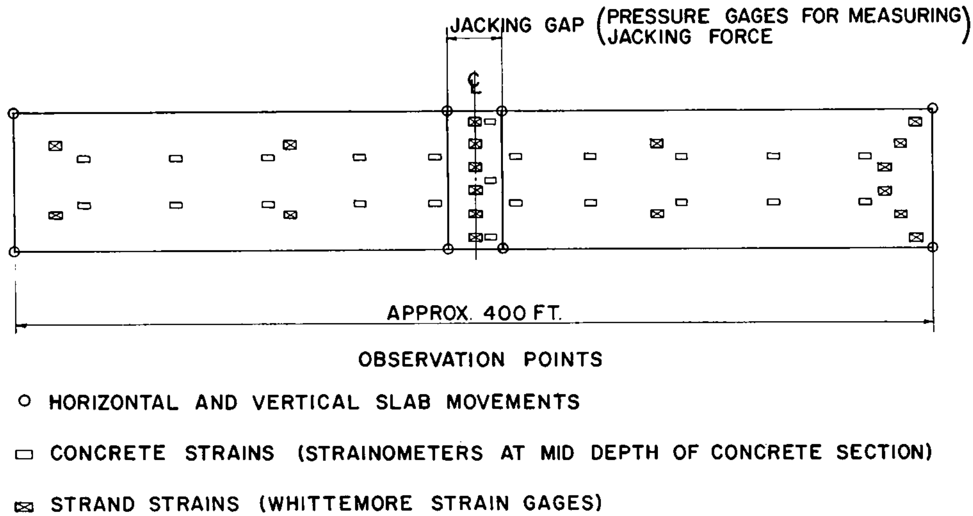

Figure 8. Location of instrumentation during jacking.

	SLAB WEIGHT (KIPS)	COEFF. OF STATIC FRICTION	COEFF. OF SLIDING FRICTION
EAST SLAB	146.6	0.64	0.34
WEST SLAB	147.4	0.71	0.60

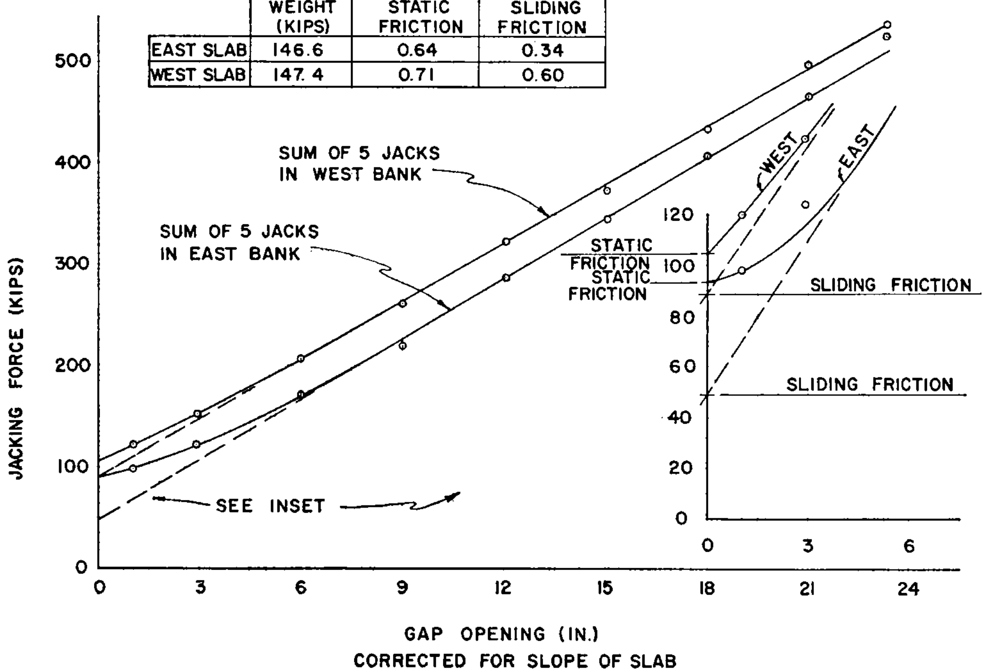

Figure 9. Jacking force vs gap opening during prestressing.

for zero opening determines the force of the jacks against the east and the west slabs required to initiate sliding. Two different curves are involved, one for the east slab and one for the west slab. The separation is due to the fact that a slight difference existed between the frictional resistance under the east and the west slabs. Also, the slabs were on a 0.75 percent grade, which meant the resistance to movement of the downhill slab was reduced by 1.1 kips, the gravity component. The difference between the two forces pushing on the east and the west slabs was absorbed in the jack anchoring slab.

The inset of Figure 9 shows the method used to derive the coefficients of static and sliding friction of the east and west slabs, and the values obtained. The static friction coefficient for the west slab is somewhat larger than for the east slab and reduction in the coefficient under sliding conditions is not as great. The reason for this variation is not known, but it indicates that the value is probably not constant. From the data it may be concluded that the coefficient of static friction under a concrete slab, when adequate precautions are taken, will vary in the range of 0.64 to 0.71 and that this value drops off after initiation of movement. The decrease in friction under motion was repeatedly confirmed during the several other slab jacking operations performed in the experiment.

Figure 10A charts measurements of concrete stresses, tendon stresses, and jacking pressure. It should be noted that the slopes of the lines showing the strand stresses indicate a small loss from the center each way to the quarter points; from the quarter point to the end of slab the loss is much greater. This resulted from the known misalignment of the tendons at the heavily instrumented load stations, which were located between the quarter points and the ends.

Table 7 gives this in terms of percent loss per foot of length. Again, the very small loss in the center portion and the relatively large loss in the end areas is apparent. This shows that the positioning method was very effective until the discontinuity at the instrumentation station was reached. In practice it should be possible to hold losses to about 0.15 percent per foot. The data are for 7-wire strand in flexible steel conduit. If straight wire tendons are used the friction probably will be somewhat less.

TABLE 7
FRICTION LOSS BETWEEN TENDON AND CONCRETE

Location	Loss, %/ft: Before Jack Release	After Jack Release
Center to ¼ point, east slab	0.158	0.053
Center to ¼ point, west slab	0.125	0.010
Average	0.142	0.032
¼ point to east end	0.200	0.177
¼ point to west end	0.286	0.234
Average	0.243	0.206

Figure 10B shows that the friction during motion which has to be overcome is considerably greater than the residual friction restraint under the at-rest conditions shown in Figures 10C, 10D, 10E, and 10F. Figure 10E shows that the friction between the slab and the subgrade is practically negligible while the jacks hold the gap open. This is almost a dynamic condition, with the extended tendons acting against the jacks as a long spring.

Figure 10F shows the friction between the slab and the subgrade after the slight return movement that occurred when the jacks were released. This movement amounted to about $\frac{3}{32}$ in., which was sufficient to cause a considerable readjustment of concrete prestress.

The shape of the curves would change somewhat under more normal construction practice and the condition shown can be considered as less than ideal. Reduction of the tendon friction losses would tend to rotate the ends of the curves closer to the horizontal and result in a higher minimum and a more uniform concrete prestress.

Static Load Testing

A total of 16 static load tests was conducted on the experimental slab at three loading stations (see Fig. 11). Two of the stations were at the quarter points

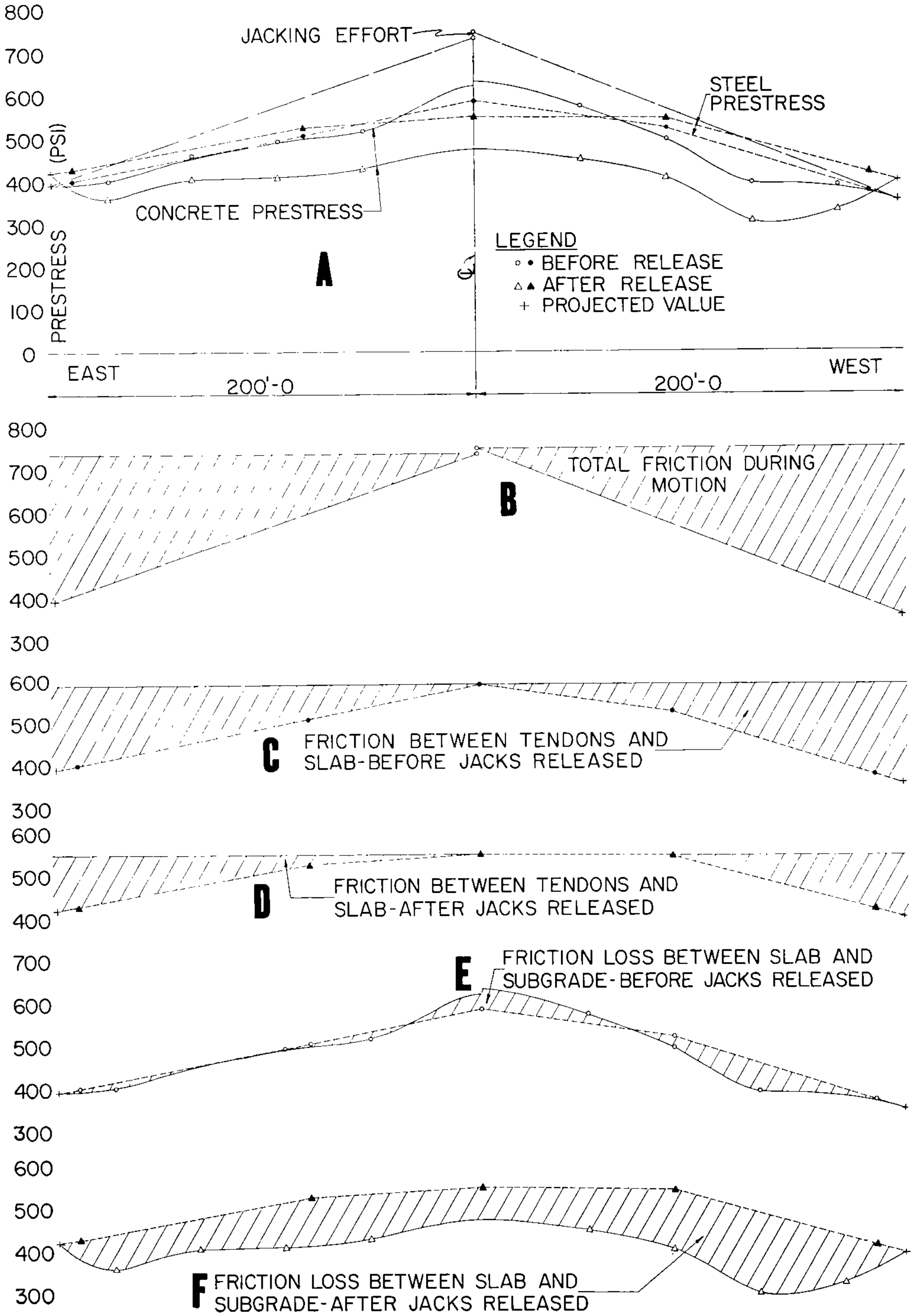

Figure 10. Measurements during prestressing.

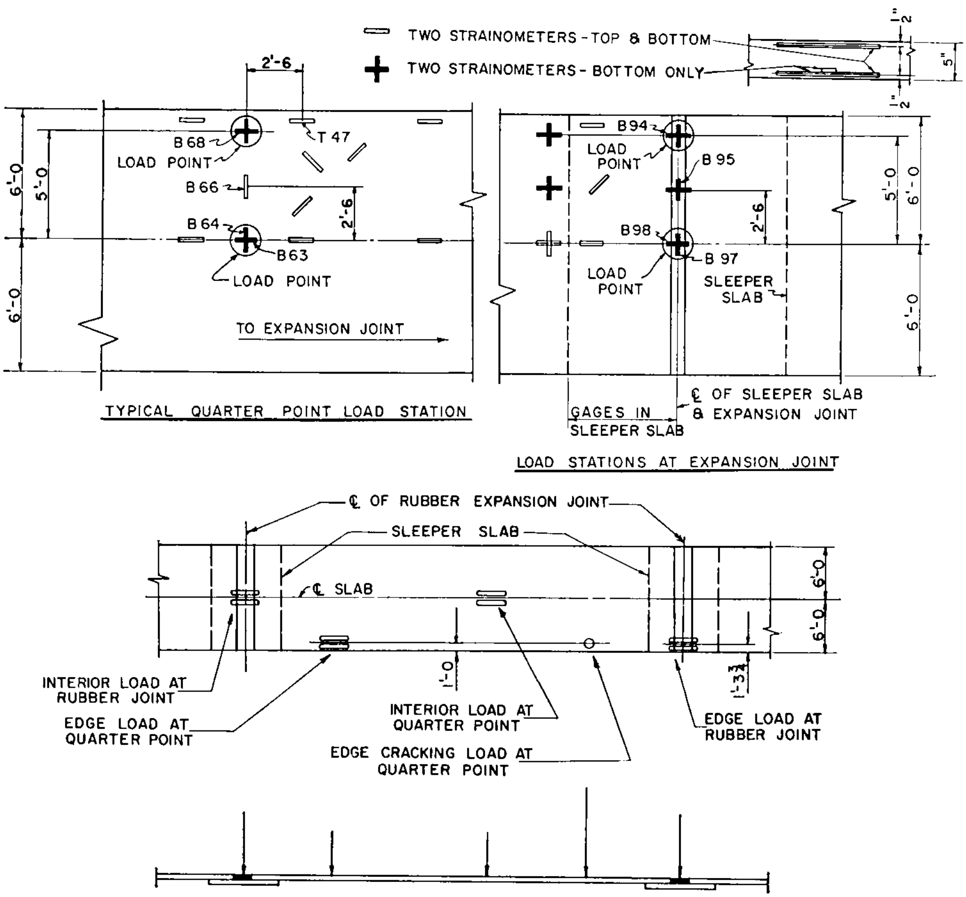

Figure 11. Static wheel load testing set-up: (top) typical load stations and (bottom) application positions. The latter are diagrammatic in length.

of the slab, and one was at the west end directly over the rubber expansion joint. Concrete strainometers, positioned to measure bending strains, were placed in the main slab at the quarter point stations and the sleeper slab at the joint. Incremental loads were applied within the elastic range. Subsequently, cracking loads were applied. In addition to concrete strain measurements, deflections were measured with precise level clinometers. The static loads were applied by jacking against a loaded steel platform which spanned the slab (Fig. 12). Two 35-ton hydraulic jacks reacted on a special wide flange beam axle through a dual wheel on which were mounted a set of standard 10 × 20 truck tires. The load was thus applied to the pavement in a manner similar to an actual truck wheel load. The magnitude of the load was determined by using two SR-4 load cells. About one-half of these tests were made at the interior of the slab, the other half at the edge. Positioning of the loads is shown in Figure 11.

The results showed that the 5-in. thick prestressed slab is entirely adequate to carry the design wheel load of 16,000 lb. Maximum deflection for this load was 0.048 in., which is well within the elastic range of the subgrade. As pointed out earlier, increased deflection of these elastic pavements is not necessarily any disadvantage. For all practical purposes, the pavement behaved perfectly elasti-

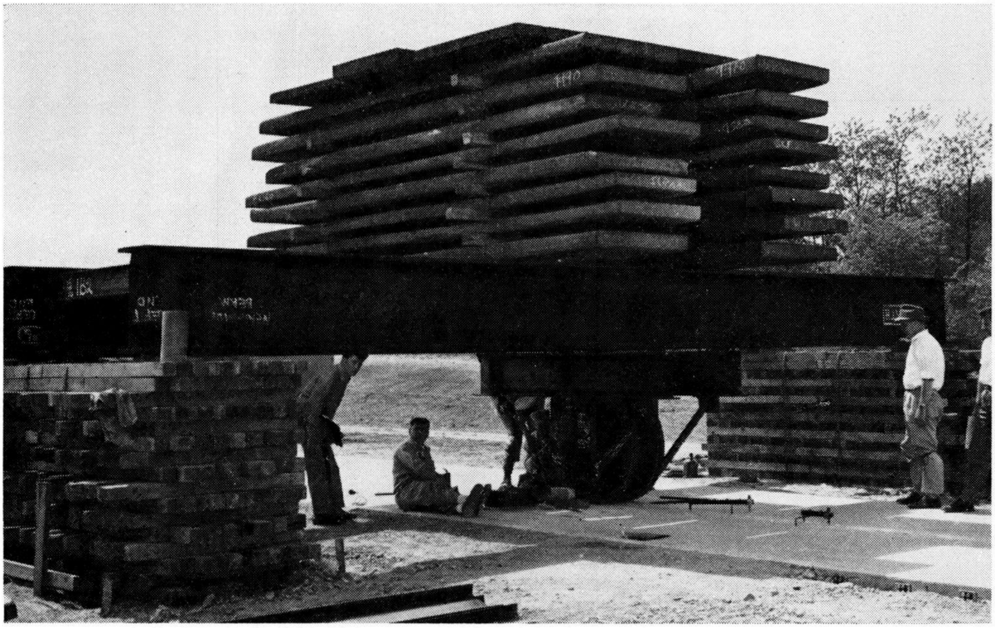

Figure 12. Loading platform and dual wheel used for static load tests.

cally in this range of loading (Fig. 13a), and no cracking of the concrete was observed.

Deflection curves for both transverse and longitudinal lines were obtained with the clinometers for the 16,000-lb load both for the interior and the edge loading condition. Typical curves for these cases are shown in Figure 14.

Concrete bending strains were found to be closely concentrated in the vicinity of the load. For all practical purposes, areas 3 to 4 ft away from the load application point were found to be unstressed, which is additional evidence of the flexibility of prestressed pavement. However, the concrete which was most highly stressed (beneath the load point) behaved elastically. Figure 13b shows a typical load-concrete stress curve for the 16,000-lb interior and edge load. One of the interior quarter point load tests produced erratic results. The strains directly beneath the load in the longitudinal direction were less than in the transverse direction. This is contrary to all past experience with narrow slabs loaded with dual truck tires and was contrary to the results obtained at the other interior load station. It has been assumed that no valid results were obtained from data at this particular loading point because of some unknown inequality of the construction or some error of instrumentation.

The load-deflection curve (Fig. 15) and the load-concrete stress curves (Fig. 16) for the interior point at which valid measurements were obtained, show linearity up to a load of 40,000 lb, which is evidence that the slab did not actually crack under this loading. In addition, no surface cracking was observed although none would be expected in this instance since the high tensile stresses were on the under side of the slab. This load was applied through the tires and no further attempt was made to crack the slab.

Figures 15 and 16 also show the load-deflection and load-concrete stress relationship for high-level loading. As shown by Figure 15, the load-deflection relationship is virtually linear up to the 66,000-lb loading. The load-concrete stress diagram for interior loads (Fig. 16) shows that the relationship is linear up to the 40,000-lb maximum loading used.

Some hypothecation had to be made as to the shape of the curve for gage B68A, the longitudinal gage under the edge steel plate loading. No readings were available between the 18,000-lb and the 30,000-lb wheel load. It was obvious that cracking had occurred at some level between these loads. From the linearity of the curves to a level of 1,100 or 1,200 psi for B63, the interior longitudinal gage, and for the longitudinal edge gage (B68B) at the B loading station (under rubber tire loading) it was hypothecated that the break occurred at the same stress level for B68A. Comparison of curves for B68A and B68B shows that the circular steel plate loading produced much higher stresses than did the rubber tires.

These are key curves for the development of design criteria for prestressed concrete pavement. Loads stressing the pavement from zero to the prestress level do not produce tension in the concrete. The curve indicates that for the steel

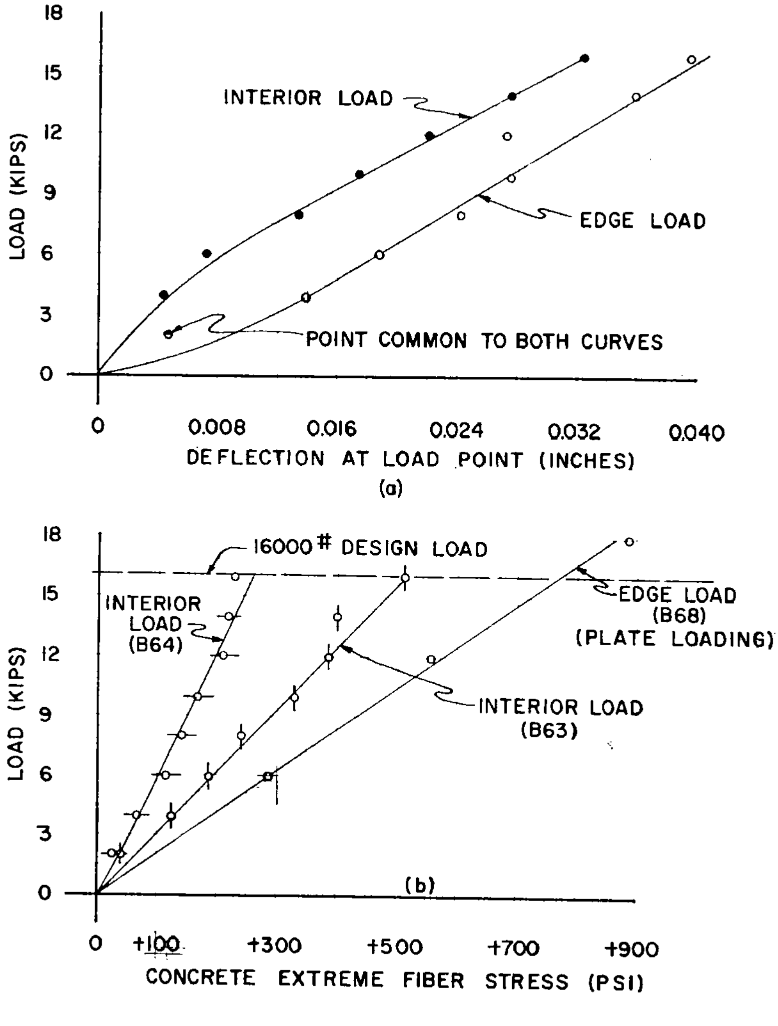

Figure 13. Typical load-deflection and load-stress curves under static loads; loading through rubber tires except as noted.

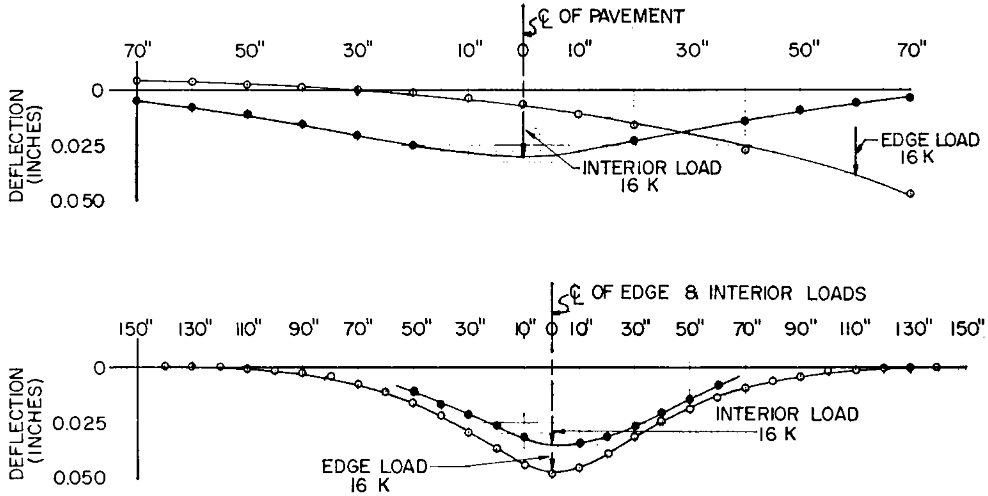

Figure 14. Typical transverse and longitudinal deflection curves under static loads.

plate loading at the 16,000-lb load about 50 percent of the tensile strength of the concrete is utilized. As shown on the curve for B68B under rubber tire loading, the tension in the concrete at the design load level of 16,000-lb is very low. Thus, even though, under extreme conditions, some of the prestress of 400 psi may be lost due to subgrade friction, the maximum concrete tensile stress will still be less than 50 percent of the modulus of rupture.

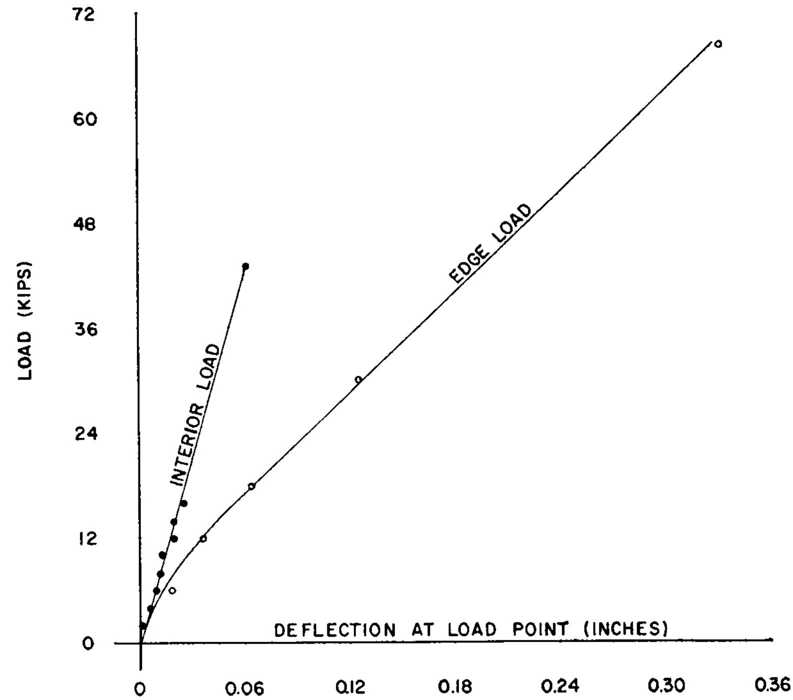

Figure 15. Typical load-deflection curve under cracking loads.

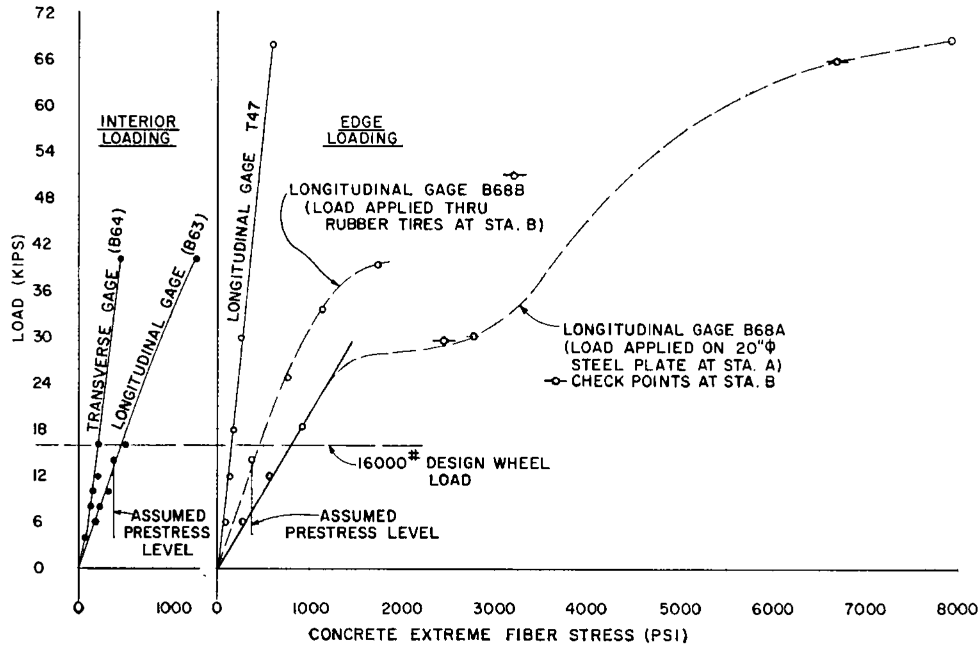

Figure 16. Typical load-stress curve under cracking loads.

The curves confirm that the edge loading condition is critical. The stresses for interior loading for both longitudinal and transverse loading are at very conservative levels. These tests indicate that millions of load repetitions well in excess of the 16,000-lb wheel load can be withstood without fatigue failure. Further, the existence of initial shrinkage cracks in a pavement will not result in subsequent additional cracking.

Edge loadings of 40,000 lb were applied at both the quarter point load stations with the tires. It was considered unsafe to exceed 40,000 lb on the tires for fear of bursting them. It was at this point that the steel bearing plate was substituted. At 45,800 lb, vertical cracking at the bottom of the edge became visible. As the load was increased, this crack progressed vertically toward the top of the slab and transversely toward the center of the slab. The concrete strainometer located beneath the load revealed that the crack had progressed to the loading point at 66,000 lb. At this time the vertical edge crack had developed to within 1 in. of the slab's surface. Simultaneously a half-moon or crescent-shaped surface crack (Fig. 17) appeared around the circular bearing plate.

The deflection at the edge of the pavement just adjacent to the bearing plate was 0.33 in. at this time. The deflection curve along the edge is shown in Figure 18, together with the recovery curve. This load, although four times the design load, was still not a true ultimate or destructive load, because upon removal of the 66,000-lb load the slab immediately recovered 90 percent of the total deflection, the vertical cracks at the slab's edge immediately disappeared, and the surface cracking was significantly reduced. Most important, within two to three days after this test, all surface cracking had disappeared.

Identical phenomena were observed at each of the two edge load points. The loads, strains, deflections, and crack dimensions were practically the same. These tests show that a longitudinally prestressed slab provides ability to recover, even when subjected to severe overloads.

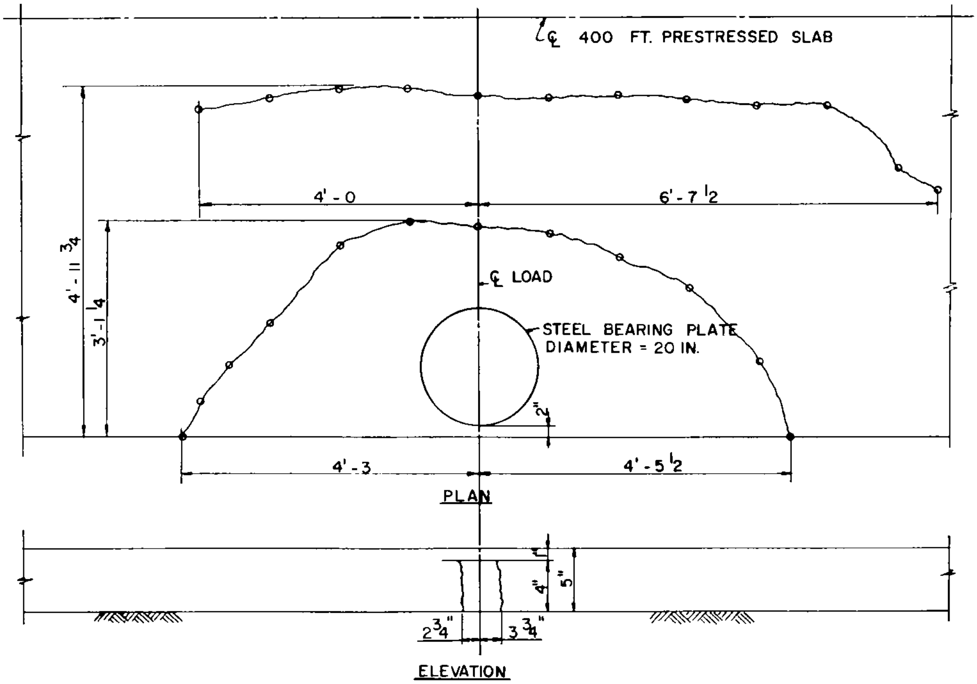

Figure 17. Crack pattern developed under 66,000-lb edge load.

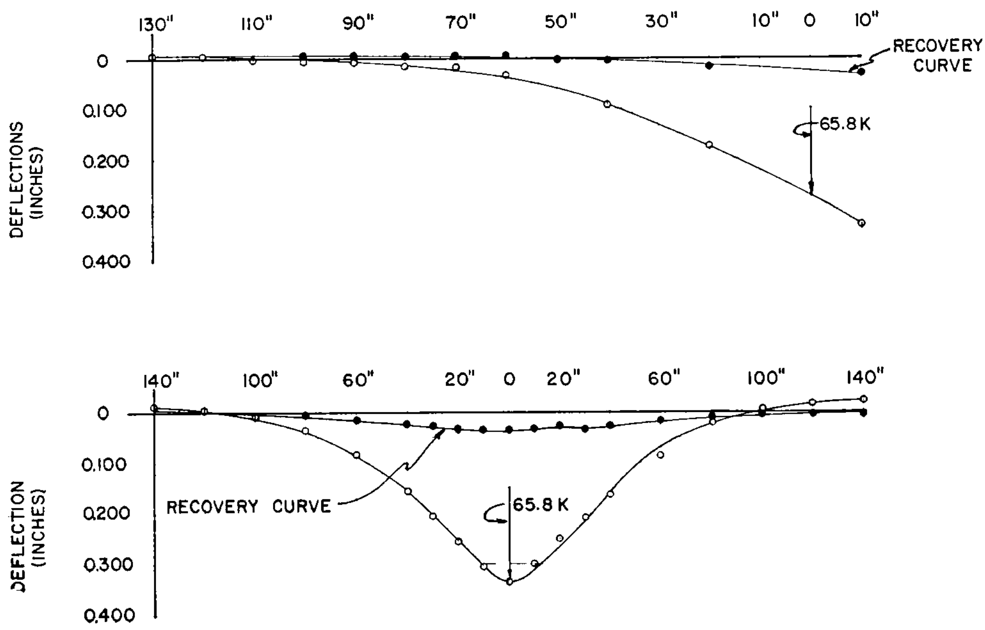

Figure 18. Transverse and longitudinal deflection curves under cracking load.

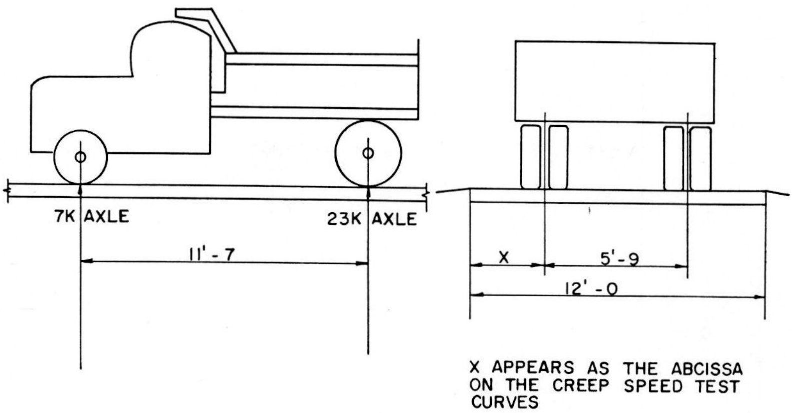

Figure 19. Weight and dimensions of test vehicle.

Creep Speed Load Tests

A total of 16 creep speed load tests was conducted on the experimental slab. A dump truck with dual rear wheels was loaded with sand until the rear axle load was 23,000 lb. Dimensions of the vehicles are shown in Figure 19. The truck was driven along the slab in various transverse paths. Strains in the concrete, and vertical deflections at the edge, were recorded with oscillograph equipment. Figure 20 shows the test set-up, with the truck making a pass.

The first series of tests was done at one of the quarter point load stations to determine typical behavior of the slab at a considerable distance from a joint. Figure 21 shows the edge deflection at the quar-

Figure 20. Typical creep speed load test for edge deflection.

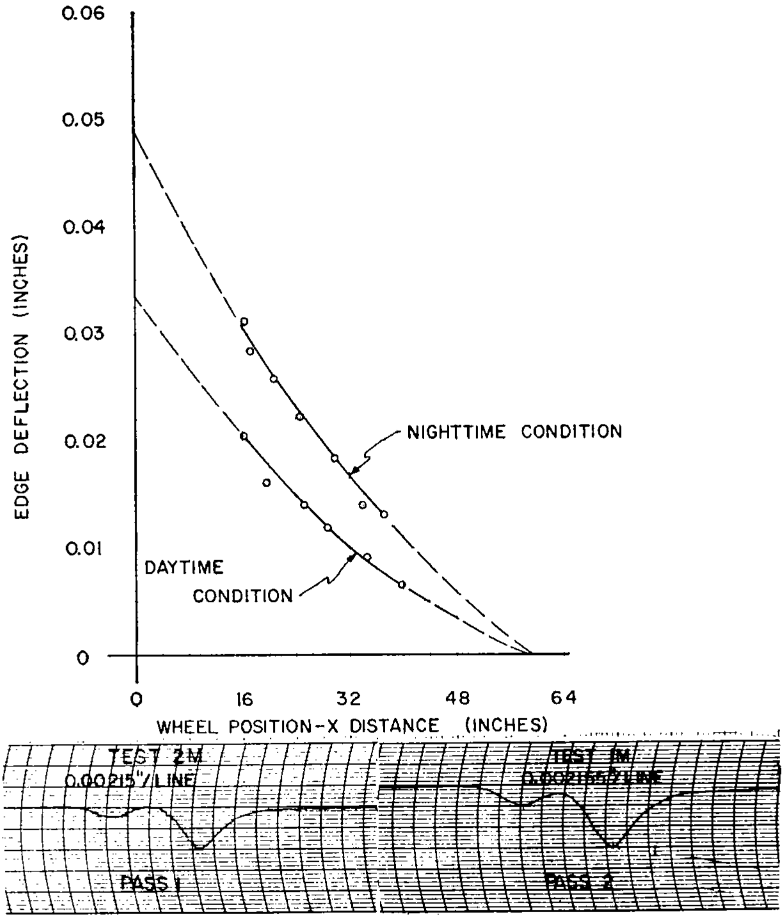

Figure 21. Edge deflection at quarter point load station under creep load.

ter point of the 400-ft pavement. The bottom curves are typical oscillograph recordings for two passes of the test vehicles. The small deflections to the left of the larger ones represent the effect of the front wheels passing the instrumented location. The portion of the curve to the right of the point of maximum deflection is a true influence line for the rear axle load; the reason is that at the instant the rear wheels are at the instrumentation point, the front wheels are 11 ft 7 in beyond the point and thus have no measurable influence.

The upper curves show the effect of positioning the test truck in various paths transversely. Points on the curve were obtained from the peak oscillograph values for different passes. Under normal traffic conditions the X distance (see Fig. 19) would be greater than 2 ft for 92 percent of the heavy commercial vehicles (*8*). Thus, for 92 percent of truck traffic this pavement would be deflected, at most, by the amount indicated on the curve at $X = 24$ in. As can be seen from the curves (projected) this deflection is in the order of one-half that which extreme edge loading would produce.

The magnitude of load stress in the transverse direction was given particular study in the moving load tests. The static load tests had shown that the longitudinal prestressing had a definite beneficial effect in rectifying load cracking after it had developed due to edge loading. It was, nevertheless, desirable to determine these stresses under moving load. The results of this study are shown in Figure 22. The two strainometers shown were buried ½ in. from the bottom of the 5-in. thick concrete. Obviously, the maximum tension should occur when the wheel is directly over the gage. The slight horizontal misalignment in the curves may have been due to the mislocation of the gages in the concrete, from the theoretical location on the center line, or from an error in measuring the location of the truck wheel as it passed.

The shapes of the two curves are essentially similar, but the transverse bending stresses are from 100 to 200 psi less than the longitudinal. The compression stress in the transverse direction at the bottom of the slab occurred as the rear wheels straddled the center line of the pavement where the gages were located. It should be noted that the magnitude of the tensile stress in the transverse direction is relatively small (+175 psi). This value must be combined with the restrained warping stress developed in the transverse direction.

Longitudinal tensile stresses at the edge are shown in Figure 23 in combina-

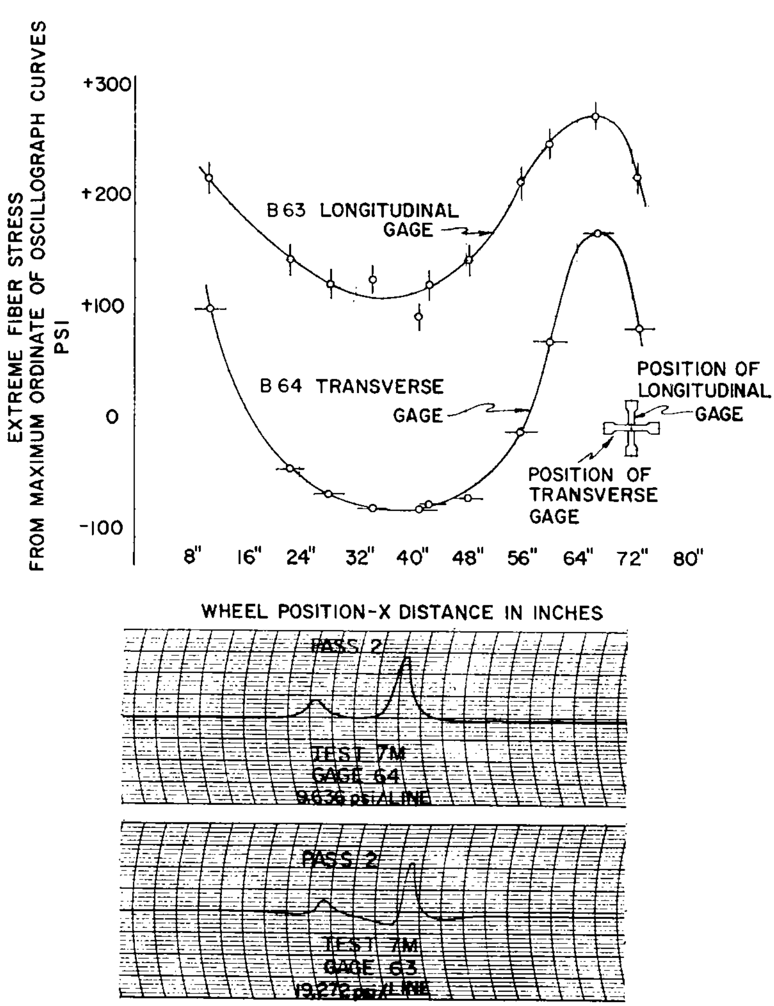

Figure 22. Stresses at quarter point load station interior gages under creep load.

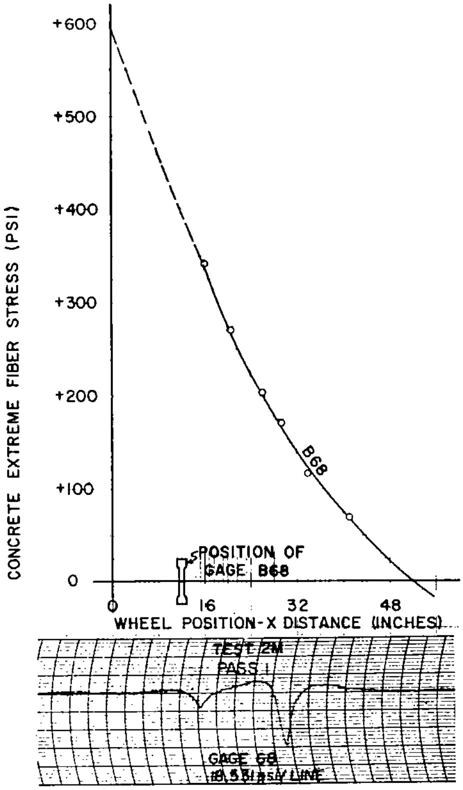

Figure 23. Stress vs X distance for gage B68.

tion with one of the typical oscillograph recordings from which the transverse distribution curve was obtained. If the pavement is elastic under the moving load, the concrete stress should be proportional to the deflection. If points are taken from deflection vs X (Fig. 21) and f_c vs X (Fig. 23) for the same X, and f_c is plotted against deflection, the curve in Figure 24 is obtained. The point where the deflection was measured was 2 in. from the extreme edge, whereas the point where the strain was measured was 12 in. from the edge. It can be seen that this plot is virtually a straight line.

Variation of transverse bending stress at a point 3 ft 6 in. from the pavement edge is shown in Figure 25. The stress reverses, as would be expected. It should be noted, however, that the magnitude of the stresses is quite low compared to the modulus of rupture of the concrete.

To evaluate the action of the sleeper slab under moving load, extensive oscillograph recordings were made in this vicinity. A technique was devised which used two deflectometers to simultaneously measure the deflection of the sleeper slab and the main slab with a single passage of the load. Figures 26 and 27 show the results of these tests. It is significant that the vertical spread between these curves is small. Thus, for all practical purposes the two slabs deflected together for various transverse positions of the load both at the center line of the sleeper slab and at the end of the sleeper slab.

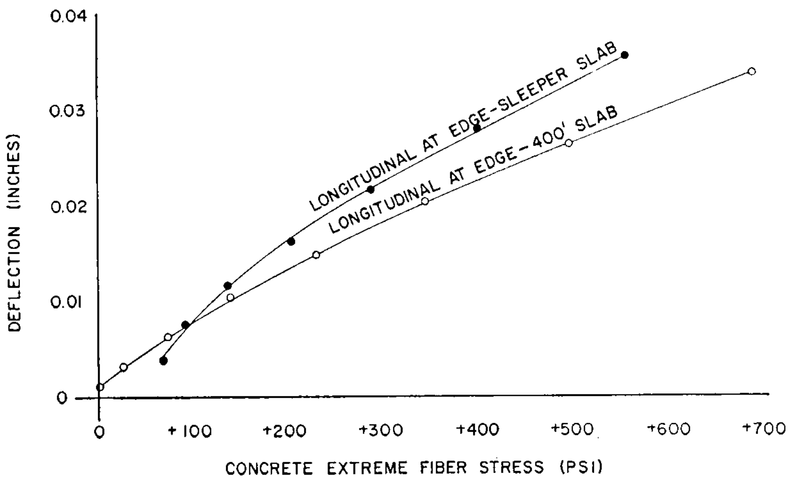

Figure 24. Deflection vs longitudinal stress at edge, sleeper slab and interior location.

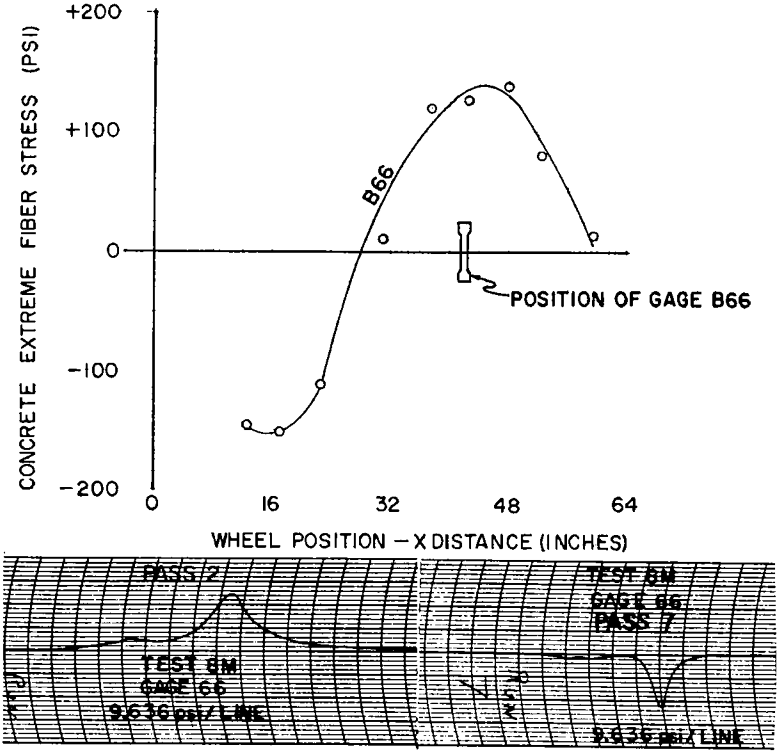

Figure 25. Intermediate gage at quarter point under moving load.

Figure 28 superimposes the oscillograph recordings. Both cases are for a truck wheel near the edge of the pavement. The difference in ordinates between these two curves represents the differential deflection of the pavement and the sleeper slab as the truck travels down the roadway. In addition, the oscillograph recording for edge deflection at the quarter point load station is superimposed on these curves. At first glance one might conclude that there is considerable difference between the continuous prestressed slab and the discontinuous joint; but, allowing for the exaggerated vertical scale, there is some similarity between the deflection characteristics of the continuous slab edge and the discontinuous joint.

The curve for the sleeper slab shown in Figure 24 indicates, as does that for the 400-ft slab, that the sleeper slab behaves elastically.

Thus, it can be concluded that the rubber joint-sleeper slab combination has a tendency toward providing a partial continuity at the joint. Because no traffic test was run on this pavement, the behavior of this joint under traffic is not known. It is understood that joints of similar design are to be constructed in a project in Maryland, which will provide a time service test of the rubber joint sleeper slab expansion joint design.

Additional data confirming that the sleeper slab behaves in a manner similar to the continuous slab are shown in Figure 29. The variation in transverse and longitudinal stress with passage of moving load is plotted for gages B-97 and B-98 (see Fig. 11). These two gages correspond to gages B-63 and B-64 in the main slab (Fig. 22). The similarity in the shape of the two longitudinal and the two transverse curves is apparent.

This same reasoning applies to the longitudinal gage in the bottom edge of the sleeper slab and the main slab. The

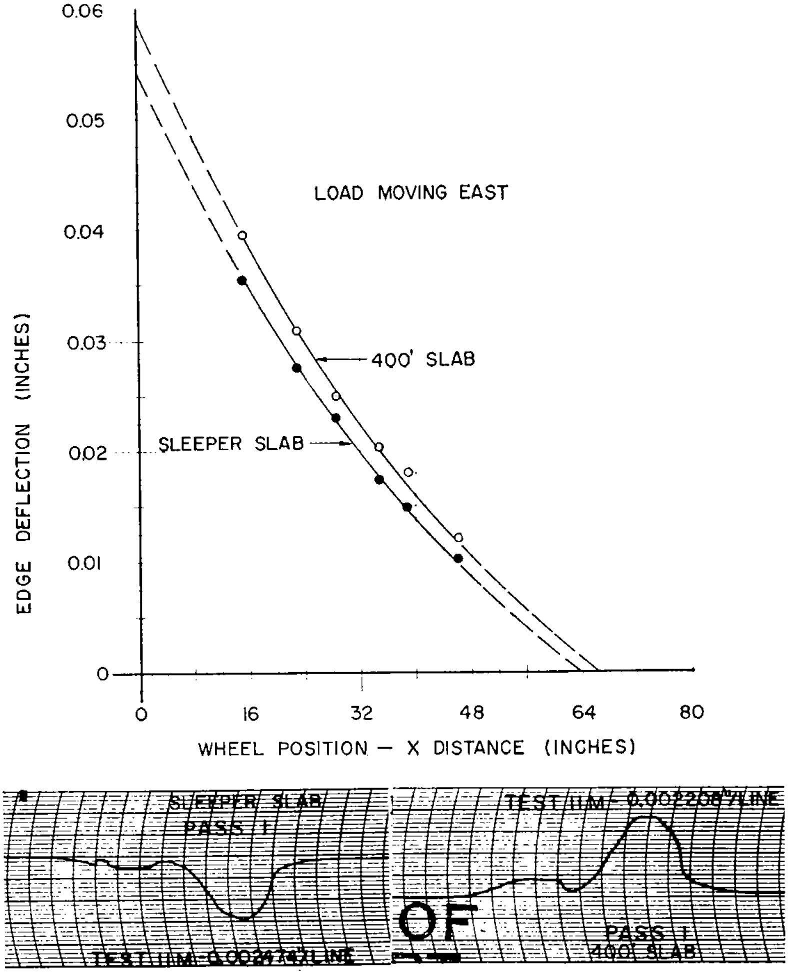

Figure 26. Simultaneous deflection of sleeper slab and 400-ft slab under moving loads.

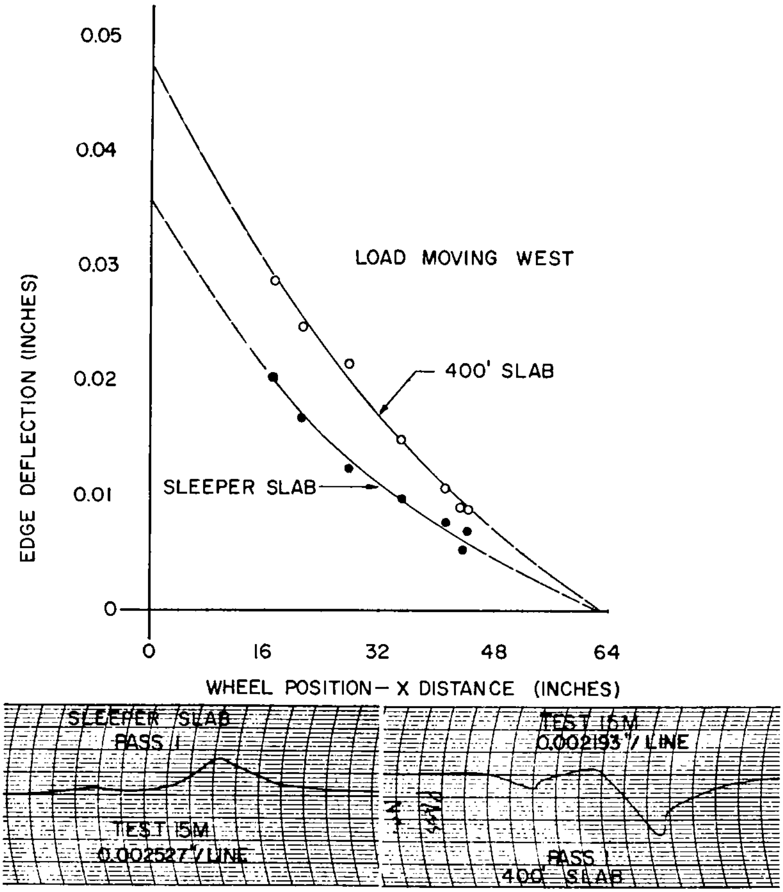

Figure 27. Simultaneous deflection of sleeper slab and 400-ft slab under moving loads.

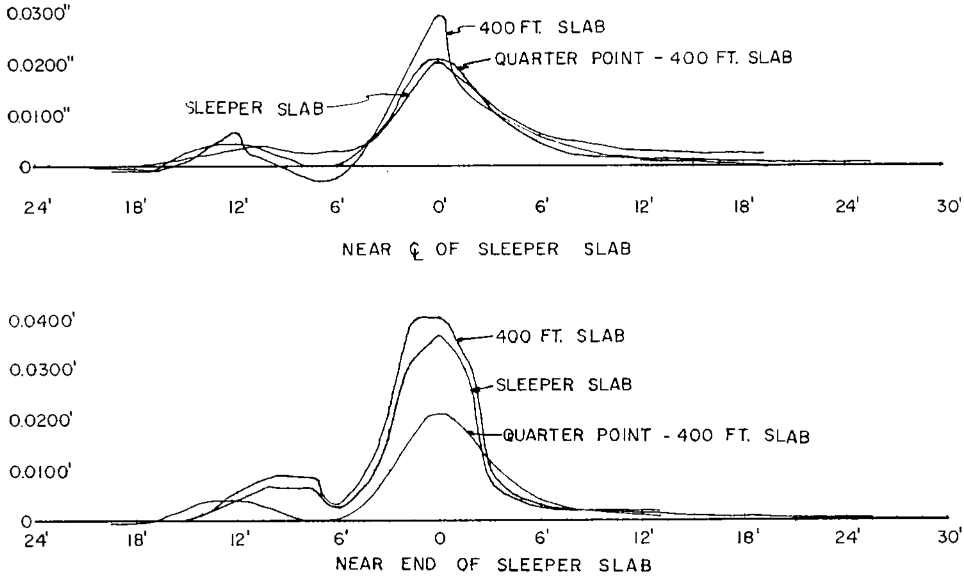

Figure 28. Comparison of deflection at expansion joint and quarter point.

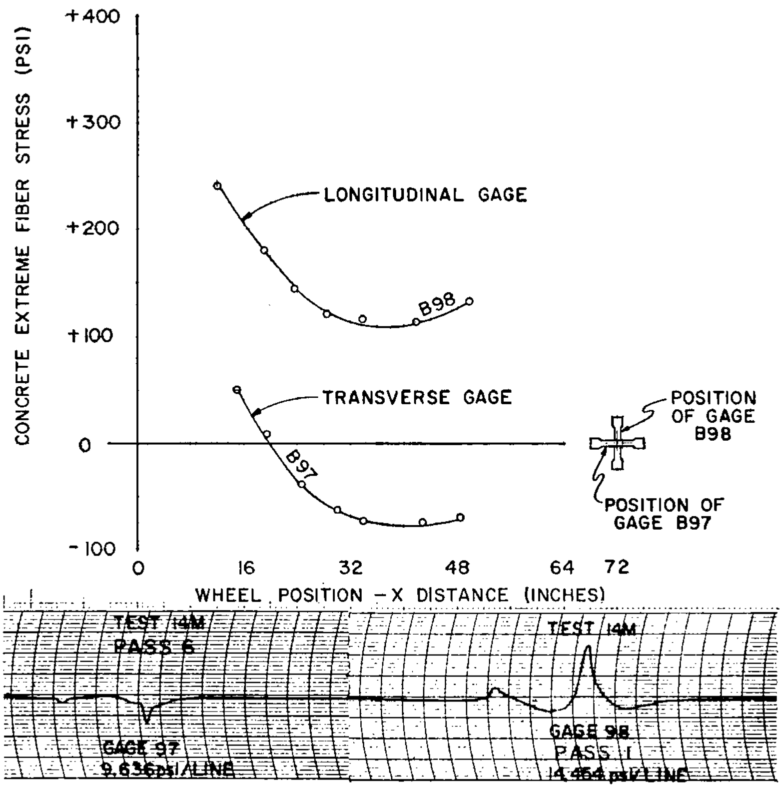

Figure 29. Stress vs X distance for interior gages in sleeper slab under moving loads.

curves for gage B-68 at the quarter point of the main slab are shown in Figure 21 and the curve for gage B-94 at the sleeper slab center line (Fig. 11) is given in Figure 30.

Oscillograph recordings were made for all of the gages in the sleeper slab shown in Figure 11. No valid readings were obtained for gage B-95 so it is not possible to compare it with B-66 in the 400-ft slab. The additional gages shown in the sleeper slab indicated small strains, as one would expect, because they are near a free end of the slab. The maximum recorded strain in this group of end gages corresponded to a concrete stress of 50 psi.

Table 8 shows the ratio of stress produced by the moving load to the stress produced by an equal static wheel load. An 11.5-kip static load was taken off the curve for the comparison inasmuch as this was the wheel load of the test vehicle.

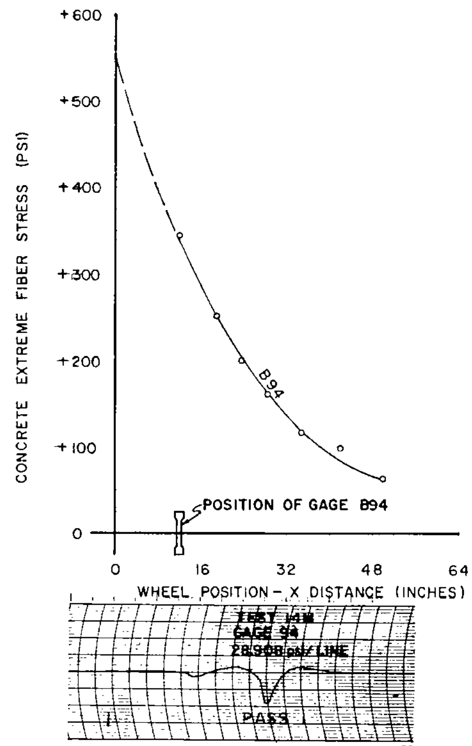

Figure 30. Stress vs X distance for edge gage in sleeper slab under moving loads.

TABLE 8

COMPARISON OF CONCRETE EXTREME FIBER STRESSES DUE TO MOVING AND STATIC LOADING (WHEEL LOAD = 11.5 KIPS)

Location and Direction of Stress	Extreme Conc. Fiber Stress, psi		
	Moving Loading	Static Loading	Ratio, Moving Static
Longitudinal, 1 ft from edge, 400-ft slab	+ 422	+ 555	0.76
Longitudinal, center of slab, 400-ft slab	+ 280	+ 369	0.76
Transverse, center of slab, 400-ft slab	+ 175	+ 185	0.94
Longitudinal, 1 ft from edge, sleeper slab	+ 285	+ 368	0.77

It can be seen from this that the moving load produces less stress than the same static load. This appears to result from the pavement and subgrade not having time to react at a given point before the load has passed.

Warping Study

In the warping study the temperature gradient in the slab and the deflected shape of the slab at two locations were obtained simultaneously (as nearly as practical). One of these locations was near the center of the slab, the other at the end adjacent to the rubber joint (see Fig. 31). The temperature gages, which were very sensitive electrical resistance thermometers utilizing SR4 instrumentation, were located within the depth of the slab as also shown in Figure 31. The sensitivity of these gages can be realized from the fact that 0.000164 in. of indicated strain corresponded to a 1 F temperature change. The temperature warping changes in elevations were determined with the clinometers, which are sensitive to 0.0001 in.

Because the differential temperature was actually measured between gages 86 and 83, which were 3¾ in. apart, the assumption had to be made that the total differential was proportional to depth. Table 9 shows the variation of the differential thus computed with time. The maximum daytime gradient of 1.84 F per in. compares with 2.18 F per in. for 6-in. slabs in the Arlington study (*9*). This is based on an average for September 1, October 11, and November 4 taken from Table 1 of the Arlington report.

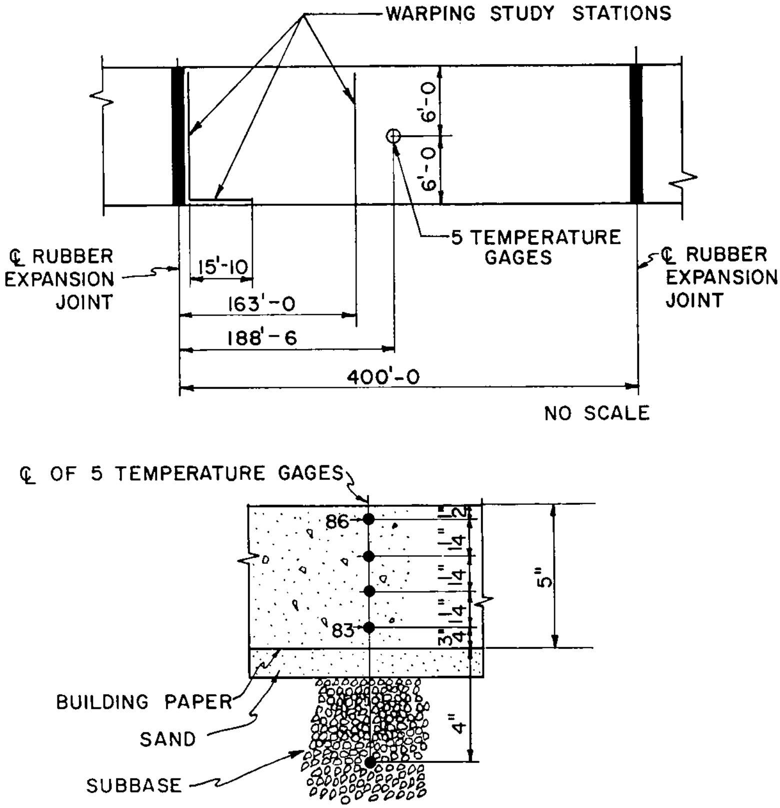

Figure 31. Scope of warping study.

TABLE 9

TEMPERATURE GRADIENTS IN 5-IN. PRESTRESSED SLAB

Date	Time	Temp., °F: Top, Gage 86	Temp., °F: Bottom, Gage 83	Temp., °F: Diff.[1]	Gradient,[1] °F/in.
9/25/57	1:42 PM	87.7	80.8	+ 6.9	+ 1.84
	3:22 PM	89.4	84.6	+ 4.8	+ 1.28
	3:39 PM	88.2	84.8	+ 3.4	+ .91
9/26/57	12:37 AM	65.2	67.8	− 2.6	− .69
	1:33 AM	64.3	67.3	− 3.0	− .80
	2:43 AM	63.3	66.7	− 3.4	− .91
	5:10 AM	61.6	65.1	− 3.5	− .93
	1:09 PM	80.9	74.9	+ 6.0	+ 1.60
	2:14 PM	82.1	78.3	+ 3.8	+ 1.01
9/27/57	1:25 PM	77.0	70.9	+ 6.1	+ 1.63
	2:13 PM	78.7	73.9	+ 4.8	+ 1.28
10/1/57	2:39 PM	83.4	80.3	+ 2.9	+ .77
	4:41 PM	83.7	81.3	+ 2.4	+ .64
	5:02 PM	79.6	80.3	− .7	− .19
	5:30 PM	77.7	79.5	− 1.8	− .47
	6:12 PM	74.5	77.7	− 3.2	− .85

[1] + = top warmer than bottom; − = top cooler than bottom.

Total warping movements are plotted in Figure 32. The longitudinal and transverse condition at the corner represents a change in gradient from +0.64 F per in. to −0.85 F per in., or a total change of 1.49 F per in. Because the upper surface was cooling, the end and edge were both curling up and the zero line is below the total deflection points. Similarly, curling at the transverse section near the center of the 400-ft slab shows an upward movement of the edge. The change in gradient was from +0.77 F per in. to −0.47 F per in., or a total change of 1.24 F per in.

The maximum movement at the edge was 0.014 in. for a gradient of 1.24 F per in.; the maximum at the corner adjacent to the rubber joint was 0.024 in. for a gradient of 1.490 F per in. In the case of the 6-in. slab tested at Arlington Farms (*9*) with a gradient of 3.67 F per in., the edge deflection was 0.015 in. and the corner deflection was 0.039 in. It can be seen that the 5-in. slab warps more for the same temperature gradient. This may be because the 5-in. slab weighs less and is thus freer to curl. It does not mean that the warping stresses are more, however, because it is well known that warping stresses increase with thickness (*10*).

Another interesting item is that the ratio of corner curling to free edge curling for the 6-in. slab is 2.6, whereas it is only

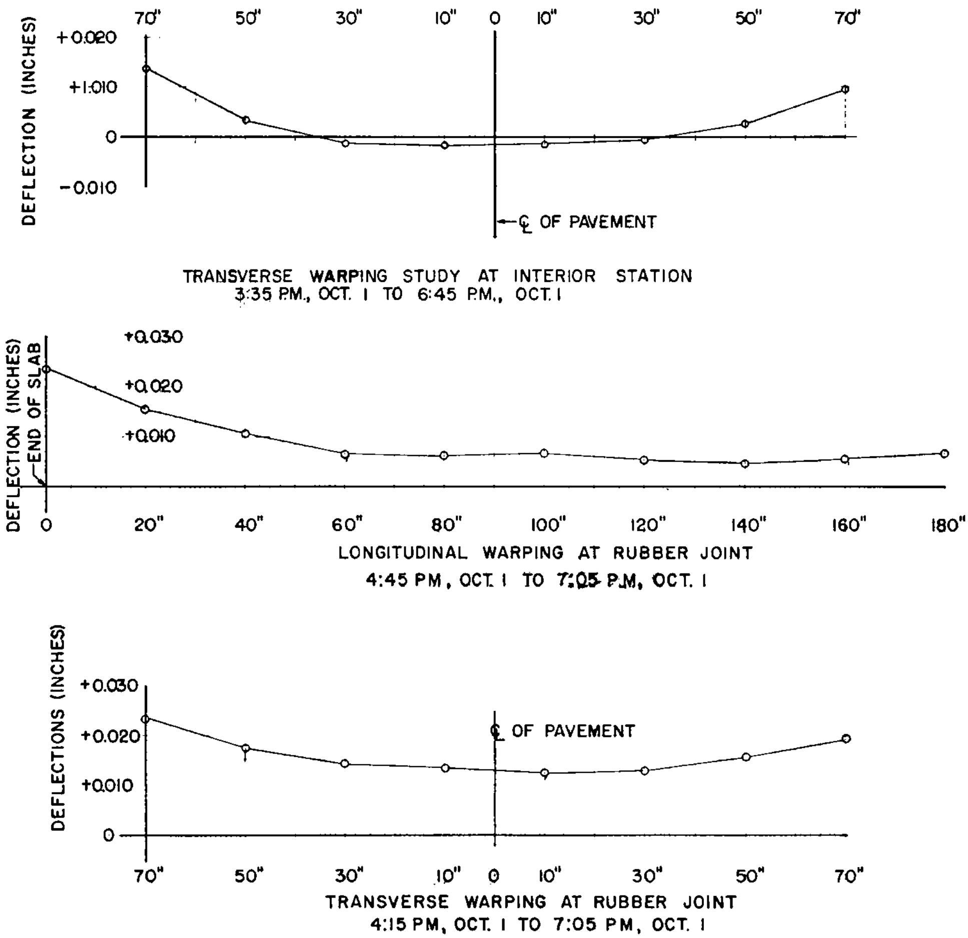

Figure 32. Total warping movements.

1.7 for the 5-in. slab. This leads to speculation that the rubber expansion joint may have had some influence in reducing the corner movement in the 5-in. slab.

TEST OF NEOPRENE RUBBER EXPANSION JOINT

Comprehensive testing was conducted to determine whether the new joint would meet the specifications on serviceability, load transfer, and accommodations of horizontal movement.

Neoprene rubber of the proper composition is well known for its superior resistance to attack from all of the expected deteriorating agents. Tests were run at the B. F. Goodrich plant to supplement data of known authenticity. Low-temperature tests were run to confirm the resiliency of the rubber under conditions which may be met in the field.

Static testing at the rubber expansion joint demonstrated the capacity of the joint to transmit load to the sleeper slab. The tests showed that the load was transferred through the joint to the sleeper slab with practically no effect on the adjacent pavement ends. Figure 33 shows the load-deflection and load-concrete stress relationship in the sleeper slab for interior and edge loading directly over the rubber expansion joint. It is interesting to note that the vertical deflections were only 0.08 in. at the interior and 0.126 in. at the edge, even though the loads were for 150 percent of the design load. These deflections were taken on the rubber surface of the joint at the center

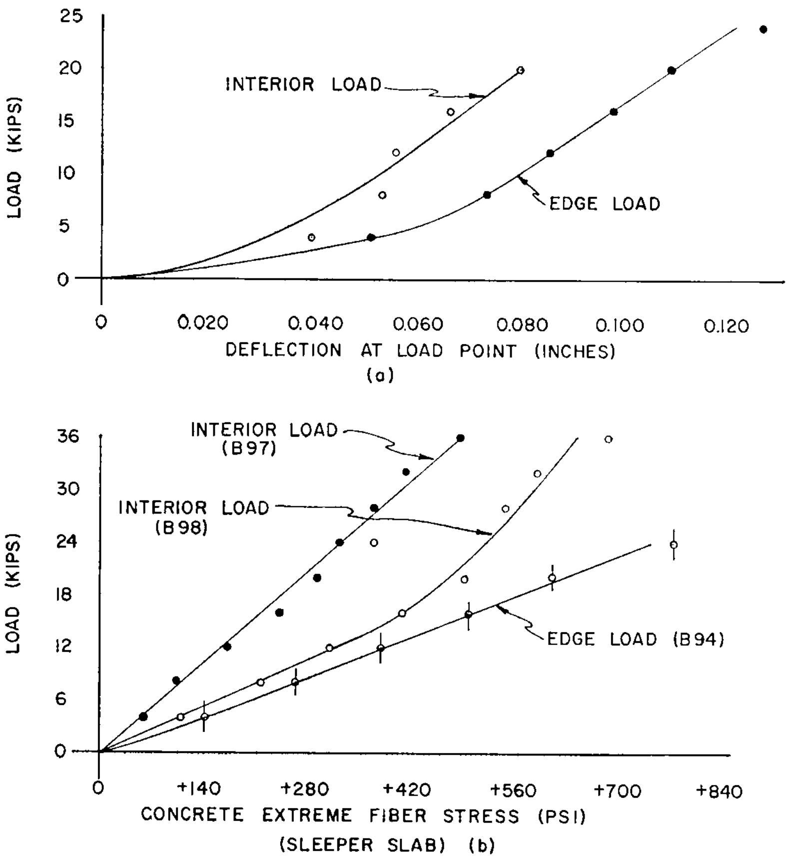

Figure 33. Load-deflection and load-stress relationships at rubber expansion joint.

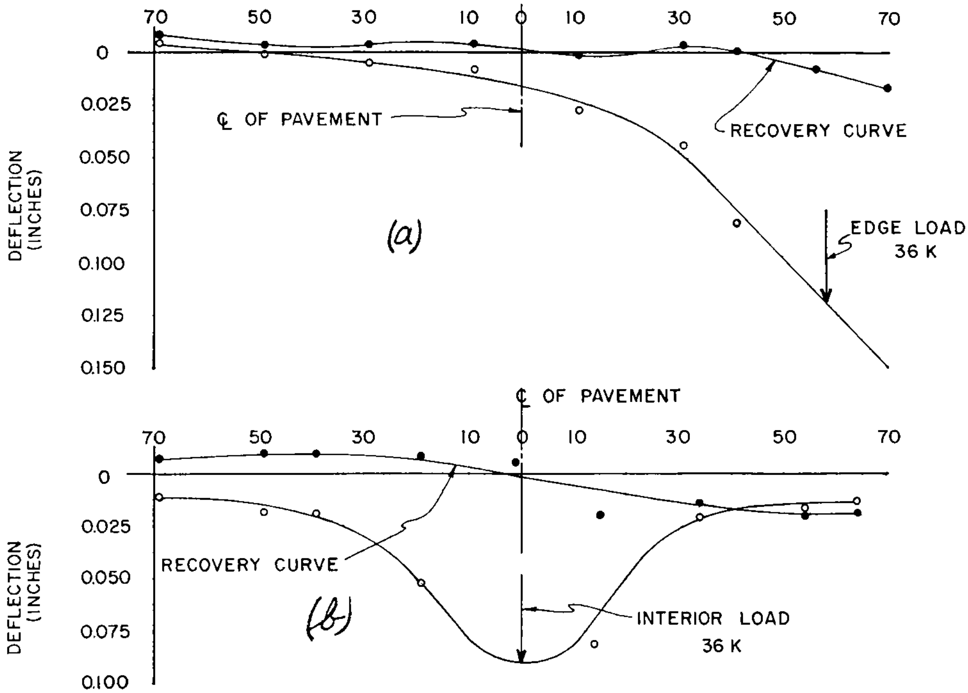

Figure 34. Transverse deflection of top of rubber expansion joint under static loads.

of application of the load. The deflections represent the sum of the deflection of the rubber joint and the structural deflection of the sleeper slab.

Surface deflections of the rubber joint under 36-kip edge and interior loading are shown in Figure 34. The high degree of recovery should be noted, as well as the relatively small deflections under this extreme loading. Again, these deflections represent the sum of the deflection of the rubber joint and the structural deflection of the sleeper slab.

The adequacy of the joint in transmitting loads vertically into the sleeper slab was shown on the moving load test oscillographs. Motion pictures of a truck wheel moving over the joint demonstrate that the deflections are small.

The efficiency of the joint in accommodating the required movement is shown by Figure 35. The expansion joint tested is not what could be considered a "production run" specimen. The manufacturer plans to construct this joint in 6-ft sections when commercially produced. Because of the experimental nature of the test specimen it was necessary to construct it of short (14-in.) sections bolted and cemented together into the required length; consequently wide variations were expected. However, Figure 35 shows that the maximum rise at one point was only about 0.18 in. when compressed 3 in. The average rise measured at 30 points was 0.11 in.

The originally planned specification for the expansion joint, allowed a surface movement of 0.125 in. above and below a normal level, or a total of 0.250 in. As shown in Figure 35 the average rise was less than one-half the total permitted. Even the maximum rise of 0.18 in. at the single point is less than the specified total. It can be expected that production runs can be held closely to the 0.11 average rise shown. As a result there is good assurance that the surface variation limit of ±0.125 in. will be readily complied with in practice.

Figure 36 shows the force required to compress the joint. It is noteworthy that

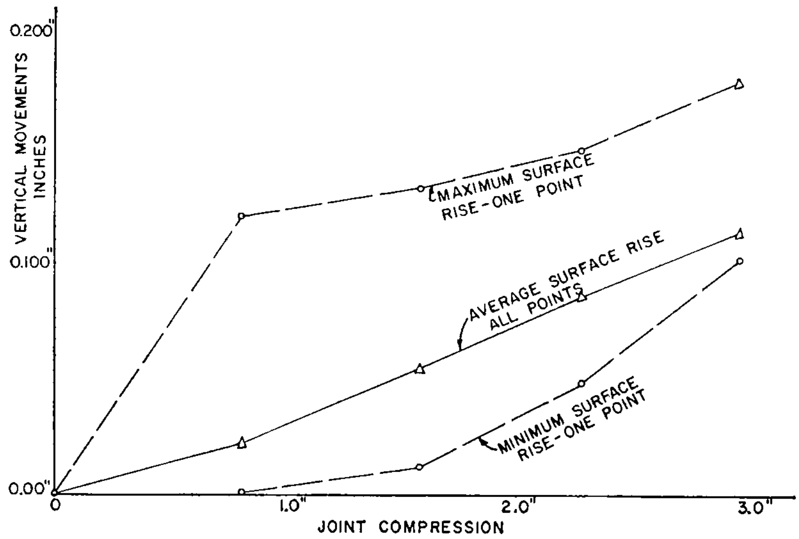

Figure 35. Vertical movements of top of rubber expansion joint.

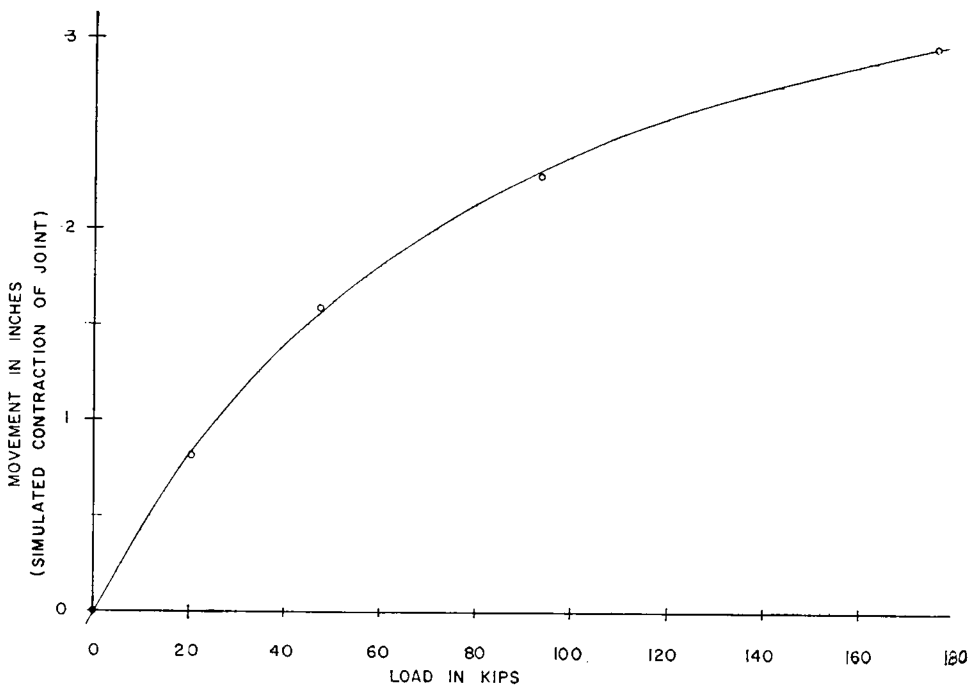

Figure 36. Horizontal movement of rubber joint under jacking loads.

180 kips were required to compress the joint the full 3 in., equivalent to 250 psi in the concrete. This stress will tend to supplement the prestress force at high ambient temperatures. Provided vertical curves are not excessive, the force will not be sufficient to produce blow-ups; on horizontal curves the transverse component of the force will not be sufficient to force the pavement out of line.

It will be necessary to compress the joint during the jacking operation an amount required by the slab temperature at the time of jacking. Sufficient jacking capacity must be provided so that the anticipated maximums can be obtained.

From the data obtained in the study of slab expansion and contraction (Table 1), it is considered necessary to provide for only one-half of the calculated temperature correction.

To perform this full-scale compression test of the expansion joint, a 30-ft length of prestressed pavement slab was constructed west of the main 400-ft section. Through the use of hydraulic jacks reacting against a pile of steel slabs, it was possible to obtain the necessary jacking effort. Loads were measured with SR4 load cells.

The results of laboratory testing of prototype sections and field testing of three different full-sized joints may be summarized as follows:

1. The joint is adequate to transmit heavy vertical loads.
2. The joint will accommodate a 3-in. movement. This is more than adequate to provide for the required expansion.
3. The joint sealed the pavement against penetration by moisture and dirt.
4. The rubber used in the joint was made of compounds designed to withstand attack from the elements.
5. The riding qualities are excellent. It is smoother than most mastic joints found in conventional concrete pavements.

PRACTICAL DESIGN PROPOSAL

Obviously all of the design, construction, and testing involved in the experiment were pointed toward development of a method which can be translated into a practical system for constructing prestressed concrete pavements for traffic use. It also is evident that further testing by installation of prestressed concrete in typical traffic test strips a mile or more in length will be necessary. The experience with the experimental slab can be invaluable for the purpose of designing such test strips.

Although some of these experiences have been mentioned, it will be useful to describe some of them in detail, as well as some not previously mentioned. The experiment has provided solutions to some of the problems involved in prestressed concrete pavement construction. Solutions to other problems are proposed.

It is proposed that the anchoring slab be eliminated and the jacking be done from above the pavement. This would simplify the operation and result in several economies. A large manufacturer of jacking equipment has made preliminary studies which indicate the feasibility of carrying in the jack cylinder the large bending moments resulting from the eccentricity of the jacking force. The pavement could be placed with no interruption for the full length of a section except for a bond-breaking bulkhead at the jacking points. The jacks would work in recesses provided by steel pipe inserts placed in the pavement.

Obviously, it is preferable to make the prestressed pavement unit lengths as long as possible. The experiment has shown that increasing the length beyond 400 ft is practical. Lengths of 600 ft, or even 800 ft, can be used. The value of the coefficient of friction is reasonably well established. By ordinary care in construction the coefficient may be held to about 0.70. Measurements of the dimension changes occurring with temperature and time have been taken. The required expansion in an 800-ft length can be readily accommodated by the joint designed for a 3-in. movement.

Should the length of unit be increased to either 600 ft or 800 ft, it will probably be necessary to gap jack each section at two points to limit the throw of the jacks, because the manufacturers of jacks consider a 3-ft or 4-ft throw impractical. Jacking at two points in each unit would not be without advantage, as it would provide a good means of controlling the geometry of the jacking operation.

It should be pointed out that the placement width for prestressed pavements will have to be two lanes: it is not practical to individually jack 12-ft lanes of concrete pavement placed side by side. This presents no problem, inasmuch as a 24-ft wide pavement can be readily gap jacked.

The neoprene rubber expansion joint used in the experiment performed so satisfactorily that it undoubtedly can be used without change except for minor modifications to adapt it to practical full-scale construction. The rubber joint in combination with the sleeper slab offers a promising solution to the difficult expansion joint problem.

A minor point worth mentioning was encountered during the construction of the experimental slab. In the jacking operation during which the 200-ft half sections of pavement were each moved about 12 in., the west end raised about 3⁄8 in. It was possible to jack the elastic prestressed slab vertically off the sleeper slab for inspection. It was found that dimples produced by the action of a vibrator were the cause of this trouble. This can be avoided by using a short piece of 1⁄4-in. plywood on the subgrade adjacent to the sleeper slab. Incidentally, it was possible to grind off the projections under the experimental slab. It is obviously necessary to finish off the sleeper slab as flat and as free from irregularities as possible.

The experimental study has indicated that some reduction is possible in the amount of prestressing steel. It is proposed that the amount of prestressing steel be reduced from the 6.94 lb per sq yd used on the experiment to about 5 1⁄4 lb per sq yd. This is possible for several reasons. First, the steel used on the experiment was not used to the greatest possible efficiency. It is felt that the strains should be made sufficient to produce initial stresses up to 75 to 80 percent of the ultimate. Second, the steel in the experiment was not placed accurately. At each instrumentation station the tendons were hand-placed and were not located accurately. As a result, bends were introduced which resulted in high friction losses, as shown in Figure 10. In field practice this difficulty resulting from instrumentation would not be encountered. Finally, the test results show that excess bending strength existed in the slab.

To secure additional economy it is recommended that each tendon consist of seven 0.250-in. diameter stress relieved wires. This will provide the recommended amount of 5 1⁄4 lb per sq yd of pavement. Because wire is less costly than strand, obvious economy will result. Previous applications of wire where anchorages through bond were obtained by proper looping in the concrete prove that no particular difficulty need be anticipated.

In a full-scale project the tendons would arrive on the construction site fully fabricated to exact length and complete with enclosing duct, looped ends, and grout fittings. Each tendon would be delivered complete in coil form. These would be uncoiled onto the subbase, then positioned in the concrete exactly as was done in the experiment.

The character of a practical full-scale prestressed concrete highway pavement may be outlined as follows:

1. The pavement unit would be 24 ft wide by 5 in. thick. Unit lengths would be at least 600 ft and could very well be made 800 ft.

2. The tendons would consist of stress relieved high-strength wire. The amount of wire used would be about 5 1⁄4 lb per sq yd. Tendons would be delivered to the site in coils, fully fabricated.

3. The basic gap jacking practice used on the experiment would be utilized, except that jacking would be made an above-the-pavement operation.

4. The neoprene rubber and flexible sleeper slab solution to the expansion joint problem would be used.

This system offers a satisfactory solution to the problem of obtaining a highway pavement free of cracks with desirable elastic qualities.

ECONOMICS

It was recognized that any construction would have to be virtually competitive with conventional reinforced concrete highway construction. Although the construction might prove superior to conventional concrete pavement, both in load-carrying ability and in maintenance cost, it would be difficult, if not impossible to specify it if the cost materially exceeded the cost of the conventional construction.

Several differing forms of prestressed concrete construction were explored which were estimated to cost excessively. The form used in the experiment was chosen because preliminary estimates showed good promise of providing a pavement which could be constructed competitively with conventional 9- or 10-in. thick pavement.

Subsequent to the time when the contractor had completed the 530 ft of 12-ft wide pavement on the experiment, he was asked to carefully estimate costs of a practical paving project of typical scope. This was made on the assumption that design modifications indicated by the results of the test program would be incorporated and that the construction would include methods suitable for a large construction project.

The estimates (Table 10) were made for construction as follows:

1. The project would involve 4-lane highway for a length of approximately 5 miles.
2. The pavement would be dual lane, of 24- or 25-ft width.
3. Tendons would be supplied fully fabricated, with flexible duct applied, in coils, to the contractor. The tendon weight per square yard would be 5.26 lb.
4. Supplementary steel reinforcing would be limited to 2 lb per sq yd (average).
5. Jacking would be simplified by the

TABLE 10

ESTIMATED COST (IN $ PER SQ YD OF PAVING) OF PRESTRESSED CONCRETE HIGHWAY PAVEMENT 25 FT WIDE AND 5 IN. THICK

Item	400-Foot Units				600-Foot Units				800-Foot Units			
	Material	Equipment	Labor	Total	Material	Equipment	Labor	Total	Material	Equipment	Labor	Total
Concrete in slab [1]	2.13	0.27	0.61	3.01	2.13	0.27	0.61	3.01	2.13	0.27	0.61	3.01
Sand layer, 1 in. anti-friction	0.13	0.01	0.03	0.17	0.13	0.01	0.03	0.17	0.13	0.01	0.03	0.17
Paper layer under slab	0.21	—	0.01	0.22	0.21	—	0.01	0.22	0.21	—	0.01	0.22
Tendons, 5.26 lb per sq yd	0.84	0.03	0.10	0.97	0.84	0.03	0.10	0.97	0.84	0.03	0.10	0.97
Flexible duct	0.34	—	—	0.34	0.34	—	—	0.34	0.34	—	—	0.34
Reinforcing 2 lb per sq yd	0.22½ [2]	—	0.10 [2]	0.32½ [2]	0.18	—	0.08	0.26	0.18	—	0.08	0.26
Sleeper slab	0.11½	—	0.02	0.13½	0.08	—	0.01½	0.09½	0.06	—	0.01	0.07
Rubber expansion joint	0.99½	0.02½	0.03	1.05	0.66½	0.01½	0.02	0.70	0.50	0.01½	0.01½	0.53
Concrete patch in gap	0.01	—	0.00½	0.01½	0.01½	—	0.00½	0.02	0.01	—	0.00½	0.01½
Grouting operation	0.16½	—	0.22½	0.39	0.16½	—	0.22½	0.39	0.16½	—	0.17	0.33½
Jacking operation	—	0.02½	0.03½	0.06	—	0.03½	0.04½	0.08	—	0.02½	0.03½	0.06
Stabilized shoulder	0.20	0.03	0.06½	0.29½	0.20	0.03	0.06½	0.29½	0.20	0.03	0.06½	0.29½
Total				6.98				6.55				6.27½

[1] At $14.50 per cu yd, allowing 5 percent waste.
[2] 2½ lb per sq yd.

use of special jacks designed to operate above the pavement and to hold the gap open while the concrete is placed. Such jacks have been found practical and are being designed.

6. The jack anchoring slab can be omitted using revised jacking technique.

The estimate considers three unit lengths: 400 ft (the length of the experimental section), 600 ft, and 800 ft. The design considerations involved in the use of the longer lengths are covered elsewhere in this paper.

The first five and the last items on the cost tabulations are virtually unaffected by length of unit used. The other six items are vitally affected by the unit length.

Attention is drawn to the fact that advantage was taken of the lesser stabilized shoulder cost resulting from reduction of the pavement thickness to 5 in. Table 11 gives cost data for conventional construction for direct comparison. The unit costs were taken from those on a typical project in northwestern Pennsylvania and are identical, for appropriate items, with the unit prices used in Table 10.

Direct costs only are shown. It is recognized that variations in unit costs in different parts of the country would affect the balance. An increased cost of concrete and a reduced cost of field labor would make the construction appear more favorable, and *vice versa.* It is believed that the comparison can be considered fairly typical.

The importance of the cost study lies

TABLE 11

UNIT DIRECT COSTS ON TYPICAL PENNSYLVANIA STANDARD 10-IN. THICK HIGHWAY PAVEMENT [1]

Item	Cost, $ Total	Per sq yd
Concrete, 28,529 cu yd [2]	413,671	4.23
Reinforcing, 97,812 sq yd	86,075	0.90
Labor	59,589	0.61
Equipment rent, repair, fuel	26,542	0.27
Stabilized shoulder		0.49
Total		6.50

[1] Northwest Pennsylvania pavement and shoulder costs; total area 97,812 sq yd.
[2] At $14.50 per cu yd, allowing 5 percent waste.

in the fact that prestressed concrete paving can be constructed within the range of costs currently found for high-class pavements. The improved structural action can thus be obtained without excess cost.

In considering the cost studies shown, it should be remembered that prices of fabrication and field labor on complicated post-tensioned bridge work were not considered appropriate for use on typical large-scale highway construction. It was necessary to project imaginative thinking to ways in which mass production techniques could be utilized. Even in the experimental construction every effort was used to demonstrate the feasibility of applying current highway construction methods in prestressed concrete highway construction.

Until contractors have become familiar with the new techniques involved, favorable unit prices cannot be expected. However, as they become familiar with the work and the economies possible, costs may be expected to stabilize at competitive levels.

The expected economies resulting from elimination of joint maintenance and from the superior protection of the subgrade can be evaluated from the results of traffic strip tests. The increase in pavement life resulting from superior structural action better protection of the subgrade, elimination of any transverse joints at which failure can occur, and improved properties of possible resurfacing can be expected to be appreciable. The very favorable flexibility of the prestressed pavement will permit conformation with subgrade settlement without damage. Even the absence of the possibility of reflection cracks in resurfacing is a point worthy of evaluation.

SUMMARY AND CONCLUSIONS

This experiment in prestressed concrete pavement was conducted with the objectives of determining the structural adequacy, the construction practicability, and the economic feasibility of prestressed concrete highway pavements.

The conclusions may be briefly summarized as follows:

1. Longitudinally reinforced prestressed concrete highway pavement is structurally sound construction.
2. The method used is practical for field application.
3. The construction can be made competitive with standard high-type reinforced concrete pavements when mass production techniques are used.
4. The solution to the difficult expansion joint problem as used on the experiment shows promise of being an excellent one.

ACKNOWLEDGMENTS

The authors wish to express their appreciation for the advice and assistance of the many individuals and organizations who worked on the experiment. Although it is not possible to give individual acknowledgment to all, a few are mentioned because of the importance of their contributions. Among these are the personnel of the Physical Research Division of the Bureau of Public Roads including Harold Allen, Chief, and H. D. Cashell, Highway Physical Research Engineer, for their advice and assistance; the personnel of the Fritz Engineering Laboratory at Lehigh University, including W. J. Eney, Director, and I. J. Taylor, Instrumentation Engineer for the design and fabrication of the instrumentation; the B. F. Goodrich Industrial Products Company personnel for the rubber joint development; and D. H. Camilli, Chief Engineer of Allegheny Contracting Industries, whose personal interest in the construction and help in cost studies were most valuable.

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DISCUSSION

Phillip L. Melville, *Civil Engineer, Airfields Branch, Office of Chief of Engineers.* — The authors have provided the engineering profession with some worthwhile information on the design and construction of prestressed concrete pavements. The Jones & Laughlin highway project was obviously designed for lighter loads than the current airfield investigational pavements at the Rigid Pavement Laboratory of the U. S. Army Corps of Engineers. As a matter of information, Table 12 gives pertinent factors used in the design of both pavements. A more complete description of prestressed concrete airfield pavements was presented

TABLE 12

COMPARATIVE DESIGN FACTORS FOR TEST HIGHWAYS

Item	Corps of Engineers	Jones & Laughlin
Depth, in.	4.4	5
Supporting medium:		
Base pavement, in.	6.0 PCC	—
Subgrade modulus, pci	65	200
Dimensions, ft	60 × 65	12 × 400
Method of prestressing	Pretensioned	Post-tensioned
Type of units	¼-in. wire	7/8-in. strand
Amount of prestress, psi:		
Longitudinal	425	450
Transverse	390	0
Concrete strength, psi:		
Flexural	725	725
Compressive	4,500	3,400

by Director Frank Mellinger of the Rigid Pavement Laboratory at the World Conference on Prestressed Concrete, held in San Francisco in 1957.

Both pavements were tested under static and either dynamic or creep loadings. Although the conditions and magnitude of these loadings are of different order, the results obtained present interesting parallels.

Table 13, prepared by Carl Renz, Chief of the Structures Branch at the Rigid Pavement Laboratory, compares the two test pavements by loading and deflections. A similar comparison for dynamic loading is given in Table 14.

The matter of traffic coverages is of particular interest because the Jones & Laughlin project did not include large numbers of load repetitions. Thus, the statement made by the authors that the prestressed pavement is "entirely adequate to carry the design wheel load" must be related to the number of load repetitions anticipated during the design life of the pavement. Failure of the Corps of Engineers prestressed overlay pavement occurred after 6,200 traffic coverages with a 75-kip load and after 12,000 coverages with a 60-kip load. It is estimated that the pavement had a capacity of 45 to 50 kips for 30,000 load repetitions with a final deflection of ½ in.

Based on the foregoing data and model studies performed by Paul Carlton, Chief of the Research Branch at the Rigid Pavement Laboratory, an evaluation of the Jones & Laughlin test slab indicates an ability to carry an unlimited number of repetitions of the 16-kip wheel load and that the pavement was substantially over-designed for this loading.

The analysis is further verified by the essentially elastic behavior of the slab at the design load. To realize the full load-carrying capacity of a prestressed pavement, the design should be based on the assumption that tensile cracking will occur in the bottom of the slab. These cracks produce: (a) a plastic hinge action in the area through which they extend; (b) an increased deflection due to a reduction in slab rigidity in the cracked areas; (c) a redistribution of bending moments which results in a substantial increase in the negative radial moment. Such action is more correctly elasto-plastic than elastic in nature.

The authors have stated that, with respect to negative moments, their design "utilizes the reserve capacity of the prestressed slab in the unloaded portion." Actually, the magnitude of the negative radial moment can be determined for the elasto-plastic condition of loading. Thus, it is possible to adjust the design so that the ratio between the maximum negative

TABLE 13

COMPARISON OF TEST PAVEMENT DEFLECTIONS UNDER STATIC LOADING

Corps of Engineers Overlay Pavement				Jones & Laughlin Pavement			
Static Load, kips	Deflection, in.[1]			Static Load, kips	Deflection, in.		
	18-In. Plate	12-In. Plate	Twin-Wheel		20-In. Plate	Dual Wheels: Interior	Dual Wheels: Edge
45	0.11	0.15	0.08	16	—	0.02	0.06
100	0.26	0.34	0.21	45	—	0.06	0.21
150	0.49	0.56	—	66	0.33	—	0.31
190	0.70	—[2]	—				

[1] Interior loading over expansion joint in base slab.
[2] Failure of both overlay and base slab by punching shear.

TABLE 14

COMPARISON OF TEST PAVEMENT DEFLECTIONS UNDER DYNAMIC LOADING

Traffic Coverages	Corps of Eng. Overlay Pavement Deflection, in.[1]				Jones & Laughlin Pavement Deflection, in.[2]	
	45-Kip Load		100-Kip Load		16-Kip Load	
	Interior	Joint [3]	Interior	Joint [3]	16 In. from Edge	48 In. from Edge
1	0.01	0.08	0.0	0.21	0.02	0.003
1,000	0.02	0.17	0.17	0.38	—	—
2,000	0.02	0.18	0.30	0.64 [4]	—	—
3,000	0.09	0.17	0.39 [5]	—	—	—

[1] Twin-wheel static loading.
[2] Dual-wheel creep loading; edge deflection.
[3] At expansion joint in PCC base slab.
[4] Failure at 2,068 coverages.
[5] Failure at 3,790 coverages.

moment that the slab can sustain without cracking and the negative moment produced by the wheel load will permit the slab to carry the required number of load repetitions. The relationship between wheel load and negative radial moment has been determined by means of small-scale model tests by Carlton at the Rigid Pavement Laboratory, as well as on a purely theoretical basis by Franco Levi.

BEN MOREELL, JOHN J. MURRAY, AND JOHN E. HEINZERLING, *Closure.*—The extensive and excellent experimental work done by the Corps of Engineers in prestressed airport runways has been extremely interesting. It is especially useful to find, even under somewhat diverse conditions, that parallels can be drawn.

The Corps of Engineers was able to provide repeated loading tests which the J & L tests were unable to provide. The conclusion drawn by Mr. Melville that the J & L pavement would withstand an unlimited number of repetitions of loading is encouraging. Obviously it is hoped to obtain proof of this contention through traffic strip testing carried out by state highway departments.

It has been the authors' feeling that Mr. Melville's statement that the pavement is "substantially over-designed for this (16-kip wheel) loading" is correct. As a matter of fact, in the paper a reduction in percentage of prestressing steel is suggested. In view of the variation in highway loadings and the tendency to run overloaded trucks on highways, it is impossible to say how far the reduction in steel may be carried. This will require further study and experimentation by others.

Results of the current work by the Corps of Engineers at Mariemont (in Cincinnati, Ohio) will be awaited with considerable interest. It is hoped that the Corps may see fit to explore the possibility of one-way (ribbon) prestressing in airport pavement. Work to date apparently has been limited to two-way prestressing.