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Prestressed Concrete Pavement Research

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Review of French and British Procedures In the Design of Prestressed Pavements

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This paper presents a review of current practices in the design of prestressed pavements for roads and airports in France and Britain. Major prestressed pavements are considered and analyzed, outlining design trends in relation to United States methods, and showing procedures for computing slab thickness and amount of prestress as a function of load, traffic, subgrade, and climate. Design of rigid pavements based on Westergaard's and Boussinesq's methods is often used in Western Europe as a guide for the determination of slab thickness for runways and taxiways. For lighter loads, which are encountered in highway design, the thickness of pavements seems to be determined largely by the need to provide adequate cover for the tensioning material (tendons) and six inches has been commonly used. Although many methods have been tried, two-directional prestressing is most often used, with the amount of prestressing having been gradually lowered in recent projects to values near 200 psi. Most pavements have been built on fair to very poor subgrades but high type bases, such as macadam, are in common use to improve bearing power. The use of friction reducing layers of sand and kraft paper is an established practice although the resulting values of the coefficient of subgrade restraint are the subject of widespread research and speculation. The most commonly accepted value of the coefficient is between 0.5 and 1.0.

● THE INHERENT advantages in prestressing concrete are now well established. The success of many structures built in the United States since World War II, which capitalizes on the advantage of prestressing, makes superfluous a panegyric of this new method of utilizing concrete. However, while prestressed concrete frames are becoming established among American engineers, prestressed concrete pavements are still considered by most designers as experimental novelties. With a few minor exceptions, all prestressed pavements are to be found in Western Europe — essentially in Britain and France.

Although the first prestressed concrete pavement on record appears to be a bridge approach at Luzancy (France), prestressed pavements did not become firmly established until the construction of the well-known Orly runway in 1947. This runway was designed to meet the requirements of modern aviation. The design gear load was 185 kips with a tire inflation pressure of about 115 psi (this is for the dynamic loading, the static gear load being 150 kips).

The object of this paper is to review British and French practice and progress in the realm of prestressed pavement design for both roads and airports as reported in available published and unpublished technical literature from the two countries. Construction practices are not within the scope of this paper which analyzes only the basis of design, and design of the pavements. All equations are given in the metric system of units unless otherwise noted.

DESIGN THEORIES

Static Stresses

Theoretical design of prestressed slabs supported continuously and uniformly in two rectangular directions has been based mainly on two theories. The well known Westergaard theory, which assumes "spring" support (ideally: a slab of ice floating on water), has been used widely. Peltier (13) states that with the increased loads and resulting increased slab thicknesses encountered in airport design, Westergaard's theory will yield increasingly lower stress values than Boussinesq's. Thus there has been a tendency to accept Boussinesq's theory for a layered system as more representative in airport pavement design. Burmister's equations are used in solving the often complex derivation encountered in Boussinesq's theory (see Fig. 1).

According to Peltier (13), the maximum bending moment is given by Westergaard as:

$$M = -\frac{P(1-\mu)}{4\pi}(\log Y - 0.618 - 0.098\,Y^2)$$

with

$$Y = r\sqrt[4]{\frac{k(1-\mu^2)}{E\,I}}$$

and by Burmister as:

$$M = -\frac{P(1-\mu)}{4\pi}(\log Z - 0.618 - 0.151\,Z^2)$$

with

$$Z = \frac{r}{h}\sqrt[3]{6\,\frac{E'}{E}\,\frac{1-\mu^2}{1-\mu'^2}}$$

where P is the load on a circular footprint of radius r, h the slab thickness, μ, and μ' the Poisson's ratios of the concrete and the subgrade, respectively, and E and E' the corresponding values of the moduli of elasticity (the logarithms are naperian).

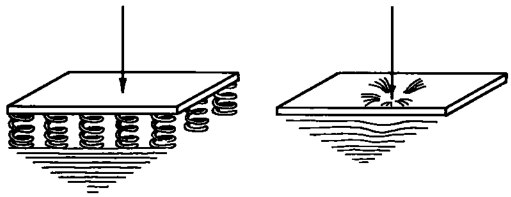

Figure 1.

Actually, both of the above equations are only applicable to a uniform, homogeneous medium where both the slab and subgrade stresses remain within the elastic range. As soon as a crack appears at the lower face of a slab, the problem becomes more complex. For an uncracked slab, Boussinesq's theory has been verified experimentally at the Laboratoire Central des Ponts et Chaussées by photoelasticity using a mirror on a cork subgrade.

Peltier (13) gives the following equation for maximum bending moment for edge loading using Boussinesq's theory:

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

The preceding equations do not apply in the case of a prestressed slab which has been loaded beyond the cracking point at the lower face of the slab. Cot and Becker (5) derived a series of equations for the case of a prestressed slab with radial cracks from the point of load application within a radius c. In that case, the deformation is mostly plastic and the moments at a distance c are as follows: M_1 per unit of length in a tangential direction and M_2 the corresponding radial moments.

$$M_1 = T\cdot c\left(1 - \frac{2}{3}\,\frac{r+h}{c}\right) + \frac{R_1}{2\mu}\left(0.666 - \frac{2}{3}\,\frac{r+h}{c}\right) + \frac{R_0}{2\mu}\left(\frac{1}{2} - \frac{2}{3}\,\frac{r+h}{c}\right) + 4200$$

Where T is the shear per unit length in a radial direction; R_0 and R_1 being the soil reactions under the center of the circular foot print and at a distance c from that point, respectively. The soil reaction resulting from a deformation W is, in turn, related to the modulus of subgrade reaction k as follows:

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

and

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

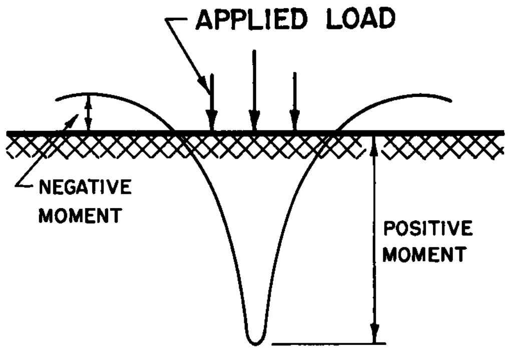

Figure 2. Distribution of moments in a slab caused by an applied load (from Stott).

and for M_2 the equation is

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

Where n is the prestress, f_t the tensile strength of concrete and λh the distance from the surface to the fiber subjected to a tensile stress equal to f_t.

If the stresses outside the previously described circle of radius c are desired, then Cot and Becker (5) use Lagrange's equation which gives for the deflection

$$W = \frac{2\rho^2}{Eh} (d\,C_0 - b\,D_0)$$

Where C and D are functions obtained from tables prepared by de l'Hortet and

$$\rho = \sqrt[4]{\frac{EI}{k(1-\mu^2)}}$$

In that area, the deformations, being proportional to stresses, are plastic. The values of d and b may be determined by taking into consideration the deflection W on the circle of radius c by equating the values of W given for the "inside and "outside" conditions.

The method of design presented above is essentially the one used to determine stresses under live load at Orly Airport. Thirty-nine in. diameter plate load tests are reported by Netter (12) for a test slab 6½ in. thick similar to the actual runway, with a prestress of 570 lb per sq in. The cracking load (at the upper face) was 165 T on natural subgrade, 270 T on a 30½-in. sand subgrade. Netter concludes the following: "For a prestressed pavement, it is not necessary to require non-cracking of the concrete under load for positive moment. Such cracking is unimportant because it will disappear with the load. It becomes a natural happening which results in an increased negative moment without consequence. The negative moments being about ⅕ of the positive moments can be taken care of safely without cracking." (See Fig. 2.) "Cracking at the upper face is an indication of imminent failure. In the case of Orly (thickness 6¼ in., prestress 470 lb/sq in., modulus of subgrade reaction 540 lb/sq in./in.) such cracking would only take place under loads equal to twice the design load. If the increase from 350 to 550 lb/sq in. in tensile strength of the concrete due to curing under stress is taken into account in addition to the prestress, the total stress of the concrete is 570 + 470 or 1,040 lb/sq in. almost three times the resistance of unstressed concrete. Since the bending moments, negative rather than positive, are only ⅕ of those of a standard slab, the prestress slab can safely carry a load 15 times as high." It is the practice of the S.T.U.P. (Société Technique pour l'Utilisation de la Précontrainte, Paris) to take advantage of the tensile strength of the concrete by special jointing; e.g., Orly, Maison-Blanche which may be considered as "continuous".

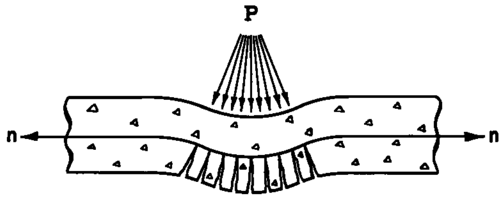

Figure 3. Formation of plastic hinges.

Westergaard's equation was also used by Franco Levi (9) in his work at Milan where he confirmed Guyton's intuitive explanation of the high load-carrying capacity of the Orly runway. In the elastic range, Levi uses the following derivation of Westergaard's equation:

$$M = \frac{P(1+\mu)}{4} \frac{1}{\pi} \left[\frac{1}{2} - \rho \frac{\gamma c}{2} + \frac{\pi}{32} c^2\right]$$

Where P is the uniform load on a circular footprint 11 in. in diameter, γ is Euler's constant. (Note: In field tests, the plate was square 11 inches on the side.) Levi's conclusions are as follows: "After complete consolidation of the subgrade, the slabs under limited loads may be satisfactorily analyzed by the usual theory of slabs resting on an elastic medium. The extent of the purely elastic phase is relatively limited, especially when prestress is used. For a non-prestressed slab, the end of the elastic phase seems to appear for a load about equal to the design load in the case of the Milan runway. The end of elastic phase seems to be marked by the appearance, on the lower face, of symmetrical cracks, the consequence of which is equivalent to that of a positive distortion of small radius. Only a test to destruction could clear up the details of the phenomenon. It appears that cracking is preceded by a local reduction of the apparent modulus of elasticity in tension but that the action is almost entirely reversible. The main effects of the positive distortion are as follows: 1-increased deflection near the loaded area; 2-movement toward the center of the point of zero ordinate 3-decrease of the bending moments in a radial direction in the distorted zone; 4-increase in the negative bending moments and movement toward the center of the area subjected to maximum negative moments. Points 3 and 4 are the most important ones, because they deal on the one hand with the trend of the most intense stresses to move into a region which has a reserve of strength; and on the other hand with the automatic protection which tends to slow down the opening of the first cracks." (See Figure 3.)

"In this discussion, the circumferential moments have not been covered although similar conclusions could be reached: the gradual increase of the positive distortion and the corresponding increase of the negative moments, determine at a certain time the appearance of a new distortion of opposite sign and greater diameter.

"The main effects of the negative distortion are as follows: 1-decrease of deflection in the distorted zone; 2-appearance of positive moments in the zone between the center and the distortion. This later phenomenon is manifested in particular in the positive distortion area and its re-increase will ensue. The stresses of the outer zone return toward the loading area and this movement marks the extreme limit of adaptation of the slab. In certain ways, the behavior of the slab is rather like that of an indeterminate prism which develops a series of plastic swivels upon loading."

From the above findings, Levi (9) concludes that it would be "absurd" to design prestressed pavements strictly in the elastic range. The limit loading will be reached when, after an important negative distortion, the amplitude of the positive distortion begins to increase rapidly and any coefficient of safety would be applied from that point.

Dynamic Stresses

It is recognized that traffic will cause stresses which cannot be duplicated by static load tests. In the case of airfields, French engineers increase the design load by 25 percent over the static gear load for taxiways and aprons. In an attempt at determining the exact effect of repeating wheel loads, repetitive load tests where performed at Orly by Cot, Becker, and Lorin (4) who derived a logarithmic relation

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

where

W = vertical deformation after n cycles
W_1 = vertical deformation after 1 cycle
a = a constant for the pavement
n = number of cycles

This equation reflects the fairly well established principle that the fatigue of the concrete is a logarithmic function of the loading cycles and many an equation has been developed in research laboratories in the United States as well as in Europe to express it. In particular Peltier (13) has proposed:

$$W_n = \rho \left(A + B \log \frac{T_n}{T_0}\right)$$

Where ρ is the tire pressure, T_n the traffic in tons since construction and T_0 the heaviest wheel load in tons using the pavement. A and B are soil constants which may be determined from bearing tests.

The effect of traffic and of the number of heavy wheel loads is handled in the design of prestressed pavement as it would be for plain or reinforced pavement. It is essentially a fatigue failure phenomenon in the concrete and/or of the pavement-subgrade complex.

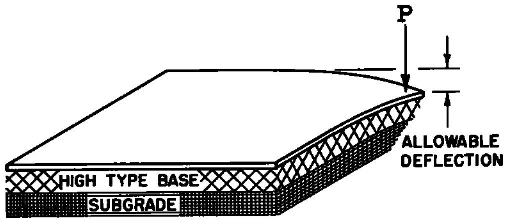

Figure 4.

Subgrade Stresses

Although it is true that most prestressed pavements in Western Europe have been built on fair to poor subgrades, it is also true that high type bases have been used to improve the foundations. The most recent major project, the Maison-Blanche Airport, has been reported by Pousse (15) as being built on an alluvial clay containing 5 to 15 per mil of organic matter (Atterberg's limits of PL = 18 to 23, PI = 30 to 37 and gradation curves indicate that the soil is a CH according to the Unified Soil Classification System). The subgrade CBR was 7 when compacted to 90 percent of the standard Proctor density. The base course system consisted of unsorted river gravel $3\frac{1}{3}$ in., limestone macadam base course $3\frac{1}{3}$ in., fine beach sand $1\frac{1}{4}$ in., kraft paper. The fines in the base course had a PI of less than 4. The Orly runway was built on a lean clay subgrade. (Atterberg's limits of LL = 31, PI = 11 and gradation curves indicate that the soil is a CL according to the Unified Classification System). The base course system was constructed using 13 in. of clay-gravel and 2 in. of river sand covered with a layer of bituminous paper.

Foundations for roads have also been improved by the use of high type bases. For example, in Britain, 3 in. of lean concrete were used at Wexham Springs in 1951 and 12 in. of crushed slag on 9 in. of crushed limestone used at Port Talbot in 1954. In France, 4 in. of lean concrete were used at Esbly in 1949 and 10 in. of gravel at Bourg-Servas in 1954. In all cases a friction reducing layer was provided, usually consisting of about one inch of sand or stone chippings and a layer of kraft or bituminous paper.

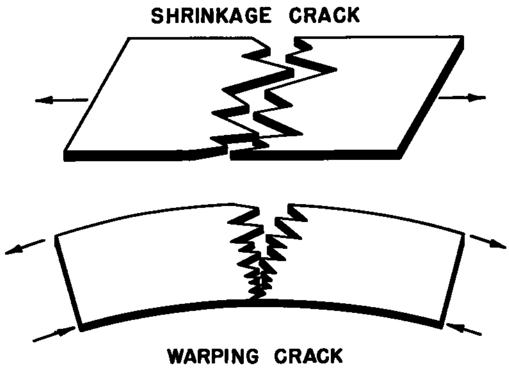

Figure 5.

The information available on the bearing power of the soils is often sketchy. In addition, any bearing index will be influenced by the type of bearing test which has been used and by the size of the loading plate in particular. For example, loading tests were made by Netter (12) for the Orly runway with a 39-in. circular plate, by Cot and Becker (5) for the Orly taxiway with a $29\frac{1}{2}$-in. circular plate and by Levi (9) for the Turin runway with an 11-in. square plate. These tests yielded subgrade bearing capacities, in terms of k, as follows: 55 lb/sq in./in., 100 lb/sq in./in. and 725 lb/sq in./in., respectively.

Finally, the subgrade stresses are interrelated with the static and dynamic stresses on account of the deflection caused by wheel loads (see discussion of the deformation W under "Static and Dynamic Stresses"). Allowable amount of deflection rather than the cracking or ultimate load may be the governing criterion in designing a prestress slab. The allowable deflection is especially critical in airport construction and, for example, the French Technical Service of Air Bases (10) limits corner deflection to 5 mm under static design load and the British LCN (16) system is based on a critical figure of 0.2 in. net settlement for 10,000 load repetitions (see Fig. 4).

Hygrothermal Stresses

Freyssinet (12) stated that "it is said that cracking (of concrete slabs) is caused by shrinkage. This is not a sufficient explanation. The coefficient of friction of a slab on its foundation is, in general, less than 1 and could be reduced . . . A runway which contracts uniformly is subjected at a distance L from its nearest edge to a tension less than 0.24 L or a maximum of 170 lb/sq in. at the center of a 330 ft slab. Thence, if (shrinkage) was the only stress, the spacing of joints could be increased twenty-fold . . . The sole cause (of cracking) is the variation between the top and bottom of a slab. The variations of the temperature of the surface are those of the ambient air temperature added to the effect of sunlight, radiation and wind-caused evaporation . . . The lower face is subjected to different conditions." Freyssinet concludes that the bending stresses caused by the hygrothermal differences between top and bottom of a slab will increase with its thickness, and critical negative moments, which are especially high near joints, will cause early failure. Freyssinet brings out that in prestressed slabs "the hygrothermal differences simply cause a limited vertical movement of the center of the prestress." (See Figure 5.)

The S.T.U.P. has taken hygrothermal changes in slab length into consideration in various ways. For the "accumulation in reserve" or "storing of variations" in prestress due to resulting changes in length, Freyssinet designed accordingly the diagonal jointing of Orly Runway and the sliding abutments of Maison-Blanche Runway. He stated that "the stresses must be regularized by absorbing the energy liberated during expansion of the runway with springs or storage tanks which are in turn capable of restoring (this energy) during shrinkage."

In prestressed slabs designed by Freyssinet, the stressing method (cables or elastic abutments) was engineered to offer some self-compensation for the variations in stress due to hygrothermal changes, which can reach a total of 600 to 700 lb/sq in.

The variations in length due to hygrothermal changes will obviously be interconnected with the friction impeding them. Morice (11) gives the maximum stress f_s for a slab of length l and thickness t as

$$f_s = \frac{Fl}{2t}$$

Where F is the subgrade restraint. Then if f_g is the maximum tensile stress caused by bending "the maximum induced tensile stress due to both effects will occur at the center of the slab length having an arithmetical value f given by:"

$$f = f_s + f_g$$

The distribution of temperatures in a slab was further studied by Thomlinson (22) and values of f_g are of the order of 100 lb/sq in. Similarly Peltier (13) studying thermal effects on concrete slabs gives the following equation for the temperature θ at one point

$$\theta = \theta_0 + Ae^{-Jh} \sin(\alpha t - Jt)$$

where θ_0 is the mean temperature; A is the half amplitude; $J = \sqrt{\frac{\alpha c w}{2\lambda}}$, in which $\alpha = \frac{2\pi}{T}$ with T the period; c is the specific heat; w is the specific weight; λ is the coefficient of conductibility; and t is the time.

Friction Stresses

Friction will play an important role in the design of prestressed pavements at two locations. Friction will occur at the lower face of slab and along the stressing cables in post-tension construction. The former one, commonly known as "subgrade friction", being more critical, will be discussed first.

It is questionable whether friction actually takes place although, as Stott (20, 21) states "restraint exists between any concrete slab and the base on which it rests. This the difference between the 5th and 55th percentiles was a representative range, although it was not capable of rigorous statistical use. The first 5 percent are likely to be exceptional drivers, probably driving well in excess of the speed limit, and, if considered, are likely to prejudice any judgment made on the average driver in the main body of the platoon. Therefore, this range , which has an added advantage of containing half of the arrival times, was investigated.

The 55th, 50th, 5th and 0 (the latest time after the green signal up to which no vehicles have arrived) percentiles were calculated from the vehicle arrival time frequency distributions for each of the five stations. These times were plotted with respect to station and a linear relationship was found to obtain for time for each percentile with respect to distance (Figure 9). The time increment between the 0 and 5 percent lines is much greater than that between the 50 and 55 percent lines, supporting the theory that the first 5 percent should probably not be catered to in timing a signal progression. From Figure 9 the time for the P_x percent of vehicles in a platoon to pass a given point was abstracted, where x is the value of the range either 0 to 50 percent or 5 to 55 percent (Figure 10). Though both lines include 50 percent of the vehicles passing a given station, the time increment for the 0 to 50 range is as much as 8.38 sec greater at 0.65 mi from the signal. The slope of either of these lines, but preferably the 5 to 55 percent range, could be taken as a measure of rate of decay of platooning. If the slope of the line were zero the vehicles would be platooned to the same extent at 0.50 mi as they were at 0.10 mi, however the line shows nothing about the extent of platooning at any one point. Definition of this latter phenomenon was not within the scope of the examination. The linearity of the relationship between time for the range (P_5 to P_{55}) to pass a given point versus distance was examined, the correlation coefficient gave a probability level of 1.12 percent, which is significant.

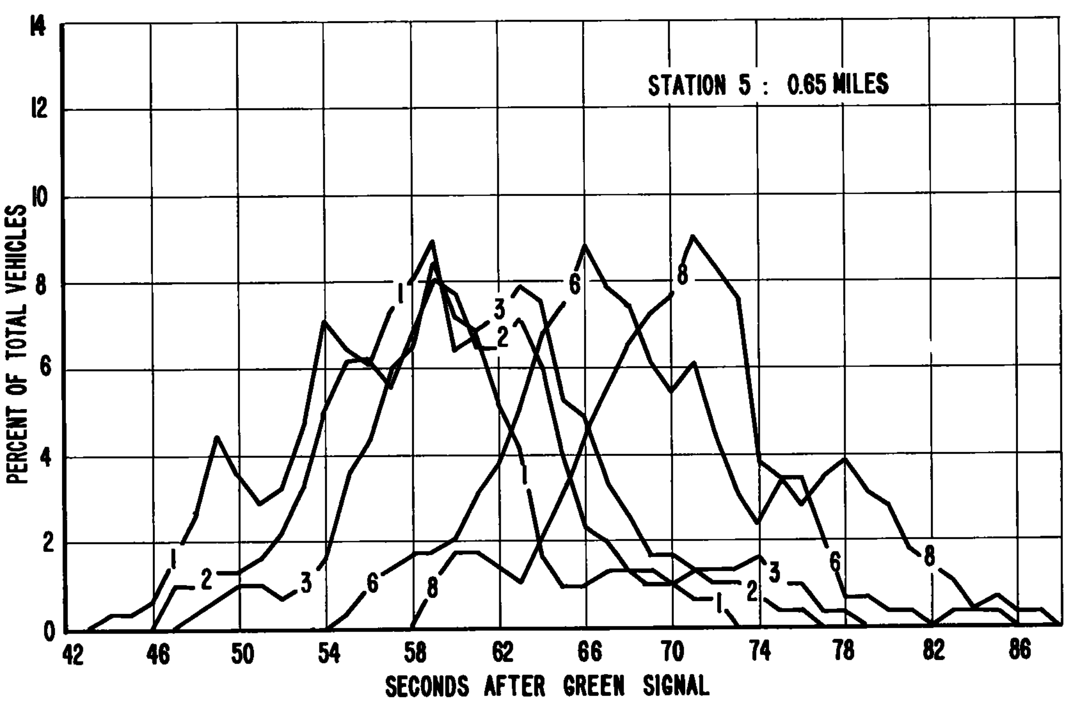

Figure 7. Frequency distributions of arrival times of nth vehicle in a platoon.

Examination of the frequency diagrams of the arrival times of the nth vehicle at each of the stations does not reveal any reason why these distributions should be other than normal. If the equivalent normal distributions are fitted to the data for each of these points perhaps a relationship between them can be determined (7, 8).

For a given distribution, say for the nth vehicle at the mth station, with mean μ and standard deviation σ, the equation of the equivalent normal curve is given by

$$y = \frac{1}{\sigma\sqrt{2\pi}} \cdot e^{-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2} \quad (1)$$

The maximum value of y at the mode is given when $x = \mu$ hence

$$y = \frac{1}{\sigma\sqrt{2\pi}} \quad (2a)$$

or

$$y = \frac{0.399}{\sigma} \quad (2b)$$

that is,

$$y = \frac{\text{constant}}{\sigma} \quad (2c)$$

For each distribution σ is known, hence a curve of $\frac{\text{constant}}{\sigma}$ can be plotted against distance for each station for a given value of n. For ease of computation and graphical presentation the value of the constant was arbitrarily taken as 10 and curves were plotted of $(y = \frac{10}{\sigma})$ versus distance (d) for each value of n.

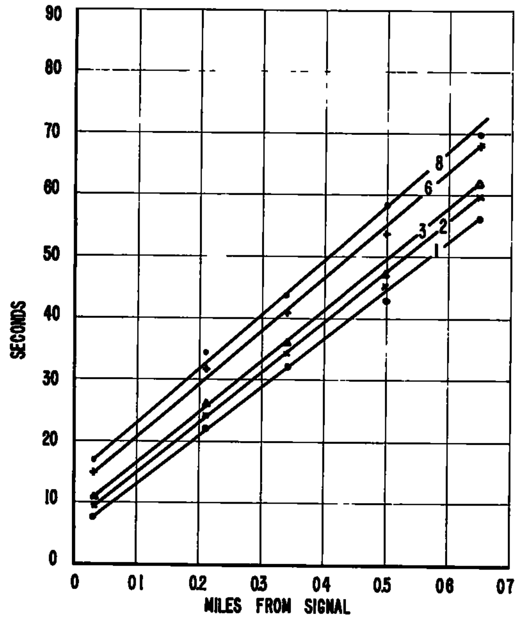

Figure 8. Mean arrival time of nth car versus distance.

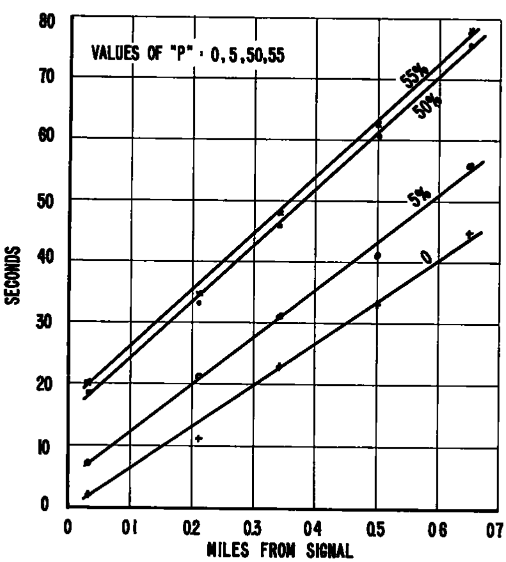

Figure 9. Time for the Pth percentile of vehicles in a platoon to pass a point at a given distance.

It was found that a linear relationship obtained between y and d. Analysis of the correlation coefficient for each line showed a high degree of significance for each line except for n = 6, where the level of probability was 10.01 percent which is not significant. Inspection of the data shows that this rejection is probably due to the value of the standard deviation for station 2. The correlation of the line for n = 8 is good, with a probability level of 1.30 percent, which is significant. There does not appear to be any particular reason why the correlation for n = 6 should not be as good as for other values of n.

ESTABLISHMENT OF A SIGNAL TIMING DIAGRAM

The analysis shows that it would be possible to construct a signal timing diagram for the highway similar to the type used for signal progressions on urban streets (1). Although it would not be possible to cater to a smooth progression of all vehicles at the greater distances, it is possible to arrange the timing of main street green to coincide with the time of greatest vehicle flow density. From the distributions of arrival times of all the vehicles at the five stations, the shortest intervals were calculated in which 50, 70, and 85 percent of the vehicles in a platoon could pass a given station. The lower and upper limits of this interval were designated t_1 and t_2 respectively. For a given percent interval, say the 50 percent interval, the values of t_1 and t_2

To complete the picture on the selection of prestresses in existing roads and airport pavements, it is necessary to discuss the specified wheel loads used in design. For international airports such as Heathrow (London), Orly (Paris) and Maison-Blanche (Algiers), the criteria are said to be essentially in conformance to the regulations of International Civil Aviation Organization (8) as presented in Table 1.

TABLE 1

AERODROMES — INTERNATIONAL STANDARDS

Code	Selected Single Isolated Wheel Load (lb)	Associated Tire Pressure (psi)
1	100,000	120
2	75,000	100
3	60,000	100
4	45,000	100
5	30,000	85
6	15,000	70
7	5,000	35

The Organization further states that "for the computation of the equivalent single isolated wheel load the following assumption has been made: in the case of single main wheels, the single isolated wheel load has been taken as being 0.45 of the gross weight; in the case of dual wheels as being 0.35 of the gross weight; in the case of dual tandem wheels as being 0.22 of the gross weight; and for 8 wheel bogey main under-carriage, 0.18 of the gross weight. It should be noted, however, that the equivalent single isolated wheel load for a given aircraft will vary depending on the type, thickness and quality of the combination of pavement and subgrade encountered, so that the above assumptions give only approximate figures . . . Account is usually taken of the established fact that slow moving or stationary aircraft impose higher stresses than fast moving ones, in the following manner: the strength of the taxiways and of those portions of the aprons that accommodate the heaviest aircraft intended to be served, is computed on the basis of a single isolated wheel load equal to 125 percent of that on which the main runway strength is computed."

Prior to 1951 and the adaption of the International Standards, the French Service des Bases Aériennes specified a load-carrying capacity on a 100 lb/sq in. tire for an aircraft weighing 135 metric tons. The gear load, including the 25 percent increase for taxiways and aprons, becomes 185 kips for Orly Airport.

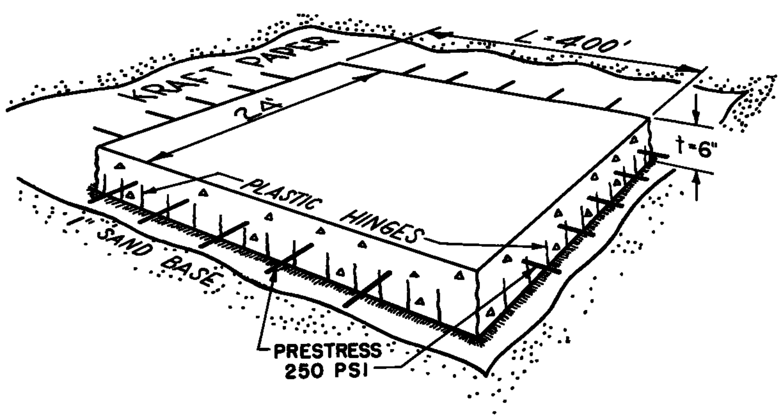

Figure 7. An average prestressed slab.

As far as roads are concerned, the standard is usually a 22 kip load per half-axle without specifying the tire pressure, but including an impact factor of the order of 40 percent.

In the construction of prestressed roads there is the additional problem of dealing with horizontal and vertical curves. Horizontal curves have been handled experimentally in England by considering them as a series of tangents or by balancing outward movement with subgrade friction on superelevated curves (see Basildon and Port, Talbot Roads). Vertical curves are more critical in view of the tendency of prestressed slab to buckle up as described by Dollet and Robin (6) at Bourg-Servas and by Pousse at Maison-Blanche which led to the use of lapped saw-tooth joints although such arrangement may not be needed for slabs 7 in. or more in thickness. Stott (21) suggests designing the vertical curves as two-pin arches, the thrust being equal to the unit weight of the slab multiplied by the radius of curvature of the subgrade. The longer the slab (e.g. runway) the more critical will the curvature be, and it is important to check on the stability of the pavement as discussed by the French Technical Service of Air Bases (10).

Finally the problem of "how to prestress" should be discussed — it is beyond the scope of this paper to comment on the sundry devices and materials currently available to obtain the necessary compressive forces and to maintain them for the expected life of the pavement. Since it has been established theoretically and experimentally that the maximum stresses on a slab are caused by "edge loading" and that the longitudinal and transverse stresses are approximately the same, depending on the footprint, it is logical to anticipate that prestressing will be required in two directions. Although width and length are obvious directions, diagonals can, of course, be used to advantage.

In the case of lighter loads found on highways, it is possible to use a reduced transverse prestress. The Road Research Laboratory (17) proposes 50 lb/sq in. for narrow slabs and no transverse prestress at all for slabs less than 15 ft wide if the possibility of longitudinal cracking under overload is an accepted calculated risk. Obviously, this risk is small in view of the fact that current construction procedures require a superfluous thickness of concrete over that strictly needed by design, to furnish sufficient protective cover to the tendons and for construction convenience.

CORRELATION WITH UNITED STATES PRACTICE

With a few minor exceptions, prestressed pavements in this country have been experimental slabs built by governmental agency, in particular by the Bureau of Yards and Docks and by the Corps of Engineers. In its final report of the test slab at Patuxent River N.A.S. (1), the Department of the Navy concurs with Western Europe in the need for a sand and paper restraint reducing layer giving a coefficient of friction of about 0.5. The design was based on Westergaard's theory. The Bureau concluded that the 6½- to 7½-in. slab was over-designed for a 100 kip gear load. Any reduction in slab thickness would be limited in order to provide minimum cover for the tendons and to maintain a sufficient cross-sectional area to limit slab deflection and unit tensile friction stresses. Thus it may be concluded that the slab was over-designed just as much as the Orly Runway was found over-designed, and this should be related to Levi and others' conclusions, discussed previously, on the formation of minute cracks at the lower face of the slab and the creation of a plastic hinge. In tests performed at the Ohio River Division Laboratories of the Corps of Engineers the subgrade restraint was also found highly variable. Tests in progress of a prestress overlay of a rigid pavement under heavy moving aircraft wheel loads will yield information on a thin prestress pavement over a rigid base. This is of importance to obtain an economical pavement with a deflection limited to allowable values. Currently available design information in the United States have been analyzed by Friberg (7). Good agreement between practices as reported in his paper and French and British procedures should be noted.

CONCLUSIONS

French and British prestressing practices may apparently be used to advantage in furthering United States design of prestressed pavements. Specifically this review indicates that:

1. Current prestressed pavements are being overdesigned from the point of view of load-carrying capacity to control deflection, furnish sufficient cover to the tendons and prevent tensile cracks due to subgrade restraint.

2. Westergaard's theory is used in design although it will give conservative results. Boussinesq-Burmister's equations may be more representative of actual conditions.

3. The formation of lower surface cracks (plastic hinges) should be anticipated and will increase load-carrying capacity without permanent harm.

4. The subgrade restraint should be reduced as much as possible with a layer of about one inch of fine sand covered with kraft paper. The resulting coefficient is of the order of 0.75 although highly variable.

5. The prestress need not be very high (of the order of 250 psi). It should be in two directions except for very light loads or very narrow lanes.

6. Slabs of 400 to 500 ft in length are now in use but increased lengths to about 1,000 ft appear more efficient and may be envisaged from the above conclusions. Thicknesses for both roads and airports are of the order of 6 in. Consequently, prestress may be only economically justifiable for heavy wheel loads.

7. Designers take advantage of the tensile strength of concrete (expected to be increased by curing under compression) to reduce prestress. This necessitates special measures to prevent early shrinkage cracks, especially in the longer slabs.

8. Although poor subgrades are used for the foundation of prestressed pavements, European practice to use high type bases renders a correlation of design with bearing power of the subgrade rather difficult. In addition, the effect of the shape and size of the loading plates used for bearing test, although known to exist, is not always resolved.

In conclusion, as it has been stated (3) "the work already done throughout the world has shown that prestress concrete pavements have definite advantages . . . but despite the work done . . . research is still needed . . . Work done in Europe . . . indicates that this new material can offer great savings in materials and material cost."

ACKNOWLEDGMENT

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Theoretical and Practical Aspects of Prestressed Concrete for Highway Pavements

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● THE INCREASING and successful use of prestressed concrete in this country directed interest toward the possible adoption of this method to highway and airfield pavements.

The apparent theoretical advantages of this novel pavement are still not formulated to the extent where they could be put into practical application in the most economical manner. Theories which were developed to predict the behavior of pavements under loads and volumetric changes therein, proved to be satisfactory only within certain limitations. The task to develop new theories to express the behavior of prestressed concrete pavements becomes even more complex, since it must deal with an increased number of variables.

Testing of such pavement seemed to be the only reasonable way to determine its merits. This made it also possible to cast some light on the practical problems which are to be encountered in future construction.

Following this course of thought, the experimental prestressed concrete slab built at the Patuxent River Naval Air Station was designed and constructed with the main purpose to obtain both theoretical and practical information. This test program, sponsored by the Bureau of Yards and Docks, Department of the Navy, was set up to investigate the adaptability of this new method to airfield pavements. The test results and experience obtained should be valuable, however, for highway engineers as well.

GENERAL THEORY OF PRESTRESSED CONCRETE PAVEMENTS

The apparent advantages of using prestressed concrete for highway pavements are twofold.

First, by using prestressing, pavements can be made continuous, without joints, within certain limitations. Second, by using prestressing, pavements can be made stronger to resist high loads or can be built thinner to resist the same loads, than conventional concrete pavements.

Actually both these features of prestressed concrete pavements can be explained by the inherent compressive energy in the concrete provided by external means. Such external means can be wires or strand under tension and imbedded in the concrete or hydraulic jacks, which can be used with fixed abutments to create compression in the pavement.

The additional strength offered by prestressing tendons when they are used cannot be compared simply with an increase in strength of the concrete. Cracking in a prestressed concrete section does not represent failure. The cracked section is still able to resist increasing loads until the limit condition is reached, similar to that occurring in reinforced concrete slabs.

The increased strength of prestressed concrete pavements points, of course, to the fact that such pavements can be produced with a thinner concrete section than conventional concrete pavements. Such pavements result in greater flexibility and therefore should adapt themselves with less detriment to uneven base conditions. Also, they should be less susceptible to differential thermal conditions, which create severe stress conditions in thick pavements.

The permanent compressive forces applied on prestressed slabs make it possible to build pavements continuous and eliminate thereby a great number of joints. Prestressing also provides a spring effect which will restore continuity in the event cracking in the slab should take place due to accidental reasons such as extreme thermal stresses, shrinkage or excessive loads.

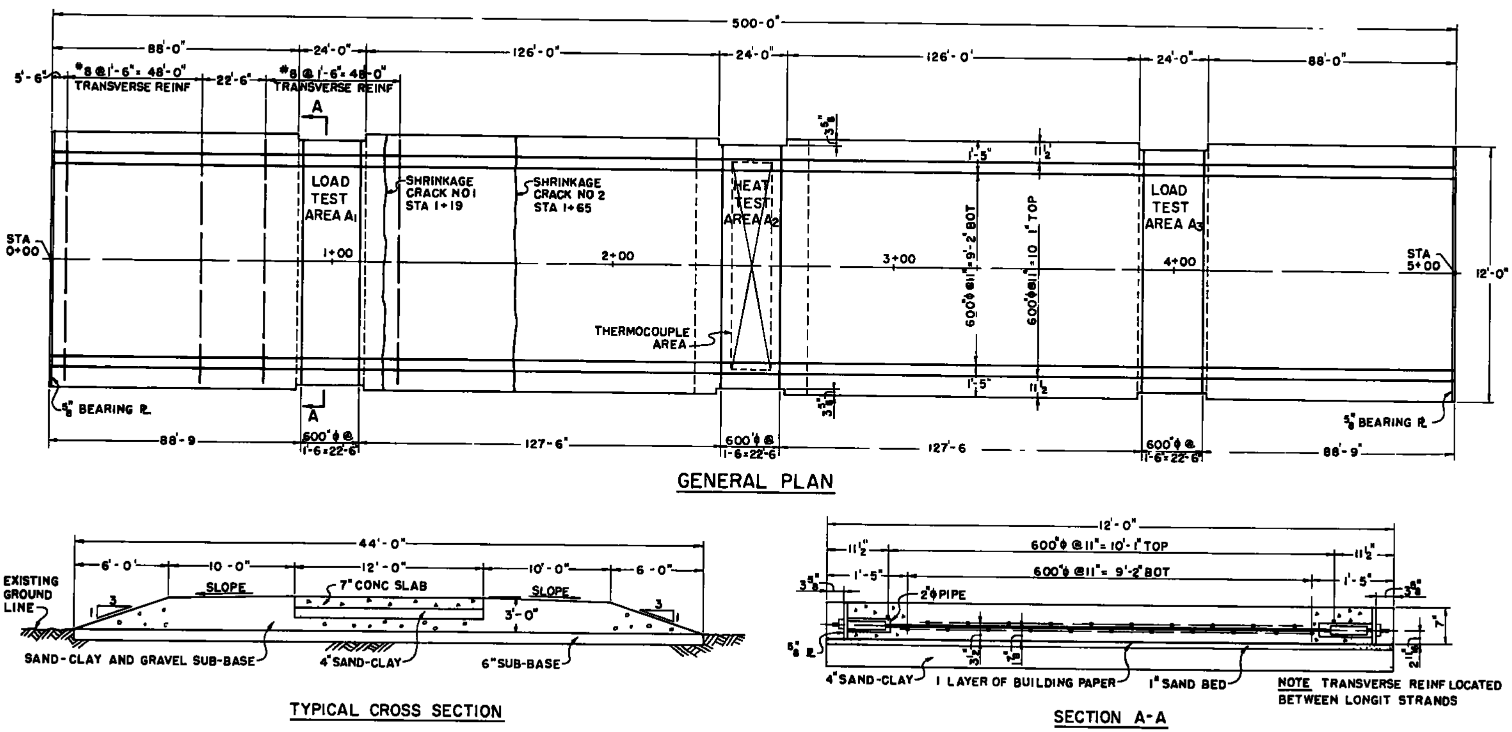

Figure 1. General plan (drawing — courtesy of American Concrete Institute).

DESCRIPTION OF THE TEST SLAB

The test at Patuxent (1) was set up to investigate the general behavior of prestressed concrete pavements with and without loads.

The slab itself is 12 ft wide, 500 ft long and 7 in. thick. Prestressing therein was provided longitudinally by two sets of 0.6 in. dia. strand, which consisted of seven hot-galvanized wires. One of the sets consisted of eleven strands placed at the center of the slab and is referred to as upper ("U") set. The lower ("L") set consisted of twelve strands placed 1⅞ in. above the bottom of the slab. (See Fig. 1.)

Transverse prestressing was provided in three locations, called test areas, where the same strands as used for the longitudinal direction were placed between the two longitudinal strands.

All strands had standard end fittings for jacking and anchorage. They were designed to allow complete stressing of the strands from either of their ends. It should be noted that the longitudinal strands had to be stressed 36 in. to produce their design load to 30,000 lb.

The majority of strands was covered prior to placing with plastic tubing, the seams of which were sealed with mastic tape. Six longitudinal strands, as shown on Figure 2, were wrapped in "Sisalkraft" paper.

The strands were designed to take an initial stress of 140,000 psi, representing a total of 30,000 lb. Such a load created a compression in the concrete of 360 psi and 330 psi when the upper and lower sets were stressed independently. Applying the same load on each transverse strand exerted 235 lb on each square inch at the test areas.

In addition to the prestressing elements, the slab was reinforced in certain sections with transversally place No. 8 bars, spaced 1 ft 6 in. o.c. as shown in Figure 1.

Concrete used in the slab was designed with seven bags of cement and 34.5 gallons of water for each yard. Crushed limestone with maximum size of 1.5 in. was used for coarse aggregate. To increase strength and workability, ¾ lb of Plastiment per bag of cement and Dairex to produce between four and five percent entrained air was also used.

This design gave a rather harsh mix with 1- to 1½-in. slump which was gradually increased to 3½ in. in the course of the concreting operation.

Results of an extensive test program made at the site are summarized in Table 1.

The slab was placed on a 2 ft 6 in. deep prepared subbase which consisted of a sand and gravel mix. Directly beneath the slab a four inch layer of sandclay was placed.

All material used for the subbase was placed in lifts not exceeding 5 in. and had to be compacted to 100 percent maximum density according to the modified Proctor compaction test. The specification called for a well finished surface, allowing not more than ⅛ of an inch projections or depressions when measured with a 10-ft straight edge.

The friction reducing layer used under the slab consisted of one inch local sand, covered with building paper. The sand was ideal for this purpose, because of its uniform grain composition. Actually 94 percent of the material was retained on the sieves with meshes from 40 to 80.

CONSTRUCTION OF THE SLAB

During the preparation of the subbase, work began to cover the strands with plastic tubing and paper respectively. This operation proved to be rather time consuming and pointed to the necessity that such work should be eliminated in the field. Tendons if used as post-tensioning elements should be provided with a conduit around them in a factory and shipped so covered to the construction site.

The 500 ft long elements were assembled and covered on a work bench adjacent to the slab-bed. They were handled from there and placed by hand, another operation which created difficulties. Improved methods, such as feeding these elements from reels mounted on a platform, will be necessary in future applications.

Prior to placing the strands, the fine sand layer and sheets of building paper thereon, were laid on the top of the 4 in. sand-clay layer.

To secure proper positioning and alignment of the strands, temporary anchorages were installed at both ends of the slab. By using these anchorages, the strands were

TABLE 1

CONCRETE DATA

AGE OF CONCRETE (DAYS)	FLEXURAL STRENGTH (P.S.I.)	COMPRESSIVE STRENGTH (P.S.I.)	MODULUS OF ELASTICITY (P.S.I.)	SHRINKAGE (MILLIONTHS)	THERMAL EXPANSION (IN./IN.)
3	395	3 070	—	—	—
7	610	3 560	—	440	—
14	720	4 830	3 240 000	510	—
28	805	5 180	3 350 000	560	.00000616
60	850	5 300	3 560 000	500	—
90	875	5 370	3 730 000	520	—
180	885	5 970	4 070 000	410	—
360	965	6 470	4 220 000	330	.00000654

NOTES: 1. AVERAGE VALUES SHOWN
2. SHRINKAGE VALUES REPRESENT INCREASES AND WERE OBTAINED FROM FIELD CURED SPECIMENS

pulled straight and slightly stiff, applying approximately 1,000 lb on each strand. In addition the strands were supported by metal chairs spaced 6 ft apart.

This arrangement seemed to be satisfactory during casting, as long as concrete was spread in the longitudinal direction of the slab. Unfortunately, since most of the spreading was done by hand, the side pressure of the concrete dislocated some of the strands in the transverse direction. This was corrected as much as possible to maintain the proper alignment of the prestressing elements.

For easier placing and finishing of the concrete, its slump was gradually increased to 3½ in. With such consistency of concrete the operation proved to be much more satisfactory. The casting of the 500 ft long slab took 11½ hours.

The daily temperatures were considerably high, with a maximum of 84 deg F in the shade. The actual temperature in the sun well exceeded 100 deg F.

The slab was cured with paper and burlap for about five days.

The surface of the slab was inspected the morning following casting. At that time two shrinkage cracks developed across the entire width of the slab at Stations 1 + 19 and 1 + 65. No other cracking developed thereafter other than due to the various loadings.

PRESTRESSING OF THE SLAB

The first day following casting of the slab all temporary anchors were released and each strand was slackened to allow a minimum of one inch between the anchor-nut and the anchor-plate. The strands were pulled one by one through their conduits thereafter by using the stressing jack. The gage of the jack was carefully watched to observe any load indications. In none of the cases was any load observed, which would have indicated binding and bounding of the strands in their conduits.

Prestressing of the strands started the third day following casting. First all longitudinal strands were stressed and left under such condition for a period of 35 days. The first stressing of the strands was completed over a longer period and in several steps, mainly because of the various measurements made during these operations.

The lower and upper strands were stressed and released several times after their first stressing and released as shown in Figure 5. This was accomplished partly to create different stress conditions in the slab for the numerous load tests and partly for the general study of the distribution of the introduced stresses along the longitudinal direction of the slab.

As a result of the investigations made during the first stressing operation the following sequence was adopted for stressing the strands thereafter:

1. Introduce one-half of the required load on one end (e.g., Station 0 + 00).
2. Check load on other end (i.e., Station 5 + 00).
3. Introduce difference of final load and checked load at same end.
4. Check load at end where first prestressing was applied at Station 0 + 00.
5. Introduce difference, after checking, for final condition.

The equipment for stressing and release consisted of one hand and one electrically operated hydraulic jack. The latter offered advantages and resulted in considerable time saving.

For future use, jacks are recommended with maximum ram movements equal to the necessary stretching of the prestressing tendons, so that the time consuming re-settings can be eliminated. The average time required to produce 36 in. of elongation of the strands was 80 minutes. With easy jack operation and follow-up of the anchor-

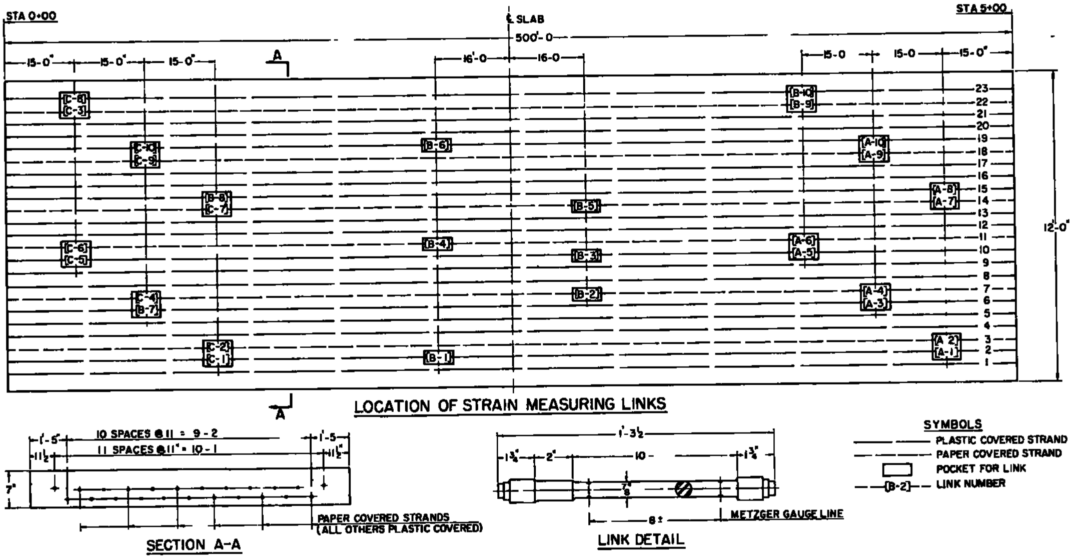

Figure 2. Strand layout.

nut this time could be reduced to about 10 minutes, including connecting and disconnecting the jacks to the elements as used.

TEST PROGRAMS

The test program was centered around two basic problems:

1. How to secure maximum efficiency in providing prestressing in the slab?
2. What is the minimum of prestressing required to secure continuity in the slab and its load-carrying capacity?

The efficiency of prestressing in the slab is affected by the distribution of loads on the prestressing tendons and by the friction resistance of the base underlying the slab.

The distribution of loads on the strands were investigated with the help of calibrated strain measuring links. These links were inserted in certain strands as shown in Figure 2. Loads were measured on these links with Metzger gages which indicate strain variations and these can be converted into loads.

The results of these tests were quite conclusive and pointed to considerable losses in certain cases.

The strain measuring links were mounted on strands belonging to four distinguished groups:

1. Three strands in the upper layer covered with plastic tubing.
2. Three strands in the upper layer wrapped in paper.
3. Same as 1 above, but in the layer above, supporting reinforcing bars and transverse strands in test areas A_1, A_2 and A_3.
4. Same as group 3, but wrapped in paper.

The best results were obtained in the case of strands in the upper layer covered with plastic tubing. The greatest losses were observed in strands in the lower strands covered with paper. Strands belonging to the two other groups showed about the same losses. The losses are expressed in percentage of the differences measured at the two ends, with relation to the introduced loads at the jacking end of the strands. These percentages are:

Group 1 - 5 percent

Group 2 - 19 percent

Group 3 - 15 percent

Group 4 - 62 percent

The results obtained from strands belonging to group 4, point to the importance of providing a proper conduit around the prestressing elements. Paper obviously did not resist the pressure of the concrete, as compared to the more favorable performance of the plastic tubing.

The higher values registered in case of the lower strands are the result partly of the lower position of the strands, partly of the pressure exerted by the transverse reinforcement which was placed on them.

It is believed, that with presently available sheetmetal conduits applied around the strands in the factory, the losses could have been reduced to much lower value. The actual or possible friction losses should be determined with consideration to the length and position of the elements in future applications.

Since all strands were stressed partly from their both ends, it must be anticipated, that in case of differentials and resulting friction losses the intermediate section of the elements carried less loads than their corresponding ends. This difference produced an equalizing tendency which was observed during the course of testing.

The losses in prestressing after the loads were applied were studied also. Of course in cases where losses due to conduit friction were significant the result would be affected greatly by the leveling off of the differentials.

Strands belonging to group 1, with their average conduit friction loss of 5 percent should, however, give satisfactory results.

Losses after prestressing are due to shrinkage and creep in the concrete and creep in the strands. The results represent losses over a period of sixty days, for strands belonging to group 1 and give an average value of 10 percent.

Total shortening of the slab during this sixty day period was not more than two inches. Since prestressing of all strands was completed only after the first 30 days, it may be reasonably assumed that the average shortening of the slab affecting the strands was not more than one inch. Such shortening would represent a loss of approximately 3 percent in the initial load. The relaxation of steel should lead to 6-7 percent loss, and the total result comes close to the measured 10 percent loss.

Losses due to base friction seem to be of greater importance and considerably more difficult to determine. The development of prestressing in the slab creates stresses in the slab accompanied by strain. Series of these strains lead to shortening of the slab, which means that the slab has to move actually in order to develop prestressing in its entire length. Such movements are obviously restrained by the base friction. The lower this base friction, the higher the efficiency of prestressing will be.

The problem of the resistance to prestressing offered by the base is a rather complex one. It is affected by temperature conditions, moisture content of the base, and not only by absolute values, but possibly even more by the gradient of these variables. It is also affected by thermal differentials existing between the slab and the concrete, and

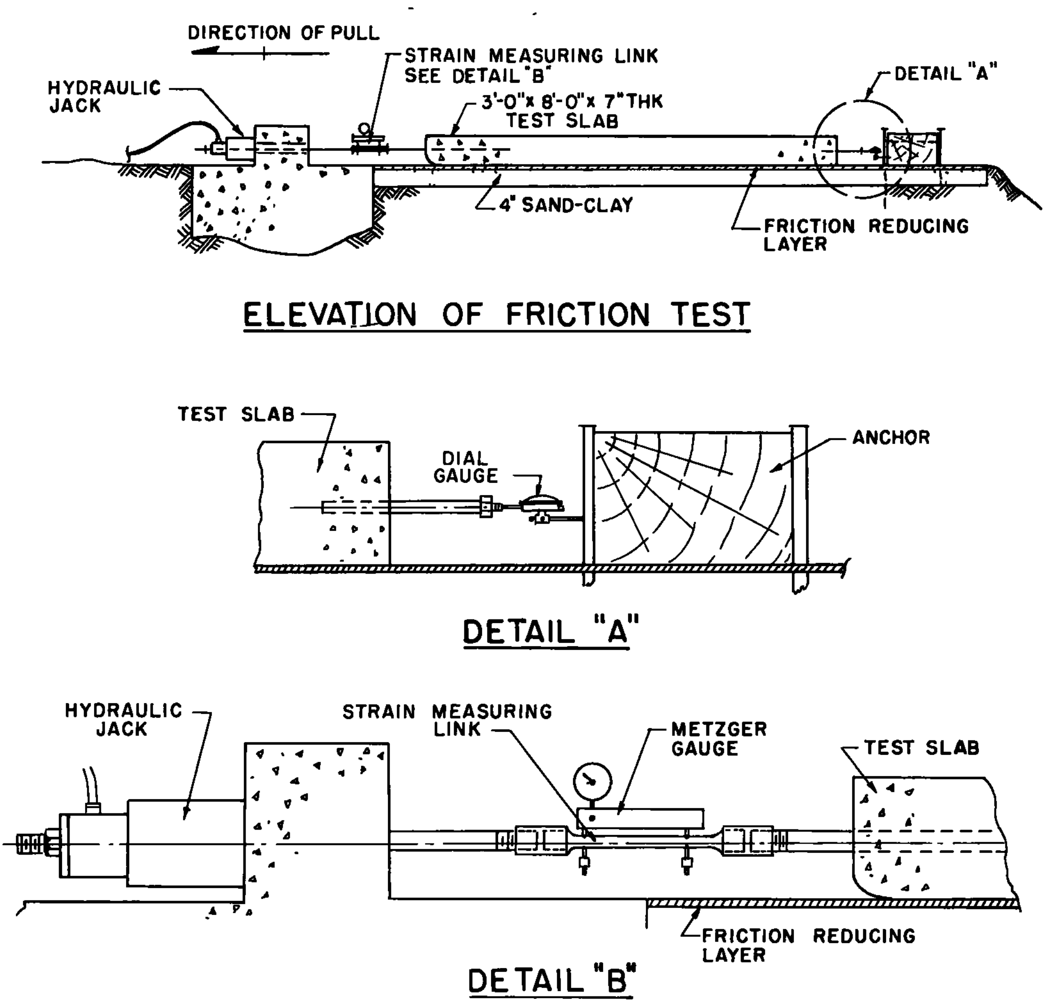

Figure 3. Friction test.

in the concrete itself. The magnitude of the prestressing forces will also have bearing on the friction losses.

It was of primary concern in the design stage of this experimental slab, to find a friction reducing expedient which will offer the lowest friction resistance at minimum construction cost. This problem was deemed important enough to conduct some preliminary testing before the construction of the actual slab.

Therefore concurrently with the base preparation, three experimental slabs, 8 ft by 3 ft and 7 in. thick were built on various friction reducing devices (Fig. 3). A fourth slab was placed directly on the prepared base. The three friction reducing devices were as follows:

1. One inch sand covered with one layer of building paper.
2. Two layers of building paper.
3. Two layers of copper-clad Sisalkraft paper, the copper sides facing and powdered soapstone between them.

The friction values obtained during these tests are tabulated in Table 2. They indicate the extremely high friction coefficient for the slab placed directly on the prepared base. The values obtained for the three slabs with the friction reducing expedient under them seem to be comparable with slab 1, giving the lowest friction values.

Considering cost, the sand and paper layer seemed to be economical and therefore it was decided, that the 500-ft long slab be built on one inch of sand covered with building paper.

It is obvious, that the friction values obtained during these tests are only indicative as to the friction conditions existing under a 500-ft long prestressed concrete slab.

Failure in the friction layer will occur progressively under a long slab, more than in case of an 8-ft long slab. The presence of such successive friction failure will lower therefore the friction coefficient and probably could be expressed by an average value of those obtained at failure and sliding thereafter.

TABLE 2

FRICTION TESTS

Slab No.	Coefficient of Friction		Movement Before Failure
	At Failure	During Sliding	in.
1	0.72	0.60	0.045
2	1.13	0.55	0.004
3	0.77	0.63	0.018
4	5.15	1.10	0.012

The nature of the forces applied during the preliminary friction tests and of those present in a prestressed concrete slab are quite different also. In a prestressed slab the loads are sustained and therefore the yielding of the base should reduce the actual friction values. The exact determination of the magnitude of the effect of base yielding seems to be extremely difficult because of the presence of several factors.

These are temperature and moisture content variations in both base and concrete.

Test results made on field-cured specimens indicated maximum volumetric changes in the concrete in the order of 470 millionths. Average actual strains measured in the slab showed a maximum variation of 430 millionths.

Compared with these figures prestressing of 300 psi would produce a maximum of 75 millionths, if the modulus of elasticity of 4 millions, as found for the concrete in the slab, was used. This should clearly illustrate that the precision with which losses in prestressing due to base friction can be determined, is rather undetermined, mainly because of the considerable influence on this problem by factors mentioned above.

In addition to these problems, the effect of initial shrinkage developing immediately following casting of the slab must be considered also.

This shrinkage takes place prior to the application of the prestressing forces. It will develop as freely as allowed by base friction. Actually due to the resistance of the base, tensile stresses will develop in the concrete and they will be in equilibrium with the friction forces at any moment. Assuming that the slab remains continuous (i.e., no shrinkage cracks develop and that the friction values are low) the initial shrinkage will dissipate all friction resistance.

Temperature conditions in the concrete are not constant either before the application of the prestressing forces. If they have a decreasing tendency, they will create contraction in the slab and will be similar in effect to shrinkage. Increasing temperatures naturally will decrease the effect of shrinkage. In any case, the combined effect of initial shrinkage and temperature conditions will create a stress condition in the base and the concrete which will be in equilibrium at any time.

If these stresses dissipate the friction resistance of the base, prestressing can be applied, as a superimposed load virtually without loss. This would indicate that losses in prestressing due to base friction cannot be separated from the effect of shrinkage and temperature variations which take place in the slab prior to the application of prestressing.

This rather complex problem was investigated with the help of sixteen strain meters ("L" meters) placed in groups of four at stations as shown in Figure 4. These meters indicated not only strain variations but also temperatures.

Results obtained in connection with the first prestressing of the slab were far from being conclusive. Therefore, a detailed program was set up to study this problem. During this program prestressing was released, then applied again, first by all upper

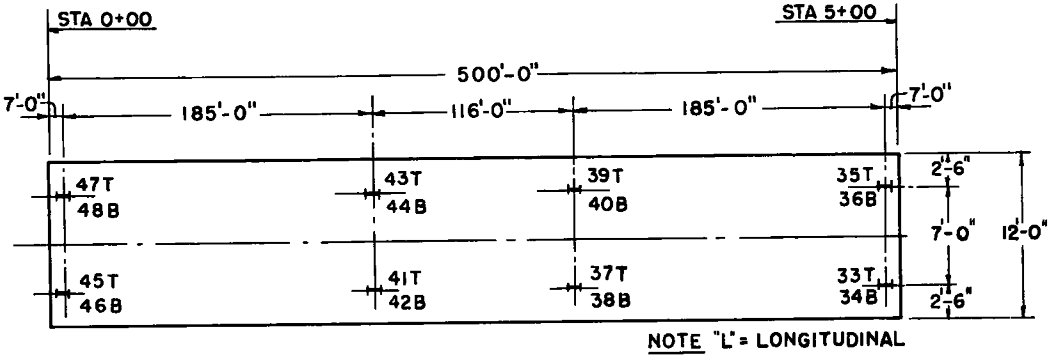

Figure 4. Location of "L" strain meters.

strands, then all lower strands and finally by all strands, with sufficient time intervals provided in between the various stressing operations. Strain meter readings were made after and before the slab was either stressed or released and usually conducted for a period long enough to determine a satisfactory strain condition under day - night variations.

Using the average results obtained from the four meters located at four various stations, the strain variations at the center of the slab were determined by extrapolation.

The results of these tests indicated practically no loss in prestressing, when the set of upper strands were stressed, February 5, 1955. Similar results were obtained when the lower strands were stressed there after April 1, 1955.

At the release of all strands on April 20, 1955, the strain meter readings indicated a drop in strains in the intermediate locations of the slab approximately 12 percent higher than the values observed at the ends of the slab. This greater drop in stresses is obviously the result of restrained volumetric expansions, which were released together with the stresses due to prestressing.

The slab was left unstressed for a week thereafter, and stressed to the maximum possible value of 690 psi, on April 27th. The results registered immediately thereafter indicated a loss of approximately 23 percent at the center of the slab. It should be noted that at this time the tendency of temperature variations was increasing and therefore prestressing forces were opposed by those caused by restrained volumetric changes.

The following release and repeated stressing of strands lead to findings which indicate clearly the effect of time on the friction losses. Immediate observations made after the release of May 1st indicated considerable amount of residual stresses remaining in the center portions of the slab in the order of 45 percent. When the slab was stressed again on May 4th, the maximum losses amounted to approximately 27 percent. The difference should be explained by a drop in residual stresses left after the release.

Repeated prestressing as performed on the slab points, however, to an important problem which if exploited correctly should add to the efficiency in developing prestressing. If prestressing is applied in a slab and released after the introduced stresses and friction forces reached a final state of equilibrium (i.e., after most of the yielding of the base took place) stresses will remain in the center portion of the slab. This will occur because friction will develop and will restrain the expansion of the slab. These interlocked stresses can be increased by applying repeated prestressing in short cycles, allowing time for development of prestressing, but eliminating the drop in residual compression in the unstressed slab. The results obtained clearly indicated the advantageous behavior of a prestressed concrete slab in this respect.

Thermal conditions of the concrete should offer further assistance to secure the most efficient conditions for the development of prestressing forces. Prestressing should be applied whenever temperatures have a decreasing tendency. Seasonal changes seem to have more influence than the day - night variations.

The over-all results did not lead to the finding of an absolute friction coefficient applicable to this test slab. It was found, that proper provisions must be made to keep friction at a low value when prestressing is introduced, and that this should be applied when the volumetric changes in the concrete create contraction in the slab. A recommended friction coefficient of .5 should be used rather as a yardstick to provide sufficient prestressing, than an absolute figure expressing friction losses.

The effect of prestressing was demonstrated at the two shrinkage cracks at Stations 1 + 19 and 1 + 65. These cracks opened up to somewhat more than $\frac{1}{16}$ of an inch before prestressing and closed gradually thereafter. They remained closed and tight whenever the slab was under compression.

The over-all length variation of the slab was determined from strain measurements performed on the "L" meters. These readings were not continuous and did not coincide with extreme conditions. They should, however, give fairly accurate results.

The maximum shortening of the slab was registered in December, 1954, with 300 psi prestressing in the slab. This amounted to a weighted average of 430 millionths, totaling 2.6 in.

LOAD TESTS

A total of 29 tests were performed on the test slab. These were made under different conditions, varying the amount of prestressing plate size and location. The summary of these tests is shown in Table 3.

The loading equipment with 100,000 lb capacity used at the beginning had to be replaced during the tests with another one having a capacity of 200,000 lb.

Most of the tests were performed at the centers of the two load tests, Areas A_1 and A_3, where strain meters to register concrete stresses and pressure meters to determine base reaction, were placed. Deflections in the area affected by the loads were measured also.

Other tests were made at the edges of the slab, at areas where no transverse strands were placed and also at the two shrinkage cracks.

The first seven load tests were performed on test areas A_1 and A_3, with varying amount of prestressing and plate sizes of 8 and 20 in. in diameter.

The results obtained during these tests did not seem to be affected by the differences in amount of prestressing and size of plate on which the various loads were applied. Deflections and loads were in proportion in the available load range.

To produce a more severe loading condition, the available load of 100,000 lb was applied on a 3 in. and then on a 2 in. plate in diameter. These tests were made at Station 1 + 43, where only 300 psi longitudinal, but no transverse prestressing was available.

Deflections were considerably higher, but recovery of the slab was better than 85 percent after the load was removed. The effect of these loads with extremely high concentration became apparent several months after, when a longitudinal crack was observed on both sides of the point of load application. It is most likely that it was the result of certain initial cracking, which developed on the bottom of the slab and worked its way through the entire section as a result of cyclical volumetric changes.

As soon as the new loading equipment with 200,000 lb capacity became available load tests were performed on test areas A_1 and A_3. The results of two tests, Nos. 15 and 27 were in close agreement. These tests were performed on a 20 in. plate. Two other tests (Nos. 17 and 22) performed at two different locations but on the same 8 in. plate were in agreement also; but the effect of increased load concentration resulted in higher deflections, as compared to those obtained with a 20-in. plate.

It should be noted that during test No. 17, the 8-in. plate punched through the slab under a load of 189,000 lb. This should not be considered as an indication of the strength of the slab, because this failure took place in a location where the concrete section was considerably weakened by the height concentration of various instruments embedded therein.

The effect of the amount of prestressing was not evident during these tests. The registered difference measured during load tests Nos. 15 and 27 is so small that it hardly could be explained by the fact that the amount of prestressing was increased from 250 psi to 600 psi. It is felt that the amount of prestressing would have demonstrated its true effect under considerably higher loads than those available.

Two load tests were performed at portions of the slab, where no prestressing w

TABLE 3

SUMMARY OF LOAD TESTS

	TEST NUMBER	DATE	LOCATION	APPLIED LOAD WITH INCREMENT (KIPS)	PLATE	PRESTRESSING IN SLAB (KIPS)		MAXIMUM DEFLECTION (INCHES)	MAXIMUM "K" Psi per in.
						LONGITUDINAL STRANDS	TRANSVERSE STRANDS		
LOAD APPLIED AT INTERIOR OF SLAB	1	10-20-53	A_3	10-20-30-40-50	20"	550	200	036	—
	3	12-17-53	A_3	20-35-50-75-100	20"	250	200	063	—
	4	12-18-53	A_3	10-20-30-40-50	8"	250	200	032	270
	5	4-27-54	A_3	25-50-75-100	8"	250	200	063	420
	2	11-17-53	A_1	10-20-30-40-50	20"	550	200	030	—
	6	4-28-54	A_3	12^5-25-37^5-50-62^5-75-87^5-100	20"	250	200	055	—
	7	4-28-54	A_1	25-50-75-100	8"	250	200	053	420
	11	4-29-54	STA 1+43	10-20-30-40-50-60-70-80-90-100	3"	300	0	088	—
	12	4-29-54	STA 1+43	50-60-70-80-90-100	2"	300	0	076	—
	13	4-30-54	STA 1+43	10-20-30-40-50-60	2"	300	0	045	—
	15	12-8-54	A_1	50-100-125-150-182-200	20"	250	240	157	280
	17	12-10-54	A_3	50-75-100-125-150-175-189	8"	250	240	~ 150	400
	18	2-1-55	STA 3+60	50-100-150-185	20"	0	0	~ 459	—
	21	2-3-55	STA 0+83	100-125-150-175	20"	0	0	282	—
	22	3-29-55	A_1	200 (8 TIMES)	20"	280	240	200	250
	23	3-30-55	STA 3+94	200 (12 TIMES)	20"	280	240	287	—
	24	4-19-55	STA 3+94	200 (18 TIMES)	20"	600	240	234	—
	26	4-20-55	A_1	200 (10 TIMES)	20"	600	240	157	—
	29	4-20-55	A_1	50-100-150-200	8"	600	240	177	—
	27	4-20-55	A_1	50-100-150-200	20"	600	240	154	450
AT SHRINKAGE CRACKS	8	4-28-54	STA 1+65	12^5-25-37^5-50-62^5-75-87^5 100	8"	300	0	103	—
	9	4-29-54	STA 1+65	12^5-25-37^5-50-62^5-75-87^5-100	8"	300	0	086	—
	10	4-29-54	STA 1+65	12^5-25-37^5-50-62^5-75-87^5-100	8"	300	0	091	—
	16	12-9-54	STA 1+65	50-75-100-125-150-175-200	20"	250	0	288	—
	20	2-2-55	STA 1+19 875	25-50-75-100-125-150-180	20"	0	0	262	—
EDGE LOADING	14	4-30-54	STA 1+45	20-40-60-80-95	8"	300	0	~.250	—
	19	2-1-55	STA 3+60	25-37^5-50-100	8"	0	0	192	—
	25	4-19-55	STA. 3+94	50-75-100-125-130	8"	600	240	~ 177	—
	28	4-20-55	STA 1+00	50-100-132	8"	600	240	249	—

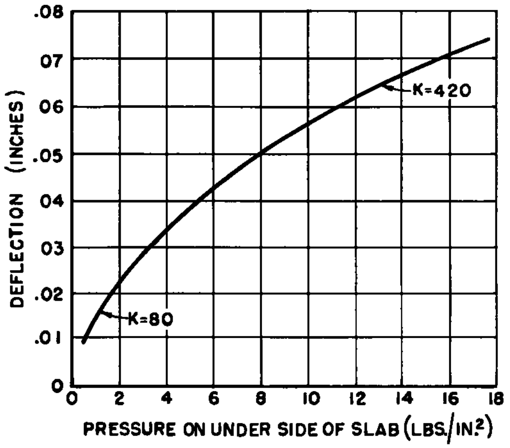

Figure 5. Variation of "K", load test No. 5.

available. Both tests were performed on 20-in. plates. The locations of these tests were selected so that one was performed where the slab was reinforced transversely with No. 8 bars. The results of these tests indicated the beneficial contribution of reinforcement. The slab without prestressing failed under a load of approximately 125,000 lb, when the deflections increased suddenly. Deflections measured during load test No. 21 were considerably lower, but indicated rapidly increasing deflections and a cracking pattern developing similar to that observed during load test No. 18.

Comparing the results obtained on sections with and without prestressing, it is evident that the prestressed sections had an extremely high load-carrying capacity. Deflections remained fairly proportional in the available range and the instantaneous recovery of the slab was never less than 85 percent after the load was removed. There was no cracking observed on the surface, however indications by strain meters located in the bottom part of the slab point to some cracking on the bottom of the slab.

Even if such cracking would have developed, it would not have meant failure and the slab could have resisted considerable additional load before reaching the ultimate condition. It would have been of great interest to obtain information on the behavior of the loaded slab nearing limit conditions. Unfortunately, the estimated loads to produce such conditions were so high that immediately practical means were not at hand.

It is assumed that the slab with prestressing would have offered considerable resistance even after cracks fully developed top and bottom. Such cracking obviously creates a redistribution of moments in the slab, and therefore loads still could be increased considerably thereafter and before failure either in steel or in concrete would take place. If cracks develop without reaching the limit conditions, they will close as soon as the loads are removed and continuity will be restored by the prestressing forces in the slab.

Based on the various deflection characteristics obtained during the various load tests it can be predicted that the slab would not fail under load up to 350,000 lb with 300 psi of prestressing.

Barring the possibility of obtaining the loading equipment of such high capacity, the slab was loaded with 200,000 lb in short cycles during the final plan of the testing.

The measured deflections were higher than those obtained with single loadings during previous tests. It should be noted, however, that in certain tests the load was sustained for a considerable time, as during load test No. 23. In another test, No. 26, when the 200,000 lb load was applied in 10-min. intervals the over-all maximum deflection still remained in the order of those registered with single loadings. As it can be seen the slab did not lose its high recovering ability and in load test No. 26 the instantaneous recovery after the tenth application of the load was over 95 percent.

The deflections observed during these tests may be considered excessive from the point of view of general practice. It was not expected that the base could have followed the concrete slab in its deflections and recoveries. A thorough study was made to determine the magnitude of base reaction under loads of such high order. The pressure meters placed at the two test areas gave valuable information in this respect.

Readings on these meters were made

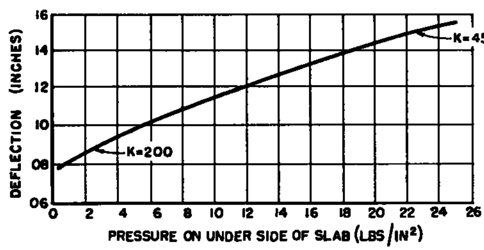

Figure 6. Variation of "K", load test No. 27.

at each load application and thereby relation between loads, deflections and base reactions were established. The relation between pressures and deflections developing under the slab, known as the K value (psi, per in. of deflections) or the supporting value proved to be variable and showed an increasing tendency, with increasing pressures in accompanying each deflection increment.

This finding was in contradiction with the Westergaard theory which assumes a straight propertion between pressures and deflections. A typical case is shown in Figure 5, load test No. 5, where the relation between deflections and pressure indicates a K of 80 in the considerably higher K of 420 for deflections exceeding 0.05 in.

It was also found that after applying high loads, deflections became so excessive that very little recovery took place in the base, and as a result actually a void developed

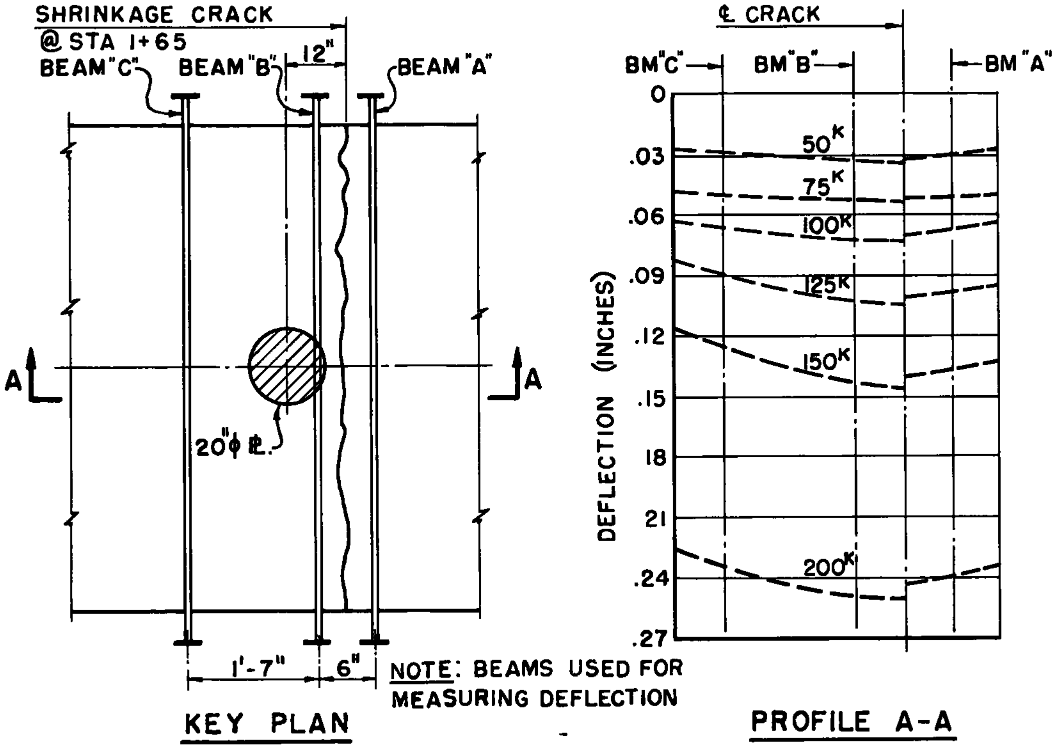

Figure 7. Deflections, load test No. 16; longitudinal prestressing = 250 psi, transverse prestressing = 0 psi.

between the bottom part of the slab and the deflected base. This was clearly demonstrated during load test No. 27, when the 200,000-lb load was applied in several increments, measuring simultaneously the maximum deflections and maximum pressure adjacent to and directly under the load. The results revealed that there was no contact between the face of the pressure meter at the bottom surface of the slab and the base until deflections reached 0.08 in. under an approximate load of 100,000 lb. At this point, the initial K was 200 and increased to 450 under the maximum load of 200,000 lb and a deflection of 0.154 (Fig. 6). It should be noted that load test No. 27 was performed immediately after load test No. 26, at the same location, where the maximum load of 200,000 lb was applied ten times in short cycles.

The five load tests performed at the two shrinkage cracks proved also the beneficial contribution of prestressing.

During load test No. 16, which was performed at Station 1 + 65, there was no indication of failure and only a slightly noticeable shifting took place across the crack (Fig. 7).

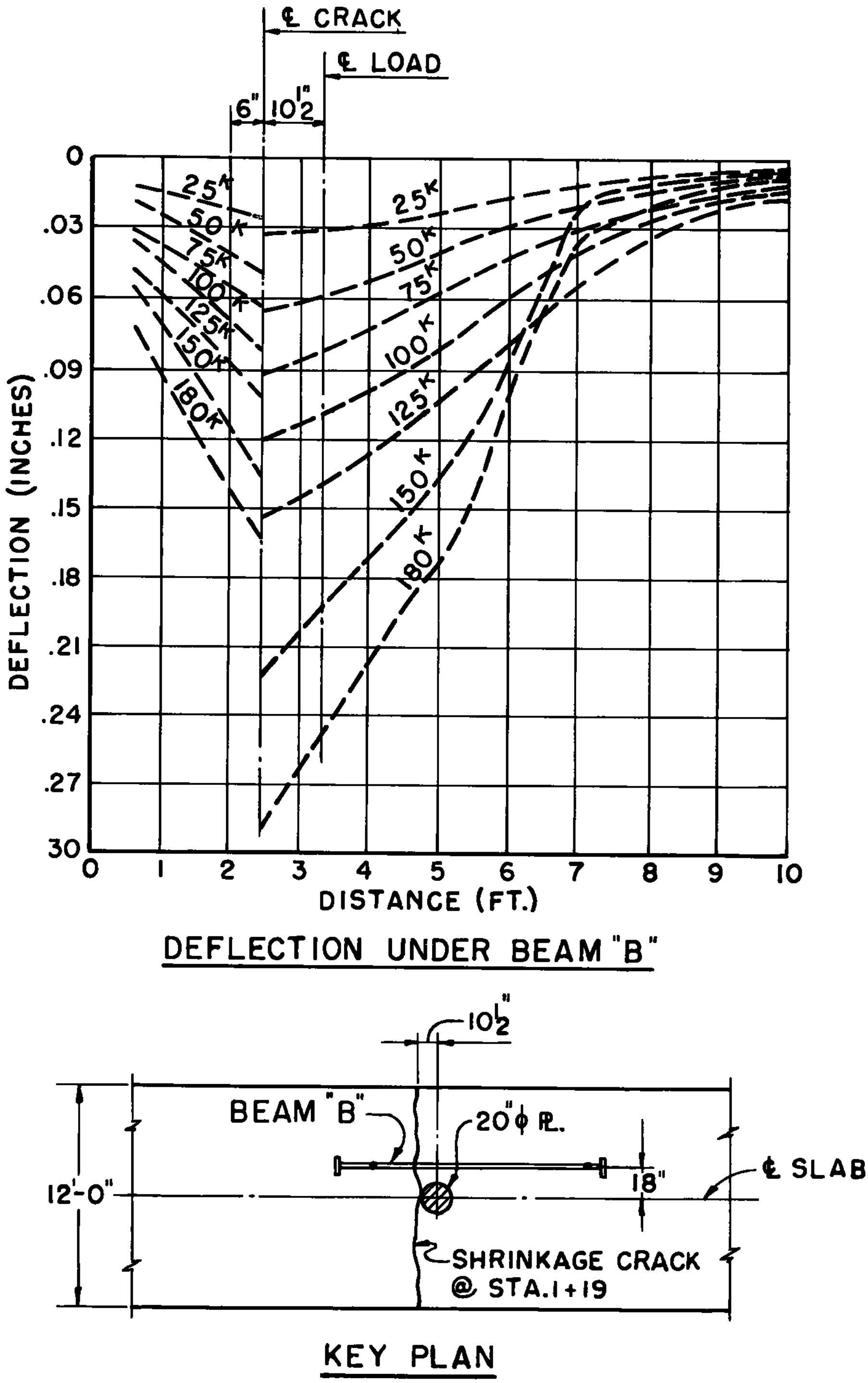

Figure 8. Deflections, load test No. 20; no prestressing in slab.

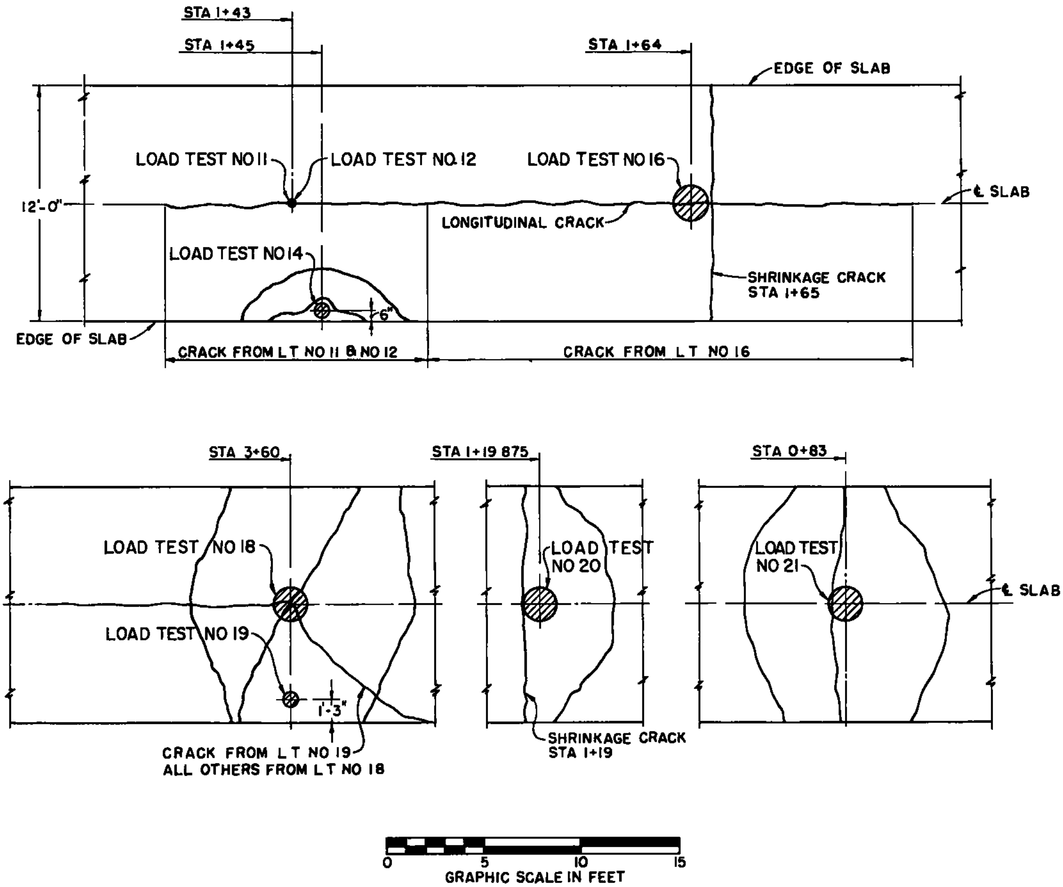

Figure 9. Cracking pattern.

The measured deflections were higher than those registered under similar loads, but at uncracked sections. It should be noted, however, that there was no transverse prestressing at this location. Recovery after removal of the maximum load was 86 percent, which by itself should be good indication as to the strength of the slab.

The results obtained during load test No. 20 indicated shifting across the shrinkage crack after the first load increment was applied. The deflection differential increased thereafter, until the slab failed under a load somewhat over 125,000 lb. The location of the crack which developed can be easily determined from the deflection profile (Fig. 8).

Load tests performed at the edges of the test slab lead to some information on the effect of prestressing on that portion of the slab.

All these tests were performed on an 8-in. plate. During load test No. 14, when the slab was prestressed only longitudinally to about 300 psi, failure occurred after the load exceeded 75,000 lb and deflection 0.12 in. The general cracking pattern is shown in Figure 9. There was no cracking transversely on the bottom of the slab.

The results obtained during load test No. 19 unfortunately cannot be considered of full value because the load was applied in an area where cracking occurred from previous loadings (Fig. 9). It should also be noted that in this case the load was placed 1 ft 3 in. from the edge of the slab as compared to 6 in. in the other cases.

CONCLUSIONS OF THE TESTS

The test indicated the over-all superior behavior of the prestressed concrete slab. Sand and building paper proved an efficient solution to reducing base friction.

Losses due to conduit friction of the strands can be kept to a minimum providing stiff covering material and using suitable placing and spreading of concrete.

Losses in the strands thereafter can be expected not to exceed 10 percent, even in extreme cases.

The problem of developing prestressing in the concrete has not been completely clarified, mainly because of the presence of several factors affecting base friction. It was found, however, that prestressing can be developed under given circumstances in the most efficient manner when the continuously changing volumetric changes in the concrete have a decreasing tendency (i.e., when the concrete tends to contract).

The effect of repeated prestressing in short cycles proved also beneficial.

As far as load-carrying capacity is concerned, the slab could have resisted extremely high loads. The added compressive energy in the concrete provided by prestressing, coupled with its spring effect would make considerable reduction in pavement thickness possible.

Load tests performed at the shrinkage cracks and at the edges also proved the beneficial contribution of prestressing in concrete pavements.

GENERAL CONCLUSIONS

The test program and the following conclusions of course did not cover all problems related to this novel pavement. The possibilities of most economical and most efficient means of providing prestressing in a concrete pavement have not been investigated.

Prestressing in a pavement can be provided either externally by using hydraulic jacks or internally by prestressing elements which are embedded in the slab.

The use of jacks seems to be economical, especially if they could be used in between a series of slabs, with few abutments. The cost of expensive abutments can thereby be spread over a considerable length of pavement section. The cost of wires or strands would be higher, but such elements would offer greater advantages than those obtained by jacks.

External prestressing represents only an additional compressive and flexural strength in the concrete section, which once exceeded by either loads or stresses created by restrained volumetric changes, would be completely lost and would lead to the failure of the slab. On the other hand, the presence of internal elements represent a tension field in the concrete section, which, coupled with the resistance of the concrete in compression, will be capable of resisting higher loads. When such loads are applied the section will behave as a reinforced concrete section and will be capable of carrying additional loads until ultimate conditions are reached either in the concrete or steel.

Volumetric changes which develop mainly as the result of temperature variations have little effect on the prestressing forces if they are created by internal steel elements because the thermal expansion coefficient of steel and concrete differ only slightly. The preservation of constant prestressing forces, as applied by hydraulic jacks, would become problematic in view of the continuous volumetric changes, which will take place in the concrete. Assuming a modulus of elasticity of 4 million and a thermal expansion coefficient of 6 millionths and no friction under the slab, a prestressing of 300 psi could be offset by a temperature drop of 12.5 deg F.

It is felt that prestressing provided by hydraulic jacks should be applied only as an auxiliary means in addition to internal steel elements.

No preference can be given to wires or strands, because from the theoretical point of view both satisfy equally the requirements of efficient prestressing. The selection of one or the other must be made with regard to the over-all economy in their respective application. In addition to material cost, expenses in connection with their handling, placing, stressing and anchoring should be considered. It is, of course, assumed that wire if used, would be grouped into units of several wires and protected so by adequate tubing.

The elements should be placed prior to placing of the concrete if post-tensioning is used. Conduits provided in the concrete and pulling the prestressing elements through them after the concrete has hardened seems to be an unnecessary complication. The tests indicated that with sufficiently stiff tubing around them and after careful placing, the conduit resistance in these prestressing elements will be negligible.

It seems to be unlikely that pretensioning of pavements will be adopted. More consideration should be given, however, to the possibility of whether the post-tensioned elements should be left free in their conduits, or should be grouted after stressing.

The primary concern is, of course, corrosion which may necessitate the grouting of these elements to secure long life to prestressed concrete pavements. It is also evident that for ultimate stress conditions grouted prestressed concrete sections offer a higher safety limit than comparable sections which have ungrouted tendons. This matter of safety, however, is of lesser importance because as is evident there is no need for such extra safety margin. Ungrouted prestressing elements furthermore will offer an over-all spring effect to pavement sections and will secure over-all continuity of the slab more likely than grouted elements.

The grouting of conduits several hundred feet long will, of course, increase the cost also. Therefore, it is felt that finding lasting lubricants with corrosion resisting properties would offer the best solution for future applications.

The cardinal problem in designing future prestressed concrete pavements will be the complex problem of determining the amount of prestressing and the thickness of the pavement. There are no theories available for prestressed slabs supported by an elastic foundation. Even if the various available theories would be further developed to include the effect of prestressing on deflections and stresses, they could not give sufficient indication as to the behavior of the slab after initial cracking took place and certain sections of the slab lost their homogeneous characteristics. Empirical solutions will, therefore, be the only ones at hand and should be accepted until better means are found.

As emphasized in the preceeding paragraphs, the main advantage of prestressing pavements is that they become continuous and by virtue of their inherent compressive energy they remain so, even if accidental reasons such as overloads or excessive restrained stresses create temporary discontinuity.

Therefore, the minimum of prestressing applied should be sufficient to maintain and restore continuity in the pavement. It is felt that for such purpose 100-150 psi prestressing is adequate in addition to the prestressing which may be lost due to base friction. The over-stressing of the slab does not seem to be justified for several reasons.

Considering conventional pavements of 6 to 8 in. in thickness, the actual saving in concrete when prestressing would be used could not affect the additional expense of providing the prestressing element with present prices and construction techniques in view. This is mainly true because of limitations set for reducing pavement thickness. With conduits to be used of approximately one inch in diameter and providing sufficient clearances for possible transverse reinforcement and adequate cover for protection, the practical limitation for pavement thickness would be 5 rather than 4 in. Pavements with less thickness would require high prestressing in the order of 500-600 psi to offer sufficient resistance to loads. With such higher prestressing the anchorage of the prestressing elements would become problematic because of the high concentration of loads. High prestressing would also increase the number of tendons to be used and would thereby create increased difficulties for placing concrete.

Pavements, if highly prestressed, would carry loads on average subbases with considerable deflections without detriment to the concrete slab but definitely causing permanent deformations in the supporting material.

The amount of prestressing necessary to overcome friction of the base is a function of length of the slab and of the unknown friction coefficient. This coefficient should be in the order of 0.5. To secure the proper development of prestressing, it should be applied with temperatures showing a decreasing tendency and in repetition (i.e., the prestressing elements should be released and restressed once but preferably twice after the first stressing).

It was assumed in the preceeding paragraphs that prestressing is to be applied only in one direction. This is recommended for highway pavements. There seems to be no need for additional strength transversely except for some light transverse reinforcement. This reinforcement should be placed above the longitudinal prestressing elements.

The prestressing elements should be placed directly under the neutral axis of the slab.

The preferred position of these elements cannot be precisely defined. The test performed at Patuxent did not lead to the clarification of this problem. Unfortunately, with the limited capacity of the loading equipment, it was impossible to produce limit conditions which would have shed some light on the effect of strands placed at various levels.

The term continuity used in connection with prestressed concrete pavement should by no means indicate that such pavements could be built practically without any limitations in length. Also, not all joints will be eliminated. It is more reasonable to speak about continuous panels, with special joints provided between them which will permit the free movements of the ends of such panels.

Prestressed concrete pavements are undoubtedly superior in general behavior to conventional concrete pavements. They seem to provide not only more strength, but also reduce problems which are connected or created by pavement thicknesses, joints, cracking and some other phenomena to be dealt with. As a matter of fact, they seem to offer answers to a good many problems for which highway engineers seek solutions.

It would be unwise, however, to overlook important problems of rigid pavements for which even prestressed concrete cannot offer a satisfactory solution.

First of all, it must be emphasized that it would be a fallacy to believe that the adoption of prestressed concrete pavements would lead to the acceptance of poorer subbase than those presently used. Economical prestressed concrete pavements must be thin and thereby more flexible. Such flexibility itself will result in greater deflections. In this case, attention must be directed to the permissible limits of base deflections in determining the quality of material underlying prestressed concrete pavements.

Pumping, which is one of the paramount problems of rigid pavements is not going to be solved by prestressed concrete pavements either. As a matter of fact, decreased pavement thicknesses and increased deflections resulting therefrom could add to this problem.

There are a good many other problems which must be closely investigated and solved before the adoption of this novel pavement.

Joints, as mentioned previously, cannot be eliminated completely. Their number can be reduced, but by doing so, the anticipated magnitude of movement at such joints may be in the order of 2-3 in. Therefore, special expansion joints will have to be devised, which by itself will require some ingenuity. In addition, such joints will represent free edges in the pavement right there, where most likely heavy concentrations of applied prestressing forces are present.

The construction of such pavements will also create problems not experienced before. Irregular pavement sections to be formed at intersections, pavement widenings, etc., together with horizontal and vertical curves will present design and construction details which will not always be solved easily.

Damage to such pavements, especially to their prestressing elements will be more detrimental and will call for more elaborate repair procedures.

Economy above all will probably be the deciding factor. Prestressed concrete pavements should be considered economical only if the savings in reduced concrete thicknesses will not be offset by the cost of prestressing elements and the cost of related new construction methods. In such comparison, the probable increased life-time of prestressed concrete pavements should also be taken into consideration.

The few attempts made, mainly in France and England, point to the future possibilities of such pavements. It should be noted that most of the experimental work in the field of prestressed concrete pavement was aimed at the improvement of airfield pavements. There loads and resulting pavement thicknesses increased to such an extent that the advantages of prestressed concrete, both theoretically and economically, could be utilized in satisfactory manner. The elimination of joints, furthermore, would solve maintenance problems of pavements where jet planes are in great use.

Therefore, it is believed that highway engineers should follow with great anticipation and interest the various experimental projects which are planned or conducted by various agencies to investigate the adoptability of prestressed concrete to airfield pavements.

There is need, of course, for more research and development to solve problems of this novel pavement related directly to highway applications. Whether prestressed concrete will be used in the future for the improvement of concrete highway pavements can be decided only after such extensive investigations.

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Model Studies of Prestressed Rigid Pavements for Airfields

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● GROSS WEIGHTS of current and proposed military aircraft have reached such proportions that as much as 30 in. of plain concrete may be required to provide an adequate design for heavy-duty airfield pavements. The search for an improved method of constructing such heavy-duty pavements has led to the consideration of prestressing rigid pavements; that is, subjecting rigid pavements to compressive stresses of sufficient magnitude to reduce significantly the critical tensile stresses normally produced by service loadings. Little data, however, are available upon which to base a rational method of design for prestressed pavements, since the use of prestressing in the United States has been confined almost entirely to structures other than airfield and highway pavements. In 1953, the Corps of Engineers initiated studies to determine the feasibility of utilizing prestressed pavements for military airfields and to develop procedures for the design and evaluation of such pavements. These studies are being made as a part of the Rigid Pavement Investigational Program conducted by the Ohio River Division Laboratories.

In one phase of the ORDL program, small scale models have been used to provide basic information concerning the behavior of prestressed rigid pavements under various conditions of loading. This paper presents the significant information obtained from the model tests completed to date. As such, this paper should be considered a progress report rather than a final report of the model tests.

TEST PROGRAM

In the model, prototype conditions were simulated through the use of small prestressed gypsum cement slabs placed on an artificial subgrade of natural rubber. Static loads were applied to the slabs by means of single footprints of various sizes. Whenever possible, comparable tests were made on plain slabs having no prestressing and on slabs having unequal amounts of longitudinal and transverse prestressing so that the performance of these types of pavements could be compared with pavements having equal longitudinal and transverse prestressing. In the testing completed to date, the following types of observations and measurements have been made: (a) crack patterns and crack development for various conditions of loading, (b) deflections measured for interior and edge loadings for elastic and elasto-plastic conditions in the slabs, (c) maximum strains and strain distribution measured for interior and edge loadings within the elastic limit of the slabs, and (d) ultimate load-carrying capacity of the slabs for various conditions of loading. A general view of the model and the auxiliary equipment is shown in Figure 1.

DESIGN OF THE MODEL

In order that Westergaard's theoretical analyses of rigid pavement behavior could be applied to certain phases of these studies, it was necessary to consider the simulation of some of the basic assumptions inherent in his analyses. The theory assumes that the materials comprising the slab and subgrade are homogeneous and isotropic, that the slab is of uniform thickness, and that the slab is large enough to act as though infinite in horizontal extent. Since Westergaard's analyses are valid only where critical stresses in the slab and subgrade remain within their respective elastic limits, no correlation between the theory and the model existed for conditions of inelastic action in the slabs.

The model slabs were cast from Hydrocal gypsum cement. Each slab was 16.6-in. square and approximately 0.20-in. thick. For this thickness, the radius of relative

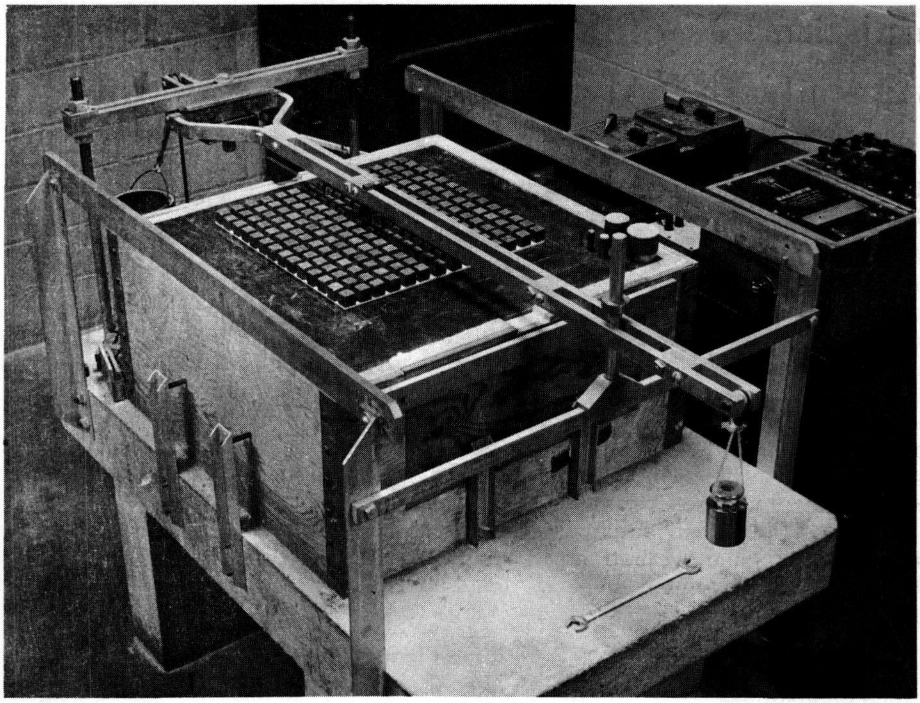

Figure 1. Model table with prestressed slab in place for testing.

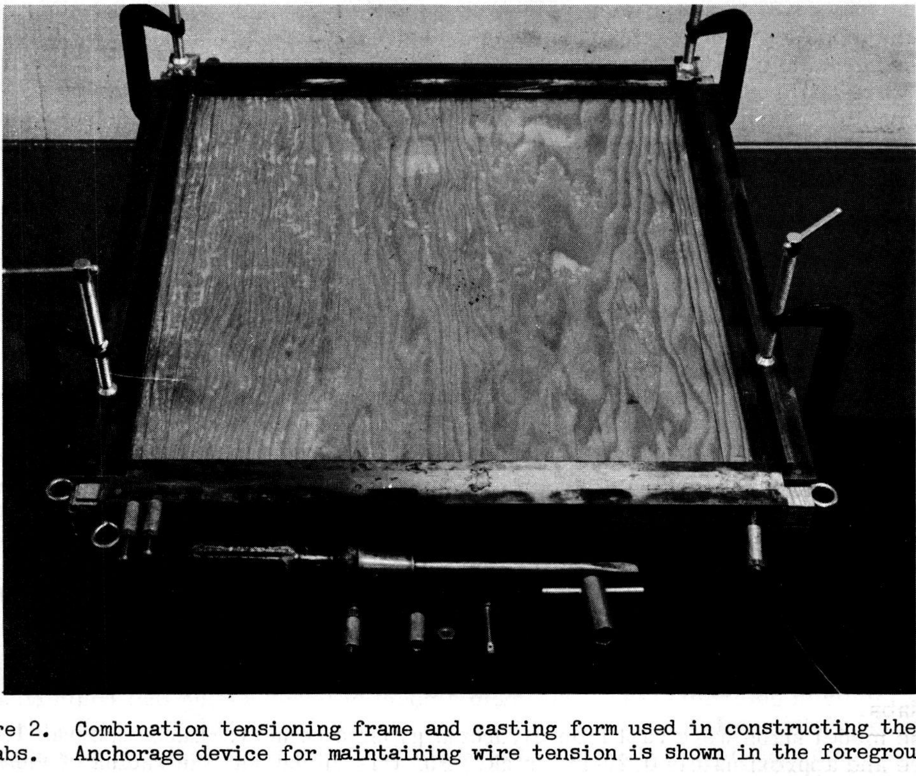

Figure 2. Combination tensioning frame and casting form used in constructing the test slabs. Anchorage device for maintaining wire tension is shown in the foreground.

stiffness, l, of each slab was 2.4-in. making the horizontal dimensions of each slab 7 l by 7 l. For all practical purposes, slabs of this size act as though infinite in extent as required by the Westergaard analyses. To increase the effective mass of the slabs and thereby insure intimate contact at the interface between the slabs and the subgrade, a layer of ¾-in. lead cubes was distributed uniformly over the surface of each test slab.

For the hydrocal, values of the various physical properties used in the analysis of the model tests were determined to be:

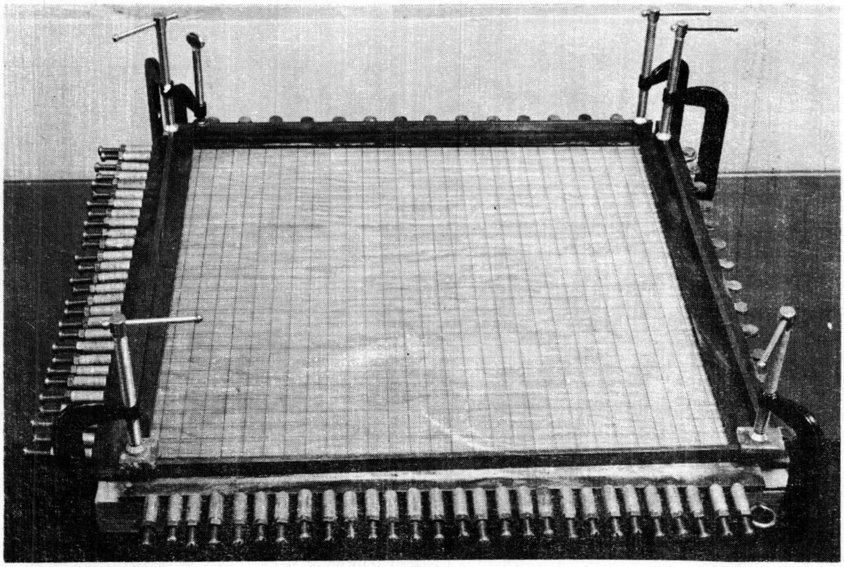

Figure 3. Longitudinal and transverse wires in place ready for the final tensioning.

Modulus of elasticity - 1.50×10^6 psi,

Poisson's ratio - 0.225,

Flexural strength - 940 psi,

Compressive strength - 3,200 psi.

A complete description of the above model techniques as applied to the study of plain concrete pavements has been given in (1) and (2).

Prestressing

The slabs were prestressed longitudinally and transversely by tensioned music wires positioned at the neutral axis of each slab. The decision to place the stressing wires at the mid-depth of the slabs was based largely on the fact that for testing within the elastic limit of the slabs, the presence of the wires should have no effect on altering the stiffness of the slabs. Wire spacing, wire diameter and the range of loads applied to the wires used in the model were chosen arbitrarily to provide a range of conditions of prestressing similar to that anticipated for prototype pavements. Although prestressing in the prototype may be effected by tensioning the stressing tendons either before or after the placing and curing of the concrete, only the "pre-tensioning" technique was used in the construction of the model slabs.

Inasmuch as the bond developed between the Hydrocal and the music wire was an important factor in determining the maximum amount of prestressing that could be applied to the slabs, pull-out tests were made on various lengths of wire embedded in Hydrocal beams one inch square in cross-section. From these tests, it was concluded

TABLE 1

SUMMARY OF DESIGN VARIATIONS IN THE PRESTRESSED SLABS

Slab Number	Slab Thickness in.	Wire Spacing, in.		Prestressing, lb/in.^{2}		Radius of Relative Stiffness, in.		Remarks
		Long.	Trans.	Long.	Trans.	Interior k = 35	Edge k = 65	
1-X	0.204	0.6	0.6	150	150	2.8360	2.4296	Preliminary-Hydrostone
2-X	0.201	0.6	0.6	600	600	2.3509	2.0140	Cracked diagonally in releasing
1	0.202	0.6	0.6	600	600	2.3596	2.0215	
2	0.200	0.6	0.6	600	600	2.3422	2.0066	
3	0.206	0.6	0.6	600	600	2.3946	2.0515	
4	0.202	0.6	0.6	600	600	2.3596	2.0215	
5	0.206	1.2	1.2	360	360	2.3946	2.0515	
6	0.204	1.2	1.2	360	360	2.3771	2.0365	Two small edge cracks in releasing
7	0.214	1.2	0.6	360	600	2.4641	2.1110	Three wires failed in bond-replacement slab being constructed
8	0.205	0.6	0.6[a]	600	360	2.3860	2.0441	
9	0.206	0.6	None	600	0	2.3946	2.0515	
10	0.204	0.6	0.6	600	600	2.3771	2.0365	
11	0.208	1.2	1.2	360	360	2.4121	2.0664	
12	0.208	0.6[b]	0.6	360	600	2.4121	2.0664	
13	0.201	0.6	0.6	600	600	2.3509	2.0140	
14	0.208	0.6	0.6	600	600	2.4121	2.0664	

[a] 45 lb per wire — all others 75 lb per wire

[b] 0.14-in. diameter wire — all others 0.20-in. diameter wire

Slabs constructed of Hydrocal (except No. 1-X), $E = 1.5 \times 10^6$, $\mu = 0.225$

TABLE 2

SUMMARY OF PRESTRESSED STRAINS, TEST SLAB NO. 12

Gage No.	Measured Strain x 10^6 in./in.					
	0 hr	6 hr	24 hr	30 hr	48 hr	Average 48 hr
1	52	62	134	111	123	
5	158	146	166	182	167	
6	36	50	98	113	124	138
2	243	256	295	294	314	
3	305	331	357	361	373	
4	345	437	460	466	469	385
7	200	222	225	239	231	
8	206	223	242	251	248	240

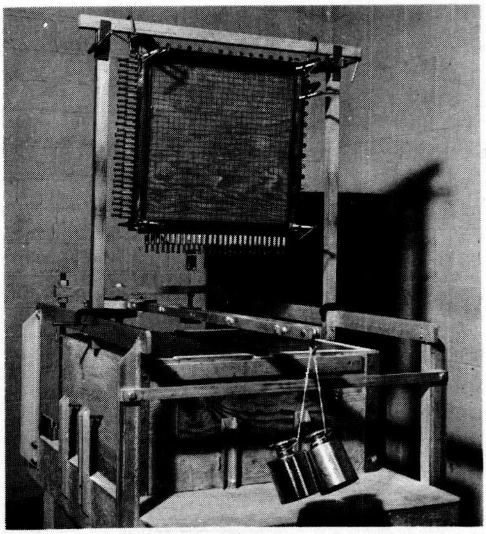

Figure 4. Arrangement for applying the desired tension to the prestressing wires. Tension developed in the wires by means of the reaction beam is maintained by tightening the lock nut on each anchorage device.

that the individual wires in the model slabs could be pretensioned as much as 75 and 50 lb for 0.020- and 0.014-in. diameter wire, respectively, without danger of exceeding the bond strength.

Most of the model slabs constructed to date have been stressed with 0.020-in. diameter (No. 8) wire which had an average tensile strength of 366,000 psi. The maximum loading of 75 lb produced an initial tensile stress of 239,000 psi or approximately 65 percent of the ultimate tensile strength of the wire. For a uniform wire spacing of 0.6-in. center-to-center and a pretensioning load of 75 lb per wire, a maximum prestress of 600 psi was developed in the model slabs.

Subgrade

In the model, simulation of the prototype subgrade was made by using a 24-in. square block of natural rubber 12-in. in depth. This block of rubber was supported rigidly by a concrete table and the sides were restrained laterally by a rigid casting. Since, in terms of l, this subgrade was 5 l deep, it acted as though infinite in depth as required by the Westergaard analyses. The modulus of subgrade reaction, k, of the rubber was determined to be 65 lb per cu in. for edge loadings and 35 lb per cu in. for interior loadings.

Loading

Service loadings on the prototype were simulated in the model through the use of various sizes of circular and elliptical footprints. The footprints were loaded statically by means of the reaction beam as shown in Figure 1. For the tests reported herein, only single-wheel loadings were investigated.

Instrumentation

Instrumentation used in the testing was limited to that necessary for observing strains and deflection developed in the model slabs. Strain measurements were made using

TABLE 3

COMPARISON OF FAILURE LOADS FOR PLAIN AND PRESTRESSED SLABS LOADED AT THE INTERIOR

Footprint Radius (in.)	Load (lb)			
	Plain		Prestressed 600 psi x 600 psi	
	First Crack	Ultimate Failure	First Crack	Ultimate Failure
0.2	30	74	42	95
0.3	36	72	50	125
0.4	40	86	59	150
0.5	44	90	66	180
0.75	54	104	88	228

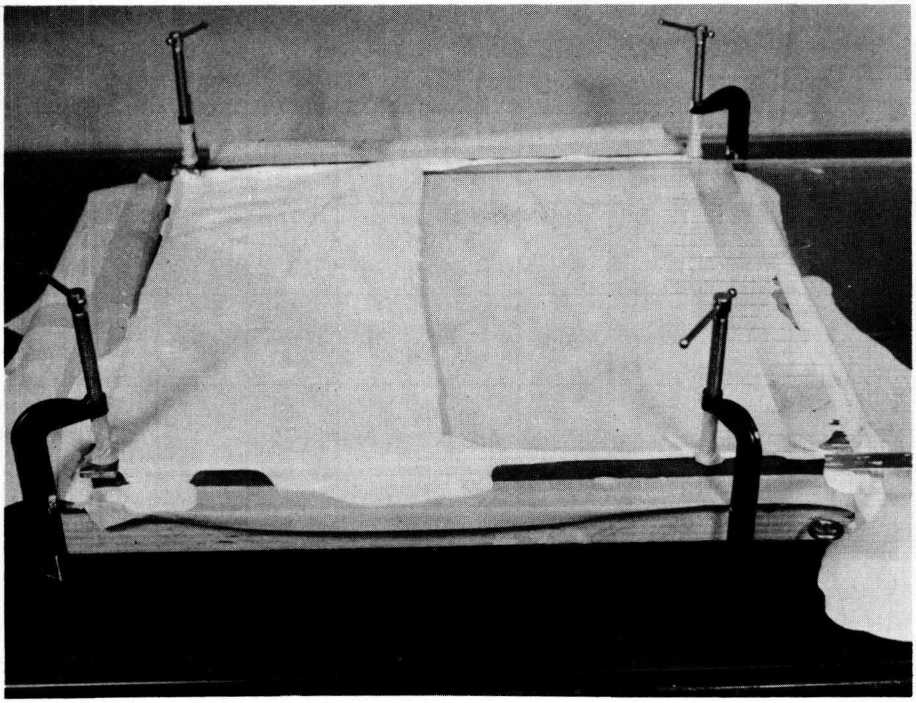

Figure 5. Striking off the top surface of the slab using a $\frac{1}{2}$-in. thick glass plate.

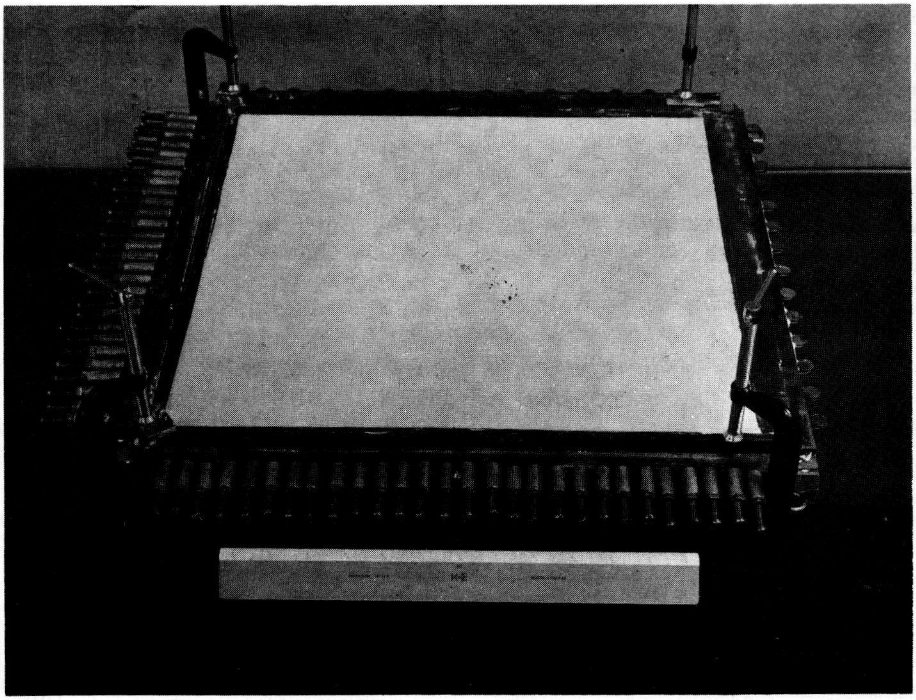

Figure 6. Test slab approximately 45 min after casting. Prestressing applied and casting frame removed after a minimum curing period of 7 days.

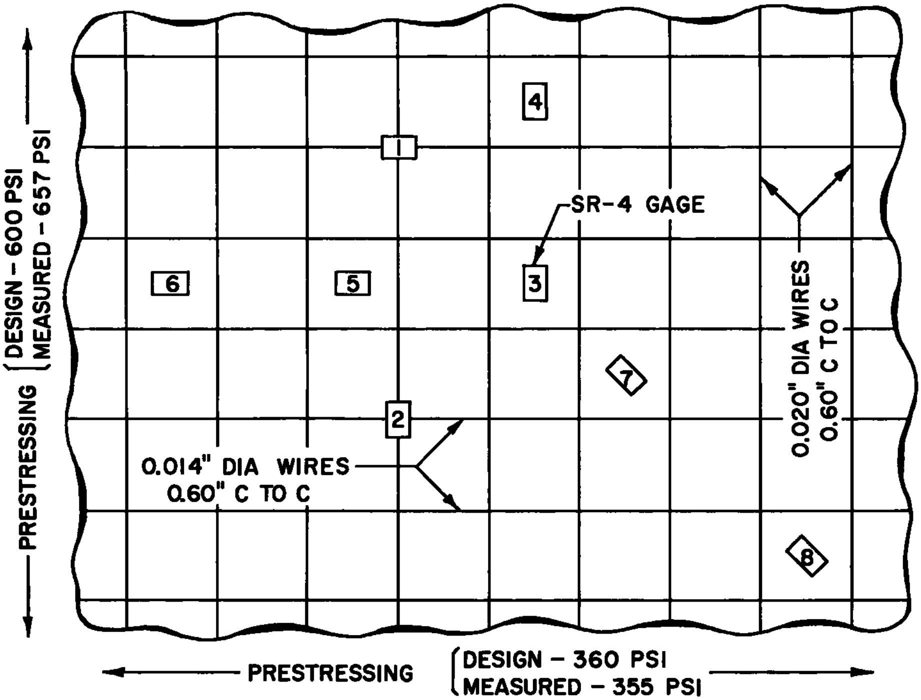

Figure 7. Arrangement of SR-4 strain gages for measuring effective prestress - Slab No. 12.

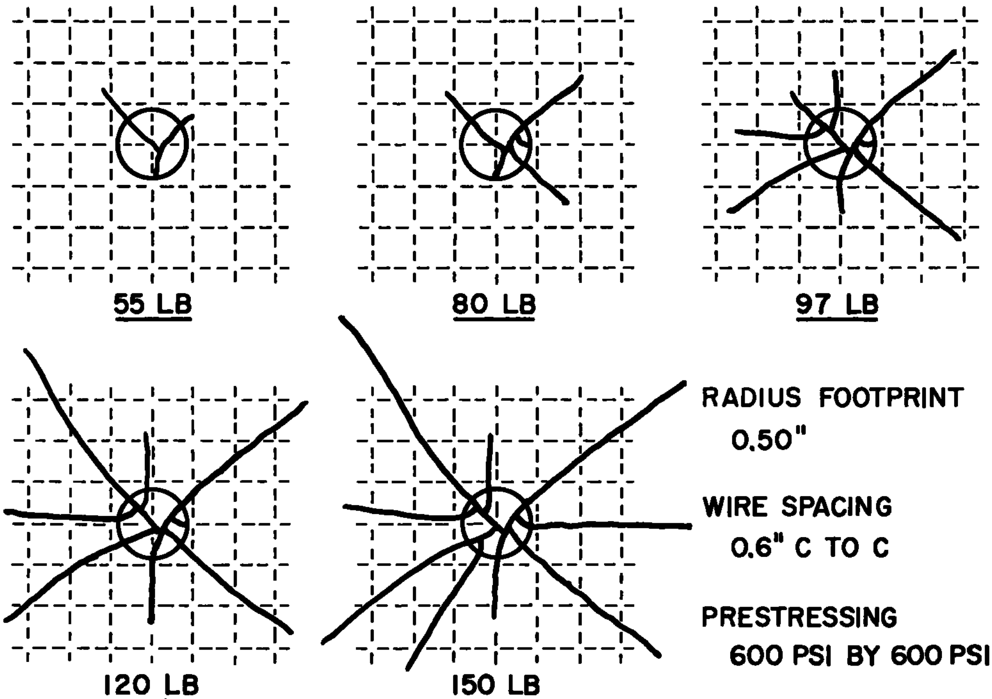

Figure 8. Progression of cracking for interior loading - Slab No. 1.

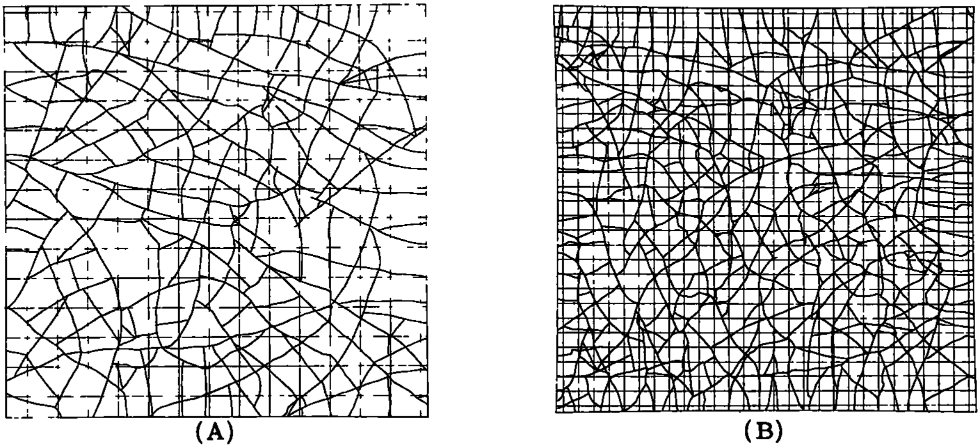

Figure 9. (A) Test slab No. 5, prestressing 360 psi by 360 psi, wire spacing 1.20 in. by 1.20 in., 75-lb load; (B) Test slab No. 10, prestressing 600 psi by 600 psi, wire spacing 0.60 in. by 0.60 in., 90-lb load. Crack patterns developed in the bottom side of two test slabs. Slabs loaded using an 0.75-in. radius footprint applied at 1-in. intervals over the entire area of the slab.

Type A-7, SR-4 gages and a Baldwin SR-4 Control Box. Dial gages reading direct to 0.0001-in. were used in measuring all deflections.

CONSTRUCTION OF THE MODEL SLABS

The model slabs were cast in a form constructed of four 18- by 1.5- by 0.20-in. steel bars clamped to a base consisting of a glass plate cemented to a 1½-in. thick piece of plywood. The steel side forms were fabricated with 0.031-in. diameter holes at mid-depth and spaced 0.6-in. center-to-center. The stressing wires were strung through these holes so that all wires in the same direction were in the same plane and the neutral axis of the slab was at the point of tangency of the two layers of wires. Each wire was secured by an anchoring device which permitted adjustment of the tension in the wire to any desired value. Details of the stressing frame, anchorage devices and sequence of placing the wires are shown in Figures 2 and 3.

The required tension in each stressing wire was obtained by using the reaction beam of the model table to apply the proper load, as illustrated in Figure 4. Correct tension was maintained in the wire by tightening the lock nut on the anchorage device. To compensate for the deflection of the steel frame which resulted from tensioning the wires, it was necessary to repeat the application of the load on each wire. Normally, two checks on the tension in the wires were made prior to casting the slab.

Following the final tensioning of the wires, the slab was cast from Hydrocal gypsum cement having a water-cement ratio of 54 percent. The strike-off of the top side of the slab was made with a ½-in. thick glass plate as shown in Figure 5. The use of glass plates to form the top and bottom sides of the test slabs produced uniformly thick slabs having smooth plane surfaces.

Approximately 45 min after the slab was cast, the strike-off glass was removed and the excess Hydrocal cleaned from the side forms and wire anchorages. A completed slab approximately one hour after casting is shown in Figure 6. The slab was then allowed to cure in air under controlled temperature (73 deg F.) and humidity (50 percent) for a minimum period of seven days. Following the curing period, the prestressing was applied to the slab by releasing the wires from the anchorages and removing the slab from the form. For those slabs were SR-4 strain gages were used to check the effective prestressing developed in the slab, the gages were cemented to the top surface of the cured slab and allowed to air-dry 48 hr before the application of the prestressing.

A total of 16 test slabs have been constructed to date for this study. Several variations in prestressing have been incorporated in these slabs to provide data on the relative effects of equal longitudinal and transverse prestressing, unequal longitudinal and transverse prestressing, and prestressing in one direction only. These variations were accomplished by altering the spacing and diameter of the stressing wires, and by changing the tension in the wires. Table 1 is a summary of the physical characteristics and conditions of prestressing for each slab.

TEST RESULTS

Check on Prestressing

For several slabs an attempt was made to determine the net amount of prestress developed by the method of construction employed. This was done by mounting SR-4 strain gages on the top surface of the slab prior to the release of the wires. The arrangement of the strain gages for Slab No. 12, as shown in Figure 7, was typical for all such testing.

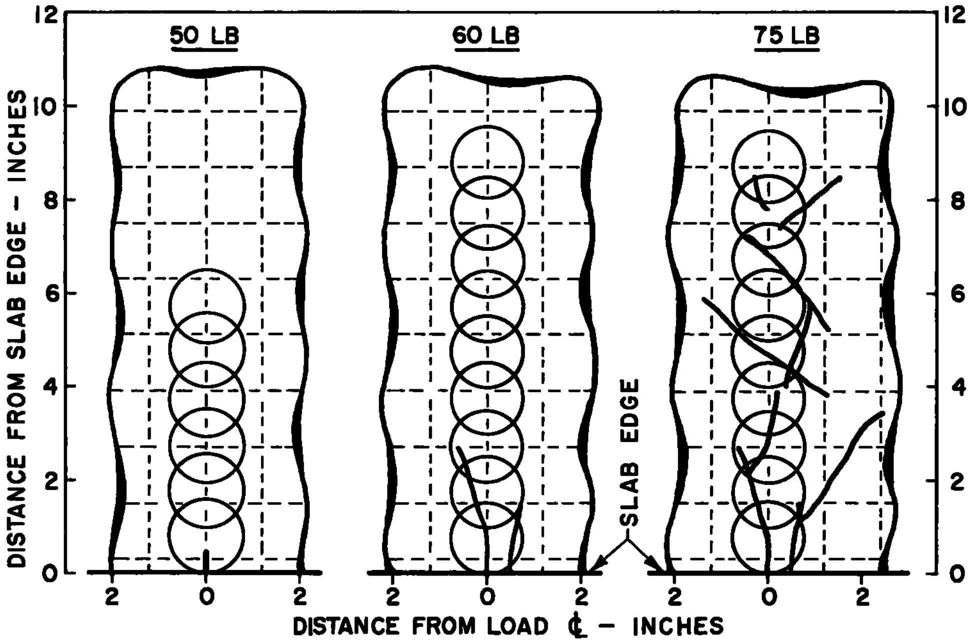

Figure 10. Development of cracking for simulation of wheel-load moving from a free edge to interior — Slab No. 6, 0.75-in. radius footprint.

Strains were read immediately following the release of the stressing wires and again at various intervals for a period of 48 hr thereafter. The prestressing was computed on the basis of the average strains obtained from each group of parallel gages. The results of these measurements indicated that the full amount of prestressing was not realized immediately, but reached a maximum approximately 36 to 48 hr after application. The initial prestress immediately after releasing the wires was generally 75 percent of the ultimate amount, with 95 percent of the ultimate being reached within 24 hr after releasing. Typical strain data, as obtained from Slab No. 12, are shown in Table 2. Based on the average strains measured 48 hr after releasing, the prestressing was 355 psi longitudinally and 657 psi transversely as compared to design values of 360 and 600 psi, respectively.

Crack Patterns

One of the attributes of testing with a small scale model is the fact that the slab can be removed from the model table for examination after being subjected to various loadings. This made it possible to observe the development of crack patterns in the bottom side of the slab as loadings were carried beyond the elastic limit.

In order to study the progression of cracking for a single position of the load in the interior of a slab, the load was applied in increasingly greater amounts until the initial crack occurred. The slab was then removed from the model table and the location and extent of the crack noted. After replacing the slab on the subgrade, the load was increased further until additional cracking occurred. Typical of the results obtained using this procedure are the crack patterns shown in Figure 8 for loadings on an 0.5-in. radius footprint positioned near the center of Slab No. 1.

As can be seen in Figure 8, the cracking is predominantly radial in nature and originates under the center of the loaded area. Although the 150 lb load represented approximately 85 percent of the ultimate load for Slab No. 1, at no time were any cracks observed in the top side of the slab. Testing of this type was repeated on other slabs having wire spacings and prestressing different from that in Slab No. 1 without significant differences in the crack patterns formed. Similarly, it was observed that the pattern of cracking was not affected by centering the load over a wire intersection or between the wires.

Somewhat different crack patterns were formed in Slabs Nos. 5 and 10 as the result of applying a load large enough to cause cracking at all points over the surface of each slab. These patterns, shown in Figure 9, were developed by loading each slab with an 0.75-in. radius footprint at one-inch intervals over the entire slab area. Loading of this nature more nearly simulates the loading experienced in the prototype than does the idealized loading shown in Figure 8. A comparison of the crack patterns for Slabs Nos. 5 and 10 reveals a closer spacing of cracks for a closer spacing of stressing wires. The exact effect of the difference in wire spacing was obscured to a degree by the fact

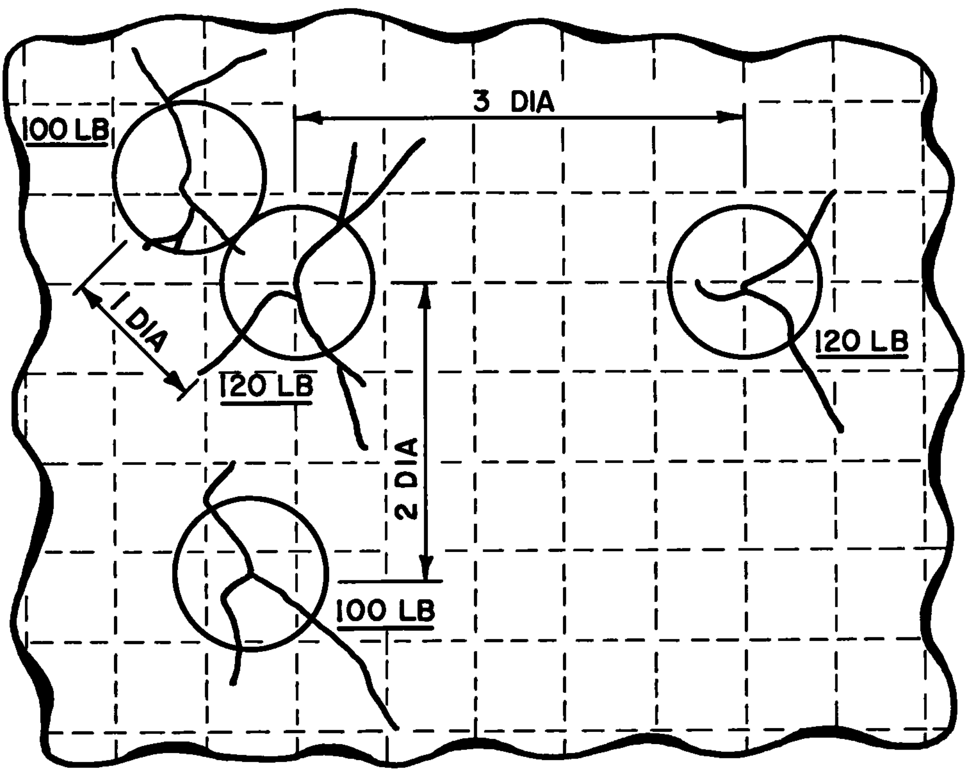

Figure 11. Formation of independent plastic hinges for interior loading - Slab No. 14.

that the greater prestress in Slab No. 10 required a heavier load to produce cracking. Again, none of the cracking was visible on the top side of either slab.

In an attempt to obtain information pertaining to the relative load-carrying ability of a prestressed slab subjected to edge loading and interior loading, Slab No. 6 was loaded in a manner as shown in Figure 10. Using an 0.75-in. radius footprint, the slab was loaded initially with the footprint tangent to a free edge and then moved at intervals of one inch to the center of the slab. This procedure was repeated for three loadings, beginning with the load producing the initial edge crack. The data presented in Figure 10

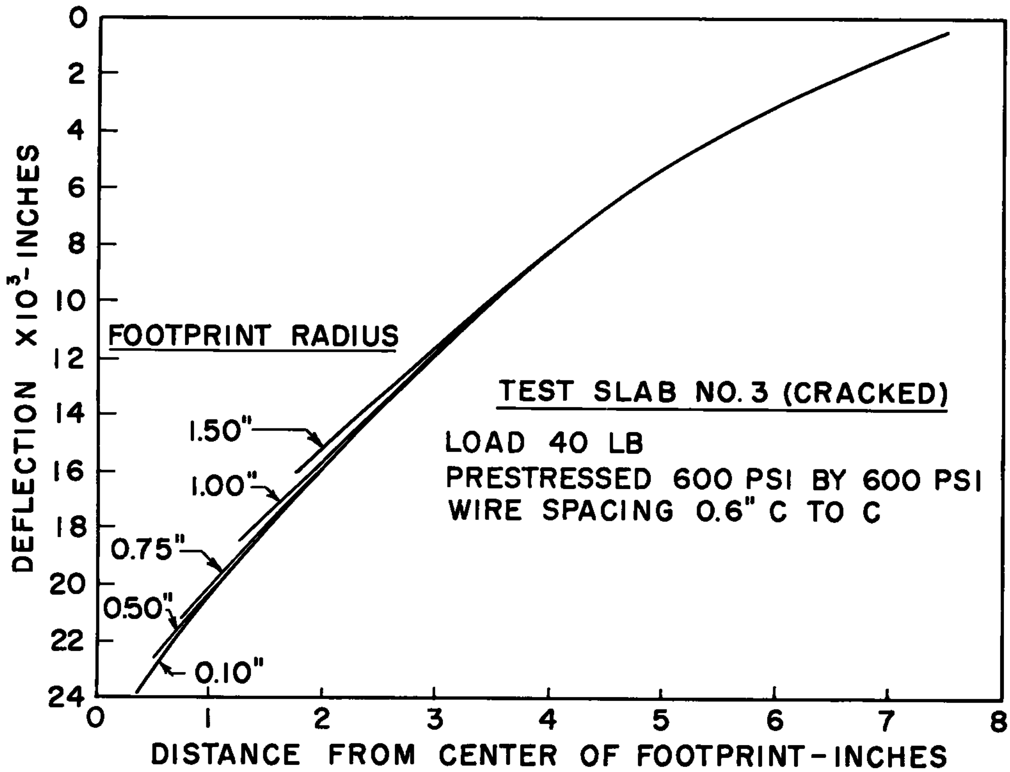

Figure 12. Effect of footprint size on deflections for interior loading of cracked slab.

clearly indicates the inherently greater load-carrying ability of the slab when subjected to interior loadings as compared to loadings at a free edge.

Basic information regarding the formation of plastic hinges within a prestressed slab was obtained from tests on Slab No. 14. Using an 0.5-in. radius footprint, loadings were made to determine the minimum distance between load applications that would result in the formation of independent plastic hinges. With the footprint positioned near the center of the slab, a load was applied until the initial crack was formed in the bottom side of the slab. The footprint was then moved, in order, to alternate positions three, two, one, and three-quarters diameters distant from the original load position. These alternate load positions were selected so that the cracking resulting from the original load was tangential, insofar as possible, to them. At each alternate position of the footprint, the slab was loaded until new cracking occurred. The relative positioning of the footprint for loadings producing independent cracking and the cracks developed at each of these points of loading are shown in Figure 11. Since independent cracks were formed in all cases except when the loads were spaced three-quarters of a diameter apart, it was indicated by these tests that independent plastic hinges could be developed in a prestressed slab for loadings spaced one or more diameters apart, center-to-center.

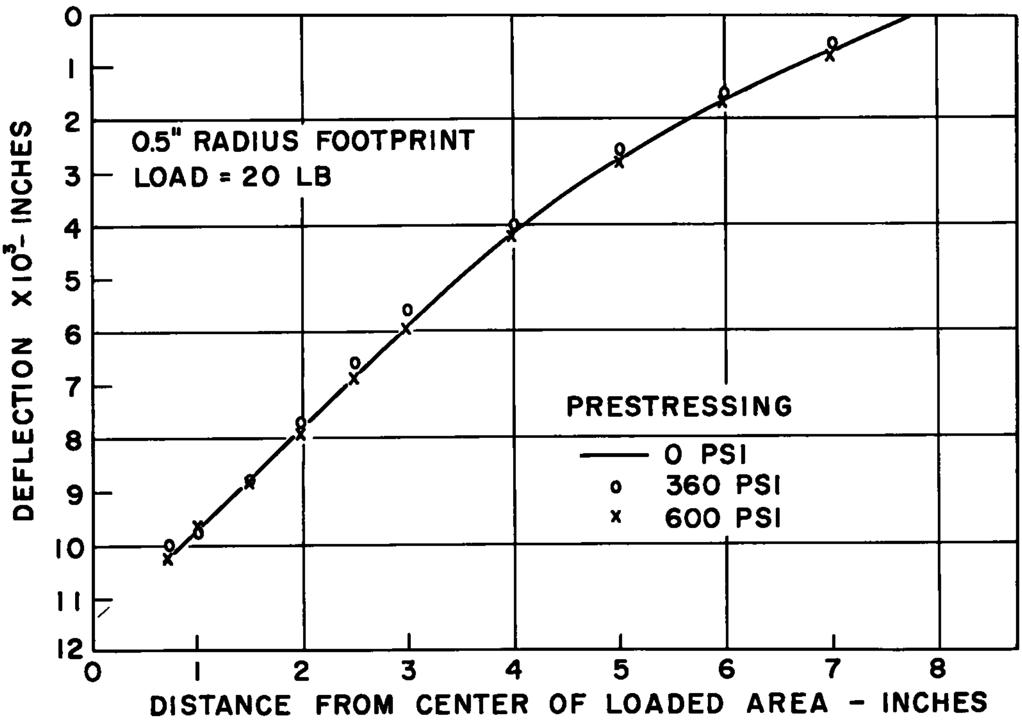

Figure 13. Effect of different prestressing on deflections within the elastic range.

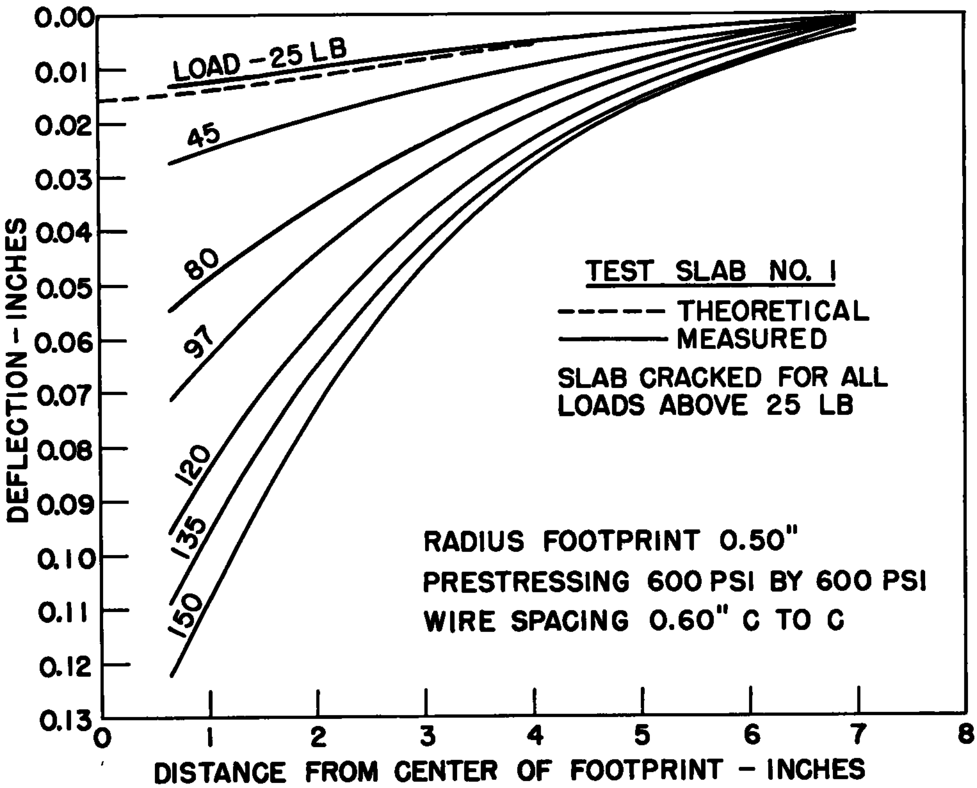

Figure 14. Deflection curves for interior loadings.

Deflections

Due to the construction of the footprints and the method of loading used, it was not possible to obtain deflection measurements within the contact area of the load. Therefore, all deflection data are for measurements at the footprint edge and beyond.

Deflection measurements were made to study the effects of such variables as: footprint size, prestressing, load, and degree of cracking. The results of tests to observe the effect of footprint size on slab deflections for a partially cracked slab are shown in Figure 12. Footprints ranging in radius from 0.10- to 1.50-in. were positioned in the center of a cracked area approximately 2½-in. in diameter in the interior portion of Slab No. 3. Each footprint was loaded to 40 lb and deflections were measured from the edge of the footprint to the edge of the slab. As can be seen from Figure 12, footprint size had little or no effect on deflections beyond the loaded area. Since these deflections are in substantial agreement with Westergaard's analyses, it was indicated that the presence of the cracking had little influence on the deflections for this loading.

In order to determine the effect of the magnitude of prestressing on slab deflections, tests were made on three slabs having different ratios of longitudinal to transverse prestressing as follows: 0 to 600 psi, 360 to 600 psi, and 600 to 600 psi, respectively. Identical loadings of 20 lb on an 0.5-in. radius footprint were made on each slab. The deflections shown in Figure 13 are those measured in the longitudinal direction and indicate that, within the elastic range, the variation in prestressing had no influence.

In conjunction with the study of crack pattern development for loading near the center of Slab No. 1, slab deflection measurements were made for comparison with the progression of cracking with increasing load. These deflections are shown in Figure 14. For the 25-lb load, the critical stresses in the slab did not exceed the elastic limit of the slab and the measured deflections were in good agreement with those computed on the basis of the Westergaard analyses. The deflections measured for loads greater

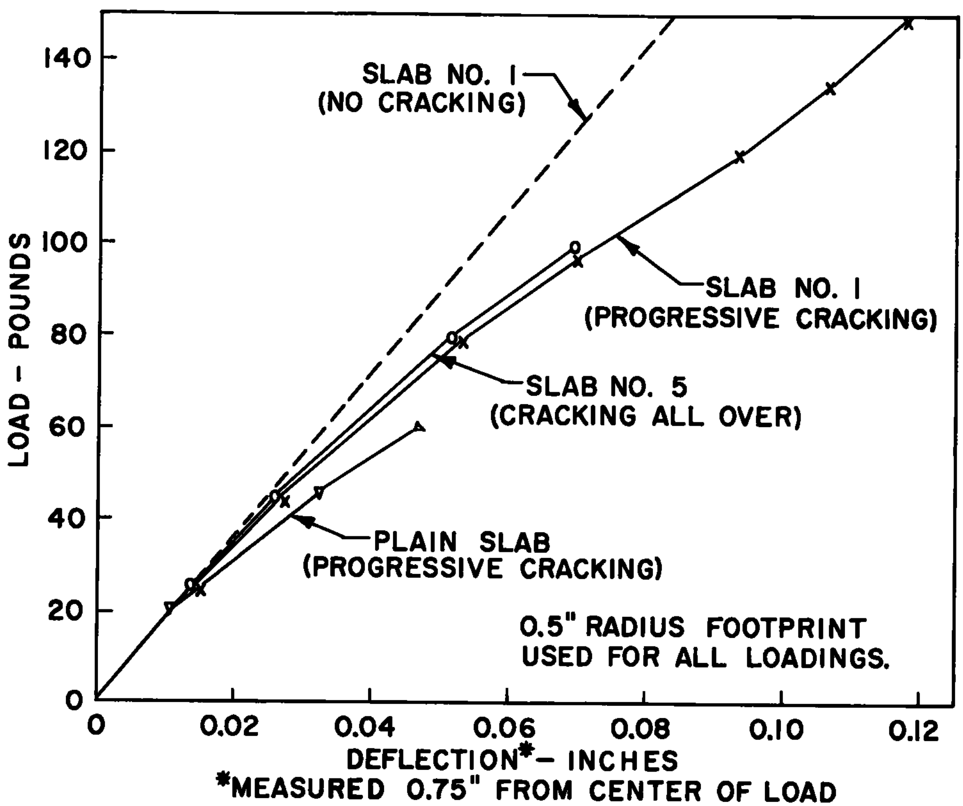

Figure 15. Deflections for interior loadings on plain and prestressed slabs.

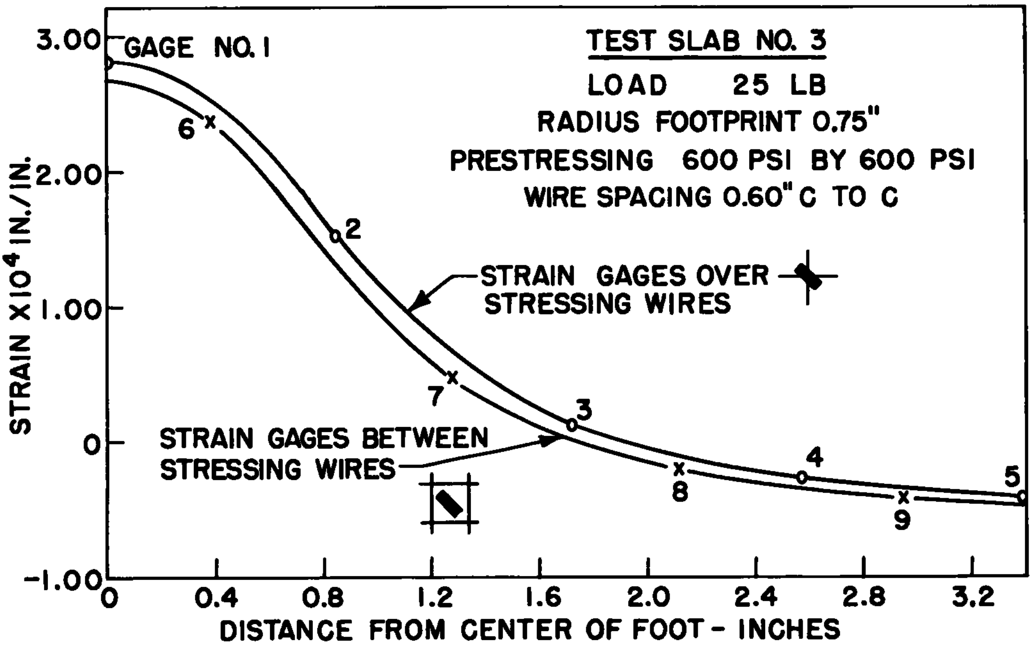

Figure 16. Effect of strain gage orientation on measured strains.

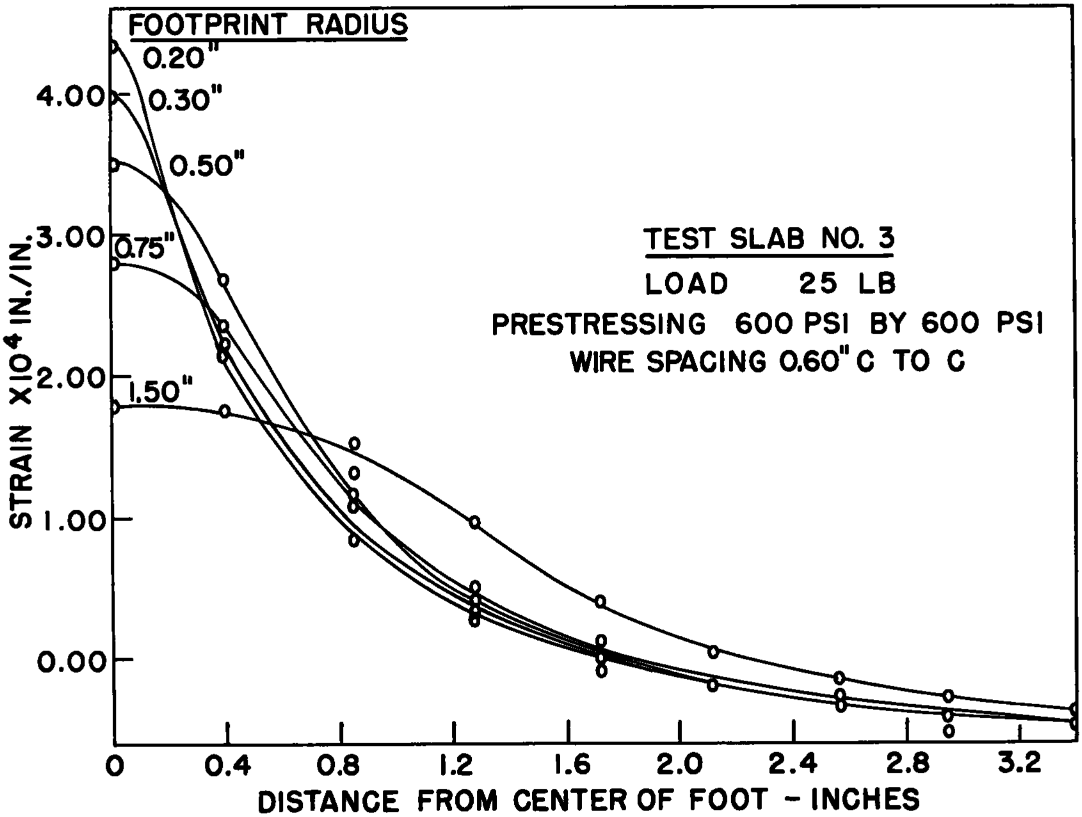

Figure 17. Effect of footprint size on strain distribution for interior loadings within the elastic range.

than 25 lb were for the progressively greater degree of cracking associated with each loading. Based on maximum deflections obtained from extrapolation of the curves plotted in Figure 14, it appears that the degree of cracking observed for a load of 150 lb (see Fig. 8) permits approximately 50 percent greater deflections than would be experienced if no cracks were present in the slab.

Figure 15 is a comparison of deflection versus load curves for two prestressed slabs and one plain slab, each loaded near the center with an 0.5-in. radius footprint. In this figure, the increase in deflection due to cracking is indicated by the difference between the curves of measured deflection and the dashed line representing an extrapolation of the deflection for the condition of no cracking in the slab. The deflections measured for Slab No. 1 and Slab No. 5 represent the effects of two different types of

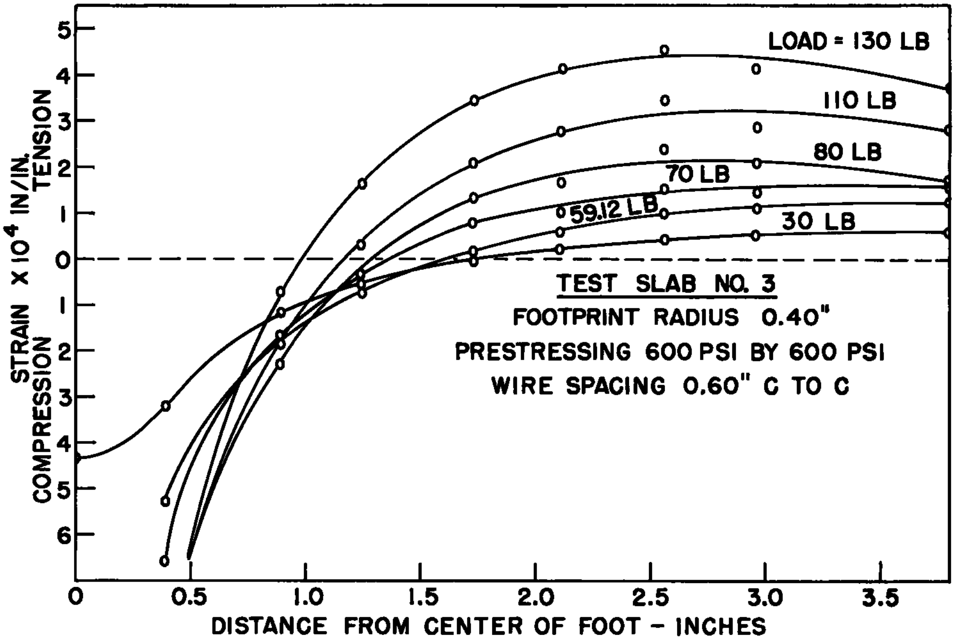

Figure 18. Distribution of radial strains in surface of model slab for interior loadings beyond elastic range.

cracking (see Figs. 8 and 9). Compared to the two prestressed slabs, the somewhat larger deflections measured in the plain slab indicated that the stressing wires had appreciable ability to reduce the deflection for loadings beyond the elastic range.

Strain Distribution

During the early stages of the model studies, an effort was made to determine what the positioning of the SR-4 gages relative to the stressing wires might have on the indicated strains. In these tests, various combinations of strain gage - stressing wire orientation were investigated. Typical of the results obtained are those shown in Figure 16 for loadings on Slab No. 3. By cementing a line of gages to the top side of the slab so that the gages were positioned alternately over the intersection of two stressing wires and between the stressing wires, it was possible to develop strain distribution curves for each gage orientation. In this case, the gages positioned directly over the wires indicated strains approximately 5 percent greater than did the gages located between the wires. Tests on other gages failed to indicate effects due to orientation greater than plus or minus 5 percent of the measured strains. Consequently, it was decided to omit any corrections for gage orientation and, wherever possible, to center the gages and footprints over wire intersections.

For loadings producing critical stresses within the elastic limit of the slab, the distribution of strain in the slab around the loaded area was found to be similar to that for a plain slab. Curves showing the distribution of radial strain for a 25-lb loading on various sizes of circular footprints are presented in Figure 17. These curves indicate that footprint size has little or no effect on strains beyond the contact area of the footprint for radii of 0.75-in. and smaller. The lack of conformity shown by the 1.50-in. radius foot suggests that the slab did not perform as though infinite in horizontal extent for a footprint of that size.

For loadings producing critical stresses behond the elastic limit of the slab, the distribution of strain was found to be a function of the magnitude of the loading. Figure 18 shows the distribution of radial strains in the surface of Slab No. 3 for various loadings on an 0.4-in. radius circular footprint. The curve for the 30-lb loading represents the distribution for elastic behavior of the slab. The curves for loadings greater than 30 lb represent conditions in which cracking existed in the bottom side of the slab, the

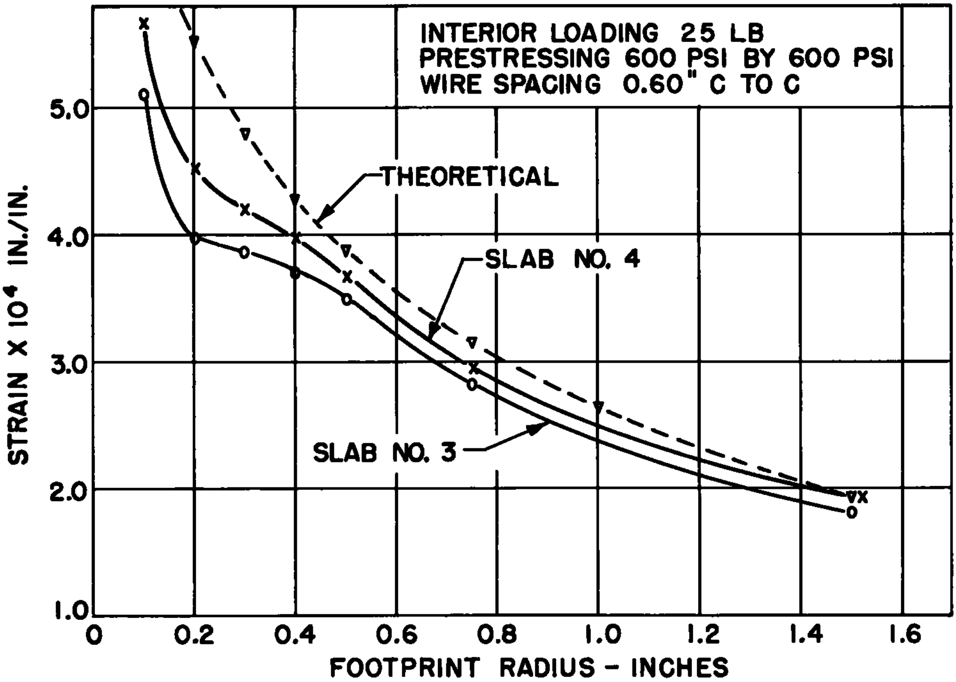

Figure 19. Effect of footprint size on maximum strain for interior loadings within the elastic range.

degree of cracking increasing with increasing load as shown previously. It can be seen from this figure that as the load was increased, both the point of maximum strain and the point of zero strain in the top of the slab moved toward the center of loading. The loading was discontinued at 130 lb when it became apparent that cracking in the top of the slab would not occur prior to complete failure of the slab at approximately 150 to 160 lb. On the basis of the strains measured in Slab No. 3, it was estimated that the maximum negative strain in the top of the slab for an ultimate load of 160 lb would be approximately 75 percent of the strain required to produce cracking.

Maximum Strains

In comparing maximum strains versus footprint radius for loadings within the elastic limit, certain sizes of footprints indicated a tendency to deviate from the relationship obtained with plain slabs. In Figure 19, curves of maximum strain versus footprint radius are shown for two different test slabs for comparison with the curve as given by the Westergaard analyses. In previous testing with plain slabs, it has been shown

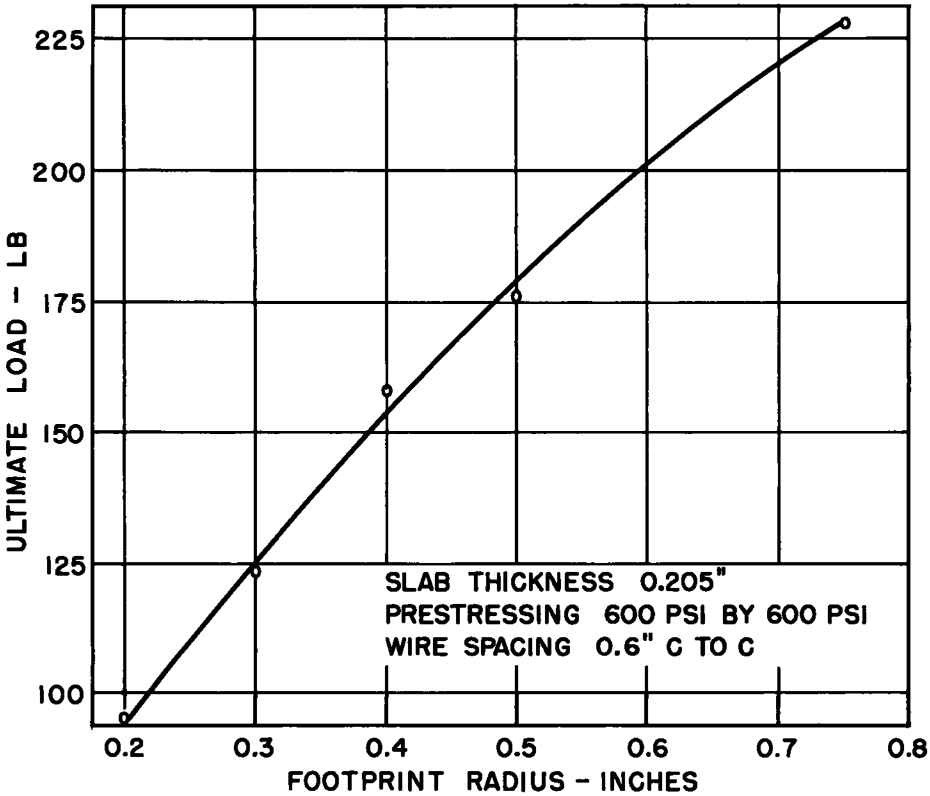

Figure 20. Effect of footprint size on ultimate load for interior loadings.

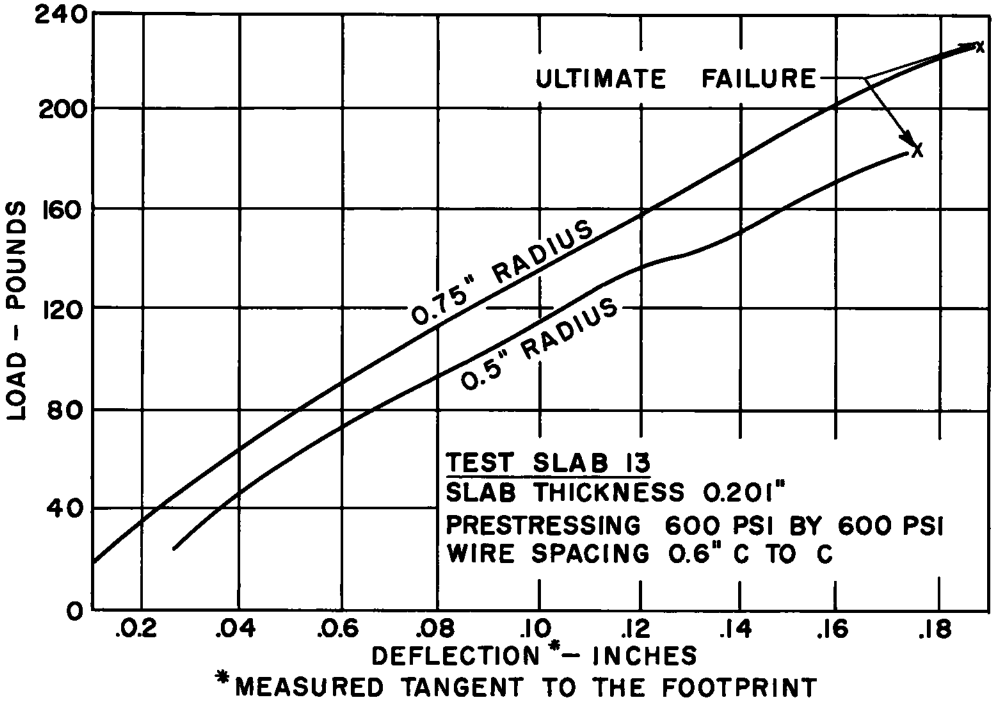

Figure 21. Deflection curves for interior loadings to ultimate failure.

that data obtained from the model closely paralleled the theory. In Figure 19, however, it is evident that the 0.2-, 0.3- and 0.4-in. radius footprints gave lower than normal values for the maximum strain. Although no definite reason for this deviation could be determined, it was believed to be related to the spacing of the stressing wires.

Ultimate Loading

In the testing completed to date, several of the model slabs have been loaded to complete failure. By using various sizes of footprints, the relationship between footprint size and ultimate load was developed, as shown in Figure 20, for model slabs having equal prestressing of 600 psi longitudinally and transversely. In the model tests, the failures were always characterized by the footprint punching through the slab and into the subgrade. Load-deflection curves for 0.50- and 0.75-in. radius footprints are shown in Figure 21 for ultimate load tests on Slab No. 13. Maximum deflections at incipient failure approximated 90 percent of the slab thickness.

A comparison of loads producing initial cracking and ultimate failure for both plain slabs and slabs prestressed to 600 psi longitudinally and transversely are presented in Table 3. Ultimate failure in these tests, defined as the load at which the footprint punched through the slab, does not necessarily represent failure in the prototype. However, these data afford some measure of the relative load-carrying capacities of prestressed pavements as compared to plain rigid pavements. In addition to the advantage of greater load-carrying capacity gained through the use of prestressing, it should be pointed out that for the design of airfield pavements prestressing will permit criteria based on the less severe interior loading as compared to the present criteria based on edge loading of plain rigid pavements.

SUMMARY OF FINDINGS

1. For pretensioning, the full amount of prestress was developed over a period of 36 to 48 hr rather than immediately following the release of the stressing wires.
2. For stressing wires placed at mid-depth in the slab and for loadings within the elastic range, prestressing had no effect on the structural rigidity of the slab. Measured strains and deflections were similar in both plain and prestressed slabs for comparable loadings.
3. Prestressing permitted the slabs to sustain greater loads prior to initial cracking for both edge and interior loadings.
4. Prestressed slabs maintained a substantial portion of their structural integrity following loadings beyond the range of elastic behavior.
5. Initial cracking occurred in a radial pattern originating in the bottom side of the slab and under the center of the loaded area.
6. Prior to complete failure, cracking was confined to the bottom side of the prestressed slabs.
7. Crack patterns were dependent upon both the manner of loading and the spacing of the stressing wires, and were not effected significantly by the magnitude of the prestressing.
8. Independent cracking occurred for loads spaced at intervals of not less than one footprint diameter.
9. Footprint size had little effect on deflections beyond the loaded area for both elastic and inelastic conditions of loading.
10. For loadings beyond the elastic range, greater deflections were observed in the plain slabs than in prestressed slabs subjected to the same loading.
11. Prestressed slabs were capable of sustaining greater deflections prior to ultimate failure than were the plain slabs.
12. For loadings beyond the elastic range, the point of maximum negative strain in the surface of a prestressed slab moved toward the center of the loaded area as the load was increased.
13. For loadings beyond the elastic range, negative strains in the surface of a prestressed slab increased at a greater rate than the loads producing the strains.

CONCLUSIONS

These studies have shown that small scale models of prestressed rigid pavements can be constructed successfully, and that these slabs, when subjected to controlled static loadings, exhibit characteristics believed to be analogous to those of the prototype. Therefore, it is concluded that this type of testing can be a useful and valid tool for investigating the behavior of prestressed airfield pavements for various conditions of loading and prestressing.

Since the construction and testing of the models is continuing, much of the data presented herein are incomplete. It is hoped that further study now programmed will contribute to the present knowledge of prestressed pavements.

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