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).

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.

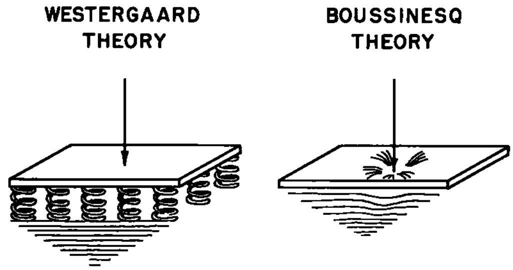

Figure 1.

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

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

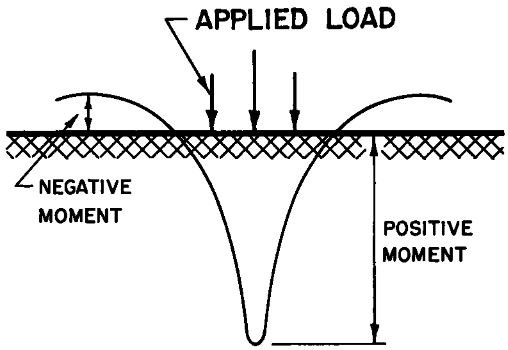

Figure 2. Distribution of moments in a slab caused by an applied load (from Stott).

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

$$W = \frac{2\rho^2}{E h} \left(d\, C_0 - b\, D_0\right)$$

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

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

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".

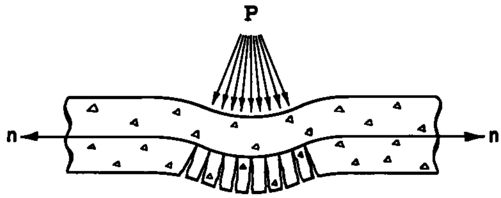

Figure 3. Formation of plastic hinges.

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:

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

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.

Subgrade Stresses

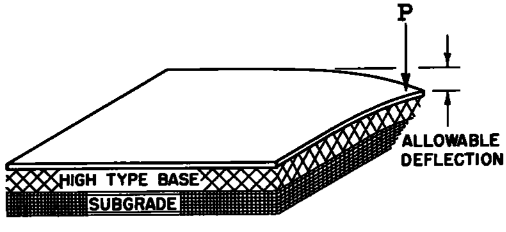

Figure 4.

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 3⅓ in., limestone macadam base course 3⅓ in., fine beach sand 1¼ 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.

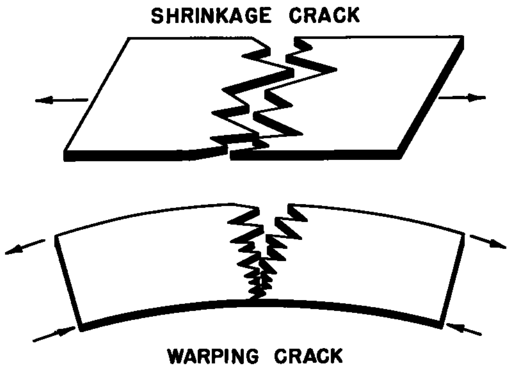

Figure 5.

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 29½-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.

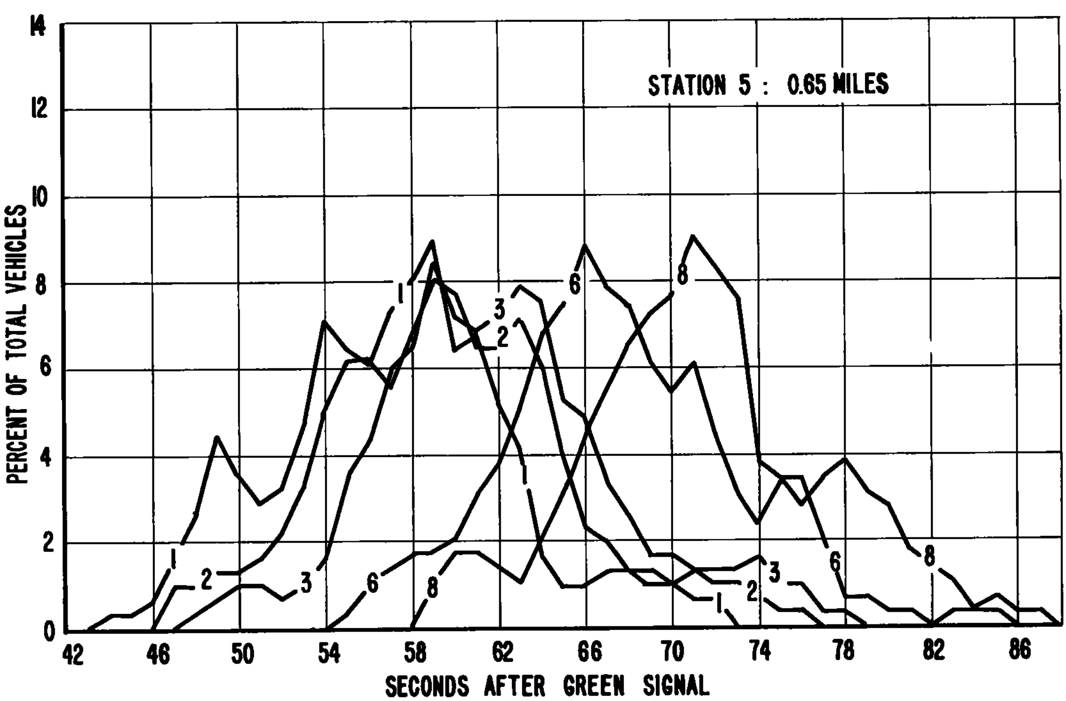

Figure 7. Frequency distributions of arrival times of nth vehicle in a platoon.

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

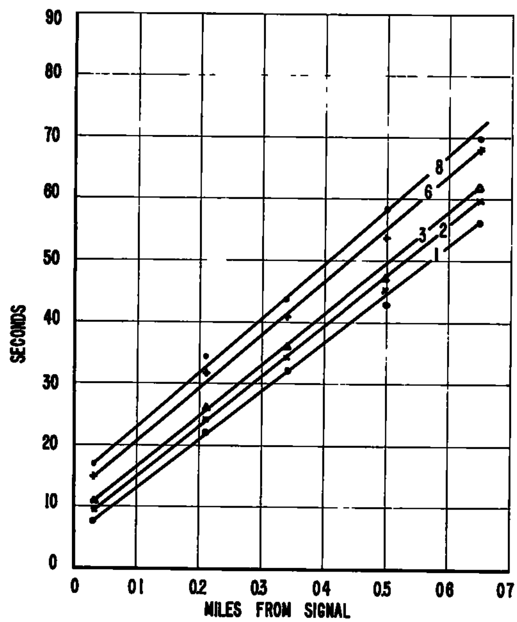

Figure 8. Mean arrival time of nth car versus distance.

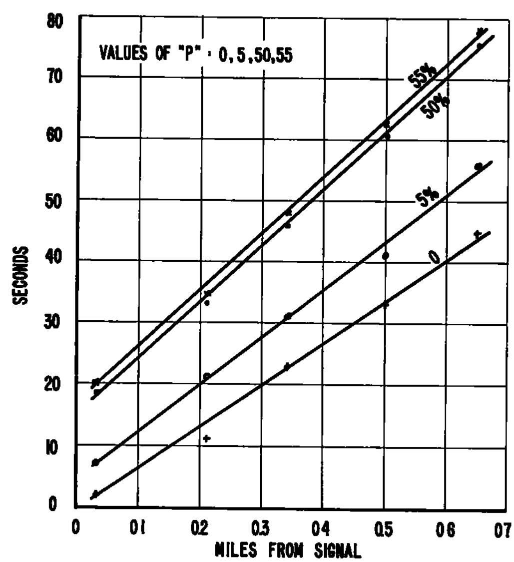

Figure 9. Time for the Pth percentile of vehicles in a platoon to pass a point at a given distance.

curves were plotted of ($y = \frac{10}{\sigma}$) versus distance (d) for each value of n.

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.

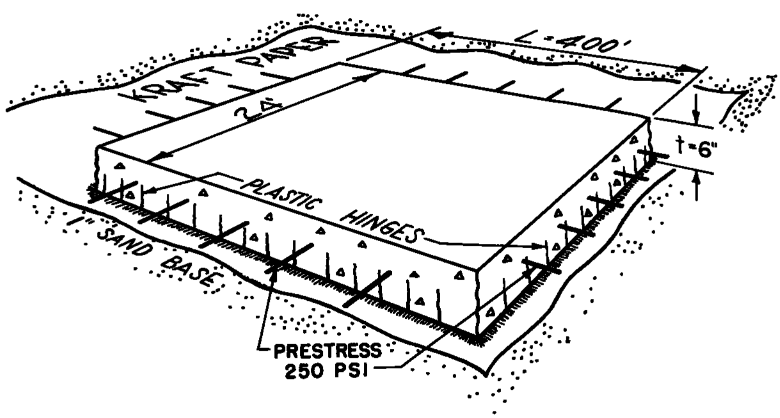

Figure 7. An average prestressed slab.

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."

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