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CONTRIBUTION TO THE STUDY OF PRESTRESSED CONCRETE ROADS

by

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(Translation made by C.V. Amerongen)

CEMENT AND CONCRETE ASSOCIATION
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CONVERSION FACTORS

Length	1 cm	=	0·394 in.
	1 m	=	3·281 ft
Weight	1 kg	=	2·205 lb
	1 metric ton	=	0·984 long ton
Stress	1 kg/cm^2	=	14·22 lb/in^2
	1 kg/mm^2	=	0·635 ton/in^2
Bending moment	1 kg m	=	7·233 lb ft

CONTRIBUTION TO THE STUDY OF PRESTRESSED CONCRETE ROADS

This is an abridged translation. Readers can see from the number of chapters and sections where most of the omissions have been made. All four Appendixes have been omitted. The full translation can be seen on request by visitors to the C & CA Library.

I. OBJECT OF THE PRESENT PAPER

Before the last war, concrete pavements had undergone considerable development in France, especially in the Nord region. Over the last decade, however, only a very few kilometres of concrete road have been built, though this type of pavement has continued to be favoured by engineers for the construction of airfields.

In recent years of course, concrete pavements have been greatly improved in many ways, and the adoption of concrete for the Paris South Motorway (Autoroute du Sud de Paris) perhaps marks the beginning of a new boom in conventional concrete pavements. Many engineers believe, however, that these improvements are not good enough and are seeking new ways of using concrete in modern pavements. Thus, in France the use of two-course pavements for heavily loaded runways is under consideration, and in Germany attempts are at present being made to achieve by new methods the best possible combination of the flexible and the rigid form of construction.

Another new form of construction would be prestressed concrete roads. As is well known, tests and construction work in this field have been carried out in France on airports (Orly and Maison-Blanche, Algiers) in the post-war years, and these have had wide repercussions. Experimental roads have also been built, both in France and abroad. A great many observations have been made on these structures. Thanks to these we are beginning to understand the functioning and behaviour of this new type of pavement, and in this respect French engineers appear to be somewhat ahead of their foreign colleagues.

My object in the present paper is to indicate the various problems presented by this new technique and how the resources that are available to the laboratory may be brought to bear on them. Furthermore, I thought it would be useful to give an account, however brief, of these problems for the benefit of all engineers; this would not only inform them of the new methods that highway engineering will shortly be able to employ, but would enable them, should the occasion arise, to take part in developing this technique. It will, in particular, be evident from this paper how much remains to be done in the way of devising and developing mechanical contrivances capable of maintaining the prestress and forming perfect joints or economical and stable abutments; in this domain, especially, it would be desirable and valuable to arrange a large number of competitions between engineers.

II. HISTORICAL SURVEY OF PRESTRESSED CONCRETE PAVEMENTS

II-1

The technique of prestressed concrete was conceived and developed in France by Freyssinet before the last war. It was not until after the war, however, that this technique was applied to pavements.

The first tests with prestressed concrete pavements, though only on a small scale, were carried out at Luzancy and Esbly by Dollet, at that time Engineer in the Bridges and Highways Department of Seine-et-Marne. But the first big experimental job was the 420 m long section of runway at Orly airport, which was designed by Freyssinet and built in 1947. We shall return to this structure in due course.

It roused considerable interest, not only in France but in other countries as well, and tests with this new method of construction were conducted in quite a number of places. Of these tests (at any rate, so far as they have come to my knowledge) the following call for mention:

a road 50 m long and 6 m wide at Esbly, built in 1947;

a road 300 m long and 7 m wide at Bourg, built in 1952;

a taxiway 430 m long and 25 m wide at Orly, built in 1953;

an experimental road 50 m long at Orly, built in 1957.

In Britain a good many experimental sections have been built:

a road 120 m long and 6 m wide at Crawley, in 1950;

a section of runway 100 m long and 36 m wide at London Airport, in 1951 (the 1947 Orly technique was used);

a road 1 km long at Woolwich, in 1952;

a road 450 m long at Port Talbot, in 1954.

The following experimental sections also call for mention:

In West Germany:

a section of road 240 m long and 7.50 m wide, built in 1953.

In Austria:

a section of runway 200 m long and 60 m wide, built in 1954;

a road 130 m long and 6.25 m wide, built in 1957.

In the United States:

a section of road 167 m long and 4 m wide.

In Italy:

a road 500 m long and 7.50 m wide at Cesena.

In Switzerland:

a section of road 500 m long and 2.50 m wide.

a section of road 334 m long and 2.50 m wide.

Finally, other tests have been carried out in Australia and in Holland, and further tests are in progress in several countries.

The above jobs were no more than experimental sections. In 1955, however, a complete scheme was executed in prestressed concrete, namely, a runway 2,400 m long and 60 m wide, together with its parallel taxiway, at Maison-Blanche, Algiers.

It appears that at the present time upwards of 250,000 m^2 of prestressed concrete pavement has been constructed in various parts of the world, nearly 90% of which is French.

<u>II-2</u>

All these pavements were the subject of numerous tests and measurements and yielded an abundant crop of informative literature. Unfortunately, however, there still exists a large number of problems that are still unsolved or not entirely solved.

These structures moreover vary considerably as regards thickness, quality of concrete, nature of the soil, climate, magnitude of the prestress, and, especially, the method of prestressing.

As for the last-mentioned point, there are structures in which the longitudinal and the transverse prestress are produced by cables running in the longitudinal and transverse directions; in others, the cables are placed obliquely. At Orly, in 1947, only transverse cables were installed, which produce the transverse prestress but which also indirectly produce the longitudinal prestress, thanks to the arrangement of roller joints at 45° to the axis of the runway. The longitudinal thrust obtained in this way is resisted by an abutment constructed at each end of the runway.

In other pavement structures (at Algiers, in particular) the transverse prestress is produced by means of cables, and the longitudinal prestress is produced by jacks in conjunction with end abutments for taking up the longitudinal thrust. In Switzerland, concrete wedges, installed between two lengths of pavement, have been used in place of jacks. In other cases the transverse prestress has been omitted altogether.

In Italy, the pavement was provided longitudinally with pre-tensioned wires which were embedded in the concrete, whereas in the transverse direction the wires were enclosed in sheaths and were post-tensioned, i.e. after the concrete had hardened.

<u>II-3</u>

Most of these pavement structures have displayed good or, indeed, very good behaviour, both during tests and under working load. At Orly, in particular, the tests carried out by Becker showed that prestressed concrete pavements were able, without fracturing, to carry loads far in excess of those for which they had been designed. Thus, the runway built in 1947, with a prestress of 80 kg/cm^2, could withstand concentrated loads of 140 tons without fracturing, although the concrete was of mediocre quality (200 kg/cm^2 compressive strength at 28 days) and the soil was definitely bad (modulus of sub-grade reaction 3 kg/cm^3). Cracks did not appear until the prestress was reduced to 10 kg/cm^2 and the concentrated load had been increased to 130 tons. Besides, these cracks closed up again as soon as the load was removed. Generally speaking, prestressed concrete possesses a remarkable degree of strength and versatility in adapting itself to poor soils. We shall refer to this point again in due course.

Nevertheless, mishaps and setbacks have occurred which need to be pointed out and investigated. I shall confine myself to French tests, which are the only ones concerning which I have sufficiently precise information at my disposal.

The first mishap occurred, in 1955, on the experimental taxiway at Orly (430 m long by 25 m wide, prestressed longitudinally by means of jacks) which, during a period of frost and dry weather (which caused drying shrinkage to be superimposed upon thermal shrinkage), cracked transversely. This was indicative of an inadequate minimum prestress. But when, with the coming of milder weather, the prestressing jacks were "pumped up" again, the crack closed up and this pavement has since behaved in a perfectly satisfactory way.

A more serious mishap occurred at Orly, at the beginning of 1958, on the first prestressed concrete runway, built in 1947. Indeed, it was so serious that this length of runway had to be overlaid with a surfacing of ordinary concrete. The haste displayed in covering up the old pavement was mainly due to the need for reducing to a minimum the time during which this much used and very important runway was out of action.

As is well known, this experimental length of runway was built by assembling on the site a number of small square precast slabs each having a side of 1 m and a thickness of 16 cm. These were joined together with mortar to form large monolithic isosceles triangles. These triangles each had a height of 60 m and a base 120 m in length (see Figures A and 1). The small component slabs along the two inclined sides of the triangle, which sloped at 45^{o} to the axis of the runway, each had a side sloped likewise at 45^{o}. These joints at 45^{o} between the triangles were made as mobile as possible by providing them with little vertical steel rollers which rolled on vertical steel plates affixed to the concrete faces of the joints. The joints were filled up with bitumen in order to protect the steel.

Figure A: Orly Airport. Prestressed concrete experimental length of runway, 1947.

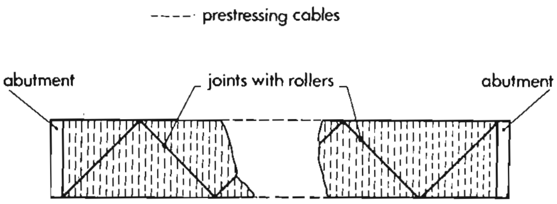


Figure 1: Diagram of prestressed concrete runway at Orly, 1947.

The prestress was produced by means of transverse steel wires tensioned between the two sides of the runway (and therefore crossing the joints). Apart from loss due to friction, the joints transformed this transverse prestress into a longitudinal prestress. The wires were placed in grooves that had been formed between the small component slabs and which, after the prestress had been applied, were plugged with mortar.

We can now, of course - thanks to what we have since learned - criticize that method on a number of points. In particular, it may be supposed that this type of joint is bound to function badly in winter when the bitumen has become hard as a result of the cold; and the arrangement adopted for the cables did not permit of subsequent adjustment and did not provide adequate mortar cover. It must be borne in mind, however, that this was a first attempt and that, by behaving perfectly for eleven years, it performed its role as an experimental section quite well.

The severe cold of February 1956 does not appear to have had any particular effect on the runway. Nevertheless, in the last months of 1957 some cracks appeared in the mortar of the grooves between the small slabs. This occurred in the central zones of the triangles. These cracks suddenly became more pronounced at the beginning of 1958, and the mortar broke up into chunks the size of a man's fist. The Central Laboratory of the Bridges and Highways Department (Laboratoire Central des Ponts et Chaussees)was called in to perform an "autopsy" on the runway and carried out a number of tests on specimens of the wire which were removed from the grooves between the component slabs. Each groove contained 30 wires, arranged in three groups of 10. Some of the wires were found to have fractured in consequence of corrosion. Only a small number of wires was thus affected, however, - something like 3%. Broadly speaking, the wires appeared to be intact, except in the zones where the mortar had cracked considerably. At these points, water had started to seep in (not long previously, it seemed) and caused incipient rusting of the wires.

On the other hand, all the wires were found to be in a more or less unstressed condition. It was possible to measure the residual stress in them by making them vibrate transversely and measuring the frequency. About sixty wires were examined in this way, and the residual stresses were found to be between 0 and 60 kg/mm^2, with an average value of 20 kg/mm^2. Actually, the stress should have been something like 100 kg/mm^2.

It is not surprising that a loss of prestress of this magnitude - which was even greater in the longitudinal direction on account of the poor efficiency of the roller joints in winter - resulted in insufficient and indeed almost zero prestress in the central zones of the triangles - where the subgrade restraint caused the greatest reduction in the prestress. The abutments do not appear to have moved appreciably; but considerable displacements would have been necessary to account for such a large loss of prestress.

It therefore appears to have been quite clearly demonstrated that the destruction of the pavement was caused by the loss of stress in the wires.

As regards the underlying cause of such loss, however, we can only make conjectural assumptions. We may lay the blame on slip of the wires in the anchorage cones, since the latter were not so good in 1947 as they are now; besides, they were not grouted with cement, but with bitumen, which may have had a lubricating effect. We may also blame excessive relaxation of the prestressing wires; this is quite likely, having regard to the quality of steel wires available in the early post-war years.

However that may be, and despite its final failure, the Orly experimental pavement must be regarded as having constituted a successful test, thanks in particular to the wealth of information it has provided on this new technique. It would perhaps be wise, however, to keep a close watch on the similar experimental pavement constructed at London Airport.

PART I

PROBLEMS PRESENTED BY PRESTRESSED CONCRETE

IV. STRENGTH OF SLABS

IV-1

The principle of prestressed concrete consists in exerting a permanent compression on concrete structural members which, under working conditions, are liable to be subjected to bending or tension. Ordinary concrete is not good at resisting tensile or flexural stresses. Prestressed concrete, on the other hand, will not fracture until the tensile stress exceeds the permanent precompression applied to the member plus, perhaps, the tensile strength of the concrete itself.

That is the principle. But actual reality has turned out to be even more favourable where prestressed concrete pavements are concerned. And failure is not often observed to occur, even though the tensile stress on the underside of the slabs may considerably exceed the tensile strength of the concrete plus the prestress.

To explain this phenomenon, Becker expressed the view that, from an early age, the strength of the concrete was increased in consequence of the prestress (which is frequently applied to the concrete when it is only 24 hours old) and, furthermore, that the prestress prevented the formation of the tensile micro-cracks that occur on the surface of ordinary concrete and initiate tensile failure.

This explanation is undoubtedly valid to quite a considerable extent, for laboratory tests have shown that concrete specimens which have hardened under load have a distinctly higher flexural strength (20 - 40%) than have similar specimens which have hardened without being subjected to load. Besides, these latter specimens were protected from drying out, otherwise the superficial micro-cracking, which occurs extensively when non-prestressed concretes are exposed to the air, would have caused the difference in strength to be even greater.

Moreover, this phenomenon could be further accentuated by increasing the cement content of the concrete. As is known, the cement content for concretes used in highway construction is limited by the considerable increase in micro-cracking which occurs at the surface of the concrete as soon as a cement content of 350 kg/m^3 is exceeded; it follows that, for air-cured concretes, the flexural strength reaches a maximum for a cement content around this value.

But if the concrete is prestressed, then, owing to the elimination of this superficial micro-cracking, the flexural strength no longer displays a maximum. It continues to increase with the cement content of the concrete, and it is therefore advantageous to increase the latter to 400 and perhaps even to 450 kg/m^3. With such concretes, having a high cement content and prestressed at an early age, one may hope to obtain flexural strengths of 100 kg/cm^2 and upwards.

IV-2

We can calculate what happens in an uncracked prestressed concrete slab, laid on an elastic soil (in the sense conceived by Boussinesq), by using Hogg's formula for loads at the centre or the results of Dantu's experiments for loads at an edge. Hogg compares the concrete slab to an elastic plate, as conceived by Navier, with infinite dimensions, and he assumes it to rest without friction on a semi-infinite elastic solid medium bounded at the top by a plane.

The principles of Hogg's analysis are given in Appendix I of the original article. The results are presented somewhat differently from the form in which Hogg has given them, because, in order to ensure the homogeneity of the formulae giving the maximum flexural stress, they have been referred to a relative thickness h which is linked to the actual thickness e of the slab, to the radius R of the impact area of the load (which is assumed to be circular, with a constant pressure p), and to the elastic characteristics E and η of the concrete and E$'$ and η' of the soil by the following relation:

$$h = \frac{e}{R}\sqrt[3]{\frac{1}{6}\,\frac{E}{1-\eta^2}\,\frac{1-\eta'^2}{E'}} \quad \ldots\ldots\ldots\ldots (1)$$

The maximum stress is then given by the formula:

$$N = 3p\,(1+\eta)\left(\frac{1}{6}\,\frac{E}{1-\eta^2}\,\frac{1-\eta'^2}{E'}\right)^{\frac{2}{3}}\Phi(h) \quad \ldots\ldots\ldots\ldots (2)$$

The function Φ (h) has been calculated, on the one hand, by means of Hogg's formula, which is valid for large values of the relative thickness h; on the other hand, for the other values of h, it has been calculated either with the aid of an electronic computer (an I.B.M.704 computer was employed) or, for low values of h, by direct observation of the deformation of the soil.

The results are given in a Table on p.63 of the original article and are plotted in the curve of Figure 2. They cover all possible values of h. Hence we need no longer worry about the range of validity as in Hogg's original formulae.

This was essential when dealing with prestressed concrete because, with the small thicknesses involved, the conditions were sometimes outside Hogg's range of validity.

Consider, for example, a 15-ton axle (i.e. P = 7.5 tons) with an impact radius R = 18 cm (tyre inflation pressure 7.5 kg/cm^2):

slab thickness e = 12 cm

E' (soil) = 600 kg/cm^2

E (concrete) = 450,000 kg/cm^2

η' = 0.25

Whence we obtain

N = 54 kg/cm^2

Hence we see that in the middle of the slab (i.e. away from the edges) a 7.5-ton wheel load could be easily supported and that with a small prestress it would even be possible to go to twice that load.

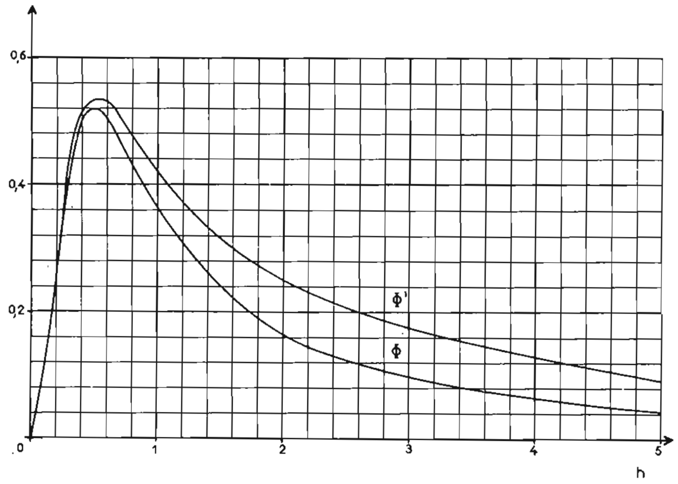


Figure 2: Diagram for the functions Φ(h) and Φ'(h).

IV-3

Actually, this analysis for the middle of the slab is too favourable. The most unfavourable case is where the load is placed at the edge of the slab. In the analysis of conventional slabs the three following loading conditions are considered:

load placed at the centre;

load placed at the edge;

load placed at a corner.

The stresses produced in the slab are higher under edge loading than under central loading, and they are still higher under corner loading.

When considering prestressed concrete pavements there is no need to concern oneself with corner loading, since corners occur only if there are joints. In a prestressed pavement the joints have been virtually eliminated, and to deal with the few joints that do occur the slabs can be strengthened locally (this will be discussed more fully later on).

The most unfavourable loading condition for prestressed concrete is that of edge loading.

It has not yet been possible to establish an accurate analysis for this case. It can be demonstrated, however, that the maximum tensile stress at the point O of the edge (Figure 3) acts on the underside of the slab, tends to produce cracks along OA, and is expressed by a formula having the following form:

$$N' = 3\,p\,(1+\eta)\left(\frac{1}{6}\,\frac{E}{1-\eta^2}\,\frac{1-\eta'^2}{E'}\right)^{\frac{2}{3}}\quad \Phi'(h)\ \ldots\ldots\ldots\ldots(3)$$

The values of Φ' are given in terms of h in the same Table and in the same diagram as the values of Φ. These values for Φ' are only approximate, however. They have been obtained, on the one hand, for the zone corresponding to low values of h by considering that here it is virtually the deformation of the soil alone that matters (hence $N' = N$), and on the other hand, for large values of h by taking account of the results obtained experimentally by Dantu which yield the expression

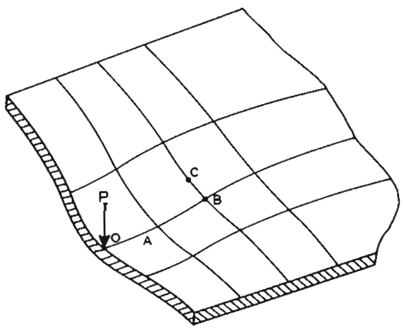
$$\frac{N'}{N} = \frac{\Phi'}{\Phi} = \frac{3.60\,h^2 + 12\,h}{1.70\,h^2 + 4.1\,h + 10.3}\ \ldots\ldots\ldots\ldots(4)$$

As soon as h exceeds 4, this ratio is practically equal to 2 (Figure 4).

The above formula is not, however, valid below h = 2. It has accordingly been necessary to find empirical intermediate values to link up with the values calculated for low values of h.

It will be noted that in the numerical example given in section IV-2 (above), we should arrive at

$N = 98\ kg/cm^2$



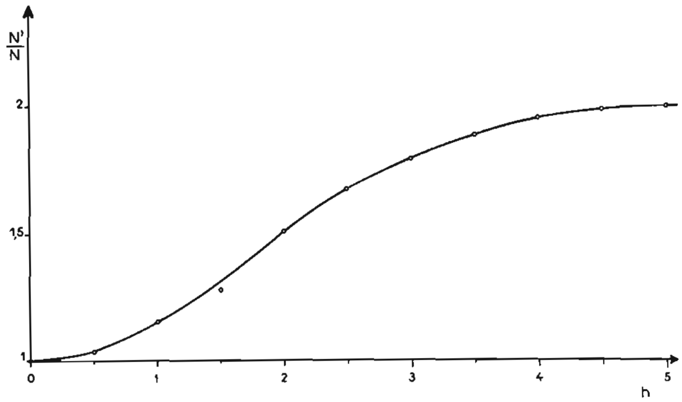


Figure 3 (left): Diagram illustrating the deformation of a slab loaded at an edge.

Figure 4 (right): Variations of the ratio N'/N as a function of h.

It will also be noted that the load at the edge will produce at B, on the upper surface of the slab, tensile stresses that tend to crack the slab along BC, i.e. in the longitudinal direction. It is not possible to calculate the values of this stress N" accurately. It can be ascertained, however, that it is less than a quarter of N'. It would not, therefore, have to be taken into consideration except where the slab is only prestressed longitudinally; this case will be examined later on.

IV-4

But though this may be the ultimate strength of the concrete in bending, it is by no means the ultimate strength of the pavement. It is undoubtedly one of the main advantages of prestressed concrete that it continues to function structurally, and indeed to do so quite effectively, when it is cracked.

It has actually been found (as witness Becker's tests at Orly) that prestressed concrete surfacings can go on resisting well beyond the flexural strength. No cracks are then observed on the upper surface of the concrete, but the surfacing is found to display a greatly increased degree of flexibility, with considerable deflexions which may sometimes be several centimetres. It is not until much later that the cracks appear at the surface and lead to final complete failure of the slab.

It would thus seem possible to distinguish three stages in the static loading tests on prestressed concrete pavements.

The first stage corresponds to loads which are so low that the stresses as calculated in accordance with sections IV-2 and IV-3 do not reach the flexural strength.

In the second stage, for larger loads, flexural failure (cracking) appears to have occurred on the underside of the slab; the prestress prevents the cracks spreading to the upper surface, however.

In the third stage, for still larger loads, cracking reaches the upper surface of the slab, causing its complete failure.

This may be summed up as follows: with conventional concrete slabs, flexural failure of the concrete constitutes the ultimate strength of the slab; with prestressed concrete, on the other hand, it merely constitutes a kind of elastic limit beyond which the pavement is still capable, without fracturing, of carrying much larger loads and, especially, of undergoing much larger deformations.

IV-5

Let us try to understand what happens in prestressed concrete beyond this cracking limit. It must be assumed that cracks are formed on the underside of the slab. Although these cracks are only partial, in the sense that they do not affect the full thickness of the concrete, they produce a considerable increase in the flexibility of the pavement. This in turn causes severer stressing of the sub-grade, but considerably reduces the stresses in the concrete. The concrete is thus enabled to resist these stresses, even in the cracked parts; this prevents the cracks from spreading.

To reduce the problem to simple terms, let us suppose that extensive cracking has occurred on the underside of the slab. Broadly speaking, the situation will now be as if the thickness of the slab, which originally was e, had become x, the depth of the cracks being e-x. We know that the maximum stress on the underside of a slab whose thickness x is varied displays the behaviour represented in Figure 2. For x = 0 the stress is zero, for the slab is infinitely flexible and can easily follow all the deformations of the sub-grade. For increasing values of x the stress increases, passes through a maximum, and then diminishes asymptotically to zero.

In the normal condition pavements are usually so constituted that they are, functionally speaking, on the right-hand side of the maximum. But if the cracks are large enough, the condition may move to the left of the maximum, and the stress in the slab will then undergo a substantial reduction. Now the precompression of the concrete, due to the prestress, will increase when x decreases, because the prestressing force, which is constant, is distributed over a smaller depth of slab. The mean value of the prestress is

$$f' = f \frac{e}{x}$$

where f is the initial prestress.

These two phenomena (reduction of the flexural stress and increase of the prestress) both co-operate in helping the slab to resist the loads and stopping the cracks from spreading. It will thus be understood how a prestressed concrete pavement slab, even when it is cracked on the underside, can continue to withstand loading well beyond the flexural strength of the concrete. This has been amply confirmed by experimental means, particularly by Becker's tests at Orly.

What really happens is somewhat more complex: for one thing, the cracks are localized in certain zones; furthermore, the cracks spread by a sort of tearing action, and the behaviour of the slab as a whole is no longer elastic. Becker has been able to analyse what happens at this stage by basing himself on the observed fact that, when the slab fails, a circular crack is formed which corresponds, on the upper surface, to the boundary of the loaded area. He thus distinguishes two different parts of the pavement:

- the part situated outside the above-mentioned circular area, which continues to exhibit elastic behaviour;
- the part within the circular area, which cracks and exhibits plastic behaviour.

There is found to be remarkably good agreement between the behaviour of the slab calculated by this method and the actual behaviour under static loads.

IV-6

Another and simpler method for evaluating what happens in this second stage consists in adopting the simplified representation given in section IV-5 (above). In this representation, the thickness of the cracked concrete is assumed to be constant over the entire area of the slab and its effect is ignored. For a given load P we can then derive formulae giving N or N' and the depth of the cracks (e-x), (at the cracks we must replace e by x) assuming that at the bottom of the cracks the flexural stress must be exactly equal and opposite to the mean prestress q'.

When x is known, it is possible to determine the compressive stress in the concrete and, furthermore, the bearing pressure on the sub-grade. The former is obtained by adding together the absolute values of N (or N') and q'; the latter is calculated with the aid of formulae which will be dealt with in chapter V.

It may be that this compressive stress and this bearing pressure are below the elastic limits of the concrete and of the soil respectively. The slab, though cracked on its underside, will then still exhibit elastic behaviour, but its capacity for deformation will be appreciably greater than in the first stage.

If the load is further increased, the elastic limit of the soil will be exceeded, especially if repetitions of load occur. The behaviour of the pavement will then no longer be elastic: depressions and irregularities may appear, and other more serious ones liable to cause "pumping" may follow. In this second stage, therefore, it is the soil that plays the leading part. The pavement will have to be designed to withstand these effects, and it may be necessary to provide a suitable base for the purpose.

If the load increases still more, the soil, no longer elastic, will behave as though it had a diminished modulus of elasticity E'. There will then be a rapid increase in the compressive stress in the concrete, which may cause it to crush. The complete failure of the pavement will then have been achieved.

IV-7

This simplified representation of what happens in a prestressed concrete pavement - and which, indeed, enables us to evaluate the stresses and deformations that occur in the second stage of a loading test - gives prominence to the significance of two "thresholds".

The first of these corresponds to the end of the first stage and may be called the "cracking threshold";

The second, which corresponds to the elastic limit of the soil, represents the limiting value of the load that the pavement can support under conditions of elastic behaviour. On a road, on account of the live load repetitions involved, this value must be quite close to the "failure threshold".

In actual practice it is undoubtedly necessary to design prestressed concrete roads so that the maximum design loads will not exceed the cracking threshold and will, indeed, have a small factor of safety in relation to that threshold. But it is also necessary to ensure that the failure threshold is sufficiently far away from the cracking threshold. The engineer can, if conditions require it, achieve this by providing a suitable foundation under the concrete slab.

There still remain a great many questions to be answered. In particular: Will the effect of traffic loads be similar to that of a loading test? Will it not be necessary to apply a dynamic impact factor to the live loads?

Such questions can be solved only by full-size tests on certain sections of road. Indeed, the main object of the present analysis is to provide a set of guiding rules for designing and interpreting these tests in the best possible way.

IV-8

To simplify the problem we have, so far, considered a prestressed concrete pavement as consisting merely of a slab resting on a homogeneous subgrade. Actually, we should have used the representation adopted by Jeuffroy and Bachelez comprising:

an elastic plate resting without friction on its foundation;

a base (foundation course under the slab);

a homogeneous sub-grade; the underside of the base is assumed to be "stuck" to this homogeneous sub-grade.

The foregoing calculations would therefore have to be done again, with the help of graphs prepared by Jeuffroy and Bachelez. But the results would be little different from those obtained by Hogg's method, at least for the cracking threshold.

IV-9

Let us examine the particular case where a depression exists in the foundation, preventing the concrete from properly resting on it over a certain area. To enable us to do a calculation and thus to ascertain the order of magnitude of the stresses, let us assume this area of the depression to be circular (radius = L) and the slab to be simply supported at the edges thereof.

When the load P, which is assumed to be distributed over a circle with radius R, is placed at the centre of the depression, the flexural stress in the slab will have a maximum value

$$N = \frac{3P}{8\pi e^2}\left[4\,(1+\eta)\log\frac{L}{R} + 4 - (1-\eta)\,\frac{R^2}{L^2}\right]$$

For $\eta = 0.25$ the expression in square brackets has the following values:

3.35	for	L = R
7.277	for	L = 2R
9.410	for	L = 3R
12.017	for	L = 5R
15.505	for	L = 10R

We see, therefore, that even a thin slab can, under these conditions, perfectly well withstand the passage of heavy loads. For example: for P = 7,500 kg, e = 12 cm, R = 20 cm, and L = 1 metre, we obtain

$N = 75\ \text{kg/cm}^2$.

V. PRESSURES ON THE SUB-GRADE. PROBLEM OF THE BASE

V-1

The deflexions and the pressures on the sub-grade may be computed either according to Hogg's theory (a plate resting on a homogeneous sub-grade) or Jeuffroy-Bachelez's theory (a plate resting on an elastic base adhering to the homogeneous sub-grade).

In Appendix I* to the present treatise the results of the calculations based on Hogg's method are indicated. The Jeuffroy-Bachelez graphs furthermore permit the determination of the deflexions and pressures on the basis of the second of the above-mentioned theories.

It will be noted that if the concrete slab is thick and rigid, which is the case in the first stage of the behaviour of the pavement as considered in section IV-5 (above) - i.e. when the concrete is not cracked - there is very little difference between the two methods. In that case, therefore, we may use Hogg's method, which is the simpler of the two. In the second stage of

* See note on first page of text, beneath title.

behaviour it is essential to use the Jeuffroy-Bachelez method, as the cracked concrete has a much increased capacity for deformation.

V-2

With Hogg's hypothesis (with the concrete uncracked) the pressures on and in the sub-grade can be investigated by assuming the load to be a concentrated load, since there is, for practical purposes, no difference between the case of the concentrated load and that of the distributed load, even with the very low-pressure tyres used nowadays.

The calculations then become simpler (see Appendix I), and we find that the bearing pressure under the slab has its maximum value under the load, viz.:

$$q_o = \frac{\pi}{3\sqrt{3}} \frac{p}{h^2} \quad \ldots\ldots\ldots\ldots \quad (5)$$

where p is the tyre pressure and h is the relative thickness as defined in section IV-2.

It can also be written in the following form, which shows that the tyre pressure does not enter into it:

$$q_o = \frac{P}{3\sqrt{3}\ e^2 \left[\frac{1}{6} \frac{E}{1-\eta^2} \frac{1-\eta'^2}{E'}\right]^{\frac{2}{3}}} \quad \ldots\ldots\ldots\ldots (6)$$

For example: if $P = 7{,}500$ kg, $e = 12$ cm, $\eta = \eta'$, $E = 450{,}000$ kg/cm^2, and $E' = 600$ kg/cm^2.

we obtain $q_o = 0.400$ kg/cm^2.

We thus see that these thin slabs have a considerable load-spreading capacity.

Farther down in the sub-grade, at a depth z below the slab, the pressure exerted on a plane horizontal element of soil diminishes rapidly. We then have the relation

$$q = q_o\, f(Z) \quad \ldots\ldots\ldots\ldots \ (7)$$

where Z denotes the relative depth, as expressed by

$$Z = \frac{z}{e \sqrt[3]{\frac{1}{6} \frac{E}{1-\eta^2} \frac{1-\eta'^2}{E'}}}$$

The numerical values of f(Z) are given in the following Table:

Z	0	0.10	0.50	1	2	5	10
f(Z)	1	0.920	0.680	0.488	0.278	0.093	0.024

It is worth noting that up to the value $Z = 5$, we may with fair approximation write:

$$f(Z) = \frac{1}{1 + \frac{5}{6} Z + \frac{2}{9} Z^2} \quad \ldots\ldots\ldots \ldots \ (9)$$

This is quite suitable for all practical calculations.

V-3

We can thus calculate the thickness that we must give the base, by comparing these pressures with those that the soil actually supports in the C.B.R. theory.

We know that this C.B.R. theory assumes, on the one hand, that the pressure distribution in the sub-grade of a flexible pavement conforms to Boussinesq's theory regarding the transmission of stress in an elastic and homogeneous medium, and, on the other hand, that the thickness z that we must give the pavement (in terms of the weight P per wheel and the C.B.R. value F of the soil) is given by

$$z = \frac{100 + 150\sqrt{P}}{F + 5} \quad \ldots\ldots\ldots\ldots(10)$$

(this formula merely expresses the values given in the relevant American graphs.)

Furthermore, the maximum pressure at a depth z is given, in Boussinesq's theory, by

$$q' = \frac{3\,P}{2\,\pi\,z^2} \quad \ldots\ldots\ldots\ldots(11)$$

for a concentrated load, and by

$$q' = \frac{P}{2\,\pi}\;\frac{3\,z^2 + R^2}{\left[z^2 + R^2\right]^2} \quad \ldots\ldots\ldots\ldots(12)$$

if the load P is distributed over a circle of radius R.

By combining these various formulae, we can compute the thickness that we must give the base in order to ensure that the pressure on the sub-grade will never exceed the pressure that it would support under the same loads if the pavement were of the flexible type.

But on carrying out calculations of this kind it becomes apparent that prestressed slabs, when uncracked, possess so considerable a load-spreading capacity that it is, generally speaking, not necessary to provide a base under the slab.

For instance, let us consider the very unfavourable case of a soil, very clayey and saturated with water, having a C.B.R. of 1. According to equation 10, a flexible pavement for withstanding loads of 7,500 kg would have to be made 85 cm thick. According to equation 11 this would correspond to a bearing pressure on the sub-grade of $q' = 0.500$ kg/cm^2 if the load is assumed to be concentrated, or $q' = 0.465$ kg/cm^2 (from equation 12), if the load P is distributed over a circular area of radius R = 18 cm.

Hence we see that these pressures are larger than those which occur under a 12 cm thick uncracked concrete slab, as in the example given in section V-2 ($q_o = 0.400$ kg/cm^2).

On the assumption that the concrete is not cracked, the base is therefore of practically no significance.

V-4

This is theoretical, however: in actual practice, the loads are not static, and it may be necessary, on account of the relative thinness of prestressed concrete pavements, to introduce an impact factor to take account of the dynamic effects. Experience alone can guide us in the matter.

But in any case this calculation appears to be of somewhat secondary importance if, as has been suggested in chapter IV, the analysis for the foundations is based on the second stage of behaviour of the pavement, where the concrete is considerably cracked.

In this latter case we must start by calculating the thickness x of the uncracked concrete, by equating the prestress calculated by means of equation 2 (where e is replaced by x) to the mean value $f \frac{e}{x}$ of the prestress, where f denotes the initial prestress.

As the function $\Phi(h)$ of equation 2 is defined only in graphical form, this calculation can be done only by trial and error. For instance, with the numerical values of the previous example and assuming a mean prestress of f = 20 kg/cm , we obtain x approximately equals 13 mm.

It should be noted that the compression in the concrete can then attain 400 kg/cm^2, a value which high-strength concrete as employed for prestressed concrete is able to withstand. In this connexion it should be borne in mind that pavements are always designed with very low factors of safety.

It should also be noted that for thin pavements, even if they have a high modulus of elasticity, the assumption of the concentrated load that was made for establishing equations 5 and 6 is no longer a valid one.

On the other hand, calculation shows that we can apply equation 12, derived from Boussinesq's theory, but that z must be replaced by $z + z_o$, where z_o is, as it were, the equivalent thickness of soil that replaces the slab. The analysis shows that

$$z_o = 1.60\ e \sqrt[3]{\frac{1}{6} \frac{E}{1-\eta^2} \frac{1-\eta'^2}{E'}} \quad \ldots\ldots\ldots\ldots (13)$$

This expression is valid only so long as the relative thickness h given by equation 1 is less than 1.

For cracked slabs this equation becomes

$$z_o = 1.60\ x \sqrt[3]{\frac{1}{6} \frac{E}{1-\eta^2} \frac{1-\eta'^2}{E'}} + e \quad \ldots\ldots\ldots\ldots (14)$$

and in the foregoing example we should thus obtain

$$z_o = 22 \text{ cm.}$$

Finally we must deduct z_o from the thickness of the flexible pavement as determined by the C.B.R. method, and thus obtain the thickness that the base (foundation course between slab and sub-grade) must have.

With the numerical values of the previous example the thickness of this base would be:

85 - 22 = 63 cm for a soil with C.B.R. = 1
64 - 22 = 42 cm for a soil with C.B.R. = 3
51 - 22 = 29 cm for a soil with C.B.R. = 5
34 - 22 = 12 cm for a soil with C.B.R. = 10
0 cm for a soil with C.B.R. = 18

Note: This method of calculation is undoubtedly a very conservative one, because in actual practice the cracking that will occur on a road will never be so intensive and so widespread as that considered in the analysis based on the second stage of behaviour. Even if the underside of the slab were broken by cracking into blocks the size of large paving setts, the bearing pressure on the underlying soil would be very substantially reduced.

But this calculation, though based on the most unfavourable assumptions, nevertheless yields acceptable values for the thickness of the base, and it is quite likely that these values are in any case necessary as a protection against frost and the movements of the sub-grade itself. Hence there would be no great advantage in discarding this very cautious method of calculation as proposed here. Experience alone will perhaps enable us to be a little bolder in the future.

VI. SLIDING OF THE SLABS ON THE SUB-GRADE

VI-1

One of the most important and most difficult points in prestressed concrete road construction is that of the sliding of the slabs on the sub-grade. It is through the sliding that the prestress is transmitted, because even in the so-called "continuous" or "immobile" form of construction (i.e. in the absence of an arrangement for automatically maintaining the prestress) it is from time to time necessary to re-inflate the jacks in order to restore the prestress which the creep and shrinkage of the concrete tend to reduce.

This sliding of the slab on the sub-grade was first thought of in terms of friction, or, to be more precise, as follows: as the slab is generally cast on a thin bed of sand, and as the underside of the slab is sufficiently rough to make the friction between concrete and sand greater than the internal friction of the sand, it was supposed that a sliding surface occurred within the sand. Under these conditions the coefficient of friction would have to be equal to $\tan\varphi$, where φ denotes the angle of internal friction of the sand.

For instance, for a very dense sand with $\varphi = 40^\circ$ we obtain $\tan\varphi = 0.839$; for a fine open-graded sand similar to Fontainebleau sand with $\varphi = 32^\circ$ we obtain $\tan\varphi = 0.600$.

It will readily be seen that, in these circumstances, if we exert a prestress f_0 at the end of the slab, then at a distance x from that end the prestress will only be

$$f = f_0 - \delta x \tan\varphi$$

where δ is the specific gravity of the concrete.

We thus see that with $\varphi = 32^\circ$ we should be losing 0·15 kg/cm^2 of prestress per metre length of pavement, i.e. a loss of prestress of 15 kg/cm^2 per 100m. Hence, if we wish to maintain a minimum prestress of 15 kg/cm^2 at all points along the slab, and if the initial prestress is 30 kg/cm^2, it will be necessary to place the prestressing devices for maintaining or "pumping up" the prestress not more than 200 m apart.

VI-2

Unfortunately, the tests that have been made on actual slabs, and the observations made on experimental stretches of pavement, have not confirmed this simple theory.

Thus, when it was attempted to force two slabs apart (one slab being short in relation to the other) by inserting a flat jack between them and inflating it, appreciable displacement of the smaller slab was not obtained until forces corresponding to friction coefficients in excess of 1 were applied.

Furthermore, some very interesting tests have been carried out by French Air Base engineers on the taxiway at Maison-Blanche airport, Algiers. These tests enabled a certain number of the parameters affecting the behaviour of prestressed concrete pavements to be ascertained with precision. At the time of constructing the taxiway, joints for "pumping up" the prestress were provided at intervals of 130 m. Each of these joints was equipped with three flat jacks.

The first jack was inflated at the very outset in order to produce the initial prestress and was filled with cement grout under pressure.

The second jack was used for a general "pumping-up" of the prestress in February 1956, as the prestress was found to have dropped to 10 kg/cm^2 at a temperature of 10° C in consequence of creep and shrinkage of the concrete. The jack was filled with cement grout at this stage.

The third jack was held in reserve for subsequent "pumping up" of the prestress if that should become necessary. By slightly inflating it with a liquid medium it was possible to use it as a pressure gauge, and it thus provided a means of accurately measuring the variations of the prestress in the taxiway.

Thus it was ascertained, during the "pumping up" effected in 1956, that the thrust exerted at a joint was practically not transmitted to the next joint (i.e. over a distance of 130 m), although the "pumping up" pressure was calculated so as to increase the prestress from 10 kg/cm^2 at 10° C (as it was at the time) to 110 kg/cm^2 at 30° C. Hence it followed that, if there really was friction, the coefficient of friction was higher than 2.

VI-3

Actually, however, what occurs is not simple sliding friction, but a rather more complex phenomenon similar to that observed in shear tests on soils carried out with Casagrande's shear box.

In that test (Figure 5) the soil sample to be tested is placed in a box consisting of two parts which can slide over each other. A compressive force N is exerted on the sample by means of a piston; then, while one half of the shear box is held fast, the other half is pulled with a force T acting in the sliding plane of the two halves of the box. A constant speed is maintained, and T is measured as a function of the displacement. For sands the operating speed, provided that it is low, can be chosen at random, since it does not affect the results obtained.

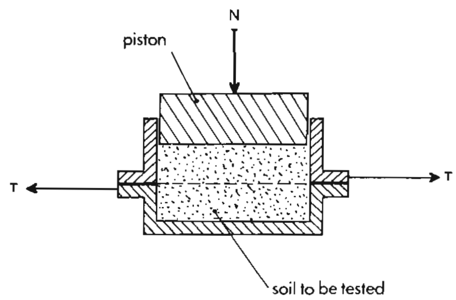


Figure 5: Shear test (Casagrande's shear box).

For a given value of N, the value of T is found to vary, when plotted against the displacement, in accordance with one of the curves given in Figure 6. Curve I rises, passes through a maximum, and then declines asymptotically to a horizontal limiting value, which is the value of T that is actually taken into account for calculating Φ . Curve II does not pass through a maximum, but it likewise tends asymptotically towards a horizontal limit.

Curve I is observed with densely compacted sands and with sands having a close grading or containing fines or calcareous particles.

Curve II is observed with the same sands if they are loose or with clean open-graded siliceous sands irrespective (or almost irrespective) of their compaction.

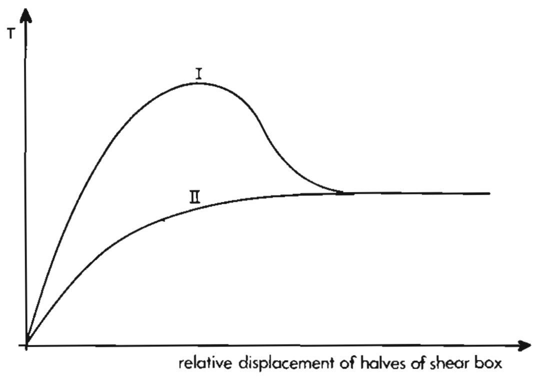


Figure 6: Typical shear curves.

Under a concrete slab, which in general is intensively vibrated, the sand will be considerably compacted. Hence, with the not specially selected sands which are usually employed for making the bases under prestressed slabs, we should observe curves of type I.

Now the sliding of a slab on its base is identical (apart from the scale) with the shear box test carried out in the laboratory. We must therefore observe phenomena similar to those represented by curve I or curve II. This is indeed confirmed by tests made in situ on isolated slabs. To be more precise: we are here dealing with an "elementary" phenomenon, i.e. it is valid for an elementary area of the slab, and it must be extended, by integration, to the entire area of the slab. If the displacement were the same at all points of the slab, the phenomenon itself would also be the same at all points, and the above-mentioned integration would simply consist in multiplying the elementary force by the area involved. Actually, however, the compressibility of the concrete causes the displacement to vary from one point to another, and it thus becomes necessary to integrate a differential equation.

VI-4

In addition, another unfavourable phenomenon plays a part, namely, the cohesion of the sand. In principle, the cohesion of sand is zero; but in reality this is only so under laboratory test conditions, in which the sand is either completely dry or completely immersed in water. In actual fact, if the sand is closely graded or if it contains fine particles or (which comes to the same thing) friable constituents which will produce fine particles by attrition (calcareous materials), such sand will have a certain slight cohesion in consequence of the small menisci of water that are concentrated at the points of contact between the grains. It is not possible to prevent the formation of these menisci either by condensation of atmospheric moisture or by water rising up by capillary action.

Besides, it must be noted that the pressure on the sand under the slab - except for very short times during the passage of vehicles, and indeed even during these times - will be very small in magnitude, so that even a relatively low cohesion is liable to have a considerable effect.

It must be borne in mind that the shear formula actually is

$$T = C + N \tan \varphi$$

Let us consider the case of a concrete slab 12 cm thick and assume the sand to have an angle of internal friction $\varphi = 32^o$ and a very low cohesion $C = 25$ g/cm^2. The force T required to displace one square metre of slab will be: T = 100 x 100 x 0.025 + 2,500 x 0.12 x 0.600 = 250 + 180 = 430 kg. We thus see that the effect of shear, viz. 180 kg, despite a relatively large angle of internal friction, is less than the effect of cohesion, viz. 250 kg, even though this cohesion is very small and can hardly be measured with laboratory instruments.

VI-5

Can this friction of the slabs on the sub-grade be reduced? This is undoubtedly one of the main points that will have to be cleared up by forthcoming tests on experimental sections of road. But we can already, in the light of the foregoing analysis, attempt to establish some of the principles involved.

To begin with, it is clear that a cohesionless soil is required, because even the slightest cohesion (as in a clay saturated with water) would have a more unfavourable effect than the internal friction, as we have just seen. We should not even rely on the plastic flow properties of such a soil because, for such flow to occur, it is necessary to exceed a certain value of the force T. Besides, the variations in the moisture content of such a soil, or the effect of frost, may completely upset its function as a sliding surface.

It is unwise to rely on oiled sand, because the oil is liable to harden after a time and it is then not possible to replace the base without having to destroy the concrete.

The only material that remains available for the purpose is sand. But it must be perfectly clean, (sand equivalent > 90) entirely siliceous (so as not to be friable), and open-graded. Sands of the Nemours type would be suitable.

It would obviously be necessary to take precautions to prevent pollution of this sand either by the concrete (at the time of casting the slab) or by the soil below.

Conversely, it may be advantageous to reduce the amount of sliding to a minimum in certain zones. It would seem that this can easily be achieved by choosing a close-graded sand containing some fines and incorporating between 100 and 150 kg of cement per cubic metre. In these zones the underside of the concrete slab would have to be made as rough as possible.

VI-6

The investigation of the sliding of a long slab on the ground, when it is pushed at one of its ends, involves some mathematical calculations because with these long slabs their elastic deformation plays a part and has the effect of causing the displacement to vary from point to point along the slab.

In order not to overweight the present text with mathematics, the analysis of the sliding of slabs on the sub-grade is given in Appendix II* to this article. The analysis is possible only if the resistance of the soil to sliding is simplified. For this purpose the following three simple assumptions have been made.

The first assumption is simply that the friction on the soil is constant.

The second assumption is that the relation between the sliding resistance T and the displacement V is represented by two straight lines: one of these passes through the origin and is sloping; it corresponds to a sort of bending of the layers of soil before actual sliding occurs; the other line is horizontal and corresponds to sliding with constant friction.

This second assumption is in general not a very good approximation of what really happens; that is why a third assumption is introduced, which is somewhat more complex but much closer to actual conditions: it consists in distinguishing two zones in the actual curve representing T as a function of V, the first zone being a quarter of a sinusoidal wave, and the second a horizontal straight line.

On experimental roads this third assumption will have to be adopted, because it is a close approximation to reality. It is possible, however, that it can be simplified when we have gained a better understanding of the phenomenon in question.

It must be noted that there is no point in producing too elaborate a mathematical analysis because, for one thing, soils are never very homogeneous materials and, for another, the assumption that the general phenomenon is merely the sum of the elementary phenomenon already constitutes a fairly rough approximation. In other words, we have found an elementary law which is valid for the shear box test in the laboratory, or for a small slab tested in situ, and we wish to integrate over the whole slab. To do this, an approximation has been made which is similar to that made by Westergaard who, in his slab analysis, replaced the soil by a large number of small independent springs, whereas actually the deformation of the soil at any particular point affects the soil as a whole, in accordance with Boussinesq's theory. In the present case, however, the simplification is a much more valid one than in Westgaard's analysis because the sliding movements affect only a fairly thin sand base under the slab.

<u>VI-7</u>

Besides, these calculations assume that the modulus of elasticity E does not vary with time. Actually, however, (as we shall see in due course) this modulus varies considerably if the duration of the application of the load varies.

We have, furthermore, so far only considered one single application of compression to the slab. In actual fact there will be several such applications at different times ("pumping up" of the prestress) and, indeed, in different directions (variations due to moisture and temperature effects). Now these are not elastic phenomena; they may consist solely of permanent deformations. Hence it follows that the state of deformation and of prestress of the pavement will depend upon the history of the applications of prestress to it. This type of pavement can thus be said to possess a "memory" and, as it were, to "remember" its previous movements before undergoing a fresh movement.

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* See note on first page of text, below title.

These "memories" can be wiped out only by large movements of the slab as a whole, comprising nothing but constant-friction sliding over its entire extent.

It should, finally, be noted that in actual practice the problem is complicated by the fact that there is not only a thrust acting at the end of the slab, but that there also occurs expansion due to variations of temperature and moisture content throughout the concrete.

The problem then becomes a very complex one. We have accordingly confined ourselves, in Appendix III*, to investigating some cases which are considered to be most unfavourable and most useful from the practical point of view. It is certain, however, that each individual case will require a special analysis.

VII. BEHAVIOUR AND RHEOLOGY OF PRESTRESSED CONCRETE

In the foregoing treatment of the subject the concrete has been considered as a perfectly elastic material which does not undergo deformations other than elastic ones. In actual practice, however, concrete subjected to loads of long duration will, in addition to elastic deformation, undergo deformations due to creep. Besides, other phenomena play a part in addition to the loads and likewise cause deformation, viz. variations of temperature and moisture content. These phenomena have a very considerable effect on the behaviour of prestressed concrete, and it is therefore worth our while to study them in detail.

VII-1

In principle, prestressed concrete is subject only to compressive stresses. Even if flexural stresses occur, the prestress should be of such magnitude as to ensure that no tension develops at any point. This is not quite true, however, of road pavements; we have seen that in some cases the loads can produce tensile flexural stresses in them. But, at all events, these tensile stresses are of short duration in comparison with the prestress, which is permanent, and with the thermal stresses, whose large cyclic variations are annual in character. There thus remain the creep phenomena to consider.

If a permanent compression is exerted on a concrete specimen which is kept at constant temperature and under saturation conditions of humidity, the specimen is found to undergo shortening which, as a function of time, exhibits a behaviour corresponding to one of the curves in Figure 7.

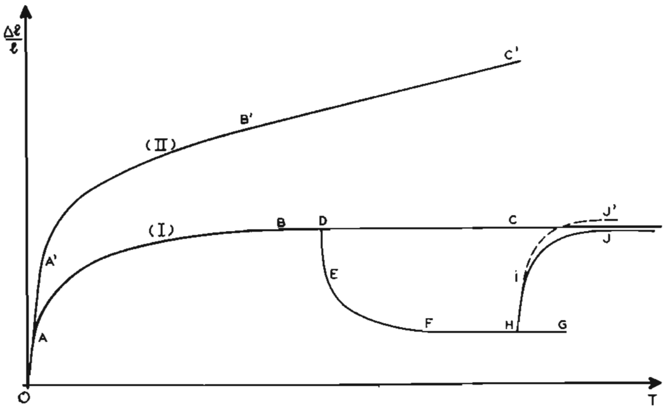

Figure 7: Strain of concrete under permanent loads.

Curve I comprises an initial portion OA representing elastic deformation; this is followed by a delayed deformation AB at a gradually diminishing rate. The total deformation thus tends towards a limiting value which, for practical purposes, it reaches at B.

* See note on first page of text, below title.

Under large loads the deformation will correspond to curve II, which likewise exhibits a delayed deformation AB' which is of approximately the same duration as the delayed deformation of curve I. But in this case the total deformation, instead of tending towards a horizontal asymptote BC, now has an inclined asymptote B'C'. In other words, the total deformation goes on increasing indefinitely. The straight portion B'C' of the curve corresponds to creep occurring at a constant rate. It is this phenomenon that is generally referred to as "flow".

The higher the load is, the more rapid will the creep be, i.e. the steeper will be the slope of the line B'C'. The creep limit is the load beyond which creep (in the sense of true "flow") occurs: that is to say, it is the limiting value of the load that separates the curves of type I from the curves of type II. *

This load constituting the creep limit is, however always higher than one-third of the ultimate strength of the concrete. Hence we need not concern ourselves with that limit when dealing with prestressed concrete pavements, in which the stress is always below one-third of the strength, especially if a high cement content is employed (which, as has already been mentioned, is advisable in order to increase the flexural strength).

We shall therefore normally have to deal only with curves of type I.

VII-2

Let us consider these curves more closely. The portion OA is proportional to the instantaneous modulus of elasticity (E) of the concrete on the day of loading. Highway concrete has a high modulus of elasticity (of the order of 450,000 kg/cm^2). The modulus can be increased to as high as 500,000 kg/cm^2 by raising the cement content above 400 kg per cubic metre of concrete. This high value of the modulus is reached only after a certain length of time, however (something like three months, depending on the climate); with young concrete the modulus may be much lower (even as low as one-half).

There is a tendency to apply the prestress to concrete pavements at an age of only one or two days in order to prevent cracking. In that case the strain corresponding to OA is likely to be quite considerable - at any rate, in relation to the rest of the curve, which corresponds to older and harder concrete.

With old concrete, both the elastic strain and the creep strain† are smaller than with young concrete. The first-mentioned strain is, by definition, proportional to the load; but, broadly speaking, the creep strain is likewise proportional to the load (at least, in the range of prestress and stresses normally employed).

For instance, at Maison-Blanche the concrete had an instantaneous modulus of elasticity of 450,000 kg/cm^2; the long-term value of the modulus of elasticity over one year was 170,000 kg/cm^2, and over two years it was 140,000 kg/cm^2. This reduction of the value of E is quite general, and we may say that in concrete pavements the creep strain is approximately twice as large as the instantaneous elastic strain.

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* French has the same word ("fluage") for "creep" and "flow"; the author's "limite de fluage" could therefore also be rendered as "flow limit" in the sense that true plastic flow occurs only above this limit. (Translator's note)

† In common with other French writers on the subject, the author uses "deformation differee" ("deferred strain") for the creep occurring in type I curves; the term "fluage" is reserved, in the more precise sense, for type II. (Translator's note).

The duration of the creep strain varies with the climate and with the dimensions of the test specimen or of the structure. With small specimens it may come to an end in six months; on a large structure it may last from 3 to 5 times as long.

With stresses of the magnitude usually employed, the instantaneous deformation or strain OA may be considered as being perfectly elastic, i.e. proportional to the stress and reversible. It is not so with the creep strain, which may produce a considerable permanent deformation. Thus, if we unload the specimen at a point D on the original curve, the unloading curve will, to start with, have a straight portion DE which is equal to OA and has the same slope, if the concrete has not meanwhile matured. But if the initial load was applied to a young concrete, the instantaneous recovery DE in the same concrete at an older age may be less than OA (to an extent depending on the ratio of the modulus of elasticity values at the two ages concerned). This is followed by a creep recovery EF which tends fairly rapidly towards a limit FG, which is located above the origin O.

Unfortunately, little information is available regarding the value of the permanent strain F. And even less is available regarding what happens in a case where the initial prestress is "pumped up", as represented by HI. Some investigators believe that the curve IJ must tend asymptotically to the original limit CD; but others believe that the creep curve IJ' will intersect the original limit CD. This second assumption as to the probable behaviour appears to be confirmed by the fact that repetitions of the same load accelerate and probably even increase the creep strain.

In actual practice the problem is rather more complex, because the magnitude of the thrust exerted on the concrete slab is not constant, but varies with the humidity.

It therefore appears essential to construct, as soon as possible, some experimental lengths of road on which the behaviour of the concrete will be studied with the most up-to-date facilities at the disposal of a large testing laboratory competent to deal with the subject. The experience provided by the runway at Algiers will be particularly valuable, because this runway is regularly examined by the Air Base engineers with the aid of, inter alia, the convenient method of using the third flat jack (hitherto kept in reserve for "pumping up" the prestress) for the measurement of the thrusts exerted.

Tests of this kind are an essential requirement for devising an approximate rheological model for concrete, which will be the only means of obtaining a proper understanding of the phenomena concerned and of evaluating them in advance.

With regard to the creep phenomena of concrete it should finally be noted that their investigation cannot be carried on exclusively in terms of the modulus of elasticity E, but that Poisson's ratio η also comes into the picture. A knowledge of the laws governing the variations of Poisson's ratio would be essential to the study of pavements prestressed in one direction only (see chapter IX). Little is known about the subject, however. All we know is that in the instantaneous elastic strain of concrete, Poisson's ratio is always something like 0.25 - 0.30. On the other hand, it would seem that this ratio has a very low value during the creep stage. Here, indeed, is a point that needs clearing up as quickly as possible.

VII-3

So far, we have treated the deformation of concrete under load as something separate from its deformation due to variations of temperature and moisture content. But this distinction is a somewhat academic one, and certain authors even believe that creep and shrinkage of concrete are merely two aspects of the same phenomenon. It would, perhaps, be more correct to say that they are two closely related phenomena.

Thus, we no longer speak of creep and shrinkage as two quantities that can simply be added together to obtain a certain total deformation. The assumption that they can be added is only a rough approximation and is, indeed, sometimes entirely incorrect if the concrete is young. This is probable due to the internal changes taking place in the concrete (in particular, the hardening process); we have already noted that hardening under load enhances the tensile strength of concrete (and also its compressive strength).

To study the combined development of the two phenomena would call for a great many laboratory investigations or, preferably, tests on experimental sections of pavement.

In "Traité de matériaux de construction" by Duriez (Vol. I, pp. 492 - 498) a report is given on the tests carried out at the Laboratoire Central des Ponts et Chaussées (Central Laboratory of the Bridges and Highways Department), over a period of five years, on two concretes used for the Vienne Bridge over the Rhone. The curves for the shrinkage of thest two concretes under load and in the absence of load are given in Figures 8, 9, 10, and 11.

We see that the total values for the strain of these two concretes under load were, at five years,

$$470 \times 10^{-6} \quad \text{and} \quad 610 \times 10^{-6}$$

The corresponding shrinkage strains were

$$280 \times 10^{-6} \quad \text{and} \quad 310 \times 10^{-6}$$

It will be noted that from an age of 2 years onward, the compressive strains and the shrinkage strains followed two exactly parallel curves. It would thus seem that creep must have been completed at that point and that only the shrinkage went on increasing.

It should also be noted that the tests were carried out in a deep and relatively damp cellar. That is why the shrinkage took place so slowly; in pavements in the open air, the entire shrinkage would be accomplished in at most two or three years.

Finally, it should be borne in mind that this was not highway concrete, but a concrete of much poorer quality.

VII-4

It would be desirable for prestressed highway concretes to have as little shrinkage as possible, and this is all the more desirable because reduced shrinkage means reduced creep. Hence it is worth noting that the shrinkage of concrete depends essentially on four factors:

the quality of the cement and, in particular, its fineness;

the cleanness of the sand (i.e. its sand equivalent);

the grading of the concrete mix;

the water content of the mix.

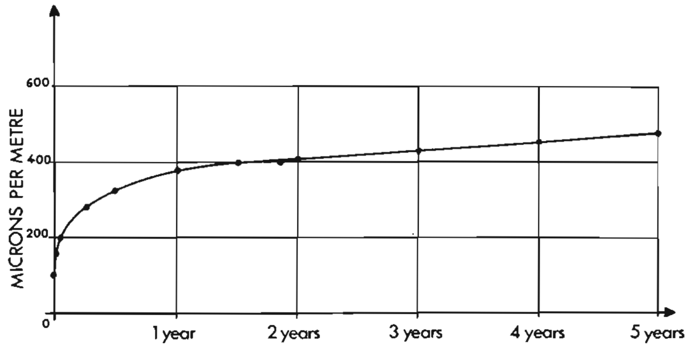

Figure 8: Total strain under compressive load. "Porte de France" cement.

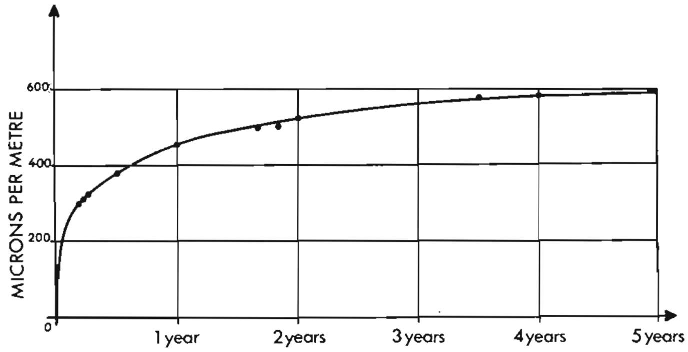

Figure 9: Total strain under compressive load. Lafarge cement.

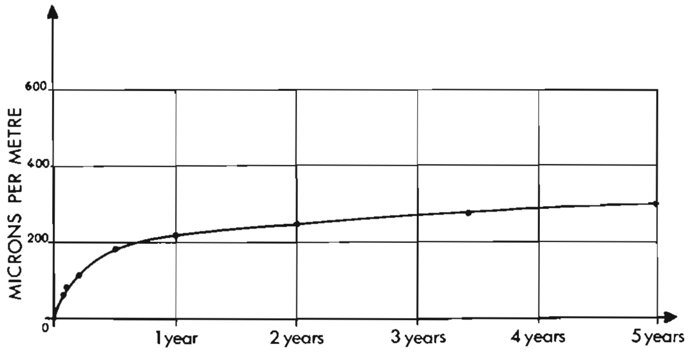

Figure 10: Shrinkage in the absence of load. Lafarge cement.

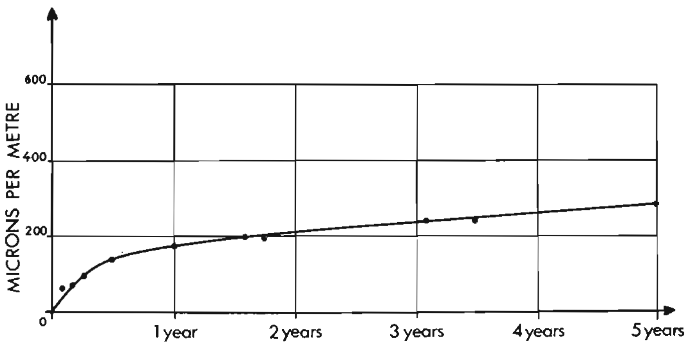

Figure 11: Shrinkage in the absence of load. "Porte de France" cement.

To reduce shrinkage, we must therefore:

use a cement having a low shrinkage of its own;

use a very clean sand (if possible, sand equivalent = 85 and even 90);

adopt a very carefully designed grading with the soundest possible structural skeleton, i.e. a gap grading;

use the lowest possible water content, which involves using powerful means of vibration and, perhaps, the admixture of plasticizers.

Incidentally, it should be noted that these rules for making concrete will also produce concrete possessing a very high strength, especially tensile strength. Hence one should not hesitate to specify them and see that they are carried out.

A further point to be noted is that the above rules say nothing about the cement content. Contrary to the belief held by many engineers, the cement content of the concrete is only a secondary factor with regard to shrinkage (provided that a high quality cement is employed). And even the creep depends mainly on the ratio between the applied load and the ultimate load; and increasing the cement content increases the ultimate load.

Actually there must be an optimum cement content; but it would seem that this optimum is somewhere in the range of 450 - 500 kg/m^3 for highway concretes. For this reason, too, it is desirable to increase the cement content, as has been suggested above.

The deformations due to variations of moisture content can occur in two directions. They are proportional to the relative humidity (expressed as a percentage of saturation). The proportionality factor varies from 2 for excellent concretes (see above) to 4 for bad concretes. It was 3 for the concrete of the Vienne Bridge which was the subject of the creep tests mentioned above.

VII-5

The thermal variations of concrete are better understood, it seems. For one thing, we can calculate the propagation of temperatures in concrete, thanks to Fourier's equations which yield a sinusoidal solution as a function of time and an exponential solution as a function of the depth. Besides, a number of satisfactory experimental checks have been made on such calculations.

The result is, apparently, that for thin pavements such as prestressed concrete pavements the daily variations in temperature should be taken into account and that we must not, as when dealing with thick pavements, merely content ourselves with considering the variations of the daily averages. This is, perhaps, a point where roads differ from runways.

We must therefore calculate or estimate, on the one hand, the annual maximum variations of the average daily temperature (average based on time) and, on the other hand, the daily maximum variations of the average of the temperature of the slab (average taken over the full thickness of the slab). But we must not simply add these two variations together, because they do not act in the same way: the moduli of elasticity of the concrete that have to be considered in each of these two cases correspond to very different durations of time.

Let λ denote the coefficient of thermal expansion of the concrete, T the range of temperature variation involved, and E the modulus of elasticity of the concrete. The concrete is assumed to undergo these thermal variations, but is prevented from expanding by abutments or indeed merely by the sub-grade restraint. Stresses will occur in the concrete, which are expressed by the formula

$$\frac{dL}{L} = \frac{N}{E} = \lambda T$$

and therefore

$$N = \lambda E T$$

It is therefore necessary to know the λE of the concrete with respect to these thermal variations.

For example, if the variation in temperature took place instantaneously, we should have $E = 450,000 \ kg/cm^2$, i.e. for normal concrete with $\lambda = 8 \times 10^{-6}$ we thus obtain $\lambda E = 3.6$. This would give rise to very high stresses which might cause buckling of the pavement.

Fortunately, however, the modulus of elasticity is reduced where the load is applied slowly. But what value should we then adopt for λE ?

It would seem that the tests which are more or less permanently in progress at Maison-Blanche, Algiers, can supply some interesting particulars with regard to this point. The following values were found:

$\lambda E = 1.5$ with respect to annual variations;

λE varying from 2.5 in winter to 3.6 in summer with respect to daily variations, i.e. an average value of 3.

These values cannot be directly applied to other runways, because the Maison-Blanche concrete, which is very calcareous in character, has a low coefficient of thermal expansion ($\lambda = 6 \times 10^{-6}$). The siliceous concretes usually employed in France have a higher coefficient of expansion ($\lambda = 8 \times 10^{-6}$, and even as high as 9×10^{-6} for flint concretes). The values of λE as indicated above would therefore have to be increased in proportion.

For example, let us suppose that we must make prestressed concrete in the Paris region, using concrete made with flint aggregate. For the purpose of this example we shall assume the following values:

annual variations: $\lambda E = 2$

$dT = 40^{\circ}$

daily average variations:

$\lambda E = 4$

$dT = 10^{\circ}$

i.e. the total variation of the prestress for a pavement of the "continuous" (or "immobile") type is:

$$N = 2 \times 40 + 4 \times 10 = 120 \ kg/cm^2$$

(The variations of temperature must be estimated in each actual case, and be based on the meteorological data of the district and also on calculations or measurements of the transmission of daily variations of temperature).

VIII. RISK OF BUCKLING OF THE PAVEMENT

We see that prestressed concrete pavements - and especially highway pavements, being thinner than runways - have to withstand very high thermal stresses.

It is therefore of some importance to study the buckling resistance of these pavements. This matter has been investigated in a fairly complete manner in Appendix IV* to the present article.

The results show that, even with the high compressive stresses occurring in pavements of the "continuous" type, there is no need to fear buckling if the slab thickness is more than 16 cm. From 12 to 16 cm buckling is possible, but should not occur if the pavement is constructed very carefully. Below 12 cm thickness of the concrete the risk of buckling becomes considerable, and the "continuous" form of construction would no longer appear to be a feasible solution.

It should be noted that these conclusions have been verified by experience. No buckling has ever been reported in slabs more than 15 cm thick. On the other hand, in the Orly tests carried out on slabs with thicknesses of 8, 10, and 12 cm, the prestress during the tests had to be limited to 75 kg/cm^2, because movement at the joints had been detected.

SECOND PART

DESIGN OF PRESTRESSED CONCRETE ROADS

VIII-2

Two alternatives are available to the designer of prestressed concrete roads:

should the pavement be provided with a transverse as well as a longitudinal prestress, or may one content oneself with a longitudinal prestress only?

should the "continuous" (or "Immobile") or the "individual slab" (or "mobile") type of solution be adopted for producing and maintaining the prestress?

It is these two questions that we shall examine in the following.

IX MUST A TRANSVERSE PRESTRESS BE PROVIDED?

Longitudinal prestressing is undoubtedly the most effective and the least expensive form of prestressing.

IX-1

It is the most effective form of prestress because, as we have already seen when considering the mechanical strength of slabs (see chapter IV), the most unfavourable loading condition is that of loads situated at the edges of the slabs: cracks tend to develop in the transverse direction. To put it more precisely: the longitudinal bending moment is higher than the transverse bending moment; the ratio between these two moments is about 2:1 in thick slabs, and it gradually diminishes to 1:1 for very thin slabs. These two values have

* See note on first page of text, below title.

been calculated in chapter IV (and in Appendix I).

For the slab thicknesses with which we are, for practical purposes, concerned here, this ratio is always fairly close to 2:1.

In non-prestressed slabs the concrete is required to have a 28-day flexural strength of 45 kg/cm^2 (i.e. at least 60 kg/cm^2 at the end of 6 months), and practical working stresses of 35 kg/cm^2 (not including thermal stresses) are adopted. It would seem that with prestressed slabs this stress could be increased to 75 kg/cm^2, to which must be added the minimum residual prestress (not including thermal stresses); this will leave a small margin of safety, but sufficient to ensure that a prestressed concrete slab, though cracked on its underside, is still far from being destroyed.

If the slab is prestressed in the longitudinal direction only, the working stress could be 75 kg/cm^2, plus, for example, 15 kg/cm^2 of minimum residual prestress, i.e. 90 kg/cm^2 in that direction; in the transverse direction the working stress would then be 45 kg/cm^2. Hence the transverse prestress would merely serve to raise the working stress 10 kg/cm^2 (as compared with the 35 kg/cm^2 permitted in non-prestressed slabs), which is not much.

In reality, however, the transverse prestress will afford an additional amount of safety inasmuch as it enables the slabs to go on functioning satisfactorily beyond cracking. But the same advantage can be obtained by installing ordinary transverse reinforcement or, better still, a reinforcement consisting of deformed bars ("Tor" steel, indented bars, etc.) which will limit the width of possible cracks to capillary dimensions.

IX-2

Let us examine the difference between the two solutions from the point of view of economy. To do this, we shall consider a certain specific case, with the following numerical values: $E = 450{,}000$ kg/cm^2 $E' = 600$ kg/cm^2
$\eta = \eta' = 0.25$ $P = 7.5$ tons (= 7,500 kg) $R = 20$ cm
(i.e. $p = 6$ kg/cm^2)

The calculations then show that in order to resist this load placed at the edge, a slab prestressed in both directions must have a thickness of 13 cm. If the slab is prestressed in the longitudinal direction only, a thickness of 16 cm will be necessary. Therefore, the transverse prestress permits a saving of only 3 cm of concrete, corresponding to something over 300 francs per square metre at present-day prices.

The transverse prestress will also permit a saving in the deformed transverse reinforcing bars. If there would have been five 10 mm diameter bars per linear metre, the saving in steel thus effected would be 3 kg/m^2. At 150 francs per kilogramme this represents a saving of 450 francs per square metre, so that the total saving becomes 750 francs per square metre.

On the other hand, the prestressing steel will entail additional expense. Even a residual prestress of only 10 kg/cm^2 (which is the very minimum) would require about 1.5 kg of high-tensile steel for every square metre of slab. The problem therefore consists in determining whether the transverse prestress will cost less than 500 francs per kilogramme of steel, or less than 750 francs per square metre of concrete.

IX-3

It is doubtful whether it will. Besides, the solution with prestress in the longitudinal direction only, has the further advantage that its greater thickness of slab practically rules out any risk of buckling and permits a reduction of the impact factor (if applicable).

In addition, this thickness of 16 cm can be constructed with present-day road-making plant and does not require a higher degree of accuracy in levelling the surfaces than is at present normally employed.

Indeed, this being so, the thickness of 16 cm could even be standardized, so that the contractors would thus avoid the need for having several different types of road forms at their disposal. If a poor sub-grade was encountered, it would merely be necessary to provide a gravel type base of modest thickness under the concrete slab to bring the working stress in the concrete down to an acceptable value. Yet this thickness of 16 cm is sufficiently small to leave the surfacing enough flexibility to enable it to adapt itself to any movement that may occur within the sub-grade itself.

It should also be noted that, in consequence of lateral strain (Poisson's ratio effect) and the use of transverse bars possessing excellent bond properties, the longitudinal prestress can create an "induced" transverse prestress in the slab. We should, however, have no illusions about the magnitude of this transverse prestress because, for one thing, poisson's ratio for concrete appears to be quite low in the case of creep and since the third dimension (vertical) is unrestrained, the expansion of the concrete will naturally occur predominantly in the vertical direction.

X. PRODUCING THE LONGITUDINAL PRESTRESS

There are two ways of producing the longitudinal prestress, corresponding to the "continuous" (or "immobile") and to the "individual slab" (or "mobile") form of construction.

In the first of these two solutions the concrete slab is rendered immobile in the sense that variations of temperature cannot cause movements of the slab in relation to the sub-grade. In the second solution the slab can slide on the ground (base or sub-grade) in consequence of such movements.

X-1

In the first solution the slab is really immobile only as regards variations of temperature and possibly seasonal or daily variations of moisture content. But the slab must be able to slide on the ground, both initially so that the prestress can be produced, and subsequently so that the prestress can be "pumped up" when it has diminished as a result of creep and shrinkage of the concrete.

At the present time the best method of producing the prestress would appear to be the method developed for the taxiway at Orly and subsequently applied at Maison-Blanche. It consists in inserting a set of at least three Freyssinet flat jacks between two lengths of pavement (Figure 12). The desired prestress can be obtained by inflating these jacks with water; this prestress can then be "locked" in the slabs by filling the jacks with cement grout which, on hardening, eliminates any risk of loss of prestress due to leakage of a jack.

For practical purposes the first prestressing operation, in which one of the jacks in each set is used, is carried out when the concrete is very young (e.g. at an age of 48 hours), as soon as the concrete has become hard enough to resist the pressure and a length of pavement has been constructed that is

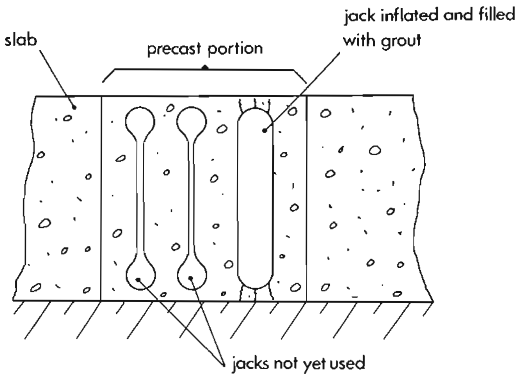

Figure 12: Joint incorporating flat jacks.

sufficient to absorb the thrust. In consequence of shrinkage and creep of the concrete it is necessary to "pump up" the prestress after a few months (generally in winter): this operation "uses up" the second jack of the set. There then remains the third jack (and possibly a fourth, if provided) for subsequent "pumping up" operations which are carried out at longer intervals. (Thus, at Algiers, after 4 years the third jack of each set has not yet been inflated with grout).

The jacks held in reserve may be gently inflated with water and can thus serve as pressure gauges for checking the evolution of the prestress as a function of time and of variations in temperature and humidity as has been done by the Air Base engineers at Algiers. Thus one can tell exactly when "pumping up" should be done, and it is even possible to predict the optimum date of this operation from the trend of the curves. But it is necessary to check the temperature curve within the thickness of the slab, because the measurements of thrust have a precise significance only if the temperatures within the slab are uniform or nearly uniform (which, in theory, occurs twice a day).

Indeed, it should be possible to devise simpler and more reliable appliances for keeping a permanent check on the prestress.

Basing oneself, in particular, on the principles set forth in Appendixes II and III, one can forecast the dates and the probable effects of "pumping up" and ascertain the optimum distance between jacking joints.

It must be borne in mind, however, that - at any rate, in their present form - these jacks have only a fairly small travel; for practical purposes it is therefore necessary for these joints to be fairly close together (130 m apart at Maison-Blanche, Algiers).

It should also be noted that these jacks, when inflated with water, constitute almost perfect articulated joints. In these circumstances, buckling at the end of the slab is quite liable to occur. Hence it is necessary to provide anti-buckling devices, such as the "combs" used at Algiers (Figure 13).

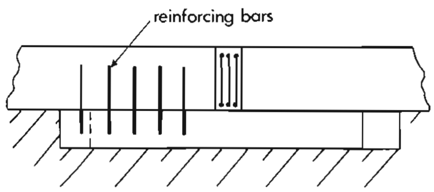

Figure 13: "Comb" device

This device comprises a number of concrete beams laid side by side on the ground in the longitudinal direction and located under the future joint. These concrete beams are provided with projecting reinforcing bars whereby they can be rigidly connnected to the overlying slabs when the latter are concreted. The beams are alternately connected to each of the two slabs on either side of the jacking joint, so as to form a system consisting of two interlocking "combs", each rigidly connected to one slab and extending some distance under the other. Thus, it is not possible for one of the slabs to lift without lifting the other. Besides, as the underlying sub-grade prevents the "combs" from dropping, they transmit (partially, at any rate) the bending moments from one slab to the other and thereby prevent the formation of an articulation at the joint.

The free upper surface of the "combs" should be smooth or, better still, coated with bitumen to facilitate the sliding of the slab over it. This sliding movement may have quite a considerable magnitude: it may be of the order of one-thousandth of the distance between two joints, e.g. 10 cm if the joints are spaced 100 m apart.

X-2

In the "individual slab" type of construction the joints are spaced at regular intervals in the concrete and are provided with devices for automatically maintaining the prestress.

Whereas in the "continuous" type the slab is kept fixed in relation to the sub-grade and thus has to take up very high compressive stresses in the warm season (up to 140 kg/cm^2 in our climate), in the "individual slab" type it is the prestress that is kept constant (or almost so) at the end of the slab, and the slab slides on the ground according to the movements produced by variations of temperature and humidity.

Thus the prestress can - near the ends of the slabs, at any rate - be limited to relatively low values. Owing to sub-grade restraint, however, variations in the prestress will still occur in the centre portions of the slabs, and these variations will be larger according as the slabs are longer. These variations can be calculated by proceeding as indicated in Appendix III. Unfortunately, the numerical coefficients that govern the sliding of slabs on the sub-grade are not yet properly known; a certain factor of safety should therefore be introduced, and it would seem that, for the time being, it is not feasible to go beyond slab lengths of 150 - 200 m.

In this type of construction it is necessary to prevent any movement of the pavement as a whole. This can be achieved either by anchoring it in places by means of special abutments or, if the slabs are of sufficient length, by improving the contact between the slab and its base by making the sand slightly coherent. The addition of a small quantity of cement will have this effect.

This "individual slab" type of construction would appear to be necessary for thin slabs that would otherwise be in danger of buckling, e.g. doubly prestressed slabs (longitudinally and transversely prestressed).

At Maison-Blanche the "continuous" solution was adopted for the central zone, and the "individual slab" solution at the ends. The ends are connected to buried abutments by means of long prestressing cables, which provide an approximately constant thrust.

It might be considered, in theory at least, that the ends of the runway slab could bear directly against the abutments, so as to obtain a "continuous" solution over the whole length of the runway. For that purpose, however, one would have to be certain that the abutments cannot move, because even a slight displacement could produce a local loss of prestress in the slab.

In road construction it should be possible, and indeed it is essential, to use devices that are less expensive and, above all, easier to repair.

There thus remains, it would seem a good deal to be done in designing anchorage devices and devices for maintaining the prestress.

As regards maintaining the prestress, a solution might be sought in the use of a kind of flat jack inflated with a fluid at constant pressure (liquefiable gas, for example) or in the use of devices utilizing appropriate plastics (nylon, rislan, etc.). These are mechanical problems for which it should be possible, with a little imagination, to find simple and inexpensive solutions.

XI MISCELLANEOUS PROBLEMS

XI-1

Among the problems presented by the construction of prestressed concrete roads, one of the most important is that of anchoring the pavement, so as to prevent any movement, especially in curves.

In curves there may be a considerable "unbalanced thrust" (radial thrust due to curvature), even if the radius of curvature is more than 500 m, as it will be on trunk roads. For example, consider a carriageway 10.50 m wide and 16 cm thick, of the "continuous" type, subject to its maximum working stress of 140 kg/cm^2.

The corresponding longitudinal thrust in the slab is 2,350 tons.

The slab can be provided with a sort of dwarf wall forming an abutment to take up the thrust on the outside of the curve. The thrust per linear metre of wall would thus be

$$P = \frac{P}{R} = \frac{2,350}{500} = 4.7 \text{ tons/m}$$

This is not a very large thrust, and by making use of the passive earth pressure it should be possible to provide economical structures that will easily be able to take up these thrusts.

The most difficult point, it seems, is to prevent appreciable movements of the supports at the instant of application of the load, because such movement would produce an irregular distribution of the prestress over the width of the slab. Such movements can be prevented by amply dimensioning the dwarf walls and also by installing flat jacks between the road slab and the walls, the purpose of these jacks being to compensate for the effect of any movement that may occur.

It is also necessary to minimize the friction of the road slab against the dwarf wall, because this friction, which in curves increases exponentially, is liable quite soon to cancel the prestress at such points.

In the "individual slab" type of construction the maximum thrusts are much smaller, so that the anchorage and thrust-resisting devices provided at curves can be less costly. The expense entailed by these devices does not appear to have any considerable effect on the total cost of the road, however.

XI-2

In both of these solutions (i.e. "continuous" and "individual slab") abutments will be necessary, either at the ends for forming the junctions with the conventional highway pavements, or at intervals along straight stretches of considerable length in order to prevent "creeping" (progressive movement) of the surfacing as a whole. If the alinement contains curves, the thrust-resisting dwarf walls at these curves will likewise serve to prevent "creeping".

Experience alone can tell us how far apart the abutments can be spaced on straight lengths of road; it would appear to be at least a kilometre. At Algiers this spacing is 2,400 m, and the runway is behaving very well. It may in the future even be possible to dispense with the intermediate abutments altogether; for the time being, however, it is advisable to provide them.

These abutments should be of a simpler type and more easy to repair than those which have been used at Algiers. They could, it seems, be designed in the form of hollow plain or reinforced concrete box-type structures that take the fullest possible advantage of the passive earth pressure. If necessary, the soil could be improved or replaced by specially selected soil in the neighbourhood of the abutments in order to increase the passive pressure as much as possible.

Simple and robust devices for maintaining (and, possibly, for measuring) the prestress should be installed between the road slab and the abutment.

XI-3

There remains the question of estimating the saving that could be expected from prestressed concrete roads in comparison with conventional roads. That would, however, be outside the purely technical scope of this paper. Besides, we do not appear, as yet, to possess all the data required for drawing up an accurate balance sheet on the subject.

It may be noted, however, that prestressing effects a saving of some ten centimetres in the thickness of the concrete (16 as against 26 cm for the Paris South Motorway), and although the concrete concerned is not of quite the same quality in both cases, the economy achieved would appear to be about 750 francs/m^2. The elimination of the joints that have to be provided in conventional concrete roads will undoubtedly also yield a substantial saving, but this may be almost entirely cancelled by the special devices necessitated by prestressed concrete: flat jacks, abutments, dwarf walls, etc. And then there are the high-tensile deformed reinforcing bars that have to be installed transversely in the prestressed slab; as indicated above, their cost may be estimated at around 450 francs per square metre. The total saving that can be expected from the use of prestressed concrete instead of conventional concrete, if the reduced cost of the base (which is simpler) is also taken into account, should be at least 10%.

In addition, there will be a much greater saving in the cost of maintenance, which would appear to be neglible for prestressed concrete, to judge from the tests and lengths of prestressed road that have been executed to date. A further advantage is the superior technical quality of the surface, particularly as regards riding quality.

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