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PRESTRESSED CONCRETE ROADS AND AIRFIELDS IN HOLLAND AND ABROAD

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

PRESTRESSED CONCRETE ROADS AND AIRFIELDS IN HOLLAND AND ABROAD

INTRODUCTION

Prestressed concrete, which has already been in extensive use in bridge construction, etc. for a good many years, has more recently been introduced into highway engineering. The application of prestressed concrete to this branch of engineering has been found to offer such advantages that it certainly deserves to have considerable attention paid to it.

What are these advantages?

(a) Prestressing a concrete road surfacing greatly reduces the risk of cracking.

(b) The prestressed surfacing is better able to withstand temporary overloading by a very heavy load than any other type of surfacing, because when a crack occurs, it will close up again once the load has passed. Ordinary concrete roads are unable to withstand overloading.

(c) The design of an ordinary concrete road is in a great measure determined by the temperature stresses that occur. These stresses make it necessary to provide expansion and contraction joints, which require a good deal of maintenance.

If prestressing is employed, the contraction joints can be dispensed with altogether, and the number of expansion joints can, to an extent depending on the prestressing system used, be greatly reduced so that the ideal of the "jointless road" can be nearly attained.

(d) To increase the load-carrying capacity of an ordinary concrete road, with or without reinforcement, it will usually be necessary to increase the thickness of the slab. As the rigidity of the surfacing is thereby considerably increased, the sub-grade will develop reactions which will produce larger bending moments. The advantage of the thicker concrete slab will therefore be lessened.

With prestressed concrete a stronger and more flexible structure is obtained. A surfacing of this kind adapts itself better to the subsidence movements of the sub-grade and the stresses due to changes of temperature in a thin prestressed slab are smaller than in a thick slab of the ordinary kind.

There are, in other countries, numerous examples of very successful airfield and highway pavements constructed with prestressed concrete.

The fact that prestressed concrete roads, despite their great advantages and successful applications, are not yet employed on a large scale is attributable, on the one hand, to the relatively high cost of the jobs that have been carried out and, on the other hand, to the unwarranted lack of interest hitherto displayed by highway designers and contractors.

PRINCIPLE OF THE PRESTRESSED PAVEMENT

Under an unloaded slab the sub-grade reacts with a pressure which is, at each point, equal to the weight of the slab. A load applied to the slab will cause it to deflect locally. The distribution of the bearing pressure with which the sub-grade counteracts the load will depend on the magnitude of the deflexion. The bearing pressure as a whole is equal to the load.

Positive and negative bending moments are produced in the slab. The shape of the bearing pressure pattern, which can be calculated in various ways, is determined by the rigidity ratio between slab and sub-grade. Without going further into this question, it can be stated that, for a given load, the bending moments will be larger according as the slab is more rigid and the sub-grade is more yielding.

The slab must therefore be so dimensioned that, on the one hand, the counteracting bearing pressure produced by a load is not distributed over too large an area (in order to limit the bending moments to low values) and that, on the other hand, the distribution area is not too small (in order to avoid local overloading of the sub-grade). A prestressed concrete surfacing is thinner than a surfacing constructed of ordinary concrete. Hence the load-spreading action will be less. This is one reason why it is considered desirable to provide a base under a prestressed slab.

Three stages can be distinguished in the loading of a prestressed concrete slab.

Stage 1

The load is acting on the slab, but there is as yet no cracking.

Stage 2

In consequence of the load, cracking begins to develop on the underside of the slab. When a crack occurs, the deflexion increases and the sub-grade pressure tends to concentrate at the crack. The bending moments, and therefore also the tensile stresses in the concrete, diminish in magnitude, but the bearing pressure per unit area increases. Besides, the total prestress, which initially acted upon the entire section, will then be concentrated on the still uncracked part of the section, which is therefore greatly strengthened. The closing of the crack after the load has passed is greatly assisted. Hence in a prestressed slab this stage is far less dangerous than in an ordinary concrete slab. All the same, the designer should endeavour to ensure that this stage will not occur under working load.

Stage 3

The load causes a complete crack to develop. This stage will be reached only if the compressive strength of the concrete is exceeded at the top of the slab or if considerable deflexion occurs.

It can furthermore be stated that many experiments in other countries (including those conducted by Becker at Orly in France) have shown the flexural strength of prestressed concrete to be much higher than that of non-prestressed concrete, which is probably because the microscopic cracks that occur in the concrete during the hardening process are pressed shut by the prestress.

In view of the foregoing it can therefore be said that the bearing capacity of a prestressed concrete surfacing is many times greater than a calculation based entirely on static deformations (in respect of which a great many assumptions and approximations have to be made) would lead us to suppose.

CHOICE OF SYSTEM

Abroad a good many projects have been carried out in prestressed concrete. The three following systems have been used.

(a) The concrete is prestressed by means of post-tensioned cables.

(b) The slabs are prestressed between abutments by means of jacks, wedges, etc. without the incorporation of steel in the concrete.

(c) The concrete is prestressed by means of pre-tensioned wires, which are released after the concrete has hardened.

PRINCIPAL APPLICATIONS IN OTHER COUNTRIES

In the following, the principal foreign applications of the systems referred to under (a), (b) and (c) will be briefly discussed.

France

The first experimental sections with post-tensioned cables were constructed near Luzancy in 1945-1946. The slabs are approximately 24 and 20 m long and 6 m wide, with a thickness of 16 cm at the centre and 20 cm at the edges. The prestressing cables are spaced at intervals of 50 cm in both directions and are laid at an angle of 45° to the axis of the road. The cables are each composed of 10 wires of 5 mm diameter; they were tensioned to produce prestresses ranging from 17 to 21 kg/cm^2.

The slab at Esbly (Figure 1), which was constructed in 1949, is approximately 48 m long and 16 cm thick. (Like the above-mentioned slabs, the present slab is installed on an approach to a bridge.) In this case the slab contains an orthogonal system of cables, each consisting of 12 wires of 5 mm diameter, spaced at intervals of 1 m in both directions and laid at an angle of 45° to the axis of the road. The prestress is 16 kg/cm^2.

Despite poor sub-grade conditions, the slabs mentioned above have, so far, behaved satisfactorily.

One of the best known French experimental schemes is the runway at Orly (Figure 2), which was constructed in 1947. For this a different method of prestressing was used. This runway, 420 m long and 60 m wide, is composed of triangular slabs with a side length of 120 m at the edge of the runway and a thickness of 16 cm. Abutments are provided at each end of the runway. The joints between the slabs are at an angle of 45° with the longitudinal axis and are so constructed as to be virtually frictionless.

The prestress (33 kg/cm^2) is produced by means of Freyssinet cables, each composed of 30 wires of 5 mm diameter, spaced at intervals of 1 m and running at right-angles to the axis of the runway. The thrust exerted by the triangular slabs in the longitudinal direction is resisted by the above-mentioned abutments.

In 1952 an experimental prestressed concrete multiple slab, 300 m in length and 12 cm thick (Figure 3), was constructed on national highway N 83 (Bourg-Lyons) near Bourg-Servas. This slab was subsequently covered with an asphalt surfacing 4 cm thick and incorporated with the general asphalt surfacing of the adjacent stretches of road.

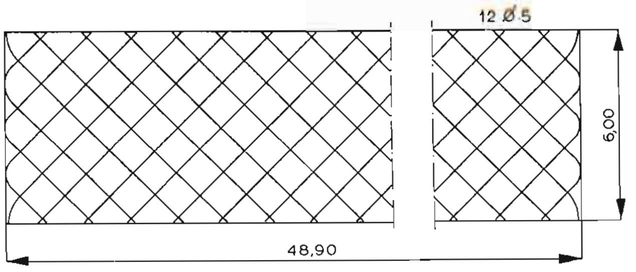


Figure 1. Diagram of the experimental prestressed slab at Esbly (diagonal arrangement of cables)

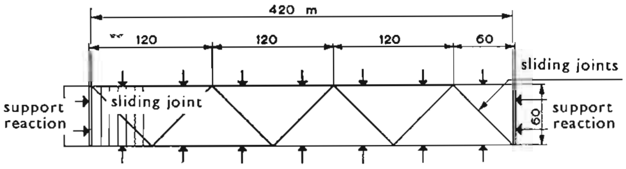


Figure 2. Diagram of the 420 m long runway at Orly (triangular slabs, transverse cables)

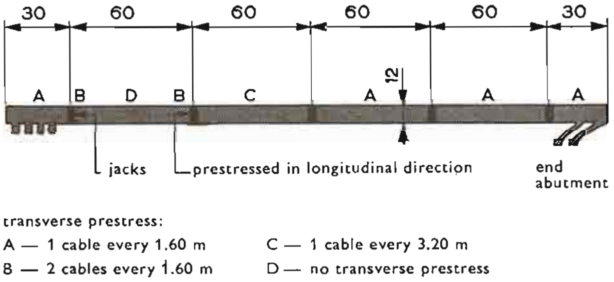


Figure 3. Diagram of prestressed concrete road at Bourg-Servas (jacks installed in joints)

At one end of the slab is an abutment (Figure 4) installed in disturbed soil. It consists of a horizontal slab under which are two circular-curved ribs placed in a vertical plane. At the other end of the slab the abutment consists of a concrete slab with locally thickened transverse ribs (Figure 5); this abutment is located at a spot where an existing road surfacing was demolished to make room for it. The stretch of road concerned is composed of four slabs, each 60 m in length, with an abutment slab 30 m long at each end. The longitudinal prestress (80 kg/cm^2) is produced by means of flat jacks placed in the joints (Figure 6) and subsequently embedded in concrete. In addition, three of the 60 m long slabs have been provided with a transverse prestress of 6 or 12 kg/cm^2 by means of post-tensioned cables. This system of construction offers the following advantages:

(a) the length of the slabs can be kept within the desired limits;

(b) no steel is required to produce the longitudinal prestress;

(c) as soon as the concrete has hardened, it can be given a small prestress to prevent cracking;

(d) the distance between end abutments can be chosen to suit the requirements of the job, so as to fit in with the location of bridges and other civil engineering works;

(e) the effect of friction in the prestressing operation becomes less important.

A similar type of construction is presented by the runway constructed at Maison-Blanche Airport, Algiers, in 1954 (Figure 7), where the longitudinal prestress (70 kg/cm^2) is likewise produced by flat jacks.

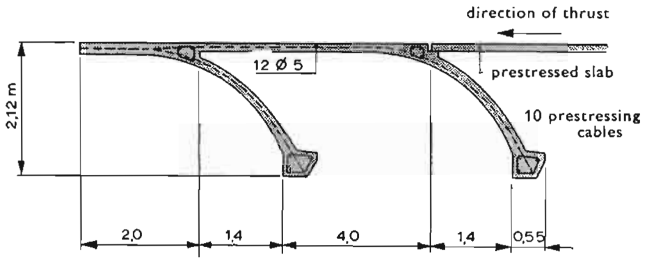


Figure 4. Diagram of one of the abutments at Bourg-Servas

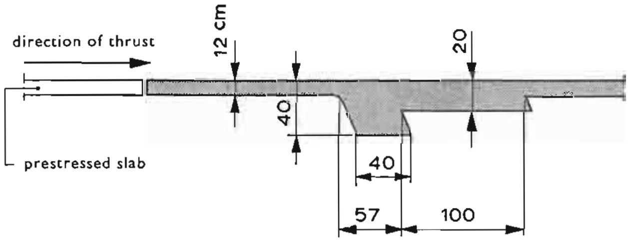


Figure 5. Diagram of the other abutment at Bourg-Servas (installed on site of demolished old road surfacing)

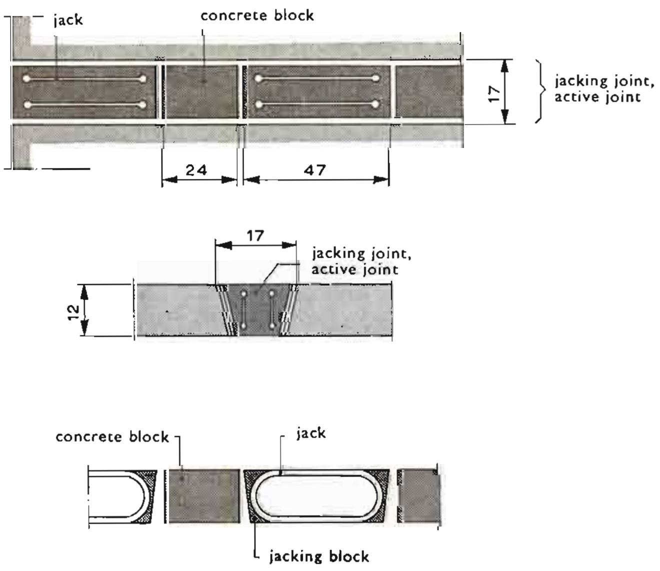


Figure 6. Sections through jacking joint with jacks (Bourg-Servas)

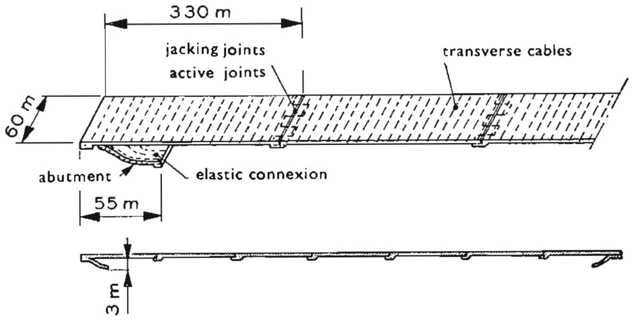


Figure 7. Diagram of the prestressed concrete runway at Maison Blanche (jacks in joints, transverse cables)

This scheme comprises a runway (2,430 m long, 60 m wide) with, to one side of it, a taxiway with a length of 2,050 m and a width of 25 m. The slabs are 330 m long and 18 cm thick; the runway and taxiway have been designed for a load of 135 tons. The transverse prestress (18 kg/cm^2) is produced by cables, at intervals of 1.33 m, anchored at the edges. Each transverse cable is composed of 12 wires of 7 mm diameter. As appears from Figure 7, the abutments employed in this case have a shape which is the opposite of that adopted for the Bourg-Servas abutments (Figure 4). In the present arrangement the abutments have been so constructed as to be loaded in tension; an elastic connexion is provided, which serves to take up part of the slab movement. Because of this arrangement, the abutment is never loaded to very high values. In this respect it presents a contrast to the foregoing compression-type abutment which, as a result of the temperature effects (which cause expansion and contraction of the slab), is continually subject to varying loads.

The jacking joints ("active" joints) are spaced approximately 300 m apart. Because of the great distance between these joints, temporary intermediate joints at intervals of 100 m were provided, in which additional jacks could be installed for producing the due amount of prestress in the slab. The temporary joints were subsequently filled with concrete.

To obviate the danger of buckling, the jacking joints are provided with comb-type devices with interlocking teeth which prevent the slab ends from lifting (Figures 7 and 8).

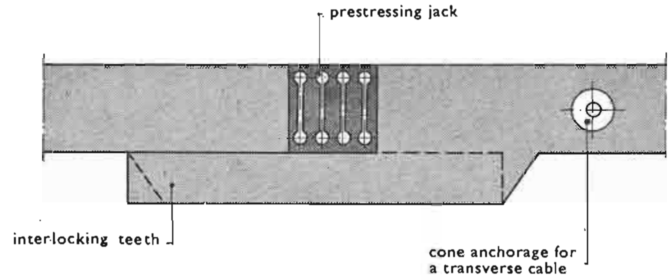


Figure 8. Jacking joint with interlocking "teeth" under the slabs (precaution against buckling) (Maison-Blanche)

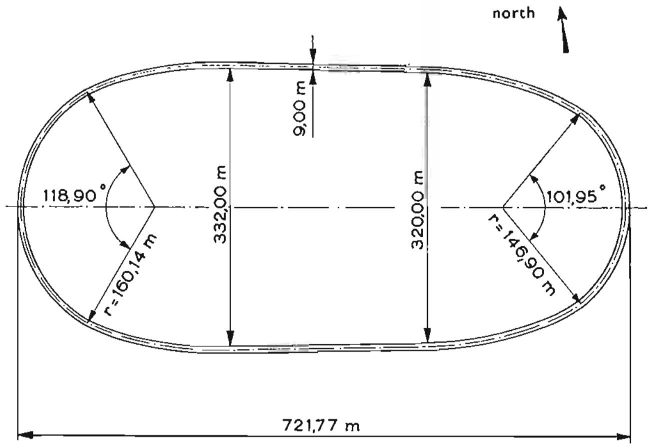
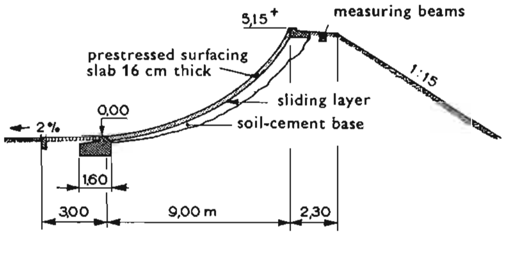
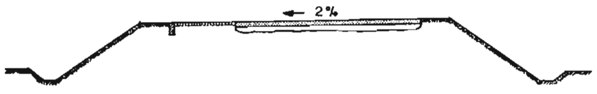


Figure 9. Diagram of the motor-car testing track of the Volkswagen works, Wolfsburg



section through east curve



section through straight intermediate portion

Figure 10. Sections through Volkswagen testing track

Further particulars regarding the schemes that have been described, as well as some schemes not described in the foregoing, are given in Table 1.

Germany

In Germany, too, various projects — roads as well as airfield runways — have been carried out in prestressed concrete with varying degrees of success. In practically every case post-tensioned cables have been employed.

Data concerning the principal projects are given in Table 2.

In addition, the following points call for mention.

The test track of the Volkswagen motor-car works at Wolfsburg is very well known. It is composed of four slabs laid on a soil-cement base; it comprises two straight portions, each 230 m long, and two curved portions with lengths of 650 and 690 m (Figure 9); in addition, transition curves have been provided. A very considerable superelevation (56° at the edge) has been employed in the curves (Figure 10). The slab is 16 cm thick. Prestress is provided both in the transverse and in the longitudinal direction. In the joints concrete wedges have been installed, which are not used for producing the prestress but for bridging the joints; they also permit a certain amount of movement.

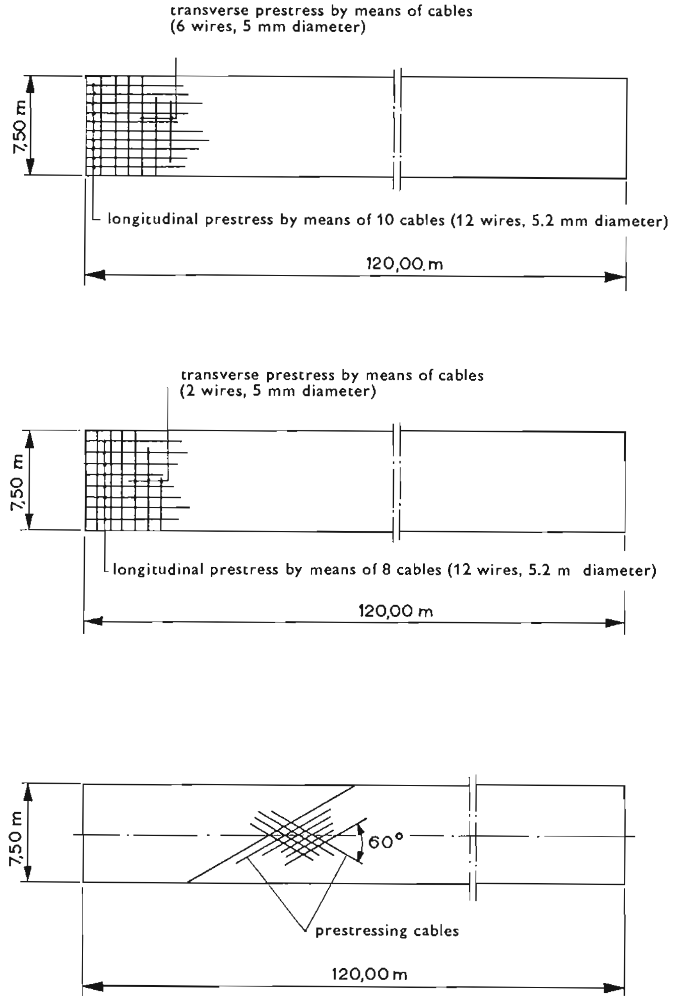


Figure 11. Diagrams of the prestressed concrete slabs at Mergelstetten (from top to bottom: slab I, slab II, slab III)

Table 1: French applications of prestressed concrete to roads and airfields

place	year of construction	total length (m)	number of slabs	dimensions of slabs		thickness (cm)	method of prestressing	longitudinal prestress (kg/cm^2)	transverse prestress (kg/cm^2)
				length (m)	width (m)				
Orly (experimental slab)	1946	14	1	14	12.50	16	cables (12 wires, 5 mm diameter) at 15 cm centres in both directions	between 0 and 40	between 0 and 80
Luzancy (road)	1946	24.90	1	24.90	6.00	16 (at the centre) 20 (at the edge)	cables (10 wires, 5 mm diameter) at 50 cm centres in both directions, at 45° to centre-line of road	17-21	17-21
Esbly (road)	1949	48.90	1	48.90	5.80	15	cables (12 wires, 5 mm diameter) at 1 m centres in both directions, at 45° to centre-line of road	16	16
Orly (runway)	1947	420	triangular slabs with sides of 120 m		60	16	two fixed abutments; cables (30 wires, 5 mm diameter) at 1 m centres, at right-angles to centre-line	33	33
Orly (runway)	1953	430	4	107	25	18	two elastic abutments; longitudinal prestress with flat jacks; transverse prestress with cables (12 wires, 9 mm diameter) at 1.33 m centres	70	18

Table 1: French applications of prestressed concrete to roads and airfields (Contd)

place	year of construction	total length (m)	number of slabs	dimensions of slabs		thickness (cm)	method of prestressing	longitudinal prestress (kg/cm²)	transverse prestress (kg/cm²)
				length (m)	width (m)				
Bourg-Servas (road)	1952	300	4 2	60 30 (abutment)	6.8	12	one fixed and one elastic abutment; longitudinal prestress with flat jacks; transverse prestress: over a distance of 180 m by means of cables (12 wires, 5 mm diameter) at 1.6 m centres; over a distance of 60 m by means of cables (12 wires, 5 mm diameter) at 3.2 m centres; over remaining distance of 60 m only a light mild-steel reinforcement	80	12 6 0
Maison-Blanche (runway)	1954	2430	7	ca. 330	60	18	two elastic abutments; longitudinal prestress with flat jacks; transverse prestress with cables (12 wires, 7 mm diameter) at 1.33 m centres	70	18

Table 2: German applications of prestressed concrete to roads and airfields

place	year of construction	purpose of the structure	length (m)	spacing of joints (m)	thickness (cm)	longitudinal prestress			transverse prestress		
						direction	kg/cm^2	quantity of steel (kg/m^2)	type	kg/cm^2	quantity of steel (kg/m^2)
Mergelstetten I	1953/'54	National Highway	120	120	15	longitudinal direction	20	2.7	transverse direction	10	1.3
Mergelstetten II	1953/'54	National Highway	120	120	15	longitudinal direction	16	2.2	transverse direction	3	0.5
Mergelstetten III	1953/'54	National Highway	120	120	15	diagonal direction	26	5.8	transverse direction	9	
Memmingen	1956	runway	300	100	14 (at the centre) 20 (at the edge)	longitudinal direction	15	3.0	diagonally	14	2.7
Wolfsburg (Volkswagen testing track)	1957	testing track for Volkswagen cars	1800	30 to 70	16	longitudinal direction	30	5.0	transverse direction	10	1.5
Diepholz		runway	1275	206	14	longitudinal direction	30	5.0	transverse direction	10	

Table 2: German applications of prestressed concrete to roads and airfields (Contd)

place	year of construction	purpose of the structure	length (m)	spacing of joints (m)	thickness (cm)	longitudinal prestress			transverse prestress		
						direction	kg/cm^2	quantity of steel (kg/m^2)	type	kg/cm^2	quantity of steel (kg/m^2)
Speyer	1954	experimental section	1700	40 to 70	20	longitudinal direction	average 15 (6 at top, 24 at bottom)	2.14	none		
Dietersheim	1958	road	900	150	16	longitudinal direction	46 at the end, 32 at the centre	8	transverse direction	15	
Niedersachsen	1958	airfield	7260 m^2 21600 m^2	242 150	9 14	longitudinal direction pre-tensioned	15 16	2 3	transverse direction post-tensioned		

Buckling of the slabs is prevented by their own weight. The measured value for the coefficient of friction is 0.3, this low value being obtained by the application of a "lubricating layer" to the base. The longitudinal cables are continuous over the entire length of the track; the transverse cables are at intervals of 1.50 m. The cables in both directions were tensioned to 35 tons. Up to the present, the track has behaved very satisfactorily.

As a result of faulty execution, a number of cracks occurred at an early stage on the experimental sections at Mergelstetten (Figure 11). A good many transverse and longitudinal cracks have already had to be repaired. In this instance the arrangement which consisted in concentrating the cables at the edges of the slab, proved unsuccessful.

The experimental stretch of road at Speyer has been tolerably satisfactory, although cracks have occurred. The joints were found to be incorrectly spaced, so that the road sustained serious damage in places, especially where additional concrete had subsequently been placed.

On account of the prestressing system used in the construction of the experimental section at Dietersheim (Held & Francke system) it was necessary to provide jacking gaps 1.70 m in width. Subsequently a 1.20 m wide portion of each gap was concreted and likewise prestressed. The remaining 0.50 m was filled with a very expensive American joint-filling unit of composite steel and neoprene (synthetic rubber) construction (Figure 12). The experience with this section of road has, so far, been very favourable: no cracks have as yet occurred.

On the basis of the experience gained in Germany, the Forschungsgesellschaft für das Strassenwesen (the German Road Research Association) has given some directives for the construction of prestressed concrete roads, particularly with regard to the stresses that are liable to occur and for which provision must be made:

approximate flexural tensile stress due to unequal heating:	35 kg/cm^2
approximate flexural tensile stress due to live loads:	20 kg/cm^2
tensile stress due to friction (sub-grade restraint) for joint spacing of 150 m:	10 - 15 kg/cm^2
Total:	70 kg/cm^2

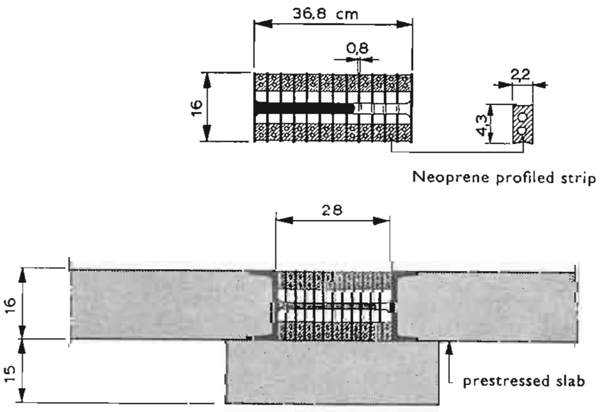


Figure 12. Neoprene/steel joint filling (Dietersheim)

The minimum flexural strength of the concrete is assumed to be approximately 50 kg/cm^2. With a longitudinal prestress of 30 kg/cm^2 the factor of safety against cracking is 1.25; with a longitudinal prestress of 40 kg/cm^2 it is 1.50. In view of the above considerations a longitudinal prestress of at least 30 kg/cm^2 is recommended.

The minimum slab thickness recommended for trunk roads (Federal highways) is 15 cm, and the recommended minimum length of slab is 150 m. Steel stresses up to 65 - 70 % of the ultimate tensile strength are permitted.

If the prestress is produced by means of jacks* or wedges it should be at least 70 kg/cm^2; the minimum slab thickness should be 16 cm and the minimum length of slab 150 m. The stressing should be carried out at the lowest possible temperature.

In all cases it is essential to apply some prestress as soon as possible, in order to prevent cracking due to shrinkage.

The Forschungsgesellschaft is of the opinion that, although measurements indicate the contrary, the base employed should not be any weaker than that provided under an ordinary concrete road.

With the exception of the Niedersachsen airfield, the Dywidag prestressing system has been used in all the prestressed airfield pavements that have been constructed in Germany.

At Niedersachsen a large number of cracks have occurred, especially over the cables; these cracks are attributable to faulty execution of the job. At this airfield observations led to the conclusion that the tensile stress in the concrete is independent of the absolute value of the drop in temperature. The full sub-grade restraint occurs already after a drop of 3 - 4°C. It was necessary to provide part of the prestress within 24 h of concreting, i.e. by the time the heat of hydration had been dissipated. It was furthermore necessary to precast the anchorage blocks.

The airfields at Memmingen (Figure 13), Wunsdorf, Diepholz, Northolz, Hopsten, and Cologne-Wahn have, up to the present, all remained in very fair condition.

The general conclusion in Germany is that, although the prestressed pavements initially gave rise to many problems, the experimental stage can now be said to have been passed.

Britain

The British experimental sections are, generally speaking, similar to those constructed in France; post-tensioned cables have been used in all of

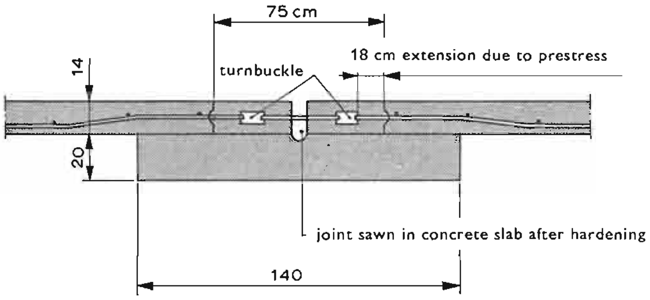


Figure 13. Diagram of joint construction at Memmingen

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* This refers to "gap-jacking", e.g. by means of flat jacks. (Translator's note)

them. Some of the jobs are mentioned in Table 3.

In general terms it can be said that in Britain the experimental stage has not yet been passed. Besides, practically all the prestressed pavements have been installed in places where there is little or no traffic, so that it is difficult to gain experience in this respect. At South Benfleet a slab thickness of only 10 cm was employed; the prestress at first was found to be inadequate, with the result that cracks developed, but these disappeared after re-tensioning had been carried out.

Numerous cracks have occurred on other experimental sections as well (Port Talbot, Ringwood), and it has not always been possible to ascertain their cause.

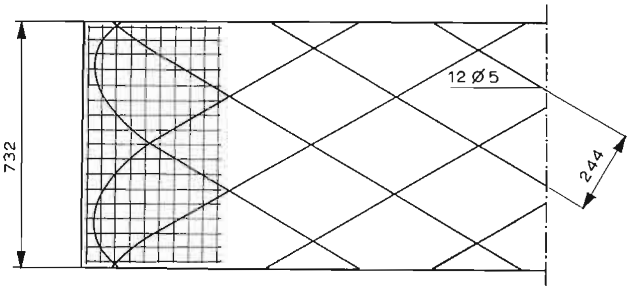


Figure 14. Diagram of prestressed slabs at Crawley (diagonal cable arrangement)

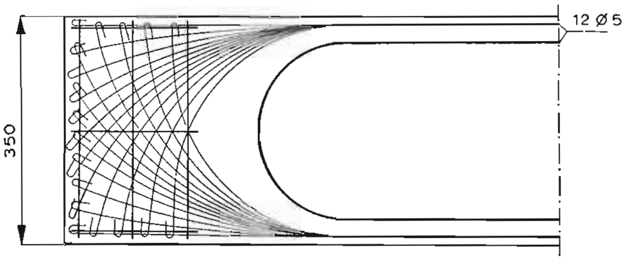


Figure 15. Diagram of the prestressed slab at Wexham Springs (perimetral cables)

Switzerland

In 1955 two experimental sections of road were constructed near Naz, one section being prestressed by means of cables and the other by means of jacks. The section provided with cables (Freyssinet system) is 334 m in length, comprising four slabs of 70 m and two of 27 m. The slabs are 2.50 m wide and 12 cm thick. The cables are continuous across the joints. At the edges of the slabs the cables have been placed closer together. Jacks and wedges were installed in the joints so that an even higher prestress was obtained over the slab as a whole. The second experimental section near Naz is 500 m long (comprising six slabs of 70 m and two of 40 m) and is provided with two fixed abutments. The prestress, produced by gap-jacking, is 60 kg/cm^2; in addition, a transverse prestress of 4 kg/cm has been provided by means of single prestressing wires 1 m apart, each with a diameter of 7 mm.

The experimental section at Möriken-Brunegg (1956) is 505 m in length (including a curved portion 145 m long), 5.5 m in width, and 12 cm thick. It is provided with a fixed abutment and an abutment with ribs embedded in an old existing road. The prestress of 70 kg/cm^2 is produced by means of concrete wedges with lateral surfaces (steel plates lubricated with graphite) specially constructed to minimize friction.

Table 3: British applications of prestressed concrete to roads and airfields

place	year	length (m)	width (m)	thickness (cm)	longitudinal prestress (kg/cm^2)	transverse prestress (kg/cm^2)	direction of prestressing	condition
Crawley (Fig. 14)	1950	120	6	15	15	1.6	diagonally	good
Wexham Springs (Fig. 15)	1951	33	3	15	13	1.6	length	good
Buckinghamshire	1951	39	3	15	9	1.6		
		60	3.5	15	18	1.6		
London Airport	1951	100	3.6	16	40	40	triangular	
John Laing's Ltd.	1951	15 x 50 = 750	3.6	25	28	40	length	
Basildon Essex	1952	2 x 50 = 100	4	15	21	40		
Woolwich	1952	4 x 60 = 240	5	15	20	40		
Port Talbot	1954	1000	5-7	15	18	2	diagonally	
South Wales		450	6	15	20	various	longitudinally and diagonally with jacks	
South Benfleet	1954	100	6	10	38	3.5	jacks in longitudinal direction, cables in transverse direction	

Italy

An experimental section of road has been constructed on the Via Amilia near Cesena (Figure 16). It is 500 m long, 7.50 m wide, and 20 cm thick, situated between two abutments (Figure 17). The joints are spaced 50 m apart, but at the centre of the section there is one length of slab 100 m long. The longitudinal prestress is provided by 5 mm diameter pre-tensioned wires. In addition, there is a transverse prestress, produced by post-tensioned cables. The prestress both in the longitudinal and in the transverse direction is 25 kg/cm^2. A bituminous layer has been provided under the slab in order to reduce friction.

Austria

The prestressed portion of the runway of Vienna Airport is 1,000 m long, 45 m wide, and 15 cm thick (Figure 18). The transverse prestress (10 kg/cm^2) is produced by means of post-tensioned cables. In the longitudinal direction pre-tensioned cables have been provided, which were initially anchored to intermediate abutments. When the concrete had attained a certain strength, the pre-tensioned tendons were gradually released — in order to prevent shrinkage cracks from occurring — and in this way a longitudinal prestress of 16.5 kg/cm^2 was obtained. The force in the wires was then transferred to the end abutment.

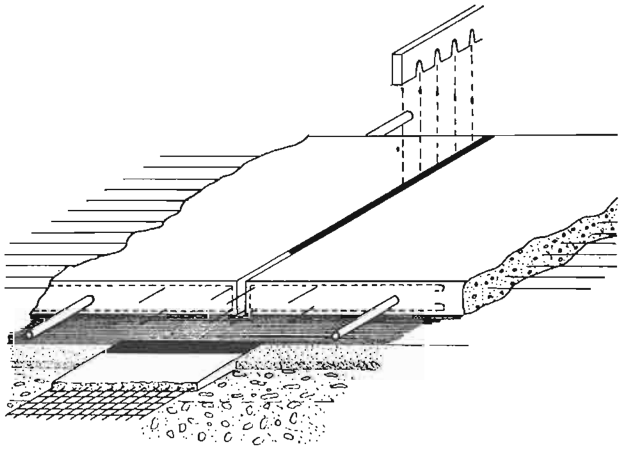


Figure 16. Sections through prestressed concrete slab at Cesena

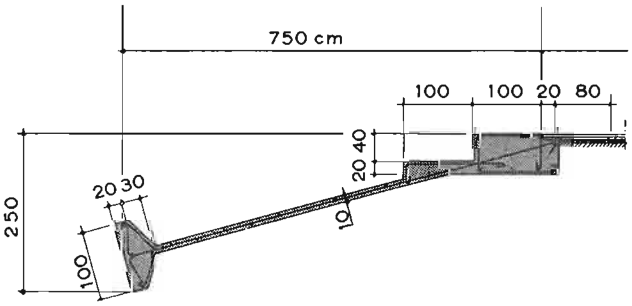


Figure 17. Abutment at Cesena

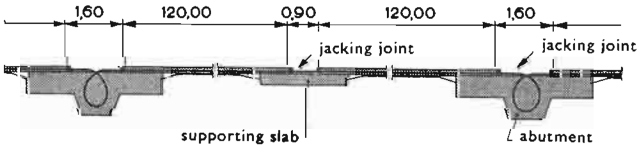


Figure 18. Diagram of prestressed concrete runway, Vienna

Jacking joints 1.50 m in width were provided at intervals of 250 m. When the creep of the concrete had ceased, jacks were installed in these joints for the purpose of producing the rest of the prestress required. These jacks were subsequently removed and the joints filled with concrete. The thrust due to changes of temperature is resisted by end abutments extending into the ground to a depth of 2 m.

Near Salzburg an experimental section 800 m in length (100 m + 3 x 200 m + 100 m), 7.50 m in width, and 16 cm thick has been constructed on the motorway (Figure 19). In one portion the prestress is produced by means of "folding wedges" (Rella-BBRV system), which are prestressed laterally and introduce a prestress of 80 kg/cm^2 into the road slab which is resisted by abutments. In consequence of variations of temperature, compressive stresses of up to 165 kg/cm^2 are liable to occur. On the east carriageway an experimental section with a length of 780 m (65 m + 5 x 130 m + 65 m), a width of 7.50 m, and a thickness of 20 cm has been constructed; the prestressing system used is identical with that of Professor Leonhardt (Figure 20). The prestress is 70 kg/cm^2, but stresses of 115 - 130 kg/cm^2 may arise in consequence of variations of temperature.

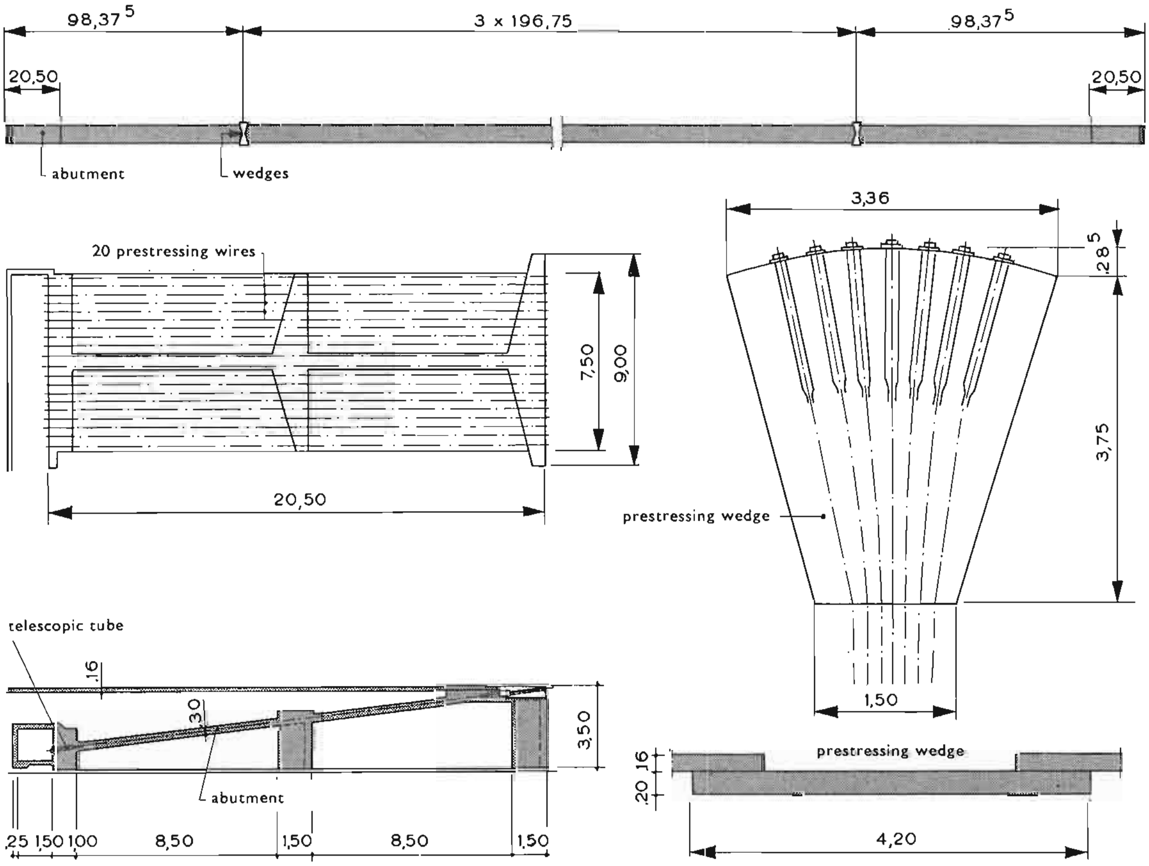


Figure 19. Diagram of a prestressed concrete experimental section with abutments, etc. on the Salzburg motorway

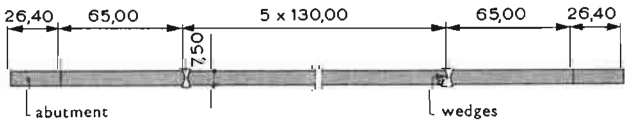


Figure 20. Diagram of the other prestressed concrete experimental section with abutments, etc. at Salzburg

Belgium

A few schemes have been carried out in Belgium, but operations are still in the experimental stage in that country.

First, there is a taxiway for aircraft at Melsbroek airport. It is 350 m long, 23 m wide, and 10 cm thick. In the longitudinal direction the concrete slab is subdivided into strips 1.24 m wide (Figure 21) which are composed of precast prestressed slabs with oblique ends. At the edges of the taxiway a special strengthening strip has been installed under the slabs. Post-tensioned wires, placed transversely, produce the transverse prestress and (in consequence of the oblique slab ends) also the longitudinal prestress. The transverse prestress is 36 kg/cm^2; this, in turn, produces a longitudinal prestress of 30 kg/cm^2.

As this longitudinal prestress adversely affects the prestress of the precast slabs in the central part of the taxiway, these slabs were (at the time of their manufacture) given a prestress of 30 + 36 = 66 kg/cm^2, so that the value of the prestress in that region will never descend below 36 kg/cm^2. The prestress in the precast slabs was produced by pre-tensioned 5 mm diameter wires of which there were 32, 38, or 44 wires in a slab, according to its location. The length of the slabs (12 m) was determined by considerations of handling.

A fairly recent experimental section is that which has been constructed on the Zwartberg-Meeuwen road. It is over $3\frac{1}{2}$ km in length and includes a curved portion 340 m long with a radius of 1,000 m, and a curved portion 50 m long with a radius of 500 m. Further details are given in Table 4. The slabs forming this 7 m wide stretch of prestressed road are unreinforced in the longitudinal direction, the prestress being produced by flat jacks. A sort of tunnel has been constructed under each jacking joint so as to permit subsequent adjustment of the jacks without causing inconvenience to the traffic using the road (Figures 22 and 23). The forces are taken up by two abutments

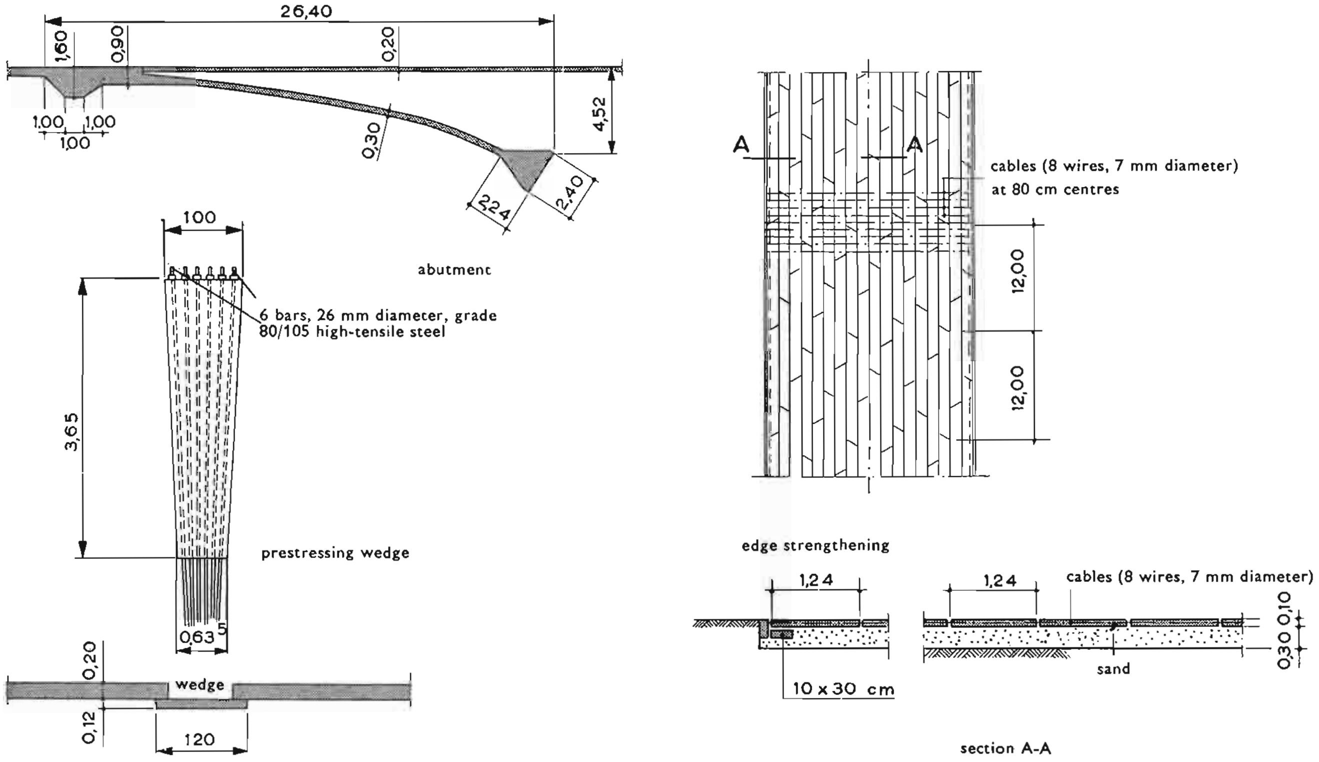
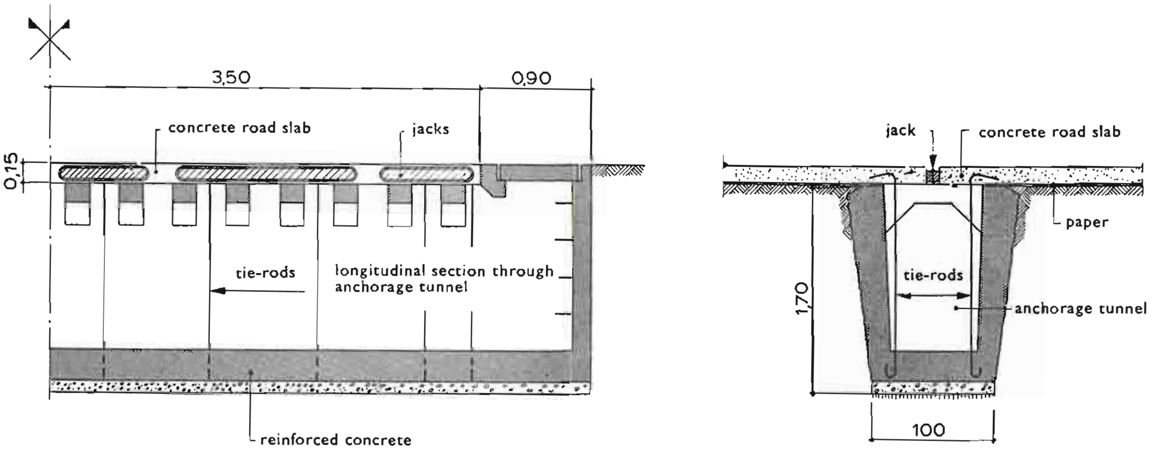


Figure 21. Diagram of taxiway at Melsbroek airport

(Figures 24 and 25), which are of different design. Lateral abutments have been provided locally in the curves (Figure 26) in order to prevent lateral movements of the slabs. Experience to date has been only moderately favourable; numerous cracks have already developed.

Table 4: Lengths and additional data of experimental sections on Zwartberg-Meeuwen road (Belgium)

no.	km	thickness (cm)	longitudinal prestress at + 15° C (kg/cm^2)	transverse prestress or transverse reinforcement
1	4,898	10	20	none
2	5,033	10	20	prestress 15 kg/cm^2
3	5,168	10	20	prestress 7.5 kg/cm^2
4	5,303	10	20	mild steel, 5 bars, 6 mm diameter, per lineal metre
5	5,438	10	30	mild steel, 5 bars, 6 mm diameter, per lineal metre
6	5,573	10	30	prestress 15 kg/cm^2
7	5,708	10	30	prestress 7.5 kg/cm^2
8	5,843	10	30	none
9	5,978	10	40	none
10	6,113	10	40	prestress 15 kg/cm^2
11	6,248	10	40	prestress 7.5 kg/cm^2
12	6,383	10	40	mild steel, 5 bars, 6 mm diameter, per lineal metre
13	6,518	12	30	mild steel, 5 bars, 6 mm diameter, per lineal metre
14	6,653	12	30	prestress 7.5 kg/cm^2
15	6,788	12	40	prestress 7.5 kg/cm^2
16	6,923	12	40	mild steel, 5 bars, 6 mm diameter, per lineal metre
17	7,058	12	30	prestress 15 kg/cm^2
18	7,193	12	30	none
19	7,328	12	20	none
20	7,463	12	20	prestress 15 kg/cm^2
21	7,598	12	20	prestress 7.5 kg/cm^2
22	7,733	12	20	mild steel, 5 bars, 6 mm diameter, per lineal metre
23	7,868	8	40	mild steel, 5 bars, 6 mm diameter, per lineal metre
24	8,003	8	40	prestress 15 kg/cm^2
25	8,138	8	30	prestress 15 kg/cm^2
26	8,273 8,408	8	30	mild steel, 5 bars, 6 mm diameter, per lineal metre



Figures 22-23. Sections at jacking joint in experimental section Zwartberg-Meeuwen

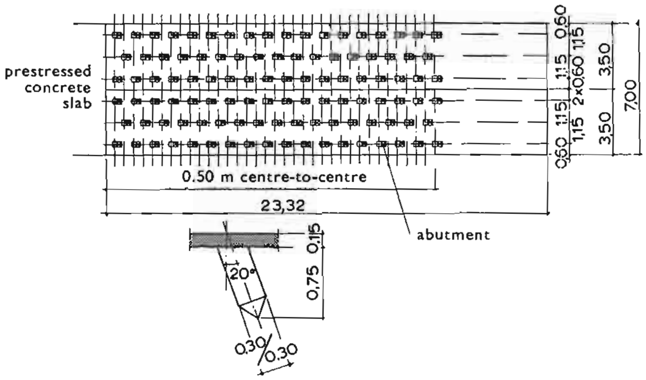


Figure 24. Abutment in experimental section Zwartberg-Meeuwen

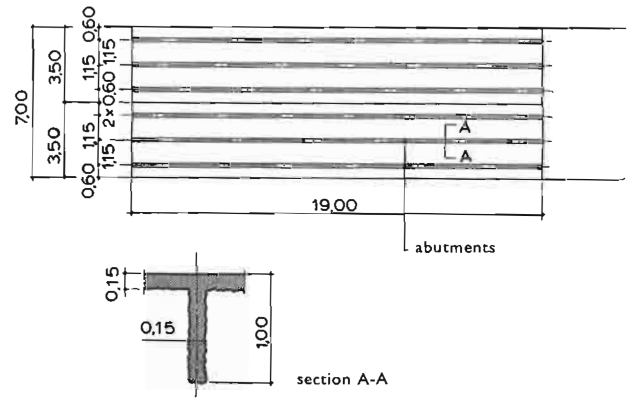


Figure 25. Abutment in experimental section Zwartberg-Meeuwen

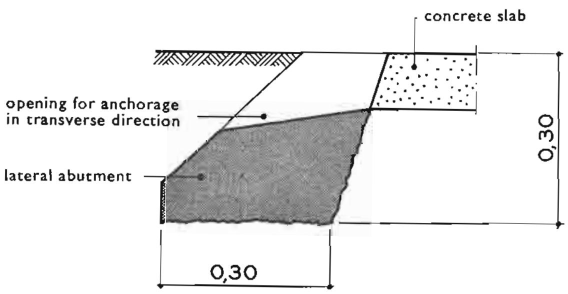


Figure 26. Lateral abutment in experimental section Zwartberg-Meeuwen

United States and Australia

In the United States, too, a few prestressed concrete experimental sections have been constructed, including one on a motorway near Pittsburgh consisting of slabs 120 m long, 3.60 m wide, and 12 cm thick, with a longitudinal prestress of 31.5 kg/cm^2 produced by post-tensioned cables (4 wires of 11 mm diameter per cable).

From Australia it is reported that an experimental length of pavement with a concrete thickness of only 6 cm is under construction.

Conclusions

What conclusions can be drawn from all this experience gained abroad?

(a) Except in France, considerable initial difficulties were encountered in all countries.

(b) These difficulties are primarily attributable to faulty design, but also in some instances (e.g. in Germany) to major faults in the execution of the jobs. Things subsequently became better.

(c) The curing of the concrete during the first 24 h was inadequate in a good many cases.

(d) Cracks often occurred within a short time (and quite frequently within 24 h of concreting) both on sections with post-tensioned cables and on those prestressed by means of jacks or wedges. With post-tensioned cables the cracks usually developed at the cables.

(e) There is a very considerable variety of methods, types of abutment, values of prestress employed, etc.

(f) The savings effected are small as compared with ordinary concrete roads and are due only to the difference in thickness of the slabs (about 10 - 15 cm). Against this must be set the much higher cost due to the prestressing.

(g) With airfield pavements the savings are much greater (corresponding to 30 - 45 cm thickness) and it is possible to produce a design which is competitive in relation to ordinary concrete or bituminous construction. This has, inter alia, been demonstrated in Germany and France in cases where the contractors invited to tender were free to submit variant designs.

(h) The most recent schemes carried into effect, especially in Germany and France, have proved very satisfactory.

From the Dutch point of view the following general conclusion can be drawn: all the foreign schemes are far too expensive, this being due to the design (involving post-tensioned cables, jacks, or wedges) and the complex and numerous manipulations and operations resulting therefrom.

The only sort of design that has any chance of being employed with success in Holland is one that is simple in its conception and which can be applied in normal road-building practice expeditiously and without involving any delays.

With this in mind I consider that Ir Obertop, of the National Highways Laboratory (Rijkswegenlaboratorium), has made a contribution of great value by drafting the first directives to that end. (These have been discussed in his article in "Wegen"(1).)

Designs based on these indications have been carried into effect on National Highway No. 4A. They will be dealt with in the following part of this article.

EXPERIMENTAL SECTIONS AT LEIDSCHENDAM AND LEIDERDORP

The design

a. Principal dimensions

The slabs of the first three sections are 100 m in length, this value having been chosen as a result of studying the foreign designs. They are 7.25 m wide, with a longitudinal joint at the centre, and 12 cm thick. It appeared from the results of tests conducted abroad and from the calculations given below that this thickness would be adequate. In addition, this thickness satisfied the requirement of economy with which the design had to comply, namely, the cost of the prestress to be provided had to be roughly equal to the cost of a concrete slab with a thickness of 23 - 12 = 11 cm.

b. Base

To permit a comparison of various possible solutions with one another, two of the slabs had a base of compacted sand and one slab had a base of cement-stabilized sand 15 cm thick.

c. Stresses due to loading

A concrete slab of finite size may be loaded at the corners, at the edges, or at some distance from the corners and edges, i.e. in the central area.

The stresses associated with these different loading conditions have to be calculated. It has been found that the stresses at the corners and edges are most frequently the decisive ones. Calculation of the stresses occurring under loads applied to the central area of the slab is therefore omitted.

Corner loading

The many American investigations and tests that have been carried out have shown that Westergaard's formulae, as modified by Pickett, provide the best indications of the actual state of affairs.

In the present design no load transfer from one slab to the other is able to occur at the transverse joints. The following formula should therefore be employed in this case:

$$\sigma_{tensile} = \frac{4.25\ s\ P}{h^2}\left(1 - \frac{\sqrt{a/l}}{0.925 + 0.22\ a/l}\right) = \text{tensile stress}$$

where

$$l = \sqrt[4]{\frac{E_b\ h^3}{12(1 - \mu^2)k}} = \text{relative rigidity factor}$$

h = slab thickness (= 12 cm)

P = load on the slab (= 4,000 kg)

s = impact factor (= 1.2)

E_b = modulus of elasticity of concrete (= 350,000 kg/cm^2)

μ = Poisson's ratio for concrete (= 0.15)

k = modulus of sub-grade reaction (sand-cement stabilized base) (= 25 kg/cm^3)

a = radius of loaded area

Determination of l :

$$l = \sqrt[4]{\frac{350{,}000 \times 12^3}{12(1 - 0.15^2) 25}} = 37.8 \text{ cm}$$

The magnitude of a may be determined by means of the following formula given by the tyre-manufacturing industry:

$$a = 10.0 + 2.7\,P - 0.1\,P^2 = 10.0 + 2.7 \times 4 - 0.1 \times 16 = 19.2 \text{ cm}$$

(where P = 4,000 kg = 4 metric tons*)

Hence we obtain (for the slab on a cement-stabilized sand base):

$$\sigma = \frac{4.25 \times 1.2 \times 4{,}000}{12^2} \left(1 - \frac{\sqrt{19.2/37.8}}{0.925 + 0.22 \times 19.2/37.8}\right)$$

$$= 43.5 \text{ kg/cm}^2$$

For the experimental slabs constructed on a compacted sand base: $k = 10$ kg/cm^3 and $l = 47.6$; therefore $\sigma = 52.5$ kg/cm^2.

Edge loading

The formula developed by Teller and Sutherland has been found to provide the best method of calculating the stresses due to loading applied to the edge of the slab. The formula is:

$$\sigma = 0.57185 \frac{P}{h^2} (4 \log_{10} l/b + \log_{10} b)$$

where $b = \sqrt{1.6\,a^2 + h^2} - 0.675\,h$ if $a < 1.724\,h$, and $b = a$ if $a > 1.724\,h$.

In the present case (for the slab on a cement-stabilized sand base):

$a = 19.2$ and $1.724\,h = 20.7$; hence:

$$b = \sqrt{1.6 \times 19.2^2 + 12^2} - 0.675 \times 12 = 18.85$$

therefore:

$$= 0.57185 \times \frac{4{,}000}{12^2} (4 \log_{10} 37.8/18.85 + \log_{10} 18.85)$$

$$= 47.5 \text{ kg/cm}^2$$

---

* In this translation "ton" denotes the metric ton = 1,000 kg = 2,205 lb. (Translator's note)

For the experimental slabs constructed on a compacted sand base:

$$\sigma = 55.6 \text{ kg/cm}^2$$

For the design of the slab as a whole, the design has been based generally on the edge stress.

d. Stresses due to difference in temperature between top and bottom of slabs

In 1956 measurements of temperature were made on the concrete surfacing of National Highway No. 4A (Figure 27a). The maximum difference was 0.4° C per cm (total difference approximately 10° C over a slab thickness of 23 cm), it being assumed for simplicity that the variation of temperature is linear. The measurements carried out on the concrete surfacing of National Highway No. 12 in 1939 (Figure 27b) indicated, however, that a difference of 0.6° C per cm (approximately 10.5° C over 18 cm) is liable to occur. This last-mentioned result is in agreement with the results of German measurements.(2) Measurements carried out in Washington, U.S.A., likewise yielded a difference ranging from 0.4 to 0.7° C per cm.

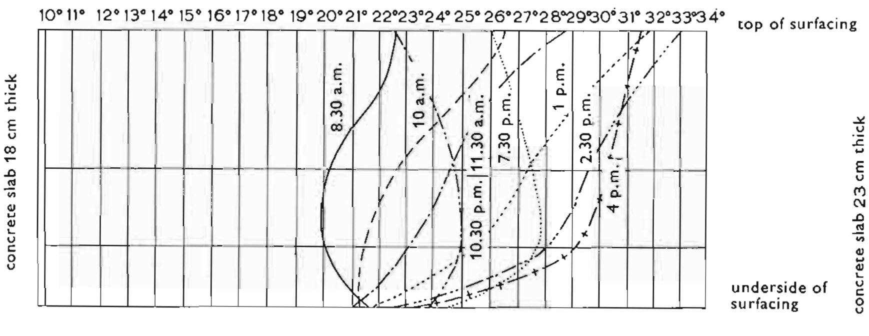
For the purposes of the present calculation the maximum anticipated temperature gradient has been taken as 0.6° C per cm and the variation within the slab is assumed to be linear. The tensile stress occurring in the 12 cm thick concrete slab is given by the formula:

$$\sigma = \frac{E_b \ \lambda \ \Delta T}{2(1 - \mu)} = \frac{350{,}000 \times 10^{-5} \times 0.6 \times 12}{2(1 - 0.15)} = 14.8 \text{ kg/cm}^2.$$

where ΔT denotes the difference in temperature between the top and bottom of the slab (= 12 x 0.6° C), and λ denotes the coefficient of expansion of the concrete (= 10^{-5} per ° C).

e. Length of slab; friction

The maximum permissible slab length for a prestressed concrete surfacing is determined by the sub-grade restraint (the friction between the slab and its base) and by the magnitude of the prestress. The friction must primarily be overcome by the prestress when this is applied to the sufficiently hardened concrete. Besides, if the slab undergoes sudden cooling, reduction of the prestress is liable to occur at the centre of the slab where there is



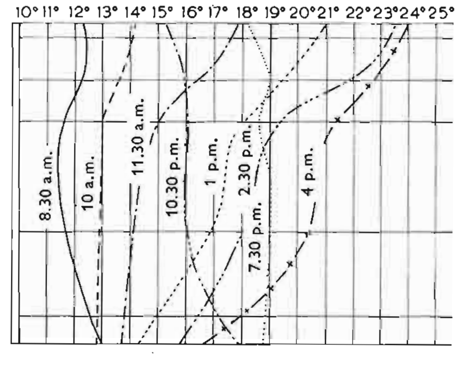


Figure 27a (left). Differences of temperature in a concrete road (National Highway No. 12) on a warm summer's day (20-21 August 1939)

Figure 27b (right). Ditto on National Highway No. 4A (31 August 1957) maximum force due to friction. Sufficient prestress must remain available should that occur. Conversely, if the slab undergoes a sudden rise in temperature, an increase in compressive stress, corresponding to the maximum force due to friction, is liable to occur. With the slab lengths adopted in the present case the concrete is perfectly capable of resisting these forces. If free expansion of the ends of the slabs is prevented, very high compressive stresses may indeed arise; this problem will, however, be further discussed with reference to the joints.

The experimental slabs are approximately 100 m long. The measured coefficient of friction was found to have a maximum value of 1.0. Hence the reduction in prestress due to friction of the slab on its base may have the maximum value:

$$\sigma = f\ \gamma\ l = 1.0 \times 2.4 \times 10^{-3} \times 50 \times 10^{2} = 12 \text{ kg/cm}^{2}$$

where γ is the bulk density of the concrete (in kg/cm^3) and l is half the length of the slab (in cm).

A thin layer of bitumen and a layer of paper were placed between the concrete surfacing and its base. In other countries efforts have been made to reduce the coefficient of friction by the use of various kinds of material between the surfacing and the base. In a few cases laboratory tests showed the coefficient of friction to be as low as 0.3 - 0.5. When these same materials were applied in actual practice, however, the coefficient was found to be approximately equal to unity.

In the present case the cement-stabilized sand base was sprayed with a bitumen emulsion, which was thereupon covered merely by a layer of paper. The value adopted for the coefficient of friction is f = 1. This coefficient is assumed to be constant over the entire area of the slab. This is not quite correct, however.(3,4) The effect of the error on the total stresses is small, however.

f. Most unfavourable stresses

The anticipated maximum stresses are therefore:

	Type of base	
	compacted sand	cement-stabilized sand
due to 4-ton wheel load (see point c)	56	47
due to restrained warping by temperature (see point d)	14	14
due to friction (sub-grade restraint) (see point e)	12	12
	82	73 kg/cm^2

These values may be compared with the flexural strength of the concrete, which is 60 kg/cm^2 (maximum).

g. Prestress provided

The prestress would thus have to be at least 22 kg/cm^2 for a base of compacted sand and 13 kg/cm^2 for a base of cement-stabilized sand. Allowance must be made for losses of prestress due to shrinkage, creep, and elastic shortening of the concrete and to relaxation of the steel.

According to data obtained by the Nederlandse Spanbeton Maatschappij (Netherlands Prestressed Concrete Company), Alphen, a value of 8 kg/cm^2 would have to be adopted for these losses.

On this assumption it would therefore be necessary to provide an initial prestress of at least 30 kg/cm^2, which means that the factor of safety of this design would be approximately equal to unity.

In his article(1) Ir Obertop had arrived at considerably lower stresses in his calculations. From considerations of structural safety, however, it was decided to adopt the above values of 22 and 13 kg/cm^2.

h. Prestressing steel provided

The slabs have been provided with 60 Demka high-tensile steel wires of 5 mm diameter (Q.V.B 130).

The total prestressing force in a cross-section of a strip of slab 3.625 m wide (corresponding to a prestress of 22 kg/cm^2) is 12 x 362.5 x 22 kg = 95,700 kg.

The wires are located centrally within the depth of the slab. To produce the eventual prestress of 22 kg/cm^2 in the concrete, the force per wire has to be 95,700/60 = 1,595 kg. This corresponds to a stress in the steel of $1{,}595/(\frac{1}{4} \times 3.14 \times 5^2) = 81.3$ kg/mm^2.

An initial stress in the steel of approximately 110 kg/mm^2 was adopted, corresponding to an initial prestress of about 30 kg/cm^2 in the concrete.

i. Transverse reinforcement

A transverse reinforcement consisting of 10 mm diameter bars at 30 cm centres has been provided. This reinforcement is likewise located centrally in the slab.

From a constructional point of view it would have been better to install the mild-steel reinforcement at the bottom of the slab, but this was not practicable on account of the limited thickness of the slab; it would have made the execution of the job very awkward.

The function of the transverse reinforcement was intended to be as follows:

1. to prevent any longitudinal cracks that might appear from spreading through the entire thickness of the slab;

2. to take up tensile stresses which will develop in this reinforcement if cracks of limited depth occur and which will alter the effective structural section of the slab.

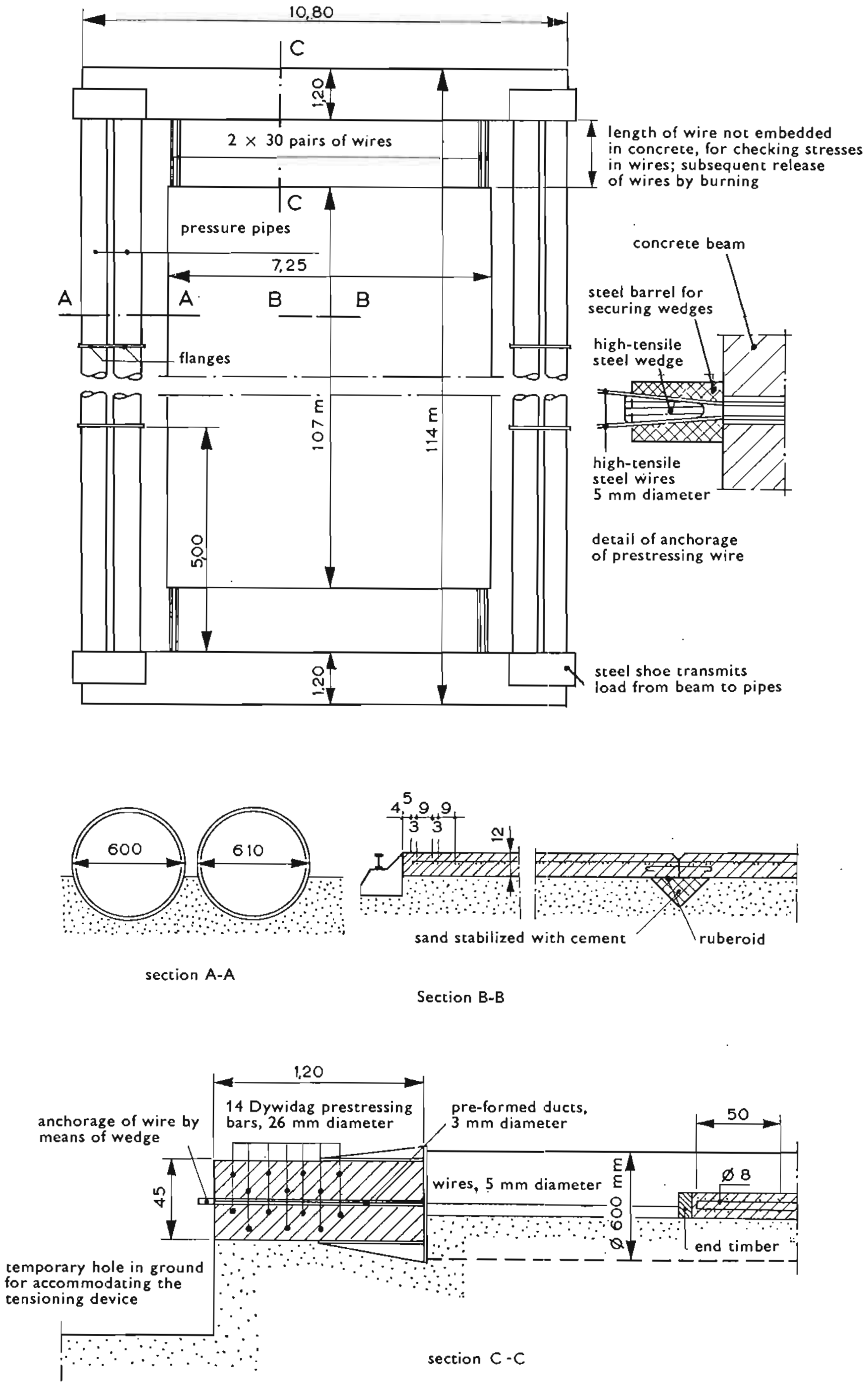


Figure 28. Diagram showing sections through prestressed experimental slab and pre-tensioning frame at Leidschendam

j. Local strengthening of the slab ends

On closer investigation it was found to be necessary to provide extra strengthening at the ends of the slabs (see Figure 28, section C-C, and Figure 30), for the following reasons.

1. After the concrete has hardened, the prestress is transferred to it by burning through the pre-tensioned wires. The forces thus suddenly applied to the concrete produce splitting stresses which the concrete is unable to withstand.

Figure 29. Concreting the experimental slab at Leidschendam

Figure 30. Detail of end of slab

2. The stress in the steel is transferred to the concrete by the bond of the concrete to the steel. The sudden force that is applied when the wire is burnt through, however, is liable to destroy the bond over a certain length. This will cause the wire to "slip" in the concrete, with the result that loss of stress will occur at the end. The concrete at the end of the slab will therefore be without prestress or the prestress will be reduced: it is necessary to compensate for this. For this reason the ends of the slabs have been strengthened by the provision of mild-steel mesh reinforcement both at the top and at the bottom. This reinforcement is provided over the entire width of the slab and extends a distance of 0.50 m in the longitudinal direction.

k. Paper

Just as in ordinary concrete roads, the prestressed slabs are separated from the base by a paper underlay.

l. Longitudinal joint (Figure 28, section B-B)

The two strips of surfacing each 3.625 m wide are separated from each other by a vertical longitudinal joint. Connexion across the joint is provided by 10 mm diameter tie-bars 50 cm in length and spaced at 30 cm centres.

Where the slabs are laid on sand, a strip of ruberoid 30 cm wide is placed under the longitudinal joint and overlying a strip of sand-cement (sand stabilized with cement).

EXECUTION OF THE WORK

Applying the prestress

As it was possible that various unexpected difficulties might crop up during the execution of the work, it was decided to construct the first slab on a branch road near the viaduct on the Landscheidingsweg, and not on the main road.

Favourable experience having been gained with this slab, two experimental slabs were then constructed on the main road.

Pre-tensioning was employed, i.e. the prestressing wires were tensioned before the concrete was placed. With this method, therefore, it was necessary to provide some means of taking up the prestressing force until the concrete had gained sufficient strength to have this force transferred to it (see Figures 28 and 31).

A prestressed concrete beam (with a cross-section of 120 x 45 cm and a prestressing force of 420 tons) was installed in front of each end of the slab. The prestress in these beams was produced by means of the Dywidag system, 14 prestressing bars of 26 mm diameter being provided in each beam. With this prestressing system the bars are tensioned to the required amount, and a nut mounted on the threaded end of each bar is then tightened against a special anchor plate which serves to spread the prestressing force to the concrete.

Two rows of pressure piping (formed of lengths of pipe of 60 cm diameter and 5 mm wall thickness) were installed along each of the two longitudinal sides of the future slab, between the two prestressed end beams. These pipes were flanged together. Next, the high-tensile wires were laid out in pairs

Figure 31. Detail of pre-tensioning frame

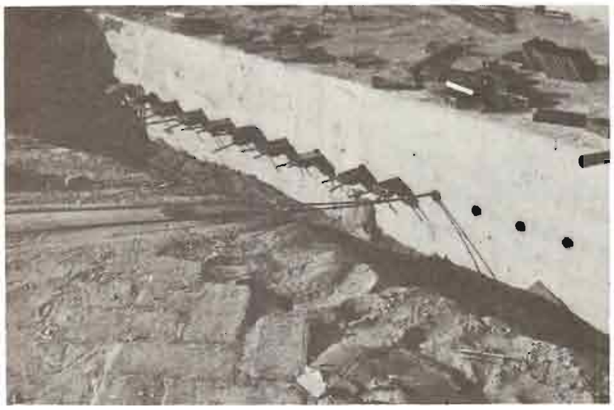


Figure 32. Detail of end beam with anchorages of wires

longitudinally between the pipes and threaded through holes formed in the prestressed beams. After being tensioned, the wires were anchored against these beams with the aid of high-tensile steel wedges in steel blocks (Figure 32). Tensioning the wires and forcing the anchor wedges in between the tensioned wires was performed by two hydraulic jacks combined into one appliance (Figure 33). The tensioned wires produced flexural load in the prestressed beams and compression in the lateral pipes. To enhance the buckling resistance of the pipes, they were filled with water.

The deflexion of the end beams, the elastic shortening of the pipes, and the compression of the jointing material between the pipe flanges added up to a maximum value of about 8 cm. The first wires to be tensioned consequently suffered (as a result of the tensioning of the subsequent wires) a loss of stress corresponding to this 8 cm. All the wires were therefore given a predetermined extra elongation. The total extension was, on an average, 65 cm (for a wire length of 113 m). The stress finally obtained in the tensioned wires was checked with the aid of a measuring device which operates on the following principle (see Figures 34 and 35).

The device is clamped to a wire at two points. Midway between these points the wire is given a deflexion. The force needed for producing this deflexion is measured. The stress in the wire can now be calculated from the gauge length (the distance between the two clamped points), the deflexion of the wire, and the value of the force.

There was a certain complicating factor affecting the constructional procedure employed. For each traffic lane (i.e. each strip of surfacing on either side of the longitudinal joint) the procedure consisted in tensioning the prestressing wires on one day and placing the concrete on the following day. It was necessary to wait until this concrete had acquired some degree of strength before it was possible to tension the wires for the second lane. Tensioning these last-mentioned wires brought about some of the above deformation of the prestressed end beams and pipes, and this movement was shared by the ends of the already tensioned and anchored wires of the first traffic lane, thus causing them to transfer part of their stress to the concrete. Measurements and calculations showed this premature prestress to have an average value of 12 kg/cm^2 at the slab ends, and in order to be able to resist this stress it was therefore necessary for the concrete to have acquired some strength.

During tensioning, timbers were laid over the tensioned wires as a precaution against possible wire fractures or faulty anchorages. This prevented the wires from leaping up, which would have been highly dangerous. Fracture or sudden release of a wire did occur a few times in the course of the job, and the precautions that had been taken proved to be very effective. All the same, the men tended to omit these precautions because they were something of a nuisance, and it was therefore necessary to keep a close check to ensure that the safety rules were being observed.

To maintain the wires at the correct level, wooden spacer blocks were temporarily placed under them and were removed during concreting. On the second and third experimental sections, concrete spacer blocks (which were left embedded in the slab) and supporting stools made from reinforcing bars were used instead.

Prestressing was carried out by the Nederlandse Spanbeton Maatschappij, Alphen.

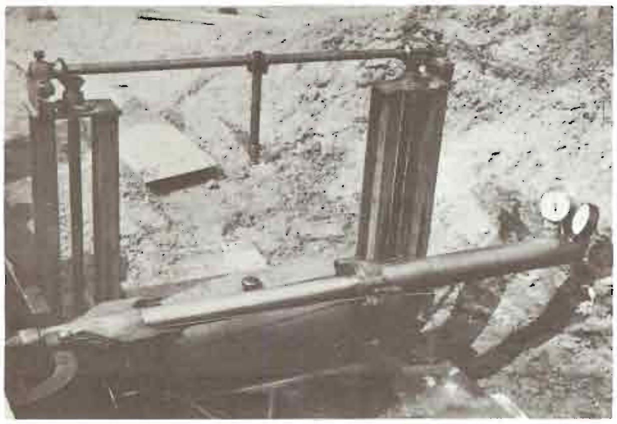


Figure 33. Hydraulic jack for tensioning and anchoring the wires

Figure 34. Device for measuring tension in wires

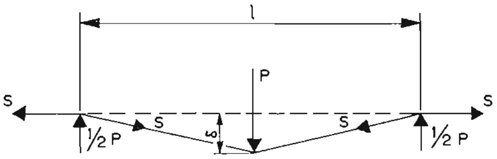


Figure 35. Diagram illustrating method of functioning of device. The tensioned wire is given a known deflexion. The force required for producing this deflexion is measured (force P). The stress S in the wire is obtained from: $\frac{1}{2} P \times \frac{1}{2} l = S \delta$ (this expression is valid only if the wire thickness is small in relation to l).

The formwork used consisted of the standard (23 cm high) forms for concrete roads, which were buried 11 cm in the sub-grade. At the ends of the slab the formwork was a timber beam provided with holes, drilled at the correct centres, through which the prestressing wires were passed.

Transverse reinforcement

In the first lane the mild-steel transverse reinforcement was attached to the underside of the high-tensile steel prestressing wires. It often occurred during concreting that the transverse bars fixed in this way were detached by men treading on them. For this reason it was decided to place the mild steel on top of the wires in the second lane.

Concrete

The standard mix for ordinary concrete roads was employed, with a cement content of 350 kg/m^3.

The concrete for the first traffic lane was placed by hand; a spreading machine was used for the second lane. In both lanes a vibrating screed was employed for compacting and finishing the concrete. Additional compaction was applied by means of a small manual vibrator at the ends of the slabs where the local mild-steel mesh reinforcement had been provided.

The concrete was covered with a layer of sand which was kept wet for 21 days.

Transfer of the prestress to the concrete was done six days after concreting the second lane, the wires being severed by burning. During these six days no big drops in temperature occurred, so that the risk of cracking of the long and, as yet, unstressed slabs was greatly reduced.

MEASUREMENTS

Introduction

It was essential to carry out measurements in order to ascertain the stress distribution in the concrete slab. The programme of measurements was drawn up in consultation with T.N.O.*, which organization also undertook to do the measuring. Furthermore, some investigations were carried out by the National Road Research Laboratory (Rijkswegenbouwlaboratorium). The measurements conducted on the first experimental section were for general guidance only, with a view to obtaining an idea of the problems involved. In the first place it was of great importance to ascertain, immediately after transfer of the prestress, whether the prestress would be able to overcome the friction between the concrete and its base, i.e. whether the central part of the slab would indeed be prestressed.

As it was found desirable to do so, more elaborate measurements were made on the other two experimental sections, and these measurements were continued over a longer period of time, in order to determine also the effect of creep, shrinkage, and friction on the stress distribution in the slab in the event of variations of temperature.

In addition, test loadings were made on that part of the first experimental section which will not be used by road traffic.

The procedure adopted for the taking of measurements, and the results of the measurements, are given in a comprehensive report issued by T.N.O.

These matters will be summarized and, where necessary, provided with explanatory comment in the following sections of this article. In addition, the modifications applied to the design in consequence of the results of the measurements will be discussed.

Procedure adopted

The stresses in the concrete were calculated from the strains, which in turn were determined from lengths and curvatures measured at different times. The temperature of the concrete was not always the same at those times, and it was therefore necessary to make corrections to the measured values of the lengths and curvatures so as to allow for the differences in temperature. It was also necessary to make corrections for shrinkage and creep.

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* Industrial Organization for Applied Scientific Research. (Translator's note)

To calculate the stresses it was, in addition, essential to know the modulus of elasticity and the state of stress at the instant when the lengths and curvatures were measured for the first time.

The quantities to be measured were therefore: changes of length; changes of curvature; moduli of elasticity; temperatures; shrinkage; creep. In addition, the stress in the wires was measured before they were severed by burning.

The apparatus and the procedure for measuring these quantities are discussed below.

Linear variations (changes of length) were usually measured by means of the demountable strain gauge and curvimeter (Figures 36 and 37a-b). For using this device, it was necessary to establish gauge lengths by fixing rows of small brass locating discs, spaced approximately at 50 cm centres, to the surface of the concrete. The distances between the locating points on these discs were then measured with the strain gauge. To do this, the fixed pin B and the horizontal movable pin A of the gauge were inserted into the locating discs. The horizontal displacement of the pin A in relation to a zero position determined by a reading on an Invar bar was indicated by the dial gauge I. The gauge length of this strain gauge is approximately 50 cm. The accuracy of the dial guage readings is 0.001 mm. For a modulus of elasticity of 300,000 kg/cm^2 for concrete this corresponds to an accuracy of 1 kg/cm^2.

Strain measurements were also made in connexion with the loading tests that were carried out. In order to locate accurately the formation of the first crack, a short demountable strain gauge was used (with a gauge length of about 10 cm).

Under the bearing plate for the jack used for applying the load, fixed strain gauges had to be employed. These were secured to two points on the surface of the concrete and thus enabled the variations in the distance between these two points to be measured.

Changes of curvature were likewise measured by means of the strain gauge and curvimeter shown in Figures 36 and 37, as is indeed apparent from the name of this composite device. Midway between the brass locating plates of the strain gauge small smooth glass plates were fitted.* With the aid of the vertically movable pin C (Figure 37) it was thus possible to measure the deflexion of the concrete slab in relation to these plates. The reading was taken on dial gauge II, which likewise permitted measurements with an accuracy of 0.001 mm. The relation between the curvature and the bending moment associated with it has been derived in Figure 37.

Figure 36. Combined strain gauge and curvimeter

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* The "small smooth glass plates" are evidently some kind of locating disc for measurements of curvature. (Translator's note)

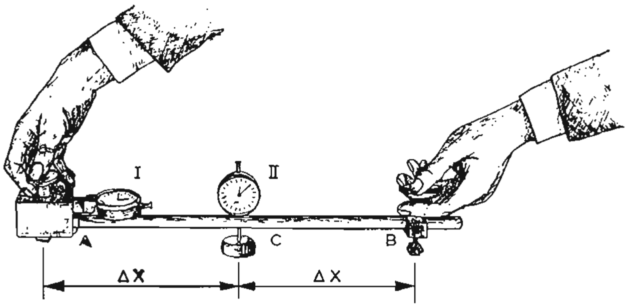

Figure 37a. Diagram of strain gauge and curvimeter

Calculation of stress from measured elongation:

$$\sigma = E\varepsilon = \frac{E\,l}{\Delta l} = E\;\frac{2\Delta X}{\text{reading}}$$

Calculation of stress from measured curvature:

slope of deflexion curve at point D =

$$\left[\frac{dW}{dX}\right]_D \propto \frac{W_o - W_1}{\Delta X}$$

slope of deflexion curve at point E =

$$\left[\frac{dW}{dX}\right]_E \propto \frac{W_2 - W_o}{\Delta X}$$

radius of curvature at B =

$$\rho = \left[\frac{d^2W}{dX^2}\right]_B \propto \frac{\left[\frac{dW}{dX}\right]_E - \left[\frac{dW}{dX}\right]_D}{\Delta X} \propto \frac{W_1 - 2W_o + W_2}{\Delta X^2}$$

$$\propto \frac{2 \text{ x reading}}{\Delta X^2}$$

$$\rho = -\frac{M}{E\,I} \quad \text{or} \quad \sigma W = M = -E\,I\left[\frac{d^2W}{dX^2}\right]_B =$$

$$= -\frac{2\,E\,I}{\Delta X^2} \text{ x reading}$$

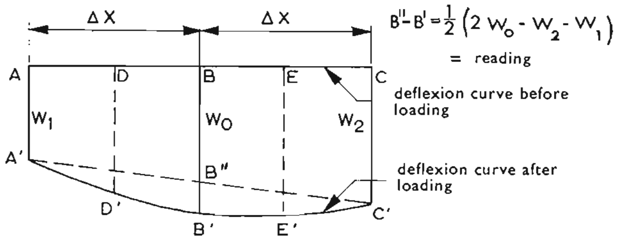

Figure 37b.

The modulus of elasticity of the concrete was determined from compressive tests on cubes which had been made at the time of concreting.

Temperatures of the concrete were measured with the aid of so-called N.T.C. (negative temperature coefficient) resistances. These are small units (8 mm diameter, 12 mm long) consisting of pressed sintered metal oxides. In an electrical circuit the resistance of this material diminishes as the temperature rises, whereas with most other materials the electrical resistance increases a little at higher temperatures. The changes in resistance were measured by means of a device fitted up as a Wheatstone bridge (Figure 36). The relation between resistance and temperature is different for each individual N.T.C. resistance, and each resistance therefore had to be calibrated in advance. With the aid of the calibration curves and measured values of the electrical resistance it was possible to determine the temperatures in the concrete slabs. These measurements were accurate to within approximately $\frac{1}{2}^{\circ}$ C. The N.T.C. resistances were embedded in the concrete at different points and at different levels in the slabs.

Differences of temperature can give rise to stresses in two different ways. If the temperature at the top of the slab differs from that on the underside, flexural stresses will occur, since the self-weight of the slab prevents curvature from developing. Furthermore, flexural and direct stresses will arise in the slab if deformations due to variations of temperature are prevented from taking place by frictional restraint.

Shrinkage and creep were — as has already been mentioned — not measured during the maturing of the first experimental section. The effect of these factors was, however, quite noticeable if two successive linear measurements were carried out with an interval of a few days between them. In particular, this was observable at the ends of the slabs, where there was no reduction of prestress due to frictional restraint. Nevertheless strains were found to occur there which were attributable only to shrinkage and creep. For this reason it was decided to include measurements of shrinkage and creep in the programme of measurements for the second and third sections. For this purpose four small concrete slabs were cast near each of these experimental sections. One of these slabs was not prestressed and served for measuring the shrinkage. The creep function, i.e. the relation between the creep strain and time (for a given value of the prestress), was determined by measurements on the other three slabs, which were prestressed. The magnitude of the prestress was different for each slab, in order that the relation between the prestress and the creep might be ascertained. It was desirable to determine this relation because the magnitude of the prestress in the road surfacing could not be regarded as a constant value.

Loading tests were carried out with the aid of the apparatus shown in Figures 38 - 41. The load applied at the selected points was steadily increased until the first crack occurred. After each increment measurements of strain were made in two mutually perpendicular directions intersecting each other directly under the centre of the jack (Figure 38). Fixed strain gauges were employed under the actual bearing area of the jack; a demountable strain gauge was used outside this area. In order to locate accurately the first crack, which manifested itself in a discontinuity in the strains measured, the short strain gauge (with a gauge length of about 10 cm; see Figure 41) was used instead of the strain gauge shown in Figure 36. The direction of the crack was determined with the aid of Figure 42.

Figure 38. Steel circular sectors for bearing plate of jack (with four strain gauges)

Figure 39. General view of loading test in progress

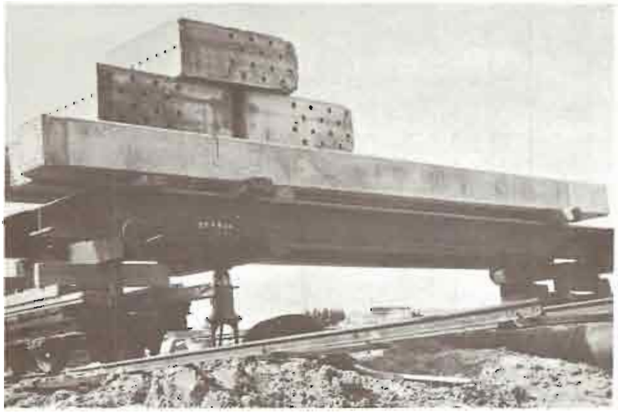

Figure 40. Close-up of loading test rig

Figure 41. The jack is seen on the right; the short demountable strain gauge is seen to the left of the jack.

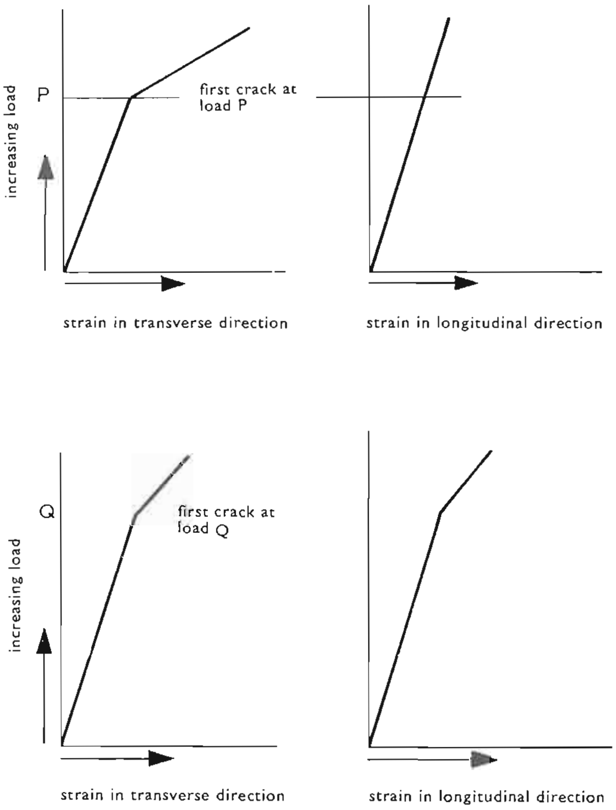

Figure 42. Diagrams showing results of strain measurements made during a loading test

RESULTS OF THE MEASUREMENTS

Properties of the material

The modulus of elasticity was 320,000 kg/cm^2 at 7 days, 340,000 kg/cm^2 at 14 days, and 360,000 kg/cm^2 at 100 days.

The measurements on the shrinkage test slabs indicated an elongational strain of 7×10^{-5} at 95 days.* The measurements were discontinued before the final value was attained. The maximum measured value for creep was 0.7. On account of the considerable scatter of the results, it was not possible to make an accurate assessment of the creep function for the experimental sections of road. It was, moreover, not possible to determine the final creep, because the measurements were prematurely discontinued for lack of funds, which was most regrettable.

Loading tests

Fracture or incipient cracking occurred at the edges at a load of 4 - 6 tons, and at some distance from the edges at a load of 8.5 tons. The low value of the cracking load at the slab edges was due, inter alia, to the poorer quality of the concrete at the edges, which in turn was due to "teething troubles" in the execution of the job. These things were considerably improved in the construction of the two other experimental sections. The transmission length of the wires at their ends, which was approximately 2 m in the first section, was reduced to 0.50 m in the other two. In addition, the ends of the second and third experimental sections were given a transverse prestress, and extra prestressing wires were installed longitudinally along the edges of the slabs.

Stresses in the concrete

After the transfer of the prestress to the concrete, compressive stresses of 28 - 34 kg/cm^2 were measured in the longitudinal direction at the slab ends. At the centre of the slabs the compressive stress in the concrete, in the second and third sections, varied between 20 and 35 kg/cm^2.

In these sections the edge stresses at the ends of the slabs locally reached values as high as 50 kg/cm^2 in consequence of the presence of the extra prestressing wires. The transverse prestress produced compressive stresses of 5 - 15 kg/cm^2.

Coefficient of friction

The difference in compressive stress between the ends and the centre of the slab is determined by the magnitude of the coefficient of friction (sub-grade restraint). As calculated from the results of the measurements, this coefficient was found to have an average value of 0.8.

Conclusions

The results of the measurements largely confirmed the assumptions made for the preliminary design. The magnitude of the coefficient of friction is

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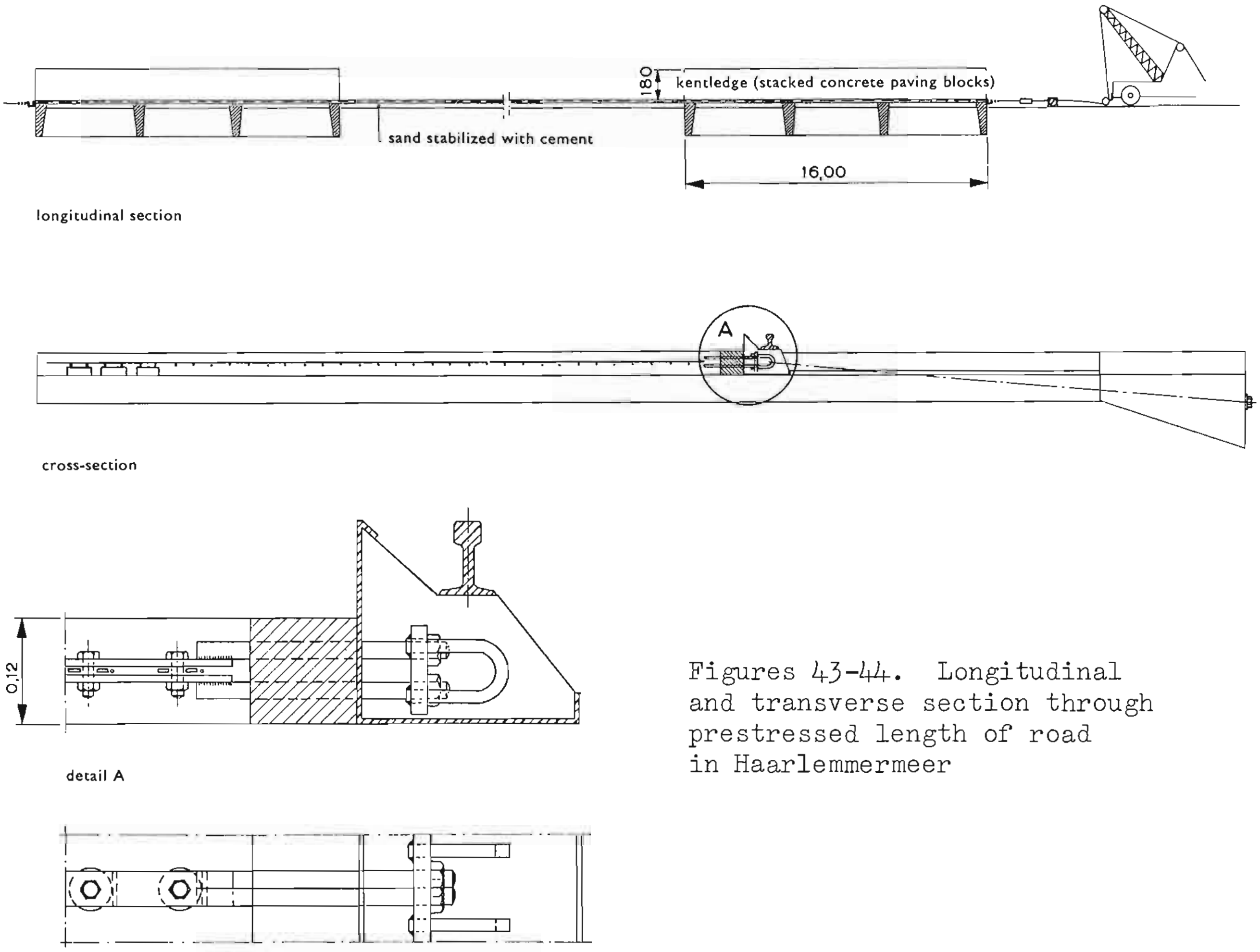
*It is explained later on that the phenomenon measured was swelling, not shrinkage. (Translator's note) in agreement with the value adopted in the calculations. The prestress — which was determined with reference to the coefficient of friction, the slab length, and the loading — was found to have the correct value. The experimental prestressed road surfacing has been used by traffic since the beginning of 1958 and has been entirely satisfactory.

In addition, particularly with the second and third experimental sections, the execution of the work was found to be simple. Concreting can be done with the type of plant used in ordinary concrete road construction.

PRESTRESSED CONCRETE ROAD SURFACING 1,100 m IN LENGTH

Introduction

In view of the favourable experience gained, it was decided to construct a stretch of prestressed road surfacing 1,100 m long in 1960 (Figures 43 and 44).



Figures 43-44. Longitudinal and transverse section through prestressed length of road in Haarlemmermeer

The road selected for the purpose was National Highway No. 4A in the Haarlemmermeer* between the aqueduct and the Lisserdwarsweg. This surfacing, which was likewise given a slab thickness of 12 cm, was constructed on a base 15 cm thick consisting of cement-stabilized sand (modulus of sub-grade reaction $k = 25\ kg/cm^3$).

The stresses were calculated in exactly the same manner as described above.

Length of slab

In foreign literature (e.g. in the German directives for prestressed concrete road surfacings) it is recommended, on the basis of experience, that the length of the slabs should be not more than 150 m. In view of the experience that had already been gained with the experimental sections described in the foregoing pages, it was considered perfectly feasible to construct slabs of this length.

The design therefore provided for 52 slabs per carriageway, of lengths ranging from 130 to 200 m. In addition, the construction of an electrically heated experimental slab, 160 m long, was included in the design. For the different lengths of slab the reduction in stress at the centre of the slab in consequence of the maximum friction (on the assumption that f = 1) is as follows:

130 m	...	15.6 kg/cm^2
150 m	...	18.0 kg/cm^2
170 m	...	20.4 kg/cm^2
190 m	...	22.8 kg/cm^2

Prestress

The prestress in the experimental sections described above was found to be adequate for the length of slab concerned. The maximum stresses liable to occur in the experimental sections were as follows:

	Type of base		
	compacted sand	cement-stabilized sand	
due to 4-ton wheel load	56	47	kg/cm^2
due to restrained warping by temperature	14	14	kg/cm^2
due to friction (sub-grade restraint)	12	12	kg/cm^2
Total	82	73	kg/cm^2

Against this we have:

flexural strength of concrete	60	kg/cm^2
final prestress (after all losses)	25	kg/cm^2
	85	kg/cm^2

As compared with the foregoing experimental sections, the only difference in the present design is in the stress due to frictional restraint, which in this case is higher on account of the greater lengths of slab. To compensate for this, the prestress has had to be increased. A value of 30 kg/cm^2 has

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*The Haarlemmermeer ("Haarlem Lake") is an area of land reclaimed from a lake in the last century. It is located between, and to the south of, Haarlem and Amsterdam. Lisserdwarsweg is the name of a road. (Translator's note) therefore been adopted for the final prestress. The stresses liable to occur for the different lengths of slab are accordingly as follows:

Length of slab (m)	130	150	170	200
Most unfavourable combination of stresses (kg/cm^2)	77	79	81	85
Flexural strength + prestress (kg/cm^2)	90	90	90	90

With regard to the difference between the flexural strength + prestress and the most unfavourable stress combination it can be stated that:

(a) for each length of slab this difference is more favourable than the same difference in the experimental slabs laid on a sand base;

(b) only for the 200 m long slab is it less favourable than the same difference in the experimental slab laid on a sand-cement base.

The factor of safety emerging from the stresses calculated above has the following values:

(a) 1 - 1.1 if the most unfavourable stress combination is considered;

(b) 2 if the stress due to wheel loads only is considered.

The actual values of the factor of safety are believed to be higher. In France several comprehensive loading tests have been carried out on prestressed concrete pavements. In those tests it was repeatedly demonstrated that no cracks occurred if the tensile stress due to the applied load exceeded the flexural strength + prestress. To explain this phenomenon it has been suggested that the application of a prestress to concrete that has not yet fully hardened increases the flexural strength, and the formation of microscopically small shrinkage cracks, which initiate the development of cracks in the event of overloading of the structure, is prevented.

Prestressing steel

In each carriageway the prestress is produced by 7 mm diameter strand-type tendons (core wire 2.4 mm diameter, surrounded by six wires of 2.3 mm diameter) spaced at 10 cm centres. The initial stress in the steel was 110 kg/mm^2, corresponding to 70 % of the tensile strength. This value of the initial stress had also been used for the experimental sections. (The German regulations and the STUVO directives specify respectively that the initial stress in the steel shall not exceed 70 and 67 % of the tensile strength.)

The initial prestressing force per tendon was 4.5 tons, producing a stress of 36 kg/cm^2 in the concrete. This initial value of the prestress subsequently diminished in consequence of creep, shrinkage, elastic shortening of the concrete, and relaxation of the steel.

Shrinkage

The choice and mix proportions of the cement, aggregates, and water must be such that shrinkage is reduced as much as possible. According to the STUVO directives the final shrinkage strain to be adopted for structures in the open air is 25×10^{-5}.

Measurements carried out by T.N.O. on small experimental slabs which were allowed to harden under conditions similar to those for the experimental prestressed slabs constructed on National Highway No. 4A indicated, however, that swelling occurred. The same phenomenon was observed in the course of measurements carried out on ordinary concrete slabs in 1956. This swelling is probably due to the particularly high humidity during the maturing period, the slabs having been concreted in the autumn and kept wet for three weeks. The duration of the measurements was too short, however, to ascertain whether permanent swelling had occurred. Swiss observations on the prestressed concrete road at Naz yielded the following results: the swelling of prisms stored in water was constant at the end of 200 days and amounted to 19×10^{-5}; the shrinkage of prisms stored in the open air was constant at the end of 250 days and amounted to 20×10^{-5}.

In view of the above data it appears likely that the shrinkage of slabs constructed during a dry period will not exceed 10×10^{-5}.

Elastic shortening

The prestress was transferred to the slabs 4 - 7 days after they had been concreted. At that stage the modulus of elasticity had not yet attained its maximum value, but was 300,000 kg/cm^2, according to measurements carried out by T.N.O. on the concrete of the experimental slabs on National Highway No. 4A. For a prestress of 30 kg/cm^2 acting over the entire length of the slab the elastic shortening will be:

$$\varepsilon_{el} = \frac{30}{300{,}000} = 1 \times 10^{-4}$$

Creep

According to the STUVO directives the creep should be calculated with the aid of the following formula:

$$\varepsilon_{cr} = \frac{2.5\,\sigma_{bdg}}{E} = 2.5\,\varepsilon_{el}$$

where σ_{bdg} is the average final stress in the member under working load.

The creep measured in the course of the above-mentioned Swiss investigations, for a prestress of 20 - 40 kg/cm^2, was $1.95\,\varepsilon_{el}$. The creep measured by T.N.O. on the experimental slabs was less. The duration of the measurements was too short, however, to enable the complete creep to be ascertained.

The value for creep indicated in the STUVO directives was adopted for the purpose of the present design.

The final value of the modulus of elasticity was also measured by T.N.O. and was found to be 350,000 kg/cm^2.

Therefore:
$$\varepsilon_{cr} = \frac{2.5 \times 30}{350{,}000} = 2 \times 10^{-4}.$$

Relaxation of the steel

The STUVO directives state that the loss of stress due to relaxation of the steel must be reckoned at 10 % of the initial stress. Some foreign directives specify lower values for this loss. According to the Nederlandse Spanbeton Maatschappij, who carried out the tensioning operations for the experimental sections, a value of 8 kg/mm^2 must be adopted for the loss of stress in the steel due to relaxation.

Thus the total loss of stress in the steel due to creep, shrinkage, elastic shortening, and relaxation is:

$$\begin{array}{lr} (2 + 1 + 1) \times 10^{-4} \times 19{,}200 & = \;\; 8 \text{ kg/mm}^2 \\ \text{relaxation} & = \;\; \underline{8 \text{ kg/mm}^2} \\ \text{total} & \underline{16 \text{ kg/mm}^2} \end{array}$$

The final stress in the steel will therefore be 94 kg/mm^2, and the final stress in the concrete will be 30 kg/cm^2.

Transverse reinforcement

From foreign literature on the subject it appears that no transverse prestress is required if the road surfacing is provided with a longitudinal joint. The correctness of this assertion was confirmed in the experimental sections described above: no longitudinal cracks appeared. A longitudinal joint was likewise provided in the 1,100 m length of prestressed concrete road under present discussion. For this reason the slabs have merely been provided with mild-steel reinforcement (8 mm diameter bars at 15 cm centres) fixed to the prestressing wires in the form of welded mesh. The transverse reinforcement is thus located practically at mid-depth in the slab and will contribute to the strength thereof only after cracking has occurred. To install the transverse reinforcement at the bottom of the 12 cm thick slab would have made the execution of the work difficult and expensive. The transverse reinforcement has been interrupted at the longitudinal joint.

Longitudinal joint

The longitudinal joint can be formed in two ways:

(1) the full width of the carriageway is concreted in one operation, and the longitudinal joint is subsequently formed by sawing;

(2) each strip of carriageway, on either side of the longitudinal joint, is concreted separately, thus automatically producing the joint.

The second method was adopted, because the contractor's plant was suitable only for concreting half the carriageway width at a time. This procedure has the following disadvantages as compared with the first method:

(1) more formwork has to be fixed and adjusted;

(2) special tie-bars are needed, which twice have to be bent to different positions;

(3) the prestressing wires in the second strip cannot be tensioned until the formwork for the first strip, along the longitudinal joint, has been removed;

(4) the tensioning operations are more lengthy, more complicated, and more expensive.

Slab ends

The end portions differ in the following respects from the rest of the slab.

(1) The loading is more severe on account of a higher value of the impact factor.

(2) A point load applied to a corner of the slab produces a higher tensile stress in the concrete than does a point load acting elsewhere on the slab.

(3) The prestress is lower in consequence of slip at the ends of the tendons. The bond between the concrete and the steel is unable to prevent the ends of the tendons from losing some of their extension, resulting in a reduction of the prestress. With careful workmanship, especially with regard to compaction of the concrete, the zone in which the prestress is thus diminished can be confined to a length of 0.5 - 1 m, as was demonstrated by the earlier experimental sections. In this end zone, therefore, the prestress varies from zero (at the end face of the slab) to the required value of 30 kg/cm^2.

To compensate for the above factors the following precautions were adopted.

(1) The ends of the slabs have been provided, top and bottom, with a mild-steel reinforcement having a width equal to that of the carriageway and extending a distance of 2 m in the longitudinal direction (so-called hairpin reinforcement).

(2) The ends of the slabs were given a transverse prestress. In the experimental sections, some of the ends had likewise been transversely prestressed by means of eight post-tensioned wires of 5 mm diameter which were inserted into ducts formed by embedded plastic tubes. The prestress was measured by T.N.O. immediately after the wires had been tensioned and was found to be 12 - 16 kg/cm^2 a short distance from the ends (i.e. from the outside edges of the slab) and 6 - 8 kg/cm^2 at the longitudinal joint. In designing the 1100 m long stretch of prestressed concrete road it was considered necessary to increase somewhat the magnitude of this transverse prestress: ten post-tensioned wires of 7 mm diameter (spaced at 25 cm centres) were accordingly installed at the slab ends. In view of the earlier measurements, the initial prestress at the outer edges of the slab will probably be about 21 kg/cm^2, and at the longitudinal joint it will be about 10 kg/cm^2.

Transverse joints (Figure 45)

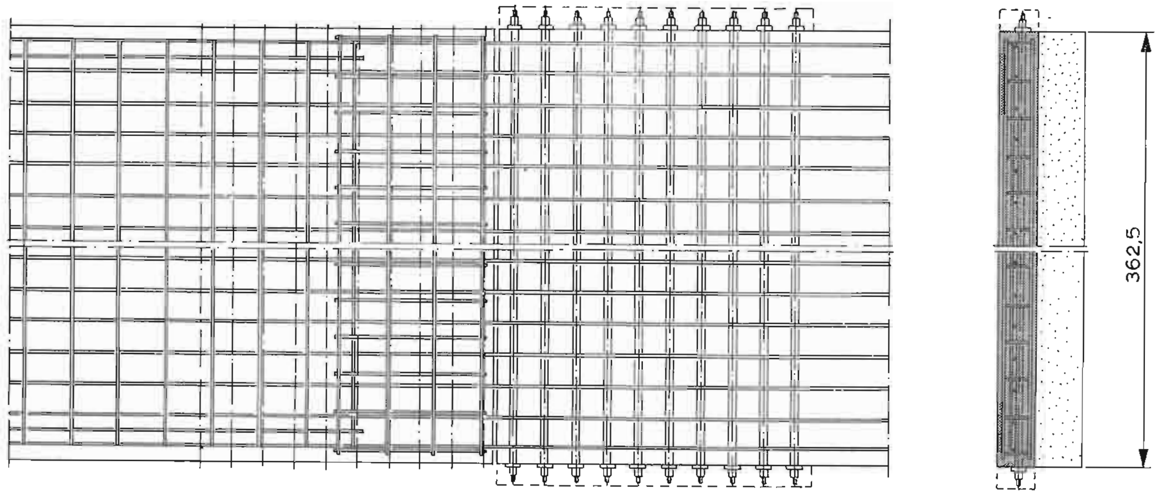
The following two types of joint occur in the design:

joints between the concrete slabs and the abutments;

joints between adjacent concrete slabs.

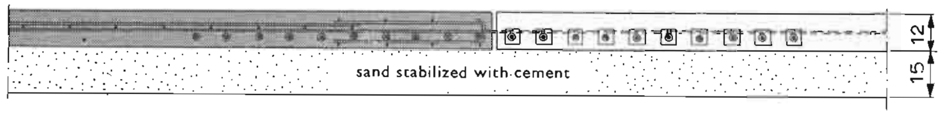
The joints between the slabs and the abutments are wide and have been filled with concrete paving blocks. The transverse joints between the slabs were sawn after the concrete had hardened sufficiently; the tendons were severed in these joints at the same time, thus causing the prestress to be transferred to the concrete. When the wires had thus been severed, the joint width underwent an increase. The maximum length of a slab terminating at a transverse joint was 200 m. If the temperature remains constant after concreting, the width of the joint increases in consequence of shrinkage, creep, and elastic shortening:

initial width of joint		=	0.3 cm
elastic shortening	$1 \times 10^{-4} \times 20{,}000$	=	2.0 cm
shrinkage	$1 \times 10^{-4} \times 20{,}000$	=	2.0 cm
creep	$2 \times 10^{-4} \times 20{,}000$	=	4.0 cm
	total		11.3 cm



plan of half carriageway at joint

cross-section



longitudinal section

elevation

Figure 45. Diagram of reinforcement at transverse joint

This eventual width of the joint will be smaller according as the slab is shorter and also if the shrinkage, creep, and elastic shortening have been overestimated.

After these deformations had taken place, the joints — now approximately 11 cm wide — were filled with concrete. It had to be borne in mind that variations of temperature would subsequently occur which might necessitate the placing of additional concrete in the joints and would cause changes in the stresses.

It is necessary to take account of a temperature of the slab that may vary approximately between -13 and +27° C. It is essential to avoid the occurrence of tension in the concrete due to a lowering of the temperature. Theoretically the joints should therefore have been filled solid at -13° C: changes of temperature would then have only produced varying values of compressive stress in the slab. But as it was intended to fill the joint with concrete, the considerations had to be based on a temperature of about +4° C.*

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*Because the placing of concrete at temperatures below +4° C is deprecated. (Translator's note)

When the temperature drops from +4 to -13° C, the joint filled with concrete will therefore open out, the theoretical increase in width being:

$$\alpha bt = 10 \times 10^{-6} \times 17{,}000 \times 17 = 2.9 \text{ cm}$$

(for a slab length of 130 m the corresponding value is 2.2 cm)

When the temperature of the slab rises to +27° C, the increase in compressive stress will be:

$$E\alpha t = 350{,}000 \times 10^{-6} \times 23 = 81 \text{ kg/cm}^2.$$

The total compressive stress will therefore (if the prestress is 30 kg/cm^2) be 121 kg/cm^2.

Ordinary concrete surfacings, 23 cm thick, without expansion joints have been constructed on National Highways Nos. 4A and 16. These surfacings have displayed no buckling phenomena. In the present 1,100 m length of prestressed surfacing, however, the loading conditions are considerably more unfavourable because:

(1) the slab is thinner, so that the resistance to buckling is lower;

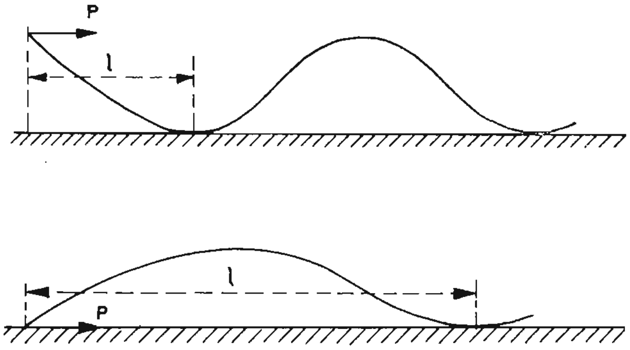
(2) the maximum compressive stress includes not only the stress due to temperature but also the prestress;

(3) if the above-mentioned procedure is employed, the compressive stress is liable to become much higher than that in the ordinary concrete surfacing, because the latter was constructed at a temperature of about 17° C, whereas the joints of the prestressed surfacing were filled with concrete at 4° C.

In Peltier's article(3) it is stated that in the course of tests no buckling occurred in slabs more than 15 cm thick, but that it did occur in slabs 8, 10, and 12 cm thick at compressive stresses of approximately 75 kg/cm^2. A number of "blow-ups" were investigated, which were attributed to an eccentricity of the compressive force amounting to 2.5 cm. In the article in question the author has calculated the buckling stresses associated with this eccentricity of the force; in slabs with lengths corresponding to those under consideration these stresses are as follows.

	Buckling stresses (kg/cm^2) for slab thickness of	
	10 cm	15 cm
If buckling occurs at the end of the slab and the end raises itself from the ground (see Figure 46)	71.6	86
As above, but the end of the slab does not raise itself (see Figure 47)	97.5	139
If buckling occurs in the middle of the slab, away from the ends	134	205

If the prestressing tendons are carefully installed at the correct height and if the joints are filled with concrete in such a way as to ensure that the compression due to a rise in temperature will act at, or a few millimetres above, the centre, then the value of 121 kg/cm^2, as calculated above, need not be exceeded. However, if sand and gravel get into the joints (which is liable to occur at temperatures between +4 and -13° C, when the joints open out), the compressive stress at 27° C will be higher than that value, and it may occur that the point of application of the pressure will be so displaced as to increase the eccentricity. On the other hand, the assumptions on which this calculation is based may quite conceivably be too unfavourable.



Figures 46-47. Buckling at end of slab.
Top: the end is lifted up.
Bottom: the end is not lifted up.

Abutments (see Figure 43)

The initial prestressing force per carriageway is 130 tons which is produced by the pre-tensioned strand tendons of 7 mm diameter. The concrete cannot take up the prestressing force until it has sufficiently hardened. It is therefore temporarily necessary to provide some other means of resisting this force.

As has already been described, in the earlier experimental sections a sort of prestressing frame was constructed around the slab. The long sides of the frame consisted of a double row of pressure pipes, while the short sides were formed by prestressed concrete beams. In the present 1,100 m length of prestressed surfacing, however, the design required the tensioning of tendons 1,100 m in length, so that this solution had to be ruled out as impracticable.

In other countries (Russia, Italy) a solution has been tried in which the prestressing force is resisted by wall-type abutments installed in the ground, these abutments being connected by tie-rods to a beam placed on the road base. The tendons are anchored to this beam. Under the conditions encountered on National Highway No. 4A, however, this solution presents the following drawbacks.

(1) To obtain sufficient anchorage resistance to take up the prestressing force, the abutments would have to be buried in the ground to a depth of about 3.5 m. (In addition there would be a 2 m depth of sand fill extending for a distance of 4 m in front of the abutment.) This would involve a considerable depth of excavation, and this excavation would have to be carried out twice if it was decided subsequently to remove the abutments. The resulting disadvantages are:

(a) in places the abutment would have to be installed below the level of the ground-water;

(b) the abutment would be buried in disturbed ground, which would adversely affect the force it could resist;

(c) the abutment would be liable to undergo a considerable amount of movement in the disturbed ground;

(d) considerable settlement is likely to occur in the surfacing subsequently laid over the abutment.

(2) The abutment can function in one direction only. To provide anchorage for tendons in both directions, it would be necessary to install two abutments close together. In view of these drawbacks, this type of abutment was not adopted in the present scheme.

A different design was adopted, which is shown in Figure 43. Essentially the abutment consists of an inverted open concrete box whose side walls, as well as the internal diaphragms with which it is provided, are buried in the ground. For this purpose slots have to be hacked in the cement-stabilized sand base. There are three longitudinal and four transverse diaphragms, 1.65 m in depth and 0.45 m thick. The reinforcement for the diaphragms is assembled in advance and placed in position as a single whole.

A kentledge consisting of concrete paving blocks, stacked to a height of about 2 m, is placed on the top slab of the abutment. The prestressing force is transmitted to the abutment as a compressive force, its permissible magnitude being equal to the failure load of the abutment divided by the factor of safety. The failure load is equal to the lesser of the two following forces:

the force at which the concrete structure fails;

the force at which the earth resistance is overcome.

In determining the location of the abutments the following considerations applied:

owing to the presence of civil engineering structures, the length of tendon to be tensioned must not exceed approximately 1,100 m;

it is possible to tension tendons in horizontal curves.

To minimize the risk of complications, however, the procedure should be kept as simple as possible. Hence the abutments have been so arranged that not more than one curve occurs in the length to be tensioned and that this curve is situated, where possible, at the end of that length. If the tendons are tensioned from the other end, their extensional movements in the curve will be small.

Horizontal curve

The length of road under consideration contains a horizontal curve. In the curve the tendons are laid to a polygonal profile (as they were in the experimental slabs) and are maintained in position by lateral forces acting at certain points.

The length of the sides of the polygonal profile should be as large as possible in order to limit the number of lateral bearing points for the tendons. The maximum length of side is determined by the radius of the curve, the minimum cover of concrete required (5 cm), and the maximum permissible distance between the outermost tendon and the edge of the concrete slab (20 - 25 cm) (see Figure 48). In the design for the 1,100 m length of road, a certain predetermined length of side was assumed for the polygonal tendon profile, and the distance between the outermost tendon and the edge of the slab associated with this length was determined. The calculated value for this distance had to be less than about 25 cm. Hence, in the curve, the width of the group of tendons as a whole undergoes a reduction equal to b-a (see Figure 48).

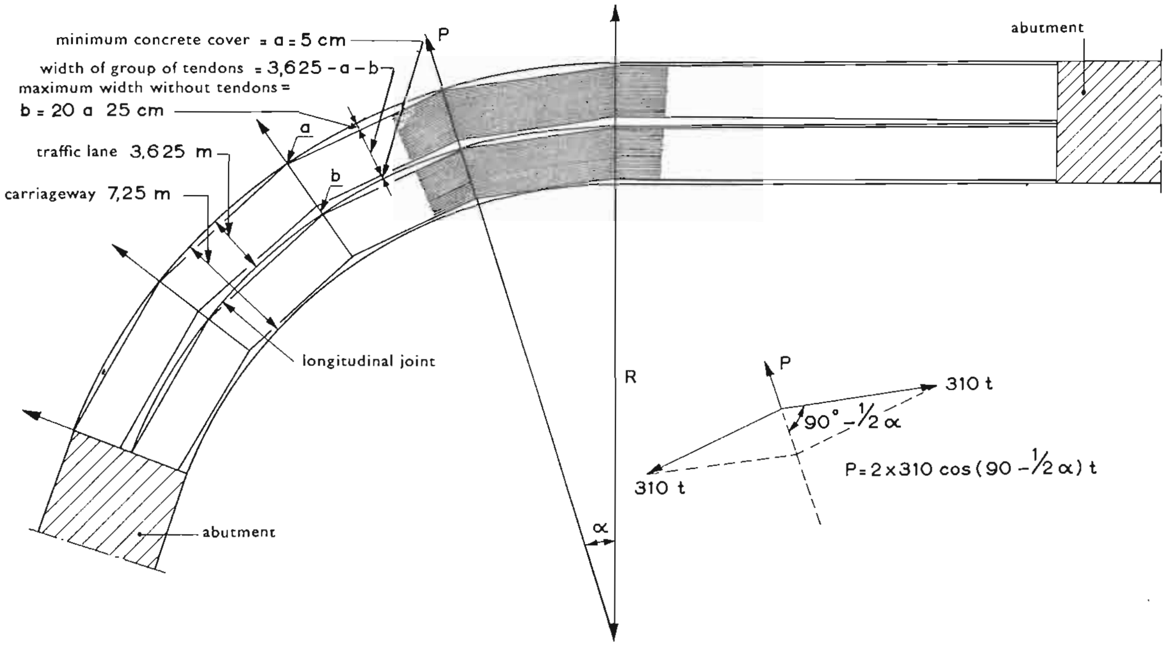


Figure 48. Diagram of a length of surfacing comprising a horizontal curve

The magnitude of the lateral forces to be resisted at the corners of the polygon may be calculated from the formula:

$$P = 2 \times 310 \times \cos\,(90^{\circ} - \tfrac{1}{2}\,\alpha) \quad \text{tons}$$

$$P_{max} = 8 \text{ tons}$$

The value of α is obtained from the length of the sides of the polygon and from the radius of the curve. These lateral forces are resisted as follows. During the tensioning process, the wires move on rollers installed at the corners (i.e. the points where changes in the direction of the tendons are introduced) and mounted between steel strips located between the side forms. The lateral force acting on the strips is transmitted by a tie-rod, welded to the strips, to a lateral abutment installed outside the slab formwork (see Figure 44, detail A). One such abutment (a block of concrete) is provided at each corner of the polygonal profile and is located beside the stabilized sand base of the slab. The tie-rod is anchored into this abutment which therefore exerts a thrust against the base.

The lateral force at a corner may have a maximum value of 8 tons. The concrete block forming the abutment is thus subjected to this force and to a vertical force V. For an abutment installed in the central reservation V = 800 kg, and for an abutment in the outer hardened shoulder V = 400 kg. Overturning of the concrete block about the point M is prevented by the weight of the block if: $Ga > Vb$.

In the central reservation:

$$0.56 \times 0.35 \times 1 \times 2.4 \times 2 = 0.94 \text{ ton} > 0.8 \text{ ton}$$

In the outer shoulder:

$$0.43 \times 0.225 \times 1 \times 2.4 \times 1.8 = 0.42 \text{ ton} > 0.4 \text{ ton}$$

The pressure on the stabilized base is:

central reservation: $\frac{8,000}{200 \times 15} = 2.7 \text{ kg/cm}^2$

outer shoulder: $\frac{8,000}{180 \times 15} = 3.0 \text{ kg/cm}^2$

The compressive strength of the sand stabilized with cement is 15 - 40 kg/cm^2, which is therefore amply sufficient.

Tensioning tendons in vertical curves

Two questions had to be considered.

(1) Will the tendons in a vertical curve at the foot of an approach to a bridge not be located too high above the surface of the stabilized base?

(2) Will it be possible to overcome sub-grade restraint where the tendons have to be tensioned in a vertical curve over a viaduct (i.e. over a "summit")?

In answer to question (1): the tendons will not be located at too high a level above the base.

The maximum difference in level occurs on the north side of bridge No. 46. A tangent drawn from the abutment V to the vertical curve over the bridge touches this curve at a point which is located at a horizontal distance of 710 m and a vertical distance of 6.40 m from that abutment. The maximum distance between this tangent and the surface of the approach is 2.40 m.

The maximum deflexion (sag) of a horizontal steel wire of 5 mm diameter, stretched horizontally over a distance of 710 m with a force of 2,150 kg, can be calculated from:

$$\frac{1}{8} q l^2 = P y_{max}$$

where

q = weight of the wire (0.154 kg/cm)

l = 710 m

P = 2,150 kg

For these values we obtain: $y_{max} = 4.5$ m.

Hence the tensioned wires will not hang high above the ground, but will be resting on the road base.

In these calculations the slight slope of the straight line between the points of support has been neglected.

In answer to question (2): the radii of the curves are large and the slopes are flat, so that the frictional resistance will be small.

Supporting the tendons

To ensure that the tensioned tendons were embedded at the correct level in the slab, they were supported on stools formed from mild-steel bars.

Formwork

The same type of formwork as normally used for concrete roads was employed for the construction of this 12 cm thick surfacing. A satisfactory degree of regularity was obtained by the use of a finishing machine behind the compacting machine.

Prevention of shrinkage cracks

Especially during the time that elapses between the placing of the concrete and the transfer of the prestress to the slab there is a danger that shrinkage cracks will develop. At this stage the 12 cm thick slabs, with lengths ranging from 130 to 200 m, are not yet prestressed, and the tensile strength of the concrete during the first few days is still very low. In this condition the slabs have to overcome the sub-grade restraint if they undergo deformation due to drying-out or a drop in temperature.

A rise in temperature is not so dangerous, since the compressive strength of the concrete increases fairly rapidly. The tensioned tendons will, to some extent, act as a shrinkage reinforcement and they will, together with the sub-grade restraint, tend to prevent the formation of cracks.

It has occurred both in Holland (Schiphol) and abroad (e.g. Switzerland, Germany) that in such cases the concrete was unable to undergo the deformations without developing shrinkage cracks.

In the experimental slabs on National Highway No. 4 A (near Leidschendam and Leiderdorp) no shrinkage cracks appeared but no large variations in temperature occurred during the critical period. On the other hand, with regard to the 1,100 m long stretch of road, it was necessary to reckon with the possibility of less favourable conditions.

The precautions taken to prevent shrinkage cracking were as follows.

(1) A layer of sand 25 cm thick was placed on the freshly placed concrete. This sand was kept wet for some weeks.

(2) The concrete in each 1,100 m long "tensioning length" was placed as quickly as possible. The surfacing was concreted at a rate of, on an average, 300 - 400 m per day, so that it took 3 - 4 days to concrete the entire length of road. The transfer of the whole of the prestress to the concrete could not be effected until about 10 days after concreting. Hence, for the first lengths of slab to be concreted, there existed a danger of shrinkage cracking for a period of 13 - 14 days. For the other lengths of slab, cast on subsequent days, this period was shorter.

Figure 49. General view of prestressed road with tendons (some of which are already tensioned) and transverse mesh reinforcement.

Figure 50. Havoc caused by a fractured tendon

Figure 51. Tensioning a tendon with the aid of a pulley-block mounted on a lorry

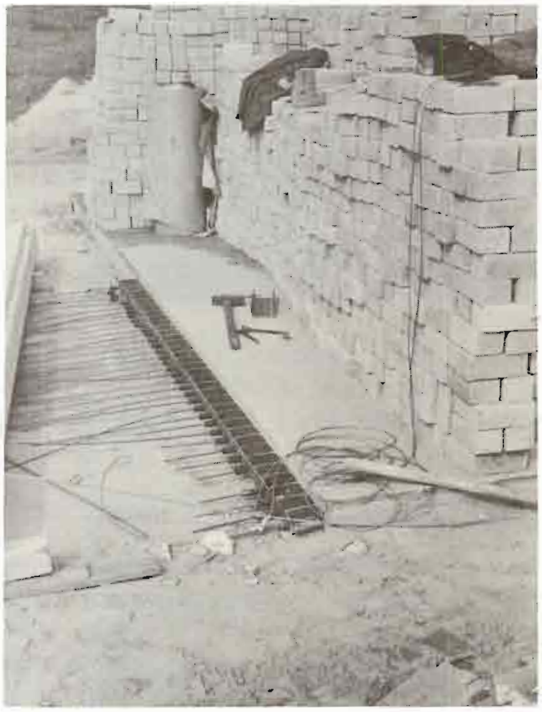


Figure 52. Abutment with kentledge of stacked blocks; the anchorages of the tendons are distinctly visible.

Figure 53. General view of the tendons, transverse reinforcement, hairpin reinforcement, etc. at a transverse joint. (See also Figure 45.)

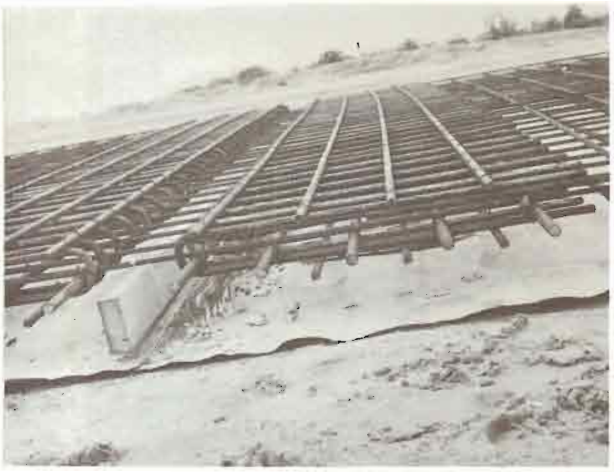


Figure 54. Side view of a future joint showing tendons, transverse reinforcement, hairpins, timber strip, etc. (See also Figure 45.)

Figure 55. Lateral support of tendons in a horizontal curve. (See also Figure 44.)

Figure 56. Sawing the concrete and the tendons at a joint

Execution of the job (Figures 49 - 56)

For the first carriageway it was decided to lay the concrete slab direct on the cement-stabilized sand base sprayed with a bituminous emulsion.

The tendons, coiled on a horizontal reel, were brought to the site on a lorry. The lorry was driven at walking pace, and the tendons were unwound from the reel and laid in position by hand. At intervals timbers provided with protruding nails were installed to serve as spacers for the tendons. In the horizontal curve the tendons were passed around rollers mounted on a steel strip laid on the road base; a second (i.e. upper) strip was then fitted in position over the rollers.

After the tendons had been threaded through ducts formed by tubes embedded in the horizontal slabs of the abutments they were anchored by means of anchor grips at one end and were tensioned from the other end with the aid of a pulley-block mounted on a lorry. The procedure was as follows. The lorry travelled a distance of 4 m at walking pace; chocks were placed behind the wheels, and the next 4 m of extension were obtained by means of the pulley-block equipment. When the required extension had thus been attained, the tendon was anchored to the abutment with the aid of an anchor grip. The tensioning force applied (approximately 4.5 tons) was measured by a gauge. On completion of tensioning, the spacing timbers were replaced by supporting stools.

As a precaution against wires shooting out of their ducts in the event of fracture, a small concrete beam was placed across the end of the slab.* For the same reason mats of mild-steel mesh reinforcement were laid on the tendons at various points along their length. These precautions proved to be effective when one tendon fractured.

It proved possible to run out and tension the tendons in a very short time. Should large jobs of this type be carried out in the future, the method can be applied without entailing any hold-up of the concreting operations.

After the mild-steel mesh reinforcement had been installed, and the "hairpin" bars and ducts for the transverse prestress at the joints had been fixed, concreting was begun.

As the compacting and finishing machine had to operate at a distance of 11 cm below the top level of the 23 cm high standard side forms, a finishing machine† was additionally employed. This new Belgian machine was a great success and produced a very even finish.

In order to determine the correct time at which to saw the joints and thus cut the tendons, T.N.O. carried out some pull-out tests on tendons embedded in small test beams. These tests were performed at 3, 7, and 10 days. The results showed that it was necessary to allow the concrete to harden for 10 days before sawing the joints.

Sawing was done with an ordinary joint-sawing machine. To minimize the thickness of concrete to be sawn, a timber strip was embedded in the concrete at each joint.

The results of the measurements showed that in the present scheme the prestress did not reach the central parts of the slabs as quickly as it had done in the 100 m long experimental sections. To speed up the process, it was decided to lay ruberoid under the surfacing of the second carriageway. It was then found that, within a few hours of sawing the joints, the full prestress had developed in the slabs. From this it can be inferred that it is not advisable to omit the provision of a sliding layer under the slabs.

The precaution of covering the freshly placed concrete with a layer of wet sand 25 cm thick proved to be very effective. The measured differences in temperature between the top and the bottom of the slab were small, and no shrinkage cracks were observed.

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*This beam is presumably the object visible in the extreme left of Figure 52 and appears to have been laid across the protruding ends of the tendons.

†The term used by the author is "correctrice", a French word; it is not clear whether this is indeed a separate machine or merely some kind of attachment to the compacting machine; nor is it clear from the context whether the "new Belgian machine" refers to the compacting machine or the "correctrice".

(Translator's notes)

Measurements

As has already been stated, measurements were carried out by T.N.O. with a view to determining the magnitude of the prestress in the slabs. In addition, differences of temperature between the top surface and the underside of the slabs were measured. Loading tests will also be carried out, and load cells have been installed.

Final conclusions

As regards the technique of execution the scheme is to be regarded as successful. A period of, say, five years will have to elapse, however, before it will be possible to assess the long-term behaviour of the prestressed surfacing.

The cost was approximately 15 % higher than that of an ordinary concrete road. For a large job the cost could be substantially reduced because:

(a) the abutments could be placed as much as 2,500 m apart (this limit is determined by the maximum available length of the wires, viz. 2,500 m);

(b) the abutments could be made to provide anchorage in two directions;

(c) it might be possible to reduce the magnitude of the prestress employed (i.e. to reduce the requisite quantity of prestressing steel);

(d) the base consisting of sand stabilized with cement could be omitted since joints would be practically eliminated. (This is borne out by the fact that the two 100 m long experimental slabs at Leidschendam, which were constructed merely on sand, are behaving quite as satisfactorily as the slab at Leiderdorp, which was constructed on a sand-cement base.)

VARIANT SCHEME

The contractors for the job, the Hollandsche Beton Maatschappij, asked to be allowed to construct an experimental section of road, 100 m in length, according to their own design (Strabed system). In order to gain experience with this alternative scheme, permission was duly granted.

Characteristic features of the Strabed system are the fact that the wires are tensioned one at a time, and the very simple manner in which each wire is embedded and tensioned. The system is, briefly, as follows.

The 9 mm diameter high-tensile steel wires are anchored in the concrete at one end and are tensioned from the other end by means of a jack.* The ends of the wires are provided — on site, with the aid of a special device — with one or more "button heads" formed by cold upsetting. One such button head is formed at the "dead" end, and one or more at the "live" end (jacking end), the number depending on the length of the wire. Before embedment in the concrete, each wire is wrapped with anti-corrosive tape, which prevents the development of bond with the concrete, so that no difficulties are encountered in tensioning the wires after the concrete has hardened.

Tensioning a wire is effected by inserting one of the button heads into the gripping device on the jack, which is operated by hand. The stress and

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* When the author speaks of "wires", he may be referring to strand tendons, though this is not explicitly stated with regard to the Strabed system. (Translator's note)

the extension of the wire can be constantly supervised. When the correct required stress has been reached, the wire is kept under tension by inserting behind one of the button heads a spacer sleeve consisting of two halves and provided externally with a screw thread. An internally threaded barrel is thereupon screwed on to the spacer sleeve until it bears against the concrete (through a steel washer).

With this single-wire prestressing system many of the drawbacks associated with the post-tensioning of thin slabs are eliminated, while the advantages are fully retained. Thus, there is no appreciable weakening of the concrete section, since no relatively large ducts have to be formed, while the greasy tape wrapping prevents excessive friction during the tensioning process.

No grouting is necessary, but the "live" ends with the sleeves and barrels must be properly cased in concrete. The bearing plates need not be excessively large, and the wires are spaced fairly close together.

The Strabed system is particularly suitable for a prestressed road surfacing which is required to have a prestress in the transverse as well as in the longitudinal direction. This can be achieved by arranging the wires in a diagonal pattern; the angle of intersection of the diagonals determines the magnitude of the transverse prestress.

This system enables the constructional operations to proceed continuously, without interruptions for installing abutments, etc. Horizontal and vertical curves present no difficulties. The prestress can be applied in stages, e.g. 20 % of the final value may be applied within 24 hours of concreting and 100 % at 28 days. After about six months or so, when most of the shrinkage has taken place, the wires may, if necessary, be re-tensioned for the last time. When this has been done, the anchorages can be finally cased in concrete. Tensioning the wires need only be effected from one side of the road slab (Figure 57), since the wires can quite suitably be curved back (with a radius of, say, 2.5 m), as indicated in Figure 58.

In the present scheme a thickness of 12 cm was adopted for the slab. The coefficient of friction for movements of the slab on its base was taken as being approximately equal to unity, i.e. f = 1.0. With reference to this

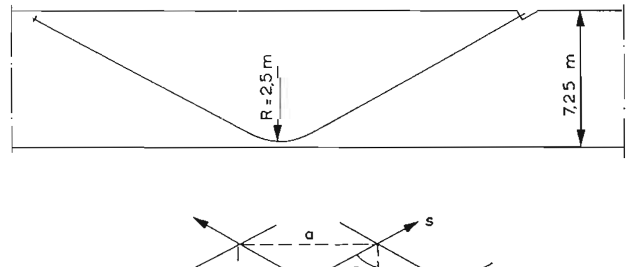

Figure 57. Diagram showing variant scheme (designed by Hollandsche Beton Maatschappij

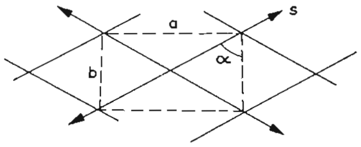

Figure 58. Diagram showing arrangement of tendons in variant scheme (designed by H.B.M.)

value for f the magnitude assigned to the longitudinal prestress was 25 kg/cm^2; 7 - 10 kg/cm^2 was adopted for the transverse prestress. The 9 mm diameter wires can be tensioned to an initial force of 6.1 tons, leaving a residual permanent force of 5.5 tons. The anchorage bearing plates must have a minimum area of $\frac{6,100 \text{ kg}}{25 \text{ kg/cm}^2} = 25 \text{ cm}^2$.

The friction of the slab on its base (sub-grade restraint), determines the practical length of the slab. For a slab length of 100 m the theoretical stress in the concrete at the centre due to this friction will already have the value $\sigma = \frac{100}{2} \times 2.5 \times 1.0 \times \frac{1}{10} = 12.5$ kg/cm^2 (tensile or compressive stress), diminishing linearly to zero at the ends of the slab.

From Figure 58 it appears that:

$$\sigma_{\text{longit.}} = 2\text{ S } \sin\alpha/\text{bd}; \qquad \sigma_{\text{transv.}} = 2\text{ S } \cos\alpha/\text{ad}$$

$$\frac{\sigma_{\text{longit.}}}{\sigma_{\text{transv.}}} = \frac{\sin\alpha}{\cos\alpha}\,\frac{a}{b} = \tan^2\alpha\,; \qquad a/b = \tan\alpha\,; \quad b = a\,\text{cotan}\,\alpha$$

The values obtained at the ends of the slab are:

$$\tan^2\alpha = 25/7 = 3.57; \qquad \alpha = 62^\circ 10'$$

$$a = \frac{2 \times 5500 \times \sin\alpha}{25 \times 12 \times \text{cotan}\,\alpha} = 61.3; \qquad b = 32.4$$

Quantity of steel per square metre = $\frac{2 \times 0.63}{0.613 \times 0.324} = 7 \text{ m} \simeq 3.5$ kg

Having due regard to the stress of 12.5 kg/cm^2 produced by friction, we obtain at the centre of the slab:

$$\tan^2\alpha = \frac{25 + 12.5}{10.5} = 3.57; \qquad \alpha = 62^\circ 10'$$

$$a = \frac{2 \times 550 \times \sin\alpha}{37.5 \times 12 \times \text{cotan}\,\alpha} = 40.9; \qquad b = 21.6$$

Quantity of steel per square metre = $\frac{2 \times 0.462}{0.489 \times 0.216} = 10.40 \text{ m} \simeq 5.2$ kg

Average quantity of steel = $\frac{1}{4} \times 5.2 + \frac{1}{4}\left(\frac{5.2 + 3.5}{2}\right) \simeq 4.6$ kg/m^2

Owing to friction, the longitudinal stress at the centre of the slab varies from 25 to 50 kg/cm^2 (compression). The maximum flexural stress due to unequal temperature of the slab plus live load is 6 + 35 = 41 kg/cm^2 in the longitudinal direction, and 6 + 26 = 32 kg/cm^2 in the transverse direction.

The excess stresses are permissible in relation to the flexural strength of the concrete; the maximum compressive stress is likewise permissible.

Since, with post-tensioning, only half the shrinkage need be taken into account, the movement at the joint will not be particularly great either.*

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* This sentence would merely appear to state that in a post-tensioned slab the shrinkage that takes place after transfer of the prestress will, in general, be less than a pre-tensioned slab (and is assumed to be only half as much, presumably in accordance with the relevant Dutch regulations).

(Translator's note)

PRESTRESSED CONCRETE ROAD SURFACING WITH CONTINUOUS TENDONS, ACCORDING TO THE SAME SYSTEM

The fact that the amount of prestressing steel in the central region of each slab has to be increased by about 40 % and that an expansion joint has to be provided at intervals of 100 m gives rise to the question whether it would not be possible to devise some way of continuing the prestress across the joints and thus keeping them closed after most of the shrinkage has taken place.

In such long jointless slabs (with or without reinforcement) as have already been constructed it has been found that a length of some 125 - 150 m at the ends does indeed continue to undergo movements, but that the intermediate portion of the slab is to be regarded as completely restrained and immovable. In this intermediate portion, therefore, we need not take account of frictional resistance to movement, though stresses produced by differences of temperature will still have to be considered.

For determining these stresses it is essential to know the value of the modulus of elasticity E. For the normally occurring variations of stress (short-term variations, as it were) we may assume $E_b = 325{,}000$ kg/cm^2, and for creep (long-term deformation) we may assume $E_{kr} = E_b/2.5 = 0.4\ E_b = 130{,}000$ kg/cm^2.

For rapid variations of temperature, e.g. occurring over a period of a few days, we should adopt $E_b = 325{,}000$ kg/cm^2, assuming a possible difference of about 10° C in the most unfavourable case.

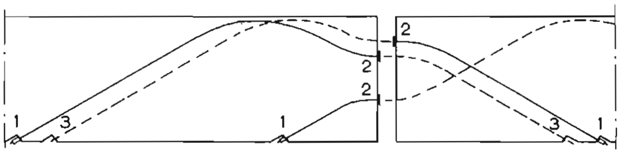
For the extreme differences between summer and winter temperatures, ranging from, say +30 to -10° C, a total range of 40° C, we can safely assume the appropriate value for E to be somewhere between E_b and E_{kr} (= $0.4\ E_b$), i.e. between 325,000 and 130,000 kg/cm^2. We shall adopt the following value $E_{b(slow)} = 0.7\ E_b = 225{,}000$ kg/cm^2. The coefficient of linear thermal expansion of concrete is 1×10^{-5}.

During construction in the most favourable months of the year the average temperature of the concrete slab will, in general, be about 15° C. In order to keep the tensile stresses that will occur in the event of a drop in temperature as low as possible, the joints should be filled at the lowest possible temperature. For practical purposes this is likely to be around 5° C.

This means that, in the most unfavourable case, a possible drop in temperature to -10° C may produce a tensile stress $\sigma_t = 10 \times 1 \times 10^{-5} \times 325{,}000 + 5 \times 1 \times 10^{-5} \times 225{,}000 = 32.5 + 11.5 = 44$ kg/cm^2; and a rise in temperature to + 30° C may produce a compressive stress $\sigma_d = 10 \times 1 \times 10^{-5} \times 325{,}000 + 5 \times 1 \times 10^{-5} \times 225{,}000 = 32.5 + 33.5 = 66$ kg/cm^2.

The concrete subsequently placed in the joints is likely to have a low tensile strength, and it is advisable to provide an amount of reinforcement equal to $100 \times 12 \times 50/10{,}000 = 6$ cm^2 per lineal metre of joint. Cut-off waste lengths of prestressing wires can suitably be used for the purpose.

In order to eliminate the danger of buckling at the joints the prestressing wires should be continued across the joints. This is technically quite feasible, as indicated in Figure 59. The wires are tensioned alternately, e.g. in each case the portion 1 - 2 is tensioned, while the portion 2 - 3 of the wire remains untensioned until after the joint has been filled. In consequence of this procedure, the ends of the slabs will initially be given only half the intended prestress.



portions 1—2 (drawn in full lines) to be tensioned first; portions 2—3 (dotted) to be tensioned later, after joint has been filled.

Figure 59. Proposed variant scheme with continuous slab (designed by H.B.M.)

When the joints are filled it is not directly possible, by tensioning the hitherto untensioned lengths of wire, to produce the correct amount of prestress in all parts of the slabs. Increasing the stress will tend to produce a shortening. Because of the closure of the joint this is no longer possible, and consequently tensile stresses will develop in the intermediate portion of the slab. These are undesirable because they cause a reduction of the prestress.

To prevent losses of stress, the sides of the joints should be forced apart by means of jacks until a stress equal to half the prestress is attained. Now if the joint is filled and the remaining tendons are tensioned, the prestress will have, or will acquire, an almost uniform distribution over the entire slab.

By filling the joints between the slabs the surfacing becomes one continuous whole. The ends, however, continue to undergo movements over a length of about 150 m, as has already been mentioned. In these end portions it is not the variation of temperature but the frictional resistance that must be considered. The latter produces a stress which increases from zero to $150 \times 2.5 \times 1.0 \times \frac{1}{10} = 37.5$ kg/cm^2 (tension or compression). This value is rather lower than the stresses due to changes of temperature calculated above, which must therefore be adopted as the determining design values.

The jointless road surfacing will therefore require a higher prestress than the value of 25 kg/cm^2 which was adopted for the surfacing with joints.

Assuming a prestress of about 40 kg/cm^2 in the longitudinal direction, and of about 9 kg/cm^2 in the transverse direction, we obtain the following values:

$$\tan^2 \alpha = 40/9 \simeq 4.5; \qquad \alpha = 65^\circ$$

$$a = \frac{2 \times 5{,}500 \times \sin 65^\circ}{40 \times 12 \times \cot 65^\circ} = 44.5 \text{ cm}; \qquad b = 20.7$$

$$\text{Quantity of steel per square metre} = \frac{2 \times 0.49}{0.445 \times 0.207} = 10.6 \text{ m} \simeq 5.6 \text{ kg}$$

Compared with the 4.8 kg/m^2 of steel usually required for the surfacing with joints, this represents an increase of only 20 %.

At very low temperatures the flexural tensile stresses due to live load will have to be resisted entirely by the flexural strength of the concrete. If the surfacing has been laid on a stabilized base, this requirement will not be unduly difficult to fulfil.

In each particular case the advantages and disadvantages will have to be weighed before deciding whether or not to provide expansion joints.

Execution

In the execution of the job the operation of wrapping the wires with "Denso" tape was found to be time-consuming. On bigger jobs, however, this operation could be done by a machine.

After the wires had been installed in position, the slab was concreted over its full width (7.35 m). The lateral "notches" for anchoring the wires are located outside the standard carriageway width of 7.25 m and will be filled with concrete in due course.

Advantages and disadvantages as compared with pre-tensioned construction

Advantages are that no abutments are necessary and that the wires can be tensioned a little after only a few hours (so that the danger of shrinkage cracking is reduced). It is possible to re-tension the wires after six months or so.

A disadvantage exists in the complicated formwork for the "notches", which must, moreover, be located outside the normal carriageway width. The number of manipulations (wrapping "Denso" tape round the wires, placing the wires in position and tensioning them) is very large, and it remains to be seen whether or not, on a large job, this will cause delays in the concreting operations.

On the other hand, it seems likely that, in a large-scale application of the method, improvements would be made. On the whole, therefore, this variant scheme is to be regarded as an interesting project, the further development of which can be awaited with confidence.

Left: Laying the 9 mm diameter wires for post-tensioning by the Strabed-H.B.M. system

Right: The 100 m long experimental section prestressed by means of the Strabed-H.B.M. system: view before commencement of concreting

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