EXPERIMENTAL PRESTRESSED CONCRETE OVERLAY FOR Y-TUNNEL ACCESS-ROAD IN THE CITY OF AMSTERDAM

Ir. G.H. Kellersmann, Public Works Amsterdam
F. Smits, Cobeton - Concrete Pavement Company Netherlands

This paper contents a description of the design and construction of an experimental prestressed cement-concrete test-section which was constructed in 1980 to examine the practibility of the application for highway pavements.
The test-section consists of 3 slabs, each with a length of 30 m and 7,50 m wide.
In order to avoid the stressing actions within the expansion joints the stressing steel is arranged in the diagonal direction. The length was kept reduced in order to keep the expansion joints as simple as possible.
The structural design is based on fatigue theory making use of the Palmgren-Miner rule.
However the test-section is located on an urban road it has been designed for the northern access road to the Y-tunnel which carries about 70.000 vehicles per day on four lanes.
Finally the paper gives a description of some economic aspects.

1. Preface

One of the authors of this report has been involved strongly in the introduction, design and construction of prestressed concrete pavements on the International Airport of Amsterdam from 1960 till 1971.

Now that these pavements are in use still satisfying for almost 20 years, one can say that it has been a wise decision to build prestressed concrete pavements on such a large scale.

Why prestressed concrete pavements are seldom applied on motorways otherwise than as an experiment? One of the problems is that the longitudinal prestressing requires special joint devices in order to have space for the stressing action with hydraulic jacks.

In Amsterdam in 1980 a test-section was built in order to test the practicability of a design, which had the aim of solving the problem.

One of the advantages of prestressed pavements is the low number of joints. On the other hand too wide spacing of the joints will cause very complicated joint constructions.

In this case a slablength of 30 m with a slab width of 7.50 m and a slab thickness of 160 mm was chosen. The prestressing elements were installed in the diagonal direction, so that the jacks could be placed at the free edges of the slabs. No concreting action in the joints was needed afterwards.
Of course other combinations of length and width are possible. After an introduction about the location of the aimed project this report deals with the structural design.

Further a description of the construction of the test section is given. Finally follows a consideration of some economic aspects comparing an unreinforced pavement with a prestressed pavement under the assumption of a sufficient scale job.

2. Location

The northern part of the province Noord Holland is separated from the rest of the country by the North Sea Canal and the Y.

The increase of motortraffic caused increasing problems at the crossings of these waterways for sea-going vessels.

Inside the Municipality of Amsterdam there are three connections between the embankments. For the local traffic the Y-tunnel is the most important one. The average daily traffic through the tunnel amounts about 70,000 vehicles, of which 14% are commercial vehicles, less than on the State Highways.

Nevertheless axle-load measurements made clear that the share of the axle-loads exceeding the legal 100 kN is 22% of the total number of equivalent standard axle-loads.

The Y-tunnel and its access-roads were opened for traffic in 1967. Besides local repairs and limited overlays until 1981 no structural maintenaince had to be carried out.

However, increasing rutting - even up to 80 mm depth - led to an extensive investigation of the properties of the asphalt-pavement and its base-course and subgrade in order to establish the residual life.

The result of this investigation was the conclusion that of about 1 km out of 6 km length of carriageway the residual life was nil. This is the carriageway between the last intersection before the entrance of the Y-tunnel in downtown direction (southbound).

As can be seen from fig.1, this roadsection is very close to the Noord Hollands Canal and trough that the ground-water table is rather high.

From calculations it was concluded that asphalt-overlays could not offer a longterm solution. And the goal was to obtain a design-life of at least 20 years.

The following alternatives were studied.

1. To break up the existing pavement and replace it by a 250 mm lean concrete sub-base covered with 150 mm asphalt-layers.

2. To break up the existing pavement and replace it by a 320 mm unreinforced cement concrete pavement on a 150 mm crushed stone base.

3. To build a continuous reinforced cement concrete overlay. This alternative was rejected soon as a concequence of the short length.

4. To build a 160 mm thick prestressed cement concrete overlay. This alternative was worked out and this led to the construction of a 90 m long test-section, which is the subject of this report. The test-section was built in 1980 on a local road in the northern part of the city of Amsterdam. The objective of the test-section was not the structural behaviour but only the practicability of the design.

5. To build a 230 mm unreinforced cement concrete overlay. The calculations for the structural design were carried out with the help of the Association of the Dutch Cement Industry.

Finally it was decided to choose for the last alternative. This decision was based upon practical reasons. Decisive was the lack of working space as it was not possible to use one lane of the north-bound carriageway.

The first and the second alternative showed to be too expensive and besides that needed too much construction time where only 5 weeks were available.

3. Structural design

The structural design for the experimental section followed the design method as reported to the Second European Symposium on Concrete Roads 1973 in Bern.(1)

The calculation was based on the data and conditions of the aimed project near the Y-tunnel. As the axle load distribution was not known by that time it had to be estimated.

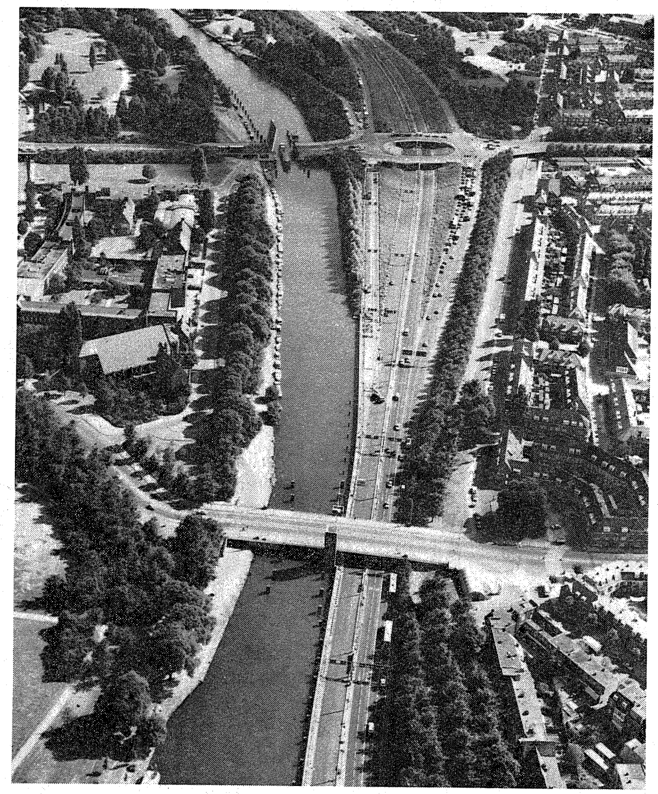
The designed pavement structure showed to be safe for more than 500,000 load repetitions of 100 kN axle loads in the heavy traffic lane, where these loads were supposed to be single wheel-loads.

The punch-out load was calculated as approximately 830 kN.

Later on a recalculation has been made, making use of the fatigue theory as given by Palmgren/Miners rule. By that time better knowledge was available about the axle-load distribution.

The calculation starts with the determination of the stresses in the concrete.
These stresses are caused by:

Figure 1. The Y-tunnel access-road is located very close to a canal.



- the prestressing forces
- the friction between the slab and the sub-base
- the temperature gradient
- the traffic

The prestressing forces

Naturally these stresses are present permanently. They are introduced by ½" strands offering a permanent force each of 100 kN after relaxation. (see fig. 2).

The longitudinal component is 0.97 and the cross component 0.24.

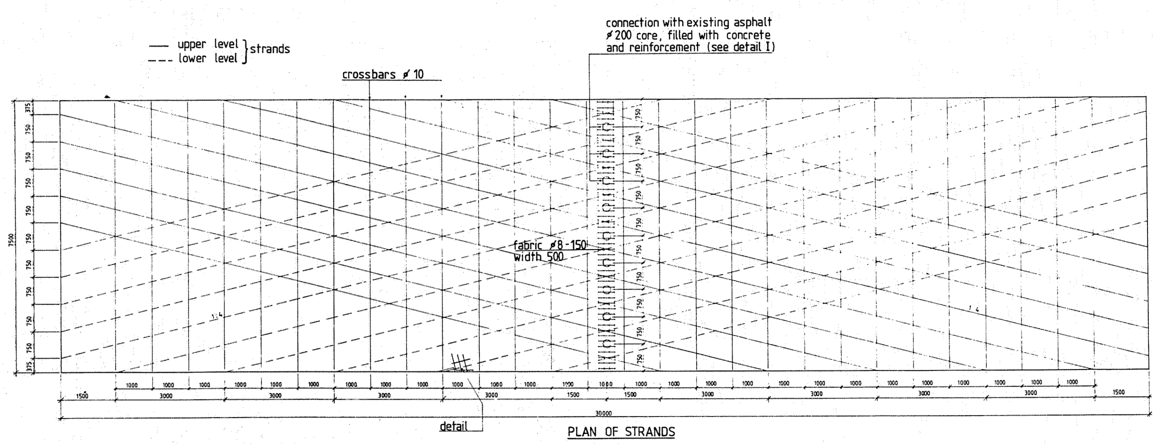
Thus the stresses in the longitudinal direction vary from 0.81 MPa at the ends of the slab to 1.46 MPa in the central part. The stresses in the cross direction vary from 0.05 MPa at the ends of the slab near the corner to 0.20 MPa in the central part.

The friction forces

The coefficient of friction between the concrete slab and the substructure can be kept as low as 0.6. This value is known from measurements which were carried out in the past at Schiphol Airport. If the temperature gradient is positive (the upper- side of the slab is warmer than the bottom) the slab will expand and the friction forces cause compressive stresses.

Otherwise the friction forces will cause tensile stresses in the case of a negative temperature gradient (upper-side colder than the bottom). Tensile stresses as well as compressive stresses are nil at the ends of the slab and maximum at half length.

Figure 2. Arrangement of prestressing steel.



The magnitude of the stresses (tensile as well as compressive) is:

$$\sigma_f = 0.5*L*\gamma*f = 0.5*30,000*2.4*10^{5} = 0.216 \text{ MPa}$$

The friction forces in the cross direction are neglected.

The temperature gradient

The temperature gradient varies naturally with the time. It is assumed that the variation in time can be outlined.

From several observations it is known, that the maximum gradient in the western part of the Netherlands which shows a moderate sea climate is 0.05°C/mm.

The assumption is:

ΔT = + 0.05 °C/mm during 5% of time (upper side warmer)
ΔT = + 0.03 °C/mm during 30% of time
ΔT = 0.0 °C/mm during 30% of time
ΔT = - 0.03 °C/mm during 30% of time (bottom side warmer)
ΔT = - 0.05 °C/mm during 5% of time

The stresses caused by the temperature gradient are:

$$\sigma_{\Delta T} = \frac{E_{stat} * \alpha_T * \Delta T * h}{2(1 - \mu)} =$$

$$\frac{2.8*10^4*10^{-5}*\Delta T * h}{2(1 - 0.15)} = 26.4\,\Delta T \text{ MPa}$$

with $\Delta T = \pm\ 0.05$ °C/mm $\quad \sigma_{\Delta T} = \pm\ 1.32$ MPa
$\Delta T = \pm\ 0.03$ °C/mm $\quad \sigma_{\Delta T} = \pm\ 0.79$ MPa

It is assumed that these values appear in longitudinal as well as in cross direction.

However some of these stresses vary in time the variations pass so slowly that they can be considered as static loads.

So in different periods of time there are different static stress-conditions on which the dynamic traffic loads must be superposed.

The possible static stress combinations are:

$\sigma = \sigma_p + \sigma_f + \sigma_{\Delta T}$ The values are listed in tables 2 to 5.

The traffic

For some years already the average daily traffic on the Y-tunnel access road is 70,000 vehicles. As a consequence of the traffic saturation of the city and the construction of a new tunnel in the ringroad around the city, no further growth of the traffic is expected.

With help of a weighing mat (see fig. 3) under traffic running with normal speed, a rather accurate estimation of the axle-load distribution could be obtained. Naturally the measured data represent dynamic loads.

Figure 3. Weighing mat.

Table 1 Symbols and Values

f	- coefficient of friction between concrete slab and base
ΔT	- temperature gradient : + means upperside warmer - means upperside colder
σ_f	- stress induced in the concrete by friction
L	- slab length = 30 m
h	- slab thickness = 160 m
w	- slab width = 7.50 m
γ	- weigth by volume of concrete = $2.4 * 10^{-5}$ N/ mm3
E_{stat}	- modules of elasticity for static loads = $2.8. * 10^{4}$ MPa
E_{dyn}	- modules of elasticity for dynamic loads = $3.5. * 10^{4}$ MPa
K	- bedding constant = 0.2 N/mm3
μ	- Poissons ratio = 0.15
α	- temperature coefficient of expansion $=10^{-5}$ m/m/ °C
l	- elastic length
q	- contact pressure of wheelloads
N_c	- number of squares covered by wheel-prints oninfluence chart nr 2 of Pickett and Ray
m	- bending moment per unit of length in Nm/m
n_i	- expected number of load repetitions in a certain axle-load group
N_i	- allowable number of load repetitions at a certain stress ratio
$\sum \frac{n_i}{N_i}$	- Miner-sum, giving the consuption of the resistence against fatique (maximum = 1)
σ_{min}	- performing static stress in the concrete
σ_{max}	- dynamic stress induced by traffic loads + static stress
f_{ct}	- characteristic flexural strength of concrete = 4.4 MPa *
f_{cc}	- characteristic compressive strength of concrete = 45 MPa *
	* values required for concrete quality class B 45 according to the Dutch Standards
σ	- stress induced by the traffic

Table 2. longitudial stresses near the end of the slab:

ΔT =+0.05 °C/mm	-0.81 + 0 + 1.32 = + 0.51 MPa
ΔT =+0.03 °C/mm	-0.81 + 0 + 0.79 = - 0.02 MPa
ΔT = 0 °C/mm	-0.81 + 0 + 0 = - 0.81 MPa
ΔT =-0.03 °C/mm	-0.81 + 0 - 0.79 = - 1.60 MPa
ΔT =-0.05 °C/mm	-0.81 + 0 - 0.32 = - 2.13 MPa

Table 3. cross stresses near the end of the slab:

ΔT =+0.05 °C/mm	-0.05 + 0 + 1.32 = + 1.27 MPa
ΔT =+0.03 °C/mm	-0.05 + 0 + 0.79 = + 0.74 MPa
ΔT = 0 °C/mm	-0.05 + 0 + 0 = - 0.05 MPa
ΔT =-0.03 °C/mm	-0.05 + 0 - 0.79 = - 0.84 MPa
ΔT =-0.05 °C/mm	-0.05 + 0 - 1.32 = - 1.37 MPa

Table 4. longitudial stresses in the central part of the slab:

ΔT =+0.05 °C/mm	-1.46 - 0.22 + 1.32 = - 0.36 MPa
ΔT =+0.03 °C/mm	-1.46 - 0.22 + 0.79 = - 0.89 MPa
ΔT = 0 °C/mm	-1.46 + 0 + 0 = - 1.46 MPa
ΔT =-0.03 °C/mm	-1.46 + 0.22 - 0.79 = - 2.03 MPa
ΔT =-0.05 °C/mm	-1.46 + 0.22 - 1.32 = - 2.56 MPa

Table 5. cross stresses in the central part of the slab:

ΔT =+0.05 °C/mm	-0.20 + 0 + 1.32 = + 1.12 MPa
ΔT =+0.03 °C/mm	-0.20 + 0 + 0.79 = + 0.59 MPa
ΔT = 0 °C/mm	-0.20 + 0 - 0 = - 0.20 MPa
ΔT =-0.03 °C/mm	-0.20 + 0 - 0.79 = - 0.99 MPa
ΔT =-0.05 °C/mm	-0.20 + 0 - 1.32 = - 1.52 MPa

Table.6 load distribution.

axle load group	number of loads in 20 years	number of loads cross distributed
< 5 kN	$2.50 * 10^{8}$	$2.00 * 10^{8}$
5 - 20 kN	$1.00 * 10^{8}$	$0.80 * 10^{8}$
20 - 40 kN	$1.00 * 10^{7}$	$0.80 * 10^{7}$
40 - 60 kN	$1.25 * 10^{7}$	$1.00 * 10^{7}$
60 - 80 kN	$0.60 * 10^{7}$	$0.48 * 10^{7}$
80 - 100 kN	$1.10 * 10^{6}$	$0.88 * 10^{6}$
100 - 120 kN	$0.40 * 10^{6}$	$0.32 * 10^{6}$
120 - 140 kN	$0.07 * 10^{6}$	$0.56 * 10^{6}$
140 - 160 kN	$0.83 * 10^{4}$	$0.66 * 10^{4}$
160 - 180 kN	$0.31 * 10^{4}$	$0.25 * 10^{4}$
> 180 kN	$0.21 * 10^{4}$	$0.17 * 10^{4}$

Based on these data the total number of axle loads on the heavy trafficked lane in axle load groups of 10 kN could be calculated. The cross distribution factor is estimated to be 0.8 (see table 6.)

Further on the following assumptions are made:

Axle load		Contact pressure
< 5 kN	Single wheels	q = 0.2 MPa
5 - 40 kN	" "	q = 0.4 MPa
40 - 80 kN	" "	q = 0.8 MPa
> 80 kN	Dual wheels	q = 0.8 MPa

The distribution in time during the periods with different temperature gradients is assumed to be as follows:

10% of the traffic during $\Delta T = + 0.05$ °C/mm
40% of the traffic during $\Delta T = + 0.03$ °C/mm
30% of the traffic during $\Delta T = 0$ °C/mm
10% of the traffic during $\Delta T = - 0.03$ °C/mm
10% of the traffic during $\Delta T = - 0.05$ °C/mm

The load repetitions for each of the considered temperature gradients are listed in table 7.

The calculations of the moments caused by the traffic are made with the help of the influence charts of Pickett and Ray.

The elastic length:

$$l = \sqrt[4]{\frac{E * h^3}{12(1 - \mu^2) k}} = 497 \text{ mm}$$

The wheelprints are supposed to be rectangles with dimensions a * b, where $a = \frac{2}{3} b$.

The number of squares on chart 2 are given in table 8.

The moments are calculated by:

$$m = \frac{q * l^2}{10^4} N_c$$

The stresses are calculated by:

$$\sigma = \frac{6 * m}{h^2}$$ and listed in table 9.

These dynamic stresses are superposed on the earlier calculated static stresses (see fig. 4).
The capability of the pavement to carry load-repetitions is given by Palmgren/ Miners rule:

Figure 4. Superposition of static and dynamic stresses.

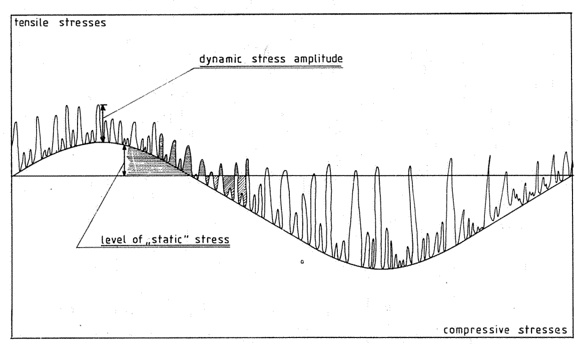


Table.7

Number of load repetitions coinciding with a temperature gradient of:					
axle load	T= 0.05°C/mm	T= 0.03°C/mm	T= 0 °C/mm	T= -0.03°C/mm	T= -0.05°C/m
< 5 kN	$2.00 * 10^7$	$8.00 * 10^7$	$6.00 * 10^7$	$2.00 * 10^7$	$2.00 * 10^7$
5 - 20 kN	$.80 * 10^7$	$3.20 * 10^7$	$2.40 * 10^7$	$.80 * 10^7$	$.80 * 10^7$
20 - 40 kN	$.80 * 10^6$	$3.20 * 10^6$	$2.40 * 10^6$	$.80 * 10^6$	$.80 * 10^6$
40 - 60 kN	$1.00 * 10^6$	$4.00 * 10^6$	$3.00 * 10^6$	$1.00 * 10^6$	$1.00 * 10^6$
60 - 80 kN	$.48 * 10^6$	$1.92 * 10^6$	$1.44 * 10^6$	$0.48 * 10^6$	$.48 * 10^6$
80 -100 kN	$.88 * 10^5$	$3.52 * 10^5$	$2.64 * 10^5$	$.88 * 10^5$	$.88 * 10^5$
100 -120 kN	$.32 * 10^5$	$1.28 * 10^5$	$.96 * 10^5$	$.32 * 10^5$	$.32 * 10^5$
120 -140 kN	$.56 * 10^4$	$2.24 * 10^4$	$1.68 * 10^4$	$.56 * 10^4$	$.56 * 10^4$
140 -160 kN	$.66 * 10^3$	$2.64 * 10^3$	$1.98 * 10^3$	$.55 * 10^3$	$.66 * 10^3$
160 -180 kN	$.25 * 10^3$	$1.00 * 10^3$	$.75 * 10^3$	$.25 * 10^3$	$.25 * 10^3$
> 180 kN	$.17 * 10^3$	$.68 * 10^3$	$.51 * 10^3$	$.17 * 10^3$	$.17 * 10^3$

Table.8

axle load group kN	kN/wheel	contact area mm2	a * b mm2	N_c cross	N_c longitudinal
< 5 kN	2	10,000	81 * 122	96	108
5 - 20 kN	6	15,000	100 * 150	144	156
20 - 40 kN	15	37,000	158 * 235	284	300
40 - 60 kN	25	31,250	144 * 216	252	268
60 - 80 kN	35	43,750	66 * 99	316	344
80 -100 kN	45	56,250	2 * 137 * 206	252	350
100 -120 kN	55	68,750	2 * 152 * 226	300	406
120 -140 kN	65	81,250	2 * 165 * 246	338	466
140 -160 kN	75	93,750	2 * 177 * 266	360	496
160 -180 kN	85	106,250	2 * 188 * 283	390	554
> 180 kN	100	125,000	2 * 204 * 306	426	620

Table.9

axle load group kN	m cross Nm/m	m long Nm/m	σ_{cross} MPa	σ_{long} MPa
< 5 kN	474	333	0.11	0.12
5 - 20 kN	1423	1541	0.33	0.36
20 - 40 kN	2806	2964	0.66	0.70
40 - 60 kN	4980	5296	1.17	1.24
60 - 80 kN	6224	6797	1.46	1.59
80 -100 kN	4979	6916	1.17	1.62
100 -120 kN	5928	8022	1.39	1.88
120 -140 kN	6679	9208	1.56	2.16
140 -160 kN	7114	9800	1.66	2.30
160 -180 kN	7706	10947	1.81	2.57
> 180 kN	8418	12251	1.97	2.87

$$\frac{n_1}{N_1} + \frac{n_2}{N_2} + \ldots\ldots \frac{n_j}{N_j} = \sum_{i=1}^{i=j} \frac{n_i}{N_i} = 1$$

In this formula n_i is the expected number of load repetitions per axle load group and N_i the allowable number of load repetitions untill rupture on each of the considered stress-levels. This stress ratio is found by deviding the performing stress by the flexural strength of the concrete.

To determine the allowable number of load repetions for each of the performing stress ratio's two different formulas have been used:

one for the tensile/tensile stress situation and the other for the tensile/compressive stress situation.

Ofcourse there are also compressive/compressive stress situations, but as the compressive stress levels are very low in comparison with the compressive strength these load repetitions can be neglected.

For the tensile/tensile stress variations:

$$\log N = 12.6 \left[1 - 0.8 \frac{\frac{\sigma_{max}}{f_{ct}} - \frac{\sigma_{min}}{f_{ct}}}{0.8 - \frac{\sigma_{min}}{f_{ct}}} \right]$$

and for the tensile/compressive stress variations:

$$\log N = 9.36 - 7.93 \left| \frac{\sigma_{max}}{f_{ct}} \right| - 2.59 \left| \frac{\sigma_{min}}{f_{cc}} \right|$$

In this formulas N is the allowable number of load repetitions for the considered stress ratio

σ_{max} = the highest stress level caused by the dynamic loads + static stress

σ_{min} = the static stress

f_{ct} = the characteristic flexural strength (see table 1)

f_{cc} = the characteristic compressive strength

The first formula is derived from a litterature-study by the Association of the Dutch Cement Industry (2) and the second from a test-program carried out by the Stevin Laboratory of Delft University of Technology.(3)

Originally this second formula is derived for pure tensile stresses, but it is the opinion of the authors that it is allowed to introduce flexural strength instead of tensile strength.

Moreover the results are on the safe side since in practice the loadrepetitions are spaced in time by longer periods of rest so that there is some regeneration. This is confirmed by some investigations where the Miner-sum showed to be bigger than 1.

Calculations have been carried out for two different places on the slab: (see fig. 5)

- in the wheel-track in the central part for cross as well as for longitudinal stresses,
- in the wheel-track near the end of the slab also for cross and longitudinal stresses.

An example of the calculations of $\sum \frac{ni}{Ni}$ is given in table 10 for the stresses in the case of the cross direction near the end of the slab.

As the load-repetitions during negative temperature gradients don't contribute to the Miner-sum they are omitted from the table.

This table must be read as follows: in the column-heads are the relevant ΔT levels given in combination with the belonging "static" stress levels (as listed in table 3). For each of the groups of axle loads in the columns 4 numbers are given:

- upper left the dynamic stress level
- upper right the allowable number of load repetitions n_i
- lower left the expected number of load repetitions N_i
- lower right $\frac{n_i}{N_i}$

As can be seen from this table the value of

$\sum \frac{n_i}{N_i}$ for all relevant stress situations is 0.929.

For the longitudinal stresses near the end of the slab $\sum \frac{n_i}{N_i} = 0.284$

For the cross direction stresses in the central part of the slab $\sum \frac{n_i}{N_i} = 0.302$

and for the longitudinal stresses $\sum \frac{n_i}{N_i} = 0.215$

Figure 5. Locations of calculated stresses.

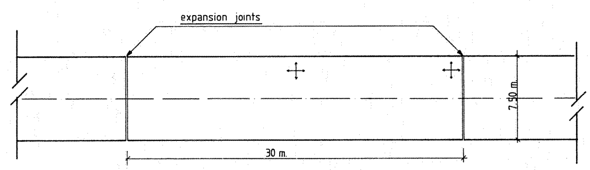


fig. 5 + calculated stresses

The conclusion can be that the design is safe. Another conclusion could be that some less longitudinal prestressing and some additional prestressing in the cross direction, especially near the ends of the slab could be profitable.

On the other hand one must realise that design methods like the described show how sensitive pavement structures are for slight changes in load patterns or concrete quality deficiencies. With a characteristic flexural strength 4.0 MPa instead of 4.4 MPa for instance the Miner-sums would have exceeded 1.

The expansion joints

The maximum temperature changes from season to season are assumed to be 50 °C. (from -15 °C to +35 °C).

Thus the maximum movement in the expansion joint will be

$$50 * 30{,}000 * 10^{-5} = 15 \text{ mm}.$$

A 50 mm neoprene joint-strip needs a maximum joint-width of 40 mm.

After shrinkage and creep of the concrete pavement about 7 mm additional joint-width can be expected.

Hence the designed joint-width is 18 mm at - 15 °C and 33 mm at + 35 °C.

At the moment of installation the joint-width depends on the actual temperature.

Spacing and diameter of the dowels are calculated in the traditional way.

4. Construction

The construction of this trial section, consisting of three diagonally pre-stressed concrete slabs measuring 30 m * 7.5 m, has taken place in October 1980.

The work was split up in 3 phases:

A. Work preparation
- sub-base
- ground anchors
- placing of moulds
- joint construction
- placing of reinforcement
- placing of anchors

B. Casting concrete
- preliminary survey plus concrete composition
- processing equipment
- subsequent processing
- quality control

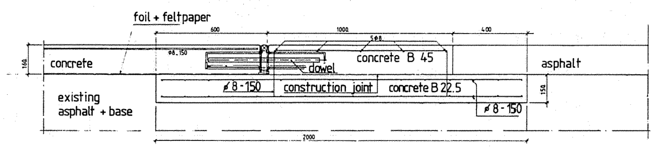
C. Subsequent stressing of reinforcement plus finishing
- stressing in 3 phases
- stressing equipment
- injection
- placing of joint profiles

A. Work preparation

The existing pavement - consisting of blast furnace slags and 120 mm gravel asphalt- concrete - was broken up and replaced by a layer of concrete rubble 0/40 mm with 80 mm gravel asphalt-concrete on top, applied in 2 layers (in respect of levelness).

At the point where the trial section joins the existing asphalt pavement ground plates were cast of reinforced concrete B 22.5 (see fig. 6)

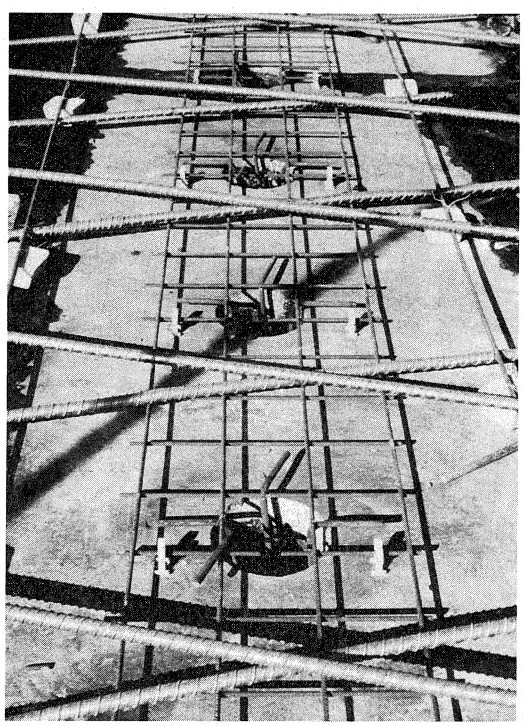
Figure 6. Reinforced ground plates.



In the centre of each prestressed slab ground anchors were fitted by means of drilling 9 holes ø 200 mm in the asphalt base. (see fig. 7)

These ground anchors ensure that a possible expansion or contraction of the prestressed slabs takes place from the centre, so that any movement in the joints will be restricted.

Figure 7. Ground anchors.



The drilling holes were reinforced and shortly before the actual casting of concrete filled up with a concrete mix and compacted.

The forms consisted of 165 mm high and 4,00 m long steel moulds with a rail profile on which the concrete-processing equipment could run.

These moulds have been adjusted very accurately, both vertically and lengthwise, in order to obtain a uniform thickness of 160 mm.

After the moulds had been placed, they were bedded in mortar in order to prevent movement c.q. subsidence during the casting of the concrete.

After the moulds had been placed the expansion joints were fixed.

These consist of two steel plates 140 mm x 10 mm with a mutual distance based upon the temperature on the day of casting.

This distance is realised by temporarily welding the two plates together with small steel plates.

These have to be removed immediately after the concrete has been cast in view of the contraction of the concrete.

The expansion joints were also provided with steel dowels ø 25 mm with mutual centre distance of 375 mm (see fig. 8)

The expansion joints have been entirely prefabricated and were placed on location in a finished state, subsequently fixed in the asphalt layer and adjusted for height (2 mm below the top of the forms).

The actual joint space between the two steel plates was filled up with jute in order to prevent penetration of concrete.

Figure 8. Expansion joint with dowels.

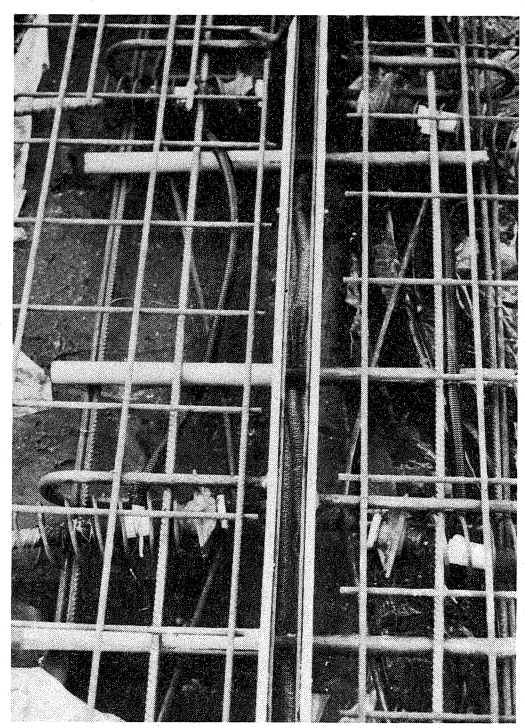


Table.10 Miner-sum for end of slab - cross direction

axle load group kN	ΔT = 0.05°C/mm σ_{min} = 1.27 MPa				ΔT = 0.03°C/mm σ_{min} = 0.74 MPa				ΔT = 0 °C/mm σ_{min} = -0.05 MPa			
< 5	1.38	$1.00 * 10^{12}$	$2.00 * 10^{7}$	0.0	0.85	$1.82 * 10^{12}$	$8.00 * 10^{7}$	0.0	0.06	$2.69 * 10^{12}$	$6.00 * 10^{7}$	
5 - 20	1.60	$1.00 * 10^{11}$	$0.80 * 10^{7}$	0.0	1.07	$2.88 * 10^{11}$	$3.20 * 10^{7}$	0.0	0.28	$6.30 * 10^{11}$	$2.40 * 10^{7}$	
20 - 40	1.93	$3.23 * 10^{9}$	$0.80 * 10^{6}$	0.0	1.40	$1.82 * 10^{10}$	$3.20 * 10^{6}$	0.0	0.61	$7.08 * 10^{10}$	$2.40 * 10^{6}$	0.0
40 - 60	2.44	$1.55 * 10^{7}$	$1.00 * 10^{6}$	0.064	1.91	$2.45 * 10^{8}$	$4.00 * 10^{6}$	0.016	1.12	$2.51 * 10^{9}$	$3.00 * 10^{6}$	0.001
60 - 80	2.73	$7.59 * 10^{5}$	$0.48 * 10^{6}$	0.632	2.20	$2.19 * 10^{7}$	$1.92 * 10^{6}$	0.088	1.41	$3.71 * 10^{8}$	$1.44 * 10^{6}$	0.004
80 - 100	2.44	$1.55 * 10^{7}$	$0.88 * 10^{5}$	0.005	1.91	$2.45 * 10^{8}$	$3.52 * 10^{5}$	0.001	1.12	$2.51 * 10^{9}$	$2.64 * 10^{5}$	0.0
100 - 120	2.66	$1.54 * 10^{6}$	$0.32 * 10^{5}$	0.021	2.13	$3.98 * 10^{7}$	$1.28 * 10^{5}$	0.003	1.34	$5.89 * 10^{8}$	$0.96 * 10^{5}$	0.00
120 - 140	2.83	$2.63 * 10^{5}$	$0.56 * 10^{4}$	0.021	2.30	$9.55 * 10^{6}$	$2.24 * 10^{4}$	0.002	1.51	$1.90 * 10^{8}$	$1.68 * 10^{4}$	0.0
140 - 160	2.93	$9.33 * 10^{4}$	$0.66 * 10^{3}$	0.007	1.43	$3.16 * 10^{6}$	$2.64 * 10^{3}$	0.001	1.61	$1.00 * 10^{8}$	$1.98 * 10^{3}$	0.0
160 - 180	3.08	$1.95 * 10^{4}$	$0.25 * 10^{3}$	0.013	2.55	$1.15 * 10^{6}$	$1.00 * 10^{3}$	0.001	1.76	$3.71 * 10^{7}$	$0.75 * 10^{3}$	0.0
>180	3.24	$3.63 * 10^{3}$	$0.17 * 10^{3}$	0.047	2.71	$3.02 * 10^{5}$	$0.68 * 10^{3}$	0.002	1.92	$1.29 * 10^{7}$	$0.51 * 10^{3}$	0.0
$\sum \frac{n_i}{N_i}$ = 0.929	$\sum \frac{n_i}{N_i}$ = 0.810				$\sum \frac{n_i}{N_i}$ = 0.114				$\sum \frac{n_i}{N_i}$ = 0.005			

Placing of the prestressing reinforcement

A system of prestressing with adhesion was selected which implies that the space between the ducts and the stressing cables are to be injected after stressing.

The stressing cables consist of ½" strands, system BBRV (1 strand = 7 wires), situated in ducts ∅ 23 mm, type Drosbach.

Before laying the strands with ducts, the underlying asphalt layer was covered with bitumen paper and plastic foil.

Each of the three concrete slabs 30 m * 7,5 m contained 20 strands, which were placed in two layers (see fig. 9)

These two layers were accurately laid out and adjusted for height by means of spacers and supports.

An intentional space has been left between the two layers of pre-stressing reinforcement in order to prevent that, during the process of injection, the injection mortar runs from one stressing-canal into another.

Figure 9. Arrangement of prestressing steel.

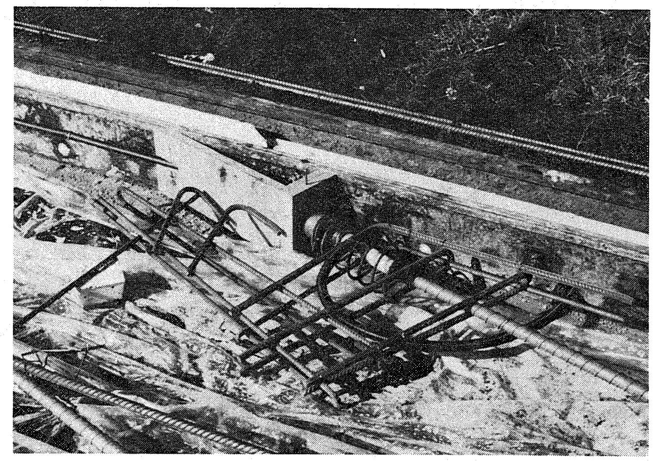
Stressing anchors.

The purpose of the system of diagonal prestressing is to keep the joints between the slabs as small as possible.

This implies that stressing takes place from the sides of the concrete slabs.

Therefore, two types of anchors are used: the so-called "blind" anchors at the heads of the concrete slab (against the expansion joints); the "active" anchors at the sides of the slab (see fig. 10)

Figure 10. Recesses for anchors.



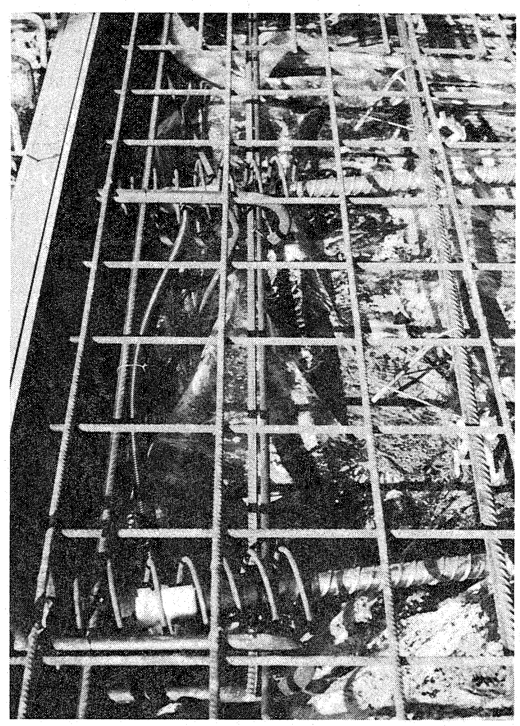
In order to be able to stress accurately in a direct line with the stressing cable, recesses had to be made within the actual forms, in which the (active) anchors were fastened (see fig. 10)

These recesses were filled up again with a special repair mortar after stressing and injection.

Since the injection of the stressing canals was only possible from the sides of the slab, the ducts near the blind anchors have been mutually connected, so that the injected mortar was returned to the adjoining canal and could run out again at the side.

In critical places, such as the blind and active anchors, a mild-steel reinforcement has been applied. (see fig. 11)

Figure 11. Mild-steel reinforcement in critical places.



B. Casting concrete

It appeared from the dimensioning that the quality of concrete should be a type B 45 with a characteristic flexural strength of at least 4.4 MPa.
Through preliminary tests a concrete mix was selected which met the abovementioned requirements.

The mix was composed as follows:

gravel	5/30 mm	941 kg/m3
porphyry	4/ 8 mm	479 kg/m3
sand	0/ 4 mm	478 kg/m3
water		127 kg/m3
Portland cement		375 kg/m3

The porphyry 4/8 mm was added in order to ensure roughness on a long-term basis.

The wet concrete was supplied on normal lorries from a ready-mix concrete plant.
The wet concrete was tipped into a container, from which it was fed into the concrete spreader by means of a mobile hydraulic excavator.

The concrete was processed by a so-called "concrete train", consisting of the following machines: (see fig. 12)

- a concrete spreader, type ABG
- a concrete compacting machine with a finishing part, type ABG
- a concrete finishing engine "right corrector", type ABG
- a brushing bridge

The processing of the wet concrete took place - as planned - in one day.
Unfortunately, the casting of the concrete was interrupted by some heavy showers, which made the concrete too wet in some places.
This resulted in a.o. the reflection of the prestressed reinforcement on the concrete surface.
During the execution of subsequent project it appeared that this reflection of the reinforcement could be substantially prevented by using a so-called oblique corrector as the final concrete finishing engine. (see fig. 13)
After finishing the concrete, the surface was provided with a broomtexture in aid of the required roughness and surface- drainage.
Subsequently, the concrete surface was sprayed with a curing-compound and covered with insulation blankets in order to prevent possible fluctuations of temperature during the hardening process.

Quality control

During production of the concrete it was regulary inspected as to composition and consistency.
Moreover, a series of test cubes was manufactured.
For each slab 4 test cubes were made to assess the final compressive strength after 28 days, and 9 test cubes to check the stressing behaviour in the initial phase in aid of the stressing.
Also 2 test beams were made per slab to assess the flexural strength.

The final results of the test cubes and beams were:

characteristic compressive strength 51,1 MPa.
characteristic flexural strength 6,91 MPa.

The levelness achieved in the trial section, despite the abovementioned reflections of the reinforcement in the surface, was satisfactory.

C. Stressing of the cables, etc

After the concrete had been cast and the first hardening had taken place, the stressing cables of the active anchors were tensioned.
The total amount of stress of 135,5 kN was applied in 3 phases. (see table 11)
The first phase of subsequent is done in order to prevent contraction-cracks in the concrete.

The stressing was carried out with a stressing-jack with a stressing-pipe whereby the amount of stress could be read out on a manometer.(see fig.14)
Moreover, the stress was checked by measuring the elongation of the strands after stressing.
All details were recorded in stress reports.

Table.11

Fase	Stressing power	Manometer	Minimum compressive strength
1 (25%)	33,9 kN	138 ato	4,3 N/mm2
2 (50%)	67,8 kN	275 ato	8,4 N/mm2
3 (100%)	135,5 kN	550 ato	20,4 N/mm2

Because of the acute angle under which the (active) anchors were projected, the moment of subsequent stressing, especially during the first phase, proved to be extremely critical.
Too late subsequent stressing could result in possible contraction-cracks in the slab.
Too early stressing could result in cracks near the anchors.
The latter has in fact occurred in a few places (see fig. 15)
The cause of this was that the actual strength of the slabs did not correspond with the hardened cubes. This implies that stressing took place to soon. This could have been prevented by using pressure-strength test methods in situ, like e.g. the Lok-test and/or B.O.test.

The damage caused by too early stressing (crumbling round the stressing-anchors) was repaired with a very fast hardening contraction-free repair mortar on a cement- basis.
After all the cables had been stressed 100%, the ducts were injected with an injection-mortar of the following composition:

50 kg Portland cement
20 lt water
0,5 kg intrusion-aid

This injection-mortar was forced into the stressing-canals by means of a plunger-pump.

On the place of the blind anchors de- aerating hoses had been fixed, which emerged trough the expansion joints.
After the injection of the ducts and the hardening of the injection-mortar, the recessing near the active anchors were filled up with a contraction-free repairmortar on a cement-basis.

Figure 12. Concrete train.

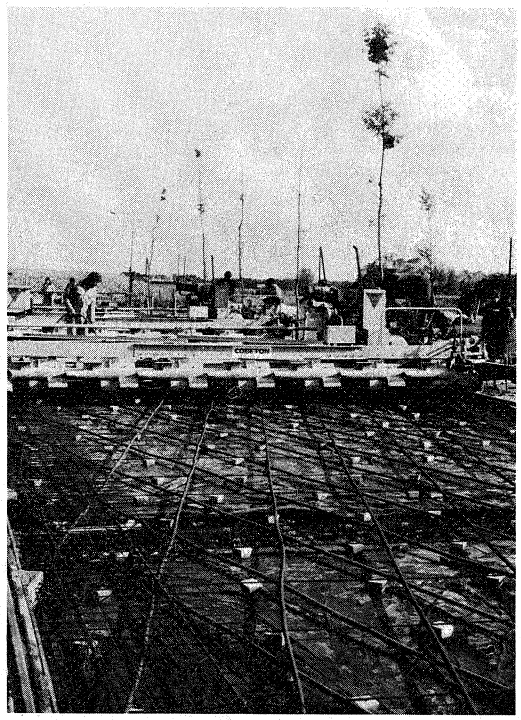


Figure 13. Texture and reflection.

Figure 14. Stressing jack.

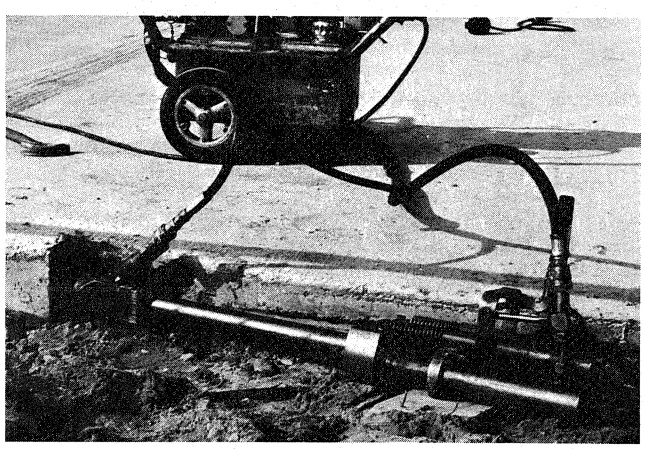


Figure 15. Damage caused by too early stressing.

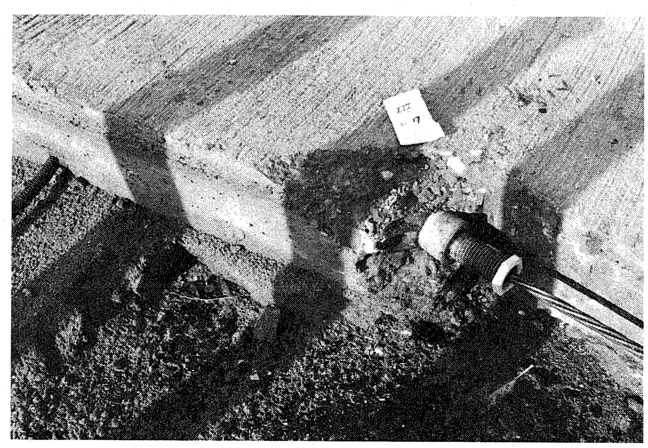
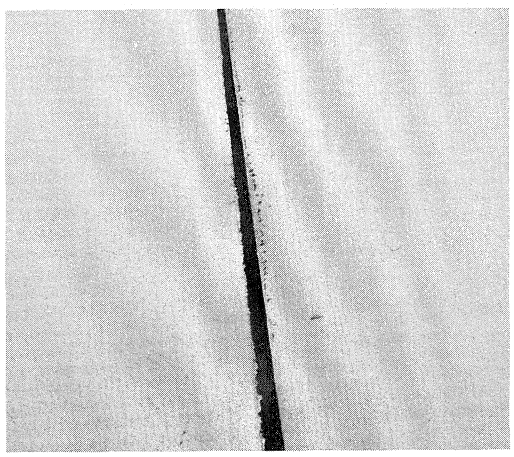


Figure 16. Expansion joint.



The part of the stressing-cable still protruding from the anchor was hereby used as a reinforcement.

As a final operation the still open expansion-joints were filled up with so-called ACME-30 neoprene profiles, wich, compressed between the steel plates, were bonded over the total width of 7,5 m. (see fig. 16)

5. Economic aspects

This aimes to compare both the initial costs as well as the predicted maintenance costs of a 10 km road-section of a 230 mm thick non-reinforced concrete pavement or a 160 mm thick prestressed concrete pavement on the same location. Numbers are based on the price level of 1983 in Amsterdam. Users costs are not taken into account.

Initial costs of non-reinforced concrete pavement.

	* Dfl. 1000,--	%
form	504	10.5
dowels	483	10.0
concrete	2,875	59.6
joints	458	9.5
various	500	10.4
	4,820	100.0

Initial costs of prestressed concrete pavement.

	* Dfl. 1000,--	%
form	504	9.8
steel, stressing, injection	1,753	34.0
concrete	2,000	38.8
joints, sides	398	7.7
various	500	9.7
	5,155	100.0

The difference of initial costs is about 7%.

For the estimation of maintenance costs the following assumptions are made:

1. the life span of both pavements is 20 years.
2. deflection measurements at the joints are carried out once per 5 years.
3. bituminous joint-filler in the non- reinforced concrete pavement is replaced once per 5 years.
4. the amount of joints to be repaired after 5, 10 and 15 years is resp. 5%, 10% and 15% for both types of pavements.
5. the amount of surface repairs is neglected for both pavements.
6. in the non-reinforced pavement after 5, 10 and 15 years resp. 0%, 5% and 10% of the slabs will show cracks that have to be repaired;
   the prestressed concrete pavement will not be cracked.

The present value of the maintenance cost is based on an interest-rate of 5%, the year of construction is 1984.

- Non-reinforced concrete pavement.

Amounts in Dfl.

	1989	1994	1999
deflection measurements	50,000	50,000	50,000
joint repair	75.000	150,000	225,000
joint cleaning, refilling	150,000	150,000	150,000
crack repair	-	25,000	50,000
traffic regulation	100,000	100,000	100,000
	375,000	475,000	575,000
	$* \left(\frac{1}{1.05}\right)^{5}$	$* \left(\frac{1}{1.05}\right)^{10}$	$* \left(\frac{1}{1.05}\right)^{15}$
present value	294,000	291,600	276,600

Total present value of maintenance costs Dfl. 862,200

- Prestressed concrete pavement.

	1989	1994	1999
deflection measurements	10,000	10,000	10,000
joint repair	15,000	25,000	40,000
traffic regulation	50,000	50,000	50,000
	75,000	85,000	100,000
	$* \left(\frac{1}{1.05}\right)^{5}$	$* \left(\frac{1}{1.05}\right)^{10}$	$* \left(\frac{1}{1.05}\right)^{15}$
present value	58,800	52,200	48,100

Total present value of maintenance costs Dfl. 159,100

Thus we can make the conclusion that the total expenses (initial costs plus present value of maintenance costs) are for
- non-reinforced concrete Dfl. 5,682,000
- prestressed concrete Dfl. 5,314,000

So however the intitial costs of the prestressed concrete pavement are about 7% higher, the total costs, including the estimated costs of maintenance will be about 7% lower.

6. Conclusions and Recommandations

This trial section, consisting of 3 diagonally prestressed slabs with a length of 30 m and a width of 7,5 m, has been constructed in order to establish whether such a system can also be applied in larger projects.

It can be said that this is possible indeed.
From a technical point of view a few aspects of the execution could still be improved.
Earlier it has been already suggested that as far as the levelness is concerned, it is recommendable to use a so-called "oblique corrector" as a final concrete-finishing machine.
In order to avoid risks, especially during the initial phase of stressing, it is necessary to carry out hardening tests with a method whereby the strength can be directly determined in-situ.

As there is a lack in large-scale and longterm experience one could say that there is an incertainty in the cost estimations. However, the small-scale test-section as described in this paper shows no need for any maintenance after 4½ years, where it has to be granted that it does not carry highway traffic.

It was the aim of this report to show that from a technical point of view it is possible to build prestressed concrete pavements and that from an economic point of view the costs wil not be prohibitive, not only on airports, but also on motor-traffic roads.

References.

1. A new sandwich construction method for prestressed concrete airfield pavements.
G.H. Kellersmann, Dienst der Publieke Werken, Amsterdam.
2nd European Symposium on Concrete Roads
Report B 8
Bern 1973

2. Vermoeiing van ongewapende betonverhardingen, dimensionering en onderzoek.
Dr. Ir. H.A.W. Cornelissen, Technische Hogeschool, Delft
Ir. M. Leewis, Vereniging Nederlandse Cementindustrie
Betonwegennieuws nr. 55, april 1984

3. Cornelissen, H.A.W.
Constant amplitude tests on plain concrete in uniaxial tension and tension-compression.
Report no. 5 - 84 - 1 1984 Stevin Laboratory
Delft University of Technology