TRANSPORT and ROAD
RESEARCH LABORATORY

Department of the Environment

TRRL REPORT LR 564

LONG-TERM STRAINS IN A SUPPORT CANTILEVER OF THE CHISWICK-BOSTON MANOR VIADUCT M4

by

R.G. Tyler BSc, CEng, MICE, MIStructE

Bridge Behaviour Division
Structures Department
Transport and Road Research Laboratory
Crowthorne, Berkshire
1973

CONTENTS

LONG-TERM STRAINS IN A SUPPORT CANTILEVER OF THE CHISWICK-BOSTON MANOR VIADUCT M4

ABSTRACT

Acoustic strain gauges and thermocouples were placed in a reinforced concrete cantilever of the Chiswick-Boston Manor viaduct (M4) during 1964 with a view to providing information on the creep and shrinkage of a normal reinforced concrete construction. Readings have been taken over 5 years following casting.

The gauges indicated a strain variation of ± 100×10^{-6} across the section during the setting of the cement. Tensile strain changes occurred on the outside of the cantilever on heating up and on the inside on cooling down. This offers an explanation for the cracking that occurs in large concrete members soon after casting.

The total measured movement in the compression flange of the cantilever, inclusive of creep, shrinkage and elastic strain, was less than 200×10^{-6} in 5 years, which is less than the value for shrinkage alone specified in CP 115. The recorded shrinkage due to moisture loss was small. It is estimated to be about 30×10^{-6} in 5 years. Creep factors, measured as the ratio of creep to elastic strain after 5 years, were 0.67 and 1.02 and creep appears to have stabilised. They compare with a range of values of 1.0 to 2.0 measured in other concrete structures recently.

1. INTRODUCTION

A support cantilever of the Chiswick-Boston Manor Viaduct (M4) was instrumented with acoustic strain gauges and thermocouples in 1964 to obtain information on the shrinkage and creep of a normal reinforced concrete structure. The cross-section of the cantilever was large, being 8 ft 4 in (2.54 m) deep by 9 ft (2.75 m) wide at its root and heavily reinforced in the tension flange. The investigation was carried out by means of acoustic strain gauges and thermocouples as in similar investigations carried out by the Laboratory[1].

2. INSTRUMENTATION

Acoustic gauges of the buried type having plastic barrels were briquetted in concrete[1] and fixed to the reinforcement in the chosen cantilever (No. 91) at two cross-sections prior to casting, one over the centre of the pier and the other about half-way along the cantilever (A-A and B-B in Fig. 1, and Figs 2 and 3). Most of the gauges were placed parallel to the length of the cantilever in order to measure the effects of bending due to the long-term dead loads, but a single gauge was placed transversely at the tip of the cantilever, just clear of the main longitudinal reinforcement in the cantilever and entirely clear of transverse reinforcement, in order to measure the free shrinkage of the concrete.

Three thermocouples were fixed down the vertical centre line of the central cross-section over the pier (Fig. 1) in order to provide an indication of the temperature within the concrete on casting. As these thermocouples did not provide sufficient information more thermocouples were fixed in another cantilever (No. 104), which was cast a few days later.

Companion specimens, 18 x 6 x 6 in (458 x 153 x 153 mm), were cast to provide a measure of free shrinkage, together with beam specimens 20 x 4 x 4 in (508 x 102 x 102 mm) and cubes of 4 in (102 mm) side for the determination of the dynamic modulus of elasticity and the crushing strength respectively. The Cawkell equipment, in which beams specimens are set in longitudinal resonance, was used for the determination of the dynamic modulus.

3. CONSTRUCTION DETAILS

3.1 Construction sequence

The column to the chosen cantilever was cast first, leaving starter bars to project into the cantilever cross head. About 3 weeks later the steel shutters to the crosshead were fixed in position followed by the reinforcement cage. The concrete was cast and the shutters left in position for about a week.

About a month after casting the precast deck beams were placed and in the following month the in-situ deck was completed. The loads transmitted to the cantilever by the various constructional operations up to the completion of the viaduct are given in Table 1.

TABLE 1

Loading of cantilever No. 91

Date	Operation	Resultant load on cantilever 91 (kN)
17.3.64	Cantilever poured	2600
15.4.64	Deck beams placed on East side	1460
16.4.64	Deck beams placed on West side	1460
2.5.64 } 13.5.64 }	Insitu deck constructed	1600
(about Sept. 1964)	Surfacing	680
	TOTAL	7800

3.2 Concrete properties

The mix used was nominally of medium strength, proportioned as follows for a 1.53 m^3 (2 cu yd) mix:

1760 kg	5 - 20 mm flint-gravel aggregate
1150 kg	sand
522 kg	cement
50 mm	slump
0.45	water/cement ratio

This gave a mean cube strength on the site at 7 days of 39.5 MN/m^2, for concrete from the chosen cantilever (No. 91), and of 35.6 MN/m^2 for the adjacent cantilever (No. 104).

The mean cube strength of 4 in (102 mm) cubes taken by the Laboratory was 51.4 MN/m^2 at 28 days for cantilever 91 and 60.3 MN/m^2 at 3 months (Table 2).

The dynamic modulus of elasticity was 47.1 GN/m^2 at 28 days and 49.3 GN/m^2 at 3 months. Allowing a reduction of about one-sixth gives a value for the instantaneous mechanical modulus, i.e., about 39 GN/m^2 at 28 days and 41 GN/m^2 (6.0 x 10^6 lbf/in^2) at 3 months.

TABLE 2

Results of tests on small specimens

Age	Crushing strength 4 in (102 mm) cubes MN/m^2	Dynamic modulus of elasticity on 20 x 4 x 4 in (508 x 102 x 102 mm) specimens GN/m^2
28 days	49.3 52.6 43.6 52.5 39.8 55.1 53.3 56.2	46.5 47.3 47.5 47.0
	Mean 51.4	Mean 47.1
3 months	60.4 59.0 59.3 62.2	48.8 49.7 49.4 49.2
	Mean 60.3	Mean 49.3

4. MEASURED STRAINS AND TEMPERATURES ON CASTING

As cleavages had been observed at the surface of the concrete around the perimeter of the reinforcement cage at the construction joint at the top of the columns soon after casting, readings were taken on the strain gauges and thermocouples in the cantilever from the time of casting with a view to finding out whether the strain pattern was also likely to cause cracking in the cantilever.

The casting of the concrete took two and a half hours and a set of zeros was taken on the strain gauges and thermocouples about half an hour afterwards. A plot of the temperature and strain changes which occurred in the first 24 days is shown in Fig. 4. These changes arose from the setting of the concrete and the associated heating and cooling over the first seven days and from the cantilever taking up its own dead weight.

The more comprehensive thermocouple readings obtained from the adjacent cantilever (No. 104) are also shown in Fig. 4, plotted to indicate the variation in temperature down the depth of the cross-section at various times after casting. Whereas the maximum temperature in the instrumented cantilever (No. 91) reached 49°C, that in cantilever 104 only reached 38°C, which was apparently caused by the mix for cantilever 91 being at a slightly higher temperature on placing than that for cantilever 104. The results for cantilever 104 show that, at the time of the peak hydration temperature 27 hours after casting, the concrete temperature at the inside of this section was about 20°C higher than at the outside and, as the measured temperature in the instrumented cantilever 91 was somewhat higher, it is likely that the difference there was about 25°C. Thus, as the inside of the section was hotter than the outside during the first day after casting, the inside would tend to expand more than the outside due to temperature rise, and during cooling on the following days would tend to contract more. This would give rise to different strains within the cross-section. During the first day a compressive strain would be expected to develop on the inside of the section and a tensile strain on the outside, whereas on cooling a reverse trend would apply, but under changed conditions as by then the concrete would be hard.

The strain curves (Fig. 4) appear to confirm this pattern; gauge 11, the only gauge near the centre of the cross-section, indicates an initial compressive strain followed by a tensile increment, whereas the other gauges located at the extreme points in the cross-section indicate a reverse movement. The interpretation of the results however must take into account the characteristics of the acoustic strain gauge. During the initial temperature rise, it is generally accepted that the coefficient of expansion of the wet mix is greater than that of the wire of the gauge owing to the presence of free water[2]. In consequence the gauge records the excess expansion of the mix over that of the gauge, plus superimposed strains arising from the stresses induced. The stiffness of the gauge tube also influences the reading[3]. Stress values would be difficult to assess during this phase as the concrete gradually sets, to harden on or just before the peak hydration temperature, and much plastic flow is possible to equalise differential expansion. At this time "plastic cracking"[4] in tension zones is possible, which probably accounts for the cleavages observed at the column tops.

On cooling, however, the hardened concrete controls the gauge movement. Moreover, the coefficient of expansion of the wire of the gauge is about the same as that of the hardened concrete, i.e. about 12×10^{-6} per °C, and in consequence, the recorded strain is a measure of the creep, shrinkage and elastic strain in the concrete. Thus a calculation of stress changes is possible if the modulus of elasticity and creep characteristics of the concrete are determined from companion specimens under the same curing conditions. In this instance for the microstrain change from -105 to -30, i.e. $+75$ tensile, as recorded by gauge 11 from day 3 to day 6 (Fig. 4), the corresponding stress change is $+75 \times 10^{-6} \times E_f$, where E_f is the effective modulus of elasticity of the concrete over three days allowing for creep, shrinkage being negligible for proper curing. Tests on small specimens in other investigations suggest that E_f may be about 20 GN/m^2 (2.9×10^6 lbf/in^2), which gives a stress increment on cooling, at the position of gauge 11, of 1.50 MN/m^2 (218 lbf/in^2) tension, which is not a large value in this instance and would be partially neutralised by any compressive stress remaining from the expansion phase. The gauges around the perimeter of the section show corresponding compressive increments of strain during the same period.

The strain gauge readings indicate that the cantilever may have been taking up strains due to its own dead weight before the shutters were stripped. Thus gauges 1, 2, 5 and 6 at the bottom of the section show greater compressive strain increments than the gauges 7, 8 4 and 10 after the first day. Gauge 12, the remaining operative gauge over the column might have been expected to behave the same way, but its reading was probably reduced by the mass of the column immediately below. Gauge 4 did not indicate expansion on the first day as gauge 10 on the other side of the section, but both gauges were in an array of closely packed 1.1/8 in (32 mm) bars and gauge 4 may have been shielded from the initial expansion effects. It did perform the same after day 1 however.

5. STRAINS AFTER STRIPPING SHUTTERS

5.1 Development of strains with time

The strain changes for 5 years following day 6 are given in Fig. 5 for the central cross-section and in Fig. 6 for the remainder of the gauges. The zero was chosen on day 6 as the steel shutters were stripped on the following day when the cantilever carried its own dead weight. In addition the strain changes due to temperature differential on casting had almost ceased, as shown in Fig. 4 by gauge 11, which is near the centre of the section and roughly on the neutral axis in bending.

The structure was almost complete 50 days after stripping the shutters: by then the precast deck beams and the in-situ deck slab had been placed (Table 1). The strains,however, steadily increased for about a year as creep developed, but thereafter stabilised, so that 5 years later the strains are very similar to those that existed after 1 year.

The gauge placed at the cantilever tip (No. 9) clear of reinforcement in order to measure free shrinkage, recorded 20 microstrain up to the time the gauge connection failed about 250 days after casting. It is not likely that the shrinkage increased very much thereafter in view of the small strain changes in the other gauges. A value of 30×10^{-6} has been taken for the shrinkage up to 5 years in the later calculations. The companion specimens however have shrunk about 250 microstrain in the 5 years after casting (Table 3). The difference in shrinkage values between the full-scale structure and the small specimens has been noted in other investigations[5,6] and is caused by the size effect.

TABLE 3

Shrinkage of companion specimens, 14 x 6 x 6 in (458 x 153 x 153 mm)
Cast 17/3/64 with cantilever

Specimen No.	Exposed outside						
	19.3.64	1.7.64	20.7.64	28.7.64	19.11.64	16.3.65	6.9.68
1	0	−61	−90	−112	−141	−90	−245
2	0	−50	−80	− 98	−132	−78	−238
	Stored in constant temperature and humidity room (20°C ± 1°C 80% R.H.)						
	19.3.64	14.4.67*	12.9.68	12.2.69			
3	0	−309	−237	−250			
4	0	−293	-	−237			

* low humidity (about 50%) caused by draining curing tanks in room for a few days.

5.2 Position of neutral axis

The measured strains have been plotted down the depth of the sections to indicate the position of the neutral axis in bending Figs 2 and 3). The measured strains for the three different time intervals were chosen as follows:

i) Strain change on adding beams and in-situ deck in the period of 35 days from 9.4.64 (day 24 after casting; see Table 4) to 14.5.64.

ii) Strain change in the two months from 23.3.64 (day 6) to 14.5.64, when the beams and in-situ deck were in position.

iii) Strain change in two years from day 6. (A two year time interval was chosen as most of the strain gauges were still operating after 2 years. Thereafter many of the gauges became open-circuit owing to failure of the particular type of connection used to extend the leads within the concrete.)

Gauge 12 was not used in plotting as it was immediately over the column top and its readings appear to have been reduced by the proximity of the mass of the column.

From the linear strain lines the neutral axis for both the instrumented sections is seen to be approximately at the step in the cross-section and alters very little with increasing time. This confirms that the shrinkage was small as a large overall shrinkage in 2 years would have caused the neutral axis to be raised significantly as compared with that obtained for loading over 35 days.

The readings may also indicate that the section remains substantially uncracked under the action of the upper tensile stress as the position of the neutral axis is even higher than the theoretical for the uncracked section, which is indicated on the diagrams for modular ratios of 6 and 10. Theoretical strains for the cracked and uncracked sections are compared with the measured values in Table 5. The theoretical moment was calculated at Section CC, at the root of the cantilever by the column edge, (Fig. 1), and was assumed to have a constant value across the width of the column. The theoretical and measured readings on the tension side (i.e. columns 6 and 7, Table 5, respectively) appear to confirm the uncracked section hypothesis, as the calculated strains for the uncracked section agree quite well with the measured. On the other hand, on the compression side the theoretical values are very much less than the measured particularly for the mid-section (columns 9 and 10, Table 5, respectively), as would be expected from the high position for the neutral axis for the measured strains. Shrinkage would be included in the measured readings on the compression side, which may account for part of the difference, and for which values of 20 to 40 microstrain are indicated by gauge 9 over the selected time intervals (Table 4); no account of this was taken in the calculations.

5.3 Creep of concrete

In view of the possibility of a progressive transfer of load to the tension steel on the upper side of the cantilever the creep of the concrete will only be considered on the lower (compression) side. The magnitude of the creep will be expressed as a creep factor measured as the ratio of creep to elastic strain in the manner of current European practice[7,8].

5.3.1 Central cross-section - bottom fibres

5.3.1.1 Elastic strain

The mean recorded strain change on the lower strain gauges (Nos 1 and 2 in Fig. 2), as the result of the addition of the beams and the in-situ deck to the structure, was 73.5×10^{-6} over 35 days (Table 4). This value of strain will include a small amount of shrinkage, as the operations took place in mid-April at the start of the drying period in the year, together with creep. The results of other investigations[5,9] suggest that, over this period of time, about four-fifths of this strain value, i.e. $0.80 \times 73.5 \times 10^{-6} = 59 \times 10^{-6}$, will be elastic strain and the remainder creep and shrinkage.

TABLE 4

Strain gauge readings
(microstrain, to a zero on placing concrete and two other arbitrary zeros)

Gauge No. Date hrs	17.3.64 1900	18.3.64 1130	18.3.64 1530	20.3.64 1105	23.3.64 1200	31.3.64	6.4.64	9.4.64	4.5.64	14.5.64	1.7.64	31.7.64	9.9.64	23.11.64	2.6.65	31.8.66	18.5.67	16.9.68	21.7.69
Days	0	1	1	3	6 0	8	14	0 17	23 42	33 52	100	130	170	245	426	881	1241	1628	1932
Central Section																			
4	-	0	0	− 18	− 44 0	+ 8	+ 26	0 + 35	+ 89	+ 64 + 99	+ 99	+ 118	+ 118	+ 118	+ 108	+ 108	-	+ 127	-
10	0	+ 67	+ 77	+ 57	+ 48 0	+ 9	+ 19	0 + 29	+ 48	+ 39 + 68	+ 88	+ 113	+ 98	+ 98	+ 98	+ 88	+ 88	-	-
11	0	−82	−103	−103	− 28 0	0	+ 6	0 + 11	0	− 5 + 6	0	− 11	− 11	− 11	− 11	− 27	− 27	− 35	− 27
1	0	+ 92	+ 87	+ 22	− 77 0	−35	−20	0 −30	−84	− 73 −103	−140	−136	−156	−173	−200	−159	−168	−182	−182
2	0	+ 32	+ 26	− 30	−113 0	−24	−10	0 −24	−74	− 74 − 98	−129	−129	−138	−147	−173	−164	−160	−168	−181
3 12	U.S. 0	+ 31	+ 26	0	− 50 0	+ 15	+ 20	0 + 35	−30	− 79 − 44	− 78	− 97	−106	−106	−101	−151	-	-	-
Mid section																			
7	0	+111	+ 81	+ 73	+ 22 0	+ 7	+ 22	0 + 29	+ 51	+ 22 + 51	+ 81	+ 66	+ 59	+ 66	+ 74	-	-	-	-
8	0	+104	+ 96	+ 48	+ 24 0	+ 8	+ 16	0 + 16	+ 16	+ 40 + 56	+ 72	+ 80	+ 80	+ 97	+ 114	+ 105	+ 114	+ 105	+ 114
5	0	+ 47	+ 41	− 11	−100 0	−20	−20	0 −29	−53	− 38 − 67	− 91	− 77	− 91	−109	−132	−109	−109	−118	−121
6	0	+ 30	+ 20	− 49	−125 0	−23	−23	0 −28	−64	− 54 − 82	−115	− 95	−100	−117	−151	−121	-	−138	−164
Transverse gauge at cantilever tip																			
9	0	+ 55	+ 48	+ 48	+ 41 0	0	+ 3	+ 14	− 7	− 7	− 7	− 28	− 28	− 21	-	-	-	-	-
	Cantilever poured				Shutters in place.	Shutters stripped 24.3.64		Deck beams placed on E. side 15.4.64. Ditto W. side 16.4.64		In-situ deck constructed 2.5.64 to 13.5.64									

TABLE 5

Calculated dead load stresses and strains at gauge positions

Modular ratio $\frac{Es}{Ec}$ (1)	I (m^4, concrete units) (2)	Theoretical position of neutral axis above base (m) (3)	Top gauge positions				Bottom gauge positions		
			Concrete stress (MN/m^2) (4)	Steel stress (MN/m^2) (5)	Theoretical strain ($x10^{-6}$) (6)	Measured strain after 5 years from day 6 ($x10^{-6}$) (7)	Concrete stress (MN/m^2) (8)	Theoretical strain ($x10^{-6}$) (9)	Measured strain in 5 years from day 6 ($x10^{-6}$) (10)
Root section - uncracked									
6	3.29	1.18	+ 3.60	+ 21.7	+ 105	+127 (1968) (gauge 4)	−3.08	− 90	−182 (gauges 1 & 2)
10	3.67	1.23	+ 3.10	+ 31.0	+ 150		−2.90	−145	
Root section - cracked									
6	1.78	0.72	0	+ 54.3	+ 263		−3.34	− 97	
10	2.57	0.88	0	+ 568	+ 274		−2.86	−136	
Mid-section - uncracked									
6	1.79	0.93	+ 1.52	+ 9.10	+ 44	+114 (gauge 8)	−1.17	− 34	−121 (gauge 5) −164 (gauge 6)
10	1.92	0.95	+ 1.38	+ 13.8	+ 67		−1.14	− 35	
Mid-section - cracked									
6	0.46	0.40	0	+ 57.9	+ 280		−1.82	− 53	
10	0.71	0.50	0	+ 53.1	+ 258		−1.46	− 71	

The bending moment at the root of the cantilever associated with the addition of the beams and in-situ deck was 5.96 MNm and that for the total construction weight was 9.20 MNm. Thus, by proportion, the total elastic increment for all the works operations, i.e. stripping and removing shutters to cantilever and adding beams, in-situ deck and surfacing, is likely to have been $\frac{9.20}{5.96} \times 59 \times 10^{-6}$, i.e. 91×10^{-6}.

5.3.1.2 Creep factor

The total measured strain on the lower gauges at the cantilever root over 5 years was 182×10^{-6}. For an elastic strain of 91×10^{-6} and shrinkage about 30×10^{-6}, this gives a net creep of 61×10^{-6} and a creep factor, measured as the ratio of creep to elastic strain, of $61/91 = 0.67$.

5.3.2 Mid-section of cantilever

5.3.2.1 Elastic strain

Following the same argument as for the root section of the cantilever, the mean recorded strain change for the lower strain gauges (Nos 5 and 6 in Fig. 3) as the result of the addition of beams and deck was 46×10^{-6} over 35 days (Table 4) of which about $0.80 \times 46 \times 10^{-6} = 37 \times 10^{-6}$ was elastic strain.

The bending moment at this section associated with the addition of the beams and the in-situ deck was 1.59MNm and that for the total construction was 2.40 MNm. By proportion this gives a total elastic increment of $\frac{2.40}{1.59} \times 37 \times 10^{-6}$, i.e. 56×10^{-6}.

5.3.2.2 Creep factor

The total measured strain for the lower gauges was 143×10^{-6}. For the elastic strain of 56×10^{-6} and shrinkage of 30×10^{-6}, the creep is 57×10^{-6} and the creep factor is $57/56 = 1.02$.

6. DISCUSSION

As the range of the strain gauge readings recorded in the six days following casting (Fig. 4) is at least half that recorded for the dead loading (Figs 5 and 6), the importance of the casting period from the point of view of cracking and the need for crack-control steel is demonstrated.

The records show that, during the setting of a large concrete member, either plastic cracking (cleavages) can be induced at the outside of the section by the rapid rise of temperature at the inside, or the more usual type of cracking can occur at the inside of the section on cooling, since the inside cools from a higher temperature than the outside. A hazard occurs when shutters are stripped too soon when cracks appear in the outside of the section due to cooling in air, or when cold water hoses are appplied on curing. It seems desirable to keep the shutters on until the section has largely cooled down in order to keep the temperature as uniform as possible across the section, as was done in this instance; there were no visible cracks in the outer surfaces of the cantilever.

The overall expansion of the mix on setting was less than has been noted in other investigations[2]. The water-cement ratio was a little higher and the cement content a little lower than the concretes previously investigated.

The implied value of 30×10^{-6} for shrinkage is small compared with values of up to 200×10^{-6} measured in other bridge structures[9], and reflects the large size of the cantilever. The creep values of 0.67 and 1.02 are also at the low end of the range, 1.0 to 2.0, so far recorded for prestressed concrete structures[10].

As the total movement, inclusive of creep, shrinkage and elastic strain, measured in the compression fibres of the instrumented section was less than 200 x 10^{-6} in 5 years, which is less than the value for shrinkage alone specified in CP 115, it is apparent that some account should be taken of size in the calculation of movement. A suggested method is given in the Joint FIB.CEB Recommendations[8] in terms of a 'theoretical thickness' of member.

7. CONCLUSIONS

1. A strain variation of ± 100 x 10^{-6} occurred across the section during the setting of the cement. The strain pattern showed that, for this type of section, cracking could either occur at the outside on heating up, or on the inside on cooling down should crack control steel be inadequate. Cracking could also occur at the outside of the section by stripping the shutters too early, thereby promoting rapid cooling of the concrete surface.

2. The total measured movement in the compression flange of the cantilever, inclusive of creep, shrinkage and elastic strain was less than 200 x 10^{-6} in 5 years, which is less than the value for shrinkage alone specified in CP 115. The recorded shrinkage due to moisture loss was small. It is estimated to be about 30 x 10^{-6} in 5 years.

3. The creep factors for the two instrumented sections, measured as the ratio of creep to elastic strain after 5 years of service, are estimated to be 0.67 and 1.02 for the compression side of the two instrumented sections, and the creep appears to have stabilised. They compare with a range of values of 1.0 to 2.0 measured in other concrete structures recently.

4. The measured values of tensile strain at the top of the instrumented sections, and the corresponding positions of the neutral axis, suggest that the tension flange of the cantilever has remained uncracked in service.

8. ACKNOWLEDGEMENTS

The author is grateful to Mr M W R Capps and Miss M Emerson of the Laboratory's staff, who carried out the temperature measurements, to Mr G R Bunn who assisted in the strain gauge readings, and to Sir Alexander Gibb and Partners, who supplied details of the cantilever. The Report was prepared in the Bridges Behaviour Division of the Structures Department.

9. REFERENCES

1. TYLER, R G. Developments in the measurement of strain and stress in concrete bridge structures. Ministry of Transport RRL Report No. 189, Crowthorne, 1968.

2. TYLER, R G. An interpretation of the results from vibrating wire strain gauges in fresh concretes. Highway Research Record No. 218, Highway Research Board, Washington, D.C., 1968.

3. BROWNE, R D. Properties of concrete in reactor vessels. Conf. on prestressed concrete pressure vessels. Proc. Inst. Civ. Engs. March, 1967. London.

4. HUNT, J G. Temperature changes and thermal cracking in concrete pavements at early ages. Technical Report No. 42.460, Cement and Concrete Association, April 1972. London.

5. TYLER, R G. Determination of stress in concrete structures. Site method used in precast unit for Western Avenue Extension. Civ. Eng. and P.W. Rev. *64* Nos 755-6. June and July 1969. Further notes *66* Nos 779 and 780. June and July 1971.

6. HANSEN, T C, and A H MATTOCK. Influence of Size and Shape of Member on the Shrinkage and Creep of Concrete, A.C.I. Journal, Proceedings *63*, No. 2, February 1966.

7. GERMAN STANDARD DIN 4227. Spannbeton, Richtlinien fur Bemessung und Ausfuhrung (Prestressed Concrete, Rules for Design and Manufacture) Berlin 1953. (Translation through B.S.I., London.)

8. FEDERATION INTERNATIONALE DE LA PRECONTRAINTE, COMITE EUROPEEN DU BETON. International recommendations for the design and construction of concrete structures. Cement and Concrete Association, London. June, 1970.

9. TYLER, R G. Full-scale tests on Mancunian Way, Suppl. to Proc. Inst. Civ. Engrs. Paper 7196S, May, 1969.

10. PRICE, W I J, and R G TYLER. Effects of creep, shrinkage and temperature on highway bridges in the United Kingdom. Proceedings, I.A.B.S.E. Conference on the Design of Concrete Structures for Creep, Shrinkage and Temperature. Madrid, 1970.

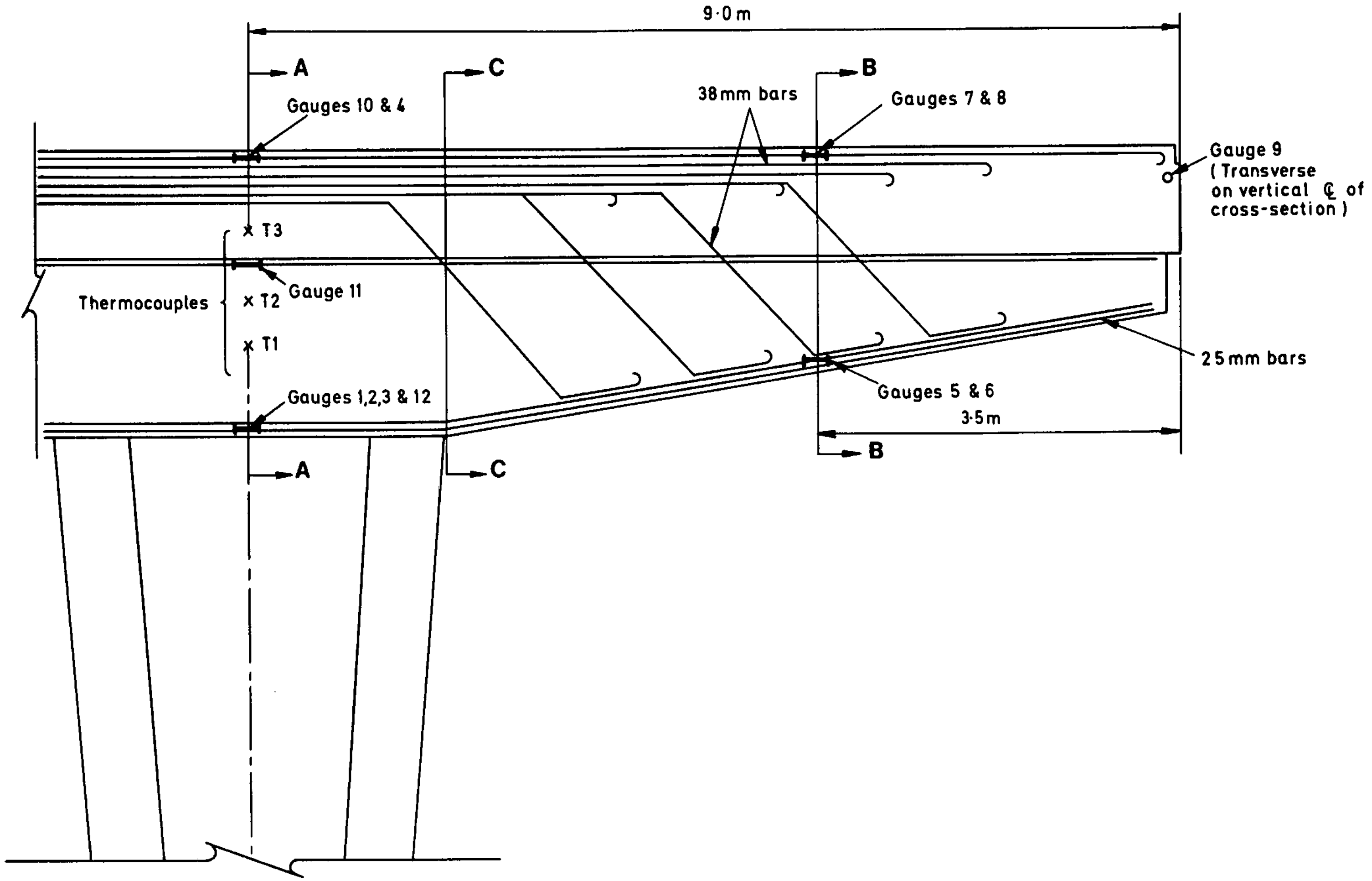

Fig. 1 ELEVATION OF CANTILEVER SHOWING INSTRUMENTATION

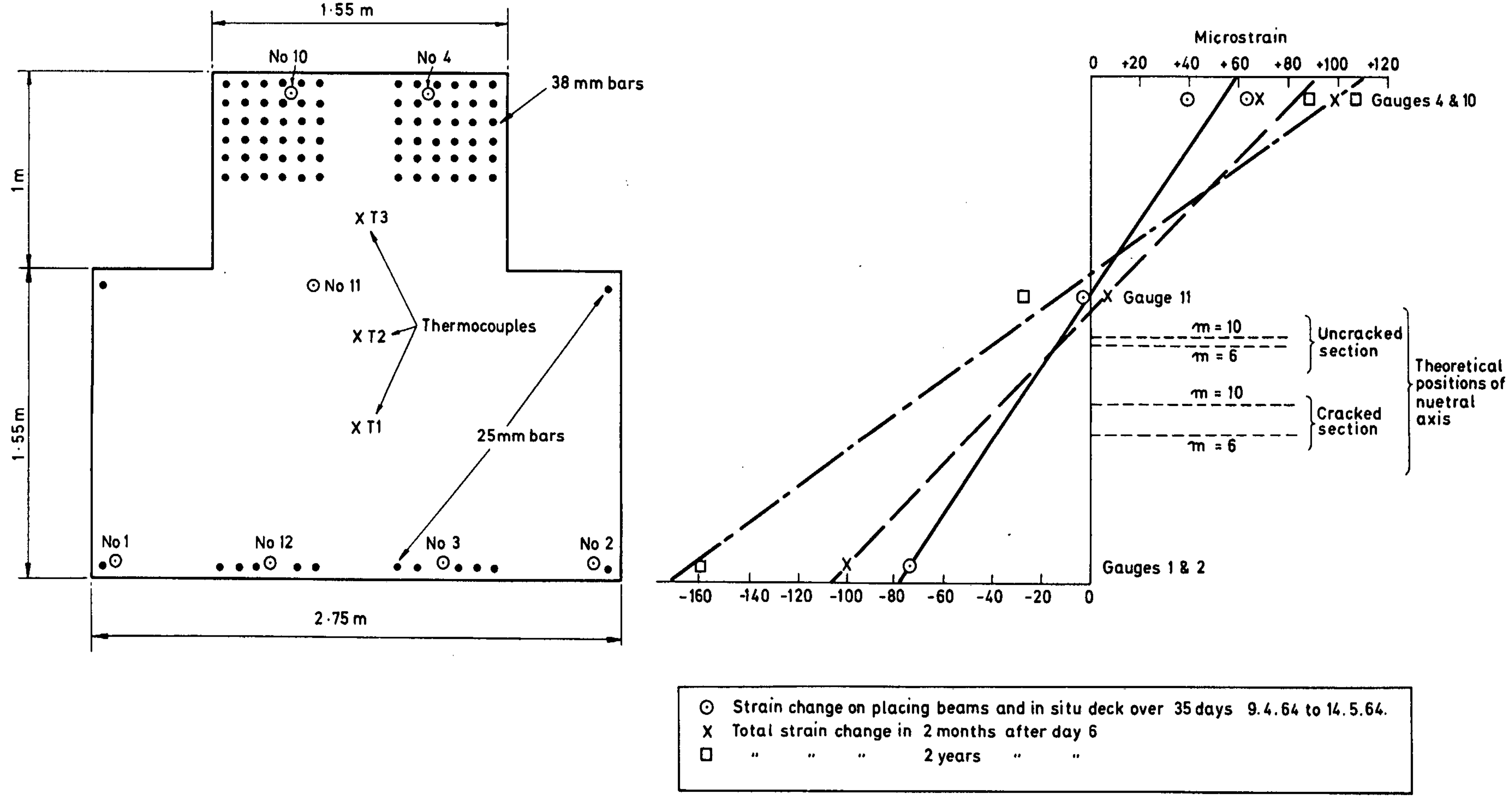

Fig. 2. STRAIN DISTRIBUTION AT CENTRAL SECTION A-A

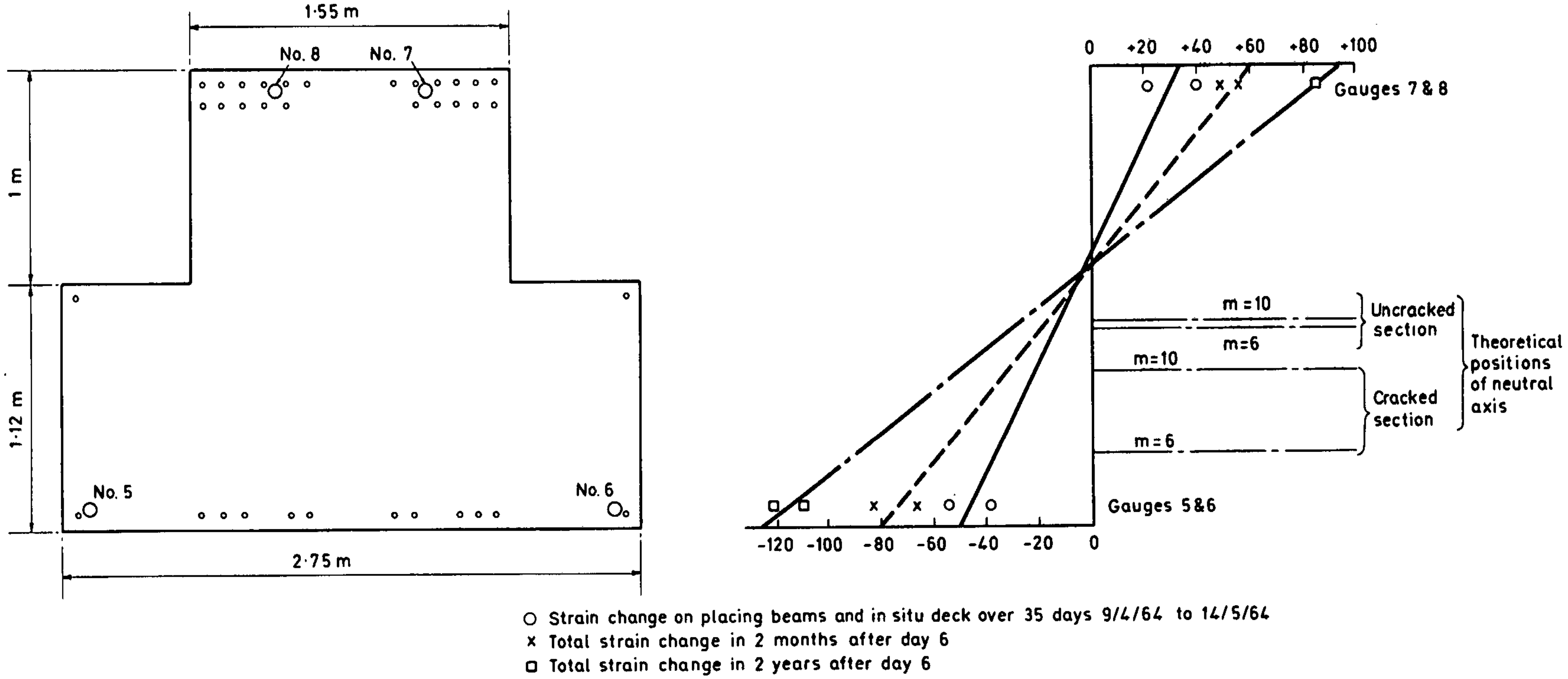

Fig. 3. STRAIN DISTRIBUTION AT MID-SECTION B-B

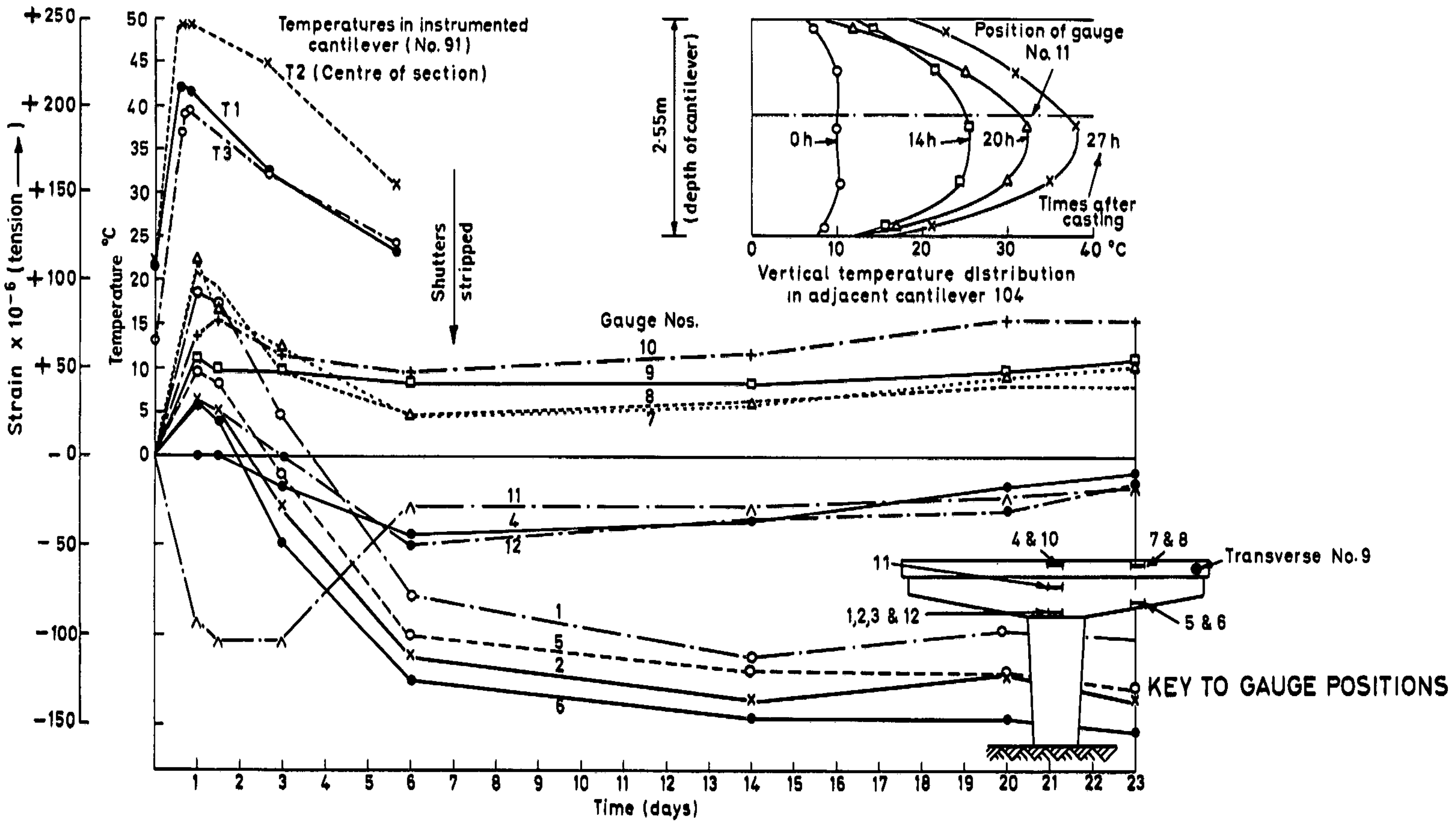

Fig. 4. STRAINS AND TEMPERATURES AFTER CASTING

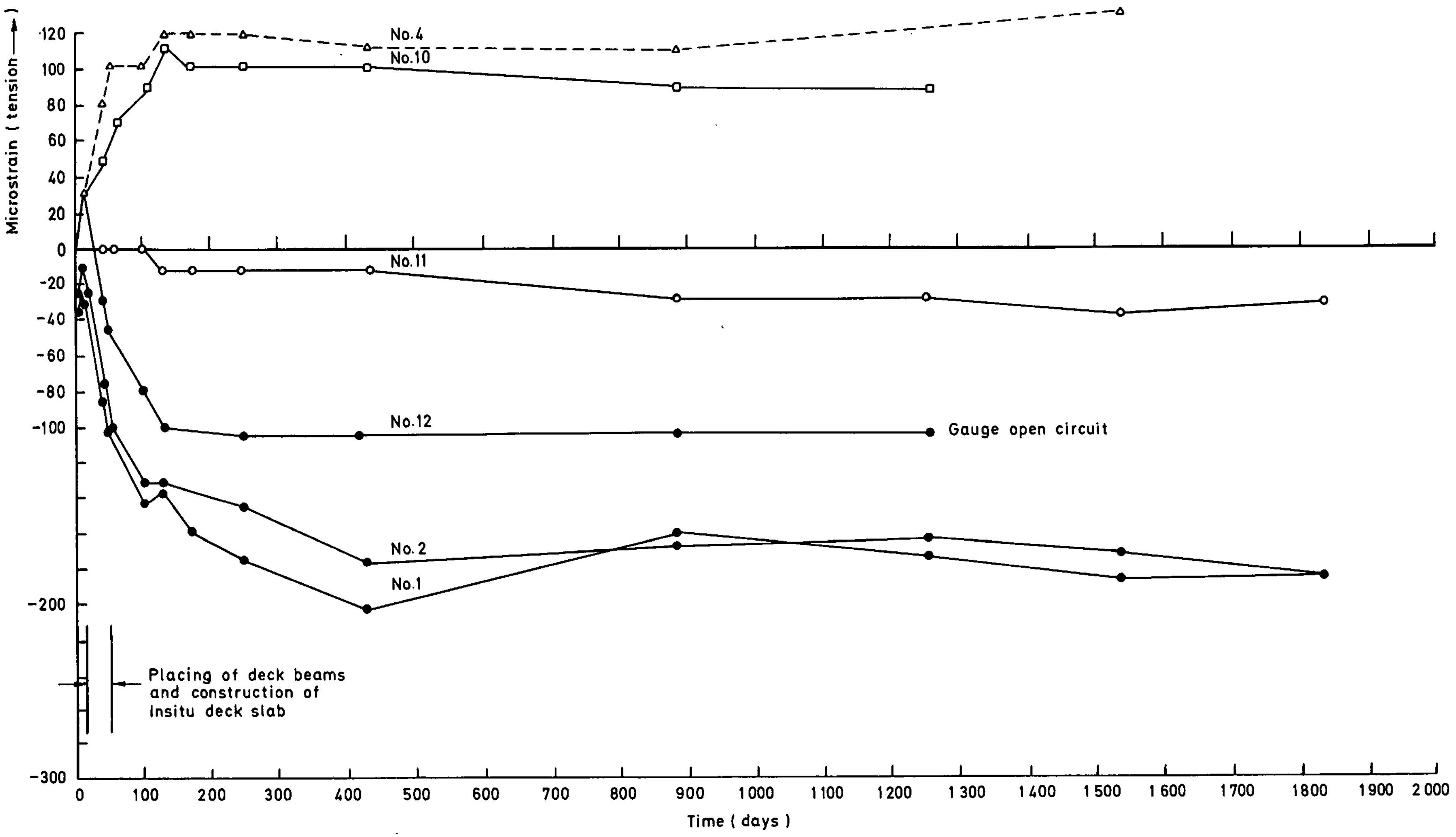

Fig. 5 STRAIN CHANGES AT ROOT CROSS SECTION (SECTION A-A) FOLLOWING STRIPPING OF SHUTTERS

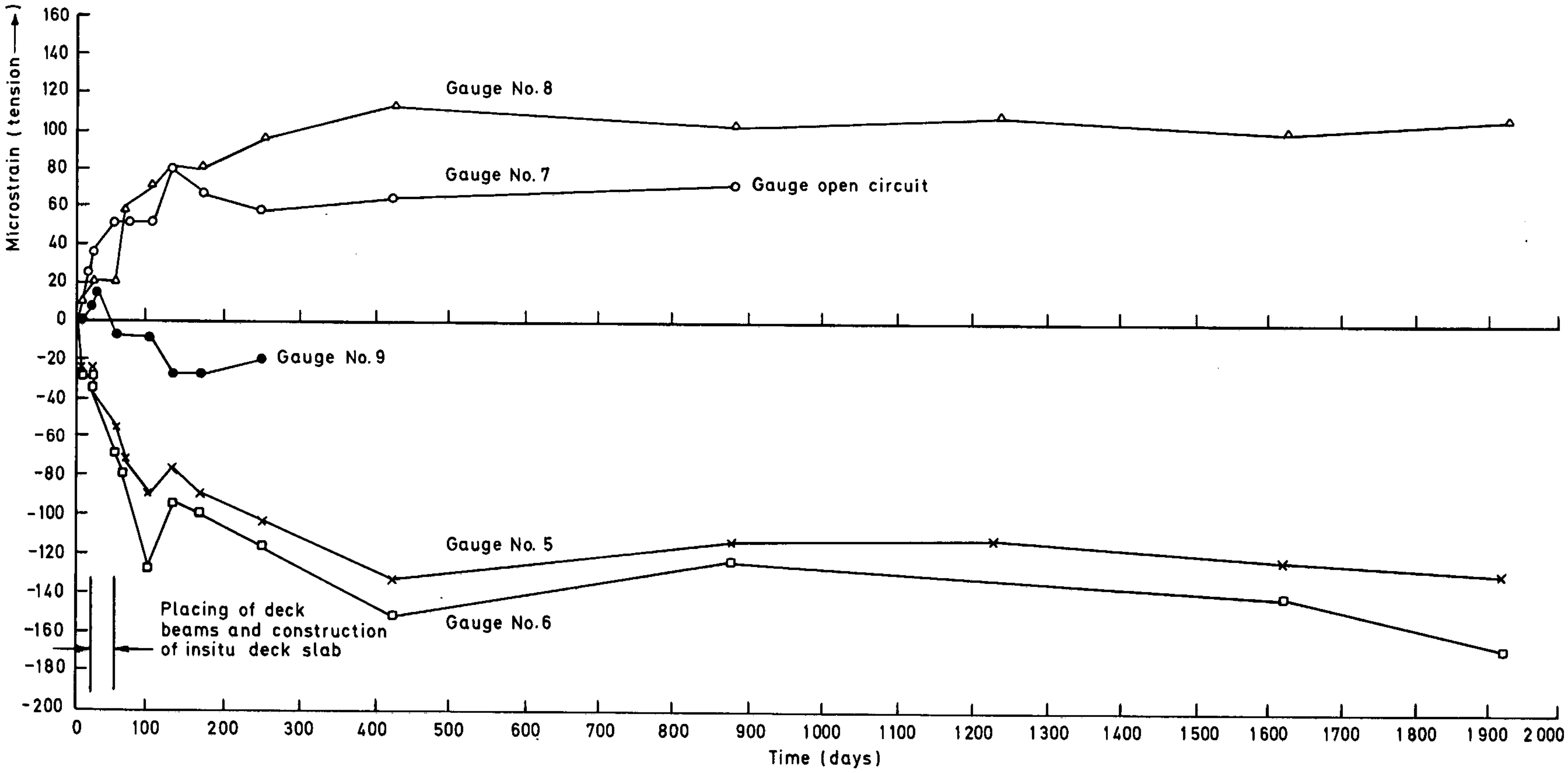

Fig. 6. STRAIN CHANGES IN CANTILEVER ARM FOLLOWING STRIPPING OF SHUTTERS (SECTION B-B)

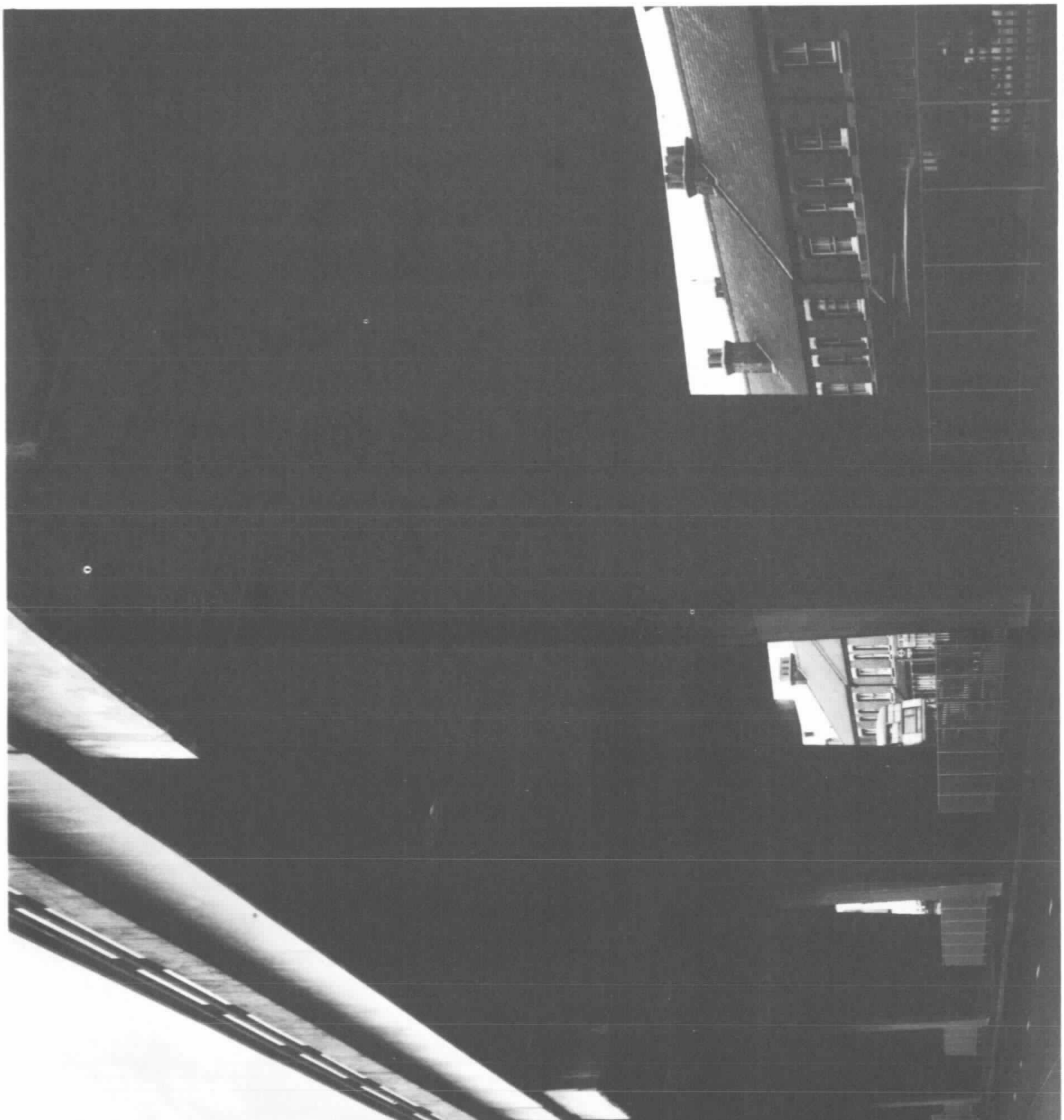

PLATE 1

The Chiswick overhead road

Neg No B 3803/65

(1430) Dd635221 3,250 7/73 H P Ltd, So'ton G1915
PRINTED IN ENGLAND

ABSTRACT

Long-term strains in a support cantilever of the Chiswick-Boston Manor viaduct M4: R G TYLER, BSc, CEng, MICE, MIStructE: Department of the Environment, TRRL Report LR 564: Crowthorne, 1973 (Transport and Road Research Laboratory). Acoustic strain gauges and thermocouples were placed in a reinforced concrete cantilever of the Chiswick-Boston Manor viaduct (M4) during 1964 with a view to providing information on the creep and shrinkage of a normal reinforced concrete construction. Readings have been taken over 5 years following casting.

The gauges indicated a strain variation of ± 100×10^{-6} across the section during the setting of the cement. Tensile strain changes occurred on the outside of the cantilever on heating up and on the inside on cooling down. This offers an explanation for the cracking that occurs in large concrete members soon after casting.

The total measured movement in the compression flange of the cantilever, inclusive of creep, shrinkage and elastic strain, was less than 200×10^{-6} in 5 years, which is less than the value for shrinkage alone specified in CP 115. The recorded shrinkage due to moisture loss was small. It is estimated to be about 30×10^{-6} in 5 years. Creep factors, measured as the ratio of creep to elastic strain after 5 years, were 0.67 and 1.02 and creep appears to have stabilised. They compare with a range of values of 1.0 to 2.0 measured in other concrete structures recently.

ABSTRACT

Long-term strains in a support cantilever of the Chiswick-Boston Manor viaduct M4: R G TYLER, BSc, CEng, MICE, MIStructE: Department of the Environment, TRRL Report LR 564: Crowthorne, 1973 (Transport and Road Research Laboratory). Acoustic strain gauges and thermocouples were placed in a reinforced concrete cantilever of the Chiswick-Boston Manor viaduct (M4) during 1964 with a view to providing information on the creep and shrinkage of a normal reinforced concrete construction. Readings have been taken over 5 years following casting.

The gauges indicated a strain variation of ± 100×10^{-6} across the section during the setting of the cement. Tensile strain changes occurred on the outside of the cantilever on heating up and on the inside on cooling down. This offers an explanation for the cracking that occurs in large concrete members soon after casting.

The total measured movement in the compression flange of the cantilever, inclusive of creep, shrinkage and elastic strain, was less than 200×10^{-6} in 5 years, which is less than the value for shrinkage alone specified in CP 115. The recorded shrinkage due to moisture loss was small. It is estimated to be about 30×10^{-6} in 5 years. Creep factors, measured as the ratio of creep to elastic strain after 5 years, were 0.67 and 1.02 and creep appears to have stabilised. They compare with a range of values of 1.0 to 2.0 measured in other concrete structures recently.