Effects of creep, shrinkage and temperature on highway bridges in the United Kingdom

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L'influence du fluage, du retrait et de la température sur les ponts routiers en Grande Bretagne

Der Einfluß des Kriechens, des Schwindens und der Temperatur auf Straßenbrücken in Großbritannien

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1. Introduction

The scarcity of information on the effects of creep, shrinkage and temperature on the behaviour of actual bridges encouraged the Road Research Laboratory, with the co-operation of many enlightened designers, to undertake measurements on certain bridges built in the United Kingdom since 1962. In this way results of direct value to the practising designer were obtained quickly and formed the basis for the development of laboratory and theoretical methods for the estimation of movements. The main conclusions hitherto are summarised in this paper.

2. Early age movements

For the type of mix used in the United Kingdom, an expansion normally occurs during the setting of the concrete up to the time of the peak temperature of hydration, which is usually from 12 to 24 hours after casting. This is followed by a thermal contraction in the hardened concrete together with shrinkage due to moisture loss.

The characteristic temperature and movement curves are illustrated in Fig. 1, which shows the results obtained from a selection of different types of acoustic strain gauge in an insulated concrete block in the laboratory[1]. A flint-gravel aggregate was used with a water : cement ratio of 0.4.

The shrinkage movements following the peak hydration temperature are shown to be small but it is worth noting that the thermal contraction of the concrete for the drop in temperature of 20°C following setting, for a coefficient of expansion of 13×10^{-6}, was approximately 260×10^{-6}, which is much greater than the shrinkage during the same period. It follows, therefore, that quite small temperature changes in concrete on the full-scale will produce comparatively large thermal strains between one part and another, which could give rise to cracking a few days after casting.

The full-scale condition is illustrated by the results from a large reinforced cantilever[2], having a cross section 2.55 m deep by 2.75 m wide at the root, constructed on the M4 Motorway at Chiswick London. Because of the size of the cross section, the temperature at the centre rose more rapidly than at the outside as indicated by the temperature curves (Fig. 2). The strain gauge at the centre of the cross section, No. 11, indicated a different stress pattern from the remainder which were around the perimeter. During the first day the centre of the section tended to expand more rapidly than the outside and gauge No. 11 indicated compression as the centre was restrained by the remainder of the concrete. During cooling, on the other hand, tension is induced at the centre and compression at the outside, as indicated by the remainder of the gauges. It is likely that there was a residual tensile strain at the centre of the cross section.

In contrast to the large thermal strains soon after casting, the shrinkage strain due to moisture loss, was only about 30×10^{-6} after 1 year from the time of cooling down following casting[2].

3. Shrinkage arising from moisture loss

On the full-scale structure, there is difficulty in separating out shrinkage from creep, while small specimens cast with a structure indicate more shrinkage than the structure itself because of the size effect[3,4]. On the Mancunian Way, a continuous prestressed concrete structure[5], a precast segment, which was reinforced in the same way as the structure, was instrumented with 23 acoustic strain gauges buried in the concrete and left on an exposure site at the Laboratory to indicate shrinkage[6]. The unit is shown in Fig. 3, together with the smaller specimen slabs and columns used for more general studies of shrinkage and creep effects. A limestone aggregate was used for the concrete with a water : cement ratio of 0.38.

Curve 1 (Fig. 4) shows the mean movement of the unit which is actually a small expansion over a period of 5 years from the time of casting. Small specimens on the other hand, shrank up to 300×10^{-6} on outside exposure. It can be concluded, therefore, that the size effect was large. However, for other aggregates on other structures, shrinkage has been indicated in similar experiments. The readings for an unreinforced slab cast with the Western Avenue Extension, a new overhead road in London, and having the same thickness as the root of the cantilever of the structure, indicate a small shrinkage of about 20×10^{-6} after one year (Fig. 5). This is continuing. A sea-dredged gravel was used in the concrete mix with a water : cement ratio of 0.38. The shrinkages of small specimens, 508 x 102 x 102 mm, waterproofed in various ways (Fig.3), are also given in Fig. 5. It is seen that the size effect is large as the peak shrinkage in an unwaterproofed specimen (curve 5) was 100×10^{-6} in 1 year. The seasonal effect is shown, i.e. the tendency of the concrete to expand in the winter following shrinkage in the summer. This effect is also observed in full-scale structures. (Curves (2) and (3) in Fig. 4; curves (1) and (2) in Fig. 7).

The results of a similar investigation for the new London Bridge[7] are shown in Fig. 6. Readings from a slab 1.5 x 1.5 x 0.3 m cast with a unit of the structure and reinforced in the same way (curve 1) are compared with readings obtained in the unit itself (curves 2, 3 and 4) and also with small specimens (curves 5, 6 and 7). For this concrete there is a greater tendency to shrink, the peak reading in the unwaterproofed specimen being 240×10^{-6} in the first year following casting. The mix included a greater fraction of sand than for the mixes for the Mancunian Way and Western Avenue, which is likely to account for the greater shrinkage[8].

While it is a little early to predict the final conclusions for the magnitude of shrinkage of full-scale bridge structures in the United Kingdom constructed to British Codes of Practice, it is likely that this will be in the range 50 to 200×10^{-6} at 10,000 days with the possible exception of limestone concrete, which may indicate no shrinkage under most exposure conditions. The small shrinkage values observed on the full-scale as compared with other countries may be attributed to the mild and humid British climate.

4. Creep movements

A preliminary analysis of the records of total strain observed at a section of one of the main cantilevers of the Medway Bridge has indicated that the creep factor, following the method of DIN 4227, is likely to be approximately 2.0 at 10,000 days when the shrinkage is 100×10^{-6}. Following the completion of the bridge the total strain for the top and bottom slabs (curves 1 and 2 Fig. 7) show that the sum of the creep and shrinkage is proportional to the logarithm of time, although it must be remembered that the prestressing force is decreasing because of relaxation in the prestressing bars. As the shrinkage of small specimens, 710 x 152 x 152 mm, housed within the box girders of the bridge (curve 3), is nearly equal to the sum of the elastic strain, shrinkage and creep for the loaded bottom slab, it is confirmed that the size effect between small specimens and the full-scale structure is large.

For the Mancunian Way (Fig. 4), a preliminary analysis shows that $\phi = 1.0$ approximately at 10,000 days and again total strain appears to be proportional to the logarithm of time. There was a static period during the first winter but the low value of ϕ is attributed to the use of limestone aggregate for the concrete. Flint-gravel was used for the Medway Bridge. The preliminary analysis of the results from other bridges suggests that ϕ is likely to be in the range 1.0-2.0 for the British climate at 10,000 days.

5. Correlation of movements between small specimens and the full-scale

It has been found that small specimens kept outside are useful in predicting movements on the full-scale as the effect of climate is allowed for, although the movements in the specimens are larger because of the size effect. To correct for the size effect, attempts were made to reduce the magnitude of the movements in the specimens by waterproofing them. In this way creep and shrinkage on the full-scale could be predicted by loading small specimens.

Using specimens 508 x 102 x 102 mm, waterproofing was first carried out by leaving strips untreated down two opposite faces in such a way that the volume/surface area ratio was the same as that of the structure. It was found that these specimens moved more than the structure (curve 4, Fig. 5). Waterproofing was later carried out by banding the specimens with a waterproofing membrane around their centres, in such a way that the depth of waterproofing was equal to the mean thickness of the box sections of the bridge. This gave similar shrinkage movements (curve 3 as compared with curve 1, Fig. 5 and curves 5 and 6 as opposed to curves 1, 2, 3 and 4, Fig. 6). It was inferred that because shrinkage movements were the same, creep movements in loaded specimens would also be the same.

Having achieved similar movements in the specimens to those in the full-scale structure, a method was devised for the determination of stress in concrete structures[9]. Specimens were loaded in creep rigs (Fig. 8) to give the same strain at all times as in an element of the structure by incrementally adjusting the load in the rigs. The stress in the specimen was then the same as that in the structure (Fig. 9).

6. Temperature Movements and Stresses

Temperature movement in bridges can be divided into two main components. Axial movement along the neutral axis and rotational movement caused by thermal bending. The resultant thermal stresses, f, developed in a beam with the temperature, T, varying as a function of its depth, T (z), is given by:-

$$f = \alpha E T(z) + \frac{1}{A} \int \alpha E T(z) b(z) dz + \frac{z}{I} \int \alpha E T(z) b(z) z dz \qquad (1)$$

where α = thermal coefficient of expansion

E = elastic modulus

$T(z)$ = temperature change as a function of beam depth

A = cross sectional area of beam

$b(z)$ = Width of the beam as a function of its depth

I = second movement of area of the section about the transverse axis through its centroid.

The datum for assessment of axial and bending thermal strains is the strain distribution derived from the actual temperature distribution, and is given by the first term. This implies that non-linearity of thermal gradients introduces stresses which are algebraically summed with axial and rotational stresses.

The middle term, divided by E, determines the axial movement: it disappears from equation (1) when axial movement is completely restrained. The last term divided by E, calculates the rotational movement: it drops out when the beam is restrained in bending. The full-scale investigation has yielded the following general conclusions:-

6.1 The thermal coefficient of expansion, determined from measurements of movements at expansion joints and the distribution of temperatures in cross sections of bridge superstructures varied between $(7.5 \pm 1) \times 10^{-6}$ for the Mancunian Way (limestone aggregate concrete) to $(13 \pm 1) \times 10^{-6}$ per ^{o}C (Hammersmith Flyover, flint-gravel aggregate concrete). Concretes having granite aggregates gave intermediate values. Tests on small specimens gave a similar range of values.

6.2 Over a 24 hour period, the mean temperature of the superstructure is lowest at 08.00 ± 1 hour (GMT) and the overall length is a minimum; the superstructure reaches its highest mean temperature and maximum length at 18.00 ± 1 hour (GMT).

6.3 The mean temperature of the superstructure at about 08.00 is usually within ± $3^{o}C$ of the air temperature in the shade at the same time. This is the only time when the bridge temperature correlates regularly with the air temperature and this fact is worth noting for setting bearings and expansion joints. Because of this correlation it follows that seasonal and weekly fluctuations in bridge temperature follow closely the changes in the mean air temperature[10]. Also at this time the temperature is approximately uniform through the superstructure. Because of thermal lag in the mass of the bridge structure the diurnal range of concrete bridge movements in the United Kingdom is unlikely to exceed the equivalent of a 6^{o} change in bridge temperature above the minimum at 0800 hrs even when the diurnal fluctuation is $20^{o}C$ above the same minimum.

6.4 Both experimental and theoretical results indicate that the range of extreme mean temperatures of the superstructure is approximately the same as the range of extreme shade temperature in the same area[11]. It should be emphasized that this conclusion applies only to the extreme values and cannot be used to determine mean temperatures of the superstructure at a particular time. However, it does enable the range of longitudinal movements of a bridge in a particular location to be calculated from the data on shade air temperature for that location. The actual values for the range will depend on the period to be considered in design and the choice of period will be governed by the seriousness of the consequence of failure. In the United Kingdom, main members of the superstructure will usually be assumed to have a design life of 120 years and it will be appropriate when guarding against structural damage to consider what the extreme bridge temperature range over this period is likely to be. For the south-eastern part of the country, a range of $-20^{o}C$ to $+40^{o}C$ is not likely to be exceeded. When damage due to temperature extremes is likely to be confined to non-structural finishes or to components whose failure have no serious consequences, then a shorter period of say 20 years may be acceptable. The corresponding range of extreme mean temperatures then becomes $-15^{o}C$ to $35^{o}C$ for design purpose. This introduces a probabilistic concept into the specification of extreme temperature ranges, but the amount of data available is inadequate for a precise statement of probability of unserviceability or failure.

6.5 A theoretical method can be used for estimating extreme ranges of mean temperatures of superstructures[11]. It can also be used for predicting extreme temperature distributions and gradients across a section of a box girder. The method assumes that the mean temperature across a section is a minimum at 08.00 (GMT) and that there is no temperature gradient at this time. These assumptions are justified by the observations in paragraphs 6.1 and 6.2 above. Starting from 08.00, the temperature distribution in the concrete is then calculated for subsequent intervals of time by solving iteratively the differential equation for the transient conduction of heat within a solid. The boundary conditions at the upper surface of the bridge deck are given by the energy balance between incoming solar radiation, the heat transfer between deck surface and the air, and the conduction of heat into the surface of the deck. On the underside of the superstructure, the heat lost to the air by convection is equated to the heat conducted into the concrete. The heat exchange between the internal surfaces of the box section is taken into account. To solve the differential equation numerically, requires that the variation of maximum solar radiation and maximum air temperature with time, for one day, is known and that the absolute maximum for both occur on the same day. This coincidence has not yet been recorded, so that there will be a tendency for thermal ranges and gradients to be overestimated by the theoretical method.

6.6 Rotational movements give rise to such effects as additional longitudinal movements away from the neutral axis, e.g. on joint sealants; the drooping of cantilevers; and the inducing of significant hogging moments on hot days. The magnitude of thermal rotational movement is determined from the third term in equation (1) and the resulting stresses for various degrees of restraint are shown in Fig. 10 for a box section, on a very hot afternoon. In continuous structures, the hogging tendency is restrained by continuity at the supports, and this leads to the redistribution of moments shown in Fig. 11. The resultant stresses, determined by integration of equation (1), are shown for Sections B and C in the same Figure. At Section A, conditions approach the distribution of stress without restraints, shown in Fig. 10; whilst at Sections D and E conditions approximate to the case of full restraint in bending, with expansion allowed.

Experimental results and calculations indicate that the thermal stresses in box structures, both longitudinally and transversely, are nearly as large as the stresses induced by the design loading. They may, therefore, have a significant effect on the design of concrete sections and their reinforcement.

7. Acknowledgements

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8. References

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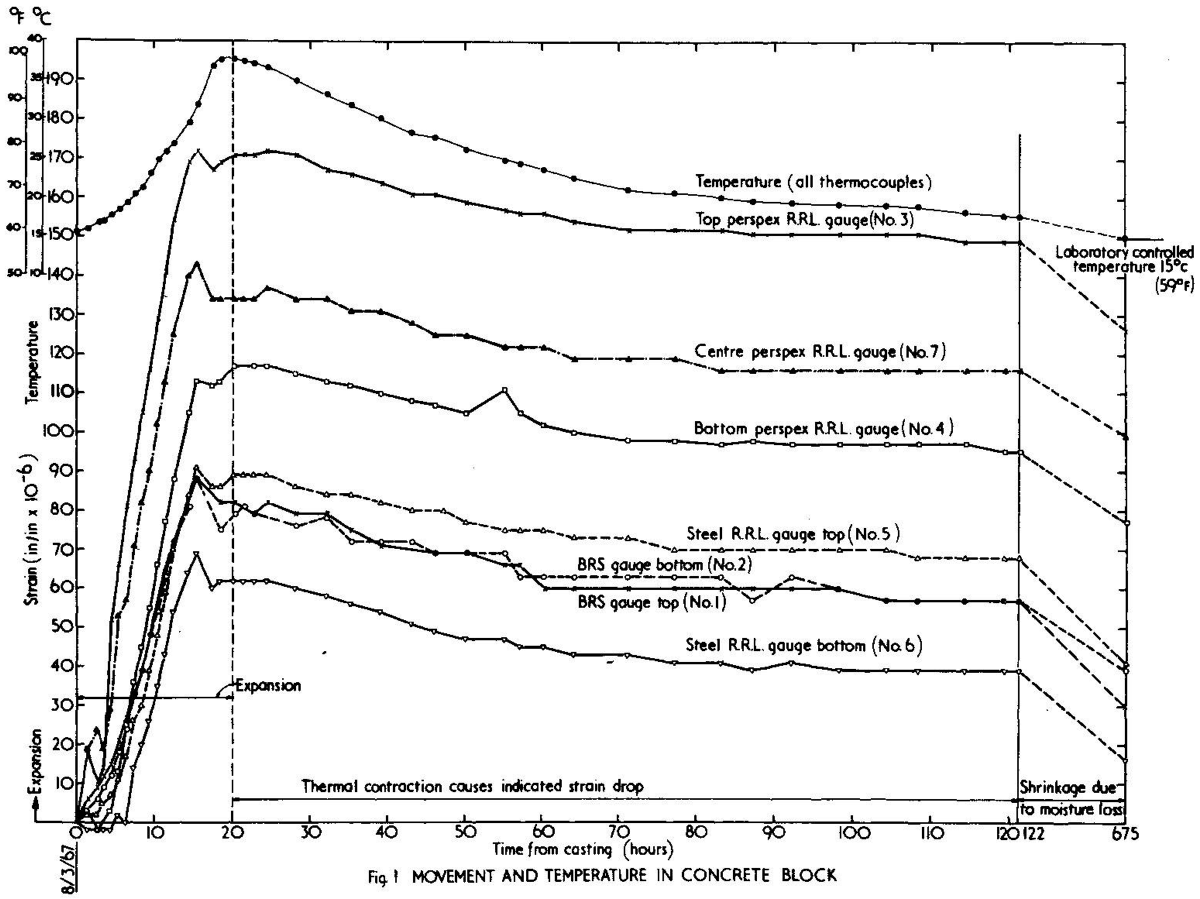
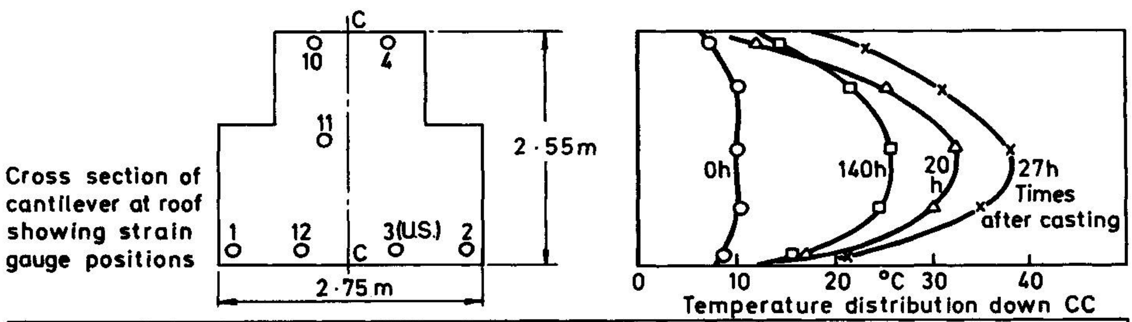


Fig. 1 MOVEMENT AND TEMPERATURE IN CONCRETE BLOCK



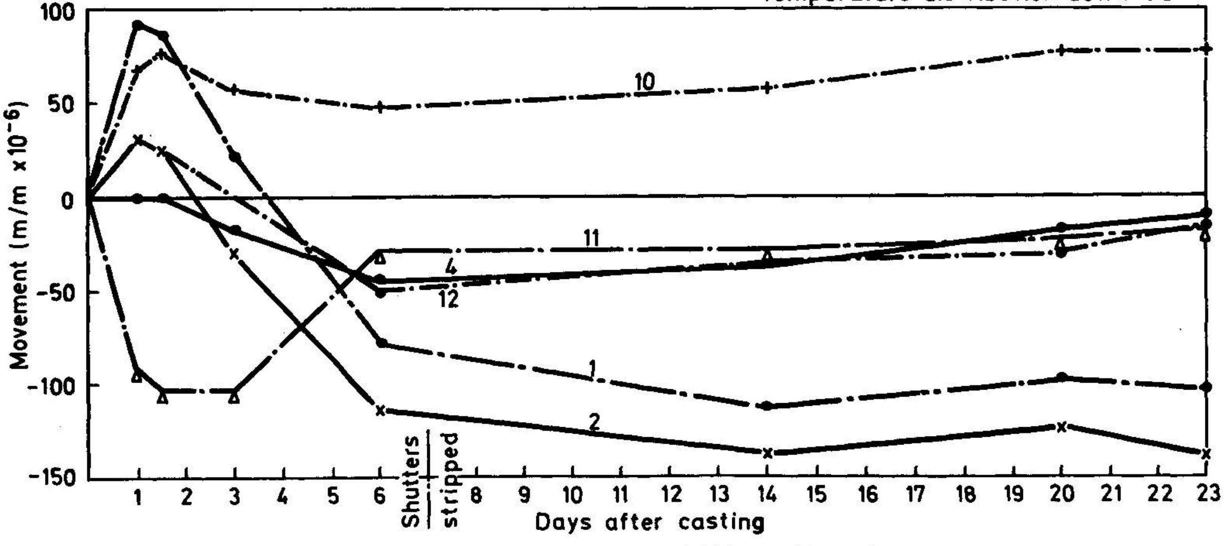


Fig. 2. EARLY-AGE MOVEMENTS IN REINFORCED CANTILEVER

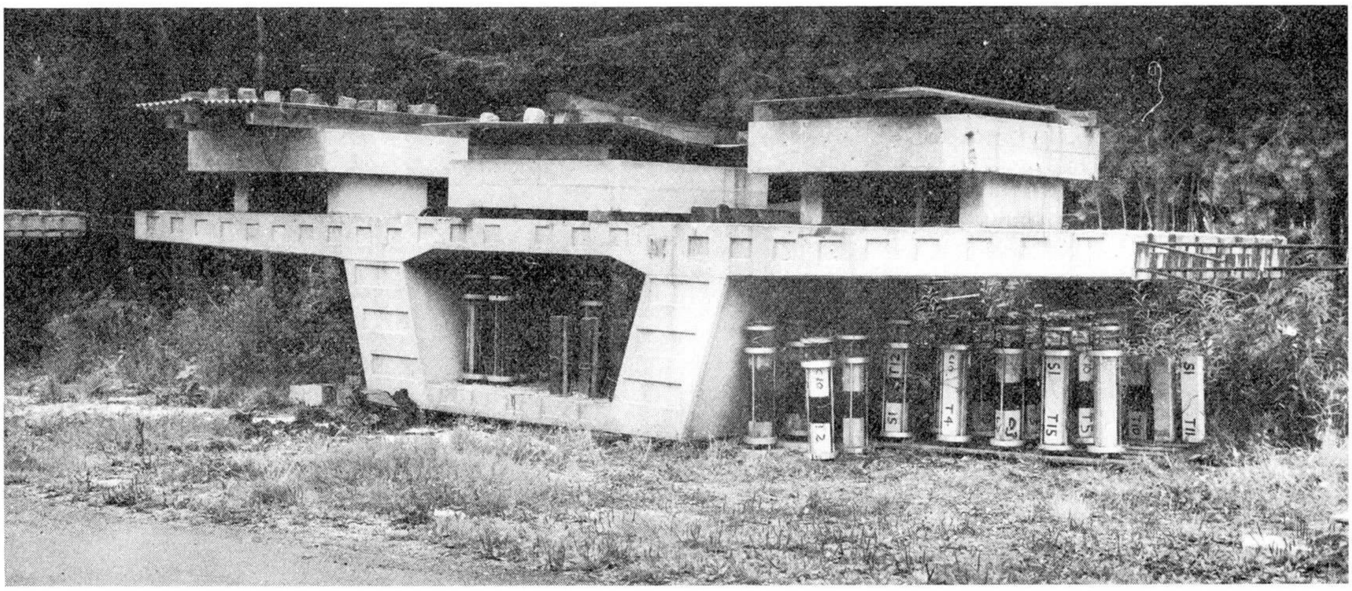


Fig. 3: Unit from Mancunian way and creep and shrinkage specimens

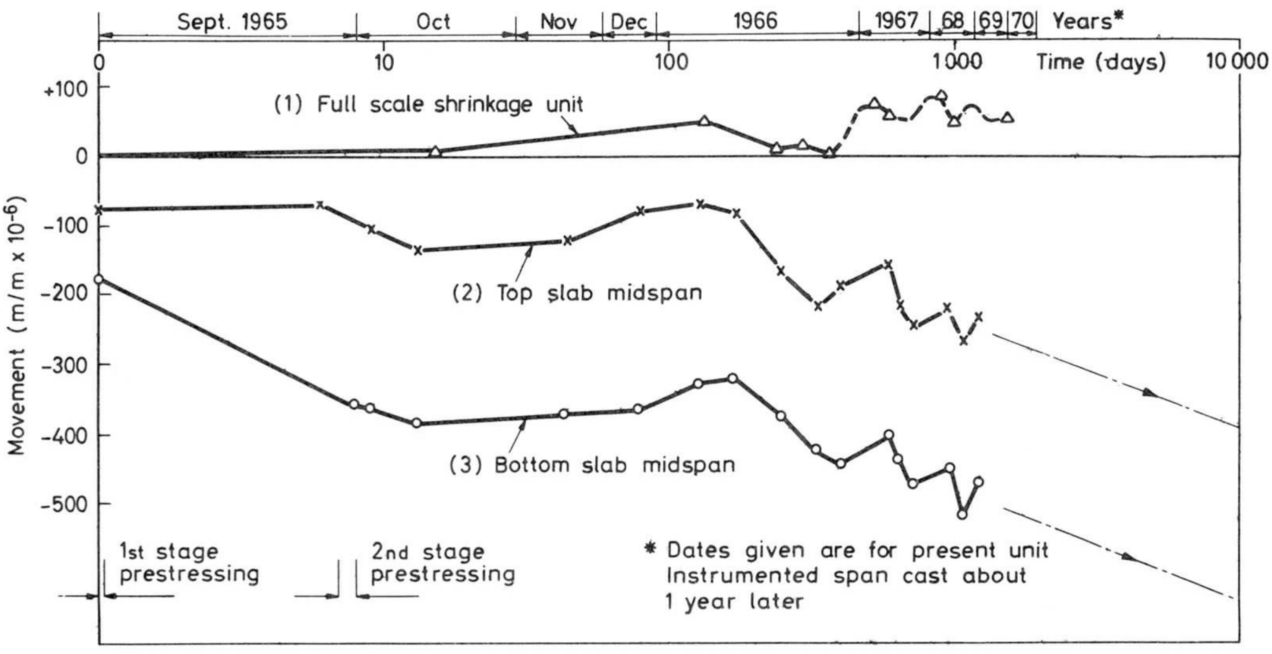


Fig. 4. MOVEMENTS IN SHRINKAGE UNIT AND STRUCTURE OF MANCUNIAN WAY

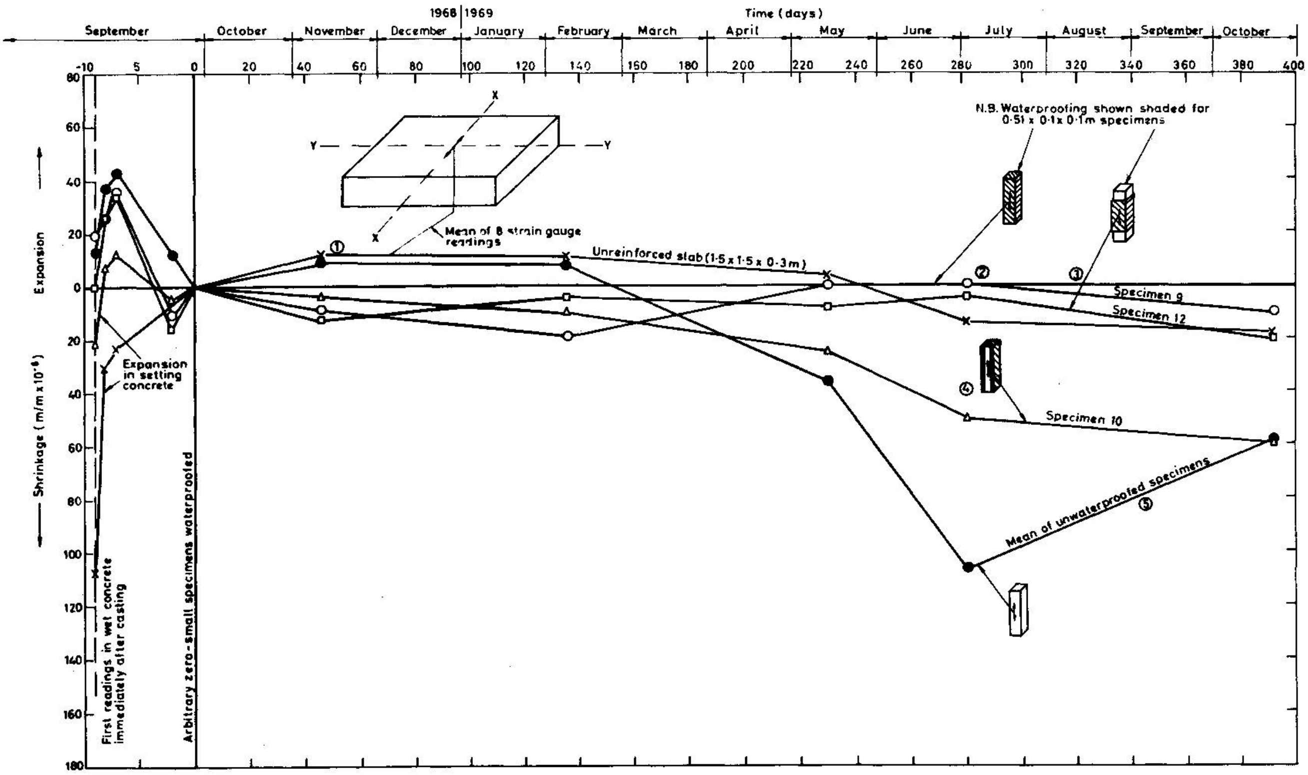


Fig. 5. SHRINKAGE IN SLAB AND SMALL SPECIMENS - WESTERN AVENUE

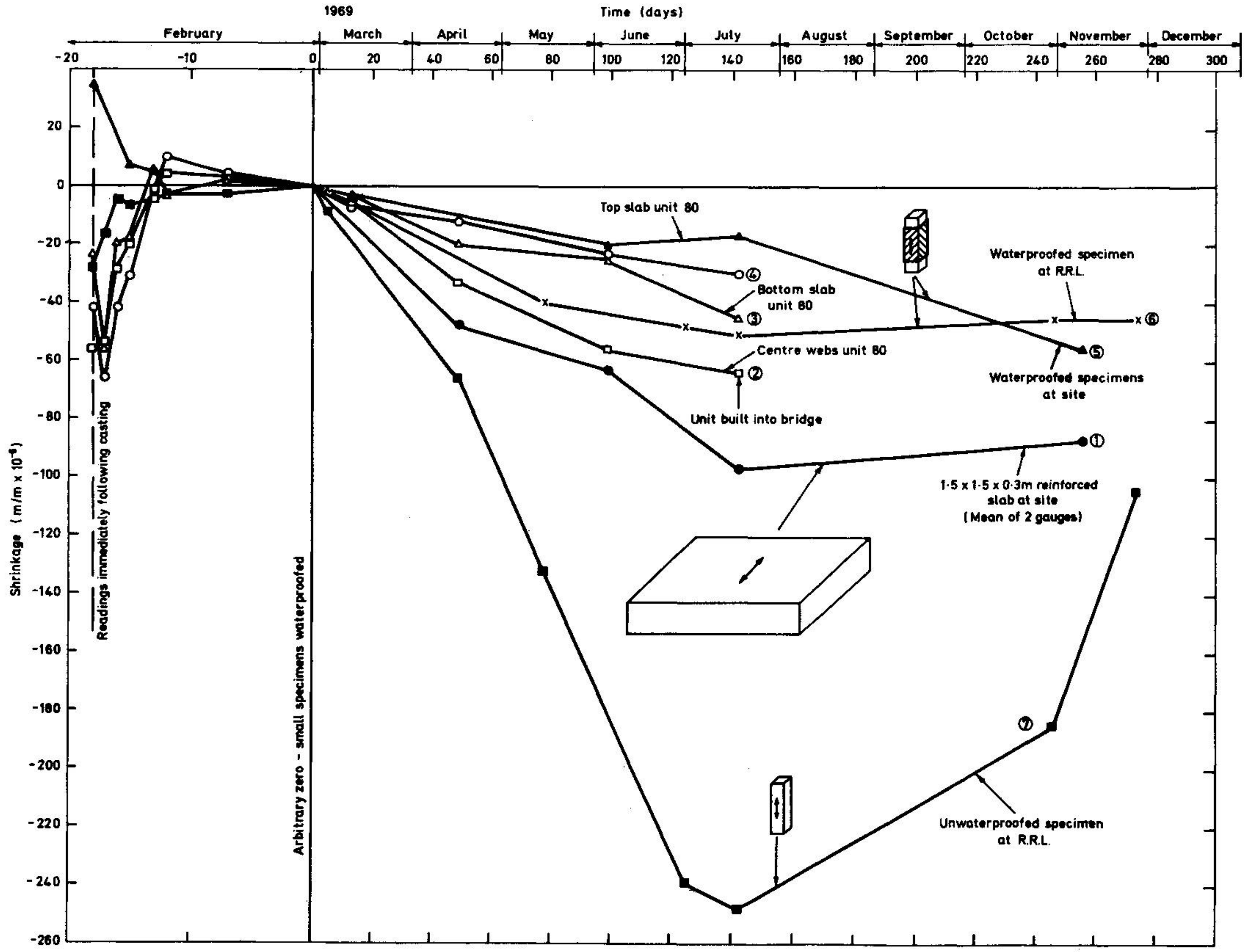


Fig. 6. SHRINKAGE OF SPECIMENS AND UNIT No.80, LONDON BRIDGE

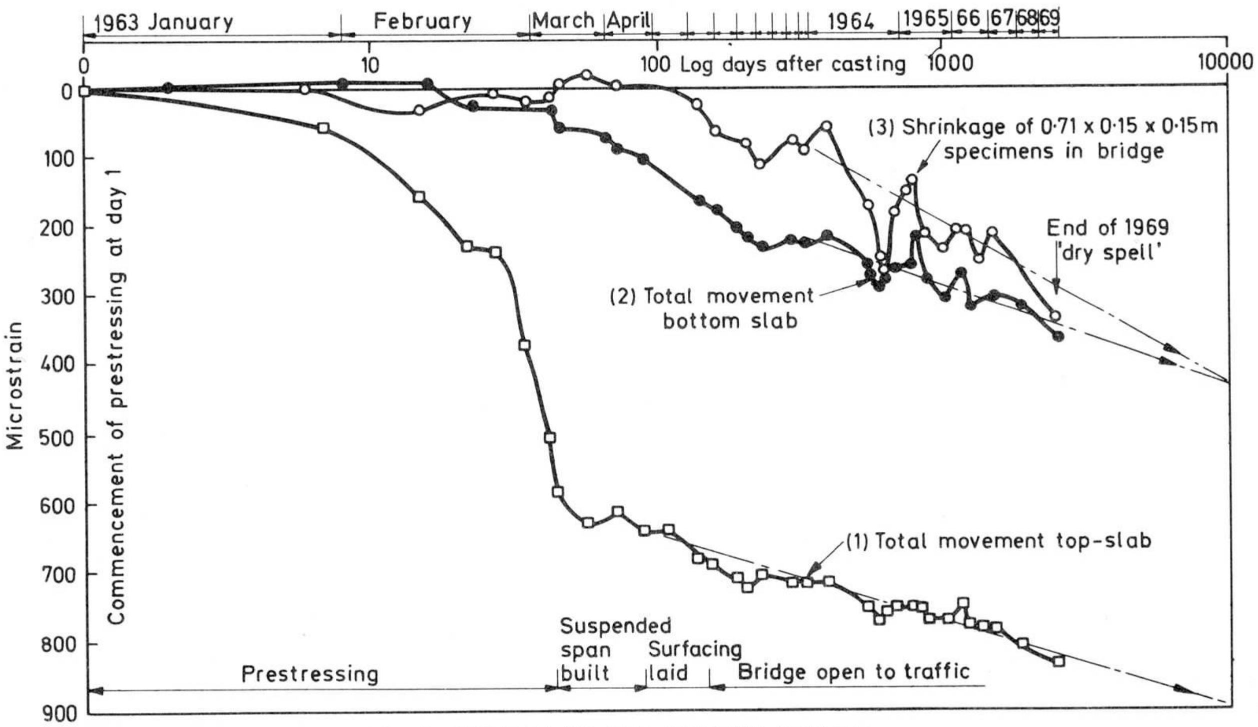


Fig. 7. MOVEMENTS FOR MEDWAY BRIDGE

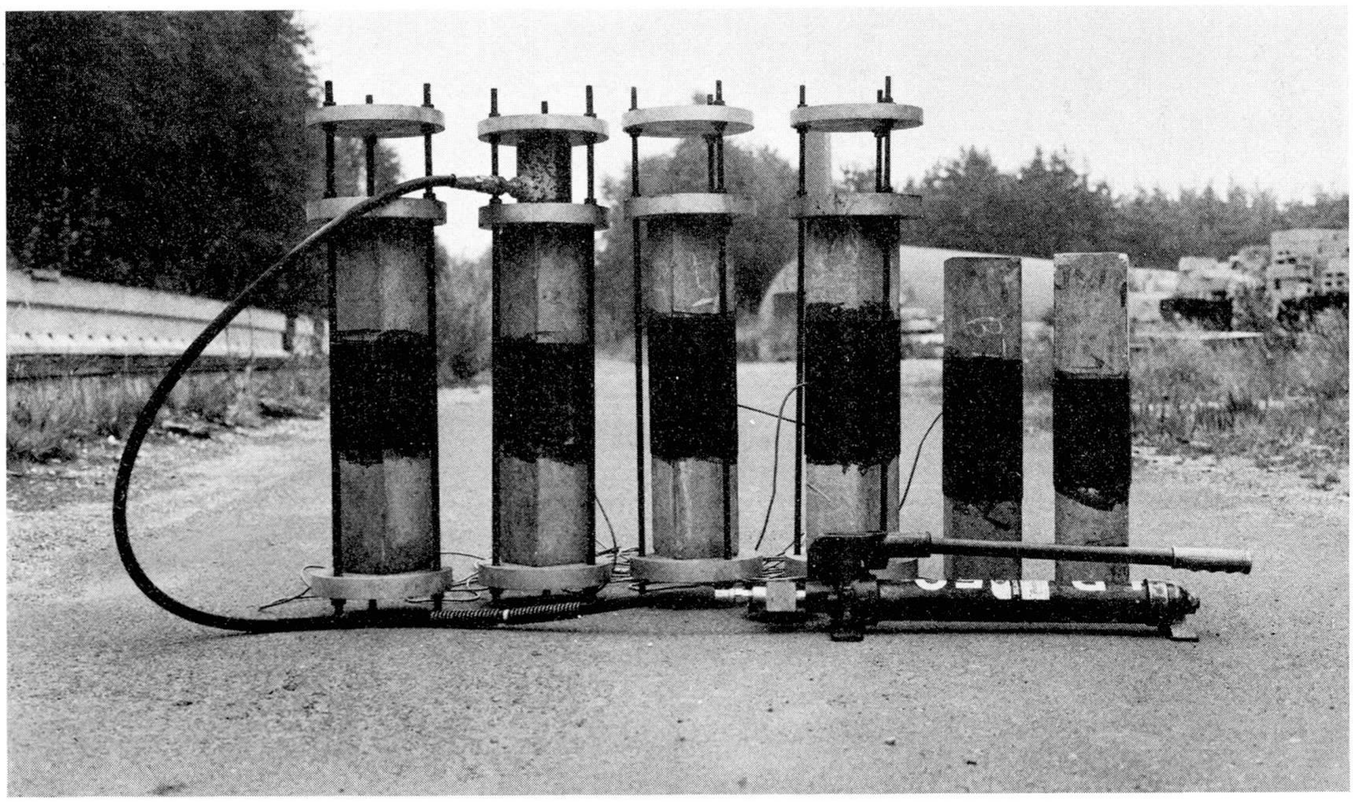


Fig. 8: Creep rigs and specimens

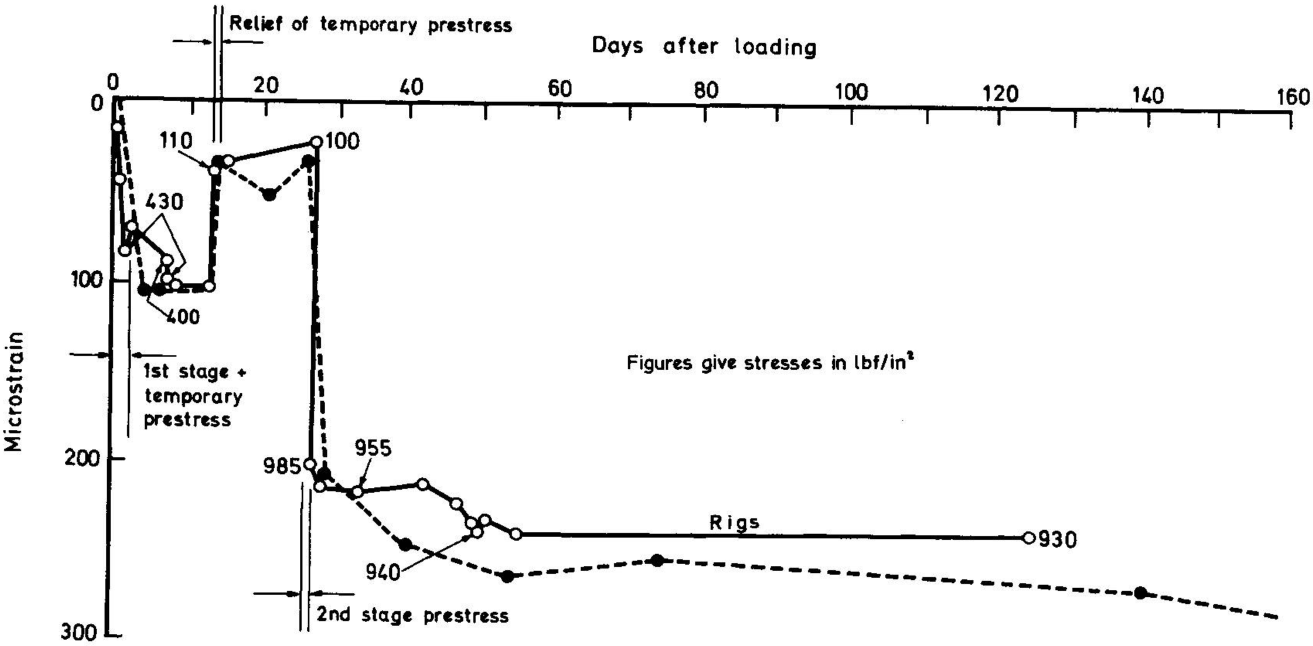


Fig.9. SECOND ASSESSMENT OF STRESS IN LOWER SLAB

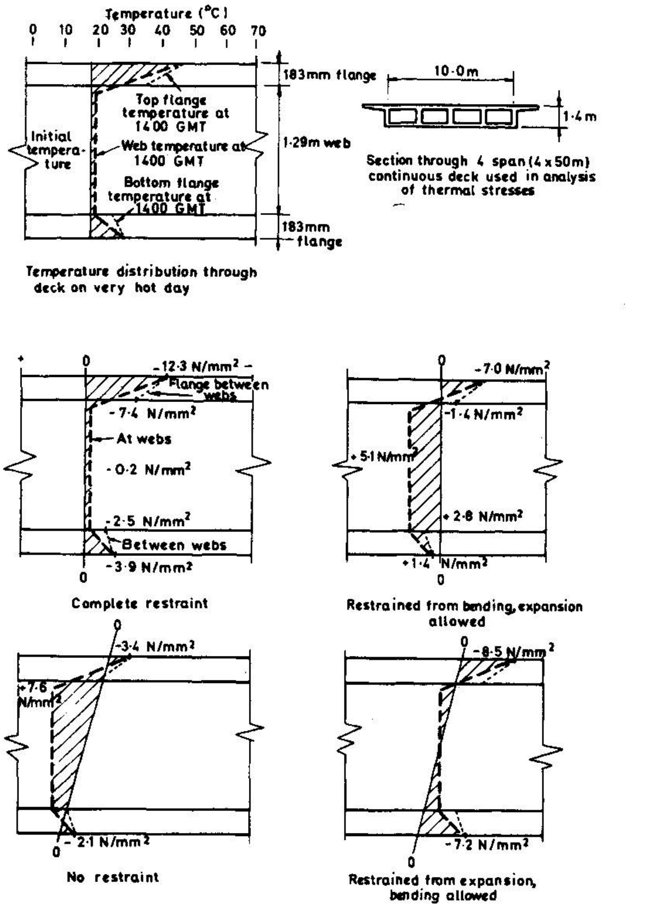


Fig.10 STRESSES DUE TO TEMPERATURE

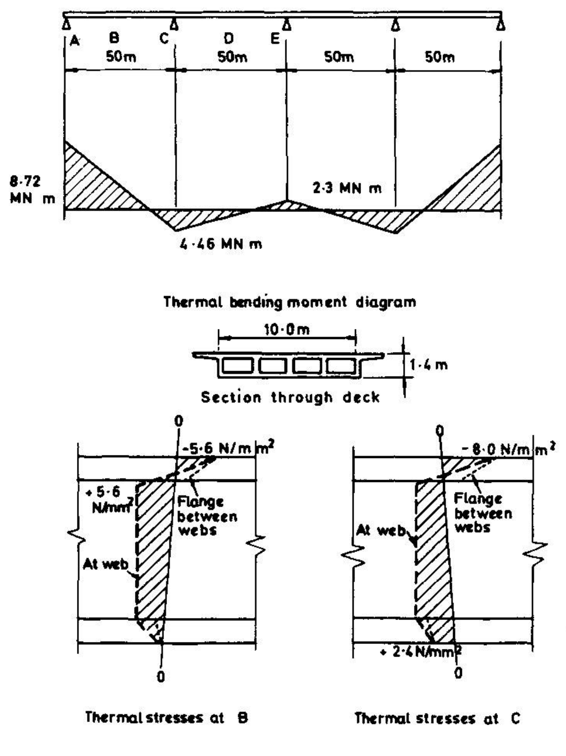


Fig.11. THERMAL STRESSES IN CONTINUOUS MULTI-CELLULAR BOX DECK

SUMMARY

Measurements of shrinkage, creep and temperature made on bridges in the U.K. are discussed. It is shown that the magnitude of the thermal movements on setting may well exceed that due to the later shrinkage arising from moisture loss. The creep factor Ø to DIN 4227, has been found to be in the range 1.0 to 2.0 at 10,000 days and the total movement was proportional to the logarithm of time. Correlation has been achieved between small specimens and the structure by partially waterproofing the specimens. The range of thermal movement in bridges is described and it is shown that thermal stresses are sometimes comparable with stresses due to live loads.

RESUME

Les auteurs décrivent des mesures de retrait, de fluage et de température faites sur des ponts au Royaume Uni. Ils démontrent que la grandeur des mouvements pendant le durcissement peut bien dépasser ceux dûs au retrait ultérieur qui résulte d'une perte d'eau. Le facteur du retrait Ø selon DIN 4227 se trouvait entre 1,0 et 2,0 à 10.000 jours et le mouvement total était proportionnel au logarithme du temps. Pour pouvoir faire une correlation entre les petites éprouvettes et la structure on a dû imperméabiliser partiellement les éprouvettes. On décrit l'étendue du mouvement thermique et on démontre que les contraintes thermiques sont quelquefois comparables aux contraintes dues aux surcharges.

ZUSAMMENFASSUNG

Besprochen werden Kriech-, Schwind- und Temperaturmessungen an Strassenbrücken in Grossbritannien. Es wird gezeigt, dass die Höhe der thermischen Bewegungen während des Erhärtens jene übersteigt, die aus dem Schwinden infolge Feuchtigkeitsverlust auftreten. Der Kriechmodul Ø nach DIN 4227 ergab sich zu 1,0 bis 2,0 bei 10 000 Tagen und die gesamte Bewegung war proportional zum Logarithmus der Zeit. Korrelation zwischen kleinen Proben und dem Bauwerk wurde durch teilweise Wasserdichtung der Proben erreicht. Der Bereich der thermischen Bewegungen in Brücken wird beschrieben und gezeigt, dass die Spannungen durch Temperatureinflüsse manchmal mit jenen aus Nutzlast verglichen werden können.

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