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Model Study of a Prestressed Concrete Box-Girder Bridge Under Thermal Loading

Etude sur modèle d'un pont en béton précontraint à section en caisson, soumis à des variations de température

Modellversuch einer vorgespannten Hohlkastenbetonbrücke unter Temperaturbelastung

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1. INTRODUCTION

Stresses are induced in bridge structures by diurnal temperature fluctuations. Although bridges have long been designed with expansion joints to cope with axial deformations due to temperature changes, effects of vertical and horizontal temperature gradients have largely been ignored, or dealt with by over-simplified theories.

The non-uniform temperature distribution induced in bridges by solar radiation has two main longitudinal effects. First, internal stresses are induced due to the non-linear vertical temperature gradient. Second, the increase in temperature of the bridge deck with respect to the soffit temperature results in an upwards hogging of the bridge. If the bridge is continuous, restraint of the upward deflection is provided at the internal supports, and the resulting reactions induce continuity stresses of appreciable magnitude.

Recent examples of distress, particularly in prestressed concrete box-girder bridges have forced engineers to investigate the problem in more detail. In New Zealand, damage to the Newmarket Viaduct, a large urban motorway bridge, caused by vertical temperature gradients, necessitated remedial action costing more than US$300,000. In consequence, considerable interest has been stimulated in New Zealand in thermal stress problems with the result that research projects have been initiated at the University of Auckland, and at the Ministry of Works Central Laboratories, specifically investigating thermal response of box-girder bridges.

Although some measurements made on concrete bridges have been reported (1,2,3) these have been limited in scope. Temperatures have been measured at comparatively few points, and correlations between stress and temperature, deflection and temperature, or theory and experiment do not appear to have been made. The experimental programme described in this paper is aimed at producing a more comprehensive and cohesive body of data obtained under laboratory controlled conditions.

2. THEORY

Various simplified design methods exist for calculating thermal stresses in box-girder bridges. See References 4 and 5 for example. However, the temperature distributions used are over-simplified, and produce inaccurate results. Further, the simplifications made are unnecessary as consideration of basic equilibrium requirements leads to simple equations capable of handling any form of temperature distribution for any section shape.

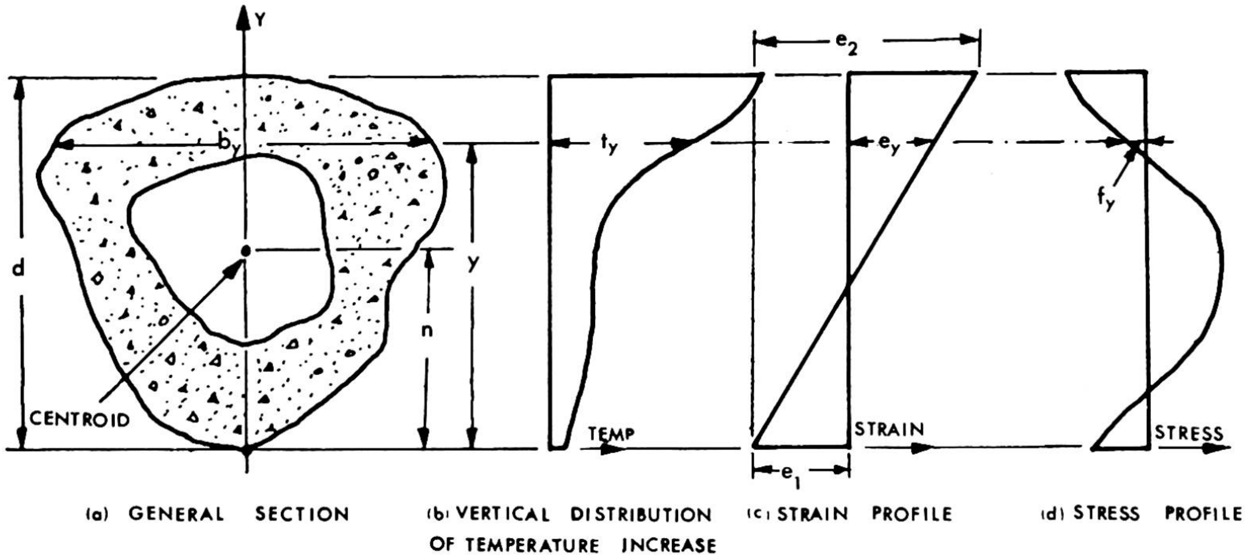


FIG 1. GENERAL SECTION WITH ARBITRARY VERTICAL DISTRIBUTION OF TEMPERATURE

Consider Fig.1 where an arbitrary section shape (Fig.1a) is subjected to an arbitrary vertical temperature distribution, (Fig.1b). Assuming the Euler-Bernoulli 'plane sections' hypothesis holds, the final vertical distribution of longitudinal strains will be of the form shown in Fig.1c. The stress induced by the temperature increase t_y will be given by

$$f_y = E(e_1 + e_2 \frac{y}{d} - \alpha . t_y) \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad 1$$

Consideration of longitudinal force and moment equilibrium leads (6) to the following equations

$$e_1 + e_2 \frac{n}{d} = \frac{\alpha}{A} \int_0^d t_y . b_y . dy \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad 2$$

$$\frac{e_2}{d} = \frac{\alpha}{I} \int_0^d t_y (y-n) b_y . dy \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad 3$$

where α is the linear coefficient of thermal expansion, A is the cross section area, and I is the moment of inertia about the horizontal axis through the centroid.

Solution of equation 3 directly yields the curvature of hogging. Substitution into equation 2 and solution provides the second unknown, e_1. Finally substitution of e_1 and e_2 into equation 1 produces the internal stresses. For a continuous bridge the additional stresses induced by continuity may be found by calculating (a) the notional deflections induced at the internal supports by the curvature e_2/d, (b) the forces necessary at the internal supports to reduce these notional deflections to zero and (c) the moments and stresses induced by these forces.

3. THE MODEL

3-1 MODEL DIMENSIONS

A model study offered the advantages over a full-scale prototype site investigation of a controlled rather than an unpredictable thermal input, an accelerated time scale, the separation of thermal effects from other load cases, reduced cost, and ease of instrumentation and data acquisition. A typical single-cell trapezoidal box-girder section was chosen for the model. The section, whose prototype dimensions are given in Fig.2, is typical of many urban motorway off-ramps in New Zealand.

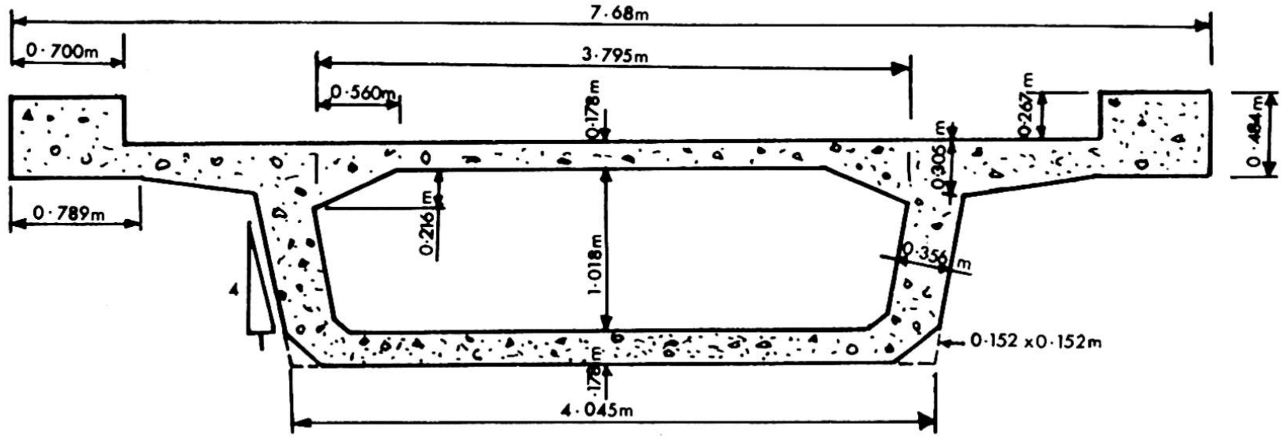


FIG.2. PROTOTYPE SECTION DIMENSIONS

A quarter-scale model representing a simply supported span of 30.5m was chosen. The scale was dictated by the physical size of the strong-floor testing area available, and by the availability of suitable steel to model mild-steel reinforcing. Fig.3 gives the overall model dimensions.

3-2 MODEL MATERIALS

The model was cast in six 1.22m segments with two solid 0.305m end diaphragms. This segmental form of construction was chosen to facilitate placing straingauges and temperature transducers. Each segment weighed about 570Kg. and was cast from a scaled concrete mix designed to have a 28 day cylinder crushing strength of 41.3N/mm^2. Maximum aggregate size was 4.75mm, and slump was between 50 and 75mm. Eighteen 300mm x 150mm dia. cylinders were cast with each segment and cured under the same conditions as the model, to provide sufficient control specimens

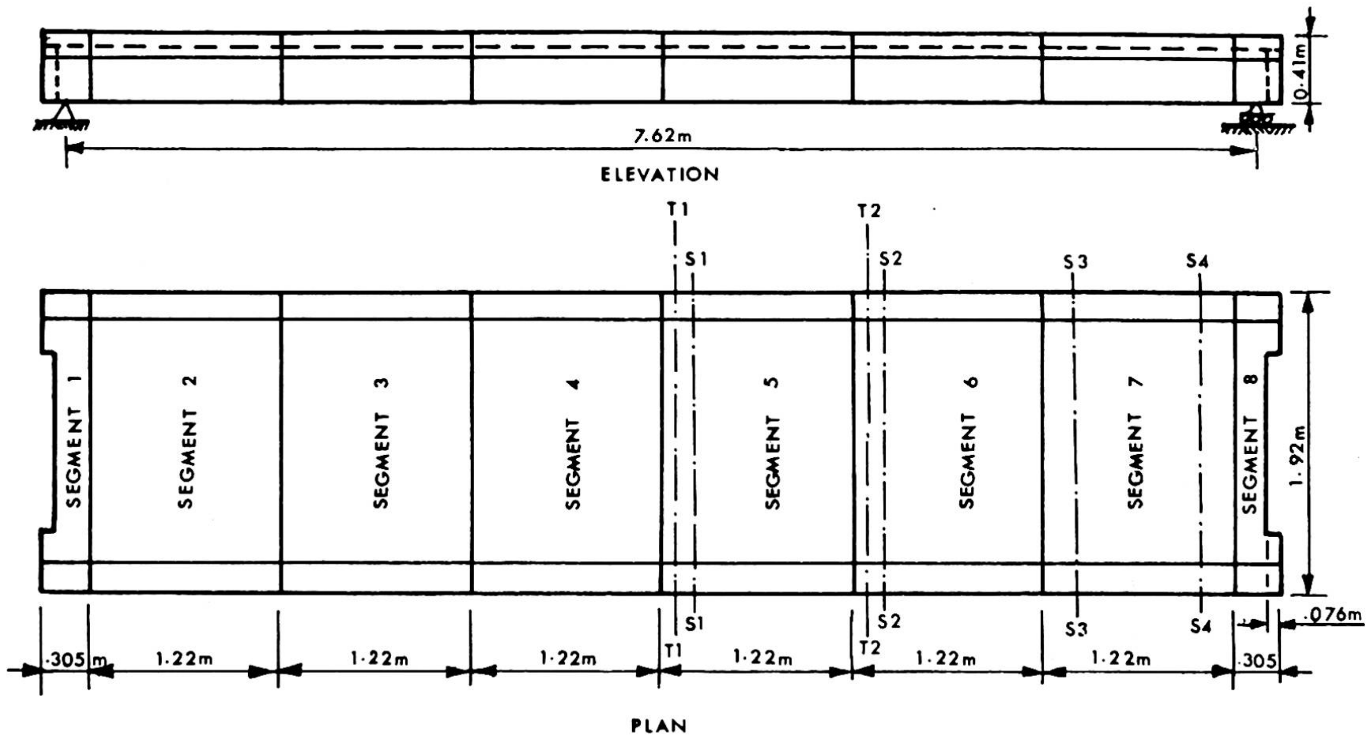


FIG 3. MODEL DIMENSIONS

for all stages of the testing programme.

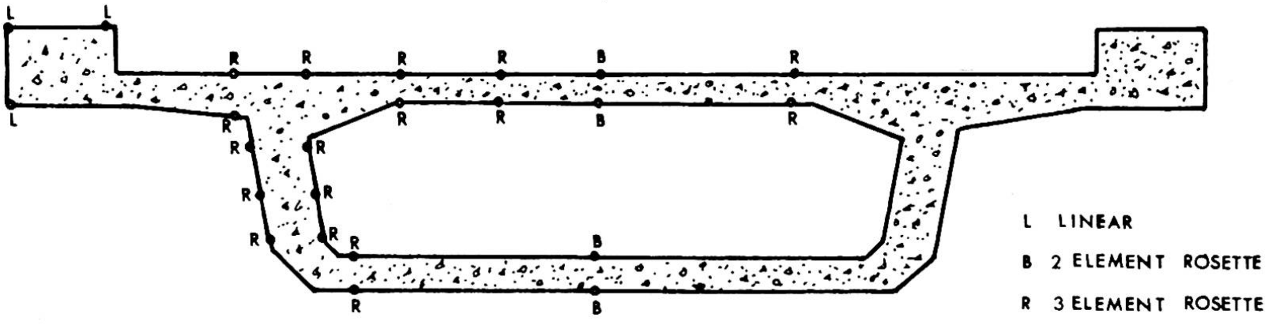
Typical mild-steel reinforcing was included in the model by use of 4.83mm and 3.18mm dia. black annealed tie-wire, and 6.35mm dia. mild steel rod.

The model was prestressed by ten draped cables; five in each web. A BBR system was used which gave a cable force of 142N after transfer, and an average prestress of 4.16N/mm^2. At the centre of the bridge theoretical prestress stresses varied between 0.57N/mm^2 tension at the curbs to 10.23N/mm^2 compression at the soffit.

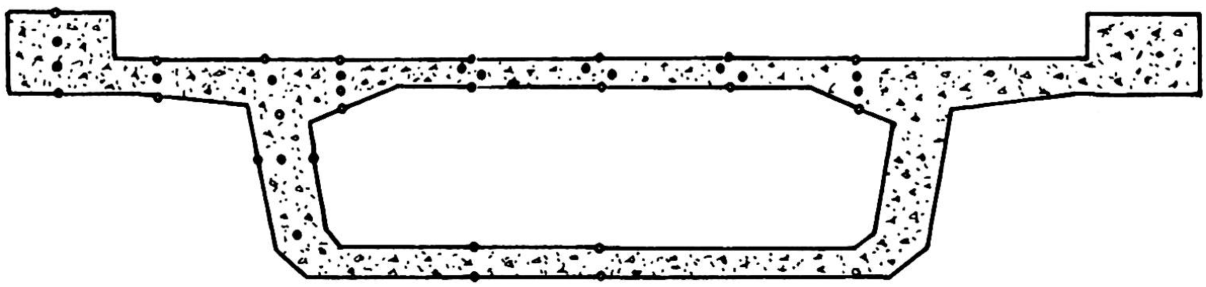
The eight precast concrete sections forming the model were glued together using an epoxy mortar. Requirements for this mortar were that it should have a high compression strength, a high modulus of elasticity, should gain strength rapidly, should stick to a vertical surface without slumping, should 'wet' the surface and should have a similar colour to the concrete. A mix based on CIBA's Araldite AW103/Hardener HY956 was adopted. Mix proportions were AW103:HY956:Silica Sand:Talc = 1.00:0.19:3.0:1.0. This mix produced 24 hr. cylinder strengths of about 40N/mm^2 with a 28 day cylinder strength in excess of 60N/mm^2. Modulus of elasticity exceeded 1.24 x 10^{4}N/mm^2 at 28 days.

3-3 INSTRUMENTATION

Electric resistance straingauges, resistance thermometers and thermocouples, and straingauged deflectometers were used to measure strain, temperature and deflection of the model. Because of symmetry it was only necessary to gauge one quarter of the bridge and use a few additional gauges to ensure that behaviour was in fact symmetrical. Fig.4a shows the distribution of straingauges at each of sections S1 to S4 defined in Fig.3. In addition to these gauges, a large number were located in the vicinity of one end of the model to check the distribution of prestress stresses. These have been reported elsewhere (7).



(a) STRAINGAUGE POSITIONS SECTIONS S1 TO S4



(b) THERMOMETER POSITIONS SECTIONS T1, T2

FIG 4. INSTRUMENTATION FOR THERMAL – LOAD TESTS

In Fig.4a L denotes a linear Kyowa KF-20-C8-11 foil straingauge, B denotes a two-element 90° Kyowa KP-20-B2-11 rosette and R denotes a three-element 90°-45° Kyowa KP-20-B3-11 rosette. All gauges had a gauge length of 20mm, and had transparent backings to ensure that gauge and concrete colours would be as close as possible, resulting in equal absorption of radiant energy. In all, 560 straingauge elements were fixed to the model.

Resistance thermometers and thermocouple positions at sections T1 and T2 of Fig.3 are shown in Fig.4b. The gauges were located in holes drilled 75mm into the ends of the segments prior to gluing. A mortar slurry was used to fill the holes after placing the gauges. Further thermocouples were distributed over the deckslab surface to provide a means for checking the uniformity of surface temperature.

Vertical deflections were measured by straingauged deflectometers on the soffit at nine points along the longitudinal centre-line. All data was monitored by a 200 channel Dynamco Datalogger and recorded on paper tape at a rate of two channels/sec. Since the number of gauges greatly exceeded the capacity of the data-logger it was necessary to duplicate tests with different batches of gauges. Data reduction by computer required the minimum of control data and enabled data manipulation by matrix commands through free-format programming.

4. THERMAL LOADING

For the thermal load tests the model was enclosed within a controlled environment box consisting of sides and top of soft-board surfaced with insulating paper covered with aluminium foil. One hundred 375 watt infra-red light bulbs fixed to the ceiling of the box were connected into eight separate banks, each producing a uniform heat pattern on the deckslab of the model. Consequently the radiant intensity applied to the model could be controlled in a step-wise fashion. Convective cooling of the top surface was provided by two 457mm diameter propeller fans located at one end of the environment box, capable of producing a 3m/sec wind along the bridge deck. When forced cooling was required the end wall of the box opposite from the fans was removed and an exterior roller-door was opened, allowing the hot air contained by the box to be exhausted to atmosphere.

In all tests the model was subjected to a cycle representing four consecutive days of identical heat input and cooling. Heating and forced cooling were controlled for the first day to produce deckslab temperatures corresponding with surface temperature cycles recorded by Dickinson (8,9) on Australian bituminous concrete surfaces. This pattern of heating and cooling was then repeated for the next three days. Examination of the scaling laws for thermal loading (10) indicates that for identical prototype and model materials a model:prototype time scale equal to the square of the length scale is required to produce model: prototype temperatures and stress scales of unity. Consequently a prototype diurnal cycle required a model duration of 90 minutes, and the four-day cycles represented required only 6 hours continuous testing.

Fig.5 shows a photograph of the model within the environment box.

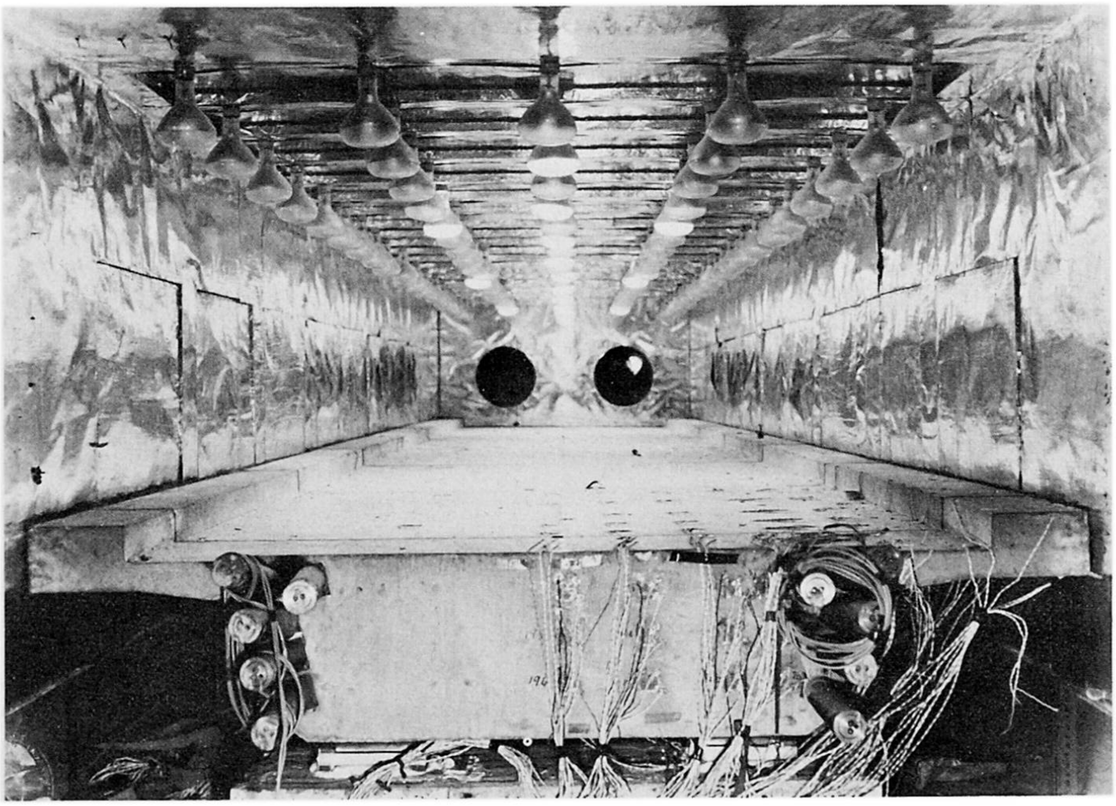


Fig.5 QUARTER SCALE MODEL IN ENVIRONMENT BOX

5. RESULTS

5-1 THERMAL RESPONSE OF MODEL

Fig.6 shows heat input:time and typical temperature:time curves in prototype time scale for a simulated four-day test cycle. As will be seen the heat input, which has only been expressed as a percentage of the maximum heat input was identical for the four days. Temperatures are all arbitrarily referred to a base of 20°C.

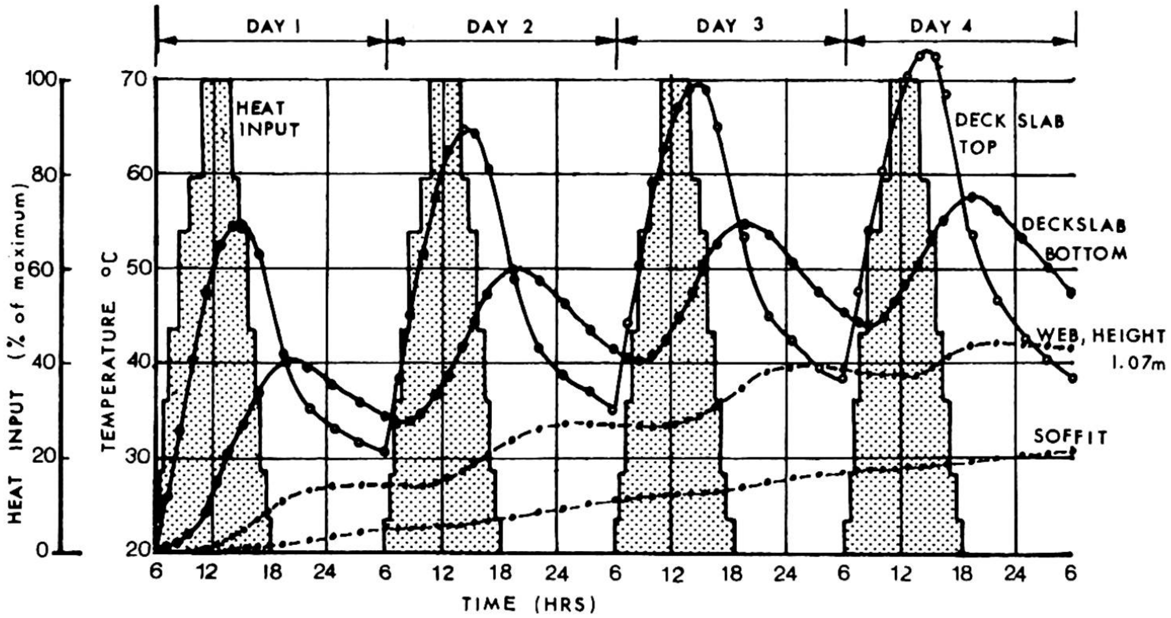


FIG. 6 TYPICAL THERMAL RESPONSE FOR 4-DAY CYCLE

Despite the convective cooling of the deckslab provided by the 3m/sec wind the peak temperatures continued to rise throughout the four-day cycle. This was particularly the case for the deck-slab above the enclosed air cell where temperatures rose to higher values than in the concrete above the webs or in the cantilevered areas. The temperature above the enclosed air cell also dropped more slowly in the cooling periods, than in the other areas. This behaviour appears to result from the insulating behaviour of the entrapped air cell. There was more resistance to vertical heat flow from the bottom of the deck-slab into the stagnant air mass than down the concrete webs, hence the higher temperatures above the air cell. Total heat stored will be similar in the volume below any given area of surface. In the cooling cycle heat could flow up and down the webs away from maximum temperature areas, whereas above the air mass heat flow would mainly be vertically up, away from the air mass, and therefore temperatures remained higher longer in the deckslab area above the air mass. This is illustrated by comparing the temperature profiles

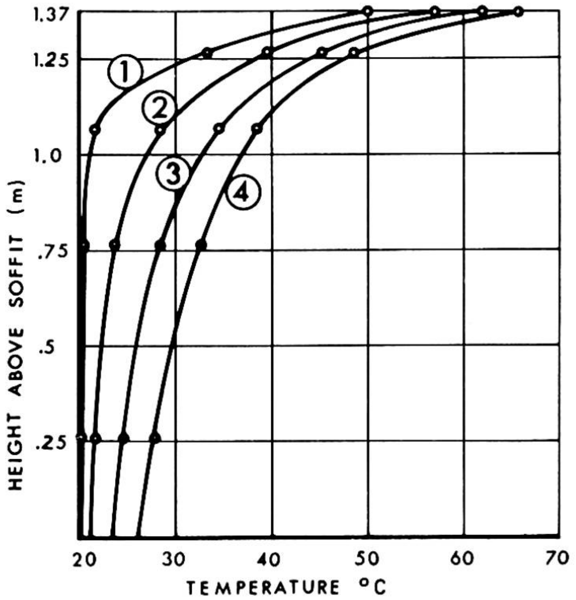


FIG. 7 TEMPERATURE PROFILES DOWN WEB AT 14:00 HRS ON 4 SUCCESSIVE DAYS

down the web, as shown in Fig.7, with the peak values in Fig.6. Fig.6 indicates a maximum deckslab temperature, above the air cell, of 73°C on the fourth day, in comparison with a peak of 66°C above the webs, as indicated in Fig.7.

5-2 COMPARISON BETWEEN THEORY AND EXPERIMENT

In order to compare experimental deflections and stresses with those predicted by Equation 1 to 3 above, temperatures measured at the positions shown in Fig.4b were used to define the temperature distribution t_y. The section of the model was considered to be divided into a number of different areas, each of

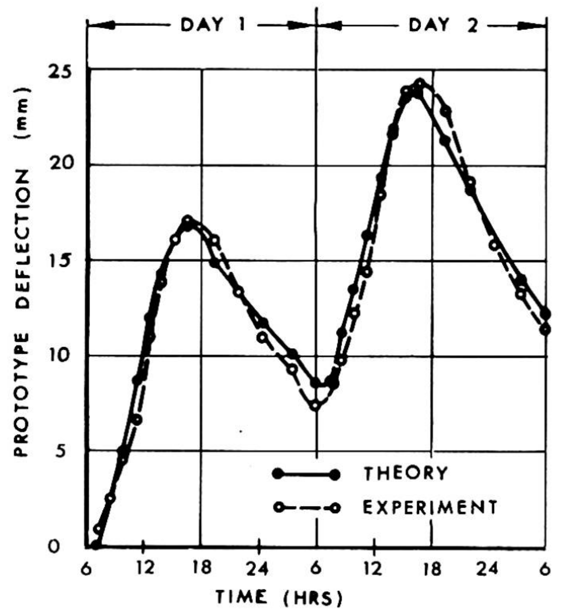


FIG. 8 CENTRAL DEFLECTION vs TIME

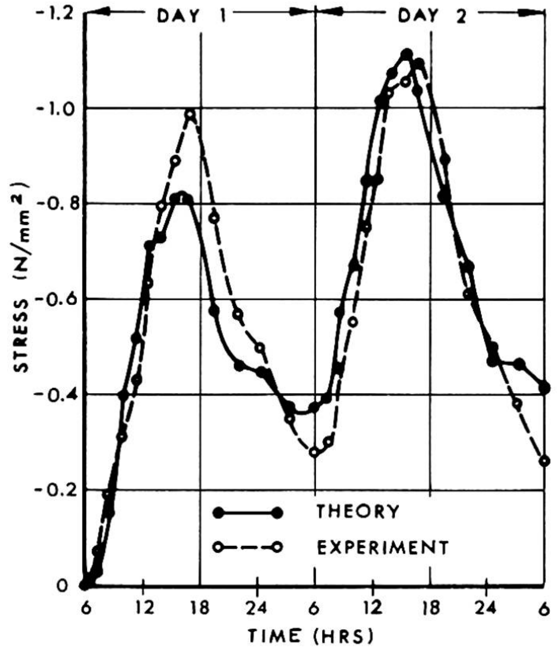


FIG. 9 SOFFIT STRESS vs TIME

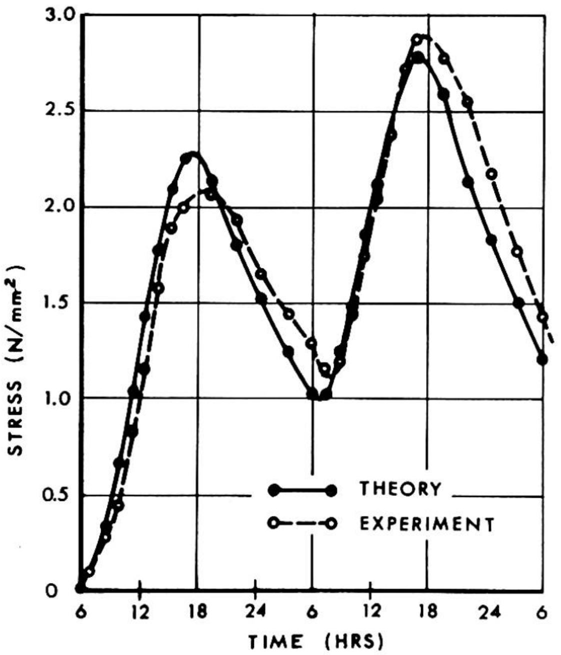


FIG. 10 WEB STRESS vs TIME at Height=870 mm

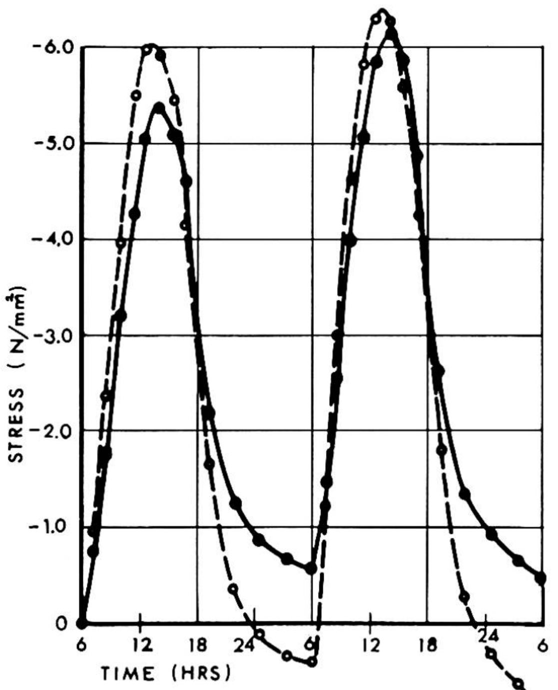


FIG. 11 DECKSLAB STRESS vs TIME

which contained a temperature transducer. At any given time the temperature indicated by a particular gauge was considered to be constant throughout the relevant area. Equations 1 to 3 were put into summation rather than integral form and solved by computer, which produced graphs of theoretical central deflection, and stress at salient points versus time, for the four-day cycles. These were then compared with experimental values.

In predicting the theoretical behaviour, experimentally obtained values for the coefficient of thermal expansion and modulus of elasticity of $10.65 \times 10^{-6}/^{\circ}C$ and $2.34 N/mm^2$ respectively, were used.

Figs. 8 to 11 show comparisons between theory and experiment for central deflection, soffit stress, deckslab stress, and web stress at a prototype height of 870mm above the soffit. In all cases only the first two days of the four-day cycle have been compared. Time is expressed in prototype scale, and the stress convention used is tension positive, compression negative.

Fig.8 indicates excellent agreement between theoretical and experimental deflections. The agreement between theoretical and experimental stress shown in Figs.9 to 11 is also good, and is in general within the expected experimental accuracy of $\pm 0.2 N/mm^2$. However, Fig.11, which shows deckslab stresses, exhibits much higher discrepancies, though experimental and theoretical curve shapes are very similar. It is thought that the discrepancy is largely due to the difficulties in obtaining accurate strain readings on the surface of the deckslab. Although the strain-gauges were compensated for a thermal coefficient of expansion of $10.8 \times 10^{-6}/^{\circ}C$, the differences in surface colour of the gauges and concrete deck may have induced artifically high temperatures in some gauges. Nevertheless, in all cases the predicted stresses exhibit satisfactory agreement with experimental values.

It is of interest to calculate the continuity stresses that would have been induced in the model if it had had a central support. At 16:30 hrs. on the second day of the thermal cycle, the central displacement was 24.2mm. If this displacement were restrained by an internal support, stresses of $3.40 N/mm^2$ compression at the top of the deckslab and $6.95 N/mm^2$ tension at the soffit would be induced. These should be added to the internal stresses indicated by Figs.9 and 11 to give final stresses of $7.60 N/mm^2$ compression at the deckslab and $6.15 N/mm^2$ tension at the soffit.

6. DISCUSSION AND CONCLUSIONS

The results presented above show that equations developed from consideration of simple statics accurately predict longitudinal stresses and curvatures in box-girder bridges. Transverse stresses resulting from restrained transverse expansion and hogging of the deckslab can also be of significant magnitude and should always be considered in design.

The temperatures induced in the model and reported herein represent extremely severe conditions. Although the surface temperatures sustained on the first day of the four-day cycle compare with those reported by Dickenson (8,9), the rather low wind velocities and the lack of forced cooling on the soffit during the cooling periods of each cycle will tend to make the temperatures on the successive days artificially high.

At this stage of testing, the model has only been tested with a bare concrete deckslab. Further series of tests with a scaled bituminous wearing surface, and finally with a white painted surface will be carried out to investigate the influence of surface colour on thermal response. This model study is to be followed by a comprehensive prototype investigation into thermal stresses in a two-span three-cell prestressed box-girder bridge.

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SUMMARY

A quarter scale prestressed concrete model of a single cell trapezoidal box-girder bridge is described. The principal aim of the study is the investigation of structural response to diurnal temperature fluctuations. Results of temperatures, stress and deflections induced by thermal loading are reported. Good agreement is obtained between experimental stresses and deflections and values predicted by a theory based on simple equilibrium criteria.