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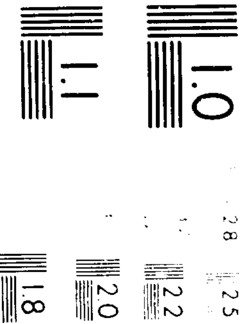

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POST TENSION SEGMENTAL BRIDGES: SOME DESIGN ASPECTS OF A 215 METRE SPAN PRESTRESSED CONCRETE BOX GIRDER BRIDGE *

by

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The Queensland Main Roads Department is the design and construction authority for a central city bypass project in Brisbane. A major component of the project is the New Farm Bridge. This bridge will be of balanced cantilever construction, some 608 m (2,000 ft) long, with a 215 m (705 ft) main span for navigable river clearance. The bridge centre line lies on a 2,300 m (7,500 ft) radius horizontal curve, and is generally 20.1 m (65.9 ft) wide except toward the southern end where it widens by 2.5 m (8.2 ft). It will be of precast segmental construction, each 3 m (9.8 ft) length of box being lifted in two halves. This paper describes the selection of the articulation, longitudinal profile, the transverse bending analysis of the superstructure, and the main pier analysis where the webs are 12 m (40 ft) deep.

INTRODUCTION

1. The New Farm Bridge is the key link in the proposed Central Freeway, now in its planning stages, which will join the Northern and South-Eastern Freeways to form the major north-south bypass of the central city area of Brisbane. Until this connection is completed, traffic congestion in the Fortitude Valley area cannot be relieved. The planning is accordingly being given a high priority.

2. The ultimate freeway development at the bridge is the provision of four lanes in each direction. However, in the initial stages, the first bridge will be planned for a restricted 3 + 3 lane configuration with a second structure to be built at a later date. This paper discusses the choice of bridge type, articulation, and methods of analysing the super-structure.

CHARACTERISTICS OF SITE

3. The northern bank of the river is a low flood plain with rock at about 26 m (about 85 ft) depth, while the southern bank rises 20 m (65 ft) above water level with weathered conglomerate exposed on the surface *(Fig. 1)*. Adjacent to the southern abutment is Wynnum Road, a major arterial serving the south-eastern suburbs.

4. Geometrical design of the freeway sited the bridge just upstream of a right-angle bend in the river *(Fig. 2)*. This increased the main span quite considerably, as it placed the bridge on a 30° skew to the shipping channel.

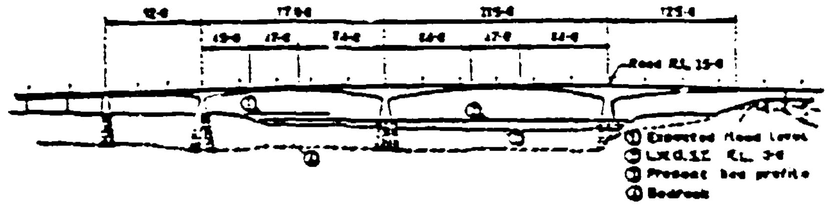


Fig. 1 — Elevation of New Farm Bridge

DESIGN REQUIREMENTS

5. The major requirements of the bridge were to:

(a) provide a road facility 20.1 m (65.9 ft) wide between outer parapets,

(b) conform to the freeway alignment, which placed the bridge on a horizontal curve of 2,300 m (7,500 ft) radius,

(c) provide river clearance to allow the passage of a partially completed ship 250 m (820 ft) long by 33 m (108 ft) wide moved by tugs on each side. Vertical clearance for this vessel is 21.3 m (70 ft),

(d) keep the river open to normal traffic at all times, and

(e) minimise disruption to traffic on Wynnum Road.

6. Item (c) above combined with the river bend necessitated a main span of 215 m (705 ft) with both piers 2 and 3 in the water and hence subject to ship impact. Items (c) and (e) lead to a structure with the roadway at approximately RL 33 m (108 ft) at centre span.

7. Because of their position, the piers were designed to withstand the forces caused by impact of a ship of 32,000 t (31,448 ton) displacement travelling at tidal velocity, and also for impact of normal river traffic (5,000 t displacement) at cruising speed.

8. Adequate protection of the piers using large diameter sheet pile caissons as recommended by Ostenfeld (1965) was not

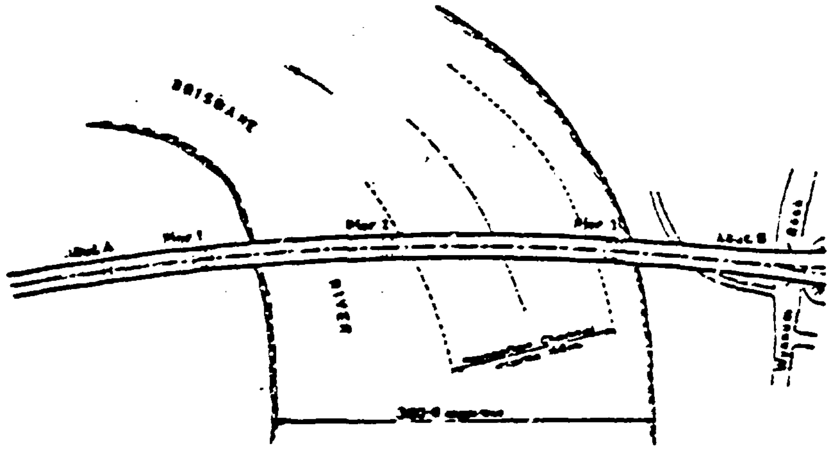


Fig. 2 — Plan of bridge site

economical because of the increased span required.

STRUCTURAL ARRANGEMENTS

9. Since a heavy sub-structure was necessary for impact resistance, a logical form of super-structure was a balanced cantilever erection out from the piers without-of-balance dead- and live-load moments directly to the sub-structure. This allowed cantilever erection out from the piers without any intermediate props in the river, as would be required in an anchor-cantilever span arrangement. Because of the loads and height (27 m (90 ft)), such props would be expensive and in constant danger of ship impact.

10. Suspended spans varying in length from zero to 60 m (200 ft) were investigated, and over this range there was little effect on total cost, including erection trusses. The central hinge arrangement, used on the Bendorf and other European bridges was considered to have several disadvantages, the major ones being the sharp change in vertical alignment at the pin due to creep and shrinkage, and the moment reversal which occurs in the cantilevers under live load.

11. The structural depth at midspan was selected as 2.5 m (8.20 ft). A depth much less than this was structurally possible, but would have lead to aesthetic problems in a 215 m (705 ft) span. The length of the suspended span was selected as 47 m (155 ft) after considering erection truss requirements, economic span/depth ratios, and the modular arrangement of box segments in the super-structure.

12. To save weight in the most critical region, the suspended span has no transverse diaphragms, and the bottom flange of the centre cell was removed leaving a flexible twin-box configuration (*Fig. 3*).

13. To limit the weight and size of elements being lifted, each box segment will be made in two sections, lifted into position on the erection cradle and the longitudinal joint concreted at the same time as the transverse joint. The full width cantilever box is then stressed onto the previous section. Precast segments are 2.5 m (8.52 ft) long, allowing a reinforced joint of 0.4 m (1.32 ft) between units. All precast segments are rectangular in plan and the horizontal curvature is produced by varying the joint width by approximately 30 mm (1.2 in).

14. Several alternative designs have been considered, including a composite steel box-concrete deck suspended span to save weight, a full steel box girder arrangement, and a cable stayed concrete box design. None of these alternatives was

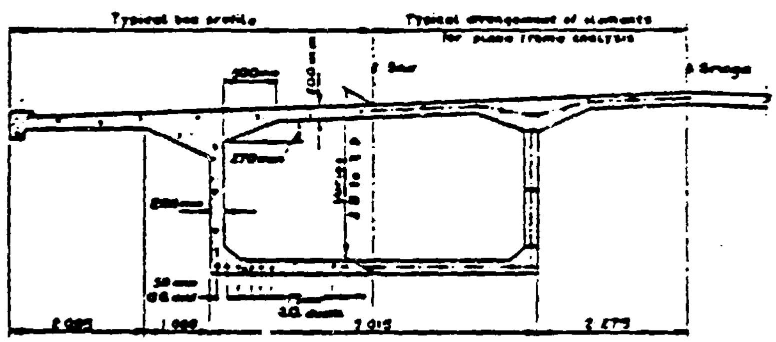


Fig. 3 — Section of suspended span box

economically competitive with the full concrete box girder arrangement.

SELECTION OF BOX CROSS-SECTION FOR MAIN SPAN CANTILEVERS

15. Preliminary designs indicated the general magnitude of prestress required at the base of the cantilever. From experience with previous bridges and studies by O'Connor (1972), it can be shown that the optimum cross-section is generally composed of minimum thickness webs and flanges.

16. The top flange was made 200 mm (8 in) thick between the large fillets at the top of the webs which contain all the prestressing cables and anchorages. The webs of the cantilever spans are in general 250 mm (10 in) thick, except at the top where they thicken to reduce the principal tensile stresses to within allowable limits (*Fig. 12*).

17. The cantilever box formwork is of constant shape, the only variable dimensions being the depth of the straight portion of web and bottom flange thickness. The bottom flange varies from 180 mm to 620 mm (7 to 24 in). This compares with a maximum bottom flange thickness of 2.5 m (8.20 ft) for the Bendorf Bridge which has a 208 m (682 ft) span.

LONGITUDINAL PROFILE OF MAIN SPAN

18. The structural depth of the box section was defined by a parabolic curve,

$$\text{depth} = ax^b - c\ (m) \qquad (1)$$

where x is the distance from span centre line, and a, b and c are constants, which were varied to give different profiles and depths at the pier. Preliminary designs were completed for many different profiles, and from these it was obvious that the main control on the cantilever was the compressive stress in the bottom flange.

19. The most significant properties of the section were the bottom flange section modulus Z_{bf}, box depth, bottom flange thickness and unit weight of cross-section. Using a programmed desk calculator, these variables were plotted over the practical range of values, as illustrated in *Fig. 4*. On this graph, various longitudinal profiles could be plotted.

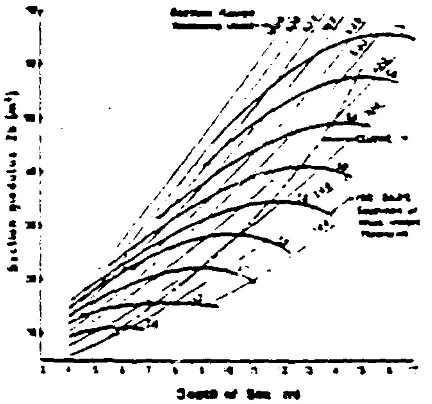


Fig. 4 — Optimisation of cantilever profile

20. The minimum weight profile can be selected directly from this graph, curve M, which passes through the peaks of the 'isobars' giving maximum section modulus for minimum weight. However, curve M leads to much larger girder depths than is necessary. This same effect was noted by O'Connor (1972).

21. Since the river clearance depends on the depth of the super-structure adjacent to the piers, the most economical pier depth must be less than the minimum-weight depth. Knowing the minimum-weight profile, the box depth at the pier was reduced until the total weight of the cantilever began to rise dramatically (*Fig. 5*). From these considerations, a web depth of 12 m (39.36 ft) at the pier centre line was selected.

22. Variation of the index b in eqn (1) leads to different profiles (as shown in *Fig. 6*). The greater the value of b, the flatter the soffit profile near centre span, and hence the optimum saving of dead weight. How-

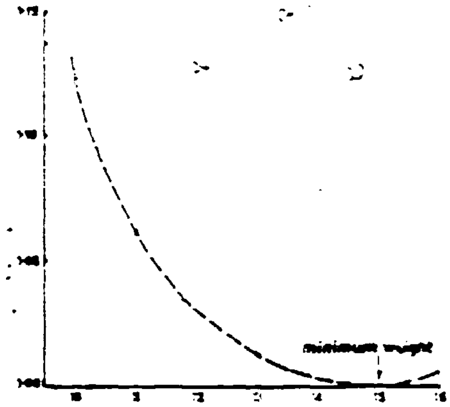


Fig. 5 — Variation of total weight of cantilever vs depth of box at pier

ever, the flatter profiles lead to shear problems towards the tip of the cantilever which require increased web thickness, or depth, or vertical prestressing. To avoid additional complications in box profile, the depth was varied, leading to a value of index b slightly greater than 2.

TRANSVERSE ANALYSIS OF BOXES

23. Even with the most sophisticated computers and programs, it is not possible to investigate with a single analytical model the *following*:

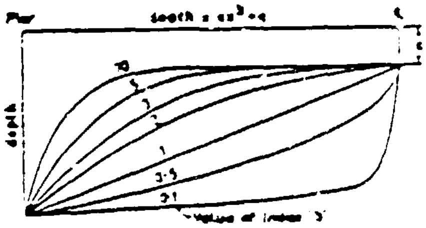
(a) local bending effects of wheel loads,
(b) local distortion of a box cross section, and
(c) interaction of transverse and longitudinal bending.

24. The analysis was thus divided into several sub-sections. A broad outline of the procedure for the suspended spans follows.

(a) A transverse section of the structure was analysed as a plane frame. From this analysis, stiffness values were obtained for use in subsequent phases. Where there was some doubt as to the correct stiffness for a particular mode, upper and lower limits were investigated.
(b) A finite element analysis of a section of the deck slab was bounded by two planes of symmetry. The deck is supported by the webs which have infinite vertical stiffness and a transverse rotational stiffness equivalent to the real box web. This analysis gives the 'local-effect' bending moments in the deck.
(c) Reactions to the webs from (b) were divided into various components as illustrated in *Figs 3* and *9*. The component of load, *Fig. 9(b)* causes distortion of the box cross section. The associated transverse bending moments were calculated using the 'beam on elastic foundation' (BEF) analogy developed by Wright *et al.* (1967).
(d) The differential bending of the two boxes of the suspended span, *Fig. 5(b)* was analysed by an idealised space-frame and by an analytical solution to the differential equations developed by Mehmel and Beck (1953). Close correspondence was achieved between the two methods.
(e) Reactions, rotations and displacements from (b), (c) and (d) were fed back into the planeframe analysis (a) to obtain the various components of the transverse bending moments for various critical points in the cross section.

PLANEFRAME ANALYSIS

25. A typical arrangement of planeframe elements used to derive the stiffness



Soffit Profiles

Fig. 6 — Variation of girder profile with index 'b'

properties of the suspended span is shown in *Fig. 3*. Behaviour of the full cross-section (two boxes) was calculated by adding symmetrical (no rotation at centre line) and anti-symmetrical (no vertical displacement at centre line) load components. The rotational stiffness of the webs was obtained by applying unit torque to each of the web-flange intersections in turn. This also gives the moments at any point in the cross-section due to unit rotations of the web-flange nodes, and hence the moments for a particular load case if these nodal rotations are known. A simple program was written on a desk computer to print all critical moments for given rotations.

FINITE ELEMENT ANALYSIS OF DECK

26. The local bending moments in the deck slab were analysed using a finite element model shown in *Fig. 7*. Since the webs have a vertical stiffness in the order of one hundred times that of the deck slab with respect to local bending, the webs may be assumed as rigid line supports. Relative deflections of the webs due to distortion, etc. are treated separately. Symmetry and anti-symmetry were used to reduce the model to one quarter size. The length of model was selected so that the 'free' end had no effect on the peak bending moments, and so that the effect of wheel loads at the normal spacing of standard truck axles could be obtained.

27. Axle loads were placed in 12 different positions across the half-width of deck, and influence lines drawn for eight critical sections. From these, critical load combinations could be clearly seen. However, since these local bending moments due to wheel loads were only one of several components of the total transverse bending moments, a tedious process of summing all possible combinations was still necessary. The effect of trucks on the other side of the longitudinal centre line was obtained directly by subtracting, rather than adding the symmetrical and anti-symmetrical load components.

28. Since there is no simple analytical method which can take account of double-tapered cantilever slabs (Sawko and Mills 1971), a separate finite element model with a much finer mesh was used to analyse the cantilever slabs, including the region around expansion joints where an edge stiffening is required. The 3 m (9.84 ft) parapet units were not included in these analyses to allow for construction and repair conditions. The parapet units do not act in the overall bending of the super-structure due to the gaps left between units. They do have a marked local stiffening effect on the cantilever, but as this tends to reduce the peak negative bending moments over the adjacent web, the effect can be safely ignored. The edge stiffening also causes significant positive bending moments in the cantilever slab.

29. The small longitudinal edge beam integral with the cantilever slab is very effective in distributing concentrated loads

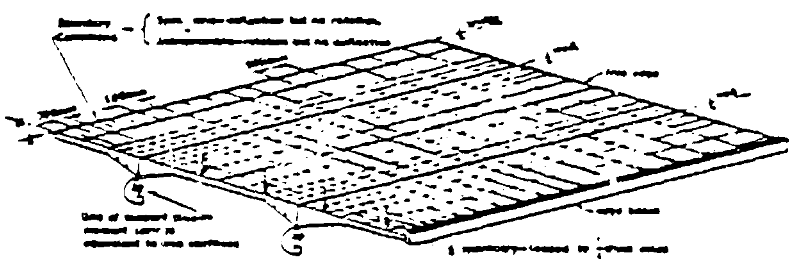


Fig. 7 — Finite element model of portion of deck slab

and has the same effect as extending the cantilever slab as suggested by Somerville *et al.* (1965). Comparing the finite element results with simple theories such as the NAASRA formula and Westergaard, it was clear that the finite element method is the only one suitable for a reasonably accurate analysis of variable thickness cantilever slabs.

DISTORTION ANALYSIS

30. The transverse distortional characteristics of the suspended span box were obtained from the planeframe model. The depth of the section changes only slightly over the length of the suspended span. Hence the box was treated as a constant cross section.

31. The distortional loads were obtained by summing the web reactions due to the truck loads on the deck finite element model over a length equal to the depth of the section. These were then divided into various components as in *Figs 8* and *9*. The distortional component of truck loading was then applied to the analogous beam on elastic foundation (BEF).

32. The most important characteristic of the BEF analogy is the βl factor, a non-dimensional measure of the relative transverse and longitudinal stiffnesses of the box. The βl value for the suspended span box was approximately eight, which signifies a high transverse stiffness. This means that longitudinal bending shears are equally distributed to both webs of a box, and that at least four diaphragms are needed to have any significant effect on the peak transverse bending moments. Since the transverse bending moments due to distortion were not large enough to control the cross-section shape, transverse diaphragms were unnecessary.

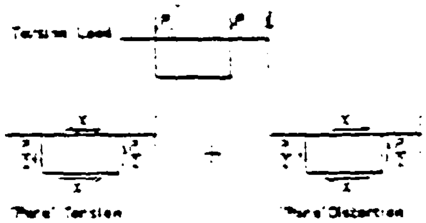


Fig. 9—Torsion and distortion in a single cell

33. After calculating influence curves on a desk computer, the effect of a series of axles representing a standard truck could be easily calculated. In this particular case, a series of three point loads P, P and 0.25P at 14 ft (4.27 m) spacing, had the same effect as a distributed load of 0.051P per foot run over the whole span. Hence, knowing the transverse bending moments due to unit distortion of the planeframe, the effect of truck loads could be calculated. It was found that the peak transverse bending moments due to distortion are essentially

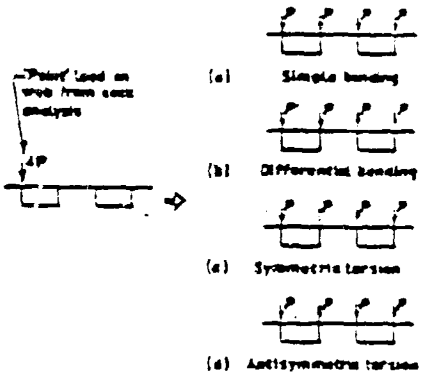


Fig. 8—Load components of suspended span box

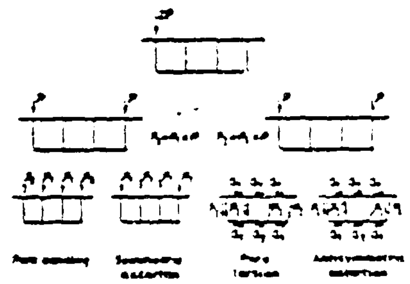


Fig. 10—Load components in three-cell cantilever box

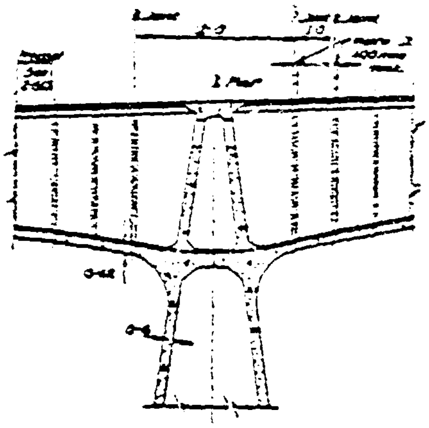


Fig. 11 — Elevation of Pier 2

constant over the central 30 per cent of the span, dropping to zero at the ends.

34. The transverse analysis of the three-cell cantilever boxes followed a similar procedure to that outlined above, except that the differential bending analysis is replaced by a more complicated distortion analysis. The designers have been unable to find a comprehensive analysis of the distortion of a three-cell box in the literature, and were forced to improvise using methods by Knittel (1965) and Wright et al. (1967).

35. Knittel deals rationally with the components of a 'fairly-uniformly-distributed' load on a three-cell box, but does not consider concentrated loads, while Wright analyses only the single cell case under uniform and point loading using the BEF analogy. In this case, it was assumed that the BEF analogy could be applied to an individual cell using the maximum distortional load component for that cell, and then summing adjacent cells to form the complete box. The load components are shown in *Fig. 10*.

36. The cross section of the cantilever box varies considerably along its length, and application of the BEF analogy was more complicated than in the suspended span case. There is no analytical solution to a variable stiffness beam on a variable stiffness foundation, so a numerical solution using a standard planeframe program and spring supports was used to find the maximum distortion of the cantilever box under point loadings.

ANALYSIS OF MAIN PIERS

37. As mentioned in para. 9, the superstructure consists of balanced cantilevers integral with the pier stem. The dead load moments at the piers balance, and apply only an axial load of 12,000 t (11,808 ton). However, construction conditions, and out-of-balance live loads can apply considerable bending moments to the pier. Simple bending theory gives a fairly accurate picture of the stress distribution in the cantilevers and

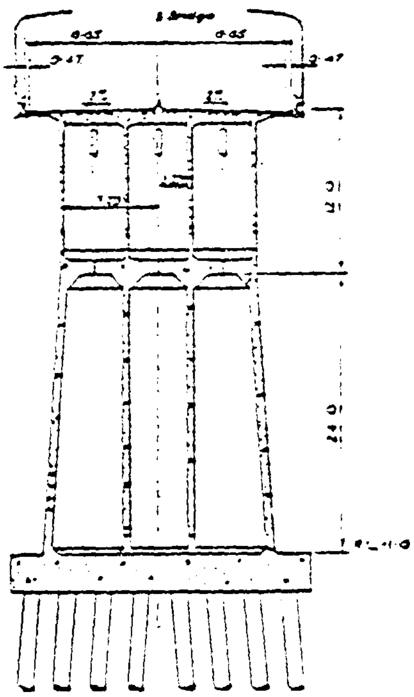


Fig. 12 — Section at Pier 2

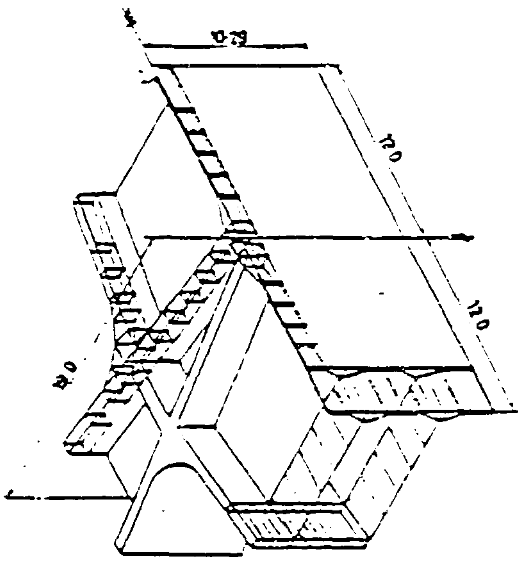


Fig. 13 — Finite element model of Pier 2 showing typical elements

pier stem, but there is no simple analytical method for the region surrounding the A frame at the pier-cantilever junction *Figs 11* and *12*.

38. Because of the size of the structure and the large loads, a three-dimensional finite element analysis was undertaken using the Stardyne program on the CDC 6600. *Fig. 13* shows the finite element model with some of the elements. The size of the model was determined by the need to apply the loads far enough away from the region of interest (St-Venant's principle) and the need for economy in model size. Only the shear and in-plane stresses were of interest and three degree-of-freedom nodes were used in conjunction with solid 'cube' elements and plane quadrilateral elements. The model contained 1032 nodes and 700 elements. The analysis showed the need to thicken the webs within the A frame from 250 mm (10 in), which is the standard web thickness, to 350 mm (14 in), in order to reduce the principal tensile stresses.

CONCLUSION

39. This paper has described some of the design considerations of a major prestressed concrete bridge. Structures of this size should be analysed by the most accurate methods possible, and suitable techniques are presently available. However, there are still many areas where new theories, more efficient programs and larger computers need to be developed.

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ADDENDUM

COMPARISON OF CANTILEVER BENDING MOMENTS

40. A comparison of design moments from finite element analyses and the NAASRA Highway Bridge Design Specification 1970, Clause 3.2.3(a) is given below. See also TABLE I and *Fig. 14*.

(a) Wheel load P = 72 kN + 30 per cent impact = 94 kN.

(b) Wheel load spread over 600 × 300 mm patch in finite element analysis.

(c) Clause 3.2.3(a) distribution width E = 0.3X + 1.1 (m).

TABLE I

BENDING MOMENTS IN kN m/m AT

SECTION AA

Cantilever type	X	Analysis (a) finite element	Analysis (b) code	b/a
NFB	1.45	25.7	60.3	2.25
FB1	1.3	30.0	68.2	2.27
FB2	0.9	21.9	46.5	2.12

SECTION BB

NFB	(2 wheels) 2.45 3.65	82.0	73.2 37.7 112.0	1.36
FB1	2.5 0.7	82.3	73.8 39.5 115.4	1.40
FB2	1.57	47.2	62.7	1.33

(d) Transverse bending moment/unit width $M = \frac{PX}{E}$

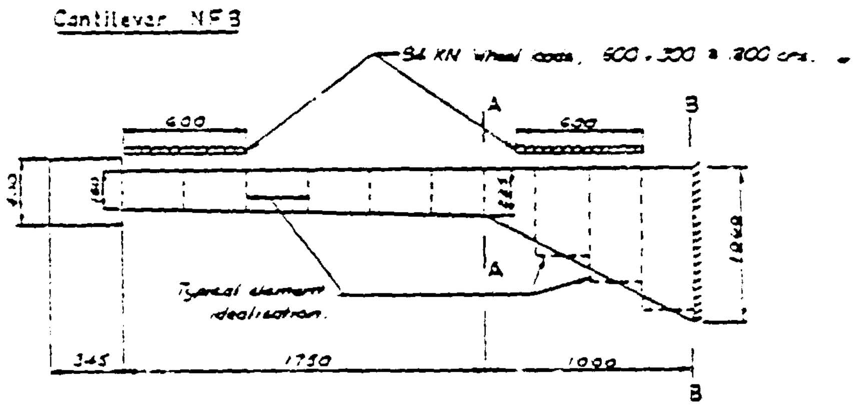
where X = distance from centre of wheel to support.

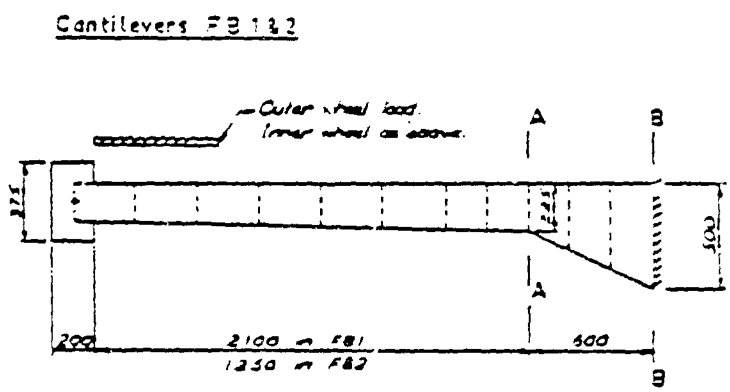
(e) Cantilever built in at support.

COMMENT

41. The large difference in design bending moments at Section AA is primarily due to the distributive effects of the edge beams which are ignored in the simple code formulae.

42. The finite element analysis is conservative because (a) it ignores the further stiffening effect of the concrete parapets placed on the edge beams, and (b) the cantilevers are not actually built in and the rotational capacity of the web/flange intersection will further reduce the peak bending moments.





DISCUSSIONS

L. G. BUCKLE
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43. A point of information that the authors may be interested to have is that the New Zealand National Roads Board is currently conducting through the Road Research Unit, comparative studies of various analytical methods for box girder bridges. Methods under study include simple grillage theory, the beam-on-elastic foundation analogy, folded plate theory, finite strip and finite element techniques. Application of these methods to simply supported and continuous, single and multi-cell, straight and curved box girder bridges is giving valuable data as to the most suitable method for a given structure. Results of this study are being published in a series of reports by the Road Research Unit.

44. A further comment might be made on the accuracy of the finite element method used by the authors. It has been well established that the assumed properties of the bending and plane-stress elements are the most significant factors controlling accuracy. Recently developed elements give excellent results for box girder analyses, even for coarse meshes. This is amply demonstrated in the reports referred to above.

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45. The authors are to be complimented on the efforts taken to obtain an optimum section. However, I would like to take issue with the statement in para. 23. In fact, 3-D finite element programmes utilising plate elements with membrane and plate-bending components (five or six degrees of freedom per node) have been specifically developed for complex box-girders in many countries, including New Zealand. The size and complexity of the New Farm Bridge would be within the capacity of existing programmes and computers. Comparison with experimental results has shown that behaviour is predicted well by the three-dimensional finite element methods, but that simplified methods of the type described in this paper tend to overestimate transverse stresses and underestimate the transverse distribution of longitudinal stress under eccentric live load.

46. Were differential thermal gradients through the deckslab considered in transverse section analysis? I imagine web bending stresses would be high.

AUTHORS' CLOSURE

To N. C. HAYLOCK
(See Introductory Remarks)

47. The longitudinal analysis of the Captain Cook Bridge was essentially the same as that used on the proposed New Farm Bridge. As the New Farm Bridge was longer, wider and of thinner cross section at many points, more attention was paid to the transverse analysis. The transverse analysis of the Captain Cook Bridge was based on a plane frame analysis of transverse bending using estimated effective

slab widths. Since the two cross sections were sufficiently different in the number of cells and thickness of members, no direct comparison was made of the overall results of the two different analyses. A comparison of design moments for cantilevers from finite element analyses and the NAASRA Code is given in an addendum (see paras 40–42).

48. In general, the main cantilevers of the bridge were analysed by the same methods as used for the suspended span except as mentioned in para. 34 of the paper. To the authors' knowledge (1975) no one has yet published a comprehensive theory of distortion for three-cell boxes and so some assumptions have had to be made. The analysis of single-cell boxes is well documented and relatively simple.

49. As mentioned in the reply to Dr Priestley, differential temperature stresses were considered in the design of the box. In the longitudinal analysis the tensile stresses in the webs lead to thickening of the upper 1.4 m of web to reduce principle tensile stresses to allowable values. This thickening from 250 to 330 mm was not shown in *Fig. 12* of the paper due to its small scale.

50. The major part of the temperature differential and the peak tensile stress occurs in the top metre or so of the box and had negligible effect on the pier-girder intersection where maximum stresses occurred in the region of the bottom flange.

To M. J. N. PRIESTLEY

51. Dr Priestley disputes the statement in para. 23 of the paper and the authors agree that he makes a valid point. However, the authors stand by the original statement *for a structure of this size*. To illustrate this point, consider the size of the problem created by extending the three-dimensional model in *Fig. 13* to cover the full 84 m of one cantilever with an element mesh fine enough to give information on both local bending effects of several axle loads at various points across the width, and the stress concentrations at the pier. Using a combination of meshes from *Figs 7* and *13*, and keeping the aspect ratio of elements down to a reasonable maximum of 3:1, the model contains some 5,000 nodes with six degrees of freedom.

52. The biggest computer commercially available to the authors in 1973 was the CDC6600 which offered the Stardyne finite element suite of programmes. This package had a limit of 15,000 degrees of freedom (i.e. 2,500 nodes of six degrees of freedom). Even if the model could be condensed to fit this limit, the cost would be enormous in both labour and computing fees. Having waded through the masses of output from several 1,000 node analyses, the author is convinced that analysing a structure several times bigger, and trying to extract design data for numerous load cases, would be more trouble than it was worth.

53. For a smaller, single cell, uniform box section, we agree it would be possible, but still very tedious. Remember that the designer is concerned not with an idealised point or uniformly distributed load, but with critical loads for numerous design sections considering lane loads, axle loads, standard trucks and abnormal vehicles in any position on the deck.

54. Differential temperature gradients were considered in both the longitudinal and transverse design using methods published by Priestley (1971). The conditions considered were maximum live load bending moments plus dead load plus 30°F (16.6°C) differential temperature at 125 per cent working stress, and 45°F (25°C) differential plus factored live load, dead loads, etc. at ultimate. These conditions did require extra transverse steel at some points.

REFERENCE

PRIESTLEY, M. J. N. (1971). Effects of transverse temperature gradients on bridges. Ministry of Works Central Laboratory Report No. 394. New Zealand.