

Strength Evaluation of the Manotick Bridge Piers

by Gilbert A. Hartley, Gerhard T. Suter,
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Synopsis: A structural investigation of a bridge pier using the finite element method is presented. For the pier, supported on three caissons with loads applied at the top, the structural action is similar to that of a deep beam. The purpose of this investigation is to examine possible causes of existing cracks in the pier. The simplicity of the application and the accessibility of the computer program are emphasized. Once the finite element model is prepared, experimentation is possible with the effects of various loading conditions, all feasible support conditions and the application of external posttensioning to prevent further cracking and to relieve internal tension stresses during remedial work.

Keywords: bridge piers; caissons; computer programs; cracking (fracturing); deep beams; evaluation; finite element method; reinforced concrete; stress analysis

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INTRODUCTION

The theoretical basis for applying the finite element method in a structural investigation has been around for a long time. The availability of computers and appropriate software in the market has made it feasible for anyone, with a computing budget and a knowledge of finite element theory, to perform such an analysis. Since many papers have been published in this field, it is essential to preview without delay those aspects of the present work which are of special interest.

The computer program used is available in a published textbook (Ref. 1) and consists of less than 500 FORTRAN statements. It is inexpensive to run and produces excellent results in applications similar to the one described here. A brief explanation of the theory behind the simple finite element used in this paper is provided for the interested reader.

The pier investigation pertains to an orthogonally reinforced concrete deep beam structure which is approximately 30 feet (9 m) wide by 18 feet (5.5 m) deep, and is 2 feet (0.61 m) thick. The beam is supported on 3 caissons. A series of finite element stress analyses has been carried out to examine possible causes of existing cracks in the beam; to determine the caisson loads; and to investigate the effect on the beam of external posttensioning during repair work.

DESCRIPTION OF MANOTICK BRIDGE

The Manotick bridge, constructed in 1955, carries County Road No. 7 over the east channel of the Rideau Canal at Manotick, Ontario, Canada. The structure is a three-span continuous steel girder bridge with an 8 in. (20 cm) reinforced concrete deck. The two reinforced concrete piers are arranged symmetrically about the midspan of the bridge as shown in the general view of Fig. 1. Each pier is supported by three caissons topped with a reinforced concrete pier cap as indicated in Fig. 2. The caissons consist of rectangular concrete filled sheet piling.

In October 1978, a structural investigation of the piers was initiated because of the existence of pier cracking and the deterioration of the caisson pier cap system. Only the typical major pier cracking pattern is shown in Fig. 2. The major cracks, in general, did extend through the piers. The widest crack widths varied between 0.005 in. (0.13 mm) and 0.010 in. (0.25 mm). A severe case of concrete deterioration had taken place in the pier cap and caissons. Concrete had eroded in the pier cap in a pattern similar to that indicated in Fig. 2. The concrete in the piers, however, was not in an eroded state, and by visual inspection, it appeared to be generally of good quality, including that in the region where the cracking had taken place. In addition, at the time of this investigation the soundness of the caissons and the supporting bedrock were not known. The exact state of support of the pier at the time of pier cracking was also unknown.

This paper relates to the finite element stress analysis of the pier in its own plane. The key questions to be answered by the structural analysis were these:

- (i) Could the pier cracking have been caused by the application of normal dead and live loading or else by an extra heavy live loading ?
- (ii) What are the effects of different pier support conditions on pier stresses and possible pier cracking ?
- (iii) Could the pier cracking be the result of the central caisson having settled and therefore being structurally ineffective ?
- (iv) Do caisson forces vary substantially between the outer caissons and the central caisson, a situation that may lead to unequal settlements ?
- (v) While remedial work would be carried out on the pier foundations, what magnitude of external posttensioning of the piers would be required to keep the piers from further cracking ?

FINITE ELEMENT ANALYSIS

Finite Element Method

The finite elements used in this study are triangles having 3 nodes each, a single node in each vertex. A node is a point of connection with adjacent elements. For the particular element used here, the in-plane x and y displacements are linear functions of the corresponding nodal displacements. The x displacement function u , for example, has the form

$$u = a + bx + cy$$

where a , b and c could be calculated for the element from a set of nodal values of the displacements. In finite element analysis, the problem is formulated so that the nodal displacements are calculated from a set of simultaneous equations.

Different linear displacement functions apply in each finite element. This means that mathematical 'corners' in the displacement functions occur across the boundaries. These 'corners' tend to smooth out as more elements are used in the finite element analysis.

Strains are derivatives of displacements, and for the linear displacement model used here, these strains are constants in each element. In the computer program which is listed and documented in Ref. 1, linear elastic behaviour is assumed.

Finite Element Model

The following points relate to reasons for selecting the finite element mesh shown in Fig. 3.

Only one-half of the symmetrical pier has been modelled. Nodes 1 through 10 are constrained for both symmetrical and anti-symmetrical loading cases.

Most nodes are on a 2 foot (0.61 m) by 2 foot grid permitting most of this data to be produced automatically. Some deviations occur to allow nodes to be placed at load points.

The union jack pattern formed by a 4 foot by 4 foot square of elements has not been maintained throughout in order to keep triangle aspect ratios as large as possible.

A more detailed stress pattern immediately under load points and above points of support could have been achieved by concentrating elements in these areas, however, these were not regions of interest in this investigation.

The adequacy of the finite element grid refinement was also investigated, and this is reported in a later section of the paper.

The analysis pertains to a linear elastic homogeneous 2-dimensional solid. There is no attempt to model the cracking in the lower central region. Numerical analysis is still some years away from modelling, in a practical way; the crack interface transfer of force in reinforced concrete, i.e., the combination of friction, bearing and reinforcement forces at a crack. This does not, however, necessarily invalidate the use of linear elastic homogeneous models in this type of investigation.

The cracking in the pier is very limited, and unless the numerical analysis indicates a state of high tension stress, which tends to open the cracks, then no advantage is gained in using a cracked model. The uncracked analysis also demonstrates whether a static state of stress caused the existing cracking. This was one of the requirements of the investigation.

The orthogonal reinforcement, consisting of a single grid layer at each face of #5 horizontal bars and #7 vertical bars 12 inches on centres, is too light to be considered in the analysis and is not modelled here.

The concrete design cylinder strength, f_c' , of 4 ksi (27.5 MPa) was assumed together with a modulus of elasticity of 3620 ksi (25,000 MPa) and a Poisson's ratio of 0.15. The tensile strength, f_t' , was taken as 0.41 ksi (2.8) MPa.

Three different support configurations were selected for study. In one, the stiffening effect of the pier cap is modelled by a fully supported base. In another, the caisson support is applied directly to the pier. In view of the nature and extent of pier cap deterioration, this may or may not be a more appropriate model. In the third model, the center caisson is not present.

Loading Cases Investigated

In the course of the structural investigation, the pier was analyzed for three different conditions of loading:

- (i) The factored design loading consisting of five 252 kip (1120 kN) point loads were applied as shown in Fig. 2. Each of these loads represents a 228 kip dead load and a 24 kip live load. For the symmetrical dead and live loading, all three support conditions were investigated.
- (ii) A heavy lane loading of 200 kips (890 kN) was investigated as shown in Fig. 4(a), as a feasible severe loading condition on the structure. The two 100 kip loads were placed directly on the finite element model as shown in Fig. 4(b). This approximation to the true loading introduces fictitious local distortions of the pier directly under the loads and therefore stresses are not correct in

these areas. In order for the symmetrical finite element mesh to be usable for this case, symmetrical and antisymmetrical cases were superimposed as shown in Fig. 5.

- (iii) The purpose of applying a posttensioning system to the pier is to reduce or eliminate horizontal tension in the pier and to strengthen the pier against the possibility of damage during subsequent remedial work on the caissons and foundation. The tendons are applied at top and bottom pier levels as shown in Fig. 6. Three point loads of 52.2 kips (232 kN) were applied top and bottom as shown in Fig. 7. Each set of 3 point loads is equivalent to 3 pairs of 5/8 in. tendons tensioned to $0.6 f_{pu}$, and applied over a 2 foot depth. It was necessary to convert these to nodal load equivalents for analysis as shown in Fig. 7.

FINITE ELEMENT RESULTS

Principal Stresses

The principal stresses at each element centroid were plotted on a calcomp plotter using a program written in an APL graphics language. Each principal stress was plotted as a scaled line, with appropriate orientation, and arrowhead turned inward for compression, and outward for tension.

The results for dead and live loading under three different support conditions are presented in Fig. 8. The pier outlines, load vectors and support symbols were added to the computer plots later by hand. Note that the highest compression stresses of 9.55 ksi (3.8 MPa) result with outer caisson support only in Fig. 8(a). Small horizontal tension stresses exist in the lower half of the pier with partial base support as shown in Figs. 8(a) and 8(b). With full base support in Fig. 8(c) these stresses are reduced to compression values. The highest tension stresses occur at the top of the pier between the applied loads. The tension stresses at the top are basically the same in each of the three cases and have a maximum value of 0.20 ksi (1.4 MPa).

In summary, the results for all three support conditions indicate that for 4 ksi (27.6 MPa) concrete all compressive stresses are localized. High tensile stresses are less than 50% of ultimate and do not occur in the cracked region. In the cracked region, in Fig. 8(a), the tensile stresses are a maximum of 0.09 ksi (0.62 kPa) at the centerline bottom and decrease to 0.03 ksi (0.21 MPa) in the next stack of elements. These values are not regional maxima since they occur not on the bottom edge, but rather at bottom element centroids. The actual tensile stresses in the bottom edge could be about 20% higher as indicated by a deep beam study reported later in this paper. Even with this 20% increase the maximum, highly localized, tension stresses

cracked region is less than 30% of ultimate. All of these results indicate that for 4 ksi concrete, the given loading, and any of the feasible support conditions, pier cracking would not be expected to occur.

The principal stress results for the lane loading showed moderate compression and tension stresses. The tension stresses in the cracked region became smaller suggesting no pier cracking under a severe lane loading. Hand calculations were required for this loading, since the rectangular stresses from the two finite element models for the two support conditions shown in Fig. 5, had to first be combined. The stress results calculated in the cracked region were found by combining directly the results represented by Figs. 5(a) and (b) for right of centerline; and combining the results represented by Fig. 5(a) and the negative of Fig. 5(b) for left of centerline.

The principal stresses resulting from the application of the posttensioning system, in combination with dead and live loading and caisson support, are given in Fig. 9. The top and bottom tendon forces have eliminated the bottom tension stresses and reduced the top stresses.

The total experimentation with the finite element model is reported in detail in Ref. 2.

Caisson Forces

Bearing stresses at the caisson points of support are proportional to the caisson forces. By balancing these forces with the total gravity load, the caisson reactions were calculated. From the numerical finite element output, the results of these bearing force calculations are as follows:

Dead load plus live load-- Center caisson: pier bearing stress = 0.19 ksi (1.3 MPa); caisson reaction = 328 kips (1459 kN)
Outer caisson: pier bearing stress = 0.27 ksi (1.9 MPa); caisson reaction = 466 kips (2073 kN)

Lane load plus dead load-- Center caisson; pier bearing stress = 0.21 ksi (1.4 MPa); caisson reaction = 299 kips (1330 kN)
Outer caisson: pier bearing stress = 0.36 ksi (2.5 MPa); caisson reaction = 521 kips (2317 kN).

The difference in caisson forces clearly indicates the potential for differential settlements of the caissons.

VALIDATION OF RESULTS

A verification of the validity of the results of a finite element analysis is especially important when a problem type is solved for the first time using a particular element. A convergence study is carried out by subdividing the mesh, and monitoring the convergence of some aspect of the structural model's response.

for example the strain energy content, the deflection at a point, and so on. Strain energy was not used as a basis for comparison here, because of the fact that the singularities at the points of support were not modelled. Since the areas of interest were somewhat remote from these points, centerline stress was considered an appropriate basis for convergence.

The verification of the model is provided by both a direct comparison with results published by Leonhardt (Ref. 3) for a similar structure; and also by mesh subdivision. The comparison example, having a shear span similar to that of the Manotick Bridge pier, is shown in Fig. 10.

A comparison of horizontal stresses at the centerline is given in Fig. 11. As expected, the Navier solution is not valid but the finite element analysis and the Leonhardt solution are in close agreement. Note that the finite element results are not taken from the exact centerline of the beam but from a stack of elements close to the centerline. These element centroids plot on a zig-zag line from the bottom to the top of the beam.

A comparison of results along the centerline is given in Table 1 for 2 successive mesh subdivisions and the results from Leonhardt (3). A further subdivision to 360 elements was attempted, but evidence of numerical roundoff errors appeared, and it was necessary, therefore, to maintain an upper limit on the number of elements. The finite element stresses for the 90 element mesh are considered to have converged and are expected to be slightly different from Leonhardt's because they are not exactly on the centerline. A comparison with results, such as those of reference 3, should be sought whenever possible, since these do provide a basic correct solution to the homogeneous linear elastic problem.

CONCLUSIONS

Based on the total results of this study, outlined in general in this paper, the following conclusions and recommendations are made:

- (i) The results of the analyses for dead plus live load indicate that the pier would not crack under any of the three assumed support conditions. This statement holds even for the most critical situation where the pier cap and center caisson are structurally ineffective.
- (ii) The application of a severe lane loading produced no major tensile stresses which could lead to pier cracking.
- (iii) The structural analysis results show that substantially higher forces exist in the outer caissons which in turn could lead to significant differential settlements. The relatively rigid piers would be sensitive to even small differential foundation movements. In combination with

vibration brought about by loose seating of the caissons, these factors could account for the present cracked state of the piers.

- (iv) Posttensioning cables applied at the top and bottom of the pier are recommended as a safeguard against additional cracking before remedial work on the caisson and pier cap is implemented.

This paper has outlined a numerical investigation of a structural element for which a simple rational theory does not exist. The finite element analysis and the code design checks given in detail in Ref. 2 indicate that the pier was well designed. However, when problems occur in such a structure some of the more rugged procedures available for design are not suitable for investigation. It is fortunate that a simple finite element program is capable of producing cheap, good quality results. This procedure enabled a degree of experimentation with several loading cases and all feasible support conditions. It also made possible the accurate calculation of caisson forces and a study into the effect of posttensioning.

The finite element program used here, and several others having the same capabilities are readily available and can be run on personal computers with a FORTRAN compiler. This type of structural investigation is thus within the means of the smallest structural consulting offices.

REFERENCES

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- (3) Leonhardt, F. and Walther, R., Wandartige Traeger (wall-like elements), Deutscher Ausschuss fuer Stahlbeton, Nr. 178, 1966.

Table 1 Finite Element Convergence Study

	Photoelasticity*	180 elements	90 elements
Position of centerline neutral axis	0.28	0.27 h	0.27 h
σ_x value at bottom centerline	1.60	1.56 $\frac{q}{b}$ **	1.55 $\frac{q}{b}$

* Leonhardt's results

** Uniform load intensity, q divided by out of plane beam width, b.

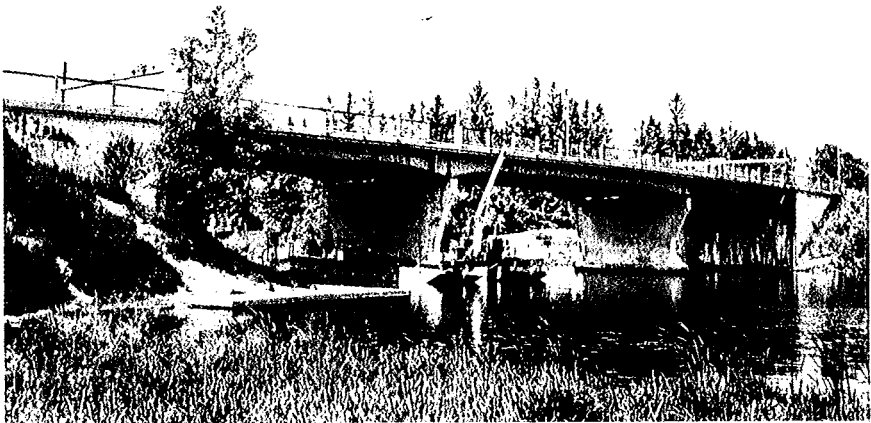


Fig. 1--Manotick bridge

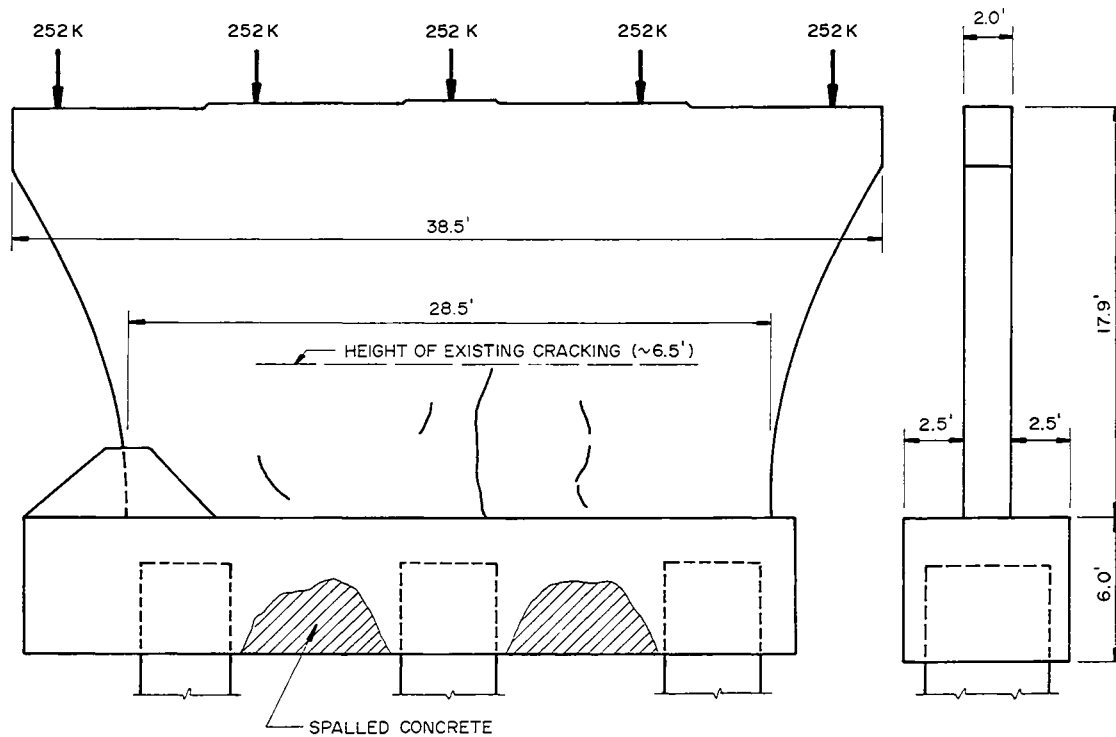


Fig. 2--Manotick bridge pier

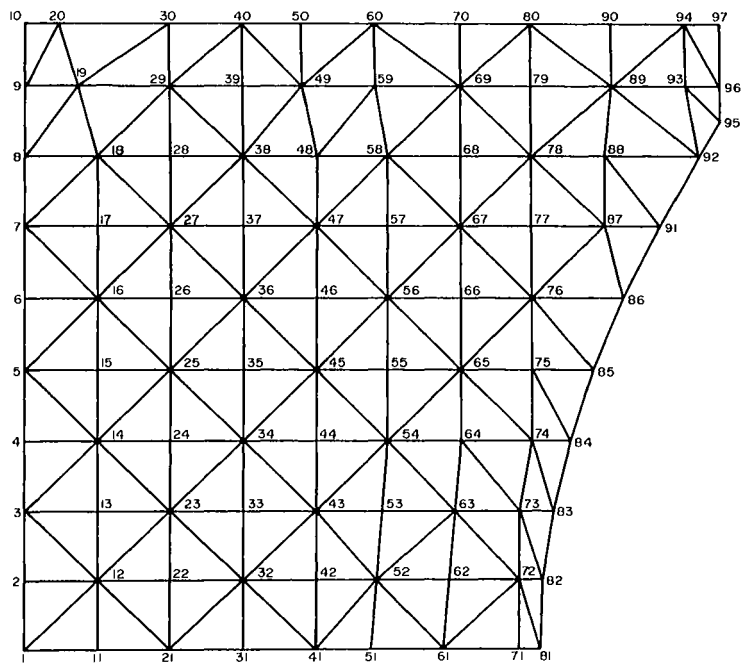
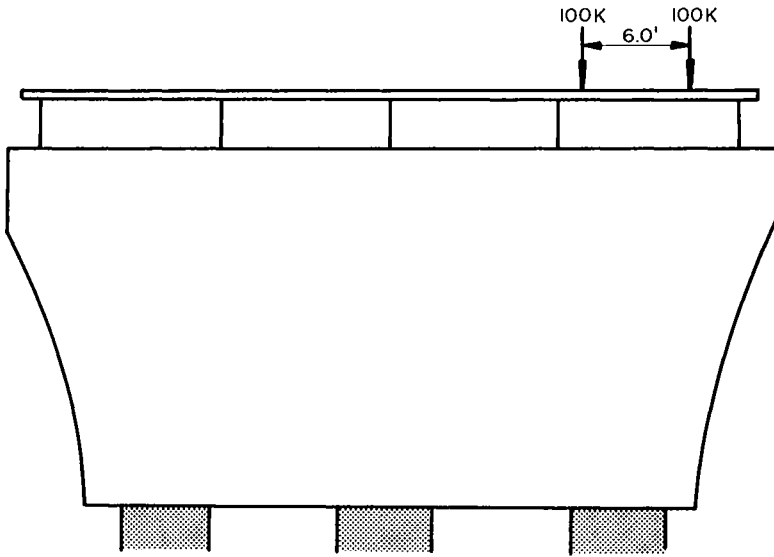
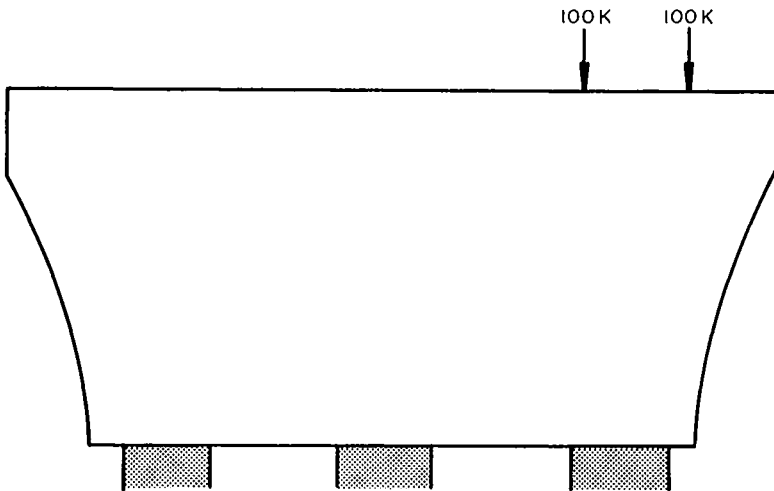


Fig. 3--Finite element mesh, node numbers



(a) ACTUAL LOADING CONFIGURATION



(b) ASSUMED LOADING CONFIGURATION

Fig. 4--Lane loading

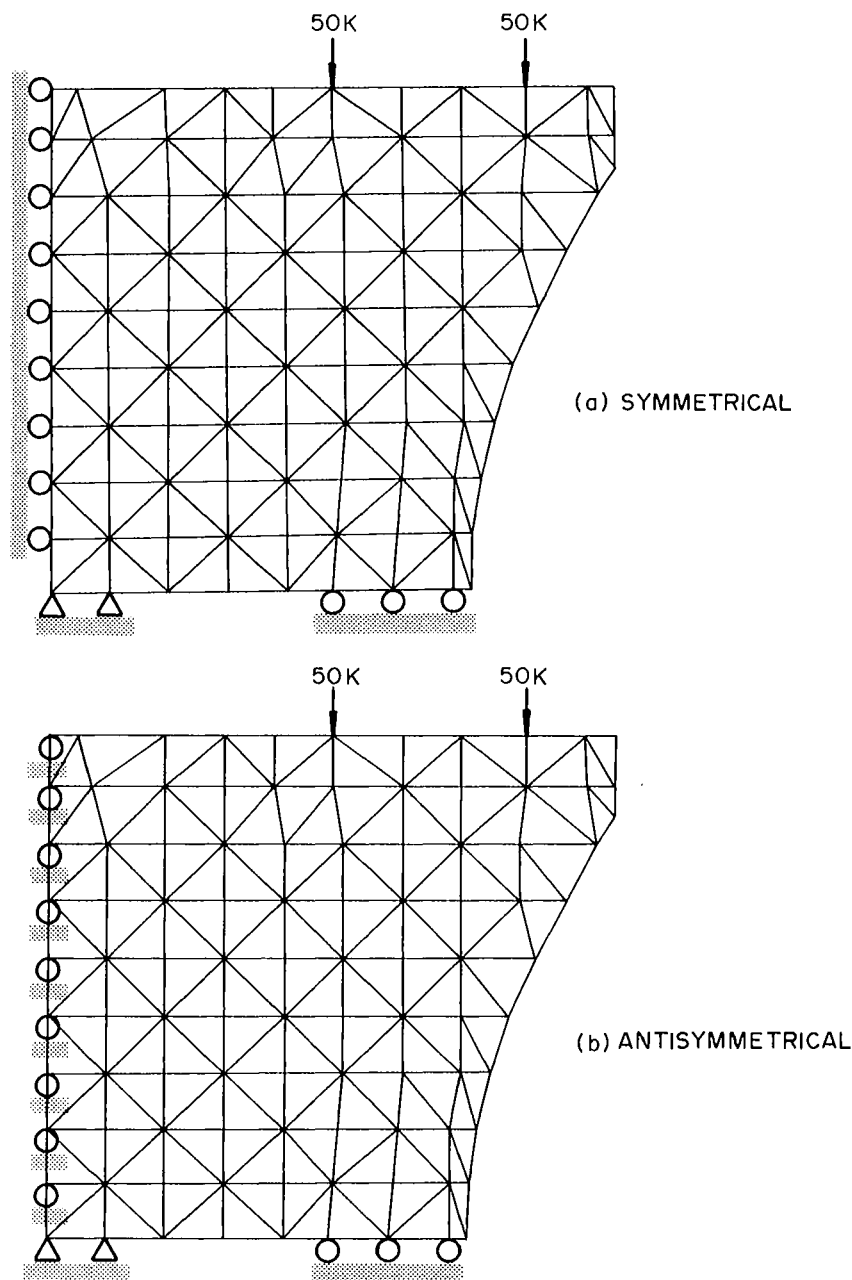


Fig. 5--Symmetrical and antisymmetrical cases to be superimposed for unsymmetrical loading

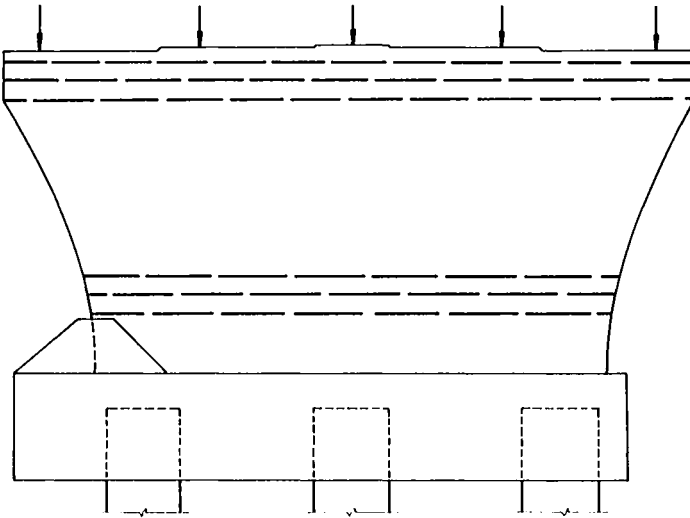


Fig. 6--Application of posttensioning system

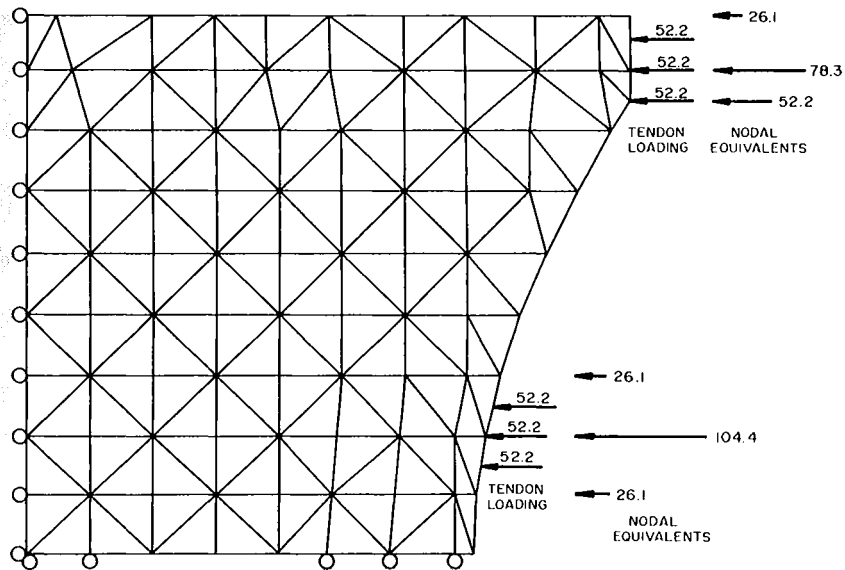


Fig. 7--Tendon loading

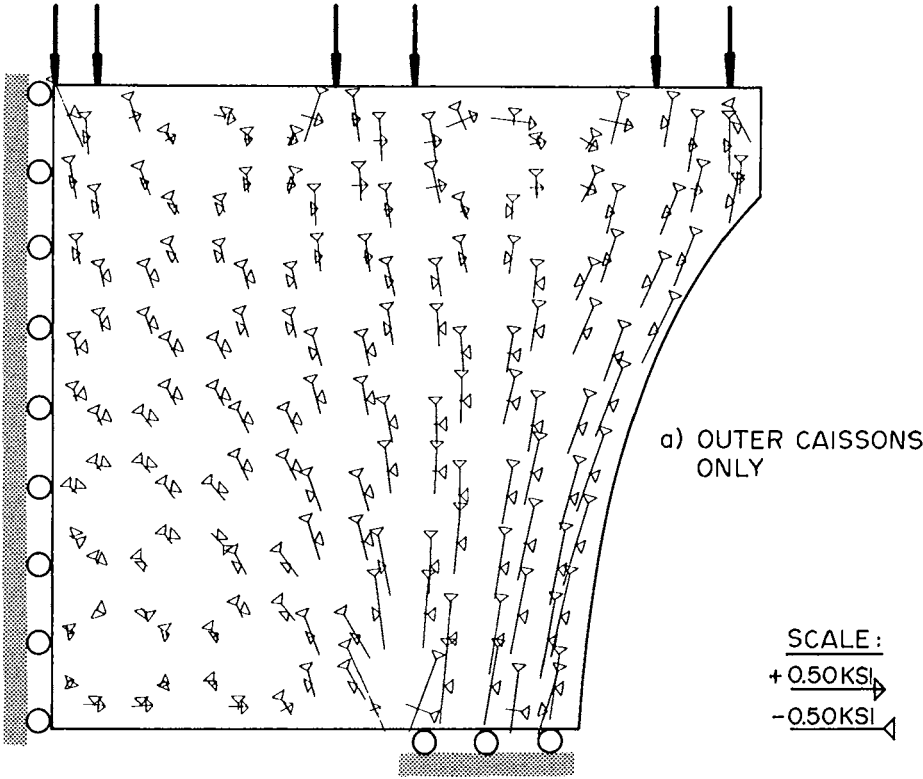


Fig. 8--Principal stresses

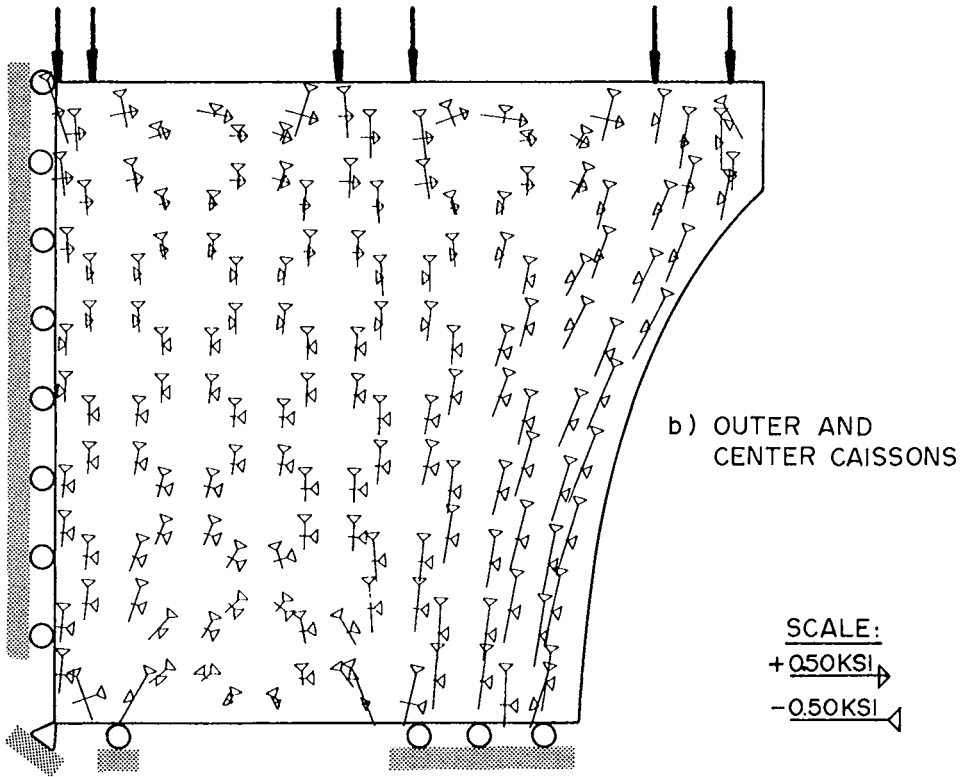


Fig. 8--Principal stresses (continued)

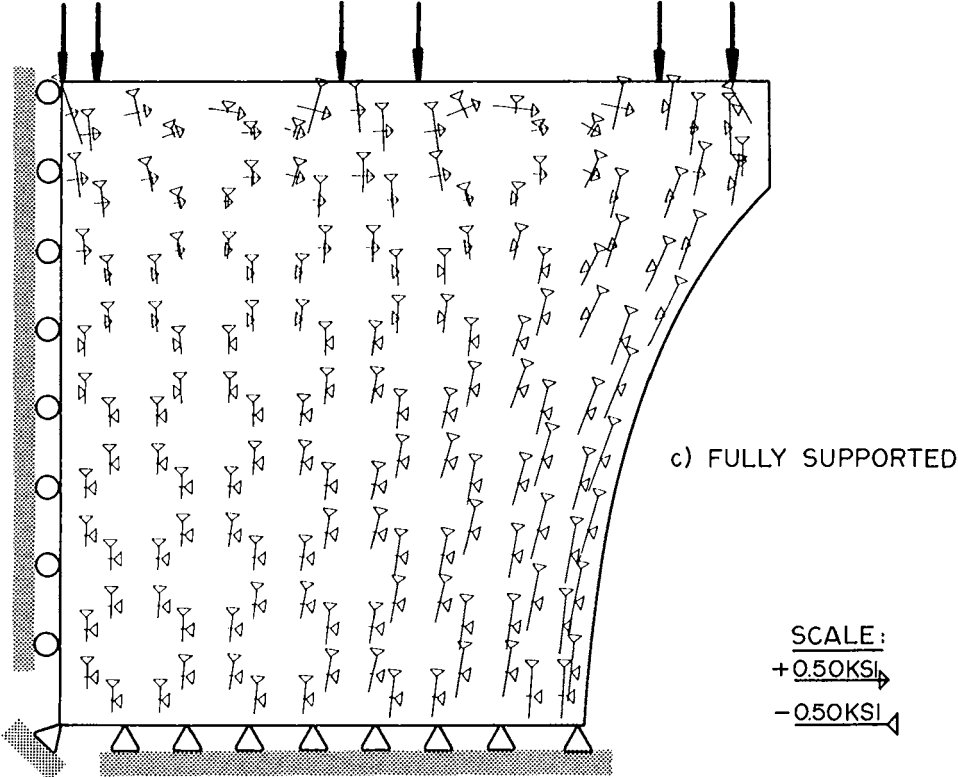


Fig. 8--Principal stresses (continued)

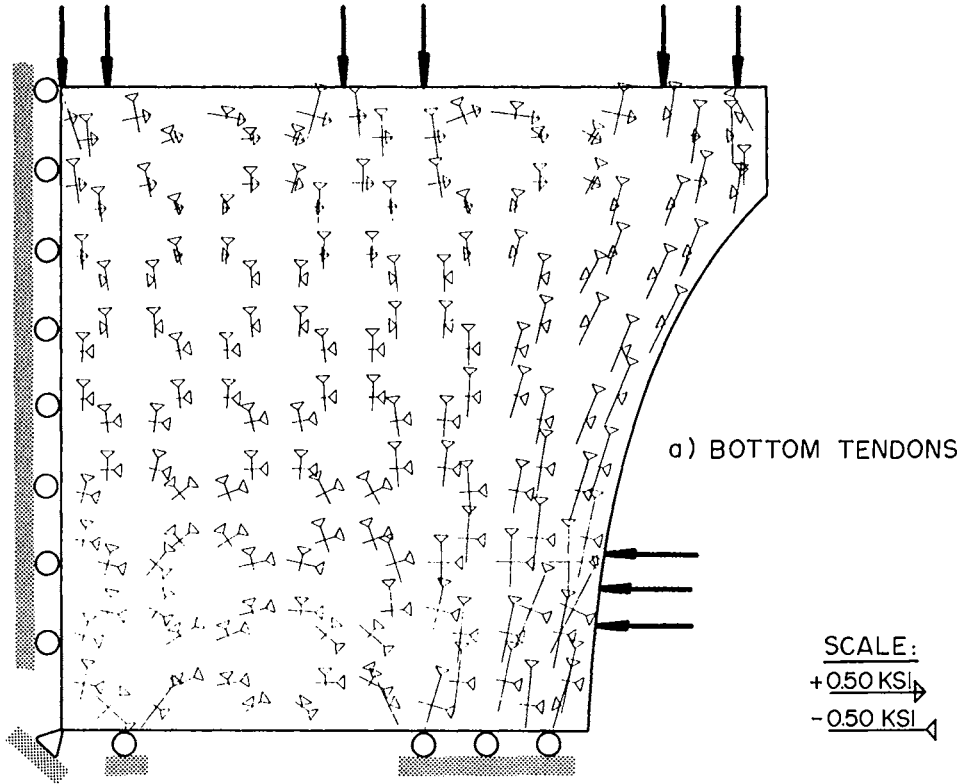


Fig. 9--Principal stresses due to posttensioning

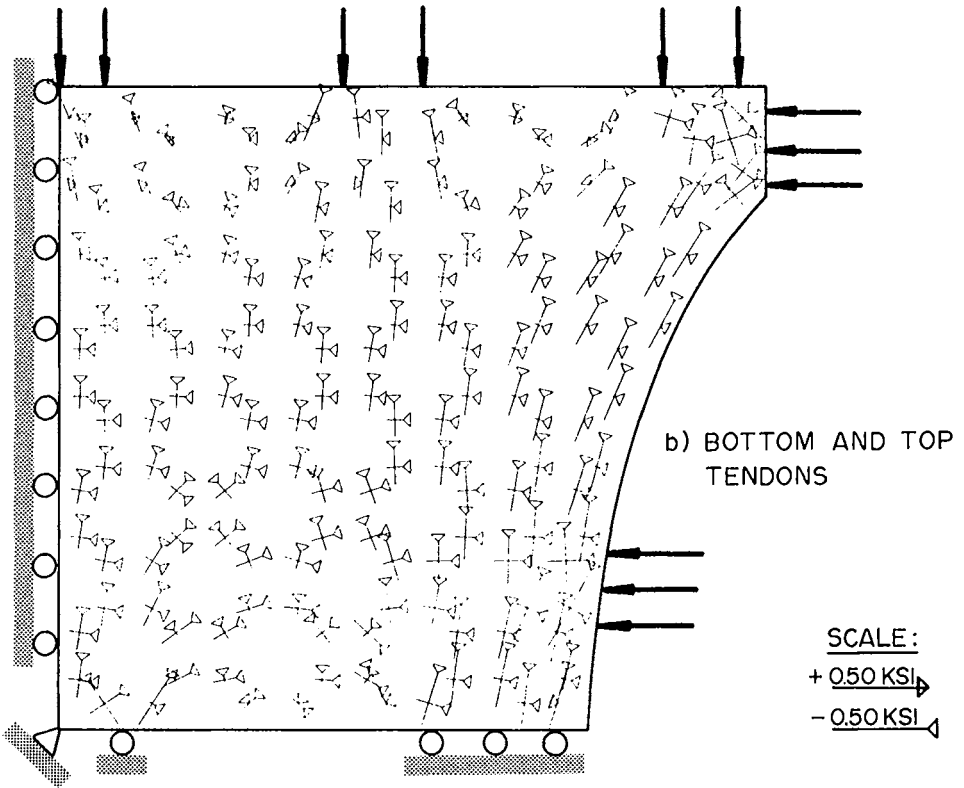


Fig. 9--Principal stresses due to posttensioning (continued)

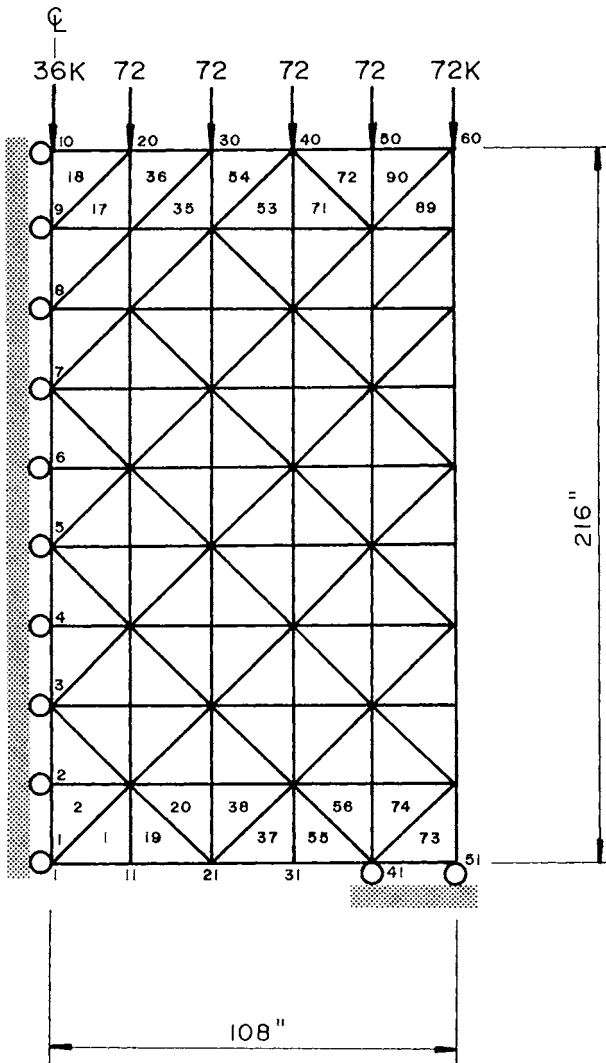


Fig. 10--Comparison example

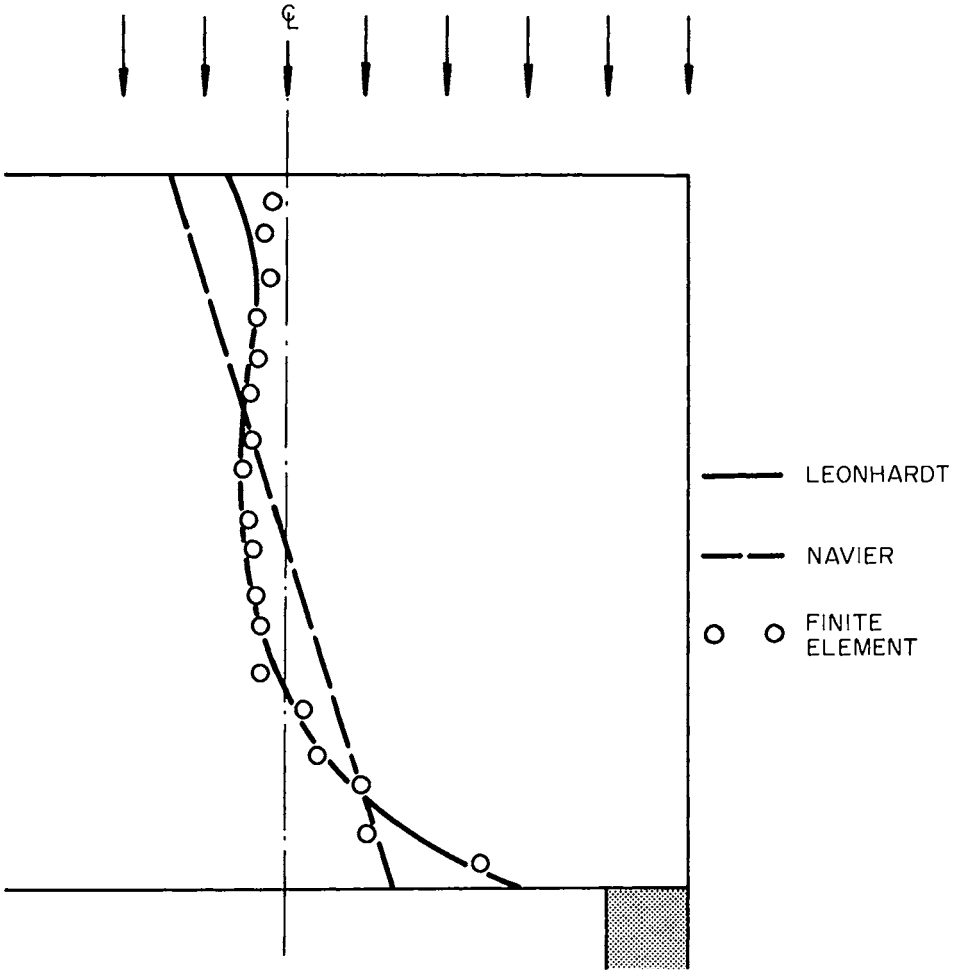


Fig. 11--Comparison run

Load Tests on Prestressed Concrete Multi-Girder Bridge Decks

by J. Kirkpatrick, A. E. Long,
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Synopsis: The results of load tests on four prestressed concrete multi-girder bridge decks are presented. The girders have been used to form both tee and pseudo box decks each of which has its own distribution characteristics. The measured distribution of beam soffit stresses are compared with predictions using a grillage analysis. For the beam construction good correlation was obtained but for the pseudo box deck measured stresses were less than those predicted. In the latter case good correlation was obtained by substantially increasing the torsional stiffness of the transverse grillage members.

Keywords: beams (supports); bridge decks; bridges (structures); loads (forces); load tests (structural); precast concrete; pre-stressed concrete; stiffness; structural analysis

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Dr W.M.C. Stevenson is Superintending Engineer (Structures) in the Roads Service of the Department of the Environment for N. Ireland having previously worked for Sir William Halcrow and Partners for 10 years. Dr Stevenson received his BSc and PhD at Queen's University, Belfast and he is currently responsible for the design and maintenance of all bridges in N. Ireland.

Mr A. Thompson is a Senior Research Officer in the Civil Engineering Department at Queen's University, Belfast. He has been responsible for the instrumentation of various tests on site and in the laboratory and has developed new equipment.

INTRODUCTION

In the United Kingdom precast prestressed concrete bridge beams (1) cover the range of spans from 15 to 29 m. As a high proportion of highway bridges lie within this range of spans they are widely used as they offer an economic solution to most railway/river crossings and busy road intersections where interference must be kept to a minimum.

These standard beams have a bottom flange approximately 1 metre wide and are generally referred to as M beams. They act compositely with a cast in-situ slab and a grillage analysis is often used for analysis. The accuracy of this analysis depends on the idealisation of the grid mesh and the method utilised for the calculation of the section properties of the grid members. These have been largely based on the results of model tests and few, if any, full scale tests have been carried out to check their validity.

As this type of beam was being used extensively in N. Ireland four bridges were tested to establish their load distribution characteristics with the objectives of improving current design procedures and reducing construction costs.

FORMS OF CONSTRUCTION

The beams are normally placed side by side with a minimum gap and can be formed into two distinct types of deck which may be defined as:-

- (1) Inverted Tee (Fig. 1(a)) where a top slab is cast on permanent formwork.
- (2) Pseudo Box (Fig. 1(b)) where a bottom slab is formed as shown and reinforced with high yield bars which extend through preformed holes in the web of the beams. The top slab is then added as for tee beam construction.

To complete the deck an in-situ concrete upstand is cast on the edge beam to accommodate the parapet rail. This edge feature contributes to the stiffness of the edge beam and influences the distribution of live load.

TEST PROGRAMME

The bridge decks tested had previously been designed by the Department of the Environment for N. Ireland. Four bridges were tested covering both tee and pseudo box construction, each having one right and one skew deck. Details of the bridges are given in Table 1.

Concrete Control Data

In decks of this type a high strength concrete with a cube strength of at least 52.5 N/mm^2 ($f'_c = 42 \text{ N/mm}^2$) at 28 days is usually specified for the prestressed beams. These act compositely with a lower grade in situ slab with a specified strength of 37.5 N/mm^2 ($f'_c = 30 \text{ N/mm}^2$) at 28 days. In order to ensure that the beams were satisfactory one beam in each bridge was nominated for load testing. This involved the application of concentrated loads at third span and the measurement of mid span deflections. For an acceptable test a substantially straight load/deflection curve and a recovery of 90% of the deflection was required. From the measured deflections values of the modulus of elasticity ranging from 37,000 to 49,000 N/mm^2 were calculated and found to be well above the value predicted using the design recommendations and the specified cube strengths (2). Load tests on the bridges were not carried out until the in situ slab had achieved its 28 day cube strength. Details of concrete strengths are given in Table 2.

It should be noted that as beams with deflected tendons were not available in N. Ireland a system of debonded tendons was adopted in all cases.

Instrumentation

The strain distribution across the deck was monitored using Tyler surface mounting recoverable vibrating wire gauges of 140 mm gauge length which were attached to the beam soffits at mid span. Usually every beam in one half of the deck was gauged as well as alternate beams in the other half, however, all beams were gauged on the Tullyear Bridge as it had only 11 beams in total. Typical gauge positions for Granville and Tullyear Bridge decks are shown in Fig. 2.

In order to monitor the variation in temperature during the tests thermocouples were mounted on the beams adjacent to the strain gauges. It should be noted that all tests were carried out on dull, overcast days with little wind so as to minimise strains caused by temperature variations.

To monitor bearing deflections dial gauges were mounted on the abutment as close as possible to the bearing pads and on the centre line of each beam.

Test Loads

Static loading was applied to the deck by two 6-wheel trucks, loaded with crushed stone, the total weight of which had been accurately measured prior to arrival on site. However, as accurate estimates of individual wheel loads were required, for use in a theoretical grillage analysis, these were measured using a Digital Portable Weighbridge (3).

The loading applied to all decks consisted of two trucks each weighing about 30 tonnes. Five loading positions were considered: 1, 2 and 3 at mid span and 4 and 5 local to the bearings. Strain gauge measurements were taken for cases 1 to 3 and bearing deflections for all 5 cases. Typical loading positions for the Granville and Tullyear Bridges are shown in Fig. 2.

Test Procedure

Prior to the tests the various positions for the trucks were carefully marked out on the deck to ensure the locations of the wheel loads were known precisely. In addition, before the trucks were driven onto the deck zero load readings were taken on the strain recorder, the deflection gauges and the thermocouples. The trucks were then driven onto the deck and positioned as required. A complete set of readings was then taken for each loading position and also when the trucks were removed.

In order to establish the influence of the concrete edge feature on the performance of the deck this procedure was carried out before and after the upstand was cast.

PRESENTATION AND DISCUSSION OF RESULTS

All period readings from the strain meter were converted into strains and as outlined below into concrete stresses. Although a value for the elastic modulus was calculated at the time of the individual beam tests for each bridge a considerable period of time had elapsed before the deck loading test was carried out. As a consequence of this and as the beams were then composite with the relatively variable strength of concrete in the slab a straight conversion of strain to stress by the elastic modulus was considered unsatisfactory. The method adopted was to use a nominal elastic modulus to convert strains to stresses and then to use the nominal composite beam section modulus to convert this to individual beam moments. These moments were then summed across the deck and the result compared with the total bending moment applied to the deck at mid span by the two trucks. The final individual beam bending moment was then calculated as follows:-

$$\text{Final bending moment in each beam} = \frac{M_t}{M_m} \times \text{original nominal bending moment in each beam}$$

Where M_t = Total bending moment applied to the deck from the truck wheel loads

M_m = Sum of the beam bending moments based on a nominal elastic modulus

This adjustment ensured that the mid span section under consideration was in equilibrium and to some extent took into account variations in the modulus of elasticity of the concrete and irregularities due to construction tolerances.

Theoretical Analysis

A theoretical analysis of all the test bridges was carried out using a grillage analysis (4) which allowed both series of tests with and without the parapet upstand to be considered. The bridge decks were idealised and section properties calculated in accordance with the recommendations by West (5) which suggest that the longitudinal grillage beams should be formed from about two physical beams and the overall grid mesh should correspond approximately to the aspect ratio of the deck. Section properties are proportioned in a similar manner.

Tee Beam Decks

The Granville Road Bridge and Bann River Bridge are both of this type and the mid span distribution of beam soffit stresses for the former are given in Fig. 3. The results of tests with and without the parapet upstand are shown and these are compared with the theoretical grillage analysis. Good overall correlation has been found to result for this type of deck indicating that the method of idealisation is realistic.

However, in the immediate vicinity of the wheel loads the test results show a more localised effect than that predicted by the grillage analysis which was based on about two physical beams to a grillage beam. This local effect may be seen, on the distribution of beam soffit stresses, as high peaks on the beams on which the axle loading occurred. When a more refined grillage of one physical beam to one grillage beam is used, as can be seen in Fig. 3, it shows a much better prediction of these peak stresses. However, in practice the use of a fine mesh will increase both data preparation and computer time considerably and if storage is a problem only a limited number of loading cases may be run at any one time. For design purposes the coarse mesh as suggested by West (5) will give a reasonably accurate estimate of the distribution of load.

In Fig. 3, Loading Case 1, it can be seen that the effect of edge stiffening by the parapet upstand is to reduce the beam soffit stresses in the beams near the edge. It is of interest to note that for this loading case the soffit stresses from the grillage analysis, without the upstand, are of the same order as those obtained from the test with the upstand included. This would suggest that this relatively simple analysis is adequate which would have the advantage that it avoids the complexity of assigning appropriate section properties for the upstand in the alternative grillage analysis. On this basis it is recommended that the deck analysis should be based on the properties of the composite beams alone. Another factor favouring this approach is that it provides conservative estimates of beam moments. This particular loading case can be seen to be the most critical and indeed will in almost all circumstances be utilised for the selection of the beam size and for that matter the levels of prestress of all the beams in a deck.

Pseudo Box Decks

The Tullyear and A50 Road Bridges were both of this form of construction and the distribution of beam soffit stresses for the Tullyear without the parapet upstand is given in Fig. 4 and with the upstand in Fig. 5. For the tests without the parapet it may be seen that for loading case No. 1, which is the most critical, the measured stresses on the soffit of beam No. 1 were about 20% less than those predicted by the grillage analysis based on the recommendations of West (5). This indicates that the pseudo box deck has a greater transverse torsional stiffness than had been assumed in West's idealisation. On further investigation it was found that an under-estimate of the torsional inertia of the idealised transverse grillage members occurs when calculated by the method of Kollbrunner (6) which is the method recommended by West (5). A good correlation of the test results is achieved by increasing the torsional inertia of the transverse grillage beams by a factor of about 7 and the results of this analysis are shown in Figs. 4 and 5.

It should be noted that West's recommendations for this type of deck were based on a one quarter scale model using Inverted Tee beams (7) which are approximately two thirds the width of the standard M beam. This means that for the same width of deck the model tested had significantly more cells than would have been the case if M beams had been used. Also, if cracking occurs between each beam due to transverse bending, more cracks would be present which would adversely effect the distribution properties of the deck. Therefore it is not unreasonable that the deck with broader beams should show considerably stiffer transverse distribution properties which result in a reduction in the magnitude of the longitudinal bending moment and superior load distribution characteristics. However, it is worthy of note that the two 30 tonne trucks used to load the test bridges may not have been sufficient to cause cracking of the in situ concrete between the beams and a high estimate of the transverse deck stiffness may have been obtained relative to its long term performance. In contrast in the one quarter scale model of West extensive cracking had developed due to the application of a test load equivalent to an abnormal vehicle of 180 tonnes. It is hoped that after a period of service these bridges will be tested again to establish if any change in the transverse deck stiffness has occurred due to cracking between the beams.

With the addition of the edge feature the measured stresses on the beam soffit of the edge beam show very little difference and it would seem that the edge stiffening does not have a very significant effect. This is to be expected as the pseudo box deck behaves in a similar manner to a solid slab with a very flat distribution of load and the addition of an area of slab would not make any appreciable difference to the distribution of stress in the beams near the edge.

Again, the analysis of this type of deck may be simplified in a similar manner to that for the tee beam deck. As may be seen from Figs. 4 and 5 (loading cases No. 1 and 3) the beam soffit stresses from the grillage analysis without the upstand are of the same order as the test results with the upstand. Therefore, it is again suggested that an analysis based only on the composite beams will give a satisfactory design in the region of the edge beams.

The Tullyear Road Bridge has only 11 beams and strain gauges were attached to all of them which allowed a check on the symmetry of results. Loading case 2 is symmetric and loading cases 1 and 3 are anti-symmetric and all of these cases gave good agreement as regards the symmetry of results which was a good check on the performance of the strain gauges and monitoring equipment.

Bearing Deflections

Typical bearing deflections for the Granville Road Bridge are shown in Fig. 6. In comparison with the results for the grillage analysis there is a similarity of deflected shape, however, in general the use of dial gauges was considered to be unsatisfactory

and at best gave only a general indication of the deflection pattern.

CONCLUSIONS

Loading tests carried out on bridges immediately after construction provide an inexpensive and effective means of assessing the behaviour of multi-girder bridge systems. When the test results are compared with grillage analyses predictions the following conclusions can be drawn:

- (1) For tee beam decks, the use of a grillage analysis with a relatively coarse mesh gives good agreement for both decks tested, Although the test results did show local peak stresses under the wheels, it was found that the results obtained were adequate for design purposes.
- (2) For pseudo box decks measured beam soffit stresses were about 20% less than those predicted by grillage analysis. However, good correlation of beam soffit stresses was obtained when the torsional inertia of the idealised transverse grillage beams was increased by a factor of about 7.
- (3) For both forms of construction the effect of edge stiffening associated with the standard edge feature need not be considered and the analysis can be based on the stiffness of the composite beams alone. This will provide a conservative estimate of beam moments in beams close to the edge of the deck.

Bridge Detail	Bann River	Granville Road	A50 Road Bridge	Tullyear Road Bridge
Type of Construction	Tee	Tee	Pseudo Box	Pseudo Box
Effective Span - m	24	23.03	18.6	16.6
Beam Size Depth - mm	M8 1200	M7 1120	M5 960	M2 720
No. of beams	17	20	21	11
Skew ^o	0	14	26	0

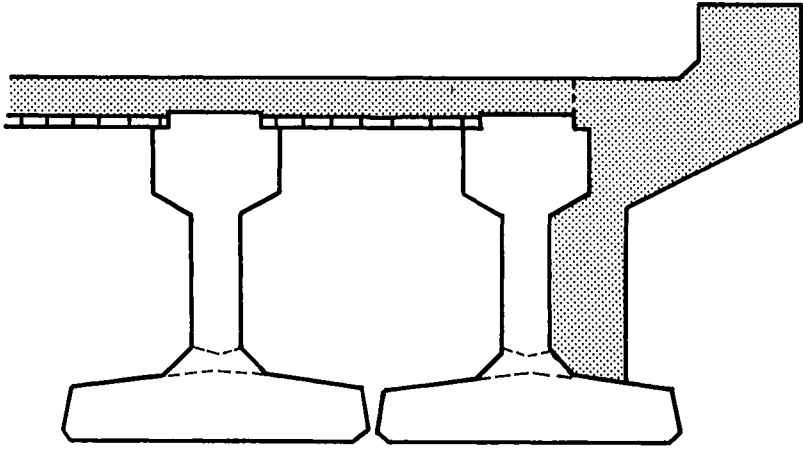
Table 1 Details of Test Bridges

Bridge	Bann River	Granville Road	A50 Road	Tullyear Road
<u>Beams</u>				
Design 28 days cube strength - (N/mm^2)	52.5	52.5	52.5	52.5
Average 28 day cube strength of concrete (N/mm^2)	60.5	57.0	56.0	62.5
Range of cube strength of concrete (N/mm^2)	56.6-60.5	56.3-62.1	55-59.3	57-64
Age at time of test (days)	28	30	48	64
Modulus of Elasticity (E) at time of beam test (kN/mm^2)	37.2	40.4	39.0	49.2
Age at time of deck test				
No. 1	362	276	294	209
No. 2	405	326	348	314

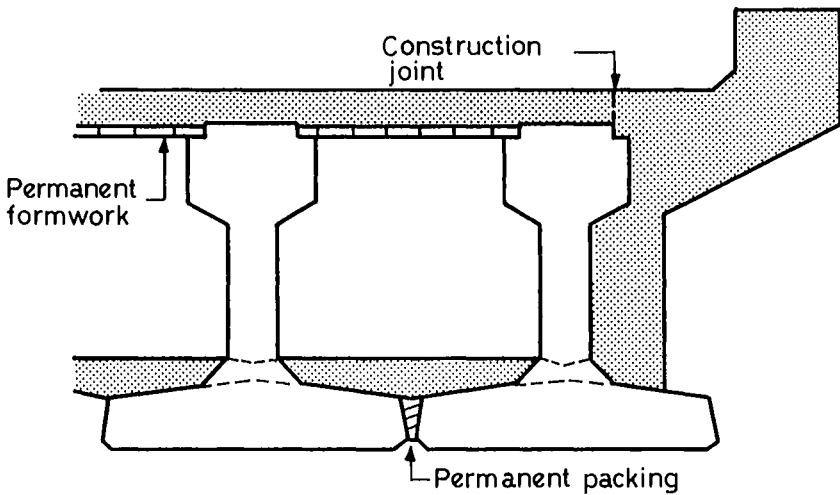
Table 2 Concrete Beam Data

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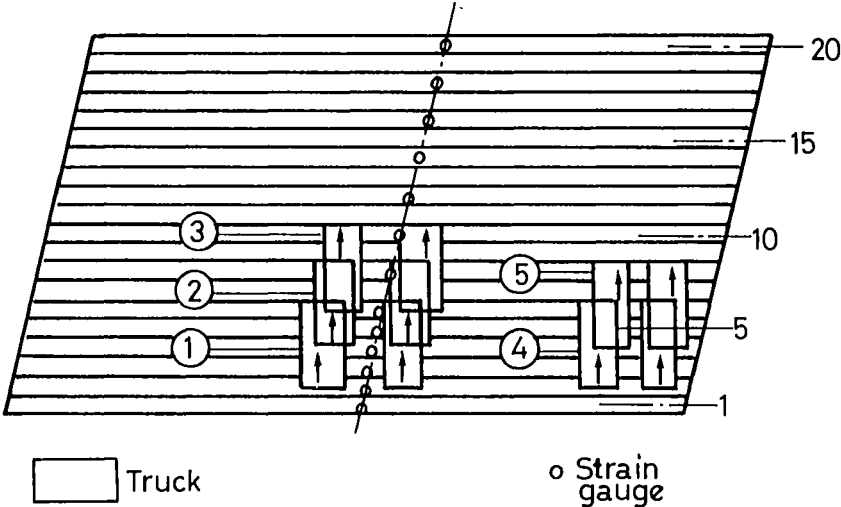


(a) T - BEAM

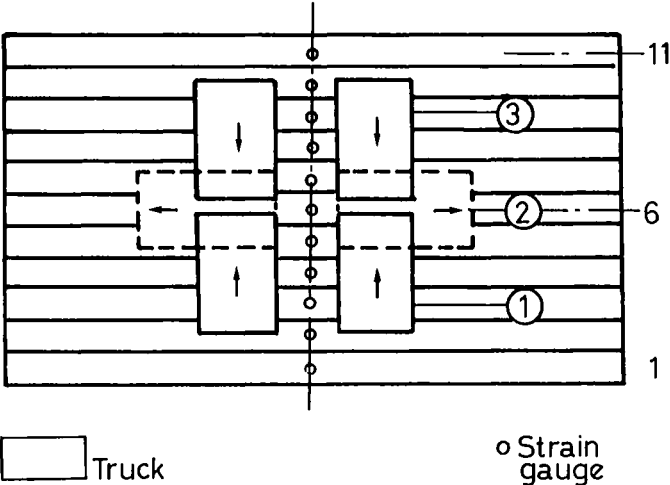


(b) PSEUDO - BOX

Fig. 1--Standard forms of deck construction using M-beams



GRANVILLE ROAD BRIDGE



TULLYEAR ROAD BRIDGE

Fig. 2--Strain gauge locations and loading positions for trucks

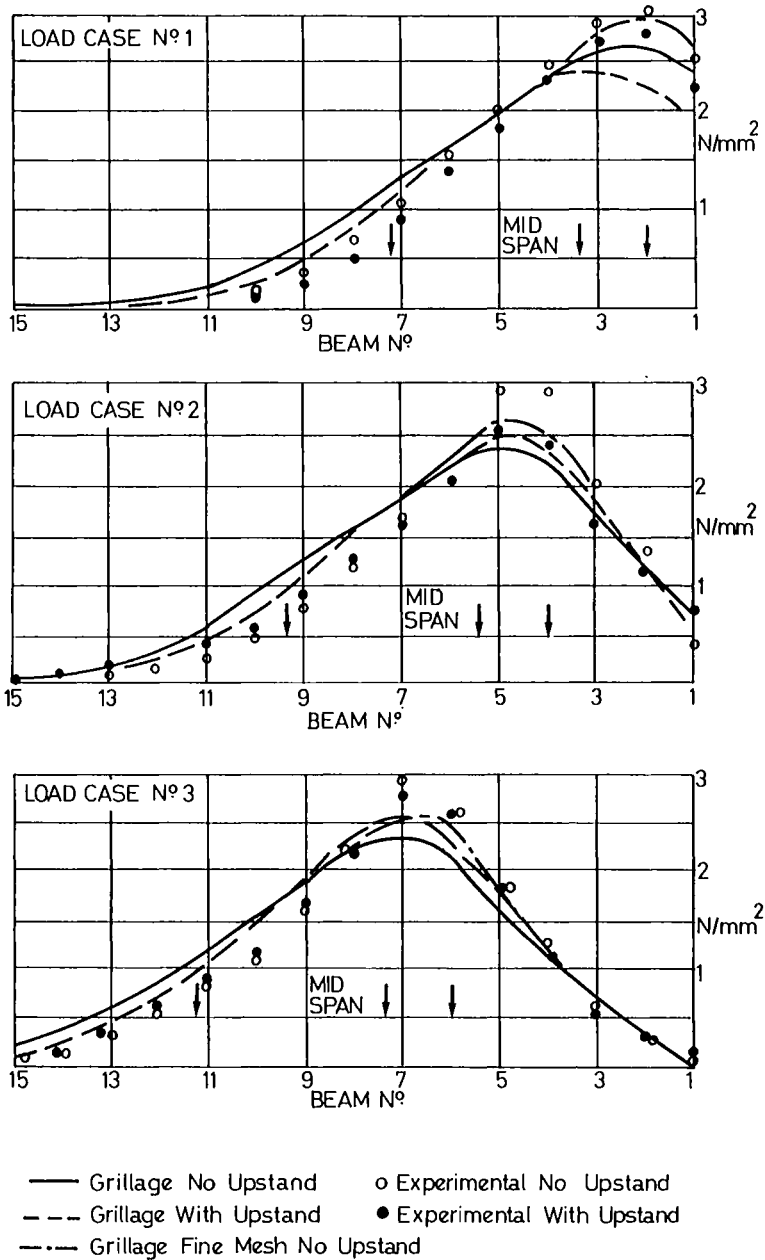


Fig. 3--Granville Road bridge (T-beam) beam soffit stresses

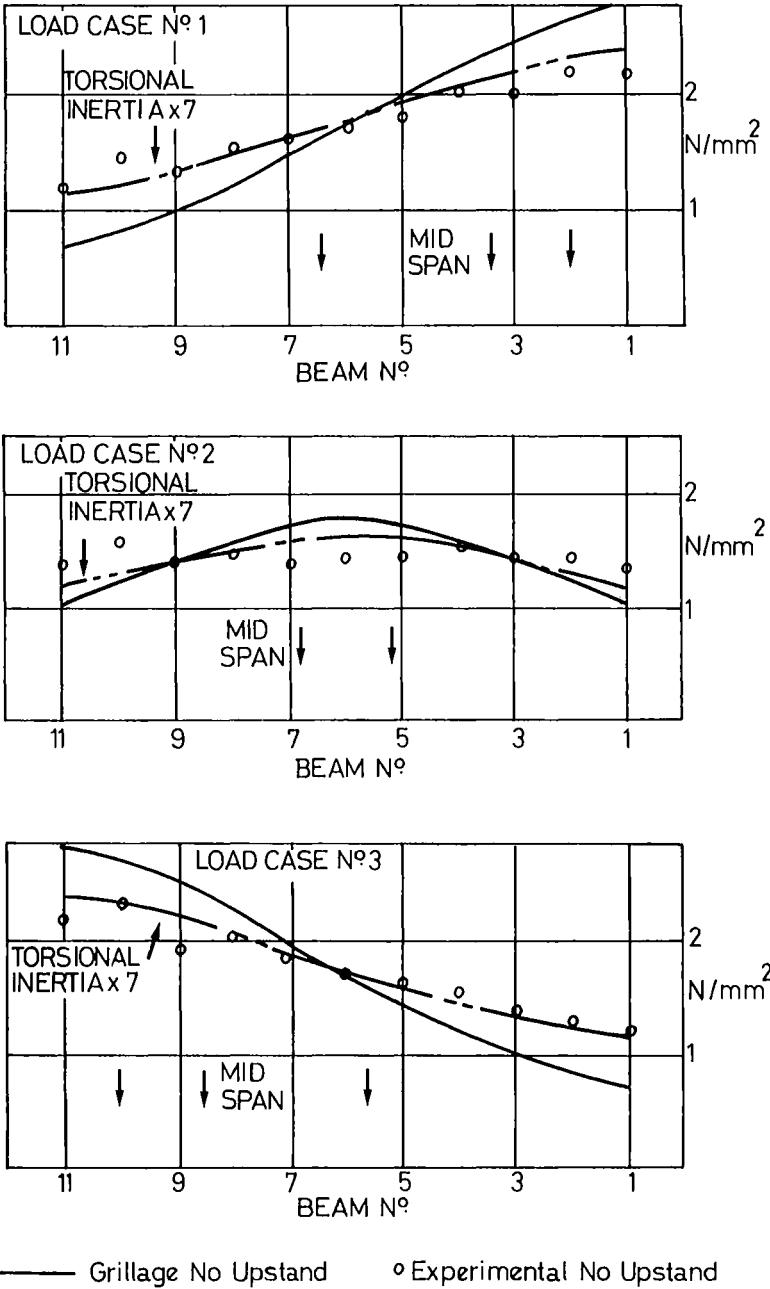


Fig. 4--Tullyear Road bridge (pseudo-box) beam soffit stresses

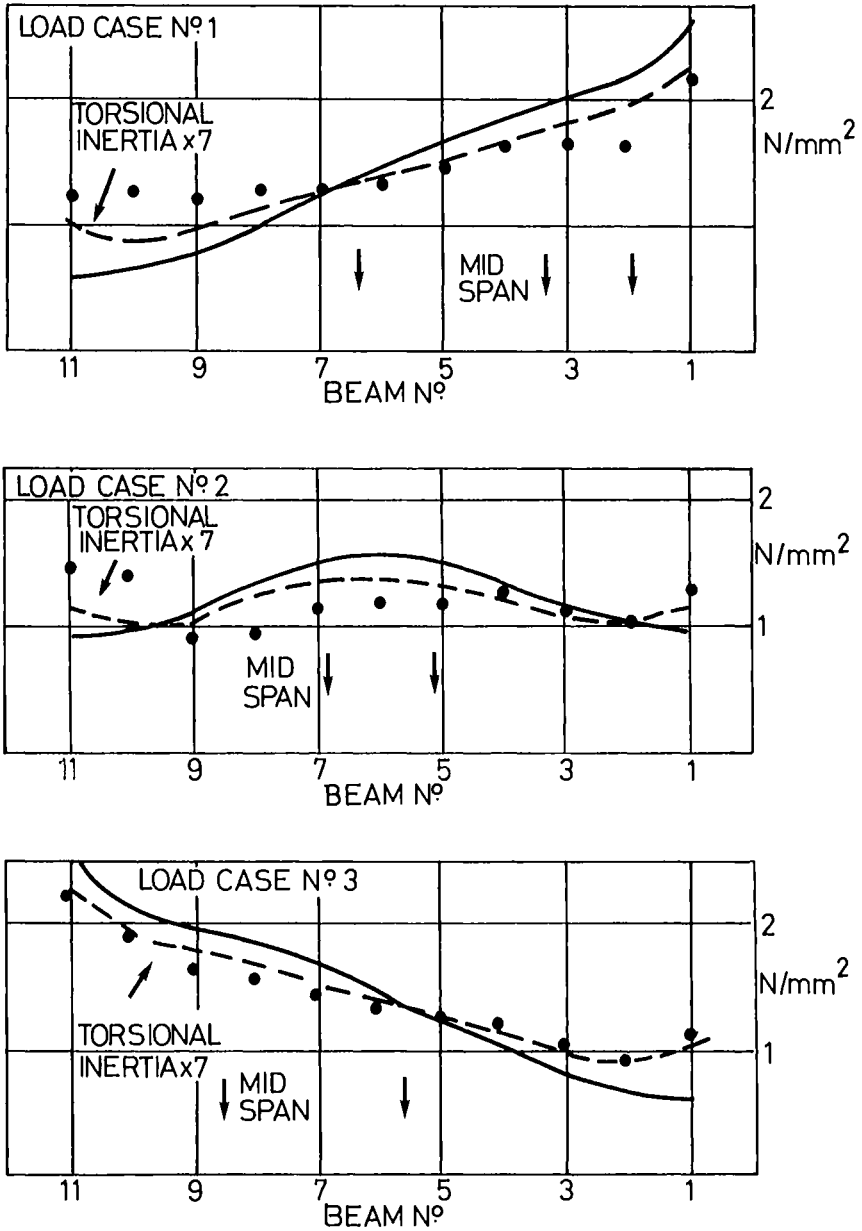


Fig. 5--Granville Road bridge bearing deflections

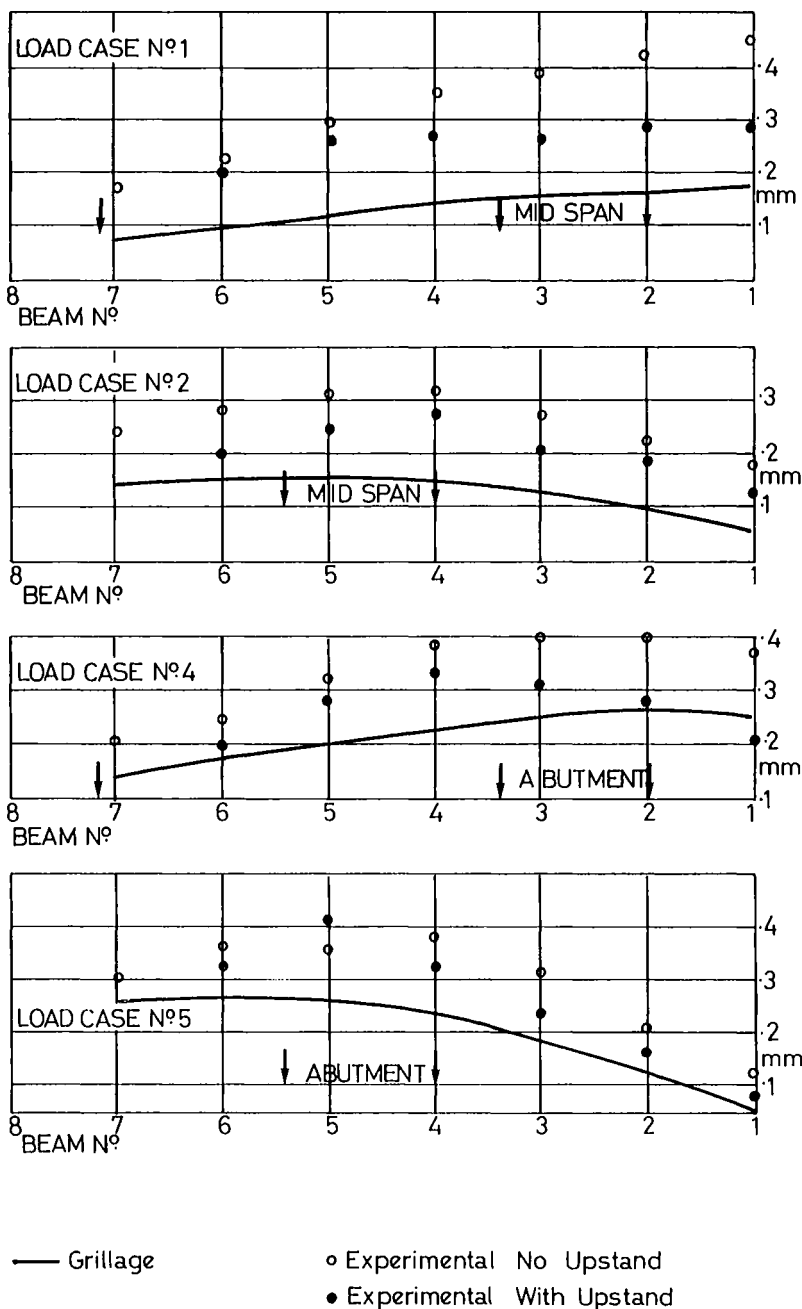


FIG. 6

Structural Performance and Bending Test of Deteriorated Reinforced Concrete Bridges

by Takeshi Oshiro and Sumio Hamada

Synopsis: Recently, significant deterioration of concrete surface and corrosion of reinforcing bars have been observed, and the necessity for the structural evaluation of such bridges becomes urgent for safety considerations.

The primary objective of this study is to investigate the structural performance of two deteriorated reinforced concrete bridges and isolated test beams cut out of the same bridges at the time of replacement. Field and bending tests were conducted from which the strength of existing bridges and isolated beams was observed. Correlations of the values of strength obtained from these two tests give important data to be used for evaluation of existing deteriorated bridges.

From the results of general investigations, field tests and bending tests, the following conclusions are drawn; (1) From the results of field tests of Nakama Bridge, strains of reinforcing bars and deflections are rather close to Theory II (uncracked gross section), whereas those of Yaka Bridge are located between Theories I (cracked section) and II. This indicates the varying effects of deterioration on main beams. (2) Beams in bending tests yield larger strains and deflections compared with those from tests; this shows that the rigidity of a beam that is part of an existing bridge is much higher than that expected from an isolated beam. (3) According to general investigations, these two bridges were highly deteriorated in appearance. However, ultimate strength of test beams is higher than that computed by ACI Code, indicating no significant losses of strength.

Keywords: beams (supports); bend tests; bridges (structures); corrosion; cracking (fracturing); deflection; deterioration; field tests; performance; reinforced concrete; reinforcing steels; safety factor; strains; strength

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INTRODUCTION

Recently, significant deterioration of concrete surface and corrosion of reinforcing bars have been observed(1), and the necessity for the structural evaluation of such bridges becomes urgent for safety considerations. The specific reasons for such deterioration are warm climate in a subtropical zone, effects of salt water due to nearness of the ocean and poor quality of concrete. Because of these unfavorable conditions a number of cracks and spalls of concrete and progressive corrosion of reinforcing bars can be observed, thus causing an urgent need for the evaluation of existing bridges.

The primary objective of this study is to investigate the structural performance of two deteriorated reinforced concrete bridges and isolated test beams cut out of the same bridges at the time of replacement. Field and bending tests(2, 3) were conducted from which the strength of existing bridges and isolated beams was observed. Correlations of the values of strength obtained from these two tests give important data to be used for evaluation of existing deteriorated bridges.

GENERAL INVESTIGATION

Most of the main bridges in Okinawa prefecture were constructed by the U.S.Army Engineer District during the U.S. government control of Okinawa from 1945-1972. In evaluating old bridges, it is very difficult to find bridge information because the design materials such as drawings and calculations were not properly transferred to the Japanese government at the time of Okinawa's reversion to Japan.

Two national highway bridges, Yaka (constructed in 1945) and Nakama (constructed in 1955), were selected for this study because they were being replaced at the time of this research project. Figs. 1 and 2 illustrate dimensions of the test bridges which were determined from field investigations. The general description of these bridges is simple span reinforced concrete T-beams, monolithic construction and they are representative of a large number of bridges presently in a deteriorated condition. The

number of reinforcing bars was assumed at first and checked after the bending tests.

Field observations showed that the development of surface spalls and hollow areas was a result of expansive pressures caused by corrosion of main reinforcing bars. The worst condition was found on down stream surfaces facing the ocean, which indicates the most influential factor seems to be salt water. As shown in Fig. 5, surface of these bridges were in poor condition where progressive deterioration due to cracks and spalls was observed and also, bending cracks were found on beams of Yaka Bridge. The degree of deterioration may be determined from the reduced amount of main reinforcing bars and lack of bond between bars and concrete. These investigations are still limited and the present definition of degree of deterioration is determined from only surface concrete cracks.

MATERIAL TESTS AND RESULTS

Concrete cores and specimens of reinforcing bars were tested to find the physical properties of the materials. Tests of concrete were made using approximately 9.5 cm (3-3/4 in) diameter cores which were cut from the existing concrete deck. Table 1 shows the test results of compressive strength and the modulus of elasticity of concrete, and yield stress and tensile strength of reinforcing bars.

STATIC FIELD TEST

Test Procedure

One or two trucks were placed on each bridge to investigate the behavior of existing bridges and the truck or the trucks were positioned to cause maximum positive moment at the midspan. To investigate lateral distribution of loads three ways of loading were determined, that is, one truck on each lane, on one lane only and at the centerline of the bridge.

Test Results

Truck loads were distributed laterally based on the Guyon-Massonnet(4) and the conventional methods. These methods are commonly used in the design of Japanese reinforced concrete bridges. The Guyon-Massonnet method based on the plate theory may be efficient unless deterioration of the slab is very serious. The moments calculated from these distributed loads are given in Table 2, where design moments on the basis of H20-44 of AASHTO Specifications(5) employed in the original design, and L-20 of Japanese Highway Association (J.H.A) Bridge Specifications(6) are also given.

Computation of theoretical values of strain and deflection was made applying the distributed moments based on the Guyon-Massonnet method. Two elastic theories were employed: Theory 1

assumes uncracked gross section and Theory II assumes cracked section and has been applied conventionally in reinforced concrete beam design. In these computations, elastic modulus of concrete obtained from the material tests were applied.

Figs. 3 and 4 illustrate the relations between computed and measured deflections and strains at the midspan for two loadings, i.e., one truck on one lane and one truck on each lane.

The test results show that the strains and deflections of Yaka Bridge are rather close to Theory I; those of Nakama Bridge to Theory II. The difference in the behavior of the two bridges is caused by the varying degree of deterioration; Yaka Bridge was in much worse condition than Nakama Bridge.

BENDING TEST OF ISOLATED BEAMS

For bending tests up to failure, two main beams were cut out of each bridge at the time of replacement. The test beams of Yaka and Nakama bridges have a rectangular and a T-section, respectively. The dimensions of test beams are illustrated in Figs. 1 and 2. Terms, Y-A and B, N-A and B, are given for the test beams of Yaka and Nakama bridges, respectively.

Test beams were tested using two different benches where 50-ton and 100-ton oil jacks were facilitated. Spans were taken as $L=8.00$ m (26'3") for beams Y-A and B and 10.00 m (32'8") for beams N-A and B. Loading was placed on beams Y-A and B by two points loading of 45 cm (1'6") distance by means of a distribution beam at the midspan. Beams N-A and B were loaded through a rubber plate 2 cm (3/4") thick and 20 cm (7-7/8") wide plated on a concrete deck in a transverse direction at the midspan. Loads were applied by an oil jack in increments of 20 kN to 50 kN up to nearly yield moment and gradually increased up to failure. Fig. 6 illustrates a set of a beam on the test bench. At each loading stage, strains of reinforcing bars and concrete, deflections and crack width were measured by wire strain gauges, dial gauges and microscopes, respectively.

RESULTS OF BENDING TESTS

BENDING MOMENT-STRAIN RELATIONS(M- ϵ RELATIONS)

Moment and strain relations are illustrated for reinforcing bars and deck concrete in Fig. 7, in which two elastic theories, Theories I and II, are noted. Fig. 7 indicates that the strains of reinforcing bars of beams Y-A and B increase linearly up to a moment of 660 kN·m where these values are rather close to Theory I. When a moment exceeds the value mentioned above, a significant increase of strains is observed. These beams, Y-A and B, failed in a flexural mode at moments of 806 kN·m and 802 kN·m, respectively. Compressive concrete strains of beam Y-A are rather close to Theory II in the elastic range, whereas those of beam Y-B are close to Theory I. It should be noted that beam Y-B was in the

poorest condition of the four test beams and that some initial cracks were observed in addition to surface deterioration. These cracks are likely to contribute to the reduction of concrete strain.

Fig. 7 also illustrates the relations between moment and strain of beams N-A and B. The general behavior is similar to beam Y-A mentioned above, where yield of reinforcing bars initiates at a moment of 1100 kN.m.

Fig. 8 illustrates relations between strain ratio and moment where a strain ratio is defined as a value of a measured strain divided by a computed one. The strain ratio of test beams becomes higher than 0.8 for the design moments given in Table 2. The upper limit of strain ratio was reported(7) to be 0.8 for the most deteriorated beams. Therefore, this comparison indicates that the test beams herein belong to a highly deteriorated one.

DEFLECTION

Fig. 9 illustrates load-deflection relations at the midspan. The measured deflections of beams Y-A and B and beams N-A and B increase linearly up to a load of 350 kN and 400 kN, respectively, then increase rapidly, which is a pattern similar to that mentioned in M- ϵ relationships. The test results show that measured deflections of beams Y-A and B are close to Theory I, whereas those of beams N-A and B are located between Theories I and II within the design load and gradually increase to be close to Theory I after passing the loads. The results show that the rigidity of beams N-A and B was not reduced highly in spite of some surface deterioration.

ULTIMATE STRENGTH

As mentioned previously, all test beams failed in a typical flexural failure mode as shown in Figs. 10 and 11. Theoretical and experimental values of yield moment are given in Table 3, where a yield moment based on the elastic theory is defined as the moment at which the strain of reinforcing bars reached the yield strains obtained from the material tests.

ACI strength method(8) of design for reinforced concrete structures is adopted for the theoretical ultimate strength. Experimental ultimate strengths are almost equal to theoretical ones for beams Y-A and B, and 10%, 4% higher for N-A and B, respectively. From these results, beams N-A and B have higher ultimate strength than theoretical ones. When specified stress of concrete and yield stress of reinforcing bars are applied, experimental ultimate strength results in 28%, 27%, 46% and 37% higher than the theoretical ones for beams Y-A, Y-B, N-A, and N-B, respectively. From these comparisons the theoretical ultimate moment based on specified values of the materials indicates quite smaller ultimate strength than actual strength and guarantees the safety of these deteriorated bridges.

To check the safety of these bridges, measured ultimate moments in Table 3 are compared with design moments based on specifications for Highway Bridges of J.H.A.. The results are listed in Table 4. It is concluded that Nakama Bridge has high safety factors compared with Yaka Bridge, and the safety factors of Yaka Bridge are in the lower limit specified in the present specifications. Values of μ in Table 4 are obtained from the tests. If T-sections were used in place of rectangular sections of Yaka Bridge, the values of μ might be 6.1% higher. This results in 1.89 of μ/M_t and 2.07 of $(\mu-1.4M_d)/M_l$.

RELATIONS BETWEEN FIELD AND BENDING TEST RESULTS

From the results of field and bending tests, Figs. 12 and 13 illustrate relations between load and deflection, and between moment and strain. The results show that the strains and deflections of bending tests are close to Theory I up to the load applied in the field, whereas those of field tests are rather close to Theory II. These values of field and bending tests could be equal if all conditions and assumptions were the same. However, the dimensions and the constrained conditions of a main beam, which is a part of a whole bridge, and the test beams cut out of the same bridge were not the same. Therefore the results show these effects from varying conditions.

A beam, being a part of an existing bridge, gives much greater rigidity than an isolated one; some contributions of railings, curbs, pavement and constraints of end beams are considered.

CONCLUSIONS

From the results of general investigations, field tests and bending tests, the following conclusions are drawn; (1) Lateral load distribution obtained from field tests shows that the Guyon-Massonnet method is applicable in evaluation for elastic behavior of deteriorated bridges, unless the slab is seriously deteriorated. (2) From the results of field tests of Nakama Bridge, strains of reinforcing bars and deflections are rather close to Theory II, whereas those of Yaka Bridge are located between Theories I and II. This indicates that varying effects of deterioration on main beams are clear. (3) Beams in bending tests yield larger strains and deflections compared with those from field tests; this shows that the rigidity of a beam that is a part of an existing bridge is much higher than that expected from an isolated beam. (4) According to general investigations, these two bridges were highly deteriorated in appearance. However, ultimate strength of test beams is higher than that computed by ACI Code, indicating no significant losses of strength.

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The opinions, findings and conclusions expressed in this paper are those of the authors and not necessarily those of the Highway Maintenance Section.

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Table 1 Results of Material Tests

			YAKA BRIDGE	NAKAMA BRIDGE
CONCRETE*	COMPRESSIVE STRENGTH	kgf/cm ² (psi)	302 (4295)	388 (5517)
	ELASTIC MODULUS	kgf/cm ² (ksi)	3.13x10 ⁵ (4450)	3.40x10 ⁵ (4834)
REINFOR- ING BAR**	YIELD STRESS	kgf/cm ² (ksi)	2800 (39.8)	3500 (49.8)
	TENSILE STRENGTH	kgf/cm ² (ksi)	5130 (75.4)	4640 (66.0)

* Averages of three cores

** Averages of three test specimens

Table 2 Maximum Moments(kN·m) for Computations

	LOADING (1)*	LOADING (2)**	AASHTO ⁺	J.H.A. ⁺⁺
YAKA BRIDGE	318	185	217	313
NAKAMA BRIDGE	218	181	237	217

* One truck on each lane at the midspan

** One truck on one lane

+ H20-44 Specifications for Highway Bridges, AASHTO

++ L-20 Specifications for Highway Bridges, Japanese Highway Association

Table 3 Comparison of Measured and Computed Ultimate Moments, Yield Moments, Deflections and Strains

		YAKA BRIDGE			NAKAMA BRIDGE		
		THEORY	TEST		THEORY	TEST	
			Y-A	Y-B		N-A	N-B
Mu	kN·m	806 (672)	806	802	1248 (1074)	1395	1300
My	kN·m	700 (600)	708	661	888 (761)	960	1020
Δ^*	mm	22	20	17	19	20	20
ϵ^*	10^{-6}	800	900	960	504	280	400

* Deflections and concrete strains when yielding of reinforcing bars started

() Values computed by the specified yield stresses of reinforcing bars of 2400 kgf/cm² (35.2ksi) for Y beams and of 3000 kgf/cm² (42.7ksi) for N beams and concrete strength of 210 kgf/cm² (2986psi)

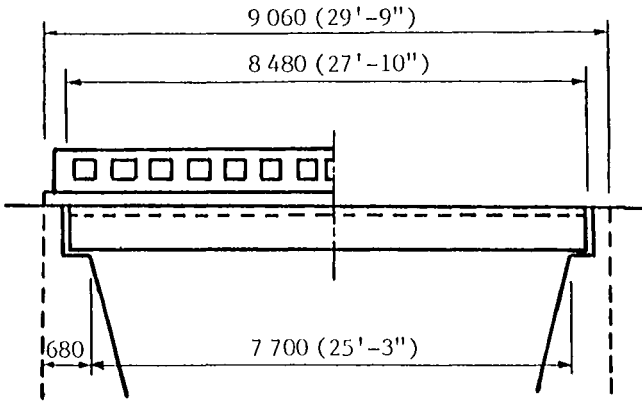
Table 4 Safety Factors Based on Test Results

	Mu	Mt		Mu/Mt	$\frac{Mu-1.4Md}{M1}$
		M1	Md		M1
YAKA BRIDGE	802	313	138	1.78	1.95
NAKAMA BRIDGE	1300	217	194	3.15	4.74

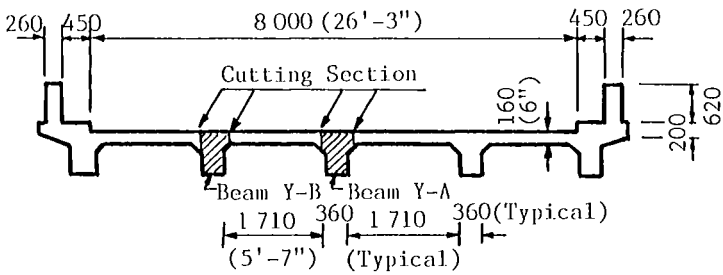
Mu The smaller measured ultimate moment for tested sections listed in Table 3

M1,Md Design moments based on Specifications for Highway Bridges, J.H.A.

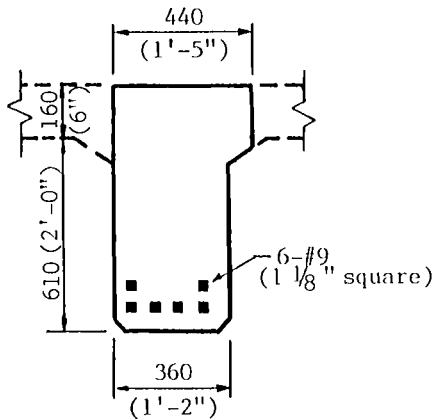
Mt M1 + Md



(a) Elevation

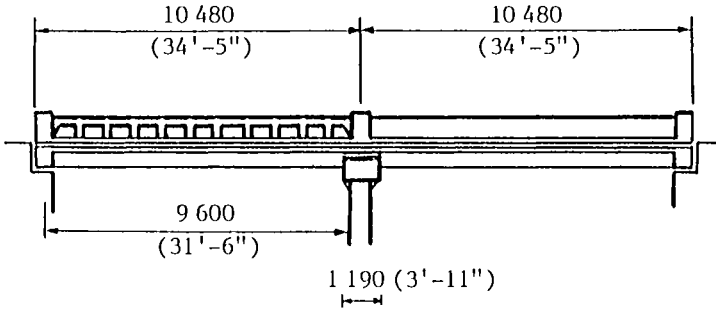


(b) Cross Section

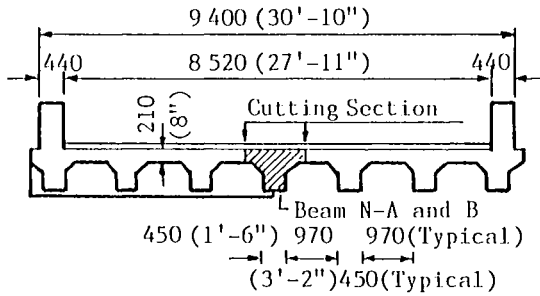


(c) Cross Section of Test Beams Y-A and B

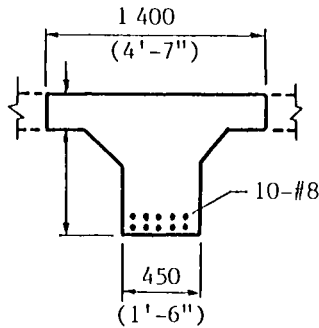
Fig. 1--Dimensions of Yaka bridge



(a) Elevation



(b) Cross Section



(c) Cross Section of Test Beams N-A and B

Fig. 2--Dimensions of Nakama bridge

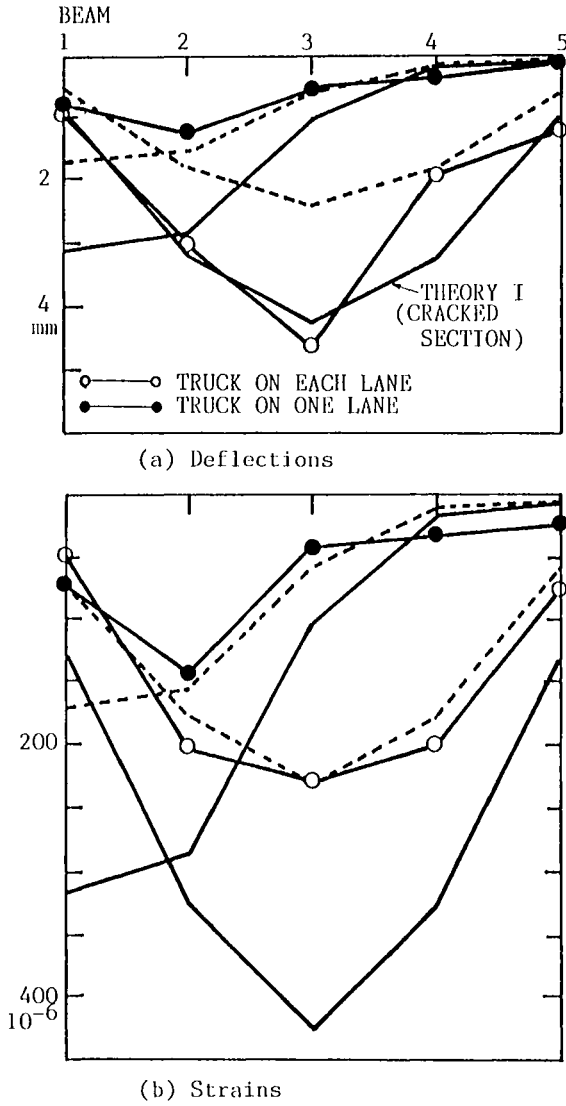


Fig. 3--Deflections and strains of Yaka bridge in field tests