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Fatigue analysis of stringer connection angles in a deck-truss bridge.

Peter B. Keating

FATIGUE ANALYSIS OF STRINGER CONNECTION ANGLES IN A DECK-TRUSS BRIDGE

by

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A Thesis

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Certificate of Approval

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Abstract

A detailed inspection of a riveted deck truss bridge revealed fatigue cracked roadway stringer connection angles. The stringer-floorbeam joint, designed as a simple shear connection, is required to carry a significant amount of bending moment due to the interaction between the truss and the stringers. The stiffness of the floorbeam web at the stringer influences the load distribution in the truss. Also, the stiffness is directly proportional to the amount of bending moment developed in the stringer connection. Three linear finite element models are used to describe this behavior. A repair procedure to release the moment connection back to a simple connection is provided.

1. Introduction

Roadway stringer connections are generally designed as simple shear connections but, depending upon the design of the bridge and the particular detail, may actually transfer moment through the joint. This situation can lead to an overstressing of the connection components which were designed with only shearing forces in mind. A detailed inspection of a riveted truss bridge revealed fatigue cracked stringer connection angles in the deck truss spans. Specifically, the affected angles were those for the roadway stringers located directly above each truss. In addition, these joints were observed to have loose rivets and generally much bleeding of rust indicating they were carrying load for which they were not designed for.

Three linear finite element models are used to analysis the behavior of the truss and the connection between the roadway stringer and the floorbeam web [1]. Load redistribution in the truss span is studied with the addition of the stringers on the truss. Also, a detailed stress distribution is developed for the connection angles from loadings on the span. Finally, a recommendation is made as to the repair procedure required to correct the stringer connection detail.

2. Background

2.1 Bridge

The bridge was originally designed in 1949 and construction was completed in 1952. The structure consists of a 256 meter cantelever truss main span flanked by two pairs of simple deck truss spans, each individual deck truss being 54.9 meters in length. Girder and stringer spans make up the approaches (see Fig. 1). The roadway consists of a 16 meter wide reinforced concrete deck that carries four lanes of traffic.

2.2 Connection Detail

The deck trusses are primarily comprised of built-up box sections connected rigidly at the panel points with riveted gusset plates (see Fig. 2). Each truss is supported at the two lower end panel points by a pinned bearing and a roller bearing respectively. The floorbeams are placed on top of the top chord of the trusses at each joint. A set of eight roadway stringers are connected to the floorbeam webs by means of a simple shear connection which consists of two 4 X 3-1/2 X 7/16 clip angles and a single line of six rivets. The connection to the floorbeam web is contrary to the more recent design of placing the stringers on the top of the floorbeam top flange, resulting in a true simple support. The last 0.25

meters of the top flange of the stringers are coped to allow the top flange of the stringer to be above the floorbeam top flange. This allows clearance for the continuous concrete deck slab.

Directly above each truss the floorbeam web is stiffened by two pairs of 6 X 4 X 1/2 stiffener angles (see Fig. 3). These are to maintain floorbeam web stability since the floorbeams (therefore all liveload and roadway deadload) are supported by the trusses at these two points. The stringer connection is placed in between the stiffeners and this results in a relatively rigid configuration. The other six stringer connections do not have these stiffeners, making for a more flexible joint.

As indicated by the design drawings the concrete deck is not rigidly connected to the top flange of the stringer, thus, does not provide for composite behavior between the two. This was verified in the field by the observance of rust bleeding out from inbetween the stringer top flange and the concrete deck. This indicates that the concrete deck is relatively free to slide along the stringer top without providing any stiffness to the stringer. Also, the date of both design and construction is prior to the use of shear connectors that would result in a composite behavior.

3. Inspection Evaluation

Although the entire bridge was given an in-depth inspection this section will concern itself with the inspection findings of the roadway stringer connection angles and are summarized in Fig. 4.

The inspection of the deck truss floorsystem was conducted primarily from a truck mounted inspection snooper. Access to the stringers by free climbing the truss members was limited due to the low clearance between the stringers and the top chord of the truss (approximately 0.3 meters).

The major finding of the inspection in this area of the bridge is the indication that many of the stringer connection angles are being overstressed. This is occurring at stringer number 2 and 7, the two stringers directly above the trusses. At these locations several of the connection angles have fatigue cracks running down through the fillet at the top of the angle (see Fig. 5). One crack was found to be 127 mm in length. Approximately half of the connections show signs of working such as bleeding rust and halos around the rivets (see Fig. 6). This is occurring along the line of rivets that connect the angle to the web of the floorbeam. Also, the floorbeam web stiffeners show signs that they are working, such as bleeding rust, indicating a resistance to joint rotation. It must be noted that these observations are unique to the stringers above the trusses and were not found at any of the other location.

4. Review of Literature

Fatigue problems with stringer-floorbeam connection angles were first noted by W. M. Wilson during the 1940's [2]. The study dealt primarily with a through-truss railroad bridge and the effect of a passing train had on the stringer connections. He noted that the stringer connections were stressed in addition to that of the designed shearing stress. This was due to the deformation of the truss and by the flexural rotation of the stringer end.

Two types of stresses occur in the stringer connection, direct and indirect. Direct stresses, or load stresses, result from the direct loading of the stringer. These stresses are in the form of shear and, due to the rigidity of the joint, flexural. The indirect stresses, or deformation stresses, result from the overall loading of the structure. Although indirect, their magnitude can be greater than the direct or load stresses. These indirect stresses result from the interaction of the various components of the bridge which were originally designed individually due to the highly redundent nature of the analysis. Usually too little attention was given to the combined action of bridge members.

Under normal passage of a train there will be little direct axial loading of the stringers since the loading is predominantly vertical in nature. But the stringer actually incurs an axial loading from the deformation of the truss. As the bridge is loaded the bottom chord, being in tension, will elongate longitudunally deflecting the floorbeams. Depending upon the degree of flexibility in the floorbeam-truss panel connection, the stringers are translated and stressed, this stress being axial in nature. This axial force must be resisted in the outstanding legs of the connection angles.

Since the elongation of the bottom chord produces an axial force in the stringers, the neutral axis for the stringers of a through-truss can not coincide with the center of gravity of the cross section. The superposition of the axial force with that of the resisting bending moment caused by a load on the stringer shifts the neutral axis away from the mid-depth of the cross section. This results in an increase in the tensile stress towards the top of the connection. This has been documented in many briges where the floorbeam web plate has developed a horizontal crack directly above the stringer connection angle [3]. This condition is aggrevated by the presence of corrosion usually found at this location.

The bridge of this study differs from that of the type of bridge studied by Wilson in that it is a deck truss while the latter was a through truss. The geometry of a deck truss results in different deformation stresses in the stringers, mainly, compressive instead of tensile. This is due to the fact that the floor system is located above the top chord of the truss, as opposed to the bottom chord in a through truss. Although one might presume that the compressive force would help improve the fatigue resistence, its magnitude is not large enough to offset the direct, loading stresses. Its effect is to cause a slight shift in the neutral axis of the stringer connection, a shift upwards. This can only influence the mean stress and not the stress range which would govern in fatigue.

5. Finite Element Analysis

5.1 Introduction

Three different finite element models are used to simulate the behavior of the stringer connection; global, stringer end, and connection angle. An effort was made to make each model as simple as possible yet provide an accurate estimation of stresses and deformations. The results from one model are used as input for the next more detailed model providing a more flexible analysis that allows the study of several different areas of interest.

Loading for the models is accomplished by either inputting concentrated loads or by forcing known displacements. The only loading condition considered in the study is live load since this is basically a fatigue analysis. The major concern is with the structural behavior of the truss and members and not with the actual stress values. Impact is not considered for this reason and because there are speed restrictions imposed by the grade of the approaches and by the confining nature of the roadway.

In all three models the clamping action of the rivets is not considered. It is assumed that all rivets transmit force by bearing only. Although this does not give an accurate stress distribution in the area surrounding the rivets, the distribution away from the rivets in a given plate member is well defined. The rivets in the outstanding leg of the connection angle are defined as single nodal points. The forces from these are used to transmit forces and loadings between the second and third model.

5.2 Description of Models

5.2.1 Model - A

The deck truss span is modeled in only two dimensions since all primary deflections are in the plane of the truss and the area of concern is the stringer connections immediately above the truss (see Fig. 7).

The truss members are modeled with beam elements to allow for rigid joints as is the case with the actual structure. The stringers are modeled using beam elements. Each of the stringers is divided into fourteen sections to allow for an accurate representation of the stresses near the ends and also to provided uniformly spaced loading points. Stringer ends are modeled with cross sectional properties of the combined coped stringer and connection angle legs. The adjacent element is modeled with properties of that of the coped stringer end only. The remaining elements have the properties of a full section W24X76 wide flange beam member. The constraining effect of the dead load of the concrete deck on the top flange of the stringers is not considered. This means that the upward deflection of the stringer nodes is not resisted. But the primary loading and, therefore, deflections are in the downward direction. An effective length of floorbeam was estimated and is modeled as a beam element in the vertical direction. The floorbeam web stiffeners are combined with the floorbeam web into one beam element.

Although all eight stringers contribute to the stiffness of the deck, the major contribution comes from the stringers directly above the truss. Other stringers are less restrained since the floorbeam to which they are connected is not stiffened with web stiffeners nor is the bottom flange of the floorbeam restrained by being connected to the truss. This allows out-of-plane rotation of the floorbeam web and rotation of the stringer joint. Also, these stringers are not influenced by the axial shortening of the truss top chord since the floorbeam is permitted to deflect laterally in its weak direction to compensate for the difference in length between the stringers and top chord.

The global model is used to measure the behavior of the stringers in relationship to the truss. As a load travels across the span the stringers are stressed from the load immediately on the particular stringer and also from the truss deflecting under the load causing indirect stresses in the members. Values obtained in this model, more specifically, stringer bending moments and nodal deflections, are used as input for the next model.

Loading of the model is provided by a unit load placed successively across the span for the development of influence lines for the stringer flanges near the end connections.

5.2.2 Model - B

The second model is a detailed discretization of the coped stringer end, again, modeled in only two dimensions. This model is used to develop an approximation as to the amount of moment the shear connection carries and to provide input for the third model by using the force distribution on the rivets.

The coped stringer model is simulated with the use of two types of elements; plane stress elements for the stringer web and truss elements as the stinger flanges (see Fig. 8). Only the last 1.9 meters of the stringer end is modeled. Loading is accomplished by the placement of concentrated loads at the nodes of the free end or by forcing these nodes through a specified displacement. The reactions to the loadings are then recorded by the use of boundary elements at the nodes that correspond to the six connecting rivets.

5.2.3 Model - C

The third model is that of the connection angle and is modeled in three dimensions. This model is used to study the effects of the moment transmitted to the connection from the deformation of the stringer. Two areas of interest are the stress distribution in the vicinity of the top fillet and the stress distribution around the back rivets that might be causing the observed prying action.

Two element types are used to model the connection angle, plate bending and plane stress (see Fig. 9). The plane stress elements are used on the outstanding leg of the angle, the leg to which the stringer is attached. A two dimensional element can be used here since the displacement of the leg is forced in a planar motion by the restrained condition of the stringer. The angle leg connected to the floorbeam web is modeled with plate bending elements since its primary distortion is out-of-plane resulting from the deformation of the adjacent outstanding leg. The translation of nodes of elements corresponding to the connecting rivets is fixed in all three directions, simulating the riveted connection to the floorbeam web. Loading, forces obtained from the coped stringer model, are input as concentrated loads at the nodes of the outstanding leg which correspond to actual rivet locations.

5.2.4 Floorbeam Web Stiffener Model

Preliminary runs of the computer model A revealed the fact that the results are sensitive to the value given to the stiffness of the stiffened floorbeam web. A simple, three dimensional, finite element model was developed in order to determine the actual stiffness of the floorbeam-stringer connection. Both the floorbeam web and stiffener angles are modeled as plate bending elements. The two angles on either side of the web are combined into one element of double thickness since this does not affect the out-of-plane stiffness of the connection. It is assumed that portions of the web plate that extend beyond the stiffener angles do not contribute to the stiffness and are ignored in the model. Since the ends of the stiffeners are finished to bear on the floorbeam flanges they are not rigidly connection and must be considered as such. Therefore the compression side of the model has the stiffener ends fixed while the tension side is left free to move vertically. A unit load is placed at the top center node in the horizontal direction towards the fixed stiffeners (see Fig. 10).

Two cases were studied, one with the stiffener ends fixed and the other with only one end fixed. The moment of inertia for the completely fixed case can easily be determined and is valued at 7.08-5 m^4. The partially fixed case gives a value of 2.10-5 m^4 when only the stiffener angles on one side are considered. This gives a lower bounds on the stiffness of the connection since the effect of the unfixed stiffener angles are not considered which add stiffness along the length of the floorbeam web. Therefore, in order to arrive at a correct value two cases are considered and their deflections compared. The stiffened floorbeam web can be considered a cantelevered beam with bottom flanges being fixed and being load by the stringer. The deflection of the free end is given by:

$$\Delta = \frac{PL^3}{3EI}$$

Since the moment of inertias are directly proportional the deflections of the two cases can be compared so as to determine the moment of inertia of the partially fixed case. Therefore, the actual stiffness of the connection detail can be given by:

$$I_{partial} = I_{fixed} \times \frac{\Delta_{fixed}}{\Delta_{partial}}$$

This results in a value of 3.87-5 m^4, which is used in Model - A and for subsequent models.

5.3 Finite Element Results

5.3.1 Model - A

The consideration of stringers in the global analysis of the truss reveals the fact that both the stringers and truss members are interdependent with regard to their structural behavior. The rigid placement of stringers above the truss causes a redistribution of forces in the truss members and at the same time the flexible supports provided by the truss for the stringers results in complicated stressing of the stringers and floorbeam web area.

A comparitive analysis was made between the truss with stringers and without stringers using a floorbeam web moment of inertia of 3.87-5 m^4. By loading the truss with an 356 kN load, placed at the upper middle joint (U3), member forces were obtained for both cases (see Fig. 11 and Fig. 12). With stringers considered, force redistribution occurred primarily in the chord members. The greatest change is in the two middle panels (U2-U3 and U3-U2') where the bottom chord force decreased 3 percent from 534 kN to 518 kN tension. The top chord force decreased 31 percent from 356 kN to 246 kN compression. The additional resistance was provided by a 101 kN compressive force in the stringers. Adding the top chord and stringer forces together yields 347 kN as compared to the original 356 kN force without the stringers. The moment is balanced by considering the moment arm provided by the additional height of the stringers above the top chord. In another area of the truss the false top chord (member U0-U1), used to provided joint stability and normally unstressed, developed a 27 kN tensile force. Evidence of this member carrying load was found during the inspection in the form of a working connection bracket (see Fig 13). The increase in the stiffness of the truss with the stringers is evidenced by a 8 percent decrease in the center span deflection, from -9.17 mm to -8.46 mm.

As stated earlier, truss member and stringer forces are sensitive to the value given to the stiffness of the floorbeam web. An analysis was made by varying the stiffness and comparing resulting member forces and truss deflection. Four different cases of the global model (Model - A) were made, each with a different value for the out-of-plane moment of inertia of the stiffened floorbeam web. The first cases has a value of 3.87-5 m^4, obtained as mentioned previously, by running a separate finite element model of the web detail. For the second case a value of 7.08-5 m^4 is used, simulating the condition where all stiffener ends are fixed, the most rigid condition (see Fig. 14). In the third case a value of 2.10-5 m^4 is used, assuming only one pair of stiffener angles on one side of the web plate contributes to the stiffness of the detail (see Fig. 15). The fourth case has a value that approaches zero, similar to that provided by an unstiffened floorbeam web (see Fig. 16).

The finite element results show the influence the web stiffeners have on the axial force in the stringers. As the stiffness increases the axial load in the stringers increase. For the midspan panels the increase is 140 percent between the two extreme cases, 48 kN to 115 kN compression. For the end panels the increase is 62 percent, 16 kN to 27 kN compression. The axial force in the top chord changes accordingly. Neglectible changes occur in the other truss members.

The above analysis was made by only loading the truss, thus, any stressing of the stringers was due to the deflection of the truss. When a load is placed on a particular stringer it is stressed by both the bending moment caused by the loading and also by the deflection of the truss due to the load being transferred to the adjacent truss joints. Influence lines were constructed for the bending moment resisted by the stringer in the vicinity of the floorbeam connection at joint U1 (see Fig. 17 and Fig. 18). From these it is observed that one stringer is affected by the loading of the other stringers, especially the adjacent stringer framing into the same floorbeam connection. And, more importantly, it is the top flange of stringers near the floorbeam that is in tension bearing evidence to the fact that the stringer-floorbeam joint is rigid and carries moment. Reversal in the flange stresses occurs only when the load is two or more panels from the point in question. The tensile force in the top flange is consistent with the fatigue cracks found in the top of the connection angle fillet. It must be remembered that these top flange tensile stresses have been reduced by the compressive action imposed on the stringers by the deflection of the truss as noted earlier.

5.3.2 Model - B

The coped stringer model is used to determine the percentage of moment carried by the shear connection. By using the full length stringer as a simple beam with fixed ends and displacing one end vertically 25.4 mm, an end moment of 318 kN-m is computed. The deflection and rotation are computed at the section 1.9 meters from the floorbeam connection corresponding to the free end of Model - B. The model free end is then forced into this displaced shape by means of boundary elements. By summing the moments from each rivet node reactive force about the centroid of the rivet group a resisting moment of 87.1 kN-m is developed in the connection. This amounts to 27 percent of the applied moment being transferred through the connection (see Fig 19) and is the amount by which the moment capacity of the joint is reduced to by the coped section.

Deflections and rotations from Model - A were used as input for the coped stringer model which are the result of a 44.5 kN load. The section chosen for analysis was panel 0-1, at joint U1. This forced displacement results in a moment of 37.6 kN-m developing in the connection. The corresponding stress distribution is shown in Fig. 20. It is this bending moment which must be carried by the connection angles, a force for which it was not originally designed for. The nodal forces (rivet reactions) are used to load the connection angle, Model - C.

5.3.3 Model - C

With the nodal foces obtained from the previous model used as input into the connection angle model at rivet nodes the stress distribution along the fillet was developed (see Fig. 21). This analysis gives an approximately linearly varying stress distribution through the length of the fillet. This is indicative of a moment carrying cross section. At the top end of the fillet there is an apparent stress concentration which raises the stress level by 75%, or to 108 MPa tension. This stress level is high enough to be the cause of the observed fatigue problem.

The resulting boundary element forces indicate that there is a significant force on the top rivet closest to the top fillet (see Fig. 22). The force on the nodes along the vertical line adjacent to the angle fillet are tension, while along the line passing through the center of the rivets they are compression. The exterior line of nodes has a marked decrease in the magnitude of force, by an average of 80 percent. The force distribution at a rivet has both tension and compression values simulating prying action on the rivets.

6. Examination of Results

6.1 Load distribution

As indicated by the original design stress sheets the deck truss members were analyzed and designed as being pin ended and no consideration was given to the stringers. But the aforementioned finite element analysis, using Model - A, reveals the fact that the roadway stringers behave as a part of the truss. There is a significant redistribution of load from the top chord of the truss to the stringers above. It is to the degree that the top chord in the midspan panels has a smaller load than the adjacent panels, normally they would be equal. Although this decrease the stress levels in the top chord, thereby increasing its capacity in carrying bridge loads, the increase in the stringer axial stress should be accounted for in the design. This increase amounts to approximately 6.9 MPa at the center panels. Though the increase is slight at the full sectin of the stringer it is magnified at the connection, both at the reduced section and in the connection components.

One of the most noticable effects of the load redistribution to the stringers is at the end panel points of the truss (U0 and U0'). The false top chord (member

U0-U1) under simple truss analysis carries no load and is included in the truss geometry to provide stability at joint U0 and also for aesthetic reasons. Therefore, for this bridge no consideration was given in the original design for the tensile force actually carried by the member. This was observed during the inspection in the form of overstressed connection bolts and the generally poor condition of the joint (see Fig. 13).

6.2 Rigidity of Floorbeam Web

The amount of load that is carried by the stringers is influenced by the stiffness of the floorbeam-stringer connection, the load being a combination of axial and bending. The axial load is compressive and results in an axial shortening of the stringers. This shortening could possibly have a detrimental effect depending upon the ratio of the stringer depth to floorbeam depth. As this ratio approaches the value of one the distance between the bottom of the floorbeam-stringer connection and the bottom flange of the floorbeam decrease. This would make the web gap in the area susceptable to out-of-plane induced displacement and cracking. This being dependent on the relative difference in the axial shortening of the top chord and the stringers. As the stiffness of the connection decreases the amount of axial load carried by the stringers decrease. This results in greater discrepancy in the axial shortening and, therefore, a greater tendency to push the floorbeam web out of plane as the top chord shortens a greater amount. The amount of displacement would be zero at the midspan panel point and would increase towards either end of the truss as this difference accumulates.

The stiffness of the floorbeam web at the stringer connection affects the amount of bending moment carried by the stringer. From the finite element analysis, using both Model - A and Model - B, it has been shown that as the floorbeam web stiffness increases the bending moment at the stringer end increases (see Fig. 23). If the floorbeam is unstiffened, without web stiffener angles, the moment is 30.2 kN-m for a unit loading of the truss. Assuming the computed stiffness of the detail, the value increases to 33.6 kN-m, an increase of 11 percent. Assuming an upperbound on the stiffness results in a bending moment of 35.1 kN-m or an increase of 17 percent. It is therefore evident that the floorbeam stiffness contributes to the elevated stress level in the floorbeam-stringer connection.

6.3 Coped Stringer End - Moment Capacity

The results from Model - B show that even though the joint was designed as a shear connection it is in fact transferring a bending moment to the connection angles. Although the geometry of the detail, such as coped flanges, will influence the degree of moment capacity, it is influenced more by the fact that the stringer end is restrained from rotation in the plane of the stringer. The restraint is provided by the line of six rivets at the connection and the connection angles. Free rotation of the stringer with no bending stresses could only be accomplished by either pin-connecting the stringer end or by placing the stringer end on a seat and only attaching the bottom flange.

The degree to which the moment capacity of the stringer connection is detrimental is dependent on the joint detail. This is supported by the observation that the stringer connections that are not located above the trusses showed no sign of fatigue damage. These stringers have identical connection details, such as, the coped top flange and the two connection angles with six rivets through the stringer web. The difference at these locations is that the floorbeam web is not stiffened and that the truss top chord is not immediately below. So, even though the stringer connection is capable of carrying moment, the joint detail at these locations allow for joint rotation and the release of the bending stress.

6.4 Connection Angle

The combined action of the loading and deformation stresses in the stringers results in a flexure of the legs of the connection angles in the direction of the stringer. The connection angles are not flexible enough to compensate for the elastic rotation and change in length of the stringers. This leads to the overstressing of the connection, resulting in the prying action around the outmost rivets and the cracking of the angle fillet. The prying action results from the counteracting forces on adjacent vertical lines near the fillet and along the rivet [4]. The degree of prying action is dependent on the flexibility of the angle legs. As the thickness of the leg is increased the prying action would be reduced since there would be a reduction in the bending deformation around the rivets. Though, the increase in the angle thickness would increas the moment capcatity and stiffness of the joint, thereby increasing the stress in the angle fillet.

7. Recommendations

The main concern with this study is the analysis of the fatigue cracking of the connection angle and a satisfactory solution for repair. The analysis has shown that the stiffened shear connection tramsmitted a significant amount of moment through the joint. Since the joint is stiffened it is unable to rotate and thus produces a bending stress distribution. The stress must be resisted by the connection angle, a function which it was not originally designed for. The geometry and configuration of the connection are such that increasing the member size to reduce stresses is both impractical and undesirable. The confinement of the detail would not allow for a heavier sectioned angle. In addition, a larger angle would only result in stiffening the joint even more, thus causing the joint to carry an increasing amount of moment through the joint.

The ideal repair would be to redesign the connection so that it is not required nor able to transmit moment. It is therefore recommended that a beam seat be placed below each stringer end. With this the original shear connection could be released or eventually allowed to fail. Full support would be provided from the bearing of the stringer bottom flange on the beam seat. This would result in a true simple connection, practically all moment would be released at the end of the stringer. Web stability of the stringer must now be considerd since the bearing force is resisted by the web. This is helped if the connection angles are left in place without the rivets through the stringer web so that they prevent the lateral motion or buckling of the stringer web. Stringer web stiffeners might be necessary. The bottom flanges of the stringers would be slot bolted through to the beam seat.

The beam seat repair could be performed without the closure of the bridge to traffic since the stringer could first be supported by the beam seat before the shear connection is released. The change out of the connection angles would require that the stringers be unloaded during the repair, thereby necessitating the closing of the bridge to all traffic. Also, the stringer would still have to be supported, though temporarily, during the repair. All of which would require a considerable more amount of time and effort than the beam seat.

With the release of the end moment consideration must be given to secondary effects. Since the end would be simple the deflections of the stringers would increase, increasing the transverse bending in the reinforced concrete slab above, though this would be slight since these deflections would be consistent with those of the original design. Also, since the original connection increased the stiffness of the truss, the flexibility would now be increased with the beam seat type of connection. Though the new detail requires consideration of new areas, the original detail as it stands now is inadequate. Failure of the connection by the shear failure of the connection angle would lead to a large unsupported length of the concrete slab and possible failure of the deck.

8. Summary and Conclusions

There is considerable work that can still be done on this study such as fine tuning the model to accurately define the boundary conditions. This would result in a more accurate estimation of stress values. But, as stated earlier, the main objective is the analysis of the structural behavior of a simply supported truss with stringers connected to the floorbeam above the upper joint of the truss. It has been shown that both the roadway stringers and the truss influence each other's behavior, especially for the stringers directly above a truss.

Normally, the originally designed shear connection is adequate for a roadway stringer, even if the joint is capable of transmitting moment. This is evidenced by the fact that 12 out of 16 connections at a given floorbeam are functioning properly. This is because the joint's ability to rotate has not been prevented. But with the placement of floorbeam web stiffeners, as is the case above each truss, the flexibility of the joint is decreased and the connection undertakes a significant amount of moment and consequently damaging stress levels. As the floorbeam web stiffness increases both the amount of bending moment and axial force in the stringers increase. The use of a three dimensional mathematical model of the detail allows for an accurate estimation of the forces and a proper evaluation of its behavior with interdependent members.

FIGURES

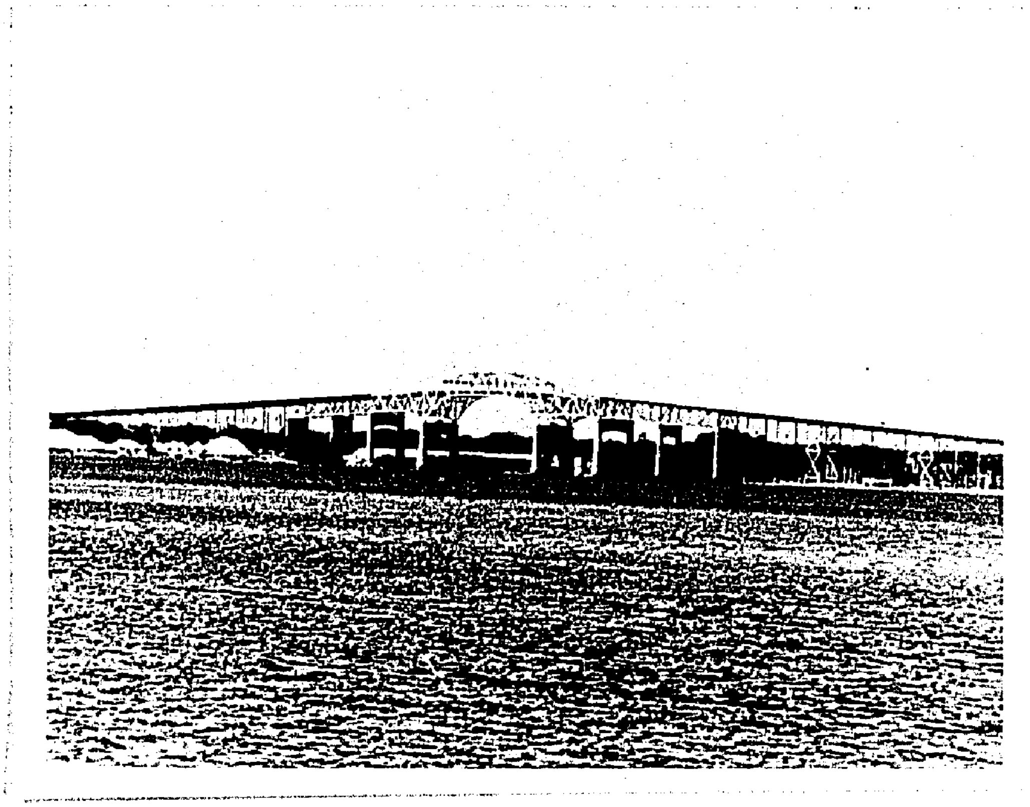

Fig. 1 View of Bridge

MICRODEX CORRECTION GUIDE (M-9)

CORRECTION

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FIGURES

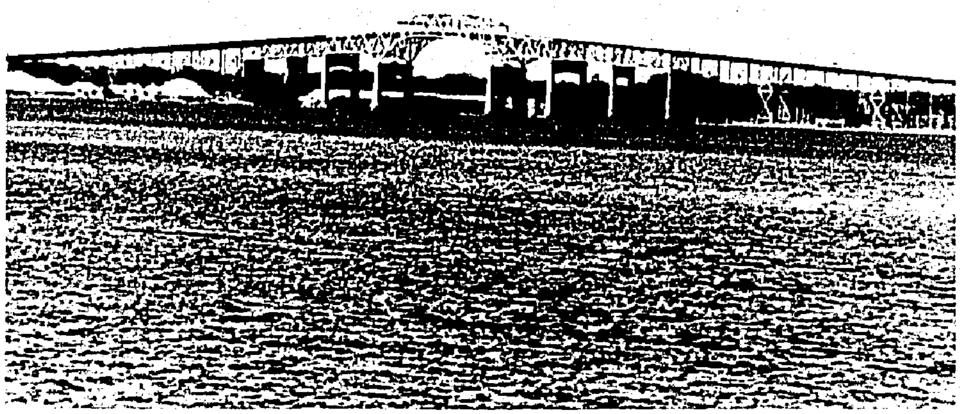


Fig. 1 View of Bridge

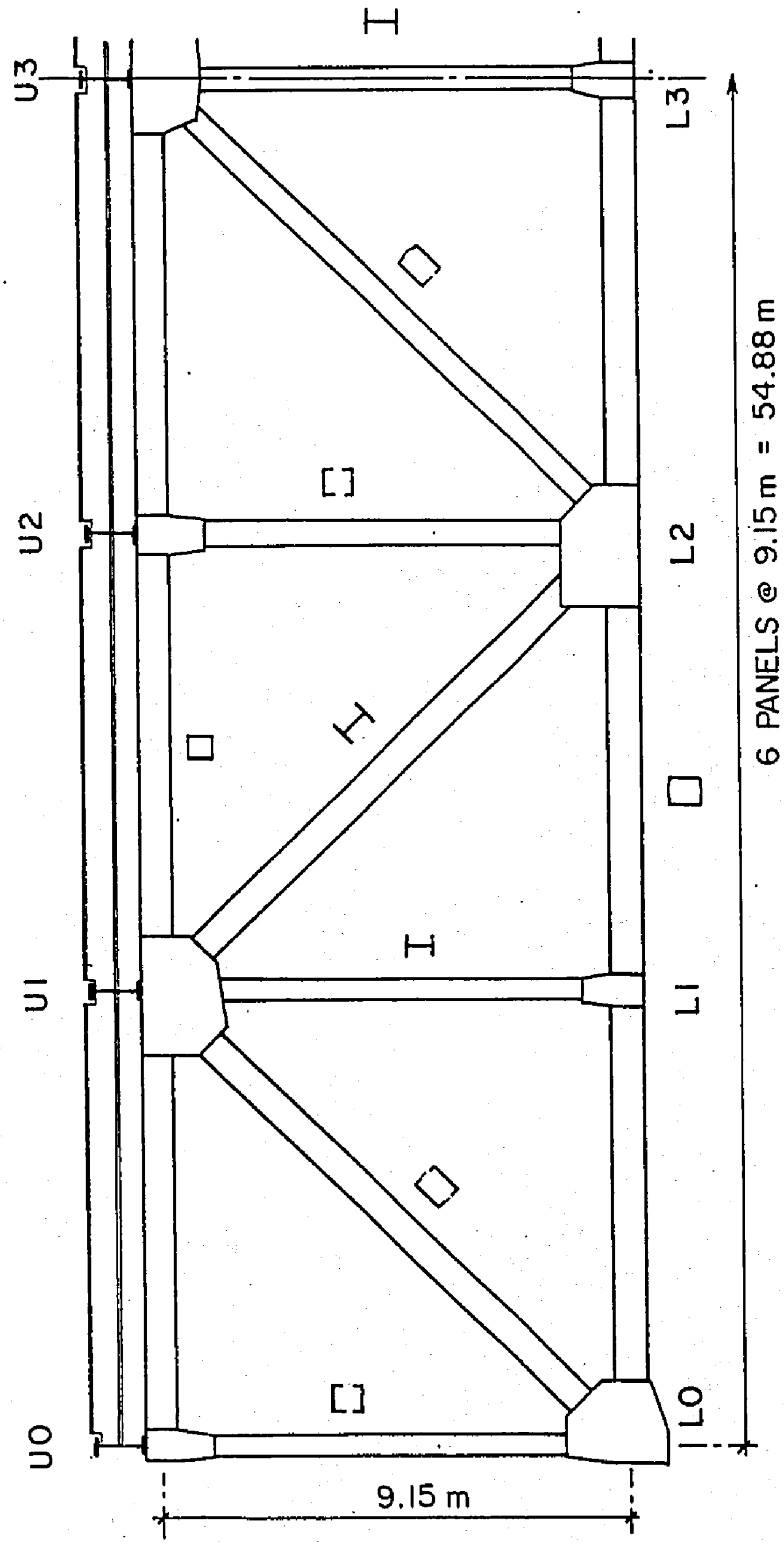


Fig. 2 Deck Truss Plan - General Plan

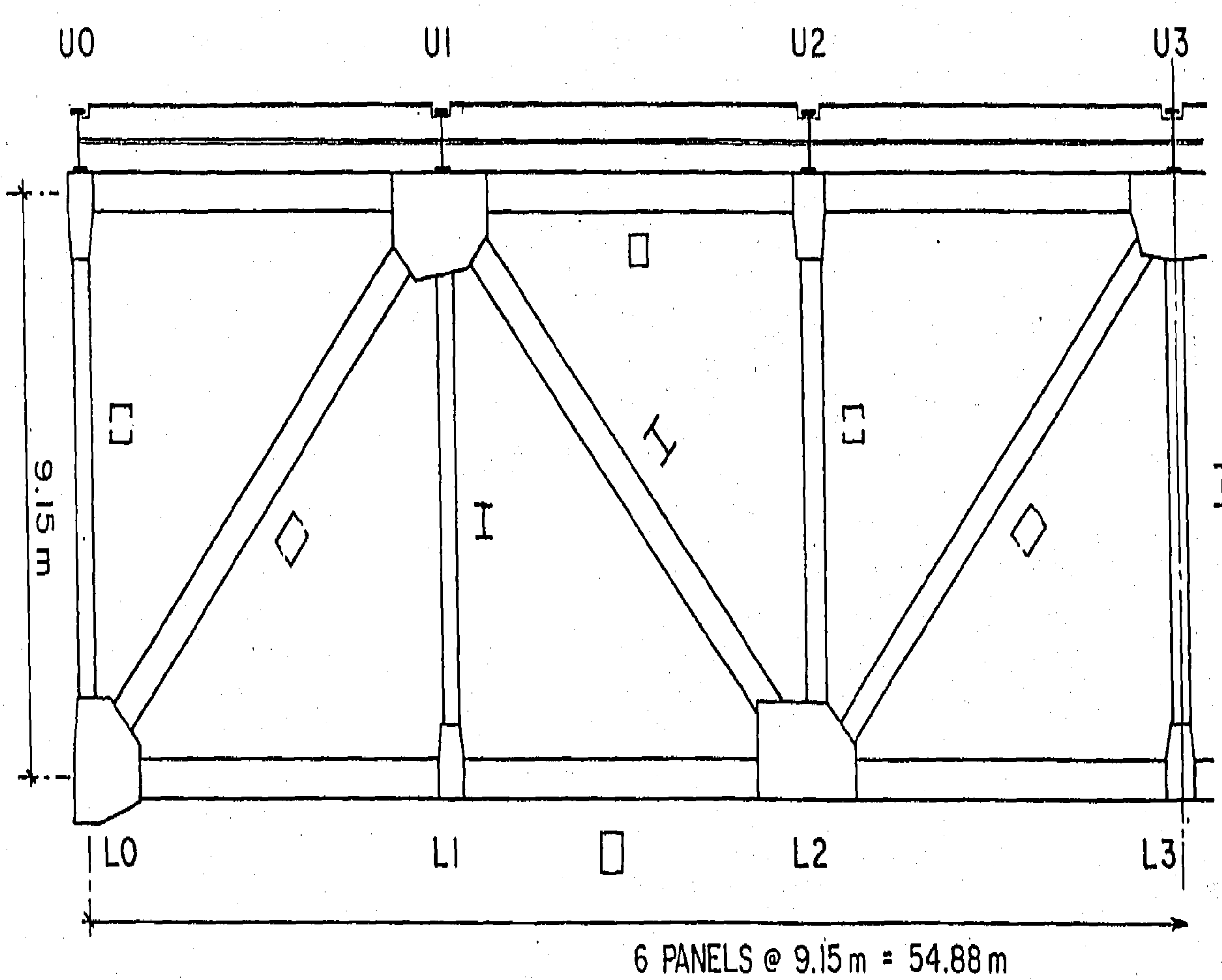


Fig. 2 Deck Truss Plan - General Plan

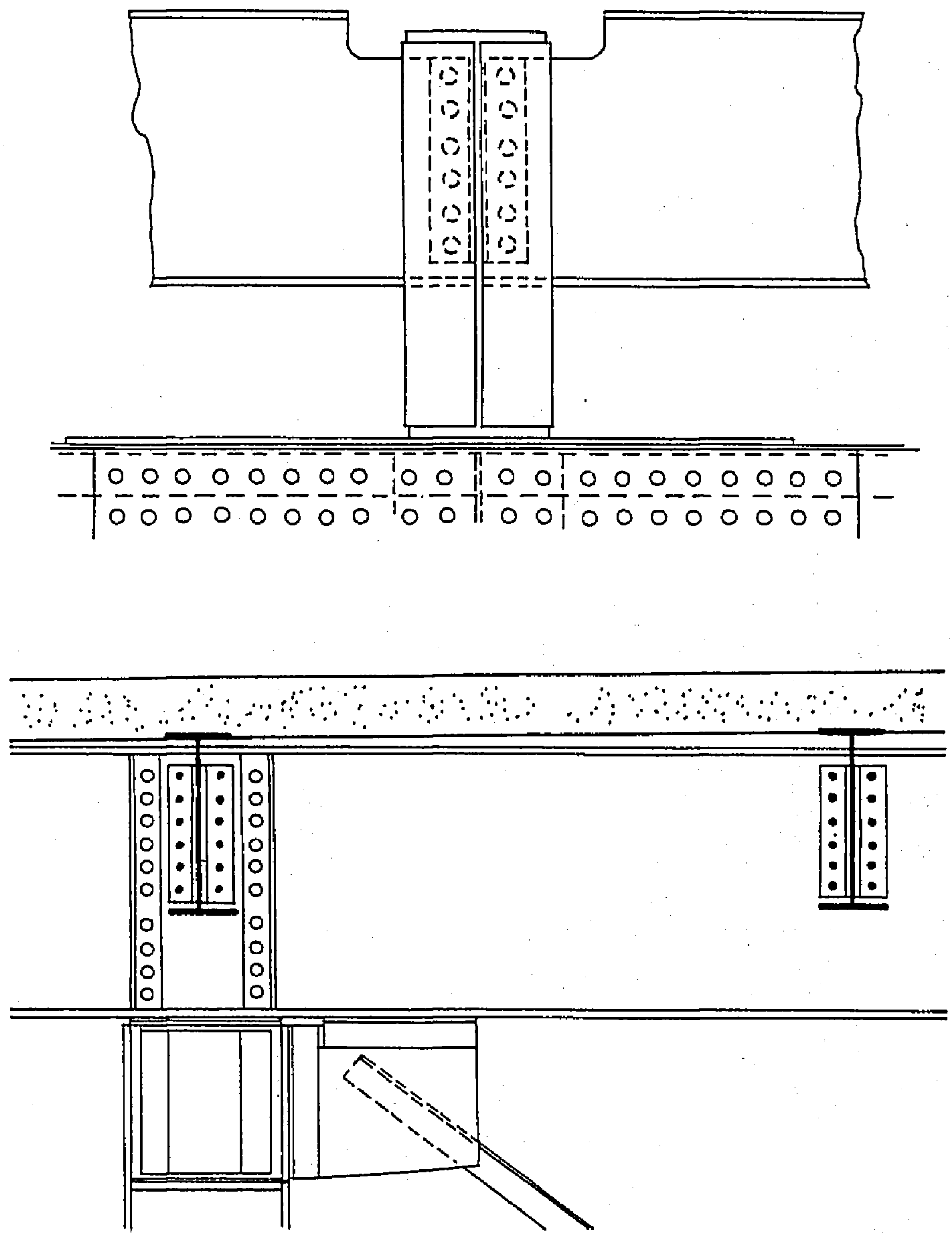


Fig. 3 Stringer - Floorbeam Details

SPAN	PANEL	JOINT	REMARK
I-II	1-2	1 (7)	Cracked fillet 70 mm
		2 (7)	Top rivet loose
			Halo around rivet
	2-3	2 (2)	Working connection angle
		2 (7)	Top and bottom rivets working
	2'-1'	2'(2)	Stiffener end working
		2'(7)	Halo around bottom rivet
	1'-0'	1'(2)	Stiffener end working
II-III	0-1	0 (2)	Cracks in both fillets 76 mm and 127 mm
			Stiffener end working
		1 (2)	Stiffener end working
	1-2	1 (7)	Cracked fillet 38 mm
	3-2'	2'(7)	Halo around bottom rivets
VI-VII	1-2	1(7)	Cracked fillet 44 mm
		2 (7)	Halo around top rivet
	3-2'	2'(7)	Halo around top rivet
	2'-1'	1'(2)	Stiffener end working
	1'-0'	1'(2)	Stiffener end working
		0'(2)	Stiffener end working
			Halo around rivets
		0'(7)	Cracks in both fillets 57 mm and 89 mm
VIIVIII	1-2	1 (2)	Stiffener end working
	2-3	2 (7)	Halos around all rivets
		3 (2)	Halo around bottom rivet
	2'-1'	1'(7)	Stiffener end working

Fig. 4 Summary of Inspection Findings

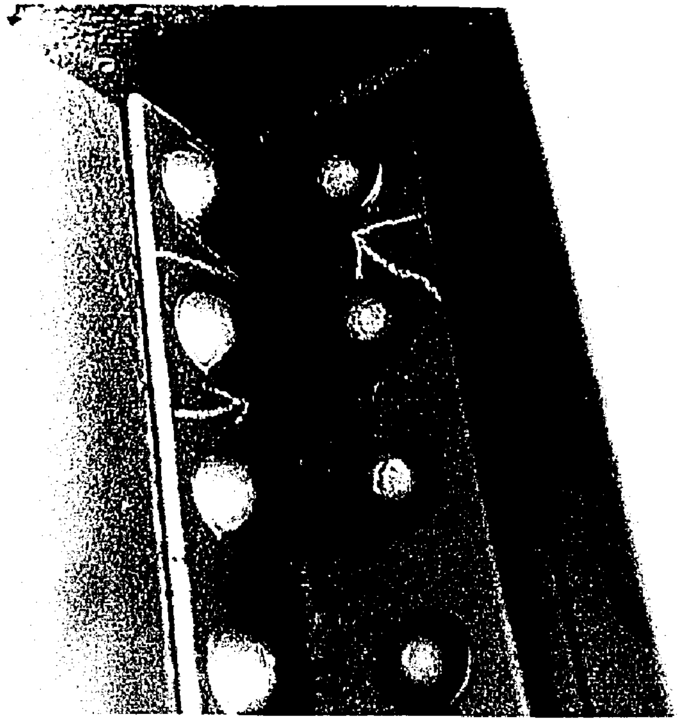


Fig. 5 View of cracked Connection Angle

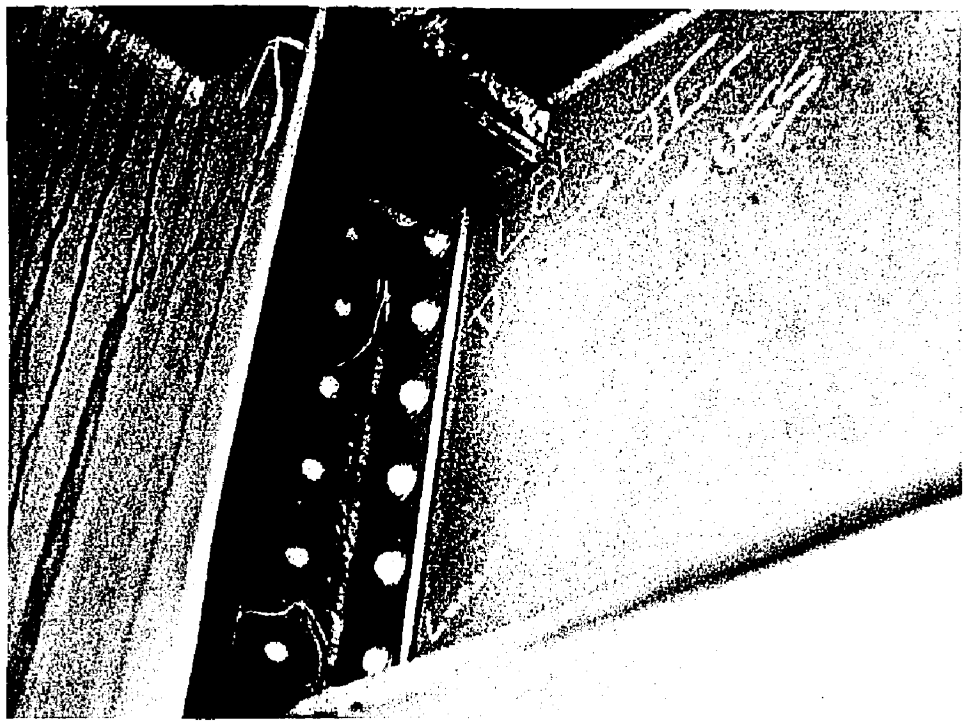


Fig. 6 View of Connection Angle with Working Rivets

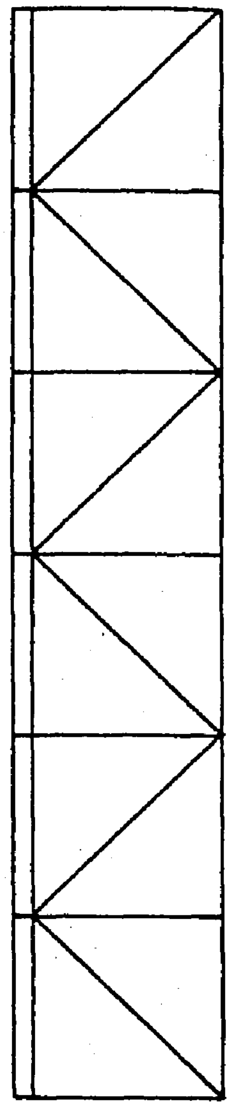


Fig. 7 Model - A Deck truss

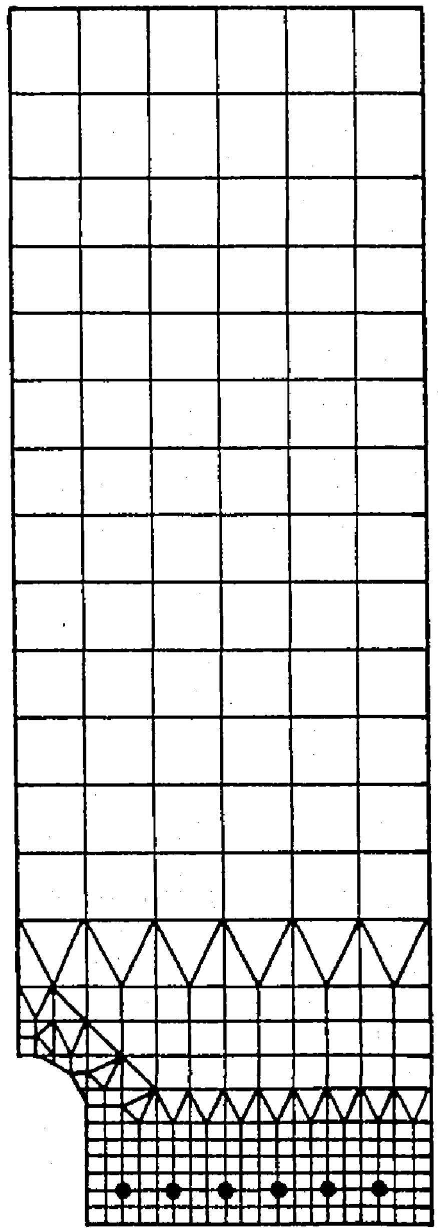


Fig. 8 Model - B Coped Stringer End

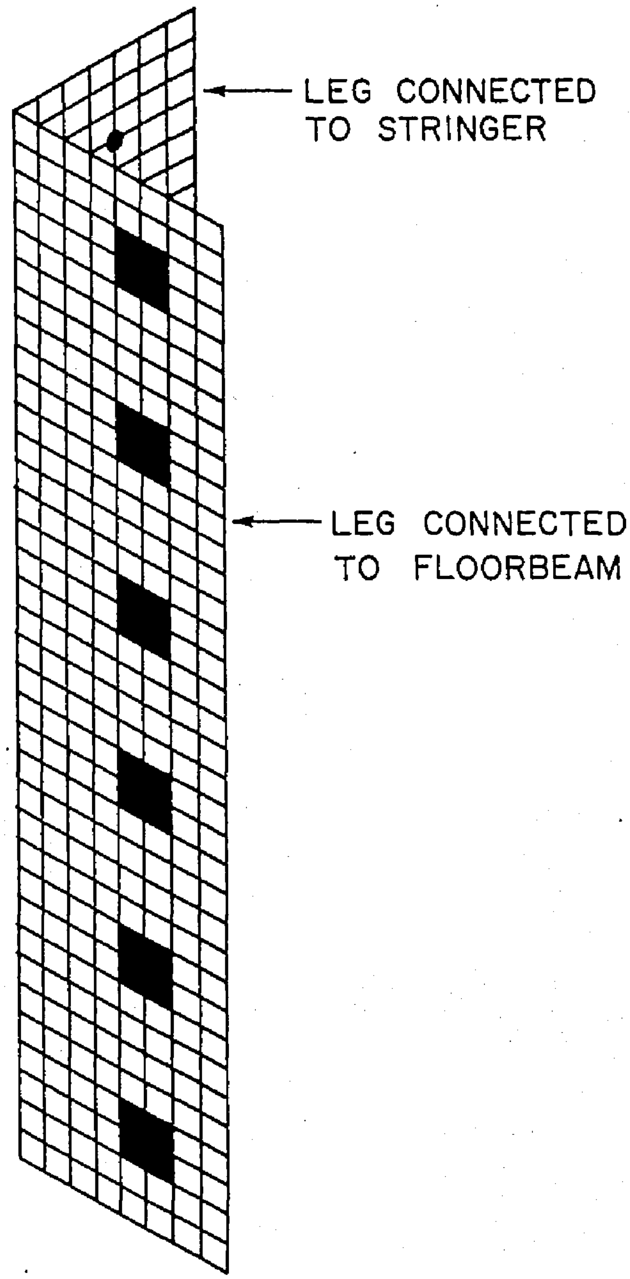


Fig. 9 Model - C Connection Angle

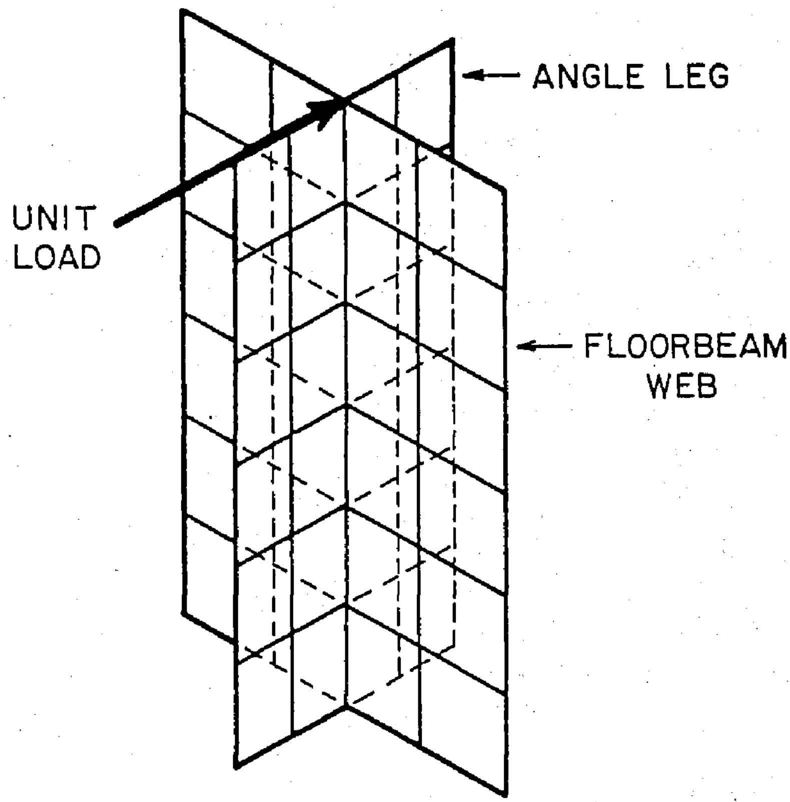


Fig. 10 Floorbeam Web Stiffener Model

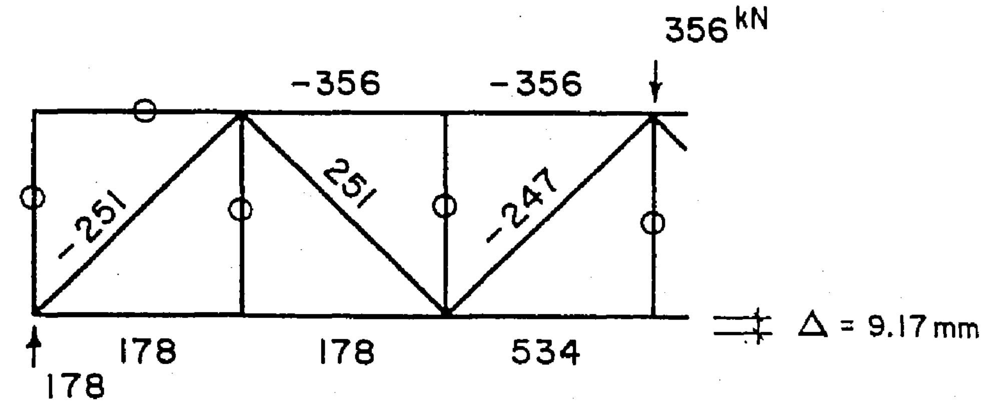


Fig. 11 Global Analysis - without stringers

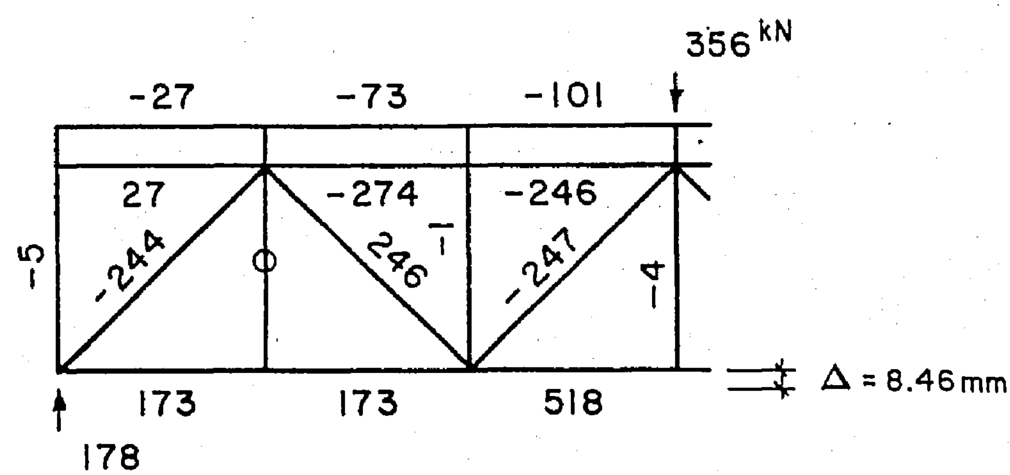


Fig. 12 Global Analysis - with stringers

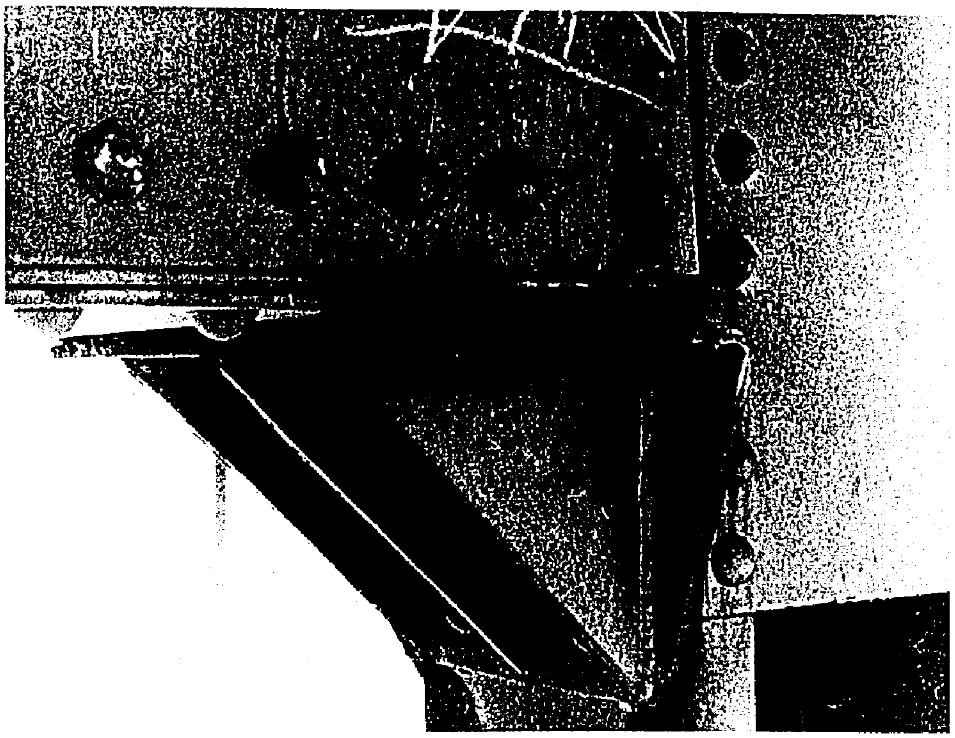


Fig. 13 View of Beam Bracket at End Upper Joint

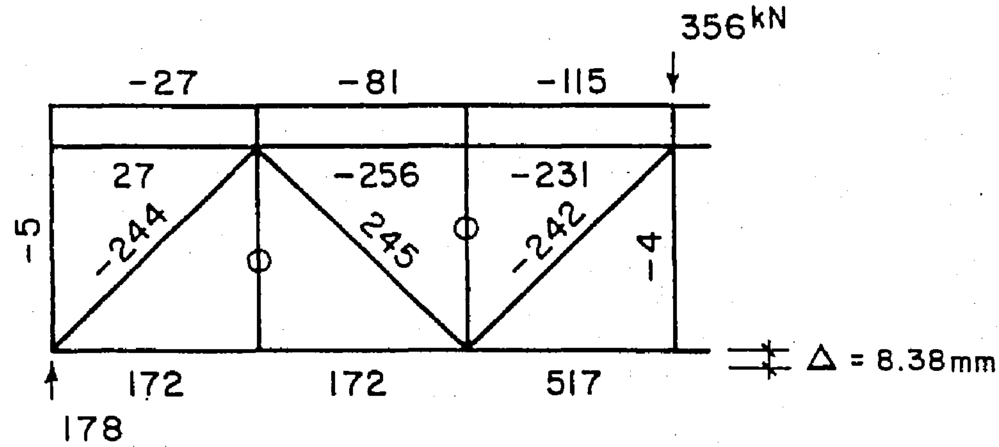


Fig. 14 Global Analysis - I=7.08-5m^4

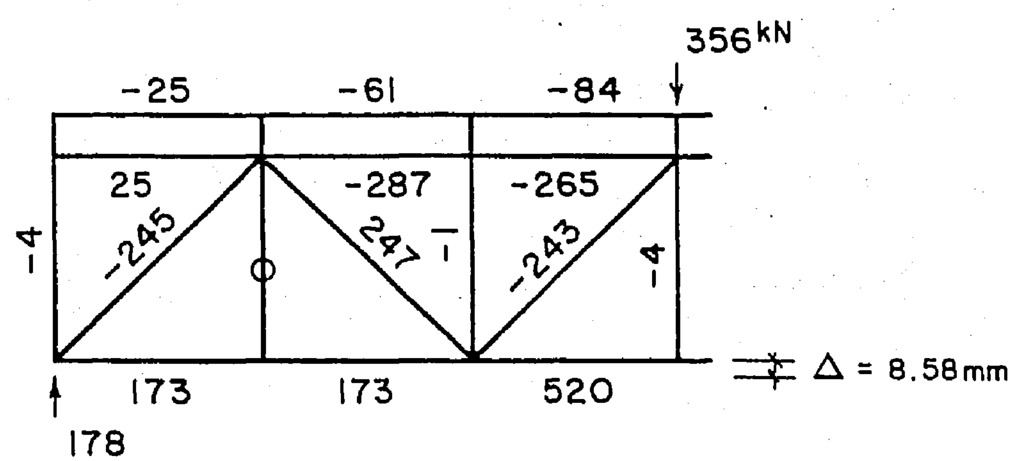


Fig. 15 Global Analysis - I=2.10-5m^4

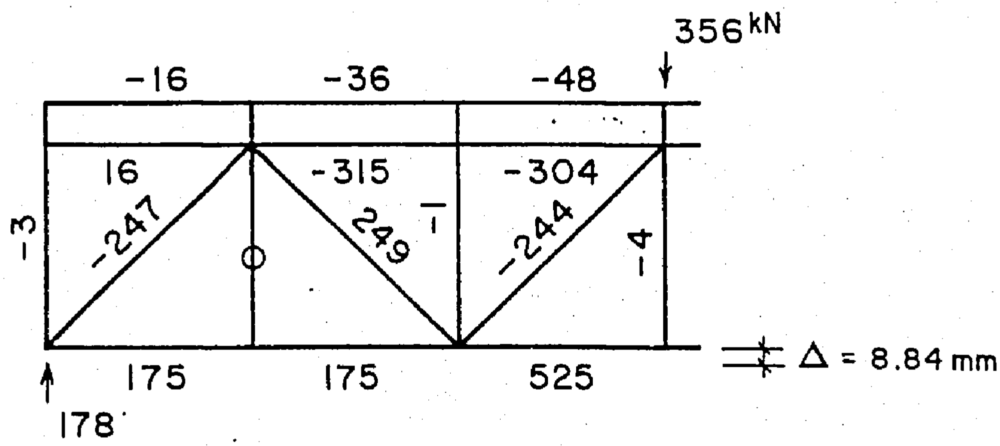


Fig. 16 Global Analysis - $I=4.16\text{-}7 m^4$

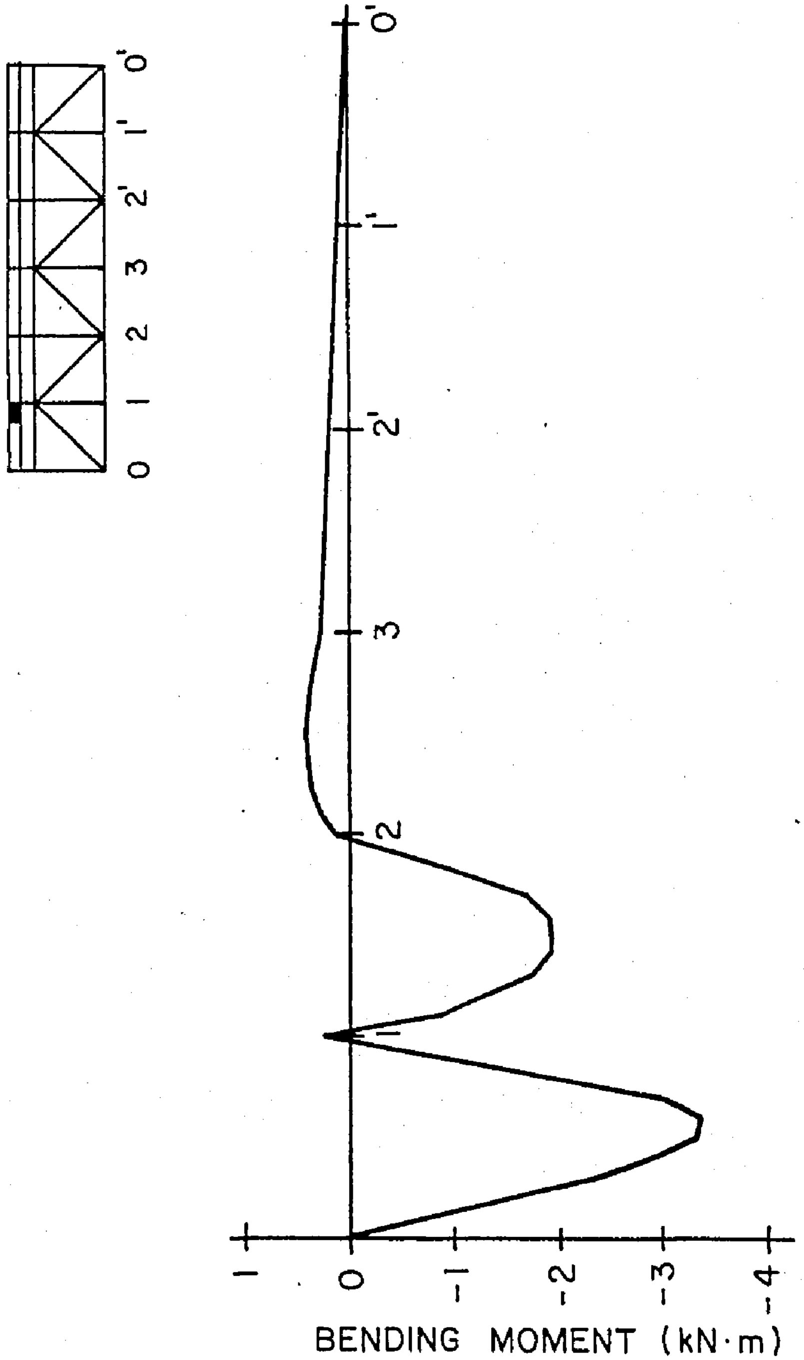


Fig. 17 Influence Line - U0U1 @U1

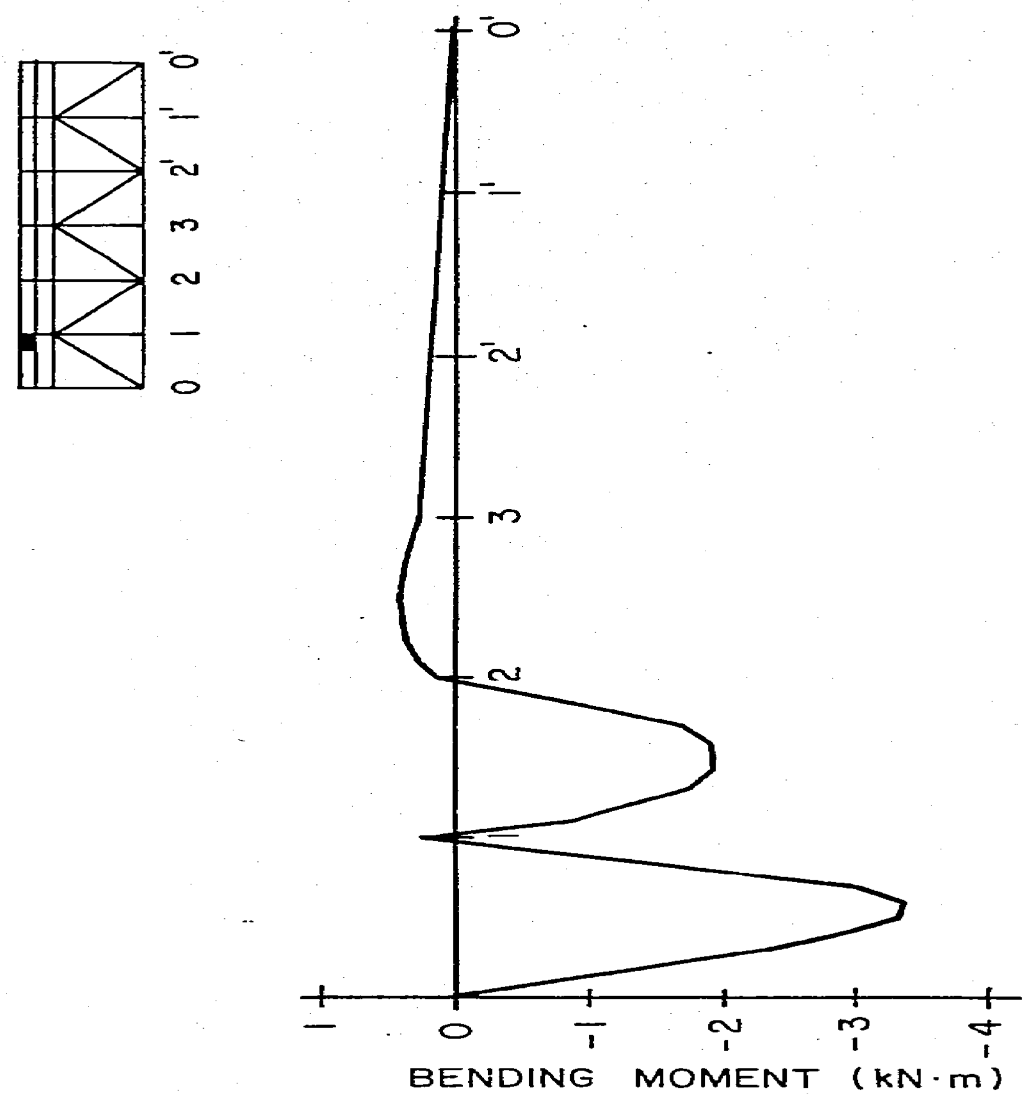


Fig. 17 Influence Line – U0U1 @U1

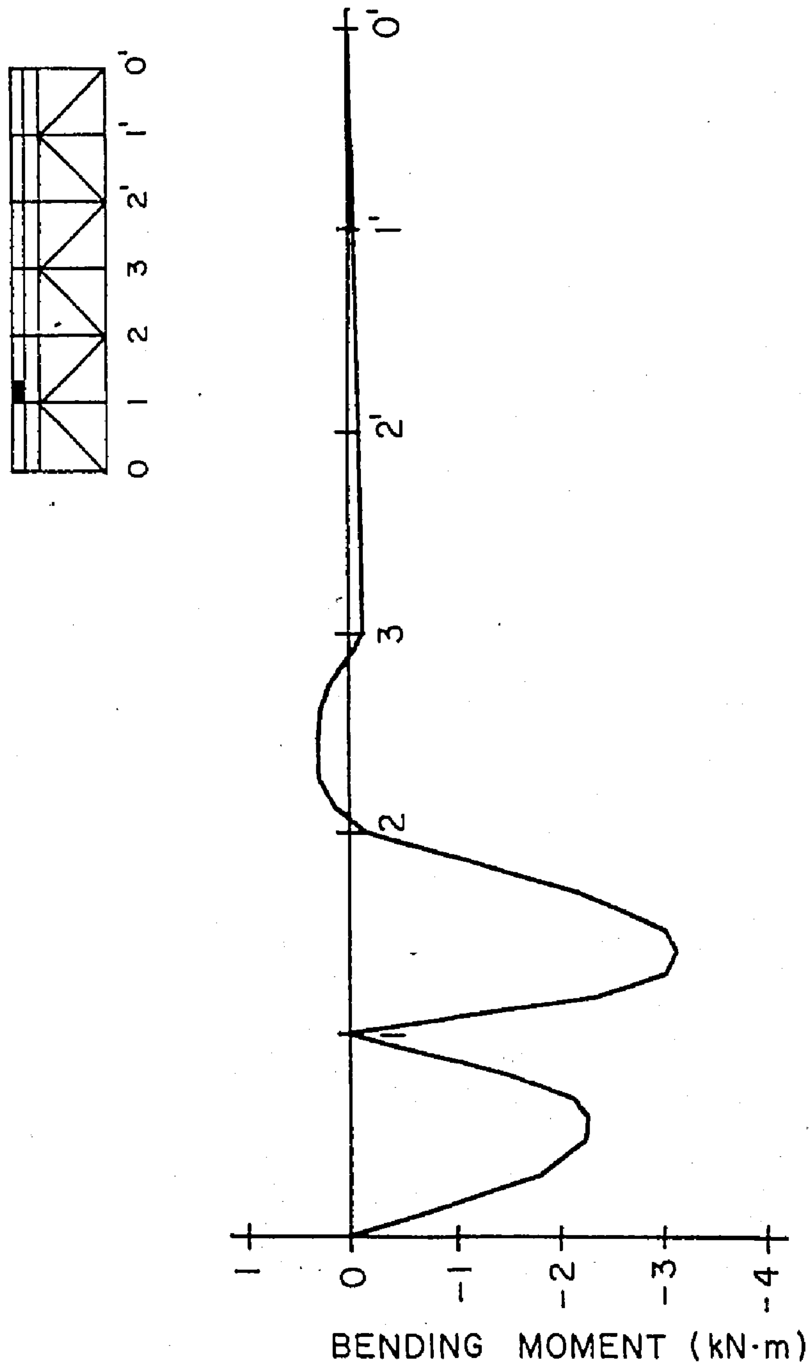


Fig. 18 Influence Line - U1U2 @U1

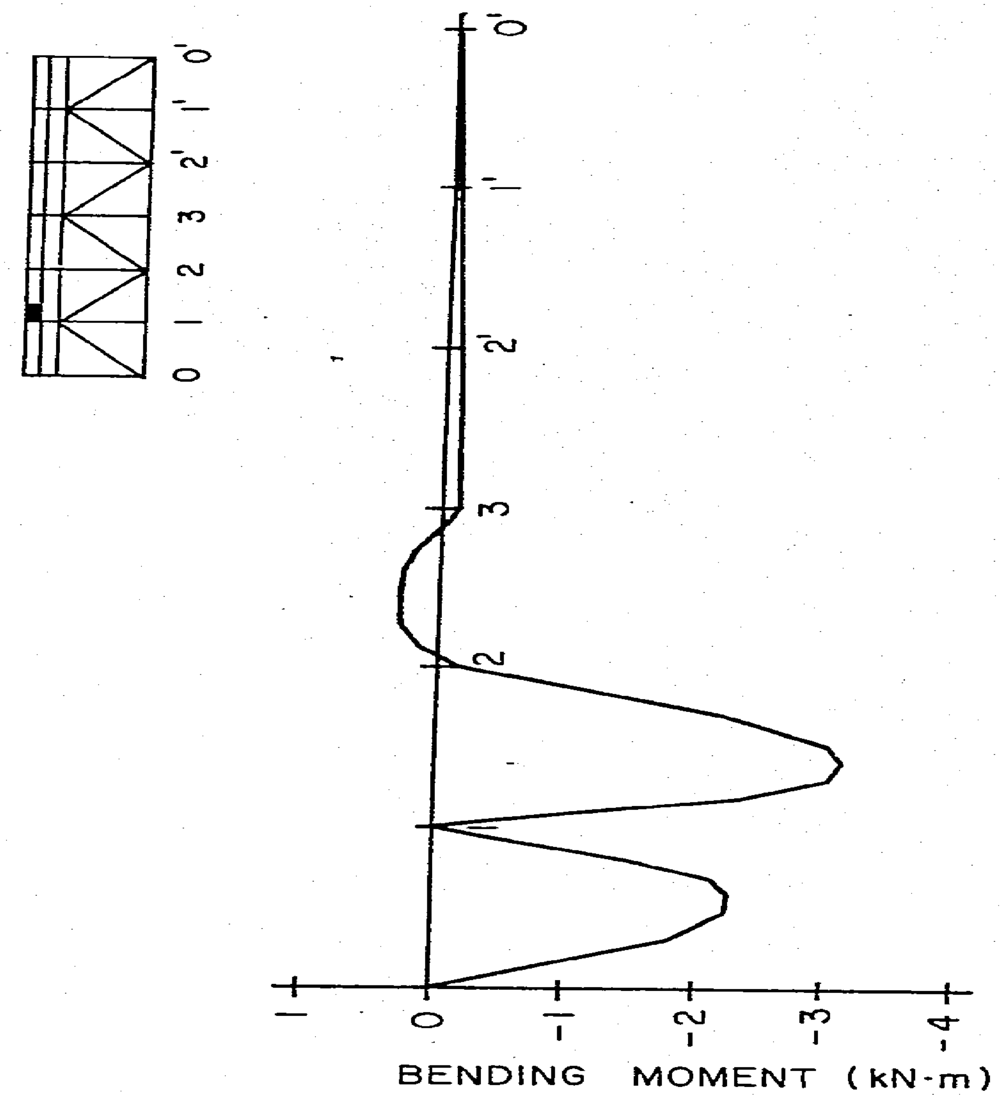
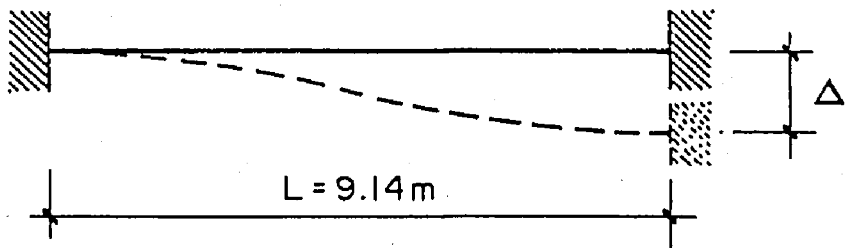
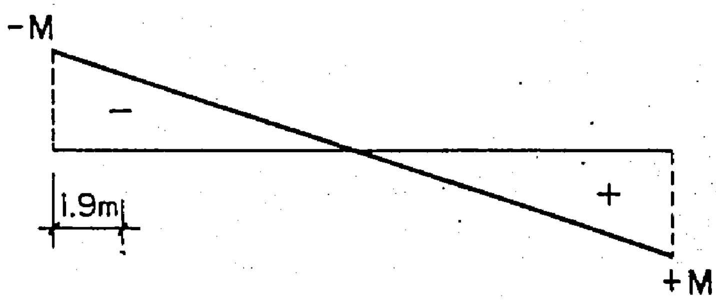


Fig. 18 Influence Line – U1U2 @U1

FORCED DISPLACEMENT



$$M = \frac{6EI\Delta}{L^2} = 12.50\Delta$$



ASSUME 25.4mm DEFLECTION

$\therefore M = 318\ kN\cdot m$

Fig. 19 Fixed-End Moment Analysis

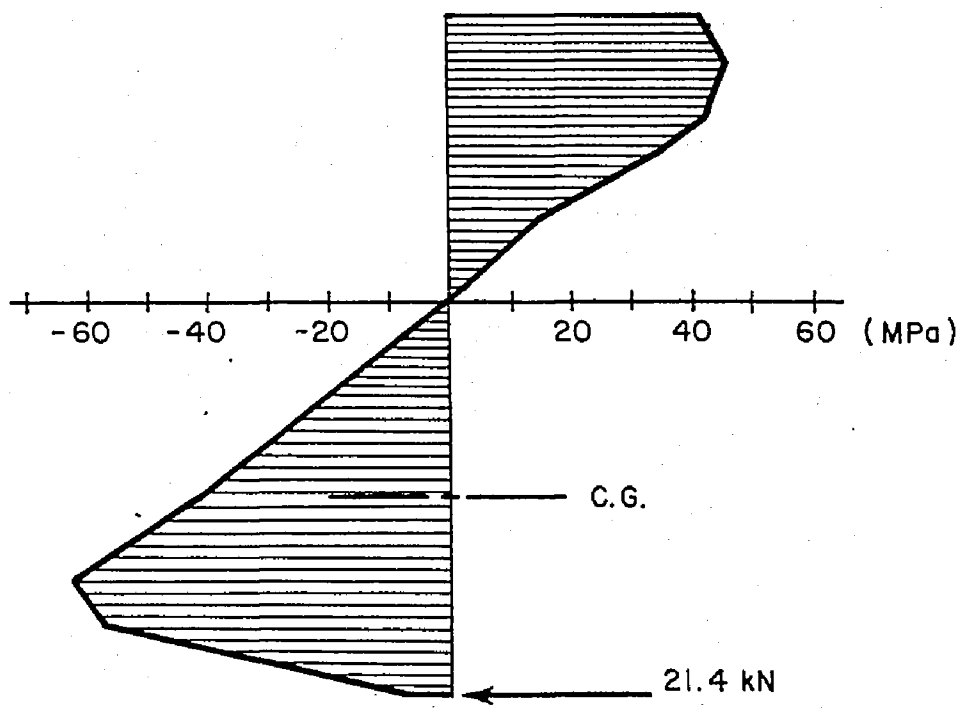


Fig. 20 Stress Distribution in Coped Stringer

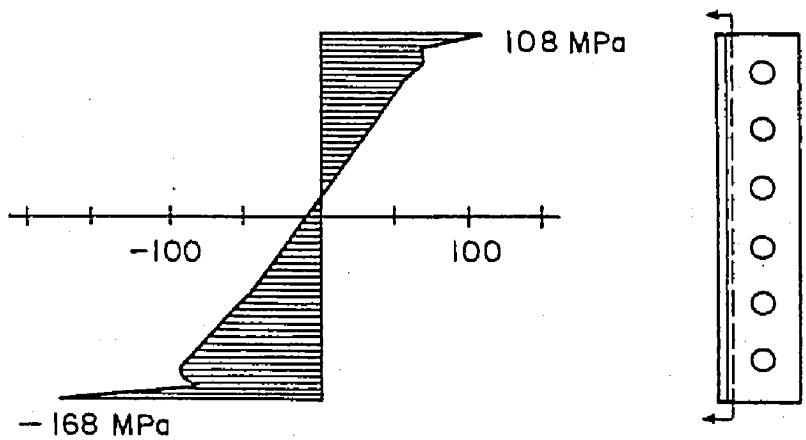


Fig. 21 Stress Distribution In Angle Fillet

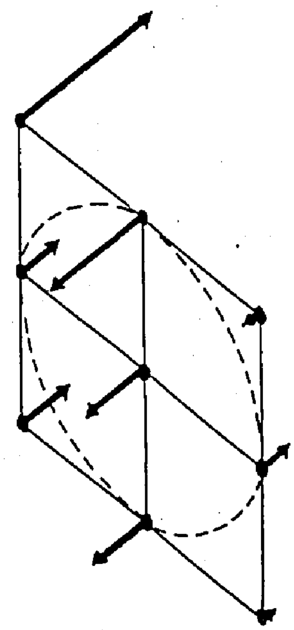


Fig. 22 Prying Action on the Top Rivet

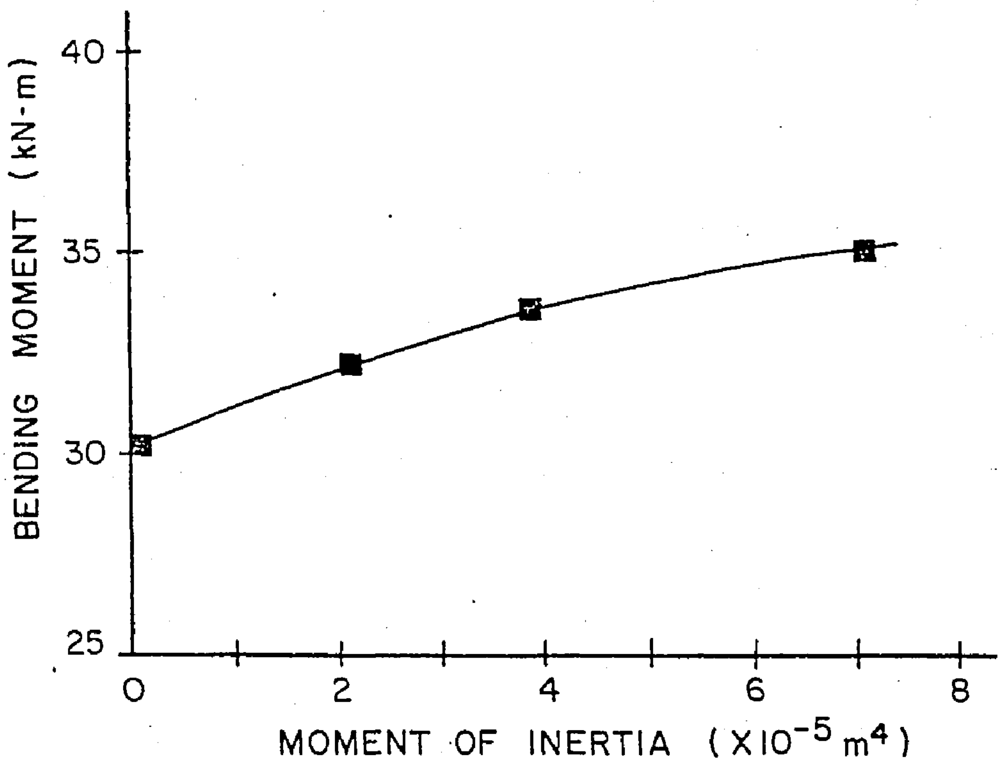


Fig. 23 Floorbeam Web Stiffness vs Moment

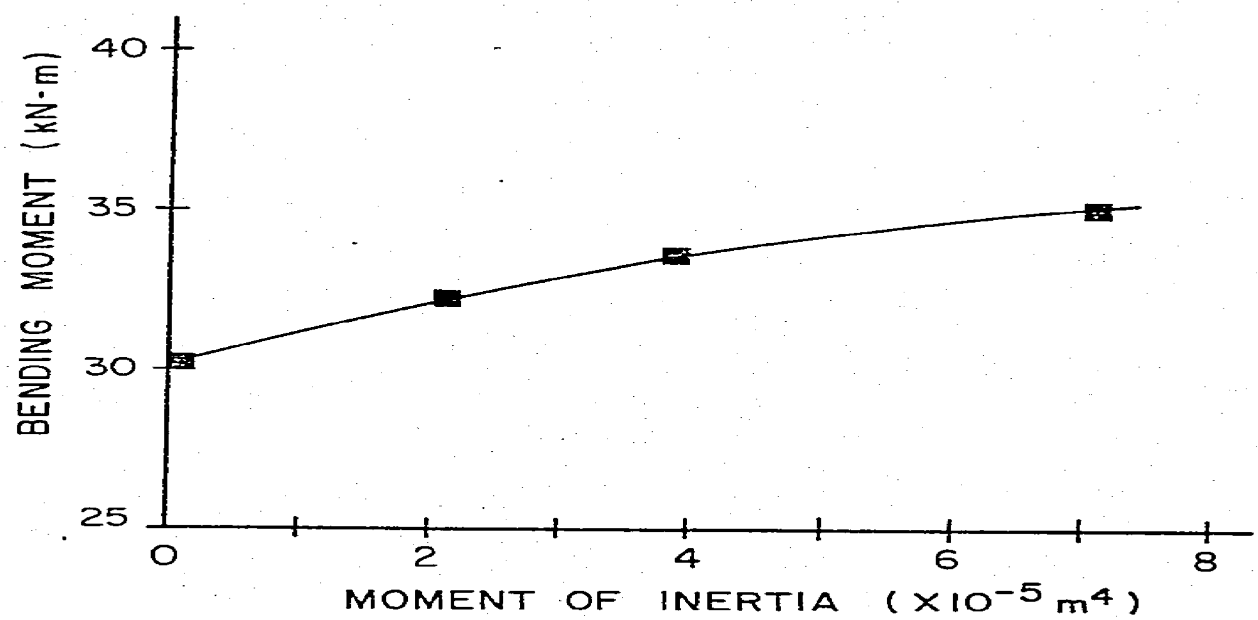


Fig. 23 Floorbeam Web Stiffness vs Moment

References

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Vita

The author was born to Richard and Barbara Keating in New York City on May 24, 1957. He grew up in the New York suburb of Ridgewood, New Jersey and attended Ridgewood High School. Upon finishing high school he entered Lehigh University in the Fall of 1975 where he majored in both civil engineering and architecture. He received a Bachelor of Science Degree in Civil Engineering in June of 1980 and a Bachelor of Arts in Arts and Sciences in January of 1981.

Graduate study in civil engineering began in the Spring 1981 semester at Lehigh while as a teaching assistant. He took the next school year off in order to work and gain practical experience. He was employed by the bridge engineering firm of Modjeski and Masters, in New orleans, Louisiana. While there he was involved with the inspection and evaluation of long-span highway and railroad bridges. He returned to Lehigh to continue graduate study as a teaching assistant in the Fall of 1982. He became a research assistant in November of that year and worked on the fatigue analysis of the Norfolk and Western Bridge 651 at Hannibal, Missouri. He will graduate in October of 1983 with a Masters of Science Degree in Civil Engineering and then continue graduate work as a PhD candidate under the supervision of Dr. John Fisher.