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REDUNDANCY OF WELDED STEEL I-GIRDER BRIDGES

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COMPUTER STUDY OF REDUNDANCY OF A SINGLE SPAN WELDED STEEL TWO-GIRDER BRIDGE

Interim Report

by

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This report describes a computer study investigating the hypothesis that a welded steel 2- girder bridge, commonly thought to be nonredundant, actually possesses significant load redistribution capability provided by such secondary members as the floor beams, cross frames, and bottom laterals. A finite element model of a real simple-span right 2- girder bridge is developed and subjected to dead load while imposing a full depth main girder crack at midspan. The results provide significant insights into the structural behavior and load redistribution mechanisms of the damaged bridge under dead load.

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ABSTRACT

The AASHTO Highway Bridge Specifications penalize nonredundant steel members in bridges but present only rough and conservative guidelines for actually determining if a structure is redundant. These guidelines are based on the usual steel bridge design procedures, which in turn are based on oversimplified 2- dimensional idealizations of complex 3- dimensional structures. There is reason to believe that secondary members not specifically designed for vertical load actually contribute greatly to the redundancy of the bridge, providing a contribution to load redistribution capability not currently accounted for in design.

This report describes a computer study investigating the hypothesis that a welded steel two- girder bridge, commonly thought to be nonredundant, actually possesses significant load redistribution capability provided by such secondary members as the floor beams, cross frames, and bottom laterals. A finite element model of a real simple-span right 2- girder bridge is developed and subjected to dead load while imposing a full depth main girder crack at midspan. The results provide significant insights into the structural behavior and load redistribution mechanisms of the damaged bridge under dead load.

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1. INTRODUCTION

1.1 Problem Statement

A nonredundant structure is one in which the failure of a single component may result in the collapse of the entire structure.

That structural engineers regard redundancy as a contributor to overall bridge safety is reflected in the AASHTO Specification requirements regarding the influence of repetitive live loads.[1]* The particular allowable stress range used in design against fatigue depends on whether the bridge is considered to be redundant or non-redundant: significantly higher allowable stress ranges are specified for redundant bridges than for nonredundant bridges. Examples of redundant and non-redundant structure types are presented in the footnotes to Table 10.3.1A of the 1983 AASHTO Specifications.[1] These are based on beliefs commonly held by bridge designers and specification writers.

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*References are listed in Chapter 7 of this report.

The problem is that these beliefs are based on steel bridge design procedures for which over-simplified 2-dimensional analyses are made. Bridges are in reality constructed as 3-dimensional structures, although they are not normally analyzed as such in design. The deck, stringers, floor beams, cross frames and lateral bracing all participate in carrying the dead and live loads. The redundancy of the bridge may increase due to the participation of these elements whose contributions are not normally considered in design. Therefore, the more stringent and uneconomical stress range restrictions for the so-called non-redundant bridges may not be warranted in all cases. The actual capability of 3-dimensional welded steel I-girder bridges to resist catastrophic failure, should a main load-carrying member fail, is not well known.

1.2 Background

While much study has been done of progressive collapse in buildings(2), little has been done to investigate the redundancy of bridges by accounting for 3-D interactions of the components. One study has investigated the redundancy of a deck truss bridge.(3) Other studies with simplified 3-D finite element models suggested that girder bridges do possess residual sources of load-carrying capacity that are not currently accounted for in design.(4,5) This work confirms the notion that to determine the behavior of such flexural systems, an analysis of the system as a 3-dimensional entity is required.[6]

The work described in this interim report is part of a research project to develop a framework to facilitate decisions regarding the realistic adequacy of welded steel 2-girder bridges to resist catastrophic failure in the event of the failure of a critical member. The bridges being investigated in this project include simple- span right, simple- span skew and 2-span continuous right bridges.

1.3 Purpose and Scope

The purpose of this interim report is to describe a computer simulation of an actual simple-span welded steel 2-girder right bridge under dead load only, subjected to a mid-span fracture of one of the two main girders. Live loading will be discussed in a future report.

The bridge investigated is part of the Betzwood bridge in Montgomery County, Pennsylvania. The girder crack is considered to extend through the bottom flange and full depth of the web without penetrating the top flange. It is assumed that fracture has already occurred due to the previous loading history of the bridge. Crack propagation and its driving force are not the main focus here. Dynamic effects at the instant of girder fracture are neglected. The damage criterion chosen for this study was arbitrarily selected to represent a possible realistic worst case. On a similar bridge, in-service fatigue cracking at a mid-span detail did lead to unstable crack growth and a full depth fracture in a main girder.[7] Other damage criteria are possible but are not investigated in this study.

The goal of this study is to assess the adequacy of the simple- span, 2-girder right bridge to resist collapse under the described damage criterion, recognizing that accounting for the complexity of the 3-dimensional interaction of members requires a synthesis of bridge-related expertise, understanding of structural behavior, and computer modelling.

1.4 Objectives of this Study

Specific objectives include the following:

1. Develop a 3-dimensional computer model for the bridge.
2. Obtain base-line stress resultants for the undamaged bridge model under dead loading.
3. Obtain stress resultants and deflections for the same bridge with a full-depth crack at midspan under dead loading.
4. Determine whether the bridge is non-redundant or redundant under dead load. If redundant under dead load, apply live load to find out how much

live load the damaged bridge model can sustain.

2. ANALYTICAL MODELING

2.1 Description of the Study Bridge

Figure 1 shows a partial plan and elevation of the Betzwood Bridge. The 90 ft. southbound bridge shown in the figure is used in this study. Designed for HS-20 live loading using the 1961 AASHO Specifications, the bridge was built in 1964 with an A36 steel superstructure and noncomposite 8" thick reinforced concrete deck.

Figure 2 shows the transversely and longitudinally stiffened girder. Each stiffened girder web is 92 " deep and 3/8" thick. The top and bottom flanges are 17-in. wide and change thickness from 2-in. to 1-1/2 in. as shown in the figure. A typical cross section is shown in Fig. 3. There are 6 cross frame locations and 5 bays equally spaced at 17'-10". The study bridge is the Southbound 2-girder bridge which is shown on the right in the figure. Figure 4 shows a plan view of the bottom lateral bracing. Figure 4b shows how the bottom lateral bracing frames into gusset plates welded to the girder web and floor beam connection plate 5 inches above the top of the bottom girder flange. Figure 4c shows the connection detail where bottom laterals cross. Only one of the lateral bracing members is continuous through the connection.

2.2 Description of the Analytical Model

For the computer simulation of the 3-dimensional structure a finite element model was constructed using the Computer- Aided Engineering system for Structural Analysis, GTSTRUDL.[8] Figure 5 shows the finite element model of the span which is used in the analysis. Figure 5a is a plot of the finite element discretization, looking from the west (see Fig. 1) from a position slightly below the span. Major structural components and bay numbering are also shown in the figure. Figure 5b is a partial view of the same span from below. This view is in the same orientation as the model in Fig. 5a and should clarify the viewing orientation employed there. The model is significantly more complex and thus more realistic than those used for the overall structure models shown in Refs. 4 and 5. Table 1 summarizes the finite element types employed for the various components of the bridge. These are described in greater detail in the following articles.

Since the bridge is constructed with a non- composite deck a critical question is the degree of composite interaction. Analytical and experimental experience has indicated that for load levels up to the elastic limit, one can assume complete interaction between the girders and the deck.[6] Without reliable criteria for slip, incomplete interaction cannot be modelled. It was decided to assume that the deck is composite with the girders and stringers. This assumption is consistent with either a similar bridge actually built composite or with this bridge retrofitted with a new composite deck to increase its load carrying capacity. Complete interaction is assumed through the full range of behavior.

2.2.1 Main Girders and Stringers

Figure 6 shows the finite element discretizations employed for the main girders and stringers in a typical bay. Each of the 5 bays shown in Fig. 5 is modelled the same. The flanges of the main girders and stringers are modelled with 3-D beam elements. Plane stress elements are used to model the webs. Although the out-of-plane degrees of freedom of the web elements are undefined, the girders and stringers in the model are still able to move freely in 3-D space.

Transverse and longitudinal stiffeners in the girders are also modelled using 3-D beam elements. Since the focus of this study is on the 3-dimensional behavior of the structure as a whole, the discretization neglects the gaps at the ends of the transverse stiffeners and floor beam connection plates. These elements are modelled as being fully attached to the girder flange. This approach assumes that the local web gap detail has a negligible effect on the global stiffness of the bridge.

The main girder crack is imposed in bay 3 of the west girder, as shown in Fig. 7. The crack is assumed to pass through the bottom flange and the full depth of the web, but not through the top flange.

2.2.2 Cross Section at Floor Beam Location

Figure 8 shows the finite element discretization employed at a cross section at a floor beam location. There are six such floor beam locations along the bridge, as shown in Fig. 5. The modelling considerations for the floor beams and cantilever outrigger brackets are similar to those for the girders and stringers. Flanges and stiffeners are modelled using 3-D beam elements. The floor beam flanges are considered not to be coped where the floor beam is attached to the girders. Webs are modelled with plane stress elements.

2.2.3 Cross Frames and Bottom Laterals

Discretization of a cross frame is also shown in Fig. 8. The plane of the bottom laterals is also shown in Fig. 8 (also refer to Fig. 5). The members in the cross frames and bottom laterals can be considered to have negligible depth and can thus be modelled with 3-D beam elements and truss elements. Beam elements are used for the horizontal cross frame members and the bottom laterals. Truss elements are used to model the cross frame diagonals.

All bottom laterals are assumed to be continuous from one girder to another. The corresponding detail is shown in Fig. 4c. The assumption that both members are continuous, instead of only one, has little effect on overall structural stiffness.

2.2.4 Concrete Deck

Figure 9 shows the finite element discretization of the bridge deck. The modelling considerations for the deck can become quite complex if an attempt is made to simulate its structural behavior exactly. There are several limit states to consider, such as crushing and cracking, as well as a significant range of nonlinear load-deformation behavior. This is discussed in greater detail in Chapter 3 of this report. It is evident that the element must account for in-plane stresses as well as bending stresses, since the deck functions as a top flange when it is assumed to act compositely with the girder. Since it was decided not to monitor the progression of concrete cracking through the deck, the use of layered elements was ruled out.[9,10] Thus, a flat thin- shell element is employed to model the deck.

Since the deck is not heavily reinforced, the reinforcing steel has a negligible effect on the stiffness of the deck in the uncracked condition. The presence of the reinforcing steel is therefore neglected in the computation of element properties. As a byproduct of this assumption, concerns about modelling such things as bond degradation, dowel action, and tension stiffening can be neglected. Cracking due to creep and shrinkage is also considered not to affect deck stiffness.

The entire deck is modelled as 8" thick flat plate elements having the properties of plain concrete. Each element is considered effective until the limiting surface tensile stress is reached.

Complete composite interaction is modelled by having the deck elements share nodes with the top flange of the girders and stringers. A side-effect of this approach is to lower the center of gravity of the deck. This modelling approximation is conservative and is assumed not to affect the results significantly.

2.2.5 <u>Bearings</u>

A significant modelling issue is the number of degrees of freedom to specify at supports. Modelling of supports is known to have a significant influence on stress resultants for horizontally curved girder bridges.[11] Modelling of supports has less sensitivity on straight girder bridges. Once a midspan girder crack is imposed, however, the sensitivity to boundary condition idealization is not well known. The bridge becomes asymmetrical, and the stress resultants may be significantly affected by the support conditions.

To investigate the sensitivity to support modelling assumptions, a comparative study was performed, and the results are summarized in Fig. 10. In Fig. 10, support reactions occur where support restraints are defined. The support reactions were found to be significantly affected by the choice of boundary conditions. The constraint condition shown in Fig. 10a was used in the initial models and is discussed further in Chapter 3.

3. ANALYSIS METHODOLOGY

3.1 General Description

The approach taken in this computer study was to perform static finite element analyses to compute deformations and stresses, first in the undamaged bridge model and then in the bridge model containing the full-depth girder crack at midspan. Initially, several linear elastic models were analyzed, with no limit states imposed. There were several reasons for these analyses:

1. To provide a reference against which the validity of the analytical approach may be assessed later.
2. To test the sensitivity of the damaged bridge model to the specification of the support conditions, as described in Art. 2.2.5.
3. To get a feel for the role of the cross frames, bottom laterals, deck and floor beams.

An analysis for redundancy requires procedures for determining the load-carrying capacity of damaged structural systems.[6] For this reason, limit state criteria must be defined. Once this is done, the first linear elastic analysis serves to identify the first limit state to be exceeded. The model must then be revised accordingly and re-analyzed in order to identify the next limit state exceedance. The revised model is itself revised and the process continues until either no further limit states are exceeded or excessive deflections appear, indicating nonredundancy.

Each solution procedes from zero load up to the next limit state. It is thus assumed that the behavior of individual components is linear up to the "failure" prescribed by the limit state condition. The major criterion for making this and other assumptions is to capture the overall behavior of interest while being conservative, that is, erring on the side of nonredundancy.

3.2 Limit State Definition

Table 2 summarizes the limit state criteria employed for the various components of the Betzwood bridge model. Although the bridge was designed in accordance with the 1961 AASHO Specifications, the limit states have been formulated wherever possible according to the intent of the 1983 AASHTO Load Factor Design provisions.

3.2.1 Cross Frames

A typical cross frame is shown in Fig. 11. The compression limit state for the diagonals (modelled as truss elements) is the inelastic column buckling strength, as specified by AASHTO (10-151). The tension limit state is taken to be the yield strength. For the horizontal members, both the beam-column stability and strength are checked. The horizontal member is considered to be braced by the presence of the walkway shown in Fig. 11.

3.2.2 Bottom Laterals

Both tension and compression limit states must be defined for the bottom laterals. The only bending that they are considered to carry is due to their own weight. In tension, the limit state is taken to be the yield strength. In compression, the column buckling limit state takes into account the influence of the other bottom lateral member crossing at midspan, which is always in tension, thereby contributing partial lateral support at mid-length. The approach taken follows that of Ref. 12. The effective length of the compression member is reduced by 50%, increasing the elastic buckling load fourfold. This detail is shown in Fig. 4c. End connections are assumed to be strong enough not to fail before the member itself fails.

3.2.3 Flexural Members

In the stringers and floor beams, the plastic moment M_p, or the reduced plastic moment M_{pc}, reduced due to the presence of axial force, is taken to be the limit criterion. M_{pc} is also taken to be the limit criterion in the top flange of the west girder above the full- depth girder crack.

The plate girder, on the other hand, is not expected to reach M_p. The finite element analysis results show that the plate girder does not enter an inelastic range of behavior.

3.2.4 Concrete Deck

Ideally, post-elastic modelling of the reinforced concrete deck should at least account for the nonlinear nature of the load-deformation behavior in compression, the concrete crushing in compression, the concrete cracking in tension, and the reinforcing steel yielding in tension.

In bending, the behavior of reinforced concrete slabs can be approximated as trilinear in nature as shown in Fig. 12. The moment- curvature relationship can be further idealized as elastic-plastic, with the elastic slope corresponding to the cracked section. This is consistent with the elastic-plastic behavior assumed for the steel components in this bridge model. A basic question surfaces on how to determine the moment- curvature relationship for the situation arising in the finite element model of the bridge. The problem is that the use of moment curvature relations for the bridge deck requires the use of moment-**thrust**-curvature relations due to the presence of axial forces in the bridge superstructure. In addition, the biaxial bending of the deck slab requires the adoption of two-dimensional moment-curvature relationships.[10] Needless to say, these are not applicable for general usage. Thus, a limit state criterion based on some simplified moment-curvature relationship is not available.

The approach taken in this study is to use a simple limit state criterion to deal with this highly indeterminate and iterative complex situation, yet err not too much on the side of nonredundancy. Therefore concrete tension cracking is defined as the limit state. When the surface tensile stress exceeds $7.5\sqrt{f'_c}$ in an element, discrete cracks are imposed at the appropriate edge of that element in the finite element model. Subsequent analyses consider the deck to be ineffective in transmitting forces across these localized discrete cracks. This approach conservatively neglects the post-cracking stiffness of the cracked deck elements and assumes a constant value for the limit state. The contribution of the steel reinforcing bars is neglected.

3.2.5 Bearings

Limit state criteria are needed for the lateral load capacity of the bearings. As will be shown in Chapter 4, imposing the through-depth girder crack causes extremely high support reactions in the horizontal plane. Conservatively ignoring the restraining effects of friction, the only resistance to horizontal forces at the fixed bearings is provided by two 1-1/4" diameter anchor bolts as shown in Fig. 13. The shear capacity of these bolts, shown in Table 2, dictates the fixed bearing capacity. At the expansion bearings, the only resistance to lateral forces is provided by the keeper plates at the top of the rocker, as shown in Fig. 14. The keeper plate capacity shown in Table 2 is determined from a yield line analysis.

3.3 Analysis Scheme Employed

Analyses at varying levels of complexity are possible. The approach actually taken utilizes "small" strain, "small" displacement finite element analysis and is summarized as follows.

1. Perform an elastic finite element analysis of the undamaged bridge. This gives the "base-line results."
2. Impose the through-depth girder crack at midspan. Perform an elastic finite element analysis.
3. Compute limit state values for the various components of the bridge. Identify in which elements the limit values are exceeded by the results of the preceding step.
4. Modify the finite element model by reducing the stiffness of the components with the highest limit value exceedances.

5. Perform elastic finite element analyses of the revised model. Each such analysis starts from zero load.
6. If instability or excessive deflections result, consider the model to be nonredundant.
7. Otherwise, return to step 3.

This approach, in combination with the above-defined limit states, can be viewed as a lower bound plasticity analysis with the additional assumption that the behavior is linearly elastic up to the limit criterion (e.g., yield or buckling in steel, cracking in concrete). This additional assumption follows from the avoidance of a more involved incremental nonlinear finite element analysis approach. According to the lower bound approach, if there is a way for the structure to carry a load, then the structure will carry at least that load (although not necessarily in the same manner).

4. RESULTS AND DISCUSSION

4.1 Base-Line Results (Undamaged Bridge)

Major results for the undamaged bridge are summarized in Table 3. The midspan deflection is 0.62", and the tensile stress in the bottom flange of the main girder is 9.6 ksi. The results compare favorably with calculations based on treating the entire bridge as a single beam subjected to a uniform dead load due to its self-weight. Even without the girder crack, the bottom laterals and some cross frame members carry significant forces, the highest at midspan as shown in Figs. 15 and 16.

4.2 Full Depth Main Girder Crack at Midspan

4.2.1 Response Under Dead Load Alone

Table 4 summarizes the sequence of model changes made to reflect limit state exceedances and the major changes that occur in the results for the revised models. The initial finite element models correspond to step 1, incorporating the girder crack but not imposing any other component failures. The results of the initial models corroborate those obtained from the I79 Backchannel bridge.[13] The full-depth girder crack used in the present computer study is very similar to that encountered on the

I79 bridge, although that bridge was 3-span continuous. On that bridge, an analysis showed no overstresses in the main girders, but stresses approaching the yield point were discovered in the floor beams and cross frames nearest the fracture.(13) On the study bridge, very high compressive forces and significant moments develop in the cross frames nearest the girder crack, as shown in Fig. 17. These forces are apparently the result of the large increase in the forces in the bottom laterals as shown in Fig. 18.

Some other noteworthy developments occur in these initial models. Figure 10 shows the high support reactions in the horizontal plane, critically dependent on modelling decisions discussed in Art. 2.2.5. These figures illustrate how assumptions regarding support modeling are critical to the stress resultants, even before consideration of limit states.

Particularly surprising are the high longitudinal reactions acting at the fixed bearings (see Fig. 10a). These can be rationally explained, but only when the full 3-dimensional interaction of the bridge components is considered. When the midspan girder crack is imposed, the cracked girder deflects downward. Since the girder has finite depth, the downward deflection is accompanied by a longitudinal displacement at the expansion bearing. This can be seen by comparing the longitudinal displacements in Fig. 10 to the longitudinal displacement of the undamaged bridge, 0.24". The downward deflection is resisted primarily by warping action in the deck and by the cross frames, in which forces are induced by the differential deflection of the main girders. The longitudinal displacement is resisted primarily by the bottom lateral bracing system, which attempts to transfer this longitudinal movement to the bottom flange of the uncracked girder. If no lateral restraint exists at the expansion bearings, the reactions and support displacements shown in Fig. 10b are predicted. With lateral restraint at the expansion bearings, lateral support reactions occur, as indicated in Figs. 10a and 10c. Moment equilibrium about a vertical axis then requires the high longitudinal support reactions at the fixed bearings.

Figures 17 and 18 show how high tensile forces in the bottom laterals in the middle bays of the bridge induce high compressive forces in the cross frame horizontals nearest the girder crack. The horizontal plane containing the bottom lateral bracing system functions as a truss incorporating the bottom laterals and the cross frame horizontals. This truss system can be viewed as a backup bottom flange that becomes activated when the main girder crack is imposed. It can thus be said to be an "alternate load path," transferring forces that the cracked girder tension flange would otherwise have sustained, across to the uncracked girder and into the bearings.

4.2.1.1 Progression of "Failures"

It is unlikely with such a girder crack that the bridge will remain elastic. Localized failures will occur due to exceeding limit states such as those described in Art. 3.2, and the load will redistribute throughout the remaining components of the bridge according to their relative stiffnesses.[6] It is already apparent at step 1 (see Table 4) that redundancy will depend on the capacity of the following components: deck, bearings, bottom laterals and cross frames, including floor beams.

It is concluded that a fixed bearing would fail before the keeper plate on the expansion bearing would fail. This follows from the observation that the longitudinal support reactions shown in Fig. 10a greatly exceed the limit state values for the fixed bearings shown in Table 2. Thus, after imposing deck cracks in step 2 (see Table 4), longitudinal restraints at a fixed bearing are removed entirely in step 3. The remaining lateral restraints at this point in the model are just sufficient to prevent rigid body motion in the horizontal plane. The cracked girder deflection increases from 1.71" to 1.98" and the cross frame forces increase.

At this stage, high compressive forces are induced in the cross frame horizontals. The bottom laterals in the middle bays are sustaining very high tensile forces induced by the opening of the girder crack, as discussed in Art. 4.2.1. The bridge elements available to resist the lateral component of these forces are the cross frame horizontals and the bottom flange of the uncracked girder. The cross frame horizontals, resisting these forces axially, are relatively more stiff than the girder flange resisting these forces by bending about the vertical axis. The high compressive forces indicate that beam-column instability will occur in the cross frame horizontals nearest to the uncracked girder, in the middle two cross frames, cross frames 2-3 and 3-4 (see Fig. 5).

The result of "removing" these two cross frame horizontals in step 4 is to increase the downward deflection. Figures 19 through 23 summarize some of the important results at this stage of the analysis. Comparing these with Figs. 10, 15, 17, and 18 gives some indication of how the dead load is being redistributed as members fail. For example, comparing Fig. 19 with Fig. 10a shows how support reactions have changes in proceding from step 1 to step 4 (see Table 4). The loss of lateral support restraints caused by the bearing failure in step 3 has allowed the vertical reactions to balance out somewhat. Comparing Fig. 20 to Fig. 18 indicates that tension forces decrease in the middle bay bottom laterals since the load paths provided by the cross frame horizontals are in effect no longer there- those members have buckled. Substantial tension forces are still maintained, however, because the bottom flange of the uncracked girder is still there to provide a path for these forces. The uncracked girder deflects toward the cracked girder 0.41" as it develops weak-axis moments about the strong axis of the bottom flange to sustain the bottom lateral tension forces shown in the middle bays of Fig. 20.

Comparing Fig. 21 to Fig. 17 shows how the cross frame forces in step 4 differ from those in step 1. The middle two cross frames, 2-3 and 3-4, have much lower forces in them since one of the horizontals has buckled. The end cross frames, 0-1 and 5-6, sustain slightly higher forces in step 4 than in step 1. The other cross frames, 1-2 and 4-5, develop only fairly small forces.

Deflections of supports, laterals, and cross frames after step 4 are shown in Figs. 19, 20, and 22 respectively. The increased downward deflection at midspan induces distortions causing sufficiently high compressive forces in the end cross frames to cause buckling of cross frame horizontals. These distortions are shown in Fig. 22. Step 5 (see Table 4) "removes" these end cross frame horizontals from the model, resulting in slightly increased cracked girder deflection and additional cracking in the deck. Work is currently under way in step 6, investigating the effect of imposing further deck cracks.

4.2.1.2 Redundancy Assessment

The roles of so-called secondary members such as cross frames and bottom lateral bracing have been described in the preceding section. Redundancy depends critically upon the performance of these components.

The bottom lateral members supplied may be adequate as long as they are detailed to be continuous at the crossing connection as shown in Fig. 4c and the end connections are properly designed and detailed to transfer the high forces induced. Other components such as the cross frames appear to be inadequate, as several of the horizontals buckle.

4.3 Load Path Identification

Although it has been used in this report and in the technical literature,[1,6] the phrase "alternate load path(s)" has not been rigorously defined. Phrases used in a recent report[6] such as "structures are said to possess multiple load paths..." and "so-called redundant load paths" indicate the lack of a clear definition of the term. In essentially one-dimensional members which carry primarily axial stresses, the notion of load path seems intuitively clear. A load path through such a member in a structural system transfers forces from one end to the other. But in the more general case of a 2-dimensional component such as a bridge deck, the meaning of the term "load path" is not clear. It appears that, in general, what is meant by the phrase "alternate load paths" is actually load redistribution capability.

The load redistribution capability of the study bridge has been alluded to in the discussion of Art. 4.2.1. With a full-depth crack imposed at midspan, the girder sheds the load that it had previously carried. What had been tension in its bottom flange is partially redistributed to the bottom lateral bracing system, which transfers forces over to the intact girder. The tension forces in the bottom lateral bracing system also induce high compressive forces in the bottom horizontals of the cross frames. At the same time, the downward deflection of the cracked girder induces high forces and moments in the middle two cross frames and warping action and cracking in the deck, due to the differential deflection of the two girders. Redundancy requires that these components have sufficient capacity to resist these induced forces.

5. SUMMARY AND CONCLUSIONS

5.1 Summary of Approach Taken

The approach taken in this study has been to develop a finite element model of a real bridge and subject it to dead load while imposing a full-depth main girder crack at midspan. The main interpretation task has been to trace the redistribution of load-carrying capacity and identify the mechanisms of load transfer. A lower-bound plasticity approach rather than an incremental nonlinear computational scheme has been adopted to handle limit state exceedances and their effect on the behavior of the model.

5.2 Conclusions

5.2.1 Redundancy of the Actual Study Bridge

The finite element analyses described in this report indicate that a critical role is played by components and details that are not rigorously designed for the induced loadings they encounter in this study. Questions about keeper plate capacity on the expansion bearings, anchor bolt shear in the fixed bearings, and the beneficial effect of member continuity on the buckling strength of the bottom laterals illustrate some of the parameters that become relevant when a main girder crack is imposed on the model of a simple span 2-girder bridge. For the model of the study bridge under dead load, imposing the girder crack is followed by cracking and warping in the deck, failing of a fixed bearing, and buckling of several cross frame horizontal members.

Although the question of redundancy/nonredundancy of this bridge is not yet resolved, significant insights have been gained into the structural behavior and load redistribution mechanisms of the damaged bridge under dead load. For the specific scenario of a through-depth girder crack at midspan of a simple- span welded steel 2- girder right bridge, high forces are induced in what are thought to be "secondary" members during design. If such components as bottom laterals, cross frames and deck are intentionally designed to resist forces induced upon the failure of the girder, redundancy is assured.

5.3 Further Work

The current research contract, of which this study is a part, may deal with the following further study:

1. Investigation of the simple span bridge with failed main girder when subjected to live load, if it is judged to be redundant under dead load alone.
2. Investigation of a single span skew bridge and a 2-span continuous right bridge with failed main

girder when subjected to dead and live load.

3. Use of upper bound plasticity analysis to complement finite element analysis in determining the redundancy/ nonredundancy of all three bridges.
4. Comparison of the lower bound finite element analysis results with the upper bound plasticity analysis results.

6. TABLES AND FIGURES

Table 1 SUMMARY OF FINITE ELEMENTS EMPLOYED

Bridge Component	Finite Element Type	GTSTRUDL Designation
Girder		
flange	3-D beam	SPACE FRAME
web	plane stress	PSHQ
stiffeners	3-D beam	SPACE FRAME
Stringer		
flange	3-D beam	SPACE FRAME
web	plane stress	PSHQ
Floor Beam		
flange	3-D beam	SPACE FRAME
web	plane stress	PSHQ
Cantilever Bracket		
flange	3-D beam	SPACE FRAME
web	plane stress	PSHQ, CSTG
stiffeners	3-D beam	SPACE FRAME
Deck	flat shell	SBHQ6
Bottom Laterals	3-D beam	SPACE FRAME
Cross Frames		
horizontals	3-D beam	SPACE FRAME
diagonals	3-D truss	SPACE TRUSS

Notes:

1. The PSHQ (2 DOF/node) hybrid quadrilateral element assumes a quadratic field for stresses within the element and linear variation of displacements on the boundaries.

2. The SBHQ6 (6 DOF/node) hybrid stretching and bending quadrilateral combines:
   - -in-plane PSHQ
   - -bending BPHQ (quadratic stress field within the element, cubic transverse displacement along the boundaries, and linear normal rotations along the boundaries, a compatible element)
   - -a fictitious rotational stiffness for suppressing instabilities in shell problems.

   The in-plane and bending stiffnesses are uncoupled. The SBHQ6, like the PSHQ, is a hybrid "Reissner" element.

Table 2 LIMIT STATE CRITERIA EMPLOYED

Component	Parameter of Interest	Limit Criterion	Value
expansion bearing keeper plate	lateral load capacity	plastic capacity (yield line analysis)	11.9 k (1)
fixed bearing anchor bolts	anchor bolt capacity	bolt shear capacity	42 k (2)
bottom lateral	compressive capacity	buckling load	varies (3)
bottom lateral	tensile capacity	ultimate tensile strength	281 k (4)
cross frame diagonal	compressive capacity	inelastic column buckling strength	58.1 k (5)
cross frame horizontal	beam-column capacity	stability limit	varies (6)
		strength limit	varies (7)
RC deck	cracking stress	tensile cracking	440 psi (8)

Notes:

(1) Behavior is assumed rigid to failure.

(2) Assumes A307 steel.

(3) Must account for intermediate support due to the presence of the crossing member. The limit value varies depending on the tension in the crossing member.

(4) AASHTO 10.46

(5) AASHTO (10-151)

(6) AASHTO (10-155)

(7) AASHTO (10-156)

(8) AASHTO 8.15.2.1.1

Table 3 SUMMARY OF BASE-LINE RESULTS (UNDAMAGED BRIDGE)

Parameter	Location	Finite Element Model (2)	Hand Calculation (1)
vertical deflection	at midspan	0.62"	0.49"
tensile stress in bottom flange of girder	at midspan	9.6 ksi	9.3 ksi
longitudinal displacement	at expansion bearing	0.24"	0.12"
longitudinal in-plane stress in deck	at midspan	0.24 ksi compression	0.21 ksi compr.
maximum axial force in cross frame horizontal	in cross frame 2-3	15.5 k compression	---
maximum axial force in bottom lateral	in bay 3	10.0 k tension	---

Notes:

(1) Based on treating the entire bridge section (including the deck) as a single composite beam, subjected to uniform dead load.

(2) The depth of the entire bridge cross section in the finite element model is less than that assumed for the "hand calculation," since the center of gravity of the deck is lower for the finite element model, as described in Art. 2.2.4.

Table 4 PROGRESSION OF FAILURES IMPOSED

Description	Rationale	Resulting Changes in Behavior of Model
STEP 1: Impose girder crack.	"worst case" fracture-critical member	Cracked girder deflects 1.65". High longitudinal reactions. High forces in cross frames and bottom laterals.
STEP 2: Impose transverse deck crack above cracked girder	extreme exceedance of cracking stress	Cracked girder deflects 1.71". No major changes.
Impose longitudinal deck crack along uncracked girder	marginal exceedance of cracking stress	
STEP 3: Remove failed fixed bearing	anchor bolt shear exceeded	Cracked girder deflects 1.98". Higher forces in cross frames. Forces in bottom laterals take on "mirror image" pattern.
STEP 4: "Remove" two buckled cross frame members (in cross frame 2-3 and 3-4)	AASHTO beam-column stability limit exceeded	Cracked girder deflects 2.23". Forces in bottom laterals decrease in middle bay, increase in other bays. Bottom flange of uncracked girder kicks towards the cracked girder.
STEP 5: "Remove" two buckled members in end cross frames	AASHTO beam-column stability limit exceeded	Cracked girder deflects 2.29". Forces in bottom laterals decrease slightly in end bays. Other changes are negligible.
STEP 6: Impose additional deck cracks	slight exceedance of tensile cracking stress.	Currently under investigation.

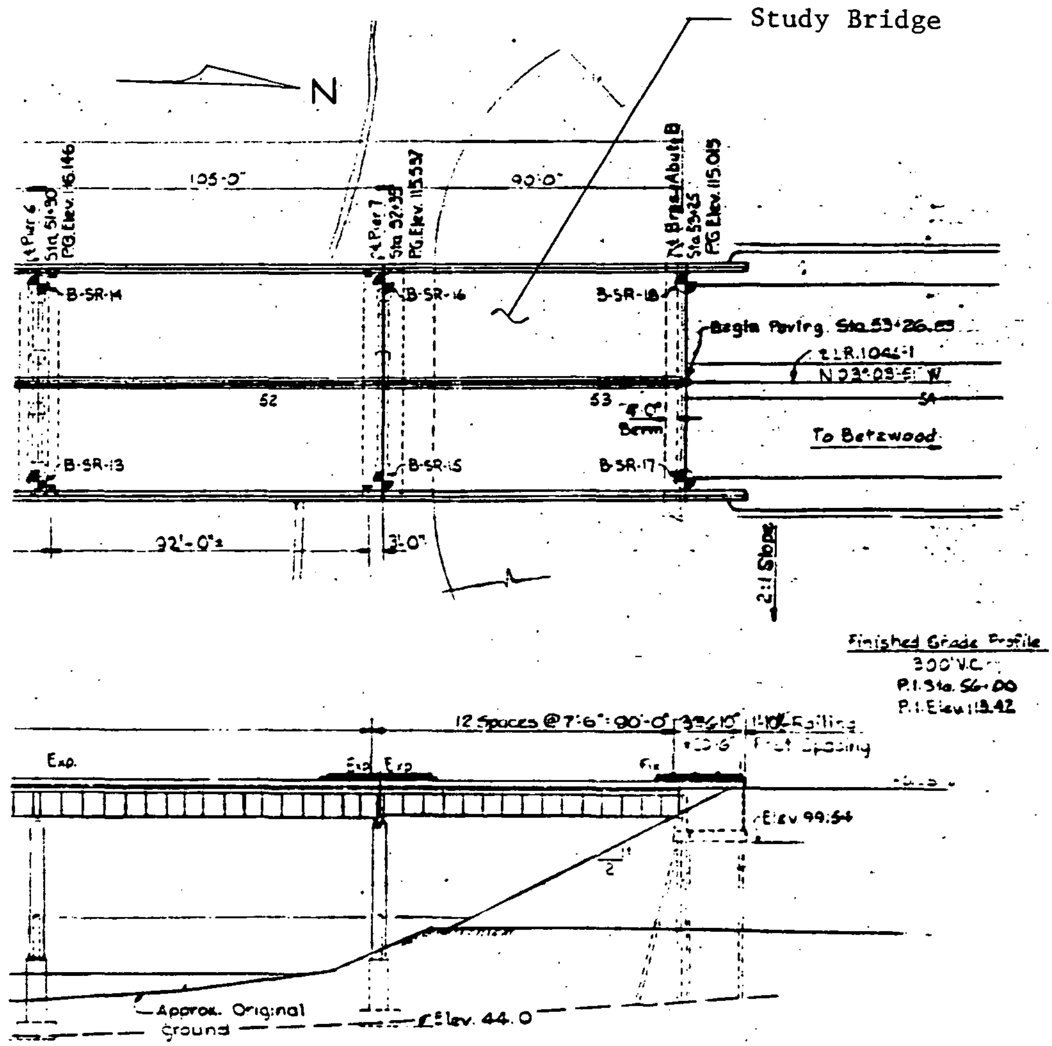


Fig. 1 Partial Plan and Elevation of the Betzwood Bridge

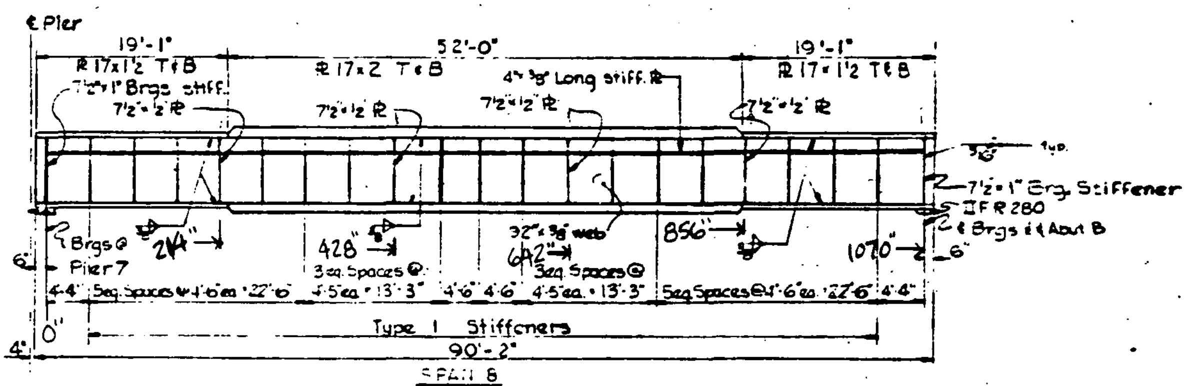


Fig. 2 Elevation of Transversely Stiffened Girder

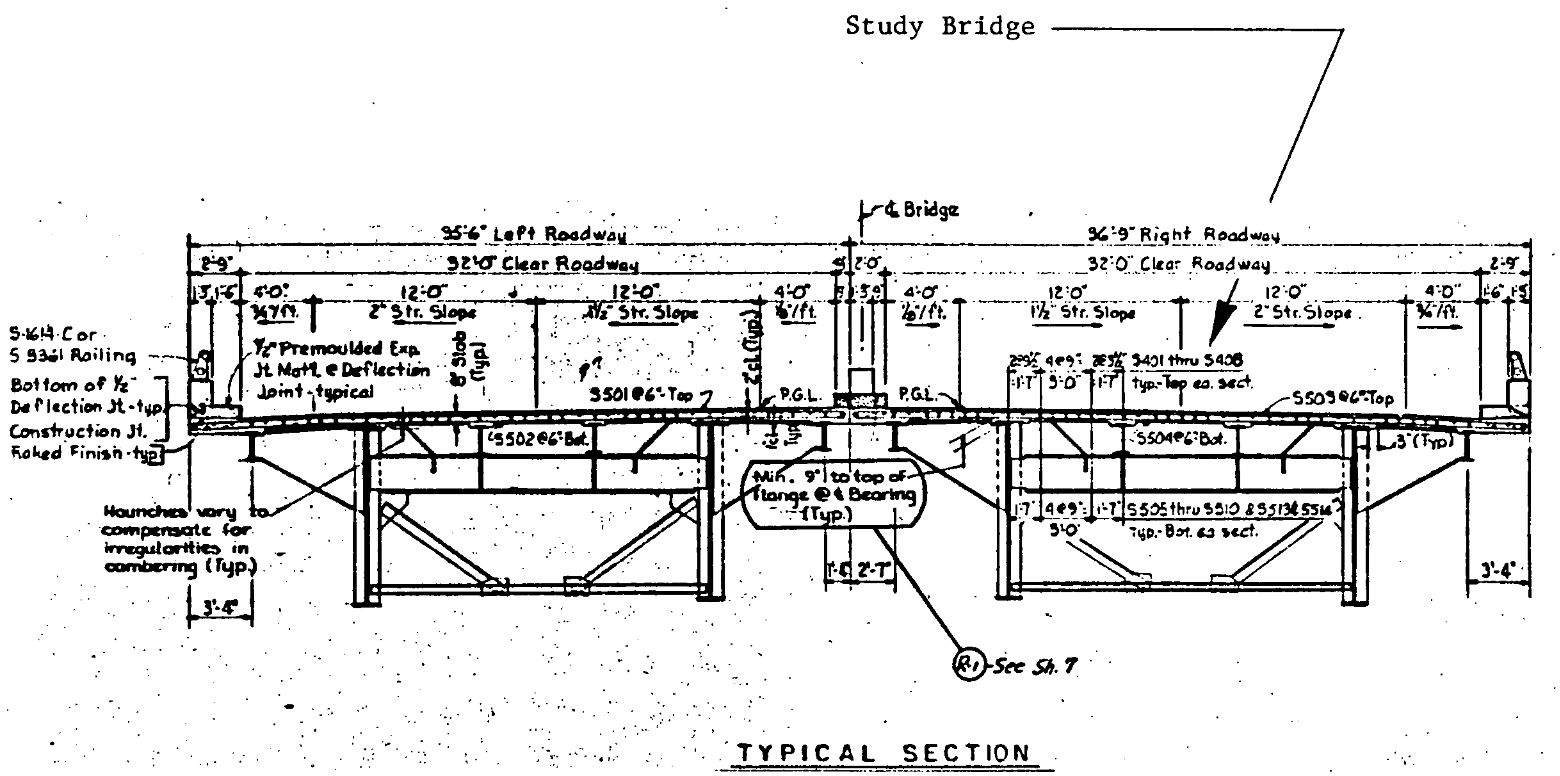
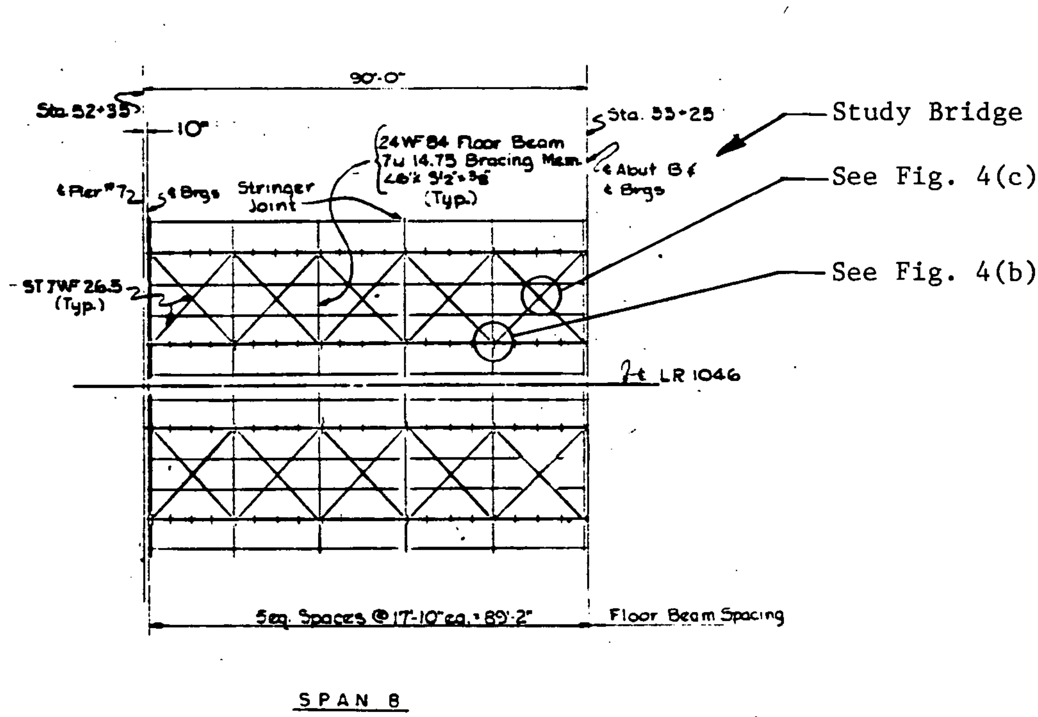
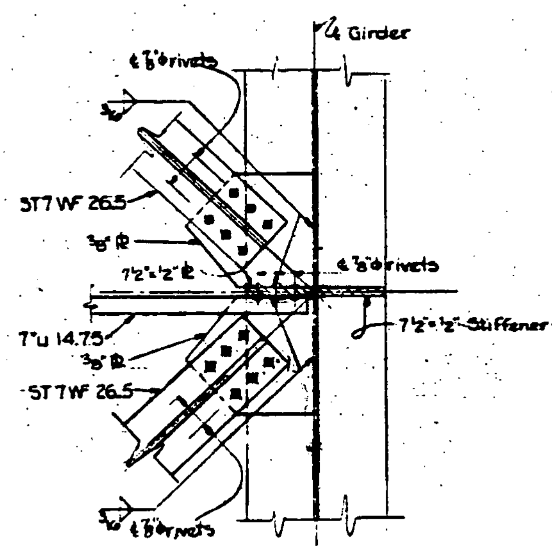


Fig. 3 Typical Cross Section of Bridge

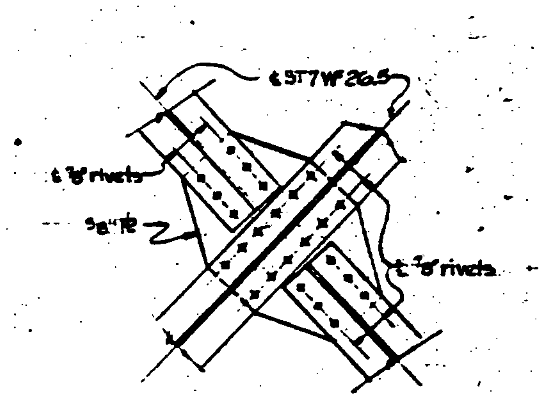


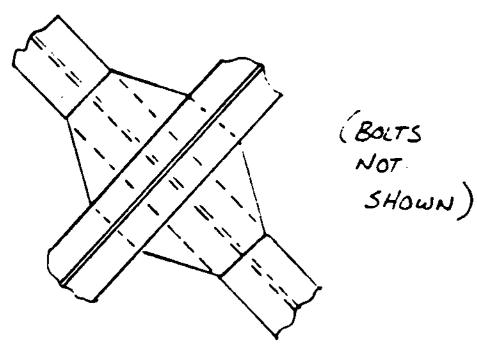
a) Plan View of Bottom Lateral Bracing Arrangement



b) Detail at Connection to Girder

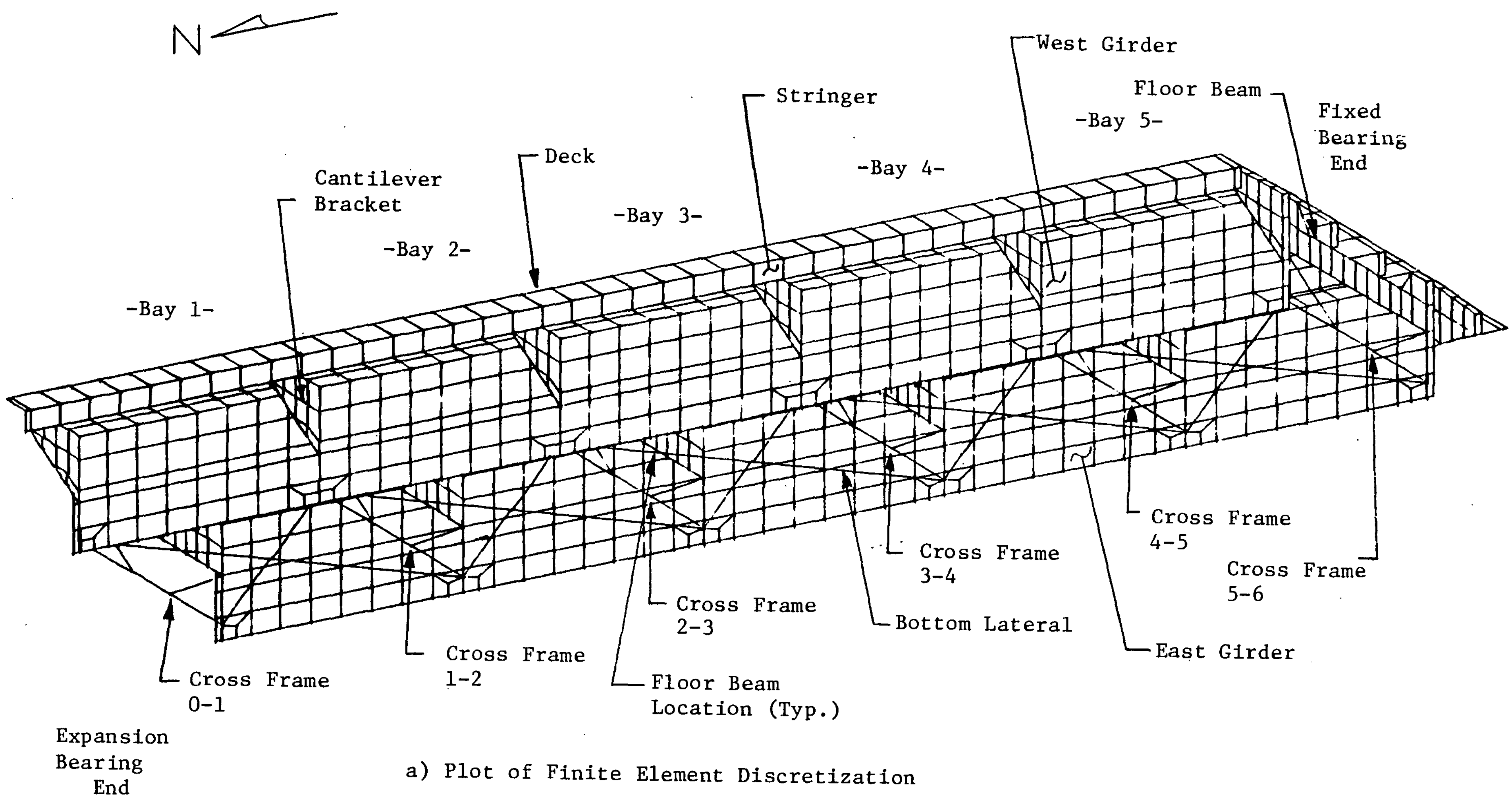
Fig. 4 Plan Views of Bottom Lateral Bracing





Revised Detail

Fig. 4(c) Connection Detail Where Bottom Laterals Cross



a) Plot of Finite Element Discretization

Fig. 5 Finite Element Model of Span
Used in the Computer Analysis

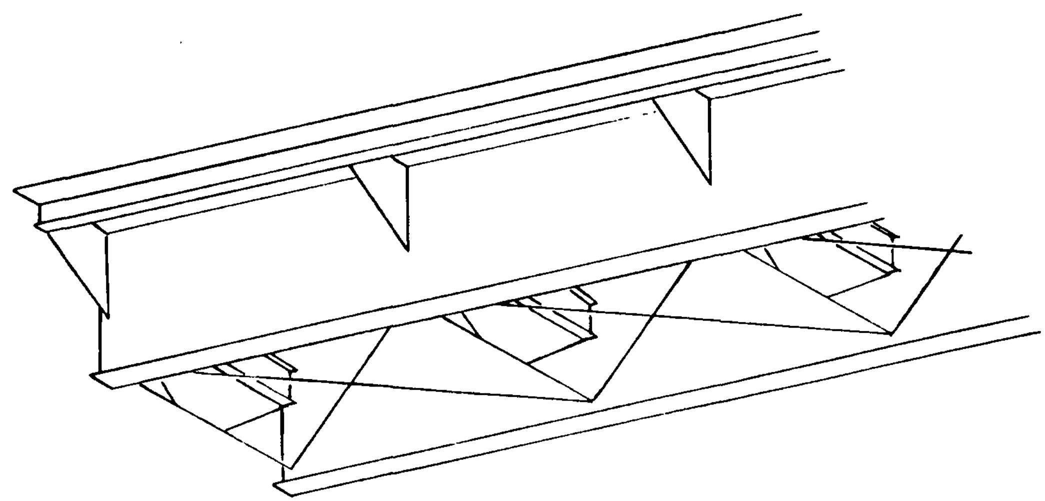


Fig. 5(b) Partial View of Span from Below

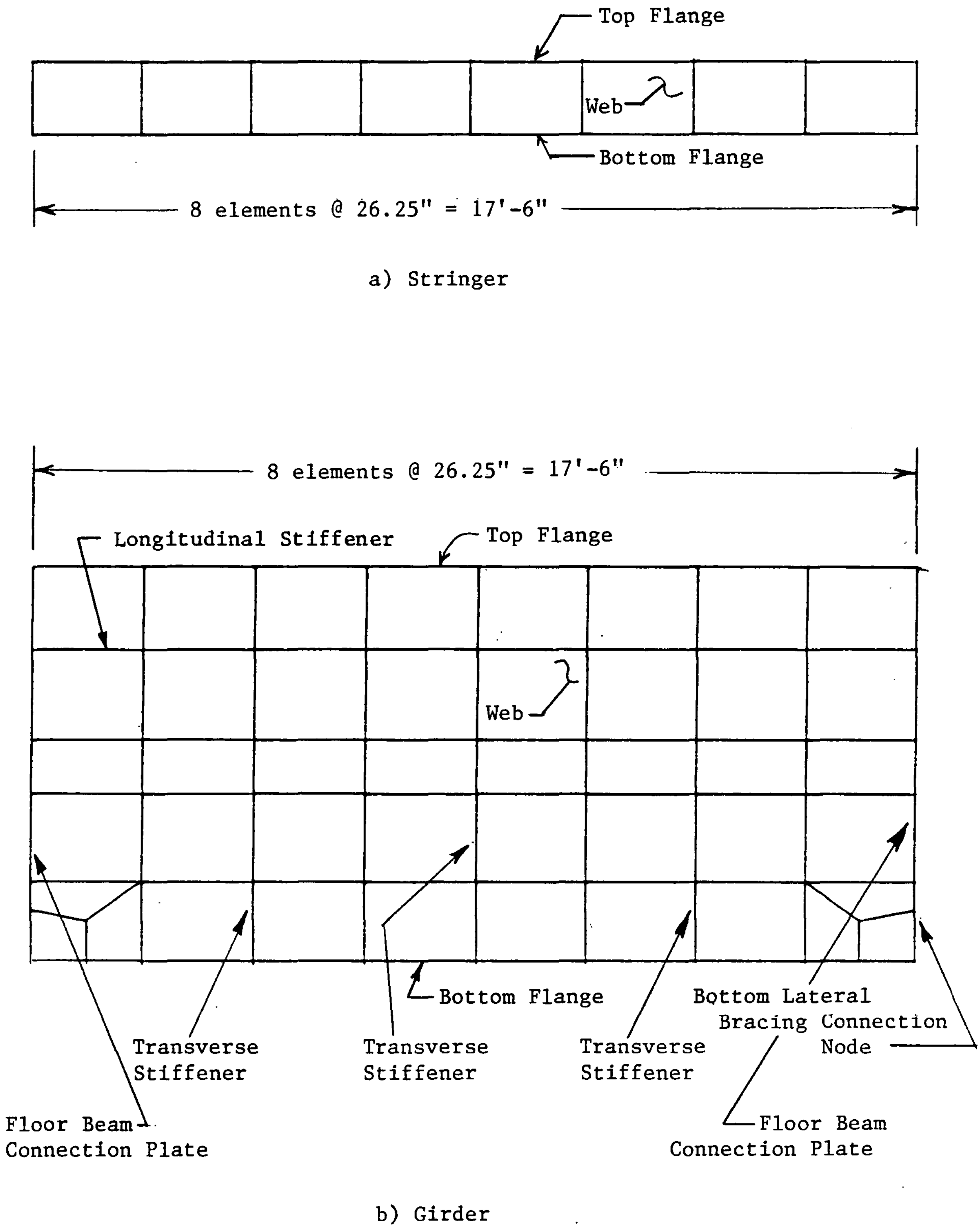


Fig. 6 Finite Element Discretization of Stringer and Girder in a Typical Bay

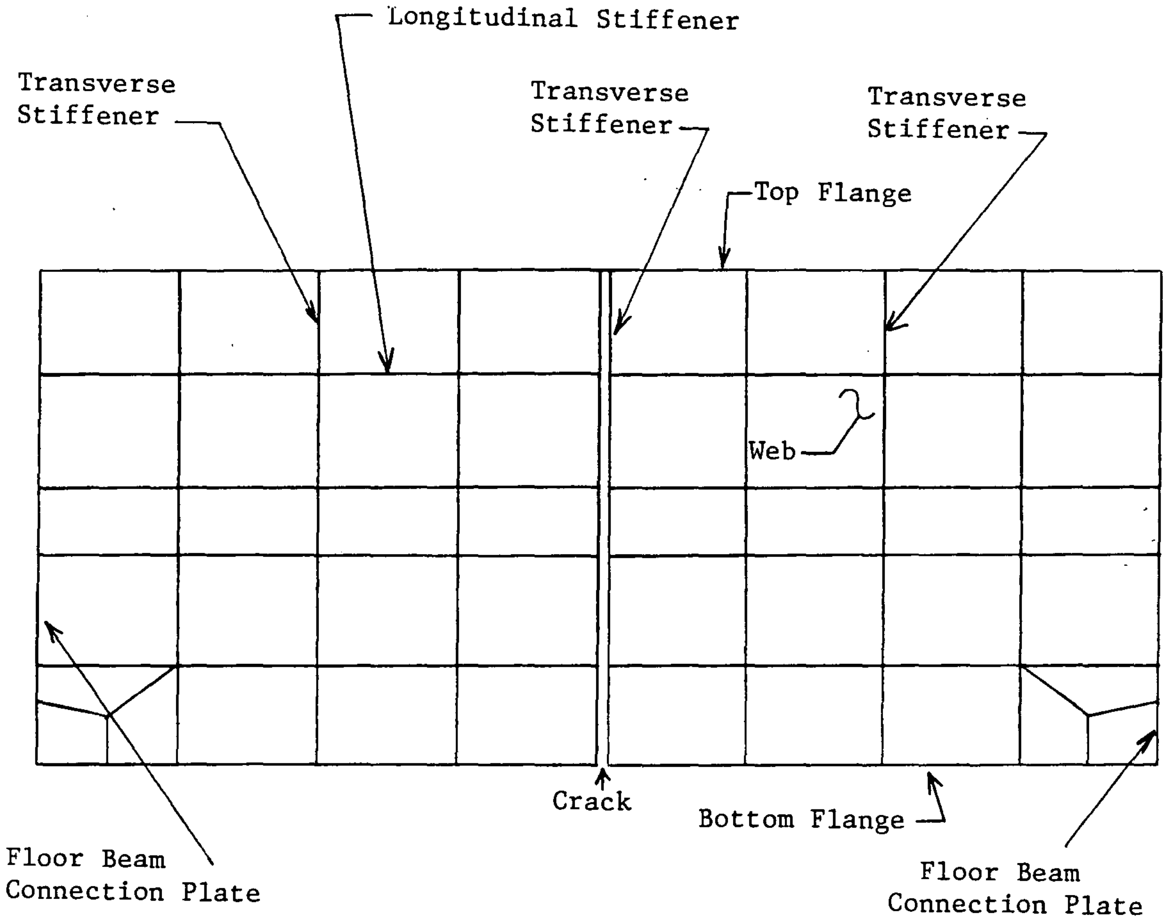


Fig. 7 Finite Element Discretization of Cracked Girder in Bay 3 (See Fig. 5 for Bay Numbering)

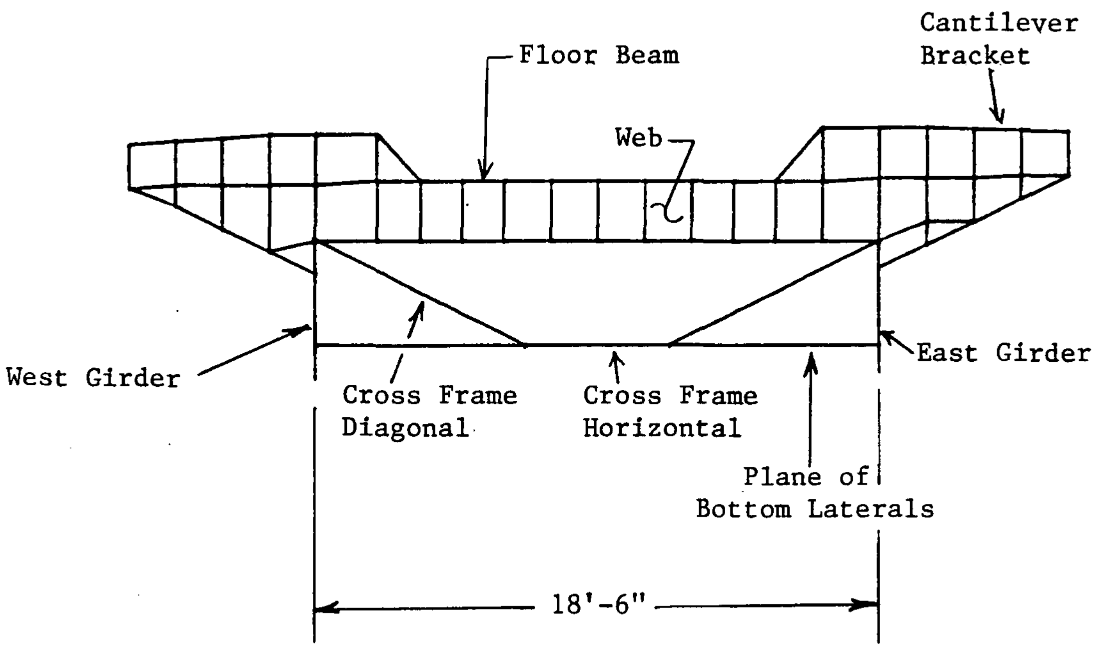


Fig. 8 Finite Element Discretization of a Cross Section at a Floor Beam Location

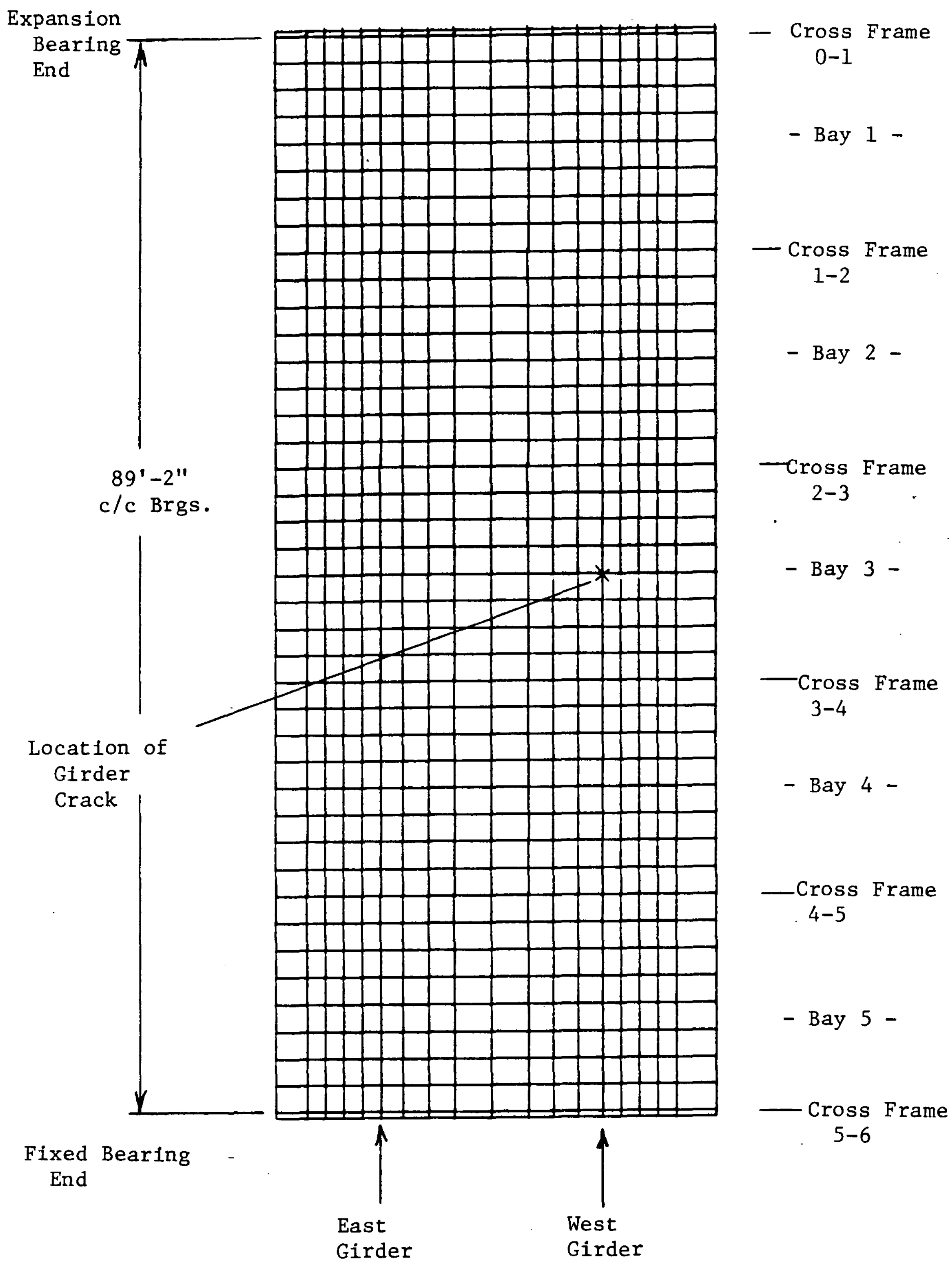
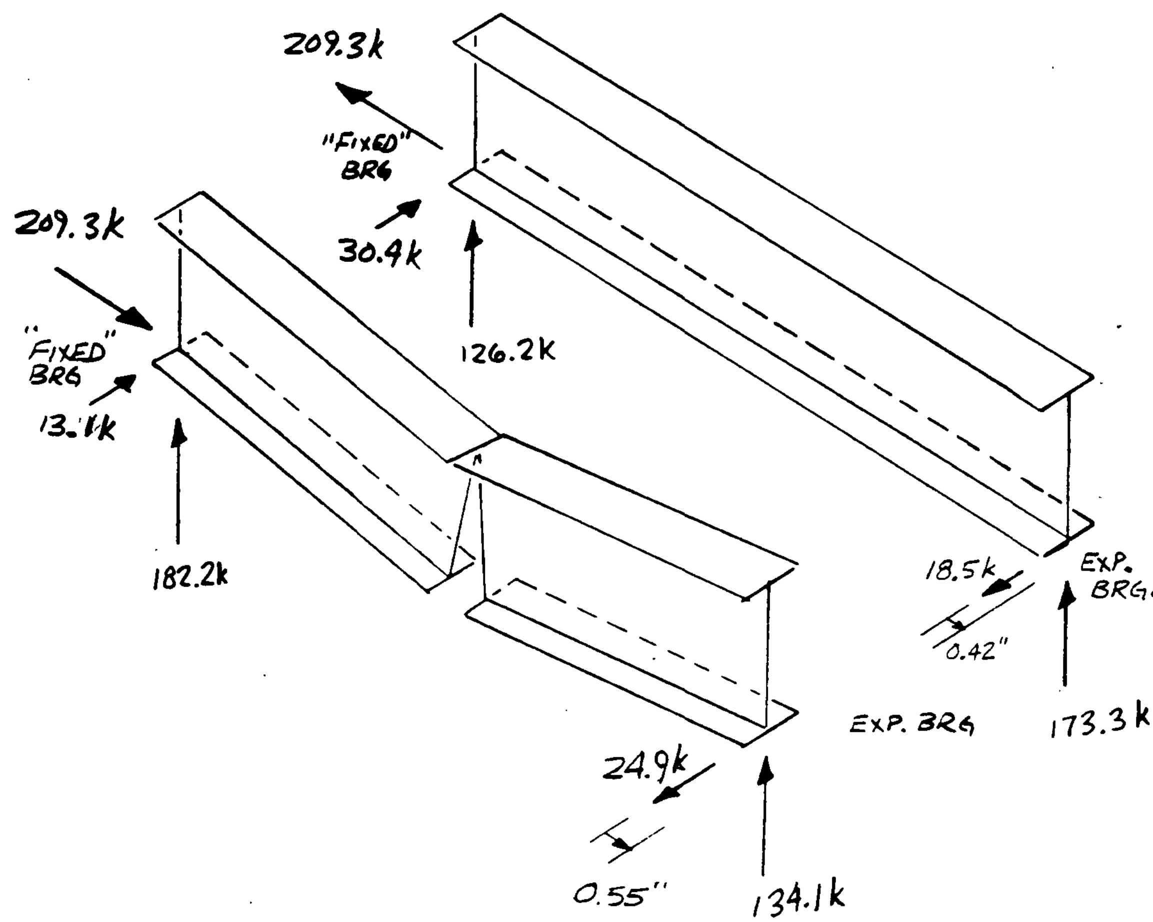
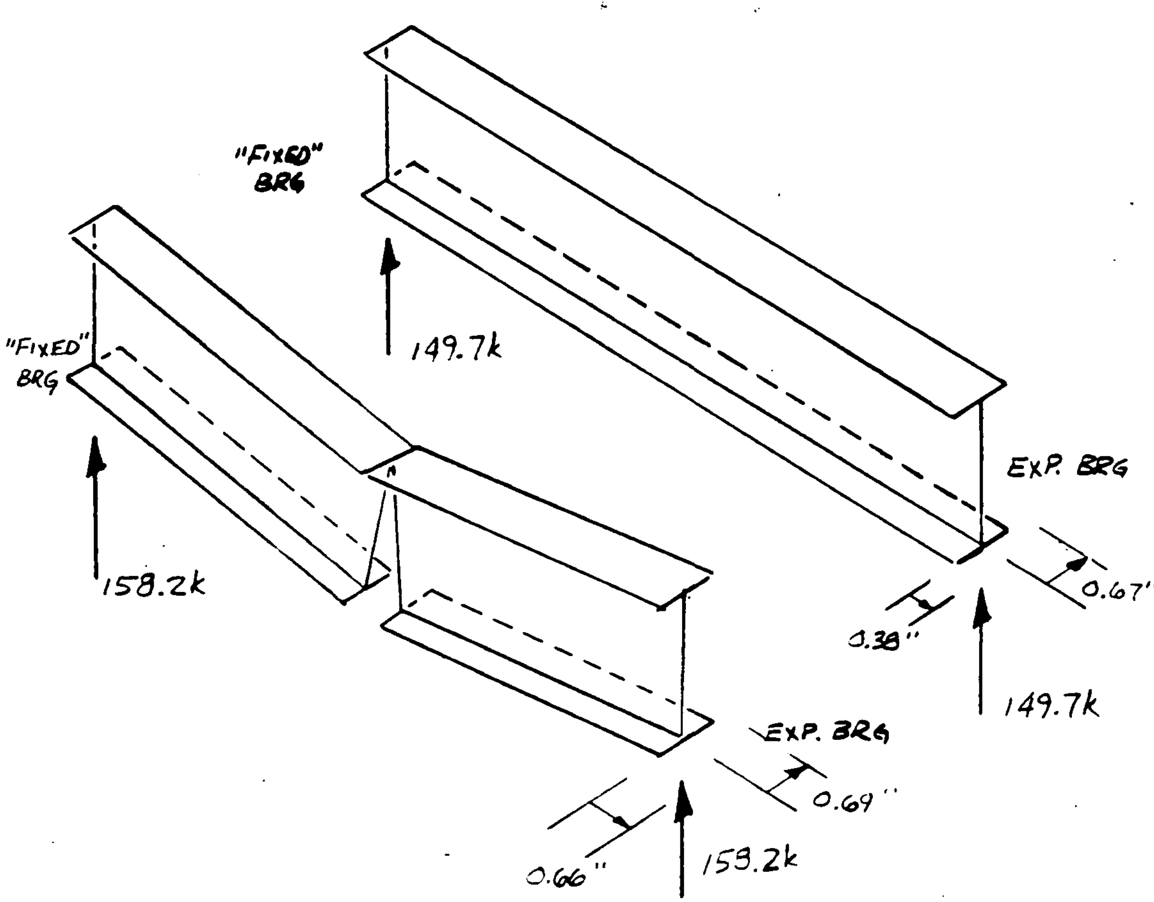


Fig. 9 Finite Element Discretization of Bridge Deck



a)

Fig. 10 Support Modeling Assumptions and Results



b)

Fig. 10 (Continued)

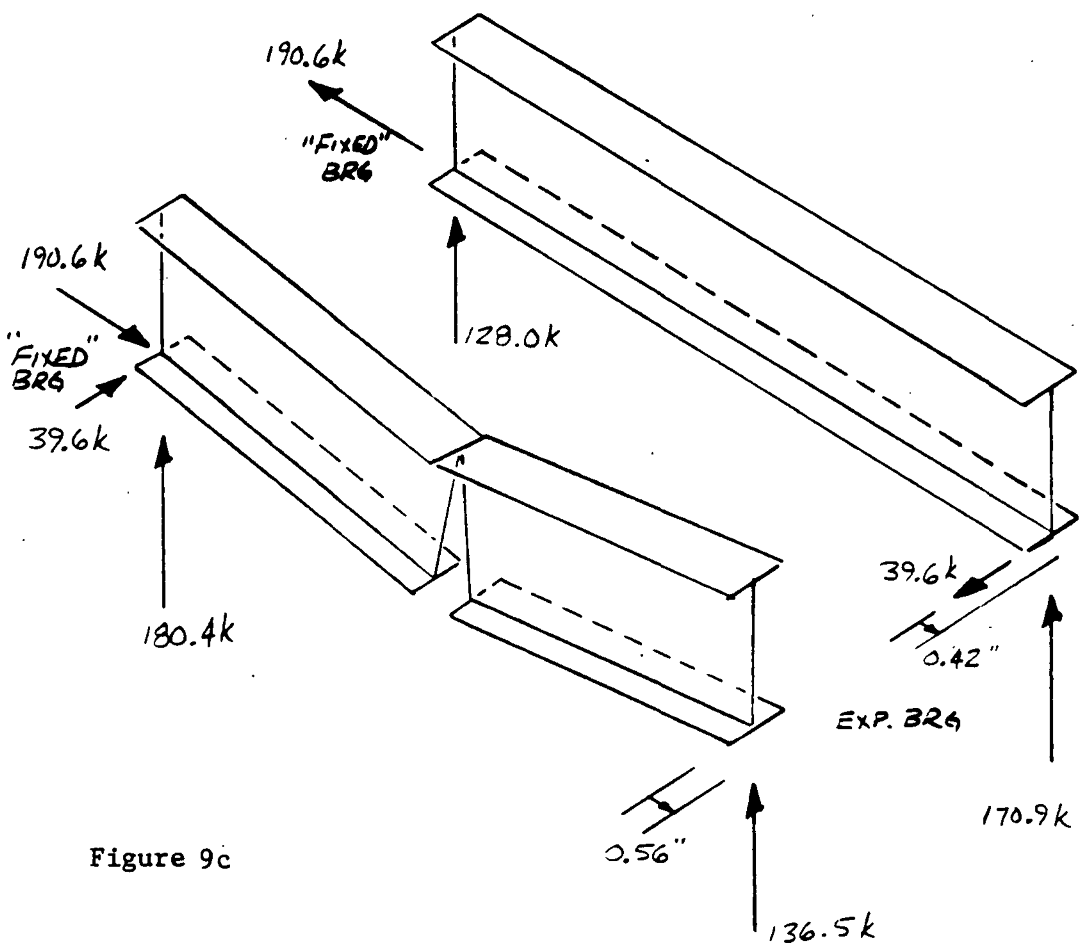


Figure 9c

c)

Fig. 10 (Continued)

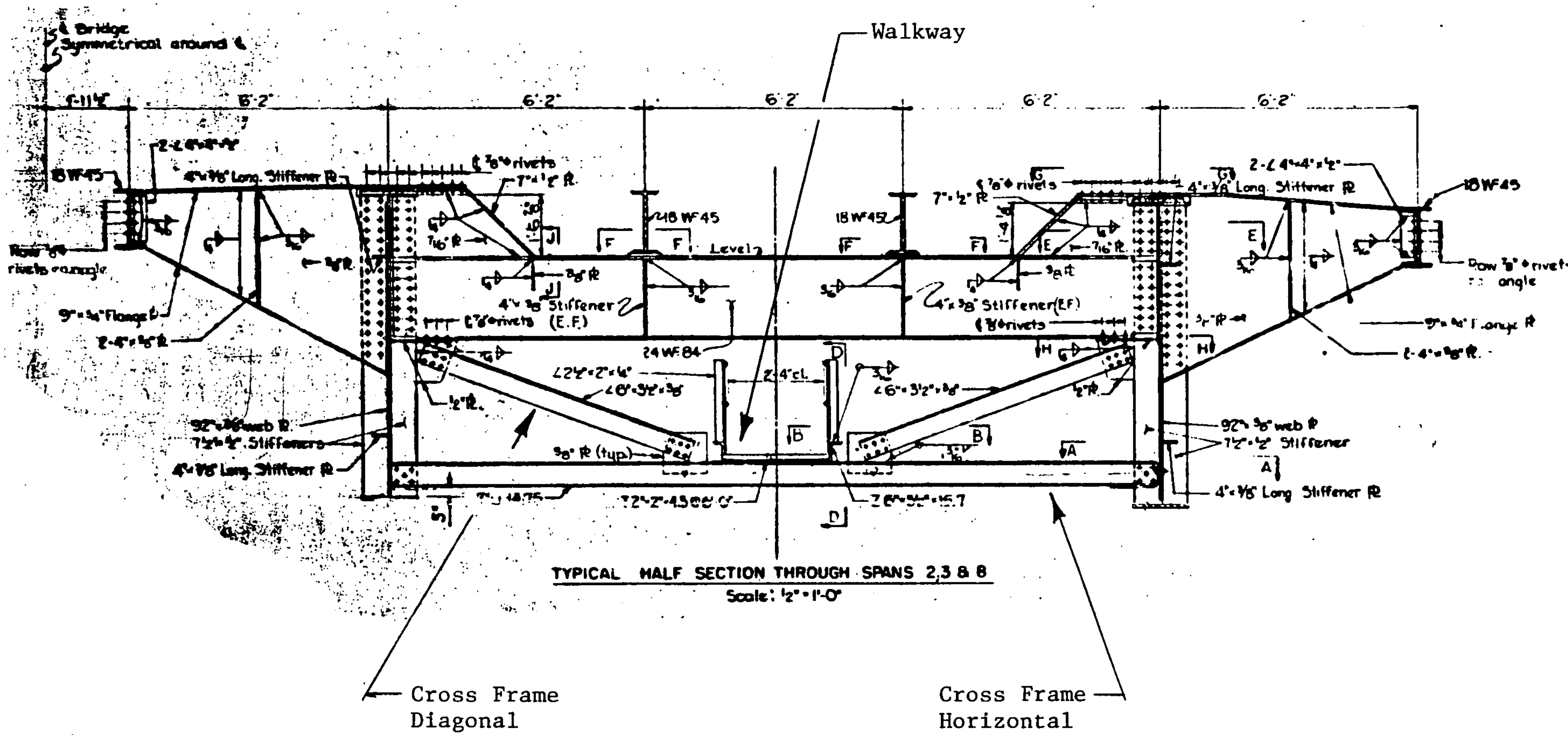


Fig. 11 Typical Cross Frame

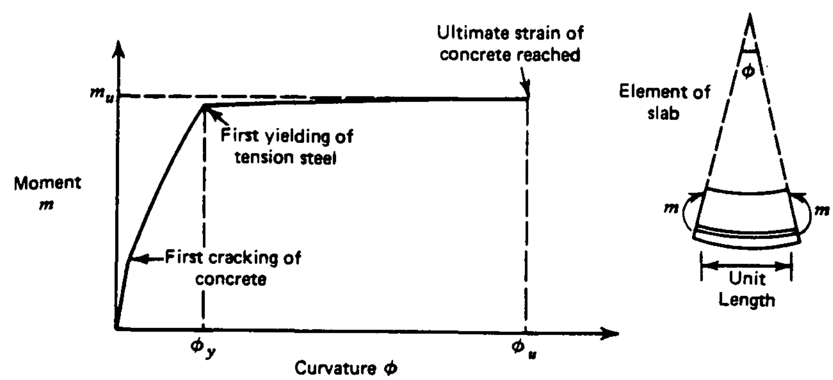


Fig. 12 Moment-Curvature Relationship of Reinforced Concrete Element

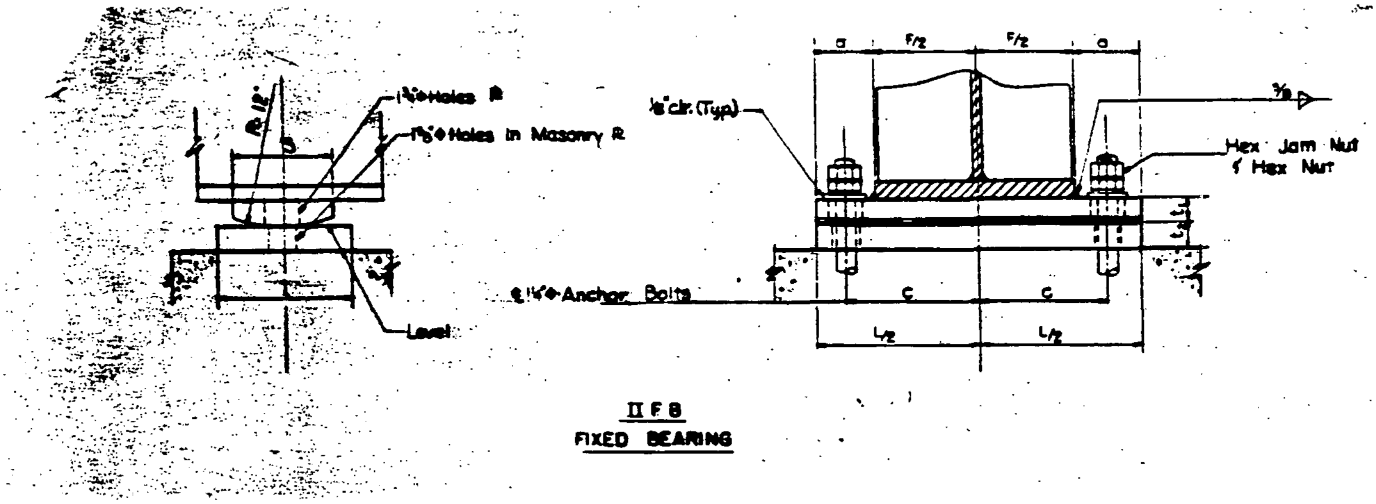


Fig. 13 Fixed Bearing Detail

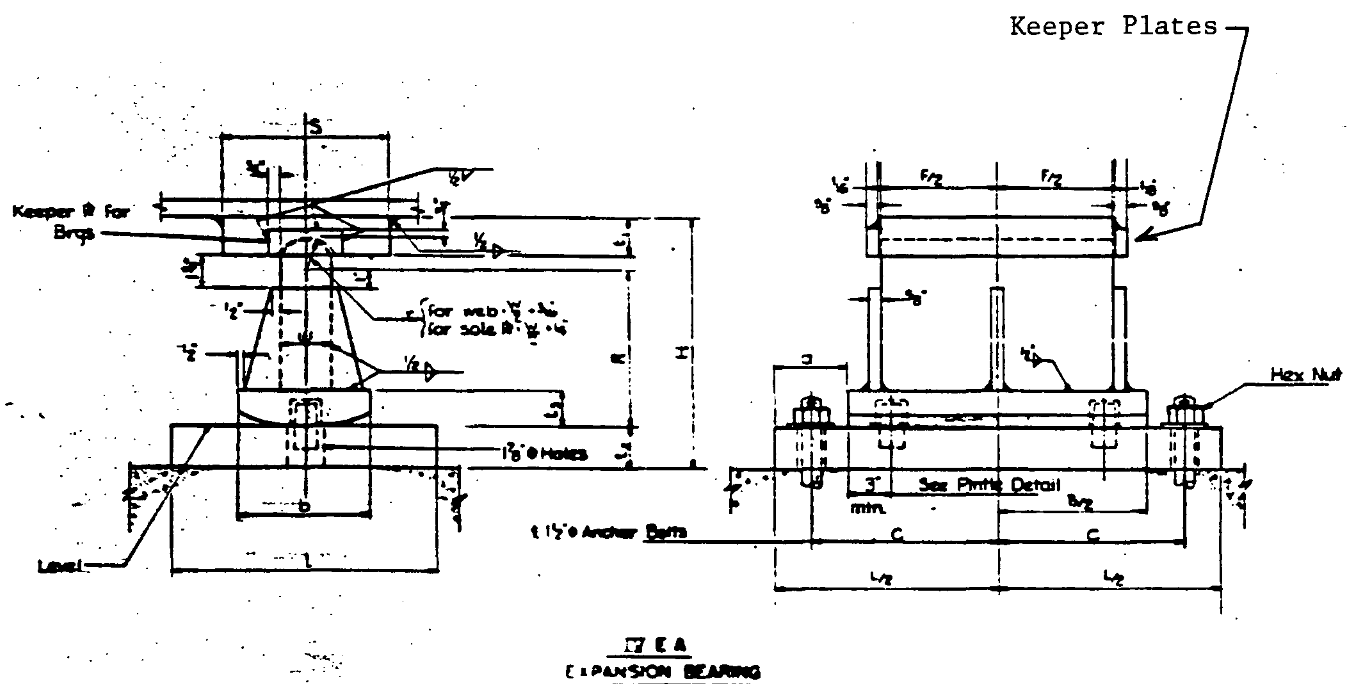


Fig. 14 Expansion Bearing Detail Showing the Keeper Plates

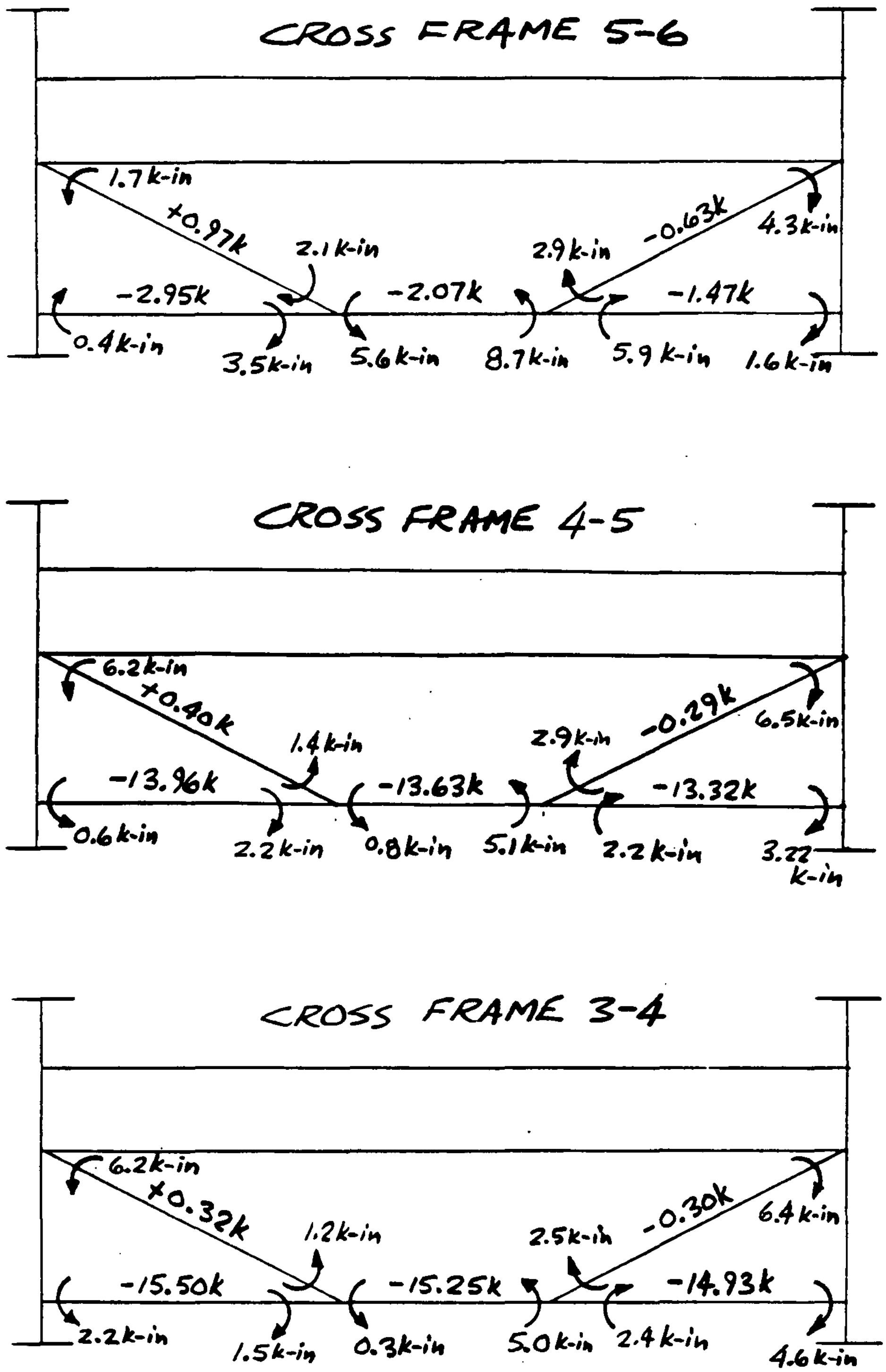


Fig. 15 Cross Frame Forces in Undamaged Bridge Model

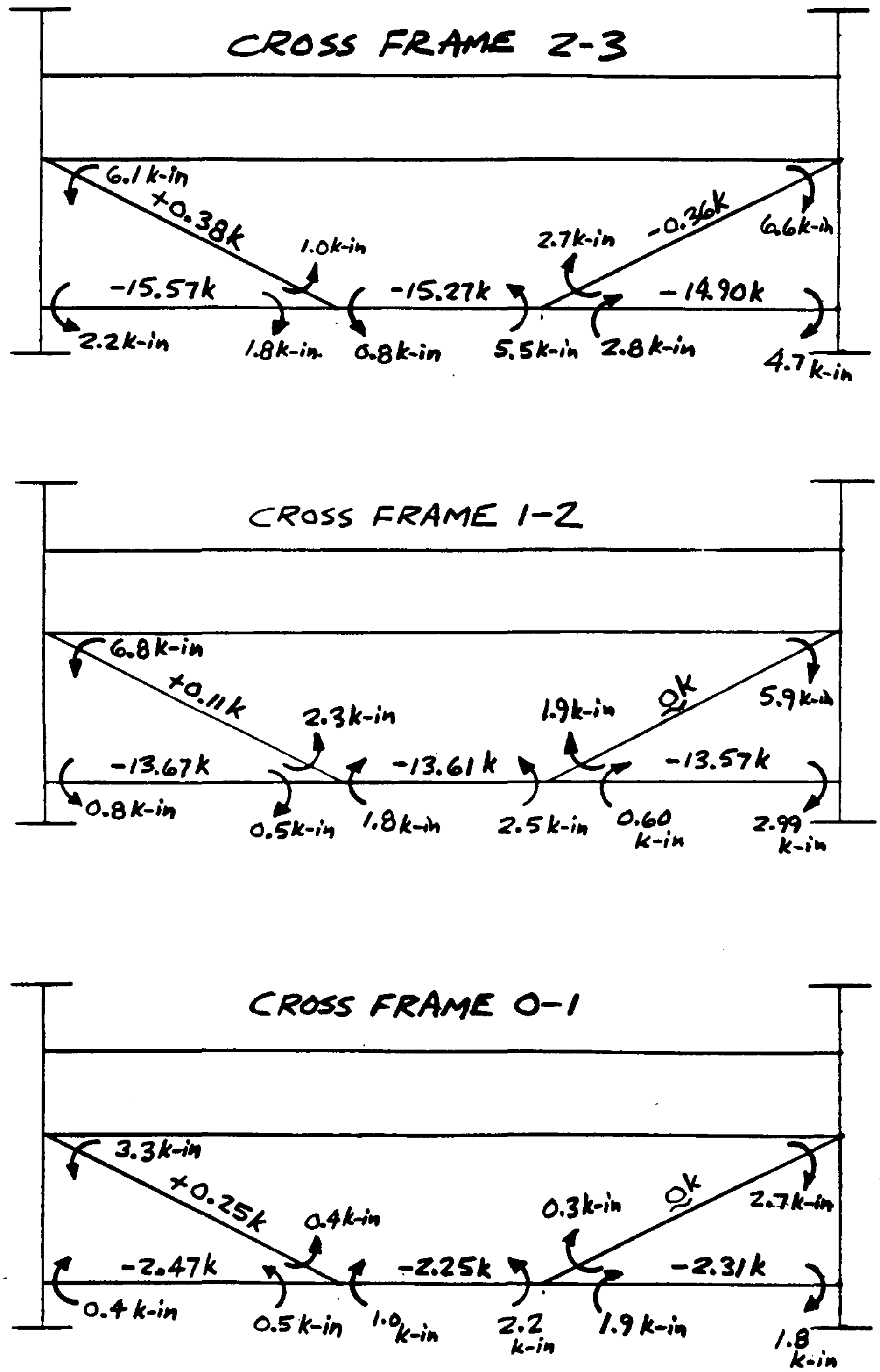
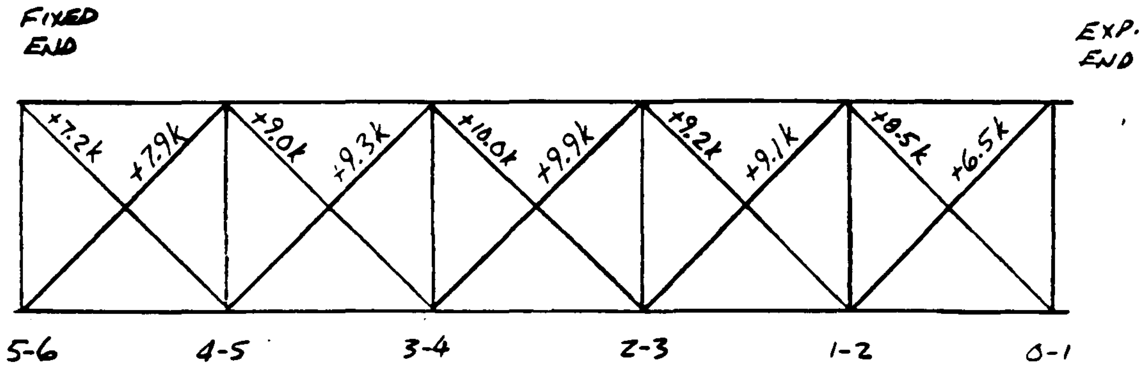
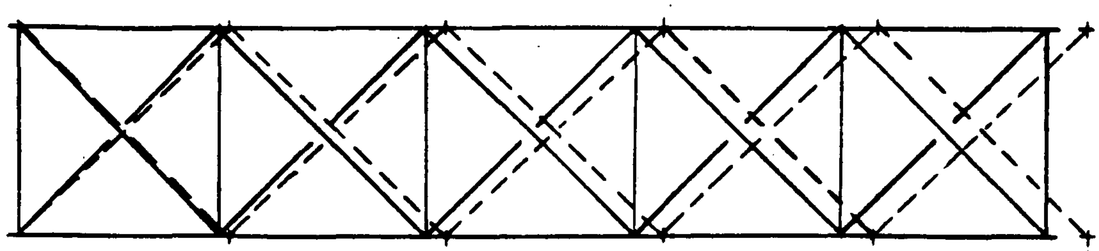


Fig. 15 Cross Frame Forces in Undamaged Bridge Model (Continued)



a) Forces in Bottom Lateral Members



b) Lateral Deflections (x200)

Fig. 16 Forces and Deflections in Bottom Laterals of Undamaged Bridge Model

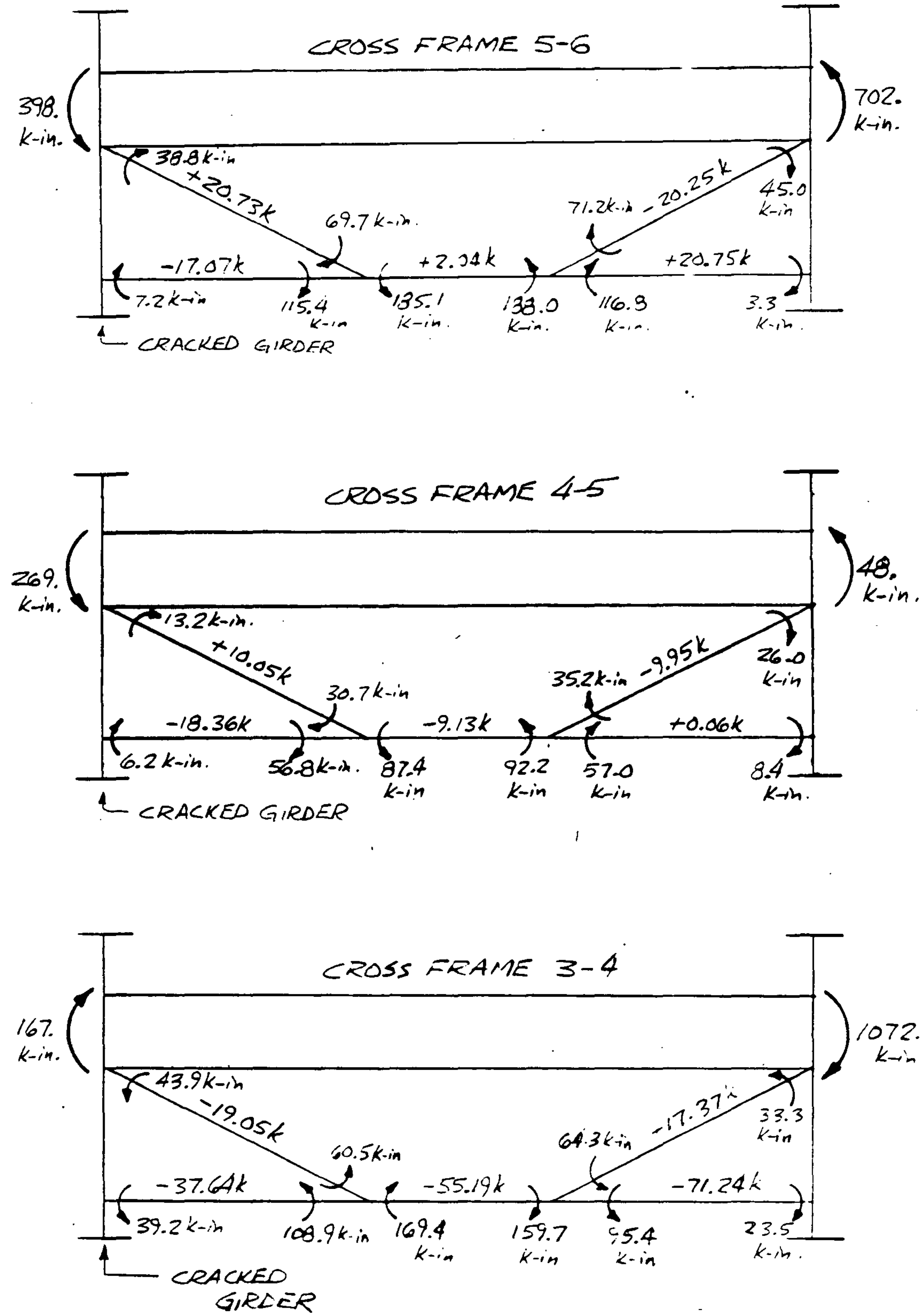


Fig. 17 Cross Frame Forces in Damaged Bridge Model

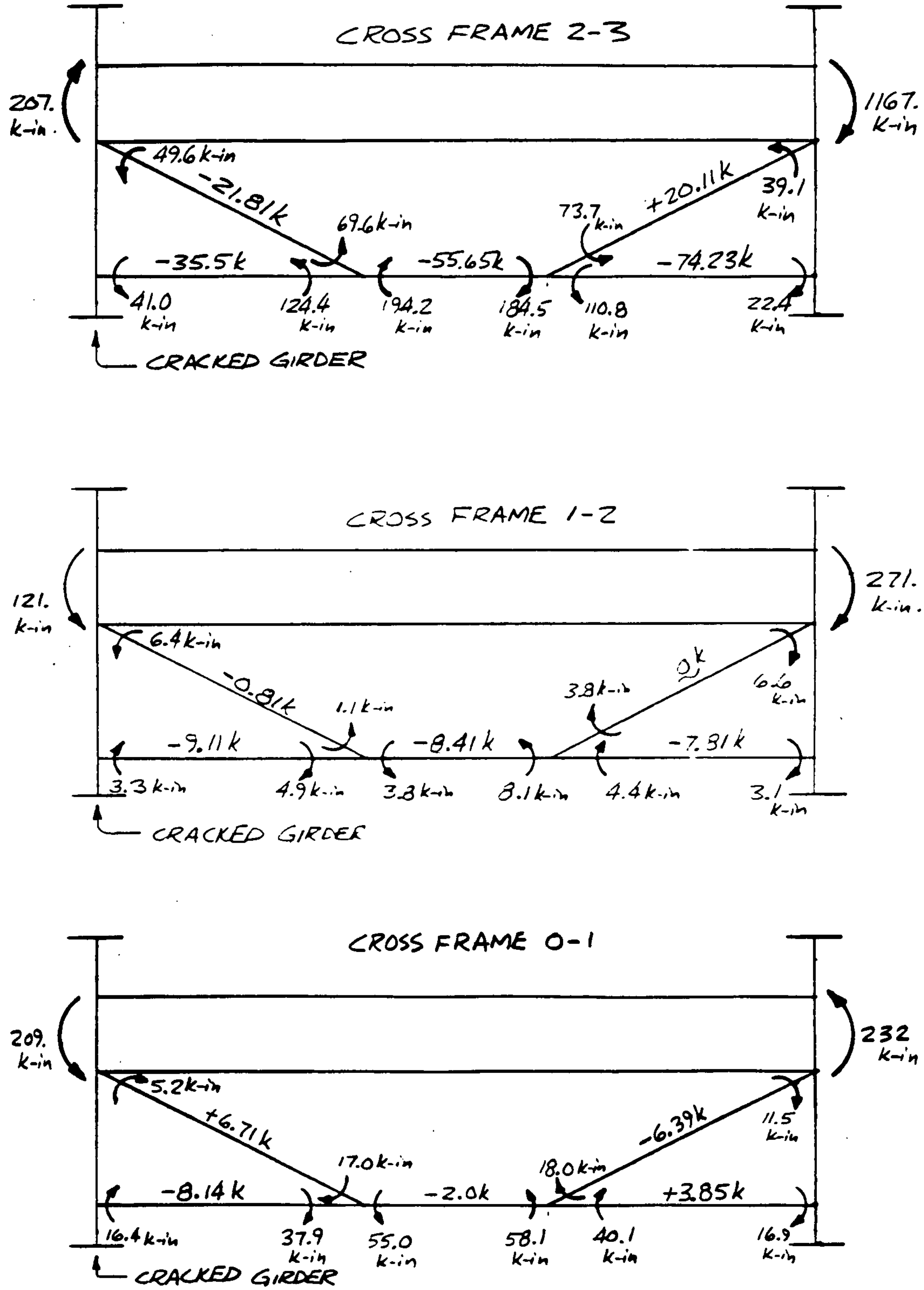
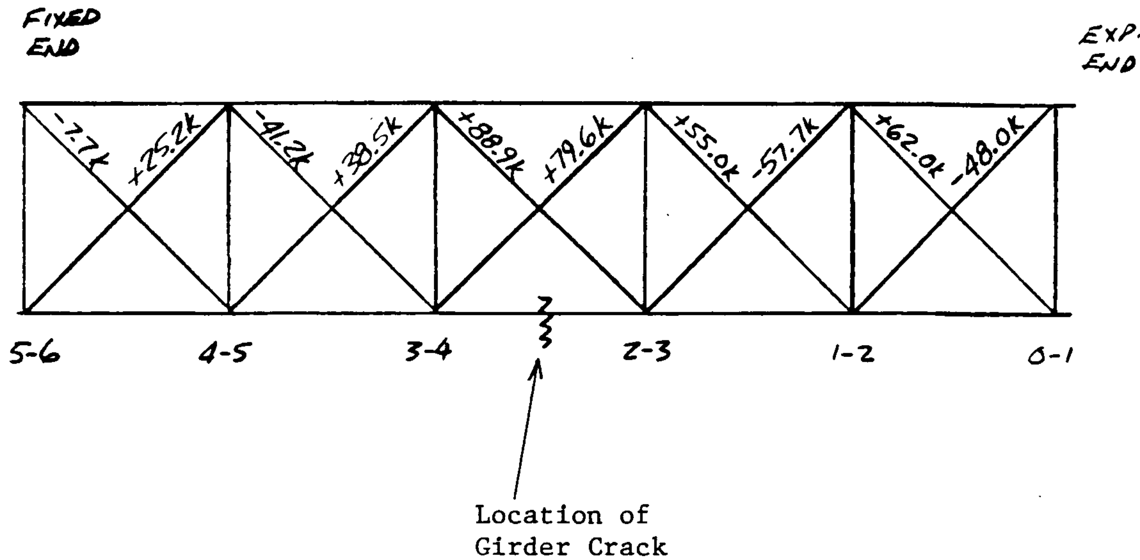
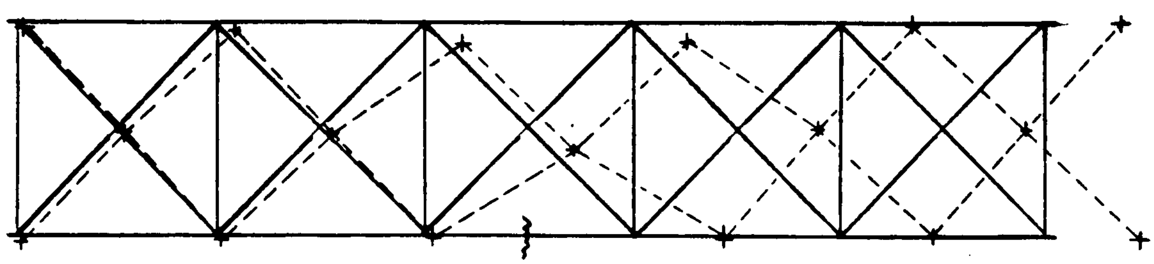


Fig. 17 (Continued)



a) Forces in Bottom Lateral Members



b) Lateral Deflections (x200)

Fig. 18 Forces and Deflections in Bottom Laterals of Damaged Bridge Model

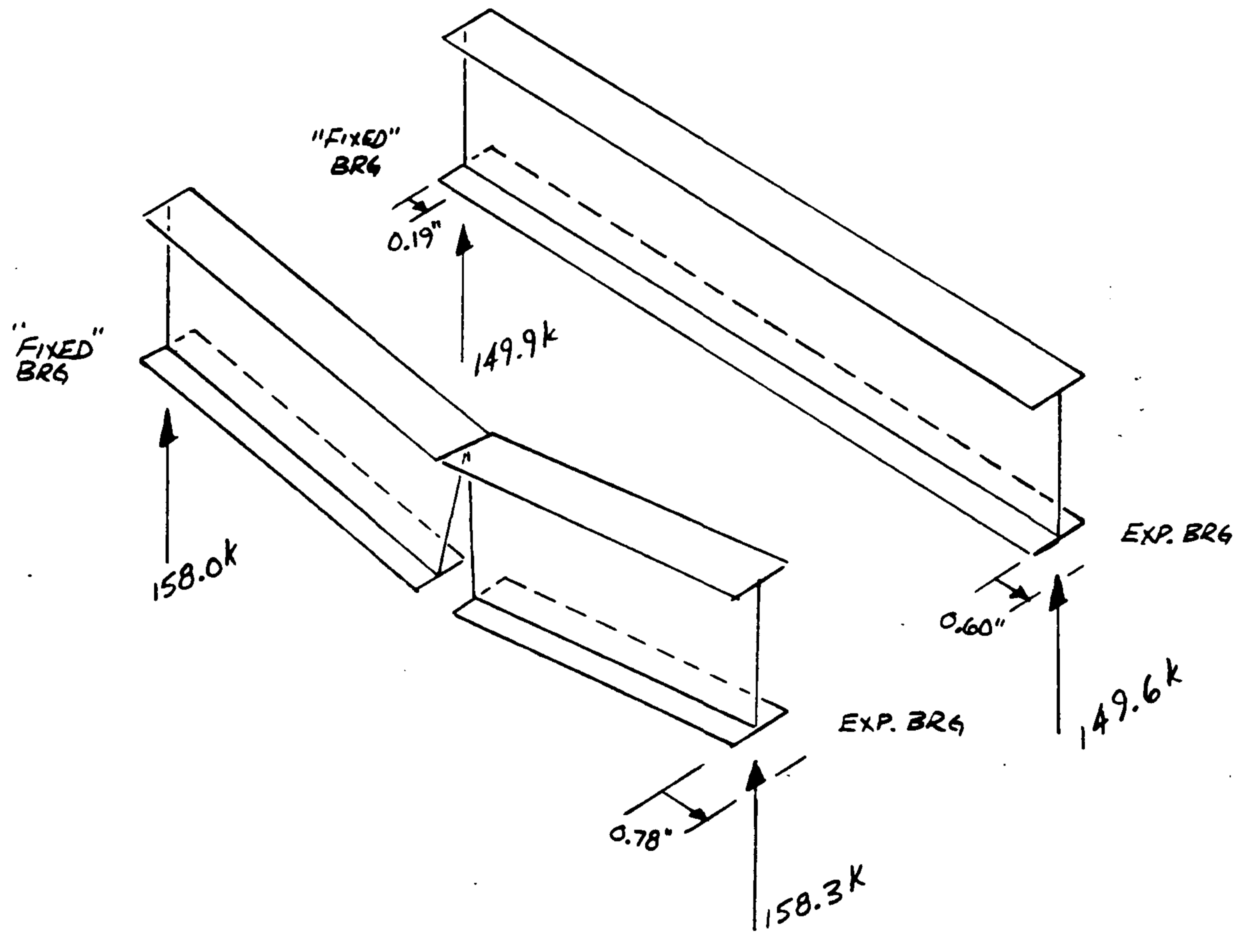
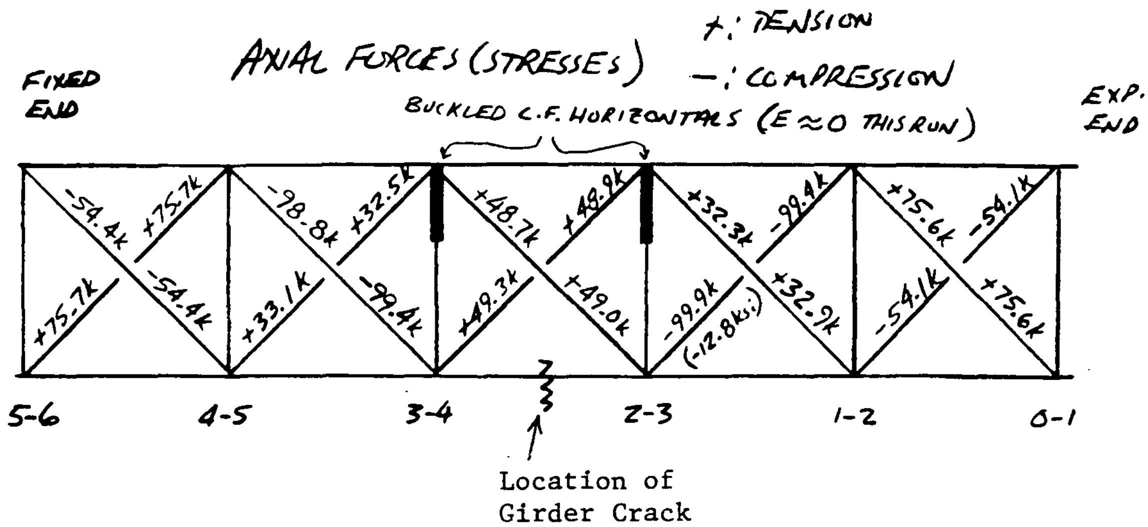
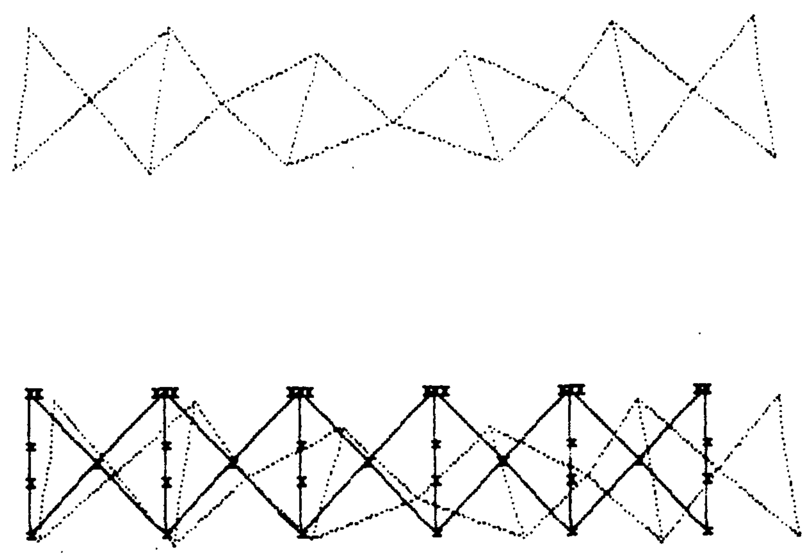


Fig. 19 Reactions and Support Displacements
After Step 4 (see Table 4)



a) Forces in Bottom Lateral Members



b) Lateral Deflections (x200)

Fig. 20 Forces and Deflections in Bottom Laterals in Damaged Bridge Model After Step 4 (see Table 4)

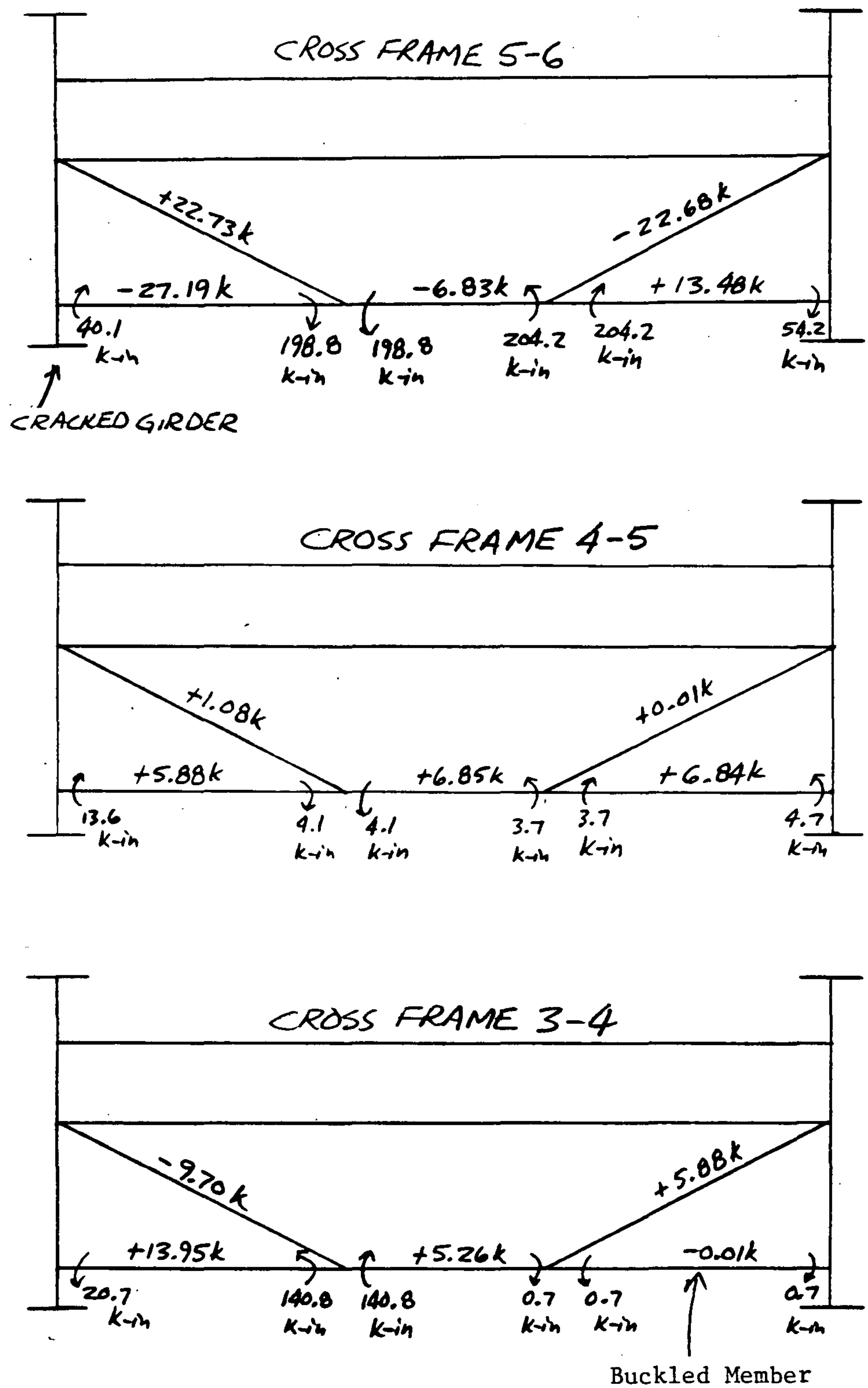
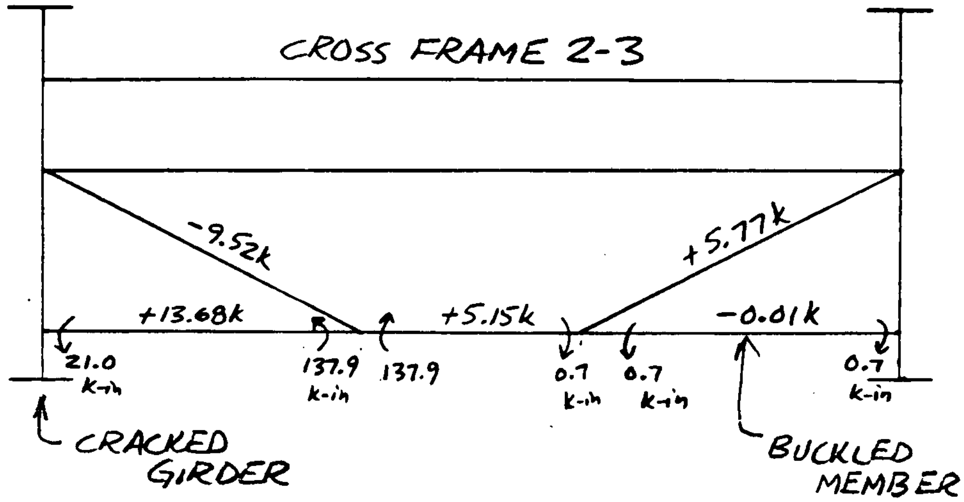
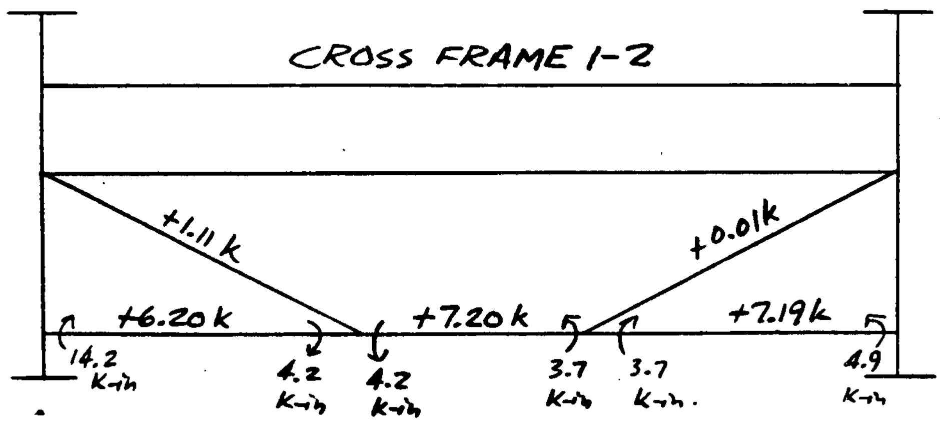


Fig. 21 Cross Frame Forces After Step 4





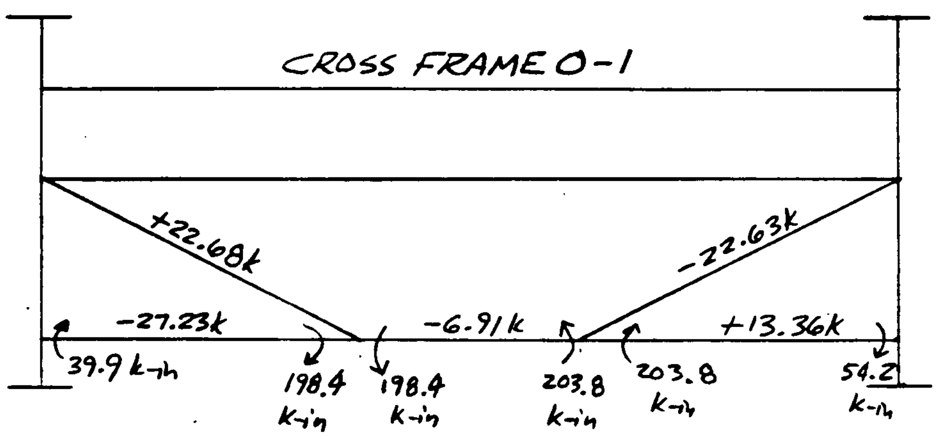


Fig. 21 Cross Frame Forces After Step 4 (Continued)

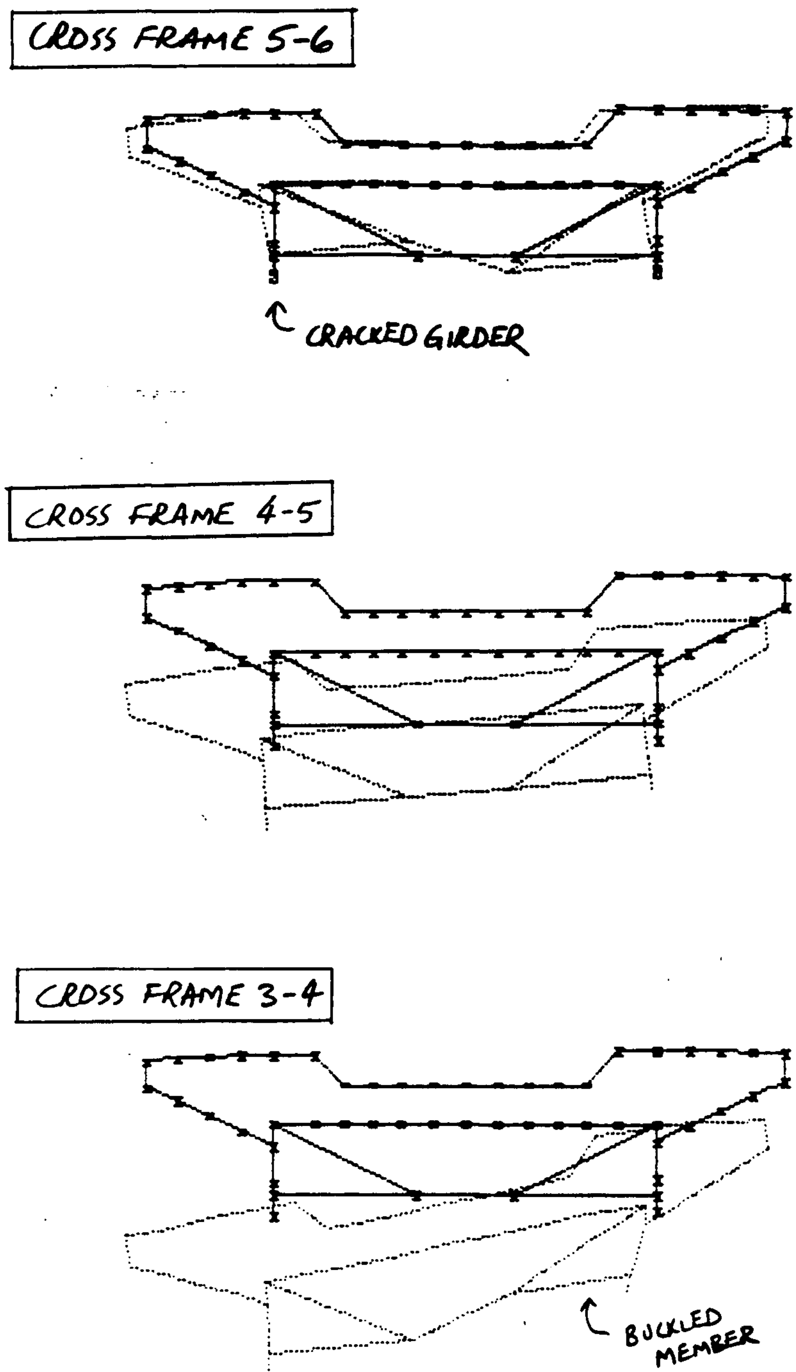


Fig. 22 Deflections in Cross Frames After Step 4 (x 50)

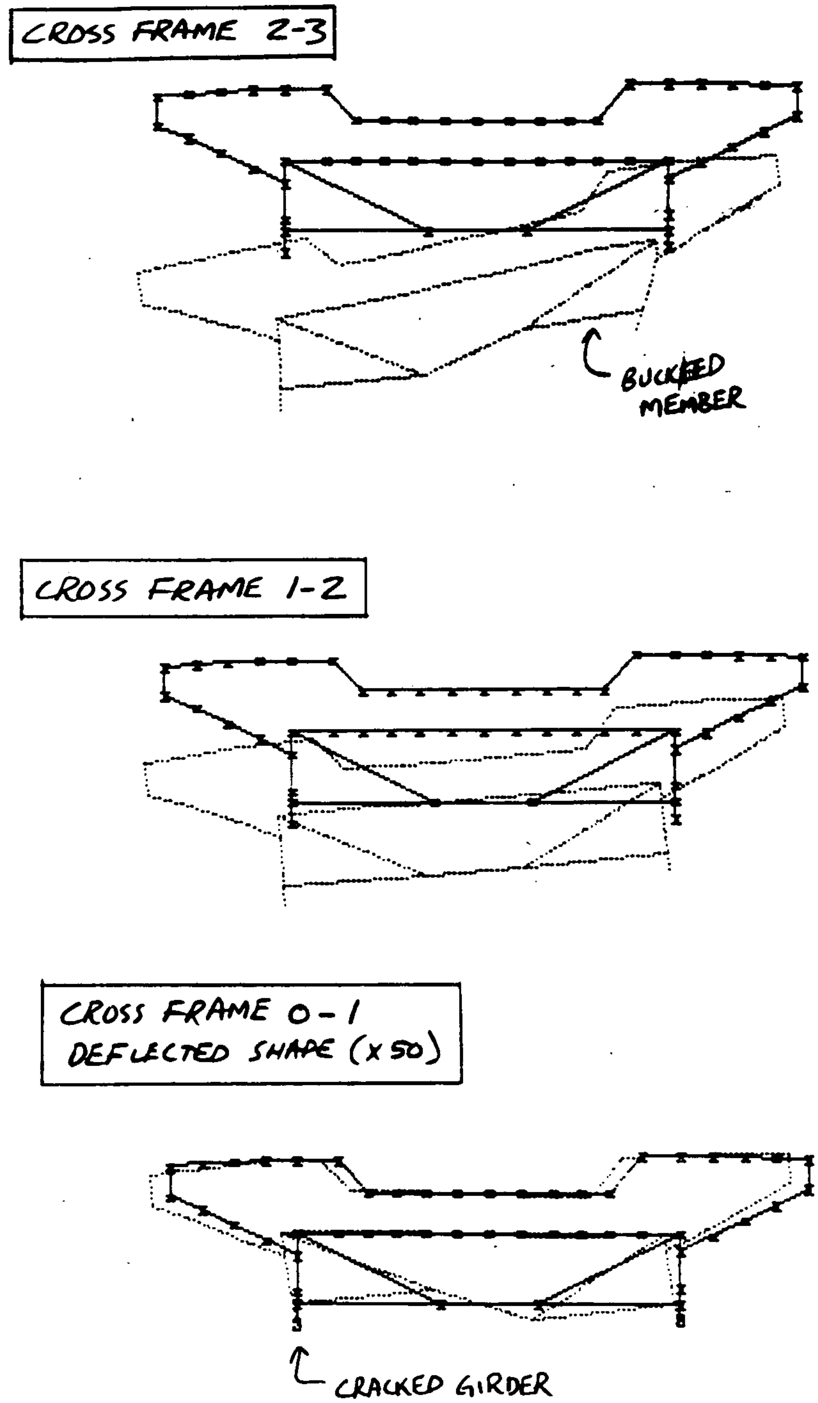


Fig. 22 Deflections in Cross Frames After Step 4 (Continued) (x 50)

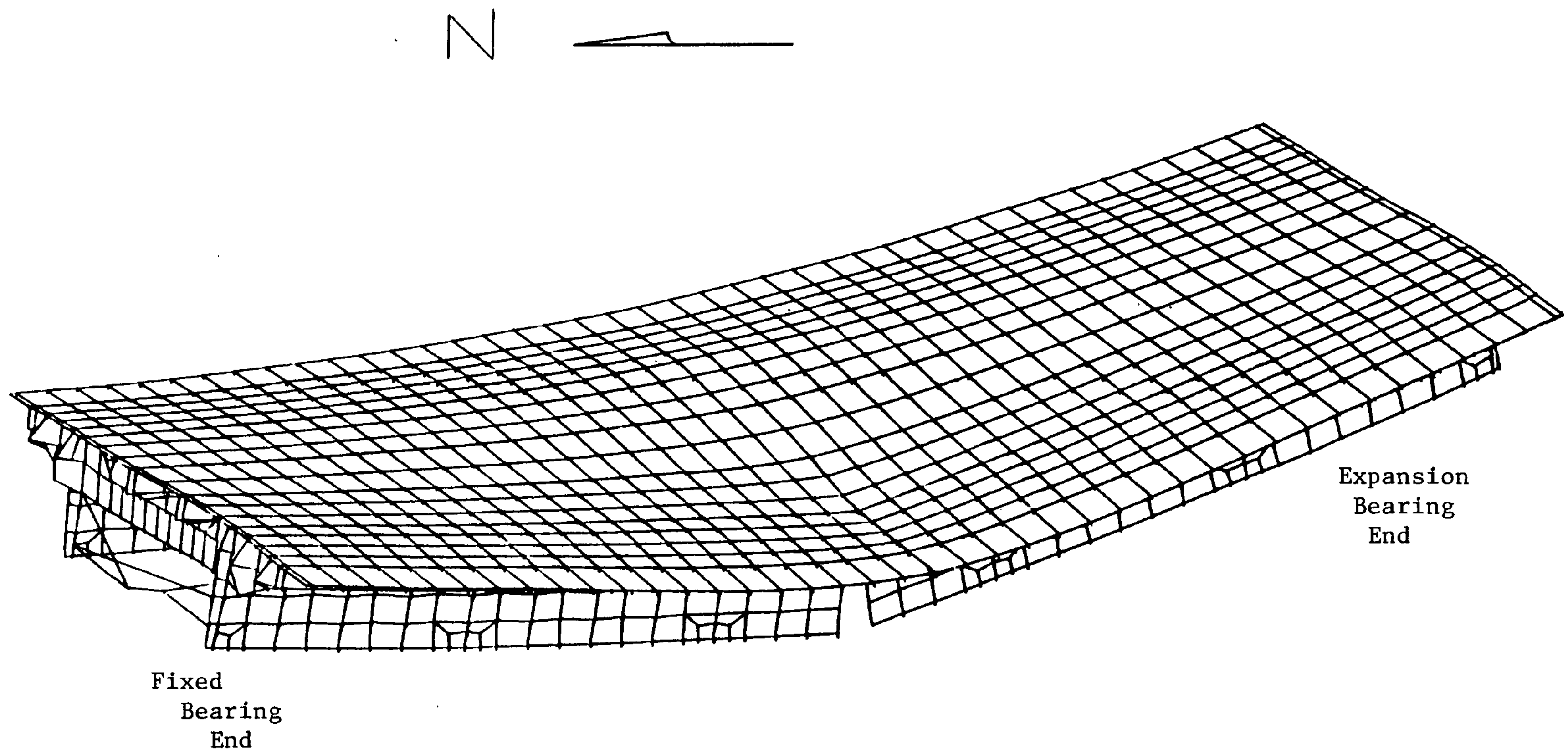


Fig. 23 Deflected Shape of the Simple-Span Bridge Model After Step 4
(x 50)

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