EXPANSION JOINT ELIMINATION FOR STEEL HIGHWAY BRIDGES

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16. Abstract

This study evaluates several popular techniques for the elimination of expansion joints on existing steel highway bridges. Based on a literature search and survey of transportation agencies five specific connection details are examined. These include a deck only connection, a deck and top flange connection, a deck and bottom flange connection, a deck, top and bottom flange connection, and a full moment splice. A sixth control case, no joint connection, was added to the analysis to gauge the previous five connection schemes.

Four typical Rhode Island bridges are used to evaluate the performance of these connection details. In addition, a parametric study of typical bridges with various span lengths is undertaken to evaluate the range of applicability of the connection details. The stress at the deck's top mat reinforcement as well as the crack width are calculated along with any increase in the load carrying capacity of the bridge.

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EXECUTIVE SUMMARY

The present study looks at the elimination of expansion joints in steel girder - slab bridges.

A literature search was completed and a survey of United States and Canadian departments of transportation was compiled. The literature search uncovered very little research on the topic of eliminating joints on existing steel bridges even though much work has been completed for prestressed concrete bridges. The papers found concerning the subject were concentrated mostly on field experience and the general benefits of eliminating joints.

The survey of U.S. and Canadian transportation agencies revealed much about the use of joint elimination details for steel bridges. Sixty-four agencies were surveyed of which twenty-two are involved with the elimination of joints on steel bridges. Well over 500 steel bridges have been made continuous or semi-continuous. Several agencies have connected five and six spans together at once.

The most popular method of joint connection among the agencies is Deck Only. It was reported to be just as effective as any other method used while being the most cost effective. Several responding agencies mentioned that they retard the concrete or pour the deck joint section last in order to eliminate the dead load stresses from the joint connection.

Four Rhode Island bridges were analyzed. These bridges included: Garden Street Bridge #547, Pine Street Bridge #548, Hartford Avenue - West Bridge #608, and Broad Street - South Bridge #657. Bridge properties are included in Appendix C. Slab

reinforcement included top and bottom mats of 60 ksi #4 rebar with 2 in concrete cover. Connection details found promising from the literature search and survey were used for the analysis. These connection schemes include: a) Deck Only, b) Deck and Top Flange, c) Deck and Bottom Flange, d) Deck, Top and Bottom Flange, and e) Full Moment Splice. A sixth control case, No Joint connection, was added to the analysis to gauge the previous five connection schemes. The stress at the bridge deck's top mat of reinforcing and the crack width were recorded along with any increase in the load carrying capacity of the bridge.

Garden Street Bridge #547's top reinforcing mat was stressed up to 21 ksi for the Deck and Bottom Flange connection. The cracking potential was greatest for the Deck Only and the Deck and Top Flange connection schemes.

Pine Street Bridge #548's top reinforcing mat was stressed up to 22 ksi for the Deck and Bottom Flange connection. Again, the greatest potential for cracking was for the Deck Only and the Deck and Top Flange connection schemes.

Hartford Avenue - West Bridge #608's top reinforcing mat was stressed up to 44 ksi for the Deck and Bottom Flange connection. The Deck Only and the Deck and Top Flange connection schemes showed the greatest potential for cracking.

Broad Street - South Bridge #657's top reinforcing mat maximum stress was 16 ksi for the Deck and Bottom Flange connection. The Deck Only and the Deck and Top Flange exhibited the greatest potential for cracking.

By comparing the mid-span moments on the loaded span, equivalent truck load ratings were calculated. The Deck Only and the Deck and Top Flange connection schemes did not increase the load carrying capacity of the bridge significantly. However, the remaining connection schemes appeared to increase the load carrying capacity from 13% to 75% with the highest increases observed for the Full Moment Splice connection.

The range of applicability for the connection schemes was also investigated. Realistic bridge models were developed to test span connections from 95' to 35'. Equal spans were connected for 95', 75', and 55' span lengths'. Unequal spans were connected in the following span configurations: 90' - 45', 80' - 40', and 70' - 35'. Properties of these models are included in Appendix C. Slab reinforcement included top and bottom mats with 60 ksi #5 rebar with 2 in concrete cover. Rebar spacing is indicated in Appendix C.

The maximum top mat reinforcing stresses for the equal 95' span structure was 46 and 44 ksi for the Deck and Top Flange and the Deck and Bottom Flange connections. The greatest potential for cracking was for the Deck Only and the Deck and Top Flange connections.

The maximum top mat reinforcing stresses for the equal 75' span structure was 41 and 39 ksi for the Deck and Bottom Flange and the Deck and Top Flange connection. The Deck Only and the Deck and Top Flange connection details greatest the greatest potential for cracking.

The maximum top mat reinforcing stress for the equal 55' span structure was 27 ksi for the Deck and Bottom Flange connection. Again, the greatest crack potential was created by the Deck Only and the Deck and Top Flange connection.

For the unequal span structures the greatest top mat reinforcing stress levels came from the Deck and Top Flange connection detail, 40, 35, and 32 ksi, respectively. The greatest cracking potential came from the Deck Only and the Deck and Top Flange connection schemes.

The Deck Only and the Deck and Top Flange connection schemes did not significantly increase the load carrying capacity of the structures. However, the remaining schemes did increase the load carrying capacity from 16% to 82%. The Full Moment Splice created the greatest potential in this regard.

The Deck Only connection scheme appears to be the most popular, most cost efficient, and easiest to construct. However, it does not improve the load carrying capacity of the structure and it has the most potential for deck cracking. Based on the results of this study it can be applied to bridges with a span up to 55 ft without causing excessive stresses in the reinforcement or substantial cracking over the joint. The potential for deck cracking may be minimized by installing elastomeric bearings and providing a sealed control notch in the concrete deck.

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

A bridge structure is subject to thermal expansion and contraction. As a result, a gap must be provided between two adjacent spans or an abutment to allow for movement. In the past, water and debris was allowed to flow freely through the gap. However, as early as 1914, joint systems were designed to seal this gap to keep the structure below dry and free from water and debris.

After World War II, the United States began a highway building program that would eventually cross the nation east to west and north to south. Interstate highways were designed and built at a rapid pace creating thousands of new bridges. Because computers were either unavailable or very expensive, hand analysis of continuous spans using moment distribution, introduced by Hardy Cross was used. This type of analysis is very laborious and time consuming, and most designers opted to design multi-span bridges as simple spans. Expansion joints were installed between adjacent spans over the abutments.

During the early 1950's, the need for safe travel during winter months became apparent and a dry-pavement policy was adopted by many transportation departments. This policy was implemented with deicing chemicals such as salt. The consequences of this decision was not fully understood until the late 1950's and early 1960's when bridges began exhibiting rapid rates of corrosion. This deterioration was especially evident below the bridge joints where not only water and debris fell, but now salt and deicing chemicals collected. A search for waterproof deck joints and better bridge designs began in earnest. About this time, computers started to become more available making the Hardy Cross analysis method more popular, thereby increasing continuous bridge construction. Unfortunately, tens of thousands of multi-simple span steel bridges had already been constructed, each with several joints.

A well constructed, installed, and properly functioning joint is cause for celebration, as one researcher declared. It is logical to conclude, from this and other research that is presented later, that the vast majority of bridge deck joints leak and cause large amounts of infrastructure deterioration at an immense cost to the public. Any reduction in the number of these bridge joints would, therefore, lessen the amount of corrosion to beams, bearings, and concrete piers and abutments. Clearly, this reduction would also lessen the amount of construction traffic delays, cost of bridge repair and replacement, and provide a smoother riding surface.

Burke (1990) stated the following: "Because design and construction of fully continuous bridges have become routine and continuous conversion of simple spans in new construction is becoming more commonplace, it is surprising that similar conversion techniques are not used more often to convert existing jointed bridges to continuous bridges. Presumably, the next decade or two will see a burgeoning in retrofitting simple multiple span bridges to continuous bridges and from non-integral to integral abutments. When more information on the operating stress levels of integral bridges has been developed and when more fully described design details and procedures for integral conversions have become available, bridge engineers will be able to more fully justify their consideration of such construction. Until then, much intuition and prudent judgement will continue to be used to ensure that integral construction and conversion techniques will provide the service life needed to justify their adoption and continued use."

Jointless integral bridges, those newly constructed bridges designed without any joints or joints only at the abutments, have received much attention (Burke, 1993). On the other hand, retrofitting existing bridges with continuous deck details has received little notice and research. This research is intended to answer some of the questions concerning the operating stress levels of bridges retrofitted with a joint elimination scheme.

1.2 LITERATURE REVIEW

Most of the studies on expansion joint elimination relate to new integral bridges. The conversion of existing multi-simple span bridges to continuous or "semi"-continuous has been less studied. Such conversion began in Wisconsin and Massachusetts in the 1960's. Europe and Japan are also involved with the conversion of existing bridges. This rehabilitation procedure has reduced maintenance, improved riding quality, lowered impact loads, reduced snow plow damage, and improved seismic resistance.

In 1990, a FHWA Technical Advisory (Leathers, 1990) concerning the rehabilitation and retrofitting of bridge deck expansion joints was distributed. According to the Advisory for joint elimination, provisions should be made for changes in movement, simple spans should be grouped into a continuous unit, elastomeric bearings should replace obsolete or deteriorated bearings, and a fixed integral condition should be investigated.

Fujiwara (1992), discusses Japan's successes in eliminating expansion joints on existing bridges. For small displacement joints, the "Buried Type No Joint Method" is used to absorb or disperse the displacement. This method is generally used for short spans of 100 ft (30m) or less that are made of concrete or prestressed concrete having light traffic conditions. Flexible rubberized asphalt is used to absorb the displacement across the joint. A more successful system uses shear plates installed between the deck and surfacing with reinforcement materials within the surfacing material.

A second method called the "Connection Type No Joint Method" is also used in Japan. This method can be applied to steel bridges, longer span concrete bridges, and heavily traveled routes that require durability. This method is classified into two categories. The first restrains expansion/contraction and rotation at the girder ends by using a full moment splice and using elastic supports to reduce local stresses and to disperse horizontal earthquake forces. A second connects just the deck and, in some cases, the upper flange, allowing the girder ends to rotate to a certain degree. For steel bridges, the decks and upper flanges are connected. For prestressed concrete girders, only the decks are connected. Bearings are changed to elastomeric for the same reason given above. A bridge must satisfy the following criteria to be considered: girders must be in line, must have consecutive simple spans, and stiffness of piers and abutments are relatively the same.

Most of the studies on expansion joint elimination relate to reinforced and particularly prestressed concrete bridges (Oesterle et. al., 1989; Hambly and Nicholson, 1990). Little has been written about the conversion of existing multi-simple span steel bridges to a continuous or semi-continuous structure, except to comment on their field performance and obvious benefits. One item of interest is the use of concrete retarding agents for deck and continuous joint placement. This allows dead loads (ie. concrete deck) and superimposed dead loads (ie. sidewalks and parapets) to be resisted by the existing composite steel beams only. The continuous deck joint is then constructed and cured so that it resists only the live loads imposed by vehicular traffic (Massoni and Bacho, 1993).

Recently, the Connecticut DOT eliminated expansion joints on a bridge consisting of three short simple steel spans. Bottom flanges of opposing beams were welded together, the deck was made continuous, and the bearings were replaced to allow for the new configuration. Analysis of the partial continuity over the piers indicated that the deck and welded bottom flange were incapable of handling the stresses developed by the dead load and live load together. Therefore, a design and construction sequence was developed providing continuity for only live load and minor superimposed dead load. Once ConnDOT's bridge deck was cured, traffic was routed over the bare deck before a bituminous overlay was added. After several months of monitoring, cracks opening to a maximum of ten-thousandths of an inch running parallel to the piers were observed in the bare deck. This type of cracking was considered normal with respect to the stress in the slab reinforcements, and problems are not expected to develop.

Some agencies limit the use of continuity to those bridges whose girders are in line, have consecutive simple spans, and whose piers and abutments have relatively the same stiffness. Replacing existing steel bearings with elastomeric bearings is recommended to reduce the local stresses and to disperse horizontal earthquake forces. The bearings should also be reconfigured to allow the new continuous structure to move along the entire length in reaction to thermal forces.

A recently popular bridge joint rehabilitation method consists of replacing the expansion joint mechanism with a flexible asphaltic plug. Kauffman *et. al.* (1990), discusses the Thorma-Joint flexible bridge expansion joint system that is composed of a rubber asphalt binder and stone aggregate. In July 1988, it was experimentally installed on six bridges in New Jersey. A two month post-installation inspection found the joints to be in fair to average condition with no signs of cracking or water leakage. However, some joints were rutting, shoving, and delaminated. This could have been attributed to the 20 days over 90 degrees immediately after installation. At a one year inspection, the overall condition had improved slightly due to compaction under traffic. This system is not exactly joint elimination but appears to be a promising technique for joint rehabilitation of short span bridges.

Though the design and construction of fully continuous bridges and the conversion of newly constructed concrete structures are routine, conversion of existing steel jointed bridges is uncommon. The conversion techniques for steel bridges have been based mostly on intuition and prudent judgement. To provide extended service life and to justify continuity conversion, information on the operating stress levels, the development of design details, and procedures for conversion are needed. A comparison of the relative efficiency of various joint conversion schemes is presented in this work.

1.3 SCOPE

The scope of this research is to investigate the elimination of expansion deck joints on existing concrete decked, multi-simple span steel bridges. The investigation concentrates on studying the state-of-the-art in retrofitting this type of bridges using a continuous deck joint detail. Several methods of connection are currently being used. The most common continuity schemes are as follows: a) deck only, b) deck and top flange, c) deck and bottom flange, d) deck, top and bottom flanges, and e) deck and moment splice. Currently, no information or data is available to compare their relative efficiency or the effects that each has on a bridge's components and properties. A sixth "no deck joint" scheme is included in the analysis to gauge the other five schemes, making a total of six schemes studied and analyzed.

Four Rhode Island bridges are examined as part of this study. Specifically, Bridge #547 (Garden Street Bridge), Bridge #548 (Pine Street Bridge), Bridge #608 (Hartford Avenue - West Bridge), and Bridge #657 (Broad Street - South Bridge). The feasibility of increasing the bridge load rating is also assessed.

Besides the study of the specific bridges, the range of applicability of the continuity schemes is also investigated. As with RIDOT's bridges, the effects of each connection detail is recorded and assessed.

It was believed that several states and, perhaps, Canadian provinces had experience with eliminating existing joints in multi-simple span steel bridges. Therefore, a survey of United States and Canadian departments of transportation needed to be completed to determine the state-of-practice.

CHAPTER 2

NORTH AMERICAN SURVEY

A survey of the United States and Canadian departments of transportation was made in order to investigate the current state-of-practice related to expansion joint elimination. Sixty four agencies returned the survey. A summary of the results can be found in Appendix A.

As part of this survey, a short questionnaire of twenty questions was mailed to transportation agencies asking a range of questions concerning their experience with retrofitting existing multi-simple span steel bridges with a continuity detail. When developing the questionnaire, a balance was struck between simple direct questions and essay type questions in order to obtain meaningful data and subjective interpretations of the methods used.

Twenty-two agencies are involved with this type of retrofit and two have their first retrofit under design. The twenty two agencies involved include: California, Colorado, Connecticut, Georgia, Illinois, Indiana, Kansas, Kentucky, Massachusetts, Maine, Maryland, New Hampshire, New York, Pennsylvania, Rhode Island, South Dakota, Tennessee, Utah, Vermont, Virginia, Ontario, and Saskatchewan. All sixty-four agencies responding to the survey expressed an interest in learning more.

In the United States and Canada, approximately 500 or more bridges have been made continuous to date (California did not specify any number). With 200 bridges made continuous, Utah appears to have the largest number of conversions. Most of the subject construction appears to have taken place within the last five years, Fig. 2.1.

The exact methods of continuity used for retrofit varied from agency to agency, but most have tried only one continuity scheme within their region. Ten agencies have connected only the deck while removing the joint. This appears to be the most popular method tried. The maximum span length connected using the deck connection scheme appears to be 130 feet (Ontario).

Three agencies have tried connecting the deck and top flange. Two agencies have tried connecting the deck and both top and bottom flanges. Two agencies have tried connecting the deck and bottom flange. Seven agencies have used a full moment splice. Still yet, three agencies have poured integral pier caps to eliminate joints in steel bridges. Four agencies have made bridge girders integral with abutments.

Six spans was the maximum number of spans connected, with a span configuration of 84 - 82 - 83 - 96 - 71 - 73 feet (Tennessee). There were four other agencies that have connected up to five spans at once. Only three agencies have standard connection details.

The next several questions of the survey discussed the last bridge the agency made continuous. Five agencies have made at least one bridge continuous in the last two years. Nine of the latest bridges were connected only at the deck, one connected the deck and top flange, none were connected at the deck, top and bottom flange, one was connected at the deck and bottom flange, six were connected with a full moment splice, two used an integral pier cap connection, and one used a web cleat angle near the bottom flange. Example plans were submitted by some states. Details of these plans are shown in Figures 2.2 through 2.8. Figure 2.9 compares what schemes have been used.

When asked why the bridge was retrofitted with a continuous deck joint, nearly all responded that the purpose was to reduce the number of joints. Some replied that they did so to increase the live load capacity of the bridge. These respondents used full moment splices as a means of retrofit on some of their bridges. New Hampshire and California said they also did it to improve the seismic resistance. Figure 2.10 illustrates these responses.

An essay portion of the questionnaire asked about construction sequencing, problems with construction, service, or maintenance, and if there were any conflicts between the design and field performance of a bridge. Detailed responses are included in Appendix A, however, the following includes some of the more interesting comments made.

Ontario indicated that they had tried several schemes (full depth concrete diaphragm, welding bottom flanges), many of which were costly. However, Metro Toronto has been very successful using deck only continuity.

It appears that most agencies, when specifying a continuity detail, replace the bearings. However, some just encase them in concrete when pouring an integral pier cap connection. In addition, some mentioned that they retard the concrete or cast the continuous deck joint segment last in order to make the deck joint subject to live loads only.

Most agencies noted no construction problems. However, New York noted that the alignment of the girders should be field measured before fabricating splice plates.

Few service problems were noted. Indiana went as far to say that no problems have occurred in the last ten years. Colorado noted that their integral piers and abutment diaphragms develop vertical cracks that extend upward from the bearing region through the diaphragm concrete.

Colorado mentioned that since retrofitting its bridges, drainage that previously found its way through expansion joints is now redirected towards the abutments where it erodes the fill slopes. California noted only one bridge with a maintenance problem but gave no further details on what those problems were. No other agencies noted any other maintenance problems.

Agencies were asked to discuss any conflict between design and field performance of the continuous deck schemes. Ontario is now in the process of developing standards and design guidelines. They found that since there is no consistent design approach, there appears no way to compare design and field performance.

Though no state has instrumented any of their steel bridges using the subject retrofit details, Rhode Island is considering a limited instrumentation program.

In summary, approximately 500 bridges have been made continuous or semi-continuous with maximum span lengths up to 300 feet and up to six spans connected in a structure. Very few problems have been noted with any of the connection schemes. However, the deck only connection scheme appears to be the most popular and most cost effective.

CHAPTER 3

BRIDGE MODELING

3.1 MODEL DEVELOPMENT

As part of this study, four Rhode Island bridges were analyzed. These bridges were: Garden Street Bridge #547, Pine Street Bridge #548, Hartford Avenue - West Bridge #608, and Broad Street - South Bridge #657. In addition, the range of applicability for the continuity schemes was desired, therefore, several bridge structures had to be developed in order to model the properties of realistic bridges.

Bridge plans were obtained from the Rhode Island Department of Transportation to determine the properties and coefficients of the four Rhode Island bridges. The American Iron and Steel Institute's *AISI Short-Span Steel Bridge Plans* book was used to determine the wide range of steel bridge span data needed for the parametric models.

ANSYS (1995), a professional finite element analysis package, was used for the analysis of the bridge structures for this study. Two dimensional finite element models made of beam and spring elements were used to model composite steel beams and the continuous joint schemes. Fig. 3.1.1 shows a simple model of a typical case. The continuous joint is composed of the deck reinforcing (top and bottom mat) and one or more of the following: top flange splice plate, bottom flange splice plate, a full moment steel beam splice, or no other connection. A "no joint connection" was also studied as the control case. An approximate offset of the reinforcement and splice plates from the neutral axis was modeled using a relatively stiff beam element.

Instead of creating separate models for each RIDOT bridge continuity scheme, one bridge model was created for each structure with all the continuity connections, Fig. 3.1.2. Axial spring elements were placed at the location of top mat reinforcement, bottom mat reinforcement, top flange splice plate, and bottom flange splice plate. A beam element representing the moment splice was placed between the two spans. To simulate a connection scheme, the connecting elements were turned "on" and "off" for each scheme by negating the property characteristics. Except for the No Joint Connection scheme, the deck reinforcement spring elements remained "on" at all times.

The beam element properties were calculated using the composite girder properties of the structures shown on the RIDOT and AISI plans. The moment splice beam element properties were considered to be equal to the properties of the lesser of the two non-composite beams being connected. The flange splices' properties were equal to the lesser of the two beam flanges being connected. The top and bottom deck reinforcing mat spring constants were calculated using the individual steel cross sectional properties and the minimum development length as specified in the American Association of State Highway and Transportation Officials' (AASHTO) *Standard Specifications for Highway Bridges* (1996). Since the centroids of all the composite girders are relatively close, each model span was placed at the same elevation

As normal for this type of rehabilitation, bearings are reconfigured to allow for the structure to expand and contract. The boundary conditions for all the models used one pin (fixed bearing) and rollers (expansion bearings) for the remaining conditions, Fig. 3.1.1.

The application of live load was in accordance with AASHTO's *Standard Specifications for Highway Bridges* (1996), Section 3: Loads. In general, the truck loading case controls over the lane loading case except for very long spans. An HS-20 truck configuration and corresponding lane loading is shown in Figs. 3.1.3 and 3.1.4. However, an analysis of standard truck loading versus lane loading, as stipulated in AASHTO (1996) Section 3.7, was completed because of the unusual structural connections. The truck loading was moved at half foot intervals to maximize the effects on the joint. It was learned at this time that loading the largest span created the maximum effect on the connection details. The truck loading case was shown to control over lane loading as expected.

In order to facilitate the modeling of both HS-20 and HS-25 vehicles, a unit weight HS vehicle wheel load was moved over the model structures maximizing the effects on the structure. The results were then multiplied by an appropriate factor to account for the actual weight of the vehicle, impact factor, and distribution factor as stipulated in AASHTO (1996) specifications. The HS-25 vehicle uniformly weighs 25% more than the HS-20 and has become the standard design vehicle for RIDOT and other departments of transportation.

It is assumed that only live load will be applied to the continuity schemes. In practice, this is normally accomplished by retarding the concrete or by constructing the continuity connection after all the dead load and superimposed dead load has been placed. Therefore, no dead load or superimposed dead load has been considered during the modeling of these structures. Concrete strengths were assumed to be 4,000 psi which is RIDOT's standard strength for concrete bridge decks. The reinforcing steel yield strength was taken to be 60,000 psi.

Only two spans were connected at one time during the final modeling phase. It was determined that little effect is transmitted to the far end of the unloaded second span from a loading in the first span. Therefore, a distant third span would see little effect from a loaded first span. In addition, if the second span of a three span configuration was loaded, the loading would be split between the two connections.

Temperature induced movements need also be considered in bridge rehabilitation studies. These are relatively small for the bridge spans considered. Also, the study did not attempt to produce a fully integral bridge. Intermediate bridge joints were eliminated but longitudinal movements still need to be accommodated in the abutments using for example an integral abutment detail.

3.2 RHODE ISLAND BRIDGE MODELS

Garden Street Bridge #547 (4 span), Pine Street Bridge #548 (4 span), Hartford Avenue - West Bridge #608 (2 span), and Broad Street - South Bridge #657 (3 span) were evaluated. Table 3.1 lists the span lengths of these four bridges. Figs. 3.2.1 through 3.2.4 show the plan elevations of each bridge.

Bridge	Span 1 (ft)	Span 2 (ft)	Span 3 (ft)	Span 4 (ft)
Garden Street (#547)	35' 6"	58' 1"	58' 0"	26' 6"
Pine Street (#548)	31' 4"	57' 11"	57' 11"	36' 11"
Hartford Avenue-West (#608)	93' 0"	84' 6"		
Broad Street - South (#657)	74' 0"	72' 9"	47' 9"	

Table 3.1 Span lengths of the four Rhode Island Bridges Considered

The model for Bridge #547 spans 1 and 2 is presented in Figures 3.2.5. It is noted that the arrows in the figure represent the location of the truck wheels as positioned for maximum effect. Support conditions are shown via small triangles. Two perpendicular triangles signify restriction of movement in both directions, i.e. fixed support. The particular model shown in Fig. 3.2.5 has two spans, the support conditions of the first span are fixed-expansion whereas the support conditions of the second span are expansion-expansion. The connection between the two spans is shown in Figures 3.2.6 through 3.2.8 which show the node, element, and property numbers, respectively. Note that in Fig. 3.2.8 material 1 represents span 1 of bridge #547, material 2 represents span 2, materials 8 and 5 are springs modeling the top and bottom mat reinforcement, material 4 corresponds to the top plate, material 9 corresponds to the bottom splice plate and material 7 models a full splice whose property is taken as the minimum property of the two joined girders. Material 6 models a very stiff element used to introduce offsets in the model. By assigning zero or nonzero properties to these materials the various connection details can be modeled.

The remaining bridge models are shown in Figures 3.2.9 through 3.2.13. The locations of the truck as well as the support conditions are shown in the same way as in Fig. 3.2.5 explained earlier. The connection model is similar to plots shown for Bridge #547 spans 1 and 2. It should be noted that since Bridge #548's spans 3 and 4 are nearly identical to Bridge #547's spans 1 and 2, no separate analysis was needed for Bridge #548's spans 3 and 4. Also, in the case of bridge #547, support conditions were such that the bearings over the pier between spans 2 and 3 were both fixed. No model/connection between spans 2 and 3 of that bridge was made although the parametric studies include span lengths of similar magnitude. Deck thicknesses, span lengths, and girder sizes were taken from RIDOT bridge plans. The properties of the structure were determined using the AISC's *Manual of Steel Construction*, Nilson and Winter's *Design of Concrete Structures*, and Salmon and Johnson's *Steel Structures*. Concrete slab reinforcement was considered per the actual plans of the bridges. It consists of top and bottom mats with #4 bars. Concrete cover is taken as 2 in. All rebar has a yield strength 60 ksi. Details of the properties for all Rhode Island bridge models are included in Appendix C.

For each combination of bridge and connection detail, the unit truck had to be moved along the largest span to create to greatest effect.

3.3 PARAMETRIC MODEL

The *AISI Short-Span Steel Bridge Plans* were used to determine section properties of steel bridge models. For modeling simplicity, the plans without cover plates were chosen. It was assumed that a 40 ft roadway width would realistically simulate a typical highway bridge that would adequately carry two highway lanes of traffic, two shoulders, and two sidewalks, Fig 3.3.1. Span lengths from 30 feet to 95 feet were combined in tandem to evaluate the relative effects of each of the continuity schemes. Steel reinforcement in the deck consists of #5 rebar with 60 ksi yield strength. The top mat includes 9X#5 bars whereas the bottom mat has 14X#5 bars within the effective length of the slab. Top mat concrete cover was taken as 2 in. Details of the properties are provided in Appendix C.

Unlike the RIDOT bridge models, only one model was created to simulate numerous span length configurations, girder sizes, continuity connection locations, and material properties. Fig. 3.3.2 shows the parametric bridge model. Figs 3.3.3, 3.3.4, and 3.3.5 show the connection detail with node, element, and property numbers, respectively. In Figure 3.3.5 up to 18 property numbers are indicated. Property numbers 1 and 2 correspond to the first and second spans. Property 4 represents a stiff element used to model offsets of the connection elements. The remaining property numbers represent various connection elements. Note that not all of them are active simultaneously. For example, in Figure 3.3.5 the only available properties are ones are property 3 (moment splice), property 5 (top mat reinforcement), property 11 (bottom mat reinforcement), property 12 (top flange splice), and property 17 (bottom flange splice).

Six span configurations and six continuity schemes were simulated using the one model shown in the previous figures by moving the boundary conditions and changing the element properties. The spans had a wide variation in depth of steel section (38 - 24 inches) but had a constant deck thickness (10 inches) and constant reinforcing size and spacing. Therefore, the relative position of the composite girder centroid to deck reinforcement, top flange, and moment splice varied from the deepest to the shallowest section.

The numerous connections between the two adjacent spans created a complicated modeling process by having to switch "on" and "off" several connections on each span and connection configuration. However, this was facilitated by creating a bookkeeping table indicating which elements were to be switched "on" and "off" during the modeling process.

The parametric models do not include any dead load, similarly to the Rhode Island bridge models. The unit truck was moved along the longest span to create the greatest effect.

CHAPTER 4

ANALYTICAL RESULTS

4.1 OVERVIEW

During the analysis, specific areas of concern became apparent: the stress levels in the top mat of deck reinforcement over the joint connection and the relative potential for deck cracking over the joint.

After analyzing the bridge structures using a unit truck, a truck factor (TF) was used for multiplying the results to obtain the stresses and forces created by an HS-20 and an HS-25 vehicle. This factor consisted of an impact factor (I), distribution factor (DF), and truck wheel line weight factor.

The impact factor (I) was calculated using AASHTO (1996) Section 3.8 equation,

$$I=\frac{50}{SpanLength+125}\leq 0.30$$

The values for impact ranged from 0.23 to 0.30 for spans of 95 to 35. For simplification, a single impact factor of 0.30 was used for all span lengths.

The distribution factor (DF) was calculated using AASHTO's (1996) Table 3.23.1,

$$DF=\frac{GirderSpacing}{5.5}$$

Girder spacing for the Garden Street Bridge #547 was 6'-1.5", the Pine Street Bridge #548 was 6'-1.5", the Hartford Avenue - West Bridge #608 was 7'-2", the Broad Street - South

Bridge #657 was 7 feet, and the parametric model's girder spacing was 11'-6". The distribution factors were 1.1, 1.1, 1.3, 1.3, and 2.1, respectively.

The weight of an HS-20's front wheel is 4 kips and middle and rear wheels are 16 kips. An HS-25's front wheel is 5 kips and its mid and rear wheels are 20 kips. By dividing each HS truck's wheel weights by a wheel weight factor (WWF) of 4 or 5, the unit truck wheel weights became 1 kip, 4 kips, and 4 kips for the respective wheels. A truck factor was then produced by multiplying the impact factor (I), the distribution factor (DF), and the HS vehicle's wheel weight factor (WWF).

$$TF = WWF*(1+I)*DF$$

The HS-20 truck factors (TF) for the models become 5.79 for Garden Street Bridge #547 and Pine Street Bridge #548, 6.76 for Hartford Avenue - West Bridge #608 and Broad Street - South Bridge #657, and 10.87 for the parametric structures. The HS-25 truck factors (TF) are 1.25 times the HS-20 truck factors (TF).

This procedure was used to determine the relative effects of HS vehicles with different weights on the bridge components without having to solve each of the 72 structure models twice for HS-20 and HS-25 vehicles. All figures and tables are for the HS-25 truck.

When retrofitting a bridge with a connection detail, it is prudent to design the remaining expansion/contraction joints to absorb the combination of connected spans' movement.

4.2 ANALYSIS OF RHODE ISLAND BRIDGE MODELS

Six separate cases were considered: 1) Garden Street Bridge #547 spans 1 and 2, 2) Garden Street Bridge #547 spans 3 and 4, 3) Pine Street Bridge #548 spans 1 and 2, 4) Hartford Avenue - West Bridge #608 spans 1 and 2, 5) Broad Street - South Bridge #657 spans 1 and 2, and 6) Broad Street - South Bridge #657 spans 2 and 3. For each one of these six cases six connections schemes were utilized for a total of 36 models. The six connection schemes used are: 1) Deck Only connection, 2) Deck and Top Flange connection, 3) Deck and Bottom Flange connection, 4) Deck, Top and Bottom flange connection, 5) Full Moment Splice connection, 6) No Joint connection.

Figure 4.2.1 shows the deformation for Bridge #547 spans 1 and 2 for the deck only connection scheme. Note that the displacements have been exaggerated for effect only. Similar deformation plots can be made for the remaining 35 models.

Top reinforcing stresses for bridges #547 and #548 connection schemes are listed in Table 4.2.1. The top reinforcing stress ranged from 8% to 38% of yield. The highest stress levels were recorded for the Deck and Bottom Flange connection. The calculated crack widths are also included in Table 4.2.1. Figures 4.2.2 and 4.2.3 show in a bar graph format these values. The largest potential for cracking came from the Deck Only and Deck and Top Flange connection schemes. It should be noted that these values (0.3 inches) may be distributed over an area of concrete and may not occur in one location over the joint. The lowest potential was recorded for the Deck, Top, and Bottom flanges connection.

Figure 4.2.4 shows the reaction at the bearing for the various connection schemes. In case of full moment connection uplift forces are developed which need to be resisted. In interpreting these large uplift forces one should realize that this is a retrofit. Two existing simply supported beams are fully connected over a pier but no elimination of one of the two supports over the pier was considered. This results in a very short beam-like segment between the two supports over the pier. The resulting model is a three span continuous beam where the mid span is very short, of the order of one foot or so. As beam curvature is forced to reverse within that short span large uplift forces result. The recommendation here is that if such a retrofit is attempted the two supports over the pier should be replaced by a single support. That would eliminate the uplift force.

Table 4.2.2 presents top reinforcing stresses and crack widths for Bridge #608. The top reinforcing stresses ranged from 5% to 73% of yield. The highest stress was observed for the Deck and Bottom Flange. The greatest potential for cracking was from the Deck Only and the Deck/Top Flange connection schemes.

Table 4.2.2 also includes results for Bridge #657 spans 1 & 2 and spans 2 & 3. Similar observations can be made for the top reinforcement stresses as well as the expected crack widths.

The case of the No Joint connection corresponds to the current situation where a mechanical expansion joint or seal separates the two spans. The reported crack width is the expected opening of the joint as a truck goes over the bridge. The cracks reported are due only to truck traffic. An overall analysis of the bridge would need to be made to make sure that temperature induced movements are accommodated using appropriate bearing details and far end connections.

Moments for the loaded spans were recorded for bridges 547, 548, 608, and 657. A clear trend became apparent when the moment of an HS-25 (45 ton) vehicle passing over the unmodified bridge (No Joint scheme) was compared to the other mid-span moments created by other connection details, Tables 4.2.3 and 4.2.4. The Deck Only and Deck and Top Flange connection schemes produced no significant differences in the carrying capacities of the bridges. The remaining schemes, however, improved the bridge rating from 45 tons to an HS vehicle weighing from 51 to 79 tons. The largest improvements, came from the Full Moment Splice connection. Tables 4.2.3 and 4.2.4 also include the percentage increase of the moment capacity of the bridges. It is shown that in one case, a full moment splice increased the capacity by 75%. It is important, though, to note the stresses in other bridge components, if a decision is made to connect spans.

Another observation is that the effects of the various connection schemes depend mostly on the largest span. For example the results for Bridge #657 spans 1 & 2 and Bridge #657 spans 2 & 3 are very similar although in the first case the span lengths are almost equal (73' 12" - 72' 9"), whereas in the second case the spans are unequal (72' 9" - 47' 9").

No obvious relation between top reinforcing stress and expected crack width can be drawn. There are many variables including the changing of the section neutral axis as various connection schemes are implemented. It should be noted that the reported crack values do not constitute a single crack. This is an estimated total value to be distributed over a long area depending on the particular detailing.

Model	Connection Scheme	Top Reinf. Stress (ksi)	Deck Crack Width (in)
Bridge 547 Spans 1 & 2 (35' 6" - 58' 1")	Deck Only	5.335	0.320
	Deck/Top Flange	12.101	0.305
	Deck/Bottom Flange	21.079	0.065
	Deck/Top and Bottom Flanges	5.555	0.028
	Full Moment Splice	4.946	0.054
	No Joint	0.000	0.321
Bridge 547 Spans 3 & 4 (58' - 26' 6")	Deck Only	5.318	0.319
	Deck/Top Flange	12.190	0.312
	Deck/Bottom Flange	19.358	0.060
	Deck/Top and Bottom Flanges	5.131	0.025
	Full Moment Splice	7.949	0.036
	No Joint	0.000	0.321
Bridge 548 Spans 1 & 2 (31' 4" - 57' 11")	Deck Only	5.287	0.317
	Deck/Top Flange	14.383	0.307
	Deck/Bottom Flange	22.487	0.069
	Deck/Top and Bottom Flanges	6.264	0.031
	Full Moment Splice	4.893	0.053
	No Joint	0.000	0.320

Table 4.2.1 - RIDOT Bridge Results Summary

Model	Connection Scheme	Top Reinf. Stress (ksi)	Deck Crack Width (in)
Bridge 608 Spans 1 & 2 (93' - 84' 6")	Deck Only	8.054	0.387
	Deck/Top Flange	19.404	0.381
	Deck/Bottom Flange	44.022	0.097
	Deck/Top and Bottom Flanges	6.981	0.022
	Full Moment Splice	3.101	0.021
	No Joint	0.000	0.383
Bridge 657 Spans 1 & 2 (73' 12" - 72' 9")	Deck Only	4.933	0.222
	Deck/Top Flange	13.013	0.234
	Deck/Bottom Flange	16.579	0.039
	Deck/Top and Bottom Flanges	6.039	0.019
	Full Moment Splice	1.830	0.018
	No Joint	0.000	0.231
Bridge 657 Spans 2 & 3 (72' 9" - 47' 9")	Deck Only	4.033	0.228
	Deck/Top Flange	10.671	0.220
	Deck/Bottom Flange	16.081	0.041
	Deck/Top and Bottom Flanges	5.994	0.022
	Full Moment Splice	1.861	0.019
	No Joint	0.000	0.217

Table 4.2.2 - RIDOT Bridge Results Summary

Model	Connection Scheme	HS-25 Moment (kip-/in)	Equivalent HS-Truck (tons)	% Increase
Bridge 547 Spans 1 & 2 (35' 6" - 58' 1")	Deck Only	8,227	45	0
	Deck/Top Flange	8,160	46	2
	Deck/Bottom Flange	6,961	54	20
	Deck/Top and Bottom Flanges	6,777	55	22
	Full Moment Splice	6,315	59	31
	No Joint	8,292	45	
Bridge 547 Spans 3 & 4 (58' - 26' 6")	Deck Only	8,344	45	0
	Deck/Top Flange	8,313	45	4
	Deck/Bottom Flange	7,223	51	13
	Deck/Top and Bottom Flanges	7,075	52	16
	Full Moment Splice	5,679	66	47
	No Joint	8,299	45	
Bridge 548 Spans 1 & 2 (31' 4" - 57' 11")	Deck Only	8,329	45	0
	Deck/Top Flange	8,275	45	0
	Deck/Bottom Flange	7,051	52	16
	Deck/Top and Bottom Flanges	6,924	54	20
	Full Moment Splice	5,439	69	53
	No Joint	8,282	45	

Table 4.2.3 - HS Truck Equivalent Ratings

Model	Connection Scheme	HS-25 Moment (kip-in)	Equivalent HS-Truck (tons)	% Increase
Bridge 608 Spans 1 & 2 (93' - 84' 6")	Deck Only	17,437	46	2
	Deck/Top Flange	17,320	46	2
	Deck/Bottom Flange	14,672	54	20
	Deck/Top and Bottom Flanges	13,949	57	26
	Full Moment Splice	10,137	79	75
	No Joint	17,692	45	
Bridge 657 Spans 1 & 2 (73' 12" - 72' 9")	Deck Only	13,117	45	0
	Deck/Top Flange	13,027	46	2
	Deck/Bottom Flange	10,860	55	22
	Deck/Top and Bottom Flanges	10,627	56	24
	Full Moment Splice	8,408	71	58
	No Joint	13,221	45	
Bridge 657 Spans 2 & 3 (72' 9" - 47' 9")	Deck Only	12,637	45	0
	Deck/Top Flange	12,572	46	2
	Deck/Bottom Flange	10,483	55	22
	Deck/Top and Bottom Flanges	10,260	56	24
	Full Moment Splice	8,111	71	57
	No Joint	12,773	45	

Table 4.2.4 - HS Truck Equivalent Ratings

4.3 PARAMETRIC MODELS

The parametric models were split into those with equal spans and those with unequal spans. The equal span configurations consisted of two equal 95, 75, and 55 foot span combinations. The unequal span configurations consisted of the following: a) 90' - 45', b) 80' - 40', and c) 70' - 35' spans. For each one of these six cases six connections schemes were utilized for a total of 36 models. The six connection schemes used are: 1) Deck Only connection, 2) Deck and Top Flange connection, 3) Deck and Bottom Flange connection, 4) Deck, Top and Bottom flange connection, 5) Full Moment Splice connection, 6) No Joint connection.

Table 4.3.1 presents the top reinforcement stress and the deck crack width for the equal span models under the HS-25 truck. Table 4.3.2 shows the same quantities for the unequal span models. Figures 4.3.1 and 4.3.2 show in a bar graph format the top mat reinforcing stress and the expected total crack opening for the equal span cases. Similar graphs can be generated for the unequal span using the values in table 4.3.2. There are similar trends between the equal span and unequal span results. One observation is that the Deck/Top Flange connection raises by 42% the stress in the top reinforcement as compared with the deck only connection for the equal span cases. For the unequal span models this increase is 35%. It should be pointed out that the connections of the bottom flanges are not realistic in case of highly unequal spans since the two girders would probably have different depths. In this study it was assumed that the girders have the same depth.

Tables 4.3.3 and 4.3.4 present the maximum moments of the loaded span as well as the HS equivalent ratings for the equal and unequal spans. The percentage increases in moment capacity observed with the different connection details are also included. No notable improvement in the load rating capacity was recorded for the Deck Only and the Deck and Top Flange connections. However, other details improved the rating up to 82 tons. Stress levels in other bridge components should be noted before choosing a particular detail for retrofit design.

Another observation is that the various observed quantities (stresses, crack widths, maximum moments) at the bridges of the parametric study are higher than corresponding values of the Rhode Island bridges of equivalent span. For example one can compare Bridge #608 spans 1 & 2 (93' - 84' 6"), with the 95' - 95' parametric model. For deck only connection the top reinforcement stress, expected crack width, and maximum moment for Bridge #608 are 8.054 ksi, 0.387", and 13,950 kip/in respectively. The corresponding values for the 95' - 95' parametric model are 26.047 ksi, 0.529" and 23,003 kip-in. These differences are due to the spacing of the girders which in turn affects the girder distribution factors. The bridges used for the parametric study have much larger girder spacing than the Rhode Island bridges. The parametric study bridge geometries were taken as mentioned earlier from the AISI bridge tables to avoid redesigning several bridges for the parametric study. Note however, that the changes due to the various connection schemes are similar. Also, the bridge rating is very similar for both cases, irrespective of the girder spacing.

Model	Connection Scheme	Top Reinf. Stress (ksi)	Deck Crack Width (in)
95 - 95	Deck Only	26.047	0.529
	Deck/Top Flange	45.520	0.508
	Deck/Bottom Flange	43.531	0.089
	Deck/Top and Bottom Flanges	7.012	0.024
	Full Moment Splice	6.921	0.021
	No Joint	0.000	0.543
75 - 75	Deck Only	22.928	0.466
	Deck/Top Flange	39.242	0.448
	Deck/Bottom Flange	40.742	0.092
	Deck/Top and Bottom Flanges	9.756	0.035
	Full Moment Splice	7.308	0.025
	No Joint	0.000	0.480
55 - 55	Deck Only	11.726	0.238
	Deck/Top Flange	19.245	0.232
	Deck/Bottom Flange	26.733	0.072
	Deck/Top and Bottom Flanges	10.952	0.043
	Full Moment Splice	7.200	0.027
	No Joint	0.000	0.244

Table 4.3.1 - Parametric Model Results Summary

Model	Connection Scheme	Top Reinf. Stress (ksi)	Deck Crack Width (in)
90 - 45	Deck Only	25.022	0.508
	Deck/Top Flange	39.503	0.488
	Deck/Bottom Flange	34.726	0.088
	Deck/Top and Bottom Flanges	14.736	0.055
	Full Moment Splice	7.136	0.022
	No Joint	0.000	0.530
80 - 40	Deck Only	22.732	0.462
	Deck/Top Flange	34.878	0.442
	Deck/Bottom Flange	28.686	0.079
	Deck/Top and Bottom Flanges	13.821	0.054
	Full Moment Splice	21.143	0.072
	No Joint	0.000	0.483
70 - 35	Deck Only	21.691	0.440
	Deck/Top Flange	32.298	0.422
	Deck/Bottom Flange	26.295	0.080
	Deck/Top and Bottom Flanges	14.395	0.060
	Full Moment Splice	20.302	0.073
	No Joint	0.000	0.465

Table 4.3.2 - Parametric Model Results Summary

Model	Connection Scheme	HS-25 Moment (kip-in)	Equivalent HS-Truck (tons)	% Increase
95 - 95	Deck Only	28,754	46	2
	Deck/Top Flange	28,495	46	2
	Deck/Bottom Flange	23,372	56	24
	Deck/Top and Bottom Flanges	22,576	59	31
	Full Moment Splice	16,150	81	80
	No Joint	29,225	45	
75 - 75	Deck Only	21,596	46	2
	Deck/Top Flange	21,386	46	2
	Deck/Bottom Flange	17,004	57	27
	Deck/Top and Bottom Flanges	16,318	60	33
	Full Moment Splice	12,306	80	78
	No Joint	21,911	45	
55 - 55	Deck Only	14,513	45	0
	Deck/Top Flange	14,423	45	0
	Deck/Bottom Flange	11,932	55	22
	Deck/Top and Bottom Flanges	11,493	57	27
	Full Moment Splice	8,802	75	67
	No Joint	14,613	45	

Table 4.3.3 - HS Truck Equivalent Ratings

Model	Connection Scheme	HS-25 Moment (kip-in)	Equivalent HS-Truck (tons)	% Increase
90 - 45	Deck Only	26,880	46	2
	Deck/Top Flange	26,686	46	2
	Deck/Bottom Flange	22,734	54	20
	Deck/Top and Bottom Flanges	22,401	55	22
	Full Moment Splice	15,016	82	82
	No Joint	27,384	45	
80 - 40	Deck Only	23,381	46	2
	Deck/Top Flange	23,222	46	2
	Deck/Bottom Flange	20,179	52	16
	Deck/Top and Bottom Flanges	19,969	54	20
	Full Moment Splice	14,055	76	69
	No Joint	23,738	45	
70 - 35	Deck Only	19,820	45	0
	Deck/Top Flange	19,683	46	2
	Deck/Bottom Flange	17,097	52	16
	Deck/Top and Bottom Flanges	16,946	54	20
	Full Moment Splice	12,111	75	67
	No Joint	20,083	45	

Table 4.3.4 - HS Truck Equivalent Ratings

CHAPTER 5

INSTRUMENTATION PROGRAM FOR GARDEN STREET AND PINE STREET BRIDGES

As part of this study a proposal for an instrumentation plan for the Garden Street Bridge (Bridge #547) and the Pine Street Bridge (Bridge # 548) was developed. Although the plan refers to these two bridges, the instrumentation can be adapted easily for other similar bridges in Rhode Island. The instrumentation can be implemented during the rehabilitation of a bridge when elimination of expansion joints is warranted. The complete instrumentation work-plan can be found in Appendix B.

The instrumentation plan was submitted to four potential installers of the sensors in order to estimate costs. Specifically, the plan was submitted to:

Construction Technologies Laboratory (CTL)

Law Engineering

A.G. Lichtenstein & Associates, Inc.

Paul Aldinger and Associates

A copy of the solicitation letter is found in Appendix B. Three of the companies responded and provided cost estimates to implement the proposed instrumentation.

CHAPTER 6

INSTRUMENTATION FOR BRIDGES UNDERGOING JOINT ELIMINATION

In this section a generic instrumentation for monitoring a newly rehabilitated bridge is outlined. Since the exact type and location of the instrumentation depends to a great extent on the specific situation, a general monitoring system is described here. For a specific implementation of such a system refer to Appendix B where a proposed instrumentation for two actual bridges is outlined. Note that this is a case of rehabilitation where expansion joints are eliminated using several connection details. Dead loads are already supported and hence the sensors need to be sensitive enough for live load measurements.

Ideally the following items need to be measured/monitored:

Material Properties:

The properties of all materials used for the rehabilitation should be known as installed. That includes the stress strain curve of any reinforcing bars spanning the old expansion joint as well as of any plates connecting the steel beams. In case that the concrete deck is made continuous over the joint the properties of concrete should also be known including compression strength, modulus of elasticity, Poisson's ratio, and coefficient of thermal expansion. Every effort should also be made to have an accurate knowledge of the properties of the existing materials, including steel girders, concrete and steel reinforcement.

Strain Monitoring:

Strain monitoring is needed to evaluate the detailing of the joint elimination scheme and to find the effect of the joint elimination on the overall bridge behavior. Here, the specific situation dictates the exact measurements to be made. In the most common situation where the deck slab is made continuous over the joint, electrical resistance strain gages can be attached to the **reinforcing bars** to determine the stress levels that they are subjected. Similarly, any **steel plates** connecting the steel girders need to be instrumented with gages to evaluate the stress levels at the plates. In the case of the reinforcing bars, it may be difficult to attach the gages in the field. Bars instrumented at the laboratory can be used and attached at the steel cage overlapping the joints. This will also insure that no cover damage due to installation occurs at epoxy covered rebars.

When some connection scheme is followed during joint elimination, it is desirable to evaluate the effect on overall bridge performance. Strain gages need to be installed at the **steel girders** at mid-span and at the supports to measure the stress levels. Such stresses can deduce the degree of continuity of the scheme selected and provide information on any additional capacity of the bridge resulting from a partial span continuity.

Optionally, **concrete strains** need to be measured over the old joints in case that the deck is made continuous. Carlson strain meters are well suited for such applications and can provide information on concrete strains due to seasonal temperature variations.

Another optional monitoring includes **slope measuring** of the girders using tiltmeters. These can be installed at the ends of the girders on both sides of the expansion joint which is eliminated. In cases of joints which are just filled with asphaltic plugs, girder rotations can provide useful information on the potential for cracking of the asphaltic material as well as they can be used to establish performance standards of the material as far as the required range of deformability.

Temperature Measurements:

Temperatures are needed for correcting strain readings. The carlson strain meters have a built-in system for providing the temperature at the gage. When strains are measured using electrical resistance strain gages, thermocouples need to be positioned at the same locations for temperature corrections.

Data Acquisition:

All sensors need to be routed to a central location. A manual readout station at that location needs to be established. It is advised that an Automated Data Acquisition System (ADAS) is also installed for long term monitoring.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

A study was completed to determine the state-of-art and practice for eliminating joints on existing steel bridges. A literature search was completed and a survey of United States and Canadian departments of transportation was compiled.

The literature search uncovered very little research on the topic of eliminating joints on existing steel bridges even though much work has been completed for prestressed concrete bridges. The papers found concerning the subject were concentrated mostly on field experience and the general benefits of eliminating joints.

The survey of U.S. and Canadian transportation agencies revealed much about the use of joint elimination details for steel bridges. Sixty-four agencies were surveyed of which twenty-two agencies are involved with the elimination of joints on steel bridges. Approximately 500 steel bridges have been made continuous or semi-continuous to date. Utah alone has done 200 conversions. Five agencies have connected five and six spans together at once.

The most popular method of joint connection among the agencies is Deck Only. It was reported to be just as effective as any other method used while being the most cost effective. Several responding agencies mentioned that they retard the concrete or pour the deck joint section last in order to eliminate the dead load stresses from the joint connection. At the request of RIDOT, four bridges were analyzed. These bridges included: Garden Street Bridge #547, Pine Street Bridge #548, Hartford Avenue - West

Bridge #608, and Broad Street - South Bridge #657.

Connection details found promising from the literature search and survey were used for the analysis. These connection schemes included: 1) Deck Only, 2) Deck and Top Flange, 3) Deck and Bottom Flange, 4) Deck, Top and Bottom Flange, and 5) Full Moment Splice. A sixth control case, No Joint connection, was added to the analysis to gauge the previous five connection schemes. The stress at the deck's top mat reinforcement as well as the crack width were recorded along with any increase in the load carrying capacity of the bridge.

Garden Street Bridge #547's top reinforcing mat was stressed up to 21 ksi for the Deck and Bottom Flange connection. The cracking potential was greatest for the Deck Only and the Deck and Top Flange connection schemes.

Pine Street Bridge #548's top reinforcing mat was stressed up to 22 ksi for the Deck and Bottom Flange connection. Again, the greatest potential for cracking was for the Deck Only and the Deck and Top Flange connection schemes.

Hartford Avenue - West Bridge #608's top reinforcing mat was stressed up to 44 ksi for the Deck and Bottom Flange connection. The Deck Only and the Deck and Top Flange connection schemes showed the greatest potential for cracking.

Broad Street - South Bridge #657's top reinforcing mat maximum stress was 16 ksi for the Deck and Bottom Flange connection. The Deck Only and the Deck and Top Flange exhibited the greatest potential for cracking.

By comparing the maximum moments on the loaded span, equivalent truck load ratings were calculated. The Deck Only and the Deck and Top Flange connection schemes did not increase the load carrying capacity of the bridge significantly. The Deck and Bottom Flange connection increased the moment capacity by 13% - 22%, the Deck and Top and Bottom Flanges connection increased the moment by 16% - 26%, and the Full Moment Splice connection increased it by 31% - 75%.

The range of applicability for the connection schemes were also determined. Realistic bridge models were developed to test span connections from 95' to 35'. Equal spans were connected for 95', 75', and 55' span lengths'. Unequal spans were connected in the following span configurations: 90' - 45', 80' - 40', and 70' - 35'.

The maximum top mat reinforcing stresses for the equal 95' span structure was 46 and 44 ksi for the Deck and Top Flange and the Deck and Bottom Flange connections. The greatest potential for cracking was for the Deck Only and the Deck and Top Flange connections.

The maximum top mat reinforcing stresses for the equal 75' span structure was 41 and 39 ksi for the Deck and Bottom Flange and the Deck and Top Flange connection. The Deck Only and the Deck and Top Flange connection details exhibit the greatest potential for cracking.

The maximum top mat reinforcing stress for the equal 55' span structure was 27 ksi for the Deck and Bottom Flange connection. Again, the greatest crack potential was created by the Deck Only and the Deck and Top Flange connection.

For the unequal span structures the greatest top mat reinforcing stress levels came from the Deck and Top Flange connection detail, 40, 35, and 32 ksi, respectively. The greatest cracking potential came from the Deck Only and the Deck and Top Flange connection schemes.

The Deck Only and the Deck and Top Flange connection schemes did not significantly increase the load carrying capacity of the structures. However, the remaining schemes did increase the load carrying capacity from 16% to 82%. The Full Moment Splice created the greatest potential in this regard.

In summary, the techniques used for the elimination of joints in steel bridges have been based nearly exclusively on field experience and intuition. There is no common design approach among and within transportation agencies. The literature search supports the lack of research in this area and the survey shows the different techniques used from agency to agency.

The analysis has shown the stress levels and the potential for cracking increase significantly depending on the connection detail, though no agency in the survey reported any significant problem by using any of these techniques for eliminating joints. It was also shown that the load carrying capacity can be increased by connecting independent spans into a continuous or semi-continuous structure.

Table 7.1 compares all the connection techniques. The Deck Only connection scheme appears to be the most popular, most cost efficient, and easiest to construct. However, it does not improve the load carrying capacity of the structure and it has the most potential for deck cracking. The potential for deck cracking, though, may be minimized by installing elastomeric bearings and providing a sealed control notch in the concrete deck.

Moment Splice

- Increases live load capacity the most
- Low top mat reinforcing stresses for equal spans, however, stresses for unequal spans may be higher
- Low potential for cracking
- Most costly and most difficult to construct

Deck, Top and Bottom Flange

- Significant increase in live load capacity
- Low top mat reinforcing stresses
- Low potential for cracking
- Costly and difficult to construct

Deck and Bottom Flange

- Increases live load capacity
- High top mat reinforcing stresses
- Low potential for cracking
- Costly and difficult to construct

Deck and Top Flange

- Little increase in live load capacity
- High top mat reinforcing stresses
- High potential for cracking
- Costs more and is more difficult to construct than Deck Only scheme

Deck Only

- No increase in live load capacity
- Slight increase in top mat reinforcing stresses
- Highest potential for cracking
- Lowest cost and easiest to construct

Table 7.1 - Connection Scheme Comparison

REFERENCES

Burke, M.P., "Integral Bridges", *Transportation Research Record 1275 - Bridges and Structures: Bridge Research*, 1990, p. 53-61.

Burke, M.P., Integral Bridges: Attributes and Limitations , *Transportation Research Record 1393*, 1993, p.1-8.

Fujiwara, M., Kazuhiro, N., and Yomamoto, S., "Elimination of Expansion Joints in Highway Bridges", *Proceedings of the 8th U.S.-Japan Bridge Engineering Workshop*, May 11-12, 1992.

Hambly, E.C. and Nicholson, B.A., "Prestressed Beam Integral Bridges", *The Structural Engineer*, Volume 68, No. 23, 1990.

Kauffman, G.J. and Mottola, V., "An Evaluation of Thorma Joint - A Flexible Bridge Expansion Joint System", *New Jersey Department of Transportation, Bureau of Transportation Structures Research*, March 1990.

Leathers, R.C., "Bridge Deck Joint Rehabilitation", *FHWA Technical Advisory T5140.16*, U.S. Department of Transportation, Federal Highway Administration, March 26, 1990.

Massoni, D.J., Egan, M.P., and Bacho, L.D., "Live Load Continuity Retrofit", *The Welding Innovation Quarterly*, Volume X, Number 2, p. 15-16, 1993.

Oesterle, R.G., Glikin, J.D., and Larson, S.D., "Design of Precast Prestressed Bridge Girders Made Continuous", *National Cooperative Highway Research Program Report 322*, November 1989.

American Association of State Highway and Transportation Officials (AASHTO), *Standard Specifications for Highway Bridges*, 1996.

American Institute of Steel Construction, Manual of Steel Construction: Load & Resistance Factor Design, First Edition, 1986.

American Iron and Steel Institute, Short Span Steel Bridge Plans, 1995.

ANSYS 5.2, ANSYS, Inc., 1995.

FIGURES

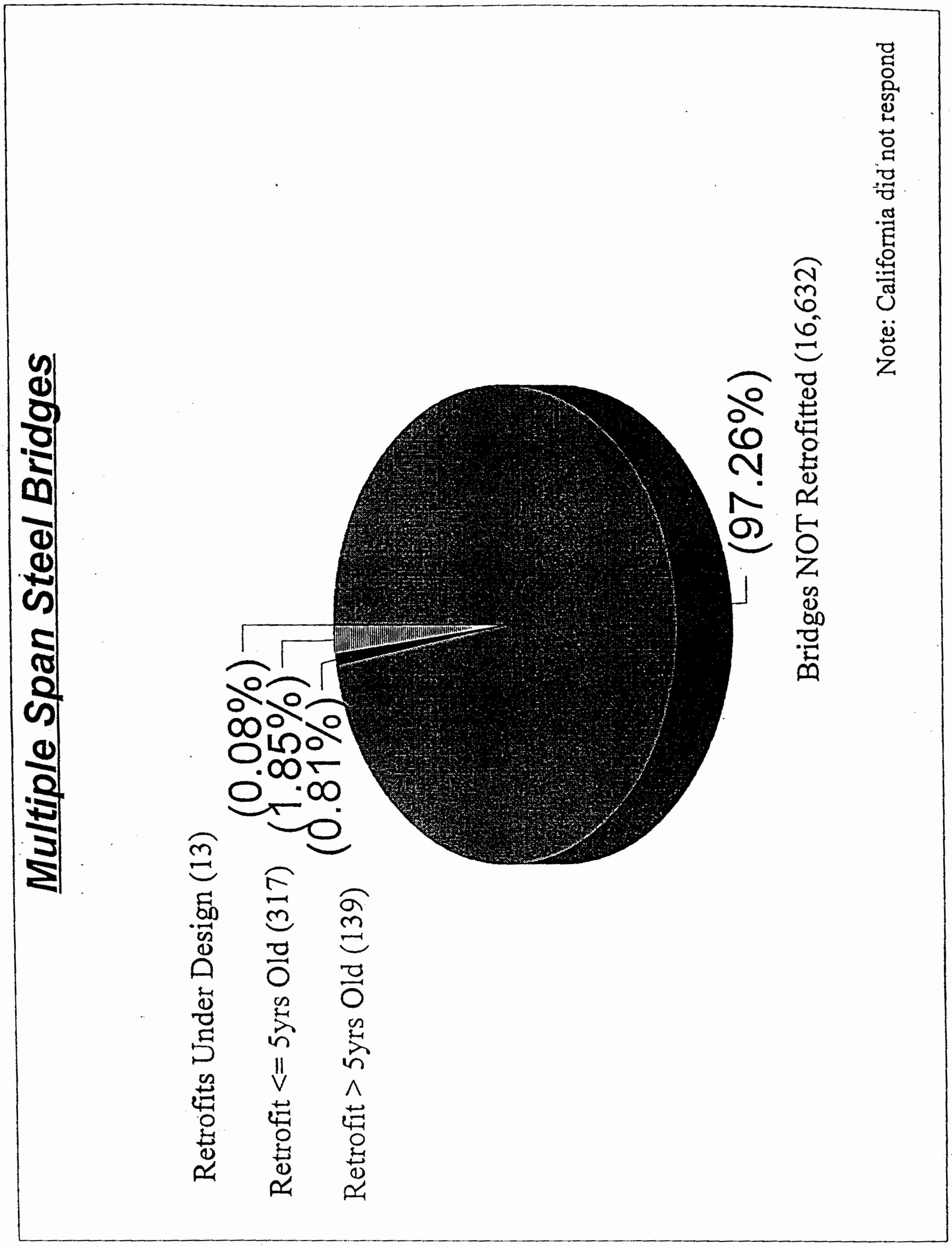


Figure 2.1 - Number of Retrofits

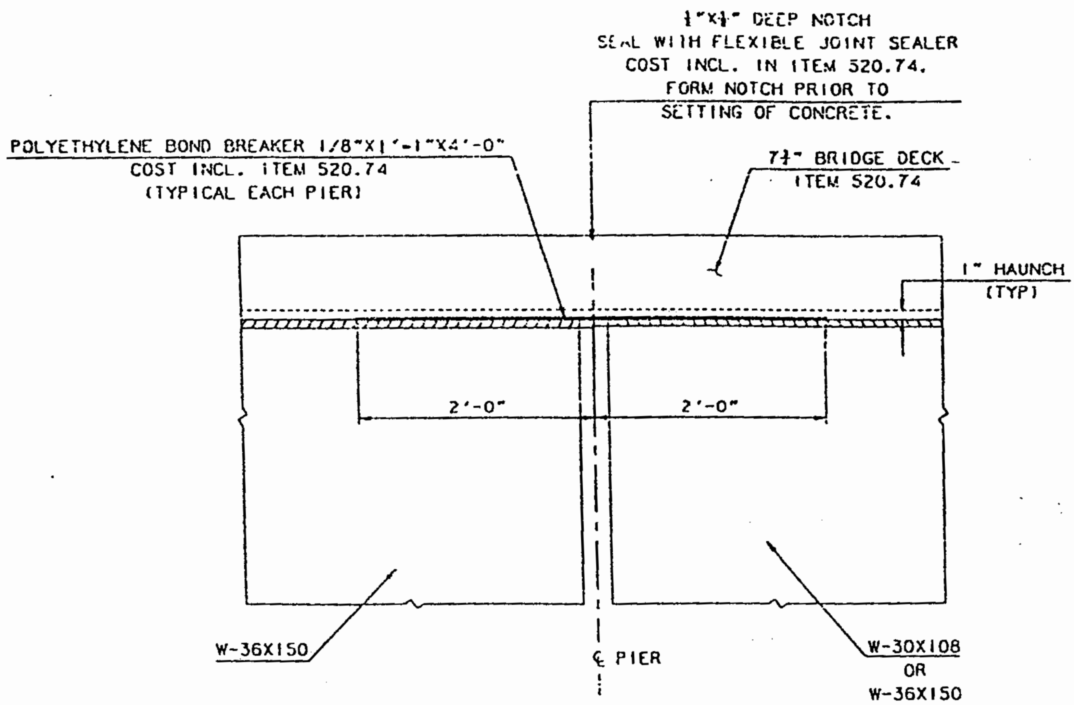


Figure 2.2 - Deck Only Connection: New Hampshire

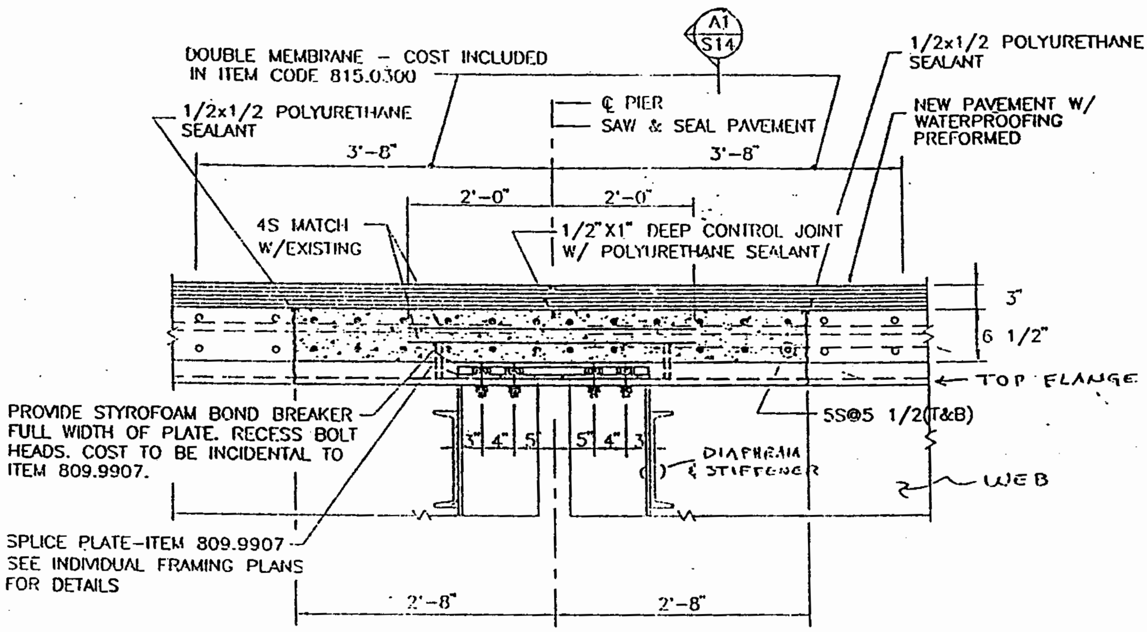
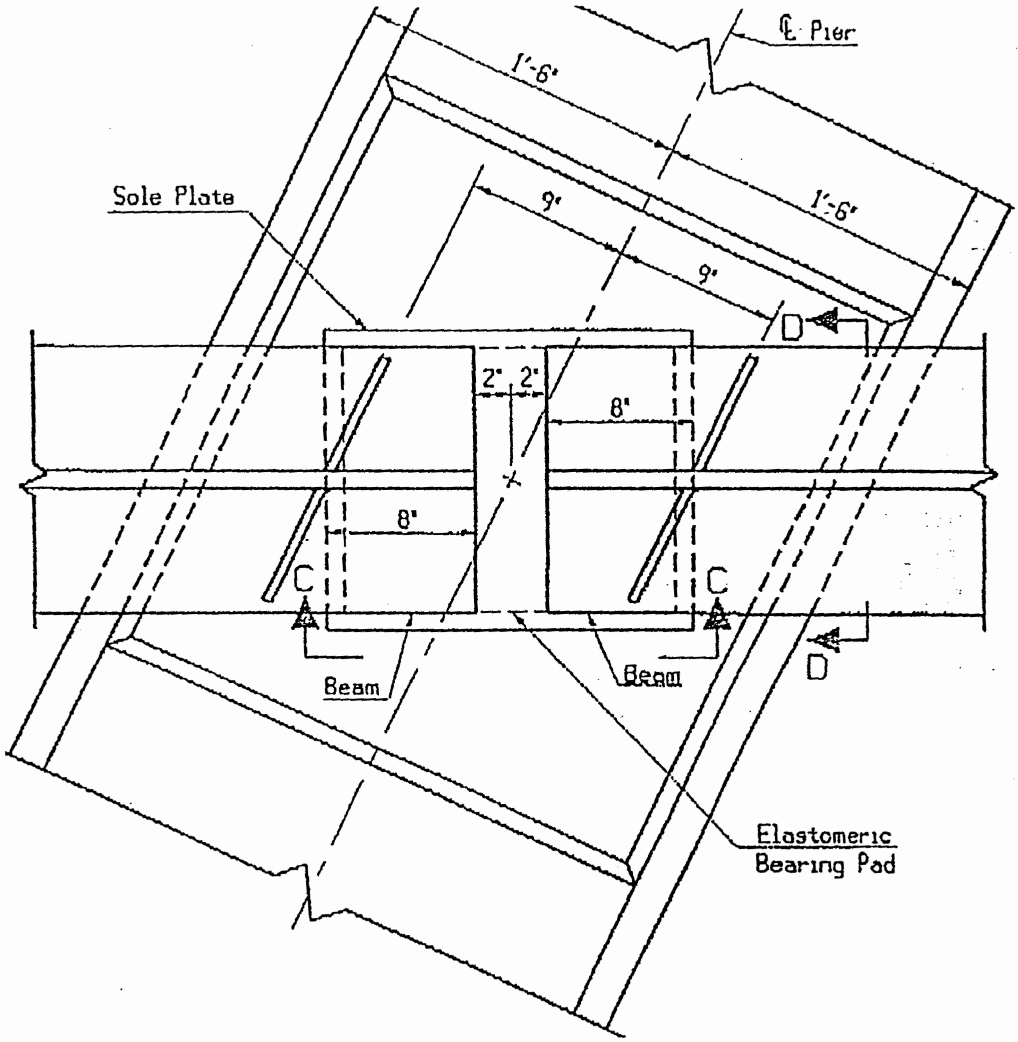


Figure 2.3 - Deck and Top Flange Connection: Rhode Island



PLAN

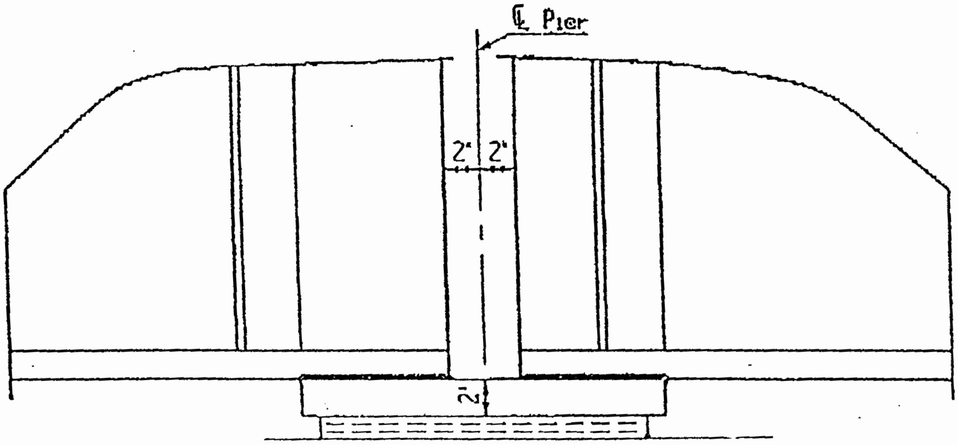


Figure 2.4 - Deck and Bottom Flange Connection: Connecticut

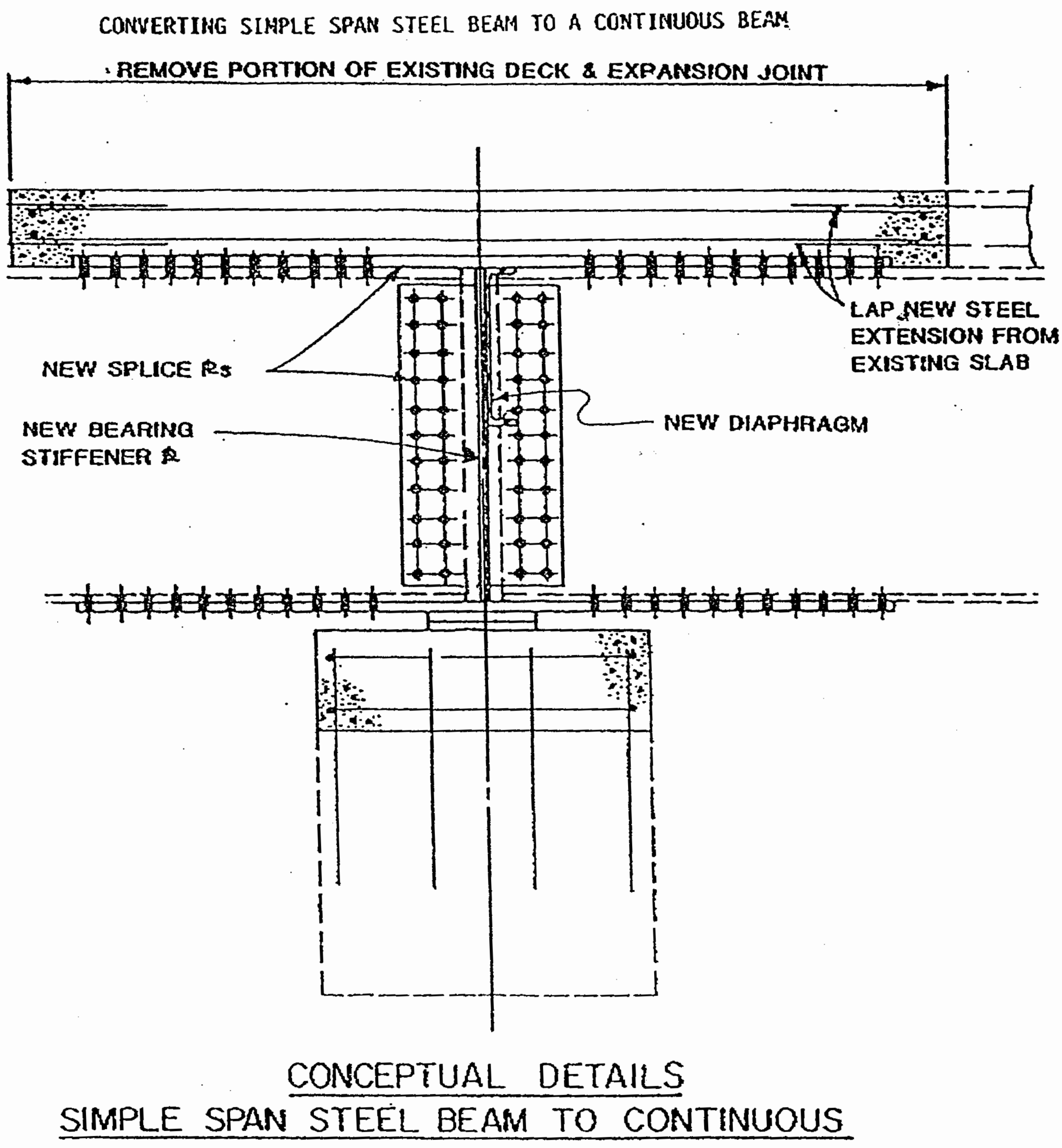


Figure 2.5 - Full Moment Splice: Ohio

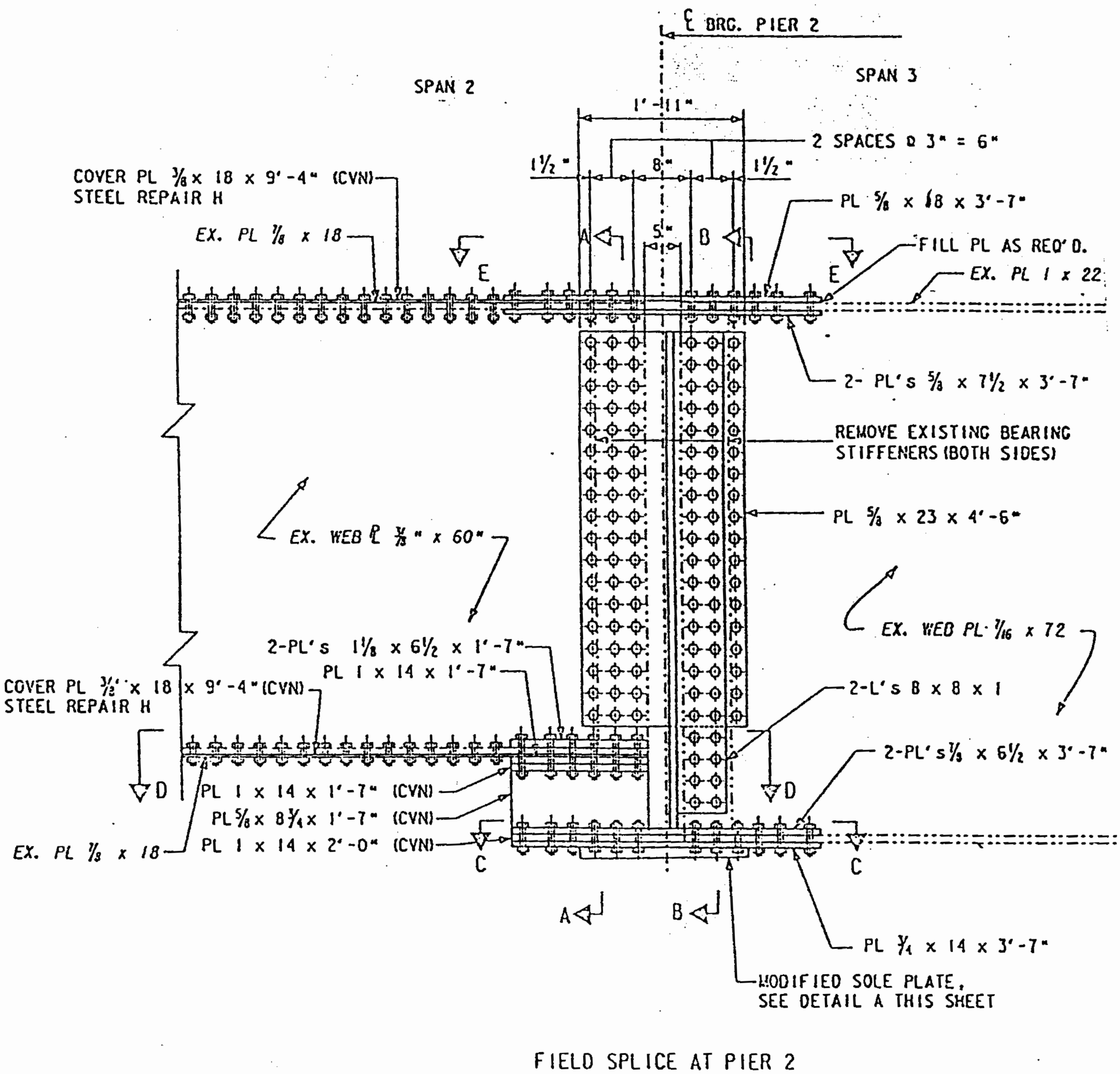


Figure 2.6 - Full Moment Splice: Pennsylvania

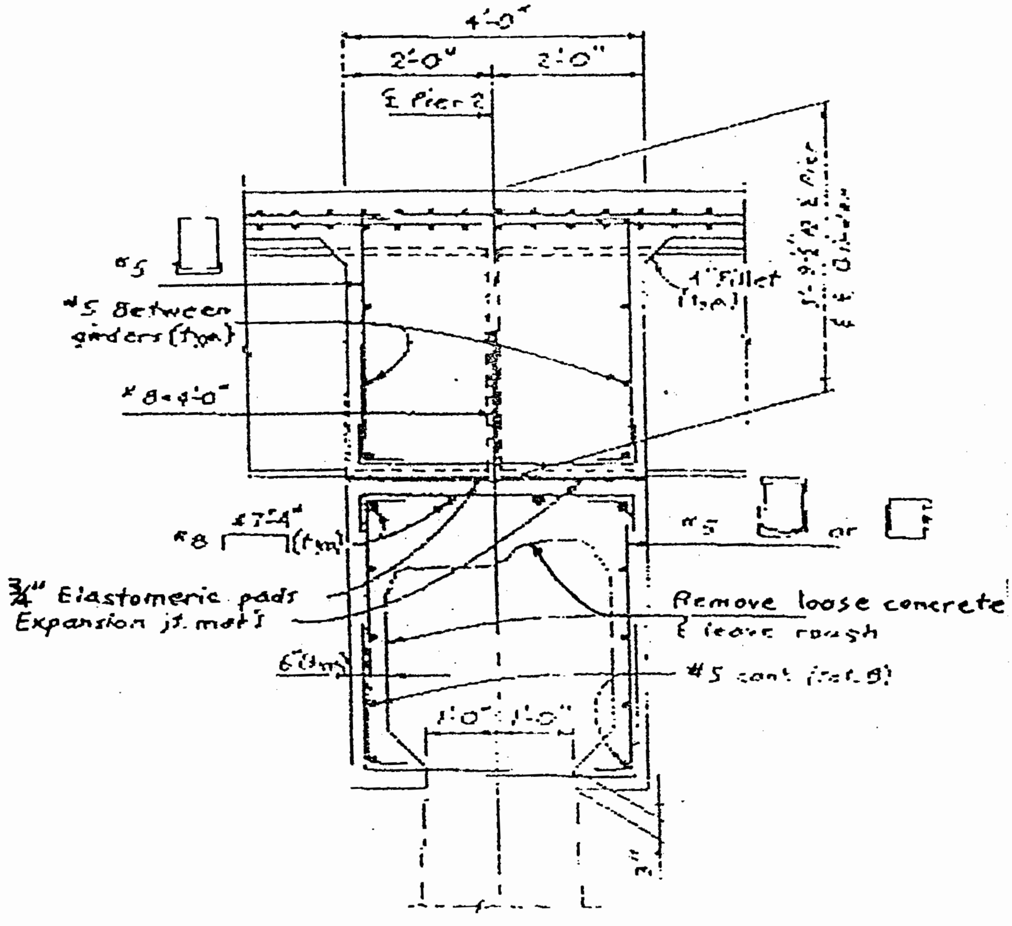


Figure 2.7 - Integral Pier Cap Connection: Colorado

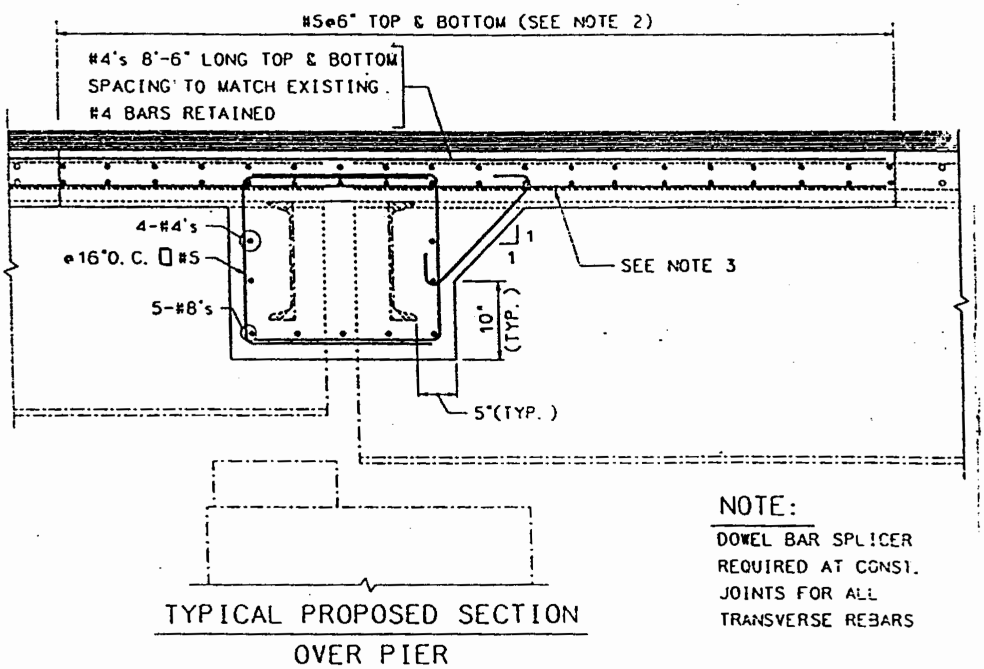


Figure 2.8 - Concrete Encasement Connection: Massachusetts

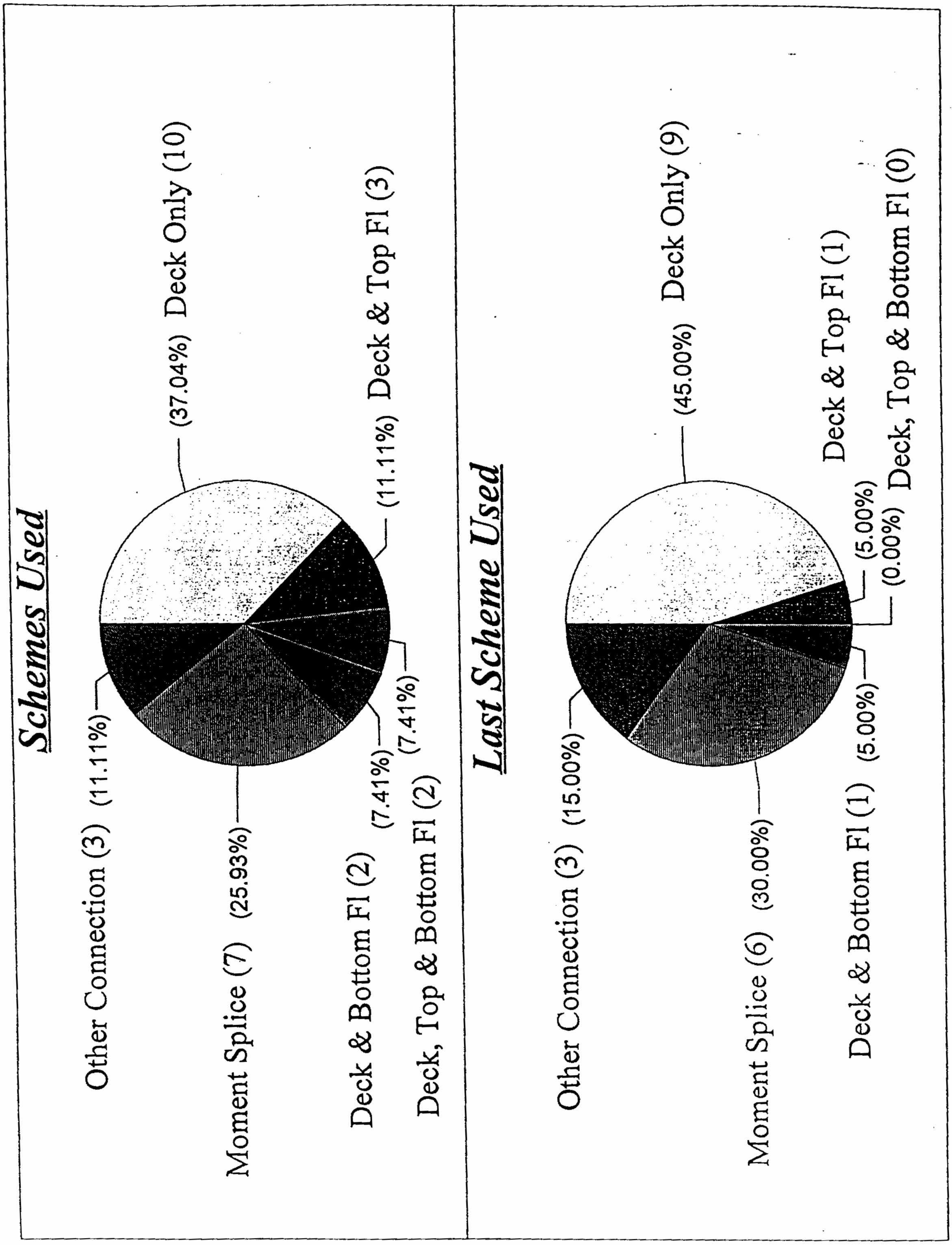


Figure 2.9 - Comparison of Schemes Used

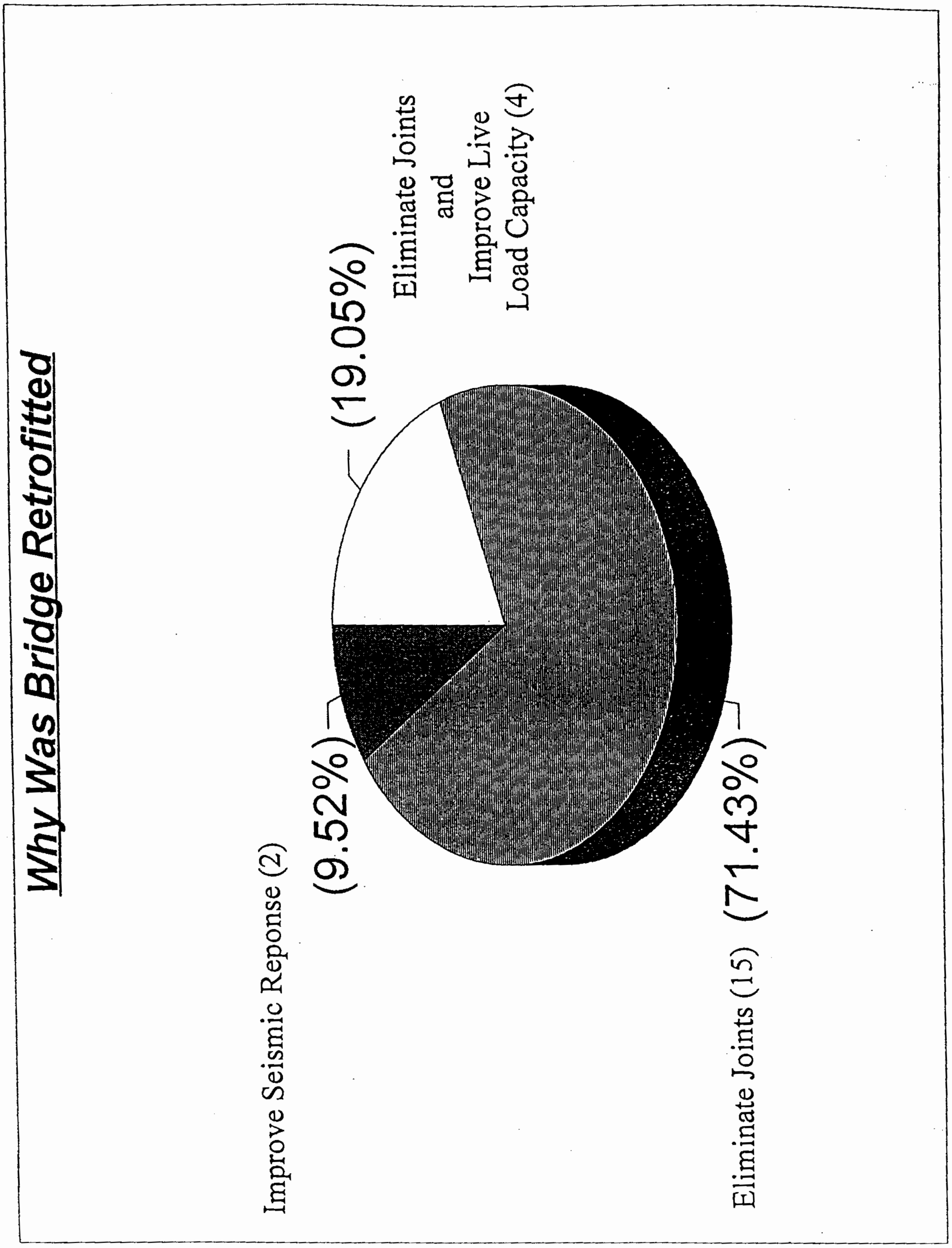


Figure 2.10 - Why Was Bridge Retrofitted

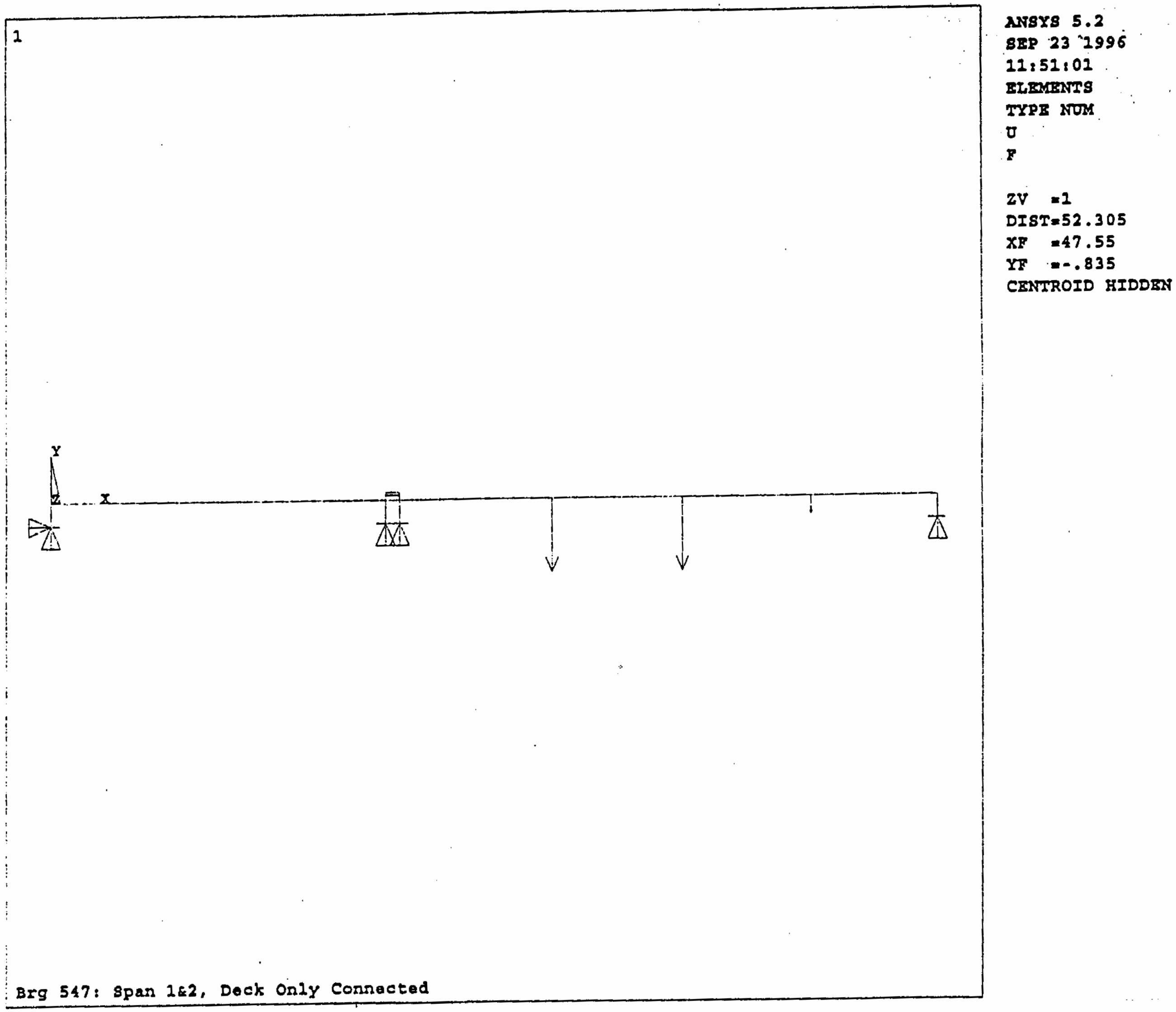


Figure 3.1.1 - Bridge #547: Span 1 and 2 Model

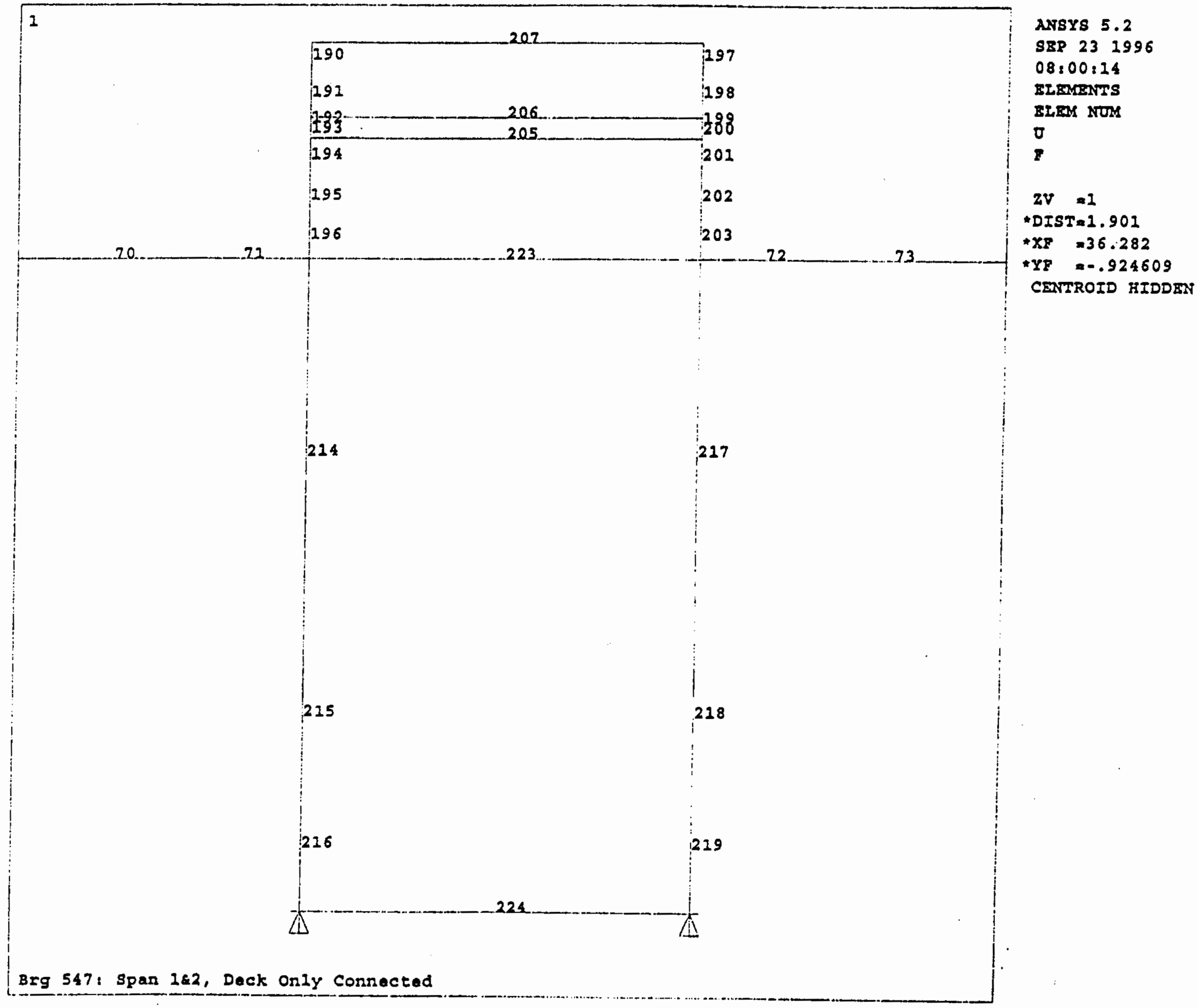


Figure 3.1.2 - Bridge #547: Span 1 and 2 Connection Detail

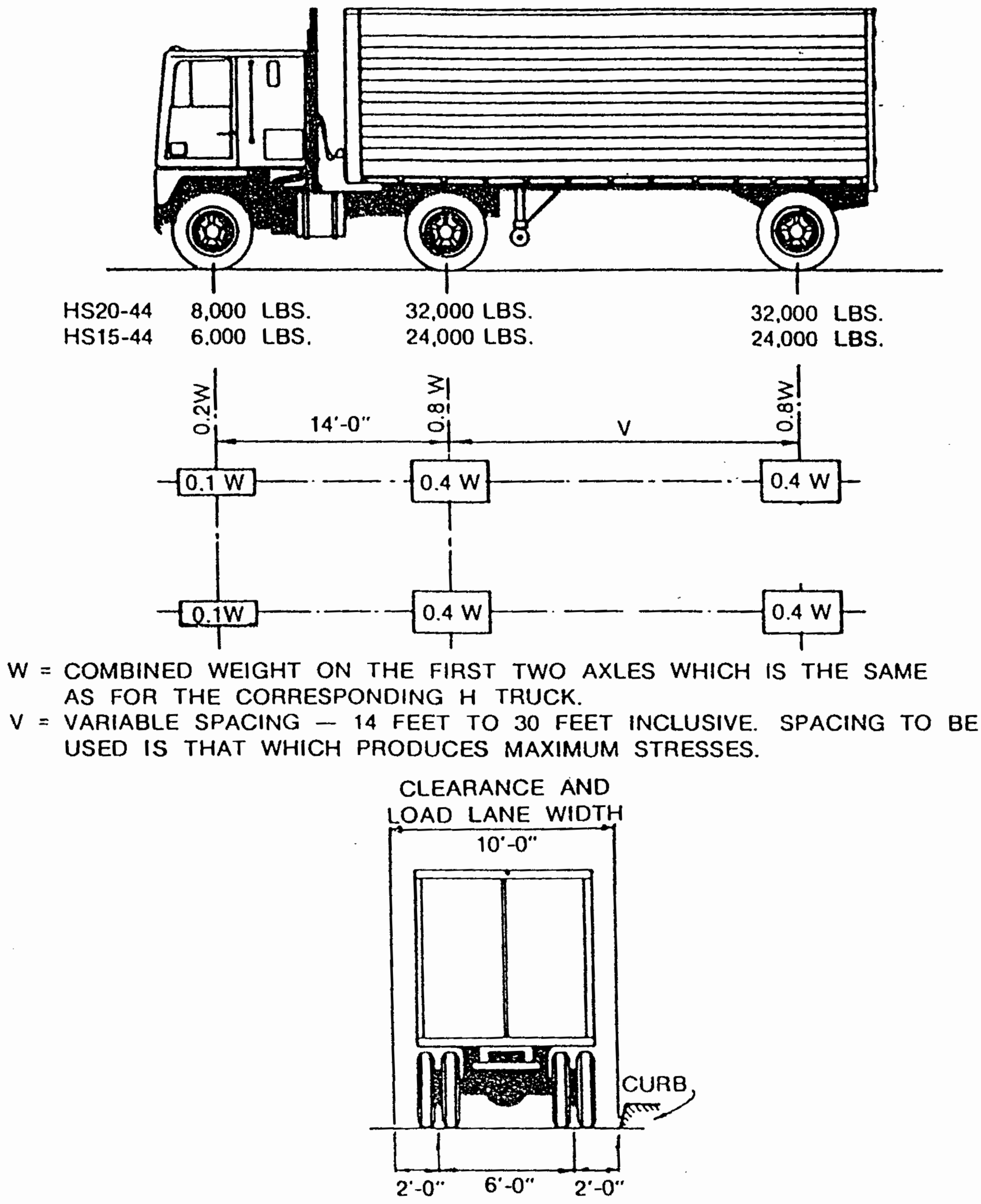
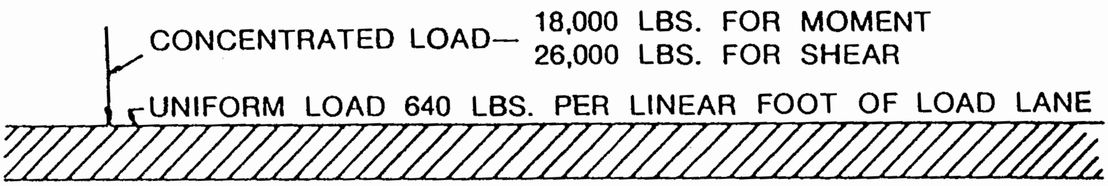
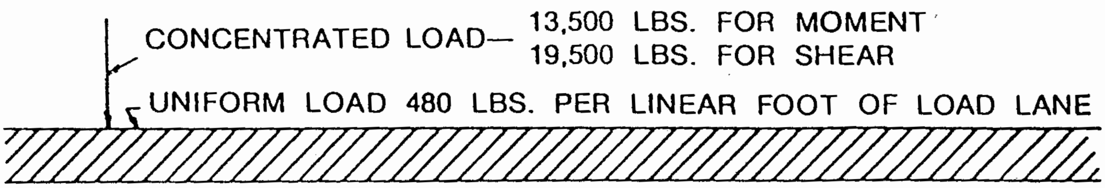


Figure 3.1.3 - AASHTO HS Truck Configuration



H20-44 LOADING
HS20-44 LOADING



H15-44 LOADING
HS15-44 LOADING

Figure 3.1.4 - AASHTO Lane Loading

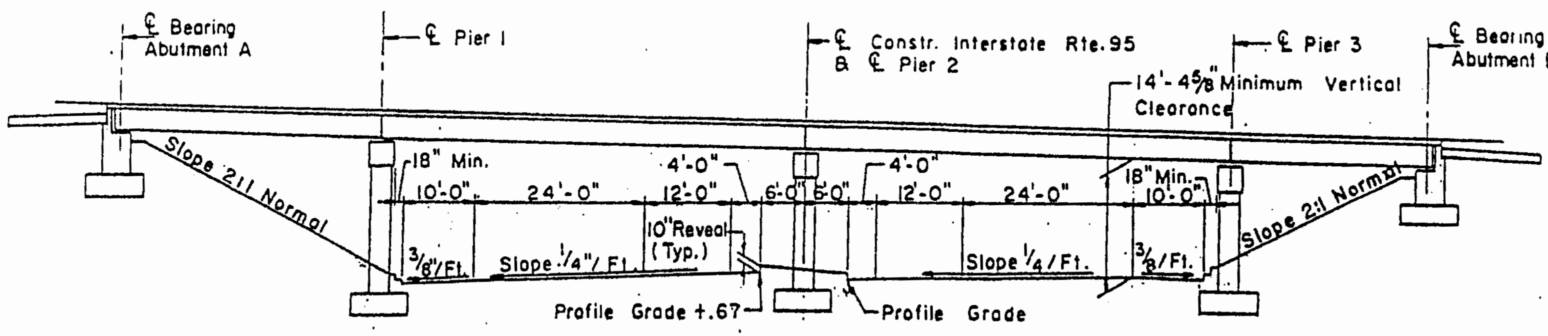


Figure 3.2.1 - Garden Street Bridge #547: Bridge Plan

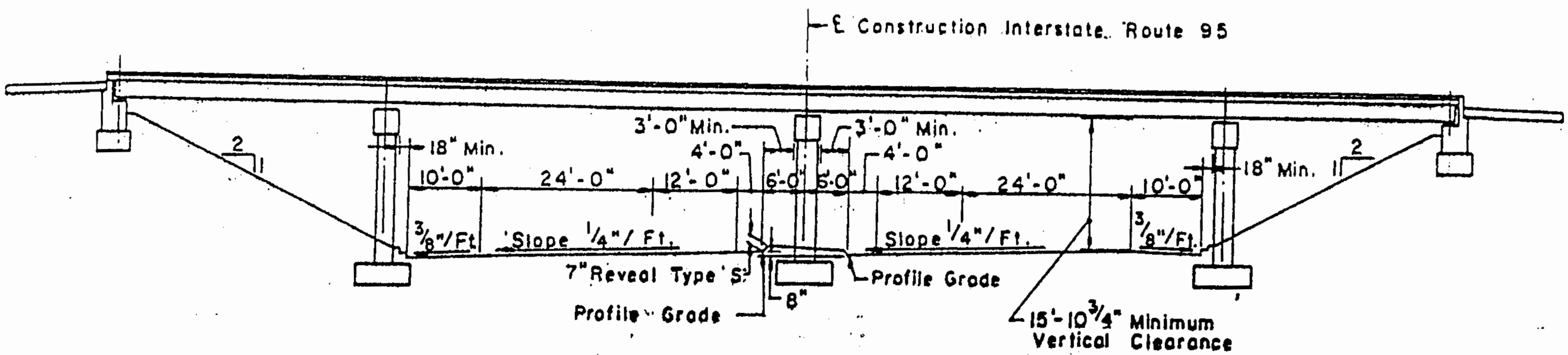


Figure 3.2.2 - Pine Street Bridge #548: Bridge Plan

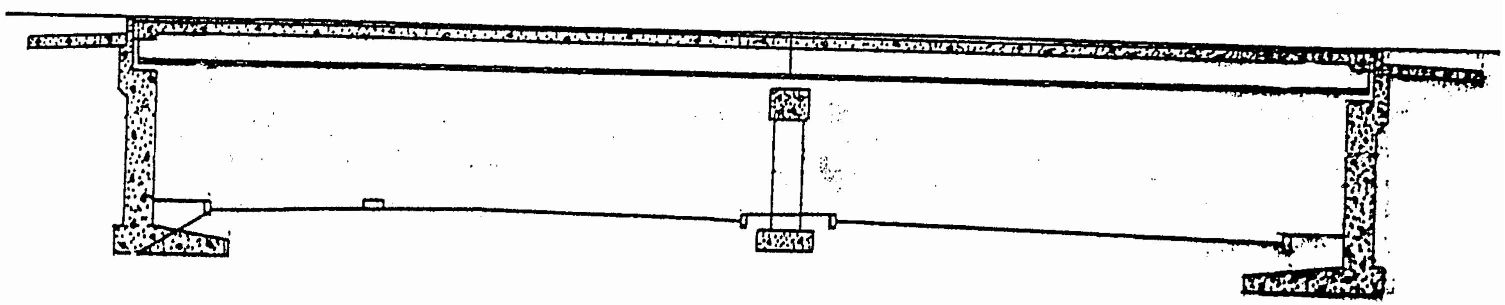


Figure 3.2.3 - Hartford Ave. - West Bridge #608: Bridge Plan

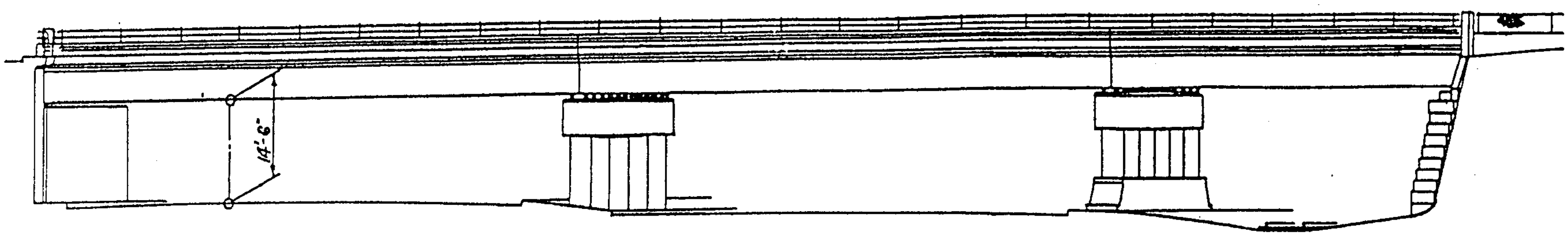


Figure 3.2.4 - Broad Steet - South Bridge #657: Bridge Plan

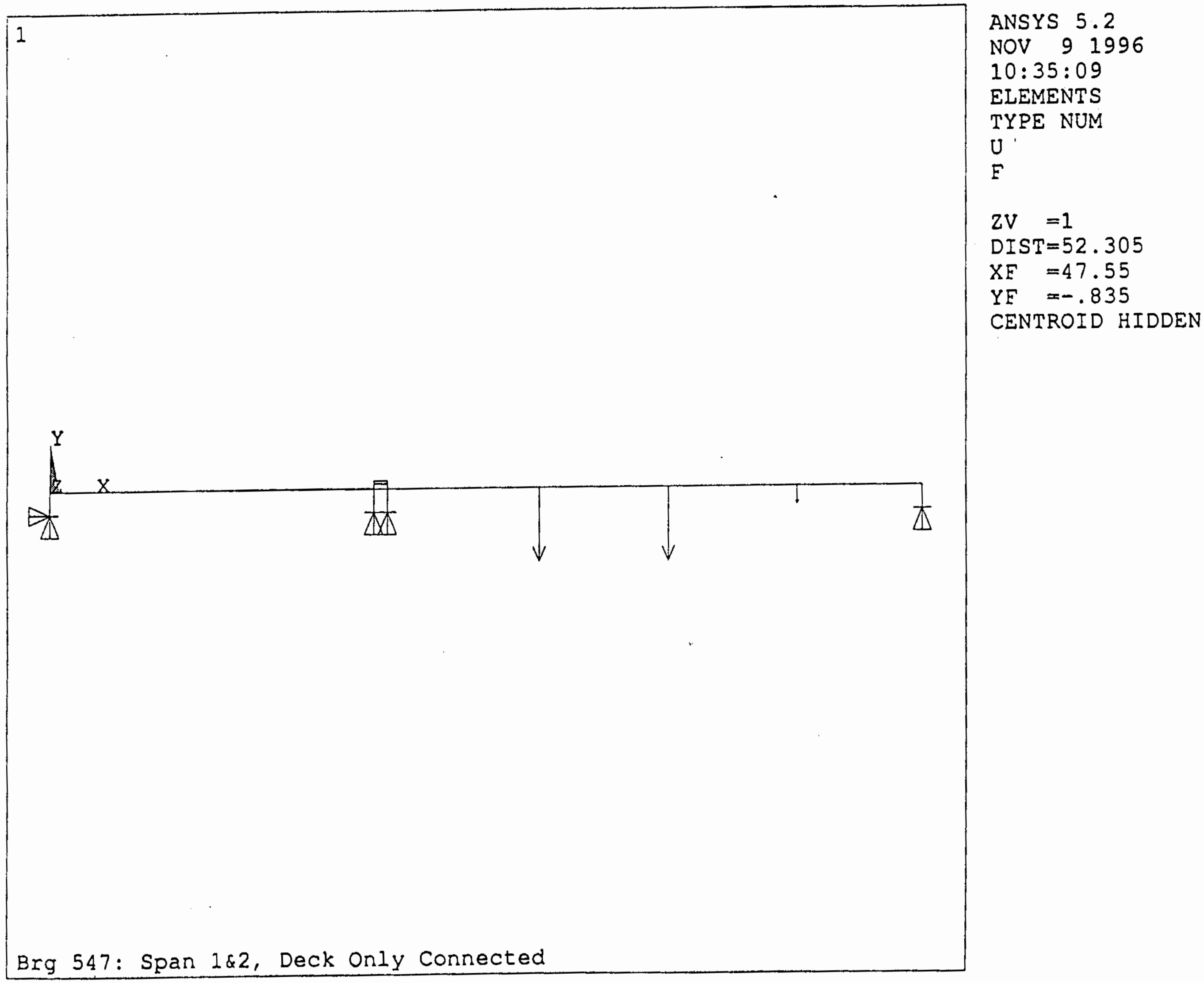


Figure 3.2.5 - Bridge #547: Span 1 and 2 - Model Plot

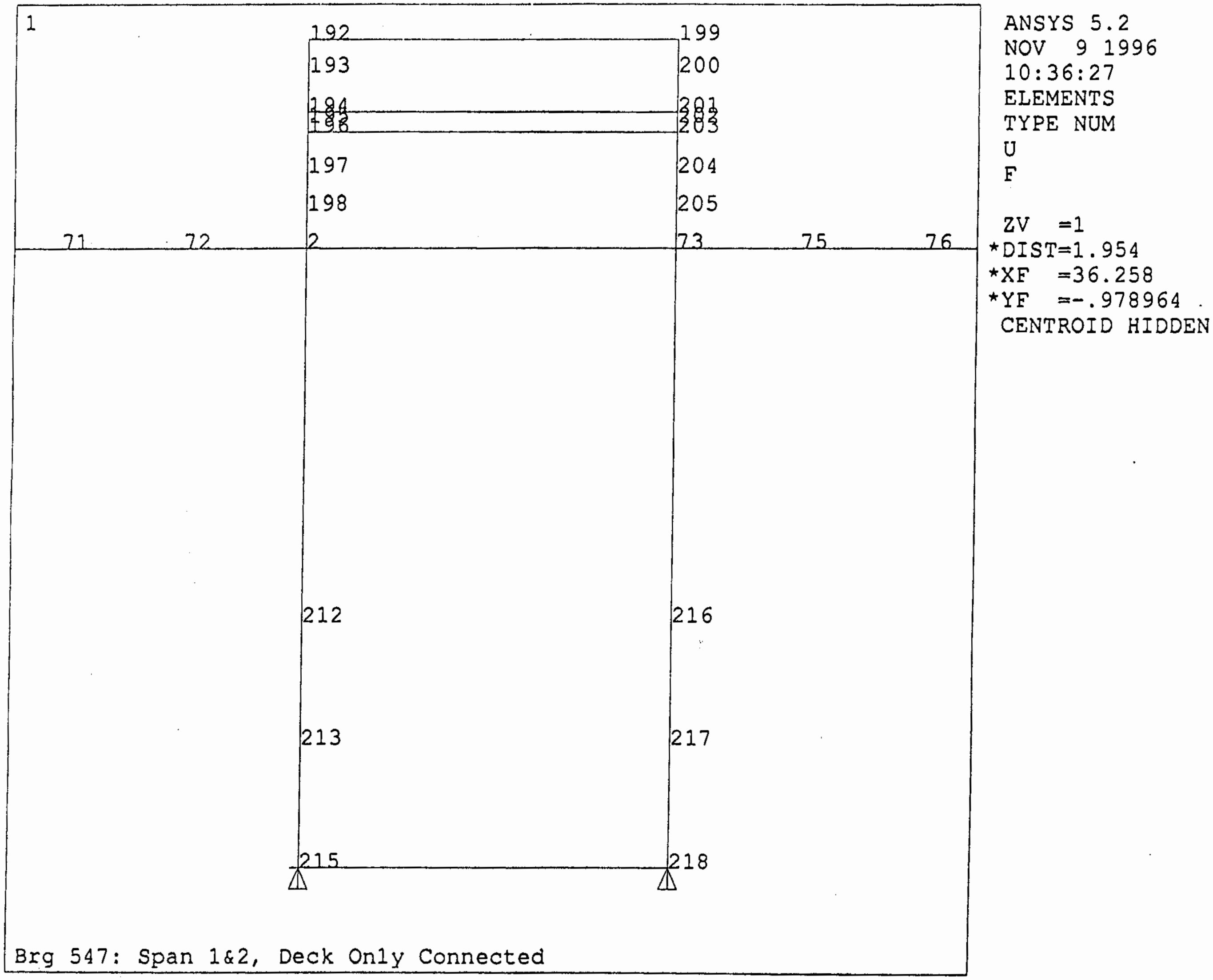


Figure 3.2.6 - Bridge #547: Spans 1 and 2 - Connection Detail Node Plot

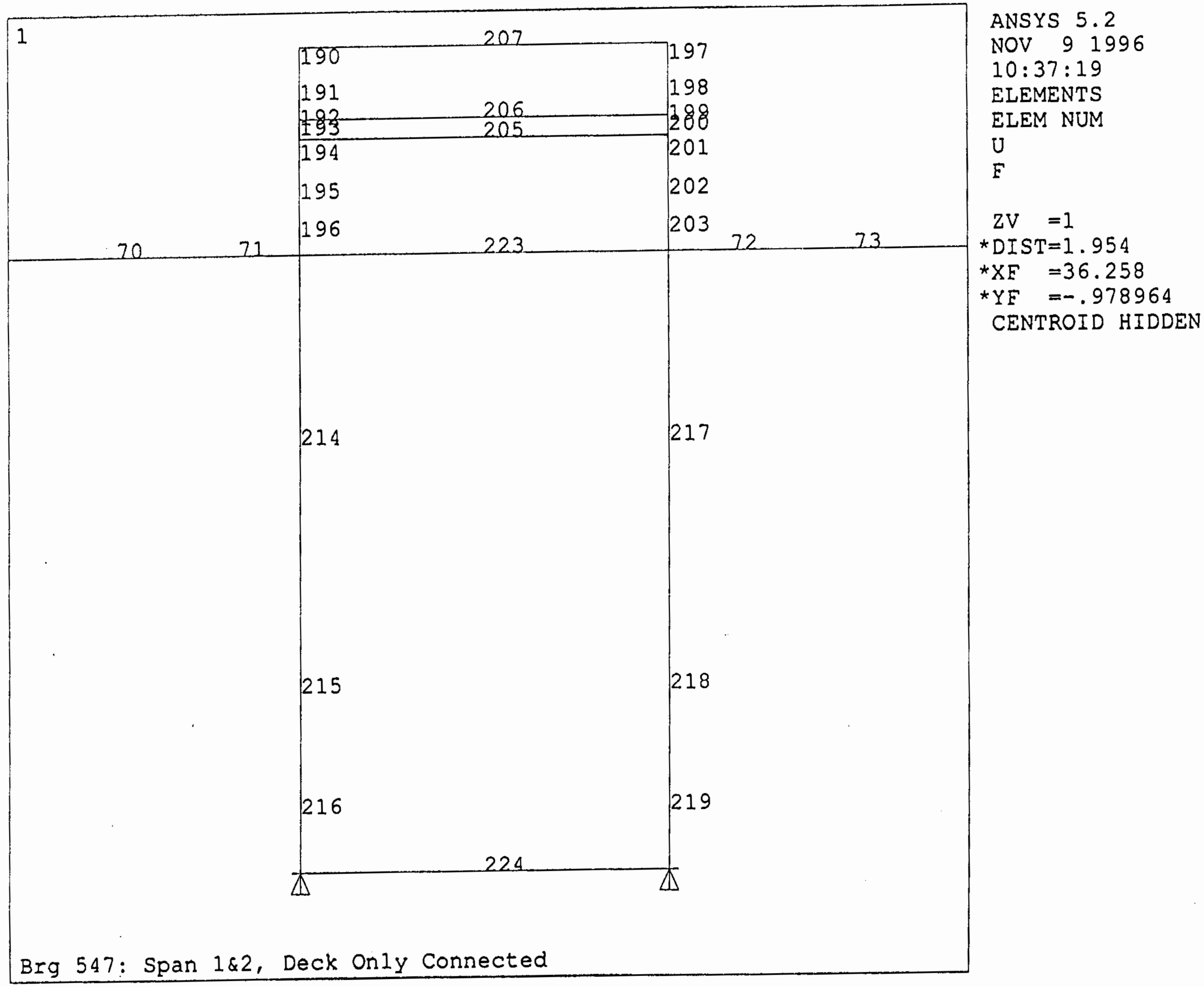


Figure 3.2.7 - Bridge #547: Spans 1 and 2 - Connection Detail Element Plot

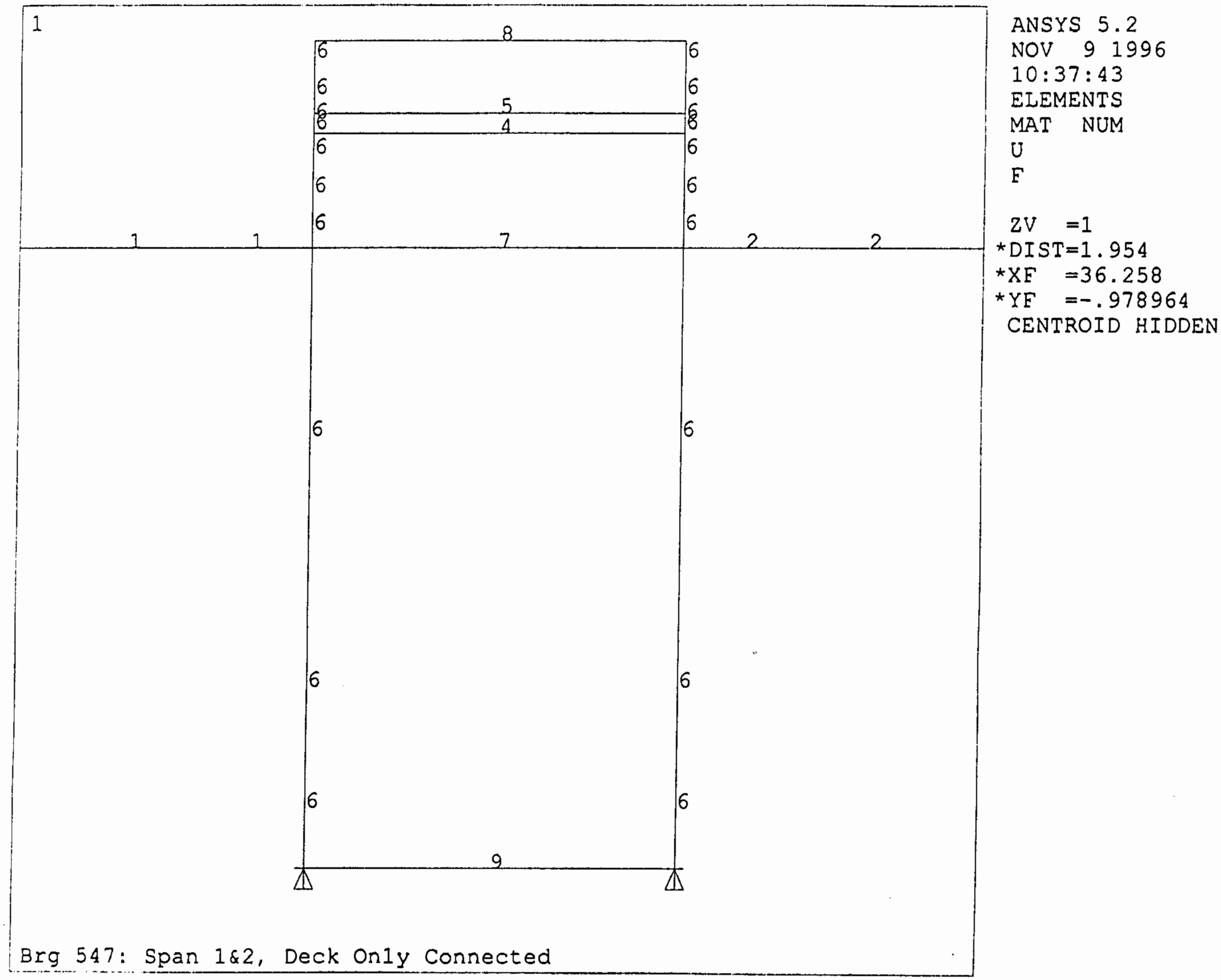


Figure 3.2.8 - Bridge #547: Spans 1 and 2 - Connection Detail Material Plot

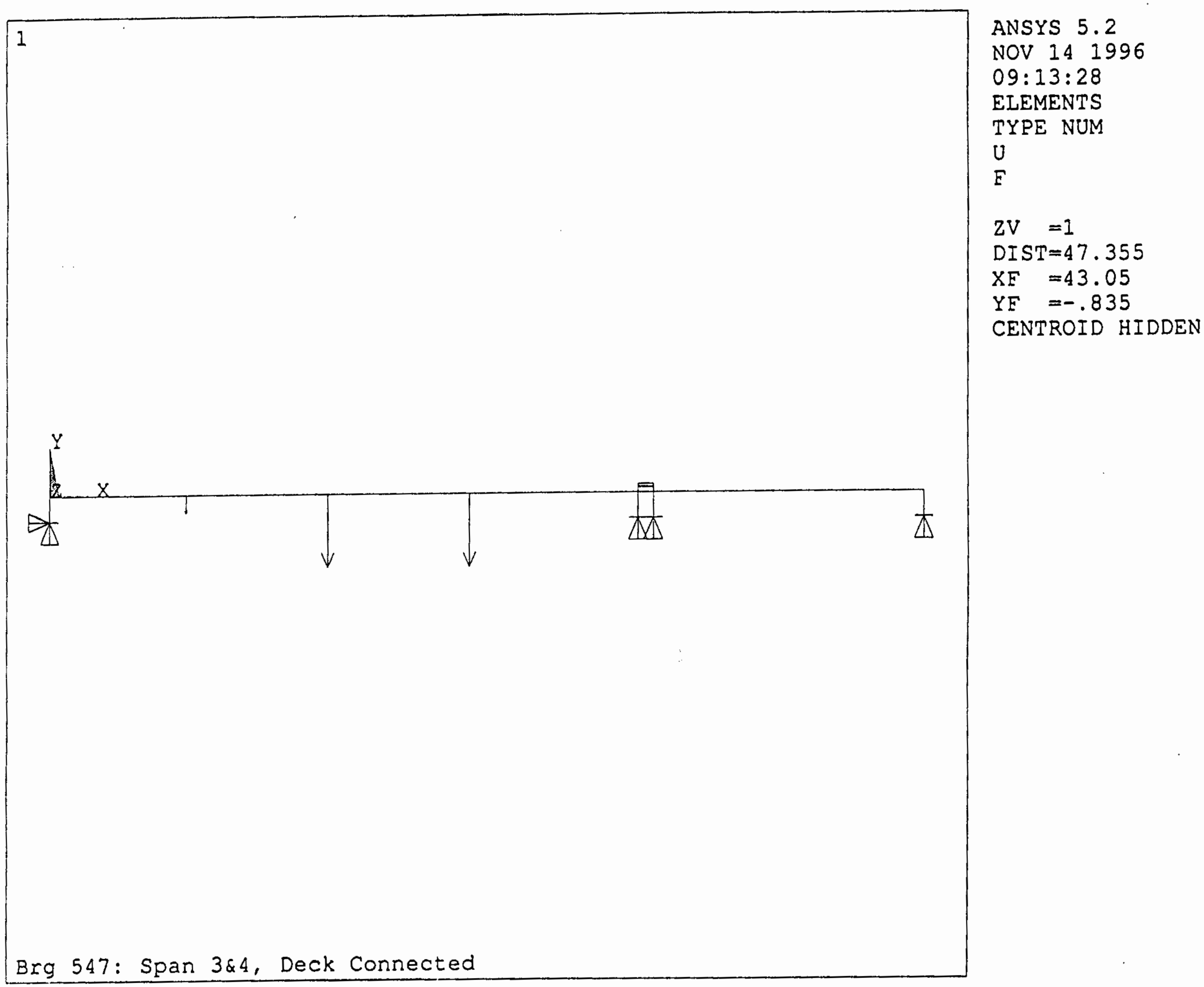


Figure 3.2.9 - Bridge #547: Spans 3 and 4 - Model Plot

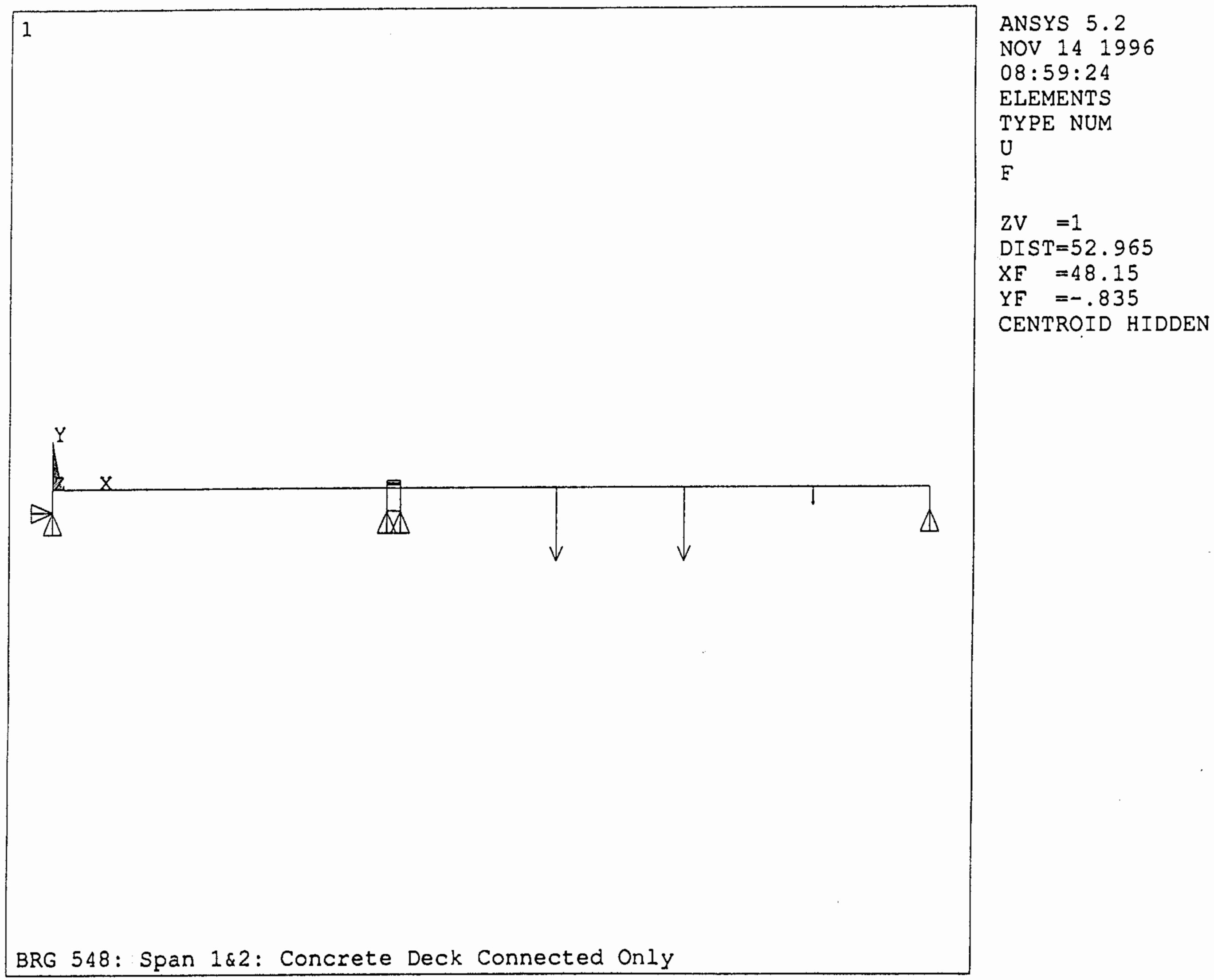


Figure 3.2.10 - Bridge #548: Spans 1 and 2 - Model Plot

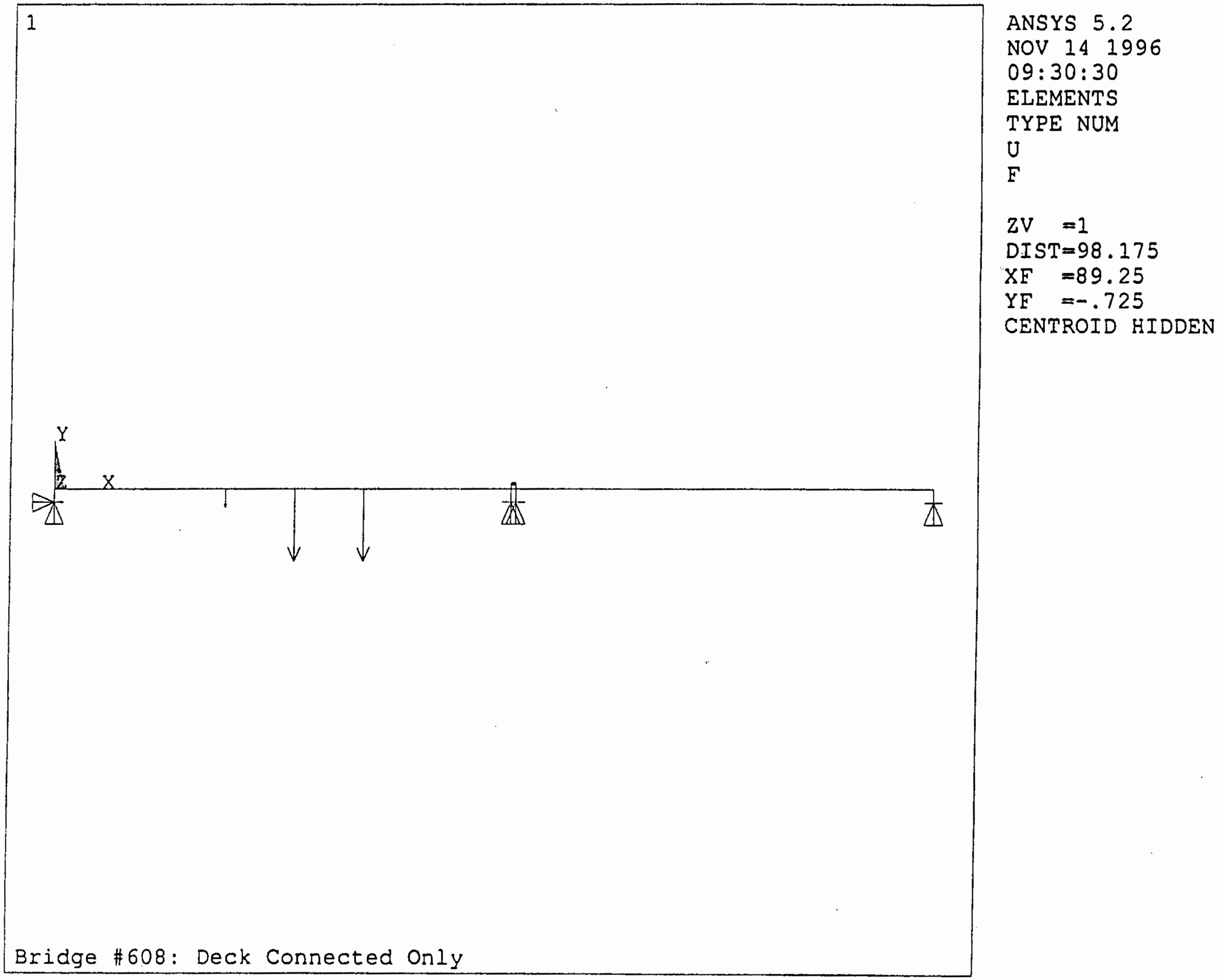


Figure 3.2.11 - Bridge #608 - Model Plot

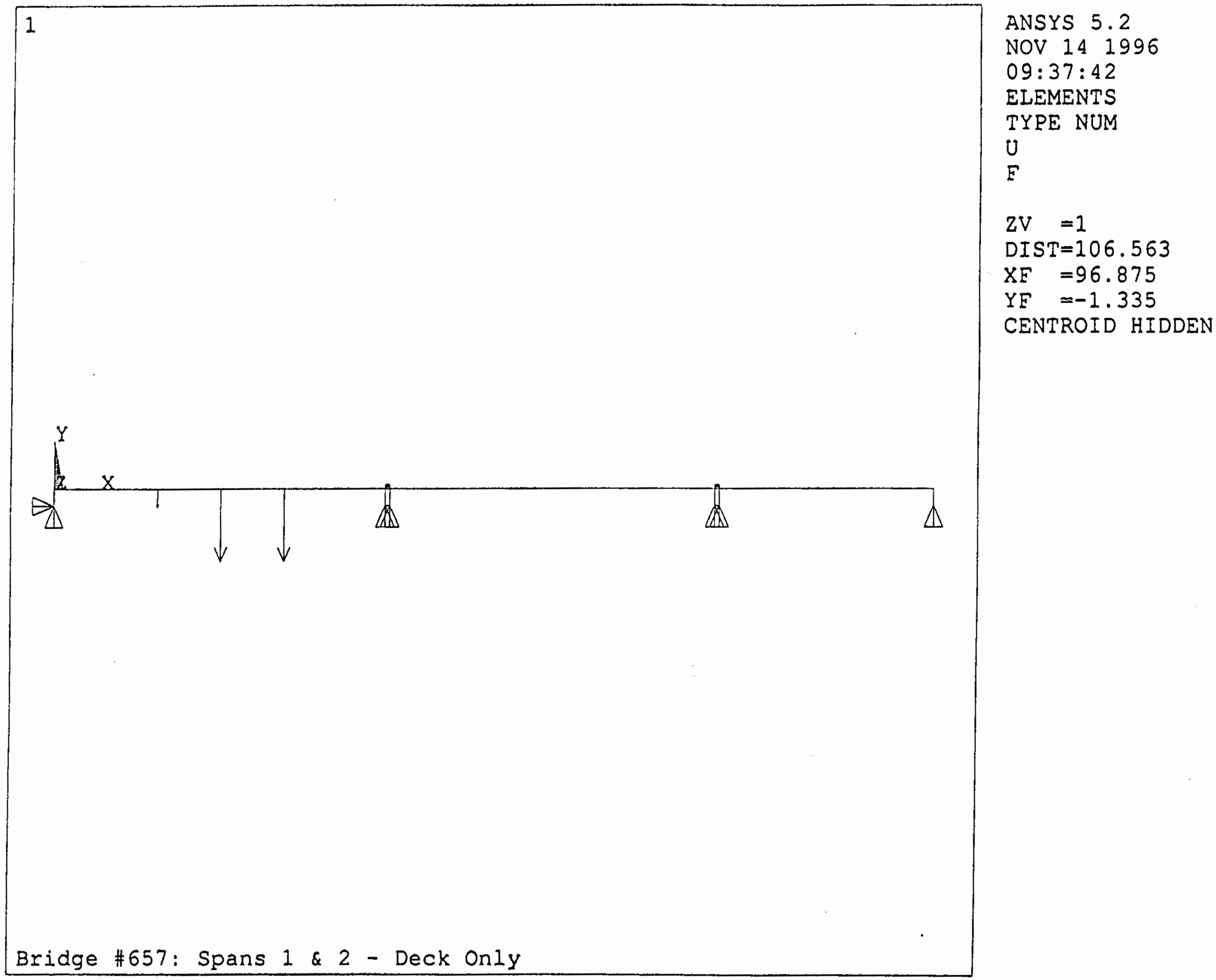


Figure 3.2.12 - Bridge #657: Spans 1 and 2 - Model Plot

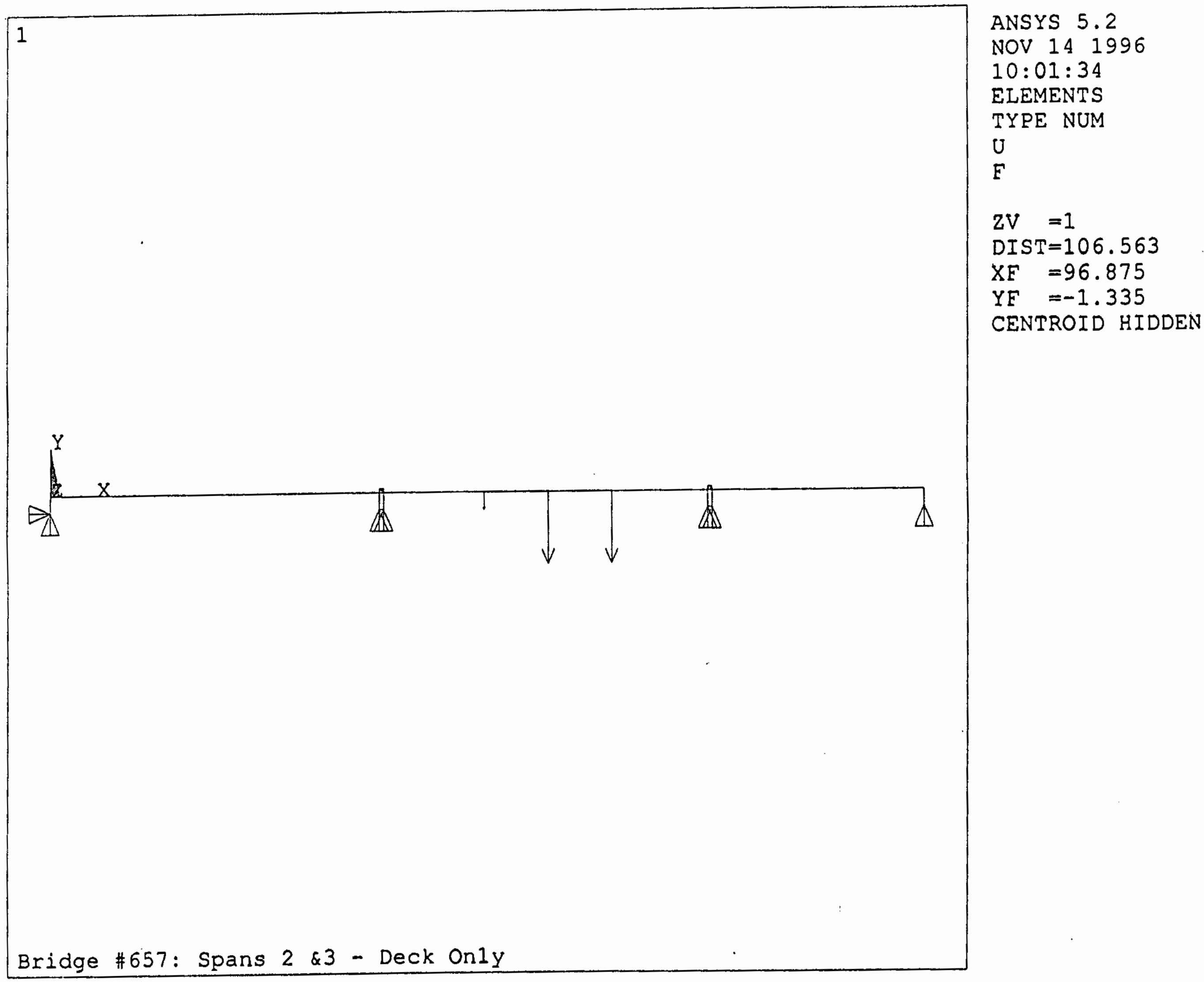
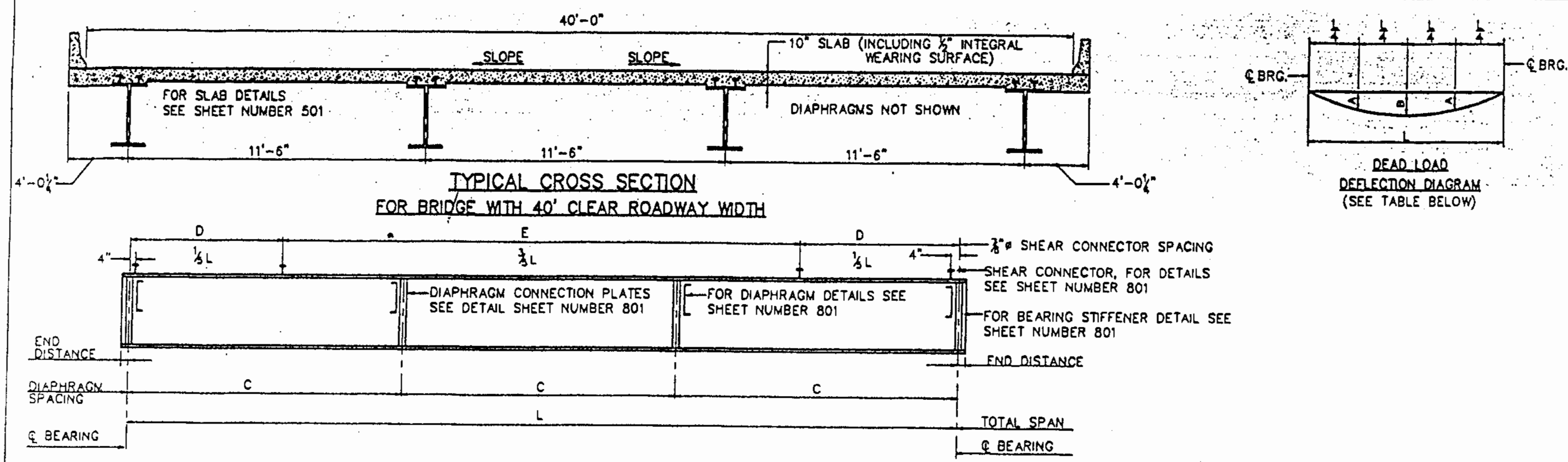


Figure 3.2.13 - Bridge #657: Spans 2 and 3 - Model Plot



COMPOSITE ROLLED BEAMS WITHOUT COVER PLATE

SPAN (L) - Ft.	REQUIRED BEAM	SIZE OF DIAPHRAGM	DIAPHRAGM SPACING (C) - Ft.	NUMBER OF DIAPHRAGMS	DEAD LOAD DEFLECTION - In.		SUPERIMPOSED DEAD LOAD DEFLECTION - In.		SHEAR CONNECTOR MAX. SPACING - In.		REACTION-Kips. (UNFACTORED)			MOMENTS Kip-Ft. (UNFACTORED)		
					A	B	A	B	D	E	DEAD LOAD	SUPERIMPOSED DEAD LOAD	LIVE LOAD WITH IMPACT	DEAD LOAD	SUPERIMPOSED DEAD LOAD	LIVE LOAD WITH IMPACT
30	W 24x76	C 12x20.7	15.00	3	3/8	9/16	0	1/16	4	6	27.1	6.9	92.6	195.5	49.7	478.2
35	W 30x90	C 15x33.9	17.50	3	3/8	9/16	1/16	1/16	5	7	31.7	8.1	94.1	266.1	67.6	608.6
40	W 30x108	C 15x33.9	20.00	3	9/16	13/16	1/16	1/16	5	6	36.6	9.2	98.2	366.0	92.0	747.2
45	W 33x130	C 15x33.9	22.50	3	5/8	7/8	1/16	1/16	5	7	42.1	10.4	100.9	473.3	116.4	895.9
50	W 36x135	MC 18x42.7	25.00	3	13/16	1 3/16	1/16	1/8	5	7	47.0	11.5	102.7	587.5	143.8	1041.3
55	W 36x160	MC 18x42.7	18.33	4	15/16	1 3/8	1/8	1/8	5	7	52.2	12.6	104.1	718.4	173.9	1185.1
60	W 36x182	MC 18x42.7	20.00	4	1 3/16	1 11/16	1/8	3/16	5	7	57.3	13.8	105.2	859.5	207.0	1327.4
65	W 36x210	MC 18x42.7	21.67	4	1 7/16	2	3/16	1/4	5	7	62.7	14.9	106.0	1019.3	242.9	1468.5
70	W 36x230	MC 18x42.7	23.33	4	1 11/16	2 7/16	3/16	1/4	5	7	68.6	16.1	106.7	1200.5	281.8	1608.4
75	W 36x260	MC 18x42.7	25.00	4	2	2 13/16	1/4	5/16	5	7	74.6	17.3	107.2	1399.2	323.4	1747.1
80	W 36x300	MC 18x42.7	20.00	5	2 3/16	3 1/8	1/4	3/8	6	7	81.2	18.4	107.6	1624.0	368.0	1884.8
85	W 36x328	MC 18x42.7	21.25	5	2 9/16	3 11/16	5/16	7/16	6	7	87.6	19.5	107.9	1860.4	415.4	2021.6
90	W 36x359	MC 18x42.7	22.50	5	3	4 1/4	3/8	9/16	6	7	94.5	20.7	108.1	2126.3	465.8	2157.4
95	W 36x393	MC 18x42.7	23.75	5	3 3/8	4 13/16	7/16	5/8	6	7	99.8	21.9	108.3	2369.1	518.9	2292.5

NOTES:
1. FOR BEARINGS AND END DISTANCES SEE SHEET NUMBER 807.
2. THE DEAD LOAD IS THE ADDITION OF THE LOADS IMPOSED BY THE BEAM/GIRDER, THE CONCRETE DECK SLAB, THE STAY-IN-PLACE DECK FORM AND THE CONCRETE HAUNCH. THE SUPERIMPOSED DEAD LOAD IS THE ADDITION OF THE LOADS IMPOSED BY THE CONCRETE PARAPETS AND THE FUTURE WEARING SURFACE.

STATE OF MARYLAND
CHARLES ROBERT KENNEDY
REGISTERED PROFESSIONAL ENGINEER

AISI SHORT SPAN
STEEL BRIDGES

COMPOSITE ROLLED
BEAMS WITHOUT COVER PLATE
NORMAL WEIGHT CONCRETE

DATE: SEPTEMBER, 1994	DO NOT SCALE	HS 25-44 LOADING	
DRAFTED BY: S.L.P.	DESIGNED BY: S.T.	CHECKED BY: C.R.K.	40'-0" ROADWAY 504

Figure 3.3.1 - AISI Plan: Composite Rolled Beams without Cover Plate / Normal Weight Concrete

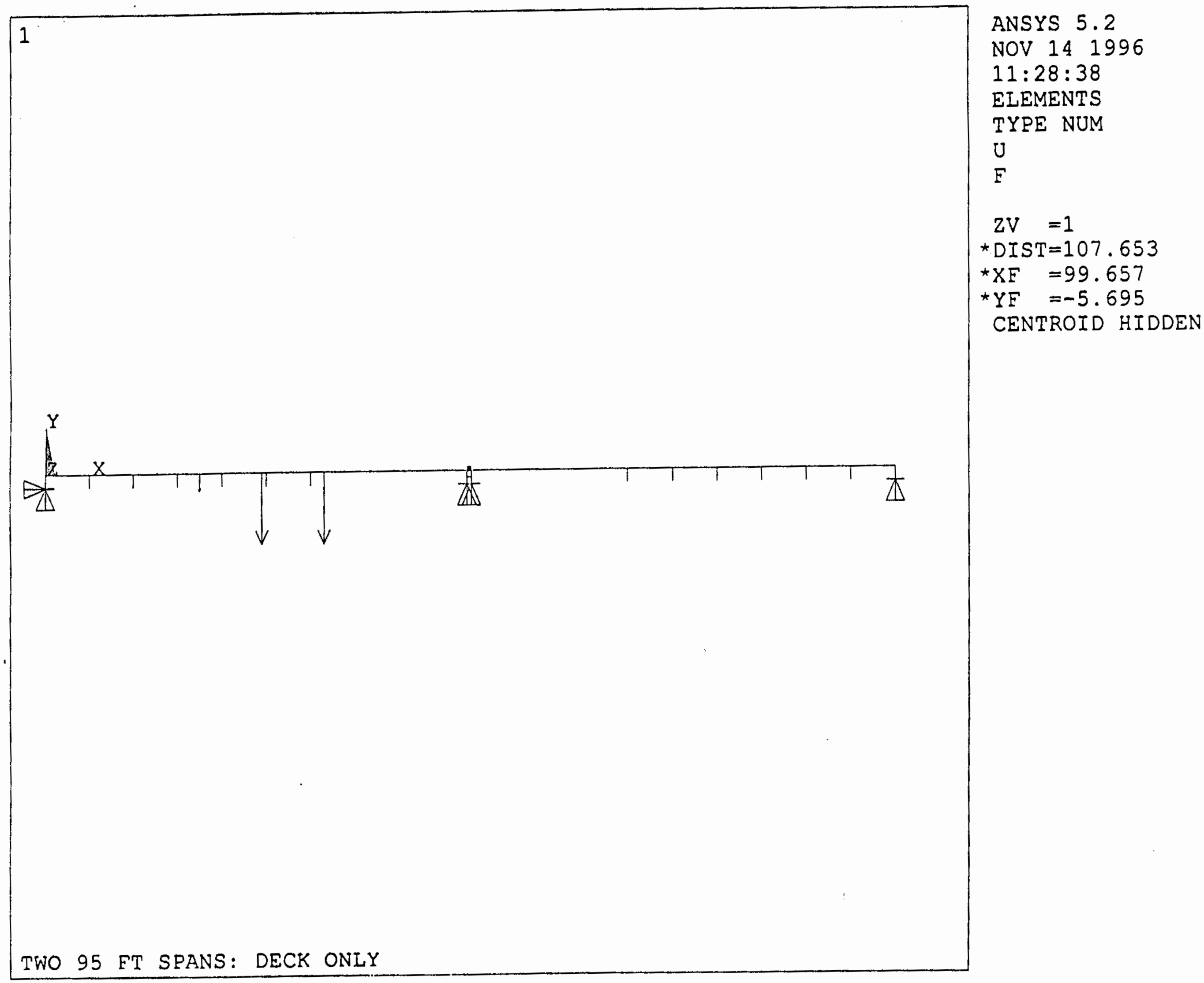


Figure 3.3.2 - 95' - 95' Model

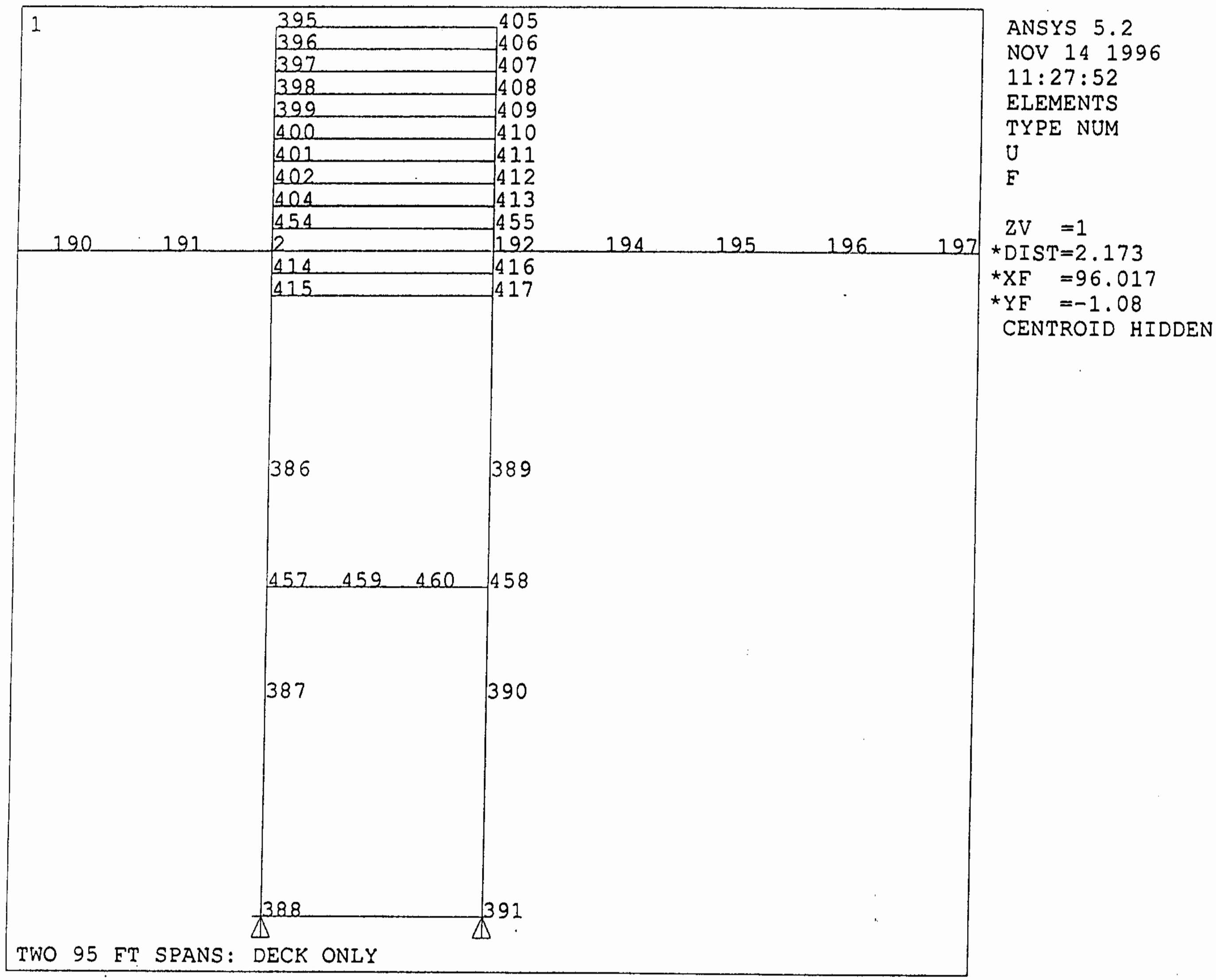


Figure 3.3.3 - 95' - 95' Model: Connection Detail - Node Numbers

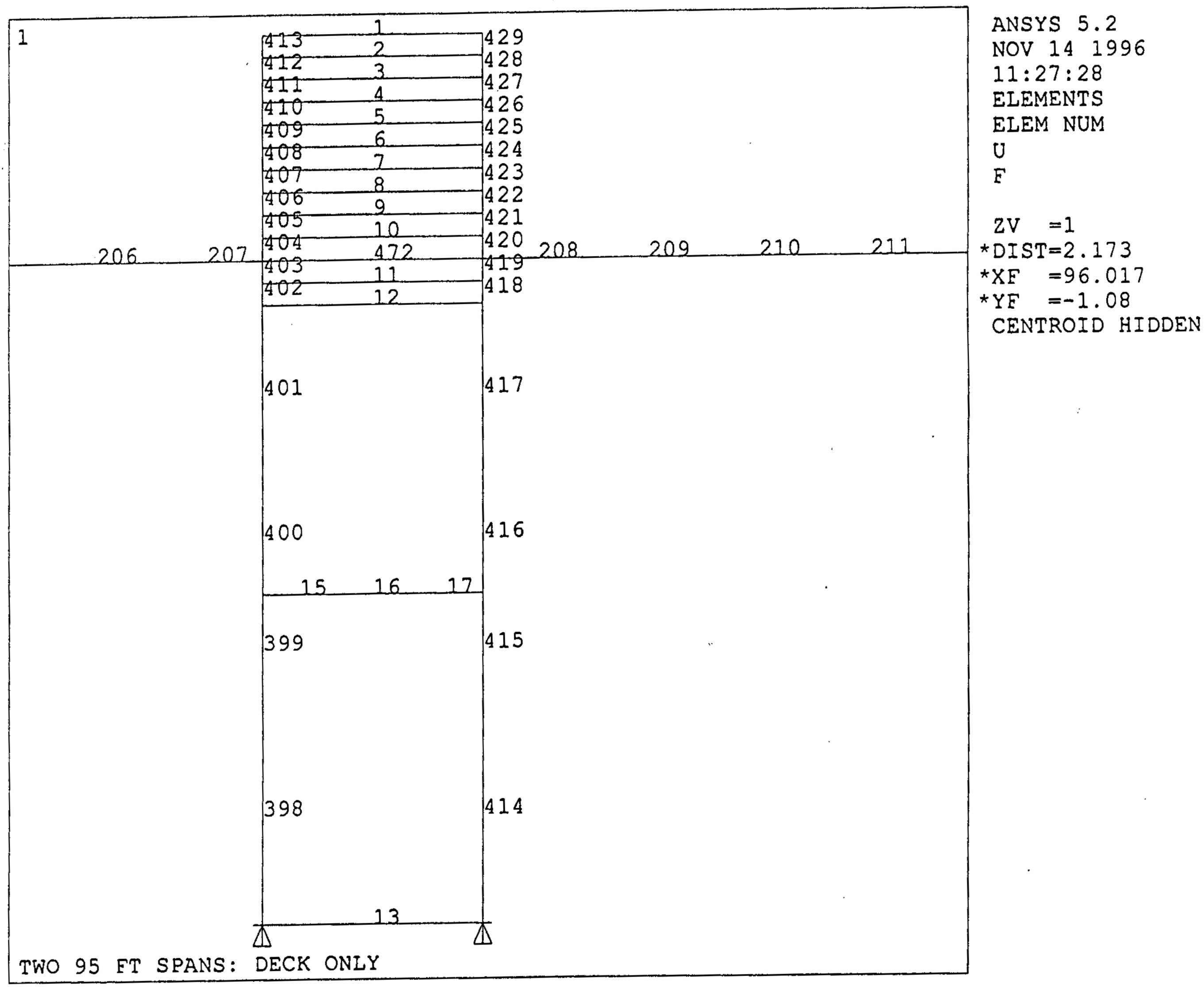


Figure 3.3.4 - 95' - 95' Model: Connection Detail - Element Numbers

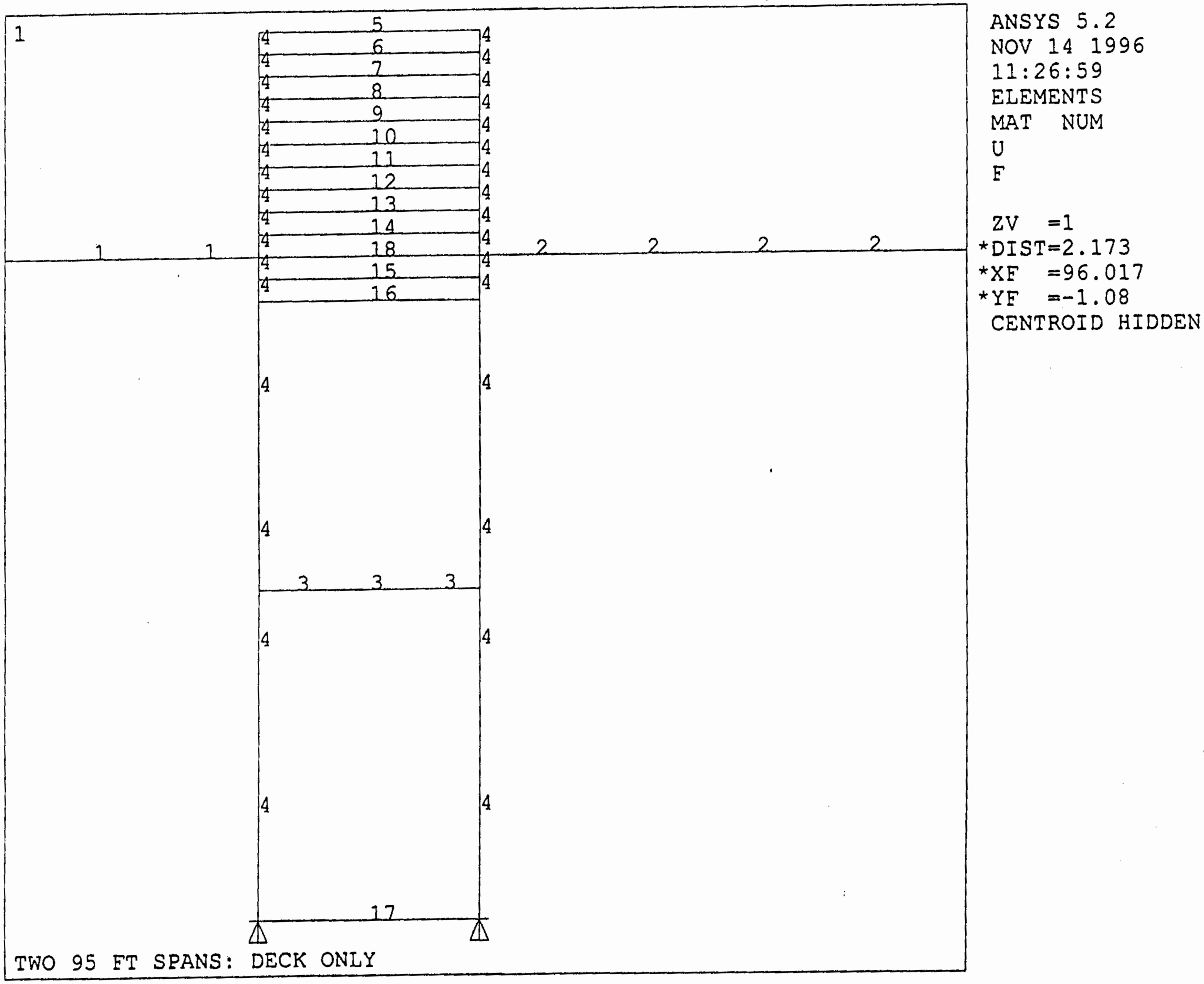


Figure 3.3.5 - 95' - 95' Model: Connection Detail - Material Numbers

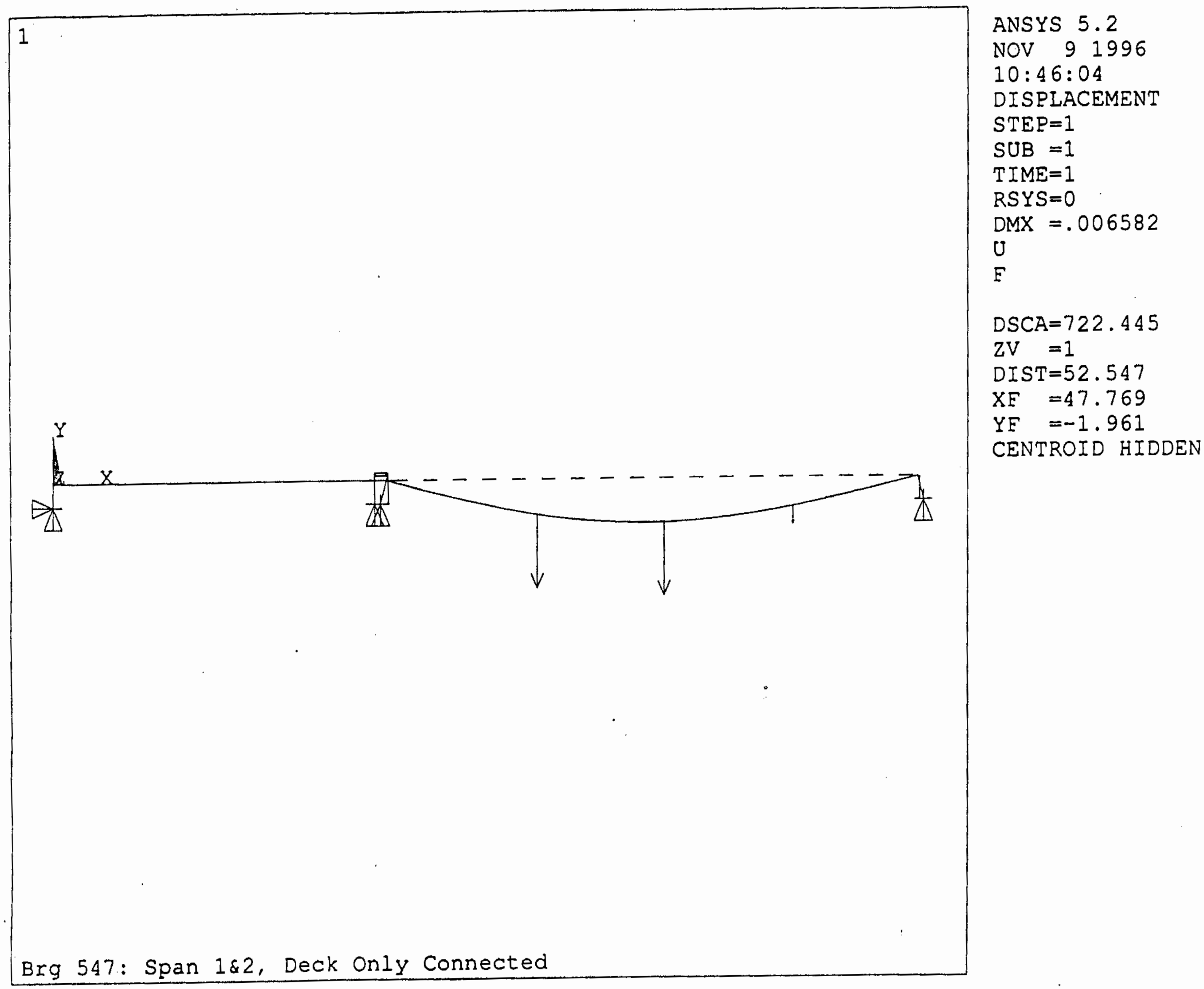


Figure 4.2.1 - Bridge #547: Span 1 & 2 - Deck Only Connection Scheme Deformation

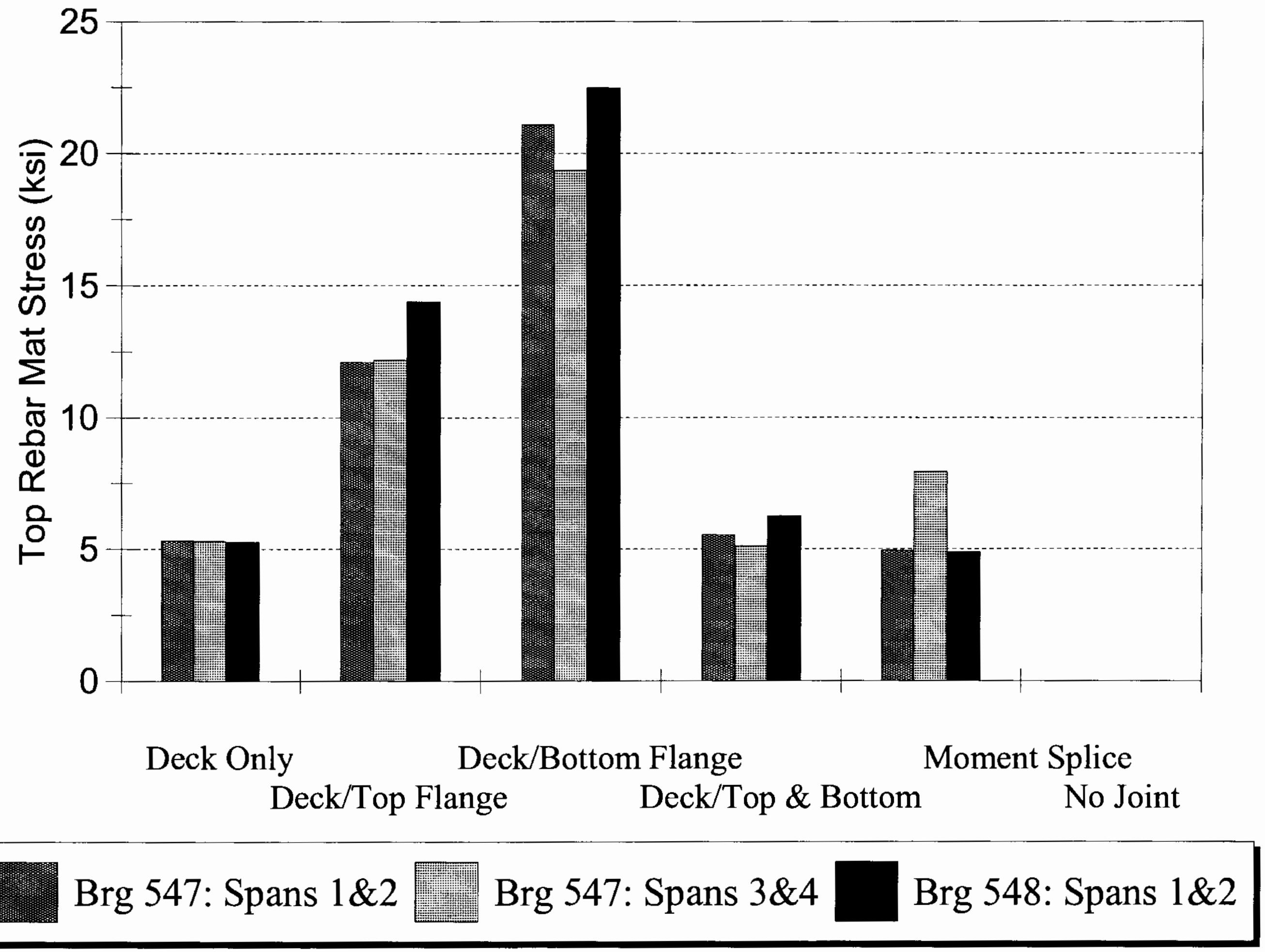


Figure 4.2.2 - Bridge Nos. 547 & 548: Top Mat Reinforcement Stress

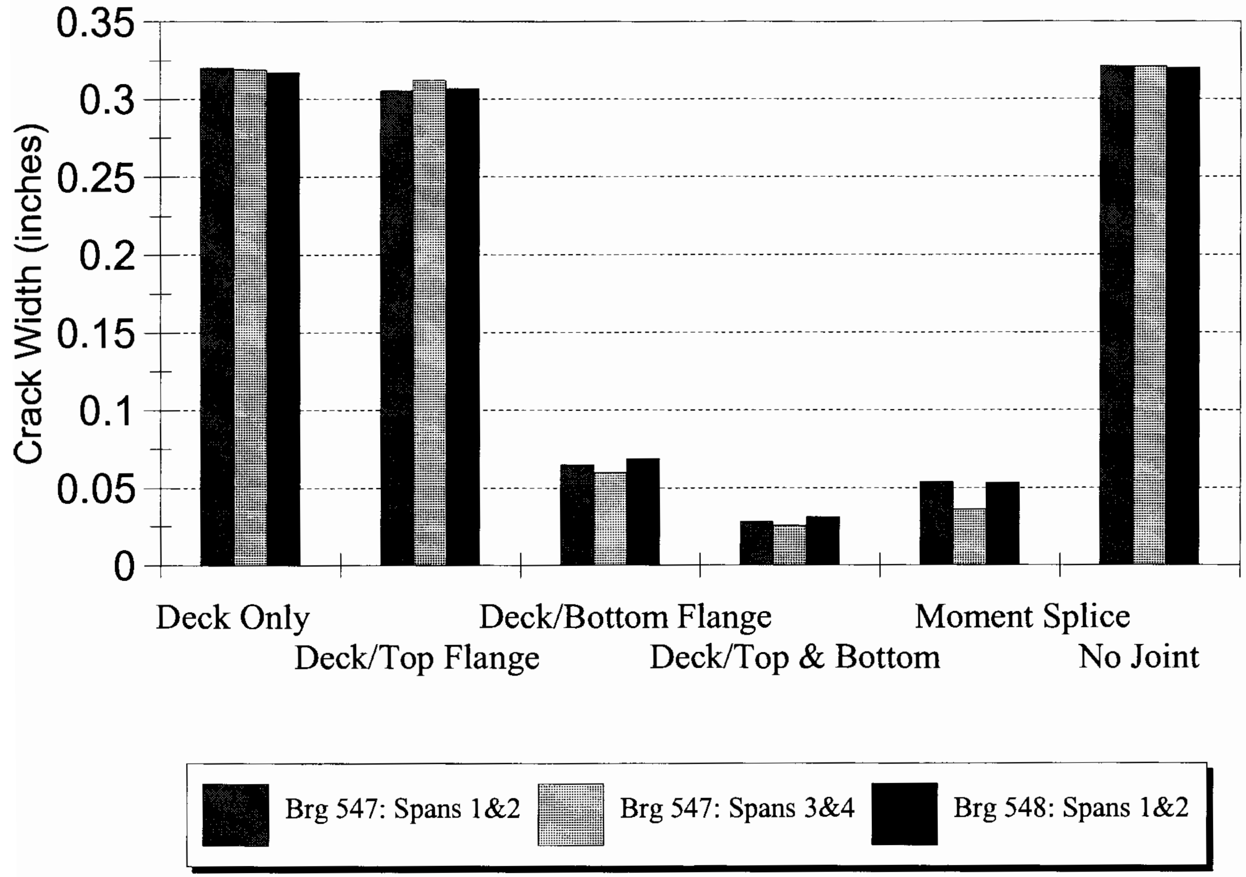


Figure 4.2.3 - Bridge Nos. 547 & 548: Crack Width

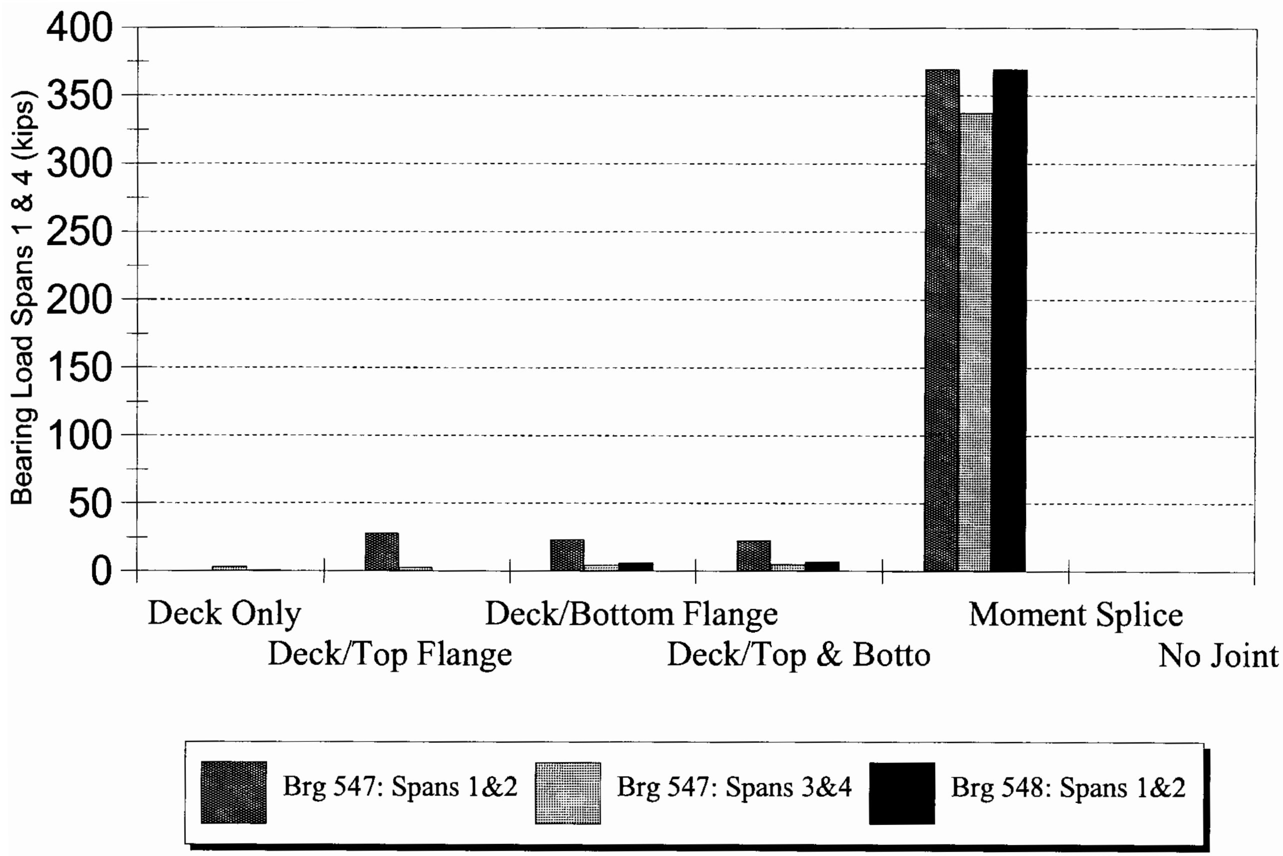


Figure 4.2.4 - Bridge Nos. 547 & 548: Bearing Load

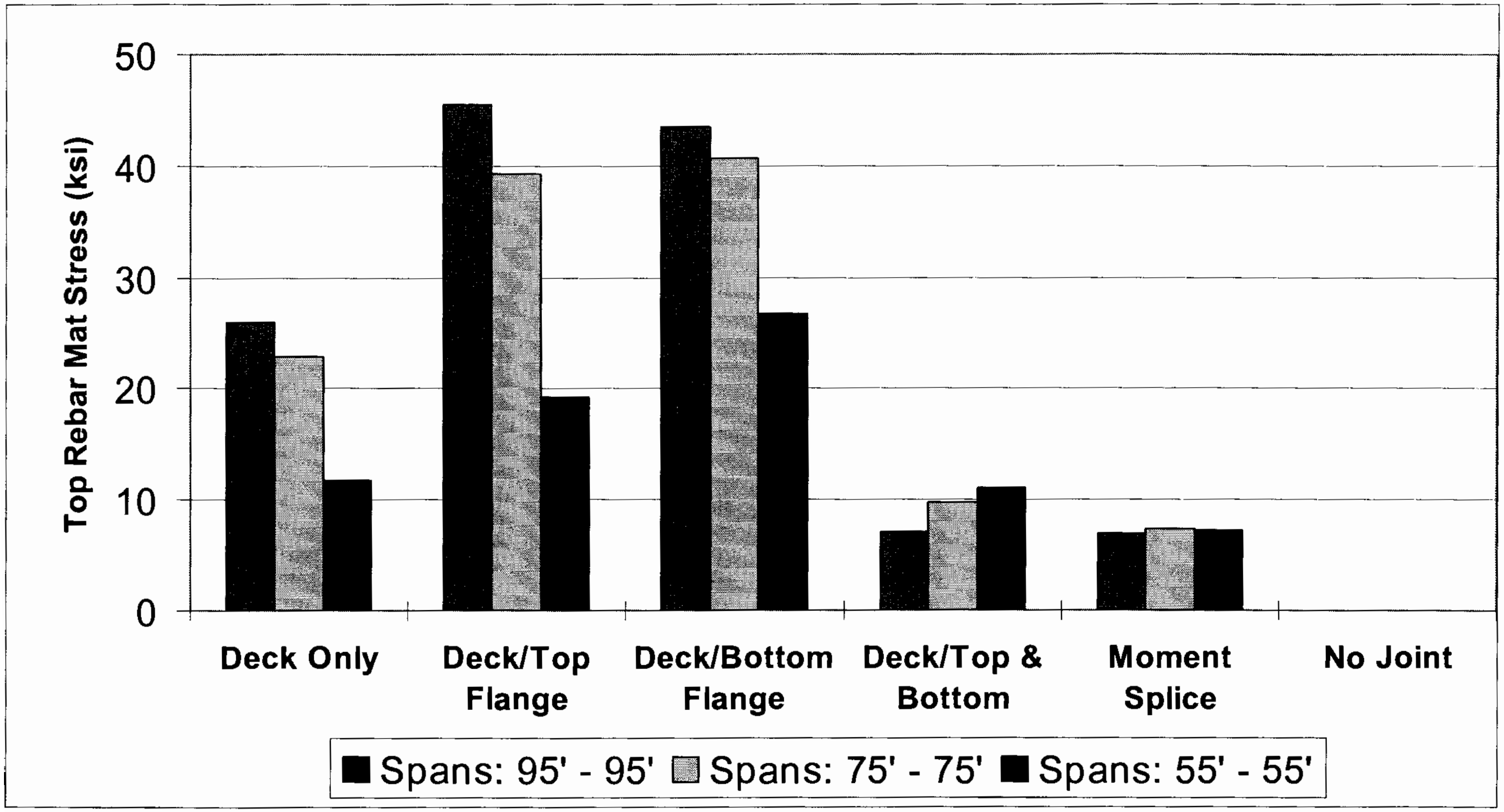


Figure 4.3.1 - Parametric Model - Equal Spans: Top Mat Reinforcement Stress

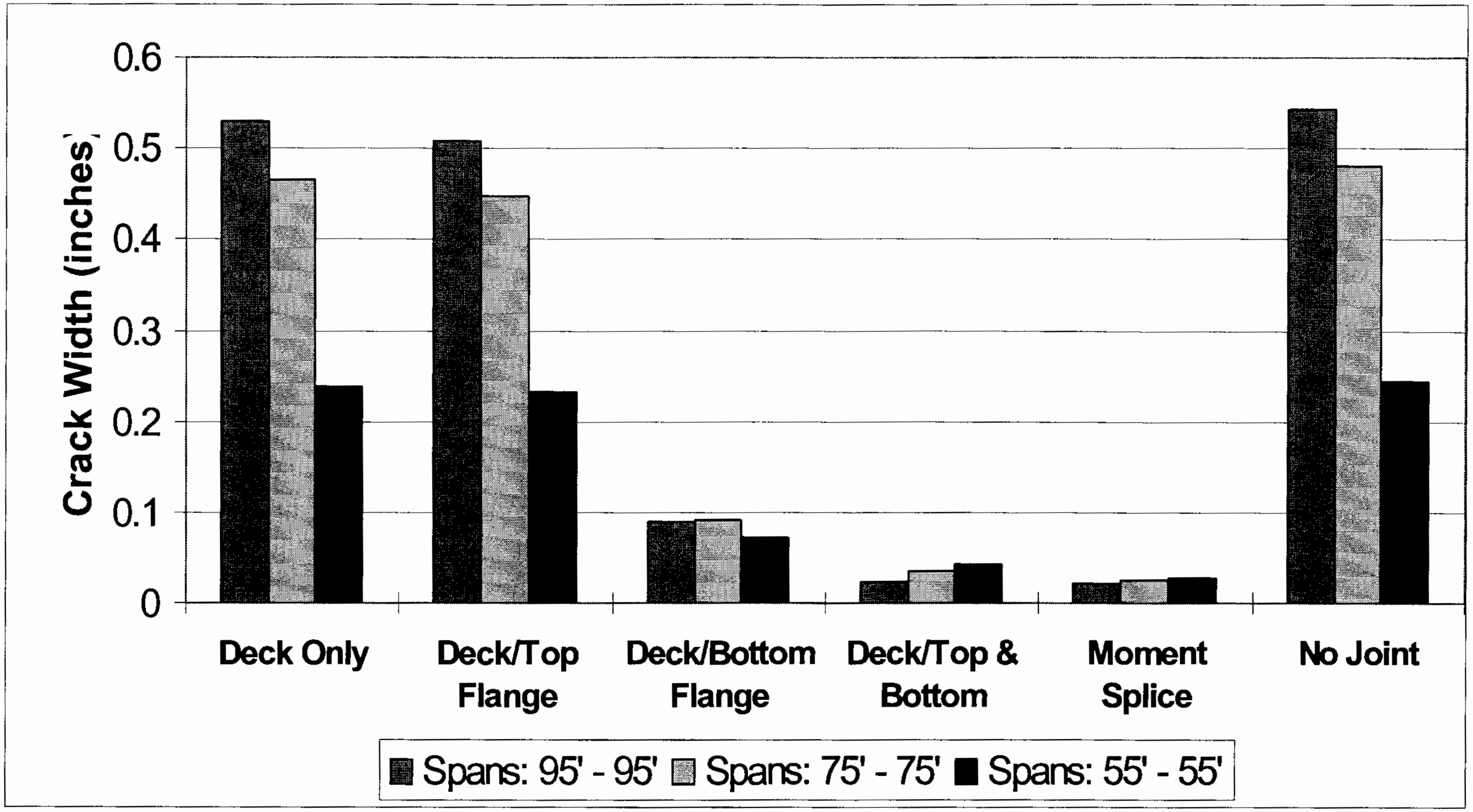


Figure 4.3.2 - Parametric Model - Equal Spans: Crack Width

APPENDIX A: NORTH AMERICAN SURVEY

CONTINUOUS BRIDGE DECK RETROFIT
OF MULTI-SIMPLE SPAN STEEL BRIDGES

Questionnaire

July 20, 1995

Name: ____________________
Title: ____________________
Organization: ____________________
Address: ____________________
City/Town: ____________________ State: ______ Zip Code: __________
Phone: (______) ______________ Fax:(______) ______________

Please return this questionnaire to:

William G. Boardman, P.E.
Project Engineer/Bridge Engineering
Rhode Island Department of Transportation
Providence, RI 02903-1124
Phone: (401)277-2053 X4068 Fax: (401)277-1271

The Rhode Island Department of Transportation is working with the University of Rhode Island to determine the state-of-the-art in continuous bridge deck retrofit. The State currently has a consultant under contract to design a retrofit scheme that will eliminate joints in several multi-simple span steel bridges. The design calls for connecting the top flanges and pouring a continuous deck where an expansion joint now exists. Bearing replacement will also be required. The State is contemplating using a joint elimination scheme on other bridges and is interested in learning more about the experiences of other agencies in this matter.

Do you wish to have the results of this questionnaire mailed back to you (Yes/No)? ________

PLEASE MAIL OR FAX THE RESULTS OF THIS QUESTIONNAIRE BY SEPTEMBER 30, 1995.

1. Has your organization ever been involved with the design or construction of retrofitting EXISTING multi-simple span STEEL bridges made continuous or semi-continuous by joining two or more adjacent simple spans (Yes/No)? ________

   If not, please return this questionnaire because this response is also <u>very</u> important and you may obtain a copy of the results of this survey. You may disregard the remaining questions. Thank you for your time.

2. How many multiple simple span steel bridges are within your area? Please define your area (state, county, province, etc.)? __________/______________________________

3. What is the approximate total number of steel bridges made continuous? __________

4. Approximately how many existing bridges have been made continuous within the last 5 years:
   Designed: __________ Constructed: __________

5. What is the maximum span length and structure length made continuous (please specify units)?
Deck Only: __________
Deck/Top Flange: __________
Deck/Top & Bottom Flange: __________
Deck/Bottom Flange: __________
Deck/Top & Bottom Flange and Web: __________
Deck/Other connection scheme (ie. link, please specify): ________________________________

6. What is the maximum number of spans made continuous and individual span lengths? ___/___

7. For those bridges made continuous, have you ever made the steel girders integral with the abutments (Yes/No)? __________

8. Do you use standard details for your continuous deck connection (Yes/No)?

9. Would you please enclose a copy of any continuous scheme details or attach a sketch (Yes/No)? __________

**** *Please answer the following questions concerning the last steel bridge made continuous:*

10. When were the deck joints (or project) completed (date)? __________

11. What component(s) were made continuous (check all that apply):
_____ Deck
_____ Top Flange
_____ Bottom Flange
_____ Web
_____ Full Moment Splice (deck, top flange, bottom flange, and web)
_____ Other (please specify): ________________________________

12. Please answer the following details:

a. Bridge Name:

b. Location:

c. Route:

d. Approximate AADT:

e. Design live load (before/after):

f. Number of spans:

g. Span length configuration:

h. Expansion/fixed joint configuration before and after (Span1 E/F, Span2 E/F, ...):

i. Bearing type (elastomeric, sliding plate, etc.):

j. Type of girders (shape, size, yield strength, etc):

k. Girder spacing:

l. Depth and material of deck:

m. Wearing surface:

13. Why did you retrofit the bridge with a continuous retrofitting detail?
_____ To improve the live load carrying capacity
_____ To eliminate deck joints
_____ Other: ____

**** ***These questions pertain to all continuous retrofits. If you need more room to answer the following questions, please write on the back or attach a separate page.***

14. Please describe the typical construction sequence for continuous retrofitting, including any bearing modifications:

____

____

____

____

15. Please describe problems encountered during the construction with their resolution:

____

____

____

____

16. Please describe any service problems that occurred after the structure was back in service:

____

____

____

17. Please describe any maintenance problems that have occurred:

18. Has your organization ever instrumented an existing steel bridge made continuous (Yes/No)? __

19. If you answered "Yes" to question 18, what was measured and for how long?

20. Have the designs of the continuous deck schemes ever contradicted the field performance?

To increase the knowledge base and to collect as much information as possible about this very interesting topic, we are asking for additional names of people involved with the design or construction of the retrofit of existing multi-simple span steel bridges with a continuous detail.

Name: ______________________________
Title: ______________________________
Organization: ______________________________
Address: ______________________________
City/Town: ______________________________ State: ____ Zip Code: ________
Phone: (___) ____________ Fax: (___) ____________

North American Survey Results

	GENERAL QUESTIONS	**CA**	**CO**	**CT**	**GA**	**IL**	**IN**	**KS**	**KY**	**MA**	**MD**
2	# multi-simple bridges	500+	229	500	4000	350	few	26	250	907	1593
3	# of bridges made continuous	unknown	60	10	5	30	5	2	25	1	6
4	# made continuous last 5 years: designed	unknown	1	10	5	10	0	0	5	2	----
	constructed		1	8	5	8	0	0	5	1	60
5	max span length & structure length made continuous										
	Deck only								40	60	88/230
	Deck / Top flange		80/---		150		200				
	Deck / Top & Bottom flange										
	Deck / Bottom flange			120/300							
	Deck/Top & Bottom Flange and Web		80/---				197	51/204			
	Deck/Other Connection Scheme		Integral Pier Cap			87/324					Integral Pier Cap
6	max # of spans made continuous with span lengths	unknown	---	4	3	-	3	4	5	3	3
				4@80	3@50		54	4@62	30-40	30	71
							89		30-40	60	88
							54		30-40	30	71
									30-40		
									30-40		
7	any girders integral with abutments	NO	YES	NO	NO	NO	NO	YES	NO	NO	NO
8	standard details	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO
	LAST BRIDGE MADE CONTINUOUS										
10	Date Project Completed	NA	1987	Constructn		1988	1984	1990	1994	1993	1993
11	Components Made Continuous										
	Deck only								X	X	X
	Deck / Top flange				X						
	Deck / Top & Bottom flange										
	Deck / Bottom flange			X							
	Deck/Top & Bottom Flange and Web						X	X			
	Deck/Other Connection Scheme		Integral Pier Cap			Integral Pier Cap					
12	(specific bridge information left out for brevity)										
	max # of spans made continuous with span lengths		2	2	3	4	7	4	3	3	2
			29@59.8	2@91	3@40	75		4@51	220	30	88
						2@87			320	60	88
						75			220	30	
13	why was the bridge retrofitted										
	improve live load capacity		X				X				X
	eliminate deck joints		X	X	X	X	X	X	X	X	X
	other	Seismic									

North American Survey Results Continued

		UNITED STATES OF AMERICA									**CANADA**	
	GENERAL QUESTIONS	**NH**	**NY**	**PA**	**RI**	**SD**	**TN**	**UT**	**VT**	**VA**	**ONT**	**SAS**
2	# multi-simple bridges	120	3454	1020	167	177	600+	-	100+	2951	unknown	4
3	# of bridges made continuous	2	30	NA	0	30	20	-	10	2	10	3
4	# made continuous last 5 years: designed	2	25	5	3	30	8	-	5	5	10	1

	constructed	2	25	5	0	25	8	200	5	2	10	1
5	max span length & structure length made continuous			-								
	Deck only	45.5/107.5				40/-	40/169	<300		75/150	spans 66 to 130	56/168
	Deck / Top flange											
	Deck / Top & Bottom flange		110/250				74/322					
	Deck / Bottom flange						96/489					
	Deck/Top & Bottom Flange and Web		140/325						88/817			
	Deck/Other Connection Scheme	83/325										
6	max # of spans made continuous with span lengths	5	5	2	2	3	6	4	5	2	3	3
		55	5@73	92	36	3@32	84	75-50	88	2@75	3@72	3@56
		72		130	57		82		69			
		83					83		3@84			
		72					96					
		44					71					
							73					
7	any girders integral with abutments	NO	NO	NO	NO	NO	YES	NO	NO	NO	YES	NO
8	standard details	NO	NO	YES	NO	NO	NO	YES	NO	NO	being developed	NO
	LAST BRIDGE MADE CONTINUOUS											
10	Date Project Completed	1993	1994	1996	in design	1995	1992	1995	1990	1995	1995	1992
11	Components Made Continuous											
	Deck only					X	X	@ abutments		X	X	X
	Deck / Top flange											
	Deck / Top & Bottom flange											
	Deck / Bottom flange											
	Deck/Top & Bottom Flange and Web	X	X	X					X			
	Deck/Other Connection Scheme										web cleat angle near bottom flange	
12	(specific bridge information left out for brevity)											
	max # of spans made continuous with span lengths	5	2	4		3	2	1	5	2	4	3
		55	126	2@92		3@32	?	134	88	2@75	40	3@56
		72	113	130					69		50	
		83		131					3@84		107	
		72									40	
		44										
13	why was the bridge retrofitted											
	improve live load capacity								X			
	eliminate deck joints		X	X	X	X	X	X	X	X	X	X
	other	seismic								test details		

DISCUSSIONS:

14 Sequence

CA This is not a standard procedure, therefore the construction sequence varies with the design, traffic, access, and other considerations

CO Remove existing deck, rebuild pier cap, add compression block at piers, and pour deck & diaphragms.

CT Remove deck, replace bearins, cast new deck continuous, retard concrete or cast pier segment last (live load retrofit)

GA Remove the deck, weld or bolt pates to steel, modify bearings, & repour deck

IL Weld studs to webs, install form work and rebars, pour concrete. Bearings are encased in concrete diaphragm.

KS Remove deck, replace bearings with one elastomeric bearing

KY Slab made continuous btwn stringer expansions. Typically just place additional #5 bars @12" across joint and pour continuous

MA Bearings were replaced prior to construction of slab continuity detail.

MD Remove steel cross frame at pier, remove rivets and cut stiffener angles and the fill plates to allow construction of seat angle, weld studs onto existing stiffener, pour concrete deck and diaphragm for full width of bridge, placing concrete between stiffener angles.

NY Expose top flange @ pier/ jack girder ends/ remove bearings, end stiffeners, & diaphragm/ install splice plates, bearings, set girder, install new pier diaphragms/ repair deck

PA The latest retrofitted bridge involves four spans, however, the first & last two spans were already continuous.

SD Construction sequence has no special details apart from performing required work. Most projects are completed one half roadway width at a time.

VT Remove deck, add bolted splice @ piers, replace bearings, remove shear studs, & replace w/ new, pour deck

ON Have tried several schemes (full depth concrete diaphragm, welding bottom flange) that have been costly. Metro Toronto has been successful using just deck continuity.

SK Remove concrete to 1" below top mat. Place rebar @8" spacing. Alternate ends at 6'-8" and 10'-0" from centerline of pier. Place low slump concrete deck.

15 Construction Problems

CA None apparent

CO Redistribution of loads may over stress piere sections if 'drop-in' spans are made continuous

CT Minor cracking in slab

GA Traffic handling during construction

NY Misalignment of adjacent span girder ends. Had to either jack girders into alignment or use shim plates. Alignment should be field measured before fabricating splice plates.

VT No problems

16 Service Problems

CO Some structures with integral pier and abutment diaphragms develop vertical craks that extend upward from the bearing region through the diaphragm concrete.

IN No problems after 10 years

ON Butyl rubber membrane reinforcement for waterproofing over flexible link causes debonding of asphalt

17 Maintenance Problems

CA One bridge report indicated . However, no additional problems have been reported.

CO Drainage that previously found its way through expansion joints is redirected to abutments or ends of wingwalls where erosion of fill slopes has increased

18 Instrumentation of steel bridge made continuous

No Agencies have instrumented this type of retrofit, however, RI has a program under design

19 What was instrumented and how long

No replies

20 Design vs. field performance conflict

CO We have used the integral pier cap scheme for many years with steel and concrete girders without notable problems.

ONT Process of developing standards and design guidelines. There seems to be no consistent design approach and, therefore, no direct comparison with field performance

Transportation Agencies NOT Involved with Continuous Deck Retrofits

AL, AK, AR, AZ, DC, DE, FL, HI, ID, IA, LA, ME*, MA TURNPIKE AUTH, MI, MN, MS, MO, MT, NE, NV, NJ, NJ H.A., NJ T.A., NM, NY&NJ P.A.O., NC, ND, OH, OK, OR, PR, SC, TX, WA, WI, WV, WY,

ALBERTA, BRITISH COLUMBIA, MANITOBA, NEW BRUNSWICK, NORTHWEST TERRITORIES, NOVA SCOTIA

* - Agencies with continuous deck joint retrofits under design

Transportation Agencies NOT Responding to Questionnaire

NEWFOUNDLAND, QUEBEC

These agencies are encouraged to respond before the results are officially published.

APPENDIX B

INSTRUMENTATION PROGRAM

FOR GARDEN STREET AND PINE STREET BRIDGES

WORKPLAN

INSTRUMENTATION PROGRAM

FOR GARDEN STREET AND PINE STREET BRIDGES

Purpose:

The purpose of this study is to provide a proposal for an instrumentation plan for the Garden Street Bridge (Bridge #547) and the Pine Street Bridge (Bridge # 548), both in Pawtucket, RI. The instrumentation will be performed during the rehabilitation of the bridges and readings will be taken manually by the installation contractor on a weekly basis for a period of one month following the installation of the individual monitoring devices to insure proper readings. The scope of this field instrumentation is to evaluate the performance of the expansion joint elimination of the above two bridges.

Bridge Descriptions:

Garden Street Bridge is located in Pawtucket, RI and carries Garden Street over I-95 northbound and southbound. It is a composite steel stringer bridge with concrete deck and bituminous overlay. It consists of four simple spans, 27', 59', 59', and 36', north to south. It has a skew angle of 5 degrees. The structure width is 54' but the south approach span is flared to 64'. It carries three one-way lanes of southbound local traffic on 40' wide pavement with two 7' sidewalks and concrete parapets with fence and railing. Figs. 1 and 2 show a typical cross section as well as a longitudinal section of the bridge.

The superstructure consists of nine in-line steel stringers (not including the flared section). It is supported on stub abutments and three intermediate concrete multi-column piers. The four spans are simply supported with the following bearing conditions:

Span 1:	Expansion	-	Fixed
Span 2:	Expansion	-	Fixed
Span 3:	Fixed	-	Expansion
Span 4:	Fixed	-	Expansion

The existing joint configuration is as follows:

Abutment A:	Expansion
Pier 1:	Expansion
Pier 2:	Fixed
Pier 3:	Expansion
Abutment B:	Expansion

Banks of telephone and electric utility ducts are located within their respective sidewalks. A 6" gas line is located below the west sidewalk.

Pine Street Bridge is located in Pawtucket, RI and carries Pine Street over I-95 northbound and southbound. It is a composite steel stringer bridge with concrete deck and bituminous overlay. It consists of four simple spans, 32', 59', 59', and 38', north to south. It has no skew angle. The structure width is 54' but the south approach span is flared to 62'. It carries three one-way lanes of northbound local traffic on 40' wide pavement with two 7' sidewalks and concrete parapets with fence and railing. Figs. 3 and 4 show a typical cross section as well as a longitudinal section of the bridge.

The superstructure consists of nine in-line steel stringers (not including the flared section). It is supported on stub abutments and three intermediate concrete multi-column piers. The four spans are simply supported with the following bearing conditions:

Span 1:	Expansion	-	Fixed
Span 2:	Expansion	-	Fixed
Span 3:	Fixed	-	Expansion
Span 4:	Fixed	-	Expansion

The existing joint configuration is as follows:

Abutment A	Expansion
Pier 1:	Expansion
Pier 2:	Fixed
Pier 3:	Expansion
Abutment B	Expansion

Conduits for fire alarm, traffic signal, and lighting along with banks of telephone and electric utility ducts are located within their respective sidewalks. An 8" water line is located below the east sidewalk.

For both bridges, the existing deck joints are in disrepair and adjacent concrete is deteriorated. The expansion bearings are corroded and frozen. The concrete piers and abutments are deteriorated and spalling.

Bridge Rehabilitation:

For the bridges to be instrumentated, the concrete deck will be repaired and patched where needed. The existing deck expansion joints over the abutments will be removed and replaced. The fixed joint at the center of the structure will be replaced in kind. The intermediate expansion joints over Piers 1 and 3 will be made continuous by use of a semi-continuous scheme made up of a steel top flange splice plate and continuous concrete deck. A length of about 5 ft of the concrete deck will be replaced at these two locations. Existing fixed bearings on piers 1 and 3 will be replaced with expansion elastomeric bearing pads. At the abutments, the deck will be extended over the backwall and a new expansion joint will be installed to absorb the anticipated increased expansion movement.

In addition, repairs to the sidewalks, parapets, railings, and fence will be performed. A new bituminous overlay will be placed. The steel stringers will be painted. Concrete repairs will also be done on the piers and abutments.

Instrumentation Plan:

Task One:

Physical properties of the new concrete, the existing concrete, and the reinforcing bars will be determined at the location where the expansion joint will be removed and partial continuity induced (over the first pier). Tests to be conducted on the concrete will include compression strength, modulus of elasticity, Poisson's ratio, and coefficient of thermal expansion. The prime contractor should prepare sixteen control specimens (6" x 12" cylinders) for testing by the instrumentation subcontractor. Eight cylinders should be tested at 28 days and eight at 3 months. In addition cores of the existing concrete deck should be taken at six random locations by the prime contractor for testing by the instrumentation subcontractor. Six longitudinal reinforcing bars at the continuous deck joint location will be retained by the prime contractor and will be provided to the instrumentation subcontractor for stress-strain tests to determine the modulus of elasticity and yield strength of the reinforcing steel. Reinforcing bars for each bridge should come from the same lot.

Three tension specimens will be cut from the same steel as the splice plates by the prime contractor and will be tested by the instrumentation subcontractor to determine the modulus of elasticity and the yield point.

All test specimens should comply with AASHTO specifications. All specimens will be recovered, tested, and a report submitted by the instrumentation contractor.

(16 cylinders, 6 reinforcing bars, 6 concrete cores, and 3 tension specimens per bridge)

Task Two:

Steel Girder strains: Strain[1] gages will be installed on four of the continuous steel stringers of each bridge. Three gages will be installed at each of three sections: at the negative moment region over pier 1 and at mid-span of the two adjacent spans. Approximate locations of the gages are indicated in Figs. 5 and 6. Actual locations will be determined and reported.

(36 gages per bridge)

Splice Plate Strains: Strain gages will be installed on the top surface of the splice plates at the locations of the new expansion joints over pier 1. Two gages per splice will

---

[1] The type of gage must be approved by RIDOT.

be placed on the four splices corresponding to the stringers that are instrumented (see above). Approximate locations of the gages are indicated in Figs. 5 and 7. Actual locations will be determined and reported.

(8 strain gages per bridge)

Reinforcing Steel Strains: Strain gages will be placed on the new longitudinal deck reinforcement over Pier 1 (twenty on the top mat and twenty on the bottom mat) for a total of 40 gages. Approximate locations are indicated in Fig. 5. Actual locations will be determined and reported. Since the rebars are epoxy coated, care must be taken that corrosion protection is present after the gages are attached.

(40 gages per bridge)

Task Three:

Concrete Strains: Seven longitudinal strain gages will be embedded in the new concrete over Pier 1. Four of these gages should be directly over the instrumented girders and three located between the girders. It is suggested that Carlson strain meters be used, but other strain gages may be approved by the engineer. Approximate locations of the gages are indicated in Fig. 5. Actual locations will be determined and reported. *(7 gages per bridge)*

Task Four:

Beam Rotations: Eight tiltmeter gages will be installed at the ends of the instrumented girders on both sides of Pier 1. Approximate locations of the gages are indicated in Fig. 5. and 6. Actual locations will be determined and reported. *(8 tiltmeter gages per bridge)*

Overall Longitudinal Movement: The total bridge deck movement between Abutment A and pier 2 should be measured. Extensometers or other approved devices should be installed on each of the first interior girders.

(2 extensometers per bridge)

Task Five:

Temperature Measurements: The Carlson meters which will monitor the long term concrete strains can also be used for temperature measurements in the concrete deck. In addition, to measure steel girder temperatures, four thermocouples will be installed on the instrumented girders at midspan of span 2. Approximate locations of these thermocouples are indicated in Fig. 5. Actual locations will be determined and reported.

(4 steel thermocouples per bridge)

Task Six:

A central station will be installed by the installation subcontractor at a convenient location on or near the bridge from which all readings can be made by the instrumentation subcontractor. The central control station shall be secure (vandal proof and lockable) and access must be easily made without traffic control. RIDOT will approve the location. Readings will begin with the installation of all transducers and continue at weekly intervals for one month. All data will be reduced. At the completion of this project all data and recording equipment will be turned over to RIDOT with instructions for operation. In addition, a final report describing the installation and including the data will be submitted. RIDOT and any applicable research personnel will be trained for the monitoring operation by the installation subcontractor at the RIDOT building. Manuals shall be provided for execution and interpretation of output.

Installation Procedure:

It is expected that installation will be performed on short notice and the ability of the installation subcontractor to coordinate with RIDOT and the prime contractor during all phases of the rehabilitation is essential. In particular coordination with the prime contractor to provide for adequate protection of all sensors during construction must be insured. The instrumentation will be installed using Maintenance and Protection of Traffic Plans provided by the prime contractor .

Four (4) meetings will be held prior to the installation between RIDOT, contractors, the Design Consultant and the installation contractor. The installation contractor will be expected to attend a bid-opening, a pre-construction meeting and two (2) RIDOT/Design Consultant (Maguire)/ coordination meetings.

Qualifications:

The instrumentation contractor will submit proof of qualifications and provide a list and scope of recent related work.

TYPICAL SOLICITATION LETTER FOR INSTRUMENTATION WORK-PLAN

Mr. Andrian Ciolko
Structural Engineering Laboratory
Construction Technologies Laboratory (CTL)
5420 Old Orchard Road
Skokie, IL 60077

December 21, 1995

Dear Mr. Ciolko:

We are involved in a research project, funded by the Rhode Island Department of Transportation, which evaluates expansion joint elimination during bridge rehabilitation. As part of this project we are performing analytical studies using finite element models of various typical bridges in Rhode Island. In order to validate our analytical calculations field monitoring of actual rehabilitated bridges is very useful. Fortunately, RIDOT is planning to eliminate a number of expansion joints in two of its bridges. These are Bridge No. 547, Garden Street Bridge and Bridge No. 548, Pine Street Bridge. Both are in Pawtucket, RI over I-95.

We are preparing an instrumentation proposal to RIDOT for instrumentation and field monitoring of the above bridges. Attached is a draft of our suggested instrumentation program. Mntion is made of a prime contractor who is the overall contractor of the rehabilitation of the bridges, an installation subcontractor who will install the sensors and related devices and an instrumentation subcontractor (this is our group) who will perform testing of specimens, analyze field data and compare with analytical results. Also attached are existing and proposed plans of the subject bridges showing joint and continuity details.

Your firm was suggested to us by colleagues as one with experience in installation of monitoring devices in bridges. I am trying to solicit some preliminary information which will help us in preparing the budget of our proposal. We would appreciate if you could provide us in the attached table your best estimate for installing the sensors and performing the tasks. That would give us an idea whether we are proposing a reasonable amount of instrumentation. If such detailed numbers are not readily available, an overall estimate of the whole instrumentation as outlined in our draft would still be very useful. In addition, any suggestions for improving the instrumentation program are welcomed. In such a case you may want to also provide some cost estimate of your suggestions. As you realize, the installation subcontractor will have to work closely with the prime contractor.

Incidentally, the prime contractor will provide all maintenance protection of traffic. Since the bridges are over I-95 lane closures will have to be kept at minimum with no more than one lane closed during off peak hours.

I would finally ask that you send us some brief information of your experience in this type of bridge instrumentation. We would appreciate your response by January 10. Once the responses are here and a final work plan is prepared, it will be sent to all qualified bidders for formal proposals.

We do thank you for your time spent reviewing the enclosed draft and providing some cost estimates. If you have any questions, please contact Prof. Tsiatas at (401)-792-5117. FAX machine number should you need it is (401) 792-2786 and e_mail is tsiatas@uriacc.uri.edu.

Sincerely,

George Tsiatas, Ph.D.
Associate Professor

Attachments

INSTRUMENT INSTALLATION ESTIMATE BREAKDOWN
FOR GARDEN STREET BRIDGE NO. 547
AND PINE STREET BRIDGE NO. 548

ITEM	BRIDGE NO. 547	BRIDGE NO. 548	TOTAL
Task Two			
36 strain gages for steel girders			
8 strain gages for splice plate			
40 gages for reinf. bars			
Task Three			
7 gages for concrete			
Task Four			
8 tilt gages			
2 extensometer gages			
Task Five			
4 thermocouples in steel			
Task 6			
central station			
report			

Other Costs such as mobililization to go to site (describe)

____

In case you cannot provide details as requested above please provide an approximate overall estimate per bridge ____________________________

APPENDIX C

RIDOT BRIDGE PROPERTIES

Section Properties

2/22/96

Garden Street Bridge #547

deck =	6.50	b eff =	73.50
haunch =	1.25		
Es =	29,000,000.00	Ec =	3,834,253.51
f'c =	4,000.00	n =	7.56
w =	150.00		

Span 1: Moment of Inertia

W 30 X 99	Area =	29.10	Ix =	3,990.00
	d =	29.65		

Element	*Transformed Area* *A*	*Moment Arm from Centroid* *y*	*Ay*	*Ay^2*	*Io*
Slab	63.17	19.33	1,220.68	23,589.71	222.40
W 30 X 99	29.10	0.00	0.00	0.00	3,990.00
	92.27		1,220.68	23,589.71	4,212.40
	0.6407				

Ix = Io + Ay^2 =	27,802.10			
Centroid = Ay/A =	13.23			
Itr = Ix - Ay^2 =	11,652.41	0.5619	yt/rebr =	7.34
y top =	9.34		yb/rebr =	3.84
y bottom =	28.06	2.3379	y splice =	1.97

Span 2 & 3 - No Cover Plate: Moment of Inertia

W 33 X 130	Area =	38.30	Ix =	6,710.00
	d =	33.09		

Element	*Transformed Area* *A*	*Moment Arm from Centroid* *y*	*Ay*	*Ay^2*	*Io*
Slab	63.17	21.05	1,329.33	27,975.73	222.40
W 33 X 130	38.30	0.00	0.00	0.00	6,710.00
	101.47		1,329.33	27,975.73	6,932.40
	0.7046				

Ix = Io + Ay^2 =	34,908.12			
Centroid = Ay/A =	13.10			
Itr = Ix - Ay^2 =	17,492.29	0.8436	yt/rebr =	9.19
y top =	11.19		yb/rebr =	5.69
y bottom =	29.65	2.4705	y splice =	3.82

Span 2 & 3 - Cover Plate: Moment of Inertia

W 33 X 130	Area =	38.30	Ix =	6,710.00

Section Properties

2/22/96

Garden Street Bridge #547

d = 33.09

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	63.17	21.05	1,329.33	27,975.73	222.40
W 33 X 130	38.30	0.00	0.00	0.00	6,710.00
Cover Plate	7.5	(16.92)	(126.90)	2,147.15	0.00
	108.97		1,202.43	30,122.88	6,932.40
	0.7567				

Ix = Io + Ay^2 =	37,055.27			
Centroid = Ay/A =	11.03			
Itr = Ix - Ay^2 =	23,786.59	1.1471	yt/rebr =	11.26
y top =	13.26		yb/rebr =	7.76
y bottom =	11.03	0.9196	y splice =	5.89

Span 4: Moment of Inertia

W 24 X 76 Area = 22.40 Ix = 2,100.00
d = 23.92

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	63.17	16.46	1,039.71	17,113.67	222.40
W 24 X 76	22.40	0.00	0.00	0.00	2,100.00
	85.57		1,039.71	17,113.67	2,322.40
	0.5942				

Ix = Io + Ay^2 =	19,436.07			
Centroid = Ay/A =	12.15			
Itr = Ix - Ay^2 =	6,802.52	0.3281	yt/rebr =	5.56
y top =	7.56		yb/rebr =	2.06
y bottom =	24.11	2.0092	y splice =	0.18

Continuous Joint: Spring Constants

	Steel Area (sq. in.)	2 X Ld (inches)	Length (inches)	Equivalent Stiffness (lb/in)	Equivalent Stiffness (k/ft)
Top Mat = 9 X #4	1.77	10.63	24.00	2,138,750.00	25,665.00
Bottom Mat = 8 X #4	1.57	10.63	24.00	1,897,083.33	22,765.00
Splice Plate = 12" X 3/4"	9.00		14.00	18,642,857.14	223,714.29

Development Length = Ld = 0.028 * (Area of bar) * (fy) / (f'c)^0.5
"Design of Concrete Structures", Nilson
Length = 2 X Development Length or 2 X 12" minimum
* Reinforcement fy = 60,000 psi

Section Properties

2/22/96

Pine Street Bridge #548

deck =	6.50	b eff =	73.50
haunch =	1.25		
Es =	29,000,000.00	Ec =	3,834,253.51
f'c =	4,000.00	n =	7.56
w =	150.00		

Span 1: Moment of Inertia

W 30 X 116	Area =	34.20	0.2375	Ix =	4,930.00	0.2378
	d =	30.01				

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	63.17	19.51	1,232.05	24,031.20	222.40
W 30 X 116	34.20	0.00	0.00	0.00	4,930.00
	97.37		1,232.05	24,031.20	5,152.40
	0.6762				

Ix = Io + Ay^2 =	29,183.60			
Centroid = Ay/A =	12.65			
Itr = Ix - Ay^2 =	13,593.40	**0.6555**	yt/rebr =	8.10
y top =	10.10		yb/rebr =	4.60
y bottom =	27.66	**2.3049**	y splice =	2.73

Span 2 & 3 - No Cover Plate: Moment of Inertia

W 33 X 130	Area =	38.30	Ix =	6,710.00
	d =	33.09		

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	63.17	21.05	1,329.33	27,975.73	222.40
W 33 X 130	38.30	0.00	0.00	0.00	6,710.00
	101.47		1,329.33	27,975.73	6,932.40
	0.7046				

Ix = Io + Ay^2 =	34,908.12			
Centroid = Ay/A =	13.10			
Itr = Ix - Ay^2 =	17,492.29	**0.8436**	yt/rebr =	9.19
y top =	11.19		yb/rebr =	5.69
y bottom =	29.65	**2.4705**	y splice =	3.82

Span 2 & 3 - Cover Plate: Moment of Inertia

W 33 X 130	Area =	38.30	Ix =	6,710.00
	d =	33.09		

Section Properties

2/22/96

Pine Street Bridge #548

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	63.17	21.05	1,329.33	27,975.73	222.40
W 33 X 130	38.30	0.00	0.00	0.00	6,710.00
Cover Plate	7.5	(16.92)	(126.90)	2,147.15	0.00
	108.97		1,202.43	30,122.88	6,932.40
	0.7567				

Ix = Io + Ay^2 =	37,055.27			
Centroid = Ay/A =	11.03			
Itr = Ix - Ay^2 =	23,786.59	**1.1471**	yt/rebr =	11.26
y top =	13.26		yb/rebr =	7.76
y bottom =	11.03	**0.9196**	y splice =	5.89

Span 4: Moment of Inertia

W 30 X 99	Area =	29.10	Ix =	3,990.00
	d =	29.65		

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	63.17	19.33	1,220.68	23,589.71	222.40
W 30 X 99	29.10	0.00	0.00	0.00	3,990.00
	92.27		1,220.68	23,589.71	4,212.40
	0.6407				

Ix = Io + Ay^2 =	27,802.10			
Centroid = Ay/A =	13.23			
Itr = Ix - Ay^2 =	11,652.41	**0.5619**	yt/rebr =	7.34
y top =	9.34		yb/rebr =	3.84
y bottom =	28.06	**2.3379**	y splice =	1.97

Continuous Joint: Spring Constants

	Steel Area (sq. in.)	2 X Ld (inches)	Length (inches)	Equivalent Stiffness (lb/in)	Equivalent Stiffness (k/ft)
Top Mat = 9 X #4	1.77	10.63	24.00	2,138,750.00	**25,665.00**
Bottom Mat = 8 X #4	1.57	10.63	24.00	1,897,083.33	**22,765.00**
Splice Plate = 12" X 3/4"	9.00		14.00	18,642,857.14	**223,714.29**

Development Length = Ld = 0.028 * (Area of bar) * (fy) / (fc)^0.5

"Design of Concrete Structures", Nilson

Length = 2 X Development Length or 2 X 12" minimum

* Reinforcement fy = 60,000 psi

Section Properties

2/22/96

Hartford Ave. - West Bridge #608

Span 1: End Sections

				Limits of b eff
span (ft) =	93.00			
deck depth (in)=	7.00	b eff (in)=	86.00	279.00
haunch (in)=	1.25			86.00
Es (psi) =	29,000,000.00			
f'c (psi) =	4,000.00	Ec (in) =	3,834,253.51	
wgt of conc. (#/ci) =	150.00	n =	7.56	

Plate Girder:

	height (in)	width (in)
Web	40.00	0.38
Top Flange	1.00	16.00
Bottom Flange	1.88	16.00

Area (sq in) = 61.00
d (in) = 42.88

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	79.59	47.63	3,790.66	180,529.96	325.01
Web	15.00	21.88	328.13	7,177.73	2,000.00
Top Flange	16.00	42.38	678.00	28,730.25	1.33
Bottom Flange	30.00	0.94	28.13	26.37	8.79
	140.59		4,824.91	216,464.32	2,335.13

	inches	feet
Total Trans Area =		0.9763
Ix = Io + Ay^2 =	218,799.45	
Centroid = Ay/A =	34.32	
Itr = Ix - Ay^2 =	53,218.11	2.5665
y top section =	16.81	
y bottom flange =	34.32	2.86
yt/rebr =	14.81	1.23
yb/rebr =	11.81	0.98
y splice =	9.06	0.75

Span 1: Middle Section

				Limits of b eff
span (ft) =	93.00			
deck depth (in)=	7.00	b eff (in)=	86.00	279.00
haunch (in)=	1.25			86.00
Es (psi) =	29,000,000.00			
f'c (psi) =	4,000.00	Ec (in) =	3,834,253.51	
wgt of conc. (#/ci) =	150.00	n =	7.56	

Plate Girder:

	height (in)	width (in)
Web	40.00	0.38
Top Flange	1.38	16.00
Bottom Flange	1.88	22.00

Section Properties

2/22/96

Hartford Ave. - West Bridge #608

Area (sq in) =	78.25
d (in) =	43.25

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	79.59	48.00	3,820.50	183,384.15	325.01
Web	15.00	21.88	328.13	7,177.73	2,000.00
Top Flange	22.00	42.56	936.38	39,854.46	3.47
Bottom Flange	41.25	0.94	38.67	36.25	12.08
	157.84		5,123.67	230,452.60	2,340.56

	inches	feet
Total Trans Area =		1.0961
Ix = Io + Ay^2 =	232,793.16	
Centroid = Ay/A =	32.46	
Itr = Ix - Ay^2 =	66,476.57	3.2059
y top section =	19.04	
y bottom flange =	32.46	2.71
yt/rebr =	17.04	1.42
yb/rebr =	14.04	1.17
y splice =	11.48	0.96

Span 2: End Sections

span (ft) =	84.46			Limits of b eff
deck depth (in)=	7.00	b eff (in)=	86.00	253.38
haunch (in)=	1.25			86.00
Es (psi) =	29,000,000.00			
f'c (psi) =	4,000.00	Ec (in) =	3,834,253.51	
wgt of conc. (#/ci) =	150.00	n =	7.56	

Plate Girder:

	height (in)	width (in)
Web	40.00	0.38
Top Flange	0.75	16.00
Bottom Flange	1.50	16.00

Area (sq in) =	51.00
d (in) =	42.25

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	79.59	47.00	3,740.91	175,822.74	325.01
Web	15.00	21.50	322.50	6,933.75	2,000.00
Top Flange	12.00	41.88	502.50	21,042.19	0.56
Bottom Flange	24.00	0.75	18.00	13.50	4.50
	130.59		4,583.91	203,812.17	2,330.07

Section Properties

2/22/96

Hartford Ave. - West Bridge #608

	inches	feet
Total Trans Area =		0.9069
Ix = Io + Ay^2 =	206,142.24	
Centroid = Ay/A =	35.10	
Itr = Ix - Ay^2 =	45,244.70	2.1819
y top section =	15.40	
y bottom flange =	35.10	2.93
yt/rebr =	13.40	1.12
yb/rebr =	10.40	0.87
y splice =	7.52	0.63

Span 2: Middle Section

				Limits of b eff
span (ft) =	84.46			
deck depth (in)=	7.00	b eff (in)=	86.00	253.38
haunch (in)=	1.25			86.00
Es (psi) =	29,000,000.00			
f'c (psi) =	4,000.00	Ec (in) =	3,834,253.51	
wgt of conc. (#/ci) =	150.00	n =	7.56	

Plate Girder:

	height (in)	width (in)
Web	40.00	0.38
Top Flange	1.00	16.00
Bottom Flange	1.50	22.00

Area (sq in) =	64.00
d (in) =	42.50

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	79.59	47.25	3,760.81	177,698.17	325.01
Web	15.00	21.50	322.50	6,933.75	2,000.00
Top Flange	16.00	42.00	672.00	28,224.00	1.33
Bottom Flange	33.00	0.75	24.75	18.56	6.19
	143.59		4,780.06	212,874.48	2,332.53

	inches	feet
Total Trans Area =		0.9972
Ix = Io + Ay^2 =	215,207.01	
Centroid = Ay/A =	33.29	
Itr = Ix - Ay^2 =	56,084.89	2.7047
y top section =	17.46	
y bottom flange =	33.29	2.77
yt/rebr =	15.46	1.29
yb/rebr =	12.46	1.04
y splice =	9.71	0.81

Continuous Joint: Spring Constants

Section Properties

2/22/96

Hartford Ave. - West Bridge #608

	Steel Area (sq. in.)	2 X Ld (inches)	Length (inches)	Equivalent Stiffness (lb/in)	Equivalent Stiffness (k/ft)
Top Mat = 5 X #4	0.98	10.63	24.00	1,184,166.67	14,210.00
Bottom Mat = 9 X #4	1.77	10.63	24.00	2,138,750.00	25,665.00
Top Splice Plate = 16" X .75"	12.00	12.00	12.00	29,000,000.00	348,000.00
Bottom Splice Plate = 16" X 1.5"	24.00	12.00	12.00	58,000,000.00	696,000.00

Development Length = Ld = 0.028 * (Area of bar) * (fy) / (f'c)^0.5
"Design of Concrete Structures", Nilson
Length = 2 X Development Length or 2 X 12" minimum
* Reinforcement fy = 60,000 psi

Moment Splice

				Limits of b eff
span (ft) =	84.46			
deck depth (in)=	0.00	b eff (in)=	86.00	253.38
haunch (in)=	1.25			86.00
Es (psi) =	29,000,000.00			
f'c (psi) =	4,000.00	Ec (in) =	3,834,253.51	
wgt of conc. (#/ci) =	150.00	n =	7.56	

Plate Girder:

	height (in)	width (in)
Web	40.00	0.38
Top Flange	0.75	16.00
Bottom Flange	1.50	16.00

Area (sq in) = 51.00
d (in) = 42.25

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	0.00	43.50	0.00	0.00	0.00
Web	15.00	21.50	322.50	6,933.75	2,000.00
Top Flange	12.00	41.88	502.50	21,042.19	0.56
Bottom Flange	24.00	0.75	18.00	13.50	4.50
	51.00		843.00	27,989.44	2,005.06

	inches	feet
Total Trans Area =		0.3542
Ix = Io + Ay^2 =	29,994.50	
Centroid = Ay/A =	16.53	
Itr = Ix - Ay^2 =	16,060.21	0.7745
y top section =	26.97	
y bottom flange =	16.53	1.38

Section Properties

2/22/96

Broad St. - South Bridge #657

Span 1: End Sections (0.00' - 11.2' & 62.2' - 73.3')

span (ft) =	73.31				Limits of b eff
deck depth (in)=	7.00	b eff (in)=		84.00	219.94
haunch (in)=	2.00				84.00
Es (psi) =	29,000,000.00				
f'c (psi) =	4,000.00	Ec (in) =		3,834,253.51	
wgt of conc. (#/ci) =	150.00	n =		7.56	

Plate Girder:

	height (in)	width (in)
Web	48.00	0.38
Top Flange	0.50	12.00
Bottom Flange	1.00	12.00

Area (sq in) = 36.00
d (in) = 49.50

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	77.74	55.00	4,275.85	235,171.96	317.45
Web	18.00	25.00	450.00	11,250.00	3,456.00
Top Flange	6.00	49.25	295.50	14,553.38	0.13
Bottom Flange	12.00	0.50	6.00	3.00	1.00
	113.74		5,027.35	260,978.33	3,774.57

	inches	feet
Total Trans Area =		**0.7899**
Ix = Io + Ay^2 =	264,752.91	
Centroid = Ay/A =	44.20	
Itr = Ix - Ay^2 =	42,547.31	**2.0519**
y top section =	14.30	
y bottom flange =	44.20	**3.68**
yt/rebr =	12.30	**1.03**
yb/rebr =	9.30	**0.78**
y splice =	5.55	**0.46**

Span 1: Middle Section (11.2' - 62.2')

span (ft) =	73.31				Limits of b eff
deck depth (in)=	7.00	b eff (in)=		84.00	219.94
haunch (in)=	2.00				84.00
Es (psi) =	29,000,000.00				
f'c (psi) =	4,000.00	Ec (in) =		3,834,253.51	
wgt of conc. (#/ci) =	150.00	n =		7.56	

Plate Girder:

	height (in)	width (in)
Web	48.00	0.38
Top Flange	0.75	12.00
Bottom Flange	1.88	12.00

Section Properties *2/22/96*

Broad St. - South Bridge #657

Area (sq in) =	49.50
d (in) =	50.63

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	77.74	56.13	4,363.31	244,891.02	317.45
Web	18.00	25.88	465.75	12,051.28	3,456.00
Top Flange	9.00	50.25	452.25	22,725.56	0.42
Bottom Flange	22.50	0.94	21.09	19.78	6.59
	127.24		5,302.41	279,687.64	3,780.46

	inches	feet
Total Trans Area =		0.8836
Ix = Io + Ay^2 =	283,468.10	
Centroid = Ay/A =	41.67	
Itr = Ix - Ay^2 =	62,508.39	3.0145
y top section =	17.95	
y bottom flange =	41.67	3.47
yt/rebr =	15.95	1.33
yb/rebr =	12.95	1.08
y splice =	9.33	0.78

Span 2: End Sections (74.3' - 84.5' & 135.5' - 145.7')

				Limits of b eff
span (ft) =	71.38			
deck depth (in)=	7.00	b eff (in)=	84.00	214.13
haunch (in)=	2.00			84.00
Es (psi) =	29,000,000.00			
fc (psi) =	4,000.00	Ec (in) =	3,834,253.51	
wgt of conc. (#/ci) =	150.00	n =	7.56	

Plate Girder:

	height (in)	width (in)
Web	48.00	0.38
Top Flange	0.50	12.00
Bottom Flange	1.00	12.00

Area (sq in) =	36.00
d (in) =	49.50

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	77.74	55.00	4,275.85	235,171.96	317.45
Web	18.00	25.00	450.00	11,250.00	3,456.00
Top Flange	6.00	49.25	295.50	14,553.38	0.13
Bottom Flange	12.00	0.50	6.00	3.00	1.00

Broad St. - South Bridge #657

	113.74		5,027.35	260,978.33	3,774.57

	inches	feet
Total Trans Area =		**0.7899**
Ix = Io + Ay^2 =	264,752.91	
Centroid = Ay/A =	44.20	
Itr = Ix - Ay^2 =	42,547.31	**2.0519**
y top section =	14.30	
y bottom flange =	44.20	**3.68**
yt/rebr =	12.30	**1.03**
yb/rebr =	9.30	**0.78**
y splice =	5.55	**0.46**

Span 2: Middle Section (84.5' - 135.5)

				Limits of b eff
span (ft) =	71.38			
deck depth (in)=	7.00	b eff (in)=	84.00	214.13
haunch (in)=	2.00			84.00
Es (psi) =	29,000,000.00			
fc (psi) =	4,000.00	Ec (in) =	3,834,253.51	
wgt of conc. (#/ci) =	150.00	n =	7.56	

Plate Girder:

	height (in)	width (in)
Web	48.00	0.38
Top Flange	0.75	12.00
Bottom Flange	1.88	12.00

Area (sq in) = 49.50

d (in) = 50.63

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	77.74	56.13	4,363.31	244,891.02	317.45
Web	18.00	25.88	465.75	12,051.28	3,456.00
Top Flange	9.00	50.25	452.25	22,725.56	0.42
Bottom Flange	22.50	0.94	21.09	19.78	6.59
	127.24		5,302.41	279,687.64	3,780.46

	inches	feet
Total Trans Area =		**0.8836**
Ix = Io + Ay^2 =	283,468.10	
Centroid = Ay/A =	41.67	
Itr = Ix - Ay^2 =	62,508.39	**3.0145**
y top section =	17.95	
y bottom flange =	41.67	**3.47**
yt/rebr =	15.95	**1.33**
yb/rebr =	12.95	**1.08**
y splice =	9.33	**0.78**

Section Properties

2/22/96

Broad St. - South Bridge #657

Span 3: (146.7' - 193.75)

				Limits of b eff
span (ft) =	47.04			
deck depth (in)=	7.00	b eff (in)=	84.00	141.13
haunch (in)=	2.00			84.00
Es (psi) =	29,000,000.00			
f'c (psi) =	4,000.00	Ec (in) =	3,834,253.51	
wgt of conc. (#/ci) =	150.00	n =	7.56	

Plate Girder:

	height (in)	width (in)
Web	48.00	0.38
Top Flange	0.50	12.00
Bottom Flange	0.75	12.00

Area (sq in) = 33.00
d (in) = 49.25

Element	*Transformed Area* *A*	*Moment Arm from Centroid* *y*	*Ay*	*Ay^2*	*Io*
Slab	77.74	54.75	4,256.42	233,038.89	317.45
Web	18.00	24.75	445.50	11,026.13	3,456.00
Top Flange	6.00	49.00	294.00	14,406.00	0.13
Bottom Flange	9.00	0.38	3.38	1.27	0.42
	110.74		4,999.29	258,472.28	3,774.00

	inches	feet
Total Trans Area =		**0.7690**
Ix = Io + Ay^2 =	262,246.28	
Centroid = Ay/A =	45.14	
Itr = Ix - Ay^2 =	36,561.79	**1.7632**
y top section =	13.11	
y bottom flange =	45.14	**3.76**
yt/rebr =	11.11	**0.93**
yb/rebr =	8.11	**0.68**
y splice =	4.36	**0.36**

Span 1&2: Full Moment Steel Splice Section

				Limits of b eff
span (ft) =	73.31			
deck depth (in)=	0.00	b eff (in)=	84.00	219.94
haunch (in)=	0.00			84.00
Es (psi) =	29,000,000.00			
f'c (psi) =	4,000.00	Ec (in) =	3,834,253.51	
wgt of conc. (#/ci) =	150.00	n =	7.56	

Plate Girder:

	height (in)	width (in)
Web	48.00	0.38
Top Flange	0.50	12.00
Bottom Flange	1.00	12.00

Area (sq in) = 36.00

Section Properties

2/22/96

Broad St. - South Bridge #657

d (in) = 49.50

Element	Transformed Area A	Moment Arm from Centroid y	Ay	Ay^2	Io
Slab	0.00	49.50	0.00	0.00	0.00
Web	18.00	25.00	450.00	11,250.00	3,456.00
Top Flange	6.00	49.25	295.50	14,553.38	0.13
Bottom Flange	12.00	0.50	6.00	3.00	1.00
	36.00		751.50	25,806.38	3,457.13

	inches	feet
Total Trans Area =		**0.2500**
Ix = Io + Ay^2 =	29,263.50	
Centroid = Ay/A =	20.88	
Itr = Ix - Ay^2 =	13,575.94	**0.6547**
y top section =	28.63	

Span 2&3: Full Moment Steel Splice Section

				Limits of b eff
span (ft) =	73.31			
deck depth (in)=	0.00	b eff (in)=	84.00	219.94
haunch (in)=	0.00			84.00
Es (psi) =	29,000,000.00			
fc (psi) =	4,000.00	Ec (in) =	3,834,253.51	
wgt of conc. (#/ci) =	150.00	n =	7.56	

Plate Girder:

	height (in)	width (in)
Web	48.00	0.38
Top Flange	0.50	12.00
Bottom Flange	0.75	12.00

Area (sq in) = 33.00

d (in) = 49.25

Element	Transformed Area A	Moment Arm from Centroid y	Ay	Ay^2	Io
Slab	0.00	49.25	0.00	0.00	0.00
Web	18.00	24.75	445.50	11,026.13	3,456.00
Top Flange	6.00	49.00	294.00	14,406.00	0.13
Bottom Flange	9.00	0.38	3.38	1.27	0.42
	33.00		742.88	25,433.39	3,456.55

	inches	feet
Total Trans Area =		**0.2292**
Ix = Io + Ay^2 =	28,889.94	
Centroid = Ay/A =	22.51	
Itr = Ix - Ay^2 =	12,166.81	**0.5867**
y top section =	26.74	

Section Properties

2/22/96

Broad St. - South Bridge #657

Continuous Joint: Spring Constants

	Steel Area (sq. in.)	2 X Ld (inches)	Length (inches)	Equivalent Stiffness (lb/in)	Equivalent Stiffness (k/ft)
Top Mat = 8 X #4	1.57	10.63	24.00	1,897,083	**22,765.00**
Bottom Mat = 15 X #4	2.94	10.63	24.00	3,552,500	**42,630.00**
Span 1 & 2: Top Splice Plate = 12" X 0.5"	6.00	12.00	12.00	14,500,000	**174,000.00**
Bottom Splice Plate = 12" X 1"	12.00	12.00	12.00	29,000,000	**348,000.00**
Span 2 & 3: Top Splice Plate = 12" X 0.5"	6.00	12.00	12.00	14,500,000	**174,000.00**
Bottom Splice Plate = 12" X 0.75"	9.00	12.00	12.00	21,750,000	**261,000.00**

Development Length = Ld = 0.028 * (Area of bar) * (fy) / (f'c)^0.5

"Design of Concrete Structures", Nilson

Length = 2 X Development Length or 2 X 12" minimum

* Reinforcement fy = 60,000 psi

APPENDIX D

PARAMETRIC STUDY BRIDGE PROPERTIES

Section Properties

5/23/96

Spans 95' to 30'

AISI - 40 ft roadway with four composite beams w/o cover plates

95 foot span

span (L) =	95.00				Limits of b eff
deck depth =	10.00	b eff =	138.00		285.00
haunch =	1.25				138.00
Es =	29,000,000.00	Ec =	3,834,253.51		
f'c =	4,000.00	n =	7.56		
wgt of conc. (w) =	150.00				

W 36 X 393	Area =	115.00	Ix =	27,500.00
	d =	37.80	bf =	16.83
			tf =	2.20

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	182.46	25.15	4,588.81	115,408.53	1,520.48
W 36 X 393	115.00	0.00	0.00	0.00	27,500.00
	297.46		4,588.81	115,408.53	29,020.48
	2.0657				

Ix = Io + Ay^2 =	144,429.01			
Centroid = Ay/A =	15.43	**1.29**		
Itr = Ix - Ay^2 =	73,638.54	**3.5512**	yt/rebr =	12.72
y top =	14.72		yb/rebr =	5.72
y bottom =	34.33		y splice =	4.57

Continuous Joint: Spring Constants	Steel Area (sq. in.)	Computed Length (inches)	Design Length (inches)	Equivalent Stiffness (lb/in)	Equivalent Stiffness (k/ft)
Top Mat = 9 X #5	2.76	16.47	24.00	3,335,000.00	**40,020.00**
Bottom Mat = 14 X #5	4.30	16.47	24.00	5,195,833.33	**62,350.00**
Splice Plate (use least for unequal spans)	37.03		14.00	76,696,714.29	**920,360.57**
Moment Splice (ft): Ix =	**1.3262**			Area =	**0.7986**

90 foot span

span (L) =	90.00				Limits of b eff
deck depth =	10.00	b eff =	138.00		270.00
haunch =	1.25				138.00
Es =	29,000,000.00	Ec =	3,834,253.51		
f'c =	4,000.00	n =	7.56		
wgt of conc. (w) =	150.00				

W 36X359	Area =	105.00	Ix =	24,800.00
	d =	37.40	bf =	16.73
			tf =	2.01

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	182.46	24.95	4,552.32	113,580.30	1,520.48
W 36X359	105.00	0.00	0.00	0.00	24,800.00

Spans 95' to 30' AISI - 40 ft roadway with four composite beams w/o cover plates

	287.46		4,552.32	113,580.30	26,320.48	
	1.9962					
Ix = Io + Ay^2 =		139,900.78				
Centroid = Ay/A =		15.84	**1.32**			
Itr = Ix - Ay^2 =		67,808.10	**3.2701**	yt/rebr =	12.11	
y top =		14.11		yb/rebr =	5.11	
y bottom =		34.54		y splice =	3.87	

Continuous Joint: Spring Constants	Steel Area (sq. in.)	Computed Length (inches)	Design Length (inches)	Equivalent Stiffness (lb/in)	Equivalent Stiffness (k/ft)
Top Mat = 9 X #5	2.76	16.47	24.00	3,335,000.00	**40,020.00**
Bottom Mat = 14 X #5	4.30	16.47	24.00	5,195,833.33	**62,350.00**
Splice Plate (use least for unequal spans)	33.63		14.00	69,656,550.00	**835,878.60**
Moment Splice (ft): Ix =	**1.1960**			Area =	**0.7292**

85 foot span

span (L) =	85.00			Limits of b eff
deck depth =	10.00	b eff =	138.00	255.00
haunch =	1.25			138.00
Es =	29,000,000.00	Ec =	3,834,253.51	
fc =	4,000.00	n =	7.56	
wgt of conc. (w) =	150.00			

W 36 X 328	Area =	96.40	Ix =	22,500.00
	d =	37.09	bf =	16.63
			tf =	1.85

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	182.46	24.80	4,524.04	112,173.47	1,520.48
W 36 X 328	96.40	0.00	0.00	0.00	22,500.00
	278.86		4,524.04	112,173.47	24,020.48
	1.9365				

Ix = Io + Ay^2 =	136,193.95			
Centroid = Ay/A =	16.22	**1.35**		
Itr = Ix - Ay^2 =	62,798.42	**3.0285**	yt/rebr =	11.57
y top =	13.57		yb/rebr =	4.57
y bottom =	34.77		y splice =	3.25

Continuous Joint: Spring Constants	Steel Area (sq. in.)	Computed Length (inches)	Design Length (inches)	Equivalent Stiffness (lb/in)	Equivalent Stiffness (k/ft)
Top Mat = 9 X #5	2.76	16.47	24.00	3,335,000.00	**40,020.00**
Bottom Mat = 14 X #5	4.30	16.47	24.00	5,195,833.33	**62,350.00**
Splice Plate (use least for unequal spans)	30.77		14.00	63,728,535.71	**764,742.43**
Moment Splice (ft): Ix =	**1.0851**			Area =	**0.6694**

Spans 95' to 30' AISI - 40 ft roadway with four composite beams w/o cover plates

80 foot span

span (L) =	80.00				Limits of b eff
deck depth =	10.00		b eff =	138.00	240.00
haunch =	1.25				138.00
Es =	29,000,000.00		Ec =	3,834,253.51	
f'c =	4,000.00		n =	7.56	
wgt of conc. (w) =	150.00				

W 36X300	Area =	88.30	Ix =	20,300.00
	d =	36.74	bf =	16.66
			tf =	1.68

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	182.46	24.62	4,492.11	110,595.64	1,520.48
W 36X300	88.30	0.00	0.00	0.00	20,300.00
	270.76		4,492.11	110,595.64	21,820.48
	1.8803				

Ix = Io + Ay^2 =	132,416.12			
Centroid = Ay/A =	16.59	**1.38**		
Itr = Ix - Ay^2 =	57,888.15	**2.7917**	yt/rebr =	11.03
y top =	13.03		yb/rebr =	4.03
y bottom =	34.96		y splice =	2.62

Continuous Joint: Spring Constants	Steel Area (sq. in.)	Computed Length (inches)	Design Length (inches)	Equivalent Stiffness (lb/in)	Equivalent Stiffness (k/ft)
Top Mat = 9 X #5	2.76	16.47	24.00	3,335,000.00	**40,020.00**
Bottom Mat = 14 X #5	4.30	16.47	24.00	5,195,833.33	**62,350.00**
Splice Plate (use least for unequal spans)	27.98		14.00	57,959,400.00	**695,512.80**
Moment Splice (ft): Ix =	**0.9790**			Area =	**0.6132**

75 foot span

span (L) =	75.00				Limits of b eff
deck depth =	10.00		b eff =	138.00	225.00
haunch =	1.25				138.00
Es =	29,000,000.00		Ec =	3,834,253.51	
f'c =	4,000.00		n =	7.56	
wgt of conc. (w) =	150.00				

W 36 X 260	Area =	76.50	Ix =	17,300.00
	d =	36.26	bf =	16.55
			tf =	1.44

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*

Spans 95' to 30' AISI - 40 ft roadway with four composite beams w/o cover plates

Slab	182.46	24.38	4,448.32	108,449.94	1,520.48
W 36 X 260	76.50	0.00	0.00	0.00	17,300.00
	258.96		4,448.32	108,449.94	18,820.48
	1.7983				

Ix = Io + Ay^2 =	127,270.42			
Centroid = Ay/A =	17.18	**1.43**		
Itr = Ix - Ay^2 =	50,858.24	**2.4527**	yt/rebr =	10.20
y top =	12.20		yb/rebr =	3.20
y bottom =	35.31		y splice =	1.67

Continuous Joint: Spring Constants	Steel Area (sq. in.)	Computed Length (inches)	Design Length (inches)	Equivalent Stiffness (lb/in)	Equivalent Stiffness (k/ft)
Top Mat = 9 X #5	2.76	16.47	24.00	3,335,000.00	**40,020.00**
Bottom Mat = 14 X #5	4.30	16.47	24.00	5,195,833.33	**62,350.00**
Splice Plate (use least for unequal spans)	23.83		14.00	49,366,285.71	**592,395.43**
Moment Splice (ft): Ix =	**0.8343**			Area =	**0.5313**

70 foot span

span (L) =	70.00			Limits of b eff
deck depth =	10.00	b eff =	138.00	210.00
haunch =	1.25			138.00
Es =	29,000,000.00	Ec =	3,834,253.51	
f'c =	4,000.00	n =	7.56	
wgt of conc. (w) =	150.00			

W 36X230	Area =	67.60	Ix =	15,000.00
	d =	35.90	bf =	16.47
			tf =	1.26

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	182.46	24.20	4,415.47	106,854.46	1,520.48
W 36X230	67.60	0.00	0.00	0.00	15,000.00
	250.06		4,415.47	106,854.46	16,520.48
	1.7365				

Ix = Io + Ay^2 =	123,374.94			
Centroid = Ay/A =	17.66	**1.47**		
Itr = Ix - Ay^2 =	45,407.27	**2.1898**	yt/rebr =	9.54
y top =	11.54		yb/rebr =	2.54
y bottom =	35.61		y splice =	0.92

Continuous Joint: Spring Constants	Steel Area (sq. in.)	Computed Length (inches)	Design Length (inches)	Equivalent Stiffness (lb/in)	Equivalent Stiffness (k/ft)
Top Mat = 9 X #5	2.76	16.47	24.00	3,335,000.00	**40,020.00**
Bottom Mat = 14 X #5	4.30	16.47	24.00	5,195,833.33	**62,350.00**
Splice Plate (use least for unequal spans)	20.75		14.00	42,986,700.00	**515,840.40**
Moment Splice (ft): Ix =	**0.7234**			Area =	**0.4694**

Spans 95' to 30' AISI - 40 ft roadway with four composite beams w/o cover plates

65 foot span

				Limits of b eff
span (L) =	65.00			
deck depth =	10.00	b eff =	138.00	195.00
haunch =	1.25			138.00
Es =	29,000,000.00	Ec =	3,834,253.51	
f'c =	4,000.00	n =	7.56	
wgt of conc. (w) =	150.00			

W 36 X 210	Area =	61.80	Ix =	13,200.00
	d =	36.69	bf =	12.18
			tf =	1.36

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	182.46	24.60	4,487.54	110,371.15	1,520.48
W 36 X 210	61.80	0.00	0.00	0.00	13,200.00
	244.26		4,487.54	110,371.15	14,720.48
	1.6962				

Ix = Io + Ay^2 =	125,091.63			
Centroid = Ay/A =	18.37	**1.53**		
Itr = Ix - Ay^2 =	42,645.66	**2.0566**	yt/rebr =	9.22
y top =	11.22		yb/rebr =	2.22
y bottom =	36.72		y splice =	0.65

Continuous Joint: Spring Constants	Steel Area (sq. in.)	Computed Length (inches)	Design Length (inches)	Equivalent Stiffness (lb/in)	Equivalent Stiffness (k/ft)
Top Mat = 9 X #5	2.76	16.47	24.00	3,335,000.00	**40,020.00**
Bottom Mat = 14 X #5	4.30	16.47	24.00	5,195,833.33	**62,350.00**
Splice Plate (use least for unequal spans)	16.56		14.00	34,312,800.00	**411,753.60**
Moment Splice (ft): Ix =	**0.6366**			Area =	**0.4292**

60 foot span

				Limits of b eff
span (L) =	60.00			
deck depth =	10.00	b eff =	138.00	180.00
haunch =	1.25			138.00
Es =	29,000,000.00	Ec =	3,834,253.51	
f'c =	4,000.00	n =	7.56	
wgt of conc. (w) =	150.00			

W 36X182	Area =	53.60	Ix =	11,300.00
	d =	36.33	bf =	12.08
			tf =	1.18

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	182.46	24.42	4,454.70	108,761.55	1,520.48

Spans 95' to 30' AISI - 40 ft roadway with four composite beams w/o cover plates

W 36X182	53.60	0.00	0.00	0.00	11,300.00
	236.06		4,454.70	108,761.55	12,820.48
	1.6393				

Ix = Io + Ay^2 =	121,582.03			
Centroid = Ay/A =	18.87	**1.57**		
Itr = Ix - Ay^2 =	37,516.23	**1.8092**	yt/rebr =	8.54
y top =	10.54		yb/rebr =	1.54
y bottom =	37.04		y splice =	(0.12)

Continuous Joint: Spring Constants	Steel Area (sq. in.)	Computed Length (inches)	Design Length (inches)	Equivalent Stiffness (lb/in)	Equivalent Stiffness (k/ft)
Top Mat = 9 X #5	2.76	16.47	24.00	3,335,000.00	**40,020.00**
Bottom Mat = 14 X #5	4.30	16.47	24.00	5,195,833.33	**62,350.00**
Splice Plate (use least for unequal spans)	14.25		14.00	29,514,750.00	**354,177.00**
Moment Splice (ft): Ix =	**0.5449**			Area =	**0.3722**

55 foot span

span (L) =	55.00			Limits of b eff
deck depth =	10.00	b eff =	138.00	165.00
haunch =	1.25			138.00
Es =	29,000,000.00	Ec =	3,834,253.51	
f'c =	4,000.00	n =	7.56	
wgt of conc. (w) =	150.00			

W 36 X 160	Area =	47.00	Ix =	9,750.00
	d =	36.01	bf =	12.00
			tf =	1.02

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	182.46	24.26	4,425.51	107,340.71	1,520.48
W 36 X 160	47.00	0.00	0.00	0.00	9,750.00
	229.46		4,425.51	107,340.71	11,270.48
	1.5935				

Ix = Io + Ay^2 =	118,611.19			
Centroid = Ay/A =	19.29	**1.61**		
Itr = Ix - Ay^2 =	33,257.17	**1.6038**	yt/rebr =	7.97
y top =	9.97		yb/rebr =	0.97
y bottom =	37.29		y splice =	(0.77)

Continuous Joint: Spring Constants	Steel Area (sq. in.)	Computed Length (inches)	Design Length (inches)	Equivalent Stiffness (lb/in)	Equivalent Stiffness (k/ft)
Top Mat = 9 X #5	2.76	16.47	24.00	3,335,000.00	**40,020.00**
Bottom Mat = 14 X #5	4.30	16.47	24.00	5,195,833.33	**62,350.00**
Splice Plate (use least for unequal spans)	12.24		14.00	25,354,285.71	**304,251.43**
Moment Splice (ft): Ix =	**0.4702**			Area =	**0.3264**

Spans 95' to 30'

AISI - 40 ft roadway with four composite beams w/o cover plates

50 foot span

				Limits of b eff
span (L) =	50.00			
deck depth =	10.00	b eff =	138.00	150.00
haunch =	1.25			138.00
Es =	29,000,000.00	Ec =	3,834,253.51	
f'c =	4,000.00	n =	7.56	
wgt of conc. (w) =	150.00			

W 36X135	Area =	39.70	Ix =	7,800.00
	d =	35.55	bf =	11.95
			tf =	0.79

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	182.46	24.03	4,383.54	105,314.63	1,520.48
W 36X135	39.70	0.00	0.00	0.00	7,800.00
	222.16		4,383.54	105,314.63	9,320.48
	1.5428				

Ix = Io + Ay^2 =	114,635.11			
Centroid = Ay/A =	19.73	**1.64**		
Itr = Ix - Ay^2 =	28,140.41	**1.3571**	yt/rebr =	7.29
y top =	9.29		yb/rebr =	0.29
y bottom =	37.51		y splice =	(1.56)

Continuous Joint: Spring Constants		Steel Area (sq. in.)	Computed Length (inches)	Design Length (inches)	Equivalent Stiffness (lb/in)	Equivalent Stiffness (k/ft)
Top Mat = 9 X #5		2.76	16.47	24.00	3,335,000.00	**40,020.00**
Bottom Mat = 14 X #5		4.30	16.47	24.00	5,195,833.33	**62,350.00**
Splice Plate (use least for unequal spans)		9.44		14.00	19,555,321.43	**234,663.86**
Moment Splice (ft):	Ix =	**0.3762**			Area =	**0.2757**

45 foot span

				Limits of b eff
span (L) =	45.00			
deck depth =	10.00	b eff =	135.00	135.00
haunch =	1.25			138.00
Es =	29,000,000.00	Ec =	3,834,253.51	
f'c =	4,000.00	n =	7.56	
wgt of conc. (w) =	150.00			

W 33 X 130	Area =	38.30	Ix =	6,710.00
	d =	33.09	bf =	11.51
			tf =	0.86

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	178.49	22.80	4,068.70	92,746.13	1,487.43
W 33 X 130	38.30	0.00	0.00	0.00	6,710.00

Spans 95' to 30' AISI - 40 ft roadway with four composite beams w/o cover plates

	216.79		4,068.70	92,746.13	8,197.43	
	1.5055					
Ix = Io + Ay^2 =		100,943.55				
Centroid = Ay/A =		18.77	**1.56**			
Itr = Ix - Ay^2 =		24,582.68	**1.1855**	yt/rebr =	7.03	
y top =		9.03		yb/rebr =	0.03	
y bottom =		35.31		y splice =	(1.80)	

Continuous Joint: Spring Constants	Steel Area (sq. in.)	Computed Length (inches)	Design Length (inches)	Equivalent Stiffness (lb/in)	Equivalent Stiffness (k/ft)
Top Mat = 9 X #5	2.76	16.47	24.00	3,335,000.00	**40,020.00**
Bottom Mat = 14 X #5	4.30	16.47	24.00	5,195,833.33	**62,350.00**
Splice Plate (use least for unequal spans)	9.84		14.00	20,385,032.14	**244,620.39**
Moment Splice (ft): Ix =	**0.3236**			Area =	**0.2660**

40 foot span

span (L) =	40.00			Limits of b eff
deck depth =	10.00	b eff =	120.00	120.00
haunch =	1.25			138.00
Es =	29,000,000.00	Ec =	3,834,253.51	
fc =	4,000.00	n =	7.56	
wgt of conc. (w) =	150.00			

W 30X108	Area =	31.70	Ix =	4,470.00
	d =	29.83	bf =	10.48
			tf =	0.76

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	158.66	21.17	3,358.01	71,072.34	1,322.16
W 30X108	31.70	0.00	0.00	0.00	4,470.00
	190.36		3,358.01	71,072.34	5,792.16
	1.3219				

Ix = Io + Ay^2 =	76,864.50			
Centroid = Ay/A =	17.64	**1.47**		
Itr = Ix - Ay^2 =	17,627.67	**0.8501**	yt/rebr =	6.52
y top =	8.52		yb/rebr =	(0.48)
y bottom =	32.56		y splice =	(2.35)

Continuous Joint: Spring Constants	Steel Area (sq. in.)	Computed Length (inches)	Design Length (inches)	Equivalent Stiffness (lb/in)	Equivalent Stiffness (k/ft)
Top Mat = 9 X #5	2.76	16.47	24.00	3,335,000.00	**40,020.00**
Bottom Mat = 14 X #5	4.30	16.47	24.00	5,195,833.33	**62,350.00**
Splice Plate (use least for unequal spans)	7.96		14.00	16,490,642.86	**197,887.71**
Moment Splice (ft): Ix =	**0.2156**			Area =	**0.2201**

35 foot span

Spans 95' to 30' AISI - 40 ft roadway with four composite beams w/o cover plates

span (L) =	35.00			Limits of b eff
deck depth =	10.00	b eff =	105.00	105.00
haunch =	1.25			138.00
Es =	29,000,000.00	Ec =	3,834,253.51	
f'c =	4,000.00	n =	7.56	
wgt of conc. (w) =	150.00			

W 30 X 90	Area =	26.40	Ix =	3,620.00
	d =	29.53	bf =	10.40
			tf =	0.61

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	138.83	21.02	2,917.44	61,309.94	1,156.89
W 30 X 90	26.40	0.00	0.00	0.00	3,620.00
	165.23		2,917.44	61,309.94	4,776.89
	1.1474				

Ix = Io + Ay^2 =	66,086.83			
Centroid = Ay/A =	17.66	**1.47**		
Itr = Ix - Ay^2 =	14,573.04	**0.7028**	yt/rebr =	6.36
y top =	8.36		yb/rebr =	(0.64)
y bottom =	32.42		y splice =	(2.59)

Continuous Joint: Spring Constants		Steel Area (sq. in.)	Computed Length (inches)	Design Length (inches)	Equivalent Stiffness (lb/in)	Equivalent Stiffness (k/ft)
Top Mat = 9 X #5		2.76	16.47	24.00	3,335,000.00	**40,020.00**
Bottom Mat = 14 X #5		4.30	16.47	24.00	5,195,833.33	**62,350.00**
Splice Plate (use least for unequal spans)		6.34		14.00	13,141,142.86	**157,693.71**
Moment Splice (ft):	Ix =	**0.1746**			Area =	**0.1833**

30 foot span

span (L) =	30.00			Limits of b eff
deck depth =	10.00	b eff =	90.00	90.00
haunch =	1.25			138.00
Es =	29,000,000.00	Ec =	3,834,253.51	
f'c =	4,000.00	n =	7.56	
wgt of conc. (w) =	150.00			

W 24X76	Area =	22.40	Ix =	2,100.00
	d =	23.92	bf =	8.99
			tf =	0.68

Element	*Transformed Area A*	*Moment Arm from Centroid y*	*Ay*	*Ay^2*	*Io*
Slab	118.99	18.21	2,166.88	39,458.92	991.62
W 24X76	22.40	0.00	0.00	0.00	2,100.00
	141.39		2,166.88	39,458.92	3,091.62

Section Properties

5/23/96

Spans 95' to 30' AISI - 40 ft roadway with four composite beams w/o cover plates

	0.9819				
Ix = Io + Ay^2 =	42,550.54				
Centroid = Ay/A =	15.33	**1.28**			
Itr = Ix - Ay^2 =	9,342.80	**0.4506**	yt/rebr =	5.88	
y top =	7.88		yb/rebr =	(1.12)	
y bottom =	27.29		y splice =	(3.03)	

Continuous Joint: Spring Constants	Steel Area (sq. in.)	Computed Length (inches)	Design Length (inches)	Equivalent Stiffness (lb/in)	Equivalent Stiffness (k/ft)
Top Mat = 9 X #5	2.76	16.47	24.00	3,335,000.00	**40,020.00**
Bottom Mat = 14 X #5	4.30	16.47	24.00	5,195,833.33	**62,350.00**
Splice Plate (use least for unequal spans)	6.11		14.00	12,663,057.14	**151,956.69**
Moment Splice (ft): Ix =	**0.1013**			Area =	**0.1556**

Development Length = 0.028 * (Area of bar) * (fy) / (f'c)^0.5

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Computed Length = 2 X Development Length

* Reinforcement fy = 60,000 psi