



A feasibility study of the use of torsional and moment carrying external diaphragms on a double steel box girder bridge system to reduce live load moment
by Todd Sean Nottingham

A thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering MONTANA STATE UNIVERSITY Bozeman, Montana December 1986
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Abstract:

The purpose of this paper was to determine the feasibility of using external load carrying diaphragms for reduction in live load bending moment produced from eccentric loading.

In the present study the most effective system for intermediate span box girder bridges of 120 and 180 feet was found to include one intermediate moment carrying diaphragm located at the center of the bridge. Design bridges were compared with bridges that did not have the diaphragm and a reasonable savings in material was observed for the 180 foot span which used the diaphragm.

A FEASIBILITY STUDY OF THE USE OF TORSIONAL AND MOMENT
CARRYING EXTERNAL DIAPHRAGMS ON A DOUBLE STEEL BOX GIRDER
BRIDGE SYSTEM TO REDUCE LIVE LOAD MOMENT

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MONTANA STATE UNIVERSITY
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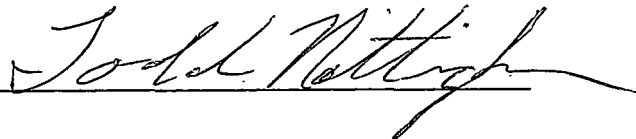
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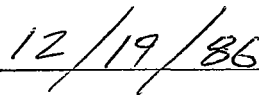
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ABSTRACT

The purpose of this paper was to determine the feasibility of using external load carrying diaphragms for reduction in live load bending moment produced from eccentric loading.

In the present study the most effective system for intermediate span box girder bridges of 120 and 180 feet was found to include one intermediate moment carrying diaphragm located at the center of the bridge. Design bridges were compared with bridges that did not have the diaphragm and a reasonable savings in material was observed for the 180 foot span which used the diaphragm.

CHAPTER 1

INTRODUCTION

Description of a Steel Box Girder Bridge

A box girder bridge may be constructed as simply supported, continuous, cable stayed, or as a suspension bridge. A box girder bridge cross-section usually consists of a single or a multi-cell box with a concrete deck that may be composite.

The shape of the box girders used in a bridge vary from a rectangular pattern to a trapezoidal shape with the webs inclined so that the top flange is the larger of the two flanges.

The number of girders used for a bridge vary from one to many. When only one girder is used the top plate of the girder is often used as the roadway deck.

The use of large thin plates for the construction of box girders often leads to the use of stiffeners to prevent buckling.

Description of Steel Box Girder Development

Steel box girder development is relatively new. This is primarily due to the development of welding practice, which has aided in the connection of the thin plates of the girder.

The concept of box girders is fairly old. Among the early box girders were those proposed by Roebling and Homere¹ in 1839, to span the Mississippi River. The proposed project which consisted of three tubular spans of 500 feet was not realized. The first tubular bridge built was the Britannia Bridge¹ built in England in 1845-1850, by

Robert Stephenson. After this first bridge several other bridges were designed by Robert Stephenson and others in the time period up to 1870.

After 1870 no box girders were built until the Cologne-Deutz Bridge¹ was built in Germany over the Rhine in 1948. Since that time there have been many box girder bridges built around the world.

Advantages of Box Girder Bridges

The steel box girder has several distinctive and preferred characteristics that make it a desirable structural member. Among these characteristics are:

- a) The potential of using larger unsupported flange widths than the conventional I-shaped girder. This allows for a greater span to depth ratio to be utilized, which can be an advantage where clearance is a problem.
- b) The box shape has a high torsional rigidity, which can be utilized to provide support for eccentric loading and for distribution of those loads in the transverse direction.
- c) Most of the fabrication of the box girder can be done in the shop permitting better control and minimizing the more costly field erection.
- d) The slender uncluttered appearance of a box girder can be regarded as one of the most aesthetic shapes, which can be pleasing in most rural and urban environments.
- e) The closed shape of the box girder reduces the effect of corrosion which occurs on outstanding edges, and for larger girders the interior can be utilized as a service passage.

Development of the Box Girder's Strengths

Box girders have many inherent strengths due to their configuration. Among these strengths is a strong torsional rigidity as well as strong bending rigidity. These strengths can be combined if the bridge girders are connected structurally in a grid pattern so that the girders act as a complete unit and not as separate elements.

The purpose of this paper is to determine the feasibility of using moment and torsional carrying external diaphragms on a double steel box girder bridge to aid in reducing live load bending moment in the girders produced from eccentric loading.

CHAPTER II

ANALYSIS

Beam Element Model

A double steel box girder bridge as shown in Figure 1 can be easily modeled using beam elements. The bridge deck cross section in Figure 1 shows eccentric loading resulting from a typical design truck loading. This eccentric loading will cause differential deflections (ΔA and ΔB) and rotations ($\Theta B1$, $\Theta B2$, $\Theta A1$, and $\Theta A2$) of the respective girders, as can be seen in Figures 2 and 3. The magnitude of the loads from the deck to each girder can be found by assuming simple beam supports at the centerline of the girders.

The differential deflections and rotations between the two girders can be used to develop a shear and couple transfer between the main girders. This shear and couple transfer is accomplished by use of external load carrying diaphragms. A grid model of the modified bridge showing the relative locations of the external diaphragms can be seen in Figure 4.

Two types of external diaphragms will be used in the model. The first is type "E" which is a box section with high torsional rigidity, and the second is type "I" a wide flange shape with high bending rigidity. Between girders A and B there is a differential rotation which is greatest at the ends of the girders. If the type "E" external box diaphragm is rigidly attached to the girders at these locations, a torque will develop in the diaphragm as can be seen in

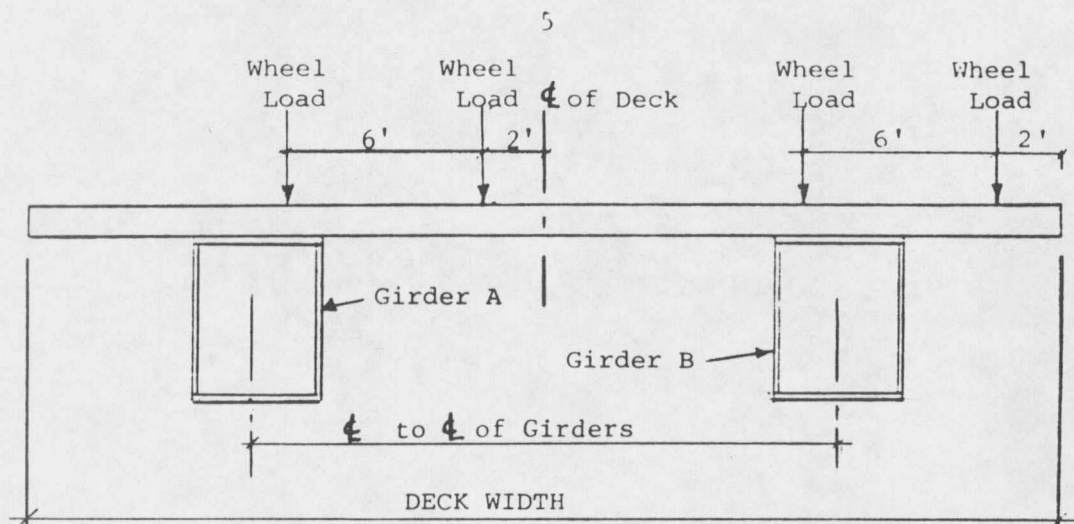


Figure 1. Cross Section of Bridge Deck Showing Eccentric Loads.

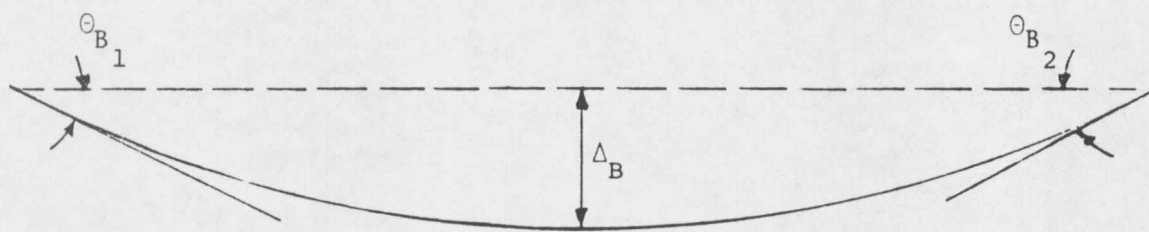


Figure 2. Diagram of Elastic Curve of Girder B.

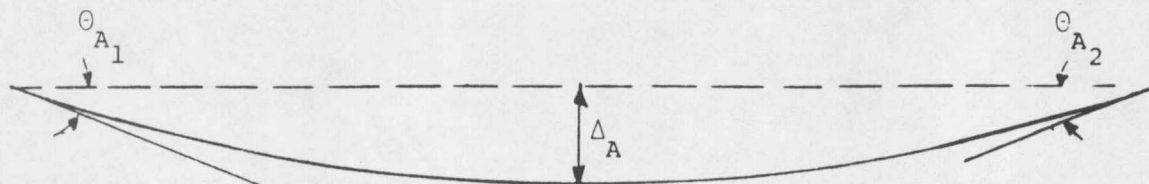


Figure 3. Diagram of Elastic Curve of Girder A.

Figure 5. The magnitudes of the couples can be determined from the relative rotations of the member ends as discussed by Weaver².

$$1) \quad ME1 = (\theta B - \theta A) * G * R / L$$

where:

θA = End rotations of girder A of the modified structure.

θB = End rotations of girder B of the modified structure.

G = Shear modulus

R = Torsional resistance factor

L = Length of diaphragm

If the "I" type diaphragms are rigidly attached near the center of the girders moments and shears will develop as can be seen in Figure 6(a). The mechanics of Figure 6(a) can be better understood by breaking the diagram into two respective components (Figures 6(b) and 6(c)). The magnitudes of these shear forces and moments can be found from the relative deflections and rotations of the member ends².

$$2) \quad MI = MA - M\theta$$

$$MA = \Delta I * 6 * E * I / L$$

$$M\theta = \theta I * 6 * E * I / (L * * 2)$$

$$3) \quad VI = VA - V\theta$$

$$VA = I * 12 * E * I / (L * * 2)$$

$$V\theta = I * 12 * E * I / (L * * 3)$$

where:

θI = Rotation of box girder about the x axis

ΔI = Relative z deflection between girders

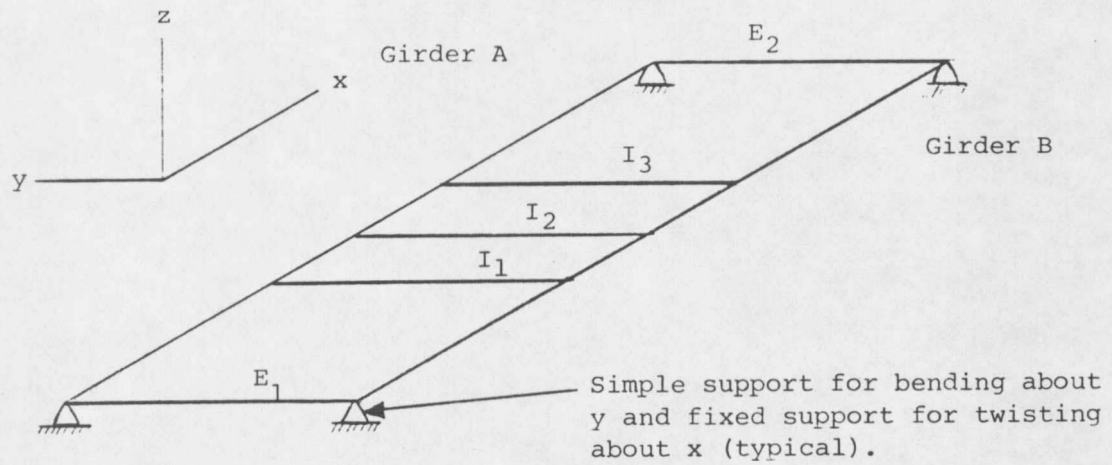


Figure 4. Modified Bridge Model with External Diaphragms.

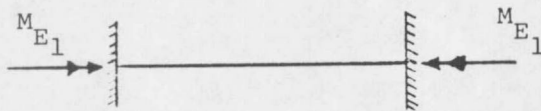


Figure 5. Diagram of Element E_1 and Loads.

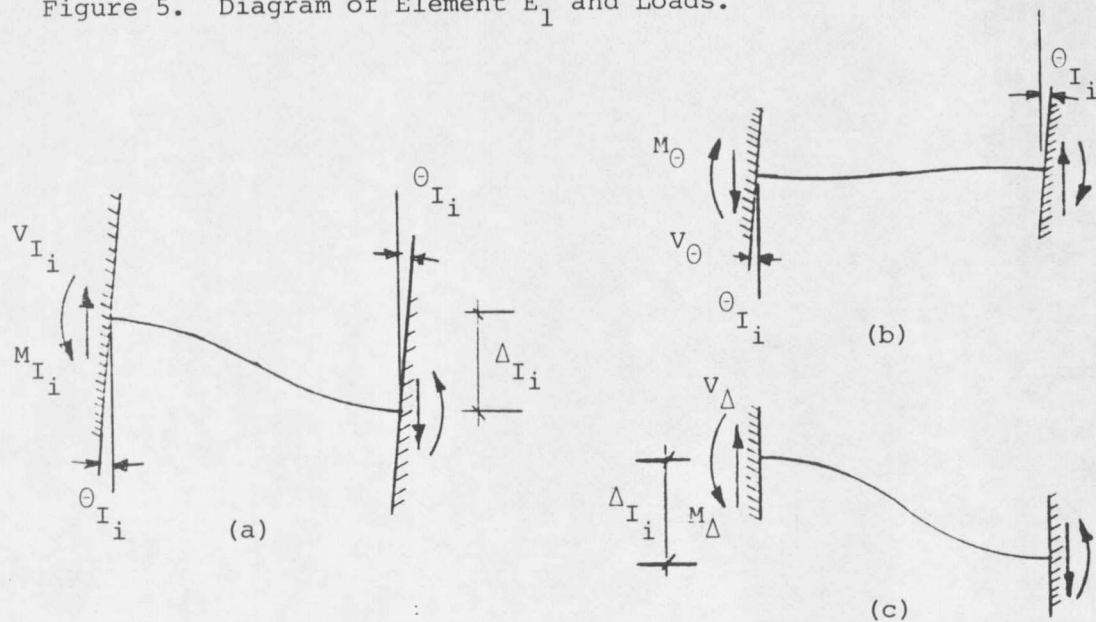


Figure 6. Diagrams of Element I_i Showing Loads and Load Components.

I = Moment of inertia

L = Length of Diaphragm

E = Modulus of Elasticity

The rotation of the box girder about the x axis will be small due to the torsional stiffness of the box shape. Therefore, the controlling component in the intermediate diaphragm shear and moment equations is the relative z deflection between the girders.

The effect of the moment and shear transfer between the two girders by means of the diaphragms is illustrated in Figures 7 and 8. As can be seen, the effect on the more heavily loaded girder B is to reduce the internal moment. While on the lighter loaded girder the effect is to increase the internal moment. This balancing of moments can nearly negate the effect of eccentric loading.

120 Foot Bridge Test Model

From the general model discussed, a model of working size was designed to test the interaction of the various bridge components.

The 120 foot test bridge has a 32 foot wide deck with the centerline to centerline spacing of the main box girders at 18 feet. The deck which is assumed to be made of precast concrete panels has a strength of $f'_c = 4000$ psi. The effective structural depth for the composite deck is 8 inches. The magnitude of the live loads to each girder can be found by assuming simple beam supports for the deck at the centerline of each girder.

Load Factor Design. The bridge will be designed using AASHTO (American Association of Highway and Transportation Officials)³ load factor design. Load factor design factors loads depending upon the

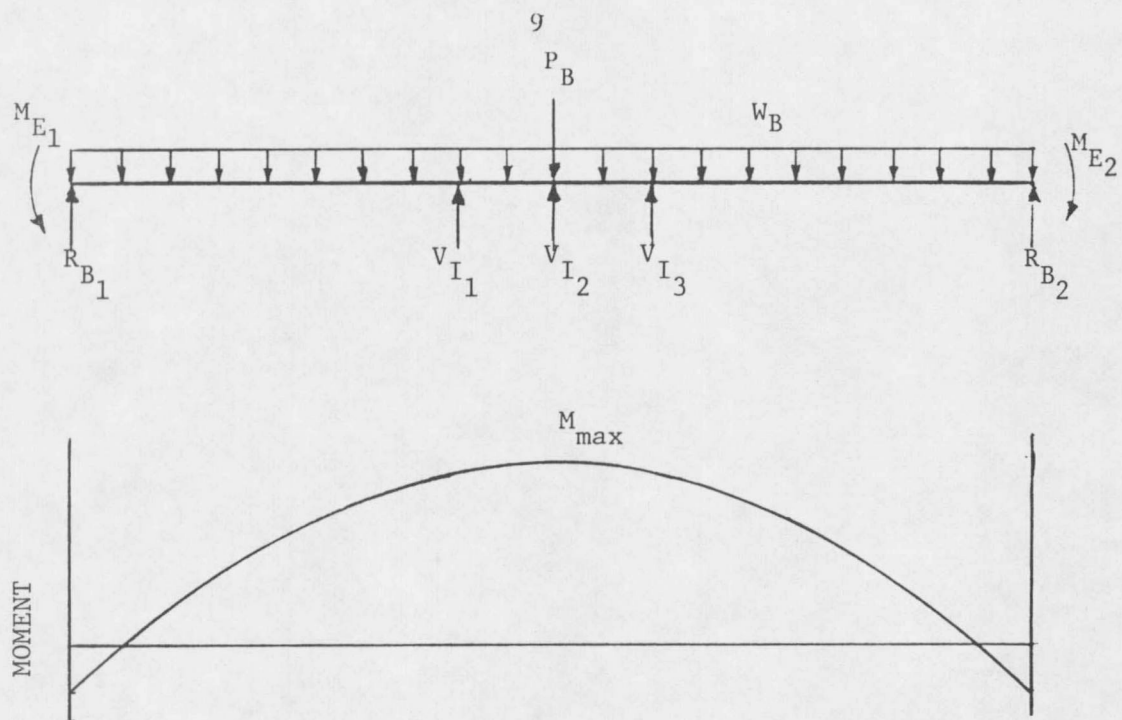


Figure 7. Girder B with Loads and Moment Diagram.

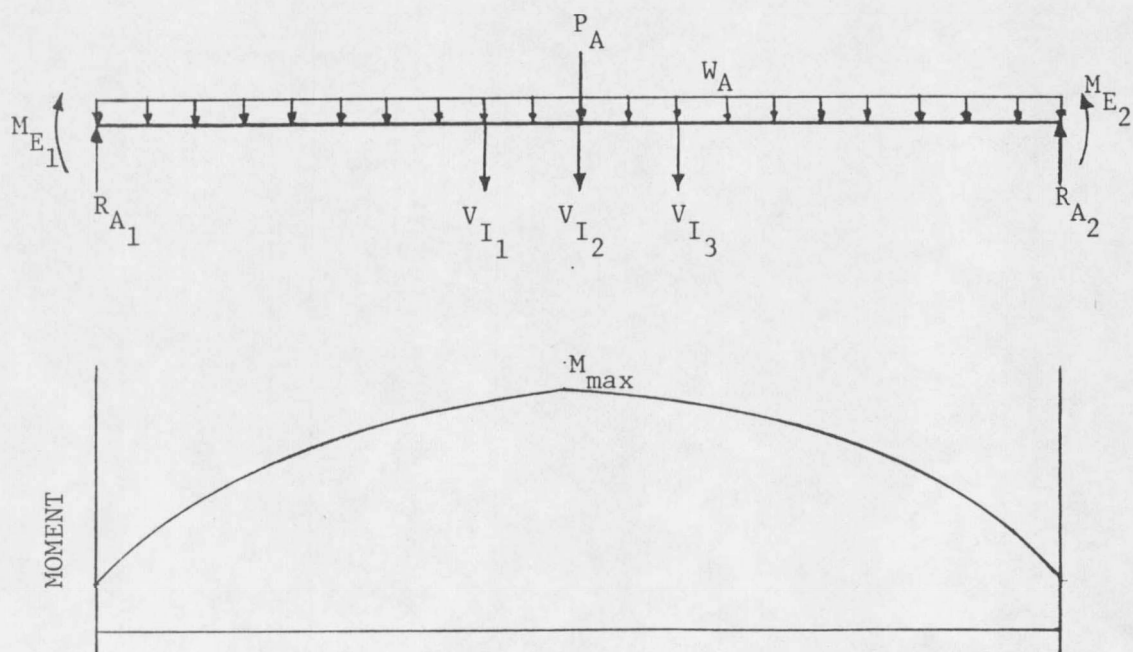


Figure 8. Girder A with Loads and Moment Diagram.

uncertainty of the load. For example, the dead load is fairly well known, therefore the load factor applied to this load is small. However, live load is highly variable so the load factor to this is higher. For this preliminary design work a combination of only dead plus live plus impact load will be reviewed. The total factor multiplied to the dead load will be 1.3. The total factor multiplied to the live plus impact loading will be 2.2.

The live load for case of simplicity was chosen as H20-44, which for a 120 foot structure consists of a uniform distributed load of 640 lb/ft and a concentrated load of 18,000 lb. per lane of traffic. Deck and rail weight of the bridge was taken as 4.4 kips/ft. Because the box girders that will be dealt with in this paper are non-compact sections, the maximum load for the girders was based on first yield of the flanges, which was calculated by:

$$f = \frac{MDL}{SS} + \frac{MLL}{SC}$$

where:

MDL = Moment due to dead weight of deck, rail, and girder

MLL = Moment due to HS20-44 truck loading

SS = Section modulus of steel box section

SC = Section modulus of composite section

Section Properties. The section properties of the cross sections of the main box girders can be calculated by the means shown:

$$I_{ST} = 2(bt_f d^2 + \frac{t_w d^3}{12})$$

$$I_c = I_{st} + y^2 A_{st} + (d + 4-y)^2 A_{eq}$$

$$A_{st} = 2(bt_f + dt_w)$$

$$A_{eq} = 96 \text{ in}^2$$

$$R_c = \frac{4b_c^2 d_c^2}{c}$$

as found in Blodgett (4)

Dimensions indicated can be seen in Figures 9-11. The equivalent transformed area A_{eq} was found by means outlined in the AASHTO code. The effective flange width was determined by the smallest of the following:

1. One-fourth of the span length for the girder.
2. The distance center to center of girders.
3. Twelve times the least thickness of the slab.

Condition 3. controlled, and the effective width was found to be 96 inches. Secondly, the transformed area was found by ratio of the modulus of elasticity of the steel versus the concrete, which was 8. The transformed area was 96 inches square located as seen in Figure 11.

Limitation on Flange Plate. As can be seen in the equations for calculating section properties, the width of the flange plate has a large impact on the torsional resistance factor. This factor is important for the torsional strength of the box section, so it is important to deal with wide plates.

Assuming that there will be no shoring of the bridge during construction, the compression flange which is unsupported during this time will be subject to plate buckling. The stress that must be carried by the unsupported compression flange results from the entire dead load. The governing criterion for design of the compression

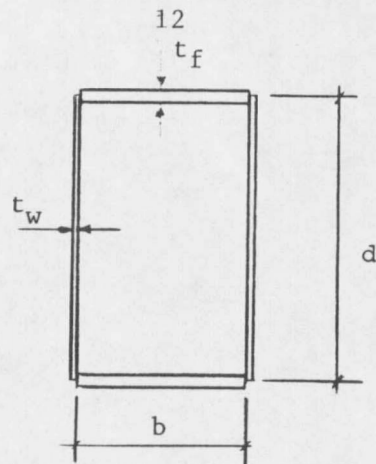


Figure 9. Typical Cross Section of Steel Box.

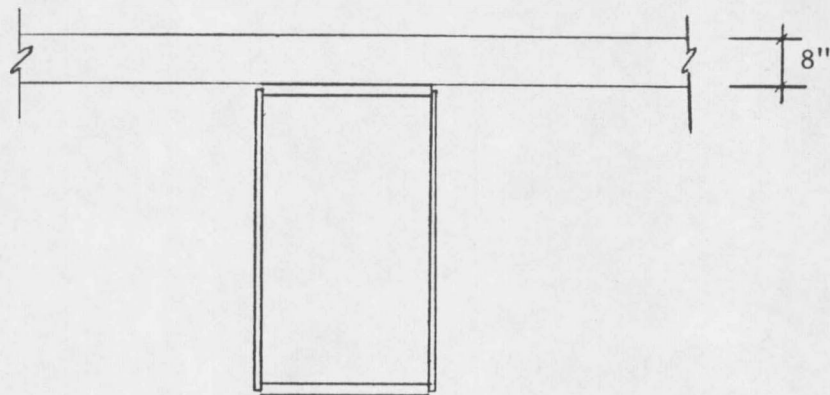


Figure 10. Typical Cross-section of Steel Box with Composite Deck.

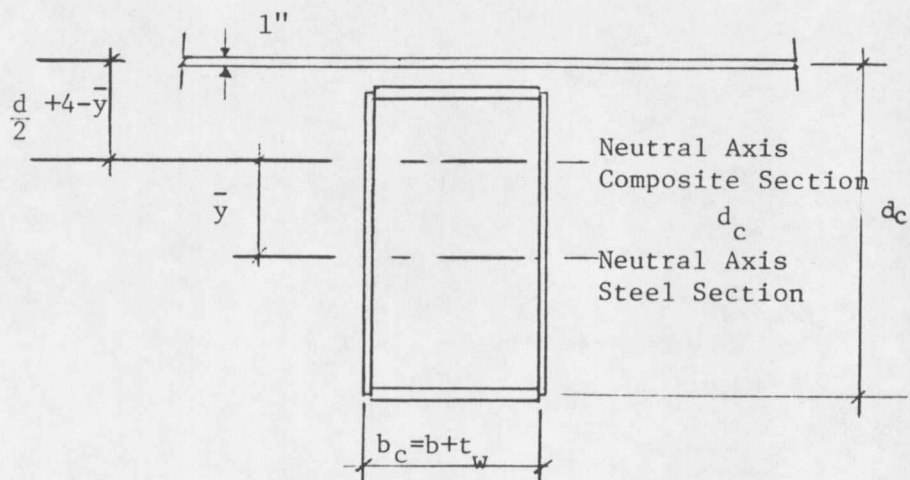


Figure 11. Test Model Box Section with Transformed Concrete.

flange during dead load loading will be initial elastic buckling. The flange width to flange thickness (b/t) ratio versus stress plot of critical buckling can be found by using the current AASHTO formulas or by theoretical formulas⁴ as can be seen on Figure 12. Plots of these formulas can be found in Figure 12. For the purposes of this design, the AASHTO formulas will be used.

Analysis of Structure by Stress

Once the configuration of the bridge has been approximated, a quick and accurate means of analysis for the structure is needed. This need is met by STRESS 11⁵.

Stress 11 (Structural Engineering Systems Solver) utilizes the direct stiffness method for solving deflections, internal forces, and external forces. The routine that was used for this bridge structure was the grid model. This model utilizes line elements for the structural member. These line elements are an approximation of the actual structure, because they neglect member and joint size effects. However, for the case of this design and many others, the scope of this program is adequate for the required analysis.

Combined Stresses. Once the internal forces are known, combinations of the forces can be analyzed to determine the stresses in the structure.

The stresses in the main box girder as discussed by Heins⁶ result from bending forces and torsional forces. The bending forces cause two main types of stress which are:

- a) Bending shear stress = V/A_w

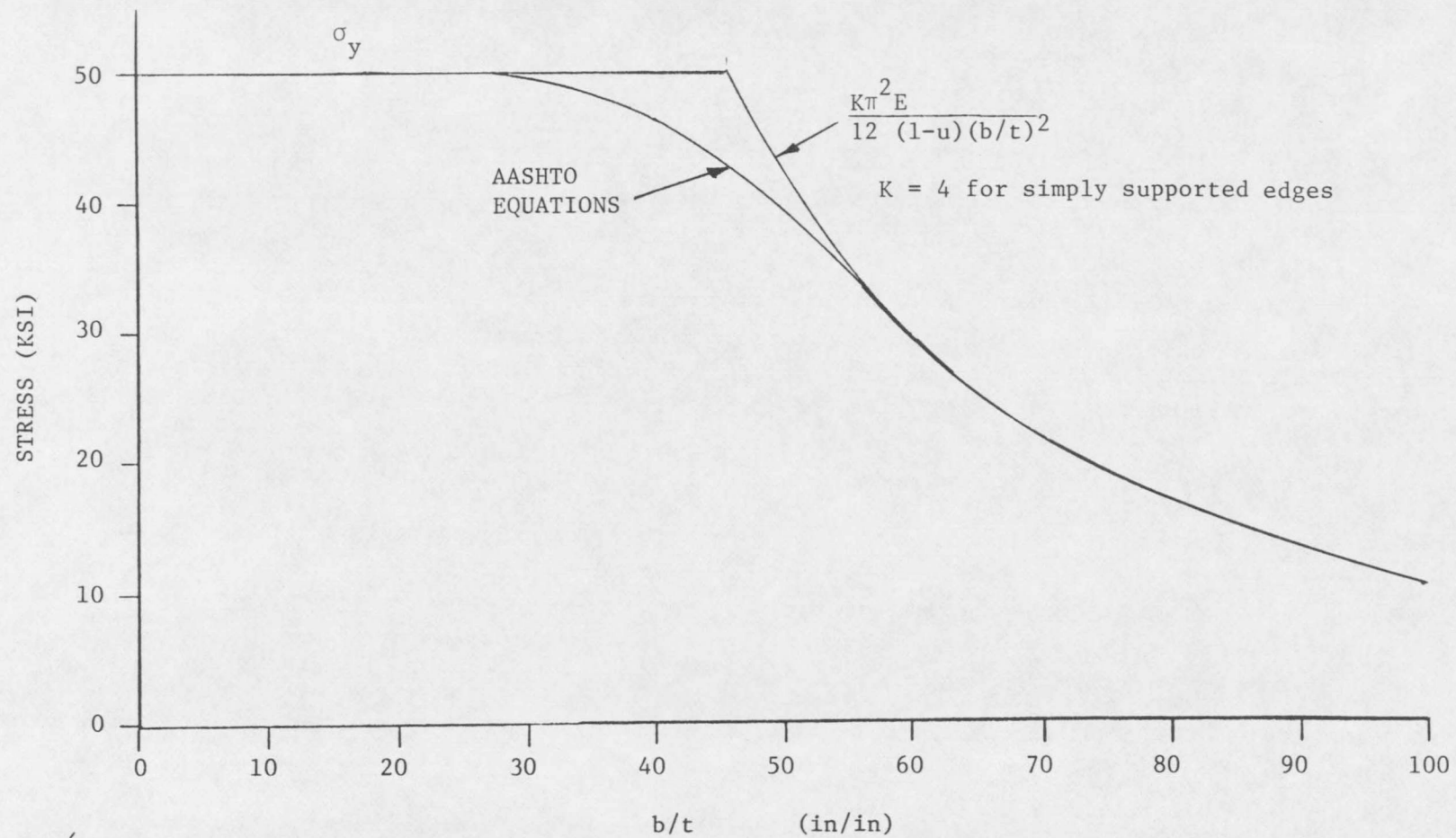


Figure 12. Graph of Equations Relating Elastic Buckling Stress to b/t . For 50 KSI Steel.

b) Bending normal stress = M/S

where:

V = shear force

A_w = area of web

The torsional loads result in three main types of stress:

- a) Torsional warping normal stress
- b) Torsional warping shear stress
- c) Torsional pure shear = $T/(2A*t)$

where:

T = Applied torque

A = Area enclosed by the box section

t = Thickness of the plate

The warping normal and warping shear stresses in the case of a box girder of this type are very small and will be neglected. Also, the combination of the bending shear and torsional shear stresses will not be of much concern because the maximum stresses in each case occur under different loading conditions.

Parameter Variation. Certain structural components of the members of the model bridge affect the behavior of the structure. These components affect the torsional and bending stiffnesses of the members.

To determine the best configuration of the bridge, certain components of the members were varied to determine the relative effects.

The components were:

- 1. l/d - Ratio of span length to depth of the main girders.
- 2. Torsionally stiff end diaphragms.

3. R - Torsional stiffness factor for main girder.
4. I - Moment of inertia of intermediate diaphragm.
5. Number of intermediate diaphragms.

The basis for determining the best configuration of the bridge was the maximum internal live load plus impact bending moment (MW) for the heavier loaded girder during eccentric loading. As a base of reference the live load plus impact loading internal moment for a girder (MWO) without external diaphragms was used. A ratio (MW/MWO) between these two moments then gives a good indication of the relative effect of the component in question.

The first parameter variation was the span to depth ratio of the main girders. Actual girders (Figure 13) were designed using the design guidelines previously stated. Results indicate that for the range of girders tested, the lower span to depth ratio will obtain a lighter section as can be seen from the graph (Figure 14). However, the costs external to that of the deck are not reflected in the cross sectional area of the girder. These additional costs include those associated with clearance excavation, and abutments. Thus, an arbitrary span to depth (L/d) ratio of twenty-five was chosen as desirable for the test model.

The second parameter variation was the inclusion or exclusion of the torsionally rigid end diaphragms. The girders obtained from the previous analysis were used to obtain the effectiveness of the end members. The results are demonstrated (Table 1) as a ratio of (MW/MWO) for the various (L/D) ratios with and without end diaphragms, but maintaining the middle diaphragms. As can be seen, there was no

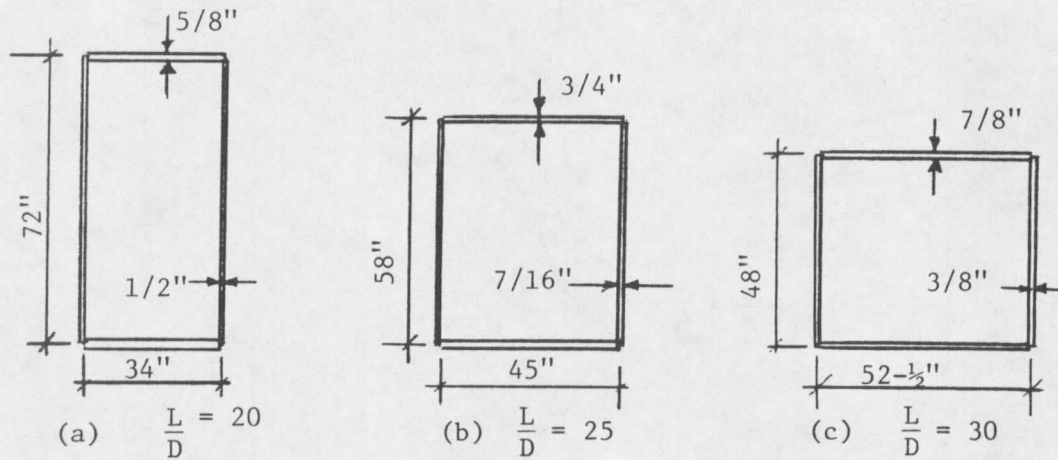


Figure 13. Design Girders for L/D Variation.

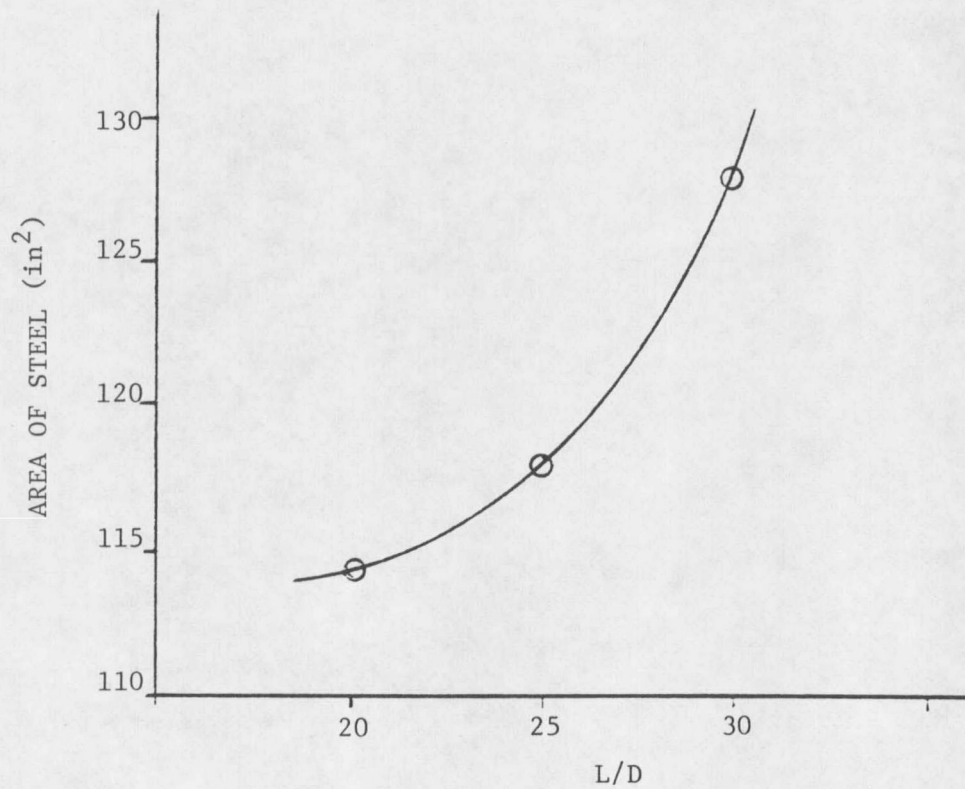


Figure 14. Graph Demonstrating Area of Steel Required Versus L/D.

beneficial effect of the end diaphragms on the structure. So the torsionally rigid end diaphragm was not included in the rest of the parameter study.

Table 1. Effect of end diaphragms on girder moments.

		L/D		
		20	25	30
$\frac{MW}{MWO} \times 100\%$	Bridge with Middle & End Diaphragm	79.3	79.9	81.2
	Bridge with Middle & without End Diaphragm	79.0	79.7	81.1

The next parameter variation was of the torsional resistance factor (R). The test model at this point is of a girder with an (l/d) ratio of twenty-five, three intermediate diaphragms, and no torsional end diaphragms. The results of the analysis can be seen in Figure 15. The various flange width to thickness (b/t) ratios for some of the obtained resistance factors are also shown. As can be seen, the higher the torsional stiffness, the better the load transfer. However, the larger the (b/t) ratio, the less the unsupported compression flange can carry during construction. Therefore, a (b/t) ratio of approximately sixty will be maintained on the rest of the parameter study.

The next parameter variation was the stiffness of the intermediate diaphragms. The stiffness component that was varied was the moment of inertia of the diaphragm. As can be seen in Figure 16, the larger the stiffness the better the load transfer. However, after

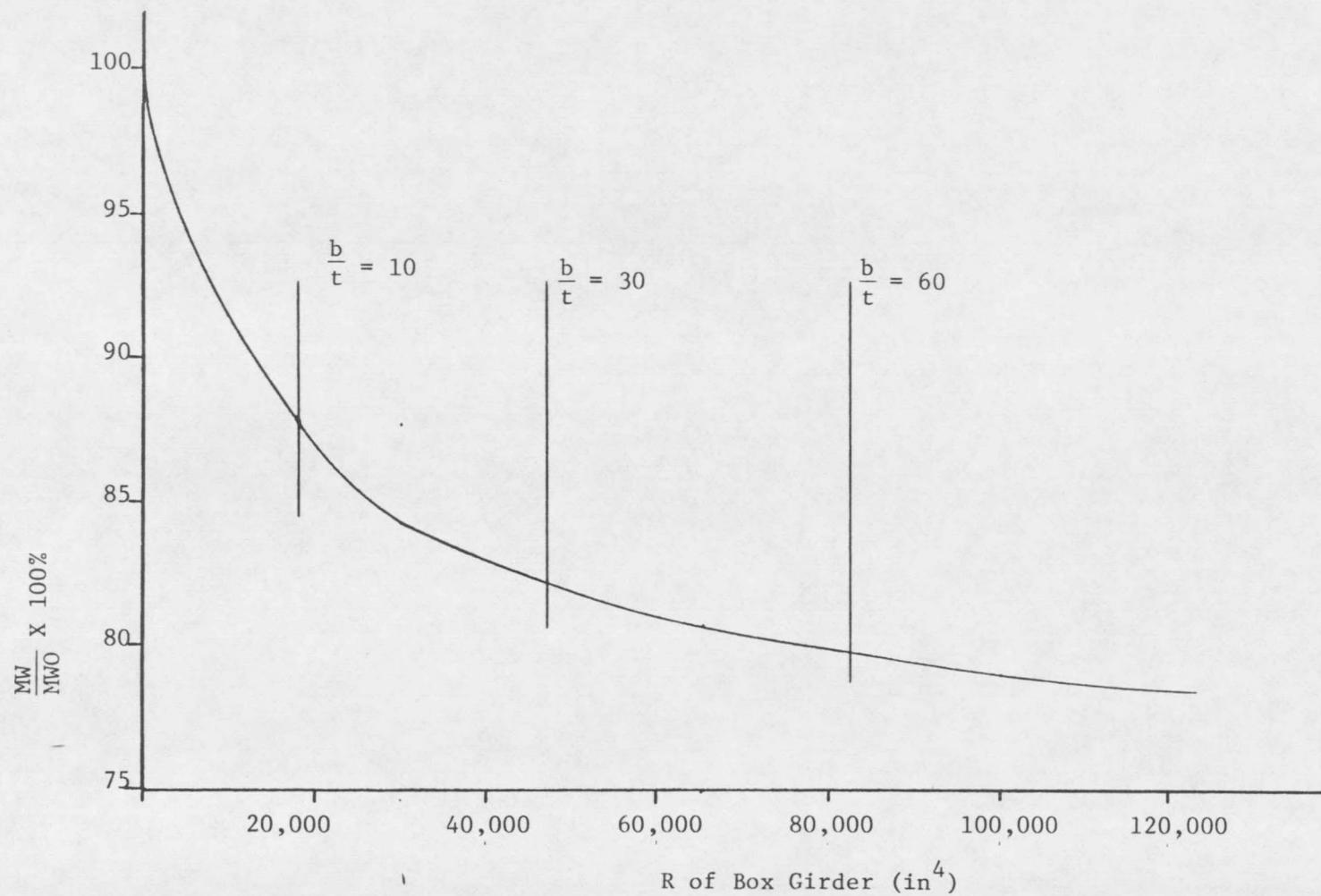


Figure 15. Effect of Torsional Resistance Factor R on Live Load Moment.

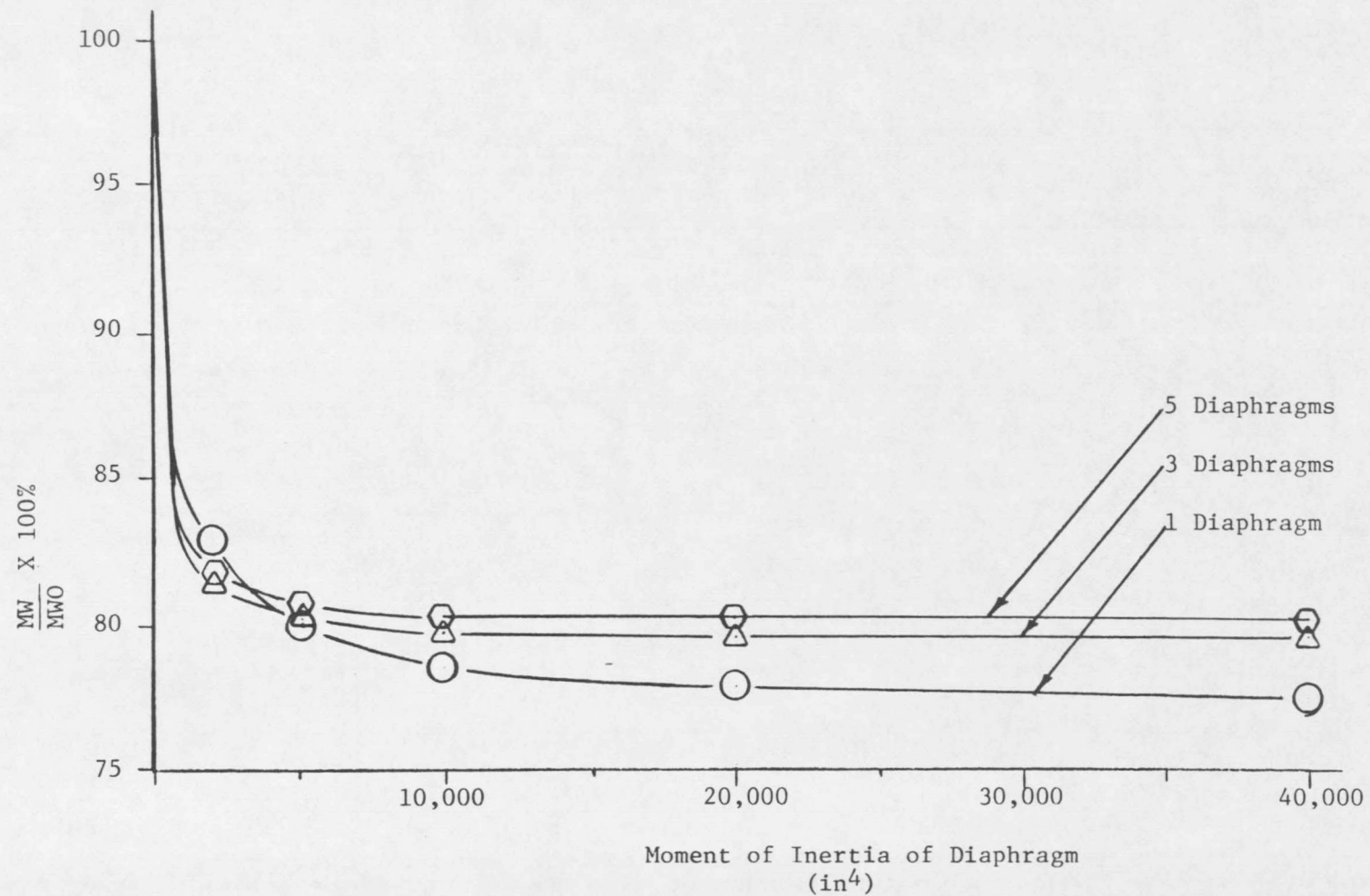


Figure 16. Effects of Moment of Inertia and Number of Diaphragms on Live Load Moment.

a moment of inertia of approximately $10,000 \text{ in}^4$ the effects of continued stiffness increases are negligible.

Also shown on Figure 16 is the variation in the number of intermediate diaphragms. One, three, and five diaphragms were considered. The results showed that the best number and spacing of the diaphragms was one diaphragm spaced directly at the centerline of the bridge.

Based on the results of the parameter study, the following general observations can be made:

1. The torsional box end diaphragm has no relative effect when used in conjunction with the intermediate diaphragms. Therefore, the torsional box end diaphragm is not considered useful for this type of structure.
2. The larger the torsional stiffness factor (R), the larger the decrease in live plus impact bending moment. However, care must be taken so that buckling of the compression flange does not occur when the b/t ratio is increased. A b/t ratio of 70 will be considered as a useful maximum.
3. The size of the intermediate diaphragms after a moment of inertia of $10,000 \text{ in}^4$ is reached does not affect the moment reduction to any significant degree.
4. The number of intermediate diaphragms for best results is one located at the center of the girders.

CHAPTER III

PRELIMINARY DESIGN

To demonstrate the improvement in design that the use of the intermediate diaphragm has on the structure, two bridges of span lengths 120 feet and 180 feet were designed. Moment envelopes for the live plus impact loading for these bridges were developed and compared with the moment envelopes for the same bridges without the intermediate diaphragms. From the moment envelope comparison, the magnitude of the beneficial effect was measured. Also, fabrication and construction methods and problems were discussed.

120 Foot Bridge Design

The 120 foot bridge was designed using HS20-44 loading. The bridge consisted of a precast concrete deck that acts compositely for live loading, two steel box girders, and one intermediate diaphragm.

A precast deck was chosen for this structure because of the span to form the deck between the girders is long, and also because another aspect of this type of design is to simplify field construction. Diagrams illustrating the precast panels can be seen in Figures 17-19. Variation in design of a deck of this type should be considered. For example, post tensioning the deck after grouting the joints could result in a lighter deck and improve servability.

The 120 foot bridge designed can be seen in Figures 20 and 21. Three different flange sizes were used for the bridge to aid in economy. Grout spacing was used to provide better continuity between

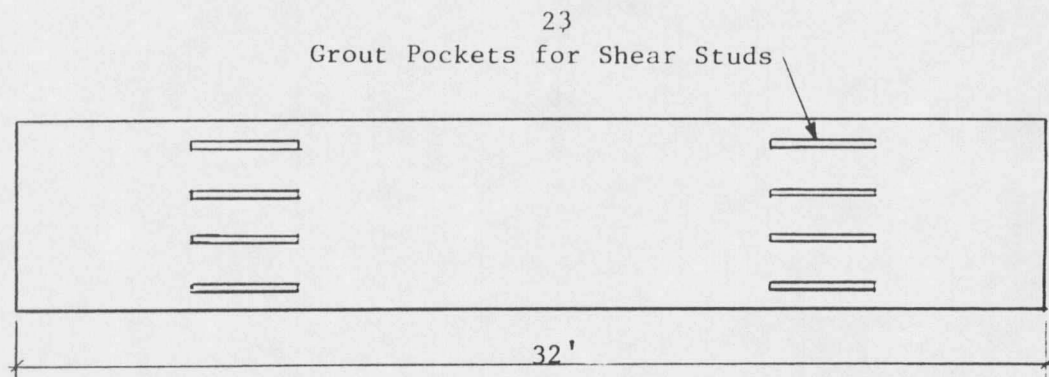


Figure 17. Plan View of Concrete Deck Panel.

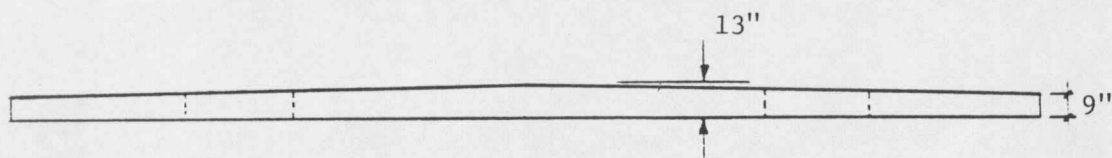


Figure 18. Elevation of Concrete Deck Panel.

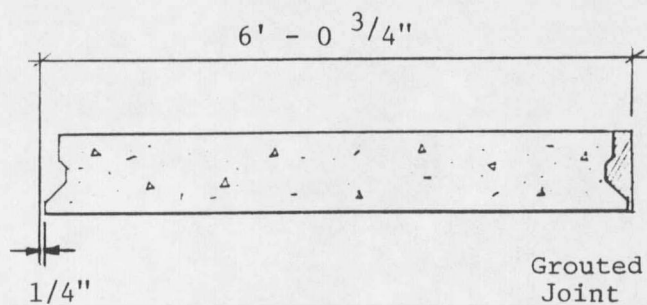


Figure 19. Cross-section Concrete Panel Showing Centerline to Centerline of Dimensions of Panel.

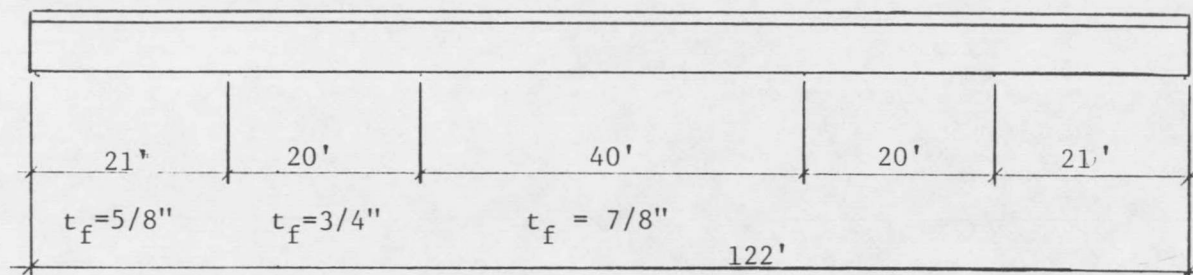


Figure 20. Elevation of 120 Foot Girder Showing Flange Size Changes.

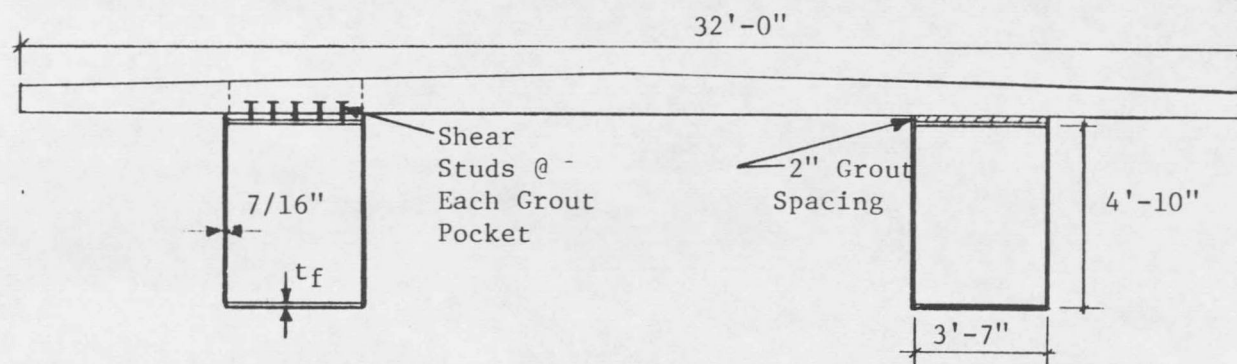


Figure 21. Cross-section of 120 Foot Bridge.

the deck and girders. The central intermediate diaphragm required a completely rigid connection to the main girders.

Two examples of possible intermediate diaphragm connections can be seen in Figures 22-24. Figure 22, a plan view of the connection, demonstrates flange transition, which is particularly needed in the tension region for fatigue reasons. Figures 23 and 24 show two possible configurations for the internal diaphragm which is needed to maintain the box configuration of the girder during torsional loading. Since the stress range for live load is not great and does not demonstrate complete reversal on the main girders, welding to the tension flange can be used for the connection. Figure 23 has the internal diaphragm welded on all four sides of the box. This connection can be difficult to fabricate so an alternate connection is also shown. This alternate (Figure 24) uses an internal diaphragm which is not welded to the exterior web of the girder, but does have a stiffener for the free edge of the internal diaphragm. These connections are of a preliminary nature and further design is needed for sizing.

The effect of the intermediate diaphragm can be demonstrated by comparing moment envelopes of the designed bridge with the same bridge without the intermediate diaphragm. These moment envelopes can be seen in Figure 25.

A sizing of a bridge that does not have the intermediate diaphragm can be developed by comparing the design live load moments of the two bridge types from the moment envelopes. Each section of the bridge without the diaphragm will require an increase in area of

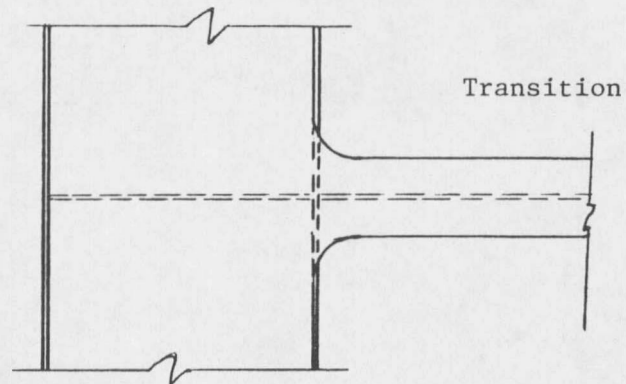


Figure 22. Plan View of Diaphragm Connection Showing Flange Transition.

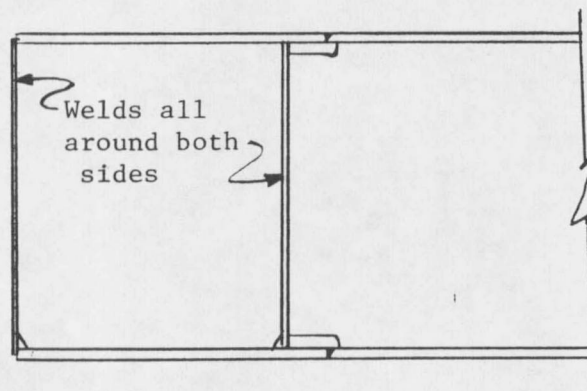


Figure 23. Cross-section of Girder Showing Possible Diaphragm Connection.

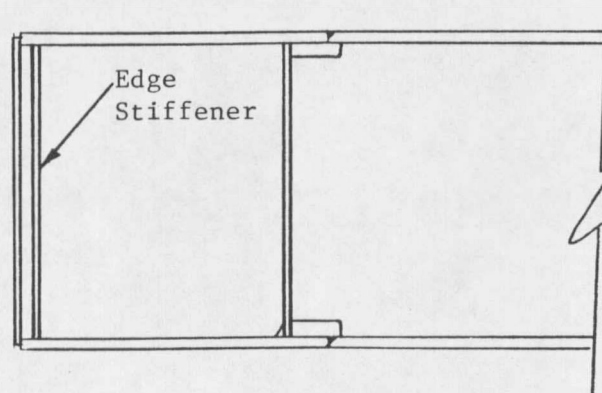


Figure 24. Cross-section of Girder Showing Alternate Diaphragm Connection.

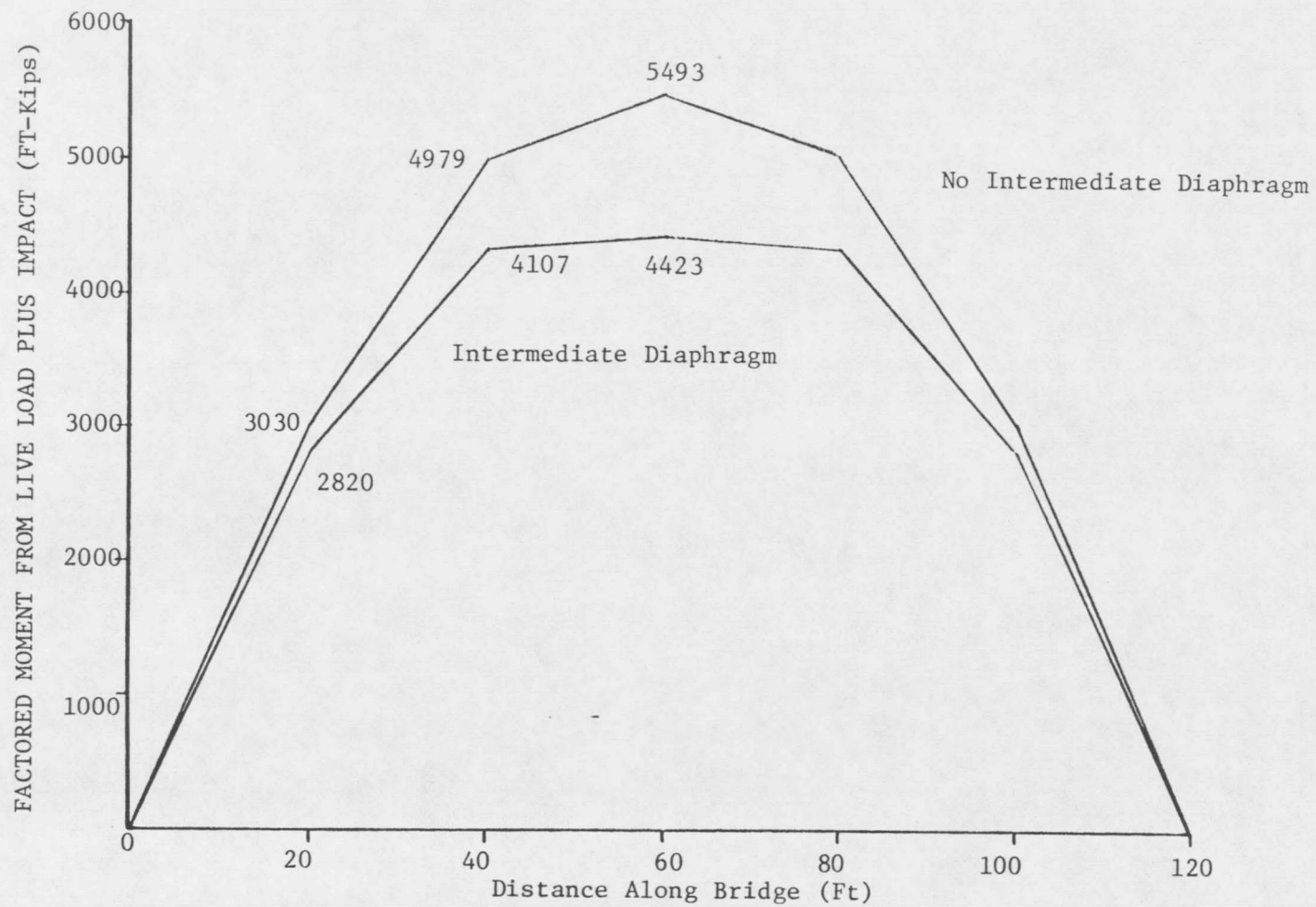


Figure 25. Comparison of Moment Envelopes of 120 foot Girder With and Without a Central External Diaphragm.

the flange size proportional to the increase in moment compared to the original. Assuming that the live load stress is 40% of the total stress of the section, the increase in flange sizes can be determined as follows:

Table 2. Increased flange area required if intermediate diaphragms are omitted.

<u>SECTION</u>	<u>FLANGE AREA</u>	<u>MOMENT CHANGE</u>	<u>NEW FLANGE AREA</u>
1	53.8 sq. in.	7.45%	55.4 sq. in.
2	64.5 sq. in.	15.1%	68.4 sq. in.
3	75.3 sq. in.	24.2%	82.6 sq. in.

The average flange area has increased from 64.5 sq. in. to 68.8 sq.in. or a change of 4.3 sq.in. for the whole bridge. The flange area for the designed bridge is 56.1% of the total area, therefore a total change of 3.7% in total area of the bridge is needed if an intermediate moment carrying diaphragm is not used in the structure. A quantity takeoff of steel for both bridge girders minus any stiffener and miscellaneous steel yields 94,800 lbs. of steel. The savings in quantity of steel for the girders is then 3,500 lbs. The weight of the intermediate diaphragm is estimated at 146 lbs/ft. over a distance of 14 feet. The total savings in steel is then approximately 1500 lbs. or 1.5%.

180 Foot Bridge Design

The 180 foot span with a span to depth ratio of thirty (Figures 26 and 27) was developed using the same deck as on the 120 foot structure and HS20-44 loading. The loading that was used was an equivalent lane loading that controls for this span length. However, due to the shape of the curvature diagram due to the lane loading and the presence of the intermediate diaphragm, the equivalent lane loading actually produced a higher moment in the lighter loaded girder (an explanation for this can be found in the Appendix). This was deemed not to represent the actual condition. A corrected loading was determined by the following procedure:

1. A truck loading was placed in each lane to determine the maximum moment in the heavier loaded girder.
2. A ratio of moments between the girders was found from the truck loading.
3. The ratio was used to determine the amount of moment to each beam from the equivalent lane loading.

Lane loading was then used to further determine the moment envelope for other points on the 180 foot structure (Figure 28).

Following the same procedure done for the 120 foot structure and assuming that factored live load stress result in 33% of the total stress, steel savings for the main girders was 4.6%. The total weight of the girders minus miscellaneous steel is 230,400 lbs. This means that a total weight reduction of 10,600 lbs. for the main girders was realized. The diaphragm weight was established at 275 lbs./ft. over a

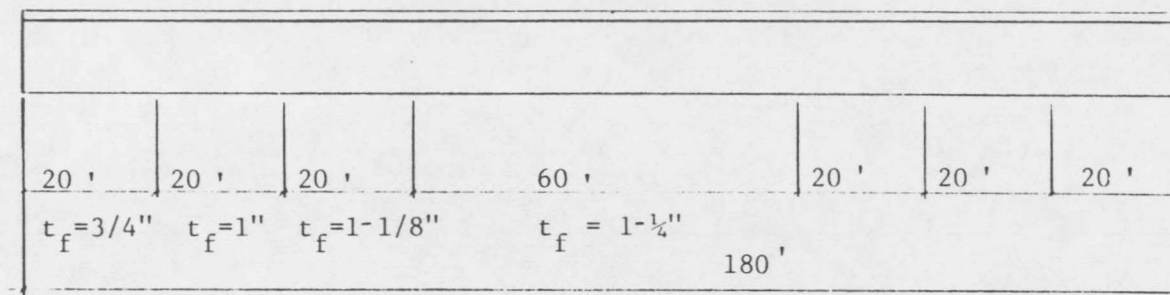


Figure 26. Elevation of 180 Foot Bridge.

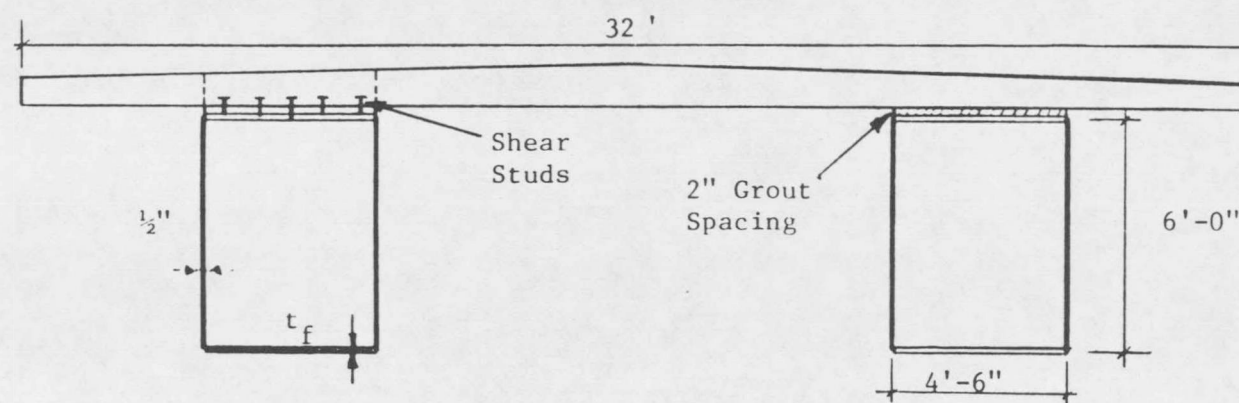


Figure 27. Cross-section of 180 Foot Bridge.

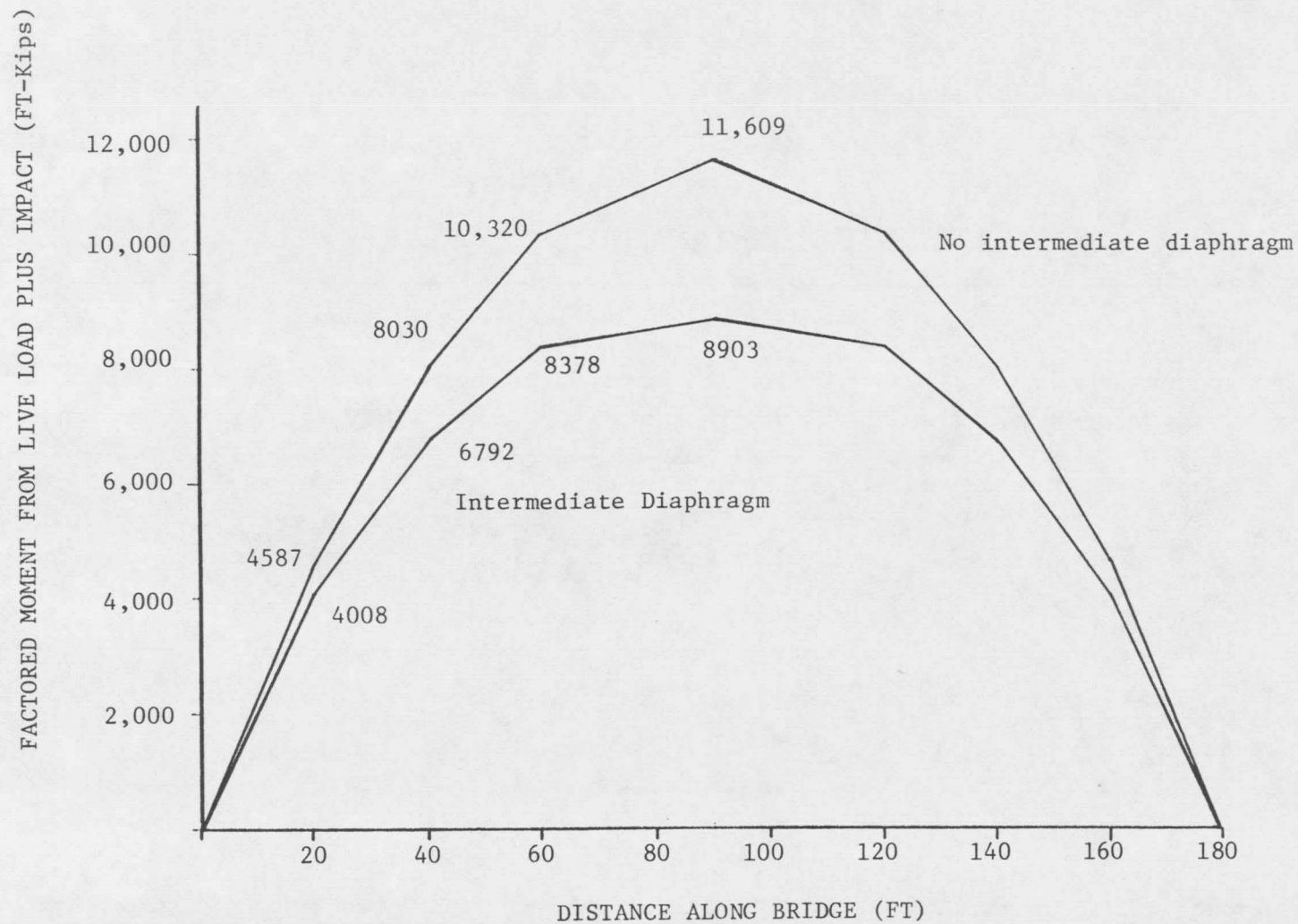


Figure 28. Comparison of Moment Envelopes of 180 Foot Girders With and Without A Central External Diaphragm.

length of 13.5 feet. The net savings in steel for this bridge was then 6900 lbs. or 3.0%.

Fabrication and Construction

The reasons for development of this type of bridge were not only to demonstrate cost effectiveness of the intermediate diaphragm, but also to demonstrate an easily fabricated and constructed bridge alternative.

Fabrication as discussed with Barz⁷ of the closed shape box girder closely resembles the procedure for that of a plate girder. First the flanges where thickness changes occur are butt welded, preferably in a double V groove weld to minimize warping. The webs cut to a preferred camber are then welded to the flanges with minimum sized fillet welds. The intermediate diaphragm is then welded half to each girder to later be field bolted at its center after girders are placed at the abutments. Shear studs can be placed in the shop or in the field by the contractor.

To minimize transportation difficulties, both the 120 and 180 foot girders can be fabricated in three sections which can be field bolted at the construction site.

The box girders that have been developed are ideally suited for remote locations, because the prefabricated nature of the girders and deck minimize the field work necessary to complete the superstructure.

Total estimated cost of the 120 foot span as discussed with Hanson⁸ can be broken into the costs in Table 3 which are for rural area of the State of Montana.

Table 3. Cost estimate for a 120-foot span girder.

Fabrication - \$0.62/lb for A588 grade 50 weathering steel for	
102,000 lbs.	\$63,900
Construction -	
Concrete deck-shop	\$ 43,000
grout	4,000
transportation 200 miles	2,000
erection	8,000
girders-transportation 200 miles	2,000
erection	7,000
Total cost of superstructure excluding guard rails	<u>\$129,900</u>

Estimated costs are presented in order that the reader may make comparisons with alternative design options. It should be noted that these estimated costs are for a preliminary design, and could decrease for a final design.

CHAPTER IV

SUMMARY AND CONCLUSION

An effort was made to utilize the torsional strength of the box girder to improve the design of a steel double box girder bridge. The torsional strength of the box girder was used by addition of external diaphragms on an eccentrically loaded box girder bridge. The diaphragms reduced the internal bending moment on the heavier loaded girder.

The purpose of this paper was to determine the feasibility of using external load carrying diaphragms for reduction in live load bending moment produced from eccentric loading. Two types of external diaphragms were investigated. One type was a torsionally rigid diaphragm located at the ends of the main girders. The second type was an intermediate wide flange shape diaphragm with high bending rigidity. Various structural arrangements for a 120 foot test bridge were investigated. The performance of each structure was based on the internal live plus impact loading bending moment. General observations obtained from the various structural arrangements can be made:

1. The torsional box end diaphragms have no relative effect when used in conjunction with the intermediate diaphragms. Therefore, the torsional box end diaphragm is not considered useful for this type of structure.

2. The larger the torsional stiffness factor (R), the larger the decrease in live plus impact bending moment. However, care must be taken so that buckling of the compression flange does not occur when the b/t ratio is increased. A b/t ratio of 70 will be considered as a useful maximum.
3. The size of the intermediate diaphragms after 10,000 in 4 does not affect the moment reduction to any significant degree.
4. The number of intermediate diaphragms for best results is one located at the center of the girders.

Two bridges of span lengths 120 and 180 feet were designed using the general guidelines. A comparison based on the live plus impact bending moment between the designed girders with one intermediate external diaphragm and the designed girders without external diaphragms was done. The comparison resulted in an estimate of the savings resulting from the use of the diaphragm. The 120 foot structure showed a material savings of 1.5%. The 180 foot structure showed a material savings of 3.0%.

In conclusion, the double steel box girder system developed could be a competitive design particularly for use in remote areas. The use of the intermediate diaphragm for economical moment reduction in the cases reviewed does show a low savings potential. However, for higher span to depth ratios, as for the 180 foot structure, the moment carrying diaphragm is a greater recognizable benefit.

REFERENCES CITED

1. Yampa, F.X.T., Buckling Behavior of Steel Box Girder Bridges, Thesis in the Faculty of Engineering, Concordia University, Montreal, Quebec, Canada (April, 1981).
2. Weaver, W. Jr., Gere, J. M., Matrix Analysis of Framed Structures, D. Van Nostrand Company, New York, New York (1965).
3. Standard Specifications for Highway Bridges, The American Association of State Highway and Transportation Officials, Washington, D.C. (1984).
4. Blodgett, O. W., Design of Welded Structures, James F. Lincoln Arc Welding Foundation, Cleveland, Ohio (1966).
5. STRESS II, Civil Engineering and Engineering Mechanics Department, Montana State University, Bozeman, Montana (December 1985).
6. Heins, C. P., Bending and Torsion Interaction of Box Girders, ASCE Journal of the Structural Division Vol. 105, No. ST5, New York, New York (May 1979).
7. Barz, A., Engineer, Roscoe Steel, Billings, Montana, Private Communication (1986).
8. Hanson, H., Engineer, Cop Construction, Billings, Montana, Private Communication (1986).

REFERENCES

- Bezzone, A.P., A Comparison of Working Stress and Load Factor Design for a Composite Box Girder Bridge, Division of Highways, State of California (no publication date).
- Croce, C. A., Live Load Stresses in a Straight Box Girder Bridge, New York State Department of Transportation, Albany, New York (December 1973).
- Dowling, D. J., Strength of Steel Box Girder Bridges, ASCE Journal of the Structural Division Vol. 101, No. ST9, New York, New York (September 1975).
- Heins, C.P., Box Girder Bridge Design - State of the Art, AISC Engineering Journal, Vol. 15, No. 4, New York, New York (1978).
- Heins, C.P., Steel Box Girders - Design Guides and Methods, AISC Engineering Journal Vol. 20, No. 4, New York, New York (1983).
Manual of Steel Construction, American Society of Steel Construction, Chicago, Illinois (1980).
- McDonald, R. E., Chen, Y., Yilmaz, L., Yen, B.T., Open Steel Box Sections With Top Lateral Bracing, ASCE Journal of the Structural Division, Vol. 102. No. ST1 (January 1976).
- Nottingham, W. D., Engineer, Peratrovich, Nottingham and Drage, Inc., Anchorage, Alaska Private Communication (1986).
- Steel Box Girder Bridges - Ultimate Strength Considerations, by the Subcommittee on Ultimate Strength of Box Girders of the ASCE-ASSHTO Task Committee on Flexural Members of the Committee on Metals of the Structural Division. ASCE Journal of the Structural Division, Vol. 100, NO ST12 (December 1974).
- Wolchuk, R. Proposed Specifications for Steel Box Girder Bridges, ASCE Journal of the Structural Division, Vol. 106, No. ST12 (December 1980).

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