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FEASIBILITY STUDY OF STEEL-CONCRETE COMPRESSION FLANGE IN STEEL BOX GIRDERS

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16. Abstract

The feasibility of reduction and elimination of haunches through the use of composite compression flanges over the negative moment region of continuous steel box girders is studied. Three designs are used as the basis for the examination. Factors which may influence the strength of the steel-concrete composite compression flange are examined herein, include the thickness of steel plates, concrete slab thickness and length, strength, weight, shrinkage and creep of concrete and the requirements of shear connectors. Sequence of construction is recognized as a prime factor. Comparison of costs is made on the basis of cost per unit weight of fabricated steel and concrete components. The results of the examination and comparison indicate strongly the feasibility of the procedure.

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1. INTRODUCTION

1.1 Background and Objective

Steel box girders are often used for medium and long span continuous bridges. [1.1] In the negative moment regions over the interior supports, the bottom flange of the box girder is subjected to compressive forces. Because the bridge bending moment is normally highest at the piers and the compressive strength of steel bottom flange plates are relatively low on account of buckling, thick steel plates with longitudinal stiffeners are necessary. For long span continuous box girders, a haunched profile is also often necessary so as to keep the compressive stresses in the compression flange within safe limit.

The use of longitudinal stiffeners on haunched box girder bottom flanges increase the cost of fabricating the box girder and erecting the bridge. Coupled with the relatively inefficient utilization of strength of steel, with respect to yielding, this condition results in very high cost of steel box girder bridges. Therefore, new arrangements for improving the efficiency of the negative moment area is urgently needed.

One approach to the solution is the utilization of steel-concrete composite compression flange. The technique has been adopted for the construction of bridges in Europe. [1.2, 1.3] In one case, the haunched profile of the box girder is maintained. [1.2] In the other case, the construction procedure for the roadway deck was the main feather.

The primary objective of this feasibility study is to examine the possibility of elimination of haunches through the use of composite compression flanges. Three examples of continuous steel box girder designs are used as the basis for examination. Some essential details of the bridge designs are reproduced as Figs. 1.1 to 1.10.

1.2 Influencing Factors

The factors which may influence the strength of the steel-concrete composite compression flange include the following:

a. Thickness and width of steel plate

b. Thickness of concrete slab

c. Spacing of shear connector

d. Strength and weight of concrete

e. Amount of reinforcement, if needed

f. Spacing of longitudinal stiffeners, if needed

g. Shrinkage and creep of concrete under long term compression.

These factors are examined in this study, and the results are reported in the subsequent chapters. First the possibility of eliminating or reducing the haunches is evaluated in Chapter 2, assuming plain concrete slab for the composite flange. The thickness of steel plate and concrete slab are examined in this chapter. Then the strength of the compression flange is examined in Chapter 3 for an assumed sequence of construction. The compression flange consists of a steel plate which, with or without longitudinal stiffeners, must sustain the weight of wet concrete before developing into a composite flange. The requirements of shear connectors are reviewed and discussed in Chapter 4. The influence of concrete properties and effects of shrinkage and creep are discussed in Chapter 5. Also discussed are the alternate procedures of cast-in-place and precasting of the concrete slab. Chapter 6 presents a comparison of costs for the "original" design of the bridges and the "alternative design" resulting from this study. The comparison is made on the basis of cost per unit weight of fabricated structure. Finally, the findings of this study are summarized in Chapter 7, concluding the feasibility of using composite compression flanges to eliminate haunches in steel box girder bridges.

2. REDUCTION OF HAUNCHES

2.1 Review of "Benchmark" Designs

The three continuous box girders designated for this feasibility study are the following:

a. West Seattle Bridge

b. Columbia River Bridge and

c. Tennessee Tombigbee Waterway Bridge

The general plan, elevation, cross section, and some details are shown in Figs. 1.1 to 1.10. Some geometrical dimensions are listed in Table 2.1. Each of these bridges has its own specific features such as rectangular or trapezoidal boxes, double or single boxes, etc. For the objective of this study, the most important dimensions are the depth of the boxes (D_c and D_p) and the thickness of the bottom flange steel (t_B)

Because the West Seattle bridge has the longest span, highest depth at pier, highest haunch ratio of depth at pier to depth at center of span (D_p/D_c), and the widest bottom flange, it is chosen for a more intensive examination. The results from altering the original or "benchmark" design can then be used as guidance for the other two bridges. However, before attempting examination of adding concrete slab on the bottom flange plate, results of a parametric study [2.1] on the effects of haunched box girder dimensions are briefly reviewed here so as to gain insight of stresses in haunched box girders.

If all other component dimensions of a box girder remain unchanged, but the depth of the haunch at a pier is decreased, the stresses in the bottom compression flange over the piers increase, so do the stresses in the bottom tension flange at the center of a span. This trend is clearly depicted in Figs. 2.1 to 2.3. Each solid curve line in these figures represents the change of compressive stress in the bottom flange over the pier if the box girder depth at the pier is changed alone. The dotted lines are for tensile stresses in the bottom tension flange at midspan. All curves indicate increasing of stresses when the haunch ratio is reduced. A haunch ratio of unity represents a constant depth box girder. Obviously decreasing the girder depth at piers reduces the moment of inertia and thus increases the bending stresses.

Figure 2.1 shows the effects of changing bottom flange thickness and the haunch ratio. For simplicity, uniform bottom flange thickness is used in the comparison. If the bottom flange plate thickness alone is changed, the stresses in the flange plate increase and decrease with the plate thickness. A decrease of flange thickness must be accomplished by an increase in haunch ratio (depth at pier) in order to maintain the same level of stress in the flange. For example, a change from a 2 in. plate to a 1 in. plate requires an increase of haunch ratio from 1.6 to about 2.5 to keep the stress at about 0.58 F_y. If the thickest available plate (say 3 in.) is already adopted and the stress level is to be kept very low (say 0.3 F_y), then a very high haunch ratio must be employed. This condition requires that the depth of the box girder at the pier be much higher than that at the midspan, about 3 times higher in this example. From the solid curves of Fig. 2.1 it can be seen that for increasing values of haunch ratio the slopes of the curves are decreasing. This implies that a large difference between box girder depth at the pier and at midspan is not an efficient way of reducing stresses in the bottom flange compression plate. A lower haunch ratio is more proficient.

To achieve a low haunch ratio, one approach is to adopt higher midspan depth. Figure 2.2 shows the effects of varying the midspan depth and haunch ratio. If the values of all parameters including the haunch ratio of (D_p/D_c) are kept constant and the midspan depth alone is increased, the bottom flange stresses are reduced, as it is shown in the figure. However, if the depth of box girder at midspan (D_c) is increased while the depth (D_p) at the pier is maintained, the compressive bottom flange stresses at the pier remain practically unchanged. This condition is depicted in Fig. 2.4, plotting stresses versus midspan depth D_c for a constant D_p. Increase of midspan depth reduces the bottom flange tensile stress at the midspan.

From the above review of results from a parametric study, the following conclusions can be drawn:

a. To reduce the compressive stresses in the bottom flange over the pier, thicker bottom flange plates and moderate haunch ratios would be more efficient.

b. To reduce the tensile stresses in the bottom flange at midspan, thicker bottom flange plate and higher depth of box girder at midspan would be more effective.

Figure 2.3 indicates that the web thickness has very limited effect on the flange stresses. Therefore, to achieve an efficient design of continuous span steel box girder, appropriate selection of girder depth at pier and at midspan (D_p and D_c) and the thickness of bottom flange (t_B) is essential.

2.2 Haunched Box Girders with Compressive Composite Bottom Flange

When concrete slab is added to the bottom flange steel plate over the piers and the two materials work compositely, the effects on the stresses in the steel plates are two-fold. First, the composite compression flange has an equivalent plate thickness higher than that of the steel plate alone. This is equivalent to using a thicker flange plate, an efficient procedure as it has been shown in the last section of the report. Second, the composite flange should eliminate possible buckling of the steel flange plate and increase its usefulness to the yield stress of the steel. This combination may reduce or even eliminate the need of a haunched profile for the box girder bridge.

The influence of concrete slab thickness and length on the stresses in haunched box girders is examined here by changing the dimensions of the "benchmark" design of the West Seattle Bridge. The results are shown in Figs. 2.5 to 2.7. For these figures, a concrete strength of 4,000 psi (n = 8) is assumed.

Figure 2.5 shows the effects of concrete slab thickness and girder haunch ratio on the bottom flange stresses. For this comparison, it is arbitrarily assumed a thickness of 1 inch for the steel bottom flange and that the concrete slab tapers from a maximum thickness at the pier to zero at about 5/16 of the center span and 5/8 of the side span. When composite action between the concrete slab and the bottom flange steel is assured, the box girder has an equivalent bottom flange thickness higher at the pier and lower at the center of span. The resulting stress versus haunch ratio relationship for the extreme fiber of the composite bottom flange is similar to that for the steel box without concrete slab (Fig. 2.1), but the composite stress over the pier is much lower. This is expected since the equivalent bottom flange thickness is 1 + (12/8) = 2.5 in. for the case of 1 ft. thick concrete slab and 4 in. and 5.5 in. for the 2 ft. and 3 ft. concrete slab, respectively.

Obviously, a concrete slab of 2 ft. or 3 ft. is very thick and the steel plate may not be able to support the weight of the concrete. The capacity of the steel flange plate to carry transverse loads in addition to the in-plane forces from the box girder will be discussed in Chapter 3. It is important to conclude from the results in Figs. 2.1 and 2.5 that the adoption of a non-uniform thickness bottom flange with thickness tapering towards the center of the span will reduce the need of using a high haunch ratio. The utilization of concrete and composite flange can provide equivalent steel plate thickness higher than those commercially available steel plates. These conditions confirm favorably the concept of composite bottom flanges for long span continuous steel box girders.

The effects of concrete slab length and haunch ratio are summarized in Fig. 2.6 for a maximum slab thickness of 2 ft. over the piers. Reducing the concrete slab length decreases the compressive stress in the bottom flange over the pier and increases the tensile stress in the bottom flange at mid-span. The amount of change, however, is quite small. The thickness of bottom flange at a cross-section between the pier and the center of span affects the stresses of the cross-section, but has only minor influence on the stresses at the pier and the center of span.

Because placement of concrete slab over the bottom flange in the negative moment region increases the tensile stresses in the bottom flange of the positive moment region, the box girder depth at midspan may need to be increased. The effects of increasing midspan depth (D_c) are shown in Fig. 2.7. The reduction of stresses at midspan with the increase of depth D_c is similar to that of Fig. 2.2 for a steel box girder without concrete slab. The reduction of stresses at the pier, however, is very small when there is a concrete slab over the pier acting compositely with the steel plate.

It can be seen in Fig. 2.7 that from the geometrical configuration and dimensions studied a constant depth box girder can be selected for which both the compressive stress and tensile stress in the bottom flange are within those of the original "benchmark" design.

2.3 Constant Depth Box Girders, Alternative Designs

Among the three example steel box girders for examination in this study, the West Seattle Bridge and the Columbia River Bridge have haunched profiles whereas the Tennessee Tombigbee Waterway Bridge is of constant depth. Therefore, based on the results of evaluation in Section 2.3, the advantage to be gained by the addition of a concrete slab to the bottom compressive flange would be expected to be more for the West Seattle Bridge and the Columbia River Bridge.

For the determination of an alternative design of each bridge, geometrical dimensions are arbitrarily assigned and component sizes are similarly chosen. The analysis of the bridge is then made using the load-factor design approach. The conditions and assumptions associated with the analysis are the following:

a. Yield strength of steel: $F_y = 50$ ksi

b. Concrete Strength: $f_c' = 4000$ psi

$n = 8$

c. The Top flange of the original design is adequate for the alternative designs.

d. Buckling of the bottom compression flange in the negative moment region is prevented by the addition of the concrete slab.

e. Steel reinforcing bars, if used in the bottom flange concrete slab, have little effect on the overall behavior of the box girder.

f. The concrete slab is in complete composite action with the steel bottom flange.

g. Flexural stresses dominate; torsional stresses due to live loads are minor.

2.3.A West Seattle Bridge

The three-span West Seattle Bridge has a haunch ratio of $(D_p/D_c) = 2.16$ and a bottom steel flange thickness of 2 in. over the piers. The bottom flange stresses are computed to be 32.7 ksi and 47.9 ksi, respectively at the pier and in the middle of the center span of the "benchmark" design. A few trials of constant depth boxes are made by increasing the midspan depth and the bottom flange thickness without the addition of concrete slab. By using the results of these few trials as guides, concrete slab is then added. The results of all these trials are listed in Table 2.2.

Examination of the computed bottom flange stresses reveals that the alternative designs, without use of the bottom flange concrete slab (Trials 1 to 6) all have bottom flange compressive stress higher than that of the original design. By appropriate arrangement of stiffeners for the steel compression flanges, their strength could be made sufficiently higher than the computed stresses. Trials 2, 3 and 6 could then be considered as acceptable designs with constant box girder depth.

Trial 7 corresponds to Trial 1 but with a bottom flange concrete slab. Addition of the slab increases dead weight and the stresses in the bottom flange. With the concrete slab, the strength of the composite flange at the pier should be higher. The tensile stress, however, is higher than the steel's yield strength. The trial design is not considered acceptable.

Trial 8 incorporates the same dimensions of components as for Trial 7, but has a higher depth of the box girder. Stresses in the bottom flange are lower than those of the original design. The estimated midspan deflection is within the guideline of 1/800 of the span. The trial design is acceptable but the component dimensions may be reduced.

Trials 9 to 15 adopt different combinations of box girder depth, concrete slab length, and steel flange thickness. All have the original steel plate thickness of 2 in. over the pier and a 1.5 ft. (18 in.) depth of concrete directly above. All except Trial 12 have bottom flange compressive stress lower than that of the original design and bottom flange tensile stresses at midspan within the yield strength of 50 ksi. Therefore, all these combinations, Trials 9 to 11 and 13 to 15, are possible alternative designs.

For these possible alternative designs, a reduction of the concrete slab thickness may be taken. A reduction of concrete thickness alone is accompanied by an increase of compressive stress in the composite bottom flange and a slight increase of tension stress in the steel bottom flange at center span, see Fig. 2.5. However, without knowing the strength of the composite compression flange, reduction of the concrete slab thickness (for example, to 15 in.) is not fully justified. What is important is that not only the elimination of the haunches is possible, but there are also different combinations of component dimensions for fine adjustment of stresses in the constant depth box girder.

The question to be answered, therefore, is whether the elimination of the haunches is economical.

For a comparison of approximate cost in Chapter 6, the possible alternative designs are examined for least weight. Trial 14, with a depth of 15 ft. and the shortest length of concrete slab in the bottom flange, would weigh the least. However, in consideration of uncertainties such as the strength of composite compression flange and the influence of box girder depth on pier top elevation, Trial 11 is arbitrarily chosen for the subsequent discussions in this study.

The dimensions of the chosen alternative design are shown in Fig. 2.8.

2.3.B Columbia River Bridge

The original design of this five span bridge adopts a haunched profile with a box girder depth of 16 ft. at the first piers and 21 ft.-4 in. at the interior piers. The depth at center of span is 10 ft.-4 in. so the higher haunch ratio is 21.3/10.3 = 2.06, a fairly high value comparable to that of the West Seattle Bridge. The center span has a length of 450 ft. The cross-section of the bridge is a twin-cell single box of trapezoidal shape. The total width of the steel bottom flange is 357 ft., 178.5 fr. for each cell. Other dimensions of the box girder are listed in Table 2.1.

The bottom flange stresses in the center span are 38.8 ksi over the piers and 41.5 ksi at midspan by the computation of this study. Trial designs are made with constant box girder depth and arbitrarily selected concrete slab thickness and length. The composite compressive bottom flange should have strength higher than that of the original 1.5 in. steel plate over the piers. Conservatively, the original stress of 38.8 ksi is used as a reference. The trial dimensions and resulting bottom flange stresses are summarized in Table 2.3.

or the twelve trials, two box girder depths are used. The depth of 10 ft.-4 in. is the original value at midspan whereas the 12 ft. depth is a small increase at midspan, but a large reduction at the interior piers. The concrete slab depth at the piers is assumed to be 1.5 ft. or 2 ft. as guided by the results of the West Seattle Bridge. The bottom flange steel plate thickness is either kept at 1.5 in. or increased to 2 in. All twelve trials appear to be acceptable, with bottom flange compressive stress within the arbitrary reference value of 38.8 ksi and bottom flange tensile stress less than the yield stress.

Trial 9 is arbitrarily chosen as the alternative design for cost comparison later. The dimensions of this trial design is shown in Fig. 2.9. Some dimensions are also listed in Table 2.4 with those of the alternate design of the West Seattle Bridge.

2.3.C Tennessee Tombigbee Waterway Bridge

Tennessee Tombigbee Waterway Bridge is a three span twin box design with trapezoidal box cross-section and constant depth over the entire length of the bridge. The center span is 400 ft. long and the depth is 12.5 ft. The bottom flange is 2 in. thick over the piers and 1.375 in. at midspan. This data is listed in Table 2.1.

As it has been discussed earlier, the adoption of a deeper girder depth and a thicker flange plate at the pier is to keep flange stresses within the strength of these flanges. The use of a thicker flange plate alone is sufficient to achieve this goal for the Tennessee Tombigbee Waterway Bridge. Therefore, adding of a concrete slab would not be as advantageous as for the other two bridges.

Table 2.5 lists the geometrical dimensions of the box girder and concrete slab for nine trial designs. Trials 1 and 2 add concrete slab to the bottom flange of the original design, thus reducing the bottom flange stresses. This is not necessary. Trials 3 to 9 adopt a thinner uniform thickness bottom flange steel plate and add concrete slab, resulting in the condition that the midspan bottom flange tensile stresses are higher than the yield strength. Other trials can be made but are not expected to improve on the original design.

In comparing the results of analysis of the Tennessee Tombigbee Waterway Bridge with those of the other two bridges, it confirms that the addition of concrete slab is efficient for reduction of haunches. If the strength of the composite steel-concrete compression flange can be utilized fully, alternative designs can be even more efficient. The strength of composite flanges is discussed next.

3. STRENGTH OF COMPRESSION FLANGES

3.1 Introduction

The incorporation of a concrete slab into the compression flange over the negative moment region of continuous steel box girders is for increasing the strength of the compression flange. Because the ratio of live-load stress to dead-load stress in long span box girders is relatively low, the major function of the compressive bottom flange in a completed bridge box girder is to sustain dead-load stresses. However, depending upon the method and sequence of construction and erection, the dead weight and construction load may be significant while the compressive bottom flange has only the steel plate portion. Therefore, it is essential to evaluate the strength of the compression flange at all stages of construction.

The stages of construction are assumed in this study to be a successive addition of box segments from a pier, forming a balanced double cantilever. A bottom flange concrete slab is added to a box segment following the attachment of the next box. This sequence is illustrated in Fig. 3.1.

Consequently, the bottom steel compression flange alone carries stresses of two steel box segments, then carries additional stresses due to the wet concrete on the steel flange, and thereafter combines with the concrete slab to form a composite compression flange.

The conditions of the bottom flange which need to be examined are the following: (a) the strength of the steel compression flanges, (b) the strength of steel plates under wet concrete, that is, under combined in-plane loading and lateral load, and (c) the strength of steel-concrete compressive flanges. There exists in literature only very limited information as the basis for examination of these conditions. This chapter provides a very brief discussion.

3.2 The Strength of Steel Compression Flanges Alone

During construction of the bridge, before the concrete slab is placed on a steel compression flange plate, it acts alone to resist in-plane stresses. The steel plate is thus "conventional" for which there are existing rules and guidelines of design.[3.1, 3.2] The basic concept is that the steel compression flange should provide sufficient margin of safety against buckling or yielding.

The buckling and yield strength of unstiffened compression plates are described by the following equations:

a. AASHTO 10.51.5 [3.1]

For $\frac{b}{t} \leq \frac{6140}{\sqrt{F_y}}$

$$F_u = F_y \tag{3.1A}$$

For $\frac{6140}{\sqrt{F_y}} \leq \frac{b}{t} \quad \frac{13300}{\sqrt{F_y}}$

$$F_u = 0.592\ F_y\ (1 + 0.687\ .\ \sin \frac{c\pi}{2} \tag{3.1B}$$

$$\text{with} \qquad C = \frac{13300 - \frac{b}{t}\sqrt{F_y}}{7160}$$

For $\frac{b}{t} \leq \frac{13300}{\sqrt{F_y}}$

$$F_u = \frac{105}{(b/t)^2} \times 10^6 \tag{3.1C}$$

in which b = width of bottom flange plate between the webs (in.)

t = thickness of the steel plate (in.)

F_y = yield strength of the steel (psi)

F_u = (buckling) strength of the plate (psi)

b. Proposed Specification 1.7.205 [3.2]

For $\lambda_{pl} \leq 0.65$

$$F_u = F_y \tag{3.2A}$$

For $0.65 < \lambda_{pl} \leq 1.5$

$$F_u = F_y \; [0.50 + 0.43 \; (\lambda_{pl} - 1.73)^2] \tag{3.2B}$$

For $\lambda_{pl} \leq 1.5$

$$F_u = F_y \; (0.82 - 0.20 \; \lambda_{pl}) \tag{3.2C}$$

in which

$$\lambda_{pl} = \sqrt{\frac{F_y}{F_u}} = \frac{b}{t} \sqrt{\frac{10.92 \; F_y}{\pi^2 \; K \; E}} = \frac{b/t}{1.9} \sqrt{\frac{F_y}{E}}$$

F_{cr} = elastic buckling stress of plate panel

K = plate buckling coefficient (takes a 4 for simply supported panel)

Figure 3.2 compares the above provisions using a yield point of 50 ksi. It is obvious that for bottom flange plates with high width to thickness ratios, the buckling stresses are low. However, as long as the stresses in the plate are lower than the buckling stress at all times, there is no need to add longitudinal stiffener.

The computed stresses in the steel bottom flange of the alternatively designed West Seattle Bridge are listed in Table 3.1 as examples. By following the erection sequence of Fig. 3.1, there appears to be no need of bottom flange stiffener since the maximum in-plane stresses in the compression are always lower than the buckling stress. Similar conditions exist for the Columbia River Bridge.

If erection sequence other than that of Fig. 3.1 is adopted, the bottom flange stresses could be higher and longitudinal stiffeners would then be necessary. Furthermore, the more critical condition is when wet concrete is placed on the steel plate and before the two materials act together compositely. This condition is examined next.

3.3 Strength of Compression Flange Steel Plates Under Web Concrete

When wet concrete is poured onto the compression flange steel plate during erection, lateral loads are applied to the steel plate causing additional stresses in these plates. The approximate loading condition of the steel plate is depicted in Fig. 3.3 in which p is the in-plane loading and q the lateral load from the wet concrete and the steel plate itself. The boundary conditions along the plate length, a, and width, b, depend on the conditions of the adjacent components of the box girder. Conservatively, all edges can be considered as simply supported.

Approximate solutions can be obtained by using the curves of W. Guffel. [3.3] These curves are shown in Fig. 3.4. Two arrangements of the bottom flange in the alternately designed West Seattle Bridge are examined as examples, one without longitudinal stiffeners and one with such stiffeners for the steel plate.

3.3.A Without Longitudinal Stiffeners

Thickness of steel plate = 2 in.

Thickness of Concrete = 18 in.

Plate panel length, a = 177 in.

b/t_s = 120

a/b = 0.74

F_y = 50 ksi

Plate buckling stress

without lateral load, K = 40

$$F_{cr} = \frac{K \pi^2 EZ}{12 (1 - \nu^2)(b/t)^2} = 7.3 \text{ ksi}$$

$$\text{Lateral load } q = \frac{t_c}{12} \times \frac{150}{144} + \frac{t_s}{12} \times \frac{490}{144}$$

$$= 2.13 \text{ psi} = 2.13 \times 10^{-3} \text{ ksi}$$

Maximum plate stress due to web concrete:

$$\sigma_{x1} = 3.2 \text{ ksi (from Table 3.2)} = \sigma_{x1} = \frac{3.2}{7.3} = 0.44\ F_u$$

From Fig. 3.4

$\alpha = 0.06$

$\beta_x = 0.60$

$\beta_y = 0.40$

Therefore, Max. lateral deflection, $W_{max} = \alpha \frac{q\ b^4}{Et^3} = 1.8$ in.

$$\sigma_{xmax.} = \sigma_{x1} + \sigma_{x2}$$

$$= 3.2 + \beta_x \frac{q\ b^2}{t^2} = 21.6 \text{ ksi} < F_y$$

$$\sigma_{ymax.} = \beta_y \frac{q\ b^2}{t^2} = 12.3 \text{ ksi} < F_y$$

The above computation indicates that the bottom flange plate of the alternatively designed West Seattle Bridge is adequate with respect to placement of wet concrete. However, the lateral (downward) displacement of 1.8 in. at the center of the flange plate is about equal to the thickness of the plate, and could be considered not acceptable. Consequently, longitudinal stiffeners may need to be used.

3.3.B With Longitudinal Stiffeners

The addition of longitudinal stiffeners to the steel compression flange results in stiffened compression plates, of which there is no simple, ready solution for combined in-plane and lateral loads. [3.4] One logical approximation is to consider the steel plate between longitudinal stiffeners as supported by elastic beams and to consider the stiffeners with part of the plate (the effective width) as beam columns.

a. The elastic buckling coefficient, K, of plates with boundary elements (Fig. 3.3, with A_s and I_s along length, a) can be evaluated by the formula [3.5]

$$K_{min.} = 4 \left(1 - \frac{1.5}{2\gamma - 8\delta}\right) \qquad (3.3)$$

in which $\delta = \frac{A_s}{bt}$

$$\gamma = \frac{EI_s}{D_b} = \frac{12\ (1 - \nu^2)\ I_s}{b\ t^3}$$

A_s = area of longitudinal stiffener

I_s = corresponding moment of inertia

For the alternately designed West Seattle Bridge, ST 10 x 45 is used as longitudinal stiffeners. By assuming that only one such stiffener is placed at mid-width of the plate, the following computation can be made:

$b = 120$ in. $\qquad a/b = 177/120 = 1.475$

$t = 2$ in.

$A_s = 14.1\ \text{in.}^2$

$I_s = 143\ \text{in.}^3$

and

$$\delta = \frac{A_s}{bt} = 0.059$$

$$\gamma = \frac{12\ (1 - \nu^2)}{bt^3} = 1.63$$

$$K_{min.} = 4 \left(1 - \frac{1.5}{2\gamma - 8\delta}\right) = 1.85$$

$$F_{cr} = \frac{K\ \pi^2\ E}{12\ (1 - \nu^2)(b/t)^2} = 13.5\ \text{ksi}$$

$$\sigma_{x1} = 3.2\ \text{ksi (from Table 3.2)}$$

$$= 0.24\ F_{cr}$$

From Fig. 3.4

$\alpha = 0.11$

$\beta_x = 0.40$

$\beta_y = 0.60$

with $q = 2.13 \times 10^{-3}$ ksi, as before

Maximum deflection, $W_{max.} = \alpha \frac{q b^4}{E t^3} = 0.21$ in.

$$\sigma_{xmax.} = \sigma_{x1} + \sigma_{x2}$$

$$= 3.2 + \beta_x \frac{q b^2}{t^2} = 6.3 \text{ ksi} < F_y$$

$$\sigma_{ymax.} = \beta_y \frac{q b^2}{t^2} = 4.6 \text{ ksi} < F_y$$

These results indicate that the deflection of the steel plate under concrete would be about one-tenth of the plate thickness, and the maximum stresses in the steel plate are quite low. If the longitudinal stiffeners are adequate, the sequence of construction is acceptable. In fact, it may even be possible to add concrete in two consecutive segments simultaneously, providing flexibility of construction scheme.

b. The elastic strength of the longitudinal stiffener acting as a "beam-column" can be evaluated using the following interaction formula [3.6]

$$\frac{M_o}{M_y} \left[\frac{1 + 0.0281 \, (P/P_E)}{1 - (P/P_E)}\right] + \left(\frac{P}{P_E}\right) \left(\frac{P_E}{P_y}\right) = 1 \qquad (3.4)$$

with $M_o = \frac{1}{8} q L^2$

q = lateral load

M_y = yield moment = $S F_y$

P = maximum axial force corresponding to q

P_E = Euler's buckling load =

$$\frac{\pi^2 E}{L^2}$$

$P_y = A\ F_y$

Equation 3.4 can be solved for $\frac{P}{PE}$, resulting in

$$\frac{P}{P_E} = \frac{1}{2} (\alpha_1 - \sqrt{\alpha_1^2\ 4\ \alpha_2^2}) \tag{3.5}$$

in which

$$\alpha_1 = 0.0281 \frac{M_o}{M_y} \frac{P_y}{P_E} + 1 + \frac{P_y}{P_E}$$

$$\alpha_2 = (1 - \frac{M_o}{M_y}) \frac{P_y}{P_E}$$

The strength of the stiffener in terms of maximum average compression stress is

$$F_u = \frac{P_E}{2A} (\alpha_1 - \sqrt{\alpha_1^2 - 4\alpha_2^2}) \tag{3.6}$$

For the alternately design West Seattle Bridge compression flange with an ST 10 x 48 stiffener, assuming that the effective plate width to be b_e = 60 in.

I = 995 in.^{4}

S = 96.5 in.^{3}

M_o = 1/8 x (2.13 x 10^{-3}) x 177^2 = 500 k-in.

M_y = SF_y = 96.5 x 50 = 4825 k-in.

$\frac{M_o}{M_y}$ = 0.104

A = $A_s + t_s\ b_e$ = 14.1 + 2 x 60 = 134.1 in.^{2}

P_E = 9.1 x 10^3 kips

$$P_y = 134.1 \times 50 = 6.7 \times 10^3 \text{ kips}$$

$$\frac{P_y}{P_E} = 0.74$$

$$\alpha_1 = 0.0281 \times 0.104 \times 0.74 + 1 + 0.74 = 1.74$$

$$\alpha_2 = (1 - 0.104)\ 0.74 = 0.663$$

$$F_u = \frac{9.1 \times 10^3}{2 \times 134.1} \left(1.74 - \sqrt{1.74^2 - 4 \times 0.663^2}\right)$$

The maximum stress in the flange under the weight of concrete is $\alpha_{xy} = 3.2$ ksi (Table 3.2), which is well within the strength of the stiffener beam-column. Results of computation show that if a concrete slab is added to two consecutive segments simultaneously, the maximum stress will be $\alpha_{x1} = 5.7$ ksi, still well within the strength of the beam-column.

With the compression flange steel plate capable of carrying the in-plane stresses and the wet concrete, and the plate deflection within nominal range, the sequence of erection as depicted in Fig. 3.1 is judged acceptable.

3.4 Composite Flange Under Dead Weight and Live Load

The composite action between the steel plate and the concrete slab relies on positive bonding or anchorage between the two materials. Besides the requirement of "shear connectors" for the development of complete interaction, there are other conditions which influence the behavior of the composite compression flange. Among the questions which need to be answered are the following.

a. Are the ends of the concrete slab in bearing against transverse stiffeners or diaphragm plates? Are the forces in the concrete slab transmitted from the steel plates?

b. What are the effects of concrete shrinkage and creep on the behavior of the composite flange?

c. Are reinforcing bars necessary for the development of strength of the composite compression flange?

The requirements of shear connectors will be examined in Chapter 4 and the effects of shrinkage and creep in Chapter 5. The question of reinforcing bars are examined below.

As long as the concrete slab and the steel compression plate are subjected to box girder flexural strain at the ends of the composite flange, each material is subjected to axial compressive force. There is no need of reinforcing bars in the concrete if its stresses are within limits. Since the geometrical configuration of the box girder and the thickness of the steel plate and concrete slab have been proportioned to ensure that stresses in all parts are within their appropriate strength, reinforcing bars are not needed in the concrete slab.

However, the composite compression flange has initial downward deflections due to the weight of the concrete. Under additional dead weight of the bridge and live load, the axial forces in the compresion flange and the initial deflection produce a bending moment in the composite plate. This changes the distribution of stresses in the vicinity and the strength of the composite flange needs to be examined. Unfortunately, there is no available procedure or guideline for such a strength evaluation. Very conservatively, the composite plate may be considered as a composite beam for a gross examination.

During erection of the box segments, the stresses in the steel bottom plate increases as each segment of box is added. (Table 3.3, Fig. 3.5) The maximum compressive stress of 29.4 ksi in the steel bottom plate occurs over the pier when the bridge is complete and under live load (Table 2.2). The increase of stress between placement of concrete in the first segment (Table 3.2) and the service condition (Table 3.3) is 29.4 - 3.1 = 26.3 ksi. This generates a resultant force and moment

$$R = 19{,}100 \text{ kips}$$
$$M = 24{,}830 \text{ kip-in.}$$

for the 1.8 inch plate deflection at center of the bottom flange plate with no longitudinal stiffener. This combination of force and bending moment is within the permissible region of the force-moment interaction diagram of the bottom flange composite beam as it is depicted in Fig. 3.

Therefore, the alternately designed constant depth West Seattle Bridge as presented in Chapter 2 has sufficient strength of the composite bottom flange without longitudinal stiffener. If one longitudinal stiffener is used, as discussed in Section 3.3B, the initial plate deflection is only 0.21 in. and the corresponding "beam-column" effect will be negligible.

In conclusion, it can be stated that the dimensions and geometry of the bottom flange composite plate can be properly arranged to assure its strength in all stages of the box girder bridge construction and service.

4. SHEAR CONNECTOR REQUIREMENTS

4.1 The Need of Positive Anchorage

The composite action between a concrete slab and the steel compression plate below relies on positive connection if there is transfer of shear forces between the two materials. In the case of a wide flange shape and a reinforced concrete slab combining to form a composite beam, there is such a transfer of shear force that shear connectors are needed. [4.1,4.2] Provisions for shear connectors are included in AASHTO Specifications [4.3] and corresponding specifications in other countries. [4.4,4,5]

In the case of reinforced or prestressed concrete decks placed above compressive top flanges of steel box girders, the condition of shear force transmittal between the steel plate and the concrete deck is similar to that of composite beams. Shear connectors are required. There is no existing design specification in this country defining the requirements of connector spacing or total number between the steel and concrete plates. The British standards, on the other hand, considers lag effects in the steel plate and establishes an expression for estimating the shear force to be transmitted by a connector. [4.4,4.6]

The concrete slab on the compressive steel bottom flange of a steel box girder may or may not require transmittal of shear forces between the slab and the steel plate. The most important factor is whether the concrete slab inside the box is under direct compression simultaneously with the steel plate. If so, the two materials share the function of resisting compressive forces, and shear connectors may be omitted. If not, then the situation is similar to that of the composite compressive top flange and shear connectors are required.

4.2 Concrete Slab in Bearing

Assuming that the concrete slab in the bottom flange is in direct bearing with transverse diaphragms or transverse stiffeners, the concrete slab is, while supported laterally by the steel plate, subjected to longitudinal strains proportioned to the distance to the neutral axis of the box girder cross-section. As long as the stresses in the concrete slab are within the concrete strength limit and the steel plate provides adequate lateral support, the concrete slab does not need to be connected to the steel plate below.

The steel compression plate is subjected to axial forces and lateral load from the concrete slab above. The situation is similar to the case of wet concrete on steel plate as discussed in Chapter 3 but with a higher magnitude of axial stresses in the steel plate. Consequently, the buckling strength of the steel plate becomes the governing condition. Anchorage of the steel plate, not shear connectors for shear force transmittal, may be necessary.

Buckling of steel compression plate under lateral load is discussed in Section 3.3, with the loading condition shown in Fig. 3.3. If stud connectors are employed as anchorage between the steel plate and the concrete slab, the loading condition for the portion of steel plate between the anchor points is slightly different. Two models are suggested in Fig. 4.1, one assumes no plate rotation on the edges of the plate "panel" and the other assumes no restraints at all.

The solution for these models are not available. Assumptions are made here so as to arrive at some conservative estimates.

a. At buckling of the steel plate between the anchors, the steel plate panel separates from the concrete slab and the lateral load of the concrete slab does not act on the steel plate.

b. The relative lateral deflection between the center of the steel plate panel and the anchorage is small. Small deflection theory can be utilized.

c. Axial forces (q_x) in the steel plate is uniform along the edge (Fig. 4.1).

d. Stud connectors remain in a plane.

The solution for the buckling coefficient, K, in the buckling formula,

$$F_{cr} = \frac{K \pi^2 E}{12(1 - \nu^2)(b/t)^2} \tag{4.1}$$

is plotted in Fig. 4.2 The lowest value of the buckling coefficient is

$$K = 2.5$$

corresponding to a ratio of (a/b) = 1.0 and rotation force boundaries. By substituting

$$F_{cr} = F_y = 50 \text{ ksi}$$

$$E = 29{,}000 \text{ ksi}$$

$$\nu = 0.3$$

and the value of K into Eq. 4.1, it is obtained

$$b/t \leq 36.2 \tag{4.2}$$

For the alternative design of the West Seattle Bridge, the thickness of bottom flange steel plate is t = 2 in. Therefore, the minimum spacing of anchors is about 70 in.

When the steel compression plate is anchored to the concrete slab according to Eq. 4.2, the steel plate and the concrete slab can work together and share the duty of carrying compressive forces. The over-buckling of the steel plate in a segment of box girder is prevented because of the anchors. The overall strength of the most severely loaded composite compression flange over the pier has been examined in Section 3.4 and is adequate.

Therefore, when the concrete slab is in direct bearing against the transverse diaphragms and participate in direct compression, the anchorage between the concrete slab and the steel plate is adequately defined by Eq. 4.2.

4.3 Shear Connectors Spacing for Composite Compression Flange

If the concrete slab on the bottom flange steel plate is not in direct bearing against the transverse diaphragms, force shear transmittal between the steel plate and the concrete slab requires shear connectors. The AASHTO provisions for composite beams [4.3] and the British Standards or compressive upper flange decks [4.4] can be temporarily used.

The AASHTO Specifications, Section 10.38.5, requires that the shear connectors be spaced according to fatigue and ultimate strength of the type of shear connectors, with a maximum spacing (pitch) of 24 in.

British Standards Institute BS5400 specifies in Part 5, Section 5.3.3.1, that the longitudinal shear force (Q_x) on a shear connection at a distance x from the box girder web be determined from [4.4]

$$Q_x = \frac{q}{n} [K \quad (1 - \frac{x}{b_w})^2 + 0.15] \tag{4.3}$$

where

q = design longitudinal shear per unit length of box girder

n = total number of shear connectors in the unit length

K = Coefficient, a function of number of shear connectors placed within a short distance from the web

x = distance from the web (in millimeter)

b_w = b/2

The maximum spacing is specified as 600 mm (24 in.) which is the same as for composite beams.

Because the maximum permissible spacing of 24 in. is more severe than the anchorage requirements as described by Eq. 4.2, shear connectors for the development of interaction between the concrete slab and the steel plate of the bottom flange are also sufficient for the anchorage of the steel plate.

4.4 Pull-Out of Anchors

The direct pulling of the anchors between the steel plate and the concrete slab must be such that the strength of a full shear cone is not exceeded. The full cone for a stud connector is depicted in Fig. 4.3. The strength is defined by [4.6]

$$P_{uc} = 4\ A_{fc}\ \sqrt{f'_c} \tag{4.4}$$

with

$A_{fc} = \sqrt{2}\ \pi\ L_e\ (L_e + D_s)$, the area of full conical surface

L_e = emedment length of anchor

D_s = diameter of stud connector head

f'_c = concrete strength

At the maximum spacing of 24 in. (2 ft.), the 2 in. thick plate of the West Seattle Bridge exerts a maximum downward force of

$$2 \times 2 \times \frac{2}{12} \times 490 = 327 \text{ lbs.}$$

Assuming a 4 in. long stud with a 1 in. head in 4,000 psi concrete, the cone strength is

$$P_{uc} = 4\ [\ 2\ \pi \times 4\ (4 + 1)] \qquad \sqrt{4000}$$
$$= 22.4 \text{ ksi}$$

which is much higher than pulling force. Even if the maximum spacing is increased after a thorough investigation of the shear connector requirements, the cone strength is more than sufficient to anchor the steel plate.

5. EFFECTS OF CONCRETE SLAB PROPERTIES

5.1 Effects of Shrinkage and Creep

The long term shortening of the concrete slab due to shrinkage may cause separation of the concrete slab from the transverse diaphragms, thus changing the nature of loading in the composite bottom flange. The long term shortening of the concrete slab due to creep is expected to transfer stresses from the concrete slab to the steel flange plate.

Several factors influence these long term effects. The concrete slab is cast on the steel flange plate and between the webs, longitudinal stiffeners and transverse diaphragms, resulting in only an exposed top surface. This condition and the relatively constant and moderate humidity inside the steel box girder inhibit the evaporation of moisture from the concrete slab. The rates of shrinkage and creep are reduced. Also, the stresses in the concrete slab are gradually increased as additional box girder segments are attached. Appropriate scheduling of erection of steel boxes and placing of concrete can lead to not only lower stresses in the compressive bottom flange, but also more favorable effects of shrinkage and creep.

Under nominal conditions, the long term shrinkage and creep coefficient are estimated from the expressions [5.1,5.2]

$$\varepsilon_{sh} = 1080\ e^{-0.12(\frac{V}{S})} \tag{5.1}$$

and

$$C_u = 1.8 + 1.77\ e^{-0.54(\frac{V}{S})} \tag{5.2}$$

where

ε_{sh} = long term shrinkage for 7 days concrete

C_u = creep coefficient

V/S = effective volume to surface ratio

Correction factors are applied to the estimated values to account for humidity and other factors.

For the composite bottom flange of the West Seattle Bridge, the concrete slab is 18 in. thick. V/S = 18 in. Assuming that the relative humidity is 70%, the correction factors for shrinkage and creep are 0.70 and 0.80, respectively. The long term shrinkage is then

$$\varepsilon_{sh} = (1080\ e^{-0.12 \times 18})(0.70)(1.20)$$
$$= 105 \times 10^{-6} \text{ in/in.}$$

The factor 1.2 accounts for shrinkage of the first seven days.

Under the full factored load of the completed bridge, the maximum compressive stress in the composite bottom flange is 29.4 ksi over the piers (Table 2.2) The stress in the steel plate at the time of placing concrete is 3.1 ksi (Table 3.2). Therefore the increase of stress in the steel plate of the composite flange is 29.4 - 3.1 = 26.3 ksi and the corresponding concrete stress is

$$26.3/8 = 3.29 \text{ ksi} < 0.85\ f_c'$$

Under service load conditions, the stress in the steel plate is about 29.4/1.5 = 19.6 ksi and the concrete stress is 19.6/8 = 2.07 ksi. The total axial load in the composite flange is

$$F_f = 19.6 \times 2 + 2.07 \times 18 = 76.5 \text{ kips/in.}$$

Because the stresses in the steel and concrete components are gradually increased according to erection scheme and shrinkage and creep take place, an approximate value of F_f = 40 kips/in. is arbitrarily chosen for the evaluation of strains in the composite flange.

$$A_{composite} = (18/n) + 2 = (18/8) + 2$$
$$= 4.25 \text{ in.}^2\text{/in.}$$

$$\text{Stress in steel plate} = 40/4.25$$
$$= 9.4 \text{ ksi}$$

$$\text{Strain in steel} = 9.4/E_s = 3.25 \times 10^{-6} \text{ in/in.}$$

This estimate average strain in the steel plate is larger than the maximum long term shrinkage strain of 105×10^{-6} in/in. for the concrete slab. In other words, for the completed bridge, there is no shrinkage gap between the concrete slab and the transverse diaphragm.

The long term creep coefficient is estimated from Eq. 5.2.

$$C_u = (0.8)(1.8 + 1.77\, e^{-0.54 \times 18})$$

A correction factor of 0.8 is applied for a 70% relative humidity. The long term modulus of elasticity for the concrete is then

$$E_{ct} = \frac{E_c}{1 + C_u} = \frac{E_s/n}{1 + C_u} \tag{5.3}$$

$$= \frac{29 \times 10^6/8}{1 + 1.44} = 1.49 \times 10^6 \text{ psi}$$

The total concrete strain due to shrinkage and creep is

$$\varepsilon_c' = 105 \times 10^6 + \frac{\sigma_c'}{E_{ct}} \tag{5.4}$$

in which σ_c' is the long term stress corresponding to ε_c'. From compatibility of strain between the components of the composite flange.

$$\varepsilon_c' = \varepsilon_s'$$

$$105 \times 10^6 + \frac{\sigma_c'}{E_{ct}} = \frac{\sigma_s'}{E_s} \tag{5.5}$$

and from equilibrium

$$A_c\, \sigma_c' + A_s\, \sigma_s' = F_f \tag{5.6}$$

Equations 5.5 and 5.6 combine to give

$$\sigma_c' = 0.60 \text{ ksi} = 600 \text{ psi}$$

$$\sigma_s' = 14.6 \text{ ksi}$$

The total strain in concrete is, by Eq. 5.4,

$$\varepsilon_c' = 105 \times 10^6 + \frac{600}{1.49 \times 10^6} = 510 \times 10^{-6} \text{ in/in.}$$

The total long term concrete strain from the completion of the bridge is

$$(510 - 325) \times 10^{-6} = 185 \times 10^{-6} \text{ in/in.}$$

The magnitude of long term concrete strain is not expected to present performance difficulties to the composite flange.

It must be pointed out, again, that the amount of shrinkage and creep strains are very conservatively estimated. The phenomenon of shrink and creep of composite steel-concrete compression plate have not been studied. The calculations given above only serve as a very brief and very rough guideline for this feasibility study. The conclusion is positive that composite steel-concrete compression flange in box girders can be developed.

5.2 Effects of Strength and Weight of Concrete

Obviously, the strength and weight of concrete are expected to have some effect on the stresses in the steel box girder components. Different strength and weight of concrete of the same thickness over the bottom flange of the negative moment region is equivalent to slightly different thicknesses of a chosen concrete. The results of this equivalent change in concrete slab thickness are slight changes of stresses in the composite compression flange, as it has been shown in Chapter 2.

To confirm this conclusion, two different concrete strengths and two different weights of concrete are used for the alternative design of the West Seattle Bridge. The computed bottom flange stresses are listed in Table 5.1.

In this table, the case with 4000 psi concrete is the alternative design of Table 2.2. An increase in concrete strength reduces the compressive stress in the composite flange and changes the tensile stress at the bottom flange of midspan very little. The use of lightweight concrete changes the weight of the concrete slab about 20%, but the total weight of the bridge and the bending moments are affected very little. Because the modulus of elasticity of lightweight concrete is lower, the effects of the modulus ratio (n) is more pronounced. The net results are that lightweight concrete causes higher compressive stresses in the composite flange. There is no advantage of using lightweight concrete for the box girder segments.

5.3 Casting of Concrete Slab

It has been pointed out that segmential casting of concrete slab inside the erected box girder segments is a very important part of the adoption of the compressive composite flange. Casting of concrete in-situ has been shown to be acceptable with regard to steel plate buckling and shrinkage and creep. Another procedure of developing the composite flange is by attachment of precast concrete slab.

Adoption of precast concrete slabs as the concrete component of the composite flange has the following advantages.

a. Reduction of amount of work on the partially erected bridge.
b. Reduction of time between erection of box girder segments.
c. Reduction of amount of shrinkage and creep from those of cast-in-place slabs.
d. Reduction of concrete slab thickness and weight by a small percent because reinforcements are most likely needed for the precast slab for handling or hoisting.

The disadvantages are:

e. The added uncertainty of shear connector requirement and the behavior of the precast slabs.
f. Necessary grouting between the precast slab and the steel components (webs, stiffeners and diaphragms).
g. Requirement of reinforcing bars for the slab.

Whether the advantages outweighs the disadvantages depends on the length and geometry of the bridge as well as on its location. The important point is that the process of placing precast decks on bridges has been successfully tried, and it should be possible to be used for composite compression flanges.

6. COST COMPARISON

The primary purpose of redesigning the three sample bridges is to achieve efficient and economical design through the use of composite concrete slab over the negative moment region. Whereas it has been demonstrated that efficiency can be gained, the economy of the scheme is not easily assessed.

The total cost of a bridge is the sum of costs for various materials and for labor of fabrication, transportation and erection of all parts. The differences between an "original" design of a bridge and its alternative design includes not only the profile and height of the continuous box girder - thus the amount of material and labor for the superstructure - but also the elevation of the pier top. The reduction or elimination of the haunch over the piers necessitates the increase of pier height in order to maintain the appropriate clearaace or navigational channel.

Without exerting extra effort to acquire information for the evaluation of foundation and pier costs as part of the total cost, estimates are made of the total cost of the superstructure and the increase of pier height for the box girder bridges. Furthermore, instead of estimating costs by counting the weight of various materials and the man-hours required for fabrication, transportation and erection, a unit price for each fabricated material is assumed. By employing a wide range of unit prices according to current market conditions [6.1], it is believed that fair comparisons can be made on cost of the "original" and alternative designs.

Tables 6.1 and 6.2 list the total costs of the West Seattle Bridge and the Columbia River Bridge. There is found no structural advantage in using composite compression flanges in the Tennessee - Tombigbee Waterway Bridge so no alternative design is made.

For the twin box, rectangular cross-sectional West Seattle Bridge, Trail 11 (Table 2.2) is chosen as the alternative "new" design, arbitrarily on lowest stress instead of on lowest weight. The total weight of the steel portion is estimated to be 7027 kips as compared to 7270 kips for the original or "old" design. The composite compression flanges require 170 cubic yards of concrete and the added height of the piers need 17 cubic yards. Table 6.1 itemizes the various combinations of unit costs for fabricated steel and concrete portions, and provides the estimated total costs in the last column.

Because of the elimination of haunches, the fabrication of the steel box girder segments is very much simplified. The transportation and erection of uniform depth box segments are also much easier than of the haunched portions. Therefore, the unit cost of the "new" steel superstructure is expected to be lower than that of the old, original superstructure. Case 1, 3, 5 and 7 in Table 6.1 assume this condition. Case 2, 4, 6 and 8 assume the same unit price. The concrete in the steel boxes is assumed to be without reinforcement and that in the pier requires special formwork. Therefore, the prices are different.

For all cases of unit cost combination, the total cost of the new alternative design is lower than the original design with haunched profile.

In the case of the Columbia River Bridge, the original design has a trapezoidal, single box, twin cell cross-section. Constant depth Trial 9 is chosen as the alternative design on the basis of lower weight. The original design, with 5/8 in. bottom flanges at midspans is lighter than the new, alternative design. In addition, concrete is needed in the box and for the increased pier height. However, the change from a haunched profile to that of a uniform depth significantly reduces the cost of steel fabrication. The unit price for steel is expected to be much lower for the alternative design.

Cases 2, 4, 6 and 8 of Table 6.2 compare the total cost of the Columbia River Bridge on the basis of equal unit price for steel, fabrication, transportation and erection. For these cases, the alternative new design is higher in cost. Cases 1, 3, 5 and 7 assume a lower unit price of steel superstructure. The resulting cost is lower for the new design.

7. SUMMARY AND CONCLUSIONS

In summary the following can be stated.

1. Reduction of box girder depth over the piers can be made with an increase of bottom flange plate thickness in order to keep the stresses within permissible limits.
2. Placement of composite concrete slab on the steel compression flange over the piers is equivalent to increasing the steel plate thickness and permits the reduction or elimination of the haunch profile.
3. Elimination of the haunch profile may require also an increase of midspan box girder depth.
4. For safe construction of steel box girders with composite concrete slabs in the negative moment region, the sequence of erecting box girder segments and placing of concrete slab is very important. Buckling of steel compression flange plates under lateral and axial loads during construction must be prevented.
5. The composite compression flange under lateral and axial loads during and after construction must also be checked to ensure strength and stability.
6. Anchors or shear connectors are needed to anchor the steel plate below the concrete slab and to transfer forces between the steel plate and the concrete slab. Not much information is available concerning composite action of steel-concrete plates. The existing provisions for shear connection in concrete decks can be temporarily adopted.
7. The effects of concrete shrinkage are found to be not governing. The strains due to creep of concrete generate differential strains in the concrete slab and the steel plate. Again, little information is available on this behavior.

8. The strength and weight of concrete have some moderate effect on the compressive stress in the composite bottom flange but affects little the stresses in the midspan sections.
9. The elimination of haunched profile permits easier fabrication of the steel box girders and result in lower unit cost for the steel superstructure. The weight of the steel superstructure may or may not be reduced. Concrete slab is added to the superstructure while additional height of pier also may be needed. The resulting total cost is lower for one of the sample bridge designs and is expectedly lower for the second sample bridge design. A third sample bridge design has uniform depth along its length and is found to have no need for composite compressive bottom flange over the piers.

In examining the feasibility of the steel-concrete composite compression flange for continuous box girders, it is realized that prestressed concrete deck can also be made composite with the box girder top flange over the piers. This condition may add to the efficiency of the composite compression flange, at least for moderately long and medium length continuous box girders. Study of this approach and of the strength of composite compression plates are suggested.

The conclusion, at this time, is that it is feasible structurally and economically to construct composite steel-concrete compression flanges over the negative moment region of continuous steel box girders.

8. ACKNOWLEDGMENTS

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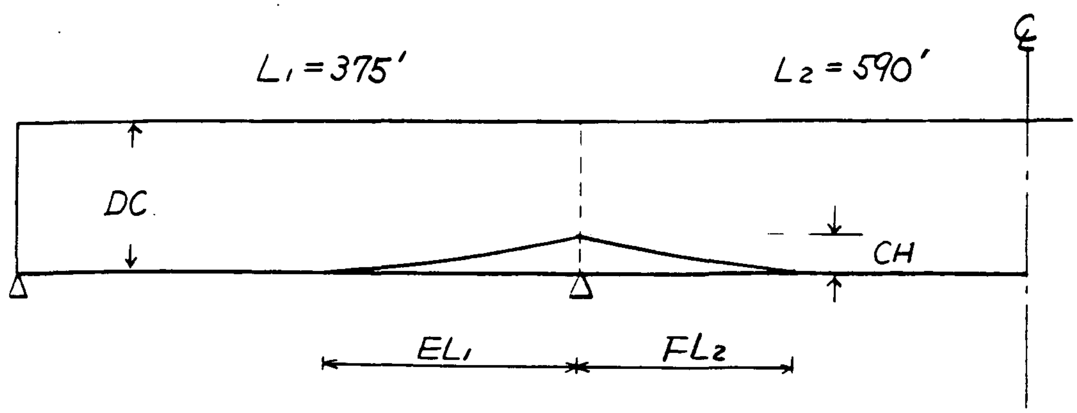
TABLE 2.1 DIMENSIONS OF ORIGINAL DESIGN

BRIDGE	WEST SEATTLE BRIDGE	COLUMBIA RIVER BRIDGE	TENNESSEE TOMBIGBEE WATERWAY
TYPE	Haunched Twin Rectangular Box	Haunched Single Trapezoidal Box (Twin cells)	Constant Depth Twin Trapezoidal Box
L SPANS	375'- 590' -375'	310'-400'-450'-400'-310'	200'-420'-200'
D_p DEPTH AT PIER	27'	16' , 21'-4"	12'-6"
D_c DEPTH AT CENTER OF SPAN	12'-6"	10'-4"	12'-6"
B_b WIDTH OF BOTTOM FLANGE	240"	178.5" each cell	66"
T_w WEB PLATE THICKNESS	$\frac{5}{8}$ " - 1"	$\frac{1}{2}$ " - $\frac{3}{4}$ "	$\frac{1}{2}$ "
T_b BOTTOM PLATE THICKNESS	$\frac{1}{2}$ " - 2"	$\frac{9}{16}$ " - $1\frac{1}{2}$ "	$\frac{3}{8}$ " - 2"
LONGITUDINAL STIFFENER AT PIER	6- ST 10* 47.5	10 WT 8* 28.5	2- WT shape varies in size
DIAPHRAGM SPACING	14'-9"= 177"	24'-9"= 297"	25'= 300"

TABLE 2.2 TRIAL DESIGNS, WEST SEATTLE BRIDGE

Comparison — West Seattle Bridge



- During the comparison only DC, CH, EL1, EL2, TB (thickness of bottom flange) are changed

TABLE 2.2 TRIAL DESIGNS, WEST SEATTLE BRIDGE (continued)

Number	DC	CH	E	F	TB pier	TB center	σ-	σ+	Comments
Original Design	Haunched Section 12.5'~27'				2"	7/8"	32.7	47.9	LFD
1	12.5'	–	–	–	2.5"	1.5"	53.2	52.6	NG
2	16'	–	–	–	"	"	41.4	40.7	
3	19'	–	–	–	"	"	36.2	34.1	
4	12.5'	–	–	–	"	1"	54	66.8	NG
5	16'	–	–	–	"	"	42	51.2	NG
6	19'	–	–	–	"	"	35.4	42.4	
7	12.5'	1.5'	0.3	0.2	"	1.5"	34.4	50.4	NG
8	14'	"	"	"	"	"	30.6	44.8	
9	16'	"	0.62	0.3125	2"	1"	30.8	47.8	
10	16'	"	0.4	0.25	"	1.5"	29.8	39	
11	16'	"	0.3	0.2	"	"	29.4	39.3	
12	15'	1.5'	0.62	0.3125	2"	1"	32.9	51.3	NG
13	15'	"	0.4	0.25	"	1.5"	31.9	41.6	
14	15'	"	0.3	0.2	"	"	31.5	42	
15	16'	1.5'	0.4	0.25	2"	1"	30	48.3	

TABLE 2.3 TRIAL DESIGNS, COLUMBIA RIVER BRIDGE

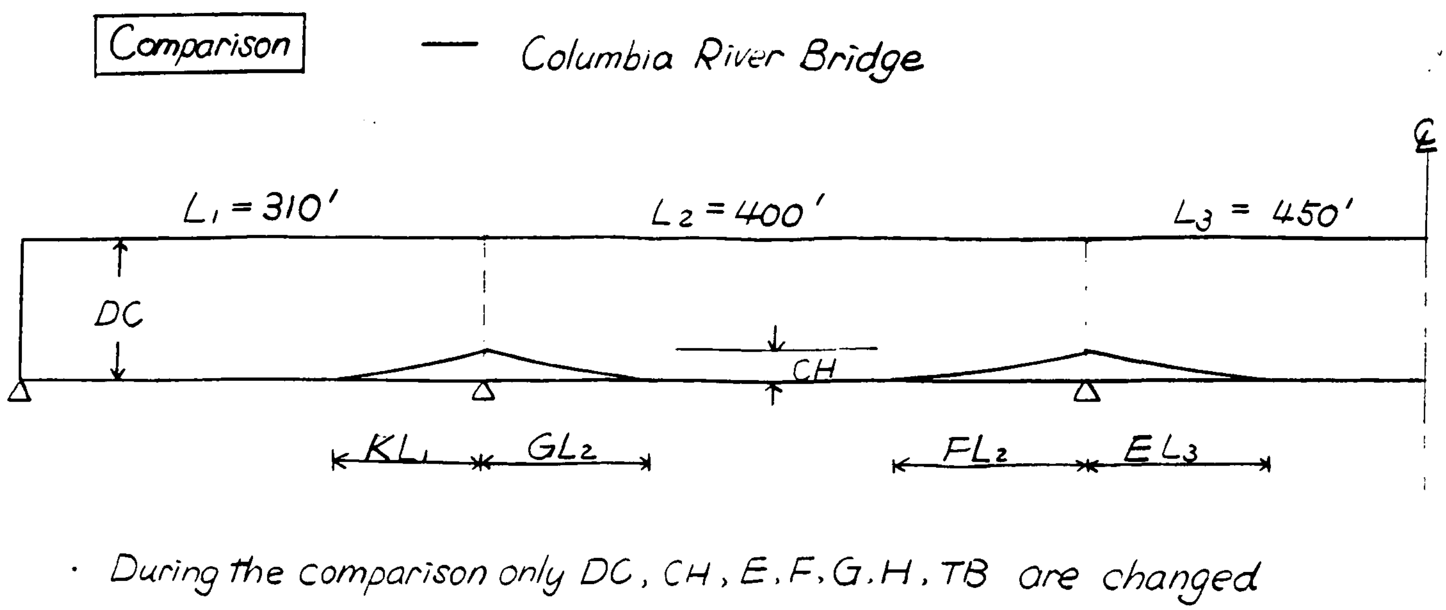


TABLE 2.3 TRIAL DESIGNS, COLUMBIA RIVER BRIDGE (continued)

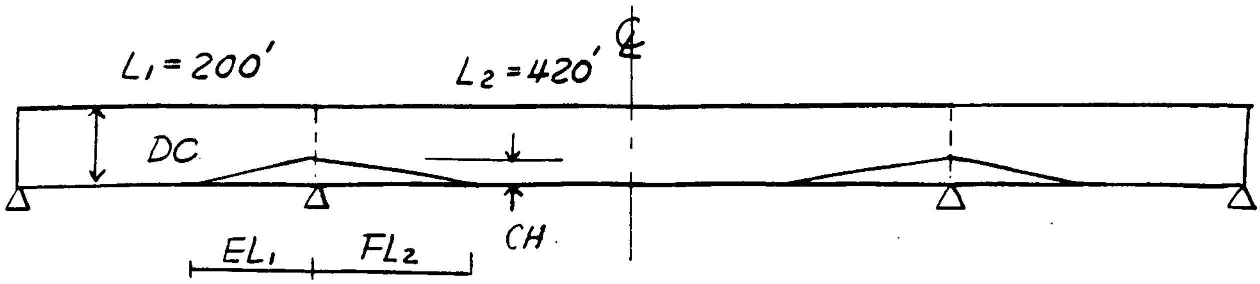
Number	DC	CH	E	F	G	K	TB (at L_3, L_4) pier	TB (at L_3, L_4) center	σ_-	σ_+	Comments
Original Design	Haunched Section 10.33' — 16' — 21.33'						1.5"	5/8"	38.8	41.5	LFD
1	10.33'	2'	0.298	0.335	0.235	0.303	"	"	36.2	48.8	
2	12'	"	"	"	"	"	"	"	30.4	41.5	
3	10.33'	"	0.2	0.225	0.16	0.2	"	1"	35.2	45	
4	12'	"	"	"	"	"	"	"	29.5	38.6	
5	10.33'	"	"	"	"	"	2"	"	32	44.4	
6	12'	"	"	"	"	"	"	"	26.9	38.2	
7	10.33'	1.5'	0.298	0.335	0.235	0.303	1.5"	5/8"	38.5	49.2	
8	12'	"	"	"	"	"	"	"	32.6	41.9	
9	10.33'	"	0.2	0.225	0.16	0.2	"	1"	37.6	45.3	
10	12'	"	"	"	"	"	"	"	31.8	38.9	
11	10.33'	"	"	"	"	"	2"	"	33.8	44.6	
12	12'	"	"	"	"	"	"	"	28.7	38.4	

TABLE 2.4 DIMENSIONS OF ALTERNATIVE DESIGN

BRIDGE	WEST SEATTLE BRIDGE	COLUMBIA RIVER BRIDGE
TYPE	Constant Depth Twin Rectangular Box	Constant Depth Single Trapezoidal Box (Twin cell)
L SPANS	375'-590'-375'	310'-400'-450'-400'-310'
D_c DEPTH OF BOX	16'	10'-4"
B_b WIDTH OF BOTTOM FLANGE	240"	178.5" each cell
T_w WEB PLATE THICKNESS	$\frac{5}{8}$" - 1"	$\frac{1}{2}$" - $\frac{3}{4}$"
T_b BOTTOM PLATE THICKNESS	$1\frac{1}{2}$" - 2"	1" - $1\frac{1}{2}$"
LONGITUDINAL STIFFENER AT PIER	No stiffener needed	No stiffener needed
DIAPHRAGM SPACING	14'-9"=177"	24'-9"=297"
CONCRETE THICKNESS AT PIER	1'-6"	1'-6"
CONCRETE LENGTH	112' , 118'	62',64',90',90'

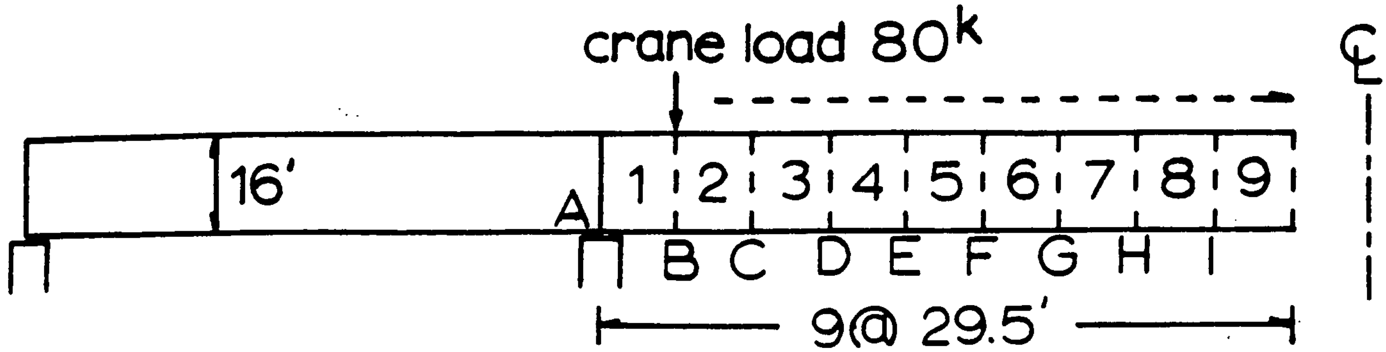
TABLE 2.5 TRIAL DESIGN, TENNESSEE TOMBIGBEE WATERWAY BRIDGE

$\boxed{\text{Comparison}}$ — Tennessee Tombigbee Waterway.



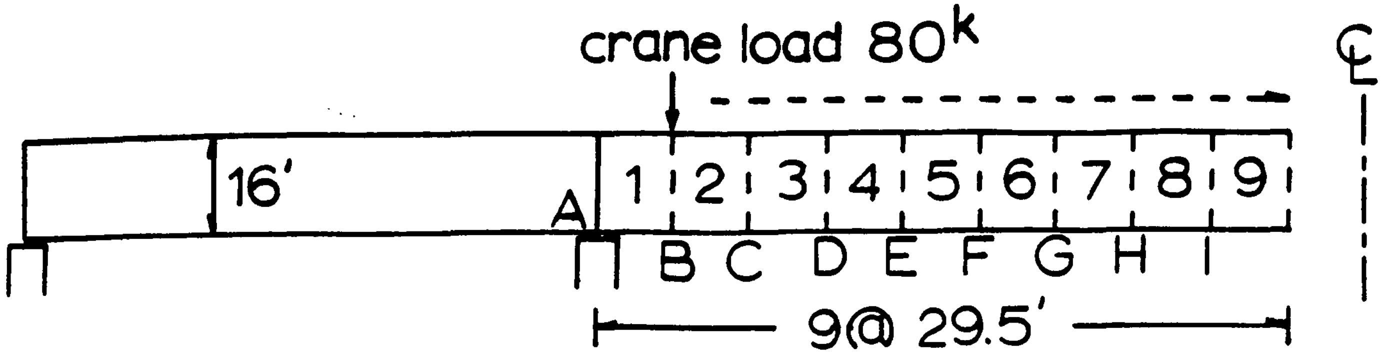
Number	DC	CH	E	F	TB pier	TB center	σ-	σ+	Comments
Original Design	Constant depth section DC = 12.5'				2"	1.375"	39.8	40.7	LFD
1	12.5'	1'	0.49	0.3	"	"	29	39	
2	"	2'	"	"	"	"	25.1	38.5	
3	"	–	–	–	1"	1"	86.3	68.9	NG
4	"	1'	0.49	0.3	"	"	45.7	62.1	NG
5	"	2'	"	"	"	"	35.5	60.1	NG
6	"	3'	"	"	"	"	32.2	59.6	NG
7	10'	1'	"	"	"	"	59	82.3	NG
8	"	2'	"	"	"	"	46.6	80	NG
9	"	3'	"	"	"	"	43.6	79.6	NG

TABLE 3.1 BOTTOM FLANGE STRESSES DUE TO WEIGHT OF STEEL BOXES



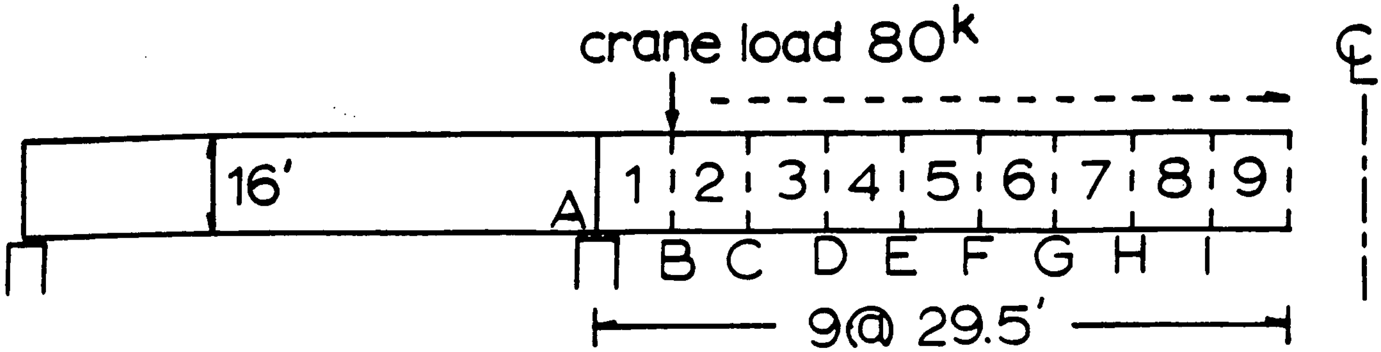
segment \ stress ksi	A	B	C	D	E	F	G	H	I
1	1.1								
2	2.9	1.1							
3		2.9	1.2						
4			3.1	1.2					
5				2.9	1.5				
6					3.8	1.5			
7						3.8	1.4		
8							3.3	1.3	
9								3.4	1.3

TABLE 3.2 BOTTOM FLANGE STRESSES DUE TO WEIGHT OF STEEL BOXES AND WET CONCRETE



segment \ stress ksi	A	B	C	D	E	F	G	H	I
1									
2	3.1								
3		3.							
4			3.2						
5				2.9					
6									
7									
8									
9									

TABLE 3.3 COMPOSITE FLANGE STRESSES DURING CONSTRUCTION



segment \ stress ksi	A	B	C	D	E	F	G	H	I
1									
2	1.8								
3	3.5	2.							
4	5.4	3.8	2.6						
5	7.4	5.8	4.6	2.7	1.5				
6	9.7	8.2	7.1	5.	3.8	1.5			
7	12.2	10.8	10.	7.9	5.4	3.8	1.4		
8	15.1	13.8	13.5	11.3	11.1	6.9	3.3	1.3	
9	18.2	17.3	17.5	15.3	16.	11.	6.9	3.4	1.3

TABLE 5.1 EFFECTS OF CONCRETE STRENGTH AND WEIGHT

WEST SEATTLE BRIDGE

Case	f'_c (ksi)	n	w_c (lb/ft³)	σ_- (ksi)	σ_+ (ksi)
Original	-	-	-	32.7	47.9
Trial 11	4	8	150	29.4	39.3
	6	6.5	150	26.8	38.8
	4	11	120	32.9	39.5
	6	9	120	30.7	39.4

$D_c = 16'$

$T_c = 18''$

TABLE 6.1 COST COMPARISON FOR BRIDGE SUPERSTRUCTURE (WEST SEATTLE BRIDGE)

CASE		Steel Weight (Kips)	Unit Price ($/Kips)	Cost of Steel	Concrete in Flange (yd^3)	Unit Price ($\$/yd^3$)	Concrete in Pier (yd^3)	Unit Price ($\$/yd^3$)	Cost of Concrete	TOTAL COST
1	old design	7270	3000	21810000	-	-	-	-	-	21810000
	new design	7027	2500	17567500	170	100	17	150	19550	17587050
2	old design	7270	2500	18175000	-	-	-	-	-	18175000
	new design	7027	2000	14054000	170	100	17	150	19550	14073550
3	old design	7270	2500	18175000	-	-	-	-	-	18175000
	new design	7027	2500	17567500	170	100	17	150	19550	17587050
4	old design	7270	2000	14540000	-	-	-	-	-	14540000
	new design	7027	2000	14054000	170	100	17	150	19550	14073550

TABLE 6.1 COST COMPARISON FOR BRIDGE SUPERSTRUCTURE (WEST SEATTLE BRIDGE) (continued)

CASE		Steel Weight (Kips)	Unit Price (\$/Kips)	Cost of Steel	Concrete in Flange (yd^3)	Unit Price (\$/yd^3)	Concrete in Pier (yd^3)	Unit Price (\$/yd^3)	Cost of Concrete	TOTAL COST
5	old design	7270	2000	14540000	-	-	-	-	-	14540000
	new design	7027	1500	10540500	170	100	17	150	19550	10560050
6	old design	7270	1500	10905000	-	-	-	-	-	10905000
	new design	7027	1500	10540500	170	100	17	150	19550	10560050
7	old design	7270	1500	10905000	-	-	-	-	-	10905000
	new design	7027	1000	7027000	170	100	17	150	19550	7046550
8	old design	7270	1000	7270000	-	-	-	-	-	7270000
	new design	7027	1000	7027000	170	100	17	150	19550	7046550

TABLE 6.2 COST COMPARISON FOR BRIDGE SUPERSTRUCTURE (COLUMBIA RIVER BRIDGE)

CASE		Steel Weight (Kips)	Unit Price ($/Kips)	Cost of Steel	Concrete in Flange (yd^3)	Unit Price ($\$/yd^3$)	Concrete in Pier (yd^3)	Unit Price ($\$/yd^3$)	Cost of Concrete	TOTAL COST
1	old design	7093	3000	21279000	–	–	–	–	–	21279000
	new design	7196	2500	17990000	170	100	117	150	34550	18024550
2	old design	7093	2500	17732500	–	–	–	–	–	17732500
	new design	7196	2000	14392000	170	100	117	150	34550	14426550
3	old design	7093	2500	17732500	–	–	–	–	–	17732500
	new design	7196	2500	17990000	170	100	117	150	34550	18024550
4	old design	7093	2000	14186000	–	–	–	–	–	14186000
	new design	7196	2000	14392000	170	100	117	150	34550	14426550

TABLE 6.2 COST COMPARISON FOR BRIDGE SUPERSTRUCTURE (COLUMBIA RIVER BRIDGE) (continued)

CASE		Steel Weight (Kips)	Unit Price ($/Kips)	Cost of Steel	Concrete in Flange (yd^3)	Unit Price ($\$/yd^3$)	Concrete in Pier (yd^3)	Unit Price ($\$/yd^3$)	Cost of Concrete	TOTAL COST
5	old design	7093	2000	14186000	–	–	–	–	–	14186000
	new design	7196	1500	10794000	170	100	117	150	34550	10828550
6	old design	7093	1500	10639500	–	–	–	–	–	10639500
	new design	7196	1500	10794000	170	100	117	150	34550	10828550
7	old design	7093	1500	10639500	–	–	–	–	–	10639500
	new design	7196	1000	7196000	170	100	117	150	34550	7230550
8	old design	7093	1000	7093000	–	–	–	–	–	7093000
	new design	7196	1000	7196000	170	100	117	150	34550	7230550

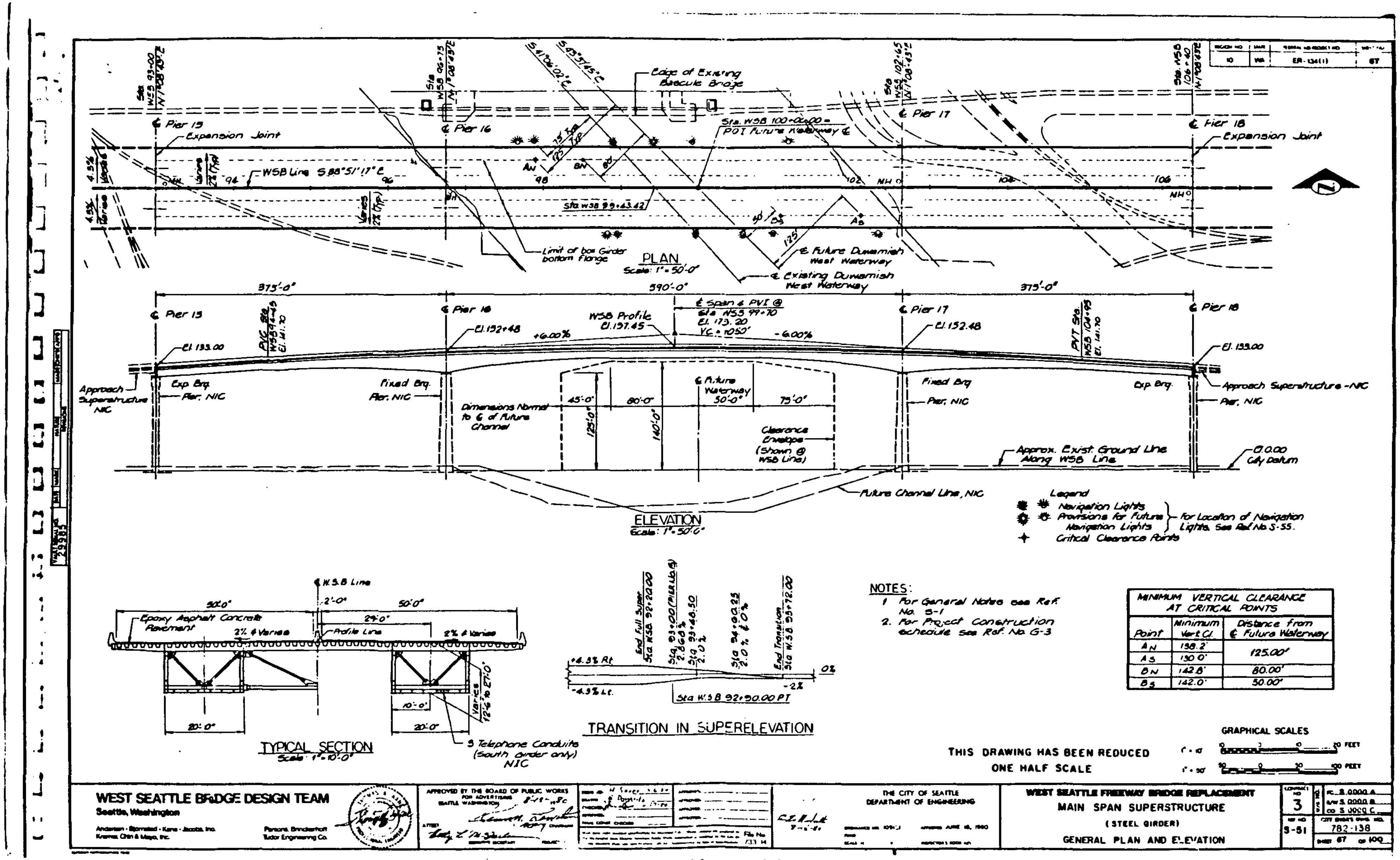


Fig. 1.1 General Plan and Elevation, West Seattle Bridge

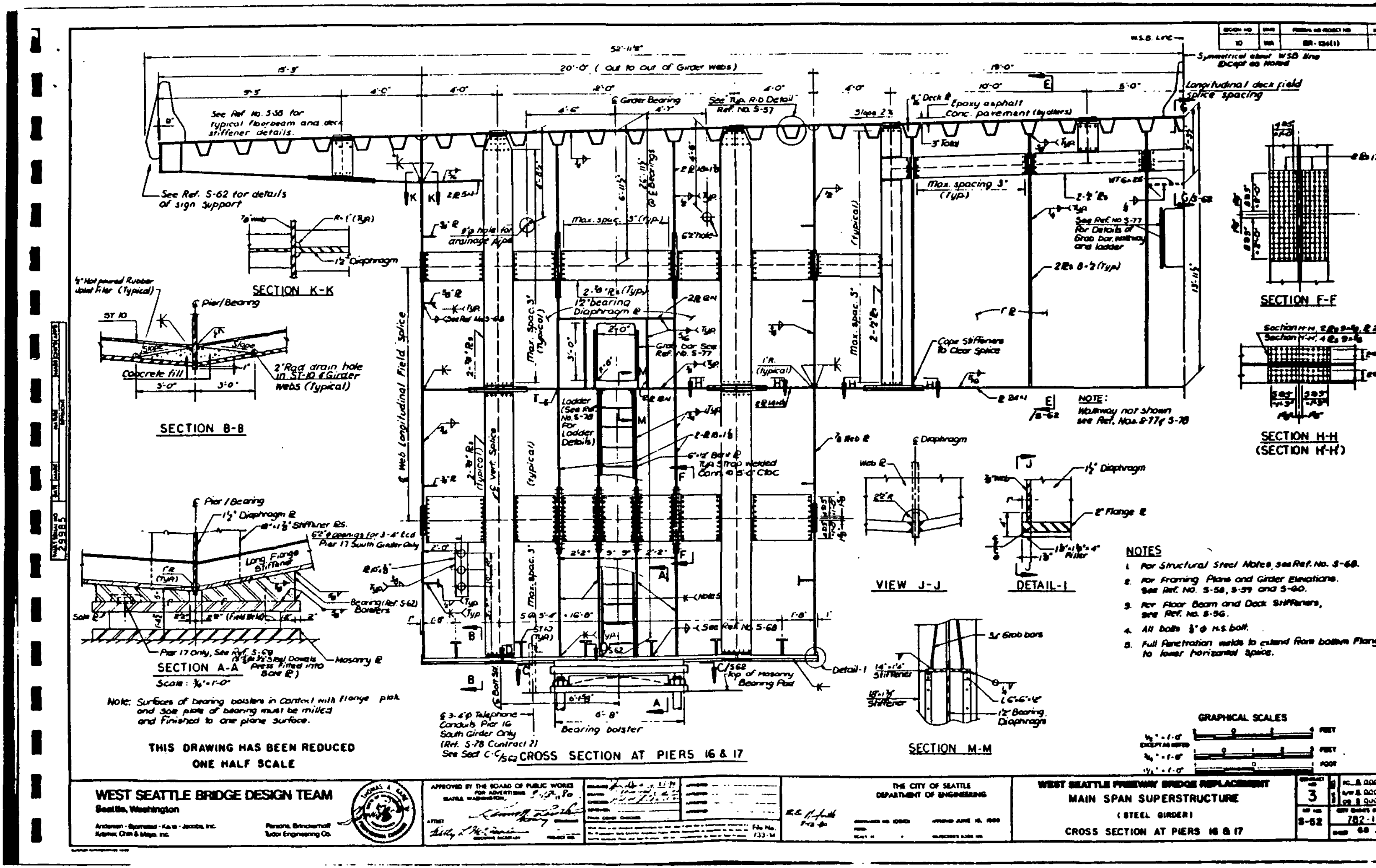


Fig. 1.2 Cross-sections of West Seattle Bridge

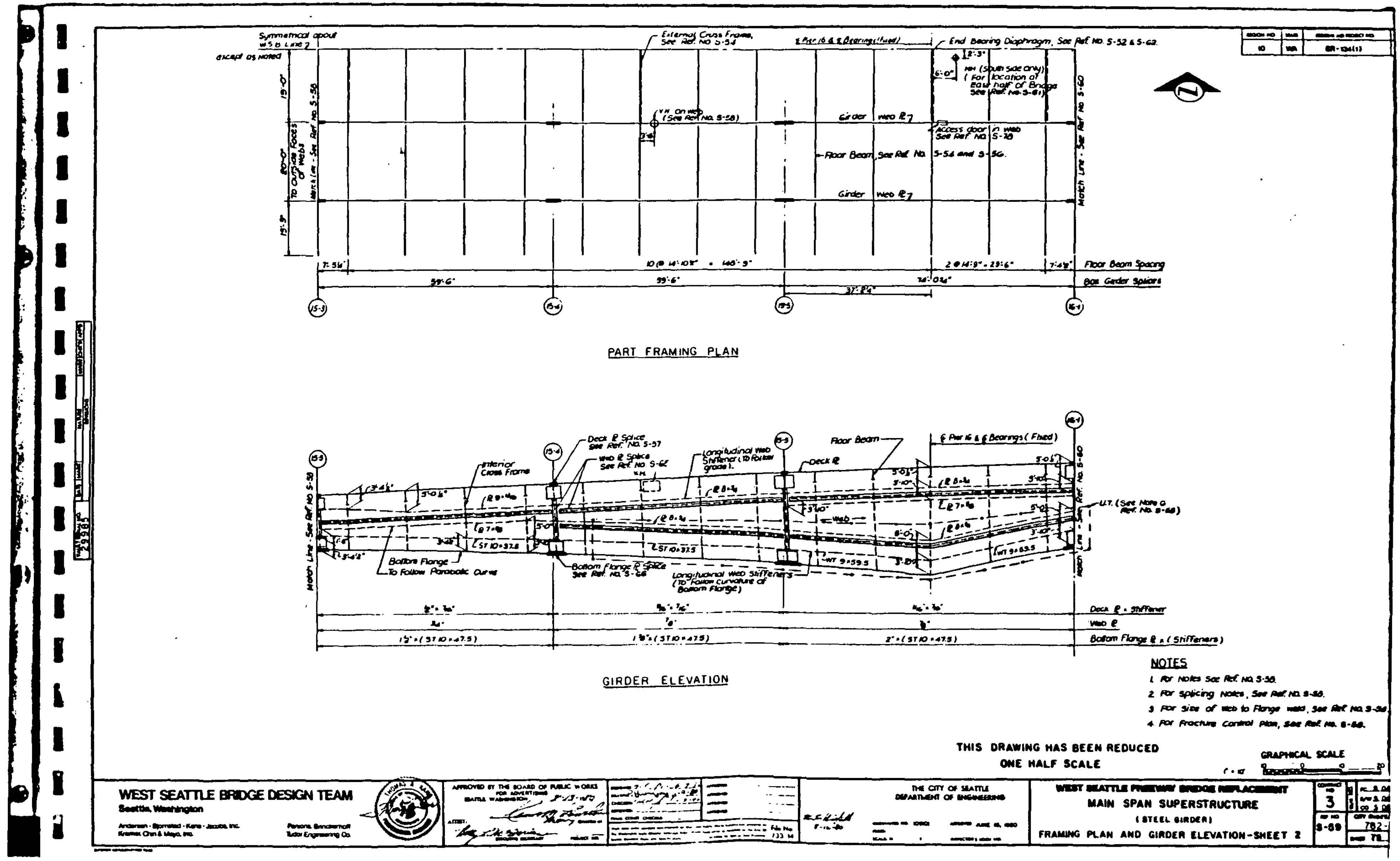


Fig. 1.3 Framing Plan and Girder Elevation, West Seattle Bridge

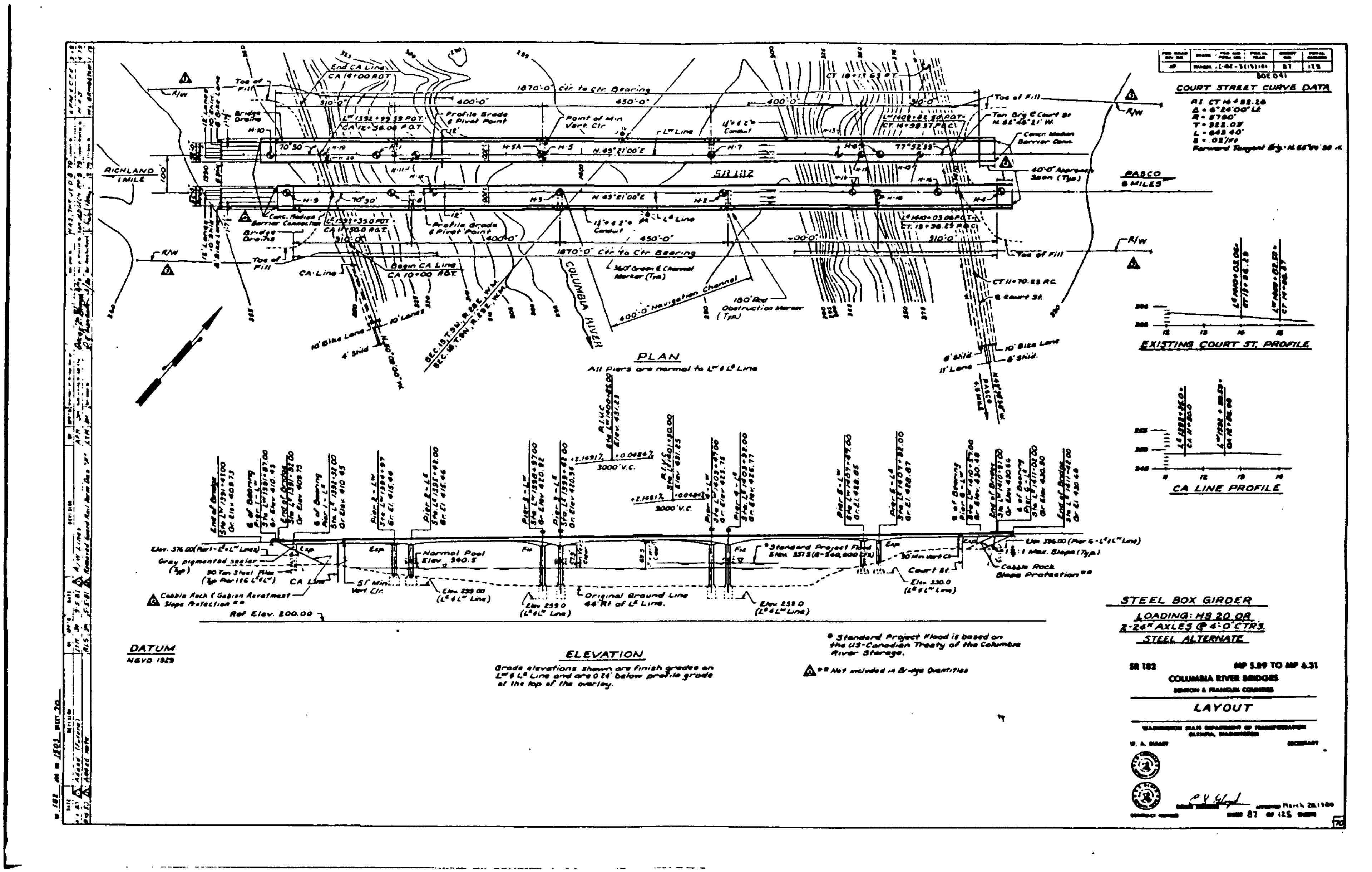


Fig. 1.4 General Plan and Elevation, Columbia River Bridge

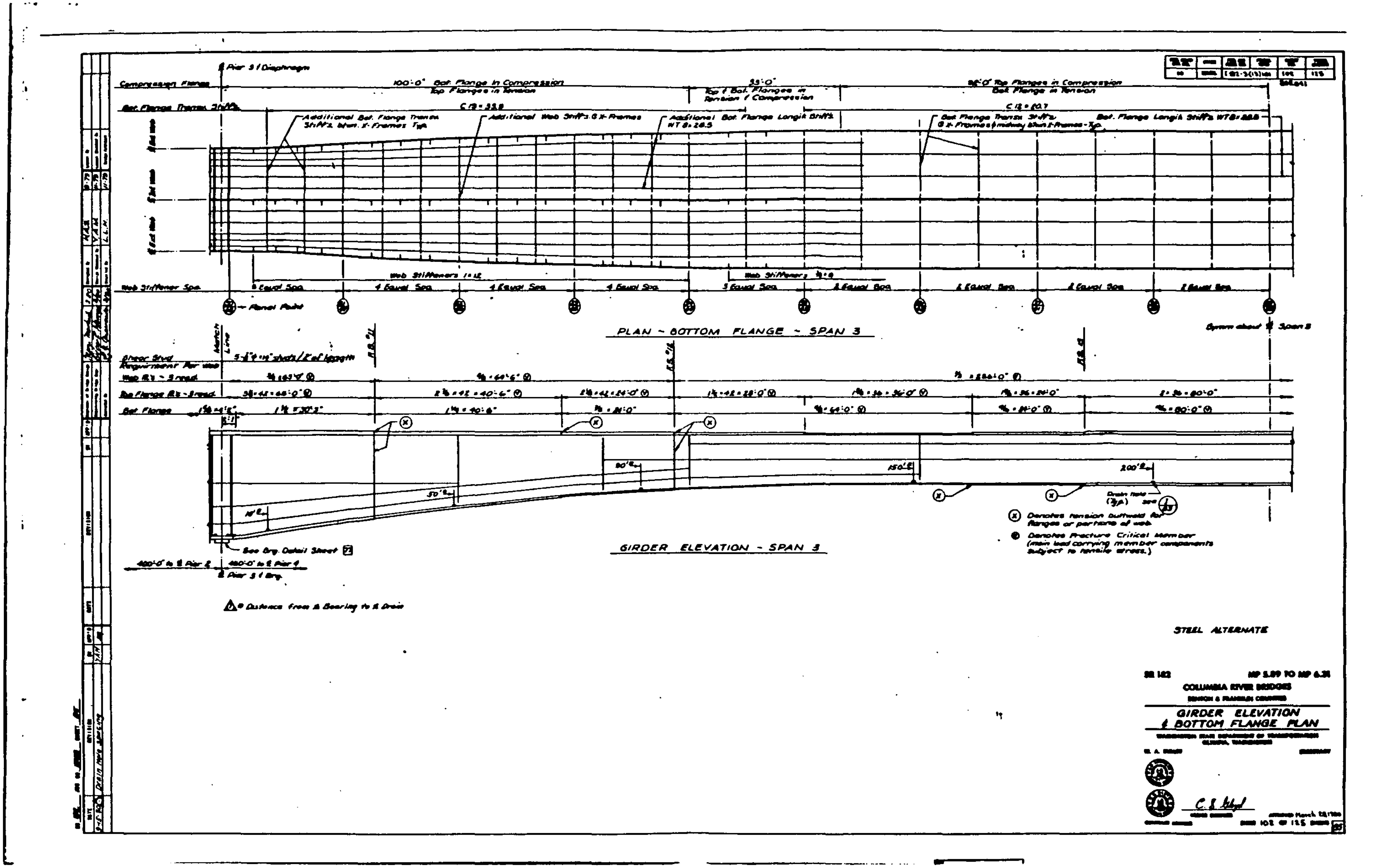


Fig. 1.5 Framing Plan and Girder Elevation, Columbia River Bridge

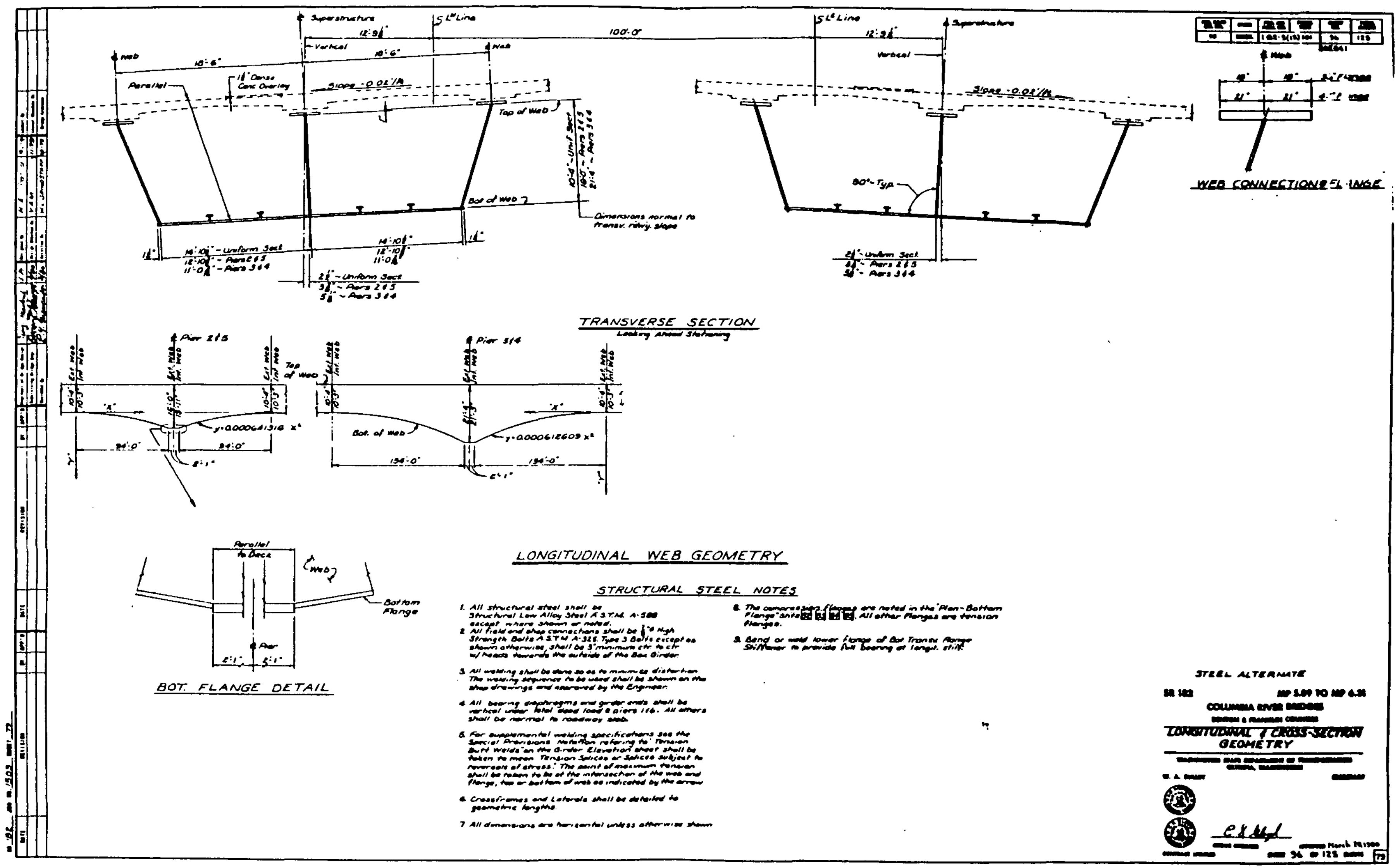


Fig. 1.6 Cross-section Geometry, Columbia River Bridge

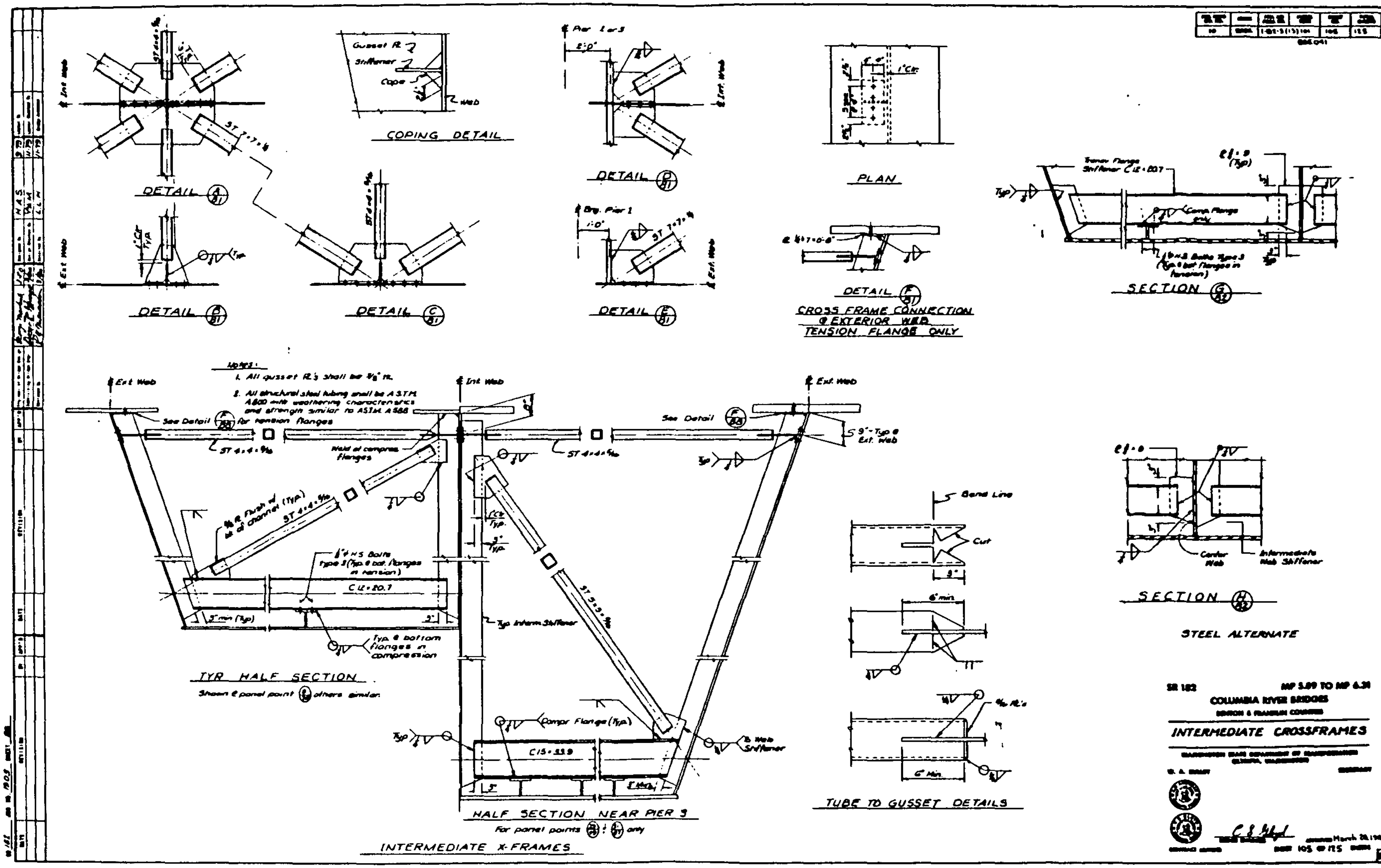


Fig. 1.7 Intermediate Cross Frames, Columbia River Bridge

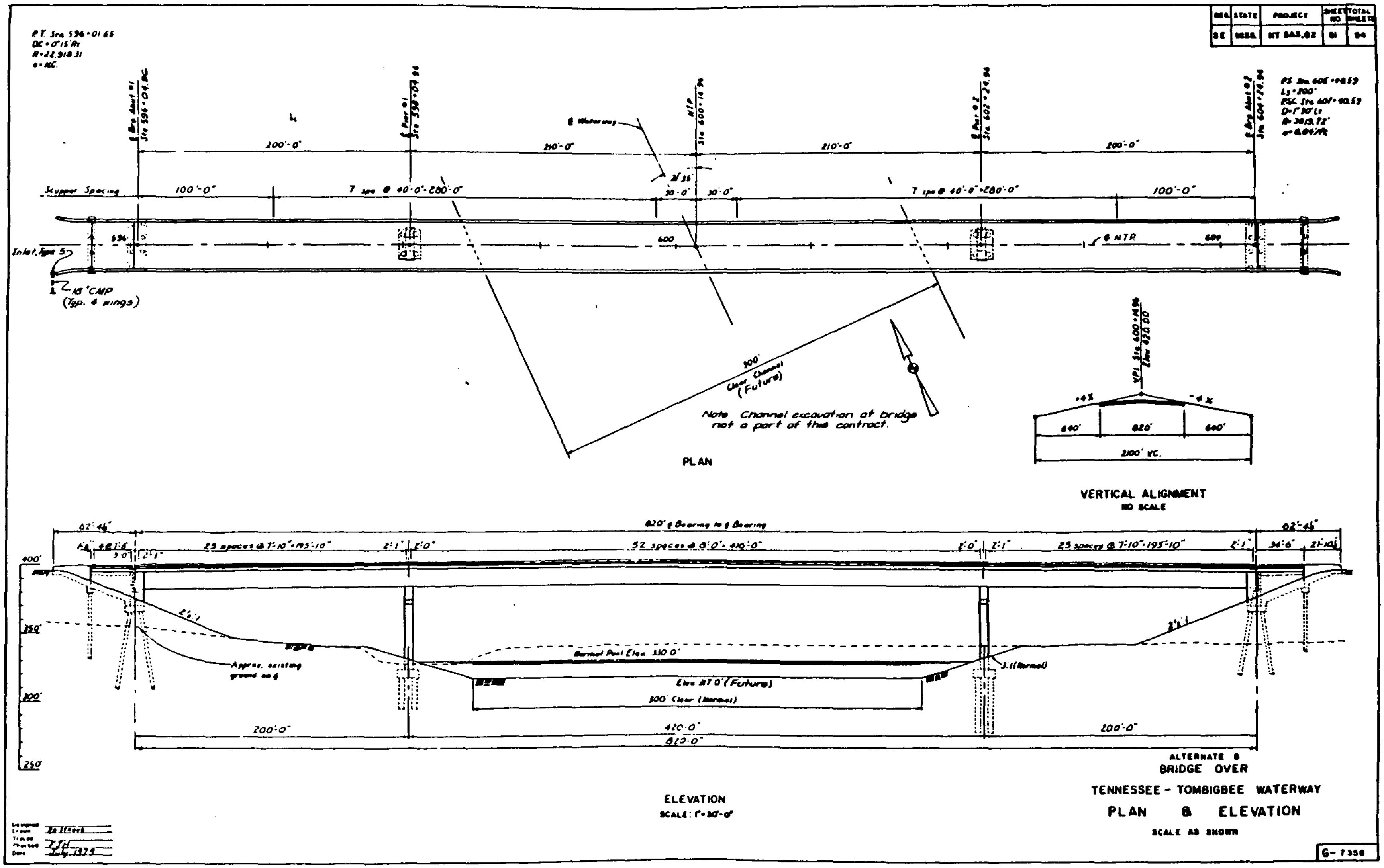


Fig. 1.8 Plan and Elevation, Tennessee - Tombigbee Waterway Bridge

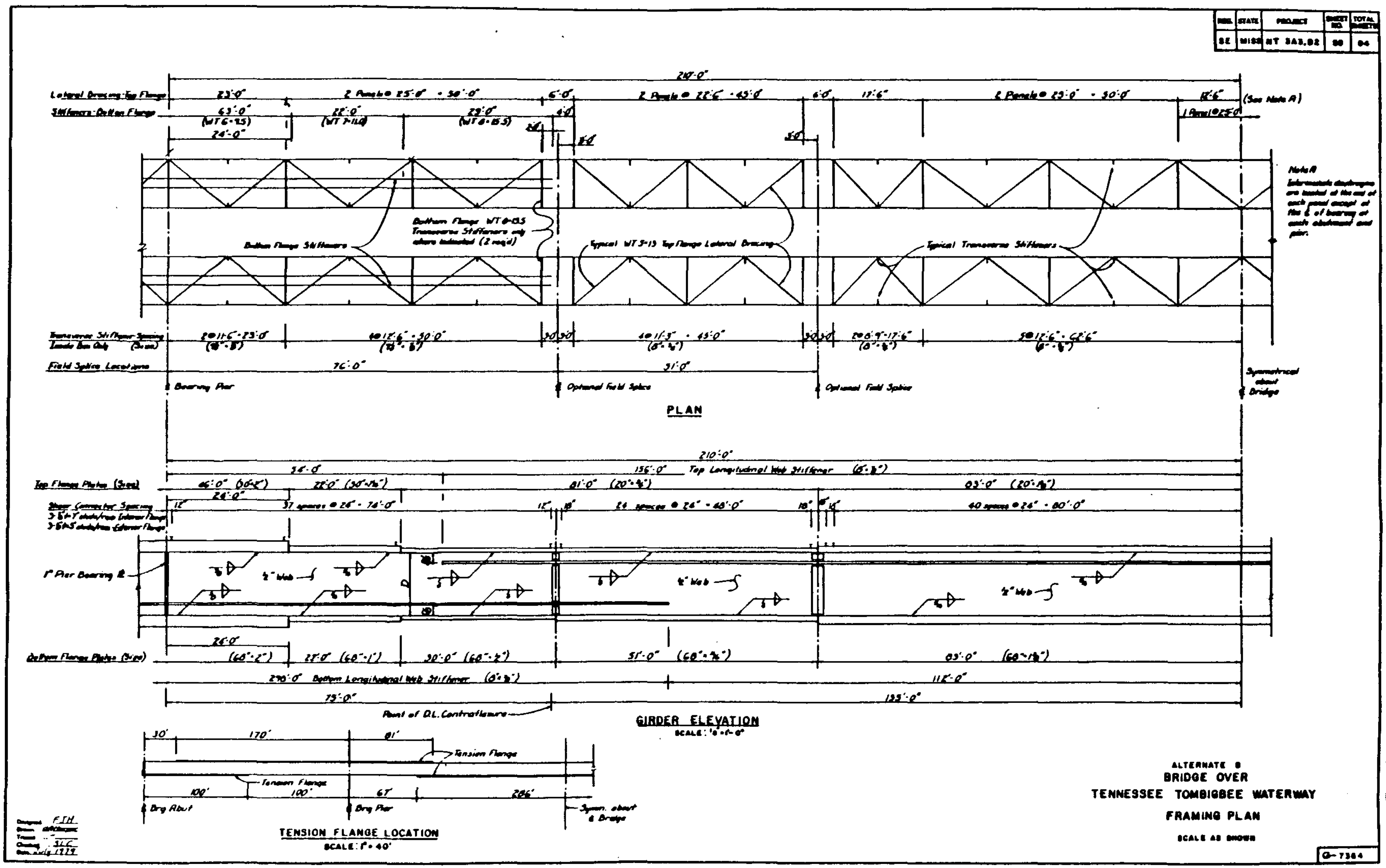


Fig. 1.9 Framing Plan and Elevation, Tennessee - Tombigee Waterway Bridge

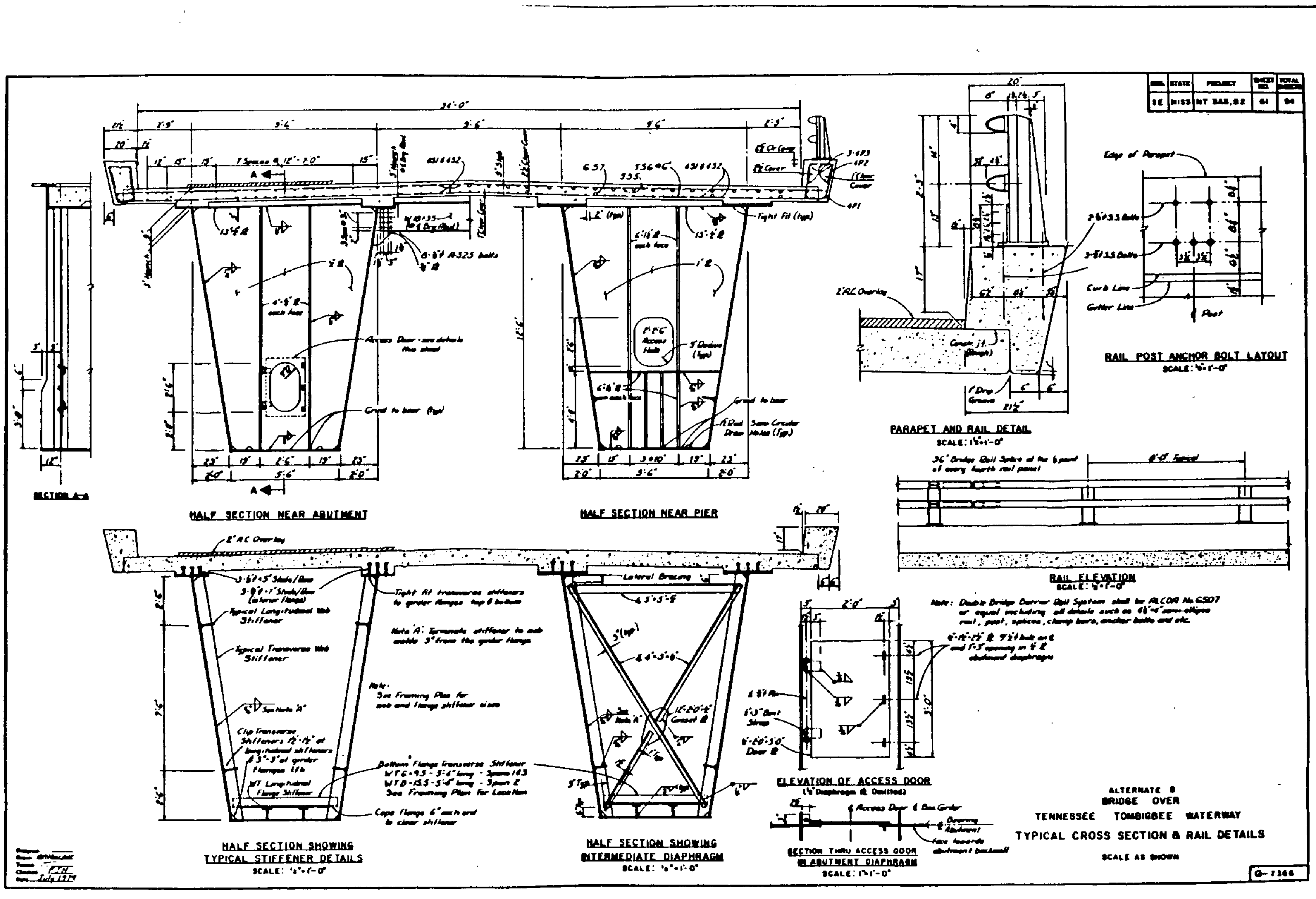


Fig. 1.10 Typical Cross-section, Tennessee - Tombigee Waterway Bridge

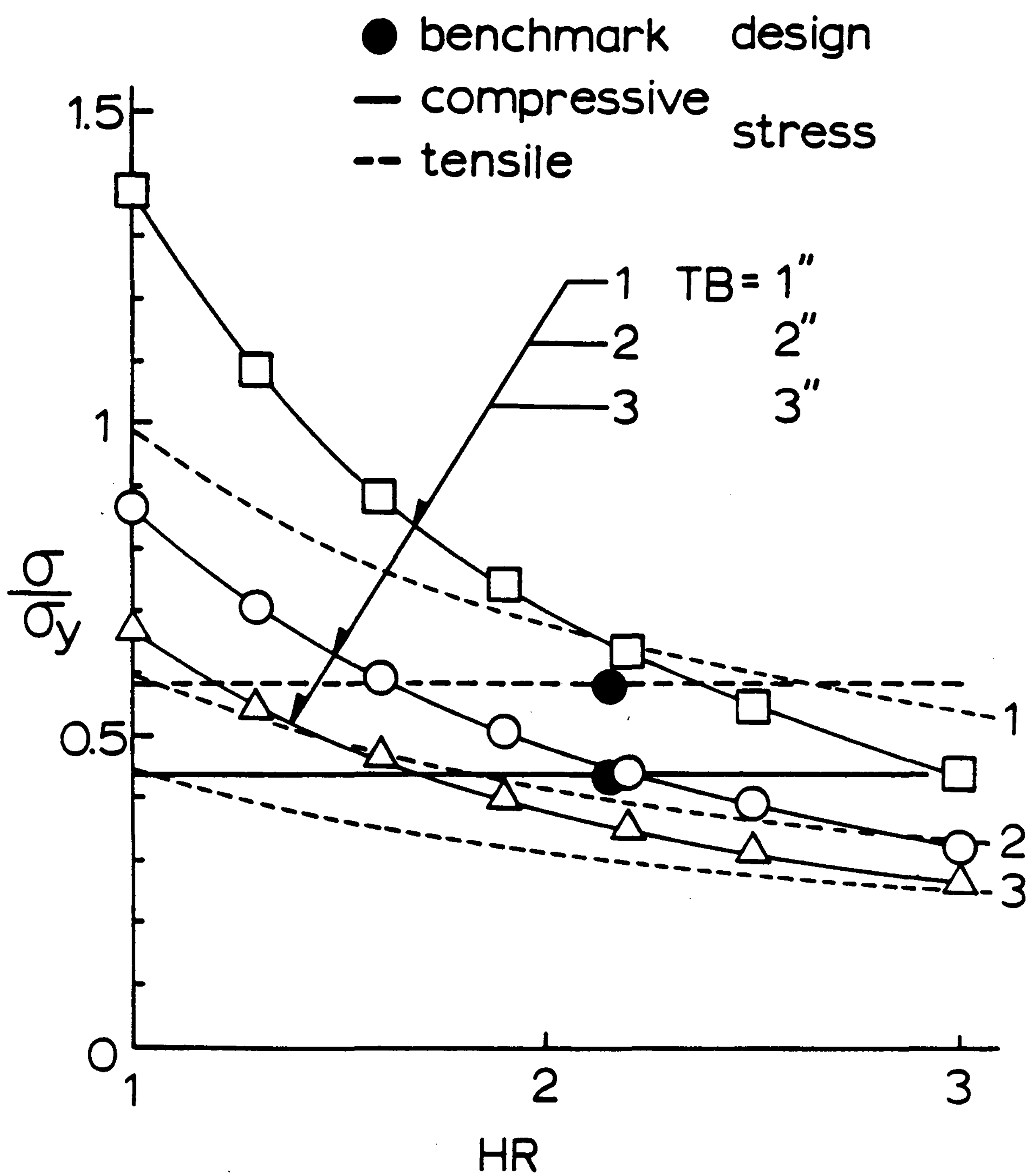


Fig. 2.1 Effect of Bottom Flange Thickness on Flange Stresses

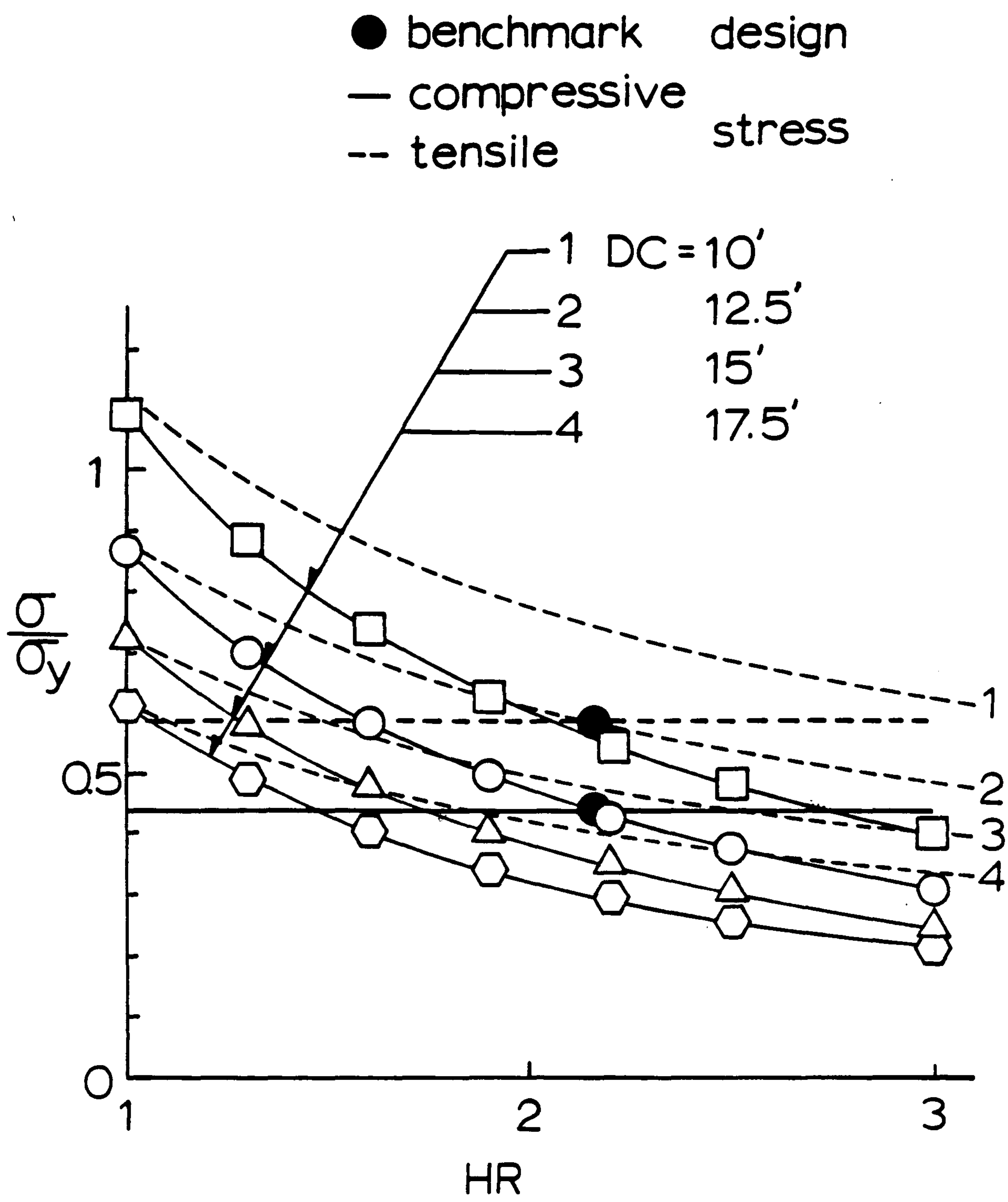


Fig. 2.2 Effect of Midspan Depth on Flange Stresses

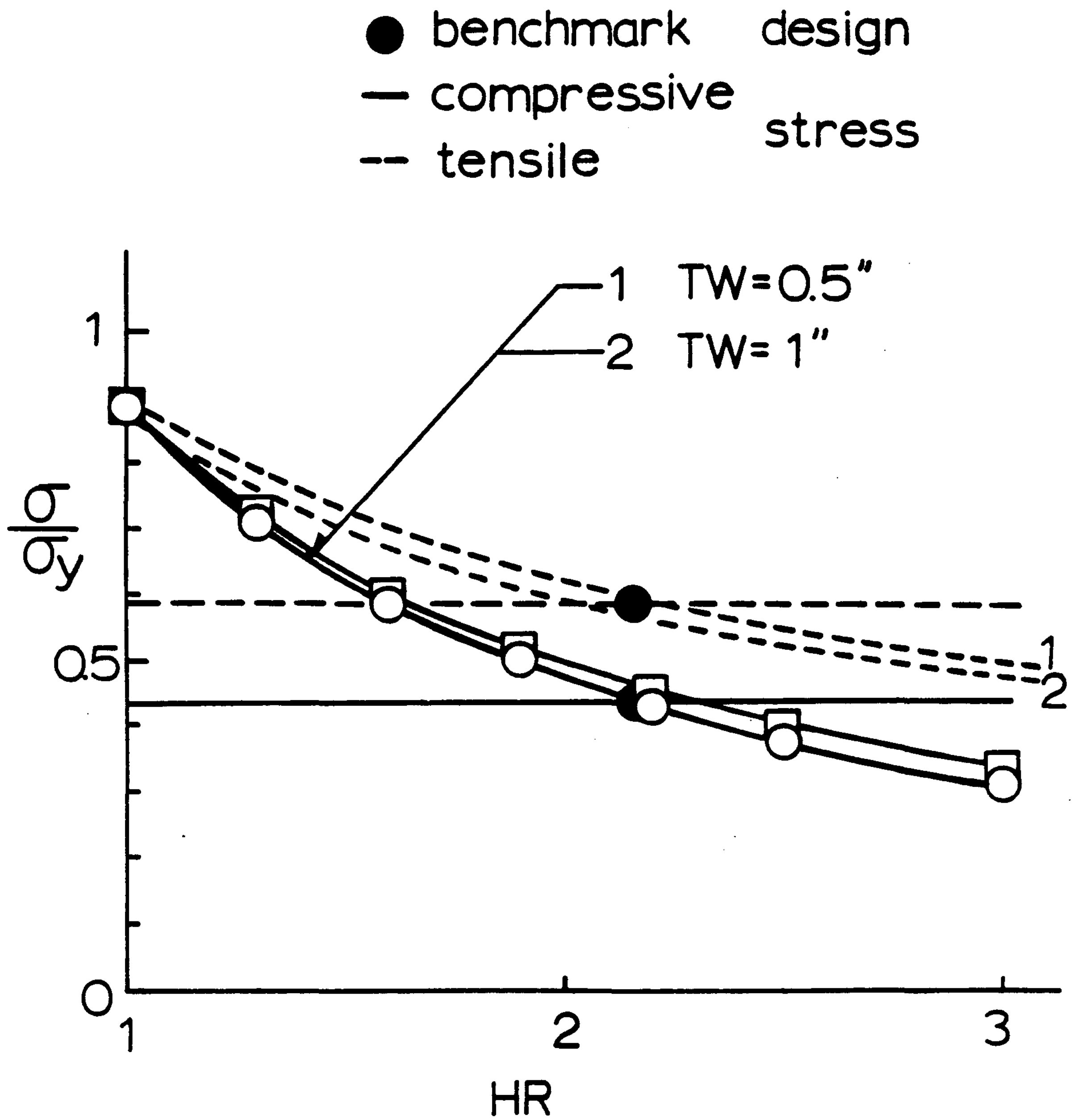


Fig. 2.3 Effect of Thickness of Web Plates on Bottom Flange Stresses

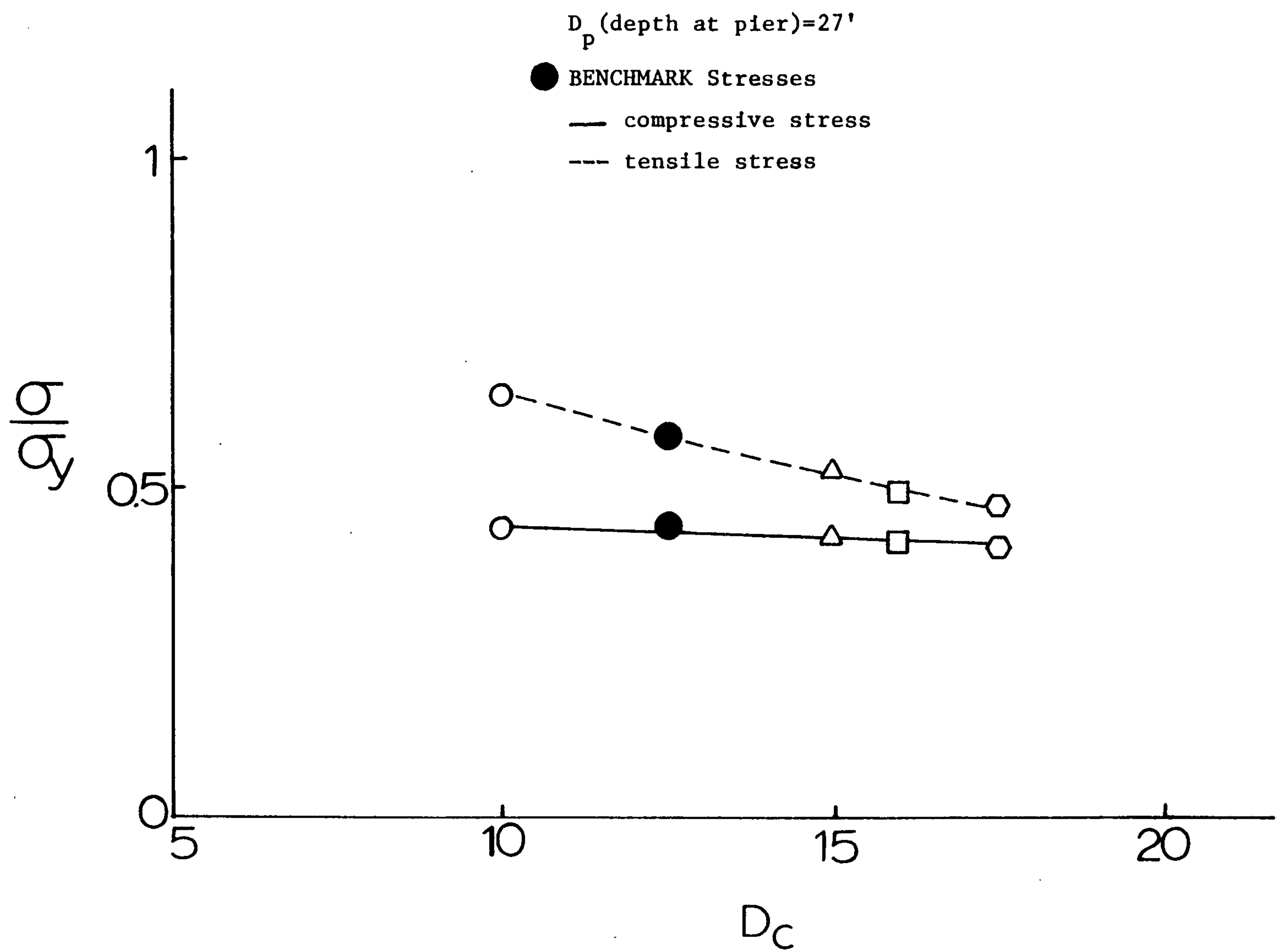


Fig. 2.4 Effect of Midspan Depth on Bottom Flange Stresses

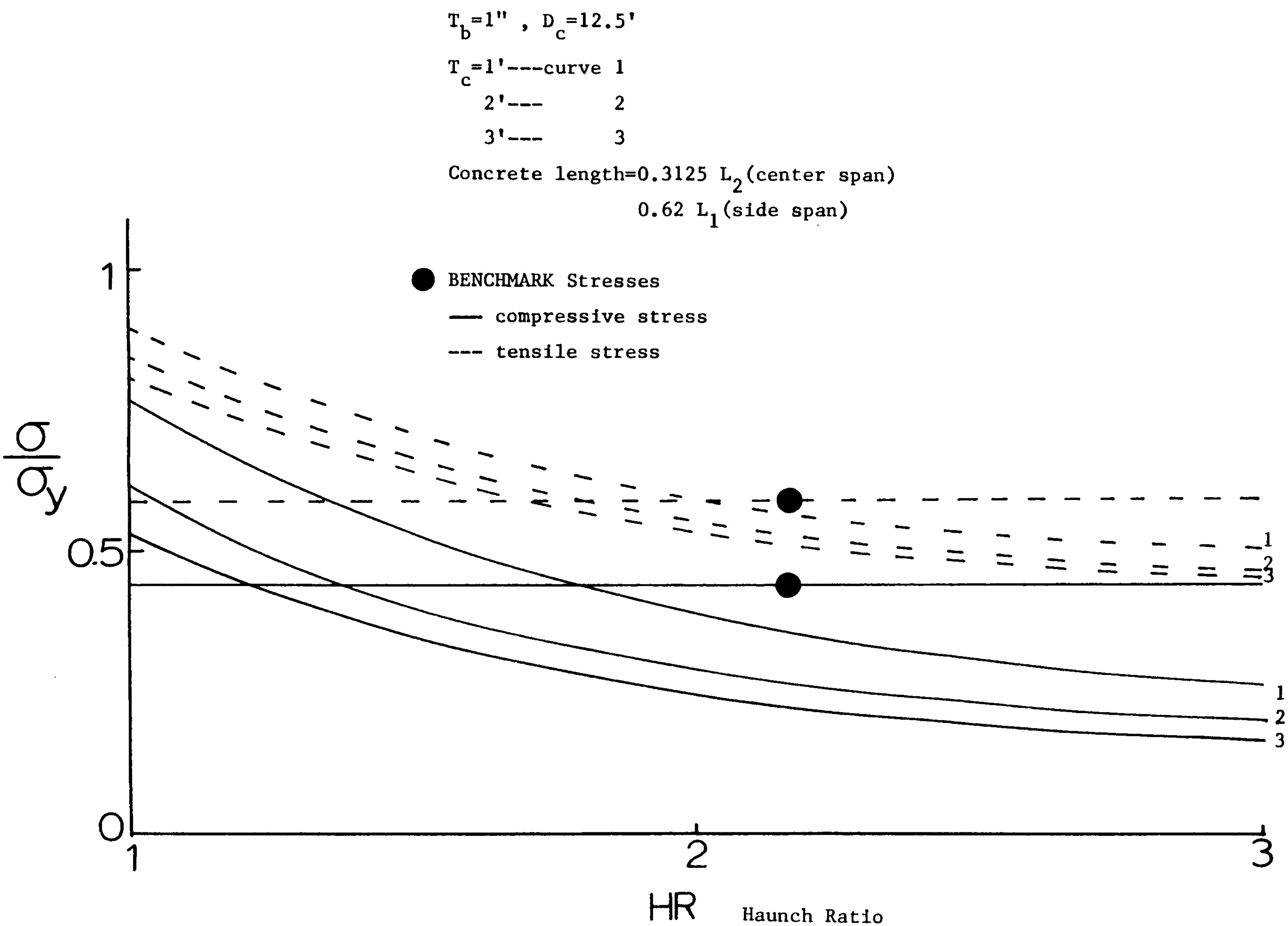


Fig. 2.5 Effect of Concrete Slab Thickness on Bottom Flange Stresses

Curve 1: Concrete length=0.3125 L_2(center span), 0.62 L_1(side span)
2: 0.2 L_2 , 0.4 L_1
3: 0.1 L_2 , 0.2 L_1

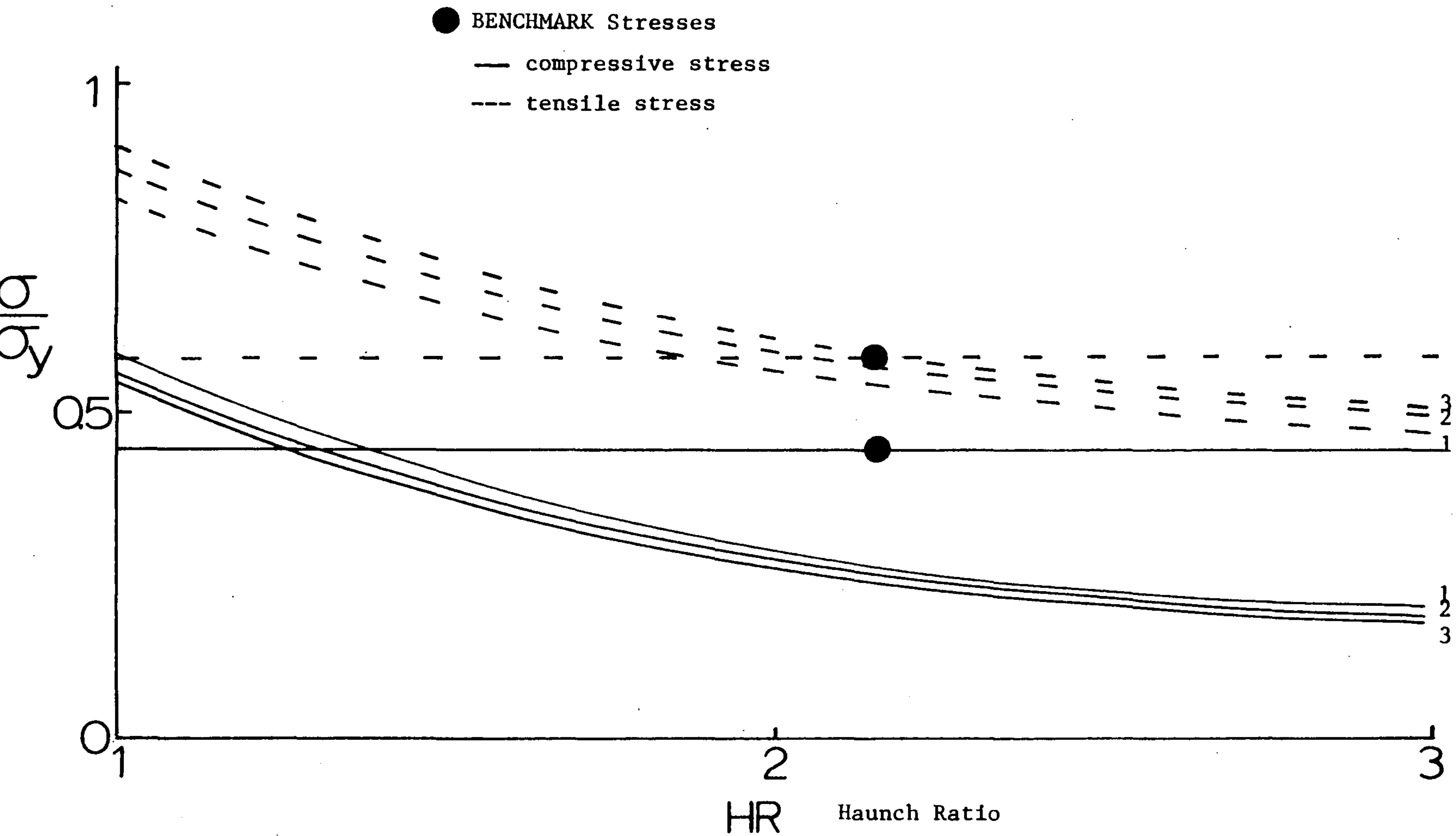


Fig. 2.6 Effect of Concrete Slab Length on Bottom Flange Stresses

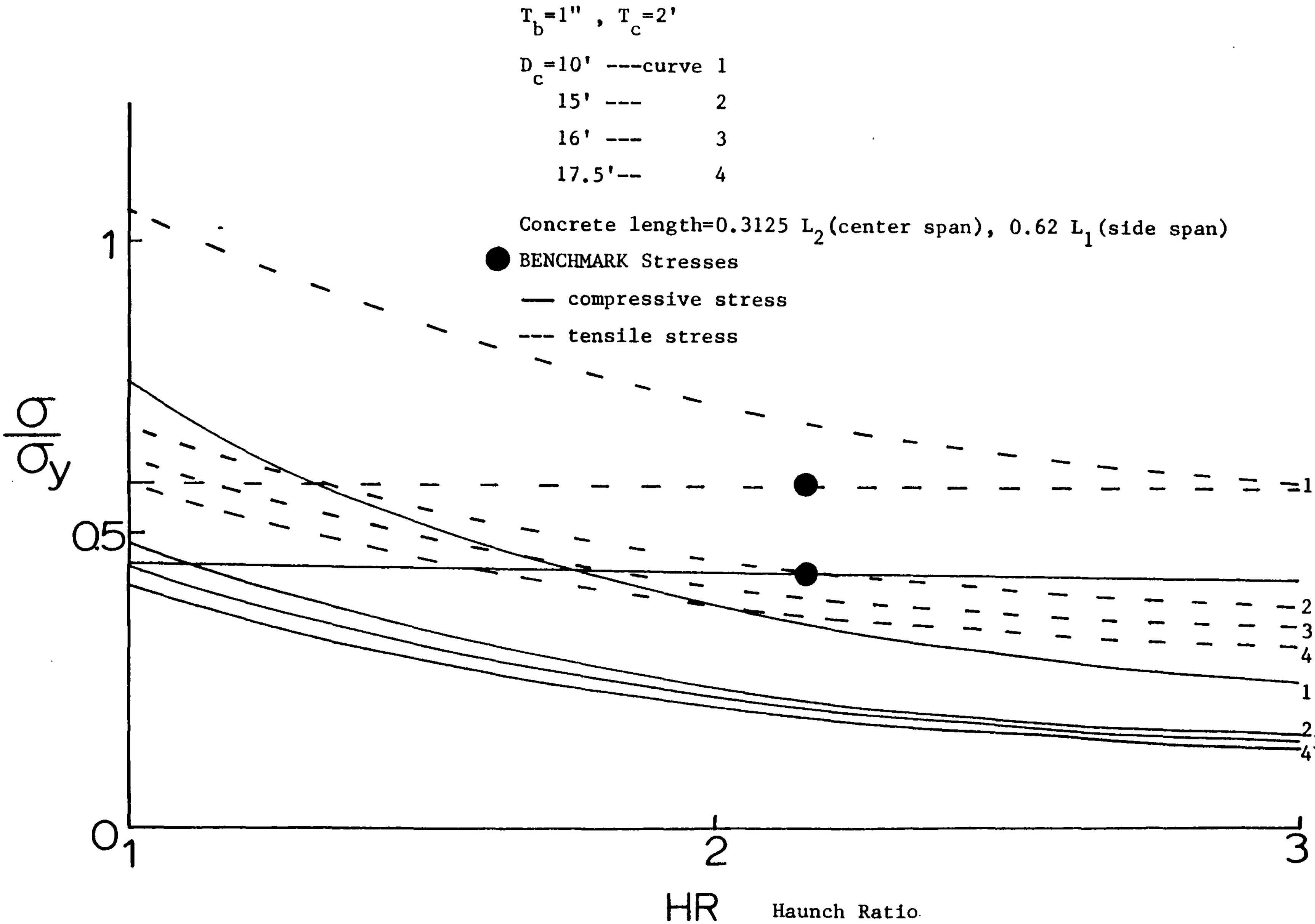


Fig. 2.7 Effect of Midspan Depth on Stresses in Flanges with Composite Slab

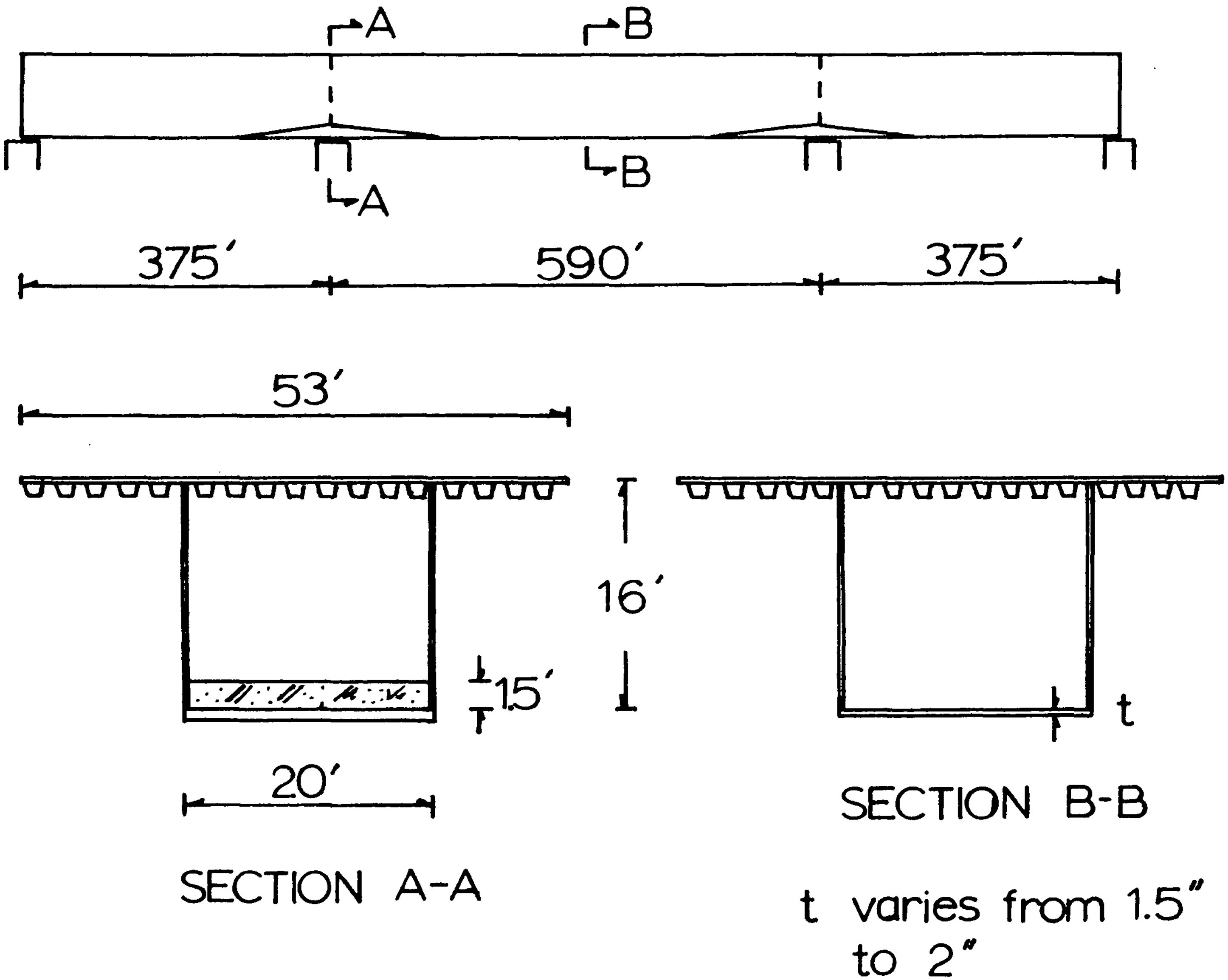


Fig. 2.8 Alternative Design, West Seattle Bridge

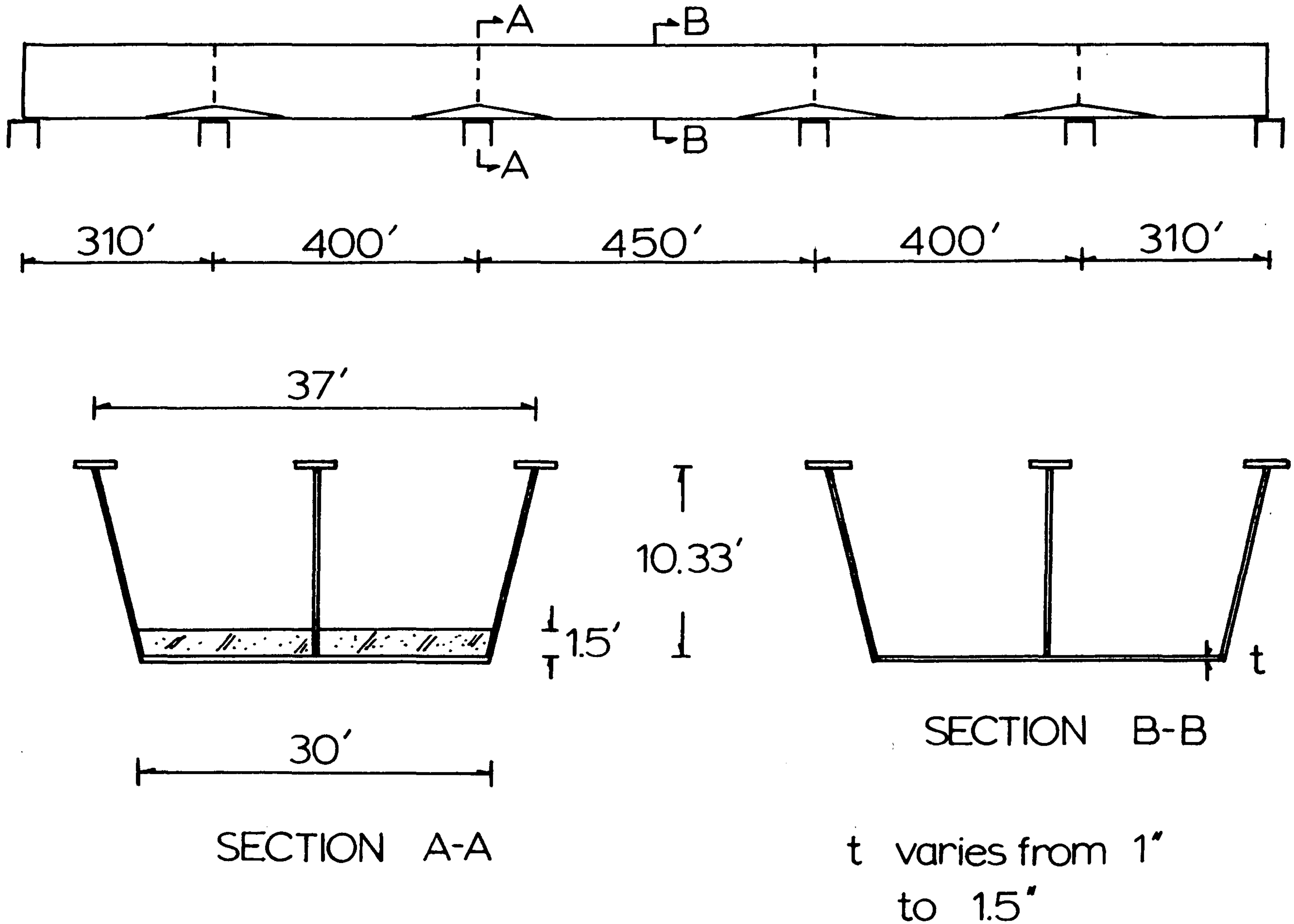


Fig. 2.9 Alternative Design, Columbia River Bridge

SUGGESTED ERECTION SEQUENCE

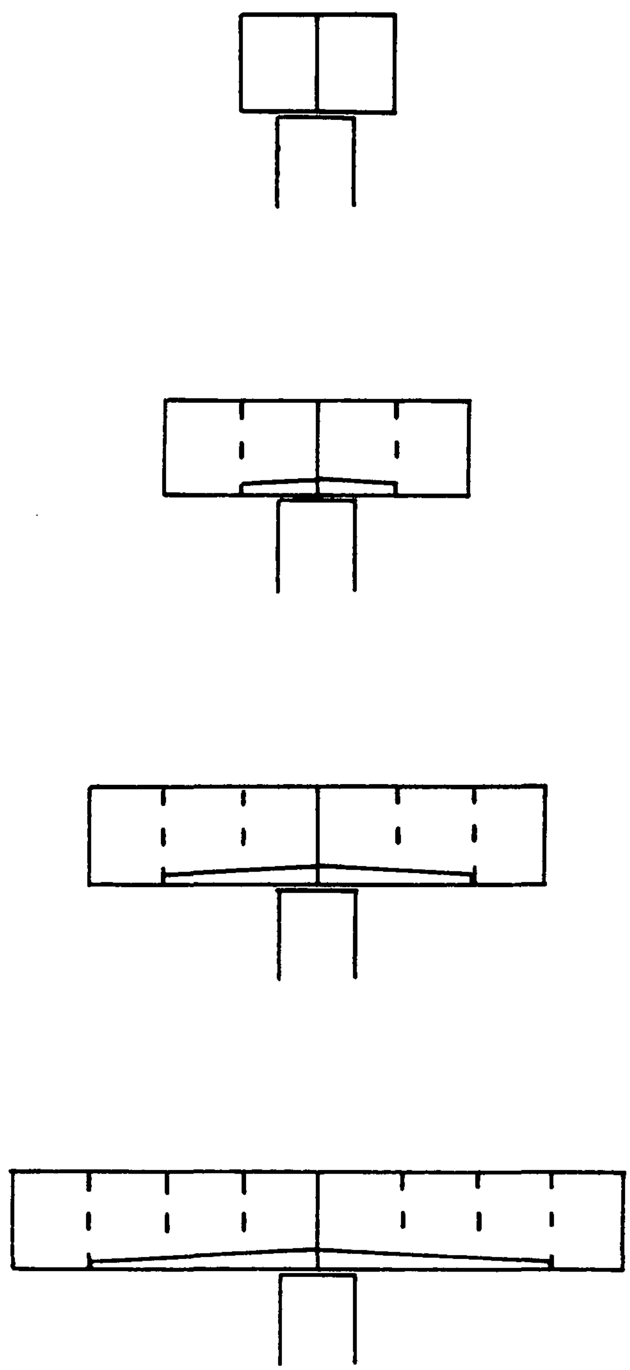


Fig. 3.1 Suggested Erection Sequence

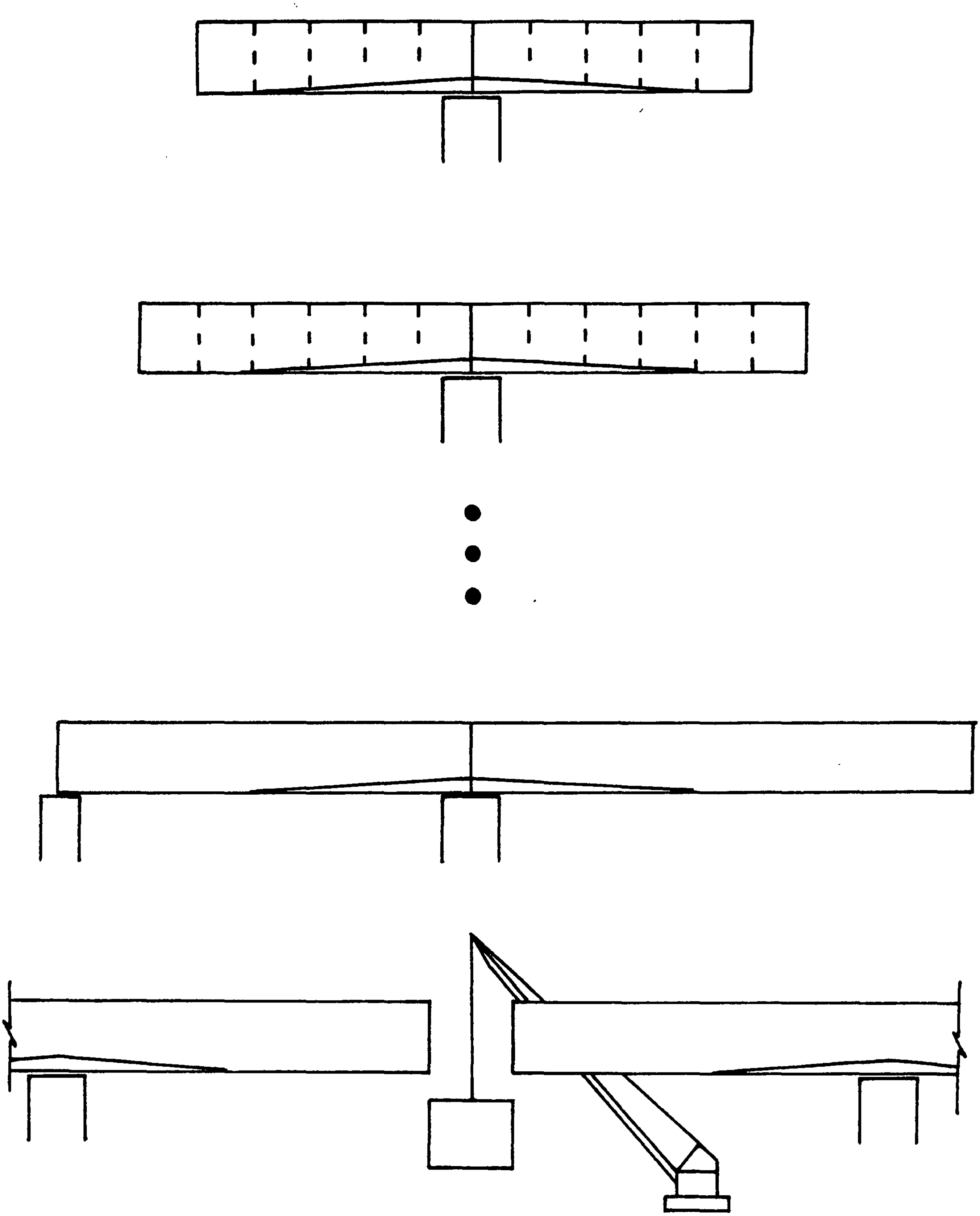


Fig. 3.1 (continued) Suggested Erection Sequence

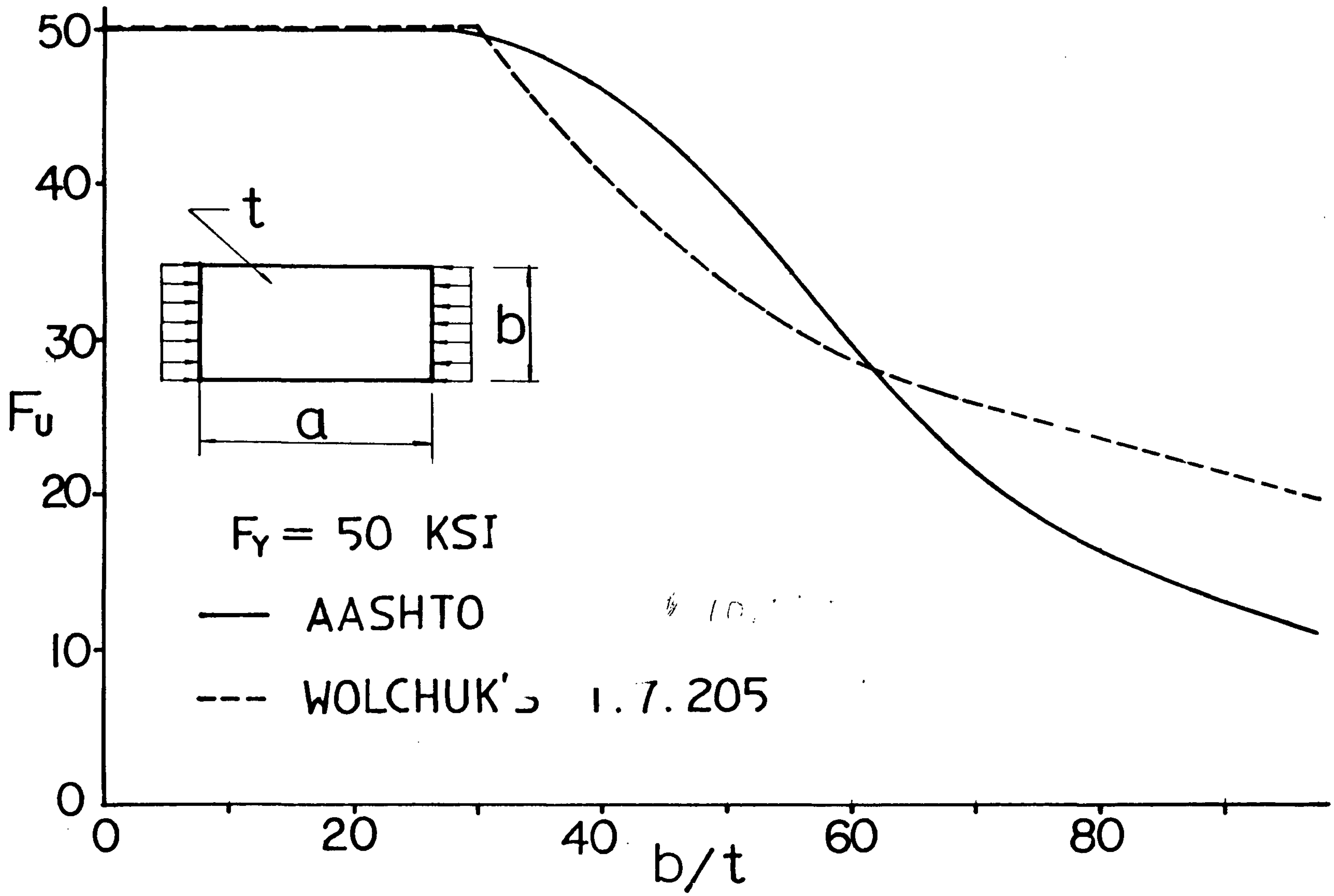


Fig. 3.2 Comparison of Steel Plate Buckling Strength

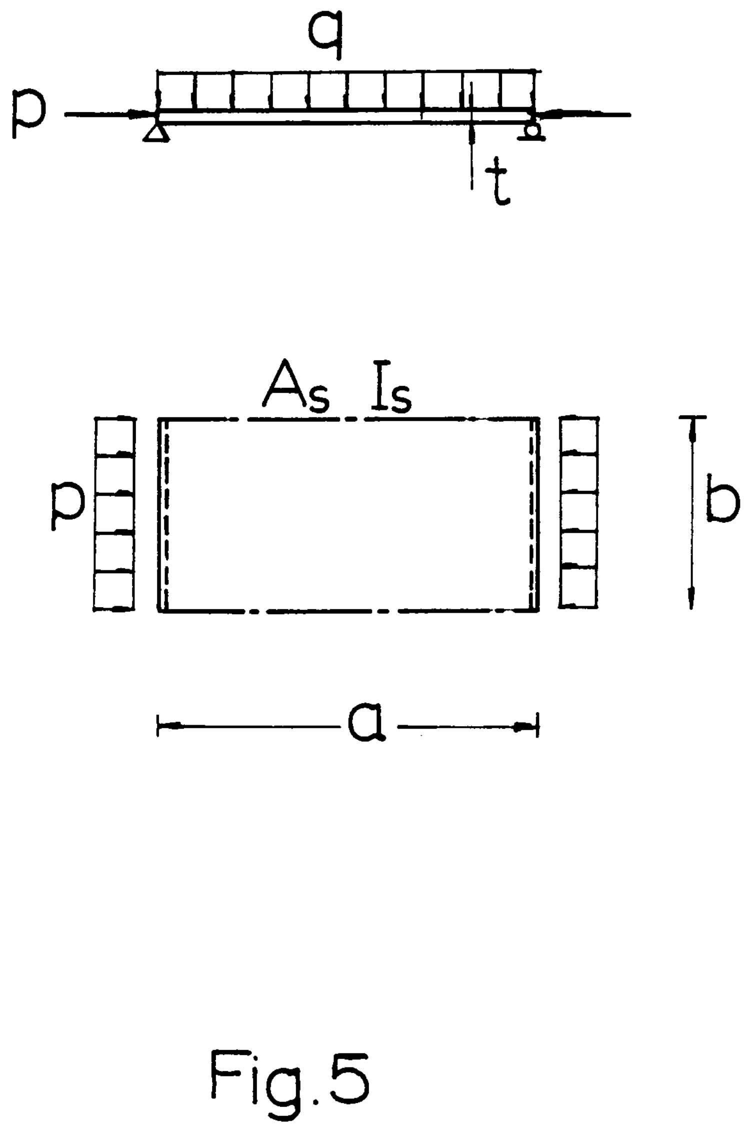
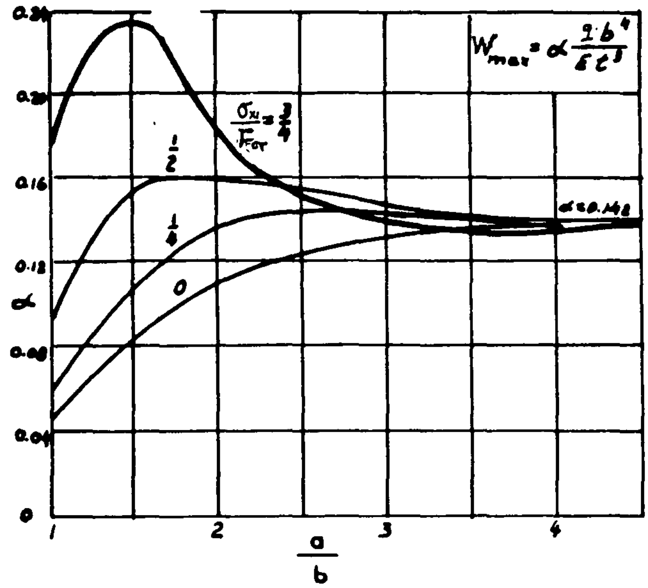
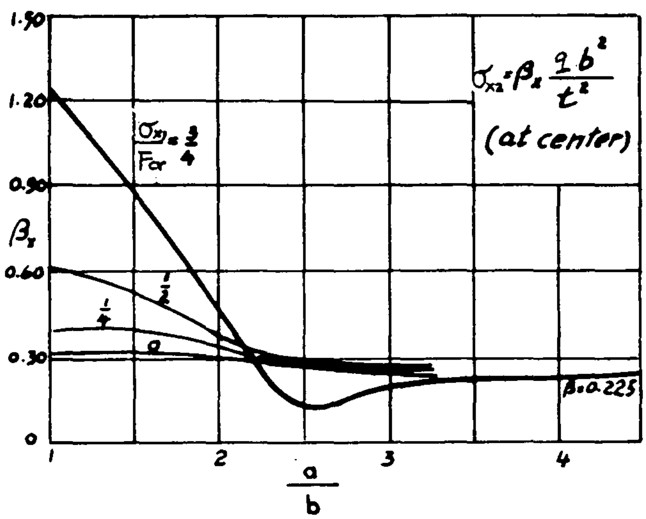


Fig. 3.3 Loads on Steel Compression Plate

Curves from Reference 3.3





$$F_{cr} = \frac{K \pi^2 E}{12(1-\nu^2)(b/t)^2}$$

$$\sigma_{x\,max} = \sigma_{x1} + \sigma_{x2}$$

$$\sigma_{y\,max} = \sigma_{y2}$$

σ_{x1} = applied in plane stress

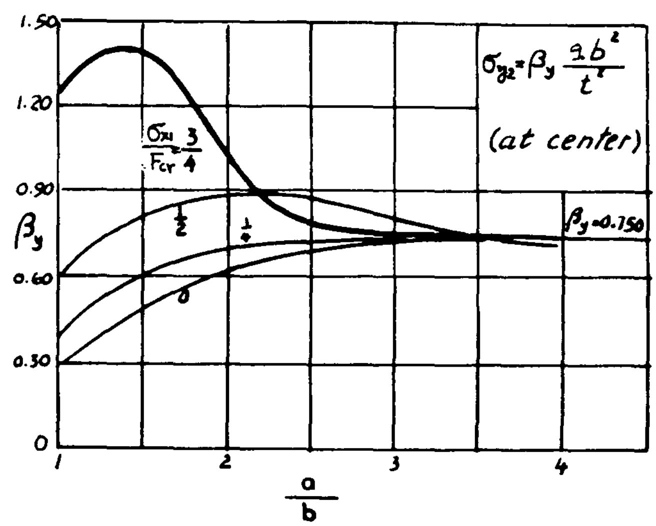


Fig. 3.4 Coefficients for Plate Analysis

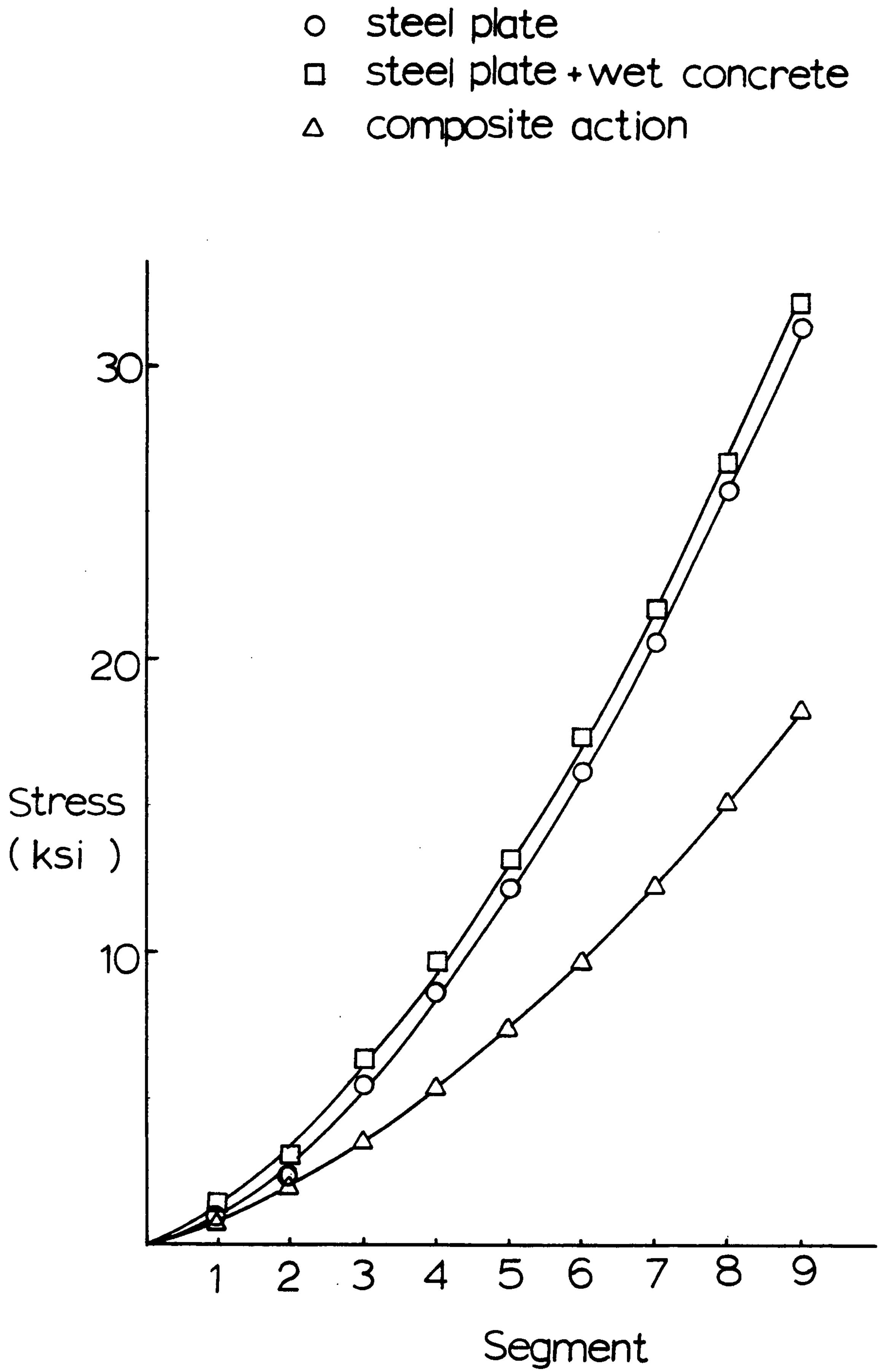
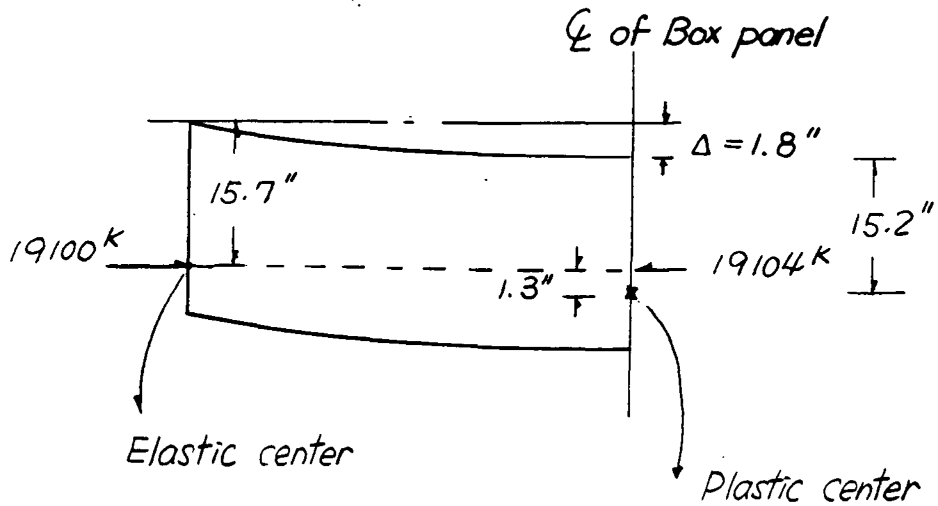
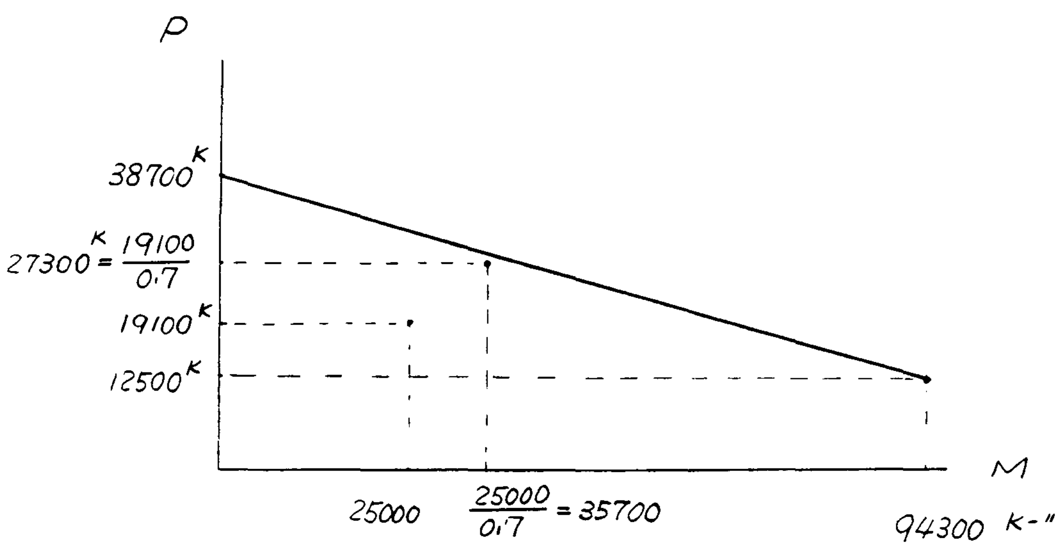


Fig. 3.5 Bottom Flange Stresses During Construction

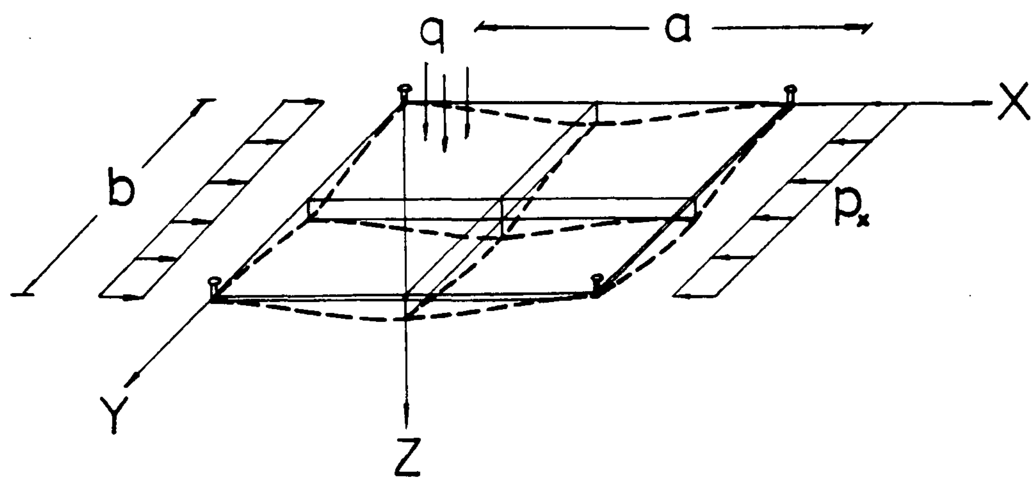


Interaction Curve



* apply 0.7 for uncertainty

Fig. 3.6 Force-Moment Interaction Diagram of Bottom Flange Composite Beam



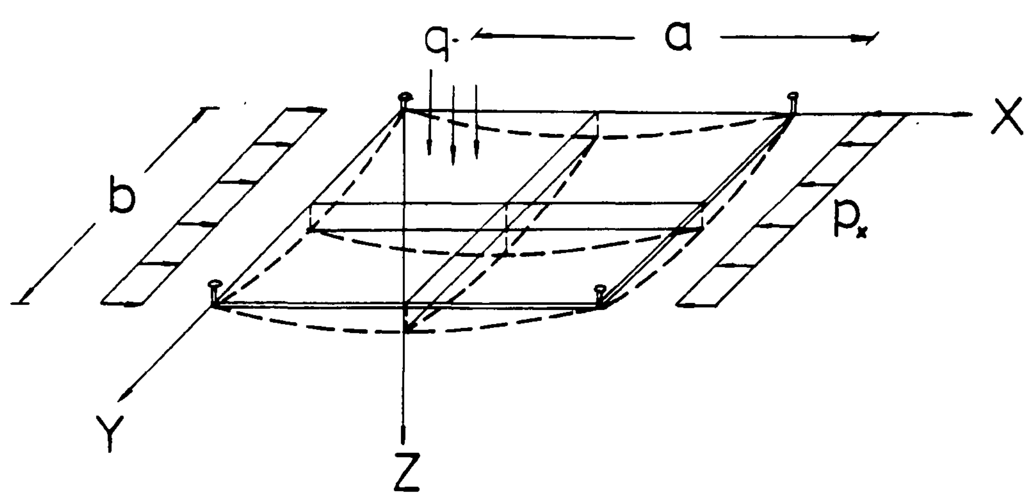


Fig. 4.1 Model of Steel Plate Buckling Between Anchor Points

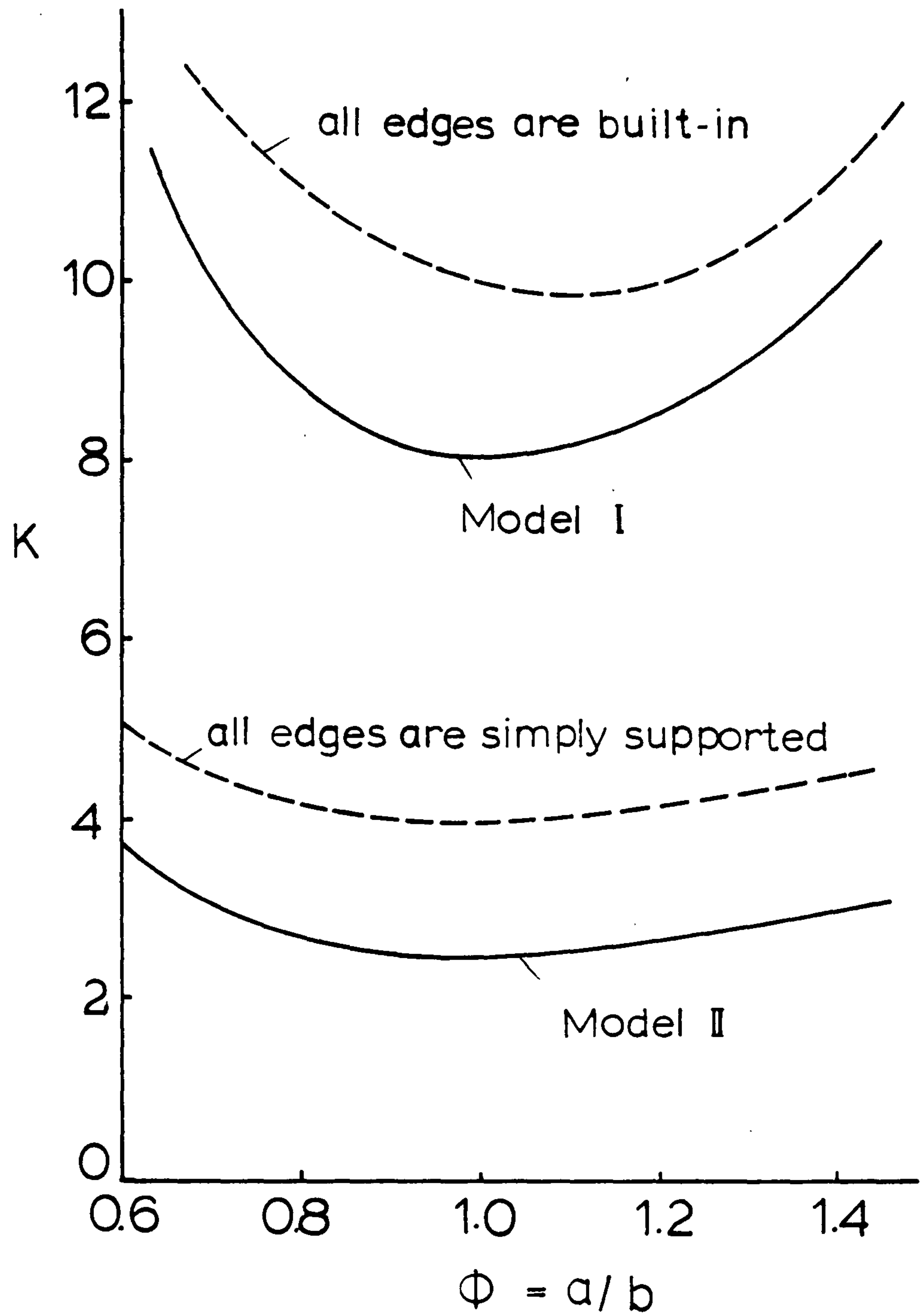
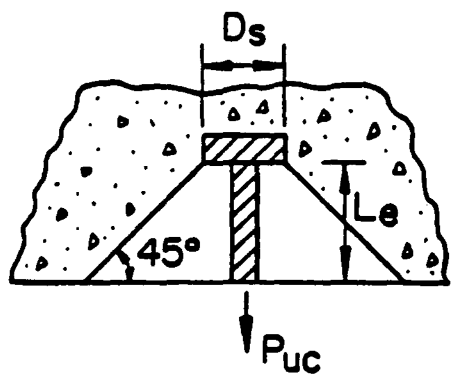


Fig. 4.2 Buckling Coefficients



(a) Full Shear Cone

Fig. 4.3 Shear Cone

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