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Clyde W. Ellis

Richard Sause

ATLSS

LEHIGH University

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Clyde W. Ellis

Richard Sause

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ATLSS is a National Center for Engineering Research on Advanced Technology for Large Structural Systems

117 ATLSS Drive
Bethlehem, PA 18015-4729

Phone: (610)758-3525
Fax: (610)758-5553

www.lehigh.edu/~inatl/inatl.html
Email: inatl@lehigh.edu

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The findings and conclusions presented in this report are those of the authors, and do not necessarily reflect the views of the sponsors.

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ABSTRACT

Recent developments in steel production practices in the United States have resulted in new high performance steels (HPS) for use in bridges. With the development of HPS comes a need for research to enable bridge engineers to take advantage of these steels. Composite steel I-girder bridges are the most common type of steel highway bridge. Therefore, the use of high performance steel in these bridges is of significant practical interest.

This research investigates the influence of certain design parameters, including the number and arrangement of the cross frames, the use of flange rotational restraint braces (FRRBs) with the cross frames, the use of intermediate transverse stiffeners, and the connection details of these stiffeners, on the weight of composite HPS I-girders for bridges.

A series of design studies were conducted. The studies focused on a prototype single-span simply-supported composite HPS I-girder bridge. The design of the I-girders for this bridge considered strength, construction, service, and fatigue limit states. Changes in the weight of the I-girders with changes in the design parameters, and the associated fabrication effort, were determined from the design studies. The results show the possible trade-offs between increasing/decreasing girder weight and increasing/decreasing fabrication effort.

Several results are as follows: (1) girder weight increases as the number of cross frames decreases; (2) cross frame arrangements with larger spacing between the end cross frames and the first intermediate cross frame lead to lighter weight girders while reducing the number of cross frames; (3) the need for intermediate transverse stiffeners can be eliminated with increases in web thickness and girder weight; upgraded fatigue details (Category C′ to Category B) allow decreases in girder weight; (4) girders with fewer cross frames and FRRBs weigh nearly the same as girders with more cross frames and no FRRBs.

Rational design of composite HPS I-girders involves careful consideration of the trade-offs between reductions in I-girder weight and reductions in I-girder fabrication effort. Variations in the arrangement of cross frames, the use of FRRBs, the use of intermediate transverse stiffeners, and the connection details of these stiffeners influence both the weight of steel I-girder bridges and the effort required to fabricate them.

CHAPTER 1

Introduction

Recent research as well as recent developments in steel production practices in the United States have resulted in new high performance steels (HPS) for infrastructure applications (Krpuse et al. 1998). For example, ASTM A709 HPS 70W steel is a recently developed high performance steel, with a nominal yield stress of 70 ksi (485 MPa), intended for use in bridges. With the development of these new high performance steels comes a need for research to enable bridge engineers to take advantage of the properties of these steels. The research reported herein addresses this need.

The properties of high performance steel include good fracture toughness, weldability, and weathering characteristics, in addition to strength greater than the steel used in current, conventional bridge design practice (i.e., ASTM A709 50W). Composite steel I-girder bridges are the most common type of steel highway bridge. Therefore, the use of high performance steel in these bridges is of significant practical interest.

Homma (1994) and Czaplicki et al. (1996) investigated the use of high performance steel in composite I-girder highway bridges. They found that fatigue design limits and stability design limits are barriers to the full utilization of the strength of high performance steel, and suggested the need for design innovations to overcome these barriers. Czaplicki et al. (1996) investigated one innovation, the composite web, which addresses I-girder web stability limits. Murphy (1997) investigated another design innovation, flange rotational restraint braces (FRRBs), which addresses the stability of the compression flange of composite steel I-girders under construction conditions before the flange is composite with the concrete bridge deck.

The research reported herein focuses on the use of several specific modifications to typical composite steel I-girder bridge design practice to explore the trade-off between reductions in I-girder weight and reductions in I-girder fabrication effort. Certain specific design parameters are investigated, including, the number and arrangement of cross frames, the use of FRRBS in conjunction with the cross frames, the use of intermediate transverse stiffeners, and the connection details of these stiffeners. The influence of changes in these design parameters on the weight of I-girders designed for a typical composite steel I-girder bridge is studied.

1.1 Objectives

The objectives of this research are:

(1) To investigate the influence of certain I-girder bridge design parameters, including the number and arrangement of the cross frames, the use of flange rotational restraint braces (FRRBs) with the cross frames, the use of intermediate transverse stiffeners, and the connection details of these stiffeners, on the weight of composite high performance steel I-girders for bridges; and,

(2) To study the trade-offs between reductions in I-girder weight and reductions in I-girder fabrication effort as decisions about these design parameters are made.

1.2 Scope

The research consists of a series of design studies focused on a prototype single-span simply-supported composite steel I-girder bridge. The bridge has a 131.24 ft (40 m) span with no skew. The I-girders of the bridge are designed using HPS 70W steel according to the AASHTO LRFD bridge design specifications (AASHTO 1998). The design of the I-girders considers strength, construction, service, and fatigue limit states. Changes in the weight of the I-girders with changes in the design parameters, and the associated fabrication effort, are determined from the design studies.

1.3 Thesis Organization

The remaining chapters of the thesis are organized as follows. Chapter 2 reviews background on lateral torsional buckling of bridge I-girders under construction conditions, which is related to the number and arrangement of cross frames and the use of flange rotational restraint braces (FRRBs) with the cross frames. Chapter 2 also reviews research by Murphy (1997) on FRRBs. Chapter 3 reviews additional background needed for the design studies presented in the following chapters, including a description of the prototype bridge, the design criteria, the design and analysis approach, the design parameters, and the practical issues considered in the design studies.

Chapter 4 presents a design study of the influence of the number and arrangement of cross frames on the design of the I-girders of the prototype bridge. Chapter 5 presents a design study that investigates the shear design of the I-girders of the prototype bridge and the influence of web thickness and the use of intermediate transverse stiffeners. Chapter 6 presents a design study of the connection details used for the transverse stiffener/cross frame connection plates and the influence of the fatigue resistance of these connection details on the design of the I-girders of the prototype bridge. Chapter 7 presents a design study that investigates the use of FRRBs on the prototype bridge. Finally, Chapter 8 summarizes the results of these design studies and provides conclusions.

CHAPTER 2

Lateral Torsional Buckling Background

In the positive moment regions of composite steel I-girder bridges, the I-girders are often designed with a top (compression) flange that is smaller than the bottom (tension) flange. After the concrete bridge deck cures, and the girder top flange is made composite with the bridge deck, the top (compression) flange is continuously braced. These conditions eliminate the potential for lateral torsional buckling under positive moment. On the other hand, before the concrete deck cures, the compression flange is braced only by permanent cross frames (or other diaphragms) and by any temporary bracing included for the construction stage. Under these conditions, lateral torsional buckling can occur.

Lateral torsional buckling combines lateral movement of the compression flange with torsion (twisting) of the cross-section. Figure 2.1 shows that the lateral torsional buckling response of an I-girder involves a combination of vertical and lateral deflections. Variables which influence the potential for lateral torsional buckling are the dimensions of the girder compression flange, the distance between locations where the flange is laterally braced, and the compression stress in the flange.

2.1 Lateral Torsional Buckling Capacity of Non-Composite I-Girders

Figure 2.2 shows schematically a plot of the nominal moment resistance versus unbraced length for an I-girder when lateral torsional buckling may control the moment resistance. The unbraced length, L_b, is the distance between locations where the compression flange is braced against lateral deflection. Figure 2.2 shows that I-girders with relatively short unbraced lengths have a nominal moment resistance, M_n, equal to the cross-section yield moment, M_y, as shown by segment AB in the figure. It should be noted that, if the cross-section has flanges that are compact, M_n is equal to the cross-section plastic moment, M_p. On the other hand, if the compression flange or web is slender, M_n is less than M_y. In this discussion, it is assumed that the compression flange and web are not slender, but are non-compact as discussed below. Furthermore, M_y is taken as the maximum nominal moment resistance that should be considered for the construction stage. Figure 2.2 shows that the nominal moment resistance for I-girders with long unbraced lengths is controlled by elastic lateral torsional buckling as shown by segment CD. The moment resistance in this region ranges from one-half M_y to a much smaller value. Inelastic lateral torsional buckling controls I-girders in segment BC.

The AASHTO LRFD bridge design specifications (AASHTO 1998) include equations to calculate the nominal moment resistance for non-compact, non-composite bridge I-girders in Article 6.10.10.2. A typical non-composite steel I-girder is shown in Figure 2.3. The nominal moment resistance of the I-girder depends on the web slenderness, compression flange slenderness, and compression flange bracing. Web slenderness requirements for a non-compact I-girder are addressed in Article 6.10.10.2.2. The compression flange slenderness requirements for a non-compact I-girder are addressed in Article 6.10.10.2.3. The compression flange bracing requirements are addressed in Article 6.10.10.2.4. If these three requirements are satisfied, the nominal moment resistance, M_n, is as follows:

$$M_n = R_b \cdot R_h \cdot M_y \qquad \text{(Eq. 2.1)}$$

where, R_b and R_h are flange stress reduction factors defined in Article 6.10.5.4. Equation 2.1 provides the moment resistance for I-girders that fall in segment AB of Figure 2.2.

If the compression flange bracing requirements are not satisfied, the nominal moment resistance is reduced by lateral torsional buckling. Article 6.10.6.4 addresses the lateral torsional buckling resistance of girders that do not satisfy the compression flange bracing requirements. These girders are controlled by either elastic or inelastic lateral torsional buckling corresponding to segments CD or BC, respectively in Figure 2.2. The calculated lateral torsional buckling resistance depends on the depth of web in compression. Girders which satisfy the following web slenderness limit:

$$\frac{2 \cdot D_c}{t_{web}} \leq \lambda_b \cdot \sqrt{\frac{E}{f_c}} \qquad \text{(Eq. 2.2)}$$

are considered to have stocky webs and the St. Venant torsional constant, J, is included in the calculation of the lateral torsional buckling capacity. In Equation 2.2, D_c is the depth of web in compression, t_{web} is the web thickness, E is the modulus of elasticity, f_c is the stress in the compression flange, and λ_b is a coefficient related to the boundary conditions provided to the web by the flanges. The nominal moment resistance, M_n, is as follows:

$$M_n = 3.14 \cdot E \cdot C_b \cdot R_h \cdot \left(\frac{I_{yc}}{L_b}\right) \cdot \sqrt{0.772 \cdot \left(\frac{J}{I_{yc}}\right) + 9.87 \cdot \left(\frac{d}{L_b}\right)^2} \leq R_h \cdot M_y \qquad \text{(Eq. 2.3)}$$

where, I_{yc} is the moment of inertia of the compression flange about the vertical axis, d is the depth of steel section, and C_b is a moment gradient correction factor. The commentary to the AASHTO LRFD bridge design specifications (AASHTO 1998) and Equation 2.3 indicate that the AASHTO LRFD specifications do not consider inelastic lateral torsional buckling for girders with stocky webs.

Girders that do not satisfy Equation 2.2 are considered to have slender webs, and both elastic and inelastic buckling are considered. In addition, the St. Venant torsional constant is taken as zero. To determine whether elastic or inelastic buckling controls, two unbraced lengths must be calculated, namely the maximum unbraced length to reach the yield moment, L_p, and the unbraced length at the transition between elastic and inelastic buckling, L_r. L_p is the unbraced length corresponding to point B in Figure 2.2 and is calculated as follows:

$$L_p = 1.76 \cdot r' \cdot \sqrt{\frac{E}{F_{yc}}} \qquad \text{(Eq. 2.4)}$$

where, r' is the radius of gyration of the compression flange about the vertical axis and F_{yc} is the yield stress of the compression flange. The transition unbraced length, L_r, is calculated as follows:

$$L_r = \sqrt{\frac{19.71 \cdot I_{yc} \cdot d \cdot E}{S_x \cdot F_{yc}}} \qquad \text{(Eq. 2.5)}$$

where, S_x is the section modulus of the girder about the horizontal axis. For girders with slender webs, and with L_b between L_p and L_r, the nominal moment resistance is calculated as follows:

$$M_n = C_b \cdot R_b \cdot R_h \cdot M_y \cdot \left(1 - 0.5\left(\frac{L_b - L_p}{L_r - L_p}\right)\right) \leq R_b \cdot R_h \cdot M_y \qquad \text{(Eq 2.6)}$$

When the unbraced length, L_b, exceeds L_r the girder is controlled by elastic torsional buckling and the nominal moment resistance is calculated as follows:

$$M_n = C_b \cdot R_b \cdot R_h \cdot \frac{M_y}{2} \cdot \left(\frac{L_r}{L_b}\right)^2 \leq R_b \cdot R_h \cdot M_y \qquad \text{(Eq. 2.7)}$$

It should be noted that according to the commentary of Article 6.10.10.6.4.1 in the AASHTO LFRD specifications, when the St. Venant torsional resistance is neglected (i.e., J=0) or girders with stocky webs, the results from Equation 2.3 are identical to the results from Equation 2.7

2.2 Flange Rotational Restraint Braces

Flange rotational restraint braces (FRRBs) can be used in conjunction with standard cross frames to provide an increase in the lateral torsional buckling resistance of an I-girder for a given cross frame spacing (unbraced length), or to increase the cross frame spacing for a given required moment resistance (Murphy 1997). An FRRB is a single plate or T-section beam spanning between the compression flanges of two adjacent girders as shown in Figure 2.4. The FRRB is bolted or welded to the compression flanges with a rigid connection. Figure 2.4 shows the position of a FRRB with respect to the two adjacent girders and standard cross frame. Figure 2.5 shows an isometric view of FRRBs along the length of the girders.

2.2.1 FRRB Advantages

The use of FRRBs provides several advantages. The primary advantage is the possibility of increasing the cross frame spacing without decreasing the lateral torsional buckling moment resistance of the I-girders in a bridge (Murphy 1997). Increasing the cross frame spacing results in fewer cross frames. This may result in a substantial reduction in cross frame fabrication and erection costs. In addition, the connection plates between the I-girders and the cross frames are usually designed with Category C′ fatigue details. Reducing the number of cross frames may allow the connection plates to be moved to regions of lower live load stress, and, therefore, fatigue design limits may be less critical in the design of the girders.

2.2.2 FRRB Behavior

When a FRRB is rigidly connected to the top (compression) flange of an I-girder, it acts as a rotational spring restraining the in-plane rotation of the compression flange at the location of the FRRB. The rotational restraint of an FRRB results in conditions less prone to lateral torsional buckling. Figure 2.6 shows the lateral deflected shape of the compression flanges of adjacent I-girders braced by conventional cross frames during lateral torsional buckling. Note the rotation (slope) of the compression flange at the cross frame (brace) location. When FRRBs are used at these locations, the rotation of the top flange at the brace point is restrained and a higher buckling load is reached. Figure 2.7 shows the influence of the FRRBs. Figure 2.7(a) shows the lateral deflected shape without FRRBs. Inflection points in the deflected shape occur only at the brace points. Figure 2.7(b) shows the lateral deflected shape with FRRBs. The rotation of the compression flange occurring at the braced points is reduced. In addition, inflection points (curvature reversals) are located at the brace point and at locations away from the brace point. The distance between these additional inflection points is the effective unbraced length, $L_{b,eff}$.

The interaction of the FRRBs and the I-girder compression flanges closely resembles the interaction of the beams and the columns in a frame with rigid connections. The compression flange behavior is similar to that of a column in compression and the FRRB behavior is similar to that of a beam when it restrains buckling of the column (Murphy 1997). The rotational restraint provided by the FRRB reduces the unbraced length to an effective unbraced length, $L_{b,eff}$, shown in Figure 2.7(b). This effect of the FRRBs can be quantified by an effective length factor K, where $L_{b,eff} = K \cdot L_b$ (Murphy 1997).

2.3 Calculation of K Factors for Compression Flanges With FRRBs

2.3.1 Finite Difference Method

The following two coupled differential equations describe the equilibrium of an I-girder in a lateral torsional deflected position (Murphy 1997):

$$E \cdot I_y \cdot u^{iv} + M_x \cdot \phi'' + 2 \cdot M'_x \cdot \phi' = 0 \qquad \text{(Eq. 2.8)}$$

$$E \cdot I_w \cdot \phi^{iv} - (G \cdot J + M_x \cdot \beta_x)\phi'' - M'_x \cdot \beta_x \cdot \phi' + M_x \cdot u'' = 0 \qquad \text{(Eq. 2.9)}$$

where, I_y is the moment of inertia with respect to the y (vertical) axis, M_x is the applied moment about the x axis, I_w is the warping moment of inertia, G is the shear modulus, β_x is a coefficient related to symmetry about the x axis, and M'_x is the gradient of the applied moment. Equations 2.8 and 2.9 do not apply when the I-girder is prone to inelastic lateral torsional buckling.

For the boundary conditions provided by conventional bracing, closed-form solutions to Equations 2.8 and 2.9 have been developed to provide the lateral torsional buckling moment (Salmon and Johnson 1971). The boundary conditions provided by FRRBs make a numerical solution to these differential equations more appealing. Murphy (1997) used the finite difference method to develop approximate solutions to these differential equations.

Murphy (1997) used finite difference method solutions to Equations 2.8 and 2.9 to estimate the effective length factor, K, for a bridge girder braced by cross frames with FRRBs. A two step approach was used. First, the girder is analyzed without FRRBs. The lateral torsional buckling resistance is calculated as the unbraced length is varied using the finite difference method. Figure 2.8 shows that the results of the finite difference method are in good agreement with the AASHTO LRFD equations (Equation 2.7) and the classical closed-form solution to Equations 2.8 and 2.9, thus validating the finite difference method solution. Then, the lateral torsional buckling resistance for the girder with FRRBs is calculated using the finite difference method. Comparison of the lateral torsional buckling resistance for the girder without FRRBs to the girder with FRRBs, as shown in Figure 2.9, allows the effective length factor, K, to be determined. The results in Figure 2.9 are for the maximum moment region of the girder with FRRBs at the cross frames at both of the unbraced length. The K factor is calculated by relating the effective unbraced length (i.e., the unbraced length without FRRBs providing the same lateral torsional buckling moment resistance) to the unbraced length with FRRBs as follows:

$$K = \frac{L_{b,eff}}{L_b^{\ FRRB}} \tag{Eq. 2.10}$$

K factors as low as 0.66 to 0.68 can be achieved. Murphy (1997) provides detailed FRRB and girder cross-section properties supporting the K values, shown in Figure 2.9.

As stated earlier, Equations 2.8 and 2.9 do not consider inelastic lateral torsional buckling. The AASHTO LRFD bridge specifications (AASHTO, 1998) provide approximate results (Equation 2.6) for girders with slender webs which have unbraced lengths between L_p and L_r for which inelastic lateral torsional buckling governs. For a girder with FRRBs and with an effective unbraced length that falls between L_p and L_r, the K factor for elastic buckling is used within Equation 2.6 to determine the inelastic lateral torsional buckling capacity.

2.3.2 Alignment Chart Method

An alignment chart is a simpler way to determine the K factor than the finite difference method. The rotational stiffness provided by the FRRB to the compression flange closely resembles the restraint provided by the beams to the columns in a frame with rigid connections. This analogy is reasonable if the connection between the FRRB and compression flange is rigid enough to provide rotational restraint. With a few modifications to the rotational stiffness equations that are used with alignment charts, approximate K factors for the unbraced lengths of I-girders with FRRBs can be determined.

Conventional alignment charts for determining the K factors for the columns in a frame consider two cases; one chart exists for cases when sidesway is allowed and another chart exists for cases when sidesway is restrained. The alignment chart for cases when sidesway is restrained is used in the following discussion. When sidesway is restrained, the K factors range from 0.5 to 1. A K factor of 1 corresponds to conditions where the end rotation of the unbraced length is completely unrestrained. A K factor of 0.5 corresponds to conditions where the end rotation of the unbraced length is completely restrained with the effective length being reduced to half the original unbraced length.

In the conventional application of alignment charts, the rotational stiffness parameters Ψ_1 and Ψ_2 reflect the restraint provided to a column by the rigidly connected beams. For a girder braced with FRRBs, these parameters are calculated at the locations where the compression flange is braced by the FRRBs and cross frames. The rotational stiffness parameters are calculated to reflect the restraint of the FRRB on the compression flange as follows:

$$\Psi = \frac{\sum\left(\frac{E \cdot I_{yc}}{L_b}\right)_{comp.flange}}{\sum\left(\frac{E \cdot I_y^{FRRB}}{L_s}\right)_{FRRB}} \qquad \text{(Eq. 2.11)}$$

where, L_s is the spacing between girders, and I_y^{FRRB} is the moment of inertia of the FRRB flange. The upper summation reflects a possible variation in the dimensions of the compression flange between the two unbraced lengths on either side of the location of the FRRB. The lower summation reflects the possibility that two FRRBs may be attached to the interior girder of a bridge, one associated with each cross frame at the cross frame location. Equation 2.11 can be simplified if only one FRRB is attached to the girder and if the moment of inertia of the compression flange is constant. The modulus of elasticity can be omitted if it is the same for both the flange and the FRRB.
Figure 2.10 shows a pair of adjacent girders with four cross frames and three unbraced lengths L_{b1}, L_{b2} and L_{b1}. As shown in the figure, FRRBs are located at B and C. The simplified expression for the rotational stiffness parameter, Ψ_1, at a location with an FRRB (e.g., location B) is as follows:

$$\Psi_1 = \frac{I_{yc} \cdot L_s}{I_y^{FRRB}} \cdot \left(\frac{1}{L_{b1}} + \frac{1}{L_{b2}}\right) \qquad \text{(Eq. 2.12)}$$

At location A, where an FRRB is not located, a conventional cross frame is assumed to provide very little rotational restraint and Ψ_2 is assumed equal to zero. With Ψ_1 and Ψ_2 determined, Figure 2.11 can be used to determine K.

To determine the rotational stiffness or K factor more accurately, Equation 2.13 can be used in place of the alignment chart (Salmon and Johnson 1971).

$$\frac{\Psi_1 \cdot \Psi_2}{4} \cdot \left(\frac{\pi^2}{K^2}\right) + \left(\frac{\Psi_1 + \Psi_2}{2}\right) \cdot \left(1 - \frac{\pi / K}{\tan \pi / K}\right) + \frac{2}{\pi / K} \cdot \tan \frac{\pi}{2 \cdot K} = 1 \qquad \text{(Eq. 2.13)}$$

Given Ψ_1 and Ψ_1, the K factor is obtained by solving Equation 2.13.

Table 2.1 compares K factors and the corresponding nominal moment resistances for two bridge girders studied by Murphy (1997) using the finite difference and alignment chart methods. The K factors from the finite difference method are determined as discussed in Section 2.3.1. The lateral torsional buckling resistances are given for two girders spanning 110 and 165 ft (33.5 and 50 m) with a nominal yield stress of 70ksi (485 MPa). First, lateral torsional buckling resistances were calculated without FRRBs for an unbraced length of 19.23 ft (5.86 m) for the shorter girder and 23.62 ft (7.2 m) for the longer girder. Second, FRRBs were added to the girder designs. The lateral torsional buckling resistance was kept nearly constant while the unbraced lengths (distance between cross frames) was increased. An unbraced length of 29.66 ft (9.04 m) was achieved in the maximum moment region located at midspan of the shorter girder and an unbraced length of 36.10 ft (11 m) was achieved for the longer girder. Table 2.1 shows that the K factors from the finite difference method are slightly larger than the K factors from the alignment chart method. This indicates the alignment chart gives slightly less conservative results than the finite difference method. Murphy (1997) used these K factors with the lateral torsional buckling resistance formulas from the AASHTO LRFD bridge design specifications (AASHTO 1998) to calculate the lateral torsional buckling moment resistance. Again, the alignment chart approach is less conservative resulting in slightly higher estimates of the lateral torsional buckling resistance.

2.4 Summary

The use of FRRBs with conventional cross frames can improve the lateral torsional buckling resistance of the positive moment regions of bridge I-girders during the construction stage. FRRBs can provide advantages such as an increased lateral torsional buckling resistance, an increased cross frame spacing, and a reduced number of cross frames. The rotational restraint provided by an FRRB reduces the effective unbraced length of the top (compression) flange. Effective length factors, K, can be used to account for this effect. K factors can be determined using finite difference method solutions to the differential equations for lateral torsional buckling of I-girders or by using an alignment chart. Table 2.1 shows that the alignment chart method produces less conservative K factors resulting in slightly higher estimates of the lateral torsional buckling moment resistance. The lateral torsional buckling resistance of I-girders with FRRBs can be calculated using the lateral torsional buckling resistance formulas from the AASHTO LRFD bridge design specifications (AASHTO 1998) in conjunction with the appropriate K factor.

Table 2.1 Comparison of alignment chart and finite difference methods by Murphy (1997)

Span ft	My kip-ft	K factor for mid segment		Design Moment kip ft	Lateral torsional buckling moment (kip-ft)		
		Finite Difference	Alignment Chart		Original L_b K = 1	L_b^{FRRB} with K from Finite Difference	L_b^{FRRB} with K from Alignment Chart
110	5850	0.66	0.64	3455	3885	3805	3935
165	11870	0.68	0.66	7690	8320	8015	8400

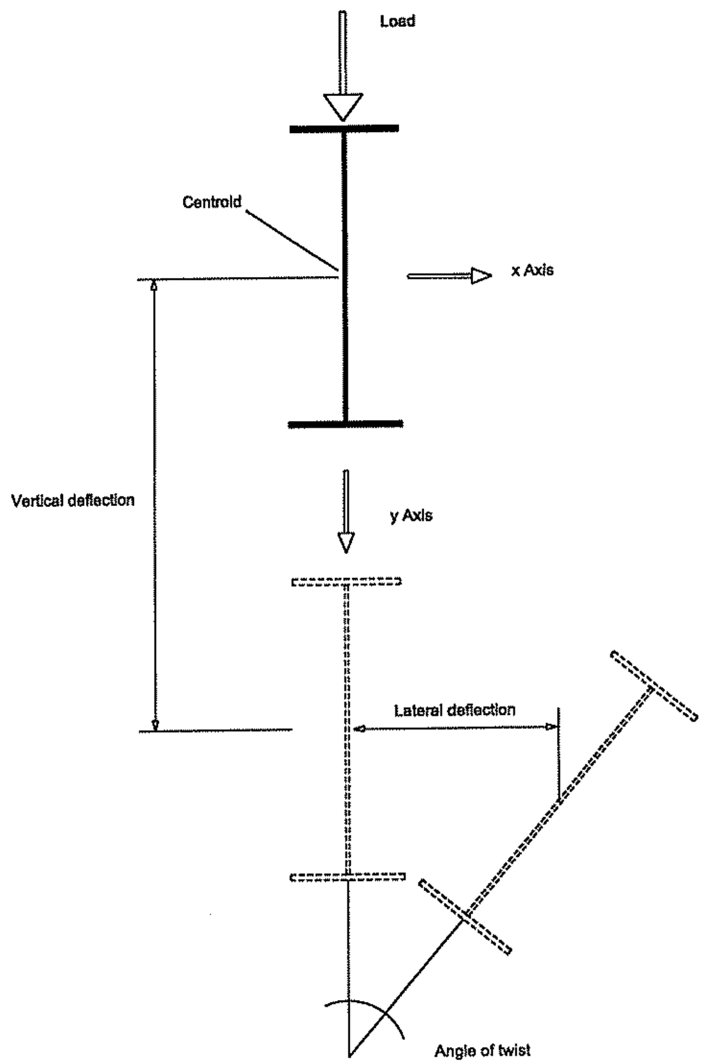


Figure 2.1 Deflections caused by vertical load (Galambos 1968)

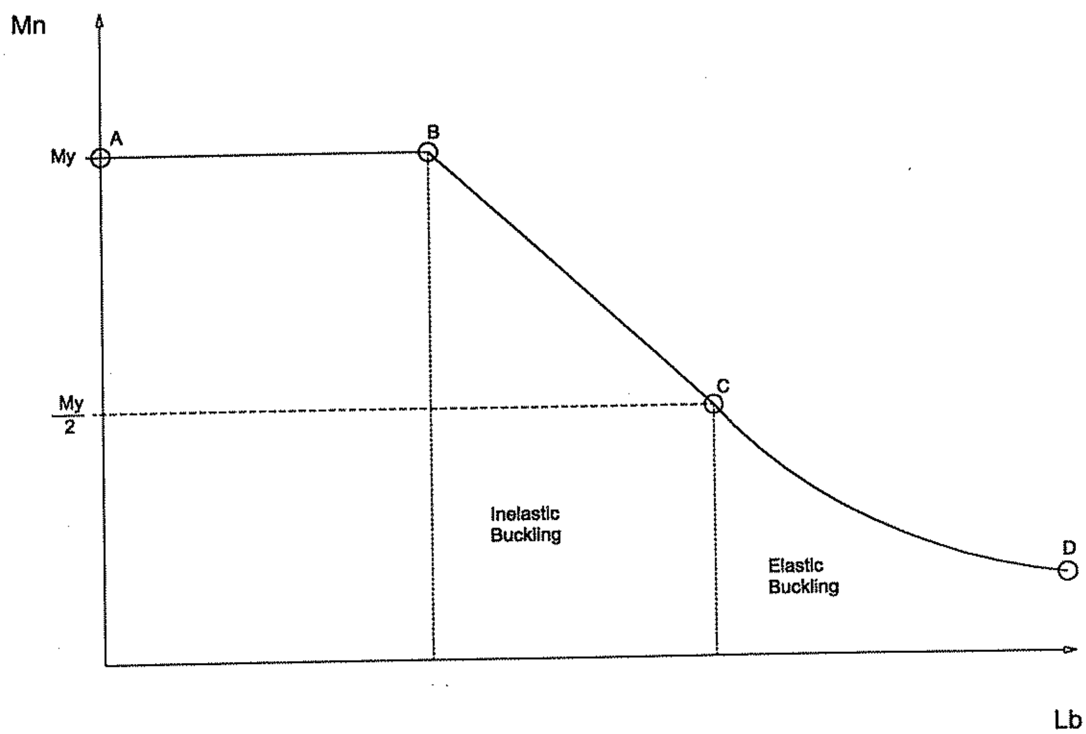


Figure 2.2 Moment resistance versus unbraced length

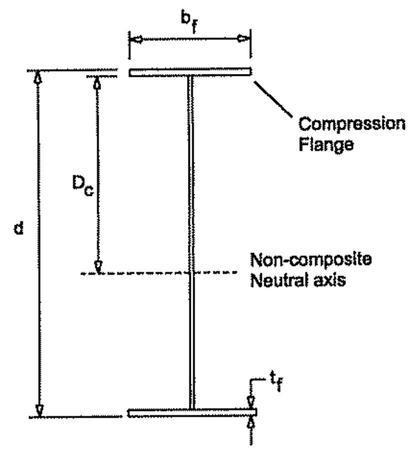


Figure 2.3 Girder cross-section

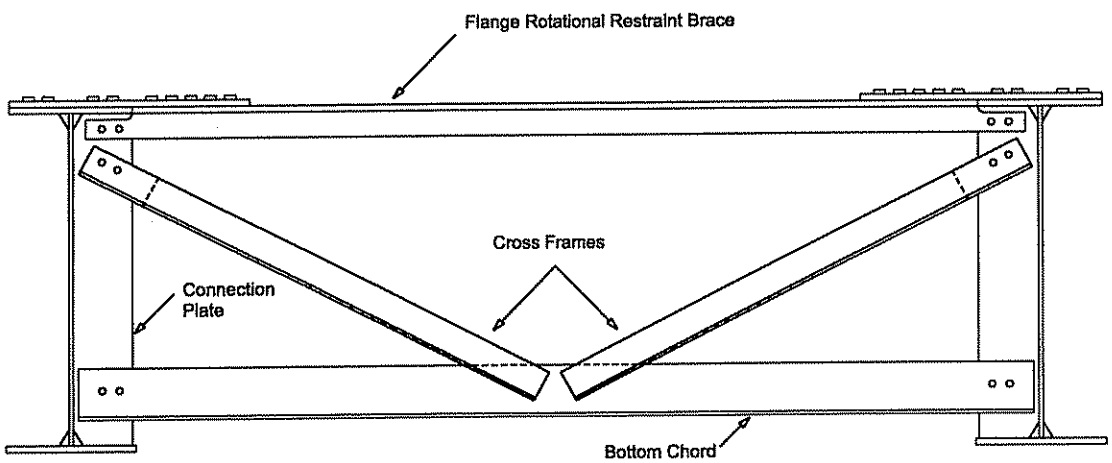


Figure 2.4 Pair of adjacent girders with cross-frame and flange rotational restraint brace

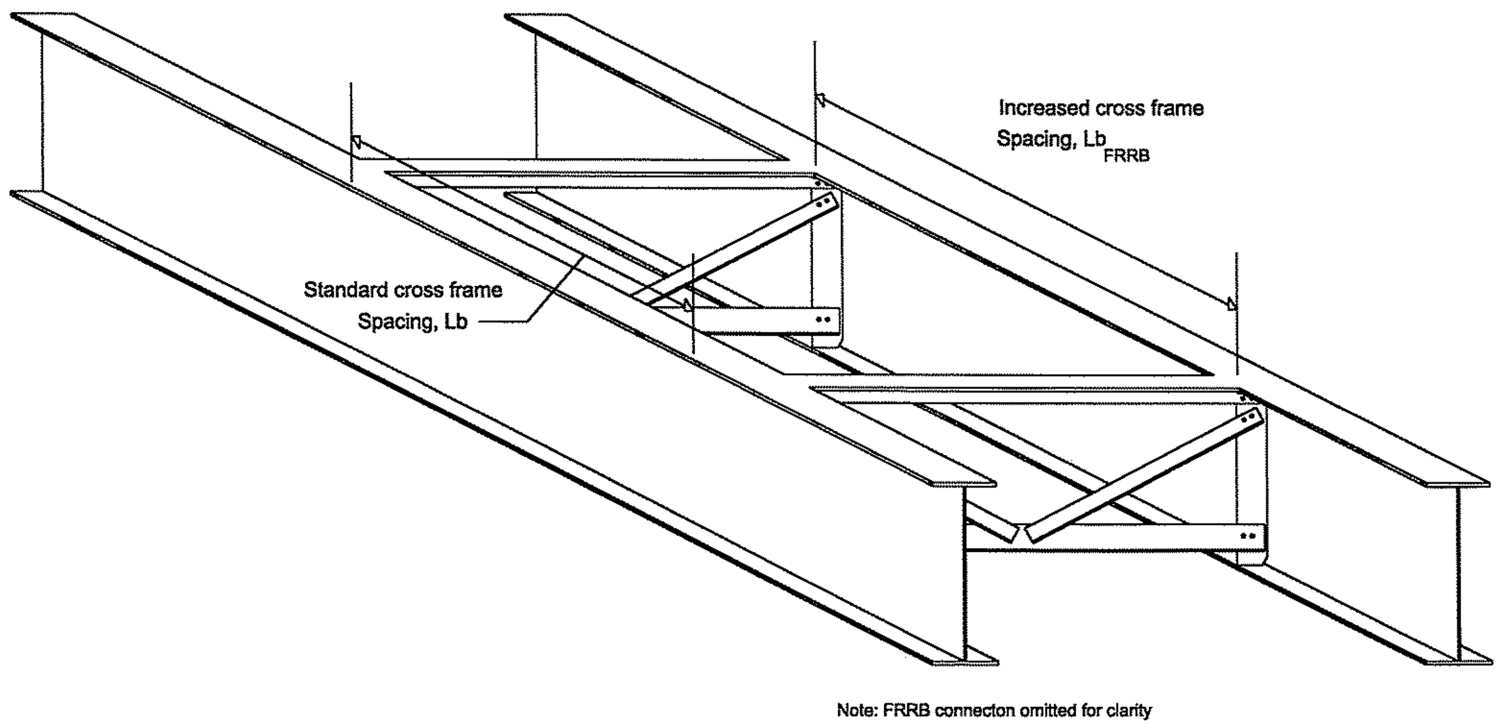


Figure 2.5 Pair of adjacent girders with cross frames and flange rotational restraint braces

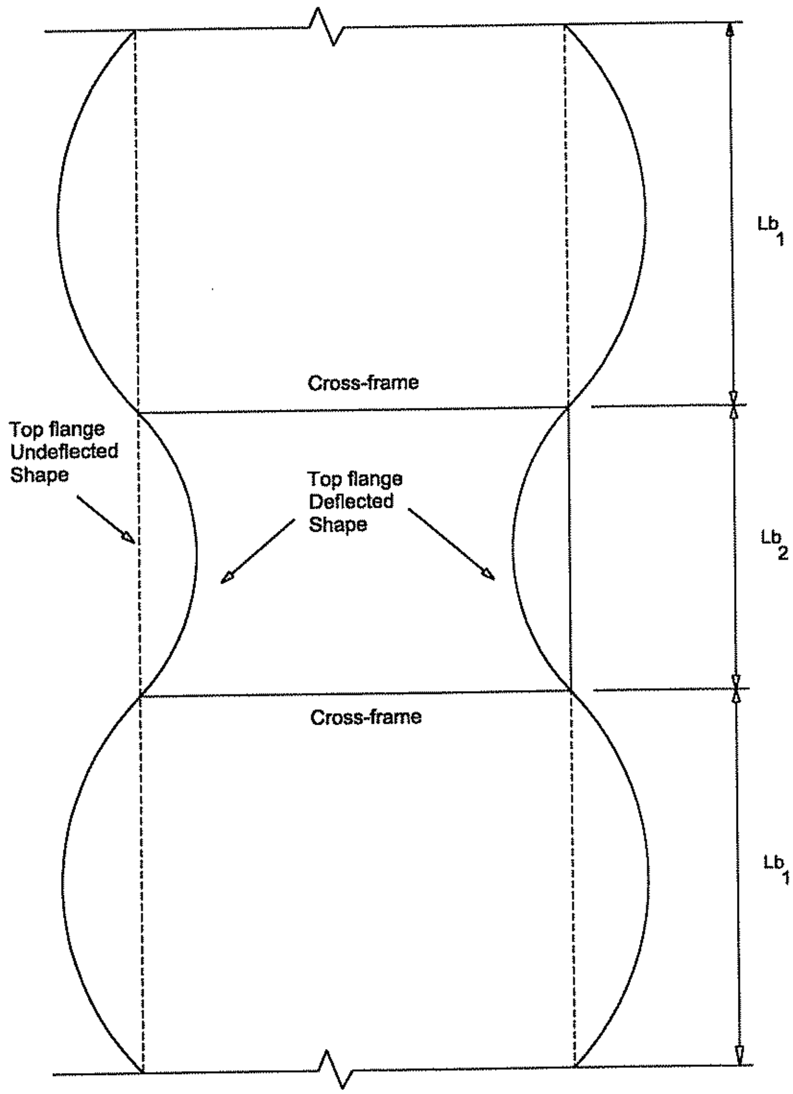


Figure 2.6 Lateral deflected shape of adjacent I-girder compression flanges without FRRBs during lateral torsional buckling

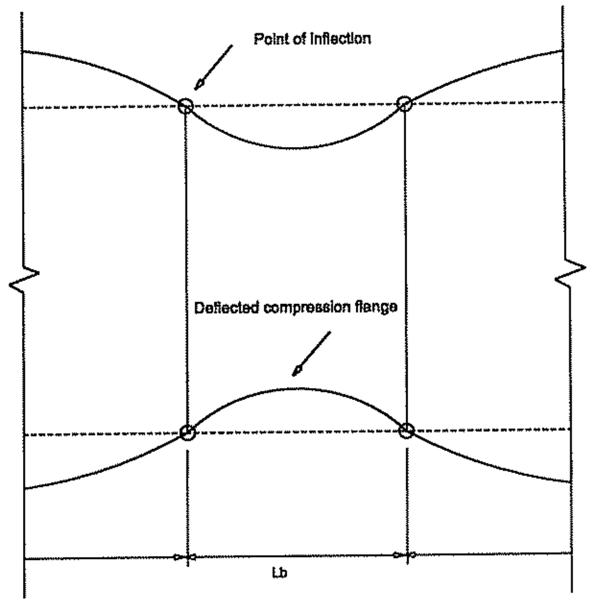


Figure 2.7(a) without FRRBs

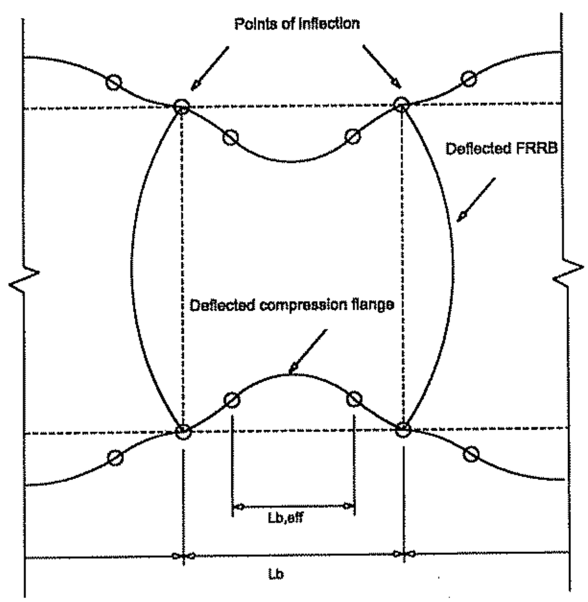


Figure 2.7(b) with FRRBs

Figure 2.7 Lateral deflected shape of compression flanges of adjacent I-girders

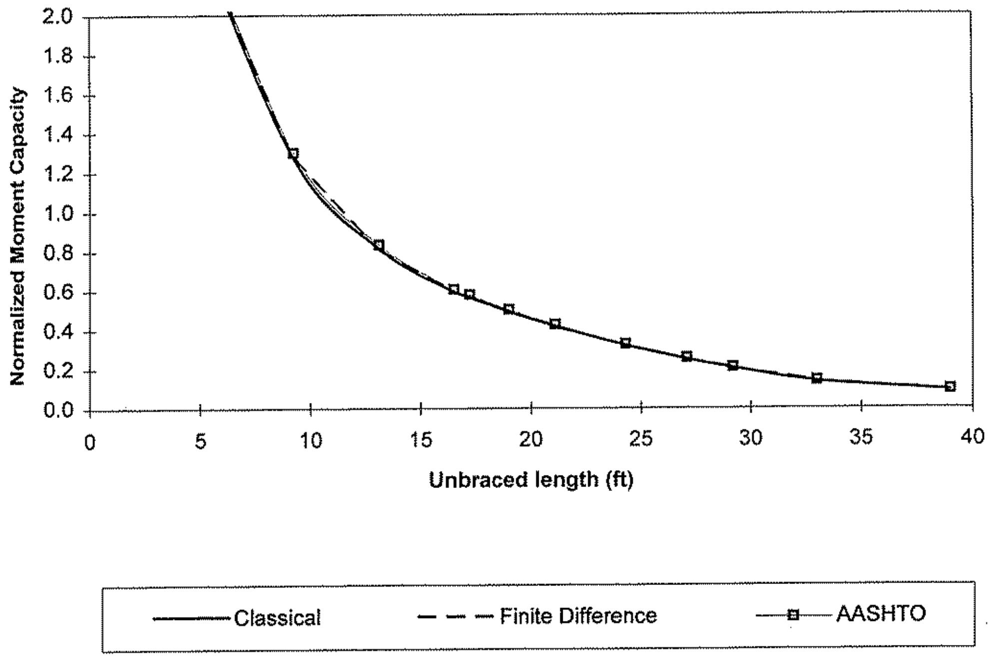


Figure 2.8 Compairson of the classical and finite difference solutions with the AASHTO design equations for a 110 ft span 70 ksi yield strength girder (Murphy 1997)

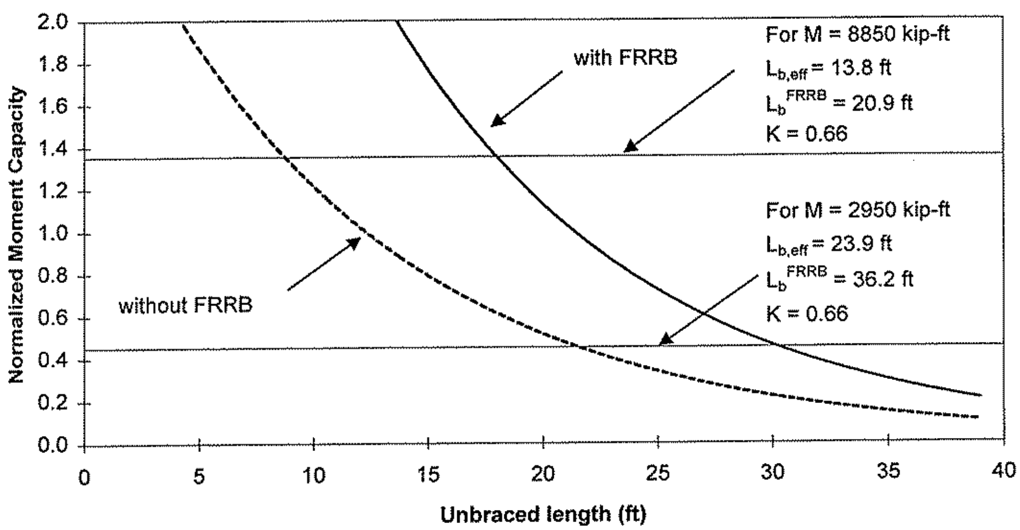


Figure 2.9 Comparison of finite difference solutions for L_b and L_b^{FRRB} to estimate the K factor for a 110 ft 70 ksi yield strength girder (Murphy 1997)

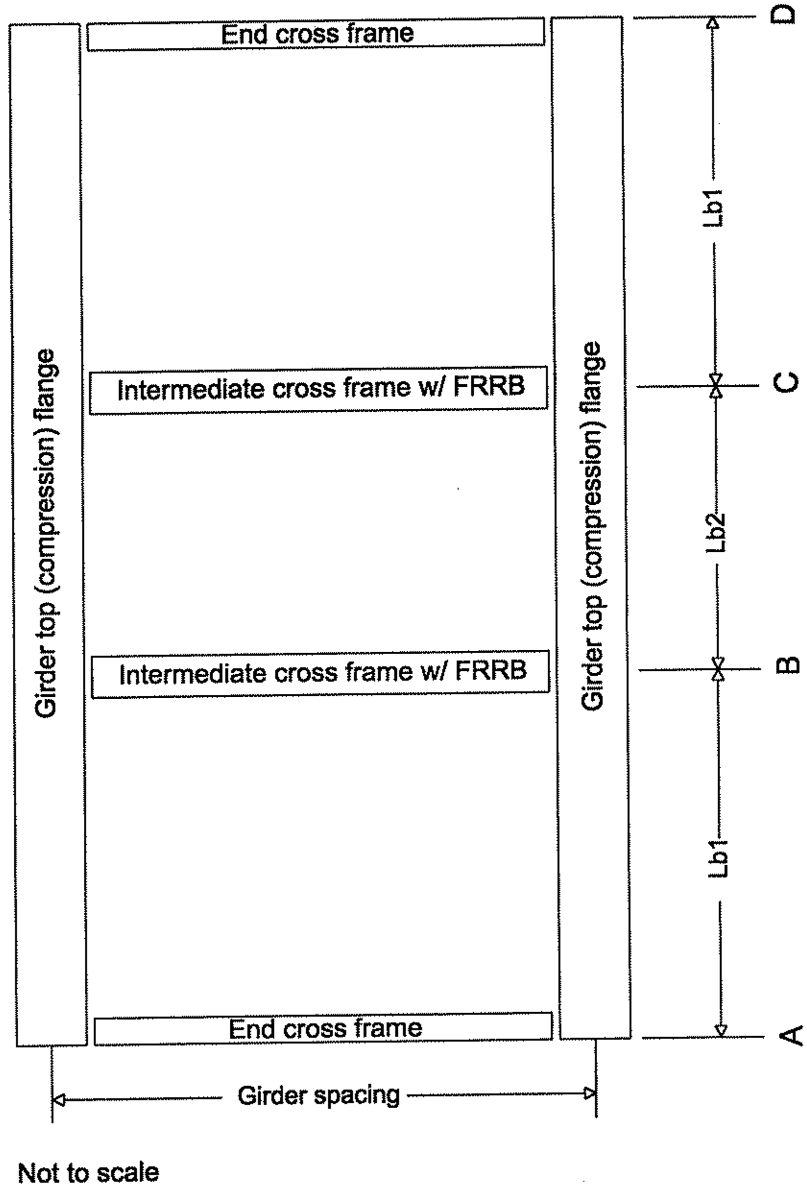


Figure 2.10 Pair of adjacent girders with cross-frames

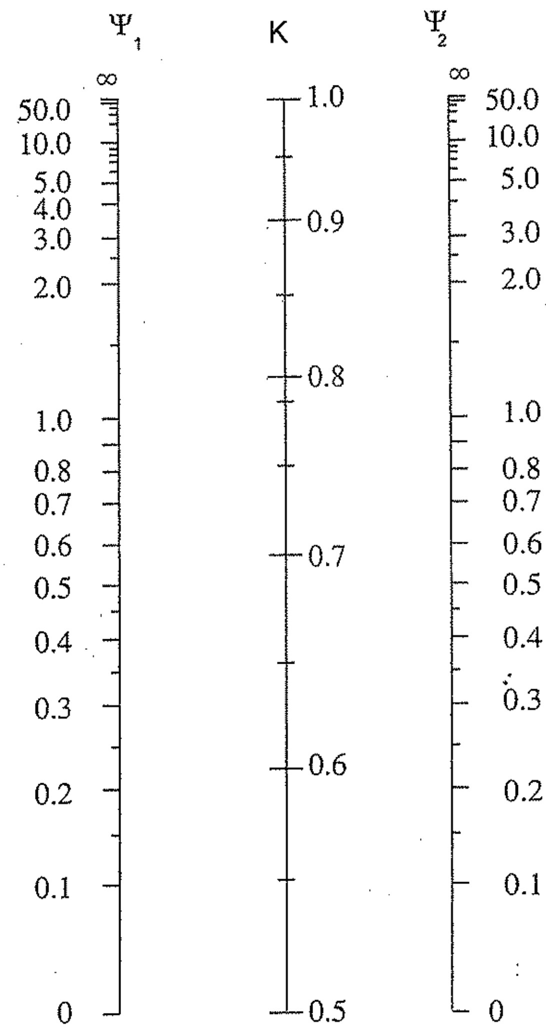


Figure 2.11 Alignment chart with sidesway restrained (Salmon and Johnson 1996)

CHAPTER 3

Background to Bridge Girder Design Study

This chapter presents the background for the design studies presented in the following chapters. These design studies will investigate the influence of cross frame spacing and several related parameters on the weight of bridge I-girders designed for minimum weight. The prototype bridge, the design criteria, the design and analysis approach, the design parameters, and the practical issues considered in the design studies are outlined in this chapter.

The girders designed in the design studies adhere to the AASHTO LRFD bridge design specifications (AASHTO 1998). Cross-section properties were carefully chosen to achieve minimum weight girder designs, considering commercially available plate thicknesses.

The results of the design studies presented in the following chapters are similar to those presented in Figure 3.1, which shows how girder weight is influenced by web depth for bridge girders with web depths ranging from 38 in (965 mm) to 50 in (1270 mm).

3.1 Prototype Bridge

The design studies described in the following chapters focus on a prototype composite steel I-girder bridge. The superstructure of the bridge consists of main and secondary steel components. The main structural steel components, which include the I-girder web and flanges and the FFRB plates, when FRRBs are used, are assumed to be made from ASTM A709 HPS 70W steel which is a high performance, weathering steel with a nominal yield stress of 70 ksi (485 MPa). The secondary steel components, which include stiffeners, connection plates, and cross frames, are assumed to be made from conventional weathering steel (ASTM A709 50W) with a nominal yield stress of 50 ksi (345 MPa).

The prototype bridge is a single span bridge with a simply supported 131.24 ft (40 m) span. The bridge width is 50 ft (15.2 m) including two 8 ft (2438 mm) shoulders and two 12 ft (3657 mm) lanes. The parapet and sidewalk combined are 5 ft (1524 mm) wide. The bridge has no skew. The concrete deck is 10 in (254 mm) thick and is composed of normal strength concrete with a specified minimum compressive strength of 4 ksi (27.6 Mpa). The bridge has four straight I-girders spaced at 12.5 ft (3810 mm) centers with 6.25 ft (1901 mm) deck overhangs. The deck is composite with the I-girders. The typical cross-section of the prototype bridge is shown in Figure 3.2.

3.2 Design and Analysis Approach

The AASHTO LRFD bridge design specifications (AASHTO 1998) present design criteria in the following form:

$$\eta \cdot \sum \gamma_i \cdot Q_i \leq \phi \cdot R_n \qquad \text{(Eq.3.1)}$$

The left side of Equation 3.1 represents the factored load effects to be carried by the structural components and the right side represents the factored resistance, or strength, of the structural components. On the load side of the equation, the sum of various load effects Q_i are multiplied by partial safety (load) factors γ_i to obtain factored design demand. On the resistance side, the nominal resistance R_n is multiplied by a partial safety (resistance) factor ϕ. The factored load effects from loads expected during the construction stage and during the service life of a bridge are not permitted to exceed the factored resistance.

3.2.1 Limit States

The AASHTO LRFD bridge design specifications (AASHTO 1998) consider both the loading conditions of the bridge and the limit states of the bridge structural components of the bridge. Limit states are "those conditions of a structure at which it ceases to fulfill its intended function" (Salmon and Johnson 1996). The AASHTO LRFD bridge design specifications consider four limit state categories: strength, serviceability, fatigue and fracture, and extreme event. Extreme event limit states were not considered in this study.

Strength limit states are considered to ensure that the strength and stability of the bridge enables it to safely resist combinations of loads much larger than those that the bridge can be expected to experience during its service life. These loads may lead to extensive distress and structural damage, but the overall structural integrity is expected to be maintained.

Strength limit states during construction are also considered, to ensure that the bridge can safely carry the dead load of the bridge and the transient loads produced by construction workers and equipment during erection of the bridge and placement of the bridge deck.

Service limit states are considered to ensure acceptable performance of a bridge during its service life. Design for service limit states restricts stresses, deformations, and cracking under service conditions.

Fatigue limit states are considered to prevent premature failure of a member or a connection under the action of repeated live loads during the service life of the bridge. Fracture limit states are considered by satisfying material toughness requirements and fabrication requirements for fracture critical members.

All of the limit states listed above are considered in the design of a bridge. The bridge designer determines the proper load factors, resistance factors, nominal loads, and nominal resistances to ensure that these limit states are not exceeded.

3.2.2 Design Loads

Bridges are subjected to various types of gravity and lateral loads. The gravity loads include the dead weight of the bridge, superstructure and substructure, superimposed dead loads, and live loads. The lateral loads include wind and earthquake loads.

The dead loads considered when designing bridge girders include the weight of the components of the superstructure, appurtenances and utilities, wearing surfaces and future overlays, and planned widenings of the deck. These dead loads are divided into two categories: the dead load of the structural components, D_c, and the dead load of the wearing surfaces and similar items, D_w. D_c consists of the weight of the structure except for the weight of non-integral wearing surfaces, appurtenances, and utilities. D_w consists of the weight of non-integral wearing surfaces, appurtenacnes, and utilities. The dead loads used in the design of the girders of the prototype bridge are shown in Table 3.1 as load per length acting on the girders. Assumptions about the weight of items, including the stay-in-place forms, the parapet, the future wearing surfaces, and so on, are from (Salmon and Johnson 1996). The uniform dead loads D_c and D_w are 2 k/ft (29.2 kN/m) and 0.5 k/ft (7.3 kN/m) respectively.

Live loads acting on highway bridges primarily involve vehicles such as cars, trucks, and military vehicles. The AASHTO LRFD bridge design specifications (AASTHO 1998) provide standard design loads that include a truck load, a lane load and a tandem (military) load.

The truck load is taken from a load classification system that describes the truck type and gross weight in tons. The AASHTO LRFD bridge specifications (AASHTO 1998) specify the use of an HS-20 design truck. However, the prototype bridge is located in Pennsylvania, and the Pennsylvania Department of Transportation (PennDOT 1993) requires that all new bridges, regardless of roadway class or funding source, be designed for an HS-25 truck (i.e., 125 percent of an HS-20 truck). Figure 3.3 shows the HS-25truck load axle loads of 10 kips (44.5 kN) and 40 kips (177.9 kN). Only one truck is assumed to be on the bridge at a time.

The lane load simulates a string of cars in a 10 ft (3048 mm) design lane over the entire bridge span. Figure 3.4 shows the uniform distributed lane load of 0.64 k/ft (9.3kN/m) per lane specified by the AASHTO LRFD bridge design specifications (AASHTO 1998).

Design for the tandem load provides highway bridges with the capacity for certain heavy military vehicles. This loading consists of two axles spaced 4 ft (1220 mm) apart with each axle carrying 30 kips (133.4 kN). This load produces live-load moments in spans less than 40 ft (12.2 m) that are higher than those produced by the truck load.

The truck, lane, and tandem loads are not directly applied to the girders. The concrete deck distributes these loads to the adjacent girders. The AASHTO LRFD bridge design specifications (AASHTO 1998) provide approximate methods of analysis to determine the distribution of live loads to the individual girders. The distribution factors vary with the type of deck and girders used in the bridge, the number of design lanes, the number of lanes loaded, and whether the girder is an exterior or interior girder. The design study focuses on the exterior girders of the prototype bridge. The interior girders are assumed identical to the exterior girders.

A dynamic load allowance (impact factor) is used to account for the dynamic response of vehicles riding over discontinuities in the deck surface such as potholes and deck joints. The dynamic load allowance applies to truck and tandem loads where a factor of 1.15 is applied for the fatigue limit state and a factor of 1.33 is applied for all other limit states.

The live load effects are calculated as follows: (1) select either the truck or the tandem load, which ever produces the largest moment or shear; (2) multiply the truck or tandem load by the impact factor; (3) add the lane load to the truck or tandem load; and (4) multiply this combination by the appropriate live-load distribution factor.

The AASHTO LRFD bridge design specifications (AASHTO 1998) specify an HS-20 fatigue design truck as the fatigue limit state live load. As discussed below, the load factor for the fatigue limit state is 0.75 and this results in a fatigue load equivalent to an HS-15 truck. The HS-20 truck is similar to the truck shown in Figure 3.3 except the rear axle has a fixed spacing of 14 ft (4267 mm), and the front and rear axle loads are 8 and 32 kips (35.6 and142.4 kN), respectively.

Figure 3.5 and Figure 3.6 summarize the unfactored dead and live load moment envelopes and shear demands used to design the I-girders for the prototype bridge.

3.2.3 Load Factors and Load Combinations

I-girders for the prototype bridge were designed to resist the loads described in the previous sections. Many of these loads act on the bridge simultaneously. In addition, these loads are nominal loads, not factored loads. Therefore, the I-girders were designed for factored demands which are combinations of the load effects due to the nominal loads with appropriate load factors.

The load combinations used in this design study correspond to the strength I, service II, and fatigue limit states of the AASHTO LRFD bridge design specifications (AASHTO 1998). Strength I is a load combination for the maximum dead and live load on the bridge without wind loads. A separate construction load combination was developed to consider the structural component dead load acting on the girders under construction conditions. Service II is a service level dead and live load combination for which permanent deformation of the structure should not occur. The fatigue load combination considers the live load and dynamic load allowance due to trucks on the bridge. The load combinations and load factors are shown in Table 3.2.

3.2.4 Design and Analysis Procedure

Formulas to calculate the flexural and shear resistance of composite steel I-girders are given by the AASHTO LRFD bridge design specifications (AASHTO 1998). Each composite I-girder design is checked for the strength, construction, service, and fatigue limit states discussed earlier. The procedure for these checks generally follows the design procedures used in (AISI 1996). The design and analysis procedure begins by selecting a set of I-girder cross-section dimensions that should result in the factored resistance exceeding the factored design demands. These dimensions are selected iteratively with the objective of minimizing the girder weight. For each set of cross-section dimensions, the following checks are performed:

Strength I limit state check

Flexure

The nominal moment resistance, M_n is taken as the plastic-moment capacity, M_p, which is calculated according to Article 6.10.5.1.3. To ensure adequate ductility of the composite section, M_n may be reduced to a value less than M_p according to Article 6.10.5.2.2. The strength I performance ratio for flexure is calculated as $M_{strength\ I} / M_n$.

Shear

The nominal shear resistance, V_n, for an unstiffened web is determined according to Article 6.10.7.2, as discussed in detail in Chapter 5. The strength I performance ratio for shear is calculated as $V_{strength\ I} / V_n$, where V_n is for an unstiffened web. If the performance ratio is greater than 1, intermediate transverse stiffeners are needed to increase the shear resistance. If intermediate trasverse stiffeners are needed, the spacing is determined according to Article 6.10.7.3.3.

Construction check

The flexural strength of the non-composite I-girder under construction loads is checked. If the cross-section satisfies the compression flange slenderness, compression flange bracing, and web slenderness criteria according to Article 6.10.5, the nominal moment resistance, M_n, of the steel cross-section is taken as the yield moment, M_y. If the compression flange bracing criteria is not satisfied, M_n is determined considering lateral torsional buckling according to Article 6.10.6.4.1. The performance ratio for lateral torsional buckling is calculated as $M_{construciton} / M_n$.

Servcie II limit state check

The flexural stress in the flanges of the composite I-girder are limited to prevent permanent deflections. The flexural stresses are calculated as the sum of the stress due to D_c acting on the non-composite girder cross-section, the stresses due to D_w acting on the long-term composite girder cross-section and the stresses due to the live loads acting on the short-term composite girder cross-section. These flexural stresses are checked against the allowable stress, $f_{allowable}$, which is determined according to Article 6.10.3.2. The performance ratio for the service II limit state is calculated as $f_{flange} / f_{allowable}$.

Fatigue limit state check

The connection of the transverse stiffener/cross frame connection plates to the web and flange of the I-girder is checked for fatigue. The stress range due to the fatigue load, Δf, at the connection detail is limited to be less than the nominal fatigue resistance, $(\Delta F)_n$, determined according to Article 6.6.1.2. This is discussed in more detail in Chapter 6. The performance ratio for the fatigue limit state is calculated as $(\Delta F)_n / \Delta f$.

The results of the design and analysis procedure for a typical girder are summarized in Table 3.3. The cross-sectional properties along with the flange and web dimensions are shown. Comparisons between the factored design demands (from the combinations of factored load effects) and the factored resistances are presented. The results for each limit state are expressed in terms of a performance ratio. In addition, the compression flange slenderness and web slenderness of the cross-section are compared with the appropriated limits. The performance ratio for these comparisons is the ratio of the plate slenderness to the slenderness limit in the AASHTO LRFD bridge design specifications (AASHTO 1998). The performance ratio for the compression flange bracing is defined as the ratio between L_b and L_p (which are defined in Chapter 2). When this ratio is greater than one, lateral torsional buckling controls the flexural resistance under construction loads.

3.3 Influence of I-Girder Design Parameters

The design studies in the following chapters investigate the influence of several I-girder designs parameters including cross frame spacing (Chapter 4), web shear strength (Chapter 5), fatigue details (Chapter 6), and FRRBs (Chapter 7) on the weight of I-girders designed for the prototype bridge. MathCAD 3.1 is the numerical mathematics computer software that was used to check the girder designs. A program was written in MathCAD 3.1 to follow the design and analysis procedure described in Section 3.2.4. Using the MathCAD program, each set of I-girder cross-section dimensions was checked as the dimensions were varied to minimize the girder weight.

To illustrate the use of the MathCAD program in the design studies presented in later chapters, several examples from these studies are presented herein. I-girders were designed for the prototype bridge considering three different cross frame arrangements. Figure 3.7 shows the cross frame locations for three cross frame arrangements with 6, 5, and 4 cross frames. Figure 3.8 shows the influence of the cross frame arrangements on the total steel girder weight for the four girder prototype bridge. Increasing the cross frame spacing, and hence decreasing the number of cross frames, results in a small increase in girder weight. The increase in girder weight is a result of an increase in the dimensions of the compression flange required to avoid lateral buckling under construction loads (see Chapters 4 and 7). In the design study presented in Chapter 5, the bridge girders of the prototype bridge are designed with the web thickness as a variable ranging from 1/16 (2 mm) to much thicker. However, the typical web thickness was 3/8 in (10 mm). Figure 3.9 shows that as the web thickness is increased up to 3/8 in (10 mm), the steel weight decreases. The design study discussed in Chapter 5 shows that as the web thicknesses is increased beyond 3/8 in (10 mm) the girder weight increases.

The design study presented in the Chapter 6 consider the influence of the fatigue resistance of connection details on the design of the I-girders for the prototype bridge. The connection of the transverse stiffeners/cross frame connection plates to the girder web and tension flange is the connection of interest. When this connection is made with a fillet weld, Category C′ with the corresponding fatigue resistance is assigned to this connection. The use of a Category C′ fatigue detail limits possible decreases in girder weight because it prevents a significant reduction in the tension flange area. The result is tension flange area that is not necessary for the strength or service limit states. Figure 3.10 shows the girder weight reduction that results from using upgraded (Category B) fatigue details. This will be discussed further in Chapter 6.

Chapter 7 discusses a design study that investigates the potential savings in girder weight that result from including FRRBs with the cross frames of the prototype bridge. The FRRBs provide increased lateral torsional buckling moment resistance during the construction stage. I-girders are designed for the prototype bridge with four cross frames braced at 0, 45.93, 85.30, and 131.24 ft (0, 14, 26, and 40 m) with twoFRRBs located at the intermediate cross frames. Figure 3.11 shows the use ofFRRBs results in a 7 kip (31.2 kN) decrease in girder weight when compared to similar design without FRRBs.

3.4 Flange and Web Splices

Steel I-girder bridges typically include field splices and shop splices. Field splices are needed to limit the lengths of the field segments of long girders to lengths that can be transported from the steel fabricator to the bridge site. The location of field splices is chosen to avoid areas of high bending moment. Shop splices are used in fabrication when the length of field segments exceed the available plate lengths and when the girder cross section dimensions should be decreased as the bending moment decreases.

The use of field and shop splices can significantly increase fabrication costs. These fabrication costs include labor and material costs associated with the splice plates and bolts holes needed for bolted field splices or the labor and material costs associated with full penetration groove welds for welded shop splices. Therefore, splices not required for length considerations are desirable only when the cost savings associated with decreased girder weight exceed the increased fabrication costs.

The prototype girder includes two welded shop splices, creating three shop welded girder segments. A design study of the prototype bridge was conducted to determine an appropriate splice location. Three girder designs with cross frames at 32.81 ft (10 m) intervals are studied. Girder designs 1, 2, and 3 have splices located 19.69 ft (6 m), 26.25 ft (8 m), and 32.81 ft (10 m) from the bearings, respectively as shown in Figure 3.12. Figure 3.13 shows that design 2, with splices located 26.25 ft (8 m) from the bearings, has the least girder weight. Therefore, the design studies in the following chapters will, in most cases, include flange and web splices at 26.25 ft (8 m) from the bearings.

3.5 Practical Limitations on Girder Dimensions

The AASHTO LRFD bridge design specifications (AASHTO 1998) include several practical limitations on cross-section dimensions. The Article 6.7.3 of the AASHTO LRFD specifications states that the minimum thickness for structural steel used in bridge girders, cross frames, and gusset plates is 5/16 in (8 mm). The AASHTO LRFD specifications suggest a traditional span-to-depth ratio of 30:1 for a simple span, steel I–girder bridge in Article 2.5.2.6.3. The design studies of the prototype bridge will maintain, approximately, the suggested span-to-depth ratio by considering typical web depths ranging from 51 in (1295 mm) to 56 in (1422 mm).

Other limitations on plate dimensions exist. Structural steel plate elements must be within the capabilities of U.S. steel producers. The availability of flange and web plate length is restricted for HPS 70W. Steel mill reheating furnaces restrict HPS 70W plate lengths from exceeding 50 ft (15.2 m) (Bethlehem Steel Corporation 1994).

Available plate thicknesses in a plate steel specification guide (Bethlehem Lukens Plate Steel Specification Guide 1998) were used to select I-girder dimensions in the design studies. The plate thickness increments used in the design studies are as follows. The flange width and web depth are selected considering 1 in (25 mm) increments. Flange thicknesses less than 1½ in (38 mm) are selected considering 1/8 in (3 mm) increments. Flange thicknesses greater than 1½ in (38 mm) have a 1/4 in (6 mm) increment. The web thickness increment is 1/16 in (2 mm). Figure 3.8 shows how girder weight is influenced by the plate dimension increments. As the web depth increases, the girder weight decreases. Inconsistent decreases in girder weight result from the increments at which the flange and web plate dimensions were varied.

Table 3.1 Dead loads

Type	Component	Calculation	Load / length
DC	Slab	$0.15\ kcf * \frac{10\ in}{12\ in} * 12.5\ ft$	$1.60\ \frac{k}{ft}$
DC	Concrete haunch	$0.15\ kcf * \frac{20\ in}{144\ sq.\ in} * 3\ in$	$0.06\ \frac{k}{ft}$
DC	Steel girder (Assume 80 sq in steel area)	$0.49\ kcf * \frac{80\ sq.\ in}{144\ sq.\ in}$	$0.26\ \frac{k}{ft}$
DC	Secondary steel	$0.26\ \frac{k}{ft} * 10\ \%$	$0.03\ \frac{k}{ft}$
DC	Stay-in-place forms	(Estimated)	$0.05\ \frac{k}{ft}$
DW	Future wearing surface	(Estimated)	$0.315\ \frac{k}{ft}$
DW	Miscellaneous (parapet, railing, lights,elec.)	(Estimated)	$0.185\ \frac{k}{ft}$

Table 3.2 Load factors and load combinations

Limit state	DC	DW	LL+IM
Strength I	1.25	1.50	1.75
Construction	1.25	-	-
Service II	1.00	1.00	1.30
Fatigue	-	-	0.75

Table 3.3 Summary of I-girder design and analysis

(a) Cross section properties

Dimensions	end section	mid section
Length (ft)	26.25	78.75
Ttf (in)	0.875	1.125
Btf (in)	14.0	19.0
Tbf (in)	1.125	1.750
Bbf (in)	19.0	21.0
Tweb (in)	0.375	0.375
Dweb (in)	51.0	51.0
Atf (in^2)	12.25	21.38
Abf (in^2)	21.38	36.75
Aweb (in^2)	19.13	19.13
Total (in^2)	52.75	77.25
Span-depth ratio	29.9	29.4

(b) Performance ratios

Limit States and Criteria	end section	mid section
Strength Limit State		
Flexure Resistance	0.95	0.88
Ductility Requirement	0.97	0.95
Shear Resistance	3.75	2.55
Construction		
Web Slenderness	0.98	1.00
Compression Flange Slenderness	0.85	0.90
Compression Flange Bracing	2.48	1.74
Flexure Resistance	0.82	0.93
Service II Limit State		
Perm. Deflection (tension flange)	0.94	0.94
Fatigue Limit State		
Bottom flange connection plate weld	1.01	1.01

Loads (kip-ft) and section location (ft)	end section	mid section
Section location	26.25 ft	66 ft
Component Dead Load, Dc	2755	4305
Wearing Surface Load, Dw	690	1075
Live Load + Impact, LL+IM	3170	5090
Fatigue Live Load	1170	1810

Nomenclature

Ttf = top flange thickness

Btf = top flange width

Tbf = bottom flange thickness

Bbf = bottom flange width

Tweb = web thickness

Dweb = web depth

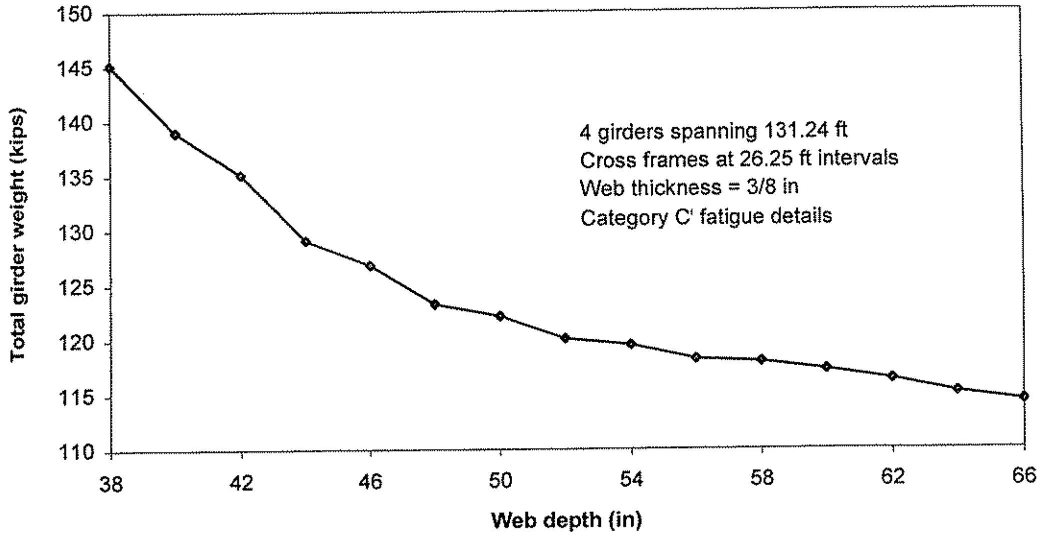


Figure 3.1 Influence of web depth on total girder weight of a bridge

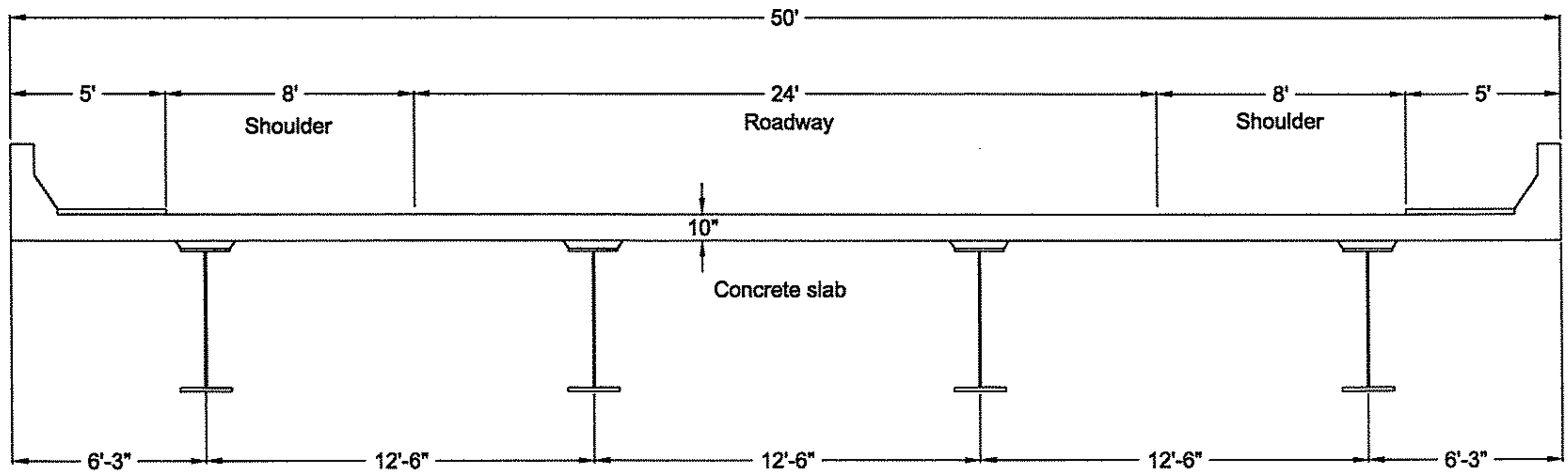
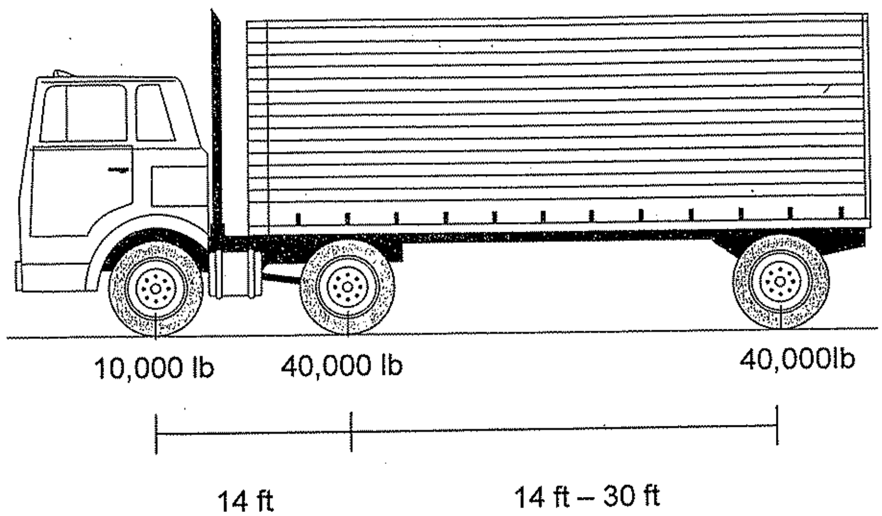


Figure 3.2 Prototype bridge cross section

(a) Front view



(b) Back view

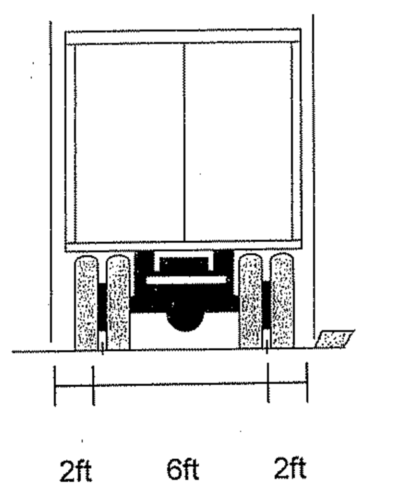


Figure 3.3 HS truck load (Tally 1998)

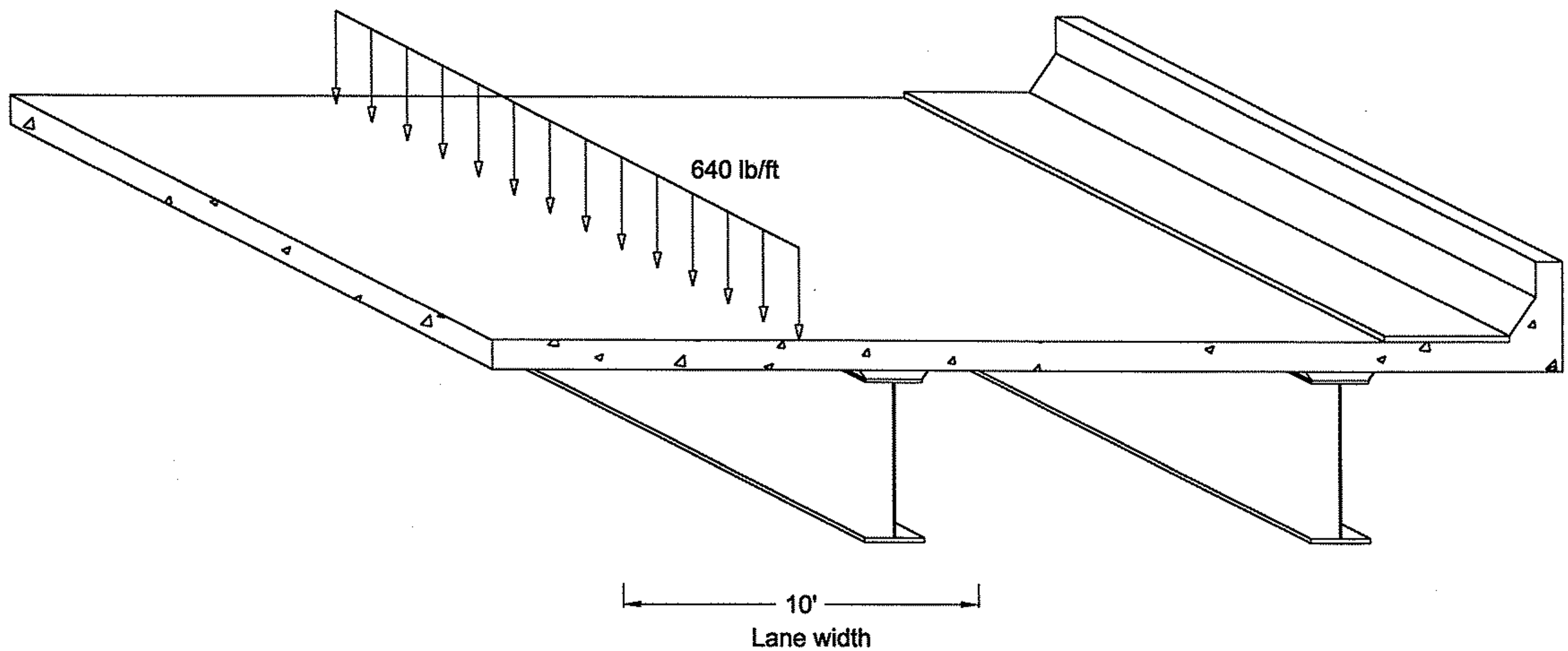


Figure 3.4 Lane loading (AASHTO 1998)

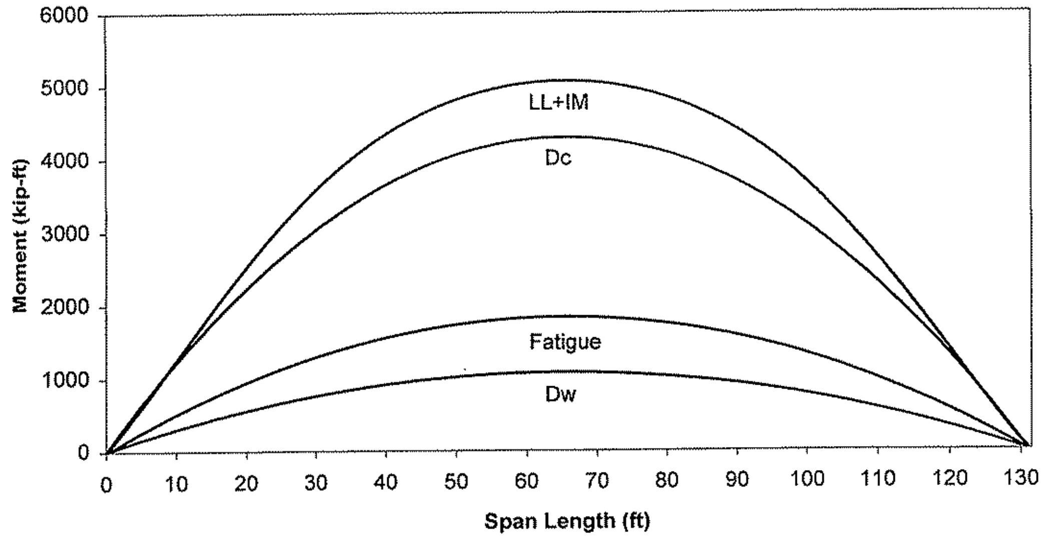


Figure 3.5 Unfactored dead and live load moment envelopes

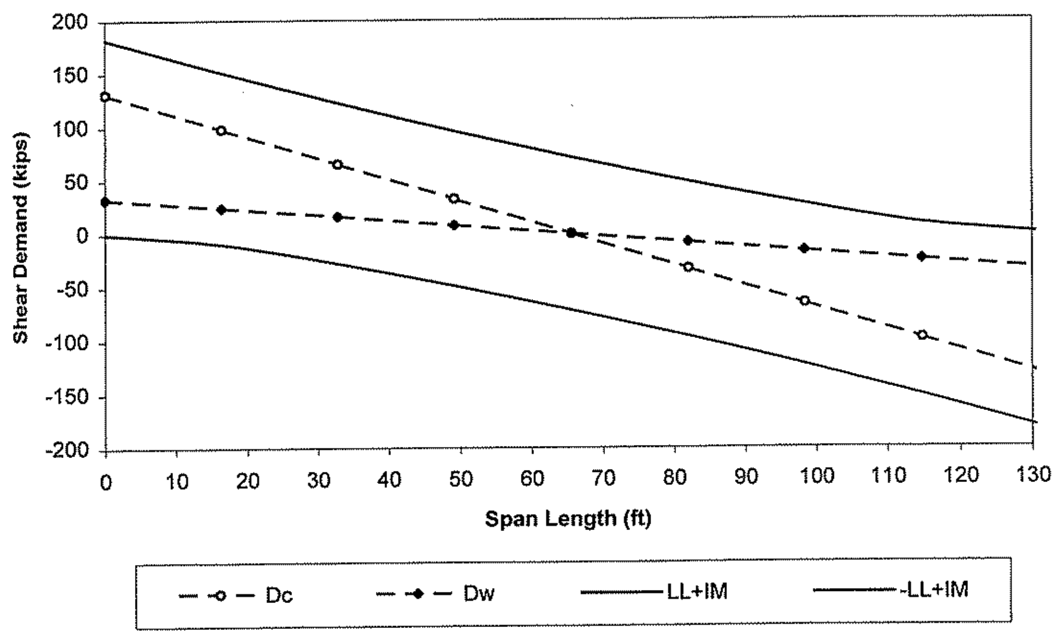


Figure 3.6 Unfactored dead and live load shear demands

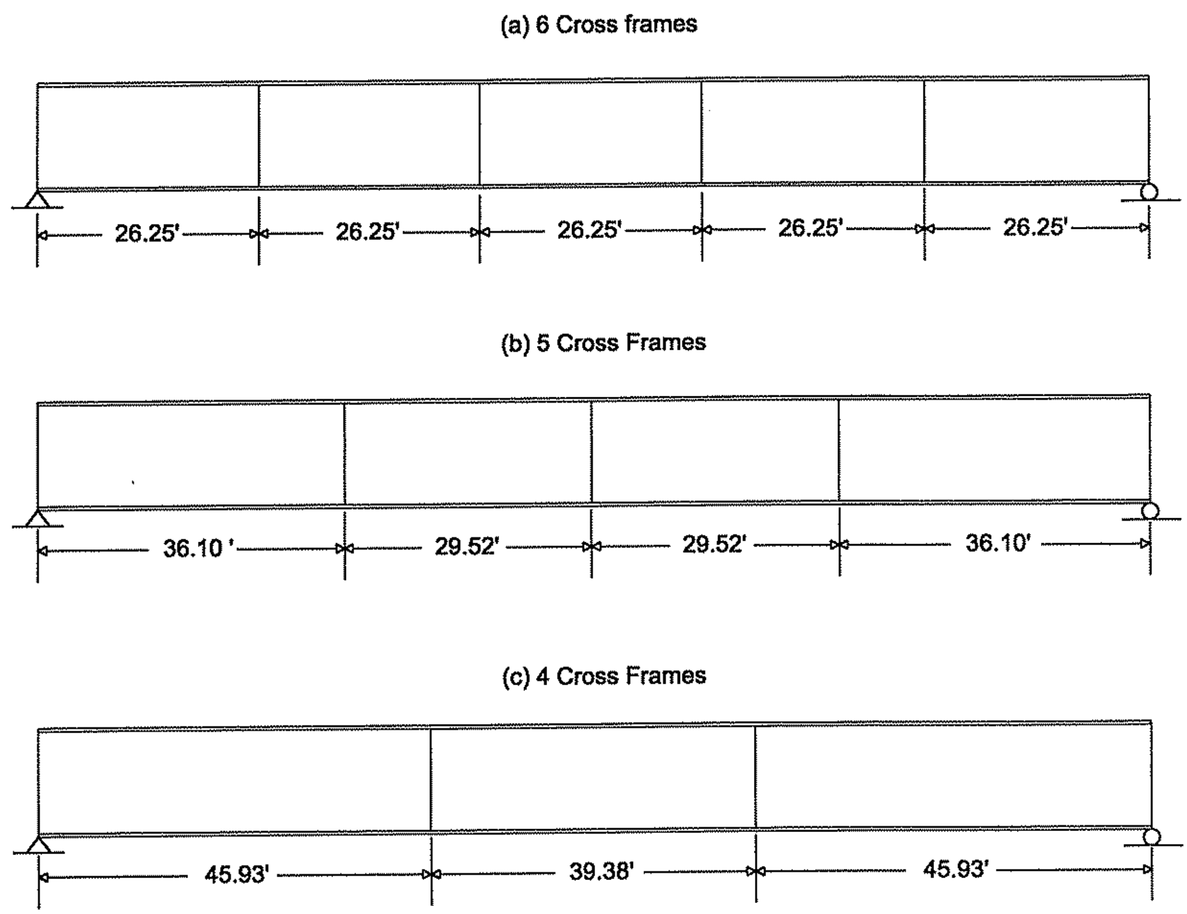


Figure 3.7 Girder cross frame locations

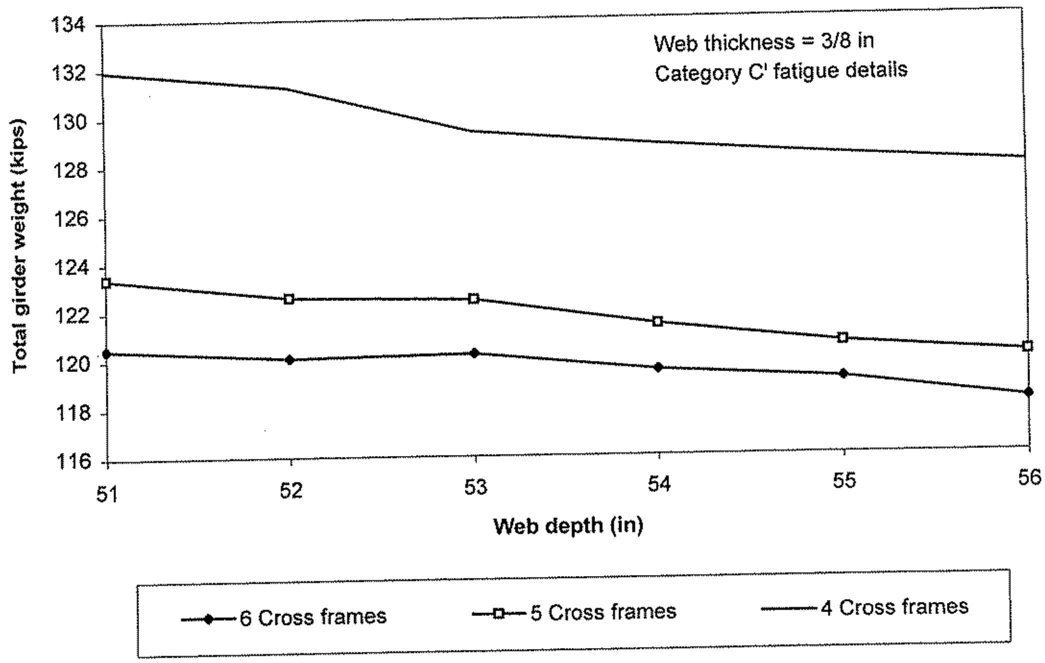


Figure 3.8 Influence of cross frame spacing on total girder weight

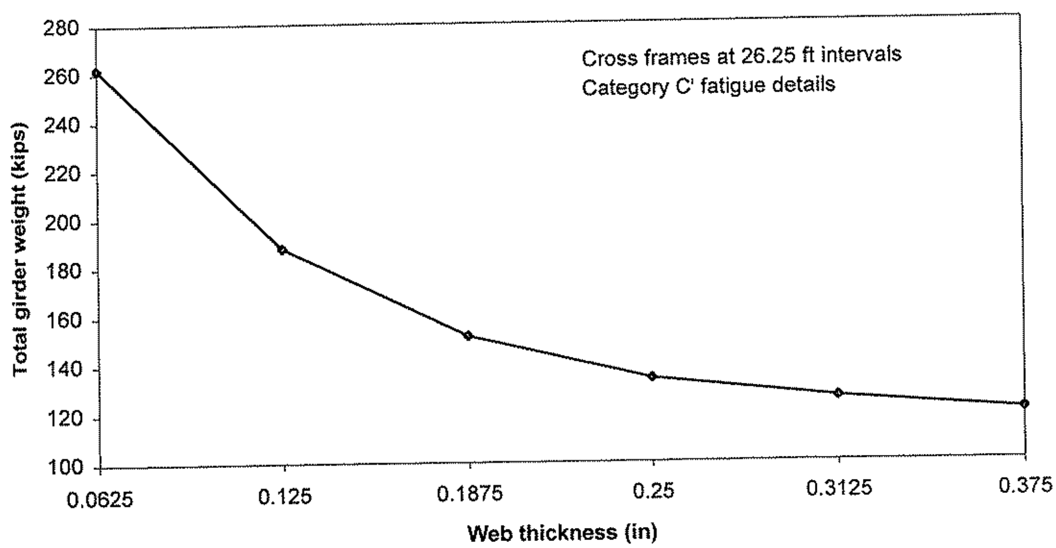


Figure 3.9 Influence of web depth on total girder weight

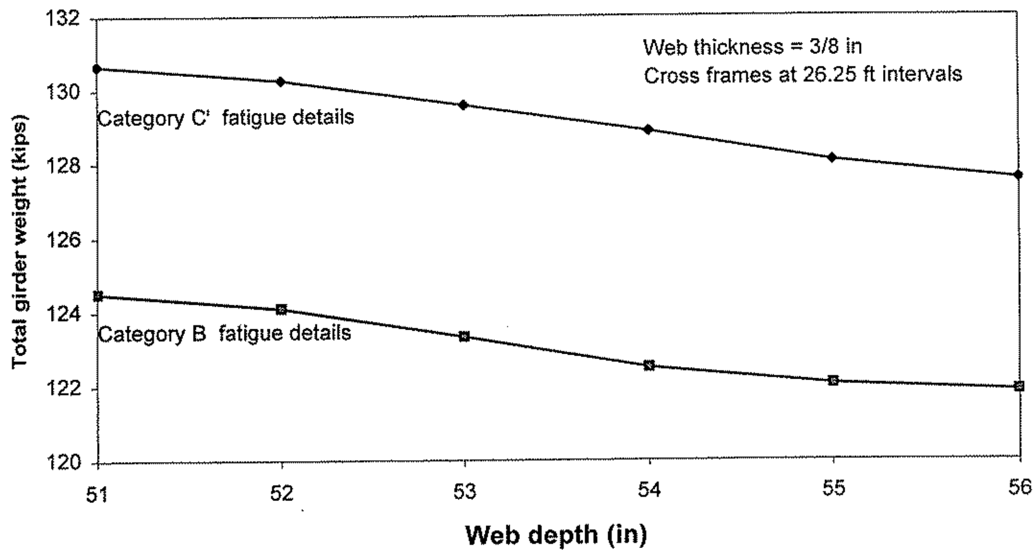


Figure 3.10 Influence of fatigue details on total girder weight

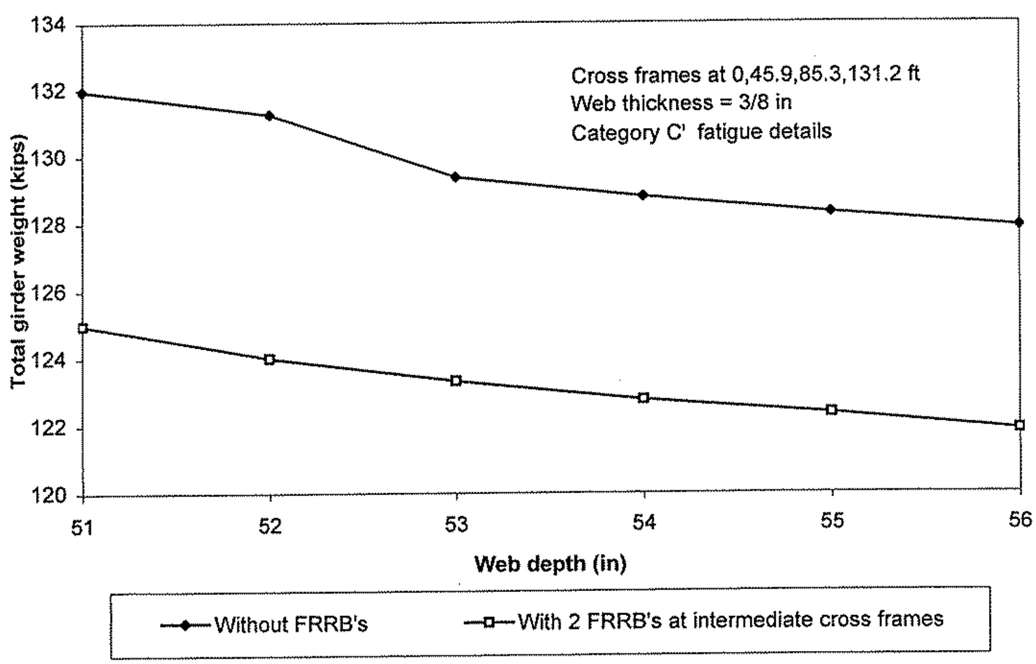


Figure 3.11 Influence of the use of FRRBs on total girder weight

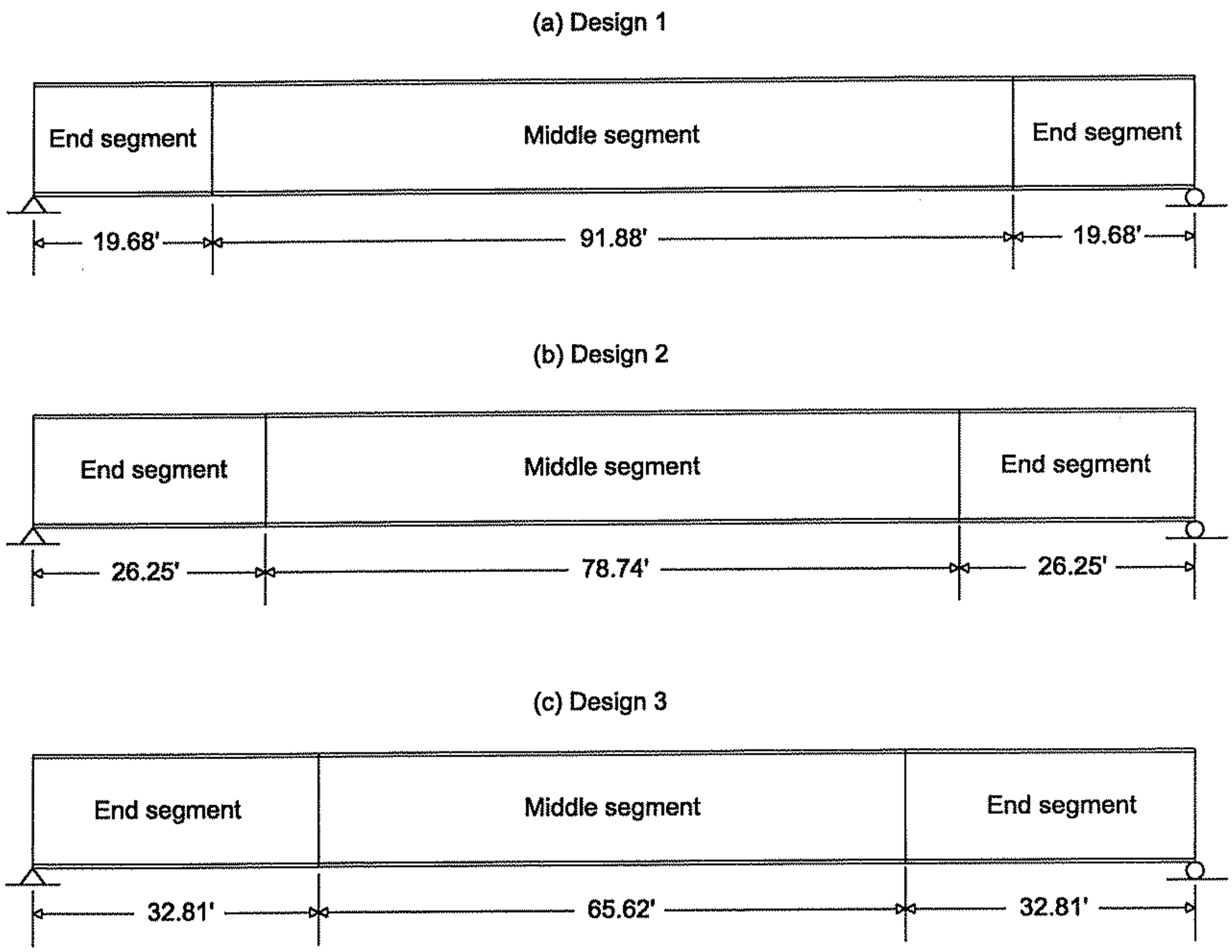


Figure 3.12 Shop splice locations

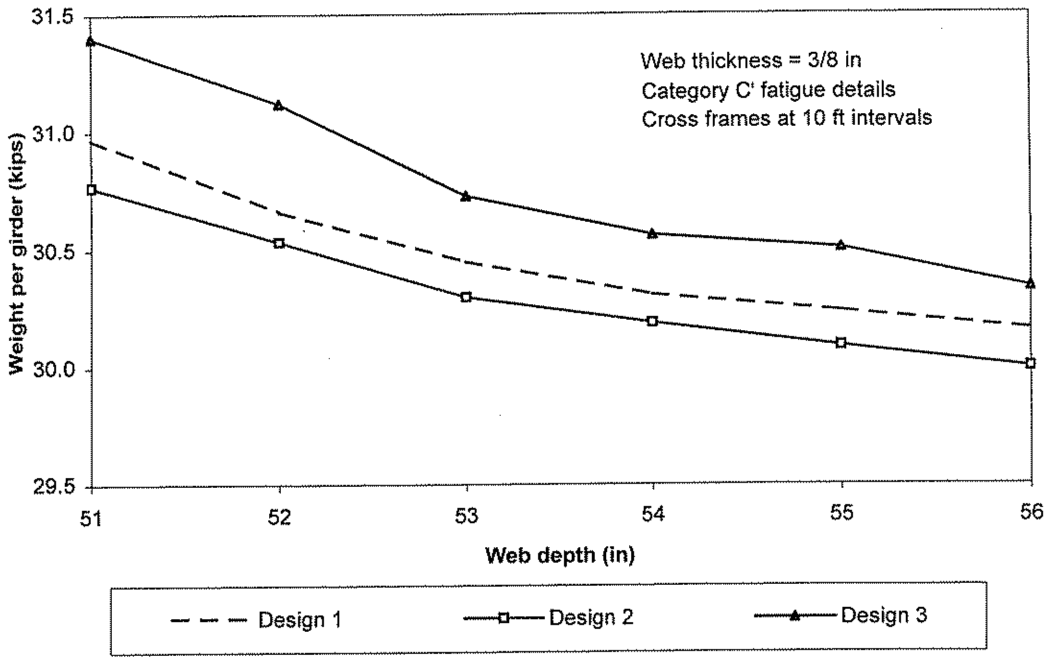


Figure 3.13 Influence of splice locations on total girder weight

CHAPTER 4

Design Study of I-Girders With Variation of Cross Frame Spacing

In composite steel I-girder bridges, cross frames are connected to adjacent I-girders at various intervals along the span. Cross frame spacing can have a significant effect on the stability of bridge I-girders. In current practice, cross frames are usually spaced at roughly equal distances apart using a spacing less than 25 ft (7620 mm). However, according to the current AASHTO LRFD bridge design specifications (AASHTO 1998), the distance between cross frames, L_b, shown in Figure 4.1, can exceed 25 ft (7620 mm) and can vary along the girder length. This chapter presents a study of the influence of the number, spacing, and arrangement of cross frames on the design of the I-girders of the prototype bridge.

4.1 Background

In a typical I-girder bridge, two distinct types of cross frames can be identified depending on their location. Cross frames located at the bridge bearings are end cross frames and those located between the bearings are intermediate cross frames.

Intermediate cross frames are constructed with top and bottom chords and diagonals as shown in Figure 4.2. These members are often equal leg single angles or double angles. The bottom and top chord are usually larger than the diagonals because they are usually designed for larger forces. The diagonals can be arranged in a cross or chevron configuration.

End cross frames are constructed of members similar to those used in intermediate cross frames but larger member sizes are often used as shown in Figure 4.3. The larger member sizes are required because large lateral forces must be transferred to the bearings from the deck by the end cross frame. In addition, the top chord of an end cross frame has to carry a small portion of the dead weight of the concrete deck. Therefore, a channel or wide flange shape is often used as the top chord.

Cross frames must be properly connected to the I-girders. The complexity of this connection can vary depending on the magnitude of the forces transmitted by the connection and the geometry of connecting members. The diagonals and top and bottom chords are attached to an I-girder with a cross frame connection plate that is similar in size to a transverse stiffener.

Cross frames must be designed for loading conditions in the construction stage and the service stage. During construction, cross frames are the only permanent members available to prevent the I-girders from deflecting independently between the bearings. Cross frames control differential vertical deflections as well as lateral deflections during the construction stage. As a result, cross frames must be designed to prevent buckling of the girders during construction. The loads acting on the girders during the construction phase include: the weight of the girders and cross frames, construction workers, equipment, concrete forms, wet concrete, and wind. During the service stage, the combination of the cross frames and the concrete deck control differential vertical deflections as well as lateral deflections. The loads acting on the bridge during the service stage include: the weight of the girders, cross frames, and deck, superimposed dead loads, live loads, wind, snow, and earthquake loads.

4.2 Cross Frame Design

Guidelines for cross frame member sizes, based on the I-girder spacing, have been developed by the American Institute of Steel Construction (AISC Short Span Steel Bridges 1994). In practice, cross frame member sizes are often determined based on previous experience. This approach stems from uncertainty in the magnitude of the forces acting on the cross frame during the construction and service stages.

A logical approach to cross frame design requires the engineer to design cross frames for lateral loads, such as wind and earthquake loads, and to consider the influence of cross frame stiffness and strength on I-girder stability. The design of cross frames for wind loads is relatively simple, and, therefore, it is included in the current discussion. Lateral loads other than wind are not considered here.

A simple approach to design cross frames for wind loads is outlined in the AASHTO LRFD bridge design specifications (AASHTO 1998). Figure 4.4 shows conceptually wind loads acting on a composite steel I-girder bridge. The exterior I-girder, deck, parapet, vehicles, and so on are exposed to a horizontal wind pressure. The design wind pressure depends on the design wind velocity (AASHTO 1998). The wind pressure on the deck, parapet, vehicles, and upper half of the I-girders is assumed to be transmitted directly to the deck, which acts as a lateral diaphragm carrying this load to the end cross frames, and thus to the bearings as discussed later. The wind pressure on the lower half of the I-girder is applied laterally to the I-girder bottom flange. This pressure can be converted into a wind load per unit length acting on the bottom flange as shown in Figure 4.5 as follows:

$$w = \frac{P_D \cdot d}{2} \qquad \text{(Eq. 4.1)}$$

where, P_D is the design wind pressure and d is the depth of the I-girder. The horizontal wind force applied to the brace point is computed as follows:

$$P_{wind} = w \cdot L_b \qquad \text{(Eq. 4.2)}$$

The bottom flange is assumed to carry the wind load to the nearest cross frames. The diagonals transmit the forces up to the deck as shown in Figure 4.6. The bridge deck carries these loads in shear to the end cross frames. The end cross frames transmit the force to the bearings as shown in Figure 4.7.

The AASHTO LRFD bridge design specifications (AASHTO 1998) include wind load in the strength III load combination with a load factor of 1.4. However, as discussed in Chapter 3, this load combination is not included in the present design study.

The stiffness and strength of intermediate and end cross frames influence the stability of the girders in a composite steel I-girder bridge, especially during the construction stage, before the concrete deck cures. Cross frames should be designed with sufficient stiffness and strength to adequately brace the I-girders. Stiffness and strength requirements have been derived considering the lateral torsional buckling behavior of the girders.

Taylor and Ojalvo (1973) developed the following equation for the bending moment at buckling of a doubly symmetric I-girder under uniform moment with continuous torsional bracing:

$$M_{cr} = \sqrt{M_o^2 + \overline{\beta}_b \cdot E \cdot I_y} \qquad \text{(Eq. 4.3)}$$

where, M_o is the buckling capacity of the unbraced girder, $\overline{\beta}_b$ is the continuous torsional brace stiffness, E is the modulus of elasticity, and I_y is the moment of inertia of the girder with respect to the y (vertical) axis.

Cross frame stiffness requirements can be determined with modifications to Equation 4.3. The first variable, M_o, representing buckling capacity of the girder, is neglected since its contribution is significantly less than the effect of the braces (cross frames). Millner (1977) replaced $\overline{\beta}_b$ with $\overline{\beta}_T$, a continuous brace stiffness that includes cross-section distortion. Equation 4.3 can be adapted for discrete braces by replacing $\overline{\beta}_T$ with $n \cdot (\beta_T / L)$ which is an equivalent continuous brace stiffness equal to the sum of the effective stiffnesses of the discrete braces along the span of the girder divided by the span.

After these modifications are made, Equation 4.3 can be solved to determine the required effective brace stiffness $(\beta_T)_{required}$. Yura and Helwig (1996) include the following changes to Equation 4.3 before solving for $(\beta_T)_{required}$: a C_{bb} factor is included to take into account the moment gradient between braces (cross frames); a factor of 1.2 is included to consider the effect of flange loading on the top flange rather than at the centroid; a factor of 2 is included to stiffen against the effects of initial out-of-straightness of the girder; and I_y is changed to I_{eff}, which is calculated as follows:

$$I_{eff} = I_{yc} + \frac{t}{c} \cdot I_{yt} \qquad \text{(Eq. 4.4)}$$

where, I_{yc} is the y axis moment of inertia of the compression flange, t / c is the ratio of distances from the extreme tension and compression fibers to the steel centroid of the girder, and I_{yt} is the y axis moment of inertia of the tension flange. The required brace stiffness is as follows Yura and Helwig (1996):

$$(\beta_T)_{required} = \frac{1.2 \cdot 2 \cdot L \cdot M_f^{\,2}}{n \cdot E \cdot I_{eff} \cdot C_{bb}^2} \qquad \text{(Eq. 4.5)}$$

where, n is the number of braces, L is the span of the girder, and M_f is the maximum moment in the beam.

In the development of bracing design recommendations by Yura and Helwig (1996), it was suggested that other factors had a significant effect on the bracing provided by discrete braces by reducing the torsional stiffness. Yura and Helwig (1996) adjusted the brace stiffness to include the effect of the web and stiffeners β_{sec}, the discrete brace stiffness (i.e., cross frame) β_b, and the girder stiffness β_g. These adjustments account for cross-section distortion at the brace point. The effective brace stiffness, β_T, is determined as follows (Yura and Helwig 1996):

$$\frac{1}{\beta_T} = \frac{1}{\beta_b} + \frac{1}{\beta_{sec}} + \frac{1}{\beta_g} \qquad \text{(Eq. 4.6)}$$

The effective brace stiffness β_T should be greater than the required brace stiffness $(\beta_T)_{required}$ when designing cross frames for stiffness.

Designing cross frames for strength requires calculation of the moment acting on the brace, M_{br}. M_{br} is obtained from Equation 4.5 by assuming the brace (cross frame) is subjected to a one degree (approximately 1 / 60 radians) twist.

$$M_{br} = \frac{0.04 \cdot L \cdot M_f^2}{n \cdot E \cdot I_{eff} \cdot C_{bb}^2} \qquad \text{(Eq. 4.7)}$$

The moment M_{br} is developed as a force couple:

$$M_{br} = F_{br} \cdot h_b \qquad \text{(Eq. 4.8)}$$

where, h_b is the height of the brace (cross frame) and F_{br} is the force in the brace. The internal forces in the cross frame members (in equilibrium with F_{br}) can be determined from statics.

4.3 Advantages of Increasing Cross Frame Spacing

The number of intermediate cross frames and the corresponding cross frame spacing affects the cost of fabricating and erecting a steel I-girder bridge. A recent technical bulletin related to the costs of steel I-girder bridges, produced by the Bethlehem Steel Corporation, pointed out the importance of fabrication and erection labor costs. "In the past, the steel industry encouraged minimizing the amount of material in order to provide the most economical girder. Lower material costs typically offset any expense for additional fabrication and erection labor. However, today the reverse is true. Steel material costs remain very near to those of ten years ago, while labor costs continue to increase" (Huzzard and Montgomery 1996).

The AASHTO standard specifications (AASTHO 1984) required cross frames at each bearing and spaced at intervals not to exceed 25 ft (7620 mm). This requirement precluded the possibility of reducing the number of cross frames. The initial AASHTO LRFD bridge design specifications (AASHTO 1994) removed this requirement and replaced it with a requirement to establish the need for cross frames by rational analysis.

Typical cross frames can cost between $500 and $1000 (Huzzard and Montgomery 1996). Thus it is important to omit unnecessary intermediate cross frames. However, intermediate cross frames can not be completely eliminated from composite steel I-girder bridges, because they are needed to brace the girders of a composite I-girder bridge (in the positive moment region) prior to placing and curing of the deck. In continuous I-girder bridges, cross frames are needed in the negative moment region to brace the I-girders before and after the deck is placed.

Figure 4.8 shows the results of a brief design study illustrating two concepts: (1) the influence of cross frame spacing on the weight of the steel girders in an I-girder bridge, (2) the effect of cross frame spacing on lateral torsional buckling resistance during construction. The prototype bridge described in Chapter 3 is the bridge used in the study.

Figure 4.8 is a plot of total girder weight versus the number of cross frames. The arrangements of cross frames range from schemes with two end cross frames to schemes with two end cross frames and 23 intermediate cross frames. It is assumed that the cross frames are equally spaced. For each cross frame arrangement, a minimum weight exterior girder is designed according to the AASHTO LRFD bridge design specifications (AASHTO 1998). The interior girders are assumed to be identical to the exterior girders. Figure 4.8 indicates 120 kips (535 kN) is the minimum total girder weight when the bridge has 25 cross frames. This minimum weight is nearly reached with only eight cross frames. This minimum weight solution will have a high labor cost, thus, this solution is not efficient. Reducing the number of cross frames to two end cross frames will reduce the labor cost, but will increase the total girder weight to 180 kips (801 kN). The 33% increase in girder weight can be attributed to the area needed in the top (compression) flange to overcome lateral torsional buckling under construction loads. Table 4.1 shows the dimensions of the girder cross-sections for each case. As the number of cross frames decreases, the area of the compression flange increases for both the end and middle segments.

These findings indicate that it is possible to reduce the number of cross frames by increasing the cross frame spacing. However, the effect on the girder weight cannot be overlooked.

4.4 Design Study of Cross Frame Arrangements for Prototype Bridge

A design study of cross frame arrangements for the prototype bridge was conducted. Different cross frame arrangements were considered. For each cross frame arrangement, minimum weight I-girders were designed for the bridge using the procedure described in Chapter 3. A "base" design, consisting of six cross frames spaced equally at 26.25 ft (8 m) was studied first. Then, arrangements of five and four cross frames with variable spacing were investigated.

As noted in Chapter 3, the prototype bridge has a single span of 131.24 ft (40 m). Plate transitions at 26.25 ft (8 m) or 32.81 ft (10 m) from the end bearings were used. The girder segments between the bearings and the plate transitions are identified as the end segments, and the segment between the plate transitions is the middle segment. A description of the eight cross frame arrangements used in this design study is presented in Table 4.2.

The cross frame arrangement called cross frame scheme 1 has six cross frames. Cross frame schemes 2 through 5 have five cross frames. These four different schemes were studied to see how increasing or decreasing the cross frame spacing influences the cross-section dimensions. Cross frame schemes 6 through 8 with four cross frames were also studied.

4.5 Design Study Results

Figure 4.9 shows the relationship between the cross frame arrangement and the total girder steel weight for a girder web depth ranging from 51 in (1295 mm) to 56 in (1425 mm). The total girder weight is the weight of four identical girders for the prototype bridge. The girders are minimum weight designs for the exterior girder of the bridge produced using the procedure described in Chapter 3. The interior girders are assumed to be identical to the exterior girders.

Figure 4.9 shows the steel I-girder weight increases when the number of cross frames is reduced. Cross frame scheme 1 with six cross frames is the least weight design. All other schemes have fewer cross frames. Cross frame scheme 3 has the lightest girder weight of the cross frame arrangements with five cross frames. Cross frame scheme 7 and scheme 8 have the lightest girders of the cross frame arrangements with four cross frames. Cross frame scheme 8 will be used as the best arrangement of the four cross frames, since scheme 7 and scheme 8 result in nearly identical girder weights.

The total girder weight varies for the different cross frame arrangements because of unequal trade-offs in steel weight between the end and middle girder segments. When the unbraced length of the end segments increases (as the cross frame spacing increases) so does the steel weight of these segments. At the same time, a decrease in steel weight of the middle segment will occur as its unbraced length is decreased. If the decrease in weight of the middle segment offsets the increase in weight of the end segments, the cross frame arrangement will result in a lower total girder weight. Notice that the lighter weight cross frame arrangements tend to have larger unbraced lengths (larger cross frame spacing) between the end cross frames and the first intermediate cross frames. This is a result of the moment gradient factor, C_b, which is large for the first unbraced length of the girder nearest the bearings.

Figure 4.10 shows the weight per girder of the end segment for the different cross frame arrangements. Cross frame scheme 3 has the lightest end segment of the arrangements with five cross frames because its first unbraced length (cross frame spacing) of 32.82 ft. (10 m) is the shortest of all the arrangements with five cross frames. Cross frame scheme 6 has the lightest end segment of the arrangements with four cross frames because scheme 6 has a shorter first unbraced length of (45.93 ft (14 m)) than scheme 7 and scheme 8 (49.21 ft. (15 m) and 50.85 ft. (15.5 m), respectively).

Figure 4.11 shows the weight per girder of the middle segment for the different cross frame arrangements. The largest unbraced length in the middle segment varies between 26.25 ft. (8 m) and 39.38 ft. (12 m) for these cross frame arrangements. Cross frame scheme 2 has the lightest middle segment, because in this case the flange plate transitions are longer (32.38 ft. (10 m)) from the end bearings than all other cases. This result is appealing, but when coupled with the heavier weight of the longer end segments (see Figure 4.10), the benefit is nullified. Cross frame scheme 8 has the lightest middle segment of the arrangements with four cross frames because the unbraced length of 29.54 ft. (9 m) is the shortest.

4.6 Summary

Eight cross frame arrangements for the prototype bridge, with four, five, or six cross frames were studied. The spacing between cross frames, which determines the unbraced length, was varied. The spacing generally increased as the number of cross frames was reduced. In addition, for a given number of cross frames, the spacing was non-uniform and was varied to determine the influence of the cross frame arrangement on the weight of the I-girders of the prototype bridge. It was found that the total girder weight increases when the number of cross frames is reduced. The cross frame spacing heavily influences the trade-off in steel girder weight between the end and middle segments. Figure 4.12 shows the cross frame arrangements that resulted in the lightest total girder weight. The results show that when the arrangement of cross frames in a bridge is being considered, the influence of cross frame spacing on the steel girder weight cannot be neglected.

Table 4.1 Girder cross section dimensions versus number of cross frames

Number of cross frames	Segment designation	Segment length (ft)	Top flange dimensions (in)	Bot. flange dimensions (in)	Top flange area (in^2)	Bot. flange area (in^2)	Segment weight (kips)	Weight per girder (kips)	Total girder weight (kips)
25	End	26.25	13 x 7/8	24 x 7/8	11.4	21.0	9.3	29.9	119.7
	Middle	78.75	18 x 1 1/8	21 x 1 3/4	20.3	36.8	20.6		
8	End	26.25	14 x 7/8	24 x 7/8	12.3	21.0	9.5	30.4	121.6
	Middle	78.75	19 x 1 1/8	21 x 1 3/4	21.4	36.8	20.9		
4	End	26.25	22 x 1	24 x 7/8	22.0	21.0	11.2	32.7	130.9
	Middle	78.75	21 x 1 1/8	21 x 1 3/4	23.6	36.8	21.5		
3	End	26.25	26 x 1	24 x 7/8	26.0	21.0	12.0	35.2	140.6
	Middle	78.75	24 x 1 1/4	21 x 1 3/4	30.0	36.8	23.2		
2	End	26.25	40 x 1 1/8	24 x 7/8	45.0	21.0	15.3	45.0	179.9
	Middle	78.75	36 x 1 1/2	21 x 1 3/4	54.0	36.8	29.6		

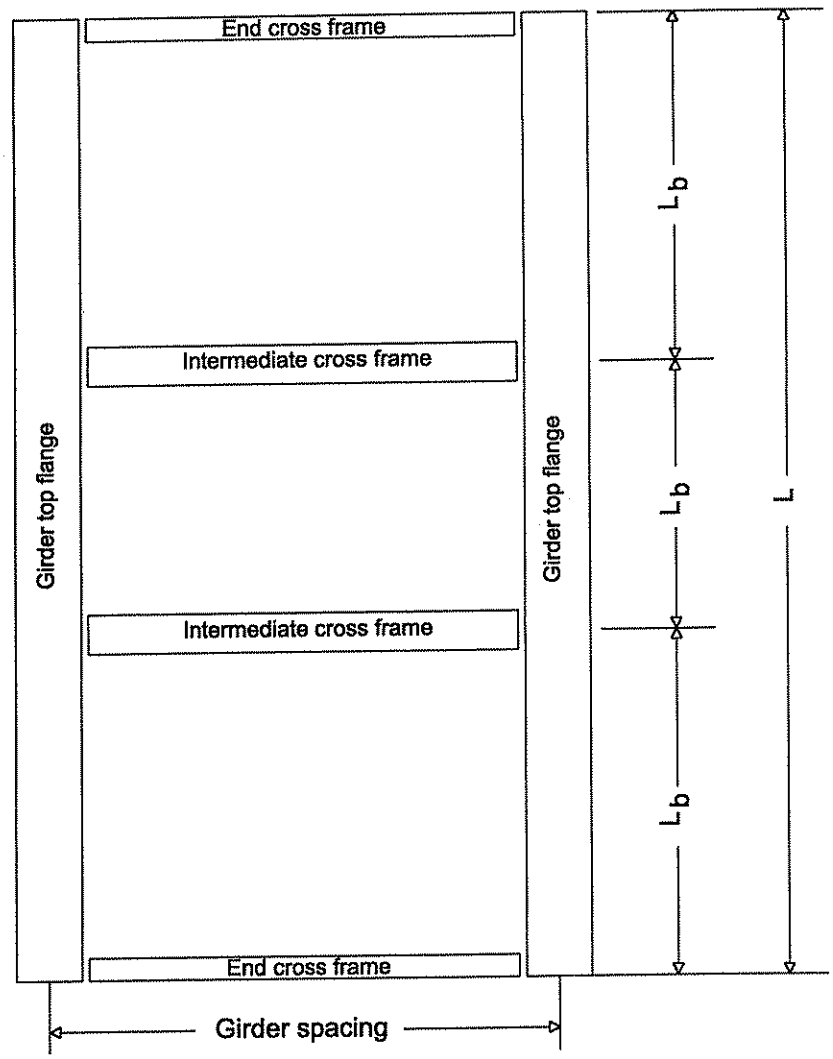
Note: The web dimensions are 53 x 3/8 in.

Table 4.2 Description of cross frame arrangements

Description		Scheme1	Scheme 2	Scheme 3
Number of cross frames		6	5	5
Cross frame location	ft	0, 26.25, 52.49, 78.75, 105.00, 131.24	0, 32.81, 65.62, 98.43, 131.24	0, 32.81, 65.62, 98.43, 131.24
	(m)	(0) (8) (16) (24) (32) (40)	(0) (10) (20) (30) (40)	(0) (10) (20) (30) (40)
Plate transition	ft (m)	26.25 (8) and 105.00 (32)	32.80 (10) and 98.40 (30)	26.25 (8) and 105.00 (32)

Description		Scheme 4	Scheme 5	Scheme 6
Number of cross frames		5	5	4
Cross frame location	ft	0, 39.37, 65.62, 91.86, 131.24	0, 36.09, 65.62, 95.14, 131.24	0, 45.93, 85.30, 131.24
	(m)	(0) (12) (20) (28) (40)	(0) (11) (20) (29) (40)	(0) (14) (26) (40)
Plate transition	ft (m)	26.25 (8) and 105.00 (32)	26.25 (8) and 105.00 (32)	26.25 (8) and 105.00 (32)

Description		Scheme 7	Scheme 8
Number of cross frames		4	4
Cross frame location	ft	0, 49.21, 82.02, 131.24	0, 50.85, 80.38, 131.24
	(m)	(0) (15) (25) (40)	(0) (15.5) (24.5) (40)
Plate transition	ft (m)	26.25 (8) and 105.00 (32)	26.25 (8) and 105.00 (32)



Not to scale

Figure 4.1 Plan view of top flange indicating the unbraced length between cross frames

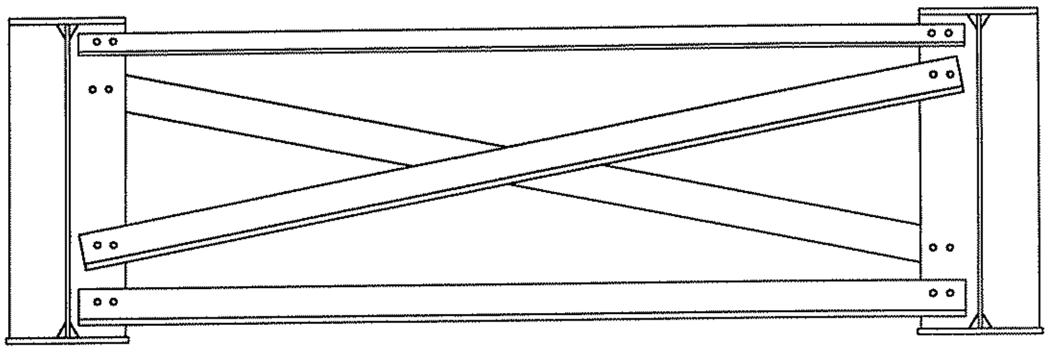


Figure 4.2 Typical intermediate cross frame

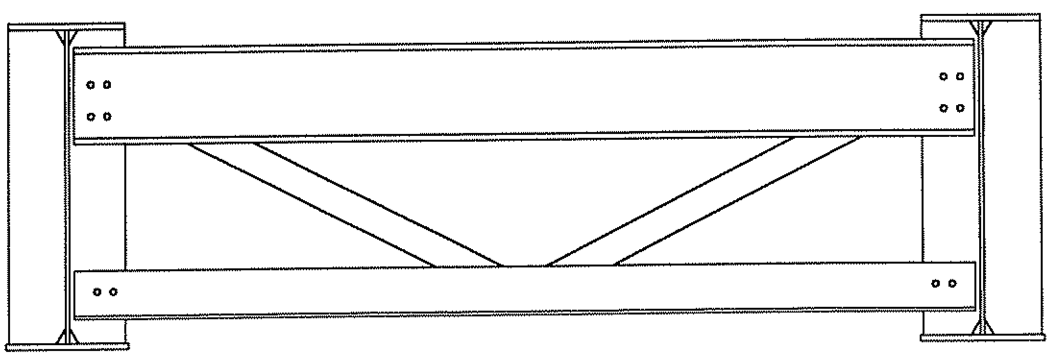


Figure 4.3 Typcial end cross frame

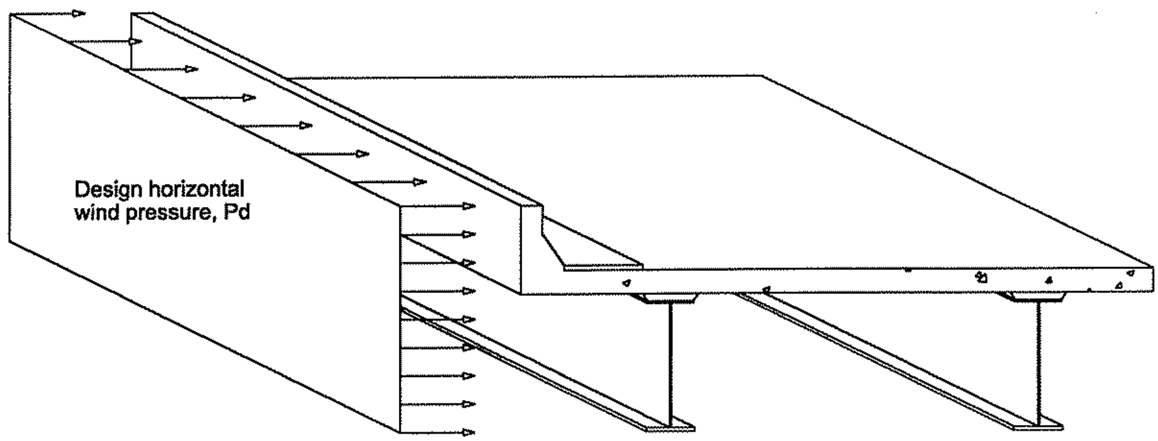


Figure 4.4 Wind pressure acting on bridge

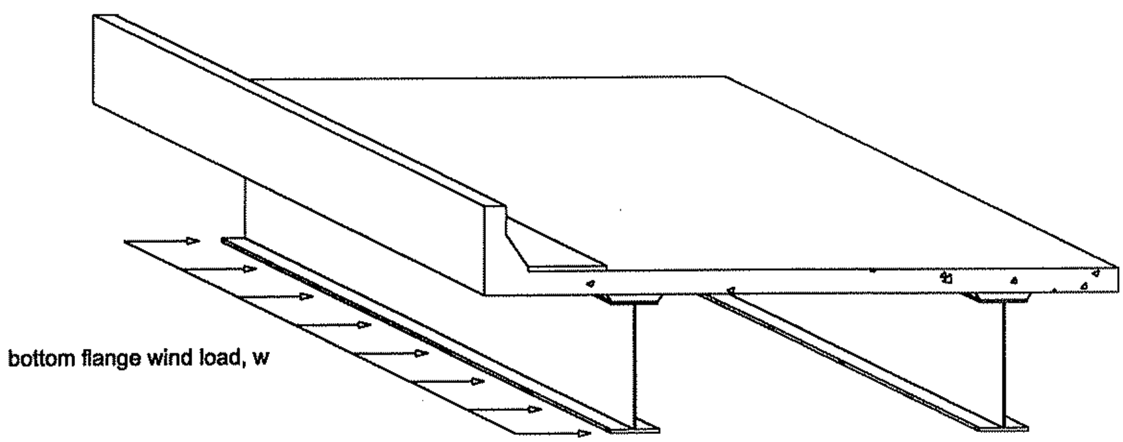


Figure 4.5 Wind design load on girder bottom flange

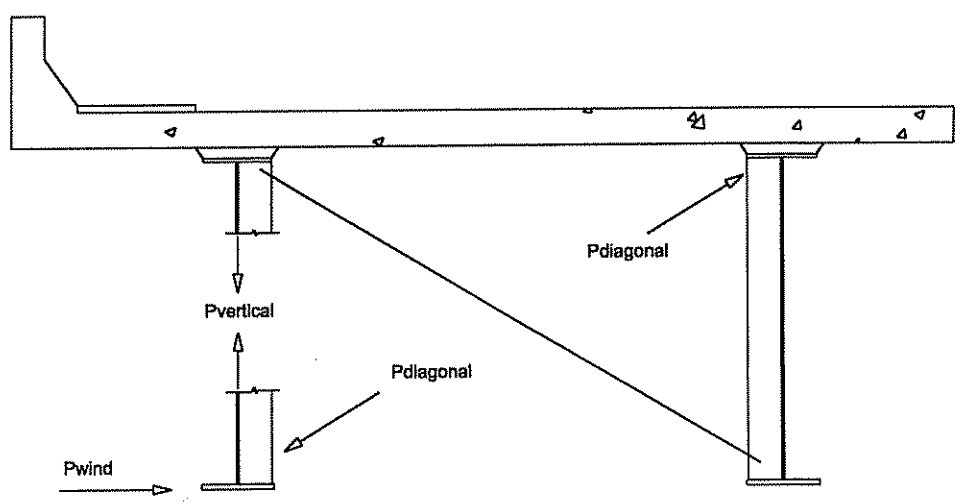
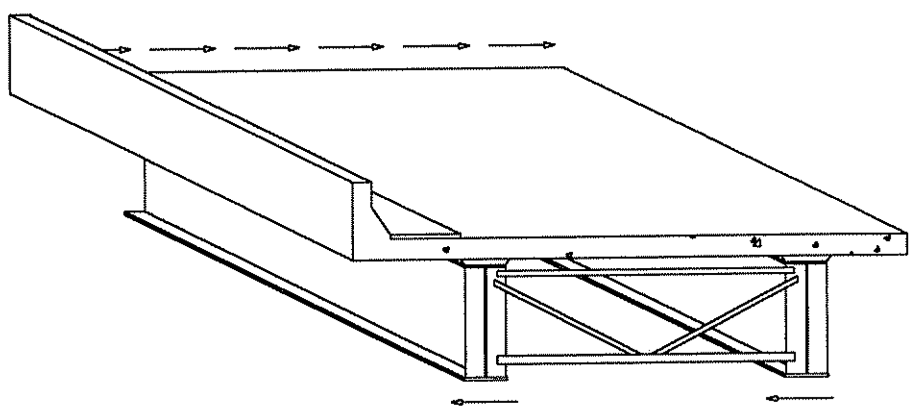


Figure 4.6 Wind load transmitted to deck at intermediate cross frame

(a) Overall free body



(b) Free bodies of end cross frame components

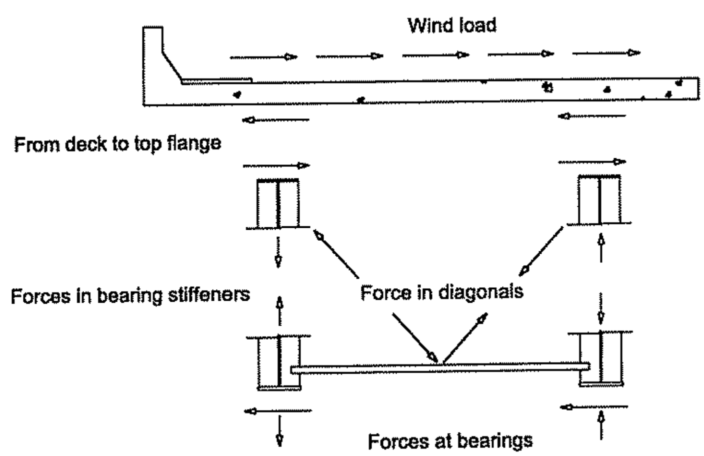


Figure 4.7 Wind load transmitted from the deck to the bearings by end cross frames

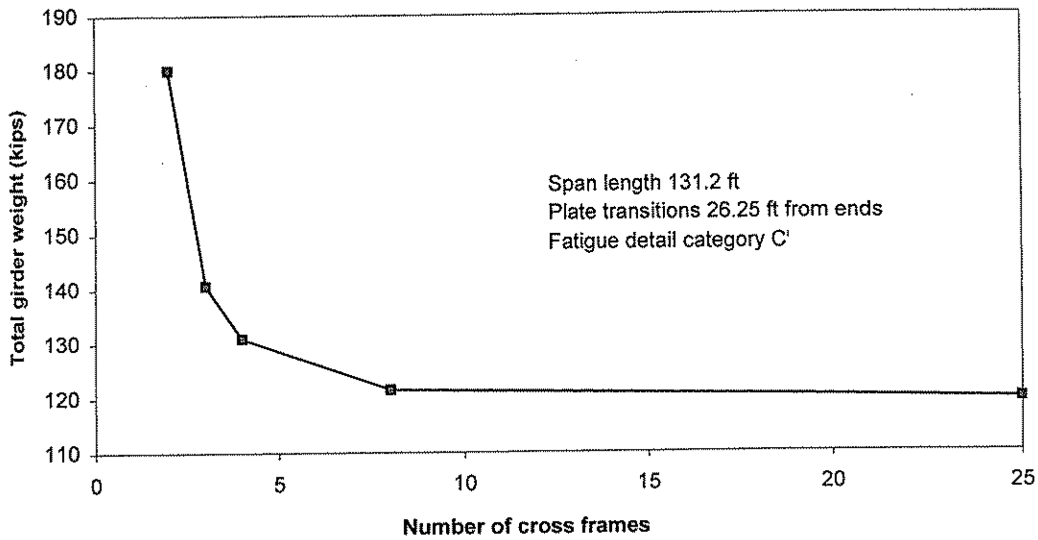


Figure 4.8 Influence of the number and corresponding spacing of cross frames on prototype bridge girder weight

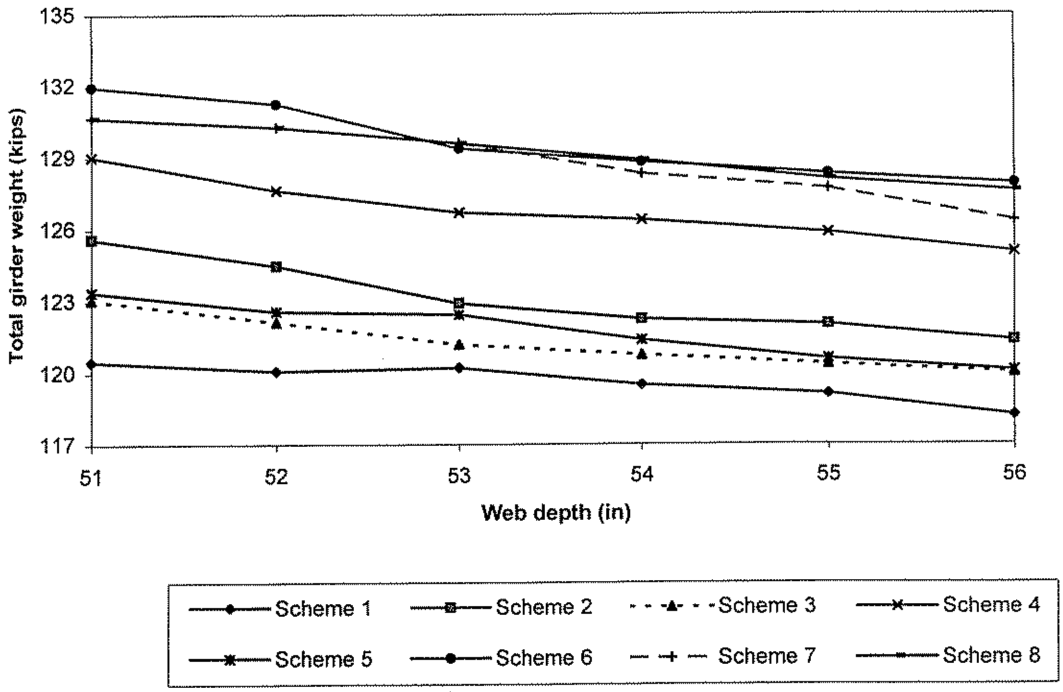


Figure 4.9 Influence of cross frame arrangement on girder weight

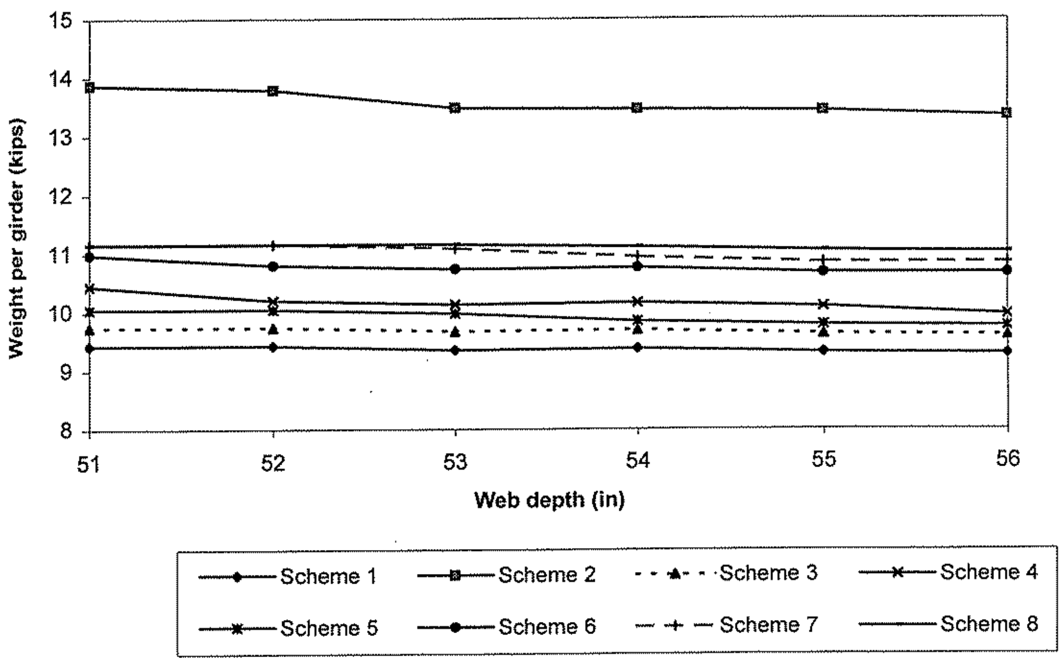


Figure 4.10 Influence of cross frame arrangement on the weight of the girder end segments

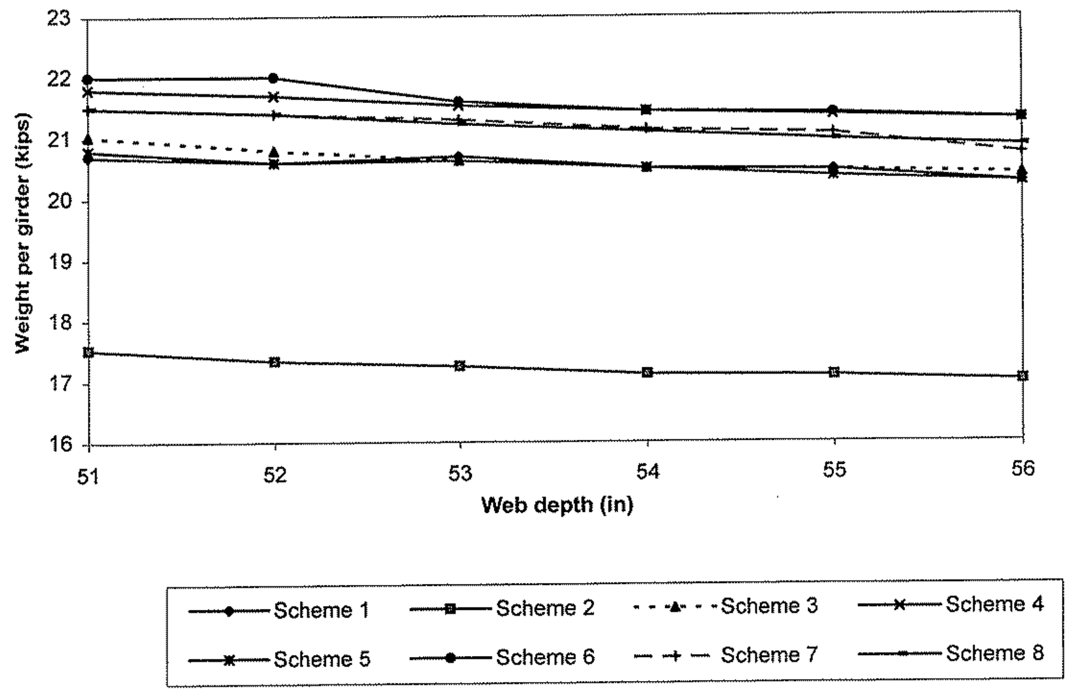


Figure 4.11 Influence of cross frame arrangement on the weight of the girder middle segment

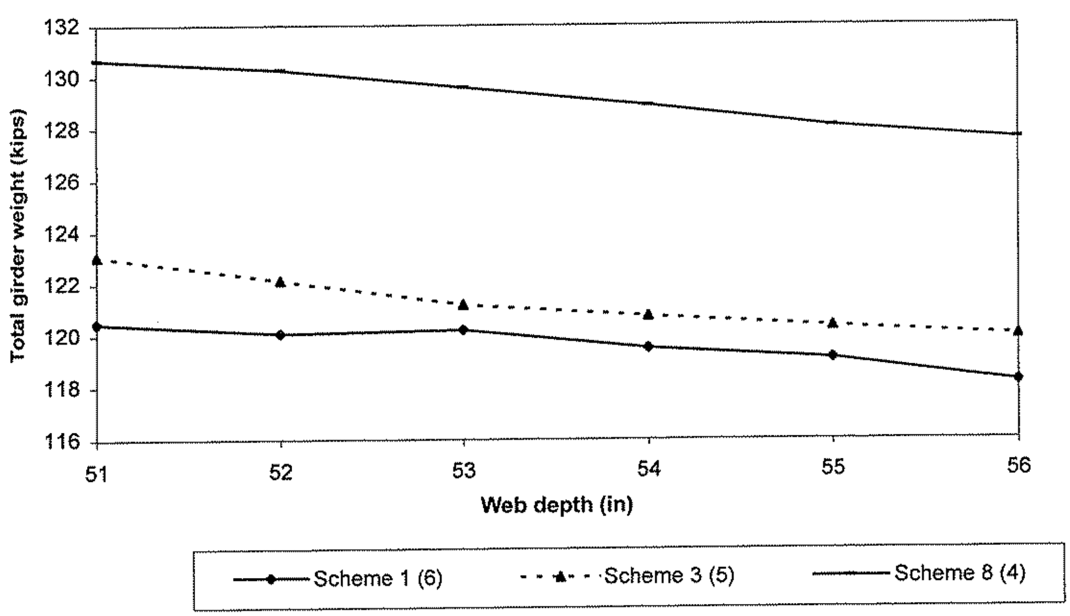


Figure 4.12 Girder weight for the best arrangements of 6, 5, and 4 cross frames

CHAPTER 5

Design Study of I-Girders Considering Web Shear Strength

A steel I-girder bridge is designed to resist both moment and shear under construction and service loads. The web of the girder carries most of the shear. This chapter presents a design study that investigates the shear design of the I-girders of the prototype bridge.

5.1 Background

Modern steel I-girder bridges are often designed with relatively stocky webs to reduce the need for intermediate transverse stiffeners. Older steel girder bridges often have relatively slender webs with many intermediate transverse stiffeners along the span between bearing stiffeners. The use of intermediate stiffeners to provide adequate shear resistance involves additional fabrication cost. However, the use of stocky webs involves additional material cost. To understand the use of stocky webs versus the use of thin webs, the shear resistance of an I-girder web is examined.

Figure 5.1 shows the shear resistance of a steel I-girder without transverse stiffeners. The nominal yield strength of the steel is 70 ksi (485 MPa). The figure shows that the shear resistance of an I-girder is heavily influenced by the web slenderness ratio, D / t_{web}, where D is the web depth and t_{web} is the web thickness. Three web slenderness ranges have been identified that determine the shear resistance of the web (AASHTO 1998):

range 1 $$\frac{D}{t_{web}} \le 2.46 \cdot \sqrt{\frac{E}{F_y}}$$

range 2 $$\frac{D}{t_{web}} > 3.07 \cdot \sqrt{\frac{E}{F_y}}$$

range 3 $$2.46 \cdot \sqrt{\frac{E}{F_y}} < \frac{D}{t_{web}} \le 3.07 \cdot \sqrt{\frac{E}{F_y}}$$

where, E is the modulus of elasticity, and F_y is the nominal yield stress of the web.

When the web slenderness is in range 1, the shear strength of the web is the shear yield stress $F_y / 3^{1/2}$ multiplied by the web area. The nominal shear resistance V_n is more commonly written as follows:

$$V_n = 0.58 \cdot F_y \cdot A_{web} \qquad \text{(Eq. 5.1)}$$

where, A_{web} is the area of the web equal to $D \cdot t_{web}$. As indicated in Figure 5.1, the nominal shear stress, V_n / t_{web}, is 40.6 ksi (280 MPa) for a 70 ksi (485 MPa) web when the web slenderness ratio is 50.1 or less.

If the web buckles before reaching the shear yield stress, the shear buckling may be either inelastic or elastic buckling. The shear stress at shear buckling is less than the shear yield stress.

When the web slenderness is in range 2, the shear resistance is controlled by elastic shear buckling. Figure 5.1 shows that the nominal shear resistance for a 70 ksi (485 MPa) web with a slenderness ratio of 62.5 and greater is as follows:

$$V_n = \frac{4.55 \cdot {t_{web}}^3 \cdot E}{D} \qquad \text{(Eq. 5.2)}$$

When the web slenderness is in range 3, the shear resistance is controlled by inelastic shear buckling. The slenderness limit between elastic buckling and inelastic buckling is the web slenderness when the elastic buckling stress is 80 percent of the shear yield stress at $D / t_{web} = 3.07 \cdot (E / F_y)^{1/2}$. The slenderness limit between inelastic buckling and yield is when the web slenderness is 80 percent of the $3.07 \cdot (E / F_y)^{1/2}$ limit at $D / t_{web} = 2.46 \cdot (E / F_y)^{1/2}$. When the web slenderness falls in range 3, the nominal shear resistance is as follows:

$$V_n = 1.48 \cdot t_{web}^2 \cdot \sqrt{E \cdot F_y} \qquad \text{(Eq. 5.3)}$$

The combination of all three ranges forms a shear resistance curve. There is a discontinuity in the shear resistance curve of the AASHTO LRFD bridge design specifications (AASHTO 1998) at the limit between range 1 (shear yield) and range 3 (inelastic shear buckling). The reason for this discontinuity appears to be that the nominal shear resistance calculated by Equation 5.3 and the web slenderness limits for range 3 were developed using a shear yield stress of $0.6 \cdot F_y$. Hence using Equation 5.1, where the shear yield stress equals $0.58 \cdot F_y$ or $F_y / 3^{1/2}$, results in a decrease in shear resistance at a web slenderness of 50.1.

The shear resistance given by Equations 5.1, 5.2 and 5.3 determines if, for any given shear stress demand, transverse stiffeners are needed. For example, if the shear stress demand is 20 ksi (138 MPa), the largest web slenderness ratio is 80 for a 70ksi (485 MPa) web without transverse stiffeners. Beyond this web slenderness ratio, transverse stiffeners are needed.

Appropriately spaced and sized transverse stiffeners can provide considerable post-buckling strength through tension field action. The post buckling strength can provide shear resistance that approaches the shear yield strength.

When stiffeners are used, the nominal shear strength is usually calculated as the sum of the shear buckling force and post buckling tension field force (Salmon and Johnson 1996). The post buckling strength contribution to the nominal shear resistance is controlled by two terms, the coefficient, C which is equal to the ratio of the shear buckling stress to the shear yield strength, and the ratio d_o / D, where d_o is the transverse stiffener spacing. The length of the web between transverse stiffeners is a stiffened web panel with increased shear resistance. Figure 5.2 shows a plot of shear resistance of a web panel considering post-buckling strength when the maximum moment in the web panel due to factored loads is less than $M_p / 2$ (AASHTO 1998). If the transverse stiffeners are more widely spaced, with $d_o / D = 1$, a smaller increase in shear resistance results. If the transverse stiffeners are more closely space, with d_o / D = 3/4, a larger increase in shear resistance is provided. When the maximum moment in the web panel due to factored loads is greater than M_p / 2, the AASHTO LRFD bridge design specifications require a moment-shear interaction factor to be considered (AASHTO 1998).

5.2 Influence of Web Thickness Changes

The previous section describes the influence of web slenderness on the shear resistance of an I-girder. It was shown that the use of appropriately spaced transverse stiffeners can increase the shear resistance. Another means of increasing the shear resistance is to increase the web thickness. An increased thickness will decrease the web slenderness ratio; and thus will increase the shear stress capacity when the shear stress is controlled by buckling. The increased thickness will also increase the web area which also increases the shear resistance.

An increased web thickness influences the design of steel I-girders in two ways (1) increasing the shear resistance may eliminate the need for transverse stiffeners, and (2) the weight of the steel girder may be increased.

A brief design study of I-girders designed for the prototype bridge was conducted to illustrate how increases in web thickness influence the I-girder weight. The web depth of the girders was set at 51 in (1295 mm) and a variable web thickness ranging from 1/16 in (2 mm) to 5/8 in (16 mm) was used.

Figure 5.3 shows that a web thickness 1/4 in (6 mm) or less results in a very large total girder weight for the prototype bridge. This results from heavy flanges needed to reduce compression bending stresses in the web to satisfy web slenderness criteria, as discussed below.

Girders designed with a web thickness of 3/8 in (10 mm) or less are controlled by the fatigue limit state and web slenderness criteria discussed below. Fatigue criteria require large tension flange dimensions such that the live load stress range does not exceed the nominal fatigue resistance (see Chapter 6). The top flange dimensions are controlled by web slenderness criteria, in particular, the web slenderness limit for non-compact sections (AASHTO 1998). The web slenderness limit for non-compact sections must be satisfied by the non-composite I-girder under construction loading. This limit is as follows:

$$\frac{2 \cdot D_c}{t_{web}} \leq 6.77 \cdot \sqrt{\frac{E}{f_c}} \qquad \text{(Eq. 5.4)}$$

where, D_c is the depth of web in compression, and f_c is the stress in the compression flange. The use of larger compression flange dimensions helps to satisfy this web slenderness limit as follows: (1) the steel centroid moves toward the compression flange reducing D_c, and (2) f_c is reduced, increasing the web slenderness limit. Increasing the web thickness decreases the importance of this web slenderness limit but this option may not be cost-effective.

Figure 5.3 shows that the total girder weight decreases as the web thickness increases up to a thickness of 3/8 in (10 mm). The reason for the weight decrease is that the decrease in flange area is significantly larger than the increase in web area. As the web thickness approaches 3/8 in (10 mm) the trade-off in flange area to web area becomes less favorable.

A web thickness greater than 3/8 in (10 mm) results in a slight increase in steel I-girder weight. This occurs because, as the web area increases, the compression flange area decreases until the compression flange area can no longer be reduced because the dimensions of the compression flange become controlled by lateral torsional buckling under construction loads. The tension flange dimensions cannot be reduced because they are controlled by the service II limit state.

Figure 5.3 shows that if lateral torsional buckling under construction loads and the service II limit state are not considered, the trend of decreasing girder weight with increasing web thickness would continue.

It appears that the optimal web thickness for the I-girders of the prototype bridge is 3/8 in (10 mm). The disadvantage of this web thickness is that the web shear resistance is less than the shear demand, unless intermediate transverse stiffeners are used. Figure 5.4 shows a plot of web thickness versus the number of transverse stiffeners needed for one girder to meet the shear demand. For a web thickness of 3/8 in (10 mm), approximately 20 transverse stiffeners per girder are needed. This significant number of stiffeners will impact the girder fabrication costs. A more economical choice may be to select a web thickness considering both the weight of the girders and the number transverse stiffeners. For example, the increase in girder weight from using a web thickness of 5/8 in (16 mm) may be economical when the decreased fabrication costs associated with a decrease in the number of transverse stiffeners is considered. A more thorough design study was conducted to investigate this.

5.3 Design Study of Web Thickness and Number of Transverse Stiffeners

This section describes a design study that investigates how web thickness and transverse stiffeners influence I-girder weight. The design study considers different webthicknesses for the girders of the prototype bridge resulting in variations in the need for transverse stiffeners to carry shear.

The web depth of the prototype bridges is varied from 51 in (1295 mm) to 56 in (1422 m). Three different arrangements of cross frames, called cross frame schemes 1, 3, and 8 in Chapter 4, are used.

Minimum weight girders for the prototype bridge were designed according to the procedure described in Chapter 3. Figure 3.6 shows the unfactored shear force demands for an exterior girder resulting from the dead and live loads acting on the bridge. These shear force demands were factored and superimposed to form a shear force demand envelope for the strength I limit state. The shear force demand envelope, shown in Figure 5.5, indicates the shear demand along the girder span. When the web is sufficiently thick so that the shear resistance (without stiffeners) is greater than the demand, stiffeners are not needed. For example, Figure 5.5 shows results for a web depth of 51 in (1295 mm) and a web thickness of 3/8 in (10 mm). If the shear resistance of an unstiffened web is less than the demand, then intermediate stiffeners are neccessary. However, if the web thickness is increased to 5/8 in (16 mm) the shear resistance of the unstiffened web exceeds the shear demand over the entire girder span, indicating intermediate stiffeners are not needed. For this reason, these cases are identified as the "stiffened" and "unstiffened" cases corresponding to the need for transverse stiffeners.

Another possibility is for the shear resistance of an unstiffened web to exceed the shear demand over a portion of the span. Figure 5.6 shows, for a web depth of 51 in (1295 mm) and a web thickness of 1/2 in (13 mm), the shear resistance is adequate at the midspan region but transverse stiffeners are needed near the bearings. This case is identified as "partially stiffened".

The stiffened case considered in the design study, shown in Figure 5.7(a), has a constant web thickness of 3/8 in (10 mm) throughout the girder span. Stiffeners are needed since the unstiffened shear resistance is significantly less than the shear demand.

The unstiffened case is shown in Figure 5.7(b). The girder has four plate transitions. The web transitions allow the web thickness to be set in each of the five segments so the unstiffened shear resistance exceeds the demand. It was assumed that the flange transitions occur at the same locations as the web transitions. Figure 5.8 explains how the locations of the web plate transitions and corresponding web thicknesses were chosen for a girder with a web depth of 51 in (1295 mm).

The partially stiffened case has two plate transitions as indicated in Figure 5.7(c). The locations of the transitions and the web thicknesses were chosen based on the unstiffened shear resistance. A significant difference between the partially stiffened and the unstiffened case is the elimination of two web and flange plate transitions in the middle segment of the girder. The web thickness in the middle segment provides an unstiffened shear resistance less than the shear demand for small portions of the middle segment. Therefore, transverse stiffeners are needed for a small portion of the middle segment. Depending on the cross frame locations, the cross frame connection plates may be located in the region where the shear demand exceeds the shear resistance, and, in this case, the connection plate can serve a dual purpose, as an intermediate transverse stiffener and as a connection of the cross frame to the girder. Figure 5.9 explains the partially stiffened case.

5.4 Design Study Results

Figure 5.10 shows the variation of the total girder weight when the web thickness is increased to reduce the need for intermediate transverse stiffeners. Figure 5.10 compares the partially stiffened case and the stiffened case when cross frame schemes 1, 3 and 8, with six, five and four cross frames, respectively, are used. An approximate 6.25 kip (28 kN) increase in steel girder weight occurs for all three cross frame arrangements when the web thickness is increased to reduce the need for stiffeners. Also, as discussed in Chapter 4, the steel weight increases as the number of cross frames decreases.

Figure 5.11 compares the stiffened, unstiffened, and partially stiffened cases for cross frame scheme 1 with six cross frames. As discussed previously, the stiffened case is significantly lower in weight by 6 to 7 kips (27 to 31 kN) than both the partially stiffened and unstiffened cases. The reason for the increased steel weight is shown in Figure 5.3. When the web thickness is increased from 3/8 in (10 mm) to provide greater shear resistance, the increase in web area is greater than the decrease in the flange area; hence, the girder cross-section area increases. Figure 5.11 also shows that the girder weight for the partially stiffened and unstiffened case is similar.

Figure 5.12 compares the stiffened, partially stiffened, and unstiffened cases for cross frame scheme 8 with four cross frames. The results are similar to those for scheme 1, except that the weight of the partially stiffened case is approximately 1 percent greater than the weight of the unstiffened case. This is related to the location of the plate transitions and the cross frame spacing between the end and intermediate cross frames. When a plate transition occurs within an unbraced length, the cross-section properties from the end and middle segments are combined to determine the lowest calculated lateral torsional buckling moment resistance (see Chapter 7). The cross-section dimensions used to determine the lateral torsional buckling moment resistance for the outer unbraced length are taken from the compression flange dimensions of the middle segment. Therefore the increased flange dimensions for the 78.74 ft (24 m) long middle segment BC of the partially stiffened case (see Figure 5.7) produce a heavier girder than the increased flange dimensions for the 27.89 ft (8.5 m) long middle segment BC of the unstiffened case.

5.5 Summary

The results in this chapter show the web thickness influences the design of steel I-girders in two ways: (1) increasing the shear resistance may eliminate the need for transverse stiffeners, and (2) the weight of the steel girder may be increased. Girder designs for the prototype bridge with a web depth of 51 in (1295 mm) and a web thickness up to 3/8 in (10 mm) show a decrease in steel girder weight. As the web thickness is increased beyond 3/8 in (10 mm), the girder weight increases due to an unfavorable trade-off between the web and flange area. Girders with slender webs may have a shear resistance less than the shear demand unless intermediate transverse stiffeners are used. The need for intermediate transverse stiffeners increases girder fabrication costs. Girders designed with stocky webs will have an increase in girder weight, but the need for intermediate transverse stiffeners may be avoided.

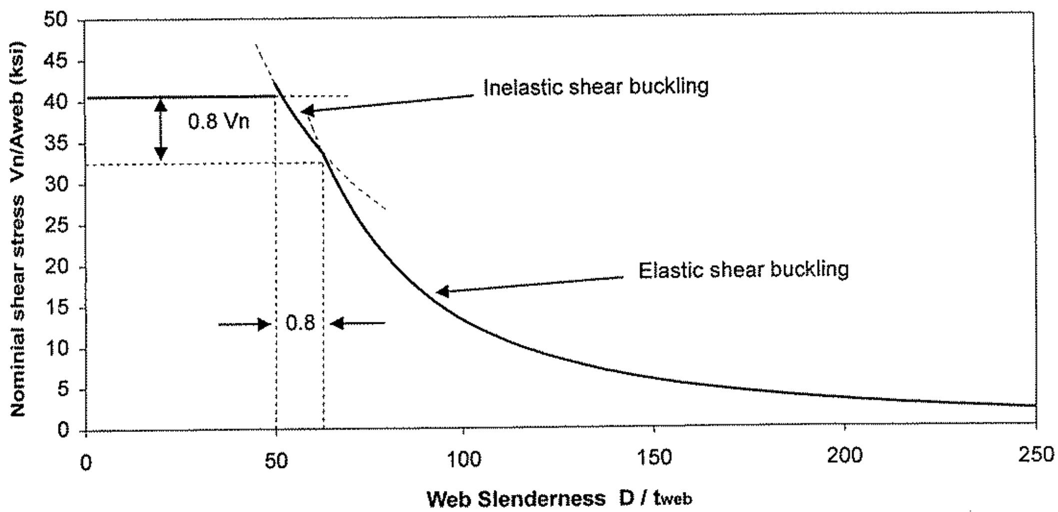


Figure 5.1 Shear resistance of 70 ksi (485 MPa) girders without stiffeners

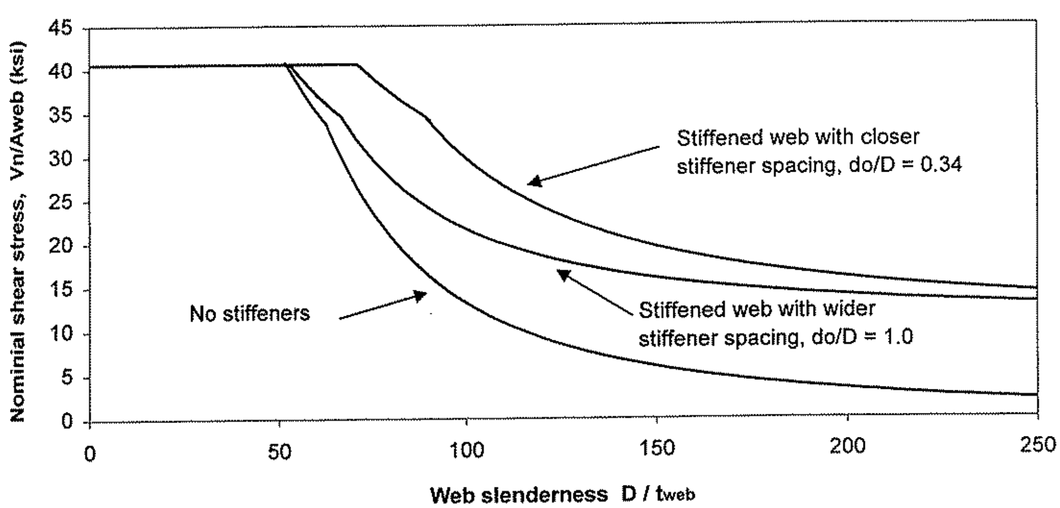


Figure 5.2 Shear resistance of 70 ksi (485 MPa) girders with stiffeners

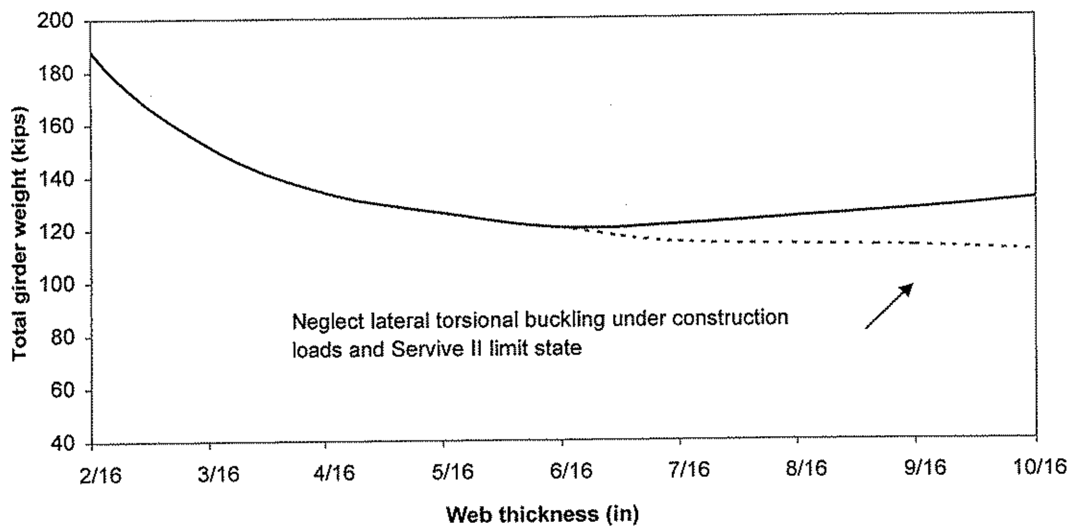


Figure 5.3 Influence of web thickness on prototype bridge girder weight

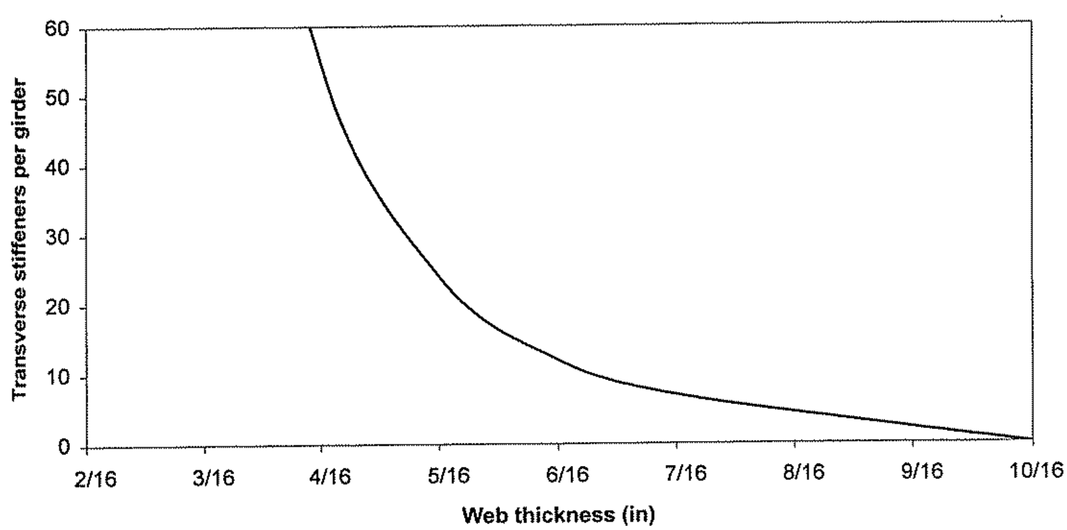


Figure 5.4 Influence of web thickness on the need for stiffeners on prototype bridge girders

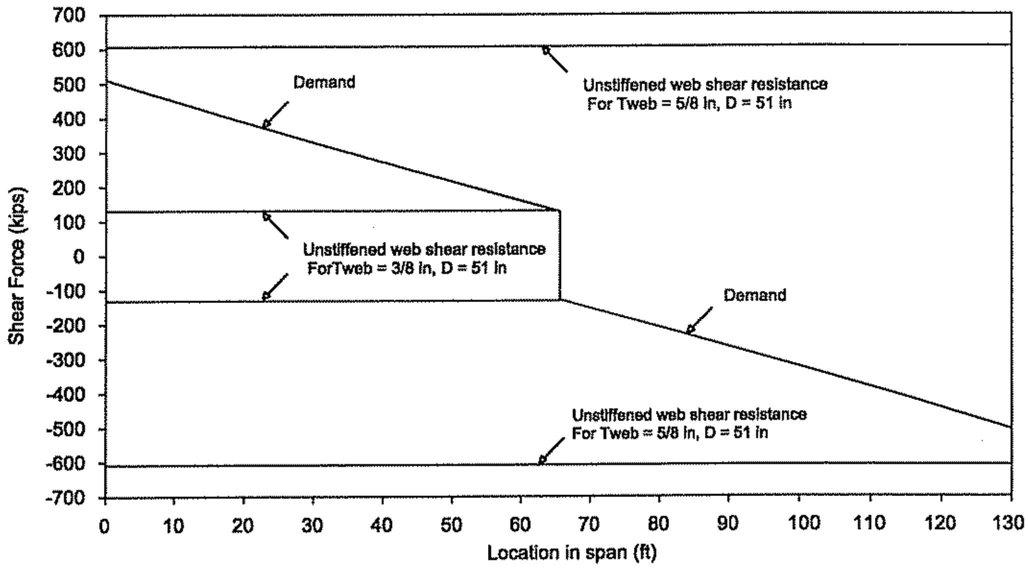


Figure 5.5 Strength I limit state shear force demand envelope and unstiffened web shear resistance

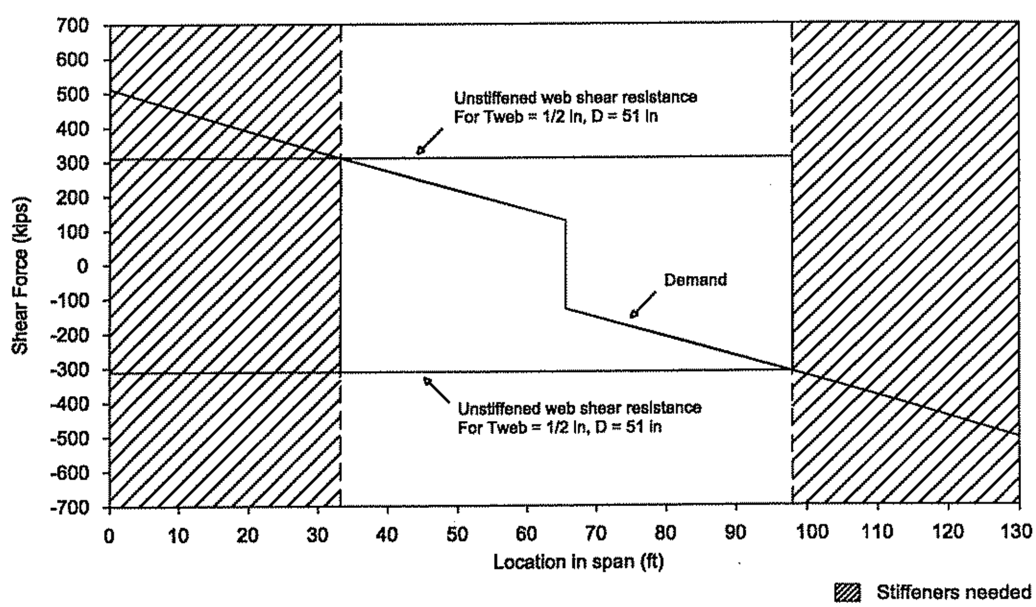


Figure 5.6 Strength I limit state sheare force demand envelope and partially stiffened option

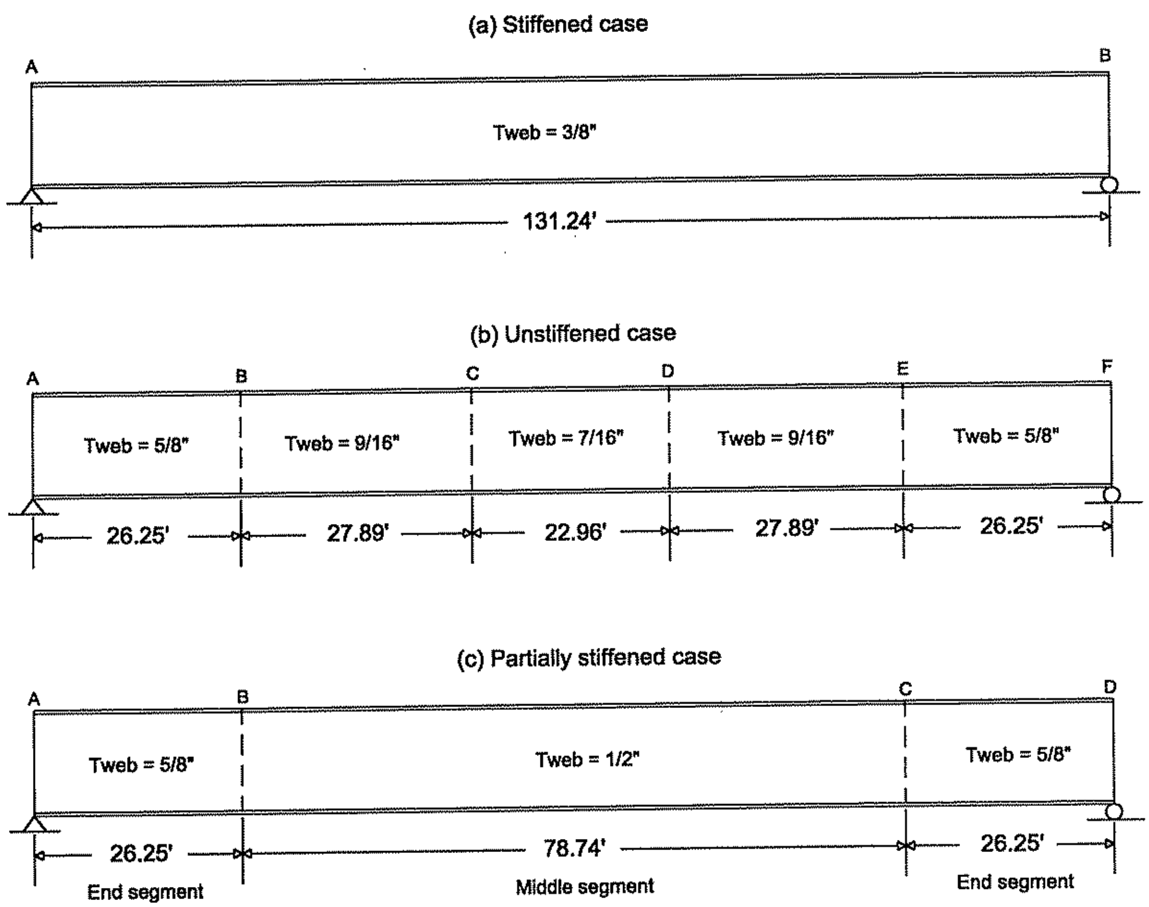


Figure 5.7 Classification of girders designed with stiffened, unstiffened, and partially stiffened webs

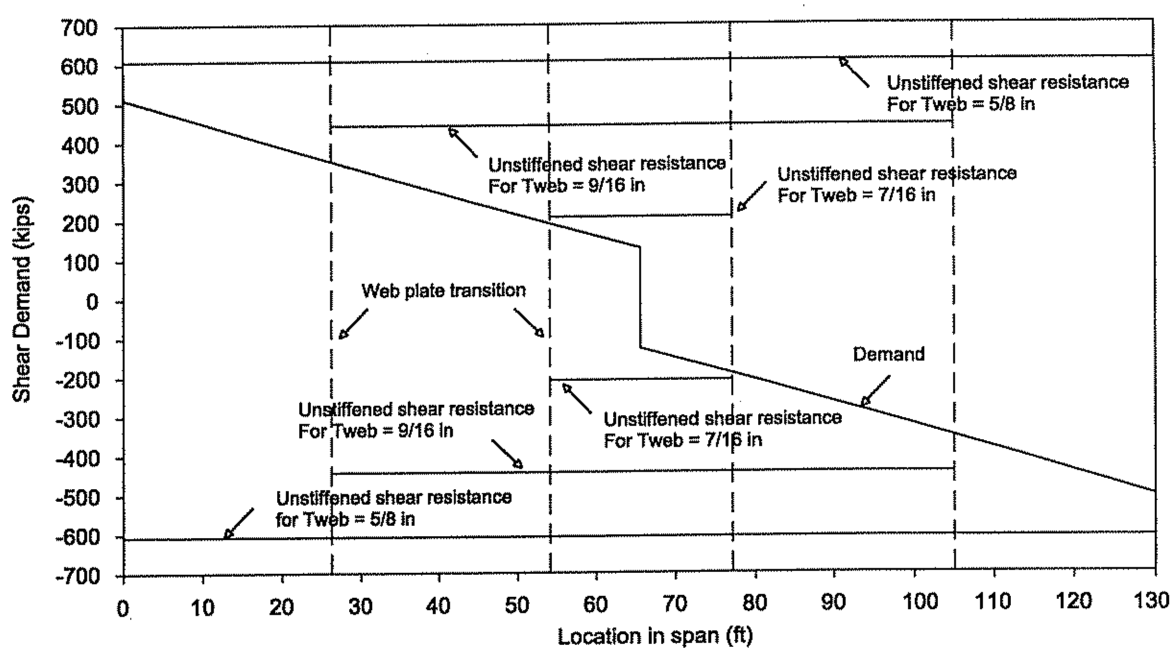


Figure 5.8 Description of unstiffened case for a web depth of 51 in (1295 mm)

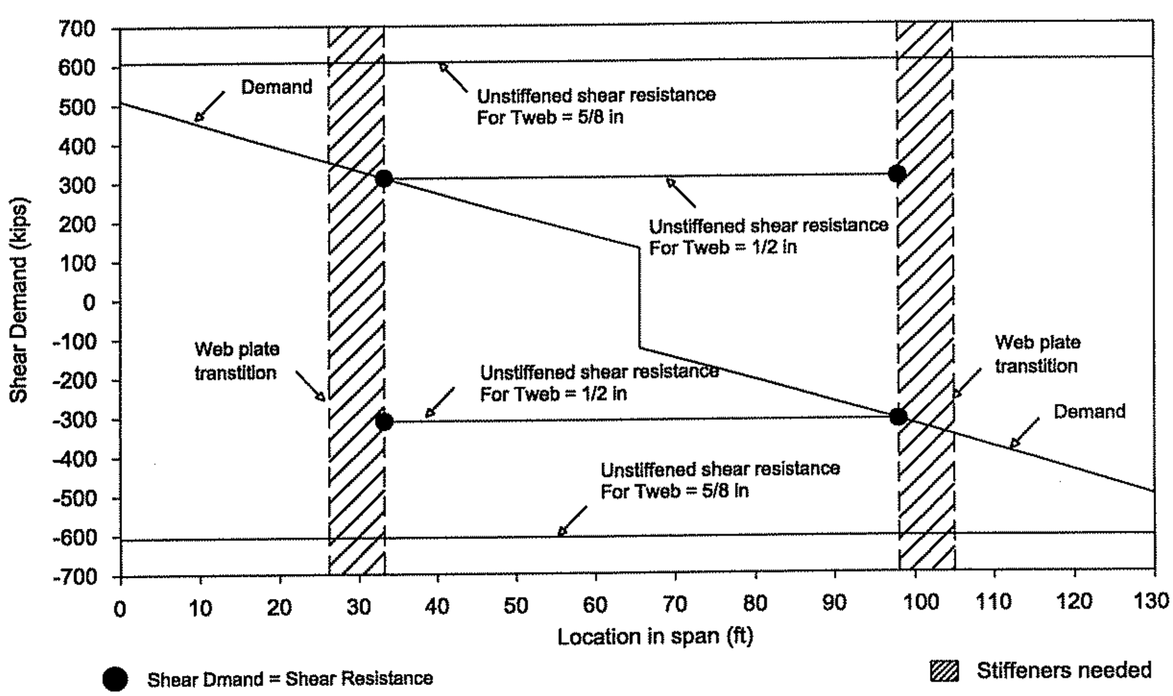


Figure 5.9 Description of partially stiffened case for a web depth of 51 in (1295 mm)

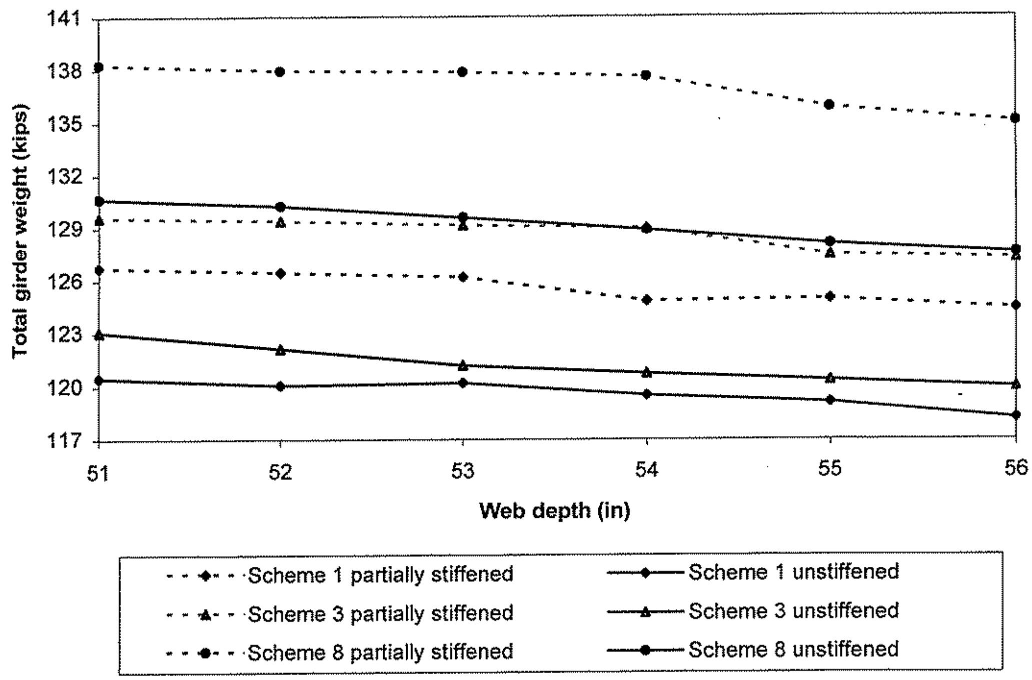


Figure 5.10 Influence of the use of stiffeners for shear strength on girder weight

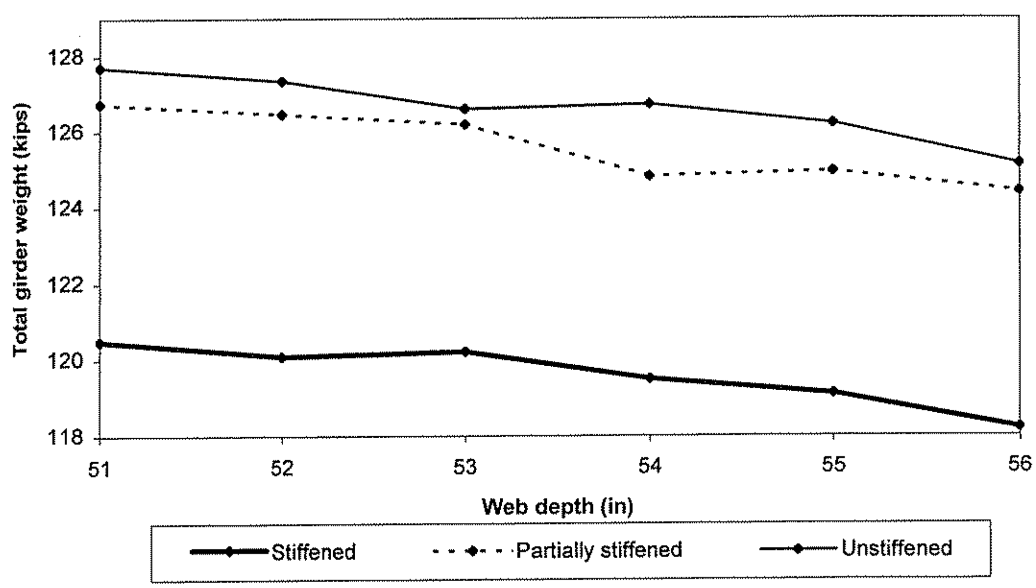


Figure 5.11 Comparison of total girder weight for cross frame scheme 1 with stiffened, partially stiffened and unstiffened webs

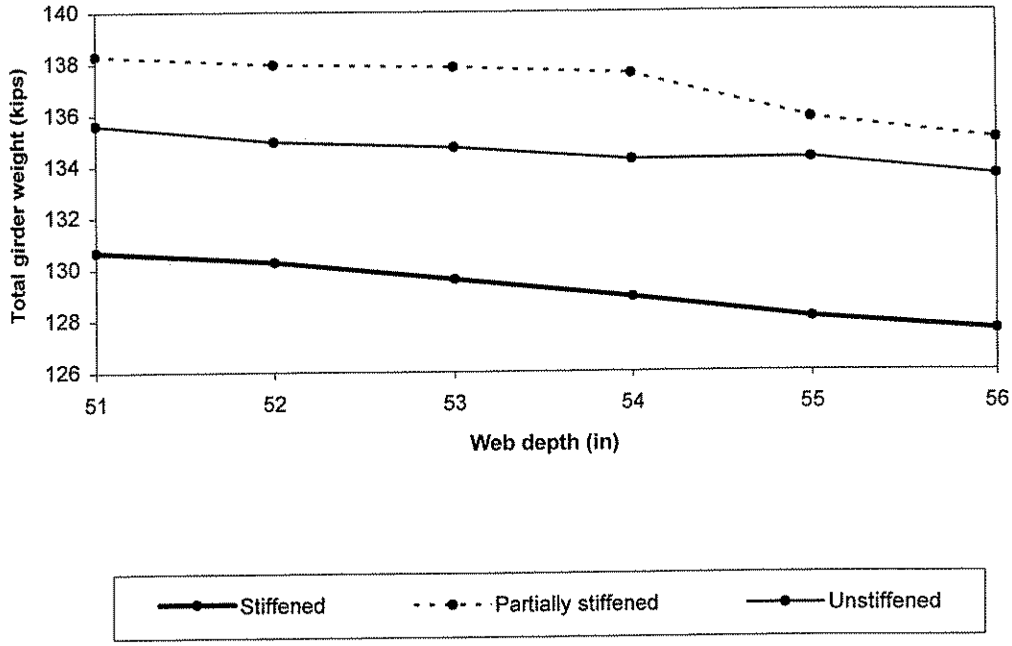


Figure 5.12 Comparison of total girder weight for cross frame scheme 8 with stiffened, partially stiffened and unstiffened webs

CHAPTER 6

Design Study of I-Girders Considering Upgraded Fatigue Resistance

Fatigue of steel I-girder bridges results from repeated stress cycles caused by vehicular live loads, in particular, truck loads. The fatigue resistance of a steel I-girder bridge is influenced primarily by the connection details. This chapter presents a design study that investigates the influence of the fatigue resistance of the connection details on the design of I-girders for the prototype bridge.

6.1 Fatigue in Steel Bridges

Highway bridge girder design considers both load-induced and distortion-induced fatigue (Fisher et al., 1997). Load–induced fatigue results from repeated loading by loads smaller than those required for failure under static loads. Load-induced fatigue is related to vehicular live loads, and depends heavily on the connection details used in the bridge.

Distortion-induced fatigue results from stresses induced by unequal deflections and deformations of adjacent bridge superstructure components (e.g. adjacent girders) under live loads. For example, when adjacent I-girders have different deflections, the cross frames between the girders will develop forces that can produce local distortion of the webs of the girders near the cross frame connections.

6.2 Fatigue Design Approach

The fatigue design approach begins by identifying specific members and connection details that require a fatigue check. For example, the top flange of a simple span bridge girder that is subjected to only compression stress under live loads is not checked for fatigue. However, the bottom flange, which is always in tension should be checked for fatigue.

Fatigue resistance is governed by the stress range at the location of connections and the type of connection detail. The AASHTO LRFD bridge design specifications (AASHTO 1998) provide nominal fatigue resistance for various types of connection details. There are eight connection detail categories ranging from A to E'. Each category has an associated fatigue resistance as shown in Figure 6.1. When a fatigue-sensitive connection detail is checked for fatigue, the associated S-N curve is used to determine the nominal fatigue resistance as discussed below.

For a steel highway bridge I-girder designed without cover plates or other unusual attachments, the fillet-welded connections of transverse stiffeners/cross frame connection plates to the girder are often the most critical fatigue sensitive connection detail. This is a category C' detail which include fillet-welded connections with welds perpendicular to the direction of primary stress. Figure 6.2 shows a typical Category C' fatigue detail commonly found on steel bridge I-girders.

The fatigue resistance of each fatigue-sensitive connection detail is checked against the stress range produced by the "fatigue truck" as it passes over the bridge. The "fatigue truck" in the AASHTO LRFD bridge design specifications (AASHTO 1998) is based on the HS-20 truck as discussed in Chapter 3. The primary stress in the element (e.g., girder flange or web) adjacent to the connection detail is checked for stresses resulting from the HS-20 truck with the load factor of 0.75 (i.e., the fatigue load) as shown in Table 3.2.

When designing for load induced-fatigue each connection detail must satisfy the following requirement:

$$\Delta f \leq (\Delta F)_n \qquad \text{(Eq. 6.1)}$$

where, $(\Delta F)_n$ is the nominal fatigue resistance and Δf is the live load stress range due to the fatigue load. The nominal fatigue resistance is determined as follows:

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}} \geq \frac{1}{2} \cdot (\Delta F)_{TH} \qquad \text{(Eq. 6.2)}$$

where, A is a constant depending on the fatigue detail category, N is the number of cycles for a 75-year design life, and $(\Delta F)_{TH}$ is the constant amplitude fatigue threshold (CAFL) for the fatigue detail category. Calculation of the number of cycles, N is determined as follows:

$$N = 365 \cdot 75 \cdot n \cdot ADTT_{SL} \qquad \text{(Eq. 6.3)}$$

where, $ADTT_{SL}$ is the single lane average daily truck traffic which is an estimate of the number of trucks per day in a single lane, and n is the number of stress cycles per truck passage taken as 1.0 for simple span girders spanning over 40 ft (12.2 m).

The lines on the S-N curve with negative slope are based on the term $(A / N)^{1/3}$ in Equation 6.2. These lines indicate the stress range and number of cycles at which fatigue failure occurs for fatigue detail categories A through E′. The horizontal lines on the S-N curve indicate the CAFL for Categories A through E′. If a connection detail is to provide infinite fatigue life, the stress range due to the fatigue load must be less than or equal to one-half the CAFL. For example, if the maximum stress range is 20 ksi (138 MPa) for a Category B fatigue detail, the fatigue life is approximately 800,000 cycles. If the stress range is 7 ksi (48.3 MPa), which falls below one-half the CAFL (i.e., ½ of 16 ksi (110 MPa) equals 8 ksi (55.2 MPa)) for a Category B fatigue detail, the detail will have infinite fatigue life.

When designing for distortion-induced fatigue, flexure and shear requirements should be satisfied. These requirements address the out-of-plane distortion of the web due to flexure or shear under live loads. For flexure the following expression must be satisfied:

$$\text{if} \quad \frac{2 \cdot D_c}{t_{web}} \leq 5.70 \cdot \sqrt{\frac{E}{F_{yw}}}, \qquad \text{then} \quad f_{cf} \leq F_{yw} \qquad \text{(Eq. 6.4)}$$

$$\text{if} \quad \frac{2 \cdot D_c}{t_{web}} > 5.70 \cdot \sqrt{\frac{E}{F_{yw}}}, \qquad \text{then} \quad f_{cf} \leq 32.5 \cdot E \cdot \left(\frac{t_{web}}{2 \cdot D_c}\right)^2 \qquad \text{(Eq. 6.5)}$$

where, D_c is the depth of web in compression and f_{cf} is the bending stress in the compression flange due to the unfactored fatigue load (no impact factor) and dead load. For shear the following expression must be satisfied:

$$V_{cf} \leq 0.58 \cdot C \cdot F_{yw} \qquad \text{(Eq. 6.6)}$$

where, V_{cf} is the maximum shear stress in the web due to the unfactored fatigue load (no impact factor) and dead load and C is the ratio of the shear resistance (as discussed in Chapter 5) to the shear yield strength $(0.58 \cdot F_{yw})$.

6.3 Advantages of Using Connection Details with Upgraded Fatigue Resistance

Figure 6.1 shows that as the fatigue detail category increases from E' to A, the fatigue resistance increases, and thus, the element adjacent to the connection detail can be subjected to a higher stress range by the fatigue load. As noted in Section 6.2, the fillet-welded connection between a transverse stiffener/cross frame connection plate and an I-girder flange or web is a Category C' fatigue detail. If this connection detail can be upgraded from Category C' to a Category B, a savings in girder weight can be achieved (Sause and Fisher 1995).

The transverse stiffener/cross frame connection plate connection detail can be upgraded from a Category C' to a Category B by using a bolted connection or other connection scheme. Figure 6.3 and Figure 6.4 show alternative Category B fatigue details.

The use of a connection detail with greater fatigue resistance for the transverse stiffener/cross frame connection plate may results in a savings in girder weight because the design of girders made of 70 ksi (485 MPa) with Category C' details is often governed by the fatigue limit state. Table 6.1 summarizes the design of an I-girder for the prototype bridge. The performance ratio (discussed in Chapter 3) for the fatigue limit state at the location of the stiffener/cross frame connection plate is 1.01 for both the middle and end segments of the girders. This shows the fatigue limit state is controlling the dimensions of the girder bottom flange. If Category B fatigue details are used, the fatigue limit state no longer controls the girder design, as shown in Table 6.2. Rather the service II limit state controls the design and a decrease in the dimensions of the girder bottom flange is possible. Figure 6.5 shows that an approximate 6.5 kip (28.9 kN) decrease in girder weight is achieved for this example.

The advantage of reducing the steel girder weight can be offset by the cost of upgrading the fatigue detail. Therefore the use of Category B fatigue details should be limited to regions of a girder where the bending moment due to the fatigue load is high. The cost of the Category C' fatigue detail shown in Figure 6.2 has been estimated at \$6 per stiffener/cross frame connection plate, while the cost of the Category B details shown in Figures 6.3 and 6.4 have been estimated at \$69 and \$88 per connection, respectively (Huzzard and Montgomery 1996).

6.4 Design Study Description

A design study has been conducted to investigate the advantages of using upgraded fatigue details. The study is based on the prototype bridge discussed in Chapter 3. The girder designs are generated using the procedure presented in Chapter 3. The decrease in girder weight from using Category B fatigue details rather than Category C' details will be shown. As noted earlier, the connection detail of concern is the attachment of the transverse stiffener/cross frame connection plate to the girder bottom flange. In the design study, the fatigue limit state is checked at the locations of the cross frames and transverse stiffeners.

For simplicity only the cross frame arrangements called scheme 1, scheme 3 and scheme 8 in Chapter 4 are included in the study.

6.5 Design Study Results

Figure 6.6 compares the total girder weight for the prototype bridge when the girders are designed with Category C′ fatigue details and Category B fatigue details. Decreases in girder weight are achieved for each cross frame arrangement.

Figure 6.7 shows the decreases in the total girder weight for theprototype bridge for cases in which the web is stiffened (as discussed in Chapter 5). Results for cross frame schemes 1, 3 and 8 are shown. For cross frame scheme 1, the decrease in girder weight is largest, approximately 7 kips (31 kN). For cross frame schemes 3 and 8, the decrease in girder weight is smaller, approximately 6 kips (26 kN). The reason for the larger decrease when cross frame scheme 1 is used is as follows. When a Category C′ fatigue detail is used web slenderness criteria (under construction loading) control the dimensions of the top flange (Chapter 5). To satisfy the web slenderness limit, the top flange dimensions are increased to reduce the depth of the web in compression and the stress in the compression flange (see Equation 5.4). The fatigue limit state controls the dimensions of the bottom flange where increasing the bottom flange dimensions reduces the stress range due to the fatigue load. Using Category B fatigue details allows for a reduction in the girder bottom flange dimensions moving the girder centroid toward the top flange and decreasing the depth of the web in compression. This makes the web slenderness limit less critical. Therefore, the top flange dimensions can also be reduced. This is the situation whenthe top flange dimensions are not controlled by lateral torsional buckling.

When cross frame schemes 3 and 8 are used, the girder top flange dimensions are controlled by lateral torsional buckling under construction loading. That is, lateral torsional buckling controls when the spacing between cross frames (i.e., the girder unbraced length during construction) is substantially increased.

The decrease in the weight of the end and middle segments of the girders for each cross frame arrangement is presented in Figure 6.8. This figure indicates greater reductions occur in the middle segment where the fatigue moment demand is high. Therefore, theuse of Category B fatigue details in this segment results in greater weight savings.

Figure 6.9 shows how decreases in total girder weight from using upgraded fatigue details are influenced when the web thickness is increased. Results for unstiffened cases (as discussed in Chapter 5) are shown in the figure. Figure 6.9 shows decreases in girder weight that are less than those shown in Figure 6.7 for the stiffened cases. The reduced weight savings results from an unfavorable trade-off between the web area and the flange areas. For example, to produce weight savings similar to the stiffened cases,the increased web area of the unstiffened cases should be offset by reduced flange areas. However, the fatigue design requirements, design for construction conditions (in particular, design against lateral torsional buckling) and other requirements prevent the flange area from decreasing as the web area is increased for the unstiffened cases.

For the unstiffened web cases, cross frame schemes 1 and 8 have larger weight decreases than cross frame scheme 3. This is influenced by the location where the fatiguelimit state is checked. If connection details are at locations along a girder segment where the fatiguelimit state governs, then larger decreases in weight are obtained by using upgraded fatigue details. For example, Figure 6.10 shows the cross frame arrangement for scheme 1 with connection plates at 26.25 ft (8 m) intervals. The fatigue limit state governs the dimensions at the bottom flange of both the end segment and middle segment 2 because connection plates fall within these segments. Therefore using a Category B fatigue detail within these segments results in weight decreases throughout a significant portion of the girder length.

6.6 Summary

This chapter presents the results of a design study using connection details with upgraded fatigue resistance in the prototype bridge. The connection of the transverse stiffener/cross frame connection plate to the girder bottom flange is the fatigue sensitive detail that the study focused on. If this fatigue detail is upgraded from Category C′ to a Category B decreases in girder weight resulting from reduced bottom flange dimensions can be achieved. This result is limited to girders with the bottom flange dimensions governed by the fatigue limit state. When the dimensions of the top flange are not governed by lateral torsional buckling under construction loading, decreases in both the top and bottom flange weight can be achieved by upgrading the fatigue details from Category C′ to Category B. The advantage of reducing the girder weight can be offset by the cost of upgrading the fatigue detail. Therefore the use of Category B fatigue details should be limited to regions of a girder where the bending moment due to the fatigue load is high.

Table 6.1 Design summary for prototype bridge girder with cross frame scheme 1 and category C ' fatigue details

(a) Cross section properties

Dimensions	end section	mid section
Length (ft)	26.25	78.75
Ttf (in)	0.875	1.125
Btf (in)	14.0	19.0
Tbf (in)	1.125	1.750
Bbf (in)	19.0	21.0
Tweb (in)	0.375	0.375
Dweb (in)	51.0	51.0
Atf (in^2)	12.25	21.38
Abf (in^2)	21.38	36.75
Aweb (in^2)	19.13	19.13
Total (in^2)	52.75	77.25
Span-depth ratio	29.9	29.4

(b) Performance ratios

Limit States and Criteria	end section	mid section
Strength Limit State		
Flexure Resistance	0.95	0.88
Ductility Requirement	0.97	0.95
Shear Resistance	3.75	2.55
Construction		
Web Slenderness	0.98	1.00
Compression Flange Slenderness	0.85	0.90
Compression Flange Bracing	2.48	1.74
Flexure Resistance	0.82	0.93
Service II Limit State		
Perm. Deflection (tension flange)	0.94	0.94
Fatigue Limit State		
Bottom flange connection plate weld	1.01	1.01

Nomenclature

Ttf = top flange thickness
Btf = top flange width
Tbf = bottom flange thickness
Bbf = bottom flange width
Tweb = web thickness
Dweb = web depth

Loads (kip-ft) and section location (ft)	end section	mid section
Section location	26.25	65.62
Component Dead Load, Dc	2755	4305
Wearing Surface Load, Dw	690	1075
Live Load + Impact, LL+IM	3170	5090
Fatigue Live Load	1170	1810

Table 6.2 Design summary for prototype bridge girder with cross frame scheme 1 and Category B fatigue details

(a) Cross section properties

Dimensions	end section	mid section
Length (ft)	26.25	78.75
Ttf (in)	0.875	1.125
Btf (in)	13.0	19.0
Tbf (in)	1.5	2.0
Bbf (in)	13.0	17.0
Tweb (in)	0.375	0.375
Dweb (in)	51.0	51.0
Atf (in^2)	11.38	21.38
Abf (in^2)	19.50	34.00
Aweb (in^2)	19.13	19.13
Total (in^2)	50.00	74.50
Span-depth ratio	29.7	29.3

Nomenclature

Ttf = top flange thickness

Btf = top flange width

Tbf = bottom flange thickness

Bbf = bottom flange width

Tweb = web thickness

(b) Performance ratios

Limit States and Criteria	end section	mid section
Strength Limit State		
Flexure Resistance	0.99	0.92
Ductility Requirement	1.01	1.01
Shear Resistance	3.75	2.55
Construction		
Web Slenderness	1.00	0.99
Compression Flange Slenderness	0.81	0.90
Compression Flange Bracing	2.70	1.74
Flexure Resistance	1.01	0.93
Service II Limit State		
Perm. Deflection (tension flange)	0.94	1.01
Fatigue Limit State		
Bottom flange connection plate weld	0.83	0.84

Loads (kip-ft) and section location (ft)	end section	mid section
Section location	26.25	65.62
Component Dead Load, Dc	2755	4305
Wearing Surface Load, Dw	690	1075
Live Load + Impact, LL+IM	3170	5090
Fatigue Live Load	1170	1810

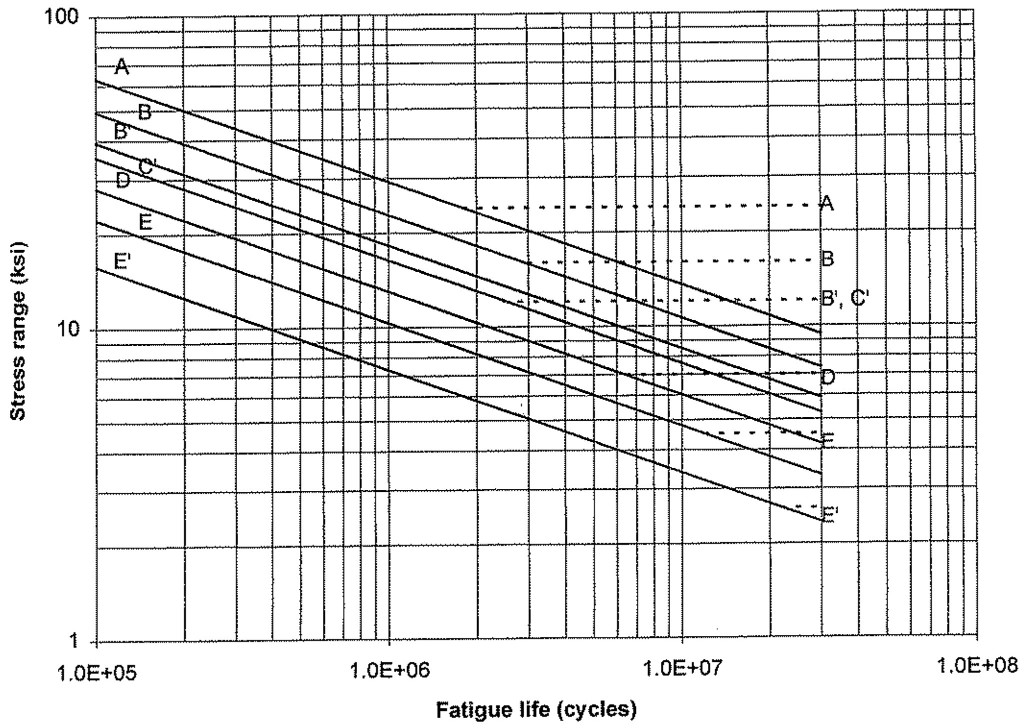


Figure 6.1 S-N curve (AASHTO LRFD bridge design specifications 1998)

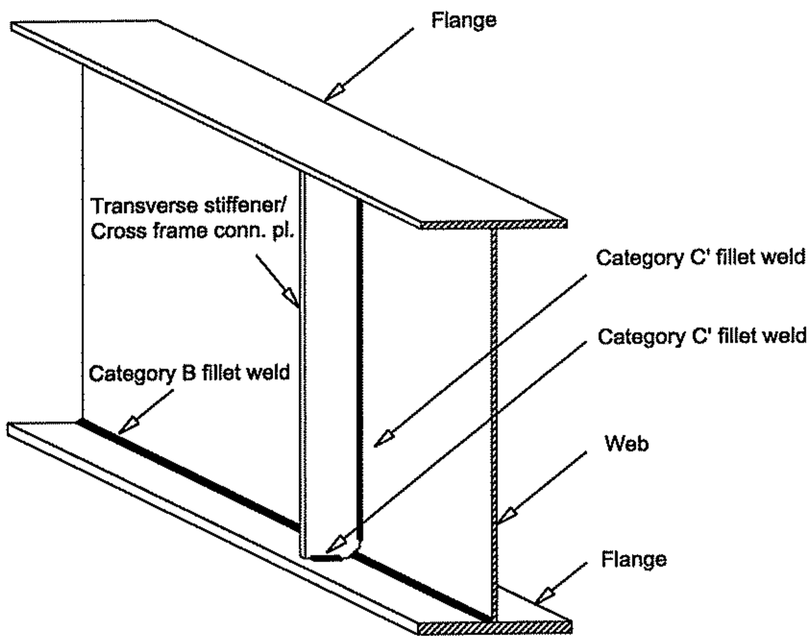


Figure 6.2 Typical Category C' fatigue detail

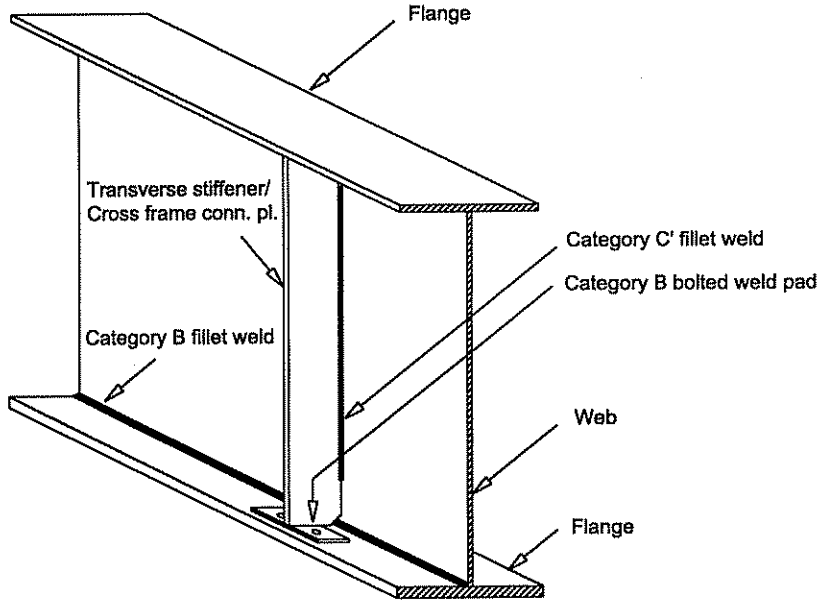


Figure 6.3 Category B fatigue detail with welded and bolted connection
(Bethelehem Steel Corporation 1996)

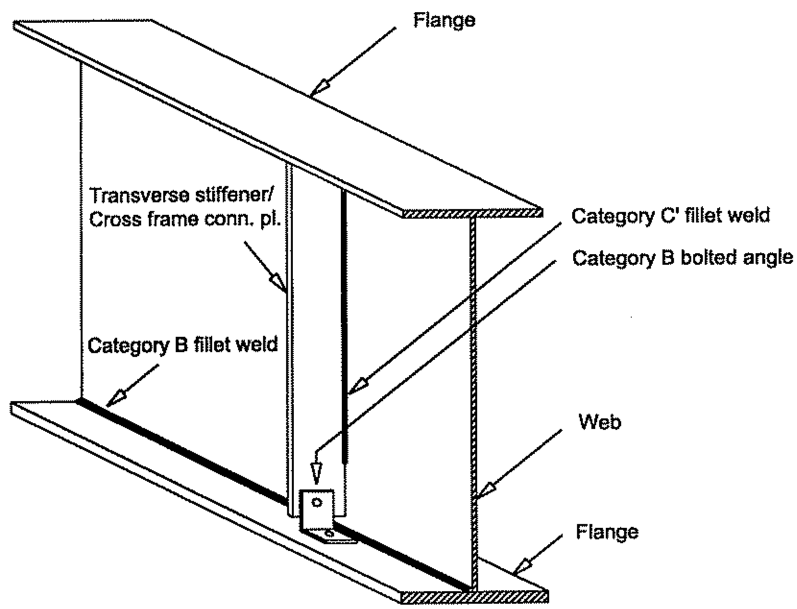


Figure 6.4 Category B fatigue detail with bolted connection
(Bethelehem Steel Corporation 1996)

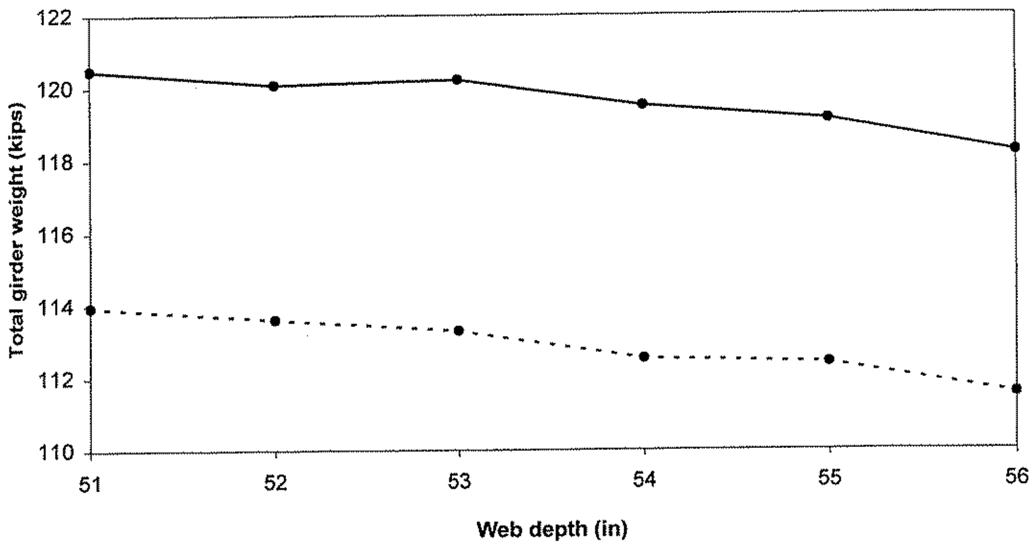


Figure 6.5 Influence of fatigue details on prototype bridge girder weight

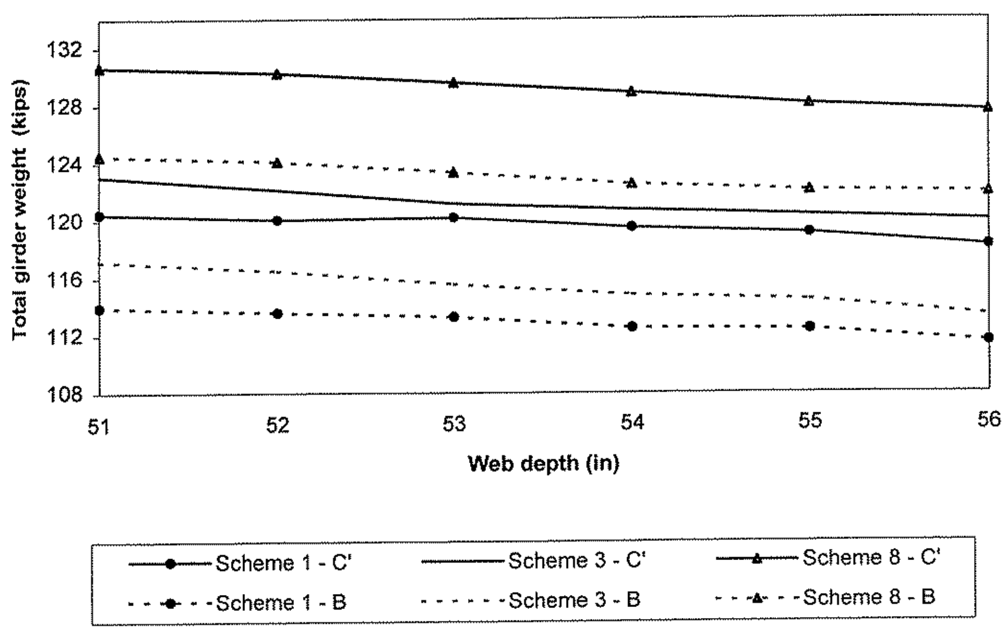


Figure 6.6 Girder weight using Category C ' and Category B fatigue details

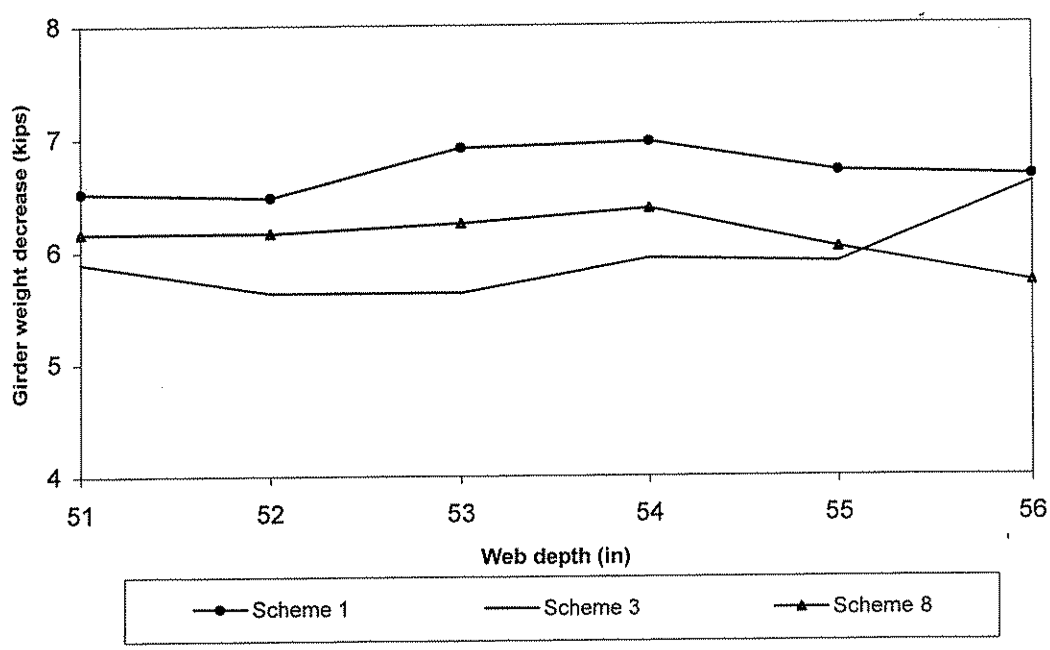


Figure 6.7 Decrease in total girder weight using upgraded fatigue details for the stiffened web cases

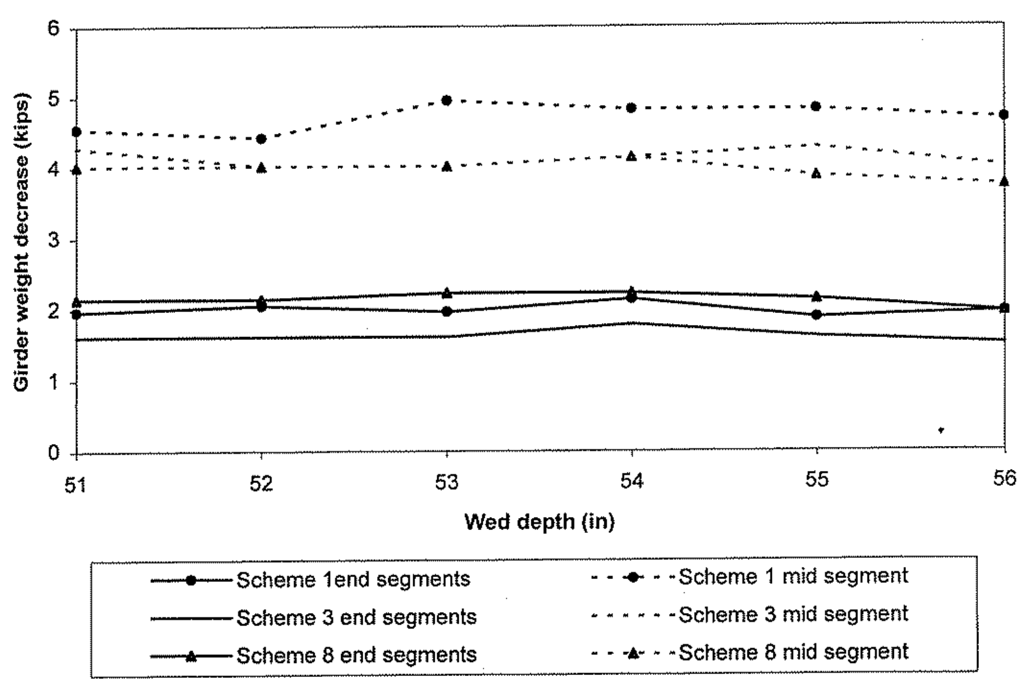


Figure 6.8 Decrease in weight of the end and middle segments using upgraded fatigue details for stiffened web cases

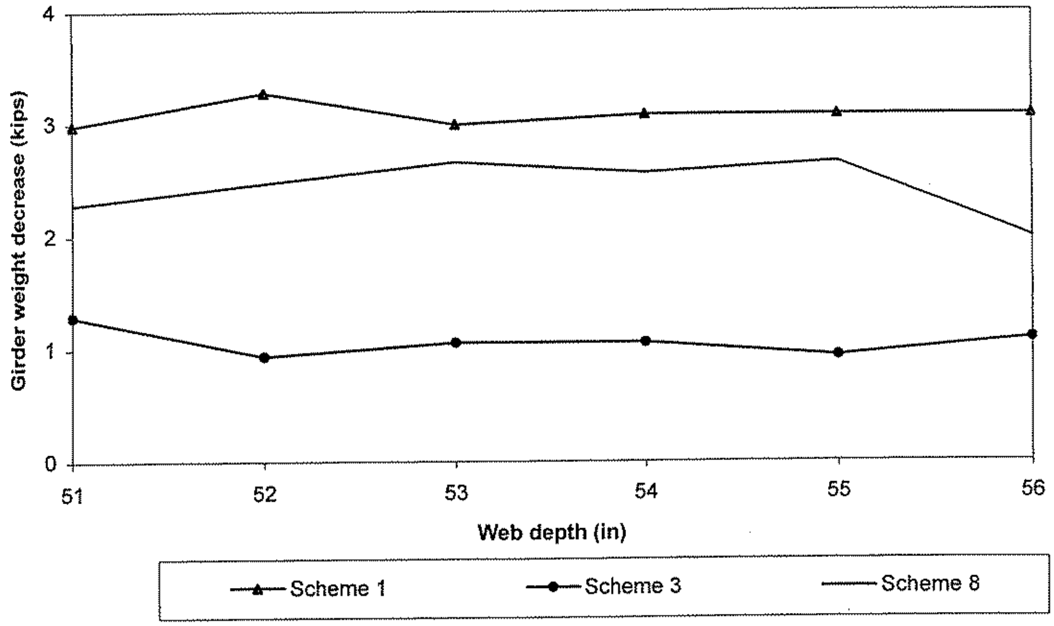


Figure 6.9 Decrease in total girder weight using upgraded fatigue details for the unstiffened web cases

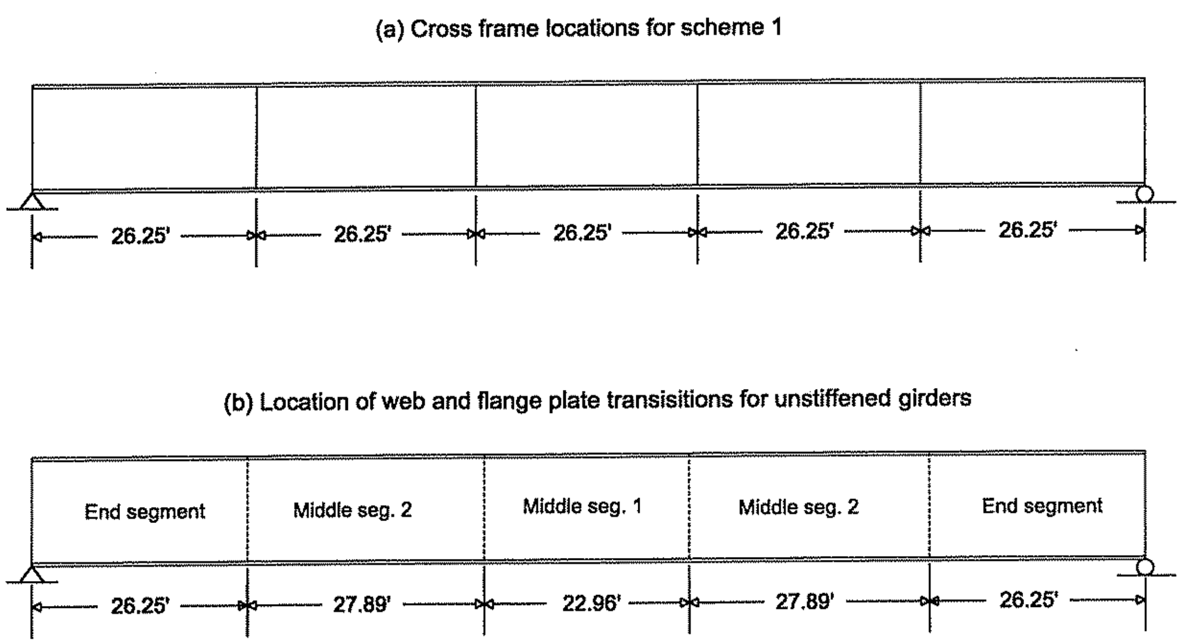


Figure 6.10 Cross frame locations and web and flange transitions

CHAPTER 7

Design Study of I-Girders With FRRBs

As discussed in Chapter 2, the design of composite steel bridge I-girder bridges should consider the potential for lateral torsional buckling of the girders in the positive moment region during construction before the concrete deck cures. One method of increasing the lateral torsional buckling resistance of a steel I-girder is to provide in-plane rotational restraint to the compression flange at cross frame locations with flange rotational restraint braces (FRRBs). This chapter presents a design study that investigates the use of FRRBs on the prototype bridge. The savings in girder weight from using FRRBs is studied for several different cross frame arrangements.

7.1 Lateral Torsional Buckling During Construction

Construction loads on steel I-girders include the weight of the I-girders and cross frames and of equipment, workers, and materials used to construct the bridge deck. These loads must be considered in design according to the AASHTO LRFD bridge design specifications (AASHTO 1998). Table 3.2 describes the load combination used for construction loads in this study.

In conventional steel I-girder bridges, cross frames are spaced no more than 25 ft (7620 mm) apart. Indeed, this maximum cross frame spacing has been a part of the AASHTO standard specifications (AASHTO 1984) for many years. When this cross frame spacing is not exceeded, the potential for lateral torsional buckling of the bridge girders is less significant. However, this approach to cross frame spacing can result in a bridge with unnecessary cross frames whose structural contribution is questionable under service loads. Reducing the number of cross frames increases the unbraced length, L_b, hence, lateral torsional buckling may control the design of the girder top flange.

7.2 Effective Unbraced Length With FRRBs

In Chapter 2, methods for determining the effective unbraced length of steel bridge I-girders with FRRBs were reviewed. As shown, FRRBs influence the lateral deflected shape of the compression flange. Additional points of inflection (change in curvature) reduce the unbraced length, L_b, to an effective unbraced length, $L_{b,eff}$. Effective length factors, K, can be used to quantify the unbraced length, $L_{b,eff} = K \cdot L_b$. The value of the K factor depends on the unbraced length, L_b, the moment of inertia of the compression flange, I_{yc}, the moment of inertia of the FRRB, I_y^{FRRB}, the length of the FRRB, L_s, and the number of FRRBs attached to the compression flange at a cross frame location. For example, Figure 7.1 shows two girders spanning 65.63 ft (20 m) with two FRRBs at the intermediate cross frames. If the top flange and FRRB both have a width of 9.5 in (241 mm) and a thickness of 1 in (25 mm), the corresponding K factor for the end segment and middle segment is 0.82 and 0.71, respectively.

The use of FRRBs allows the spacing between cross frames to be increased without a reduction in the lateral torsional buckling moment resistance. The increase in cross frame spacing depends on the bending moment in the corresponding region of the girder. For the interior regions of a simply supported girder with the largest bending moment, the use of FRRBs can increase the cross frame spacing by 50 to 60 percent. When the cross frames are originally spaced evenly along the span, the cross frame spacing near the bearings of a simply supported girder can be increased by over 100 percent (Murphy 1997).

Figure 7.2 compares the total girder weight for the prototype bridge for two cases. In one case, the bridge is designed with FRRBs located at the intermediate cross frames, and in the other case, the bridge is designed without FRRBs. The unbraced length for both designs are similar with cross frames located at 0, 45.93, 85.31 and 131.24 ft (0, 14, 26, and 40 m). Figure 7.2 shows a 7 kip (31.1 kN) savings in girder steel weight for web depths ranging from 51 in (1295 mm) to 56 in (1422 mm), when FRRBs are used. The addition of the FRRBs increases the lateral torsional buckling moment resistance allowing the top flange area to be reduced.

7.3 Design Study of Prototype Bridge With and Without FRRBs

A design study of the influence of FRRBs on the total girder weight of the prototype I-girder bridge is presented in this chapter. Cross frame schemes 6, 7 and 8, described in Chapter 4, are studied. Each cross frame arrangement consists of four cross frames. Lateraltorsional buckling under construction loads controls the dimensions of the top flange of the girders when these cross frame arrangements are used.

For the first stage of the design study, FRRBs are arranged at the intermediate cross frames, as shown in Figure 7.1, except that the total span and unbraced lengths correspond to those of cross frame schemes 6,7 and 8. A K factor of 0.71 is assumed for the middle segment where FRRBs are located at both ends of the unbraced length. A K factor of 0.82 is assumed for the end segments where FRRBs are located at one end of the segment (at the intermediate cross frame only). For a second stage of the design study, FRRBs are included at all four cross frame locations. In this case FRRBs are located at both ends of the end segments and a K factor of 0.71 is assumed for these segments.

Lateral torsional buckling under construction loads is checked at the location of maximum moment in the unbraced length. When web or flange transitions occur within an unbraced length, the section properties are selected from the girder segments on either side of the transition to produce the lowest calculated lateral torsional buckling moment resistance.

7.4 Results

This section presents results from the design study outlined in Section 7.3. I-girders for the protoype bridge were designed with a web depth ranging from 51 in (1295 mm) to 56 in (1425 mm) according to the procedure discussed in Chapter 3. Section 7.4.1 presents the design study results for I-girder with FRRBs compared to I-girders without FRRBs. Section 7.4.2 presents results for designs with 2 FRRBs, located at the intermediate cross frames, compared to designs with 4 FRRBs, located at both the intermediate and the end cross frames. Figure 7.3 describes cross frame schemes 6, 7, and 8 and the nomenclature used in this section.

7.4.1 Designs with Two FRRBs

This section presents results for I-girders designed for the prototype bridge with two intermediate cross frames with FRRBs and two end cross frames without FRRBs. Figure 7.4 shows the total girder weight for cross frame arrangements with and without FRRBs. The girder webs are stiffened and Category C′ fatigue details are used. A comparison of the weight for girders with FRRBs to the weight of girders without FRRBs shows a typical decrease in girder weight of 6.4 kips (28.5 kN) when cross frame scheme 6 is used with FRRBs. For cross frame schemes 7 and 8, the typical weight decrease is 4.6kips (20.5 kN) when FRRBs are included.

The larger decrease in girder weight for cross frame scheme 6 results from the following. The middle unbraced length (unbraced length BC in Figure 7.3) for cross frame scheme 6 is larger than unbraced length BC for schemes 7 and 8. As a result, the dimensions of the top flanges of girders with cross frame scheme 6 are controlled by lateral torsional buckling under construction loads. The dimensions of the top flanges of girders with schemes 7 and 8 are controlled by other conditions as discussed below. The use of FRRBs has a greater influence on girder weight when the designs are controlled by lateral torsional buckling, in particular, buckling of unbraced length BC.

The decrease in the weight of the middle segment of the girders with cross frame scheme 6 is greater than for girders with schemes 7 and 8, as shown in Figure 7.5. The weight of the middle segment can be influenced by the calculation of the lateral torsional buckling resistance of both the middle unbraced length (unbraced length BC in Figure 7.3) and the outer unbraced lengths (unbraced lengths AB and CD) as follows. When the plate transition occurs within an unbraced length, the cross-section properties from both the end and middle segments are combined to determine the lowest calculated lateral torsional buckling moment resistance. That is, the smallest cross-section properties are used in the calculations. This occurs in the two outer unbraced lengths (unbraced lengths AB and CD) for cross frame schemes 6, 7 and 8.

The cross-section dimensions used to determine the lateral torsional buckling moment resistance for the outer unbraced lengths (unbraced lengths AB and CD) for cross frame scheme 6 differ from those used for schemes 7 and 8. The y axis moment of inertia of the compression flange, I_{yc}, for scheme 6 is taken from the end segment, whereas, I_{yc} for schemes 7 and 8 is taken from the middle segment. The length of the unbraced length BC has a strong influence on this result. When unbraced length BC is decreased, as is the case for schemes 7 and 8, the dimensions of the compression flange in the middle segment are decreased because lateral torsional buckling is less critical for this unbraced length. The smaller compression flange dimensions for the middle segment results in an I_{yc} that is less than I_{yc} of the end segment. Therefore the design of the outer unbraced lengths for lateral torsional buckling is controlled by the compression flange dimensions (I_{yc}) of the middle segment. The middle segment does not fully benefit from the FRRBs (with a full decrease in compression flange area) because its compression flange dimensions are controlled by compression flange dimensions of the outer unbraced lengths. As a result, the weight savings in the middle segment of the girders with cross frame schemes 7 and 8 are less than for girders with scheme 6. Weight savings for the end segment of girders with cross frame scheme 6, 7, and 8 are similar, as shown in Figure 7.6.

Figure 7.7 compares the total weight of girders with FRRBs using cross frame schemes 6, 7 and 8 to the total weight of girders without FRRBs using cross frame schemes 1 and 3, which have more cross frames. The weight of girders using cross frame scheme 6 with four cross frames and two FRRBs is approximately 2 kips (8.9 kN) heavier than the weight of girders using scheme 3 with five cross frames. Therefore, girders for the prototype bridge with cross frame scheme 6 and FRRBs at the intermediate cross frames weigh approximately the same as girders for the prototype bridge with scheme 3 which has one more cross frame but has no FRRBs. The girder weight using scheme 6 with FRRBs is 4 kips (17.8 kN) heavier than the girder weight using scheme 1 with six cross frames.

Figure 7.8 shows the total weight of girders with cross frame schemes 6,7 and 8, with and without FRRBs, and with stiffened webs and Category B fatigue details. The weight of these girders is compared with that of girders without FRRBs using cross frame schemes 1 and 3. The results are similar to those discussed previously but the influence of the upgraded fatigue details is shown. Girders with cross frame scheme 6 have the greatest decrease in weight from using FRRBs, 6.3 kips (28 kN). The decrease in girder weight with schemes 7 and 8 is 5.2 kips (23.1 kN). The use of upgraded fatigue details does not have a significant influence on the weight savings from using FRRBs.

Figure 7.9 shows the total weight of girders with cross frame scheme 6 with and without FRRBs, and with Category C′ fatigue details. The use of stiffeners to provide part of the shear strength is considered, and results for stiffened, partially stiffened, and unstiffened webs are shown. The weight of girders with partially stiffened webs decreases 7.6kips (33.8 kN) from using FRRBs. The weight of girders with stiffened and unstiffened webs decreases 6.4 and 5.2 kips (28.5 and 23.1 kN), respectively. An explanation of the design criteria that control the girder designs with and without FRRBs is needed to explain the larger weight savings for the cases with partially stiffened webs. The designs of the girders withoutFRRBs are heavily influenced by lateral torsional buckling under construction loads because only four cross frames are used in cross frame scheme 6. Lateral torsional buckling influences the designs of the girders with FRRBs, but other criteria such as web and compression flange slenderness limits also influence the designs of the girders with stiffened webs. Girder designs controlled by lateral torsional buckling with and without FRRBs receive the full benefit of the FRRBs. Girder designs that become controlled by other criteria when FRRBs are included receive only a partial benefit of the FRRBs.

Figure 7.10 shows the total weight of girders with bracing Scheme 6, with or withoutFRRBs, and with Category B fatigue details. The girders are designed with stiffened, partially stiffened, or unstiffened webs. Girders with partially stiffened webs have the greatest decrease in weight from using FRRBs of approximately 7.0 kips (31.1 kN). Girders with stiffened and unstiffened webs have decreases in weight of 6.3 and 5.9 kips (28.0 and 26.2 kN), respectively. The use of upgraded fatigue details (Category B rather than Category C′) does not influence the weight savings for designs with stiffened webs. The use of upgraded fatigue details for cases with partially stiffened webs and unstiffend webs show some variation in weight savings (comparing results from using Category B details and using Category C′ details). This result can be attributed to the increments in the flange plate dimensions used in the design study (discussed in Chapter 3).

7.4.2 Designs with Four FRRBs

This section presents results for girders designed for the prototype bridge with two intermediate cross frames with FRRBs and two end cross frames withFRRBs. A total of four FRRBs are used. Results for girders with stiffened, partially stiffened, andunstiffened webs are presented. Cross frame scheme 6 is the only scheme considered since girders with this cross frame arrangement had the greatest decrease in girder weight from using FRRBs compared to girders designed using other arrangements of four cross frames.

Figure 7.11 compares the total weight of girders without FRRBs, with two FRRBs and with four FRRBs. The girders have stiffened webs and Category C′ fatigue details. As discussed previously, the weight of girders with cross frame scheme 6 decreases by approximately 6.4 kips (28.5 kN) from using two FRRBs. When four FRRBs are used, the typical decrease in girder weight is 7.8 kips (35.7 kN). A decrease of only 1.4 kips (6.2 kN) results from adding two FRRBs. Comparing the results of using two FRRBs to results using four FRRBs indicates the additional two FRRBs do not result in twice the weight savings. Nonetheless, the weight of girders with 5 cross frames and no FRRBs (cross frame scheme 3) is similar to the weight of girders with four cross frames and four FRRBs (cross frame scheme 6).

Figure 7.12 compares the weight of girders with stiffened webs and Category B fatigue details. The girders are designed without FRRBs, with two FRRBs, and with four FRRBs. Typical weight savings of 6.3 (28.0 kN) and 7.5 kips (33.4 kN) is achieved using two FRRBs and four FRRBs, respectively. The use of upgraded fatigue details does not influence the comparison of girders designed using two and using four FRRBs.

Figures 7.13 and 7.14 compare the weight of girders with Category C′ fatigue details, and with partially stiffened and unstiffened webs. The results are similar to those presented above. The use of four FRRBs, rather than two FRRBs does not double the weight savings. Figure 7.15 and 7.16 show the use of upgraded fatigue details does not significantly change the results.

7.5 Summary

This chapter presents the results of a design study of the advantages of using FRRBs in the prototype bridge. The use of FRRBs allows the spacing between cross frames to be increased without a reduction in the lateral torsional buckling moment resistance. The possible increase in cross frame spacing depends on the level of the bending moment in the corresponding region of the girder.

Since the addition of the FRRBs increases the lateral torsional buckling moment resistance, the top flange dimensions can be reduced resulting in decreased girder weight when lateral torsional buckling under construction loads control these dimensions. The decrease in girder weight from using FRRBs is largest when girder designs are controlled by lateral torsional buckling both with and without FRRBs. The design study shows that girders for the prototype bridge with four lines of cross frames (scheme 6) and FRRBs at the interior cross frames, weigh approximately the same as girders with five lines of cross frames (scheme 3). The use of upgraded fatigue details does not have a significant influence on the decreases in weight from using FRRBs. Comparing the effect of using two FRRBs, and using four FRRBs indicates the additional two FRRBs do not produce significant weight savings.

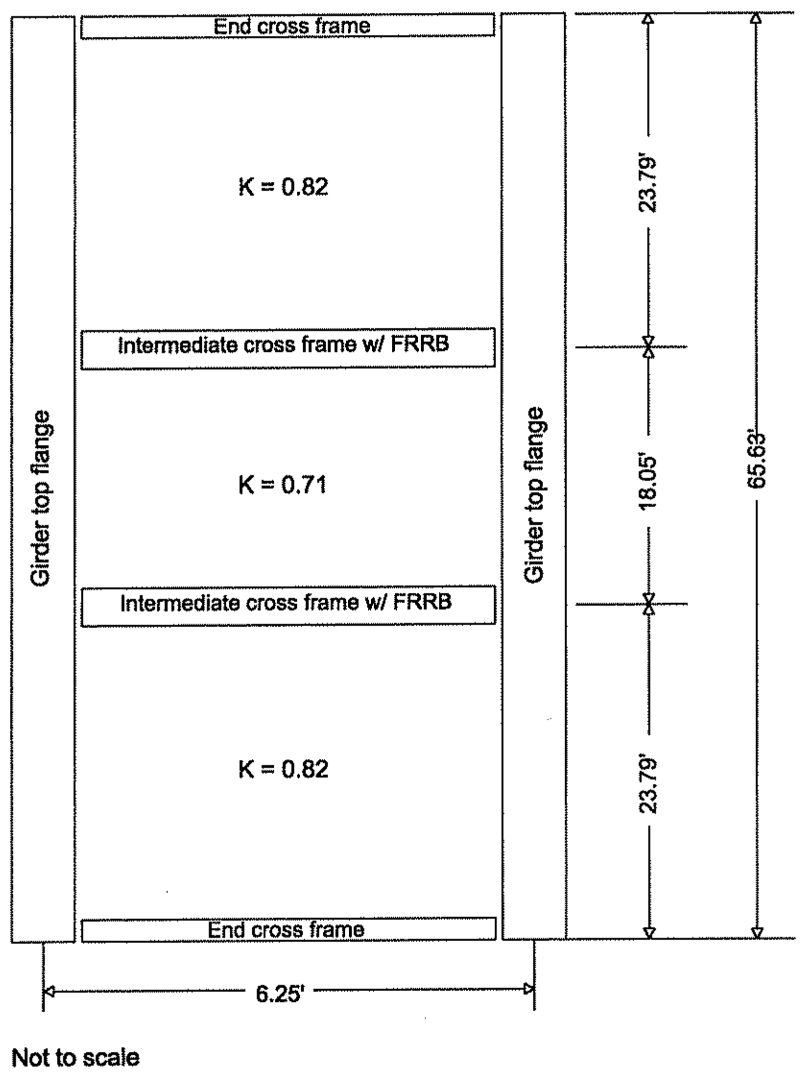


Figure 7.1 Cross frame arrangements and effective length factors

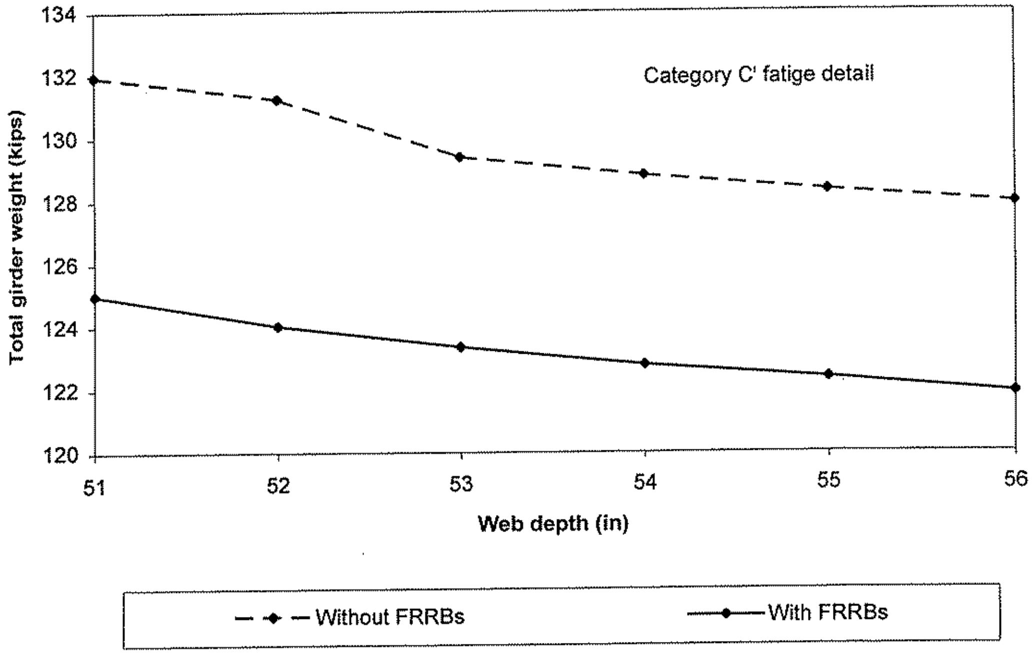


Figure 7.2 Typical weight savings using FRRBs

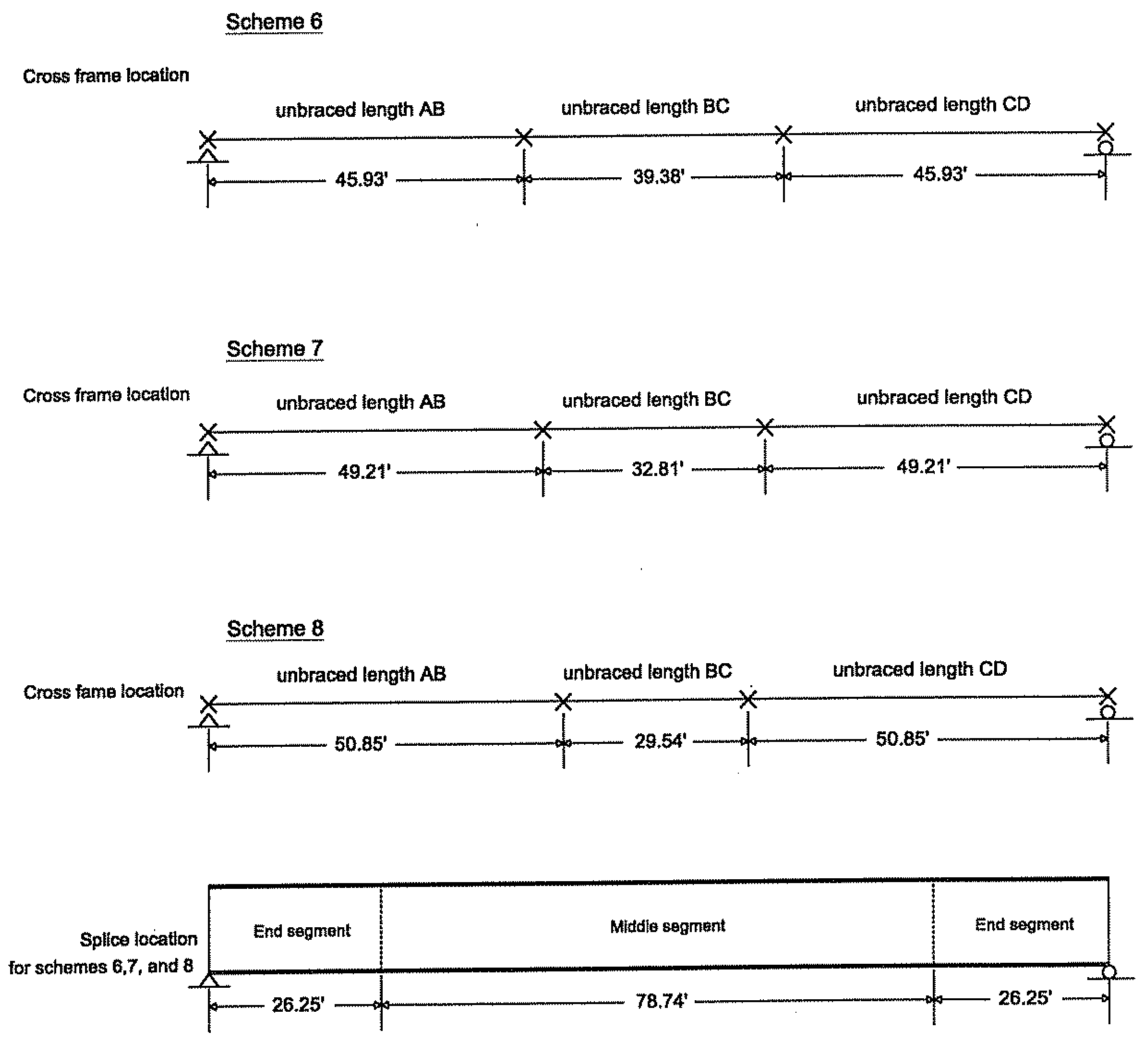


Figure 7.3 Cross frame schemes 6,7 and 8

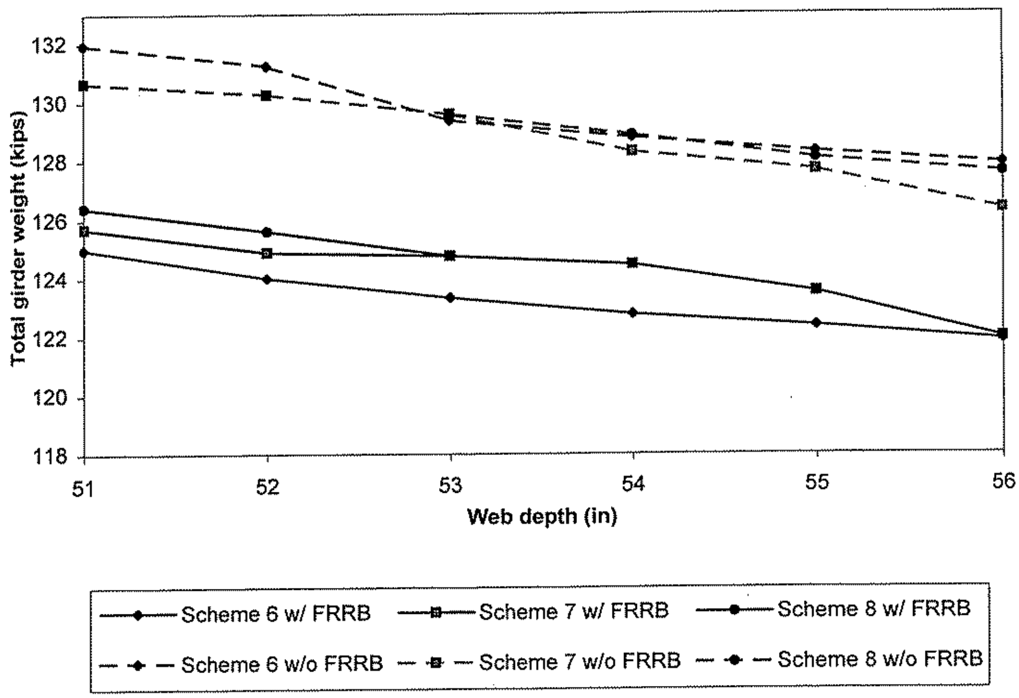


Figure 7.4 Influence of FRRBs on girder weight for girders with stiffened webs and Category C' fatigue details

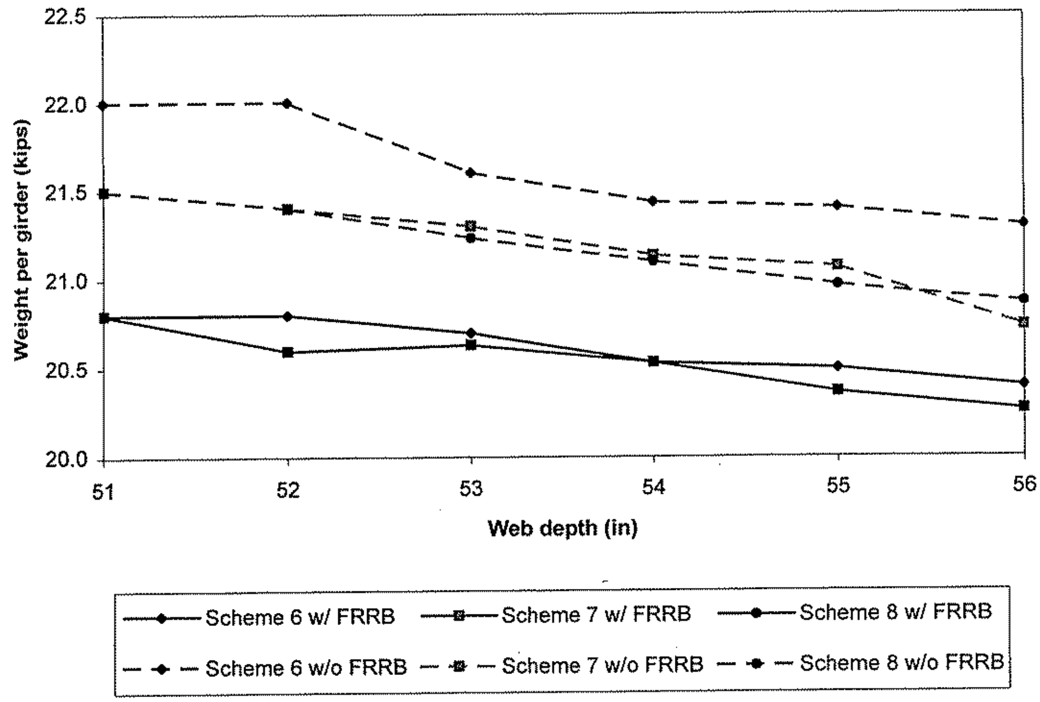


Figure 7.5 Influence of FRRBs on weight of middle segments of girders with stiffened webs and Category C' fatigue details

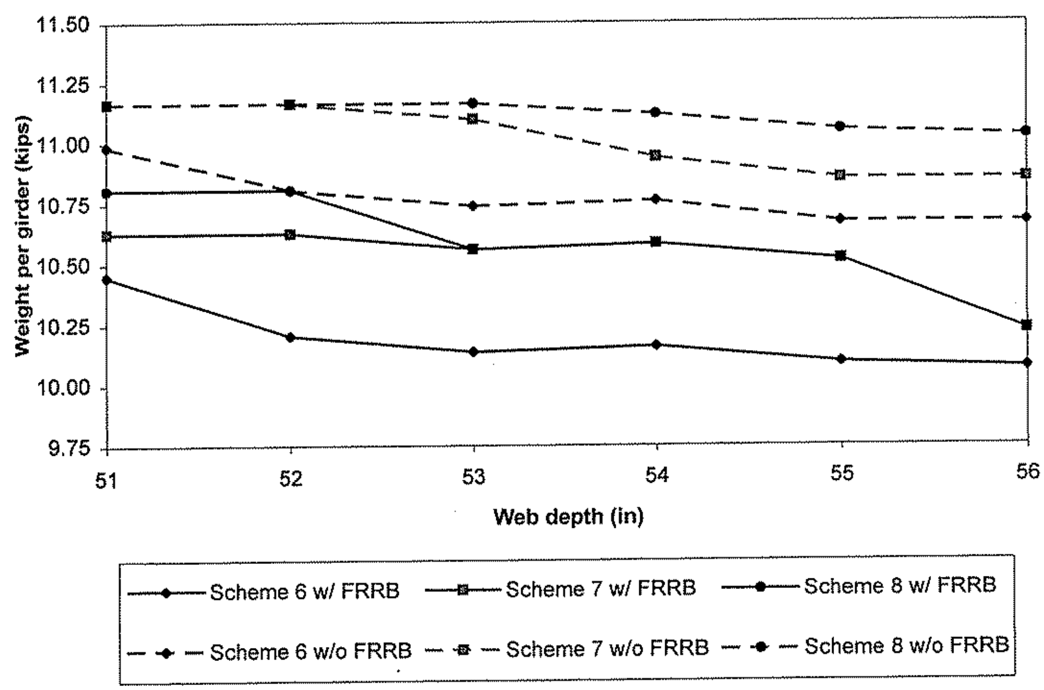


Figure 7.6 Influence of FRRBs on weight of end segments of girders with stiffened webs and Category C' fatigue details

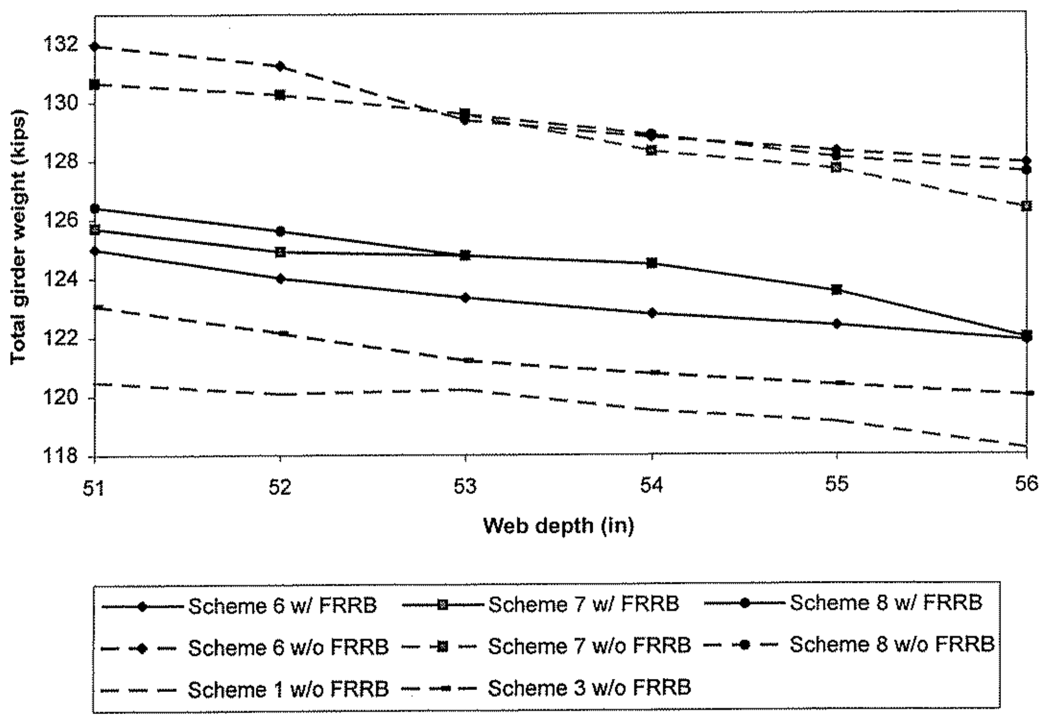


Figure 7.7 Comparison of stiffened girders with Category C' fatigue details designed with FRRBs, without FRRBs, and without FRRBs but with more cross frames

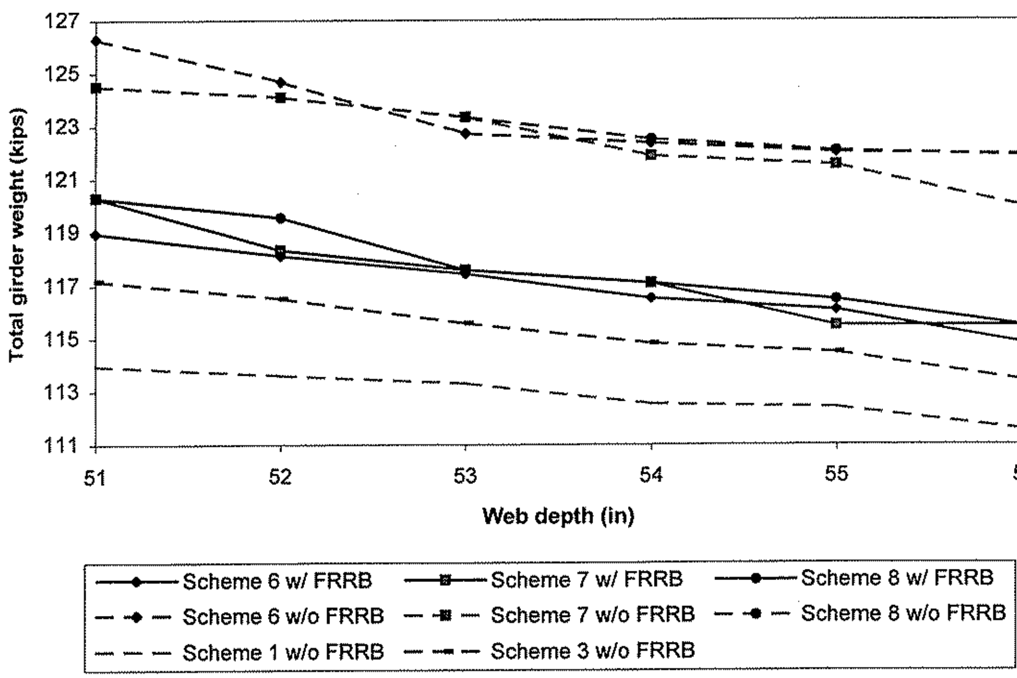


Figure 7.8 Comparison of stiffened girders with Category B fatigue details designed with FRRBs, without FRRBs, and without FRRBs but with more cross frames

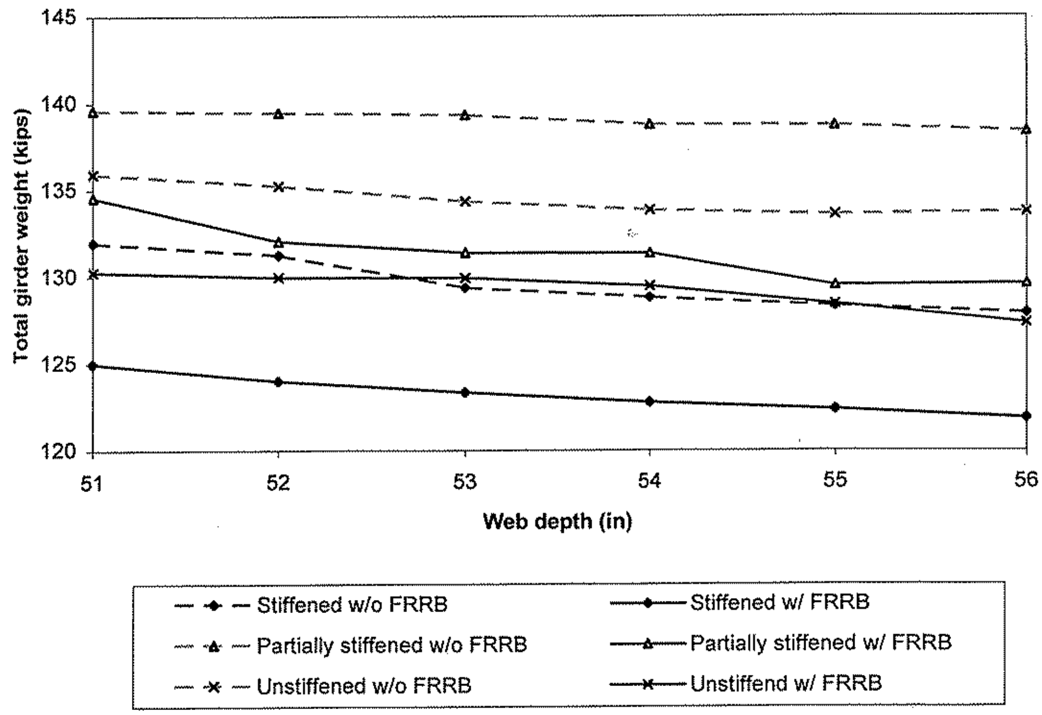


Figure 7.9 Influence of the use of stiffeners for shear strength on the weight of girders with and without FRRBs for cross frame scheme 6 and Category C' fatigue details

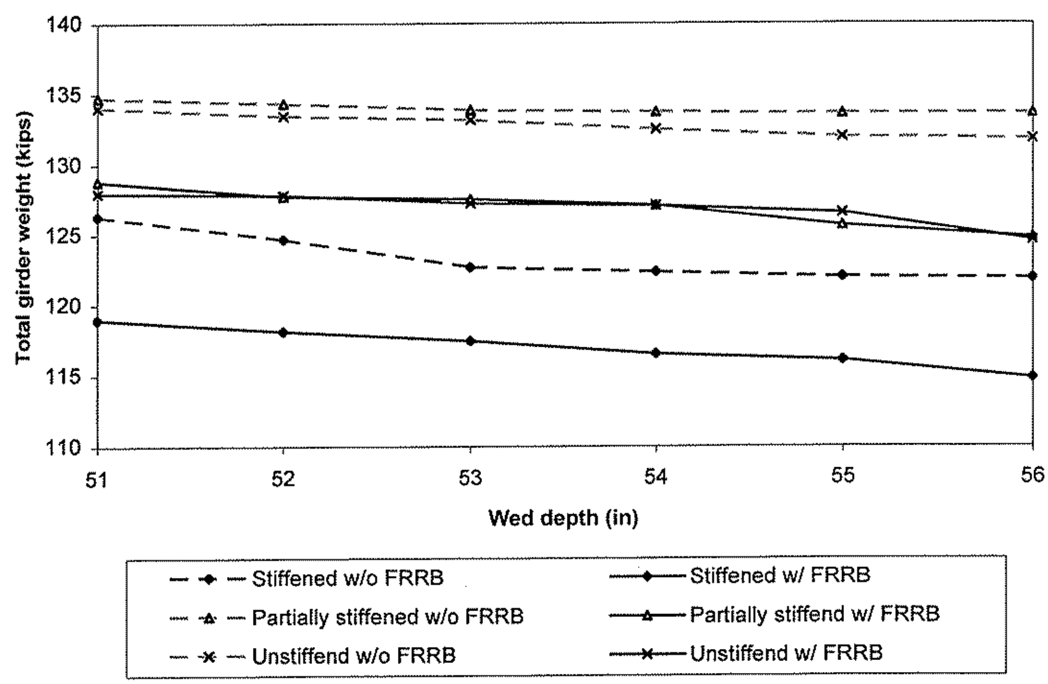


Figure 7.10 Influence of the use of stiffeners for shear strength on the weight of girders with and without FRRBs for cross frame scheme 6 and Category B fatigue details

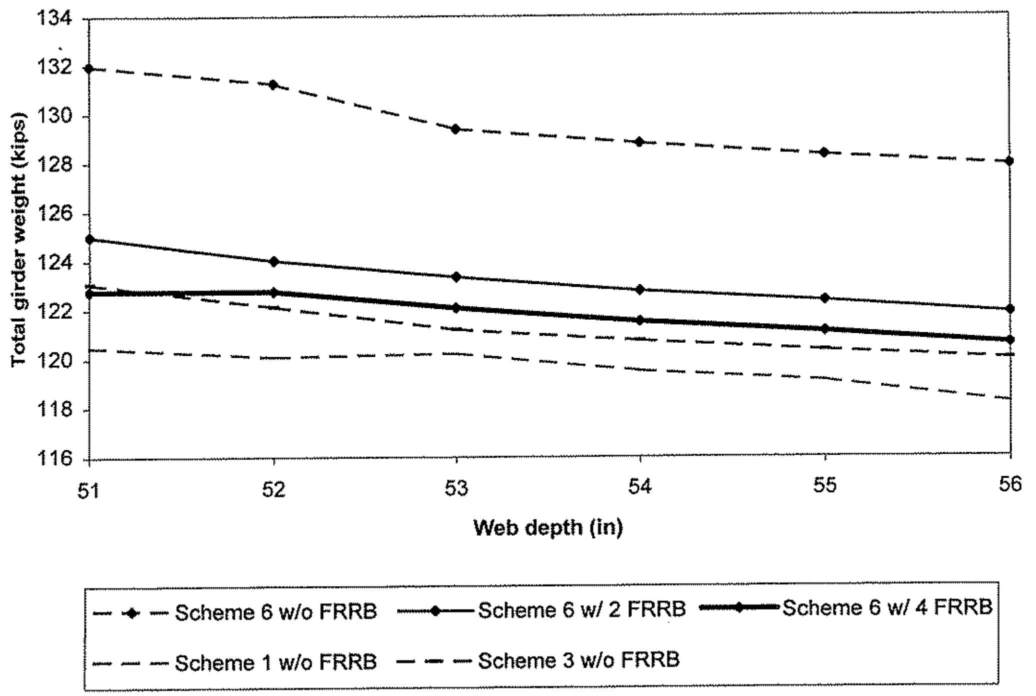


Figure 7.11 Comparison of stiffened girders with Category C' fatigue details designed with two FRRBs and with four FRRBs

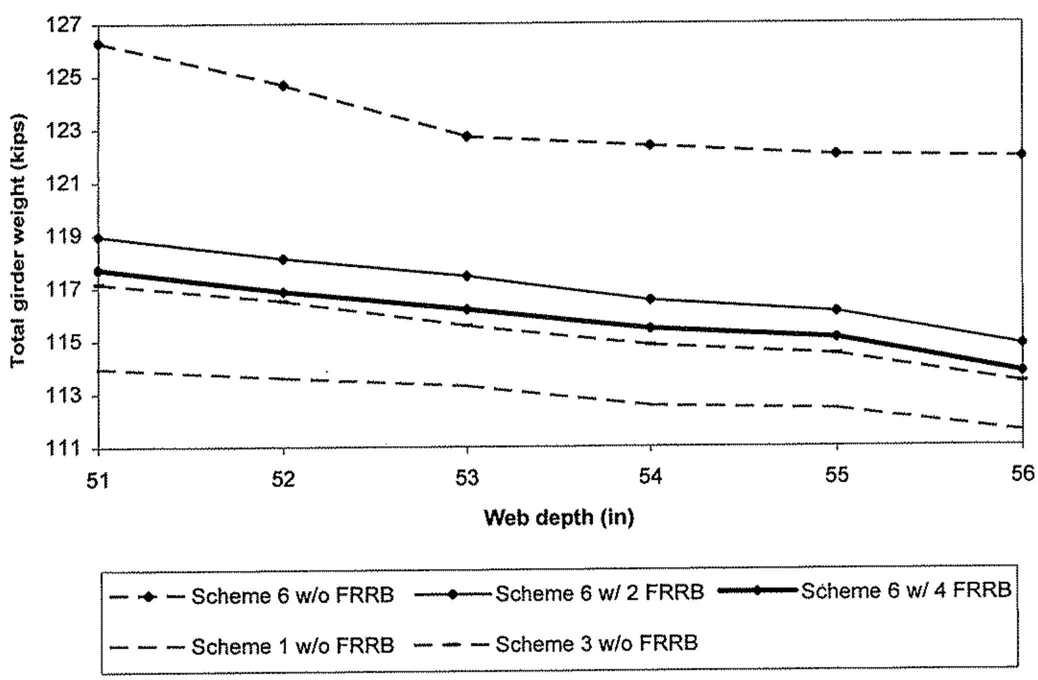


Figure 7.12 Comparison of stiffened girders with Category B fatigue details designed with two FRRBs and with four FRRBs

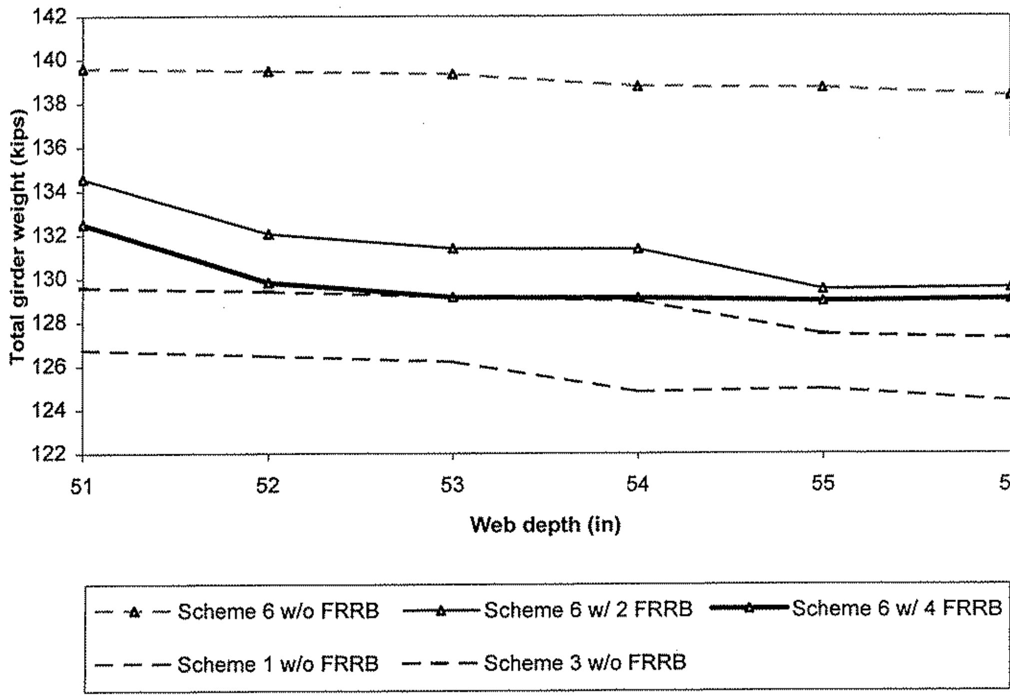


Figure 7.13 Comparison of partially stiffened girders with Category C' fatigue details designed with two FRRBs and with four FRRBs

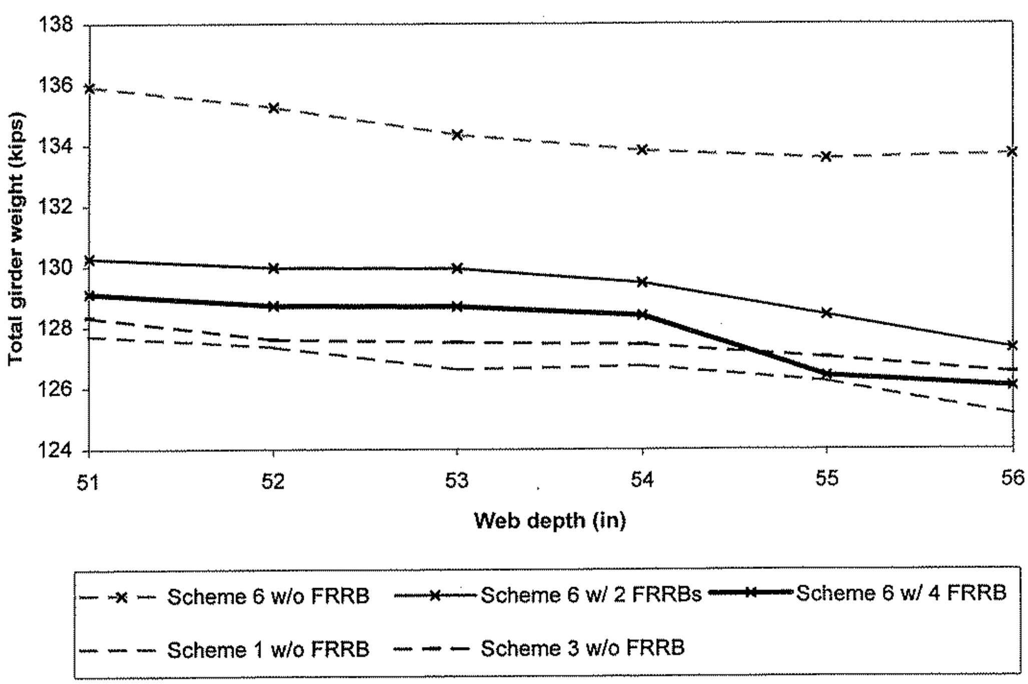


Figure 7.14 Comparison of unstiffened girders with Category C' fatigue details designed with two FRRBs and with four FRRBs

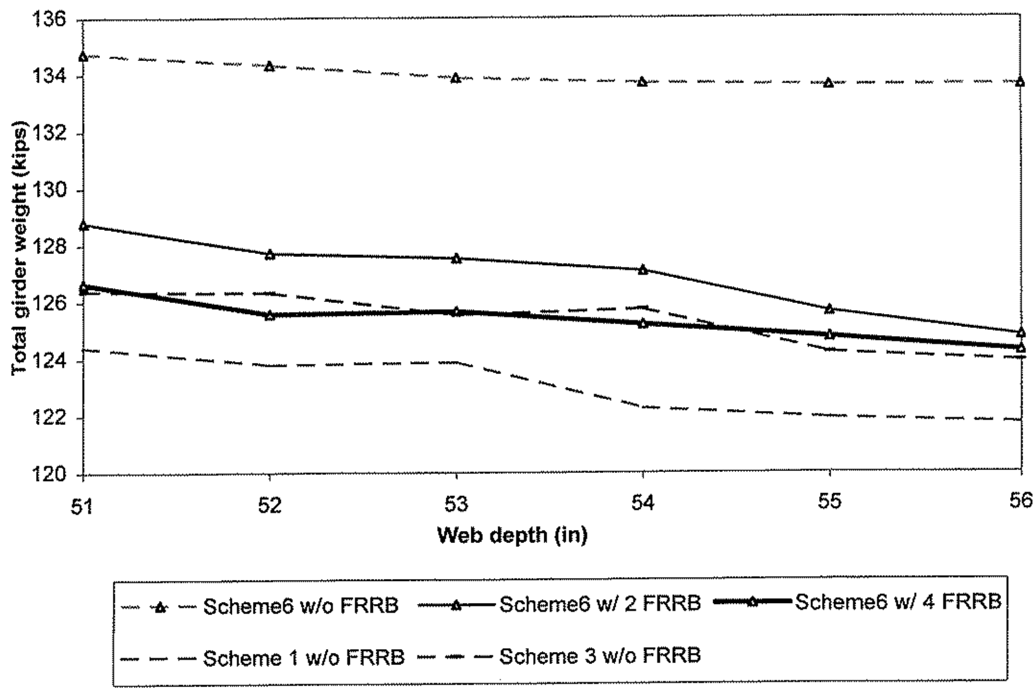


Figure 7.15 Comparison of partially stiffened girders with Category B fatigue details designed with two FRRBs and with four FRRBs

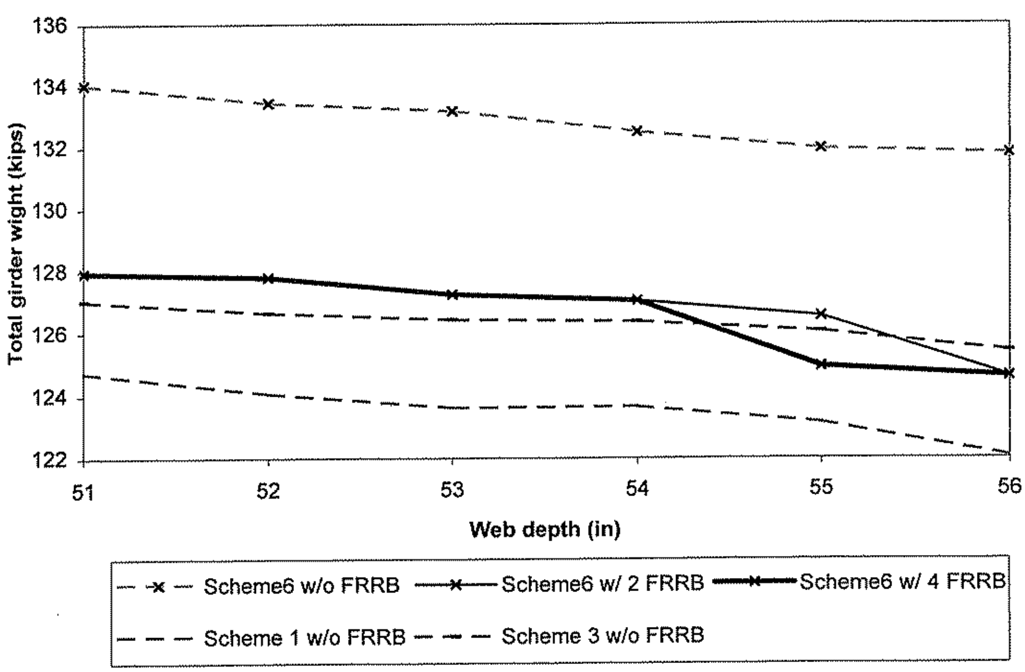


Figure 7.16 Comparison of unstiffened girders with Category B fatigue details designed with two FRRBs and with four FRRBs

CHAPTER 8

Summary and Conclusions

8.1 Summary

The objectives of the research presented in this thesis are: (1) to investigate the influence of certain I-girder bridge design parameters, including the number and arrangement of the cross frames, the use of flange rotational restraint braces (FRRBs) with the cross frames, the use of intermediate transverse stiffeners, and the connection details of these stiffeners, on the weight of composite high performance steel I-girders for bridges; and (2) to study the trade-offs between reductions in I-girder weight and reductions in I-girder fabrication effort as decisions about these design parameters are made.

The research consisted of a series of design studies focused on a prototype single span, simply supported composite steel I-girder bridge. The design of the I-girders for this bridge considered strength, construction, service, and fatigue limit states. Changes in the weight of the I-girders with changes in the design parameters, and the associated fabrication effort, were determined from the design studies. The results of the design studies show the possible trade-offs between increasing/decreasing girder weight and increasing/decreasing fabrication effort.

Chapter 1 introduced the design studies that were conducted. Chapter 2 provided background information on lateral torsional buckling of bridge I-girders under construction conditions. Information needed to design I-girders using cross frames with FRRBs was reviewed, including the concepts and methods for calculating the effective unbraced length factors for I-girders with FRRBs were also reviewed.

Chapter 3 described the prototype composite steel I-girder bridge used as the basis for the design studies. The prototype bridge was designed with ASTM A709 HPS 70W steel with a nominal yield stress of 70 ksi (485 MPa). The prototype bridge is a single span, simply supported bridge with a 131.24 ft (40 m) span, a 50 ft (15.2 m) width, and no skew. The bridge has four straight I-girders spaced at 12.50 ft (3810 m) centers. Exterior girders were designed according to the AASHTO LRFD bridge design specifications (AASHTO 1998) with the intent that all four girders of the bridge have the same cross-section. Chapter 3 reviewed the loads, the design criteria, the design and analysis procedure, the design parameters, and the practical considerations used in the design studies.

Chapter 4 presented a design study of the influence of the number, spacing, and arrangement of the cross frames on the design of the I-girders of theprototype bridge. Eight different cross frame arrangements were considered. For each cross frame arrangement, minimum weight I-girders were designed for the bridge. Arrangements of six, five, and four cross frames with different cross frame spacing were investigated to show how the cross frame arrangement influences the total weight of the girders in the bridge.

Chapter 5 presented a design study of the influence of parameters that determine the shear strength of an I-girder on the weight of I-girders designed for the prototype bridge. The design study varied the web thickness and the use of intermediate transverse stiffeners. Several different cross frame arrangements were used. The design study compared the weight of I-girders designed with stocky webs and no intermediate transverse stiffeners to the weight of I-girders designed with thin webs and many stiffeners.

Chapter 6 presented a design study of the connection details used for the transverse stiffener/cross frame connection plates on the prototype bridge. The advantages of using upgraded connection details with improved fatigue resistance were studied. Several cross frame arrangements were used. The study shows that upgrading the fatigue resistance of the connection details of the transverse stiffener/cross frame connection plates results in decreases in the weight of I-girders designed for the prototype bridge.

Chapter 7 presented a design study of the influence of using flange rotational restraint braces (FRRBs) on the prototype bridge. Several cross frame arrangements with four cross frames were studied. The study shows that decreases in the weight of I-girders designed for the prototype bridge are possible using FRRBs.

8.2 Summary of Findings

The design studies presented in Chapters 4, 5, 6, and 7 investigated the influence of the arrangement of cross frames, the use FRRBs, the use of intermediate transverse stiffeners, and the connection details of these stiffeners, on the weight of composite steel I-girders designed for the prototype bridge. Trade-offs between reductions in I-girder weight and reductions in I-girder fabrication effort were observed. The findings of the design studies are summarized using Figures 8.1, 8.2 and 8.3.

Figure 8.1 compares the total weight of some of the lightest and the heaviest I-girders designed for the prototype bridge. The I-girder designs represented in Figure 8.1 are minimum weight designs, however, design parameters such as the cross frame arrangement, the use of stiffeners, and so on were varied. The lightest girders were designed with cross frame scheme 1 without FRRBs, with Category B fatigue details, and with a stiffened web. For these girders, the total girder weight for the prototype bridge is approximately 114 kips (507.3 kN). The light weight results from the upgraded fatigue details and the close cross frame spacing. Some of the heaviest girders were designed with cross frame scheme 8 without FRRBs, with Category C′ fatigue details, and a partially stiffened web. For these girders, the total girder weight for the prototype bridge is approximately 138 kips (614.1 kN). The heavy weight results from the wide cross frame spacing which requires a heavy top flange to avoid lateral torsional buckling under construction loads and from the need for a heavy web. The 24 kip (106.8 kN) difference in total weight between these two cases shows the influence of the parameters that were considered.

Figure 8.2 compares the total weight of girders requiring different levels of fabrication effort. The girders requiring the most fabrication effort are those with Category B fatigue details and stiffened webs. As indicated above, the use of cross frame scheme 1 without FRRBs, Category B fatigue details, and a stiffened web results in the lightest I-girders designed for the prototype bridge with a total girder weight of approximately 114 kips (507.3 kN). Girders designed with cross frame scheme 3 without FRRBs, with Category B fatigue details, and with a stiffened web weigh approximately 117 kips (520.7 kN). Thus, the girders with cross frame scheme 3 are approximately 3 kips (13.4 kN) heavier than the girders with cross frame scheme 1, but one less cross frame is required. Girders designed with cross frame scheme 6 with FRRBs, with Category B fatigue details, and with a stiffened web weigh approximately 118 kips (525.1 kN). These girders are approximately 4 kips (17.8 kN) heavier than the lightest girders (with cross frame scheme 1), but two cross frames are omitted. The comparison of these three different cross frame arrangements shows that girder weight is not significantly increased as the number of cross frames is reduced and if FRRBs are included when needed.

Figure 8.2 shows the influence of using upgraded fatigue details. For girders designed with cross frame scheme 3 without FRRBs, upgrading the fatigue details from Category C′ to Category B results in a 6 kip (26.7 kN) decrease (from approximately 123 kips (547.4 kN) to approximately 117 kips (520.7 kN) in total girder weight. The results for girders designed with cross frame scheme 6 with FRRBs are similar, with the total girder weight decreasing from approximately 124 kips (551.8 kN) with Category C′ fatigue details to approximately 118 kips (525.1 kN) with Category B fatigue details. Upgrading the fatigue details results in a 6 kip (26.7 kN) decrease in total girder weight. On the other hand, however, the fabrication effort of upgrading these fatigue details can be eliminated with a 6 kip (26.7 kN) increase in total girder weight.

Figure 8.2 shows how girder weight is influenced when web thickness is increased to reduce the need for intermediate transverse stiffeners. Girders designed with cross frame scheme 3 without FRRBs, with Category B fatigue details, and with a stiffened web have a total girder weight of approximately 117 kips (520.7 kN). With a partially stiffened web, the total girder weight is approximately 126 kips (560.7 kN). The total girder weight increases 9 kips (40.1 kN) as the fabrication effort for most of the transverse stiffeners is eliminated. The results for girders designed with cross frame scheme 6 are similar, with the total girder weight increasing from approximately 118 kips (525.1 kN) with a stiffened web to approximately 128 kips (569.6 kN) with a partially stiffened web. Thus, the increase in total girder weight from changing from a stiffened web to a partially stiffened web with significantly fewer intermediate transverse stiffeners is approximately 10 kips (44.5 kN).

The results in Figure 8.2 are summarized as follows. The girders requiring the least fabrication effort are those with the fewest cross frames (cross frame scheme 6) and partially stiffened webs. The range of total girder weight shows that a reduction in fabrication effort is accompanied by measurable increases in the total girder weight.

Figure 8.3 focuses on the influence of cross frame arrangements on total girder weight. Girders designed with cross frame schemes 1, 3, and 8 without FRRBs, with Category B fatigue details, and with stiffened webs are compared. As stated above, cross frame scheme 1 with six cross frames results in the lightest weight girders. Reducing the number of cross frames from six (scheme 1) to five (scheme 3) results in a 3 kip (13.4 kN) increase in total girder weight. Further reducing the number of cross frames from five (scheme 3) to four (scheme 8) results in a 7 kip (31.2 kN) increase in total girder weight.

8.3 Conclusions

From the design study of the influence of the arrangement of cross frames on the design of I-girders for the prototype bridge, the following conclusions are drawn:

- The girder weight increases as the number of cross frames decreases.
- Lateral torsional buckling of the girders under construction conditions may significantly influence the top flange dimensions when the cross frames are spaced greater than 26.25 ft (8 m).
- Cross frame arrangements with larger spacing between the end cross frames and the first intermediate cross frame lead to lighter weight girders while reducing the number of cross frames by taking advantage of the moment gradient C_b factor.

From the design study of the influence of web thickness and the use of intermediate transverse stiffeners on the design of I-girders for the prototype bridge, the following conclusions are drawn:

- An increased web thickness may eliminate the need for intermediate transverse stiffeners.
- Girders with a 3/8 in (10 mm) web thickness have the lightest weight but need a significant number of transverse stiffeners to provide the required shear strength.
- Girders with a 5/8 in (16 mm) web thickness have increased girder weight but the need for intermediate transverse stiffeners is eliminated.

From the design study of the influence of the fatigue resistance of the connection details used for the transverse stiffener/cross frame connection plates on the design of I-girders for the prototype bridge, the following conclusions are drawn:

- Using upgraded fatigue details (Category C′ to Category B) allow decreases in the weight of the girders.
- When the top flange is not governed by lateral torsional buckling under construction conditions, both the top and bottom flange dimensions can be reduced when the fatigue details are upgraded from Category C′ to Category B.
- Category B fatigue details should be limited to regions of high bending moment demand under the fatigue load.

From the design study of the influence of using FRRBs on the prototype bridge, the following conclusions are drawn:

- FRRBs allow for an increase in cross frame spacing without compromising the lateral torsional buckling resistance under construction conditions.
- Girders with four cross frames and FRRBs weigh nearly the same as girders with five cross frames and no FRRBs.
- FRRBs are used most effectively in regions of the girders where the dimensions of the compression flange are controlled by lateral torsional buckling.

Finally, rational design of composite high performance steel I-girders can be achieved through careful consideration of the trade-offs between reductions in I-girder weight and reductions in I-girder fabrication effort. Variations in parameters such as the arrangement of cross frames, the use of FRRBs, the use of intermediate transverse stiffeners, and the connection details of these stiffeners, influence both the weight of steel I-girder bridges and the effort required to fabricate them.

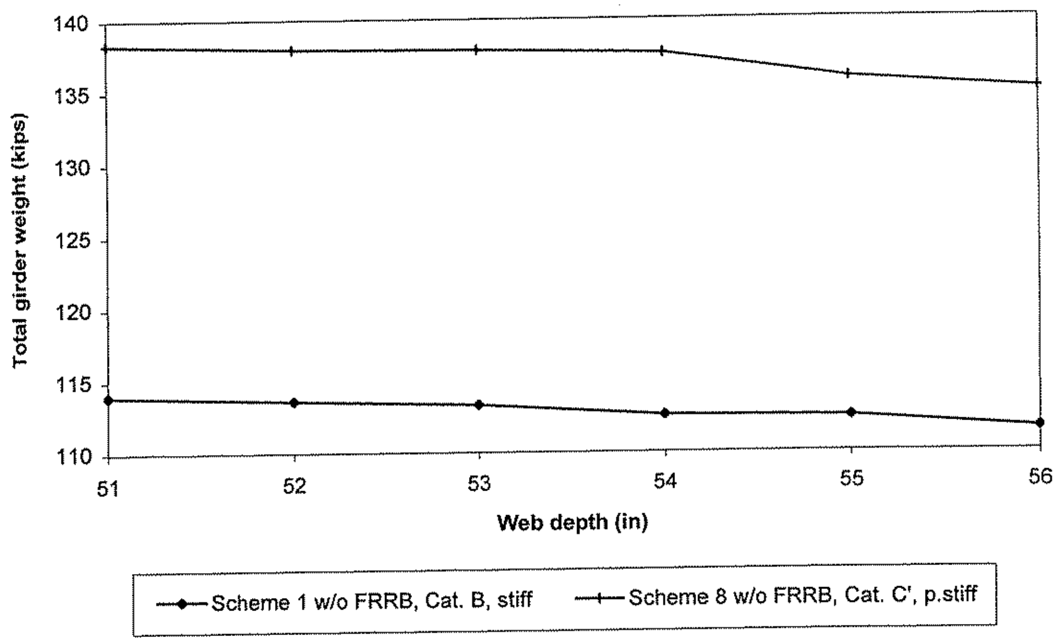


Figure 8.1 Range of total girder weight from design studies of the prototype bridge

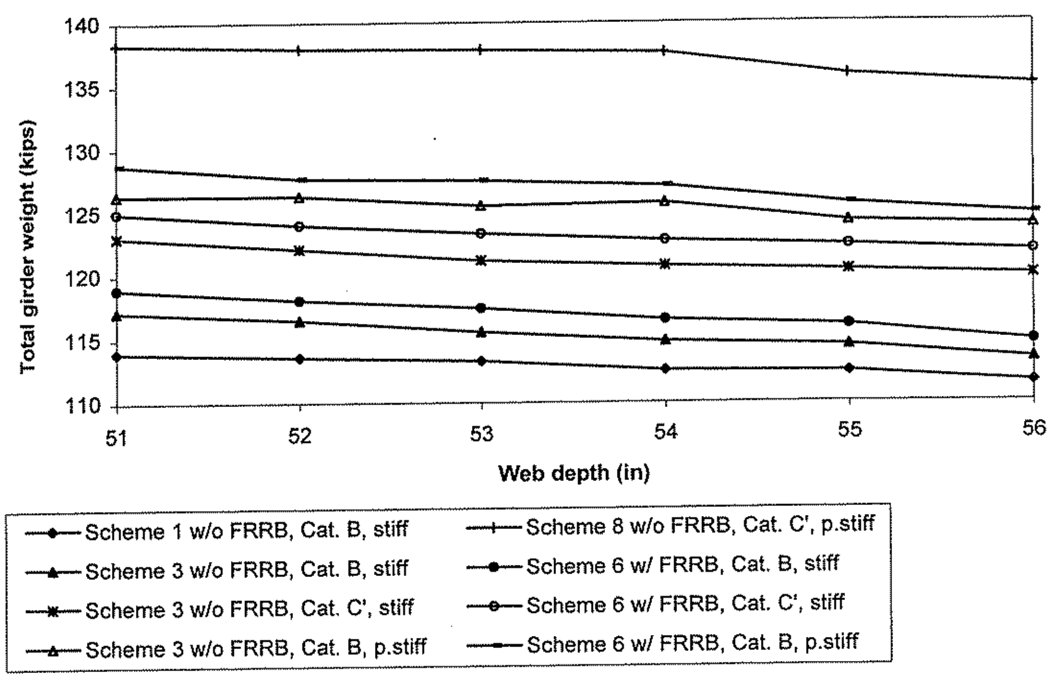


Figure 8.2 Influence of parameters related to fabrication effort on the total girder weight

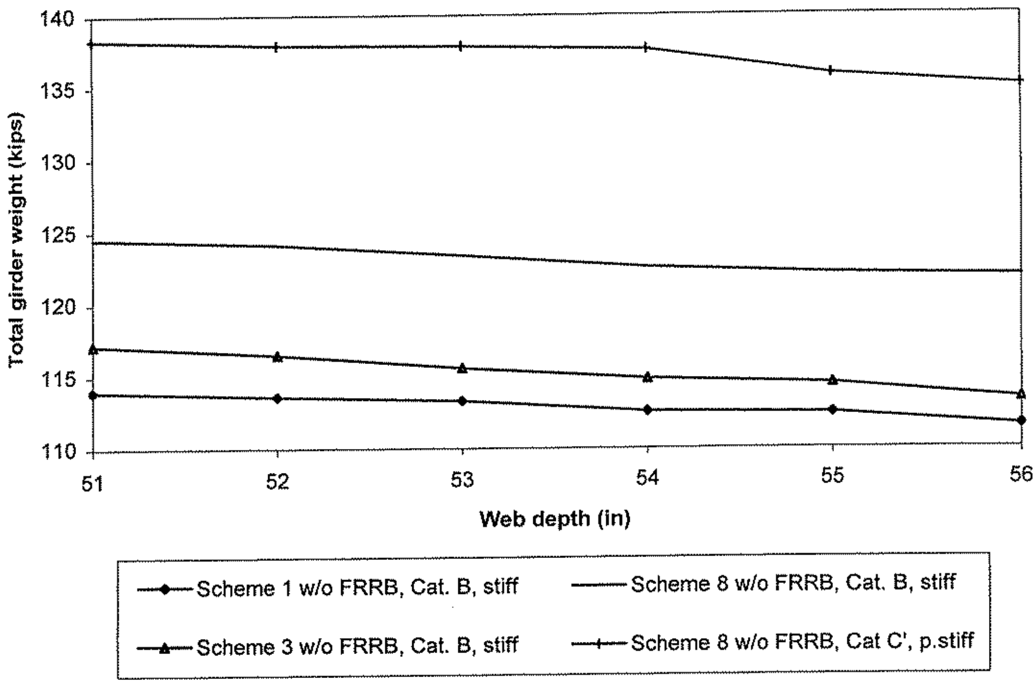


Figure 8.3 Influence of cross frame arrangements on the total girder weight of the prototype bridge

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