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Design of HPS bridge girders with tubular flanges

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DESIGN OF HPS BRIDGE GIRDERS WITH TUBULAR FLANGES

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Design of HPS Bridge Girders with Tubular Flanges

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ABSTRACT

The lateral-torsional buckling resistance of I-girders can be increased by replacing the conventional flat-plate compression flange with a concrete filled steel tube. This increased lateral-torsional resistance can be important during the construction of single-span bridges, before the compression (top) flange is braced by the deck. As a result, the tubular-flange girder requires less steel and fewer cross-frames to resist construction loads than the a conventional I-girder.

This research investigates the use of girders with tubular flanges in highway bridges. Four combinations of prototype bridges and girder types are studied: (1) a four-girder prototype bridge with composite I-girders, (2) a four-girder prototype bridge with composite tubular flange girders, (3) a four-girder prototype bridge with non-composite tubular flange girders, and (4) a through-girder prototype bridge with tubular flange girders. The design criteria used in the study were based on the AASHTO LRFD code (AASHTO LRFD 1998). This study also analyzed the influence of certain design parameters on the girders designed for these cases. These design parameters include the number of interior cross-frames, the number of intermediate transverse stiffeners, and the fatigue category of the connection details.

The designs considered strength, construction, service, and fatigue limit states. Minimum weight girder designs were developed for all the prototype bridges, girder types, and design parameters that were considered. The girder weight and the fabrication effort required for each case were then compared.

The results of the study demonstrate that the girder weight decreases as the number of cross-frames and transverse stiffeners increase. Increasing the fatigue resistance of the connections can also reduce the required girder weight. However, many of these changes in the design parameters increases the required fabrication effort. The study also demonstrates that the tubular flange girders require less steel and fewer cross-frames than the conventional I-girders, and therefore, they are a viable option in bridge design.

CHAPTER 1

INTRODUCTION

1.1 INTRODUCTION

Interior cross-frames have traditionally been spaced along the length of a bridge at intervals less than 25 ft (7620 mm). The AASHTO LRFD specifications (AASHTO LRFD 1998) lifted the cross-frame limit of 25 ft (7620 mm) and replaced it with a requirement to perform a rational analysis of the required cross-frame spacing. In a straight, simply-supported steel girder bridge, the primary functions of cross-frames are to brace the girder compression flange when the bridge is under construction, before the deck braces the girder, and to help in resisting wind loads.

A decrease in the number of cross-frames in a steel girder bridge decreases the required fabrication effort, and as a result, decreases the cost of the bridge. The locations of cross-frames are also associated with fatigue issues. In particular, the welds of the cross-frame connection plates to the girder are susceptible to fatigue.

Ellis (Ellis 1999) studied the effect of interior cross-frames on the weight of composite I-girders designed with ASTM A709 HPS-70W steel. Ellis (1999) found that increases in the size of the compression flange are needed as the number of cross-frames decreases, and the associated spacing between cross-frames increases. These increases in the compression flange are needed to address the increases tendancy for lateral-torsional buckling under construction conditions. Ellis (1999) found that I-girder weight increases only 9% when the number of interior cross-frames in a 131.24 ft (40000 mm) simply-supported bridge is decreased by half.

The study reported herein considers the use of steel bridge girders with a concrete filled tube as the compression flange. The concrete filled tubular compression flange has significant torsional stiffness which increases the lateral-torsional stability of the girder. The increased lateral torsional stability of the tubular flange girder decreases the interior cross-frames when the presence of cross-frames is controlled by lateral-torsional buckling under construction conditions.

This study investigates how the design of tubular flange girders is influenced by bridge design parameters such as cross-frame spacing and stiffener spacing. Tubular flange girders are designed to be either composite or non-composite with the deck. The designs are compared with conventional I-girder designs. Also, a through-girder bridge is studied. Minimum weight designs are generated, and from the results of the study, conclusions regarding the advantages of tubular flange girders are made.

1.2 OBJECTIVE

The objective of this study is to investigate the possible advantages of tubular flange girders, when used with high performance ASTM A709 HPS 70W steel. Advantages investigated include a reduction in the need for cross-frames, a reduction in girder weight, and a reduction in the need for stiffeners. The possible use of tubular flange girders in a through-girder bridge for tight clearance conditions was also considered.

1.3 SCOPE

This study consists of design studies that focus on two prototype bridges: (1) a four-girder prototype bridge and (2) a through-girder prototype bridge. Both prototype bridges are single-span, simply supported steel girder bridges with a span length of 131.24 ft (40000 mm). The bridges are designed using AASHTO LRFD specifications (AASHTO LRFD 1998). Strength, service, and fatigue limit states are considered in the design studies. The studies produced minimum weight tubular flange girders that are compared as bridge design parameters, such as cross-frame spacing, are varied.

1.4 OUTLINE OF THESIS

The remaining chapters of the thesis are as follows. Chapter 2 gives a summary of some past research concerning the uses of cross-frames and the effect on the bridge girders when the cross-frames are removed. Chapter 3 describes the prototype bridge dimensions, and the loads that were assumed to act on the girders. Also, the limit states and load combinations affecting the bridge are discussed. Chapter 4 lists and describes the design parameters that were varied in each design study. Chapter 5 describes the approach taken in each design study, and also gives the results from the design studies. Chapter 6 summarizes and presents the conclusions of the study.

CHAPTER 2

BACKGROUND INFORMATION

2.1 INTRODUCTION

In modern steel bridges, cross-frames are connected to adjacent steel girders at certain intervals along the span. A typical interior cross-frame is shown in Figure 2.1. Traditionally, these cross-frames have been spaced evenly along the length of the span at a distance no greater than 25 ft (7620 mm), in accordance with AASHTO design specifications (AASHTO, 16th edition 1996). The 25 ft (7620 mm) limit on cross-frame spacing has been replaced by a requirement to determine the number and spacing of cross-frames by a rational analysis (AASHTO LRFD 1998). A rational analysis will often lead to a decrease in the number of cross-frames, which will reduce the number of fatigue-prone attachment details where the cross-frames connect to the girders (AASHTO LRFD 1998).

Interior cross-frames serve two primary functions in a straight, steel girder bridge: (1) distributing loads, and (2) bracing the girders compression flanges. The load distribution function includes the distribution of the lateral (wind, earthquake, collision, etc.) loads from the girders to the deck, and ultimately, to the bridge bearings, and the distribution of the gravity (dead and live) loads among the girders. The bracing function involves bracing the top flange of a composite steel girder in the positive moment region when it is in compression during construction (before it is composite with the deck), or bracing the bottom flange of a multi-span continuous girder in a positive moment region during construction and under service conditions. Cross-frames also aid in erection of the girders during the construction of the bridge.

2.2 LOAD DISTRIBUTION

One of the main functions of cross-frames is to assist in distributing loads among adjacent girders. It is unclear how much cross-frames contribute to the load distribution. Previous research has tried to determine the contribution of cross-frames to load distribution and to establish how many interior cross-frames are actually required for this function.

2.2.1 LATERAL LOAD DISTRIBUTION

Lateral loads include wind loads and collision loads. These loads act on the fascia girders. Interior cross-frames are usually designed to carry these loads from the bottom flange of the fascia girder to the concrete deck. A conceptual view of this load path can be seen in Figure 2.2. After the load is carried to the deck, the deck transmits the load to the bearings through the cross-frames at the piers and abutments. The load path from the deck to the bearings is shown in Figure 2.3. If no interior cross-frames are present, and if the bottom flange of the fascia girder is large enough, lateral bending moment and shear in the bottom flange will transmit the forces to the bearings.

When considering wind loading, it is often assumed that the concrete deck will directly resist the wind that acts on the upper half of the fascia girder. The wind loads acting on the bottom half of the fascia girder are assumed to be resisted by bending and shear in the bottom flange. With interior cross-frames in place, the bottom flange spans between the cross-frames. Without interior cross-frames, the flange spans between the cross-frames at the bearings. Interior cross-frames are usually designed to carry the wind load to the deck via the top flange of the adjacent girder, as shown in Figure 2.2. Other load paths can be designed to carry wind loads if cross-frames are not present in the bridge. The fascia girder bottom flange can be designed to carry the wind load to the bearings, without any interior cross-frames (Mertz 1999). The wind load design should not be a governing factor in determining the spacing of the interior cross-frames for typical lengths of steel bridges (Stallings 1996). More often, the stability of the compression flange is the governing factor for the cross-frame spacing.

Collision loads from over-height vehicles crossing under the bridge are another lateral load to consider. In the case of a collision load, closely spaced cross-frames may be harmful to the bridge. If the collision occurs near a cross-frame, the steel near the cross-frame connection has a tendency to tear (Mertz 1999). If the collision occurs between the cross-frames, the flange tends to plastically deform in bending. When the collision occurs near a cross-frame, the tear in the girder must be removed and replaced using field welding. This repair may reduce the fatigue resistance of the girder. When the collision occurs away from a connection, the plastic deformation can be more easily and more reliably repaired thought heat straightening (Mertz 1999). Therefore, the collision load design should not be a governing factor in the cross-frame spacing.

2.2.2 LIVE LOAD DISTRIBUTION

Interior cross-frames participate in the distribution of live loads and dead loads among the girders. The cross-frames contribution to the live load distribution is not clear, and a few recent studies have investigated the effect of cross-frames on load distribution. This was done by measuring the stresses in steel bridge girders before and after the cross-frames were removed.

In a study by Stallings (1996) of a single-span composite bridge with 9 rolled-steel, wide-flange girders, removing the cross-frames resulted in an increase in the maximum bottom flange stresses of the most heavily loaded girder of less than 15%. The strains in the cross-frames were also measured, but the magnitudes of measured strains were so small that they were unreliable (Stallings 1996). When the cross-frames were removed, the deflection of the girders directly under the load slightly increased, while the deflection of the girders away from the load slightly decreased. Similar results were obtained when the cross-frames were removed from a three-span, continuous, non-composite steel girder bridge. The stress in the bottom flange increased as much as 15% in the most heavily loaded girder. The change in deflections, and the strain measured in the cross-frames were similar to those measured in the single span bridge (Stallings 1999). Azizinamini (1994) had similar results in laboratory tests of a three-girder system, where removing the diaphragms resulted in only a slight difference in the stresses and deflections. Again, the level of stress in the diaphragms was very small. A finite element study was also carried out and similar results were found.

These results show that interior diaphragms influence the load distribution among steel bridge girders, but the influence is not great. These studies also show that removing the cross-frames does not dramatically decrease the live load capacity. The increased stresses in the most heavily loaded girder, after the removal of the cross-frames, did not exceed the stresses calculated using AASHTO specifications (Stallings 1996).

The current AASHTO LRFD live load distribution specifications (AASHTO LRFD 1998) ignore the contribution of interior cross-frames. A provision is included where the live load distribution factors may be calculated by assuming that the bridge acts as a rigid cross-section if it is known that regularly spaced cross-frames will be present. Alternatively, acceptable and conservative distribution factors can be calculated using the AASHTO LRFD specifications without regard for the presence of cross-frames.

The studies outlined above and the AASHTO specifications suggest that cross-frames are not important to the load distribution of either lateral or live loads. The stiffness of the slab is sufficient in order to distribute the live loads among adjacent girders (Azizinamini 1994). Other load paths can be designed to carry the lateral loads. Thus, load distribution should not be the factor that governs interior cross-frame spacing in a straight, steel girder bridge.

2.3 COMPRESSION FLANGE BRACING

Compression flange bracing is the most significant function of cross-frames (Mertz 1996). This function includes bracing the top flange of a composite steel girder in a positive moment region during construction, and bracing the bottom flange in a negative moment region during construction and during service.

In a single span bridge with straight girders, bracing other than that supplied by a composite deck is not required during service conditions. Cross-frames are only required during the construction of the bridge, before the deck fully cures. The AASHTO LRFD specifications (AASHTO LRFD 1998) state that "Diaphragms or cross frames required for conditions other than the final condition may be specified to be temporary bracing." The use of temporary diaphragms may decrease the number of fatigue prone attachments, and therefore, may allow smaller steel sections to be used. If temporary bracing is used, future deck replacement projects would have to re-install temporary bracing during reconstruction.

In a multi-span continuous bridge, bracing is required for the compression flange (bottom flange) in a negative moment region throughout the life of the bridge. The distance between the cross-frames should be determined by rational analysis considering lateral-torsional buckling.

Increasing the size of the compression flange may also increase the lateral-torsional buckling strength of a girder, thereby decreasing the need for cross-frames in either a positive or negative moment region. A study by Ellis (1999) investigated how the number and spacing of interior cross-frames affect the design of I-girders made from ASTM A709 HPS 70W steel section. Eight different cross-frame arrangements were studied. These arrangements are described in more detail in Chapter 4. The steel I-girder weight increased 9% when the spacing between the cross-frames was increased from approximately 26 ft (8000 mm) (scheme 1, with six lines of cross-frames) to a maximum of approximately 50 ft (15500 mm) (scheme 8, with four lines of cross-frames). The cost of increasing the size and weight of the compression flange may or may not outweigh the cost of fabricating and installing cross-frames to brace the compression flange.

Thus, although the cross-frames perform an important function in bracing the compression flange, there are ways to decrease the number of permanent cross-frames that may be more cost efficient than using permanent cross-frames spaced at 25 ft (7620 mm) or less.

2.4 ERECTION AIDS

Cross-frames are used during in the erection of steel girders in four ways: (1) bracing the compression flange, (2) maintaining the horizontal spacing between girders, (3) maintaining the cross-slope or vertical alignment of the girders, and (4) maintaining the plumbness of the girders (Mertz 1999).

With the cross-frames spaced at every 25 ft (7620 mm), erectors have installed only every second or third cross-frame as the compression flange bracing for the self-weight of the girder. The remaining cross-frames are then installed to provide the compression flange bracing needed for the girder self weight plus the weight of the wet concrete deck (Mertz 1999). This implies that, if the girders are designed to support the construction loads with fewer cross-frames, erection could be preformed with cross-frames at only 50 ft (15240 mm) or 75 ft (22860 mm).

2.5 CONCLUSIONS

The use of cross-frames spaced at more than the traditional 25 ft (7.6 m) offers several technical advantages for steel bridges: (1) fewer fatigue-prone attachments to the girders, and (2) more easily repaired damage from vehicle collisions with the bottom flanges. Eliminating cross-frames may lead to increases in the weight of the girders as the girder cross-section dimensions are increased to provide the required lateral-torsional buckling resistance. This increase in weight may not be significant (Ellis 1999).

Fabricating and installing cross-frames is an expensive process. Decreasing the number of cross-frames may decrease the erected cost of a bridge. If temporary cross-frames are used, the costs required to remove the temporary cross-frames must be considered. Also, the cost of re-installing the cross-frames when the deck is replaced must also be considered. In areas where deck replacements are infrequent, the cost of addressing possible fatigue cracking at cross-frame connections may outweigh the cost of temporary bracing when it is required.

Each option, and the related economic and safety benefits, should be weighed (Mertz 1996). The costs of cross-frames differ between fabricators and erectors, and it is difficult for a designer to choose the least cost solution in the typical design-bid-build process. However, all these items should be looked at to design the most economical cross-frame arrangement.

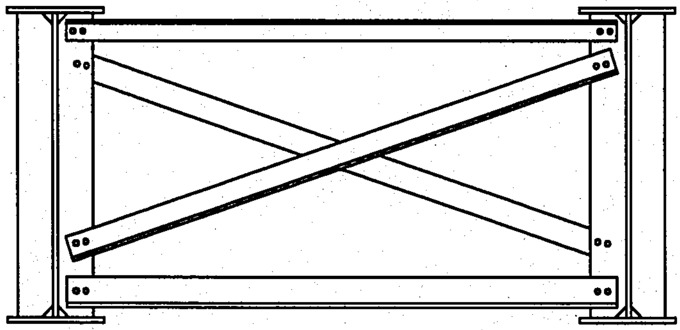


Figure 2.1 Typical Intermediate Cross-Frame

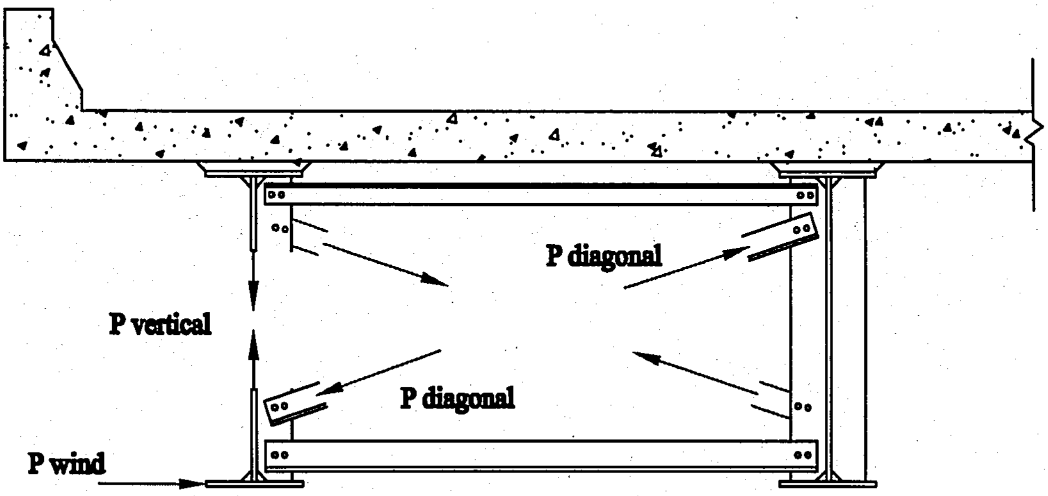


Figure 2.2 Wind Load Transmitted to the Deck at Intermediate Cross-Frames

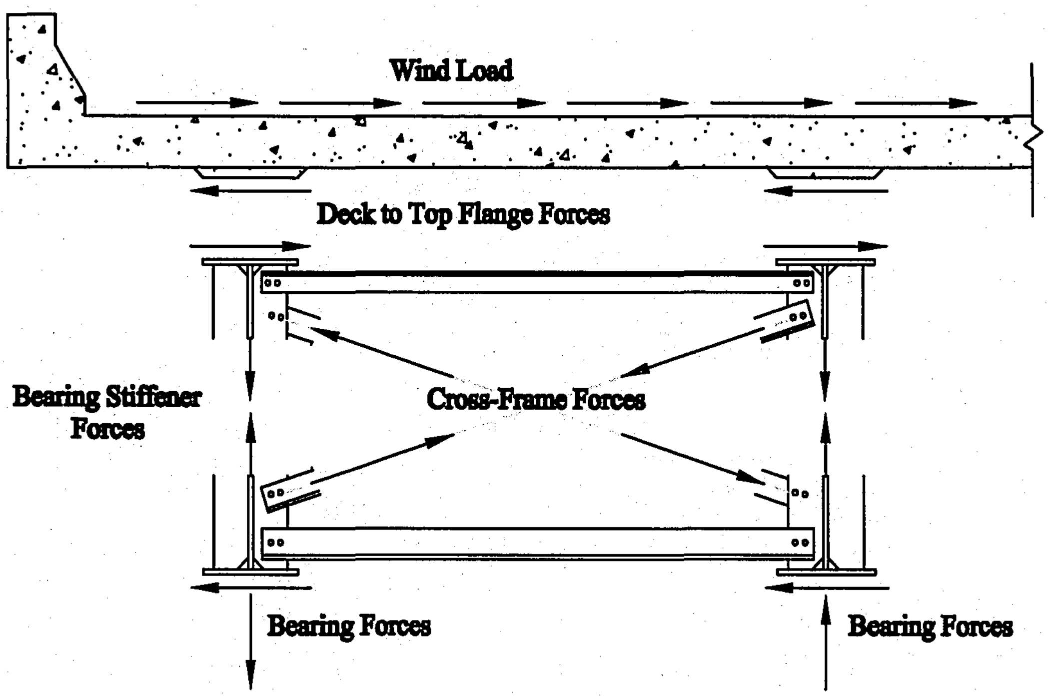


Figure 2.3 Wind Load Transmitted from the Deck to the Bearings by the End Cross-Frames

CHAPTER 3

PROTOTYPE BRIDGES, DESIGN LOADS, LOAD COMBINATIONS, AND LIMIT STATES

3.1 INTRODUCTION

This chapter outlines the basis for the design study of bridge girders with tubular flanges. The prototype bridges are described first. Then, the dimensions and layout of the bridges, as well as the dead and live loads for each bridge are outlined. The dead and live load effects (the bending moments and shears) that are used to design the girders are also discussed.

The chapter then outlines the information used to design the girders. This includes the load combinations and the limit states and related design criteria. The AASHTO LRFD bridge design specifications (AASHTO LRFD 1998) that were used in the design of the girders are summarized.

3.2 PROTOTYPE BRIDGE

The prototype bridges used in the design study are simply-supported, single span bridges. Two prototype bridge configurations are studied. One has four equally spaced girders supporting a concrete deck. The other is a through-girder bridge with two girders. Both prototype bridges have spans of 131.24 ft (40000 mm).

As shown in Figure 3.1, the four-girder bridge has a total width of 50 ft (15240 mm). The bridge is intended to carry two 12 ft (3658 mm) lanes of traffic with 13 ft (3962 mm) on each side for a shoulder and a parapet. However, the bridge was designed for four 11.5 ft (3505 mm) traffic lanes with 2 ft (610 mm) for a shoulder and a parapet on each side, in order to produce the maximum load effect on the girders. 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 (30 MPa). The four girders are spaced 12.5 ft (3810 mm) apart and the deck overhang is 6.25 ft (1905 mm). The bridge was designed with either conventional I-girders or tubular flange girders. The conventional I-girders were assumed to be fully-composite with the deck. The tubular flange girders were assumed to be either fully-composite with the deck or to be non-composite. Figure 3.1 shows a typical cross section with four tubular flange girders.

The second prototype bridge is a through-girder bridge. This bridge has two girders and was designed with tubular flange girders only. The total width of the deck of the two-girder bridge is 32 ft (9754 mm). There are two 12 ft (3658 mm) traffic lanes and two 4 ft (1219 mm) shoulders with parapets. The deck is assumed to be made of precast, prestressed segments that are 13 in (330 mm) thick. The two girders are assumed to be spaced 32 ft (98754 mm) apart. A typical cross section of the two-girder bridge can be seen in Figure 3.2.

The I-girders 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 tubular flange girders were assumed to be made from either ASTM A709 HPS 70W, or a high performance steel with a yield stress of 100 ksi (690 MPa). A combination of the two steels, with the tubular flange made from ASTM A709 HPS 70W and the web and bottom flange made from 100 ksi (690 MPa) high performance steel was also considered. When the tubular flange girders use only HPS 70W, the concrete in the tube is a normal strength concrete with a compressive strength of 4 ksi (30 MPa). When the 100 ksi (690 MPa) high performance steel is used, the concrete in the tube is a high strength concrete with a compressive strength of 8 ksi (55 MPa). The secondary steel components, including the stiffeners, the connection plates, and the cross frames, are assumed to be made from a conventional weathering steel, such as ASTM A709 50W, with a nominal yield stress of 50 ksi (350 MPa).

3.3 DESIGN LOADS

Each prototype bridge was designed for various dead and live load conditions. Lateral loads such as wind loads and earthquake loads were not considered in every bridge, although wind loading was considered for the four-girder prototype bridge with composite tubular flange girders.

The dead loads considered include the weight of all components of the structure, the wearing surface, and the attached appurtenances. The dead load is divided into two categories: (1) the weight of the bridge components and attachments to the girders (D_c) and (2) the weight of the wearing surfaces and utilities attached to the deck (D_w). D_c includes the weight of the deck, the cross-frames, the stay-in-place forms, and the self-weight of the girders. D_w includes the weight of the non-integral wearing surface and the parapets. The dead loads were computed as a weight per linear foot of bridge girder, and vary according to the characteristics of the prototype bridges. For example, the self-weight of the slab of the through-girder prototype bridge is larger than the self-weight of the slab of the four-girder prototype bridge. The numerical values of these loads are summarized in Tables 3.1, 3.2, 3.3, and 3.4.

The live loads (LL) consist of either a truck or tandem load acting (in some cases) coincident with a uniformly distributed lane load. The AASHTO LRFD bridge design specifications (AASHTO LRFD 1998) specifies the values and positions of these loads. Either one design truck or one design tandem is assumed to be on the bridge at a time, and this load is placed on the bridge to cause the greatest load effect on each girder.

The AASHTO LRFD design truck is an HS-20 truck. The Pennsylvania Department of Transportation (PennDOT Bridge Design Manual Part 4 1993) specifies 125% of the HS-20 loading (an HS-25 truck) for bridges in Pennsylvania. Since the prototype bridges are assumed to be in Pennsylvania, the bridges are designed for this loading. The HS-25 truck includes three axle loads, the first is 10 kips (43.75 kN), and the second and the third are 40 kips (181.25 kN) (The metric units are directly from the SI version of AASHTO LRFD (1998)). There is 14 ft. (4300 mm) between the first and second axle and 14 to 30 ft (4300 to 9000 mm) between the second and the third axle. The distance between the second and third load is varied to cause the greatest load effect on each girder.

The tandem load is a military loading which consists of a pair of 30 kip (110 kN) axles spaced 4 ft (1200 mm) apart. These loads are 125% of the AASHTO LRFD design tandem. The design tandem produces live load effects greater than the design truck in spans under 40 ft (12192 mm). Since the design span of the prototype bridges is 131.24 ft (40000 mm), the design truck governs.

A design lane load must also be applied to the prototype bridge. The lane load is used to model a line of cars along the lane. The lane load is a 0.64 k/ft (9.3 N/mm) force distributed along the length of the bridge and across a 10 ft (3000 mm) design lane.

An HS-20 truck with the axle spacing fixed at 14 ft (9000 mm) is used to design for fatigue. The HS-20 truck includes three axle loads, one 8 kip (35 kN) load and two 32 kip (145 kN) loads. The fatigue load consists of one HS-20 truck placed where it causes the greatest load effect on each girder. The design lane load is not included in the fatigue load.

The live loads are increased by a dynamic load allowance to account for the dynamic response. For most load cases, the effects of the design truck or tandem are increased 33% (AASHTO LRFD 1998). The dynamic load allowance is 15% for the fatigue load effects. The lane load is not increased by a dynamic load allowance.

The live loads are not directly applied each girder. The loads are transmitted though the deck to the girders, and then to the supporting substructure. The AASHTO LRFD specifications (1998) provide guidelines on the distribution of each axle load to the girders. Distribution factors are applied to the live loads to determine the load applied to a girder, and these distributed loads are used to calculate values of moment and shear for the girders. The distribution factors can be calculated by using formulas found in the specifications or by utilizing the lever rule. The distribution factor formulas depend on the type of deck and the spacing between the girders. The lever rule is only a function of the spacing between the girders. In the lever rule, the fraction of live load distributed to each girder is calculated by placing the loads on the bridge and summing moments about the adjacent girder line. The lever rule must be used if the characteristics of the bridge do not meet the requirements of the AASHTO LRFD distribution formulas. For the through-girder prototype bridge, the lever rule must be used, because the distribution factor formulas require that the bridge have three or more girders (PennDOT Bridge Design Manual Part 4 1993).

For the four-girder prototype bridge, the distribution factor for an exterior girder was used to design the girders. The interior girders were assumed to be similar to the exterior girders

Figures 3.3, 3.4, 3.5, 3.6, and 3.7 summarize the unfactored dead and live load moment envelopes and shear envelopes used to design the prototype bridges. The moment envelopes for the four-girder bridge with I-girders are shown in Figures 3.3 and 3.4. The envelopes for live load plus dynamic load allowance (IM), for fatigue plus dynamic load allowance (IM), and for dead load due to wearing surface and parapets (D_w) are the same for all the four-girder prototype bridges that were studied. The envelopes for dead load due to bridge components (D_c) vary for the different types of girders used in the bridge. The D_c moment envelop for each type of girder is shown in Figure 3.5. The moment and shear envelopes for the through-girder prototype bridge are shown in Figures 3.6 and 3.7.

3.4 DESIGN CRITERIA

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

$$\eta \Sigma \gamma_i Q_i \leq \varphi R_n \qquad \text{(Eq. 3.1)}$$

where, Q_i is the force effect, R_n is the nominal resistance, γ_i is the statistically based load factor, φ is the statistically based resistance factor, and η is the load modification factor.

The AASHTO LRFD bridge design specifications (1998) include four categories of limit states: (1) strength limit states, (2) service limit states, (3) fatigue and fracture limit states, and (4) extreme event limit states.

For each of the limit states, the load effects from the design loads previously described are combined and factored to calculate the total force effect on the left-hand side of Eq. 3.1. Each limit state requires a specific load combination, with different load factors being applied to the design loads. The load combinations used in this design study correspond to the strength I, service II, and fatigue limit states. These combinations are as follows:

Strength I Load Combination

$$1.25D_c + 1.50D_w + 1.75(LL + IM) \qquad \text{(Eq. 3.2)}$$

Service II Load Combination

$$1.00D_c + 1.30D_w + 1.00(LL + IM) \quad \text{(Eq. 3.3)}$$

Fatigue Load Combination

$$0.75(LL + IM) \quad \text{(Eq. 3.4)}$$

The strength I load combination relates to normal use of the bridge. To check the constructability of the prototype bridges, a construction limit state load combination is developed from the strength I load combination by eliminating the D_w and LL terms. The service II limit state includes a load combination intended to control yielding of the steel girders. The fatigue load combination considers live (truck) loads on the bridge.

The strength III and strength V limit states of the AASHTO LRFD specifications (1998) were checked for the four-girder prototype bridge with composite tubular flange girders. The strength III and strength V load combinations include wind load effects (WS) as follows:

Strength III Load Combination

$$1.25D_c + 1.50D_w + 1.40WS \quad \text{(Eq. 3.5)}$$

Strength V Load Combination

$$1.25D_c + 1.50D_w + 1.35(LL + IM) + 0.40WS \quad \text{(Eq. 3.6)}$$

The strength III limit state includes a load combination to account for very high wind load effects and very little live load effect, assuming that high winds will keep much of the traffic off of the bridge. The strength V load combination relates to normal vehicle use of the bridge with a average wind load.

The service I load combination relates to normal operational use of the bridge. This load combination was used to calculate the deflections of the bridge, and is as follows:

Service I Load Combination

$$1.00D_c + 1.00D_w + 1.00(LL + IM) \qquad \text{(Eq. 3.7)}$$

3.4.1 STRENGTH I LIMIT STATE

For a steel girder bridge, the factored bending moments and shear load effects (from Eq. 3.2) must be less than the factored nominal resistance of the steel girder section. Other factors, such as stability of the girders, are also checked. The nominal strength of the girders is calculated according to AASHTO LRFD specifications (AASHTO LRFD 1998). These specifications are summarized in this section, along with any modifications made for tubular flange girders. The current AASHTO LRFD specifications for web stability and shear strength are used in this study without modification.

Flexural Resistance – Compact Section

The design criteria used in this study for I-girders are identical to those given in the AASHTO LRFD specifications (AASHTO LRFD 1998). The criteria used for the tubular flange girders are similar to those in the AASHTO LRFD specifications. The I-girders and tubular flange girders designed for the four-girder prototype bridge are compact sections (under service conditions) and appropriate design criteria were developed for the strength I limit state and used as discussed in this section.

For the composite I-girders of the four-girder prototype bridge, the webs were designed to be compact (local buckling of the compression flange is not considered for positive moment regions). For the tubular flange girders, the web and tube were stocky enough to be compact. The composite I-girders and composite tubular flange girders are fully braced by the deck (in service). The non-composite tubular flange girders were assumed to be sufficiently connected to the deck to be braced by the deck in service. Therefore, all of the girders designed for the four-girder prototype bridge were treated as compact sections for the strength I limit state.

The non-composite I-girders of the through-girder prototype bridge were not braced by the deck and were not treated as compact sections. Flexural criteria for these girders are described in the following subsection.

The factored flexural resistance for compact sections is expressed in terms of the following equation:

$$M_r = \varphi_f M_n \qquad \text{(Eq. 3.8)}$$

where, φ_f is the resistance factor for flexure (1.00), M_n is the nominal moment resistance, and M_r is the factored moment resistance. As shown in Eq. 3.1, the factored flexural resistance must be less than the factored load effects. For the strength I limit state, the factored load effects are given by Eq. 3.2.

In the AASHTO LRFD specifications (1998) for I-girders, the nominal moment resistance, M_n, of a composite girder in positive flexure, depends on the ratio of D_p to D', where D_p is the distance from top of the slab to the neutral axis at the plastic moment, and D' is the ductility factor defined by Wittry(1993). The ductility factor, D', is based on the strain required for strain hardening to occur in the bottom fibers of the steel girder. If the ratio of D_p to D' is less than or equal to 1.0, M_n is equal to the calculated plastic moment. If the ratio of D_p to D' is equal to 5.0, the nominal moment resistance is 85% of the calculated yield moment. For ratios between 1.0 and 5.0, a linear transition defines the nominal moment resistance. The purpose of this reduction of M_n below the plastic moment is to ensure adequate ductility of the composite section.

In this study, the nominal moment resistance under positive flexure of a composite or non-composite tubular flange girder is based on an equivalent rectangular stress block for the concrete and elastic perfectly plastic stress-strain behavior for steel, as shown in Figure 3.8. For a composite tubular flange girder, with the tube in compression, the maximum usable strain at the extreme concrete compression fiber, which is at the top of the deck, is assumed to be 0.003, the strain at which concrete is assumed to begin to fail. For a non-composite section, the maximum usable strain is assumed to be 0.003 at the top of the concrete in the steel tube.

Flexural Resistance – Non-Compact Section

During construction of composite I-girders and composite tubular flange girders, the compression flange is not braced by the deck and lateral-torsional buckling of the girders should be considered. When the girders become composite with the deck, the deck braces the girders and, assuming local buckling of the web and compression flange (of tube) is prevented, the girders can be designed as compact. All of the girders designed for the four-girder prototype bridge are assumed to be braced by the deck under service conditions, and the criteria outlined in the previous subsection were applied for the strength I limit state. For construction conditions, the criteria in this subsection were applied. For the through-girder prototype bridge, the girders are not braced under service conditions and the criteria given below were applied for the strength I limit state.

For non-compact sections designed by the AASHTO LRFD specifications (1998), yielding or lateral-torsional buckling may control the flexural resistance. The unbraced length of a girder, L_b is the distance between cross-frames when the girder is not braced by the bridge deck. When L_b is less than L_p, the lateral bracing limit for flexural resistance governed by yielding, lateral-torsional buckling does not control, and the nominal bending moment can be taken as either the yield moment, M_y, or the plastic moment, M_p. In the AASHTO LRFD specifications (1998), the limit for non-compact sections is M_y. If L_b is greater than L_p, lateral-torsional buckling must be considered. L_p is calculated as follows:

$$L_p = 1.76 r_t \sqrt{\frac{E}{F_{yc}}} \qquad \text{(Eq. 3.9)}$$

where, r_t is the radius of gyration of the compression flange of the girder taken about the vertical axis, E is the modulus of elasticity, and F_{yc} is the specified minimum yield strength of the compression flange.

To determine whether elastic or inelastic buckling controls, the unbraced length should be compared to L_r, the lateral bracing limit for flexural resistance governed by inelastic lateral torsional buckling. For I-girders, L_r is calculated using the AASHTO LRFD specifications (1998). For tubular flange girders, L_r is calculated as follows:

$$L_r = \sqrt{\frac{4\pi^2 E I_{yc}}{M_y^2}\left[GK_T + \sqrt{(GK_T)^2 + \left(\frac{dM_y}{2}\right)^2}\right]} \qquad \text{(Eq. 3.10)}$$

where, I_{yc} is the moment of inertia of the compression flange about the vertical axis, G is the shear modulus, K_T is the St. Venant torsional constant, d is the depth of cross-section, and M_y is the yield moment of the cross-section. To calculate M_y, an equivalent rectangular stress block is used for concrete within the steel tube.

When the unbraced length, L_b, is greater than L_r, elastic lateral-torsional buckling controls the girder. The nominal flexural resistance is calculated as follows:

$$M_n = 3.14EC_bR_h\left(\frac{I_{yc}}{L_b}\right)\sqrt{0.772\left(\frac{K_T}{I_{yc}}\right)+9.87\left(\frac{d}{L_b}\right)^2} \leq R_hM_y \qquad \text{(Eq. 3.11)}$$

where, C_b is the moment gradient correction factor and R_h is the hybrid factor.

When L_b is between L_p and L_r, inelastic lateral-torsional buckling controls the girder and the nominal flexural resistance is calculated as follows:

$$M_n = C_bR_hM_y\left[1-0.5\left(\frac{L_b-L_p}{L_r-L_p}\right)\right] \leq R_hM_y \qquad \text{(Eq. 3.12)}$$

where, L_p is calculated by using Eq. 3.9.

The AASHTO LRFD specifications treat I-girders with stocky webs differently than those with slender webs. For the web to be considered stocky, the following condition must be satisfied:

$$\frac{2D_c}{t_w} \leq \lambda_b\sqrt{\frac{E}{F_{yc}}} \qquad \text{(Eq. 3.13)}$$

where, D_c is the depth of web in compression, t_w is the web thickness, and λ_b is a coefficient related to the boundary conditions provided to the web by the flanges.

For stocky web I-girders, cross section distortion is neglected, and the St. Venant torsional resistance, represented by K_T, is included in calculating the elastic lateral-torsional buckling resistance (as shown in Eq. 3.11). In addition, inelastic lateral torsional buckling is ignored for stocky web I-girders by the AASHTO LRFD specifications (1998). For slender web I-girders, the St. Venant torsional stiffness is neglected in calculating the elastic lateral-torsional buckling resistance. In addition, the inelastic lateral-torsional buckling resistance is approximated by a straight line transition between the elastic buckling resistance and the yield moment, for slender web I-girders (AASHTO LRFD 1998).

For tubular flange girders, the torsional stiffness is much greater than that of I-girders, and should not be neglected. To take advantage of the torsional stiffness, the web should be stocky, to avoid cross-section distortion. However, inelastic lateral-torsional buckling should not be neglected. Eq. 3.12 is a straight line transition proposed by Kim (2001) for stocky web girders. Figure 3.9 shows the proposed straight line transition (PSLT) that extends from the elastic buckling resistance at an unbraced length L_r to the yield moment (M_y). The figure shows that by neglecting inelastic lateral-torsional buckling of stocky web girders, the AASHTO LRFD specifications (1998) assume a much higher lateral-torsional buckling resistance than various other specifications. In this study, the proposed straight line transition is used with stocky web tubular flange girders to provide a more reasonable estimate of the lateral-torsional buckling resistance for L_b between L_p and L_r.

To take advantage of torsional stiffness and resulting lateral-torsional buckling resistance, the tubular flange girders are designed to have stocky webs in this study.

Shear Resistance

The factored shear resistance, V_r, is expressed as

$$V_r = \varphi_v V_n \qquad \text{(Eq. 3.14)}$$

where, φ_v is the resistance factor for shear (1.00) and V_n is the nominal shear resistance. Again, the factored shear resistance must be greater than the factored load effects (Eq. 3.2) as shown in Eq. 3.1.

The current design specifications (AASHTO LRFD 1998) are used to determine the nominal shear resistance of the web for both I-girders and tubular flange girders. The web is assumed to carry all of the shear force in the tubular flange girders, which is a conservative assumption. The web shear resistance mainly depends on the web slenderness ratio, D/t_w, where D is the web depth and t_w is the web thickness. For an unstiffened web, the shear resistance is categorized by the following equations:

if $\frac{D}{t_w} \leq 2.46\sqrt{\frac{E}{F_{yw}}}$, then:

$$V_n = 0.58 F_{yw} D t_w \qquad \text{(Eq. 3.15)}$$

if $2.46\sqrt{\frac{E}{F_{yw}}} < \frac{D}{t_w} \leq 3.07\sqrt{\frac{E}{F_{yw}}}$, then:

$$V_n = 1.48 t_w^2 \sqrt{E F_{yw}} \qquad \text{(Eq. 3.16)}$$

if $\frac{D}{t_w} > 3.07\sqrt{\frac{E}{F_{yw}}}$, then:

$$V_n = \frac{4.55 t_w^3 E}{D} \qquad \text{(Eq. 3.17)}$$

where, F_{yw} is the specified minimum yield strength of the web.

The shear resistance of stiffened webs is also given by equations provided in the AASHTO LRFD specifications (1998). The AASHTO LRFD specifications account for tension field action in stiffened interior web panels (the panels away from the ends of a bridge girder). The shear resistance in the end panels adjacent to the end of a bridge girder is less than in the interior panels because tension field action is not included.

3.4.2 STRENGTH III LIMIT STATE AND STRENGTH V LIMIT STATE

The strength III and strength V limit states consider load combinations with wind load effects (Eq. 3.5 and Eq. 3.6). The strength III and strength V limit states were checked for only the composite tubular flange girders.

The current design specifications (AASHTO LRFD 1998) are used to determine the wind load effects and the nominal moment resistance for tubular flange girders for the strength III and the strength V limit states. The wind load is carried by lateral bending of the bottom flange of the girder, either between the cross-frames, or between the bearings (if no interior cross-frames are present). When interior cross-frames are present, the maximum lateral moment in the flange due to the factored wind loading is as follows:

$$M_w = \frac{W \cdot L_b}{10} \qquad \text{(Eq. 3.18)}$$

where, W is the factored wind force per unit length applied to the flange and L_b is the spacing of the cross frames.

If no interior cross frames are present, the maximum lateral moment in the flange due to the factored wind loading is as follows:

$$M_w = \frac{W \cdot L}{8} \qquad \text{(Eq. 3.19)}$$

where, L is the span of the bridge. The denominator of Eq. 3.18 (10) is different from the denominator of Eq. 3.19 (8) because the cross-frames act as intermediate supports to the flange, and the moment is assumed to be in between the maximum moment in a beam with two pinned supports under uniform load and the maximum moment in a beam with two fixed supports under uniform load. When the flange spans between the bearings, the flange is assumed to act like a beam with two pinned supports under uniform load.

The calculated lateral wind moment is carried by a pair of fully yielded segments at either tip of the bottom flange. The size of each of these segments is calculated according the AASHTO LRFD specifications (1998). The width of the segment on either side of the flange, is given by:

$$b_w = \frac{b_{fb} - \sqrt{b_{fb}^2 - \dfrac{4M_w}{t_{rb}F_{yb}}}}{2} \leq \frac{b_{rb}}{2} \qquad \text{(Eq. 3.20)}$$

where, b_{fb} is the width of the bottom flange, t_{fb} is the thickness of the bottom flange, F_{yb} is the specified yield strength of the bottom flange, and M_y is the maximum lateral moment as calculated from Eq. 3.18 and Eq. 3.19.

These yielded segments are then discounted from the section, and the reduced section is assumed to resist the vertical loads. The resistance to the vertical loads is calculated as discussed in the previous section.

3.4.3 SERVICE II LIMIT STATE

To prevent permanent deflections, current AASHTO LRFD design specifications (1998) for I-girders define limit states for both composite girders and non-composite girders in terms of stress. For sections that will be composite in the final condition, the flange bending stresses are calculated superposing the stress due to the component dead loads (D_c) acting on the non-composite girder cross-section (before the girder is composite with the deck), the stress due to the superimposed dead loads (D_w) acting on the long-term composite girder cross-section, and the stress due to live loads (LL) acting on short-term composite girder cross-section. To calculate the flange stresses of a section that will be non-composite in the final condition, the dead loads and live loads are considered to be acting on the non-composite girder cross-section. In calculating these bending stresses, the load effects from D_c, D_w, and LL are factored according to Eq. 3.3. The factored stresses should be less than the factored allowable stresses, as shown in Eq. 3.1.

In this study, two different methods are used to check the service II limit state. For a composite tubular flange girder, a superposition approach is used to calculate flange stresses, which is similar to the approach in the AASHTO LRFD specifications (1998). However, to calculate the flange bending stresses due to D_c acting on the non-composite girder cross-section (before the girder is composite with the deck) an equivalent rectangular stress block for the concrete within the tube is used. To calculate the stresses due to D_w, which acts on the long-term composite girder cross-section, and due to the live load, which acts on the short-term composite girder cross-section, the concrete in the tube is neglected. This approach is based on the assumption that the concrete is already fully stressed due to D_c alone. For a non-composite tubular flange girder, the dead and live loads are considered to act on the non-composite girder cross-section, and an equivalent stress block is used for the concrete within the tube.

The design criteria for the service II limit state is expressed as follows:

$$f_f \leq 0.95 R_b R_h F_{yf} \qquad \text{(Eq. 3.21)}$$

where, f_f is the flange bending stress caused by the factored loading, R_b is the load-shedding factor, and F_{yf} is the specified minimum yield strength of the tension flange.

3.4.4 SERVICE I LIMIT STATE

The service I limit state includes a load combination (Eq. 3.7) that is applied to limit the deflection of the bridge under live load. The live loads used when calculating the deflection is a result of either the design truck alone, or the result of 25% of the design truck along with the design lane load. In all cases in this study, the design truck alone governed the deflection.

When calculating the deflection, all the girders are assumed to act together, all design lanes are loaded, and all the components are assumed to deflect evenly. For composite design, the cross-section used in the deflection calculations includes the entire width of the roadway and the structurally continuous portions of the roadway and parapets, while for non-composite design, the cross-section used in the deflection calculations includes only the girders.

The AASHTO LRFD specifications (1998) and PennDOT bridge design specifications (PennDOT Bridge Design Manual 4, 1993) provide a limit on elastic deflections under live load. For bridges with pedestrian traffic, the recommended maximum deflection is 1/1000 of the span of the bridge, and for bridges without pedestrians, the recommended maximum deflection is 1/800 of the span (PennDOT Bridge Design Manual 4 1994).

The prototype bridges were not designed to meet the criteria of the deflection specifications. After the design of the prototype bridges with tubular flanges was completed using the strength I, service II, and fatigue limit states, the deflection of the tubular flange prototype girders was calculated and compared to the allowable limits.

3.4.5 FATIGUE LIMIT STATE

Fatigue is categorized as either load-induced fatigue or distortion-induced fatigue. In this study, only load-induced fatigue is considered. The attachment of the transverse stiffener or a cross-frame connection plate to the tension flange is the fatigue detail that is checked. Using the load factor given in Eq. 3.4, Eq. 3.1 can be written in terms of fatigue load and fatigue resistance for the fatigue limit state as follows:

$$\gamma(\Delta f) \leq (\Delta F)_n \qquad \text{(Eq. 3.22)}$$

where, γ is the load factor (0.75, as shown in Eq. 3.4), (Δf) is the live load stress range due to the passage of the fatigue load, and $(\Delta F)_n$ is the nominal fatigue resistance.

The AASHTO LRFD specifications (1998) provide eight fatigue detail categories. The categories range from E′, which has the worst fatigue resistance, to A, which has the best fatigue resistance. For example, the ends of a partial-length cover plate without end welds are a category E′ detail. Each detail category has an associated fatigue resistance. The fatigue resistance is quantified as the number of cycles at a specific stress range that leads to fatigue failure. This is summarized in an S-N curve, where the S is the stress range, and the N is the number of cycles. The nominal fatigue resistance for each category as a function of the total number of stress cycles as can be seen in the S-N curve of Figure 3.10.

The horizontal lines on the S-N curve (Figure 3.10) are the constant-amplitude fatigue thresholds (CAFL). If the stress range for a fatigue detail is below the CAFL ($(\Delta F)_{th}$), the detail is expected to have an infinite fatigue life. Table 3.5 lists $(\Delta F)_{th}$ for each detail category. For bridges with high volumes of truck traffic, fatigue design requires that the maximum stress range at a critical fatigue detail be less than the $(\Delta F)_{th}$. The maximum stress range is taken as twice the factored live load stress range (i.e. $2(\Delta f)$) due to the passage of the fatigue load, or, alternatively, the nominal fatigue resistance is taken as ½ $(\Delta F)_{th}$. (AASHTO LRFD 1998)

The S-N curve can be expressed by the following equation for the nominal fatigue resistance:

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}} \geq \frac{1}{2}(\Delta F)_{TH} \qquad \text{(Eq. 3.23)}$$

where, A is the 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.

The number of cycles a bridge is subjected to during a 75 year design life, N, is calculated as follows:

$$N = 365 \cdot 75 \cdot n \cdot ADTT_{SL} \qquad \text{(Eq. 3.24)}$$

where, n is the number of stress range cycles per truck passage, and $ADTT_{SL}$ is the single-lane average daily truck traffic.

In this study, the prototype bridges are assumed to have high volumes of truck traffic, therefore, the nominal fatigue resistance is taken as ½ $(\Delta F)_{th}$.

Table 3.1 Dead Loads for Prototype Bridge with Four I-Girders

Type	Component	Calculation	Load/Length
DC	Slab	$0.15\frac{k}{ft^3}*\frac{10in}{12\frac{in}{ft}}*12.5ft$	$1.56\frac{k}{ft}$
DC	Concrete Haunch	$0.15\frac{k}{ft^3}*\frac{20in}{144\frac{in^2}{ft^2}}*3in$	$0.06\frac{k}{ft}$
DC	Steel Girder (assume 80 in^2 steel area)	$0.49\frac{k}{ft^3}*\frac{80in^2}{144\frac{in^2}{ft^2}}$	$0.27\frac{k}{ft}$
DC	Secondary Steel	0.10*steel girder wt.	$0.03\frac{k}{ft}$
DC	Stay-in-place forms	$0.015\frac{k}{ft^2}*12.5ft$	$0.19\frac{k}{ft}$
DW	Future Wearing Surface	$0.035\frac{k}{ft^2}*12.5ft$	$0.42\frac{k}{ft}$
DW	Miscellaneous (Parapet, railing, lights, etc.)	(assumed)	$0.25\frac{k}{ft}$

Table 3.2 Dead Loads for Prototype Bridge with Four Composite Tubular Flanges

Type	Component	Calculation	Load/Length
DC	Concrete Haunch	$0.15\frac{k}{ft^3}*\frac{14.25in}{144\frac{in^2}{ft^2}}*3in$	$0.04\frac{k}{ft}$
DC	Steel Girder (assume 80 in^2 steel area)	$0.49\frac{k}{ft^3}*\frac{80in^2}{144\frac{in^2}{ft^2}}$	$0.27\frac{k}{ft}$
DC	Concrete Tube (assuming 6in radius)	$0.15\frac{k}{ft^3}*\pi*\frac{(6in)^2}{144\frac{in^2}{ft^2}}$	$0.12\frac{k}{ft}$

Table 3.3 Dead Loads for Prototype Bridge with Four Non-Composite Tubular Flange Girders

Type	Component	Calculation	Load/Length
DC	Concrete Haunch	$0.15\frac{k}{ft^3}*\frac{16.25in}{144\frac{in^2}{ft^2}}*3in$	$0.05\frac{k}{ft}$
DC	Steel Girder (assume 140 in^2 steel area)	$0.49\frac{k}{ft^3}*\frac{140in^2}{144\frac{in^2}{ft^2}}$	$0.48\frac{k}{ft}$
DC	Concrete Tube (assuming 8in radius)	$0.15\frac{k}{ft^3}*\pi*\frac{(8in)^2}{144\frac{in^2}{ft^2}}$	$0.21\frac{k}{ft}$

Table 3.4 Dead Loads for Through-Girder Prototype Bridge with Tubular Flange Girders

Type	Component	Calculation	Load/Length
DC	Slab	$0.15\frac{k}{ft^3}*\frac{13in}{12\frac{in}{ft}}*\frac{32ft}{2}$	$2.6\frac{k}{ft}$
DC	Steel Girder (assume 140 in^2 steel area)	$0.49\frac{k}{ft^3}*\frac{140in^2}{144\frac{in^2}{ft^2}}$	$0.48\frac{k}{ft}$
DC	Secondary Steel	0.10*steel girder wt.	$0.05\frac{k}{ft}$
DC	Concrete Tube (assuming 11.5in radius)	$0.15\frac{k}{ft^3}*\pi*\frac{(11.5in)^2}{144\frac{in^2}{ft^2}}$	$0.43\frac{k}{ft}$
DW	Future Wearing Surface	$0.035\frac{k}{ft^2}*\frac{32ft}{2}$	$0.49\frac{k}{ft}$
DW	Miscellaneous (Parapet, railing, lights, etc.)	(assumed)	$0.57\frac{k}{ft}$

Table 3.5 - Constant Amplitude Fatigue Thresholds
(AASHTO LRFD Bridge Design Specifications 1998)

Detail Category	$(\Delta F)_{th}$ (ksi)	$(\Delta F)_{th}$ (MPa)
A	24.0	165
B	16.0	110
B'	12.0	82.7
C	10.0	69
C'	12.0	82.7
D	7.0	48.3
E	4.5	31.0
E'	2.6	17.9

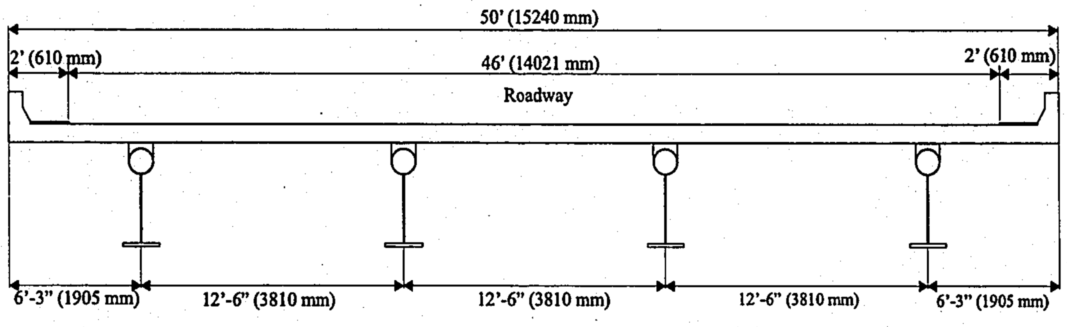


Figure 3.1 Four-Girder Prototype Bridge

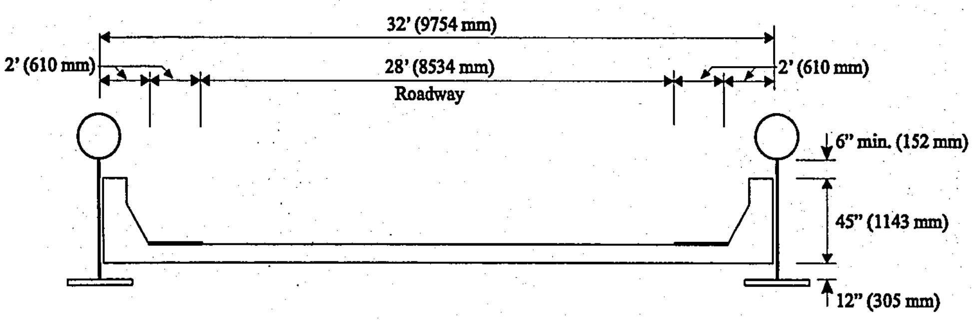


Figure 3.2 Two-Girder Prototype Bridge

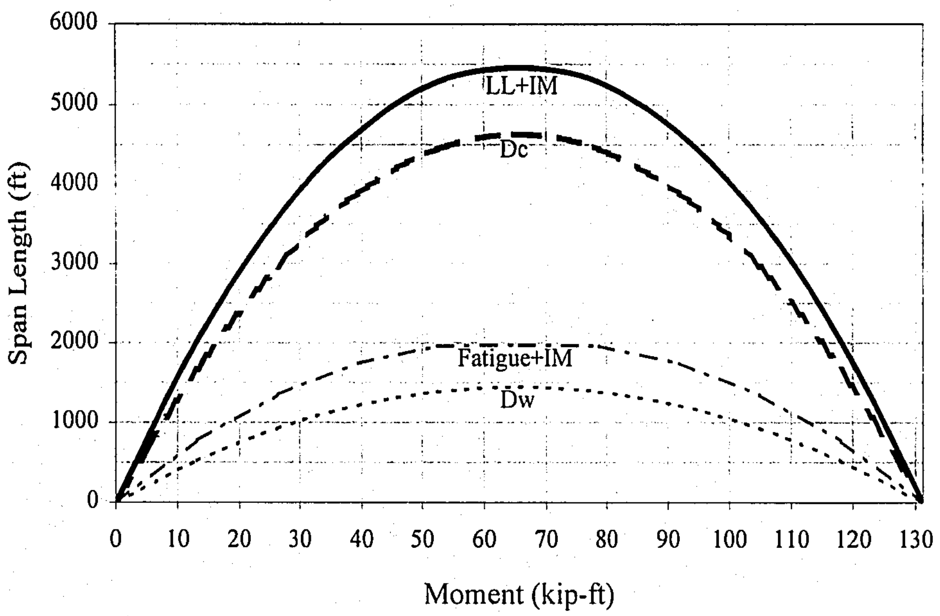


Figure 3.3 Unfactored Dead and Live Load Moment Envelopes for Four-Girder Prototype Bridge with I-Girders

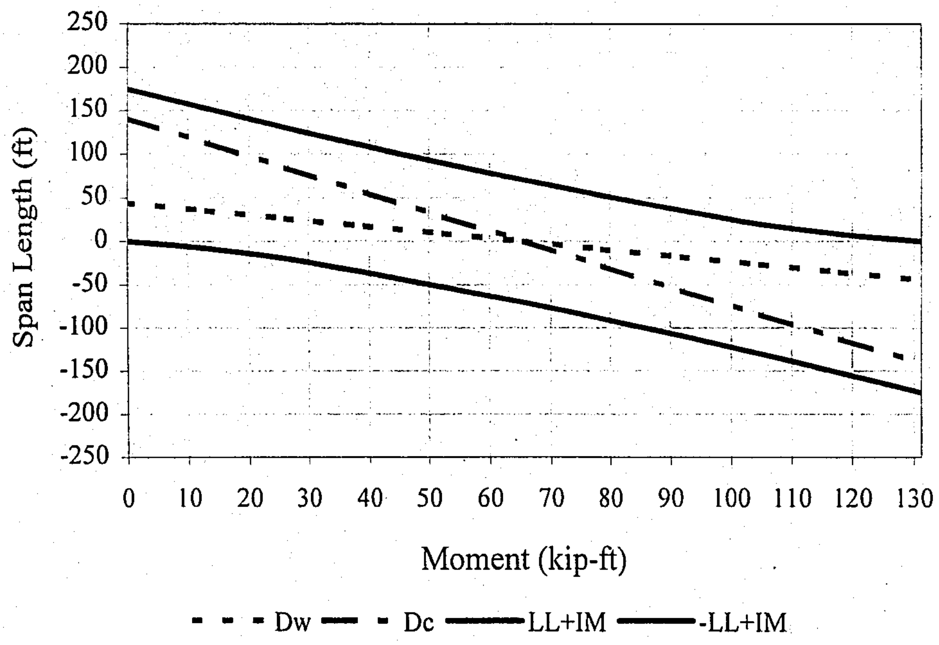


Figure 3.4 Unfactored Dead and Live Load Shear Envelopes for Four-Girder Prototype Bridge with I-Girders

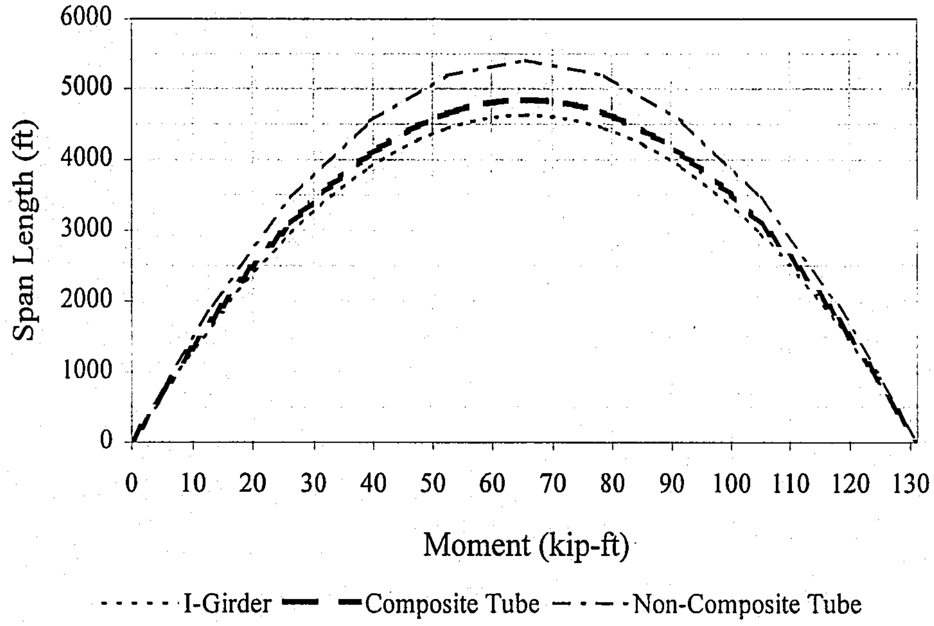


Figure 3.5 Unfactored D_c Moment Envelopes for Four-Girder Prototype Bridge

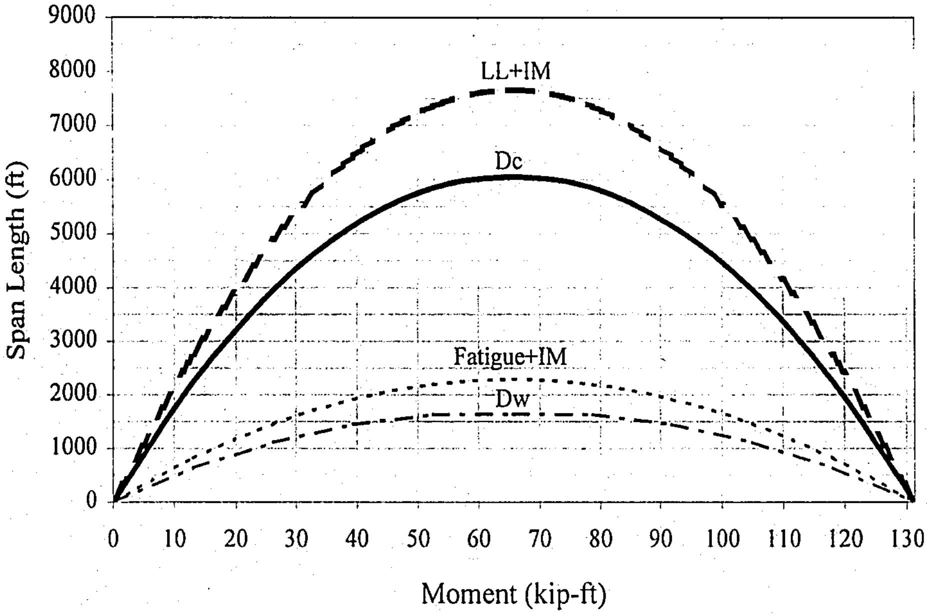


Figure 3.6 Unfactored Dead and Live Load Moment Envelopes for Two-Girder Prototype Bridge

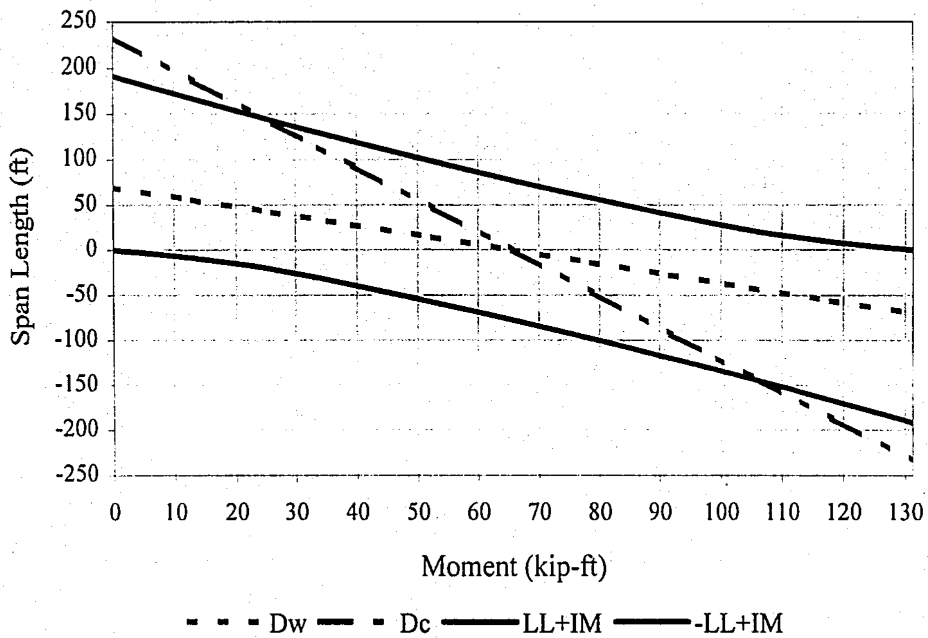


Figure 3.7 Unfactored Dead and Live Load Shear Demands for Two-Girder Prototype Bridge

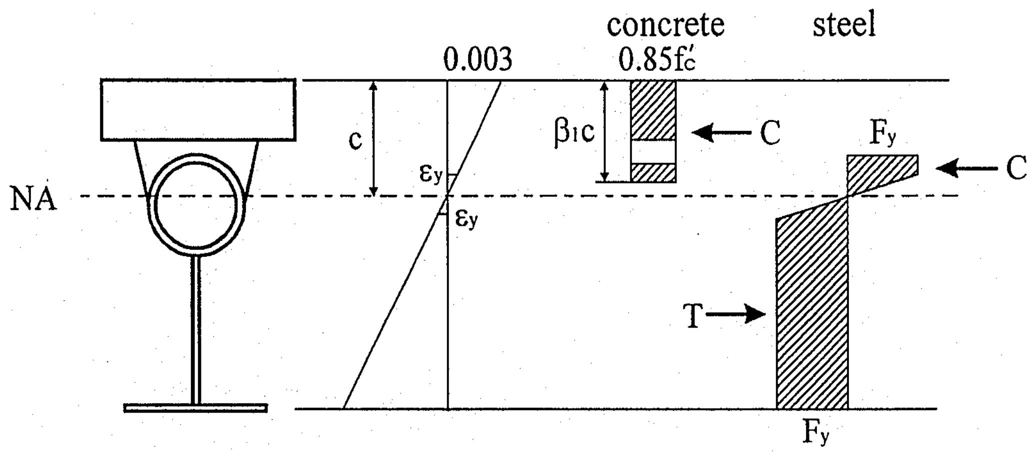


Figure 3.8 Positive Nominal Moment Resistance of the Composite Section

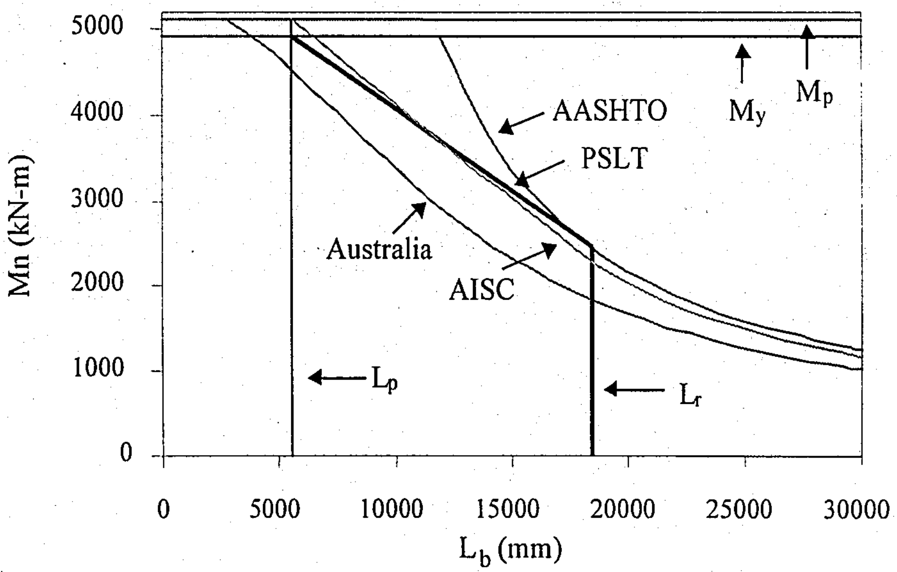


Figure 3.9 Lateral-Torsional Buckling Resistance

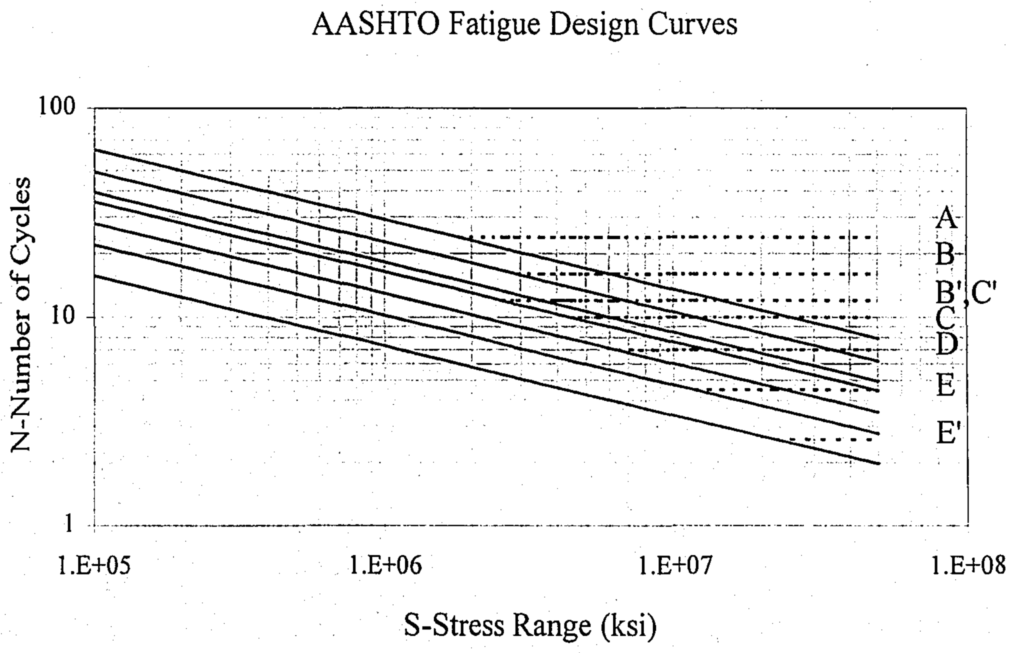


Figure 3.10 S-N Curve (AASHTO LRFD Bridge Design Specifications 1998)

CHAPTER 4

PROTOTYPE BRIDGE DESIGN PARAMETERS

4.1 INTRODUCTION

As described in Chapters 5 and 6, design studies of the prototype bridges were carried out. The design studies developed minimum weight designs of the prototype bridges as parameters such as the number of cross-frames and stiffeners were varied. The minimum weight designs were compared to show the potential advantages and disadvantages of using tubular flange girders in the prototype bridges.

There are five parameters considered in the design study and discussed in this chapter: (1) arrangement of cross-frames, (2) locations of plate transitions, (3) arrangement of stiffeners, (4) fatigue resistance (fatigue detail category) for stiffener and cross-frame connection plate welds, and (5) plate thickness. These parameters are discussed below.

4.2 CROSS-FRAME ARRANGEMENT

The number of interior cross frames (cross-frames between the bearings) affects the cost of fabricating and erecting a steel bridge. In the past, an economical bridge used the minimum amount (weight) of steel. As the cost of labor has increased, the fabrication effort has become equally important. The cost of fabricating and installing cross-frames is an important consideration in bridge design. Interior cross-frames cannot be omitted, however, because they still serve important functions as discussed in Chapter 2. The function of cross-frames as girder compression flange bracing (in the positive moment region) during construction is the only function considered in the design study.

Ellis (1999) studied the influence of cross-frame arrangement on the weight of I-girders designed for a 131.24 ft (40000 mm) four-girder prototype bridge. Ellis studied 8 different cross-frame arrangements, from scheme 1, which has 6 cross-frames with a 26.25 ft (8000 mm) spacing, to scheme 8, which has 4 cross-frames, with a maximum spacing of 50.85 ft (15500 mm). These cross-frame arrangements are considered in the present study. Two additional cross frame arrangements have been added for the present study: scheme 9, which has 3 cross-frames, one at each end and one at the midspan, and scheme 10, which has 2 end cross-frames and no interior cross-frames. A summary of the cross-frame arrangements can be seen in Table 4.1. Figure 4.1 shows schematics of some of the cross-frame arrangements. Schemes 1, 8, and 9, which are shown in Figure 4.1, are the arrangements most often used in this study.

Ellis (1999) found that the girder weight increases as the number of cross-frames decrease. The weight increase is related to the function of the cross-frames as girder compression flange bracing. During construction, only the cross-frames brace the girders. As the number of cross-frames decreases and the spacing between the cross-frames increases, more steel is required in the compression flange to prevent lateral-torsional buckling of the girders under construction loads. Girders with tubular compression flanges have greater lateral-torsional stability than traditional I-girders. This stability is particularly important for the through-girder prototype bridge which does not have any cross-frames and does not allow the deck to brace the compression flange under service conditions.

4.3 PLATE TRANSITIONS

The girders of the prototype bridges have has three segments with two shop splices. Ellis (1999) designed I-girders for a 131.24 ft (40000 mm) four-girder bridge with plate transitions at different locations along the span, as seen in Figure 4.2. Ellis (1999) found that placing the flange plate transition 26.25 ft (8000 mm) from the ends of the girders (Figure 4.2 b) results in minimum weight I-girders for the four-girder prototype bridge. The girders in the present study have the same arrangement of plate transitions with three segments, two end segments 26.25 ft (8000 mm) long and one center segment 78.74 ft (24000 mm) long.

In the present study, the thickness and width of the flanges change at the plate transitions. The web height is constant over the length of the girders. The web thickness is constant if the web is stiffened, but is allowed to vary if the web is unstiffened. When the tubular flange girders were used, the tube thickness and diameter are constant over the length of the girders.

In this study, it is assumed that the plate transition occurs a specific location along the length of the girder. In practice, the transition does not occur at one location, but, rather, the individual plates transition in width or thickness over a length of 1 to 2 ft (300 to 600 mm) (Tonias 1995). In addition, the web plate shop splice and flange plate shop splice may not occur at exactly the same location.

4.4 TRANSVERSE STIFFENERS

Transverse stiffeners may be added to bridge girders at the bearings and at interior points between the bearings (intermediate transverse stiffeners). Girders can be designed with transverse as well as longitudinal stiffeners, however, only transverse stiffeners are used in this study. The primary function of intermediate transverse stiffeners is to increase the shear capacity of the girders. Girders with fewer, more widely-spaced stiffeners may require thicker web plates to carry the required shear forces.

Modern bridge I-girders have relatively stocky webs to minimize the need for transverse stiffeners. In the past, I-girders were designed with thinner webs and more stiffeners. Increases in the number of stiffeners decreases the amount of material in the girder, but the fabrication effort increases. In this study, the prototype bridge girders are designed with and without transverse stiffeners to show the influence of stiffeners on the girder weight. Transverse stiffeners are not designed in detail, but are assumed to conform to AASHTO LRFD specifications (AASHTO LRFD 1998).

According to the AASHTO LRFD specifications (1998), the spacing between transverse stiffeners must be less than three times the web depth to contribute to the shear resistance. For the prototype bridge girders, the lengths between the cross-frames were divided into equal segments, with each segment being less than three times the web depth, to determine the stiffener spacing.

The spacing of the stiffeners in the regions near the bearings at the end of the girders was less than near the middle of the span, since the maximum shear occurs near the bearings. The AASHTO LRFD specifications (1998) require the first stiffener to be placed a maximum of 1.5 times the web depth from stiffener near the end of the girder.

4.5 FATIGUE

Fatigue of bridge girders results from the repeated stress cycles and deformation cycles caused by vehicular loads, in particular, truck loads. Locations where attachments are made to the primary plate elements (webs and flanges) of the girders are possible locations of fatigue damage. For typical bridge girders, the details where the cross-frames and the stiffeners are attached to the tension flanges are the most critical.

The cross-frame connection plates and intermediate transverse stiffeners designed in this study use either category B or category C′ fatigue details, as described in the previous chapter. A typical category C′ fatigue detail is shown in Figure 4.3. The attachment of connection plates and stiffeners can be upgraded to a category B fatigue detail by using a bolted connection, as shown in Figures 4.4 and 4.5.

By upgrading a fatigue detail to category B, the stress range for N cycles and the constant amplitude fatigue threshold ($(\Delta F)_{TH}$) at the location of the fatigue detail increases. This upgrade enables a decrease in the amount of steel required in each girder, however category B details require more fabrication effort than category C′ connection details.

4.6 PRACTICAL LIMITS ON PLATE DIMENSIONS

In the design studies, the plate dimensions were varied as minimum weight bridge girders were designed. Limits were placed on the dimensions of the prototype bridge girders. For the four-girder system with conventional I-girders, the web depth was varied from 52 in (1321 mm) to 64 in (1626 mm), in increments of 2 in (51 mm). For the four-girder system with tubular flange girders, the combined web depth plus tube diameter was varied from 52 in (1321 mm) to 64 in (1626 mm), in increments of 2 in. These girder depth ranges were chosen because they are slightly greater than the minimum depth for a constant depth girder (0.033 times the span length), suggested by the AASHTO-LRFD specifications (1998). For the prototype bridge, the minimum girder depth is slightly smaller than 52 in (1321 mm).

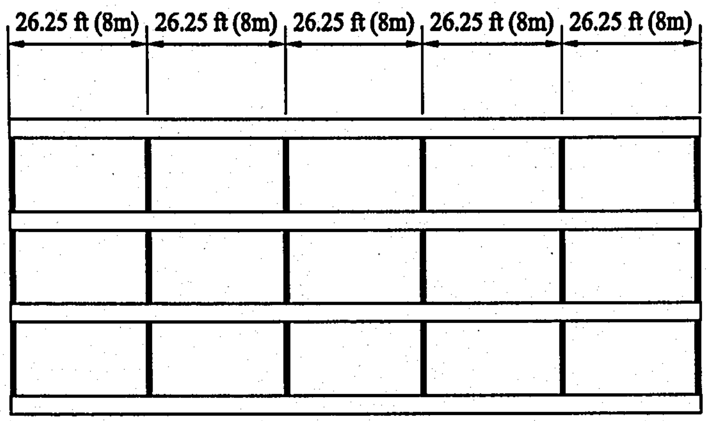
For the through-girder system, the combined web depth plus tube diameter was varied from 83 in (2108 mm) to 95 in (2413 mm) in increments of 4 in (102 mm). This depth allows a 12 in (305 mm) gap between the deck and the bottom flange, a 13 in (330 mm) thick deck, a 32 in (813 mm) tall parapet, a 6 in (152 mm) gap between the top of the parapet and the bottom of the tube, and a minimum tube diameter of 20 in (508 mm).

For all of the girders, the web thickness varied between 0.438 in (11.1 mm) and 1 in (25.4 mm) in increments of 0.063 in (1.6 mm). The flange thickness varied between 0.75 in (19.1 mm) and 3.0 in (76 mm), with increments of 0.125 in (3.2 mm) between 0.75 in (19.1 mm) and 1.5 in (38 mm), and with increments of 0.5 in (12.7 mm) between 1.5 in (38 mm) and 3.0 in (76 mm). The flange widths were varied from 12 in (305 mm) to 36 in (914 mm), in increments of 2 in (51 mm). The flange width and thickness were both permitted to vary at the plate transitions. The flange widths on either side of the plate transition were designed to be as close to each other as possible.

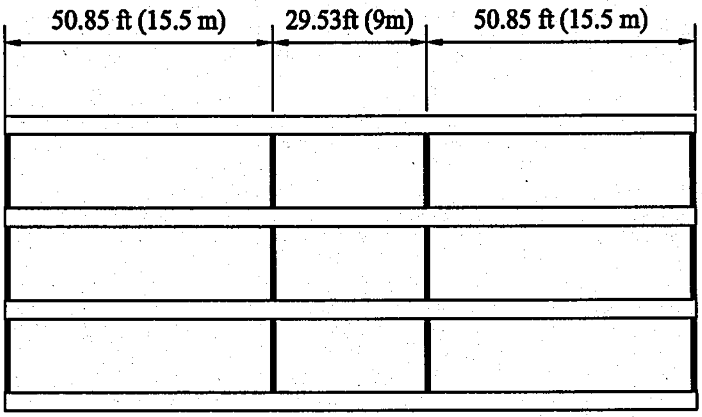
For the tubular flange girders, the tube diameter was varied from a minimum of 12 in (305 mm) to one-half of the combined web depth plus tube diameter height. The tube diameter increased in increments of 2 in (51 mm). The thickness of the tube varied from a minimum of 0.25 in (6.4 mm) in increments of 0.063 in (1.6 mm).

Table 4.1 Desciption of cross-frame arrangements

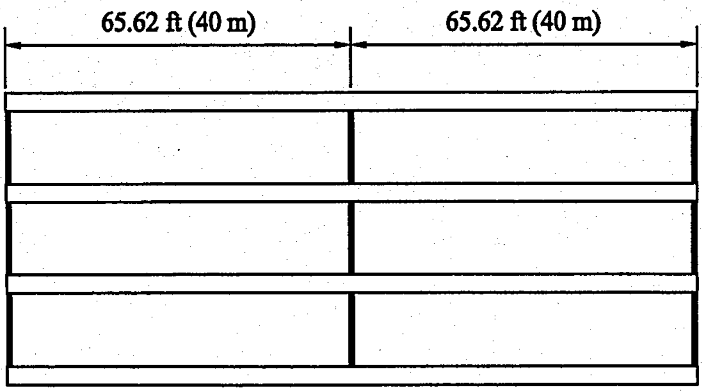
Description	Number of cross-frames	Cross-frame location		Plate transition	
		ft	(m)	ft	(m)
Scheme 1	6	0, 26.25, 52.49, 72.75, 105.00, 131.24	(0), (8), (16), (24), (32), (40)	26.25, 105.00	(8), (32)
Scheme 2	5	0, 32.81, 65.62, 98.43, 131.24	(0), (10), (20), (30), (40)	32.80, 98.40	(10), (30)
Scheme 3	5	0, 32.81, 65.62, 98.43, 131.24	(0), (10), (20), (30), (40)	26.25, 105.00	(8), (32)
Scheme 4	5	0, 39.37, 65.62, 91.86, 131.24	(0), (12), (20), (28), (40)	26.25, 105.00	(8), (32)
Scheme 5	5	0, 36.09, 65.62, 95.14, 131.24	(0), (11), (20), (29), (40)	26.25, 105.00	(8), (32)
Scheme 6	4	0, 45.93, 85.30, 131.24	(0), (14), (26), (40)	26.25, 105.00	(8), (32)
Scheme 7	4	0, 45.21, 82.02, 131.24	(0), (15), (25), (40)	26.25, 105.00	(8), (32)
Scheme 8	4	0, 50.85, 80.34, 131.24	(0), (15.5), (24.5), (40)	26.25, 105.00	(8), (32)
Scheme 9	3	0, 65.62, 131.24	(0), (20), (40)	26.25, 105.00	(8), (32)
Scheme 10	2	0, 131.24	(0), (40)	26.25, 105.00	(8), (32)



(a) Scheme 1



(b) Scheme 8



(c) Scheme 9

Figure 4.1 Cross-Frame Arrangements

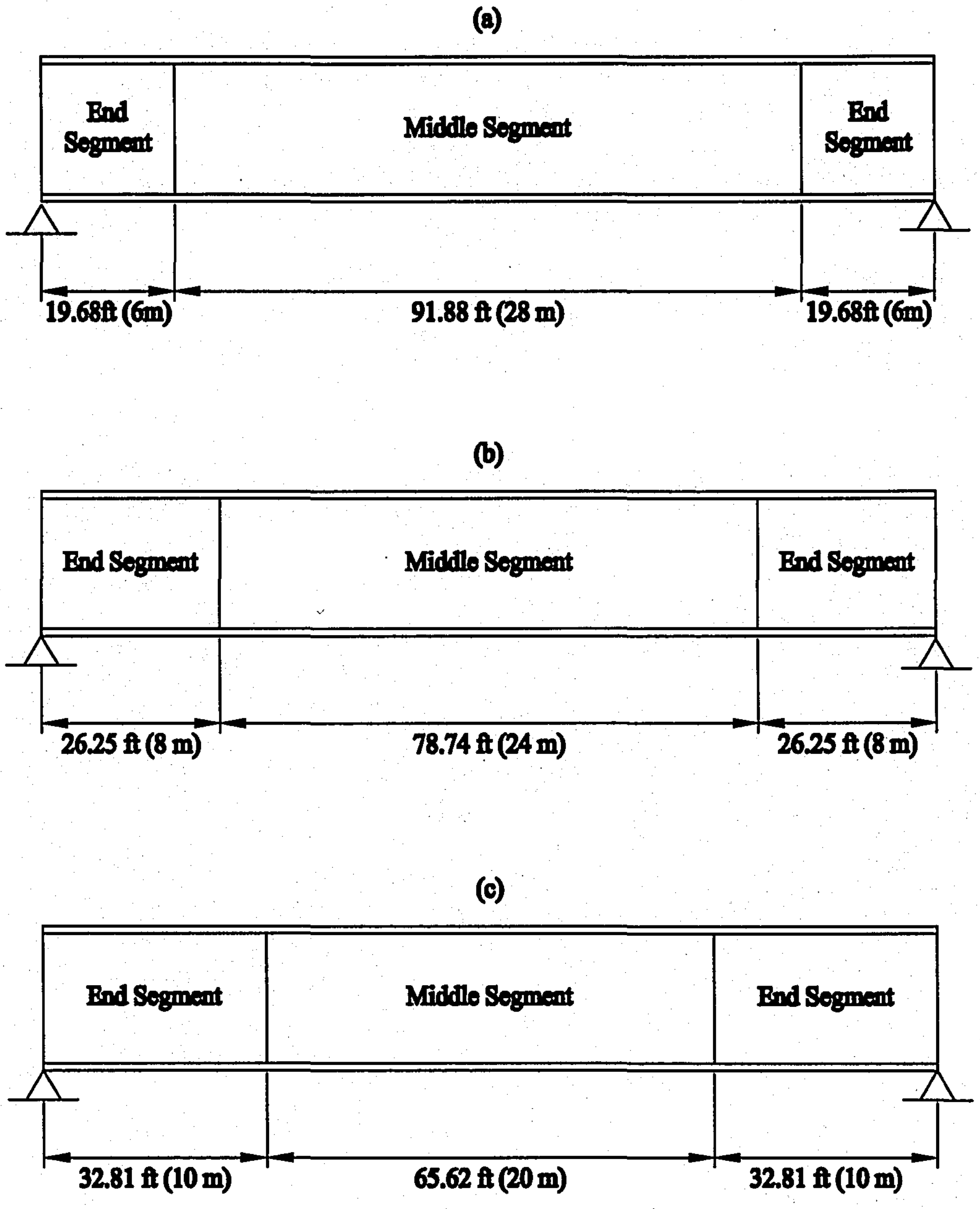


Fig. 4.2 Plate Transition Locations

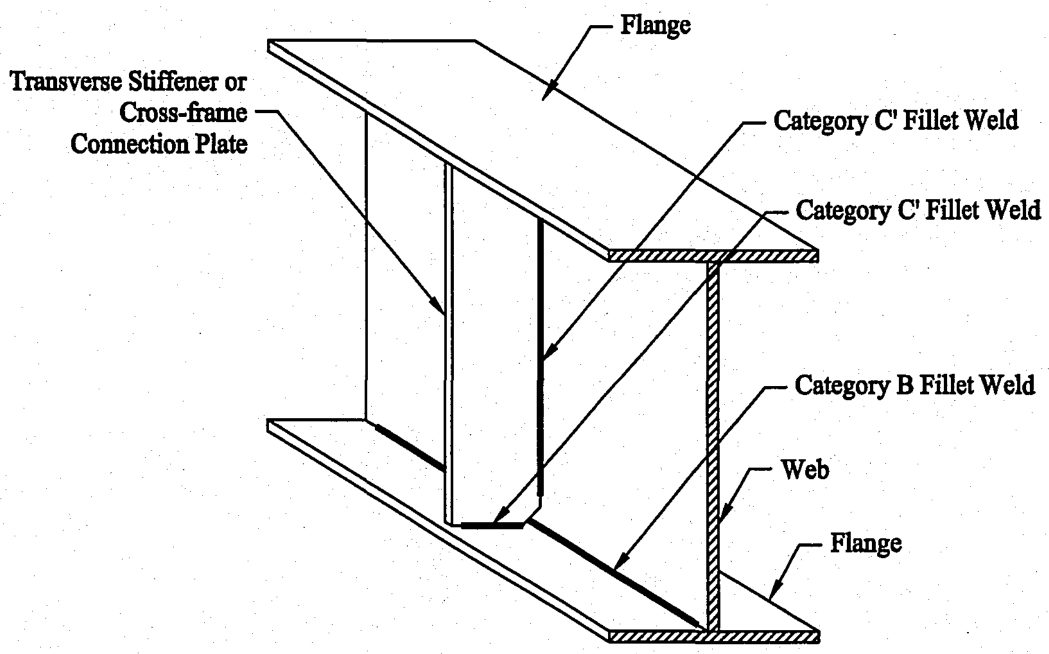


Figure 4.3 Typical Category C' Fatigue Detail

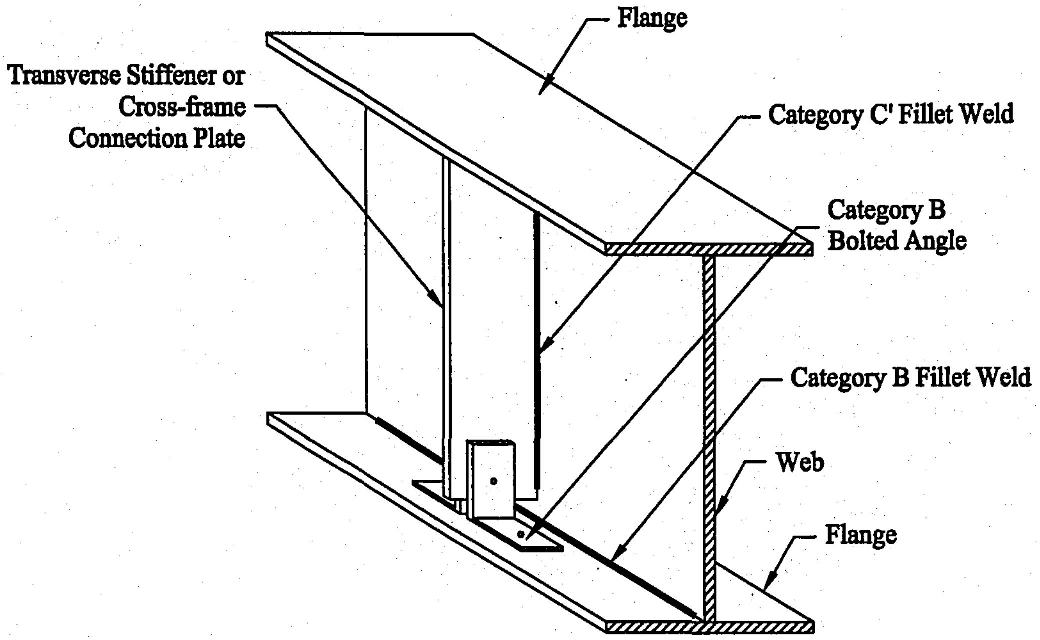


Figure 4.4 Typical Category B Fatigue Detail With
Bolted Connection

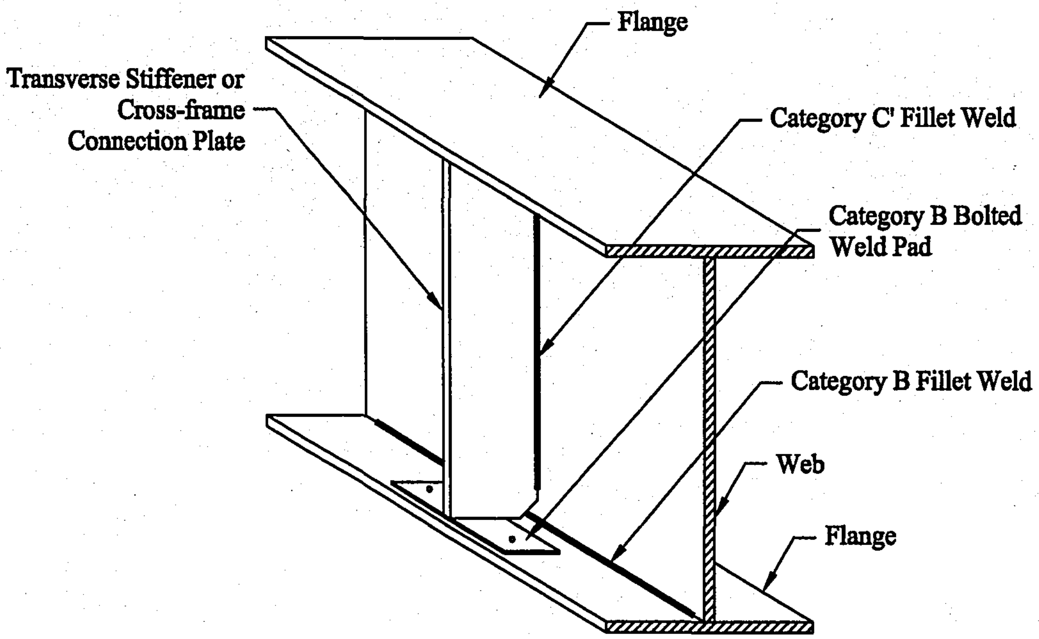


Figure 4.5 Typical Category B Fatigue Detail
With Welded And Bolted Connection

CHAPTER 5

DESIGN METHODS AND RESULTS

5.1 INTRODUCTION

This chapter outlines how girders were designed for the prototype bridges. As noted previously, I-girders, composite tubular flange girders, and non-composite tubular flange girders were designed for the four-girder prototype bridge. Non-composite tubular flange girders were designed for the through-girder prototype bridge. The design parameters that were applied in each girder design are described, and the results of the design studies are given. Some construction and fabrication concerns will also be discussed. Finally, the results of the design studies for the different girders will be compared to each other.

5.2 I-GIRDERS

I-girders were designed for the four-girder prototype bridge considering two cross-frame arrangements: scheme 1 (4 interior cross-frames) and scheme 8 (2 interior cross-frames). For scheme 1, three arrangements of stiffeners were considered. The stiffeners were arranged either every 8.75 ft (2667 mm) or every 13.13 ft (4000 mm), or the webs were unstiffened. For scheme 8, three different arrangements of stiffeners were considered: (1) stiffeners were spaced at 8.48 ft (2583 mm) between the end cross-frames and the interior cross-frames and 9.85 ft (3000 mm) between the interior cross-frames, (2) stiffeners were spaced at 12.71 ft (3875 mm) between the end cross-frames and the interior cross-frames and 14.77 ft (4500 mm) between the interior cross-frames, or (3) the webs were unstiffened. For all these cases, either category B or category C′ fatigue details were used.

The I-girder design method used in this study was based on a method devised by Ellis (1999). Ellis (1999) encoded relevant AASHTO LRFD specifications (AASHTO LRFD 1998) into a Mathcad, version 8 (1998) program. The prototype bridge design parameters (e.g., unfactored load effects), I-girder cross-section properties, and stiffener and cross-frame spacing are entered into the program, and the program, with assistance from the user, checks required design criteria and determines if the design is acceptable. A few changes were made to the Mathcad program created by Ellis (1999) for the current study. These changes include altering the live load distribution factors and altering the values of the applied dead load to the values given in Chapter 3.

The first step in generating a design is to choose values for overall parameters, such as cross-frame arrangement, fatigue details, and arrangement of stiffeners. Considering the parameters discussed above, twelve different types of I-girders were designed for the four-girder prototype bridge. The minimum weight designs of each I-girder type were determined by varying the size of the girder plates to minimize the weight while satisfying the design criteria. The web depth was chosen to be a value between 52 in (1321 mm) and 64 in (1626 mm), as described in the previous chapter. Then, the web thickness was chosen to be just large enough to carry the shear. The bottom flange was then given an approximate size so that the top flange could be designed. The top flange width and thickness was chosen to satisfy non-compact section requirements, and to satisfy strength design criteria for the construction loading

($1.25\ D_c$). The bottom flange design was controlled as follows: if the girder was being designed with category C′ fatigue details, the fatigue limit state governed the size of the bottom flange, while if the girder was designed with category B fatigue details, the service II limit state governed the bottom flange. The final girder design was also checked to make sure it was acceptable for the strength I limit state, as well as the service II and fatigue limit states.

The arrangement of cross-frames and stiffeners, and the fatigue detail category (B or C′) affect the size of the plates that make up the girder. Tables 5.1-5.8 list the minimum weight designs for the I-girders as these parameters are changed. Also listed in the table are the performance ratios for several of the design limit states. The performance ratio is the required strength divided by the available resistance. A performance ratio less than one means that the design is acceptable for that design limit state.

Table 5.1 lists the I-girder design and the performance ratios for the I-girders designed with the scheme 1 cross-frame arrangement, category C′ fatigue details, and a web depth of 52 in (1321 mm). 52 in (1321 mm) is the smallest web depth used for the I-girders. Table 5.2 lists the results for the same design parameters, but with the web depth equal to 64 in (1626 mm), which is the largest web depth used for the I-girders. Both Tables 5.1 and 5.2 show the results for the three stiffener arrangements that were used. As the stiffener spacing increases, the top and bottom flanges stay fairly constant, but the web thickness increases, causing the total girder weight to increase. Figure 5.1 plots the total girder weight (the steel weight of four-girders) of all the minimum weight I-girders designed with the scheme 1 cross-frame arrangement and category C′ fatigue details versus web depth.

Tables 5.3 and 5.4 show the I-girder designs and performance ratios for I-girders designed with scheme 1 cross-frames and category B fatigue details. Table 5.3 is for a web depth of 52 in. (1321 mm) and Table 5.4 is for a web depth of 64 in. (1626 mm). Tables 5.3 and 5.4 show that the service II limit state performance ratios are equal to, or close to, 1.0, indicating that the size of the bottom flange is governed by the service II limit state. For the I-girders designed with category C′ fatigue details (Tables 5.1 and 5.2), the fatigue limit state performance ratios are equal to or close to 1.0, indicating that the fatigue limit state governs the size of the bottom flange. Thus, the choice of fatigue details influences the I-girder weights.

Figure 5.2 is a plot of the total girder weight versus the web depth. Also plotted for comparison are the results for girders designed with category C′ fatigue details and stiffeners every 8.75 ft (2667 mm).

Tables 5.5, 5.6, 5.7, and 5.8 show the I-girder designs and the performance ratios for I-girders designed with the scheme 8 cross-frame arrangement. Tables 5.5 and 5.6 are for girders with category C′ fatigue details, and Tables 5.7 and 5.8 are for girders with category B fatigue details. When the spacings of cross-frames is increased by switching from the scheme 1 arrangement to the scheme 8 arrangement, the size of the top flange increases, because of the required increase in lateral-torsion buckling resistance. The increased size of the top flange causes the total girder weight to increase about 8.5%. Again, it can be seen that when the stiffener spacing is increased, the web thickness increases, and when the fatigue details are upgraded from category

C′ to category B, the size of the bottom flange decreases. Figure 5.3 and 5.4 show plots of the total girder weight versus web depth for girders designed with the scheme 8 cross-frame arrangement. Also plotted for comparison are the results for girders designed with the scheme 1 cross-frame arrangement, category C′ fatigue details, and stiffeners at 8.75 ft (2667 mm).

5.3 COMPOSITE TUBULAR FLANGE GIRDERS

The composite tubular flange girders were designed for the four-girder prototype bridge considering three cross-frame arrangements: scheme 1 (4 interior cross-frames), scheme 9 (1 interior cross-frame), and scheme 10 (no interior cross-frames). For scheme 1 and scheme 10, only an unstiffened web was considered. For scheme 9, two arrangements of stiffeners were considered. The stiffeners were arranged either every 9.37 ft (2857 mm) or the webs were unstiffened. Both category B and category C′ fatigue details were used for the cases with the scheme 9 cross-frame arrangement. Only category B fatigue details were used with the scheme 1 and scheme 10 cross-frame arrangements.

Considering these parameters, six different types of composite tubular flange girders were designed for the four-girder prototype bridge. The size of the girder plates and the tubular flange was varied to minimize the weight while satisfying the design criteria. The combined depth (the web depth plus tube diameter) was chosen to be a value between 52 in (1321 mm) and 64 in (1626 mm). Then, the web thickness was chosen to be just large enough to carry the shear. The bottom flange was then given an approximate size so that the tubular flange could be designed. The tube diameter and thickness were chosen to satisfy tube local buckling requirements, and to satisfy strength design criteria for construction loading. The tube local buckling requirement is discussed in more detail in Section 5.4. Then, the bottom flange was designed, as described for the I-girder prototype bridge. The final girder design was also checked to make sure it was acceptable for the strength I limit state, as well as the service II and fatigue limit states.. Because of the long distance between cross-frames (131.24 ft (40000 mm)), the tubular flange girders with the scheme 10 arrangement of cross-frames were also checked for the strength III and strength V limit states, which include wind loading, as described in chapter 3.

The minimum weight designs and the performance ratios for the composite tubular flange girders can be seen in Tables 5.9, 5.10, 5.11, and 5.12. Tables 5.9 and 5.11 show the values for the shallowest girders that were designed (52 in (1321 mm)), and Tables 5.10 and 5.12 show the values for the deepest girders that were designed (64 in (1626 mm)).

Table 5.9 shows the plate and tube sizes and performance ratios for the composite tubular flange girders designed with the scheme 9 cross-frame arrangement and a combined web depth plus tube diameter of 52 in (1321 mm). Table 5.9 shows that the size of the tubular flange remains constant as the stiffener arrangement and the fatigue category vary. Table 5.10 shows the plate and tube sizes for girders with a combined web depth plus tube diameter of 64 in (1626 mm). Again, the size of the tubular flange remains constant with a diameter of 14 in (356 mm) and a thickness of 0.25 in (6.4 mm). The web thickness increases as the number of stiffeners decreases, and the bottom flange size increases as the fatigue detail category changes from B to

C′. The bottom flange size does not increase as dramatically as for the I-girders when the fatigue detail category changes, because when category C′ fatigue details are used, either the fatigue limit state or the service II limit state govern the design of the bottom flange.

Tables 5.11 and 5.12 show the minimum weight designs and performance ratios for the composite tubular flange girders designed with scheme 1 and scheme 10 cross-frame arrangements. The girders designed with the scheme 1 cross-frame arrangement are exactly the same as the girders designed with the scheme 9 cross-frame arrangement. When the interior cross-frames are removed for scheme 10, and the unbraced length becomes the entire span of the bridge, the tubular flange size increases, since strength (considering lateral torsional buckling) under the construction loading governs the size of the tube.

Tables 5.11 and 5.12 also list the performance ratios for composite tubular flange girders designed with the scheme 10 cross-frame arrangement when they were checked for the strength III and strength V limit states. The performance ratios are acceptable (< 1), so wind loading does not affect the design of these girders. Since the scheme 10 cross-frame arrangement has the largest unbraced girder length, which results in the largest wind force effects on the girders for all cases considered, the strength III and strength V limit states should not govern the design of any of the composite tubular flange girders.

Figures 5.5 and 5.6 are plots of the total weight of the girders (including only the steel weight of the four girders) versus the combined web depth plus tube diameter. Figure 5.5 shows the results for the composite tubular flange girders designed with the scheme 9 cross-frame arrangement. The increase in the girder weight as stiffeners are removed is smaller for the composite tubular flange girders than for the I-girders. The weight increases only 2% as the tubular flange girder design changes from stiffeners every 9.37 ft (2857 mm) to unstiffened, while the weight increases about 4% as the I-girder design changes from stiffeners every 8.75 ft (2667 mm) to unstiffened.

Figure 5.6 shows results for the scheme 10, scheme 9, and scheme 1 arrangements of cross-frames. The total steel weight of the four girders for the four-girder prototype bridge with the scheme 10 cross-frame arrangement is about 5% greater than the total girder weight for the scheme 1 cross-frame arrangement. The total girder weight for the composite tubular flange girders with the scheme 1 cross-frame arrangement is essentially the same as the total girder weight with the scheme 9 cross-frame arrangement. An unusual result occurs when the web depth plus tube diameter equals 58 in (1473 mm). The total girder weight for the scheme 1 arrangement of cross-frames is greater than the scheme 9 arrangement. This is due to the C_b factor, which equals 1.3 for the scheme 9 arrangement and 1.0 for the scheme 1 arrangement. The C_b factor accounts for the moment gradient, and increases the lateral-torsional buckling strength, as seen in Eq. 3.11 and 3.12.

The deflection of the composite tubular flange girder bridge under dead and live load was also calculated. These values can bee seen in Tables 5.9, 5.10, 5.11, and 5.12. The AASHTO LRFD specifications (1998) and the PennDOT bridge design specifications (PennDOT Bridge Design Manual 4 1993) limit the live load deflection to 1.57 in (40 mm) (1/1000 of the span length) if there will be pedestrian traffic on the bridge, or 1.97 in (50 mm) (1/800 of the span length) if there will be no pedestrian traffic. The maximum calculated live load deflection is 0.61 in (15.5 mm), which falls well below both limits.

The use of the composite tubular flange girders creates some construction issues that must be resolved. One of these issues is the connection of the tubular flange girders to the deck. One solution is shown in Figure 5.7. Light-weight angles could be welded to the sides of the tube and the stay-in-place forms can be placed on the angles. Shear studs can be welded onto the top of the tube to create composite action between the girder and the deck. The deck can then be placed onto the forms.

5.4 NON-COMPOSITE TUBULAR FLANGE GIRDERS

Tubular flange girders were designed for the four-girder prototype bridge assuming that the girders would not be composite with the bridge deck under service conditions. This design approach would eliminate the need for using shear studs and simplify the attachment of a precast concrete deck to the girders. To be economical, the tube and the concrete within the tube must provide most of the compression capacity normally provided by the deck in a composite girder bridge.

The non-composite tubular flange girders were designed considering two cross-frame arrangements: scheme 1 (4 interior cross-frames) and scheme 9 (1 interior cross-frame). Only an unstiffened web and category B fatigue details were considered. The girders designed with the scheme 9 cross-frame arrangement were designed with three different combinations of steel strength and concrete strength, as follows.

The non-composite tubular flange girders were first designed with materials similar to those used in the composite case, with ASTM A709 HPS 70W steel used for the girder and normal strength concrete with a compressive strength of 4 ksi (30 MPa) in the tube. This choice of materials led to a total girder weight (steel weight of the four girders) that was about twice as large as for the composite case. To reduce the total girder weight, the girder was designed with high performance steel with a yield stress of 100 ksi (690 MPa), along with high strength concrete with a compressive strength of 8 ksi (55 MPa) in the tube.

For the strength limit I limit state, the strain limit for the concrete is taken as 0.003. For 100 ksi steel, the yield strain is 0.003448 (ε_y=Fy/E), and thus, under a positive moment, the top of the tube, which is relatively thin, will not reach the yield strain, because of the strain limit for the concrete in the tube is 0.003. As a result, the increased strength of the 100 ksi (690 MPa) steel is not fully utilized in the steel tube. The 70 ksi (ASTM A709 HPS 70W) steel (with a yield strain of 0.00241) is more effectively used in the tubular flange.

Therefore, hybrid girders with a combination of steel were used in the design of non-composite tubular flange girders. The hybrid girders have a web and bottom flange made of 100 ksi (690 MPa) steel and a tubular flange made of 70 ksi (485 MPa) steel. The concrete in the tube is high strength concrete with a compressive strength of 8 ksi (55 MPa).

The design of the non-composite tubular flange girders was similar to that of the composite tubular flange girders. Based on the different materials that were investigated, four cases were considered: (1) 70 ksi (485 MPa) steel for the tube, web, and flange with 4 ksi (30 MPa) concrete, scheme 9 cross-frame arrangement, and unstiffened; (2) 100 ksi (690 MPa) steel for the tube, web, and flange with 8 ksi (55

MPa) concrete, scheme 9 cross-frame arrangement, and unstiffened; (3) 70 ksi (485 MPa) for the tube, 100 ksi (690 MPa) for the web and flange (i.e., hybrid), with 8 ksi (55MPa) concrete, scheme 9 cross-frame arrangement, and unstiffened; (4) 70 ksi (485 MPa) for the tube, 100 ksi (690 MPa) for the web and flange (i.e. hybrid), with 8 ksi (55MPa) concrete, scheme 1 cross-frame arrangement, and unstiffened. The material properties and the cross-frame arrangement were decided first. Then the plate dimensions and the tubular flange dimensions were chosen to minimize the weight and satisfy the design criteria, as described for the composite tubular flange girder.

Tables 5.13 and 5.14 summarize the minimum weight designs and the performance ratios for the non-composite tubular flange girder. Table 5.13 describes the results for girders with a combined web depth plus tube diameter of 52 in (1321 mm), and Table 5.14 describes the results for girders with a combined web depth plus tube diameter of 64 in (1626 mm). Only girders designed with the scheme 9 cross-frame arrangement are shown in these tables. The girders designed with a scheme 1 cross-frame arrangement had the same dimensions as the prototype bridge with a scheme 9 cross-frame arrangement (using the same steel and concrete materials).

Tables 5.13 and 5.14 show that the sizes of the flange, web, and tubular flange get smaller as the steel strength increases from a yield stress of 70 ksi (485 MPa) to 100 ksi (690 MPa). When the combined web depth plus tube diameter is 52 in (1321 mm), the total girder weight (steel weight of four girders) does not change as the steel yield stress changes from 100 ksi (690 MPa) to the combination of 70 ksi (485 MPa) and 100 ksi (690 ksi). But when the combined web depth plus tube diameter is 64 in (1626 mm), the girder weight actually *decreases* when the yield stress of the steel changes from 100 ksi (690 MPa) to the combination of 70 ksi (485 MPa) and 100 ksi (690 MPa). The reason for the decrease in girder weight is the tube local buckling requirement. The AASHTO LRFD specifications (1998) require the ratio of the tube diameter to the tube thickness be less than a constant that is inversely proportional to the yield stress of the tube, as follows:

$$\frac{D_t}{T_t} \leq 2.8\sqrt{\frac{E}{Fy}} \qquad \text{(Eq. 5.1)}$$

where, D_t is the tube diameter, T_t is the tube thickness, E is Young's modulus for steel, and Fy is the yield stress for steel. For a constant D_t, T_t can be smaller for a 70 ksi (485 MPa) steel tube than a 100 ksi (690 MPa) steel tube.

Tables 5.13 and 5.14 show the calculated deflections of the non-composite tubular flange girders under dead and live load. The non-composite tubular flange girders have the largest deflection of all the girders designed in this study. The calculated live load deflections vary from 1.45 in (37 mm) to 1.90 in (48 mm). Almost all the live load deflections are greater than the deflection limit of the AASHTO LRFD specifications (AASHTO LRFD 1998) and the PennDOT bridge design specifications (PennDOT Design Manual 4 1993) for a bridge with pedestrian traffic, which is 1.57 in (40 mm). All of the live load deflections are, however, under the limit for a bridge with no pedestrian traffic, which is 1.97 in (50 mm).

Figure 5.8 plots the total weight of the non-composite tubular flange girders versus the combined web depth plus tube diameter. The figure shows that the girders made of 70 ksi (485 MPa) steel have a total girder weight 30% greater than the total girder weight of girders made with 100 ksi (690 MPa) steel. The hybrid girder with a 70 ksi (485 MPa) steel tube and a 100 ksi (690 MPa) steel web and flange is the lightest.

5.5 THROUGH-GIRDERS

In areas where under-clearance is a concern, a through-girder bridge can be considered. The through-girder bridge is designed with an unbraced length equal to the length of the bridge.

The through-girder prototype bridge is designed with girders made of three different combinations of steel strength and concrete strength, similar to the girders used in the four-girder prototype bridge with non-composite tubular flange girders. Three cases were considered: (1) 70 ksi (485 MPa) steel for the tube, web, and flange with 4 ksi (30 MPa) concrete, (2) 100 ksi (690 MPa) steel for the tube, web, and flange with 8 ksi (55 MPa) concrete, and (3) 70 ksi (485 MPa) for the tube, 100 ksi (690 MPa) for the web and flange (i.e., hybrid), with 8 ksi (55MPa) concrete. The through-girders are designed with an unstiffened web and category B fatigue details, and is designed similarly to the previously designed four-girder prototype bridges. The material strengths were first chosen, then web thickness was chosen to support the required shear force. The tubular flange was then chosen to support the strength I load case, including the live (LL) and dead (D_w) loads as well as the construction (D_c) loads. The tubular flange also had to satisfy the tube local buckling requirement. The bottom flange was then designed to satisfy the fatigue and service II limit states, and strength I limit state. The strength I limit case and the service II limit state governed the size of the bottom flange.

The sizes of the plates and the size of the tubular flange of the through-girders can be seen in Tables 5.15 and 5.16. Table 5.15 shows results for the minimum web depth plus tube diameter that was designed (83 in (2108 mm)), while Table 5.16 shows results for the largest web plus tube diameter (95 in (2413 mm)). Table 5.16 shows that the tube local buckling requirement causes the thickness of the tube composed of 100 ksi (690 MPa) steel to be larger than the thickness of the tube composed of 70 ksi (485 MPa) steel. By using the hybrid girder, the thickness of the 70 ksi (485 MPa) tubular flange can be reduced, while the bottom flange and web utilize the strength of the 100 ksi (690 MPa) steel, as shown in Table 5.16. Table 5.15, for the girders where the web depth plus the tube diameter equals 83 in (2108 mm), shows that the total weight of the girders does not increase as the girder material changes from entirely 100 ksi (690 MPa) steel to the combination of 100 ksi (690 MPa) for the web and bottom flange and 70 ksi (485 MPa) steel for the tube when the tube local buckling requirement does not govern the thickness of the tubular flange.

The total girder weight plotted against the combined web depth plus tube diameter for the through-girders can be seen in Figure 5.9. The through-girders have the least total girder weight of all the girders designed for the prototype bridges. However, the total girder weights cannot be directly compared between prototype bridges, because the through-girder bridge is designed with only two traffic lanes, as opposed to four lanes for the four-girder prototype bridge, and the through-girder bridge has a narrower deck.

The calculated live load deflections of the two-girders are less than the limits given by AASHTO LRFD specifications (1998) and PennDOT bridge design specifications (1993) for a bridge with pedestrian traffic (1.57 in (40 mm)). The live load deflection of the two-girder prototype bridge varies between 1.07 in (27 mm) and 1.34 in (34 mm) as seen in Tables 5.15 and 5.16.

Corrosion of the girders must be considered when designing the through-girder prototype bridge. Weathering steels, like the steels considered in this study, are susceptible to corrosion damage if moisture, wet leaves, and other debris are allowed to build up against the girders. Two possible designs for the through-girder prototype bridge are outlined. The first design, shown in Figure 5.10, limits the possibility of moisture being trapped against the girder web for an extended period of time by placing the parapets away from the girder webs, allowing enough space for the water to evaporate or to run-off the bridge. Care must be taken in this design to make sure that debris does not build up between the parapets or the deck and the girders.

The second design places the parapets up against the girder webs, as shown in Figure 5.11. The joint between the parapet and the girder can be sealed to prevent water seeping against the web. The parapet shown in the figure is designed so water runs away from the girder, and onto the deck.

Three methods of connecting the deck to the girders have been considered. The methods include, installing a bracket to hold the deck (as shown in Figure 5.10), bolting the deck directly to the web (as shown in Figure 5.11), or placing the deck directly on the bottom flange. Each of these methods introduces interactions between the girders and the deck that should be considered in design.

5.6 SUMMARY OF FINDINGS

The lightest I-girders designed for the four-girder prototype bridge were designed with the scheme 1 cross-bracing arrangement, stiffeners every 8.75 ft, and category B fatigue details, as seen in Figure 5.2. Although these designs use the least amount of steel, fabricating the large numbers cross-frames and stiffeners, and using category B fatigue details, instead of the category C′ details, increases the fabrication effort. These I-girders will be the most labor intensive.

Similar results are observed for the tubular flange girder designs. As the number of cross-frames and transverse stiffeners increases, and as the fatigue details are upgraded (from category C′ to category B), the total girder weight decreases. However, these parameters increase the fabrication effort.

The design of the composite tubular flange girders have a similar total girder weight whether the scheme 1 cross-frame arrangement or the scheme 9 cross-frame arrangement is used, as seen in Figure 5.6. The total girder weight of the composite tubular flange girder designs is less than the total girder weight of the I-girder designs. A comparison of the fabrication effort of the tubular flange girders and I-girders suggests that the tubular flange girders require less fabrication effort because of the decreased number of cross-frames, however, the I-girders themselves requires less fabrication effort, because they are easier to fabricate than the tubular flange girders..

Figure 5.12 is a plot that summarizes the total weight of all the girders that were designed. The figure shows the total girder weight divided by the deck area plotted versus either the web depth (for I-girders) or the web depth plus tube diameter (for tubular flange girders). By normalizing the girder weight this way, the designs for the four-girder prototype bridge can be compared to the designs for the through-girder prototype bridge. In the figure, the tubular flange girder designs use the scheme 9 cross-frame arrangement, while the I-girder designs use the scheme 1 cross-frame arrangement. The figure shows that the composite tubular flange girders have the lightest total girder weight. However, the construction effort required to make the tubular flange girders composite with a concrete deck may not make these designs the most economical.

The non-composite tubular flange girders have the largest total girder weight, but less effort is required to construct the bridge. For example, a precast deck can be more easily installed if the girders are designed to be non-composite.

The through-girders have a total girder weight (normalized by deck area) that approximately equals that of the girders designed for the four-girder prototype bridge. The through-girder prototype bridge was designed to be constructed in areas where the under-clearance would be a concern. The bottom flanges of the girders are at a level approximately equal to the level of the deck, in contrast to the four-girder prototype bridges, where the girders are placed completely under the deck. The through-girder bridge should have a construction effort less than or equal to the four-girder bridge. The through-girder bridge was designed for a precast deck, which should reduce the construction effort.

Table 5.1 Summary of I-Girder Designs: Scheme 1, Category C' Fatigue Details
Web Depth = 52 in.

	Scheme 1		Scheme 1		Scheme 1	
	Category C'		Category C'		Category C'	
	Stiffened at 8.75 ft		Stiffened at 13.13 ft		Unstiffened	
	Middle Section	End Section	Middle Section	End Section	Middle Section	End Section
Web Depth (in.)	52	52	52	52	52	52
Web Thickness (in.)	8/16	8/16	9/16	9/16	9/16	10/16
Top Flange Thickness (in.)	1.125	0.75	1	0.75	1	0.75
Top Flange Width (in.)	19	14	19	14	19	14
Bottom Flange Thickness (in.)	2.25	1.375	2.5	1.5	2.5	1.75
Bottom Flange Width (in.)	17	17	15	15	15	12
Cross Section Area (in^2)	85.62	59.88	85.75	62.25	85.75	64.00
Total Weight (kips) - 4 Girders	**134.55**		**136.38**		**137.63**	
Performance Ratios						
Strength Limit State						
Flexural Resistance	0.86	0.75	0.85	0.74	0.85	0.73
Ductility Requirement	0.94	0.83	0.93	0.82	0.93	0.82
Shear Resistance	0.55	0.90	0.79	0.81	0.79	0.84
Construction						
Web Slenderness	0.76	0.80	0.71	0.68	0.71	0.59
Compression Flange Slenderness	0.84	0.96	0.95	0.91	0.95	0.87
Flexural Resistance	0.91	0.97	0.85	0.74	0.97	0.94
Service II Limit State						
Perm. Deflection (tension flange)	0.95	0.93	0.97	0.93	0.97	0.96
Fatigue Limit State						
Bottom Flange Conn. Plate Weld	1.00	1.00	0.99	0.99	0.99	1.00

1 in = 25.4 mm

1 kip = 4.45 kN

Table 5.2 Summary of I-Girder Designs: Scheme 1, Category C' Fatigue Details
Web Depth = 64 in.

	Scheme 1		Scheme 1		Scheme 1	
	Category C'		Category C'		Category C'	
	Stiffened at 8.75 ft		Stiffened at 13.13 ft		Unstiffened	
	Middle Section	End Section	Middle Section	End Section	Middle Section	End Section
Web Depth (in.)	64	64	64	64	64	64
Web Thickness (in.)	8/16	8/16	9/16	9/16	9/16	11/16
Top Flange Thickness (in.)	1	0.75	1	0.75	1	0.75
Top Flange Width (in.)	18	14	17	13	17	13
Bottom Flange Thickness (in.)	2.25	1.375	1.75	1	1.75	0.75
Bottom Flange Width (in.)	13	12	16	15	16	17
Cross Section Area (in^2)	79.25	59.00	81.00	60.75	81.00	66.50
Total Weight (kips) - 4 Girders	**127.10**		**130.22**		**134.33**	
Performance Ratios **Strength Limit State**						
Flexural Resistance	0.81	0.70	0.80	0.69	0.80	0.66
Ductility Requirement	0.87	0.76	0.86	0.75	0.86	0.72
Shear Resistance	0.26	0.84	0.47	0.88	0.97	0.78
Construction						
Web Slenderness	0.84	0.79	0.75	0.69	0.75	0.53
Compression Flange Slenderness	0.88	0.88	0.81	0.79	0.81	0.73
Flexural Resistance	0.90	0.80	0.98	0.98	0.98	0.98
Service II Limit State						
Perm. Deflection (tension flange)	0.95	0.92	0.95	0.94	0.95	0.94
Fatigue Limit State						
Bottom Flange Conn. Plate Weld	0.99	1.00	1.00	1.00	1.00	0.99

1 in = 25.4 mm

1 kip = 4.45 kN

Table 5.3 Summary of I-Girder Designs: Scheme 1, Category B Fatigue Details
Web Depth = 52 in.

	Scheme 1		Scheme 1		Scheme 1	
	Category B		Category B		Category B	
	Stiffened at 8.75 ft		Stiffened at 13.13 ft		Unstiffened	
	Middle Section	End Section	Middle Section	End Section	Middle Section	End Section
Web Depth (in.)	52	52	52	52	52	52
Web Thickness (in.)	8/16	8/16	9/16	9/16	9/16	10/16
Top Flange Thickness (in.)	1.125	0.75	1	0.75	1	0.75
Top Flange Width (in.)	19	14	19	14	19	14
Bottom Flange Thickness (in.)	1.375	0.875	1.125	1	1.125	0.875
Bottom Flange Width (in.)	26	24	31	20	31	22
Cross Section Area (in^2)	83.13	57.50	83.13	59.75	83.13	62.25
Total Weight (kips) - 4 Girders	**130.18**		**131.78**		**133.57**	
Performance Ratios						
Strength Limit State						
Flexural Resistance	0.90	0.80	0.89	0.78	0.89	0.77
Ductility Requirement	0.98	0.88	0.98	0.87	0.98	0.85
Shear Resistance	0.55	0.98	0.99	0.96	0.79	0.84
Construction						
Web Slenderness	0.75	0.78	0.69	0.67	0.69	0.58
Compression Flange Slenderness	0.84	0.96	0.95	0.92	0.95	0.87
Flexural Resistance	0.93	0.98	0.99	0.96	0.99	0.96
Service II Limit State						
Perm. Deflection (tension flange)	1.00	1.00	1.00	1.00	1.00	1.00
Fatigue Limit State						
Bottom Flange Conn. Plate Weld	0.80	0.81	0.80	0.81	0.80	0.80

1 in = 25.4 mm

1 kip = 4.45 kN

Table 5.4 Summary of I-Girder Designs: Scheme 1, Category B Fatigue Details
Web Depth = 64 in.

	Scheme 1		Scheme 1		Scheme 1	
	Category B		Category B		Category B	
	Stiffened at 8.75 ft		Stiffened at 13.13 ft		Unstiffened	
	Middle Section	End Section	Middle Section	End Section	Middle Section	End Section
Web Depth (in.)	64	64	64	64	64	64
Web Thickness (in.)	8/16	8/16	9/16	9/16	9/16	11/16
Top Flange Thickness (in.)	1	0.75	1	0.75	1	0.75
Top Flange Width (in.)	18	13	17	13	17	13
Bottom Flange Thickness (in.)	2.25	1.25	1.125	0.875	1.125	0.75
Bottom Flange Width (in.)	12	12	23	15	23	15
Cross Section Area (in^2)	77	56.75	78.88	58.88	78.88	65.00
Total Weight (kips) - 4 Girders	**123.08**		**126.60**		**130.98**	
Performance Ratios **Strength Limit State**						
Flexural Resistance	0.84	0.73	0.83	0.73	0.83	0.69
Ductility Requirement	0.90	0.79	0.90	0.79	0.90	0.75
Shear Resistance	0.46	0.84	0.47	0.88	0.97	0.78
Construction						
Web Slenderness	0.83	0.80	0.74	0.68	0.74	0.52
Compression Flange Slenderness	0.88	0.84	0.81	0.80	0.81	0.73
Flexural Resistance	0.90	1.00	0.99	0.98	0.99	0.98
Service II Limit State						
Perm. Deflection (tension flange)	1.00	0.98	1.00	1.00	1.00	0.99
Fatigue Limit State						
Bottom Flange Conn. Plate Weld	0.79	0.79	0.79	0.81	0.79	0.79

1 in = 25.4 mm

1 kip = 4.45 kN

Table 5.5 Summary of I-Girder Designs: Scheme 8, Category C' Fatigue Details
Web Depth = 52 in.

	Scheme 8		Scheme 8		Scheme 8	
	Category C'		Category C'		Category C'	
	Stiffened at 8.48 ft/9.85 ft		Stiffened at 12.71 ft/14.77 ft		Unstiffened	
	Middle Section	End Section	Middle Section	End Section	Middle Section	End Section
Web Depth (in.)	52	52	52	52	52	52
Web Thickness (in.)	8/16	8/16	9/16	9/16	9/16	10/16
Top Flange Thickness (in.)	1.125	1	1.125	1	1.125	0.875
Top Flange Width (in.)	22	23	22	22	22	23
Bottom Flange Thickness (in.)	2.25	1.375	2.5	1.5	2.5	1.5
Bottom Flange Width (in.)	17	17	15	15	15	14
Cross Section Area (in^2)	89.00	72.37	91.50	73.75	91.5	73.62
Total Weight (kips) - 4 Girders	**147.10**		**150.76**		**150.68**	
Performance Ratios **Strength Limit State**						
Flexural Resistance	0.86	0.74	0.84	0.72	0.83	0.72
Ductility Requirement	0.93	0.82	0.92	0.81	0.92	0.81
Shear Resistance	0.54	0.88	0.52	0.81	0.79	0.84
Construction						
Web Slenderness	0.69	0.51	0.60	0.46	0.60	0.42
Compression Flange Slenderness	0.91	0.88	0.87	0.82	0.87	0.98
Flexural Resistance	0.89	0.89	0.88	0.89	0.84	0.72
Service II Limit State						
Perm. Deflection (tension flange)	0.95	0.89	0.95	0.89	0.95	0.91
Fatigue Limit State						
Bottom Flange Conn. Plate Weld	1.00	1.00	0.99	0.99	0.99	1.00

1 in = 25.4 mm

1 kip = 4.45 kN

Table 5.6 Summary of I-Girder Designs: Scheme 8, Category C' Fatigue Details
Web Depth = 64 in.

	Scheme 8		Scheme 8		Scheme 8	
	Category C'		Category C'		Category C'	
	Stiffened at 8.48 ft/9.85 ft		Stiffened at 12.71 ft/14.77 ft		Unstiffened	
	Middle Section	End Section	Middle Section	End Section	Middle Section	End Section
Web Depth (in.)	64	64	64	64	64	64
Web Thickness (in.)	8/16	8/16	9/16	9/16	9/16	11/16
Top Flange Thickness (in.)	1	0.875	1	0.875	1	0.875
Top Flange Width (in.)	21	22	21	22	21	22
Bottom Flange Thickness (in.)	2.25	1.25	1.75	1	1.75	0.75
Bottom Flange Width (in.)	13	13	16	15	16	17
Cross Section Area (in^2)	82.25	67.50	85.00	70.25	85.00	76.00
Total Weight (kips) - 4 Girders	**136.38**		**141.30**		**145.41**	
Performance Ratios **Strength Limit State**						
Flexural Resistance	0.81	0.69	0.80	0.68	0.80	0.65
Ductility Requirement	0.86	0.75	0.86	0.74	0.86	0.72
Shear Resistance	0.94	0.89	0.46	0.86	0.94	0.90
Construction						
Web Slenderness	0.77	0.57	0.67	0.50	0.67	0.39
Compression Flange Slenderness	0.97	0.96	0.93	0.92	0.93	0.86
Flexural Resistance	0.96	0.96	0.95	0.91	0.95	0.91
Service II Limit State						
Perm. Deflection (tension flange)	0.94	0.89	0.95	0.89	0.94	0.90
Fatigue Limit State						
Bottom Flange Conn. Plate Weld	0.99	1.00	1.00	1.00	1.00	0.99

1 in = 25.4 mm

1 kip = 4.45 kN

Table 5.7 Summary of I-Girder Designs: Scheme 8, Category B Fatigue Details
Web Depth = 52 in.

	Scheme 8		Scheme 8		Scheme 8	
	Category B		Category B		Category B	
	Stiffened at 8.48 ft/9.85 ft		Stiffened at 12.71 ft/14.77 ft		Unstiffened	
	Middle Section	End Section	Middle Section	End Section	Middle Section	End Section
Web Depth (in.)	52	52	52	52	52	52
Web Thickness (in.)	8/16	8/16	9/16	9/16	9/16	10/16
Top Flange Thickness (in.)	1.125	1	1.125	1	1.125	0.875
Top Flange Width (in.)	22	22	22	22	22	23
Bottom Flange Thickness (in.)	1	0.75	1.375	0.75	1.375	0.75
Bottom Flange Width (in.)	35	26	25	25	25	24
Cross Section Area (in^2)	85.75	67.50	88.37	70.00	88.37	70.62
Total Weight (kips) - 4 Girders	**140.14**		**144.74**		**145.18**	
Performance Ratios **Strength Limit State**						
Flexural Resistance	0.91	0.81	0.89	0.79	0.89	0.78
Ductility Requirement	1.00	0.90	0.98	0.88	0.98	0.87
Shear Resistance	0.54	1.00	0.52	0.88	0.79	0.84
Construction						
Web Slenderness	0.68	0.50	0.59	0.44	0.59	0.41
Compression Flange Slenderness	0.91	0.85	0.88	0.82	0.88	0.98
Flexural Resistance	0.91	0.96	0.90	0.95	0.90	0.96
Service II Limit State						
Perm. Deflection (tension flange)	1.00	1.00	1.00	1.00	1.00	1.00
Fatigue Limit State						
Bottom Flange Conn. Plate Weld	0.82	0.85	0.80	0.84	0.80	0.83

1 in = 25.4 mm

1 kip = 4.45 kN

Table 5.8 Summary of I-Girder Designs: Scheme 8, Category B Fatigue Details
Web Depth = 64 in.

	Scheme 8		Scheme 8		Scheme 8	
	Category B		Category B		Category B	
	Stiffened at 8.48 ft/9.85 ft		Stiffened at 12.71 ft/14.77 ft		Unstiffened	
	Middle Section	End Section	Middle Section	End Section	Middle Section	End Section
Web Depth (in.)	64	64	64	64	64	64
Web Thickness (in.)	8/16	8/16	9/16	9/16	9/16	11/16
Top Flange Thickness (in.)	1	0.875	1	0.875	1	0.875
Top Flange Width (in.)	21	22	21	22	21	22
Bottom Flange Thickness (in.)	0.875	0.75	0.875	0.875	0.875	0.75
Bottom Flange Width (in.)	30	18	29	14	29	14
Cross Section Area (in^2)	79.25	64.75	82.37	67.50	82.37	73.75
Total Weight (kips) - 4 Girders	**131.20**		**136.52**		**140.99**	
<u>Performance Ratios</u> **Strength Limit State**						
Flexural Resistance	0.86	0.75	0.84	0.73	0.84	0.69
Ductility Requirement	0.92	0.81	0.90	0.80	0.90	0.76
Shear Resistance	0.45	0.81	0.46	0.86	0.97	0.91
Construction						
Web Slenderness	0.75	0.55	0.65	0.48	0.65	0.38
Compression Flange Slenderness	0.97	0.96	0.93	0.92	0.93	0.86
Flexural Resistance	0.98	0.92	0.97	0.91	0.97	0.91
Service II Limit State						
Perm. Deflection (tension flange)	1.00	0.99	1.00	1.00	1.00	0.98
Fatigue Limit State						
Bottom Flange Conn. Plate Weld	0.81	0.84	0.81	0.83	0.87	0.80

1 in = 25.4 mm

1 kip = 4.45 kN

Table 5.9 Summary of Composite Tubular Flange Girder Designs: Scheme 9
Web Depth Plus Tube Diameter = 52 in.

	Scheme 9		Scheme 9		Scheme 9		Scheme 9	
	Category C'		Category C'		Category B		Category B	
	Stiffened at 9.37 ft		Unstiffened		Stiffened at 9.37 ft		Unstiffened	
	Middle Section	End Section	Middle Section	End Section	Middle Section	End Section	Middle Section	End Section
Web Depth (in.)	36	36	36	36	36	36	36	36
Web Thickness (in.)	8/16	8/16	8/16	9/16	8/16	8/16	8/16	9/16
Tube Thickness (in.)	6/16	6/16	6/16	6/16	6/16	6/16	6/16	6/16
Tube Diameter (in.)	16	16	16	16	16	16	16	16
Bottom Flange Thickness (in.)	2.5	1.375	2.5	1	2.5	1.375	2.5	1
Bottom Flange Width (in.)	16	17	16	22	16	16	16	21
Cross Section Area (in^2)	76.41	59.78	76.41	60.66	76.41	58.41	76.41	59.66
Total Weight (kips)	**24.61**		**125.23**		**123.63**		**124.52**	
Dead Load Deflection (in.)	6.09		6.10		6.11		6.14	
Live Load Deflection (in.)	0.61		0.61		0.61		0.61	
Performance Ratios **Strength Limit State**								
Flexural Resistance	0.87	0.76	0.87	0.76	0.87	0.79	0.87	0.78
Shear Resistance	0.79	0.96	0.79	0.81	0.79	0.96	0.79	0.81
Construction								
Web Slenderness	0.37	0.22	0.37	0.19	0.37	0.21	0.37	0.18
Tube Thickness Requirement	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Flexural Resistance	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Service II Limit State								
Perm. Deflection (tension flange)	1.00	0.95	1.00	0.96	1.00	0.99	1.00	1.00
Fatigue Limit State								
Bottom Flange Conn. Plate Weld	0.96	0.99	0.96	1.00	0.72	0.77	0.72	0.77

1 in = 25.4 mm

1 kip = 4.45 kN

Table 5.10 Summary of Composite Tubular Flange Girder Designs: Scheme 9
Web Depth Plus Tube Diameter = 64 in.

	Scheme 9		Scheme 9		Scheme 9		Scheme 9	
	Category C'		Category C'		Category B		Category B	
	Stiffened at 9.37 ft		Unstiffened		Stiffened at 9.37 ft		Unstiffened	
	Middle Section	End Section	Middle Section	End Section	Middle Section	End Section	Middle Section	End Section
Web Depth (in.)	50	50	50	50	50	50	50	50
Web Thickness (in.)	8/16	8/16	9/16	10/16	8/16	8/16	9/16	10/16
Tube Thickness (in.)	4/16	4/16	4/16	4/16	4/16	4/16	4/16	4/16
Tube Diameter (in.)	14	14	14	14	14	14	14	14
Bottom Flange Thickness (in.)	1.125	0.75	1.75	0.875	1.125	1	1.75	0.875
Bottom Flange Width (in.)	27	22	17	16	27	16	17	16
Cross Section Area (in^2)	66.17	52.30	68.67	56.05	66.17	51.80	68.67	56.05
Total Weight (kips)	**108.29**		**113.65**		**107.94**		**113.65**	
Dead Load Deflection (in.)	4.93		4.88		4.93		4.88	
Live Load Deflection (in.)	0.51		0.50		0.51		0.50	
Performance Ratios								
Strength Limit State								
Flexural Resistance	0.83	0.73	0.81	0.71	0.83	0.74	0.81	0.71
Shear Resistance	0.58	0.93	0.77	0.82	0.58	0.93	0.77	0.82
Construction								
Web Slenderness	0.57	0.37	0.50	0.29	0.57	0.36	0.50	0.29
Tube Thickness Requirement	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99
Flexural Resistance	0.99	1.00	0.97	0.98	0.99	1.00	0.97	0.98
Service II Limit State								
Perm. Deflection (tension flange)	0.99	0.93	0.98	0.95	0.98	0.95	0.98	0.95
Fatigue Limit State								
Bottom Flange Conn. Plate Weld	0.74	1.00	0.97	1.00	0.74	0.76	0.73	0.75

1 in = 25.4 mm

1 kip = 4.45 kN

Table 5.11 Summary of Composite Tubular Flange Girder Designs: Scheme 1 and 10
Web Depth Plus Tube Diameter = 52 in.

	Scheme 10		Scheme 1	
	Category B		Category B	
	Unstiffened		Unstiffened	
	Middle Section	End Section	Middle Section	End Section
Web Depth (in.)	34	34	36	36
Web Thickness (in.)	8/16	8/16	8/16	9/16
Tube Thickness (in.)	6/16	6/16	6/16	6/16
Tube Diameter (in.)	18	18	16	16
Bottom Flange Thickness (in.)	2	1.25	2.5	1
Bottom Flange Width (in.)	20	18	16	21
Cross Section Area (in^2)	77.76	62.39	76.41	59.66
Total Weight (kips)	**127.93**		**124.52**	
Dead Load Deflection (in.)	6.12		6.11	
Live Load Deflection (in.)	0.60		0.61	
Performance Ratios **Strength I Limit State**				
Flexural Resistance	0.87	0.75	0.87	0.78
Shear Resistance	0.74	0.79	0.79	0.81
Strength III Limit State				
Flexural Resistance	0.46	0.34	-	-
Strength V Limit State				
Flexural Resistance	0.79	0.65	-	-
Construction				
Web Slenderness	0.27	0.13	0.37	0.18
Tube Thickness Requirement	0.84	0.84	0.75	0.75
Flexural Resistance	0.94	0.95	0.95	0.66
Service II Limit State				
Perm. Deflection (tension flange)	0.99	0.94	1.00	1.00
Fatigue Limit State				
Bottom Flange Conn. Plate Weld	0.72	0.73	0.72	0.77

1 in = 25.4 mm

1 kip = 4.45 kN

Table 5.12 Summary of Composite Tubular Flange Girder Designs: Scheme 1 and 10
Web Depth Plus Tube Diameter = 64 in.

	Scheme 10		Scheme 1	
	Category B		Category B	
	Unstiffened		Unstiffened	
	Middle Section	End Section	Middle Section	End Section
Web Depth (in.)	46	46	50	50
Web Thickness (in.)	8/16	10/16	9/16	10/16
Tube Thickness (in.)	6/16	6/16	4/16	4/16
Tube Diameter (in.)	18	18	14	14
Bottom Flange Thickness (in.)	1.5	0.75	1.75	0.875
Bottom Flange Width (in.)	19	16	17	16
Cross Section Area (in^2)	72.26	61.51	68.67	56.05
Total Weight (kips)	**121.41**		**113.65**	
Dead Load Deflection (in.)	5.05		4.88	
Live Load Deflection (in.)	0.48		0.50	
Performance Ratios **Strength I Limit State**				
Flexural Resistance	0.84	0.72	0.81	0.71
Shear Resistance	1.00	0.76	0.77	0.82
Strength III Limit State				
Flexural Resistance	0.47	0.33	-	-
Strength V Limit State				
Flexural Resistance	0.77	0.63	-	-
Construction				
Web Slenderness	0.29	0.12	0.50	0.29
Tube Thickness Requirement	0.88	0.84	0.99	0.99
Flexural Resistance	0.79	0.80	0.97	0.98
Service II Limit State				
Perm. Deflection (tension flange)	1.00	0.98	0.98	0.95
Fatigue Limit State				
Bottom Flange Conn. Plate Weld	0.76	0.79	0.73	0.75

1 in = 25.4 mm

1 kip = 4.45 kN

Table 5.13 Summary of Non-composite Tubular Flange Girder Designs: Scheme 9, Category B Fatigue Details
Web Depth Plus Tube Diameter = 52 in.

	Scheme 9		Scheme 9		Scheme 9	
	Category B		Category B		Category B	
	Fy=70 ksi; f'c=4 ksi		Fy=100 ksi; f'c=8 ksi		Fy=70 ksi & 100 ksi; f'c=8 ksi	
	Unsiffened		Unstiffened		Unstiffened	
	Middle Section	End Section	Middle Section	End Section	Middle Section	End Section
Web Depth (in.)	40	40	26	26	26	26
Web Thickness (in.)	9/16	9/16	7/16	8/16	7/16	8/16
Tube Thickness (in.)	41/16	41/16	12/16	12/16	12/16	12/16
Tube Diameter (in.)	12	12	26	26	26	26
Bottom Flange Thickness (in.)	3	1.375	3	1.125	3	1.125
Bottom Flange Width (in.)	21	26	25	23	25	23
Cross Section Area (in^2)	161.48	134.23	145.87	98.37	145.87	98.37
Total Weight (kips)	**268.97**		**226.63**		**226.63**	
Dead Load Deflection (in.)	8.99		10.34		10.34	
Live Load Deflection (in.)	1.90		1.83		1.83	
<u>Performance Ratios</u> **Strength Limit State**						
Flexural Resistance	0.89	0.85	0.86	0.85	0.90	0.85
Shear Resistance	1.00	1.00	0.87	0.87	0.85	0.87
Construction						
Web Slenderness	0.25	0.14	0.03	-0.09	0.03	-0.09
Tube Thickness Requirement	0.08	0.08	0.73	0.73	0.61	0.73
Flexural Resistance	0.46	0.48	0.38	0.38	0.39	0.38
Service II Limit State						
Perm. Deflection (tension flange)	1.00	1.00	0.99	0.94	1.00	0.94
Fatigue Limit State						
Bottom Flange Conn. Plate Weld	0.65	0.75	0.60	1.00	0.60	1.00

1 in = 25.4 mm

1 kip = 4.45 kN

Table 5.14 Summary of Non-composite Tubular Flange Girder Designs: Scheme 9, Category B Fatigue Details
Web Depth Plus Tube Diameter = 64 in.

	Scheme 9		Scheme 9		Scheme 9	
	Category B		Category B		Category B	
	Fy=70 ksi; F'c=4 ksi		Fy=100 ksi; f'c=8 ksi		Fy=70 ksi & 100 ksi; f'c=8 ksi	
	Unsiffened		Unstiffened		Unstiffened	
	Middle Section	End Section	Middle Section	End Section	Middle Section	End Section
Web Depth (in.)	34	34	40	40	40	40
Web Thickness (in.)	8/16	9/16	8/16	9/16	8/16	9/16
Tube Thickness (in.)	9/16	9/16	9/16	9/16	7/16	7/16
Tube Diameter (in.)	30	30	24	24	24	24
Bottom Flange Thickness (in.)	2	1.25	1.75	0.75	2.5	0.875
Bottom Flange Width (in.)	26	22	19	24	16	21
Cross Section Area (in^2)	121.02	98.65	94.66	81.92	92.39	73.26
Total Weight (kips)	**200.19**		**160.00**		**151.36**	
Dead Load Deflection (in.)	8.31		10.23		10.11	
Live Load Deflection (in.)	1.45		1.84		1.70	
Performance Ratios **Strength Limit State**						
Flexural Resistance	0.94	0.80	0.91	0.73	0.91	0.83
Shear Resistance	0.77	0.82	0.90	0.93	0.90	0.94
Construction						
Web Slenderness	0.01	-0.05	0.04	-0.02	0.08	-0.01
Tube Thickness Requirement	0.94	0.94	0.90	0.90	0.97	0.97
Flexural Resistance	0.39	0.39	0.39	0.39	0.39	0.39
Service II Limit State						
Perm. Deflection (tension flange)	1.00	1.00	1.00	0.93	1.00	0.93
Fatigue Limit State						
Bottom Flange Conn. Plate Weld	0.68	0.75	0.96	1.00	0.83	0.99

1 in = 25.4 mm

1 kip = 4.45 kN

Table 5.15 Summary of Through-Girder Designs: Category B Fatigue Details, Unstiffened
Web Depth Plus Tube Diameter = 83 in.

	Category B		Category B		Category B	
	Fy=70 ksi; f'c=4 ksi		Fy=100 ksi; f'c=8 ksi		Fy=70 ksi & 100 ksi; f'c=8 ksi	
	Unsiffened		Unstiffened		Unstiffened	
	Middle Section	End Section	Middle Section	End Section	Middle Section	End Section
Web Depth (in.)	63	63	63	63	63	63
Web Thickness (in.)	10/16	12/16	10/16	12/16	10/16	12/16
Tube Thickness (in.)	19/16	19/16	16/16	16/16	16/16	16/16
Tube Diameter (in.)	20	20	20	20	20	20
Bottom Flange Thickness (in.)	2	1	2	0.75	2	0.75
Bottom Flange Width (in.)	28	20	20	19	20	19
Cross Section Area (in^2)	165.56	137.43	139.07	121.19	139.07	121.19
Total Weight (kips)	**137.82**		**117.82**		**117.82**	
Dead Load Deflection (in.)	5.47		6.59		6.59	
Live Load Deflection (in.)	1.13		1.32		1.32	
<u>Performance Ratios</u> **Strength Limit State**						
Flexural Resistance	1.00	1.00	1.00	1.00	1.00	1.00
Shear Resistance	0.91	0.79	0.91	0.79	0.91	0.79
Web Slenderness	0.25	0.10	0.19	0.08	0.19	0.08
Tube Thickness Requirement	0.30	0.30	0.42	0.42	0.35	0.35
Construction						
Flexural Resistance	0.43	0.43	0.43	0.43	0.43	0.43
Service II Limit State						
Perm. Deflection (tension flange)	0.80	0.96	0.77	0.80	0.77	0.80
Fatigue Limit State						
Bottom Flange Conn. Plate Weld	0.36	0.48	0.47	0.56	0.47	0.56

1 in = 25.4 mm

1 kip = 4.45 kN

Table 5.16 Summary of Through-Girder Designs: Category B Fatigue Details, Unstiffened
Web Depth Plus Tube Diameter = 95 in.

	Category C'		Category C'		Category C'	
	Fy=70 ksi; F'c=4 ksi		Fy=100 ksi; f'c=8 ksi		Fy=70 ksi & 100 ksi; f'c=8 ksi	
	Unsiffened		Unstiffened		Unstiffened	
	Middle Section	End Section	Middle Section	End Section	Middle Section	End Section
Web Depth (in.)	69	69	69	69	71	71
Web Thickness (in.)	10/16	12/16	10/16	12/16	11/16	12/16
Tube Thickness (in.)	8/16	8/16	9/16	9/16	7/16	7/16
Tube Diameter (in.)	26	26	26	26	24	24
Bottom Flange Thickness (in.)	1.75	0.75	0.75	0.75	0.75	0.75
Bottom Flange Width (in.)	21	22	24	23	26	20
Cross Section Area (in^2)	119.93	108.31	106.08	113.95	100.70	100.64
Total Weight (kips)	**102.96**		**97.56**		**89.92**	
Dead Load Deflection (in.)	5.79		7.03		7.26	
Live Load Deflection (in.)	1.07		1.34		0.32	
Performance Ratios **Strength Limit State**						
Flexural Resistance	1.00	1.00	0.97	0.97	1.00	1.00
Shear Resistance	0.99	0.86	0.99	0.86	0.77	0.89
Web Slenderness	0.19	0.09	0.07	0.05	0.13	0.09
Tube Thickness Requirement	0.92	0.92	0.97	0.97	0.97	0.97
Construction						
Flexural Resistance	0.43	0.43	0.42	0.42	0.43	0.43
Service II Limit State						
Perm. Deflection (tension flange)	0.96	0.89	0.99	0.61	0.91	0.63
Fatigue Limit State						
Bottom Flange Conn. Plate Weld	0.43	0.45	0.66	0.43	0.61	0.46

1 in = 25.4 mm

1 kip = 4.45 kN

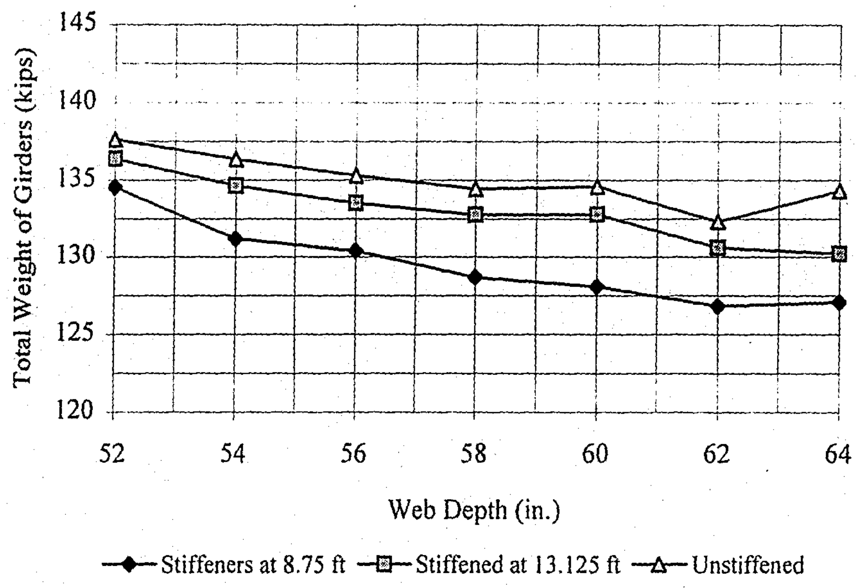


Figure 5.1 I-Girders:
Category C' Fatigue Details; Scheme 1 Cross-Frame Arrangement

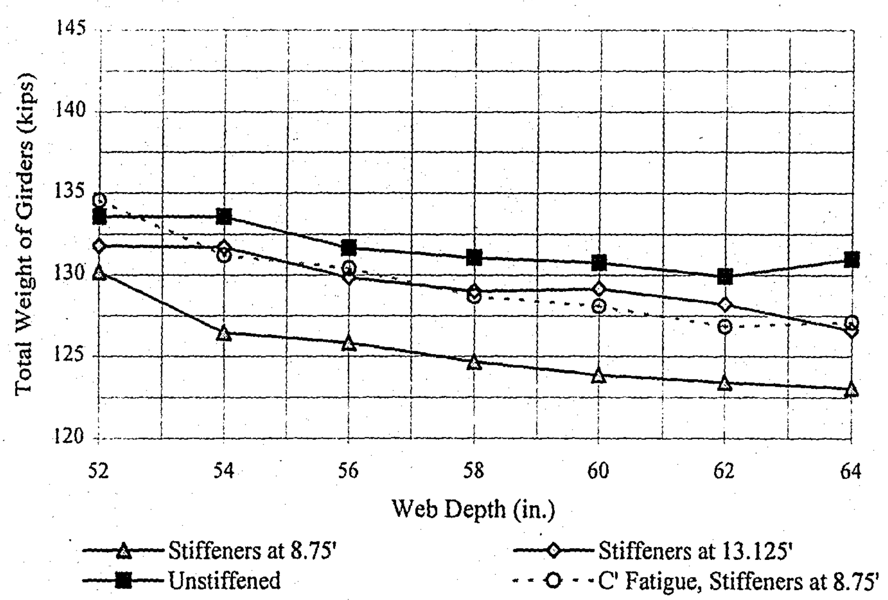


Figure 5.2 I-Girders:
Category B Fatigue Details; Scheme 1 Cross-Frame Arrangement

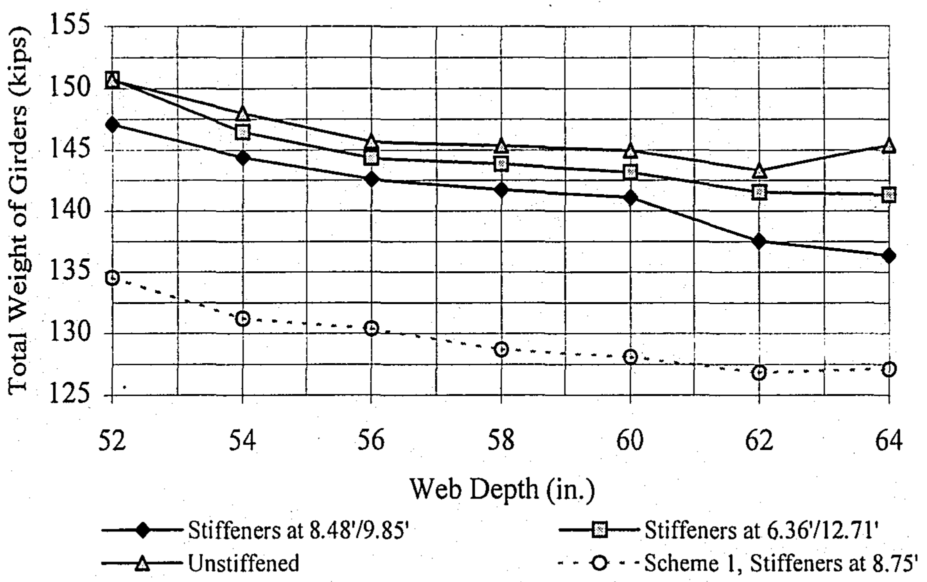


Figure 5.3 I-Girders:
Category C' Fatigue Details; Scheme 8 Cross-Frame Arrangement

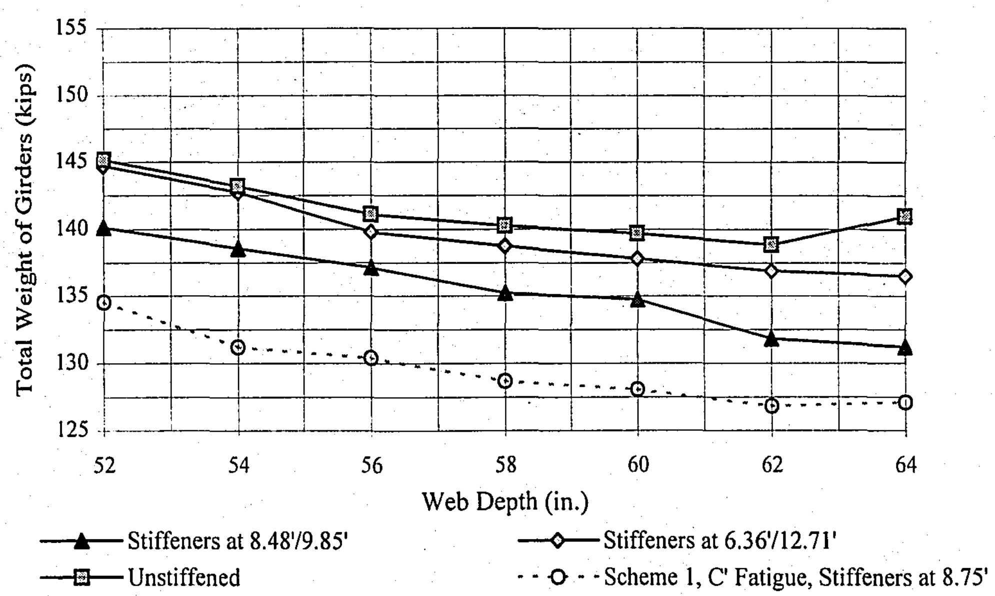


Figure 5.4 I-Girders:
Category B Fatigue Details; Scheme 8 Cross-Frame Arrangement

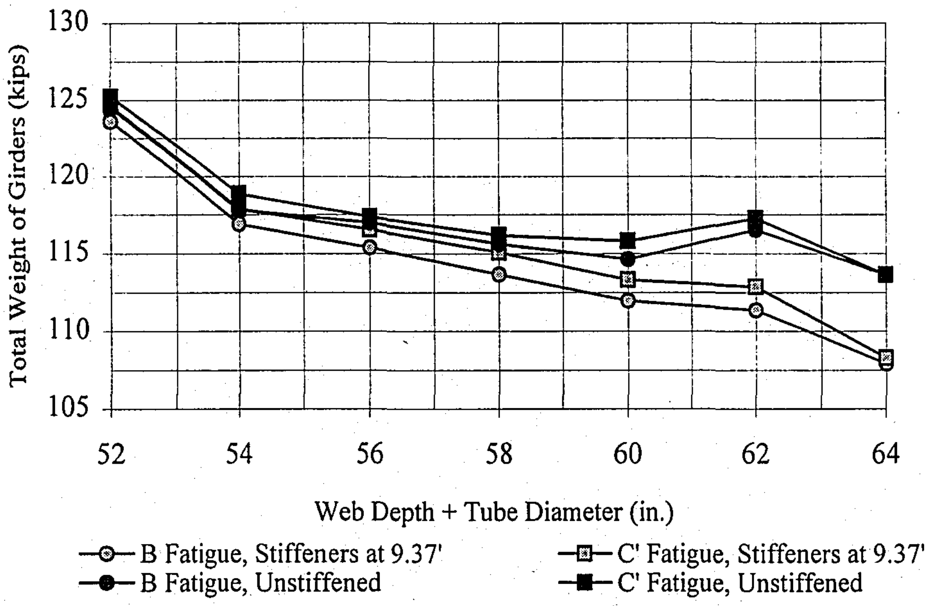


Figure 5.5 Composite Tubular Flange Girders:
Scheme 9 Cross-Frame Arrangement

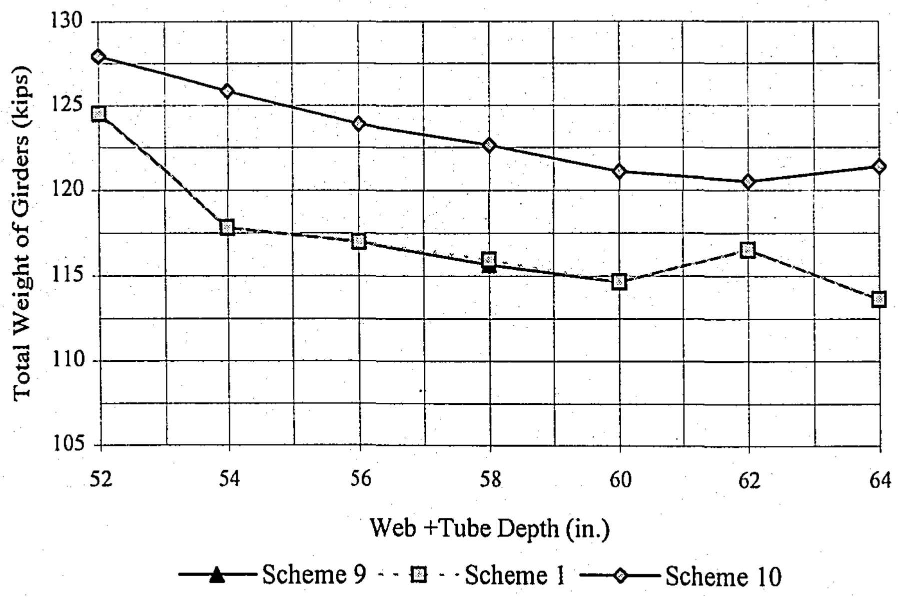


Figure 5.6 Composite Tubular Flange Girders:
Unstiffened, Category B Fatigue Details

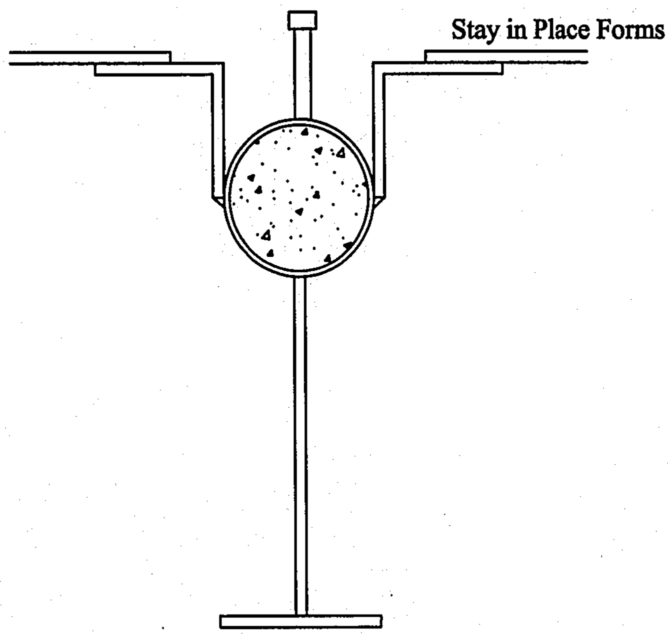


Figure 5.7 Construction of Composite Tubular Flange

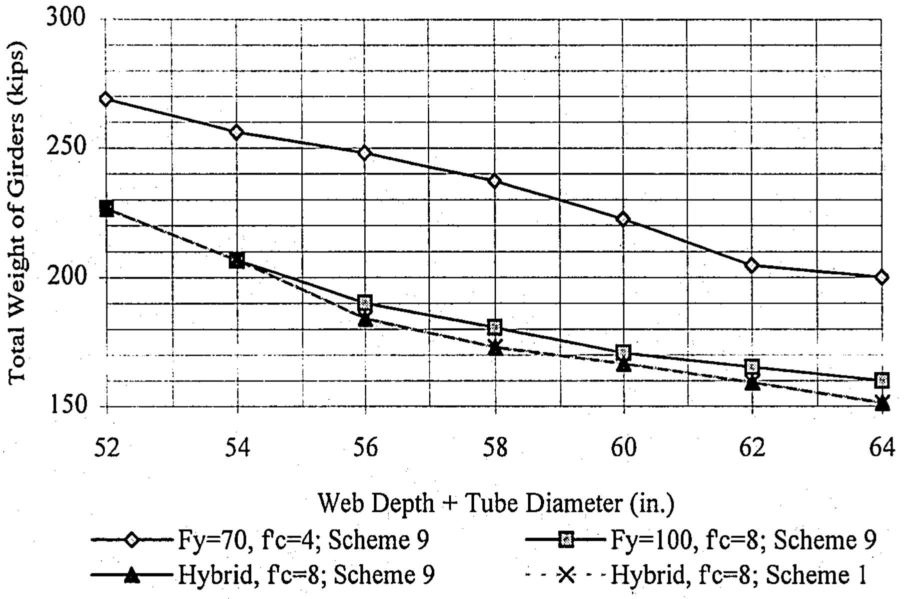


Figure 5.8 Non-Composite Tubular Flange Girders:
Unstiffened; Category B Fatigue Details

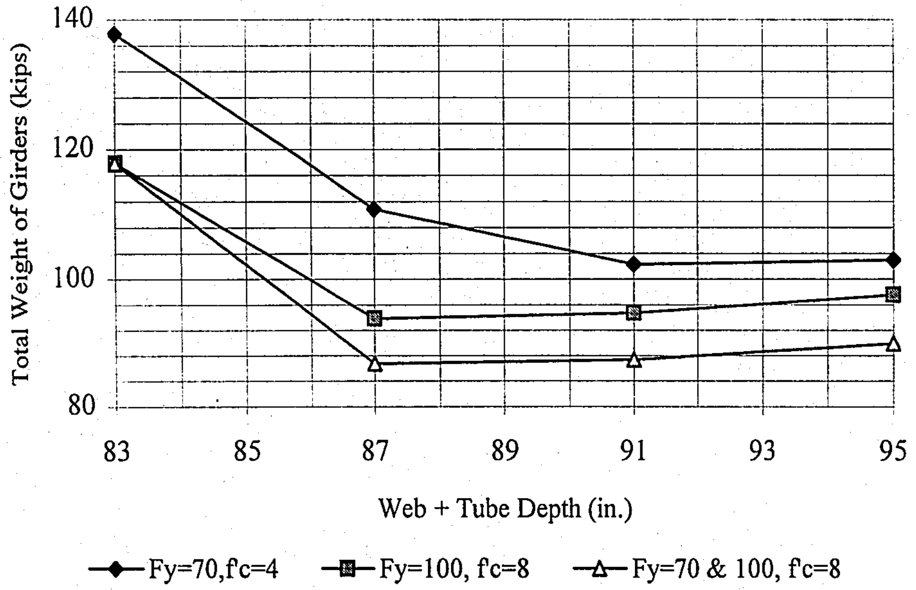


Figure 5.9 Through-Girders:
Unstiffened; Category B Fatigue Details

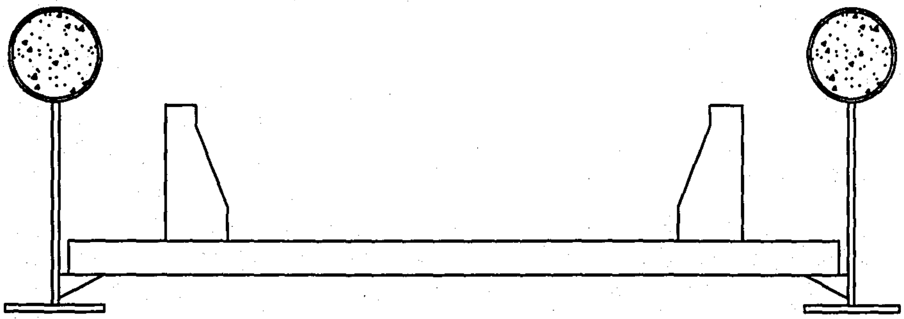


Figure 5.10 Through-Girder Bridge Design (1)

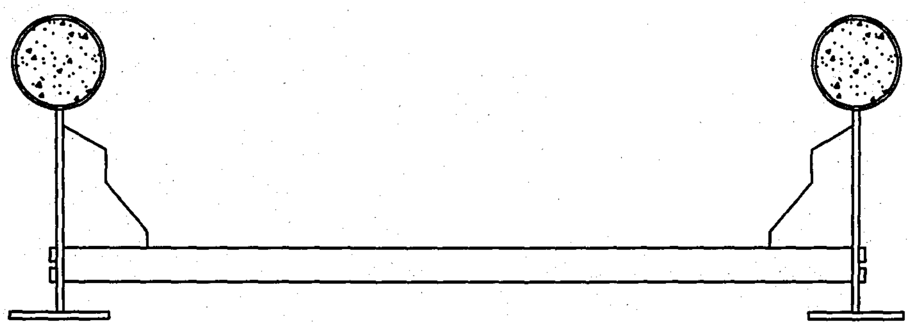


Figure 5.11 Through-Girder Bridge Design (2)

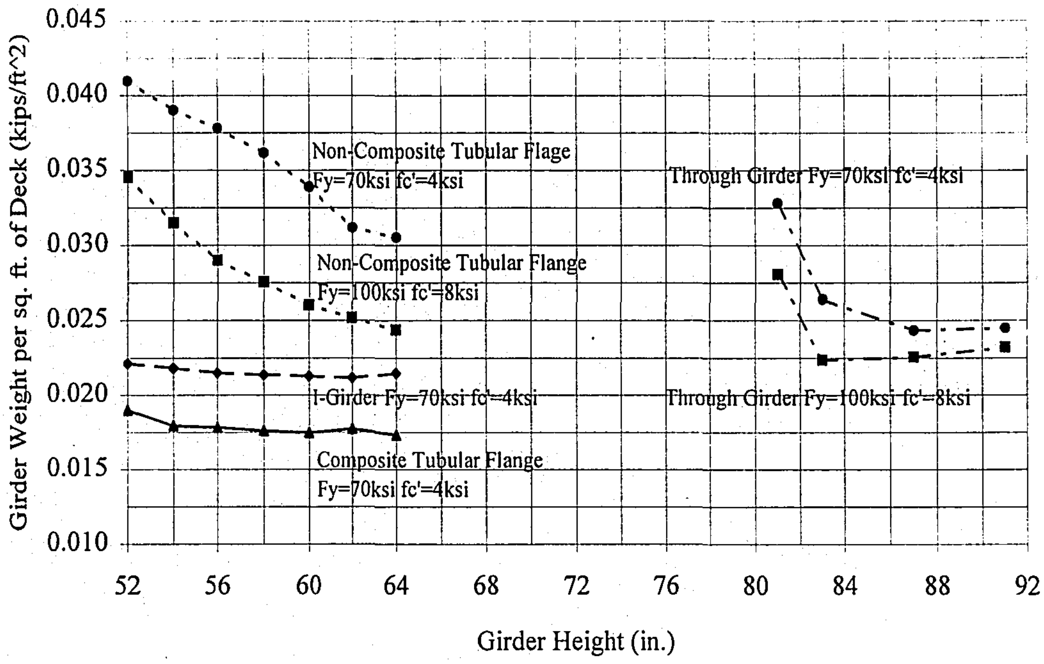


Figure 5.12 Girders Designed for Four-Girder and Through-Girder Prototype Bridges: Unstiffened; Category B Fatigue Details

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 SUMMARY

The objective of the research presented in this thesis is to study the possible advantages of tubular flange girders. I-girders and tubular flange girders were designed and the designs were compared to identify the advantages of tubular flange girders. The influence of bridge design parameters, such as cross-frame and stiffeners spacing, was investigated.

The combinations of prototype bridges and girder types that were studied include: (1) a four-girder prototype bridge with composite I-girders, (2) a four-girder prototype bridge with composite tubular flange girders, (3) a four-girder prototype bridge with non-composite tubular flange girders, and (4) a through-girder prototype bridge with tubular flange girders. The different parameters that were varied in the design studies include: (1) transverse web stiffener spacing, (2) cross-frame spacing, and (3) category of the fatigue details (B or C′). All of the prototype bridges were 131.24 ft (40000 mm) single-span bridges. Also, the type of steel and concrete used in the girder was varied in the non-composite girder and through-girder bridge designs.

All of the bridge girders have been designed according to AASHTO LRFD specifications (AASHTO LRFD 1998) for the strength I, service II, and fatigue limit states. Certain versions of the four-girder prototype bridge with composite tubular flanges were checked for the strength III and strength V limit states. Also, the service I load combination was used to calculate live load deflections.

6.2 CONCLUSIONS

The design studies showed that as the numbers of cross-frames and stiffeners increase, and as fatigue details are upgraded (from a category C′ to a category B), the total girder weight decreases. However, these changes in bridge design parameters increase the effort required to construct the bridge. The girder weight must be balanced against the construction effort to design the most economical bridge.

When considering the use of cross-frames, this study found that I-girders designed with the scheme 8 cross-frame arrangement (2 interior cross-frames) were 9% heavier than I-girders designed with the scheme 1 cross-frame arrangement (4 interior cross-frames) For the composite tubular flange girder, the total girder weight did not change as the cross-frame arrangement was varied between scheme 1 and scheme 9 (1 interior cross-frame). The reason for this result is that the tubular flange girders have enough lateral-torsional buckling resistance to be efficient with a long unbraced lengths, while I-girders become inefficient with long unbraced lengths. In regard to the effort required to fabricate and install cross-frames, as the number of interior cross-frames included in the design decreases, the required effort decreases.

Increasing the number of transverse stiffeners in the girder designs decreases the weight of the girders. However, the fabrication effort is increased as the stiffeners are added to the design. The same thing occurs when the fatigue detail category is upgraded from category C′ to category B. Category B fatigue details are more labor intensive, however less steel is needed in the girders.

The advantages of using tubular flange girders instead of I-girders include: (1) tubular flange girders require smaller girder weights than conventional I-girders, and (2) fewer interior cross-frames are required. By decreasing the number of cross-frames, the fabrication effort will decrease. However, a drawback of using tubular flange girders is the fabrication effort for an actual I-girder may be less than for a tubular flange girder. The through-girder design with tubular flanges has also been shown to be a viable option. The total girder weights for the through-girders are comparable to the total weights of the four-girder prototype bridges. This study has shown that tubular flange girders are a viable option for steel girder design.

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VITA

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END

OF

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