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Design and behavior of bearing stiffeners and splice plates used for corrugated web girders

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Design and Behavior of Bearing Stiffeners and Splice Plates Used for Corrugated...

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DESIGN AND BEHAVIOR OF BEARING STIFFENERS AND SPLICE PLATES
USED FOR CORRUGATED WEB GIRDERS

by

Tameka Clarke

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ABSTRACT

The “Pennsylvania Innovative High Performance Steel Bridge Demonstration Project” is sponsored by the Pennsylvania Department of Transportation. The purpose of this project is to investigate the use of innovative bridge member configurations, in particular corrugated web I-girders, with high performance steel (HPS). As a part of this project, the objective of this thesis research is to investigate the design and behavior of bearing stiffeners and splice plates for corrugated web girders.

Corrugated web I-girders consist of top and bottom flanges welded to a corrugated web. The web is fully corrugated over the depth of the web with trapezoidal corrugations continuous in the direction parallel to the axis of the girder. The corrugations increase the stiffness of the web and help overcome web stability, fatigue and service limit states.

The bearing stiffeners of a typical I-girder with a flat web are designed to meet the requirements of the AASHTO LRFD bridge design specifications. This study discusses the strength of bearing stiffeners for corrugated web I-girders that are designed according to the AASHTO LRFD bridge design specifications where applicable. Two test specimens were tested. The results show that the ultimate strength of bearing stiffeners designed for corrugated web I-girders equals their full plastic capacity and the nominal axial capacity of bearing stiffeners of corrugated web I-girders exceeds that of a flat web I-girder with identical stiffener parameters.

Flange and web splices can be easily designed to meet the requirements of the AASHTO LRFD bridge design specifications. This study presents the design concepts
for these splices and investigates the behavior of splices designed for corrugated web I-girders. Test results are presented, which indicate that these splices are stronger than predicted from AASHTO design equation.
CHAPTER 1 – INTRODUCTION

1.1 BACKGROUND

The research described in this thesis is part of the Pennsylvania Innovative High-Performance Steel Bridge Demonstration Project. It is sponsored by the Pennsylvania Department of Transportation, the Federal Highway Administration, and the Pennsylvania Infrastructure Technology Alliance (through a grant from the Pennsylvania Department of Community and Economic Development). This project investigates the use of high performance steel (HPS) in the design of I-girder highway bridges. The application of HPS offers potential savings over conventional steel. These can be realized with innovative bridge member configurations which maximize the potential of HPS. As part of this project, I-girders with corrugated webs have been investigated as an innovative system that utilizes the high strength of HPS.

The research presented in this thesis focuses on the design and behavior of bearing stiffeners, and bolted web and flange field splices used in corrugated web bridge I-girders. Experimental results are presented and analyzed, and design recommendations are made.

1.2 OBJECTIVE AND SCOPE

The objective of this work is to provide, through an experimental program, new information on the behavior of bearing stiffeners and bolted field splices for corrugated web bridge I-girders fabricated from HPS. To accomplish this objective, the following tasks are undertaken:
1. A study was made to compare the calculated strength of bearing stiffeners used in conventional and corrugated web I-girders.

2. Experiments to determine the strength of bearing stiffeners of corrugated web girders were conducted and the results compared with the calculated results from Task 1.

3. Conventional design criteria for flange and web splices were used to design splices for a corrugated web I-girder.

4. Experiments on flange and web splices were conducted to evaluate the applicability of conventional design criteria.

1.3 ORGANIZATION AND PRESENTATION

This thesis is presented in five chapters:

1. Introduction

2. Previous Research

3. Bearing Stiffeners for Corrugated Web I-Girders

4. Flange and Web Splices for Corrugated Web I-Girders

5. Summary and Conclusions

Chapter 2 will review previous research on the behavior of corrugated web I-girders. The first two sections of Chapter 2 will summarize previous research on the flexural, fatigue, and shear behavior of corrugated web I-girders. The next two sections will present information on an experimental investigation conducted at Lehigh University on the fatigue and shear behavior of corrugated web girders.
Chapter 3 focuses on bearing stiffeners for corrugated web I-girders. The first main section of Chapter 3 presents findings from a study of the calculated strength of bearing stiffeners. The subsequent two main sections discuss details of an experimental investigation of the strength of bearing stiffeners. The final section presents a summary and conclusions.

Chapter 4 focuses on flange and web splices for corrugated web I-girders. The first two main sections of Chapter 4 present the test concept for experiments to evaluate the behavior of the splices. Details of the experimental investigation, including the design of the flange and web splices for the experiments, are presented in the subsequent ten sections. The final section presents a summary and conclusions.

Chapter 5 summarizes the investigations, and makes recommendations.
CHAPTER 2 - PREVIOUS RESEARCH

2.1 INTRODUCTION

Corrugated web girders are being investigated (e.g., Elgaaly 1998, Abbas 2003) as a potential innovative bridge girder design that overcomes certain design limits, for example, web instability, excessive deflections and fatigue failure, that are impediments to the effective use of HPS in typical I-girders (Sause and Homma 1995, Sause and Fisher 1996). A corrugated I-girder consists of top and bottom flanges welded to a web, which has corrugations repeated in the direction parallel to the axis of the girder. The corrugations maybe trapezoidal, rectangular or curved, discrete or continuous.

Compared to typical I-girders, corrugated web girders have increased flexibility in bending due to the flexibility of the web (Abbas 2003). It is assumed that the flanges carry all the bending moment, while the entire vertical shear is resisted by the web. The corrugations of I-girders provide enhanced shear stability. This eliminates the need for transverse stiffeners with the effect of improving the fatigue behavior of the girder and also eliminating the cost and time associated with the welding of the stiffeners to the girder. The enhanced shear stability may result in a thinner, lighter web and thus, more economical girders. Previous research is discussed in the following sections pertaining to the flexural, shear and fatigue behavior of corrugated web girders.

2.2 BACKGROUND AND RELEVANT RESEARCH

In the 1980s, corrugated webs were first used in bridges. Four bridges with corrugated webs were constructed in France and at least three were constructed in Japan. The profile of the corrugations of these bridges was trapezoidal.
Corrugated webs are highly flexible in the longitudinal direction and, thus, the stresses in corrugated webs are mainly shear stresses with insignificant normal stresses due to bending. The primary failure mode for corrugated webs is shear instability which occurs as local buckling of a single fold, global buckling of an entire panel, or intermediate buckling which develops from interaction of local and global buckling.

Research related to corrugated web plates goes as far back as 1925. The most significant contributions related to this thesis are summarized below.

The flexural behavior of corrugated web girders has been studied by Aschinger and Linder (1997), Elgaaly et al. (1997), Johnson and Cafolla (1997c), Korashy and Varga (1979), Linder (1990, 1992), Linder and Aschinger (1990), and Protte (1993). These studies focused on the flexural capacity of corrugated web girders including lateral torsional buckling and local flange buckling.

The fatigue behavior of corrugated web girders has been investigated by Harrison (1965), Korashy and Varga (1979), Elgaaly et al. (2000), and Ibrahim (2001). Harrison tested two sinusoidal corrugated I-girders, Korashy and Varga tested eleven girders stiffened with discrete web corrugations, and Elgaaly studied trapezoidal corrugated I-girders. Test results indicated the fatigue strength of corrugated web girders is greater than that of I-girders stiffened with transverse stiffeners welded to the web.


From these studies, it has been observed that the shear instability failure modes of corrugated webs are local and global buckling. Local buckling involves a single fold, but, global buckling involves multiple folds, extending over the entire depth of the web. Based on these investigations, equations have been developed to predict the shear capacity of corrugated web girders (Abbas 2003).

The literature review suggests that there is a lack of information and test data on the flexural, shear, and fatigue strength of corrugated web girders for bridges with trapezoidal corrugations (Abbas 2003). At Lehigh University, further work has been
done on the flexural behavior under in-plane loading, the fatigue strength under cyclic loading, and the static shear capacity of corrugated web I-girders. This will be discussed in more detail in the following sections. Other aspects of corrugated web bridge I-girders, such as bearing stiffeners, cross frames and cross frame connection plates, and bolted splices have not been sufficiently studied. Bearing stiffeners and bolted splices are the subject of this thesis.

2.3 FATIGUE STRENGTH TESTS AT LEHIGH UNIVERSITY

Recent research at Lehigh University has investigated the fatigue behavior of corrugated web I-girders under cyclic loading (Abbas 2003). This research evaluated the factors that affected the fatigue life of corrugated web I-girders with emphasis on the fatigue strength of the flange-to-web fillet welds. Fatigue design criteria, which can be used to design corrugated web girders for highway bridges from HPS, were developed.

Fatigue strength tests were conducted on a total of six geometrically identical girder specimens (G1A through G6A). The I-girder specimens, fabricated from HPS-485W (Figure 2.1), were 7.4 m long, with a span length of 7 m. The full scale corrugated web profile (Figure 2.2) was trapezoidal in shape with a large bend radius of 120 mm. The flanges were 225 mm wide and 20 mm thick. The web was 1.2 m deep and 6 mm thick. The web-flange fillet welds (8 mm in size) were made using a semiautomatic gas metal arc welding (GMAW) process. Stiffeners, 20 mm thick, were provided at the reaction and load points. Two additional tests were conducted on two girder specimens that were repaired and retested (G4B and G1B). The damaged flanges
were replaced and welded to the web using robotic GMAW. These tests were conducted in order to evaluate the effect of the welding process on the fatigue life of corrugated web I-girders.

The specimens were tested under constant amplitude cyclic loading in four-point bending with two load and two reaction points. Under these loading conditions, a constant moment region developed near midspan which involved three full corrugation waves. This provided the potential for multiple crack initiation locations within the constant moment region. The test results are plotted in Figure 2.3 along with results from previous research on fatigue strength of corrugated web girders. The results agree with findings from previous researchers with regards to the increased fatigue strength of corrugated web I-girders compared to that of conventional stiffened web I-girders. The results indicate that the fatigue life for the robotically loaded specimens is slightly greater than that of specimens welded with semiautomatic GMAW.

2.4 SHEAR STRENGTH TESTS AT LEHIGH UNIVERSITY

Recent research at Lehigh University has investigated the shear strength of corrugated web I-girders (Abbas 2002). The research evaluated the shear capacity and presented additional information on the shear failure modes associated with web instability. Shear strength design criteria were developed, which can be used in the design of corrugated web I-girders for highway bridges fabricated from HPS.

Shear strength tests were conducted on two geometrically identical girder specimens (G7A and G8A). The I-girder specimens, fabricated from HPS-485W, (Figure 2.4) were 11.6 m long, with a span length of 11 m. The corrugated web is
shown in Figure 2.2. The flanges were 450 mm wide and 20 mm thick. The web was 1.5 m deep and 6 mm thick. The web-flange fillet welds (8 mm in size) were made using a semiautomatic gas metal arc welding (GMAW) process. Pairs of T-stiffeners were provided at the reaction and load points.

The specimens were tested under monotonic loading in three-point bending with one load point (1 m from mid-span) and two reaction points (Figure 2.4). Under these loading conditions, shear failure was forced in the shorter shear span, which had a shear span-to-web depth ratio of three. It was observed that the webs of both girders buckled suddenly. Girder G7A failed abruptly at a load level of 3892 kN (0.9τₚ, where τₚ is the shear yield stress). Failure by web buckling initially occurred in the upper half of the three longitudinal flat folds near the load point. Girder G8A had a similar mode of failure with failure occurring at a load level of 3647 kN (0.85τₚ). However, the folds that initially buckled were the three longitudinal flat folds near the support. The test results are plotted in Figure 2.5 along with results from previous research mentioned in Section 2.2.

The following equation for nominal design shear capacity, τₚ, was proposed by Abbas et al. (2002) for the entire range of shear behavior (elastic buckling, inelastic buckling and yield).

\[
τₚ = \sqrt{\left(\frac{τ_{cr,L} \cdot τ_{cr,G}}{τ_{cr,L} + τ_{cr,G}}\right)^2}
\]

(2.1)

where:

τₚ\text{,L} is the local buckling capacity from Eq. 2.2 or Eq. 2.4,

τₚ\text{,G} is the global buckling capacity from Eq. 2.3 or Eq. 2.4.
In Eq 2.1, the local and global capacities are calculated from Equations 2.2 through 2.5 using minimum values of $k_L$ and $k_G$. $k_L$ is minimized when the fold aspect ratio, $w/h_w$, is small. $k_L$ ranges from 5.34, for simply supported edges, to 8.98, for fixed edges (Abbas et. al 2002). $k_G$ is minimized for an infinitely long web. $k_G$ ranges from 31.6, for simply supported edges, to 59, for fixed edges (Abbas et. al 2002). Equation 2.2 provides the local elastic shear buckling stress, considering the elastic buckling capacity of an individual fold. The fold is assumed to be supported along its edges by the adjacent folds and the flanges.

\[(\tau_{el,L})_{el} = k_L \frac{\pi^2 E}{12(1-v^2)(w/t_w)^2}\]  

(2.2)

where:

- $E$ is the elastic modulus,
- $v$ is Poisson’s ratio,
- $w$ is the maximum fold width (considering both the inclined and the longitudinal folds),
- $t_w$ is the thickness of the web,
- $k_L$ is the boundary condition and fold aspect ratio factor.

Equation 2.3 provides the global elastic shear buckling stress, by treating the corrugated web as an orthotropic flat web.

\[(\tau_{el,G})_{el} = k_G \frac{Et_w^{1/2} b^{3/2}}{12h_w^2} \sqrt{\frac{(1+\beta)\sin^3 \alpha}{\beta + \cos \alpha}} \left(\frac{3\beta + 1}{\beta^2(\beta + 1)}\right)^{3/4}\]  

(2.3)

where:

- $h_w$ is the web height,
- $b$ is the longitudinal fold width,
- $\alpha$ is the corrugation angle,
- $\beta$ is the ratio of the longitudinal fold width to the inclined fold width,
\( k_0 \) is the boundary condition factor.

When the critical elastic buckling stress exceeds \( 0.8 \tau_y \), Equation 2.4 is used to determine the local and global buckling shear stresses. \( \tau_y \) is the shear yield stress according to Von Mises yield criterion, as defined as in Equation 2.5.

\[
(\tau_{cr})_{el} = \sqrt{0.8 \tau_y (\tau_{cr})_{el}} \leq \tau_y
\]  

(2.4)

\[
\tau_y = \frac{F_y}{\sqrt{3}}
\]  

(2.5)

where:

\( (\tau_{cr})_{el} \) is either \( (\tau_{cr,L})_{el} \) or \( (\tau_{cr,G})_{el} \) (i.e., Equation 2.4 can be used for either local or global buckling),

\( \tau_y \) is the shear yield stress,

\( F_y \) is the minimum specified yield strength of the web steel.

If one assumes that local buckling controls the shear behavior and the global buckling capacity (from Equation 2.4) is equal to the yield shear stress. Then, Equation 2.1 predicts the maximum local buckling stress (again from Equation 2.4) is \( 0.707 \tau_y \). As seen in Figure 2.5, Equation 2.1 represents a lower bound to test results controlled by local buckling. There is significant scatter in the results, which is attributed to geometric imperfections in the corrugated webs of the test specimens (Abbas et. al 2002). Considering the magnitude and distribution of imperfections that can be associated with corrugated web I-girders, Equation 2.1 provides a reasonable tool to predict the shear capacity of corrugated web I-girders.
Figure 2.1 Fatigue Test Specimen (units in mm): (a) Elevation, (b) Longitudinal

Figure 2.2 Web Profile of Corrugated I-girder Specimens (units in mm)
Figure 2.3 Fatigue Test Data

Figure 2.4 Shear Test Specimen (units in mm): (a) Elevation, (b) Longitudinal
Figure 2.5 Shear Test Data
CHAPTER 3 - BEARING STIFFENERS FOR CORRUGATED WEB I-GIRDERS

3.1. INTRODUCTION

This chapter addresses bearing stiffeners for corrugated web I-girders, focusing on their capacity under vertical loading. A brief comparison of bearing stiffeners for conventional flat web I-girders and those for corrugated web I-girders is made.

Three main topics are covered: (1) design of bearing stiffeners for corrugated web I-girders in the context of the AASHTO LRFD specifications (AASHTO 1998) for bearing stiffeners, (2) tests of bearing stiffeners for corrugated web I-girders, and (3) analysis of the test results and comparison with design calculations.

3.2. DESIGN CONSIDERATIONS FOR BEARING STIFFENERS FOR CORRUGATED WEB I-GIRDERS

For conventional I-girders with flat webs, bearing stiffeners are provided at the locations of bearings and other locations of concentrated loads. A bearing stiffener consists of two stiffener plates, with one plate attached to each side of the web. The bearing stiffener attachment to the web transmits the full force of the bearing reaction or the concentrated load. Bearing stiffeners are designed to extend to the full depth of the web. Each stiffener plate is attached to the flange by full penetration groove welds or is milled to fit against the flange. Bearing stiffeners are designed for the full bearing force due to the factored loads to prevent web crippling and to maintain cross section geometry.
The design requirements for bearing stiffeners for conventional flat web I-girders are outlined in the AASHTO LRFD specifications (AASHTO 1998). The AASHTO LRFD specifications, however, do not outline the design requirements for bearing stiffeners for corrugated web I-girders. The bearing stiffeners of test specimen G2A (see Section 2.3), used in the present study, were designed in accordance with the AASHTO LRFD specifications where applicable (Figure 3.1).

3.2.1 BEARING CAPACITY

The bearing capacity of a bearing stiffener is based on the bearing area and yield strength of the stiffener plates. The bearing area is considered to consist of the area of the stiffener plates in direct contact with the flange. Therefore, from Eq. 3.1, the bearing capacity of bearing stiffeners for corrugated web I-girders can be determined, as follows:

$$B_r = \phi_b \cdot A_{pn} \cdot F_{ys}$$

(3.1)

where:

$\phi_b$ is the resistance factor for bearing,

$A_{pn}$ is the area of direct bearing of the stiffener plates on the flange,

$F_{ys}$ is the specified minimum yield strength of the stiffener plates.

For the bearing stiffeners of test specimen G2A shown in Figure 3.1, $F_{ys} = 345$ MPa, and $A_{pn} = 2600$ mm$^2$ and with $\phi_b = 1$, $B_r = 897$ kN. If $B_r$ is less than the nominal axial resistance, $P_n$ of the bearing stiffener, it will control the stiffener design. For the
purpose of this study, the focus will be placed on the axial resistance, $P_n$, of the bearing stiffener.

3.2.2 AXIAL RESISTANCE

Introduction

In the AASHTO LRFD specifications (AASHTO 1998), bearing stiffeners of conventional flat web girders are analyzed as compression elements subject to the bearing force that develops due to the factored loads. In other words, a bearing stiffener is treated as a column, under axial force. The axial resistance of the bearing stiffener develops from a portion of the web acting in combination with the bearing stiffeners. The length of the web that contributes to resisting the axial force is assumed to be $9t_w$, on each side of the stiffener, where $t_w$ is the thickness of the web plate (AASHTO 1998). Therefore, the effective column section for a bearing stiffener for a conventional flat web girder consists of the two stiffener plates, plus $\gamma t_w$ (where $\gamma = 18$) of the web (Figure 3.2).

The bearing stiffeners for corrugated web I-girders will also be analyzed as compression elements subject to an axial force equal to the bearing force due to the factored loads. For an I-girder with trapezoidal corrugations (for example, girder G2A) with the bearing stiffener centered on the inclined fold of the corrugation (Figure 3.3), the effective column section is unknown. Assumptions regarding the effective column section will have to be made and verified. The main unknown is the amount of web that contributes to the axial force capacity of the bearing stiffener. Based on assumptions
regarding the web contribution, the axial capacity of the bearing stiffener can be predicted, and, later, compared to test results.

The following sections discuss the two assumptions that were made and used to determine the axial capacity of the bearing stiffeners for corrugated web I-girders. For consistency and clarity, the subscripts ‘C’ and ‘F’ are used to denote the corrugated web configuration and the flat web configuration, respectively.

Influence of Web Contribution on Axial Resistance of Bearing Stiffeners

Assumption 1 is that the total length of the web contribution \( \gamma t_w \) does not exceed the flat portion of the inclined fold (170 mm) on which the stiffener is centered (Figure 3.3 and Figure 3.4). For Assumption 1, the largest possible value of \( \gamma \), \( \gamma_{\text{max}} \), is 28 and the maximum length of the web contribution is 168 mm. The nominal axial capacity based on Assumption 1 was calculated three different ways: (1) using the column formula given in the AASHTO LRFD specifications (AASHTO 1998), (2) using the Euler column buckling formula, and (3) using the plastic capacity based on the gross section area.

The nominal axial resistance for bearing stiffeners of corrugated web I-girders determined from the AASHTO LRFD specifications, \( P_{nAC} \), is based on the assumption that slenderness requirements are satisfied and a compression failure governs the capacity. The slenderness requirement in the AASHTO LRFD specifications is

\[
\frac{b_t}{t_p} \leq k \cdot \frac{E}{F_{ys}}
\]

(3.2)

The nominal axial resistance from the column formula is:
\[ P \defx = 0.66^k \cdot F_y \cdot A \text{ if } \lambda \leq 2.25 \]  
\[ P \defx = \frac{0.88 \cdot F_y \cdot A}{\lambda} \text{ if } \lambda > 2.25 \]  

(3.3)

for which the slenderness parameter is:

\[ \lambda = \left(\frac{K L}{r \cdot \pi}\right)^2 \frac{F_{ys}}{E} \]  

(3.4)

where:

- \( b_t \) is the width of the stiffener plate,
- \( t_p \) is the thickness of the stiffener plate,
- \( F_y \) is the specified minimum yield strength of the effective column section,
- \( k \) is the plate buckling coefficient as defined in the AASHTO LRFD specifications (AASHTO 1998), which is given in Table 3.1,
- \( A \) is the area of the effective column section,
- \( E \) is the modulus of elasticity,
- \( L \) is the unbraced length of the effective column sections which equals the web height,
- \( K \) is the effective length factor,
- \( r \) is the radius of gyration about the axis perpendicular to the buckling plane of the effective column section.

The effective length factor in the AASHTO LRFD specifications (AASHTO 1998) is 0.75. However, for this study, the boundary conditions of the bearing stiffener are unknown. As shown in Table 3.1, \( K \) is taken as 0.75 or 1.0 to investigate the effect of \( K \) on \( P_n \).
The AASHTO column formula is applicable to an effective column section that has a uniform \( F_y \). However for test specimen G2A, the specified minimum yield strength of the web and the stiffener plates are 485 MPa and 345 MPa, respectively. Eq. 3.3 was modified as shown in Eq 3.5 to accommodate the different yield strengths of the stiffener and the web of the effective column section and, thus, provide a better estimate of the axial resistance.

\[
P_{nAC} = 0.66 \cdot \left[ \left( F_{yw} A_w + F_{ys} A_s \right) - \left( \frac{F_{yw} A_w + F_{ys} A_s - F_{ys} A}{2.25} \right) \right] \text{ if } \lambda \leq 2.25
\]

\[
P_{nAC} = \frac{0.88 \cdot F_y \cdot A}{\lambda} \text{ if } \lambda > 2.25
\]

\[\text{Eq. 3.5}\]

where:

- \( F_{yw} \) is the specified minimum yield strength of the web of the effective column section,
- \( A_w \) is the area of the web of the effective column section,
- \( A_s \) is the area of the stiffener plates of the effective column section.

The radius of gyration, \( r \), for the effective column section is determined from Eq. 3.6:

\[
r = \sqrt{\frac{I}{A}}
\]

\[\text{Eq. 3.6}\]

where:

- \( I \) is the moment of inertia of the effective column section. For conventional flat web I-girders, \( I \) is \( I_x \), the moment of inertia taken about the x-axis shown in Figure 3.2. This axis is the weak axis for the stiffener buckling, when the presence of the web is considered. For corrugated web I-girders, the axis about which buckling will occur may
not be either of the orthogonal axes (x or y) shown in Figure 3.4. For corrugated web I-girders, I is taken as $I_{\text{min}}$, the minimum moment of inertia for the effective column section. $I_{\text{min}}$ is determined from Eq. 3.7. The weak axis about which buckling of the section will occur is represented by the angle $\theta$, determined from Eq. 3.8 (measured in degrees from the y-axis in a clockwise direction).

$$I_{\text{min}} = \frac{I_x + I_y}{2} - \sqrt{\left(\frac{I_x - I_y}{2}\right)^2 + I_{xy}} \quad (3.7)$$

$$\theta = \tan^{-1}\left(\frac{-2 \cdot I_{xy}}{I_x - I_y}\right) \quad (3.8)$$

for which:

$$I_x = \frac{2}{3} t_p b_t^3 + \frac{\gamma^2 t_w^4 (\sin(\alpha))^2}{12} \quad (3.9)$$

$$I_y = \frac{1}{6} t_p b_t^3 + \frac{\gamma^2 t_w^4 (\cos(\alpha))^2}{12} \quad (3.10)$$

$$I_{xy} = -\frac{\gamma^3 t_w^4 \sin(2\alpha)^2}{24} \quad (3.11)$$

where:

$\alpha$ is the angle of the inclined fold with respect to the x-axis as shown in Figure 3.4,

$I_x$ is the moment of inertia about the x-axis of the effective column section,

$I_y$ is the moment of inertia about the y-axis of the effective column section,

$I_{xy}$ is the product of inertia of the effective column section.
The nominal axial resistance of a bearing stiffener of a corrugated web I-girder determined from the Euler buckling column formula, \( P_{nEC} \), is based on the assumption that the effective column section behaves as an ideal column in which all fibers remain elastic until buckling occurs. It is assumed that \( \lambda = 2.25 \) represents the limit of the application of the Euler formula. For \( \lambda < 2.25 \), the column becomes inelastic and the formula is no longer valid. The nominal axial resistance from the Euler column buckling formula is:

\[
P_{nEC} = \frac{\pi^2 \cdot E \cdot I_{\text{min}}}{(K \cdot L^2)} \quad (3.12)
\]

The nominal axial resistance of a bearing stiffener of a corrugated web I-girder determined from the yield capacity, \( P_{nYC} \), is based on the assumption that the bearing stiffener has developed its full plastic capacity.

\[
P_{nYC} = A_t F_{ys} + A_w F_{yw} \quad (3.13)
\]

Assumption 2 is that the total length of the web contribution, \( \gamma t_w \), is greater than 168 mm but less than 450 mm. The web geometry used to calculate the geometric properties of the effective column, is shown in Figure 3.5. Note that the large bend radius, shown in Figure 3.4, was ignored. The effective column section is shown in Figure 3.5 and Figure 3.6. Based on Assumption 2, the largest value of \( \gamma \), \( \gamma_{\text{max}} \), is approximately 75, where \( \gamma_{\text{max}} = \gamma_1 + 2 \gamma_2 \). The nominal axial resistance based on Assumption 2 was calculated as discussed in the preceding section using the AASHTO LRFD column formula, the Euler column buckling formula, and the plastic capacity.

The effective column section based on Assumption 2 differs from the effective column section based on Assumption 1. Equations 3.3, 3.4, 3.5, 3.6, 3.7, 3.8, 3.12, 3.13
and 3.14 given above are valid for this effective column section. Eq. 3.9, 3.10, and 3.11 are modified as follows:

\[
I_x = \frac{2}{3} t_p b_t^3 + \frac{3}{100} \gamma_1 t_w^3 + \frac{1}{6} \gamma_2 t_w^3 + \frac{1}{8} \gamma_2 t_w d^2 \tag{3.14}
\]

\[
I_y = \frac{1}{6} t_p b_t^3 + \frac{4}{75} \gamma_1 t_w^3 + \frac{1}{6} \gamma_2 t_w^3 + \frac{2}{25} \gamma_1 \gamma_2 t_w^2 + \frac{1}{5} \gamma_2 \gamma_1 t_w + \frac{1}{8} \gamma_2 t_w \tag{3.15}
\]

\[
I_{xy} = -\frac{1}{25} \gamma_1 t_w \tag{3.16}
\]

**Calculated Axial Resistance**

Table 3.1 shows the values for the variables introduced in the preceding section that were used to analyze the test specimen. Table 3.2 and 3.3 and Figures 3.7 to 3.10 show the effect of varying K and \( \gamma t_w \) on \( P_n \) of the bearing stiffener of test specimen G2A. Though the AASHTO LRFD specifications (AASHTO 1998) indicate that K should be taken as 0.75, the results are presented for values of K equal to 0.75 and 1.0. In the tables and figures, the subscripts AC, EC, and YC refer to results from Eq. 3.5, 3.12 and 3.13 for corrugated web I-girders respectively. Also, note that for \( \gamma t_w \) less than and equal to 168 mm, the results are based on Assumption 1. For \( \gamma t_w \) greater than or equal to 170 mm, the results based Assumption 2.

The nominal axial resistance, \( P_n \), of a bearing stiffener with identical parameters, but attached to a flat web I-girder is also shown in Tables 3.2 and 3.3. These results, labeled as \( P_{nAF} \), were calculated using the effective column section formula for a conventional flat web I-girder in accordance with the AASHTO LRFD specifications (AASHTO 1998).
Results from the Euler column buckling formula and the plastic capacity were not calculated for the flat web I-girder. Therefore, only the $P_{nAF}$ results are used as a basis for comparing the strength of bearing stiffeners for flat web I-girders and corrugated web I-girders.

$P_{nAC}$ and $P_{nAF}$ are both calculated using the AASHTO effective column formula. A comparison of $P_{nAC}$ and $P_{nAF}$ in Tables 3.2 and 3.3 shows that the calculated strength of bearing stiffeners for a corrugated web I-girder, $P_{nAC}$, will always be less than $P_{nAF}$, regardless of the assumed $\gamma tw$ and $K$ when only values obtained for $P_{nAF}$ with a $\gamma tw \leq 108$ are considered. Values not considered are shown shaded in Tables 3.2 and 3.3.

Comparing $P_{nEC}$ and $P_{nAC}$, $P_{nEC}$ is greater than $P_{nAC}$. However, the Euler buckling formula is not considered valid for values of $\lambda$ less than 2.25. Values obtained for $P_{nEC}$ with a $\lambda$ less than 2.25, shown shaded in Tables 3.2 and 3.3, are not considered further. $P_{nYC}$ is calculated assuming the bearing stiffener develops its full plastic strength, therefore, $P_{nYC}$ exceeds $P_{nAC}$.

Plots of the normalized $P_n$ versus $\gamma tw$ are shown in Figures 3.7 and 3.8. In the plots, $P_{nAC}$, $P_{nEC}$ and $P_{nAF}$ are normalized by $P_{nYC}$. The figures indicate how closely each calculated $P_n$ approaches the plastic capacity of the bearing stiffener for a given $\gamma tw$. As $\gamma tw$ increases, $P_{nAC}$ approaches $P_{nYC}$. From these values it can be seen that if $\gamma tw$ is large, $P_n$ can be approximated by $P_{nYC}$. Plots of $P_{nAC}$, $P_{nEC}$, $P_{nYC}$ and $P_{nAF}$ vs. $\gamma tw$ are shown in Figure 3.9 and 3.10 for $K = 0.75$ and $K = 1.0$. Each calculated $P_n$ is greater when $K = 0.75$ compared to $K = 1.0$, at a given $\gamma tw$. As $\gamma tw$ increases, each calculated $P_n$ increases.
Values of $\theta$ of Tables 3.3 and 3.4, gives the angle the axis of buckling makes with the $y$-axis, measured in the clockwise direction. It is observed that this angle increases as $\gamma t_w$ increases.

The results of the bearing stiffener compression tests presented in the following sections will be analyzed in light of the calculated results presented in this section.

### 3.3 TEST SET-UP, INSTRUMENTATION AND PROCEDURE

This section discusses the test set-up, instrumentation and test procedure for the bearing stiffener tests.

#### 3.3.1 TEST SET-UP

Each bearing stiffener of fatigue test specimen, G2A, was tested in compression (Compression Test 1) in the SATEC universal testing machine with a capacity of 2670 kN at the ATLSS Center, Lehigh University. Later, each bearing stiffener was again tested (Compression Test 2) in the 22,240 kN Baldwin universal testing machine in Fritz Engineering Laboratory, Lehigh University. The bearing stiffeners, Test Specimen 1 and Test Specimen 2 were cut from the west and the east end of test specimen G2A, respectively. Test Specimens 1 and 2 are shown in Figure 3.11 and 3.12.

In Compression Test 1, the compressive load was applied using a 254 mm diameter hardened swivel head placed concentric to the bearing stiffener of each test specimen, as shown in Figure 3.13. The flanges were braced by 127 mm by 127 mm by 45 mm angles welded to a 19 mm and 25 mm thick plates at the base and top, respectively. For Compression Test 2, the same test set-up was used under the 22,240 kN Baldwin universal testing machine as in Compression Test 1.
3.3.2 INSTRUMENTATION

Instrumentation was used to provide data on the load, vertical deflection, the out-of-plane deflections of the stiffener and web, and the strains on the web and stiffener at the neutral axis. The load was measured by the test machine. The vertical deflection was measured using displacement transducers placed on both stiffener plates and the south end of the web plate of Test Specimen 1. Displacement transducers were placed on both stiffener plates and both ends of the web of Test Specimen 2. The locations of the displacement transducers are shown in Figure 3.14 (a), (b) and Figure 3.15 (a), (b).

Strain gages were used to measure the stiffener and web strains. Uniaxial strain gages were positioned at the mid height of the web and the stiffener. Strain gages were placed on both sides of the stiffener plates of Test Specimen 1, and this strain data was recorded for Compression Test 1 but not for Compression Test 2. Strain gages were placed on both sides of the stiffener plates and both sides of each end of the web of Test Specimen 2. This strain data was recorded for both compression tests. When the strain gages were placed on both sides of the plate, they were placed back-to-back. The locations of the strain gages are shown in Figure 3.14 (c), (d) and 3.15 (c), (d).

3.3.3 PROCEDURE

For each test specimen, it was initially thought that failure would occur before the 2670 kN capacity of the SATEC universal testing machine was reached (during Compression Test 1). The values obtained in Table 3.2 and Table 3.3 for $P_{nAC}$ and $P_{nYC}$
show that the 2670 kN capacity is exceeded only by $P_{nYC}$, when $\gamma_t w$ is 450 mm (essentially the full length of the specimen). When failure of the specimens did not occur before 2670 kN, testing to failure was completed using the 22,240 kN Baldwin universal testing machine. Therefore, two compression tests were done per test specimen. Each compression test involved two stages: initial loading; and loading up to the test machine capacity or up to failure. The initial loading stage involved three elastic cycles to 445 kN. The cycles were for alignment and seating of the test set-up and to check instrumentation. For the second stage, the instrument readings were taken at a load of zero and the specimen was statically loaded.

**Test Specimen 1**

Three attempts were made to fail Test Specimen 1 in the SATEC universal testing machine. For each attempt during Compression Test 1, the test specimen had an initial load rate of 95 kN/min up to 445 kN. Thereafter, a displacement rate of 0.25 mm/min was applied to the test specimen. The three attempts were made to fail the specimen by increasing the load until the capacity of the test machine was reached and, then, unloading to 0 load. Only the results from the first attempt are shown in this report. Compression Test 2 involved loading to failure with the same load and displacement rates using the 22,240 kN Baldwin universal testing machine.
**Test Specimen 2**

Compression Tests 1 and 2 involved loading to failure at an initial load rate of 95 kN/min up to 445 kN, and thereafter at a displacement rate of 0.25 mm/min up to approximately 2670 kN and, then, unloading to 0 load.

During the loading, the stiffener and web were inspected for out of plane deflections. Strain gage and displacement transducer readings were taken every 2 seconds.

### 3.4 TEST RESULTS AND ANALYSIS

The objective of the bearing stiffener compression tests was to determine their axial force capacity. The test specimens were designed to satisfy the slenderness requirements of the AASHTO LRFD specifications (AASHTO 1998). This ensured that the specimen developed its yield strength before the onset of local buckling of the stiffeners. Therefore, it was expected that the test specimens would fail by combined buckling of the stiffener plates and the web plate along their weak buckling axis.

The results from the compression tests of the two bearing stiffener specimens are presented and discussed in the following sections. The results of the first attempt of Compression Test 1 and the results of Compression Test 2 are presented for Test Specimen 1. Similar results (Compression Test 1 and Compression Test 2) are presented for Test Specimen 2.
3.4.1 TEST RESULTS

Test Specimen 1

The load, deflections and web and stiffener strains recorded during both compression tests are presented briefly in this section. Figures 3.16 and 3.17 show load vs. cross head travel of Test Specimen 1 for Compression Test 1 and Compression Test 2, respectively. Figures 3.19 to 3.21 show load vs. vertical deflection of the west and east stiffener plates, and load vs. vertical deflection of the south end of the web plate for Compression Tests 1 and 2. Figure 3.22 plots load vs. strain measured at the mid height of the stiffeners for Compression Test 1.

Test Specimen 2

The load, deflections and web and stiffener strains recorded during both compression tests are presented in this section. Figure 3.25 shows load vs. vertical deflection of Test Specimen 2. This figure was created by assuming that the unloading curve for Compression Test 1 can be represented by the loading curve of Compression Test 2. As a result, the load curve of Compression Test 2 was offset to represent this assumption. Figures 3.27 to 3.28 show load vs. deflection of the east and the west end stiffeners, and load vs. vertical deflection of the north and the south ends of the web plate. Figures 3.29 to 3.30 plot load vs. strain measured at the mid height of the web and stiffeners.
3.4.2 ANALYSIS OF TEST RESULTS

The results obtained from the compression tests conducted on Test Specimens 1 and 2 will be discussed in this section.

Test Specimen 1

The overall load-deflection plots of the test specimen, Figures 3.16 and 3.17, show that the test specimen experiences some seating in the initial loading stages, then exhibits linear elastic behavior up to approximately 1500 kN. At this point, the test specimen shows an initial reduction in stiffness. The ultimate load, \( P_u \), was 2915 kN (Figure 3.17), which is much larger than the results for \( P_n \) given in Table 3.2 and Table 3.3. The value of \( P_n \) closest to \( P_u \) is \( P_{nY}=2682 \) kN (\( \gamma_{tw}=450 \) mm and \( K=0.75 \)). The maximum calculated \( P_n \) is 8.0% less than \( P_u \).

Figure 3.23 shows plots of the average strain of each bearing stiffener plate. The theoretical yield strain of the stiffener plates is 1725 micro strain. The average strain reached this strain level at 2025 kN and 2168 kN for the west and the east bearing stiffener plates, respectively (see Figure 3.23), suggesting that yielding occurred at this load level. However, this assumes an initially perfect stiffener plate up to theoretical yield. Residual stresses and imperfections in the flatness of the stiffener plate and the web will cause nonlinear behavior to occur prior to theoretical yield. Both stiffener plates exhibit nonlinear behavior before theoretical yield. Plots of the strains on the surface of each stiffener plate are shown in Figure 3.22. The loads at which the measured strain reaches the yield strain for the north and the south faces of the west stiffener plate are 2277 kN and 1752 kN, respectively. The loads at which the measured
strain reaches the yield strain for the north and the south faces of the east stiffener plate are 2189 kN and 2235 kN, respectively. Therefore, initial yield of the west stiffener plate occurred at a load of 1752 kN, while initial yield of the east stiffener plate occurred at a load of 2189 kN.

Out-of-plane deformation in the stiffener plates is indicated by the difference in strain (strain separation) between the gages on opposite faces of each stiffener plate. Visible strain separation occurred at approximately 1400 kN and 1371 kN for the west and east stiffener plates, respectively (see Figure 3.22 (a) and (b)). At initial yield of the west and east stiffener plates, the strain separations were 622 and 224 micro strain, respectively. This indicates that the stiffener plates began to bend before initial yielding of the plates occurred. The bending of the stiffener plates can be attributed to growth of initial imperfections in the plate flatness, and to the softening of the plates from yielding of regions with residual compressive stresses.

Figure 3.24 shows plots of curvature vs. load for the bearing stiffener plates. The curvature of the stiffener plates is calculated by subtracting the strain of the south face from the strain on the north face and dividing the result by the plate thickness. At initial yield, curvature magnitudes of 31.1 mm\(^{-1}\) and -11.2 mm\(^{-1}\) were recorded for the west and east stiffener plates, respectively. These values indicate the west stiffener plate was deflecting out of plane, concave toward the north and the east plate was deflecting concave towards the south. Evidence of this is seen in Figure 3.24.

The bending of the stiffener plates relative to each other (i.e. bending of the effective column) is seen clearly in a plot comparing the average strains in the west and east stiffener plates (Figure 3.23). This plot shows evidence of bending when the
separation of the average strains occurs. Bending begins soon after a load of 1650 kN is applied. The west stiffener plate has greater compression strain than the east stiffener plate.

Permanent vertical deflection of the stiffener plates occurred during both tests (Figure 3.19 and 3.20). At initial yield (during the first attempt to fail Test Specimen 1 in Compression Test 1), the vertical deflections of the west and east stiffener plates were 1.5 mm and 2.2 mm, respectively. At \( P_u \), a deflection of 6.9 mm and 6.0 mm was observed in the west and east stiffener plates, respectively. Permanent vertical deflection of the south end of the web was also observed (Figure 3.21). When the first stiffener plate experienced initial yield (i.e., the west plate), the vertical deflection of south end of the web plate was 0.6 mm. At \( P_u \), a deflection of 9.6 mm was observed for the south end of the web plate.

The vertical deflection of the web when the first (west) stiffener plate experienced initial yield is small compared with the corresponding deflections of the stiffener plates. This is attributed to the higher yield strength of the web, and the fact that the area of the web where the deflection was measured was not directly under the bearing plates, while, the stiffeners were (see Figure 3.13). However, at the peak load for Compression Test 2, the web vertical deflection was larger than that of the stiffeners.
Test Specimen 2

The overall load-deflection plot of the test specimen shows that the test specimen experiences some seating in the initial loading stages, then exhibits linear elastic behavior up to approximately 1800 kN. At this point the test specimen shows a small change in stiffness. The ultimate load, $P_u$, was 2695 kN (Figure 3.25), which is much larger than the results for $P_n$ given in Table 3.2 and Table 3.3. The value of $P_n$ closest to $P_u$ is $P_{nYc} = 2682$ kN ($\gamma t_w = 450$ mm and $K = 0.75$). The maximum calculated $P_n$ is 0.4% less than $P_u$.

Figure 3.31 shows plots of the average strain of each bearing stiffener plate for both compression tests. The plots show that theoretical yielding of the bearing stiffener plates occurred at 1829 kN and 1862 kN for the west and the east bearing stiffener plates, respectively (see Figure 3.31 (a)). Both stiffener plates had begun to exhibit a small degree of nonlinear behavior before theoretical yield. The loads at which the measured surface strain reaches the yield strain for the north and the south faces of the west stiffener plate are 1726 kN and 2271 kN, respectively. The load at which the measured surface strain reaches the yield strain for the north and the south faces of the east stiffener plate are 1873 kN and 1854 kN, respectively, therefore, initial yield of the west stiffener plate occurred at a load of 1726 kN, while, initial yield of the east stiffener plate occurred at a load of 1854 kN.

Strain separation occurred at 347 kN for the west stiffener plate during Compression Test 1 (see Figure 3.29 (a)). Strain separation at such a low value of load is attributed to the presence of significant imperfections in the flatness of the stiffener plate. Strain separation of magnitude of 414 micro strain was recorded at initial yield.
and 1824 micro strain at the peak load. For Compression Test 2, separation of the strains for the west stiffener plate is seen soon after load is applied to the specimen (Figure 3.29 (b)), as a result of the out-of-plane deformations from Compression Test 1. Only very small strain separation was initially observed for the east stiffener plate at a load value of 1800 kN (Figure 3.29 (c)) for Compression Test 1. Strain separation at initial yield was 77 micro strain.

Figure 3.33 shows a plot of curvature vs. load for the bearing stiffener plates. The magnitude of curvature recorded at initial yield for Compression Test 1 was 20.7 mm⁻¹ for the west stiffener plate and 3.8 mm⁻¹ for the east stiffener plate. These values indicate the west and east stiffener plates were deflecting out-of-plane concave toward the north and that the curvature of the west plate was greater that of the east stiffener plate, (see Figure 3.33 (a)).

The bending of stiffener plates relative to each other is seen clearly in a plot comparing the average strains in the west and east stiffener plates (Figure 3.31 (a)). This plot shows evidence of relative bending when the separation of the average strains occurs. Relative bending begins soon after a load of 1500 kN is reached. The strains in the west stiffener plate continue to increase relative to those of the east stiffener plate. The curvature in the east stiffener plate was observed to be small (Figure 3.33 (a)) and that means little out of plane deflection was occurring. Therefore, unlike the west stiffener, compression rather than bending controls the behavior of the plate, and the east stiffener plate appears to be stiffer (with less average strain).

After yielding of the stiffener plates and as the out of plane deflections become significant, the stiffness of the test specimen decreases. However, the load continues to
increase up to the ultimate load, $P_u$, of 2695 kN. The loss in load resistance beyond the peak in related to distortion of the web. Figure 3.30 shows load vs. strain in the web plate at both ends. For Compression Test 1, the strains at the north end of the web plate do not separate significantly until just before the peak load. This strain separation repeats for Compression Test 2. For the south end of the web plate there is little strain separation during Compression Test 1, but significant strain separation near the peak load of Compression Test 2. Near the peak load, the strain separation becomes sudden and the load begins to decrease.

The theoretical yield strain of the web plate is 2414 micro strain. Figure 3.32 shows plots of the average strain of each end of the web plate for both compression tests. The average strain plot for the north end of the web plate (Figure 3.32 (a)) indicates that it did not yield for Compression Test 1. The average strain plot for the south end of the web plate reaches 2414 micro strain, suggesting that yielding occurred during Compression Test 1, at a load value of 2228 kN. The south end of the web plate had begun to exhibit a small degree of nonlinear behavior before theoretical yield which can be attributed to residual stresses, plate flatness and imperfections. Based on surface strain gages (Figure 3.30), the loads at which theoretical yielding of the west and east faces of the south end of the web plate were observed are 2262 kN and 2197 kN, respectively. Therefore, initial yield of the south end of the web plate occurred at a load of 2197 kN. At initial yield of the south end of the web plate, the stiffeners had already yielded and had experienced strain separations of 1610 micro strain (west stiffener) and 47 micro strain (east stiffener).
It should be noted that in Figure 3.30 (b) where web strains are plotted for Compression Test 2, the east face (in compression) of the north end of the web plate had yielded, while, the west face (in tension) did not. The east face of the north end of the web plate reached the yield strain at a load of 2667 kN. This is very close to \( P_u = 2695 \) kN, suggesting that when initial yielding of the north end of the web plate occurs, the ultimate strength was reached (the south end of the web and the stiffeners had already yielded).

During Compression Test 1, strain separation for the north end of the web plate (see Figure 3.30) was seen at a load of approximately 1800 kN. At a load of 2460 kN, significant strain separation occurred, indicating the formation of a local buckle during Compression Test 1. The strain separation for the north end of the web plate at the peak load for Compression Test 1 (\( P = 2663 \) kN) was 1176 micro strain. Strain separation for the south end of the web plate was seen at a load of approximately 1000 kN. The magnitude of strain separation for the south end, at initial yield was 167 micro strain. At the peak load for Compression Test 1, the strain separation was 1033 micro strain. The north and south end of the web plate began to deflect out-of-plane before yielding of the each plate occurred. Out-of-plane deflections of the web plate can be attributed to the growth of initial plate flatness imperfections and the softening of the plates from yielding of regions with residual compressive stresses.

Figure 3.34 shows a plot of curvature vs. load for each end of the web plate. The sharp increase in curvature for the north end of the web plate correlates with the load at which the local buckle forms. The plots of curvature indicate both ends of the web were
deflecting out of plane, concave toward the west. Evidence of this is seen in Figure 3.34 (a).

The bending of both ends of the web relative to each other is seen clearly in a plot comparing the average strains (Figure 3.32). This plot shows evidence of relative bending when the separation of the average strains occurs. Relative bending begins soon after application of load for Compression Test 1.

For Compression Test 1, curvature of the web grew more rapidly than curvature of the stiffener plates, though yielding of the web occurred after yielding of both stiffener plates (compare Figure 3.33 and Figure 3.34). This can be attributed to the slenderness of the web. At $P_u$, the curvature of the web was greater than that of the stiffener plates.

permanent vertical deflection of the stiffener plates occurred (Figure 3.27) during both tests. For Compression Test 1, at initial yield, the vertical displacements of the west and east stiffener plates were 0.3 mm and 2.0 mm; at peak load, the deflections were 2.0 mm and 7.6 mm, respectively. Permanent vertical deflection of the north and south end of the web was also observed (Figure 3.28). For Compression Test 1, at the peak load the vertical deflection of the north end of the web plate was 1.3 mm. At initial yield, the deflection of the south end of the web plate was 2.1 mm; at peak load, the deflection was 4.0 mm.
3.5 SUMMARY AND CONCLUSION

$P_n$ obtained for Test Specimen 1 was 7.6% greater than that obtained for Test Specimen 2. The major difference in Test Specimen 1 and Test Specimen 2 is the amount of web that was included on the north end of the specimen. The total length of web included in Test Specimen 1 was twice the length included in Test Specimen 2, however, the increase in $P_u$ was not proportional to the increased length of the web. Therefore, it can be assumed that not all the web contributed to the ultimate strength of the bearing stiffener of Test Specimen 1.

The maximum calculated $P_n$ (i.e., $P_n = P_{nYC} = 2682$ kN) using a $\gamma_{tw}$ of 450 mm is 8.0% less than $P_u$ for Test Specimen 1 and 0.4% less than $P_u$ for Test Specimen 2. Therefore, it can be concluded that the ultimate strength of a bearing stiffener of a corrugated web I-girder, $P_u$, can be approximated by the full plastic capacity of the section (Eq. 3.13). The effective column section can be assumed to include the stiffener plates and a length of web equal to $75t_w$ (i.e. $\gamma = 75$) that is centered on the stiffener plates.

As discussed in Section 3.2.2, the nominal axial resistance of a bearing stiffener centered on the inclined fold of a corrugated web I-girder can be calculated from column strength equations (Eq. 3.3). The minimum radius of gyration should be used, as discussed in Section 3.2.2, and the appropriate effective length factor, $K$, is 0.75. Based on the test results of Test Specimen 1 and Test Specimen 2, the nominal axial resistance based on Eq. 3.3 calculated with a web contribution of $24t_w$ (i.e. $\gamma = 24$) is approximately equal to the limit of linear behavior of a bearing stiffener. This limit of
linear behavior is approximately 50% of the ultimate strength, where the ultimate
strength is calculated from the plastic capacity.

The AASHTO design specifications (AASHTO 1998) states that the nominal
axial resistance a bearing stiffener of a flat web I-girder should be calculated with a web
contribution of $18t_w$ (i.e. $\gamma = 18$) centered on the stiffener plates. The test results of Test
Specimen 1 and Test Specimen 2 show that the nominal axial resistance of a bearing
stiffener of a corrugated web I-girder with the bearing stiffeners centered on the
inclined fold can be calculated with a web contribution of $24t_w$ (i.e. $\gamma = 24$). As a result,
the nominal axial resistance of a bearing stiffener of a corrugated web I-girder will be
greater than that of a conventional flat web I-girder with identical stiffener parameters.
Table 3.1 Bearing Stiffener Variables Defined for G2A

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<td>$F_{yw}$</td>
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Table 3.2 Effect of $\gamma_{tw}$ on $P_{nAC}$, $P_{nEC}$, $P_{nYC}$ and $P_{nAF}$ for $K = 0.75$

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<th>$K$</th>
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<th>$\theta$ (°)</th>
<th>$\lambda_C$</th>
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<th>$P_{nEC}$ (kN)</th>
<th>$P_{nYC}$ (kN)</th>
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* $\gamma = 18$

** $\gamma = 75$
Table 3.3 Effect of $\gamma_t w$ on $P_{nAC}$, $P_{nEC}$, $P_{nYC}$ and $P_{nAF}$ for $K = 1.0$

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<th>$\gamma_t w$ (mm)</th>
<th>$\theta(\degree)$</th>
<th>$\lambda_C$</th>
<th>$\lambda_F$</th>
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<th>$P_{nYC}$ (kN)</th>
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* $\gamma = 18$

** $\gamma = 75$
Figure 3.1 Bearing Stiffener For Test Specimen G2A (units in mm): (a) Corrugated Web I-girder, (b) Section Showing Bearing Stiffener, (c) Individual Bearing Stiffener Plate
Figure 3.2 Bearing Stiffener for Conventional Flat Web I-girder
Figure 3.3 Bearing Stiffener for Test Specimen G2A (units in mm)
Figure 3.4 Variables for Assumption 1
Figure 3.5 Equivalent Column Section (units in mm) for Assumption 2

Figure 3.6 Variables for Assumption 2

\[ \gamma = \gamma_1 + 2\gamma_2 \]
Figure 3.7 Effect of Web Contribution on Normalized $P_n$ for $K=0.75$

Figure 3.8 Effect of Web Contribution on Normalized $P_n$ for $K=1.0$
Figure 3.9 Effect of web contribution on $P_n$ for $K=0.75$

Figure 3.10 Effect of web contribution on $P_n$ for $K=1.0$
Figure 3.11 Bearing Stiffener Test Specimen 1 (units in mm)

Figure 3.12 Bearing Stiffener Test Specimen 2 (units in mm)
West ~ SATEC - machine head

- swivel head Φ = 254
- steel PL 508 x 508 x 25.4
- steel plate (typ.) 229 x 229 x 25.4
- bearing stiffener 100 x 20 x 1200
- angles (typ.) 127 x 127 x 45
- hydrostone (typ.) steel PL 915 x 813 x 19

(a) South Elevation of Test Set-Up for Test Specimen 1 and 2 (units in mm)

Figure 3.13 Test Set-Up for Bearing Stiffeners of G2A
(b) East Elevation of Test Set-Up for Test Specimen 1 (units in mm)

Figure 3.13 Test Set-Up for Bearing Stiffeners of G2A (cont.)
Figure 3.13 Test Set-Up for Bearing Stiffeners of G2A (cont.)

(c) East Elevation of Test Set-Up for Test Specimen 2 (units in mm)
(d) Photograph of Set-Up of Bearing Stiffener Test Specimen

Figure 3.13 Test Set-Up for Bearing Stiffeners of G2A (cont.)
(d) Photograph of Set-Up of Bearing Stiffener Test Specimen

Figure 3.13 Test Set-Up for Bearing Stiffeners of G2A (cont.)
(a) East Elevation of Displacement Transducer Layout for Test Specimen 1 (units in mm)

Figure 3.14 Instrumentation for Test Specimen 1
Figure 3.14 Instrumentation for Test Specimen 1 (cont.)

(b) South Elevation of Displacement Transducers Layout for Test Specimen 1 (units in mm)
(c) East Elevation of Strain Gage Layout for Test Specimen 1 (units in mm)

Figure 3.14 Instrumentation for Test Specimen 1 (cont.)
(d) South Elevation of Strain Gage Layout for Test Specimen 1 (units in mm)

Figure 3.14 Instrumentation for Test Specimen 1 (cont.)
(a) East Elevation of Displacement Transducer Layout for Test Specimen 2 (units in mm)

Figure 3.15 Instrumentation for Test Specimen 2
(b) South Elevation of Displacement Transducer Layout for Test Specimen 2 (units in mm)

Figure 3.15 Instrumentation for Test Specimen 2 (cont.)
(c) East Elevation of Strain Gage Layout for Test Specimen 2 (units in mm)

Figure 3.15 Instrumentation for Test Specimen 2 (cont.)
(d) South Elevation of Strain Gage Layout for Test Specimen 2 (units in mm)

Figure 3.15 Instrumentation for Test Specimen 2 (cont.)
Figure 3.16 Load vs. Cross Head Displacement for Test Specimen 1 from Compression Test 1
Figure 3.17 Load vs. Cross Head Displacement for Test Specimen 1 from Compression Test 2
Figure 3.18 Test Specimen 1 After Compression Test 2
Figure 3.18 Test Specimen 1 After Compression Test 2
Figure 3.19 Vertical Deflection of West Stiffener Plate

(a) Compression Test 1

(b) Compression Test 2
Figure 3.20 Vertical Deflection of East Stiffener Plate
Figure 3.21 Vertical Deflection of South End of Web Plate

(a) Compression Test 1

(b) Compression Test 2
Figure 3.22 Strain in West and East Stiffener Plates

(a) Compression Test 1 - West Stiffener Plate

(b) Compression Test 1 - East Stiffener Plate
Figure 3.23 Compression Test 1 - Average Strain in West and East Stiffener Plates

Figure 3.24 Compression Test 1 - Curvature of West and East Stiffener Plates
Figure 3.25 Compression Tests 1 and 2 - Load vs. Cross Head Displacement of Test Specimen 2
Figure 3.26 Test Specimen 2 After Compression Test 2
Figure 3.26 Test Specimen 2 After Compression Test 2
Figure 3.27 Vertical Deflection of West and East Stiffener Plates

(a) Compression Test 1 - West Stiffener Plate

(b) Compression Test 2 - West Stiffener Plate
Figure 3.27 Vertical Deflection of West and East Stiffener Plates (cont.)

(c) Compression Test 1 - East Stiffener Plate

(d) Compression Test 2 - East Stiffener Plate
Figure 3.28 Vertical Deflection of North and South End of Web Plates

(a) Compression Test 1 - North End of Web Plate

(b) Compression Test 2 - North End of Web Plate
Figure 3.28 Vertical Deflection of North and South End of Web Plates (cont.)

(c) Compression Test 1 - South End of Web Plate

(d) Compression Test 2 - South End of Web Plate
(a) Compression Test 1 - West Stiffener Plate

(b) Compression Test 2 - West Stiffener Plate

Figure 3.29 Strains of West and East Stiffener Plates
(c) Compression Test 1 - East Stiffener Plate

(d) Compression Test 2 - East Stiffener Plate

Figure 3.29 Strains of West and East Stiffener Plates (cont.)
(a) Compression Test 1 - North End of Web Plate

(b) Compression Test 2 - North End of Web Plate

Figure 3.30 Strains of North and South End of Web Plates
Figure 3.30 Strains of North and South End of Web Plates (cont.)

(c) Compression Test 1 - South End of Web Plate

(d) Compression Test 2 - South End of Web Plate
Figure 3.31 Average Strain of West and East Stiffener Plates

(a) Compression Test 1

(b) Compression Test 2
Figure 3.32 Average Strain of North and South End of Web Plate
Figure 3.33 Curvature of East and West Stiffener Plates
(a) Compression Test 1

(b) Compression Test 2 – Web Plate

Figure 3.34 Curvature of North and South End of Web Plate
CHAPTER 4 - FLANGE AND WEB SPLICES FOR CORRUGATED WEB I-GIRDERS

4.1 INTRODUCTION

Previous research at Lehigh University has addressed the shear strength and the flexural strength of corrugated web I-girders. However, the important aspects of bolted field splices of corrugated web I-girders have not been studied. This chapter investigates the behavior of bolted flange and web splices for corrugated web bridge I-girders with emphasis on girders fabricated from HPS.

In this study, it is assumed that the bending is carried by the flanges, while, the shear is carried by the web (Abbas 2003). To adequately study the behavior of bolted flange and web splices, experimentally, separate tests were conducted. The experimental program had only enough corrugated web I-girder material (as discussed below) to make one specimen. The two tests were conducted sequentially using this test specimen. In the first test, the highest possible bending moment was generated in the flanges at the splice location (the Flexural Strength Test). In the second test, a large shear develops in the web at the splice location (the Shear Strength Test). The Flexural Strength Test (FST) and the Shear Strength Test (SST) were designed to investigate the performance of the flange splice and the web splice, respectively.

The test specimen for the FST and SST was created from two girders, G7A and G8A, tested previously in the shear tests described in Section 2.4. Figure 4.1 shows approximately the initial and subsequent failure regions of G7A and G8A. Only one test specimen could be created from the regions of G7A and G8A that had not failed during the shear tests. The longest possible test specimen had to be created in order to generate
the highest possible bending moment for the FST. To achieving proper alignment of the webs at the splice, the girders were cut as shown in Figure 4.2. The test specimen was created from splicing together girder sections, G7Ar and G8Ar, shown in Figure 4.2.

The test specimen was used for both the FST and the SST. Therefore, the FST had to be completed without structurally damaging the two test specimen components G7Ar and G8Ar. The critical limit states to avoid during the FST were shear failure of the web and flexural failure of the flanges. As a result, neither the nominal shear strength \( V_n \) nor nominal flexural strength \( M_{nf} \) of the test specimen could be exceeded during the FST.

4.2. TEST CONFIGURATION FOR THE FLEXURAL STRENGTH TEST

4.2.1 OBJECTIVE OF THE TEST

The objective of the Flexural Strength Test (FST) was to experimentally investigate the behavior of bolted flange splices designed for corrugated web I-girders. To achieve this objective, the largest possible moment had to be developed in the flanges at the location of the splice without exceeding the nominal flexural strength \( M_{nf} \) or the nominal shear strength \( V_n \) of the test specimen. The acceptable failure mode of the test specimen was limited to failure of the flange splice plates.

4.2.2 TEST CONFIGURATION

The FST was performed in Fritz Engineering Laboratory at Lehigh University under the 22,240 kN Baldwin universal testing machine. The test configuration is shown in Figure 4.3. The girder was simply supported at the locations of T-stiffeners
with a span length of 12 m. The load points were at location A (load point on G8Ar) and location B (load point on G7Ar). A T-stiffener was already present at B. A new plate stiffener was designed and fabricated to accommodate the load at the load point A. To maximize the bending moment, while accommodating the length and location of the flange splice plates, the load points were located as far as possible from the supports. Points A and B are located on either side of the splice location, C.

Free body diagrams of the test configuration of the FST are shown in Figure 4.4 (a). From the test configuration, shear and bending moment diagrams can be generated. These are shown in Figures 4.4 (b) and (c). The shear and bending moment diagrams are based on loads, F_a and F_b, applied to the test specimen to develop the largest possible moment between locations A and B without exceeding M_nf or V_n. In particular, it was found that V_n was more restrictive than M_nf for the test specimen. Therefore, the loads F_a and F_b, and their locations, were selected to produce equal shear force, V, at each end of the specimen. From the shear and bending moment diagrams, the following can be observed:

1. The largest values of shear, V, develop between the supports and points A and B.
2. The largest moment develops between A and B. The region between A and B represents a region of relatively constant moment.
3. The largest moment that develops at C (M_splice) is proportional to the largest shear (V).
From the third observation, the following equations relating $V$ to the loads, shear forces, and bending moments that develop in the test specimen can be derived (with shears and loads in kN and the moments in kN.mm).

\[ V_{\text{splice}} = 0.20V \]  \hspace{1cm} (4.1)
\[ P = 2V \] \hspace{1cm} (4.2)
\[ F_a = 1.20V \] \hspace{1cm} (4.3)
\[ F_b = 0.80V \] \hspace{1cm} (4.4)
\[ M_a = 5000V \] \hspace{1cm} (4.5)
\[ M_b = 4500V \] \hspace{1cm} (4.6)
\[ M_{\text{splice}} = 4750V \] \hspace{1cm} (4.7)

where:

$V$ is the shear that develops in the web between the supports and locations A and B

$V_{\text{splice}}$ is the shear that develops at location C.

$P$ is the load applied from the test machine to the spreader beam ($W \, 14 \times 398$)

$F_a$ is the load applied to the test specimen at location A

$F_b$ is the load applied to the test specimen at location B

$M_a$ is the value of moment at location A

$M_b$ is the value of moment at location B

$M_{\text{splice}}$ is the value of moment at the splice, location C.

The largest value of $V$ that was allowed for the FST is $V_n = 1772$ kN. This value was calculated by multiplying $\tau_n$ (from Eq. 2.1) by the cross sectional area of the web.
\[ A_w = 9000 \text{ mm}^2 \] (Abbas 2003). Table 4.1 shows the corresponding values for the above variables when \( V = V_n \). These values indicate that the largest load \( (P_{\text{max}}) \) that can be applied to the test specimen without the possibility of failing the test specimen in shear is 3554 kN.

From the bending moment diagram shown, the largest moment develops at \( A (M_a) \). To prevent flexural failure of the girder (not the splice) during the FST, the nominal flexural resistance of the flanges (the flange force, \( R_{nf} \)) should be greater than the force that develops in the flanges at location \( A \) (from \( M_a \)). The required flexural resistance of the flanges was determined after the design of the flange splice plates for the FST was completed. The relationship between the nominal flexural resistance \( (R_{nf}) \) and the nominal flexural strength of the girder \( (M_{nf}) \) is defined by Eq. 4.8., where as discussed in Section 2.1, it is assumed that the corrugated web does not contribute to resisting the bending moment.

\[
M_{nf} = d_{ma}R_{nf} \quad (4.8)
\]

where:

\( d_{ma} \) is the distance between the middle surfaces of the flange plates (1550 mm).

4.3. TEST CONFIGURATION FOR THE SHEAR STRENGTH TEST

4.3.1 OBJECTIVE OF THE TEST

The objective of the Shear Strength Test (SST) was to investigate the behavior of bolted web splices designed for corrugated web girders. To achieve this objective, the largest possible shear had to be developed in the test specimen. The acceptable
failure mode of the test specimen was failure of the web by buckling or failure of the web splice.

4.3.2 TEST CONFIGURATION

The SST was also performed under the 22,240 kN Baldwin universal testing machine in Fritz Engineering Laboratory. The test configuration is shown in Figure 4.5. The girder was simply supported with a span length of 7.5 m. The load point and the reaction points were at T-stiffener locations. The load and reaction points were chosen in order to develop the largest possible shear in the test specimen. The shortest shear span, which is between location A and location B, (shear span AB) is subject to this maximum shear (see Figure 4.6 (a)).

Figure 4.6 (a) shows a free body diagram for the SST. Shear and bending moment diagrams for the SST are shown in Figures 4.6 (b) and (c). The following relationship exists between the shear in span AB ($V_{ab}$) and the load applied to the test specimen ($P$).

$$V_{ab} = \frac{3}{5}P$$  \hspace{1cm} (4.9)

The largest possible value of $V_{ab}$ is the shear causing shear failure of the web.

According to Eq. 2.1, the nominal shear strength is $V_n = 1772$ kN. As discussed in Chapter 2.3, the results from previous shear test results were higher, and the web shear capacity of G7A ($V_{G7A}$) was 2294 kN, while, that of G8A ($V_{G8A}$) was 2153 kN. Table 4.2 shows the corresponding values of $P$, when $V_{ab}$ is assumed equal to $V_n$, $V_{G7A}$ and $V_{G8A}$. These values of $P$ suggest the likely failure load of the test specimen.
4.4 FLANGE SPLICE FOR THE FLEXURAL STRENGTH TEST

4.4.1 OBJECTIVE OF THE FLANGE SPLICE DESIGN

The flange splice designed for the Flexural Strength Test (FST) was designed to fail before the onset of structural damage to test specimen components G7Ar and G8Ar. As shown in Section 4.2.2, the maximum value of $M_{\text{splice}}$, which should be considered in designing the splice, is $M_{\text{splice, max}} = 8,417,000 \text{ kN.mm}$.

4.4.2 DESIGN OF THE FLANGE SPLICE

The flange splice was designed with both inner and outer splice plates. Unlike a conventional flat web, the position of a corrugated web changes with respect to the flange along its length (refer to Figures 2.1 and 2.2). This aspect of a corrugated web limits the geometry of the inner splice plates. The inner splice plates could be designed with a varying width that matches the corrugated web profile, as shown in Figure 4.7 (a). However, considering the fabrication effort, among other things, the inner splice plates were designed with a constant width, as shown in Figure 4.7 (b). The maximum available width of the inner splice plates (126 mm) was used as shown in Figure 4.7 (b). A tolerance of 13 mm between the inner splice plates and the flange-web fillet web was introduced to account for any inconsistencies in the web profile. The width of the outer splice plate was designed equal to the width of the flange (450 mm) as shown in Figure 4.7 (c).

The flange splice was designed to satisfy both service and strength requirements according to the AASHTO LRFD specifications (ASSHTO 1998). The flange splice was designed as a slip critical connection for the flange force ($R_{\text{nl}}$) that develops from
the flexural moment induced stress ($F_s$) at the Service II limit state. $R_{nf}$ at the service limit state was calculated from multiplying $F_s$ by the cross sectional area of the flange ($A_f = 2250 \text{ mm}^2$). $F_s$ was assumed to be equivalent to $(1.15/1.5)$ times one half the yield stress of the flange of the test specimen ($0.5F_yf$). This estimate was based on $F_yf = 485$ MPa and represents a relatively low stress compared to those that can develop.

The connection was designed using A 325 - 22 mm diameter bolts. The bolt layout was chosen and bolt regions were identified, as shown in Figure 4.8. The splice spacing was designed to be 6 mm. There exists a single slip plane in Region B while in Regions A and C there are two slip planes. Bolts were included in region B to avoid violating the maximum bolt spacing requirement of the AASHTO LRFD specifications (AASHTO 1998) for the outer plate. For the given bolt layout, the specifications for minimum and maximum spacing, and edge distances were satisfied.

The flange splice was designed preliminarily for flange force ($R_{nf}$) that develops from the flange stress ($F_{cf}$) at the strength limit state. $F_{cf}$ is equal to the average of the flexural moment induced stress ($f_{cf}$) and the flexural resistance of the flanges of the test specimen ($F_{yf}$). $R_{nf}$ at the strength limit state was calculated from multiplying $F_{cf}$ by $A_f$. $f_{cf}$ is assumed to be one half the yield stress of the flange, i.e, $f_{cf} = 0.5F_{yf}$, and therefore $F_{cf}$ equal to $0.75F_{yf}$. For a flange splice with inner and outer plates, $R_{nf}$ at the strength limit state has to be proportioned respectively between each plate. The distribution of flange force between the inner and outer plate could not be determined as stated in the AASHTO LRFD specifications (AASHTO 1998), which distribute the force between both plates under the assumption that each bolt passes through both inner and outer plates. However, the bolts in region B only pass through the outer plate, while, the bolts
in Region A and C pass through both the inner and outer plates. Therefore for design purposes, assumptions were made concerning the distribution of forces between the inner and outer plates.

The forces were distributed among the flange splice plates based on the assumption that $R_{nf}$ at the strength limit state is distributed to the inner and outer plates via the bolts, and the proportion of $R_{nf}$ distributed to a region is determined by the number of bolts ($nb$) and slip planes ($Ns$) in that region. The proportion of $R_{nf}$ in a given region was determined as follows in Eq. 4.10 to Eq. 4.12:

$$
\text{Ratio}_A = \frac{nb_A \cdot Ns_A}{nb_A \cdot Ns_A + nb_B \cdot Ns_B + nb_C \cdot Ns_C} \quad (4.10)
$$

$$
\text{Ratio}_B = \frac{nb_B \cdot Ns_B}{nb_A \cdot Ns_A + nb_B \cdot Ns_B + nb_C \cdot Ns_C} \quad (4.11)
$$

$$
\text{Ratio}_C = \frac{nb_C \cdot Ns_C}{nb_A \cdot Ns_A + nb_B \cdot Ns_B + nb_C \cdot Ns_C} \quad (4.12)
$$

where:

$\text{Ratio}_A$ is the proportion of $R_{nf}$ at the strength limit state in the bolts in Region A

$\text{Ratio}_B$ is the proportion of $R_{nf}$ at the strength limit state in the bolts in Region B

$\text{Ratio}_C$ is the proportion of $R_{nf}$ at the strength limit state in the bolts in Region C

$N_{sA}$ is the number of slip planes in Region A

$nb_A$ is the number of bolts on one side of the splice in Region A

$N_{sB}$ is the number of slip planes in Region B

$nb_B$ is the number of bolts on one side of the splice in Region B

$N_{sC}$ is the number of slip planes in Region C
\( n_{BC} \) is the number of bolts on one side of the splice in Region C.

The number of slip planes and bolts in Region A and C are equal, therefore, the ratios are equal (i.e., \( \text{Ratio}_A = \text{Ratio}_C \)).

The proportion of \( R_{nf} \) at the strength limit state distributed to one inner plate, \( F_{IP} \), and to the outer plate, \( F_{OP} \), respectively, was determined as follows:

\[
F_{IP} = \frac{\text{Ratio}_A \cdot R_{nf}}{N_{SA}}
\]

\[
F_{OP} = R_{nf} - 2 \cdot F_{IP}
\]

The outer flange splice plate was designed for a nominal force of \( F_{OP} \), while, each inner splice plate was designed for a nominal force of \( F_{IP} \). The design of the flange splice plates is shown in Figure 4.9. Table 4.3 indicates \( R_{nf} \) at a given limit state for the flange splice when the nominal yield strength of the splice plates is 485 MPa. The results from the table show that the failure of the flange splice plates by fracture on the net section is the controlling factor for design. The flange splice plates were, therefore, designed to fail by fracture on the net section when a moment of \( M_{\text{splice fup}} \) develops at the splice. This value is shown in Table 4.4. This preliminary splice design cannot be used because the values of \( M_{\text{splice fup}} \) based on the factored and nominal resistances (i.e., 8,536,000 and 10,672,000 kN.mm) exceed \( M_{\text{splice max}} \). For the flange splice plate design for the FST to be successful, this design had to be revised for \( M_{\text{splice fup}} < M_{\text{splice max}} \).

The design was altered by using flange splice plates of with a yield strength of 345 MPa. This lowered the resistances of the flange splice plates. Table 4.5 indicates \( R_{nf} \) at a given limit state when the nominal yield strength of the splice plates is 345 MPa. This design satisfies the objective of the design of the flange splice plates for the
FST because as seen in Table 4.6, $M_{\text{splice max}}$ exceeds the values of $M_{\text{splice fnp}}$ based on the factored and nominal resistance (i.e., 6,166,000 and 7,707,000 kN.mm).

To meet the objective of the FST, neither $M_{nf}$ nor $V_n$ could be exceeded in order to maintain the integrity of the specimen for the SST. The results from Table 4.6 show that the controlling factor for $M_{nf}$ of the test specimen is fracture of the flange on the net section. Therefore, $M_{nf}$ is calculated from Eq. 4.8 where $R_{nf}$ is the flange force that develops when the flange fractures at the net section. Table 4.6 and Table 4.1 show that $V_{fnp} < V_n$ and that $M_{a \text{ max}} < M_{nf}$. Therefore, the best suited design would be as shown in Figure 4.8 using steel plates with a nominal yield strength of 345 MPa. It is expected from this design that the flange splice plates should fracture before the failure of the web or flanges during the FST.

Tension coupon tests were conducted on the flange splice plate material. Table 4.5 shows $R_{nf}$ at different limit states for the measured material properties. The corresponding values of load, shear, and moment obtained from $R_{nf}$ are presented in Table 4.7. The table shows that the value of $V_{fnp}$ based on the factored $R_{nf}$ is less than $V_n$, however, the opposite is true for the value based on the nominal $R_{nf}$. Therefore, shear failure at the web appears possible during the FST.

4.5 WEB SPLICE FOR THE SHEAR STRENGTH TEST

4.5.1 OBJECTIVE OF THE WEB SPLICE DESIGN

The web splice for the Shear Strength Test (SST) was designed to resist the shear and the moment due to the bolt eccentricity that would develop during the SST.
The web splice was not designed for a web shear larger than the shear strength of the web.

4.5.2 DESIGN OF THE WEB SPLICE

The web splice was designed according to the AASHTO LRFD specifications (AASHTO 1998) except as follows:

1. According to the AASHTO specifications web splices should not have less than two rows of bolts on either side of the splice. The web splice was designed with one row of bolts on each side of the splice.

2. The bolt tightening clearances of the AASHTO specifications were not considered in the design of the web splice. These requirements can be easily satisfied by revising the design.

Figure 2.2 shows that the width of the flat portion of the longitudinal web folds is 220 mm. To ensure proper alignment of the web splice plates, the width of the web splice plate ($W_{ws}$) was limited to 220 mm. Two rows of 22 mm diameter bolts would not fit on either side of the splice under the spacing and edge distance requirements of AASHTO LRFD specifications (AASHTO 1998). Therefore, only one row of 22 mm diameter bolts was used on either side of the splice. The final web splice design is shown in Figure 4.10.

The splice was designed for the web shear under two load conditions: the service limit state ($V_{uw\text{serv}}$) and the strength limit state ($V_{uw\text{stren}}$). The splice connection was designed as a slip critical connection for the web shear force at the service limit state ($V_{uw\text{serv}}$). This shear was assumed to be $(1.15/1.5)$ times the web shear at the splice.
due to the factored loads $V_u$ for the Strength I load combination. $V_u$ was assumed to be $0.8V_r$; where $V_r$ is defined as the factored shear resistance of the web. As discussed in Section 2.4, the value of $V_r$ for corrugated web I girders, can be determined from Eq. 2.1. Therefore, $V_r$ is defined as follows:

$$V_r = \phi_v 0.707\tau_y A_w$$

(4.15)

where:

$\phi_v = 1.0$

$$\tau_y = \frac{F_{yw}}{\sqrt{3}}$$

$F_{yw} = 485$ MPa

$A_w = 9000$ mm$^2$

The bolt layout is shown in Figure 4.10 (b). Based on this layout, the connection was checked to satisfy the requirements of a slip critical connection under the maximum resultant bolt force that develops due to the eccentricity of the shear.

The bolted connection was designed at the strength limit state to resist a web shear force ($V_{uwstren}$) equal to the average of the shear due to factored loads ($V_u$) and the factored shear resistance of the web at the point of splice ($V_r$).

$$V_{uwstren} = \frac{V_r + V_u}{2}$$

(4.16)

For design, it was assumed that $V_{uwstren}$ was distributed equally between the splice plates on either side of the web. Therefore, the design force for one web splice plate is defined as follows

$$F_{wp} = \frac{1}{2}V_{uwstren}$$

(4.17)
The web splice plate design is shown in Figure 4.10 (b). Table 4.8 indicates the shear force that develops in the web at a given limit state. Based on the design, the governing failure mode for the SST is expected to be failure of the web in shear because the shear resistance of the splice exceeds the nominal web shear strength \( V_n = 1772 \) kN. The shear failure of the bolts is also considered critical for the design of the web splice because of the web shear at bolt failure is close to the web shear strength. To avoid the hazardous failure of the bolts, the value of \( P \) during the SST should be carefully monitored. The value of load at which failure of the bolts will occur, \( P_{bs} \) is shown in Table 4.9.

Figure 4.11 and Figure 4.12, show a layout of the final design of the flange and web splice plates for the SST. It can be observed in this figure that the bolt tightening clearance requirements are not satisfied. In order to satisfy these requirements, the length of the web splice plate should have been reduced and the bolt spacing and edge distances adjusted accordingly.

**4.6 DESIGN OF THE FLANGE SPLICE FOR THE SHEAR STRENGTH TEST**

The flange splice for the Shear Strength Test (SST) was designed to resist the largest \( M_{splice} \) that could develop during the SST. Figure 4.6 (c) shows the moment diagram for the SST. The largest \( M_{splice} \) for which the flange splice plates would be designed would be greater than that which could develop when \( V = V_{G7A} \) (see Table 4.9).

The design considerations of the flange splice for the SST are similar to those discussed for the FST in Section 4.4. The design of the flange splice plates for the SST
is shown in Figure 4.13. Table 4.10 indicates $R_{nf}$ at different limit states for a flange splice plates designed with a yield strength of 345 MPa. The controlling design limit is failure of the flange splice plates by fracture of the net section. It can be observed from the Table 4.11 that $M_{\text{splice}} (V = V_{G7A}) < M_{\text{splice fhp}}$. Therefore, based on this design, failure of the web by shear during the SST is the governing failure mode.

4.7 DESIGN OF WEB SPLICE FOR THE FLEXURAL STRENGTH TEST

From Table 4.1, the largest shear that can develop at the splice before the onset of any structural damage of the test specimen during the FST is 443 kN. The web splice design, discussed in Section 4.5.2, was adequately designed to resist 443 kN of web shear force. Therefore, the plates designed for the SST were used for the FST.

The design of the web splice plates for the SST was modified for use in the FST. The expected failure mode for the FST was fracture of the net section of the flange splice plates. During the FST, the splice spacing could increase to accommodate elongation of the bottom flange splice plates prior to fracture. As a result, deformation of the bolt holes in the web splice near the bottom flange could occur. To avoid damage of the web splice plates during the FST, the three pairs of bolts closest to the bottom flange were removed from the web splice plate.

Figure 4.14, shows the web splice designed for the FST. Table 4.12 shows the web shear force at a given limit state for the web splice designed for the FST. The critical design limit is shear failure of the bolts. From Table 4.12 and Table 4.1, $V_{\text{splice bs}} > V_{\text{splice}} (V = V_n)$ i.e., 1404 kN $> 354$ kN. Therefore, though three pairs of bolts were removed from the web splice, it was adequately designed for the FST.
4.8 STRESS-STRAIN PROPERTIES OF SPLICE PLATES

The splice plates had a nominal yield strength of 345 MPa. The flange splice plates had a nominal thickness of 19 mm and 32 mm, while the web splice plates had a nominal thickness of 6 mm.

Tensile coupons were cut from portions of the plates used to make the splice plates. The coupons had gage lengths of 203 mm and were fabricated according to ASTM E8 (ASTM 1994). The tensile coupons were tested in the ATLSS Center, Lehigh University in the 2,700 kN SATEC universal testing machine. Tests were performed on three 19 mm thick, two 32 mm thick, and three 6mm thick splice plate coupons.

Table 4.13 shows the results from the splice plate tension coupon tests. The average values given in each table are the properties used in this report. For the flange splice plate steel with a thickness of 19 mm, the average yield stress is 420.7 MPa, the ultimate tensile stress is 526.1 MPa, and the ratio of the yield stress to ultimate tensile stress is 0.80. For the flange splice plate steel with a thickness of 32 mm, the average yield stress is 380.1 MPa, the ultimate tensile stress is 497.9 MPa, and the ratio of yield stress to ultimate tensile strength is 0.77. For the web splice plate steel the average yield stress is 410 MPa, the ultimate tensile stress is 483.3 MPa, and the ratio of yield stress to ultimate tensile stress is 0.85. The elastic steel modulus was assumed to be the usual value for conventional steel of 200 GPa.

Figure 4.15 (a), (b), and (c) show a typical stress strain curve for the splice plate steel with thicknesses of 19 mm, 32 mm, and 6 mm, respectively. This figure shows that
each plate steel has a well defined yield point at the end of the linear elastic range (yield stress). The curve exhibits a yield plateau before the onset of strain hardening.

4.9. TEST SET-UP, INSTRUMENTATION AND PROCEDURE FOR FLEXURAL STRENGTH TEST

This section discusses the test set-up, test procedure and instrumentation for the Flexural Strength Test (FST).

4.9.1 TEST SET-UP

As mentioned in Section 4.2, the corrugated web I-girder splice test specimens were tested in the 22,240 kN Baldwin universal testing machine in Fritz Engineering Laboratory. The test configuration for the FST, shown in Figure 4.3, was briefly discussed in Section 4.2.2. A compressive load was applied to a 521 mm diameter hardened steel cylindrical roller. A spreader beam (W 14 X 398) was used to transfer the load from the 521 mm diameter roller to two 152 mm diameter hardened steel cylindrical rollers at locations A and B. The test specimen was placed in the test machine with the longitudinal axis in the east west direction. The specimen was simply supported with 292 and 254 mm diameter rollers at the west and east end of the test specimen, respectively. Lateral bracing was provided for the specimen, as shown in Figure 4.16. Figure 4.18 shows a photograph of the test specimen in the test machine.
4.9.2 TEST INSTRUMENTATION

The applied load was obtained from the load cell in the testing machine. Instrumentation measured three aspects of the test specimen behavior: (1) vertical deflection; (2) slip and elongation of the flange splice plates; and (3) strains in the flange splice plates and the flange.

The vertical deflection was measured by displacement transducers at the bottom flange of the girder at the locations shown in Figure 4.18. Slip of the flange splice plates was monitored by displacement transducers centered at the ends of the splice plates of the bottom flange (Figure 4.19). Each displacement transducer was numbered as shown in Figure 4.19. Elongation of the bottom flange splice plates was measured by displacement transducers placed on the splice plates, at four locations as shown in Figure 4.19 (g).

High elongation uniaxial strain gages were placed on the splice plates as shown in Figure 4.20. Strain gages were not placed in pairs, with one gage on each face of the splice plate, and only a single gage was placed on the outer face of a splice plate at each location. The strain gages were placed at locations along the plate where fracture was likely during the FST, i.e., on the net section of the splice plate bolt holes closest to the splice and at the splice location, as shown in Figure 4.9 and Figure 4.20. Each strain gage was numbered as shown in Figure 4.20.

High elongation linear strain gages were placed on the flanges of the test specimen as shown in Figure 4.21.
4.9.3 TEST PROCEDURE

The procedure used for the FST was composed of two steps: (1) three elastic cycles to 1780 kN, and (2) loading until failure. The initial loading cycles up to 1780 kN were used to align and seat the test specimen, check the instrumentation, and monitor possible lateral or longitudinal movement of the test specimen. Load was applied at a rate of 25 kN/min during the elastic cycles until the test was terminated and the specimen unloaded. For the loading until failure, the test specimen was loaded to 3852 kN. Load was initially applied at a rate of 25 kN/min up to 1780 kN. Thereafter, a displacement rate of approximately 0.6 mm/min was used until 3852 kN, at which point the test was terminated before structural damage of test specimen components G7Ar and G8Ar occurred. At several points during the test, the loading was stopped temporarily so that photographs could be taken. Loading at the displacement rate of 0.6 mm/min was then resumed.

4.10 TEST RESULTS AND ANALYSIS

The flange splice plates designed for the FST were designed to fail before the onset of structural damage to test specimen, components G7Ar and G8Ar. As discussed in section 4.4.1, the expected mode of failure was fracture on the net section of the bottom flange splice plates.

Load vs. deflection of the test specimen, longitudinal strains that developed in the flange splice plates and flanges, and the slip and elongation of the plates during the FST are discussed in this section.
Figure 4.22 shows the overall load-deflection of the test specimen. The test specimen experiences some seating in the initial loading stages, then exhibits linear elastic behavior up to approximately 3,000 kN. At this point, the girder shows a small change in stiffness. However, the load continues to increase significantly until the test is stopped at a load $P = 3852$ kN.

Figure 4.23 shows plots of load vs. vertical displacement at locations A, B and C (see Figure 4.3 for locations). The vertical deflection of locations A and B are almost identical during the test (Figure 4.23 (a)). The plots of the north and south vertical deflections of location C indicate that soon after the start of the test, the vertical deflections of the north and south flange tips were different. Larger vertical deflections were seen per unit load for the south flange tip deflection, as shown by the smaller slope in Figure 4.23 (c). However, after $P = 3,000$ kN, the vertical deflections per unit load were approximately the same, as shown by the similar slopes in Figure 4.23 (d). Thus, during the test, the specimen was deflecting slightly more on the south at the location of the splice (location C).

Figure 4.24 shows plots of elongation of the bottom flange splice plates, Splice Plates 1, 2, and 3. Based on Figure 4.24, elongation of Inner Splice Plate 1 occurred earlier (at a lower load) than elongation of Inner Splice Plate 3. Splice Plate 1 was located on the north side of the bottom flange, while, Splice Plate 3 was located on the south side. Also, elongation of the north side of Outer Splice Plate 2 preceded that of the south side. Figure 4.24 indicates that elongation of the plates occurred in the following order: north side Outer Splice Plate 2, south side Outer Splice Plate 2, Inner Splice Plate 1 (north side) and Inner Splice Plate 3 (south side).
If the test specimen behavior is symmetric during testing, elongation of the inner (or outer) splice plates on the north and south side should occur at the same time. However, the FST results given above show a lack of symmetry. Lack of symmetry is also seen in the results of Table 4.14. Table 4.14 shows the load, \( P \), at which local yielding of the splice plates at the strain gage locations occurred. The theoretical yield strain is 2105 micro strain and 1901 micro strain for the outer and inner plates, respectively (based on the properties from tensile coupon tests). From the locations of the strain gages and the sequence in which yielding at each location occurred, a pattern of local yielding of the splice plates progressing from north to south was observed.

When all the strain gages in a line across the width of an outer splice plate reach the yield strain, the plate can be assumed to have yielded at the cross section along the line of the gages. Also, when all the strain gages along the same line across the width of the two inner splice plates on the same flange reach the yield strain, the two inner plates can be assumed to have yielded at the cross section. Table 4.15 shows the load, \( P \), at which yielding of the flange splice plates occurred. Table 4.15 shows that the first line of gages to yield was along Outer Splice Plate 2 located at the bottom flange. This set of gages was centered under the splice (a location without bolt holes) and yielded at \( P = 3089 \) kN.

The value of \( P \) at yielding of Splice Plate 2 along the gross section and net section was \( P = 3089 \) kN and \( P = 3132 \) kN, respectively. It was expected that yielding and fracture of the net section of the splice plates would precede yielding of the gross section. However, the strain was measured on only the exposed surface of each splice plate. Therefore, the assumption that the entire cross section of a splice plate yields
when the surface strains reach yield does not appear to be valid, especially for the cross section of Outer Splice Plate 2 directly under the splice, where yielding of the gross section before yielding of the net section of the plate is not expected. The effect of plate bending appears to be significant at this location.

The calculations done, however, to determine the load, \( P \), at which fracture of the net section or yielding of the gross section occurred are based on the assumption that each plate is in pure tension. The effect of the bending of the plate was not considered into these calculations, although, high bending strains could develop in the splice plates directly at the splice, but are not expected elsewhere on the splice plates where the plate is bolted firmly against the flange.

The flange splice plates were designed in Section 4.4.2 assuming a distribution of the flange force \( R_{nf} \) between the inner and outer splice plates. The distribution between the plates is based on the assumption that each plate is subject to pure tension without bending. From Eq. 4.13 and 4.14, the ratio of \( R_{nf} \) that is distributed to the outer plate is 0.544, while the ratio of \( R_{nf} \) that is distributed to the two inner plates is 0.456.

Table 4.16 shows estimates of the forces in the outer and inner plates at a given load \( P \) for several limit states. In this table, the first nine rows focus on forces on the splice plates at yield of the net section. The last three rows focus on fracture of the net section. Bold font is used in the table for values that are the basis for calculating the remaining numbers in the table.

The first set of three rows in Table 4.16 focuses on estimating the yield resistance of the net section of the splice plates. The yield resistance of the net section of the outer splice plate is 2654 kN, while, that of the two inner plates is 1917 kN. The
The first row of the table shows that if it is assumed that the outer plate is at yield on the net section ($F_{OP} = 2654 \text{kN}$) and that the force ratios are 0.544 for the outer plate and 0.456 for the two inner plates, then the force that develops in the two inner plates is 2224 kN, and the applied load is $P = 3184 \text{kN}$. However, the calculated force of 2224 kN for the inner splice plates is well above the calculated net section yield resistance of the two inner plates, 1917 kN, so this result is not correct. The second row of the table shows that if it is assumed that the two inner plates are at yield on the net section ($2F_{IP} = 1917 \text{kN}$) and that the force ratios are 0.544 for the outer plate and 0.456 for the two inner plates, then the force that develops in the outer plate is 2287 kN, and $P = 2744 \text{kN}$. The third row shows that when the inner and outer plates are at yield on the net section, and the assumed force ratios are not used, the resulting force ratios are 0.581 for the outer plate and 0.419 for the two inner plates, and $P = 2983 \text{kN}$. Unfortunately, these results do not agree well with the test results. Table 4.15 shows that, during the test, the applied load was 3132 kN and 3150 kN, respectively, at yielding of the east (Event 2) and west (Event 4) net sections of the outer splice plate. These values are quite close to the estimated value of 3184 kN (which was unacceptable because it required the inner splice plates to carry a force of 2224 kN), but are much greater than the other net section yield force estimates of 2744 kN and 2983 kN.

The second set of three rows in Table 4.16 provides estimates of the splice plate forces at Event 2, yielding of the east net section of the outer splice plate, when $P = 3132 \text{kN}$ and the corresponding flange force is $R_{nf} = 4799 \text{kN}$. The first of these three rows shows that when the force ratios of 0.544 and 0.456 are used to distribute $R_{nf} = 4799 \text{kN}$ to the splice plates, the resulting forces are $F_{OP} = 2611 \text{kN}$ and $2F_{IP} = 2188 \text{kN}$. 
This result for $F_{OP}$ is close to the calculated net section yield resistance of the outer plate, 2654 kN, but the result for $2F_{IP}$ is well above the calculated net section yield resistance of the two inner plates, 1917 kN, so these results are not correct. The second of the three row set for Event 2 shows that if it is assumed that the outer plate is at yield on the net section ($F_{OP} = 2654$ kN), and the remainder of $R_{nf}$ is distributed to the two inner splice plates, then the resulting force ratios are 0.553 and 0.447, and $2F_{IP} = 2188$ kN, which, again, is well above the calculated net section yield resistance of the two inner plates, 1917 kN, so these results are not correct. The third of these three rows considers the possibility of strain hardening in the outer plate. Table 4.17 shows that at Event 2, when $P = 3132$ kN, two strain gages on the surface of the outer plate at the east net section (at locations 6 and 8) have strains well into the strain hardening range of the steel (see Figure 4.15), while two other strain gages on the outer plate at the east net section (locations 10 and 13) are just past yield. Table 4.17 also shows that the strain gages on the surface of the inner plates at the east net section (at locations 3 and 16) have strains below yield. The calculations for the third row for Event 2 in Table 4.16 are based on the assumption that the stress at the net section of the outer plate is the average of the yield (420.7 MPa) and a strain hardening value of 511 MPa, which is estimated from the strains at strain gage locations 6 and 8. The resulting outer plate force is $F_{OP} = 2939$ kN, and when the remainder of $R_{nf}$ is distributed to the two inner splice plates, the resulting force ratios are 0.612 and 0.388, and $2F_{IP} = 1860$ kN, which is below the calculated net section yield resistance of the two inner plates, 1917 kN. Thus, the third row for Event 2 provides a plausible distribution of flange force between the inner and outer plates when the outer splice plates have strains above yield at all
strain gages on the net section. This row suggests that, at Event 2, portions of the net section of the outer plate are strain hardening when other portions are reaching yield, and that the inner plates are below yield at the net section.

The third set of three rows in Table 4.16 provides estimates of the splice plate forces at Event 4, yielding of the west net section of the outer splice plate, when $P = 3150$ kN and the corresponding flange force is $R_{nf} = 4827$ kN. The calculations are very similar to those discussed above for Event 2. Similarly, the third row for Event 4 provides a plausible distribution of flange force between the inner and outer plates when the outer splice plates have strains above yield at all strain gages on the west net section. The third row for Event 4 suggests that, at Event 4, portions of the net section of the outer plate are strain hardening when other portions are reaching yield, and that the inner plates are below yield at the net section.

The final set of three rows in Table 4.16 focuses on estimating the fracture resistance of the net section of the splice plates. The fracture resistance of the net section of the outer splice plate is 3319 kN, while that of the two inner plates is 2511 kN. The first of the three rows for fracture on the net section shows that if it is assumed that the outer plate is at fracture on the net section ($F_{OP} = 3319$ kN) and that the force ratios are 0.544 for the outer plate and 0.456 for the two inner plates, then the force that develops in the two inner plates is 2782 kN, and the applied load is $P = 3982$ kN. However, the calculated force of 2872 kN for the inner splice plates is well above the calculated net section fracture resistance of 2511 kN, so this result is not correct. The second row for fracture on the net section shows that if it is assumed that the two inner plates are at fracture on the net section ($2F_{IP} = 2511$ kN) and that the force ratios are
0.544 for the outer plate and 0.456 for the two inner plates, then the force that develops in the outer plate is 3004 kN, and $P = 3599$ kN. This applied load is well below the maximum load applied during the test ($P = 3852$ kN). The third row shows that when the inner and outer plates reach fracture on the net section at the same time, and the assumed force ratios are not used, the resulting force ratios are 0.569 for the outer plate and 0.431 for the two inner plates, and $P = 3805$ kN. This result is slightly below the maximum force applied during the test, $P = 3852$ kN, and, thus, the third row for fracture on the net section provides a reasonable distribution of flange force between the inner and outer plates at the fracture limit state.

The plot of load vs. overall vertical deflection of the test specimen in Figure 4.22 shows that the load continued to increase after yielding of the cross section(s) of the outer splice plate. At $P = 3000$ kN, not all of the flange splice plate net section had yielded. The net section area that had yielded by $P = 3000$ kN reached the strain hardening range of behavior. Figure 4.25 shows photographs of the flange splice plates. The plates are centered on the center of the splice so that the four bolt holes nearest the center of the photos are from either the east net section or west net section of the splice plates. The elongation of the bolt holes from the significant strains at the net section is usually evident.

At values of $P$ greater than $P = 3000$ kN, strain reversal was observed in a number of gages. Figure 4.26 shows a typical strain curve for a strain gage for the FST. The values of $P$ at which strain reversal occurred at each location and the corresponding strain is tabulated in Table 4.18. The strain reversals were unexpected and cannot be entirely explained. One possible cause for these reversals is failure of the strain gages.
However, there is no consistency in the values of strain at which reversal is seen (i.e., not all strain reversals were seen at high levels of strain) and there was no visible debonding of the strain gage from the surface to the plates. However, no physical explanation of the reversal of strains based on the expected behavior of the test specimen has been found.

The flange splice was designed as a slip critical connection. This design was successful. Figure 4.27 shows the typical load vs. slip curve for a splice plate that had slipped during the FST. This curve indicates that slip of the splice plates did not occur until just before yielding of the test specimen.

4.11 SUMMARY AND CONCLUSIONS FROM FLEXURAL STRENGTH TEST

Significant yielding of the test specimen during the flexural strength test (FST) occurred with yielding of the net section of the bottom flange outer splice plate. Insignificant premature yielding on the surface of the gross section of the outer splice plate was observed at the center of the splice where local bending of the plates can occur.

The test was terminated at an applied load $P = 3852$ kN. The corresponding shear of 1926 kN, and the corresponding splice moment of $9,149,000$ kN.mm were well above the test limits of $V_n = 1772$ kN and $M_{splice,max} = 8,417,000$ kN.mm (see Table 4.1 or Table 4.7) established to avoid failing the test specimen components G7Ar and G8Ar in shear. Previous test results from Abbas (2003), discussed in Section 4.3.2, indicated that these test limits could be slightly exceeded. However, due to unexplained overstrength of the splice, failure of the flange splice by fracture on the net section of
the splice plates did not occur, and the test was terminated to avoid damaging the test specimen components.

The largest flange force $R_{nf}$ observed during the FST was estimated to be 5902 kN (with a corresponding $P$ of 3852 kN). Table 4.16 shows when $R_{nf}$ reaches 5830 kN (with a corresponding $P$ of 3805 kN), fracture of both the outer and inner splice plates should occur. Thus, the actual strength of the flange splice exceeded the calculated maximum strength based on measured material properties by about 1%. The design calculations of the flange splice resistance (see Table 4.7), using assumed flange force ratios of 0.544 and 0.456 for the outer and two inner splice plates, respectively, and measured material properties, indicate a maximum flange force $R_{nf}$ of 5515 kN, (with a corresponding $P$ of 3599 kN). This unfactored resistance was exceeded by 7% during the test.

4.12. TEST SET-UP, INSTRUMENTATION AND PROCEDURE FOR SHEAR STRENGTH TEST

This section discusses the test set-up, test procedure and instrumentation for the Shear Strength Test (SST).

4.12.1 TEST SET-UP

As mentioned in Section 4.2, the corrugated web I-girder splice test specimens were tested in the 22,240 kN Baldwin universal testing machine in Fritz Engineering Laboratory. The test configuration for the SST, shown in Figure 4.3, was briefly discussed in Section 4.2.2. Figure 4.5 shows the test set up in the machine. The test
specimen was placed in the test machine with the longitudinal axes in the east west
direction. The specimen was simply supported with 292 and 254 diameter rollers at a
distance of 1750 mm west of the splice and at the east end of the test specimen. Lateral
bracing was provided for the specimen, as shown in Figure 4.28. A photograph of the
test specimen in the test machine is shown in Figure 4.29.

4.12.2 TEST INSTRUMENTATION

The applied load was obtained from the load cell in the testing machine.
Instrumentation measured four aspects of the test specimen behavior: (1) vertical
deflection; (2) shear strains that developed in the web; (3) shear strains that developed
in the web splice plate; and (4) longitudinal flange strains.

Vertical deflection was measured by displacement transducers at the bottom
flange of the girder at the location shown in Figure 4.30.

Plus and minus 45° strain rosettes were used to measure the shear strain that
developed in the web at the locations shown in Figure 4.31. The locations of the rosettes
were numbered as shown in Figure 4.31. Rosettes were intended to monitor local
distortion of the web plate. They were placed in pairs, one on each side of the plate at a
given location. Rosettes were also placed on the outside face of the web splice plates at
locations shown in Figure 4.32. The locations of the rosettes were identical for both web
splice plates.

Uniaxial strain gages were placed in pairs, shown in Figure 4.33, one on each
side of the top and bottom flange plates. The locations of these strain gages were
identical for both the top and bottom flange.
4.12.3 TEST PROCEDURE

The procedure used for the SST was composed of two steps: (1) three elastic cycles to 1780 kN, and (2) loading until failure. The initial loading cycles up to 1780 kN were used to align and seat the test specimen, check the instrumentation, and monitor possible lateral or longitudinal movement of the test specimen. Load was applied at a rate of 25 kN/min during the elastic cycle until the test was terminated and the specimen unloaded. After the elastic cycles, the test specimen was loaded to failure in shear. Load was initially applied at a rate of 25 kN/min up to 1780 kN. Thereafter, a displacement rate of approximately 0.6 mm/min was maintained until failure. At several points during the test, the loading was stopped temporarily so that photographs could be taken.

4.13 TEST RESULTS AND ANALYSIS

The web splice plates for the SST were designed to resist the shear and the moment due to eccentricity of the shear that would develop during the SST, at the point of the splice. As discussed in Section 4.5.2, the governing failure mode for the SST is failure of the web in shear.

Load vs. deflection of the test specimen, shear strains developed in the web and web splice plates, and longitudinal strains that developed in the flanges are discussed in this section.

Figure 4.34 shows the overall load-deflection of the test specimen. The test specimen experiences some seating in the initial loading stages, then exhibits linear elastic behavior up to approximately 2750 kN. At this point the girder shows a small
change in stiffness. However, significant softening does not occur until $P = 3352 \text{ kN}$.

After the peak load is attained, the load drops off, but not suddenly.

Failure of the test specimen occurred at the peak load of $P = 3352 \text{ kN}$. Buckling initiated in the top half of the longitudinal fold containing strain rosettes 4 and 5 (see Figure 4.31) and as increased load was applied, the buckled waves extended to the adjacent inclined flat fold. Figure 4.35 shows a photograph taken of the appearance of the web after the SST was completed.

The plots of load vs. average shear strains for the rosettes at locations 1 to 5 (Figure 4.37), gives an approximation of the value of load at which local yielding of the web plate and the web splice plates occurred. The average shear strains are calculated as the average of the shear strains recorded on each side of the web plate at a given location. Thus, the average strain represents the mid surface theoretical yield strain of the web is 2414 micro strain. From these plots it can be seen that yielding occurred at locations 1, 2, 3 and 5 at load values of 2589 kN, 2781 kN, 3308 kN, and 2490 kN, respectively.

The plot of load vs. average strain at location 4 (Figure 4.37 (d)) indicates that yielding of the plate did not occur at that location. From the plot of the shear strains of the rosettes on the north and south face (Figure 4.36 (d)), significant separation of strains is seen at a load value of 2750 kN. Thereafter, the strain separation increases which leads to strain reversal in the shear strain of the south face. This indicates that a local buckle formed in this region before shear yielding of the mid surface of the web plate.
Local plate curvature is obtained by subtracting the shear strains on the north face from those on the south face and dividing the result by the thickness of the web. The plot of load vs. curvature (Figure 4.38 (d)) and the plot of load vs. shear strain (Figure 4.36 (d)) at location 4 indicate that, the load value at which visible separation occurs corresponds to the value of load at which large increases in plate curvature are seen. Therefore, it can be concluded that the formation of the local buckle contributed to initial softening seen in Figure 4.34, at P = 2750 kN.

An examination of the shear strains of the rosettes at location 5 indicates that yielding as opposed to bending of the plate occurred at lower levels of applied load. This is seen in the plots of shear strains in Figure 4.36 (e) where only a small strain separation is observed at location 5 up to the peak load. Strain separation occurs later at P = 3352 kN. Therefore, it can be assumed that when the drop in load occurred at P = 3352 kN, the local buckle had extended further down the fold to involve location 5.

Though local yielding of locations 2 and 3 were observed, out of plane deflections were not. This can be observed from Figures 4.36 (b) and (c). At location 1, however, strain separation occurred soon after the initial load was applied to the test specimen (Figure 4.36 (a)). Strain separation at such a low value of load indicates that there were significant initial imperfections in the flatness of the plate that grew with an increase in load.

The web splice plates were designed to resist the load that would develop during the SST. The strains that developed during testing at locations 6, 7 and 8 were less than the shear yield strain of the web splice plate. However, observations are made of unexpected changes in shear strains during testing. Plots of load vs. shear strain at
locations 7 to 8 (Figure 4.36 (g) and (h)) shows the relative bending of the north and south face of the web splice plates relative to each other. The separation of strains gives an indication of the load at which this occurred. At the peak load, strain separations are observed at locations 7 and 8 of magnitudes -42 micro strain and 454 micro strain, respectively. These values indicate that some distortions were taking place at these locations. These results are not available for location 6 due a faulty strain rosette on the south face.

After the peak load is attained, the shear strains at locations 1, 2 and 3 continue to increase. Where buckling had occurred at locations 4 and 5, the curvature continued to grow.

4.14 SUMMARY AND CONCLUSION FROM SHEAR STRENGTH TEST

For summarizing the results of the Shear Strength Test (SST), the shear stress, \( \tau \), is approximated as:

\[
\tau = \frac{V}{D\cdot t_w}
\]  

(4.18)

where:

- \( D \) is the web depth (1500 mm),
- \( t_w \) is the web thickness (6 mm).

The peak load developed during test was, \( P = 3352 \) kN. This corresponds to \( V_{ab} = 2011 \) kN, \( \tau = 0.223 \) GPa and \( \tau/\tau_y = 0.797 \). This result from the SST is plotted in context of Figure 2.5, which is repeated as Figure 4.39 for convenience. Comparing the plots of the results obtained from G7A, G8A, and the SST test specimen; the shear
strength of G7A and G8A are 12% and 6% greater than that of the SST test specimen, respectively.

Equation 2.1 predicts the nominal shear resistance is \(0.707\tau_y\). This is less than the result obtained for the test specimen of the SST. The test specimen failed by web buckling at \(\tau = 0.797\tau_y\). While, G7A and G8A failed at \(0.907\tau_y\) and \(0.853\tau_y\), respectively. These differences are attributed to the magnitude and distribution of imperfections that can be associated with corrugated webs.

The web splice plates were designed adequately so that the corrugated web I-girder specimen could exceed its shear capacity according to the nominal capacity given by Eq. 2.1.
Table 4.1 Forces and Moments When $V = V_n$ for FST

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<tr>
<th>$V_n$ (kN)</th>
<th>$V_{splice}$ max (kN)</th>
<th>$P_{max}$ (kN)</th>
<th>$F_a \text{ max}$ (kN)</th>
<th>$F_b \text{ max}$ (kN)</th>
<th>$M_a \text{ max}$ (kN.mm)</th>
<th>$M_b \text{ max}$ (kN.mm)</th>
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<tr>
<td>1772</td>
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Table 4.2 Values of $V_{ab}$ and Corresponding Applied Load for SST

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<td>$V = V_{G7A}$</td>
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<td>3823</td>
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Table 4.3 Flange Forces for Initial Flange Splice Plate Design for FST.

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<tr>
<th>Limit States</th>
<th>$\Phi$</th>
<th>Factored $R_{nf}$ (kN)</th>
<th>Nominal $R_{nf}$ (kN)</th>
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<td>Fracture on net section of flange</td>
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<td>Yielding on gross section of flange</td>
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Table 4.4 – Corresponding Forces and Moments for Critical Limit States for Initial Flange Splice Design

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<th>$R_{nf}$ (kN)</th>
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<th>$V_{\text{max}}$ (kN)</th>
<th>$M_{\text{splice}}$ (kN.mm)</th>
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</tr>
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<td>Fracture on net section of flange</td>
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* Based on nominal flange splice plate material strength

** Based on measured flange splice plate material strength
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<th>$P$ (kN)</th>
<th>$V_{\text{max}}$ (kN)</th>
<th>$M_{\text{splice}}$ (kN.mm)</th>
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<tr>
<td>$V = V_n$</td>
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<tr>
<td>Nominal fracture on net section of flange</td>
<td>10,490</td>
<td>6846</td>
<td>3423</td>
<td>16,260,000</td>
</tr>
</tbody>
</table>
Table 4.7 – Corresponding Forces and Moments for Critical Limit States for Revised Flange Splice Design for FST Based on Measured Material Properties.

<table>
<thead>
<tr>
<th>Limit States</th>
<th>( R_{nf} ) (kN)</th>
<th>( P ) (kN)</th>
<th>( V_{max} ) (kN)</th>
<th>( V_{splice} ) (kN)</th>
<th>( M_{splice} ) (kN.mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( V = V_n )</td>
<td>5430</td>
<td>3544</td>
<td>1772</td>
<td>354</td>
<td>8,417,000</td>
</tr>
<tr>
<td>Factored fracture on net section of plates (fnp)</td>
<td>4413</td>
<td>2880</td>
<td>1440</td>
<td>283</td>
<td>6,840,000</td>
</tr>
<tr>
<td>Nominal fracture on net section of plates (fnp)</td>
<td>5515</td>
<td>3599</td>
<td>1780</td>
<td>354</td>
<td>8,548,000</td>
</tr>
</tbody>
</table>

Table 4.8 – Web Shear Forces for Web Splice Designed for SST at Critical Limit States

<table>
<thead>
<tr>
<th>Limit States</th>
<th>( \Phi )</th>
<th>Factored Web Shear (^\ast) (kN)</th>
<th>Nominal Web Shear (^\ast) (kN)</th>
<th>Factored Web Shear (^\ast) (kN)</th>
<th>Nominal Web Shear (^\ast) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolts fail in shear (bs)</td>
<td>0.65</td>
<td>2002</td>
<td>2463</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Plates fail in shear</td>
<td>1.0</td>
<td>7054</td>
<td>7054</td>
<td>8507</td>
<td>8507</td>
</tr>
</tbody>
</table>

\(^\ast\) Based on nominal web splice plate material strength

\(^\ast\) Based on measured web splice plate material strength
Table 4.9 – Corresponding Forces and Moments for Web Splice Design for SST at Critical Limit States

<table>
<thead>
<tr>
<th>Limit States</th>
<th>V&lt;sub&gt;max&lt;/sub&gt; (kN)</th>
<th>P (kN)</th>
<th>M&lt;sub&gt;splice&lt;/sub&gt; (kN.mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V = V&lt;sub&gt;n&lt;/sub&gt;</td>
<td>1772</td>
<td>2953</td>
<td>3,101,000</td>
</tr>
<tr>
<td>Bolts fail in shear (factored)</td>
<td>2002</td>
<td>3337</td>
<td>3,504,000</td>
</tr>
<tr>
<td>V = V&lt;sub&gt;G8A&lt;/sub&gt;</td>
<td>2153</td>
<td>3588</td>
<td>3,768,000</td>
</tr>
<tr>
<td>V = V&lt;sub&gt;G7A&lt;/sub&gt;</td>
<td>2294</td>
<td>3823</td>
<td>4,015,000</td>
</tr>
<tr>
<td>Bolts fail in shear (nominal)</td>
<td>2463</td>
<td>4105</td>
<td>4,310,000</td>
</tr>
</tbody>
</table>
Table 4.10 – Flange Forces for Flange Splice Designed for SST at Critical Limit States

<table>
<thead>
<tr>
<th>Limit States</th>
<th>$\Phi$</th>
<th>$R_{nf}^*$ (kN)</th>
<th>Nominal $R_{nf}^*$ (kN)</th>
<th>Factored $R_{nf}^{**}$ (kN)</th>
<th>Nominal $R_{nf}^{**}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fracture on net section of flange</td>
<td>0.8</td>
<td>8390</td>
<td>10,490</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Yielding on gross section of flange</td>
<td>0.95</td>
<td>10,310</td>
<td>10,850</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Fracture on net section of plates (fnp)</td>
<td>0.8</td>
<td>2387</td>
<td>2983</td>
<td>2648</td>
<td>3309</td>
</tr>
<tr>
<td>Yielding on gross section of plates</td>
<td>0.95</td>
<td>3389</td>
<td>3567</td>
<td>3741</td>
<td>3937</td>
</tr>
</tbody>
</table>

* Based on nominal flange splice plate material strength

** Based on measured flange splice plate material strength
Table 4.11 – Corresponding Forces and Moments for Critical Limit States for Flange Splice Design for SST Based on Measured Material Properties.

<table>
<thead>
<tr>
<th>Limit States</th>
<th>P  (kN)</th>
<th>V  (kN)</th>
<th>$M_{\text{splice}}$ (kN.mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V = V_n$</td>
<td>2953</td>
<td>1772</td>
<td>3,101,000</td>
</tr>
<tr>
<td>$V = V_{G8A}$</td>
<td>3588</td>
<td>2153</td>
<td>3,768,000</td>
</tr>
<tr>
<td>$V = V_{G7A}$</td>
<td>3823</td>
<td>2294</td>
<td>4,015,000</td>
</tr>
<tr>
<td>Factored fracture of flange splice plates on net section (fnp)</td>
<td>3846</td>
<td>2308</td>
<td>4,038,000</td>
</tr>
<tr>
<td>Nominal fracture of flange splice plates on net section (fnp)</td>
<td>4806</td>
<td>2884</td>
<td>5,046,000</td>
</tr>
</tbody>
</table>
Table 4.12. – Web Shear Forces for the Web Splice Designed for the FST at Critical Limit States

<table>
<thead>
<tr>
<th>Limit States</th>
<th>$\Phi$</th>
<th>Factored Web Shear Forces $^*$ (kN)</th>
<th>Nominal Web Shear Forces $^*$ (kN)</th>
<th>Factored Web Shear Forces $^{**}$ (kN)</th>
<th>Nominal Web Shear Forces $^{**}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolts fail-in shear</td>
<td>0.65</td>
<td>1404</td>
<td>1724</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>(bs)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Plates fail-in shear</td>
<td>1.0</td>
<td>7054</td>
<td>7054</td>
<td>8507</td>
<td>8507</td>
</tr>
</tbody>
</table>

$^*$ Based on nominal web splice plate material strength

$^{**}$ Based on measured web splice plate material strength
Table 4.13 Tensile Coupon Data for Splice Plate Steel

<table>
<thead>
<tr>
<th>Coupon ID</th>
<th>Yield Stress (MPa)</th>
<th>Ultimate Stress (MPa)</th>
<th>Ratio (Yield Stress to Ultimate Stress)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coupon 1 – 19 mm</td>
<td>421.5</td>
<td>523.8</td>
<td>0.80</td>
</tr>
<tr>
<td>Coupon 2 – 19 mm</td>
<td>424.1</td>
<td>534.8</td>
<td>0.79</td>
</tr>
<tr>
<td>Coupon 3 – 19 mm</td>
<td>416.4</td>
<td>519.6</td>
<td>0.80</td>
</tr>
<tr>
<td>Average</td>
<td>420.7</td>
<td>526.1</td>
<td>0.80</td>
</tr>
<tr>
<td>Coupon 1 – 32 mm</td>
<td>392.0</td>
<td>510.7</td>
<td>0.77</td>
</tr>
<tr>
<td>Coupon 2 – 32 mm</td>
<td>368.2</td>
<td>485.0</td>
<td>0.76</td>
</tr>
<tr>
<td>Average</td>
<td>380.1</td>
<td>497.9</td>
<td>0.77</td>
</tr>
<tr>
<td>Coupon 1 – 6 mm</td>
<td>394.8</td>
<td>490.0</td>
<td>0.81</td>
</tr>
<tr>
<td>Coupon 2 – 6 mm</td>
<td>444.8</td>
<td>481.0</td>
<td>0.93</td>
</tr>
<tr>
<td>Coupon 3 – 6 mm</td>
<td>390.4</td>
<td>478.9</td>
<td>0.85</td>
</tr>
<tr>
<td>Average</td>
<td>410.0</td>
<td>483.3</td>
<td>0.85</td>
</tr>
</tbody>
</table>
Table 4.14 - Load at Local Yielding of the Splice Plates

<table>
<thead>
<tr>
<th>Location</th>
<th>P (kN)</th>
<th>Splice Plate</th>
<th>Flange</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>2211</td>
<td>Inner - 1</td>
<td>top</td>
</tr>
<tr>
<td>2</td>
<td>2340</td>
<td>Inner - 1</td>
<td>bottom</td>
</tr>
<tr>
<td>5</td>
<td>2395</td>
<td>Outer - 2</td>
<td>bottom</td>
</tr>
<tr>
<td>4</td>
<td>2740</td>
<td>Outer - 2</td>
<td>bottom</td>
</tr>
<tr>
<td>7</td>
<td>2765</td>
<td>Outer - 2</td>
<td>bottom</td>
</tr>
<tr>
<td>16</td>
<td>2778</td>
<td>Inner - 3</td>
<td>top</td>
</tr>
<tr>
<td>6</td>
<td>2796</td>
<td>Outer - 2</td>
<td>bottom</td>
</tr>
<tr>
<td>8</td>
<td>2808</td>
<td>Outer - 2</td>
<td>bottom</td>
</tr>
<tr>
<td>9</td>
<td>2991</td>
<td>Outer - 2</td>
<td>bottom</td>
</tr>
<tr>
<td>14</td>
<td>3045</td>
<td>Inner - 3</td>
<td>bottom</td>
</tr>
<tr>
<td>13</td>
<td>3054</td>
<td>Outer - 2</td>
<td>bottom</td>
</tr>
<tr>
<td>12</td>
<td>3089</td>
<td>Outer - 2</td>
<td>top</td>
</tr>
<tr>
<td>12</td>
<td>3115</td>
<td>Outer - 2</td>
<td>bottom</td>
</tr>
<tr>
<td>10</td>
<td>3132</td>
<td>Outer - 2</td>
<td>bottom</td>
</tr>
<tr>
<td>5</td>
<td>3142</td>
<td>Outer - 2</td>
<td>top</td>
</tr>
<tr>
<td>11</td>
<td>3150</td>
<td>Outer - 2</td>
<td>bottom</td>
</tr>
<tr>
<td>11</td>
<td>3189</td>
<td>Outer - 2</td>
<td>top</td>
</tr>
<tr>
<td>1</td>
<td>3201</td>
<td>Inner - 1</td>
<td>bottom</td>
</tr>
<tr>
<td>4</td>
<td>3288</td>
<td>Outer - 2</td>
<td>top</td>
</tr>
<tr>
<td>9</td>
<td>3389</td>
<td>Outer - 2</td>
<td>top</td>
</tr>
<tr>
<td>10</td>
<td>3417</td>
<td>Outer - 2</td>
<td>top</td>
</tr>
<tr>
<td>8</td>
<td>3401</td>
<td>Outer - 2</td>
<td>top</td>
</tr>
<tr>
<td>3</td>
<td>3492</td>
<td>Inner - 1</td>
<td>bottom</td>
</tr>
<tr>
<td>6</td>
<td>3496</td>
<td>Outer - 2</td>
<td>top</td>
</tr>
<tr>
<td>7</td>
<td>3529</td>
<td>Outer - 2</td>
<td>top</td>
</tr>
<tr>
<td>15</td>
<td>3557</td>
<td>Inner - 3</td>
<td>bottom</td>
</tr>
<tr>
<td>16</td>
<td>3568</td>
<td>Inner - 3</td>
<td>bottom</td>
</tr>
<tr>
<td>13</td>
<td>3589</td>
<td>Outer - 2</td>
<td>top</td>
</tr>
<tr>
<td>15</td>
<td>3608</td>
<td>Inner - 3</td>
<td>top</td>
</tr>
<tr>
<td>14</td>
<td>3631</td>
<td>Inner - 3</td>
<td>top</td>
</tr>
<tr>
<td>1</td>
<td>did not yield Inner - 1</td>
<td>top</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>did not yield Inner - 1</td>
<td>top</td>
<td></td>
</tr>
</tbody>
</table>
### Table 4.15 Load at Yielding of Cross Sections of Splice Plates

<table>
<thead>
<tr>
<th>Event</th>
<th>Load (kN)</th>
<th>Flange</th>
<th>Splice Plate</th>
<th>Location</th>
<th>Section Yielding</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3089</td>
<td>bottom</td>
<td>outer</td>
<td>at splice</td>
<td>gross</td>
</tr>
<tr>
<td>2</td>
<td>3132</td>
<td>bottom</td>
<td>outer</td>
<td>east</td>
<td>net</td>
</tr>
<tr>
<td>3</td>
<td>3142</td>
<td>top</td>
<td>outer</td>
<td>at splice</td>
<td>gross</td>
</tr>
<tr>
<td>4</td>
<td>3150</td>
<td>bottom</td>
<td>outer</td>
<td>west</td>
<td>net</td>
</tr>
<tr>
<td>5</td>
<td>3201</td>
<td>bottom</td>
<td>inner</td>
<td>west</td>
<td>net</td>
</tr>
<tr>
<td>6</td>
<td>3529</td>
<td>top</td>
<td>outer</td>
<td>west</td>
<td>net</td>
</tr>
<tr>
<td>7</td>
<td>3557</td>
<td>bottom</td>
<td>inner</td>
<td>at splice</td>
<td>gross</td>
</tr>
<tr>
<td>8</td>
<td>3568</td>
<td>bottom</td>
<td>inner</td>
<td>east</td>
<td>net</td>
</tr>
<tr>
<td>9</td>
<td>3589</td>
<td>top</td>
<td>outer</td>
<td>east</td>
<td>net</td>
</tr>
<tr>
<td>10</td>
<td>3608</td>
<td>top</td>
<td>inner</td>
<td>at splice</td>
<td>gross</td>
</tr>
</tbody>
</table>

*Note that “Section Yielding” is determined from strain gages on one surface of the plate, and bending of the splice plate is neglected.*
### Table 4.16 Relationship Between $P$, $M_{\text{splice}}$, $R_{\text{nf}}$, FOP and FIP

<table>
<thead>
<tr>
<th>Limit States</th>
<th>$P$ (kN)</th>
<th>$M_{\text{splice}}$ (kN.mm)</th>
<th>$R_{\text{nf}}$ (kN)</th>
<th>$\frac{F_{\text{OP}}}{R_{\text{nf}}}$</th>
<th>$\frac{2F_{\text{IP}}}{R_{\text{nf}}}$</th>
<th>$F_{\text{OP}}$ (kN)</th>
<th>$2F_{\text{IP}}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outer plate at yield on the net section</td>
<td>3184</td>
<td>7,651,000</td>
<td>4878</td>
<td>0.544</td>
<td>0.456</td>
<td>2654</td>
<td>2224</td>
</tr>
<tr>
<td>Inner plate at yield on the net section</td>
<td>2744</td>
<td>6,516,000</td>
<td>4204</td>
<td>0.544</td>
<td>0.456</td>
<td>2287</td>
<td>1917</td>
</tr>
<tr>
<td>Inner and outer plate at yield at the same time</td>
<td>2983</td>
<td>7,085,000</td>
<td>4571</td>
<td>0.581</td>
<td>0.419</td>
<td>2654</td>
<td>1917</td>
</tr>
<tr>
<td>Event 2 - based on distribution of $R_{nf}$</td>
<td>3132</td>
<td>7,439,000</td>
<td>4799</td>
<td>0.544</td>
<td>0.456</td>
<td>2611</td>
<td>2188</td>
</tr>
<tr>
<td>Event 2 - outer plate at yield on net section</td>
<td>3132</td>
<td>7,439,000</td>
<td>4799</td>
<td>0.553</td>
<td>0.447</td>
<td>2654</td>
<td>2145</td>
</tr>
<tr>
<td>Event 2 - outer plate 50% at $F_y$ 50% at strain</td>
<td>3132</td>
<td>7,439,000</td>
<td>4799</td>
<td>0.612</td>
<td>0.388</td>
<td>2939</td>
<td>1860</td>
</tr>
<tr>
<td>Event 4 - based on distribution of $R_{nf}$</td>
<td>3150</td>
<td>7,481,000</td>
<td>4827</td>
<td>0.544</td>
<td>0.456</td>
<td>2626</td>
<td>2201</td>
</tr>
<tr>
<td>Event 4 - outer plate at yield on net section</td>
<td>3150</td>
<td>7,481,000</td>
<td>4827</td>
<td>0.550</td>
<td>0.450</td>
<td>2654</td>
<td>2173</td>
</tr>
<tr>
<td>Event 4 - outer plate 50% at $F_y$ 50% at strain</td>
<td>3150</td>
<td>7,481,000</td>
<td>4827</td>
<td>0.611</td>
<td>0.389</td>
<td>2948</td>
<td>1879</td>
</tr>
<tr>
<td>Outer plate at fracture on net section</td>
<td>3982</td>
<td>9,457,000</td>
<td>6101</td>
<td>0.544</td>
<td>0.456</td>
<td>3319</td>
<td>2782</td>
</tr>
<tr>
<td>Inner plate at fracture on net section</td>
<td>3599</td>
<td>8,548,000</td>
<td>5515</td>
<td>0.544</td>
<td>0.456</td>
<td>3004</td>
<td>2511</td>
</tr>
<tr>
<td>Inner and outer plate fracture at the same time</td>
<td>3805</td>
<td>9,037,000</td>
<td>5830</td>
<td>0.569</td>
<td>0.431</td>
<td>3319</td>
<td>2511</td>
</tr>
</tbody>
</table>
Figure 4.17 Loads and Strains at Given Locations

<table>
<thead>
<tr>
<th>P (kN)</th>
<th>Locations</th>
<th>Strains (µε)</th>
<th>State</th>
</tr>
</thead>
<tbody>
<tr>
<td>3132</td>
<td>3</td>
<td>1320</td>
<td>Below yield</td>
</tr>
<tr>
<td>3132</td>
<td>6</td>
<td>9389</td>
<td>Strain hardening</td>
</tr>
<tr>
<td>3132</td>
<td>8</td>
<td>8093</td>
<td>Strain hardening</td>
</tr>
<tr>
<td>3132</td>
<td>10</td>
<td>2105</td>
<td>At yield</td>
</tr>
<tr>
<td>3132</td>
<td>13</td>
<td>2863</td>
<td>At yield</td>
</tr>
<tr>
<td>3132</td>
<td>16</td>
<td>1287</td>
<td>Below yield</td>
</tr>
<tr>
<td>3150</td>
<td>1</td>
<td>1749</td>
<td>Below yield</td>
</tr>
<tr>
<td>3150</td>
<td>4</td>
<td>8728</td>
<td>Strain hardening</td>
</tr>
<tr>
<td>3150</td>
<td>7</td>
<td>9900</td>
<td>Strain hardening</td>
</tr>
<tr>
<td>3150</td>
<td>9</td>
<td>5099</td>
<td>Strain hardening</td>
</tr>
<tr>
<td>3150</td>
<td>11</td>
<td>2105</td>
<td>At yield</td>
</tr>
<tr>
<td>3150</td>
<td>14</td>
<td>2004</td>
<td>Below yield</td>
</tr>
</tbody>
</table>

Table 4.18 Load and Strains at which Strain Reversal Occurred at Given Strain Gage Locations.

<table>
<thead>
<tr>
<th>Locations</th>
<th>Load (kN)</th>
<th>Strains (µε)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>3230</td>
<td>14,083</td>
</tr>
<tr>
<td>8</td>
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CHAPTER 5 - SUMMARY AND CONCLUSIONS

The research described in this thesis is part of the Pennsylvania Innovative High-Performance Steel Bridge Demonstration Project. It is sponsored by the Pennsylvania Department of Transportation. This project investigates the use of high performance steel (HPS) in the design of I-girder highway bridges. By using innovative bridge member configurations, such as girders with corrugated webs, the potential of advantages HPS can be more fully utilized. The research presented in this thesis focuses corrugated web bridge I-girders, in particular, the design and behavior of bearing stiffeners and bolted web and flange field splices for corrugated web I-girders. The results of this research are summarized in this chapter.

Bearing stiffener specimens (Test Specimen 1 and Test Specimen 2) were cut from a pre-existing corrugated web I-girder fatigue test specimen that had been tested at Lehigh University. The bearing stiffeners of Test Specimen 1 and Test Specimen 2 were cut from the west and the east end of this corrugated web I-girder, respectively. The major difference in Test Specimen 1 and Test Specimen 2 is the amount of web that was included in the specimen. The total length of web included in Test Specimen 1 of 900 mm (i.e. 150 times the thickness of the web) was twice the length included in Test Specimen 2.

To determine the nominal axial capacity of bearing stiffeners of corrugated web I-girders, three methods of calculation were investigated: the column buckling formula (AASHTO LRFD specifications, (AASHTO 1998)), the Euler column buckling formula
and the plastic capacity formula. Tests were conducted and the results were compared with the calculated nominal axial capacity.

Test Specimen 1 and Test Specimen 2 were tested in compression. The ultimate load for Test Specimen 1 was 2915 kN, while that of Test Specimen 2 was 2695 kN. The results show that the ultimate strength of Test Specimen 1 was 7.6% greater than that obtained for Test Specimen 2. However, the increase in the ultimate strength was not proportional to the increased length of the web. The nominal axial resistance calculated from the plastic capacity formula by assuming that the full length of web of Test Specimen 2 (450 mm) contributed to the strength of the bearing stiffener was 2682 kN. This value was 0.4% less than the ultimate strength of Test Specimen 2. Therefore, the ultimate capacity of a bearing stiffener with the stiffener plates centered on the incline of the fold can be determined from the plastic capacity of the section which includes the stiffener plates and a web length of 75 times the thickness of the web.

The nominal axial capacity of the bearing stiffeners of corrugated web I-girders can be determined from the limit of linear elastic behavior of the bearing stiffener when loaded in compression. This limit is approximately 50% of the ultimate strength of the bearing stiffener, which is equivalent to the nominal axial capacity of the bearing stiffener calculated from the column buckling formula for a length of web equivalent to 24 times the thickness of the web.

According to the AASHTO LRFD specifications, the strength of a flat web I-girder is determined from the column buckling formula and the length of web included in the strength of the bearing stiffener is 18 times the thickness of the web. Therefore, the
nominal axial resistance of a bearing stiffener of a corrugated web I-girder will be greater than that of a conventional flat web I-girder with identical stiffener plate parameters.

Two separate tests of flange and web splices were conducted. The two tests were conducted sequentially using a single test specimen. In the first test, the highest possible bending moment was generated in the flanges at the splice location (the Flexural Strength Test). In the second test, a large shear force developed in the web at the splice location (the Shear Strength Test). The FST had to be completed without structurally damaging the test specimen components. The critical limit states to avoid during the FST were shear failure of the web and flexural failure of the flanges.

The flange splice was designed to satisfy both service and strength requirements according to the AASHTO LRFD specifications (AASHTO 1998). The splice was designed using A 325 - 22 mm diameter bolts. However, not all the bolts passed through both the inner and outer flange splice plates. As a result, the splice plates were designed using an assumed distribution of flange force between the inner and outer plates. The ratio of flange force distributed to the inner and outer splice plate is 0.544 and 0.456, respectively. The splice plates were designed with a nominal yield stress of 345 MPa. The outer and inner splice plates had a nominal thickness of 19 mm and 32 mm, respectively.

The flange splice plates designed for the FST were designed to fail by fracture of the net section during testing before the onset of structural damage to the main components of the test specimen. The flange splice plates were expected to fail before a load of 3805 kN was reached. However, this failure did not occur during the FST, and
the test was stopped at a load of 3852 kN, before fracture of the net section of the flange splice plates was observed. Therefore, the actual strength of the flange splice exceeded the expected strength from design based on measured material properties.

The web splice was designed according to the AASHTO LRFD specifications (AASHTO 1998) except as follows: the web splice was designed with one row of bolts on each side of the splice and the bolt tightening clearances of the AASHTO specifications were not satisfied. The splice was designed using A 325 - 22 mm diameter bolts. The splice plates were designed with a nominal yield strength of 345 MPa. The web splice plates had a nominal thickness of 6 mm.

The SST test specimen failed by web buckling and the web splice design was shown to be adequate. The peak load developed during testing was, \( P = 3352 \text{ kN} \) and the maximum shear that developed in the web was 2011 kN, which exceeded the calculated shear capacity. However, the shear strength of two similar corrugated web I-girder specimens tested previously at Lehigh University was 12% and 6% greater than that of the SST test specimen.

According to available shear strength design criteria for corrugated webs, summarized in Chapter 2, the nominal shear resistance of a corrugated web I-girder is limited to 70.7% of the shear yield capacity of the web. The SST test specimen failed at 79.7% of the shear yield capacity, while, the two previously tested specimens failed at 90.7% and 85.3% of the shear yield capacity, respectively. These differences are attributed to the magnitude and distribution of imperfections that can be associated with corrugated webs.
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END OF TITLE