STRENGTHENING T-JOINTS OF RECTANGULAR HOLLOW STEEL SECTIONS USING THROUGH-WALL BOLTS AND EXTERNALLY BONDED FRP PLATES

By

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Abstract

T-joints are common in beam-column connections of steel frames, vierendeel girders and at mid-span of N-trusses. Strengthening the members of these structures increases the demand on the joints, which may require joint strengthening. This thesis examines different strengthening techniques of T-joints of RHS members. In Phase I, the effectiveness of through-wall steel bolts is examined. This is accomplished by controlling the web outward buckling of the chord under the brace axial load. The study examined the effect of the number and pattern of bolts, as well as the web height-to-wall thickness (h/t) ratio of the chord, on strengthening effectiveness. Rectangular 203x76x(3.09, 4.5, and 5.92) mm chord members were tested. The 8 mm diameter steel bolts varied from a single bolt to 15 bolts of various distributions. The joint strength increased by 3.1%, 6.2%, and 29% for chords with (h/t) of 34, 45, and 65, respectively. The number and distribution of bolts had little effect on their effectiveness.

In Phase II, similar T-joint specimens were strengthened using adhesively bonded GFRP plates, 9.5 mm thick, of different configurations, and 2 mm thick high-modulus CFRP plates of equivalent stiffness. It was shown that strength gain increases significantly, from 9% to 38%, as (h/t) ratio of the HSS chord increases from 34 to 65. In thin-walled HSS (h/t = 65), retrofitting provided significant gains in strength but not in ductility. In thick-walled HSS (h/t = 34), retrofitting provided little strength gain, but enhanced ductility, especially with properly bonded plates extending on the brace. Generally, plates fractured under local bending or delaminated within plate layers while bond was fully intact.
In Phase III, selected configurations of the two retrofitting methods were used in additional T-joints with chord (h/t) ratio of 65, to study their effectiveness in presence of axial compression load in the chord. Two sustained load levels were induced in the chord, representing 45% and 80% of its full axial capacity. The transverse brace load was then gradually increased to failure. The through-wall steel bolts increased the joint capacity by 13% to 25%, depending on the chord’s axial load level, while the bonded GFRP plate increased the capacity by 38 to 46%.
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Chapter 1

Introduction

1.1 GENERAL

The Vierendeel truss was first developed by Arthur Vierendeel in 1896. Unlike conventional trusses with triangular voids and pin connections designed to take axial forces, this system was developed to resist bending forces at the joints, along with axial forces for the members. The Vierendeel truss is designed with chord members connected to bracing members, typically at 90° to one another (Figure 1.1). This type of connection is also known as a T-joint. For a Vierendeel truss with an even number of cells, which has the applied load at every joint, the middle brace does not exhibit bending moments, only axial forces. As expected the top chord is in compression while the bottom chord is in tension. In many cases, vierendeel truss members are made of steel Rectangular Hollow Sections (RHS). The high concentration force of the brace can cause the top chord RHS sidewalls to buckle near the joint. Since T-joints are commonly used in beam-to-column connections the load capacity of the structure may be governed by the strength of its joint, especially when thin-walled sections are used.

When structures are in need of upgrade to carry larger loads, the cost of retrofitting is typically far less than replacement. Retrofits require less time, thus reducing service interruption time. Typical repair methods of steel structures, particularly T-joints, may require bolting or welding steel heavy plates (Figure 1.2) or ‘cans’ (Figure 1.3) to existing structures. The use of ‘doubler’ or ‘collar’ plates for reinforcing T-joints has
been found to significantly strengthen the joint (Choo et al., 1998, Figure 1.4). The increase in dead load due to these techniques may limit the increase in live load carrying capacity. Furthermore, a reduction in fatigue life due to welding steel plates may reduce the durability of the structure. The use of Fibre Reinforced Polymers (FRP) can be an alternative. Although expensive compared to steel, it has quick installation time and adds relatively little weight to the structure. FRP can provide superior strength for retrofits, with little impact on aesthetic appearance. FRP is available in the form of sheet or plates. It can provide increased flexural strength, shear resistance, axial strength, and ductility.

The use of both Glass Fibre Reinforced Polymers (GFRP) and Carbon Fibre Reinforced Polymers (CFRP) is increasingly being used in retrofitting concrete bridges and structures. Due to the inherent high strength and stiffness of steel compared to that of concrete, it becomes more challenging to strengthen steel. The use of a low Young’s modulus material, used to strengthen steel may result in the new material taking on the loads only after the steel yields. It may therefore be more desirable to use high modulus CFRP instead of a low modulus GFRP. However, for thin structures (Class 3 and 4) the added stiffness brought on by GFRP may be sufficient.

Although Class 4 sections are not commonly used, they are occasionally used due to their lightness and efficiency. Furthermore, a deteriorating or corroding member of higher thickness may drop from a higher Class to a Class 4, due to a reduction in thickness.
1.2 RESEARCH OBJECTIVES

The experimental research program carried out in this thesis explores the retrofit of RHS T-joints against local crippling. Two types of retrofit options are examined, for a T-joint under brace axial loads only, using through bolts and FRP. Later, selected configurations of each retrofitting technique are used to test T-joint specimens with various axial loads in the chord also. The main topics addressed by this study are:

1. Assessment of the capacity of through-bolts, of various patterns, to resist outward local crippling of the chord sidewalls due to transverse loads applied to the chord through the brace. This is studied for chords with and without axial compression loads of various levels.

2. Assessment of the capacity of externally bonded GFRP and CFRP plates of various sizes and shapes to resist outward local crippling of the chord sidewalls due to transverse loads applied to the chord through the brace. This is also studied for chords with and without axial compression loads.

3. Effect of wall thickness of RHS on strengthening effectiveness.

4. Examining the various failure modes of RHS chord.

1.3 SCOPE

The scope of this study includes experimental investigation of RHS chord members reinforced with steel through-bolts or bonded FRP plates when subjected to transverse loads, with the chord being axially loaded, or unloaded.

The experimental investigation was intended to assess the feasibility of using through-bolts and FRP for reinforcing RHS webs against buckling. Thirty specimens
were tested in total. Fourteen specimens were tested in the first phase of testing looking at the effects of through-bolts as a means of reinforcing RHS chords. Ten specimens were tested in the second phase to look at the effects of using FRP to strengthen RHS chords. In the third phase six specimens were tested to see the effects of axial compression loads in the chord on the effectiveness of both strengthening techniques. The tests for phase one and two aimed to establish the most effective configurations of reinforcement which were then used in phase three. T-joints tested in phases one and two without axial load in the chord represents beam-column connections in frames, while T-joints tested in phase three with axial loads in the chord represents vierendeel and trussed girders.

1.4 OUTLINE OF THESIS
A manuscript format has been selected for this thesis. As such, the relevant literature review to each chapter is provided in the introduction of the chapter. The following briefly describes the contents of this thesis:

Chapter 2: Presents the first study (Phase 1) on the strengthening of T-joints in thin-walled structures using through bolts. Only the brace is axially loaded in those tests.

Chapter 3: Presents the second study (Phase 2) on the strengthening of T-joints in thin-walled structures using FRP. Only the brace is axially loaded in those tests.

Chapter 4: Presents the third study (Phase 3) on the strengthening of T-joints in thin-walled structures using selected through-bolt and FRP configurations from chapters 2 and 3. However, both the brace and chord are axially loaded.
Chapter 5: Presents the conclusions found from all three studies.

References

Appendix: Presents the calculations and analysis involved in design of the test specimens.
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Chapter 2

Strengthening T-Joints of Rectangular Hollow Steel Sections against Web Crippling under Brace Axial Compression using Through-Wall Bolts

2.1 INTRODUCTION

Rectangular Hollow Steel Sections (HSS) are commonly used in trusses, vierendeel girders, and frames. The connection between the chord and vertical member (brace) in vierendeel girders and at mid-span of N-trusses, or between the beam and column in frames, take the form of a T-joint. Increasing the strength of joints may be essential in certain structures, especially if the individual members have been strengthened. This becomes even more crucial in thin-walled members. In this case, the bearing of the vertical member, or brace, on the chord produces web crippling of the thin walls of the chord. Unlike W- and S-sections, it is not possible to install web stiffeners inside the HSS sections. As such, mitigation of web crippling must be through external means.

The most commonly applied concept in the offshore industry is the use of a ‘can’ whereby the chord members comprised of Circular Hollow Sections (CHS) are partially thickened at the joints. Cans are usually incorporated in the design stage of the structure. However, when last-minute adjustments are required to provide additional local joint stiffening, alternative approaches may be considered (e.g. ring stiffeners, diaphragms, or grout). Another type of reinforcement, which can be used in various components in

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offshore platforms, is referred to as the ‘doubler plate’ reinforced joint. For a T-joint reinforced with a doubler plate, the brace is welded directly to the plate through a penetration weld, whereas the doubler plate is fillet welded to the chord. A slightly different scheme, referred to as a ‘collar’ may be suitable to provide reinforcement to prefabricated joints that are found to be under-designed (Choo et al., 1998). It should be noted that all these strengthening techniques are suitable for CHS commonly used in offshore structures. For Rectangular Hollow Sections (RHS), however, different strengthening techniques may be employed. Failure of the chord sidewalls by yielding or crippling is the most common for T-, Y-, and X-joints, especially when their $\beta$ ratios (breadth of brace/breadth of chord) are close or equal to unity. Filling hollow sections with concrete to improve the web crippling behaviour was found to be very efficient (Packer, 1995). Another technique was adopted (Bains, 1983, and Bradfield et al., 1994), where one bolt was used to brace the chord sidewalls, at the vicinity of the joint, against the outward crippling, and resulted in an increase of 18% in joint capacity. Another study (Zhao, 1999) recommended that a wooden brace be inserted into the RHS in addition to the through-bolt to avoid the inward crippling of the chord sidewalls.

This chapter investigates the effect of bracing the sidewalls of thin-walled RHS chord members of T-joints using various patterns and numbers of through-wall bolts. Another important parameter studied is the effectiveness of this method for various web slenderness ratios of the RHS chord, in terms of its height-to-thickness ($h/t$) ratio. The T-joints were subjected to axial compression loads applied to the brace.
2.2 EXPERIMENTAL PROGRAM

In the following sections, test specimens and parameters, material properties, fabrication of specimens, test setup and instrumentation are described in details.

2.2.1 Test Specimens and Parameters

Table 2.1 provides a summary of the test matrix. In total, fourteen T-joints were fabricated and tested under brace axial compression load. The T-joint consisted of a horizontal, 1220 mm long chord member welded to a 400 mm long brace member. Two key parameters were explored, namely, the wall thickness of chord member, and the pattern of the through-wall bolts. The chord member was a 203x76xt HSS, where the wall thickness (t) was a variable, namely, 3.09 mm (Class 4) in the first group (specimens T1 to T6), 4.5 mm (Class 4) in the second group (T7 to T12), and 5.92 mm (Class 3) in the third group (T13 and T14). The different wall thicknesses of the chord correspond to a web height-to-thickness ratio (h/t) of 65, 45, and 34, respectively. The brace member was a 76x76x8.9 mm HSS (Class 1) with a thick-wall to avoid failure of the brace. Control specimens were tested in each group of a given wall thickness, including specimen T1 in the first group, T7 to T9 in the second group, and T13 in the third group. The second group was selected to check for repeatability by testing triplets (T7 to T9) of control specimens and triplets of strengthened specimens (T10 to T12). The effect of wall thickness was studied by comparing specimens T2, (T10 to T12) and T14, all with the same pattern and number of bolts.

The two sidewalls of the chord member of each strengthened specimen were connected together using a variety of symmetrical bolt patterns in the vicinity of the
brace, as shown in Figure 2.1. Five different patterns (A to E) were examined for the first group (T2 to T6). Configuration A includes 15 bolts in three arrays below the brace member, within a 40 x 40 mm grid. This arrangement covers the critical region below the brace and likely provides the maximum possible potential of this method. Patterns B to D represent a systematic attempt to reduce the number and optimize the locations of the bolts from 15 bolts to a single bolt. In this process, selection of patterns and locations was guided by the location of the maximum outwards deflection (bulging) of the sidewalls of the chord member in control specimens, which was shown to occur 40 mm below the top flange of the chord. The effect of patterns A to D was studied using specimens T2 to T6, all with the same chord wall thickness.

2.2.2 Material Properties

2.2.2.1 Cold-formed HSS sections: Two types of HSS sections were used, a rectangular one (RHS) of three different wall thicknesses, 203x76x(3.09, 4.5 and 5.92) mm, for the chord, and a square (SHS), 76x76x8.9 mm for the brace. Both the RHS and SHS were manufactured in accordance with ASTM A500 C (2009). Uniaxial tension tests were performed according to ASTM E8/E8M-09 (2009) on six dog-bone coupons. The coupons were 200 mm long overall, with a grip width and length of 20 mm and 50 mm, respectively. A 12.5 mm thickness was used as the width in the gage length. Two coupons were cut for each chord thickness, one from the flange and one from the web. A 50 mm extensometer was used to measure and record strains. The stress-strain plots for all six steel coupons are shown in Figure 2.2. The yield strengths (offset secant at 0.2%) of flanges were 426, 423, and 449 MPa, for the 3.09, 4.5, and 5.92 mm thicknesses,
respectively, while yield strengths of the webs were 427, 438, and 443 MPa, respectively. The modulus of elasticity of both flange and web was 209 GPa. The reported yield strengths by manufacturer were 410, 430, and 389 for the three thicknesses, respectively.

2.2.2.2 Through-wall bolts: The bolts used in the strengthened specimens are standard 8 mm diameter (Grade 8) high-strength bolts with a reported ultimate strength of 1034 MPa.

2.2.3 Fabrication of T-Joint Specimens

The lengths of the chord and brace members were cut to 1220, and 400 mm, respectively. The SHS brace member was directly welded to the flange of the RHS chord member at mid-length. Cutting and welding of specimens were performed by a professional, at machine shop. The holes necessary for the bolts to pass through the chord were then hand drilled in both webs (Figure 2.1). The 8 mm Grade 8 high-strength headed bolts were then inserted into the holes and anchored from one side using a special washer and nut for a snug fit. Special care was taken in order not to over tighten the nut and cause inward crippling of the two webs. Vertical stiffener plates, 110x191x12 mm, were inserted inside the chord member at both ends (Figure 2.3) to prevent premature failure due to crippling at supports.

2.2.4 Test Setup and Instrumentation

The specimens were tested under brace concentric loading using a 1000 kN Riehle testing machine (Figure 2.3). The load was applied using stroke control at a rate of 1 mm/min up
to failure. The chord member was clamped at both ends using a special assembly of heavy SHS sections and threaded rods. First, the specimen was rested on two 150x150x12 mm HSS supports. The two supports were set apart to provide a clear span (L) of 1000 mm, which is almost five times the chord depth (h) of 203 mm. Another two SHS sections were set on top of the specimens ends. The two upper SHSs were held down and anchored to the Riehle testing machine using two vertical 25.4 mm diameter threaded rods at each support. The threaded rods were evenly hand tightened using wrenches.

This span-to-depth (L/h) ratio of the chord was carefully selected as per recommendations in the literature. On one hand, the UK Department of Energy Offshore Technology Report (1990) and other researchers (Moffat et al., 1996) suggested that (L/h) ratio should not be less than four, in order to avoid any effect of supports on the joint strength. On the other hand, others (Lalani, 1992) indicated that the (L/h) ratio should not be excessive, otherwise chord failure may occur prior to joint failure as the plastic moment of the chord cross-section at the crown location is reached. In fact, some researchers (Madros et al., 1995) indicated a limit of 5.75 for (L/h) to ensure joint failure in a simply supported chord. In this study, the (L/h) ratio of 5 was used to avoid the influence of supports on joint strength. Some end fixities were provided by clamping, to increase the load at which yield and plastic moments occur, in order to focus on the joint strength.

Two 100 mm linear potentiometers (LPs) were mounted at the upper and lower flanges of the chord member at mid-span, to measure the vertical deflection of the top and bottom flanges independently (Figure 2.3). These two deflections may differ as the
chord sidewalls buckle. The strains on the chord, in two directions, and on some through-wall bolts, were measured using 5 mm electric resistance strain gauges. The load was measured using a load cell built-in within the Riehle machine. Figure 2.4 shows the various locations of strain gauges on the chord.

2.3 RESULTS OF THE EXPERIMENTAL PROGRAM

Table 2.1 provides a summary of the experimental results, in terms of the maximum load achieved and the percentage of gain in strength of retrofitted specimens relative to their control counterparts. Figure 2.5 provides the load-deflection responses of various specimens. Figures 2.6 to 2.8 show the load-strain responses. Figure 2.9 summarizes the findings in terms of variation of percentage of gain in strength with (h/t) ratio, while Figure 2.10 demonstrates the effect of bolt configurations on the gain in strength. Figure 2.11 shows the various failure modes.

It was important to evaluate the repeatability and the level of variation of test results. This is particularly important in thin-walled tubular structures vulnerable to stability failure. Because specimens may vary in their geometric imperfections, which in this case is primarily the out-of-straightness of the webs of the chord, the load at which local instability occurs may also vary. Specimens T7, T8 and T9 were all of similar (h/t) ratio, and were used to check reproducibility of results for control specimens, while T10, T11 and T12 were all strengthened with pattern A (15 bolts). The measured maximum loads in all specimens are reported in Table 2.1. The average, standard deviation (SD) and coefficient of variation (CV) for control specimens were 280.7 kN, 9.5 kN and 3.38%, respectively, while the average, SD and CV for strengthened specimens were
298 kN, 3.6 kN and 1.21%, respectively. It was concluded that variations within results are small with respect to the differences between types of specimens. The following sections describe in details the test results and effect of various parameters on performance.

2.3.1 Load-Deflection Behaviour

Figure 2.5 shows the load-deflection responses of all test specimens. Generally, the load ascends almost linearly initially, followed by a non-linear behaviour of various degrees of nonlinearity, depending on (h/t) ratio of the web, until a peak load is reached and then a descending response can be observed. This behaviour is similar for both control and retrofitted specimens, except that the peak load is higher for retrofitted specimens. It can be also noticed that, while the retrofitting system enhances strength, it has virtually no effect on the initial stiffness nor does it influence the rate of post-peak descent. The peak load consistently corresponds to instability failure of the webs of the chord. For each specimen, the deflection of the top and bottom flanges of the chord differed slightly at any given load, as a result of the out-of-plane displacements of the vertical webs. This effect becomes more pronounced near the peak load and even more in the descending part of the response.

2.3.2 Load-Strain Behaviour

By examining the extreme fibre strains of the chord at mid-span (SG4 and SG5 in Figure 2.4) relative to the yield strain of 4200 micro-strain (corresponding to 0.2% offset secant in Figure 2.2), it can be seen that in specimens T1 to T6 with a 3.09 mm thick web
(Figure 2.6), the peak load was reached, and hence stability failure, before yielding of the chord in the vicinity of the joint. The one exception was specimen T3, where the bottom fibres slightly exceeded yielding. It can be concluded that for this (h/t) ratio, retrofitting enhanced strength significantly as will be discussed later, but did not allow the chord to develop yielding or any plasticity.

In control specimen T7 with a 4.5 mm thick web (Figure 2.7(a)), it can be seen that yielding and joint failure occurred almost simultaneously, whereas in retrofitted specimens T10 to T12 (Figure 2.7(b, c, d)), the chord yielded first before reaching the peak load when joint stability failure occurred. It can be concluded that for this (h/t) ratio, retrofitting allowed the section to exceed yielding and achieve partial plasticity. This partial plasticity is manifested in a more pronounced nonlinear response before the peak load (Figure 2.5(b)), compared to the specimens with 3.09 mm wall thickness (Figure 2.5(a)).

In both control and retrofitted specimens T13 and T14 with 5.92 mm thick web (Figure 2.8), it can be seen that the chord member has yielded well before joint failure; however, strengthening allowed the section to achieve more plasticity before the peak load, and indeed more ductility, despite the fact that the gain in strength was quite modest. This is reflected in the significant non-linear responses and extended plastic region in Figure 2.5(c).

In control specimen T1 with 3.09 mm wall, SG2 and SG6 in the vertical direction of the web near the location of maximum amplitude of crippling (Figure 2.4) show almost zero strain (Figure 2.6(a)), as would be expected, until about 70% of the peak load, when a pronounced tensile strain develops rapidly, suggesting the onset of outwards
web crippling. Similar responses of SG2 and SG6 can be observed for specimens T7 with 4.5 mm wall (Figure 2.7(a)) and T14 with 5.92 mm wall (Figure 2.8(a)), but at about 78% and 86% of their respective peak loads.

The strains developed in the steel bolts based on SGB, SGC and SGC (Figure 2.4) are generally small and did not exceed 3000 micro-strains (Figs. 2.6 to 2.8), which is well below the yield strain of the bolt. This is attributed to the size of the bolt relative to the force exerted.

2.3.3 Effect of Web Height-to-Thickness (h/t) Ratio of Chord

The effect of (h/t) ratio of chord on strengthening effectiveness using through-wall bolts can be studied by comparing specimens T1 to T2 (for h/t = 65), the average of T7, T8 and T9 to that of T10, T11 and T12 (for h/t = 45) and T13 to T14 (for h/t = 34). All retrofitted specimens have pattern A of 15 bolts. Table 2.1 shows that the percentage gain in strength increases from 3.1% to 29% as (h/t) increases from 34 to 65, which is also plotted in Figure 2.9.

2.3.4 Effect of Number and Pattern of Bolts

The effect of pattern and number of bolts was studied for specimens with (h/t) ratios of 65 since they were slender enough to benefit sufficiently from this strengthening technique. These specimens are T2 to T6 with different patterns A to D (Table 2.1 and Figure 2.1). In order to quantify the different patterns, a parameter (k) is introduced as the ratio of ‘total effective area’ (A) of all bolts located in the compression side (above the neutral axis which is assumed at h/2) to the location of the centroid of this effective area.
(y), measured from the extreme compression fibre. This concept is illustrated in Figure 2.10. The effective area of a single bolt is approximated as a diamond shape of 40x40 mm, analogous to the elliptical crippling shape of the control specimen, which occurred just below the compression flange, with its centroid (i.e. maximum amplitude of outward deflection) located approximately at 40 mm from the top flange (Figure 2.10). In multiple bolts, the overlapped area counts only once. Figure 2.10 shows the variation of the effectiveness of each pattern, in terms of the percentage of gain in strength, with the parameter (k). Interestingly, no significant variation or a correlation of effectiveness with (k) can be observed as the range of gain in strength was 24 to 29%. It is clear that as little as a single bolt is sufficient to provide the necessary bracing of the webs, provided that it has sufficient axial stiffness as the bolts used in this study. It is possible that smaller bolts would result in a more significant trend as the pattern changes, but this was not explored in this study.

2.3.5 Failure Modes

The failure mode of the control specimens T1, T7, T8, T9, and T13 was symmetrical outward crippling of the chord sidewalls (Figure 2.11(a)), however, in T13, yielding of the chord occurred well before crippling. For all the retrofitted specimens, except T5, the through-wall bolts shifted the failure mode of the chord sidewalls from the symmetrical outward crippling (i.e. lateral displacement of the webs in opposite directions), to a different mode, in which both sidewalls were forced to displace laterally in the same direction. As such, one of the sidewalls buckled outward, and the other side was forced inward by the bolts. This unsymmetrical behaviour of the two sidewalls caused the
vertical brace to rotate out of plane in some specimens, as shown in Figure 2.11 (b and d),
triggering a lateral-torsional buckling secondary failure. In T5, both webs buckled
outwards, but this was shifted downwards below the single bolt used.
Table 2.1 Test matrix

<table>
<thead>
<tr>
<th>Specimen I.D</th>
<th>Chord wall thickness (mm)</th>
<th>Retrofitting System</th>
<th>Bolt Pattern (Fig. 2.1)</th>
<th>Maximum Load (kN)</th>
<th>Gain in Strength (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>3.09</td>
<td>N/A (control)</td>
<td></td>
<td>131</td>
<td>-</td>
</tr>
<tr>
<td>T2</td>
<td></td>
<td>15 bolts</td>
<td>A</td>
<td>169</td>
<td>29.0</td>
</tr>
<tr>
<td>T3</td>
<td></td>
<td>2 bolts</td>
<td>B</td>
<td>164</td>
<td>25.2</td>
</tr>
<tr>
<td>T4</td>
<td></td>
<td>4 bolts</td>
<td>C</td>
<td>162</td>
<td>23.7</td>
</tr>
<tr>
<td>T5</td>
<td></td>
<td>1 bolt</td>
<td>D</td>
<td>164</td>
<td>25.2</td>
</tr>
<tr>
<td>T6</td>
<td></td>
<td>1 bolt</td>
<td>E</td>
<td>165</td>
<td>26.0</td>
</tr>
<tr>
<td>T7</td>
<td>4.50</td>
<td>N/A (control)</td>
<td></td>
<td>271</td>
<td>-</td>
</tr>
<tr>
<td>T8</td>
<td></td>
<td></td>
<td></td>
<td>281</td>
<td>-</td>
</tr>
<tr>
<td>T9</td>
<td></td>
<td></td>
<td></td>
<td>290</td>
<td></td>
</tr>
<tr>
<td>T10</td>
<td></td>
<td>15 bolts</td>
<td>A</td>
<td>294</td>
<td>6.2</td>
</tr>
<tr>
<td>T11</td>
<td></td>
<td></td>
<td></td>
<td>299</td>
<td>(based on avg.)</td>
</tr>
<tr>
<td>T12</td>
<td></td>
<td></td>
<td></td>
<td>301</td>
<td></td>
</tr>
<tr>
<td>T13</td>
<td>5.92</td>
<td>N/A (control)</td>
<td></td>
<td>448</td>
<td>-</td>
</tr>
<tr>
<td>T14</td>
<td></td>
<td>15 bolts</td>
<td>A</td>
<td>462</td>
<td>3.1</td>
</tr>
</tbody>
</table>
Figure 2.1 Different through-wall bolt configurations A to E in the chord member of the T-joint

Figure 2.2 Stress-strain curves of the HSS chords
Figure 2.3 Test setup of T-joints under brace axial compression concentric load

Figure 2.4 Electric resistance strain gauge configurations on specimens T1 to T14
Figure 2.5 Load-deflection responses at top and bottom chords
Figure 2.6 Load-strain responses in specimens T1 to T6 with chord wall thickness of 3.09 mm
Figure 2.7 Load-strain responses in specimens T7 to T12 with chord wall thickness of 4.5 mm

Figure 2.8 Load-strain responses in specimens T13 and T14 with chord wall thickness of 5.92 mm
Figure 2.9 Variation of the percentage gain in strength with chord wall depth-to-thickness (h/t) ratio

Figure 2.10 Effect of bolt configuration on the percentage gain in strength for h/t = 65
Figure 2.11 Failure modes: (a) T1, (b) T2, (c) T3, (d) T4, (e) T5, and (f) T6
Chapter 3

Bonded FRP Plates for Strengthening Rectangular Hollow Steel Section T-Joints against Web Crippling induced by Transverse Compression

3.1 INTRODUCTION

Significant efforts in recent years have been focused on retrofitting steel structures using adhesively bonded FRP plates and sheets (Hollaway and Cadei, 2002, Shaat et al., 2004, and Harries and El-Tawil, 2008). Most of work used carbon-FRP (CFRP) laminates and have focused primarily on retrofitting members rather than connections. Several researchers studied bond between steel and CFRP (e.g. Xia and Teng, 2005, and Nozaka et al., 2005), flexural strengthening or repair of W- and S-sections (e.g. Tavakkolizadeh and Saadatmanesh, 2003, Shaat and Fam, 2008, and Dawood et al., 2009), and strengthening slender Hollow Steel Section (HSS) columns against global buckling (Shaat and Fam, 2009). Limited work addressed local instabilities of HSS members. For example, Zhao et al. (2006) demonstrated the benefits of using CFRP wraps to control web crippling of HSS sections under end bearing. Also, Shaat and Fam (2006) studied axially loaded CFRP-wrapped short HSS columns to control local buckling. A study on FRP retrofitting of truss joints of tubular members in overhead sign supports (Fam et al., 2006) showed that FRP can indeed recover the joint tensile strength reduced by cracking.

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2 This chapter has been published (online) as the following journal paper: Aguilera, J., and Fam, A. (2012) “Bonded FRP Plates for Strengthening Rectangular Hollow Steel Section T-Joints against Web Buckling Induced by Transverse Compression”, Journal of Composites for Construction
of the weld. The study however did not address local instabilities due to compression at the joint.

Tubular members are quite common in truss and vierendeel girders, and frames. The connection between the chord and vertical member (brace) in vierendeel girders and at mid-span of N-trusses, or between beam and column in frames, take the form of a T-joint (Figure 3.1(a)). The capacity of the structure may be governed by the strength of its joints. Thus, upgrading the joint capacity may be essential in certain structures, especially if the members have been upgraded. This becomes even more crucial in thin-walled members. In this case, the bearing of the brace on the chord produces web crippling of the thin walls of the chord. Unlike W- and S-sections, it is not possible to install steel web stiffeners inside tubular sections. In offshore structures with circular tubes, commonly used concepts include the ‘can’ whereby the chord members are partially thickened at the joints, or the ‘doubler plate’, sometimes slightly modified to a system referred to as the ‘collar’, where the brace is welded directly to the doubler plate through a penetration weld, whereas the doubler plate is fillet-welded to the chord (Choo et al., 1998). In rectangular HSS members, failure of the chord side walls by yielding or crippling is the most common failure mode, especially when the breadth of brace-to-breadth of chord ratio is close or equal to unity. Conventional retrofitting techniques involve filling the joint with concrete or connecting the two side walls by steel bolts (Zhao, 1999, and Aguilera et al., 2012).

This chapter investigates strengthening using FRP plates adhesively bonded to the side walls of rectangular HSS chords in the vicinity of brace members loaded in compression in T-joints. The effects of HSS web (h/t) ratio, GFRP plate size, shape and
edge tapering, as well as thick GFRP versus thin high-modulus CFRP plates, on strengthening effectiveness are explored.

3.2 EXPERIMENTAL PROGRAM

In the following sections, material properties, test specimens and parameters, fabrication of specimen, test setup, and instrumentation are described in details.

3.2.1 Material Properties

3.2.1.1 Cold-formed HSS sections: Two types of HSS sections were used, a rectangular HSS of two different wall thicknesses (203x76x3.09 and 5.92 mm), for the chord, and a square HSS (76x76x8.9 mm) for the brace. Both sections were manufactured in accordance with ASTM A500 C (2009). Uniaxial tension tests were performed according to ASTM E8/E8M-09 (2009) on four dog-bone coupons. The coupons were 200 mm long overall, with a grip width and length of 20 mm and 50 mm, respectively. A 12.5 mm thickness was used as the width in the gage length. Two coupons were cut for each chord thickness, one from the flange and one from the web. A 50 mm extensometer was used to measure and record the strains. The stress-strain plots for all steel coupons are shown in Figure 3.2. The yield strengths (offset secant at 0.2%) of flanges were 426 and 449 MPa, for the 3.09 and 5.92 mm thicknesses, respectively, while yield strengths of the two web thicknesses were 427, and 443 MPa, respectively. The modulus of elasticity of both flange and web was 209 GPa.
3.2.1.2 GFRP Plate: Commercially available 9.5 mm thick plates were used. The plates consisted of alternating layers of unidirectional E-glass roving and random mats impregnated with polyester resin. Three coupons were cut from the plates (Figure 3.2) and tested in tension in the longitudinal direction according to ASTM D3039/D3039M (2008) (dimensions shown in Figure 3.2). The stress-strain curves are shown in Figure 3.2. The average longitudinal tensile strength and elastic modulus were 268 MPa and 20.6 GPa, respectively. The manufacturer reported longitudinal and transverse tensile strengths of 138 and 69 MPa, and moduli of 12.4 and 6.9 GPa, respectively.

3.2.1.3 CFRP Plate: A high-modulus, 2 mm CFRP pultruded plate classified as E-Plate HM1020 with unidirectional fibres, was used. The manufacturer reported a tensile strength of 1200 MPa, and a Young’s modulus of 450 GPa.

3.2.1.4 Adhesive: The adhesive used for bonding the FRP plates to the steel specimens was Weld-On SS620. Schnerch et al. (2004) examined several adhesives and found that Weld-On SS620 had one of the shortest development lengths. The adhesive is comprised of two components, namely an adhesive (SS620-A) and an activator (SS620-B) mixed at a 10:1 ratio. The reported tensile strength and lap shear strength were both 20 MPa on average.

3.2.2 Test Specimens and Parameters

Table 3.1 provides a summary of the test matrix. A total of twelve T-joints was fabricated and tested under brace axial compression load. The T-joint consisted of a horizontal,
1220 mm long rectangular HSS chord member welded to a 400 mm long square HSS brace member (Figure 3.1(b)). Four key parameters were explored, namely, the wall thickness of the chord member, in terms of web (h/t) ratio, GFRP plate configuration (shape and size), GFRP plate end condition (square and reverse tapered edges), and type of FRP plate (GFRP and CFRP). Specimens T2-a and T2-b were identical to check for repeatability.

The chord member with a 3.09 mm (Class 4) wall thickness was used in specimens T1 to T7, and the one with 5.92 mm (Class 3) wall thickness was used in T8 to T11. The different wall thicknesses of the chords correspond to (h/t) ratios of 65, and 34, respectively. The 76x76x8.9 mm HSS (Class 1) brace member had a thick-wall to avoid failure of the brace. Control specimens were tested in each group, including specimen T1 and T8.

Outwards web crippling of the chord member occurs directly below the brace member. As such, seven different FRP plate configurations (A to G) were used to cover this region, on both sides of the chord, as shown in Figure 3.3 (parallel lines indicate longitudinal direction of FRP plate). Configurations A and B use a 185x38 mm GFRP plate oriented vertically below the brace, with the A plate having square edges and the B plate having a reversed tapered edges. The plate length extends the full depth of the chord, while the width is half that of the brace. Configuration C is similar to A, except that plate width is 76 mm, equal to brace width. Configuration D is similar to C, except that plate length is 95 mm, half the chord depth. Configuration E uses a 409x182 mm GFRP plate oriented horizontally below the brace. It covers the full depth of chord and extends a length almost twice the depth of the chord. Configuration F is similar to C,
except it uses a 195x71 mm CFRP plate, also vertically oriented. Given that web local crippling essentially imposes bending on the FRP plate, the criterion for the CFRP plate design was to provide similar flexural stiffness ($E/1$) to the GFRP plate in configuration C, relative to the steel substrata, based on a 2 mm thick adhesive layer. Finally, configuration G uses a 388x77 mm GFRP plate, somewhat similar to C, except that the plate extends 185 mm up beyond the chord and on the brace for additional bond length. This configuration was only used in the thicker wall (5.92 mm) chord specimen, to avoid a peeling failure that occurred in a specimen with configuration C.

For the effect of HSS chord wall thickness on strengthening effectiveness, specimens (T4 and T9) with configuration C, as well as (T6 and T11) with configuration E were compared. For the effect of GFRP plate configuration on effectiveness, specimens (T2, T3, T4, T5, and T6) with 3.09 mm thick HSS chord, as well as specimens (T9, T10 and T11) with 5.92 mm thick HSS chord were compared. The effect of reverse tapering of the GFRP plate on bond, versus the square edge, was examined by comparing (T2 and T3). Thick GFRP and thin CFRP plates were compared through specimens T4 and T7 of configurations C and F, respectively.

### 3.2.3 Fabrication of Test Specimens

The lengths of the chord and brace members were cut to 1220 and 400 mm, respectively, and brace members were welded to the flanges of the chord members at mid-lengths, using a professional machine shop. The specimens were then sandblasted. Before FRP installation, the steel surface was cleaned with acetone to remove any dirt or debris left from sandblasting. The GFRP plates were cut to dimension using a wet table diamond
blade saw. The plates were then sanded with fine grit sandpaper to remove the smooth polymeric coating and were cleaned with isopropyl alcohol. The CFRP plates were cut from a long pultruded plate and the protective tape was removed just before installation.

The adhesive was then applied to the surfaces and the FRP plates positioned and pressed to maintain a consistent thickness of adhesive (Figure 3.1(c)). After curing, and before testing, vertical stiffener steel plates, 110x191x13 mm, were inserted inside the chord member at both ends (Figure 3.4) to prevent premature failure due to crippling at supports.

3.2.4 Test Setup and Instrumentation

The specimens were tested under brace concentric compression loading using a 1000 kN Riehle machine (Figure 3.4). The load was applied to failure using stroke control at a rate of 1 mm/min. The chord member was clamped at both ends using a special assembly of heavy HSS sections and threaded rods. First, the specimen was rested on two 150x150x12 mm HSS supports. The two supports were set apart to provide a clear span (L) of 1000 mm, which is almost five times the chord depth (h). Another two HSS sections were set on top of the specimens ends. The two upper HSSs were held down and anchored to the Riehle testing machine using two 25 mm diameter threaded rods at each support. The threaded rods were evenly hand tightened using wrenches.

The span-to-depth (L/h) ratio of the chord was carefully selected as per recommendations in literature. The UK Department of Energy Offshore Technology Report (1990) and other researchers (Moffat et al., 1996) suggested that (L/h) ratio should not be less than 4, in order to avoid any effect of supports on the joint strength. On
the other hand, others (Lalani, 1992) indicated that the (L/h) ratio should not be excessive, otherwise chord flexural failure may occur by achieving plastic moment, prior to joint failure. In this study, an (L/h) ratio of 5 was used to avoid the influence of supports on joint strength. Some end fixities were provided by the clamping system for stability of the specimen, and to increase the load at which yield and plastic moments of the chord occur, in order to focus primarily on the joint strength.

Linear potentiometers (LPs) with a 100 mm range were generally used for measuring displacements in the experiments. Two vertical LPs were used at mid span to measure independently the deflections of the upper and lower flanges of the chord member below the loaded brace. These two deflections may differ as the chord sidewalls buckle. Four LPs were mounted horizontally to measure the slip of the top and bottom flanges of the chord at each clamped end of some of the specimens. The strains in the chord webs and flanges were measured using 5 mm electric resistance strain gauges; installed on both the steel and FRP plates (Figure 3.5). The load was measured using a load cell built-in within the Riehle machine.

3.3 RESULTS OF THE EXPERIMENTAL PROGRAM

Table 3.1 provides a summary of the experimental results, in terms of the maximum load achieved and the percentage of gain in strength of retrofitted specimens relative to their control counterparts. Figures 3.6 and 3.7 show the load-deflection responses of the chords, at both top and bottom flanges, in specimens with 3.09 mm and 5.92 mm wall thickness, respectively. Similarly, Figures 3.8 and 3.9 show the load-strain responses.
Figure 3.10 shows the load-end slip responses of the chord. Figure 3.11 summarizes the findings in terms of the effect of GFRP plate configuration and HSS wall thickness on the gain in joint strength. Finally, Figures 3.12 and 3.13 show the failure modes.

### 3.3.1 Load-Deflection Behaviour

Figures 3.6 (a and b) show the load-deflection responses of specimens T1 to T7, while Figure 3.7 shows similar responses of specimens T8 to T11. Generally, the load ascends almost linearly initially, followed by a non-linear behaviour of various degrees of nonlinearity, depending on (h/t) ratio of the web, and FRP plate configuration, until a peak load is reached. Then, a descending response can be observed (except for specimens T6 and T11, where the wide GFRP plate has changed failure mode). It should be noted that while the FRP retrofitting system enhances strength, it has virtually no effect on the initial stiffness. It is also noted that in specimens T1 to T7 (except T6), with (h/t) ratio of 65, the retrofitting systems provided a significant gain in strength but not in ductility, as evident by the rapid post-peak descent of the load-deflection curves. In specimens T8 to T11 (except T11) with (h/t) ratio of 34, very little gain in strength is observed but certain plate configurations enhanced ductility by extending the plastic plateau.

For each specimen, the deflection of the top and bottom flanges of the chord differed slightly at any given load, as a result of the out-of-plane displacements of the vertical webs. This effect becomes more pronounced near the peak load and continues to increase in the descending part of the response. The peak load generally corresponds to instability failure of the webs of the chord at the joint, except for specimens T6 and T11.
It is believed that the GFRP plate in T6 and T11 was wide enough (almost 40% of the span) that it not only provided web stability as intended, but also flexural contribution. For this reason specimen T6 shows a significant gain in strength along with a plastic behaviour and failure mode changed to shear near the supports.

### 3.3.2 Load-Strain Behaviour

By examining the extreme fibre strains of the chord at mid-span of specimens T1 to T7 with (h/t) ratio of 65 (in Figure 3.5: SG4/5 for T1, SG1/2 for T2 to T5 and T7, and SG1/6 for T6), relative to the yield strain of 4100 micro-strain (corresponding to 0.2% offset secant in Figure 3.2), the effects of strengthening the specimens can be assessed (Figure 3.8). In control specimen T1, the peak load was reached, and hence stability failure, before yielding of the upper or lower flanges of the chord in the vicinity of the joint. In all strengthened specimens, except T5 and T7, the top flange reached yielding at peak load, while bottom flange slightly exceeded yielding. In specimens T5 and T7, the bottom flanges just reached yielding at peak load. It can be concluded that for this wall thickness, retrofitting enhanced strength significantly (Figure 3.6) but did not allow chords to develop much yielding. This explains the rapid post-peak decent in load-deflection responses.

Similarly, by examining the extreme fibre strains in specimens with (h/t) ratio of 34 (in Figure 3.5: SG4/5 for T8, and SG1/2 for T9 to T11), Figure 3.9 shows that control specimen T8 yielded sufficiently before reaching the peak load. Strengthening (specimens T9 and T10) allowed the section to achieve higher yielding strains, especially
at the tension flange, before reaching the peak load. This explains the gain in ductility in load-deflection responses.

In control specimens T1 and T8, SG2/6 in the vertical direction of the web near the location of maximum amplitude of web crippling (Figure 3.5) show almost zero strain (Figures 3.8(a) and 3.9(a)), as would be expected, until about 70% and 86%, respectively, of the peak loads, when a pronounced tensile strain develops rapidly, suggesting the onset of outwards web crippling. Also, Figures 3.8 and 3.9 show that the maximum strains developed in the GFRP at peak loads are small relative to the plate rupture capacity.

3.3.3 Load-Slip Behaviour of Chord

Figure 3.10 shows the load-end slip of the chord member at supports. The figure suggests that at peak load, end slip was about 1 to 1.5 mm in specimens with (h/t) of 65, and 2 to 4 mm in specimens with (h/t) of 34. As such, the end support system provided only partial fixity.

3.3.4 Repeatability of Test Results

Due to the vulnerability of thin-walled tubular structures to geometric imperfections such as out-of-straightness, which directly affects stability failure, it was important to evaluate the repeatability of test results. Specimens T2-a and T2-b were of similar (h/t) ratio, and were both strengthened with the same GFRP pattern A. Figure 3.6(b) compares the load-deflection responses of specimens T2-a and T2-b. No distinct difference can be observed in response and the peak loads were comparable, 172 and 176 kN. Also, in similar control
specimens tested by the authors (Aguilera et al., 2012), but with (h/t) ratio of 45, three repetitive tests showed a coefficient of variation of 3.38% only.

3.3.5 Effect of Web Height-to-Thickness (h/t) Ratio of Chord

The effect of (h/t) ratio of the chord on strengthening effectiveness using GFRP is demonstrated in Figure 3.11 for GFRP plate configurations C and E. It is clear that strengthening effectiveness increases significantly as the ratio (h/t) increases. For example, for configuration C, the strength gain increases from 9% to 38%, as (h/t) increases from 34 to 65. Similarly, for configuration E, the strength gain increases from 27% to 53%.

3.3.6 Effect of GFRP Plate Edge Configuration

Figure 3.6(b) compares the load-deflection responses of specimens (T2-a and T2-b) with a square edged GFRP plate to T3 with a reversed tapered edge. No distinct difference can be observed in the behaviour or maximum load. The average strength gain was 32.5% for T2 and 30.6% for T3. Also, failure modes were quite similar. As such, it appears that tapering off the GFRP plate ends is not as significant for this application, for this particular (h/t) ratio, especially since the plates did not debond from the steel specimen.

3.3.7 Effect of Type of FRP Plate

Specimens T4 and T7 had thick GFRP and thin CFRP plates, respectively, of similar equivalent flexural stiffness. Figure 3.6(a and b) compares their load-deflection
responses. The strength gains were 38 and 32%, respectively. It should be noted that the CFRP plate fractured due to local bending in the vertical direction on one side, while the GFRP plate shifted the web local crippling away from mid-span. The very high modulus of the CFRP plate limits its ultimate strain and hence it could not accommodate the bending deformation. It appears then for this application, that thick GFRP plates are more effective than thin CFRP plates of high modulus.

3.3.8 Failure Modes

The failure mode of the control specimens T1 (h/t = 65) and T8 (h/t = 34) was symmetrical outward crippling of the chord side-walls, right below the brace, with the maximum amplitude at about 40 mm from top flange (Figure 3.12(a)). This occurred in T1 while the chord was still elastic, while in T8 the chord has already yielded.

Retrofitted Specimens with (h/t) of 65: Specimens T2-a, T2-b, T3, and T4 had similar failure modes, where local crippling occurred on the top flange of the chord on one side of the brace, and this was associated with some local crippling in the web (Figure 3.12 (b,c)). It is clear that the GFRP plate shifted the crippling away from mid-span. In specimen T5, the GFRP plate extended only to mid-depth of the chord. As such, the chord webs were not restricted from outward crippling, but unlike T1, the maximum amplitude shifted to mid-depth of the web. As such, some peeling occurred at the bottom of the GFRP plates (side 1, Figure 3.12(d)). Because peeling did not occur on both sides, symmetry was lost, and that resulted in some lateral torsional buckling (end view in Figure 3.12(d)). In specimen T6, the GFRP plate was quite wide, almost 40% of the span.
As such, it shifted failure mode to a web shear failure and crippling near the support, even in presence of the steel end stiffener (Figure 3.12(e)). In specimen T7, the thin CFRP plate failed on one side by fracture of the fibres at mid-depth of the chord due to the local bending induced by web crippling in the vertical direction (side 1, Figure 3.12(f)). On the other sidewall, the CFRP plate started peeling off from the top (side 2, Figure 3.12(f)). The lack of symmetry also promoted some lateral torsional buckling, as the brace began to bend (end, Figure 3.12(f)).

Retrofitted Specimens with \((h/t)\) of 34: Specimen T9 had a symmetrical outward web crippling like control specimen T8, which resulted in significant local bending and hence top end peeling occurred within the GFRP plate itself (Figure 3.13(a)). The plate was split near its mid-thickness while the adhesive bond of the plate to the steel surface was intact. For this reason configuration G in specimen T10 was designed. As a result, specimen T10 did not have end peeling and the plates had significant bending deformations associated with the symmetric web crippling, before the plate fibres fractured (Figure 3.13(b)). This change in failure mode of T10 relative to T9, has led to significant increase in ductility (Figure 3.7). It is possible that the reverse tapering of the edges of the plate (as in T3) could have mitigated the peeling problem in T9, though this was not tested in this study. Unlike T6, the wide GFRP plate in specimen T11 did not lead to shear failure near support because of the thicker steel web of the chord. Instead, the GFRP plate suffered delamination within its thickness under the effect of in-plane compressive stresses near the top of the chord (top, Figure 3.13(c)) and tensile fracture
near the bottom edge due to in-plane tension (side, Figure 3.13(c)). The lack of symmetry also promoted lateral torsional buckling (end, Figure 3.13(c)).
### Table 3.1 Test matrix

<table>
<thead>
<tr>
<th>Specimen I.D</th>
<th>Chord wall thickness (mm)</th>
<th>Retrofitting System</th>
<th>FRP Plate Dimensions (mm)</th>
<th>FRP plate configuration (Fig. 3.3)</th>
<th>Maximum Load (kN)</th>
<th>Gain in Strength (%)</th>
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<td>T1*</td>
<td>3.09</td>
<td>N/A (control)</td>
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<td></td>
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<td>GFRP</td>
<td>38x185x9.4</td>
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<td>31.7</td>
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<td>409x182x9.4</td>
<td>E</td>
<td>568.0</td>
<td>26.9</td>
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</table>

*Aguilera et al. (2012)*
Figure 3.1 Description of test specimens and FRP installation

Figure 3.2 Stress-strain curves of the HSS chords and FRP plates
Figure 3.3 Different FRP retrofit configurations A to G in the chord member of the T-joint

Figure 3.4 Test setup of T-joints under brace axial compression concentric load
Figure 3.5 Electric resistance strain gauge configurations on specimens T1 to T11
Figure 3.6 Load-deflection responses at top and bottom chords of specimens with 3.09 mm wall thickness
Figure 3.7 Load-deflection responses at top and bottom chords of specimens with 5.92 mm wall thickness
Figure 3.8 Load-strain responses of specimens with 3.09 mm wall thickness
Figure 3.9 Load-strain responses of specimens with 5.92 mm wall thickness
Figure 3.10 Load-horizontal end displacements at top and bottom of chord members
Figure 3.11 Effect of h/t ratio of HSS chord on strengthening effectiveness
Figure 3.12 Failure modes of specimens with 3.09 mm wall thickness:

(a) T1, (b) T2a, (c) T4, (d) T5, (e) T6, and (f) T7
Figure 3.13 Failure modes of specimens with 5.92 mm wall thickness: (a) T9, (b) T10, and (c) T11
Chapter 4

Retrofitting Tubular Steel T-Joints subjected to Axial Compression in Chord and Brace Members using Bonded FRP Plates or Through-Wall Steel Bolts

4.1 INTRODUCTION

Tubular steel structures are commonly used in the form of trusses, vierendeel girders, and frames. The connection between the chord and vertical member (brace) in vierendeel girders and at mid-span of N-trusses, or between beam and column in frames, take the form of a T-joint. The capacity of the structure may be governed by the strength of its joints. Therefore, upgrading the joint capacity may be essential in certain structures, particularly if the members have been upgraded. This is crucial in thin-walled members, where the bearing of the brace member on the chord member produces web crippling of the thin walls of the chord. Unlike steel W- and S-sections, it is not possible to install steel web stiffeners inside tubular sections. In offshore structures with circular tubes, the commonly used concepts include the ‘can’ whereby the chord members are partially thickened at the joints, or the ‘doubler plate’, sometimes slightly modified to a system referred to as the ‘collar’, where the brace is welded directly to the doubler plate through a penetration weld, whereas the doubler plate is fillet-welded to the chord (Choo et al., 1998). It should be noted that all these strengthening techniques are primarily used in circular hollow sections, commonly used in offshore structures.

For Rectangular Hollow Sections (RHS), different strengthening techniques may be employed. Failure of the chord side walls by yielding or crippling is the most common

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3 This chapter has been accepted in the following journal paper:
for T-, Y- and X-joints, especially when their $\beta$ ratios (breadth of brace/breadth of chord) are close or equal to unity. Welded rectangular HSS T-joints have been studied experimentally by a number of researchers in the past (e.g. Kato and Nishiyama, 1979, and Zhao and Hancock, 1991). When $\beta$ is within 0.8 and 1.0, web crippling normally governs. When $\beta$ is less than 0.8, the failure modes depend on the chord flange width-to-chord thickness ratio. A deformation limit, in the form of the level of local indentation of flange chord face, is adopted in the design of welded tubular joints as described in the latest IIW static design procedures for welded tubular joints (Zhao et al., 2010, Lu et al., 1994, and Zhao, 2000). Filling hollow sections with concrete to improve the web crippling behaviour was found to be efficient (Packer, 1995). Another technique was adopted (Bains, 1983, and Bradfield et al., 1994), where a single bolt was used to brace the chord sidewalls against the outward buckling, and resulted in an increase of 18% in capacity. Another study (Zhao, 1999) recommended that a wooden brace be inserted into the RHS in addition to the through-wall bolt to avoid the inward buckling of the chord sidewalls. The use of externally bonded fibre reinforced polymers (FRP) laminates has also been successfully used to strengthen HSS columns against global buckling (Shaat and Fam, 2009). Limited work addressed local instabilities of HSS members. For example, a study (Zhao et al., 2006) demonstrated the benefits of using carbon-FRP wraps to control web crippling of HSS sections under end bearing. Another study (Shaat and Fam, 2006) investigated axially loaded carbon-FRP-wrapped short HSS columns to control local buckling. Two previous studies have investigated the strengthening of T-joints of tubular members using through-wall bolts and bonded FRP plates (Aguilera et al., 2012, and Aguilera and Fam, 2012) where the chord was transversely loaded through a brace member.
This chapter investigates the strengthening of RHS chords under sustained axial compression loads, in the vicinity of brace members at T-joints, where the HSS brace member is gradually loaded to failure. Both through-wall steel bolts and adhesively bonded GFRP plates techniques are investigated. Also, the effect of the level of sustained axial compression in the chord on strengthening effectiveness is studied.

4.2 EXPERIMENTAL PROGRAM
The following sections describe test specimens and parameters, material properties, fabrication of specimens, test setup and instrumentation.

4.2.1 Test Specimens and Parameters
Table 4.1 provides a summary of the test matrix. T-joints were fabricated and tested under combined brace and chord axial compression loads. The T-joint consisted of a horizontal, 1220 mm long chord member welded to a 400 mm long brace member (Figure 4.1(a)). Two key parameters were explored, namely, the type of retrofitting reinforcement (through-wall steel bolts and adhesively bonded GFRP plates (Figure 4.1(b)), and the level of axial load in the chord member. The chord member was a 203x76x3.09 mm RHS (Class 4), with an (h/t) ratio of 65. The brace member was a 76x76x8.9 mm HSS (Class 1) with a thick wall to avoid failure of the brace. The study included three control specimens, one with no axial load in the chord (T1), one with 200 kN (T2), and one with 350 kN (T3). The axial compression loads in specimens T2 and T3 represent 45% and 80%, respectively, of the pure axial strength of the chord, calculated according to CAN/CSA-S16-01 (2006). The chord loads were kept constant during gradual application of the load on the brace member to failure.
Specimens T4 to T6 are counterparts of specimens T1 to T3 and were retrofitted with the through-wall steel bolts system (configuration A in Figure 4.1(b)), where the two bolts were located directly below the brace member, 40 mm below the compression flange of the chord, where the maximum local crippling was expected in the chord. Specimens T7 to T9 are also counterparts of T1 to T3 and were retrofitted by two 76x185x9.5 mm adhesively bonded GFRP plates, one on each web, directly below the brace member (configuration B in Figure 4.1(b)).

4.2.2 Material Properties

4.2.2.1 Cold-formed tubular sections: Two types of HSS sections were used, a rectangular one (RHS), 203x76x3.09 mm, for the chord, and a square (SHS) 76x76x8.9 mm section for the brace. Both the RHS and SHS were manufactured in accordance with ASTM A500 C (2009). Uniaxial tension tests were performed according to ASTM E8/E8M-09 (2009) on dog-bone coupons cut from the flange and the web of the RHS chord. The coupons were 200 mm long overall, with a grip width and length of 20 mm and 50 mm, respectively. A 12.5 mm thickness was used as the width in the gage length. A 50 mm extensometer was used to measure and record strains. The stress-strain plots for the steel coupons are shown in Figure 4.2. The yield strength (offset secant at 0.2%) and modulus of the chord were 426 and 209 GPa. The reported yield strength by manufacturer was 410 MPa.
4.2.2.2 Through-wall bolts: The bolts used in the strengthened specimens are standard 8 mm diameter (Grade 8) high-strength bolts with a reported ultimate strength of 1034 MPa (Figure 4.2).

4.2.2.3 GFRP Plates: A 9.5 mm thick commercially available GFRP plate was used. It consists of alternating layers of unidirectional E-glass roving and random mats impregnated with polyester resin. The manufacturer reported longitudinal and transverse tensile strengths of 138 and 69 MPa, and moduli of 12.4 and 6.9 GPa, respectively. The reported longitudinal and transverse compressive strengths and moduli by the manufacturer are 165 and 110 MPa and 12.4 and 6.9 GPa, respectively. To confirm these results, three coupons were cut from the plates and tested in tension in the longitudinal direction according to ASTM D3039/D3039M (2008) (dimensions given in Figure 3.2). The stress-strain curves are shown in Figure 4.2. The average longitudinal tensile strength and elastic modulus were 268 MPa and 20.6 GPa, respectively.

4.2.2.4 Adhesive: The adhesive used for bonding the GFRP plates to the steel specimens was Weld-On SS620. It is comprised of two components, namely an adhesive (SS620-A) and an activator (SS620-B) mixed at a 10:1 ratio. The reported tensile strength is 18 to 21 MPa and the lap shear strength is 19 to 22 MPa.

4.2.3 Fabrication of T-Joint Specimens

The chord and brace members were cut to lengths of 1220 and 400 mm, respectively. The SHS brace member was directly welded to the flange of the RHS chord member at mid-
length. Cutting and welding of specimens were performed by a professional, at a machine shop. The holes necessary for the bolts to pass through the chord in retrofitting configuration A were then hand drilled in both webs. The 8 mm Grade 8 high-strength headed bolts were then inserted into the holes and anchored from one side using a special washer and nut for a snug fit. Special care was taken in order not to over tighten the nut and cause inward crippling of the two webs. This was accomplished by manual tightening of the nut until slack was removed between the washer and the web of the section, but no pretension was applied to the bolt. For retrofitting configuration B, specimens were first sandblasted to prepare the surface and were cleaned with acetone to remove any dirt or debris. The GFRP plates were then sanded with fine grit sandpaper to remove the smooth polymeric coating and were cleaned with isopropyl alcohol. The adhesive was then applied to the surfaces and the FRP plates were positioned and pressed to maintain a consistent thickness of adhesive. After curing, and before testing, vertical stiffener steel plates, 110x191x6 mm, were inserted inside the chord member at both ends (Figure 4.3) to prevent premature failure due to crippling at supports.

4.2.4 Test Setup and Instrumentation

The axial compression load in the chord was first applied through a 25 mm diameter high strength threaded rod positioned concentrically inside the RHS chord member. The rod was anchored against the RHS section on one end using a large load cell with a central hole and was tensioned from the other end against an end steel plate using a hydraulic ram (Figure 4.3(a)). At both ends, the rod was anchored using a special nut and washer system of spherical surfaces (Figure 4.3(a)), which was also lubricated. This was to
prevent any bending in the threaded rod, which could contribute to the flexural capacity of the RHS chord member during transverse loading on the brace member.

After loading the chord to the desired axial force, the brace was concentrically loaded to failure using a 1000 kN Riehle testing machine (Figure 4.3(b)), at a 1 mm/min rate. The brace was clamped at both ends using a special assembly of heavy SHS sections and threaded rods. First, the specimen was rested on two 150x150x12 mm HSS supports. The two supports were set apart to provide a clear span (L) of 1000 mm, which is almost five times the chord depth (h) of 203 mm. Another two SHS sections were set on top of the specimens ends. The two upper SHSs were held down and anchored to the Riehle testing machine using two vertical 25.4 mm diameter threaded rods at each support. The threaded rods were evenly hand tightened using wrenches. This span-to-depth (L/h) ratio of the chord was carefully selected as per recommendations in the literature. The UK Department of Energy Offshore Technology Report (1990) and other researchers (Moffat et al., 1996) suggested that (L/h) ratio should not be less than four, in order to avoid any effect of supports on the joint strength. On the other hand, others (Lalani, 1992) indicated that the (L/h) ratio should not be excessive, otherwise chord failure may occur prior to joint failure as the plastic moment of the chord cross-section at the crown location is reached. Some researchers (Madros et al., 1995) have indicated a limit of 5.75 for (L/h) to ensure joint failure. In this study, the (L/h) ratio of 5 was used. Some end fixities were provided by clamping, to increase the load at which yield and plastic moments occur, in order to focus on the stability aspect of joint strength.

Two 100 mm linear potentiometers (LPs) were mounted at the upper and lower flanges of the chord member at mid-span, to measure the vertical deflection of the top
and bottom flanges independently (Figure 4.3(b)). These two deflections may differ as the chord sidewalls buckle. Four additional LPs were mounted in a horizontal fashion to measure the slip of the ends of the chord specimens. The strains on the chord, in two directions, and on some through-wall bolts, were measured using 5 mm electric resistance strain gauges. Figure 4.4 shows the locations of strain gauges on the chord.

4.3 RESULTS OF THE EXPERIMENTAL PROGRAM

Table 4.1 provides a summary of the experimental results, in terms of the maximum load achieved and the percentage of gain in strength of retrofitted specimens relative to their control counterparts. Figure 4.5 provides the load-deflection responses of all specimens. Figure 4.6 demonstrates the stability of axial force in chord members during the transverse loading of the specimen. Figure 4.7 summarizes the study by showing the interaction diagram of axial and lateral loads in the chord at peak values for control and retrofitted specimens. Figure 4.8 shows the load-strain responses. Figure 4.9 depicts the end slip of the top and bottom flanges of the chord member at the supports. Figures 4.10 to 4.12 show the various failure modes of the control and retrofitted specimens. The following sections describe in details the test results and effect of various parameters on performance.

4.3.1 Load-Deflection Behaviour

Figure 4.5 shows the load-deflection responses of all test specimens based on both the top and bottom flanges in each specimen (except in specimen T3 in which only top flange deflection was recorded). Generally, the load ascends almost linearly initially, followed
by a non-linear behaviour until a peak load is reached and then a descending response can be observed with various degrees of nonlinearity. This general trend is similar in both control and retrofitted specimens, regardless of chord axial load level or method of retrofitting, except that the peak load was increased in retrofitted specimens. The peak load consistently corresponds to instability failure of the webs of the chord, or crippling of the top flange. It should be noted that while the retrofitting system enhances strength, it has virtually no effect on the initial stiffness. For each specimen, the deflection of the top and bottom flanges of the chord differed slightly at any given load, as a result of the out-of-plane displacements of the vertical webs. This effect becomes more pronounced near the peak load and even more in the descending part of the response.

Figure 4.6 shows the variations of the axial force in the chord with deflection. Also shown are the transverse load-deflection curves. Since the chord axial force was induced by a self-reacting system using a threaded rod anchored at both ends, deflection of the chord would lead to change in this axial force. To avoid this and maintain a sustained level of axial force, the hydraulic ram was kept within the self-reacting system (Figure 4.3(b)). The force was continuously adjusted by the ram as the brace was loaded and the chord deflected. Figure 4.6 shows that the force was maintained fairly stable in all specimens, except T2, which had a slight variation.

4.3.2 Effect of Chord Axial Load Level on Retrofitting Effectiveness

Figure 4.7 shows the variation of maximum transverse load applied to the chord with the axial load for control and retrofitted specimens. As the axial load level increases, the interaction diagram shows that the maximum transverse load reduces, which is expected.
The figure shows that the through-wall bolts have increased the maximum transverse load by 25%, 13% and 13% for chords with axial loads of zero, 200 kN and 350 kN, respectively. It is clear that the effectiveness of the system reduces as the axial load level in the chord member increases. For the bonded GFRP plate system, the figure shows that the maximum transverse load significantly increased, by 38%, 46% and 38% for chords with axial loads of zero, 200 kN and 350 kN, respectively. No specific trend appears as of how the level of axial loads affects the strengthening effectiveness in this case.

4.3.3 Load-Strain Behaviour

Figure 4.8 shows the load-strain responses of test specimens. It can be seen that for specimens with precompression in the chord, the responses are offset in the horizontal scale and starts from a negative compressive strain. By examining the extreme fibre strains of the flanges of the chord at mid-span (Figure 4.4 shows strain gauge identifications) relative to the yield strain of 4200 micro-strain (corresponding to 0.2% offset secant in Figure 4.2), it can be seen that in all control (unretrofitted) specimens, the peak load was reached, and hence stability failure, before or just around yielding of the chord in the vicinity of the joint. The exception was control specimen T3, with the highest axial compression load, where the top flange (SG1) slightly exceeded yielding. In specimens T3, T6 and T9 with the highest axial compression load, the peak loads were achieved while the bottom flange remained in compression (SG2).

In control specimen T1 with 3.09 mm wall, strain gauges SG2 and SG6 in the vertical direction of the web near the location of maximum amplitude of crippling show almost zero strain, as would be expected, until about 70% of the peak load, when a
pronounced tensile strain develops rapidly, suggesting the onset of outwards web crippling. Similar trend is shown in specimen T2.

The strains developed in the steel bolts are generally small and are well below the yield strain of the bolt. This is attributed to the size of the bolt relative to the force exerted.

### 4.3.4 Load-End Slip Behaviour of Chord

Figure 4.9 shows the load-end slip of the chord member at supports in specimens where slip was measured. The figure suggests that at peak load, end slip did not exceed 1.5 mm. As such, the end support system provided only partial fixity.

### 4.3.5 Failure Modes

In all specimens, peak loads were associated with the onset of local instabilities. The failure mode of the control specimens T1 and T2 was symmetrical outward crippling of the chord side-walls (Figure 4.10(a and b)). Control specimen T3 had an inwards local crippling on one side near the joint (Figure 4.10(c)), which caused slight out-of-plane bending. All three control specimens underwent inwards crippling of the top flange at one side of the brace.

Specimens T4 and T5 with through-wall bolts had both sidewalls forced to displace laterally in the same direction, by having one of the sidewalls crippling outward, and the other side buckled inward (Figure 4.11(a and b)). Specimen T6 had the web fail by inward crippling just below the brace and outward crippling in the adjacent region (Figure 4.11(c)).
In specimen T7 with bonded GFRP plate, the outward crippling of the web was shifted just beyond the GFRP plate on one side and occurred in both webs (Figure 4.12(a)). Also, the top flange buckled inwards. As axial load was introduced in specimens T8 and T9, web crippling shifted inwards, while top flange crippling shifted outwards (Figure 4.12(b and c)).

It should be noted that once the peak loads were reached in all axially loaded specimens, and due to local crippling occurrences, an increased overall axial shortening occurred in the chords. To avoid excessive loss of axial force due to this shortening, a faster rate of jacking was needed for the hydraulic ram to maintain the axial force. As shown in Figure 4.6, this approach led to a steady level of axial compression in all axially loaded specimens. This was particularly important at the range where peak loads occurred.
<table>
<thead>
<tr>
<th>Specimen I.D</th>
<th>Chord wall thickness (mm)</th>
<th>Retrofitting System and Configuration</th>
<th>Axial Compressi on Load of Chord (kN)</th>
<th>Maximum Transverse Load (kN)</th>
<th>Gain in Strength (%)</th>
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<td>T1 a</td>
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<td>N/A (control)</td>
<td>0</td>
<td>131.1</td>
<td>-</td>
</tr>
<tr>
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<td></td>
<td></td>
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<td>103.8</td>
<td>-</td>
</tr>
<tr>
<td>T3</td>
<td></td>
<td></td>
<td>350</td>
<td>58.3</td>
<td>-</td>
</tr>
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<td>164.3</td>
<td>25.3</td>
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<tr>
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<td>12.6</td>
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<td></td>
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<td>80.6</td>
<td>38.3</td>
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</table>

*Aguilera et al. (2012)  
*Aguilera and Fam (2012)*
Figure 4.1 (a) Test specimen, (b) Retrofitting methods:
A-Through wall bolts, and B-Externally bonded FRP plates

Figure 4.2 Stress-strain curves of the HSS chords, steel bolts and GFRP plates
(a) Applying sustained axial compression load to the chord

(b) Loading the brace member

Figure 4.3 Test setup of T-joints under brace and chord axial compression loads
Figure 4.4 Configurations of strain gauges
Figure 4.5 Load-deflection responses of top and bottom chords of test specimens
Figure 4.6 Variation of axial and transverse loads in chord members with deflection

Figure 4.7 Variation of axial and transverse loads on chord member for control and retrofitted specimens
Figure 4.8 Load-strain behaviour of test specimens
Figure 4.9 Load-end slip responses of test specimens
Figure 4.10 Failure modes of control specimens: (a) T1, (b) T2 and (c) T3
Figure 4.11 Failure modes of through-wall bolts retrofitted specimens: (a) T4, (b) T5 and (c) T6
Figure 4.12 Failure modes of bonded GFRP-retrofitted specimens: (a) T7, (b) T8 and (c) T9
Chapter 5

Summary and Conclusions

5.1 SUMMARY

The main objective of this study was to evaluate the use of through-wall bolts and FRP in retrofitting HSS T-joints. The study demonstrated the greatest success of both techniques for the thinnest HSS member tested. The experimental investigation comprised of three phases. The first phase (Phase 1) investigated the use of through-wall bolts, to connect the RHS chord sidewalls. The second phase (Phase 2) considered the effect of using FRP plates adhesively bonded to the sidewalls of the RHS chord. Both Phase 1 and 2 loaded the chord member transversely only. The final phase (Phase 3) investigated the effect of maintaining axial loads on the chord member, reinforced with either a through-wall bolt configuration (Phase 1) or a GFRP plate configuration (Phase 2). These specimens were loaded transversely until failure.

The conclusions of the experimental investigation are summarized in the following sections. However, the recommendations and conclusions are based on the following specific conditions:

1. RHS mechanical properties: (based on coupon tests)

   Manufactured in accordance with: ASTM A500 C

   Yield Strength ($F_y$):
a. 3.09 mm: 426 MPa  
b. 4.5 mm: 423 MPa  
c. 5.92 mm: 449 MPa  

Modulus of Elasticity \(E\): 209 GPa  

2. Chord span-to-depth \(L/h\) ratio: 5  

3. Brace span-to-width \(L/b\) ratio: 13.1  

4. The height-to-thickness \(h^{*}/t\) ratio tested: 34, 45, and 65  
   *where the thickness varied but not the height \(h=203\text{mm}\)*  

5. Applies to fixed end boundary conditions at supports  

6. Through-bolts: ¼” diameter Grade 8 \(F_{ult} = 1034\text{ MPa - manufacturer reported}\)  

7. GFRP properties: (effect of plate direction was not investigated)  
   Layers of unidirectional and random E-glass mats w/ polyester resin  
   Thickness: 9.5 mm used for testing (effect of plate thickness not investigated)  

Longitudinal Properties: (based on coupon tests)  

   Tensile strength: 268 MPa  
   Elastic modulus: 20.6 GPa  

Transverse Properties: (manufacturer reported)  

   Tensile strength: 69 MPa  
   Elastic modulus: 6.9 GPa  

8. CFRP properties: (manufacturer reported)  

   Tensile strength: 1200 MPa  
   Elastic modulus: 450 GPa  

9. Adhesive: Weld-On SS620 (used recommended mix 10:1)
5.2 PHASE 1 CONCLUSIONS

Based on this experimental investigation, the following conclusions can be drawn:

1. Through-wall bolts are effective in retrofitting Rectangular Hollow Sections (RHS) of chord members in T-joints under brace axial compression. The joint strength could increase significantly, depending on chord height-to-wall thickness (h/t) ratio; however, there are no effects on stiffness or on the rate of load descent beyond the peak strength.

2. The percentage of gain in strength of T-joint increased from 3.1% to 29% as (h/t) ratio of the RHS chord increased from 34 to 65.

3. In chords with (h/t = 65), the peak load was reached, indicating a stability failure of the webs, before yielding of the chord at the joint. Therefore, for this (h/t), the technique enhanced strength significantly but did not allow the chord to exceed yielding.

4. In control specimen with (h/t = 47), chord yielding and joint stability failure occurred almost simultaneously, whereas in its retrofitted counterpart, the chord yielded before reaching the peak load and web stability failure. Therefore, for this (h/t), strengthening allowed the section to exceed yielding and achieve partial plasticity.

5. In both control and strengthened specimens with (h/t = 34), the chord yielded well before joint stability failure and the peak load; however, strengthening allowed the section to achieve more plasticity and indeed more ductility, despite the fact that the gain in strength was quite modest.
5.3 PHASE 2 CONCLUSIONS

This study demonstrated the effectiveness of externally bonded FRP plates in stabilizing thin webs of HSS sections subjected to transverse loading, thereby increasing their bearing load capacity. The T-joints studied represent beam-column connections or joints in vierendeel and truss girders. Based on this experimental investigation, the following conclusions can be drawn:

1. Strengthening effectiveness increases significantly as web height-to-thickness (h/t) ratio of the HSS member increases. As (h/t) increased from 34 to 65, the strength gain increased from 9% to 38% and from 27% to 53% when narrow and wide GFRP plates, respectively, were used.

2. Strengthening effectiveness increases slightly as FRP plate area becomes larger and/or its centre becomes closer to the point of maximum bulge of web crippling. For the (h/t = 65) specimens, a plate similar in width to the brace and extending the full depth of the chord provided 38% strength gain. Reducing the plate width, or depth by half, still provides 32% and 35% strength gains, respectively.

3. In thin-walled HSS (h/t = 65), FRP retrofitting provided significant gains in strength but not in ductility, as evident by the rapid post-peak descent of the load-deflection curves. The HSS chord did not develop significant yielding as the FRP plate delayed and shifted local crippling failure away from the centreline of brace, but did not completely prevent it.

4. In thick-walled HSS (h/t = 34), FRP retrofitting provides little gain in strength, but enhances ductility by extending the plastic plateau. Delamination occurred within the GFRP plate thickness at the brace end, unless the plate had additional
bond length along the brace, in which case the plate fractured due to local bending.

5. While the FRP plates enhanced the joint strength, they had no effect on the initial stiffness.

6. In all cases bond between GFRP and steel was completely intact. As such, tapering the GFRP plate edges did not provide any additional benefit in the (h/t = 65) specimens.

7. The very high modulus of the CFRP plate limits its fracture strain, hence, it could not accommodate the web bending deformation and fractured. It appears then for this application that thick GFRP plates are more effective than thin high-modulus CFRP plates.

8. Very wide FRP plates (about 40% of span) provided flexural strengthening, in addition to stabilizing the webs, and changed failure modes from local crippling to shear failure near supports for (h/t = 65) members and to plate delamination in (h/t = 34) members.

5.4 PHASE 3 CONCLUSIONS

Based on this experimental study on T-joints in which the chord member was a Rectangular Hollow Section (RHS) with (h/t) ratio of 65 and subjected to various levels of axial compression loads, and the brace member was an axially loaded Square Hollow Section (SHS), the following conclusions are drawn:
1. Both methods investigated, namely through-wall steel bolts and bonded GFRP plates, are practical and effective in retrofitting RHS chord members in T-joints under brace and chord axial compression loads.

2. Using through-wall steel bolts retrofitting system, the joint strength was increased by 25%, 13%, and 13% when the axial compression loads in the chord were zero, 45% and 80% of its pure axial strength, respectively.

3. Using bonded GFRP plates as a retrofitting system, the joint strength was increased by 38%, 46%, and 38% when the axial compression loads in the chord were zero, 45% and 80% of its pure axial strength, respectively.

4. In general, the transverse load capacity of the chord reduces as the axial compression load increases, for both control and retrofitted specimens.

5. The peak loads reached in all control and retrofitted specimens were associated with stability failure in the form of local crippling in the webs and compression flange of the chord. In retrofitted specimens, the local crippling was shifted laterally, away from the vicinity of the brace, leading to the higher strength.

6. Generally, local crippling occurred before or just at yielding of the chord member at the joint. Therefore, for the (h/t) ratio used in this study, it can be concluded that the retrofitting technique enhanced strength significantly but did not allow the chord to exceed yielding.
References


ASTM A500 “Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes”.


Kato, B and Nishiyama, I. (1979) , The static strength of R.R. joints with large beta ratio. CIDECT program 5Y Report, Department of Architecture, Faculty of Engineering, University of Tokyo, Japan


Appendix

Design Calculations

*Properties (from Steel Tube Institute of North America)*

**Chord:**
HSS 203.2x76.2x3.2:
- Area: 1,620 mm²
- $I_x$: 8.18x10⁶ mm⁴
- $I_y$: 1.77x10⁶ mm⁴
- $S_x$: 80.5x10³ mm³
- $S_y$: 46.6x10³ mm³

$F_y = 426$ MPa (coupon tests)
Design thickness 3.0 mm

HSS 203.2x76.2x4.8: Design thickness 4.4 mm
HSS 203.2x76.2x6.4: Design thickness 5.9 mm

**Brace:**
HSS 76.2x76.2x9.5: Design thickness 8.9 mm

**Specimen Class Check:**
3mm:

*Flange:*

\[
\frac{b}{t} = \frac{76.2 - 4(3.0)}{3.0} = 21.4
\]
Web:
\[
\frac{h}{t} = \frac{203.2 - 4(3.0)}{3.0} = 63.7
\]

4.4mm:
Flange:
\[
\frac{b}{t} = \frac{76.2 - 4(4.4)}{4.4} = 13.3
\]

Web:
\[
\frac{h}{t} = \frac{203.2 - 4(4.4)}{4.4} = 42.2
\]

5.9mm:
Flange:
\[
\frac{b}{t} = \frac{76.2 - 4(5.9)}{5.9} = 8.9
\]

Web:
\[
\frac{h}{t} = \frac{203.2 - 4(5.9)}{5.9} = 30.4
\]

8.9mm:
Flange & Web:
\[
\frac{b}{t} = \frac{76.2 - 4(8.9)}{8.9} = 4.6
\]

Class Limits:
Class 1:
\[
\frac{b}{t} \leq \frac{420}{\sqrt{f_y}} \leq \frac{420}{\sqrt{426}} = 20.3
\]
Class 2:
\[ \frac{b}{t} \leq \frac{525}{\sqrt{f_y}} \leq \frac{525}{\sqrt{426}} = 25.4 \]

\[ \frac{b}{t} \leq \frac{670}{\sqrt{f_y}} \leq \frac{670}{\sqrt{426}} = 32.5 \]

3mm
∴ Flange: Class 2
    Web: Class 4

4.4mm
∴ Flange: Class 1
    Web: Class 4

5.9mm
∴ Flange: Class 1
    Web: Class 3

8.9mm
∴ Flange: Class 1
    Web: Class 1

Transverse Load Range Calculation: (Chapter 4, 3mm specimens)

- Axial Compression Resistance – Class 4

[S16-09: Cl.13.3.5.a] “Effective Area” Method – \( A_e \)

Class 3:
\[ \frac{b}{t} = 32.5 \]
\[ b = 32.5 \times t = h_e \]
\[ h_c = 32.5 \times 3.0 \]
\[ h_c = 97.5 \]

\[ A_e = A - (h - h_e) t \times 2 \]
\[ = 1620 - (191.2 - 97.5)(3) \times 2 \]
\[ = 1057.8 \text{mm}^2 \]

\[ Cr = \varnothing A_e F_y (1 + \lambda^2 n) \frac{1}{n} \]
\[ \lambda = \frac{kL}{r} \frac{F_y}{\pi^2 E} \]
\[ r_y = \frac{I_y}{A} \]

\[ \Phi = 1.0 \]
\[ A_e = 1057.8 \text{mm}^2 \]
\[ F_y = 426 \text{MPa} \]
\[ E = 209 \text{MPa (coupon test results)} \]
\[ K = 1.0 \]
\[ L = 1220 \text{mm} \]
\[ r_y = 33.05 \]
\[ \lambda = 0.53 \]

\[ \therefore C_r = 398 \text{kN} \]

[S16-09: Cl.13.8.3.b]

- **Overall Member Strength**
  i) \[ Cr: \quad K = 1.0 \]
  \[ A_e = 1057.8 \text{mm}^2 \]
  \[ L = 1000 \text{mm (fixed distance for bending)} \]
  \[ F_y = 426 \text{MPa} \]
  \[ L = 1220 \text{mm (for axial comp.)} \]
\[
r_x = \frac{I_x}{A}
\]
\[
r_x = \frac{8.18 \times 10^6}{1620}
\]
\[
r_x = 71.06
\]

\[
\lambda = \frac{kL}{r} \sqrt{\frac{F_y}{\pi^2 E}}
\]
\[
\lambda = \frac{(1)(1220)}{71.06} \sqrt{\frac{426}{\pi^2 (209000)}}
\]
\[
\lambda = 0.2467
\]

\[
C_r = \varnothing A F_y (1 + \lambda^{2n})^{-\frac{1}{n}}
\]
\[
C_r = 443 \text{ kN}
\]

\[
C_e = \frac{\pi^2 E I}{L^2}
\]
\[
C_e = \frac{\pi^2 (209000)(8.18 \times 10^6)}{(1220)^2}
\]
\[
C_e = 11,337 \text{ kN}
\]

\[
U_{1x}:
\]
\[
U_1 = \begin{bmatrix}
\omega_1 \\
1 - \frac{C_f}{C_e}
\end{bmatrix}
\]
\[
\omega_1 = 0.85
\]

\[
U_1 (C_f = 200 \text{kN}) = 0.865
\]
\[
U_1 (C_f = 350 \text{kN}) = 0.877
\]

[S16-09: Cl.14.3.4]
• Effect of thin webs on moment resistance

\[ Mr' = Mr \left[ 1 - 0.0005 \frac{A_w}{A_f} \left( \frac{h}{w} - \frac{1900}{\sqrt{\phi S}} \right) \right] \]

Where:

\[ Mr \leq \phi M_y \]

Note: due to axial compressive force in addition to moment constant 1900 is reduced by the factor:

\[ 1 - 0.65 \left( \frac{C_f}{\phi C_y} \right) \]

\[ C_y = AF_y \]
\[ C_y = 690 \text{ kN} \]

Reduction factors:
\[ 1_{(200kN)} = 0.81 \]
\[ 2_{(350kN)} = 0.67 \]

\[ M_r = \phi SF_y \]
\[ M_r = 34.29 \text{ kNm} \]

\[ Mr' = (34.29 \times 10^6) \left[ 1 - 0.0005 \frac{(2x203.2x3.0)}{(2x76.2x3.0)} \left( \frac{203.2 - 4(3)}{3} - \frac{1900(0.81^{1} or 0.67^{2})}{\sqrt{1)(80.5x10^3)}} \right) \right] \]

\[ \frac{c_f}{c_r} + \frac{U_{lx}M_{fx}}{M_{r'}} \leq 1.0 \]
Solve for $P$: (using Excel Solver)

**For $C_r = 200$ kN**

\[
\frac{200,000}{443,000} + \frac{0.865M_{fx}}{M_r'} \leq 1.0
\]

$M_r = 22.56$ kNm

Fixed Ends:

\[
M = \frac{PL}{8}
\]

$P = 180.5$ kN

Simple Supported:

\[
M = \frac{PL}{4}
\]

$P = 90.2$ kN

**For $C_r = 350$ kN**

\[
\frac{350,000}{443,000} + \frac{0.877M_{fx}}{M_r'} \leq 1.0
\]

$M_r = 8.83$ kNm

Fixed Ends:

\[
M = \frac{PL}{8}
\]

$P = 70.6$ kN

Simple Supported:

\[
M = \frac{PL}{4}
\]

$P = 35.3$ kN