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Chiew, Sing Ping; Zhao, Mingshan; Lee, Chi King

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Fatigue Performance of High Strength Steel Built-up Box T-joints

S.P. Chiew*, M.S. Zhao[#] and C.K. Lee[^]

School of Civil and Environmental Engineering, Nanyang Technological University

50 Nanyang Avenue, Singapore 639798

Emails: *cspchiew@ntu.edu.sg, [#]mzhao1@e.ntu.edu.sg, [^]ccklee@ntu.edu.sg

Abstract

In this study, experiments were conducted to study the fatigue behaviors of two full scale high strength steel built-up box T-joints. Firstly, the stress concentration factors at the joint intersection were measured. Secondly, fatigue tests were conducted with the crack growth monitored by the Alternating Current Potential Drop micro-gauge. It is found that the fatigue resistance of the high strength steel joints could be conservatively predicted by the existing fatigue design curves for normal strength steels. In addition, the higher residual stress present in the joints seems to produce little effects on the crack penetration rate in the thickness direction.

Keywords: Reheated, quenched and tempered high strength steel; Built-up box T-joint; Residual stress; Fatigue performance; Through thickness crack

1 Introduction

The quenched and tempered low alloy structural steel is one of the most favorable types of high strength steel employed to construct statically loaded structures due to its superior load carrying capacity when comparing with traditional normal strength steels. However, for applications under prolonged repeated cyclic loadings such as offshore oil and gas platforms construction, the fatigue performance will be one of the dominating factors under consideration. In the literature, two different opinions about the fatigue strength and performance of welded high strength steel structures exist. Some reports claimed that high and ultra-high strength steels manufactured by newly developed manufacturing techniques acquired better fatigue resistance than traditional steels with lower yield strength [1, 2], while the others concluded that welded high strength steels do not have better fatigue performance when comparing with conventional steels and they still need to rely on the post-welding techniques to improve their fatigue resistances [3-5].

Such contradictory opinions were the results of various arguments. Arguments favor for high strength steel include that for bare material the fatigue resistance is positively related to the strength of that material and lower stress ratio is favorable in certain cases [6], although strength is no longer dominating in the fatigue resistance. Meanwhile, it is easy to identify potential unfavorable conditions that have negative effects on the fatigue life of high strength steel structures. For example, since welding is inevitable during the fabrication of high strength steel structures as such material is often only available in plate form, the welding process may result in high residual stress [7, 8] and lead to homogeneity loss in hardness [9, 10] as well as drop of fracture toughness in the heat affected zone [10, 11]. Besides, reduction of member size and self-weight by using high strength steel often leads to reduction of stiffness and thus indirectly increases the stress fluctuations due to live load. In short, from arguments related to material property to load conditions, the accumulated favorable and unfavorable facts make the study of

fatigue performance of high strength steel structures a rather complicated problem and it is not easy to eliminate the possibility that high strength steel structures may behave differently from normal strength steel when subjected to fatigue loadings.

The main objective of this paper is to investigate the potential fatigue issues of built-up box section joints made of reheated, quenched and tempered (RQT) high strength steel RQT-S690. Experiments were carried out on two identically built-up box T-joints in order to study their fatigue behaviors: Specimen I was subjected to a high stress range while Specimen II was subjected to a lower stress range. The whole experimental study was conducted in two stages. In the first stage, static loads in three directions, namely axial force (AX), in-plane bending (IPB) and out-of-plane bending (OPB) were applied on the brace end so that the corresponding stress concentration factor (SCF) and the hot spot stresses (HSS) that eventually generated during the second stage were carefully measured. In the second stage, fatigue tests were carried out in such a way that cyclic loadings were applied and the specimens were put under the continuous monitoring of the Alternating Current Potential Drop (ACPD) crack monitoring system. In the fatigue tests, three-direction-combined cyclic loadings were applied at the brace end of the specimen to generate fatigue loadings until the specimen was failed. Finally, the test results were verified against the S-N curves developed for mild steel hot-rolled rectangular hollow section (RHS) joints with similar geometrical parameters [12].

2. Specimens and test set-up

2.1 Material and specimen fabrication

The high strength steel employed in this project, RQT-S690 structural steel plate, has nominal yield strength of 690MPa, and tensile strength between 790MPa and 930MPa. These RQT steel plates comply with the EN 10025-6 grade S690 specification [13], which is approximately

equivalent to the ASTM A514 steel [14]. The mechanical properties of the RQT-S690 steel plate obtained from standard coupon tensile test are shown in Table 1 and are compared with the properties of a typical hot-rolled rectangular hollow section manufactured to the grade S355J2H. Although the ductility of RQT-S690 is sacrificed during the hardening processes, the high strength offered better serviceability under elastic stage.

The square, high strength steel built-up box T-joints tested in this study were fabricated from RQT-S690 steel plates by flux core arc welding (FCAW). The weld profiles were designed according to the complete joint penetration groove weld method prescribed in the AWS structure welding code for steel [15]. The welding electrode employed is OK Tubrod 15.09 FCAW, which complies with EN 12535: 2000 [16]. Its mechanical properties are also shown in Table 1.

As showed in Fig. 1, the fabrication procedure of the built-up box T-joints consisted of two steps. In the first step, the steel plate was cut into long strips and they were then welded to form the brace and the chord sections. It should be noted that the backing plates is only tack-welded onto the section walls to contain the molten weld pool. The contact between the backing plates and the box-section is restricted at the weld pool area so that it has no influence on the SCF as well as the crack initiation and propagation. In the second step, the brace section was then welded onto the surface of the chord section to form the final box T-joints. The detailed dimensions and geometrical properties of final box T-joints are shown in Fig. 2.

2.2 Residual stress distribution around the weld toe

Fatigue strength is normally evaluated as the number of cycles to fracture under a given load or the endurance limit. Residual stresses are not cyclic, but they may augment or detract from applied stresses depending on their respective signs, and may affect the fatigue life. While it is commonly accepted that there is a positive relationship between strength and the level of tensile

residual stress for structural steels [3], the negative effects of residual stress on the fatigue strength of structural steel member are still arguable and cannot be confirmed completely [17]. Nevertheless, in general, it is believed that a lower level of residual stress may enhance the fatigue performance of the structure. Hence, in this study, efforts were made to measure the magnitude of residual stress. The semi-destructive hole drilling technique [18] was employed to measure the residual stress distributions around the weld toe of the brace-chord intersection after fabrication. The directions of the measured residual stresses are perpendicular to the weld toe, i.e. normal to the potential cracks. More details regarding the residual stress measurement procedures for these two joints could be found in the paper by Lee et al. [19].

The locations of the residual stress measurement and the residual stress distributions on these two specimens obtained are shown in Fig. 3. In general, the residual stress level of Specimen I was lower than that of Specimen II. In the fatigue tests, different levels of the cyclic loadings were applied on the two specimens but the load combinations are carefully designed so that the surface cracks would eventually be initiated at corner C4 for both specimens, where both the residual stress values and their trends around that corner were comparable. In this way, the effects of residual stress could then be evaluated implicitly by comparing the test results of the two specimens and the predicted fatigue lives of traditional hot-finished steel joint with same geometries but with low residual stresses, while the synergy mechanism between residual stress and different levels of tensile stresses if there is any, can be evaluated by comparing the fatigue performance of the two specimens.

2.3 Test setup-up

A specially designed test rig that could apply AX, IPB and OPB independently or any combination of these basic loads was employed to test the built-up box T-joints and is shown in

Fig. 4. This rig was capable of applying static loads to determine the SCF and HSS, as well as designated cyclic loadings to investigate the fatigue performance and fracture behavior of the joint after cracks are formed. To generate the desired combinations of different loading types, three independent actuators were installed in the AX, IPB and OPB directions (Fig. 4). Both Actuator AX and Actuator IPB had a maximum loading capacity of 250KN while Actuator OPB had a loading capacity of 150KN.

3. Static tests

3.1 Strain gauge system deployed

While in the static test the main propose was to measure the hot spot strain distribution near the brace-to-chord intersection, additional 12 strain gauges were installed at the mid-span of the brace (3 on each face) to measure the nominal stress of the brace. As the strain distribution near the weld toe is expected to be highly non-linear, the quadratic perpendicular strain/stress extrapolation method was employed to measure the SCFs. By following the CIDECT guide [12], three strain gauges were arrayed at each location along the line perpendicular to the weld toe at distances equal to $0.4t$, $0.9t$ and $1.4t$ (t is the thickness of the chord or brace). For Specimen I, 40 and 20 sets of strain gauge were installed on the chord side and the brace side, respectively. Based on the results obtained from Specimen I, for Specimen II, the strain gauge numbers were reduced so that only 28 sets and 20 sets were respectively instead on the chord side and the brace side. The detailed locations of the strain gauges installed are shown in Fig. 5.

3.2 Loadings applied and SCF calculation

In the static test, in order to obtain the SCF distribution along the brace-chord intersection, a series of AX, IPB and OPB loads and combination of them were applied. Under each load

condition, the maximum load was determined when the maximum strain readings reached roughly 70% of the yield strain, which was taken as the 0.2% strain offset strength. Six steps were taken during the loading and unloading phases for each load case. At each step, strain readings were recorded after the load being maintained for 3 minutes. The real-time strain values obtained from the data logger confirmed that both the specimens and the strain gauges were installed properly and the test rig was operating correctly for all the load combinations since almost no residual strain readings were observed after unloading.

The strain gauge readings were analyzed and the hot spot strain (HSSN) was obtained using the quadratic extrapolation method so that

$$HSSN = 2.52 \times SN(0.4t) - 2.24 \times SN(0.9t) + 0.72 \times SN(1.4t) \quad (1)$$

where $SN(0.4t)$, $SN(0.9t)$ and $SN(1.4t)$ are the strain values corresponding to the locations along the line perpendicular to the weld toe at distances equal to $0.4t$, $0.9t$ and $1.4t$, respectively.

The strain concentration factor (SNCF) was then calculated as

$$SNCF = \frac{HSSN}{SN_n} \quad (2)$$

where SN_n is the nominal strain measured in the brace and is calculated as the averaged strain recorded by the 12 strain gauges at the mid span of the brace. Assuming that $\xi_{\perp}$ and $\xi_{\parallel}$ are respectively the maximum and minimum principal strains at the weld toe, the value of SCF could be computed as

$$SCF = SNCF \frac{(1 + \nu \xi_{\parallel} / \xi_{\perp})}{(1 - \nu^2)} = SNCF \times SSCF \quad (3)$$

where $\nu=0.3$ is the Poisson's ratio and SSCF is the stress-strain conversion factor [20]. In this study the proposed value by Dutta [20] of $SSCF=1.1$, which is obtained by setting $\xi_{\parallel}=0$, was

adopted as such value has been founded to give reliable results in previous study of hollow section joints [21, 22].

3.3 SCF distributions

In all tested load cases, SCFs at the brace side were found to be smaller than those at the chord side, while the values were comparable to those of S355J2H steel RHS joints [23]. The SCF distribution for Specimen I and Specimen II under pure AX, IPB and OPB conditions are shown in Fig. 6 and Fig. 7, respectively. From Figs. 6 and 7, it can be seen that the HSS distributed symmetrically according to the IPB and the OPB axes, and concentrated at the corners.

The SCF value obtained at the corner C4 at critical locations A and E at the brace side and B, C and D (Fig. 5) at the chord side are listed in Table 2 and are compared with the values calculated by using the SCF equations suggested by Wingerde [22] and adopted by the CIDECT design guide [12]. From Table 2, it can be observed that the test values generally agreed with the equation values, while the relative percentage differences for most locations were less than 30%. More detailed analysis of SCF at other corners showed similar conclusion except that the relative percentage differences were slightly larger for corners C1, C2 and C3. On the brace side, the SCF at A and E were found to be slighter higher than predictions by the equations and other locations at the brace. Hence, it could be concluded that the SCF at A and E could represent the maximum SCFs that could appear at the brace side. As for the chord side, when AX and IPB were applied the SCF values at D appeared to be consistently less than those at B and C. However, the SCF at D was consistently higher than those at B and C when pure OPB is applied. For the maximum SCF locations, both the measured SCF distribution and the CIDECT design guide equation consistently show that they are located at B for AX and C for IPB. Note that no prediction of the maximum SCF value under OPB is given by the CIDECT design guide.

3.4 Validation of the superposition method

Based on the superposition method for SCF [21], the stress at a given point (p) under any combined load cases can be calculated as

$$\sigma(p) = SCF_{AX}(p) \times \sigma_{n-AX} + SCF_{IPB}(p) \times \sigma_{n-IPB} + SCF_{OPB}(p) \times \sigma_{n-OPB} \quad (4)$$

where $SCF_{AX}(p)$, $SCF_{IPB}(p)$ and $SCF_{OPB}(p)$ are, respectively, the SCFs at point p for AX, IPB and OPB loads. σ_{n-AX} , σ_{n-IPB} and σ_{n-OPB} are the corresponding nominal stresses. In this study, the validity of Eqn. (4) is checked by comparing the test result obtained under two load combinations with the corresponding values predicted by Eqn. (4). The two loading combinations employed are AX=120KN, IPB= -3.6KN and OPB=3.6KN for Specimen I and AX=80KN, IPB=-1.2 KN and OPB=3.6KN for Specimen II. The combined HSS distributions obtained are shown in Figs. 8 and 9, respectively. From Figs. 8 and 9, it can be seen that the stress distribution obtained by the superposition method agreed well with the experimental measurements in both cases and this confirmed that the superposition method can be applied to the high strength steel built-up box T-joint.

4. Fatigue tests

4.1 Test schedule and crack monitoring system

After the static tests were completed, fatigue test were conducted for both specimens in air using sinusoidal constant amplitude combined loadings until through thickness cracks were developed on the chord of the joints. The cyclic loading cases applied to the specimens are listed in Table 3. From Table 3, it can be seen that the zero load was applied as the minimum stress state for the AX, IPB and OPB loading cases. Hence, the corresponding HSS range distributions are the same as the maximum stress state, which are exactly corresponding to the load combinations used in

the superposition validation as shown in Figs. 8 and 9. From Fig. 8, one can see that Specimen I was tested under a higher maximum stress range of 535.3MPa which equals to 69.6% of the 0.2% strain offset yield strength. For Specimen II, Fig. 9 shows that it was subjected to a lower maximum stress range of 325.5MPa which is corresponding to a stress ratio of 42.3%. It should be noted that the maximum stress range applied to Specimen I is equivalent to 150.8% of the nominal yield strength of the S355J2H steel (355MPa). While for Specimen II, the maximum stress range applied is actually 91.7% of the nominal yield strength of the S355J2H steel.

In order to monitor and record the initiation and propagation of the surface crack, the ACPD technique [24] was employed in the fatigue tests. Based on the peak HSS locations acquired from the static test, 32 sets of ACPD probes were installed around the chord-brace intersections near corner C4, as shown in Fig. 10. The probes were placed at equal intervals of 10mm along the weld toe, making a total monitored length of 320mm. The plan views of the ACPD probes locations for both specimens are shown in Fig. 11 (with the final crack paths recorded). During fatigue tests, crack depth data were recorded automatically by the control software at an interval of 10min which is equivalent to 200 cycles of loadings for Specimen I and 300 cycles for Specimen II.

4.2 Fatigue test results and comparison with similar hot-finished mid steel joints

Fig. 12 shows the crack surface that is cut from Specimen I after the fatigue test. Similar to the case of hot-finished S355J2H square-to-square hollow section (SSHS) T-joints tested by Chiew et al. [23], the crack surface is found to be a 3D curve surface. The cracks penetrated the C4 corner of Specimen I and a through thickness crack (TTC) was eventually formed but for Specimen II no TTC was developed until the end of the fatigue test. To facilitate the description of the fatigue crack depth development, some geometrical parameters of a surface crack are

defined here first [23]. As shown in Fig. 13, a typical surface crack is initiated at the weld toe and penetrates into the chord plate. The *depth of the crack*, which is defined as the vertical distance from the crack tip to the top surface of the chord is denoted as a . The length of the crack curve (from the weld toe to the crack tip) is denoted as a' and is called the *curved crack depth*. It should be noted that the crack depth that was monitored continuously by the ACPD is the curved crack depth a' , while only the final value of a could be measured when the cracked joint was opened up for inspection after the test. Obviously, from Fig. 13, $a' \geq a$ and usually for a sufficiently deep crack, $a'/t_0 > 1$ while $a/t_0 \leq 1$ as the crack profile at any cross section of the crack surface is a 3D curve. Furthermore, at any cross section of the crack surface, the locations of the maximum a' and a are at the crack tip.

From previous studies of hot-finished S355J2H SSHS T-joints, it was found that the growth of the crack surface can be divided into two phases [12]: the *crack initiation phase*, during which micro-cracks may form, and a *crack propagation phase*, which may lead to visible cracks or even critical sizes that result in failure. In this study, the initiation and propagation of the surface cracks were monitored by the ACPD so that the curved crack depths, a' , at all monitored points (P-1 to P-32 of Fig. 11) at different load cycles were recorded. In particular, the *crack initiation phase* was defined as the number of cycles between the first load cycle to the cycle when the ACPD detected the crack (N_i), i.e. a' approximately equals to 0.1mm. The crack propagation phase was defined as the period between the crack initiation and the numbers of cycle until the formation of TTC (N_f). Or in the case of Specimen II, the numbers of load cycle when the joint is failed and the fatigue test was stopped. A summary of the test results is listed in Table 4 together with the test results obtained by Chiew et al. [23] in which two *16mm thick*, hot-finished S355J2H SSHS T-joints (S355J2H SI and S355J2H SII of Reference [23]) of similar size were

tested to failure. In addition, in order to conduct a more detailed investigation of the fatigue behaviors of the two joints, the following graphs are plotted according to the ACPD records:

- (1) The crack propagation and penetration profiles of the whole crack front: graphs of the a' development history (Figs. 14 and 15)
- (2) The crack penetration curves at the deepest point: graph of a' against N at the deepest point of the crack front, where N is the number of load cycles applied (Fig. 16).

From Figs. 14 and 15, it can be seen that while Specimens I and II were subjected to different level of stress ranges, the load combinations were designed in such a way that the peak HSS was generated at the corner C4 for both specimens. The surface cracks were initiated at the region between P-18 and P-20 for Specimen I and the region between P-7 and P-9 for Specimen II as predicted by the HSS plots (Fig. 8 and 9 respectively) and then gradually propagated towards the corner C3 along the weld toe. The ratio N_i/N_f for Specimens I and II are 27.02% and 23.5%, respectively, which was similar to that of *16mm thick S355J2H SI* (25%) specimen. The *16mm thick S355J2H SII* specimen showed a shorter fatigue crack initiation life ($N_i/N_f \approx 10\%$) because of the higher peak HSS to f_y ratio ($>90\%$) and the different loading methods applied [23]. After the crack initiation, for the high strength steel Specimens I and II, the cracks developed into the chord plate gradually with a relatively steady penetration rate for a' . However, the location of the TTC for Specimen I and the deepest crack locations for Specimen II turned out to be slightly shifted from the initiation points. The penetration rate of the deepest point in Specimen II was comparable to those in the *16mm thick S355J2H SI* and *SII* (Fig. 16) as all three joints were subjected to similar level of peak HSS (316MPa to 336MPa). From Fig. 16, it can be seen that the penetration rate of Specimen I was much higher than the other three specimens and this can be explained by the much higher peak HSS applied ($>500\text{MPa}$, Table 4). At the final stage of the

test when the remaining uncracked part of the chord became thin enough, Specimen I was penetrated with a significantly higher penetration rate. This phenomenon was not observed in the other three specimens that were not fully penetrated by fatigue cracks.

For Specimen I, the fatigue failure occurred after 158,400 cycles when the TTC was detected and this is more than 600% of the numbers of cycles predicted by the CIDECT 12mm S-N curve [12].

For Specimen II, since a lower stress range was used, the fatigue test was stopped after 390,600 cycles were applied which is equivalent to 347.37% the value predicted by the CIDECT guide.

The fatigue life of both specimens obtained from the tests were plotted in Fig. 17 together with the fatigue life calculated using the general equations provided by the American Railway Engineering and Maintenance-of-way Association (AREMA) manual (Section 1.3.13, stress category B for build-up members using the general S_R-N equation given in Section 9.7.3.4.2) [25] and the standard S-N curve of the CIDECT guide [12] for the standard mild steel RHS T-joint formed with the same geometrical parameters. For comparison, the results of the two 16mm thick hot-finished SSHS joints S355J2H SI and SII tested by Chiew et. al [23] were also plotted in Fig. 17. From Fig. 17, it is found that the fatigue life of the studied specimens predicted by AREMA manual agreed well with the CIDECT S-N curve with thickness equals to 12mm and the CIDECT guide is conservative in the prediction of the fatigue life of the high strength steel RQT-S690 specimens tested in this study. In fact, Fig. 17 shows that even the numbers of load cycles for crack initiation (N_i) of both 12mm thick RQT-S690 specimens are already close to the CIDECT 12mm fatigue life curve. When comparing the actual fatigue life, Fig. 17 shows that the fatigue life (N_f) of the 12mm high strength steel RQT-S690 Specimen II was marginally 4% and 12% lower than the 16mm S355J2H SII and SI, respectively. Finally, in Fig. 17, the horizontal line corresponding to a stress range of 355MPa was also plotted and the two data points corresponding to Specimen I (with HSS equals to 535.3MPa) are clearly above this line. This

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3 implies that if the minimum stress state is a *zero stress state*, a joint formed by using S355 (or
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5 even S460) steel will not be able to provide any significant fatigue resistance since the materials will
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7 be yielded in the first load cycle. In this case, only high strength steel such as RQT-S690 with
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9 yield stress significantly higher than the maximum tensile stress applied could only be able to
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11 provide sufficient fatigue resistance.
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17 *4.3 Effects of residual stress on the fatigue performance of RQT-S690*

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19 Similar to other common quenched and tempered steels, RQT-S690 gained high strength
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21 microstructures through complicated heat treatments but its chemical compositions are not much
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23 different from common hot-finished normal strength steels such as S355. However, these
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25 processes also bring issues of high residual stresses, especially after welding [8, 19]. On the other
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27 hand, hot-finished hollow sections usually have relatively low level of residual stresses, because
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29 they are either formed by hot rolling or through cold working followed by stress relieving heat
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31 treatments. In general, it is accepted that the maximum cyclic tensile stress affects the time to
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33 crack initiation, while the stress range controls the rate of crack growth [17]. Although residual
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35 stresses are not cyclic, they may augment or detract from applied stresses depending on their
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37 respective signs, and might affect the fatigue life. Hence, it is suspected that the synergy between
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39 residual stress caused by welding and the tensile HSS caused by external loads might affect the
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41 fatigue life of the RQT-S690 box joints.
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47 The effect of residual stress on the fatigue performance is further studied by investigating the
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49 crack development histories of the tested specimens. Figs. 18 and 19 respectively show the crack
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51 penetration curves of three locations on Specimen I (P-18, P-23 and P-28) and Specimen II (P-8,
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53 P-13 and P-18) where residual stress values are measured. From Figs. 18 and 19, it could be seen
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55 that while the numbers of cycle needed for crack initiation varied greatly due to the different
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3 levels of tensile HSS applied at different locations, the crack penetration curves for all locations
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5 selected were roughly parallel for the whole crack propagation stage. To investigate more closely
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7 the effects of residual stress on the crack penetration rate, the crack penetration rates (calculated
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9 as mm of a' per 10^5 cycles) of the first 3mm curved crack depth (where residual stress is normally
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11 highest in the depth direction) for the six selected locations are calculated and listed in Table 5.
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13 From Table 5, it can be seen that for Specimen I, while the levels of residual stress at P-18 and P-
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15 23 are similar, P-23 showed a much higher crack penetration rate. In addition, P-23 and P-28
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17 showed almost the same crack penetration rate despite that they were under sufficiently different
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19 level of residual stresses. For Specimen II, Table 5 again shows that no obvious relationship
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21 could be detected between the residual stress and the initial crack penetration rate. In addition, by
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23 considering the fact that the crack shape parameters (a'/t_0 and a/t_0 , Table 4), the N_i and N_f values
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25 of the RQT-S690 Specimen II and the S355J2H SI are very similar despite that the level of
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27 residual stress for the RQT-S690 Specimen II should be higher than that for the S355J2H SI, it
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29 can be concluded that residual stress showed no sufficient effect on the crack penetration rate for
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31 these two joints. Finally, it should be noted that although the residual stress levels at corner C4 of
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33 both RQT-S690 specimens were comparable (Fig. 3), the fatigue life of Specimen I should be
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35 less affected by the residual stress due to the higher HSS, since it is well known that the effect of
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37 residual stress decreases with increased of applied stress [17].
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47 5. Conclusions

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49 In this study, the fatigue performance of two full-size built-up box section joints made of high
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51 strength steel RQT-S690 was studied experimentally. Firstly, static tests were carried out to study
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53 the stress concentration phenomena near the weld toe of the brace-chord intersection. The results
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55 indicated that while the CIDECT guideline for welded hollow section joint was still generally
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conservative, the measured SCFs were not very close to the predicted values by the CIDECT guideline. The differences varied from 5% to 40% and this could be caused by the relatively sharp corners of the built-up sections. It was also shown that the superposition method could be used to predict the SCF and HSS distributions for combined loadings for high strength steel built-up box T-joints. Secondly, fatigue tests were conducted for the two specimens fabricated and the crack propagation and penetration rates were recorded by the ACPD crack monitoring system. Specimen I was subjected to a higher stress range (535.3MPa) while Specimen II at a lower stress range (325.5MPa). Similar to the case of hot-finished section S355J2H T-joint, the fatigue test results showed that the HSS distribution was again a very good indicator to predict the crack initiation position. The fatigue test also indicated that the crack propagation behaviors of the high strength steel built-up box T-joints were comparable to those of hot-finished S355J2H SSBS T-joints with similar geometries. Furthermore, it was also found that the fatigue life of the high strength steel built-up box T-joints tested could be predicted conservatively by using the CIDECT guide S-N curve developed based on lower strength steels. Finally, further analysis of the crack propagation curves at different locations showed that residual stress seems to have little effects on the crack depth development and the crack penetration rate, as residual stress was supposed to be released when the crack was formed.

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Table 1 Mechanical properties of RQT-S690 and S355J2H steel and the electrodes used

	f_y or $f_{0.2}$ (MPa)	$f_{2.0}$ (MPa)	f_u (MPa)	E (GPa)	Elongation
RQT-S690	769.0	817.1	849.8	201.3	14.7%
S355J2H	423.4	461.6	535.1	204.5	34.2%
Electrode	780.0	-	840.0	-	19.0%

Table 2 Values of SCF under pure AX, IPB and OPB at critical locations near Corner C4

Locations (Fig. 5)	AX					IPB					OPB	
	SI	SII	Eqn.	Diff %		SI	SII	Eqn.	Diff %		SI	SII
				SI	SII				SI	SII		
A (brace)	11.2	9.9	<u>12.8</u>	14.3	29.3	10.1	<u>11.7</u>	8.4	-16.6	-28.0	<u>10.3</u>	8.4
E (brace)	11.2	9.1	<u>12.8</u>	14.3	40.7	<u>11.8</u>	10.7	8.4	-28.8	-21.5	<u>11.4</u>	8.8
B (chord)	22.1	18.6	<u>24.4</u>	10.4	31.2	<u>17.1</u>	13.6	14.7	-13.9	8.3	<u>10.3</u>	8.4
C (chord)	19.8	15.1	<u>21.0</u>	5.8	38.8	17.2	18	<u>19.0</u>	10.2	5.3	<u>11.4</u>	8.8
D (chord)	11.9	<u>12.3</u>	9.6	-19.2	-21.8	7.6	<u>10.6</u>	9.4	23.5	-11.4	14.2	<u>16.5</u>

Maximum values at a given location are underlined

Maximum values for the brace and the chord SCFs are bolded.

SI: Results from Specimen I

SII: Results from Specimen II,

Eqn.: From SCF equations suggested in References 12 and 13.

Table 3 The fatigue load cases applied to the investigated joints

	Specimen I (KN)	Specimen II (KN)
AX	$AX = 60 \pm 60 \sin \theta$	$AX = 40 \pm 40 \sin \theta$
IPB	$IPB = -1.8 \pm 1.8 \sin \theta$	$IPB = -0.6 \pm 0.6 \sin \theta$
OPB	$OPB = 1.8 \pm 1.8 \sin \theta$	$OPB = 1.8 \pm 1.8 \sin \theta$
Frequency	1/3 Hz	1/2 Hz

Table 4 Summary of the fatigue test results

	RQT-S690 Specimen I	RQT-S690 Specimen II	S355J2H SI	S355J2H SII
Chord dimensions (mm)	300×300× 12	300×300× 12	350×350× 16	350×350× 16
Brace dimensions (mm)	200×200× 12	200×200× 12	250×250× 16	200×200× 16
f_y (MPa, measured value)	769.0	769.0	366.0	370.0
Peak HSS and stress range (MPa)	535.3	325.5	315.9	336.2
Peak HSS/ f_y	69.6%	42.3%	86.3%	91%
No. of cycles when crack initiated (N_i)	42,800	91,800	100,000	44,600
No. of cycles upon TTC formed or fatigue failure occurred (N_f)	158,400	390,600	407,517	446,508
N_i/N_f	27.02%	23.50%	25%	10%
No. of cycles to failure predicted by CIDECT	24,428	112,444	94,700	78,561
Test/CIDECT	648.44%	347.37%	430.32%	568.36%
Final a' (mm)	20.6	15.9	20.0	20.5
Final a'/t_0	1.69	1.32	1.25	1.28
Final a/t_0	1.0	0.82	0.75	0.79

For RQT-S690, the 0.2% strain offset strength $f_{0.2}$ is adopted as f_y , since its stress-strain curve has no distinguishable yield stage.

Table 5 The initial crack penetration rates of selected locations

Specimen I	P-18	P-23	P-28
Residual stress (MPa)	63.6	69.4	95.7
Crack Penetration rate for the first 3mm (mm/ 10^5 cycles)	15.40	8.98	9.02
Specimen II	P-8	P-13	P-18
Residual stress (MPa)	33.5	94	121.5
Crack Penetration rate for the first 3mm (mm/ 10^5 cycles)	3.43	6.55	4.55

(a) Step 1: forming the box sections

(b) Step 2: assembling the T-joint

Fig. 1 Fabrication procedures for the built-up box T-joints
(All dimensions are in mm)

Geometry	b_o	t_o	b_1	t_1	$\beta = \frac{b_1}{b_o}$	$2\gamma = \frac{b_o}{t_o}$	$\tau = \frac{t_1}{t_o}$
Parameters	[mm]	[mm]	[mm]	[mm]			
	300	12	200	12	0.667	25	1.0

Fig. 2 Specimen geometry

(a) locations for residual stress measurements

(b) The measured residual stresses distributions

Fig. 3 Residual stress distributions around the weld toe of the specimens

Fig. 4 Test rig with test specimen installed

(a) Locations of strain gauges on Specimen I

(b) Locations of strain gauges on Specimen II

Fig. 5 Strain gauge system installed to capture the SCF around the brace-chord intersection

(a) SCF on the brace side

(b) SCF on the chord side

Fig. 6 SCF on brace and chord of Specimen I

(a) SCF on the brace side

(b) SCF on the chord side

Fig. 7 SCF on brace and chord of Specimen II.

Fig. 8 Hot spot stress distribution for Specimen I

Fig. 9 Hot spot stress distribution for Specimen II

Fig. 10 ACPD crack microgauge system

(a) Specimen I

(b) Specimen II

Fig. 11 Plan views for ACPD probes locations

Fig. 12 The crack surface along the weld toe

Fig. 13 Geometrical parameters of a surface crack

Fig. 14 The crack depth development curves of Specimen I

Fig. 15 The crack depth development curves of Specimen II

Fig. 16 The crack penetration curves of the crack tips

Figure 17 Fatigue test results in S-N diagram

Fig. 18 The crack depth development curves for certain locations on Specimen I

Fig. 19 The crack depth development curves for certain locations on Specimen II