CFRP STRENGTHENING OF STEEL BEAMS
WITH WEB OPENINGS

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<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
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<tbody>
<tr>
<td>$A_r$</td>
<td>Area of stiffening</td>
</tr>
<tr>
<td>$b_r$</td>
<td>Width of the stiffener</td>
</tr>
<tr>
<td>$b_f$</td>
<td>Flange width</td>
</tr>
<tr>
<td>CFRP</td>
<td>Carbon fibre reinforced polymer</td>
</tr>
<tr>
<td>$D$</td>
<td>Local damage within an element in the finite element method</td>
</tr>
<tr>
<td>$d$</td>
<td>Web height</td>
</tr>
<tr>
<td>$d_r$</td>
<td>Stiffener’s depth from the flange</td>
</tr>
<tr>
<td>$EDC$</td>
<td>End debonding of CFRP</td>
</tr>
<tr>
<td>ENA</td>
<td>Equivalent neutral axis</td>
</tr>
<tr>
<td>$E_{CFRP}$</td>
<td>Elastic modulus of carbon fibre reinforced polymer</td>
</tr>
<tr>
<td>$E_{steel}$</td>
<td>Elastic modulus of steel</td>
</tr>
<tr>
<td>ETF</td>
<td>End-two-flange</td>
</tr>
<tr>
<td>$E_x$</td>
<td>Elastic modulus in the longitudinal direction</td>
</tr>
<tr>
<td>$E_y, E_z$</td>
<td>Elastic modulus in the transverse directions</td>
</tr>
<tr>
<td>$e$</td>
<td>Opening eccentricity</td>
</tr>
<tr>
<td>FRP</td>
<td>Fibre reinforced polymer</td>
</tr>
<tr>
<td>$f$</td>
<td>Member natural frequency</td>
</tr>
<tr>
<td>$f_y$</td>
<td>Yield stress</td>
</tr>
<tr>
<td>GFRP</td>
<td>Glass fibre reinforced polymer</td>
</tr>
<tr>
<td>$G_{xy}$</td>
<td>In-plane shear modulus</td>
</tr>
<tr>
<td>$G_{xz}$, $G_{yz}$</td>
<td>Shear modulus in the transverse directions</td>
</tr>
<tr>
<td>HM – CFRP</td>
<td>High modulus carbon fibre reinforced polymer</td>
</tr>
<tr>
<td>$h$</td>
<td>Beam depth</td>
</tr>
<tr>
<td>$h_{adh.}$</td>
<td>Adhesive height</td>
</tr>
<tr>
<td>$h_{CFRP}$</td>
<td>Height of carbon fibre reinforced polymer</td>
</tr>
<tr>
<td>$h_o$</td>
<td>Opening depth</td>
</tr>
<tr>
<td>ITF</td>
<td>Interior-two-flange</td>
</tr>
<tr>
<td>$l_F$</td>
<td>Stress-based failure criterion of FRP</td>
</tr>
<tr>
<td>$K$</td>
<td>Beam’s stiffness</td>
</tr>
<tr>
<td>LTB</td>
<td>Lateral torsional buckling</td>
</tr>
<tr>
<td>LSB</td>
<td>Lite steel beam</td>
</tr>
<tr>
<td>LVDT</td>
<td>Linear variable displacement transducer</td>
</tr>
</tbody>
</table>
Anchorage length of the stiffener
Bending moment
Nominal bending capacity of a steel member with a web opening
Bending capacity of a steel member without an opening
Multi-points constraint
Primary beam
Quad damage stress criteria (damage initiation)
Rectangular hollow section
Reinforced opening with carbon fibre reinforced polymer plates
Reinforced opening with steel stiffeners
Longitudinal shear strength
Secondary beam
Stress degradation (damage evolution scalar)
Square hollow section
Shear zone
Top flange yielding
Adhesive thickness
Thickness of carbon fibre reinforced polymer
Flange thickness
Nominal stress in three directions
Stiffener thickness
Web thickness
Ultra-high modulus carbon fibre reinforced polymer
Vierendeel failure
Longitudinal compressive strength
Yielding of opening corner
Web local buckling
Web post buckling
Stress limit in x-direction
Stress limit in y-direction
Longitudinal tensile strength
Transverse compressive strength
Transverse tensile strength
Z \quad \text{Plastic modulus of steel section}

\textbf{Greek symbols:}

\begin{align*}
\delta_m^0 & \quad \text{Effective relative displacement at the initiation of failure} \\
\delta_m^f & \quad \text{Effective relative displacement at the end of failure} \\
\delta_m^{\text{max}} & \quad \text{Maximum effective displacement} \\
\Delta A_s & \quad \text{Change in shear area} \\
\Delta F & \quad \text{Change in force} \\
\Delta U & \quad \text{Change in displacement} \\
\varepsilon_1 & \quad \text{Strain in the longitudinal direction} \\
\varepsilon_2 & \quad \text{Strain in the transverse direction} \\
\varepsilon_{\text{nominal}} & \quad \text{Nominal strain} \\
\varepsilon_{\text{true}} & \quad \text{True strain} \\
\mu \varepsilon & \quad \text{Microstrain} \\
\sigma_1 & \quad \text{Stress in the longitudinal direction} \\
\sigma_2 & \quad \text{Stress in the transverse direction} \\
\sigma^{\text{max}} & \quad \text{Tensile strength of the adhesive} \\
\sigma_{\text{nominal}} & \quad \text{Nominal stress} \\
\sigma_{\text{true}} & \quad \text{True stress} \\
\sigma_{xx} & \quad \text{Longitudinal stress in x-direction} \\
\sigma_{yy} & \quad \text{Longitudinal stress in y-direction} \\
\sigma_{xy} & \quad \text{Shear stress in x-y direction} \\
\tau_{\text{max}} & \quad \text{Maximum shear strength of the adhesive}
\end{align*}
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ABSTRACT

During the service life of a building structure, the introduction of web openings into existing steel floor beams is often required to allow for new services such as air conditioning, sprinkler systems, telecommunications etc. However, the presence of large openings in the webs can significantly reduce the shear and bending strength capacity of the beams. Traditionally, the welding of additional steel plates around the opening areas is adopted as a means of strengthening and stiffening. This not only presents practical difficulties but can induce residual stresses which weaken fatigue performance of the section.

The aim of this study is to explore the applicability of externally bonded carbon fibre reinforced polymer composites (CFRP) as an alternative means of strengthening for web openings in steel flexural elements.

A numerical and experimental investigation was employed in the research reported in this thesis to achieve this aim. Using a non-linear finite element approach, the effects of strengthening arrangements and CFRP lengths were investigated with a view to determine the most structurally efficient layout of CFRP strengthening. The experimental tests were conducted later on four specimens, one control specimen (without opening and un-strengthened) and the rest with different web opening positions and CFRP strengthening.

In order to further understand the limits of applicability of this approach, further numerical modeling was also performed to assess the proposed strengthening method when applied to full-scale steel beams with web openings at mid-span or in the high shear zone. The series of beams examined comprised the types of spans which are common in commercial frame buildings.
The outcomes of this research show that the CFRP strengthening method is capable of recovering and in some cases exceeding the strength of the beam to that before the introduction of web openings. Similarly, the strengthening method increases the stiffness of the altered beam, thus bringing deflections back to a level similar to the beam before introduction of openings. In many of the strengthened cases, a reduction in ductility was observed; this can in part be due to over-specification of strengthening thickness and thus demonstrates the importance of choosing the optimum strengthening arrangement. In parallel with the reduction in ductility, it was observed that changes in failure mode and position can occur with certain strengthening arrangements in comparison to the unaltered beam. In the application of the method to full scale beams, the results of the simulations suggest that the technique of CFRP strengthening can be used effectively as a realistic and practical alternative solution to retrofitting of existing steelwork. When the CFRP strengthened beams were compared to those with traditional welded steel plate strengthening designed according to SCI P355 (2011), the results were found to be similar in terms of load capacity and overall failure mode.
DECLARATION

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DEDICATED

TO

MY PARENTS, WIFE AND KIDS

FOR THEIR

LOVE, SACRIFICES AND PRAYERS
ACKNOWLEDGEMENTS

First and foremost, I wish to give all the praise to ALLAH Almighty for giving me the patience, strength and time to complete this research.

I would like to express my personal gratitude to my supervisors, Dr Lee S. Cunningham and Dr Martin Gillie for their expert supervision, continued guidance, consistent motivation and valuable advices during all phases of the research work. They provided me with all kinds of support during my PhD study.

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Mohammed Altaee
PUBLICATIONS


CHAPTER 1

INTRODUCTION

1.1 Background

Towards extending service life and future adaptation, existing steel structures may often require the creation of voids in the web of floor beams to accommodate services such as ventilation ducts, electrical and data communication systems, fire protection systems, heating and cooling systems and instrumentation cables. However, the existence of web openings may have a significant detrimental effect whereby the beam may be weakened in the vicinity of the opening due to the reduction in section. Traditionally the welding of steel plates around openings is used to compensate for the loss of section as outlined in SCI P355 (Lawson & Hicks 2011). This conventional approach induces residual stresses which can weaken the fatigue performance (Fattouh & Shahat, 2015). Further disadvantages of this strengthening method are the practical and safety issues associated with welding at height.

The bonding of fibre reinforced polymer (FRP) composite plates or sheets represent a potentially advantageous alternative to the traditional method of strengthening steelwork. FRP composites have many advantages over welding steel plates. FRP, depending on the constituents, usually has a much higher strength-to-weight ratio, making it a much easier material to handle. This technique also avoids welding at height issues making it overall much quicker and easier to install. The enhanced
durability aspects of FRP materials are also attractive particularly in regard to extended service life of structures.

### 1.2 Fibre reinforced polymers

Fibre reinforced polymer (FRP) is a composite material that is composed of two components: fibre reinforcement and polymeric resin (matrix), as shown in Figure 1-1. The fibres enhance the strength and stiffness of the FRP composites whilst the matrix allows the load transfer between the individual fibres, and protects them from mechanical and environmental damage. The properties of the FRP composite depend on the properties of the matrix and the fibre as well as the fibre volume ratio and the fibre orientation.

![Figure 1-1 Schematic and microscopy image of FRP composite (Abed, 2012).](image)

The most commonly used fibre constituents for FRP in civil engineering applications are glass fibre (resulting in GFRP) or carbon fibre (resulting in CFRP). Other less commonly used fibres include aramid fibre, while newer naturally derived fibres such as basalt fibres are starting to receive attention in the research community. These types of composite have a wide range of material properties and prices. Carbon fibre polymer has been used heavily for strengthening of steel structures, primarily
because CFRP materials have relatively higher stiffness. CFRP laminates for application on steel structures are available as pultruded plates and sheets as shown in Figure 1-2. Pultruded plates are typically bonded to the steel using a two part epoxy adhesive, while the sheets can be bonded the same way or with a compatible adhesive film (Hollaway & Cadei 2002).

![CFRP sheet](image1.png) ![CRRP plate](image2.png)

(a) CFRP sheet  (b) CRRP plate

Figure 1-2 CFRP products used in strengthening.

Depending on the modulus of the CFRP material, researchers have used terms such as normal, high and ultra-high modulus to categorise laminates as given in Table 1-1.

<table>
<thead>
<tr>
<th>Category</th>
<th>$E_{CFRP}$ to $E_{steel}$</th>
<th>$E_{CFRP}$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low modulus</td>
<td>$E_{CFRP} \leq 0.5E_{steel}$</td>
<td>$\leq 100$</td>
</tr>
<tr>
<td>Normal modulus</td>
<td>$0.5E_{steel} \leq E_{CFRP} \leq E_{steel}$</td>
<td>100-200</td>
</tr>
<tr>
<td>High modulus</td>
<td>$E_{steel} &lt; E_{CFRP} \leq 2E_{steel}$</td>
<td>200-400</td>
</tr>
<tr>
<td>Ultra-high modulus</td>
<td>$E_{CFRP} &gt; 2E_{steel}$</td>
<td>400</td>
</tr>
</tbody>
</table>

Table 1-1 Classification of CFRP according to modulus of elasticity (Zhao & Zhang 2007).
1.3 General application of FRP-strengthening of steelwork

In general, strengthening of an existing steel structure may be required for a number of reasons e.g. change of use and load requirements, geometric modifications or damage e.g. introduction of web penetrations, loss of section from corrosion etc (Cadei et al. 2004).

As previously mentioned FRP composite is the most favoured material in many strengthening applications because it is light-weight, has suitable strength and stiffness properties and is easy to install on site. Externally-bonded FRP-strengthening is a powerful technique of extending the life of structures including those made of steel. The overall cost of the whole strengthening job using FRP materials can be as competitive as using conventional materials, in addition to being quick and easy to handle on site with minimum interruption to the use of the facility. In some situations, FRP composites are the only plausible material that could be used for strengthening especially in the places where neither access of heavy machinery is possible nor closure of the facility is practical.

1.4 Enhancing stability of steel sections using FRP

FRP composite materials externally bonded to the surface of steel members have been shown to enhance strength and stiffness by generating composite action and creating an enlarged section. In situations where the stability of un-strengthened steel members have been compromised, FRP strengthening in local (flange or web) or global (sectional) buckling situations has proved to be effective. Stability enhancement through the application of FRP to steel members is primarily a research topic and there are few known applications in practice at this time. Ekiz et al. (2004) report that the presence of the CFRP wrap increased the size of the yielded plastic
hinge region, inhibited the occurrence of local buckling, and delayed the onset of lateral torsional buckling. Accord and Earls (2006) demonstrated that the presence of GFRP strips enhanced the structural ductility of the cross-section as a result of providing effective bracing of the flange outstands, and thus inhibiting the formation of the local buckles in the compression flange of the cross-section. Ekiz [(2007) demonstrated large improvements in the buckling and post-buckling response of full-scale double angle brace members subjected to reversed cyclic loading.

1.5 Force transfer mechanism in adhesive joints

There are several possible failure mechanisms in FRP strengthened steelwork; however the weakest link in the strengthening system is the adhesive bond between the steel and the FRP. De-bonding failure can happen in situations where both the FRP and steel are relatively lowly stressed in comparison to their respective capacities. Irrespective of the load type applied on a joint, the stresses in an adhesive joint can be decomposed into two main stress components: shear stress (τ) and normal stresses (σ). Normal and shear stresses are caused by forces acting perpendicular and parallel to the element under consideration in the adhesive layer, respectively. Normal stresses result in tension or compression in the material, generating normal strain while shear stresses cause diagonal deformation in the material known as shear strain. Stresses and strains are simply related to each other by the modulus of elasticity of the adhesive material.

Figure 1-3 presents a schematic of how the stresses in the joint are developed and interaction between them. The load transfers in adhesive from one adherent to another is achieved via shear action in the adhesive layer Figure 1-3a). These shear stresses vary along the laminate and decrease with increasing distance from the end.
of the FRP plate as shown in Figure 1-3b. Moreover, the shear action in the joint causes a secondary bending effect in the FRP as illustrated in Figure 1-3c. This bending moment in the laminate is counteracted by through-thickness force in the adhesive layer, referred to as peeling force or peeling stress which is distributed as shown in Figure 1-3d.

![Diagram showing shear and peeling stresses](image)

Figure 1-3 Formation of (a) shear and (c) peeling stresses in adhesive joint and corresponding (b) shear and (d) peeling stress distributions (Karbhari, 2014).

### 1.6 Failure modes of FRP-steel joint

In order to fully achieve the benefits of adhesive bonding, an understanding of the failure behaviour of adhesively bonded joints is needed. Due to the disparity between the strains of the steel substrate and the FRP reinforcement, high stress (both peeling and shear, see Figure 1-3) is concentrated in the adhesive joints. FRP-strengthened structures usually fail due to the failure of these joints. Although de-bonding is
usually the most prevalent, there are several typical failure modes which shown schematically in Figure 1-4 can be summarised as:

1. Interlaminar failure of CFRP-CFRP failure.
2. CFRP rupture.
3. CFRP-adhesive interface debonding (adhesion failure).
4. Adhesive failure (cohesion failure).
5. Steel-adhesive interface debonding (adhesion failure).

![Image of failure modes for steel-CFRP bond system](image)

Figure 1-4 Failure modes for steel-CFRP bond system (Zhao and Zhang, 2007).

1.7 Objectives of research

Currently no published research work exists on the application of FRP strengthening in beams with web openings. In order to investigate the feasibility of the carbon fibre reinforced polymer (CFRP) strengthening technique with a particular view to recovering the strength and stiffness of steel beams after the introduction of web openings, the following objectives are considered:
1. Studies of both un-strengthened and FRP-strengthened steel beams using FE modelling validated using experimental investigations in the literature to develop understanding of the behaviour of the structural system.

2. Further corroboration of the findings of the FE study by physically testing 3m strengthened steel beams with different web opening locations and CFRP configuration.

3. Identification of optimum CFRP configuration for a given arrangement of web openings.

4. Investigation of potential size effects and limits of applicability of the method.

1.8 Layout of the thesis

This thesis consists of eight chapters set out as follows:

Chapter one presents an introduction to the research and describes briefly the principle of FRP strengthening and general failure mechanisms of FRP strengthened steelwork. It also describes the aims and objectives of the research. Finally, it outlines the content of the thesis.

Chapter two presents a review of the literature relevant to steel beams with web openings and the strengthening of steel beams and composite beams in general using FRP techniques. Finally, an overview of the existing design guidelines that are applicable to the use of FRP with steel structures is given.

Chapter three describes the details of a validation study in which the FE analyses of relevant previous test specimens were carried out using ABAQUS software
release 6.13-1. It includes a comprehensive description of the finite element tools that have been used in the validation process such as element selection, material modelling and contact modelling, etc. Finally the results of the FE analyses are compared to relevant experimental tests.

**Chapter four** presents the details of an experimental investigation comprising the tests of four specimens, one un-strengthened control specimen and the three CFRP strengthened specimens with different locations of web opening. The test rig, testing procedure, the strengthening techniques employed and CFRP composites used for the strengthening of the specimens are described.

**Chapter five** presents the experimental results of the CFRP strengthened steel beams subjected to static load. In addition, the comparisons between the test and theoretical ultimate loads are presented in this chapter.

**Chapter six** presents the details and results of FE analyses carried out on models of the test specimens. The ultimate loads, modes of the failure, load-deflection responses and locations of the plastic hinges developed in specimens obtained from the FE analyses are compared to those in the tests.

**Chapter seven** presents a numerical parametric study on full-scale steel floor beams for three different spans, 7.5m, 10m and 15m, distributed between primary and secondary beams. Optimum CFRP configurations following introduction of web openings and potential limits of applicability are investigated.

**Chapter eight** summarises the main findings and conclusions of the research with suggestions for possible future work.
CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

The retrofitting of metallic structures has increased significantly since the late 20th century due to the pressing need to extend the life of existing steel building and bridge structures. This is attributed to a number of reasons such as modifications in their geometry. In most cases, maintaining this infrastructure through retrofit to extend their service life is more economical than replacing them. This chapter first presents an overview of metallic structural materials, followed by a brief summary of conventional methods currently used in retrofitting steel structures. A more detailed discussion on Fibre Reinforced Polymer (FRP) materials and their use in retrofitting metallic structures, along with recent research advances in this field, will follow.

2.2 Steel structures

Although structural steel has been mass produced in Britain since 1860, it took about 50 years to entirely replace wrought iron (Cadei et al., 2004). The mechanical properties of steel depend on the chemical composition, the heat treatment and the manufacturing process. Increasing the carbon percentage increases the strength of the steel but decreases its ductility. Modern steel is produced with a maximum carbon content of 0.25%, which provides the best combination of strength and
ductility. Developments in the manufacture and the control of the quality and strength of steel materials have motivated engineers to create extremely challenging and spectacular structures which are either difficult or costly to produce with other materials.

2.3 Unstrengthened steel beam with web opening

2.3.1 Failure modes

In the absence of overall instability, which is caused by lateral torsional buckling, steel beams with web openings have three basic modes of collapse, depending upon the geometry and the position of the web opening as follows:

2.3.1.1 Vierendeel or shear failure

This mode of failure is associated with high shear forces acting on the beam which form four plastic hinges at the opening edges, causing the perforated section to deform as a parallelogram (Kerdal & Nethercot, 1984). After that, the T-section above the web openings is deformed to a stretched shape, as shown in Figure 2-1.

Figure 2-1 Shear failure mode and corresponding Von Mises stress (MPa).
2.3.1.2  Flexural failure

Due to introducing an opening in the pure bending region (mid-span of a simply supported beam), the top T-section above the opening is vulnerable to buckling failure due to high compression stresses, as shown in Figure 2-2.

![Figure 2-2 Bending failure mode and corresponding Von Mises stress (MPa).](image)

2.3.1.3  Web-post buckling

The horizontal shear force in the web-post is associated with a double curvature bending over the height of the web-post; one inclined edge of the web opening will be stressed in tension, and the opposite edge in compression, and so buckling will cause the web-post to twist along its height, as shown in Figure 2-3.
2.3.2 Failure mode studies

Many studies have been conducted on the failure modes of steel beams with web penetrations without FRP; this research started in the middle of the last century. Redwood & McCutcheon (1968) presented three types of failure mechanisms for beams with rectangular openings in the web: single opening failure, interaction failure including two or more holes and failure through shear in the web post. The results reported the ultimate load necessary for each failure mechanism to occur, and assumed that the beam being analysed would fail through the mechanism which required the smallest ultimate load to form.

In the same context, the failure mechanisms were also studied by Hosain & Spiers (1971), but for simply supported castellated steel beams under various load systems, to investigate the yielding and rupture of the welded joints of these beams. Local buckling was prevented by providing full depth stiffeners and sufficient lateral bracings were also provided to prevent premature lateral buckling. Sudden weld rupture was the common mode of failure for all beams.
Hosain & Speirs (1975) later investigated the failure modes of 10 castellated beams with different beam length, web opening geometry and spacing between openings. All beams were simply supported while adequate lateral bracing and full depth stiffeners were provided. Five beams failed due to web buckling directly under the point of load application, three beams failed due to the formation of plastic hinges at the corners of the web openings where both shear and moment forces were acting, and the last two beams had flexural failure.

The buckling of the web-post between web openings of castellated beams was studied by Zaarour & Redwood (1996) for 12 beams with different depths, experimentally and numerically. Lateral support was provided in the form of a vertically aligned greased bearing plate to prevent lateral torsional buckling. FE analysis of the web post, taking into account inelastic action, was used to predict the web-post buckling loads. The results showed that the possibility of web-post buckling as a governing design criterion increased significantly in the deeper beam, as shown in Figure 2-4.

![Figure 2-4 Web-post buckling failure (Zaarour & Redwood 1996).](image)

In an experimental study, four perforated beams with identical cross sections and opening configurations were tested by Demirdjian (1999). All the beams were
simply supported and a central single concentrated load. The main focus of this experimental study was to investigate the buckling failure of the web-post between web openings. The results showed that all the beams failed due to web-post buckling. Test conditions were then simulated by elastic finite element analysis; the maximum test loads of all the beams exceeded the buckling loads obtained from finite element analysis with margins ranging from four to 14%.

Furthermore, a numerical study on steel beams with various shapes and sizes of web openings was conducted by Liu & Chung (2003) to investigate shear-moment interaction and failure mode. All simply supported beams were under uniformly distributed load with a span of 12m. The web openings were concentric to the mid-height of the sections, located at different positions along the beams’ length. The perforated beam at location 1 was under pure shear without any global moment, unlike position 4, while interaction between global shear force and global moment at positions 2 and 3 was significant, as shown in Figure 2-5. It was found that the yield patterns were common of failure modes for all steel and the most important parameter in assessing the structural behaviour of perforated beams was the length of the T-sections above and below the web opening, as it controls the magnitude of the local Vierendeel moments acting on the T-sections. Furthermore, the global shear-moment interaction curves were a similar shape, and thus it is possible to derive empirical shear-moment interaction curves to assess the load capacities of all steel beams with web openings of various shapes and sizes.
Finally, an experimental and analytical study on the failure mode of steel beams with web openings was presented by Tsavdaridis & D’Mello (2011). Twelve specimens with different web opening shapes were tested to investigate the failure mode and load strength of the web-post between two adjacent web openings. Test results were developed and analysed by the finite element method and the results were compared with the full-scale experiments. The effect of web opening spacing/web opening depth was studied to investigate the effective ‘strut’ action of the web-post buckling. The effect of the web opening depth/web thickness was also studied to investigate the stability (slenderness) of the web-post subjected to vertical shear load.

From these previous studies, although they used different parametric studies in terms of opening shape, opening size, single opening or multi-openings, and most of them involved the failure mode shapes, none of them studied the possibility of maintaining the beam strength and stiffness using FRP plates or section after inserting the web opening.
2.3.3 Approaches to reinforcing the web opening

2.3.3.1.1 Hicks & Lawson approach (SCI, 2011)

Using a steel stiffener is the only technique that has been applied to strengthening the web opening in steel beams. SCI publication P355 (Hicks & Lawson 2011) suggested a design guide using horizontal stiffeners formed from steel plates and welded to the top and bottom edges of the openings on one or both sides, as shown in Figure 2-6. This technique can help to transfer the forces around the openings and prevent local web buckling. In addition to the limitations in Figure 2-6, the following rules may be used in selecting the stiffener size:

\[
\frac{t_r}{t_w} \leq 1.2 \left( \frac{l_w}{2b_r} \right)
\]

Where:
- \(l_w\) is the anchorage length of the stiffener
- \(t_r\) is the thickness of the stiffener, \(8\text{mm} \leq t_r \leq 20\text{mm}\)
- \(t_w\) is the thickness of the web
- \(b_r\) is the width of the stiffener, \(b_r \leq 10t_r\epsilon\), \(\epsilon = \sqrt{\frac{235}{f_y}}\)

Figure 2-6 Detailing the limits of the reinforced web opening.
2.3.3.1.2 Darwin approach (AISC, 1990)

Instead of giving the stiffening geometry, the American Institute of Steel Construction adopts the calculation of the nominal capacity of a steel member with a web opening in pure bending, $M_n$ is expressed in terms of the capacity of the member without an opening, $M_p$.

For $t_w e \geq A_r$

$$M_n = M_p \left[ 1 - \frac{\Delta A_s \left( \frac{h_o}{4} + e - \frac{A_r}{2t_w} \right)}{Z} \right]$$  \hspace{1cm} 2-2

For $t_w e < A_r$

$$M_n = M_p \left[ 1 - \frac{t_w \left( \frac{h_o^2}{4} + h_o e - e^2 \right) - A_r h_o}{Z} \right]$$  \hspace{1cm} 2-3

In which:

$M_p = f_y Z$

$\Delta A_s = h_o t_w - 2A_r$

$h_o$: opening depth

$t_w$: web thickness

$e$: opening eccentricity, see Figure 2-7.

$Z$: plastic modulus of section without opening.

Figure 2-7 Reinforced opening configurations for steel beams.
The drawback of this approach is that there is not enough information about the stiffener geometry, such as the anchorage length of the stiffener and the stiffener position, which can make it more difficult to use it as a technical guide compared to the Hicks & Lawson Approach. Therefore, the Hicks & Lawson Approach will be used in this study in Chapter 7 to design steel stiffeners and compare it with the CFRP strengthening study approach, for the steel beams with web openings.

### 2.4 FRP-strengthening of metallic structures

The success of FRP composites in strengthening concrete structures encourages engineers to use them for the strengthening and repair of steel structures in place of conventional methods using steel strengthening. This allows welding requirements such as large scaffolding and heavy lifting to be avoided. Although the use of FRP composite materials for the strengthening of steel structures started in the late 20th century, it has increased rapidly since the start of the 21st century. A review of the use of FRP composites for strengthening steel structures is given below.

#### 2.4.1 FRP strengthening of existing steel bridges

Several metallic beams in bridges and other structures have been strengthened with CFRP composites worldwide to increase the live load and necessary expansions. In the UK, the Hythe Bridge over the River Thames was strengthened with normal modulus CFRP plates, 112GPa, to increase the load-carrying capacity in bending from 7.5 tonnes to 40 tonnes (Hollaway & Cadei, 2002). A London underground
steel bridge at Acton in west London was also strengthened with HM-CFRP plates, 310GPa, to reduce the live load stresses by 25% and improve its fatigue resistance (Moy & Bloodworth, 2007). The King Street Railway Bridge in Mold was strengthened with HM-CFRP strips, 360GPa, which helped strengthen six cast iron girders to allow 40-tonne vehicles to use the bridge (CNR-DT 202, 2007).

In Delaware, in the USA, two bridges, I-704 and I-95, were strengthened using normal modulus CFRP plates, 112GPa (Miller et al., 2001). After the retrofitting, the measured strain in the tension flange of the steel girder was reduced by 15% and the girder stiffness was increased by 12%. The Sauvie Island Bridge in Washington was strengthened using a normal modulus CFRP strip, 138GPa, bonded to an aluminium honeycomb core (Mosallam, 2007). The aluminium core was added to increase the distance of the CFRP strip from the neutral axis of the steel beam, thus increasing the stiffness of the member.

In Japan, the Takiguchi Bridge in Tokyo was strengthened using UHM-CFRP plates, 450GPa. The bridge girders were strengthened by bonding 4mm thick laminates along the bottom of the tension flange to create a maximum laminate thickness of 14 mm at mid-span (Peiris, 2011).

### 2.4.2 Previous studies on FRP strengthening of steel structures

Experimental and analytical work was conducted for six beams by Sen et al. (2001) to investigate the feasibility of using CFRP strips to repair steel-concrete composite bridge members. To simulate this effect, all the specimens were pre-loaded under a four-point load to give permanent deformations in each member before CFRP repairing. The average pre-loading was 142 kN and 187 kN for the specimens with the steel yield strengths of 310 MPa and 370 MPa respectively. The beams were
named S1 to S6, and were strengthened with two thicknesses of CFRP and with different bonding methods, as given in Table 2-1. The test specimens were obtained by cutting each of two steel-concrete composite bridge models into three parts, as shown in Figure 2-8(a). The resulting cross section is shown in Figure 2-8(b). The normal weight concrete used had an average compressive strength of approximately 50 MPa.

It can be noticed from the results that the bonding method has more effect than the CFRP thickness in repairing the damaged steel beam and the repairing process is more vital with a lower steel grade.
Table 2-1 Details of beams with comparison of ultimate loads (Sen et al. 2001).

<table>
<thead>
<tr>
<th>Beam No.</th>
<th>Yield strength of steel (MPa)</th>
<th>CFRP plate thickness (mm)</th>
<th>Bonding method</th>
<th>Strengthened beam ultimate load/control beam ultimate load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control 1</td>
<td>310</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>S1</td>
<td>310</td>
<td>5</td>
<td>Epoxy only</td>
<td>0.86</td>
</tr>
<tr>
<td>S2</td>
<td>310</td>
<td>5</td>
<td>Epoxy &amp; bolting</td>
<td>1.52</td>
</tr>
<tr>
<td>S3</td>
<td>310</td>
<td>2</td>
<td>Epoxy &amp; bolting</td>
<td>1.21</td>
</tr>
<tr>
<td>Control 2</td>
<td>370</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>S4</td>
<td>370</td>
<td>2</td>
<td>Epoxy only</td>
<td>1.09</td>
</tr>
<tr>
<td>S5</td>
<td>370</td>
<td>2</td>
<td>Epoxy only</td>
<td>1.09</td>
</tr>
<tr>
<td>S6</td>
<td>370</td>
<td>5</td>
<td>Epoxy &amp; bolting</td>
<td>1.32</td>
</tr>
</tbody>
</table>

Figure 2-8 (a) Cutting of bridge into three beam sections and (b) simply supported composite bridge section (Sen et al. 2001).

For the repairing of damaged steel-concrete composite girder beams, Al-Saidy et al.'s (2004) study also applied CFRP strips to the tension side. To simulate the field corrosion, damage to three of the four damaged beams was induced by removing 50% of the area of the cross section of the bottom flange and to the remaining one by removing 75% of the area of the cross section.
The experiment comprised six steel-concrete composite specimens in total: two undamaged control beams, two un-repaired damaged beams with two percentages of damage, 50% and 75%, and two CFRP-repaired damaged beams. The beams consisted of a 76 mm thick by 812 mm wide concrete slab connected to a 3.4 m long W8x15 structural steel beam by shear connectors, as shown in Figure 2-9. The normal weight concrete used had an average compressive strength of approximately 33 MPa, the A572 steel had a yield strength of 364 MPa and the pultruded CFRP strips had a modulus of elasticity of 200 GPa.

The repair of the three damaged beams was achieved by attaching 1.4 mm thick CFRP strips to the tension side of the steel sections. Two of the three repaired beams had 50% and 75% of the bottom flange area removed respectively. All the six beams were tested in a four-point static-loading system. The test results showed that the strength of the CFRP-damaged repaired beams was not only fully restored compared to that of the original undamaged beams, but it was also further increased by 4 to 20%. The CFRP-repaired beams failed due to slip at the steel-concrete

Figure 2-9 CFRP strengthening schemes.
surface followed by the crushing of the concrete slab. Neither adhesive breakdown nor CFRP rupture was observed at failure of the repaired beams.

Vatonec et al. (2002) investigated the behaviour of steel tubes strengthened with different configurations of pultruded CFRP strips. The configurations comprised bonding the CFRP strips on the top, the bottom and both the top and bottom walls of the steel tubes. Ten steel tubes, two un-strengthened and eight CFRP-strengthened, were tested. All specimens were 3.35 m long, TS 6x6x3/16 grade A500 steel tubes. The CFRP strips had modulus of elasticity and ultimate strengths of 165 GPa and 2800 MPa respectively. In tests of the first two specimens, the local buckling of the flanges limited the full potential for the CFRP mobilisation. Therefore, the middle-half length of the remaining eight specimens was filled with normal-weight concrete to eliminate the flange local buckling. All the beams were tested in a four-point loading system. Test results showed that the specimen with the single top CFRP strip exhibited the minimum increase of 6% in flexural strength, while that with the single top and double bottom strips exhibited the maximum increase of 26%, compared to that of the un-strengthened specimens. Furthermore, the CFRP strips debonded in tests of all the strengthened tubes and the top strips debonded prior to the bottom strips in all the cases.

Colombi & Poggi (2006) conducted an experimental and numerical programme to characterise the static behaviour of steel I-beams strengthened by pultruded CFRP strips. The test group comprised four beams, one un-strengthened and three CFRP-strengthened. The control specimen, TR0, was a 2500 mm long HEA 140 steel beam. Both the TR1 and TR2 beams had one layer of CFRP strips bonded with the bottom flange, 60 mm wide and 1.4 mm thick, using the epoxies Sikadur30 and SikadurR330 respectively, as shown in Figure 2-10. The third strengthened beam,
TR3, had the bottom flange bonded with two layers of the CFRP strips using epoxy resin Sikadur R30, as shown in Figure 2-11. The two epoxies differed in mechanical properties. The elastic modulus and tensile strength for Sikadur R30 were 4500 and 24.8 MPa and those for Sikadur R330 were 3800 and 30 MPa respectively.

Figure 2-10 Specimen details for beams TR1 and TR2 (Colombi, P. & Poggi, 2006).

Figure 2-11 Specimen details for beam TR3 (Colombi, P. & Poggi 2006).

Three-point bending tests were performed using the test frame shown in Figure 2-12. The specimens TR0, TR2 and TR3 were provided with lateral supports to prevent lateral torsional buckling, which occurred when testing specimen TR1, which had no lateral support.
The test results showed that the ultimate loads of the CFRP-strengthened beams, TR1, TR2 and TR3, were increased by 14%, 31% and 40% respectively, compared to that of the control specimen, TR0. Due to the provision of lateral support and use of a different epoxy adhesive, specimen TR2 had achieved double increase ratio in the ultimate load than TR1.

Another application using FRP to recover the strength and stiffness of damaged steel beams was conducted by Photiou et al. (2006). A combination of CFRP and GFRP laminates was used in two geometric shapes to repair the artificially damaged, rectangular hollow section (RHS) steel beams. Half of the thickness of the tension (bottom) flange was removed to simulate the real damage. The FRP repaired beams had been divided into two repaired sets, two beams for each, and the control set had an undamaged-unrepaired beam and damaged-unrepaired beam. All of them were tested in a four-point loading system and no control beam was tested. The first set was repaired by bonding U-shaped FRP laminate unit layers to the tension flange, while the second set was repaired by bonding a flat plate FRP
lamine unit. In each set, two layers – each of an ultra-high modulus CFRP, UHM-CFRP, and a high modulus CFRP, HM-CFRP – were used respectively with a combination of the layers of a low modulus GFRP laminate, as shown in Figure 2-13. The laminates were bonded together using Sikadur 31 epoxy adhesive. The ultimate loads of the undamaged and damaged steel beams were determined using the elasto-plastic analyses of the beams.

Test results showed that, due to removing half the thickness of the bottom flange, the ultimate load of the damaged beam was reduced by approximately 18% compared to that of the undamaged beam. The ultimate load of the damaged beam was not only fully restored to that of the undamaged beam using the CFRP-repair, either U-shaped or flat plate laminates, but it was also further increased by up to 30%. It also showed that the ultimate loads of the repaired beams using the HM-CFRP laminates were approximately 11% higher than those using the UHM-CFRP laminates because the latter had a lower ultimate strain of 0.4%; therefore, the failure of the HM-CFRP-repaired beams was ductile, while that of the UHM-CFRP-repaired beams was brittle.
Figure 2-13 Schematic diagrams of RHS steel beams repaired using (a) U-shaped CFRP/GFRP and (b) flat plate pre-peg laminates (Photiou et al. 2006).

The effect of CFRP-strengthening on the web-buckling capacity of light steel beams was investigated by Zao & Al-Mahaidi (2008). The test comprised 28 specimens comprising seven control beams and 21 CFRP-strengthened beams that were tested under compression load. The seven control specimens, LSB1 to LSB7, had web slenderness ratios ranging from 62.5 to 125. Three strengthening techniques were used for each type of specimen. The unidirectional fibre orientation CFRP plates were used with appropriate adhesive bonded to their webs on the outer sides, the inner sides or both sides, as shown in Figure 2-14. The direction of the CFRP fibre was kept perpendicular to the longitudinal axis of the beam.
Figure 2-14 CFRP-strengthening of LSBs (a) outer side, (b) inner side and (c) both sides (Zhao & Al-Mahaidi 2008).

The test results showed a significant ultimate load increase in each technique compared to the un-strengthened beams where the ultimate loads of the beams were increased by 40% to 200%, using the CFRP strengthening on the outer sides of the web; using the CFRP strips on the inner sides of the web increased the ultimate loads by 140% to 300%, and using the CFRP strips on both sides of the web showed the highest increases of 250% to 500% in the ultimate loads of the strengthened beams. With regard to failure mode, the debonding of CFRP strips was a dominant failure mode for all the three types of strengthening, as shown in Figure 2-15.

These results help to understand that the use CFRP in both sides to strengthen the web is more important than using it in only on one side and the bond breakdown is a failure mode in web strengthening.
Figure 2-15 Modes of failure of LSBs (a) un-strengthened beam, (b) CFRP on outer side, (c) CFRP on inner side and (d) CFRP on both sides (Zhao & Al-Mahaidi 2008).

FRP-strengthening also means additional out-of-plane stiffness is provided to the steel-plate girders that are prone to web buckling. Two specimens were tested by Okeil et al. (2009), an un-strengthened control beam, OB1, and a GFRP-strengthened beam, OB2. The steel-plate girder beam was 2083 mm long, 532 mm deep with a 3.2 mm thick web, and 279 mm wide and 13 mm thick flanges, as shown in Figure 2-16. It was divided into four panels by five vertical steel stiffeners, 9.5 mm thick and 114 mm wide, on both sides at an equal spacing of 521 mm. The investigated mode of failure of the plate girder was web buckling; therefore, all other possible modes in the plate girder causing failure were designed to be avoided, such as flange buckling and stiffener buckling.

The two vertical GFRP-pultruded T-sections, GFRP stiffener in Figure 2-16, were applied to the specimen OB2 at one end of the plate girder as the additional stiffeners on both sides of the web. A single-point load was applied at the first internal stiffener on one end of the plate girder.
Test results showed that the ultimate load, 389 kN, of the GFRP-strengthened specimen, OB2, was 40% greater than that, 278 kN, of the control specimen, OB1. If loads at the initiation of web buckling are considered, there was an improvement of 56% because the buckling initiated at 389 kN (87.5 kip) with failure in the GFRP-strengthened specimen and at 249 kN in the un-strengthened specimen. Failure in the GFRP-strengthened specimen was initiated by a breakdown of the steel-GFRP bond followed by immediate buckling of the web, as shown in Figure 2-17.
The suitability of using FRP composites to resist global buckling in steel T-sections was investigated by Harries et al. (2009). Twenty columns were tested, four un-strengthened columns and 16 FRP-strengthened columns. The test specimens were T steel sections, 155 x 10.5, grade 345MPa steel, as shown in Figure 2-18(a). CFRP strips 1.4mm thick and GFRP strips 1.4mm thick were placed on top of each other and bonded adhesively to both sides of the web in one layer and two layers respectively, with details as shown in Figure 2-18. The tensile strength and modulus of elasticity of the CFRP strips were 2790 MPa and 155 GPa respectively, while those for the GFRP strips were 895 MPa and 41.4 GPa respectively.
In the first set of tests for the elastic buckling, the columns were 1664 mm long. Five steel sections, one un-strengthened and four FRP-strengthened, one of each case, were tested. The strong and weak-axis bifurcation loads were assessed as the loads at which an abrupt change in lateral displacement occurred about these axes. Results showed that the strong and weak-axis bifurcation loads of the un-strengthened specimen were increased by up to 60% and 13% respectively by the FRP-strengthening. However, a very small increase, up to 9%, in the axial capacity in the FRP-strengthened sections was observed as compared to that of the un-strengthened section. In the second set of tests for the inelastic buckling, the length of the specimen was reduced to 356 mm to avoid the flange local and torsional buckling. Fifteen specimens, three un-strengthened and 12 FRP-strengthened, three of each case, were tested. FRP application led to improvements of nine to 17% in web local buckling (WLB) bifurcation loads, compared to those of the control specimens. An increase of four to 14% was observed in the axial capacity due to FRP-strengthening. The specimens with two 25.4 mm wide FRP strips performed
better than those with one 50.8 mm strip. Debonding of FRP strips occurred in all tests, usually at about 75% of the peak loads, see Figure 2-19.

![Figure 2-19 Debonding of CFRP strips in specimen CFRP-2 (Harries et al., 2009).](image)

In the same context, the bonded pultruded CFRP strip was used at the beam web for shear strengthening of steel I-beams, see Figure 2-20. Narmashiri et al. (2010) had previously implemented FE analyses and experimental testing to investigate this. Five specimens, as shown in Table 2-2, one control (NB1) and four CFRP-strengthened (NB2 to NB5), were tested in a four-point loading system. The shear zone was the web region of surrounded by two partial height stiffeners and two flanges near the supports and was 200 mm wide and 130 mm deep, as shown in Figure 2-20. All specimens were a 1.3 m long beam, 150 mm deep with a 6.6 mm thick web and 100 mm wide and 10 mm thick flanges.
Table 2-2 Specimens used in Narmashiri et al.’s (2011) test.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Number of CFRP Strips</th>
<th>Strengthening Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>NB1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>NB2</td>
<td>3</td>
<td>At each shear zone at each end of the beam on both sides of the web</td>
</tr>
<tr>
<td>NB3</td>
<td>2</td>
<td>At each shear zone at each end of the beam on both sides of the web</td>
</tr>
<tr>
<td>NB4</td>
<td>3</td>
<td>At one shear zone at each end of the beam on the one side of the web</td>
</tr>
<tr>
<td>NB5</td>
<td>2</td>
<td>At one shear zone at each end of the beam on the one side of the web</td>
</tr>
</tbody>
</table>

Figure 2-20 CFRP-strengthening and loading of specimen NB2 (Narmashiri et al, 2010).
The test results showed that the increase in the ultimate load of the strengthened specimens NB2 and NB3, using three and two CFRP strips respectively on both sides of the web, was the same, approximately 52%, compared to that of the control specimen (NB1) while, for the strengthened specimens NB4 and NB5 using three and two strips respectively on one side of the web only, the increases in the ultimate loads were 43% and 35% respectively, compared to that of the control specimen (NB1). They also showed that the control specimen failed due to flange twisting and web crippling, as shown in Figure 2-21(a). With regard to failure mode, the results showed that either longitudinal delamination of the CFRP strips in the area near the applied loads or debonding of the CFRP strips in the strengthened specimens was initiated in the strengthened specimens, as shown in Figure 2-21(b). Finally, the CFRP strengthening decreased the shear buckling of the web.

![Figure 2-21 Mode failure: (a) control specimen (b) strengthened specimen (Narmashiri et al, 2010).](image)

This study provides a good indication about using the CFRP strips in strengthening the web of an I-section beam against shear buckling, and the significant achievement is that attaching CFRP strips at both sides of the web gave a significant increase in beam strength compared with only strengthening one side.
Different FRP composites, adhesives and surface treatment methods were used by Islam & Young (2013) to investigate their effect on the behaviour of stainless steel tubular sections. A total of 57 specimens were tested under end-two-flange (ETF) and interior-two-flange (ITF) loading conditions, as shown in Figure 2-22. Nine of them were un-strengthened controls and 48 were FRP-strengthened. Specimens were either rectangular hollow sections (RHS) or square hollow sections (SHS).

The tests results showed that the web buckling capacity was increased by up to 50% due to FRP strengthening. Furthermore, the FRP debonding was the dominant failure mode and the carbon laminate ‘Sika Carbodur H514 carbon’ and the adhesive ‘Araldite 2015’ showed a better performance than the others by giving the higher ultimate loads. Finally, slight differences were observed in the ultimate loads between using a sander treatment and a grinder treatment.

Figure 2-22 Test set-up of ETF and ITF loading conditions (Islam & Young 2013).
In conclusion, it has been observed that the majority of the published work has focused on the use of GFRP and CFRP composites in the tension regions of the steel beams in order to increase the beams’ flexural strength. Although there has been some use of the GFRP pultruded sections as intermediate and diagonal stiffeners to strengthen the steel beams at their webs, where the failure is initiated by out-of-plane buckling, their use as stiffeners at web openings instead of steel stiffeners has not been investigated.

2.5 Guidelines for strengthening steel structures

It is well known that design guidelines and standards for the use of FRP strengthening with steel structures are not as advanced as those for concrete constructions, although a few guidelines for the use of FRP in strengthening metallic structures have already been published throughout Europe as well as in the USA.

The first design guidance was published in the UK by the Institution of Civil Engineers (Moy, 2001). Elastic range and an elastic-plastic analysis were adopted to evaluate the strength of a metallic structure strengthened with FRP materials. Furthermore, it suggested partial factors for material to consider the effects of short-term and long-term behaviour.

Another design guideline was also published in the UK, by CIRIA C595 (Cadei et al., 2004) for strengthening metallic structures using FRP materials. In this guidance, elasto-plastic analysis was also used beyond the elastic range for steel beams and steel-concrete composite girders. It also reports that the debonding failure of the strengthening system can be prevented if the maximum principle
stress in the adhesive joint does not exceed the strength of the adhesive obtained from lap-shear tests. In addition, the material partial factor was considered to include the environment and time effect.

In 2006, Schnerch et al. (2006) proposed design guidelines in the USA for strengthening steel concrete composite beams with high modulus CFRP composites. The guidelines stated that the allowable increase in live load for a strengthened girder should be selected to satisfy serviceability, safety and redundancy conditions.

Recently, the Italian National Research Council published state-of-the-art guidelines on the design, installation and monitoring of an externally bonded FRP system for strengthening existing metallic structures (CNR-DT202, 2007). The guidelines recommend the use of material partial factors to ensure a structure’s safety. The stresses analyses are similar to those proposed by the CIRIA C595 (Cadei et al., 2004) guidance.

No guidance was given for the FRP-strengthening of the web opening in the existing steel members where the failure is related to the location of the opening.
2.6 Summary

The review has revealed for FRP strengthened steel beams the following:

1. Vierendeel, bending and web-post buckling were the main modes of failure of steel beams with web openings.
2. The rectangular web openings have more detrimental effect on beam strength than circular openings.
3. Little attention has been given to apply the FRP composites on steel beam webs as a stiffener against the out-of-plane buckling. However, the use of FRP composites as stiffeners at web openings instead of steel stiffeners has not been investigated.
4. Since the failure mode for FRP-strengthened steel structures is generally initiated by a breakdown of the adhesive bond, the adhesive properties have a great influence on the joint durability.
5. The surface preparation is one of the most important factors to be considered in order to achieve a strong adhesive bond.
6. The adhesive thickness has only a small influence on the strength of the adhesive joint.
7. In order to further strengthen the bond between the FRP composites and the steel surface, after bonding the adherents should be held together until the curing of the adhesive is complete.

Most of the previous studies focused at the effect of FRP composites configurations and material properties on the flexural and shear strength of the steel beams. Therefore, there were no numerical and experimental studies conducted to reinforce the web opening of the steel beam. This Thesis will adopt this experimentally and numerically in the next chapters.
CHAPTER 3

DESCRIPTION OF NUMERICAL MODEL AND VALIDATION

3.1 Introduction

The chapter starts by giving an overview of the adopted numerical modelling procedure in ABAQUS, covering the main parameters such as solver type and element description. This is followed by comparative studies between the ABAQUS simulation and three different test results for steel beams with and without strengthening. The experimental tests used for the validation study are namely those of: (Tsavdaridis & D’Mello 2011), (Linghoff et al. 2009) and (Deng & Lee ,2007).

3.2 Solver type

Standard analysis in ABAQUS consists of two main analysis approaches: Implicit and Explicit. Implicit uses the Newton-Raphson iteration method to solve the equation $\Delta u = K^{-1}\Delta F$ with relatively large time steps, while this equation is solved directly with very small time steps by the Explicit solver.

One advantage of the explicit procedure over the implicit procedure is the greater ease with which it resolves complicated FRP debonding problems (Chen et al., 2015). In addition, as models become very large, the explicit procedure requires fewer system resources than the implicit procedure.
Applying the explicit dynamic procedure to quasi-static problems requires some special considerations to achieve a solution in a short amount of time and without dynamic effect.

Use the frequency analysis to find the member natural frequency corresponding to the expected failure mode.

Use the smooth load amplitude called SMOOTH STEP using *AMPLITUDE, DEFINITION=SMOOTH STEP, ABAQUS/Explicit, which can automatically connect each of the data pairs with curves whose first and second derivatives are smooth and whose slopes are zero at each of the data points.

Use the suitable time in this amplitude, which can be obtained from:

\[ t = \frac{1}{f} \]

Where \( f \) is the member natural frequency of the desired failure mode extracted from the frequency analysis.

Therefore, in this research, the explicit solver was employed to conduct 3-D nonlinear FE simulations. Both material and geometric nonlinearities were considered. In order to capture the large deformation and local instability effects in the 3-D FE models, the geometric nonlinearity was taken into consideration by activating the optimal parameter “NLGEOM” in ABAQUS.

### 3.3 Material modelling

#### 3.3.1 Steel

Either shell or solid elements may be used for modelling steel beams. Use of shell elements is preferred here owing to their superior computational efficiency. S4 and S4R are the two quadrilateral shell elements in ABAQUS with full and reduced integration respectively: S4 having four integration points and S4R having one.
Since the S4 element has more integration points that take longer to run the simulation, it was decided to use the S4R element, which has four nodes and six degrees of freedom (three translations and three rotations) in each node.

3.3.2 Cohesive

To model the steel/adhesive/CFRP interface, there are two approaches in ABAQUS to model the adhesive behaviour in composite layers, cohesive elements and cohesive surfaces.

3.3.2.1 Cohesive surfaces approach

This approach is called the surface-based cohesive approach; it is represented by the surface interaction properties that are assigned to a contact pair using the finite-sliding, node-to-surface formulation. A cohesive surface is more desirable to begin the analysis with the surfaces just touching each other. Consequently, cohesive surfaces are never affected by interface thickness (ABAQUS, 2013). Although the surface-based cohesive approach is widely used, its utilisation is limited to cases in which the adhesive thickness is negligibly small. Moreover, it is not supported in the ABAQUS/Explicit solver.

3.3.2.2 Cohesive elements approach

ABAQUS offers an alternative approach to define adhesive joints with ABAQUS/Implicit and ABAQUS/Explicit solver. The main aspect of this approach is that constitutive thickness has a noticeable effect on the interface behaviour because the nodal coordinates of the cohesive elements are calculated based on the initial thickness. Thus, the adhesive material properties such as stiffness and strength are available in modelling the cohesive. This is implemented by using the tie constraint of the cohesive element, COH3D8, having eight nodes, three degrees
of freedom in each node, surfaces to steel substrate and CFRP plate. The debonding growth occurs along the layer of cohesive elements and without deformation into the adjacent parts. Thus, the cohesive element approach can predict the bond behaviour from the initial loading to the initiation of damage and then the damage propagation.

3.3.2.3 Damage modelling of adhesive

The failure response of the bond can be modelled in both of the aforementioned approaches using traction-separation models.

A basic bilinear traction-separation law frequently used in calculations can be seen in Figure 3-1, where the softening after damage initiation is linear. Another frequently used model is the one seen in Figure 3-2, where the softening after damage initiation is described by an exponential function. In both, the damage model consists of two stages: a damage initiation and a damage evolution. The initial response of the cohesive surfaces is linear: after a damage criterion is met, the traction-separation response can be defined to a user-defined damage evolution law.

Figure 3-1 Illustration of a simple bilinear traction-separation law (ABAQUS, 2013).
3.3.2.4 Damage initiation

ABAQUS provides four damage initiation criteria, two of them based on the stress and the other two based on the separation; examples are the maximum nominal stress criterion, the maximum nominal strain criterion, and the quadratic nominal stress or strain criterion (see Figure 3-3).

In this study, the quadratic nominal stress criterion is used as the damage initiation criterion, which can be represented as:
\[
\left( \frac{\langle t_n \rangle}{\sigma_{\text{max}}} \right)^2 + \left( \frac{t_s}{\tau_{\text{max}}} \right)^2 + \left( \frac{t_t}{\tau_{\text{max}}} \right)^2 = 1
\]

where,

\(t_n, t_s\) and \(t_t\) are peak values of the nominal stress.

\(\sigma_{\text{max}}\) is the tensile strength of the adhesive, and

\(\tau_{\text{max}}\) is the shear strength of the adhesive; the symbol < > represents the Macaulay bracket which is used to signify that compressive stresses do not initiate damage (i.e. \(t_n\) is negative and thus <\(t_n\)> is equal to zero).

### 3.3.2.5 Damage evolution

After damage initiation, a scalar damage variable \(D\) is introduced as the overall damage in the material. The range of \(D\) is from 0 to 1, with 0 representing the undamaged case and 1, the total separation. Subsequently, the corresponding stress components are then degraded as follows:

\[
I_n = \begin{cases} 
(1 - D) \bar{t}_n, & \bar{t}_n \geq 0 \\
\bar{t}_n, & \bar{t}_n < 0
\end{cases}
\]

\[
t_t = (1 - D) \bar{t}_t
\]

\[
t_s = (1 - D) \bar{t}_s
\]

where, \(\bar{t}_n, \bar{t}_t\) and \(\bar{t}_s\) are the stress components predicted by multiplying the initial stiffness and the current relative displacements.

For the linear softening law, the damage index \(D\) can be expressed as:

\[
D = \frac{\delta^f_m (\delta^\text{max}_m - \delta^0_m)}{\delta^\text{max}_m (\delta^f_m - \delta^0_m)}
\]
where $\delta_{m}^{\text{max}}$ is the maximum effective relative displacement attained during the loading history, and $\delta_{m}^{0}$ and $\delta_{m}^{f}$ are the effective relative displacement at the initiation and end of failure respectively. The effective relative displacement $\delta_{m}^{f}$ can be written as

$$\delta_{m}^{f} = \sqrt{\langle \delta_{e}^{f} \rangle^2 + \delta_{e}^{c} + \delta_{t}^{f}}$$

3.3.3 CFRP modelling

FRP composites are materials that consist of two constituents. The constituents are combined at a macroscopic level and are not soluble in each other. One constituent is the reinforcement fibre, which is embedded in the second constituent, a continuous polymer called the matrix (Cadei et al., 2004). The reinforcing fibres, which are typically stiffer and stronger than the matrix, take up to 70% of the compound volume. The unidirectional lamina has three mutually orthogonal planes of material properties, where the x direction is in the same direction as the fibres, and the y and z directions are perpendicular to the x direction. Therefore, it is considered as an orthotropic material. This orthotropic material is also transversely isotropic, where the properties of the FRP composites are nearly the same in any direction perpendicular to the fibres. Thus, the properties in the y direction are the same as those in the z direction. Thus, this material is so-called a specially orthotropic material (Teng et al., 2015). An S4R shell element was also used to simulate the CFRP in the numerical analysis.

The rupture of the FRP reinforcement was defined by specifying a stress-based failure criterion: Tsai-Hill failure theory (Kollár & Springer, 2003). The input data required for this failure envelop are tensile and compressive stress limits, $X_t$ and $X_c$. 

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in the $x$ direction; tensile and compressive stress limits, $Y_t$ and $Y_c$, in the $y$ direction; and shear strength (maximum shear stress), $S$, in the $X-Y$ plane. The Tsai-Hill failure criterion requires that:

$$I_F = \frac{\sigma_{xx}^2}{X^2} - \frac{\sigma_{xx}\sigma_{yy}}{X^2Y^2} + \frac{\sigma_{yy}^2}{Y^2} + \frac{\sigma_{xy}^2}{S^2} < 1$$  \hspace{1cm} (3.7)$$

If $\sigma_{xx} > 0$, then $X = X_t$; otherwise, $X = X_c$. If $\sigma_{yy} > 0$, then $Y = Y_t$; otherwise, $Y = Y_c$.

### 3.4 Tests conducted by Tsavdaridis and D’Mello (2011)

#### 3.4.1 Test description

A cellular steel beam was tested by Tsavdaridis and D’Mello (2011) to investigate the failure mode and load strength of the web-post between two adjacent web openings. The simply supported steel beam has a length of 1.9m (with 1.7m clear span between the supports) and a cross section of type UKB 457x152x52; the dimensions and the support conditions of the steel beam are shown in Figure 3-4. The steel material properties are shown in Table 3-1. Thick steel plate stiffeners were welded to each beam at the mid-span and supports to avoid premature flange local buckling.
Figure 3-4 Details of the tested beam.

Table 3-1 Steel test results.

<table>
<thead>
<tr>
<th></th>
<th>Modulus of Elasticity (GPa)</th>
<th>Yield Strength (MPa)</th>
<th>Tensile Strength (MPa)</th>
<th>Strain</th>
<th>Poisson Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Yielding %</td>
<td>Ultimate %</td>
</tr>
<tr>
<td>Web</td>
<td>200</td>
<td>375.3</td>
<td>492.7</td>
<td>0.187</td>
<td>25</td>
</tr>
<tr>
<td>Flange / Stiffener</td>
<td>200</td>
<td>359.7</td>
<td>480.9</td>
<td>0.18</td>
<td>24</td>
</tr>
</tbody>
</table>

3.4.2 FE modelling

In this simulation, the general-purpose shell element S4R with reduced integration was adopted for both the steel section and the CFRP plate. Both material nonlinearity and geometric nonlinearity were considered in the FE models. The three full-depth stiffeners were provided on the two sides of the web at the applied...
load and supports. Each stiffener was tied (tie option in FE) to the top flange, the bottom flange and the web of the cross section. The load and the support conditions were applied at the same positions in the test as shown in Figure 3-5. The material properties of steel were the same in the test, Table 3-1, with bi-linear isotropic model. The linear buckling analysis was implemented before the nonlinear analysis to select the web-post failure mode as happened in the test. The initial out-of-plane geometric of web was assumed the imperfection of 1mm. In the experimental study of Tsavdaridis and D’Mello (2011), no measured initial imperfection was reported. Redwood and Demirdjian (1998) recorded a measured maximum imperfection of the web of about 1 mm. The validated mesh size was 15mm with uniform distribution as shown in Figure 3-5.

Figure 3-5 Load, support and mesh conditions of FE model.
3.4.3 FE results and discussion

As the extracted results from experiment were the deflections of the beam at two locations and the failure mode, they were compared with FE results. It can be noticed from Figure 3-6a that the ultimate load obtained from FE simulations was slightly higher than in the test by around 7% for both dial gauges, 1 and 2, and the stiffness obtained from the FE simulation was much greater than that from the test. This level of accuracy between test and FE results was achieved by Narmashiri et al. (2012).

This can be attributed to the residual stresses in the beam left by the manufacturing process and this effect of these stresses is not considered in the FE model.

Regarding to the failure shape, the simulated perforated steel beam was failed by the web-post buckling due to high shear stresses. This S-shape buckling deformation predicted in the FE simulation agreed with the test results as shown in Figure 3-6b.
(a) Load versus deflection

(b) Failure modes

Figure 3-6 FE results compared with Experiment results.
3.5 Tests conducted by Linghoff et al. (2009)

3.5.1 Test description

The selected steel beam tested by Linghoff et al. (2009) was constructed of a standard HEA 180 profile steel beam with 200GPa as the modulus of elasticity and was subjected to four-point bending. The net span of the beam was 1800 mm and simply supported boundary conditions were provided. To prevent local buckling of the web, full-depth 4 mm thick stiffeners were provided on each side of the web at all support and loading points. To prevent lateral buckling of the beam, lateral supports were provided at the mid-span. Two strips of 1.4 mm thick CFRP laminate, 200GPa of elasticity modulus and 3300MPa of tensile strength, were bonded to the lower flange using epoxy, 7GPa of elasticity modulus and 25MPa of tensile strength, in the arrangement shown in Figure 3-7.

Figure 3-7 Details of the specimen tested by Linghoff et al. (2009).
3.5.2 FE modelling

Both material nonlinearity and geometric nonlinearity were considered in the FE models. The load was applied on the top flange of the steel section at the two loading points. The four full-depth stiffeners were provided on the two sides of the web at the applied load and supports. Each stiffener was tied (tie option in FE) to the top flange, the bottom flange and the web of the cross section. The load and the support conditions were applied at the same positions in the test as shown in Figure 3-8. Since the beam was also laterally restrained along the span to prevent out of plain torsional buckling, there was no need to implement the buckling analysis to get the proper failure. In the absence of an experimental stress-strain curve, a bi-linear (i.e. elastic-hardening) stress-strain model as given in Figure 3-9 was adopted. The general-purpose shell element S4R with reduced integration was adopted for both the steel section and the CFRP plate, while the adhesive layer was modelled using the cohesive element COH3D8. The top and bottom surfaces of the adhesive layer were tied to the bottom surface of the steel beam and the top surface of the CFRP plate respectively. Due to the limited information of the CFRP and adhesive, both of them are assumed to be linear elastic until failure. The CFRP plate was modelled as an orthotropic (Teng et al., 2015) elastic material, see Table 3-2, while the adhesive was modelled as an isotropic elastic material, see Table 3-3.

<table>
<thead>
<tr>
<th>Material property</th>
<th>CFRP plates of FE model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal modulus of elasticity, GPa</td>
<td>200</td>
</tr>
<tr>
<td>Transverse in-plane modulus of elasticity, GPa</td>
<td>20</td>
</tr>
<tr>
<td>In-plane shear modulus, GPa</td>
<td>3.7</td>
</tr>
<tr>
<td>Out-of-plane shear modulus, GPa</td>
<td>2.6</td>
</tr>
<tr>
<td>Major in-plane Passion’s ratio</td>
<td>0.3</td>
</tr>
<tr>
<td>Out-of-plane Passion’s ratio</td>
<td>0.2</td>
</tr>
</tbody>
</table>
Table 3-3 Material properties the adhesive material of FE model.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elasticity modulus, GPa</td>
<td>8</td>
</tr>
<tr>
<td>Shear modulus, GPa</td>
<td>2.6</td>
</tr>
<tr>
<td>Tensile strength, MPa</td>
<td>29</td>
</tr>
<tr>
<td>Shear strength, MPa</td>
<td>26</td>
</tr>
</tbody>
</table>

Figure 3-8 Load, support and mesh conditions of FE model.
3.5.3 FE results and discussion

The mesh size of the previous validation, 15mm, with different mesh sizes have been investigated for unstrengthened beam to find the most efficient mesh. Despite the refining mesh is needed in high stress gradient such as the corners and sharp places, the uniform distribution can also help in saving the computing time; therefore, it is preferable numerically to start with the uniform and global mesh first and then check the need to the localised mesh.

It can be seen, see Figure 3-10, that all mesh configurations are in convergence with the test result with no need to localise the meshes. However, the intermediate sizes, 15 mm and 25 mm, presents more levels of acceptance, see Figure 3-11. In addition, this convergence was also noticed for strengthened beam, as shown in Figure 3-12. As commented in the previous modelling, the slight differences in results of the test and FE model can be attributed to the residual stresses in the beam left by the manufacturing process and this effect of these stresses is not considered in the FE model.
Figure 3-10 Mesh size convergence.

Figure 3-11 Load–strain curve of control steel beam.
Nevertheless, strain readings were also obtained for several points along the laminate, 180mm from the mid-span, and compared with those of the tested beams for different magnitudes of applied load (Figure 3-13). These were less than 5% as a difference in the strain along the distance.
Moreover, the adhesive damage has been checked in terms of damage initiation (QUADSCRT) and damage evolution scalar (SDEG) to verify the ability of the finite element to capture the damage mechanism, see Figure 3-14. It can be noticed that the maximum value of the aforementioned parameters at the beam mid-span at
the adhesive edges due to a plastic hinge forming in the steel beam. Therefore, these parameters have been investigated along the adhesive edge.

(a) Damage initiation (QUADSCT) of adhesive at failure load.

(b) Damage evolution scalar (SDEG) of adhesive at failure load.

Figure 3-14 the adhesive damage.
Finally, the top flange yielding failure, see Figure 3-15, has been reported as a failure mode in the test work, which is confirmed in FE prediction with no CFRP debonding. This provides that the use of enough CFRP length can prevent debonding and gives a significant strength increase.

![Figure 3-15 Von Mises stress (MPa) and corresponding deflected shape at failure.](image)

3.6 Tests conducted by Deng and Lee (2007)

3.6.1 Test description

The simply supported steel beam that was tested by Deng and Lee (2007) has a length of 1200mm (with 1100mm clear span between the supports) and a cross section of type 127x76x13UKB; the dimensions and the boundary conditions of the steel beam are shown in Figure 3-16. Grade 275 steel was used with a corresponding tensile elastic modulus of 205 GPa. The CFRP plates had a thickness of 3 mm, a width of 76 mm, 500mm length and an elastic modulus in the fibre direction of 212 GPa. The epoxy resin used to bond the CFRP to the steel has a
tensile strength of 29.7MPa, tensile elastic modulus of 8GPa and shear modulus of 2.6GPa. Two 4mm thick steel plate stiffeners were welded to each beam at the mid-span, one on each side of the web, to avoid premature flange local buckling.

![Diagram](image)

Figure 3-16 Details of the tested specimen and mesh distribution.

### 3.6.2 FE modelling

As the experimental stress-strain curve of the steel was not given, a bi-linear (i.e. elastic-hardening) stress-strain model as given in Figure 3-17 was adopted. The general-purpose shell element S4R with reduced integration was adopted for both the steel section and the CFRP plate, while the adhesive layer was modelled using the cohesive element COH3D8. The load and the support conditions were applied at
the same positions in the test as shown in Figure 3-18. Since the beam was also laterally restrained along the span to prevent out of plain torsional buckling, there was no need to implement the buckling analysis to get the proper failure.

The two full-depth stiffeners were provided on the two sides of the web in the mid-span region, and three sides of each stiffener were tied (using tie option in ABAQUS) to the top flange, the bottom flange and the web of the cross section respectively. Similarly, the top and bottom surfaces of the adhesive layer were tied to the bottom surface of the steel beam and the top surface of the CFRP plate respectively. Due to the limited information of the CFRP and adhesive, both of them are assumed to be linear elastic until failure. The CFRP plate was modelled as an orthotropic elastic material, see Table 3-4, while the adhesive was modelled as an isotropic elastic material, see Table 3-5. The validated mesh size was 15mm with a quadric distribution as shown in Figure 3-18.

![Figure 3-17 FE Model of steel material properties.](image-url)
Figure 3-18 Load, support and mesh conditions of FE model.

Table 3-4 Material properties CFRP plates of FE model.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal modulus of elasticity, GPa</td>
<td>212</td>
</tr>
<tr>
<td>Transverse in-plane modulus of elasticity, GPa</td>
<td>21</td>
</tr>
<tr>
<td>In-plane shear modulus, GPa</td>
<td>4</td>
</tr>
<tr>
<td>Out-of-plane shear modulus, GPa</td>
<td>2.8</td>
</tr>
<tr>
<td>Major in-plane Passion’s ratio</td>
<td>0.3</td>
</tr>
<tr>
<td>Out-of-plane Passion’s ratio</td>
<td>0.2</td>
</tr>
</tbody>
</table>

Table 3-5 Material properties the adhesive material of FE model.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elasticity modulus, GPa</td>
<td>8</td>
</tr>
<tr>
<td>Shear modulus, GPa</td>
<td>2.6</td>
</tr>
<tr>
<td>Tensile strength, MPa</td>
<td>29</td>
</tr>
<tr>
<td>Sear strength, MPa</td>
<td>26</td>
</tr>
</tbody>
</table>
3.6.3 Results and discussion

The predictions are compared with the experimental load-displacement curve of the control beam (un-strengthened beam) in Figure 3-19. The FE result is seen to agree with the test result, with small differences between the two curves. In both the experimental and numerical models, the beam failed by compression flange buckling.

![Figure 3-19 Load displacement curve of control steel beam.](image)

In the case of the strengthened beam, failure in the experiment was initiated by end debonding of the CFRP plate at 149kN. The same failure mode was predicted by the FE model, which failed at 138kN. The load-deflection curve obtained from the FE model is compared with the experimental curve in Figure 3-20. The FE model is close to the experimental curve except it has a lower value at ultimate load by 7%, which represents the debonding load. This value of accuracy between the experiment and FE results was achieved by Narmashiri et al. (2012).
As explained before, these differences between test and FE results may attribute to the residual stress in test’s beam which is not included in FE model.

Figure 3-20 Load displacement curve of strengthened steel beam.

Moreover, the strain distribution was measured experimentally along the CFRP and is compared with FE results, as shown in Figure 3-21 to Figure 3-24. It is believed that these results reflect an acceptable agreement. It can be seen clearly that the CFRP strain decreases from the centre to the ends where the load is applied.
Figure 3-21 Strain readings on CFRP plate at strain gauge G3 in experiment and FE models.

Figure 3-22 Strain readings on CFRP plate at strain gauge G5 in experiment and FE model.
Finally, the failure mode was CFRP debonding followed by top flange yielding, which was also reported in the test work as shown in Figure 3-25.
Using the model approach described in the preceding sections, an investigation of CFRP strengthening configurations will now be conducted for the case of a beam with web openings.

3.7 Investigation of CFRP strengthening for steel beams with web openings

In this section, the ability of strengthening steel beams is considered after introducing web openings using CFRP plates in different arrangement configurations. This study helps to give a clear idea about the experiment which was subsequently conducted.

3.7.1 Model description

A simply supported 457*152.4*13 UKB with 3m clear length was used with rectangular web openings in different locations to examine the possible failure modes (see 3.7.3) and suggest the most effective methods of strengthening. The beam section size was chosen as being typical of floor beam sections used in practice for multi-storey frame buildings. The beam is prevented from lateral...
translation in the $z$-direction and is uniformly loaded in $y$-direction, as shown in Figure 3-26.

Figure 3-26 Model description and boundary condition.

3.7.2 FE modelling

A bi-linear stress-strain model as was adopted with nominal yield strength of 275MPa with an elastic modulus of 200GPa as shown in Figure 3-27. The CFRP plates were 25mm wide and CFRP equal angles were 25*25mm with material properties as shown in Table 3-4. The elastic modulus in the fibre direction was 212GPa with 3mm thickness in all cases. Coupled cohesive behaviour of the adhesive layer with tensile strength of 29.7MPa, tensile elastic modulus of 8GPa and shear modulus of 2.6GPa was adopted, see Table 3-5.

The general-purpose shell element S4R with reduced integration was adopted for both the steel section and the CFRP plate, while the adhesive layer was modelled using the cohesive element COH3D8. The load was applied uniformly on top flange.
while the support conditions, vertical restraint, were applied at the beam ends using MPC constraint option in ABAQUS that shares the entire cross section to prevent the stress localisation, see Figure 3-28. Since the beam was also laterally restrained along the span to prevent out of plain torsional buckling, there was no need to implement the buckling analysis to get the proper failure except it was needed in web-post buckling failure.

The top and bottom surfaces of the adhesive layer were tied to the steel beam’s surface and the CFRP surface respectively. The CFRP plate was modelled as an orthotropic elastic material, see Table 3-4, while the adhesive was modelled as an isotropic elastic material, see Table 3-5. As previously, the mesh was based on a 15mm element grid as shown in Figure 3-28.

![Figure 3-27 FE Model of steel material properties.](image)
Figure 3-28 Load, support and mesh conditions of FE models.
3.7.3 Failure modes

3.7.3.1 Vierendeel failure

To simulate this type of failure as shown in Figure 3-29, an opening in the high shear region is introduced. Transfer of the shear force across the opening causes secondary moments in the T-section above the opening. Six different configurations, A to F, using plates, pultrusion or a combination of both have been modelled to find the most efficient strengthening system.

Table 3-6 and Figure 3-30 show the CFRP plate and angle configurations that have been used for strengthening, each with the same thickness.

Figure 3-29 Shear failure mode and corresponding Von Mises stress (MPa) for beam without strengthening.

Figure 3-30 CFRP locations at opening of the beam.
Table 3-6 CFRP configuration of strengthening.

<table>
<thead>
<tr>
<th>Specimen Description</th>
<th>CFRP Length</th>
<th>Symbol</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP plate strengthening above and below the opening</td>
<td>4×opening length</td>
<td>A</td>
<td></td>
</tr>
<tr>
<td>CFRP plate strengthening above and below the opening, and beneath the top and bottom flange.</td>
<td>4×opening length</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>CFRP plate strengthening above and below the opening, and beneath the top and bottom flange.</td>
<td>3×opening length</td>
<td>C</td>
<td></td>
</tr>
<tr>
<td>CFRP plate strengthening above and below the opening, and beneath the top and bottom flange.</td>
<td>2×opening length</td>
<td>D</td>
<td></td>
</tr>
<tr>
<td>CFRP angle strengthening above and below the opening.</td>
<td>4×opening length</td>
<td>E</td>
<td></td>
</tr>
<tr>
<td>CFRP angle strengthening above and below the opening, and CFRP plate strengthening beneath the top and bottom flange.</td>
<td>4×opening length</td>
<td>F</td>
<td></td>
</tr>
</tbody>
</table>
The use of these configurations gives different percentages of the strength and stiffness recovery of the beam after introduction of the opening, depending on the strengthening location and strengthening stiffness, as shown in Figure 3-31. Although the specimens B, C and F have different strengthening lengths, they all obtained a significant strength and stiffness recovery. This was achieved before the first onset of de-bonding, which occurred at a load of 865kN for the horizontal strengthening at top and bottom flanges and this also coincided with top flange yielding. Specimen F, which incorporates angle strengthening, gives the same strength recovery that has been achieved in specimens B and C with plate strengthening; in all three cases, failure was due to debonding at the same load level. However, this strength and stiffness recovery was not observed in the other beams, which failed in a brittle manner due to the absence of CFRP in the top and bottom flanges.

Figure 3-31 Load deflection curve for the steel beam with different configurations of strengthening.
3.7.3.2 Flexural failure

The buckling T-section above the web opening is the common failure of this opening location type, as shown in Figure 3-32.

![Figure 3-32 Bending failure mode and corresponding Von Mises stress (MPa) for beam without strengthening.](image)

Figure 3-32 Bending failure mode and corresponding Von Mises stress (MPa) for beam without strengthening.

Figure 3-33 illustrates the load displacement curves of the same configurations shown in Table 3-6 that have been utilised to strengthen the steel beam with web opening at mid-span. It can be seen that the configurations B and F have demonstrated significant strength increases of 20% and 17% respectively, more than the solid beam strength but with different behaviour for both: ductile for B and brittle for F; this is due to the different CFRP elements in the latter debonding simultaneously. Configuration C, which has the same layout as B with a lesser bonded length, showed a good strength recovery with a slight increase until debonding occurred at the top flange strengthening; this layout showed a reasonable level of ductility. The configurations A and C exhibited full strength recovery but with earlier debonding of the strengthening system.
3.7.3.3 Web-post buckling failure

In web-post buckling, three plastic hinges are generated at the middle and ends of the top T-section in addition to the plastic hinge in the middle of the web-post, as shown in Figure 3-34; therefore, these zones may benefit from an increase in effective section by addition of CFRP sections and profiles with different configurations, as mentioned in Table 3-6. With a view to significantly increasing the effective section properties profiled, pultruded angles (25*25mm) were added to the configuration; this is subsequently referred to as B+web angle.
Figure 3-34 Web post buckling failure mode and corresponding Von Mises stress (MPa) for beam without strengthening.

Figure 3-35 illustrates the load-deflection behaviour of the beam in each case; note that for all configurations the same effective CFRP thickness of 3mm was adopted. It can be noticed that there are different percentages of strength and stiffness recovery of the beam depending on the strengthening location and strengthening stiffness. Configurations B, C, E and F have provided a significant stiffness recovery for the beam with the opening compared with the solid beam. The significant strength recovery has been recorded with configurations B, B+web angle and F with nearly the same value of 15%, while the A configuration has given just 5%. Despite the fact that the addition of the pultrusion angle in beam B+web angle provided a considerable strength recovery, it could not significantly reduce the stiffness loss, especially when the web post angle started to debond. Neither the C configuration, which has a CFRP bonded length of three times the opening length, nor the configuration D, which has a CFRP bonded length of two times the opening length, showed strength recovery, but there was a slight stiffness increase with the C configuration. It is worth mentioning that non-ductile failure is dominant in this type of opening configuration due to the sudden buckling of the web post.
Figure 3-35 Load deflection curve of beam with two central openings and different configurations of strengthening.
3.8 Assumptions and limitations

The numerical results obtained in this chapter are based on the following assumptions:

1. A uniform mesh throughout has been used for all FE models. This has been adopted for expediency of the analysis and has been shown to be adequate in terms of accuracy for the beams examined. However, a uniform mesh may not be able to capture accurate stress distributions in areas of high stress gradient and concentration such as the ends of FRP plate and the corner of openings, in these cases a localized, refined mesh may be more appropriate.

2. An idealised bi-linear (i.e. elastic-hardening) stress-strain relationship was adopted to model the steel.

3. The general-purpose shell element S4R with reduced integration was adopted for both the steel section and the CFRP plate.

4. Initial imperfections of steel beams are considered in the modelling of perforated beam which was conducted Tsavdaridis and D'Mello(2011), 1mm imperfection was used in the modelling of web-post buckling.

5. The explicit solver was employed to conduct 3-D nonlinear FE simulations with smooth step to achieve quasi-static analysis. Failure criteria for each material were also adopted in the analysis.

6. Both material and geometric nonlinearities were considered.

7. The material properties of steel, CFRP plate and adhesive are assumed to be as per the respective manufacturers’ declared properties.
3.9 Summary

This chapter has presented the results of a numerical model validation and sensitivity study. Three different experimental tests reported in the literature were simulated, one beam with web openings and two beams with CFRP strengthening. Following this, the ability of CFRP strengthening of steel beams after introducing web openings has been investigated numerically using the validated model. The major achievements from this chapter can be summarised as:

1. Overall, the simulation results were in acceptable agreement with the published experimental results in terms of the load-displacement relationships, strain distribution and failure modes including the crucial phenomenon of debonding.

2. Based on these validation study results, the same modelling techniques will be applied for all future numerical models in this thesis.

3. A series of CFRP strengthening configurations have been investigated in order to identify the most effective in terms of overall performance enhancement, structural efficiency and practicality. Three opening scenarios were considered i.e. central web opening, opening near the beam end and then double opening at the centre. Generally the configuration noted as type B was found to be the optimum in most cases.

4. For the case of the beam with the double central opening where web post buckling failure dominated, a variant of type B strengthening which included a CFRP angle section along the height of the web post was found to be the most effective strengthening solution. This however may not be the most practical configuration due to the relative scarcity of CFRP angle sections compared to CFRP plate.
Since the numerical study has suggested the type B configuration is most suitable in general, a series of beams will be tested experimentally using this configuration in the next chapter. The three different web positions previously outlined will be examined in order to gain greater insight into the strengthened beams’ performance.
CHAPTER 4

EXPERIMENTAL STUDY

4.1 Introduction

The Finite Element investigation in Chapter 3 showed different strengthening configurations for a steel beam with opening in different locations. One of these configurations helped to recover the beam’s strength and stiffness in significant manner. In the current study, this configuration of strengthening was used in the experimental investigation. This experimental work comprises the tests of four specimens, one control (without openings and un-strengthened) and the other three with different web opening positions and the configuration of CFRP strengthening which approved in Chapter 3. This chapter presents the structural tests, including details of test specimens, experimental parameters, preparation methods, instrumentation, experimental setup, and testing procedure.

4.2 Experimental setup

4.2.1 Specimen description

The experimental series consisted of 4 no. 305×102×25 UKBs with 3m clear span; one without an opening acting as the control (B0), and three with rectangular web openings in different locations (B1-RO, B2-RO and B3-RO). These specimens were strengthened by CFRP plates as shown in Figure 4-1.
Figure 4-1 Layout of strengthened specimens and boundary conditions.
4.2.2 Loading scheme

The typical uniform load delivered by a floor slab in practice was represented using a rigid steel frame comprising three UC beams, one UC 203×203×60 as primary beam and two UC 152×152×30 as secondary beams, joined together by hinges, see Figure 4-2. All the specimens were tested under 6-point bending to approximate a uniformly distributed load.

![Load frame assembly](image)

Figure 4-2 Load frame assembly.

4.2.3 Boundary conditions

The ends of the specimens were placed on two supports. Both supports restrained the specimen vertically and were free to rotate and move horizontally. The steel roller, 50mm, were used at each specimen end to provide this horizontal translations. In addition, 130mm wide bearing plates at the supports were added to prevent premature web bearing and buckling failure, in accordance with Eurocode 3 (BSI, 2005), with details and dimensions shown in Figure 4-3.

Triangulated steel frames were fabricated from I-sections (178×102×28 UKB) to form lateral restraints, these were distributed along the beam length at 900mm c/c as shown
In Figure 4-4. In order to allow the specimen to deflect vertically and minimise friction effects between the lateral support and the specimen, rigid steel spacers in combination with 10mm thick PTFE plates were adopted.

Figure 4-3 Lateral support description and end support condition.
Figure 4-4 Test rig arrangement.
4.2.4 Specimen preparation

Each specimen had a 3000mm clear span. Either one, or two, 210 x 185mm web openings were formed in different places over the specimen length using an electric cutter. Prior to cutting, 5mm holes were formed at the opening corners to avoid notch formation.

In order to achieve the best bond between steel and CFRP, mechanical grinding was employed to remove the weak oxide layer, see Figure 4-5. Then, the ground surface was cleaned with acetone to remove any grease or oil before applying the epoxy and CFRP layer.

![Flange preparation](image1)

(a)Flange preparation

![Flange preparation](image2)

(a)Flange preparation

Figure 4-5 Specimens preparation.
### 4.2.5 Specimen material properties

To determine the mechanical strength of the steel I-sections used for these tests, tensile coupon tests were carried out to the specifications and guidelines in BS EN 10002-1 (EN, 2001). Three coupons were taken from two un-yielded locations (one flange and one web) of the four steel beams tested. The average yield and the average ultimate stresses for flange and web were determined by averaging the results obtained from all the tensile coupon tests as shown Table 4-1. The average stress-strain curve for each series is shown in Figure 4-6. Unidirectional pultruded carbon-fiber-reinforced polymer (CFRP) plates, 3mm thick, were provided by Weber (Weber Building Solution, 2017). The CFRP modulus of elasticity was 200 GPa, and the tensile strength was 2970MPa, based on the manufacturer's details. The CFRP plates were manufactured using a pultrusion process and had a fibre volumetric content of 70% in an epoxy resin matrix. Two components of epoxy were used to bond the CFRP plates to the specimens. This epoxy (Araldite 420) is manufactured by Araldite Co (Huntsman 2009) with properties as shown in Table 4-2.

<table>
<thead>
<tr>
<th></th>
<th>Young’s Modulus (GPa)</th>
<th>Yield stress (MPa)</th>
<th>Ultimate tensile stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flange</td>
<td>206</td>
<td>412</td>
<td>566</td>
</tr>
<tr>
<td>Web</td>
<td>210</td>
<td>435</td>
<td>569</td>
</tr>
</tbody>
</table>

Table 4-1 Summary of the mechanical properties of steel (mean values shown).
Figure 4-6 Mean tensile stress-strain relationship from coupon tests.

Table 4-2 Summary of the mechanical properties of Araldite 420 epoxy adhesive when cured at room temperature for 24 hours (Huntsman 2009).

<table>
<thead>
<tr>
<th>Young’s Modulus (GPa)</th>
<th>Ultimate Tensile strength (MPa)</th>
<th>Ultimate Shear strength (MPa)</th>
<th>Strain at rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>29</td>
<td>26</td>
<td>4.6</td>
</tr>
</tbody>
</table>
4.2.6 Installation of CFRP plates

The CFRP plates were first cut to the desired length and their edges were ground to obtain a flat, smooth surface. Then the resin/hardener components (A/B) of the epoxy were mixed together with ratio of 1:2.5 by weight (Huntsman 2009). 1mm glass beads were used in the mix at a 1% dosage to achieve a constant thickness for the epoxy layer.

After cleaning the CFRP plates from dust, the adhesive was applied in a triangular form to allow even dispersal and then the plate was placed immediately on the desire place. Next, when the CFRP was placed on the steel beam, the adhesive was squeezed out from the centre to the sides, thereby reducing the possibility of air suction into the adhesive layer. The composite was left to cure before bonding the other face of the specimen.

In order to prevent any sliding or shifting in the CFRP plate positions, a 10mm steel plate was placed over the CFRP with a PTFE separation layer to avoid unintentional bonding as shown in Figure 4-7.
(a) Fixing of CFRP plates showing temporary weight during curing

(b) Upturned specimens during curing of adhesive

Figure 4-7 CFRP plate application process.
4.2.7 Strain and deflection measurement

Strain gauges were mounted on every CFRP strip parallel to the fibre direction, and on each specimen at 20mm from the plate end in view of the findings of Deng et al. (2004) and Linghoff et al. (2009) in which they recorded maximum shear stresses in this location, details are shown in Figure 4-8. Linear Variable Displacement Transducer (LVDT) was positioned underneath the specimens at mid-span to measure the vertical displacement during the test. Moreover, rosette strain gauges were mounted on the steel web near the opening corner to measure the steel strain in three directions X, Y and XY. All beams were tested under displacement control with a rate of 2mm/min using a 500kN capacity INSTRON Universal Testing Machine. Loads and dial gauge readings were recorded at each of these increments.

Figure 4-8 Strain gauge layouts, specimens B1-RO to B3-RO.
4.3 Summary

This chapter has presented the details of the experimental work comprising the testing of four specimens, one control (without opening and un-strengthened) and the rest three with different web opening positions and a CFRP strengthening configuration corresponding to type B (as presented in Chapter 3, Table 3-6).

A purpose built test-rig which provided lateral restraint to the top flange of the test beams at discrete points has been described. For the test series, steel coupon tests from the parent steel were conducted to derive the stress-strain behaviour including yield and ultimate strength. Mechanical properties of the CFRP and adhesive used based on the respective manufacturers’ published values have been described. The preparation and fixing process for the CFRP strengthening was also explained. Finally, a detailed description of the experimental setup and instrumentation including strain gauge locations, LVDT locations, etc., was given.

The next chapter presents the results obtained from the test series along with detailed discussion on each specimen.
CHAPTER 5

EXPERIMENTAL RESULTS

5.1 Introduction

This chapter deals with the experimental data acquired from the steel specimens tests discussed in Chapter 4. The various categories of data are presented and typical types of behaviour are identified instead of considering each individual test on its own. Any deviations from the general behaviour are explained. Initially, the modes of failure and load capacity are discussed. Then, the CFRP strengthening and steel strain, deflection, torsional buckling and steel yielding are examined.

5.2 Results and discussion

The following section describes the results from the test series. For economy, only the strengthened beams with openings were tested experimentally. In order to compare the performance, the un-strengthened versions of B1-RO, B2-RO & B3-RO (denoted B1-UO, B2-UO & B3-UO respectively) were modelled using non-linear finite element analysis via ABAQUS. The model was validated in previous studies by the authors, Altaee et al. (2016). The experimental behaviour of the control beam B0 provided further opportunity to validate the model for the unstrengthened beams.
5.2.1 Unstrengthened beams B0, B1-UO, B2-UO & B3-UO

The numerical load displacement relationship for B1-UO to B3-UO together with the experimental model and numerical result for the control beam B0 is presented in Figure 5-1.

For benchmarking purposes, the Eurocode 3 (BSI, 2005) derived design strength of B0 has been included in Figure 5-1. According to Eurocode 3 (BSI, 2005), for a yield strength of 412 N/mm², and effective length of 900mm, the moment capacity of B0 should be around 130kNm (fully restrained i.e. zero effective length moment capacity = 140 kNm), which corresponds to an ultimate total load of around 322kN for the 6-point bending configuration examined. At the early stages of the experiment on B0, a linear elastic response was observed up to around 300 kN. Beyond this point, the onset of localised yield in the top flange at mid-span was observed. Concurrent with this, visible rotation of the top flange at mid-span was noted. On further loading, lateral torsional buckling in the constant moment zone between the central lateral restraints was observed as the beam eventually reached a peak load of 406kN. At this point a full plastic hinge was developed at the mid-span top flange (see Figure 5-2a). The experimental unloading branch of the beam was captured following displacement control procedure until the rotation of the top flange reached a point at which the test was stopped. The numerical model shows acceptable agreement with the experimental results (Figure 5-1), with occurrence of yield in the top flange and onset of lateral torsional buckling between the mid-span restraints at around 350kN.

As would be presumed, the FE results show that beams B1-UO, B2-UO and B3-UO all failed at lower load levels than B0, around 15% on average and in the case of B2-UO, a noticeably less stiff response was observed. With its central opening, B1-UO represents a 15% reduction in the plastic modulus (and associated reduction in local moment capacity) compared to B0. In the case of beam B1-UO, after linear load
behaviour up to around 350kN, significant torsional rotation started in the top T-section above the central opening, this also corresponded with the development of a plastic hinge in the top T-section at mid-span. After peaking at around 357kN, unloading of the beam was accompanied by further lateral buckling of the section between the central restraints, see Figure 5-3. For beam B2-UO, yielding occurred in both corners around the opening at a load of around 330kN, consistent with the Vierendeel action. In beam B3-UO, yielding was observed in the top flange around both openings after yielding in the central portion, web-post buckling was initiated at a load of around 350kN.

5.2.2 Strengthened beams B1-RO, B2-RO & B3-RO

Figure 5-4 compares the experimental load-displacement response of strengthened beams B1-RO, B2-RO and B3-RO with the unstrengthened control beam B0. In all strengthened cases a stiffer response is obtained accompanied by an increase in peak load. In each case, the degree of lateral movement of the top flange at ultimate load was more pronounced than for B0, which meant that the experiments were stopped at an earlier stage in the unloading branch. A summary of the results is given in Table 5-1.

Specimen B1-RO

Due to presence of CFRP plates, the stiffness of B1-RO was significantly increased in comparison with un-strengthened specimen B0. B1-RO exhibited a fairly linear trend up to a load of approximately 435 kN. Beyond this load level, there was evidence of the onset of plastic hinge formation in the top flange near the end of the CFRP plate. This is corroborated by the strain gauge output from FS1 (Figure 5-5a), which suggests onset of
steel yielding (2000 microstrain), assuming strain compatibility between the CFRP and the steel. Lateral torsional buckling of the section between the central lateral restraints was observed to start at around 470 kN. At this point, debonding of the CFRP commenced in the top flange at the extreme edge away from the opening. Strain gauges FS2, 3 and 4 all recorded comparatively low strains indicating the CFRP strengthening in these locations was not working hard. The test was continued until a load level of 488kN, beyond this point the test was stopped due to excessive lateral deflection. The final deflected shape of the beam, including the plastic hinge location is shown in Figure 5-2b. The strain rosette readings from SS1 indicate that yielding did not take place in the web, Figure 5-6a.

**Specimen B2-RO**

In comparison to B1-RO, a less-stiff response was obtained. Lateral torsional buckling commenced at around 405kN in the zone between the central lateral restraints. Onset of yielding was recorded at the opening corners before reaching the ultimate load, see FS4 and SS1-XY in Figure 5-5b and Figure 5-6b respectively. As in the case of B1-RO, the test was stopped due excessive lateral displacement of the top flange and an accompanying drop in load at around 424 kN. The final deformed shape of the beam is shown in Figure 5-2c. The failure mode of B2-RO was different than to those for the numerical model of the corresponding unstrengthened beam B2-UO. In B2-RO plastic hinge formation at the mid-span top flange and lateral torsional buckling controlled, while Vierendeel action and associated yielding around the corner opening dominated the behaviour of B2-UO. This change in failure pattern demonstrates the influence of the CFRP strengthening on the stress state of the member. Observation of Figure 5-1 shows that B2-UO achieved an ultimate load 334 kN (compared to 405.8 kN for B0).
Use of the CFRP strengthening arrangement allowed B2-RO to recover and surpass the full strength of B0 resulting in a 5% strength enhancement. Although FS4 indicated the strain in the CFRP at this location was relatively high, no debonding occurred in the test.

**Specimen B3-RO**

B3- RO exhibited the same failure mode as B1- RO where lateral torsional buckling of the section between the lateral restraints occurred at around 420kN. At this load level, plastic hinge formation was also observed in the top flange near the corner of the opening close to the mid-span, Figure 5-2d. Yielding was recorded at the opening corners before reaching the ultimate load, see FS1 in Figure 5-5c. In similarity to B1-RO, end debonding of the top flange CFRP commenced at ultimate load, 442kN. The final deflected shape of the beam, including the plastic hinge location is shown in Figure 5-2d. The strain rosette readings from SS1 indicate that yielding did not take place in the web, Figure 5-6c. In contrast to the corresponding un-strengthened beam B3- UO, web post buckling was not observed in the experiment. These suggests the CFRP strengthening on the flanges and at the central web post were sufficient to delay onset of this mode, allowing a more global failure mode to develop. Strain gauge FS5 which was mounted at the centre of the web-post strengthening returned modest strains during the test indicating a low utilisation ratio for the CFRP at this location, Figure 5-5c. Observation of Figure 5-1 shows that B3- UO achieved an ultimate load of 334 kN (compared to 405.8 kN for B0). Use of the CFRP strengthening arrangement allowed B3- RO to recover and surpass the full strength of B0 resulting in a 10% strength enhancement.
Figure 5-1 Load versus vertical displacement at mid-span: specimens B0 & B1-RO to B3-RO.
Figure 5-2 Failure patterns for Specimens B0 & B1-RO to B3-RO.
Figure 5-3 Failure modes and corresponding Von Mises stress (MPa) at ultimate load.
Figure 5-4  Load versus vertical displacement at mid-span for specimens B0 & B1-RO to B3-RO.

Table 5-1  Comparison of specimen ultimate load and failure mode.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Ultimate load (kN)</th>
<th>Strength difference with B0 (%)</th>
<th>Failure mode* In sequence</th>
</tr>
</thead>
<tbody>
<tr>
<td>B0</td>
<td>406</td>
<td>-</td>
<td>LTB+TFY</td>
</tr>
<tr>
<td>B1-UO</td>
<td>357</td>
<td>-11.8</td>
<td>LTB+TFY</td>
</tr>
<tr>
<td>B2-UO</td>
<td>334</td>
<td>-17.5</td>
<td>YOC+VF</td>
</tr>
<tr>
<td>B3-UO</td>
<td>351</td>
<td>-13.3</td>
<td>WPB+TFY</td>
</tr>
<tr>
<td>B1-RO</td>
<td>488</td>
<td>20</td>
<td>LTB+ TFY +EDC</td>
</tr>
<tr>
<td>B2-RO</td>
<td>424</td>
<td>5</td>
<td>LTB+TFY</td>
</tr>
<tr>
<td>B3-RO</td>
<td>442</td>
<td>10</td>
<td>LTB+ TFY +EDC</td>
</tr>
</tbody>
</table>

Figure 5-5 CFRP strain gauge output.
Figure 5-6 Steel strain gauge output.
5.3 Assumptions and limitations

1. The experimental tests were subjected to static load via six equally spaced point loads.

2. The experiments were conducted on 305×102×25 UKB with 3m clear span. Different failure mechanisms might govern the behaviour of longer and deeper beams.

3. All beams were prevented to move laterally using lateral supports distributed along the beam length at 900mm c/c. The lateral restraints were in contact with the top flange of the beams only.

4. It is assumed that a simply supported boundary condition was created in the test set-up. In terms of vertical movement, the bearing points of the beams were assumed to be perfectly rigid, in reality, some minor vertical displacement may occur.

5. The CFRP was oriented in one dimension, 3mm thick. The CFRP length was four times the opening length. In reality the CFRP is made to a certain degree of tolerance as specified by the manufacturer.

6. A uniform thickness of adhesive was maintained by use of glass sphere in the matrix. This provided a practical level of uniformity, in reality the 100% uniform thickness is impossible to achieve in practice.
5.4 Summary

This chapter has shown the results of the experimental part of this thesis. The main findings of this experimental work can be drawn as:

1. This method is likely to be a useful approach in practice to recover the stiffness and strength of beams where in-service web openings are introduced.

2. The beam’s strength was reduced after introducing of the web openings by 11.8%, 17.5 and 13.3 for B1-UO, B2-UO and B3-UO respectively.

3. This method not only helped to recover the beam's strength but also it gave an increase with 20%, 5% and 10% for beams B1-RO, B2-RO and B3-RO respectively compared with control beam, B0.

4. In all cases, the strengthened beams showed stiffer response and greater load capacity than the un-strengthened beams with web openings.

5. The experiments reveal that this approach of strengthening changed the failure location and mechanism of the strengthened beams which were not always the same as in the un-strengthened cases, or the control case. Therefore, care must be taken to identify and check potential failure other than those that might be expected with strengthening.

6. No debonding was observed in the tests except end debonding which occurred after steel yielding. Therefore, full composite action is achieved by using this approach with Araldite 420 as the adhesive layer and implementing adequate prior surface preparation.

Knowing the material properties of steel, CFRP and adhesive can encourage finding more information that could not acquire from tests using FE modelling. Nevertheless, more parametric can be investigated for CFRP strengthening of steel beams with web openings such as the optimum CFRP thickness. This is what will be conducted in the next chapter.
CHAPTER 6

NUMERICAL MODELLING OF EXPERIMENTAL WORK

6.1 Introduction

In order to provide further insight into the behaviour of the specimens described in chapters 4 and 5, a detailed finite element (FE) study is presented here. The models are tested using ABAQUS 6.13-1, adopting the approaches described in Chapter 3. The objectives of carrying out the FE analyses of the test specimens are:

- Due to the limitation in the experimental programme, some parameters that were not included in the experimental testing will be examined in this chapter to provide greater insight.
- Once the composite behaviour is fully understood, it can start to be used in further parametric studies.
6.2 Loading and boundary conditions

All four models were analysed under the same loading and boundary conditions which were used in the experiments, as shown in Figure 6-1.

The vertical load was applied directly to the beam at the same test places after using the rigid plates, and the models were supported at both ends after placing the bearing plates with the same details as the test bearing plates. Hard interaction was implemented as contact behaviour between these plates and the specimen with different friction coefficients. Due to the smooth surface and lubricating effect which was used in the experiment for the support plates to allow smooth longitudinal movement, 0.16 was used as friction coefficient between the end support’s plates and the specimen while 0.8 (Bowden & Tabor, 2001) was used between the load plates and the specimens that had normal steel roughness.
Figure 6-1 Load, support and mesh conditions of FE models.
6.3 Material modelling

6.3.1 Steel properties

As described in Chapter 3, the steel was modelled as an isotropic material, and true stress-strain relationships were used where the following equations were used to convert nominal stress and nominal strain to true stress and true strain:

\[ \varepsilon_{True} = \ln(\varepsilon_{nominal} + 1) \]  \hspace{1cm} 6-1

\[ \sigma_{True} = \sigma_{nominal}(\varepsilon_{nominal} + 1) \]  \hspace{1cm} 6-2

A four-node shell element S4R, as approved in FE verification in Chapter 3, was selected in modelling the steel member.

6.3.2 CFRP Material Properties

The linear behaviour was used in modelling the CFRP plates in the numerical simulation, as explained previously in detail in Chapter 3. Similar to the steel beam, the CFRP was modelled with a four-node shell element, S4R. The material properties of CFRP were provided by the supplier (Weber Building Solution, 2014), modulus of elasticity, tensile strength and Poisson ratio, for one direction. Due to the need for properties in all directions to feed the CFRP damage’s criteria, explained in Chapter 3, the material properties of other directions have been taken from a previous study (Abdullah, 2010) which used the same material with same thickness and from same supplier source. A summary of the material properties used for the modelling is given in Table 6-1.
Table 6-1 Material properties for FRP composites (Abdullah, 2010).

<table>
<thead>
<tr>
<th>Elastic Modulus (MPa)</th>
<th>Poisson Ratio</th>
<th>Shear Modulus (MPa)</th>
<th>Tensile/shear strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_x = 200000$</td>
<td>$\nu_{xy} = 0.29$</td>
<td>$G_{xy} = 5127.5$</td>
<td>$X_t = 2400$</td>
</tr>
<tr>
<td>$E_y = 14050$</td>
<td>$\nu_{xz} = 0.29$</td>
<td>$G_{xz} = 5127.5$</td>
<td>$X_c = 2250$</td>
</tr>
<tr>
<td>$E_z = 14050$</td>
<td>$\nu_{yz} = 0.6$</td>
<td>$G_{yz} = 4390.6$</td>
<td>$Y_t = 69$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$Y_c = 350$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$S = 87$</td>
</tr>
</tbody>
</table>

$E_x$ Elastic modulus in the longitudinal direction.

$E_y$ and $E_z$ Elastic modulus in the transverse directions.

$G_{xy}$: In-plane shear modulus.

$G_{xz}$ and $G_{yz}$: Shear modulus in the transverse directions.

$X_t$: Longitudinal tensile strength.

$X_c$: Longitudinal compressive strength.

$Y_t$: Transverse tensile strength.

$Y_c$: Transverse compressive strength.

$S$: Longitudinal shear strength.

6.3.3 Adhesive properties

Cohesive element COH3D8, an eight-node, three-dimensional cohesive element, was used to model the adhesive material. Surface-based tie constraints were applied at interfaces, adhesive/CFRP and adhesive/steel. The materials used are reported in Table 6-1. Traction-separation behaviour and proper failure criteria were as described in Chapter 3, section 3.3.
6.3.4 Geometric imperfection

The out-of-straightness imperfection values were introduced into the FE analysis to initiate the overall buckling mode of failure, indicated by the buckling analysis (first stage analysis). For each beam, an initial geometric imperfection of span/1000 based on BS EN 1090-2 (2008) was adopted in the analysis.

6.3.5 Mesh

To arrive at an efficient mesh which can be used in the numerical modelling of the test series, four mesh schemes, with quad-dominated shape and distribution as shown in Figure 6-1, were examined by varying the number of elements along the width of the flange, web and along the beam length for the control beam (B0-FE), as shown in Figure 6-2 and Table 6-2. The results obtained for the mid-span deflection versus the load for the selected meshes showed a good convergence in terms of stiffness and ultimate load, except for mesh 4, which was unable to match the experimental ultimate load and produced a less stiff post-peak response compared to the other mesh arrangements. Given the lack of difference between meshes 1, 2 and 3, it would suggest that mesh 3 is the most efficient mesh density.

Table 6-2 Number of divisions for the mesh schemes of beam B0-FE.

<table>
<thead>
<tr>
<th></th>
<th>Number of elements along the web height</th>
<th>Number of elements across the flange width</th>
<th>Number of elements along the beam length</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>30</td>
<td>10</td>
<td>300</td>
</tr>
<tr>
<td>2</td>
<td>20</td>
<td>6</td>
<td>200</td>
</tr>
<tr>
<td>3</td>
<td>12</td>
<td>4</td>
<td>120</td>
</tr>
<tr>
<td>4</td>
<td>9</td>
<td>2</td>
<td>85</td>
</tr>
</tbody>
</table>
6.4 Simulation results

In addition to the numerical verifications that have been presented in Chapter 3, the numerical results are now also compared with the actual experimental results reported in Chapter 5, to obtain further information not provided by the experiments.

6.4.1 Unstrengthened beams

6.4.1.1 Beam B0-FE

The numerical load displacement relationship for B0-FE exhibit a linear elastic response up to around 350 kN, see Figure 6-3, and a slight stiffness which led to the yielding being delayed, which was at around 300 kN of load. Beyond this point, a localised yield in the top flange at mid-span began. Subsequently, the yielding of the top flange allowed a plastic hinge to occur, which caused lateral torsional buckling at mid-span at 407kN of load, see Figure 6-5, as was observed in the experiment with a
slight difference in maximum load (it was 406kN in the experiment). The model was aborted after excessive distortion.

6.4.1.2 Beam B1-UO-FE

Due to introducing the web opening at mid-span, see Figure 6-3, the FE results show that beam B1-UO-FE failed at lower load levels than beam B0-FE around 12%, while the stiffness was maintained. After linear load behaviour up to around 340kN, which was when the yielding started, significant torsional buckling started in the top T-section above the central opening. After peaking at around 357kN, unloading of the beam was accompanied by further lateral buckling of the section between the central restraints, see Figure 6-6.

6.4.1.3 Beam B2-UO-FE

Although this beam showed less stiffness than beam B1-UO-FE, linear load behaviour up to around 325kN, which was when yielding occurred in both corners around the opening, was consistent with the Vierendeel action, see Figure 6-7. The existence of opening at high shear area led to this reduction in stiffness and 18% in strength of the beam see Figure 6-3.

6.4.1.4 Beam B3-UO-FE

In this beam, two web opening were introduced at mid-span. The FE results show that beam B3-UO-FE failed at around 14% lower load levels than B0-FE, see Figure 6-3, while the stiffness was maintained. Yielding was initiated at a load of around 336kN in the top flange around both openings. After this, the lateral buckling started in the top flange above the openings and was accompanied by web-post buckling in the central part between the two openings. This lateral movement increased greatly, see Figure 6-8, until the ultimate load, 351kN, after which the load dropped and the model was stopped.
Figure 6-3 Load versus vertical displacement at mid-span: specimens B0-FE to B3-UO-FE.

Figure 6-4 Von Mises stress (MPa) of specimen B0-FE at 350kN of load.
Figure 6-5 Failure patterns for specimens B0-FE at 407kN of load and B0 at 406kN of load.

Figure 6-6 Von Mises stress (MPa) at ultimate load for Beam B1-U0-FE at 357kN of load.
Figure 6-7 Von Mises stress (MPa) at ultimate load for Beam B2-UO-FE at 335kN of load.

Figure 6-8 Von Mises stress (MPa) at ultimate load for Beam B3-UO-FE at 351kN of load.

6.4.2 Strengthened beams

6.4.2.1 Beam B1-RO-FE

Figure 6-9 presents the load deflection results at the beam’s mid-span obtained from the numerical simulations of the strengthened specimen, B1-RO-FE. Due to the presence of CFRP plates, the beam showed a 40% strength increase and stiffer
response than beam B1-UO-FE, without CFRP strengthening. Beam B1-RO-FE exhibited a fairly linear trend up to 444 kN of load. Beyond this load level, the onset of plastic hinge formation was in the top flange near the end of the CFRP plate. Consequently, the lateral buckling began at 478 kN of load. At this point, end debonding of the CFRP commenced in the top flange, see Figure 6-10. The ultimate load capacity was recorded as 503 kN; beyond that the load dropped dramatically, which allowed a new stage of lateral deformation to start. This stage was not available in the experiment because it was stopped after load dropping to maintain the test rig.

Nevertheless, the adhesive damage can be calculated from the numerical model using the damage variable of the adhesive, $D$, which reflects the damage evolution of the adhesive, as explained clearly in Chapter 3. This parameter can be found using eq. 3-5 and it can be seen from eqs. 3-3 and 3-4 that, if this parameter is equal to 1 for any element, this means that there are no stresses and the element has failed, after which it will be deleted. It is expressed by SDEG in the numerical output, which means stress degradation. It can be noticed from Figure 6-11 that SDEG or the D parameter reached a value of 1 in specimen B1-RO-FE at the adhesive end.

With regard to strain distribution on the CFRP, a lot of data can be gained from the FE model but, due to the maximum strain values that were recorded for that attached to the upper flange, it was selected to acquire the strain along which were not gained from the experiment, except for one strain gauge, FS1, in Figure 6-12.

The strain values along the CFRP are shown in Figure 6-13. As discussed in Chapter 1, the ends of the strengthening plate are usually subject to high shear stresses and co-existing normal stresses, these stress concentrations can initiate de-bonding. Due to the presence of these very localized stress concentrations, non-zero longitudinal strain values can be recorded close to the edge of the plates as found experimentally by Deng & Lee, (2007) and Linghoff et al., (2009). Since the presented strain in this case is
numerical, the value represents the solution at the given integration point of the finite element (spatially inset from the element edge) and is hence not the true edge of the plate. Improved levels of accuracy in this case may be obtained by using a much finer localized mesh at the plate ends. The maximum occurring value was 2100 microstrain at the CFRP end, which is about 18% of its maximum strain capacity, 12000 microstrain. This can be considered to be the utilization ratio i.e. the maximum occurring CFRP strain/maximum CFRP strain capacity. The utilization ratio can be a useful indicator as to how hard the strengthening plate is working, however the parameter has limitations since it is unable to give any indication of the spatial distribution i.e. where and over how much of the plate are high strains occurring and should therefore be viewed in that context.

Regarding the steel strain at the opening corner, no yielding state was indicated in any direction, X, Y or XY, as shown in Figure 6-14.

In terms of the numerical compatibility with the experiment results, B1-RO-FE and B1-RO showed a reasonable agreement in the stiffness aspect while a slight difference was noted in the ultimate load where, in the numerical simulation, the ultimate load was more than that obtained from the test by 3%. In addition, the strain from the FE strain gauges mounted on the CFRP and steel was in convergence with the experiment results, as shown in Figure 6-12 and Figure 6-14 respectively.
Figure 6-9 Load vs. mid-span deflection of beam B1-RO-FE.

Figure 6-10 Von Mises stress (MPa) and failure pattern at ultimate load for Beam B1-RO-FE at 503kN of load.
Figure 6-11 Adhesive damage parameter, SDEG, of beam B1-RO-FE at 503kN of load.

Figure 6-12 CFRP strain values of B1-RO-FE and B1-RO.
Figure 6-13 Strain value on CFRP at top flange in beam B1-RO-FE at ultimate load.

Figure 6-14 Steel strain values at opening corner of B1-RO-FE and B1-RO.
6.4.2.2 Beam B2-RO-FE

Beam B2-RO-FE showed a 31% strength increase and a stiffer response than beam B2-UO-FE, without CFRP strengthening, as shown in Figure 6-15. Beam B2-RO-FE also exhibited a linear trend up to 380kN of load. Beyond this load level, the onset of plastic hinge formation was in the top flange at the beam mid-span. Consequently, lateral buckling commenced at 396kN of load between the central lateral supports. The beam carried the load until 440kN, after which the load dropped. Finally, a plastic behaviour was noticed after that during loading due to excessive lateral deformation, as shown in Figure 6-16, which was not in the experiment because it was stopped once load dropping occurred.

Because this failure was away from the strengthening area, CFRP debonding did not happen. The adhesive was not damaged, which can be proved by the damage variable, \( D \) or \( SDEG \), where all its values were less than 1, as shown in Figure 6-17.

Regarding the strain values on the CFRP, the maximum strain values were recorded for that attached at the web below the opening. Therefore, it was selected to acquire the strain along which was not achieved from the experiment, except for one strain gauge, FS4, as shown in Figure 6-18. The strain distribution along the CFRP that attached at the web below the opening, see Figure 6-19, showed that the high strain values were at the opening edge which is close to the end support, while low values were at both ends of the CFRP. It can also be mentioned here that the maximum CFRP strain, 1610 microstrain, is about 13% of its maximum strain capacity, 12000 microstrain, which also indicates the lower utilisation ratio. Finally, steel strain at the opening corner confirmed that the yielding happened at the upper corner at the end support side where the high shear occurred, as shown in Figure 6-20.

In terms of the numerical compatibility with the experiment results for B2-RO-FE and B2-RO, a reasonable agreement was again recorded in the stiffness aspect, while a
slight difference in maximum load was observed in the numerical simulation, Figure 6-15, the ultimate load was more than that obtained from the test by 4%. This accuracy of the maximum load was also achieved by Seleem et al. (2010) and Fernando (2010). Also, the acquired FE strain and the test’s stain gauges mounted on the CFRP and steel was in agreement, as shown in Figure 6-18 and Figure 6-20 respectively.

Figure 6-15 Load vs. mid-span deflection of beam B2-RO-FE.
Figure 6-16 Von Mises stress (MPa) and failure pattern at ultimate load for Beam B2-RO-FE at 440kN of load.

Figure 6-17 Adhesive damage parameter, SDEG, of beam B2-RO-FE at 440kN of load.
Figure 6-18 CFRP strain values of B2-RO-FE and B2-RO.

Figure 6-19 Strain value on CFRP attached at the web below the opening in beam B2-RO-FE at ultimate load.
6.4.2.3 Beam B3-RO-FE

Figure 6-21 presents the load deflection at the beam’s mid-span in the numerical simulation of strengthened beam B3-RO-FE. Due to the presence of CFRP plates, the beam showed a 34% strength increase and a stiffer response than beam B3-UO-FE, without CFRP strengthening. Beam B3-RO-FE exhibited a fairly linear trend up to 415kN of load. Beyond this load level, the onset of yielding was in the top flange near the end of the CFRP plate at 448kN of load. Consequently, the lateral buckling started at 471kN of load, after which the load dropped with plastic lateral deformation. At this point, end debonding of the CFRP commenced in the top flange, see Figure 6-22.

Similar to B1-RO-FE, this debonding can be achieved clearly from the numerical model using the damage variable of the adhesive, SDEG, or D parameter. It can be noticed from Figure 6-23 that the SDEG parameter reached a value of 1 in specimen...
B3-FE at the adhesive end, which refers to adhesive failure. Hence, the debonding of the CFRP happened there.

According to the strain distribution on the CFRP, the maximum strain values were recorded for that attached to the upper flange. Therefore, the numerical model was used to acquire the strain along it. This was not gained from the experiment, except for one strain gauge, FS1, in Figure 6-24. As in B1-RO-FE, the strain values along the CFRP showed that the high strain values were also at the ends of the CFRP, as shown in Figure 6-25, and the maximum strain value was 1746 microstrain at the failure side before debonding occurred. This maximum strain value forms about 15% of its maximum strain capacity, 12000 microstrain, which also reflects the low utilisation ratio. Regarding the steel strain at the opening corner, SS1-FE, shown in Figure 6-26, no yielding state was reported in any direction, X, Y or XY. In comparison with the experiment results, the numerical results showed about a 6% difference in ultimate load, Figure 6-21.

Strain values of the CFRP and steel were in agreement with the test results except the strain value of SS1 was in difference with the test’s value. This may be attributed to the bad contact of this strain with the beam in test, as shown in Figure 6-24 and Figure 6-26 respectively.
Figure 6-21 Load vs. mid-span deflection of beam B3-RO-FE.

Figure 6-22 Von Mises stress (MPa) and failure pattern at ultimate load for Beam B3-RO-FE at 471kN of load.
Figure 6-23 Adhesive damage parameter, SDEG, of beam B3-RO-FE at 471kN of load.

Figure 6-24 CFRP strain values of B3-RO-FE and B3-RO.
Figure 6-25 Strain value on CFRP at top flange in beam B3-RO-FE at ultimate load.

Figure 6-26 Steel strain values at opening corner of B3-RO-FE and B3-RO.
6.5 Effect of CFRP thickness

Due to the low CFRP utilisation ratio in all specimens, which was confirmed by the numerical results, a parametric study was conducted to obtain the effective thickness of CFRP. In addition to the 3mm CFRP thickness that was investigated previously, 1 and 2mm thicknesses were also examined to identify the most effective thickness. It can be observed from Table 6-3 that there were significant increases in the strength of specimens B1-RO and B3-RO with CFRP thickness while smaller increases happened with B2-RO because a plastic hinge formed in the top flange at mid-span earlier than its formation near the end of the CFRP plate. As can be seen also from Figure 6-27 to Figure 6-29, in all specimens the increase of CFRP thickness not only helped to recover the stiffness and strength after introduction of web penetration but also gave a notable increase in stiffness and strength compared to that in the original beam and the beam with an opening.

Table 6-3 Percentage of strength achievement compared to specimen with web opening.

<table>
<thead>
<tr>
<th></th>
<th>CFRP thickness, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1mm</td>
</tr>
<tr>
<td>B1-RO-FE</td>
<td>23%</td>
</tr>
<tr>
<td>B2-RO-FE</td>
<td>28%</td>
</tr>
<tr>
<td>B3-RO-FE</td>
<td>24%</td>
</tr>
</tbody>
</table>
Figure 6-27 Load mid-span deflection response of specimen B1-RO-FE with different CFRP thicknesses.

Figure 6-28 Load mid-span deflection response of specimen B2-RO-FE with different CFRP thicknesses.
It is obvious that, by increasing the CFRP thickness, a small increase in strength is gained and also decreases in beam ductility, because the debonding takes place before the CFRP’s ultimate strengthening ability is achieved. Therefore, the use of each can provide an acceptable stiffness and strength recovery. However, the economic issue stands beside using of 1mm thick.

Because the optimum thickness of CFRP was investigated for strengthening of the section 305×102×25 UKB with web opening, it could not be used for the further sections unless it was linked to the web or flange thickness. This selection depends on whether the web opening is located at mid-span or shear side because the maximum strain on the CFRP was found at those attached to the flange for the web opening at mid-span while the maximum was recorded for those attached to the web for the web opening at shear side.

Figure 6-29 Load mid-span deflection response of specimen B3-RO-FE with different CFRP thicknesses.
Therefore, the proportion of CFRP thickness to the flange thickness (1/7) will be used for strengthening the web opening located at the mid-span while the proportion of CFRP thickness to the web thickness (1/5.8) will be used for strengthening the web opening located at the shear side for the case study in the next chapter.

6.6 Assumptions and limitations

The numerical results obtained in this study are based on the following assumptions:

1. In all the numerical models, the static load was subjected at 6 points load as in the test, it is assumed that the loads are equal.

2. The shell element, S4R, with reduced integration points was used in modelling the steel and CFRP while continuum element was used in modelling the adhesive with traction-separation behaviour.

3. The imperfections of steel beams are considered according to Eurocode 3 while the residual stress is neglected.

4. The material properties of CFRP plate and adhesive are extracted from the respective manufacturers’ published data.

5. The CFRP was unidirectional, 3mm thick and four times the opening length as a length. In reality, some degree of manufacturing and placement tolerance will be in existence as discussed in section 5.3.
6.7 Summary

The main aim of this chapter was to further validate the numerical model against the current experimental results. Additionally, due to the limitation in the experimental programme in terms of data capture, some of the details not obtained in the experimental tests were examined to provide greater insight, in particular the spatial distribution of stresses in the system. Furthermore, the validated model has been used to assess the effect of CFRP thickness on strengthening effectiveness with a view to design optimisation. From the results of this chapter, the following conclusion may be drawn:

1. Based on the simulation results of this and chapter 3, the numerical model is capable of simulating the behaviour of CFRP strengthened steel members with and without web opening to an acceptable level of accuracy. This conclusion was drawn after comparing the simulation results with all of the current experimental data in terms of load displacement response, failure modes and strain distribution.

2. The quasi-static analysis technique available in ABAQUS/Explicit can be employed to simulate the static force.

3. In comparison with the experiment results, the numerical results showed about, 3%, 4% and 6% difference for the beams B1, B2 and B3 respectively in maximum load.

4. It has been found that the highest strain values were recorded at failure at the ends of the CFRP plate due to stress concentration. This finding is consistent with that of existing experimental research e.g. (Deng & Lee, 2007), (Linghoff et al., 2009) etc.

5. Examining the strains across the CFRP strengthening plate and the associate damage parameter in the adhesive would suggest the often large parts of the
plate are lowly stressed compared to the very ends of the plate where the aforementioned stress concentrations occur. This may suggest plate thickness in many cases could be reduced, particularly if the plate and or adhesive profile at the ends can be altered to reduce stresses as discussed in the early chapters of this thesis.

6. It has been proven that CFRP can recover the strength of the steel section to that prior to introduction of the web openings. In parallel with this, the strengthening system can change the mode of failure in many cases.

7. Three different CFRP plate thicknesses were examined with a view to finding the most efficient layout. The CFRP thickness was characterised in terms of the steel flange or web thickness. For the case of openings at mid-span, a CFRP thickness of 1/7 of the flange was found to be most efficient. For the case of openings near the end of the beam, a CFRP thickness of around 1/6 of the web thickness was suggested.

The work presented here and in the previous chapters has concentrated on relatively small scale beams. In the next chapter, using the same validated model the CFRP strengthening technique will be explored within the context of larger scale beams of the size and loading common in commercial frame buildings. The suggested optimum ratios of CFRP plate to steel thickness will also be explored. For further demonstration of the practicality of CFRP plate strengthening, its performance will also be compared to that of the conventional strengthening technique for web openings i.e. welded steel plate.
CHAPTER 7

CASE STUDY

7.1 Introduction

The purpose of this chapter is to explore possible size effects, beam span and section size, and investigate whether there are practical limits of applicability for CFRP strengthening. Additionally, the performance of CFRP strengthening will be compared with conventional steel-plate strengthening.

7.2 Model description and material properties

To represent typical commercial floor arrangements, three different bay sizes of 7.5m×7.5m, 10m×10m and 15m×15m were framed using primary and secondary beams, see Figure 7-1, and designed in accordance with Eurocode 3 (BSI, 2005), see Table 7-1. All beams were designed as simply supported, carrying floor loads of 5kN/m² and 3.6kN/m² as live and dead load respectively. The concrete slab was assumed to provide full lateral restraint to the top flanges of the beams. Following initial design, each beam had a web opening introduced; the opening size was 0.75 of the beam depth as a length and 0.6 of the beam depth as a height, whether located at mid-span or near the supports, as shown in Figure 7-2.

The configuration of CFRP strengthening that was applied to the specimens in chapters 4 and 6 was also used here, with CFRP strips having a length of four times the opening length. The effective proportional ratios that have been concluded from Chapter 6 were used to select the CFRP width and thickness. Therefore, the same effective ratio,
CFRP width/flange, was used to select the CFRP width. In addition, the same effective ratios, CFRP thickness/web thickness and CFRP thickness/flange thickness, were used to select the CFRP thickness when the web opening location was at the shear side and mid-span of the beam respectively, see Table 7-2 to Table 7-4. The material properties of the CFRP and adhesive material used in this chapter were the same as those extracted from the experimental test and presented in chapters 4 and 6. The steel was assumed to be of grade S355 and the stress-strain curve is shown in Figure 7-3.

The steel density and the modulus of elasticity were 7850 kg/m³ and 200000 MPa respectively. The steel-plate strengthening has been designed accordance to SCI P355 (2011) guidance, as detailed in Chapter 2, with the same steel properties for the steel beam. The stiffeners were welded on both sides of the steel, above and below the openings.

Figure 7-1 Floor scheme.
Table 7-1 Beam sections.

<table>
<thead>
<tr>
<th></th>
<th>7.5m</th>
<th>10m</th>
<th>15m</th>
</tr>
</thead>
<tbody>
<tr>
<td>PB</td>
<td>533x312x273</td>
<td>914x419x343</td>
<td>1500x310x1020</td>
</tr>
<tr>
<td>SB</td>
<td>305x127x48</td>
<td>457x152x74</td>
<td>686x254x125</td>
</tr>
</tbody>
</table>

Figure 7-2 Opening size scheme.

Figure 7-3 Steel stress-strain used in the current simulations.
Table 7-2 Description of beams used in the 7.5×7.5m floor.

<table>
<thead>
<tr>
<th>Beam nomination</th>
<th>Beam section</th>
<th>Opening size, mm</th>
<th>Steel plate/CFRP strengthening</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Thick, mm</td>
<td>Length, mm</td>
<td>Width, mm</td>
</tr>
<tr>
<td>PB0-7.5</td>
<td>- - - -</td>
<td>-  - - -</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>PB1-UO-7.5</td>
<td>- - - -</td>
<td>-  - - -</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>PB2-UO-7.5</td>
<td>- - - -</td>
<td>-  - - -</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>PB1-ROS-7.5</td>
<td>12 800 80</td>
<td>5.4 1618 125</td>
<td></td>
<td>7.5m length primary beam with reinforced opening at mid-span by steel plates</td>
</tr>
<tr>
<td>PB1-ROC-7.5</td>
<td>3 1618 125</td>
<td>305×127×48</td>
<td>233.25×186.6</td>
<td>7.5m length primary beam with reinforced opening at high shear region by CFRP plates</td>
</tr>
<tr>
<td>PB2-ROS-7.5</td>
<td>12 800 80</td>
<td>5 470 80</td>
<td>204.63×233.7</td>
<td>7.5m length primary beam with reinforced opening at high shear region by steel plates</td>
</tr>
<tr>
<td>SB0-7.5</td>
<td>3 1618 125</td>
<td>3 933 50</td>
<td>31 ×1322×48</td>
<td>7.5m length secondary beam with reinforced opening at high shear region by CFRP plates</td>
</tr>
<tr>
<td>SB1-UO-7.5</td>
<td>3 933 50</td>
<td>3 933 50</td>
<td>204.63×233.7</td>
<td>7.5m length secondary beam with reinforced opening at high shear region by steel plates</td>
</tr>
<tr>
<td>SB2-UO-7.5</td>
<td>3 933 50</td>
<td>3 933 50</td>
<td>204.63×233.7</td>
<td>7.5m length secondary beam with reinforced opening at high shear region by CFRP plates</td>
</tr>
</tbody>
</table>
Table 7-3 Description of beams used in the 10×10m floor.

<table>
<thead>
<tr>
<th>Beam nomination</th>
<th>Beam section</th>
<th>Opening size, mm</th>
<th>Steel plate/CFRP strengthening</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>thick, mm</td>
<td>length, mm</td>
</tr>
<tr>
<td>PB0-10</td>
<td>914×419×343</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>PB1-UO-10</td>
<td>683.85×547.08</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>PB2-UO-10</td>
<td>10</td>
<td>1350</td>
<td>10</td>
<td>1618</td>
</tr>
<tr>
<td>PB1-ROS-10</td>
<td>10</td>
<td>1350</td>
<td>80</td>
<td>1618</td>
</tr>
<tr>
<td>PB1-ROC-10</td>
<td>10</td>
<td>1350</td>
<td>2.4</td>
<td>1386</td>
</tr>
<tr>
<td>PB2-ROS-10</td>
<td>10</td>
<td>1350</td>
<td>2.4</td>
<td>1386</td>
</tr>
<tr>
<td>SB0-10</td>
<td>457×152×74</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>SB1-UO-10</td>
<td>346.5×277.2</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>SB2-UO-10</td>
<td>5</td>
<td>700</td>
<td>5</td>
<td>700</td>
</tr>
<tr>
<td>SB1-ROS-10</td>
<td>5</td>
<td>700</td>
<td>2.4</td>
<td>1386</td>
</tr>
<tr>
<td>SB1-ROC-10</td>
<td>5</td>
<td>700</td>
<td>2.4</td>
<td>1386</td>
</tr>
<tr>
<td>SB2-ROC-10</td>
<td>1.4</td>
<td>1386</td>
<td>1.4</td>
<td>1386</td>
</tr>
<tr>
<td>Beam nomination</td>
<td>Beam section Opening size, mm</td>
<td>Steel plate/ CFRP strengthening</td>
<td>Description</td>
<td></td>
</tr>
<tr>
<td>-----------------</td>
<td>-------------------------------</td>
<td>--------------------------------</td>
<td>--------------</td>
<td></td>
</tr>
<tr>
<td>PB0-15</td>
<td>1500x310x1020</td>
<td>- - - -</td>
<td>15m length primary beam without opening or strengthening</td>
<td></td>
</tr>
<tr>
<td>PB1-UO-15</td>
<td>1125x900</td>
<td>- - - -</td>
<td>15m length primary beam with unreinforced opening at mid-span</td>
<td></td>
</tr>
<tr>
<td>PB2-UO-15</td>
<td></td>
<td>- - - -</td>
<td>15m length primary beam with unreinforced opening at high shear region</td>
<td></td>
</tr>
<tr>
<td>PB1-ROS-15</td>
<td>20 2200 80</td>
<td>11.4 1618 135</td>
<td>15m length primary beam with reinforced opening at mid-span by steel plates</td>
<td></td>
</tr>
<tr>
<td>PB1-ROC-15</td>
<td>20 2200 80</td>
<td>7.1 1618 135</td>
<td>15m length primary beam with reinforced opening at high shear region by steel plates</td>
<td></td>
</tr>
<tr>
<td>PB2-ROS-15</td>
<td>6 1010 80</td>
<td>2.3 933 70</td>
<td>15m length secondary beam with reinforced opening at mid-span by CFRP plates</td>
<td></td>
</tr>
<tr>
<td>PB2-ROC-15</td>
<td>6 1010 80</td>
<td>1.7 933 70</td>
<td>15m length secondary beam with reinforced opening at high shear region by CFRP plates</td>
<td></td>
</tr>
</tbody>
</table>
7.3 Material modelling, boundary conditions and meshing

The material modelling was the same as that explained in chapters 3 and 6 where a shell element S4R was employed to represent the steel and CFRP, while element COH3D8 was used for the adhesive material. The same failure criteria for each material were also adopted in the analysis.

In order to simulate the support boundary conditions, a rigid plate was defined at the ends of the beam, as shown in Figure 7-4. The plate was connected to the I-section through coupling constraints provided in ABAQUS using a reference point at the middle of the plate and mid-height of the I-section. The boundary conditions were then applied to the reference point (Najafi & Wang, 2017). For a simply supported condition, vertical and longitudinal restraints are applied at one end (to the reference point) while the other end is only restrained against vertical movement. This type of modelling can allow for any external axial load or bending moment to be applied at the support reference point without causing any eccentric loading condition. Beams were also laterally restrained along the span to prevent lateral torsional buckling.

The validated mesh described in Chapter 6 was used in these cases by assigning the number of divisions for length, height and width of beams keeping the same divisions/dimensions ratio.
7.4 Simulation results

This study comprised beam analysis of floors 7.5×7.5m, 10×10m and 15×15m, which were distributed as primary beams (PB) and secondary beams (SB). Both of them were strengthened with steel plates and CFRP plates after introducing a web opening at the bending zone (mid-span) in the first case and at the shear zone in the second case. All results have been compared with the control beams without openings and without strengthening in terms of applied load and failure mode.

7.4.1 Load mid-span deflection response

Figure 7-6 to Figure 7-15 show the total applied load versus mid-span deflection relationship for the beams (SB and PB) in the aforementioned cases. It is obvious also that the plastic behaviour, in the horizontal part of the curves, occurred for all cases.
due to the perfectly plastic behaviour assumed for steel in these cases. Firstly, the strength of both PB and SB of different floors dropped significantly after the introduction of a web opening at mid-span and shear region, as summarised in Table 7-5, while their strength was recovered completely after applying both types of strengthening, with a marginal increase in some of them. However, it can be noticed that the web opening at the shear region did affect the strength of the secondary beams and no further strength was provided by using either type of strengthening, as shown in Figure 7-8, Figure 7-12 and Figure 7-16. This can reflect that the selected opening size in this area of the beam has no effect on the shear resistance of SB2-UO-7.5, SB2-UO-10 and SB2-UO-15. Regarding the stiffness, all beams showed very similar stiffness compared to their reference beams, PB0-7.5, SB0-7.5, PB0-10, SB0-10, PB0-15 and PB0-15, after strengthening was applied.

7.4.2 Modes of Failure

Due to providing a full lateral restraint for all the beams, no lateral torsional buckling (LTB) was reported as a failure type. In addition to that, all beam sections are either plastic or compact. All reference beams failed by a conventional flexural failure mode in which a plastic hinge is formed at mid-span after top flange yielding.

Regarding the beams with a web opening at mid-span, PB1-UO-7.5, SB1-UO-7.5, PB1-UO-10, SB1-UO-10, PB1-UO-15 and SB1-UO-15, the top T-section was in compression; therefore, it was vulnerable to failing in buckling, see Figure 2-2 in Chapter 2, whereas, in the strengthened beams, PB1-ROC-7.5, PB1-ROS-7.5, SB1-ROC-7.5, SB1-ROS-7.5, PB1-ROC-10, PB1-ROS-10, SB1-ROC-10, SB1-ROS-10, PB1-ROC-15, PB1-ROS-15, SB1-ROC-15 and SB1-ROS-15, the plastic hinge formed at the end of strengthening, as shown in Table 7-6, Table 7-8 and Table 7-10.
On the other hand, the introduction of a web opening in the shear region had no effect on the beam strength and stiffness of SBs in which the failure mode was the same as that in the reference beams, SB0-7.5, SB0-10 and SB0-15, where the failure happened due to the formation of a plastic hinge in the beam mid-span. In contrast, the PBs failed in Vierendeel shear failure mode, see Figure 2-1 in Chapter 2, by developing four plastic hinges, see PB1-UO-7.5, PB1-UO-10 and PB1-UO-15 in Table 7-7, Table 7-9 and Table 7-11. However, strengthened PBs, PB1-ROC-7.5, PB1-ROS-7.5, PB1-ROC-10, PB1-ROS-10, PB1-ROC-15 and PB1-ROS-15, exhibited flexural failure and the plastic hinge formed at mid-span.

### 7.4.3 CFRP strain distribution

Table 7-12 to Table 7-14 demonstrate the strain distribution of SB and PB in different lengths and with CFRP plate strengthening for web opening at mid-span, beams PB1-ROC-7.5, SB1-ROC-7.5, PB1-ROC-10, SB1-ROC-10, PB1-ROC-15 and SB1-ROC-15, and at shear side, beams PB2-ROC-7.5, SB2-ROC-7.5, PB2-ROC-10, SB2-ROC-10, PB2-ROC-15 and SB2-ROC-15.

Due to the presence of web openings in the beams mid-spans, beams PB1-ROC-7.5, SB1-ROC-7.5, PB1-ROC-10, SB1-ROC-10, PB1-ROC-15 and SB1-ROC-15, a pure tensile strain developed in CFRP plates below the opening and a pure compressive strain was along the CFRP plates above the opening. Meanwhile, the strain distributions were different to those applied to the web openings at the shear sides, beams PB2-ROC-7.5, SB2-ROC-7.5, PB2-ROC-10, SB2-ROC-10, PB2-ROC-15 and SB2-ROC-15, where the strain localised at the openings corners in compressive and tensile values due to Vierendeel action. It can be reported here that the strain values on the CFRP were more for the latter than the former and increased with span length.
Furthermore, the utilisation ratio of CFRP, the maximum strain/allowable strain, for different beams ranged from 20-70%, 22-75% and 24-85% for beams of length 7.5m, 10m and 15m respectively, as shown in Table 7-15. These low values can be attributed to the plastic behaviour of steel materials that allows the plastic hinges to form sooner than in the strain-hardening behaviour.

It is also obvious that this ratio increased with beam length and size. It can be noticed also that it was significantly higher when the CFRP was applied at the shear region than in the mid-span. This can be attributed also to the strain localisation at the opening corners due to Vierendeel action.

### 7.4.4 Steel stiffeners’ stress distribution

As the steel plate is a ductile material, the von-Mises stress has been selected to make a comparison between the different cases. Table 7-16 to Table 7-18 illustrate the stress distribution of SB and PB in different lengths and with steel-plate strengthening for web opening at mid-span, beams PB1-ROS-7.5, SB1-ROS-7.5, PB1-ROS-10, SB1-ROS-10, PB1-ROS-15 and SB1-ROS-15, and at shear side, beams PB2-ROS-7.5, SB2-ROS-7.5, PB2-ROS-10, SB2-ROS-10, PB2-ROS-15 and SB2-ROS-15.

A uniform stress distribution can be noticed on the stiffeners applied to the web opening in the beams mid-span, beams PB1-ROS-7.5, SB1-ROS-7.5, PB1-ROS-10, SB1-ROS-10, PB1-ROS-15 and SB1-ROS-15, while stress localisation was on the stiffeners applied to the web opening in the shear side, beams PB2-ROS-7.5, SB2-ROS-7.5, PB2-ROS-10, SB2-ROS-10, PB2-ROS-15 and SB2-ROS-15. This was close to the opening’s corners due to Vierendeel action.

Regarding the utilisation ratio of the steel stiffener, the maximum stress/allowable stress, it ranged from 70-86%, 62-88% and 83-92% for beams of 7.5, 10 and 15m
length respectively, as shown in Table 7-19. It is obvious that this ratio increased with beam length and size.

In contrast, in the CFRP strengthening, even though the stress localisation was also observed in the stiffeners at the shear side, there were slight differences in values between the stiffeners applied at the shear region and those applied at the mid-span. This can be attributed to the way in which the steel stiffeners were applied, as they were welded vertically to the web at the opening edges. This can provide more stiffening for the opening edges.

7.4.5 Adhesive damage

As explained before, in chapters 3 and 6, the stress degradation, SDEG, was used to examine the damage status of the adhesive. Table 7-20 to Table 7-22 show this parameter for different beam lengths and sections. It can be seen clearly that there has been no debonding in any of the adhesive joints of all beams. This can be noticed from the values of damage parameter, SDEG, which are less than 1.0. It can also be observed that the maximum values of this parameter were at joint ends for those used to strengthen the opening in the beam mid-span, beams PB1-ROC-7.5, SB1-ROC-7.5, PB1-ROC-10, SB1-ROC-10, PB1-ROC-15 and SB1-ROC-15, while they were at the opening’s corners in those applied to the opening in the shear side, beams PB2-ROC-7.5, SB2-ROC-7.5, PB2-ROC-10, SB2-ROC-10, PB2-ROC-15 and SB2-ROC-15.
Table 7-5 Percentages of strength reduction due to web opening.

<table>
<thead>
<tr>
<th>Material Set</th>
<th>7.5%</th>
<th>10%</th>
<th>15%</th>
</tr>
</thead>
<tbody>
<tr>
<td>PB1-UO</td>
<td>12%</td>
<td>20%</td>
<td>24%</td>
</tr>
<tr>
<td>SB1-UO</td>
<td>14%</td>
<td>22%</td>
<td>13%</td>
</tr>
<tr>
<td>PB2-UO</td>
<td>25%</td>
<td>37%</td>
<td>19%</td>
</tr>
<tr>
<td>SB2-UO</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
</tr>
</tbody>
</table>

Figure 7-5 Load deflection curve of PB1-ROS-7.5, PB1-ROC-7.5, PB1-UO-7.5 and PB0-7.5.
Figure 7-6 Load deflection curve of SB1-ROS-7.5, SB1-ROC-7.5, SB1-UO-7.5 and SB0-7.5.

Figure 7-7 Load deflection curve of PB2-ROS-7.5, PB2-ROC-7.5, PB2-UO-7.5 and PB0-7.5.
Figure 7-8 Load deflection curve of SB2-ROS-7.5, SB2-ROC-7.5, SB2-UO-7.5 and SB0-7.5.

Figure 7-9 Load deflection curve of PB1-ROS-10, PB1-ROC-10, PB1-UO-10 and PB0-10.
Figure 7-10 Load deflection curve of SB1-ROS-10, SB1-ROC-10, SB1-UO-10 and SB0-10.

Figure 7-11 Load deflection curve of PB2-ROS-10, PB2-ROC-10, PB2-UO-10 and PB0-10.
Figure 7-12 Load deflection curve of SB2-ROS-10, SB2-ROC-10, SB2-UO-10 and SB0-10.

Figure 7-13 Load deflection curve of PB1-ROS-15, PB1-ROC-15, PB1-UO-15 and PB0-15.
Figure 7-14 Load deflection curve of SB1-ROS-15, SB1-ROC-15, SB1-UO-15 and SB0-15.

Figure 7-15 Load deflection curve of PB2-ROS-15, PB2-ROC-15, PB2-UO-15 and PB0-15.
Figure 7-16 Load deflection curve of SB2-ROS-15, SB2-ROC-15, SB2-UO-15 and SB0-15.
Table 7-6 Failure mode and von-Mises stresses (MPa) at maximum load of SB & PB with web opening at mid-span with a beam 7.5m in length.
Table 7-7 Failure mode and von-Mises stresses (MPa) at maximum load of SB & PB with web opening at shear region with a beam 7.5m in length.
Table 7-8 Failure mode and von-Mises stresses (MPa) at maximum load of SB & PB with web opening at mid-span with a beam 10m in length.

<table>
<thead>
<tr>
<th>Failure Mode</th>
<th>von-Mises Stresses (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SB0-10</td>
<td></td>
</tr>
<tr>
<td>PB0-10</td>
<td></td>
</tr>
<tr>
<td>SB1-UO-10</td>
<td></td>
</tr>
<tr>
<td>PB1-UO-10</td>
<td></td>
</tr>
<tr>
<td>SB1-ROC-10</td>
<td></td>
</tr>
<tr>
<td>PB1-ROC-10</td>
<td></td>
</tr>
<tr>
<td>SB1-ROS-10</td>
<td></td>
</tr>
<tr>
<td>PB1-ROS-10</td>
<td></td>
</tr>
</tbody>
</table>

Legend:
- +3.550e+02
- +3.257e+02
- +2.964e+02
- +2.670e+02
- +2.377e+02
- +2.084e+02
- +1.791e+02
- +1.497e+02
- +1.204e+02
- +9.108e+01
- +6.175e+01
- +3.243e+01
- +3.101e+00
Table 7-9 Failure mode and von-Mises stresses (MPa) at maximum load of SB & PB with web opening at shear region with a beam 10m in length.
Table 7-10 Failure mode and von-Mises stresses (MPa) at maximum load of SB & PB with web opening at mid-span with a beam 15m in length.
Table 7-11 Failure mode and von-Mises stresses (MPa) at maximum load of SB & PB with web opening at shear region with a beam 15m in length.

<table>
<thead>
<tr>
<th></th>
<th>SB0-15</th>
<th>PB0-15</th>
</tr>
</thead>
<tbody>
<tr>
<td>SB2-UO-15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SB2-ROC-15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SB2-ROS-15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PB2-UO-15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PB2-ROC-15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PB2-ROS-15</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Legend:
-3.650e+02
-3.259e+02
-2.968e+02
-2.677e+02
-2.386e+02
-2.095e+02
-1.805e+02
-1.514e+02
-1.223e+02
-9.321e+01
-6.412e+01
-3.503e+01
+5.944e+00
Table 7-12 Strain distribution on CFRP plates at maximum load of SB & PB with a beam 7.5m in length.

<table>
<thead>
<tr>
<th></th>
<th>SB1-ROC-7.5</th>
<th>PB1-ROC-7.5</th>
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</thead>
<tbody>
<tr>
<td>SB1</td>
<td>ROC-7.5</td>
<td>ROC-7.5</td>
</tr>
<tr>
<td>PB1</td>
<td>ROC-7.5</td>
<td>ROC-7.5</td>
</tr>
</tbody>
</table>

Table 7-13 Strain distribution on CFRP plates at maximum load of SB & PB with a beam 10m in length.

<table>
<thead>
<tr>
<th></th>
<th>SB1-ROC-10</th>
<th>PB1-ROC-10</th>
</tr>
</thead>
<tbody>
<tr>
<td>SB1</td>
<td>ROC-10</td>
<td>ROC-10</td>
</tr>
<tr>
<td>PB1</td>
<td>ROC-10</td>
<td>ROC-10</td>
</tr>
</tbody>
</table>
Table 7-14 Strain distribution on CFRP plates at maximum load of SB & PB with a beam 15m in length.

Table 7-15 Utilisation percentages of CFRP in different beams.

<table>
<thead>
<tr>
<th></th>
<th>7.5m</th>
<th>10m</th>
<th>15m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mid-span opening</td>
<td>Shear side opening</td>
<td>Mid-span opening</td>
</tr>
<tr>
<td>SB</td>
<td>20</td>
<td>31</td>
<td>22</td>
</tr>
<tr>
<td>PB</td>
<td>24</td>
<td>70</td>
<td>28</td>
</tr>
</tbody>
</table>

Table 7-16 Von Mises stress distribution (MPa) on steel stiffeners at maximum load of
SB & PB with a beam 7.5m in length.

Table 7-17 Von Mises stress distribution (MPa) on steel stiffeners at maximum load of SB & PB with a beam 10m in length.

Table 7-18 Von Mises stress distribution (MPa) on steel stiffeners at maximum load of
SB & PB with a beam 15m in length.

Table 7-19 Utilisation percentages of steel plates in different beams.

<table>
<thead>
<tr>
<th></th>
<th>7.5m</th>
<th></th>
<th>10m</th>
<th></th>
<th>15m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mid-span opening</td>
<td>Shear side opening</td>
<td>Mid-span opening</td>
<td>Shear side opening</td>
<td>Mid-span opening</td>
</tr>
<tr>
<td>SB</td>
<td>70</td>
<td>SB</td>
<td>77</td>
<td>SB</td>
<td>86</td>
</tr>
<tr>
<td>PB</td>
<td>79</td>
<td>PB</td>
<td>62</td>
<td>PB</td>
<td>94</td>
</tr>
</tbody>
</table>

Table 7-20 Adhesive damage parameter, SDEG, of beams 7.5m in length at maximum load.
Table 7-21 Adhesive damage parameter, SDEG, of beams 10m in length at maximum load.

Table 7-22 Adhesive damage parameter, SDEG, of beams 15m in length at maximum load.
7.5 Assumptions and limitations

The numerical results obtained in this study are based on the following assumptions:

1. Three different bay sizes of 7.5×7.5m, 10×10m and 15×15m were framed using primary and secondary beams. All beams were designed as simply supported in accordance with Eurocode 3 and carrying 5kN/m² and 3.6kN/m² as live and dead load respectively.

2. The opening size was set at 0.75 and 0.6 of the beam depth as a length and height respectively, whether located at mid-span or near the supports.

3. The configuration of CFRP strengthening (denoted as type B) in Chapter 3 was applied to the specimens having a length of four times the opening length.

4. The shell element, S4R, with reduced integrations points was used in modelling the steel and CFRP while cohesive element COH3D8 was used in modelling the adhesive with traction-separation behaviour.

5. The imperfections of steel beams are considered according to Eurocode 3 while the residual stress is neglected.

6. The material properties of CFRP plate and adhesive are extracted from the manufacturer’s declared values.

7. The CFRP thickness was proportional to flange thickness when the opening was located at mid-span, and proportional to web thickness when the web opening was located at the shear side.

8. The bi-linear stress-strain relationship of steel is used according to Eurocode 3 with elastic-perfect plastic behaviour.

9. The steel-plate strengthening has been designed accordance to SCI P355 (2011) guidance with the same material properties for the steel beam.
10. In order to create the support boundary conditions, a rigid plate was defined at the ends of the beam. The plate was connected to the I-section through coupling constraints provided in ABAQUS using a reference point at the middle of the plate and mid-height of the I-section, vertical restraint.

11. All beams were also laterally restrained along the top flange to prevent lateral torsional buckling, however no composite action with a floor slab was assumed.

7.6 Summary

Several numerical simulations have been conducted in this chapter to explore the ability of CFRP strengthening to recover the stiffness and strength of full-scale steel beams after the introduction of a web opening. In addition, the performance of CFRP strengthening has been compared with conventional steel-plate strengthening. Three types of full-scale steel beams, 7.5m, 10m and 15m in length, with two web opening locations were used. The findings of this study can be summed up as follows:

1. The CFRP-strengthening method is applicable to recover the stiffness and strength of full-scale beams with different lengths and sections after insertion of a web opening, whether at mid-span or at the beam ends in regions of high shear.

2. Although the stiffness and strength have been recovered, the full plastic behaviour of the steel material did not allow the optimum utilisation of CFRP to be achieved.

3. The utilisation ratio of CFRP strengthening is more than 40% while it is more than 60% for the steel-plate strengthening. This means that it may be possible to reduce the CFRP grade or its thickness from the values considered here.
4. The CFRP can give similar strengthening effect to that from traditional steel plate designed by the SCI P355 method.

5. No de-bonding type failure has been reported for any of the beams in the different cases.
CHAPTER 8

CONCLUSIONS

8.1 Overview of Research

This thesis has presented the details and results of an investigation into the use of carbon fibre reinforced polymer (CFRP) composites to strengthen steel beams after the introduction of web openings. The objective of this work was to devise an efficient CFRP strengthening arrangement which could recover the strength and stiffness of the steel beams after web openings were introduced. First of all, the proposed FE model was validated using previous experimental works, hence proving the model’s ability to capture the actual behaviour of steel beams with CFRP composites to an acceptable level of accuracy. Then, the validated model was used to investigate the most effective configuration, length and thickness of CFRP plates which were then used later in a series of experimental tests. The effectiveness of CFRP strengthening was investigated experimentally by testing of four specimens, one without an opening acting as the control, and three with rectangular web openings in different locations.

To gain understanding of potential size effects and the limits of applicability of the proposed strengthening system, further numerical modelling was used to explore the effect of beam span with a particular focus on spans and depths common in a typical modern frame building. Furthermore, the proposed CFRP strengthening system was compared with the industry standard approach of strengthening via welded steel plate stiffeners.
How the FRP system works: General Conclusions

For each of the different beam geometries, opening scenarios and strengthening configurations considered in this thesis, the FRP strengthening system has proven effective. The way by which this is achieved is as follows:

1. Where a single web opening is located at mid-span, the CFRP system is able to re-distribute the peak stresses away from the T-section above the opening (i.e. the flange and remaining component of web). In the un-strengthened case, yielding and/or buckling of the T-section initiates failure at load levels generally below that of the beam without openings. In the strengthened case, peak stresses are distributed to approximately the ends of the middle third span of the beam where the full depth of the steel section is mobilised, eventually forming a plastic hinge. A typical example of this trend can be seen in Table 7-6 where a 60% reduction in peak stress around the opening is observed in the strengthened 7.5m secondary beam SB1-ROC-7.5 compared to the un-strengthened case SB1-UO-7.5. This level of local stress reduction is commensurate with the level of section modulus enhancement at the T-section brought about by the addition of the CFRP. In the same strengthened beam, redistribution of stresses is observed as a plastic hinge forms some distance away from the opening at approximately the ends of the middle third span.

As discussed in Chapter 1, the most common failure mode of the strengthened system is de-bonding of the CFRP from the steel surface, this typically occurs long before rupture of the CFRP plate can occur. Crucially, the CFRP is able to act fully compositely with the steel section in the vast majority of the strengthened cases throughout the loading process since no de-bonding occurs,
thus allowing the enhanced section properties to be mobilised. The absence of de-bonding indicates the length of CFRP plate and chosen thickness dimension is sufficient to fully mobilise the section and develop full force transfer as discussed in section 1.5. It should be noted however that a few of the beam scenarios examined were close to the threshold of de-bonding at ultimate load as indicated by the numerical damage parameters. Use of thinner CFRP sections along with adoption of tapered ends may reduce the associated peeling and shear stresses in these cases.

2. In the case of single openings located near the end of the beam in the high shear zone, the dominant mode of failure in the un-strengthened case is yielding in the T-section above and below the opening due to the Vierendeel action generated. As in the case for the mid-span openings, the strengthening system is able to reduce the peak stresses around the opening and allow redistribution of stresses compared to the un-strengthened case. A typical example of this can be seen in Table 7-7 where a 50% reduction in peak stresses at the opening is observed in beam PB2-ROC-7.5 compared to the un-strengthened beam PB2-UO-7.5. Since no de-bonding occurred, the fully developed composite action and associated enhanced section modulus around the opening avoids local failure in this area and has allowed the beam to develop full plastic section capacity at the mid-span section leading to development of the plastic hinge and ultimate loads comparable to the same beam without an opening i.e. PB0-7.5.
8.3 **Experimental Work: Conclusions**

1. It is clear that for all the strengthened beams examined, a stiffer response was observed up to ultimate load compared to the un-strengthened cases. This is due to the fact that the CFRP is creating an enhanced composite I’ value which is favorably altering the stress state in the locations corresponding to the peak stresses in the un-strengthened beam, as discussed in section 8.2. Furthermore, in all cases the strengthened beams showed greater load capacity, 503kN, 440kN and 471kN for B1-RO, B2-RO and B3-RO respectively, after strengthening with web openings present compared to the control case which had no openings or strengthening. Therefore, this method is likely to be a useful approach in practice to recover the stiffness and strength of beams where in-service web openings are introduced.

2. As expected, for the un-strengthened beams where web openings were introduced, the ultimate strength reduced, i.e. by 11.8%, 17.5% and 13.3% for B1-UO, B2-UO and B3-UO respectively.

3. The proposed strengthening method not only helped to recover the beam's strength but also resulted in an increase of ultimate strength by 20%, 5% and 10% for beams B1-RO, B2-RO and B3-RO respectively compared with control beam, B0.

4. The experiments reveal that the proposed CFRP strengthening system changed the failure location and mechanism of the strengthened beams which were not always the same as the un-strengthened cases, or the control case. Therefore, care must be taken to identify and check potential failure other than those that might be expected with strengthening.
5. As discussed in Chapter 1, the phenomenon of de-bonding is the most common cause of premature failure of the strengthening system. For the beams tested experimentally, no de-bonding of the CFRP plate occurred at the target ultimate loads (i.e. the ultimate of the same beam without an opening). As referred to in section 8.2, full composite action was achieved in test series. This suggested the bond length of CFRP plate configuration was adequate. Similarly, the use of the high-modulus adhesive, in this case Araldite 420, ensured full shear transfer between steel and CFRP occurred.

8.4 Numerical Modelling of Experimental Series: Conclusions

1. When compared with previous and current experimental tests in terms of ultimate load, mode of the failure, strain distribution and load-deflection response both for un-strengthened specimens and strengthened specimens, the proposed FE model showed an acceptable level of accuracy. Based on this, the numerical models used to explore the proposed CFRP strengthening system can be viewed with a good level of confidence.

2. Crucially, the proposed FE model is shown to be capable of capturing the key phenomena of plate de-bonding and buckling of the steel section (local and global).

3. In comparison with the experiment results, the numerical results showed about, 3%, 4% and 6% for the beams B1, B2 and B3 respectively in maximum load.
8.5 Full scale numerical modelling study: Conclusions

1. In the case of full-scale steel beams, the proposed CFRP strengthening system was shown to be a suitable alternative to traditional steel stiffeners in recovering beam stiffness and strength. This applied to both scenarios of when web openings were presented at mid-span and near the end of beams.

2. Different failure modes have been observed for strengthened beams to those for un-strengthened beams. For the un-strengthened case, where the opening was a mid-span, local failure of the T-section (yielding followed by buckling) occurred. For the un-strengthened beams with openings near the ends, Vierendeel action led to localised yielding around the corners of the opening. In the CFRP strengthened cases for both opening locations, the localised failure was avoided, with plastic hinges eventually forming at ultimate loads, well away from the opening position. As in the experiments, bond breakdown was not a problem in all strengthened beams which achieved full composite action between the steel beam and the CFRP strengthening system.

3. No CFRP rupture occurred in all strengthened beams thus suggesting a lesser amount of CFRP could be used.

4. The proposed CFRP system is able to produce a strengthening performance comparable to that of the standard steel plate stiffener approach.

5. Using the proposed CFRP geometry, the utilisation ratio was more than 40%. On the other hand, the utilisation ratio was more than 60% with steel stiffener strengthening using SCI P355 to select the steel plate geometry.

6. The steel grade has a significant role to play in the optimum utilisation of CFRP.
8.6 Recommendations for Future Research

Over the limited series of tests and numerical studies undertaken, the CFRP strengthening approach has been demonstrated as alternative method to reinforce web openings as an alternative to steel stiffeners. Further research is required to be carried out in the following areas:

1. Since several of the strengthening plates appeared to be under-utilised, further in-depth study should be conducted to find the optimum plate thickness while accounting for different steel grades. Similarly, the end geometry of the plate could be studied further, including tapers etc. with a view to potential de-bonding issues at higher utilization ratios.

2. In regards to economy, further study is needed to investigate the ability of alternative composites such as GFRP to reinforce web openings.

3. The current study is limited to CFRP strengthening of one or two rectangular opening shapes. Further studies on multiple openings and other common opening shapes are necessary.

4. Further numerical modelling including finer localised meshes at plate ends and corners of openings could be conducted to gain further understanding of the role of the associated stress concentrations in failure mechanisms such as de-bonding.
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