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Master's degree Thesis

Numerical investigation of Cold-formed steel bolted moment connections

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ABSTRACT

The demand for efficient structures in terms of economic and environmental impacts is essential at present to help initialize modern sustainable infrastructures. Light steel framing and Modular modern construction systems are the best latest example where Cold-formed steel (CFS) framing systems are implemented increasingly drawing the attention in steel construction market. This is thanks to CFS flexibility in terms of wide variety of profiles, high strength to weight ratio, short time execution and the ease of transportation and erection which allows to achieve efficient and economic design solutions. At this time, CFS are used as primary structural elements especially in modular systems. However, their use is still limited to low- and mid-rise buildings due to their restricted ductility. Thus, the development of a new framing system had to be arisen. CFS connections are now the focus of recent research to widen their use as a design solution even in seismic areas. According to recent research, the proposed novel CFS beam-to-column with through plate connection has proven to have promising results in terms of strength and ductility. This research thesis investigates CFS moment resistant connections numerically by the development of an analytical model using ABAQUS validated against previous research analytical and experimental tests to follow and support the most recent studies carried out in this field and create base model for future work. In addition to that, supportive consideration regarding state-of-the-art CFS green construction practices has been conducted to illustrate the cost-effective and ecological consequences of CFS framing which form a great step towards sustainable buildings in the light of their contribution to the well-being of future occupants and the surrounding population. The results of the numerical simulations are in good agreement with prior research demonstrating the structural capacity of CFS members to be employed as principal structural members for multi-storey earthquake resisting frames. Nevertheless, the behavior of such connections needs to be enhanced to best find appropriate design solutions. The results are reported in terms of comparison with former studies revealing the gaps and discussing the most aspects that can be improved and be part of upcoming research focus.

CHAPTER 1. Introduction

1.1 General background

The use of cold-formed steel (CFS) members in building construction goes back to the middle of the 19th century in both the United States and England. However, the acceptance of such steel members as a construction material was still limited until around 1946 when the development of thin-walled cold-formed steel construction in the United States was realized by the issuance of various editions of the "Specification for the Design of Cold-Formed Steel Structural Members" of the American Iron and Steel Institute (AISI). Using this type of sections in construction has been of growing importance in the recent years due to their positive contribution to lowering environmental risks and reducing the amount of carbon emissions and construction waste compared to the typical Hot-rolled steel sections and to other materials such as concrete. Additionally, these sections can lead to economic and design solutions with less material and waste due to their higher strength-to-weight ratio and flexibility for obtaining unusual sectional shapes in comparison with hot-rolled ones (Figure 1.1). Moreover, While Hot-rolled steel profiles are formed at elevated temperature, the fabrication process of CFS counterparts can be performed at room temperature using less energy and reducing cost. Therefore, CFS structural systems are appropriate options for modular and multi-storey buildings from sustainable point of view (Figure 1.2).

CFS material properties allow them to also provide offsite construction solutions which leads to cost-effective as well as quicker construction. The environmental and social concerns call for a new material that serves advanced and efficient technology of construction and helps meet the building targets and the development of built environment. Thus, CFS framing is at the research focus and studies to promote their employment to industry.

Figure 1.1 Various shapes of cold-formed sections (Yu et al., 2019).

Figure 1.2 (a) CFS Portal frame - www.mcelroymetal.com and
(b) Multi-Storey mid-risw CFS building (Yu et al., 2019) www.buildsteel.org

1.2 Design standards background

Until 1940s, there were not standards applicable to cold–formed sections because of their relatively thin steel walls which were susceptible to buckling.

Since cold-formed steel structural members are usually made of relatively thin steel sheet and come in many different geometric shapes in comparison with typical hot-rolled sections, the structural behavior and performance of such thin-walled, cold-formed structural members under loads differ in several significant respects from that of heavy hot-rolled steel sections. In addition, the connections and fabrication practices which have been developed for cold-formed steel construction differ in many ways from those of heavy steel structures. As a result, design specifications for heavy hot-rolled steel construction cannot possibly cover the design features of cold-formed steel construction completely. It soon became evident that the development of a new design specification for cold-formed steel construction was highly desirable (Yu et al., 2019).

In the United States, the first edition of the "Specification for the Design of Light Gage Steel Structural Members" was published by the American Iron and Steel Institute (AISI) in 1946 (American Iron and Steel Institute, 1946).

In Canada, the Canadian Standards Association (CSA) published its first edition of the Canadian Standard for Cold-Formed Steel Structural Members in 1963 based on the 1962 edition of the AISI Specification with minor changes.

In Mexico, cold-formed steel structural members have always been designed according to the AISI specification. The 1962 edition of the AISI design manual was translated into Spanish in 1965.

In 1994, Canada, Mexico, and the United States implemented the North American Free Trade Agreement (NAFTA). Consequently, the first edition of North American Specification for the Design of Cold-Formed Steel Structural Members (NAS) was developed in 2001 by a joint effort of the AISI Committee on Specifications, CSA Technical Committee on Cold-Formed Steel Structural Members, and Camara Nacional de la Industria del Hierro y del Acero (CANACERO) in Mexico (American Iron and Steel Institute, 2003). It included the ASD and LRFD methods for the United States and Mexico together with the Limit States Design (LSD) method for Canada.

In 2004, AISI issued a Supplement to the 2001 Edition of the North American Specification that provides the revisions and additions for the Specification. This supplement included a new Appendix for the design of cold-formed steel structural members using the direct-strength method (DSM). The first edition of the North American Specification was revised and expanded in 2007 to the second edition of the North American Specification for the Design of Cold-Formed Steel Structural Members (American Iron and Steel Institute, 2007). The document was prepared based on the 2001 edition of the Specification, the Supplement 2004 to the 2001 Specification, and subsequent developments. The North American specification has been approved by the ANSI and is referred to in the United States as AISI S100. It has also been approved by the CSA and is referred to in Canada as S136 (Dubina et al., 2013).

In Europe, the ECCS Committee TC7 originally produced the European Recommendations for the design of light gauge steel members in 1987 (ECCS, 1987). This European document has been further developed and published in 2006 as the European Standard Eurocode 3: (CEN, 2006).

Additionally, Australian/New Zealand Standards (Australia, 2018) are amongst many other standards for CFS structures.

1.3 CFS structural members

Cold-formed steel structural members family includes sections cold formed from steel sheet, strip, plate, or flat bar of uniform thickness by Rolling, Press-Braking or Folding at ambient temperatures. Therefore, contrary to thicker hot-rolled sections, they have a very high strength-to-weight ratio and can be designed in many different configurations providing more efficient solutions (Figure 1.1).

1.3.1 Methods of forming

Cold-formed members are normally manufactured by one of the following processes (Figure 1.3):

- Roll forming
- Folding
- Press-Braking

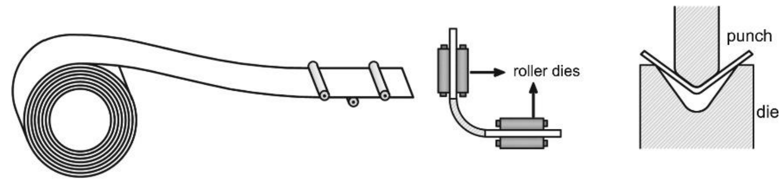


Figure 1.3 Manufacturing Process for CFS members (Amouzegar et al., 2016).

Roll forming consists of a sequence of stages where at each stage a fixed amount of deformation can be introduced to a steel strip by a pair of rolls to form the required profile. Generally, greater number of stages are needed in order to realize more complex cross-sectional shapes as shown in (Figure 1.4). This method has been widely used to produce building components such as individual structural members. (Figure 1.5) shows an industrial roll forming line for long products profiles. This process is used for large volume production.

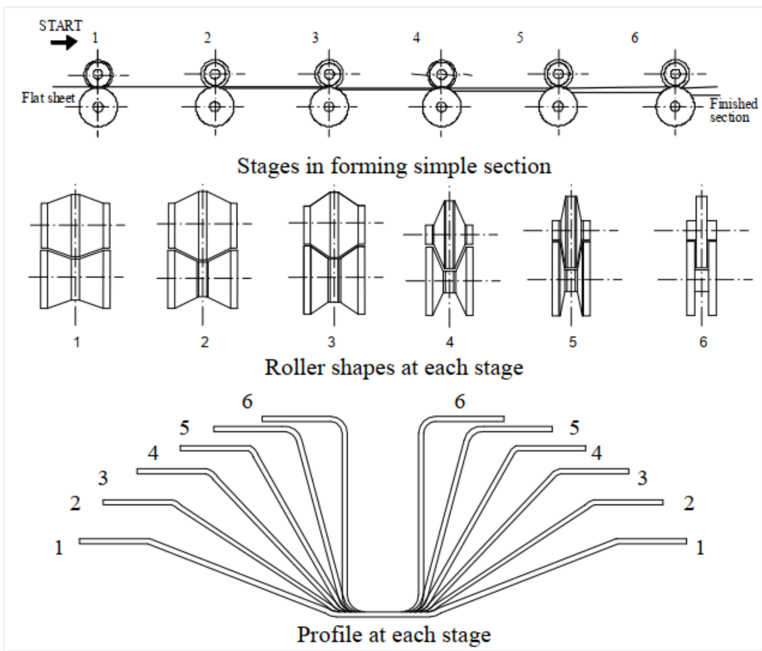


Figure 1.4 Stages in roll forming a simple section (Rhodes, 1991).

Figure 1.5 Cold-roll-forming machine - www.voestalpine.com

Folding process is a simple process if compared to roll forming. It is used to produce specimens of short length, and of simple geometry from a sheet of material by folding a series of bends (Figure 1.6). This process has very limited application (Rhodes, 1991).

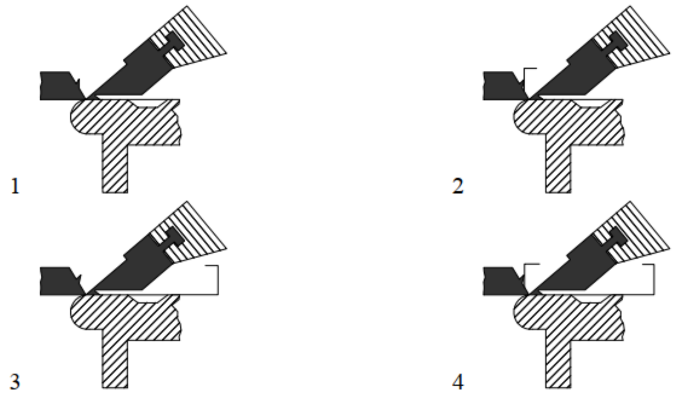


Figure 1.6 Forming of folding (Dubina et al., 2013)

Press-Braking operation is used in case of simple sections and low volume production. Greater variety of profiles can be produced by this process through which the section is formed from a strip by pressing it between shaped dies to form the profile shape (Dubina et al., 2013). (Figure 1.7) illustrates the forming steps in Press-Braking process which has limitations on the geometry and more importantly on the lengths of profiles that can be formed.

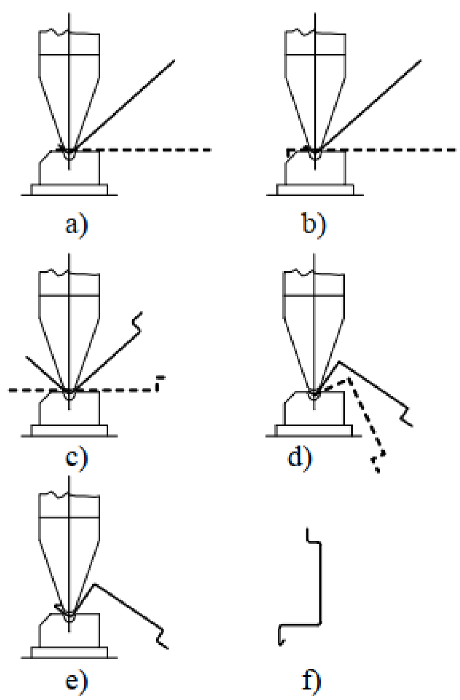


Figure 1.7 Forming steps in Press-Braking process (Dubina et al., 2013).

1.3.2 CFS Sections

The thickness of a light gauge steel sheet or strip that can be cold-formed into structural profiles ranges from 0.3 mm to about 6 mm. Sections up to 25 mm can be cold-formed due to recent developments.

Cold-formed structural members can be classified into two categories (Dubina et al., 2013):

- Individual structural framing members (Figure 1.8)
- Panels and decks

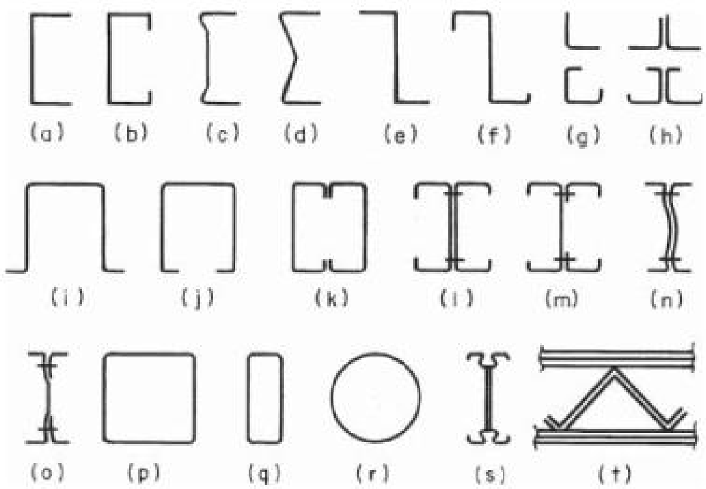


Figure 1.8 Cold-formed sections used in structural framing (Yu et al., 2019).

Structural framing members includes 3 different types as illustrated in (Figure 1.9):

a) Single open sections
b) Open built-up sections
c) Closed built-up sections

The depth of these sections ranges between 50 mm to 400 mm while the thickness can be from 0.5 to 6 mm. The usual shapes are channels (C-sections), Z-sections, angles, hat sections, I-sections, T-sections, and tubular members.

On the other hand, panels and decks depth ranges between 20 - 200 mm, while the thickness from 0.4 to 1.5 mm. (Figure 1.10) shows typical profiled sheets and linear trays from which panels and decks can be made. These kind of CFS sections can be used generally as roof decks, floor decks, wall panels, siding material, and bridge forms. They can provide surfaces on which flooring, roofing, or concrete fill can be applied in addition to structural capability of carrying loads. Moreover, they can provide enclosed cells for electrical and other conduits.

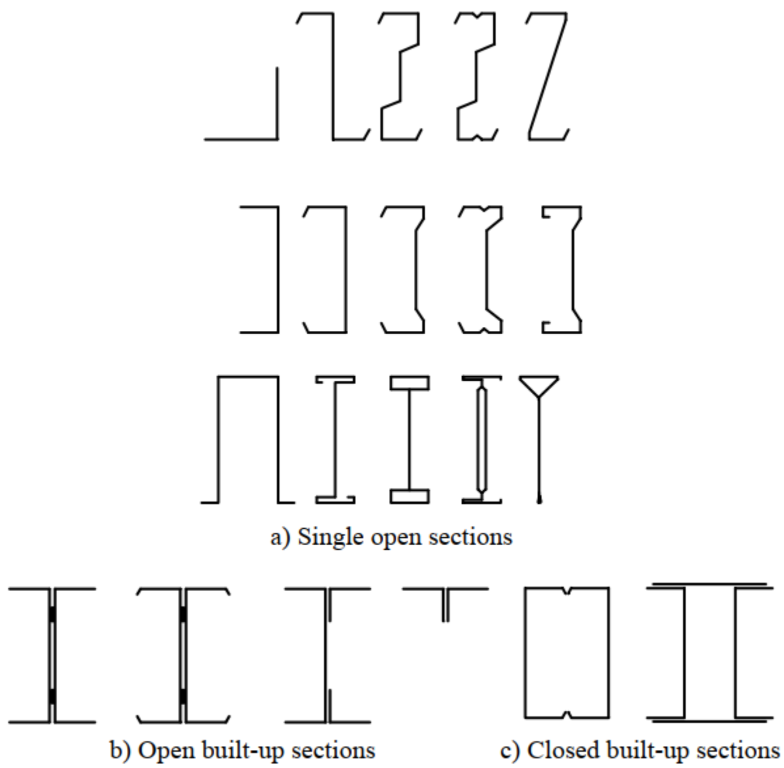


Figure 1.9 Typical forms of sections for cold-formed structural members (Dubina et al., 2013).

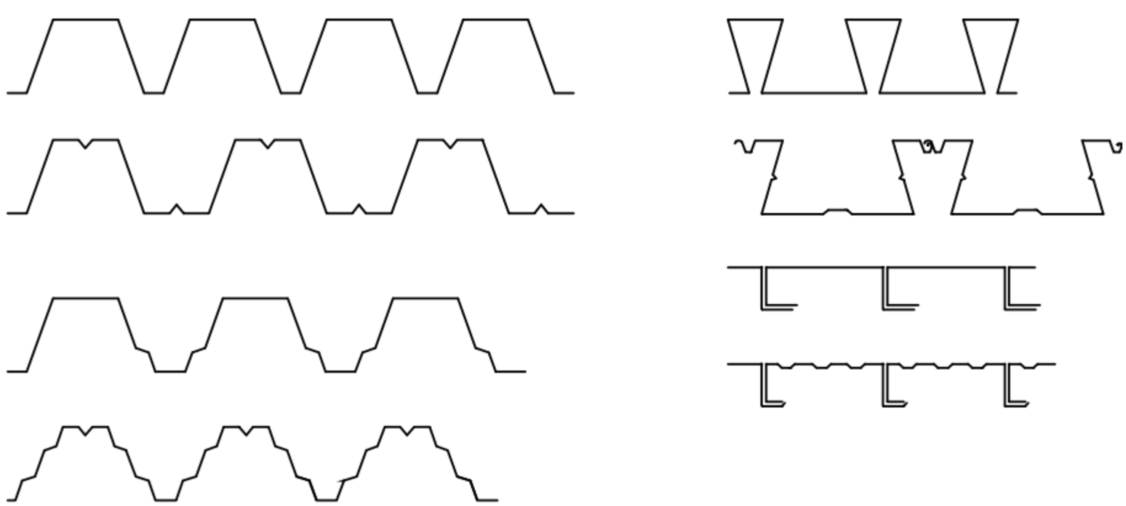


Figure 1.10 Profiled sheets (Dubina et al., 2013).

Generally, CFS structural members provide the following advantages in building construction (Yu et al., 2019):

1. CFS members are efficient in terms of strength and stiffness.
2. Cold-formed light members can be manufactured for relatively light loads and/or short spans compared with thicker hot-rolled shapes.
3. Complex and uncommon sectional configurations can be produced economically by cold-forming operations, and consequently favorable strength-to-weight ratios can be obtained (Figure 1.1).
4. Interlocking sections can be produced, allowing for developing efficient structural applications in addition to simple transportation and erection solutions.
5. Load-carrying panels and decks can provide useful surfaces for floor, roof, and wall construction, and in other cases they can also provide enclosed cells for electrical and other conduits. Additionally, they can act as shear diaphragms to resist force in their own planes if they are adequately interconnected to each other and to supporting members besides supporting normal-to-surface loads.

CFS structural members provide the following qualities, compared with other materials such as concrete and timber, which lead to cost savings in construction if combined:

- Ability to provide long spans up to 12 m
- Lightness
- High strength and stiffness
- Off-site manufacture, ease of prefabrication and mass production
- Fast and easy erection and installation with lesser labor
- Easy and economical transportation and handling
- Highly precise detailing
- Substantial elimination of delays due to weather
- Non-shrinking and non-creeping at ambient temperatures
- High durability
- Fireproof
- Fully recyclable material and reusable without loss of quality

Although CFS sections have many advantageous features, their performance can be limited due to their susceptibility to local/distortional buckling as well as global buckling due to high width-to-thickness ratios. Therefore, their buckling resistance becomes lower and consequently they will lack ductility. As a result, their behaviour under seismic events can experience high vulnerability to collapse and thus their implementation in multi-storey buildings needs to be carefully investigated. The development of CFS stiffened sections with more folds and rolled-in stiffeners have been introduced as solution to delay the inherited local/distortional buckling and enhance the buckling resistance of CFS profiles (Davies, 2000). The evolution of CFS purlins profiles from typical simple lipped channels and Z sections into more complex sections with intermediate and edge lips located on the webs and flanges due to modern fabrication technologies can be illustrated as shown in (Figure 1.11).

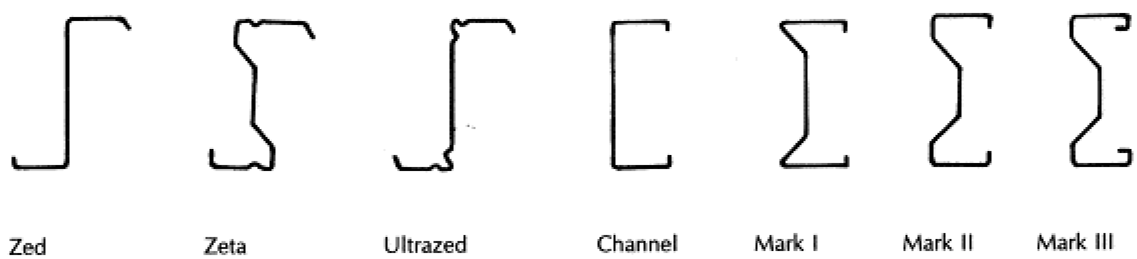


Figure 1.11 Evolution of cold-formed purlin sections (Davies, 2000).

One of the most major design considerations is Local Buckling of thin-walled element of CFS members where these elements experience buckling instability at stress levels less than the yield stress when subjected to compression, shear, bending, or bearing. Such elements may develop higher strength that allows them to carry increasing loads even if the buckling stress is reached and local buckling appears at first. This behaviour is due to post-buckling strength of compression elements (Yu et al., 2019). In general, other types of buckling that may CFS be subjected to are global, distortional and shear. Distortional buckling involves the distortion of the whole cross-section, and it is characterized by relative movement of fold-lines leading to in-plane and out-of-plane deformations of constituent plates while local buckling involves only flexural deformations of the individual plate elements (Dubina et al., 2013). The most prevalent in CFS as mentioned is the local buckling type. However, distortional buckling is having more importance as the sectional shapes have increasing complexity that might cause these two generic types of buckling to interact with each other as well as with global buckling (Davies, 2000).

1.3.3 CFS connections

Connections in CFS structures are used either to connect steel sheets to supporting structure (thin-to-thick), or for interconnecting two or more sheets (thin-to-thin), or for assembling bar members (thin-to-thin or thick-to-thick).

The most common types of CFS connections are welded, bolted, screw, and rivet connections. There are two classes for welds, arc welds and resistance welds. In welded connections arc welds are often used for erection, connecting CFS members to each other, or connecting CFS members to hot-rolled framing members while resistance welds (Figure 1.13) are mostly used for shop welding in CFS fabrication (Yu et al., 2019).

(Figure 1.12) illustrates the types of arc welds which are:

a) groove welds in butt joints
b) arc spot welds
c) arc seam welds
d) fillet welds
e) flare bevel groove weld
f) flare V-groove weld.

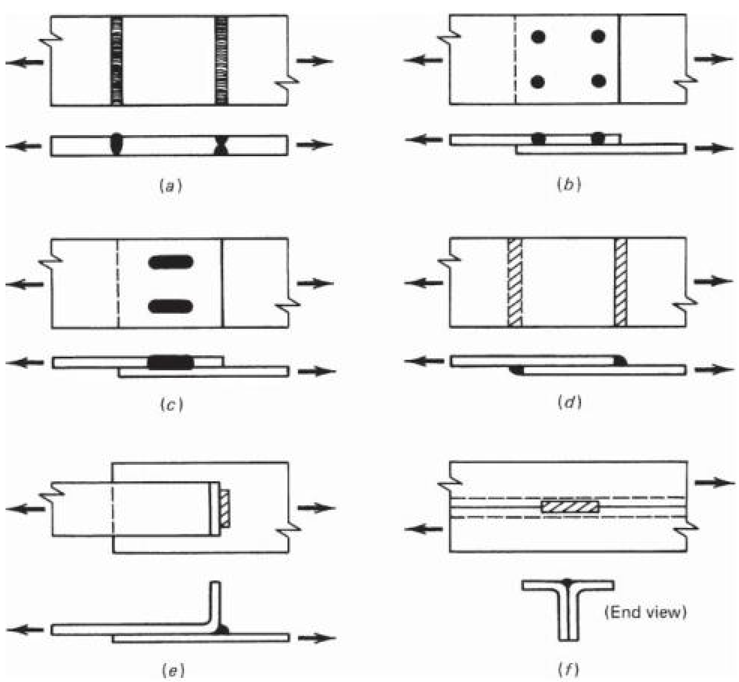


Figure 1.12 Types of arc welds(Yu et al., 2019).

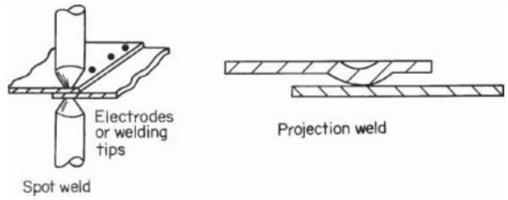


Figure 1.13 Resistance welds (Yu et al., 2019).

In bolted connections, the structural behaviour of CFS connections is influenced by the reduced stiffness of thin walls. However, it can be considered almost similar to the hot-rolled ones. For thin-walled sections bolt diameters are usually M5 to M16. The preferred property classes are (8.8) or (10.9). The basic types of failure for thin steel bolted connections in shear and tension can be considered as the following (Dubina et al., 2013):

Failure modes in shear (Figure 1.14):

a) Rupture due to shearing of the bolt
b) Crushing due to shearing of the bolt
c) Bearing and/or piling up of thin material in front of the bolt
d) Yielding of both sheets may occur together with bolt tilting when both materials are thin
e) Tearing of the sheet in the net sections
f) End failure by longitudinal shearing of thin sheet along two parallel lines

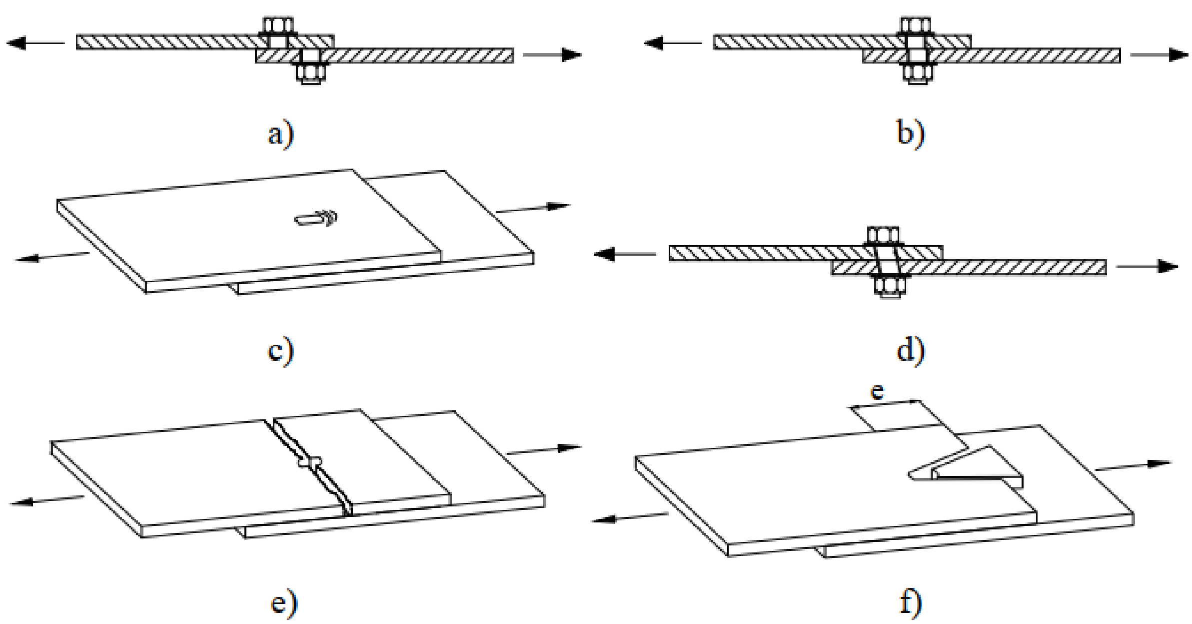


Figure 1.14 Failure modes of bolted connections in shear (Dubina et al., 2013).

Failure modes in tension (Figure 1.15):

a) Tension failure or rupture of bolt

b) Pull-through failure

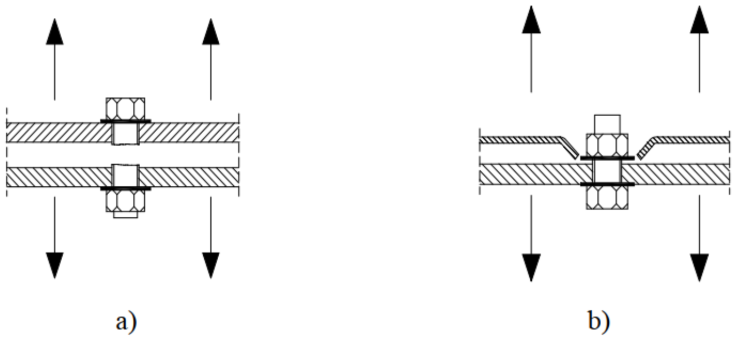


Figure 1.15 Failure modes for bolted connections in tension (Dubina et al., 2013).

In addition to welded and bolted connections screws are often used for CFS construction. The two main types of screws are self-tapping and self-drilling screws. They can provide effective and fast solutions when it comes to connecting roofing and siding sheet elements to framing members as well as siding and roofing connections. Their use is common in panelized construction for sheathing-studs or tracks connections(Yu et al., 2019). (Figure 1.16) shows some of self-tapping screws applications.

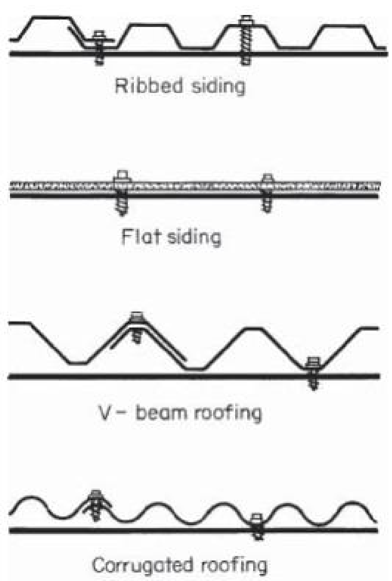


Figure 1.16 Application of self-tapping screws (Yu et al., 2019).

1.4 CFS framing applications

The range of use of CFS sections as load-bearing structural components is very wide, covering residential, office and industrial buildings, the automobile industry, shipbuilding, rail transport, the aircraft industry, highway engineering, agricultural and industry equipment, office equipment, chemical, mining, petroleum, nuclear and space industries.

The structural employment of CFS for residential and light commercial framing goes back to the mid-1990s in USA where there have been applications including wall, floor, and roof framing in a number of building types. CFS structural systems have a broad use in the construction industry due to the advantageous features of cold-formed sections. The common structural components for light-frame construction include wall studs, floor and ceiling joists, roof rafters, roof and floor trusses, decking, and panels. The primary application for CFS sections used to be as secondary structural members such as purlins and side rails (wall girts) supporting the cladding in industrial type buildings (Figure 1.17). In-site steel framing has become one of many popular applications where frames and panels for walls and roofs are assembled in site (Figure 1.18). CFS sections can be used to form wall panels that are prefabricated and then assembled on site. (Figure 1.19) shows this kind of system that is called wall stud system. They may be used as an alternative to timber joists in floors of modest span in domestic and small commercial buildings as well. They are also used as chord and web members of trusses (Figure 1.20), space frames (Figure 1.22), arches, and storage racks. Moreover, it is worth mentioning that CFS in the form of profiled decking (floor decking) as a basic component, along with concrete, in composite slabs (Figure 1.21) has gained widespread acceptance over the last two decades and this application can be considered at present prevalent in the multi-storey steel framed building market (Dubina et al., 2013).

The cold-formed structural steelwork industry is under increase in the developed world. This is due to the improving technology of manufacture and corrosion protection which leads, in turn, to higher competitiveness and quality of resulting end products as well as new applications. Recent advanced applications have involved the development of frames with bolted beam-to-column joists for industrial buildings such as portal frames (Figure 1.23).

Figure 1.17 CFS Purlins and Girts www.abtechsteelbuildings.co.uk

Figure 1.18 CFS in-site framing www.cssbi.ca

Figure 1.19 Wall stud housing system with prefabricated CFS wall panels (Dubina et al., 2013).

Figure 1.20 CFS Roof Truss (Dubina et al., 2013).

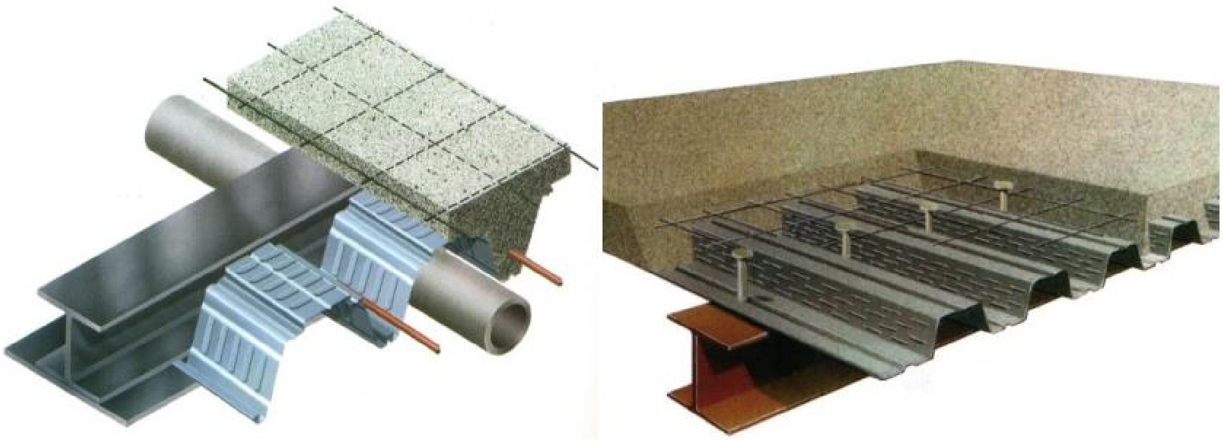


Figure 1.21 Composite steel concrete floors with sheeting and steel beams (Dubina et al., 2013)

Figure 1.22 CFS space frame www.en.wikipedia.org/wiki/File:WikiCFSbuilding.jpg

The beneficial properties of CFS promote their implementation to be suitable for fast modern methods of construction which include panelized and modular construction systems. In modular systems, three-dimensional housing unit segments are factory built (prefabricated), transported to the site, lifted into place, and fastened together while in panelized systems, flat wall, floor, and roof sections are prefabricated in a production system (see Figure 1.24), transported to the site, and assembled in place (Yu et al., 2019). Prefabricated modular units used for schools and offices are shown in (Figure 1.25) and (Figure 1.26).

Figure 1.23 roof portal frame made by built-up sections (Dubina et al., 2013).

Figure 1.24 Panelized construction – Spain (Dubina et al., 2013).

Figure 1.25 American School in Bucharest www.algeco.ro

Figure 1.26 Office building www.algeco.ro

In high-rise multistory buildings the main typical framing is of hot-rolled members while CFS members are used as secondary structural elements such as steel joists, studs, decks, or panels. In this case the heavy hot-rolled steel shapes and the CFS sections supplement each other. The employment of CFS as primary structural members is more popular in low- to mid-rise buildings that typically may range from 4 to 12 stories high. One of the major CFS applications is single-storey dwellings which can be made entirely from CFS sections.

CHAPTER 2. Literature review

This chapter addresses the most recent trends regarding CFS framing

2.1 Sustainability

Light steel framing and modular construction have recently demonstrated their significant impact on environmental, social, and economic level, hence they are in the line of sustainability requirements which leads to durable development of society. CFS section advantages such as flexibility, reusability as well as lightness guarantee economic performance besides being cost-effective in terms of construction and energy while from environmental point of view CFS framing reduces waste and construction-related carbon emissions by increasing reusable energy and materials usage. The world challenges call for a rapid implementation of modern methods of construction MMC as it helps cope with climate change and reduce consequences of natural and man-made ecological risks. In addition, it has long term impact on the national economic growth. Furthermore, it can increase EU construction industry competitiveness in the global market (Dubina et al., 2013).

Light steel framing and modular construction are favorable in terms of ecological performance and green buildings. One reason is that they perform dry construction which increases durability. Other attributes include the fact that steel is a closed material, CFS framing allows to possess 100% recyclable system with less energy consumption and waste. (Figure 2.1) illustrates the difference of CFS house and typical one in terms of environmental impact.

Green buildings can affect our lives significantly as our comfort, security, health, quality of life as well as productivity are highly related to built environment. Offsite CFS construction plays an Important role in meeting strategic plan goals In UK given an expectation of 33% reduction in the cost of construction, 50% reduction In construction time, and 50% reduction In greenhouse gas emissions in the built environment (Sustainability – www.SteelConstruction.Info).

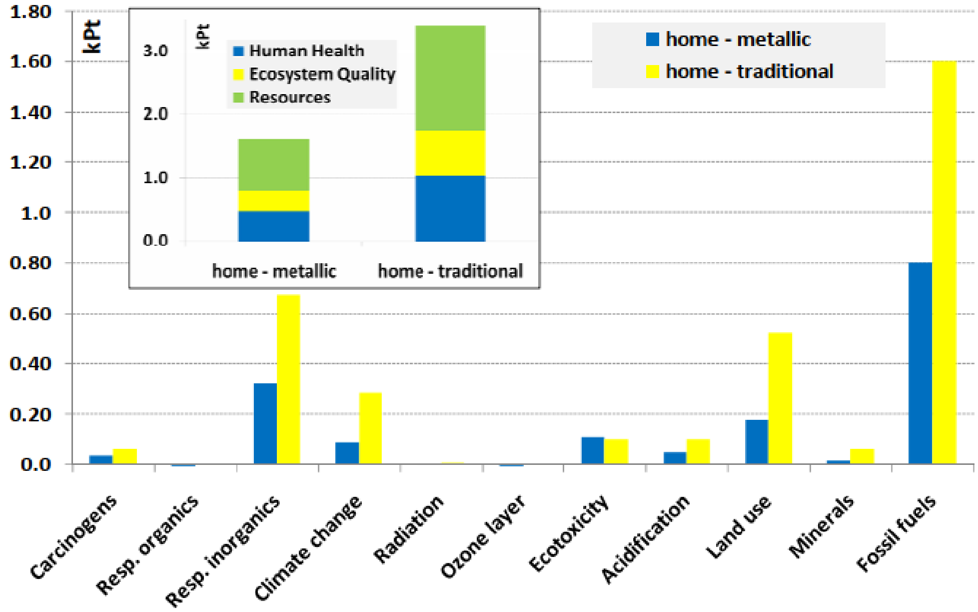


Figure 2.1 Comparison on environmental impact for light gauge steel framed house and traditional house (Dubina et al., 2013)

The "Foundation and primary stage school" BFS In Naples, Italy presents one of the major examples of cost-effective and eco-friendly structure that has been realized In Italy despite of construction site limitations. The employment of CFS structural framing systems provided fast solution with fully reusable possibility as well as high structural performance against seismic actions with reduced execution time (Figure 2.2).

Figure 2.2 The new school BFS in Naples (Fiorino et al., 2014)

This research thesis is mainly concerned with CFS bolted moment connections. Sustainable design of connections and joints is of great importance since they form a major part of design fabrication and erection which contributes to almost half of the total steelwork (Lu, 2016). Design for deconstruction (DfD) should be taken into consideration when it comes to sustainable design of connections and joints. This Is because they form essential role in life-ending of building systems in addition to their typical role as connectors in the production phase joining structural element in structural systems. With these considerations, the reusability of structures Is guarantees as long as DfD of connections and joints is taken into account. As a result, more job opportunities regarding specializing in DfD technologies, reducing waste due to reusable and recycled structures as well as cost reduction would be realized, hence, the importance of connections and joints design cannot be Ignored when need to meet sustainability targets.

2.2 CFS framing systems

2.2.1 CFS typical framing systems

In the most recent construction industry, light steel framing and modular systems are increasingly used where CFS profiles perform as principal structural elements other than the conventional use as secondary load-bearing members such as purlins and girts. The prevalent structural system adopted is composed of shear walls made of vertical studs, diagonal braces, and top and bottom tracks. Wall panels are not only used as cladding and to support vertical loads but most importantly to resist horizontal loads such as wind and seismic loads as well as to prevent lateral buckling of beams and overall buckling of columns. CFS wall panels structural performance depends on panels sectional configurations, type and arrangement of connections, material properties, geometry, loading and material fill such as concrete. Thus, numerical and experimental testing has been of high importance recently to capture the complex behavior of panels in terms of stiffness and strength (Dubina et al., 2013). Recent studies have included experimental and numerical tests on many types of lateral force-resisting framing systems to investigate their behavior under seismic actions such as:

- CFS strap-braced walls (Figure 2.3)
- CFS K-braced shear wall (Figure 2.4)
- CFS wall frames sheathed with CSB under monotonic and cyclic shear loads
- High-strength lightweight foamed concrete-filled CFS shear walls

CFS shear panels braced with many types of strap braces have been tested experimentally to evaluate the vertical and lateral performance under cyclic loads (Moghimi & Ronagh, 2009).

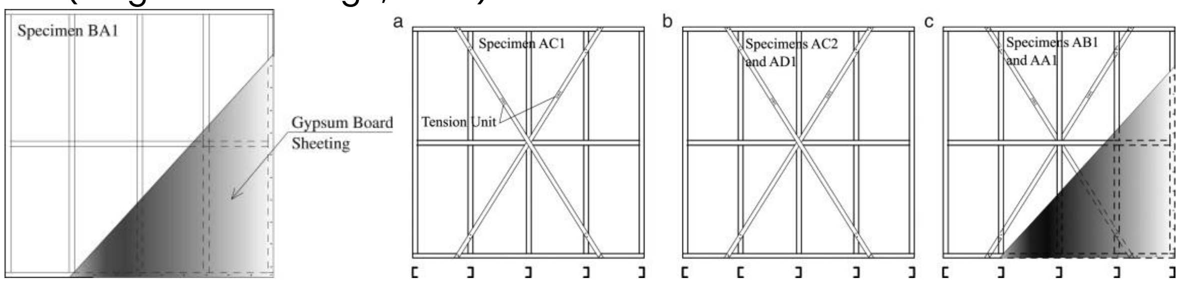


Figure 2.3 Some of CFS strap-braced stud walls test specimens (Moghimi & Ronagh, 2009)

In addition, CFS K-braced shear wall system has been tested experimentally to investigate Its seismic behaviour and results indicate their suitability for low-rise buildings in low seismic zones (Zeynalian et al., 2012).

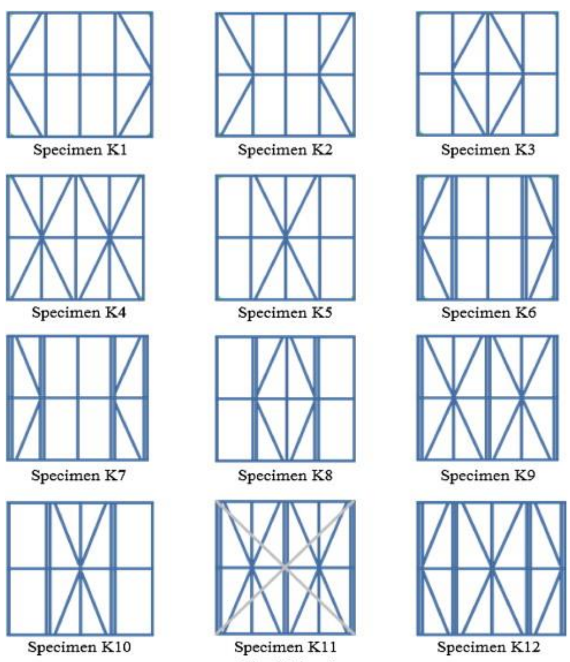


Figure 2.4 CFS K-braced shear walls specimens (Zeynalian et al., 2012).

Further experimental studies have been conducted on various structural systems including CFS shear wall sheathed with calcium silicate board (Lin et al., 2014) as well as CFS high-strength lightweight foamed concrete (CSHLFC) (Xu et al., 2018).

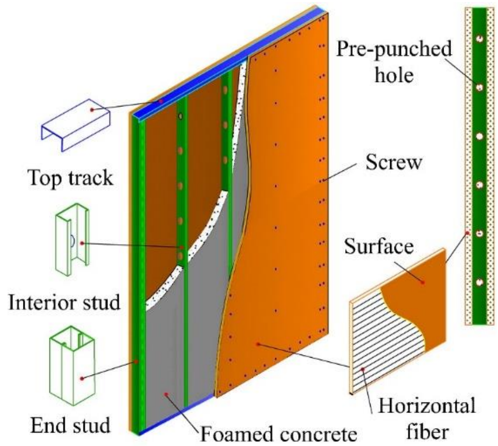


Figure 2.5 CSHLFC Detail (Xu et al., 2018).

The employment of another shear wall system stick-built system made of CFS walls braced by wood-based sheathing panels has been realized in Naples, Italy (Fiorino et al., 2014) (Figure 2.2). Two types of structural systems have been adopted, stick built and framed. Stick built systems are load-bearing systems composed of floors and walls made of CFS and oriented strand board (OSB) panels as illustrated in (Figure 2.6) and (Figure 2.7).

Figure 2.6 Stick-built Floors (Fiorino et al., 2014)

Figure 2.7 Stick-built Walls(Fiorino et al., 2014)

While framed systems are mainly lateral resisting systems where moment resisting frames consisting of portal frames and concentric bracing frames consisting of X-bracings have been used as shown in (Figure 2.8).

Figure 2.8 Framed systems (Fiorino et al., 2014)

The study presents an actual example of fast dry construction solution that meets structural seismic safety design and sustainability requirements.

2.2.2 CFS Moment resisting frames

Many research papers during the last decade started to address some new configurations for CFS bolted moment resisting connections to improve their seismic behavior. The most common CFS connection is beam-column with through plate connection which has been tested experimentally with several stiffeners' arrangements (Bagheri Sabbagh et al., 2012b). The connection experimental setup, geometry and configurations are illustrated in (Figure 2.9) and (Figure 2.10).

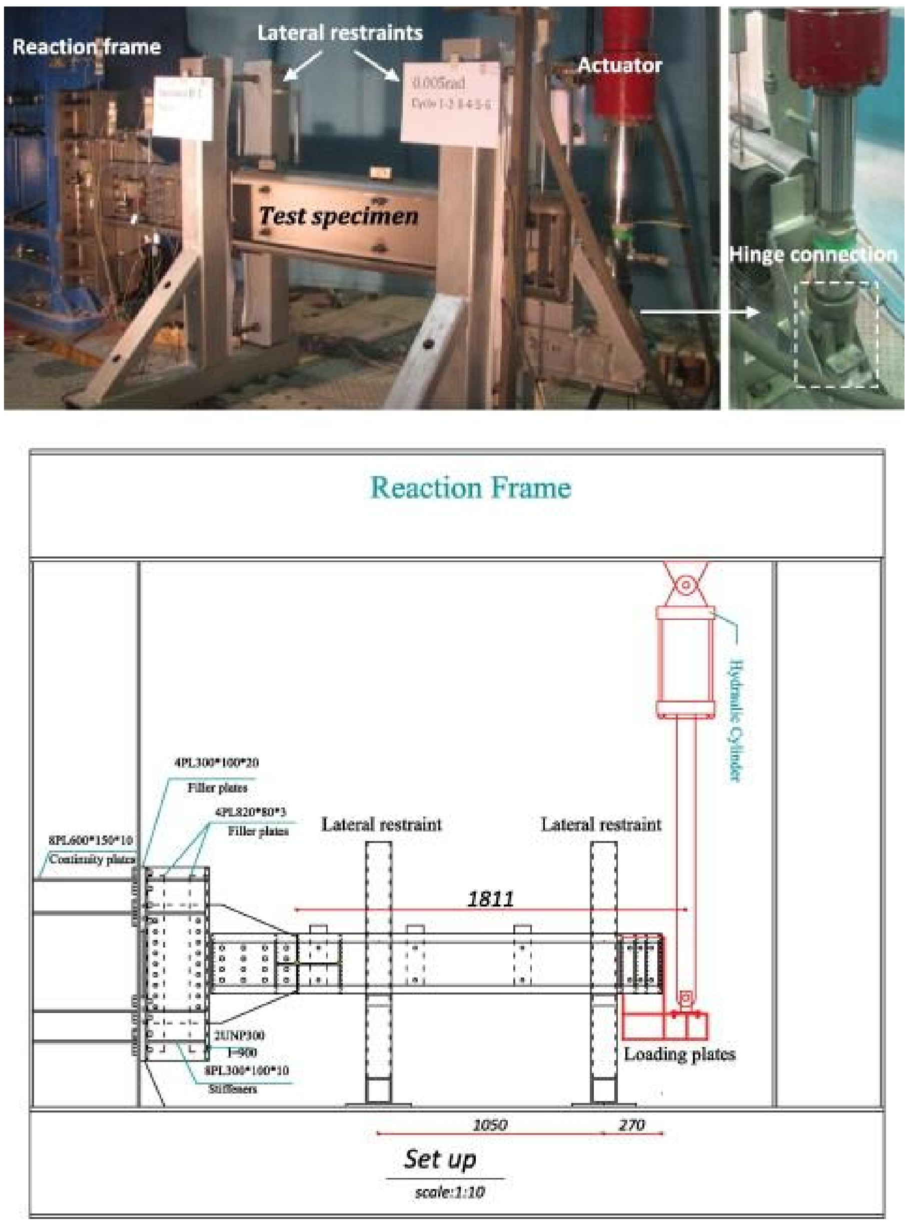


Figure 2.9 Experimental test set up (Bagheri Sabbagh et al., 2012b).

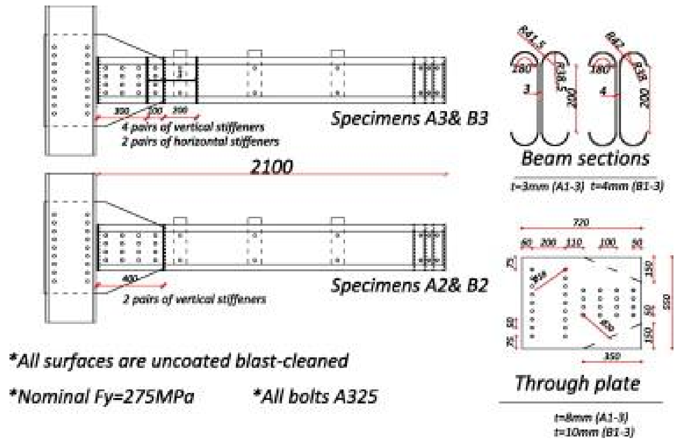


Figure 2.10 CFS Connection configurations (Bagheri Sabbagh et al., 2012b)

This study has shown that CFS curved flange beams cannot only exceed the nominal plastic moment capacity (Mp), but also sustain this capacity at large rotations thus it has demonstrated the ability of back-to-back channels with curved flanges to achieve high stiffness and ductility. However, still the use of curved flanges cannot be considered as a practical solution due to construction and manufacture limitations as these sections are hard to be fabricated, not suitable for connection with floor system and difficult to be welded to stiffeners on larger scale productions.

Further cyclic analytical studies have been performed to examine the performance of CFS beam with curved flanges and compare it with the typical flat ones (Figure 2.12) (Bagheri Sabbagh et al., 2011). The results indicated that the connection with the beam can develop a ductile behavior unlike the conventional CFS framing systems.

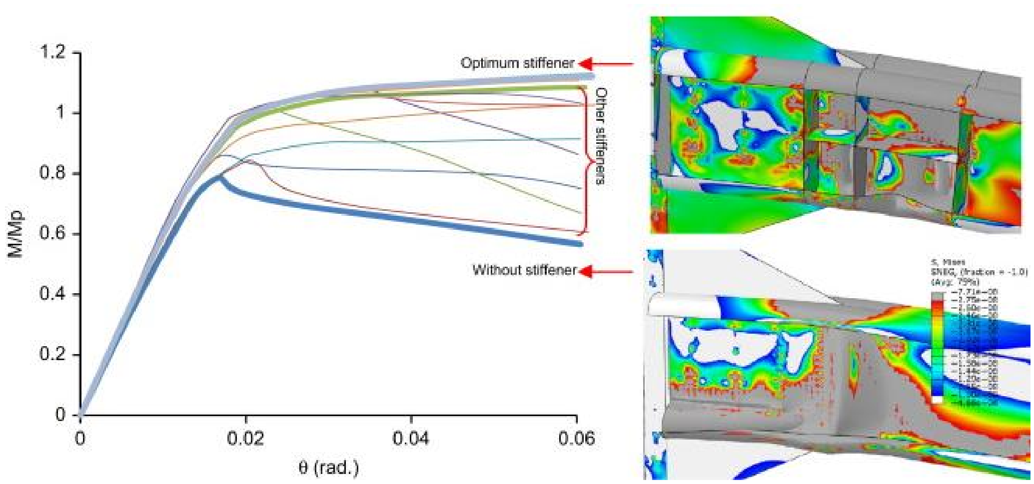


Figure 2.11 M–ϑ curves and local buckling deformation of beam without stiffener and beam with optimum stiffener (Bagheri Sabbagh et al., 2012a)

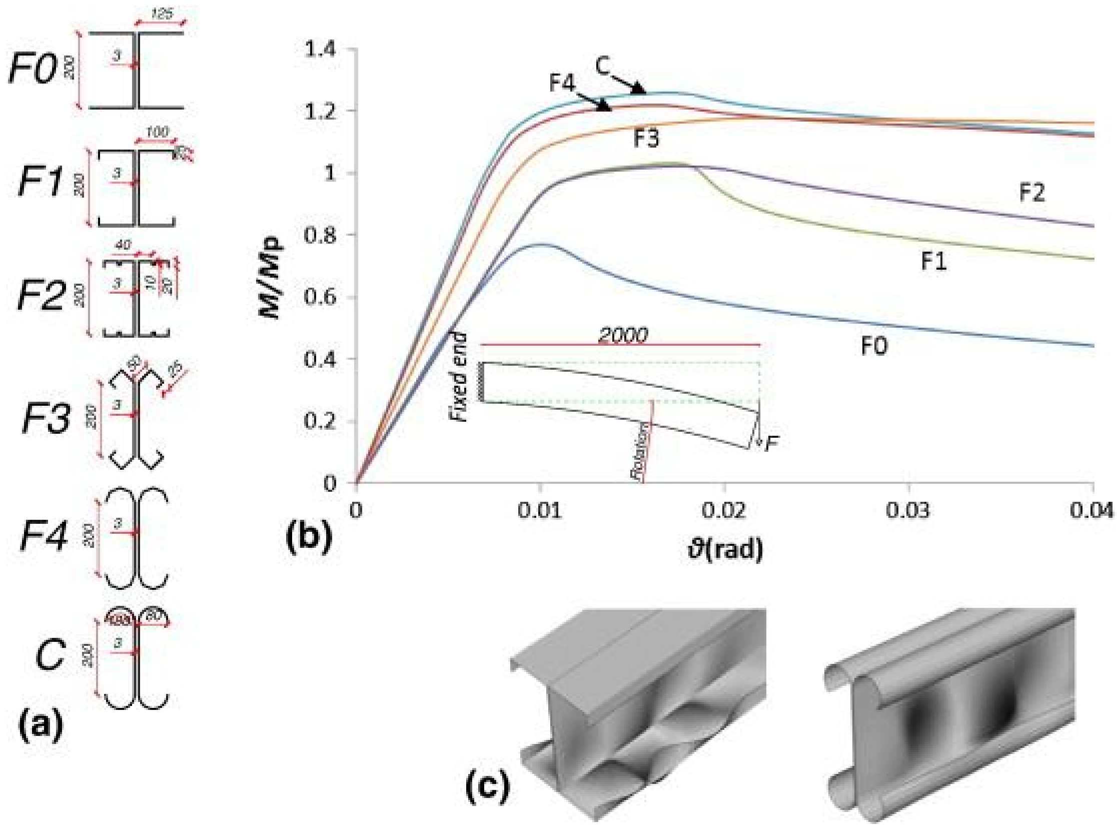


Figure 2.12 Comparison of beams with different cross sections: (a) section shapes, (b) moment–rotation diagrams for fixed end beams with cross sections F0 to F4 and C, and (c) predominant buckling modes of sections F1 and C (Bagheri Sabbagh et al., 2013)

Additionally, advanced analytical models which have been validated against the previous experimental test have undertaken different connections configurations (Figure 2.11) showing how stiffeners can play a significant role in the connection region where the web and flange buckling have been delayed providing higher strength (Bagheri Sabbagh et al., 2012a). The numerical study has been investigated carefully to better demonstrate the reliable results and prove their correspondence with the ones in experimental tests by the simulation of bolts slip and the geometrical imperfections (Bagheri Sabbagh et al., 2013). Moreover, the behavior of CFS bolted connections for moment resisting frames under seismic actions have been studied taking into account the effect of beam cross-section (Figure 2.12), bolt group arrangement and the bolts mechanism of friction-slip (Ye et al., 2019). A comparison between numerical and experimental tests has been presented as shown in (Figure 2.13).

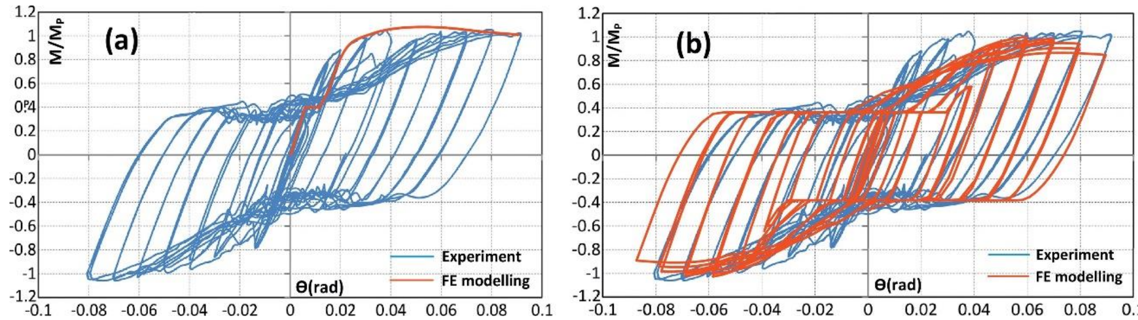


Figure 2.13 Comparison between experimental (tested by (Bagheri Sabbagh et al., 2012b) *and FE moment-rotation results under. (a) monotonic load and (b) cyclic load* (Ye et al., 2019).

Significant increase in the ductility and energy dissipation of the connection has been observed due to the bolts slip-friction mechanism which enhances this type of connection to be a suitable choice in seismic zones (Figure 2.14).

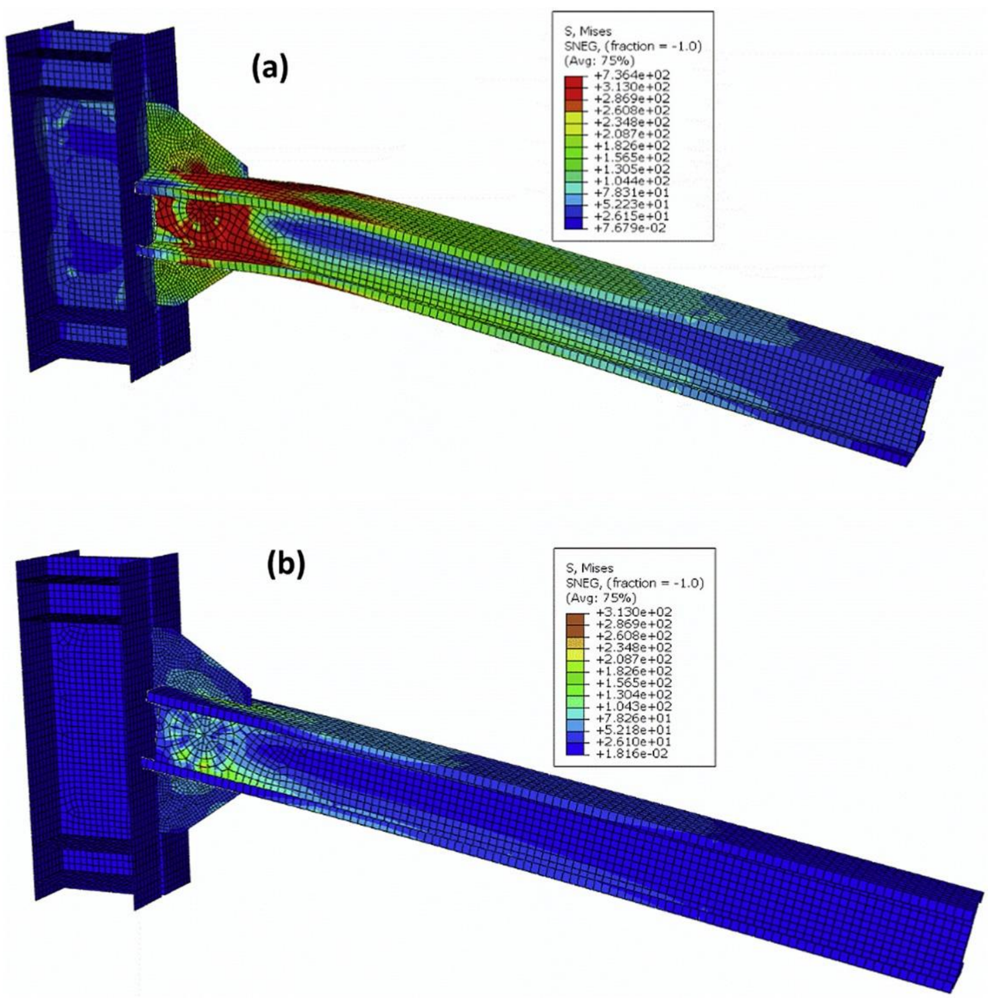


Figure 2.14 Von-Mises stress distribution and corresponding damage in the (a) normal, and (b) mobilized friction-slip connections with flat-flange beam section and circular bolt configuration (Ye et al., 2019).

The key design parameters which influence the connection seismic response as have been illustrated in previous studies so far are the following:

- Bolts distribution and bolt tightening
- Material yielding and bearing around the bolt holes
- Shape and dimensions of the beam cross section (Figure 2.15)
- Stiffeners arrangements
- Beam thickness and the gusset plate shape and thickness

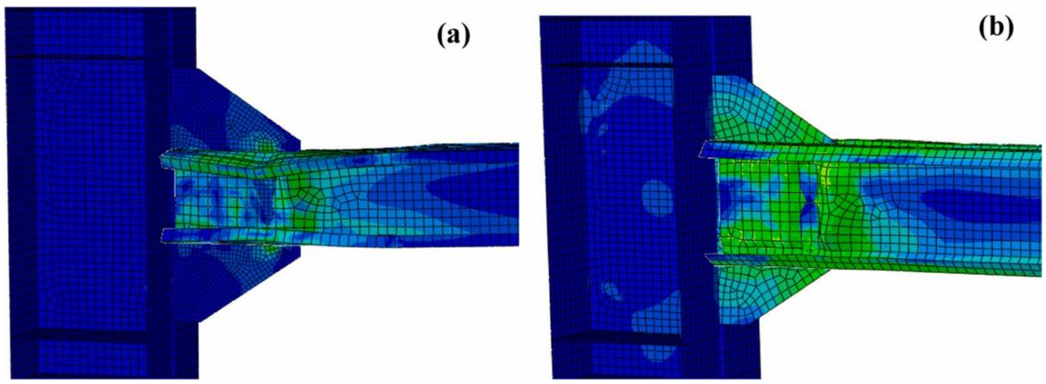


Figure 2.15 Typical failure modes:
(a) flat flange channel, (b) bent flange channel (Ye et al., 2020)

A parametric study to provide more advanced seismic design solutions have been addressed (Ye et al., 2020). More advancements have been added to the design parameters to allow the structural promotion of CFS connection with gusset plate. The folded flange beam has shown similar behavior to the curved flange one. However, it respects the construction limitations compared to the latter. Other aspects have been tested analytically combined introducing the best design configurations that respect the structural requirements in seismic regions. The most recent research study has provided several additional connections configurations considering the shape of the gusset plate, the connection type between beam and column whether the web or/and the flange are engaged as part of the joint and the thicknesses of both the beam and gusset plate (Papargyriou et al., 2022) (Figure 2.16). The behavior of the previous connection patterns has been studied to find the most adequate choice for earthquake resisting frames.

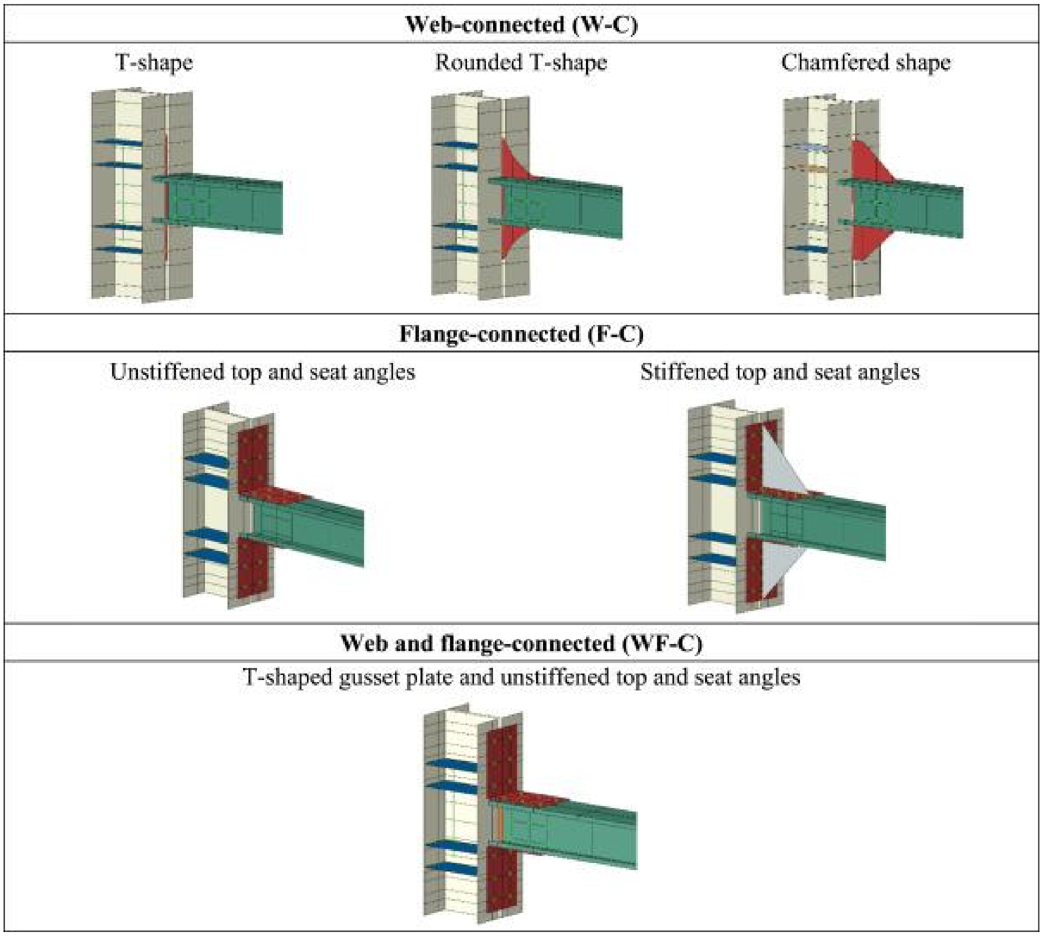


Figure 2.16 Various connection types (Papargyriou et al., 2022)

Although previous connection types and configurations might exhibit adequate behaviour under seismic actions. However, they may not be suitable for real construction practice and more experimental tests are needed to understand the behaviour of such connections. Moreover, there is lack of standards and guidelines with regard to CFS moment connections thus their implementation at present is questionable. There are many challenges that restrict the promotion of this kind of connection in industry. One is that the size of gusset plate needs to be reduced in a way that facilitates construction process. Additionally, the number of bolts is high and needs to be reduced. Furthermore, even though stiffeners have significant impact to the connection performance, still their installation process will cost and become more complicated for big projects where high number of connections is needed.

CHAPTER 3. Numerical modelling of CFS bolted MR connections

Finite Element Model of Cold-formed steel bolted beam-to-column with through plate (gusset plate) connection has been created using Finite Element Software ABAQUS following previous experimental tests carried out by (Bagheri Sabbagh et al., 2012b). In the following figure (Figure 3.1 and Figure 3.2), General 3D view, side and front view can be shown:

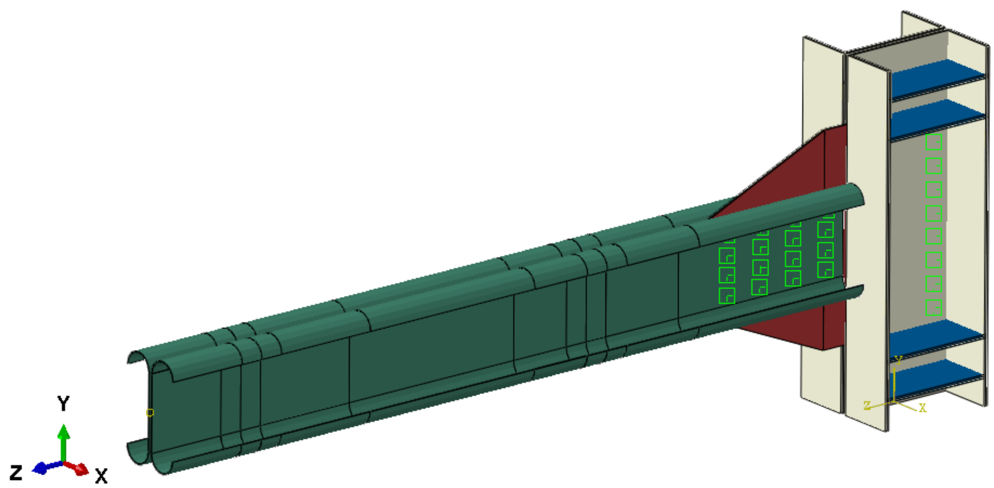


Figure 3.1 General 3D View of CFS Bolted beam-to-column connection

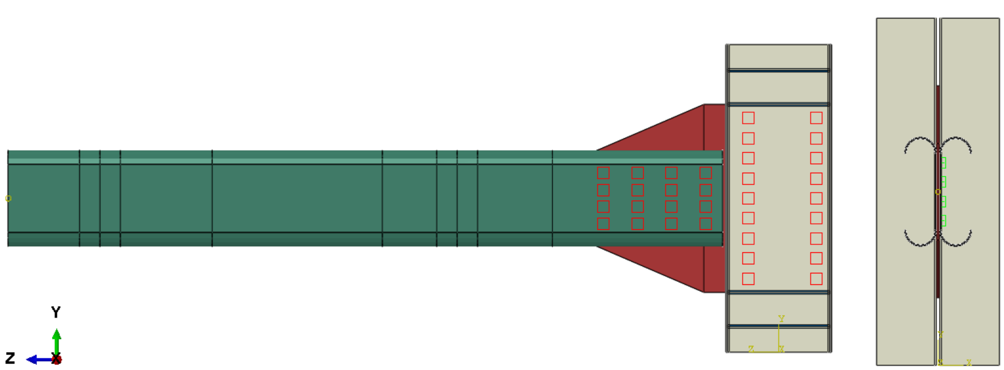


Figure 3.2 Side and front view of CFS Bolted beam-to-column connection

The simulation of the connection in terms of geometry, material, and boundary conditions has been performed to be in correspondence with research experimental test. However, loading conditions are different as the numerical test is monotonic while the experimental test was performed under cyclic loading. In addition to that, FE models have also been considered in previous studies to simulate and study the behaviour of the connection with respect to the experimental test.

The main parameters that have to be incorporated into ABAQUS to obtain a coherent model and simulate the actual behaviour are the following:

1. Geometry
2. Material
3. Bolts
4. Boundary conditions
5. Loading conditions
6. Mesh and Elements type
7. Analysis type

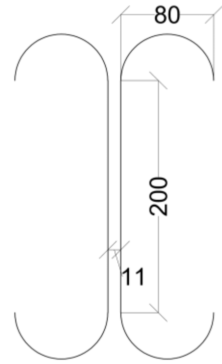
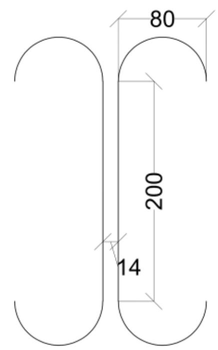
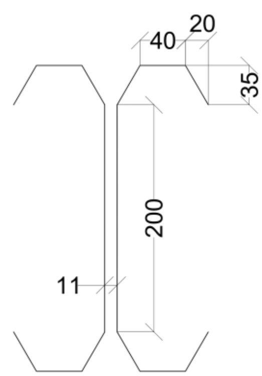
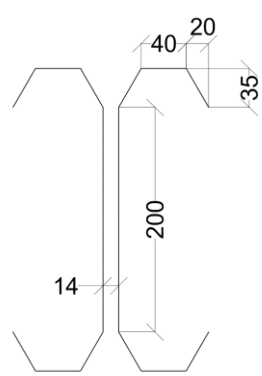
3.1 Geometry

The details of overall geometry of the experimental beam-to-column dimensions have been introduced in (Figure 2.10). Beams with curved flanges and a gusset plate (similar to experimental tests) with two different thicknesses denoted A and B have been tested. 3 Specimens for each thickness (denoted from 1 to 3) with different stiffeners arrangements have been addressed. In our studies, additional similar specimens of beam with folded flanges denoted F and G have been modelled. shows all details of specimens under study.

Table 1 Details of specimens for numerical modelling

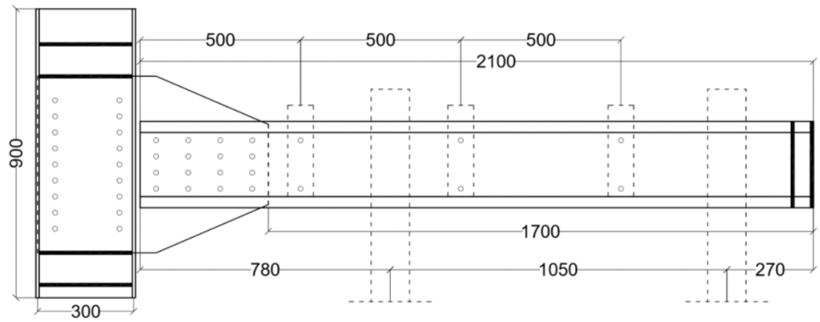
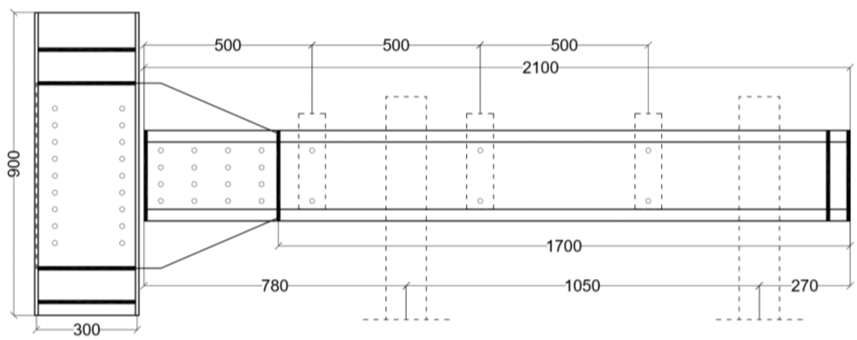
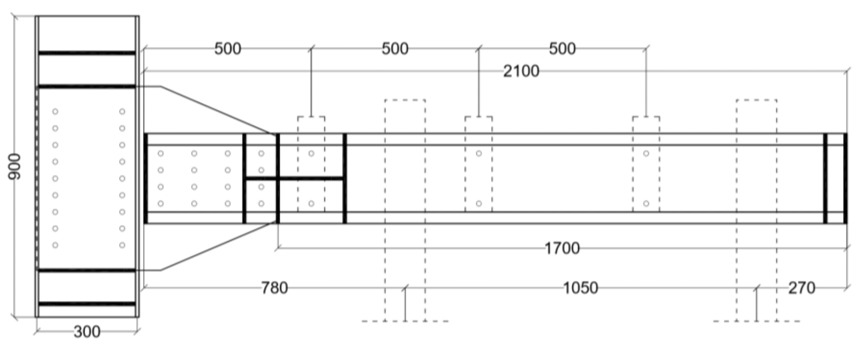
Specimens	*Beam Section [-]*	*Beam Thickness [mm]*	*Gusset Pl. Thickness [mm]*	*Beam Stiffeners Thickness [mm]*
A1-3	Curved	3	8	8
B1-3	Curved	4	10	10
F1-3	Folded	3	8	8
G1-3	Folded	4	10	10

Table 2 Beam dimensions for all specimens

Specimens	*Beam Section Dimensions in [mm]*
A1-3	
B1-3	
F1-3	
G1-3	

Beam dimensions for all specimens are shown in Table 2. The gap space between the beams in transversal direction depends on the thickness of beam as well as through plate.

Table 3 Dimensions of CFS beam-to-column connection specimens

STIFFENERS CONFIGURATIONS	SIDE VIEW OF BEAM-TO-COLUMN CONNECTION SPECIMENS ALL DIMENSIONS IN [mm]
1	
2	
3	

Since there are 3 different stiffeners arrangements considered in the previous experimental test (1 for no stiffeners, 2 for minimum stiffeners (partial) ,3 for Optimum stiffeners arrangement(full)) (Bagheri Sabbagh et al., 2012b), this study considers the same arrangements for comparison purposes as shown in Table 3.

Two lateral brace frames were used to avoid premature global instability spaced of 1.05 m as shown in connection configurations Table 3. The beam webs were connected at 3 different locations with spacing of 0.5 m to form a built-up section. Figure 3.3 shows through plate dimensions with bolt group arrangements. As illustrated in Table 1, beam stiffeners have the same thickness of gusset plate.

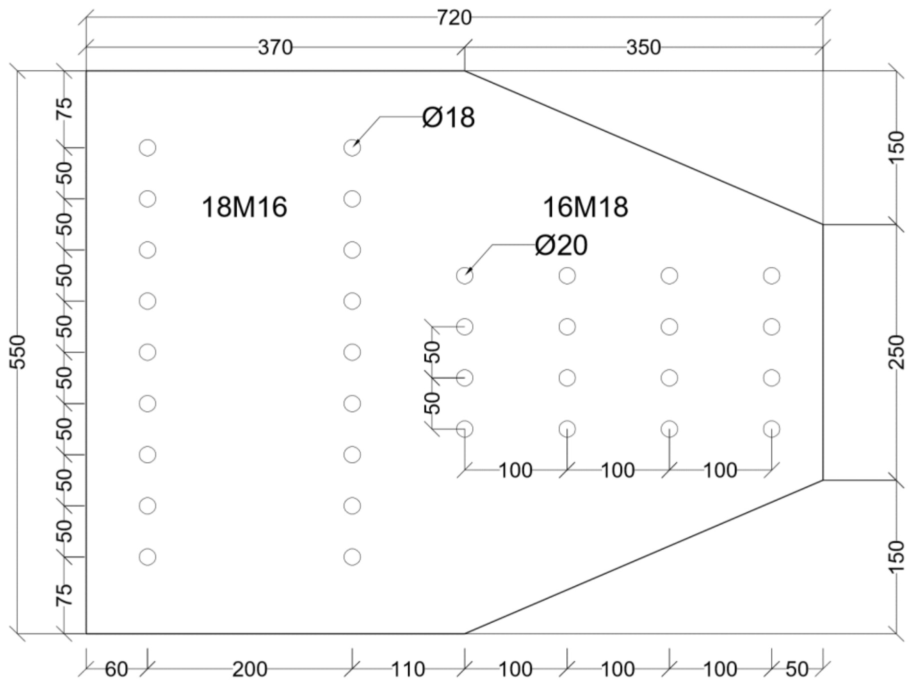


Figure 3.3 Gusset plate dimensions with bolt group arrangements - Dimensions in [mm]

3.2 Material

Nonlinear elastic-plastic stress-strain material model suggested by (Haidarali & Nethercot, 2011) is used to simulate the material behaviour of CFS beam and gusset plate (Papargyriou et al., 2022).

The mathematical expression of the constitutive model adopted is given in Equation 1.

Equation 1

$$\varepsilon = \frac{\sigma}{E} + 0.002\left(\frac{\sigma}{\sigma_{0.2}}\right)^n \qquad for\ \sigma \leq \sigma_{0.2}$$

$$\varepsilon = \varepsilon_{0.2} + \frac{100(\sigma - \sigma_{0.2})}{E} \qquad for\ \sigma \geq \sigma_{0.2}$$

where E is the elastic modulus (taken as 210 GPa), $\sigma_{0.2}$ is the 0.2% proof stress, $\varepsilon_{0.2}$ is the strain corresponding to $\sigma_{0.2}$ and n is a constant parameter used to determine the roundness of the stress-strain curve and is taken as 10 to have best agreement with coupon test reference results which carried out by (Bagheri Sabbagh et al., 2012b). In the second phase of stress-strain curve, the slope is given as $E_0 = \frac{E}{100}$. The material properties for both beam and gusset plate specimens A and B are given in the following Table 4.

Table 4 Material properties for beam and gusset plate from coupon test - specimens A & B

Specimen	*Beam A*	*Gusset A*	*Beam B*	*Gusset B*
$\sigma_{0.2}$ [MPa]	313	353	322	308
$\varepsilon_{0.2}$ [-]	0.00349	0.00368	0.00353	0.00347
E [MPa]	210000	210000	210000	210000
E_0 [MPa]	2100	2100	2100	2100

The stress-strain curves that have been used in FE model by ABAQUS are illustrated in the figures below:

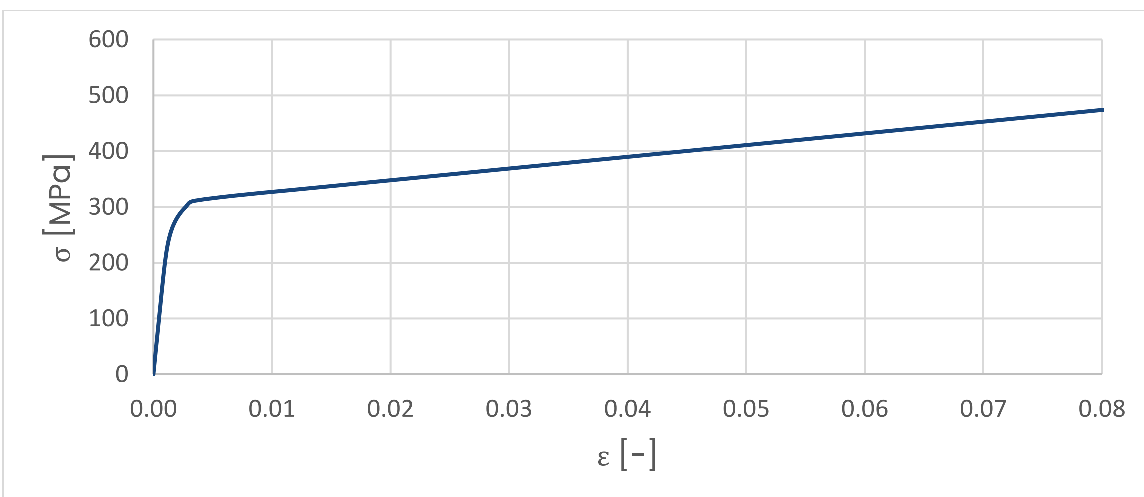


Figure 3.4 Stress-Strain curve for beam of specimen A

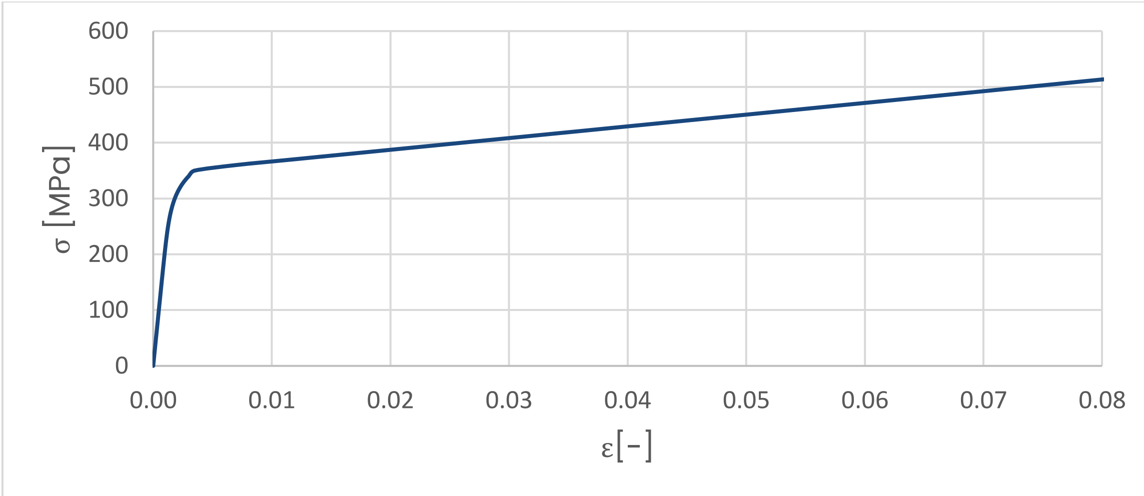


Figure 3.5 Stress-Strain curve for gusset plate of Specimen A

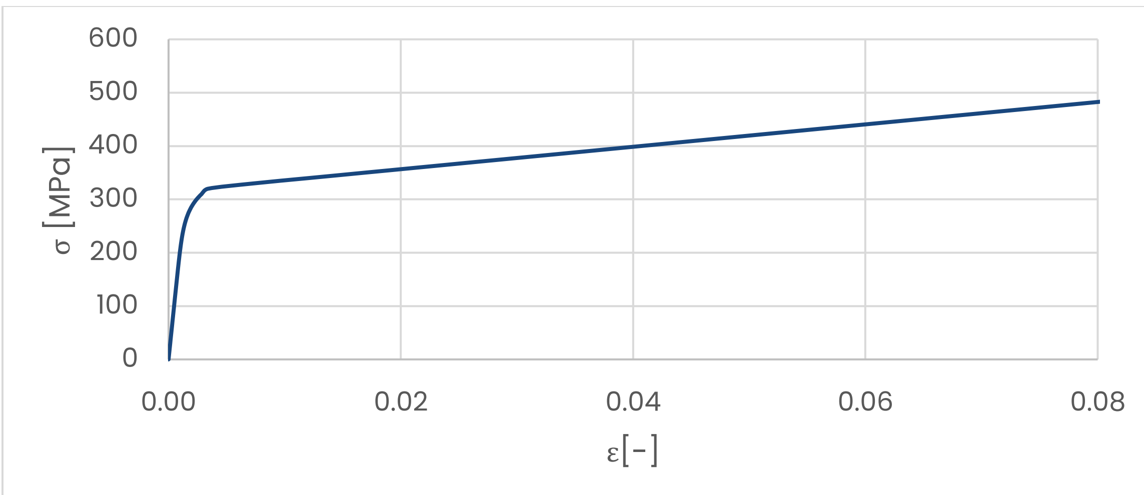


Figure 3.6 Stress-Strain curve for beam of Specimen B

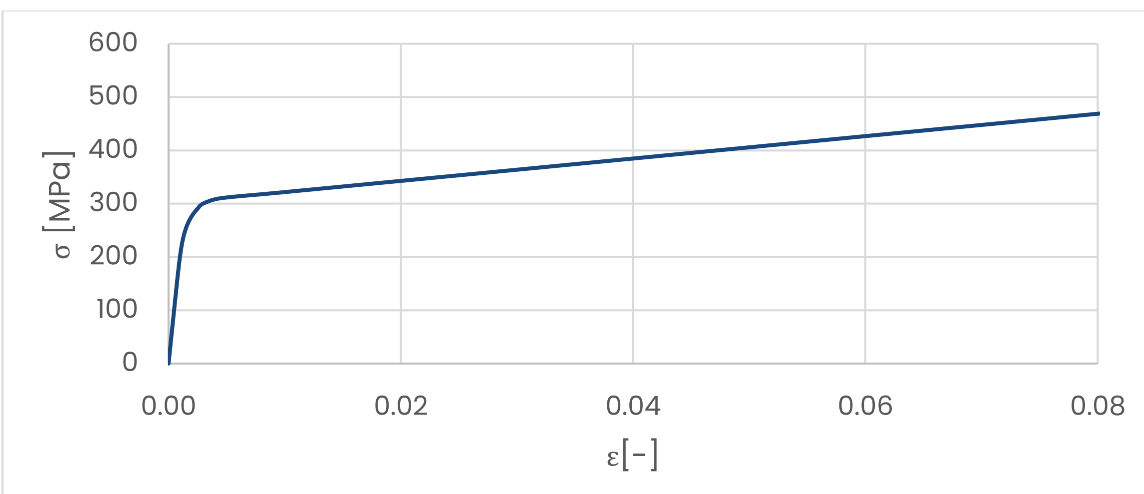


Figure 3.7 Stress-Strain curve for gusset plate of Specimen B

3.3 Bolts

Bolts arrangements details are given in (Figure 3.3). The bolt behaviour under monotonic loading was simulated by employing *point-based fasteners* with simplified connection element. This will lead to nearly accurate resaults considering lower computational time and effort compared to more complex solutions.

The modelling technique of point-based fasteners needs to identify surface layers which are beam web\column surface-gusset plate surface-beam web\column surface as illustrated in the following (Figure 3.8)and (Figure 3.9).

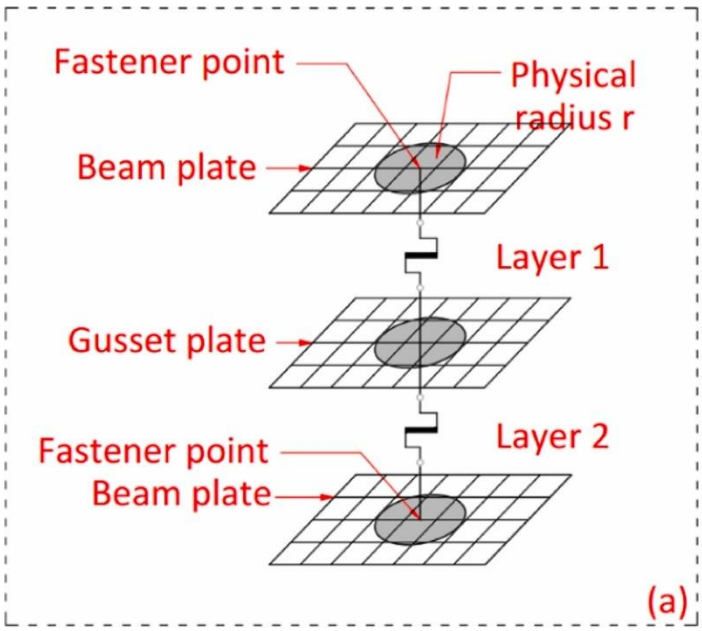


Figure 3.8 Single bolt modelling in ABAQUS: definition of fastener (Ye et al., 2020).

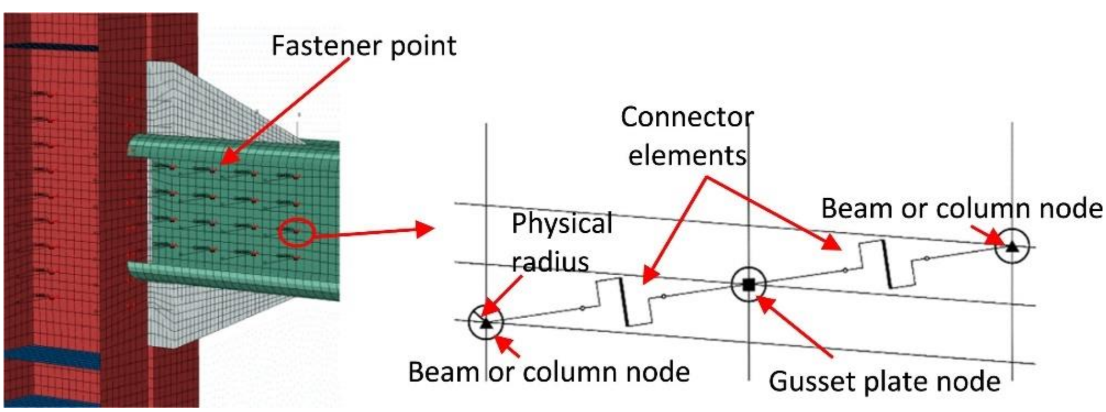


Figure 3.9 FE model of the beam-column connection with fastener definition (Ye et al., 2020).

According to (*Abaqus CAE User's Manual*, 2007), fasteners are used to model a point-to-point connection between two or more faces, such as a spot weld,

a bolt, or a rivet. Point-based fasteners make use of positioning points to create mesh-independent fasteners. Attachment points at equally spaced intervals around the edge of the bracket must be created at first used to define the location of the fastener's positioning points. A point-based fastener can connect selected faces with either connectors or rigid (beam) multi-point constraints. In this study a rigid connection is modelled by implementing rigid multi-point constraints. Rigid multi-point constraints are computationally cheaper than connectors and are less likely to result in an over constrained model when two adjacent fasteners are sharing nodes. When Abaqus detects two adjacent fasteners that are sharing nodes and using rigid multi-point constraints, it uses a penalty distributing coupling formulation that relaxes, to a small degree, the constraint between the motion of the fastening point and its coupling nodes to avoid the over constraint. A point-based fastener uses distributing coupling constraints to connect the faces regardless of how you mesh the faces. Figure 3.10 illustrates the concept of modelling point-based fasteners.

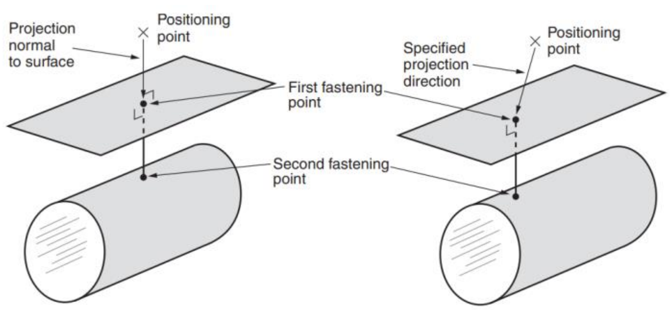


*Figure 3.10 Creating point-based fasteners (*Abaqus CAE User's Manual*, 2007)*

To model connector element a "physical radius" r is defined to represent the bolt shank radius and simulate the interaction between the bolt and the nodes at the bolt hole perimeter. Each fastener point is connected to the CFS steel plates using a connector element that couples the displacement and rotation of each fastener point to the average displacement and rotation of the nearby nodes. Hence, rigid behaviour is assigned to the local coordinate system corresponding to the shear deformation of the bolts (Ye et al., 2020). Since no failure of bolts is observed in the reference experimental tests, the failure modes of bolts are not taken into account.

3.4 Boundary and loading conditions

The boundary conditions have been introduced to the model with respect to the reference experimental test conditions (Bagheri Sabbagh et al., 2012b). For this purpose, the following conditions have been considered taking into account some reference FE models conducted previously (Ye et al., 2019), (Ye et al., 2020), and (Papargyriou et al., 2022).

- The back-to-back channel column is connected to the base by using pinned support (Ux,Uy,Uz = 0), the column base faces were coupled to the reference point RP located at the centroid of the cross-section where the boundary condition is applied.
- The translational degrees of freedom at the top face of the column are restrained (Ux,Uy = 0), the column top faces were coupled to the reference point RP located at the centroid of the cross-section where the boundary condition is applied.
- Since the back-to-back beam was assembled with bolts and filler plates in the experimental tests, the web lines are connected together in the UX, UY and UZ direction using the "Tie" constraint in ABAQUS.
- Lateral bracing in the X direction (out of plane deformation direction of the beam) is imposed at the locations of lateral frames used in the experiments to prevent lateral torsional buckling and global instability of the beam element.
- A tip displacement corresponding to rotation of the connection is applied at the Reference Point (RP) on the cross-sectional centroid of the beam end section where all degrees of freedom of the beam end section are coupled.
- The column stiffeners (used in the experiments to ensure the column remains elastic) are tied to the column surfaces using tie constraint in ABAQUS.

Figure 3.11 shows the boundary conditions adopted in the FE model according to (Papargyriou et al., 2022) while Figure 3.12 shows the boundary conditions of the FE model under study. Monotonic loading has been introduced by applying a controlled displacement at the end of the beam of 200 mm maximum while the type of analysis performed is general static analysis.

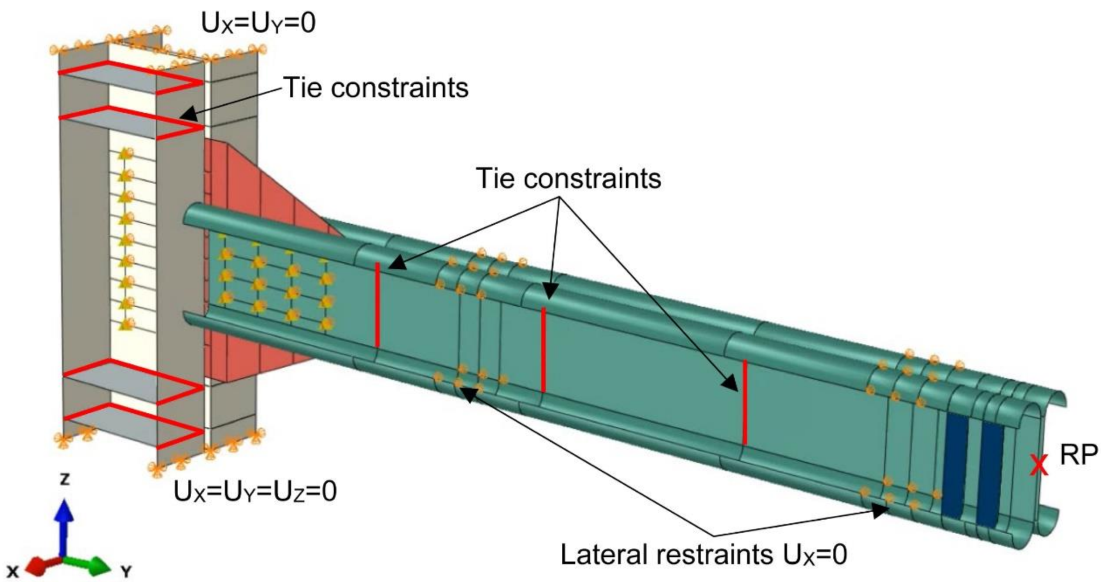


Figure 3.11 Boundary conditions of the FE model (Papargyriou et al., 2022)

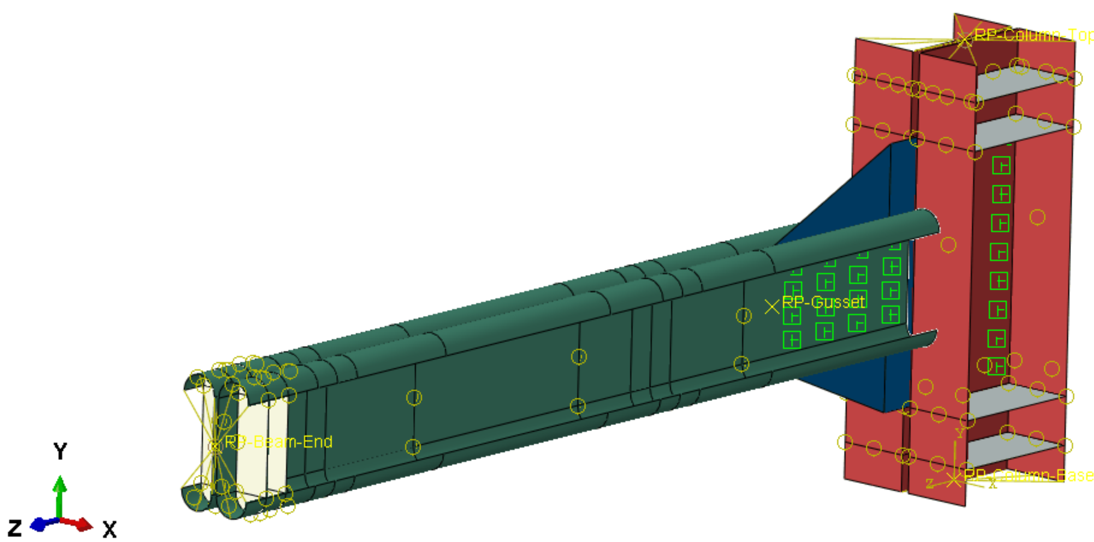


Figure 3.12 Boundary conditions of CFS beam-to-column connection

3.5 Element type and Mesh

The S4R general-purpose finite element available in ABAQUS was employed to model all connection components, it can accurately in capture the behaviour of CFS elements and connections (Papargyriou et al., 2022). This element type can account for nonlinear material properties and finite membrane strains and features reduced integration. A mesh size of 20×20 mm was selected to guarantee adequate numerical accuracy while keeping the computational time within acceptable limits.

CHAPTER 4. Results and numerical investigations of CFS bolted MR connection

4.1 Failure modes and Moment-Rotation curves

4.1.1 Specimen A1

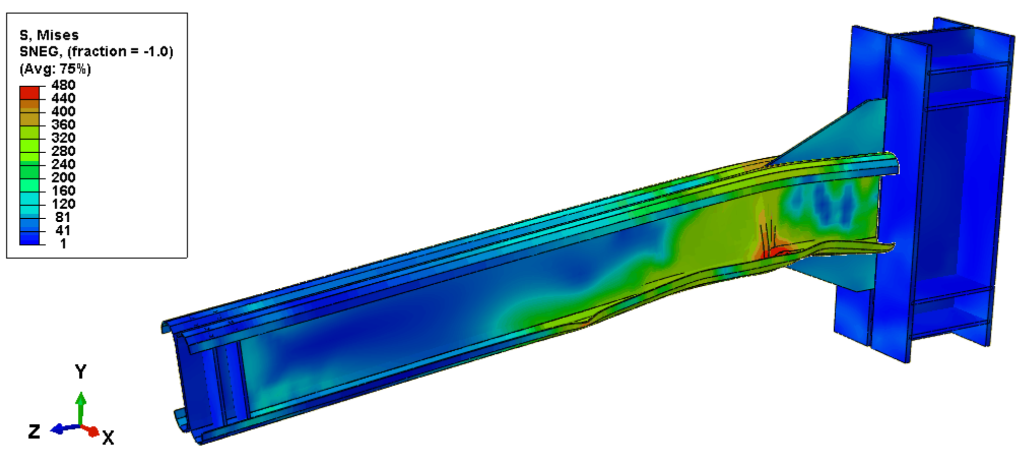


Figure 4.1 Failure mode of specimen A1

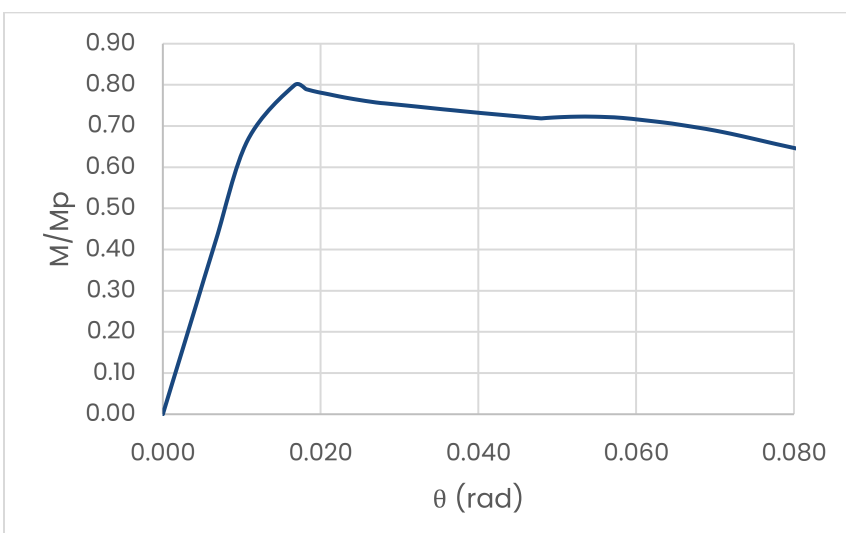


Figure 4.2 Moment-Rotation Curve of specimen A1

4.1.2 Specimen A2

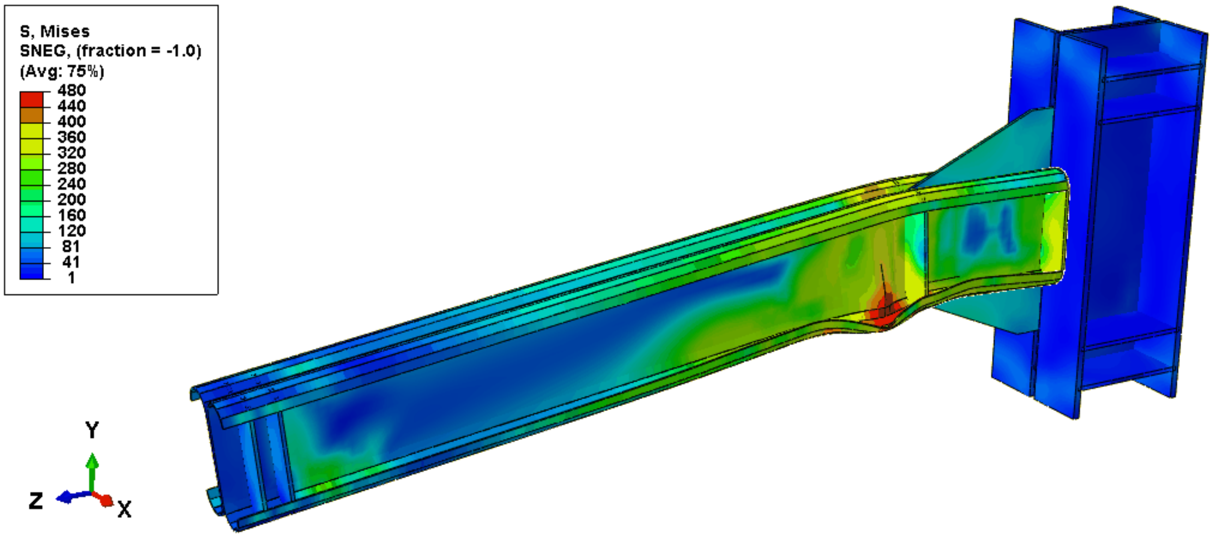


Figure 4.3 Failure mode of specimen A2

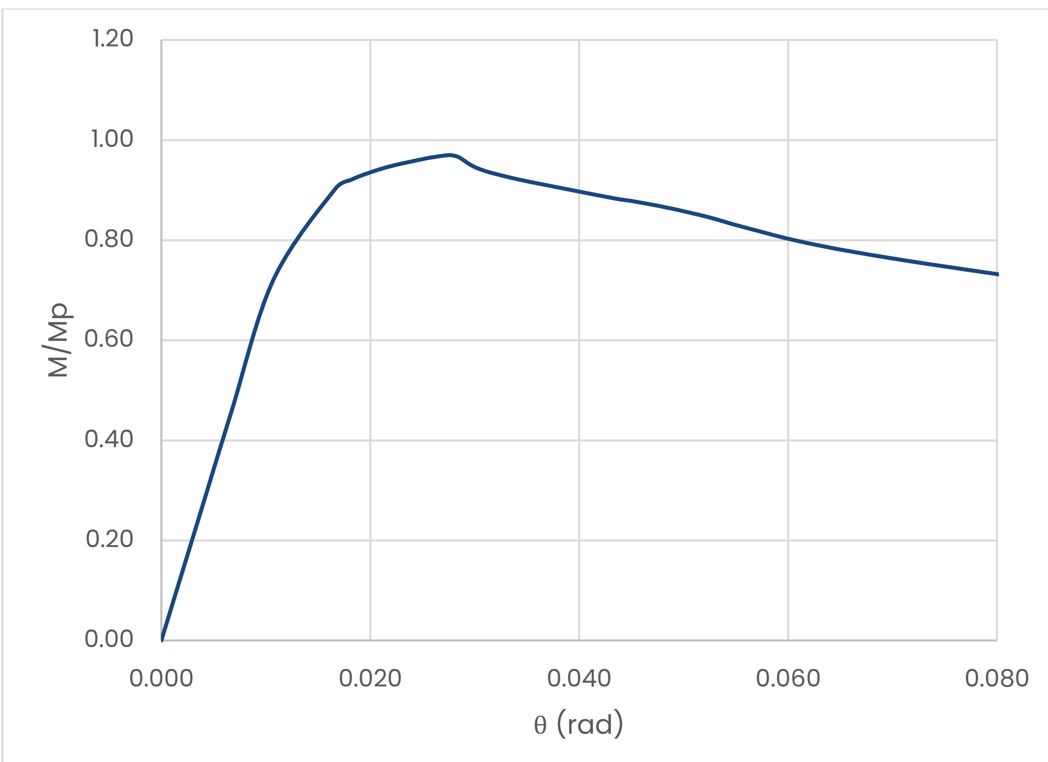


Figure 4.4 Moment-Rotation Curve of specimen A2

4.1.3 Specimen A3

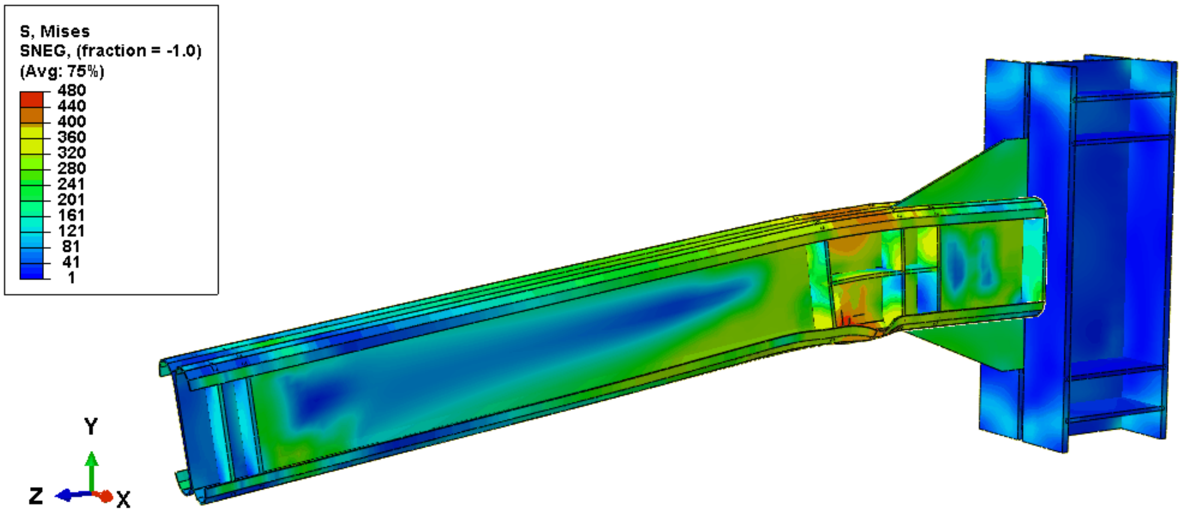


Figure 4.5 Failure mode of specimen A3

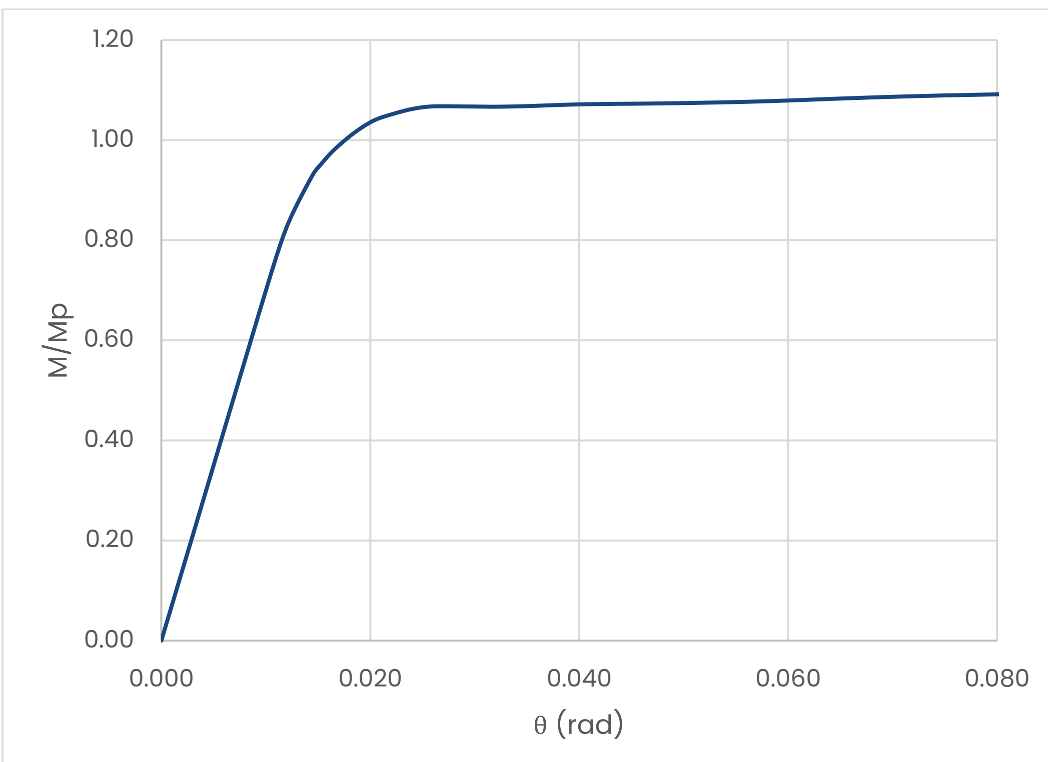


Figure 4.6 Moment-Rotation Curve of specimen A3

4.1.4 Specimen B1

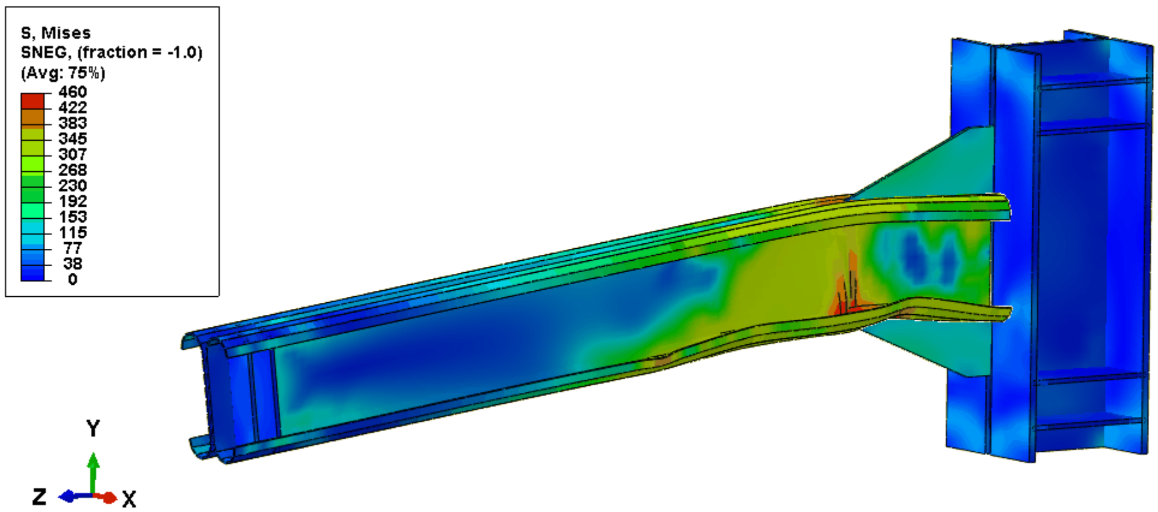


Figure 4.7 Failure mode of specimen B1

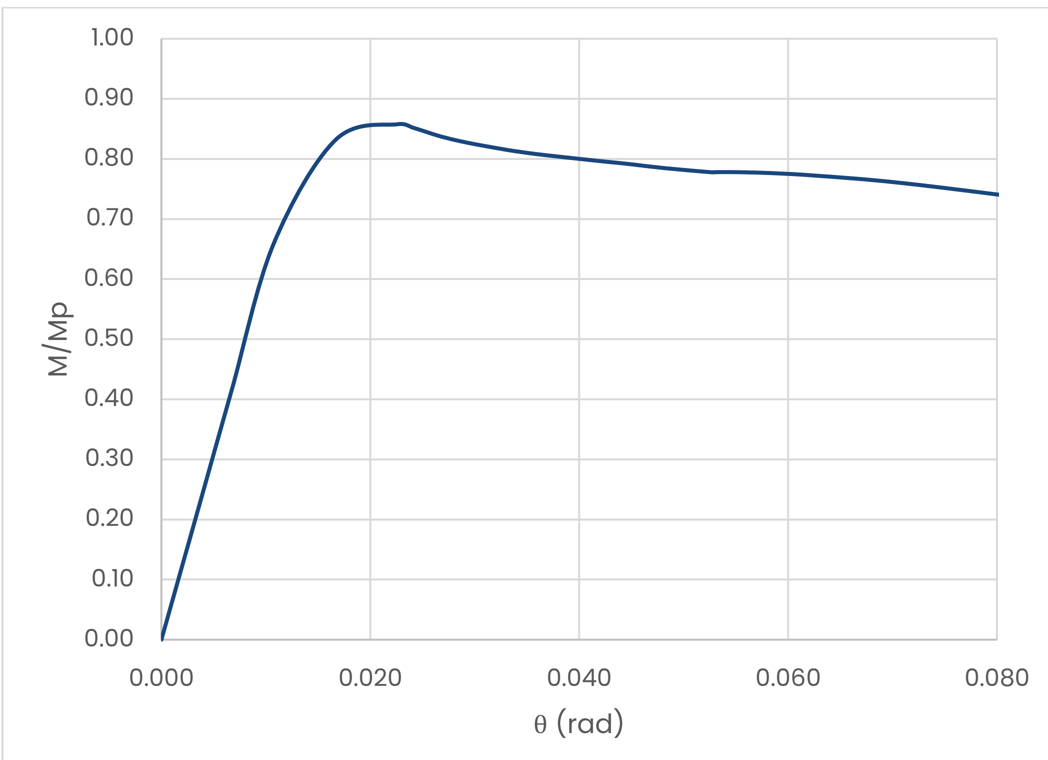


Figure 4.8 Moment-Rotation Curve of specimen B1

4.1.5 Specimen B2

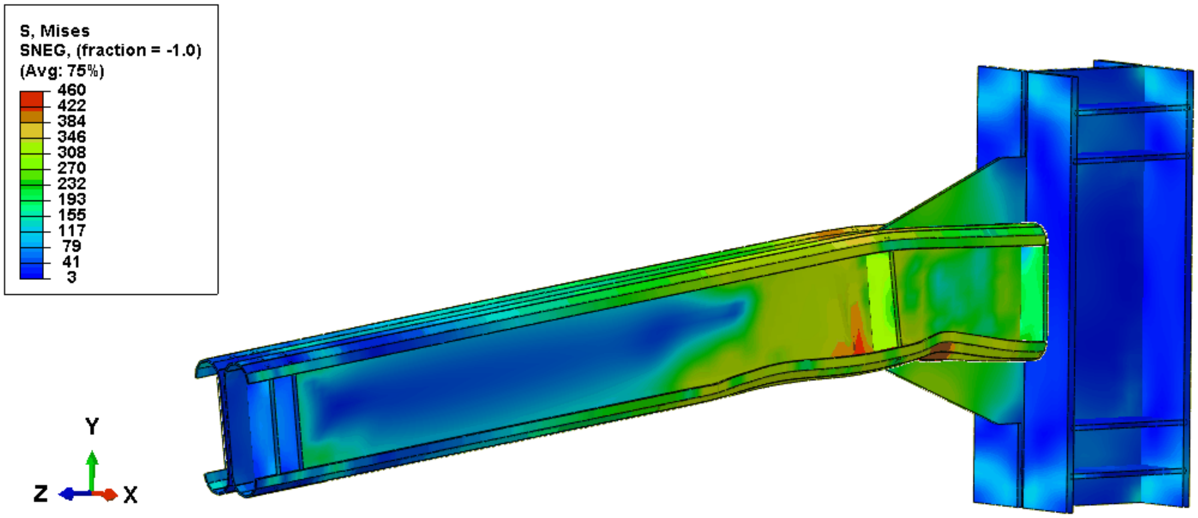


Figure 4.9 Failure mode of specimen B2

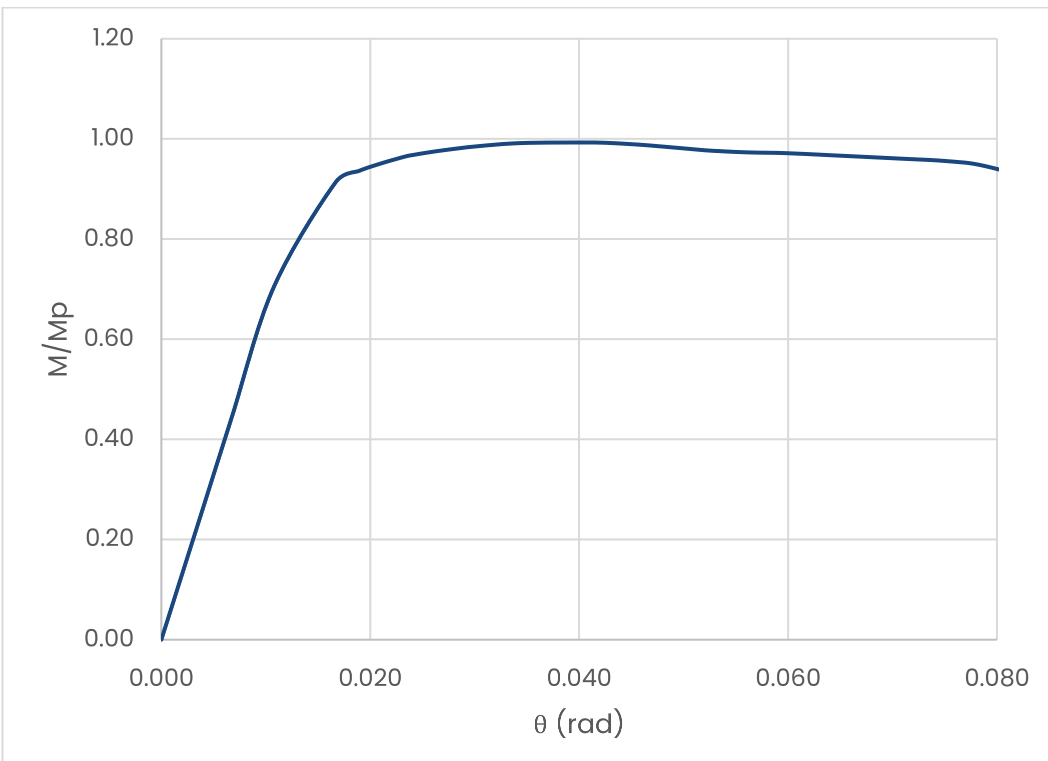


Figure 4.10 Moment-Rotation Curve of specimen B2

4.1.6 Specimen B3

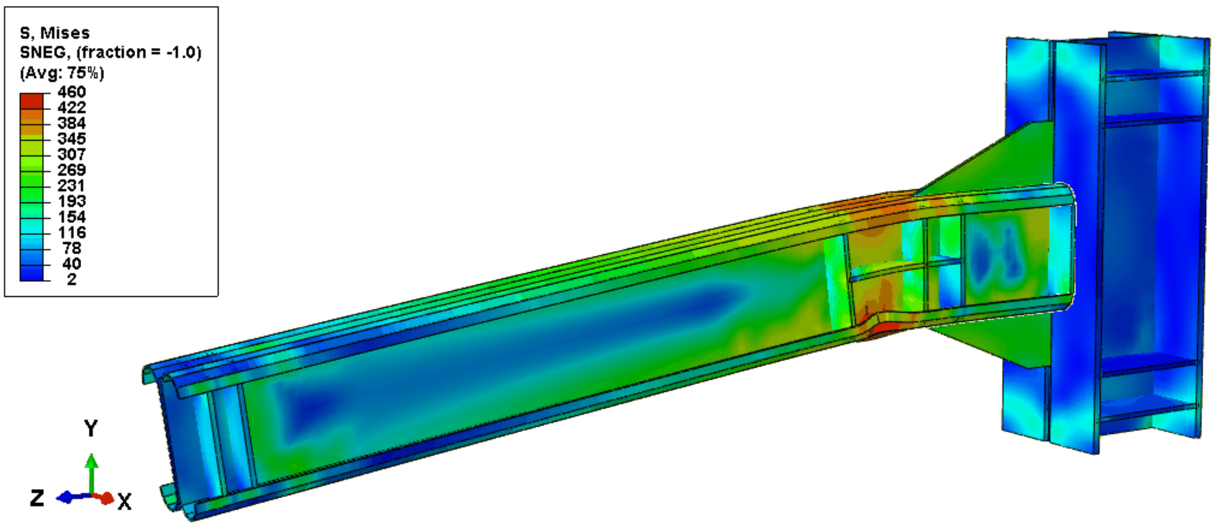


Figure 4.11 Failure mode of specimen B3

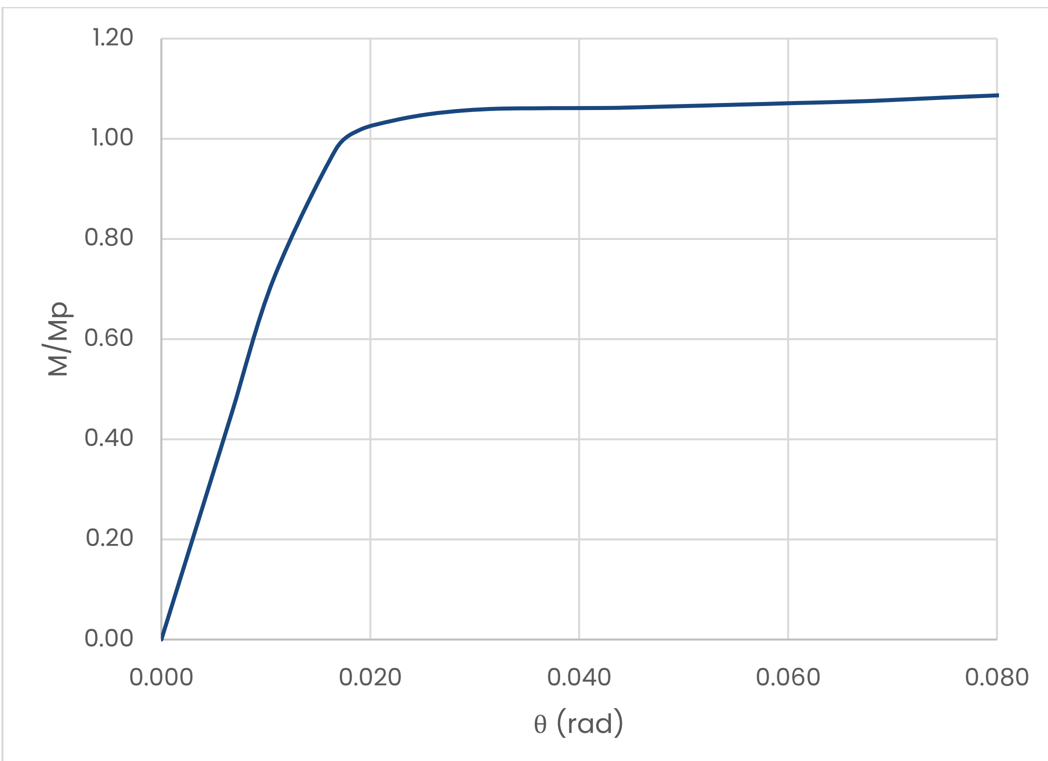


Figure 4.12 Moment-Rotation Curve of specimen B3

4.1.7 Specimen F1

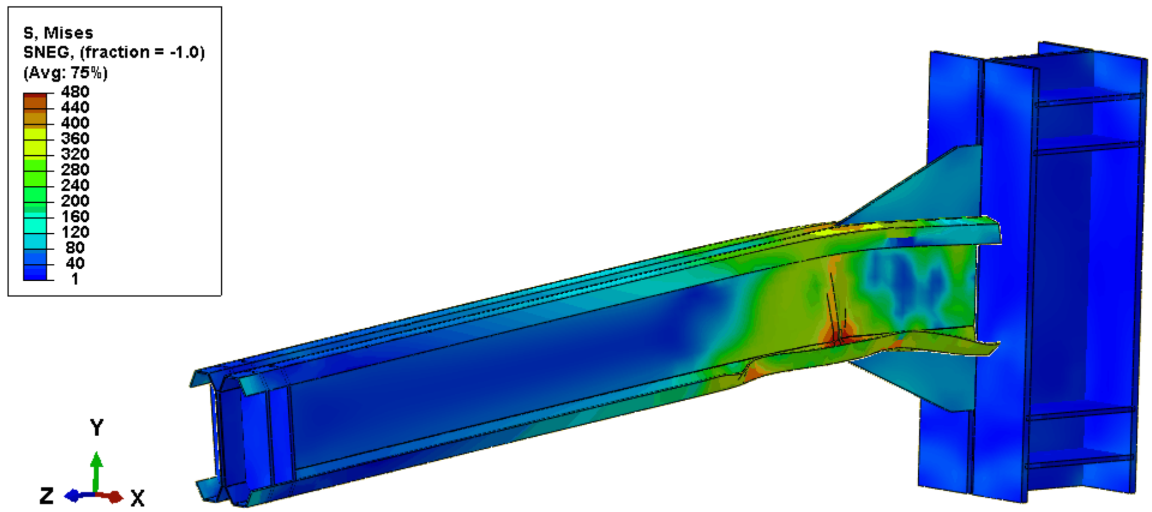


Figure 4.13 Failure mode of specimen F1

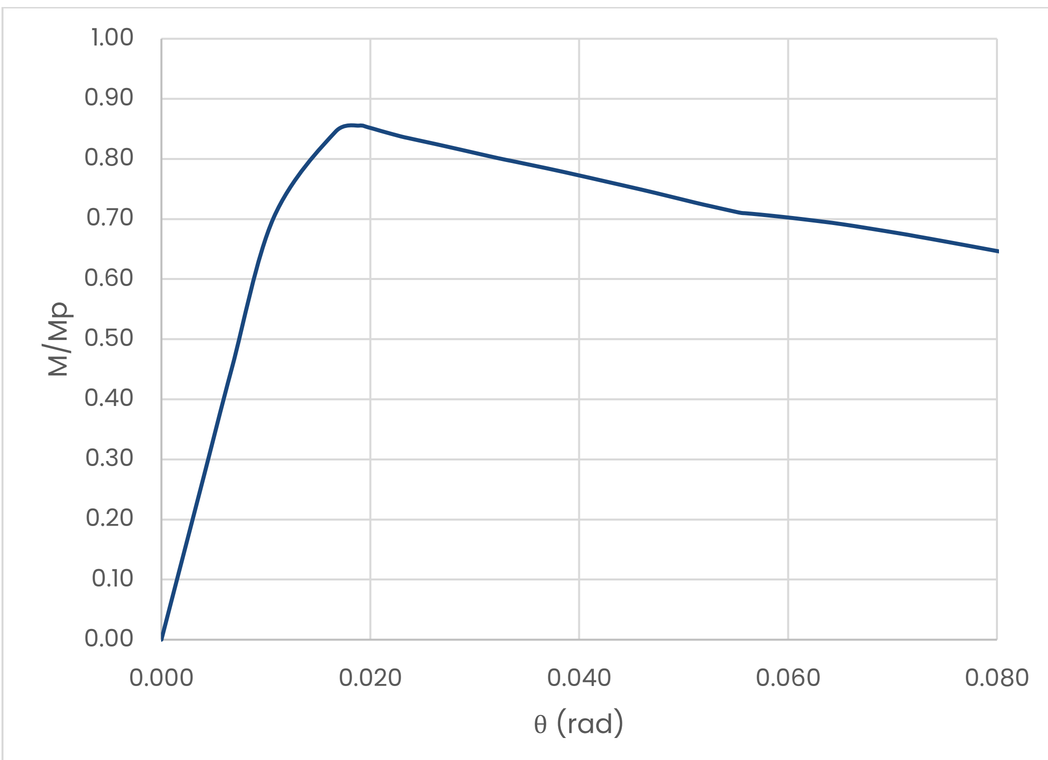


Figure 4.14 Moment-Rotation Curve of specimen F1

4.1.8 Specimen F2

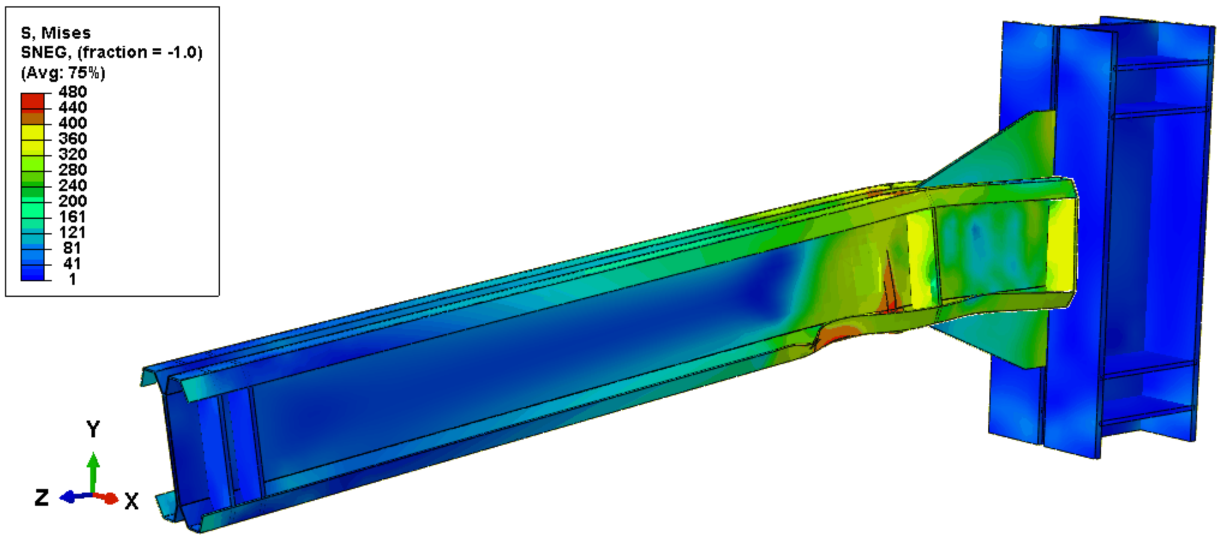


Figure 4.15 Failure mode of specimen F2

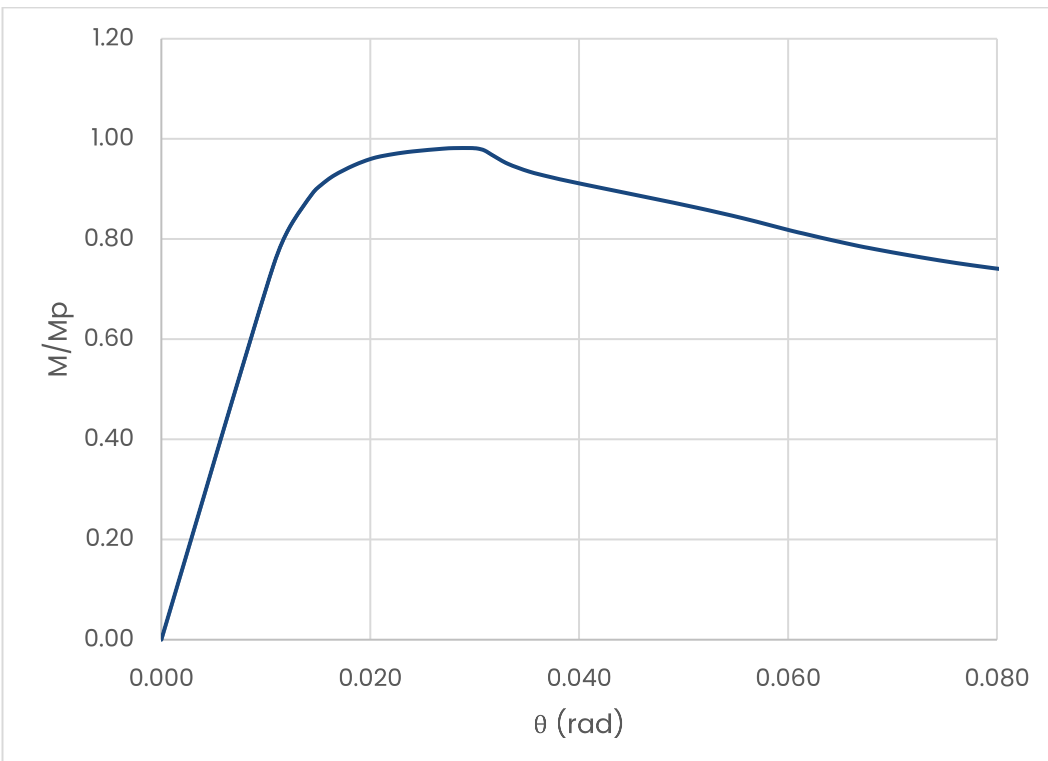


Figure 4.16 Moment-Rotation Curve of specimen F2

4.1.9 Specimen F3

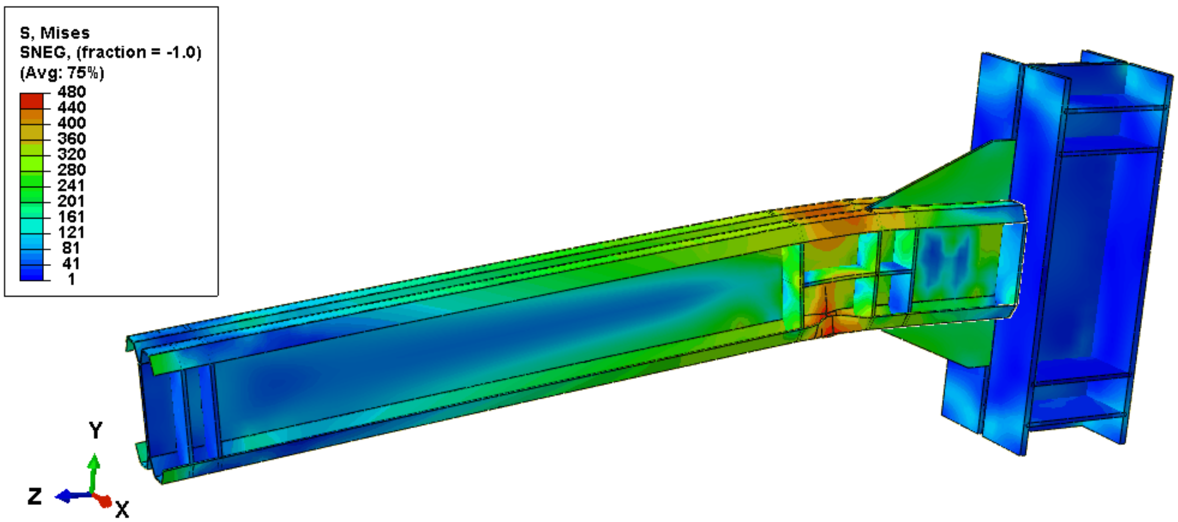


Figure 4.17 Failure mode of specimen F3

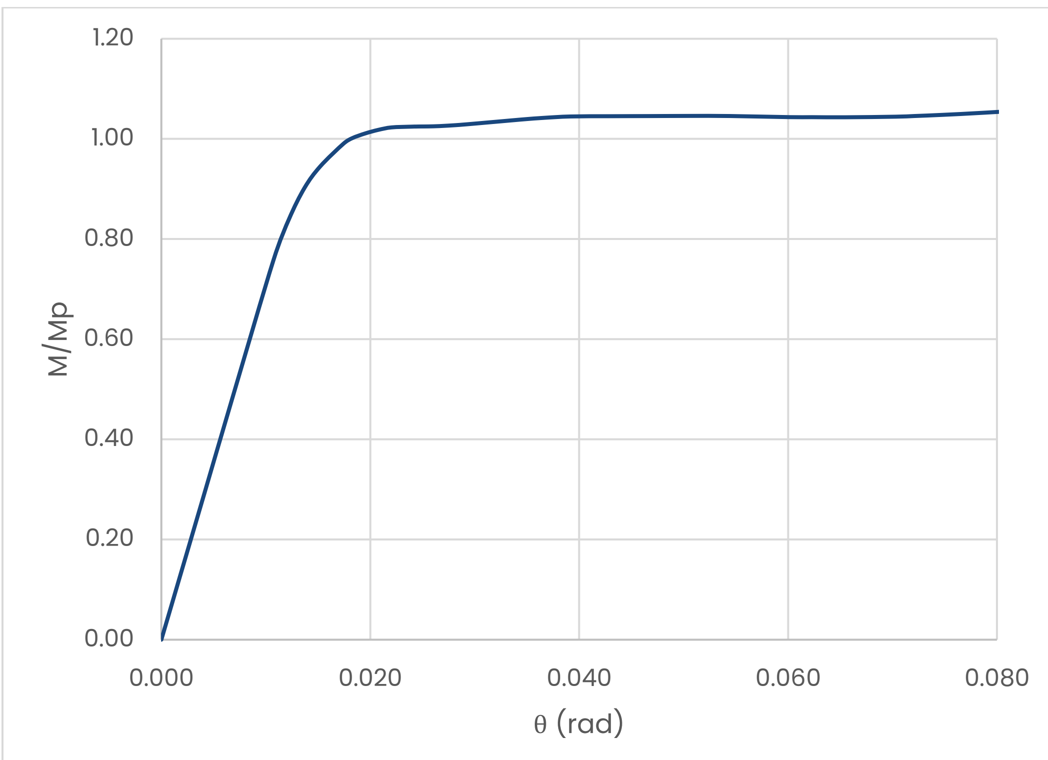


Figure 4.18 Moment-Rotation Curve of specimen F3

4.1.10 Specimen G1

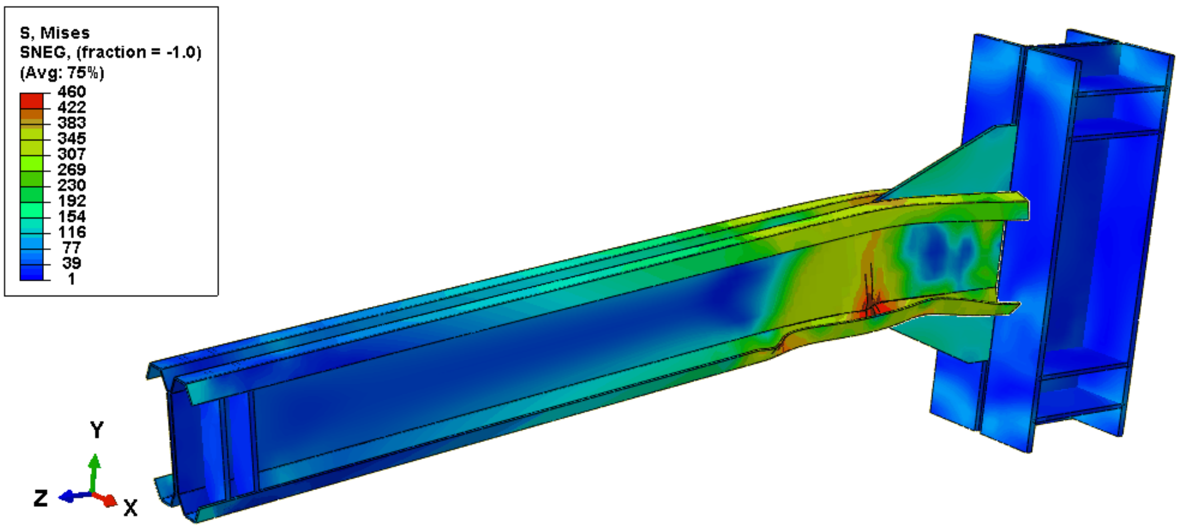


Figure 4.19 Failure mode of specimen G1

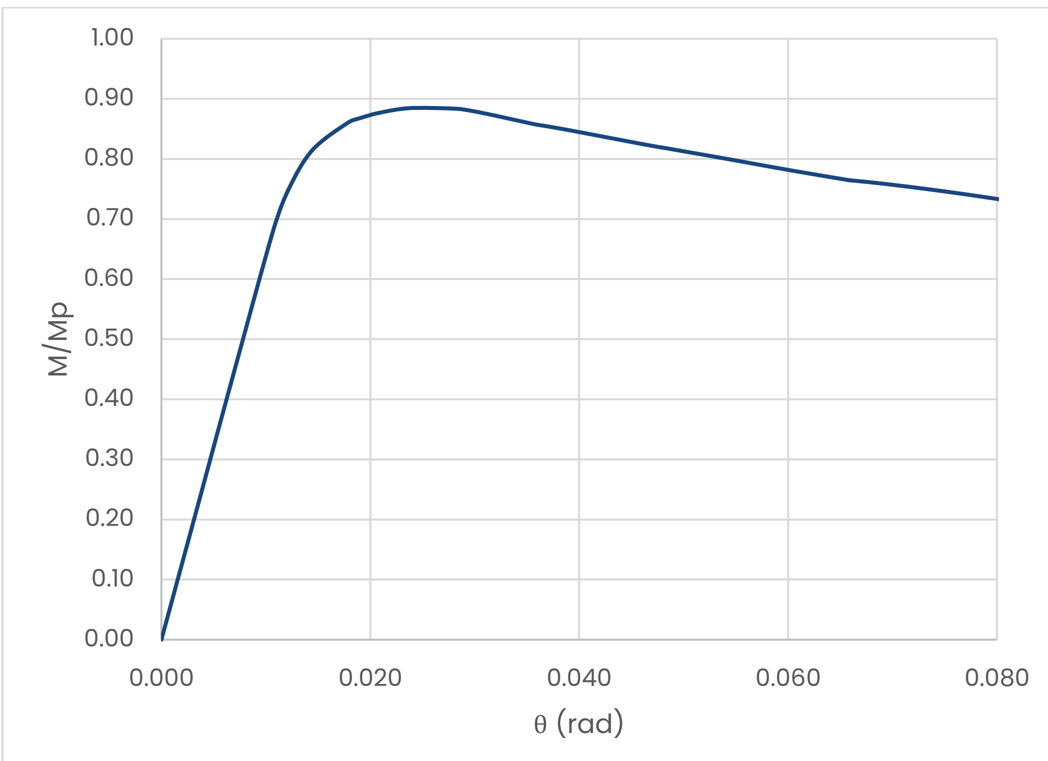


Figure 4.20 Moment-Rotation Curve of specimen G1

4.1.11 Specimen G2

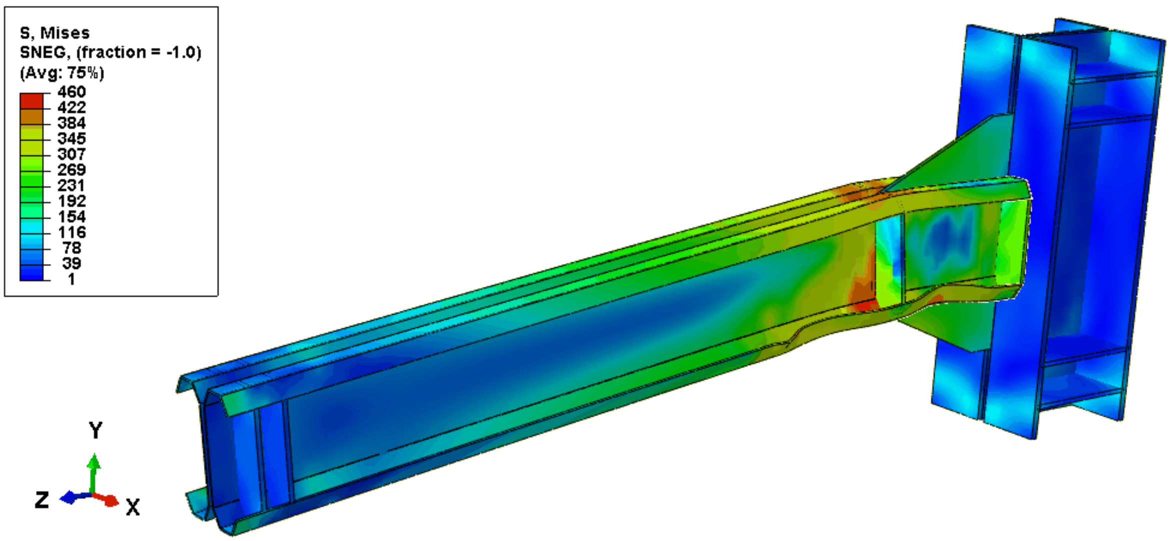


Figure 4.21 Failure mode of specimen G2

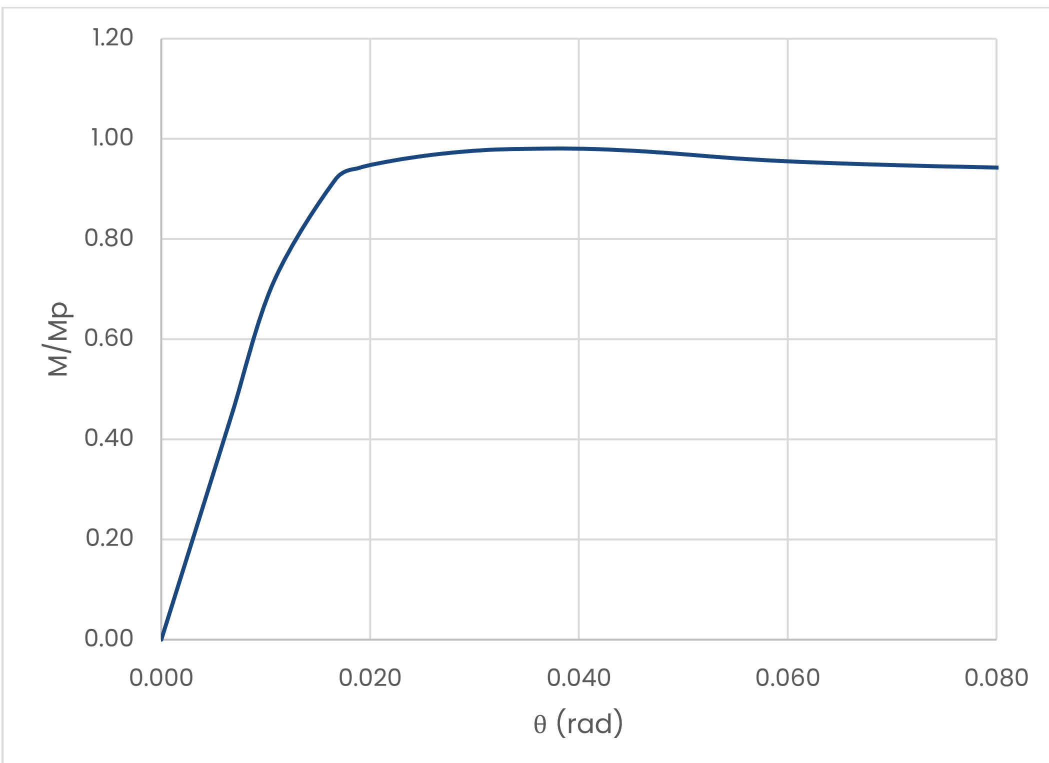


Figure 4.22 Moment-Rotation Curve of specimen G2

4.1.12 Specimen G3

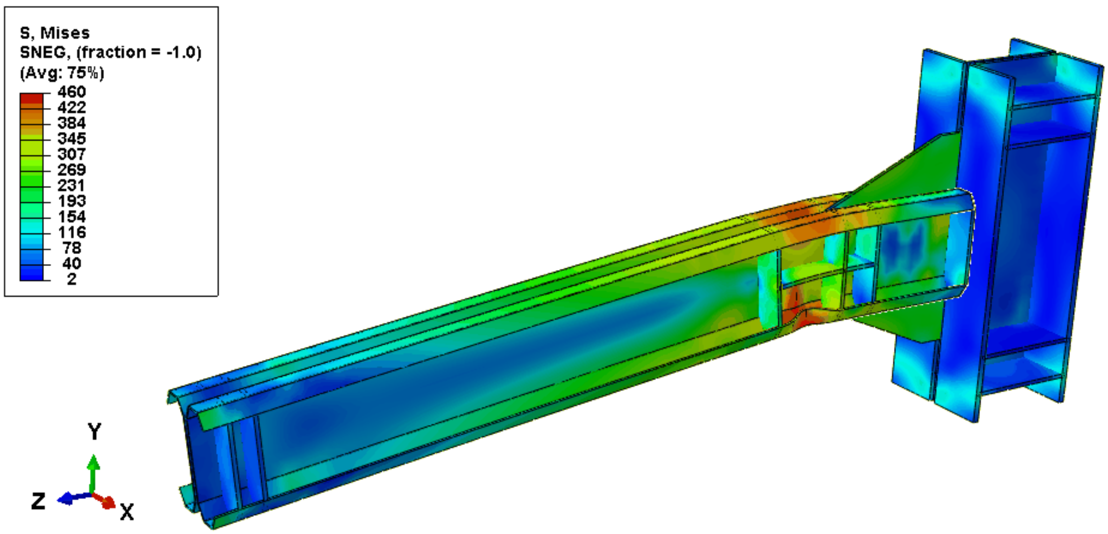


Figure 4.23 Failure mode of specimen G3

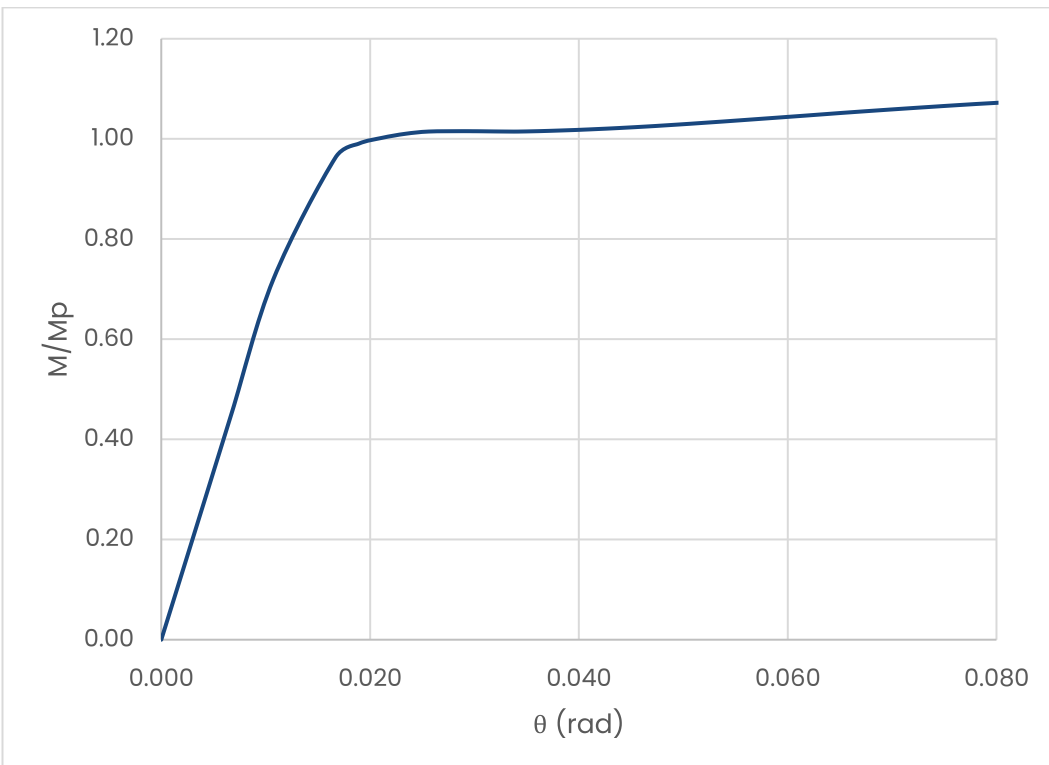


Figure 4.24 Moment-Rotation Curve of specimen G3

4.2 FE modelling validation

Failure modes for specimen A1, A2, A3, B1, B2 and B3 are reported in the following figures:

Table 5 Failure deformations of FE models vs Experimental Tests by (Bagheri Sabbagh et al., 2012b) for specimens A1, A2, A3, B1

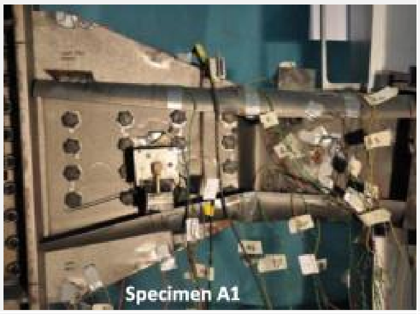
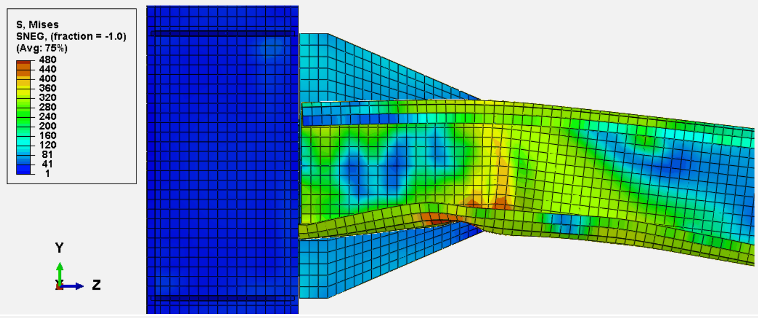
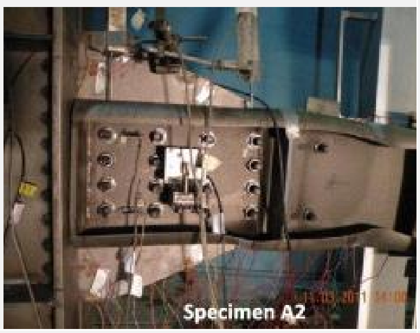
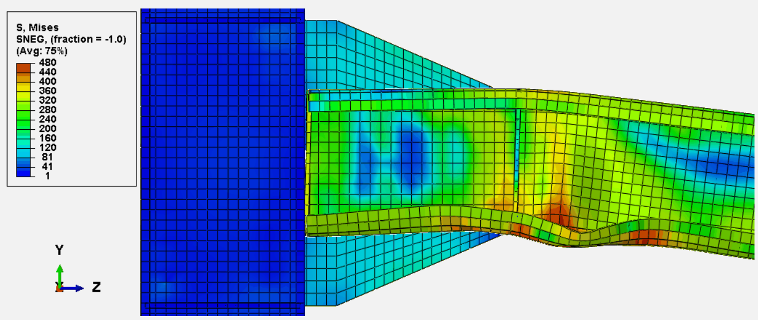
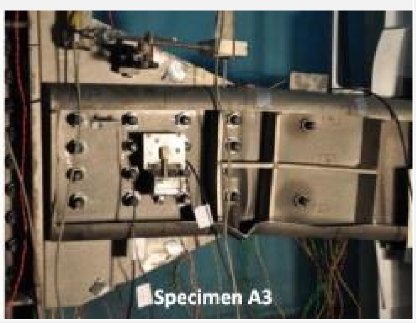
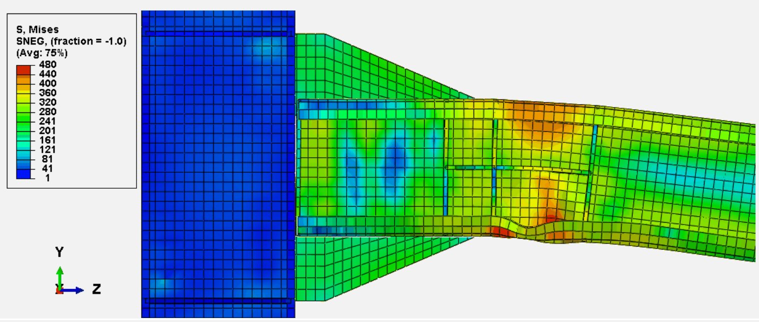
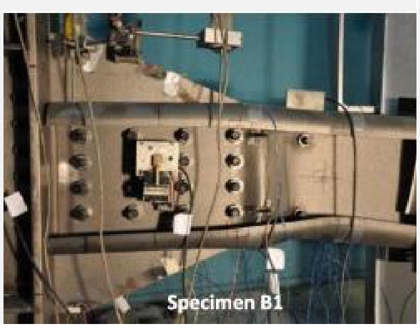
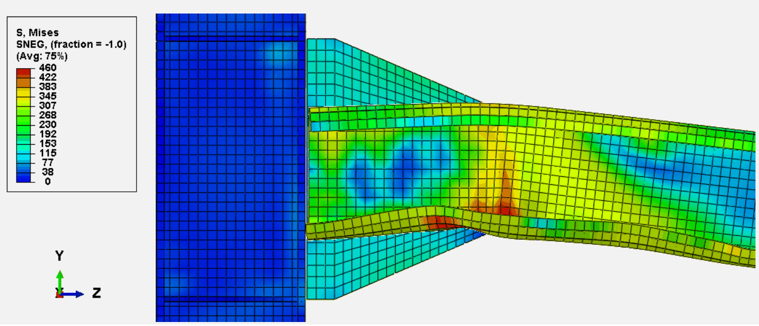
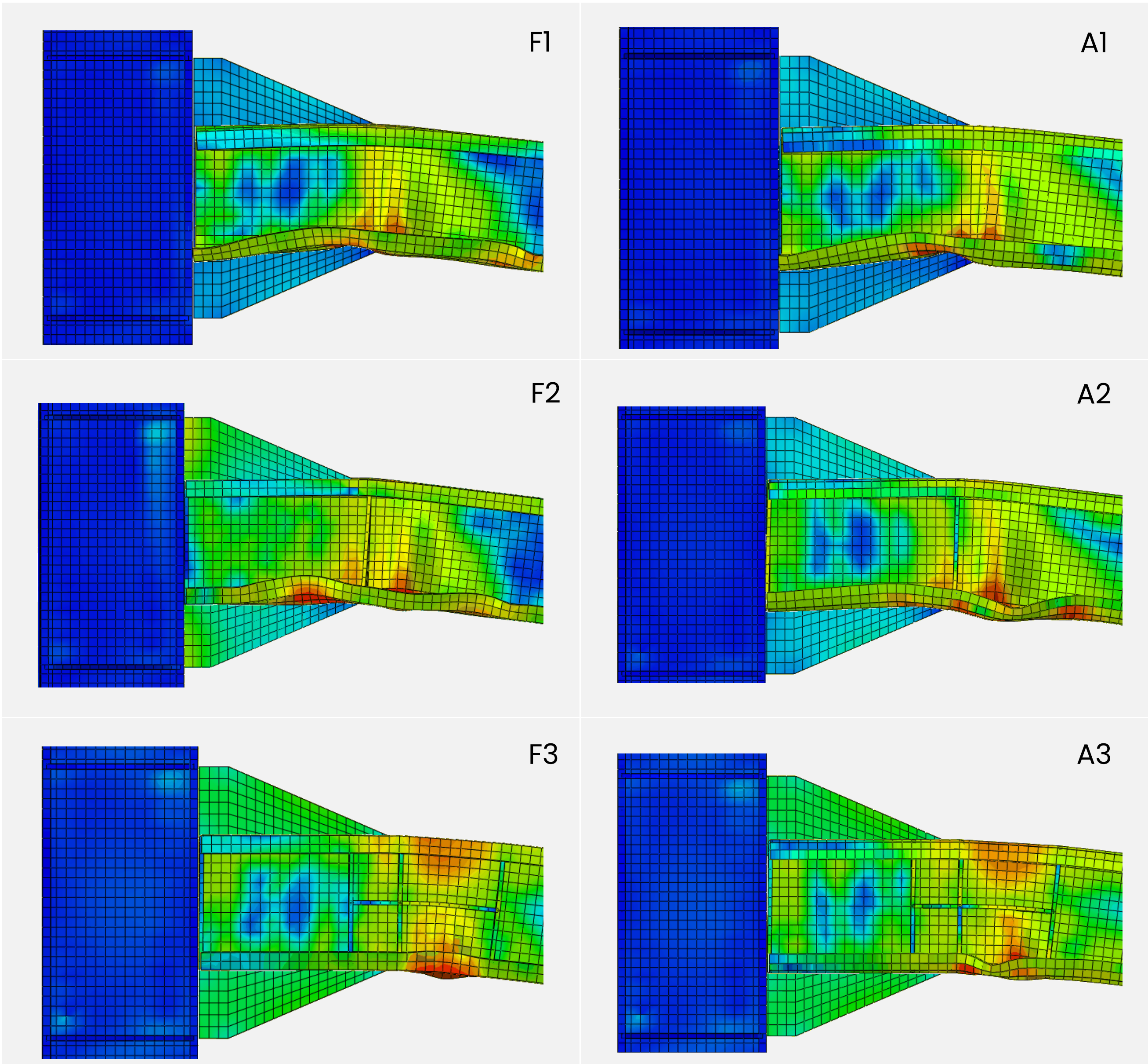
A1		
A2		
A3		
B1		

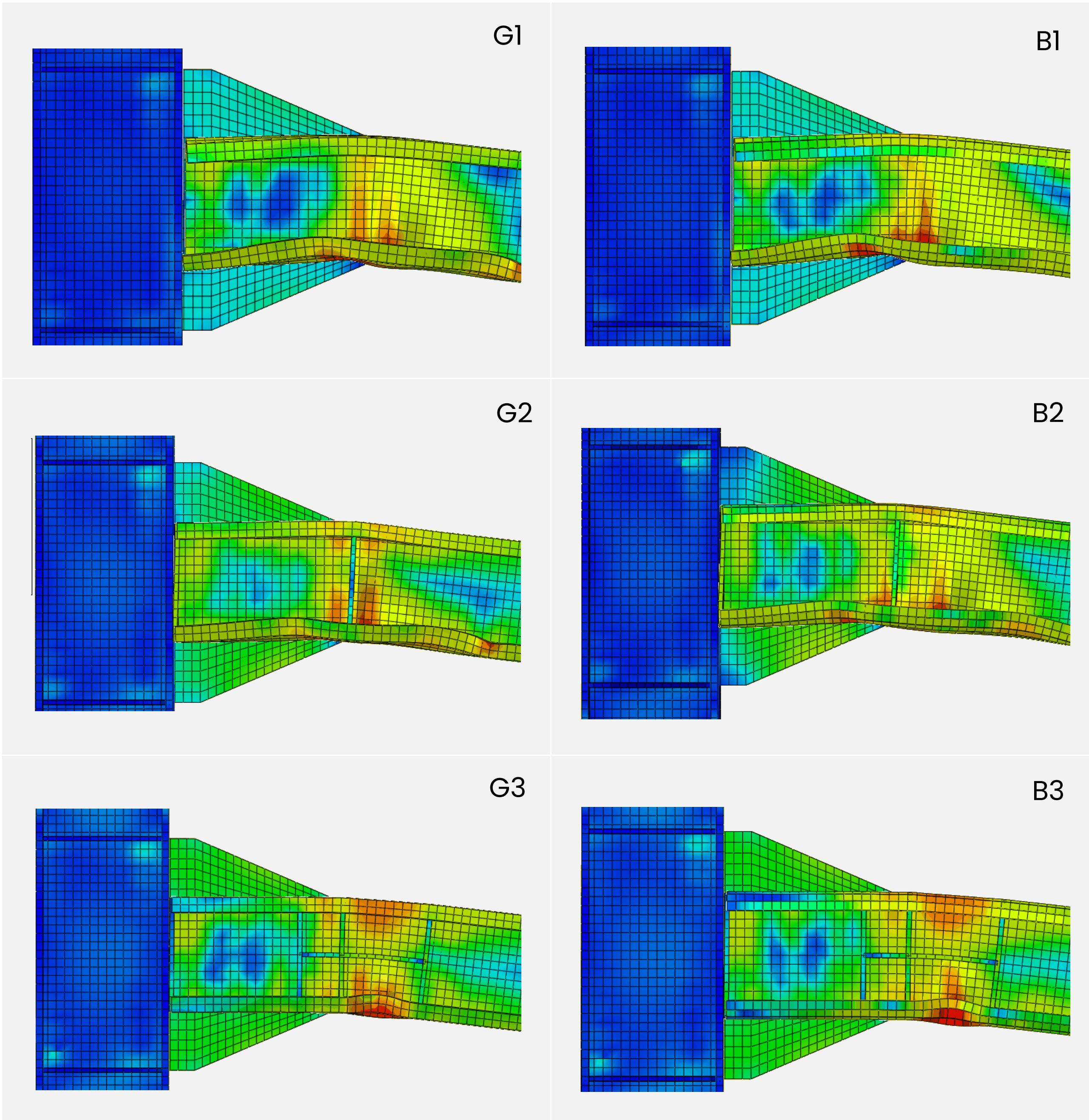
Table 6 Failure deformations for specimens A & F



Failure deformations of FE models are nearly close to the one obtained in experimental tests as shown in Table 5. The conducted experiments were under cyclic loading. However, FE models have been carried out considering monotonic loading which consequently affects the results.

Comparison in failure deformations for specimens A and F as shown in Table 6 and for specimens B and G as shown in Table 7 have been conducted using FE models created in this study.

Table 7 Failure deformations for specimens B & G



Moment-Rotation curves have been presented considering normalized value of moment with respect to plastic moment (M_p) to better compare with reference experimental and numerical results. (M_p) is the nominal moment of the curved beam section: **67 kNm** for specimens **A** and **90 kNm** for specimens **B**, all assumed with nominal yield stress **f_y = 275 MPa**. However, the actual yielding stresses of the beams based on tensile test results for specimens **A** and **B** were **313 MPa** and **322 MPa**, respectively. Therefore, the actual plastic moment strength of the beams is expected to be **76 kNm** for specimens **A** and **105 kNm** for specimens **B** (Bagheri Sabbagh et al., 2012b). The values of plastic moment were determined by using the dimensions of the beam cross sections, with the assumption of full effective width for all the sectional elements (Bagheri Sabbagh et al., 2013).

The normalized moment (**M/M_p**) has been plotted against the rotation value. The rotation of the connection was determined as the ratio of the beam tip displacement to the length of the beam up to the through plate that is **1.7 m** as shown in Table 3.

Cyclic envelopes of previous experimental moment-rotation curves have been compared with the numerical results for specimens **A** as shown in (Figure 4.25).

According to (Figure 4.25), the initial stiffness of the connections in the FE models and the maximum strength are in good agreement with reference results. However, they cannot capture the post-buckling behaviour. Still to improve the accuracy of the results, geometrical imperfections and bolts slip-critical behaviour require to be taken into consideration.

Previous cyclic envelope of numerical moment-rotation curves has been also compared to the numerical results obtained for A as well as B specimens also showing the adequate correspondence between results as reported in the chart (Figure 4.26) and (Figure 4.27). This would consider the FE models as reliable, hence, the form a good starting point for future research and experimental models validations. Note that for specimens B the comparison have been performed only for B1 and B2 as there were no existing data related to B3 FE models because B2 and B3 were tests to study specifically the effect of slip on the behaviour of the connection. Hence, the comparison can be mainly effective for specimen A1, A2, A3 and B1 (behaviour dominated by deformations) while for B2 and B3 (behaviour dominated by slip-bearing actions) more specific FE cyclic with slip critical behaviour must be conducted to capture the actual behaviour.

It can be observed that the region after reaching plastic moment have significant degradation in reference FE cyclic models. This is due to the fact that FE monotonic tests are not able to capture degradation in moment capacity due to hysteretic behaviour. Thus, conducting cyclic tests will accurately give better results to simulate the behaviour in seismic regions.

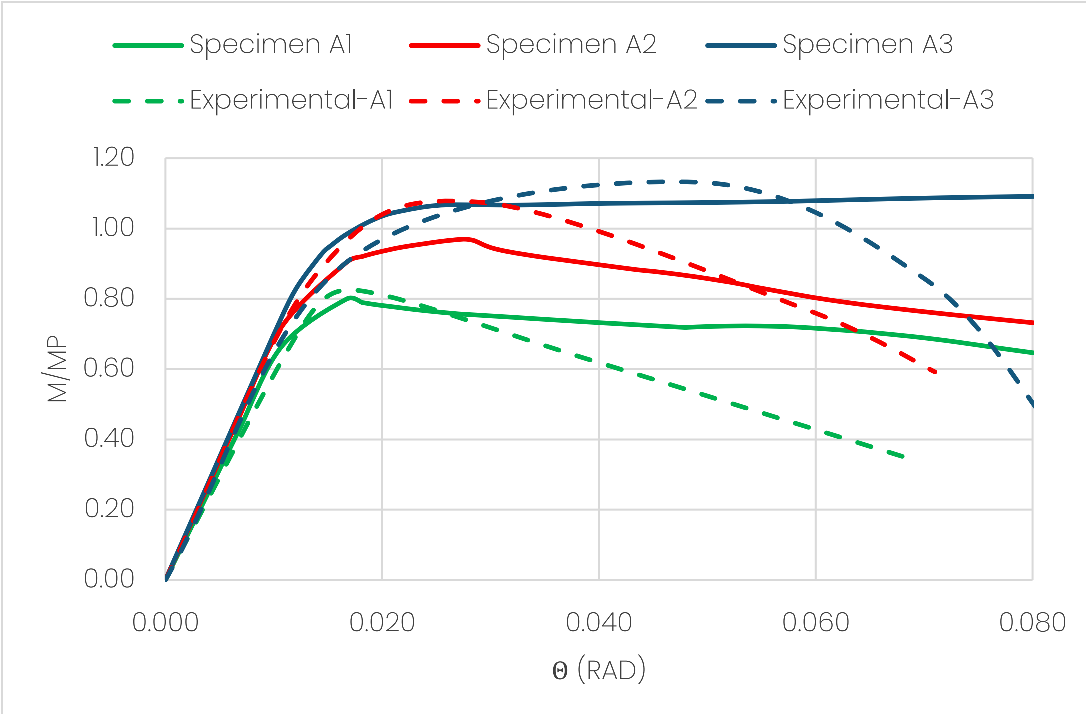


Figure 4.25 Comparison between experimental test by (Bagheri Sabbagh et al., 2012b) *cyclic envelop curves and FE models moment-rotation under monotonic loading for A*

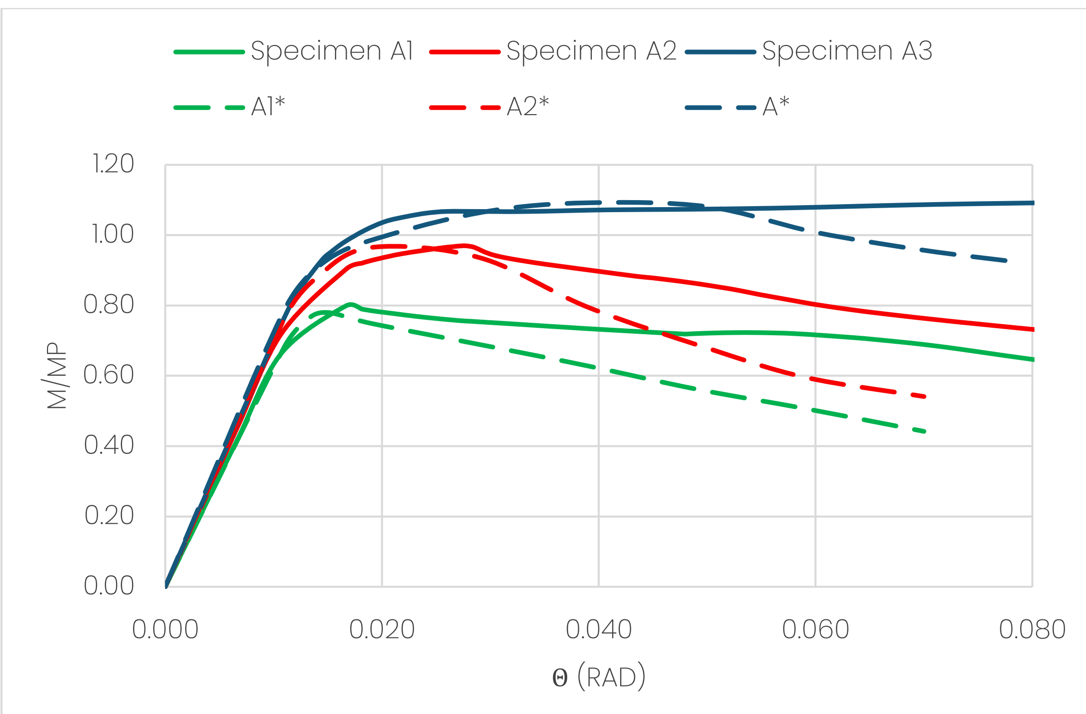


*Figure 4.26 Comparison between envelope of cyclic numerical existing tests** (Bagheri Sabbagh et al., 2013) *and FE moment-rotation results under monotonic load for A*

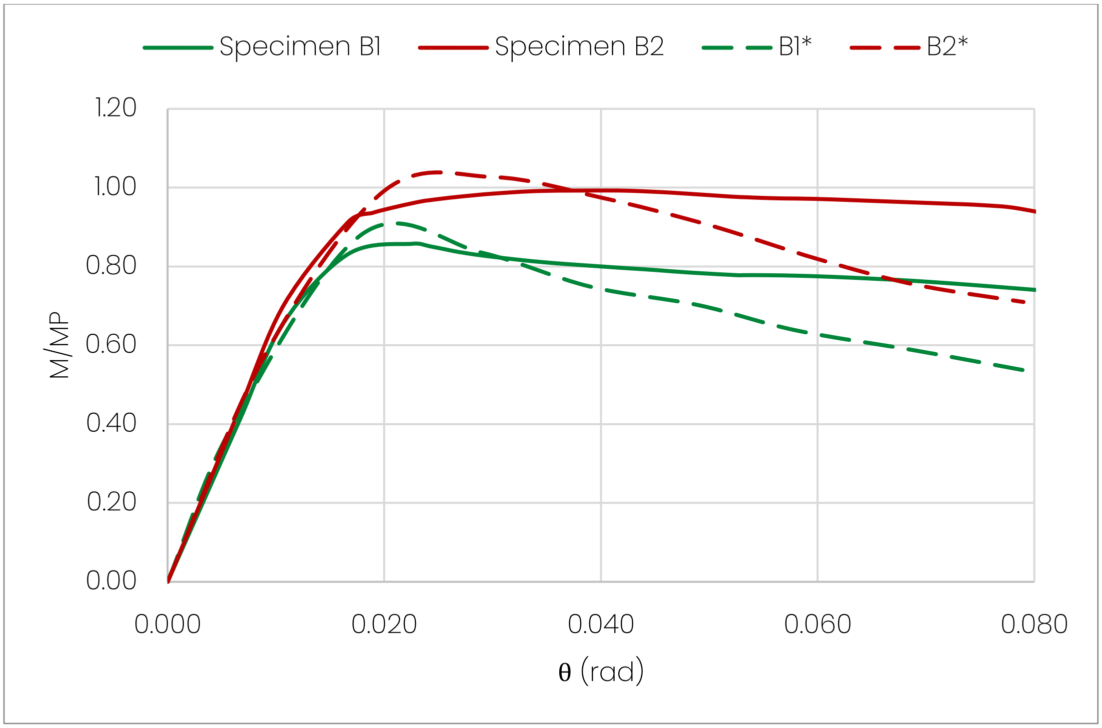


*Figure 4.27 Comparison between envelope of cyclic numerical existing tests** (Bagheri Sabbagh et al., 2013) *and FE moment-rotation results under monotonic load for B*

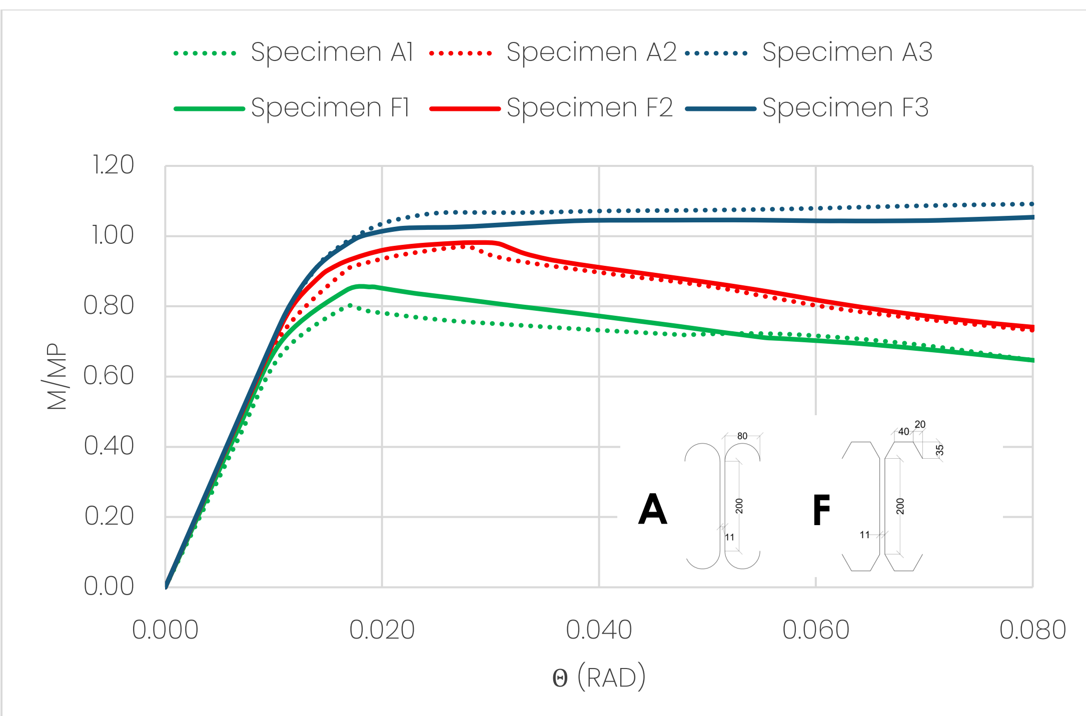


Figure 4.28 Moment-Rotation curves for connections with curved flanges beam specimens A vs connections with folded flange beam specimens F

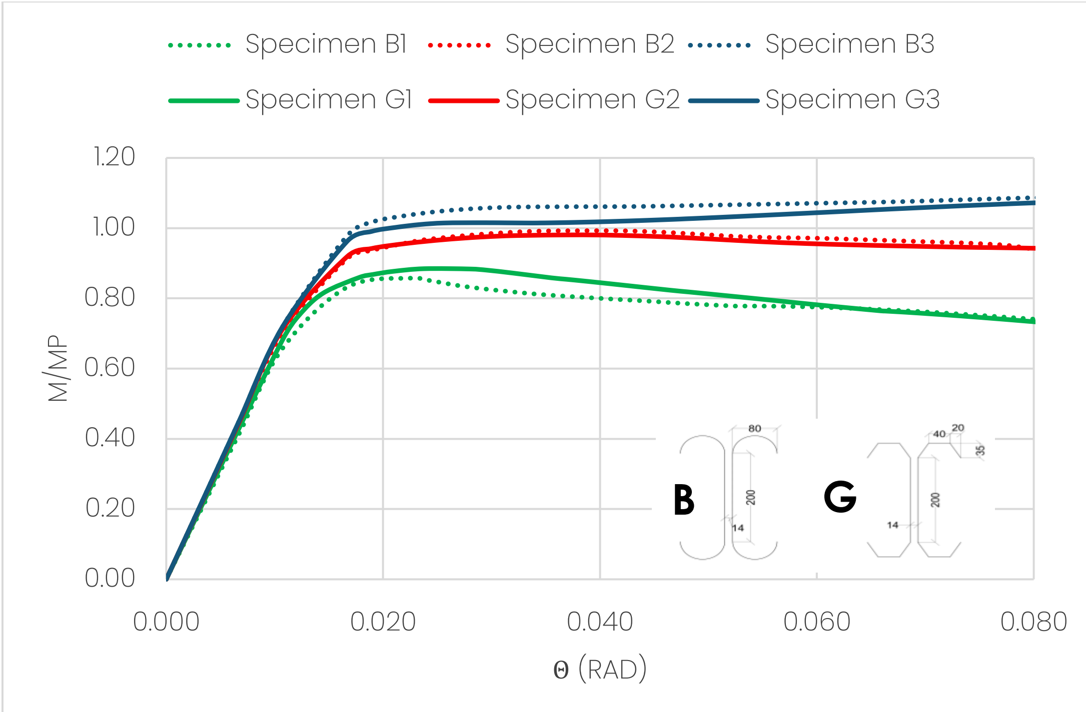


Figure 4.29 Moment-Rotation curves for connections with curved flanges beam specimens B vs connections with folded flange beam specimens G

Advanced FE models are developed for CFS bolted beam-to-column through plate moment resisting connection by replacing the curved flange beam in specimens A and B by folded flange beam in specimen F and G to better understand if folded section can be an alternative solution that meets both construction and structural requirements. The results are reported and constructed in terms of moment-rotation curves as shown in (Figure 4.28) and (Figure 4.29). It is observed that the folded flange beam-to-column connection exhibit similar behaviour and maximum capacity with respect to the curved flange beam. Additionally, Figure 4.28 and Figure 4.29 illustrated how out-of-plane stiffeners play an important role in increasing the capacity of the connection. They also affect the behaviour of the connection causing the local buckling to be delayed which allows the connection to maintain high deformation up to the plastic deformation as seen in the case of A3, B3, F3 and G3 moment-rotation curves in (Figure 4.6, Figure 4.12, Figure 4.18, and Figure 4.24). As a result, they are predicted to have good influence on ductility. Further, cyclic studies with more accurate models with geometrical imperfections and bolts behaviour mechanism modelling would be conducted to study assess the ductility and energy dissipation of such connections.

CHAPTER 5. DISCUSSIONS

With reference to CHAPTER 1, guidelines and standards do not adequately address Cold-formed steel framing systems design especially for CFS connections for moment resisting frames.

The use of CFS sections as primary structural members is in increasing manner, this is due to the numerous and competitive advantages this material offers compared to other materials as referred to in Section 1.3.2

Due to the complex behaviour of CFS members, the structural application of this material has been under investigation for decades. Nevertheless, successful examples of CFS framing have been reported recently worldwide and also in Italy as shown in Section 1.4

In the midst of climate change and homelessness crisis, it is more and more essential to think of innovative solutions for pressing global socio-economic issues. Within the construction sector, the social economic issues relating to buildings such as unavailable labour, ageing societies, and environmental concerns e.g. reducing embodied carbon, waste and efficient use of limited resources, can all be proactively addressed by the construction sector.

Working on new construction technology using high sustainable material contribute to solving socio-economic issues through a reduction in waste, a reduction in carbon, whilst utilizing natural resources more efficiently and manufacturing at lower cost and quicker speed.

Light steel framing and modular construction that are offsite construction forms can help meet building targets in a sustainable manner. This new form of construction has demonstrated the significant impact they can leave on environment, society and economy which make them in line with sustainability requirements for buildings as shown in Section 2.1 where also indicated a recent successful example of CFS framing in Italy.

Still, the implementation of CFS framing systems is restricted to low- to mid-rise buildings, especially in seismic zones. In addition to that, the lack of design guidelines and standards limit their use in industry and also limit the design of such structures to only design specialists. Therefore, more efforts have to be conducted in research through numerical and experimental testing to help promote this eco-friendly and cost-effective material and new construction systems.

In areas where seismic events are prevalent such as Italy, CFS panelised structures using CFS shear walls provide good performance under seismic events. Experimental testing and analytical studies have been performed to study their behaviour as indicated in Section 2.2.1. Nonetheless, such systems have demonstrated high performance only for low- to mid- rise buildings up to eight stories. This is due to limited ductility of such systems.

When it comes to multi-story buildings, CFS moment resisting frames are considered a good solution as lateral-load resisting systems. Section 2.2.2. shows previous studies that have been carried out in research in the past 10 years. It is observed that there is limited research in this area, especially when it comes to experimental investigations.

CFS bolted moment-resisting connections are the most essential elements of CFS moment resisting frames. Thus, studying the behaviour of connections and joints under seismic actions is of highest concerns and focus of recent research. There is lack of understanding of the behaviour of CFS connections as well as lack of design standards which makes their applications in industry much restricted. CFS connections design plays an important role towards sustainable building and has a significant impact on the lifecycle of sustainable structures. As a result, CFS connections studies and investigations cannot be ignored. Although there are limited applications of CFS moment connections over the past decades, their performance was only limited to portal frames. According to standards, connections need to be classified as rigid or semi-rigid in order to be suitable for moment resisting frames in seismic zones. Thus, the most recent studies have presented a new form of CFS moment-resisting connection that is CFS bolted beam-to-column with through plate connection with the ability to meet the requirements for moment resisting frames in seismic regions as shown in Section 2.2.2.

Previous analytical studies on CFS sections and CFS connection have been carried out to highlight the improvement the CFS sectional shape might have on the overall performance of the connection. In Section 1.3.2, the evolution of CFS sections over time have been presented. It is shown that the cross-section went from flat section to more complex stiffened sections. This is because, for CFS members, local buckling failure is dominant and hence the introduced stiffening part in the section help limit local buckling phenomena. At the end, as shown in Figure 2.12, the connection performance can be enhanced by introducing more bends to beam flanges. It was observed that the beam of curved flange is the optimum solution in terms of moment capacity and strength. However, this kind of section is difficult to manufacture and experience difficulties related to real construction practice that limit the placement of floor system.

Figure 2.9 presents the primary experimental test conducted on CFS bolted beam with curved flanges-to-column with through plate connection. The experimental study has been performed on 6 specimens as shown in Figure 2.10. The results of the experimental tests have demonstrated the ability of the connection to reach plastic moment and sustain plastic deformation. Therefore, this kind of connection can be suitable for moment resisting frames because of its ductile behaviour and maximum strength. In fact, two types of connections were tested: connections with behaviour dominated by deformations as well as connections with behaviour dominated by slip-bearing action. Each type of connection has been tested taking into account two additional out-of-plane stiffeners configurations. It is observed from the test that local buckling that occurs at the web of the beam can be delayed when using out-of-plane stiffeners thus increasing the capacity of the connection to perform plastic deformation which leads to significant improvement in terms of ductility. The arrangements of stiffeners have been reported in Table 3. This study also demonstrates the key role stiffeners can play on the behaviour of the connection as illustrated in Figure 4.28 and Figure 4.29 and has good agreement with respect to existing tests results.

Despite the enhancement contribution that can be provided by the out-of-plane stiffeners, they may address some issues and challenges including difficulties in relation to welding, especially to beam flanges. In the addressed experimental study on connections, as shown in Table 5, the stiffeners with curved edges where welded to flanges that make it very difficult to be applied when high number of connections are needed that could affect the time and cost of project and eventually hot-rolled connection might be favorable.

Moreover, the number of bolts and the gusset plate size and shape will restrict the application of such connection to a high level. As mentioned in Section 2.2.2. Analytical studies have addressed the later issues acting on many key parameters including through plate shapes, configurations, thickness and size as well as bolt group arrangements. More practical solutions have been provided and investigated. However, these studies are not complete and need to be validated by testing. One solution that is found to be serving in terms of applicability and structural performance for gusset plate is the gusset plate with curved edges that allows to have more space for construction needs.

This study aims to create a coherent numerical model for future research work in this field and preparation for experimental work in the laboratory of Politecnico di Torino. Thus, previous experimental tests were compared to the FE models conducted in this study showing high correspondence in result as shown in Figure 4.25 in Section 4.2. In fact, it is observed that the initial stiffness and the maximum strength of all specimens experience high similarities. However, the FE models could not capture the post-buckling behaviour and the capacity degradation due to cyclic action that is clear in experimental tests. This means that analytical tests with monotonic loading cannot provide reliable results for connections experiencing seismic events, hence, cyclic analytical test should be performed to better understand the behaviour of the connection.

Additionally, the existing FE models were also compered to FE models of this study, results in Figure 4.26 and Figure 4.27 indicate the correspondence in results which at the end allow this numerical models suitable for future investigations.

Finally, this study addresses analytically the connection with folded beam flanges that were suggested in previous research. The results in Figure 4.28 and Figure 4.29 show that CFS bolted connection with beam of folded flange can be a good alternative to the curved one. This study would highly suggest the use of folded sections in future experimental tests as it does not have only good structural performance but also can respects construction and practical requirements including the fact that it is easier to manufacture and allow the placement of floor systems.

CHAPTER 6. CUNCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

I. Enviromental, social and economic concerns call for a rapid implementation of new construction technology that allow the contribution of construction sector to address these issues. Light cold-formed steel framing and modular construction have proved to be in high agreement with sustainability requirements and great solution in construction industry to help cope with global issues such as climate change by limiting embodied carbon and construction waste.

II. CFS connections and joints can significantly impact sustainable building design as they form a major part of design fabrication and erection that contributes to almost half of the total steelwork.

III. CFS typical framing systems have been proved to have good seismic and structural performance. However, this is restricted to only low- to mid- rise buildings. In seismic regions including Italy, since CFS framing are becoming popular, there will be need to more sustainable multi-storey buildings where CFS moment resisting systems can be adopted. However, there are no guidelines and design standards regarding the design of CFS moment resisting structures or connections. Not to mention the lack of experimental testing. Thus, this type of system needs to be more investigated.

IV. Although CFS bolted beam-to-column with through plate connection appears to have good performance and able to experience proper ductility and strength. Still, they face limitations related to real construction practice which also limit their promotion to industry. Stiffeners numbers and arrangements, and bolts number restrict its implementation especially when high number of connections need to be installed. Thus, finding an alternative solution to stiffeners that can delay local buckling and reducing bolts number are considered of main challenges that face this type of connection. Not to mention the size and shape of the through plate which also limit the use of this connection in industry. Therefore, reduction in size or find a suitable shape of gusset plate that provides more space adds another challenge. Although previous studies presented some solutions, the studies were analytical and recommended experimental studies needed to provide better understanding of their behaviour.

V. The comparison between monotonic FE models of connections with beam of folded flanges show high correspondence with curved beam connections which promote their use and make them more suitable for practical applications including floor system placement and manufacture easiness. The outcome of this study can form the best step to design the future experimental work that needs not only to improve the structural performance of the connection but also provide appropriate practical solutions.

VI. A possible future work could be to find an alternative solution to out-of-plane stiffeners. It is possible to act on bolts arrangements and number. Moreover, gusset plate size, shape and thickness need to be under investigation.

VII. It is suggested to better predict the behaviour of such connection, it is not enough to take into account geometry, boundary conditions and non-linear material properties. Geometrical imperfections as well as bolts behaviour mechanism are recommended to be incorporated into Finite Element Analysis as shown in previous numerical studies. Moreover, cyclic tests have been proved to have more reliable results compared to monotonic ones due to their ability of capturing the capacity degradation after experiencing seismic event.

VIII. As part of Politecnico di Torino plan to promote CFS framing to industry and help build sustainable structures as well as contribute to the society development, this study aims to create a coherent FE model of CFS bolted moment connections for future research studies and experimental work at the laboratory of Politecnico di Torino.

IX. As CFS bolted moment connections is an innovative research topic and the experimental studies in this manner are limited. Thus, Politecnico di Torino aims to focus on this area of research by performing experimental tests as part of future research work and consequently get involved in innovative research globally.

X. This study has helped initiating future research work collaboration with The University of Sheffield research group for CFS multi-storey buildings that have significant relevant history in this area by presenting and communication the outcome of this study.

XI. Based on this study, communication with suppliers as well as market leaders in the industry will be initiated to advance research in the area of CFS connections. Research outcome will be presented into research journals and conferences in Italy and over Europe to get as much as possible the attention to these structures.

XII. It worth mentioning that to better predict and monitor the behaviour of such connections in the laboratory, advanced equipment will be used to capture the geometrical imperfections so that it can be incorporated into future analytical models.

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