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A review of the behaviour and analysis of bolted connections and joints in pultruded fibre reinforced polymers

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Abstract

This paper presents a literature review of the state-of-the-art experimental and analytical methodologies adopted in the construction industry for the design of mechanically fastened connections and joints in pultruded fibre reinforced polymer framed structures. Results and conclusions obtained from the published literature relating to the effects of critical parameters, which include geometry, material properties, configuration, connecting components, fasteners, lateral restraint, etc. on the mechanical behaviour and failure modes are discussed in the light of addressing gaps in knowledge that need resolving to prepare design guidance that is reliable and robust. Further research required to improve the design of composite mechanically fastened joints is identified as a result of this review.

Keywords: Design guides, Experimental data, Numerical data, Plate-to-plate connections, Pultruded FRP, Structural joints.

1. Introduction

Fibre reinforced polymer (FRP) materials are structural engineering materials comprising of continuous fibre (say 10 μm diameter) reinforcements embedded in a continuous matrix [1]. E-glass and carbon fibres are the principal synthetic fibre materials used to manufacture FRP products for use in structural engineering. Unsaturated polyester and vinyl ester resins are the most widely used in the matrix system for producing industrial and commercial FRP composites. The fibre architecture used in the pultrusion is of a symmetric and balanced laminate. In typical off-the-shelf FRP profiles the laminate lay-up consists of the two E-glass reinforcements of continuous filament mats (or, more recently, woven fabrics) and unidirectional fibre rovings. The rovings run along the pultrusion length and serve to maximize the stiffness and the strength in that direction. The principal function of the continuous filament mats is to provide transverse stiffness and strength. Typical mechanical properties for thin-walled profiles produced by the pultrusion composite processing method are listed in Table 1.4 in [1]. For an introduction to pultruded structures and frames, the text book by Bank [1] has Chapter 13 on pultruded flexural members, Chapter 14 on pultruded axial members and Chapter 15 on pultruded connections.

A detailed review of the behaviour and design of FRP joints and connections is deemed necessary for several reasons. First, one of the major accomplishments and research advances being made in the recent European PFRPStruJoin project (PIEF-GA-2012-327142) includes new contributions to knowledge and understanding of FRP structural materials in the form of advanced finite element (FE) techniques that can be used to generate parametric studies that will form the basis for the development of simple and reliable design guidelines. Because these FE simulations will be compared with a large body of experimental data, it was felt that the original works and results should be available to fully understand the analyses and follow the discussions. Second, not all of the original works are equally accessible in the literature. Third, current FE modelling techniques will serve as a basis to develop a refined three-dimensional model of joints and connections of pultruded material that is able to capture the complete moment-rotation and force-deformation curves at a level of refinement not yet attained. Fourth, current simplified design guidelines need to be revisited. Finally, the review is also intended to serve as a point of departure for those who wish to pursue the subject matter. Therefore it is designed to offer as comprehensive a coverage as possible of the current state-of-the-art in the subject of mechanically fastened plate-to-plate connections and beam-to-column joints made from pultruded material. Accordingly, significant portions of the original works have been condensed, paraphrased or expanded for inclusion so that the review will be self-contained.

One of the most challenging aspects of composite FRP mechanically fastened joints and connections is that the well-established design procedures for steel joints and connections, based on years of experience with isotropic and homogeneous materials, have to be changed in order to accommodate the anisotropic and non-homogeneous directional properties of composite materials. Also, pultruded composites have practically none of the exceptional capabilities of metals which yield and deform to redistribute loads and thus reduce the sensitivity to local stress concentrations. The inherent brittle behaviour of (linear elastic) composites renders the joints and connections susceptible to damage prior to ultimate failure as stress redistribution by plasticity is non-existent.

Experiments investigating the effects of important parameters on the mechanical performance of mechanically fastened pultruded plate-to-plate connections and beam-to-column joints are presented in Section 2. Results and conclusions obtained from the published literature relating to the effects of critical parameters, which include the joint or connection configuration and geometry, the composite material parameters, fastener configuration and joint geometry, on the mechanical behaviour and failure modes of mechanically fastened joints are discussed. The special topic of environmental effects is not included in this review.

Numerical procedures for the prediction of the behaviour of pultruded plate-to-plate connections and beam-to-column joints are presented in Section 3. These numerical results were restricted to the linear elastic domain and were mostly used to supplement test data and give a more accurate description of the loading paths in the connections components in this domain. The application of more sophisticated FE techniques that include the use of progressive damage analysis are discussed.

Section 4 summarizes current provisions and procedures for connection and joint design. It is shown that current guidelines are clearly insufficient to provide confidence in the use of this new family of materials for primary structural applications.

Further research on improving the design of composite mechanically fastened joint is discussed in the last section. This includes the numerical effort required to improve the accuracy and reliability of static resistance prediction methodologies as well as the research work required to provide a data base which is essential in developing scientifically founded rules to be included in design guidelines and codes of practice.

One difficulty experienced in collating published work was the wide variety of ways to define a number of parameters and concepts. In particular,a distinction between the concepts of joints and connections is necessary, as both terms are used interchangeably in practice. In this review, the EUR 18172 [2] and the EN 1993-1-8 [3] terminology is adopted:

- A *connection* is the location at which two or more elements meet; for design purposes it is the assembly of the basic components required to represent the behaviour during the transfer of the relevant internal forces and moments at the connection;
- A *joint* is the zone where two or more members are interconnected; for design purposes it is the assembly of all the basic components required to represent the behaviour during the transfer of the relevant internal forces and moments between the connected members.

In other words, the connection is the physical component which mechanically fastens teo plates or a beam to a column, and is concentrated at the location where the fastening action occurs. The joint is the connection plus the corresponding zone of interaction between the connected members, namely the panel zone of the column web in a beam-to-column joint.

2. Experimental tests on pultruded bolted joints

Previous research pertaining to the experimental behaviour of bolted connections made from FRP composites generally concentrates on aeronautical and aerospace applications. In this engineering sector composites are dominated by FRP reinforced with continuous carbon fibres in an epoxy matrix. The experience and understanding of physical testing of structural connections commonly used in the aerospace industry has proved to be valuable and has helped to develop simple design rules for composites in construction, such as those included in the Eurocomp Design Code and Handbook [4]. However, there is a need to focus on the unique features of pultruded composite materials. These materials are different in structure compared to advanced aerospace composites (laminates). Another additional feature of pultruded connections used in the construction sector relates to the significance of the through-thickness effects on the overall behaviour. In composite laminates, the ratio of the hole diameter to the thickness of the material is high enough to induce very little deformation of the bolt through the thickness of the material, whereas in civil engineering applications the members are relatively thick and the through-thickness effects may become relevant. Although research into the behaviour of the latter class of FRP is scarcer, there are currently more than 1000 test results reported in the literature. A comprehensive summary of these tests is presented in what follows.

The physical characteristics that can influence the behaviour of structural joints and connections include:

1. Geometric parameters such as the ratio of width-to-hole diameter w/d_0, the ratio of end distance-to-hole diameter e_1/d_0, plate thickness t, edge distance e_2, pitch and gauge length for the bolting, p_1 and p_2, respectively (Fig. 1).
2. Material parameters, including fibre types and form, matrix type, fibre orientation and laminate stacking sequence.
3. Configuration, single- or double-lap (one and two shear planes, respectively), single- or multi-bolted, number of bolt rows, number of bolts per row, among other factors.
4. Fastener parameters, including type and hole clearance.
5. Lateral restraint from bolt tightening or clamping area.
6. Design parameters, namely the loading type and direction, failure criteria.

2.1 Plate-to-plate connections

Previous short-term (static) experimental studies mostly consider single-bolted connections and concentrated on the various failure modes and the associated resistance. In-plane failure of plate-to-plate connections can be governed by four possibilities:

1. Bearing failure (B), which is characterized by failure close to the contact region at the hole edge (Fig. 2a). Bearing is likely to occur when the ratio w/d_0 is high. This mode of failure is strongly affected by the lateral constraint that delays ply delamination, i.e.

splitting between layers in the through-thickness direction. It may result in a progressive deformation with continuous localized buckling of the fibres and crushing of the matrix.

2. Net-tension failure (NT), which is characterized by sudden crack propagations transversely to the direction of the connecting force, due to a relatively small area of the plate cross-section (Fig. 2b). This is the dominant mode in multi-row bolted connections because of the high stress concentrations at the first row of bolts [2].
3. Shear-out failure (S), which occurs when the ratio e_1/d_0 is small (say less than 4). This mode of failure is mainly caused by shear stresses and occurs along shear-out planes on the hole boundary in the principal fastener load direction (Fig. 2c). In most cases, shear-out failures in single bolted row connections are a consequence of bearing failure with a short end distance.
4. Cleavage failure (C), which is essentially a mixed net-tension and shear-out failure (Fig. 2d).

Experimental research has shown that the bearing failure mode can be characterized by progressive damage growth and the other three modes are typically of a brittle nature. Thus, the bearing mode of failure possesses the highest deformation capacity and may offer a damage tolerant bolted connection. Most studies have therefore focused on the quantification of the critical ratios e_1/d_0 and w/d_0 that ensure that this failure mode is most likely to be decisive, rather than the brittle shear-out and net-tension modes.

In the following sub-sections, experimental results and conclusions obtained from published literature relating to the effects of various parameters on pultruded composite mechanically fastened joints are presented and discussed [5]. For convenience, the parameters are divided into the principal groups that influence the connection behaviour.

2.1.1 *Influence of geometry*

One of the best-recognized experimental studies on plate-to-plate connections is that of Rosner [6] that is generally acknowledged as the pioneer work with a single bolt. Rosner, under the supervision of Rizkalla [7,8], conducted a total of 102 tests in a double-lap arrangement. In the test set-up, the outer plates were of pultruded flat sheet and the inner plate was made from structural steel, so that two single-bolted tension connections were tested in parallel. The parameters varied in the study were the thickness t of the FRP plates (9.5 mm, 12.7 mm and 19.1 mm), the ratios w/d_0 (from 1.2 to 12.3) and e_1/d_0 (from 0.9 to 1.9), and the angle between the roving and tension directions that is analysed in Section 2.1.2. In this sub-section, only the results for those specimens with fibres parallel to the tension axis (the 0° direction) are considered. The flat sheet consisted of symmetrically stacked, alternating layers of identically oriented unidirectional E-glass rovings and randomly oriented continuous filament mat in a polyester matrix [6]. High-strength steel bolts 19 mm in diameter were tightened to a constant torque of 32.5 Nm in 1.6 mm clearance holes. This torque level corresponds to what Strongwell (previously MMFG) recommended for their glass FRP threaded rods [6]. The authors used this installation torque even though the fasteners were high-strength steel bolts so that a comparative study could be made with the different types of fasteners in a subsequent investigation. Several failure modes were observed in the test series.

The variation of the connection resistance with the ratio e_1/d_0 for different ratios of w/d_0 is shown in Fig. 3. The plots show that the load-carrying capacity is mostly influenced by the plate width and tends to level off past $w/d_0 > 5$. This parameter as well as the end distance also influences the failure mode. Rosner showed that (see Fig. 3):

1. For connections with relatively small end distances ($e_1/d_0 \approx 1$), the mode of failure changed from net-tension to cleavage as the width increases.
2. For large end distances ($e_1/d_0 > 4$), net-tension failure changed to bearing failure as the plates became wider.
3. When $e_1/d_0 < 4$ the mode of failure tends to change from bearing to cleavage failure.

Finally, the results shown in Fig. 4 suggest that the connection resistance increases linearly with the plate thickness. Based on the test data, Rosner and Rizkalla [8] proposed simplified formulae for the prediction of the load-carrying capacity of single-bolted connections.

Abd-El-Naby and Hollaway [9] tested 97 plate-to-plate connections in a double-lap configuration with different geometry (variation in width and end distance). The (unconventional) pultruded plate material was manufactured from E-glass fibres, with high volume fraction in the axial direction, in a polyester resin, and surrounded by two continuous filament mat layers. The bolt diameter was 9.5 mm and all bolts were hand-tightened. Clearance of the hole was not specified. In the test set-up, the inner plate was pultruded and the outer plates were made from steel, in the majority of tests. Some tests were conducted with the three plates made from pultruded material. The principal objective of the tests was to determine the critical end distance for connections made from an FRP with high proportion of axial fibres. This critical distance was defined as the end distance at which a connection of fixed width reached its maximum possible resistance. The configurations were divided into two series, with different fibre volume fractions of uniaxial and continuous filament mat layers (material G and Y). Each series focused on different combinations of the above geometrical parameters. Test results showed that an increase in the end distance would increase the resistance of the connection until a critical value had been reached (Fig. 5 – $F_{u,av}$ is the average of a batch of three tests). This critical value also increased with the plate width. The authors also highlighted the significance of the lay-ups on the failure mechanisms. In general, those specimens with increased thickness of the continuous filament mat layers allowed the plate to redistribute the stresses more evenly.

Turvey and Cooper [10] reported data derived from 63 double-lap single-bolted tension connections. In their test set-up, the inner plate was composite and the outer ones were mild steel. The composite plates were cut from the web and the flanges of standard structural wide flange pultruded sections. The plate material was 6.4 mm and 9.5 mm thick. 10 mm and 12 mm diameter mild steel bolts were used in the smaller and larger thickness material, respectively. Bolts were lightly torqued to 3 Nm, simulating the hand-tightened condition and were neat-fit to minimize the hole clearance. The principal objective of this set of experimental tests was to determine the resistance and the elastic stiffness S_{el} of the connections using different geometries. The ratio w/d_0 and e_1/d_0 were each varied from 2 to 8. Based on test results, the authors recommended the following critical ratios that ensure, for the pultruded material characterized, that failure occurred in the bearing mode:

1. For wide plates ($w/d_0 \geq 7$) with thickness of $t = 6.4$ mm, $4 < e_1/d_0 < 6$ and for $t = 9.5$ mm, $e_1/d_0 \approx 6$.
2. For plates with large end distances ($e_1/d_0 \geq 6$) with $t = 6.4$ mm, $w/d_0 \approx 4$ and for $t = 9.5$ mm, $4 < w/d_0 < 6$.

Wang's [11] experimental studies included compressive and tensile bearing tests with the following objectives: (i) to determine the bearing strength and the factors that affect the failure mechanisms, and (ii) to evaluate the effect of additional layers of plain woven fabrics on connection resistance and modes of failure. He used pultruded plates of 3.2 mm thickness fabricated with E-glass fibres in a vinyl ester matrix. Some pultruded samples were further reinforced with woven fabrics. In both cases, the specimens were cut (i) along the material longitudinal direction and (ii) along the material transverse direction (the 90° direction). The results for the tensile bearing behaviour are taken for further analysis. Wang conducted a total of 43 tests in tension (23 specimens were made from pultruded plates and 20 specimens were reinforced with surface layers of fabric). The bearing load was applied through a pin held by steel bars. There was no lateral constraint. The geometric parameters considered in his experiments were width, end distance and hole diameter. Two hole diameters were selected, namely 6.4 mm and 12.7 mm. The ratios e_1/d_0 and w/d_0 varied as follows: $1 \leq e_1/d_0 \leq 5$ and $2 \leq w/d_0 \leq 8$. The key points from the study are:

1. Ratios $e_1/d_0 > 1.5$ had little influence on the failure strength.
2. An increase in the geometric ratio w/d_0 corresponds to an increase in the ultimate connection resistance.
3. The bearing strength, defined as the relationship between the applied load and the projected cross-sectional area, was significantly influenced by the size of the hole: as the hole size enlarged, the bearing strength decreased. This can be explained by the fact that the actual failure zone did not increase with the hole size.
4. The connection resistance was higher in the tests performed with the specimens cut along the longitudinal direction than those cut along the transverse direction.
5. The bearing strength at failure was increased with the added woven fabric reinforcement to the pultruded plates. This trend was found to be dependent on the specimen geometry and direction.
6. All four failure modes were observed in the tests, as highlighted by the plotted test results in Fig. 6. Bearing failure was observed for pultruded specimens with $e_1/d_0 \geq 1.5$ and $w/d_0 \geq 4$ along the longitudinal direction; most specimens tested in the transverse direction failed in a net-tension mode. Only the relevant portion of the curves in Fig. 6 (for $F_u > 4$ kN or $F_u > 3$ kN, whereby F_u is the ultimate resistance) is shown to increase the resolution of the plots.

In a recent experimental investigation, Turvey [12] and Turvey and Godé [13] evaluated the ultimate resistance of single-lap single-bolted connections subjected to tension. In the test configuration (for batches of 3), both plates were pultruded. A set of 45 tests was completed for 15 different combinations of the ratios w/d_0 and e_1/d_0. The pultruded plate had a constant thickness of 6.4 mm. The rovings were parallel to the load direction in all tests. 9.8 mm diameter mild-steel bolts in 10 mm holes were used. Bolts were lightly torqued. It can be seen from Fig. 7 ($F_{u,av}$ is the average of a batch of three tests) that the connection resistance is uniform for $e_1/d_0 > 2.5$. For a combination of $e_1/d_0 > 2.5$ and $w/d_0 > 4$, the actual resistance is virtually the same. The authors also found that excessive bolt rotations about an axis passing through the plate where the two plates contacted and internal delamination were the characteristic features of single-lap failure modes. The crack patterns were found to be more complex than those observed in symmetric double-lap configurations.

2.1.2 *Influence of the angle between applied tension and pultrusion direction*

Rosner [6] and Rosner and Rizkalla [7], whose work also covers the effect of the fibres orientation, concluded that the resistance of the 0° connections was higher than the 45° and 90° counterparts. The overall behaviour of connections with different fibre-to-load orientation was very similar.

Fifty-four tests on hand-tightened single-bolted tension connections in 6.4 mm thick pultruded plates were reported by Turvey [14]. In these tests Turvey explored the effect of off-axis angle between the rovings and tension directions, for different connection geometries. The angles were 90°, 45° and 30°. For completeness the test data collected in Cooper and Turvey [15] for an angle of 0° is also considered. The test set-up is identical to that adopted in previous test series at Lancaster University. Four ratios w/d_0 of 4, 6, 8 and 10 were considered. High-strength steel bolts M10 in neat-fit holes (for 0.1 to 0.3 mm clearance) were used. The bolt shank was in contact with the plates. The comparison results in terms of resistance, modes of failure and elastic stiffness are shown graphically in Fig. 8, as a function of e_1/d_0 ($F_{u,av}$ is the average of a batch of two tests). Comparisons are only shown for the orthogonal loading directions of 0° and 90°. It can be seen in Fig. 8a that for $w/d_0 = 10$, specimens oriented at 90° always fail in a net-tension mode and the resistance level increases with the end distance ratio. For the other case, the resistance level becomes steady for $e_1/d_0 > 4$ and the bearing failure mode dominates. For the other width ratios, the information is rather incomplete and no inference on the connection behaviour can be made. Fig. 8b suggests that for the largest w/d_0 ratios, the average elastic stiffness is reasonably independent of the e_1/d_0 ratio.

The graphs in Fig. 9 show that the bearing failure mode does not occur even for small off-axis angles, say 30°, for a constant end distance ratio of 6. It is also shown that both resistance and stiffness steadily decrease with the off-axis angle for any of the width ratios considered in the study.

Yuan and Liu [16] conducted experimental studies in which the alignment of fibres with respect to the loading direction was varied to ascertain their significance on the overall behaviour of connections with the double-lap configuration. They used pultruded nine-layer flat sheet of 9.5 mm thickness. The matrix polymer was an isophalic polyester. Stainless steel bolts were employed in the tests. The bolt was loosely tightened. The hole clearance was lower than 0.8 mm (close fit situation). The geometry of the connection was defined to confine failure to the bearing mode, which might be damage tolerant as the connection accommodates further deformation after the damage load is reached. In this context, the following geometry was chosen for all tests: w/d_0 = 7 and e/d_0 = 3. The angle between the load and the pultrusion direction was varied as follows: 0°, 15°, 30°, 45°, 60°, 75° and 90°. The load-deformation response for all tests was characterized by an initial linear behaviour up to the damage load level, followed by a reduction in stiffness due to progressive damage growth. Test results show that both damage and ultimate resistance of the bolted connection decrease as the load direction changes from 0° to 90°. There is also a change in the failure mode: bearing failure mode dominates the behaviour for angles lower than 45° whilst for orientations greater than 45°, the net-tension failure mode is observed. Finally, the authors concluded that the failure load level varies linearly with the fibre-to-load orientation.

2.1.3 *Influence of fastener parameters*

Erki [17] tested 28 tension connections in double-lap configurations with both inner and outer plates being pultruded flat sheet. The thickness of the main plate was 25 mm and the side plates 13 mm. She used three different types of fasteners: glass FRP threaded rods, steel threaded rods and steel bolts, all having 19 mm diameter. Fastener hole clearance was 1.6 mm in all cases, following the American practice. She assessed the effect of the fastener strength and stiffness on the connection resistance. It was shown that the connections fabricated with FRP threaded rods exhibited half the resistance of the connections fabricated from steel threaded rods. The main conclusions of the experimental work can be briefly summarized as:

1. If the fastener is stronger than the plate material (e.g. steel threaded rods), the failure modes and the load-carrying capacity are determined by the mechanical properties of the FRP plates.
2. If the fastener is weaker than the plate material (e.g. FRP threaded rods), the failure modes and the load-carrying capacity are mainly influenced by the fastener with little damage of the FRP plates.

Yuan et al. [18] carried out an investigation of bolt hole clearance effect on the bearing capacity. The authors tested 25 double-lap connections with a 9.5 mm thick plate and bolts with 12.7 mm diameter in a hole clearance that varied from "no-clearance" (≈ 0) to 6.4 mm, in increments of 1.6 mm. The "no-clearance" situation corresponds to a tight-fit clearance, say of 0.1 mm to 0.3 mm. The bolt was torqued to the hand-tight state. They found that for a clearance above 1.6 mm there was a substantial decrease in connection strength with increasing clearance, as can be seen in Fig. 10 ($F_{u,av}$ is the average of a batch of five tests). For the American recommended clearance of 1.6 mm, the tests by Yuan et al. showed just a 2% load reduction compared to the "no-clearance" situation.

2.1.4 *Influence of lateral restraint*

Several experimental studies [9,15,16,18] have underlined the effect of lateral constraint for torqued bolt conditions, the effect of clamping load and the effect of secondary bending on the potential to shift the failure mode and affect the connection resistance. Some of these contributions are reviewed in this section.

Abd-El-Naby and Hollaway [9] tested 11 connections in a double-lap arrangement with the same geometry (w = 70 mm and e_1 = 40 mm). The bolt diameter was 9.5 mm and all bolts were hand-tightened. They considered three specimen groups, each being subjected to a different clamping condition. The first group used non-standard tight-fitting steel washers; the second and third groups used plates that covered the entire potential damage area. Specimens in the second group used steel plates, and smooth composite plates clamped the specimens in the third group. Abd-El-Naby and Hollaway found that (i) the connection strength increased with the confinement area and (ii) the bolt displacement was reduced by replacing the washer with a steel plate and even further reduced by replacing the steel plate with a composite plate. The specimens initially failed in bearing and ultimately by cleavage. The load-deformation curves were different. For those specimens confined using a washer, the load-deformation curves had a constant initial stiffness with a sudden drop in load at a relatively low deformation after initial (damage) failure. Further load was sustained by the connections with increasing deformations. The load-deformation behaviour of the other specimens was characterized by a nonlinear response from the commencement of loading. An initial stiff phase was followed by a second phase of much reduced stiffness after damage initiation. This suggests that material failure developed gradually and progressively.

Cooper and Turvey [15] carried out 81 tests on M10 bolted connections in double-lap configuration under three different clamping conditions: pin-bearing condition (hand-tightened bolts), lightly clamped (3 Nm) and fully clamped conditions (30 Nm). In addition, they varied the ratios e_1/d_0 and w/d_0 to get insight into the influence on failure of the geometry for different levels of bolt clamping torque. All specimens were made from 6.4 mm flat sheets having a polyester-based matrix. Cooper and Turvey analysed the results with respect to connection stiffness and resistance. They further defined two different levels of resistance: damage resistance (or incipient failure load, [15]), corresponding to the level of stiffness reduction, and failure load, corresponding to ultimate failure. At the damage load level, proposed as the design resistance, irreversible damage to the connection specimen was observed and worsened progressively until rupture. The four possible in-plane failure modes introduced in Fig. 2 occurred under the three clamping conditions, depending on the connection geometry. The authors eventually recommended critical ratios $e_1/d_0 = 3$ and $w/d_0 = 4$ for the specific flat sheet material to achieve a bearing failure mode. They also proposed a design load level for the tested plate of 15 kN, based on the lightly clamped conditions. Suggestion was made to fabricate the joint with bolts fully tightened to around 30 Nm, which corresponds to the assembled fully clamping condition of the experiments.

In their work on double-lap single-bolted connections, Yuan and Liu [16] also investigated the effect of variations of bolt torque. They used the same test configurations described in sub-section 2.1.2. In this new series, they varied the bolt torque from 0 to 34 Nm, in a total of six tests. All connections with applied torque showed signs of bearing failure and eventually failed in a brittle failure mode. Results also indicate that the connection resistance increases with the level of bolt torque.

2.1.5 *Multi-bolted connections*

The general objectives in those tests on multi-bolted connections (see Fig. 1) were to characterize (i) the failure modes when there are two or more rows of bolts and (ii) the load transfer mechanism between rows. An accurate load distribution is not possible by experimental testing and requires the use of advanced FE analyses that take into account the most influential factors.

Abd-El-Naby and Hollaway [19] presented and discussed experimental results for the behaviour of double-lap connections (21 tests) using two 9.5 mm bolts in series, i.e. two bolt-rows having one single bolt per row, considering both effects of changing the relative stiffness of the plates and the different failure modes. Bolt pitch, p_1, was kept constant in all tests. The main objective of these tests was the characterization of the mechanisms of load transfer and modes of failure. The authors observed that the load was gradually transferred from the first row to the second row, as defined in Fig. 1, and ultimately the load share of the second row of bolts became higher. For those specimens that failed in bearing, the connection damage tolerance was expected to result in equal loads on both bolts. Additionally, Abd-El-Naby and Hollaway concluded that the two-bolted connection efficiency was similar to that of a geometrically equivalent single-bolted connection, as long as failure was governed by bearing.

The work of Prabhakaran et al. [20] was aimed at studying the failure modes and developing design equations. Test specimens were fabricated from 12.7 mm thick plates and used high-strength steel bolts, with a diameter of 15.9 mm in 0.13 mm clearance holes. Bolts were hand-tightened. Specific configurations were selected to highlight the net-tension and block shear failure mechanisms. A total of 18 tests were carried out in a double-lap configuration (three tests per batch). Five different series with different number and arrangement of bolts were considered, and they are:

- Prabhakaran A: one single bolt (1×1 bolts);
- Prabhakaran B: two bolt-rows and one single bolt per row (2×1 bolts);
- Prabhakaran C: one bolt-row and two bolts per row (1×2 bolts);

- Prabhakaran D: two bolt-rows and two bolts per row in a square pattern (2×2 bolts);
- Prabhakaran E, F, G: two bolt-rows and two bolts per row with staggered holes.

Test results show that the resistance for the 2×2 bolts increases when compared to the other configurations, especially against the single-bolted configuration. For the two-bolted configurations (Prabhakaran B and C), the resistance is identical, although the failure mode changes from a pure shear-out to a mixed net-tension and shear-out mode. The response of the connection depends significantly on the number of bolts, although the increase in resistance is not directly proportional. The authors also analysed the staggering effect of a 2×2 bolt configuration with the original 2×2 bolt arrangement (Prabhakaran D). They concluded that the staggering effect was detrimental in terms of resistance.

The authors further developed a Load and Resistance Factor Design (LRFD) approach to estimate the connection resistance and the respective resistance factors that were calibrated against test data. Later, Prabhakaran and Robertson [21] extended this experimental work to another series of tests to investigate the load shared by the bolts and to assess the influence of bolt tightening and clearance on the load distribution. Specimens predominately failed in a net-tension mechanism. Test results showed that (i) the tightening torque had a marginal effect on the bolt load distribution and (ii) an increase in the clearance at the heaviest loaded bolt hole resulted in a larger overall load capacity up to a certain clearance limit.

The effect of the most influential parameters, width and end distance, bolt pattern, number of bolts and connection force orientation on the behaviour of multi-bolted double shear lap connections was investigated experimentally by Hassan and co-authors [22]. They conducted 105 tests, using the same test methodology as in Rosner [6], grouped in five different series, each series corresponding to a different bolting configuration:

- Hassan A: two bolt-rows and one single bolt per row (2×1 bolts);
- Hassan B: one bolt-row and two bolts per row (1×2 bolts);
- Hassan C: three bolt-rows and one single bolt per row (3×1 bolts);
- Hassan D: one bolt-row and three bolts per row (1×3 bolts);
- Hassan E: two bolt-rows and two bolts per row (2×2 bolts).

Standard 19 mm high strength structural bolts in 20.6 mm drilled holes were used for all connections. Bolts were tightened to a constant torque of 32.5 Nm. The pitch and gauge lengths were a constant parameter at four times the hole diameter. The pultruded plate material had a constant thickness of 12.7 mm. The load was applied at material orientations of 0°, 45° and 90°. The authors found that the modes of failure were mainly influenced by the width and the end distance and the orientation relative to the load (Figs. 11-13). To alleviate non-uniform load share per bolt in a row, these researchers took specific care when assembling the connection specimens to locate the centre of the bolt with the centre of the drilled hole. In order to predict the load distribution among the bolts, the specimens were instrumented with strain gauges. The results showed that in series B and D, with one single bolt row, the bolts shared the load equally. This was not observed for the other connections and it was found that the load share was determined by the bolt arrangement. The main conclusion of this experimental study was that the increase in the resistance and efficiency of the connection was not proportional to the increase in the number of bolts, especially if this increase is accomplished by adding rows of bolts, as can be seen in Figs. 12a and 13a and Figs. 12b and 13b. For example, results from Figs. 12a and 13a for e_1/d_0 = 5 and w/d_0 = 14.8 (F_u = 312 kN and 390 kN, respectively) show a minimal increase of 25% when an extra second row with two bolts is added to the connection configuration. This means that the second row of bolts carry much less load than the first. As a follow-up study, Hassan et al. [23] proposed a rational model to predict the design resistance of multi-bolted lap connections and the respective failure mode.

2.1.6 *Principal conclusions*

Several conclusions are drawn from this experimental review on the behaviour of plate-to-plate tension connections:

1. Plate-to-plate connections should be designed, if practical, so that the governing failure mode is the damage tolerant mode of bearing.
2. The off-axis angle between the fibres and the tensile load reduces the load-carrying capacity of the connection.
3. Although any bolt torque increases the connection capacity, over-tightening may damage the surface polymer matrix and result in premature damage of the material.
4. The resistance of multi-bolted connections is not necessarily found by multiplying the resistance of a single-bolted connection by the total number of bolts.

The main design implications are:

1. Based on the tests by Rosner [6], the following ratios are recommended $e_1/d_0 > 5$ and $w/d_0 > 7$. These results are rather conservative, especially when compared with other test results that used thinner pultruded plates.
2. Plate-to-plate connections should be designed for the bolt hand-tightened condition (no lateral restraint), in which there is little lateral restraint. The bolt installation torque should however correspond to a snug-tight condition, in order to increase the connection resistance. This is also explained in detail in Mottram and Turvey [24], who clearly state that the stress relaxation over time due to viscoelastic creep will reduce any pre-load such that the through-thickness compressive force reduces to an extent that cannot be predicted.
3. The bolt shank should be in contact with the laminate and not the threaded portion of the bolt, as standard practice when steel bolts are used. This issue has already been addressed by Mottram [25].

Some relevant issues were exposed during this investigation that warrant further consideration. They are listed below and are proposed as future research:

1. Further tests that supplement crucial data on the mechanical properties of specimens should be conducted, namely fracture toughness, and strength of the matrix and of the rovings and mats.
2. Current test results do not account properly for the plate thickness variation and this is one of the most influential parameters in connection behaviour. This clearly influences the through-thickness behaviour that is crucial when a complete characterization of the connection response is needed.
3. Current test data do not provide quantitative guidance for minimum ratios of p_1/d_0 and p_2/d_0.
4. Current test data do not provide quantitative guidance for maximum ratios of e_1/d_0, w/d_0, p_1/d_0 and p_2/d_0.
5. A clarification on staggered bolt arrangements is also needed.
6. A complete characterization of the available connection damage tolerance is timely. This requires a combined experimental-numerical research programmeme to get full insight of the problem and ultimately propose guidelines to the required connection damage tolerance.

2.2 Beam-to-column joints

Structural joints in framed structures exhibit a distinctively nonlinear behaviour when deformed by the moments and forces they transfer between members. This nonlinearity arises because a joint is an assemblage of several components that interact differently at distinct levels of applied loads. The interaction between the elemental parts includes the inherent characteristics of the material, contact, slip and separation phenomena. The analysis of this complex behaviour is usually approximate in nature with drastic simplifications. Tests (both experimental and numerical) are frequently carried out to obtain the actual response, which is then modelled approximately by mathematical expressions that relate the main structural joint properties.

Beam-to-column joints have to transfer the beam and floor loads to the columns. Generally, the forces transmitted through the joints can be axial and shear forces, bending and torsion moments. Therefore, in the structural analysis of joints a clear understanding of the load paths, i.e. the exact mechanism by which various components of the joint itself transfer load through the connection is essential. This is now firmly established for bare steel joints and so the individual bolts, welds, plate elements, etc. can be properly arranged. The various modes of failure have been identified and the general aspects of joint behaviour are well characterized. Because of this sound knowledge, most pultruded joint configurations are copied from steel design practice. However, the intrinsic characteristics of the pultruded material can be an obstacle to the efficiency of joint details inspired from this pragmatic approach.

In the framework of a limit state design concept, considerations other than static resistance (ultimate state) have to be taken into account. The design of pultruded frames, as well as the design of many steel frames, particularly those made from high strength steel (yield stress > 460 MPa), is controlled by stiffness, in the form of deflections or drift limits complying with serviceability limit states. In this context, the use of the semi-continuous/partially-restrained analysis and design seems appropriate. This requires a more comprehensive understanding of the joint response, in terms of moment-rotation (M-ϕ) characteristics, when compared to the traditional ideas of pinned or rigid joints that are often misleading in terms of the deformations and pattern of internal forces that actually develop [26]. The semi-continuous/partially-restrained behaviour can be illustrated by considering a specific example. Fig. 14a shows a uniformly loaded beam segment with semi-rigid/partial-strength connections at both ends. The term *semi-rigid* applies to the actual joint behaviour and cannot be defined straightforwardly for pultruded joints and, more importantly, univocally. In fact, a joint should not be classified *per se* but on the basis of the influence of its behaviour on the response of the whole structure. EN 1993-1-8 [3] defines criteria for such a stiffness classification. The term *partial-strength*, on the other hand, has a rather trivial definition. The resistance of a partial-strength joint is less than that of the connected members. In the case of pultruded members, the beam resistance can be taken as the elastic moment of resistance. In Fig. 14b the beam end moments are limited to the joint moment capacity, $M_{j,c}$. The beam is thus designed for less that the mid-span moment $\left(qL_b^2/8\right)$ that would be developed assuming it to have pinned-ends. Similarly, since the beam-to-column connections will possess some degree of initial rotational stiffness, $S_{j,ini}$, end restraint to the beam will reduce the span deflection from the simply supported value $\left(5qL_b^4/384EI\right)$, see Fig. 14b. Reductions in deflection due to the inherent stiffness of the joint may well allow designing the beam to carry a higher load at SLS deflection limits, and it should be noted that this is a clear economic advantage in terms of pultruded construction. Once $M_{j,c}$ is attained (Fig. 14c), further load is accommodated by further rotation of the connections and, progressively, more moment is transferred into the span to permit the development of the beam's full sagging capacity, $M_{b,max}$ (Fig. 14d). This moment redistribution requires an amount of rotation of the joint that is likely to be a limiting factor. An accurate prediction of the connection rotation capacity is thus crucial (see the recently published work by Girão Coelho [27]). The definition of $M_{j,c}$ for pultruded joints is not well established. Mottram [28] introduced the "First failure" (currently known as "Damage failure") concept to define the joint moment and rotation values at which material damage (usually of the connecting pieces) is severe enough to expose fibres. This is a rather empirical concept and has been established by visual observation of joint tests [29].

The principle of semi-continuous/partially-restrained design as applied to a single beam segment recognizes that the performance of structural analysis requires the knowledge of the M-ϕ response of the end connections, or at least the ability to approximate the key parts of the curve adequately. This M-ϕ curve describes the relationship between the applied bending moment M and the corresponding rotation ϕ between the members and defines three main structural properties: (i) moment resistance, (ii) rotational stiffness and (iii) rotation capacity. The M-ϕ behaviour of pultruded joints under short-term loading is typically nonlinear. This nonlinearity is not due to material plasticity as in steel (pultruded materials behave elastically up to failure) but rather as a result of progressive material damage and geometric changes in the joint. This behaviour is generally characterized by an initial elastic phase that is followed by a second phase of much reduced stiffness. The initial phase can be represented by a simple linear approximation which is the average of the behaviour up to damage onset. If joints are damage tolerant and can develop sufficient deformation, the ultimate failure, i.e. the point at which no further load can be applied, will be reached at which very large rotations (say > 30 mrad) will be achieved at a virtually constant moment. The collapse mechanisms of pultruded joints, which are not yet fully understood, often involve failure of the web-flange junction(s) of the profiles.

The M-ϕ behaviour of pultruded beam-to-column joints is most conveniently obtained by physical tests. Although few test series have been conducted worldwide and the availability of good quality, carefully documented test data is scarce, the authors will attempt to assemble the most relevant data into usable collections in this sub-section of the review topic. Only major axis joints are considered below. Two comprehensive background reviews on this specific subject have been published by Turvey and Cooper in 2004 [30] and Mosallam in 2011 [31]. The present review offers the first quantitative summary of the input data for future analytical work or for interpretation of tests. The principal objectives of the tests conducted on pultruded beam-to-column joints can be summarized as follows:

1. Acquisition of the M-ϕ response data from joint tests.
2. Observation of damage progression.
3. Characterization of the mode(s) of failure.

2.2.1 *Joints with angles*

Fig. 15 shows common structural details of connections joining pultruded frame members include web angles (Fig. 15a), flange angles (top and seat angles – Fig. 15b) and flange and web angles (Fig. 15c). The connecting elements can be of pultruded or steel grade material. Bolted web angle connections are simple and allow easy site installation. They are formed by bolting short angle cleats (or clips, as known in North America) between the web of the beam and the column face. Beams are normally attached to the angle cleats using two or more bolts through the web. It is a neat connection type for rectangular (orthogonal) grids in which the beams and columns all meet at right angles. Flange cleats provide a direct bearing for the beam. The use of a seating cleat provides a connection that is quick and easy to erect because the beam member can be positioned directly onto the support angle. It can be fully bolted. One disadvantage is that the seating cleat can impact on the ceiling finishes at the column position. A cleat can also be fixed to the top flange to provide additional restraint against twisting of the beam. The three connection typologies exhibit different responses when subjected to short-term loading, as shown later.

The experimental investigations on design oriented joints obtained from published literature that relate to the effects of various parameters are now presented and discussed. For convenience, the results and major findings are summarized in the form of tables. Column entries in Tables 1-3 are organized as follows: (1) test reference, (2) joint typology and angle dimensions, (3)-(5) experimental results in terms of the principal characteristics, stiffness, resistance and rotation capacity, (6) description of the observed failure modes, (7) comments on test results or some specific details of the test, (8) reference of the relevant research work, (9) beam and column sections used in the joint test, (10)-(13) present information on the bolts and their installation, (14) set-up of the test, and (15) objectives of some specific tests. Values presented in the tables are those from the corresponding authors.

Bank and his associates [32-34] conducted the first test programme on joints with bolted angles to characterize the M-ϕ response of the different configurations and see whether they had adequate performance for semi-rigid action. A summary of the test information is given in Table 1. All tests used $203 \times 203 \times 9.5$ mm wide flange profile beams and columns. A gap of 5 to 10 mm was left between the end of the beam and column flanges. Fasteners were FRP threaded rods, all having 19 mm diameter, tightened to a torque of 41 Nm. Fastener hole clearance was not specified. All connecting elements were made from pultruded material. The authors used the same simple test set-up of direct compression across the joint in the experimental programme. The first three tests (Bank 1 to Bank 3) were essentially aimed at evaluating the initial stiffness. Tests Bank 4 to Bank 11 were loaded to failure to characterize the full M-ϕ response and identify the modes of failure of the connections. Bank 5 to Bank 11 were conducted by sequentially modifying an original typology [33,34]. The following principal ultimate failure modes were observed in this second group of tests:

1. Tensile tearing of the column flange at the flange/web junction.
2. Combined flexural-tensile cracking in the heel of the angle bolted to the tension flange of the beam.
3. Specific failure modes of the non-standard connecting elements.

The M-ϕ responses for joints Bank 4 to Bank 7 is given in Fig. 16 for joints Bank 4 to Bank 7 as polynomial regression curves from the experimental data. Table 1 presents a compendium of the main test features and results. As can be seen, an improvement in the connection performance was obtained with the successive redesign of the original connection Bank 4. By adding an angle stiffener to the column (Bank 5), the ultimate resistance was increased by a factor of 1.4 and the rotation capacity increased 1.6 times. The rotational stiffness did not show much improvement in magnitude, especially in the virtual elastic range. Similar observations are drawn for joint Bank 6, now with increases of 1.3, 2.1 and 2.3 times with respect to initial stiffness, ultimate resistance and maximum rotation. The difference in the stiffness properties in joint Bank 7 among the different configurations is easily recognized from the M-ϕ plots of Fig. 16. Although this aspect can be relevant for serviceability requirements, it can also be observed that this improvement occurs at the expense of the joint deformation capacity. This can be a serious drawback in terms of joint performance, even at SLS. The fact that the connection structural details are impractical is worthy of mention.

A comprehensive study of the behaviour of pultruded cleated joints was produced at the University of Warwick, under the supervision of Mottram [29,35-39] following an initial experimental programme on the behaviour of web cleat connections as part of the project EUREKA EU468: Eurocomp [28,40]. The important feature of these studies was the identification of the modes of failure and the verification of the adequacy of some connection details. Findings from these research investigations are particularly interesting from a practical viewpoint.

The structural details of the joint tests are given in Table 2 (L: left connection and R: right connection) and the main characteristics of the tests are summarized in Table 3. As can be seen, tests Mottram 1 to 3, identified that their weak component was the top cleat. Mottram and Zheng [36] then proposed a bespoke *L* shaped top cleat piece with a gradual heel and appropriate fibre placement, manufactured by vacuum bagging to replace the pultruded top cleat cut from standard “off-the-shelf” leg-angle profile. This change in component produced a more efficient connection (Mottram 8). Unfortunately, the results were not as expected (see Table 3). The M-ϕ response of these four tests is shown in Fig. 17. The rotational characteristics from test Bank 4 are also included in the plot since this specific test has similar structural details to test Mottram 1. The differences in the two curves can be justified by the use of different fasteners and the tests set-up. The test arrangement used in Bank’s series introduces parasitic compressive forces, the effects of which on the overall joint response are unknown. Naturally, this is merely informative because these results are from a single test per detailing.

Mottram and Zheng [29,36] also tested web-cleated joints (Mottram 4 to 7, 9 and 10) to highlight the effect of the adhesive bonding. There is a clear benefit from using adhesive bonding in terms of initial stiffness, but it also induces a sudden brittle failure at lower rotations, say less than 10 mrad (see the results for Mottram 7, in particular) as in this type of connection the adhesive is often the weak link and governs the joint response. Fig. 18 summarizes the M-ϕ results of these tests. The key findings from this work are summarized below:

1. Adhesive bonding cannot be used on its own.
2. There needs to be a gap of 6 – 12 mm between the beam end and the column face to accommodate the free rotation between the connected members.

Fig. 19 compares the results for flange cleated connections and highlights the significant effect of the mechanical properties of the connecting elements on the M-ϕ behaviour. Although the elastic stiffness is not affected by the different material properties of the cleats, the moment capacity and the maximum deformation are markedly improved. Experimental tests carried by Qureshi and Mottram [38,39] have shown that this improvement in deformation capacity could be attributed to slip at the clearance holes.

Fig. 20 contains the final set of plots, which show the effect of the number of bolts per leg angle cleat and the mechanical properties of web angles on the overall joint response. In these tests, the hole clearance was minimized to limit the joint rotation due to slippage on the beam side and to develop the maximum prying action deformations for the lowest damage failure rotation. This also results in maximizing the joint stiffness that could be found in practice [38,39]. Another interesting finding was that the onset of damage occurred when the moment acting at the joint was about 55% of the joint moment capacity, $M_{j,c}$. The following general observations may be made:

1. The joint properties for the three- and two-bolted configurations are not significantly different. The middle (third) bolt is found to be redundant since it is not required to resist the design shear force.
2. Web cleats cut from pultruded shapes are not likely to have acceptable fibre reinforcement to resist the joint deformations generated from prying action and become susceptible to delamination failure when joint rotation has attained 10 mrad.
3. The governing failure mode has not been reported previously and involves fracturing within the pultruded column flange. This mode of failure occurs when steel replaces pultruded FRP web cleats of similar dimensions. Steel cleats are able to deform under bending (prying action) to resist the joint moment and so failure is shifted from the web cleating into the column.

Important work on the performance of bolted-only cleated joints was also reported by Turvey and Cooper [41,42] and Turvey [43]. The first series of tests were conducted with the objective of assessing the initial stiffness of joints with angles with different structural configurations. Results show that the initial stiffness is higher in those specimens that use thicker cleats. In the second series of tests [43] stainless steel cleats were employed. Turvey observed that the stainless steel cleats did not appear to show any signs of yielding around the bolt holes, though evidence of yielding in flexure of the angle legs was evident, especially for the stiffer joints. The modes of failure of these joints are clearly different from most of those already discussed and involve (i) shear out of the bolts in the web of the beam, (ii) failure of the tension flange of the beam, and (iii) a combination of both.

An experimental study of the behaviour of beam-to-column connections for pultruded FRP I-beams and box-sections was conducted by Smith [44] under the supervision of Parson and Hjelmstad [45,46] with the aim of comparing both structural systems, from a stiffness and resistance standpoint. Eventually, the authors proposed an innovative connection design by means of the so-called *cuff connection*. The cuff is a single monolithic unit that requires no bolting and utilizes the full column section, avoiding separation of the column web and flange at higher loads. This structural system will not be considered further.

I- and box-section beams with identical strong axis bending behaviour were selected so that the overall joint behaviour could be readily compared. The joints were constructed with pultruded angles and pultruded plates. The authors concluded that box beam connections performed much better than the I-beam connections with respect to both stiffness and resistance properties. In general, stiffness was improved by 25% and resistance by 280% by using the box sections with relatively simple connection geometry. This can be attributed to the tubular geometry that allows for the use of side plates in lieu of the web angles. These side plates are found to be structurally more effective.

The typical ultimate failure mode of joints with flange angles and web cleats made from pultruded material is that of separation of the column web from the facing flange in the region of the top angle under the action of the tensile bending forces being transmitted through the angle [45]. This mode of failure is likely to govern the joint behaviour because the web and the flange in the upper region of the column tends to act as two individual orthotropic plates that separate from each other due to insufficient fibre reinforcement in this interface to sustain the tensile forces that are transmitted by the top cleat. To avoid this mode of failure and ensure that the entire column section could contribute to the overall joint deformation, the researchers from the University of Wisconsin at Madison modified the joint typology by considering bolting through the entire column. This solution actually succeeded in restraining the facing flange against pulling away from the column web, but did not improve the stiffness or resistance of the joint. Failure itself was initiated in the web angles, which were likewise unaffected by bolting through.

2.2.2 *Other joint configurations*

Some development work on other joint configurations was also carried out in the 1990's and it may also offer a possible application for semi-rigid joint design. This work however has not been taken to a great depth and it is therefore inappropriate to present a detailed discussion in this report. A brief summary of the most relevant studies is thus presented next.

Bell, in 1992 [30], tested four pultruded corner joints. The joints connected beams and columns by means of L-shaped pultruded plates bolted through the webs of the members. Tests 2 to 4 also included a pultruded angle bolted across the heel of the joint (2nd test) and the instep of the joint (3rd and 4th tests). The plate rovings were aligned parallel to the beam pultrusion direction in the first three tests and at 45° in the fourth test. Bell found that the additional angles did not increase the elastic rotational stiffness of the connection. He also observed that the mode of failure was governed by tearing in the bolted web plates and cracking and bolt pull-through in those joints with angles [30]. Following this work, Turvey and Cooper [41] tested a pultruded frame joint by using two pairs of cruciform plates, bolted to the webs of the beams and column to evaluate the initial rotational stiffness.

Bruneau and Walker [47] discussed the cyclic behaviour of a single pultruded beam-to-column joint. The connection was comprised of bolted T-stubs connecting the top and bottom flanges of the beam to the face of the column and double-web angles. In their work, Bruneau and Walker gave a detailed description of the joint failure modes but a very limited amount of data was recorded during the test. A quantitative assessment of the M-ϕ response was therefore not possible. First failure of the joint was attributed to delamination of the bottom T-stub in the flange-web core. The ultimate failure mode was the separation of the column web-to-flange core.

2.2.3 *Principal conclusions*

Section 2.2 has attempted to provide the reader with an appraisal of the current experimental understanding for beam-to-column joints made from pultruded FRP material. The main conclusions of this review may be summarized as follows:

1. In most studies, the behaviour of joints is evaluated from individual tests that give results that are not statistically significant. Nonetheless, there is sufficient experimental data to allow numerical analyses to be performed.
2. The principal ultimate failure modes were identified:
   (a) Tensile tearing of the column’s flange from the web.
   (b) Fracturing within the FRP column flange outstands.
   (c) Delamination of FRP web cleat at the fillet radius of the cleats.
   (d) Delamination in a top FRP cleat.
3. The use of pultruded profiles for the connecting components severely limits the possibility of proper fibre orientations with respect to the load path(s).
4. There are certain combinations of stiffness and rotation at damage failure that would allow an increase in static loading for SLS deflection limits.

5. Effective connections joining pultruded members should be simple, economic, neat and easy to erect. The authors claim that steel-like joints are not necessarily going to be the best choice for this specific material.

Although careful study of these test data and selective additional testing can ensure that their value is maximized, a comprehensive coverage requires the availability of analytical and numerical techniques that can be used to generate parametric studies.

3. Numerical modelling of pultruded bolted joints

The response of joints to the applied loading results from a complex interaction between member components (webs, flanges), connecting components (plates, cleats) and the mechanical fasteners. The basic mechanism of this interaction has to be fully understood as the fundamental background for any simpler approach to analysing and capturing joint behaviour. Most studies rely on physical testing as the most appropriate means of determining the structural properties and characterizing the behaviour to ultimate failure. Whilst this has led to an improved knowledge and understanding of the joints and connections response, it is, of course, limited by the availability of reliable, relevant, and carefully documented test data. Despite the large number of tests conducted world-wide (Section 2, about 1000 for plate-to-plate connections and 100 for beam-to-column joints) the number of different types of joints and connections and the scope for variation within each type means that only a limited coverage of all practical joint details will ever be available. Although careful study of this test data and selective additional testing can ensure that their value is maximized in terms of interpolation between cases, identification of similarities, etc., a comprehensive coverage really requires the implementation of advanced FE techniques that can be used to generate parametric studies that will form the basis for the development of simple and reliable design guidelines.

For the FE modelling of pultruded FRP structural joints and connections, limited research has been done so far. A literature survey on this specific topic has been carried out and selected results are presented in the following sections. The primary areas of numerical research have been on plate-to-plate connections (especially for aerospace applications) and on supplementing stress results that are not easily obtained by experimental testing. In general, the analysis of bolted FRP structural joints and connections requires the precise modelling of: (i) geometrical and material nonlinearities of the various plate components, the members and the connection, (ii) damage initiation and growth to account for connections and joints possessing damage tolerance (iii) bolt behaviour, (iv) bolt interaction with the plate components (e.g. contact between the shank and the hole surface), (v) compressive interface stresses, (vi) possibility of slip, and (vii) presence of initial imperfections.

Pultruded structural components may exhibit significant nonlinear behaviour under various loading conditions. This nonlinearity is mainly due to second order geometric effects involving changes in stiffness due to damage initiation and propagation. Part of this nonlinearity can also be attributed to the matrix dominated stiffness. The nonlinear behaviour is particularly important near loaded fastener holes, edges, and cut-outs, that tend to amplify the nonlinearity due to stress concentrations. The response at these stress concentration areas can have crucial influence over the structural failure mechanism. This highlights the importance of requirement (i) and the need for a comprehensive nonlinear constitutive material modelling that can predict the overall behaviour under various loading conditions.

Additionally, in order to predict the joint and connection in-plane and out-of-plane response to loading with a high degree of reliability, the FE analysis has to use a progressive damage analysis methodology, which includes three basic steps: (i) stress analysis, (ii) failure criteria, and (iii) material stiffness degradation rules. The stress analysis uses standard laminated composite plate theory to describe the stress-strain behaviour of the material [48]. The failure analysis will assess the severity of the stress state for a given load level and decide which regions of the plate will fail, and in which mode of failure the damage will occur [49]. Damage can occur in one of the following three modes: matrix cracking, fibre breakage in tension and compression, and delamination. The first two modes are known as intra-laminar damage. Delamination is an inter-laminar damage mode. Several failure criteria have been proposed to determine the strength of a composite lamina [49]. A popular failure criterion is the Tsai-Wu tensor polynomial theory [50], which allows for interaction between components in a similar way to the von Mises criterion for isotropic materials. It is important to emphasize that the Tsai-Wu criterion provides first ply failure predictions, i.e. the initial localized damage. The material property degradation rules determine the post failure material properties for regions that have been subjected to damage. Kilic and Haj-Ali [51] developed an elastic degrading constitutive model specific to pultruded composites. In a follow up paper, Kilic and Haj-Ali [52] proposed a simple approach, which is readily amenable to computational procedures, for the progressive damage analysis of pultruded materials and structures, with a special emphasis on notched plates and pin-loaded bolted connections. The proposed constitutive and damage framework was integrated within the commercial FE code Abaqus [53] for a general nonlinear analysis of pultruded structures using layered shell or plate elements and was later extended to three-dimensional continuum elements [54].

Due to the complexity of the problem, most researchers have restricted themselves to only the stress analysis of plate-to-plate connections without considering failure. Stress analysis has mainly taken the form of two-dimensional analysis, despite the fact that the three-dimensional effects inherent to the problem have been known to be important for a long time. These effects include the through-thickness clamping action and element interfacing, in particular bolt/plate contact simulation. Little work has been reported on this subject. In the context of FE modelling of advanced composites some work has been carried out and added to existing two-dimensional investigations. The through-thickness clamping action was first analysed by Marshall et al. [55] who modelled four clamping conditions: (i) pin-loaded hole (no clamping over the washer area), (ii) finger-tight washer, modelled by restraining nodes in the washer area from normal movement which showed an increase in section from an initial pin-loaded run, (iii) flexible washer (uniform pressure distribution), and (iv) rigid washer (uniform displacement). The major findings of their study can be summarized as follows:

1. The stress profiles increased with clamping action.
2. With regard to the normal interface stresses, results suggest that the likelihood of delamination reduced with a reduction in the clamping ratio.
3. The high inter-laminar normal stresses associated with higher clamping ratios support of failure initiation by local bearing at high clamping action.
4. Friction has been shown to be beneficial in reducing bearing stresses in pinned connections, but it cannot be relied upon over the service life due to relaxation effects [56].

5. Less anisotropic materials have been shown to exhibit lower stress concentration factors in mechanically fastened joints.

Sun et al. [57] also focused on the three-dimensional numerical analysis of bolted plate-to-plate connections subjected to various clamp-up loads and extended on earlier work to include an appropriate progressive damage model to predict the strength of mechanically fastened joints under various clamping effects. The effects of friction and clearance, and the contact analysis were investigated by several authors; see, for example, the works of Hyer and Klang [58], Hyer et al. [59] and Dano et al. [60].

Relatively few numerical results relating to the behaviour of bolted plate-to-plate connections and beam-to-column joints made from pultruded material are available in the published literature. Most studies were restricted to linear elastic analyses to determine displacements and internal stress distributions. In this case, the joint or connection stiffness and deformation should be relatively straightforward to model. Very few authors have gone to the extent of predicting damage paths representing complete failure of a plate-to-plate connection, let alone a beam-to-column joint. A full three-dimensional stress analysis with application of three-dimensional failure criteria thus remains an open field for new research that will have an impact in developing what joint details should be used in practice.

3.1 Plate-to-plate connections

Hassan et al. [61] were the first to carry out nonlinear FE analyses of the stresses and deformations at the onset of failure in pultruded FRP double-lap single and multi-bolt tension joints to develop insight into the failure and resistance of these joints. They simplified the material constitutive behaviour by assuming that the laminates were transversely isotropic. A simple strength criterion based on the Tsai-Wu criterion [50] was implemented to assess material failure but delamination failure and damage tolerance phenomena were not accounted for. The numerical models used three-dimensional 8-noded layered shell elements. The bolt-hole contact problem was modelled with a joint mesh interface (gap elements) with no friction. They were able to reasonably predict the load-deformation response of these connection types (refer to Section 2.1). The average predicted values were 15% lower than the experimental values.

Turvey and Wang [62] built up a two-dimensional plane stress FE model to analyse the stress distribution in single-bolt tension joints of pultruded flat sheet. The plate was assumed to be a homogeneous elastic orthotropic material. The interaction between the bolt shank and the hole was modelled with surface contact elements that included the effects of friction and sliding. Bolts were assumed to behave rigidly because the authors concluded that these components had no effect on the stress distributions for the joint configurations considered in the analyses. Some general implications for design were also deduced. Park et al. [63] proposed a similar model that was verified against experimental results in terms of connection resistance.

In 2012, Feo et al. [64] reported a comprehensive numerical study to supplement a previous experimental study aimed at examining the distribution of shear stresses among the different bolts in plate-to-plate connections by varying the number of rows of bolts as well as the number of bolts per row. Five series of double shear lap connections were studied:

- Feo A: one bolt-row and one single bolt per row (1×1 bolts);
- Feo B: two bolt-rows and one single bolt per row (2×1 bolts);
- Feo C: two bolt-rows and two bolts per row (2×2 bolts);
- Feo D: three bolt-rows and three bolts per row (3×3 bolts);
- Feo E: four bolt-rows and four bolts per row (4×4 bolts);

The connections were modelled using 8-noded orthotropic bricks. The contact between the bolt and the FRP plates was simulated with one-dimensional point-contact elements. The study also considered the presence of variable diameter washers and their influence on the bearing stresses of composites with different material orientations. The results of this study showed that in multi-bolt joints, the load was not distributed equally due to varying bolt position, bolt-hole clearance, bolt-torque or tightening of the bolt, friction between member plates and at washer-plate interface. The results indicated that in the presence of washers, the stress distributions in the fibre direction, varying fibre inclinations, were decreasing for each modelled value of washer pressure. They showed that the optimum washer diameter should be taken as $2d_0$.

3.2 Beam-to-column joints

The first studies into pultruded beam-to-column joint behaviour were carried by Bank et al. [34] and were related to the joint configurations tested by the authors in the laboratory [33]. Details of the FE model and simplifications are not given in the paper. The authors limited their numerical study to a linear elastic analysis to obtain the initial M-ϕ behaviour, i.e. the linear stiffness response. Comparisons between experimental and numerical results for the initial stiffness were very poor, with differences as high as 60%.

The numerical study by Smith et al. [65] followed their experimental investigations. It had the main aim of reproducing experimentally measured linear elastic stiffness. They used shell elements. The FE model was in fact able to predict this joint property within 10% of that measured. Carrion et al. [66] further extended this FE study using 20-noded quadratic solid elements to represent a series of beam and column test frames comprising pultruded or steel box sections connected together using monolithic *cuff connections* of different thicknesses. Damage was also investigated in the models by employing the Tsai–Wu failure criterion to assess regions of the composites where ply failures were likely to occur. The models were first validated with respect to stiffness and strength by comparing them to the experimental test frames, and they were then used to proportion ‘‘improved’’ *cuff connections* for the test configuration considered.

Harte and McCann [67] developed a two-dimensional FE model (solid plane stress elements) for the analysis of semi-rigid pultruded beam-to-column joints using web and flange cleats. The model included material orthotropy. Contact conditions between all the components were modelled. The objective of this research was the comparison of the elastic rotational stiffness as predicted by the FE model with the experimental results of Turvey and Cooper [41], with good agreement shown by the numerical predictions. The full nonlinear moment-rotation response was not characterized since the model could not provide insight into the initiation and progressive failure mechanisms of the joint.

4. Design guidelines

Today most plate-to-plate connections and beam-to-column joints are detailed using pultruder-published design manuals, including Strongwell and Creative Pultrusions Inc. in the United States and Fiberline Composites A/S in Denmark, due to lack of specific standards and codes of practice to design pultruded structures. The absence of an approved code or guide for the design of this type of structures constitutes a liability problem, and so currently the design basis for pultruded structural design has to be defined by the professional engineer.

Evolving design guidance is provided by three publications that adopt the probability-based limit state design philosophy. These documents are the Eurocomp Design Code and Handbook [4], the Italian Guide for the Design and Construction of Structures made of FRP Pultruded Elements [68], and the American Pre-Standard for Load & Resistance Factor Design (LFRD) of Pultruded Fiber Reinforced Polymer (FRP) Structures [69]. None of the three sources of design guidelines has any legal standing. The Eurocomp document is from 1996 and provides some simplified design approaches for plate-to-plate connections, using bolting or adhesive bonding, but does not specifically scope the design of beam-to-column joints. There is general consensus that the procedures in Eurocomp can lead to unsafe results and should not be used in design [62]. The Italian Guide of 2008 limits the scope to (bolted or bonded) plate-to-plate connections and covers all known distinct (non-interacting) modes of failure. The resistance (or strength) formulae in these two documents for plate-to-plate connections were neither verified nor calibrated for their partial factors of resistance using test results from the sources reviewed in Section 2 of this paper. Recently, a more comprehensive American pre-standard has been prepared based on the current state-of-the-art knowledge for predicting both connection and joint behaviours. To establish the strength formulae for single- and multi-row bolted connections the test results from the literature to 2009 (see Mottram and Turvey, 2003) were used to determine the partial factors in the resistance formulae for the observed distinct modes of failure. This Pre-standard of 2010 is presently with an American Society of Civil Engineers standards committee passing through the American National Standards Institute process to its published standard. In the process of transformation to the standard a number of revisions have been made to the mandatory clauses in Chapter 8 for the design of bolted connections. It should be noted that the design guidance provided in the Pre-Standard for framed structures is limited to the case of nominally pinned joints (simple no-sway braced construction) and only relies on test results in Section 3 in terms of them showing how an adequate joint rotation (say 25 mrad) can be achieved without composite material failure. Because there is uncertainty in applying the simple design formulae and rules of thumb in universal design situations we find that all three publications are pragmatic in enabling designers to use fit-for-purpose physical testing to permit the establishment of compliance on the basis of test results; such as reviewed in Sections 2 and 3. In the case of the Pre-standard the test results must be evaluated in accordance with Section 2.3.2 for Prequalified FRP Building Products.

In 2011 the Construction Institute of the American Society of Civil Engineers published the Manual of Practice No.102 [31] for a design guide for FRP composite connections, in recognition of the need for rationale design procedures for connections and joints. Although this manual covers key issues related to the analysis and design of composite connections and joints fabricated of pultruded material, it does not, for the preparation of a design standard, compare test results for the strengths of bolted connections against what simplified formulae predict based on geometry and strength properties.

5. Conclusions and research needs

Technical observations pertaining to the behaviour of structural connections and joints made from pultruded material as a result of this review are summarized in the following:

1. Static connection/joint failure analysis for pultruded FRP, which represents the primary area of research activity in this literature review, mainly consists of detailed stress and deformation analysis.

2. The two principal methodologies include experimental testing and FE analysis. Experimental tests provide reliable results that can give an accurate description of the behaviour of connection/joint. However, due to the large number of variables and potential modes of failure, it is unlikely that all aspects of the problem have been thoroughly examined. Use of FE analyses can explore a large number of variables and failure mechanisms and complement the limited experimental studies. Three-dimensional FE analysis is required in order to capture the actual behaviour, namely the through-thickness effects, contact and interfacing and ultimate (orthotropic) failure.
3. The effects of several design parameters on the static behaviour of pultruded connections and joints have been examined. For the case of plate-to-plate connections, the possible variations in the relevant parameters were analysed and some requirements were established for the bearing mode of failure that complies with most design practices.
4. There is currently no quantitative guidance available for the design of beam-to-column joints in the context of the semi-continuous/partially-restrained philosophy. The design of pultruded joints in this framework would be most beneficial in terms of increasing loading that satisfies the SLS design limits.
5. Most research results have shown that the mimicry of bare steel joint configurations is not necessarily the most appropriate for pultruded joints, due to the orthotropic properties of the connecting parts and elements.
6. Closed sections are perhaps a more appropriate member selection.

As a result of this review, the need for further research in the following areas is identified:

1. The differences between single- and double-lap plate-to-plate connections have not been sufficiently studied. FE analyses are very suitable for this type of study, especially three-dimensional stress analysis to assess the strain and bolt-hole contact pressure.
2. The development of efficient bolted joint design is an essential part of designing minimum weight structures from pultruded FRP materials. Current practice relies on steel-like designs and this requires critical evaluation as clearly the maximum joint efficiencies in composite structures are not being attained.

3. Further research is required in the development of sophisticated FE techniques for the analysis of several aspects of connection and joint behaviour. While current techniques already seem to be suitable for predicting some features of the response of connections and joints of pultruded plate material, the characterization of the complete force-deformation and moment-rotation curves requires the ability to model (i) the three-dimensional characteristics of the joint behaviour, (ii) interaction between thin-walled panels and fasteners, and (iii) material damage initiation and growth, at a level of refinement not yet attained, to capture the actual deformation response to ultimate failure.
4. Using the capability of the FE techniques for predicting with adequate accuracy the complete joint moment-rotation curve, several studies of the sensitivity of frame response to the variation of the rotational characteristics can be carried out. The main implications for design can then be determined and combined into a complete procedure for the design of the frame using the principles of semi-continuous/partially-restrained construction for both the ULS and SLS.
5. The methodologies developed for composite materials of glass fibres and a polymer based matrix can be readily exploited when natural or bio composites have a property portfolio suitable for use in primary load bearing structures. The work can be further extended to composite components made by other processing methods (e.g. resin transfer moulding, filament winding).
6. Future programmes of work in this area ought to focus on the characterization of (i) the structural performance after connections and joints have been exposed to loadings and environmental aging that represent what the frame structures might experience during a service life extending over, say 30 years (durability design requirements) and (ii) the structural behaviour of FRP connections and joints subjected to extreme loading (e.g. seismic events).

Some of these aspects will be addressed in the current European research project PFRPStruJoin that is being conducted by the authors. The project relies on applied University-led research to produce the information that can address the above objectives, as industry cannot be expected to conduct underpinning research to assist scientifically based rules to be included in design guidelines and codes of practice.

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Figure captions list

Table captions list

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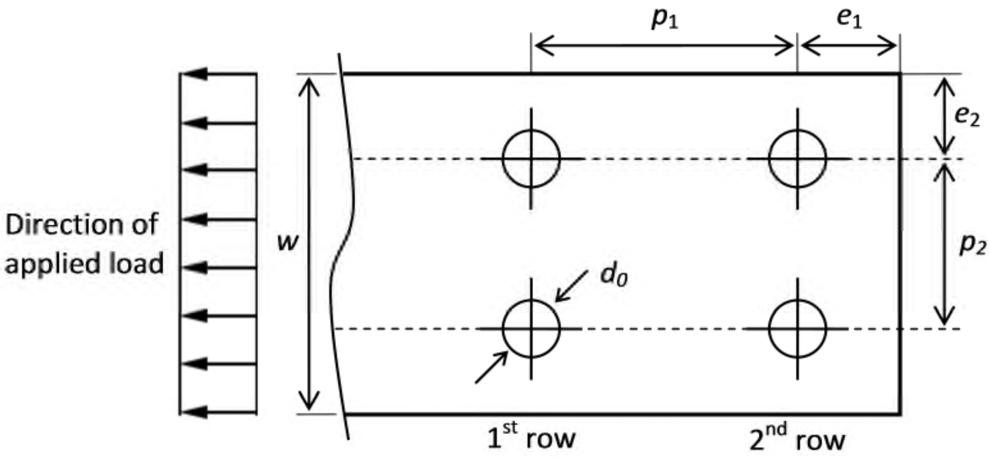
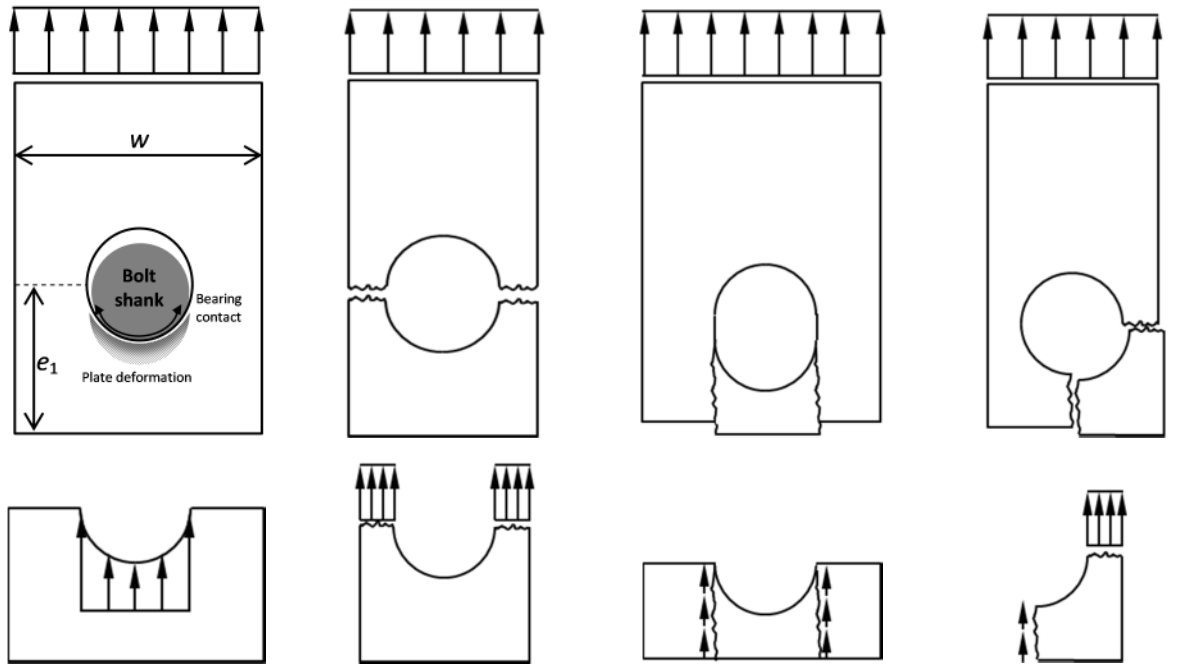
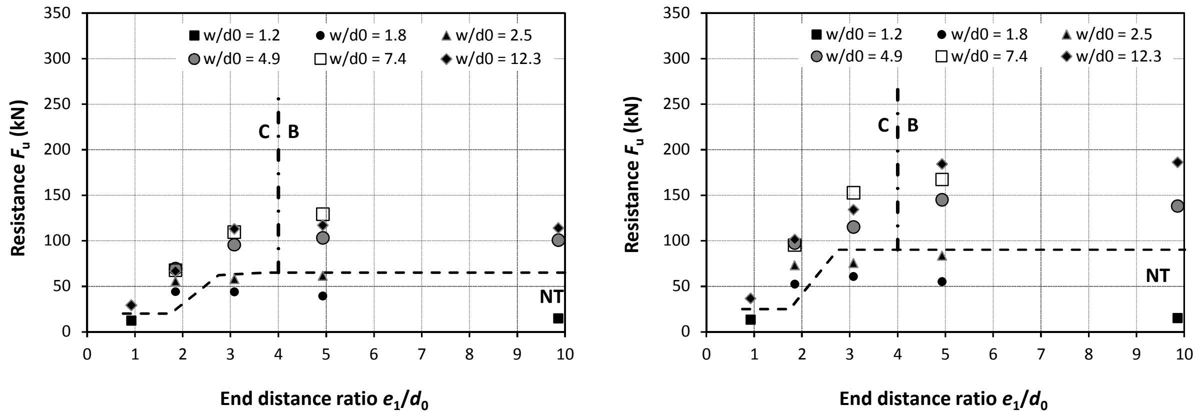


Fig. 1: Definition of the connection geometry and symbols for spacing of fasteners



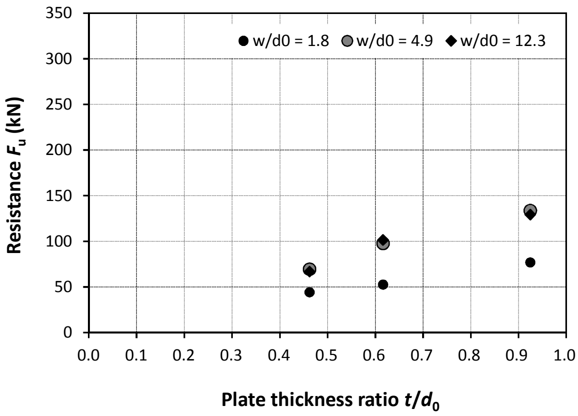
a) Bearing **b)** Net-tension **c)** Shear-out **d)** Cleavage

Fig. 2: Static in-plane failure modes of single-bolted plate-to-plate connections, with e_1 and w sized in terms of the distinct mode

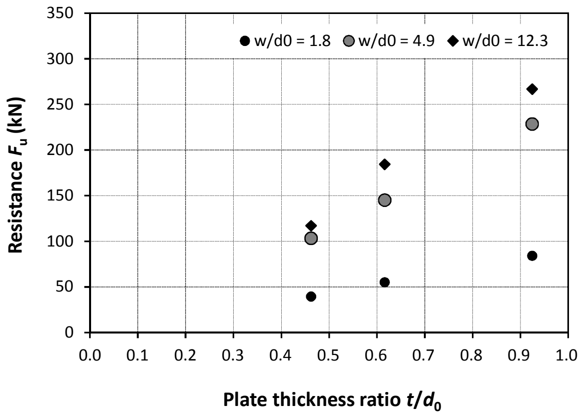


a) *t* = 9.5 mm **b)** *t* = 12.7 mm

Fig. 3: Resistance-end distance ratio results: effect of varying the width ratio with the plate thickness (data from Rosner [6])



a) e_1/d_0 = 1.8



b) e_1/d_0 = 4.9

Fig. 4: Resistance-plate thickness ratio results: effect of varying the width ratio with the end distance ratio (data from Rosner [6])

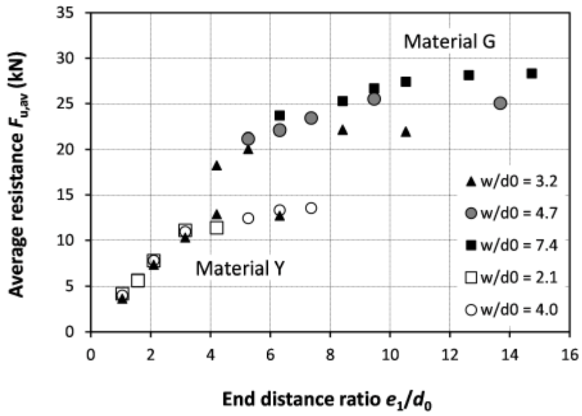
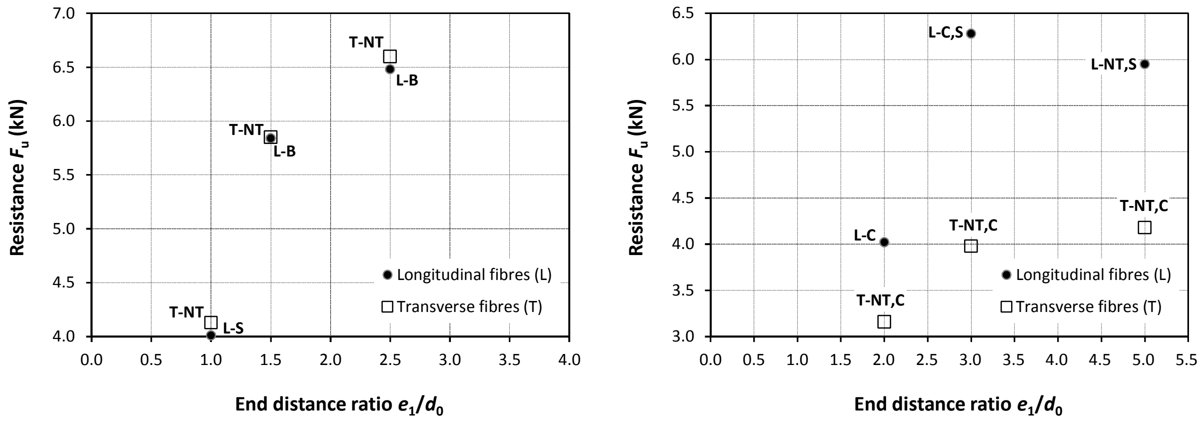


Fig. 5: Average resistance-end distance ratio results: effect of varying the width ratio for different fibre volume fraction of uniaxial and CSM layers – for material G: 73% and 27%, and for material Y: 59% and 41%, respectively (data from Abd-El-Naby and Hollaway [9])



a) d_0 = 12.7 mm **b)** d_0 = 6.4 mm

Fig. 6: Resistance-end distance ratio results: effect of varying the off-axis angle between roving and tension direction with the hole diameter for a fixed ratio w/d_0 = 4 – results are plotted for pultruded material without reinforcement (data from Wang [11])

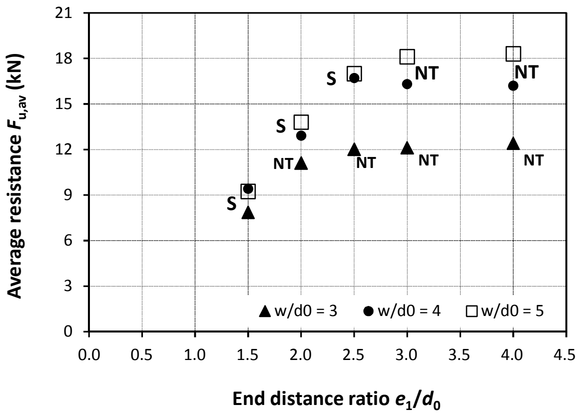
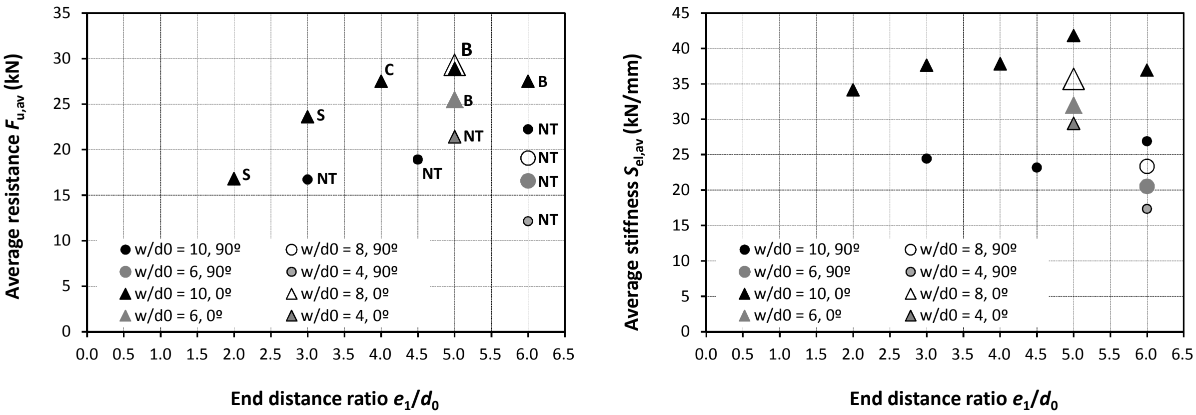


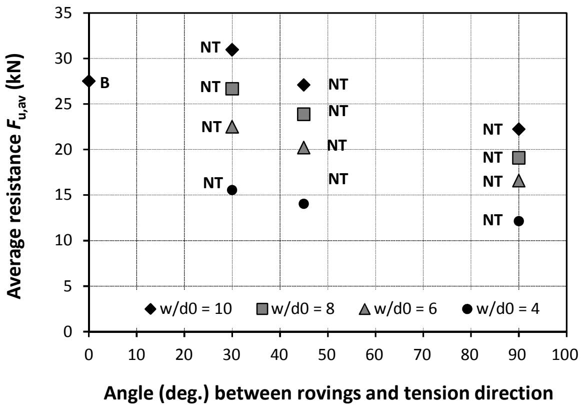
Fig. 7: Resistance-end distance ratio results: effect of varying the width ratio (data from Turvey [12] and Turvey and Godé [13])



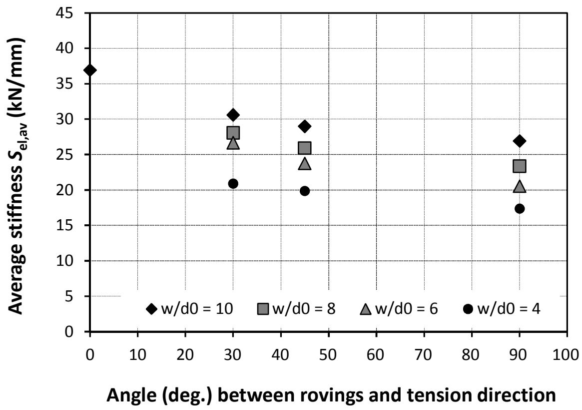
a) Resistance and modes of failure

b) Stiffness

Fig. 8: Resistance-end distance ratio results: effect of varying the roving direction (data from Turvey [14])



a) Resistance and modes of failure



b) Stiffness

Fig. 9: Connection characteristics versus off-axis angle between roving and tension direction results: effect of varying the width ratio for an end distance ratio of six (data from Turvey [14])

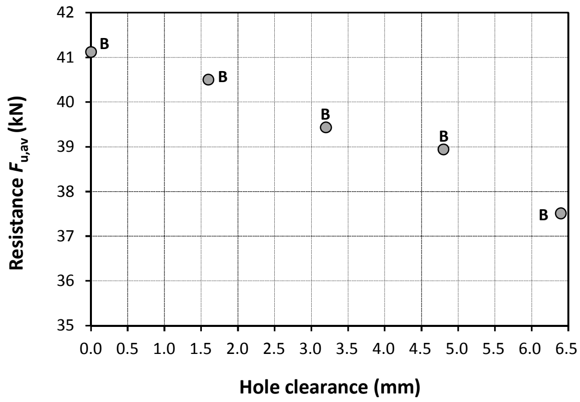
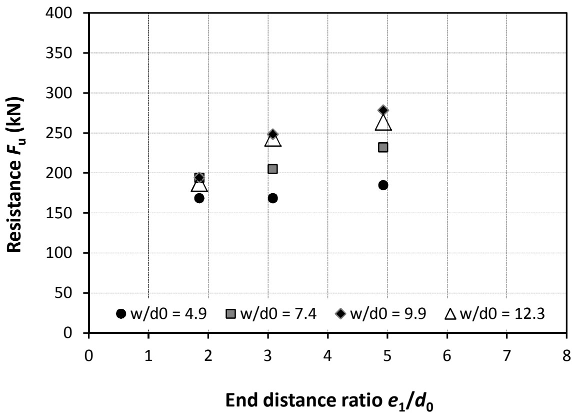
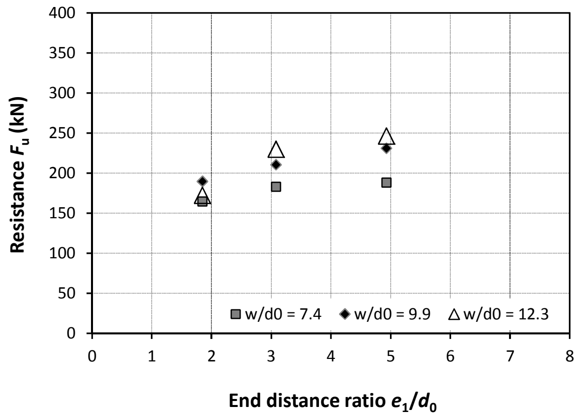


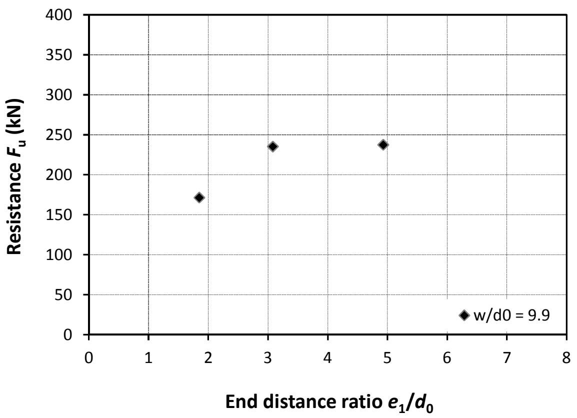
Fig. 10: Resistance-hole clearance results (data from Yuan et al. [18])



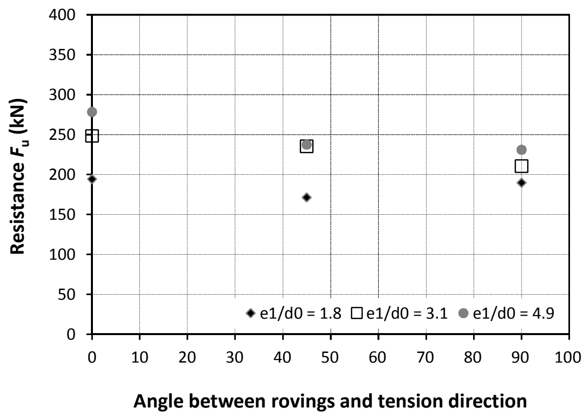
a) Roving direction 0°



b) Roving direction 90°

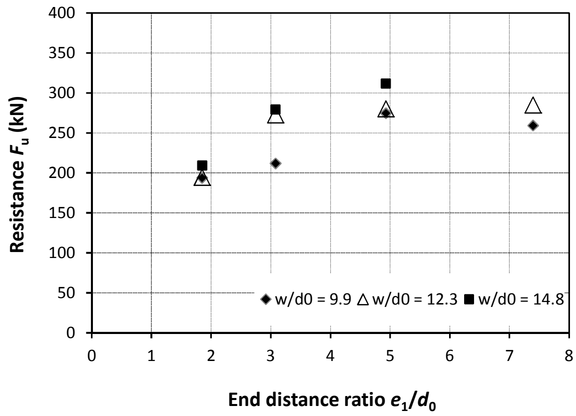


c) Roving direction 45°

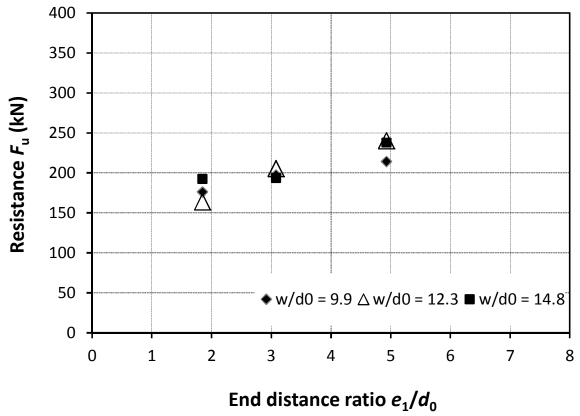


d) Comparisons for $w/d_0 = 9.9$

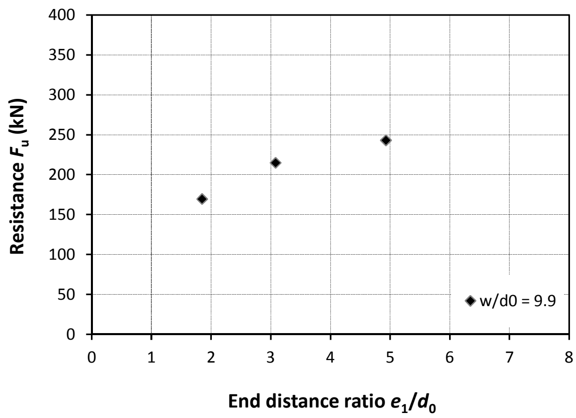
Fig. 11: Principal results for test Hassan A (2×1 bolts) (data from Hassan et al. [22])



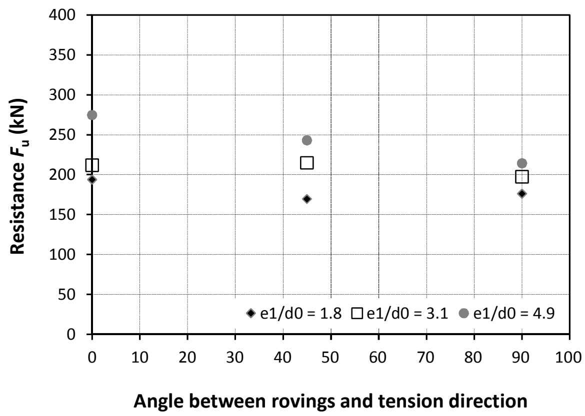
a) Roving direction 0°



b) Roving direction 90°

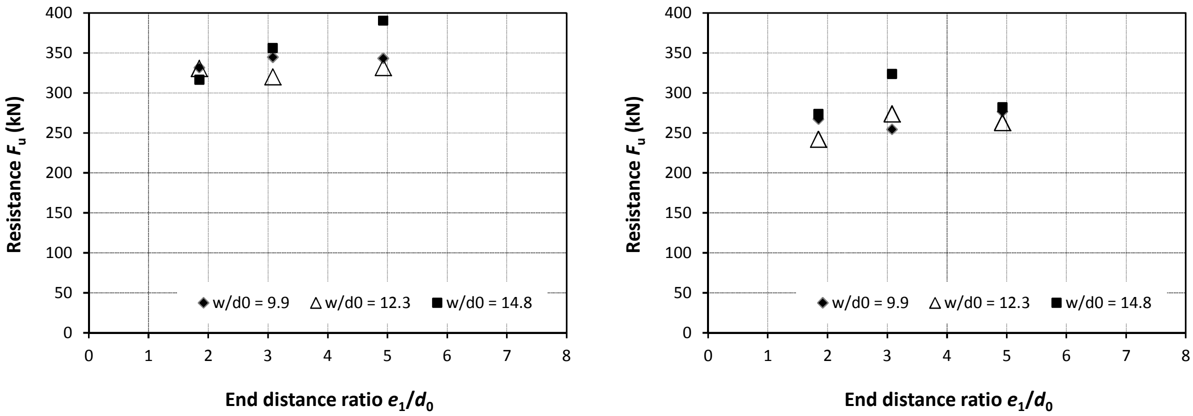


c) Roving direction 45°



d) Comparisons for $w/d_0 = 9.9$

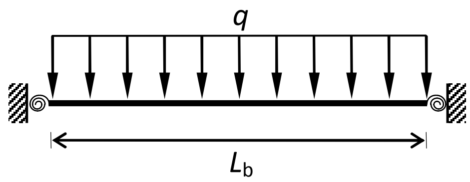
Fig. 12: Principal results for test Hassan B (1×2 bolts) (data from Hassan et al. [22])



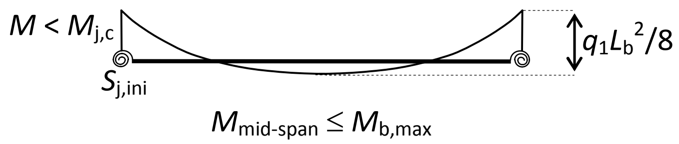
a) Roving direction 0°

b) Roving direction 90°

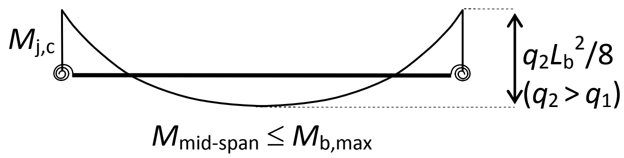
Fig. 13: Principal results for test Hassan E (2×2 bolts) (data from Hassan et al. [22])



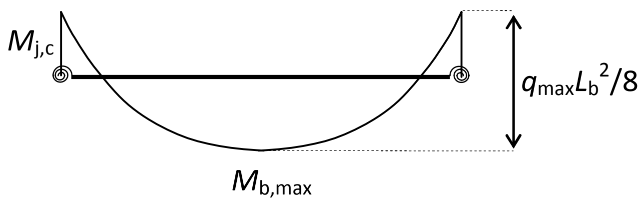
a) Beam with semi-rigid/partial-strength end joints



b) Elastic distribution of moments (SLS)

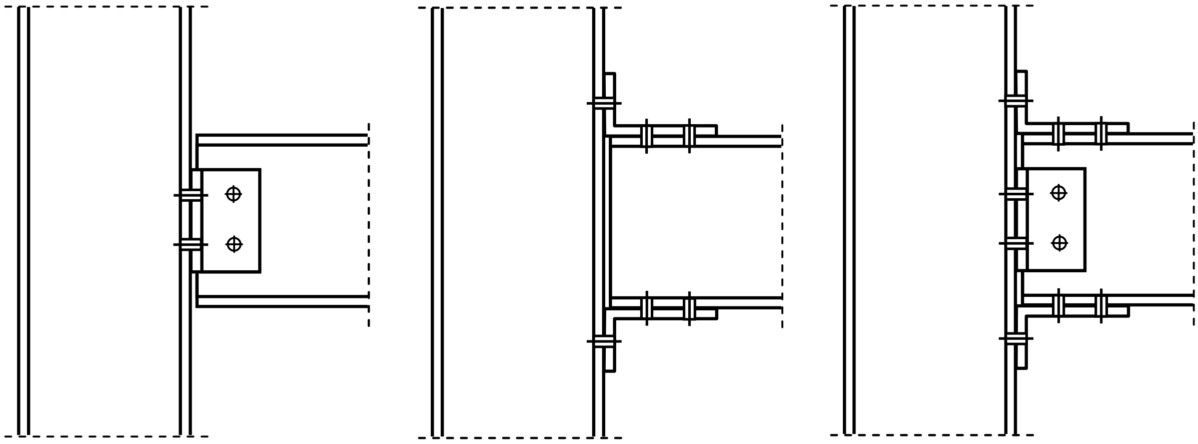


c) Joint moment capacity reached



d) Maximum moment redistribution (ULS)

Fig. 14: Illustration of the principles of semi-continuous/partially-restrained design philosophy (SLS: serviceability limit states and ULS: ultimate limit states)



a) Bolted web angles

b) Top and seat angles bolted to the column and beam

c) Top and seat bolted angles and bolted web angles

Fig. 15: Common joint typologies used in the experimental studies

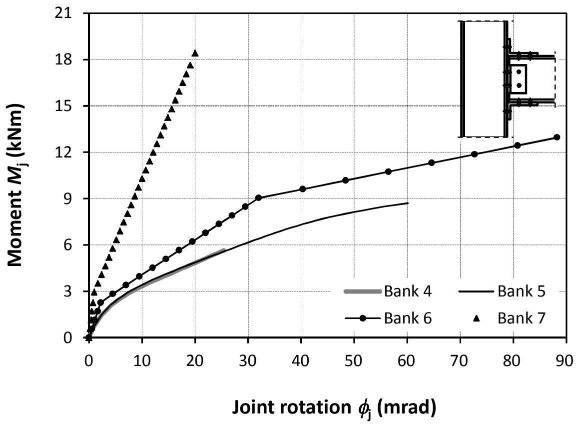


Fig. 16: Nonlinear moment-rotation curves (data from Bank et al. [33])

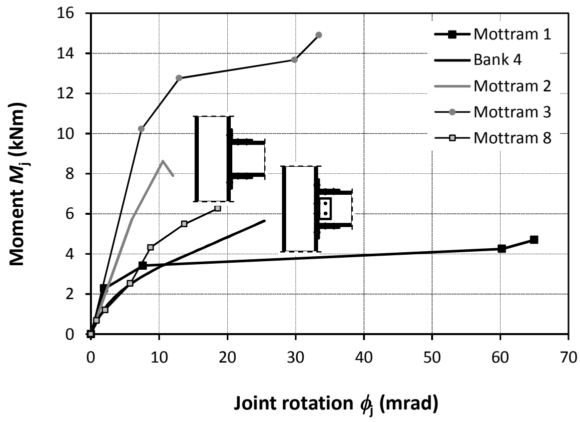


Fig. 17: Moment-rotation curves: effect of adding extra connecting elements and comparison with previous test results (data from Bank et al. [33] and Bass and Mottram [35])

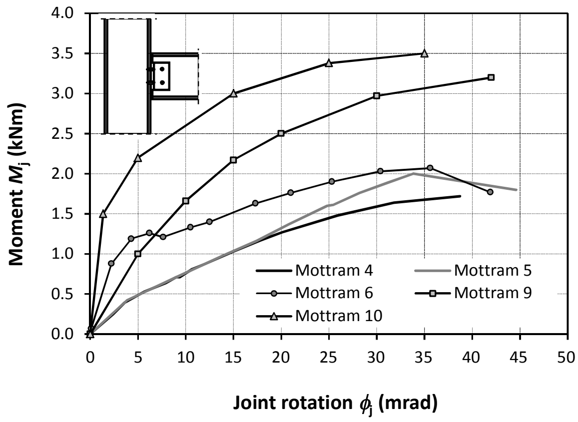


Fig. 18: Moment-rotation curves: comparisons between the behaviour of web cleated connections and effect of adhesive bonding in addition to mechanical fastening on connection performance (data from Mottram [28] and Mottram and Zheng [29])

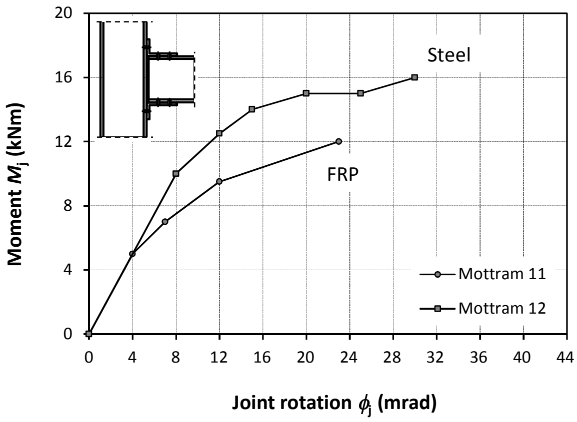
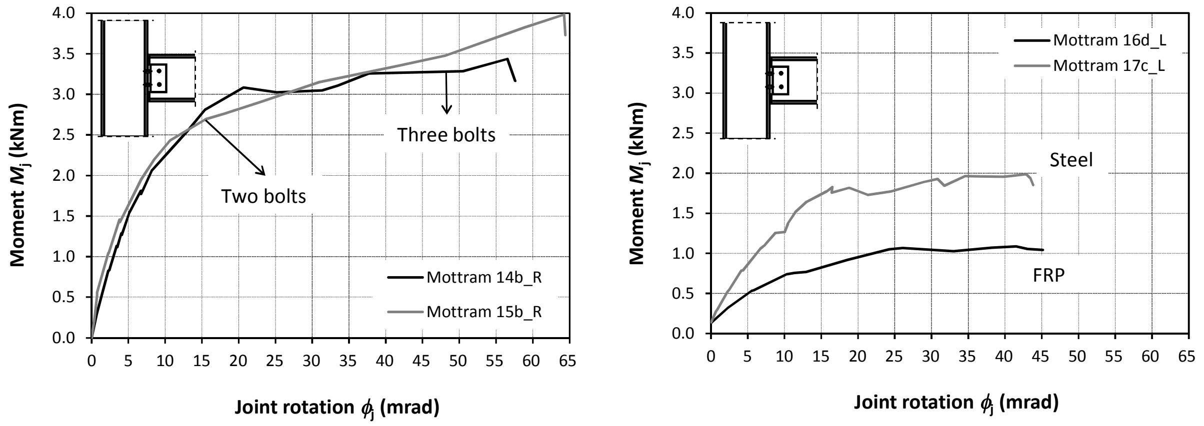


Fig. 19: Moment-rotation curves: comparisons between steel and FRP flange cleats (data from Mottram and Zheng [37])



a) Beam and column sections 254 × 254 × 12.7 mm: effect of number of bolts connecting the cleat to the beam web

b) Beam and column sections 203 × 203 × 9.5 mm: comparisons between steel and FRP web cleats

Fig. 20: Moment-rotation curves (data from Qureshi and Mottram[38,39])

Table 1: Joint tests supervised by Bank (data from [32-34])

Test	Joint typology	Experimental results				Comments	
		Initial stiffness (kNm/rad)	Resistance (kNm)	Rotation capacity (mrad)	Failure mode		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	
Bank 1	Web cleats (152x152x12.7)	27				Authors concluded that the initial stiffness of pultruded FRP joints are lower than those of steel connections but are not insignificant. For example, a bare steel connection with similar details as Bank 3 has an initial stiffness 2.9 times higher.	No adhesive bonding.
Bank 2	Web cleat and seat angle (152x152x12.7)	170					
Bank 3	Flange cleats (152x152x12.7)	678					
Bank 4	Flange and web cleats (152x152x12.7)	790	6.1	38	Separation of the column flange from its web behind the bolt-line of the top seat; through-thickness cracks developed in the top angle in the connection to the beam flange.	Prying action was observed (top angle).	
Bank 5	Flange and web cleats (152x152x12.7) and a further angle stiffener bolted to the column web and flange to reinforce the column web/flange junction.	790	8.7	59	Combination of radial tensile cracking and the nuts punching through the pultruded top angle. Web cleat rods failed in shear, with the thread being stripped from the pultruded rod core.	Failure of the rods only occurred after the top cleat failed and the load was transferred to the web angles.	
Bank 6	Same connection details as Bank 5 but the top angle was replaced with a built-up part.	1027	13.0	88	Transverse tensile failure of the T-flange web. The web angle rods failed in a similar fashion to connection Bank 5.	Built-up part consisted of two T-Flanges and a pultruded gusset plate with its longitudinal direction at 45º to the beam section.	
Bank 7	Built-up parts were used for both top and bottom connection.	2943	18.5	20	Sudden and brittle failure occurred in the adhesive bond between the pultruded plate and the slotted T-section which formed the gusset; minor through-thickness cracking occurred in the flange of the T-flange section in the top stiffener.	Same buit-up part; gusset plate bonded to the T-flanges to create a monolithic three-dimensional part; threaded rods were extended to the opposite flanges of the beam and column sections.	
Bank 8	Similar to the joint layout in Bank 7 but the gusset plates were fabricated differently; the threaded rod arrangements varied in both tests.	460	4.5	20	Through-thickness tensile failure of the wide flange used to construct the right brace.	Connection performed very poorly.	
Bank 9		678	8.5	32			
Bank 10	Similar in layout to Bank 8 but used pultruded multi-cellular elements in place of the gussets and flat plate stiffeners in place of the tubular rods.	595	30.5	47	Thread stripping of the bolts in the top brace of the connection.	No failure of the beam and column sections were noted. Plate stiffeners detach prior to failure.	
Bank 11	Similar in layout to test 8 but used wrapped angles in place of the gussets.	237	11.3	85	The top brace failed due to bolt failure.	Performance of this connection was considered impressive in a structural engineering sense considering its simplicity and minimal materials usage.	

Table 2: Joint tests supervised by Mottram: structural details (data from references in the table)

Test	Ref.	Beam and column sections	Joint typology	Bolts				Test set-up	Comments	
				Diam. (mm)	Grade	Torque (Nm)	Clear. (mm)			
(1)	(8)	(9)	(2)	(10)	(11)	(12)	(13)	(14)	(7)	
Mottram 1	[35]	203x203x9.5	FRP flange and web cleats (152x152x12.7)	16	8.8	23.8	2.0	Double cantilever beam	Same connection details as Bank 4.	Joints were fabricated using bolting and bonding. Adhesive bonding of the mating sufaces was used to increase initial connection stiffness. No gap between the beam end and the column flange. Top and bottom flange cleats were joined to the frame members by 2x2 bolts in each leg.
Mottram 2									Pultruded angles were also incorporated to prevent tearing failure at the web flange junction of the column; pairs of stacked pultruded angles used as tension flange cleats in order to stiff the joint and prevent delamination failure in the cleat heels.	
Mottram 3									Pultruded angles were also added to the lower face of the beam tension flanges and the column flanges.	
Mottram 4 (L & R)	[28,36]		FRP web cleats (152x152x12.7)	16	8.8	23.8	2.0		Cleats bolted to the beam web and column flange.	
Mottram 5 (L & R)				20			0.1 - 0.3			
Mottram 6 (L & R)				16			2.0		Cleats bolted and bonded to the beam web and column flange.	
Mottram 7 (L & R)				*Cleats bonded to the beam web and column flange*					–	
Mottram 8 (L & R)			FRP flange cleats (152x152x12.7)	16	8.8	23.8	2		Pre-peg top angle; conventional seat angle.	
Mottram 9	[29,37]	254x254x12.7	FRP web cleats (102x102x12.7)	16	-	100.0	2		Cleats bolted (3 bolts) to the beam web and column flange.	Details of these two connections are from the Strongwell Design manual. There was a gap of 10 mm between the beam end and the column flange. The bolt torque is much higher than in previous tests to try to eliminate connection slip when adhesive bonding is not present.
Mottram 10									Cleats bolted (3 bolts) and bonded to the beam web and column flange	
Mottram 11		203x203x9.5	Top and seat steel cleats (100x100x8)	16	4.6	100.0	2		Cleats were steel grade S275. Cleats were bolted to the beam flanges by means of bolts and through the column flanges by means of M20 threaded rods that crossed the column section. There was a gap of 10 mm between the beam end and the column flange. The bolt torque is much higher than in previous tests to try to eliminate connection slip when adhesive bonding is not present.	
Mottram 12			Top and seat cleats	16	4.6	23.8	2		Bespoke pressure moulded GRP cleats attached to the outer face of the beam tension flange and pultruded 152x152x12.7 angles for the inner and outer faces of the beam tension and compression flanges, respectively. Threaded rods 25.4 mm for the column stiffeners.	
Mottram 13									Similar to test 12, but without the cleats bolted to the underside of the beam tension flange.	
Mottram 14 (L & R)	[38]	254x254x12.7	Steel web cleats (100x100x10)	16	8.8	*Bolts were tightened to give a snug fit.*	0.1 - 0.3		Cleats were fabricated from steel grade S275. Joints were fabricated using bolting and bonding. 10 mm gap between beam and column. The shank bearing into FRP material is plain to avoid any localized deformation from thread indentations.	3 bolts per leg.
Mottram 15 (L & R)										2 bolts per leg.
Mottram 16 (L & R)	[39]	203x203x9.5	FRP web cleats (75x75x10)	16	8.8		0.1 - 0.3			2 bolts per leg.
Mottram 17 (L & R)			Steel web cleats (75x75x10)							2 bolts per leg.

Table 3: Joint tests supervised by Mottram: principal results; stiffness values in italic correspond to secant stiffness values rather than initial stiffness

Test		Experimental results			
		Stiffness (kNm/rad)	Resistance (kNm)	Rotation capacity (mrad)	Failure mode
(1)		(3)	(4)	(5)	(6)
Mottram 1		*1120*	4.7	65	Tearing of the column flange from its web; there was also significant delamination in the heel of the tension flange cleat.
Mottram 2		*1120*	7.9	12	–
Mottram 3		*1320*	14.9	33	Delamination in putlruded top cleat.
Mottram 4	L	*52*	1.7	39	Irreversible slip occurred due to bolt hole clearance; large web cleat prying and column flange bowing - web cleats splitting.
	R	*57*		49	
Mottram 5	L	*59*	1.9	45	No slip; Splitting at top of web cleats (delamination)
	R	*76*		30	
Mottram 6	L	*185*	2.1	42	Adhesive debond at web cleat/column flange interface; left connection progressively failing by delamination at the column web/flange interface.
	R	*172*		33	
Mottram 7	L	*369*	*1.1*	*4*	Sudden and brittle failure.
	R	*385*		*10*	
Mottram 8	L	*450*	6.3	19	Heel of the top cleat, delamination growth progressed in a sudden and brittle manner as the connection deformed.
	R	*396*		15	
Mottram 9		*220*	3.2	42	First failure: delamination crack at the top surface of the web cleats; ultimate failure: combination of delamination and tensile rupture (due to bending) in the leg angles next to the location of the top row of bolts.
Mottram 10		*990*	3.5	35	Interlaminar failure of the pultruded material, whereby fibres became exposed. This failure mechanism is typical of adhesively bonded pultruded profiles.
Mottram 11		*1100*	> 12	> 18	Flexural rupture of top beam flange at the location of the single row of bolts (cleavage type failure).
Mottram 12		*1330*	16.0	30	Delamination in pre-peg top cleat and stripping of the thread on the composite rods.
Mottram 13		*600*	–	–	Debonding of the top cleat.

Table 3: Joint tests supervised by Mottram: principal results; stiffness values in italic correspond to secant stiffness values rather than initial stiffness (cont'd)

Test		Experimental results			
		Stiffness (kNm/rad)	Resistance (kNm)	Rotation capacity (mrad)	Failure mode
(1)		(3)	(4)	(5)	(6)
Mottram 14	a_L	302	3.4	50	Delamination in the web-flange junction of the column near top bolt level and progressed downwards to bottom bolt level. Due to prying action, column flange outstands deflect outwards at the top bolt level and there was no change in the depth of the section at the bottom bolt level. Steel cleats did not noticeably deform or have yielding.
	a_R	316	3.4	59	
	b_L	649	3.4	51	
	b_R	386	3.4	55	
	c_L	913	3.3	40	
	c_R	444	3.4	57	
Mottram 15	a_L	896	3.7	59	
	a_R	398	3.7	61	
	b_L	744	4.0	64	
	b_R	386	4.0	68	
	c_L	646	3.7	62	
	c_R	393	3.7	69	
Mottram 16	a_L	87	1.1	58	Excessive delamination damage at top of cleating and near the fillet radius.
	a_R	79	1.1	53	
	b_L	80	1.0	26	
	b_R	74	1.0	30	
	c_L	73	1.1	64	
	c_R	80	1.0	28	
	d_L	81	1.0	41	
	d_R	75	0.9	33	
	e_L	72	1.0	39	
	e_R	63	1.0	56	
Mottram 17	a_L	179	1.5	15	Failure happens within the column member as significant outward flexural deformation causes internal (non-visible) fracturing.
	a_R	144	1.5	53	
	b_L	194	1.8	58	
	b_R	166	1.8	18	
	c_L	179	1.9	40	
	c_R	152	1.8	67	