GROUP EFFECT OF GFRP-TIMBER BOLTED CONNECTIONS IN TENSION

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Abstract

Steel plate bolted connections are frequently used in timber structures. However, these connections may incur durability problem due to the corrosion of the steel plates. This paper proposes using GFRP plate to replace steel plate in timber connections to mitigate the corrosion issue. The group effect of the proposed GFRP-timber bolted connection is studied. Tensile tests of GFRP-timber bolted connections were conducted considering a number of variables including bolt diameter, bolt pitch distance, bolt row number, bolt column number. The failure modes and the load-displacement curves of the connections were reported. Experimental results showed that the group effect of the GFRP-timber bolted connection is strongly affected by the bolt diameter and number of bolts, but little affected by bolt pitch distance. The effect of the number of bolt columns on the group effect was also discovered and quantified. The design equation of the timber bolted connection in Eurocode 5 was modified considering the effects of various parameters. The modified equation was then verified using the experimental data in this paper.

Finally, detailed recommendations were provided for the design of the proposed GFRP-timber bolted connection.

KEYWORDS: Group effect; Bolted joints; GFRP; Timber; Mechanical properties

1. Introduction

Timber structures are becoming more popular over the world considering the material greenness and sustainability in construction industry [1-2]. Timber structures often use bolted connections with steel plates connecting timber elements due to their reliability in force transmission [3-9]. These bolted connections with steel plates are increasingly used in heavy timber structures and more recently become the principle connecting method in modular construction of timber structures [10]. However, bolted connections with steel plates suffer from corrosion issue considering the inherent moisture in the timber. Condensed water will also accumulate on the steel surface when temperature is low making the corrosion issue become worse. In addition, steel plates have good thermal conductivity which may form a thermal bridge in timber structures, leading to higher energy consumption for maintaining consistent indoor temperature. These corrosion and thermal issues make the maintenance of timber structures become more difficult and expensive [11-12]. Glass fiber reinforced polymer (GFRP) composite has attracted increasing attentions in the construction industry considering its advantages of light weight, high strength, low cost and corrosion resistance [13-21]. Therefore, it has great potential to help mitigate the

corrosion issue of timber connections by replacing the steel elements with GFRP ones. This may have extra benefit of reducing the self-weight of the structure when improving the durability at the same time.

One of the most important characteristics of timber bolted connection is the group effect [22]. Two timber bolted connections can be used to explain the concept of group effect. For example, connection A has ‘n’ bolts and connection B only has one same bolt. The capacities of both connections are defined as F_A and F_B respectively. It has been found that F_A is not simply equal to ‘n’ times F_B. In other words, the capacity of a multi-bolt connection is not necessarily equal to ‘n’ multiplying the capacity of the same connection with only one bolt. The group effect of multi-bolt connection is a very important concept in the design of timber structures. It has been explicitly considered in some international design standards such as Eurocode 5: Design of Timber Structures [23], in which F_A is equal to ‘n_{ef}’ times F_B. n_{ef} is called effective number of bolts which is a function of the actual number of bolts ‘n’ and represents the group effect.

Most studies on the group effect in the literature focused on all FRP connections [24-33] or all timber connections [7-9, 22, 23, 34-37]. Jorissen et al. [22] were one of the first who studied and defined the group effect of multi-bolt timber connections. He used the group effect concept to describe the relationship between the load carrying capacity of a multi-bolt connection and corresponding single-bolt connection. He pointed out that the group effect was greatly affected by parameters including bolt diameter, bolt pitch distance, and number of bolts. Cramer et al. [34] proposed

another concept of bolt utilization efficiency to describe the group effect of multi-bolt connections. He found that each bolt in a multi-bolt timber connection had different contribution to the load carrying capacity. For example, the bolts close to the end of the connection were found more effective than bolts far away from the ends. Xu et al. [35] carried out a finite element analysis of the bolted timber connections and found that the stresses of the bolt holes near the end of the connection were greater than those of the bolts far away from the end. Therefore, his findings support the work by Cramer et al. [34]. Similar studies on the group effect can also be found in [24-32], which reported that different bolt row had different contribution to the load carrying capacity of FRP bolted connections. Jensen et al. [8] studied the effect of bolt pitch distance and bolt edge distance on the group effect of multi-bolt connections. He reported that the bolt edge distance exhibited greater effect on group effect than bolt pitch distance. Jockwer et al. [36] also studied the effect of bolt pitch distance and bolt edge distance of timber-steel-timber connections with dowel-type fasteners and reported that small bolt pitch distance and bolt edge distance usually results in brittle failure and sufficient distance is needed to meet the safety of the structure. Yurrita [37] and Mohammad [9] reported the failure mode of steel-timber multi-bolted connections, where splitting failure occurred more often when the number of bolts was greater than or equal to 3. Gattesco [7] studied the effect of the number of bolts in a column on the group effect. He reported that the bolt utilization efficiency decreased with the increase of bolt number. Persson [33] reported that large bolt diameters and protruding-head fastener significantly increased gross section strength, while bolt hole

machining damage and prepreg type had no effect on the bearing capacity of FRP bolted connections. It can be summarized from the published research that the key parameters affecting the group effect of multi-bolt connections include bolt diameter, bolt pitch distance, number of bolt columns and rows. These parameters are also incorporated in Eurocode 5 [23] to quantify the group effect of multi-bolt timber connections.

However, there has been no report on the group effect of bolted connections incorporating both GFRP and timber. Therefore, considering the benefit of sustainable design and anticorrosion merit of GFRP-timber bolted connection, this paper presents an experimental study on the group effect of this new type of connection. 46 GFRP-timber connections were fabricated and tested in tension. The effects of four variables on the group effect were considered including bolt diameter, bolt pitch distance, number of bolts, and number of bolt columns. The failure modes, load-displacement curves and capacities of the connections were reported. The effective number of bolts were calculated using the experimental results, and the effects of the parameters on the effective number of bolts and group effect coefficients were analyzed. The experimental results suggested that the GFPR-timber connection should use bolts with a diameter no less than 10 mm, and the number of bolt columns should be no more than three. The design equation in Eurocode 5 was modified considering the effects of various parameters on the group effect. The modified equation was verified by the experimental results. This paper contributes to

111 the understanding of group effect of GFRP-timber bolted connections and is
112 beneficial for the future development of associated design guidelines.

113 2. Experimental Program

114 2.1. Materials

115 The pultruded GFRP plates were cut from a large panel with a measured thickness of
116 2.3 mm. Tensile coupon tests were conducted according to ASTM D 3039 [38] to
117 determine the tensile Young's modulus and strength. Three coupons were cut along
118 the pultrusion direction. All coupons had the same dimension of 350 × 80 mm (length
119 × width). The coupon tests were conducted on an Instron 8802 testing machine. The
120 coupon was directly clamped on the grips of the machine with a gripping length of
121 100 mm. No slip was observed during all tests and all coupons failed away from the
122 gripping area. The measured tensile modulus and strength in the pultrusion direction
123 were 20 GPa and 348 MPa, respectively.

124 Off-axis tensile coupon tests were conducted according to ASTM D 3518 [39] to
125 determine the in-plane shear strength. Three coupons were cut in a direction 45
126 degrees from the pultrusion direction. All coupons had the same dimension of 350 ×
127 25 mm (length × width). The coupon tests were conducted on the same Instron
128 machine. The failure mode is shown in Fig. 1. The in-plane shear strength in the 45°
129 direction were 44 MPa.

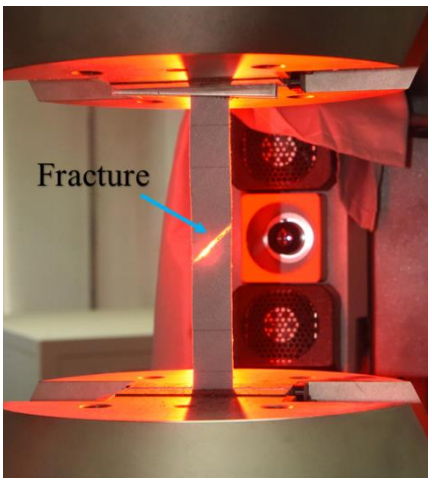


Fig. 1. Off-axis tensile tests

The matrix and the reinforcement of the pultruded GFRP plate are polyester resin and E-glass fiber, respectively. The fiber mass fraction and the reinforcement architecture were characterized using burn off test according to ASTM D 3171 [40]. Three small square samples were cut from a large GFRP panel. The mass of each specimen before and after the test was measured using a digital balance with an accuracy of 0.001 g. The fiber mass fraction is calculated and shown in Table 1. As can be seen in Fig. 2, the reinforcement architecture of this pultruded GFRP plate is symmetric and balanced with roving in the center, and a layer of chopped strand mat on each side of the section. The average measured fiber mass fraction is 62.45%. Unidirectional fiber roving accounts for about 70% of all fibers.

It should be noted that, the connections of timber structures may be subjected to complicated loading conditions. Therefore, it is recommended that the fiber layup of GFRP plates be designed and optimized according to the specified loading conditions. In other words, the multiple fiber layers may be required and they should be oriented in the most suitable directions. Pultruded GFRP plates were selected in the current

147 study because the joints were loading in tension direction. Therefore, it is the most
148 suitable to have fibers oriented parallel to the tensile loading. In addition, the focus of
149 this paper is on the effect of bolt arrangements on the group effects of bolted
150 connection. Therefore, the fiber orientation was fixed for all specimens and its effect
151 can be minimized.

152 Table 1. Measurement of fiber mass fraction

ID	Mass before test/g	Mass after test/g	Fiber mass fraction
1	3.438	2.140	62.25%
2	3.569	2.242	62.82%
3	3.647	2.271	62.27%
	Average		62.45%



153

154 Fig. 2. Fiber architecture after the resin was burnt off according to ASTM D 3171

155 Pine wood was used for the connections. Tensile coupon tests were conducted
156 according to ASTM D 3039 [38] to determine the tensile Young's modulus and
157 strength. Three coupons were cut along the grain direction. All coupons had the same
158 dimension of 360 × 30 mm (length × width). The coupon tests were conducted on an
159 Instron machine. The coupon was directly clamped on the grips of the machine with a
160 gripping length of 100 mm. No slip was observed during all tests and all coupons
161 failed away from the gripping area. The measured tensile modulus and strength in the
162 grain direction were 7 GPa and 27 MPa, respectively.

Moisture content and density of timber were measured according to ASTM D 4442 [41] and D 2395 [42] respectively. This was to ensure all timber plates of the GFRP-timber connections had similar conditions. Therefore, immediately after the connection test (see Section 2.2), at least one timber block sample was cut from every specimen and the dimensions were measured to the nearest 0.01 mm using an electronic Vernier caliper. The mass of each timber block before and after the test was measured using a digital balance with an accuracy of 0.001 g. Little variations in moisture content and density were observed in the timbers and the average measured values were 7.21% and 469kg/m^3 respectively.

The ordinary bolts were stainless steel hexagon head bolts of 40 mm long. The nominal tensile strength of this ordinary bolt is 700 MPa, which is equivalent to Grade A bolt as specified in ASTM A307 [43]. Zinc-plated fender washers were used with an inner diameter of 6.4 mm, an outer diameter of 12.5 mm, and a thickness of 1.75 mm. When used with ordinary bolts, one washer was used beneath the bolt head and another was under the nut.

2.2. GFRP-timber connections

The typical pultruded GFRP-timber bolted connection is presented in Fig. 3. Two inner timber plates were connected by two outer GFRP plates through steel bolts. The timber plate was 22 mm thick and GFRP plate has a thickness of 2.3 mm. The thicknesses were selected to make sure timber and GFRP plates have similar tensile stiffness. It can be seen in Fig. 3 that, there are three bolts aligned in a column on one side whereas six bolts are deliberately placed on the other side of the connection. The

aim is to ensure the failure only occurs on the side of interest (three bolts side), which is called observation side, and the other side is called safe side. In the following discussions, the bolt row refers to a group of bolts which are aligned transverse to the loading direction. Similarly, the bolt column means a group of bolts aligned in the loading direction.

Dimensional parameters for timber plate were designed according to the Eurocode 5 [23] , where it is required that: (1) the minimum distance between bolt holes, center-to-center, should be no less than five times the diameter of the hole; (2) the minimum end distance, center-to-end, should be no less than seven times the diameter of the hole or 80mm; and (3) the minimum side distance, center-to-side, should be no less than three times the diameter of the hole. Dimensional parameters for GFRP plate were designed according to Pre-Standard for LRFD of Pultruded FRP Structures (ACMA) [44] , where it is required that: (1) the minimum distance between holes, center-to-center, should be no less than four times the diameter of the hole; (2) the minimum end distance, center-to-end, should be no less than four times the diameter of the hole; and (3) the minimum side distance, center-to-side, should be no less than 1.5 times the diameter of the hole. Because the joints were all under tension, the bolts were in pure shearing condition (see Fig. 3). All bolts were tightened to a snug tight condition, which is defined as the tightness attained by the resistance to bolt tightening using human fingers only. This relatively low tightening torque was chosen to negate the complication of a significant long-term reduction in clamping tension from material creep and viscoelastic relaxation [45]. A drilling machine was used to

machine the holes and the dimension of the bolt holes were controlled using the drilling bits. The dimension of the holes all satisfied the requirements in the abovementioned standards.

In the following discussions, the bolt diameter is denoted by ‘d’. The bolt distance, center-to-center, measured in the loading direction is called pitch distance and is represented using ‘p’. The bolt distance, center-to-center, measured perpendicular to the loading direction is called gauge distance and is named ‘g’. In the current study, the gauge distance is kept constant as 40 mm. The edge distance from bolt center to the edge of timber plate is denoted by ‘e_t’, and the edge distance from bolt center to the edge of GFRP plate is represented by ‘e_{frp}’ (see Fig. 3).

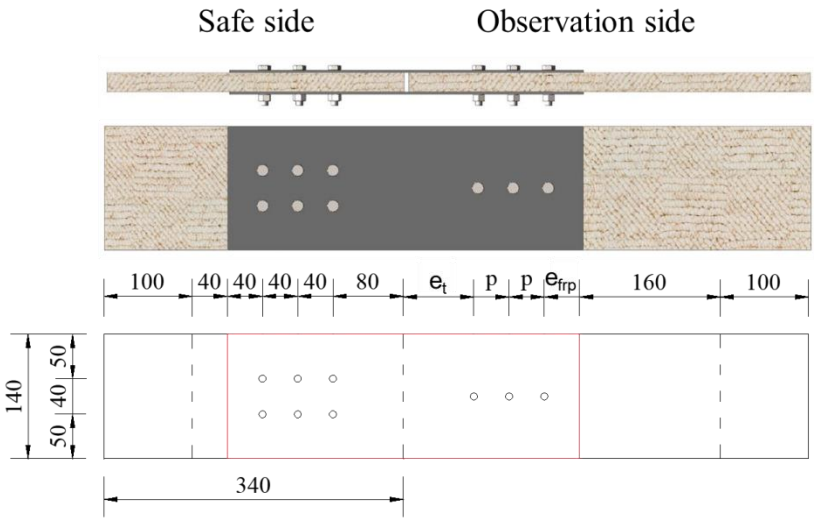


Fig. 3. Configurations and dimensions of a typical GFRP-timber connection of row number group (all dimensions are in mm)

Five groups of GFRP-timber connections were prepared as shown in Fig. 4 considering effects of different parameters on the group effect. The parameters considered in the current study include bolt diameter, bolt pitch distance, number of

223 bolt rows and number of bolt columns. The first group includes specimens with only
224 one bolt of different diameters, i.e. 6 mm to 14 mm with 2 mm increment as shown in
225 Fig. 4(a). The first group is called control group because the specimens in this group
226 only have one bolt. The specimens in other groups have multiple bolts. Therefore, the
227 group effect of bolted connections can be investigated by comparing the results of
228 other groups with those of control group.

229 The second group in Fig. 4(b) is called bolt diameter group. All specimens in this
230 group have two bolts of various diameters, i.e. 6 mm to 14 mm with 2 mm increment.
231 The effect of bolt diameter on the group effect can be studied by comparing this group
232 with control group. The third group shown in Fig. 4(c) is bolt pitch distance group.
233 All specimens in this group have two bolts of the same diameter of 8 mm, but with
234 changing bolt pitch distance i.e. from 40 mm to 120 mm with 20 mm increment. The
235 effect of number of bolt rows on the group effect is studied using the fourth group
236 shown in Fig. 4(d). The specimens in this group have one column of bolts but
237 multiple rows changing from one to four rows. The bolt diameter is 8 mm with
238 constant bolt pitch distance of 40 mm. The last group shown in Fig. 4(e) is to study
239 the effect of number of bolt columns on the group effect. The specimens in this group
240 have two rows of bolts but changing number of columns from one to four. The bolts
241 have a diameter of 8 mm and a pitch distance of 40 mm. One specimen was
242 disassembled to show the detailed elements of the GFRP-timber connection in Fig.
243 4(f).

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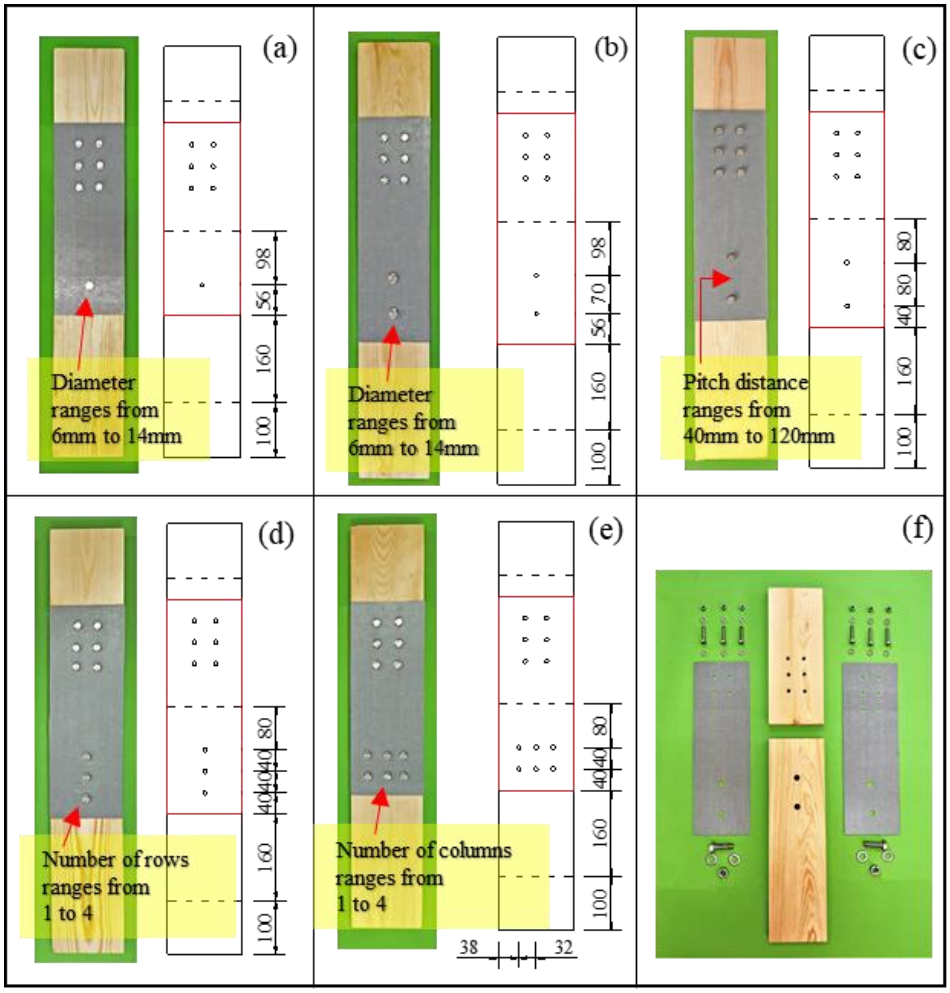


Fig. 4. Configurations and dimensions of five groups of specimens for investigation of different parameters on the group effect: (a) control group; (b) bolt diameter; (c) bolt pitch distance; (d) number of bolt rows; (e) number of bolt columns; (f) disassemble of a typical specimen showing detailed elements in the connection (all dimensions are in mm)

All specimens are listed in Table 2. For each configuration shown in Fig. 4, two identical specimens were prepared and tested for repeating purpose. Therefore, a total of 42 specimens were tested in the current study. As can be seen in Table 2, all specimens are categorized into five groups. The label "C" for specimens in the first group means control group. Similarly, the specimen labels in other groups, i.e. "Dia",

256 “Dis”, “N” and “M” represent bolt diameter, bolt pitch distance, number of bolt rows
257 and number of bolt columns, respectively.

258 Table 2. Dimensions and experimental results of GFRP-timber bolted connections

Category	ID	m	n_c	d/mm	p/mm	e_t/mm	e_{frp}/mm	F_a/kN
Control group	C-6	1	1	6	-	98	56	4.96
	C-8	1	1	8	-	98	56	8.93
	C-10	1	1	10	-	98	56	8.27
	C-12	1	1	12	-	98	56	9.69
	C-14	1	1	14	-	98	56	11.05
Bolt diameter	Dia-6	1	2	6	70	98	56	12.65
	Dia-8	1	2	8	70	98	56	15.02
	Dia-10	1	2	10	70	98	56	16.44
	Dia-12	1	2	12	70	98	56	23.12
	Dia-14	1	2	14	70	98	56	25.44
Bolt pitch distance	Dis-40	1	2	8	40	80	40	15.60
	Dis-60	1	2	8	60	80	40	13.79
	Dis-80	1	2	8	80	80	40	14.26
	Dis-100	1	2	8	100	80	40	14.62
	Dis-120	1	2	8	120	80	40	14.04
Row number	N-1	1	1	8	-	80	40	8.02
	N-2	1	2	8	40	80	40	15.60
	N-3	1	3	8	40	80	40	17.70
	N-4	1	4	8	40	80	40	29.92
Column number	M-1	1	2	8	40	80	40	15.60
	M-2	2	2	8	40	80	40	24.65
	M-3	3	2	8	40	80	40	32.84
	M-4	4	2	8	40	80	40	40.99

259 NOTE: m is the number of bolt columns and n_c is the number of bolts in one column.
260 p is the pitch distance between bolt centers in the loading direction. e_t and e_{frp} are the
261 edge distance from bolt center to the edge of timber plate and GFRP plate respectively
262 (see Fig. 3). F_a represents the average capacity of the two repeating tests.

2.3. Experimental set up and loading programs

The experimental set up is shown in Fig. 5. All tension tests were carried out using an Instron 8802 hydraulic machine. A bubble level was used to ensure the alignment and straightness of the specimen during installation. All specimens had a clamping length of 100 mm at both ends. The two grips were driven by hydraulic power and were carefully tuned to make sure there was no slip between the specimen and the clamps. The displacement of the observation side of the specimen was measured using a linear variable differential transformer (LVDT). The static tension tests were carried out under displacement control at a loading speed of 2 mm/min until final failure [25, 26, 46, 47].

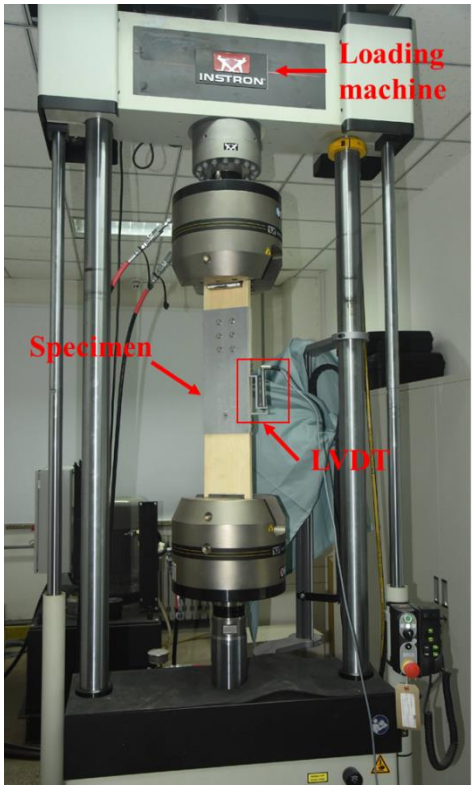


Fig. 5. Experimental set up

3. Experimental Results and Discussions

3.1. Failure modes

The specimens in the control group had one bolt of five different diameters. The typical failure mode is shown in Fig. 6(a). All the specimens showed bolt hole bearing failure in the timber plates. That is, the bolt squeezed the hole and slipped in the timber plate. The original round smooth bolt holes (see Fig. 3f) became elongated and formed a groove. The bearing failure is a progressive failure mode and can be clearly monitored. Both the bolt and the GFRP plates were not damaged.

The specimens in the bold diameter group had two bolts of five different diameters. The typical failure mode is shown in Fig. 6(b). When the bolt diameter is less than or equal to 10 mm, the specimens exhibited bolt hole bearing failure in the timber plates, the same as the control group (see Fig. 6(a)). While the GFRP plates and bolts of these specimens were not damaged. On the other hand, when the bolt diameter is greater than or equal to 12 mm, bearing failure of the holes in GFRP plates were also observed in addition to the bearing failure in the timber plate. It should be noted that, for all specimens, when further loaded after the bearing failure was observed, all specimens finally failed with cleavage cracking in the timber plates.

The specimens in the bolt pitch distance group had two bolts of the same diameter of 8 mm, but changing pitch distances of 40 mm, 60 mm, 80 mm, 100 mm, 120 mm. The typical failure mode is shown in Fig. 6(c). All the specimens initially exhibited timber bearing failure. When further loaded, bearing damage of the holes in GFRP plates started to show, and finally cleavage failure occurred in the timber plate. Therefore,

298 similar to specimens in the bolt diameter group, the mechanical behavior of the
299 connections was determined by the bearing failure.

300 The specimens in the bolt row number group only had one bolt column. The number
301 of bolts (or bolt rows) changed from 1 to 4. All bots had the same diameter of 8 mm
302 and the same pitch distance of 40 mm. The typical failure mode is shown in Fig. 6(d).
303 It was observed that, when there were only 1 or 2 bolt rows (number of bolts), the
304 connection failed by the progressive bearing failure of the holes in the timber plates.
305 However, for specimens with more bolt rows (e.g. 3 or 4 bolts), the connections failed
306 by cleavage failure in the timber plates. A crack suddenly formed connecting the bolt
307 holes in the loading direction, splitting the timber plates in halves. Different from the
308 progressive bearing failure, the cleavage failure was fast and sudden without warning
309 and the load dropped immediately when it occurred.

310 The specimens in the bolt column number group had two rows of bolts with the same
311 diameter of 8 mm. The number of bolt columns varied from 1 to 4. The typical failure
312 mode is shown in Fig. 6(e). When the specimen had only one row of bolts, bearing
313 failure was observed in the timber plates. However, when the specimen had 2 or more
314 rows of bolts, cleavage failure occurred with a crack suddenly formed in the timber
315 plate along a column of bolt holes. No damages were observed in the GFRP plates
316 and bolts.

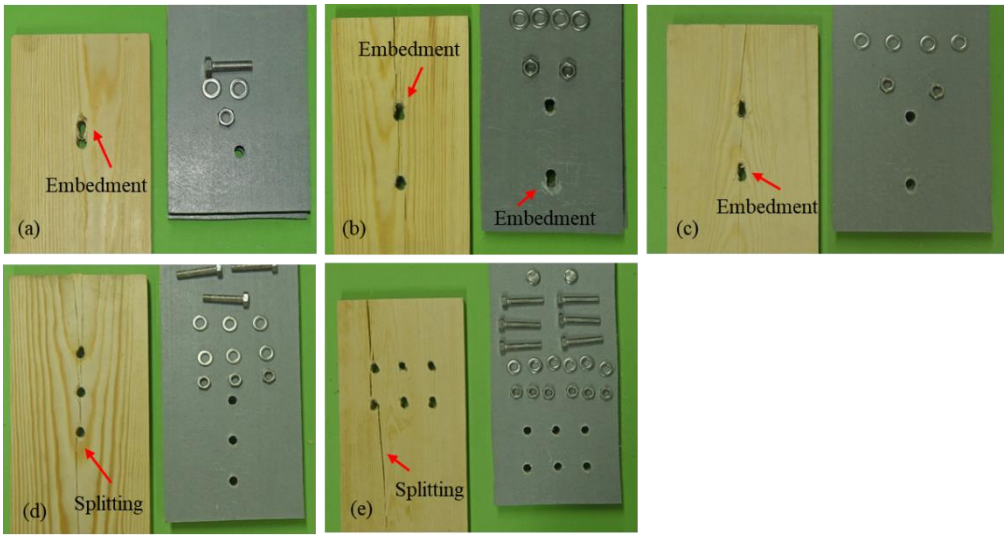


Fig. 6. Failure modes of specimens considering effects of (a) control group; (b) bolt diameter; (c) bolt pitch distance; (d) number of bolt rows; (e) number of bolt columns

3.2. Load-Displacement Curves

3.2.1 Bolt diameter group

The load-displacement curves of specimens in the bolt diameter group are shown in Fig. 7. The load initially linearly increased with displacement upto a peak point, after which the load dropped a little bit and maintained at a relatively high level with small fluctuations. No sudden load drop was observed in all curves. It can be seen in Fig. 7 that, the slope of the initial linear stage of all curves was similar. This indicates that the bold diameter had little effect on the stiffness of the connection. On the other hand, the bold diameter was linearly correlated to the peak, i.e. specimen with larger bolts achieved higher peak load.

The load-displacement behaviors in Fig. 7 are closely related to the failure mode of corresponding specimens. As described in Fig. 6, the specimens in the bold diameter group failed by bearing failure of bolt holes in the timber plates. When loaded, the

bolt shafts started contacting the inner surface of the bolt holes, and the load path started to build up to carry the tension load. Therefore, the load initially increased linearly with the displacement. When the material at the inner surface of the bolt holes became damaged due to the high pressure induced by the bolt shafts, the peak load of the specimen was achieved. After that the load fluctuation started due to the progressive nature of the bearing failure mode. This load fluctuation is also observed in the study by Mustafa [48].

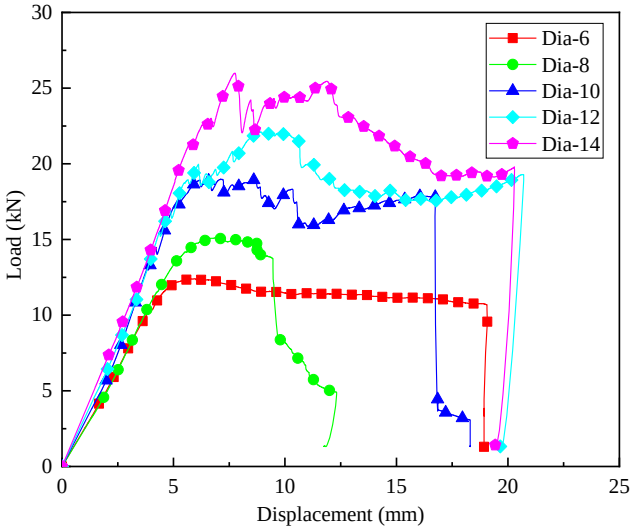
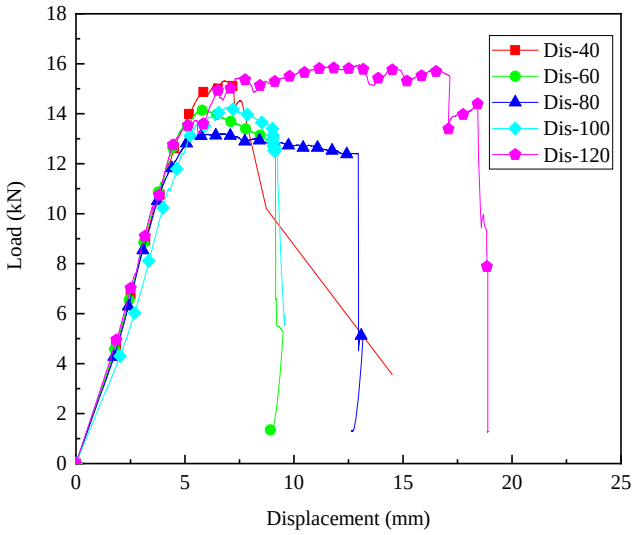


Fig. 7. Load-displacement curves of specimens in the bolt diameter group

3.2.2 Bolt pitch distance group

Fig. 8 shows the load-displacement curves of specimens in the bolt pitch distance group. Similarly, the load linearly increased with the displacement to a peak point, after which the load maintained at a high magnitude until the failure of connection. The load-displacement curves were also related to the bearing failure of the bolt holes so no sudden drop in the load was observed.

348 It can be seen in Fig. 8 that, the bolt pitch distance has little effect on the stiffness of
349 the connection considering the similar slopes of the initial linear stage of all curves. In
350 addition, it seems that the bolt pitch distance also has little effect on the peak load
351 (capacity) of the connection. This can be attributed to several reasons: (1) the
352 dimensional parameters of the connection (e.g. pitch distance, edge distance etc.)
353 satisfy all requirements of the design guideline, and all specimens have the same
354 dimensional parameters, the same number of bolts with the same diameter; (2) the
355 mechanical behavior of the connection is determined by the load carrying mechanism
356 which is related to the bearing behavior between bolt shaft and bolt hole. Because all
357 specimens have the same number of bolts with the same diameter, so the number of
358 bearing holes and the size of the bearing surfaces are all the same. Therefore, the
359 variation of the bolt pitch distance may have little effect on the capacity of the
360 connection given all other dimensional parameters remain unchanged.



361

362 Fig. 8. Load-displacement curves of specimens in the bolt pitch distance group

3.2.3 Row number group

The load-displacement curves of specimens in the bolt row number group are shown in Fig. 9. Two types of curves can be observed. For specimens with one or two bolts, their load-displacement curves exhibit two stages, i.e. initial linear stage and load fluctuation stage after the peak. This is related to the progressive bearing failure mode of bolt holes. On the other hand, when the connection has three or four bolts, only the initial linear stage can be observed in the load-displacement curves. The load dropped immediately when the peak load was reached. This is because these connections failed by cleavage failure of the timber plate, which is a type of brittle failure.

It can also be observed that the connection stiffness was positively correlated with the number of bolts. The peak load also proportionally increased when the connection had more bolts. This is reasonable because higher capacity and stiffness can be achieved when more bearing surfaces engage in carrying the load.

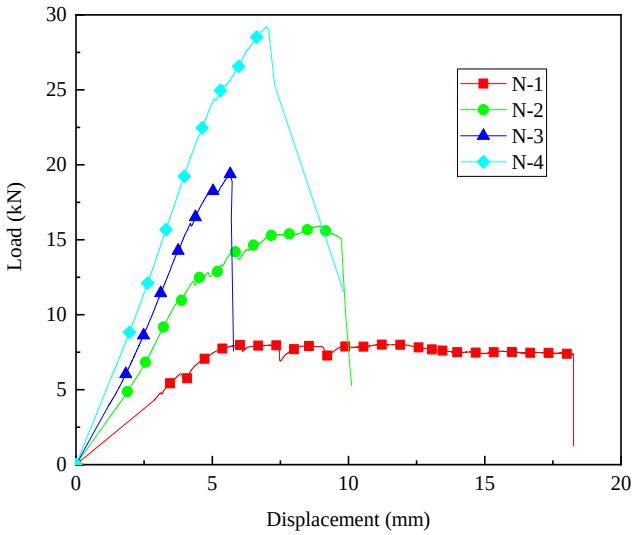


Fig. 9. Load-displacement curves of specimens in the row number group

3.2.4 Column number group

The load-displacement curves for all specimens in the bolt column number group are shown in Fig. 10. For all specimens, the load linearly increased with the displacement upto a peak point after which the load dropped suddenly. This sudden load drop is related to the timber cleavage failure which is brittle. Similar to Fig. 9, the stiffness and peak load of the connections are proportional to the number of bolt columns because more bolts engaged in load carrying mechanism.

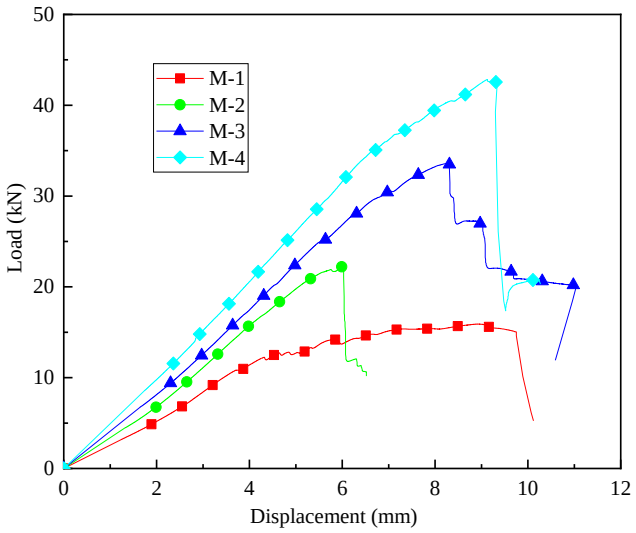


Fig. 10. Load-displacement curves of specimens in the column number group

4. Load Carrying Capacities and Group Effect of GFRP-Timber Bolted Connections

In the current study, the peak load of the load-displacement curve is defined as the load carrying capacity of the corresponding connection. The load carrying capacities of all specimens are listed in Table 2. Group effect is an important concept for design of bolted connections considering the fact that a multiple-bolt connection does not necessarily have a capacity of ‘n’ (n is the number of bolts) times the capacity of the

same connection with only one bolt. The group effect can be represented by Eq. 1, where $F_{multiple}$ is the capacity of a connection with multiple bolts, and F_{single} is the capacity of the same connection with only one bolt.

$$n_{ef} = \frac{F_{multiple}}{F_{single}} \quad (1)$$

In Eq. 1, n_{ef} is the number of bolts which effectively engage in load carrying mechanism, so it is called effective number of bolts of the connection. The mechanical behaviour of the bolted connection is affected by many factors including the material properties, arrangement of bolts etc., meaning that not all bolts are effective. So it is important to understand how different parameters affect n_{ef} so that the capacity of multi-bolt connection can be predicted.

In the current study, the effect of four parameters on the group effect of the GFRP-timber bolted connection are investigated, including bolt diameter, bolt pitch distance, bolt row number and bolt column number.

4.1. Effect of bolt diameter

The effect of bolt diameter on the group effect can be analyzed by comparing the results of the specimens in the bolt diameter group with the results of specimens in the control group. The specimens in the bolt diameter group have two bolts while specimens in the control group have only one bolt. The load carrying capacities of specimens in both groups are plotted against the bolt diameters in Fig. 11. It can be seen that, the load carrying capacity is almost linearly correlated to the bolt diameter. For the control group, the capacity increased from 4.96 kN to 11.05 kN when the bolt

diameter increased from 6 mm to 14 mm. Similarly, the specimens in the bolt diameter group showed an increase in capacity from 12.65 kN to 25.44 kN when the bolt diameter increased from 6 mm to 14 mm. The improvement in capacity is because the contact area between the bolt and bolt hole became larger when the bolt diameter increased.

n_{ef} is also plotted against the bolt diameter in Fig. 11. n_{ef} is the ratio between the capacity of the specimen in the bolt diameter group and the capacity of the corresponding specimen in the control group with a bolt of the same diameter. Fig. 11 shows that n_{ef} also increases with the bolt diameter, indicating more bolts became effective and contributed to the load carrying capacity when larger bolts were used. It should be noted that, for specimen with two 6 mm bolts, n_{ef} is 2.55, which seems unreasonable. This is most probably because the control specimen showed a very low capacity due to stress concentration at the bolt hole when a 6 mm bolt was used. It seems the stress concentration effect induced by the small diameter bolt can be minimized by using more bolts. Another interesting observation is that, for specimens with two bolts of 12 mm and 14 mm diameters, n_{ef} is greater than 2. This might be because larger bolt diameter is more helpful to reduce the stress concentration effect at the bolt hole, and to achieve a better connection integrity when engaging in load carrying mechanism. Therefore, it is recommended that bolt diameter be no less than 10 mm to avoid severe stress concentration at the bolt hole and to achieve a better connection integrity.

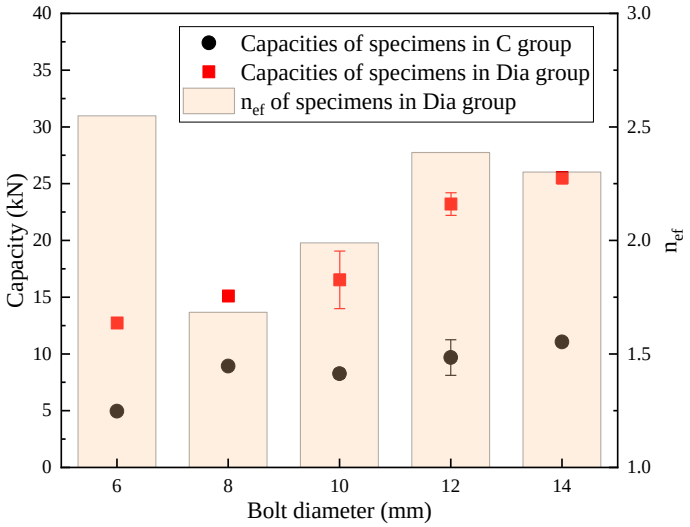


Fig. 11. Effect of bolt diameter on the load carrying capacity and n_{ef}

4.2. Effect of bolt pitch distance

n_{ef} and the load carrying capacities of specimens in the bolt pitch distance group are plotted against bolt pitch distance in Fig. 12. The specimens in the bolt pitch distance group have two bolts with a diameter of 8 mm. The pitch distance between these two bolts varied from 40 mm to 120 mm with 20 mm increment. n_{ef} is obtained by dividing the capacity of the specimen with two 8 mm bolts by the capacity of the control specimen with one 8 mm bolt.

It seems in Fig. 12 that, the bolt pitch distance has little effect on the load carrying capacity which varied between 13.79kN and 15.60 kN. n_{ef} also did not change much with the increase of bolt pitch distance. This indicates that when the bolt pitch distance satisfies the design requirements (see Section 2.2), the effect of bolt pitch distance on the group effect can be neglected. Similar observation was also reported by Jensen et al. [8]. It is also noted that, satisfying the bolt pitch distance requirement by the design standards does not guarantee all bolts are effective in carrying the load,

because n_{ef} is generally lower than 2 for all specimens in the bolt pith distance group. Therefore, it can be said that, the requirements regarding the bolt pitch distance limit in the current standards are reasonable. The experimental results showed that when the design requirement was satisfied, the load carrying capacity of the bolted connection would not be much affected by the bolt pitch distance.

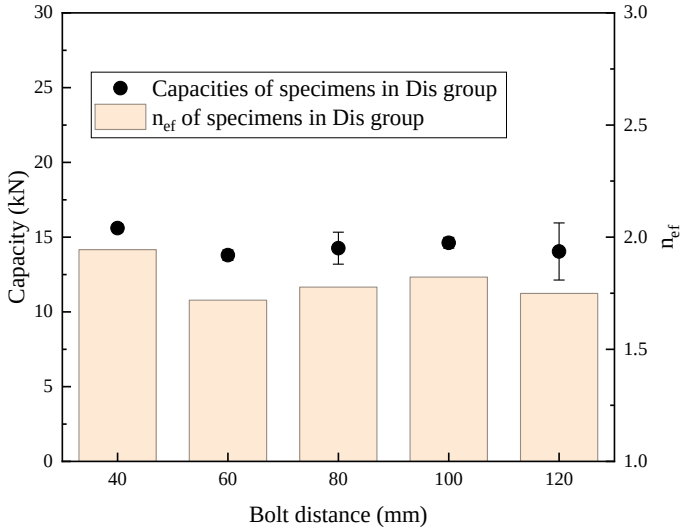


Fig. 12. Effect of bolt pitch distance on the load carrying capacity and n_{ef}

4.3. Effect of bolt row number

The load carrying capacities of specimens in the bolt row number group are presented in Fig. 13. All specimens have one column of bolts and changing number of rows from 1 to 4. The bolt diameter was 8 mm for all specimens. Therefore, n_{ef} can be simply calculated by comparing the capacity of a specimen with multi row of bolts with the capacity of the specimen with only one bolt.

It is obvious in Fig. 13 that the load carrying capacity increases with the number of bolt rows. For example, the capacity of the connection with one 8 mm bolt is 8.03 kN. It grows to 15.60 kN, 17.70 kN and 29.92 kN respectively when 2, 3 and 4 bolts are

used in the connection. Similarly, n_{ef} also increased from 1.94 to 3.73 with the row number varied from 2 to 4. It should be noted that, n_{ef} is always lower than the actual number of bolts in the connection (n) regardless the bolt row number. This observation means that simply increasing the number of bolt rows is not effective to achieve 100% utilization of the bolts.

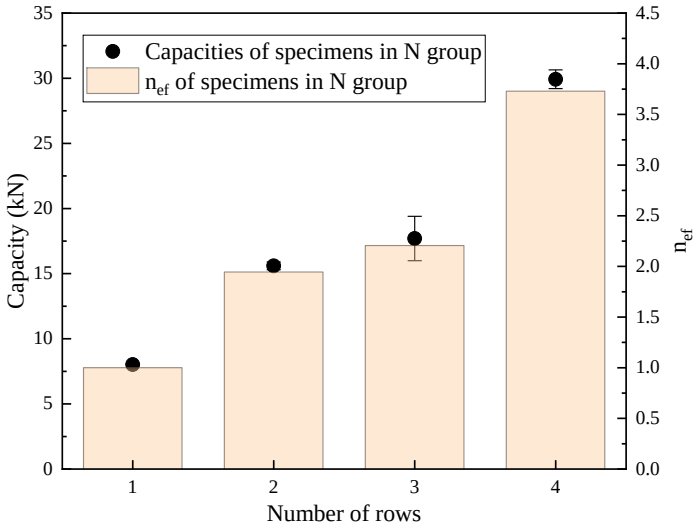


Fig. 13. Effect of bolt row number on the load carrying capacity and n_{ef}

4.4. Effect of bolt column number

The specimens in the bolt column number group have 2 rows of bolts but changing number of bolt columns from 1 to 4. All bolts have a diameter of 8 mm. The load carrying capacities of the connections are plotted against the number of bolt columns in Fig. 14. n_{ef} is calculated by dividing the capacity of specimens in bolt column number group by the capacity of the control specimen with only one 8 mm bolt.

It can be seen in Fig. 14 that, the capacity of the connection increased from 15.50 kN to 40.99 kN when the number of bolt columns grew from 1 to 4. Similarly, n_{ef} is

483 increased from 1.94 to 3.07, 4.09 and 5.11, when the actual number of bolts increased
484 from 2 to 4, 6 and 8.

485 Similar to the concept of effective number of bolts, for connections with multi number
486 of bolt columns, the effective number of bolt columns can also be defined as the
487 capacity of connection with multi number of bolt columns divided by the capacity of
488 the same connection with only one column of bolts. Therefore, the effective numbers
489 of bolt columns for connections with 2, 3 and 4 columns of bolts are 1.58, 2.11 and
490 2.63 respectively. That is to say the number of bolt columns has obvious effect on the
491 group effect of the bolted connection. This observation has not been reported in the
492 literature. Also, in the current international design standards [23], this phenomenon is
493 not considered as well. In other words, according to the current design standards, the
494 capacity of a connection with multi number of bolt columns can be calculated by
495 multiplying the number of bolt columns with the capacity of the same connection with
496 only one bolt column. According to the experimental results in this paper, the capacity
497 of the connection will be overestimated if the effect of column numbers on the group
498 effect is not considered. Therefore, the design guideline is modified based on the
499 experimental results in this paper to consider the bolt column number effect. This will
500 be further discussed in Section 5.

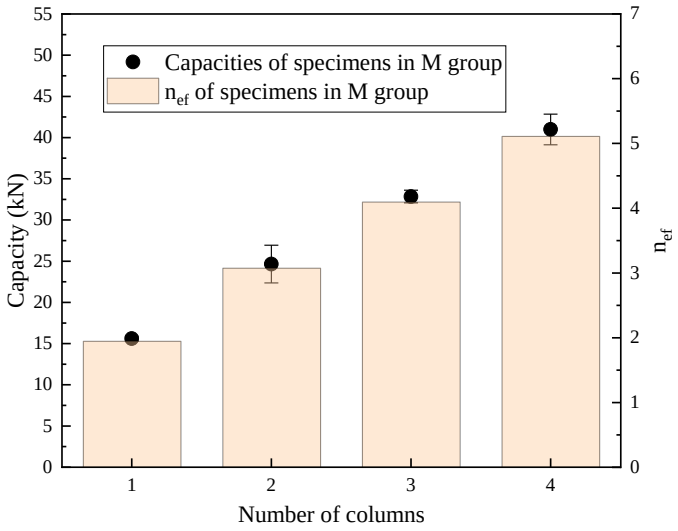


Fig. 14. Effect of bolt column number on the load carrying capacity and n_{ef}

4.5. Effect of different parameters on the group effect coefficient R

From the above discussions, the four parameters have obvious effects on the load carrying capacities and n_{ef} of the connections. In order to further investigate the four parameters on the group effect, another index is introduced as "group effect coefficient", which is given by Eq. 2:

$$R = \frac{n_{ef}}{n} \times 100\% \quad (2)$$

R reflects the connection efficiency in terms of utilizing the bolts. For example, a connection with R=80% means that any bolt in the connection only contributes 80% of its capacity. R is listed in Table 3 and plotted against the four parameters including bolt diameter, bolt pitch distance, bolt row number and bolt column number in Fig. 15(a) to (d), respectively.

For the bolt diameter group, the group effect coefficient increased from 84% to 115% when the bolt diameter increased from 8 mm to 14 mm. This shows that when the diameter of the bolt is increased, the problem of stress concentration caused by the

516 bolt pressing the bolt hole is alleviated, and the force transmission efficiency of the
517 bolt is increased.

518 As for the effect of bolt pitch distance, when it varied from 40mm to 120mm, the
519 group effect coefficient exhibited little changes, as shown in Fig. 15(b). Therefore, the
520 bolt pitch distance has negligible effect on the group effect. This is because only
521 changing the bolt pitch distance does not alter the load carrying mechanism which is
522 mainly determined by the number of contacting surfaces of the bolt holes. This
523 observation also confirms the correctness of the bolt pitch distance requirement in the
524 current design standards.

525 The effect of number of bolt rows is presented in Fig. 15(c). The group effect
526 coefficients are 100%, 97% and 93% for the number of bolt rows of 1, 2 and 4,
527 respectively. That means the bolt utilization efficiency does not change much with the
528 increase of number of bolt rows. Similar observation was also reported by Jensen and
529 Quenneville [8], who found that the group effect coefficients were 80.4%, 94.2% and
530 96.4%, respectively, when the number of bolt rows were 2, 3 and 4. Gattesco [7] also
531 reported that the group effect coefficients were 0.87, 0.87, 0.83 and 0.75, respectively,
532 when the bolt row numbers were 2, 3, 4 and 6.

533 On the other hand, The number of bolt columns exhibits obvious effect on the group
534 effect coefficient in Fig. 15(d). The group effect coefficients of the connection
535 decreased by 34% from 97% to 64% when the number of bolt columns grew from 1 to
536 4. This indicates that the more columns of bolts that a connection has, the less

effective of the bolts in terms of carrying the load. In other words, it is not effective to have more columns of bolts in a connection.

According to the observations in Fig. 15, it is recommended that the bolt diameter should not be less than 10 mm and the number of bolt columns should not exceed three for the design of a bolted connection.

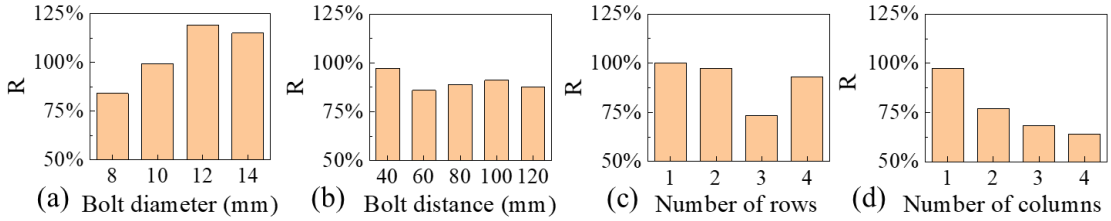


Fig. 15. Group effect coefficient of specimens considering effects of (a) bolt diameter; (b) bolt pitch distance; (c) number of bolt rows; (d) number of bolt columns

5. Design of GFRP-Timber Bolted Connections Considering Group Effect

According to the Eurocode 5 [23], if there are 'n_c' bolts in one column of a bolted connection parallel to the grain direction of the timber, the effective number of bolts 'n_{ef}' shall be calculated according to Eq. 3:

$$n_{ef} = \min \begin{cases} n_c \\ n_c^{0.9} \cdot \sqrt[4]{\frac{p}{13d}} \end{cases} \quad (3)$$

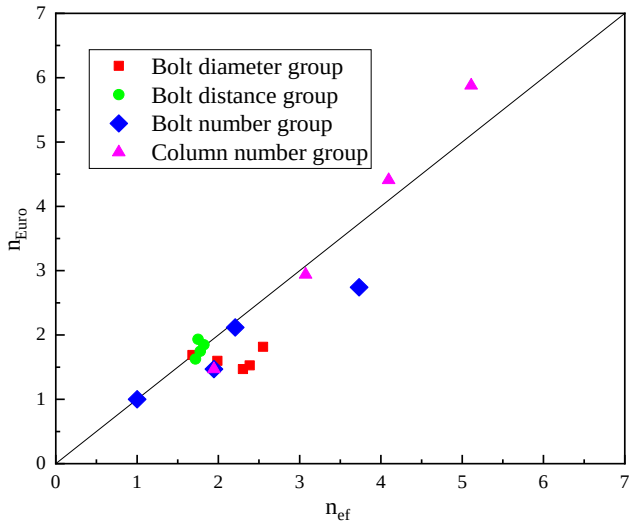
where p is the pitch distance, d is the bolt diameter. It can be seen from Eq. 3 that the Eurocode 5 considers the effects of three key parameters on n_{ef}, including the number of bolts, bolt pitch distance and bolt diameter, without considering the effect of the number of bolt columns. The number of effective bolts of each specimens was calculated using Eq. 3 and denoted as $n_{ef/Euro}$. It is compared in Table 3 and Fig. 16 with $n_{ef/EXP}$ (the effective number of bolts calculated from the experimental

555 capacities). It can be seen in Fig. 16 that the predicted $n_{ef/Euro}$ generally agree well
556 with the experimental results $n_{ef/EXP}$. The average ratio of $n_{ef/Euro}/n_{ef/EXP}$ is 0.89 with a
557 standard deviation of 0.16. This indicates the predicted $n_{ef/Euro}$ is generally
558 conservative and lower than the experimental $n_{ef/EXP}$.

559 Table 3 Effective number of bolts and group effect coefficients of all specimens

Category	ID	$n_{ef/EXP}$	R	$n_{ef/Euro}$	$n_{ef/Euro}$ /$n_{ef/EXP}$	F_c	F_c / F_a
Bolt diameter	Dia-6	2.55	127%	1.82	0.71	9.01	0.71
	Dia-8	1.68	84%	1.69	1.00	15.09	1.00
	Dia-10	1.99	99%	1.60	0.80	13.22	0.80
	Dia-12	2.39	119%	1.53	0.64	14.80	0.64
	Dia-14	2.30	115%	1.47	0.64	16.24	0.64
Bolt pitch distance	Dis-40	1.94	97%	1.47	0.76	11.79	0.76
	Dis-60	1.72	86%	1.63	0.95	13.04	0.95
	Dis-80	1.78	89%	1.75	0.98	14.02	0.98
	Dis-100	1.82	91%	1.85	1.01	14.82	1.01
	Dis-120	1.75	87%	1.93	1.10	15.51	1.10
Row number	N-1	1.00	100%	1.00	1.00	8.02	1.00
	N-2	1.94	97%	1.47	0.76	11.79	0.76
	N-3	2.21	74%	2.12	0.96	16.98	0.96
	N-4	3.73	93%	2.74	0.74	21.99	0.74
Column number	M-1	1.94	97%	1.47	0.76	11.79	0.76
	M-2	3.07	77%	2.94	0.96	24.75	1.00
	M-3	4.09	68%	4.41	1.08	32.41	0.99
	M-4	5.11	64%	5.88	1.15	40.07	0.98
Average value					0.89		0.88
Standard deviation					0.16		0.15

560 NOTE: $n_{ef/EXP}$ means the number of effective bolts of the connection calculated using
561 experimental results. $n_{ef/Euro}$ represents the number of effective bolts calculated using
562 Eq. 3. R represents the group effect coefficient. F_c means the capacity calculated by
563 Eq. 6.



564

565 Fig.16 Comparison of $n_{ef/Euro}$ and $n_{ef/EXP}$

566 As mentioned above, Eq. 3 in Eurocode 5 does not consider the effect of the number
567 of bolt columns on n_{ef}. However, the experimental results in Section 4.4 showed that
568 when the actual number of bolt columns was 2, 3, and 4, the corresponding effective
569 number of bolt columns was 1.58, 2.11, and 2.63, respectively. The effect of the
570 number of bolt columns on the group effect was also reported by Jorissen et al. [22].
571 Therefore, Eq. 3 needs to be modified to consider the effect of the number of bolt
572 columns on n_{ef}, otherwise the capacity of the bolted connection would be
573 overestimated.

574 According to the experimental results in Section 4.4, when the actual number of bolt
575 columns (m) was 1, 2, 3, and 4, the number of effective bolt columns (m_{ef}) was 1,
576 1.58, 2.11, and 2.63, respectively. Therefore, a linear relationship between m_{ef} and m
577 is proposed through curve fitting:

$$m_{ef} = \begin{cases} 1 & (m = 1) \\ 0.65 \times m + 0.8 & (m \geq 2) \end{cases} \quad (4)$$

578 Therefore, Eq. 3 for calculating n_{ef} considering m_{ef} is modified as follows:

$$n_{ef} = \min\begin{cases} n \\ m_{ef} \times n_c^{0.9} \times \sqrt[4]{\frac{p}{13d}} \end{cases} \quad (5)$$

where n is the total number of bolts; n_c is the number of bolts in one bolt column.

The load carrying capacity of a bolted connection can be calculated as:

$$F_{multiple} = n_{ef} \times F_{single} \quad (6)$$

Therefore, we only need to test the capacity of a single bolt connection (F_{single}), so that the capacity of a multi-bolt connection with the same size bolts can be predicted using Eq. 6. The load carrying capacities of all the connection in the paper were calculated using Eq. 6 and listed in Table 3. The predicted capacities are compared with the experimental capacities in Fig. 17, and a good agreement is achieved. The average ratio between the predicted and experimental capacities is 0.88 with a standard deviation of 0.14.

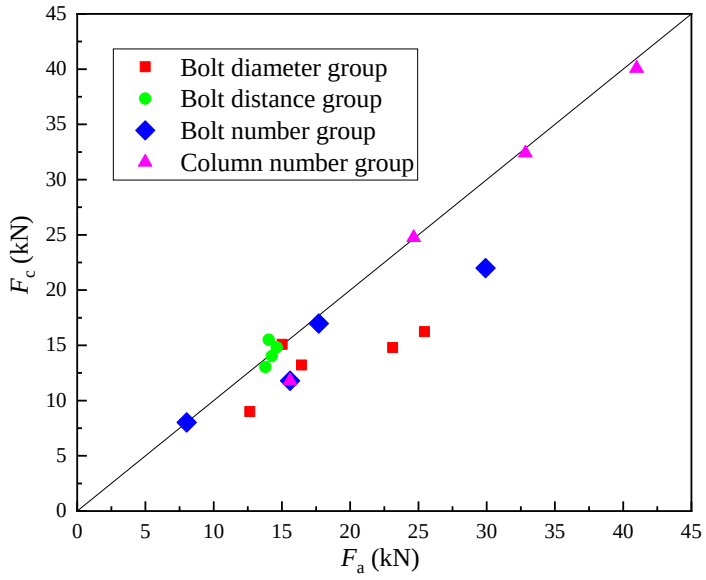


Fig.17 Comparison of F_c and F_a

6. Conclusions

This paper proposed a GFRP-timber bolted connection which can be an optional solution to resolve the potential corrosion issue of timber connections using steel plates. The group effect was investigated through experimental testing and four key parameters are considered including bolt diameter, bolt pitch distance, bolt row number and bolt column number. Based on the experimental results, the design equation of the effective number of bolts in Eurocode 5 was modified. The predicted capacities of the bolted connections agreed well with the experimental results. The following conclusions can be drawn:

(1) Most GFRP-timber bolted connections failed by bearing failure of the bolt holes in the timber plates. Bolt hole bearing failure is a progressive type of failure mode. However, when more than three bolts were used in a single bolt column, or when more than two bolt columns were used in the connection, cleavage failure was observed in the timber plate which was a sudden type of failure mode.

(2) It is recommended to use bolts with a diameter of no less than 10 mm. This is to minimize the potential stress concentration problem when small bolts contact the bolt hole. This is also beneficial to improve the integrity of the connection.

(3) It was confirmed that the requirement on the bolt pitch distance limit in the current design standard was reasonable. Because the experimental results showed that the capacity of bolted connection would not increase when the pitch distance satisfied the dimensional requirements of the design code.

(4) It was observed that the number of bolt columns played an important role of the

mechanical behavior of the bolted connection. When multi bolt columns were used in the connection, each column was not equally effective. This indicates that the number of bolt columns affects the group effect of the connection. However, this effect is not considered in the current design code, which may yield overestimated connection capacity. It was also noted that, the load carrying efficiency of each bolt decreased when more bolt columns were used in the connection. Therefore, it is recommended no more than three bolt columns be used in the connection.

(5) The equation for determination of the effective number of bolts in Eurocode 5 was modified to consider the effect of number of bolt columns. The capacities of the connections predicted using the new equation agreed well with the experimental results.

It is important to note that the experimental results of this paper are valid for the materials selected in this study and are only applicable to GFRP-timber bolted connections in tension. It is recommended that more experimental work and numerical simulations be conducted in the future to expand the database so that the design equation can be generalized and applied for GFRP-timber bolted connections.

Acknowledgments

The authors gratefully acknowledge the financial support provided by the National Science Foundation of China (51978025, 51911530208). Mr. Yao Wang and Mr. Bingchen Meng are acknowledged for their assistance of conducting the experimental tests at Beihang University.

Data Availability Statement

The raw/processed data required to reproduce these findings cannot be shared at this time as the data also forms part of an ongoing study.

REFERENCES

[1] Li JH, Rismanchi B, Ngo T. Feasibility study to estimate the environmental benefits of utilising timber to construct high-rise buildings in Australia. Build Environ 2019;147:108-20.

[2] Skullestad JL, Bohne RA, Lohne J. High-rise Timber Buildings as a Climate Change Mitigation Measure – A Comparative LCA of Structural System Alternatives. Energy Procedia. 2016;96:112-23.

[3] Awaludin A, Hirai T, Hayashikawa T, Sasaki Y. Load-carrying capacity of steel-to-timber joints with a pretensioned bolt. J Wood Sci 2008;54(5):362-8.

[4] Kharouf N, McClure G, Smith I. Postelastic Behavior of Single- and Double-Bolt Timber Connections. J Struct Eng 2005;131(1):188-96.

[5] Santos CL, De Jesus AMP, Morais JJL, Lousada JLPC. Quasi-static mechanical behaviour of a double-shear single dowel wood connection. Constr Build Mater 2009;23(1):171-82.

[6] Quenneville JHP, Mohammad M. On the failure modes and strength of steel-wood-steel bolted timber connections loaded parallel-to-grain. Can J Civ Eng 2000;27(4):761-73.

[7] Gattesco N, Toffolo I. Experimental study on multiple-bolt steel-to-timber tension joints. Mater Struct 2004;37(266):129-38.

[8] Jensen JL, Quenneville P. Experimental investigations on row shear and splitting in bolted connections. Constr Build Mater 2011;25(5):2420-5.

[9] Mohammad M, Quenneville JHP. Bolted wood-steel and wood-steel-wood connections: verification of a new design approach. Can J Civ Eng 2001;28(2):254-63.

[10] Liu W, Yang H. Research progress on modern timber structures. J Build Struct 2019;40(2):16-43.

[11] Singh PM, Anaya A. Effect of wood species on corrosion behavior of carbon steel and stainless steels in black liquors. Corrosion Sci 2007;49(2):497-509.

[12] Zelinka SL, Stone DS. The effect of tannins and pH on the corrosion of steel in wood extracts. Mater Corros 2011;62(8):739-44.

[13] Harries KA, Guo Q, Cardoso D. Creep and creep buckling of pultruded glass-reinforced polymer members. Compos Struct 2017;181:315-24.

[14] Liu T, Yang J-Q, Feng P, Harries KA. Determining rotational stiffness of flange-web junction of pultruded GFRP I-sections. Compos Struct 2020;236.

[15] Xiong Z, Liu Y, Zuo Y, Xin H. Experimental evaluation of shear behavior of pultruded GFRP perforated connectors embedded in concrete. Compos Struct

675 2019;222.
676 [16] Shekarchi M, Oskouei AV, Raftery GM. Flexural behavior of timber beams
677 strengthened with pultruded glass fiber reinforced polymer profiles. Compos
678 Struct 2020;241.
679 [17] Corradi M, Vo TP, Poologanathan K, Osofero AI. Flexural behaviour of hardwood
680 and softwood beams with mechanically connected GFRP plates. Compos Struct
681 2018;206:610-20.
682 [18] Liu T, Vieira JD, Harries KA. Lateral torsional buckling and section distortion of
683 pultruded GFRP I-sections subject to flexure. Compos Struct 2019;225.
684 [19] Zhang L, Liu W, Wang L, Ling Z. On-axis and off-axis compressive behavior of
685 pultruded GFRP composites at elevated temperatures. Compos Struct 2020;236.
686 [20] Ghadimi B, Russo S, Rosano M. Predicted mechanical performance of pultruded
687 FRP material under severe temperature duress. Compos Struct 2017;176:673-83.
688 [21] Grammatikos SA, Jones RG, Evernden M, Correia JR. Thermal cycling effects on
689 the durability of a pultruded GFRP material for off-shore civil engineering
690 structures. Compos Struct 2016;153:297-310.
691 [22] Jorissen AJ, Double shear timber connections with dowel type fasteners (Ph.D.
692 thesis), TU Delft, 1998.
693 [23] EN 1995-1-1. Eurocode 5 – Design of timber structures. CEN, Brussels, Belgium;
694 2004.
695 [24] Ascione F. A preliminary numerical and experimental investigation on the shear
696 stress distribution on multi-row bolted FRP joints. Mech Res Commun
697 2010;37(2):164-8.
698 [25] Starikov R, Schon J. Quasi-static behaviour of composite joints with countersunk
699 composite and metal fasteners. Compos Part B Eng 2001;32(5):401-9.
700 [26] Starikov R, Schon J. Quasi-static behaviour of composite joints with
701 protruding-head bolts. Compos Struct 2001;51(4):411-25.
702 [27] Feo L, Marra G, Mosallam AS. Stress analysis of multi-bolted joints for FRP
703 pultruded composite structures. Compos Struct 2012;94(12):3769-80.
704 [28] Liu F, Zhang J, Zhao L, Xin A, Zhou L. An analytical joint stiffness model for
705 load transfer analysis in highly torqued multi-bolt composite joints with
706 clearances. Compos Struct 2015;131:625-36.
707 [29] Gray PJ, McCarthy CT. A global bolted joint model for finite element analysis of
708 load distributions in multi-bolt composite joints. Compos Part B Eng
709 2010;41:317-25.
710 [30] Ekh J, Schön J. Load transfer in multirow, single shear, composite-to-aluminium
711 lap joints. Compos Sci Technol 2005;66: 875-85.
712 [31] Taheri-Behrooz F, Kashani ARS, Hefzabad RN. Effects of material nonlinearity
713 on load distribution in multi-bolt composite joints. Compos Struct 2015;125:
714 195-201.
715 [32] Lawlor VP, McCarthy MA, Stanley WF. An experimental study of bolt–hole
716 clearance effects in double-lap, multi-bolt composite joints. Compos Struct
717 2005;71:176-90.
718 [33] Persson E, Eriksson I. Fatigue of multiple-row bolted joints in carbon/epoxy

719 laminates: Ranking of factors affecting strength and fatigue life. Int J Fatigue
720 1999;21(4):337-53.
721 [34] Cramer CO. Load distribution in multiple-bolt tension joints. J Struct Eng
722 1968;94(5):1101–17.
723 [35] Xu BH, Taazount M, Bouchair A, Racher P. Numerical 3D finite element
724 modelling and experimental tests for dowel-type timber joints. Constr Build
725 Mater 2009;23(9):3043-52.
726 [36] Jockwer R, Fink G, Kohler J. Assessment of the failure behaviour and reliability
727 of timber connections with multiple dowel-type fasteners. Eng Struct
728 2018;172:76-84.
729 [37] Yurrita M, Cabrero JM, Quenneville P. Brittle failure in the parallel-to-grain
730 direction of multiple shear softwood timber connections with slotted-in steel
731 plates and dowel-type fasteners. Constr Build Mater 2019;216:296-313.
732 [38] ASTM D 3039. Standard test method for tensile properties of polymer matrix
733 composite materials. American Society for Testing and Materials; 2017.
734 [39] ASTM D 3518. Standard Test Method for In-Plane Shear Response of Polymer
735 Matrix Composite Materials by Tensile Test of a ±45° Laminate. American
736 Society for Testing and Materials; 2018.
737 [40] ASTM D 3171. Standard Test Methods for Constituent Content of Composite
738 Materials. American Society for Testing and Materials; 2015.
739 [41] ASTM D 4442. Standard Test Methods for Direct Moisture Content Measurement
740 of Wood and Wood-Based Materials. American Society for Testing and Materials;
741 2016.
742 [42] ASTM D 2395. Standard Test Methods for Density and Specific Gravity
743 (Relative Density) of Wood and Wood-Based Materials. American Society for
744 Testing and Materials; 2017.
745 [43] ASTM A 307. Standard Specification for Carbon Steel Bolts, Studs, and
746 Threaded Rod 60 000 PSI Tensile Strength. American Society for Testing and
747 Materials; 2014.
748 [44] Dittenber DB, Rao HVSG. Fatigue equation considerations for the American
749 Society of Civil Engineers pre-standard for load and resistance factor design of
750 pultruded fiber-reinforced polymer structures. J Compos Mater
751 2013;47(16):1943-9.
752 [45] Wu C, Bai Y, Mottram TJ. Effect of elevated temperatures on the mechanical
753 performance of pultruded FRP joints with a single ordinary or blind bolt. J
754 Compos Constr 2016;20(2):04015045.
755 [46] Wu C, Feng P, Bai Y. Comparative Study on Static and Fatigue Performances of
756 Pultruded GFRP Joints Using Ordinary and Blind Bolts. J Compos Constr
757 2014;19:04014065.
758 [47] Cheng X, Wang S, Zhang J, Huang W, Cheng Y, Zhang J. Effect of damage on
759 failure mode of multi-bolt composite joints using failure envelope method.
760 Compos Struct 2017;160:8-15.

[48] Ozen M, Sayman O. Failure loads of mechanical fastened pinned and bolted composite joints with two serial holes. Compos Part B Eng 2010;42.