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Title: Using self-tapping screw to reinforce dowel-type connection in a timber portal frame

Author names and affiliations:
Cong Zhang[a], Kiho Jung[b], Haibo Guo[c], Richard Harris[d], Wen-Shao Chang[e]

[a]Department of Architecture and Civil Engineering, University of Bath, Bath, UK
c.zhang@bath.ac.uk
[b]Department of Teacher Training, University of Shizouka, Shizouka, Japan
ekjung@ipc.shizuoka.ac.jp
[c]School of Architecture, Harbin Institute of Technology, Harbin, China
guohb@hit.edu.cn
[d]Time For Timber Ltd, Bath, UK
r.harris@timefortimber.co.uk
[e]Sheffield School of Architecture, University of Sheffield, Sheffield, UK
w.chang@sheffield.ac.uk

Corresponding author:

Wen-Shao Chang
Sheffield School of Architecture, University of Sheffield, Sheffield, UK
w.chang@sheffield.ac.uk

Abstract

In this study, partially threaded self-tapping screws have been used as reinforcement on timber portal frames to enhance mechanical performance of dowel-type connections. Experimental tests on unreinforced and reinforced portal frames showed that reinforced frames achieved a 31% and 51% increase in moment-resisting capacity and ultimate rotation, respectively. The test on the reinforced frames was stopped when the stroke on the hydraulic jacks had been reached, while 20% of load drop was not observed. The test results demonstrated the performance of partially threaded self-tapping screws which reduces the drive-in torque when compared to fully threaded self-tapping screws. A theoretical prediction on the characteristic moment-resisting capacity of screw reinforced portal frames is proposed.

Highlights

- Self-tapping screws improved the mechanical performance of timber portal frame
- Partially threaded screws showed a trend to effectively control crack propagation
- A theoretical prediction method is demonstrated

Keywords

Self-tapping screws, reinforcement, timber dowel-type connection, moment-resisting, theoretical prediction

1. Introduction

Timber as construction material has the advantage over concrete and steel of having a low self-weight. However, due to its low capacity of strength perpendicular to the grain, the application of tall timber structures has been limited [1]. For instance, timber dowel-type connections are commonly used in design, but their moment-resisting capacity is much lower than that of a timber member, making it the most vulnerable link in a timber structure [2].

A moment connection with sufficient capacity is vital in a portal frame structure. Therefore, in past decades, efforts have been made to strengthen the capacity of dowel-type timber connections with various types of reinforcement.

With the development of fibre reinforced polymer (FRP), considerable research has been conducted into using FRP as reinforcement for timber connections. Studies by [3, 4] reported that FRPs improved the load carrying capacity and prevented splitting failure of the connection. Haller and Wehsener [5] used FRP reinforcement combined with densified timber to improve the load carrying capacity of dowel-type connections by two times that of the unreinforced connection. In the tests of [6], timber frames reinforced by FRP and densified wood showed less reduction in structural stiffness than unreinforced timber frames. Recent works used FRP to repair damaged timber beams and successfully restored their mechanical properties [7]. Other reinforcing techniques, as summarised by Blaß and Schädle [8], used glued-on wood-based panels and truss plates. However, the above methods using different materials often require complex preparation and sufficient accessible space to conduct the work. In addition, some of them may not be feasible for repairing historic buildings as they are limited by accessibility and aesthetic requirements.

Under the construction stage of timber structures, it is always more convenient to assemble a dowel-type connection with slightly oversized holes. However, due to the gap between the drilled hole and the dowel, unexpected deformation of the structure is likely to occur; thus, [9-11] proposed the use of expanded tube fasteners combined with densified veneer wood (DVW) reinforcement in moment-resisting connections. The expanded steel tube helps to make a tight fit for the fastener so as to avoid slack load take-up, as well as enhancing stiffness [2]. The DVW reinforcement controls splitting parallel to the grain but also enhances the embedment strength of the connection. Test results from [11] show that such reinforcement significantly improved the moment-resisting capacity and stiffness of connection compared to unreinforced ones. Other researchers examined the seismic performance of this connection and results show it has very high capacity to dissipate energy [12]. However, this design can significantly increase the total thickness of the connection to over 500mm if glued laminated timber is used. Therefore, [2, 13] proposed to use thin steel plate as the middle member in order to reduce the total thickness. The loss of rotational stiffness, by replacing the middle timber member with steel flitch plate, can be compensated by decreasing the gap between the two timber side members so as to create a rotational suppressing effect. However, the procedure to fabricate such connections involves using a hydraulic jack to compress the tube to fit and attaching the DVW reinforcement is time-consuming and complex.

In the last two decades, studies by [8, 14, 15] demonstrated that self-tapping screws can effectively reduce the splitting tendency of the connections. Works by [16-18] investigated the effectiveness of using self-tapping screws as reinforcement on bolted timber connections under dynamic load. found that self-tapping screws as reinforcement could increase the moment-resisting capacity by 170% under reverse cyclic loading. Another study compared the reinforcement effectiveness of plain round rods and self-tapping screws on post-to-beam connections [19]. Their work found that screw reinforcement outperformed the plain round rods in maximum moment enhancement, ductility and energy dissipation. In [20, 21], research work showed the effectiveness of screw reinforcement and suggested the use of self-tapping screws with threads on the point end to reduce the drive-in torque of the screw in order to have a lower friction force generated during the installation process.

Currently, there is no experimental testing on timber portal frames using dowel-type connections reinforced by self-tapping screws. Therefore, the aim of this paper is to compare the mechanical performance of unreinforced and self-tapping screw reinforced portal frames. As in large timber structures, long screws are required and higher friction forces are inevitable when the screw is fully threaded. Therefore, the portal frames in this study used screws partially threaded at the point end.

2. Material and methods

2.1. Material preparation

Two timber portal frames were fabricated for the test, each frame consisting of three glulam beams made from European Whitewood and classified to GL24c. The measured average volume density was 456 kg/m^3 (CoV=1.5%) and the average moisture content was 10.2% (CoV= 17.9%). The self-tapping screw had a flange head and its drawing and specifications, according to [22], are shown in **Figure 1**.

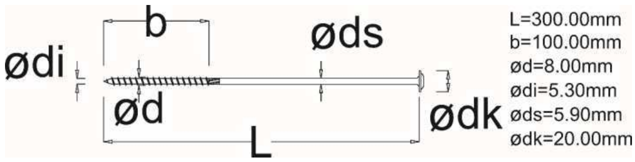

Figure 1: *The partially threaded self-tapping screw used in this study.*

The configuration of timber-steel-timber connections in the portal frame was designed according to Eurocode 5 (EC5 hereafter) [23] and the details are shown in **Figure 2**. A 3×3 fastener group consisting of 12mm dowels was adopted for the connection, the geometry of the fastener groups in the columns and beams being identical. An 8.5mm wide slot was used to accommodate the 8mm steel plate as the central member. The steel dowels and steel plates were made from bright mild steel classified to 080A15T. To ensure the 300mm self-tapping screws could be accurately installed, a pre-drilled hole with 5mm diameter and 300mm depth was prepared using a pillar drill. The self-tapping screw was placed at *1d* distance (12mm) from the dowel, so that the screw was not in contact with the dowel.

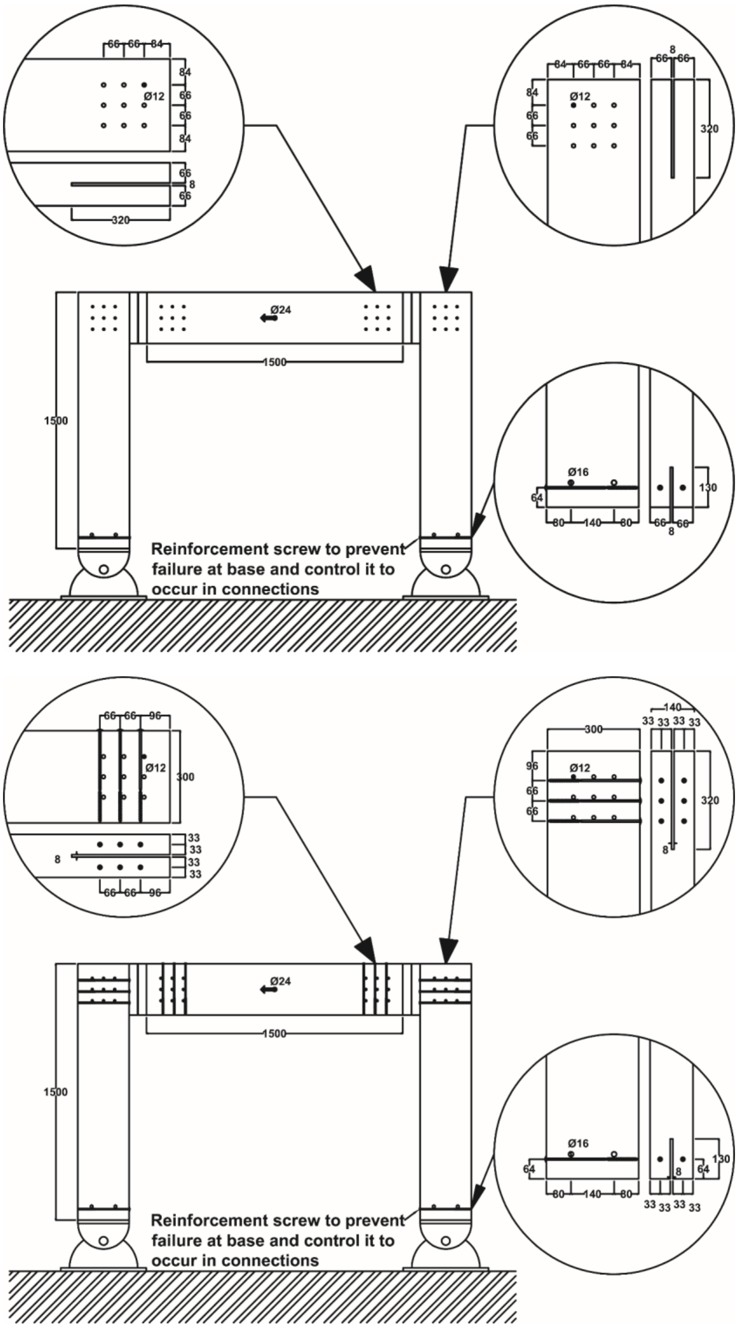

Figure 2: *Portal frame configurations: unreinforced portal frame and member configuration (top); reinforced portal frame and member configuration (bottom).*

As the portal frame was to be subjected to a horizontal force, the base of the two columns was designed to sustain an opposing force to reach equilibrium. Therefore, a shear force acting parallel to the cross section of the beam could lead to shear splitting failure of the column. To avoid failure at the base of the frame, both the unreinforced and reinforced frames were reinforced by self-tapping screws at the base, as shown in **Figure 2**, while calculation has shown sufficient shear resistance by the two 16mm dowels at base. **Table 1** gives a summary of the properties of the two tested portal frames.

Table 1: *Summary of each testing groups.*

Group	Description	Mean density(kg/m^3) (CoV)	Mean M.C.% (CoV)
UPF	*Unreinforced*	*458 (0.5%)*	*10.5 (23.5%)*
RPF	*Reinforced*	*458 (0.5%)*	*9.6 (18.2%)*

2.2. Portal frame test set-up

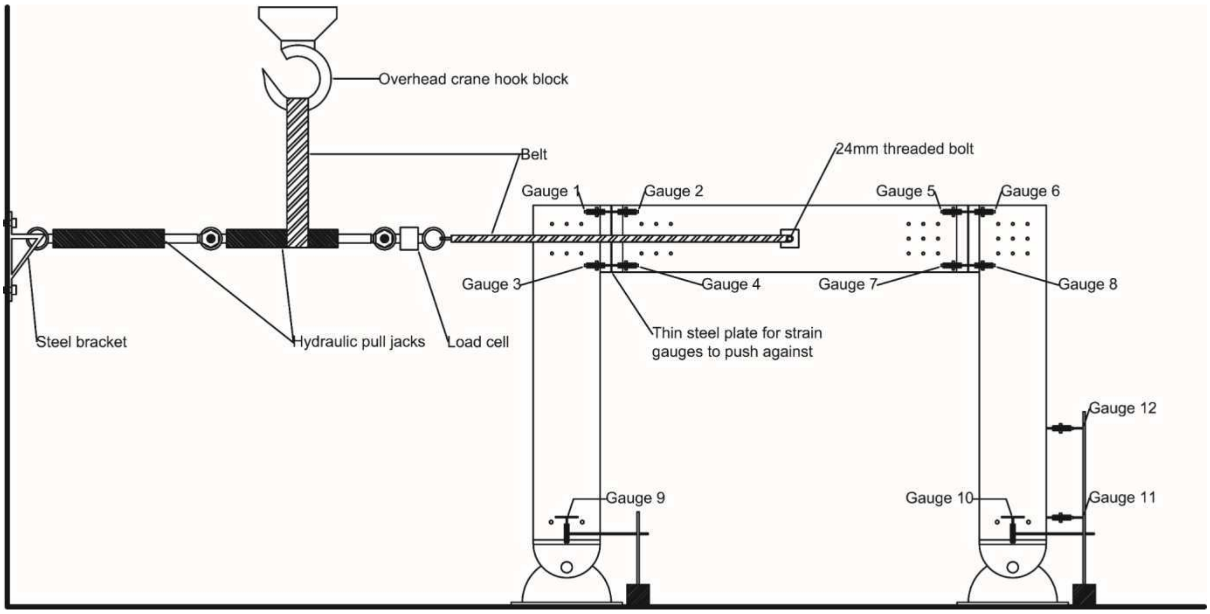

Figure 3: *Portal frame test layout.*

A general overview of the test layout is shown in **Figure 3**. The portal frames have pinned supports bolted to the strong floor and the frame is loaded horizontally by two hydraulic push jacks. The hydraulic jacks are placed in series with one end connected to a steel triangle bracket fixed to the wall and the other end connected to a load cell. One of the jacks is linked to the hook block of an overhead crane by a belt to ensure the hydraulic jacks are held in position vertically. Another belt is used to transfer the load from the hydraulic jack to the 24mm bolt installed in the frame, as demonstrated in **Figure 3**. In this static loading test, the portal frame is pushed to failure or the load is stopped when the stroke reaches the 300mm limit (with 150mm strokes for each jack). The loading of the test followed BS EN 26891:1991 [24] which describes how a pre-load should perform from 10% to 40% of the estimated load carrying capacity before the frame is ramp loaded to failure.

3. Results and discussion

3.1. Unreinforced specimen

In this study, the unreinforced portal frame was unloaded when the capacity of the frame was 20% lower than the peak load or when the stroke of the hydraulic jack was reached. During the loading stage, splitting of the timber in the two columns occurred with a large wood cracking noise. A total of 8 major cracks were found in the two columns. After failure, it was observed that cracks were located at the top and bottom rows parallel to the grain of the columns, as shown in **Figure 4**. Some of the wide and deep cracks propagated to the mid-span of the timber member. The columns rotated around the pinned supports to allow deformation of the frame, and the fasteners in the columns rotated around the centre dowel to take the moment generated during the movement of the frame. For the dowels at the corner of the square-shape fastener group in the column, the four dowels sustain the highest moment as they are located furthest from the centre of the rotation. The load on them is at 45° to the grain direction and the component force perpendicular to the grain is the cause of the splitting in the wood parallel to the grain. Cracks appeared on both sides of the columns in the unreinforced portal frame. The initiation of the crack on the right-hand side of the column appeared along the first column of dowels with an initial length of 202mm at 26kNm. At the point of failure, this crack propagated to about 551mm. A second crack appeared on the right column of dowels in the fastener group around the failure point and suddenly propagated to 407mm. The beam member, however, did not rotate substantially relative to the steel plate as the columns did (see **Figure 8**) and no cracks could be observed at the point of failure of the unreinforced frame.

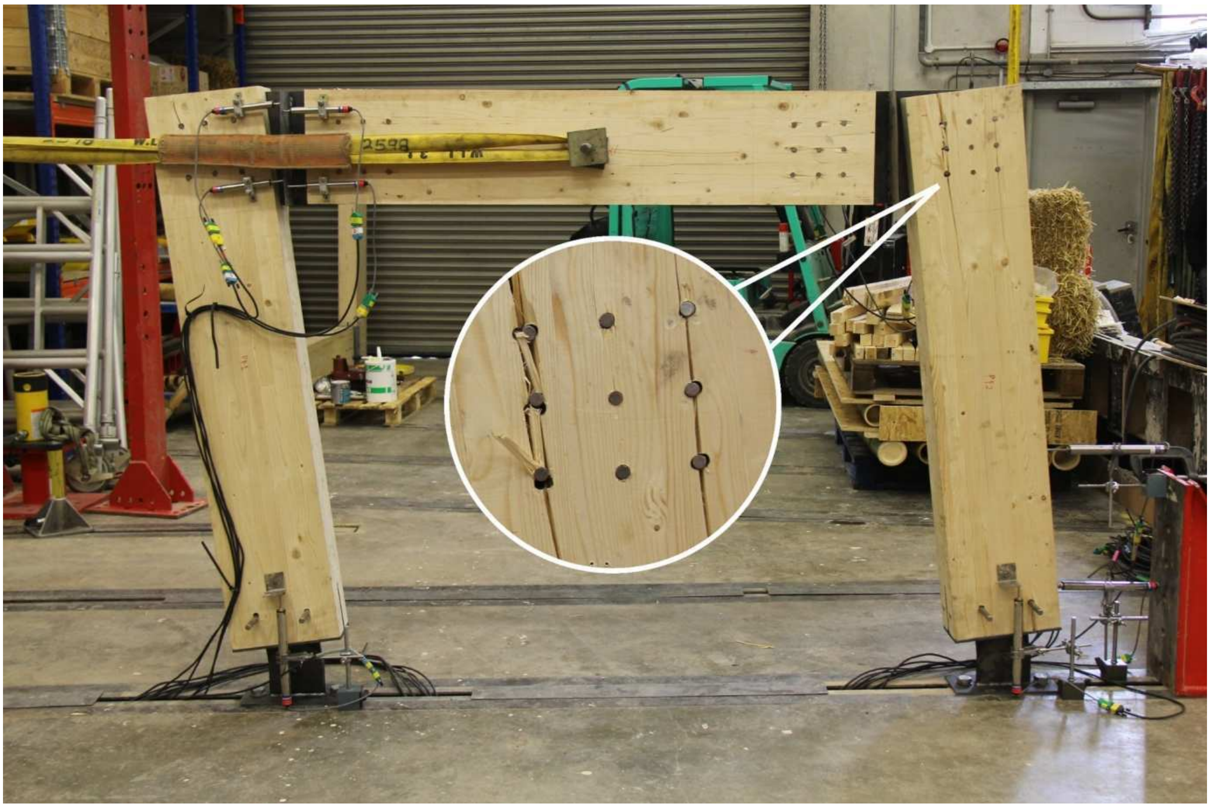

Figure 4: *Unreinforced portal frame during testing, significant cracks can be observed on columns.*

3.2. Reinforced specimen

For the reinforced portal frame, loading was stopped when the stroke had been reached. The reinforced frame did show load reduction when the timber split, but the load drop had not reached 20% of the peak load. There were 4 major cracks at the lower row of fasteners parallel to the grain of the side columns. The cracks were much shorter than those in the unreinforced frame and did not pass through all three dowels in a row, as shown in **Figure 5**. The two side columns rotated at the pinned supports and at a higher angle of rotation than the unreinforced. The central beam again had much smaller rotation relative to the steel plates and no cracks were found on the beam. In the last few minutes of the test, the 100mm gauge (No.12) exceeded the stroke capacity and the final width of the gap between the tip of the strain gauge and the surface of the column was measured to be 30mm. For the reinforced portal frame, the significant load perpendicular to the grain of the column was intended to split the wood, but the crack propagation was restricted by the screw reinforcement, as expected. The crack initiated at 33kNm with a length of 129mm. At the end of the loading, the crack length remained to be the same. A second crack initiated after 35kNm with an initial length of 108mm and propagated to 126mm at the end of the loading. The screw reinforcement effectively used the restrains provided by the embedment of the screw head and the friction between the threads and the wood. **Figure 6** shows the embedment of the screw head in the reinforced portal frame.

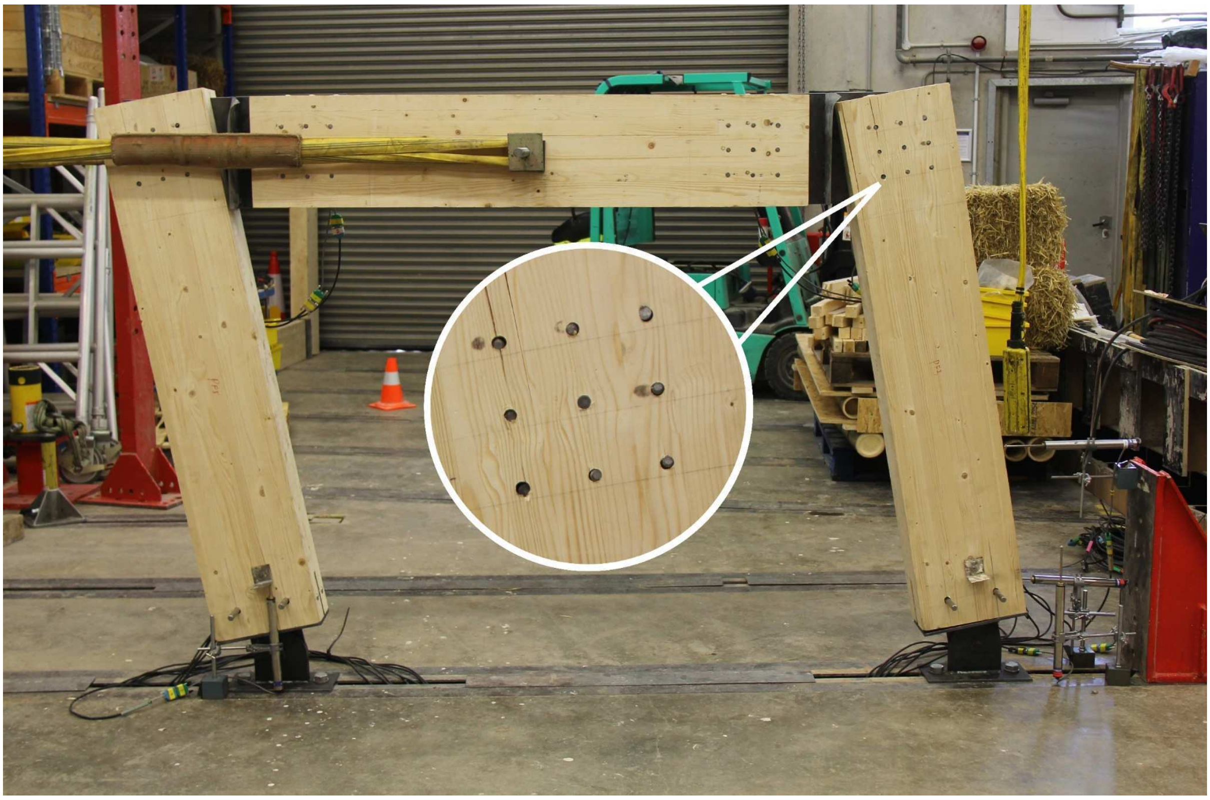

Figure 5: *Reinforced portal frame during testing, two short cracks located on the right-hand side column is zoomed.*

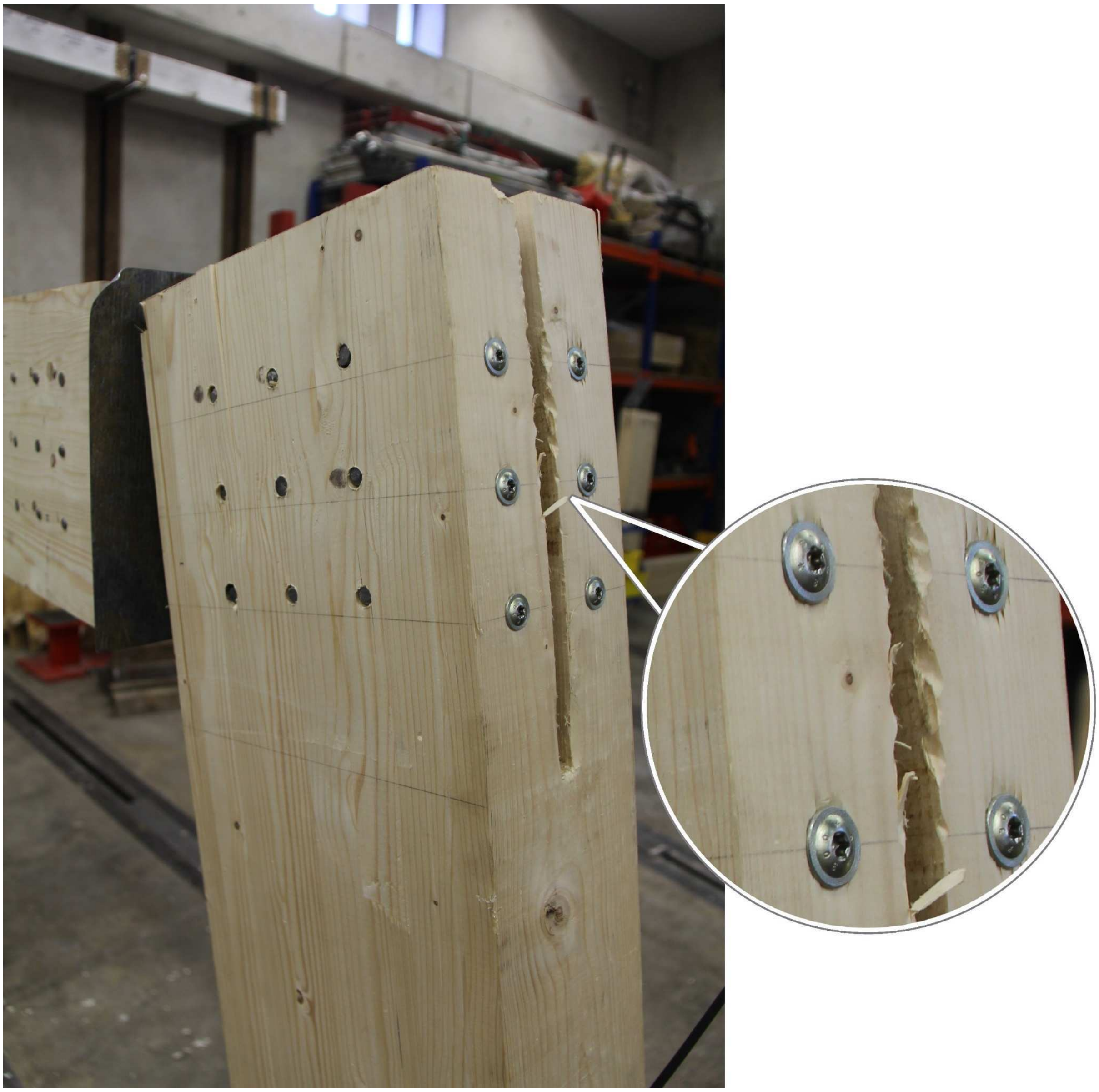

Figure 6: *Screw head embedment in the reinforced portal frame.*

3.3. Comparison between unreinforced and reinforced portal frames

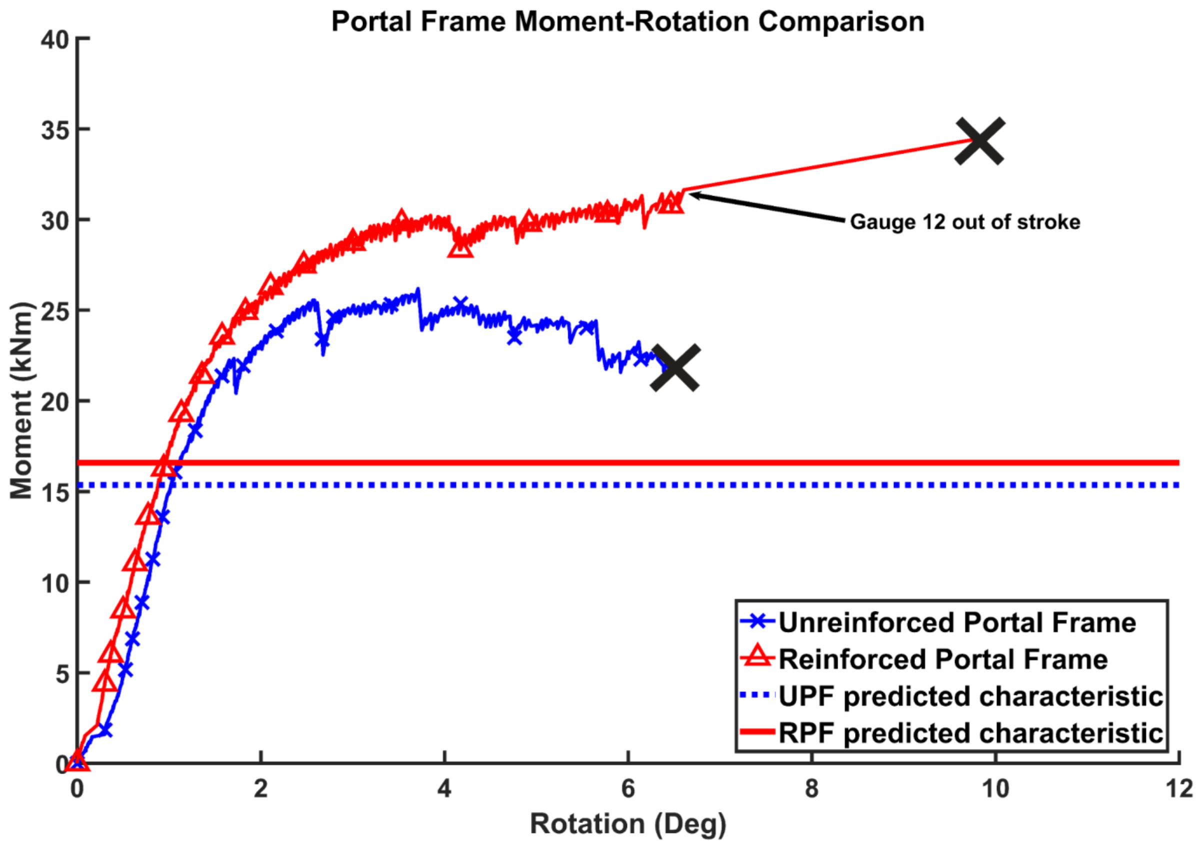

Figure 7: *Moment-rotation curves for the two tested frames. For the UPS, the black X mark indicates the 20% load drop from the peak load. For the RPF, the marker indicates the end of stroke of the hydraulic jack. The pre-loading stage is excluded in the graph.*

The moment-rotation curves for both frames are plotted in **Figure 7**. For the unreinforced frame, the moment-resisting capacity drops as the crack developed in the connections. The unreinforced frame reached its peak moment at 26.19kNm and failed soon after this value. The ultimate rotation for the unreinforced frame is 6°. As for the reinforced frame, the moment first peaked at 30kNm with a rotation of 4°. The capacity then slightly dropped with crack propagation; however, as the steel dowels were bent and started to bear on the screw reinforcement, the connection regained its moment-resisting capacity. This was calculated to happen at around 5.2° for the dowel to touch the screw. The capacity then increased to about 32kNm before the stroke of Gauge 12 (100mm strain gauge) was reached. In **Figure 7**, the straight line for the reinforced portal frame represents this stage as no data points were available. The measured 30mm increase in stroke (as found in the previous section) was added to the final point that Gauge 12 output, where loading stopped at 32kNm. At the final point, before the load was removed, the reading of Gauge 11 and the accumulated reading of Gauge 12 are used to calculate the final rotation of the frame, which is found to be 9.8°. The final point, also the measured peak moment-resisting capacity of the reinforced frame is found to be 34.47kNm and is marked with ***X*** in the graph.

All the interval points within this period are excluded from the graph and a straight line between the final point and the last validate point from the gauge is drawn in the graph. The reinforced portal frame demonstrated high moment-resisting capacity and ultimate rotation compared to the unreinforced one. A higher ultimate rotation in a dowel-type connection is an indicator for a higher ductility.

Table 2: *Summary of calculated mechanical properties for the two frames.*

Group	Maximum moment-resisting capacity (kNm)	Ultimate rotation (°)	Stiffness
UPF	*26.19*	*6.50*	*9.03*
RPF	*34.47 **	*9.80 **	*10.00*

** This is not the maximum value of the reinforced portal frame, but the final reading when the stroke on the hydraulic jack was reached.*

The calculated mechanical properties of the two frames are listed in **Table 2**. As can be seen, the screw reinforcement effectively improves the moment-resisting capacity and ultimate rotation of the frame with dowel-type timber connections. However, the stiffness does not show significant enhancement and correlates well with previous results on screw reinforced moment-resisting connections.

Figure 8 displays the beam to steel plate rotation and column to steel plate rotation for both frames. It shows that the beam did not rotate significantly around the plate, while the column in the reinforced frame shows a larger angle of rotation around the plate than that of the unreinforced frame.

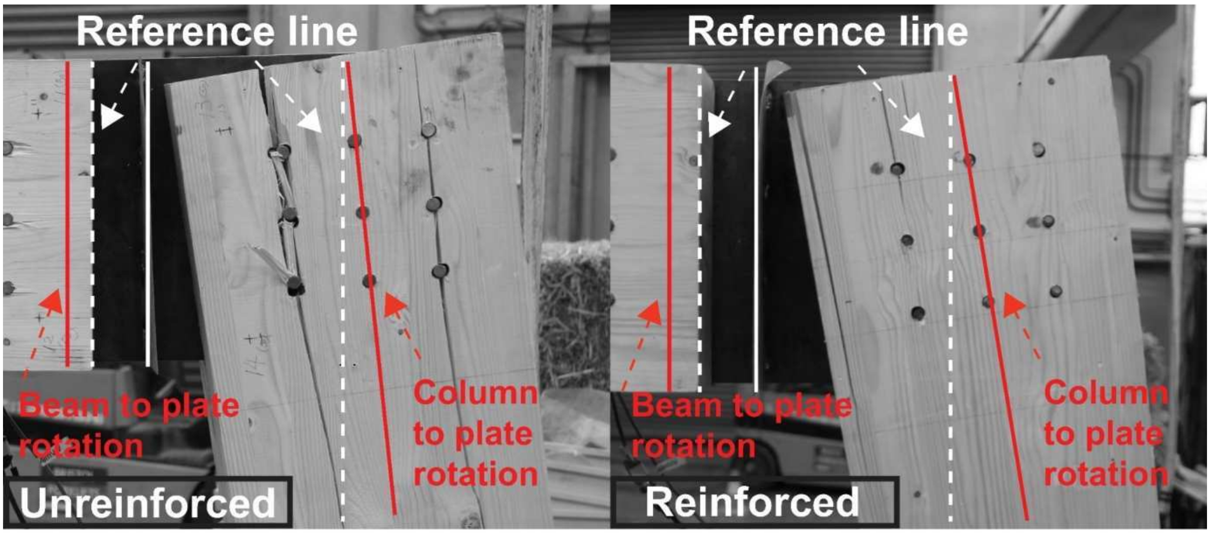

Figure 8: *Pictures showing the beam/column to plate rotation for unreinforced portal frame (left) and reinforced portal frame (right).*

3.4. Comparison with theoretical strength

Current EC5 does not provide any calculations for screw reinforced timber structures. In this study, the connections in the portal frame is assumed to be rotationally rigid where the centre of rotation is the centroid of the fastener group and remains fixed.

The calculation method is based on the model presented in Blaß [25] and Porteous and Kermani [26] and, for a three by three moment connection, is expressed as:

$$M_k = FL = [(F_A + F_C + F_E + F_G) \cdot r_{max} + (F_B + F_D + F_F + F_H) \ \cdot r_i] \cdot n_{sp} \quad (1)$$

where, M_k is the design moment-resisting capacity of the connection, F is the load acting perpendicular to the grain, F_x represents the load acting on the dowel due to the moment, see **Figure 9**, r_{max} is the maximum distance between the dowel and the centre of rotation, r_i is the distance between the dowel and the centre of rotation and n_{sp} is the number of shear planes.

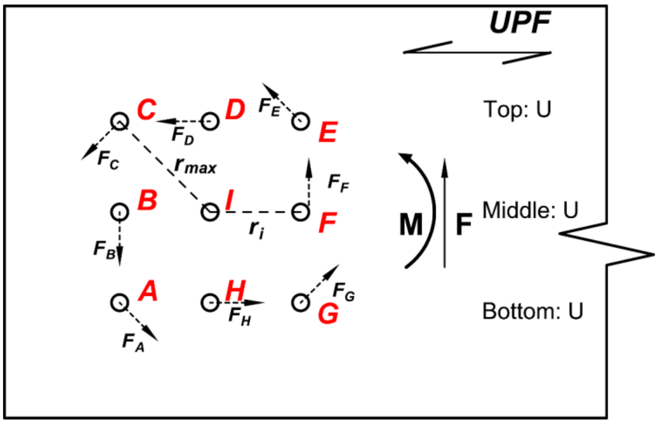

Figure 9: *The drawing indicates the unreinforced column connections on the right-hand side only and for convenience, they have been rotated 90˚ in the anti-clockwise direction. The black arrows represent the load on the dowel due to the moment.*

The calculation method assumes the connection is rigid and the dowels have same slip modulus and rotation angle. Therefore, for a three by three connection, the dowels have same perpendicular distance to the centre of rotation are subject to the same amount of load due to pure moment:

$$F_A = F_C = F_E = F_G = K \cdot r_{max} \cdot \theta \quad (2)$$

$$F_B = F_D = F_F = F_H = K \cdot r_i \cdot \theta \quad (3)$$

Thus, the equation for the characteristic moment-resisting capacity of the connection can be re-written based on the load on one dowel, taken dowel E for example:

$$M_k = FL = (4F_E \cdot r_{max} + 4\frac{F_E}{r_{max}} \cdot r_i^2) \cdot n_{sp} \quad (4)$$

The method considers the influence of a load on the connection, when also subject to a moment at the centre of rotation. The load, either in the vertical or the horizontal direction, can change the angle of the total load on the dowel, see **Figure 10**. F_{TX} represents the total load on dowel X due to the moment and the horizontal load F_h. The angle of the total load can be found by using the resolution of F_{TX}, F_h and F_x. The magnitude of the total load should not be greater than the load-carrying capacity derived using the equations in EC5.

As the embedment strength varies with the angle of load to the grain direction, the load-carrying capacity derived from each dowel will be different. Therefore, changing the magnitude of the total load on the dowel will lead to a different value of F_x, the load acting on the dowel due to the moment. Since the characteristic moment-resisting capacity of the connection can be represented by the load on a certain dowel due to the moment, such as Equation (4), the moment-resisting capacity of a connection varies with the dowel that is under consideration. Consequently, the dowels shall fail in an order.

For the reinforced connection, the order of failure is influenced by the embedment strength of the wood around the dowels. The later part of this sections explains why there are unreinforced wood in a reinforced connection and it was found that dowels bearing on unreinforced wood tend to fail first as they are calculated to have lower load-carrying capacity. Thus, the moment-resisting capacity of the reinforced connection would be underestimated if it is calculated using the load-carrying capacity of the first yielded dowel.

Therefore, the calculation method first involves finding the order of failure, then, calculate the moment-resisting capacity of the connection by considering the failure of dowels and requires considering three to four dowels to fail for an accurate prediction. The purpose for such requirement is because the dowels bearing on unreinforced wood tend to fail first and considering the failure of three to four dowels would involve at least one case of the failure of dowel bearing on reinforced wood to occur. Furthermore, the calculation methods also assume the failed dowels would maintain their peak moment resistance. A detailed calculation procedure of reinforced connection is provided in [27].

As the connections in the beam member did not have significant rotation, the theoretical prediction considers the fastener groups at the columns which subject to a horizontal load and moment.

To predict the moment-resisting capacity of the unreinforced and reinforced portal frames, characteristic embedment strength from single-dowel embedment tests are applied. The characteristic embedment values are acquired from the original data using the five-percentile method described in BS EN 14358:2016 [28]. As the characteristic values are only available for loading parallel to the grain, $f_{h,0,k}$, the characteristic embedment strength, $f_{h,1,k}$, in various loading directions (denoted as α_{TX} in **Figure 10** where X represents the fastener), can be calculated using the Hankinson formula illustrated in EC5 Clause 8.5.1.1 [23]. **Table 3** lists the characteristic values obtained from the embedment test.

Table 3: *Characteristic values calculated from embedment test.*

Group	Description	Repetition	Characteristic embedment strength (N/mm^2)
U	*Unreinforced*	*10*	*20.07*
RBS	*Reinforced by screw with one third of threads on the point end*	*10*	*24.80*

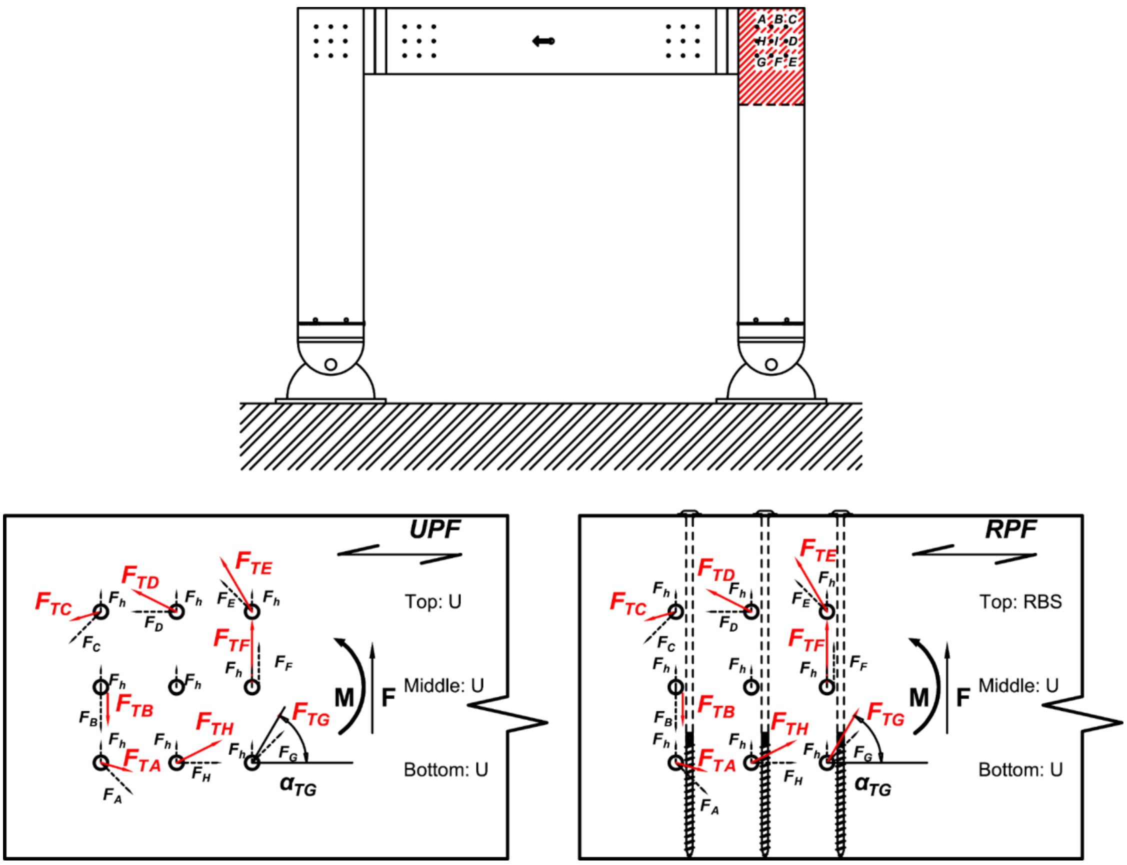

Figure 10: *The drawings indicate the column connections on the right-hand side (refer to* ***Figure 4*** *and* ***Figure 5****) only and for convenience, they have been rotated 90˚ in the anti-clockwise direction. Reinforcement scenario assigned for the two frames: unreinforced portal frame (left) and screw reinforced portal frame (right). Red arrows indicate the total loads on the dowels from the timber members due to the rotation and horizontal loading.*

For the unreinforced connection, the characteristic embedment strength for each dowel is calculated based on the unreinforced embedment test (group *U*) as shown in **Figure 10**.

In the reinforced connection, only the top row of the fasteners*, C, D and E,* have the movement bearing on the screw reinforcement. In other words, the embedment strength for these locations is enhanced. Therefore, the embedment strength of group *RBS* was applied. As for the middle and bottom dowels, their rotation directions determine that they will not bear on the self-tapping screws. Therefore, the wood at these locations are defined as 'unreinforced' and applied with unreinforced values. The load-carrying capacity of each dowel is acquired by using the equations *(f)*, *(g)* and *(h)* from clause 8.2.3 in EC5 [23].

The characteristic moment-resisting capacity of the portal frame is the sum of the capacity of the two column connections. The predicted maximum values for the unreinforced and reinforced portal frames were 15.38 and 16.60kNm, respectively; they are shown by the dotted straight lines in **Figure 7**.

In addition, the maximum moment-resisting capacity of the reinforced portal frame is about 8% higher than the unreinforced frame for the theoretically predicted values.

The 8% increase of theoretical moment-resisting capacity of the frame is a result of the higher embedment strength of the wood around the dowels *C*, *D* and *E*. As shown in **Figure 10**, only these dowels are considered to be effectively reinforced by the self-tapping screws. To further enhance the capacity of the frame, one possible method is to enhance the rest part of the connection, which is shown in **Figure 11**.

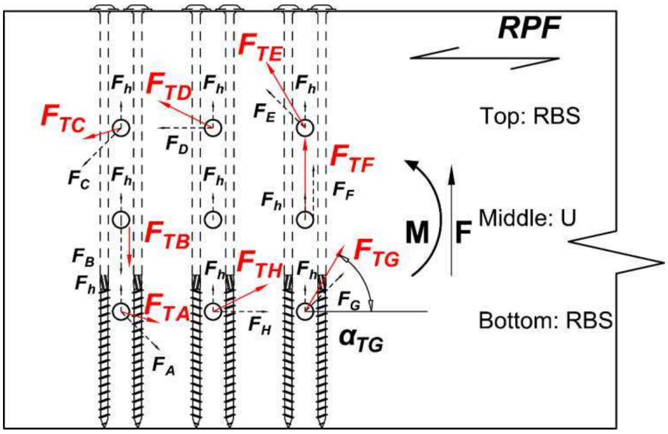

Figure 11: *Proposed reinforcement approach to further enhance the moment-resisting capacity of portal frame. Red arrows indicate the total loads on the dowels from the timber members due to the rotation and horizontal loading.*

In **Figure 11**, another 3 additional screws are placed at *1d* distance to the dowels, opposite to the existing screws. These screws are applied to enhance the embedment strength of wood around dowels *A*, *G* and *H*. For steel dowels *B* and *F*, their movements are perpendicular to the grain. As the embedment strength of the wood is the lowest in the perpendicular to the grain direction, using self-tapping screws to enhance their strength is less effective than enhancing those with higher strength parallel and 45° to the grain direction. The theoretical moment-resisting capacity of this kind of reinforced frame is 17.47kNm, 14% higher than the unreinforced frame.

The results of this study are limited by the number of frames. It is necessary that in the future, to largely increase the sample size and use the 5-percentile method in BS EN 14358:2016 [28] to calculate the characteristic moment-resisting capacity of the test results. The characteristic values can then be used to demonstrate whether the characteristic values, obtained by the proposed calculation method in this study, are appropriate and conservative.

4. Conclusion

This study compares the mechanical performance of dowel-type moment-resisting timber frames that are unreinforced and reinforced by self-tapping screws. The sample size of the experiment in this study is small and a large number of tests are required in the future for confirmation. The following points were concluded based on the results of this study:

- Self-tapping screws with one third of thread on the point end placed at one fastener spacing to the dowel showed a tendency to effectively enhance the moment-resisting capacity and ultimate rotation of timber portal frames.
- Screw reinforcement has demonstrated an effective behaviour in controlling crack initiation and propagation. The restraining force is provided through screw head embedment and thread-wood bonding.
- The study does not find a tendency for screw reinforcement to improve the structural stiffness. This result corresponds well with previous findings on moment-resisting connection tests.
- A simple calculation method for predicting the moment-resisting capacity of screw reinforced portal frames is proposed. The method uses results from embedment tests to predict the load-carrying capacity of each dowel which is assigned different reinforcement scenarios. The summation of the moment resistance of the fastener group represents a structure's characteristic moment-resisting capacity.

- The predicted values are smaller than experimental results, but a larger number of tests are required to validate whether the method is conservative. With further confirmation, the proposed method may be used to predict the moment-resisting capacity of certain types of screw reinforced dowel-type timber structures if the corresponding embedment data is available.

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