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PORTAL FRAME BRACING SYSTEMS USING FIBER REINFORCED POLYMER (FRP) IN LIGHT-WOOD FRAME BUILDINGS

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ABSTRACT: Although most wood-frame buildings are able to meet the minimum wall bracing requirements to resist seismic and wind loads, there are situations where the required length of braced walls may not be sufficient due to openings desired by architectural requirements such as large openings for multi-car garages. Hence, developing alternative bracing systems that can either replace or be used in conjunction with conventional shear wall segments without compromising the structural integrity of the building is desired. Wood portal frame systems have been identified as a viable option to meet the lateral load requirements. However, there is a need to explore the potential for improvements to the portal frame systems to make sure they can be integrated into light wood frame structures as the main lateral load resisting system in combination with conventional shear walls. Test observations have shown that the moment capacity of the corner joint between the header and narrow braced wall segment dominates the portal frame behavior. The current study focuses on optimizing the corner details using full scale joint tests. This paper reports on tests results and failure modes from corner tests on portal frame corner joints retrofitted with Fiber Reinforced Polymer (FRP). Modeling technique to develop numerical model using nonlinear finite element software SAP 2000 to validate the test results is also presented in the paper.

KEYWORDS: Portal Frame, Lateral load bracing system, Alternative bracing system, FRP, Finite element modeling

1 INTRODUCTION

Wood portal frames have been investigated with the purpose of incorporating such systems into light frame wood buildings in order to allow for more open space and larger openings. International Residential Code (IRC) in the US has allowed the use of prescriptively designed portal frames as lateral bracing systems in light frame wood building since 2006 [1]. Although research on portal frame bracing systems has been undertaken by APA- The Engineered Wood Association [2] to include the system in the IRC, very few research work [3] have been conducted to establish their performance using a mechanics-based approach. The current research work is attempting to establish and enhance the performance of portal frames so that they can be used in conjunction with or as a substitute to light frame conventional shear walls with wood sheathing panels.

Since the connection of header and narrow braced wall segment is detrimental to the lateral load resistant capacity of portal frame bracing systems, Fiber Reinforced Polymer (FRP) was used to strengthen the corner joint due to its superior tensile strength. The work reported here focuses on the investigation and comparison of the moment carrying capacity and stiffness for specimens with and without FRP application at the corner joint.

Numerical modelling with non linear finite element software SAP 2000 had been performed and output from model results compared with the full scale corner tests conducted.

2 CORNER JOINT TEST

To simulate the corner joint of a portal frame an L-shaped test assembly was developed. Although the boundary conditions of the test assembly differed from those found in the portal frame, the test was intended to quantify the potential changes in strength, stiffness and ductility due to the application of the FRP material.

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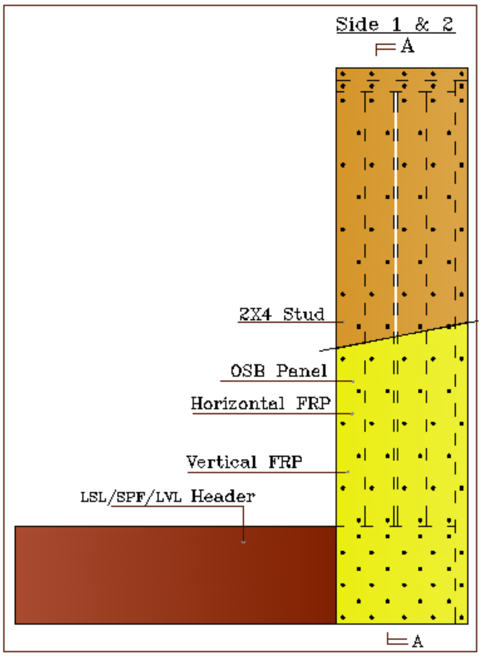
Both standard portal frame technique developed by APA [2] and midply shear wall technique developed by Forintek [4] were evaluated. In midply shear wall detailing, the studs are rotated 90-degree about the longitudinal axis as shown in Figure 1, so the number of stud become double relative to traditional light frame shear walls. The detailing allows for higher lateral load resisting capacity due to engagement of the nails in double or triple shear [4].



(a)

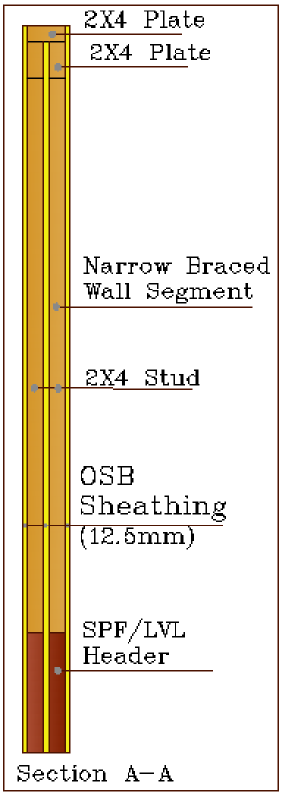


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Figure 1: *(a)* Orientation and (b) cross section of midply portal frame corner joint

2.1 TEST ASSEMBLY DETAILS

Six different configurations of portal frame had been tested as shown in Figure 4. Portal frame corner joint assembly C1, C2, FRP-1 and FRP-2 had been assembled as per standard portal frame technique and consisted of 1760 mm high and 405 mm wide wall segments connected to nominal 2x12 Laminated strand lumber (LSL) header. The narrow wall segment was constructed of 2x4 SPF No. 2 & better studs and sheathed with Oriented Strand Boards (OSB). Two headers were jointed as a built-up section to form an 89 mm (3.5”) thick header. The sheathing to lumber connection consisted of 8d common nails with a nailing schedule of 75 mm o.c. and the sheathing to the LSL header was nailed using a nail spacing grid of 75 mm.

Corner joint assembly C3 and FRP-3 had been assembled as per midply shear wall technique and consisted of 1720 mm high and 405 mm wide wall segments connected to a header consisting of a two-ply nominal 2x12 Spruce-Pine-Fir (SPF) members for test specimen C3 and nominal 2x12 Laminated veneer lumber (LVL) header for test specimen FRP-3. The 12.5 mm thick OSB, with a span rating of 2R32, was used as middle and exterior sheathing panels. 16d common nails (4.06 mm in diameter and 90 mm in length) were used to attach the sheathing to the studs. The nailing schedule used was: one row of nails spaced @100 mm on each face of stud on wall segment, and a grid pattern of nails spaced @50 mm on the header.

Six 5.12 mm x 88 mm wood screws were used to connect exterior stud with header. Simpson Strong Tie LSTA 21 (1000 lb capacity) was used to connect the header and end studs in some configurations of the corner joint. Average moisture content of the wood members in the test assemblies was 16.5% and the maximum moisture content did not exceed 19%.

The FRP membrane was applied vertically at the top of the exterior OSB. Both face of OSB sheathing was covered vertically with single sheet of FRP. Horizontal FRP membrane was applied at the end of vertical FRP to avoid delamination. Tyfo SCH-41S-1 reinforcing fabric with Tyfo S Epoxy was used. Tyfo SCH-41S-1 is a custom weave, unidirectional carbon fabric with glass crosses. Fibres are orientated in the 0^{o} direction and glass fibres are orientated at 90^{o}

2.2 TEST SETUP

The test set-up is shown in Figures 2 below. Cyclic load was applied at the top of the assembly through a 9.0 mm thick metal plate connected to an actuator. The metal plate was fastened to the top wood plates using four Ø12.7 mm lag screws. The header was secured to a heavy glulam beam using a combination of metal brackets and 12.7 mm lag bolts on both sides of the headers. The glulam beam was attached to a steel I-beam, which in turn, was anchored to the concrete foundation of the laboratory. Three displacement transducers LVDTs were used to measure the horizontal and vertical movements of the vertical section relative to the header/base.

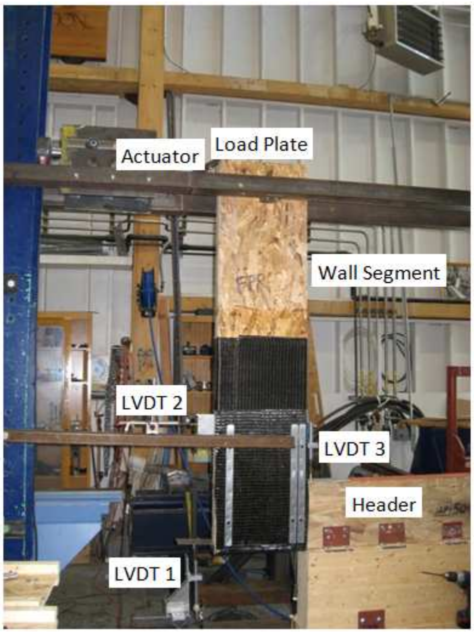


Figure 2: A picture of the test set-up of portal frame corner assembly with FRP

2.3 LOAD PROTOCOL

ASTM E2126-08 [5] CUREE protocol was used to conduct reversed cyclic loading test as shown in Figure 3. Frequency of cyclic tests was 0.25 Hz and the acquisition of data was captured at a frequency of 10 Hz. Based on the monotonic tests, the reference ultimate displacement was taken as 60 mm (2.36 inch) at the top of the wall segment which was used to develop the CUREE loading protocol.

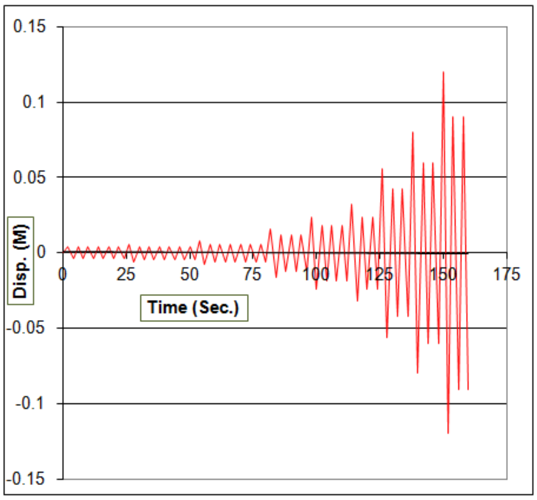
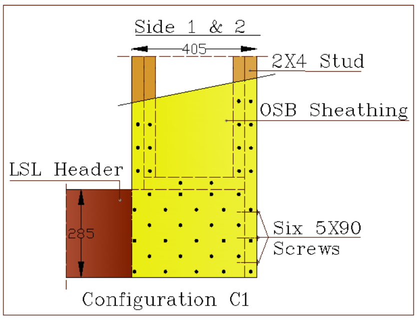
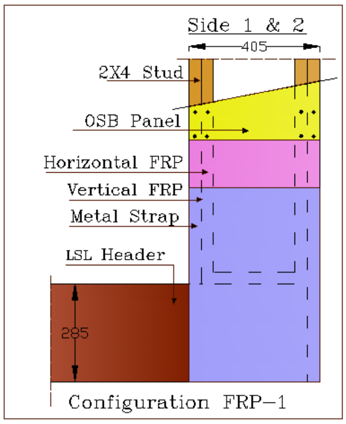


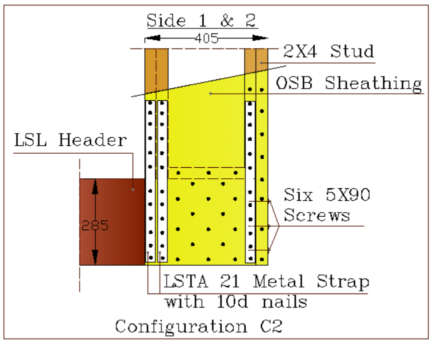
Figure 3: CUREE reversed cyclic loading protocol for the cyclic tests (ASTM E 2126-08)

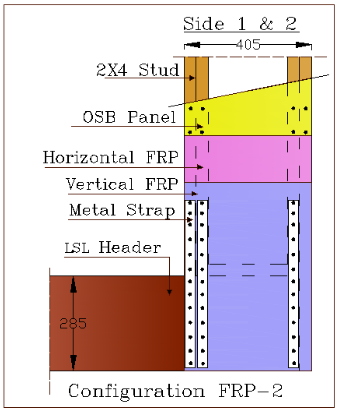
2.4 TEST MATRIX

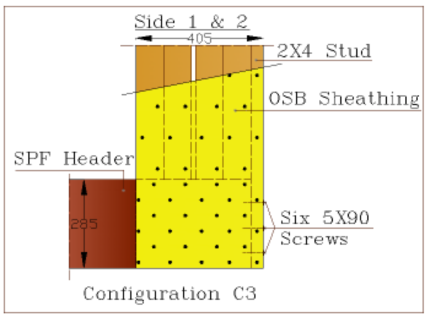
For all of the configurations with FRP single specimen was tested while all of the configurations without FRP two specimens were tested. Metal straps were nailed above the sheathing and on top of the FRP membrane. Figure 4 shows all the test configurations of corner joints.











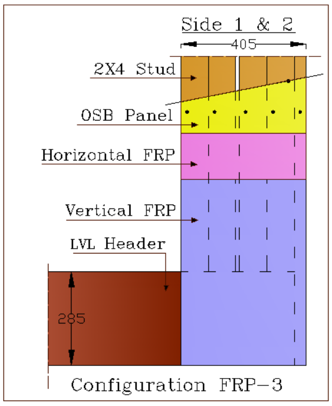


Figure 4: *Test configurations of Corner joints (all dimensions are in mm)*

3 EXPERIMENTAL TEST RESULTS

The test results were analysed according to the Equivalent Energy Elastic-Plastic (EEEP) curve in ASTM Standard E2126 [5]. Maximum lateral load capacity and corresponding displacement, initial stiffness, maximum moment and corresponding rotation as well as rotational stiffness were calculated to compare the performance of the corner joint assemblies. For moment resistance calculations, the distance between the center of the header and vertical wall segment corner to the top of bottom plate connected to the load distribution metal plate (i.e. 1.58m) was taken as the moment arm. Transducer 2 (see Figure 2) was used to measure the deflection of the vertical stud to calculate the rotation angle. A summary of the analysis results for corner joint with and without FRP with similar configurations is given in Table 1.

Table 1: *Experimental results of Portal Frame Corner joints*

			Max. Load	Max. Moment	Rotation @ Maxm. Moment	Rotational Stiffness
Specimen	Corner Joint Type	Header	KN	KN-M	Rad.	KN-M / Rad
C1	Standard	LSL	9.26	15.0	0.0471	610
FRP-1	Standard	LSL	14.26	23.0	0.0793	540
% Increase			54.0	54.0	68.4	-11.5
C2	Standard	LSL	10.96	17.7	0.0452	721
FRP-2	Standard	LSL	17.46	28.2	0.0607	715
% Increase			59.3	59.3	34.3	-0.8
C3	Midply	SPF	10.67	16.8	0.0554	599
FRP-3	Midply	LVL	17.98	28.4	0.0626	738
% Increase			68.5	68.5	13.0	23.2

3.1 MAXIMUM MOMENT RESISTANCE

The maximum moment resistance was calculated as the average of maximum positive and negative moment of each specimen. Figure 5 shows that for all the configurations FRP membrane lead to an increase of the moment resisting capacity. Compared to FRP-1, addition of 3 metal straps on each face (FRP-2) increased the moment resisting capacity of standard portal frame with FRP by 22.6%. Midply portal frame with FRP (FRP-3) has also 23.5% higher moment resisting capacity than standard portal frame with FRP (FRP-1).

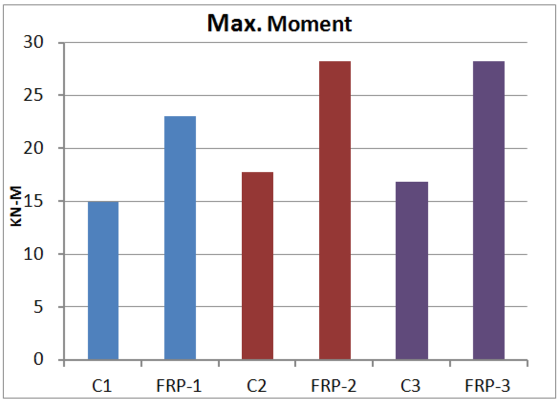


Figure 5: *Average maximum moment resisting capacity of Corner joints*

3.2 ROTATIONAL STIFFNESS

Rotational stiffness was calculated using the slope of the secant line passing between 0% and 40% of the maximum moment measured from the envelope curve. Figure 6 show that FRP membrane had no significant effect on stiffness properties. The addition of 3 metal straps on each face with FRP (FRP-2) had a slightly more profound effect and increased the rotational stiffness of standard portal frame by 32% compared to FRP-1. Midply portal frame with FRP (FRP-3) has 37% higher rotational stiffness than standard portal frame with FRP (FRP-1).

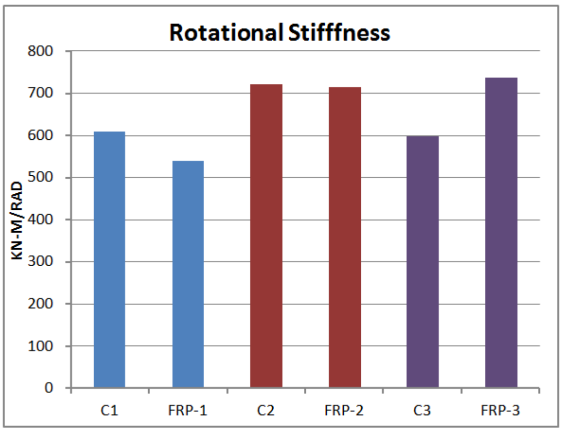


Figure 6: *Average rotational stiffness of Corner joints*

4 FAILURE MODE

Failure modes of all the corner joint configurations with and without FRP were recorded. In specimens FRP-1, the FRP started to tear at both the outermost corner at the top of header and propagated towards the centre. No failure of exterior OSB was observed.

Metal straps did not show noticeable visual elongation or rupture in specimen FRP-2. Bottom plate failed to transfer load and split at the peak load. There was no sign of exterior OSB failure. In FRP-3 failure of the test specimen was due to the rupture of outside OSB followed by failure of middle OSB sheathing. Figure 7 shows typical failure modes of corner joint configurations with FRP observed during the test.

In C1 failure occurred due to rupture of exterior OSB sheathing on both face. In C2 failure was due to the tearing of metal strap followed by the rupture of exterior OSB sheathing. In C3, the middle OSB sheathing ruptured first followed by a strength increase until rupture of exterior OSB.

(a) Failure of FRP membrane in FRP-1

(b) Failure of bottom plate in FRP-2

(c) Failure of exterior OSB in FRP-3

Figure 7: *Typical failure mode of Corner joints with FRP*

4.1 NUMERICAL MODELING

A non-linear numerical model was developed using SAP 2000 finite element software to validate the test results and behaviour of portal frame corner joint with FRP. In the numerical model, framing member of the narrow braced wall segment was modelled as a beam element, header and OSB sheathing was defined as shell element. Linear elastic behaviour was assumed for framing members and header, and the modulus of elasticity was taken from Canadian timber design code CSA O86**.** OSB was assumed to be orthotropic shell element and the properties were taken from work by [6].

For sheathing to framing and framing to framing nailed connection, a 3D link element was assigned where component 1 represent withdrawal load, component 2 representing joint loaded parallel to the grain of the framing member and component 3 representing joint loaded perpendicular to the grain of the framing member. The metal strap was modelled as a one dimensional link element. Load slip curve of nail and metal strap was also taken from [6].

In order to accurately predict the failure mode of the portal frame corner joint it was assumed that metal strap, placed over the exterior OSB, carried the entire load after rupture of the exterior OSB. Since SAP 2000 cannot remove the component automatically when OSB reached maximum capacity, splitting of OSB sheathing was assigned manually in the numerical model. It was predicted that ultimate lateral load carrying capacity of corner joint increased until the rupture of metal strap.

Test results from specimens C1 and C2 were used to validate the numerical model developed by SAP 2000. Figure 8 shows that the numerical model can effectively predict the behaviour of corner joint assembly. The same numerical model was used to validate the test result of corner joint test with FRP.

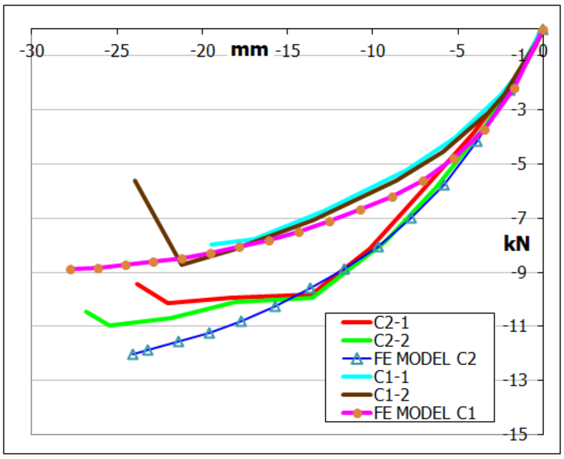


Figure 8: *Validation of FE model with corner joint test results*

From the corner joint test and failure mode it was observed that only small portion (25mm) of FRP membrane was active to take the tensile stress induced by the lateral load. As an example, during pulling of the braced wall segment, FRP membrane at the top of interior stud was active only. Thus the FRP membrane was defined in the FE model as non linear circular cable element. Diameter of the assigned FRP cable was calculated from the equivalent active area of FRP membrane from corner joint experiment. Monotonic load was applied to the numerical model to identify the behaviour of corner joint on that specific direction. The FRP properties were assigned from paper [7]. One-dimensional link was used to represent epoxy used to connect FRP to exterior OSB. To connect FRP to FRP another one-dimensional link with the same tensile properties of the circular FRP cable was used. Figure 9 shows the connection detail of FRP at the top corner of the header and narrow wall segment joint.

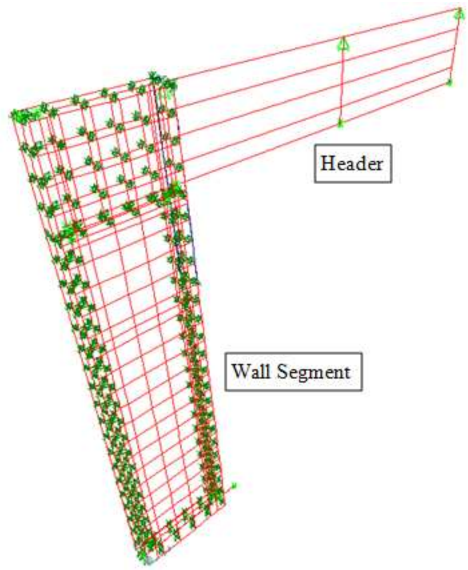


Figure 9: *FE model of Corner joint*

Figure 10 shows that FE model was able to capture the maximum capacity of the specimens with FRP. However the prediction of the initial stiffness is not very accurate.

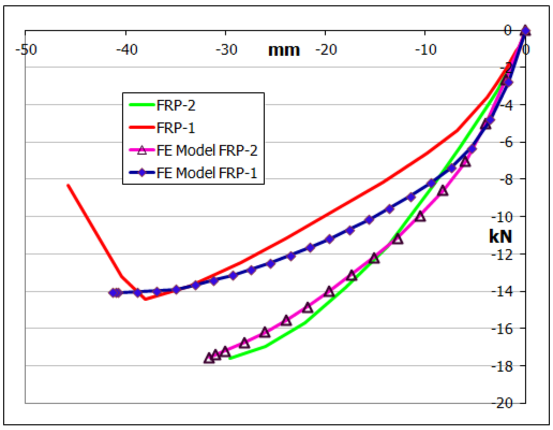


Figure 10: *Comparison of FE model with FRP corner joint*

Table 3 shows the percentage of variation of maximum load, displacement at maximum load and the initial stiffness of FE model compared to corner joint test with FRP. It is clear from this table that, where the model is able to reasonably predict the maximum capacity, the initial stiffness has not been well-predicted well. This is not uncommon for light frame wood structures, as initial stiffness is a function of complex mechanisms involving, amongst others, friction, gaps and other redundancy issue, which the model does not take into account. This could considerably affect the load path, stress distribution and ultimately, the assembly stiffness.

Table 3: *Variation of FE model and Corner joint with FRP*

		Max. Load	Disp @ Maxm. Load	Initial Stiffness
Specimen	Corner Joint Type	KN	MM	KN/MM
FRP-1	Standard	14.26	36.6	0.726
FE Model-1	Standard	14.08	41.3	1.237
% Variation		1.2	12.8	70.4
FRP-2	Standard	17.46	28.0	0.961
FE Model-2	Standard	17.54	31.6	1.179
% Variation		0.5	13.0	22.7

5 CONCLUSION

The research work confirmed that it is possible to increase the lateral load resistance capacity of portal frame by minimum 50% with the use of FRP at top of the exterior OSB. The strength can be further increased by using midply shear wall technique.

Nonlinear finite element model had been developed and validated by the test results. The same model was used to predict the behaviour of corner joint test and found that the structural behaviour of FE model resembled well with corner joint with FRP. The FE model will be used in future to predict the behaviour of full scale portal frame with different configurations of FRP at the corner joint. The optimum corner joint configuration will be tested with full scale portal frame to identify the capacity and behaviour to develop high capacity portal frame.

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