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2011

Kuramoto, H., Li, B., Meas, K., & Fauzan. (2011). Experimental and analytical performance evaluation of engineering wood encased concrete-steel beam-column joints. *Journal of Structural Engineering*, 137(8), 822–833.

<https://hdl.handle.net/10356/95533>

[https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0000334](https://doi.org/10.1061/(ASCE)ST.1943-541X.0000334)

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Experimental and Analytical Performance Evaluation of Engineering Wood Encased Concrete-Steel Beam-Column Joints

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Abstract:

The seismic performance of engineering wood encased concrete-steel (EWECS) beam-column joints is investigated and reported within this paper. Experimental and analytical investigation was carried out on a total of two interior and two exterior beam-column joints. These four beam-column joints typically consisted of an EWECS column and a wood encased steel beam. The primary parameter was the failure modes of the specimens, namely the beam flexural failure and the joint shear failure. The response of the specimens was presented in terms of their hysteresis loop behavior, crack pattern, joint shear distortion, and deformation decomposition ratios. In addition, the results obtained from a three-dimensional nonlinear finite-element analysis simulating their seismic behaviors were also compared with the test data. The finite-element analysis incorporated both bond stress-slip relationship and crack interface interaction at the unbonded connection region. The analytical prediction of joint shear strength was satisfactory for both interior and exterior joints. This validated numerical model was subsequently used to examine the contributions of the steel frame mechanism formed by the column flange, column web, and stiffener.

CE Database subject headings: Beam columns; Joints; Concrete; Steel; Drift; Finite element method; Slip.

Author keywords: Engineering wood encased concrete-steel; Beam-column joint; Drift ratio; Finite element; Bond slip.

Introduction

In high seismic regions, building structures constructed of composite steel reinforced concrete (SRC) have been reported to behave in a ductile manner by Sheikh et al. (1989). Azizinamini and Schneider (2004) reported that concrete filled steel tubes (CFT) add significantly more stiffness to frame structures by utilizing the concrete within the steel to prolong the local buckling of the steel tubes. In return, the steel provides confinement to the concrete to prevent it from spalling. It was from the SRC and CFT ideologies that a new hybrid engineering wood encased concrete-steel (EWECS) was developed for low to high seismic zones. This new composite material overcomes the limitation of the number of stories that can be constructed by utilizing wood as a material on its own. This system consists of a concrete encased steel (CES) column core wrapped with an exterior wood panel. The beam consists of a wood panel wrapped around a steel beam. The construction of this joint began by attaching the wood panel to the steel beam with wood glue. The wood panel utilized to wrap the column was initially assembled using the wood glue. The final step involved casting the concrete within the wood panel of the column. The connection between the wood panel of the beam and column was left in contact without any application of wood glue. It is believed that constructing a building with this system would be environmentally friendly and save time during construction. This is because the wood panel in this proposed system can also act as formwork, unlike buildings constructed from normal SRC. Upon completion of construction, these wood panels, apart from providing additional support, also improve the aesthetic appeal of the building.

There have been previous experimental and analytical studies conducted on EWECS columns to investigate their seismic performance in the past four years, reported by Fauzan et al. (2005, 2006). The results indicated that EWECS columns had excellent hysteresis behavior. The wood panel wrapped around the column was shown to improve its flexural capacity by up to a maximum of approximately 12%, as documented by Kuramoto and Fauzan (2005). Further studies were conducted to investigate the effect of providing a shear stud to attach the wood panel to the core concrete of the column. These studies found that the shear stud provided almost no increase to the strength of the column. However, significant improvement to the column ductility was reported by Meas et al. (2006). An experimental and analytical study was conducted to establish a deeper understanding of the behavior of such interior and exterior joints when subjected to seismic loadings. A three-dimensional (3D) nonlinear finite-element (FE) analysis was also conducted on the EWECS interior and exterior beam-column joint specimens that utilized FINAL version 99 developed by Obayashi Corp. (2004). This FEM analysis was carried out to supplement the experimental research program on the EWECS structural system.

Test Program

Test Specimens

This research program involved the testing of two interior and two exterior one-third scale beam-column joint specimens. One of each of the two types of beam-column joint specimens was designed to fail flexurally in its beam. They are labelled WJA and WJAE from the interior and exterior joints, respectively. The other specimens were designed to fail within its joint. They are labeled WJB and WJBE from the interior and exterior joints, respectively.

The column and beam section dimensions of all specimens were 400 x 400 mm and 200 x 300 mm, respectively. The difference between the specimens designed to fail by joint shear and designed to fail by flexure was the different size of steel used within the beam and in the panel zone. Detailed drawings of the interior and exterior joint specimens are shown in Fig. 1. The test programs carried out for both types of joints are listed in Table 1. Aside from the small differences in mechanical properties of the materials, the sizes of the beams and columns of the interior and exterior joints were kept constant.

Specimen Design

The wood panel wrapped around the column was transformed into a concrete section before utilizing elastic-plastic analysis to determine the ultimate flexural strength of the composite column specimen, introduced by Oehlers and Bradford (1999). The cross-sectional area of the wood panel was converted into an equivalent area of concrete by multiplying it with a transformation factor. This transformation factor is defined as the ratio of the modulus of elasticity of wood over that of concrete:

$$n = \frac{E_w}{E_c} \quad (1)$$

The column was considered to be a stocky column that was subjected to axial compressive force and bending moment about its major axis during the analysis. It was assumed during the analysis that there was full interaction between the concrete and steel. During the elastic-plastic analysis, the stress distribution of steel was assumed to be elastic-plastic, whereas the stress distribution of concrete was rigid plastic that was capable of attaining a maximum strain of 0.003.

The ultimate flexural strength of the EWECs beam was determined by utilizing the same procedure as for calculating the special case of pure bending moment of the EWECs column. The wood panel of the beam contributed to the flexural strength of the beam at a maximum strain of 0.0075 after the steel yields.

The shear strength of the joint region was determined as shown in Eq. (2) by summing up the total of the nominal horizontal shear strength provided by concrete compression strut, V_n , the concrete compression field, V'_n , and the panel zone steel, V_s , as recommended by Viest and Colaco (1997). The wood panel at the joint region was already transformed into concrete within this formula:

$$Q_p = V_n + V'_n + V_s = 1.7\sqrt{f'_c}b_ph + 0.41\sqrt{f'_c}b_0h + 0.6F_{yw}t_w h \quad (2)$$

The shear strength of each component (beam, column, and panel zone) were converted to an equivalent column shear for compatibility and ease in comparison. All converted equations were derived from the free moment diagram around the joint region.

The beam shear strength for the interior joint was converted to an equivalent column shear using Eq. (3):

$${}_cQ_b = \frac{M_{b\max}x_2}{H} \quad (3)$$

The column shear strength of the interior joint was calculated using Eq. (4):

$${}_cQ_c = \frac{M_{c\max}}{\frac{H}{2} - \frac{h}{2}} \quad (4)$$

The shear strength of joint region of the interior joint was converted to an equivalent column shear using Eq. (5):

$${}_cQ_c = \frac{LJ_b}{2L'H - LJ_b} Q_p \quad (5)$$

The beam shear strength for the exterior joint was converted to an equivalent column shear using Eq. (6):

$${}_cQ_b = \frac{M_{b\max}}{H} \quad (6)$$

The column shear strength of the exterior joint was calculated using Eq. (7):

$${}_cQ_c = \frac{M_{c \max}}{\frac{H}{2} - \frac{h}{2}} \quad (7)$$

The shear strength of joint region of the exterior joint was converted to an equivalent column shear using Eq. (8):

$${}_cQ_p = \frac{LJ_b}{L'H - LJ_b} Q_p \quad (8)$$

The columns were intentionally designed to have the highest shear strength capacity. As such, failure modes of the two specimens were determined by the ratio of shear between the joint and the beam. The approximate ultimate shear strength and ratio of each component after converting into column shear is shown in Table 2.

Mechanical Properties of Materials

Plain Concrete

Ready-mixed concrete with compressive strength of 24 and 28 MPa were cast inside the columns of interior and exterior joints, respectively.

Structural Steel

All specimens had a similar $300 \times 220 \times 10 \times 15$ mm H-section steel placed within the column. The steel within the beam and column was connected by welding. The mechanical properties of steel for interior and exterior joints are listed in Table 3.

Wood

Wood from a larch tree was used to create glue-laminated engineering wood used in the specimens. An average peak compressive stress of 45 MPa and a modulus of elasticity of 11.50 GPa were obtained from a sample compressive test conducted in a direction parallel to the annual growth ring of the wood from the interior joint. A test similar to that for exterior joint was conducted on the wood and produced an average peak compressive stress of 51 MPa and a modulus of elasticity of 11.59 GPa. The peak compressive stress obtained from a test conducted in a direction perpendicular to the ring of wood produced 5 MPa for the wood from both joints. Although the tensile strength of wood in the direction parallel to the annual growth ring from testing was approximately 60 MPa, reported by Calderoni et al. (2006), in the specimen design the contribution of the wood panel beam and column to tensile strength was too small. The glue is epoxy resin used for attaching the laminated wood together to form engineering wood. The

result from experimental testing of engineering wood under shear force in the direction parallel to the annual growth ring showed that shear cracking occurs in the wood rather than along the glued surfaces of the wood if the glue application is well prepared. The compressive strength of the glue when hardened was stronger than the wood. With the advantage of fire resistance of the glue, the engineering wood performance is more resistant to fire than normal wood.

Test Setup

The specimen was laterally loaded at the top of its column to simulate beam-column joints within a frame that produced inflection points at the midspan of the beam and at the midheight of the column. The test setup of the interior joint of the specimen is shown in Fig. 2.

The interior joint specimens were cyclically loaded with lateral shear forces while applying a constant axial load of 615 kN. This axial load was determined as $0.3 N_0$, where N_0 is the total compressive strength of the concrete column core. The exterior joint specimens were also loaded cyclically with lateral shear forces. However, the axial load that was applied was varied at $N = 0.1N_0 \pm 3Q$, where N_0 is the total axial compressive strength of the CES column and Q is the applied shear force.

The increments within the lateral load cycles for both interior and exterior joints were controlled by story drift angles, R , defined as the ratio between the relative vertical displacements measured by vertical transducers installed to a gauge holder at the end of the beam to the beam length, which is δ/L . The lateral load sequence consisted of two cycles to each story drift angle, R of 0.005, 0.01, 0.015, 0.02, 0.03, and 0.04 rad, followed by a half cycle to R of 0.05 rad.

Experimental Results and Observations

Interior Joint

Hysterisis Characteristics

Story shear versus story drift angle responses of Specimens WJA and WJB are given in Fig. 3. As shown by the continuous line in the figure, both specimens showed a stable shear versus story drift angle response. The first yield for Specimen WJA with beam flexural failure occurred on the steel flange of the beam when a load of 221 kN was applied at R of 0.004 rad. Its maximum shear capacity of 435 kN was attained at R of 0.03 rad. The specimen showed stable spindle-shaped hysteresis loops with a little decrease in strength upon attaining its maximum capacity.

The first yield of Specimen WJB with joint shear failure occurred on the steel web of the panel zone at shear force of 163 kN and R of 0.0024 rad. The hysteresis curve showed a little pinching-

shaped but stable behavior with strength degradation after attaining the maximum capacity of 393 kN at R of 0.015 rad. The specimen retained more than 75% of its peak strength at the last story drift R of 0.05 rad. The maximum capacity of this specimen was lower than that attained by Specimen WJA.

Failure Mode

The development of crack patterns observed on each specimen was different. More damage was observed on the wood panel along the front and back face of the column on Specimen WJB than Specimen WJA. The damage is visible in Fig. 4 for WJA and WJB. A little crack was observed on the column face of Specimen WJA at R of 0.03 rad. In contrast, at this same ratio, a splitting of the wood panel was observed along the top and bottom column face of Specimen WJB. In addition, both specimens had only slight damage to the wood panel around the beams. This could be attributable to the sinking (embedment) and uplifting of the wood panel of the beam that occurred at the connection of the wood panels of the beams and columns. After completing the tests, the wood panels from the front and sides of the column were removed to inspect the damage on the concrete on the joint. The in-filled concrete in Specimen WJB was crushed more severely than Specimen WJA. This is shown in Fig. 5.

Deformation Decomposition Ratio

Fig. 6 shows the contributions to the total deformation of the joint by the columns, beams, and joint regions until R of 0.02 rad. The deformation of the column and the joint region were calculated by converting into the deformation of beam. Fig. 6 clearly shows that the beam of Specimen WJA contributed significantly to its total deformation, and the joint region of Specimen WJB was the key contributor to its total deformation.

Exterior Joint

Hysterisis Characteristics

The story shear versus story drift angle responses of both specimens of exterior joints are plotted in Fig. 3. The first yield of Specimen WJAE designed to fail flexurally at its beam occurred on its steel flange at an applied load of 131.5 kN and R of -0.005 rad in its negative cycle. The shear force increased slightly up to its maximum capacity of 247.5 kN at its final story drift, R of 0.05 rad. This increase in its strength might be attributable to strain hardening of the beam steel and some contribution from the sink of the wood panel of the beam into that of the column. The specimen produced a stable spindle-shaped hysteresis loop without degradation of its load-carrying capacity until R of 0.05 rad owing to the formation of a plastic hinge in the beam that resulted in the specimen behaving in a ductile manner.

The first yield of Specimen WJBE, designed to fail by shear in its joint, occurred on its panel zone steel at a shear force of 179.3 kN at R of 0.0043 rad. The panel steel yielded first, followed by the stiffeners, and finally the column flange. This sequence of yielding is observed based on the result from the strain measurement. The hysteresis loop showed a little pinching-shape but stable behavior with strength degradation after attaining a maximum capacity of 342.5 kN at R of 0.02 rad. It retained more than 75% of its peak strength at the last story drift angle, R of 0.05 rad. The strength degradation of the specimen might be attributable to the softening of the concrete core and the cracks in the wood panel of the column, caused by the cyclic loadings.

Failure Mode

Fig. 4 shows the crack patterns on the wood panel around the column for both specimens. For Specimen WJAE, up to R of 0.05 rad, almost no crack was observed on the wood panel of the specimen. However, sink and uplift occurred at the connection between wood panels of the beam and column. This was because of the force exerted by the beam wood in the direction perpendicular to the annual growth ring of the column wood, which has low strength.

Splitting cracks were observed in the middle of the face of the wood panel wrapped around the column of Specimen WJBE at R of 0.02 rad. The strength of the specimen began to decrease after these cracks started to show. In addition, there was only slight damage present on the wood panel of the beams of this specimen.

Deformation Decomposition Ratio

Fig. 6 shows the contributions of deformation by columns, beams, and joint regions to the total deformation of the joints until R of 0.02 rad.

For Specimen WJAE, the contributions of deformation of columns, beams, and joint region at R of 0.005 rad were 15, 74, and 11%, respectively. At R of 0.02 rad, the column and the joint region contribution decreased slightly, and the contribution of the beam increased to approximately 90%.

For Specimen WJBE, the contributions of deformations of column, beams, and joint region at R of 0.005 rad were 12, 41, and 47%, respectively. Owing to yielding of the panel zone steel, the deformations of the joint region increased significantly until R of 0.02 rad.

Finite-Element Analysis

Finite-Element Modeling of the Specimen

An FE analysis was carried out on the EWECS beam-column joint by using a nonlinear FE analysis software package, FINAL version 99, developed by Obayashi Corp. (2004), to serve as a comparison with the experimental data. The specimens were modeled by using solid elements,

two-dimensional (2D) plane stress elements, and interface elements for bond stress-slip interaction and crack interaction.

The 2D plane stress element was used to model the structural steel and solid element, with eight nodes used to model beam and column steel end plates. Concrete and wood materials were also modeled by using solid element. Interface elements, which took into account the bond stress-slip interaction and crack interaction, were used to link elements together.

Figs. 7(a) and 7(b) show the finite-element idealization of the specimens for interior and exterior joints, respectively. In this figure, half of the specimen was modeled by using the symmetrical condition at the center in the longitudinal direction of the beam.

Material Modeling

Modeling of Concrete

The compressive strength of concrete used in both specimens for interior joints was 24 and 28 MPa for both exterior joints. Almost no confinement was considered for the concrete that bears against the column steel flanges and the stiffeners. Therefore, a peak concrete strain of 0.0025 was used in the analysis.

The constitutive model of concrete was the envelope curve of the model under cyclic loading, as shown in Fig. 8. The stress-strain relationship in the rising region was designed on the model developed by Saenz (1964), which was built in the program. In the softening region, the linearly decreasing model was adopted. Compression strength reduction after concrete cracks was considered by using the model proposed by Vecchio and Collins (1986). The fracture criterion of concrete was applied with the adoption of the five parameter model of William-Warnke (1975). When the material in compression was loaded or unloaded, stiffness reduction owing to cyclic stress was not considered, and the secant stiffness was adopted for the stiffness calculation. Tension was taken to be very small after cracking occurred, and the proposed model by Izumo (1989) with coefficient, c , of 1 was applied in the analysis. Moreover, when concrete in tension was loaded or unloaded, the secant stiffness was used to evaluate the stiffness, with the assumption that cracked strain was linearly reduced to zero when stress approached zero. Concrete in the joint panel was subjected to higher shear deformation than in other region. To account for shear stiffness reduction by shear crack deformation, shear transfer model developed by Al-Mahaidi (1979) was included in the analysis, with the use of modified shear transfer coefficient, β , of 0.75.

Modeling of Wood

The constitutive model of engineering wood in the paper was the envelope curve of the model under cyclic loading, as shown in Fig. 9. Owing to the design to allow the force to be applied in the direction parallel to the annual growth ring of the wood, some existing concrete models built

in the program by many researchers might be used in the analysis with some modifications. Stress-strain relationship in the rising region was modeled with the linearly increasing model and the linearly decreasing model in the softening region. The fracture criterion of wood was adopted following the rule of the five parameter model of William-Warnke (1975) for concrete with the input of wood material characteristic. When the wood material in compression was loaded or unloaded, stiffness reduction owing to cyclic stress was not considered and the secant stiffness was adopted for the stiffness calculation. The maximum tensile strength of the wood panel during the analysis was taken as 5 MPa because of the unbonded connection between the wood panel and the steel plates of the column and the beam. This value also took into consideration the lower tensile strength in the direction perpendicular to the grain. Based on data of wood material from tensile testing, the proposed equation for concrete by Izumo (1989) for tension softening after crack was also applied in the analysis with modified coefficient c of 1.5 used for the engineering wood. Compression strength reduction factor after crack was not considered because the compressive strength of the wood during testing might not be reached throughout the test. This was because the peak strain was very large, resulting in a delay of crack attributable to compression force. Experimental testing found that after shear crack on the column wood panel, the ultimate strength of the specimen started to reduce significantly. To account for shear stiffness reduction by shear crack deformation, the shear transfer model for concrete developed by Al-Mahaidi (1979) was included with the use of modified shear transfer coefficient β of 0.35 for wood.

Modeling of Structural Steel

The structural steel was modeled using 2D plane stress elements. The constitutive model of the plate was modeled with von Mises yield criterion with isotropic strain hardening. Cyclic constitutive model of structural steel is shown in Fig. 10. The mechanical properties of the steel utilized for interior and exterior joints are shown in Table 3.

Modeling of Interaction between Concrete and Steel

The behavior of the composite structure is highly influenced by the bond stress-slip effect between concrete and steel. This is because of the reduction of the composite action caused by relative displacement between these two materials. An experimental study on the bond stress-slip characteristic between concrete and steel plates under monotonic loading was conducted by Kim and Noguchi (1994). However, the experimental result showed that the bond stress in the bond stress-slip relationship at the interface between concrete and steel was very small. The data from this experiment of the bond stress-slip relationship were modified for use in this FE analysis as the bond stress-slip relationship under cyclic loading, as shown in Fig. 11; the coefficient of friction of 0.65 between these two materials was also used.

Modeling of Interaction between Wood and Steel of the Beam

The bond stress-slip relationship between the wood panel and the steel of EWECS beams was modeled by using interface elements. This bond stress-slip relationship between wood panel and steel was assigned directly during the analysis with much lower stiffness than that of the bond stress-slip relationship between concrete and steel. The bond stress-slip relationships between the wood panel and the steel of the beam and between column wood panel and the CES core of the EWECS column were assumed the same, and the coefficient of friction of 0.9 between these two materials was used in the analysis.

Modeling of Crack Interface Interaction

The crack interface element that accounted for crack interaction at the unbonded connection between the column wood panel and the beam wood panel was used. Only compression stress was considered in this element to transfer the compressive stress from the beam wood panel to the wood panel of the column. At this connection, the force exerted from the beam wood panel to the column wood panel was in the direction perpendicular to annual growth ring of the wood. To solve this problem in the FE analysis, crack interface element was used to account for the amount of uplift and sink at this connection measured during experiment. The crack interface element was the relationship between sink and compressive force, and between uplift and tensile force. In this case, tensile force was not considered because glue was not applied at this connection to allow a free opening.

Analytical Result for Interior Joints

Story Shear versus Story Drift Angle Responses

The FE analytical results of the story shear versus story drift angle responses were compared to the experimental data of EWECS beam-column joint for both types of specimens, as shown in Fig. 3. The figure shows that the analytical results for story shear versus story drift angle responses of the specimens showed a good agreement with the test results. The analytical models adequately simulated the behavior of the test specimen.

The maximum lateral shear force for Specimen WJA was obtained at a story drift angle R of 0.03 rad from the FE analysis. This was approximately 3% higher than the results obtained from the experiments. The prediction of maximum strength correlated well with the experimental results until this stage, except that the hysteresis loop formed from the FE analysis was a little fatter than that recorded from the experiment.

The maximum shear force for Specimen WJB recorded at a story drift R of 0.015 rad from the

FE analysis agreed well with the experimentally obtained data. The hysteresis loop from FE analysis also exhibited a similar pinching-shape and energy dissipation until this story drift angle.

These good comparative results confirmed the ability of the proposed numerical analysis to predict the maximum strength and behavior of EWECS beam-column joints under constant axial load and lateral load reversals with acceptable accuracy.

Principal Stress Distribution

The results of stress distribution from FE analysis for both specimens showed that the average nodal stress in concrete compression strut that was mobilized against the column flanges and stiffeners was higher than that in the concrete compression field that was away from the column flange in the diagonal direction of joint region. The minimum principal stress exceeded the uniaxial compressive strength of concrete in the compression strut region. Moreover, maximum principal stress also exceeded the tensile strength of concrete, which caused tension-splitting crack formation along the diagonal of the panel zone. Minimum principal stress distribution from analysis showed that concrete in the joint region of Specimen WJA started cracking at R of 0.015 rad, in the diagonal direction, and Specimen WJB concrete started cracking at R of 0.005 rad. The minimum principal stresses in concrete of Specimen WJB are shown in Figs. 12(a) and 12(b) in the concrete field and strut, respectively.

The wood panel of the column was the primary factor in contributing its strength to the total joint shear through its shear strength. The principal shear stresses in the wood panels of the columns of Specimens WJA and WJB are shown in Figs. 13(a) and 13(b), respectively. These principal shear stresses as shown inside the oval shape show that cracks occurred on the wood panel of the column at a shear stress of approximately 6 MPa at maximum shear force at the location where the wood panel of the column were assembled together by using wood glue. The tangential shear strength of the normal wood suggested by Calderoni et al. (2006) was averaged at 7.44 MPa. The crack occurring at this location might be attributable to the weak shear strength of the connection using wood glue during construction of the specimens. Cracks also formed at the opposite side of the column, propagated along the column height. The maximum shear strength in the wood panel of the column of Specimen WJB from the analysis was higher than that of Specimen WJA owing to significant yielding of the panel zone steel and diagonal cracking of the concrete.

Story Shear versus Joint Distortion Responses of Specimen WJB

Fig. 14 illustrates the joint distortions of Specimen WJB compared with the analytical and experimental results to a story drift angle of R of 0.02 rad. In the figure, the horizontal axis represents the joint distortion while vertical axis represents the story shear. This indicates that the

joint distortion obtained from the analytical result was similar to the experimental result, which confirmed the accuracy of the proposed numerical analysis.

Analytical Result for Exterior Joints

Story Shear versus Story Drift Angle Responses

Fig. 3 shows the comparisons made between the story shear and story drift angle responses of the experimental and analytical results of these exterior beam-column joints. The figure shows that the analytical results of story shear versus story drift responses of both specimens showed a good agreement with the test results when the applied axial force was in compression. When the applied axial force was in tension, the analytical result showed higher strength than that recorded in the experiment, which might be a result of the difference in the bond stress-slip effect among the applied axial forces in compression and tension.

When the compressive axial force was applied, Specimen WJAE attained a maximum lateral shear force of 234 kN at a story drift angle, R of 0.03 rad through the FE analysis, and experimental data recorded a maximum shear of 247.5 kN at R of 0.05 rad. The analytical result showed a reduction in the specimen strength after a story drift angle of 0.03 rad. It reached the story shear force of 216 kN at R of 0.05 rad.

In contrast, when the compressive axial force was applied on Specimen WJBE, the maximum shear force of 336 kN at R of 0.025 rad agreed well with the experimental test data which recorded a maximum shear force of 342.5 kN at R of 0.02 rad.

Principal Stress Distribution

The principal stress of Specimen WJAE was still lower than the compressive and tensile strength of concrete in both the compression struts and field in the joint region. Cracks would not propagate within the joint region along the diagonal direction. Moreover, the result of shear stress distribution in the wood panel also showed that the maximum shear stress from analysis was much lower than the tangential shear strength of the wood panel, as shown in Fig. 15(a). This indicated that the formation of cracks along the wood panel of the column and along the diagonal direction of concrete in the joint region would not occur. However, at story drift angle R of 0.03 rad, the web and flange of the steel beam exhibited significant yielding.

The minimum principal stress distribution in the concrete of Specimen WJBE showed that concrete in the joint region of Specimen WJBE started cracking at R of 0.015 rad. Moreover, the maximum shear stress in the wood panel of the column was approximately 9 MPa, as shown in the oval shape of Fig. 15(b), from the analysis when it began to crack, leading to the failure of the load-carrying capacity of the specimen. This higher shear stress resulted in shear cracks to originate from that location in the joint region. At that story drift angle R of 0.015 rad, panel

zone steel also exhibited significant shear yielding. This indicates that the specimen will fail in shear owing to the significant yielding of panel zone.

Story Shear versus Joint Distortion Responses of Specimen WJBE

The comparisons between the FE analytically and experimentally obtained joint distortions of Specimen WJBE up to a story drift ratio of 0.02 rad are shown in Fig. 14. Analytically obtained joint distortions were similar to experimentally obtained results when the applied axial force was in compression. However, when the axial force was applied in tension, the joint distortion from the FE analysis was higher than those recorded in the experiment. Moreover, the measured joint distortion exhibited a little more pinching-shape in the hysteresis loop than in the analytical results.

Parametric Study

Interior Joint

The validated FE model for the interior beam-column joint of Specimen WJB was modified by removing panel zone steel to study the contribution of the steel frame mechanism formed by flanges and webs of column steel and stiffeners in the joint panel. This study was conducted in terms of its story shear versus story drift responses. The comparative result of story shear versus story drift angle response of the validated model of Specimen WJB with and without panel zone steel web showed that without panel zone steel web, Model WJB reached its maximum shear capacity of 315 kN at story drift angle R of 0.02 rad. Comparing these two, with and without panel zone steel models, the shear capacity for Model WJB without panel zone steel was 20% less than the shear capacity of Model WJB with the steel web. This indicated that the steel web of the panel zone contributed approximately 20% of the total joint shear.

The principal stress distribution in the column steel from analysis showed that the steel frame mechanism contributed its strength to the total shear by formation of the yielding of the stiffeners, column web, and column flange near the location where the three elements connected together.

Exterior Joint

The validated FE model of the exterior beam-column joint of Specimen WJBE was further utilized to carry out a numerical analysis by removing the panel zone steel to study the contribution of the frame mechanism. The result showed that Model WJBE without the panel zone steel reached its maximum shear capacity of 276 kN at story drift angle R of 0.015 rad.

Comparing these two, with or without panel zone steel models, the shear capacity for Model WJBE without panel zone steel was found to be 18% less than the shear capacity of Model WJBE with panel zone steel web. This indicated that the steel web of the panel zone contributed approximately 18% of shear capacity to the total joint shear.

The steel frame mechanism contributed its strength to the total shear through the yielding of the stiffeners, column web, and column flange near the location where the three elements connected together. The yield was obtained from the principal stress distribution in column steel from analysis. When the panel zone steel was absent, both the interior and exterior joints failed in shear, therefore, the FEM simulation showed that the composite action combined between concrete and steel frame in the joint region solely resisted the applied shear force. Summing up the nominal shear strength of each material, the predicted ultimate shear strength of interior joint of Specimen WJB failed in shear was 37.5% lower than the experimental result. A study should be conducted about this composite action contributed to the horizontal shear force of the interior joint region. The prediction that the ultimate shear strength for exterior joint failed in shear was satisfied, 10% lower than the experimental result.

Conclusions

On the basis of the experimental and analytical investigation conducted on EWECS beam-column joints, the following conclusions can be drawn:

1. Both types of interior and exterior beam-column joints, with beam flexural failure and joint shear failure, caused little damage to the wood panel of the beam owing to the beam wood pane sinking into the column wood panel and the connection between wood panels of beam and column uplifting. The sink occurred because of the placement of the wood panel around the beam, which was placed in a direction perpendicular to the annual growth ring of the wood panel of the column.
2. EWECS beam-column joints had good structural performance with stable and ductile hysteresis behavior upon attaining their maximum strength. The maximum shear strength of both interior and exterior joints, designed to fail by shear, decreased immediately after the cracks started to form on the wood panel of the column.
3. The maximum shear strength of Specimen WJBE, the exterior beam-column joint designed to fail by shear, was 11.5% lower when a tensile axial force was applied than when a compressive axial force was applied. This was because the concrete and column wood did not contribute their tensile strength.
4. The analytical result obtained from the FE analysis was able to accurately simulate the hysteresis behavior of the EWECS interior beam-column joints.
5. The FE analytical results of the prediction of the maximum shear strength of both the exterior joints correlated well with the experimental results. However, the analytical prediction of exterior joint failure in shear was a little higher than the experimental result when the applied axial load was completely in tension. This might be attributable to the bond

stress-slip effect between the applied axial forces in tension, which was different from its behavior when in compression.

6. FE analysis showed that the tangential shear strength that caused shear cracking to form in the engineering wood was approximately 9 MPa. This was a little higher than that of normal wood, which is averagely 7.44 MPa, as reported by Calderoni et al. (2006). At the location where the wood panels of column were assembled with wood glue, the tangential shear strength causing shear cracking was approximately 6 MPa. This might be due to a weak connection that was not well prepared during construction of the specimen.
7. Experimental testing of the specimens of interior and exterior joints failed in joint shear without panel zone steel should be conducted to compare the contribution of steel frame mechanism to the total joint shear with the numerically obtained data in this paper.

Notation

The following symbols are used in this paper:

- b_0 = thickness of concrete compression field outside column flange;
- b_f = width of beam flange;
- b_p = width of column flange;
- E_c = modulus of elasticity of concrete;
- E_w = modulus of elasticity of wood;
- F_{yw} = tensile yield strength of panel zone steel;
- f'_c = concrete compressive strength;
- f_t = tensile strength of concrete;
- f'_{wc} = compressive strength of wood;
- f_{wt} = tensile strength of wood used in analysis;
- H = height of column;
- h = steel column width measured from outer face of column flange to the other outer face;
- J_b = distance from center to center of beam flange;
- L = length of column measured from pin to pin;
- L' = length of column measured from pin to column face;
- $M_{b\max}$ = flexural bending capacity of beam;
- $M_{c\max}$ = flexural bending capacity of column;
- N = column axial load;
- N_0 = total axial compressive strength of CES core of column;
- n = transformation factor for changing wood into concrete;
- Q = applied force;
- Q_p = nominal horizontal shear strength of joint;
- ${}_cQ_b$ = converted beam shear strength to column shear;
- ${}_cQ_c$ = column shear;

${}_cQ_p$ = converted panel zone shear strength to column shear;

R = story drift angle;

t_p = thickness of column flange;

t_w = thickness of panel zone steel;

V_n = nominal shear strength of concrete compression strut;

V'_n = nominal shear strength of concrete compression field;

V_s = nominal shear strength of panel zone steel;

β = shear transfer reduction factor;

γ_p = joint distortion of the joint region;

δ = vertical displacement installed to a gauge holder at the end of the beam;

ϵ_c = peak compressive strain of concrete; and

ϵ_{wc} = peak compressive strain of wood.

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Specimen		WJA and WJAE	WJB and WJBE
Type of failure		Beam flexural failure	Join shear failure
Wood panel thickness of column (mm)		45	
Thickness of panel zone steel (mm)		$H-300 \times 220 \times 10 \times 15$	$H-300 \times 220 \times 4.5 \times 15$
Column	Built-in steel (mm)	$H-300 \times 220 \times 10 \times 15$	
	Column height: h (mm)	1,300	
	Cross section: $B \times D$ (mm)	400×400	
Beam	Built-in steel (mm)	$H-300 \times 150 \times 6.5 \times 9$	$H-300 \times 200 \times 9 \times 19$
	Cross section: $B \times D$ (mm)	300×400	
Applied constant axial force for interior joint		615 kN (12% of CES core strength)	
Applied varying axial force for exterior joint		$0.1 \text{ No} \pm Q$	
No. = 5,411.75 kN (total axial compressive strength of CES core of column)		Q -applied shear force	

Table 1

Specimen	Column shear (kN)	Joint region shear (kN)	Beam shear (kN)
WJA	1,040 (2.80)	371 (1.00)	380 (1.02)
WJB	1,040 (4.24)	245 (1.00)	760 (3.10)
WJAE	1,040 (5.20)	415 (2.07)	200 (1.00)
WJBE	1,040 (3.37)	308 (1.00)	390 (1.27)

Table 2

Steel	Yield stress (MPa)	Ultimate stress (MPa)	Specimen	Note
$H-300 \times 220 \times 10 \times 15$	284.0/304.2	450.9/447.9	INTER/EXTER	Column flange
	295.5/318.9	454.9/460.7		Column web
PL-10	295.5/318.9	454.9/460.7	WJA/WJAE	Panel zone web
PL-4.5	256.7/306.9	337.5/439.4	WJB/WJBE	
$H-300 \times 150 \times 6.5 \times 9$	320.5/304.0	458.0/433.7	WJA/WJAE	Beam flange
	407.7/348.4	510.4/453.4		Beam web
$H-300 \times 200 \times 9 \times 19$	251.6/281.1	440.8/432.9	WJB/WJBE	Beam flange
	293.1/304.3	407.1/446.2		Beam web

Table 3

Fig. 1

Fig. 2

Fig. 3

Fig. 4

Fig. 5

Fig. 6

Fig. 7

Fig. 8

Fig. 9

Fig. 10

Fig. 11

Fig. 12

Fig. 13

Fig. 14

Fig. 15