International Journal of GEOMATE, June 2019, Vol.16, Issue 58, pp.1 - 7
Geotec., Const. Mat. & Env., DOI: https://doi.org/10.21660/2019.58.4778
ISSN: 2186-2982 (Print), 2186-2990 (Online), Japan

AN ANALYTICAL INVESTIGATION OF EWECS COMPOSITE COLUMN WITH AND WITHOUT SHEAR STUDS

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*Corresponding Author, Received: 01 Nov. 2018, Revised: 30 Dec. 2018, Accepted: 21 Jan. 2019

ABSTRACT: This paper explains the numerical analysis of EWECS (Engineering Wood Encased Concrete-Steel) composite columns with and without shear studs connection under constant axial and lateral cyclic loads based on the previous experiment. EWECS column is the new composite structure contains concrete encased steel (CES) core covered by a wood panel. The shear studs were used to connect between CES core and wood panel. In this study, the non-linear finite element (FE) analysis is performed by modeling the column with the 3D solid element and the friction devices of the shear studs with the two-node-link element. The results show that the hysteresis curves shape from FE analysis has similarity with those from the test data as well as on the main stiffness, ductility, and energy dissipation. The addition of shear studs increased the ductility of the EWECS column; however, it had no significant effect on the maximum flexural capacity. The wood panel had the contribution to flexural strength of the EWECS columns until large story drift, R of 5%, even though the damage of the column occurred at story drift of 4%.

Keywords: EWECS column, Shear stud, Cyclic load, Seismic behavior, Numerical analysis

1. INTRODUCTION

Wood or timber is the most commonly used building material for low rise building in Japan. More than 50 % of homes in Japan are built from wood, including log houses due to some reasons such as a good environment, has high earthquake resilience and culture. However, the number of stories for a wooden structure is strictly limited due to fire safety consideration [1]. As a solution to this problem, an engineering wood encased concrete-steel (EWECS) composite structure has been introduced, as shown in Fig. 1. The structural frame of the column consists of EWECS columns, EWES beams, and EWECS beam-column joints.

Experimental and analytical studies on seismic behavior of EWECS columns have been carried out, including the parameter of double and single H-section encased steel and shear span ratio [2-4]. The results show that EWECS had excellent seismic behavior with a stable spindle-shaped hysteresis curve. Also, no significant damage was observed on the column until large story drift, R of 5%. The wood panel gave the contribution to maximum flexural strength by around 12 %.

Generally, the efficiency of a structure will be improved through the composite action from combining the structural elements that create a single composite section. The term "full shear connection" relates to the case in which the connection between the components is able to resist the forces applied [5]. In order to improve the composite action among materials in EWECS columns, the connection between CES core and wood panel has been developed. In EWECS columns, the component that assures the shear transfer between the wood panel and the concrete encased steel core in the composite construction is the shear studs. Steel bolts with diameter 10mm were used as shear studs to connect the CES core to wood panel, as shown in Fig. 2.

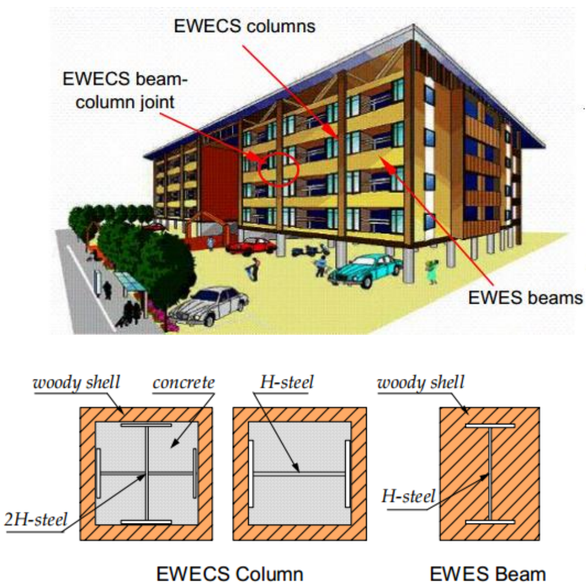
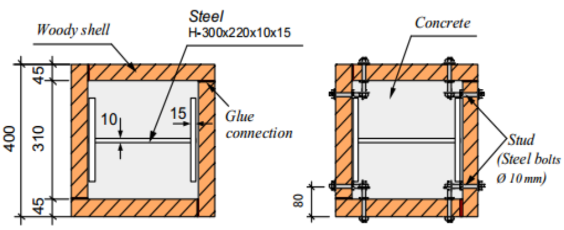


Fig.1 Schematic view of EWECS structural system [2]-[4]

In the previous experimental study [3], two EWECS column specimens were tested: Specimens WC1 (without shear studs) and WC1-S (with shear studs). The influence of shear studs was examined by comparing these two specimens. Furthermore, an analytical study is carried out using the finite element method to validate the test data. The aim of this study is to predict analytically the seismic behavior of the EWECS columns with and without shear connection between CES core and wood panel. The finite element model (FEM) was developed using ANSYS commercial software. The nonlinear concrete behavior and interface connection among the material was considered in this analysis. The analytical results were validated with data obtained from the previous experiment [3].



(a) (b)

Fig.2 The dimension and cross-section of (a) WC1 and (b) WC1-S spesimens [3]

2. ANALYTICAL WORK

Finite element analysis (FEA) is a type of computer program that uses the finite element method to analyze a material or structure in order to find applied stresses that will affect the material or structure. FEA can be used to predict the behavior of a structure under several loading conditions. In this study, FEA of the EWECS columns was carried out using the ANSYS Program APDL v. 14. [6].

2.1 The 3D Models Detail

Two EWECS columns models: Models WC1 (without shear studs) and WC1-S (with shear studs) were constructed in FEA. Both column models had 1600 mm height and 400 x 400 mm^2 cross-sections. A composite section is modeled with single H-section steel of WF 300x220x10x15, and the thickness of the wood panel was 45 mm. The model of the composite column is shown in Fig. 3.

In FEM, the structure will be divided into small elements which accurately represent the geometry of the object. The aspect ratio of plane elements, which describes the element shape in the assemblage, plays an important role in this analysis. This ratio depends on the displacement change in different directions. Totally 7810 elements were used for constructing the 3D FE model of EWECS Column.

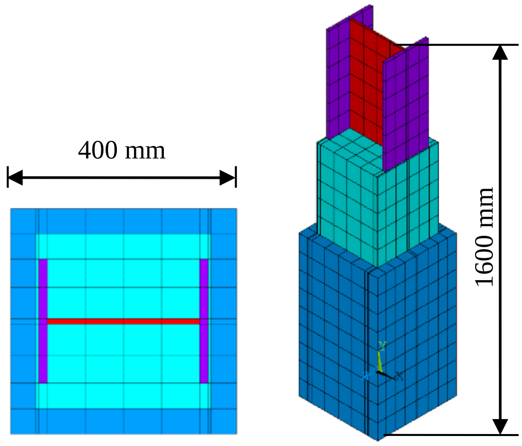


Fig.3 The 3D FE model of EWECS column

2.2 Material Properties

The characteristics of the EWECS column models and the real properties of materials are presented in Table 1.

Table 1 Material properties of EWECS column

Material	Property	Value
Concrete	Compressive strength f_c' (MPa)	35
	Tensile strength f_t (MPa)	3.1
	Young's modulus E_c (MPa)	27800
	Poisson's ratio υ	0.2
	Ultimate comp. strain	0.0025
Steel (Web)	Yield stress f_{sy} (MPa)	313.3
	Poisson ratio υ	0.3
	Young's modulus E_s (MPa)	156700
Steel (Flange)	Yield stress f_{sy} (MPa)	293.6
	Poisson ratio υ	0.3
	Young's modulus E_s (MPa)	146800
Wood	*Compressive strength f_{cw}' (MPa)	45
	Tensile strength f_{tw} (MPa)	5
	Young's modulus E_c (MPa)	11500
	Poisson's ratio υ	0.34
	Ultimate comp. strain	0.0025

Note: * the direction is parallel to the axis of grain.

2.3 Element Types Adopted

SOLID65 and SOLID185 elements were used to model the concrete and steel section, respectively. Both elements have eight nodes having three degrees of freedom at each node: translations in the nodal x, y, and z directions, as shown in Fig. 4. The SOLID65 element is capable of plastic deformations, cracking in three orthogonal directions and crushing. Meanwhile, the SOLID185 has the capability of hyperelasticity, stress stiffening, creep, large deflection and strain and plastic deformation [6].

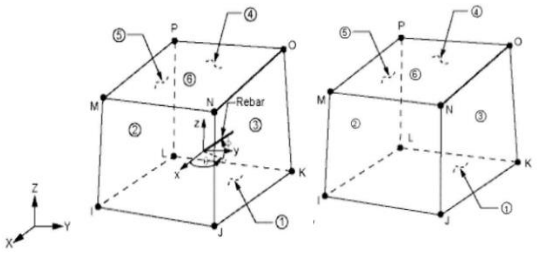


Fig.4 (a) ANSYS Solid65 and (b) Solid185 elements [6]

Elements CONTA174 and TARGE170 shown in Fig. 5 are used for interface elements: the contact and target surfaces, respectively. Contact surface was assumed as surfaces with a finer mesh, while target surfaces were surfaces with coarser meshes. In finite element analysis, if there is a separation between the surfaces in contact, the normal pressure was set to zero [7].

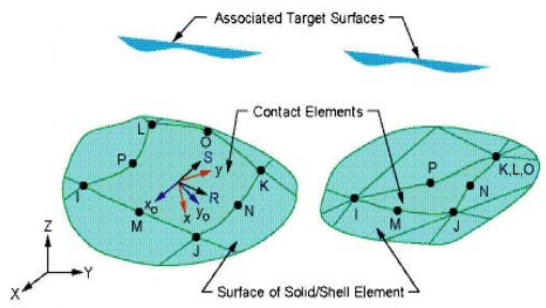


Fig.5 ANSYS Conta174 element [6]

2.4 Constitutive Model of Material

2.4.1 Concrete

Fig. 6 shows the constitutive model of concrete material used in this analysis. The model defines concrete failure based on the William-Warnke failure criterion [8].

The multi-linear isotropic stress-strain relationship for the concrete considering the plastic behavior of concrete was used for concrete in compression. The rising zone of the concrete model based on the developed model by Saenz [9], as seen in Fig. 6. The shear transfer model developed by Al-Mahaidi with the modified shear transfer coefficient, β, of 0.75 [10] was used in this analysis.

2.4.2 Steel encased

In this analysis, the steel element was considered to be a perfectly elastic material, which has similar behavior in both compression and tension. The constitutive model of the encased steel section in EWECS columns is shown in Fig. 7.

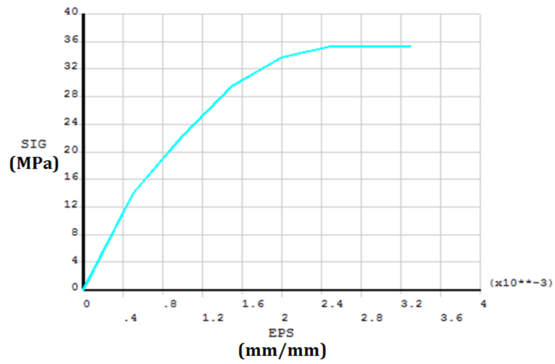
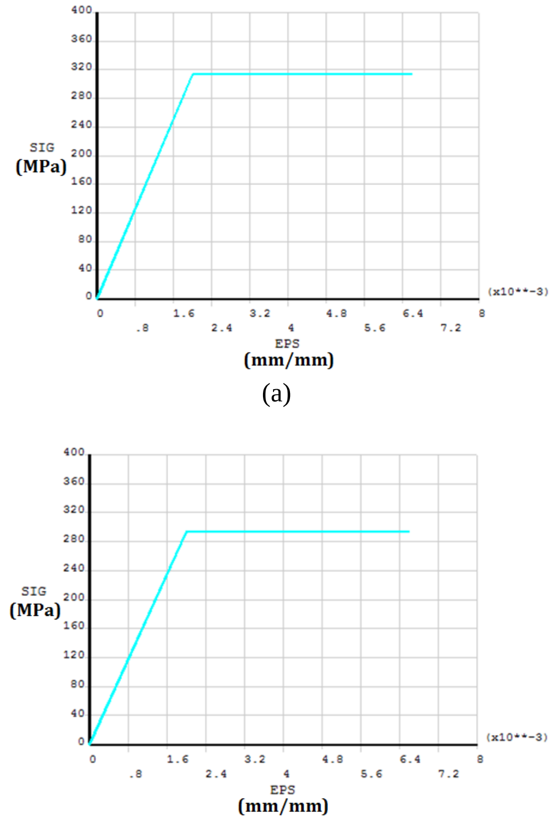


Fig.6 Compressive stress-strain relationship for concrete



(a)

(b)

Fig.7 Tensile stress-strain relationship for (a) web and (b) flange of the encased steel

2.4.3 Wood panel

Fig. 8 shows the stress-strain relationship of the wood panel that was used in this analysis. The model was modified from concrete models built in the ANSYS program. The stress-strain curve was modeled with a perfectly elastic-plastic criterion model and slightly reduced by 5% for the wood panel without shear studs [11].

The failure of wood was assumed following the William-Warnke five-parameter model [8] for concrete with the input of wood properties. Considering the shear stiffness reduction by shear crack deformation, the modified shear transfer coefficient β of 0.35 from the concrete model developed by Al-Mahaidi was used for the wood material [10]. The material interface between steel and concrete is assumed to be perfectly-bonded [12], while the interface between concrete and wood for WC1 is unbonded performed by slightly reducing the stress-strain relationship of the wood panel. The material interface between concrete and wood panel for WC1-S is assumed perfectly bond [13].

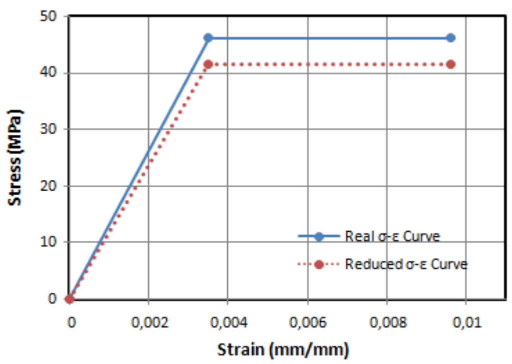


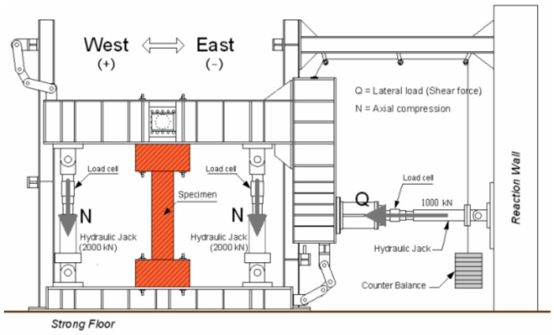
Fig.8 Compressive stress-strain relationship for wood

2.5 Boundary Conditions

The test set up of columns specimens were conducted by fixing the bottom of the column against every possible displacement, while the load was applied at the top of the column. To simulate the experimental work, the bottom end of the simulated specimens was fixed against all the degrees of freedom, as seen in Fig. 9. The applied load was performed by applying displacement to the top of the column, and the column flexural capacity was measured using a reference point at the bottom of the column [14].

2.6 Loads

The loads are made to consider the experimental test setup. The most-top columns were subjected to a constant axial load of 1031 kN represented by applying a point pressure of 1.4 kN on the stub elements with 717 nodal in the FE model. The lateral load cycles for the column were controlled by story drift, R (δ/h). Fig. 10 shows the cyclic loading history represented in the finite element model.



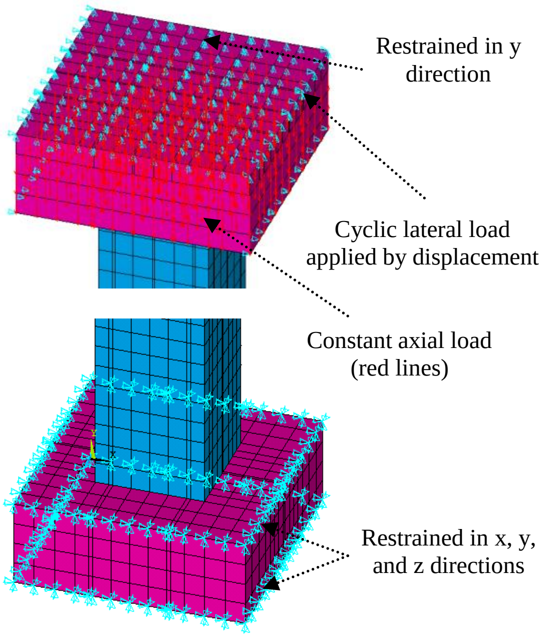


Fig.9 Boundary conditions of the FE model representing the experimental setup

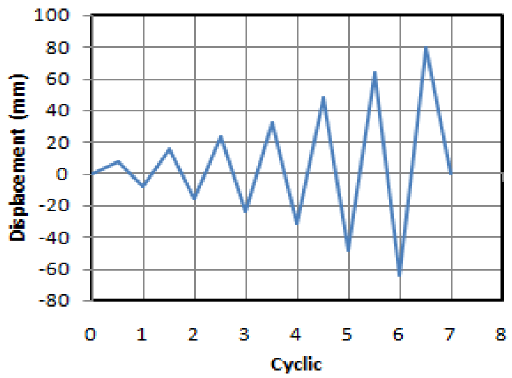


Fig.10 Lateral cyclic load (displacement)

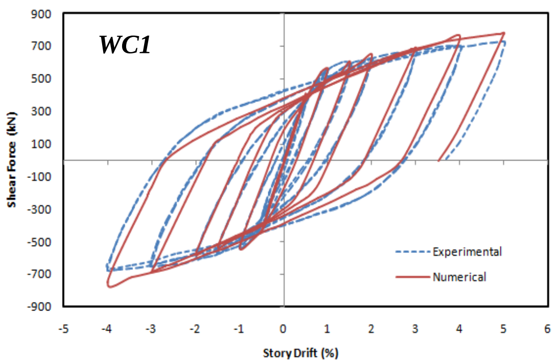
3. RESULTS AND DISCUSSION

3.1 Hysteresis Characteristics

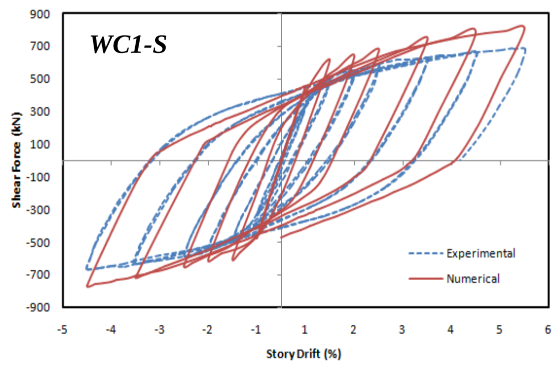
The analytical hysteresis loops (shear force versus story drift) for the WC1 and WC1-S are compared with the test results, as shown in Fig.11.

The analytical results for the hysteresis curve of the FE models matched well with the experimental data. The hysteresis curve of these models (WC1 and WC1-S) is spindle-shaped, which simulated the hysteresis curve from test data. Table 2 lists the measured strength both EWECS columns at first yield and at the maximum capacity. The analytical results show that the maximum shear force of 732 kN was obtained at R of 5% for WC1 which is around 3.1%, higher than those from the test result (709 kN). The behavior of the column in each loading cycle has almost similar between FE analysis and test results (Fig. 11a). In each loading cycle, the different result between FEA and the test data is around 8.7%.

For WC1-S, the maximum shear force from FEA is 804 kN at R of 5%, that is around 11% higher than the test result (725 kN). The hysteresis curve of FEA has higher dissipated energy than those in the test data, as shown in Fig.11 (b). The different result between FEA and the test data in each loading cycle is around 21%. These different results might be due to the assumption used in this analysis, especially the assumption of the material interfaces. The analytical results also show that the contribution of shear studs to flexural capacity is around 9.8% in maximum.



(a)



(b)

Fig.11 Comparison of hysteresis loops between experimental and numerical results of (a) WC1 and (b) WC1-S

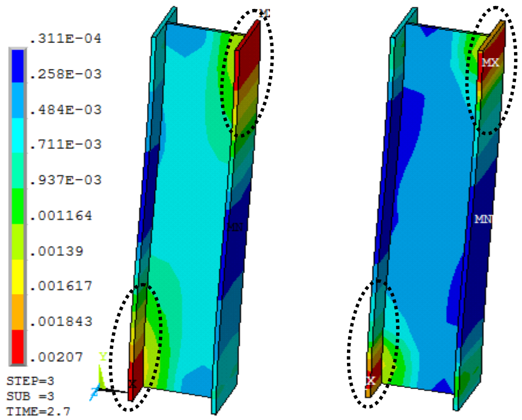
Table 2 Measured strength

Model	Study	at Yielding		at the Max. Capacity	
		Qy	Ry	Qm	Rm
WC1	Exp.	367	0.55	709	5
	Num.	373	0.50	732	5
WC1-S	Exp.	427	0.70	725	5
	Num.	419	0.74	804	5

Note: Q (kN); and R (%).

3.2 Failure Mode and Principal Stress Distribution

Fig. 12 shows the first yield of the steel section that occurred on the top and bottom of the steel. FEA results show that column models WC1 and WC1-S reached a 0.002 principal strain at R of 0.50% and 0.74%, respectively. These results are almost similar to the test data, which are respectively at R 0.55% and 0.70%.

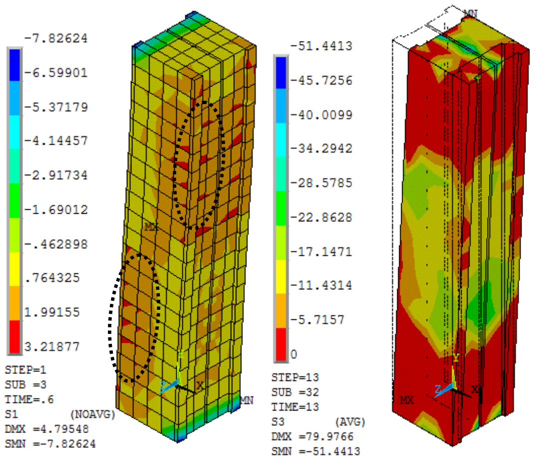


(a) (b)

Fig.12 The principal strain at first yielding of the (a) WC1 and (b) WC1-S models

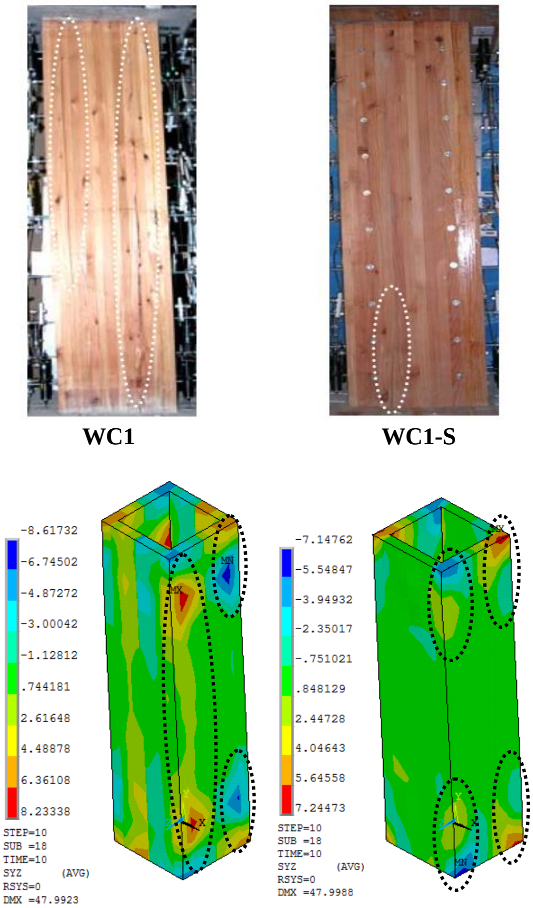
In WC1 column model, the concrete crack firstly was observed at R of 0.4%, in which the maximum principal stresses in tensile is higher than the concrete tensile strength of 3.1 MPa, as shown in Fig. 13 (a). Fig. 13 (b) shows the minimum principal stress (compressive) of the concrete at last story drift, where the value has exceeded the compressive strength of concrete (49 MPa) that resulting in a crush on the top and bottom of the concrete. The similar behavior was also observed in WC1-S.

The principal shear stress in the wood panel is shown in Fig. 14. The figure shows that the crack of the wood panels was observed at a shear stress of approximately 8.2 MPa and 7.24 MPa (maximum principal shear stress) for WC1 and WC1-S, respectively. This corresponds to the value of wood shear strength at averaged 7.44 MPa as suggested by Calderoni [15]. The cracks of wood panel are also observed at the opposite side. They propagate along a height of the column.



(a) (b)

Fig.13 The principal stress at (a) first crack and (b) crush in concrete core of the WC1 model



(a) (b)

Fig.14 Comparison of the failure mode of a wood panel at (a) WC1-S and (b) WC1 between experimental and FE results

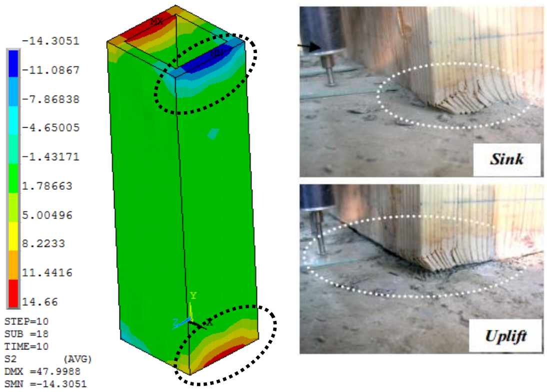


Fig.15 Comparison of the failure mode of the wood panel in terms of sink and uplift of WC1 between experimental and FE results

Fig. 15 shows the 1st principal normal stress on the wood panel for WC1 that is almost similar to those for WC1-S. From the figure, it can be seen that the concentration of stress occurs at the edge bottom and top of the wood panel for both column models (WC1 and WC1-S), indicated by the sink and uplift of the wood panel during the experiment.

4. CONCLUSION

(1) FE models of EWECS columns with and without shear studs had the excellent structural performance with ductile and stable spindle-shaped hysteresis loops until large story drift, R of 5%.
(2) The addition of shear studs increased the ductility of the EWECS column; however, it had no significant effect on the maximum flexural capacity. The increase of maximum flexural strength due to the presence of shear stud is around 9.8%.
(3) By the shear studs, the wood panel had the contribution to flexural strength of the EWECS column until last story drift of 5%, even though the damage of column occurred at story drift of 4%.
(4) The hysteresis loops from finite element analysis matched very well with the test data indicated that the proposed FEA have the ability to predict the flexural capacity and seismic behavior of EWECS columns with acceptable accuracy.

5. ACKNOWLEDGMENTS

The authors gratefully acknowledge the financial support received from Andalas University.

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