Research on seismic behavior of Wood-Concrete Hybrid Structure

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ABSTRACT:

While there is a relatively large body of technical information on the seismic performance of wood-frame buildings to resist seismic ground motions, the quantitative assessment of seismic resistance of wood-concrete hybrid structure is less well understood. In this paper, the shake table tests on 3-storey full-scale wood-concrete hybrid buildings, consisted of a symmetric two-storey wood-frame construction on top of a one-storey concrete frame were briefly introduced. A total of five specimens with different structural configurations representing different stiffness ratios between the wood-frame construction and concrete frame were tested on the shaking table. The natural frequencies, damping ratios and the seismic performances of accelerations and displacements at each floor were discussed. The elastic numerical calculations of these specimens under earthquake actions are performed. Based on these test data and calculation results, some conclusions are drawn. In general, it is found that seismic response is greater for specimens with smaller stiffness ratio.

KEYWORDS: Wood-Concrete hybrid structure, Shake table test, Wood frame construction, Dynamic behavior

1. INTRODUCTION

Wood Frame Construction (below as “WFC”) and Wood-Concrete Hybrid Structure (below as “WCHS”) are major structural types for residential and commercial buildings in North America. They were introduced to China as demonstration and promotion over the last couple of years. During the Northridge earthquake in California in 1994, huge losses resulted from failure of light wood frame structures, which led to the increasing awareness of the seismic performance of WFC and WCHS. Some research achievements are absorbed in lasted building codes, such as the US International Building Code and the National Building Code of Canada. In these codes, seismic design of wood frame construction is expatiated clearly. But, the assessment of seismic resistance of Wood-Concrete hybrid structure is less well mentioned.

In NBC and IBC, a two-stage equivalent lateral force procedure is permitted to be used for structures having a flexible upper portion above a rigid lower portion, with the stiffness of the lower portion must being at least 10 times the stiffness of the upper portion, and the period of the entire structure shall not be greater than 1.1 times the period of the upper portion considered as a separate structure fixed at the base. However, design methods are not specified for a wood-concrete building if the stiffness of concrete frame is less than 10 times the stiffness of wood frame construction. To investigate the dynamic characteristics and the seismic performances of this type of building, FPInnovations and Tongji University jointly conducted shake table tests of three-storey wood-concrete specimens with a range of different stiffness ratios between concrete frame and wood frame construction.

2. SHAKE TABLE TESTS

Five full-scale specimens with different stiffness ratios from 2:1 to 12:1 were tested. Configurations of the specimens are provided in Table 1. The dimensions of all specimens are 6.1m × 3.7m in plan, with 2.8 m storey height for wood frame construction and 3.0 m for the reinforced concrete frame in the first-storey. The cross section of the concrete columns is 270mm × 270mm, and the thickness of concrete slab is 80mm. A schematic and a photo of the specimen are shown in Figure 1.

All the specimens are symmetrical in layout. For the upper wood frame structures, it is estimated that the unit lateral stiffness of the full-height shear wall is approximately 0.5kN/mm/m[3]. Therefore, the design lateral stiffness of wood frame structures at the second-storey with the opening size of 1200mm is 4.8kN/mm for specimens M1 and M3, and 2.4kN/mm for specimens M2, M4 and M6, with opening size of 3600mm. The lateral stiffness of the reinforced concrete frame in the first-storey is adjusted with diagonal steel bracing. The design lateral stiffness for the concrete frame is approximately 10kN/mm for M1 and M2, 20kN/mm for M3 and M4, and 30kN/mm for M6.

Table 1 Specimen configurations and ratios of stiffness between concrete frame and 2nd-storey

Specimen No.	Opening size at the 2nd storey, (mm)	Reinforced concrete frame, and brace type	Stiffness (kN/mm)		Stiffness ratio RC / WFC frame
			RC Frame	WFC	
M1	1220	Only RC Frame, no brace	10.0	4.8	2：1
M2	3660		10.0	2.4	4：1
M3	1220	RC Frame with brace, type 1	20.0	4.8	4：1
M4	3660		20.0	2.4	8：1
M6	3660	RC Frame with brace, type 2	30.0	2.4	12：1

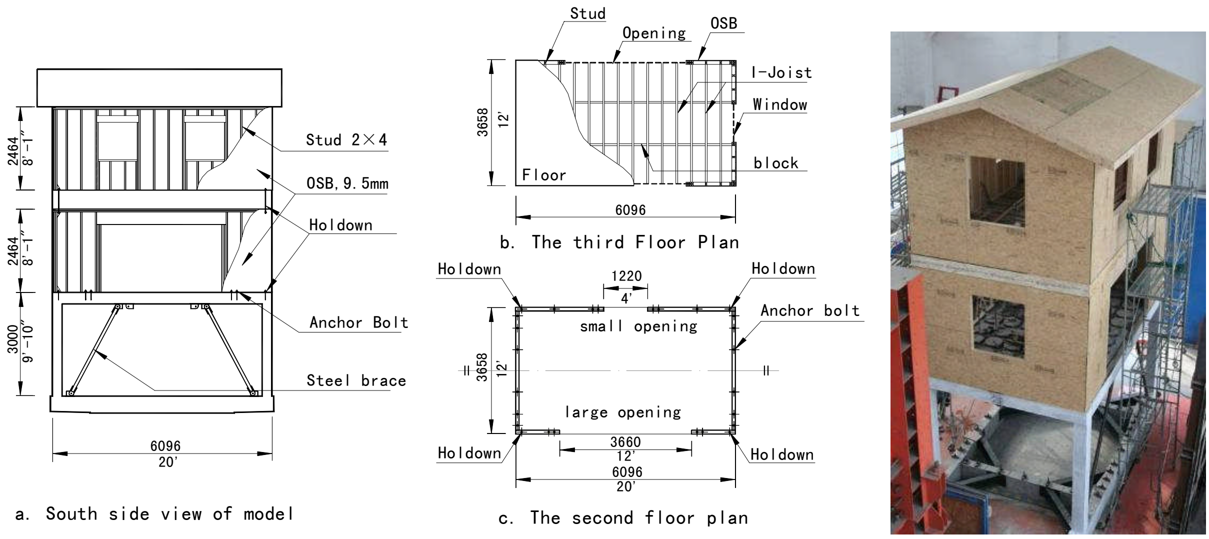


Fig.1 Schematic and photo of the wood-concrete hybrid specimen

The test specimen was built in accordance with the prescriptive requirements of the National Building Code of Canada [2] and Chinese Code for Design of Timber Structures (GB 50005-2003)[4]. The two-story wood frame consisted of studs of 38 × 89 mm No.2 and better grade SPF lumber spaced at 406 mm (16 in.) on center and 9.5 mm (3/8 in.) oriented strand board (OSB) sheathed on the outside of exterior walls. The OSB sheathing was fastened with 65 mm galvanized threaded nails of 3.2 mm diameter, spaced at 150 mm along the perimeter of the sheathing panels and 300 mm elsewhere. The floor consisted of JSI-20 wood I-joists (241 mm in depth) spaced at 406 mm on center and 19 mm (3/4 in.) T&G OSB. The roof consisted of pre-engineered trusses spaced at 600 mm on centre, sheathed with 11 mm (7/16 in) OSB. Anchor bolts of 12.7 mm (1/2 in) diameter at a nominal spacing of 1220 mm fastened the sill plate to the concrete beam. Two anchor bolts were provided within 150mm (6 in) from the opening. Commercially available hold-downs were used at the four corners of the specimens.

Fifty percent of design live load was added to floor and roof. With the assumption of 2.0 kPa for residential occupancy and 0.5 kPa for roof loading, a total weight of 24kN and 6kN were added to the 3rd floor and roof, respectively, to simulate a building with a plan dimension of 4 × 6 m. As the thickness of concrete slab is likely 100mm in real construction, the balance of the weight was added to the living load. Therefore, a total weight of 36kN was added to the 2nd floor.

Three different earthquakes (Taft, El Centro, and an artificially generated Shanghai earthquake record) were applied in three progressively larger steps of shaking intensity, from 0.1g, 0.2g, 0.3g, 0.4g and 0.5g peak ground acceleration (PGA). White noises were applied before and after every shaking to obtain the dynamic parameters of the specimens.

The test specimen was instrumented with 21 accelerometers in X direction (shaking direction) and in Y derection at each end of an exterior wall, at the base, the top of the 1st floor (the beam of concrete frame), the bottom of the 2nd floor (the sill plate), the bottom and the top of the 3rd floor (the bottom plate and the top plate of the 3rd storey frame wall), and the end of the roof ridge. A total of 11 absolute displacement transducers were placed in the direction of shaking at the base, the bottom and the top of the 2nd storey, the bottom and the top of the 3rd storey and the roof ridge. To measure slip and seperation of sill plate and concrete beam, 8 relative displacement transducers were applied in the 2nd floor between the sill plate and the stud at each exterior corner and at each door opening, and 4 at each corner in the second story. Sixteen load cells were inserted between the sill plate and the nut, one at each end of each wall and two at intermediate points or at door openings.

3. TEST RESULTS

For all the specimens, no visible damages were observed for both the wood frame structure and concrete frame up to peak ground acceleration of 0.2g. The horizontal displacements at the floors and roof were insignificant, and nails were tightly attached to the panels. Damages at the corners of wall panels were observed when specimens were loaded at peak ground acceleration of 0.4g and 0.5g. They were primarily nail shear failure, nail withdrawal and panel chip out, which is similar to the failure modes of 2-storey wood frame house. Meanwhile fine cracks were developed at the joints of steel braces in concrete beams and the end of concrete columns.
In general, it was found that seismic response is greater for specimens with smaller stiffness ratio.

3.1 Dynamic Properties

Dynamic properties of the specimens, such as the natural frequencies, damping ratios and the mode shapes were obtained by analyzing the transmission functions of acceleration response at each floor of the specimen. Figure 2 shows the initial natural frequencies (columns in grey color) and the corresponding damping ratios (column in white color) of the five specimens. As noticed, the larger the opening in the second storey, the lower the natural frequencies and the higher the damping ratio for the same concrete frame.

Figure 3 shows the first and second mode shapes of specimens. It indicates that the greater the stiffness ratio, the more flexural the mode shape curves.

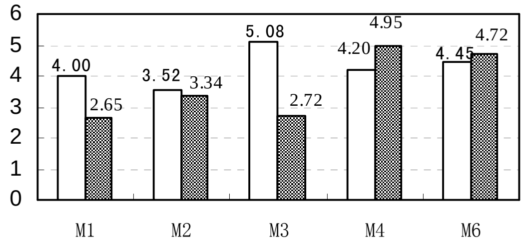
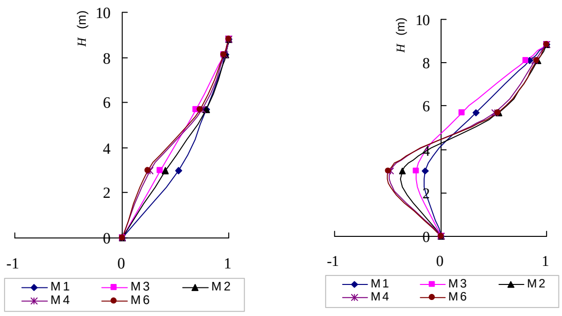


Fig. 2 Frequencies(Hz), damping ratios(％)



First-mode Second-mode

Fig. 3 Comparison of Vibration Mode Shape

Figures 4 and 5 show the change of natural frequency and damping ratio with increasing peak ground accelerations. As the stiffness is proportional to the frequency squared, the progressively lower natural frequency as the level of shaking increases within each specimen is an indication of cumulative damage that resulted in stiffness softening of the specimen. Since damages were largely located in the second storey with little or no damage in the third and concrete frame, this indicates that the actual stiffness ratio between concrete frame and wood frame is increased.

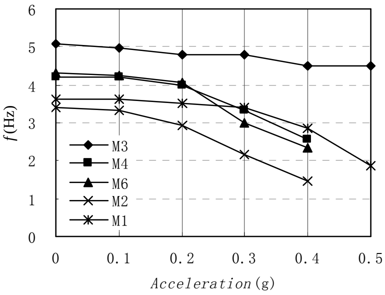


Fig. 4 The change of frequencies

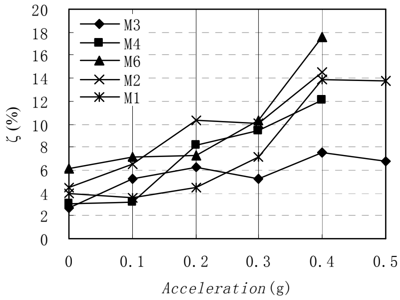


Fig. 5 The change of damping ratios

3.2 Acceleration responses

The acceleration responses of five specimens were recorded at roof, the 3rd storey, the 2nd storey, and the top of the concrete beam by accelerametors when each base excitation. The amplification factors, the ratio of peak acceleration response at each storey to the the peak acceleration of ground motion, is increases with the increase of height in the specimen. Meanwhile, the amplification factors decrease with the stiffness ratio incresed. But, the factors are different with the earthquake input. The acceleration responses are stronger for the specimen under SHW2 and Taft earthquake records than that under El Centro earthquake record.

Figures 6 and 7 illustrate the acceleration response of specimen M1 and M6 at concrete beams and at roof ridge. For specimen M1 (stiffness ratio of RC to WC is 2:1), the frequencies of acceleration responses at concrete beams and at roof ridge are similar. However, the frequencies of acceleration responses at concrete beams and at roof ridge are quite different for specimen M6 (stiffness ratio of RC to WC is 12:1). This indicates that the contribution of second vibration mode becomes more significant with increased stiffness ratio.

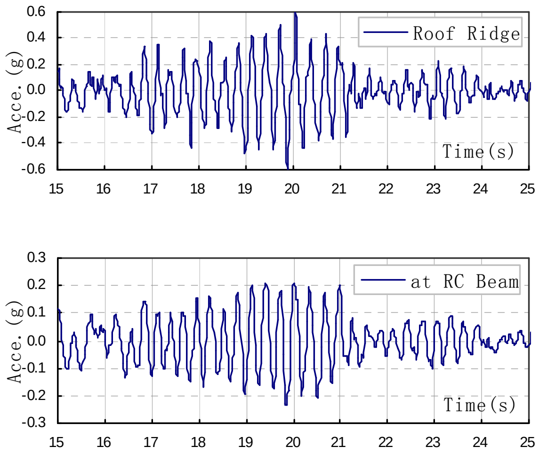


Fig. 6 Acceleration response of M1

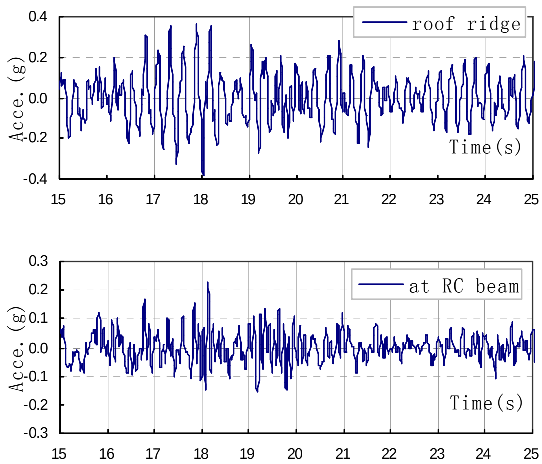


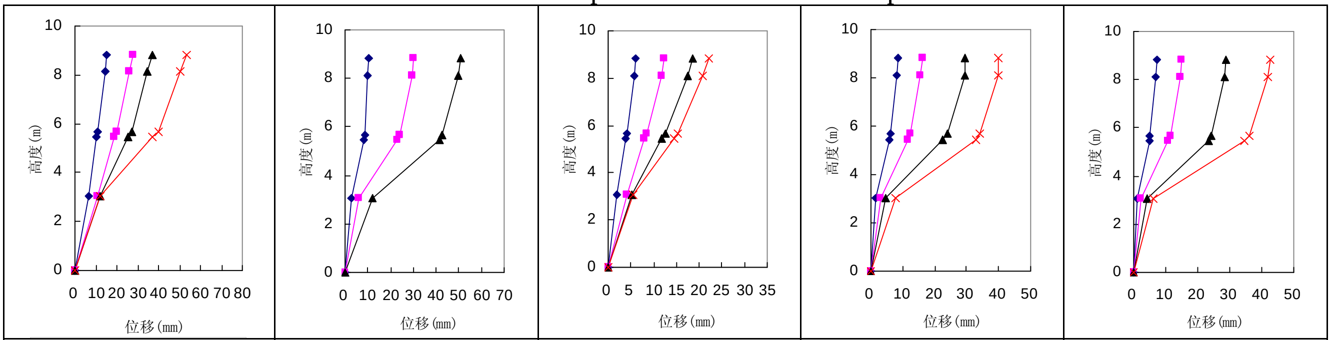
Fig. 7 Acceleration Response of M6

3.3 Displacement response

The displacement responses of five specimens were recorded at roof, the 3rd storey, the 2nd storey when each base excitation. Under the PGA of 0.2g excitation, the four specimens, except M2, are shown as elastic performance and the maximum storey drifts are within 1/250. When the PGA is large than 0.2, the plastic-elastic performance are revealed. The Maximum storey drift is almost 1/49 at the second storey for specimens M1 and M2 when the PGA of 0.5g.

Table 2 shows the maximum displacements at every storey of five specimens under the excitation of SHW2 from 0.1g to 0.5g of peak groud acceleration. The storey drift became more significant with the increasing PGA. The shape of the maximum displacement along the height of the specimens follows the first mode shape of vibration.

Table 2 Maximum displacement curve of five specimens



M1,（2:1）	M2,（4:1）	M3,（4:1）	M4,（8:1）	M6, (12:1)

Note: 0. 1g 0. 2g 0. 3g 0. 4g

4. NUMERICAL ANALYSIS

Based on the shake table tests, the specimens of wood-concrete hybrid structure response are elastic when the peak acceleration of ground motion is less than 0.2g. So, the elastic numerical methods were used to approach the dynamic behaviors and seismic performances of specimens when the PGA is less than 0.2g. In this paper, three kinds of method, which are base shear method, response-spectrum modal analysis and time history analysis are adopted in calculation of natural periods, horizontal seismic action and shear forces of every storey.

4.1 Basic Assumption

All the five kind of specimens with different stiff ratio are symmetric and regular in configuration, and the mass are concentrated at floors and the middle of roof. Table 3 lists the representative value of gravity load and the height of its location. The other configurations and stiffness information are shown in Table 1. The damping ratio of 5% is adopted in calculation, based on the 4%~5% are shown in the shake table tests.

Table 3 Representative value of gravity load（kN）and Height of its location（m）

	Mass 1 at 2nd floor	Mass 2 t 3rd floor	Mass 3 at middle of roof
Representative value of gravity load	105	35	11
Height	3.0	5.7	8.5

4.2 Natural periods

The natural frequencies of each specimen can be calculated by dynamic analysis of Multiple-Degree-Of-Freedom System. In this case, it is a three degree of freedom system and without torsion in horizontal plan. Table 4 listed the calculation values and the test data.

Table 4 The calculated natural periods (s) compared with test data (s)

Specimen		1st period	2nd period	3rd period
M 1	Calculation	0.28	0.15	0.09
	Test	0.25	0.08	—[1]
	Cal. / Test	112%	180%	—[1]
M 2	Calculation	0.32	0.17	0.09
	Test	0.28	0.11	—[1]
	Cal. / Test	113%	156%	—[1]
M 3	Calculation	0.23	0.12	0.09
	Test	0.20	0.07	—[1]
	Cal. / Test	118%	170%	—[1]
M 4	Calculation	0.30	0.13	0.09
	Test	0.24	0.09	—[1]
	Cal. / Test	126%	144%	—[1]
M 6	Calculation	0.29	0.11	0.09
	Test	0.22	0.09	—[1]
	Cal. / Test	129%	121%	—[1]

Note: 1. The period of third mode did not obtained by tests data analysis.

It is noticed that the periods of first mode are good agreement between calculated by dynamic analysis and test results, meanwhile, the periods of second mode are larger than the test data.

In code of timber structure design (GB50005-2003), a simplified formula is suggested to estimate the fundamental period of wood timber construction, $T = 0.05H^{0.75}$. In this formula, H is the height (m) of the building. Due to all five specimens are of height of 8.5m, so the fundamental periods of five specimens are same, T_1=0.25s. It is shown that this simplified approach can estimate the fundamental period of hybrid structure which of symmetric and regular configuration in reasonable way.

4.3 Shear force

Shear forces can be calculated by means of base shear methods, response-spectrum modal analysis and time history analysis.

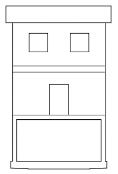
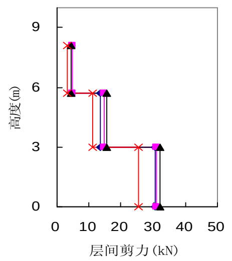
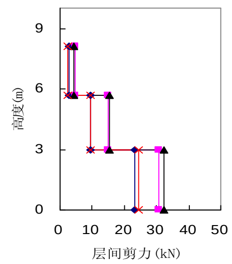
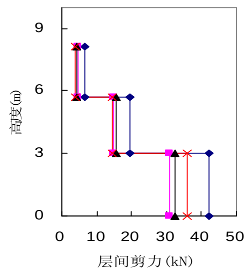
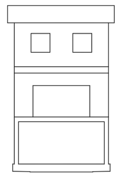
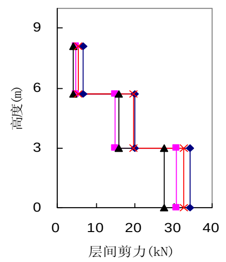
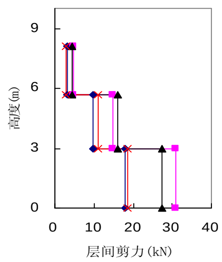
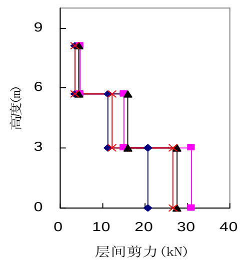
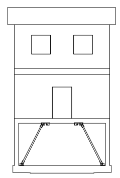
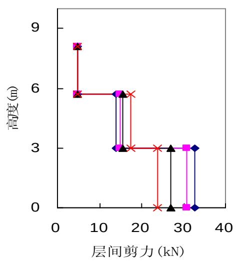
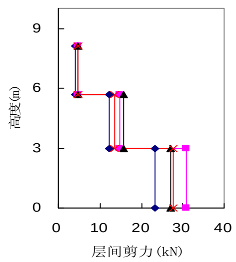
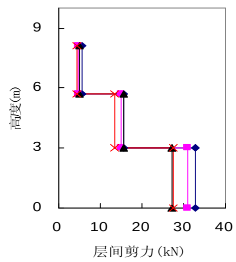
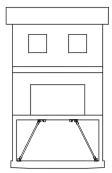
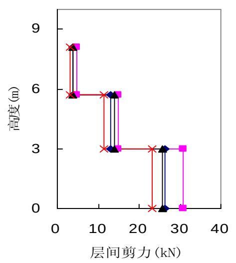
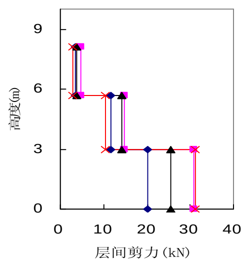
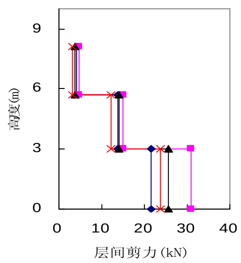
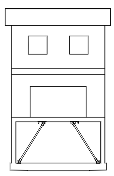
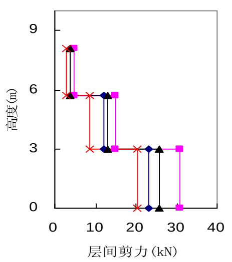
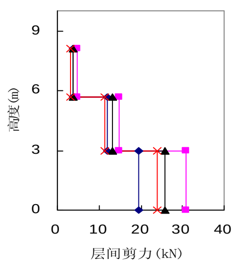
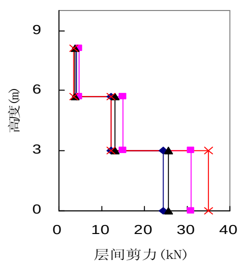
In shake table tests, the earthquake waves are Taft wave, EL Centro wave, which is near earthquake record in site II~III. The Shanghai artificial earthquake wave is made from the site IV. The earthquake coefficient factors can be obtained according to the response-spectrum in seismic design code of building structure (GB 50011-2001). It is noticed that the earthquake coefficient factors corresponding with the 1st and the 2nd periods achieved the maximum value based upon the natural periods which calculated by dynamic analysis.

Shear forces at each storey of five specimens under the excitation of 0.1g of PGA are calculated by three kinds of methods and compared with test data, listed in Table 5. Some conclusion can be obtained that:

1. The calculated values of base shear method, response-spectrum modal analysis and time history analysis are good agreement with the test results. But, to evaluate the shear forces of this kind of hybrid structure which having a flexible upper portion above a rigid lower portion, response-spectrum modal analysis and time history analysis are suggested.

2. This case demonstrates that static approach of base shear method can also evaluate the shear forces of these hybrid structures which stiffness ratio of concrete frame to woodframe construction is not large than 4, and the structure configurations are regular and symmetric in plan.

Table 5 The seismic shear force of each story (kN) of specimen

Model	Input Wave: Taft	Input Wave: El Centro	Input Wave: SHW2
 M1, SR = 2:1			
 M2, SR = 4:1			
 M3, SR = 4:1			
 M4, SR =8:1			
 M6, SR = 12:1			
Test Results Response-Spectrum Modal Analysis		Shear Base Method Time History Method	

5. CONCLUSIONS

Five specimens with different lateral stiffness ratios were designed and tested. Based on the test results and numerical analysis, following conclusions are obtained:

For the wood-concrete specimens, the upper wood frame construction has good seismic performance. The structures did not collapse with the maximum inter storey drift of 1/49 under the peak ground acceleration of 0.5g. The main damages is at the corners of wall panels, especially near the openings, which shown as primarily nail shear failure, nail withdrawal and panel chip out. Meanwhile fine cracks were developed at the joints of steel braces in concrete beams and the end of concrete columns.

The natural frequency of the wood-concrete specimens is determined by the natural frequency of the upper wood frame construction. The bigger the size of the opening, the lower the natural frequency and the larger the damping ratio. In general, the first mode shape dominates the response. It is noticed that, in the specimens with higher stiffness ratio, the contribution of second vibration mode was apparent.

For the wood-concrete specimens tested, the acceleration response, displacement response, acceleration amplification factor of upper wood frame structure decrease with the increase of stiffness ratio of concrete frame to wood frame. In general, it was found that seismic responses are greater for specimens with smaller stiffness ratio than that with larger stiffness ratio.

Dynamic analysis of Multi-degree freedom system can be used to calculate the natural periods of five specimens in this test. And the calculated values are good agreement with the test results. The simplified approach, $T = 0.05H^{0.75}$, can estimate the fundamental period of hybrid structure which of symmetric and regular configuration in reasonable way.

The shear forces of every storey can be calculated by means of base shear method, response-spectrum modal analysis and time history analysis. To evaluate the shear forces of this kind of hybrid structure which having a flexible upper portion above a rigid lower portion, response-spectrum modal analysis and time history analysis are suggested. But in this case which structure having regular and symmetric configurations, and the stiffness ration is not large than 4, static approach of base shear method can also be adapted.

6. ACKNOWLEDGEMENT

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