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SEISMIC PERFORMANCE OF MID-RISE LIGHT WOOD FRAME BUILDING CONNECTED TO A STIFF CORE

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ABSTRACT: The storey limit of residential light wood frame buildings in Canada has been raised from 4 to 6 in the Province of British Columbia since 2009. Increase in height leads to a more flexible building system, necessitating the need to rely on the stiffer elevator shaft and stairwell core to reduce building deflection under lateral loads. The seismic design requirements of this hybrid building related to the seismic force modification factors R_dR_o are currently conservative in National Building Code of Canada. Empirical equations are proposed in this paper to estimate these values of hybrid building based on the properties of its sub-systems. Incremental dynamic analyses of 67 buildings with different property combinations of sub-systems and the connection were conducted through the use of a two-dimensional numerical modelling method. This method was implemented by commercial software ABAQUS V6-10 together with a user-developed subroutine that incorporates the Bouc-Wen-Barber-Wen (BWBN) model to describe the hysteresis behaviour of various building components. The results show that a value of R_dR_o higher than the lowest value of the two sub-systems can be used to design the hybrid building system. The fundamental period of the hybrid building is linearly proportional to the stiffness ratio of masonry core to the whole building.

KEYWORDS: Mid-rise hybrid building, Seismic force modification factor, Numerical modelling, wood-masonry connection, Fundamental period

1 INTRODUCTION

In North America, light wood frame structures have traditionally been restricted to 4-storey in building codes. In design practice, the elevator shaft and stairwell core made of reinforced concrete or masonry are usually treated independently from the wood frame. To achieve this, a physical gap is often left between the wood frame and stiffer core to ensure no pounding under lateral loads. As the storey limit of residential building has been increased from 4 to 6 (mid-rise), there is a potential need to rely on the stiffer core, which is often present as elevator shaft and stairwell, to provide additional resistance to reduce building deflection under lateral loads. Physically attaching the wood frame to the stiff core needs special attention to the structural performance of hybrid building system under seismic loads as these two materials have vastly different physical and mechanical properties. Two issues arise from a structural perspective. The first one is related to the determination of seismic force modification factor. The seismic force modification factor for shear wall system made of wood is 5.1 while for moderate ductile reinforced masonry system is 3.0. According to the National Building Code of Canada (NBCC) [1], if two systems are rigidly connected, the lower value of the force modification factor, R_dR_o (R_d is the ductility-related force modification factor; R_o is the over-strength related force modification factor) of the two systems shall be used for the hybrid building system which is likely to be conservative. However, no design information is provided in the code if the two systems are connected with ductile connections. Another issue is related to the estimation of fundamental natural period of hybrid building system. In the equation suggested by Clause 4.1.8.11.3) c) of NBCC [1], the fundamental natural period of a shear wall building is only related to its height irrespective of structural material, while light wood frame building systems usually have longer natural period than masonry with the same building height. A modified equation is needed to estimate

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the fundamental period of hybrid buildings based on the properties of the sub-systems.

In this paper, a two-dimensional (2-D) model built with ABAQUS V6-10 together with a user-developed subroutine that incorporates the Bouc-Wen-Barber-Wen (BWBN) hysteresis model was used to simulate the seismic performance of multi-storey hybrid buildings with different resistance ratios and ductility properties of sub-systems and connections under earthquake load. The purpose of these analyses was to quantify the seismic force modification factor R_dR_o, fundamental period of hybrid building system and how the properties of a mid-rise hybrid building system are affected by properties of sub-systems and of the connection between them. Finally, empirical equations are proposed based on the research work.

2 SEISMIC FORCE MODIFICATION FACTOR

Seismic force modification factor R is used to reduce the design seismic load. In NBCC (2010) [1], it includes two parts R_d and R_o. R_d is the ductility force modification factor reflecting the capability of a structure to dissipate energy through inelastic behaviour and R_o is over strength-related force modification factor accounting for the dependable portion of reserve strength in a structure. There are several ways to quantify the R value. Newmark and Hall [2] related the ductility force modification factor R_d to the ductility factor μ through dynamic analysis of a perfect elastic-plastic system under El Centro earthquake:

T<0.03s, $\quad R_d = 1$ (1a)

0.1s<T<0.5s, $\quad R_d = \sqrt{2\mu - 1}$ (1b)

T>0.5s, $\quad R_d = \mu$ (1c)

where μ is expressed as the ratio of maximum permissible displacement over the yield displacement.

These equations were developed based on the analysis of a perfect elastic-plastic system without considering the pinching effect of a hysteresis performance. Eq. (1) may overestimate the R_d value of structures studied in this project that have significant pinching phenomena. Besides, the ductility factor, μ, is sensitive to the definition of the yield point. Since the real load-displacement curve does not have clear yield point, Eq. 1 can only provide a rough estimation of R_d factor. The Federal Emergency Management Agency (FEMA) [3] provides a methodology for quantifying the response parameters of a structural system for use in seismic design. This method was developed with an initial assumption of R value for a new structural system followed by a systematic assessing procedure based on the adjusted collapse margin ratio. In this method, the initial assumption of R value largely depends on the designer's knowledge and experience in structural performance under seismic load. A few iterations are usually expected before a final value is accepted. This process is obviously tedious. Ceccotti and Sandhaas [4] proposed two alternative procedures for determination of the R factor for wood structures. In method one (Fig. 1 (a)), the structure was designed elastically with an initial R=1. In the following incremental dynamic analysis, the intensity of the ground motion represented by the peak ground acceleration (PGA) was increased to a level of $I_{\text{near-collapse}}$ on that the structure reaches its ultimate deformation U_m.

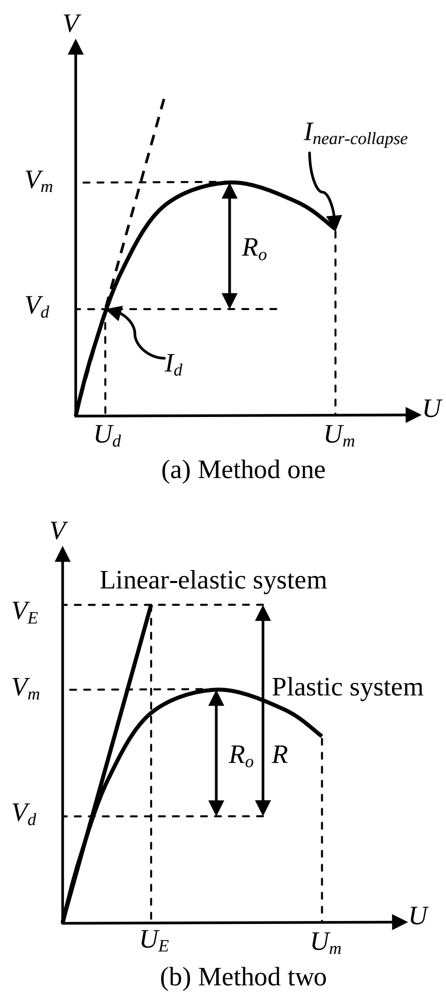
$$R = I_{near-collapse}/I_d \quad (2)$$

where I_d represents the seismic intensity at design level and $I_{\text{near-collapse}}$ represents the seismic intensity on that the non-linear system reaches its ultimate deformation.

In method two as shown in Fig. 1 (b),

$$R = V_E/V_d \quad (3)$$

Where V_E is the base shear developed in the corresponding linear-elastic system under the same level of ground motion on that the non-linear system reaches its ultimate deformation and V_d is the design base shear of non-linear system.



(a) Method one

(b) Method two

Figure 1: *Illustration of methods of determining seismic force modification factor.*

In this project, a modified method based on the method one proposed by Ceccotti and Sandhaas [4] was used. Instead of elastic design assumption with R=1, an initial $R_{\text{design}}>1$ was assigned to reduce the design seismic load of hybrid building. Thus, the fundamental period of analyzed building is closer to the one of a real structure. Another modification was related to the ground motion scaling method. The PGA based scaling method suggested by Ceccotti and Sandhaas [4] can introduce a large scatter in the analysis results [5]. So ground motion scaling based on the response spectrum was used in this project. And the R was separated into R_d and R_o with Eq. (4) and Eq. (5).

$$R_o = V_m/V_d \tag{4}$$

$$R_d = R/R_o \tag{5}$$

where V_m is maximum base shear developed at the non-linear plastic system and V_d is design base shear.

To sum up, the procedures used in this project to obtain the R_dR_o of hybrid structure are as follows:

- Design hybrid building with an initial assumption of $R_{\text{design}}>1$.
- Create 2-d numerical model based on the reduced design base shear V_d.
- Scale the intensity of ground motion to a level of I_d on which its relevant response spectrum marches the target spectrum
- Scale the intensity of ground motion at 0.05 times of I_d until near-collapse criteria were reached. The intensity is represented by $I_{\text{near-collapse}}$.
- Calculate the R with Eq. (6)
- Calculate the R_o and R_d with Eq. (4) and Eq. (5)

$$R = R_{design} \times I_{near-collapse}/I_d \tag{6}$$

3 DESIGN OF HYBRID BUILDING SYSTEM

A set of multi-storey buildings with different building height (4, 6 and 8 storeys), resistance ratio of masonry to hybrid building (0, 0.2, 0.4, 0.6, 0.8 and 1), design method (D1, D2 and D3) and ductility level of connection and masonry (moderate ductility and limited ductility) were designed in this project. In D1, the resistance of connection is equal to that of masonry wall on the same storey; in D2, the resistance of connection is equal to that of masonry wall on the bottom storey; in D3, the resistance of connection and masonry is equal to that of masonry wall on the bottom storey. Fig. 2 shows the schematic of design base shear of hybrid building system under the three different design methods. Two ductility levels of connection and masonry wall are considered in this project. The hysteresis curve of limited ductility (LD) connection and masonry are artificially scaled from the moderate ductility (MD) values of a real test (see Fig.3). The equivalent energy elastic plastic (EEEP) curves were derived according to ASTM E2126 – 09 [6].

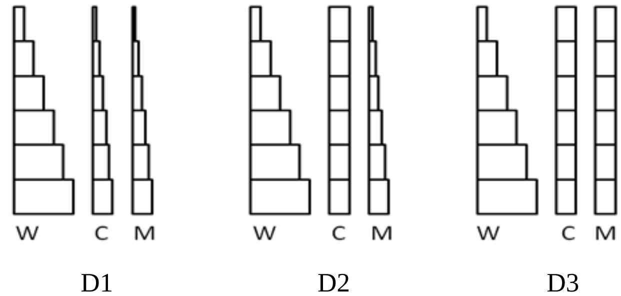
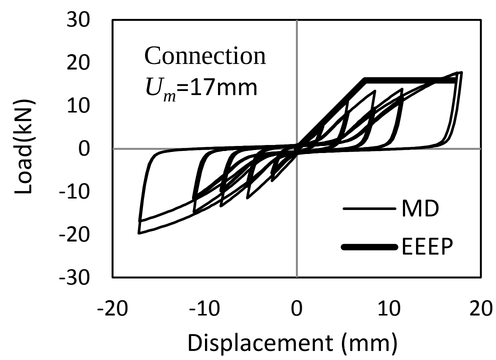
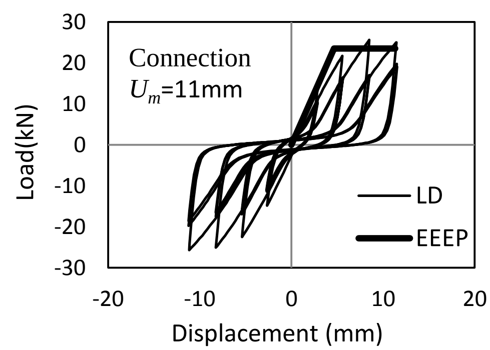


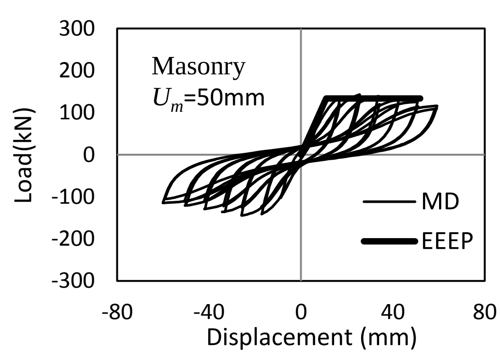
Figure 2: *Schematic of design storey base shear of hybrid building system*



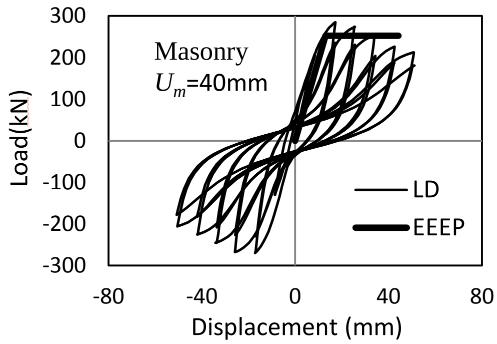
(a) ½ in. bolt connection



(b) Artificial curve of connection



(c) 1×2.8m masonry wall



(d) Artificial curve of masonry wall

Figure 3: *Hysteresis loops and EEEP curves of connection and masonry wall with moderate and limited ductility*

These buildings were assumed to be located in Vancouver (Vancouver City), Canada with site class of D (stiff soil). The total building height was 11.2m, 16.8m and 22.4m for 4, 6 and 8-storey buildings respectively with a storey height of 2.8m. Fig. 4 shows the layout of hybrid building containing wood shear walls and masonry walls. The total floor area of the building was 696 m^2. Dead load of 0.7kPa, 1.3kPa and 0.5kPa was assigned respectively to roof, floor and partition walls of these buildings. The self-weight of reinforced masonry wall is ignored here as the value is relatively small compared to the weight of wood sub-structures. To simplify the design procedure, only the seismic load applied at the East-West direction of the building is considered in this paper.

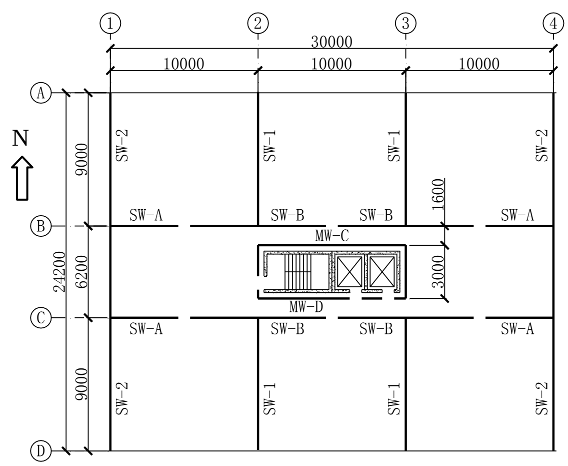


Figure 4: *Layout of the hybrid building containing wood shear walls (SW) and masonry walls (MW).*

The fundamental periods of these buildings were calculated with the empirical formula Eq. (6) provided in NBCC [1].

$$T_a = 0.05(h_n)^{3/4} \tag{6}$$

where h_n is the building height.

The initial R_{design} was assumed to be linearly proportional to the resistance ratio of masonry to whole building (see Eq. (7)). Table 1 lists the design R value of hybrid building. There were a total of 67 building cases designed in this project (see Table 2).

$$R_H = (R_w - R_m)(1 - \alpha) + R_m \tag{7}$$

where R_H is the seismic force modification factor of hybrid building; R_w=5.1 and R_m=3.0 are the seismic force modification factors of wood building and masonry building respectively, α is the resistance ratio of masonry to whole building.

For the numerical modeling purpose, fictitious shear wall length and bolted connection numbers were assigned to each storey of the building, so that the design shear resistance on each storey was equal to its design shear force. According to Clause 9.5.1 of CSA O86 and its commentary [7], the factored shear strength of wood shear wall is approximately equal to half of the measured average ultimate lateral loads from testing. Therefore the total length of the shear walls on each storey of buildings can be calculated by dividing the design storey shear with the factored shear strength. The factored design resistance of masonry shear wall and wood-masonry connection was also set to be half of their measured ultimate strengths.

Table 1: *Initial design R value of hybrid building systems*

	W	H1	H2	H3	H4	M
α	0	0.2	0.4	0.6	0.8	1
R	5.1	4.7	4.3	3.8	3.4	3.0

Note: W represents pure wood structure, Hi represents hybrid buildings with different resistance ratio of masonry to whole structure and M represents pure masonry building

Table 2: *Building cases*

4W	4H1D1	4H2D1	4H3D1	4H4D1	4M
	4H1D2	4H2D2	4H3D2	4H4D1	
	4H1D3	4H2D3	4H3D3	4H4D3	
6W	6H1D1	6H2D1	6H3D1	6H4D1	6M
	6H1D2	6H2D2	6H3D2	6H4D2	
	6H1D3	6H2D3	6H3D3	6H4D3	
6W	6H1D1Lc	6H2D1Lc	6H3D1Lc	6H4D1Lc	6M
	6H1D2Lc	6H2D2Lc	6H3D2Lc	6H4D2Lc	
	6H1D3Lc	6H2D3Lc	6H3D3Lc	6H4D3Lc	
6W	6H1D1Lm	6H2D1Lm	6H3D1Lm	6H4D1Lm	6MLm
	6H1D2Lm	6H2D2Lm	6H3D2Lm	6H4D2Lm	
	6H1D3Lm	6H2D3Lm	6H3D3Lm	6H4D3Lm	
8W	8H1D1	8H2D1	8H3D1	8H4D1	8W
	8H1D2	8H2D2	8H3D2	8H4D2	
	8H1D3	8H2D3	8H3D3	8H4D3	

Note: 4, 6 and 8 are storey numbers; W represents pure wood structure; Hi represents hybrid buildings; M represents pure masonry building; Lc represents limited ductility of connection; Lm represents limited ductility of masonry core and Di represents design methods.

4 TWO-DIMENSIONAL NUMERICAL MODELLING ANALYSIS

4.1 DESCRIPTION OF 2-D MODEL

A 2-d numerical model was used to analyze the seismic performance of the designed buildings under earthquake load. In the 2-d model, all wood shear walls in a storey were grouped into one super element. Likewise, the reinforced masonry walls were represented by another super element. The super element contains three rigid truss elements and two diagonal springs simulating the lateral hysteretic performance of the walls. The two super elements were connected by a pair of hysteretic springs that represent the bolted connections. Fig. 5 shows the schematic of the 2-d modeling approach.

This 2-D model was implemented using the commercial software ABAQUS V6.10 together with a user-developed subroutine that incorporates the Bouc-Wen-Barber-Wen (BWBN) model to describe the hysteretic elements of the building [8]. The BWBN model contains 13 parameters to describe the hysteresis performance of the wall elements and connections. Detailed explanation of the significance of each parameter and equations controlling the hysteresis loops can be found in reference [9]. This 2-D model could reasonably predict the performance of hybrid building system if reliable input properties of system elements are given. Another paper by the authors addresses the verification of the modelling approach [10].

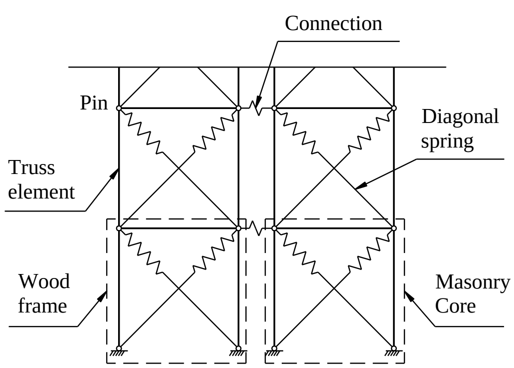
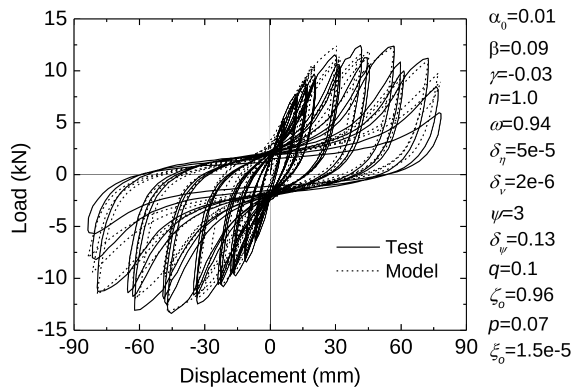


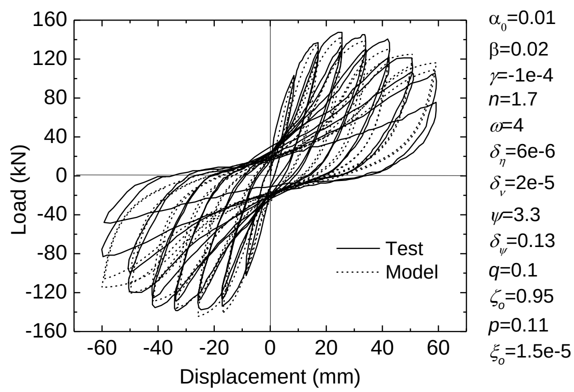
Figure 5: *Schematic of 2-d modelling approach*

4.2 INPUT DATA OF NUMERICAL MODEL

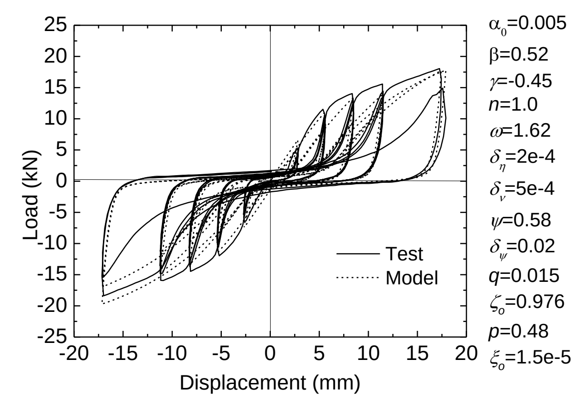
Input data for wood-masonry connection and wood shear wall were obtained from laboratory tests performed as part of this project [10]. The hysteresis loop of reinforced masonry wall was cited from the work by Shedid et al [11]. It was assumed that the lateral deflection of shear wall was proportional to its height and the shear resistance was proportional to its length. So the hysteresis loop of shear walls in this project can be scaled from these test data to meet the dimension requirements. Fig. 6 shows the scaled test data and BWBN model hysteresis loops of wood wall, masonry wall and connection used in this project. The 13 parameters of each BWBN model are calibrated with ±10% difference of dissipated energy (see Fig. 7).



(a) 1m×2.8m wood wall

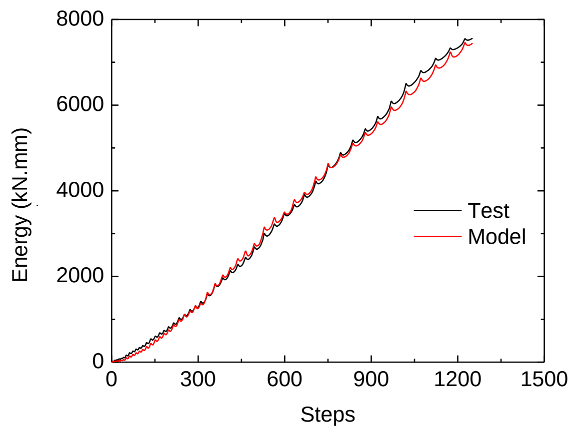


(b) 1m×2.8m masonry wall

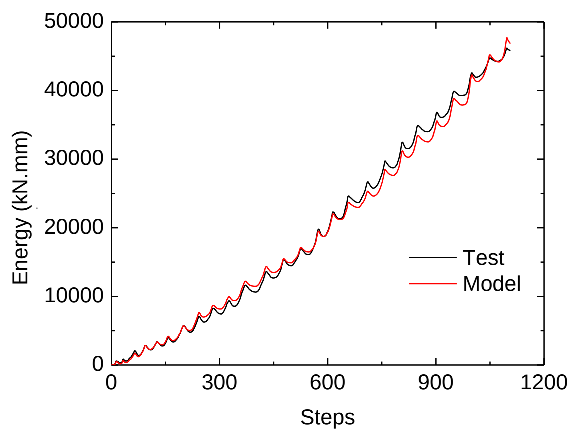


(c) 1/2in bolt wood-masonry connection

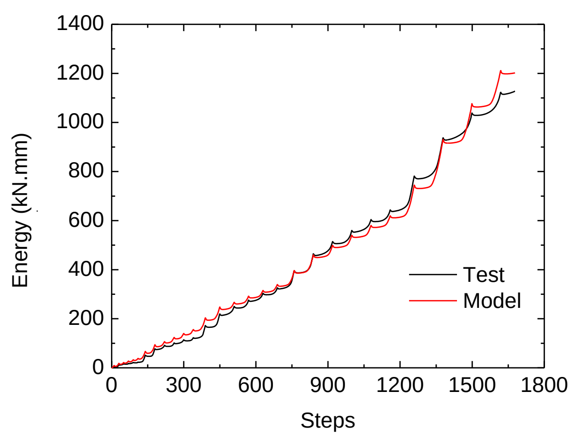
Figure 6: *Comparison of hysteresis loops of test data and BWBN model*



(a) 1m×2.8m wood wall



(b) 1m×2.8m masonry wall



(c) 1/2in bolt wood-masonry connection

Figure 7: *Comparison of energy dissipation of test data and BWBN model*

4.3 GROUND MOTION SCALING METHOD

The ground motion histories (Fig. 8) of San Fernando earthquake was downloaded from PEER Ground Motion Database [12] with an average shear wave velocity within the top 30 m thick of earth, V_{s30}=316.5 m/s (180 – 360 m/s) which meets the site class D assumption. There were two steps to scale the San Fernando earthquake ground motion. First, the intensity of the ground motion was scaled to a level on that the response acceleration was compatible to the design value of the Vancouver response spectrum at the fundamental period range of 0.2s to 2s (Fig. 9). Second, the intensity of ground motion was scaled up and down at 0.05 times of the designed level until near-collapse criteria were reached. Damping was assumed to be 5% in the non-linear dynamic analysis. Failure of the hybrid building system is defined as any part of the wood, masonry and connection sub-system reaches its ultimate deformation of the load-displacement curve. The ultimate deformation of the load-displacement curve is defined as the displacement on which the load drops to 80% of peak load, $0.8P_{peak}$, or the maximum displacement when the failure happened before it reaches $0.8P_{peak}$ (Fig. 3). The ultimate deformations of sub-systems are given in Table 3.

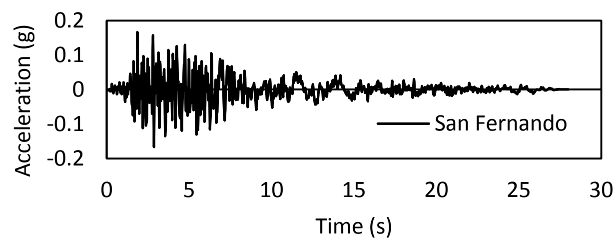


Figure 8: *Acceleration of San Fernando earthquake*

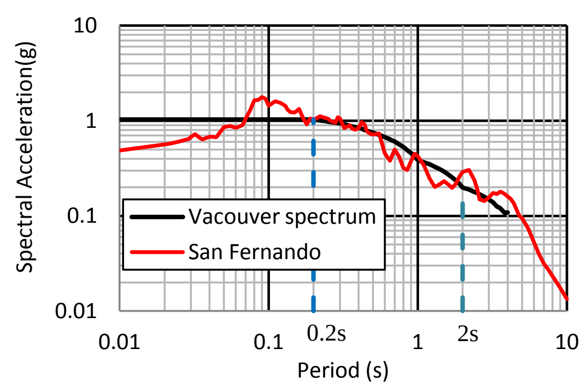


Figure 9: *Response spectrum of Vancouver City and San Fernando earthquake*

Table 3: *Ultimate deformation*

	U_m (mm)	
	MD	LD
Masonry	50	40
Connection	17	11
Wood	70	

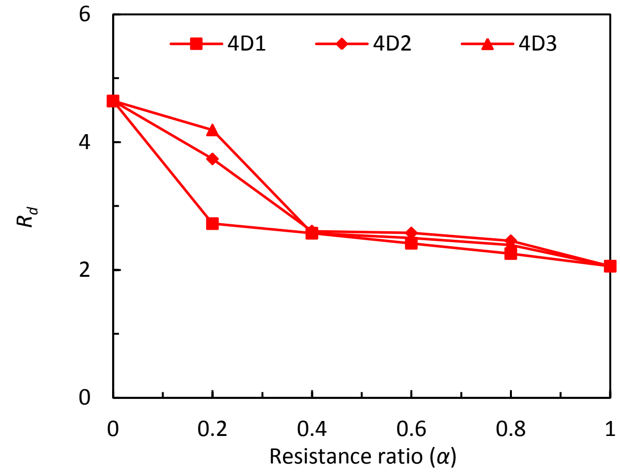
Note: MD represents moderate ductility and LD represents limited ductility.

5 RESULTS

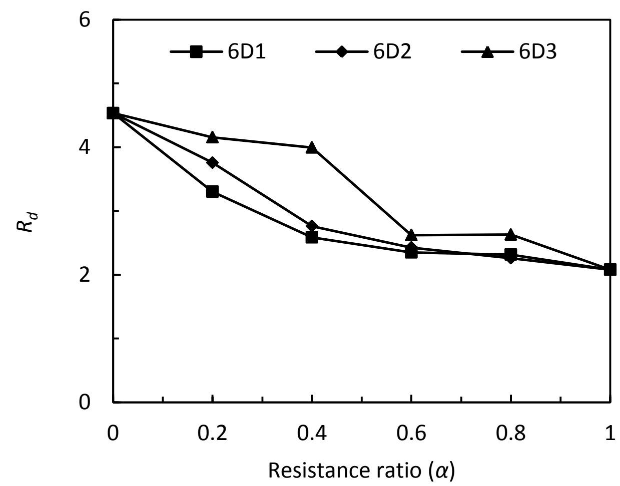
Theoretically, failure can happen firstly at any part of wood, masonry or connection system of hybrid building on any storey. It depends on the relative stiffness and ultimate deformation of the three components and mass distribution along the building height. In hybrid buildings under design method one (D1), the connection system on top storey failed first; after reinforcing the connection system on the upper storeys under design method two (D2), the top storey masonry became the weak point of the hybrid system and in design method three (D3) by reinforcing both the connection and masonry wall, the failure occurred at the bottom storey of the masonry wall. In all of the hybrid buildings under any one of the three design cases, no failure occurred first in the wood sub-system. This is expected as the ultimate deformation of shear wall used in this project is greater than the sum of the ultimate deformation of masonry and connection.

5.1 R_d and R_o VALUES OF MID-RISE HYBRID BUILDINGS

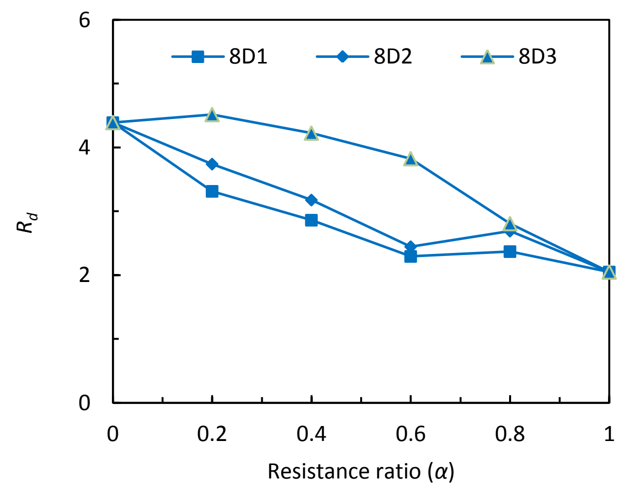
The R_d and R_o values of hybrid building can then be calculated with Eq. (4) and Eq. (5). Fig. 9 and Fig. 10 show the R_d and R_o values of the 67 building cases with different resistance ratio of masonry to hybrid building, design method, building height and ductility level of connection and masonry core. It can be seen that the R_d of hybrid building decreases with increasing of resistance ratio of masonry to hybrid buildings α, while R_o is not sensitivity to α. Both of the R_d and R_o values increase when the design methods changed from D1 to D2, and to D3. This is reasonable as the connection or both of connection and masonry were increased in D2 and D3. However, the amount of increase varies with changes of building height. When the ductility of masonry core is reduced, the relative R_d values are decreased (Fig.10 (e)), while when the ductility of connection is reduced, the R_d values are slightly increased under D1 in which the top storey connection controls the failure mode of hybrid buildings (Fig.10 (d)). This is due to the scaling method of hysteresis loops shown in Fig. 3(a) and (b). Although the ultimate deformation of connection is reduced from 17mm to 11mm, the relative stiffness of the connection is increased at the same time. In a series system consisting of the connection and masonry, less deformation is shared by the connection which leads to an increase of the R_d value. In those buildings designed under D2 and D3 where the masonry core controls the failure mode, the ductility of connection does not affect the R_d values. The R_o values are not sensitive to the ductility level of both connection and masonry core (Fig.11 (d)).



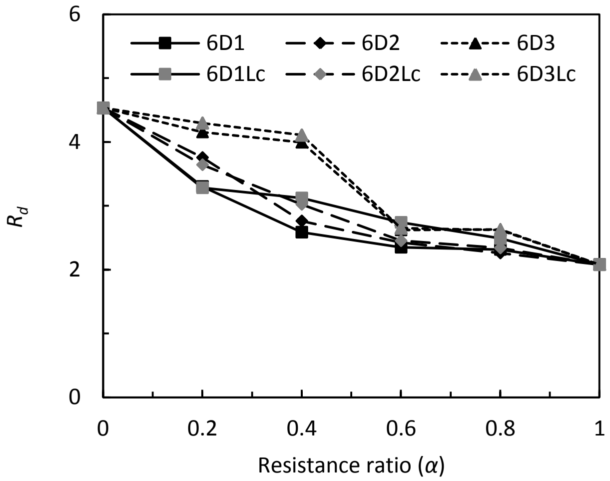
(a) 4-storey buildings



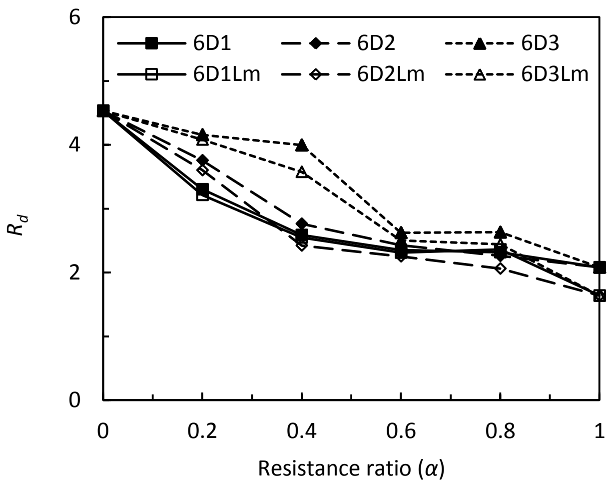
(b) 6-storey buildings



(c) 8-storey buildings

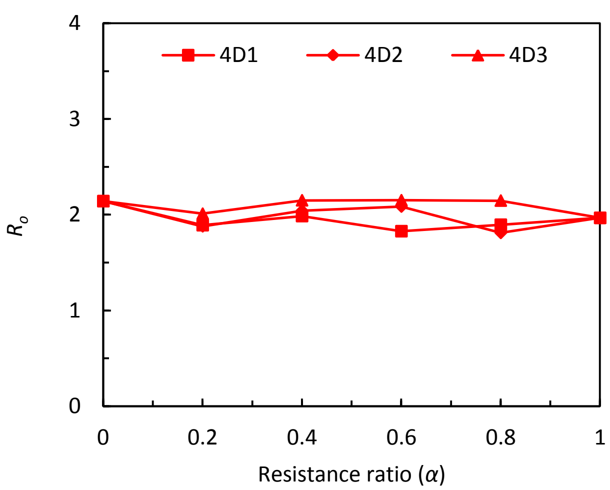


(d) Comparison of ductility of connection

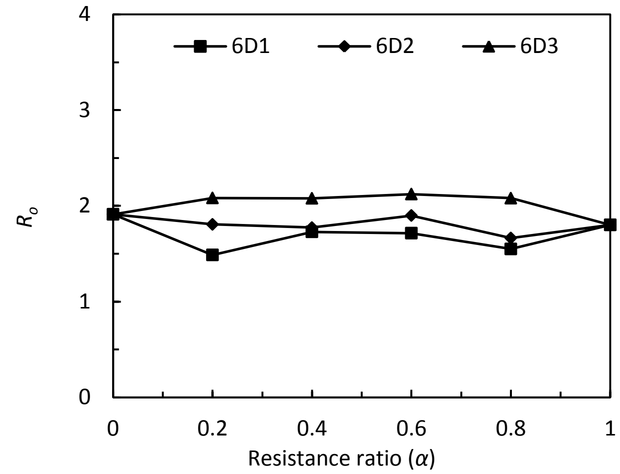


(e) Comparison of ductility of masonry core

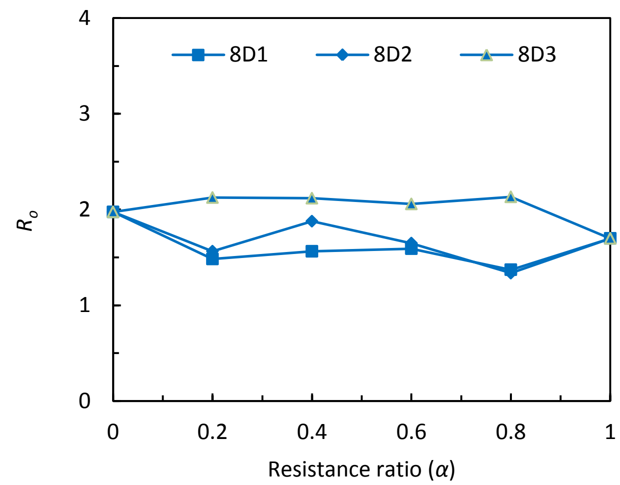
Figure 10: *R_d value of hybrid building*



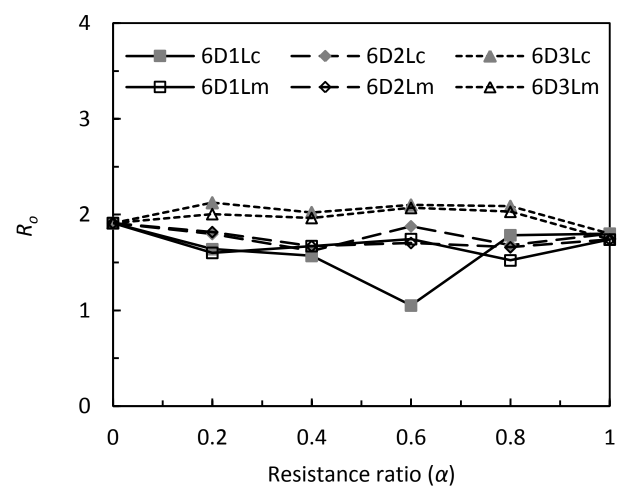
(a) 4-storey buildings



(b) 6-storey buildings



(c) 8-storey buildings



(d) Limited ductility of connection and masonry core

Figure 11: *R_o value of hybrid building*

5.2 EMPIRICAL EQUATION OF R_d, R_o

An empirical equation Eq. (9) is proposed based on the R_d values of 67 buildings derived in this project to estimate the R_d of hybrid building system. This equation demonstrates the relationship between the R_{dH} of hybrid building and R_{dw} of wood, R_{dm} of masonry structure and how the R_{dH} of hybrid building is effected by the resistance ratio of masonry to hybrid building, α; building height (represented by storey number of N) and design method (represented by design method related factor β). The value of β for design method D1, D2 and D3 are listed in Table 4. Fig. 12 shows an example of the fitting curves calculated by Eq. (9) and the R_d values derived from the numerical modelling analysis.

$$R_{dH} = (R_{dw} - R_{dm})(1-\alpha)^{\beta/N} + R_{dm} \tag{9}$$

where R_{dH} is the ductility related force modification factor of hybrid building; R_{dw} is the ductility related force modification factor of wood structure; R_{dm} is the ductility related force modification factor of masonry structure; α is the resistance ratio of masonry core to the whole building, N is story number of building. β is a design method related factor.

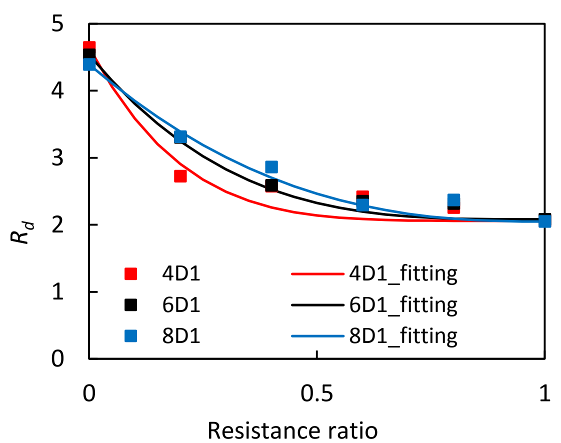


Figure 12: *Comparison of fitting curves and numerical modelling results*

Table 4: *Design method related factor*

	D1	D2	D3
β	20	15	10

According to NBCC [1], the R_{ow} of wood shear wall system is equal to 1.7 and R_{om} of masonry shear wall system with moderate ductility is equal to 1.5. The difference is only 0.2. Since R_o is not sensitive to the resistance ratio of masonry core to hybrid building, α (see Fig. 11), the average value of R_{ow} and R_{om} expressed in Eq. (10) is suggested to estimate the over-strength force modification factor of hybrid building.

$$R_{oH} = (R_{ow} + R_{om})/2 \tag{10}$$

where R_{oH} is the over-strength related force modification factor of hybrid building; R_{ow} is the over-strength related force modification factor of wood structure and R_{om} is the over-strength related force modification factor of masonry structure. So for the hybrid building system studied in this project, the R_{oH} =1.6, which is on the safe side when compared to the R_o values summarised in Table 5 that averages the R_o of six buildings with different resistance ratio of masonry to hybrid building.

Table 5: *R_o value of hybrid building*

	D1	D2	D3
4-storey	2.0	2.0	2.1
6-storey	1.7	1.8	2.0
6-storeyLc	1.6	1.8	2.0
6-storeyLm	1.7	1.8	2.0
8-storey	1.6	1.7	2.0

5.3 FUNDAMENTAL PERIOD OF HYBRID BUILDING T_a

The empirical formula provided in Eq. (6) according to the NBCC 2010 [1] shows that the fundamental period T_a of a shear wall system is only related to its height irrespective of structural material, however, light wood frame building systems usually have longer natural period than masonry with the same building height according to the study in this project (Table 6). A brief discussion about the T_a of hybrid buildings is presented here based on the frequency analysis of 71 buildings. Table 6 lists the T_a value of these buildings. It can be seen that the fundamental period T_a is not sensitive to the design resistance of connection system. This indicates that the T_a of hybrid building is mainly determined by the stiffness and mass of the wood and masonry sub-systems.

Table 6: *Fundamental period (s) of hybrid buildings*

W	H1	H2	H3	H4	M
0.70	0.67	0.64	0.60	0.57	0.50
	0.67	0.63	0.59	0.56	
	0.65	0.60	0.56	0.51	0.45
0.83	0.79	0.75	0.71	0.66	0.59
	0.79	0.74	0.70	0.65	
	0.76	0.70	0.65	0.60	0.54
0.83	0.79	0.75	0.70	0.65	0.59
	0.79	0.74	0.69	0.65	
	0.76	0.70	0.65	0.59	0.54
0.83	0.81	0.78	0.74	0.71	0.65
	0.80	0.77	0.74	0.70	
	0.78	0.73	0.69	0.64	0.59
0.95	0.90	0.85	0.80	0.75	0.67
	0.90	0.84	0.79	0.74	
	0.87	0.80	0.73	0.67	0.61

Note: The T_a value shown in Table 6 is corresponding to the building cases shown in Table 2. Four cases of pure masonry buildings by assigning the base shear on each storey equal to that of the bottom story were added in the sixth column of Table 6 shaded in gray.

Fig. 13 highlights the T_a of analyzed buildings by eliminating the cases designed under D2 or with limited ductility property of connection. The curves shown in Fig.13 indicate an almost linear relationship of T_a and α'. So Eq. (11) is suggested to calculate the fundamental period of hybrid building based on the T_a of its sub-systems.

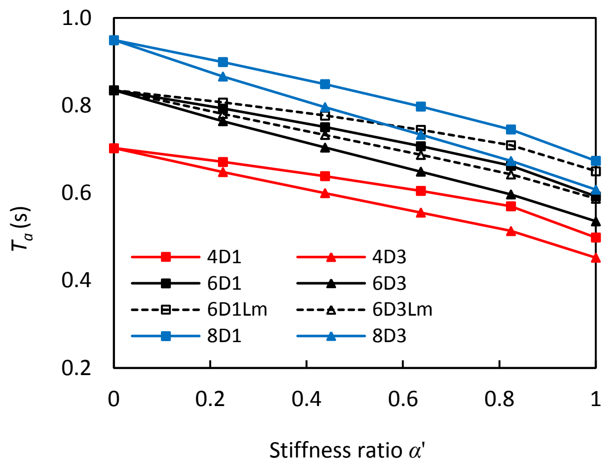


Figure 13: *Fundamental period of buildings*

$$_H = (T_w - \ _m)(1 - \alpha') + \ _m \quad (11)$$

where T_H is the fundamental period of hybrid building; T_w is the fundamental period of wood structure; T_m is the fundamental period of masonry structure and α' is the stiffness ratio of masonry to the whole building.

6 CONCLUSIONS

The seismic performance parameters (seismic force modification factor R (R_d, R_o) and fundamental period T_a) of multi-storey hybrid buildings were investigated in this project through the use of 2-D numerical modelling analyses of hybrid buildings with various sub-system property combinations. The analysis results show that failure mode of hybrid building depends not only on the ultimate deformation of each components but also the relative stiffness of the wood, masonry and connection system. An R value larger than the lowest value of the two sub-systems can be assigned to design of the hybrid building. It varies with different design method, resistance ratio of masonry core to hybrid building and building height. Empirical equations of $R_{d,}$ R_o and T_a of hybrid buildings are proposed based on the $R_{d,}$ R_o and T_a of the sub-systems respectively. The R_d value derived in this project is based on the dynamic analysis under San Fernando earthquake. The absolute value under other earthquakes may be different, while the relationship explored in the proposed empirical equations are expected to have the generality.

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REFERENCES

[1] NRC: National Building Code of Canada 2010. National Research Council (NRC), Ottawa, Canada, 2010.

[2] N.M. Newmark and W.J. Hall: Earthquake spectra and design, Engineering Monograph MNO-q, earthquake Engineering Research Institute, Berkeley, Calif, 1982.

[3] FEMA: Quantification of Building seismic performance factors, P695, Federal Emergency Management Agency, Washington, D.C., 2009.

[4] A. Ceccotti and C. Sandhaas : A proposal for a standard procedure to establish the seismic behaviour factor q of timber buildings. In: *10th World Conference in Timber Engineering*, paper no. 834, 2010.

[5] Y. C. Kurama and K. T. Farrow: Ground motion scaling methods for different site conditions and structure characteristics, Earthquake engineering and structural dynamics, 32: 2425-2450, 2003.

[6] ASTM: Standard test methods for cyclic (Reversed) load test for shear resistance of vertical element of the lateral force resisting systems for buildings, E2126 – 09. American Society for Testing and Materials (ASTM), West Conshohocken, PA, 2009.

[7] CWC: Wood Design Manual, Canadian Wood Council (CWC), Ottawa, ON, Canada, 2010.

[8] J. Xu and J. D. Dolan: Development of a wood-frame shear wall model in ABAQUS. J. Struct. Eng., 135(8): 977-984, 2009.

[9] J. Xu: Development of a general dynamic hysteretic light-frame structure model and study on the torsional behavior of open-front light-frame structures. PhD dissertation, Washington State Univ., Pullman, WA, 2006.

[10] L. Zhou, Y.H. Chui and C. Ni: Hybrid wood-masonry wall test and verification of two-dimensional modelling approach. *Proc., 13th World Conference on Timber Engineering*, Paper #442, Quebec, Canada, 2014.

[11] M. T. Shedid, R. G. Drysdale and W. W. El-Dakhakhni: Behavior of fully grouted reinforced concrete masonry shear walls failing in flexure: experimental results. J. Struct. Eng., 134(11), 1754-1767, 2008.

[12] PEERC: PEER Ground Motion Database. Pacific Earthquake Engineering Research Center, <http://peer.berkeley.edu/peer_ground_motion_database> (Apr. 30, 2013).