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AN APPROACH FOR ESTIMATING SEISMIC FORCE MODIFICATION FACTOR OF HYBRID BUILDING SYSTEMS

Zhiyong Chen[1], Ying H. Chui[2], Ghasan Doudak[3]

ABSTRACT: When hybrid buildings containing more than one type of lateral load resisting system (LLRS) are designed to resist earthquake loads, the National Building Code of Canada requires that the lowest force modification factors of all LLRSs is used for design purposes. An approach is proposed in this paper to calculate a ductility ratio, μ, and ductility-related force modification factor, R_d, for the entire hybrid structure based on strength and ductility ratio or R_d factor of individual types of LLRS. To verify the proposed approach, four-storey buildings with single- and hybrid-LLRS were designed using the proposed approach. These buildings were analysed by conducting a 3-D frequency and non-linear time history response analyses using a finite element program. The results show that using a R_d value higher than the lowest R_d factor suggested by the National Building Code of Canada is justified and that the proposed approach has the potential to be adopted for design purposes.

KEYWORDS: Hybrid structures, Lateral load resisting systems, Seismic response, Force modification factors.

1 INTRODUCTION

In seismic design of buildings using equivalent static force procedure, most national and international building codes specify force modification factors to reflect the energy absorption and ductility characteristics of the lateral load resisting system (LLRS) of the building. For instance, Eurocode 8 [1] and US codes (ASCE 7 [2] and IBC [3]) use the 'q' or 'R' factor, while the National Building Code of Canada (NBCC) [4] specifies the product of two factors, ductility- (R_d) and over-strength (R_o) related force modification factor.

In the NBCC, values for R_o and R_d factors are provided according to structural types for each LLRS [4]. For a hybrid structure consisting of more than one type of LLRS, the NBCC requires that the lowest R_dR_o value of all the LLRSs is used. Although the over-strength related force modification factor R_o factor can be expressed explicitly, all the parameters in the equation were derived statistically exhibiting the difference between design solutions and actual behaviour. As a result, this study focuses on the ductility-related force modification factor, R_d, only. The suggestion to use the lowest R_d factor for hybrid system is conservative and may result in an uneconomical design. The NBCC however, does allow for the use of a more liberal seismic force modification factor if it can be supported by appropriate engineering analyses. A suitable approach is to conduct non-linear time history analysis of a building designed with different R_d values [3-4]. However, this process is time consuming and the results are specific to the particular building under investigation.

Hence, a balanced approach that is more user friendly, while less conservative than the method proposed in the NBCC for estimating R_d factor for hybrid structures is desirable. The purpose of this paper is to present an approach of estimating the overall ductility ratio, μ, and ductility-related force modification factor, R_d, for a multi-storey hybrid buildings consisting of different types of LLRS based on the relevant mechanical characteristics of the individual LLRSs, such as ductility ratio and strength ratio.

2 FORCE MODIFICATION FACTORS IN SOME BUILDING CODES

2.1 EUROCODE 8

In Europe, the design base shear of buildings, V, can be calculated using Equation (1) [1].

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$$V = \frac{V_e}{q} \quad (1)$$

where, V_e is the elastic force obtained from spectra with probability of exceedance of 10% in 50 years (475 years return); and q is the behaviour factor. However, there is no formally recognized procedure for calculating the q factor given in Eurocode 8 [1]. Some researchers have attempted to estimate the q factor by PGA- and base shear-based methods.

For PGA-based method, the behaviour factor q is defined as the ratio of the PGA of the earthquake record that causes near collapse state of the non-linear structural model, $PGA_{u,eff}$, over the design PGA for the location, PGA_{code}, as indicated in Equation (2).

$$q = \frac{PGA_{u,eff}}{PGA_{code}} \quad (2)$$

Regarding the base shear method, the q factor is defined as a ratio of the base shear, V_e, obtained from the linear elastic dynamic analysis of the building model and the base shear, $V_{max,nl}$, from the non-linear analysis of the same model, for any particular earthquake record, as shown in Equation (3).

$$q = \frac{V_e}{V_{\max,nl}} \quad (3)$$

2.2 IBC and ASCE 7

In the USA, the seismic base shear, V, in a given direction shall be determined in accordance with Equation (4) [2-3].

$$V = C_s W \quad (4)$$

where W is the effective seismic weight; and C_s is the seismic response coefficient determined in accordance with Equation (5).

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \quad (5)$$

where S_{DS} is the design spectral response acceleration parameter with probability of exceedance of 2% in 50 years (2,475 years return) in the short period range; R is the response modification factor; I_e is the importance factor.

Determination of R factor for structural system and component are provided by FEMA P695 [5] and P795 [6], respectively. The methodology specified by FEMA P695 directly accounts for the potential variations in structural configuration of buildings, ground motions, and the quality of the available test results for the behavioural characteristics of the structural elements. However, it is a quite complex and time-consuming procedure, since a large number of non-linear dynamic analyses are required on a number of different building models with different configurations.

2.3 NBCC

In Canada, the 2010 edition of NBCC [4] states that the seismic design load, V, is determined by reducing the elastic seismic load, V_e, by ductility- (R_d) and over-strength (R_o) related force modification factor, as indicated by Equation (6).

$$V = \frac{S(T_a) M_v I_e W}{R_d R_o} \quad (6)$$

where $S(T_a)$ is the design spectral acceleration with probability of exceedance of 2% in 50 years (2,475 years return) at selected period; M_v is the higher mode effect factor; W is the seismic weight; I_e is importance factor.

To account for the various components contributing to the over-strength related force modification factor, R_o, the following formulation was used:

$$R_o = R_{size} R_\phi R_{yield} R_{sh} R_{mech} \quad (7)$$

where R_{size} accounts for the over-strength arising from restricted choices for sizes of members and elements and the rounding of sizes and dimensions; R_ϕ is a factor accounting for the difference between nominal and factored resistances, equal to $1/\phi$, where ϕ is the material resistance factor as defined in the CSA standards; R_{yield} is the ratio of "actual" yield strength to minimum specified yield strength; R_{sh} accounts for the over-strength due to the development of strain hardening; and R_{mech} is the over-strength arising from mobilizing the full capacity of the structure such that a collapse mechanism is formed. Although the R_o factor can be expressed by Eq. (7) explicitly, all the parameters in the equation were derived statistically exhibiting the difference between the actual situation and the structural design.

The ductility-related force modification factor, R_d, corresponds to the R factor in 1995 NBCC. It is directly related to structural ductility, and is dependent on the building period. However, no official guidelines on how to calculate this factor are provided. Newmark and Hall derived a relationship between the ductility ratio, μ, and ductility-related force modification factor, R_d, according to the period (T) of a structure [7]. For short period structures

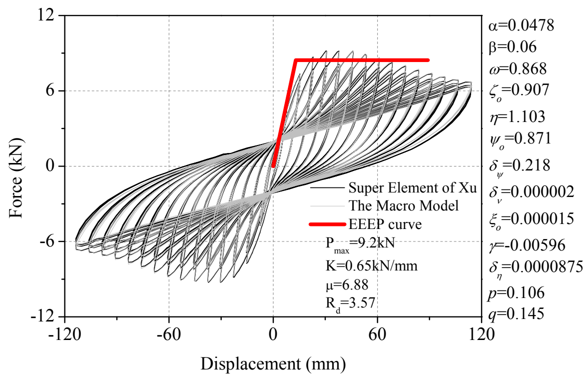
(T ≈ 0.1s to 0.5s), the equal energy principle applies, Equation (8).

$$R_d = \sqrt{2\mu - 1} \tag{8}$$

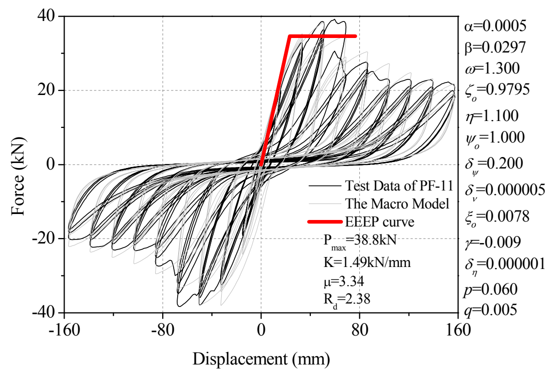
For medium and long period structures (T>0.5s) equal displacement rule applies, Equation (9).

$$R_d = \mu \tag{9}$$

Therefore, the R_d factor can be estimated as long as the ductility ratio is determined. The measured ductility, μ, is expressed as the ratio of the displacement at failure over that at yield, determined using the Equivalent Energy Elastic-Plastic (EEEP) approach [8]. Figure 1 illustrates two example of estimation of R_d. It shows the hysteresis loops and the EEEP curves of a 1220mm × 2440mm shear wall of specimen 04FAc3 in [9] and a wood portal frame (PF) specimen PF-11 in [10].



(a) Shear wall specimen 04FAc3 from [9]



(b) Portal frame specimen PF-11 from [10]

Figure 1: *Hysteresis loops of wood shear wall and portal frame and calculation of R_d*

3 PROPOSED METHOD FOR ESTIMATING R_d OF HYBRID SYSTEM

As it can be seen from Equations (8) and (9), the R_d factor is determined from the ductility ratio of the system. Therefore, establishing the relationship of ductility between the hybrid system and the sub-systems is the bridge to develop an approach to estimate the R_d factor of hybrid systems.

3.1 THEORETICAL DERIVATION OF SYSTEM μ

Figure 2 shows a hybrid structural system composed of two sub-systems with different performances, in terms of stiffness, load-carrying capacity and ductility ratio, and rigid diaphragm (in plane stiffness K_D is taken as infinite). The idealised load-displacement curves of these sub-systems under lateral load are illustrated in Figure 3.

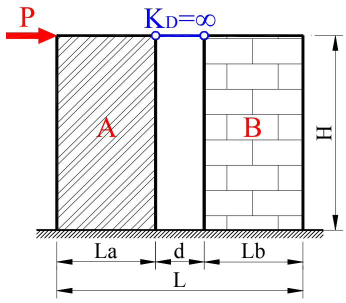


Figure 2: *Hybrid structural system*

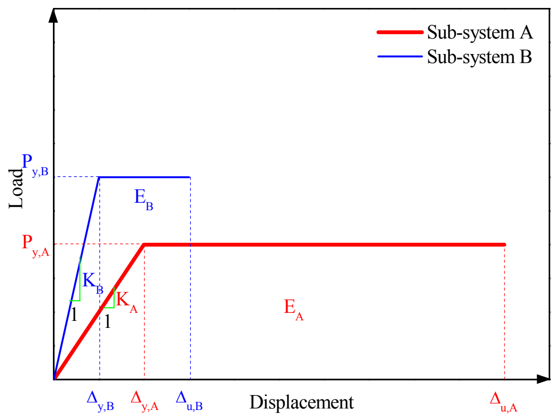


Figure 3: *EEEP curves of the two sub-systems*

Note: the K_i, $P_{y,i}$, $\Delta_{y,i}$, $\Delta_{u,i}$ and E_i are the stiffness, yield strength, yield displacement, failure displacement and energy consumption of the ith sub-system.

Since the diaphragm of the hybrid system is idealised as 'rigid', the lateral load, P, is distributed to the two sub-systems in proportion to their stiffness, and therefore, both sub-systems deform by the same amount [10, 11]. When one sub-system yields, it just can resist the yield load, while the other sub-system carries the remaining load until it yields. After that, the hybrid system yields completely until it reaches failure. The load-displacement curve of the hybrid system can be obtained by summing the curves of sub-systems, as shown in Figure 4.

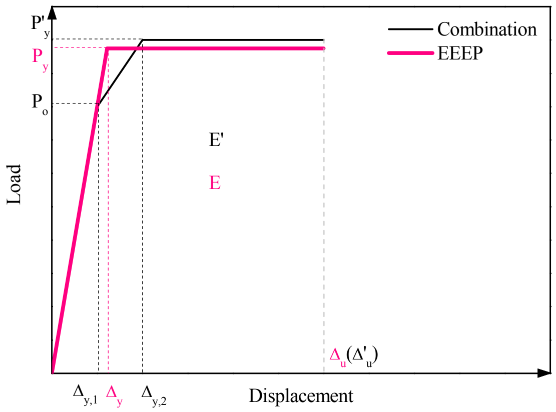


***Figure 4:** Combination EEEP curves of hybrid system*

The stiffness of the hybrid system, K, can be calculated using Equation (10).

$$K = K_A + K_B \tag{10}$$

The loads of two yield points, P_o and P_y' of the combination curve can be derived by Equations (11) and (12).

$$P_o = \frac{\Delta_{y,B}}{\Delta_{y,A}} P_{y,A} + P_{y,B} \tag{11}$$

$$P_y' = P_{y,A} + P_{y,B} \tag{12}$$

Whereas the displacement of the failure point for the combination curve can be derived by Equation (13), based on equal energy consumption, $E' = E_A + E_B$.

$$\Delta_u = \alpha_{F,A}\Delta_{u,A} + \alpha_{F,B}\Delta_{u,B} \tag{13}$$

where $\alpha_{F,i}$ is strength ratio, and is taken as $P_{y,i} \Big/ \sum_{i=A}^{B} P_{y,i}$.

The EEEP curve of the combination curve of the hybrid system is shown in Figure 4. These two curves have the same failure displacement ($\Delta_u = \Delta_u'$). In view of equal energy consumption, $E = E'$, the yield displacement, Δ_y, and load, P_y, of the EEEP curve can be calculated by Equation (14).

$$\Delta_y = \Delta_u - \sqrt{\Delta_u^2 - 2\sum_{i=A}^{B}\left(\alpha_{K,i}\frac{\Delta_{u,i}^2}{\mu_i}\right) + \sum_{i=A}^{B}\left(\alpha_{K,i}\frac{\Delta_{u,i}^2}{\mu_i^2}\right)} \tag{14}$$

$$P_y = K\Delta_y \tag{15}$$

where $\alpha_{K,i}$ is strength ratio, and is taken as $K_i \Big/ \sum_{i=A}^{B} K_i$.

Now, the ductility of the hybrid system can be derived by Equation (16).

$$\mu = \Delta_u / \Delta_y \tag{16}$$

From Equation (16) it can be seen that the ductility of the whole system is a function of stiffness ratio, $\alpha_{K,i}$, strength ratio, $\alpha_{F,i}$, ductility ratio, μ_i, and failure displacement, $\Delta_{u,i}$, of the sub-systems. Even though the relationship among the R_d factor of the hybrid system, and the stiffness ratio, $\alpha_{K,i}$, the strength ratio, $\alpha_{F,i}$, the $R_{d,i}$ factor, and the failure displacement, $\Delta_{u,i}$, of sub-systems can be derived by substituting Equation (8) or (9) into (16), it seems complicated and tedious for structural design. On the other hand, since the EEEP curve is an idealized elasto-plastic model for the envelope curve of the hysteresis loops of a structural system, the accuracy of the calculated ductility ratio, μ_i, or the R_d factor of the hybrid system is reduced.

3.2 NUMERICAL SIMULATION OF SYSTEM μ

In an attempt to derive a simpler approach to estimate the ductility ratio, μ_i, or the R_d factor of the hybrid system, a numerical investigation using finite element analysis (FEA) method was performed.

3.2.1 FEA Model

The hybrid building with rigid horizontal diaphragm (Figure 2) was analyzed under a concentrated cyclic load, P. This system-level modelling approach simulated the responses of the hybrid building using two-dimensional planar analyses, as illustrated in Figure 5.

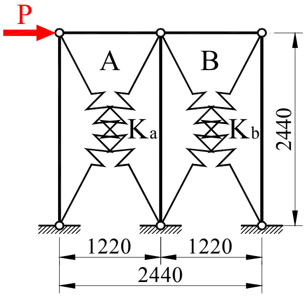


***Figure 5:** FEA models of hybrid structural systems*

The two sub-systems were simulated by the macro-element model of lateral load resisting element (LLRE) [13] within the ABAQUS [14] FEA software. This macro-element model (Figure 6) is composed of three truss elements (T2D2) with large sectional properties to represent member rigidity, and two user elements (UEL) of the modified Bouc-Wen-Barber-Noori (BWBN) spring [13].

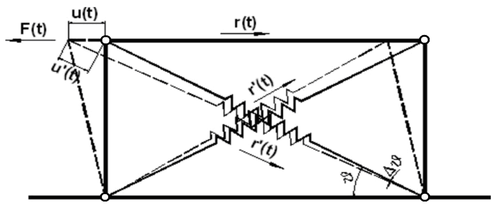


***Figure 6:** Macro model for lateral load resisting element*

In common cases, a hybrid structural system is developed by linking a stiffener or/and high-strength secondary sub-system which is usually less ductile to a ductile primary sub-system. These hybrid systems could include light-wood shear wall sub-system plus portal frame sub-system [15], masonry wall sub-system [16], or concrete shear wall sub-system. In this study, a high ductility (HD) and a low ductility (LD) sub-system were considered. The properties of the HD system were represented by the traditional wood shear wall, Figure 1. In order to evaluate the influence of individual element ductility on the resulting system ductility ratio, three artificial hysteresis loops of LD with different stiffness, maximum resistance and ductility ratio were artificially adjusted to produce three different load-deformation responses with different ductility ratios by those of wood shear wall system. In an attempt to achieve different strength ratios, three types of LD system were used. The structural performance parameters of these systems are listed in Table 1.

***Table 1:** Structural performance parameters*

No.	HD	LD_I	LD_II	LD_III
K (kN/mm)	0.65	0.86	0.65	0.36
P_y (kN)	8.4	9.3	7.4	4.4
Δ_y (mm)	13.1	10.6	11.3	12.1
P_{max} (kN)	9.2	10.4	8.2	4.9
Δ_u (mm)	89.0	17.0	28.3	43.6
μ	6.7	1.6	2.5	3.6

Six combination cases of HD and LD for the hybrid building were analyzed to investigate the effect of strength ratio, $\alpha_{F,LD}$, of LD relative to the total strength of LLRS (0.0, 0.2 … 1.0) on the system ductility ratio (μ).

3.2.2 Ductility Ratio of Hybrid Systems

The system ductility ratio, μ, was determined by subjecting the hybrid building to a reversed cyclic load, P, as shown in Figure 5. The ISO 16670 loading protocol (ASTM 2009) with one time ultimate displacement of the 1220 × 2440 mm wood shear wall [9] was used. From the skeleton curves of the resulting hysteresis loops the system μ was calculated using the EEEP approach [8]. The system ductility ratio, μ, of the hybrid building with different $\alpha_{F,LD}$ and ductility ratio of LD element, μ_{LD}, is shown in Figure 7.

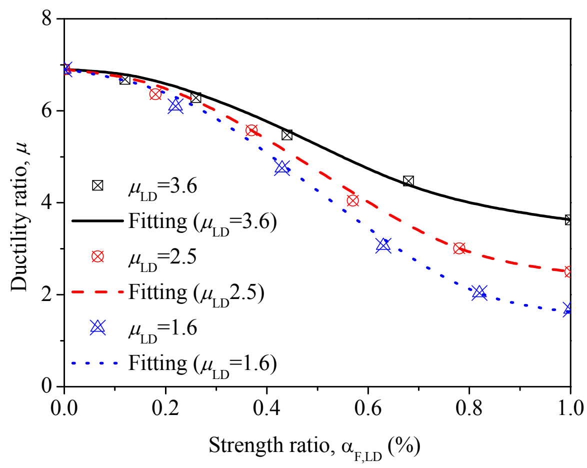


***Figure 7:** System μ of hybrid structural systems with different strength ratios*

The μ of the hybrid system decreases with increasing the strength ratio of the LD element to the LLRS. By fitting the FEA results using the least-squares method, Equation (17) was obtained to estimate the system μ of the hybrid building.

$$\mu = \mu_{LD} \sin^2\left(\alpha_{F,LD}\frac{\pi}{2}\right) + \mu_{HD}\cos^2\left(\alpha_{F,LD}\frac{\pi}{2}\right) \quad (17)$$

where μ_i is the ductility ratio of LD or HD element.

As shown in Equation (17), only the μ_i of individual LLREs and the strength ratio affect the system μ of the hybrid building. It is simpler than Equation (14).

3.3 SYSTEM R_d OF HYBRID BUILDINGS

According to the research by Newmark and Hall [7] the relationship between the ductility-related force modification factor, R_d, and the ductility ratio, μ, depends on the building period, as shown in Equations (8) and (9). However, this is problematic as buildings which have the same type of LLRS but different periods should be assigned different R_d factors. In NBCC [4], the same R_d factor is assigned to the buildings containing the same type of LLRS for any natural periods. Though a more conservative R_d would be obtained for building period greater than 0.5s, Equation (8) is consistent with the approach in the NBCC. Therefore, an equation, Equation (18), for estimating the system R_d factor of single-storey hybrid buildings was obtained via substituting Equation (8) into Equation (17).

$$R_d = \sqrt{R_{d,LD}^2 \sin^2\left(\alpha_{F,LD}\frac{\pi}{2}\right) + R_{d,HD}^2\cos^2\left(\alpha_{F,LD}\frac{\pi}{2}\right)} \quad (18)$$

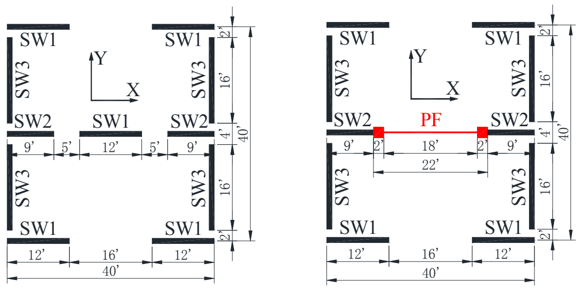
where $R_{d,i}$ is the ductility-related force modification factor of individual LLRE type. Similar to the system ductility ratio of hybrid buildings, only R_d of the individual LLREs and the strength ratio affect R_d of single-storey hybrid buildings, as shown in Equation (18). The preliminary results shown here point to the feasibility of relating the system R_d to sub-system R_d, the strength ratios.

4 RESPONSE OF HYBRID BUILDING SYSTEMS TO SEISMIC MOTIONS

4.1 STRUCTURAL DESIGN

In order to investigate the structural behaviour of hybrid buildings under earthquakes and to assess the equations for estimating R_d factor, two multi-storey LWFBs with different layouts were designed to NBCC [4] structural requirements applicable to Vancouver (PGA = 0.46g) with stiff soil condition (Site Class D).

The buildings were 4-storey high with a storey height of 2.44m, totalling 9.76 m for the four storeys. The plane dimensions of the building were 12.2m × 12.2 m (40 ft × 40 ft), as shown in Figure 8. A reference building with only one type of LLRE of HD (shear wall, SW), plus a hybrid building consisting of LD (portal frame, PF) and HD (shear wall, SW) sub-systems, were included, Figure 9. The seismic weight of the floor and roof was 1.8 and 1.36 kPa, respectively [17].



(a) Reference building *(b) Hybrid building*

***Figure 8:** Layouts of buildings*

Equivalent Static Force Procedure [4] was utilized in the seismic design. In the reference case, the building was constructed with HD (shear wall) elements only, with R_d = 3.57 which was calculated using Equation (8). In the hybrid buildings, LD (portal frame) and HD (shear wall) elements were combined in the same system. Based on the R_d of HD (3.57) and LD (2.38) elements, shown in Figure 1, and the strength ratio, the storey R_d factors were calculated using Equation (18) and are given in Table 2. The lowest storey R_d factor was taken as the system R_d factor for the multi-storey hybrid building (Table 2). The same R_o factor of 1.7 was used for the buildings of all cases.

***Table 2:** Structural design matrix of reference and hybrid buildings*

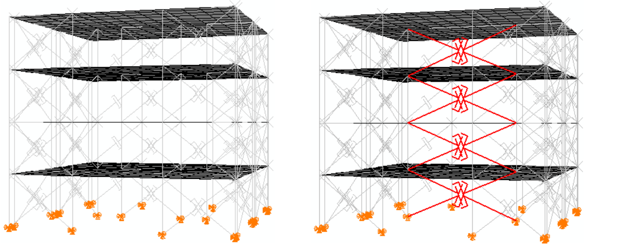
No.		Reference				Hybrid **			
		1st	2nd	3rd	4th	1st	2nd	3rd	4th
*Nail spacing (mm)**		100	100	150	150	100 (75)	100 (75)	150 (100)	150 (150)
HD / LD		H	H	H	H	H&L	H&L	H&L	H&L
R_d	Storey	3.57	3.57	3.57	3.57	3.49 (2.38)	3.49 (2.38)	3.43 (2.38)	3.43 (2.38)
	System	3.57				3.43 (2.38)			

Note: * - One OSB with thickness of 12.0 mm was used in shear wall; ** - the values listed in parentheses and highlighted in red are the design requirements in accordance with NBCC using the lower R_oR_d of the sub-systems for the hybrid system.

The natural period used for designing the base shear calculation was assumed to be twice the period determined (0.62 = 0.31 × 2) in accordance with Clause 4.1.8.11.(3)(c) of NBCC [4], since the calculated periods of the two buildings using FEA were greater than twice the empirically calculated periods. The lateral forces were distributed to each wall based on tributary area. The HD shear walls (Figure 8) were designed based on the shear resistance values in CSA O86 [18]. The LD portal frame was assumed to have a shear resistance of 26.25 kN, which was determined in accordance with AC130 [19] based on test data. The criterion of the maximum inter-storey drift at any storey was 2.5% (1/40) of storey height in the drift check. Shear walls of this building were designed with 11 mm OSB and 38 mm × 89 mm spruce-pine-fir stud grade lumber spaced 406 mm on center. The intermediate nail spacing of all the shear walls was the same, 300 mm. The edge spacing of the shear walls was identical at the same storey but varied from storey to storey, Table 2.

4.2 NUMERICAL SIMULATION

Two three-dimensional FEA models of LWFBs, Figure 9 were developed using ABAQUS [14].



(a) Reference building *(b) Hybrid building*

Figure 9: *FEA models*

Note: the highlighted dashpots belong to the portal frames.

As shown in Figure 9, the HD and LD elements were simulated by the modified macro-element model illustrated in Figure 6, similar to the 2D FE models of hybrid buildings for investigating the system ductility ratio. This macro model is composed of three truss elements (T3D2) with large sectional properties to represent member rigidity, and two user-defined elements (UEL) of the modified BWBN spring. Two dashpot elements (DASHPOTA) with a damping ratio of 1% [11] were placed in each macro-element model to take the elastic damping effect into account.

The hysteresis loops of HD elements (shear walls, Figure 1a) in the design buildings were obtained by scaling the load value of the hysteresis loops of the 1220 × 2440 mm shear wall (Figure 1) based on the ratio of shear wall design values given in Wood Design Manual [20] to that of the reference shear wall. The loops given for the portal frame shown in Figure 1b were used for the LD element. A pair of dash pot elements (DASHPOTA), as shown in Figure 9, with a damping ratio of 1% [13] were placed in each LLRE to account for the elastic damping effect.

The horizontal diaphragm was modelled using the shell element (S4R) with a thickness of 235 mm. To achieve a rigid diaphragm in plane, 10 GPa was taken as the modulus of elasticity for diaphragms. The Rayleigh damping with equivalent viscous damping ratio of 8.3% [21] was used for the diaphragms of the FE model.

The shear walls and portal frames were connected to the diaphragm through the horizontal framing member of the macro-element model (Figure 4), and the slip between the shear walls and diaphragm was negligible. The mass was uniformly distributed in the floor and roof diaphragms.

4.3 FUNDAMENTAL NATURAL PERIOD

The undamped fundamental natural periods of the reference and hybrid buildings were 0.70 s and 0.72 s, respectively, obtained by conducting frequency analysis using the LANCZOS technique [14]. The natural period of the hybrid building is generally about 3% longer than the corresponding reference building. This is obviously due to the substitution of one HD element with a LD element of which the stiffness is lower than that of some of the replaced HD elements in the hybrid building.

4.4 SEISMIC RESPONSE

The seismic behaviour of the two building models under the design hazard level was analyzed with the implicit dynamic analysis method using direct integration [14]. Twenty-two "Far-Field" earthquake records, which were obtained from sites located greater than or equal to 10 km from fault rupture [6], in the fault normal (FN) direction were scaled at the corresponding fundamental period of each building models to match the spectral acceleration, S_a, of the Vancouver design spectrum, as shown in Figure 10. Since this building has similar structural performance in the two directions, only the lateral load resisting system in the X direction was analysed.

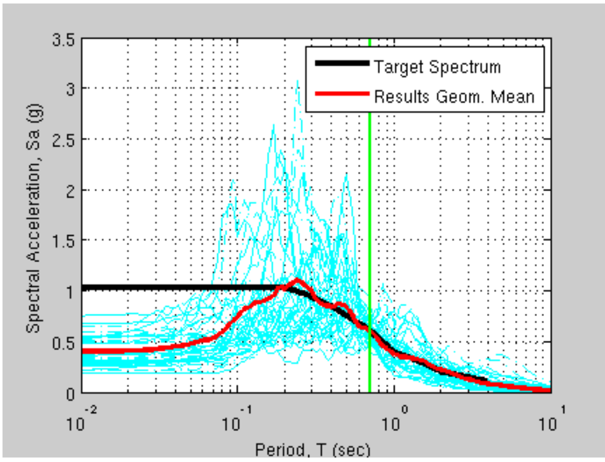


Figure 10: *Scaling earthquake records (T_o = 0.70 s)*

Altogether, 44 non-linear time history analyses were conducted and the relationship between the cumulative distribution function (CDF) and the maximum drift ratio is shown in Figure 11.

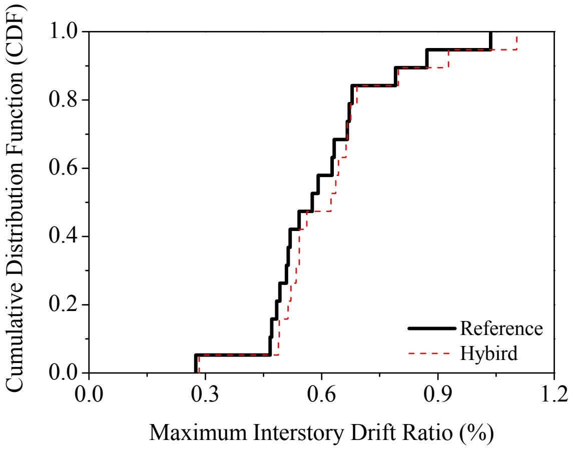


***Figure 11:** CDF's of maximum inter-storey drift ratio*

Note: "Reference" and "Hybrid" indicate building with single- and hybrid-LLRS, respectively.

As shown in Figure 11, all the maximum drift ratios of the two building models are less than the design criterion of 2.5%. It means that the design of the building models fulfilled the seismic design requirement of NBCC [4]. Meanwhile, it should be noted that the buildings were designed with a building period of 2T in accordance with Clause 4.1.8.11 of NBCC. Because of this, the responses of LLRS's were in the ranges of elasticity and yield with minimal ductility. There was no distinguishable difference in drift ratios among the reference and hybrid cases. This indicates that the seismic response of the hybrid LWFB including shear walls and portal frames designed with the R_d estimated from the proposed approach is nearly the same as the reference LWFB containing shear wall only, at the stages of elasticity and yield with some ductility. Hence the approach for estimating the system R_d factor is appropriate.

5 CONCLUSIONS

- A mechanics-based model for estimating the system ductility ratio of hybrid systems is obtained. The system ductility ratio of hybrid system can be interpreted as a function of stiffness ratio, strength ratio, ductility ratio and failure displacement of sub-systems.

- An approach for estimating the ductility ratio, μ, and the ductility-related force modification factor, R_d, of hybrid buildings containing two types of LLRS is proposed. Empirical models relating the strength and ductility ratio or ductility-related force modification factor of the individual LLRS are provided.

- A reference and hybrid models were designed and analysed to evaluate the estimated R_d from the proposed approach. Twenty-two non-linear time history analyses, without considering the torsional effects, were conducted using ABAQUS with a macro-element model of LLRE. According to the FEA results, the dynamic characteristic and seismic response of the hybrid LWFB including shear walls and portal frames designed with the R_d estimated from the proposed approach were nearly the same as the LWFB including shear wall only, and hence the replacement of shear wall with portal frame and the proposed approach for estimating the system R_d factor are appropriate.

- The results show that the current NBCC approach for assigning R_d for hybrid building is conservative and that the proposed method of estimating system R_d for a hybrid building leads to designs that provide comparable seismic performance of a similar building constructed with one material and designed according to current NBCC seismic provisions.

For further investigation, a more comprehensive analysis on hybrid buildings incorporating varying types of LLRS's will be performed. This method still needs to be investigated further for applications that involve hybrid buildings incorporating other types of LLRS's. Similar method to estimate the system ductility ratio and R_d factor of multi-storey hybrid buildings will be investigated.

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REFERENCES

[1] CEN: EN 1998-1:2004/A1:2013 Eurocode 8: Design of Structures for Earthquake Resistance – Part 1: General Rules, Seismic Actions and Rules for Buildings. European Committee for Standardization (CEN), Brussels, 2013.

[2] ASCE: Minimum design loads for buildings and other structures, ASCE 07-10. American Society of Civil Engineers (ASCE), Reston, 2010.

[3] ICC: 2009 International Building Code (IBC). International Code Council (ICC), 2009.

[4] NRC: National Building Code of Canada. Canadian Commission on Building and Fire Codes and National Research Council of Canada (NRC), Ottawa, 2010.

[5] Applied Technology Council (ATC): Quantification of building seismic performance factors. ATC-63/FEMA P–695 Project Rep., 2009.

[6] ATC: Quantification of building seismic performance factors. ATC-63/FEMA P–795 Project Rep., 2011.
[7] Newmark N.M., and Hall W.J.: Earthquake spectra and design. Earthquake Engineering Research Institute, 1982.
[8] ASTM: E2126 Standard test methods for cyclic (reversed) load test for shear resistance of vertical elements of the lateral force resisting systems for buildings. ASTM, West Conshohocken, 2009.
[9] A. J. Salenikovich: The racking performance of light-frame shear walls. Ph.D Dissertation, Virginia Polytechnic Institute and State University, Blacksburg, 2000.
[10] C. Ni, and M. Mohammad: Evaluation of lateral load prescriptive details of small wood buildings. Report prepared for Canadian Forest Service, Natural Resources Canada, Ottawa, 2011.
[11] Z. Y. Chen, Y. H. Chui, C. Ni, G. Doudak, and M. Mohammad: Load distribution in timber structures consisting of multiple lateral load resisting elements with different stiffnesses. J. Perform. Constr. Facil., ASCE,10.1061/(ASCE)CF.1943-5509.0000587, 2014.
[12] Z. Y. Chen, Y. H. Chui, M. Mohammad, G. Doudak, and C. Ni: Load distribution in lateral load resisting elements of timber structures. In: *13th World Conference on Timber Engineering*, PAP463, 2014.
[13] J. Xu, and J. D. Dolan: Development of a wood-frame shear wall model in ABAQUS. J. Struct. Eng., ASCE, 10.1061/(ASCE)ST.1943-541X.0000031, 977-984, 2009.
[14] ABAQUS: ABAQUS analysis user's manual (Version 6.11). Hibbitt, Karlsson, and Sorenson, Pawtucket, 2011.
[15] Z. Y. Chen, Y. H. Chui, C. Ni, and J. Xu: Seismic response of midrise light wood-frame buildings with portal frames. J. Struct. Eng., 10.1061/(ASCE)ST.1943-541X.0000882 , A4013003, 2013.
[16] L. Zhou, C. Ni, Y. H. Chui, and Z. Y. Chen. Seismic performance of hybrid building consisting of light wood frame and reinforced masonry core. J. Perform. Constr. Facil., ASCE. DOI:10.1061/(ASCE)CF.1943-5509.0000597, 2014.
[17] APEGBC: Structural, fire protection and building envelope professional engineering services for 5 and 6 storey wood frame residential building projects (Mid-Rise Buildings). Association of Professional Engineers and Geoscientists of BC (APEGBC) Technical and Practice Bulletin, Burnaby, 2009.
[18] CSA: CSA O86-09: Engineering design in wood, Canadian Standards Association (CSA), Toronto, 2009.
[19] ICC-ES: *AC130: Acceptance criteria for prefabricated wood shear panels*. International Code Council – Evaluation Service (ICC-ES), Whittier, 2009.
[20] Canadian Wood Council (CWC): Wood design manual 2010. Eton System, Nepean, 2010.
[21] Z. Y. Chen, Y. H. Chui, G. Doudak, C. Ni, and M. Mohammad: Simulation of the lateral drift of multi-storey light wood frame buildings based on a modified macro-element model. In: *13th World Conference on Timber Engineering*, PAP195, 2014.