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ENERGY BASED SEISMIC DESIGN OF A TIMBER CORE-WALL MULTI-STOREY HYBRID BUILDING

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Abstract

Current earthquake design philosophy in North America recommends an equivalent static force procedure (ESFP). Much research lately has been in new performance based methodologies including direct displacement based design (DDBD) and energy-based design (EBD). Research in energy-based design has not had the attention of DDBD yet now is gaining in popularity because of the methods reliance on the velocity spectrum and duration of earthquake hazard. This paper discusses an energy based methodology in designing a novel multi-storey hybrid building consisting of a timber-steel core wall system. This hybrid system combines Cross Laminated Timber (CLT) panels with steel plates and connections to provide the required strength and ductility to core walled buildings.

To improve the applicability of the hybrid system an EBD methodology is proposed to design the core-walled building. The methodology is proposed as it does not rely on empirical formulas and force modification factors to determine the final design of the structure. In order to assess the feasibility of the EBD method, it is implemented in the design of a 7-storey building based off an already built concrete benchmark building. The design is first carried out following the ESFP outlined by the National Building Code of Canada for Vancouver, BC. Nonlinear time history analysis is carried out on the ESFP design and the proposed EBD methodology using 10 ground motions selected at 2% in 50 years return period, to evaluate the suitability of the method and the results of the ESFP and EBD methodologies are discussed and compared.

Keywords: Energy Based Design, Cross Laminated Timber, Timber-Steel Hybrid Building, Performance Based Design

1. Introduction

The problem in current seismic design philosophy has been shown by past earthquakes unexpected damages, economic loss, and costly repairs of appropriately detailed and designed buildings [1, 2]. These factors need to be addressed so that future earthquakes do not result in similar outcomes; however, current design methodologies do not account for these factors due to the simplified code regulations. Current earthquake design philosophy in North America recommends an equivalent static force procedure (ESFP). The ESFP relies on spectral acceleration response spectrums to calculate the base shear required by the structure to resist. The building codes addressed life safety in seismic design controlling the damage for small and moderate earthquakes and collapse prevention in major earthquakes. Performance of the building is defined by the displacement and interstorey drift that the structure would see during the earthquake events.

To address the current seismic design problems, research has focused on specifying the desired performance of the structure and determining acceptable loss and damages an earthquake event would cause. This method was termed performance based seismic design (PBSD). PBSD allows the designer to determine the design reliability to achieve the design objectives [1]. Several fundamental reports contributed to the development of PBSD [4-7]. Although structural damage from an earthquake to a structure is caused solely by deflections, energy based methods allow for the assessment of the energy dissipated and absorbed from the earthquake event. This assessment is helpful in designing the building by balancing the energy seen by the structure [8]. In order to understand the demands an earthquake puts on a structure it is important to consider the duration effect as energy is a cumulative measure of ground shaking. By considering duration the fault type can be included in determining the seismic forces including near-fault ground motions.

Energy based design involves two key aspects, the first being establishing design earthquakes and the second being determining the actual absorption and dissipation capacity of the structure [9]. The concept of energy can be derived from the equation of motion:

$$m\ddot{u} + c\dot{u} + ku = -m\ddot{u}_g \tag{1}$$

where m is the mass of the system, c is the viscous damping coefficient, k is the stiffness, u is the displacement and u_g is the ground acceleration. By integrating each term with respect to the relative displacement, u, in the equation of motion the energy equation can be derived as:

$$\int m\ddot{u}\,du + \int c\dot{u}\,du + \int ku\,du = -\int m\ddot{u}_g\,du \tag{2}$$

The three terms on the left side of Eq. (2) are related to the structural characteristics and represent the stored kinetic energy (E_K), dissipated energy through damping (E_D), and absorbed energy (E_A), respectively. The absorbed energy; however, can be further separated into strain energy (E_S) and hysteretic energy dissipation (E_H). Strain energy represents the recoverable energy that the structure can withstand whereas the hysteretic energy dissipation represents the irrecoverable hysteretic energy which causes the damage to the structure. These energies, when summed equate to the total input energy subjected by the earthquake (E_I). Therefore, the energy equation can be simplified as:

$$E_K + E_D + E_S + E_H = E_I \tag{3}$$

Researchers have developed various design methodologies using the energy principles [10-13]. The fundamental concept in EBD is the energy-balance concept as shown in Fig.1. This figure shows that the elastic and plastic energy when summed should equate to the input energy of an equivalent elastic system at the maximum target displacement [14]. These studies displayed that the target performance was met when the designs were subjected to analytical design earthquakes.

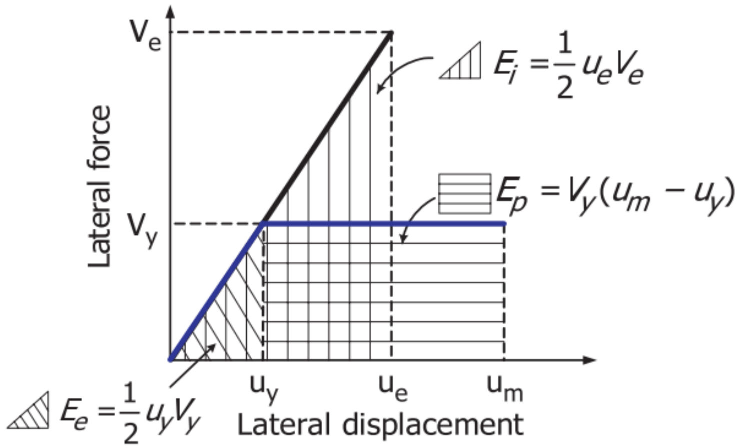

Fig. 1 – Energy balance concept with force-displacement relationship [14]

2. Timber-steel core walls

Tall wood buildings have recently become popular in Canada with the introduction of mass timber products [15]. Mid-rise timber buildings are now becoming a viable option for developers and designers in British Columbia after the 2009 BC Building Code increased the height limitation on wood-frame structures from four to six. Mass timber construction consists of smaller timber beams glued together to create a large beam or panel capable of resisting higher loads. As western Canada is densely populated with forests, mass timber products are an economic choice for builders in this region. Advantages of mass timber buildings when designed and detailed properly include: a reduced carbon footprint, constructability, aesthetics and construction time. Mass timber products are stiff and light and therefore can often result in brittle buildings with high strength and low ductility. High seismic regions require a ductile structure in order to reduce shear force demand on a building by allowing the building to deform during the excitation. To overcome these problems the proposed system utilizes CLT panels with steel plates and ductile steel connectors that transfer the seismic load from the CLT diaphragm to the foundation (Fig.2a).

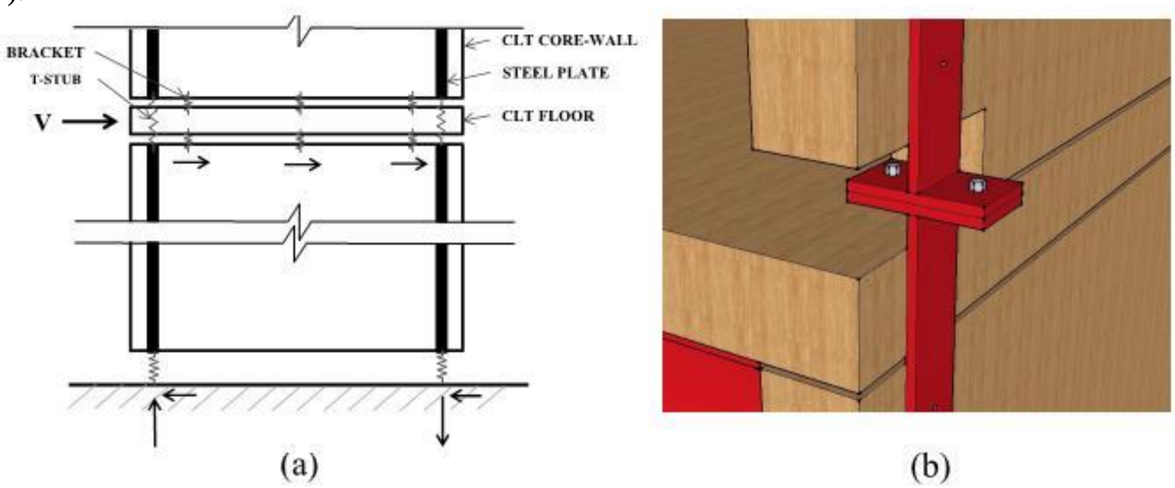

Fig. 2 – Timber-steel core system: a) layout; b) 3D rendering of t-stub connection detail

Steel t-stub connections (Fig.2b) provide the ductility to the system by plastically deforming at the base of each floor. The plates were designed to transfer the shear forces from the CLT panel to the t-stub connections. Moreover, steel brackets transfer the shear forces from the diaphragm to the walls. The proposed lateral system is supported by a steel gravity system that efficiently carries loads from the roof down to the concrete foundation. The aim of the proposed system was to maximize the strength and stiffness of the CLT panels within the core walls by using steel plates that run the height of the panels and are continuously connected to the panel [16].

3. Building studied

The timber-steel core wall system was utilized in the design of a multi-storey 7-storey benchmark building. The benchmark building was a 7-storey residential concrete building located in Vancouver, Canada. The building has structural irregularities allowing for comparisons between the built benchmark building and proposed timber-steel hybrid building. The floor plan remained constant from floors 1 through to 7; however, a mechanical room on the roof was included in the design similar to the benchmark building (Fig.3). The height of the first storey was 3.6m with each floor above having a height of 3m resulting in the total building height (including the mechanical roof top room) of 24.6m.

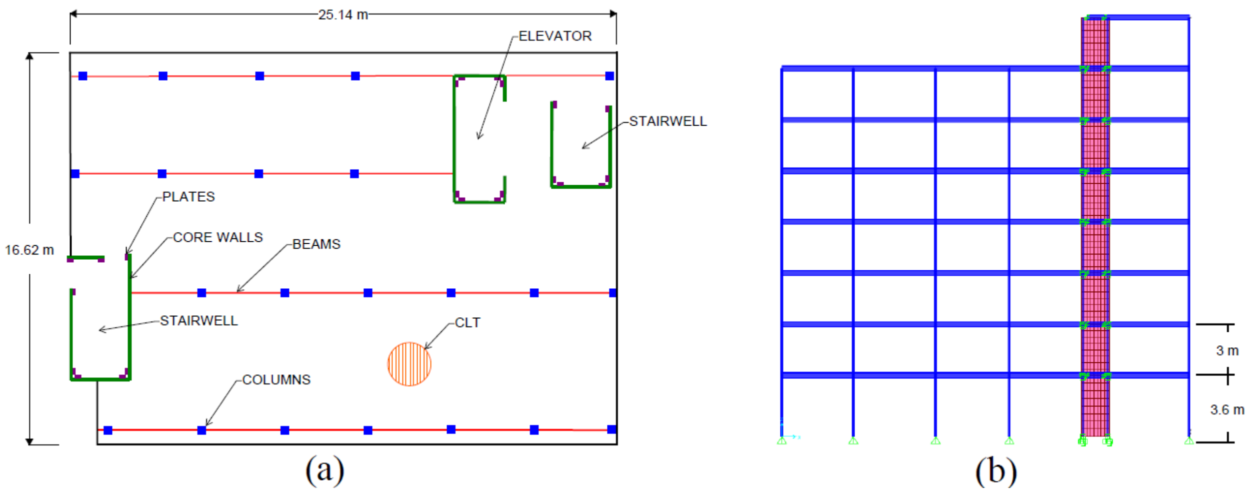

Fig. 3 – Hybrid building: a) floor plan; b) plan view

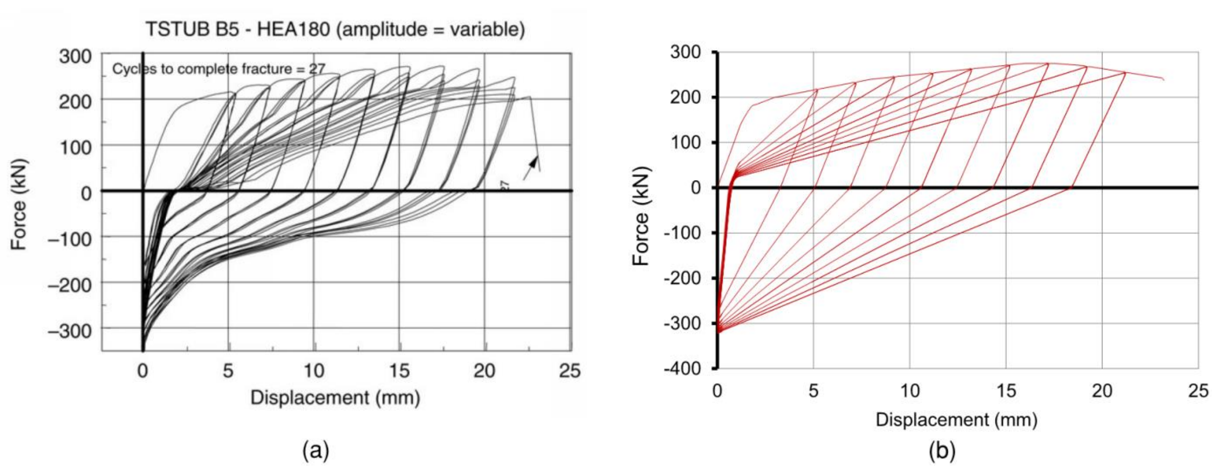

Fig. 4 – T-stub connection: a) experimental result [19]; b) pivot model calibration in SAP2000

4. Modeling

The analysis in this study was carried out using the SAP2000 commercial finite element software developed by Computers and Structures Inc. [17]. The CLT panels were modeled accordingly as orthotropic shell elements. To determine the elastic and shear moduli of the CLT panel in all planes the 'k Method' was used from the CLT Handbook [15]. All steel members were input to the model using the SAP2000 database for Canadian steel members. However, the plates were input to the model through the section designer provided in SAP2000. The frame members were discretized according to the accuracy necessary for the study. However, the meshing of the steel members has shown to have a small effect on the results, so a larger mesh size was appropriate to save computing effort.

Accurately modeling the connections is the most critical detail in the analysis of the timber-steel hybrid building and dictates whether the obtained results were correct. With extensive research into the program analysis methods the connections were modeled appropriately. Using experimental tests the connections were validated using SAP2000 and then compared with experimental work. Elastic and nonlinear spring elements were utilized in the building model. The conventional CLT hold-downs and brackets were designed to remain elastic during an extreme earthquake. Therefore, these connections were modelled as linear elastic elements. The t-stub connections were assigned multilinear plastic behaviour to allow for the plastic behaviour of the connection.

SAP2000 provides three different hysteresis types for the multilinear links: kinematic, Takeda and pivot. The hysteresis of t-stub connections from experimental tests showed to be most similar to the pivot hysteresis type. The model was developed by Dowell et al. [18] for reinforced concrete members. However, the hysteresis as shown in Fig.4 quite accurately predicted the response of the t-stub connection.

5. Energy based design

The proposed methodology was modified from the work of Choi et al. [14]. The steel-timber core wall EBD methodology follows the flow chart as shown in Fig.5.

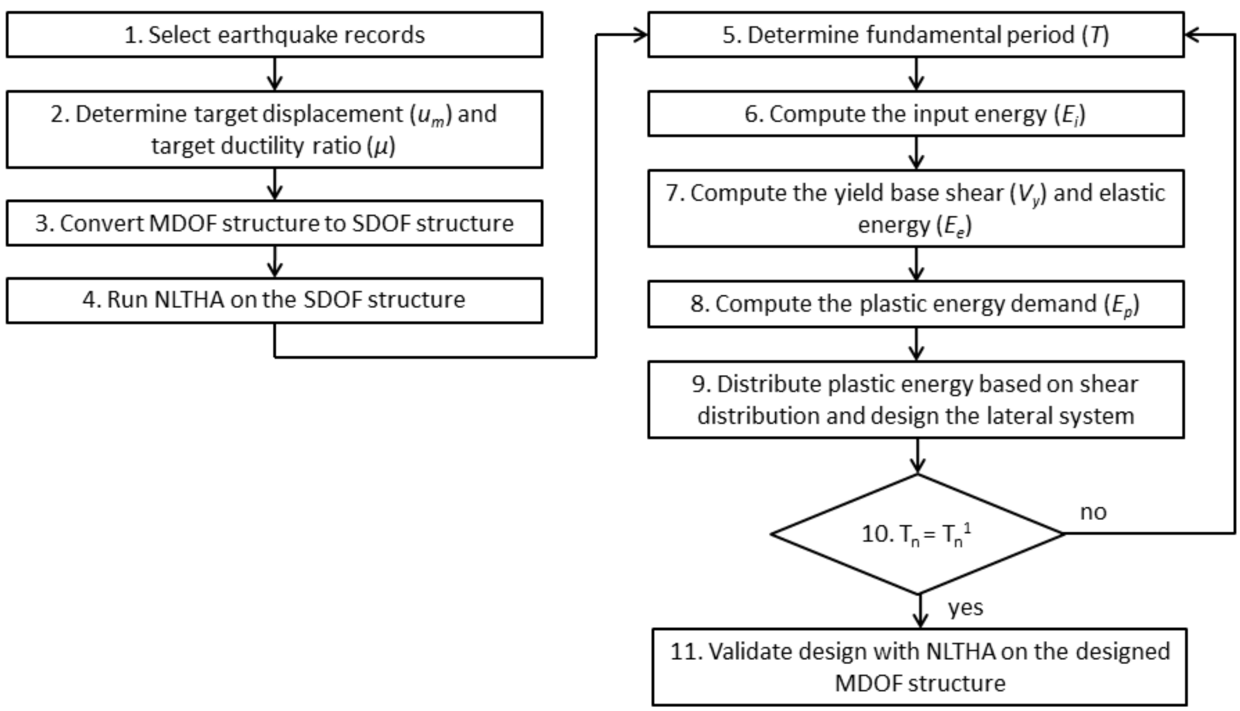

Fig. 5 – Energy based design flow chart

5.1 Step 1: Select earthquake records

Earthquake records are selected based on the location the building is to be built. These ground motions govern the design of the structure as the response spectra are developed from this set of ground motions. The ground motions were selected using a model that is based on a multiple-conditional-mean-spectra method [20, 21]. The pseudo acceleration (Fig.6) and velocity spectra were developed using the program Bispec [22]. Bispec is a nonlinear spectral analysis software that uses earthquake ground motion records to perform uni-directional and bi-directional dynamic time history analysis on SDOF systems.

5.2 Step 2: Determine target displacement and ductility ratio

This EBD methodology relies on a target displacement (u_T) to compute the ductility ratio (μ_T) based off the known yield displacement (u_y) of the system. Target drift is based on the structure type and desired performance level [1]. The yield displacement is found for the steel-timber hybrid system using the equations as described in Chapter 4 with no inelastic behaviour from the connections. The target ductility ratio is then defined as the target to yield drift:

$$\mu_T = \frac{u_T}{u_y} \tag{1}$$

For the timber-steel hybrid system the interstorey drift was limited to 2% as this would result in an overall drift of 1.5% to meet the life safe performance level [1]. Therefore, the yield drift of the system was found to be 143mm and the target drift was set as 369mm. These drifts resulted in a target ductility of 2.5.

5.3 Step 3: Convert the MDOF structure to an equivalent SDOF structure

The yield and target displacement are then used to derive the equivalent SDOF yield ($u_{y,eq}$) and target displacement ($u_{T,eq}$) [4]:

$$u_{T,eq} = \frac{u_T}{\Gamma_1 \Phi_{t1}} \tag{2}$$

$$u_{y,eq} = \frac{u_y}{\Gamma_1 \Phi_{t1}} \tag{3}$$

where Γ_1 is the modal participation factor and ϕ_{t1} is the fundamental mode shape vectors roof storey component.

5.4 Step 4: NLTHA on SDOF structure

Based on constant ductility, NLTHA is carried out on a SDOF structure with a bi-linear force displacement relationship. Damping is assumed to be 5% of the critical damping. A period range of 0.01 to 3.0 s is used to compute the acceleration, velocity and energy spectra. Again, this study used the program Bispec [22] to construct the spectra.

5.5 Step 5: Determine period

For the first iteration of the methodology the empirical formula from the NBCC for shear walls is used (Eq. (7)) After the first iteration the fundamental period will be re-iterated and therefore this empirical formula is only a starting point and does not dictate the final design.

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

where h_n is the height of the structure.

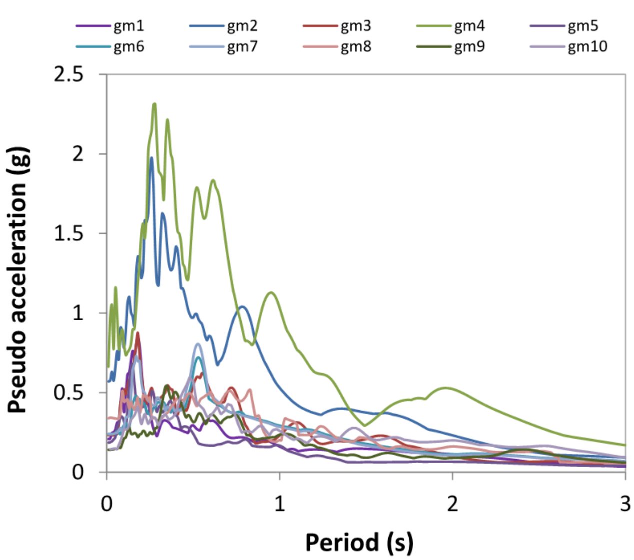

Fig. 6 – Pseudo acceleration response spectra

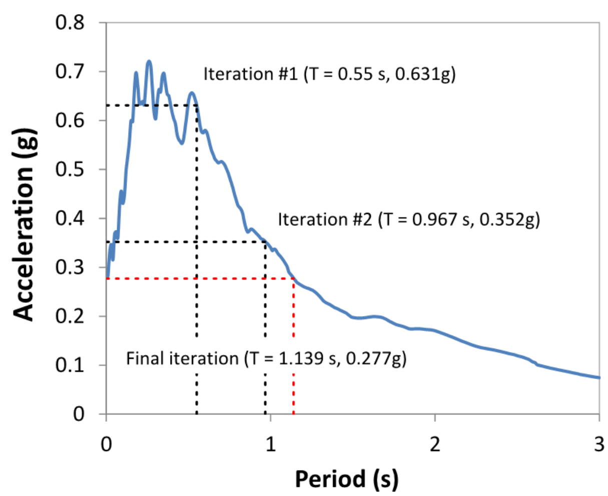

Fig. 7 – Average acceleration for each trial period

5.6 Step 6: Compute the input energy

The input energy (E_i) is estimated using the energy-balance concept as described earlier.

$$E_I = \frac{1}{2} M_1 S_v^2 = \frac{1}{2} M_1 \left(\frac{T_1 S_a}{2\pi}\right)^2 \tag{5}$$

where M_1 is the first modal mass, S_v is the pseudo velocity and S_a is the pseudo acceleration. This study uses the pseudo acceleration to estimate the input energy. The pseudo acceleration in determining the input energy is found by taking the average pseudo acceleration of the 10 ground motions. The average acceleration of the ten ground motion records is shown below in Fig.7 along with the design iterations.

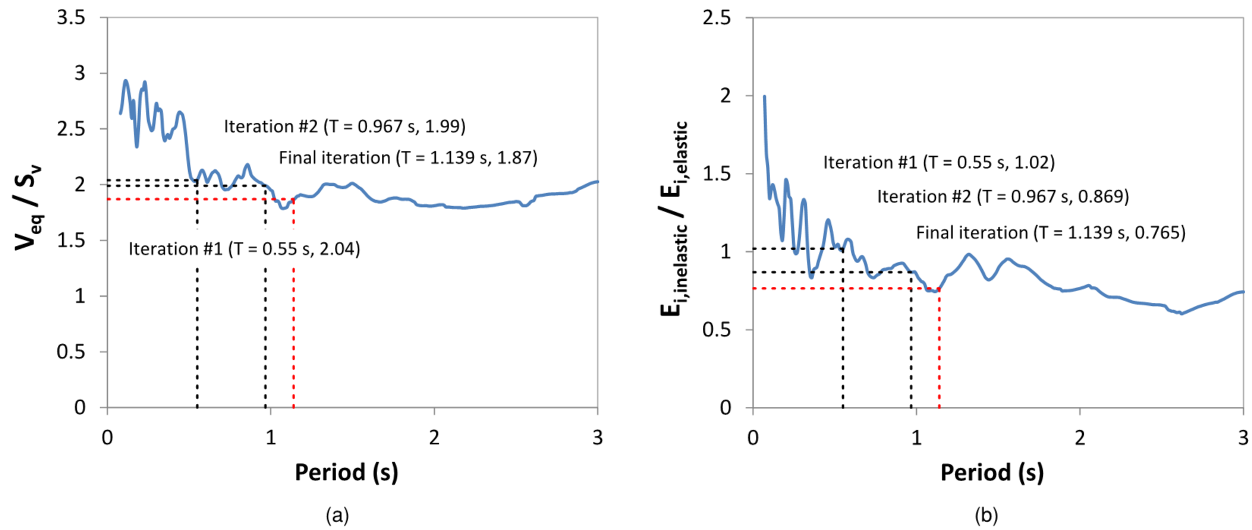

Fig. 8 – Ratio of: a) the equivalent velocity to pseudo velocity; b) inelastic to elastic input energy

However, this estimate of input energy has shown to underestimate the earthquake energy input to the structure [14]. Therefore, Choi et al. [14] recommend a modification factor (α) to estimate the correct input energy (E_i^*):

$$\alpha = \left(\frac{E_{i,inelastic}}{E_{i,elastic}}\right)\left(\frac{V_{eq}}{S_v}\right)^2 \tag{6}$$

where ($E_{i,inelastic}/E_{i,elastic}$) is the ratio of inelastic to elastic input energy for the target ductility and V_{eq} is the equivalent velocity:

$$V_{eq} = \sqrt{\frac{2E_i}{m}} \tag{7}$$

The equivalent velocity can be plotted using the above equation and the pseudo velocity as shown in Fig.8a. Furthermore, the inelastic to elastic input energy ratio is plotted in Fig.8b.

The modification factor (α) is then multiplied by the input energy (E_i) to obtain the correct input energy (E_i^*):

$$E_i^* = \alpha E_i \tag{8}$$

5.7 Step 7: Compute the yield base shear and elastic energy

The yield base shear (V_y) and elastic energy (E_e) are then determined:

$$V_y = \frac{E_i^*}{u_{Teq}\left(1 - \frac{1}{2\mu}\right)} \tag{9}$$

$$E_e = \frac{1}{2} u_{y,eq} V_y \tag{10}$$

5.8 Step 8: Compute the plastic energy demand

Finally, the plastic energy demand for the inelastic system (E_p^*) is estimated:

$$E_p^* = \beta(E_i^* - E_e) \tag{11}$$

where β is the correction factor to account for the overestimation of hysteretic energy ratio [14]:

$$\beta = \frac{E_h/E_i}{E_p/E_i} \tag{12}$$

where E_h is the hysteretic energy and found through the NLTHA in step 5. The ratio of hysteretic to input energy is plotted below in Fig. 9.

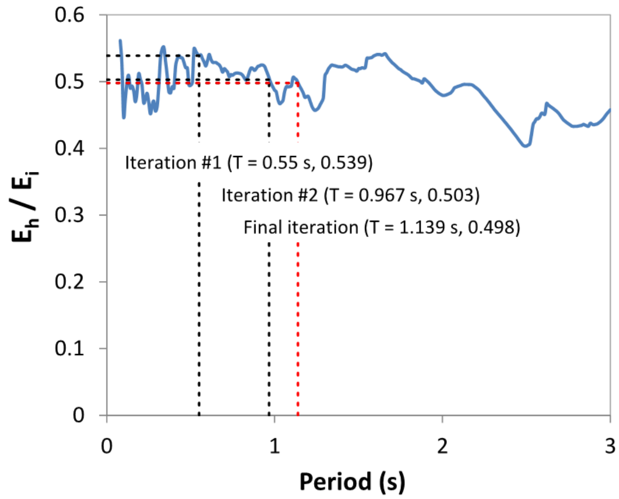

Fig. 9 – Hysteretic to input energy ratio

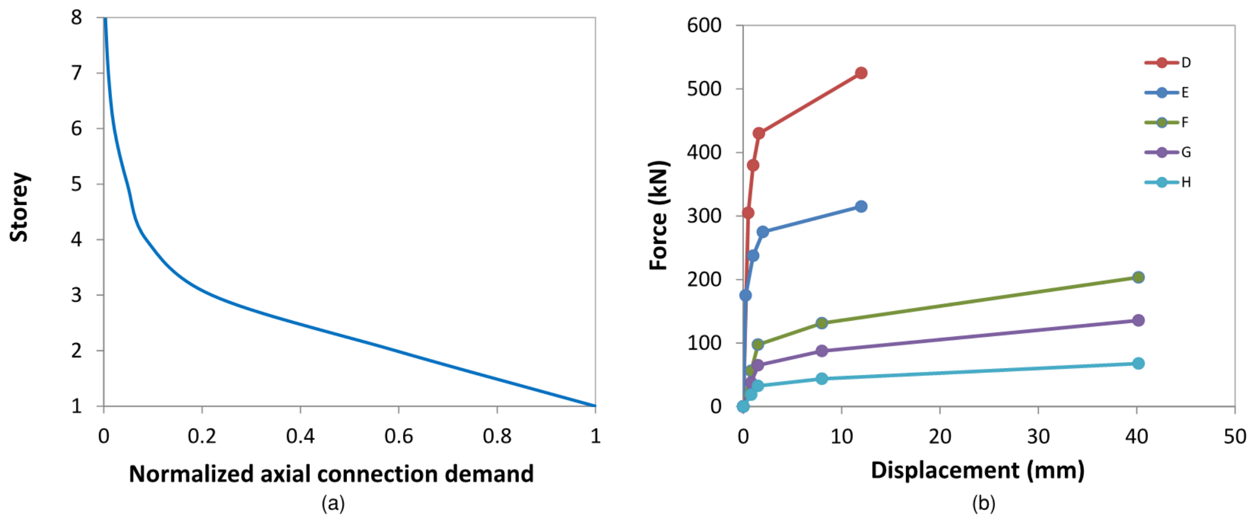

Fig. 10 – Normalized axial distribution ratio

The plastic energy demand must now be modified for the MDOF structure:

$$E_{pM}^* = \gamma E_p^* \tag{1}$$

where Υ is the ratio of plastic energy for a MDOF to an equivalent SDOF:

$$\gamma = \frac{E_{p,MDOF}}{E_{p,ESDOF}} \tag{2}$$

5.9 Step 9: Distribute plastic energy based on shear distribution

The energy distribution was determined for the proposed system according to the t-stub connections. Therefore, the axial distribution was the best representative for the dissipation of energy. The ESFP designed 2D structure is analyzed using NLTHA to determine the energy distribution. The results for each earthquake are averaged to determine the final distribution (Fig.10a). The plastic energy demand from step 8 is then applied to the structure according to the distribution and the connections are designed to dissipate this energy. Limiting factors on the connection design are the resulting interstorey drift and maximum displacement. The t-stub connections used in the 2D design are shown in Fig.10b.

5.10 Step 10: Fundamental period check

Using eigenvalue analysis in SAP2000 modal analysis is carried out to determine the fundamental period of the designed structure. If the new period from SAP2000 is the same as the assumed period from step 5 then the EBD methodology is complete. If the periods are not the same, the new period is used as the input for step 5 and steps 6-10 are performed again. These iterations continue until the fundamental periods converge. The design iterations for the 2D structure are shown in Table 1 below. The final EBD design for the 2D structure is shown in Table 2.

Table 1 – EBD process for timber-steel hybrid structure with 1.5% target drift

Trial	**1**	**2**	**···**	**Final**
Period (s)	0.550	0.967	···	1.139
Acceleration (g)	0.631	0.352	···	0.277
E_i (kN·mm)	902.8	874.9	···	740.1
V_{eq} / S_v	2.040	1.990	···	1.870
$E_{i,inelastic}$ / $E_{i,elastic}$	1.020	0.869	···	0.765
Ei^* (kN·mm)	3832.0	3010.7	···	1979.8
V_y (kN)	20.715	16.400	···	10.516
E_h / E_i	0.539	0.503	···	0.498
(E_h / E_i) / (E_p / E_i)	0.710	0.663	···	0.656
$E_{p,MDOF}$ / $E_{p,SDOF}$	0.759	0.759	···	0.759
E_{pM}^*(kN·mm)	1563.8	1146.6	···	746.5
T-stub$_7$	G	G	···	H
T-stub$_6$	G	G	···	H
T-stub$_5$	G	G	···	H
T-stub$_4$	G	G	···	H
T-stub$_3$	G	G	···	H
T-stub$_2$	F	G	···	H
T-stub$_1$	D	E	···	E

Table 2 – Final EBD design details

CLT core walls	
Floors	**Thickness (ply)**
1 - 8	5
T-stub connections	
Floors	**Type**
1 - 7	H
0	E
Plate size	
Floors	**Thickness (mm)**
3 - 8	12
2	16
1	20

5.11 Step 11: Validate the design

The final step in the methodology is to validate the EBD by applying the selected ground motions from step 1 on the MDOF structure. The results will show how the building behaves under the earthquake load and whether the target displacement is met for the system. Fig.11a shows the interstorey drift results (solid) with the ESFP results (dotted). Moreover, Fig.11b shows the maximum displacement results for the EBD methodology on the timber-steel hybrid structure.

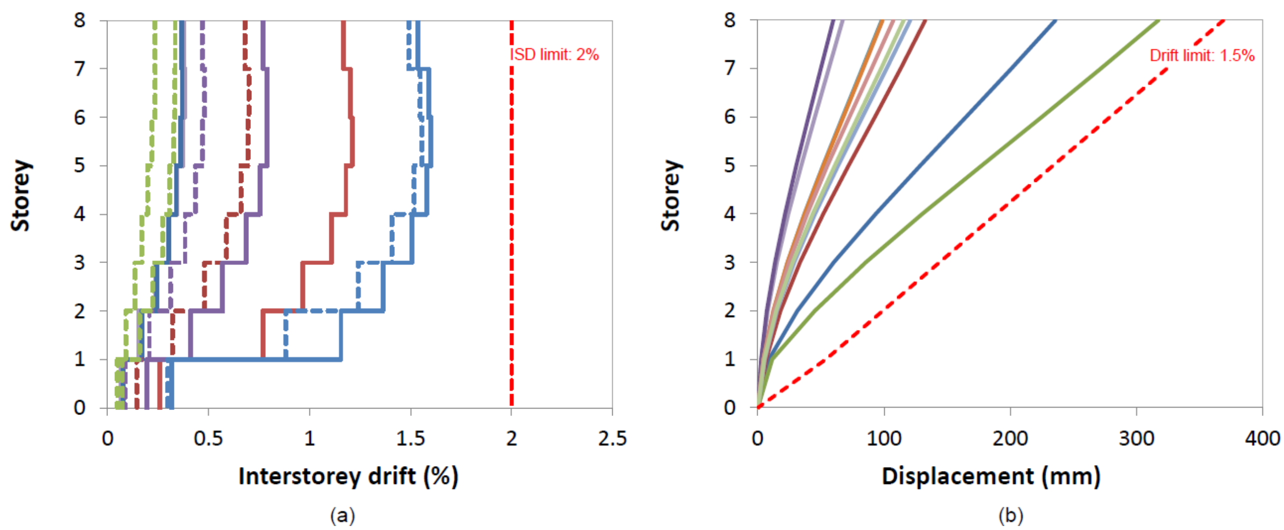

Fig. 11 – NLTHA results: a) interstorey drift results for the EBD design (solid line) compared with the ESFP design ISD results (dotted line); b) maximum displacement for EBD

6. Conclusions

Results show that the EBD performed as expected with an interstorey drift value less than 2%. Moreover, the target drift was not exceeded for the earthquakes. Therefore, the performance level desired was achieved. However, under the proposed EBD methodology some of the earthquakes should have exceeded the target drift as the average acceleration values are less than the maximum ground motions. There are a variety of ways to calculate the input energy modification factor for the actual system. This research used the same method as Choi et al. [14] suggested for BRB framed structures. Modifying this equation for a much lower ductility system will give more suitable results. When comparing to the work completed by Choi et al. [14] for BRB structures the design method should be altered as the BRB members are responsible in dissipating all the lateral forces in the system. The proposed system has many components contributing to the lateral resistance and therefore the t-stub connections do not have the same large effect on the system as the BRB members.

7. List of acronyms

BRB	buckling restrained brace	MDOF	multi degree of freedom
CLT	cross laminated timber	NBCC	national building code of Canada
DDBD	direct displacement based design	NLTHA	nonlinear time history analysis
EBD	energy based design	PBSD	performance based seismic design
ESFP	equivalent static force procedure	SDOF	single degree of freedom

8. Acknowledgements

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9. References

[1] Ghobarah A (2001): Performance-based design in earthquake engineering: State of development. *Engineering Structures*, **23**, 878–884.

[2] Pang W, Rosowsky DV (2009): Direct displacement procedure for performance-based seismic design of mid-rise wood-framed structures. *Earthquake Spectra*, **25** (3), 583–605.

[3] Boore DM, Joyner WB, Fumal TE (1997): Equations for estimating horizontal response spectra and peak acceleration from western North American earthquakes: a summary of recent Work. *Seismological Research Letters*, **68**, 128–153.

[4] ATC (1996): *ATC 40: Seismic evaluation and retrofit of concrete buildings*. Redwood City, USA.

[5] Federal Emergency Management Agency (1997): *NEHRP guidelines for the seismic rehabilitation of buildings: FEMA 273 report*. Washington, USA.

[6] Federal Emergency Managament Agency (1997): *NEHRP commentary on the guidelines for the seismic rehabilitation of buildings*. Washington, USA.

[7] SEAOC Vision 2000 Committee (1995): Performance based seismic engineering of buildings. Sacramento, USA.

[8] Mollaioli F, Bruno S, Decanini L, Saragoni R (2011): Correlations between energy and displacement demands for performance-based seismic engineering. *Pure Applied Geophysics*, **168**, 237–259.

[9] Decanini LD, Mollaioli F (1998): Formulation of elastic earthquake input energy spectra. *Earthquake Engineering and Structural Dynamics*, **27**, 1503–1522.

[10] Choi H, Kim J (2005): Energy-based seismic design of buckling-restrained braced frames using hysteretic energy spectrum. *Engineering Structures*, **28**, 304–311.

[11] Choi H, Kim J (2009): Evaluation of seismic energy demand and its application on design of buckling-restrained braced frames. *Structural Engineering Mechanics*, **31** (1), 93–112.

[12] Ghosh S, Adam F, Das A (2009): Design of steel plate shear walls considering inelastic drift demand. *Journal of Construction Steel Research*, **65** (7), 1431–1437.

[13] Leelataviwat S, Goel SC, Stojadinović B (2002): Energy-based seismic design of structures using yield mechanism and target drift. *Journal of Structural Engineering*, **128** (8), 1046–1054.

[14] Choi H, Kim J, Chung L (2006): Seismic design of buckling-restrained braced frames based on a modified energy-balance concept. *Canadian Journal of Civil Engineering*, **33** (10), 1251–1260.

[15] Gagnon S, Pirvu C (2011): *CLT Handbook: Cross-Laminated Timber*. Quebec, Canada.

[16] Goertz C (2016): *Energy Based Seismic Design of a Multi-Storey Hybrid Building : Timber Steel Core Walls*. (Masters dissertation). University of British Columbia.

[17] CSI (2006): *SAP2000 linear and nonlinear static and dynamic analysis and design of three-dimensional structures: basic analysis reference manual*. Berkeley, USA.

[18] Dowell RK, Seible F, Wilson EL (1998): Pivot hysteresis model for reinforced concrete members. *ACI Structural Journal*, **95** (5), 607–617.

[19] Piluso V, Rizzano G (2008): Experimental analysis and modelling of bolted T-stubs under cyclic loads. *Journal of Construction Steel Research*, **64** (6), 655–669.

[20] Atkinson GM, Goda K (2011): Effects of seismicity models and new ground-motion prediction equations on seismic hazard assessment for four Canadian cities. *Bulletin of the Seismological Society of America*, **101** (1), 176-189.

[21] Baker JW (2011): Conditional mean spectrum: Tool for ground motion selection. *Journal of Structural Engineering*, **137** (3), 322-331.

[22] Hachem M (2004): *BISPEC: Bidirectional linear and nonlinear spectra of earthquakes*. University of California, Berkeley, USA.