DIRECT DISPLACEMENT BASED DESIGN OF STEEL MOMENT-RESISTING FRAMES WITH CLT INFILL WALLS

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ABSTRACT: An innovative hybrid timber-steel structure, as an alternative lateral-load resisting system, is developed at The University of British Columbia. The hybrid structure incorporates Cross Laminated Timber (CLT) panels as an infill in steel moment resisting frames (SMRFs). In this paper, an iterative direct displacement based design method is developed and analytically validated for this hybrid structure. The iterative design procedure involves the following initial modeling variables: gap between CLT panel and steel frame, bracket (connection) spacing, CLT panel thickness and strength, and post yield stiffness ratio of the steel members. Subsequently, the design displacement profile is developed by assigning an initial relative strength between CLT wall and frame elements. This profile is then used to obtain the characteristics of an equivalent single degree of freedom (SDOF) system. A mathematical expression for the system ductility is formulated based on the proportions of the overturning moment resistance of the CLT wall and SMRF. A calibrated equivalent viscous damping-ductility relationship is used to obtain the energy dissipation of the equivalent SDOF system. A case study hybrid system, with 3-, 6-, and 9-storey building heights, three bays and middle bay infilled, are designed using the proposed method. Nonlinear time history analysis using twenty-earthquake ground motion records is used to validate the performance of the proposed design methodology. The results indicate the efficiency of the proposed method in controlling displacements due to seismic excitation of the hybrid structure.

1. Introduction

An innovative hybrid timber-steel structure, as an alternative lateral-load resisting system, is developed at The University of British Columbia (UBC) (Stiemer et al. 2012; Dickof 2013; Dickof et al. 2014). The hybrid structure incorporates Cross Laminated Timber (CLT) shear panels as an infill in the steel moment resisting frame (SMRF) (Fig. 1). The proposed system has higher seismic resistance and lower seismic vulnerability than bare steel moment frames (Tesfamariam et al. 2014b). L-shaped steel brackets are bolted to the steel frame and nailed to the CLT panels achieve the connection between the steel frames and CLT walls. These connections ensure full confinement between the structural elements as well as energy dissipation under seismic shaking. The interface between the CLT infill wall and the steel frame is provided with a small gap to allow the connection brackets to deform under lateral load. This allows the frame and panel to act independently and influence each other under lateral loading.

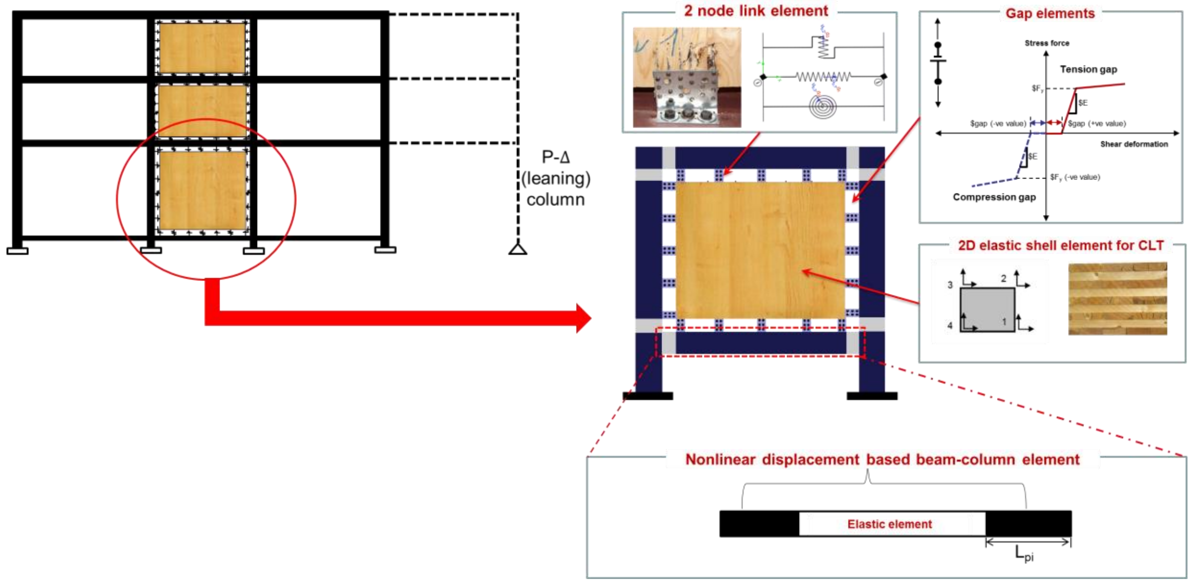


Fig. 1 - Details of CLT infilled SMRFs

The National Building Code of Canada (NBCC) (NRC, 2010) recommends an equivalent static force design with appropriate overstrength and ductility factors for the seismic design of structures. However, the NBCC (NRC 2010) does not have the appropriate overstrength and ductility factors to design the proposed hybrid structure. In this paper, for the proposed hybrid system, a direct displacement based design (DDBD) method (Priestley et al. 2007), as an alternative performance based design, is developed and analytically validated. DDBD is a performance based seismic design method where the designer, based on the desired / acceptable level of damage, defines the performance objectives (Priestley et al. 2007; Ghobarah et al. 1999). The performance objective can be specified as the maximum displacement and/or interstorey drift values. The idea of incorporating displacements in the design process of structures through a concept of substitute-structure was first implemented by Shibata and Sozen (1974). Gulkan and Sozen (1974) developed a method to estimate the design base shear of structures by considering their inelastic response. Priestley and Kowalsky (2000) then applied the DDBD method to design multi-degree freedom of system (MDOF) reinforced concrete frames and wall buildings based on an initially estimated displacement profile.

In this paper, an iterative DDBD method is developed for a CLT infilled steel moment resisting frame structure. The paper is outlined as follows. Firstly, the proposed DDBD approach is presented in a step-by-step format by designing a case study low-rise (3-storey) hybrid building. Subsequently, this method is extended to design 6- and 9-storey hybrid buildings. Thirdly, validation of the proposed method is carried out by using Nonlinear Time History Analysis (NLTHA). Finally, results and discussions are provided about the efficiency of the method by giving emphasis for second order (P-Delta) effects.

2. Proposed DDBD Approach for CLT infilled SMRFs

A thorough discussion on the fundamentals of the DDBD method for different types of structures is reported in Priestley et al. (2007). In this paragraph, the basic steps and key equations of DDBD method are highlighted. Fig. 2 shows the flowchart for the DDBD. The steps are outlined below.

- First step entails a virtual representation of the MDOF nonlinear structures with an equivalent single degree of freedom (SDOF) system through effective mass (m_{eff}) and effective height (h_{eff}) (Fig. 2a), secant stiffness K_e and equivalent viscous damping (EVD) ξ_{eq} at peak displacement Δ_d (Fig. 2b).

- In DDBD, EVD represents the energy dissipative capacity of the structures. This EVD (ξ_{eq}) contains both the elastic and hysteretic components of damping (Fig. 2c).
- Once the design displacement Δ_d and ξ_{eq} are established, the effective period (T_{eff}) can be obtained from the highly damped displacement spectra (Fig. 2d). The corresponding effective stiffness, K_{eff} (Fig. 2b), design base shear, and distributed design force vector can then be calculated using basic structural dynamics equations. A brief review and applicability of these equations is presented in subsequent sections.

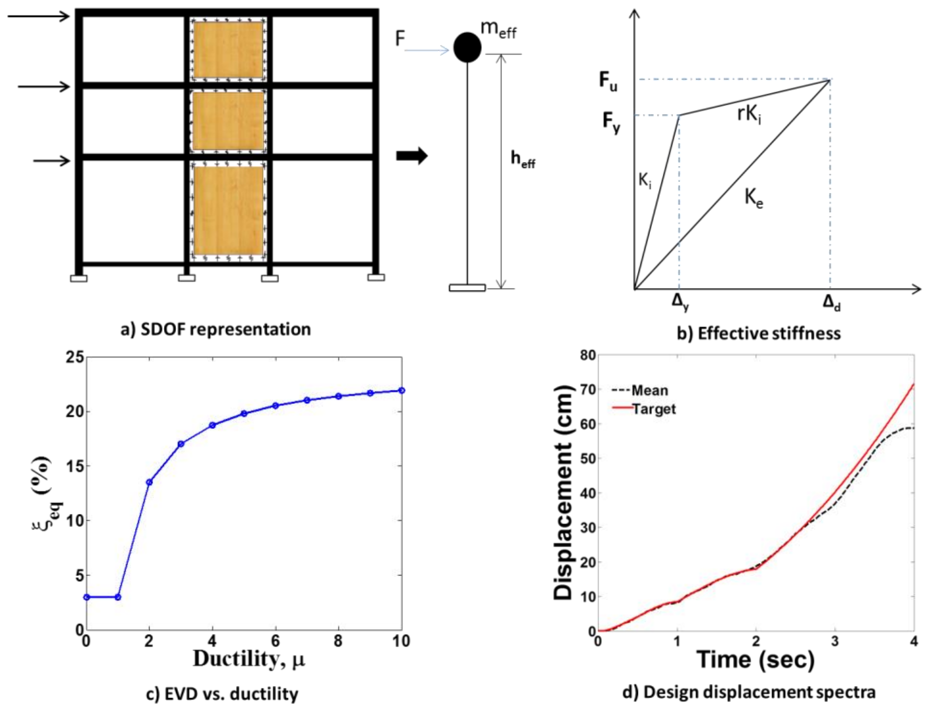


Fig. 2 - Basics of DDBD

The proposed DDBD procedure is illustrated by designing a case study building: 3 storey - 3 bays (middle bay infilled) SMRF (Fig. 2a). The height of each storey of the building is 3.2 m. A constant bay width of 6 m is used for the entire building. The building is assumed to be situated on a very dense soil and soft rock (site class C) in Vancouver, Canada. The building is modelled as a two-dimensional structure and due to its symmetry in plan, accidental torsion is neglected both in the design and analysis phase. Both beam and column elements are detailed based on CSA-S16 code (CSA-S16, 2009) with a yielding strength of Fy = 350 MPa and modulus of elasticity of Es = 200 GPa. A constant floor seismic weight (including the CLT panels) of 253 tons was considered. The proposed design methodology is comprised of 11 steps and details of each step are provided for the case study building.

Step1: Optimum modeling parameters of CLT infilled SMRFs

Tesfamariam et al. (2014a) and Bezabeh (2014) performed a multi-objective optimization analysis of drift demands of CLT infilled SMRFs. Based on their result, optimal bracket spacing of 0.8 m, panel thickness and strength of 99 mm and 17.5 MPa, respectively, are used as starting parameters for the current example.

Step 2: Lateral load proportions between CLT infill walls and steel moment frames

A concept of initial strength assignment to calculate the characteristics an equivalent SDOF is adopted from Sullivan et al. (2006). Since the CLT shear panels behave in a pure shear manner, it is reasonable to assign the shear strength proportion at the start of the process, as their bending strength is not important. As initial starting point, 70% of the total shear is directly assigned to the frames and this is iteratively changed through the design process. The shear resistance for the CLT wall is then calculated by subtracting the SMRF shear ($V_{i,frame}$) from the total shear as (Sullivan et al. 2006):

$$\frac{V_{i,CLT}}{V_b} = \frac{V_{i,total}}{V_b} - \frac{V_{i,frame}}{V_b} \tag{1}$$

where V_b is design base shear, $V_{i,CLT}$ is the shear resisted by the CLT infill panels at story *i*, and $V_{i,total}$ is the total shear at storey *i*. The total shear of the system is established as a function of V_b, storey number (*i*) and total number of storeys (*n*) as (Sullivan et al. 2006):

$$\frac{V_{i,total}}{V_b} = 1 - \frac{i(i-1)}{n(n+1)} \tag{2}$$

Step 3: Design displacement profile

The properties of an equivalent SDOF system depends on the drift limit of the lower stories of moment frames and an assumed displacement profile. This displacement profile is related to the inelastic first mode response of the structure under seismic excitation (Priestley et al. 2007). To ensure satisfactory performance of the structures under seismic event, building codes specify limits on lateral storey drift values. The NBCC (NRC 2010) specified a 2.5% interstorey drift limit to represent extensive damage on the buildings. Therefore, the drift limit Δ_d of 2.5% corresponding to lower storey drift demand of the hybrid system is selected as a target drift limit. The displacement profile is established as (Sullivan and Lago 2012):

$$\Delta_i = \omega_\theta \theta_c h_i \left(\frac{4H_n - h_i}{4H_n - h_1}\right) \tag{3}$$

where Δ_i is the displacement at level *i*, h_i is the height of i^{th} floor from the ground, H_n is the total building height, ω_θ is the factor to account for the effects of higher modes and is given as:

$$\omega_\theta = 1.15 - 0.0034H_n \le 1 \tag{4}$$

Step 4: Characteristics of equivalent SDOF system

The design displacement (Δ_d), effective mass (m_{eff}), and effective height (h_{eff}) of a substitute equivalent SDOF system are computed as (Sullivan and Lago 2012):

$$\Delta_d = \frac{\sum_{i=1}^{n} m_i \Delta_i^2}{\sum_{i=1}^{n} m_i \Delta_i} \tag{5}$$

$$m_{eff} = \frac{\sum_{i=1}^{n} m_i \Delta_i}{\Delta_d} \tag{6}$$

$$h_{eff} = \frac{\sum_{i=1}^{n} m_i \Delta_i h_i}{\sum_{i=1}^{n} m_i \Delta_i} \tag{7}$$

where m_i is lumped mass in each storey (*i*) and h_i height of each storey from the base. Table 1 summarizes the characteristics of an equivalent SDOF system of the hybrid system.

Table 1- Characteristics of equivalent SDOF

Storey	h (m)	Δ_i	Θ_i	m_i (Ton)	$m_i\Delta_i$	$m_i\Delta^2_i$	$m_i\Delta_i h_i$	Δ_d	m_{eff}	h_{eff}
3	9.6	0.196	1.59	253	49.68	9.75	476.92	0.156	680.9	7.28
2	6.4	0.145	2.04	253	36.8	5.35	235.52			
1	3.2	0.08	2.5	253	20.24	1.61	64.768			

Step 5: Hybrid system ductility (μ_{sys})

As crushing of the wood at the interface with the nails occurs at relatively low drift values, the associated ductility is large. However, for the SMRFs the inelastic response occurs at a relatively larger drift value. Therefore, for this design purpose, this effect is aggregated by taking the weighted average ductility associated with panel crushing and steel yielding as:

$$\mu_{sys} = \frac{M_{CLT}\mu_{CLT} + M_{frame}\mu_{frame,average}}{M_{frame} + M_{CLT}} \tag{8}$$

where M_{CLT} and M_{frame} are the overturning moment resistance of CLT panel and frame, respectively. The displacement ductility value of CLT panels (μ_{CLT}) is calculated from design displacement of the hybrid system as follows:

$$\mu_{CLT} = \frac{\Delta_d}{\Delta_{crush,CLT}} \tag{9}$$

where $\Delta_{crush,CLT}$ is the wood crushing displacement at the interface with the nails. With the aim of formulating the EVD of CLT infilled SMRFs, Bezabeh (2014) developed the following relationship to calculate $\Delta_{crush,CLT}$ by representing the compression force in the panels as a strut action.

$$\Delta_{crush,CLT} = \frac{d f_{CLT}}{E_o cos\theta} \tag{10}$$

where E_o, f_{CLT} , and d are the modulus of elasticity, crushing strength, and diagonal length of CLT panel, respectively.

The displacement ductility of the steel moment resisting frame is calculated using Equation 11 (Garcia et al. 2010).

$$\mu_{frame,i} = \frac{\Delta_i - \Delta_{i-1}}{h_i - h_{i-1}}\left(\frac{1}{\theta_{y,steel\ frame}}\right) \tag{11}$$

where $\mu_{frame,i}$ is the ductility demand of i^{th} storey and $\theta_{y,steel\ frame}$ is the yield drift of the steel frame as given by Equation 12 (Priestley et al. 2007b).

$$\theta_{y,steel\ frame} = 0.65\varepsilon_y \frac{L_b}{h_b} \tag{12}$$

where L_b and h_b are the beam span length and depth, respectively. For the building of this case study, the calculated μ_{sys} = 2.217.

Step 6: System equivalent viscous damping (ξ_{eq})

An expression and plots of EVD for SDOF hybrid systems are developed in Bezabeh (2014) and Bezabeh et al. (2015). Fig. 3 shows the damping ductility law of SDOF hybrid system with corresponding modeling variables specified in step 1. Derivation of the damping-ductility law shown in Fig. 3 is discussed in Bezabeh (2014). From the plot, the EVD corresponding to μ_{sys} = 2.217 (obtained in Step 5) is ξ_{eq} = 14.5%. This total ξ_{eq} is computed by adding 3% elastic damping to hysteretic damping (ξ_{hyst}).

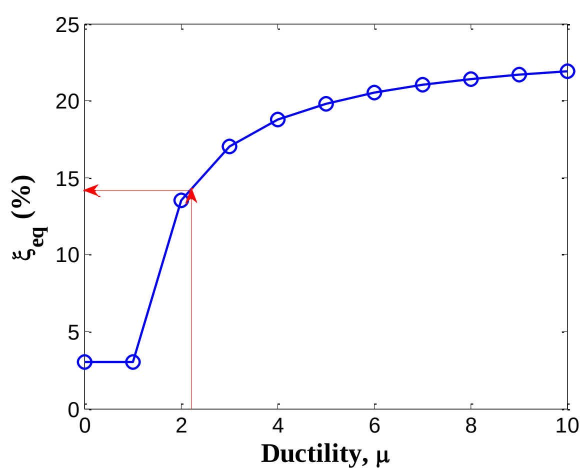


Fig. 3 - Equivalent viscous damping (EVD)-ductility relationship for the proposed hybrid system

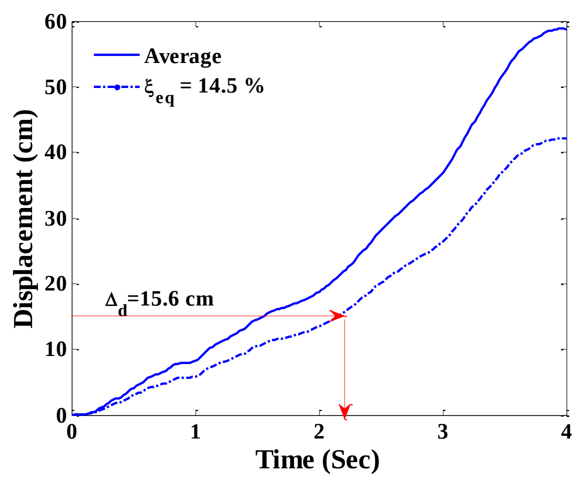


Fig. 4 - Effective period from damped displacement spectrum

Step 7: Effective period of the system (T_{eff})

The effective period T_{eff} of the equivalent SDOF system is obtained from the highly damped displacement spectrum. A scaling factor is calculated using Equation 13 of Eurocode-8 (EC8 1998) to obtain the highly damped displacement spectra corresponding to ξ_{eq} = 14.5 %. Fig. 4 shows the damped spectrum used to calculate T_{eff} = 2.26 sec.

Step 8: Effective stiffness and design base shear

The effective stiffness (K_{eff}) and design base shear (V_b) are calculated as follows:

$$\eta = \sqrt{\frac{10}{5+\xi_{eq}}} \tag{13}$$

$$K_{eff} = 4\pi^2 \frac{m_{eff}}{T_{eff}^2} = 5257.4 \text{ kN/m} \tag{14}$$

$$V_b = K_{eff}\Delta_d = 824.04 \text{ kN} \tag{15}$$

Step 9: Structural analysis

The above calculated design shear force is distributed to perform the structural analysis of the system. The design shear forces at each level of the building (F_i) is computed as (Sullivan and Lago 2012):

$$F_i = V_b \frac{m_i \Delta_i}{\sum_{i=1}^{n} m_i \Delta_i} \tag{16}$$

A portal method of analysis has been chosen to perform the structural analysis due to its simplicity and accuracy. During the analysis, the inflection point for the bottom columns is taken to be at 60% of the storey height (h_s). This provision will avoid any soft storey mechanisms (formation of yielding point on the top of the lower story columns). However, for other columns of the frames, this inflection point is set at the mid height of the given storey.

Step 10: Design of steel frame members

The plastic moment strengths for the beams and columns are calculated in step 9. Subsequently, the plastic section modulus is computed to choose an appropriate section that satisfies the demand. It should be noted at this point that the selection of steel cross sections is accomplished with only the governing load (seismic actions). This assumption is correct for building designs in high seismic region. As required by CSA S16-09 (CSA, 2010), both beams and columns are assumed to be constructed by considering bracing against lateral torsional buckling. Table 2 summarizes the selected sections for the case study building.

Step 11: Check for CLT properties

The steel connection brackets transfer the shear and axial loads from the steel frame to the CLT wall. Experimental tests performed by Schneider et al. (2014) on the bracket connections indicate that these connection types can carry up to 45 kN both in shear and axial directions. At the start of the design process, 16 (bracket spacing of 0.8 m) brackets were applied at the top and bottom of the panel. Therefore, these brackets transfer 16 x 45 = 720 kN shear and axial force without failure. The maximum shear demand on the CLT wall is less than the bracket shear force transferring capability. Therefore, the initially assumed bracket spacing is acceptable. The above steps have been followed to design middle bay CLT infilled six and nine storey 2D hybrid buildings. The final design results of DDBD of 3-, 6- and 9-storey building heights are summarized in Table 2.

3. Validation using Nonlinear Time History Analysis

The proposed DDBD method is validated by comparing its displacement responses from NLTHA with the initially assumed target displacement profile. For this purpose, NLTHA using twenty earthquake ground motion records was conducted using OpenSees software (Mazzoni et al. 2006).

Table 2- Details of DDBD

	3 storey	6 storey	9 storey	9 storey (Re-designed)
Proportion of V_b assigned for frames (%)	70	50	50	50
Design storey drift, Θ_d (%)	2.5	2.5	2.5	2.5
Design displacement , Δ_d (m)	0.156	0.28	0.409	0.409
Effective height, h_{eff} (m)	7.28	13.5	19.73	19.73
Effective mass, m_{eff} (ton)	680.8	1287.3	1887.5	1887.5
$\mu_{frame,\ average}$	1.123	1.22	1.28	1.28
μ_{sys}	2.21	7.53	10.21	10.21
ξ_{SDOF} (%)	14.5	20.5	21	21
Effective period, T_{eff} (sec)	2.26	3.2	4.2	4.2
K_{eff} (kN/m)	5257.4	4957.9	4219.9	4219.9
V_b (kN)	824.04	1399.8	1726	1726
Beam section	W310 x 52	W310 x 67	W360 x 79	W360 x 122
Interior column section	W360 x 79	W360 x 91	W360 x 110	W360 x 162
Exterior column section	W310 x 45	W310 x 52	W360 x 64	W360 x 101
Beam strength, M_{bi} (kN.m)	261	326	444	705
Interior column strength, $M_{int:col,i}$ (kN.m)	444	552	640	975
Exterior column strength, $M_{ext:col,i}$ (kN.m)	220	261	354	584

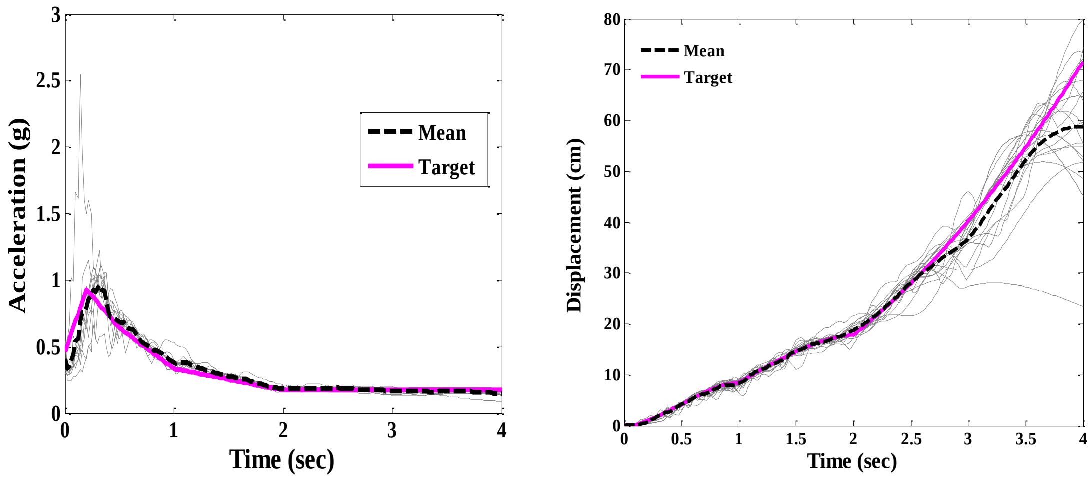
3.1. Modeling of the CLT infilled SMRFs

The Open System for Earthquake Engineering Simulation (OpenSees) (Mazzoni et al. 2006) is used to model steel frames, CLT infill, and connections. The structural frame elements have been modeled using combination of linear and nonlinear elements. Linear elastic and non-linear *displacement based beam-column* elements used for the center and end of the frame member respectively are shown in Fig. 1. Modified Ibarra Krawinkler Deterioration model (Lignos and Krawinkler 2011) used with a bilinear material property based on moment-curvature relationships are given in the ASCE 41 (ASCE 41 2006) for nonlinear parts of the frame elements with appropriate plastic hinge length. As shown in Fig. 1, a leaning column is introduced to the hybrid structure to simulate P-Delta effects. The section of CLT panel was modeled as single layer with linear elastic-isotropic wood material property of Quad elements (Dickof et al. 2014).

The CLT mechanical properties used for modeling can be found in Dickof et al. (2014). The steel bracket connections between the steel frames and CLT walls are modelled as a nonlinear spring element using OpenSees's *Pinching4* material model. Shen et al. (2013) based on the experimental results from Schneider et al. (2013) calibrated this model. The nonlinear bracket behaviour was assigned in both the shear and axial direction. Additionally, the *Elastic Perfectly Plastic Gap (EPPG)* gap property was assigned to the system in parallel formulation with the axial behaviour of bracket.

3.2. Ground Motion Selection and Matching

Once the analytical models are developed, NLTHA has been carried out using twenty ground motions records which were obtained from the Pacific Earthquake Engineering Center (PEER 2005) database by comparing the ratio of seismic motion peak acceleration (A) to peak velocity (V) to Vancouver's A/V. The details of the ground motions can be found in Bezabeh (2014). The GMRs considered were matched with the response spectra of Vancouver at the specified period range. Matching was done within the period range of (0.2T - 1.5T); 0.138 sec - 4 sec, where T is the fundamental period of the building. Fig. 5a and 5b show the scaled spectra with the mean and target spectrum for acceleration and displacement, respectively.

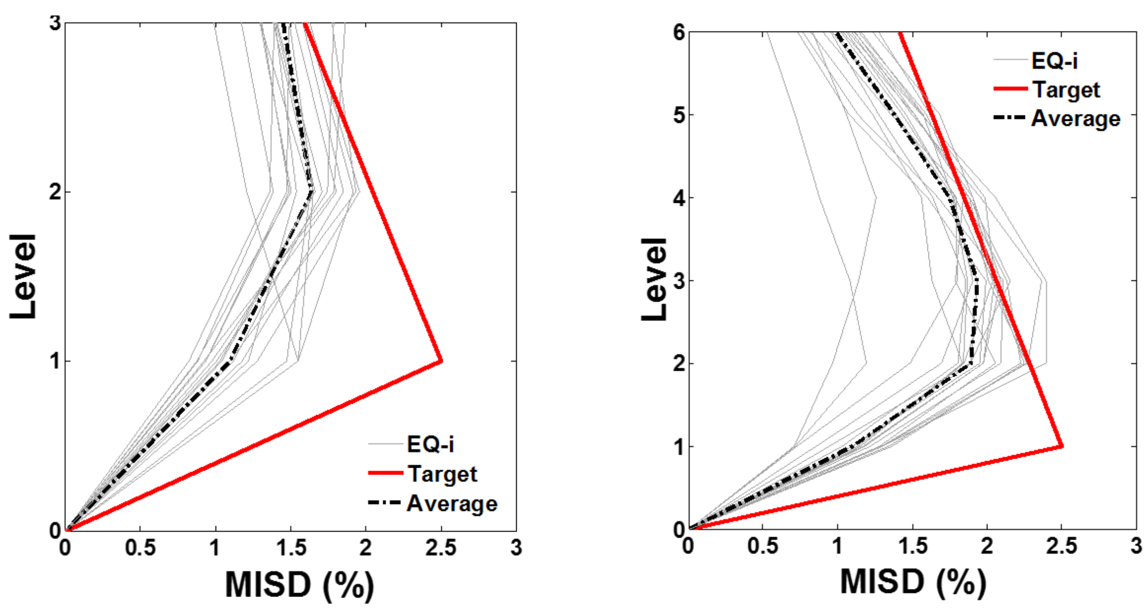


(a) (b)

Fig. 5 - a) Comparison between mean and target spectrum for selected ground motion and b) Displacement spectra at 5% damping level from the scaled ground motion

3.3. Results of nonlinear time history analysis

The Maximum Interstorey Drift (MISD) response of 3-, and 6-storey hybrid buildings are plotted in Fig. 6(a and b), respectively. As can be seen from Fig 6, irrespective of the height of the building, the average MISD response is always less than the target drift profile. In Bezabeh (2014) the same result is reported without consideration of second order effects (P-Delta). Therefore, it can be concluded that for low and med-rise hybrid buildings, the developed method is satisfactory, even without consideration of P-Delta effects during the analysis. However, Fig 7a shows the average MISD response of 9-storey hybrid building exceeded the target drift profile by 14% at the third storey. Consequently, the building is redesigned by increasing the member sizes (i.e. moment resistance of frame members) without changing the shear proportions between the walls and frames. The details of the redesigned building are summarised in Table 2 and the MISD result is shown in Fig. 7b.



(a) (b)

Fig. 6 – Maximum interstorey drift response hybrid buildings a) 3 storey building b) 6 storey building

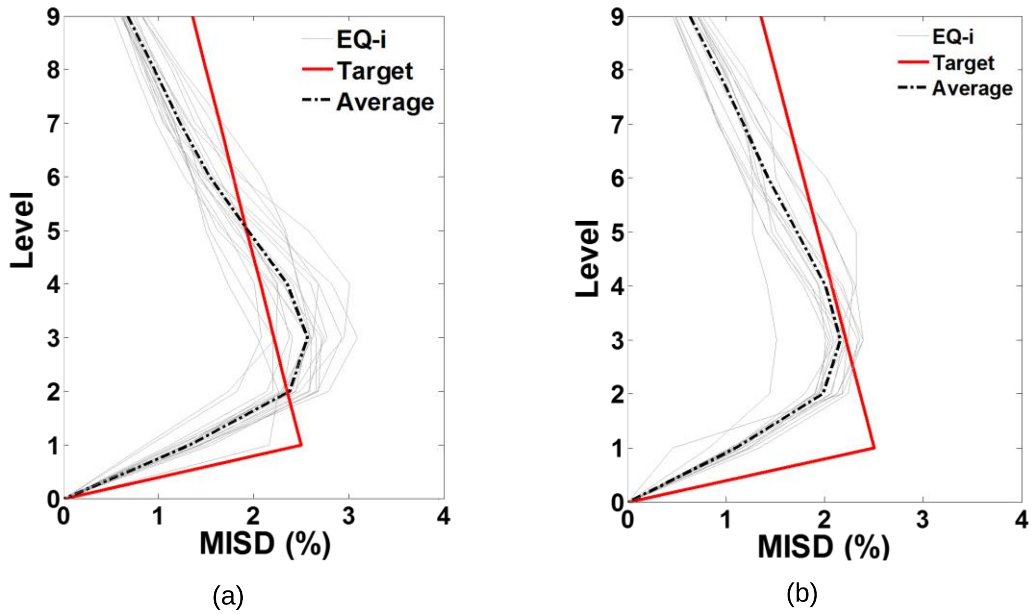


Fig. 7 - Response of 9-storey hybrid building a) First trial design (with P-Delta) and b) Redesigned building (with P-delta in the analysis)

It is evident from Fig. 7b that the MISD response of the redesigned building is close to, but less than the target drift profiles. A more robust approach to mitigate the amplifications due to second order effect is to consider P-Delta effects in the DDBD design process. Pettinga and Priestly (2008) proposed a DDBD method that accounts for P-Delta amplifications and provided design charts.

4. Conclusion

A new iterative direct displacement based design method for SMRFs with CLT infill walls has been developed and tested by designing 3-, 6-, and 9-storey hybrid buildings. In summary, the developed method proved to effectively control seismic interstorey drifts and displacements. A robust finite element model of the hybrid structure that accounts for the CLT panel and frame interactions was used for the validation process. Initial shear proportions between the wall and frame are assigned at the start of the design process. Furthermore, the system ductility and equivalent viscous damping are explicitly accounted. Better control of storey drifts and displacement were achieved for low and mid-rise hybrid buildings. From the design examples shown in this paper, for low- and mid-rise height structures, the P-Delta effect is not significant, however, for high-rise structures, P-Delta effect should be taken into consideration.

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