MITIGATING TORSIONAL IRREGULARITY USING CROSS LAMINATED TIMBER-REINFORCED CONCRETE HYBRID SYSTEM

by

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Abstract

Seismic torsional responses in buildings is a result of eccentricity in mass and stiffness distribution. Torsional irregularity is one of the major causes of severe damage and collapse of structures during an earthquake. In this study, effect of torsion on the structures is reviewed, the definition of torsional irregularity and the characteristic of the structure that leads to this type of irregularity is elaborated. The evolution of the methods to consider the effect of torsion in the National Building Code of Canada (NBCC) is reviewed and different methods to prevent torsional irregularity in the structures are discussed.

Hybridization with Cross-Laminated Timber (CLT) is suggested as a new method to rectify the effect of torsional irregularity for different performance levels. Accordingly, the definition of hybridization and hybrid structure seismic behavior, CLT material specifications and CLT seismic performance is discussed.

In order to evaluate the effect of CLT hybridization on buildings with torsional irregularity, a four-storey reinforced concrete (RC) structure with torsional irregularity is considered for Vancouver seismicity condition. SAP2000 software is used to conduct Linear Dynamic Analysis (LDA) and Non-Linear Time History Analysis (NLTHA) using eight different ground motion scaled to Vancouver design spectra.

The effect of the CLT wall panel as shear wall on the in plane seismic base shear and inter-storey drift is shown using the linear and non-linear dynamic analysis. The result from the analysis compared to the code static values.

The literature of Performance Based Seismic Design (PBSD) is reviewed. PBSD is used to determine the performance level of the original and hybrid building. The inter-storey drifts criteria defined in FEMA 356 guidelines is used for the purpose of NLTHA.

Preface

One journal papers have been prepared from this master thesis to be submitted to a peer review journal. The paper includes a variety of materials from all chapters of this thesis. Yazdinezhad, M. and Tesfamariam, S. (2016) Mitigating torsional irregularity using cross laminated timber-reinforced concrete hybrid system. *International Journal of Advanced Structural Engineering (IJASE)*, to be submitted.

The paper is written by me, I am the lead author. Dr. Solomon Tesfamariam provided review and feedback and finalized the manuscript.

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1 Chapter: Introduction

1.1 Building irregularities

Geometric irregularities, in floor plan of the buildings can result in accidental torsion of floor diaphragm during an earthquake load. The eccentricity between the center of mass and center of rigidity of the diaphragm will induce extreme additional forces due to lateral load, specifically at the edge. Figure 1-1 indicates failure of buildings subjected to strong earthquakes. Because the torsional irregularity is one of the major cause of severe damage in the building in the past few decades (Anastassiadis et al. 1998; Athanassiadou 2008; Chandler and Hutchinson 1986; Humar and Kumar 1998a; Tabatabaei 2011) understanding the behavior of the structure due to torsion is of importance to building owners and decision makers.

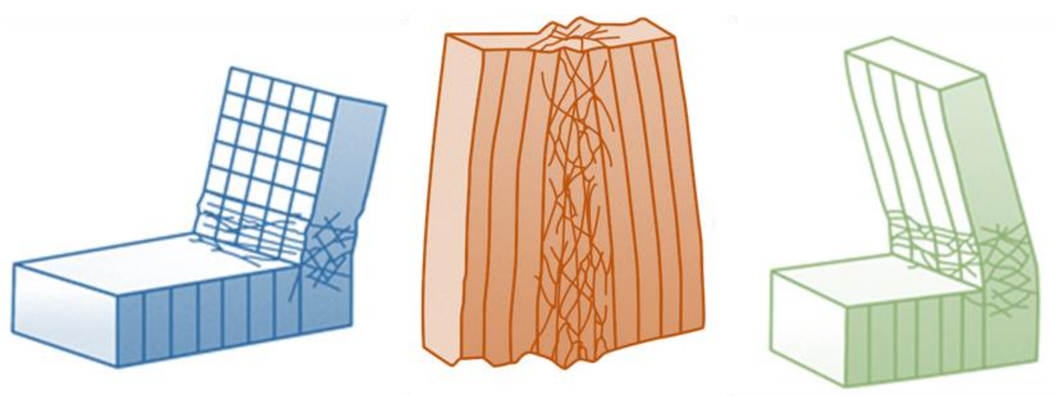


Figure 1-1 Severe damage to irregular buildings subjected to strong earthquakes (adapted from YEATS 2004)

In addition to dealing with material properties, designers often have to consider the geometrical constraints of the buildings. Based on the configuration of lateral supports or other factors regardless of type of structural system, building codes have identified these geometrical specifications as irregularities. Eight categories of irregularities has been considered in the National Building Code of Canada (NRCC1941. 1953. 1960. 1965. 1970. 1975. 1977. 1980. 1985. 1990. 1995. 2005.2010. n.d.)(NRCC 2010) as summarized in Table 1-1.

Table 1-1 Structural irregularities described in NBCC 2010

Type	***Definition***
1.Vertical Stiffness Irregularity	When the lateral stiffness of the SFRS in a storey is less than 70% of the stiffness of any adjacent storey, or less than 80% of the average stiffness of the three storeys above or below.
2.Weight (mass) Irregularity	Where the weight of any storey is more than 150% of the weight of an adjacent storey. A roof that is lighter than the floor below need not be considered.
3.Vertical geometry irregularity	When the horizontal dimension of the SFRS in a storey is more than 130% of that in an adjacent storey
4.In-plane discontinuity in vertical lateral force-resisting element	An in-plane offset of a lateral load-resisting element of the SFRS or a reduction in lateral stiffness of the resisting element in the storey below.
5.Out-of-plane offsets	Discontinuities in a lateral force path, such as pot-of-plane offsets of the vertical elements of the SFRS
6.Discontinuity in capacity- weak storey	Where the storey shear strength is less than that in the storey above
7.Torsional sensitivity- to be consisted when diaphragms are not flexible	**The ratio B calculated according to 4.1.8.11(9) exceeds 1.7.**
8.Non-orthogonal systems	When the SFRS is not oriented along a set of orthogonal axis

1.2 Torsional irregularity

In the past few decades several studies have indicated that in buildings where the center of mass (CM) does not coincide with the center of rigidity (CR), translational and torsional displacement of the floor diaphragm will cause significant rotation in diaphragm plane in an earthquake event (Humar et al. 2003; Lin et al. 2012; Miranda et al. 2012). Figure 1-2 elaborates the definition of CM and CR on a floor diaphragm. Because the earthquake-induced force will apply to the center of rigidity, from Figure 1-2, it can be seen that increase of the eccentricity between the CR and CM could cause significant absolute displacement between extreme edges of the diaphragm.

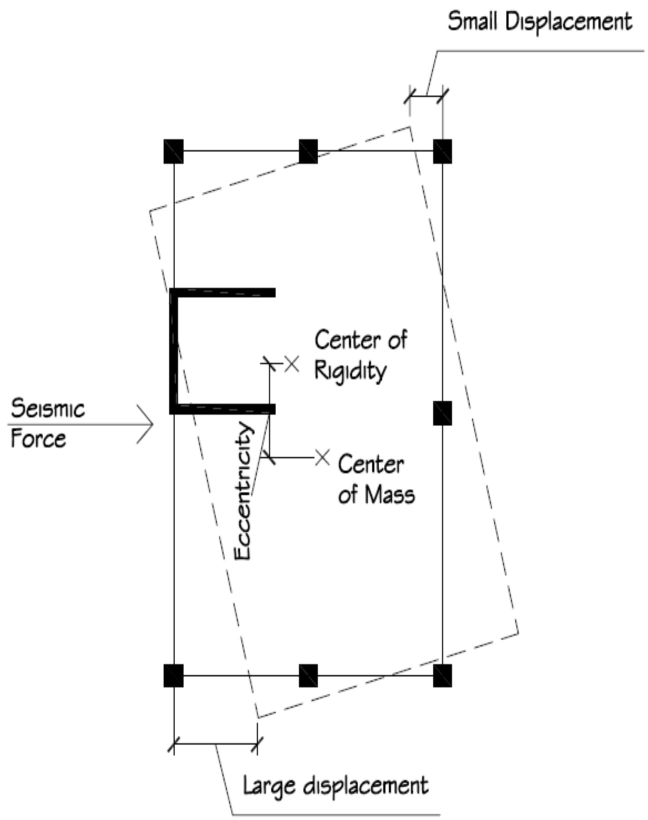


Figure 1-2 Definition of CM and CR on a floor diaphragm

This geometrical irregularity in the floor plan of buildings will cause the rotation in the diaphragm even when the earthquake induces uniform translation at the rigid base (Chandler and Hutchinson 1986; Cruz and Chopra 1986; Humar and Kumar 1998a).

In order to calculate the torsional response of a structure, it is common practice to assume all points of the foundation base are excited due to ground motion simultaneously. Accordingly, if the CR and CM of each floor coincide at the same vertical axis, the response of the structure under the horizontal component of earthquake only induces horizontal-lateral translational force on the floor diaphragm. However, if the CR and CM offset from the vertical axis there will be both horizontal translation and torsional motion upon the ground shaking at the same time. These type of structures are referred as eccentric structures (Humar 1984).

The torsional response of asymmetric structure due to seismic force leads to increase in displacement at the extreme point of the diaphragm. This may cause Seismic Force Resisting Systems (SFRSs) of the structure to undertake additional forces, especially for torsional flexible structures. In addition, the seismic responses of the torsionally flexible systems are different from the value in static loading procedure. Accordingly, building codes usually define a simple expression for eccentricity in order to consider the seismic and torsional response of structures in the elastic range. Several researches has proposed the methods of calculation of eccentricities (Chandler and Hutchinson 1992; Humar and Kumar 1998a; Thambiratnam and Corderoy 1994). A review on some of these researches is shown in the following.

In one of the earlier research Anastassiadis et al. (1998) have conducted dynamic analysis of simplified models and presented simple formulas to account the equivalent static eccentricities. Results from their parametric analyses indicated that torsional and lateral stiffness of the structure play an important role in determining the equivalent static eccentricities. In addition, they indicated that in the past seismic events in buildings where the orientation of lateral load elements were not symmetric, the torsional-translational vibration of the floors has occurred. In other words, torsional-translational vibrations require additional ductility demands on lateral elements. Anastassiadis et al. computed the design eccentricities as

$$e_{max}=e_f + e_0 \qquad (1.1)$$

$$e_{min}=e_r - e_0$$

where e_f and e_r represent the equivalent static eccentricities on the rigid and flexible side, respectively, and e_a represents the accidental eccentricity. However, they did not examine the accidental eccentricity in their research.

Anastassiadis et. al (1998) reviewed the relation between e_r and e_f with respect to e_0 in different building codes. For example they indicated that the definition of e_r and e_f in NBCC 1990(NRCC 1990), where e_f=1.5e_o and e_r=0.5e_0, showing that the strength distribution increases along the stiffness elements. In addition, more efficient ductility demand occurs as compared to the case where e_f=e_r=e_0.

In addition, Anastassiadis et. al (1998) conducted analytical examination in order to determine the torsional eccentricities using modal analysis. The detailed method of calculation of the eccentricities in both directions was explained and finally design eccentricity was introduced. They concluded that the formula on NBCC 1995 (NRCC 1995) only is suitable for the torsionally lateral and stiff structures with rectangular plan. For other type of structures, coefficient for the eccentricity will be dependent on other specifications of the plan such as r (radius of inertia), ρ_k (torsional radius of gyration) and $\mu = \rho_k/r$.

In another research, Moghadam and Tso (2000) have proposed a procedure to determine the center of mass and torsional radius for the static torsional provisions in Euro Code 8 for asymmetric multi-storey buildings. It was shown that, for torsional provisions of the code to result effectively, a minimum torsional stiffness is required. They have shown that, this minimum torsional stiffness can be specified by using the mean stiffness radius of gyration of buildings. However, to calculate the inelastic torsional response for a multi-storey building, they found that the determination of equivalent eccentricities was difficult for each storey and the cause was explained due to variation of the location of center of rigidity in each floor diaphragm.

Fajfar et al. (2005) have studied general development of the inelastic behavior of plan-asymmetric structures. They investigated the effect of seismic components on eccentricity in two orthogonal directions. The pushover analysis using N2 method was conducted to check the target displacement and deformation distribution on the building height. In addition, linear spectral dynamic analysis was conducted to determine the torsional amplifications. In order to verify their proposed pushover-based seismic analysis approach in plastic range, they studied a few test examples that included both single-storey and multi-storey models, as well as a three-storey building.

1.3 Modeling the torsional effect in structures

In order to model the effect of torsion in a structure under earthquake, full three-dimensional inelastic dynamic time history analysis is required (Miranda et al. 2012). Because of the complexity of the inelastic behavior of structures in torsion, the majority of the latest researches have investigated the torsional response of structures in *elastic range* (Macrae et al. 2008). In general, structures are categorized as "*torsionally restrained*" when the combination of an SFRS in two orthogonal directions resists the earthquake, or "*torsionally unrestrained*" when the earthquake force is carried by a SFRS only in-plane of the applied force.

According to Miranda et al. (2012) three main structural conditions can lead to torsional effects in buildings:

1) Eccentricity between Center of Mass (CM) and Center of Rigidity (CR)

2) Concentration of large mass (possibly live load) asymmetrically with respect to stiffness.

3) A combination of the two above could also result in torsional irregularity.

1.4 Torsional effect

Where the center of mass and center of rigidity do not coincide in a structure, even under pure horizontal transitional force, a coupled torsional-transitional motion occurs (Tabatabaei 2011). A study by Humar (2003) revealed that, where torsional and translational frequencies of a seismic response of a structure are close together, the torsional component has no significant contribution on overall response of structure. Humar and Kumar (1998b) past few decades' researches are in disagreement with this result.

Tabatabaei (1998) has investigated the *vibration-based analysis* to model a building that had failed because of the torsional-translation motion in Mexico City earthquake. As shown in

, the springs represented the lateral support stiffness. As the center of rigidity remains fixed, the seismic force induced on that point.

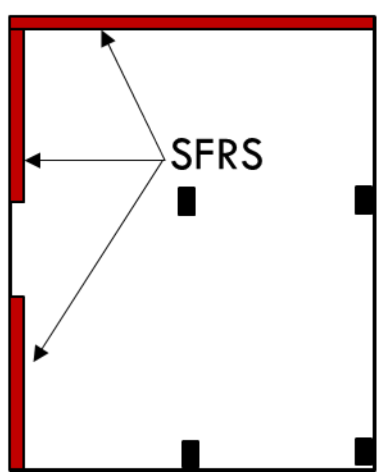


Figure 1-3 Model of one storey torsional irregular building

The torsional response of structure was defined as the frequency ratio $\Omega = \omega_\theta / \omega_x$, where ω_x represents an *uncoupled translational frequency* and ω_θ is defined as *uncoupled torsional frequency* (Tabatabaei 1998). It was shown that translational response was expected when $\Omega > 1$ and the structure classified as "torsionally flexible". On the other hand, the torsional response governs when $\Omega < 1$. Accordingly, when the structure is torsionally stiff, the typical method of elastic displacement calculation can be used by applying the force at the center of mass, as there is only one translational motion in floor. However, for the torsional flexible structure, the displacement is calculated considering the envelope of both translation and torsional mode, which cannot be estimated with elastic method.

1.5 Torsional provisions in building codes

Significant numbers of researches in the past few decades have shown the effect of torsion on buildings. However, most of the studies are inconsistent in the results (e.g. Chandler and Hutchinson 1986; Fajfar et al. 2005; Humar and Kumar 1998a; b; Lin et al. 2012; Macrae et al. 2008; Miranda et al. 2012; Moghadam and Tso 2000; Tabatabaei 1998). This can be due to the complexity of torsional motion and number of effective factors. Accordingly, various building codes have different provisions to design for torsion (Humar et al. 2003). Most building codes consider a formula to account for torsional moment due to the seismic shear force on each floor multiplied by the eccentricity between the center of mass and rigidity. Therefore, the static analysis of the structure will provide the design forces on the structural elements. In some building codes, a multiplier of design eccentricity is used to include possible dynamic amplification of torsion. Figure 1-4 indicates the maximum and average displacement in floor diaphragm on direction of lateral load.

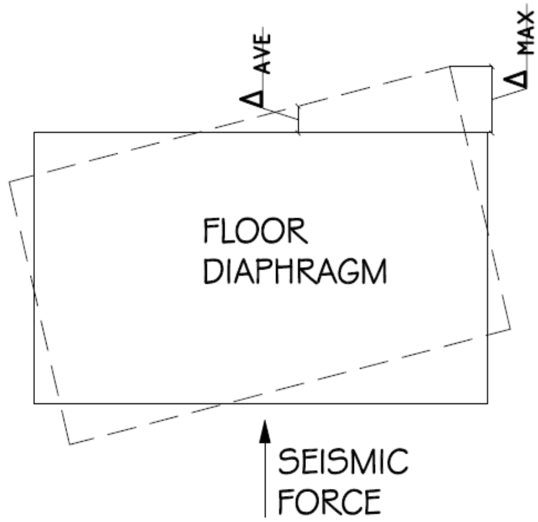


Figure 1-4 Maximum and average displacement of diaphragm

Typically, two parts are defined to explain the design eccentricity equation in building code equations (Tabatabaei 2011):

1- The effect of simultaneous action of two horizontal ground motion that causes the torsion is described by magnification factor times the eccentricity.
2- The accidental eccentricity to account for additional torsion resulting from structural elements deficiency, vibration, dead and live loads, etc.

Tso and Meng (1981) have done an extensive review of torsional provisions in different building codes at the time. By using dynamic response spectrum method they found that for the building with large eccentricities the calculated torsion from the NBC 1977 (NRCC 1997) is almost twice than the calculated by the code recommended formula.

Humar and Kumar (1998a, b) and Humar (2003, 1984) have done significant research on the torsional effect evaluation on Canadian building code during the past few decades. Torsional response of single and multi-storey for both elastic and inelastic response of structure in earthquake were investigated.

In case of symmetric system, the elastic forces resist the inertia forces from the ground motion acting on the center of mass. This leads to a pure translational motion of the structure where the elastic forces are acting on center of resistance. In asymmetric structures, the effect of torsional motion is called natural torsion. Accidental torsion is defined to account for a variety of factors such as mass and stiffness distribution. In addition, the other factor that might need to be considered is rotation of ground, which may cause twisting in the structure. Recent researches have shown the effect of ground rotation is negligible (De la Llera and Chopra 1994).

To define the elastic response of the structure to a given ground motion, similar approach as described in Tabatabaei (1998) is selected by Humar and Kumar (1998a; b) and Humar (2003, 1984). This entails calculating the elastic torsional response by defining translational and torsional frequency (ω_y and ω_θ) and $\Omega_R = \omega_\theta / \omega_y$ as frequency ratio. Using response spectrum analysis it is shown that the design eccentricities e_{fc} and e_{sc} given shown in Eq. 2.2 (NBCC 1995) (NRCC 1995) are overly conservative in flexible side of the structure, but are not adequate in stiff side of the plan especially in the cases that Ω_R <1 with small eccentricities.

$$e_{fc}=1.5e+0.1b$$
$$e_{sc}=0.5e-0.1b \qquad 1.2$$

where e_{fc} and e_{fc} are eccentricity on flexible stiff side of the plan, respectively.

As a result, they suggested new design eccentricities to be considered. Using analytical studies, they have shown the adequacy of the proposed eccentricity with different frequency ratio and plan specifications.

In another study, Humar and Kumar (1998b) have investigated the inelastic response of the structures on torsional motion for both single and multi-storey buildings. Because of the complexity of the behavior especially in multi-storey structures, it was revealed that the proposed provisions could be used for both single- and multi-storey buildings with asymmetric plan.

In addition, the effect of accidental torsion is considered due to uncertainty of the errors in estimation of mass and stiffness. Researchers have shown that these errors can be estimated in the dynamic response magnification by increasing or decreasing the eccentricity by 0.05b (de la Llera and Chopra 1994; Juan et al. 1994) . Although, traditionally it is believed that the second part of the code equation in NBCC (±0.1b) is to account for the accidental torsion. However, Humar (Humar et al. 2003) has the opinion that the interpretation of the code equation is a combination effect of natural and accidental torsion.

They have shown that because of inelastic systems, it can be assumed that some of the elements yield under the torsional motion, and therefore it is possible the center of rigidity and the center of strength not be at the same location. Accordingly, additional parameters are added to account for the effect of yielding element. Analysis has been done for single storey, five storeys and ten storeys building. Inter-storey ductility demand for each of the examples is compared with target ductility and they have shown that the results with the new proposed provision are still in conservative side particularly for the flexible edge. The new provisions are adopted in NBCC 2005 (NRCC 2005) and current 2010 version of the code in calculating the torsion.

In this research the effect of the translational to torsional frequency (Ω_R) vs. the 10 percent eccentricity proposed by Humar fort both the original RC structure and Hybrid structure using CLT will be investigated. The results are shown in Chapter 4.

1.6 Evolution of torsion in NBCC

NBCC 2005 and 2010 (NRCC 2005, 2010) are quite the same in considering the torsional effect of the structures. In this study, the method of calculation of torsion in current edition of NBCC that will be used in this research will be presented. In addition, Table 1-2 indicates a summary of the evolution of torsion calculation in NBCC. NRC (Mitchel et al. 2010) provides the evolution of consideration of torsional effect in the building code of Canada in a comprehensive report.

Table 1-2 Torsional effect evolution in NBCC

Edition	*Description*	*Torsional effect*
1960	Torsional effect was introduced with no guidelines	
1965	Torsional effect considered for the first time	$e_x= 1.5e\pm .05D$
1975	Period formula introduced, Torsion revised	$e_x=1.5e+0.05D$ $e_x=0.5e - 0.05D$
1985	Period revised, Torsional eccentricity from 0.05 increased to 0.1D	$e_x=1.5e+0.1D$ $e_x=0.5e - 0.1D$
1995	Period revised and it could be used from dynamic analysis, Torsional Moment is directly given with coefficient for the eccentricity	$T_x= F_x(1.5e_x+0.1D_{nx})$ $T_x= F_x(1.5e_x-0.1D_{nx})$ $T_x= F_x(0.5e_x+0.1D_{nx})$ $T_x= F_x(0.5e_x-0.1D_{nx})$
2005, 2010	The elimination of the coefficient for e greatly simplified the calculation of torsional effect and torsional sensitivity index is introduced	$T_x= F_x(e_x+0.1D_{nx})$ $T_x= F_x(e_x-0.1D_{nx})$ $B_x=\delta_{max} / \delta_{ave}$

As indicated in Table 1-2, the first change for calculation the seismic force in 2005 and 2010 edition is considering the spectral accelerations with a probability of occurrence of 2% in 50 years (2475-year return period). This lower probability is chosen to provide a close probability of structural failure. It is noted that the *dynamic analysis approach* became the preferred method of analysis and must be used for structures with certain irregularities.

The fundamental lateral period of vibration of a building (for example with shear walls), T_a , in seconds can be evaluated empirically as (Saatcioglu et al. 2013):

$$T_a = 0.05(h_n)^{3/4} \quad 1.3$$

where h_n is defined as total height of the building in meter. Torsional moments are applied about a vertical axis at each floor level to account the torsional effects (Humar et al. 2003):

$$T_x= F_x(e_x \pm 0.1D_{nx}) \quad 1.4$$

where T_x is torsion in diaphragm at level x, F_x is equivalent static seismic force at level x and D_{nx} is dimension of the diaphragm perpendicular to the load at level x. This will allow the designer to account for torsion directly by shifting the mass 10 percent and perform the 3-D modeling.

Table 1-2 summarizes the evolution of code considerations and limitations on torsional effect in NBCC. In the 2005/2010 code, B_x is defined as torsional sensitivity index at each level x:

$$B_x=\delta_{max} / \delta_{ave} \quad 1.5$$

where δ_{max} is the maximum storey displacement at the extreme points of the structure at level x induced by the equivalent static forces acting at distances $0.1D_{nx}$ from the center of mass at each floor, and δ_{ave} is the average of the displacements at the extreme points of the structure at level x produced by the above forces. According to the code, when B_x exceeds 1.7 and $I_EF_aS_a$ (0.2) >0.35 then a 3-D dynamic analysis is required. This is because the static analysis technique has been established assuming a regular distribution of stiffness and mass in a building structure. Therefore, statistic lateral analysis of structure could be inaccurate if the structure does not satisfy those assumptions (i.e., a regular distribution of stiffness and mass). As the torsional irregularity could be very problematic and quite catastrophic in an earth quick event, it has been treated severely in NBCC, which can be lead to a very uneconomic building structure. Therefore, there is a dire need to develop innovative, practical and cost effective techniques to rectify the torsional irregularity in a building structure, without compromising the desired architectural plan and/or increasing the cost of building.

It is necessary to emphasize that these techniques could be more appreciated if they can be implemented in existing buildings in order to rehabilitate their irregularity without a huge amount of construction works and architectural changes. One of the unique solution that has been proposed and investigated in this thesis is hybridization of an existing irregular structure by a *cost effective, lightweight, easy-to-access,* and *least-bulky* structural material.

1.7 Objective of the thesis

In this thesis, the effect of hybridization on the buildings with torsional irregularity (Type No. 7 in Table 1-1) is investigated. In Canada, since the 1960's edition of the building code torsional effect on diaphragms has been considered and the criteria for calculation of this effect was revised until 2005 NBCC where a new index (Torsional Sensitivity Index) B was introduced in the building Code. The Torsional Sensitivity Index, B, was considered as the ratio of Maximum displacement on the extreme corner of the building at each level to the average displacement of all corners under static seismic force at each floor with 10 percent eccentricity. If the value of B is more than 1.7 the building should be considered as irregular and the effect of accidental torsion and irregularity should be calculated. This will add a significant amount of load on diaphragm that should be considered in the design process. Furthermore, dynamic analysis is required to determine the base shear and period of the structure.

This study aims to investigate the effect of hybridization with the CLT (Cross Laminated Timber) panel on a four storey concrete torsional irregular structure. The performance of the structure is evaluated by determining the inter-storey drift and base shear using Linear Dynamic Analysis (LDA) and Non-Linear Time History Analysis for a building located in Vancouver, BC, Canada. In addition, the performance prior to occurrence of first plastic hinge for different performance levels IO, LS and CP according to FEMA 356 guidelines is investigated.

1.8 Organization of the thesis

This thesis is arranged in five chapters as outlined below.

In ***Chapter 1***, the effect of torsional irregularity under seismic loading on the structures is discussed. The methods of considering the torsional effect and the main cause of that is explained. A comprehensive literature review of analyzing the torsional irregular buildings and code provision formulas is conducted.

In ***Chapter 2,*** hybridization is introduced as a method of mitigating of the structure weakness in both gravity and lateral performance. Literature review on background and use of hybrid structures is done. Performance Based Earthquake Engineering (PBEE) is reviewed to elaborate the performance of the structure at different level and determine if the building satisfies the code performance requirements. The specification of Cross-Laminated Timber (CLT) as a product that recently have been introduced in North American market is reviewed. The hysteresis behavior of CLT by means of steel brackets under cyclic loading is explained based on the researches done to date. Hybridization with CLT is suggested as a new method to reduce the torsional sensitivity in the structures.

In ***Chapter 3***, the building under study is described. The procedure of designing the structural components for gravity and seismic loading is shown. SAP 2000 is introduced as analytical software in order to investigate the dynamic behavior of the structure. The method of calculations the torsion, period of the building is explained. The method of scaling the selected ground motions and the procedure of implication on the modeling program SAP2000 is explained. Furthermore, the procedure to account for the non-linear behavior of CLT panel connections is described. Linear Dynamic Analysis is used to demonstrate the behavior of the structure under design spectra for Vancouver site class C for the original and hybrid model. The change of the seismic base shear following the NBCC 2010 provisions for Equivalent Static Force and dynamic analysis is shown. In addition, Non-Linear Time History Analysis is used to evaluate the seismic performance of each structure.

In ***Chapter 4***, the analysis results for both type of selected analysis method is presented. Comparison between the analysis results and values from NBCC is shown. The performance of the structure under the subjected ground motions is evaluated. In addition, for the purpose of verifying the analysis method a parametric study is conducted in order to find the optimum location and length of the CLT panel in the building plan.

Finally, in ***Chapter 5,*** the summary of the research and the main conclusions observed in Chapter 4 is presented. In addition, the limitation of the current study and the suggestions for the improvement of the future work is described.

2 Chapter: Retrofitting of Buildings Using Hybrid Systems

In order to rectify the deficiency of the buildings in torsional motion several considerations could be made. For the new construction the building has to be designed with the seismic criteria that have been defined in the latest building code. Often this will significantly increase the size of the members and accordingly the cost of construction due to higher lateral force. Therefore, it is preferred by designers to reduce the effect of torsional motion as much as possible. One of the simple methods to reduce the effect of torsion is changing the floor layout, to move the lateral supports, in order to coincide the CM and CR. However, this method is not practical most of the time because of the architectural limitations. The other option for the existing buildings is to add lateral supports for retrofitting of the existing building to upgrade the structure to the current code. Accordingly, especially for the latter other consideration shall be made in order to rectify the torsional problem and upgrading the performance of the structure. In this chapter hybridization and hybrid structures is explained as a method to solve this problem.

2.1 Hybrid structures

Hybridization can be achieved by combination of different materials such as steel, concrete and wood. Hybrid structures, by combining different construction materials: steel, concrete and wood, have been used in North America for a long time. In recent years hybrid structures have become an efficient method in design of structures. Although, the steel and concrete combination has been utilized for different structural elements (Zona et al. 2008), hybrid wood structures with other material came to interest only in the last two decades because of special characteristics of wooden materials, e.g. (Asiz and Smith 2011; Clouston and Schreyer 2008).

Combination of steel and concrete has been used in different structural elements as each material has exceptional strength and performance (i.e. concrete is strong in compression and steel is good in both tension and compression). In addition, both materials have the same thermal coefficient which is an important benefit in hybridization.

Steel composite deck with concrete topping, steel composite beams, steel encased concrete beam and steel connections for precast concrete panels are some of the examples of using these valuable materials together (Asiz and Smith 2011; Fauzan and Kuramoto 2011; Lindt et al. 2009). Steel and wood connections have been developed in recent years at different component of structures. However, using steel fasteners such as nail and screws have a deep root in structure history. One of the common examples of hybridization of wood and steel is the steel moment resisting frame system in wood frame buildings that has been used widely for construction of multi residential housing (Aziz 2007; Dickof et al. 2012a, 2014).

Three categories can be identified in hybridization; component level, system level and building level (Dickof et al. 2012). Every structure is subjected to two main types of loads; permanent loads and rare loads which can be interpret to gravity and lateral load. Hybridization can provide additional strength to resist both gravity and lateral loads. There are different hybrid systems for resisting the gravity loads. Some of the examples of the use of steel in wood construction at the component level are steel and wood connection hangers in housing construction in North America; or some hybrid steel and wood bridges in Quebec and Northern Ontario with the timber decking and studs with concrete topping (Krisciunas 1996).

As an example for hybridization at structural level, Fauzan 2011 has conducted both analytical and experimental studies on Engineering Wood Encased Concrete-Steel (EWECS) composite columns. Four specimens have been tested and hysteric behaviors of the columns were found satisfactory for at large inter-storey drift. As a result of using the woody shell as forming during the construction, the cost of labor has been decreased. Therefore, the system provided both economical and structural benefit.

2.1.1 Hybrid steel - wood structure

Several researches have been conducted in order to investigate the behavior of hybrid structures. The performance of seven storey hybrid steel-wood structure with steel frame and light wood frame shear walls through pseudo static experiments has been investigated by He and Li (2012). They have calculated the preliminary data such as strength, hysteresis behavior and rigidity from experimental results on one storey specimen to model the seven-storey building. Monotonic loading and cyclic loading was applied on steel frame with and without wood shear walls and diaphragms. They have shown that the seismic performance of the structure has increased because of using light wood frame shear wall and wood diaphragms in combination of the steel frame proportionally. In addition, the result from experimental analysis showed that the failure in hybrid wood shear walls under cyclic loading occurred in ductile fasteners (nails) whereas at the non-hybrid single wood shear wall usually the failure will happen at the end studs. Moreover, they found that the seismic base shear has decreased, because of using light wood material. In order to understand the response of the system under cyclic loading Equivalent Energy Elastic Plastic (EEEP) curve according to ASTM E2126-09 was used by (He and Li 2012) to define the hysteresis behavior of hybrid timber-steel structure. Figure 2-1 indicates EEEP curve defined for the wood shear walls.

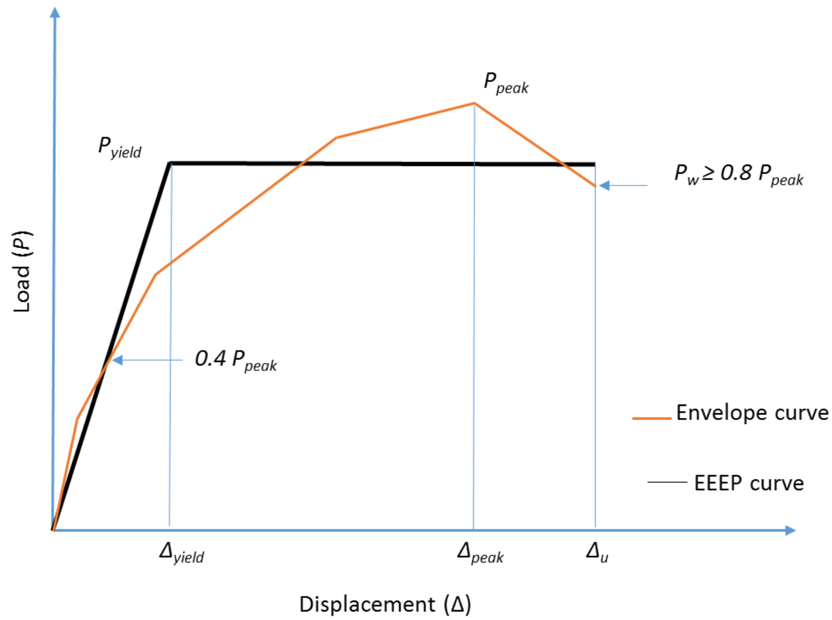


Figure 2-1: EEEP curve (One storey specimen)

2.1.2 Hybrid wood–concrete floor system

Wood-Concrete Composite (WCC) has been used around North America since early 1930's in timber bridge decks. The behavior of WCC floor systems has been investigated in composite decks by Clouston and Schreyer (2008). In that study they examined the effect of hybridization in component level. It was shown that the strength of wood beam with concrete deck has significantly improved two to four times by using shear connectors. The composite mechanics analysis was adapted from Euro Code 5 for the analysis of the shear connectors. It is shown that main advantage of this system is the composite action where the concrete slab is in compression and the wood beam is in tension that is desirable. In addition, the weight of overall section has shown to be less and more cost effective comparing to the conventional concrete beam and slab system. Several use of this hybridization method is shown to improve the vibration and deflection performance of the wood floor of existing buildings. In that study, several common connectors were described such as the HBV connector (Figure 2-2) from GmbH Company from Germany. The HBV connectors are steel mesh that is glued half in to the wood and half in the concrete.

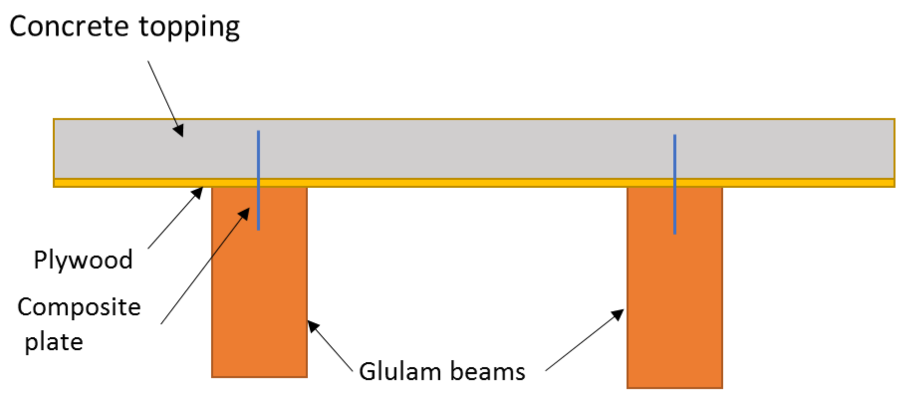


Figure 2-2 Typical wood–concrete composite floor

2.1.3 Hybrid wood-steel and concrete structure

The use of hybridization of wood and steel in members, connections and structure has been reviewed by Koshihara et al. (2009). A practical table for classification of timber based hybrid members was developed and the structural performance and fire resistance of five storey hybrid steel–wood building in Kanazawa City (Japan) was studied. Every hybrid structure like other buildings requires definitions of its main components to withstand the gravity load as well as lateral force. Furthermore, it should maintain its stability in case of fire. In order to provide a system to satisfy both mentioned requirements, the beams were consisted of steel plate confined with timber beams on each side. Drift pins provided the connection between the plate and timber. In order to avoid the timber to carry the gravity load, a 3 mm gap considered between the wood and solid steel square rod for columns. It was shown that the steel column did not buckle under the axial load because of glue laminated timber confinement. For the lateral braces same section as columns were considered except the steel was consisted of three-layer plates. Table 2-1 shows the stiffness ratio between steel and wood hybrid section.

Table 2-1 Comparison of Flexural stiffness ratio between timber and steel frame

Material type	***E (N/mm^2)***	***I (mm^4)***	***EI (Nmm^2)***	***EI/ΣEI***
Timber frame	1.05×104	5.55×108	0.583×1013	0.366
Steel frame	2.05×105	4.95×107	1.01×1013	0.634

RC shear wall for the first floor and steel braces confined with wood were considered from the second to fifth storey to create the seismic force resisting system (SFRS) for five-storey building. In order to transfer the lateral shear of the connection of the concrete floor slab to the braces was provided by lag screws (Figure 2-2). The results have shown the satisfactory performance of the hybrid structure under seismic and wind load. It is necessary to mention that, the structure of this building was the first timber structure in Japan with one hour fire rating because of hybridization of steel and wood (Koshihara et al. 2009).

2.2 Performance based seismic design

Often the main design objective in building codes is providing the life safety in moderate and minor earthquakes and preventing the collapse in major earthquakes. After several major earthquakes during the 1990's such as Northridge and Kobe, the need for more realistic based design of the structure has become in to attention to mitigate the repair cost and loss of the use of the building (Ghobarah 2001).

Several types of damages can be expected at earthquake event, which are defined as effective factors on structure vulnerability. These damages can be in the range of life fatalities, economic losses, cultural harms, political damages etc. There are specific seismic criteria that should be considered in the design process to prevent the above-mentioned damages such as; soil types, peak acceleration, duration, distance from earthquake faults etc.

Performance based seismic Design (PBSD) has been introduced as a new approach for the earthquake resistant design of structures. The purpose of PBSD is to predict the seismic performance of the structure with predefined acceptable performance objectives such as life-safety, collapse prevention, or immediate occupancy rather than following the code empirical formulas. Two leading guidelines on this subject, which extend the limit state design to cover the complicated issues that the design engineers facing, are ATC-40 and FEMA 273/274. In addition ASCE-46 and FEMA 356 are introduced to provide guidelines for the seismic rehabilitation of the structures. In order to have better understanding of the philosophy of PBSD the definition of performance objective is necessary.

In general, there are two essential parts for a performance objective, which are defined as *damage state* and *level of seismic hazard*. Seismic performance is described by designating the maximum allowable damage that states the seismic hazard level (earthquake ground motion). A performance consideration of damage states for several levels of ground motion could be termed a double or multiple-level performance objective. The target performance objective can be divided to *Structural Performance Level* (SP-n, where n is the designated number) and *Non-structural Performance Level* (NP-n, where n is the designated letter) which can be studied independently. However, the combination of the two determines performance level. Structural performance levels are categorized in following six performance levels as summarized in Table 2-2.

Table 2-2 Building performance levels

Non-structural performance levels	Structural Performance Levels					
	SP-1 Immediate Occupancy	**SP-2 Damage Control**	**Sp-3 Life safety**	**SP-4 Limited Safety**	**SP-5 Structural Stability**	**SP-6 Not considered**
NP-A Operational	1-A Operational	2-A	NR	NR	NR	NR
NP-B Immediate Occupancy	1-B Immediate Occupancy	2-B	3-B	NR	NR	NR
NP-C Life Safety	1-C	2-C	3-C Life Safety	4-C	5-C	6-C
NP-D Reduced Hazards	NR	2-D	3-D	4-D	5-D	6-D
NP-E Not Considered	NR	NR	3-E	4-E	5-E Structural Stability	Not Applicable

Legend

	Completely referenced Building Performance Levels (SP-Np)
	Other possible combinations of SP-NP
NR	Not recommended combinations of SP-NP

2.2.1 Seismic performance of hybrid structures

Performance Based Seismic Design (PBSD) was used by (Liu et al. 2008) to verify the hybridization effect on a seven storey structure. The structure was consisted of steel moment frame (SMF) on the first floor and the six floors above is light wood construction. The effective stiffness of steel frame determined by correlation of inter storey drift and desired performance. Numerical analysis was conducted using SAPWood program to model the floor details and shear wall configurations. Based on the experimental data initial stiffness K_0 and post yield stiffness $1/8K_0$ for the SMF was considered. Incremental dynamic analysis was performed using twenty earthquakes ground motion and three different hazard levels from ASCE41-06 for performance requirements was considered (Table 2-3).

Table 2-3 Required performance for SMF in ASCE 41-06 (Liu et al. 2008)

	IO	***LS***	***CP***
SMF	0.70%	2.50%	5%
Wood	1%	2%	3%

In addition, the drift requirements for NEESWood project were chosen for evaluating the performance of wood frame system. It was shown that the increase in the stiffness of the SMF will increase the peak drifts in the first storey of wood frame, but will decrease the drift in peak point of SMF.

In another research, Miranda et al. (2012) have investigated the possibility on decreasing the displacement demand in one storey building's shear walls by increasing the strength and stiffness of non-critical elements. They have shown that the increase in rotational mass or decrease the stiffness eccentricity by using non-critical wall elements would decrease the demand of critical wall elements. They indicated that the results for the multi storey building are similar to one storey model.

As a result, considering all the effective factors and the results from the converting the non-structural components (especially walls) to structural component, as an innovative solution, hybridization can be used to reduce the effect of torsion in structures considering for both new design and retrofit of the structure.

In another research, He et al. 2011 have investigated seismic performance of using wood panel diaphragms in a six-storey concrete moment frame (R/C frame). In that study, two types of rigid and flexible diaphragms were considered for structures (He et al. 2011).

Using SAP2000 to model the diaphragm stiffness, they have shown that the wood diaphragm would behave between rigid and flexible diaphragm when it is used in combination of RC frames. They concluded that this could result in reducing the load on the lateral and foundation design.

2.3 Cross Laminated Timber (CLT)

Cross-Laminated Timber (CLT) (Figure 2-3) is developed initially in Austria and Switzerland in early 1990s. However, CLT is a new building system in North American construction. CLT is a cost competitive wood-based solution that complements the existing light and heavy frame options and is a suitable substitute for some applications that currently use concrete masonry and steel. Most publications is currently are based on European experience which has originally developed in Switzerland in 1990’s. Since 2000s because of the green building movement in Europe, CTL has gained more popularity in construction industry. The European experience showed that using CLT could be competitive with other construction system especially in mid-rise and high-rise buildings.

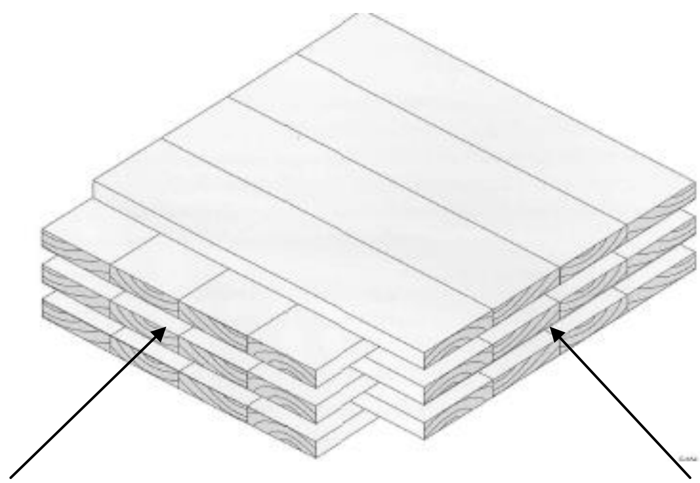


Figure 2-3 Cross panel configuration

2.3.1 CLT hybrid structures

CLT is an innovative and cost effective building system in North America (Asiz and Smith 2011). CLT is a heavy timber structural component that consists of a minimum of three cross wise layers of (usually perpendicular) wood panels that are glued together along their wide face (Pei et al. 2012) as indicated in Figure 2-3. Several researches have showed acceptable strength and stiffness of the CLT panels (Stürzenbecher et al. 2010). Recently, the behavior of CLT has been investigated under cyclic loading (earthquake) by few researchers. The satisfactory seismic performance of hybrid structures depends on the stiffness and strength of the material that used in the main structural components i.e., walls, frames, and connections. The implementation of steel and concrete hybrid systems have a well-known background and their different structural components have been studied and experimented (Shen et al. 2013). CLT has been frequently used in the multi storey buildings (up to 10 storeys) in past few years (Michael Green tall building report). It is known that the acceptable performance of CLT panels under seismic loading in addition to the lightweight will bring the benefit of the combination of this material with other structural materials. The combination of CLT walls with concrete core shear walls for design of the sky scrapers up to 150 meters has been analyzed by (Van De Kuilen et al. 2011). The result indicated that the hybridization of concrete with CLT is feasible.

2.3.2 Seismic performance of CLT

In Canada, FPInnovations launched a multi-disciplinary research program on CLT in 2005. However, CLT is not identified as a Seismic Force Reduction System (SFRS) in the current edition of the NBCC. In order to understand any material's response under cyclic loading it is necessary to model the hysteresis response of that material. Numerical studies and experimental results have shown that the Pinching4 hysteretic model is the suitable model to identify the hysteresis curve for CLT panels (Shen et al. 2013).

Pei et al. (2013) developed design modifications for three multi –storey structures and calculated a possible range of ductility factor (R_d). They have used the inter-storey drift to define the required seismic performance in high seismic area such as Vancouver. The Ten-parameter hysteretic model developed by a series of tests is used for the modeling of the connections (Pei et al. 2012). In most cases, steel brackets or other types of steel fasteners such as fastening screws or strapping (Figure 2-4) indicates the connection between a CLT panel with other panels or other structural materials. They have concluded from the previous researches that the seismic modification factor for a CLT panel is a function of ductility of the connection (Asiz and Smith 2011; Ceccotti and Sandhaas 2013; Pei et al. 2012; Shiling et al. 2012; Rinaldin et al. 2013). As a result, the lateral resistance of CLT panel is calculated by summation of load-slip resistance of the connectors that are contributing to the rocking motion of stiff CLT panel.

Pei et al. (2013) have used equivalent Static Force Procedure (EFSP) according to NBCC 2010 on three 6-, 10- and 15-storey buildings with same plan configuration with different R_d values that identified in the NBCC from 1.5 to 4. The over-strength factor R_o was considered as 1.5 same as the heavy timber factor in the code and it is assumed that the buildings are located in Vancouver seismicity. By using SAPWOOD program for all 18 models the shear demand and capacity ratio of each building at each performance level were calculated. In addition, the absolute maximum inter-storey drift of any storey for all the building models with different R values was found by conducting nonlinear time history analyses with series of 22 bi-axial ground motion scaled to Vancouver design spectra. They have concluded that if a design with certain R-value can withstand the majority of the 22-ground motion can be assumed as appropriated (Pei et al. 2013). As a result, a ductility factor of 2 is suggested.

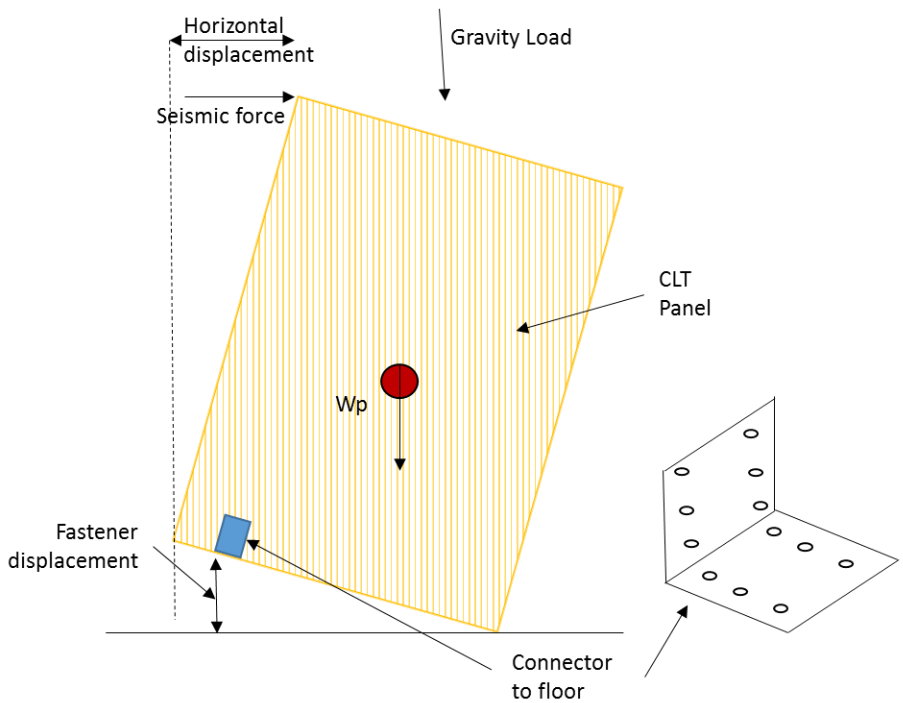


Figure 2-4 Typical CLT wall connection configuration

Rinaldin et al. (2013) have conducted a numerical model to define the hysteretic behavior of connections in CLT structures. The study assumed that one of the brackets, screw fastenings or long steel strapping has provided the connection between the wall panels in each floor to the foundation. They have done an extensive review on the previous methods of consideration of hysteresis behavior of the connections. They have suggested a new component approach to define the nonlinear multi spring elements. The numerical results were verified with a series of experimental results.

Another study by Fragiacomo et al. (2011) has defined the CLT panels as elastic shell element. They modeled the CLT panels as multi-linear elastic springs. This approach can be used in SAP 2000 software to calculate the monotonic response of structure and nonlinear pushover analyses (Figure 2-5).

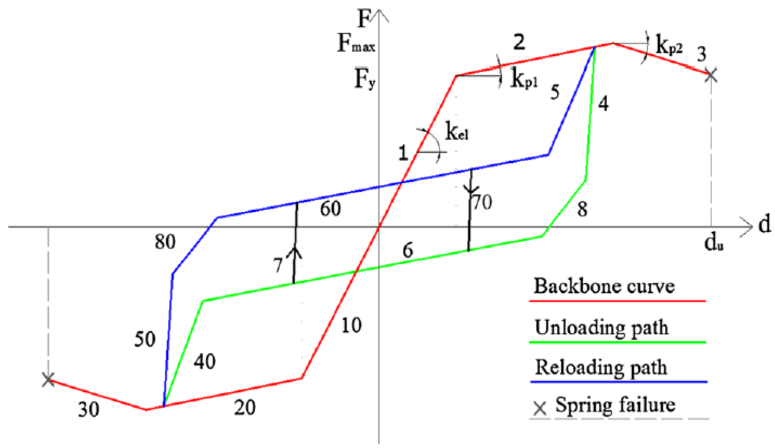


Figure 2-5 Piecewise-linear law of shear spring component

Nakashima et al. (2014) have conducted a series of testing on the effect of CLT panels in concrete and masonry buildings under cyclic loading. The quasi-static cyclic loading is conducted using shaking table test for concrete, with and without infill panels. Several advantages of using the CLT panels such as low mass of timber panels which do not have much effect to the seismic force, insulation value, openings for doors and windows and less interruption during the retrofit is mentioned. The case study structure is a three storey existing concrete building in southern Europe which has been designed only for gravity loads. Dynamic analysis and full scale test has been conducted on the building. Response spectral analysis and pushover analysis based on the N2 method have been used to determine the capacity of the structure prior to retrofitting. Two options were considered to use the CLT panels for retrofitting. The short CLT panels in exterior face did not show a good performance. This is because the behavior of the panels was mostly in bending. Therefore, significant deformation occurred in the connections to super structure. However, the long panel with more connections has shown almost 90% increase in allowable ground acceleration. In addition, experimental results have indicated that the combination of CLT with RC frame would increase the frequency of the frame and accordingly the building would be stiffer. Furthermore, the storey drift were reduced up to 30% under same ground motion.

2.4 Effect of CLT hybridization in torsional irregular building

In this chapter, hybridization in structures as an advanced method to improve the performance of the building is explained. Several researches using the hybridization at different structural level is reviewed. PBSD is described and method of calculation of performance of the buildings. Cross-Laminated Timber is described as a wood based product that has shown appropriate performances in in-plane lateral strength in several researched. The ductility of this component using steel connectors to the building has been studied and the cyclic behavior of this component is studied. However, there has not been much research about the use of hybridization with CLT to rectify the different type of irregularities. Accordingly, in the next chapter the use of this material in a torsional irregular structure is studied in combination with concrete shear wall as lateral support.

3 Chapter: Building Description and Methodology

A four-storey reinforced concrete structure is selected for this structure. The building plan is a common type structure used around Canada with concrete frame (columns and beams) and cast in place concrete slab. In order to provide maximum flexibility from architectural point of view the lateral support (concrete shear walls) has been concentrated at the core of structure around the stair well or elevator shaft. It is assumed that the building located in Vancouver, BC, which is considered as high seismic area.

3.1 Building description

The case study four-storey building has an overall height of 14.4 m (48 ft.). All the storeys height is 3.6 m (12 ft.). The building is essentially square (18 m × 18 m) in shape and has 3 equal bays in each direction. The spacing between each bay is 6 m (20 ft.) in both directions. The shear wall core is considered 3 m (10 ft.) in each direction. One side of the wall has been left open for the doorways. Figure 3-1 and Figure 3-2 show the plan and elevation view of case study building.

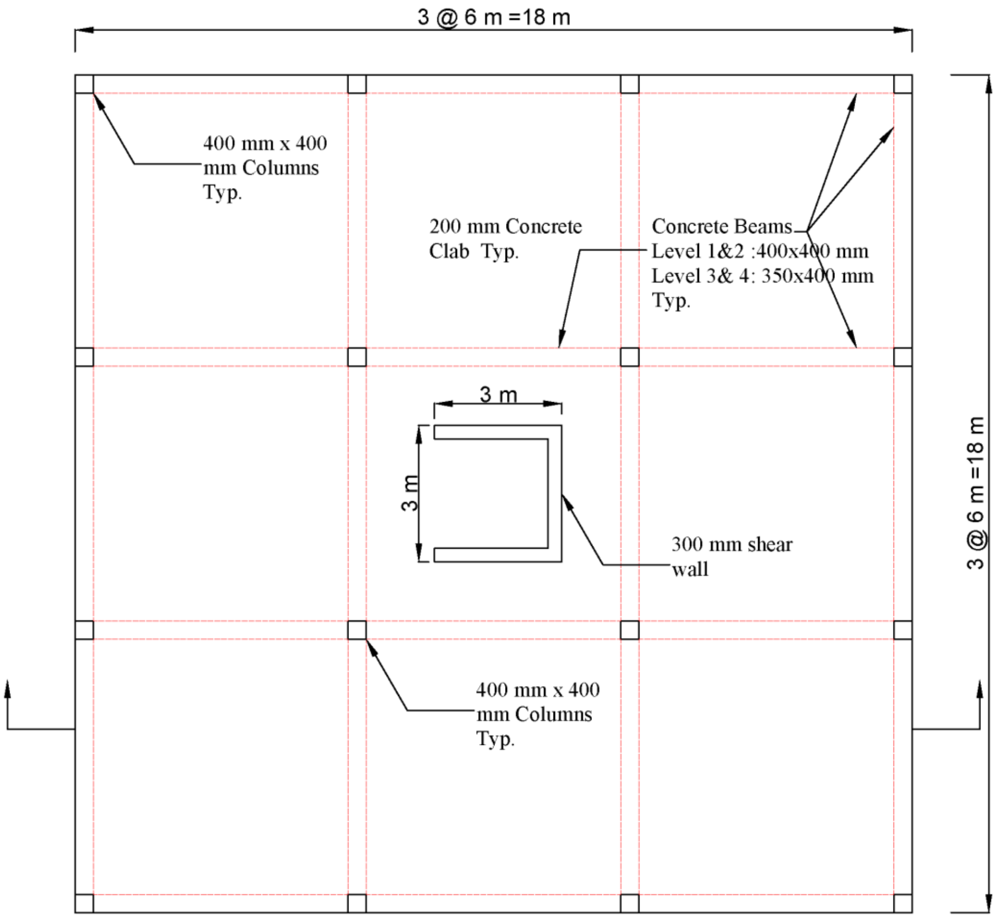


Figure 3-1 Plan view of the case study building

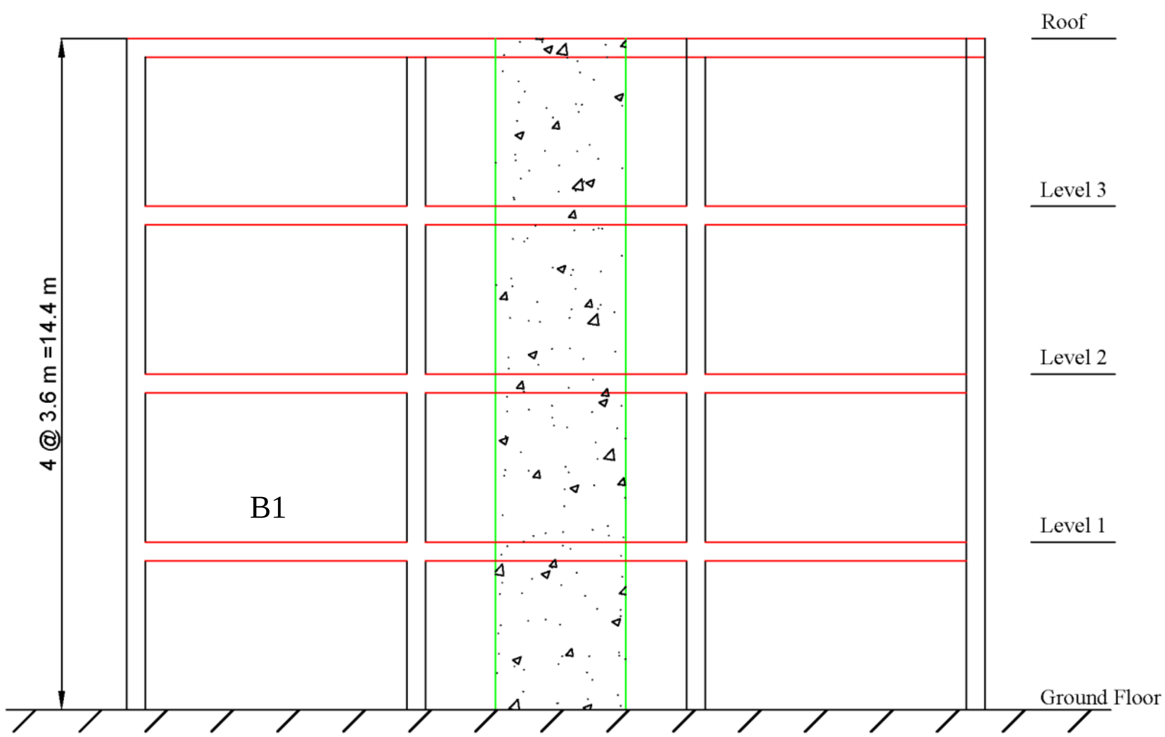


Figure 3-2 Elevation of the case study building

3.2 Design of the building structure

The case study building is designed according to the load requirement of National Building Code of Canada (NRCC 2010) and Design of Concrete Structures (CSA A23.3-04) (CSA 2004). The building is designed for the climatic data given for the city of Vancouver. Site classification C is assumed for the determination of acceleration and velocity factors in order to determine the base shear. The Seismic Force Resisting system (SFRS) of the structure is considered as Limited ductility concrete shear walls using Table 4.1.8.9 of Part 4 of the building code. Accordingly, the values for the ductility factor (R_d=2) and over-strength factor (R_o=1.4) were used from that table for the analysis. In addition to the self-weight of the structure, 0.5 kPa of partition load on the floor area was considered. Because the building is an office space, for the live load calculation 2.4 kPa was used on the floors. For the design of the roof members 1.6 kPa snow load for Vancouver was used.

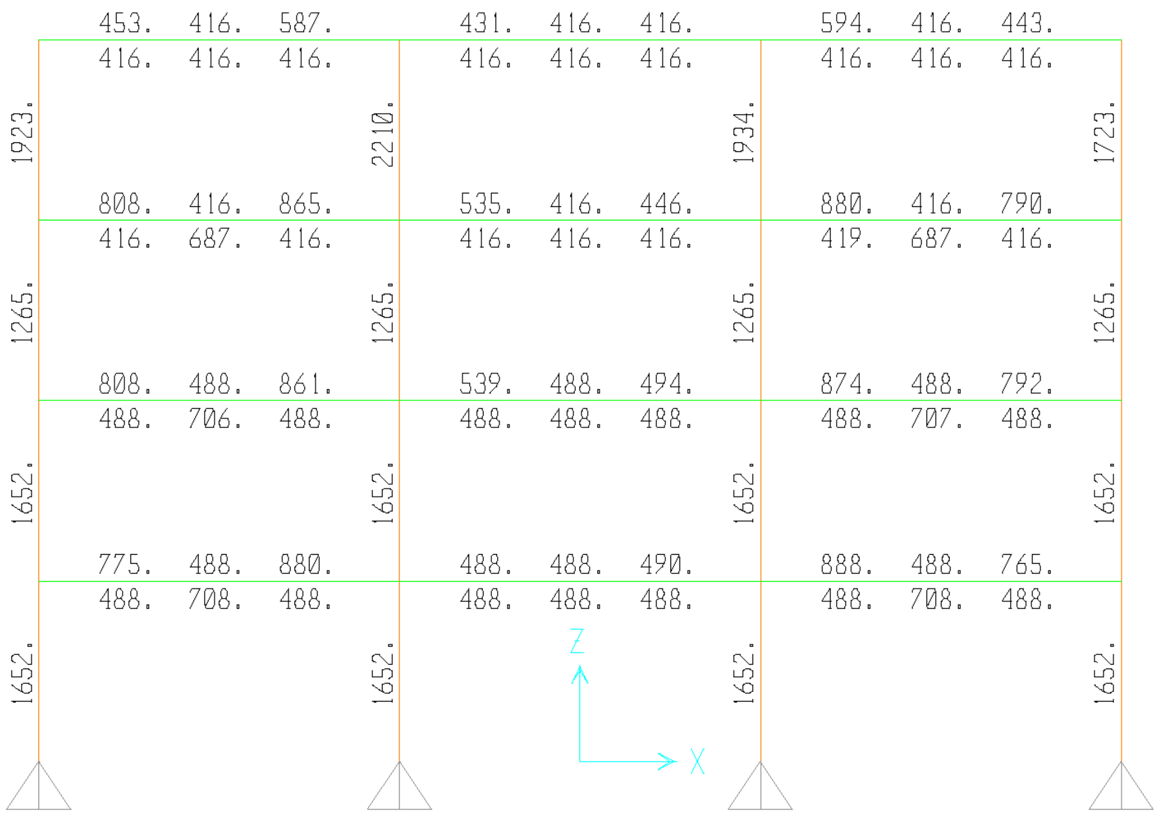


Figure 3-3 Reinforcement area (mm^2) required for the beams under the LC (1D+1ELX+0.5L)

The preliminary design was conducted using Excel spreadsheet for individual members (see Appendix B). Full modeling of the structure in SAP2000 program is done in order to calculate the reinforcement requirement and section sizes due to different load combination required in NBCC. Figure 3-3 shows the reinforcement requirement on the beams and columns in one of the elevations. It should be noted that in this study, the effect of wind load is not considered and it is assumed that the seismic load will govern the design for the lateral load.

Figure 3-4 shows the cross sectional area for floor slab, columns and Beam B1 for illustration.

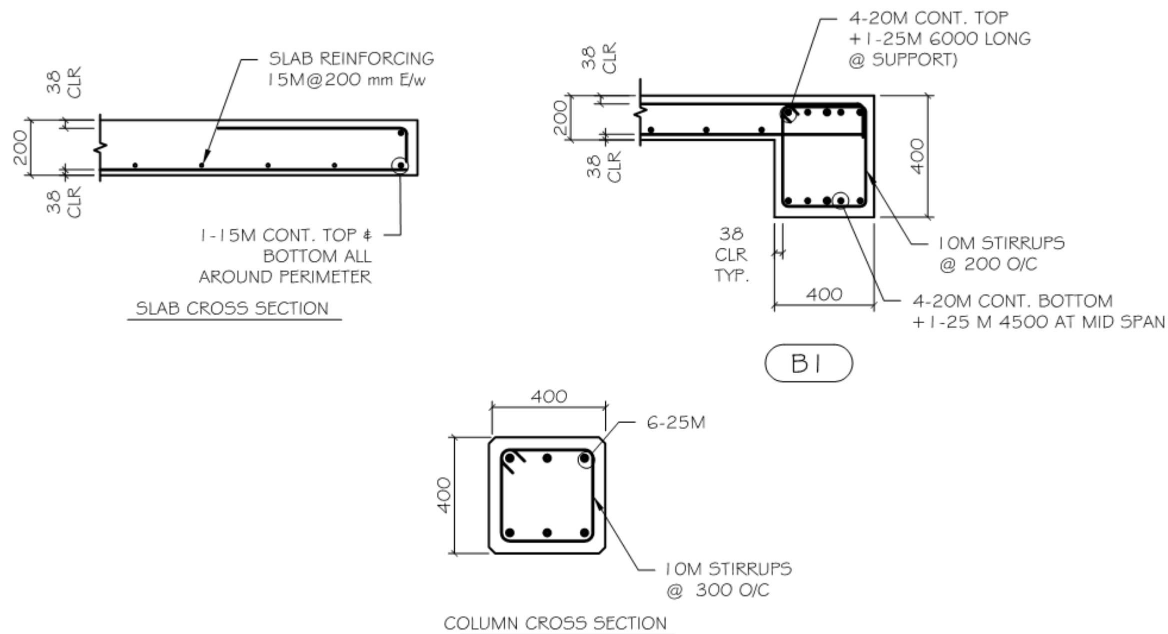


Figure 3-4 Reinforcement in floor slab, columns and beam B1

3.3 Analysis methods

According to NBCC and most common building codes, it is necessary to calculate the eccentricity between the center of mass and center of rigidity for each floor in order to account for the effect of torsion. The total torsional seismic force on each floor in NBCC 2010 is accounted with Equation 1.4. Furthermore, a limitation is defined for the ratio of maximum deflection to the average deflection for each level ($B_x=\delta_{max} / \delta_{ave}$). If the torsional sensitivity index B_x is bigger than 1.7, in cases where seismic intensity, $I_E.F_a.S_a$ (0.2) > 0.35, dynamic analysis is required to determine the design seismic base shear. The dynamic analysis procedure can be done for:

a) Linear dynamic analysis by either modal response spectrum analysis (RSA) or linear time history method.
b) Non-linear dynamic analysis.

Both methods are described in the following subsections.

3.3.1 Linear dynamic analysis

Linear dynamic analysis or response spectrum analysis is based on the modal response of the structure under certain design spectra for specific rotation. It is important that the sum of the mass in different modes calculated in specific direction to be more than 90 percent of the mass of the structure. There are two main common methods to calculate the modal combination response is CQC (Complete Quadratic Combination) and SRSS (Square Roots of Some of Squares) methods (Zhou et al. 2004) . In this research, the SRSS method was used which assumes that all the maximum modal values are statically independent. Figure 3-5 indicates the design spectra for Vancouver site class C.

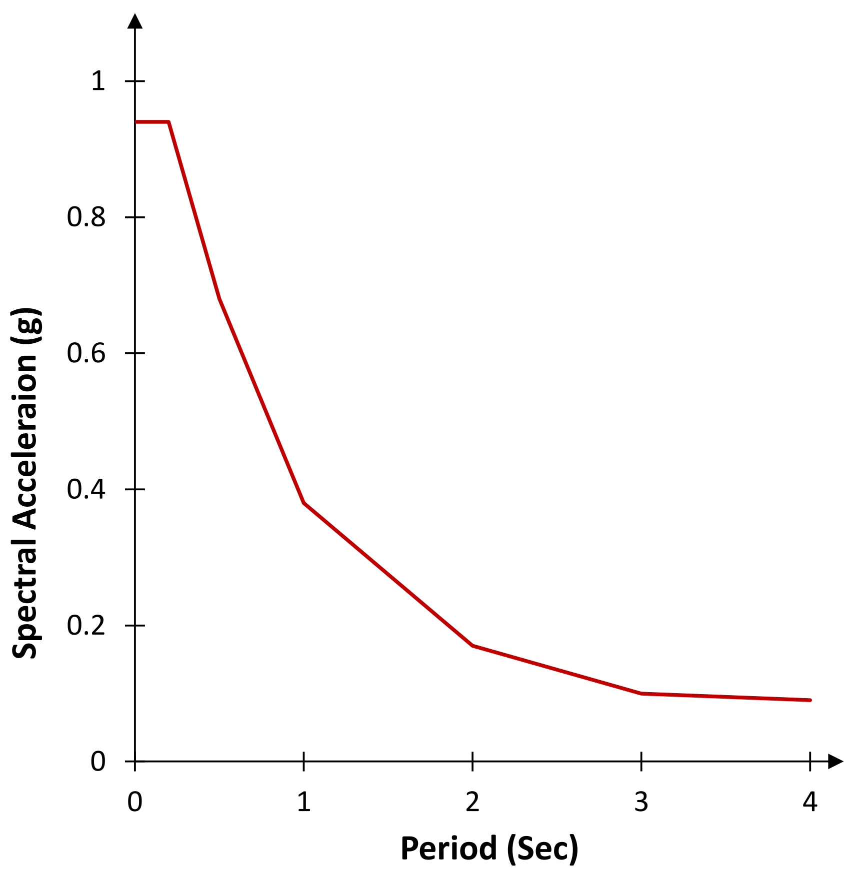


Figure 3-5 Vancouver design response spectra for site class C

The architectural plan of the building shows the shear wall core located at the center of the building. The seismic base shear of the building is calculated according to part 4 of NBCC 2010 requirement for linear dynamic analysis as summarized in Table 3-1.

Table 3-1 modal analysis period for 8 mode shapes and response spectra base shear from dynamic analysis with SRSS combination

Mode	***Period (Sec)***	***RSX/(R_d*R_o) (kN)***	***RSY/(R_d*R_o) (kN)***
1(tor)	1.134	0	90
2(x)	0.379	4806	0
3(y)	0.331	0	-2732
4(tor)	0.277	0	-1626
5(tor)	0.222	-30	1
6(y)	0.220	16.4	0
7(x)	0.218	5	0
8(tor)	0.217	0	0
SRSS combination		1714	1624

3.3.2 Period, base shear and torsion calculation based on NBCC 2010

The building code has empirical formulas to calculate the approximate periods of each type of structures such as moment frames, brace frames, shear walls. Table 3-2 indicates the period calculation of the building under study.

Table 3-2 Period calculation based on NBCC 2010

h_n	***12***	***Meter***	***Height above the base to level n***
T_a	0.3	sec	Shear wall
T_a (all.)	0.6	sec	C.4.1.8.11.3.c

In addition, equivalent static procedure is a typical method to calculate the seismic base shear in most of the building codes. Table 3-3 is a summary of seismic calculation in NBCC 2010 (NRCC 2010).

Table 3-3 Static seismic base shear

M_v=	*Sa(0.2)/Sa(2.0)*	*5.8*	*> 8*	*Factor to account for higher mode effect*
	M_v=	1		Table 4.1.8.11
	W=	8244	kN	
V_{min}=	$S(T_a)M_v$	0.52667		4.1.8.11.2
V_{min} =	$S(2.0)M_v$	0.17		4.1.8.11.2b
V_{max}=	2/3 S(.2)	0.653		4.1.8.11.2c
	I_e=	1		
	R_d=	2		
	R_o=	1.4		
	V_s=	**1153**	kN	Code static base shear

In order to calculate the effect of torsional motion and accidental torsion, first the center of mass and rigidity in each floor was calculated. The torsional sensitivity index (B_x) was calculated from the equation ($B_x=\delta_{max} / \delta_{ave}$) as it explained on Chapter 2. Because B_x> 1.7 (Table 3-4), according to the building code, a dynamic analysis must be conducted.

Table 3-4 Torsional sensitivity index and eccentricity for original model

Storey	*XCM (m)*	*YCM (m)*	*XCCM (m)*	*YCCM (m)*	*XCR (m)*	*YCR (m)*	*ex (m)*	*Tx (kN.m)*	*Bx*
STOREY4	9.17	9.14	9.17	9.14	11.74	9.14	2.56	2421	5.57
STOREY3	9.19	9.14	9.18	9.14	11.69	9.14	2.50	2441	4.38
STOREY2	9.19	9.14	9.18	9.14	11.60	9.14	2.42	2390	3.88
STOREY1	9.19	9.14	9.18	9.14	11.33	9.14	2.15	2237	3.56

From the dynamic analysis using the restraint model the period of structure, T_a=0.38 sec and accordingly V_e= 3280 kN and $V_d=V_e(I_e)/(R_dR_o)$=1598 kN >1153 kN. Table 3-5 indicates the distribution of seismic force before and after the dynamic analysis on each floor. It is shown the inertia force in each floor diaphragm will increase after dynamic analysis to the torsional irregularity in building. Table 3-4 describes the dynamic analysis procedure to calculate base shear, using SRSS combination. Table 3-5 indicates increase of design base shear using the dynamic procedure.

Table 3-5 Static and dynamic design base shear calculation

Storey	h_x *(m)*	W_x *(kN)*	$W_x.h_x$ *(kN.m)*	f_x *(kN)*	*V* *(kN)*	*Fx+T* *(kN)*	*Vdes(static)* *(kN)*	$F_d=F_e(V_d/V_e)$ *(kN)*	F_e *(kN)*	F_d+T *(kN)*
STOREY4	3.6	2034	7322	383		473		396	563	486
STOREY3	3.6	2070	7452	392	383	482	473	401	576	495
STOREY2	3.6	2070	7452	392	775	482	954	401	576	491
STOREY1	3.6	2070	7452	392	1167	473	1435	401	576	486
Total		8244	29678		1153		1908			1962

3.3.3 Non-linear time history analysis

Non-linear time history analysis is an advanced technique to calculate the response of the structure at discrete time steps using the selected ground motions from the past or artificial earthquakes. The damping matrix of the material properties will shift in each time step at deformation level near the yield deformation.

There are certain conditions that must be considered in modeling nonlinear analysis of structures. Unlike the linear analysis, method where the structure is consistently in the linear range the structure will experience the non-linear stage in this method. As a result, in this type of analysis the method of selection of load combinations is very important. FEMA 273 ATC-40 (Applied Technology Council 1997) , SEAOC (Structural Engineering Association of California and FEMA 356 have specific guidelines for this type of analysis that have been used in this research. According to these guidelines, the analytical model used for the evaluation and rehabilitation of the structure should describe the complete three-dimensional behavior of the building. This means that the model of the building structure must consider all the characteristic of the structure such as mass, strength, stiffness and deformation of the building near the performance point. In addition, by experiment on each component, the complete hysteretic of each component should be modeled.

All structural and non-structural elements contribute to the building stiffness, damping and its response to ground motion. However, not all of these elements will have significant effect due to strong earthquakes. Accordingly, these elements are divided to primary and secondary component. The latter is considered to have lower stiffness, strength and deformation capacity and usually in a new building design are not considered. FEMA 356 (FEMA 356: Federal Emergency Management Agency 2000) has defined three behavior (i.e. load/deformation curves) for primary components in a structure. These behaviors can be defined as either Force-Controlled (zero or limited ductility) or Deformation-Controlled behavior. The two types of behaviors are defined to disguise a ductile performance from a brittle performance. If a force or moment cause a noticeable non-linear deformation in a component, the performance can be assumed as deformation-controlled, e.g. beam control by flexure. Whereas, if a force or moment do not cause a noticeable inelastic response in a component, the performance can be assumed as a force-controlled. Figure 3-6 indicates load/deformation curves in a component. Type 1 curve and Type 2 curve are representing the ductile component and depending on the ratio between e and g can be determined as force or deformation-controlled.

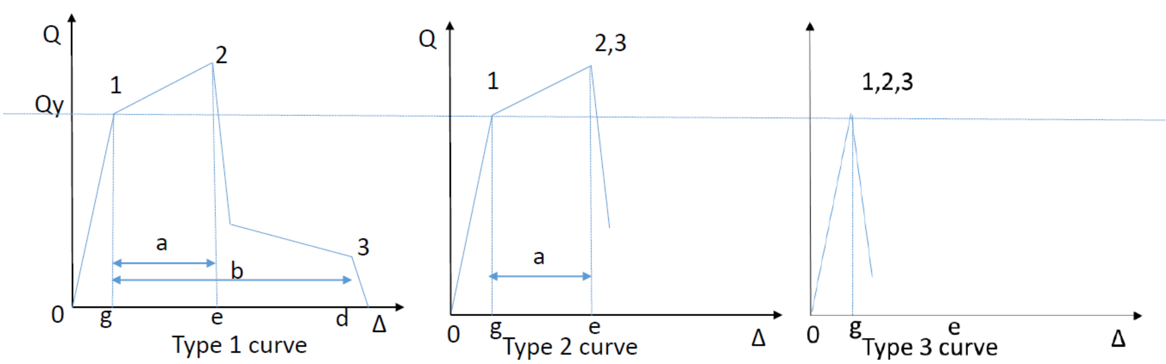


Figure 3-6 Component force vs. deformation curve (adapted from FEMA 356)

The Generalized load-deformation relation is described in Figure 3-7 that is used for all four major structural materials to model the deformation-controlled action in FEMA273 and FEMA 356. From point A to B the material behavior is linear where B is defined as yielding point (Q_y). From B to C is considered as strain- hardening with a slope between 0 to 10%. From C to D the material strength decreases significantly and there is no strength considered after point D.

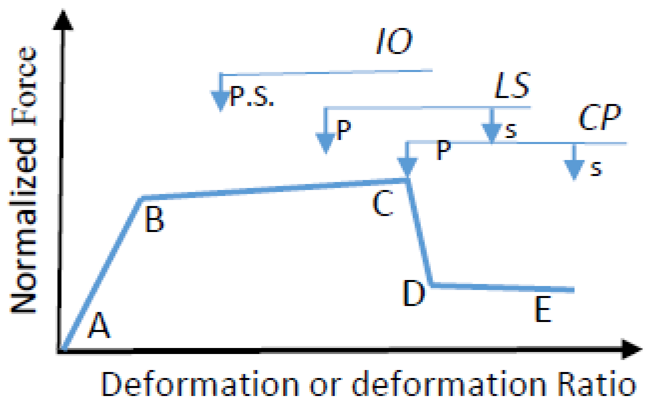


Figure 3-7 Generalized load-deformation in FEMA 356

3.4 Structural model using SAP2000

SAP2000 program is used to perform the nonlinear dynamic analysis. This program is capable of seismic analysis for planar reinforced concrete frames. The program is developed by CSI since 1975. Because the results are very sensitive to the assumptions, the user experience and judgment are essential in order to gain realistic results. Lumped plasticity model is an acceptable method that is used in SAP2000 to obtain results similar to actual structure (Hopper 2009).

Several types of hysteresis models are introduced in the program to define the nonlinear behavior of the materials such as steel and reinforced concrete for static and dynamic analysis. In addition, there is an option to define other materials with non-linearity. Three hysteresis behaviors Takeda, Pivot and Kinematic are defined for reinforced concrete structure. In this study, the Takeda model (Figure 3-8) is used for the nonlinear behavior of the RC structure. For more information regarding the hysteresis behavior refer to (Roufaiel and Meyer 1987) (Dowell and Tang 2003).

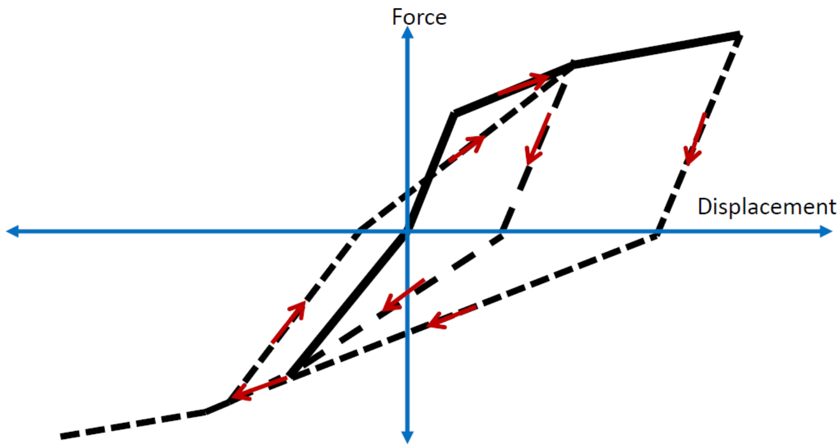


Figure 3-8 Schematic model of Takeda hysteresis behavior used in SAP2000

Mander et al. (1989) have shown the stress-strain curve relationship for the confined concrete (Figure 3-9).

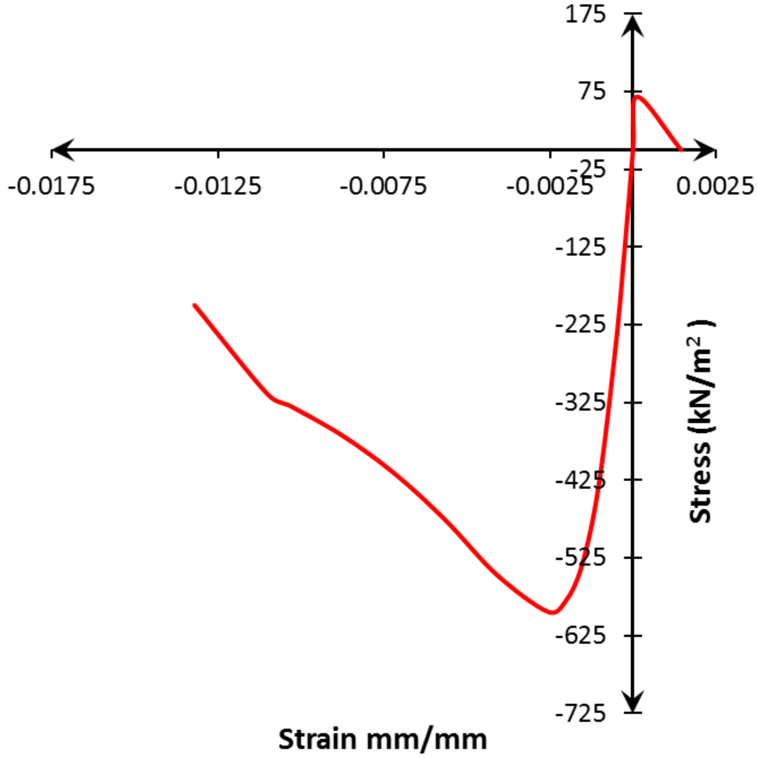


Figure 3-9 Mander Stress-Strain curve for confined concrete used in the analysis model

Moment-Rotation curve is defined for each member according to FEMA 356 guidelines to evaluate the performance of the structure. The Values for beams are from Table 3-6 for M3 (Moment on major axis) and values for columns are from Table 3-6 for P-M2-M3 (interaction of axial load and moment in two main directions) (FEMA 356, 2000). Two points of the moment-rotation curve are given to the program: yielding and ultimate. The location of performance point is shown in the diagram on Figure 3-10.

Table 3-6 Required performance drift for RC frames FEMA 356

Performance Level	***IO***	***LS***	***CP***
Beams	0.01	0.02	0.025
Columns	0.003	0.012	0.015

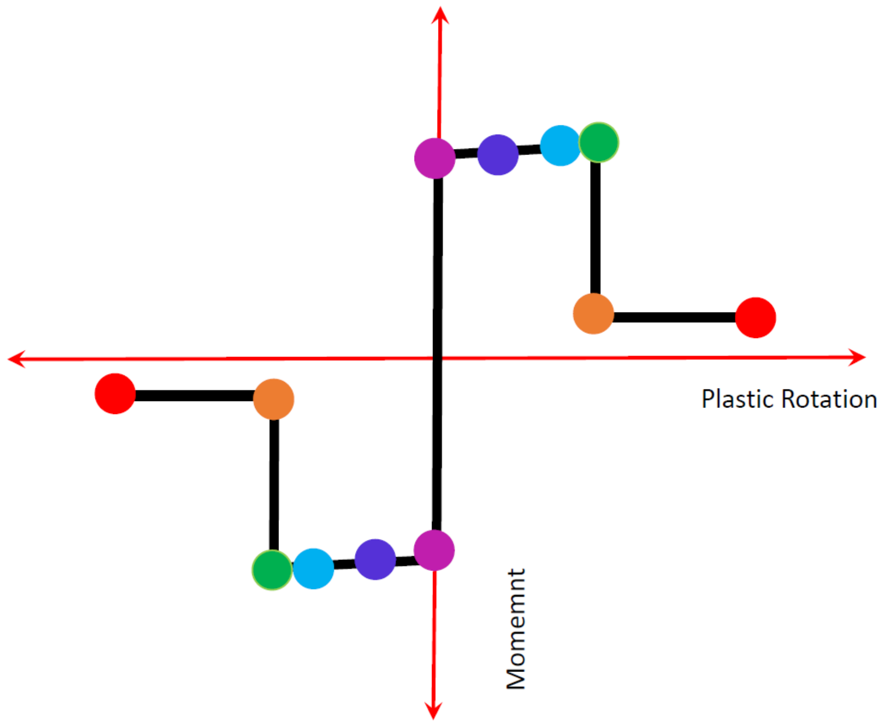


Figure 3-10 Location of performance point in moment curvature diagram in SAP2000

3.4.1 Structural modeling

There are two main methods in modeling the structural members in different software, *Fiber Model* and *Lumped Plasticity Model*. The two structural modeling methods are described in following sections.

3.4.1.1 Fiber model

In this method, every characteristic of the structural members such as dimensions and material properties is assigned to a number of fibers in the cross section of the member. In addition, the nonlinearity is distributed along the component length.

3.4.1.2 Lumped plasticity model

This method is known as a suitable model to define different response levels of the structural member from cracking to the collapse point. The basis of the lumped plasticity method is to define all the nonlinear behavior of the structural component such as beam or column at the two end hinge points (Figure 3-11). Accordingly, the rest of the member length act linear elastic. Scott and Fenves (2006) has investigated the effect of hinge integration on force-based beam-column components. Berry and Eberhard (2008) have evaluated the performance level of different bridge columns using lumped plasticity model. Moehle et al. (2008) have used this method in evaluation of nonlinear response of different RC structures.

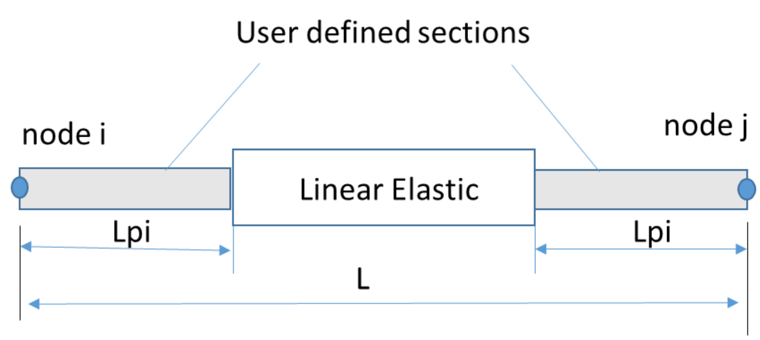


Figure 3-11 Schematic view of lumped plasticity model for structural component

3.4.2 Damping definition for modeling

It is desirable to understand the response of the structure beyond the elastic range in an earthquake event. The structural stiffness matrix is typically calculated from the stiffness of each individual member. However, it is not possible to calculate the damping matrix with the same method, because of the variation in damping properties of the materials (Figure 3-12). As a result, the damping matrix for the structure should be calculated from its modal damping ratios (Chopra 2012).

One of the methods that is used in SAP2000 for the calculation of damping matrix is Rayleighs and Caughey classical damping matrix method. The damping equation is defined as:

$$C = \alpha_0 M + \alpha_1 K \qquad 3.1$$

The coefficients α_0 and α_1 have the units of sec^{-1} and sec, respectively. It is shown that based on the virtue of the modal orthogonality the matrix C is diagonal.

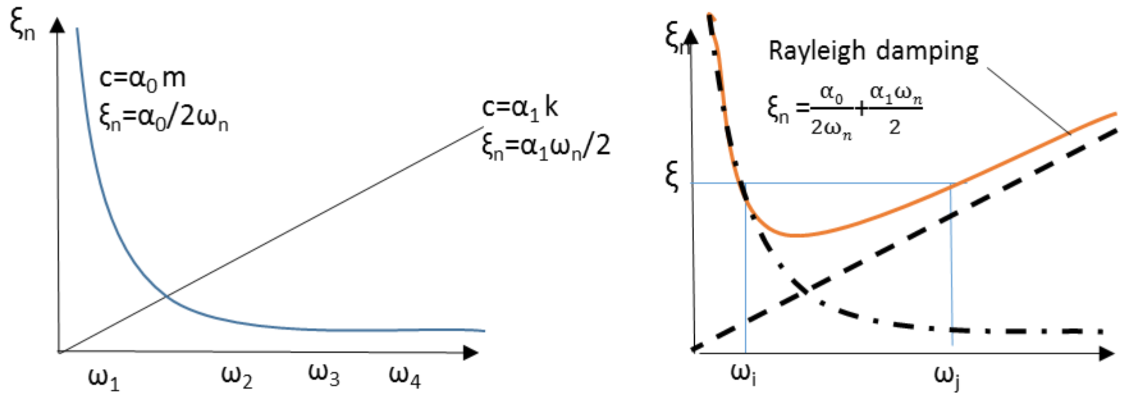


Figure 3-12 Effect of natural frequencies on variation of modal damping ratios

The values for α_0 and α_1 are calculated as:

$$\alpha_0 = \varepsilon \frac{2\omega_i \omega_j}{\omega_i + \omega_j} \qquad \alpha_1 = \varepsilon \frac{2}{\omega_i + \omega_j} \qquad 3.2$$

3.5 Ground motion selection

In the National Building Code of Canada (NBC 2010) (NRC 2010) earthquake ground motions are considered in terms of Uniform Hazard Spectrum (UHS) in the seismic provisions. The location, site condition (soil classification) are the main items to determine the target UHS (Atkinson 2009). The UHS in NBCC is based on the 2% chance of occurrence in 50 years. For linear dynamic analysis and modal dynamic analysis, UHS can be directly used. However for non-linear time history analysis the actual ground motions are required conforming to the existing site condition.

In order to define the seismic hazard for a specific building three main characteristic should be considered; the distance of the building to causative faults, the site specific geologic data and the selected level of earthquake hazard.

Because in NLDA the target displacement is determined through dynamic analysis using ground motion, the result can be highly sensitive based on each ground motion characteristics. As a result, the analysis should be carried out with more than one ground motion. Accordingly, the calculated internal force is expected to be in reasonable approximation from design earthquake. FEMA 356 required minimum three ground motion to be selected based on the site characteristic. These selected ground motions should be scaled to design spectra for the period between 0.2T seconds to 1.5T seconds (T is the fundamental period of the building.

All seismic code and guidelines require scaling of the ground motion time histories to match the period range of interest for desired design spectra. This is because of the seismic hazard at each site has been often represented by design spectra (Naeim et al. 2004).

In this research PEER database ground motion (GM) is used to find the appropriate ground motions for the building under study in Vancouver area with the probability of 2% in 50 years. Different factors in order to select the ground motions such as the magnitude, distance from the fault, fault type, etc. should be defined in order to find the appropriate GMs. Because the PGA/PGV (PGA in g and PGV in m/sec) for Vancouver is close to one the selected ground motions have an average PGA/PGV equal to 0.97. In addition, PEER has the option of uploading the desired design spectra and finding the scaled GMs for the user

(Figure 3-13). Eight ground motions are chosen for the analysis in SAP2000 for this research. Figure 3-14 indicates the ground motion acceleration used for the nonlinear-time history analysis in this research (as summarized in Table 3-7). Figure 3-15 indicates the scaled design spectra for the site class C in Vancouver that used for the analysis in this research. The non-linear analysis is done the x-direction value for the 3D analysis in both x and y direction.

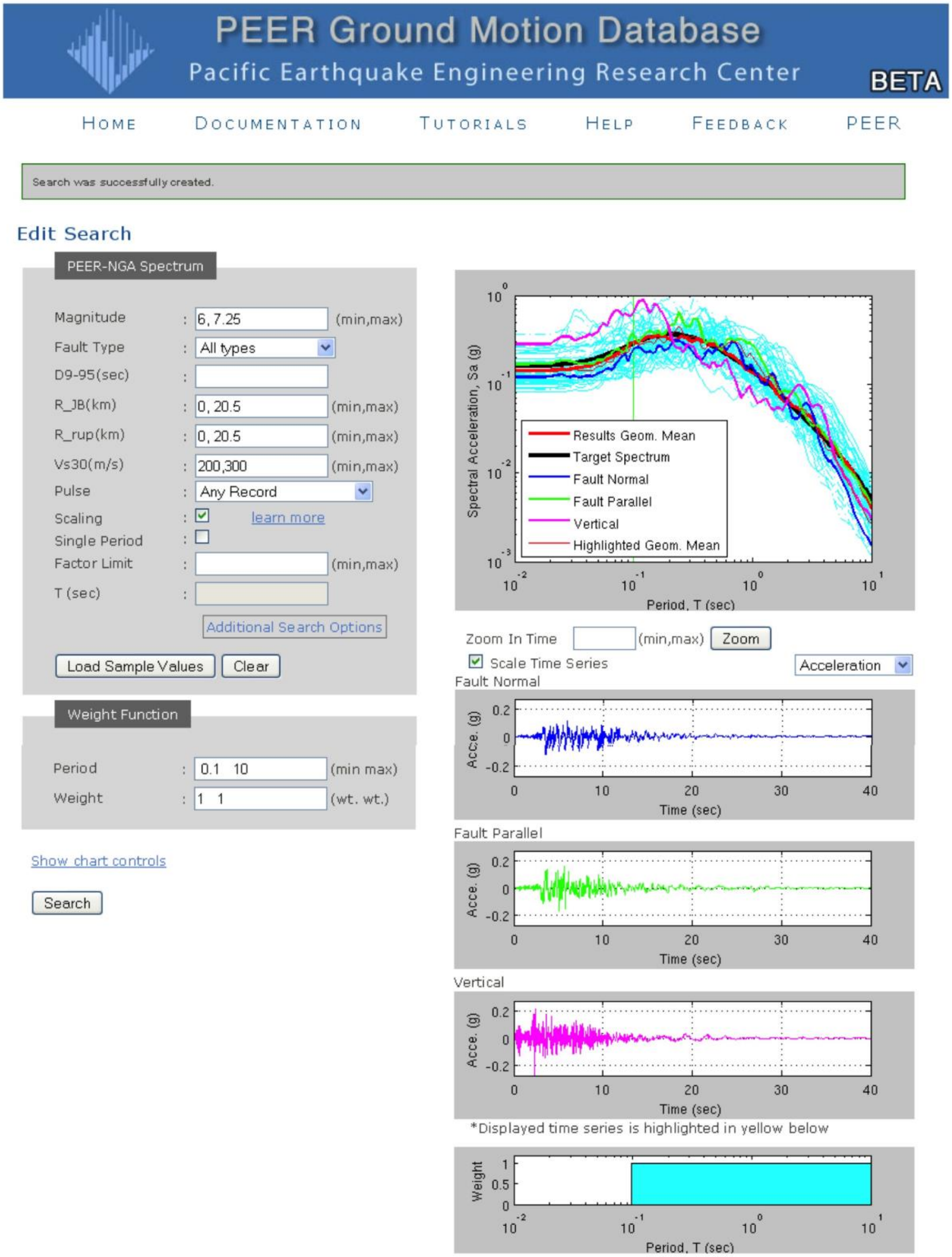
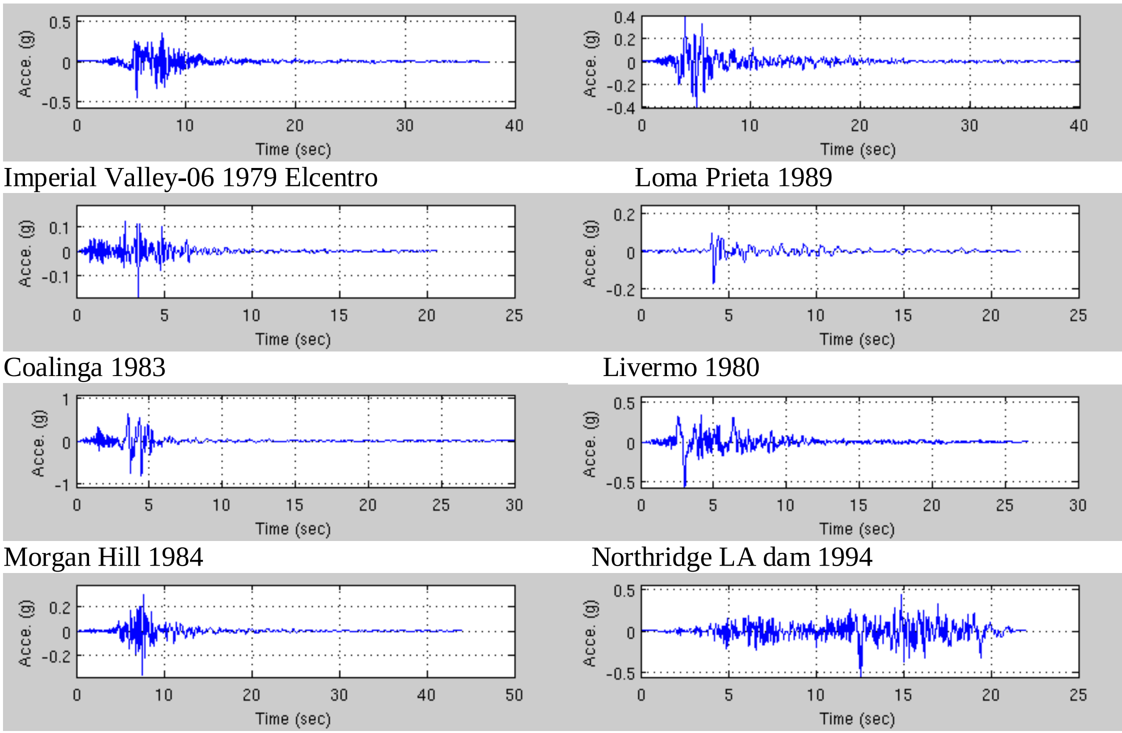


Figure 3-13 Ground motion calculation in PEER NGA WEST (http://peer.berkeley.edu/)



Imperial Valley-06 1979 Elcentro

Loma Prieta 1989

Coalinga 1983

Livermo 1980

Morgan Hill 1984

Northridge LA dam 1994

Parkfield 1966

San Fernando 1971

Figure 3-14 Sample of ground motion time history

Table 3-7 Ground motions selected for the time history analysis

Record	Earthquake	Event date	Station	M_w	PGA (g)	PGA/PGV	Epic. distance (km)
1	Imperial Valley	10/15/1979	Chihuahua	6.53	0.27	0.923	18.12
2	Park field	6/28/1966	Cholame Shandone	6.19	0.059	0.98	36.18
3	Northridge	1/17/1994	LA dam	6.69	0.229	1.03	31.45
4	Livermore	1980-07-01	Fremont mission	5.42	0.037	0.95	27.98
5	Coalinga	5/2/1983	Slack Canyon	6.36	0.153	1.016	33.526
6	Morgan Hill	4/24/1984	Fremont Mission	6.19	0.22	0.91	31.89
7	Loma Prieta	10/18/1989	Gilroy Array	6.93	0.156	1.092	34.53
8	San Fernando	1971-09-02	Lake Hughes	6.61	0.126	0.81	26.7

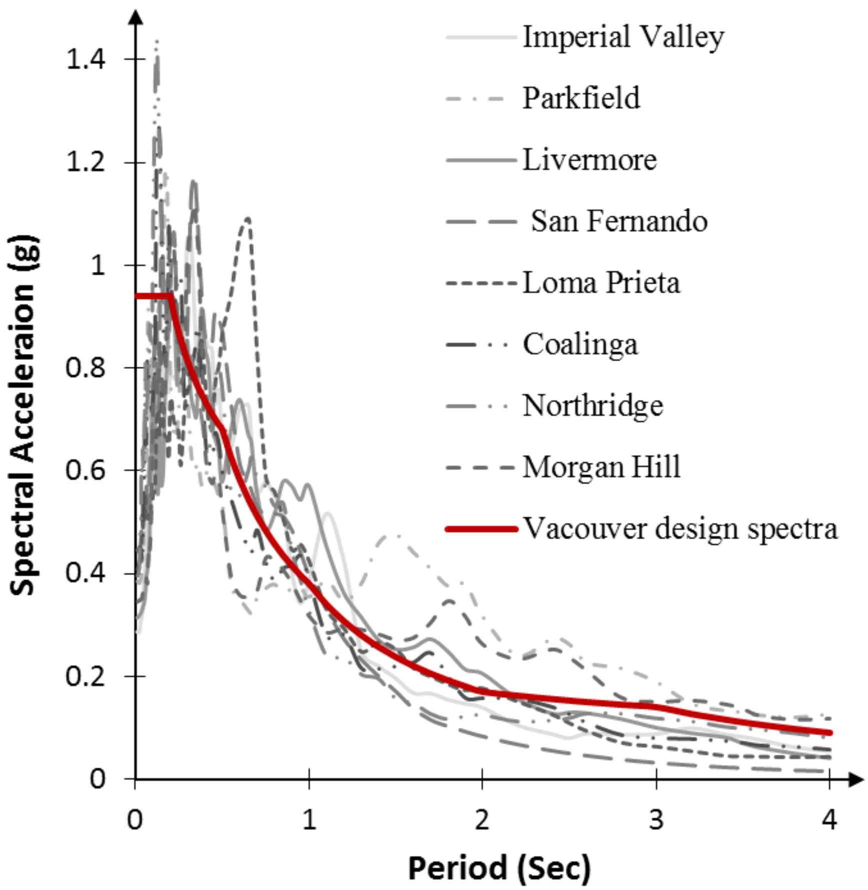


Figure 3-15 Scaled design Spectra and Target spectra for site class C in Vancouver

3.6 CLT panel definition and hysteresis behavior modeling

In order to have a better understanding of the hybridization it is necessary to identify the behavior and specification of the component of hybrid structure. Accordingly, the definition of material specification is the first step to analyze and design. Because the combination of concrete and wood will be investigated in this study each material characteristics are described briefly.

Steel is a ductile material, which its isotropic specification provides the same behavior in all direction and through the material. However, the plasticity of structural steel may not be sufficient on certain loading condition and since the strength of steel is very high, other failure such as local buckling and local, instability may occur under certain loading conditions. In addition, because of the high density of steel comparing to other materials, using the minimum amount of material in the design procedure is always considered. As a result, possible deficiencies such as width to thickness ratio of columns, flexural buckling of bracing and columns, local buckling, beam torsion in the frame and effect of P-Δ are some challenges in the design of steel structures. Moreover, brittle failure is possible in some structures such as cases of pure tensile failure of screw connection, failure of welds and fatigue stress of connection used under cyclic loads.

Accordingly, the definition of material specification is important to analyze and design of structures using materials with inelastic behavior. Hysteresis behavior is known as the best way to achieve this goal. The need to accurate modeling of hysteretic behavior of material can be summarized as analytical behavior of inelastic structures that requires elaborating the force deformation relationship, under seismic loading, that includes a number of variation factors. Some materials incorporate degradation in strength, stiffness and contribute the pinching effect due to cycling and dynamic loading which may cause to weakening and failure in the structure. Accordingly, the nonlinearity of system different models has been introduced. The most common model identification of structural materials in the Bouc-Wen system (1960's) which includes a variety of hysteretic patterns and has the versatility to be used for most of materials. This method has been applied for most of engineering problems.

Using the CLT panel as shear wall requires preventing any crushing in the panel. Fragiacomo et al. (2011) have shown that considering the flexibility for the connections at the top and bottom of the panels (brackets or hold-downs) is important, because the vibration period will be significantly underestimated.

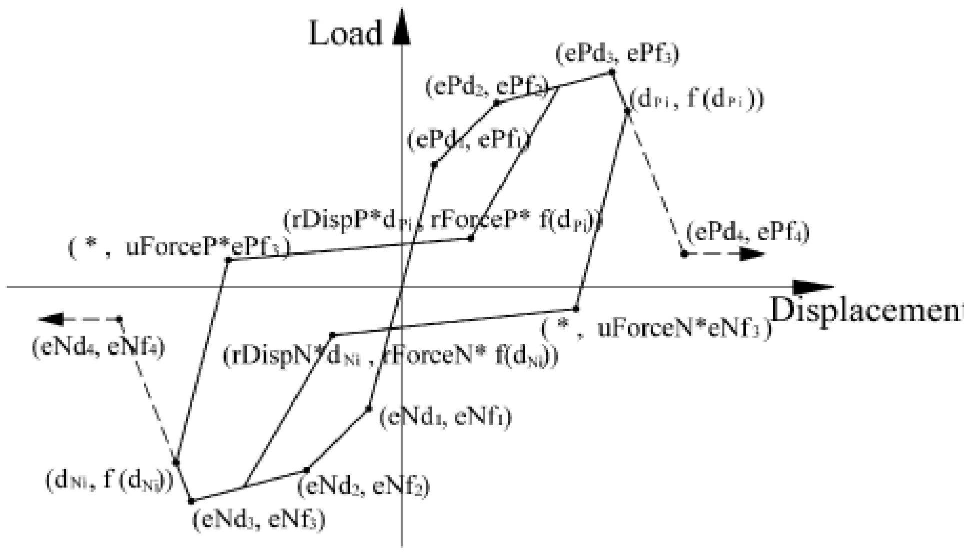


Figure 3-16 Pinching 4 model

Because the CLT panel is very stiff in plane, ductility should be provided with the connections. In order to modeling the cyclic behavior of the brackets, several researches have been done which some of them has been reviewed in Chapter 2. In this research we will use the pinching 4 model values explained by Shen et al. (2013) in order to define the hysteretic behavior of connection in SAP2000 models. Figure 3-16 shows the pinching 4-model hysteresis curve.

The benefit of using pinching4 model comparing to other method is that it can be used for asymmetric hysteresis behavior. In addition, the connection failure happens when the displacement curve exceeds the envelope curve defined. The values used for the hysteresis curve for the bracket is shown in Table 3-8.

Table 3-8 Bracket force-deformation value for pinching model

Parametrs	***Positive backbone***		***Negative backbone***	
	Longitudinal to grain	Perpendicular to grain	Longitudinal to grain	Perpendicular to grain
ePf_1 (kN)	19.5	18.68	-19.5	-18.68
ePf_2 (kN)	44.89	41.5	-44.89	-41.5
ePf_3 (kN)	49.45	46.7	-49.45	-46.7
ePf_4 (kN)	6.35	18.6	-6.35	-18.6
ePd_1 (mm)	2.15	3.7	-2.15	-3.7
ePd_2 (mm)	8	10	-8	-10
ePd_3 (mm)	20	24	-20	-24
ePd_4 (mm)	60	70	-60	-70
rDispP	0.55	0.55	0.5	0.5
fForceP	0.15	0.15	0.3	0.3
uForceP	0.03	0.03	0.05	0.05
rDispN	0.55	0.55	0.5	0.5
fForceN	0.15	0.15	0.3	0.3
uForceN	0.03	0.03	0.05	0.05

The benefit of using pinching4 model comparing to other method is that it can be used for asymmetric hysteresis behavior. In addition, the connection failure happens when the displacement curve exceeds the envelope curve defined. The values used for the hysteresis curve for the bracket is shown in Table 3-8.

4 Chapter: Result of the Analysis

In this chapter, the result from analysis on 4-storey torsional irregular building is presented. Linear dynamic analysis and nonlinear time history analysis is conducted on the original and hybrid structure to elaborate the behavior of the structure under seismic loading.

4.1 Comparison of static and dynamic base shear

The analysis result for the base shear calculation using Linear Static and Linear Dynamic methods for the main structure is presented in Chapter 3. In this chapter, the effect of hybridization is studied and the analysis results compared with the original structure.

4.2 Hybridization with CLT panels

In order to reduce the torsional sensitivity in the floor diaphragm two CLT wall panels are added to the building as shear wall (Figure 4-1). The two walls are continuous from the storey 1 to 4 from and are located on gridlines A and D, and in between gridlines 1 and 4. The multi-linear hysteresis behavior of the CLT connection to the concrete structure is not included in this analysis.

The 5-ply CLT panel (6.75 in) has been modeled in SAP2000 Program as shell element as it was described in Chapter 3. The result from the analysis indicates significant decrease on the eccentricity between center of mass and rigidity. The torsional sensitivity index B_x is less than 1.7 and as a result the building is not torsional irregular on plan (Type 7 in NBCC). The need for dynamic analysis can be waived. However, if the dynamic analysis is conducted, the resulted base shear can be decreased to 0.8 of equivalent static design force. Table 4-1 shows a summary of the results from analysis.

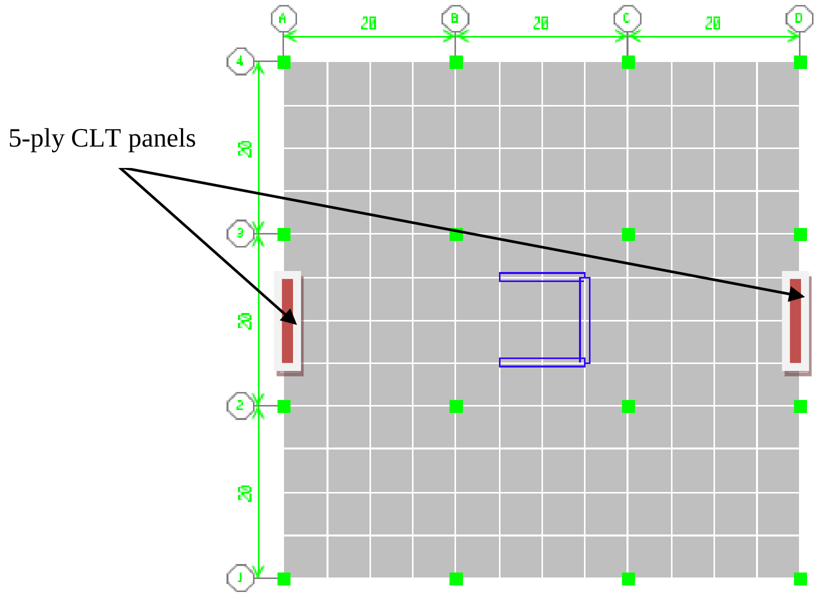


Figure 4-1 Building plan with CLT panels

Table 4-1 Torsional sensitivity index and eccentricity for the hybrid model

Storey	*XCM (m)*	*YCM (m)*	*XCR (m)*	*YCR (m)*	e_x *(m)*	*Tx (kN m)*	*Bx*
STOREY4	9.17	9.14	10.07	9.14	0.91	1051	1.11
STOREY3	9.19	9.14	10.11	9.14	0.92	1076	1.10
STOREY2	9.19	9.14	10.15	9.14	0.96	1093	1.08
STOREY1	9.19	9.1	10.15	9.14	0.92	1079	1.08

From dynamic analysis, using the restraint model, the period of structure, T_a=0.55 sec and accordingly V_e= 1656 kN and $V_d=V_e(I_e)/(R_dR_o)$=1156 kN < 1552 kN. Accordingly, 80% of static base shear is equal to 1242 kN which means the base shear needs to be scale up. Table 4-2 indicates the distribution of seismic force before and after dynamic analysis on each floor.

Table 4-2 Static and dynamic design base shear calculation for hybrid model

Storey	h_x *(m)*	W_x *(kN)*	$W_x.h_x$ *(kN.m)*	f_x *(kN)*	*V (kN)*	F_x+T *(kN)*	$V_{des}(static)$ *(kN)*	$F_d=F_e(V_d/V_e)$ *(kN)*	F_e *(kN)*	F_d+T *(kN)*
STOREY4	3.65	2029	7405	347		396		239	410	336
STOREY3	3.65	2070	7555	351	347	405	396	293	414	342
STOREY2	3.65	2070	7555	351	698	405	801	293	414	347
STOREY1	3.65	2070	7555	351	1049	405	1206	293	414	342
Total		8244	30073		1400		1611			1367

Figure 4-2 shows the inter-storey drift for the original and hybrid structure from dynamic analysis result. It is shown that the effect of adding the CLT panels will decrease the inter-storey drift significantly, more than 75%, under the code requirements (Figure 4-2).

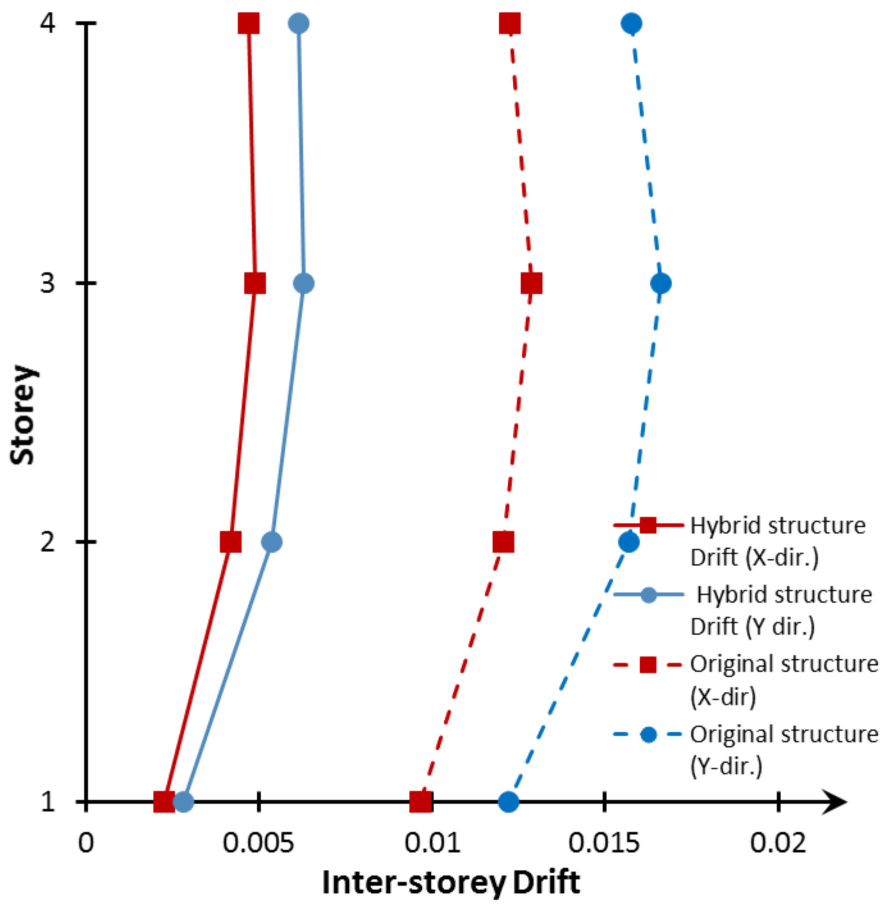


Figure 4-2 Comparison of inter-storey drift for hybrid and original structure (LDA analysis)

In addition, the reduction in seismic design base shear due to hybridization is shown in Figure 4-3. From Figure 4-3, it can be seen that the base shear has been decreased to ~30% below the base shear in the original structure.

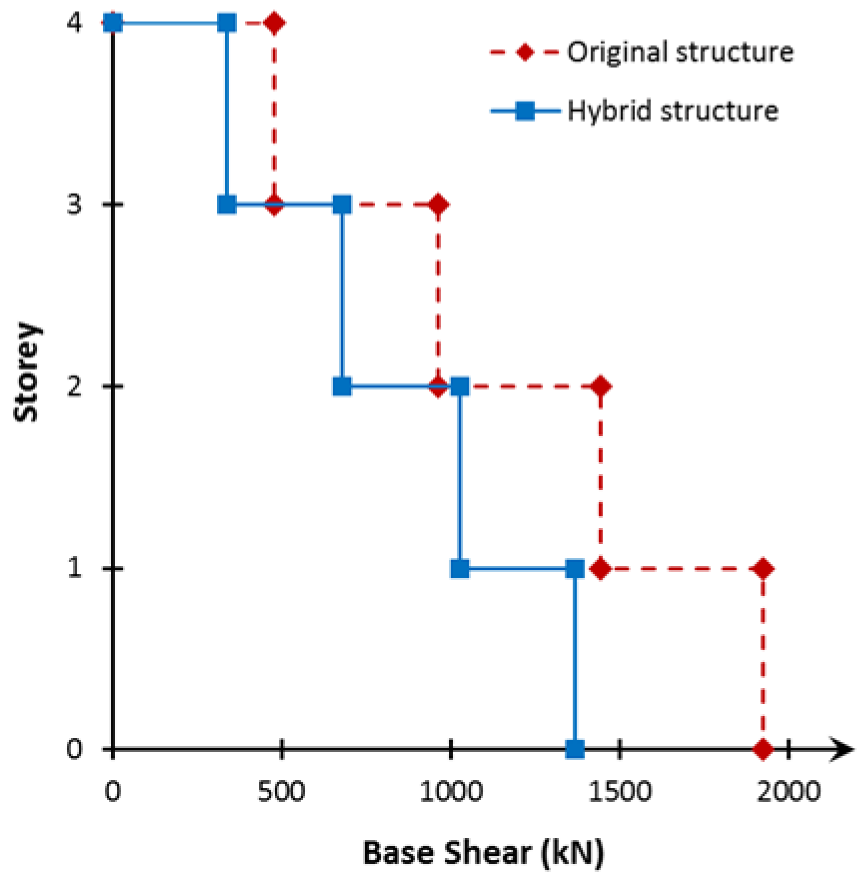


Figure 4-3 Comparison between seismic base shear for original and hybrid structures

The comparison of torsional sensitivity on each storey for original and hybrid structures is shown in Figure 4-4. It is indicated that in original structure B_x is greater than 1.7, the maximum allowable as defined by code, and as a result, dynamic analysis is mandatory. However, for the hybrid system the torsional sensitivity almost consistent along the height of the building and is lower than 1.7, the maximum allowable B_x as defined by code.

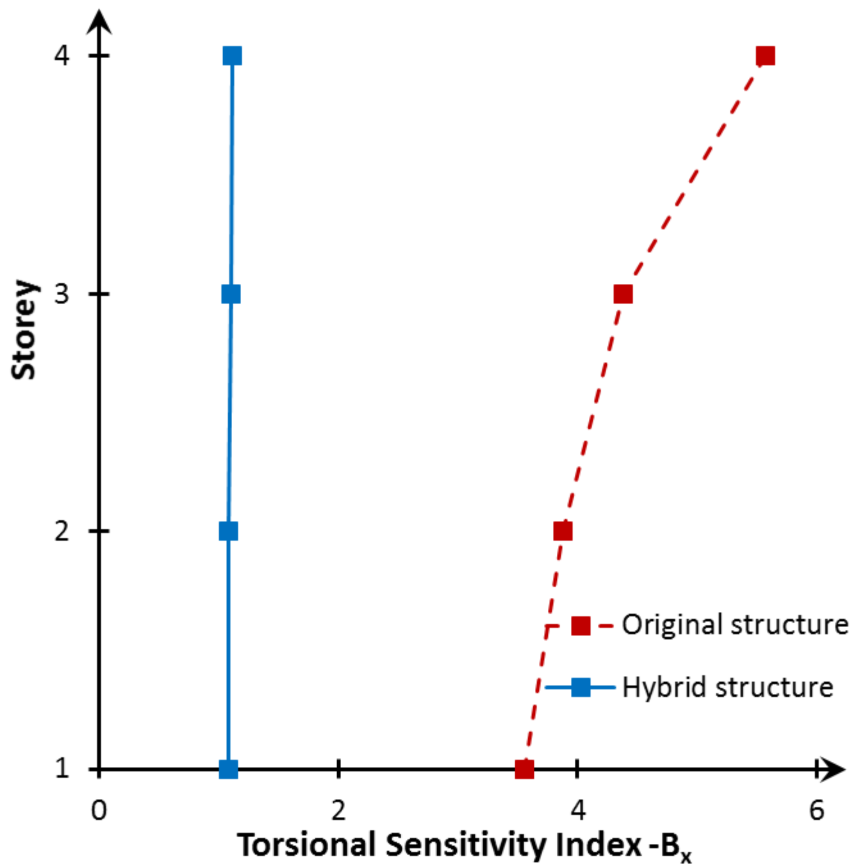


Figure 4-4 Comparison of torsional sensitivity index (B_x) for hybrid and original structures

4.3 Non-linear time history analysis

In order to have better understanding of the structure behavior before and after hybridization, a more comprehensive dynamic analysis method, which is Nonlinear Time History analysis, is conducted. This method was explained in detail in Chapter 3.

4.3.1 Parametric study

Prior to investigate the effect of hybridization on the structure, two-dimensional modeling is conducted in SAP2000 to understand the effect of CLT wall length on a four storey frame. Two models with full length of the wall as infill in the frame and the 2nd one with 3-meter panel with bracket connection is analyzed to compare the inter-storey drift (Figure 4-5 and Figure 4-6)

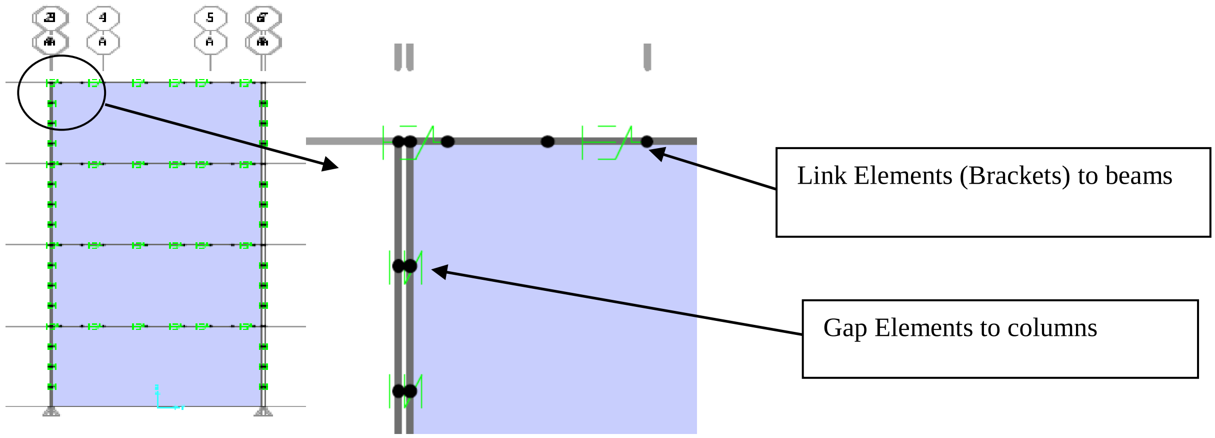


Figure 4-5 Concrete frame with full length of the wall as infill in the frame

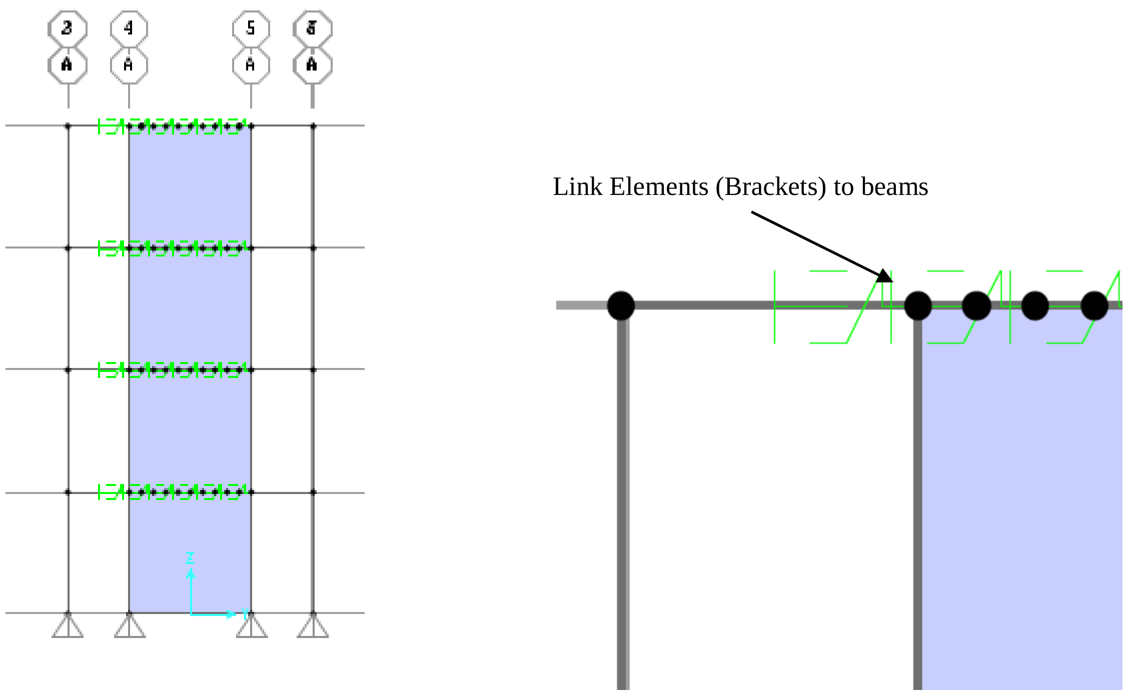


Figure 4-6 Concrete frame with 3 meter CLT with panel connections to structure

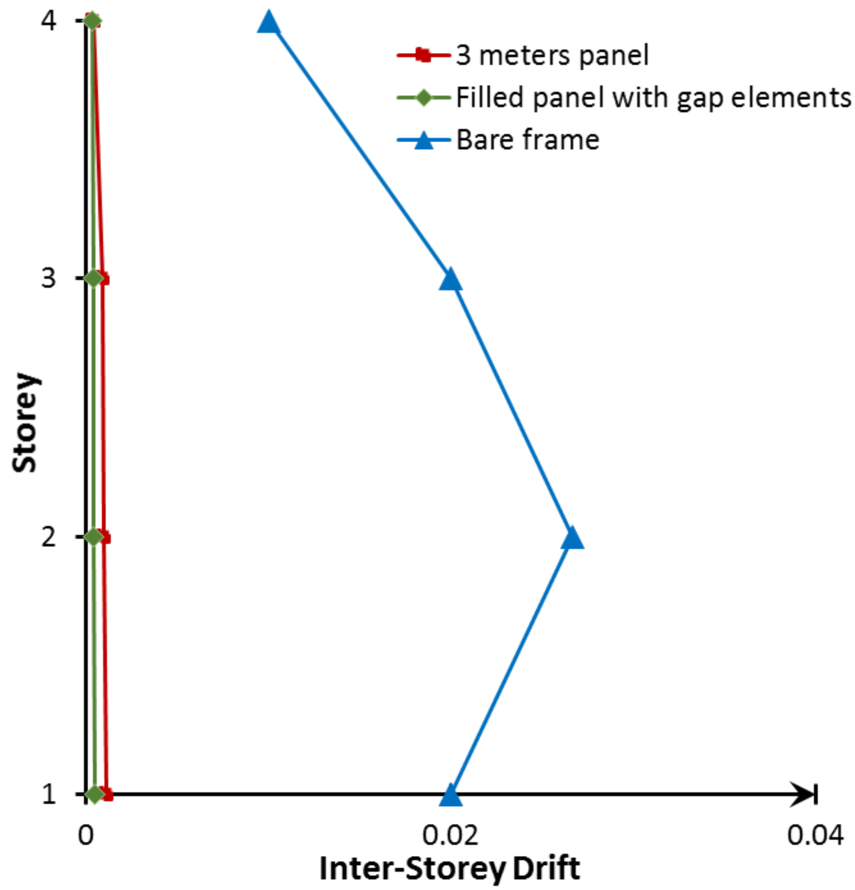


Figure 4-7 Inter-storey drift comparison for the two type infill

From the analysis result that indicated in Figure 4-7 and 4-8, it is concluded that the inter-storey drift of the both filled panel and 3-meter panel structures are very below (20 to 50 times) the bare RC moment frame. Accordingly, the inter-storey drift variation regardless of the infill panel wall length is negligible (as the larger one is ~2% of allowable drift).As a result, in this research, the 3 meter wall infill panel is used on 3D modeling and dynamic analysis. Accordingly, the effect of gap elements and CLT panel crushing due to main concrete frame movement is avoided. Figure 4-8 the inter-storey drift comparison between the two models.

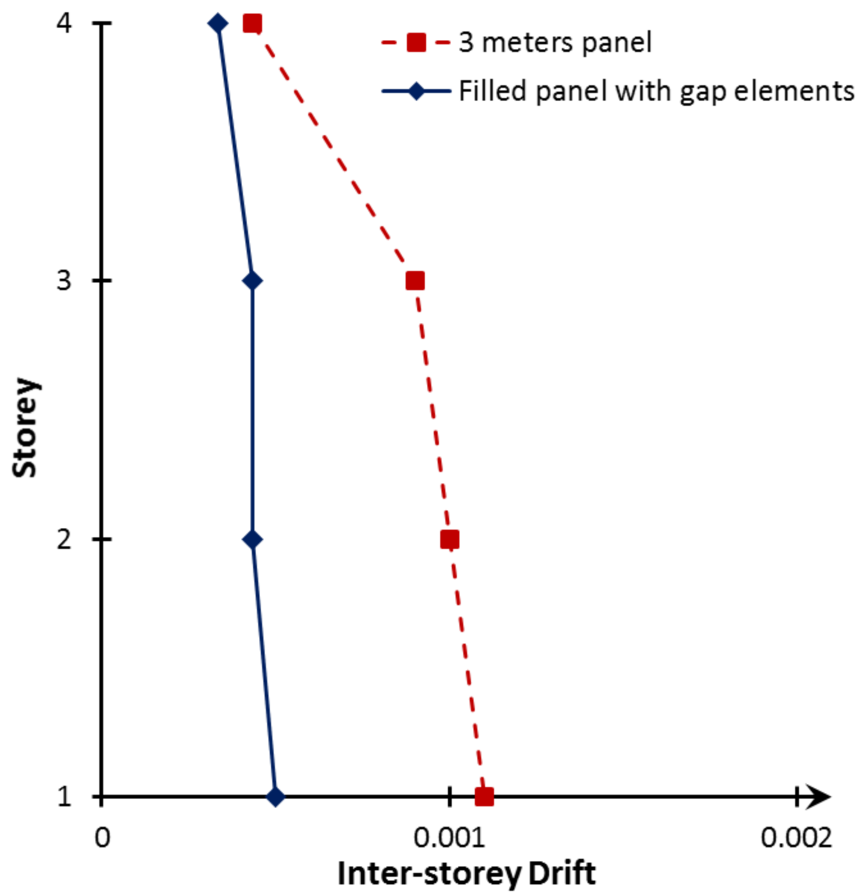


Figure 4-8 Inter-storey drift comparison for the two type infill

In addition, the average bracket force-deflection relationship from the chart below indicates that the numbers of applied brackets per panel are in the range as described in Chapter 3 (Figure 4-9).

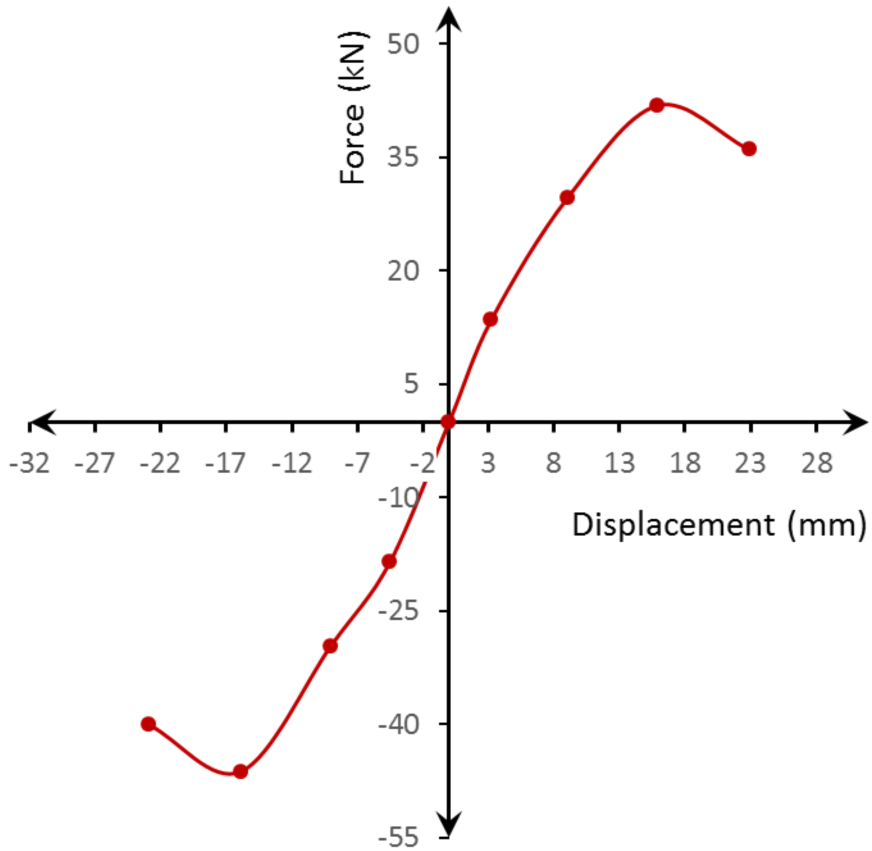


Figure 4-9 Bracket Force-Deflection ratio for 3 m panel

4.3.2 Results of non-linear time history analysis

Non-linear time history analysis is used to evaluate the performance of the structure with and without hybridization. In addition, a few options for the location of CLT panels are explored. The relationship between the eccentricity and panel location is also investigated. Furthermore, locations of the first plastic hinges are determined. The analysis is done using SAP2000 based on the procedure discussed in Chapter 3.

4.3.2.1 Comparing inter-storey drift with and without CLT

Figures 4-10 to 4-13 are the summary of the results from NLA for 8 different ground motion that are discussed in Chapter 3. The inter-storey drift relationship vs. the storey for both hybrid and original structure are shown (Figure 4-10 and 4-11).

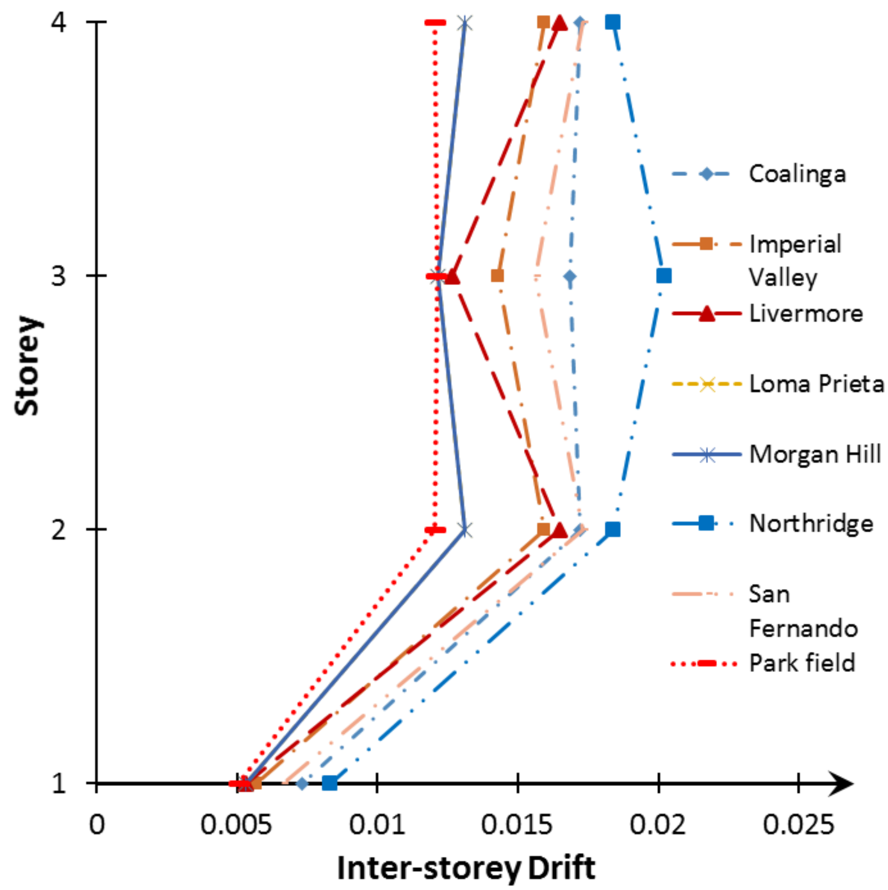


Figure 4-10 Inter-storey drifts for original structure from NLTHA analysis

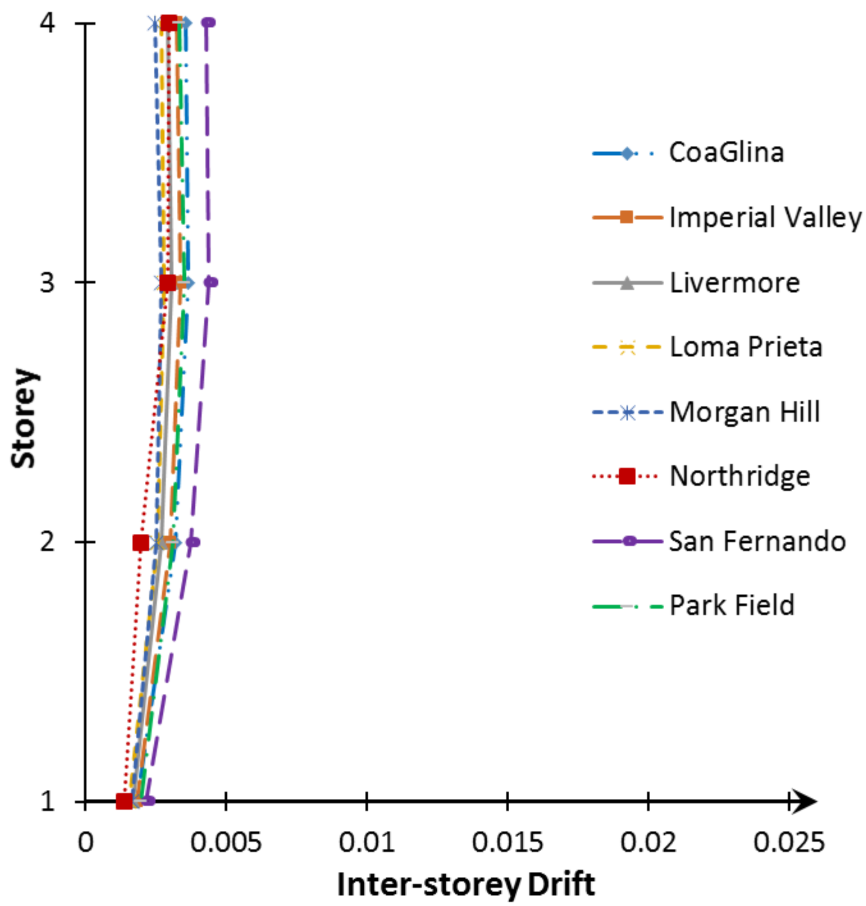


Figure 4-11 Inter-storey drifts for the hybrid structure from NLTHA analysis

It is indicated that the drift results from NLTHA for the original building in all cases are almost showing significant change for the first to second floor due to change on eccentricity. In addition, effect of torsional force can be seen on the nonlinear drift diagram. However, in the hybrid structure, because of the reduction in eccentricity and accordingly the torsional force, the change in the drift on each floor is gradually increasing and it is almost steady for the all cases.

4.3.2.2 Evaluation of performance for hybrid and original structure

According to the threshold given in FEMA-356 (explained in Chapter 3) and based on the inter-storey drift (Moment-Curvature of the structural components, beams and columns) the performance of both buildings is evaluated.

From Figure 4 12 it can be seen the original structure does not meet the life safety performance objective of the code, neither for the columns, nor for the collapse prevention which barely satisfy the limit (Figure 4-12). While, both of life safety and of collapse prevention performance level met the code requirements according to the beams. However, in the hybrid structure, the inter-storey drift for both columns and beams well satisfy the limits from the code (Figure 4-13).

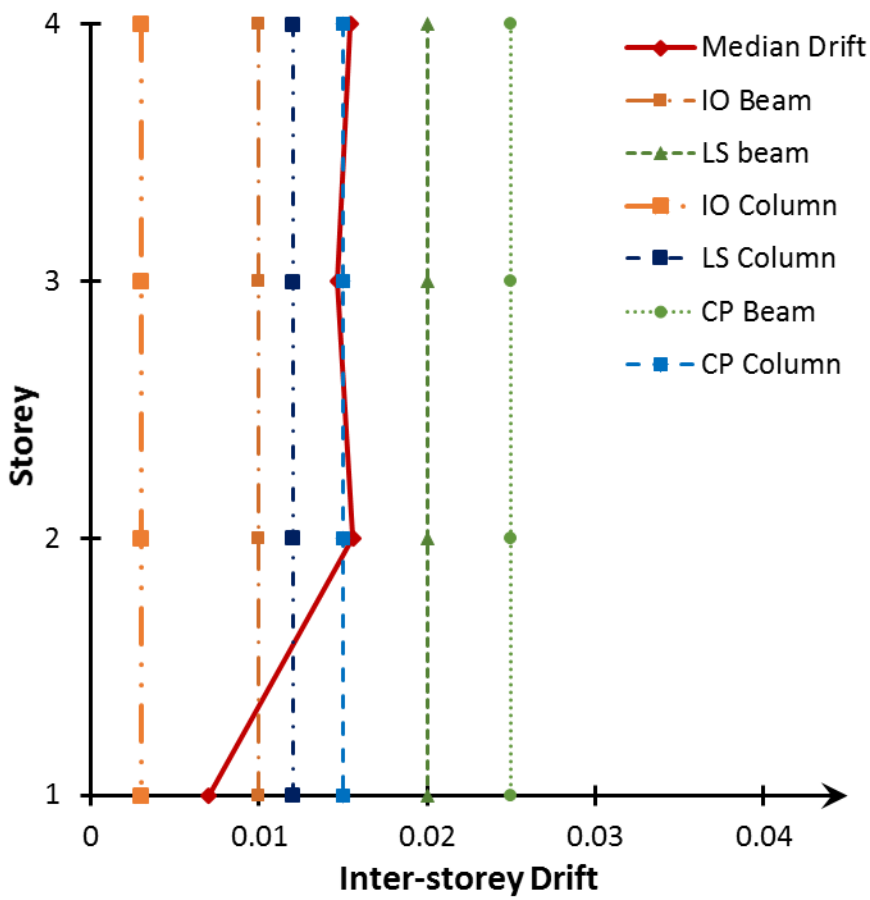


Figure 4-12 Performance level for median drift for original structure

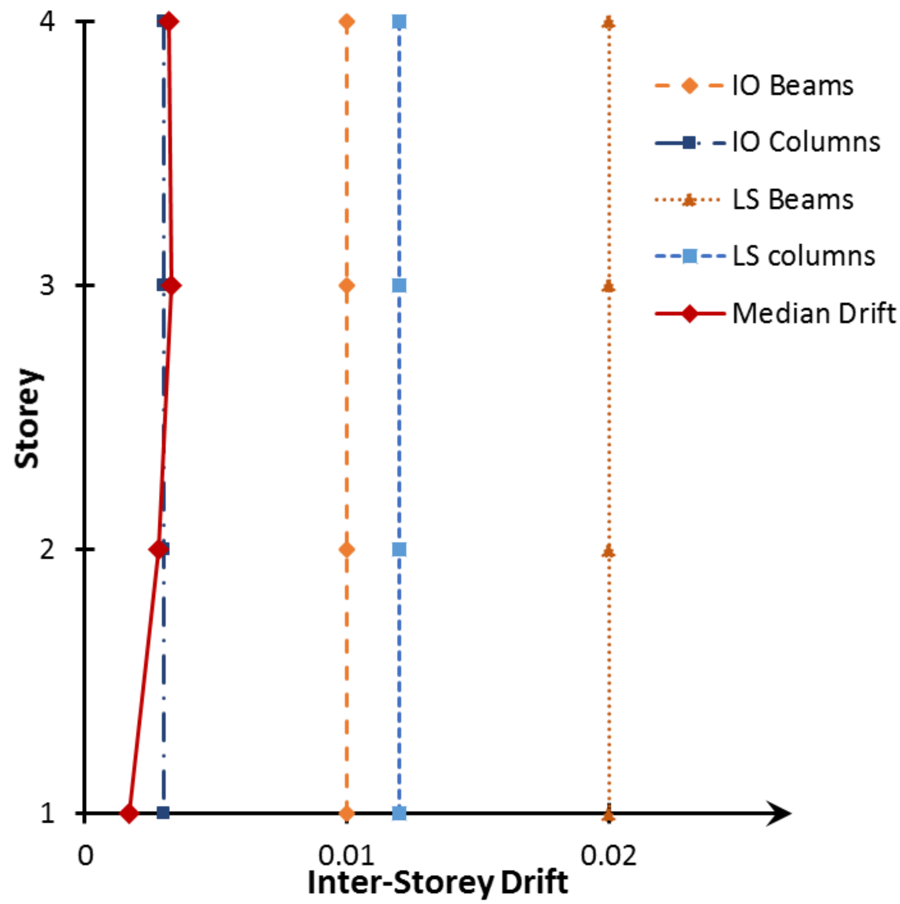
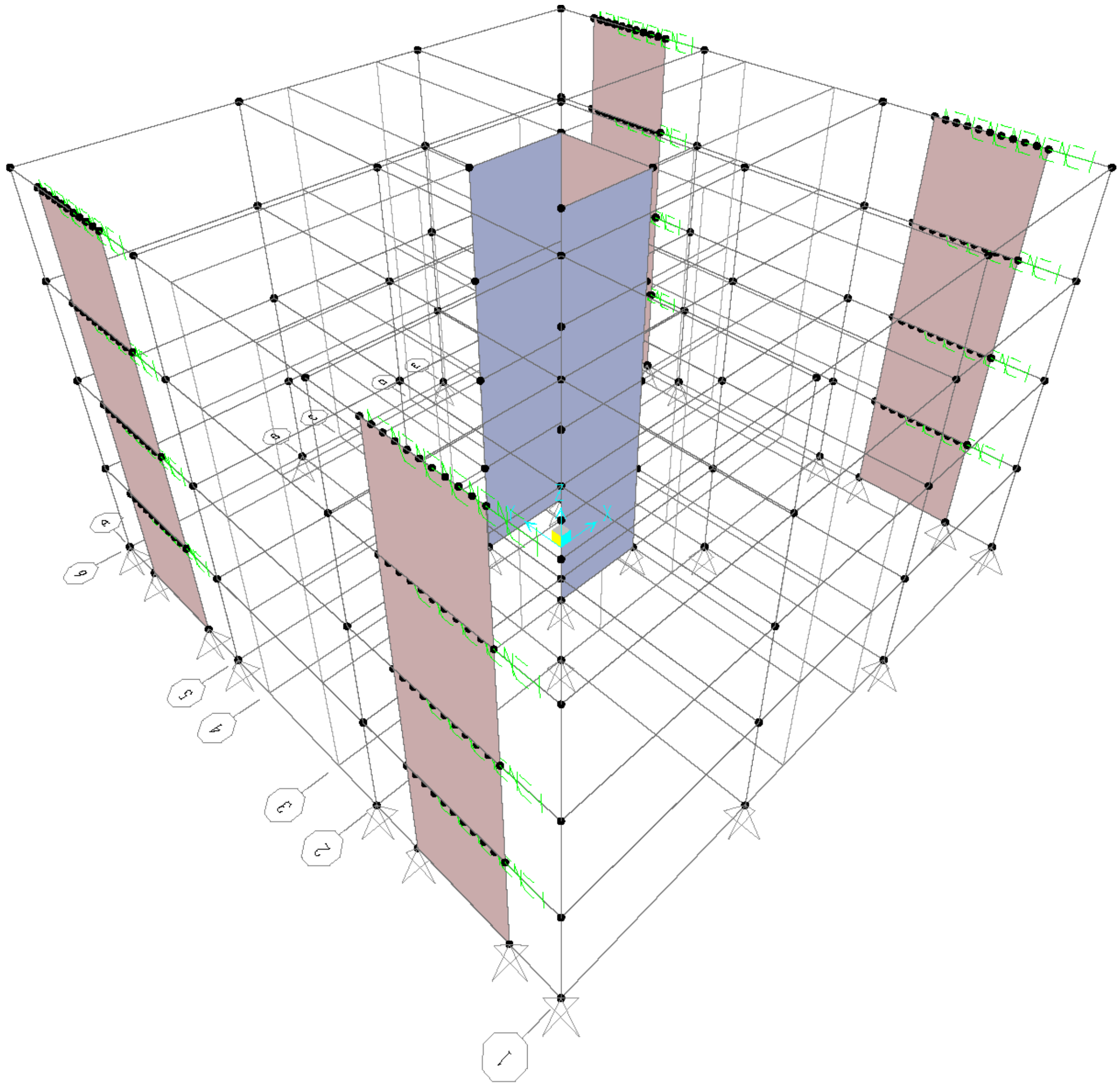


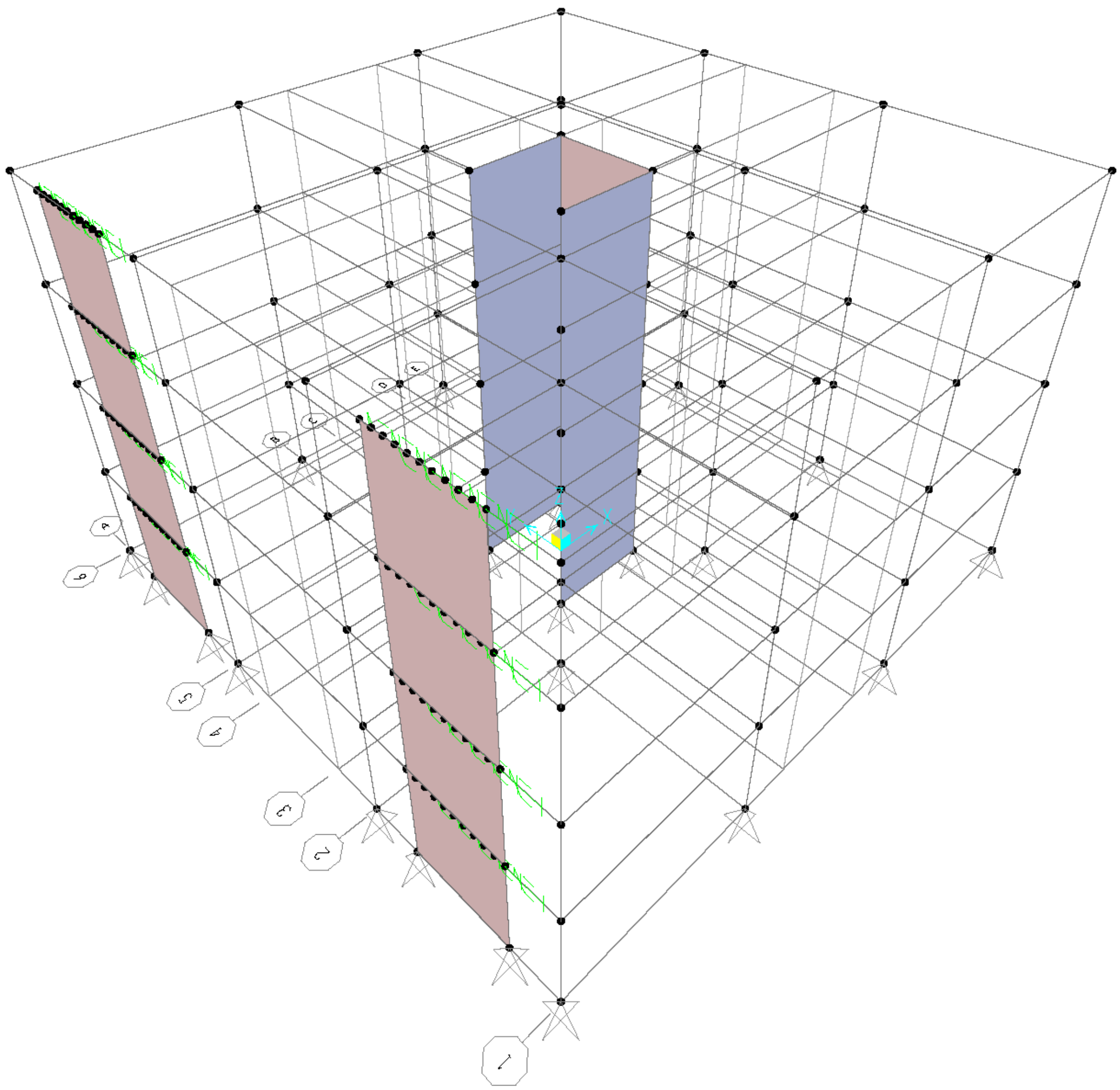
Figure 4-13 Performance level for median drift for hybrid structure

4.3.2.3 Comparison of effect of wall location on plan on the inter-storey drift

In addition, to the hybrid model under the study two other options for the location of CLT panels in the plan is investigated (Figure 4-14). The result shown in the Figure 4-15 to elaborate the relationship between inter-storey drift and eccentricity.



(a) Two side with end bay panels



(b) One side with end bay panels

Figure 4-14 Orientation of CLT shear panels

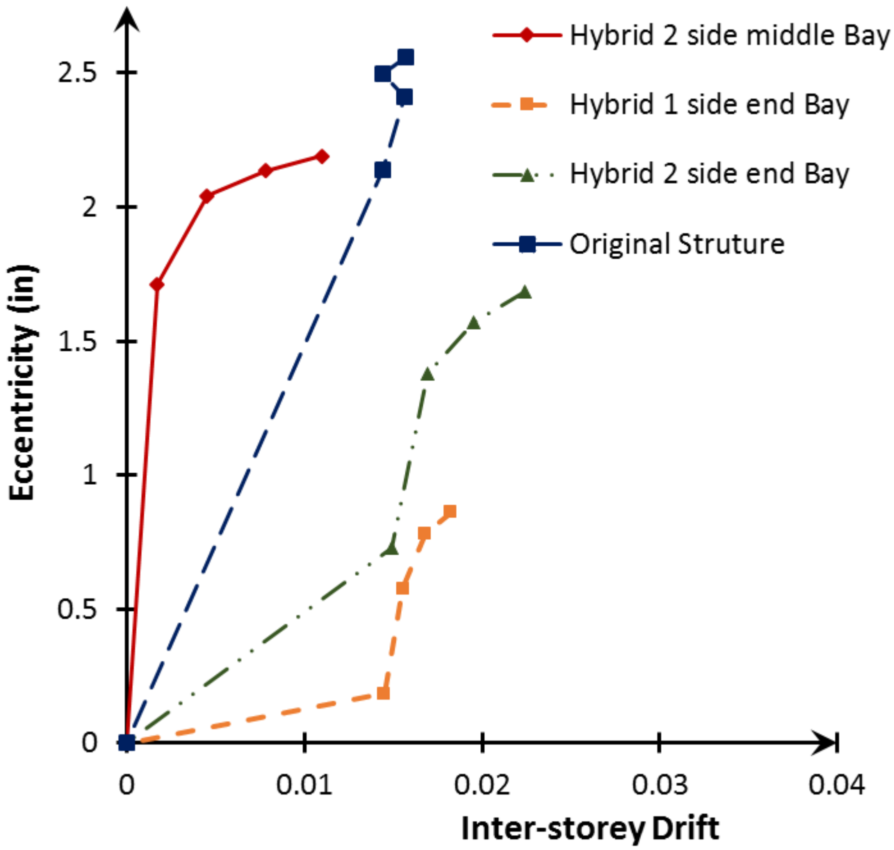


Figure 4-15 Comparison of eccentricity vs. inter storey drift

From the diagram, it is concluded that the location and orientation of the panels in the plan has a significant effect on the eccentricity and inter-storey drift. The optimum model that has been used in this analysis demonstrates an acceptable range of eccentricity and drift to retrofit the original structure.

5 Chapter: Conclusion and Future Work

5.1 Summary

Torsional irregularity is described as one of the major causes of damaging the buildings at earthquake events. A comprehensive review of methods of calculations of torsion is done. Most of the past research is based on the one storey and expanded to multi storey buildings. It is shown that the main cause of torsional effect in the buildings is because of the eccentricity between center of mass (CM) and center of rigidity (CR). Torsional provision evolution in the building codes, especially in the NBCC is reviewed and the rationale behind the formula in the code is shown by reviewing the relevant literature.

Hybridization is suggested as a method to retrofit the torsional irregular buildings. Hybrid structure and the material properties and the methods of hybridization are described. CLT panels are introduced as an alternative that can be used. The main advantages of using these panels are their low weight and rigidity in plane. A review is done the researches that have used these panels and their connections to the main structure in order to better understanding of the cyclic behavior of this material under lateral load. Seismic performance of hybrid structures in different level of hybridization reviewed.

Performance based design is described and performance objective according to NBCC 2010 is explained. In addition, the methods of considering torsion and different type of analysis for torsional irregular building are discussed.

A four storey concrete structure with shear walls is considered for this research. Linear dynamic analysis is conducted according to obtain the effect of torsional motion and irregularity on the inter-storey drift and seismic base shear. In addition, non-linear dynamic analysis is done in order to determine the performance of the building under 8 different ground motion scaled for the Vancouver seismic area. FEMA 356 guideline is used for the threshold of the allowable rotation for the beams and columns at the performance level.

5.2 Findings

- From linear dynamic analysis it is shown that the seismic base shear will increase almost 27% compared to the code static analysis for the original building to torsional effect.
- The torsional sensitivity index B_x is significantly above the code limit due to eccentricity of CM and CR in the floor diaphragm.
- It is shown that the effect of hybridization with 5 ply CLT wall on two sides of the building decreased the design base shear almost 40% less than the result from dynamic analysis on the original structure (Figure 4-3).
- Moreover, the inter-storey drifts for the original and hybrid structures were calculated. Although, both buildings drifts are below code threshold, the hybrid structure drifts for each floor are significantly lower than the original structure and it is very reasonable to assume they will satisfy the immediate occupancy performance level (Figure 4-2).
- In addition, the comparison between torsional sensitivity indexes shows that for the hybrid structure $B_x \approx 1$ which is way smaller than the code allowable 1.7, whereas for the original structure B_x is dramatically increasing from first floor (3.56) to the fourth floor (5.57).
- Parametric study is conducted to obtain an optimum pattern for the length of the CLT panel in the frame and the number and location of brackets for non-linear time history analysis. Two-dimensional analysis is used on a single bay four storey frames with 3

different panel length and connections (Figure 4-5 and 4-6). It is concluded that the inter-storey drift variation regardless of infill panel wall length is negligible (Figure 4-7). In addition, the bracket forces (Figure 4-8) has given reasonable values as compare to the given values to the program described in Chapter 3 (Table 3-10).

- The comparison for the inter-storey drift for the original and hybrid structures NLTHA for 8 different ground motion is shown (Figure 4-9 and 4-10). It can be seen that there is a gradual increase for the drift at each storey level for the hybrid model whereas the original model doesn't follow a uniform pattern for the drift. This can be resulted because of the sensitivity of the structure due to torsional motion.
- Seismic performance of each building is evaluated based on the criteria on FEMA 356 (Table 3-6). It is shown that the performance level of the original structure for median drift resulted from all ground motions does not meet the code requirement drift for the life safety of the columns. In addition, for the collapse prevention of columns the results are at boundary limit (Figure 4-11). However, the result from the median drift of the hybrid model indicates well performance of structural component for the life safety and collapse prevention.
- The effect of the location of wall panels on the inter-storey drift and eccentricity is investigated, using the result analysis from the two alternative models shown in Figure 4-13. It can be seen from the results on Figure 4-14 that the optimum orientation for the panels is the two side panel at the center bay of the building, because of the steady relationship between the eccentricity and the drift at each level. However, for the two other suggested panel location despite lower eccentricity the amount of inter-storey drift has increased significantly.

5.3 Contribution

The contribution of this research can be summarized as defining and developing a new and practical technique to reduce the extent of damage and improving the performance of the structures with torsional irregularity in an earthquake event using hybridization with CLT. The use of CLT panels to mitigate the torsional effect has never been explored in any other researches to date.

The cornerstones of the framework on this research was based on Torsional Irregularity, Hybridization, Linear Dynamic Analysis and Non-Linear Time History Analysis.

The developed framework can be applied as a typical procedure to study other types of rehabilitation techniques for variety of irregular buildings.

Using CLT in retrofitting the existing structures with torsional irregularity can reduce the need for reinforcing and redesigning the foundation. The panels have minimal weight comparing the other conventional materials (concrete, steel) and the energy will dissipate through the bracket ductile behaviour connected to the structural elements. Furthermore, in case of existing structures, interior non- load bearing walls can be replaced with the CLT panels and therefore no interior living space will be lost. CLT is accessible material in Canada and the cost of retrofit construction can be competitive comparing with other options.

5.4 Limitation of study and future work

- This study has focused on a four storey torsional irregular building which has been designed to the current code. Additional research can be done on the buildings with different storey numbers, which are not to the current code, in order to mitigate the effect of torsion. This method can be used to retrofit the existing structures and improve the performance of the existing buildings.

- An optimization tool can be developed to assist the designers to find an optimum length, location and number of brackets for the design and retrofitting the buildings with panels.
- Because the weights of the panels are negligible compare to the structure, the requirement of hold-downs and connection to the foundation can be investigated.
- For the purpose of this research only one horizontal component of selected ground motions is used. Further study can be done considering both component of the ground motion in non-linear time history analysis.

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Appendices

1.1 Appendix A

1.1.1 Material Properties

The definition of material specification is the first step to analyze and design.

Table 0-1 summarized the material properties of three common structural material; concrete, steel and wood.

Table 0-1 Material properties of steel and concrete and wood

Material properties		*Steel*	*Concrete*	*Timber*
Density (kg/m³)		7800	2400	400-600
Modulus of Elasticity (MPa)		200 000	20 000	8000-11000
Strength (MPa)	Compression	400-1000	20-40	Par:30 Perp: 8
	Tension	400-1000	2.0-5.0	Par:6 Perp: 1
	Yield	350	N/A	N/A

1.1.1.1 Concrete

Concrete is non-ductile isotropic material. This composite material is produced by mixing cement, water and granular components. Despite the strong compression, behavior concrete is weak in tension. However, because of same elongation coefficient with steel the combination of steel and concrete i.e. reinforced concrete has been used widely in structural members.

1.1.1.2 Steel

Steel is a ductile material that its isotropic specification provides the same behavior in all direction and through the material. However, the plasticity of structural steel may not be sufficient on certain loading condition and since the strength of steel is very high other failure such as local buckling and local instability may occur under certain loading conditions. In addition, because of high density of steel comparing to other materials using the minimum amount of material in the design procedure is always considered. As a result, possible deficiencies such as width to thickness ratio of columns, flexural buckling of bracing and columns, local buckling, beam torsion in the frame and effect of P-Δ are some challenges in the design of steel structures. Moreover, brittle failure is possible in some structures such as cases of pure tensile failure of screw connection, failure of welds and fatigue stress of connection used under cyclic loads.

Accordingly, the definition of material specification is important to analyze and design of structures using materials with inelastic behavior. Hysteresis behavior is known as the best way to achieve this goal. The need to accurate modeling of hysteretic behavior of material can be summarized is analytical behavior of inelastic structures which requires to elaborate the force deformation relationship under seismic loading which includes a number of variation factors. Some materials incorporate degradation in strength, stiffness and contribute the pinching effect due to cycling and dynamic loading which may cause to weakening and failure in the structure. Accordingly, to introduce the non-linearity of system different models has been introduced. The most common model identification of structural materials in the Bouc-Wen system (1960's) which includes a variety of hysteretic patterns and has the versatility to be used for most of materials. This method has been applied for most of engineering problems.

Steel structures have been around for centuries. The special behavior of steel under permanent and cyclic loading has increased the demand of using steel and established this material as considerable choice design engineers in designing different type of structures. One of the specifications of steel is high strength of steel in both compression and tension loading. However, strength degradation is an important factor that should be considered.

In steel moment frame degradation is mainly caused by buckling of the members (beams and columns) or non-ductile behavior of connection.

Researchers are indicating that the post yield stiffness doesn't change significantly because of local buckling where as a gradual degradation in strength is monitored (Ghodrati Amiri et al. 2012). However, for analytical modeling10% to 40% decrease in strength is considered.

Other type of degradation is fracture failure in beam flange weld that widely has happened in pre- Northridge connections. Fracture initiation in beam-flange connection and propagation to the column web or flange will decrease the plastic moment capacity of the connection.

1.1.1.3 Wood

Wood is an orthotropic material that means unlike steel and concrete the material property and strength will change in different orthogonal direction. Because, wood is a product that comes directly from nature, respectively the behavior of wood under loading is similar to the tree fibers. In other words, there are different variables that need to be considered in characterizing the wood behavior. In general, wood is strong parallel to grain or longitudinally and weak in perpendicular to grain and radial direction. Wood has different species that have their own characteristics such as D.Fir, SPF, Hem Fir. Growing condition also will cause different imperfections for wood such as knots(Lepper and Keenan 1986). Effect of rolling shear that is defined as shear stress causes the in plane shear strain perpendicular to the grain- direction. Due to very low rolling shear of timber, significant shear deformation is expected. Researchers have been done to determine the rolling shear modulus of timber (Blass, H. J.; Görlacher 2000).

1.1.1.4 Thermal property of steel and wood

Steel is sensitive to temperature change while wood doesn't show more expansion or shrinkage with thermal change. However the change of moisture content in wood has significant effect on its expansion and shrinkage. In the grain direction wood expands when heated and contracts when cooled. This change is referred to linear thermal effect on its expansion or contraction. In the grain direction wood changes dimension by about two millionth of its length per Fahrenheit degree change. Table A-2 summarized the effect of moisture on wood materials.

Table A-2 Effect of moisture on wood in different directions

Shrinkage	***Effect of moisture change in wood from wet to dry***
longitudinal	0.1-0.2 %
Radial	2.2-7.7%
Tangential	5-12.5%

1.2 Appendix B

CSA A23.3-04 is used in order to design the concrete beams, columns and slabs. Table B-3 and

Table B-1 summarised typical design procedures that are used for floor slab and beams in order to check the software output.

Table B-3 Typical procedure for design of floor slab

Description	***Values***	***Unit***			***Description***	***value***	***Unit***
Fy	*400*	*MPa*					
Φ_s	0.85				Dead Load	6.3	kPa
Φ_c	0.65				Live Load	2	kPa
λ	1	Normal concrete			Span	6000	mm
f'_c	35	MPa			Trib. Width	6000	mm
ε_{cu}	0.0035				M_f	49	kN.m
$E_c=4500\sqrt{f'_c}$	26622.4	MPa			V_f	196	kN
$f_r=0.6\lambda\sqrt{f'_c}$	3.5	MPa			Section	simple span	
E_s	200000	MPa			cover	25	mm
α_1	0.7975				D	175	mm
β_1	0.8825				Slab h_f	200	mm
As_{req}	914	mm^2/m			b'_T	1200	mm
#15M	5						
	Use 15M @ 200 mm						
<u>Max. Rebar spacing</u>			Spacing (S)				
3h	600		500	mm			
	500						
			S				
$1.4d_b$	27.3		30	mm			
$1.4\ a_{max}$	28						

Table B-1 Typical procedure for design of beams

General	value				Factored Load	
Dead Load	19.5	kN /m			24.4	kN /m
Live Load	7.5	kN /m			11.3	kN /m
F_y					400	MPa
Φ_s	0.85					
Φ_c	0.65					
λ_c	1	Normal concrete				
f'_c					30	MPa
α_1	0.8					
β_1	0.9					
$E_c=4500\sqrt{f'_c}$	22500	MPa				
$f_r=0.6\lambda\sqrt{f'_c}$	3.0	MPa				
E_s	200000	MPa				
Beam Flexure Reinforcement						
b	350	mm	0.5*h	233		
h	400	mm	ln/12	467		
Span	6	m				
M_f	160	kN.m				
cover	30	mm	Table A.2 for cover			
d	370	mm				
$A_{s\ req}$	1552	mm^2				
d_b	25	mm	Table A.1			
A_s rebar	500	mm^2				
# of provided	4					
$A_{s\ provided}$	2000	mm^2	>	1552		
Confirm that the strength requirement is satisfied						
a	124	mm				
M_r	**194**	**kN.m**	>	160		
Confirm max. tension reinforcement is satisfied						
$\rho=A_s/b_d$	0.015		<		$\rho_b=f'_c/1100$	0.027
			if not smaller than change the section			
Calculate Min. required bar spacing						
a_{max}	20	aggregate size				
$1.4d_b$	35	mm				
1.4 a_{max}	28	mm				
	30	mm				
S_{min}	35	mm				
Determine Min. width of beam						
b_{min}	345		<	350		
Stirrup diameter	10	mm				
(Real effective depth) d	**347**	**mm**				

Confirm min. reinforcement is satisfied

$A_{s,min}=0.2\sqrt{f'_c}/f_y*b*h$	383	mm^2	<	2000

Check the crack control parameter (Cl10.6.1)

$d_c=d_s$	52.5	mm		
$A_e=b(2*d_s)$	36750	mm^2		
A=	9187.5	mm^2		
(stress in steel) f_s	240	MPa		
$z=f_s(d_c*A)^{1/3}$	18822	N/mm	<	300000

Check the Moment considering compression reinforcement

A'_s	400	mm^2	2-15M
d'	40	mm	
c	124	mm	
ε_s	0.0022	if smaller than 0.002 repeat f'_s	
$f'_s=E_s\ \varepsilon_s$	400		
C'_r	136000		
a	99	mm	
M_r	219	**kN.m**	