Seismic Retrofit of Existing Steel Structures

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Abstract

The substantial demand for housing due to the uprising increase in population led to a surge in the number of structures built during the last century, a huge part of which are steel structures, however most of the construction occurred before the introduction of seismic codes. This has left the steel structures in the high seismic zones vulnerable. If not designed to have a fuse to dissipate the earthquake energy, the whole structure is at risk of severe damage of all of its members and ultimate collapse. With structures already erected it would not be feasible to demolish, redesign and rebuilt them using modern seismic codes. That's why seismic retrofit comes in play as an efficient and reliable solution for existing structures. Seismic retrofit could be achieved in two ways: by strengthening existing structural components or introducing new components to the existing structural system. Designers are always looking to find the most suitable retrofitting technique for vulnerable structures. This paper looks at retrofitting using steel plate shear walls, buckling-restrained braces and the strengthening of welded moment connections by welding an additional heavy shear tab or a straight haunch. In addition to that a case study reviewing 12 non-ductile concentrically braced frames to evaluate the frequency and severity of deficiencies is presented. The CBFs were reviewed based on the current special code provisions for CBFs.

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1. Introduction

1.1 History

Prior to the introduction of seismic codes all structures including steel buildings were designed according to ultimate limit state design and service limit design. This did not cause a problem especially in low risk seismic zones. Although Montreal's seismic hazard is low according to the National Building Code of Canada, structures in Montreal not meeting the modern seismic code requirements remain at high risk of failure. Moreover the structures are vulnerable because of the initial design method that didn't take into account the dissipation of energy and specifying a part of the structure as a fuse to dissipate this energy. However, the increase in population, the importance of the infrastructure, and the economic cost of such structures have led to an increased attention to preserve structures especially buildings in moderate and high seismic zones designed during the 1900s. With most buildings still fully functional the complete demolish and replacement of such structures comes at a hefty cost. Ignoring this problem can lead to safety hazards as well as to the reduction of the life span of structure. Seismic retrofit is presented as a cost effective method to modify existing structures and make them resilient to earthquake hazards. Different seismic retrofit techniques are discussed outlining their application, advantages and optimal use.

1.2 Objective

Due to the expected growth of population in Montreal, demand for residential buildings will increase. With most of the construction in Montreal completed during the 1960s and 1970s there is no need for new construction projects. Therefore attention has been shifted towards the preservation and protection of existing structures. One particular hazard is the seismic hazard, to protect old structures from earthquakes they have to be retrofitted. The objectives of this paper are to present different seismic retrofit techniques and discuss those techniques from different perspectives. The paper will discuss retrofitting by steel plate shear walls, buckling-restrained braces and the strengthening of welded moment connections (Bruneau et al., 2005). In addition to that a case study reviewing 12 non-ductile concentrically braced frames for current special seismic code provisions will be presented.

2. Case study

2.1 Non ductile CBFs

Concentrically braced frames are used to resist lateral loads due to their high stiffness achieved through brace tension and compression. The current codes include special provisions to ensure the frame behaves in a certain manner when subjected to lateral loads as seismic activity. The frame must have a component that acts as fuse to dissipate the earthquake's energy. This fuse is the brace in the braced bent, in the concentrically braced frame. In modern construction a steel honeycomb damper could be place within the bracing bent for better energy dissipation (Lee et al., 2017). If the brace capacity is higher than the probable force from the earthquake then another part of the frame will fail. Moreover if the connections in the bracing bent fail then the forces won't be transferred to the braces and another part of the structure will dissipate the energy. If another member of the structure fails, the structure is at risk of ultimate collapse during the earthquake event. To ensure this doesn't happen the current codes include special provisions for the design of bracings in concentrically braced frames designed for seismic events. Provisions for global and local slenderness limits for the brace, as well the connections are designed based on expected tensile and compressive capacities of the braces. The gusset plates are designed with a geometric proportion to allow the brace to rotate as it buckles out of plane. These provisions encourage a yield sequence that starts with the fuse element, as the first point of dissipation of energy, which is the brace. Prior to the adoption of the modern standards, CBFs were designed for gravity loads with no regards for the yielding sequence or in other words without considerations of principles based on plastic mechanisms (Grande & Rasulo, 2015). These CBFs are referred to as non-ductile CBFs or NCBFs.

2.2 Seismic evaluation of 12 NCBFs

A case study reviewing 12 NCBFs was done to evaluate the frequency and severity of deficiencies. The CBFs were reviewed based on the current special code provisions for CBFs (Sen et al., 2017). Deficiency severity was primarily evaluated using demand-to-capacity ratios DCR. Where the demand is based on the expected brace force, calculated using R_yF_y, where a probable factor (R_y) is multiplied by the yield stress of steel (F_y), to ensure that the material strength is not under estimated since the material would normally have an actual yield stress higher than the yield stress specified by the supplier, and this requirement ensures that the brace will yield. The capacity is calculated from the appropriate design expressions, including expected material properties for steel plates and structural members and neglecting resistance factors. The gusset plate connections were evaluated for the resistance limit states and geometric limits like the N_{tp} clearance as shown in Figure 1.

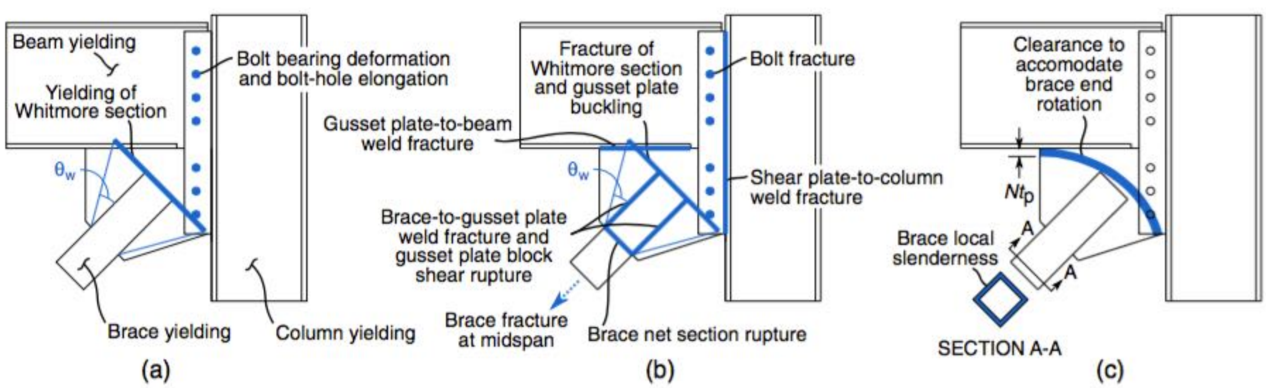

Figure 1: (a) yielding mechanism, (b) failure modes, (c) geometric limits for gusset plates

Source: Sen et al. (2017)

The gusset plate-yielding limit was calculated using a Whitmore section. The bolt and weld capacities were calculated using the Instantaneous center of rotation method. The gusset plate clearance to allow unrestricted rotation of the brace was compared to the 8 t_p elliptical clearance recommended by (Lehman et al., 2008). After studying the CBFs with their existing gusset plates and evaluating them according to the current standard, It was found that the welds and bolt groups capacities are smaller than the probable expected brace force calculated using R_yF_y and therefore the connection is at risk of failure before the brace yields. Secondly the brace local slenderness limits exceeded the current allowable limits and the gusset plate clearance to allow brace-end rotation was very small or non-existing in respect to the 8 t_P criterion, which means the plate is at risk of failure from buckling of the brace. The NCBFs were then retrofitted to address the key deficiencies listed above to improve the seismic performance of the braces and the connection configurations. The NCBF deformation capacity could be improved through the following measures: replacement of the braces with new braces meeting the current code thickness and slenderness limits, connection reinforcement with welds or bolts, and protection of welds on the gusset plates from out of plane rotation demands by using buckling restrained braces. This case study was presented to show that older structures designed prior to modern day seismic requirements in fact lack the required detailing that would ensure a yield sequence that starts with the fuse element, as the first point of dissipation of energy, which is the brace and that means that other integral structural members are at risk of failure during an earthquake event which might lead to the ultimate collapse of the structure.

2.3 Seismic retrofitting techniques

As the case study outlined the importance of revising older structure according to modern day code requirements, the next step is to retrofit those structures using one of the various seismic retrofitting techniques.

3. Modification of welded moment connections

3.1 Connections in moment frames

The poor performance of moment-resisting connections, which are designed to resist wind and seismic forces, is well documented in literature in studies following major earthquakes. Hence the seismic retrofit of these connections in existing steel structures is much needed. Since moment connections are usually employed to moment frames and since most of steel moment frame buildings have composite floor slabs, it is of great interest to study the retrofit techniques for conventional moment connections that are connecting beams, carrying a composite slab. To study the effect of the detailing of the beam to column connection on seismic performance, in an attempt to reinforce existing welded steel moment connections with highly composite floor slabs; welded flange and bolted web connections were built to replicate existing connections, and then the connections were retrofitted.

3.2 Retrofitting schemes

The retrofit techniques are heavy shear tabs, welded straight haunch and welded triangular haunch. The retrofit techniques were chosen because when modifying existing connections for improved seismic performance, the presence of a concrete floor slab often causes construction constraints and dictates modification in the bottom flange of the beam only and so the straight and triangular welded haunches are tested. Strengthening existing beam web by using heavy shear tab can be a viable alternative when even the bottom flange modification is not possible. Thus trying to retrofit the beam with heavy shear tabs is to reduce tensile strain demand on the beam flange by increasing the plastic section modulus of the retrofitted beam web, and to push the plastic hinging away from the brittle welding area to the inner ductile area. So 4 specimens were tested to compare the different modifications, the different specimens are shown in Table 1 (Kim Sung-Yong & Lee, 2017).

Table 1: Summary of specimens with their respective retrofitting scheme

/	Retrofit scheme	Figure
1	None	2
2	Heavy shear tab	3
3	Straight Haunch	4
4	Triangular Haunch	5

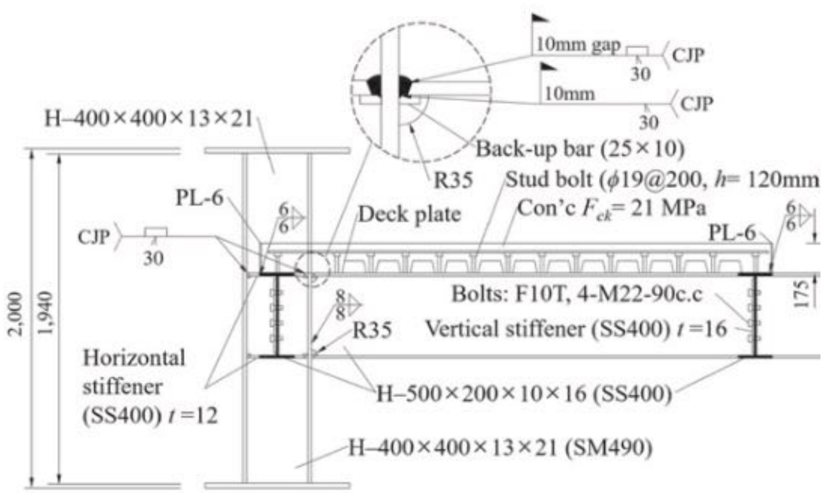

Figure 2: Beam to column connection without any retrofit

Source: After Kim Sung-Yong & Lee (2017)

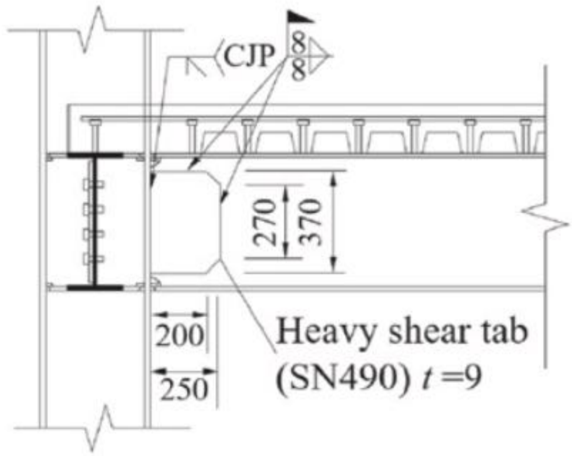

Figure 3: Heavy shear tab at the beam and column interface

Source: After Kim Sung-Yong & Lee (2017)

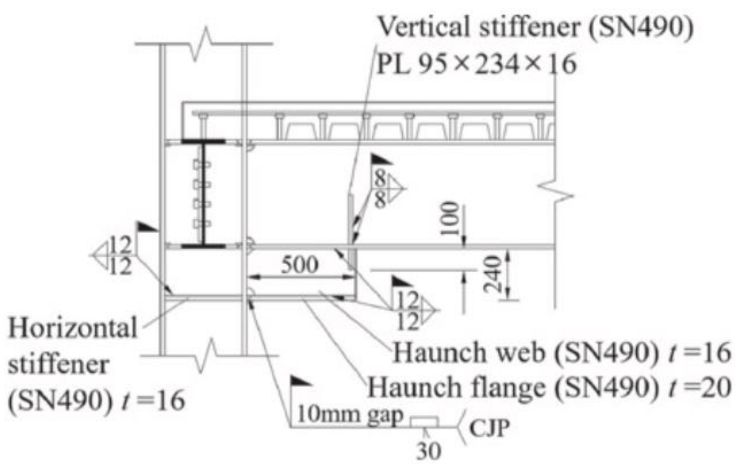

Figure 4: Welded straight haunch between beam and column

Source: After Kim Sung-Yong & Lee (2017)

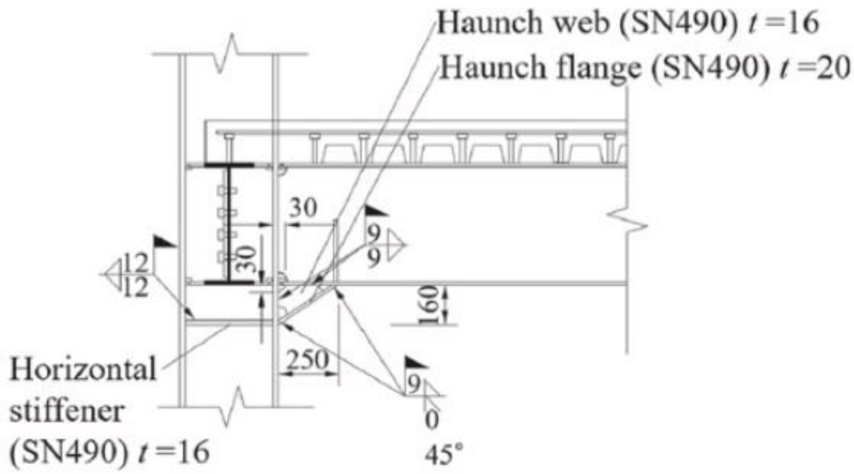

Figure 5: Welded triangular haunch between beam and column

Source: After Kim Sung-Yong & Lee (2017)

3.3 Retrofitting scheme results

The bare steel specimen exhibited connection plastic rotation of 5% rad, which is more than the 3% rad that is the minimum requirement for special moment frames. On the other hand the specimen exhibited severe local and lateral buckling at the point of hinging. For the heavy shear tab specimen, brittle fracture was delayed thanks to the web strengthening as well as the connection ductility and energy dissipation capacity was significantly improved in comparison to the bare steel specimen. The specimen showed optimal strength up to 4% rad plastic rotation and finally fractured at the very high value of 7% rad but the local and lateral buckling were less severe than the bare steel specimen. Finally both haunch-retrofitted specimens 3 and 4, showed similar and excellent plastic rotation capacity exceeding 5% rad without fracture. The plastic hinging was successfully formed outside the haunch region. Table 2 shows the positive strain-hardening factors for all specimens, in fact positive strain hardening was higher than the bare steel for all specimens.

Table 2: Positive strain hardening factors

		Computed at the tip of strengthened zone		
	Bare steel connection	Heavy shear tab	Straight haunch	Triangular haunch
Positive strain hardening	1.165	1.386	1.610	1.472

Source: After Kim Sung-Yong & Cheol-Ho (2017)

Connection improvement by using modification schemes like shear tabs or haunches provides a structural, energy dissipation fuse at the critical section of the beam. In such connections with a promoted plastic zone, the increased strain demand is relieved. The use of heavy shear tabs and triangular or straight haunches effectively pushed plastic hinging outside the strengthened region and exhibited excellent total plastic rotation capacity exceeding 4% rad enhancing the seismic performance of the connection.

4. Buckling-Restrained Braces

4.1 Applicability of BRBs

Brace replacement with a buckling restrained brace is another alternative considered for use with deficient gusset-plate with small brace-end distances, as the BRB has smaller brace end rotations. The BRB could also be used to increase the buckling capacity and so the overall capacity of the brace. BRB retrofit may be attractive because BRBs can span long distances and have greater fracture resistance and energy dissipation capacity than a standard HSS or L shaped angle. However, BRBs place large, concentric force demands on the connections, beams, and columns, which can cause unintended failure modes and to account for this, the BRBs are bolted to the gusset plates, doubler plates are welded to the beam web, to reduce local damage (Palmer et al., 2016). In cases where the column web is thinner than the gusset plate, then doubler plates are welded to the columns as well. With the use of BRBs with the modified connection configurations the brace capacity is improved and the gusset plate is at less risk of failure before the brace reaching its probable capacity.

4.2 BRBs mode of work

Conventional bracings lose a great value of their capacity in compression, due to buckling of the brace under seismic loading. BRBs have very efficient buckling behavior than traditional concentric braces. The BRB consists of a steel core, which is encased by concrete as shown in Figure 6.

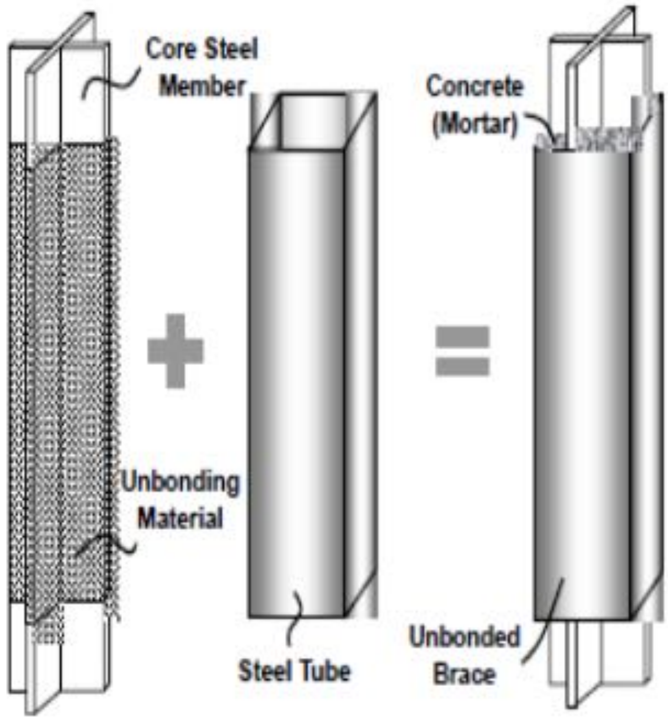

Figure 6: Schematic of BRB

<u>Source: </u>After Surendran & Varma P (2017)

The concrete filling around the steel core provides the required confinement during cyclic loading. The resisting element is the steel component, and the outer casing counteracts the overall buckling of the steel core.

4.3 BRBs vs. Conventional Concentric Braces

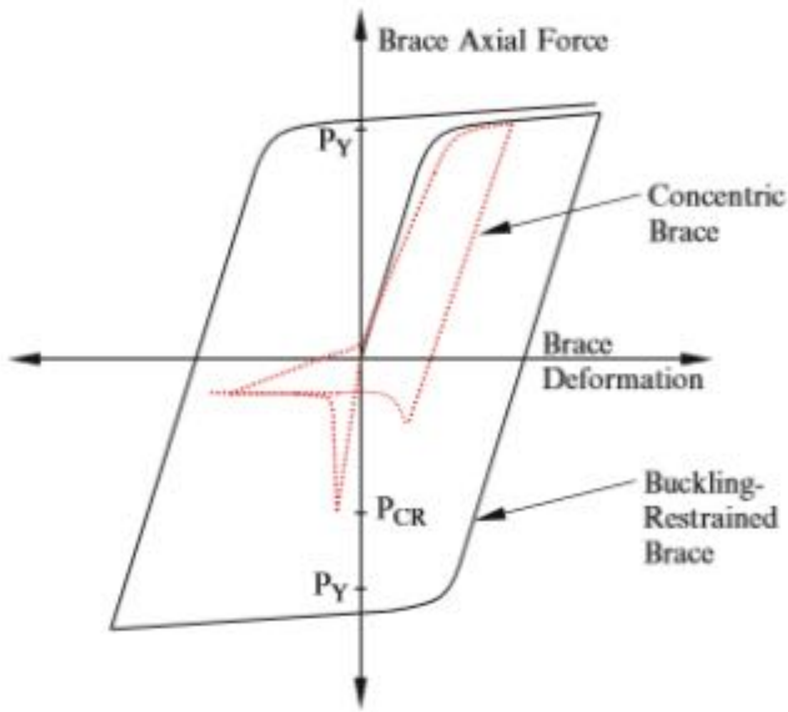

Figure 7: Hysteresis plot for BRB vs. Conventional Concentric Brace

Source: After Surendran & Varma P (2017)

As shown in Figure 7, the hysteresis behavior of the BRB is much more stable in comparison to a regular concentric brace, in other words the BRB has a more uniform force-deformation curve during tension and compression cycles. On the other hand the conventional brace performs well during the tension cycle and experiences buckling during the compression cycle, after the brace buckles, it loses strength and so the compression axial load is much lower than the BRB. But low compression cycle capacity leads to low energy dissipation and low deformation ductility in comparison to BRBs. To ensure this compression behavior of the BRB, there is only one requirement that the outer steel tube must have an elastic buckling strength greater than the yield strength of the steel core. Finally BRBs are easily connected to the existing structural systems, which means they are easy to adopt for seismic retrofitting and usually they don't require additional structural members or foundation strengthening. They exhibit stable hysteretic behavior, high-energy dissipation capacity and limited sensitivity to environmental condition changes due to protection by the outer core. In short a BRB acts as a structural fuse and during a seismic event damage is concentrated in the BRB element, as it has the ability to fully yield in tension and compression, giving the structure excess dissipation capacity unlike conventional concentric braces (Wigle & Fahenstock, 2010). A dual system consisting of the steel frame and BRBs can outperform a similar system in which the frame is assigned a larger role in seismic resistance (Terán-Gilmore & Ruiz-García, 2011).

4.4 BRBs vs. special moment frames

To sum up according to the results obtained in the study of seismic performance of BRBs and moment frames dual systems by (Mehdipanah et al., 2015), it was shown that when the relative stiffness ratio in the subsystems are set in a way that BRBFs are designed for 65% of lateral forces and 35% for the moment frame; a better seismic behavior is achieved. Which again confirms how BRBs could enhance the seismic performance of the structure.

5. Steel Plate Shear Walls

5.1 Steel plate as a shear wall for resisting lateral loads

A steel plate shear wall exhibits high stiffness when subjected to cyclic inelastic loading, behave in a very ductile manner and dissipate significant amounts of energy, which makes it suitable to resist seismic loadings. A steel plate shear wall is a lateral load resisting system that could act as a retrofit, vertical steel plates that are connected to surrounding beams and columns and installed in one or more bays along the full or partial height of the structure. The steel panels experience shear forces when subjected to lateral loading. When the web panel is subjected to shear, equal principle tensile and compressive stresses develop within the panel, when compressive stresses exceeds critical stresses, the panel buckles elastically which doesn’t limit its shear capacity, the steel panel is able to carry additional loads due to diagonal tension field action developed post buckling (Alinia et al., 2012). Current AISC and CAN/CSA standards provide clauses with the panel allowed to buckle and develop diagonal tension field. In addition to the lateral resistance provided by the shear wall post buckling, the interaction between the shear wall and the surrounding frame members provides additional lateral resistance that would improve the overall seismic resistance of the structure.

5.2 Retrofitting using a steel plate shear wall

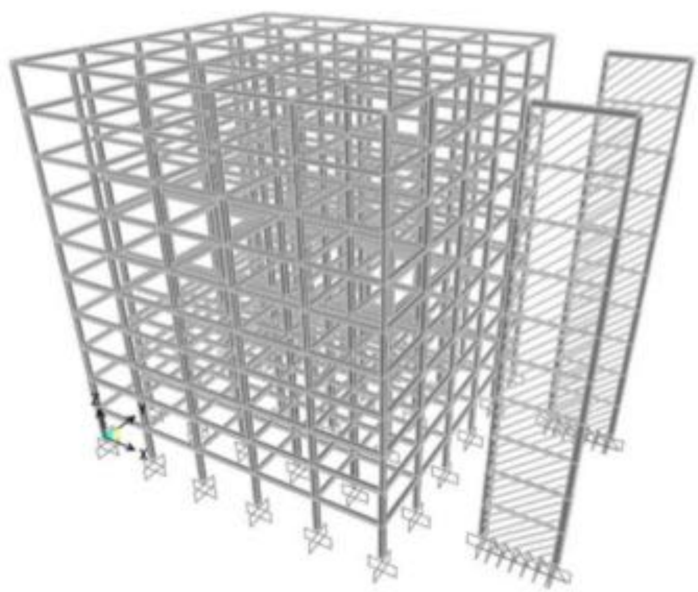

Figure 8: An overview of steel shear wall and frame combination

Source: After Mahtab & Zahedi (2008)

A 10 story steel frame structure model was studied. For seismic retrofit, two bays of the model were retrofitted using steel plate shear walls. In order for the model to represent a retrofit, the plate shear walls were designed and modeled separately, to predict their behavior and then were added to the initial 10 story structure, and the whole frame was evaluated to compare the behavior with and without the shear wall (Mahtab & Zahedi, 2008). The seismic base shear was used to conduct a non-linear static analysis using SAP2000, for evaluation the model was pushed to target displacements under the base shears and the results for the different models are shown in Figure 9.

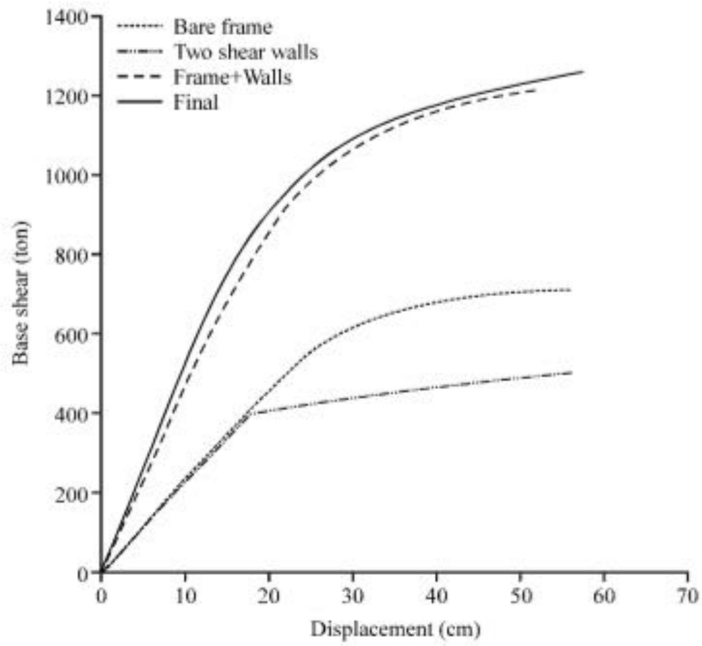

Figure 9: Target displacement vs. base shear for different model cases

Source: After Mahtab & Zahedi (2008)

As shown in Figure 9 by observing the base shear absorbed by the shear walls and the frame on its own, it is clear that the base shear carried by the frame and the walls together is much higher than the frame on its own for the same target displacement, so in fact the shear walls are providing beneficial ductility to the system hence the displacement of the frame is much less with the shear walls than with the frame on its own. This additional capacity is beneficial since the structure may encounter higher seismic fortification intensity than initially designed for (Li et al., 2012). The reaction of the structure to higher base shears is very important, and with steel plate shear walls the structure could still meet target displacements for higher base shears than what was initially adopted.

6. Conclusions

Most structures were designed and built before the introduction of seismic codes and consequently they were designed for ultimate limit state gravity loads only. This leaves such structures vulnerable to seismic events, which places the lives of the occupants at risk shall, a seismic hazard take place. As the case study outlined the importance of revising older structures according to modern day seismic code requirements, seismic retrofit is introduced as a cost-effective technique of modifying and renovating the structure without the need of demolishing it and reconstructing it. Three retrofit techniques were discussed in terms of their applicability, mode of action and seismic performance enhancement when they are added as retrofits. The use of heavy shear tabs and triangular or straight haunches effectively pushed plastic hinging outside the strengthened region and exhibited excellent total plastic rotation capacity exceeding the required 4% rad enhancing the seismic performance of the connection. BRBs perform better than concentric braces and enhance the seismic performance of moment frames when added as a retrofit. Steel plate shear walls provide additional ductility to the steel frame and the dual system meets target displacements at higher base shears in comparison to the frame on its own. The different retrofit techniques are unique; the bottom line was to show that each of those techniques would provide better seismic performance if added to the steel structure. There is no retrofit technique that is superior to the other but the choice of the retrofit would rather depend on the situation, and the nature of the structure to be retrofitted.

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