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Seismic Retrofit of Steel Frames Using Steel Plate Shear Walls

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Abstract: In this study, by retrofitting a ten story bending frame an economical comparison between two lateral load resisting system, steel plate shear wall and cross bracing system has been done. For this purpose by using a series of trial and error processes the considered structure retrofitted by two mentioned systems and evaluated by pushover analysis in accordance with FEMA 356. Finally, by comparison of these two retrofitting methods, it is observed that retrofitting by a steel plate shear wall, the use of the existing frame will be optimized and also, by using the steel plate shear wall, the consumed steel volume in retrofitting is about 30% lower than bracing and if using the minimum 3 mm thickness due to practical consideration, it will be about 15% lower than bracing.

Key words: Retrofit, non-linear static analysis, steel plate shear wall, pushover, brace

INTRODUCTION

The Steel Plate Shear Walls (SPSW) has been used as the primary lateral load resisting system in the high-rise buildings in the recent three decades. This structural system that has spread increasingly in the word has been utilized in constructing of new buildings and also in retrofitting the existing buildings, especially in countries with seismic vulnerability such as USA and Japan. Utilizing such structural system comparing to moment restraining steel frames has been resulted in 50% saving in steel construction. Considering the fact that lots of the existing buildings have been constructed in the seismic vulnerable zones without applying new standards of the building codes and they are not safe against earthquake, we can almost imagine the widespread disaster, which the cities will be faced with. So, the first reaction of the structural engineers regarding the unsafe structures against the earthquake is to retrofit the lateral load resistance of these structures. Presenting an effective option for the seismic retrofit, depends on many factors, such as construction and implementation costs, ease of implementation, availability of material and the minimum disruption to the function and occupants of an existing building. Also, the retrofitting plan should include an optimal combination of resistance, rigidity and ductility.

Using steel plate shear walls, concrete shear walls, x, k, Λ bracing and post tensioned bracings are amongst the most common retrofitting methods. Purpose of this research is economical comparison between two retrofitting methods, using cross bracing and the steel plate shear walls.

STEEL PLATE SHEAR WALLS

In general, steel plate shear wall system consist of steel plate wall, two boundary columns and horizontal floor beams. The steel plate wall and two boundary columns act as a vertical plate girder, shown in Fig. 1. The columns act as flanges of the vertical plate girder and the steel plate wall act as

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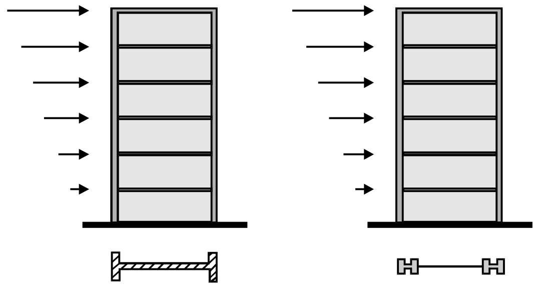

Fig. 1: Typical plate girder and steel plate shear wall

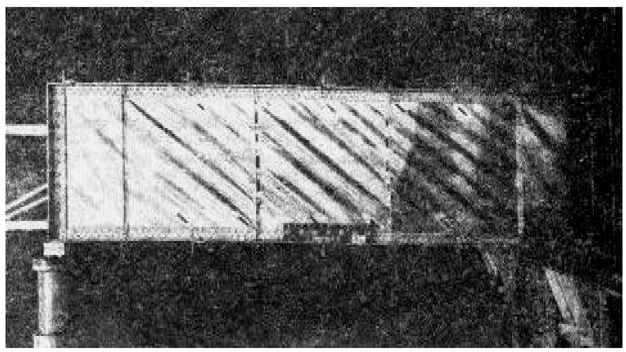

Fig. 2: Panel after the web buckling (steel plate)

its web. The horizontal floor beams act, more-or-less, as transverse stiffeners in a plate girder. In early application of steel plate shear walls, the walls had vertical and horizontal stiffeners. In Japan and United States, almost all of the steel plate shear walls are stiffened and it results in increasing the shear yield strength of the wall. But welding such stiffeners to steel wall can be costly as well as time-consuming. So, multiple studies and tests were conducted in Canada, United States and Japan on the steel plate shear walls without stiffeners.

The main idea of using the steel shear walls without stiffeners is to utilize the diagonal tension field action developed in plate after buckling of the plate (Fig. 2).

The above mentioned phenomenon is called post-buckling. This phenomenon is so well-known in plate girders. Plate buckling is not similar with failure and if the plate is adequately supported along its boundaries, as in the case of shear wall, the post buckling strength can be more than several times the theoretical buckling strength and it can provide substantial stiffness and ductility. The idea of utilizing the post buckling strength of steel plate shear walls was first formulated by Thorburn *et al.* (1993) and verified experimentally by Timler and Kulak (1993). Studies performed to evaluate strength, ductility and hysteretic behavior of such SPSW designed with unstiffened infill plate demonstrated their significant energy dissipation capabilities (Caccese *et al*., 1993; Elgaaly *et al*., 1993; Driver *et al*., 1997; Rezai, 1999; Astaneh-Asl, 2001).

Analysis and Design of Steel Plate Shear Walls-CAN/CSA S16-01

The CAN/CSA S16-01 seismic design process for steel plate shear walls follows the selection of a lateral load resisting system (i.e., shear walls with rigid or flexible beam-to-column connections), calculation of the appropriate design base shear and distribution of that base shear along the building height by the usual methods described in building codes. Preliminary sizing of members is done using a model that treats the plate at each story as a single pin-ended brace (known as the equivalent story brace model) that runs along the diagonal of the bay (Fig. 3a).

From the area of the story brace, A, determined from that analysis, an equivalent plate thickness can be calculated using the following Eq. 1 based on an elastic strain energy formulation (Thorburn *et al*., 1993):

$$t = \frac{2A_b\Omega_s \text{Sin}\theta}{L\sin 2\propto} \quad (1)$$

where, θ angle between the vertical axis and the equivalent diagonal brace, L is the bay width; Ω_s is the system overstrength factor, as defined by FEMA 369 and taken as 1.2 for SPSW (Berman and Bruneau, 2003) and α angle of inclination of the principal tensile stresses in the infill plate measured from vertical, which is given by:

$$\tan^4\alpha = \frac{1+\dfrac{t_w.L}{2A_c}}{1+t_w.h_s\left[\dfrac{1}{A_b}+\dfrac{h_s^3}{360\,I_c.L}\right]} \quad (2)$$

where, t is thickness of the plate A_c and I_c are the cross-sectional area and moment of inertia of the bounding columns, respectively; h_s is story height and A_b is beam cross-sectional area (Timler and Kulak, 1993).

CAN/CSA S16-01 also provides the following equation to ensure that a satisfactory minimum moment of inertia is used for columns in steel plate shear walls to prevent excessive deformation leading to premature buckling under the pulling action of the plates (derived from Kuhn *et al*., 1952).

$$I_c \geq \frac{0.00307\,t.h_s^4}{L} \quad (3)$$

Once the above requirements have been satisfied, a more refined model, known as the strip or multistrip model, that represents the plates as a series of inclined tension members or strips (Fig. 3b) is required for the analysis of steel plate shear walls (with α as calculated by Eq. 2). Through

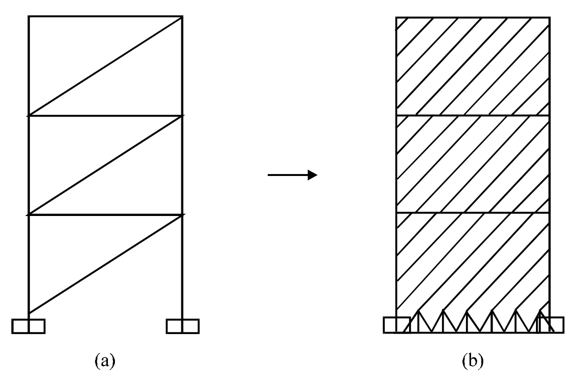

Fig. 3: The equivalent story brace and strip model

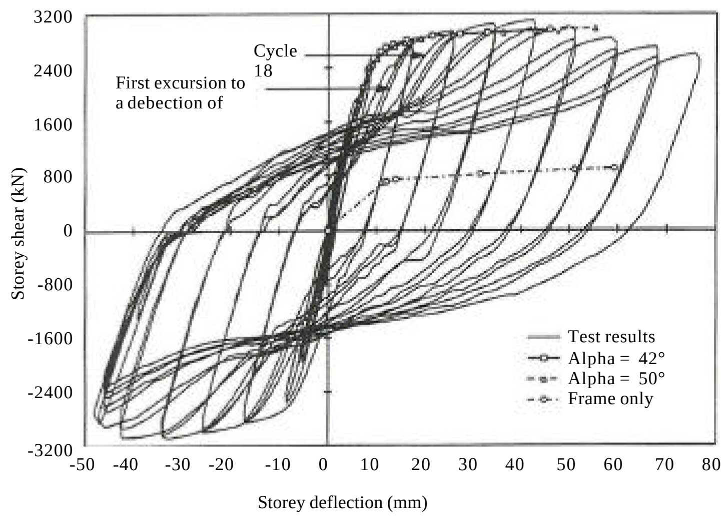

Fig. 4: Comparison between experimental results and strip model (Driver *et al*., 1997)

comparison with experimental results, the adequacy of the strip model to predict the ultimate capacity of SPSW has been verified in several studies. Figure 4, adapted from Driver *et al*. (1997) is one example of this verification.

A minimum of ten strips is required at each story to adequately model the wall. Each strip is assigned an area equal to the plate thickness times the tributary width of the strip. Drifts obtained from the elastic analysis of the multistrip model are then amplified by factors prescribed by the applicable building code to account for inelastic action and then checked against allowable drift limits. For SPSW having rigid beam-to-column connection, CAN/k#Canadian Standard Association (CSA, 2001). Limi S16-01 also requires that a capacity design be conducted to prevent damage to the bounding columns of the wall.

Due to practical considerations, infill thickness may be larger than necessary to resist the seismic loads, therefore, capacity design is required to insure a ductile failure mode (i.e., infill yielding prior to column buckling). To achieve this, the moments and axial forces (obtained from an elastic analysis) in this columns are magnified by a factor B, defined as the ratio of the probable shear resistance at the base of the wall for the supplied plate thickness, to the factored lateral force at the base of the wall obtained from the calculated seismic load. The probable resistance of the wall (V_{re}) is given by

$$V_{re} = 0.5R_y\, F_y\, t_w\, L_{cf} \sin 2\alpha$$
$$B = \frac{V_{re}}{V_u} \tag{4}$$

where, R_y is Ratio of the expected (mean) steel yield stress to the design yield stress (specified as 1.1 for A572 Gr. 50 steel) ; F_{ye} is Design yield stress of the plate and all other parameters have been defined previously.

Note that B need not be greater than the ratio of the ultimate elastic base shear to the yield base shear, which is the ductility factor R_μ specified as 5.0 by CAN/CSA S16-01. Column axial loads are found from the overturning moment BM_f where, M_f is the factored overturning moment at the bottom of the wall. Local column moments from tension field action of the plate, as determined from the elastic analysis, are also amplified by B. If a nonlinear pushover analysis is carried out, these corrections need not be done and more accurate values for the column axial forces and moments can be obtained. Since pushover capabilities are becoming more common in structural programs, this is also a viable option.

DESIGNING THE ORIGINAL MODEL

The model under study is a 10 story steel frame building with plan dimension of 25 m in both the N-S and E-W direction. The floor plan is shown in Fig. 5. The frames are moment-resisting and the floors are one way slab. Gravity loading and lateral loading is according to Iranian codes. By selecting the intermediate bending frame system the corresponding base shear calculated. Since of retrofitting process, we need structure which is capable of retrofitting, so designing was performed only for 70% of the obtained base shear. The selected profile used for the beams was of HE-A type and for the columns was of box type.

Seismic Retrofit by Steel Plate Shear Wall

For seismic retrofit of the existing model, two bay of the model retrofitted by steel plate shear wall (Fig. 6). The program conducted for seismic retrofit of considered structure by the steel plate shear wall is in such a way that at first, the walls is designed separately and then will be added to the existing frame and then the whole structure will be evaluated. This is done because the frame and wall behaviors are observed both separately and in combination with each other.

To design the steel plate shear walls, firstly we calculate the appropriate design base shear. For dual system with special steel moment frame and unstiffened steel plate shear wall, appendix R of the American Seismic Provisions, suggested response modification factor equal to $R = 8$. Thus having R, the seismic force will be calculated by the equivalent static method. Given the fact that around 30%

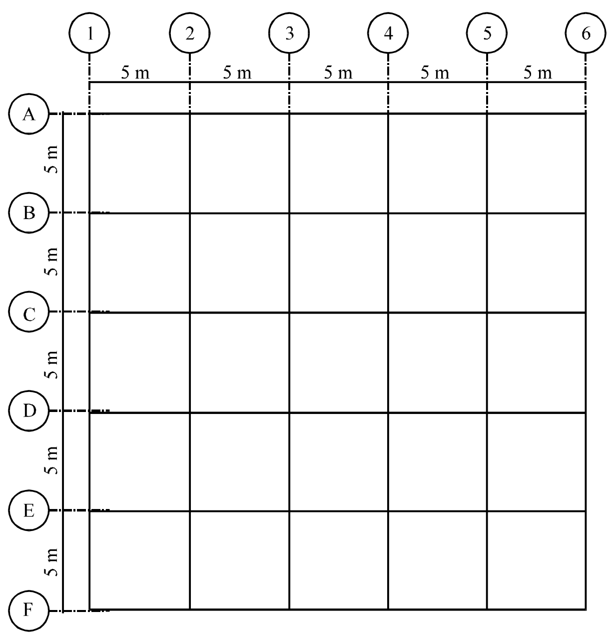

Fig. 5: A ten story building plan with an intermediate bending frame system

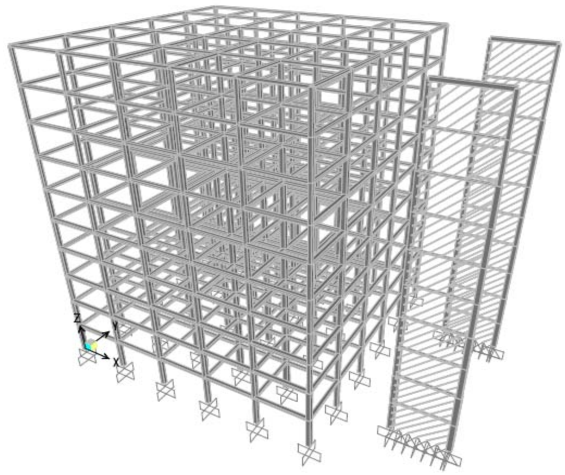

Fig. 6: A view of the steel shear wall and frame combination

of the base shear of the whole structure should be endured by steel plate shear walls, we will proceed by designing the desired walls. Because of two walls are used for retrofitting the considered structure, each wall will be designed for half of the obtained base shear. Designing of the wall is according to CAN/CSA S16-01. Designed walls will be connected to the existing structure by the diaphragm (Fig. 6), then the evaluation of the retrofitted structure would be completed.

For this purpose, non-linear static analysis were conducted, using SAP 2000 (Version 10.1) program according to provisions of FEMA 356. Also for all beams, columns and strips, non-linear hinges that define non-linear force-displacement or moment-rotation behavior, assigned to discrete locations along the length of frames. Also target displacement calculated according to structural condition and the model pushed over to that displacement.

The strip model was used to represent the infill walls. For modeling the axial elastic-plastic behavior of strips, a non-linear hinges was used. The strain hardening effects are calculated considering a 3% grade of the elastic section. Considering the evaluation of the structure in the target displacement (45 cm), lots of columns, especially those around the shear walls, did not respond to the generated forces and this suggested that the shear wall absorbs more shear because of its much more stiffness, so more destruction is taking place in it. Thus, it can be concluded that the resistance of designed wall is not sufficient and a wall with more bearing strength should be designed. So, as a new step the steel plate shear walls for two times the previous force, i.e., 60% of the base shear of the whole structure would be designed.

Again, with evaluating the retrofitted structure in the target displacement (35.6 cm), it is observed that only the first floor columns of the shear wall can not respond to the existing forces which are retrofitted again and finally they should be added them to the previous frame. Therefore, the steel plate shear wall column's cross section is added to the previous frame's columns as the equivalent plates. Also by replacing the beams, in which the moment of inertia of wall beams and the existing beams are combined, an equivalent beam was selected approximately. Graphical display of the final model in target displacement is shown in Fig. 7.

In Fig. 8, the base shear-displacement curve of the considered model before combination (frame+wall) and the behavior curve of the model after combination (final) are displayed.

Also, in order to compare, the primary base shear-displacement curve of the bending frame before retrofitting (bare frame) as well as the base shear-displacement curve of the two walls together (two

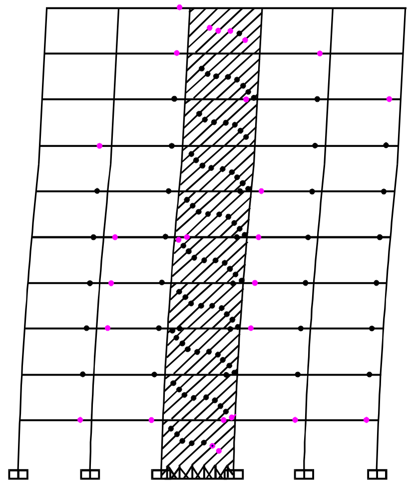

Fig. 7: Graphical display of the final model in the target displacement (final model)

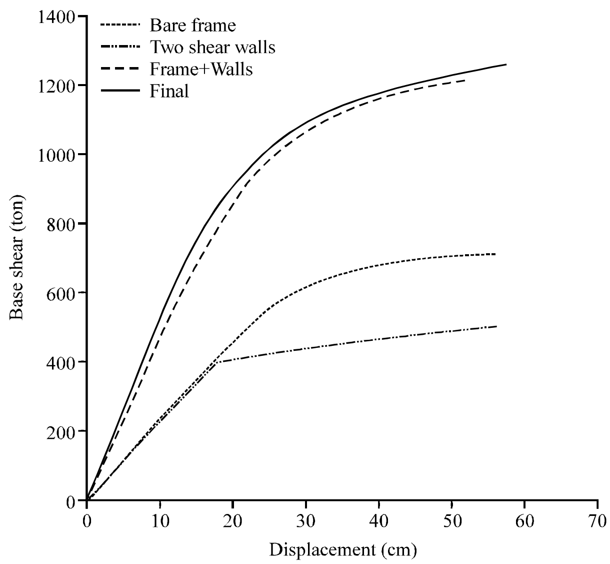

Fig. 8: The frame base shear-displacement curve alone, two walls together, combination of the frame and wall and the final design

steel plate shear walls) are shown in Fig. 8. With careful attention to the wall's behavior curve, it is observed that the value of shear yield of the steel plate shear wall is compatible with the theoretical values due to Eq. 4.

Applying a Minimum 3 mm Thickness for the Web Plate

In the two designed shear walls, the obtained thickness for the wall plate varied from 1.5- 3.5 mm. In order to apply minimum thickness of 3 mm for the practical consideration, the wall plates in the final model will be 3 mm from the third floor on. In this condition, checking and designing the columns around the wall has been like the previous stages by calculating the target displacement and pushing the model to that target displacement.

Taking control of the resulted deformation and forces in other frame members, such as beams and columns, all of them had provided acceptance limits of Life Safety (LS). Also, checking out the created strains in the web strips, the maximum strains created in the web strip didn't exceed from 5.9 Δ_y (Δ_y is yield deformation) and this amount of deformation in strips is considerably lower than the steel deformation capacity.

It should be mentioned that the regulations related to the steel plate shear walls mentioned in FEMA 356 is related to the walls with stiffeners and there is no discussion regarding the walls without stiffeners and only the method of their modeling (strip model) has been mentioned. But considering the conducted tests and researches in majority of the tested specimens, the test has ended with local buckling of the column or failure where the column joints to the base plate; while the web plate yield and its yielding (which is an energy absorption mechanism) has been done slowly and stiffness is decreased gradually during that stage. The important reason that yielding of the plate has no influence on the sudden decrease in stiffness is that the continuity of plate has a great influence on redistributing the loads in surfaces which are not yielded and this redistribution helps the lateral load bearing resistant system. Thus, in target displacement the columns will control the operation.

SEISMIC RETROFIT BY CROSS BRACING

Concentrically Braced Frames (CBFs) are commonly used in new and retrofit construction to resist earthquake by providing lateral stiffness, strength and ductility. Therefore, this system is utilized to retrofit the given moment resisting frame.

For this purpose, at the first step, only two bays of the existing structure retrofitted by concentrically braces as shown in Fig. 9a. Then the appropriate design base shear calculated and the structure designed for new position. After that the retrofitted structure evaluated by non-linear static analysis method (Pushover) accordance with previous stage.

In the evaluation of the structure in this step, it is observed that in the target displacement (32.7 cm) 70% of the bracings and 40% of the boundary columns around the bracing does not provide the acceptance limits of LS. So, the structure must be retrofitted again.

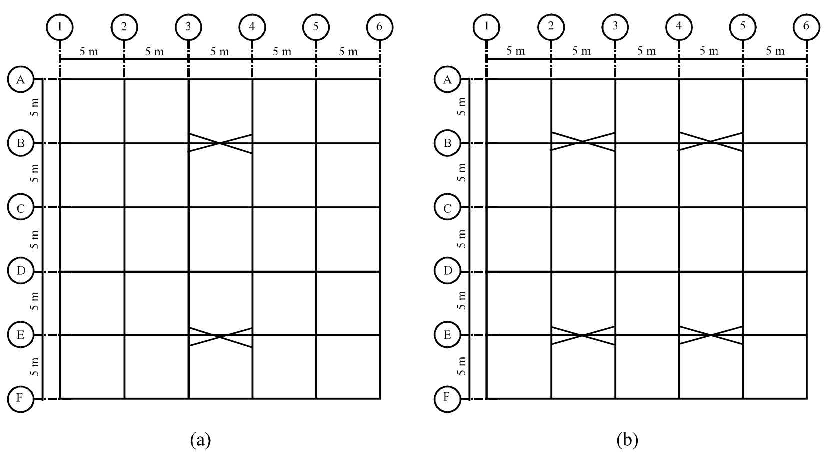

Fig. 9: Retrofitting by cross bracing (a) two days and (b) four days

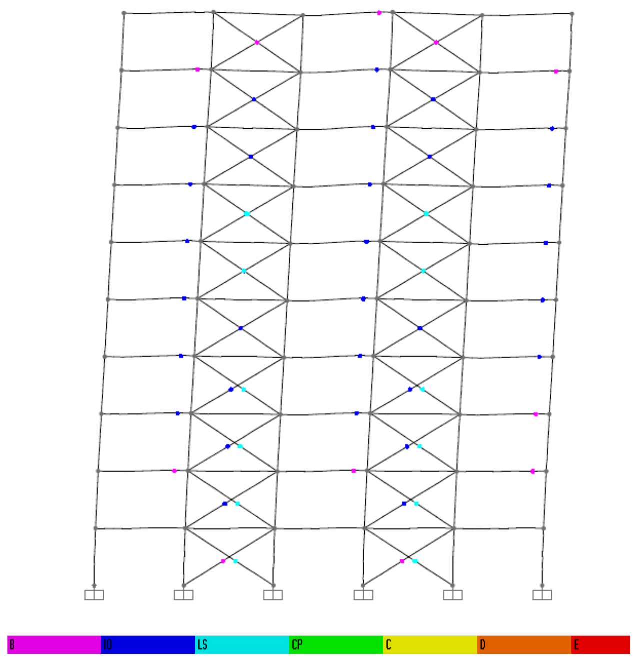

Fig. 10: Graphical display of the structure in the target displacement (reinforcement with cross bracing-the second design)

In second step, the structure is redesigned by retrofitting four bays within the whole structure, Fig. 9b (symmetrically). In evaluating the structure in the target displacement (24.6 cm), Fig. 10, only a few columns were not able to resist the generated forces, but all bracings provided acceptance limit of LS, so the second retrofitting will be acceptable.

Actually, the first retrofitting was not qualified to resist generated deformation and forces in the target displacement due to excessive drifts arose in the structure. As a matter of fact, in the first plan, the frame and bracings will undergo excessive lateral drift under the influence of the lateral load to reach to the operation point because of magic target displacement and it was influence of insufficient lateral stiffness (insufficient number of bracings) and this is undesirable. But in second step, because of sufficient number of bracings, the target displacement was satisfied, so retrofitting was acceptable.

Comparison of Absorbed Shear by the Steel Plate Shear Wall and Bracings

The absorbed shear percentage by the members in each story is actually part of the base shear which is absorbed by the bending frame or the wall or the braces. Figure 11a and b display the graphs related to the base shear absorption by the frame, wall and bracings.

As it is obvious, significant percentage of the base shear in the lower stories is supported by the wall and bracings, but with an increase in the height, their influence will be decreased and the absorbed shear by the frame will be increased which can be observed in the Fig. 11. Also, as it is observed in the graphs, the maximum absorbed shear by the wall is 63% while it is about 84% for the bracings and it shows higher stiffness of the bracing system than the steel plate shear wall. To put it another way, the absorbed shear by the frames in the model retrofitted by steel shear wall is more significant, so the use of the existing frame in steel shear wall system will be optimized.

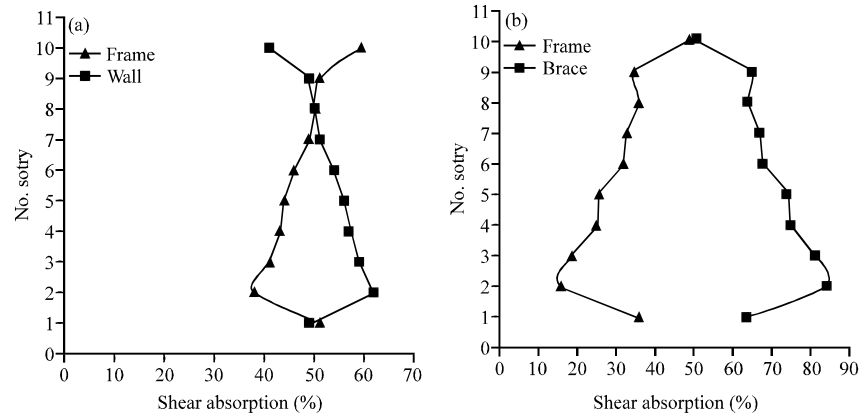

Fig. 11: The absorbed shear percentage by (a) the frame and steel shear wall, (b) The absorbed shear percentage by the frame and bracings

Table 1: Comparison of the consumed steel content between two reinforcement methods

Consumed steel volume (cm_3)	Column	Web or cross bracing	Beam	Total volume
Cross bracing	3.90×106	2.77×106	-	6.67×106
Steel shear wall	4.20×106	0.80×106	0.22×106	5.22×106
Steel shear wall with the minimum thickness of t = 3 mm	4.59×106	0.99×106	0.26×106	5.84×106

Table 2: Comparison of stiffness and period of the original structure before and after retrofitting

System	The original stiffness (kN cm^{-1})	Period
The original bending frame	22.7	2.39
Retrofitting by cross bracing	85.5	1.20
Retrofitting by steel shear wall	51.9	1.53
Retrofitting by steel shear wall with the minimum thickness of t = 3 mm	53.9	1.49

Comparison of the Consumed Steel with Two Methods

One of the effective factors in selecting the seismic retrofit plan is its economical issues. In Table 1, a comparison of the consumed steel volume between two retrofitting methods with steel plate shear wall and bracing is performed.

Considering the Table 1, the consumed steel volume for the bracings is about 2.8 times the web volume, but the consumed steel volume in shear wall columns is 1.2 times the bracing model. In fact, using larger sections is inevitable in the bracing model to prevent the buckling of the bracings, but in steel plate shear walls the whole plate capacity is used and plate buckling creates no problem in its bearing capacity and wall bearing will be in the same resistance after its buckling. However, generally, the consumed steel volume in retrofitting by steel shear wall is about 30% lower than bracing and if using the minimum 3 mm thickness, it will be about 15% lower. It should be noted that in the above evaluation, the consumed steel volume for the gusset plates in the bracing system and the gusset plate in the shear wall model as well as the required weld for connection has not been accounted for, while such issues need to be accounted for in an economical evaluation.

Comparison of Stiffness and Period

In Table 2, stiffness and period of the original model and the retrofitted models are given. As it can be observed, with retrofitting the original model by bracing, the stiffness is increased 3.7 times and in retrofitting with the shear wall (t = 3 mm) the stiffness is 2.4 times.

Minimum Disruption to the Building Service

In retrofitting by the steel shear wall, only one bay of B and E frames (totally two bays) was covered by steel walls, while for retrofitting by the bracing, two bays of B, E (totally four bays) were subject to improvement. It is obvious that in selecting the retrofitting plan, the minimum disruption to the function and occupants of an existing building is desirable.

CONCLUSIONS

Comparing the shear content absorbed by the shear wall and the bracings, it is observed that the absorbed shear share by the frame in the retrofitted model with the steel plate shear wall is more significant than the bracing model, so optimal use is made of the existing frame.

Also, comparing the consumed steel volume in two retrofitting plans, generally total consumed steel volume in the designed steel plate shear wall is about 30% lower than the bracing and in case using a minimum thickness of 3 mm, it would be 15% lower.

In retrofitting by the steel shear wall, the disruption to the function and occupants is lower and it is desirable.

This comparison and conclusion is for a 10 floor structure with average bending frame. To generalize this research, it's better to do the same on different floors and in the areas with different seismic vulnerabilities.

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