SEISMIC UPGRADATION OF NON-SEISMIC STEEL BUILDINGS USING STEEL PLATE SHEAR WALLS

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ABSTRACT:

In recent years, the awareness regarding seismic upgradation for steel buildings has increased significantly. Many old damaged and undamaged building structures do not meet the criteria of modern seismic design codes. Such structures need to be seismically enhanced. The main objective of this paper is to validate the effectiveness of steel plate shear walls (SPSW) as upgradation tools for non-seismic undamaged steel structures. Several recent research works have recommended SPSW as effective lateral load resisting components for building structures. The upgradation is carried out considering a performance based probabilistic approach, wherein Incremental Dynamic Analysis (IDA) is used. The upgradation involves a static energy based scheme for the design of SPSW, to sustain a certain amount of interstory drift demand. The proposed procedure is tested on a 7-story steel framed building structure. The results lead to substantial drift reduction overall, showing the effectiveness of the SPSW; however, in case of some earthquakes, the drifts are above the selected target drift. In such cases, the ineffectiveness of SPSW in meeting the probabilistic target is assigned to not accounting for several uncertainties properly in the upgradation framework, and to the formation of unwanted plastic hinges in the columns bounding the SPSW, leading to the loss in their shear resisting capacity. Proper design of boundary elements encompassing the SPSW can improve the behavior of the upgraded structures substantially. On the whole, the proposed procedure is found to be effective for the upgradation of non-seismically designed steel frame structures to satisfy an inelastic drift based probabilistic performance criterion.

KEYWORDS: seismic upgradation, steel plate shear walls, probabilistic performance evaluation, incremental dynamic analysis, non-seismic design

1. INTRODUCTION

In the past two decades, the awareness regarding seismic upgradation has increased significantly. Many old steel buildings were designed and constructed prior to the formulation of today's seismic design guidelines. These structures are usually very much prone to earthquake damages. Such structures suffer from the lack of sufficient strength, stiffness and ductility; required from the perspective of a current seismic design code. Hence, such damaged and undamaged structures should be seismically upgraded to make them safer against prospective earthquake hazards.

Steel plate shear walls (SPSW) have emerged as an innovative technique for lateral load resistance in buildings because of the various advantages [1] they have over other systems. Their design was implemented as early as 1970 as a primary load resisting system. Initially, only stiffened SPSW were used in order to resist the shear forces within their elastic buckling limits, as in the Sylmar Hospital in Los Angeles, the Nippon Steel Building in Tokyo, etc. [1,2]. With the analytical and experimental research carried out by Berman [3], Elgally [4], Cacesse [4] and others, it was observed that the post-buckling ductile behaviour of the unstiffened SPSW is much more effective than the elastic behaviour of the stiffened SPSW in resisting seismic forces. Also, the unstiffened plates exhibit substantial strength, stiffness, and ductility; and their hysteretic energy dissipation behaviour is stable and pronounced. These characteristics make unstiffened SPSW well suited for seismic design of new structure and for upgradation of old ones. A lot of research has gone into the study of the behaviour of unstiffened SPSW. However, a very little information regarding the use of unstiffened SPSW for seismic upgradation is available on published literature [5]. This paper focuses on the utilization of unstiffened SPSW for seismic upgradation of old steel framed buildings.

In the process of seismic upgradation, first the performance evaluation is carried out to judge either the capacity or the demand or both, of the structural system. Analysis procedures like linear static analysis, linear dynamic analysis, nonlinear static analysis and nonlinear dynamic analysis are recommended and used for performance evaluation in practice. The selection of a particular analysis procedure depends on the type of structure, seismic zone, its functionality and importance. With the advent of advanced computational technologies and the growth in computer processing power, increasingly accurate analysis techniques can now be adopted for performance evaluation. One such technique is the Incremental Dynamic Analysis (IDA), which has the essence of both nonlinear static analysis and nonlinear dynamic analysis.

IDA [6] is a parametric method wherein a structural model is subjected to one or more ground motion records, each scaled to multiple levels of intensity, thus producing one or more curves of response parameterized versus intensity level. An IDA curve is structure and ground motion record specific. The IDA being a performance evaluation technique, it is necessary to define the performance parameters which include the intensity measure (IM) and damage measure (DM). With the parameters being defined, IDA can be carried out to record the selected DM (e.g., interstory drift) of the structural model and to plot the resulting values versus corresponding IM (e.g., spectral acceleration) as continuous curves. The performance of the structure can be evaluated quantitatively from the IDA curve. This paper involves the performance evaluation of a 7-story steel moment frame considering 20 earthquake records together known as the LMSR series [7].

The main objective of this paper is to validate the effectiveness of SPSW as an upgradation tool for non-seismic undamaged steel framed building systems. The procedure selected for this validation involves the performance evaluation of the structural system using IDA. A suitable probabilistically defined seismic hazard level and target performance level are selected for the structure. The results obtained thereafter, from the performance evaluation are utilized for upgradation of the structure using unstiffened SPSW. A static energy based scheme is utilized for the design of SPSW. The modified structure is then re-evaluated using IDA to observe the upgraded performance of the structure. The proposed procedure does not involve complex probabilistic calculation. Therefore, although it fails to properly account for all the uncertainties involved, it should remain easily adoptable for the profession.

2. PROPOSED PROCEDURE FOR SEISMIC UPGRADATION

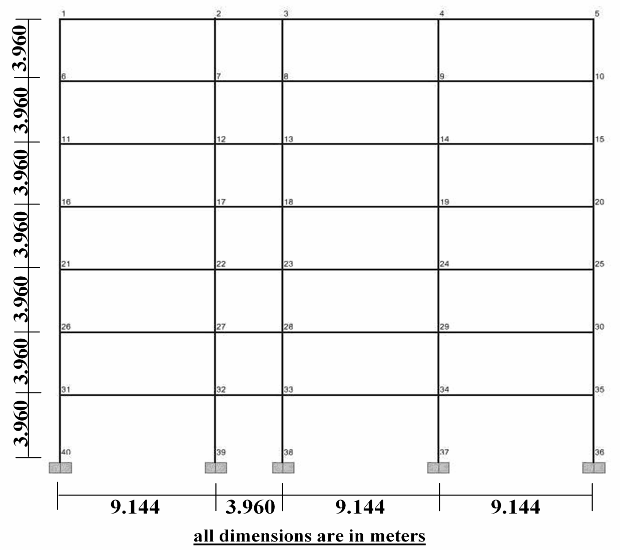
The general procedure from the initiation to the completion of seismic upgradation proposed in this paper involves the following steps:

- Setting the performance objectives for the structure which involves the selection of earthquake hazard level and target performance level.
- Carrying out the probabilistic performance evaluation of the existing structure using IDA, which involves multi-record IDA study using 20 large magnitude earthquake records.
- Designing the SPSW using a static energy based scheme, depending on the excess damage measure (DM) beyond the target performance level for the predefined hazard level.
- Upgrading the structure using the designed SPSW panels.
- Re-evaluation of the structure using probabilistic approach involving IDA to compare the damage measure, before and after seismic upgradation.

3. APPLICATION OF THE PROPOSED PROCEDURE – A CASE STUDY

An old undamaged 7-story moment frame with fixed support conditions is considered to simulate a non-seismically designed structure. The frame is assumed to be located in the state of California, USA at 33.93° N and 118.40° W. It consists of 4 bays and 7 stories as shown in Figure 1. It is assumed to be designed as per the

AISC-ASD design procedures [8] for standard gravity load combinations only. P-M interaction is considered for the columns and the column sections are checked against elastic buckling. All the structural modelling and analyses are carried out using the finite element system OpenSEES. [9]. A lumped mass model is considered with no flexibility of the joint panel zones. Force based nonlinear beam-column elements with five integration points are used for each beam or column element. A 5% Rayleigh damping is considered for the dynamic analyses and the P-Δ effects are neglected.



Outer columns: W24X250 (1-3rd story), W21X166 (4-7th story)
Inner columns: W30X326 (1-3rd story), W21X201 (4-7th story)
Beams (SPSW span, 2nd from left): W10X112
Beam (other spans): W12X210 (1-6th floor), W12X120 (7th floor)

Figure 1 Structural configuration of the 7-story steel moment frame

The structure is considered to be an “essential structure” subjected to moderate to severe earthquakes as per IBC 2006 [10]. The design earthquake hazard level is selected as having a 2% probability of exceedance in 50 years. Similarly, “life safety” is considered as the target performance level for the structure under consideration. With the performance objective thus defined, the multi-record IDA study can be carried out for this structure. The fundamental mode spectral acceleration (S_a) is considered as the intensity measure (IM) for the IDA. With reference to the United States Geological Survey (USGS) hazard maps [7], the design value of S_a is calculated using the procedure described in IBC. Also, to monitor the damage in the structural components, maximum interstory drift is selected as the damage measure (DM). The limiting interstory drift for the selected performance level, i.e., “life safety”, is set at 2.5%, similar to that of steel moment frames.

The performance evaluation of the structure is carried out by using a set of 20 strong motion records (LMSR series) [7]. For each of the earthquakes, IDA plot of 1st mode S_a versus maimum interstory drift for each story is obtained. The 95 percentile IDA curve is derived from the multi-record IDA for each story. Similarly, the 95 percentile curves of maximum story shear versus maximum interstory drift are also developed. These plots are shown in Figures 2 and 3, respectively. The objective is to have an upgraded structure having a low (5%) probability of failure, i.e., having a high (95%) probability of the interstory drift being within the target limit of 2.5%. The details regarding the drift demand and the story shear demand for the 7-story frame are presented in Table 3.1.

Table 3.1 95 percentile interstory drift and story shear demands for the original frame

Story	1st	2nd	3rd	4th	5th	6th	7th
Interstory drift (%)	2.90	3.20	3.50	3.60	3.84	4.08	3.36
Story shear (kN)	3778.50	3270.69	3619.72	2864.00	2953.05	2857.51	2149.61

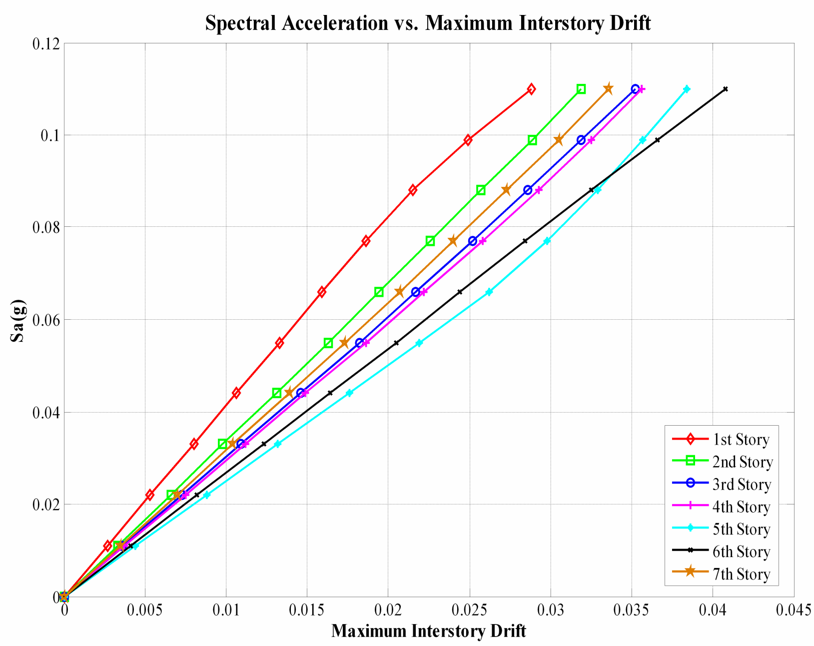


Figure 2 95 percentile S_a vs. maximum interstory drift IDA curves for the original frame

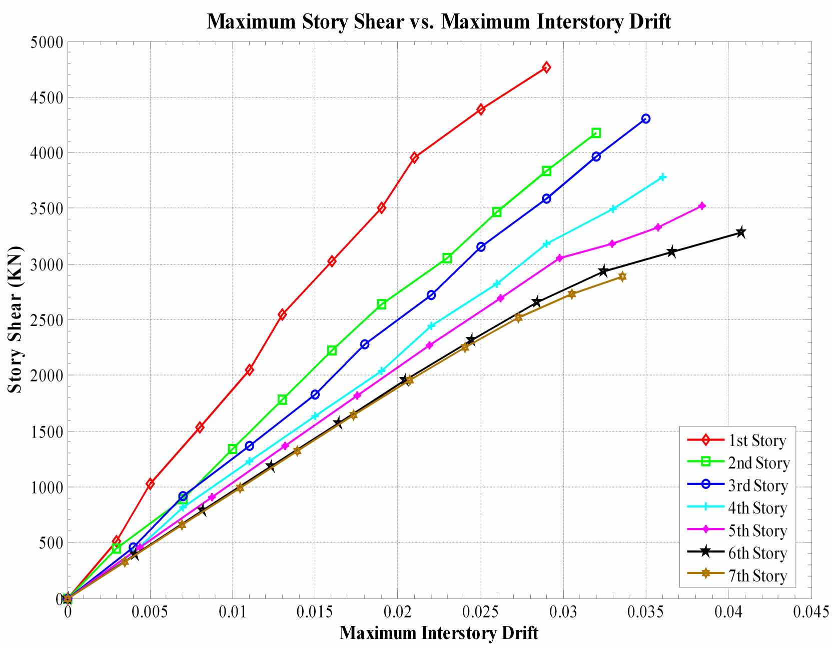


Figure 3 95 percentile maximum story shear vs. maximum drift demand curves for the original frame

The design of SPSW is based on reducing the drift demand of the system within the target drift limit. A displacement based approach is adopted; wherein the idea is to compare the static energy demands (at the peak monotonic displacement) of the original structure and the upgraded structure. The energy demand, for the same earthquake, changes from the original to the upgraded structure. A simple pseudo spectral velocity based energy formulation is considered following Akiyama's work [11]:

$$E = 0.5mS_v^{\ 2} \tag{3.1}$$

where, E = total (elastic plus plastic) strain energy demand, m = total seismic mass, and S_v = pseudo spectral velocity corresponding to the fundamental period (T_1). A factor F is used as the ratio of the energy demands imposed on the upgraded structure to that of the original structure:

$$F = \frac{E_{upgraded}}{E_{original}} = \left(\frac{S_{v-upgraded}}{S_{v-original}} \right)^2 \tag{3.2}$$

From the maximum story shear versus maximum interstory drift plots (Figure 3), the total strain energy demand imposed on the original structure is calculated as the area under this curve. Energy demand on the upgraded structure is obtained as in Eqn. 3.2. A median S_v spectrum, from all the 20 LMSR records, is used for this. The shear force demand on the SPSW is calculated by assuming an elastic-perfectly plastic monotonic behaviour for the structure. A number of iterations are carried out for the above procedure corresponding to different time periods, till the value of factor F converges. Figure 4 explains how the shear demand on the steel plate (V_{spsw}) is calculated.

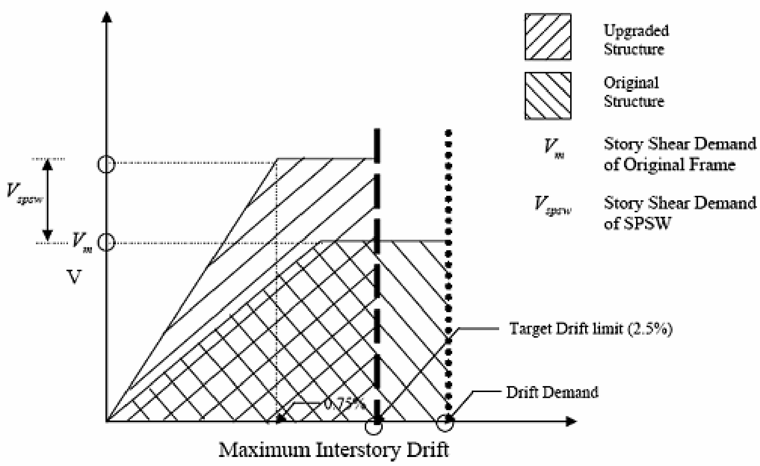


Figure 4 Static energy based scheme to calculate story shear demand of SPSW

With the shear force demand of the SPSW thus calculated, the required thickness of the steel panel in each story is calculated using elastic strain energy formulation [3]:

$$t = \frac{V_{spsw}}{0.5 F_y L \sin 2\alpha} \tag{3.3}$$

where, V_{spsw} = shear demand on the SPSW, F_y = yield stress of the plate material, L = bay width, α = angle of inclination of the principal stress in SPSW measured from the vertical. The details on this calculation are avoided here and are available in [7]. Unstiffened steel plates of the required thicknesses at each story as per Eqn. 3.3 are provided to obtain the upgraded structure.

The upgraded structure is re-evaluated for its performance under the selected hazard level. With the change in the fundamental time period of the structure, the modified design value of S_a is calculated using the procedure given in IBC 2006 [10]. The details are provided in Table 3.2. The upgraded structure is re-evaluated using the same set of 20 ground motion records. The nonlinear response history analyses are carried out using a multi-strip idealization for the unstiffened thin steel panels [3]. Details of the structure model are available in [7]. The 95 percentile IDA curves are obtained for each story of the upgraded 7-story structure (Figure 5). The interstory drift demands and the story shear demands imposed on the upgraded structure at the selected IM are presented in Table 3.3.

Table 3.2 T_1 and S_a values for the original and the upgraded steel frames

	Original	Upgraded
T_1 (s)	3.90	3.09
S_a (g)	0.11	0.14

Table 3.3 95 percentile interstory drift and story shear demands for the upgraded frame

Story	1st	2nd	3rd	4th	5th	6th	7th
Interstory drift (%)	2.80	2.90	2.60	2.63	2.60	2.80	2.40
Story shear (kN)	4402.73	3456.07	3248.84	3380.40	2949.74	2709.51	1805.92

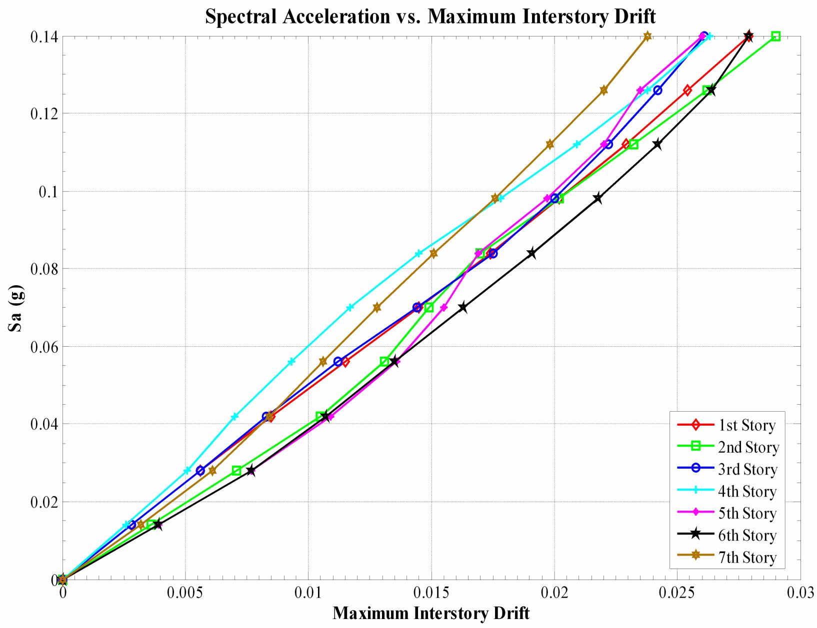


Figure 5 95 percentile S_a vs. maximum interstory drift IDA curves for the upgraded frame

4. OBSERVATIONS AND DISCUSSIONS

The primary aim of the upgradation is to satisfy a probabilistically defined performance objective for the structure. In other words, it is to limit the drift demands, with certain probability, within the target drift limit of 2.5% (i.e., performance level of "life safety") for the selected probabilistic hazard level. It is observed from the case study presented in this paper that substantial amount of reduction in the drift demand takes place in each story of the 7-story structure. The 95 percentile drift demands obtained for the upgraded structure (Table 3.3) are very close to the target drift limit and more uniform over the stories (compared to the original structure), although with some discrepancies. Figure 6 indicates that the probability of limiting the drift within the level of "life safety" is sufficiently high for all the stories. It is higher than 80% in all of the cases and even as high as 95% in some of the cases. This behaviour indicates that substantial lateral resistance is present in the upgraded structure, which in turn represents the effectiveness of SPSW in restraining the drift demands within permissible limits.

The primary reason for not being able to achieve the target is not accounting for the uncertainties properly in the procedure. Although a probabilistic target is set for the structure, detailed probabilistic calculations are avoided in the proposed procedure in order to make it easily adoptable in practice. The primary sources of uncertainty, other than the variability in earthquakes, are in i) using a static energy based scheme instead of a dynamic one, and ii) assuming the dominance of the fundamental mode in estimating the energy demand (Eqn. 3.1). The thickness calculation for the steel panel also incorporates the assumption that the columns do not have any plastic rotation other than at the base of the structure. In order to check this, numerical tests are conducted to detect the probable locations of plastic hinges within the upgraded structure. Also, story shear demands are calculated to compare them with the story shear capacities provided. Record 16 and Record 20 are selected for these tests, as the upgraded structure responded with considerably high drifts in these two cases. The observations from these tests are described in Figure 7 and Table 4.1.

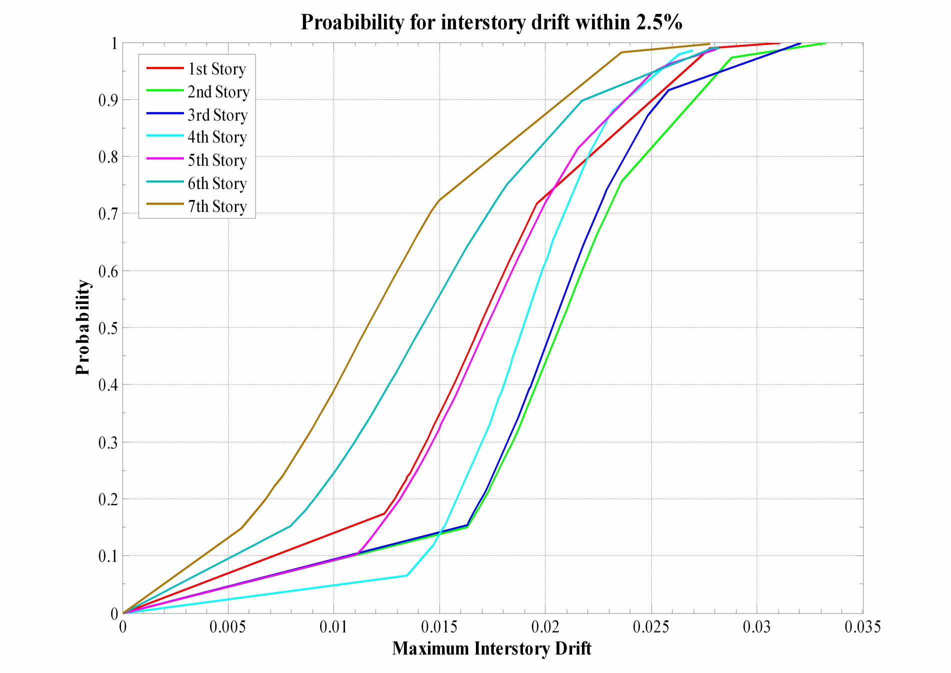


Figure 6 Cumulative probability distribution of maximum interstory drift demand on the upgraded structure

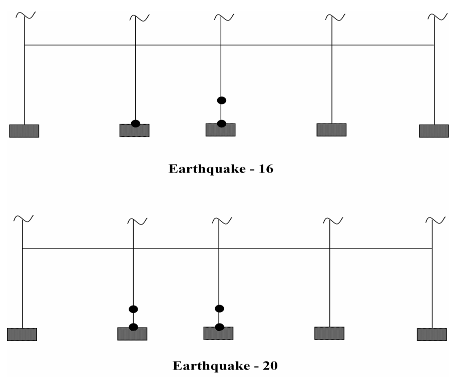


Figure 7 Plastic hinge locations in the upgraded frame

Table 4.1 Design and 95 percentile of actual story shear achieved for the upgraded frame

Story	1st	2nd	3rd	4th	5th	6th	7th
Design story shear (kN)	4737.55	4277.08	4828.18	4284.57	4599.81	4710.39	3248.47
Actual story shear (kN)	4402.72	3456.07	3248.84	3380.50	2949.74	2709.52	1805.92
% achieved	92.93	80.80	67.29	78.90	64.13	57.52	55.60

The analyses indicate that apart from the supports (where they are intended), the plastic hinges also form within the lowermost columns surrounding the SPSW. This behaviour is contradictory to the controlled mechanism assumed for the upgraded steel moment frame [7]. The 95 percentile of actual story shear values achieved are also lower than the capacities the plates are designed for (Table 4.1). This happens because the columns bounding the SPSW are not strong enough to transfer the forces from the SPSW to the support before reaching their own plastic capacities. This in turn, reduces the shear resisting capacity of the lower columns and hence, the overall behaviour of the structure is affected. It is an important problem in hand to ascertain the optimum sizes of the boundary elements encompassing the SPSW. In other words, on what criteria should the design of the boundary elements be based, which enables the SPSW to act effectively leading to satisfactory drift reduction in the structures under consideration.

5. CONCLUDING REMARKS

This paper presents an easily adoptable method of upgrading non-seismically design steel moment frame structures with steel plate shear walls to satisfy a probabilistically defined performance objective as recommended in today's advanced seismic design guidelines. This procedure can take into account a multi-earthquake based probabilistic definition of hazard, as well as an inelastic displacement based performance level, without incorporating complex probabilistic calculations. The displacement based approach for the design of SPSW is found to be much more realistic and suitable giving substantially good results. A case study on a 7-story structure shows that the procedure provides good results, although not exactly to the level required. The effectiveness of SPSW in providing lateral resistance is found to be influenced by the strength and stiffness of the boundary elements. The procedure is expected to improve if the various uncertainties can be properly accounted for in the method.

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