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SEISMIC UPGRADING OF NON-DUCTILE STEEL FRAMES USING STEEL PLATE SHEAR WALLS

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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 propose and validate the effectiveness of an upgrade scheme for non-seismic buildings using steel plate shear walls (SPSW). The upgrade is carried out considering a probability-based drift-based criterion, wherein incremental dynamic analysis is used for performance evaluation. The proposed upgrade procedure involves a static energy based scheme for the design of SPSW. It is tested on a 7- and a 5-story steel framed building structure. The results show substantial drift reduction overall, showing the effectiveness of the SPSW; however, selected target performance is not achieved exactly. Various reasons for the inability of the SPSW in meeting the probabilistic target are indicated. Overall, the proposed procedure is found to be effective for the upgrade of non-seismic steel frame structures to satisfy an inelastic drift-based probabilistic performance criterion. Need for future research works are indicated based on the shortcomings of the proposed procedure.

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

1. Introduction

In the past two decades, the awareness regarding seismic upgrade has increased significantly. Many old buildings were designed and constructed prior to the formulation of today's seismic design guidelines. These structures are usually very prone to earthquake damages, since they

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suffer from a lack of sufficient strength, stiffness and ductility that are required from the perspective of a current seismic design code. These damaged and undamaged old structures need seismic upgrade or enhancement in order to make them safer against prospective earthquake hazards. Recently, *steel plate shear wall* (SPSW) has emerged as an innovative technique for lateral load resistance in buildings because of the various advantages it has to offer over other lateral load resisting systems, such as moment resisting frames, braced frames, reinforced concrete shear walls, etc. A comprehensive review of the pros and cons of SPSW can be found in (Astaneh-Asl 2001) and various other works. SPSW were implemented in seismic structural design as the primary load resisting system, as early as the early 1970s. Initially, only stiffened SPSW with closely spaced horizontal and vertical stiffeners were used in order to resist seismic shear forces within their elastic buckling limits, as in the cases of the Sylmar Hospital in Los Angeles, USA, and the Nippon Steel Building in Tokyo, Japan. With the analytical and experimental research carried out by Thorburn et al. (1983), Timler and Kulak (1983), Caccese et al. (1993), Elgaaly et al. (1993), Driver et al. (1997), Berman and Bruneau (2003), and many 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. 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 the seismic design of new structures and for the upgrade of old ones. A lot of research has gone into the analysis and design of unstiffened SPSW, however, very little information is available in published literature regarding the use of unstiffened SPSW specifically for the purpose of seismic upgrade (Bruneau and Bhagwagar 2002). The present work focuses on the utilization of unstiffened SPSW for seismic upgrade of old non-seismic steel framed buildings.

The other major emphasis of this paper is on the adoption of a probabilistic framework for defining the target performance criterion in the proposed upgrade procedure. The process of seismic upgrade includes the performance evaluation of the existing structure and the reevaluation of performance after it is upgraded. These performance evaluations can be achieved in both a deterministic and a probabilistic framework. Analysis procedures with various levels of accuracy/idealization, such as linear static analysis, linear dynamic analysis, nonlinear static analysis and nonlinear dynamic analysis, are recommended and used for performance evaluation in earthquake engineering practice. The selection of a particular analysis procedure depends on the type of structure, seismic zone, its functionality and importance, and available tools of analysis. With the advent of advanced computational technologies and the growth in computer processing power, computation intensive 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 the nonlinear static analysis and the nonlinear dynamic analysis, and can easily fit into a probabilistic framework (Vamvatsikos and Cornell 2002).

The primary objective of this paper is to explore a methodology for upgrading non-seismic undamaged steel frames using SPSW, within a probabilistic evaluation framework. A suitable probabilistically defined seismic hazard level and target performance level are selected for the structure. A semi-probabilistic upgrade procedure is proposed here for achieving the target performance of the upgraded system. This procedure involves the performance evaluation (and reevaluation) of the structural system using IDA. The proposed method is checked against a 7- and a 5-story steel frame building.

2. Proposed Seismic Upgrade Procedure

The general procedure from the initiation to the completion of the proposed seismic upgrade procedure is provided in this section. The target performance criterion or the performance objective is defined following the general performance-based seismic design (PBSD) guidelines described in documents, such as the Vision 2000 document (SEAOC Vision 2000 Committee 1995) or FEMA-350 (FEMA 2000). These documents defined performance objective as a combination of the selected seismic *hazard level* and the intended structure *performance level*, both defined in a probabilistic sense. The design earthquake hazard level is selected as having a 2% in 50 years probability of exceedance. Similarly, “life safety” is considered as the target performance level for the structure under consideration, and inter-story drift demand is selected as the damage measure for the structure. The objective is to have an upgraded structure having a low (5%) probability of failure, i.e., having a high (95%) probability of the inter-story drift being within the target limit of “life safety”. It should, however, be noted here that the proposed method is not limited to the performance objective selected above. Any other suitable hazard level and performance level can be chosen similarly in this generic procedure.

With the performance objective of the structure thus defined, a multi-record IDA study is first performed for the performance evaluation of the old non-seismic 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, the design value of S_a is calculated using the procedure described in IBC 2006 (ICC 2006). Also, to monitor the damage in the structural components, maximum inter-story drift is selected as the *damage measure* (DM) for this IDA. The probabilistic performance evaluation of the structure is carried out by using a set of 20 strong motion records known as the “LMSR series”. The LMSR series of records represents a set of 20 “large magnitude small distance” earthquakes, which has been used in various studies in the recent past (for example, (Vamvatsikos and Cornell 2004)). These records have magnitudes in the range of 6.5-6.9 and epicentral distances in the range of 16-32 km. The selection of a particular set of critical earthquakes for carrying out probabilistic performance evaluation of structural systems depends on many different factors like location, site conditions, proximity from the nearby faults, etc. These ground motion records are considered to provide an adequate level of record-to-record randomness for seismic performance evaluation of structures on firm soil and susceptible to strong near-source earthquakes (Vamvatsikos and Cornell 2004, Dhakal et al. 2006). For each of the earthquakes, IDA plot of the 1st mode S_a versus maximum inter-story 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 inter-story drift are also developed, which are used later in the upgrade scheme. It should be noted here that a set of 20 records is not sufficient for using the 95 percentile information. However, since the focus is primarily on the method and not on the actual results, these 20 records are assumed to be adequate (Dhakal et al. 2006).

The design of SPSW panels is based on reducing the drift demand (95 percentile demand, based on the multi-record IDA) of the system within the target drift limit for each story. 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 (1985):

$$E = 0.5 m S_v^{\,2} \tag{1}$$

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:

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

From the maximum story shear versus maximum inter-story drift “IDA” plot, the total strain energy demand imposed on each story of the original structure is calculated as the area under this curve. Energy demand on each story of the upgraded structure is obtained as in Equation (2). A median S_v spectrum, from all the 20 LMSR records, is used for this. The shear force demand on a steel panel is calculated by assuming an elastic-perfectly plastic monotonic shear force-deformation behaviour for the story. Since the fundamental period for the upgraded structure is not known beforehand, a number of iterations are carried out for the above procedure corresponding to different T_1, till the value of factor F converges. Figure 1 schematically explains how the shear demand on the steel plate (V_{spsw}) is calculated for each story, based on the assumed energy formulation. The energy demands for the original and the upgraded structures, shown as shaded areas in Figure 1, are compared (with the factor *F*). It should be noted here that the bilinear elastic-perfectly plastic curves in Figure 1 are based on an idealization of the actual (95 percentile) maximum story shear vs. maximum inter-story drift plot based on the 20-record IDA. With the shear force demand of the SPSW for a selected story thus calculated, the required thickness (t) of the steel panel in each story is calculated using elastic strain energy formulation (Berman and Bruneau 2003):

$$t = \frac{V_{spsw}}{0.5 F_y L \sin 2\alpha} \tag{3}$$

where, F_y = yield stress of the plate material, L = bay width for the plate panel, α = angle of inclination of the principal stress in SPSW measured from the vertical. Unstiffened steel plates of the required thicknesses at each story as per Equation (3) are provided to obtain the upgraded structure.

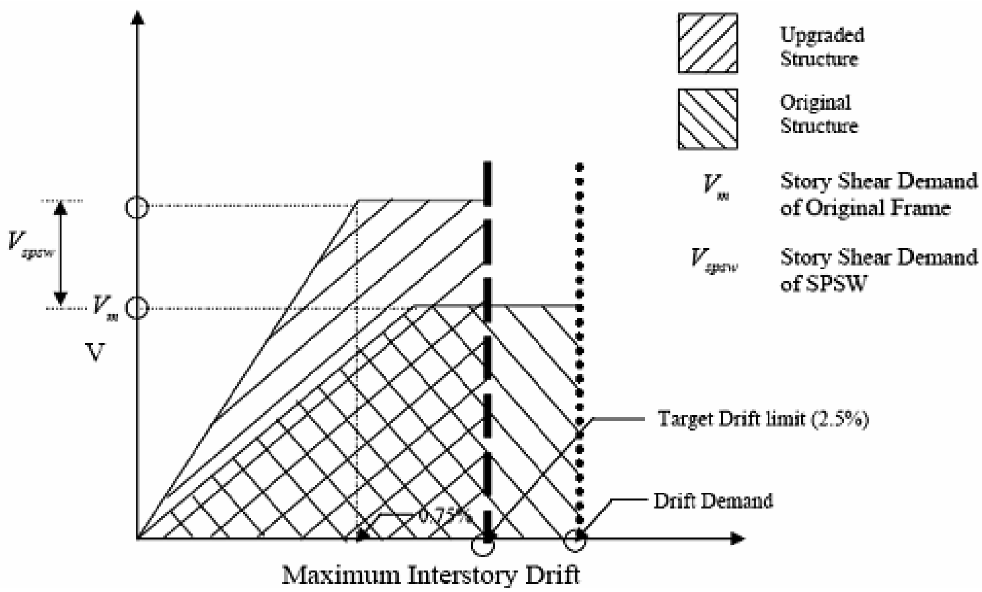


Figure 1. Schematic representation of the static energy based scheme used for upgrading

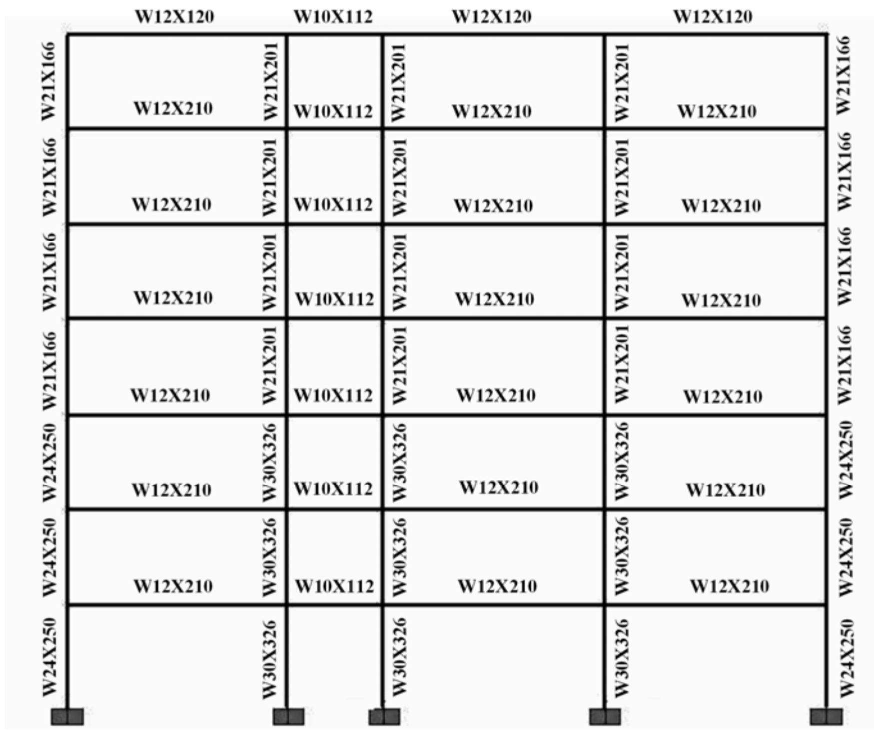
The upgraded structure is reevaluated for its performance under the selected hazard level. With the change in the fundamental time period (T_1) of the structure, the modified design value of S_a is recalculated using the procedure given in IBC 2006 (ICC 2006). The reevaluation is performed using the probabilistic approach of multi-record IDA for the same set of 20 ground motion records, and the 95 percentile inter-story drift demands are obtained for each story of the upgraded structure at the selected IM level. These inter-story drift demands are checked against the target drift values as per the selected performance level. Various aspects of this proposed upgrade procedure are reviewed again in Section 5, where the advantages and the shortcomings of the proposed procedure are discussed.

3. Application of the Proposed Upgrade Scheme

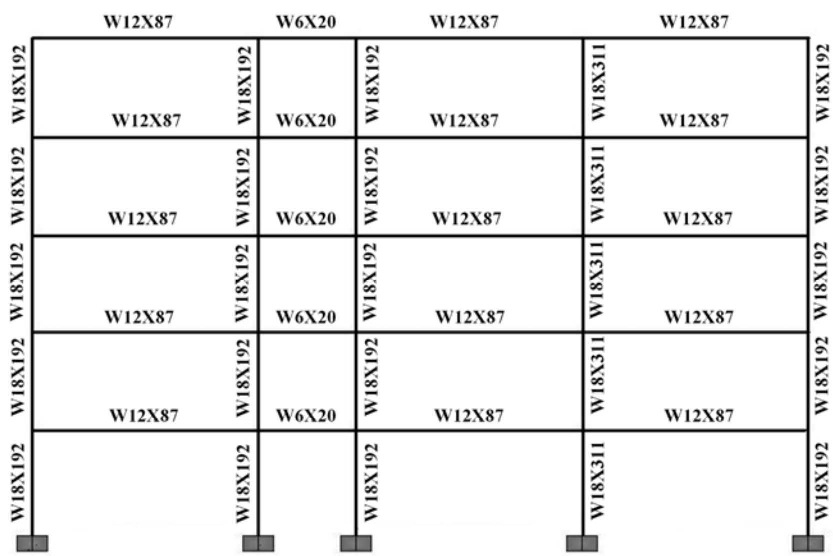
The semi-probabilistic approach of upgrade described in the previous section is tested on two steel frame buildings. This section provides the building details, analysis details, important considerations, upgrade details, significant results and observations. Two old non-seismically designed steel frames of 7- and 5-story configurations are considered as test cases. They consist of 4 bays and 7 or 5 stories, without any basement as shown in Figure 2. Both the frames have a story height of 3.960 m and bay length of 9.144 m, except for the bay second from left which is 3.960 m long. This bay is considered for putting the SPSW later for upgrading. These frames are assumed to be designed as per linear elastic allowable stress design (ASD) procedures (AISC 2005) and only for standard gravity load combinations, so that they can somewhat represent old non-seismic building designs. P-M interaction is considered in the column design and the column sections are checked against elastic buckling. Table 1 provides gravity loading details for the 7- and 5-story frames. The designed sections are shown in Figure 2. Further information on the design procedure is available in a detailed report (Bhatia 2008), which also includes all other detailed information on these two case studies. Although, it is not necessary to use moment frames for non-lateral force designs, such frames are considered here just to illustrate the procedure, since the generic upgrade scheme applies to frames with pin-connected as well as rigidly connected beams. The structures are considered to be "essential structures" subjected to moderate to severe earthquakes as per IBC 2006 (ICC 2006). For computing the ground hazard, the selected frames are hypothetically placed near Los Angeles, USA (at 33.93° N and 118.40° W), on firm soil. The design earthquake hazard level is selected as having a 2% probability of exceedance in 50 years. For this hazard level, the 1st mode spectral acceleration (S_a) is found to be 0.11g and 0.12g, respectively for the 7- and the 5-story frames (T_1 = 3.90 sec for 7-story and 3.69 sec for 5-story frame; These rather long fundamental periods of the two structures also come from the fact that the lateral stiffness of the pin-connected gravity frames are not included in the analysis). Similarly, "life safety" is considered as the target performance level for the structures under consideration. There is no specific recommendation available in standard guidelines on performance levels for steel plate shear wall systems. The performance level for steel moment frame systems as defined in FEMA-351 (FEMA 2000) is adopted for the SPSW systems considered for this case study in order to illustrate the proposed upgrade procedure. Accordingly, a limit of 2.5% inter-story drift is set as the target. However, any other limit could easily be used following the proposed procedure.

Table1. Gravity loading details for the original 7-story and 5-story frames

	External columns (kN)	Internal columns (kN)	Girders (kN/m)
Floors	105.0	154.8	14.60
Roof	90.30	137.5	12.40



(a)



(b)

Figure 2. Elevations of the original: a) 7-story; and b) 5-story frames

All the structural modeling and analyses are carried out using the finite element system OpenSEES (Mazzoni et al. 2007). A lumped mass model is considered with no flexibility of the joint panel zones. A single force-based nonlinear beam-column element with five integration points is used for modeling each beam or column element. A 5% Rayleigh damping is considered for the dynamic analyses. P-Δ effects and the stiffness contribution from the gravity frames in the building are neglected. Based on the nonlinear response history analyses under scaled ground motions, the multi-IDA curves are obtained for each story of a frame. These IDA plots have 1st mode S_a as the IM and maximum inter-story drift as the DM, as shown in Figure 3 for the 1st story of the 5-story frame. For each earthquake of the LMSR series, the IDA is carried out till selected level of hazard is reached (for example, up to $S_a = 0.12g$ for the 5-story frame). The 95 percentile IDA curve is derived from the 20-record IDA for each story. For example, Figure 4 shows the 95 percentile IDA plots for all the stories of the original 5-story frame. From the results of the same nonlinear response history analyses, similarly, 95 percentile maximum story shear versus maximum inter-story drift "IDA" plots are obtained for each story. Figure 5 shows the 95 percentile story shear versus maximum inter-story drift curves for the 5-story frame. Table 2 provides the 95 percentile values of inter-story drift demands and story shear demands for each story of the 7- and 5-story frames.

Table 2. 95 percentile values of inter-story drift demands and story shear demands for the original frames

Story	Peak inter-story drift (%)		Peak story shear (kN)	
	7-story	5-story	7-story	5-story
1	2.90	3.50	3778	3086
2	3.20	3.70	3271	2177
3	3.50	4.40	3620	1994
4	3.60	5.40	2864	2124
5	3.84	5.80	2953	2141
6	4.08		2858	
7	3.36		2150	

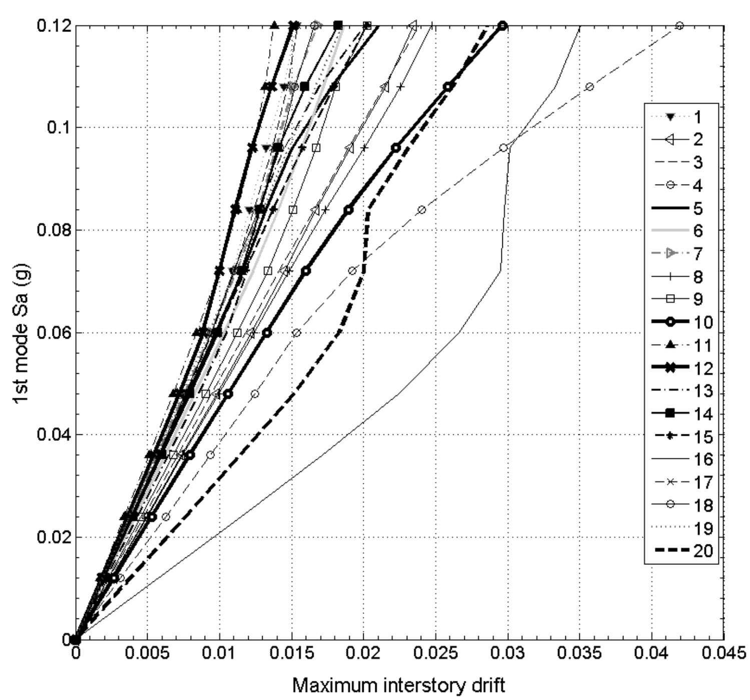


Figure 3. Multi-record IDA plot for the LMSR records for the 1st story of the original 5-story frame

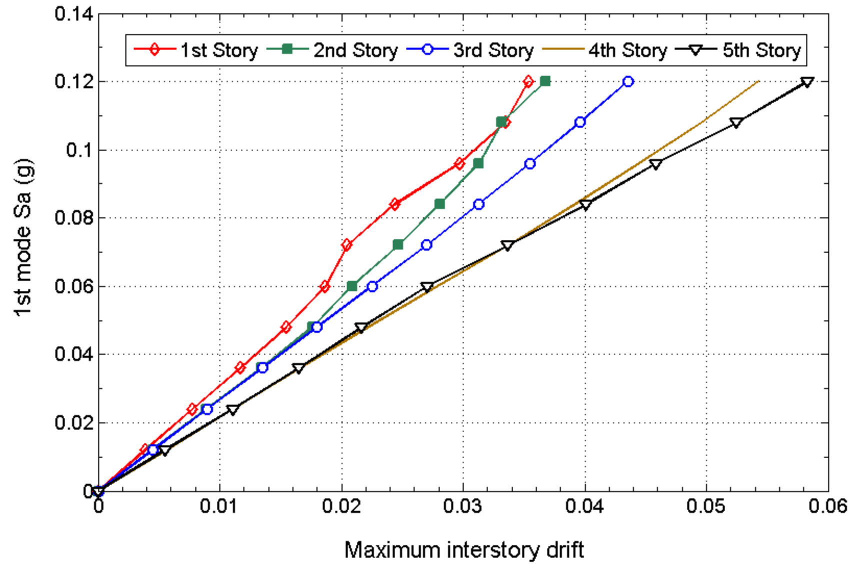


Figure 4. 95 percentile IDA curves for all the stories of the original 5-story frame

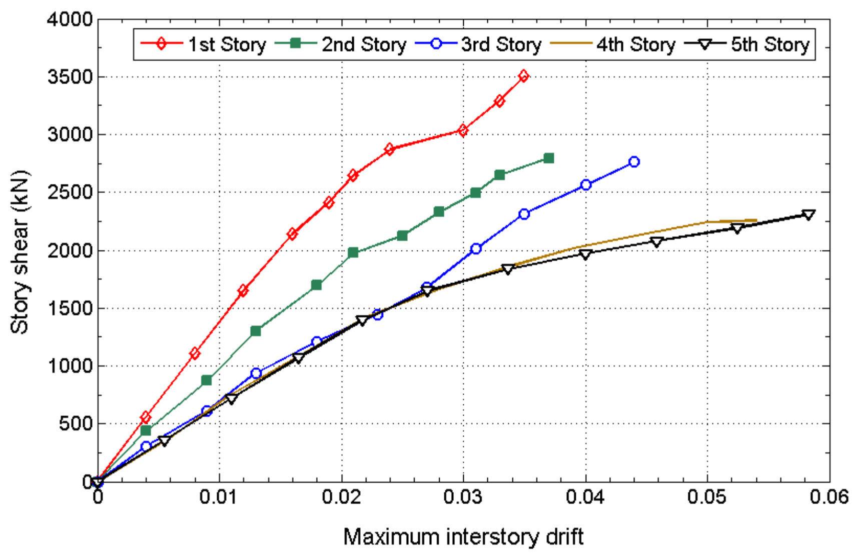


Figure 5. 95 percentile story shear versus maximum inter-story drift for the original 5-story frame

Table 2 shows that the inter-story drift demands on the original frames are beyond the target limit of 2.5% for all the stories. Hence, SPSW are needed at all levels for upgrading both the frames. For each story, the story shear versus inter-story drift plots are bilinearized (with drift demands as per Table 2) as shown in Figure 1, and the area under the curve is equated (incorporating the factor F) with the corresponding area of the proposed upgraded system for limiting drift of 2.5%. This gives the shear capacity required for each steel plate shear wall (V_{spsw}). The required thickness (t) of the steel plate is calculated using Equation (3). The angle of inclination of the principal tensile direction (α) is obtained as:

$$\tan^4\alpha = \frac{1 + tL\left(\dfrac{1}{2A_c} + \dfrac{L^3}{120 I_b h_s}\right)}{1 + th_s\left(\dfrac{1}{2A_b} + \dfrac{h_s^{\ 3}}{360 I_c L}\right)} \tag{4}$$

where, A_c = area of the boundary column section, I_c = moment of inertia of the boundary column section, I_b = moment of inertia of the boundary beam section, h_s = story height (Bruneau and Bhagwagar 2002). Due to the interrelated nature of Equations (3) and (4), the required thickness of a steel panel is obtained through iterations. Values of the required shear capacity of the steel plate, its thickness and the inclination angle are given in Tables 3 and 4 for the two frames. It should be noted that actual thickness values of SPSW as per Equations (3) and (4) are hypothetically used in these designs, irrespective of the real availability of these exact thicknesses in the market. The steel plate panel is provided in the bay second from left for all the stories.

Table 3. Details of the steel plates for the upgraded 7-story frame

Story	V_{spsw} (kN)	t (mm)	α (rad)
1	959.1	1.47	0.929
2	1006	1.55	0.933
3	1208	1.85	0.946
4	1420	2.21	0.951
5	1647	2.57	0.961
6	1853	2.90	0.968
7	1099	1.68	0.934

Table 4. Details of the steel plates for the upgraded 5-story frame

Story	V_{spsw} (kN)	t (mm)	α (rad)
1	637.1	1.17	1.11
2	799.0	1.47	1.12
3	1130	2.13	1.13
4	1867	3.58	1.14
5	2219	4.29	1.14

The performance re-evaluation of the upgraded frames is conducted again through multi-IDA with the same records. The steel plate is modeled using the *multi-strip* idealization (Thorburn et al. 1983), where each plate is modeled using 10 nonlinear corrotational truss elements, inclined at an angle α with the vertical and connecting the boundary beam and column elements. Each boundary beam and column is modeled with multiple force-based nonlinear beam-column elements with only two integration points, where each of the element spans between two nodes connecting to the truss elements. Other beams and columns (not surrounding the SPSW) are modeled with single elements with five integration points as for the original structure. The fundamental time period (T_1) of the upgraded structure is obtained from an eigenvalue analysis (T_1 = 3.09 sec for 7-story and 3.00 sec for 5-story frame). The new design S_a values for the frames corresponding to the selected hazard level of 2% in 50 years exceedance probability are obtained using IBC 2006 guidelines. For both the upgraded frames the new S_a value is 0.14g. The multi-record IDA are carried out for each frame upto this S_a value and the 95 percentile curves are obtained for each story (Figures 6 and 7). The 95 percentile drift demands for each story of the upgraded 7- and 5-story frames are provided in Table 5.

Table 5. 95 percentile values of inter-story drift demands for the upgraded frames

Story	Peak inter-story drift (%)	
	7-story	5-story
1	2.80	3.12
2	2.90	3.08
3	2.60	3.04
4	2.63	3.20
5	2.60	3.36
6	2.80	
7	2.40	

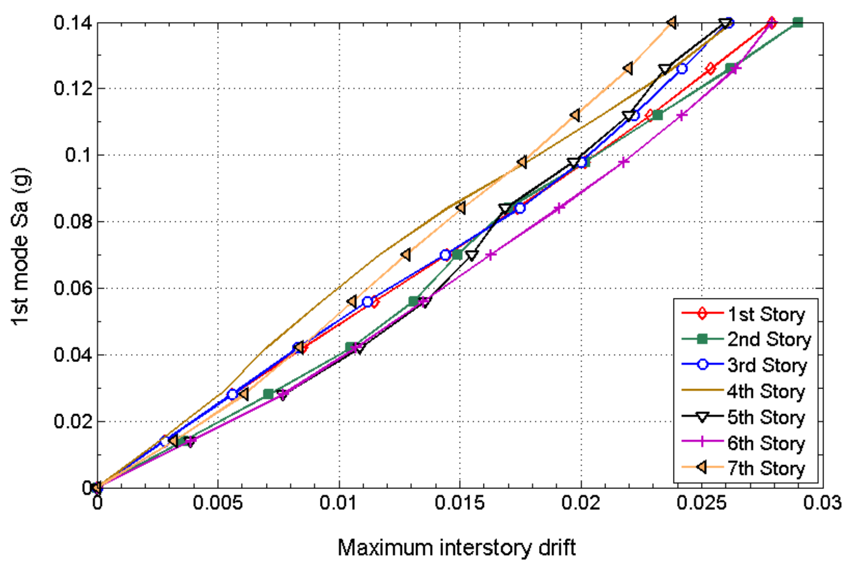


Figure 6. 95 percentile IDA curves for all the stories of the upgraded 7-story frame

4. Discussion on the Results

The results (Table 5) show that the target drift limit of 2.5% is not achieved at all the stories of the frames considered for the application of the proposed procedure. However, the SPSW are found to able to reduce the drift demands significantly from the original demands (Table 2). For the 7-story frame the average drift demand (over all the stories) reduces from 3.50% for the original structure to 2.68% for the upgraded structure. For the 5-story frame, the average drift demand reduces from 4.56% to 3.16%. In addition, the demands are almost uniform over the height of the frame for the upgraded systems. For the 5-story frame, the standard deviation in drift values over the height reduces from a huge 1.02% to only 0.126%. For the 7-story frame, this parameter reduces from 0.392% to 0.169%. The primary reasons for not achieving the intended levels of drift are postulated as:

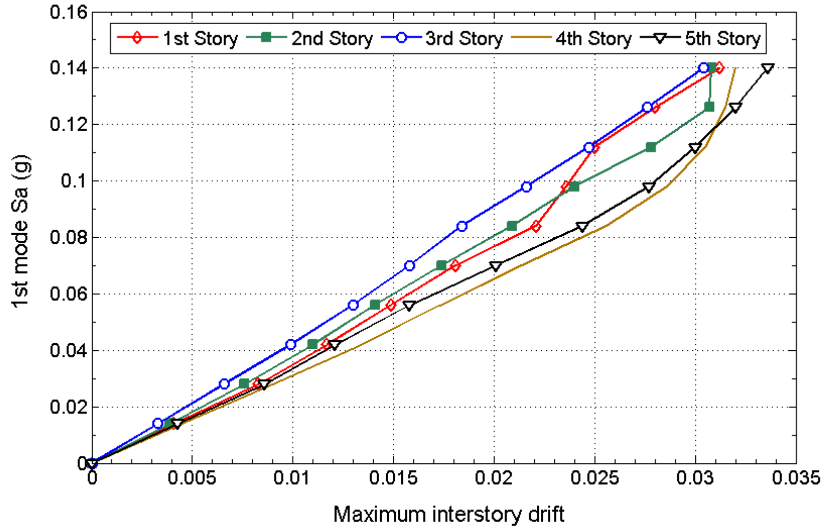


Figure 7. 95 percentile IDA curves for all the stories of the upgraded 5-story frame

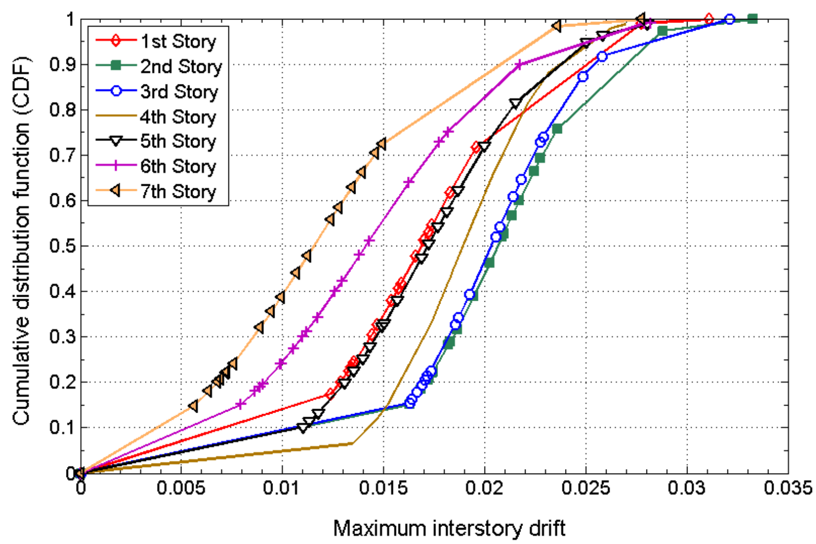


Figure 8. Cumulative distribution function of maximum inter-story drift at the selected hazard level for the upgraded 7-story frame

1. Twenty records are too low for obtaining the 95 percentile information, and a single earthquake record plays the dominant role in the 95 percentile behavior. The results could have been better for a larger set of records.

2. As opposed to the common design practice, no margin or factor of safety is applied in the upgrade scheme adopted herein.

In order to observe the results from a different perspective, the cumulative probability distribution function (CDF) is plotted for maximum drift values obtained for the 20 records at the design S_a level. For example, Figure 8 shows the CDF for maximum drift values for the 7-story frame. This plot shows that the probability of the drift value being less than or equal to 2.5% varies in the range of 82-98%, which is quite high although not upto the desired level for all cases.

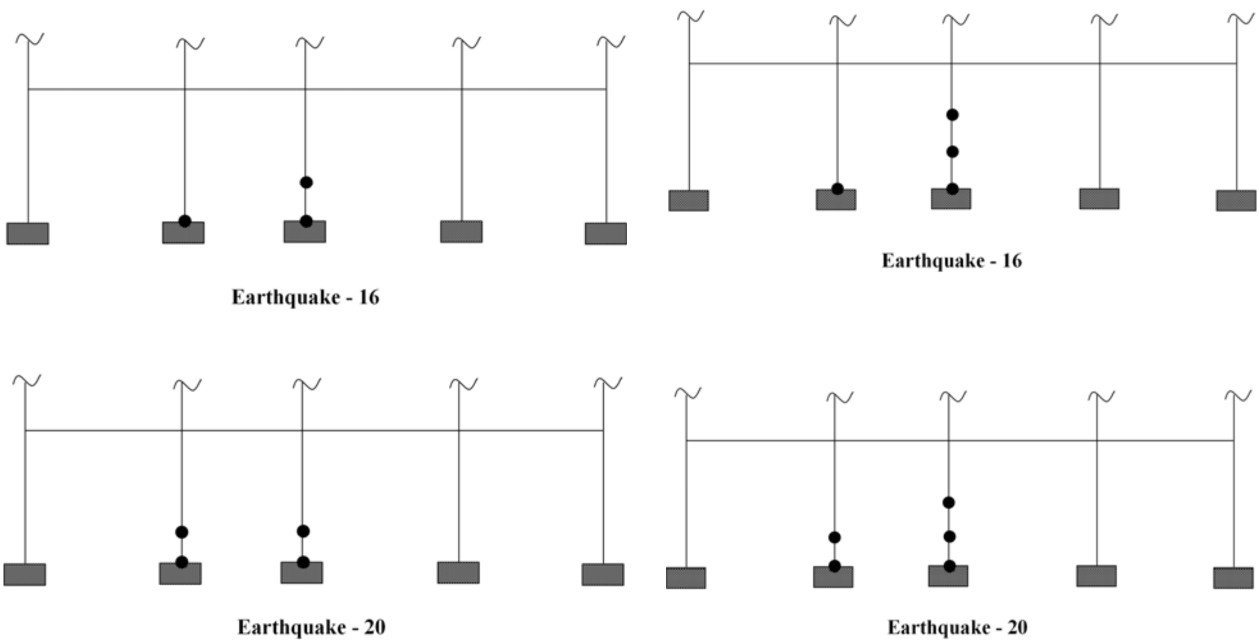
In addition to the achieved drift levels, the achieved shear forces and the formation of plastic hinges are also checked. The design story shear values (total shear for the plate and boundary elements, not only V_{spsw}) and the maximum story shear (95 percentile value from the 20 records) carried by the upgraded frames are provided in Table 6, which shows that the achieved shear carrying capacities of the SPSW system do not reach the design story shear levels. This may be detrimental to the drift reduction capacity of the SPSW as per Figure 1, since the area under the bilinear curve remains unchanged pushing the drift limit to a higher value. It is observed that the maximum story shear carried by a plate as percentage of what it is designed for is, in general, more for the 7-story frame, than the 5-story frame. This may be a reason why the 7-story results are better (closer to the target) than the 5-story results. However, one should note that the drifts result more from an overall frame behaviour than a story-wise behaviour, as evident from the fact that a lower percentage of shear force carried (for example, in the 7th story of the 7-story frame) does not correspond to a higher drift value. The locations of column plastic hinges are checked from nonlinear response-history analyses. For this, two records (record numbers 16 and 20 of the LMSR set) are selected for each frame for which the drifts are considerably high. Figure 9 shows the plastic hinge locations in the boundary columns of the SPSW. Based on the assumed uniform drift scenario, column plastic hinges should form at the base of the 1st story columns only. However, there are more plastic hinges in those columns and this may be a reason for the plates not being able to carry the full design shear. It should be noted here that there are more plastic hinges in the 5-story frame (which performs worse) than in the 7-story frame (which performs better). The formation of the plastic hinges indicates that the boundary columns are reaching their capacity before the SPSW utilize their full potential. A minimum moment of inertia of the boundary columns are provided in the upgrade scheme in order to prevent the columns from buckling before the plasticization of the steel plate, following Berman and Bruneau (2003):

$$I_c \geq \frac{0.00307 t h_s^{\;4}}{L} \tag{5}$$

However, this criterion is not intended for a full design of the column section, and the results show that it seems to be inadequate to realize the plastic shear capacity of the SPSW. More stringent requirements, for example, the criterion of a minimum plastic moment capacity of columns as proposed by Driver et al. (1997) may be needed to avoid multiple hinge formation in columns before attaining the full plate shear capacity.

Table 6. Maximum story shear carried by the upgraded frames as percentage of their design story shear

Story	7-story			5-story		
	Design story shear (kN)	Actual story shear (kN)	% achieved	Design story shear (kN)	Actual story shear (kN)	% achieved
1	4738	4403	92.93	3723	3257	87.50
2	4277	3456	80.80	2976	2622	88.10
3	4828	3249	67.29	3124	2280	72.97
4	4285	3380	78.90	3991	2378	59.58
5	4600	2950	64.13	4360	2138	49.04
6	4710	2710	57.52			
7	3248	1806	55.60			



(a) (b)

Figure 9. Plastic hinge formation in the boundary columns for the upgraded: a) 7-story; and b) 5-story frames

5. Concluding Remarks

This paper presents an easily adoptable method of upgrading non-seismic steel frame structures with steel plate shear walls to satisfy a probabilistically defined performance objective as recommended in today's advanced seismic design guidelines. The proposed semi-probabilistic procedure can take into account a multi-earthquake based probabilistic definition of hazard, as well as an inelastic displacement-based performance level, without involving complex probabilistic calculations in the upgrade procedure. The choice of a displacement-based performance level, and avoiding detailed probabilistic calculations while designing for a probabilistic performance objective make the procedure very attractive for implementing in practice. However, the omission of those complex calculations reduces the accuracy of the proposed procedure.

Through the probabilistic hazard curves and the IDA, the randomness in the record-to-record variation and in its effect on the nonlinear dynamic behaviour of the structure are properly accounted for. This is better than the practice of considering an elastic S_a with certain variation, and performing a single equivalent static analysis subjected to that (mean) spectral acceleration. However, there are other sources of uncertainty which are not incorporated in the proposed method. The primary sources of uncertainty, other than the two stated earlier, are in i) using the static energy based scheme for finding the required thickness, and ii) assuming the dominance of the fundamental mode in estimating the energy demand. These two sources are in a way connected to each other. Elastic and plastic seismic energy demands on a system is not always dominated by the fundamental mode of vibration (Prasanth et al. 2008), and Equation (1) gives a very rough estimate of the energy demand. The static monotonic energy based scheme assumes a uniform and unidirectional drift profile for the structure at the peak inelastic displacement. This, in a way, considers that the structure is vibrating with a linear mode shape (close to the case of vibrating in the fundamental mode), even when it is inelastic. It also assumes a typical yield mechanism for the system. The bi-linearization of the story shear versus inter-story drift plots in computing the energy demand introduces additional uncertainty. The proposed procedure can be improved greatly if these uncertainties can be properly quantified and incorporated in the upgrade scheme. This should give a better control on the confidence level of the upgraded structure similar to the design guidelines of FEMA-351 (FEMA 2000). Also, the fact that the results are based on a smaller statistics (of 20 records only) than necessary for the required probability levels is another reason for not achieving the target performance level accurately.

In summary, this paper proposes an innovative methodology of upgrading non-seismic structures using a rather recent structure type (steel plate shear walls) and it targets a probability-based displacement-based criterion for upgrade, while keeping the calculations simple and attractive. The emphasis of the work is in the exploration of developing such a simplistic methodology for a relatively complex problem. Two cases of application of the proposed method are illustrated, which show that the results are good but need further improvement. Various shortcomings of the proposed procedure are identified (primarily, regarding the proper design of boundary columns, accounting for all the uncertainties, consideration of a larger set of ground records, consideration of a safety factor, etc.) which can be taken up for future research in order to refine the proposed method and realize the promises shown in this work.

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