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RESEARCH ARTICLE

SEISMIC RESPONSE OF REINFORCED CONCRETE FRAMES WITH STEEL PLATE SHEAR WALLS

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ABSTRACT

This study aimed to evaluate the effect of using steel plate shear walls (SPSWs) on the earthquake response of reinforced concrete (RC) moment resisting frames. For this, two types of RC frames, namely, ductile and nominally ductile RC frames were used. Each frame structure had five storeys and three-bays. The first storey height was 4 m while the height of the other storeys was 3 m. To upgrade the performance of the existing structures under seismic actions, a special unstiffened SPSW system was incorporated into the middle-bays of each frame. SPSWs consist of an infill steel plate, horizontal and vertical boundary elements such as beams (as HBEs) and columns (as VBEs). The structures were modeled analytically using a finite element method and evaluated by both nonlinear static (pushover) and time history analyses. In the pushover analysis, the first mode load distribution and uniform load distribution were used. In the dynamic time history analysis, a set of ground motion records was employed. As a seismic hazard level, 2% probability of exceedance in 50-year period was taken into consideration. The seismic response of the ductile and nominally ductile RC frames with and without SPSWs was examined comparatively. The results indicated a considerable enhancement in the earthquake performance of both the ductile and the nominally ductile RC frame structures with SPSWs.

Keywords: Dynamic response, Nonlinear analysis, Reinforced concrete frame, Seismic performance, Steel plate shear wall

1. INTRODUCTION AND BACKGROUND

Steel plate shear walls (SPSWs) recently becomes an attractive alternative as a lateral load-resisting system against lateral loads such as wind and earthquake loads. SPSW system consists of horizontal boundary elements (HBEs), vertical boundary elements (VBEs) and infill steel plates that connected to these HBEs and VBEs over the frame bay in full height. In recent decades, the SPSW system has been employed in various buildings effectively as the primary lateral load resisting system [1-5].

The SPSWa behave in a very ductile manner, dissipate significant amount of energy, and help in satisfying design requirements in seismic zones. The SPSWs have some major advantages compared with many other systems. Due to its thin profile, this system has less weight and more architectural flexibility than comparable reinforced concrete shear walls. Moreover, it has the advantage of high initial stiffness and ultimate strength, distributed yielding over multiple stories, ease of construction, and short construction time [6]. Also shorter bays can be used and therefore it provides more options for the space usage and building configurations [7]. This designed system is very effective in limiting the drift due to relatively high initial stiffness. Moreover, reduction of the structure mass causes less seismic load [8].

When the design of the infill plates is taken into consideration, SPSWs have two types with infill plates stiffened or unstiffened [9]. In the study of Ghosh et al. [10], it was observed that due to stable hysteretic energy dissipation behaviour of unstiffened steel plates; against earthquake shaking the postbuckling ductile behaviour of an unstiffened SPSW is more efficient than that of a stiffened SPSW.

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In the literature, some researchers studied beneficial models about the lateral load resisting system and designed on the steel plate shear walls [7, 11-17]. To expand the SPSW applicability, the innovative design of SPSWs has been proposed, investigated, and experimentally validated [1-4, 13, 15, 18-27]. For instance, by using finite element analysis, Zirakian and Zhang [18] investigated the influence of the unstiffened infill plates which were classified as slender, moderate, and stocky on the behaviour and performance of SPSWs. The research of Thorburn et al. [11] introduced development of the strip model, an analytical method for modeling SPSW where the panel is represented by a series of pin-ended truss elements oriented in the direction of the tension field to carry tension force only. Moreover, Caccese et al. [22] conducted an experimental program to investigate the effects of panel slenderness ratio and beam-to-column connection type on SPSW hysteretic behaviour. Similarly, Tromposch and Kulak [23] conducted an experimental test on a single full-scale SPSW and compared the experimental results with the analytical ones. They showed that the analytical outcomes which were based on the strip model methods established by Thorburn et al. [11] and Timler and Kulak [24], were convenient with the experimentally obtained hysteretic loops. Qu et al. [25] and Vian et al. [26, 27] both conducted the analytical and experimental researches on SPSWs with reduced beam section (RBS) connections. The research conducted by Qu et al. [25] consisted of a two stage experimental program where a full scale two storey SPSW specimens with RBS connections and composite floors were tested. In addition to these, Vian et al. [26, 27] carried out the experimental and analytical investigation of special perforated SPSWs with RBS anchor beams.

However, to increase the utilization of such systems, their response under severe dynamic actions is required to be better understood. Therefore, in this study, the nonlinear static and dynamic time history responses of the reinforced concrete (RC) structures with and without SPSWs were examined comparatively. For this, two types of RC frames, namely, ductile and nominally ductile moment resisting frames were utilized. To upgrade the seismic performance of the existing structures, a special unstiffened SPSW system was incorporated into the middle-bays of each frame. The SPSWs with 2 m wall length and 3 mm thickness was utilized for all frames. The results based on the capacity curve, storey displacement, interstorey drift ratio, time history of base shear and roof displacement were evaluated for both ductile and the nominally ductile RC frame structures with and without SPSWs.

2. DESCRIPTION OF SAMPLE STRUCTURES

In this study, ductile and nominally ductile reinforced concrete (RC) moment resisting frame (MRF) buildings were selected as case study structures. Both of them were designed by Sadjadi et al. [28] considering gravity and lateral loads in accordance with Canadian Concrete Standard [29] and National Building Code of Canada [30], and in the current study these frames were used for different purposes. In modeling of the frame, gravity loads such as dead load and live load were taken into consideration. In the calculation of the dead load (DL), the weight of the structural frame elements and also the weight of the masonry infill walls were included. The live load (LL) was taken as 2.4 kN/m^2 for each story, which was typical for residential buildings [28].

The typical floor plan and elevation views of the 5-storey buildings are shown in Figure 1 [28]. Each frame is accepted as a part of the lateral load resisting system. The building has a total height of 16 m consisting of 3 m storey height except the first storey which is 4 m.

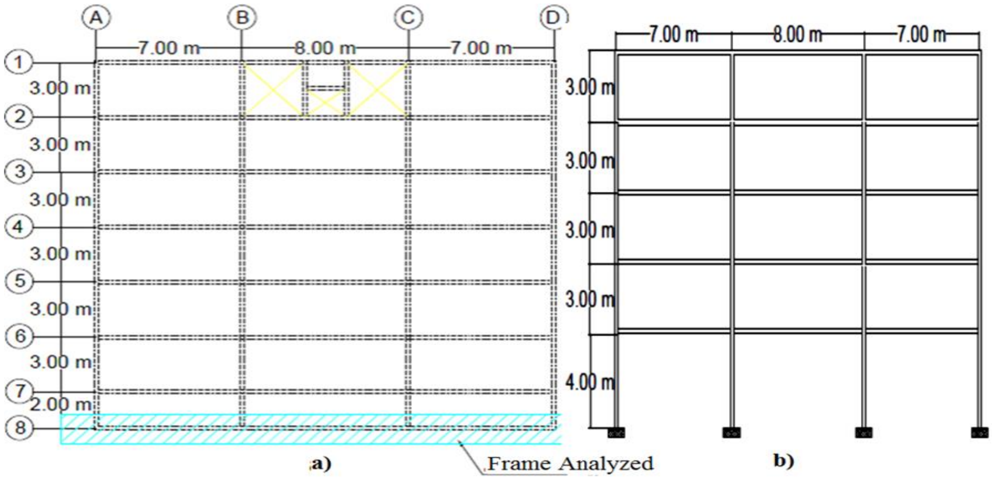


Figure 1. The five-storey RC frame (a) plan and (b) elevation views [28]

The material properties are identical throughout the height of the structure. The compressive strength of concrete is f_c'=35 MPa and the reinforcing steel yield strength is f_y=400 MPa. The details of the section properties of the ductile and nominally ductile frames are summarized in Table 1.

The cross sectional properties of the columns and the amount of reinforcement in these columns differ from each other in the ductile and nominally ductile frames. For example in the ductile frame, the interior columns have 12 No.30 bars, and exterior columns have 12 No.20 bars whereas in the nominally ductile frame, columns have 12 No.30 bars at 1st, and 2nd storey, 8 No.30 bars at 3rd, and 8 No.25 bars at 4th, and 5th stories. The section properties of beam for both frames are the same, however, these beams have different reinforcement ratio. Further detailed information regarding the design properties of the ductile and nominally ductile frames can be found in the study of Sadjadi et al. [28].

Table 1. Cross-sectional properties of ductile and nominal ductile frames [28]

<table>
<tr><th colspan="2">Type</th><th colspan="2">Storey</th><th>Dimension (mm)</th></tr>
<tr><td rowspan="4">Ductile Frame</td><td rowspan="2">Beams</td><td colspan="2">1st and 2nd</td><td>600×600</td></tr>
<tr><td colspan="2">3rd, 4th and 5th</td><td>500×500</td></tr>
<tr><td rowspan="2">Columns</td><td rowspan="2">All storeys</td><td>Interior</td><td>800×800</td></tr>
<tr><td>Exterior</td><td>600×600</td></tr>
<tr><td rowspan="4">Nominally Ductile Frame</td><td rowspan="2">Beams</td><td colspan="2">1st and 2nd</td><td>600×600</td></tr>
<tr><td colspan="2">3rd, 4th and 5th</td><td>500×500</td></tr>
<tr><td rowspan="2">Columns</td><td rowspan="2">All storeys</td><td>Interior</td><td>600×600</td></tr>
<tr><td>Exterior</td><td>600×600</td></tr>
</table>

SPSWs were designed according to the provisions AISC [31]. I sections were used to design horizontal boundary elements (HBEs) and vertical boundary elements (VBEs) for all walls of existing SPSW systems. Moreover, the steel grades; S235 and S355 were used for the infill plates (having 2 m wall length and 3 mm thickness) and for the boundary members, respectively. Boundary members were placed at all sides of infill plate at all storeys. Figure 2 shows the location of the SPSWs in the ductile and nominally ductile RC frames. As seen in the figure, the SPSWs were located at the same place in both frames.

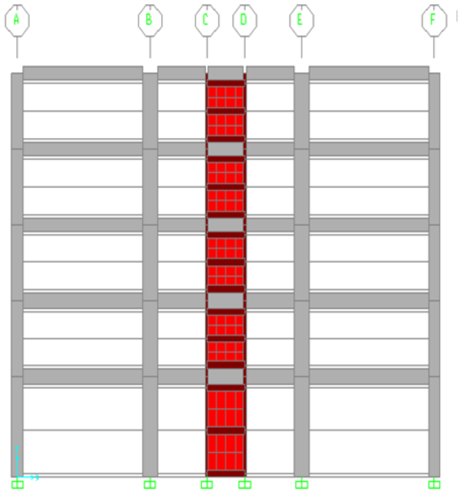


Figure 2. Elevation view of the frame with SPSWs

3. ANALYTICAL STUDY

The analytical model of the frames, including nonlinear properties of the structural members was obtained. Non-linear static analysis, which is also known as pushover analysis consists of pushing structure horizontally in one direction with a specific lateral force or the displacement distribution in order to achieve either the specified displacement or a lack of numerical stability. Increasing the lateral forces applied to the model in a systematic manner is based on the initial load pattern or mode shape. In this study, a triangular load pattern determined from the first mode shape of the structure and uniform load distribution were utilized as an initial load pattern. For each step, the forces are calculated in members, and the member stiffness is changed. When the frame becomes unstable or target displacement reaches, the process finish.

Lumped plasticity approach in which nonlinearity was considered by implementing plastic hinges with hysteretic behaviour based on FEMA 356 [32] to each end of the beam and column members was utilized. For the column members, axial force and moment hinges (P-M2-M3), and for the beams, flexural moment hinges (M3) were considered. At the end of the pushover analysis, the capacity curves illustrating the base shear-roof displacement relation were obtained for the frames with and without SPSW. Thus, the nonlinear behaviour of the frames under lateral loads and the effects of SPSW on the nonlinear behaviour of the frames were investigated through pushover analysis.

In addition to nonlinear static (pushover) analysis, nonlinear dynamic (time history) analysis in which the response of the structure either in force or in deformations was calculated as a function of time, under earthquake accelerations, were conducted. From the nonlinear time history analysis, the variation of maximum storey displacement, inter-storey drift through within the building height, maximum base shear, the variation of base shear, and some floor displacements with time were obtained for both ductile and nominal ductile frames with and without SPSW by using several ground motions.

These earthquake accelerations were generated by using PEER ground motion database [33] considering seismic hazard level of 2% probability of exceedance in fifty years period. The characteristics of the natural earthquake accelerations used such as properties of the site where accelerations recorded, their magnitude (M_w), peak ground acceleration (PGA), peak ground velocity (PGV) and peak ground displacement (PGD) were summarized in Table 2. Furthermore, the elastic spectral accelerations of the ground motions are given in Figure 3.

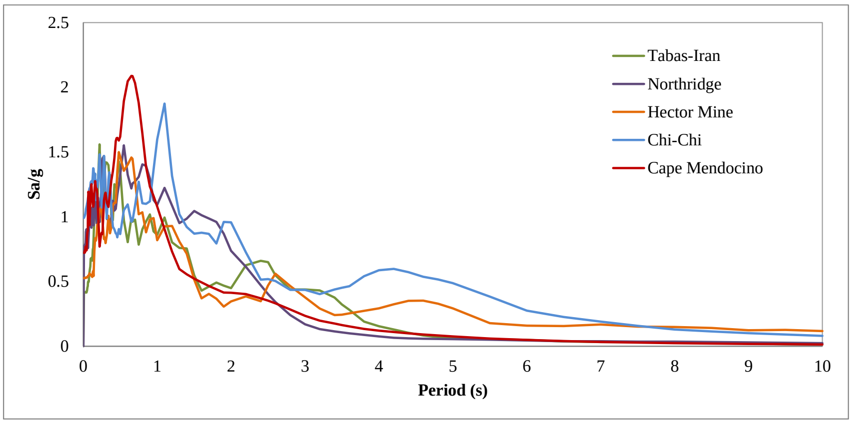


Figure 3. Elastic spectral accelerations of the ground motions

Table 2. Characteristics of the ground motions

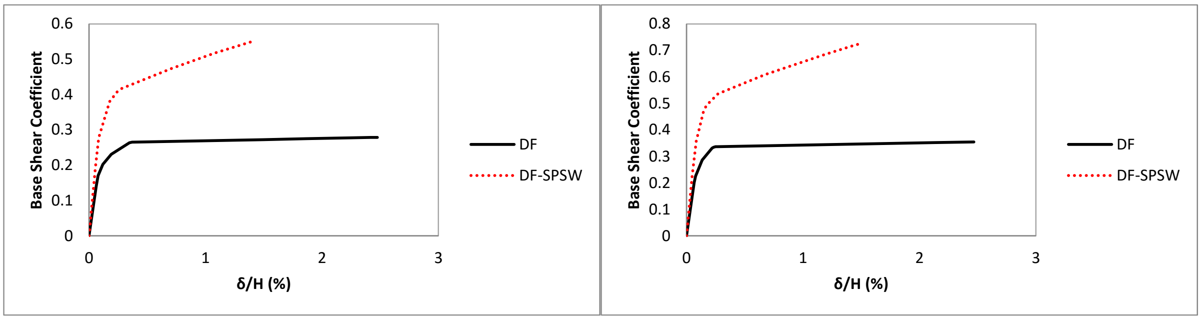
Earthquake	Year	Magnitude (M_w)	Mechanism	R_{jb} (km)	R_{ump} (km)	V_{s30} (m/s)	PGA (g)	PGV (cm/s)	PGD (cm)
Tabas-Iran	1978	7.35	Reverse	1.8	2	766.8	0.81	118.3	96.8
Northridge	1994	6.69	Reverse	0	5.3	251.2	0.80	93.3	53.3
Chi-Chi	1999	7.62	Reverse-Oblique	0.6	0.6	305.9	0.82	127.8	93.2
Cape Mendocino	1992	7.01	Reverse	0	8.2	712.8	0.62	81.9	25.5
Hector Mine	1999	7.13	Strike-Slip	195.6	195.9	294.2	0.51	100.2	146.8

4. RESULTS AND DISCUSSION

In this section, the results obtained by nonlinear static (pushover) and time-history analyses for the ductile and nominally ductile RC buildings with and without steel plate shear walls were given and discussed comparatively. Performance characteristics in terms of capacity curves, variation of storey displacement, inter-storey drift ratio, variation in the base shear and roof displacement vs. time were given below.

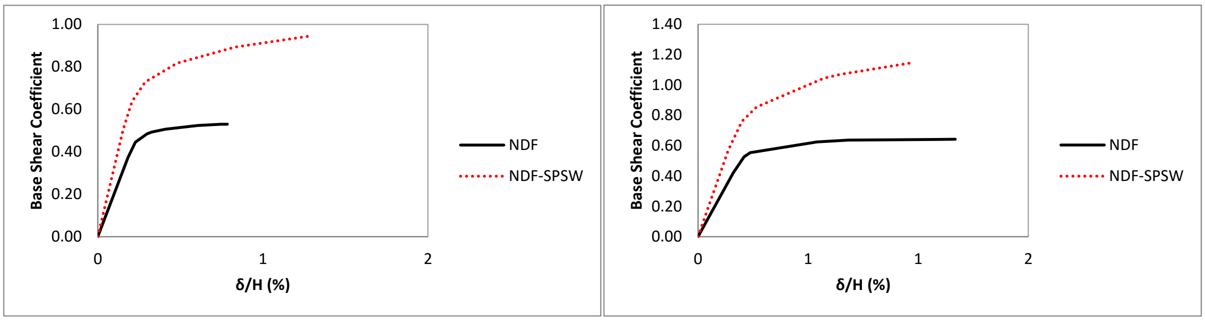
Figures 4 and 5 show the comparison of the capacity curves obtained in terms of base shear coefficient (which means total base shear normalized with the total building weight) and roof displacement ratio (which means the roof drift normalized with the building height) of the ductile and nominally ductile frames with and without steel plate shear walls. In order to evaluate the stiffness and strength characteristics of the upgraded ductile and nominally ductile RC frames with SPSW, firstly the nonlinear pushover analysis was carried out by using an inverted triangular lateral load distribution (that is based on the 1st natural mode shape) and uniform load distribution.

As seen from these figures, the frames with SPSWs had considerably greater capacity in comparison to the original ductile and nominal ductile frames. For example, under uniform load distribution, for the ductile frames, the maximum base shear coefficient of the original ductile frame was about 0.35 while that of the frame with SPSWs was nearly 0.73. This implied about 2.09 times higher lateral load carrying capacity for the ductile frame with SPSW in comparison to the original ductile frame. For the nominally ductile frames, under uniform load distribution, the maximum base shear coefficient of the original nominally ductile frame was about 0.64 while that of the upgraded frame with SPSWs was nearly 1.15. This implies about 1.8 times higher lateral load carrying capacity for the upgraded nominally ductile frame in comparison to the original frame. Hence, from these results it is examined that the steel plate shear walls effectiveness in increasing the load carrying capacity of the ductile and nominal ductile frames are almost similar independent from the shape of the load used in pushover analysis. Furthermore, it was observed that for both ductile and nominally ductile frame, SPSW caused increase in the building stiffness in comparison to the original frames.



a) b)

Figure 4. Capacity curves of the ductile frame (DF) with and without SPSW obtained under (a) 1st mode and (b) uniform load distribution.

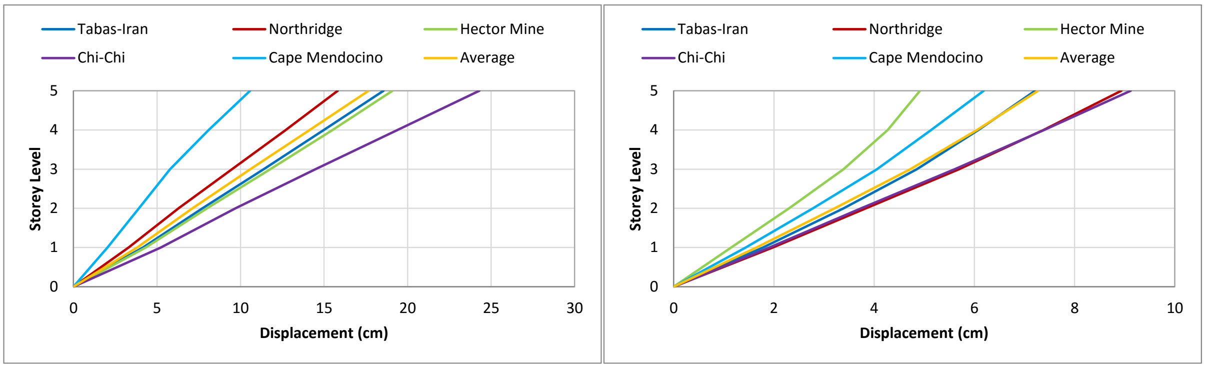


a) b)

Figure 5. Capacity curves of the nominal ductile frame (NDF) with and without SPSW obtained under (a) 1st mode and (b) uniform load distribution.

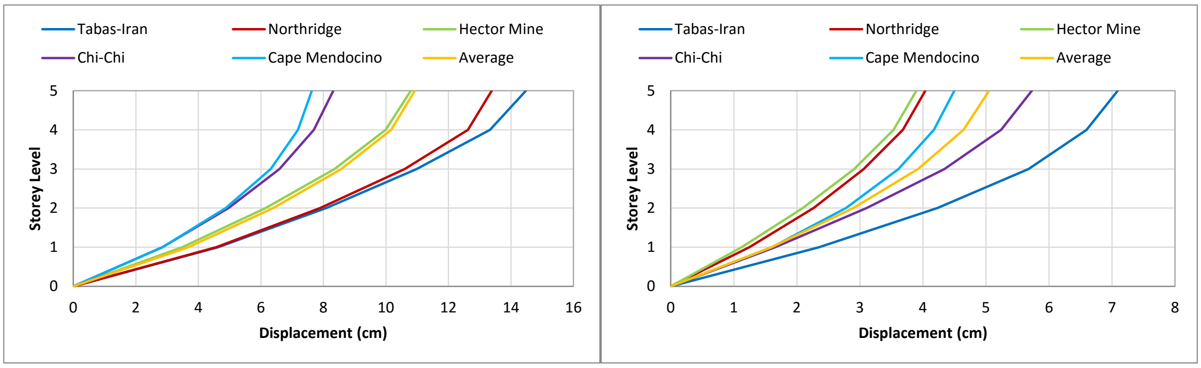
From the nonlinear time history analysis, the variation of the maximum displacements at each storey under each ground motion was obtained as given Figures 6 and 7. It was observed that the use of SPSWs remarkably decreased the value of the maximum storey displacements observed at ductile and nominally ductile frames.

As expected, the maximum storey displacement observed in ductile and nominal ductile bare frames differ from each other, such that the former had greater storey displacement than later. The effectiveness of the steel plate shear walls in mitigating the seismic response was evaluated and it was seen that SPSW systems significantly decreased the maximum displacement demand of the original ductile and nominally ductile frames. For example, as seen in Figure 6, under the Tabas-Iran earthquake, the maximum displacement of the ductile frame was obtained as 18.57 cm while the maximum displacement of the ductile frame that with SPSWs was obtained as 7.21 cm. In the case of nominally ductile frames (see Figure 7), under the Tabas-Iran earthquake, the maximum displacement of the nominally ductile frame was achieved as 14.48 cm while the maximum displacement of that with SPSWs was obtained as 7.09 cm.



a) b)

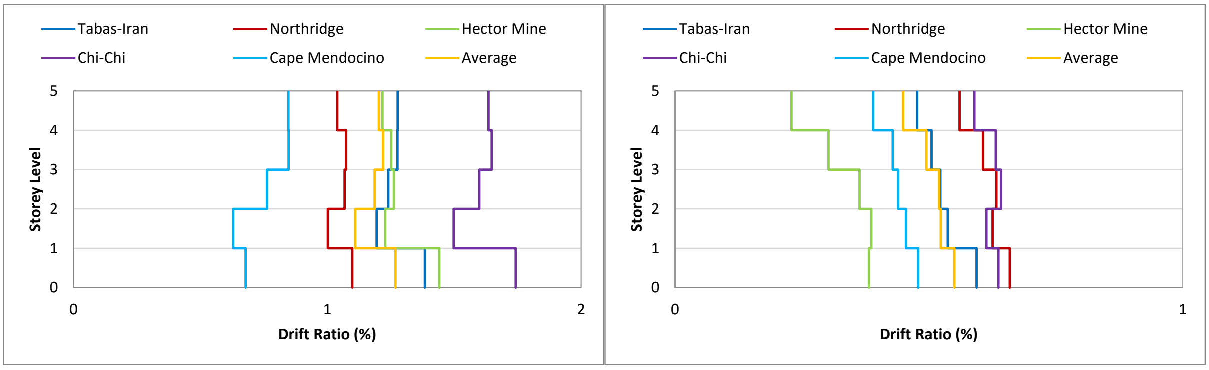
Figure 6. Storey displacement variation in ductile (a) bare frame and (b) frame with SPSW



a) b)

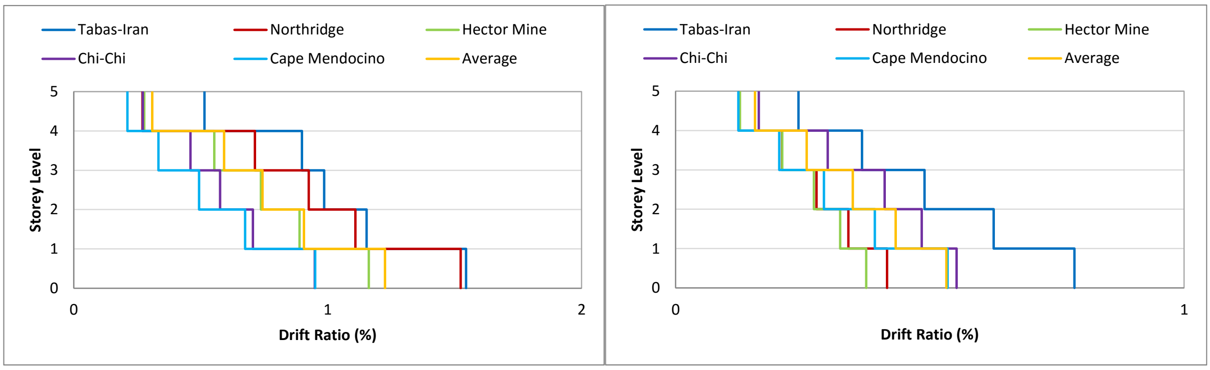
Figure 7. Storey displacement variation in nominal ductile (a) bare frame and (b) frame with SPSW

The plots showing the maximum inter-storey drift ratio variation through the height of the ductile and nominally ductile frames with and without SPSWs are given in Figures 8 and 9. According to the results of analysis, it was observed that using SPSW systems decreased significantly inter-storey drift ratio of the ductile and nominally ductile frames under the all earthquake ground motions. For example, as seen in Figure 8, for the ductile frames, under the Chi-Chi earthquake, the maximum inter-storey drift ratio of the ductile frame was achieved as 1.74% while the maximum inter-storey drift ratio of the ductile frame with SPSWs was obtained as 0.64%. Moreover, as seen in Figure 9, for the nominally ductile frames, under the Chi-Chi earthquake, the maximum inter-storey drift ratio of the nominally ductile frame was found as 0.95% while the maximum inter-storey drift ratio of the nominally ductile frame with SPSWs was obtained as 0.55%. In particular, the average maximum inter-storey drift ratio for the ductile frame with SPSWs was about 0.75% while the average maximum inter-storey drift ratio of the nominally ductile frame with SPSWs was about 0.61%. These results confirmed that SPSW were very effective in reducing the inter-storey drift demand. Furthermore, as clearly observed from the figures, the use of SPSW tends to distribute the drift demand more uniformly through the height of the frame.



a) b)

Figure 8. Variation of the maximum inter-storey drift ratio of ductile (a) bare frame and (b) frame with SPSW



a) b)

Figure 9. Variation of the maximum inter-storey drift ratio of nominal ductile (a) bare frame and (b) frame with SPSW.

Figures 10 and 11 show the variation of base shear with time for the ductile and nominally ductile frames with and without SPSWs. The maximum base shear was affected both by the frame type and properties of ground accelerations. For example, in the case of nominally ductile frame, the maximum base shear was greater than the ductile frames. It was observed that the use of SPSWs for the purpose of enhancement of seismic behaviour increased remarkably the value of the maximum base shear as compared to original frames, especially in the case of nominally ductile frames.

As seen in Figure 10, for the ductile frames, under the Hector Mine earthquake, the maximum base shear of the ductile frame was achieved as 775 kN. The maximum base shear of the ductile frame with SPSWs was obtained as 1078 kN. Moreover, as seen in the Figure 11, for the nominally ductile frames, under the Hector Mine earthquake, the maximum base shear of the nominally ductile frame was found as 1082 kN while the maximum base shear of the frame with SPSWs was obtained as 1423 kN. Thus, with the addition of SPSW, the base shear demand of the frames was increased about 1.03 to 1.59 times. With the inclusion of SPSWs, the higher base shear was induced due to relatively high lateral stiffness of the frames.

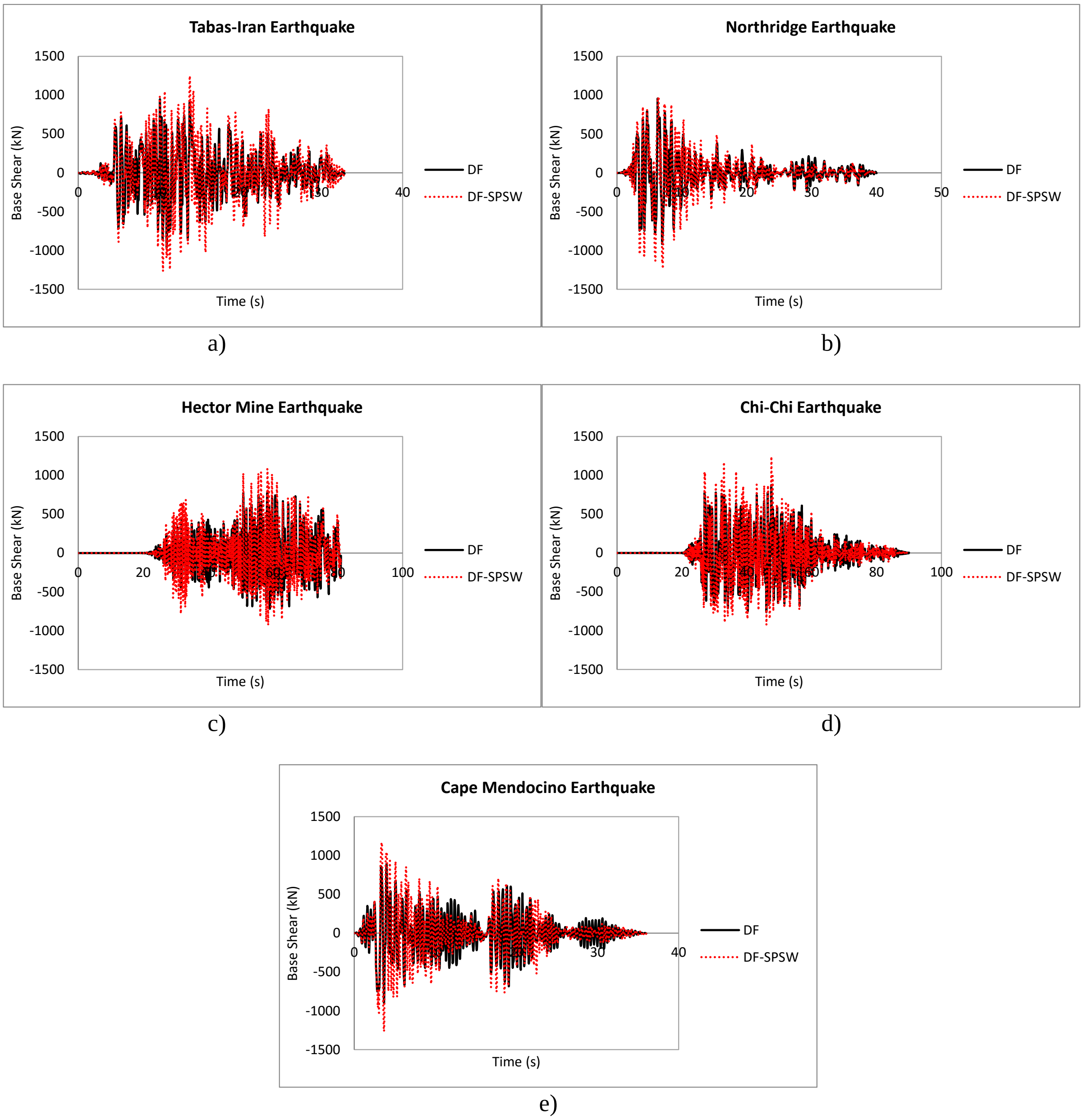


Figure 10. Base shear vs. time for the ductile frame (DF) and that with SPSWs (a) Tabas-Iran, (b) Northridge, (c) Hector Mine, (d) Chi-Chi, and (e) Cape Mendocino earthquakes.

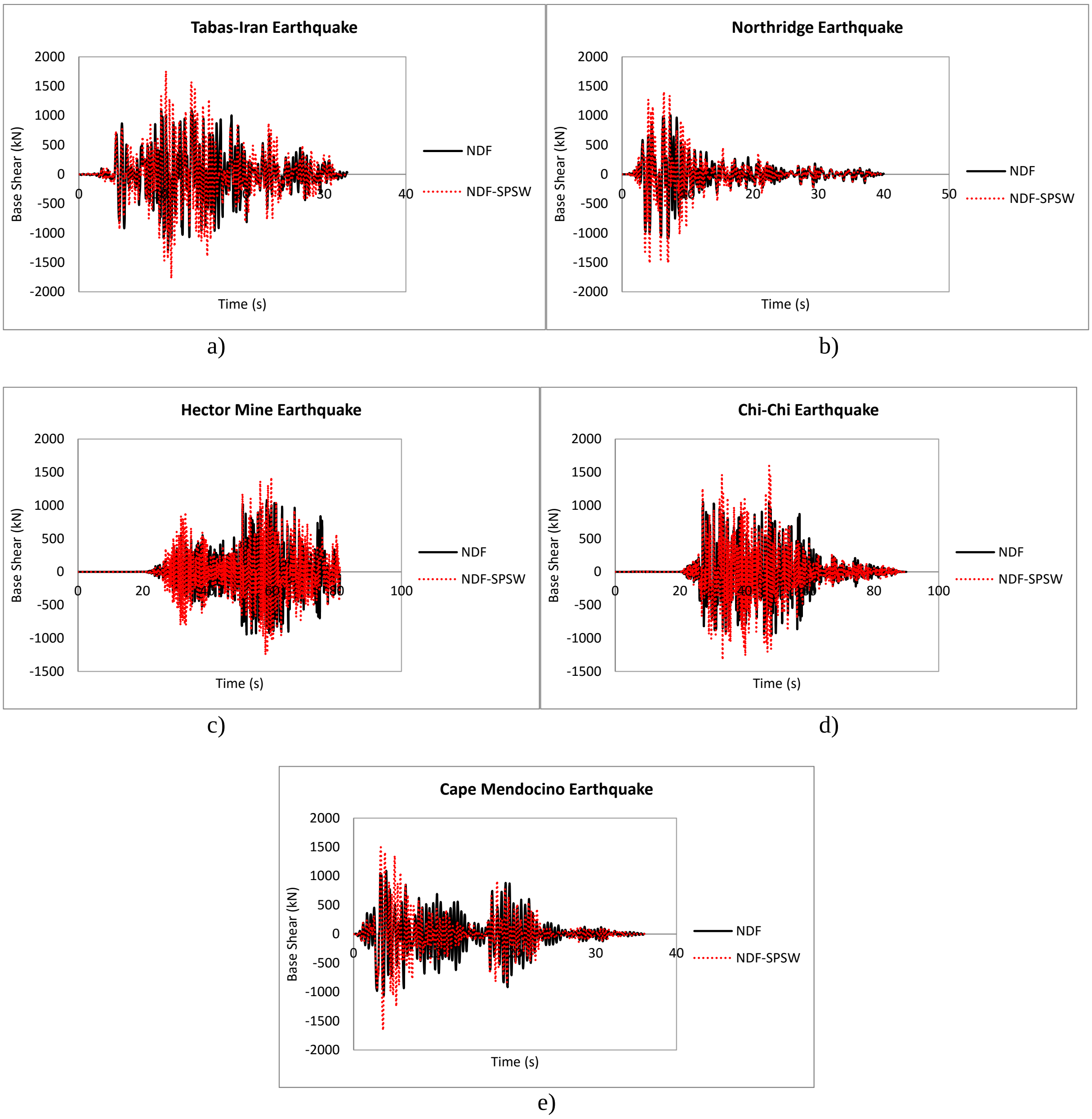


Figure 11. Base shear vs. time for the nominal ductile frame (NDF) and that with SPSWs (a) Tabas-Iran, (b) Northridge, (c) Hector Mine, (d) Chi-Chi, and (e) Cape Mendocino earthquakes.

Figures 12 and 13 show the roof displacement time history of the ductile and nominally ductile frames and that with SPSWs under the seismic excitations of Tabas-Iran, Northridge, Hector Mine, Chi-Chi, and Cape Mendocino earthquakes. According to the results obtained from the nonlinear time history analysis, the utilization of SPSWs decreased significantly roof displacement of the both existing ductile and nominally ductile frames under the all earthquake ground motions. For example, as shown in Figure 12, for the ductile frames, under the Northridge earthquake, the maximum roof displacement of ductile frame was 15.82 cm while that of the frame with SPSWs system was achieved as 8.94 cm. Similarly, as seen in Figure 13, for the nominally ductile frames, under the same earthquake, the maximum roof displacement of the nominally ductile frame was achieved as 13.39 cm whereas the maximum roof displacement of the nominally ductile frame with SPSWs was found as 4.04 cm. It was also evident from these Figures 12 and 13 that the application of SPSWs system into frames resulted in a marked maximum reduction in the roof displacement up to about 74% and 70% for ductile and nominally ductile frames, respectively. For example, under Hector Mine earthquake, for the ductile and nominally ductile frames the use of SPSW resulted in 74% and 64% reduction, respectively.

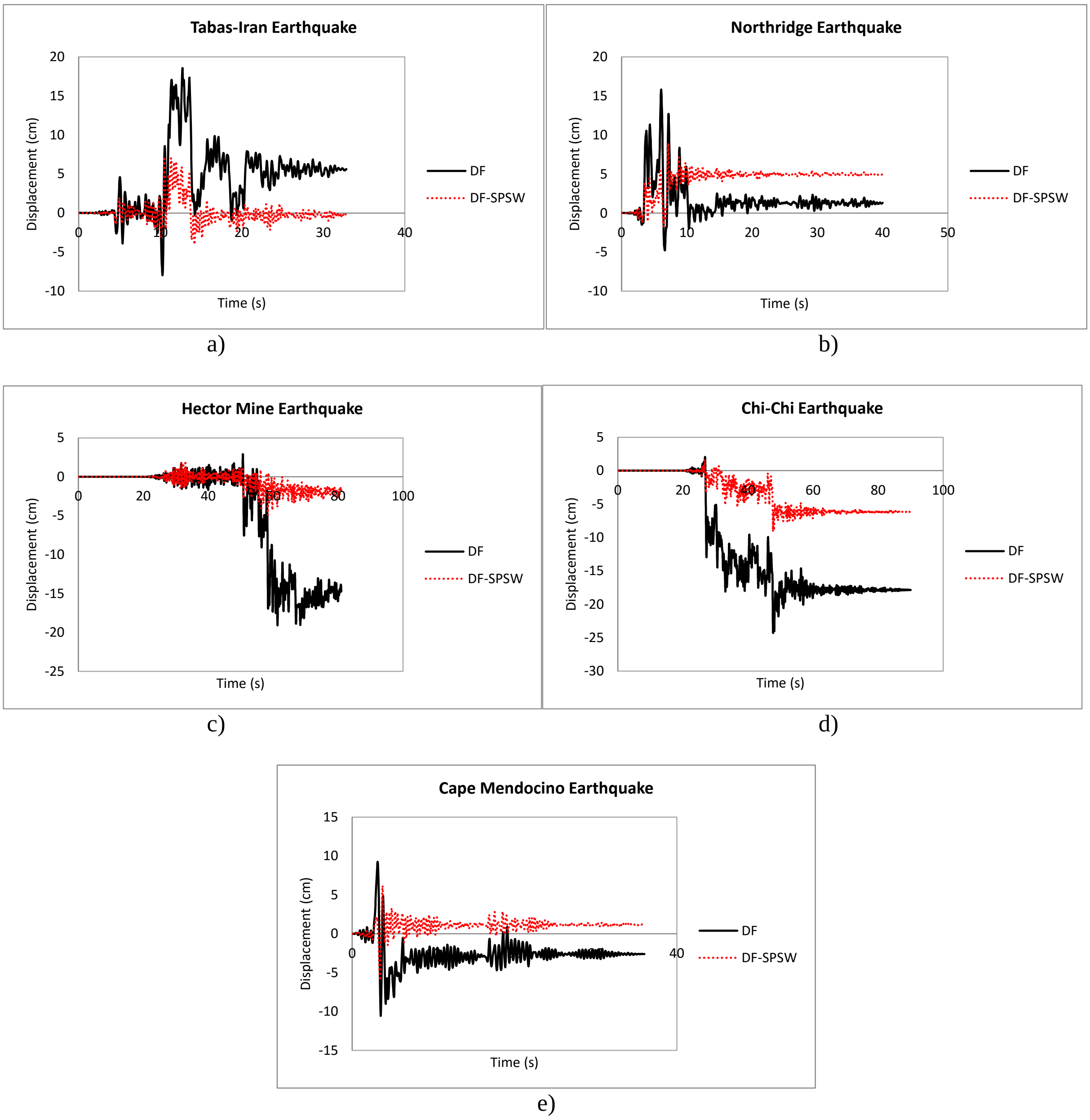


Figure 12. Roof displacement vs. time for the ductile frame (DF) and that with SPSWs (a) Tabas-Iran, (b) Northridge, (c) Hector Mine, (d) Chi-Chi, and (e) Cape Mendocino earthquakes.

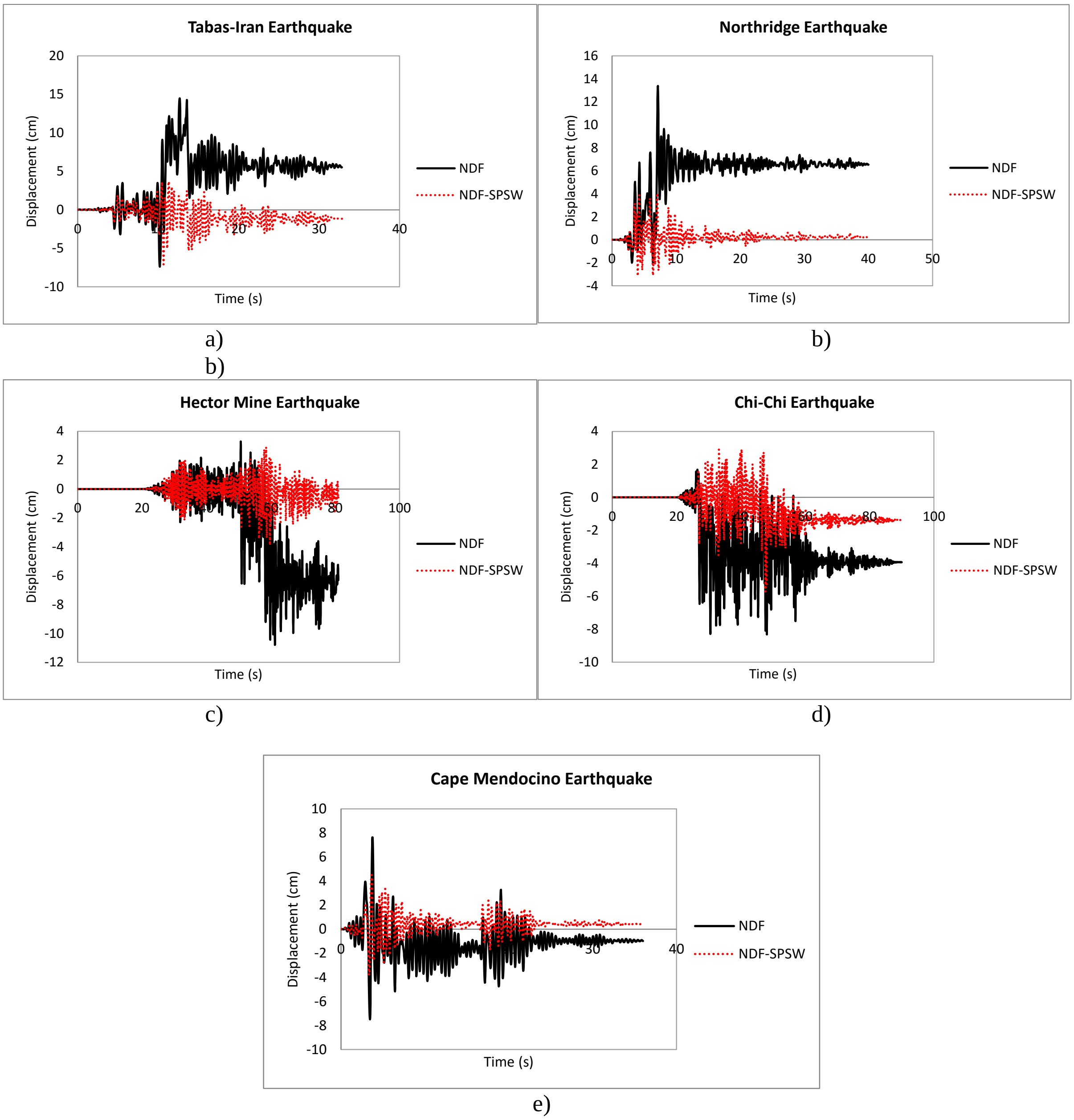


Figure 13. Roof displacement vs. time for the nominal ductile frame (NDF) and that with SPSWs (a) Tabas-Iran, (b) Northridge, (c) Hector Mine, (d) Chi-Chi, and (e) Cape Mendocino earthquakes.

5. CONCLUSIONS

This study investigated the structural performance of the existing 5-storey ductile and nominally ductile reinforced concrete buildings and those upgraded with steel plate shear walls (SPSWs). The nonlinear pushover analysis considering first mode load distribution and uniform load distribution was first performed, and then nonlinear dynamic time history analysis was conducted under natural ground accelerations.

In the pushover analysis of the capacity curves, a significant improvement in the seismic performance of the ductile and nominally ductile frames was observed. As expected, the upgraded ductile and nominal ductile frames with SPSWs resulted in higher lateral load carrying capacity than the original ductile and nominally ductile frames. The stiffness of the existing frames was also increased by applying SPSW systems. In the time history analysis, the results confirmed that the SPSWs for both ductile and nominally ductile frames were very effective in reducing the displacement and the inter-storey drift demands. Even though the frames with SPSWs was subjected to greater lateral loads under the earthquakes, their displacement demand is less and the use of SPSWs tend to distribute the drift demand more uniformly along the height of the frame, especially for the ductile case. Thus, in the case of the earthquakes having different characteristics, a substantial reduction of drift demand could be observed in the frames with SPSWs. As a result, the SPSW systems could be used effectively as an attractive alternative for upgrading the lateral load carrying capacity of the ductile and nominally ductile designed mid-rise reinforced concrete structures in seismic areas.

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