Comparison of Elastic Design and Performance Based Plastic Design Method Based on the Inelastic Response Analysis using SAP2000

Sejal P Dalal
Assistant Professor
Civil Engineering Dept
SVIT,Vasad,India

S A Vasanwala
Associate Professor
Applies Mechanics Dept
SVNIT,Surat

A K Desai
Associate Professor
Applies Mechanics Dept
SVNIT,Surat
Gujarat,India

ABSTRACT
Presented in this paper is the comparison of a steel moment resisting frame designed by the Performance based Plastic design method and conventional elastic design method based on the seismic evaluation done by both nonlinear static (Push over Analysis) and nonlinear dynamic analysis (Time history analysis) under different ground motions using the SAP2000 software. The Performance based Plastic design is a displacement based method which uses pre-selected target drift and yield mechanisms as design criteria whereas the elastic design method is based on the conventional force based limit state method. The nonlinear static pushover analysis shows formation of hinges in columns of the frame designed using elastic design approach leading to collapse. Whereas in the Performance based Plastic design method, formation of hinges is seen in the beams and bottom of base columns. Although the ground motions caused large displacements in the Performance based Plastic design frame as it was seen from the acceleration and displacement responses obtained from the nonlinear time history analysis, the structure did not lose stability. Study of hysteretic energy dissipation results reveals that the Performance based Plastic design method is superior to the elastic design method in terms of the optimum capacity utilization.

Keywords
Performance based Plastic Design, Nonlinear Static Pushover Analysis, Nonlinear Time History Analysis, Inelastic response, SAP2000.

1. INTRODUCTION
Performance based Plastic design method is a rapidly growing design methodology based on the probable performance of the building under different ground motions. The structures designed by current codes undergo large inelastic deformations during major earthquakes. The current seismic design approach is generally based on elastic analysis and accounts for inelastic behavior in a somewhat indirect manner. The inelastic activity, which may include severe yielding and buckling of structural members and connections, can be unevenly and widely distributed in the structure. This may result in a rather undesirable and unpredictable response, sometimes total collapse, or difficult and costly repair work at best (Dalal ,[1]).

It should be noted that in this design approach, the designer selects the target drifts consistent with acceptable ductility and damage, and a yield mechanism for desirable response and ease of post earthquake damage reparability. The method has been successfully applied to a variety of common steel framing systems like Steel Moment Resisting Frame (Lee and Goel, [2]), buckling restrained braced frame, Eccentrically Braced Frame (Chao and Goel, [3]), concentric braced frames(Chao and Goel , [4]) Special Truss Moment Frame (Chao and Goel , [5]), composite buckling restrained braced frame (Dasgupta et al, [6]) and, more recently, to Reinforced Concrete (RC) moment frames. Results of extensive inelastic static and dynamic analyses showed that the frames developed desired strong column-sway mechanisms, and the storey drifts and ductility demands were well within the target values, thus meeting the desired performance objectives. Comparisons of responses with corresponding baseline frames designed by current practice have consistently shown superiority of the proposed methodology in terms of achieving the desired behavior.

2. THE SEISMIC EVALUATION OF STRUCTURES DESIGNED USING PBPD METHOD
The seismic evaluation of structures designed using the PBPD method can be done either by nonlinear static (Pushover) analysis or nonlinear dynamic (Time History) analysis. It must be emphasized that the pushover analysis is approximate in nature and is based on static loading. As such it cannot represent dynamic phenomena with a large degree of accuracy. It may not detect some important deformation modes that may occur in a structure subjected to severe earthquakes, and it may exaggerate others. Inelastic dynamic response may differ significantly from predictions based on invariant or adaptive static load patterns, particularly if higher mode effects become important. Thus, performance of pushover analysis primarily depends upon choice of material models included in the study.

The time history analysis is an actual dynamic analysis that can be done for both linear and nonlinear systems. It is found that this analysis incorporates the real time earthquake ground motions and gives the true picture of the possible deformation and collapse mechanism in a structure. But, at the same time, it is a very tedious and complex analysis having a lot of mathematical calculations. Although non-linear dynamic analysis is generally considered to be the most accurate of the available analysis methods, it is cumbersome for design. Also, mathematically, nonlinear static analysis does not always guarantee a unique solution. Small changes in properties or loading can cause large changes in nonlinear response. And hence it is advisable to perform these sophisticated analyses on software. Today, various softwares are available for these complicated analyses to make our task easier and faster. SAP2000 (CSI, [8]) is one of the most sophisticated and user-friendly software which performs the non- linear static (Push Over) and non- linear Time history analysis in a very simple way.

3. NON-LINEAR STATIC ANALYSIS (PUSH OVER ANALYSIS)

The static pushover analysis is becoming a popular tool for seismic performance evaluation of existing and new structures. The expectation is that the pushover analysis will provide adequate information on seismic demands imposed by the design ground motion on the structural system and its components. A pushover analysis is performed by subjecting a structure to a monotonically increasing pattern of lateral forces, representing the inertial forces which would be experienced by the structure when subjected to ground shacking. Under incrementally increasing loads various structural elements yield sequentially. Consequently, at each event, the structure experiences a loss in stiffness. Using a pushover analysis, a characteristic nonlinear force-displacement relationship can be determined. Typically the first pushover load case is used to apply gravity load and then subsequent lateral pushover load cases are specified to start from the final conditions of the gravity.

4. INELASTIC NON-LINEAR TIME HISTORY ANALYSIS

Non-linear structural analysis is becoming more important in earthquake resistant design, particularly with the development of performance based earthquake engineering, which requires more information about the drifts, displacements and inelastic deformations of a structure than traditional design procedures. Inelastic time history analysis is dynamic analysis, which considers material nonlinearity of a structure. Considering the efficiency of the analysis, nonlinear elements are used to represent important parts of the structure, and the remainder is assumed to behave elastically. Nonlinear elements are largely classified into Element Type and Force Type.

The Element Type directly considers nonlinear properties by changing the element stiffness. SAP2000 programs use the Newton-Raphson iteration method for nonlinear elements of the Element Type to arrive at convergence. Direct integration must be used for inelastic time history analysis of a structure, which contains nonlinear elements of the Element Type.

The Force Type indirectly considers nonlinear properties by replacing the nodal member forces with loads without changing the element stiffness. For nonlinear elements of the Force Type, convergence is induced through repeatedly changing the loads. If a structure contains nonlinear elements of the Force Type only, much faster analysis can be performed through modal superposition. Iterative analysis by the Newton-Raphson method is carried out in each time step in the process of obtaining the displacement increment until the unbalanced force between the member force and external force is diminished.

The unbalanced force is resulted from the change of stiffness in nonlinear elements of the Element Type and the change of member forces in nonlinear elements of the Force Type. The analysis of a 20 storied steel moment resisting frame is done using both the methods discussed above and is described in the next section.

Moment frames are very common for steel as well as RC building structures. The Seismic Evaluation of a 20-Storey Structural Engineers Association Of California (SEAOC), The Applied Technology Council (ATC), And Consortium Of Universities For Research In Earthquake Engineering (CUREE) Steel Moment Frame (also known as SAC steel moment frame) using SAP2000 is presented in this section. The frame was designed by the PBPD method its responses under static pushover and dynamic time-history analyses due to selected set of ground motions were studied. The framing plan of the structure is shown in Figure 1. Since the original SAC frame was designed according to the Uniform Building Code (UBC) (1994), same loading and other design parameters were used for the redesigning of the frame by PBPD method. The storey heights are 18 ft for the first storey and 13 ft for all others.

According to the UBC (1994), the elastic design spectral acceleration, $S_a = ZIC$, where Z is the seismic zone factor, I is the occupancy importance factor and C is the seismic coefficient.

With $S = 1{\cdot}2$ for S2 soil type, $Z = 0{\cdot}4$, $I = 1{\cdot}0$ and estimated $T = 2{\cdot}299$ s, C =0.9 , the value of S_a turned out to be equal to 0·36.

The design base shear was determined for a 2% maximum storey drift ratio (θ_u) for ground motion hazard with a 10% probability of exceedance in 50 years (10/50 or 2/3 maximum considered earthquake (MCE)); A yield drift ratio (θ_y) of 1·0% was used, which is typical for steel moment frames. The calculated values of significant design parameters are listed in Table 1.

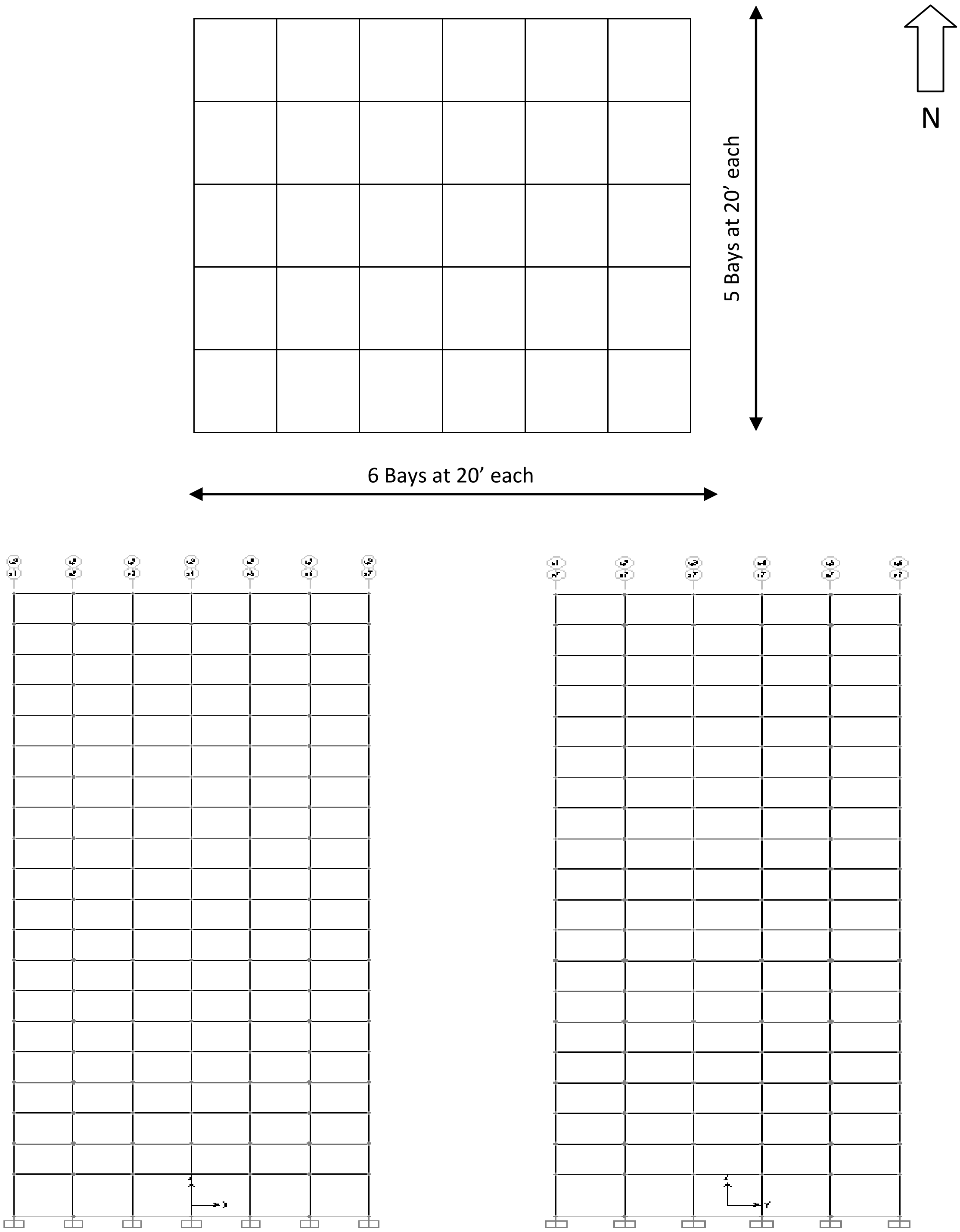


Fig. 1 : Plan and Elevation of the steel moment resisting frame

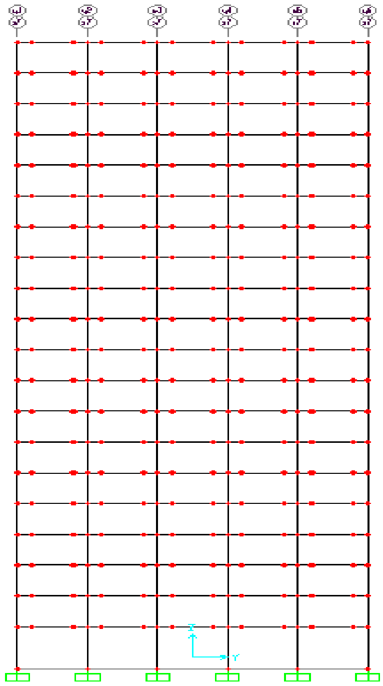
Table 1 : The design parameters of the Steel Moment Resisting Frame

Materials	Structural steel with f_y = 50 ksi
Floor Seismic Weight for Roof	645 kips
Floor Seismic Weight for Floor 2	622 kips
Floor Seismic Weight for Floor 3-20	608 kips
Seismic zone factor, Z	0.4
Importance factor, I	1
Spectral Acceleration S_a	0.36 g
Time Period T	2.99 sec
Yield drift ratio θ_y	1 %
Target drift ratio θ_u	2 %
Inelastic drift ratio $\theta_{p=}$ θ_u- θ_y	1%
Ductility factor μ_s= θ_u/ θ_y	2.0
Reduction Factor due to Ductility R_μ	2.0
Energy Modification Factor γ	0.75
Total Seismic Load W	12191 kips
Design Base shear V_y	1146 kips
V_y/W	0.094
A	0.942

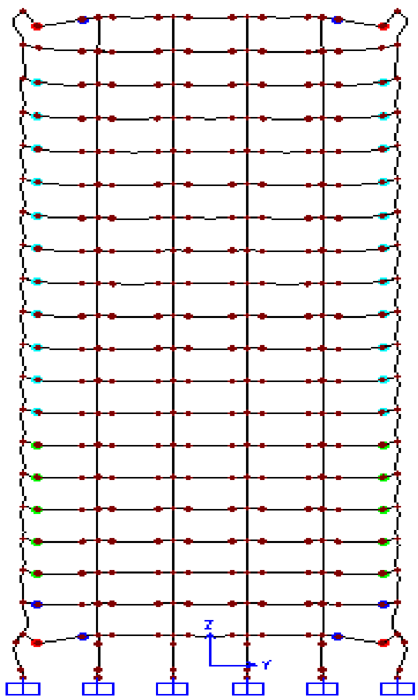
Nonlinear static (pushover) and dynamic (time-history) analyses were carried out for the steel moment resisting frame designed by both elastic design approach as well as PBPD method by using SAP2000 software. The analysis results are shown in the next section.

5. INELASTIC RESPONSE ANALYSIS OF THE STEEL MOMENT RESISTING FRAME DESIGNED USING ELASTIC DESIGN APPROACH

The steel moment resisting frame was first designed by the elastic design approach pertaining to the current UBC94 codes using the SAP2000 software. The frame was then analyzed by the nonlinear static Pushover analysis in SAP2000. In nonlinear static pushover analysis, the entire frame is carried out up to the target drift by using design lateral force distribution. Nonlinear static push over analysis was performed on this 20 storied frame by assigning the hinges at 6 inches from the column face as shown in figure 2a.



(a)



(b)

Fig. 2 : **(a) Hinges assigned in beams for applying the static Pushover Force. (b) Formation of Plastic hinges in the frame designed using elastic design approach**.

The failure mechanism of this frame obtained by SAP2000 is shown in figure 2b. The results show formation of plastic hinges in some columns of floors which may result into total collapse of the entire frame. The nonlinear Time history analysis of the frame when subjected to six different ground motions (Santa Monica, Petrolia, Lacco North 90 degrees , Lacco North 0 degrees , Corralotos, and Altedena Earthquake ground motions as shown in figure 3) was also carried out using the software. The acceleration and displacement response of this frame to these ground motions is shown in figures 4 and 5 and the hysteretic Energy dissipation curves are shown in figure 6. It could be seen in the acceleration and displacement responses of this frame that the peak values are obtained in synchronization with the ground motion. The hysteretic energy loops show that the structure remains in the elastic zone and fails before fully utilizing the capacity lying in the inelastic zone. The reason is that the columns fail first leading to the premature collapse of the structure as observed from the push over analysis.

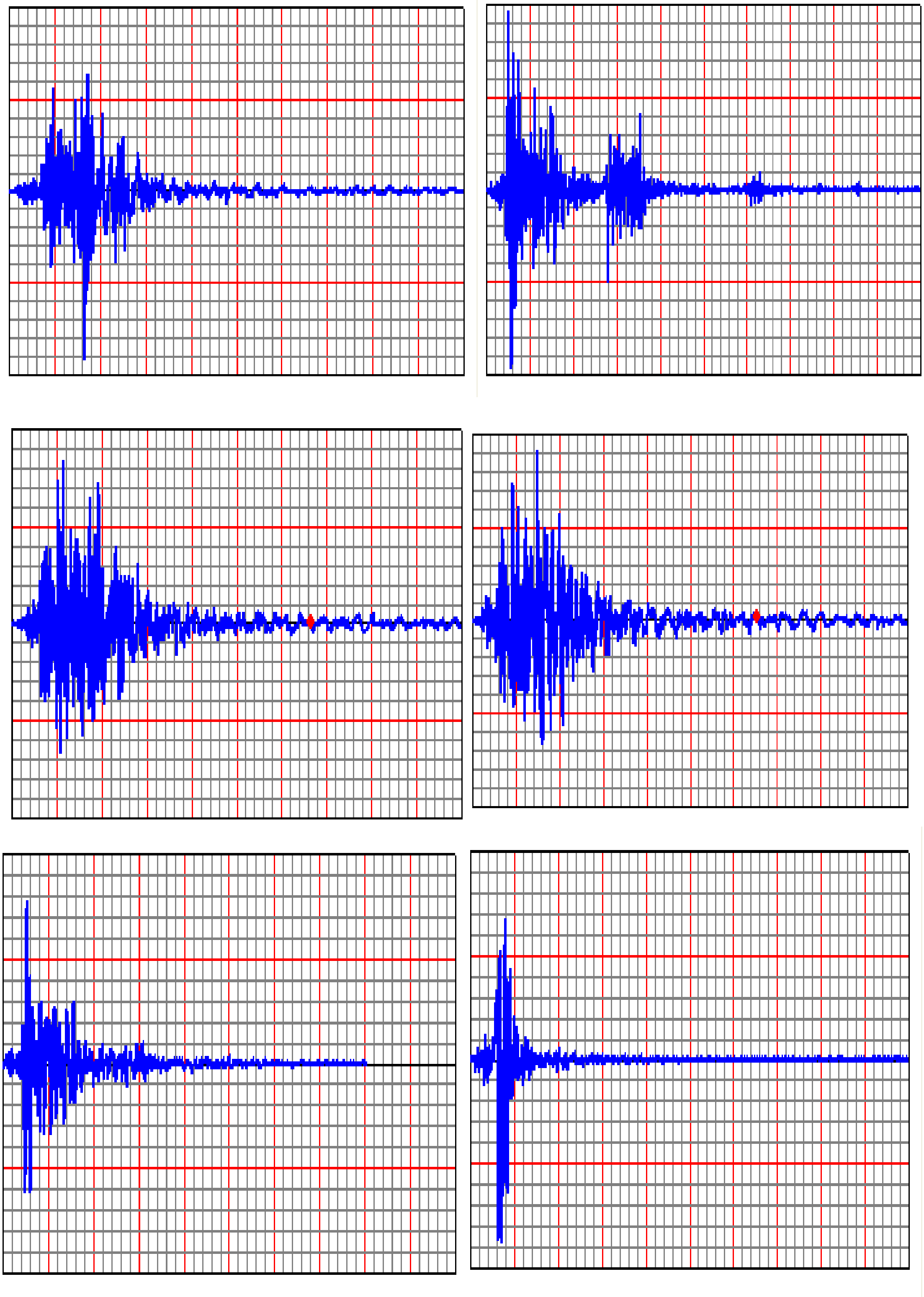


Fig. 3: The Santa Monica, Petrolia, Lacco N 90, Lacco N 0,Corralotos and Altedena Ground Motions

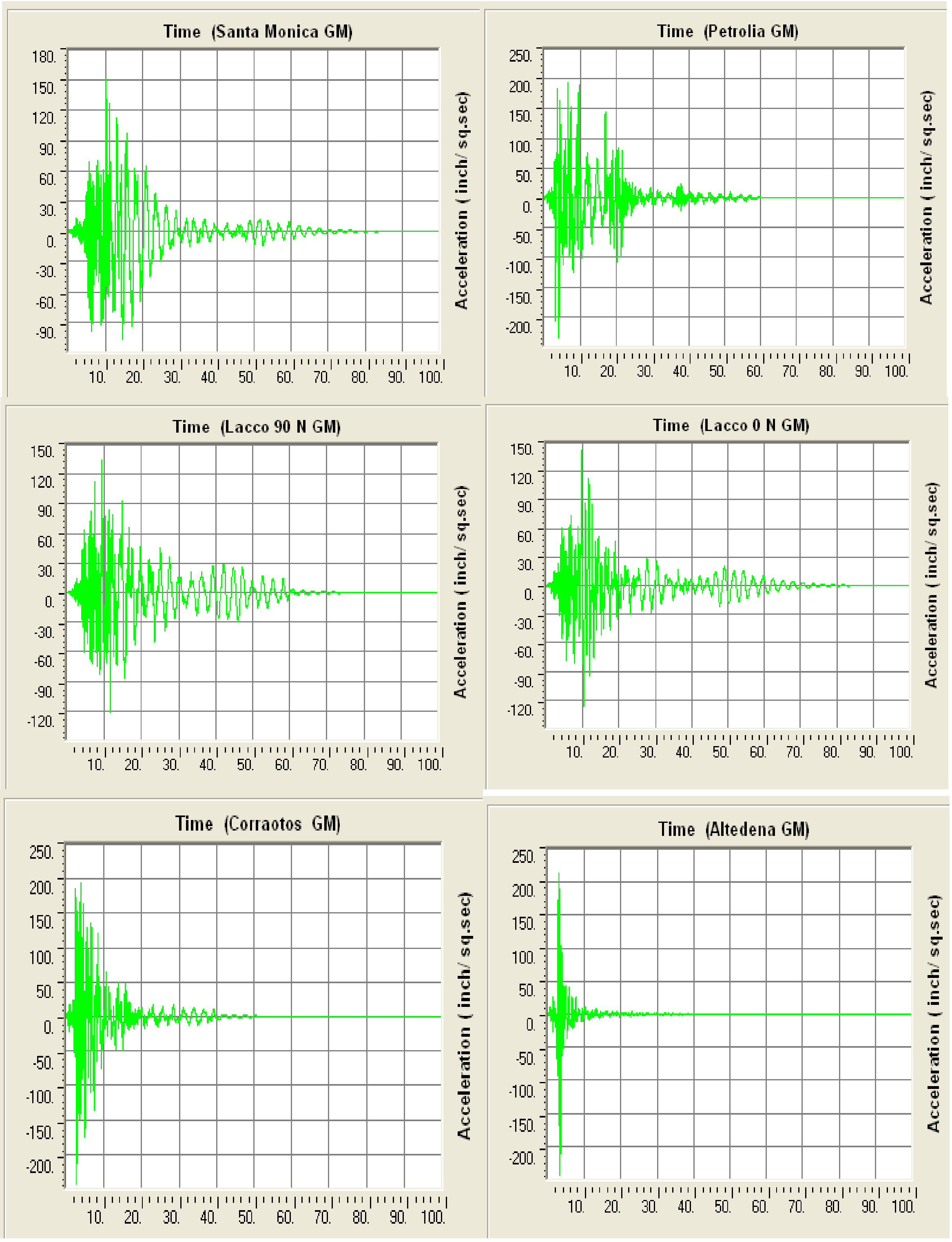


Fig. 4: The acceleration response of the frame designed using elastic design approach when subjected to different ground motions.

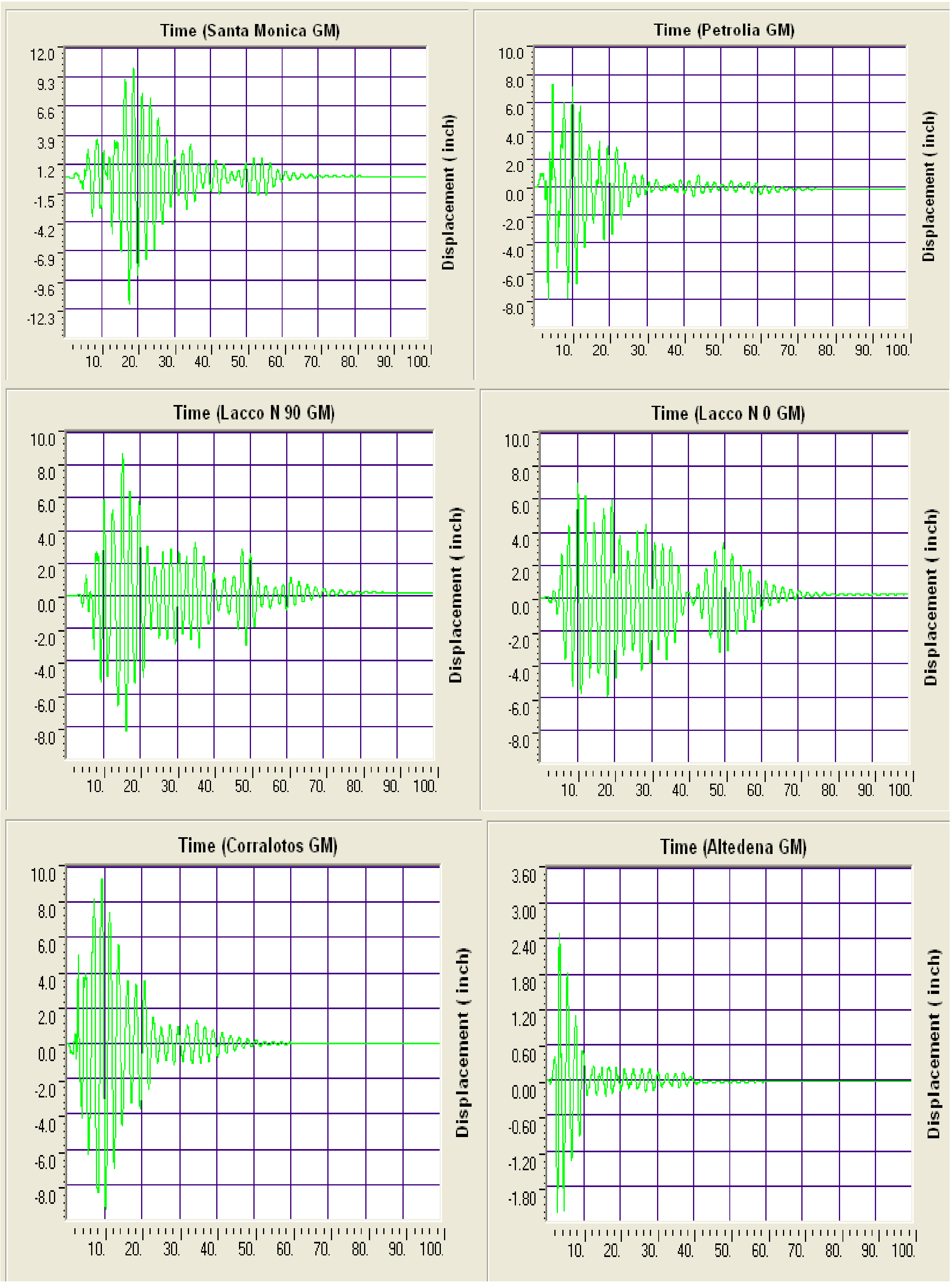


Fig. 5: The displacement response of the frame designed using elastic design approach when subjected to different ground motions.

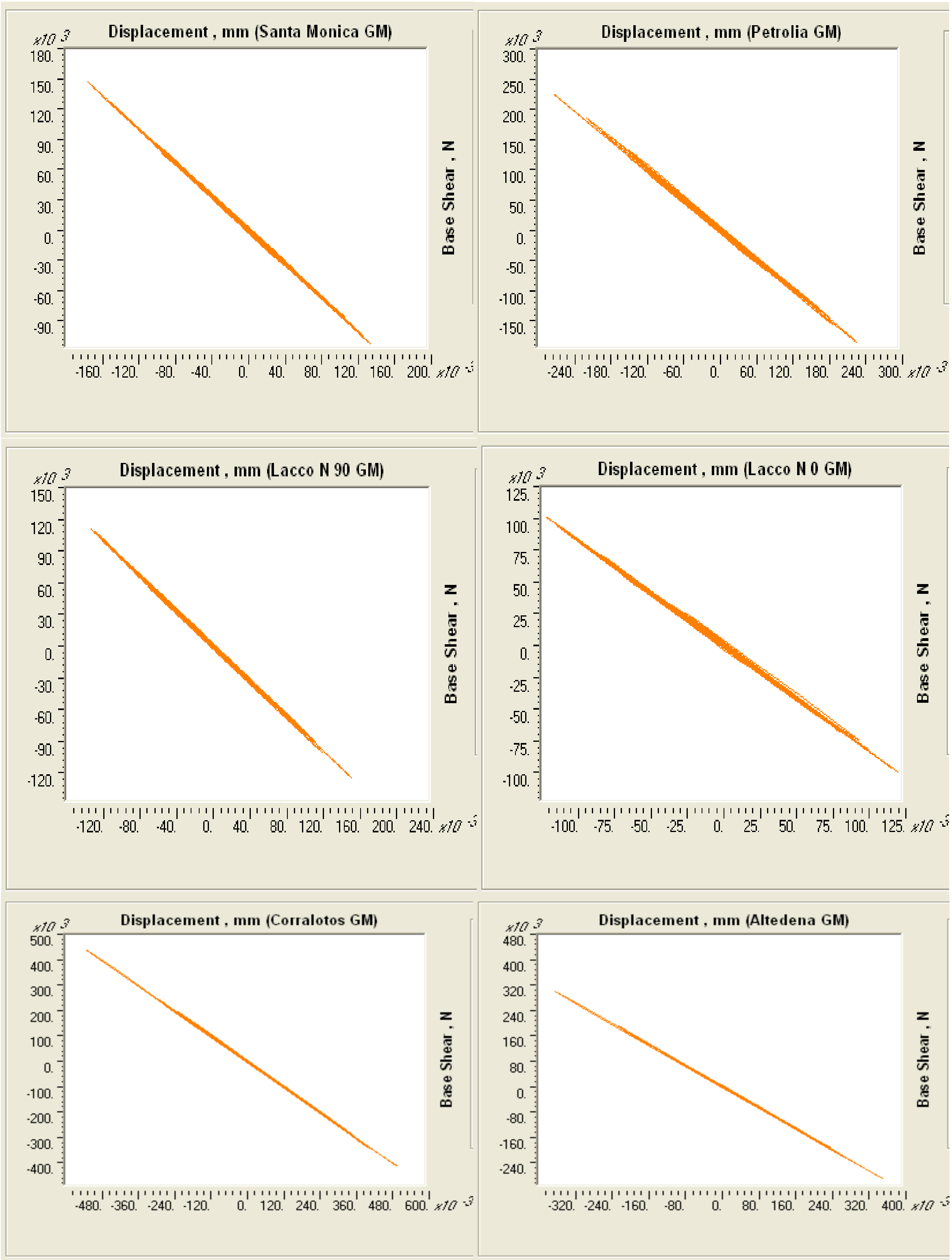


Fig. 6: The hysteretic energy dissipation of the frame designed using elastic design approach when subjected to different ground motions.

6. INELASTIC RESPONSE ANALYSIS OF THE STEEL MOMENT RESISTING FRAME DESIGNED USING PERFORMANCE BASED PLASTIC DESIGN APPROACH

In order to achieve the main goal of performance based design i.e. a desirable and predictable structural response, it is necessary to account for inelastic behavior of structures directly in the design process. Figure 7 shows the target and yield mechanism chosen for the frame while designing it using the performance based plastic design method. The hinges are to be formed at the bottom of the base column and in beams only. The beams are modeled to behave inelastically, while the columns are modeled (or 'forced') to behave elastically. P-Delta effect is captured by applying the floor gravity loads on 'gravity columns' (columns not part of the lateral force resisting frame), which can be lumped into one.

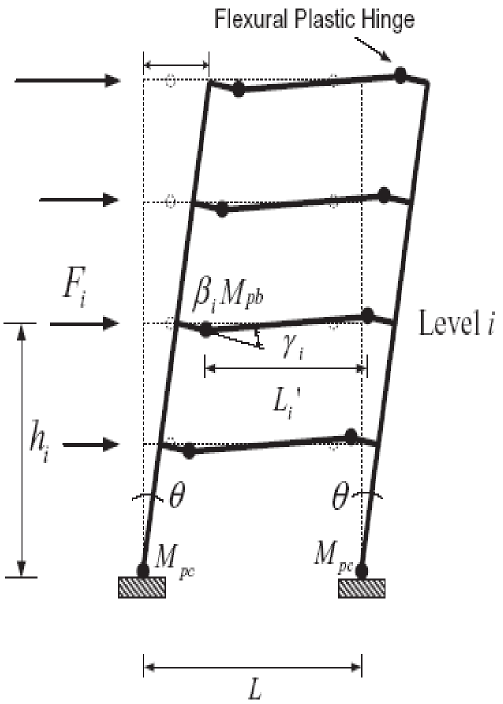


Fig. 7 : Target Yield mechanism for moment frame designed using PBPD approach.
Source: Goel et al [2]

Unlike the force distribution in the current codes, the design lateral force distribution used in the PBPD method is based on maximum story shears as observed in nonlinear Time history analysis results (Chao,2007). The design lateral force and shear distribution in the PBPD method are calculated from equations

$$\frac{V_i}{V_n} = \beta_i = \left(\frac{\sum_{j=1}^{n} w_j h_j}{w_n h_n}\right)^{0.75T^{-0.2}} V_y$$

and

$$F_i = (\beta_i - \beta_{i+1}) \times \left(\frac{w_n h_n}{\sum_{j=1}^{n} w_j h_j}\right)^{0.75T^{-0.2}} V_y$$

Where
β_i = Shear distribution factor at level i
V_i=story shear force at level i
V_n=story shear force at roof level (n^{th} level)
w_j= seismic weight at level j
h_j= height of level j from base
w_n= seismic weight at the top level
h_j= height of roof level from base
T = fundamental time period

This formula of force distribution has been found suitable for Moment Frames, Eccentrically Braced Frames, Concentrically Braced Frames and Special Truss Moment Frames.(Chao, [8]).

The current design codes obtain these lateral forces on the assumption that the structure behaves elastically and primarily in the first mode of Vibration. However, building structures designed according to these procedures undergo large deformation in the inelastic range when subjected to major earthquakes. The steel frame under this study was designed using this lateral force distribution for the PBPD method and then nonlinear static and time history analyses was carried out. In nonlinear static pushover analysis, the entire frame is carried out up to the target drift by using design lateral force distribution and thus the failure caused is shown in figure 8.

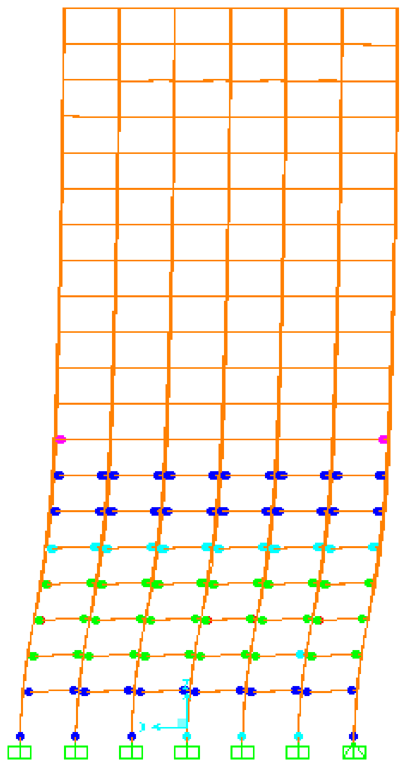


Fig. 8: Formation of Plastic hinges in the frame designed using PBPD approach.

It could be clearly seen in figure 8 that hinges are formed in beams only and the bottom of base columns which converts the whole structure into a mechanism and avoids the total collapse.

The nonlinear time history analysis of the PBPD frame shows a considerable increase in the acceleration and displacement responses as shown in figure 9 and 10 as compared to the frame designed using elastic design approach which leads to a higher hysteretic energy dissipation. The increased hysteretic energy dissipation of the frame indicates that the structure utilizes its capacity lying in the inelastic zone. The reason is that the PBPD method is based on the "strong column weak beam" concept and the beams fail first. As the structure turns into a mechanism due to formation in hinges in beams (2 in each beam) and bottom of the base columns, it undergoes large deformation before failure.

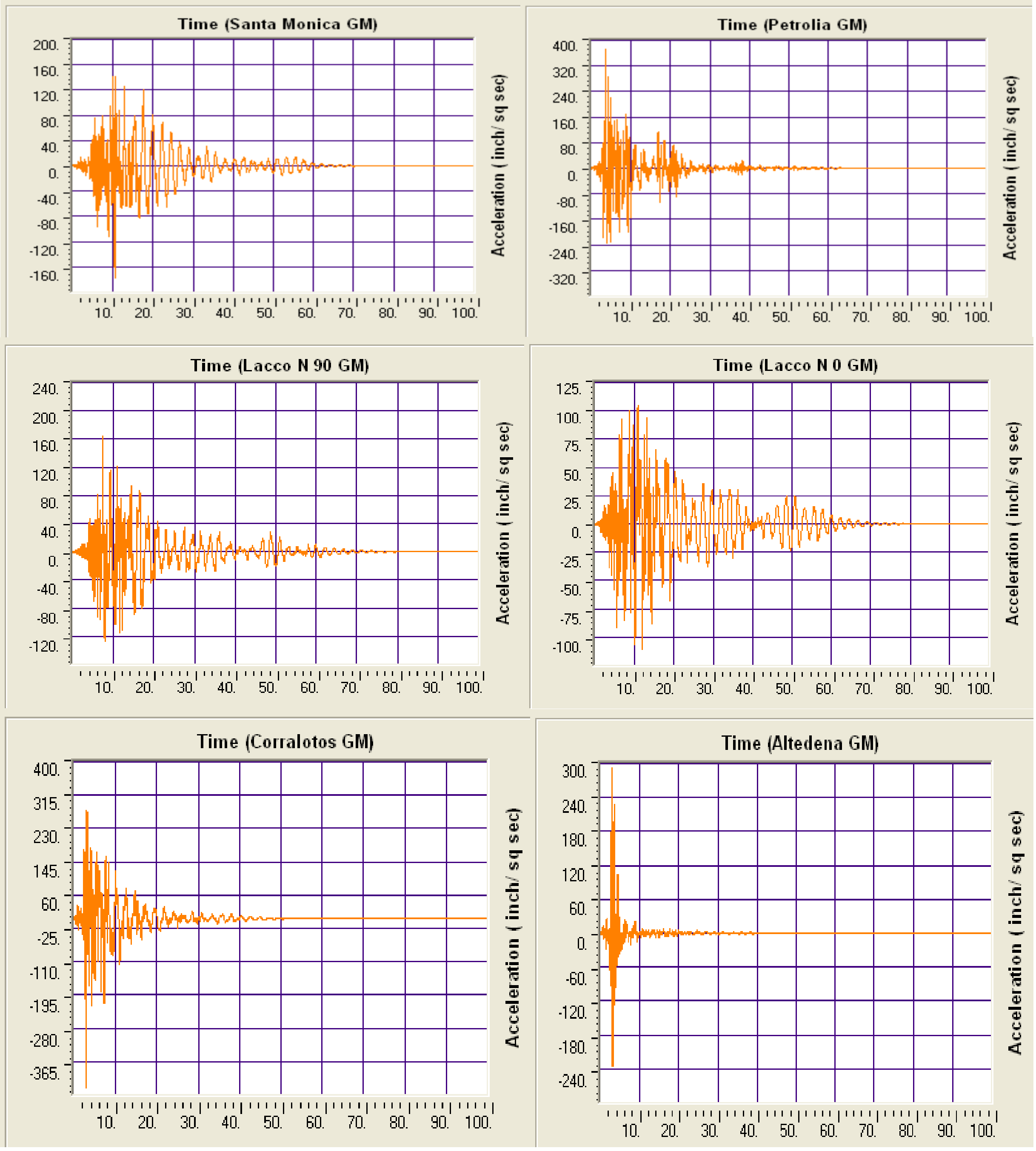


Fig. 9: The acceleration response of the frame designed using PBPD approach when subjected to different ground motions.

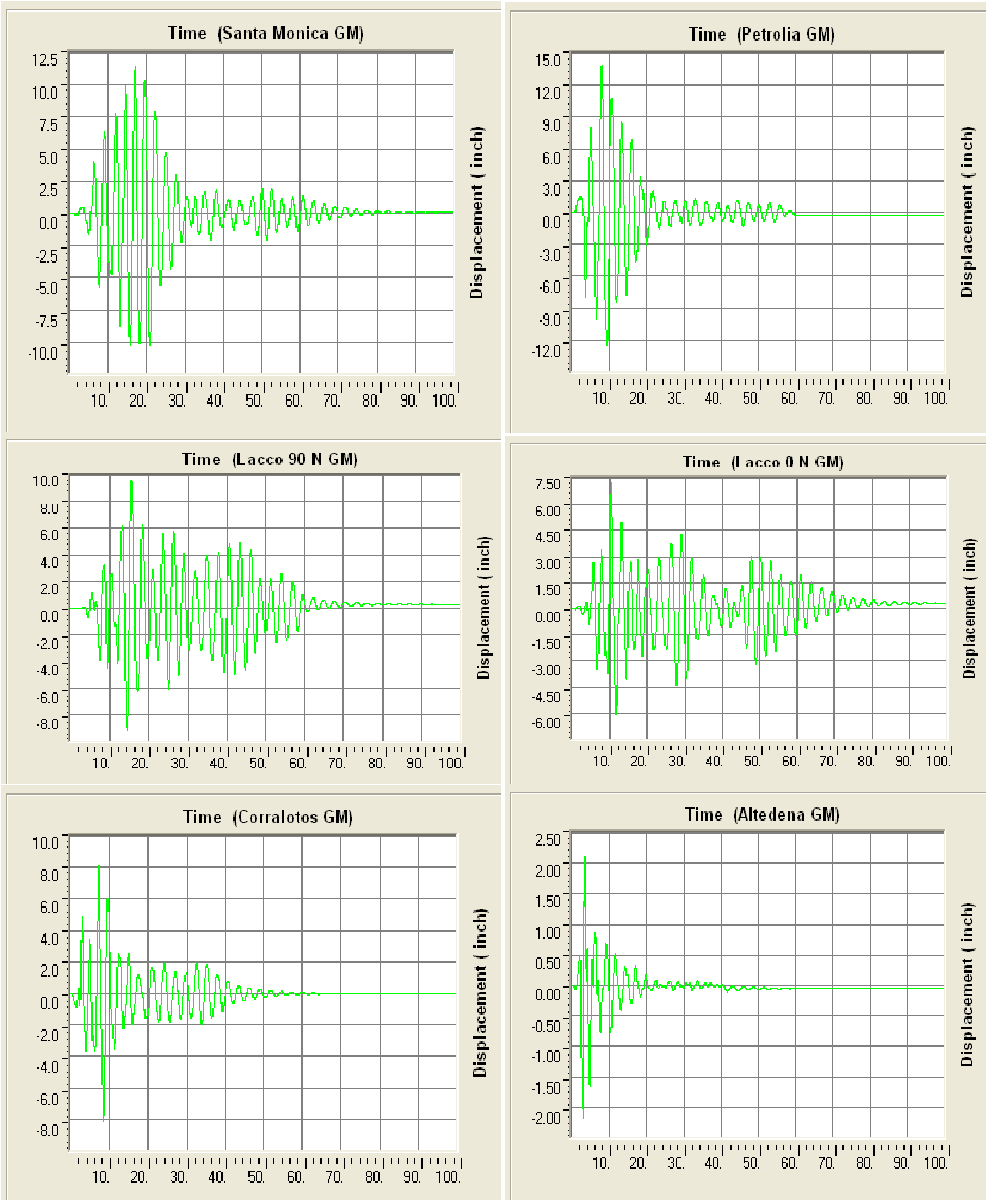


Fig. 10: The displacement response of the frame designed using PBPD approach when subjected to different ground motions.

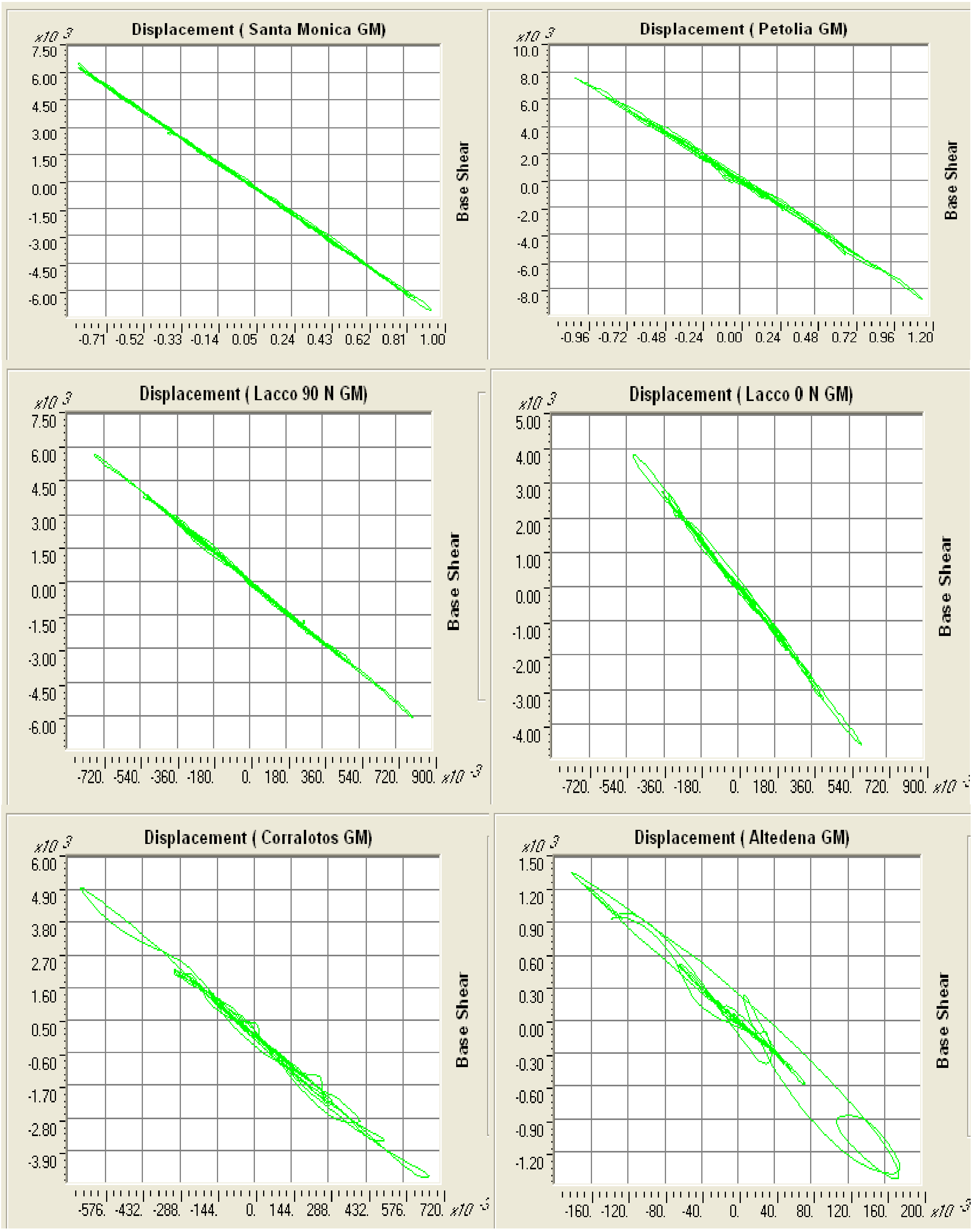


Fig. 11: The hysteretic energy dissipation of the frame designed using PBPD approach when subjected to different ground motions.

7. RESULTS AND CONCLUSION

Inelastic static and dynamic analyses of the steel frame when designed using elastic design methodology and performance based plastic design methodology were carried out for six different ground motions using SAP2000 software. The results showed very good behavior of the PBPD frame under static pushover loads. No unexpected plastic hinging was observed in the columns of the PBPD frame. The hinges are formed in beams only and the bottom of base columns which converts the whole structure into a mechanism and avoids the total collapse. Although these ground motions caused large displacements in the PBPD frame, the structure did not lose stability. Also, the increased hysteretic energy dissipation of the frame indicates that the structure utilizes its capacity lying in the inelastic zone. It can be thus concluded that the PBPD method is superior to the elastic design method in terms of the optimum capacity utilization.

List of Abbreviations

FEMA	Federal Emergency Management Agency
PBPD	Performance Based Plastic Design
SEAOC	Structural Engineers Association of California
UBC	Uniform Building Code

List of Figures

8.REFERENCES

[1] Dalal S P , Vasanwala S V , Desai A K :2011 : “Performance based seismic design of Structures: A Review” International Journal Of Civil and structural Engineering ISSN No. 0976-4399 Volume 1 No. 4 – 2011 pp 795-803.

[2] Lee SS, Goel SC. 2001. “Performance-Based design of steel moment frames using target drift and yield mechanism.” Research Report no. UMCEE 01-17, Dept. of Civil and Environmental Engineering, University of Michigan, Ann Arbor, MI.

[3] Chao SH, Goel SC. 2006a. Performance-based design of eccentrically braced frames using target drift and yield mechanism. AISC Engineering Journal Third quarter: 173–200.

[4] Chao SH, Goel SC. 2006b. A seismic design method for steel concentric braced frames (CBF) for enhanced performance. In Proceedings of Fourth International Conference on Earthquake Engineering, Taipei, Taiwan, 12–13 October, Paper No. 227

[5] Chao SH, Goel SC. 2008. Performance-based plastic design of seismic resistant special truss moment frames. AISC Engineering Journal Second quarter: 127–150.

[6] Dasgupta P, Goel SC, Parra-Montesinos G. 2004. Performance-based seismic design and behavior of a composite buckling restrained braced frame (BRBF). In Proceedings of Thirteenth World Conference on EarthquakeEngineering, Vancouver, Canada, 1–6 August 2004, Paper No. 497.

[7] Computer and Structures (2000). SAP2000 Nonlinear Version 8, Berkeley, USA.

[8] Chao SH, Goel SC, Lee S-S. 2007. A seismic design lateral force distribution based on inelastic state of structures.Earthquake Spectra 23: 3, 547–569.