Performance Based Seismic Design of Steel Structure Using Non-Linear Dynamic Analysis

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Abstract. In the recent decades, it was observed that the performance of buildings that were designed using the traditional building codes were not satisfactory under seismic effects. There was increased demand for buildings that would not only provide life safety under the design earthquake and prevent collapse under major earthquakes, but also include other performance objectives such as the time taken for the building to be occupied post-earthquake, cost of repair and so on. This resulted in the research and development of a performance based seismic design that would incorporate these demands in the design. This study presents a performance based seismic design of a 5-storey steel special moment resisting frame building by using non-linear dynamic analysis to meet multiple performance objectives. The seismic performance was evaluated based on the confidence level, which indicates the ability of the building to meet the target performance objective, determined for the inter-storey drift parameter, using the guidelines given in FEMA-350. The results show that the designed building satisfies the minimum confidence level requirement for each performance objective. The study concludes that performance based seismic design proves to be a reliable method for design of buildings that provide enhanced seismic performance.

Keywords: Performance based seismic design, Non-linear dynamic analysis, Bi-directional earthquake

1. Introduction

Several earthquakes that took place over the past few decades namely the Northridge earthquake which took place in the US in 1994 and the Kobe earthquake that took place in Japan in 1995, brought to surface the inability of the current building codes in predicting the behavior of buildings under seismic effects [1]. Buildings designed using the building codes are expected to provide life safety under design earthquake, without damage under minor earthquakes and prevent collapse under severe earthquakes. This however does not guarantee that the building would meet each of those criteria when subjected to seismic effects as the code does not explicitly include design for multiple limit states. Although buildings do survive the earthquakes, the economic losses because of the earthquakes, are unprecedented. FEMA 424 however mentions that this does not qualify buildings designed and built using the building codes as unsafe, instead the requirements in the building codes should be taken only as a minimal requirement for protecting public safety, and general welfare. There may be times when it will be beneficial or required to go beyond minimum safety requirements prescribed by codes [2]. Owing to these requirements, research for a more efficient design method that would tackle the shortcomings of the building codes and also incorporate the cost of damages, began, leading to the development of Performance Based Seismic Design (PBSD) procedure.
Several PBSD approaches were proposed such as Yield Point Spectra Method (Aschhen and Black, 2000), Modified Lateral force procedure (Englekirk, 200; Panagiotou and Restrepo, 2007), Deformation Controlled design (Panagiotakoes and Fardis M.N, 1999), Direct Displacement Based Design (DDBD) (Priestley et al., 2003-2007) and Performance Based Plastic Design (PBPD) (Goel and Chopra 2007) [3]. Among these methods, the DDBD and PBPD are the most developed methods, although they are still being improved. These PBSD approaches aim to provide feasible steps for performance-based design of buildings by incorporating maximum design features while also avoiding the need for iteration for design.

Several studies were carried out in an effort to compare between PBSD and Force Based Design (FBD) methods to weigh the pros and cons of PBSD against FBD in terms of reliability, feasibility, economy, etc. Muljati et al [4] concluded that DDBD was superior in comparison to FBD in predicting the seismic demand of structure. Sutariya and Shah [5], Gamit and Amin [6] and Singh and Raju [7], all concluded that DDBD proved to be more economical than FBD. Vivinkumar R. V. and Karthiga S. [8] found that DDBD structure showed better performance as compared to FBD structures. Dubal et al [9] observed that PBSD method significantly enhanced the performance point of frames as compared to FBD method for all cases.

Studies have also been done on PBSD of various types of buildings configurations, height, lateral force resisting systems etc. Fox et al [10] designed a 7-storey building in New Zealand, with RC coupled walls using PBSD. Karthiga et al [11] compared FBD and DDBD of two-dimensional RC frames having mass irregularity as well as geometric irregularity in vertical direction. Dubal et al [9] analysed a 10-storey RC moment resisting frame having vertical geometric irregularity and soft storey using PBSD as well as FBD. Khedkar et al [12] studied the RC moment resistant frame having vertical setback using PBSD and FBD method as per IS 1893: 2002. Ccahuana et al [13] examined the seismic responses and damping parameters of a dual concrete building and a CBF frame using DDBD procedure to compare their base load and displacements.

Besides RC structures, various studies were conducted on steel structures as well. Lee et al [14] proposed a PBSD procedure to design SMRFs by using pre-selected yield mechanism and target drift and concluded that the proposed method was able to produce structures that would meet the pre-selected performance objectives. Zhang et al [15] put forward a PBPD methodology, which is a PBSD method, for designing high rise steel frames and concluded that PBPD frames displayed excellent seismic performance. T. J. Sullivan [16] formulated a DDBD procedure for eccentrically braced frames (EBFs) and concluded that the proposed methodology was promising. Malekpour et al [17] assessed the DDBD procedure for regular steel moment resisting frames and concluded that DDBD can be considered as an alternative to the current FBD methods. Wijesundara and Rajeev [18] proposed a DDBD procedure for steel CBFs by utilizing the yield displacement profile and concluded that the linear displacement shape proposed by Priestley et al [19] for low MRFs was substantially valid for low-rise CBF structures.

Although substantial efforts have been made towards PBSD of steel buildings, there are still lesser research works available as compared to reinforced concrete structures. Therefore, this study was conducted to further understand the behaviour of steel structures designed using PBSD. The performance of buildings considering multiple limit states (e.g., collapse, immediate occupancy, life safety) and enhanced performance objectives also needs to be investigated more and hence is included in this study. Furthermore, a 3D model of the building has been considered instead of the 2D models generally used, to better incorporate the multi-directional effects during seismic activity and the
building is subjected to bi-directional earthquake effects by applying orthogonal pair of ground motions to the building, simultaneously in both directions instead of the generally used unidirectional earthquakes.

2. Overview and Design of the Building

A 5-storey special steel moment resisting frame (SMRF) office building has been designed for the study. The building has been assumed to be constructed in a region of high seismicity. The resistance to lateral seismic forces was provided by Seismic Force Resisting System (SFRS) consisting of special SMRFs arranged symmetrically at the perimeter in both the orthogonal directions, having three bays in E-W direction and four bays in N-S direction. The detailed arrangement of the SFRS has been given in figure 1.

The building considered was rectangular in plan with five bays of 28ft in E-W direction and four bays of 23ft in N-S direction, giving a total floor area of 140x92 ft². A storey height of 13ft was provided for all the levels. The building schematics has been shown in figure 1 and the typical floor plan in figure 2.

Design Loads. The dead load from self-weight of the steel frame system and concrete deck of 4.8 inch of 4000 psi was automatically calculated by the software and included in the analysis. Super imposed dead load of 650 lb/ft from façade was provided at the perimeter of each floor, except on the roof where load of 165 lb/ft from parapet has been provided at the perimeter. A finishing load of 12 lb/ft² was provided on all the
floors and load coming from the equipment and miscellaneous loads assuming to be 15 lb/ft2 was also applied on all floors. Partition wall of 250 lb/ft is applied on all the floors except the roof. Live load of 60 lb/ft2 has been provided on all the floors with a live load of 30 lb/ft2 on the roof.

Seismic Parameters. The SH provided in ASCE 7 is based on a risk-targeted design philosophy. It is defined as ground motions having 1% probability of causing total or partial structural collapse of an appropriately designed structure in 50 years. This ground motion intensity is designated in the code as MCER [20]. The SH used for the design have been given below.

- Building Risk Category: II
  ASCE 7 Table 1.5-1
- Soil site class: D, Stiff soil
  ASCE 7 Table 20.3-1
- Spectral response acceleration parameters:
  \( S_s = 1.5g \)
  \( S_1 = 0.67g \)
  \( S_{DS} = 1g \)
  \( S_{D1} = 0.6g \)
- Seismic Design Category: D
  ASCE 7 Table 11.6-1
  ASCE 7 Table 11.6-2

A preliminary design was processed using Equivalent Lateral Force method as per ASCE 7-16 in SAP2000. The building was then analysed using Non-Linear Dynamic Procedure in order to enhance the design to meet the acceptance criteria. The building’s performance has been evaluated as per FEMA-350. The guideline provides a formula for determining a level of confidence in the ability of the building to meet the desired performance objectives. FEMA-350 mentions three parameters, namely...
inter-storey drift ratio (IDR), column axial load and column splice tension that require evaluation of confidence. However, in this current report, the building was designed as per the confidence level determined by only using the IDR parameter.

**Ground Motion Data.** A set of 12 ground motion records were selected comprising of two horizontal components for each record. The ground motion characteristics have been provided in table 1.

A graph showing the target response spectrum for BSE-2 Seismic Hazard Level (SHL) along with the response spectra of the selected ground motions and their corresponding average spectrum has been shown in figure 3.

The performance objective selected is Basic Safety Objective characterized by Collapse Prevention (CP) for Risk Targeted Maximum Considered Earthquake (MCER) with probability of exceedance (POE) of 2% in 50 years, Life Safety (LS) for Design Basis Earthquake (DBE) with POE of 10% in 50 years and Immediate Occupancy (IO) for Serviceability Earthquake (SE) with POE of 20% in 50 years.

![Fig. 3. BSE-2 Response Spectra](image)
### Table 1. Ground motion data

| No | RS N | Event Name       | Station Name         | Scale Factor | Magnitude | Year  |
|----|------|------------------|----------------------|--------------|-----------|-------|
| 1  | 12   | Kern County      | LA-Hollywood Stor FF | 9.46         | 7.36      | 1952  |
| 2  | 22   | El Alamo         | El Centro Array #9   | 12.5         | 6.8       | 1956  |
| 3  | 138  | Tabas_Iran       | Boshrooyeh           | 5.26         | 7.35      | 1978  |
| 4  | 293  | Irpinia_Illay-01 | Torre Del Greco      | 11.1         | 6.9       | 1980  |
| 5  | 570  | Taiwan SMART1(45)| SMART1 C00           | 3.32         | 7.3       | 1986  |
| 6  | 573  | Taiwan SMART1(45)| SMART1 l01           | 3.33         | 7.3       | 1986  |
| 7  | 577  | Taiwan SMART1(45)| SMART1 O01           | 3.53         | 7.3       | 1986  |
| 8  | 581  | Taiwan SMART1(45)| SMART1 O07           | 4.01         | 7.3       | 1986  |
| 9  | 746  | Loma Prieta      | Bear Valley #5_Callens Ranch | 7.84 | 6.93 | 1989 |
| 10 | 775  | Loma Prieta      | Hollister-SAGO Vault | 11.2         | 6.93      | 1989  |
| 11 | 778  | Loma Prieta      | Hollister Differential Array | 2.17 | 6.93 | 1989  |
| 12 | 787  | Loma Prieta      | Palo Alto-SLAC Lab   | 2.65         | 6.93      | 1989  |

### 3. Results and Discussion

The results presented here are the average of the maximum response from all the 12 ground motion sets.
3.1 Inter-storey Drift Ratio

A summary of the mean IDRs for all the performance objectives have been shown in Table 2. Drift limits as per FEMA-273 are also provided. However, it should be noted that these drift limits were prescribed for steel moment resisting frames without consideration of the gravity frames.

| Performance Level | X    | Y    | Drift Limit |
|-------------------|------|------|-------------|
| CP                | 0.033| 0.032| 0.050       |
| LS                | 0.025| 0.024| 0.025       |
| IO                | 0.014| 0.014| 0.007       |

For CP, the IDRs were almost similar in both the orthogonal directions with maximum IDR of 0.033 and 0.032 occurring at the 3rd storey in E-W and N-S directions respectively. The maximum drift ratio limit of 0.05 given in FEMA-273 has been satisfied i.e., the drift ratio obtained was much lesser than the maximum limit of 0.05.

In the case of LS, the maximum drift limit provided by FEMA-273 is 0.025 for steel moment frames to be within the LS performance level. The result from the analysis showed the maximum IDR to be 0.025 and 0.024 for E-W and N-S direction respectively. The IDRs were at the edge of the drift ratio limit for LS performance level unlike the conservative result shown in the CP performance level.

![Fig. 4. Summary of Mean Inter-Storey Drift Ratio](image-url)
For IO performance level, FEMA-273 specifies a drift limit of 0.007 for steel moment frames. The result from analysis showed the maximum IDR 0.014 for frames in E-W and N-S directions. This showed that the drift ratios obtained for IO performance level was twice as much from the drift ratio limit specified by FEMA-273 and hence did not satisfy the criteria.

A graph showing the summary of the mean of maximum IDRs for all performance levels has been shown in figure 4.

3.2 Confidence Level

The confidence level obtained and the minimum confidence level requirement as per FEMA-350 for each performance level has been provided in table 3. The acceptance criteria for LS are not provided in FEMA-350. However, for this study, the values were interpolated and evaluated for verification (as was suggested in FEMA-350 commentary).

| Performance Level | X     | Y     | Minimum Required |
|-------------------|-------|-------|-----------------|
| CP                | >>99% | >>99% | 90%             |
| LS                | 97.77%| 98.38%| 70%             |
| IO                | 52%   | 52%   | 50%             |

From table 3, it has been observed that the minimum confidence level requirement as per FEMA-350 was satisfied for all the performance levels.

Based on the acceptance criteria for IDR parameter as per FEMA-273 and FEMA-350, the results obtained showed conservative behaviour for CP performance level. The results obtained for LS just reached the drift limit provided by FEMA-273 but was conservative as per FEMA-350. However, the minimum confidence level for LS has been interpolated for this study and hence may not be accurate. In case of IO performance level, the IDR obtained did not satisfy the IDR limit given by FEMA-273 and it only nearly satisfied the minimum confidence level requirement provided by FEMA-350. Hence it can be seen that the ability of the building to satisfy the IO performance objective posed the highest risk and so it proved to be the governing criteria for design in this study.

4. Conclusion

This study has provided a PBSD of a steel SMRF office building with its seismic performance evaluated based on IDR parameter by using a reliability-based acceptance criteria in the form of confidence levels given by FEMA-350. The design has been verified successfully with the building’s performance meeting the minimum confidence level requirements. The use of NDP, though tedious, has been proven to be the most accurate method and hence used in many experimental studies for establishing the benchmark of structure’s response.

In conclusion, performance based seismic design has been successfully evaluated in this study and has proven to be a reliable method for design, providing enhanced seismic performance at multiple limit states.
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