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Performance-Based Design of an Irregular RC Structure with and without Supplemental Damping

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Abstract

Many countries are undertaking changes in earthquake design codes, the main reason for it is that the existing building codes do not specify the performance criteria of individual structural elements under various levels of shaking. Even though some buildings performed well in life safety aspects, many of those failed in safeguarding the building from high extent of structural damage under severe shaking. Retrofitting had also proved to be uneconomical for the buildings to perform better under severe shaking. In the current earthquake codes, the displacements and forces are within the elastic limit i.e., the assumed behaviour is linear. But the response of a structure to the major earthquakes is not elastic. There is a high chance of yielding and the development of plastic hinges in the members. Therefore, it is required to perform the non-linear analysis for assessing the inelastic behaviour of the structure. In the present study non-linear static analysis was adopted to assess the seismic performance of an irregular building in plan with re-entrant corners using the capacity spectrum method. If either the drift limit of the structure or the response of the hinges is exceeding the desired level of performance, the structure is fed with supplemental damping to ensure the required performance. It was observed that the formation of hinges crossed life safety level and entered the collapse prevention state. As the response exceeded the desired level of performance a methodology to calculate the required supplemental damping for the structure is proposed. Depending on the level of safety of the structure, an appropriate supplemental damping value can be obtained from the proposed performance plots.

Keywords: Seismic performance, performance-based design, pushover, FEMA, ATC 40, R-FBI base isolator, Supplemental damping

1. Introduction

Earthquake resistant design based on traditional approaches aim to attain the prescribed limits on strength and serviceability criteria as per code provisions. Even after practicing those design practices, earthquakes incurred catastrophic damages to structures and led to huge loss of life and economy. After Loma Prieta and Northridge earthquakes that occurred in 1989 and 1994 respectively, many structural design engineers in United States started working on development of procedures giving importance to performance rather to strength.

The commitment of Performance based design in earthquake engineering is to build structures of which the seismic performance is attained as desired. Potential loss of occupancy, repair costs and life safety impacts come under the desired seismic performance. In this type of design, the decision maker- may be owner or tenant or a developer chooses the structural performance required as per his interest. The engineer uses his skills and provide a design which is capable of satisfying the decision maker requirements. Combined effort of professionals and designers is required to make this commitment into reality. Because of the advancements that took place recently in assessing seismic hazard, facilities for performing experiments and computer applications, many design engineers and developers in earthquake prone regions are attracted to Performance based seismic design. Hopefully we can say that with in a very short period of time PBSD becomes a conventional method for the design and construction of structures resistant to earthquakes. To utilize PBSD effectively we must be aware of uncertainties included in structural performance and estimation of seismic hazard.

- Important terms in PBSD
  a. Performance level: It is a maximum acceptable damage state or condition caused by physical damage within a building, the threat to building occupants life safety due to damage and serviceability of the structure post-earthquake.
  b. Seismic hazard: shaking or ground motion level at site for a given earthquake. Seismic hazard has three standard levels.

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- Serviceability earthquake, which has a probability of 50% to be exceeded in 50 years.
- Design earthquake, which has a probability of 10% to be exceeded in the 50 years.
- Maximum earthquake, which has a probability of 5% to be exceeded in 50 years.

c. Performance objective: It is the desired performance level for a defined seismic hazard
d. Capacity: It is the expected ultimate strength of the component in a structure without using any reduction factors that are used in the design
e. Demand: Unlike in linear elastic analysis where demand means the lateral force applied to the structure, in non-linear static analysis it means the deformations or displacements estimation that the structure may undergo.
f. Capacity curve: This is also called as pushover curve. It is the curve drawn between base shear acting on the structure and the corresponding roof displacement of the structure.
g. Capacity spectrum: The capacity curve which is drawn between base shear and roof displacement is transformed to spectral acceleration and spectral displacement.
h. Demand spectrum: It is the reduced response spectrum taking inelastic behavior of the structure into account to represent the seismic motion.
i. Capacity spectrum method (CSM): It is a non-linear static analysis method in which the expected performance of the structure is shown graphically by the intersection of capacity spectrum with the demand spectrum. The intersection point is called performance point. The obtained displacement coordinate of the performance point is the displacement demand of the structure estimated for a particular level of seismic hazard.

➢ Performance based seismic evaluation procedures

There are many Performance based seismic design procedures. Some of them are:

a. Applied Technology Council (ATC 40)- CSM
b. Federal Emergency Management Agency (FEMA 440)-CSM
c. Federal Emergency Management Agency (FEMA 356)-DCM
d. American Society of Civil Engineers (ASCE 41)-DCM

In this study ATC 40 procedure is used for the performance evaluation.

➢ Performance objectives selection

As stated above, Performance objective = Performance level + Seismic hazard
Building Performance level = Structural + Non-structural performance level
There are four performance levels that are commonly used

- Operational: Damage caused to the structural building is very limited. Repairs required are minor such that functionality of the building is not affected. Services like communication, utility, transportation are provided with backup system.
- Immediate occupancy(IO): Most commonly used performance level. The building system and the spaces are expected to be considerably used. No backup is provided.
- Life safety(LS): Threat to safety of life is of low probability. The threat may be due to damage of structural system or may be tipping, falling of non-structural components.
- Structural stability(CP): Stability under vertical loads is only left. No stability to address the aftershocks. Threat to life due to structural and non-structural elements may occur.

Junwon Seo *et al* (2015) concluded that there is discrepancy of the mass participation factor due to presence of shear wall and they also mentioned that first and third modes have dominant modal mass in longitudinal as well as transverse directions. Response spectra analysis provided 28.1% and 54.0% lesser values in maximum displacements and drift ratios compared to those for the Nonlinear time history analysis. They found that fragility of floor level mostly reduced for all FEMA performance level and the performance level of this structure falls under IO. Pawaar *et al* (2015) considered four different models like Bare Frame Modelled with Diaphragm discontinuity plan and E-shape Plan, Diaphragm discontinuity plan and E-shape plan modelled with flat slabs, shear walls (at corners) and flat slabs, shear walls (at re-entrant corners) respectively. It was observed that storey drifts are lesser in flat slab and shear wall. E shaped plan having shear walls at re-entrant corners and flat slab gave better results for point displacement. Overall performance is better for E shape plan model with shear wall according to this literature. Folić (2015) reviewed various codes and literatures associated with Performance based seismic design (PBSD) of reinforced concrete buildings. The purpose of this study is to assess the performance levels described in different codes across the globe. A comparison was done for Indian, Japanese and other codes. Nilesh M. Kashid (2014) worked on developing the seismic analysis method that can be included into Indian code IS 1893 by following the existing methods such as FEMA method. He developed different structural models and compared the performance levels and gave different cross-sectional areas for different performance levels. Mainly he concentrated on the ductility of the structure and nonlinear behaviour of the structure. This literature provided a detailed methodology of PBSD followed in FEMA 273. Study concluded that ductility has major contribution to response reduction factor, he mentioned that the response reduction factors for performance levels have been calculated considering time period of each level. This will overcome the deficiency of taking constant reduction factor for every design period. Researcher stressed on a point that calculation of plastic rotations & hinge type formation for each performance level and their possible locations gives a useful information for adapting special confining reinforcement in members of the structure. Gayathri *et al* (2014) compared different types structural systems such as dual systems. Pushover analysis and equivalent static analysis was carried out using ETABS to evaluate natural period, lateral displacement, storey drift and base shear. It was concluded that provision of shear walls in the buildings which are in high seismic zone reduces lateral displacement and storey drift when compared to ordinary moment resisting frames. It was also concluded that the provision of shear walls in the structure will result in life safety and collapse prevention levels under strong ground motion. Dalal *et al* (2011) reviewed performance based seismic design. Performance based plastic design (PBPD) which is also called as nonlinear analysis is explained in detail in this literature. They emphasised the performance-based design in the coming future.

2.Methodology

The following step by step process is adopted for the present study

1. G+8 RC building which is irregular in plan having Re-entrant corners confirming to clause 7.1 of IS 1893(Part 1) :2016 is modelled having floor height of 3.2 m.
2. Details of structural elements
    a. Beam- 0.23 X 0.45m
    b. Column-0.45 X 0.45m
    c. Slab-0.125m
3. Desired performance objectives are selected. Preliminary design is done to the building for linear analysis which is subjected to dead load, imposed load and seismic load.
4. The oscillation type in fundamental mode is verified by modal analysis.
5. The loads are assigned to structural elements as per IS 875 and earthquake loads as per IS 1893-2016.
6. Details of gravity loading
    - Dead (Wall) load-8KN/m and 4KN/m
    - Dead (Floor finishes) load-1KN/m^2
    - Live load-3KN/m^2
7. Details of seismic loading
    - Seismic load in both x and y directions
    - Importance factor-1.2
    - Response reduction factor-5
    - Zone factor-0.24(zone- IV)
8. It is verified whether all the structural elements are sufficient enough to carry the elastic loads.
9. Next static non-linear analysis is performed, for this a separate gravity load case is added.
10. Pushover analysis is performed in both x and y directions which starts at the end of non-linear gravity analysis.
11. The pushover analysis can be performed in two ways
    a. Force method
    b. Displacement method

    In this study displacement method is used for the analysis.
12. The pushover curve obtained is converted to Acceleration-Displacement Response Spectrum format (ADRS) format and superimposed on the demand spectrum and the performance point is obtained.
13. Using the performance point displacement of the roof and the base shear at that point is obtained.
14. The global response of the building is verified by drift limits given in Table-11.2 of ATC-40.
15. The local response of the elements is also verified by plastic hinge rotation limits in Table-11.3 for beams and Table-11.4 for columns of ATC-40 whether they are within the acceptable limit or not.
16. After verifying the limits, it is confirmed whether the structure possess the desired performance level or not. If not, the structure is redesigned or the performance of the building can be enhanced by providing supplemental damping

3. Results and observations

Pushover (Nonlinear) analysis

Figure 1 shows the variation of base shear with roof displacement for the irregular RC structure chosen. The base shear obtained at maximum roof displacement of 251.47mm is 4975kN.

The entire roof displacement is split into 31 steps. The formation of 1st plastic hinge is observed in 7th step for a drift of 127.5mm and corresponding base shear of 3653.4kN as shown in Figure 2.

Formation of hinges in all the columns at base storey is observed in the step7 as shown in Figure 3. Those hinges crossed Immediate occupancy state and are represented by Blue colour. The roof displacement and base shear at this instant are 188.7mm and 4659.2kN

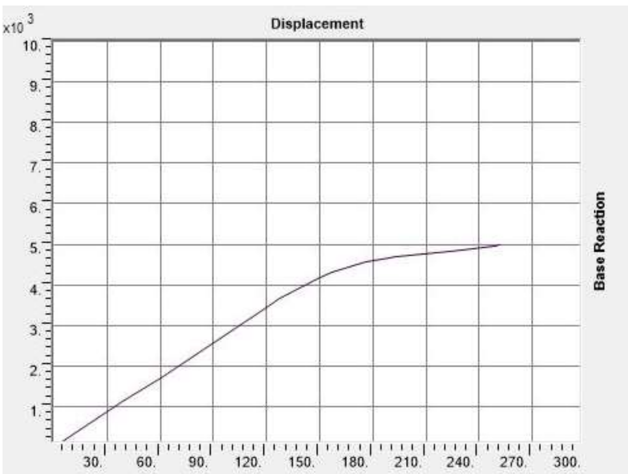

Figure 1 Base shear vs Roof Displacement

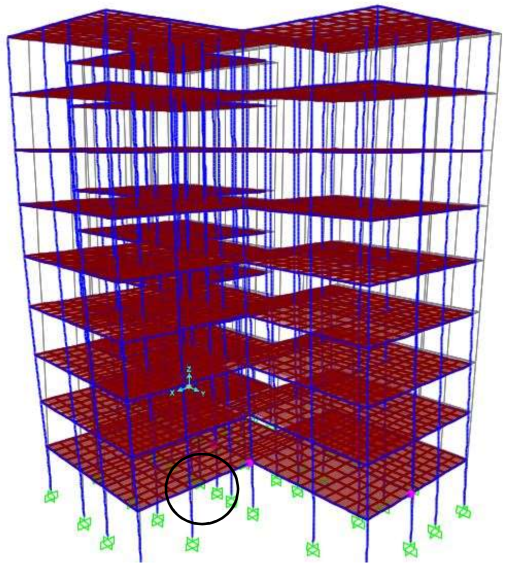

Figure 2 Formation of first plastic hinge

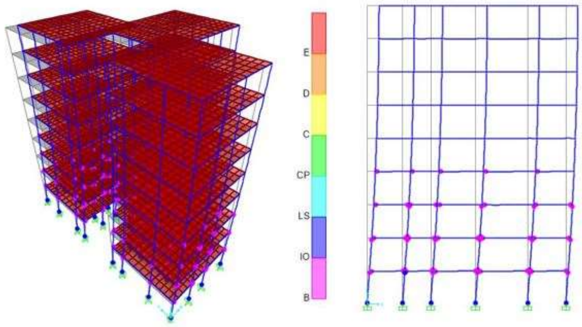

Figure 3 Formation of hinges in columns

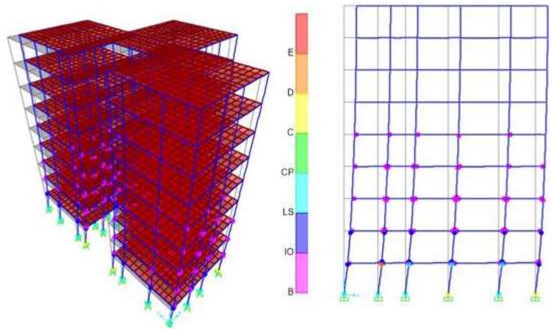

Figure 4 Formation of hinges beyond LS level

Figure 4 shows the structure at step 19 for a roof displacement of 249.3mm and base shear of 4966.3kN. It is observed that many hinges crossed life safety level and entered collapse prevention state.

Figure 5 shows the condition of the structure at maximum roof displacement i.e., 251.47mm in step 31. It is evident that many hinges are beyond collapse prevention state. So, the structure cannot take any further lateral load but stable under vertical loads.

The moment rotation curve of one of the column hinges in the base storey is shown in Figure 6. It can be seen from the highlighted circle that the rotation of the hinge exceeded collapse prevention limit. Limits as per ATC 40 code are given in table 1.

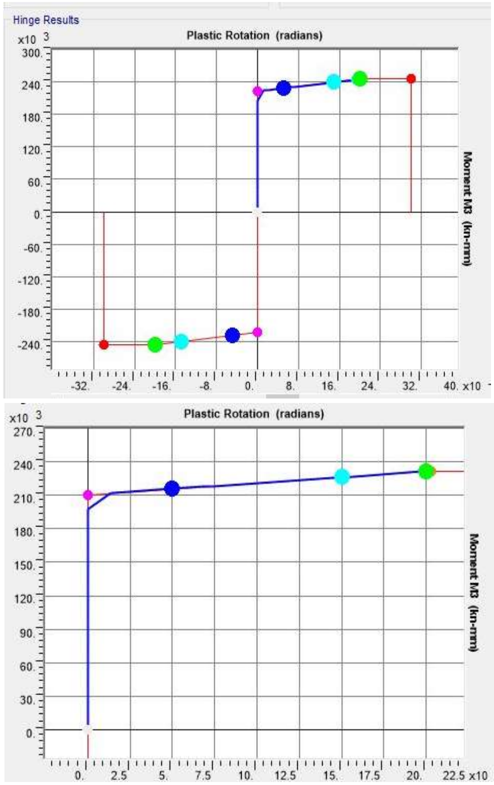

Figure 6 Moment rotation curves

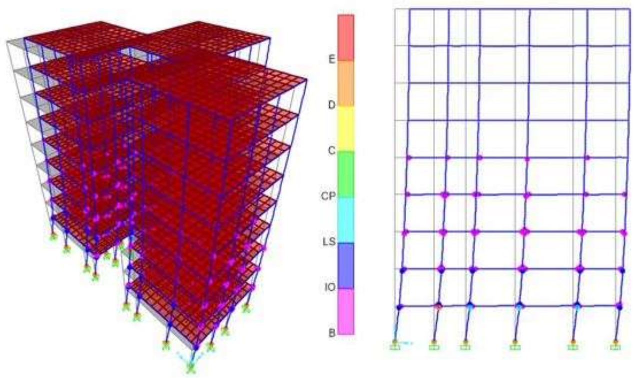

Figure 5 Formation of hinges beyond CP level

Table 1 Plastic Hinge rotational limits as per ATC 40

Performanc level	Rotation limits
Immediate occupancy (IO)	0.005
Life safety (LS)	0.01
Collapse prevention (CP)	0.02

To get the performance point demand is overlapped on the capacity curve. Both the curves as shown in Figure 7 are converted to spectral coordinates in order to get that point. In curves it is observed that the demand is very much lesser than the capacity of the building and also the demand is within the elastic range of the structure. So, the demand is increased such that the demand spectrum intersects the capacity curve in the non-linear portion. The base shear and the roof displacement at performance point are felt by the structure when an equivalent earthquake of demand spectrum is acted on it.

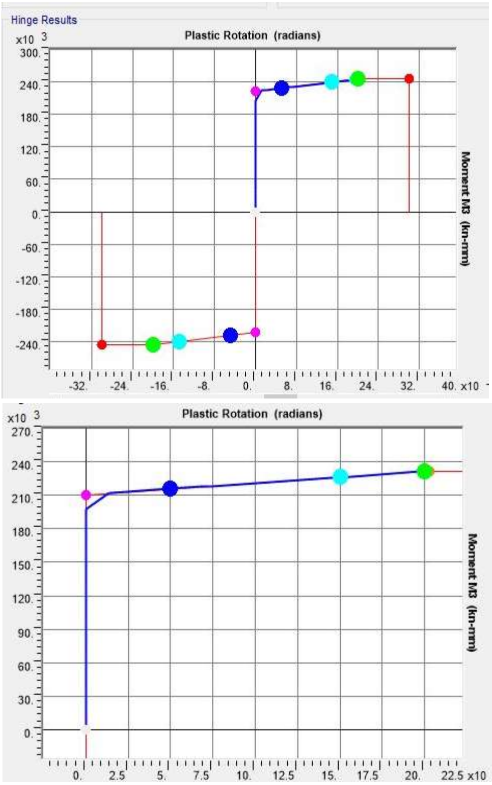

Figure 6 Moment rotation curves

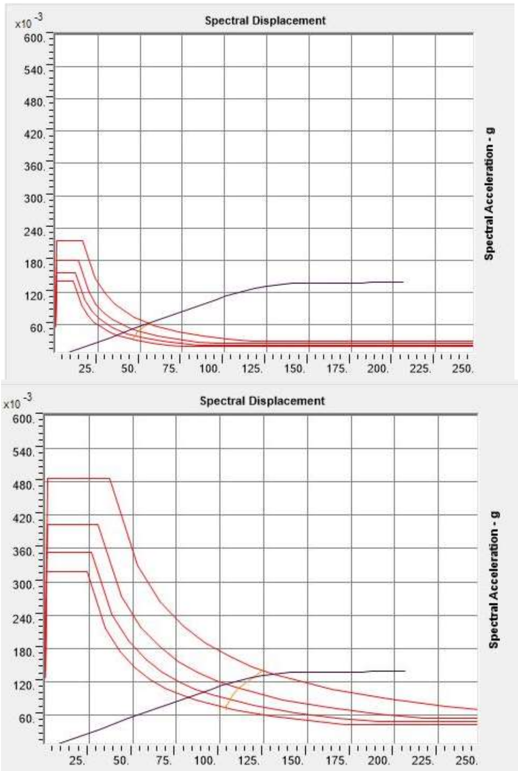

Figure 7 ADRS Curve

Performance point (V, D) = (4902.1kN, 235.81mm)

From Figure 8 the performance point in spectral coordinates (S_a, S_d) = (0.139g,192mm). The capacity spectrum is converted to bilinear form to get the spectral coordinates at yield points. Using spectral acceleration and spectral displacement at performance point and yield point the energy

Figure 8 Performance point in spectral coordinates of ADRS curve

dissipated by the inelastic behaviour of the structure can be found using the ATC 40 guidelines as given in equation below.

$$\xi_{eq} = \frac{1}{4\pi}\frac{E_{DS}}{E_S} = \frac{V_y S_{dt} - S_{dy} V_t}{\pi V_t S_{dt}}$$

V_y and V_t are the spectral acceleration at yield point and performance point respectively

S_{dy} and S_{dt} are the spectral displacement at yield and performance point respectively.

E_{DS} represents the dissipated energy

E_S represents the maximum strain energy stored

Figure 9 shows the plot between spectral displacement and time period gives the effective time period of the structure and effective damping required by the structure.

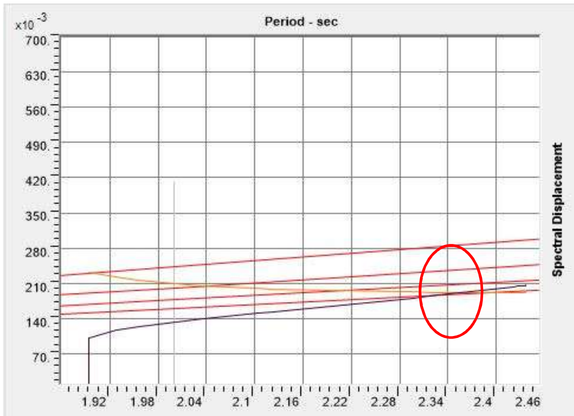

Figure 9 Spectral Displacement vs Time period

Table 2 Hinge status in pushover analysis

Step	A to <= IO	IO to<= LS	LS to CP	>CP	Total Hinges
7	1962	0	0	0	1962
12	1908	54	0	0	1962
19	1816	80	46	20	1962
22	1816	80	27	39	1962
31	1812	84	12	54	1962

Effective time period (T_{eff}) = 2.36 sec
Effective damping (β_{eff}) = 0.194

Now the amount of supplemental damping required is given by

$$\xi_d = (\xi_{eff} - \xi_{eq} - \xi_i)\frac{T_e}{T_{eff}}$$

Supplemental damping required ξ_d = 0.0946
Supplemental damping required for IO stage= 0.0946 - 0.0737 = 0.0209

4. Conclusions

The performance level chosen is IO state. The performance of the building globally is satisfying the IO state criteria with respect to drift limits. Locally many hinges are beyond IO state at performance point. To make the entire building to be in IO state the corresponding sections are to be strengthened by increasing their cross section or by providing damping.

5. Future scope

- Distribution of supplemental damping among the storeys using FEMA 1997b guidelines.
- Trilinear representation of the capacity spectrum in a way resulting in equal area under the curves.
- Comparison of seismic evaluation using ASCE 41-17 and ATC 40

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Disclosures

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