Performance Based Design of Concrete Structure

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Abstract — This paper presents a comparative PBSD analysis of G+9 regular RC frames using Response Spectrum and Time History methods in Zone IV as per IS 1893 (Part 1):2016. The primary objective of the thesis work is to verify the essential dimensions of primary structural members in the Lifeline Structures Condition of Collapse Prevention (CP), Life Safety (LS) and Immediate Occupancy (IO). The Time-History data for PGA 0.3g, 0.6g & 0.9g was used for this analysis and adaptive pushover analyses are conducted with various configurations and design parameters to calculate the lateral displacement, and the changing law of plastic hinge distribution and deformation mechanism of RC frame structures is investigated. Based on the analysis results, the displacement profile expression is proposed where the effect of elastic first order mode, maximum interstory drift, column-to-beam strength ratio, height-width ratio and the drift at first plastic hinge formation are involved. Its accuracy and reliability are validated by comparing with the results of nonlinear time history analyses.

Indexed Terms— Non-Structural Components, Performance-Based Seismic Design, Lifeline Structures, Maximum interstory drift ratio, Nonlinear time history analyses.

I. 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, many structural design engineers in the United States started working on developing procedures giving importance to performance rather than strength. To utilize PBSD effectively, we must recognize uncertainties included in the performance of structural members and estimation of seismic hazard.

PBSD allows design of new structures or upgradation of existing structures with a practical understanding of the risk, occupancy and monetary loss. It is an approach to obtain buildings that perform better than typical code conforming buildings. PBSD of structures is required to build new or hybrid systems, not envisioned by code. In PBSD, performance objectives are selected as per owner's priorities. A performance objective specifies permissible risk and the significant losses due to damages, at a specific seismic hazard. Then, through a series of analyses, the probable performance of building is estimated. The design is complete when the performance meets or exceeds performance objectives.

Performance-based seismic design (PBSD) philosophy has received increasing attention in recent years, and is becoming an integral part of the current design codes or guidelines. In the latest generation of PBSD developed by Pacific Earthquake Engineering Research (PEER) Center, various sources of uncertainty are considered in performance evaluation such that the probability of exceedance of various values of a decision variable can be estimated. The displacement-based design approaches, which provide important way to implement the PBSD idea, have been widely used in practical engineering because of its simplicity. In this methodology, the structure is designed to achieve a specified inelastic displacement state under earthquake hazard rather than to achieve a state which is less than a specified limit state. More recently, by employing the substitute structure concept, the Direct Displacement-Based Design (DDBD) developed by Priestley et al. provide an effective tool for implementing displacement- based design, where the structure is represented by a single degree-of- freedom (SDOF) system associated with the peak displacement response. When applying this method to designing a MDOF frame structure, the displacement profile of the structure under design-level earthquake should be assumed at the beginning of design. Then, an equivalent SDOF system can be established based on the displacement profile, and the lateral seismic force in design-level earthquake can be determined subsequently. Therefore, the displacement profile expression is of critical importance in DDBD, and should be developed by considering the actual inelastic deformation characteristics of structures.

II. NON-LINEAR DYNAMIC ANALYSIS

The modal load vectors are generated for a present number of modes, called Response History analysis. After applying the proper modal load vector to the structure, an estimation of its modal static response is made using static analysis for the designated mode. Then, a dynamic study of the relevant SDOF system is carried out to determine its spectral ordinates throughout each time step. The real-time response history for that modal quantity can be obtained by multiplying this spectral ordinate by the corresponding static model response at each time step. The other modes' modal response histories are derived using the same process. These modal replies are multiplied at each time step to produce the chosen response's real-time history for the obtained seismic design record.

This method is accurate for determining how a structure will respond seismically. As a result, the structure is sensitive to actual ground movement, which a graph of ground acceleration can represent over time. The ground acceleration is computed to acquire the ground motion record at a tiny time step. The design requirement is then selected from the peak value of this time history after the structural reaction has been determined at each time instant. Therefore, to determine forces and displacement, it is necessary to apply a mathematical model to an earthquake shaking reflected by the history of ground motion that explicitly links the non-linear properties of individual components and structural parts. Internally estimated forces and elastic responses mostly match the seismic design's predictions.

III. MODEL PROPERTIES

Consider a nine-story RC office building shown below. The structure is located in seismic zone IV. The soil conditions are medium stiff and the entire structure is supported on a mat foundation. The brick masonry is used to infill R. C. frames. This section only validates the appropriateness of the proposed displacement profiles derived by pushover analysis through NTHA and reveals the insensitiveness of displacement profile to higher mode effects. However, this does not mean that the determination of design lateral forces of the structure is not affected by the higher mode response when the proposed equation is used for design.

Properties of Building	
Height of building (m)	32
Plan area (sq.m)	675
Plan dimension (m)	37.5 X 18
Column size (mm)	1. 750 X 750 2. 650 X 650
Beam size (mm)	750 X 350
Thickness of slab (mm)	150
External wall width (mm)	250
Internal wall width (mm)	150
Parapet wall width (mm)	250
Parapet wall height (m)	1.2
Unit weight of concrete (kN/m3)	25
Unit weight of masonry (kN/m3)	20
Grade of Concrete	M30
Grade of Steel	Fe500

Loads Applied on Building	
Live Load Floor (kN/m)	2
Live Load Roof (kN/m)	1.5
Floor Finish (kN/m)	1.5
Roof Treatment (kN/m)	1.5
External Wall (kN/m)	13.5
Internal Wall (kN/m)	8.1
Parapet Wall (kN/m)	5

Seismic Parameters	
Seismic zone	IV
Importance factor	1
Response reduction factor	5

Type of soil	II
Damping (%)	5

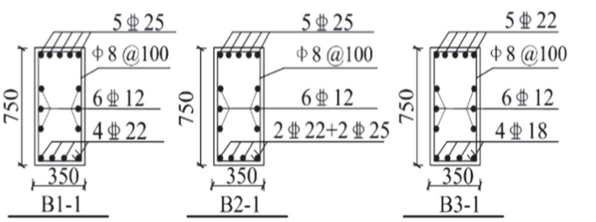
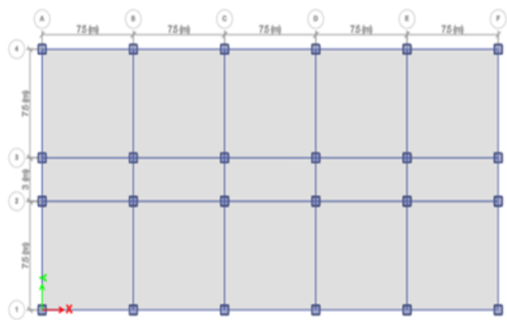


Fig. Beam Reinforcement.

Models:



Reinforcements:

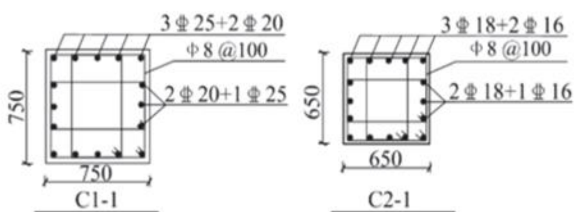


Fig. Column Reinforcement.

IV. RESULTS

Three ground motion intensity levels (i.e. PGA = 0.3 g, 0.6 g and 0.9 g) are considered. Figures. 1, 2 and 3 compares the displacement profiles obtained from NTHA and those calculated by Equations for the structure. It can be seen that the proposed Equation can accurately predict the displacement profile of RC frame structures for different θ_{max} values, and this indicates that proposed expression can be used for the design of RC frame structures considering various performance levels.

Table. 1 NTHA Vs Manual Calculated

Story	PGA 0.3g NTHA	PGA 0.3g Manual
	mm	mm
Story 9	100.52	101.5922206
Story 8	97.668	98.29360551
Story 7	92.552	93.33880113
Story 6	84.419	85.74780748
Story 5	72.998	74.90624544
Story 4	60.104	61.76725233
Story 3	44.91	46.67769083
Story 2	27.984	28.32194006
Story 1	11.68	11.8
Base	0	0

It is observed that for a given RC frame structure, the inelastic displacement profile is associated with a selected maximum inter story drift ratio and is related to the column-beam strength ratio, the height-width ratio and the drift corresponding to the first plastic hinge formation. The normalized displacement profile expression is established considering the above parameters and the verification shown that the proposed expression is capable of evaluating the lateral displacement of RC frame structures reliably. The displacement profile equation proposed in the study is suitable for the performance based seismic design and the structures dominated by the higher order mode of vibration.

Table. 2 NTHA Vs Manual Calculated

Story	PGA 0.6g NTHA	PGA 0.6g Manual
	mm	mm
Story 9	144.304	145.5922206
Story 8	139.855	140.2936055
Story 7	130.571	131.3388011
Story 6	114.219	114.7478075
Story 5	90.952	91.60624544
Story 4	68.459	68.96725233
Story 3	51.218	51.67769083
Story 2	32.517	33.32194006
Story 1	14.284	14.8
Base	0	0

Table. 3 NTHA Vs Manual Calculated

Story	PGA 0.9	PGA 0.9 Manual
	mm	mm
Story 9	210.225	211.5922206
Story 8	207.73	208.9360551
Story 7	203.052	204.3388011
Story 6	195.263	196.8478075
Story 5	181.928	182.9062454
Story 4	161.901	163.0672523
Story 3	132.482	133.7776908
Story 2	93.635	94.92194006
Story 1	48.727	49.25
Base	0	0

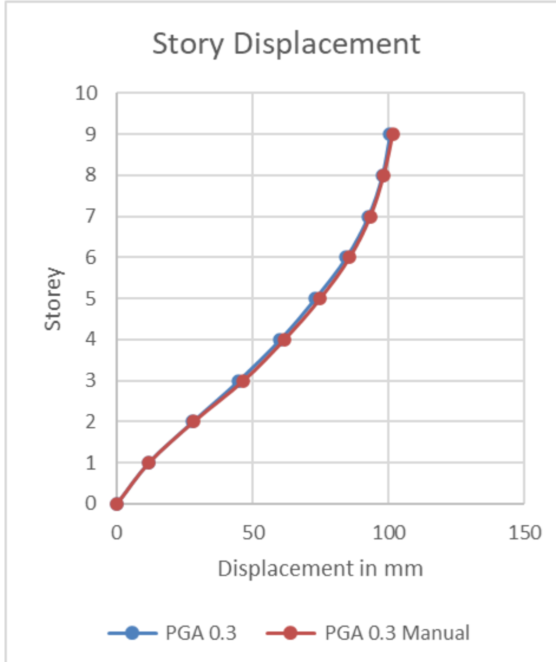


Fig. 1 NTHA Vs Manual Calculated

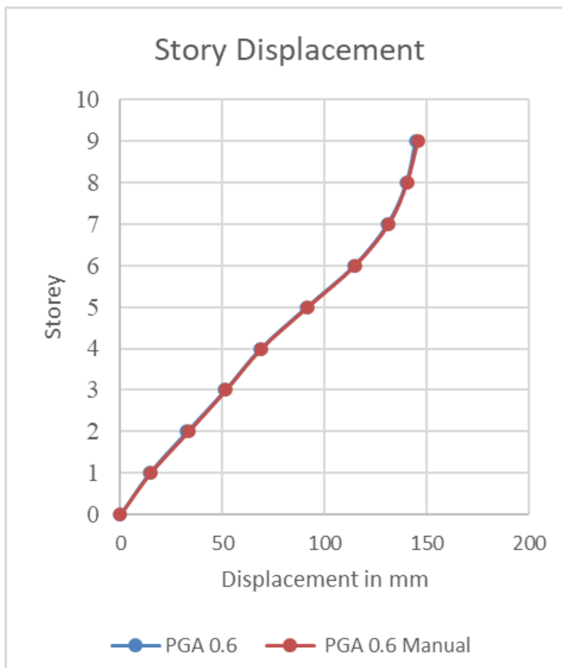


Fig. 2 NTHA Vs Manual Calculated

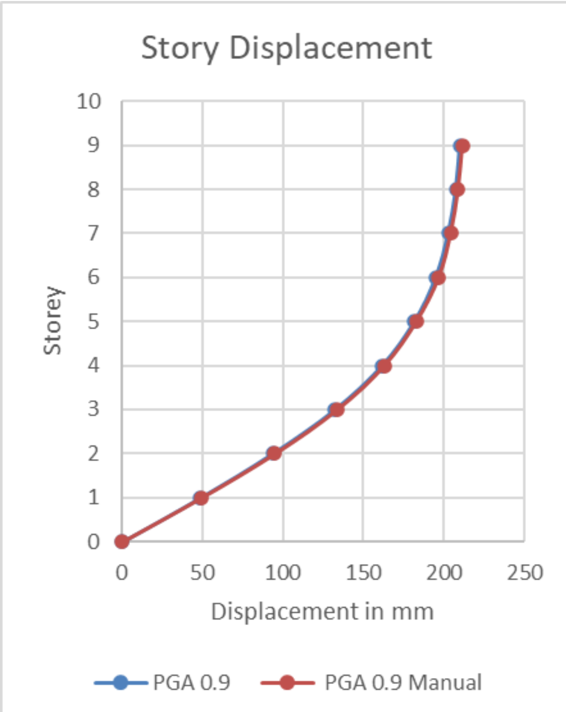


Fig. 3 NTHA Vs Manual Calculated

CONCLUSION

1. For PGA 0.3g, the maximum variation for story displacement is 2.547% between manually calculated and results of non-linear time history analysis.
2. For PGA 0.6g, the maximum variation for story displacement is 0.884% between manually calculated and results of non-linear time history analysis.
3. For PGA 0.9g, the maximum variation for story displacement is 0.805% between manually calculated and results of non-linear time history analysis.
4. The study validates the appropriateness of the proposed displacement profiles derived by pushover analysis through NTHA and reveals the insensitiveness of displacement profile to higher mode effects.

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