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PERFORMANCE BASED SEISMIC DESIGN OF A TEN STOREY RCC COMMERCIAL BUILDING USING NONLINEAR PUSHOVER ANALYSIS

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Abstract. *Performance-based design using nonlinear pushover analysis, which generally involves monotonous and intensive computational effort, is an elastic design methodology done on the probable performance of the building under input ground motions. In this Study, a ten storey commercial building is designed using ETABS and the performance based seismic design is performed by pushover analysis technique using ETABS 2015, a product of Computers and Structures International. An extensive study is conducted to investigate the effect of different parameters on the performance point. The parameters include the effect of input ground motions on the performance point, changing percentage of reinforcement in columns, size of columns and beams individually. The results of analysis are compared in terms of base shear and storey displacements.*

Keywords: Performance based design, Pushover analysis, Elastic response spectrum, Future trends

1 INTRODUCTION

Amongst the natural disaster the earthquake have the potential for a devastating damages. The basic concept of Performance-based seismic design (PBSD) is to provide engineers with the capability to design buildings that have a predictable and reliable performance in earthquake. Performance based design is an elastic design methodology which requires rigorous nonlinear analysis. Pushover analysis which is an iterative method under constant gravity loads and monotonically increasing lateral forces until a target displacement is reached is generally carried out to understand real behavior of structure during strong ground acceleration. The major outcome of pushover analysis is the capacity curve which shows the base shear vs. roof displacement.

2 LITERATURE REVIEW

Performance based design has been practiced since early in the twentieth century. The International Code Council (ICC) [1] in the United States had a performance code available for voluntary adoption since 2001 (ICC, 2001). In 1989, the FEMA-funded project was launched to develop formal engineering guidelines for retrofit of existing buildings began [2], it was recommended that the rules and guidelines be sufficiently flexible to accommodate a much wider variety of local or even building-specific seismic risk reduction policies than has been traditional for new building construction. The performance levels were generalized with descriptions of overall damage states with titles of Operational, Immediate Occupancy, Life Safety, and Collapse Prevention. Over the 10-year period after publication of FEMA 273 [3], its procedures were reviewed and refined and eventually published in 2006 as an American Society of Civil Engineers (ASCE) national standard - Seismic Rehabilitation of Existing Buildings, ASCE 41. It is considered to represent the first generation of performance-based seismic design procedures.

3 CASE STUDY

A 3-D model of ten story concrete commercial building shown in Fig.1 has been created using finite element package ETABS 2015 [4] to undertake the nonlinear analysis. The structure has designed according to the BNBC 2015[5](Bangladesh National Building Code). From soil report it has found the structure will be built in medium dense sandy soil (SC Type, SPT: 15 -50) in zone 4 (very severe seismic intensity zone) of Bangladesh Seismic map. For this zone PGA value is 0.36g. This building is 30m x 20m in plan and 3.5mm x 10 floors in elevation. There are 4 bays in the X direction and 3 bays in the Y direction.

Beams and columns are modeled as nonlinear frame elements with lumped plasticity at the start and the end of each element. ETABS 2015 provides default-hinge properties and recommends PMM hinges for columns and M3 hinges for beams as described in FEMA-356.

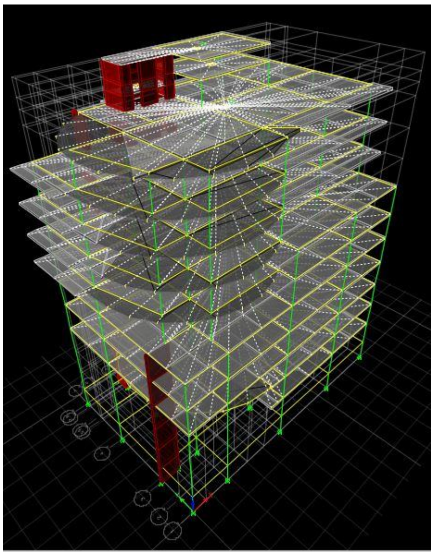

Figure 1: 3D Model of the Concrete Structures

4 PUSHOVER ANALYSIS

In the push over analysis method, earthquake load is applied on the model in an incremental basis. As the loads are increased, the building undergoes yielding at a few locations. Every time such yielding takes place, the structural properties are modified approximately to reflect the yielding. The analysis is continued till the structure collapses, or the building reaches certain level of lateral displacement. The material nonlinearities are assigned as hinges; M3 flexural hinges for beams and PMM flexural hinges for columns. Then each lateral load pattern is applied.

5 PERFORMANCE BASED DESIGN

Performance-based seismic design explicitly evaluates how a building is likely to perform; given the potential hazard it is likely to experience, considering uncertainties inherent in the quantification of potential hazard and uncertainties in assessment of the actual building response. As graphically presented in Fig. 2, the nonlinear static analysis procedure requires determination of three primary elements: capacity, demand and performance. The capacity spectrum can be obtained through the pushover analysis, which is generally produced based on the first mode response of the structure assuming that the fundamental mode of vibration is the predominant response of the structure. This pushover capacity curve approximates how a structure behaves beyond the elastic limit under seismic loadings. The demand spectrum curve is normally estimated by reducing the standard elastic 5% damped design spectrum by the spectral reduction method. The intersection of the pushover capacity and demand spectrum curves defines the “performance point” as shown in Fig. 2.

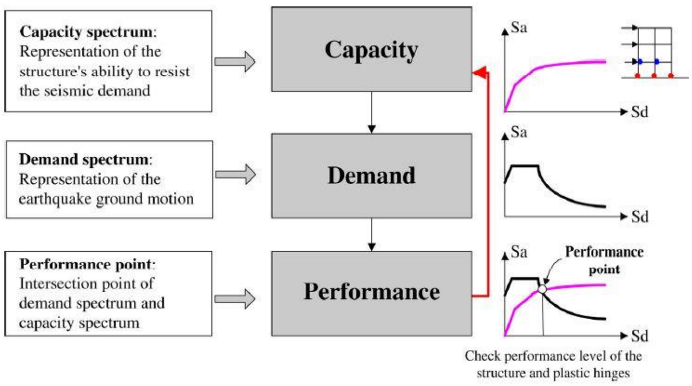

Figure 2: Nonlinear Analysis Procedure

At the performance point, the resulting responses of the building should then be checked using certain acceptability criteria. When the responses of a structure do not meet the targeted performance level, the structure needs to be resized the design process repeated until a solution for the desired performance level is reached. In general, the determination of the satisfactory performance response that fulfills both system level response and element level response requires a highly iterative trial-and-error design procedure even the aid of today’s engineering computer software. The performance target can be a specified limit on and response parameter such as stress, strains, displacements, accelerations, etc. Usually drift levels are associated with specific damage categories. Some of the subjected performance levels can group in equivalent categories as listed in Table 1. [3]

Table 1: Definition of performance level according to FEMA

Performance level	Performance description	Story drift
Fully operational	Continuous service, negligible damage	<0.2 %
Operational	Safe for occupancy, light damage, repairs for Non-essential operation	<0.5 %

Life safety	Moderate damage, life safety protection, repair may be possible but impractical	<1.5 %
Near collapse	Severe damage, collapse prevented, falling Non-structural elements	<2.5 %
Collapse		>2.5 %

6 SIMULATION AND RESULTS

To illustrate the PBD procedure for finding the performance point, a ten storey concrete frame of a commercial building as shown in Fig.1 is taken as an example. The frame is designed according to BNBC 2015 using ETABS. Structural details and natural frequencies of the concrete frame are given in Table 2 and 3 respectively. The pushover analysis is performed on the RC building and redesigning by changing the main reinforcement of various frame elements and again analyzing. For parametric studies, a total of 13 cases as per Table 4, for a particular ten storey building frame located in Zone-4 have been analyzed, changing reinforcement and sizes of different structural elements, i.e. beams and columns, in different combinations as well as at different storey levels. Roof displacement, ductility demand, performance point and effect of change in beam and column size have been illustrated in the Table 5, 6, 7 and 8, respectively.

Table: 2 Structural Details as per ETABS

S. No.	Structural Element's	Dimension (m)		Reinforcement area (mm^2)	
		Breadth	Depth		
1	C1 Column Up to Ground Floor	0.813	0.813	10862	
2	C1 Column Ground Floor to 5th Floor	0.762	0.762	10862	
3	C1 Column 6th to Roof	0.762	0.762	7250	
4	C2 Column Up to Ground Floor	0.559	0.661	7150	
5	C2 Column Ground Floor to 5th Floor	0.508	0.609	7150	
6	C2 Column 6th to Roof	0.508	0.609	6050	
				Top	Bottom
7	Grade Beam	0.310	0.534	1650	1375
8	Floor Beam	0.310	0.610	2525	2031

Table 3: Natural frequency and time periods

Mode shape	Period (sec)	Frequency (cycle/sec)
1	1.272	0.786
2	0.462	2.164
3	0.257	3.891

Table 4: Various cases for parametric studies

Serial No	case	Description of case	Serial No	case	Description of case
1	A	Basic Structure	9	H	10% decrease in column size
2	B	10% increase in column reinforcement	10	I	20% decrease in column size
3	C	20% increase in column reinforcement	11	J	10% increase in beam size
4	D	10% decrease in column reinforcement	12	K	20% increase in beam size
6	E	20% decrease in column reinforcement	13	L	10% decrease in beam size
7	F	10% increase in column size	14	M	20% decrease in beam size
8	G	20% increase in column size			

Table 5: Roof displacement for elastic and inelastic response spectra for different Performance level

Serial no	Performance Level	Roof Displacement for PGA 0.36g (mm)	
		Elastic	Inelastic
1	Operational	63.11	57.39
2	Immediate Occupancy	144.23	133.1
3	Life Safety	212.67	208.31
4	Collapse Prevention	372.72	309.11
5	Complete Collapse	∞	∞

Table 6: Ductility demand for elastic and inelastic response spectra for different Performance level

Serial no	Performance Level	Ductility Demand for PGA 0.36g (mm)	
		Elastic	Inelastic
1	Operational	1	1
2	Immediate Occupancy	3.45	3.01
3	Life Safety	11.23	10.61
4	Collapse Prevention	14.22	13.59
5	Complete Collapse	∞	∞

Table 7: Performance point

	Performance point for PGA 0.36g	
	Elastic	Inelastic
Base Shear (KN)	6821	2593
Roof Displacement	198.34	35.33

Table 8: Effect of change in beam and column size and reinforcement

Serial No	Case	% change in roof displacement	% change in base shear
1	A	-	-
2	B	3.45	-4.72
3	C	7.45	-17.34
4	D	-1.76	-4.42
5	E	-6.87	15.29
6	F	2.63	-3.19
7	G	4.34	-20.22
8	H	-2.34	7.25
9	I	-6.66	12.11
10	J	1.21	-3.92
11	K	1.45	-4.99
12	L	-0.98	2.34
13	M	-1.62	3.31

7 CONCLUSION

Based on the present study, the following conclusions can be drawn:

- Since frequencies are varied, higher modes are neglected for pushover analysis
- As the response changes from elastic to inelastic, roof displacements and ductility demands decrease for different PGA level.
- The performance point obtained satisfies the acceptance criteria for immediate occupancy and life safety limit states for various intensities of earthquake.
- The increase in reinforcement of columns results in nominal change in base shear and displacement.
- As the beam and column section size increases, the roof displacement decreases whereas base shear increases.
- As the size decreases, the roof displacement increases whereas base shear decreases.

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