Stochastic analysis for assessment of Sensitivity of pushover curve to design parameters

Neena Hede(Panandikar)
NITK, Research Scholar
Department of Civil Engineering, Surathkal

Dr.K.S.Babu Narayan
Department of Civil Engineering
NITK, Surathkal, Karnataka, India

Abstract- Nonlinear Static Pushover analysis is an inelastic static analysis procedure which gives due consideration to the material non-linearity. It is regarded as an efficient tool to evaluate the performance of the structure under lateral loads. Pushover analysis in its present form has several limitations, which needs to be addressed in order to bring the accuracy of pushover analysis at par with the nonlinear dynamic analysis. Researchers are showing lot of interest in nonlinear static pushover analysis as lot of computational time is required to perform complete dynamic analysis. It has been observed that the results of pushover analysis are very sensitive to material model adopted, geometric model adopted, modeling of joints, location of plastic hinges and in general to procedure followed by the analyzer.

A G + 2 RC Framed structure has been tested and pushover results are available. Considering these available results as basis, attempt has been made to assess sensitivity of pushover analysis results using SAP2000 software by considering user defined hinges and frame modeled with slab modeled as shell element and results compared with experimental values. Three random uncertain parameters adopted are strength of concrete, strength of steel and cover to the reinforcement .Hinge length is adopted by considering Pauley and Mattock's Hinge length formulations. Monte Carlo simulation is used and stochastic analysis is carried and generated random variables are given as input to the simulation model.

Keywords – Base shear, Displacement, moment curvature, sensitivity.

I. Introduction

Pushover analysis has been the preferred method for seismic performance evaluation due to its simplicity and has been viewed as an attractive alternative to the nonlinear time history analysis. The use of nonlinear analyses is essential to capture behavior of structures under seismic effects. (Mehmet Inel and Ozmen, 2006). However, an assessment of the uncertainty in the nonlinear pushover analysis methods must be made in order to incorporate this method in the reliability framework of performance-based design (Mathew J.Skokan and Gary C.Hart, 2000).

Lot of research has been carried out on conventional pushover analysis and after investigating deficiencies, efforts have been made to improve it. The procedure for modeling and performing three dimensional pushover analyses for a building structure using SAP2000 was proposed by Ashraf Habibullah et al [1998] .It documents the modeling procedure and defines force-displacement criteria for hinges as documented in ATC-40 and FEMA documents. They suggested that for buildings that are being rehabilitated it is easy to investigate the effect of different strengthening schemes. Also buildings can be strengthened by changing member properties and modifying the hinge acceptance criteria and rerunning the analysis. Using Sap2000 lot of research has then been carried out to seismically evaluate existing structures and retrofitting techniques are suggested if found seismically deficient.

Much development is still needed, coupled with further analytical verification and ultimately experimental confirmation. (Elnashai, 2001).Lot of structures has been analyzed using SAP2000 for carrying out seismic evaluation and if found deficient retrofitting techniques have been suggested. But actual test results to verify the analytically obtained pushover results are rarely available. In this paper attempt is being made to compare analytically obtained results with experimentally obtained results by considering a G+2 storied RCC framed structure that was tested at SERC (Structural Engineering Research Centre) Chennai.

II. EXPERIMENTAL DETAILS

The experimental setup used for conducting the test is shown in below. The test was conducted at SERC, Chennai. (Akanshu Sharma e tal). The RCC framed structure was pushed with the help of servo hydraulic actuators by taking reaction from the reaction wall. The parabolic load distribution along height was maintained.

Figure 1. Experimental Setup for Pushover Test

This test was performed under monotonically increasing Pushover Loads. The loads pattern was kept as parabolic along the height of the structure with the load ratio of 1: 4: 9 for first storey: second storey: third storey. The total base shear was thus obtained by adding the loads at different levels. Therefore if the load applied at first floor is ‘P’ kN then the loads at second and third floor are 4P and 9P kN respectively. The maximum base shear and corresponding roof displacement was found to be 286.5kN and 0.11m respectively. The authors have taken the experimental values of roof displacement and corresponding base shear as basis and simulation of the same structure is carried out in Sap-2000 so as to investigate the sensitivity of pushover curve to modeling.

III. STRUCTURAL DETAILS

A. Sectional Details and Material Properties –

Both beam and column sections are 150mm x 200mm in size with 2-12Φ bars at top and bottom in case of beam and 2-16Φ bars at top and bottom in case of columns. The transverse reinforcement for both beams and columns is provided by 2-legged 6Φ stirrups/ties @ 150mm c/c. The slab is 50 mm thick. Fig. 2 shows the section properties for the beams and columns.

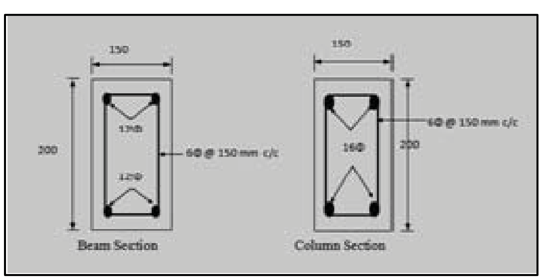

Figure 2 Section Details

The actual material properties from tests were found as
Average concrete strength = 35 MPa

Average Reinforcement yield stress = 478 Mpa

B. Details of Modelling

A basic computer model is created using the graphical interface of SAP2000 and the material properties and Geometric properties defined. The program includes several built-in default hinge properties that are based on average values from ATC-40 for concrete members and average values from FEMA-273 for steel members. But in this paper user defined hinge option has been used .To adopt user defined hinge option, moment curvature analysis has been done using compatibility and equilibrium equations. The pushover load cases are then defined. 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 pushover. Lateral load distribution across the height of the building is found out based on the formula specified in FEMA 356, given by equation(1) and then in-cooperated in the model.

$$F_x = \frac{W_x h_x^k}{\sum_{i=1}^{N} W_i h_i^k} V \qquad (1)$$

$$C_{vx} = \frac{W_x h_x^k}{\sum_{i=1}^{N} W_i h_i^k}$$

Fx is the applied lateral force at level 'x', W is the storey weight, h is the story height, V is the design base shear and N is the number of stories. Cvx is the coefficient that represents the lateral load multiplication factor to be applied at floor level 'x'.The load pattern for tests was kept as parabolic with the load value at a storey increasing in proportion to the square of the height of the floor from foundation level.

We have,

Height of 1st floor from foundation level, h1 = 1800 mm

Height of 2nd floor from foundation level, h2 = 3600 mm

Height of 3rd floor (roof) from foundation level, h3 = 5400 mm, Thus,

$\Sigma hi^2 = 1800^2 + 3600^2 + 5400^2 = 45360000$

Weightage of total force for 1st floor = $h1^2 / \Sigma hi^2 = 1800^2/45360000 = 0.0714$

Weightage of total force for 2nd floor = $h2^2 / \Sigma hi^2 = 3600^2/45360000 = 0.2857$

Weightage of total force for roof = $h3^2 / \Sigma hi^2 = 5400^2/45360000 = 0.6429$

Thus during the test carried out the load was monotonically increased in the ratio of 0.6429:0.2857:0.0714 ≈ 9:4:1 for roof: 2nd floor: 1st floor. To maintain consistency of lateral load application in experimental and numerical studies, the lateral load is applied in the same ratio to the sap model.

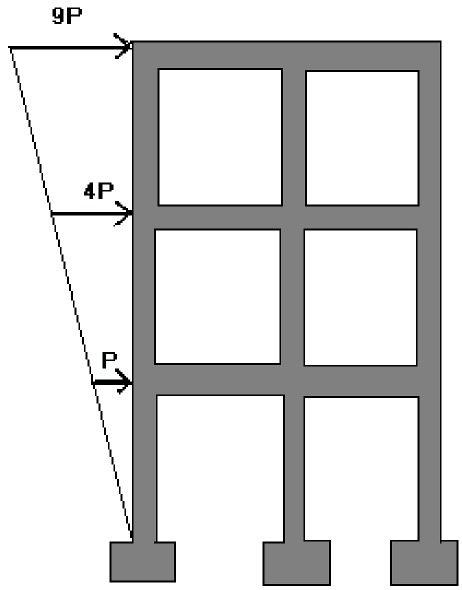

Figure 3 Photograph of loading pattern

The pushover load case PUSH for the pushover analysis was applied laterally to the model created, as a point load at each storey in the ratio of 9:4:1 for roof: 2nd floor: 1st floor as calculated using above equations. Displacement controlled Nonlinear static pushover analysis is defined for the present study.

D. Generation of moment curvature values –

Moment curvature values are obtained by considering IS recommended stress strain model for unconfined concrete shown in Fig.4 and British code recommended (CP 110-1972) stress-strain curve for steel as shown in Fig.5. (CP 110-1972) is used as it gives all the simplified general equations which can be used for any grade of steel.

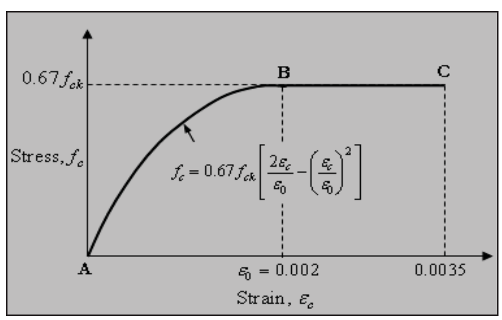

Figure. 4 IS recommended stress strain model for unconfined concrete

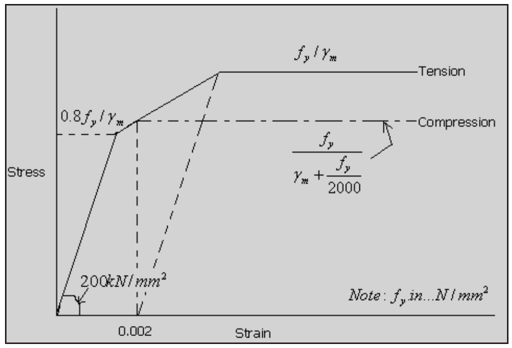

Figure .5 British code recommended (CP 110-1972) stress-strain curve for steel

.The generated moment curvature values are shown in Table 1 and Table 2 for columns and beams respectively. Actual material properties were used in performing the analysis. The moment values are in kN-m.

Table 1Moment curvature values for columns

Points	A	B	C	D	E
	Origin	Yielding	Ultimate	Strain hardening	Strain hardening
fy=478 N/mm2 **fck=35N/mm2**	M=0 Φ=0	M=21.55 Φ=0.0134	M=23.34 Φ=0.088	M=25.12 Φ=0.099	M=26.94 Φ=0.111

Table 2 Moment curvature values for beams

Points	A	B	C	D	E
	Origin	Yielding	Ultimate	Strain hardening	Strain hardening
fy=478 N/mm2 **fck=35N/mm2**	M=0 Φ=0	M=12.76 Φ=0.011	M=14.58 Φ=0.078	M=16.40 Φ=0.090	M=18.23 Φ=0.105

M3 plastic hinges (user-defined) was assigned to beam and columns to incorporate elemental nonlinearity. It represents nonlinear moment rotation characteristics.

IV. ANALYSIS

Initial study was carried out by considering the actual material properties of the test results and modeling of the frame was done in SAP2000 considering bare frame and frame modeled as rigid diaphragm and frame modeled as shell frame. User defined hinge properties are incorporated by generating moment curvature data by adopting IS recommended stress strain model for unconfined concrete and British code recommended (CP 110-1972) stress-strain curve for steel. The results of pushover Analysis carried out are shown in Figure.6 and Figure.7 respectively. (Neena etal, 2013).

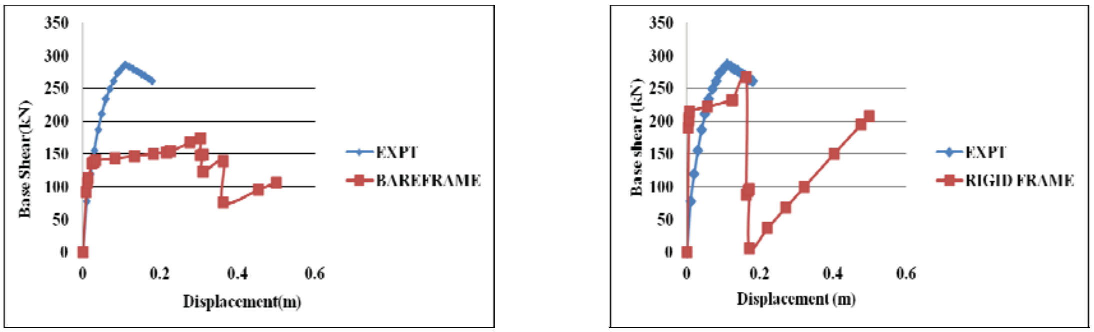

Figure.6 Comparison of experimental pushover results with analytical (Bare frame) and Rigid frame

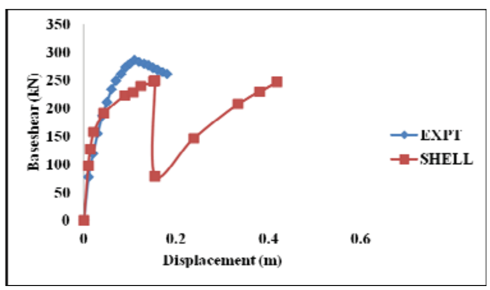

Figure. 7 Comparison of experimental pushover results with analytical (Shell frame)

From the analysis carried out it was found that for the frame modeled as bare frame the corresponding P (Base Shear) and Δ (Displacement) was found to be 173.8kN and 0.31m respectively and for frame with slab modeled as rigid diaphragm the corresponding P (Base Shear) and Δ (Displacement) was found to be 267kN and 0.16m respectively. For frame with slab modeled as shell element the corresponding P (Base Shear) and Δ (Displacement) are found to be 248kN and 0.15m respectively. Thus from above analysis it is clear that there is some variation in Base shear and Displacement values for the various geometric models considered. This indicates that pushover results are sensitive to geometric modeling. The base shear and displacement values for the above geometric models considered also vary when compared with experimental values. The percentage variation is shown in Table 3.

Table 3. Percentage variation in base shear and displacement when compared with Experimental

Frame type	(P) kN	% less than Expt	Δ(m)	% more than Expt
Bare Frame	173.8	39 %	0.31	172%
Rigid frame	267	7.3%	0.16	45%
Shell	248	15.52%	0.15	36.36%

Further study is carried out by considering randomness in design parameters and for discrete hinge locations. Three random uncertain design parameters adopted are strength of concrete, strength of steel and cover to the reinforcement. The discrete hinge locations considered are 0.0L, 0.05L, 0.1L, 0.15L and 0.2L.Hinge length is adopted by considering Pauley and Mattock's Hinge length formulations available in literature as given by equations below.

Mattock's formula

$$l_p = 0.5d + 0.05z \tag{2}$$

Pauley-Priestley formula

$$l_p = 0.08z + 0.022 d_b f_y \quad \text{(MPa)} \tag{3}$$

Where,
db = diameter of main reinforcing bars in mm
fy = yield strength of reinforcement bars, in MPa.
Z = Distance of critical section from point of contraflexure
d = effective depth of the member.

Monte Carlo simulation is used and stochastic analysis is carried and generated random variables are given as input to the simulation model. Considering lower and upper limit as 15% decrease and 15% increase in reference grade of steel (fy=478N/mm^2), reference grade of concrete (fck=35N/mm^2) and Cover(20mm), random numbers were generated and incorporated in the analysis. Programming was done in excel to get newly generated moment curvature data. For each combination of above mentioned uncertain parameters moment curvature data is generated and input in SAP2000 as user defined hinge properties in the model .About 25 iterations are carried out by using model alive and data base interactive editing in SAP2000. Base shear and Displacement values for hinge location at 0.15L is shown in Table .4.Similar iterations are carried out considering hinge locations at 0.0L, 0.05L,0.1Land 0.2L respectively. The statistical analysis for the above generated data is carried out in Minitab (version 16).The statistical analysis results are shown in Table. 5.

Table.4 Design Matrix: Base shear and Displacement values for hinge location at 0.15L

Sr .No				Pauley		Mattock	
	Fy	Fck	Cv	P(kN)	Δ(m)	P(kN)	Δ(m)
1	34.93	497.1	21.5	299.61	0.074	286.18	0.055
2	30.61	441.6	14.48	280.4	0.067	271.55	0.053
3	38.44	534.9	18.99	271.36	0.064	255.1	0.048
4	35.35	529.9	19.46	311.9	0.077	294.2	0.052
5	31.81	464.8	25	296	0.108	277	0.051
6	35.72	523	23.54	297.64	0.072	283.9	0.054
7	31.4	421.3	24.96	254.56	0.071	243.6	0.052
8	35.54	434.2	18.19	269.93	0.067	256.39	0.0505
9	30.75	454.3	10.62	286.4	0.066	274.7	0.051
10	30.13	486.2	15.83	279.7	0.064	264	0.048
11	28.71	530.3	14.91	305.4	0.069	289.7	0.05
12	38.58	517.7	15.23	290.59	0.052	293	0.052
13	29.39	484	13.92	300.6	0.07	284.4	0.052
14	35.91	540.8	26.88	312.6	0.072	292.4	0.05
15	34.68	501.3	21.15	297.3	0.073	283.8	0.054
16	36.53	485.6	20.1	290.5	0.07	277.3	0.053
17	39.5	457.3	11.74	290.51	0.07	278.89	0.053
18	32.4	468.6	19.01	284.5	0.07	269.5	0.052
19	30.55	481.4	23.98	272.7	0.068	257.6	0.05
20	29.1	480.9	16.68	294	0.071	278.3	0.052
21	31.82	501.9	28.94	270.1	0.067	258.5	0.052
22	29.50	485.7	14.50	300	0.07	284.39	0.052
23	30.96	514.1	24.11	290.96	0.07	277	0.053
24	34.78	467.7	29.65	264.4	0.071	252.96	0.053
25	35.72	527.1	10.68	319.18	0.072	271.11	0.057

Table.4 Statistical Results

Hinge location	Pauley		Mattock	
	Mean P(kN)	MeanΔ(m)	Mean P(kN)	MeanΔ(m)
@0.0L	252.24	0.109	238.3	0.073
@0.05L	251.5	0.1204	242.2	0.064
@0.1L	263.4	0.070	253	0.056
@0.15L	289.2	0.070	274.2	0.051
@0.2L	331.2	0.065	315.4	0.052

It is observed that pauley's formulation gives nearer results when compared to experimental values of base shear and displacement. Hypothetical testing of generated data considering Pauley' formulation is carried out to assess uncertainty by considering target value of base shear and displacement as the experimental values viz, P=286.5kN and Δ=0.11m.
The results of one sample t-test carried out is shown in Table 5

Table 5 Results of Hypothetical Testing (Pauley formulation)

Hinge location	Pmean	95% CI	Δmean	95% CI
@0.0L	252.24	246.80-257.67	0.10	0.10506-0.11318
@0.05L	251.48	245.49-257.46	0.12	0.0559 - 0.1850
@0.1L	263.14	257.35-269.48	0.078	0.07502- 0.08250
@0.15L	289.23	282.64-295.83	0.070	0.06686-0.07434
@0.2L	331.17	322.9-339.45	0.065	0.06210- 0.06795

IV.CONCLUSION

- It is observed that the base shear increases with increase in distance of hinge location from the end supports of the elements for both hinge length formations considered.viz Pauley and Mattock.
- For the hinge formulations considered either the base shear values are closer to experimental values or the corresponding displacement but not both for any of the hinge location considered. This is also evident from the hypothetical test results for Pauley's formulation.
- The frame with shell model with hinges formulated using Pauley's and Mattock 's formulations and located at a distance of 0.15L from the supports show the base shear values nearer to the experimental values whereas the corresponding displacement is less.

The analysis results show that there is variation between analytically obtained pushover results when compared to experimental values for frame modeled with shell slab.The variation is because the pushover results are very sensitive to geometric model, material model adopted, hinge location and hinge length adopted. Further study needs to be carried out by considering other material models and considering random hinge location and other hinge lengths formulations viz Corley, sawyer available in literature and the results compared with experimental observations.

REFERENCES

[1] Applied Technology Council, ATC-40.Seismic evaluation and retrofit of concrete .

[2] CP 110: PART 1:1972- British code of practice

[3] CSI, SAP2000 Integrated finite element analysis and design of structures basic analysis reference manual, Berkeley (CA,USA): Computers and Structures,2002. Computers and Structures

[4] Elnashai, A.S. "Advanced inelastic static (pushover) analysis for earthquake applications", Journal of Structural Engineering and Mechanics 2001.Vol 12, pp.51-69

[5] Federal Emergency Management Agency, FEMA-273.NEHRP guideline for Seismic rehabilitation of buildings Washington (DC); 1997

[6] Matthew J.Skokan and Gary C. Hart, "Reliability of Nonlinear Static methods for the Seismic Performance Prediction of steel Frame Buildings"12WCEE 2000.

[7] Mehmet Inel, Hayri Baytan Ozmen.Effects of plastic hinge properties in nonlinear analysis of reinforced concrete buildings. Engineering Structures Vol .28,pp. 1494-1502, 2006

[8] Ashraf Habibullah and Stephen Pyle , "Practical Three Dimensional Nonlinear Static Pushover Analysis", Published in Structure Magazine, winter ,1998

[9] Mattock, A.H. "Discussion of Rotational capacity of reinforced concrete beams", Journal of structural division, ASCE, Vol. 93, ST2, pp. 519-522.1967.

[10] Neena Panandikar (Hede), Babu Narayan K.S ."Stochastic modeling to investigate uncertainty in pushover analysis", *Proc. of Interantional conference on Modelling and simulation in civil Engineering*, Kollam, Kerala, pp.158-164.2013.

[11] Pauley T. and Priestley, M.J.N. "Seismic Design of Reinforced Concrete and Masonry Buildings". New York ;John Wiley and Sons; 1992.

[12] Akhanshu Sharma ,Reddy, ghosh, Kushwaha . A report on "Experimental and analytical investigation on behavior of scaled down reinforced concrete framed structure under monotonic pushover loads"

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