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Subject: Structural Engineering

Keyword: Structure

Publication Date: 2004

Original Publication: CTBUH 2004 Seoul Conference

Paper Type:
1. Book chapter/Part chapter
2. Journal paper
3. **Conference proceeding**
4. Unpublished conference paper
5. Magazine article
6. Unpublished

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Lateral Load Distribution Factor for Modal Pushover Analysis

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Abstract

A simple method for the nonlinear static analysis of complex building structures subjected to monotonically increasing horizontal loading(pushover analysis) is presented. The method is designed to be a part of new methodologies for the seismic design and evaluation of structures. A variety of existing pushover analysis procedures are currently being consolidated under programs such as ATC 40 and FEMA 273. In this paper, a modal pushover analysis using design response spectra of UBC 97 is proposed. The proposed method is compared against the method in FEMA 273 and results of time history analysis

Keywords: lateral load distribution factor, pushover analysis, modal pushover

1. Introduction

Various analysis methods, both elastic(linear) and inelastic(nonlinear), are available for the analysis of buildings. The most basic inelastic analysis method is the complete nonlinear time history analysis, which at this time is considered overly complex and impractical for general use. The most general nonlinear static procedures are Capacity Spectrum Method, as described in ATC-40(1996)[1)], and Displace Coefficient Method, as described in FEMA 273(1997)[2)] and FEMA 356(2000)[3)]. In order to determine capacities beyond the elastic limits, some form of nonlinear analysis, such as the pushover procedure, is required.

The pushover analysis is a static analysis procedure in which a lateral load profile is applied to the structure and then incrementally increased by a scaling factor until the displacement at some point on the structure reaches a limit state. However, this procedure has some deficits. First, the choice of the lateral force distribution used in the analysis is an important consideration. Furthermore, to conduct these analyses, one would need computer software that can perform nonlinear analysis. Finally, the creation of a nonlinear analysis model is much more complicated than the creation of a linear analysis model. The models must account for the inelastic load-deformation characteristics of important elements.

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The capacity curve(pushover curve) is generally constructed by the fundamental vibration mode or the way of vertical distribution of seismic forces represented in the Code. This is generally valid for buildings with fundamental periods of vibration up to about one second. For more flexible buildings with a fundamental period greater than one second or irregular buildings, the analyst should consider addressing higher mode effects in the analysis.

The pushover curve applied rectangular shape, 1^{st} mode and pattern in the FEMA 273(Fig. 1). It should be noted from Fig. 1 that the response of a building depends on the pattern of lateral force distribution.

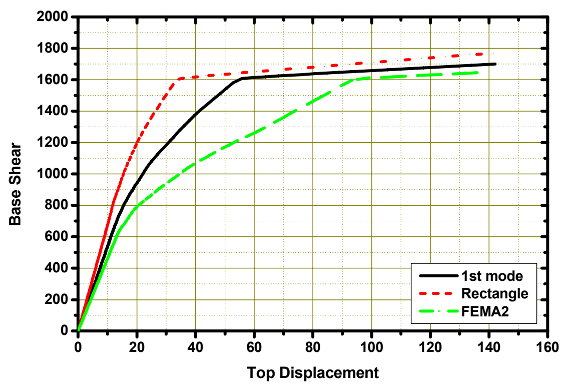

Fig. 1. Pushover curve

Thus, to represent accurate nonlinear behavior, lateral load distribution reflecting higher mode is proposed. The validity was verified to compare the proposed method with time history analysis and the method proposed in FEMA 273.

2. FEMA 273[2)] Pushover analysis

① FEMA 1(Uniform pattern)

The first pattern shall be based on lateral forces that are proportional to the total mass at each floor level. It is computed from Eq. (2.1).

$$C_{vx} = \frac{w_x}{\sum_{i=1}^{n} w_i} \quad (2.1)$$

where:

C_{vx} = Lateral load distribution factor at xth floor

n = Total floor

w_x = Weight at xth floor

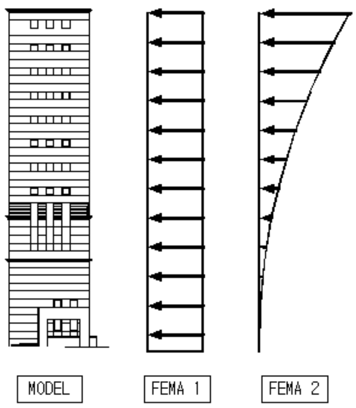

Fig. 2. FEMA 1, 2

② FEMA 2(Equivalent lateral force distribution)

The second pattern represented by values of C_{vx} given in Eq. (2.2)

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=0}^{n} w_i h_i^k} \quad (2.2)$$

where:

k = 1.0 for T≤ 0.5 second

= 2.0 for T≥ 2.5 seconds

Linear interpolation shall be used to estimate values of k for intermediate values of T.

C_{vx} = Vertical distribution factor

w_i = Portion of the total building weight W located on or assigned to floor level i

h_i = Height form the base to floor level i

The first and the second patterns are presented in Fig. 2.

③ FEMA 3(SRSS)

The calculation of the SRSS distribution is summarized as a series of steps as follows:

1. For the nth-mode calculate the lateral forces by Eq. (2.3).

$$f_{in} = \Gamma_n m_i \phi_{in} A_n \quad (2.3)$$

where:

i = floor number

A_n = Pseudo-acceleration of nth-mode SDF elastic system

$$\Gamma_n = \frac{L_n}{M_n}, \quad M_n = \sum_{j=0}^{N} m_j (\phi_j)^2, \quad L_n = \sum_{j=0}^{N} m_j \phi_j$$

2. Calculate the story shears by Equation (2.4).

$$V_{jn} = \sum_{i=j}^{N} f_{in} \quad (2.4)$$

3. Combine the modal story shears using SRSS rule.

$$V_i = \sqrt{\sum (V_{in})^2} \quad (2.5)$$

4. Back calculate the lateral forces at the floor levels from the combined story shears V_i.

5. The lateral forces are normalized by the base shear to obtain the lateral load distribution factor.

The third patterns by various earthquakes are presented in Fig. 3

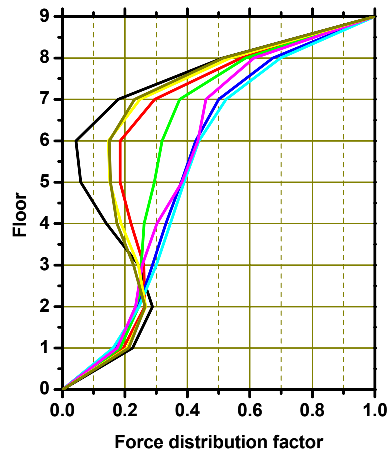

Fig. 3. FEMA 3

3. Proposed Method

The pushover curve, a plot of base shear versus roof displacement, is determined by nonlinear static analysis of the structure subjected to lateral forces with invariant distribution over height. The pattern of lateral force distribution has very important effects on the response of a building as shown in Fig. 1.

The first pattern of FEMA 273 is almost uniformly distributed over height. And the second pattern is the vertical distribution of seismic forces in the Code. This force distribution is calculated from weight, height and exponent k. These two patterns are not concerned about the contribution of higher modes. The third pattern is determined from response spectrum analysis of the building, and it is concerned about the effect of higher modes. However, because the detailed characteristics of future earthquakes are not known, it is unreasonable for the load distribution to be determined from past earthquakes. To complement this deficit, this investigation proposed the lateral load distribution factor determined from design response spectrum of UBC 97[4)](Fig. 4).

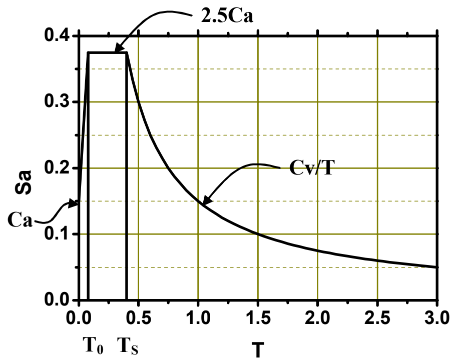

Fig. 4. The design response spectrum of UBC 97(Ca=Cv=0.15)

The following gives a detailed step-by-step determination of proposed lateral load distribution facor.

1. For the nth-mode calculate the lateral forces by Eq. (3.1)

$$f_{in} = \Gamma_n m_i \phi_{in} \psi_n \qquad (3.1)$$

where :

ψ_n : Spectral acceleration of design response spectrum of UBC 97

$$T_s = C_v / 2.5C_a \ , \quad T_0 = 0.2T_s$$

$$0 \le T < T_0 \qquad \psi = 1.5\frac{C_a}{T_0}T + C_a$$

$$T_0 \le T < T_s \qquad \psi = 2.5C_a$$

$$T_s < T \qquad \psi = \frac{C_v}{T}$$

Step 2 to 5 of the proposed method are the same as those for FEMA 3 presented in the previous section.

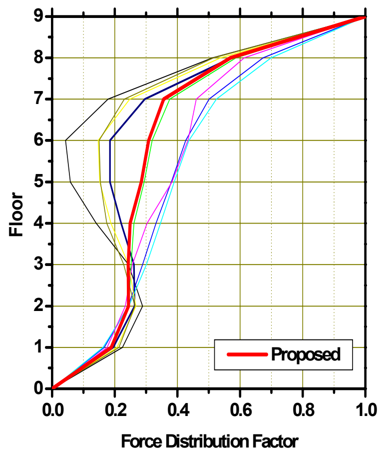

Fig. 5. Comparison of the proposed method with FEMA 3

4. Verification of proposed method

4.1 System and excitation considered

The 9-story structure, shown in Fig. 6 and 7, was designed by Brandow & Johnston Associates for the SAC Phase Ⅱ Steel Project. Although not actually constructed, this structure meets seismic code and represents typical medium-rise buildings designed for the Los Angeles, California region[5)].

The details are follows:

Beams(248MPa):

Ground-2nd level	W36×160;
3rd-6th level	W36×135;
7th level	W30×99;
8th level	W27×84;
9th level	W24×68.

Columns(345MPa):

B-1~1st	W14×500
1st~3rd	W14×455
3rd~5th	W14×370
5th~7th	W14×283
7th~9th	W14×257

column sizes change at splices

corner columns and interior columns the same, respectively, throughout elevation;

Restraints:

columns pinned at base;

structure laterally restrained at 1st level.

Splices:

denoted with ;

are at 1.83m (6ft) w.r.t. beam-to-column joint

Connections:

→← indicates a moment resisting connection.

- - indicates a simple (hinged) connection.

Table 1. Earthquake Data

Year	Location	PGA(g)	v_{max}/a	Year	Location	PGA(g)	v_{max}/a
1979	Bonds Corner EI Centro	0.778	22.65	1971	San Fernando Pacoima Dam	-0.117	38.21
1979	Coyote Lake	0.429	44.88	1966	Parkfield Cholame, Shandon	-0.275	16.42
1940	EI Centro Site	0.357	48.10	1971	San Fernando	0.315	23.82
1952	Hollywood Storage P.E	0.059	39.59	1971	San Fernando 8244 Orion Blvd.	-0.255	45.89
1979	James RD. EI Centro	-0.595	30.78	1994	Northridge, St Monica, City Hall	-0.882	18.18
1995	Kobe, Japan	0.599	49.04	1952	Taft Lincoln School	-0.156	39.45
1994	Northridge, Sylmar Country Hospital	0.344	50.01	1999	Duzce, Turkey	0.356	29.92

Dimensions:

all measurements are center line;
basement level height 3.65 m (12'-0");
Ground level height 5.49 m (18'-0");
1st-8th level heights 3.96 m (13'-0");
bay widths (all) 9.15 m (30'-0").

Seismic Mass:

including steel framing, for both N-S MRFs;
Ground level 9.65×10^{5}kg;
1st level 1.01×10^{6}kg;
2nd-8th level 9.89×10^{5}kg;
9th level 1.07×10^{6}kg;
entire structure 9.00×10^{6}kg;

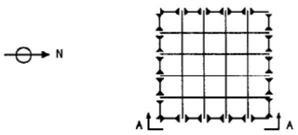

Fig. 6. Building Plan

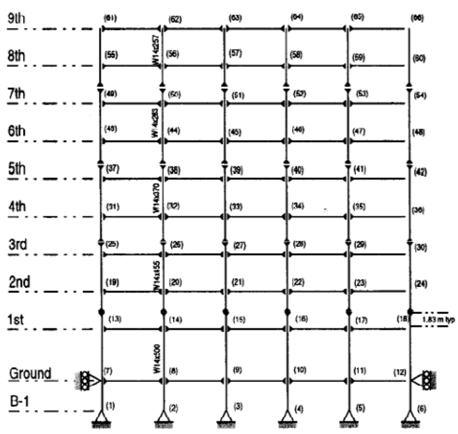

Fig. 7. Building Evaluation

A total of 14 different earthquakes with v_{max}/a ranging from 16 to 51 were used in this study(Table 1). All ground motions selected have the similar characteristic of design response spectrum of UBC 97.

To get characteristics of the design response spectrum Ca is scaled up to 1.0g. The spectral velocities are calculated from Eq. (4.1)

$$S_v = \frac{S_a}{\omega} = S_a \times \frac{T}{2\pi} \tag{4.1}$$

These data are normalized by the amplification factor(acceleration = 2.12, velocity = 1.65) for 5% damping. The v_{max}/a of design response spectrum of UBC 97 is 31.57.

4.2 Comparison of results

CANNY(version C02) program[6)] was utilized to carry out nonlinear pushover analysis.

Compared in this section are the earthquake-induced demands for the selected building determined by four analyses: mean value of time history analysis with selected earthquakes, pushover analysis using the first two force distributions in FEMA 273 and the proposed method. The results presented below evaluate the accuracy of the proposed method for a mean value of time history analysis.

For the verification, the index of performance objectives(POs) is used in the interstory drift index(IDI) proposed by Bertero[7)].

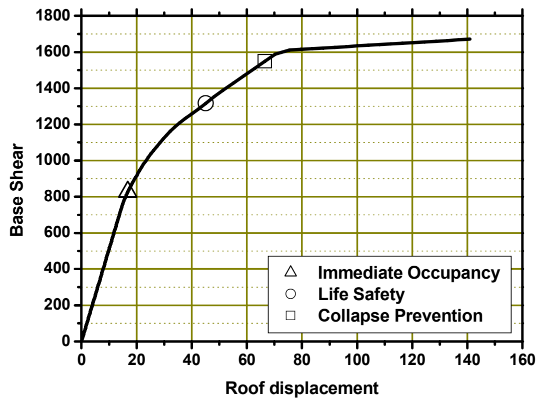

Fig. 8. Base shear of POs occurred

The principal steps of the verification procedure are as follows:

1. Perform the pushover analysis by the proposed method
2. Check the base shear of POs(Table 2).
3. Obtain maximum base shears by each ground motion.
4. Scale up or down ground motions, normalizing maximum base shear of every earthquakes with the base shear of the selected POs.
5. Obtain each story drift ratio when the maximum story drift ratio has happened.
6. Calculate the mean value and the standard deviation(SD) of each story.
7. Compare the mean value with results of

pushover analysis applying three patterns – FEMA 1 and 2 and the proposed method.

Table 2. IDI and Base shear of POs

POs	IDI(%)	Base Shear
Immediate Occupancy	0.5	827
Life Safety	1.5	1,316
Collapse Prevention	2.0	1,549

Table 3 through 5 and Fig. 9 through 11 present the results of four analyses: mean value of time history analysis with selected earthquakes, pushover analysis using the first two force distributions in FEMA 273 and proposed method. In Table 3, 4 and 5, parentheses indicte that the value is over the standard deviation.

Table 3. Results of Immediate Occupancy

Floor	Mean	SD	Proposed	FEMA1	FEMA2
1	0.358	0.095	0.452	0.442	(0.467)
2	0.433	0.095	0.500	0.438	(0.583)
3	0.496	0.074	0.503	(0.388)	(0.685)
4	0.530	0.059	0.493	(0.361)	(0.754)
5	0.487	0.068	0.433	(0.315)	(0.717)
6	0.435	0.087	0.397	(0.277)	(0.632)
7	0.434	0.127	0.437	(0.263)	(0.611)
8	0.428	0.156	0.464	(0.242)	0.583
9	0.330	0.128	0.388	(0.184)	0.432

Table 4. Results of Life Safety

Floor	Mean	SD	Proposed	FEMA1	FEMA2
1	0.560	0.240	(0.851)	(0.814)	0.799
2	0.833	0.385	(1.262)	1.052	(1.285)
3	1.035	0.490	1.453	1.018	(1.634)
4	1.142	0.538	1.520	0.861	(1.870)
5	1.062	0.510	1.429	0.594	(1.929)
6	0.899	0.352	1.277	(0.410)	(1.842)
7	0.892	0.425	1.213	(0.339)	(1.743)
8	0.809	0.639	1.157	0.306	(1.576)
9	0.648	0.719	0.905	0.231	1.217

Table 5. Results of Collapse Prevention

Floor	Mean	SD	Proposed	FEMA1	FEMA2
1	0.788	0.421	1.127	1.171	(1.221)
2	1.117	0.593	1.632	1.488	(1.903)
3	1.475	0.688	1.887	1.459	(2.388)
4	1.714	0.675	2.001	1.254	(2.732)
5	1.720	0.583	1.949	(0.871)	(2.884)
6	1.561	0.532	1.824	(0.544)	(2.866)
7	1.327	0.621	1.783	(0.377)	(2.828)
8	1.106	0.814	1.737	0.327	(2.696)
9	0.849	0.873	1.478	0.245	(2.347)

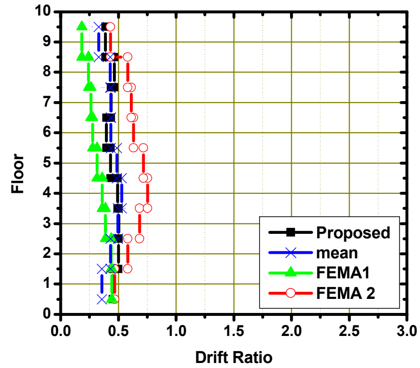

Fig. 9. Story drift ratio of Immediate Occupancy

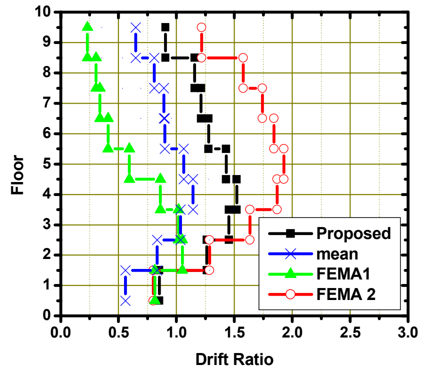

Fig. 10. Story drift ratio of Life Safety

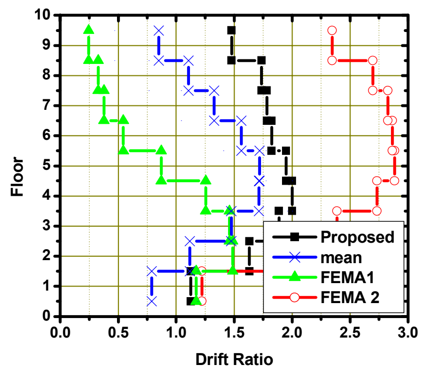

Fig. 11. Story drift ratio of Collapse Prevention

5. Conclusions

The first pattern of FEMA 273 is almost uniformly distributed over height.

The second pattern is vertical distribution of seismic forces in the Code. This force distribution is calculated from weight, height and exponent, k. This pattern is not concerned about the contribution of higher modes.

The third pattern is determined from response spectrum analysis of the building, and is concerned with the effect of higher modes. However, because the detailed characteristics of future earthquakes are not known, it is unreasonable for the load distribution to be determined from past earthquakes.

In order to complement this deficit, this investigation proposed the lateral load distribution factor determined from the design response spectrum of UBC 97.

The evaluation of the proposed method in comparison with FEMA 1 and 2 and the mean value of time history analysis led to the following conclusions

1. Immediate Occupancy level: All results of the proposed method were within the standard deviation, but the FEMA 1 and 2 were not. Maximum story drift ratio of the proposed method was nearer the mean value of the time history analysis.
2. Life Safety level: The results of the proposed method show that two stories were over the standard deviation. However, three and seven stories were over, in FEMA 1 and 2, respectively. For the maximum story drift ration, FEMA 1 had the nearest value to the time history analysis. On the other hand, for the maximum story drift ratio of the time history analysis, the proposed method and FEMA 2 occurred in 4^{th} floor while FEMA 1 occurred in 2^{nd} floor.
3. Collapse Prevention level: All results of the proposed method were within the standard deviation, while those for FEMA 1 and FEMA 2 were over at three stories and nine stories, respectively. For the Maximum story drift ratio, the error in the proposed method was similar to that of FEMA 1.

Although there are some errors on the application of the proposed method, the proposed method approximates the mean value of time history analysis

The proposed method should be evaluated for a wide range of buildings and ground motion ensembles.

Acknowledgements

This research is funded by the Samsung Corporation and by the Ministry of Construction & Transportation of Korea under Grant No C103A1040001-03A0204 -00110. This financial support is gratefully acknowledged.

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