Australian Journal of Basic and Applied Sciences, 7(4): 133-142, 2013
ISSN 1991-8178

# Effect of DifferentLateral Load DistributiononPushover Analysis

S. Taghavipour, T.A. Majid, Lau Tze Liang

School of Civil Engineering, UniversitiSains Malaysia, Seri Ampangan, 14300 Nibong Tebal,Pulau Pinang, Malaysia

**Abstract:** This study focuses on effects of different lateral load distribution in pushover analysis. In this regard, FEMA-356 guideline proposes the lateral loads using the dynamic properties of the structurefor consistency of load patterns relative to mode shape.Various lateral loads were supposed, including uniform, elastic first mode and Equivalent Lateral Force (ELF). Meanwhile, 5, 8 and 12-story frames were selected to represent the real low, medium and high rise regular reinforcement concrete structures.The results of the pushover analysis indicated that behaviour of the structures using ELF and the first mode lateral load was more realistically than those analysed using uniform lateral load.

**Key words:** Seismic Performance,Nonlinear Static Analysis,Inter-Story Drift, Coefficient Method, Roof Displacement

## INTRODUCTION

Earthquake engineering started at the end of the 19th century when some European engineering suggested designing structure with a few percent of the weight of the structure as the horizontal load. This idea of seismic design was taken up and developed in Japan at the beginning of the 20th century (Hu, Liu and Dong 1996).

For seismic performance of structure analysis is required to determine force and displacement demands in various components of the structure. A significant decision in a structural analysis is to assume whether the relationship between forces and displacements is linear or nonlinear. Linear analysis for static anddynamic loads has been used in structural design for decades. Also, nonlinearanalysis procedures were usually used, because emerging performance-basedguidelines require representation of nonlinear behaviour. Recent guidelines for seismic rehabilitation of structures pioneered the requirements for nonlinear analysis procedures, specifically FEMA 356 (2000) and the predecessor FEMA-273 (1997).

### *History of Nonlinear Static Analysis (Pushover Analysis):*

Beginning of nonlinear static analysis (Pushover Analysis) is often attributed to the work of Takeda, Sozen and Nielsen(1970), Freeman, Nicoletti and Tyrell(1975), Saiidiand Sozen(1979) and later, Fajfar and Fischinger (1988) proposed an approach in which the response of a Multi-Degree-of-Freedom (MDOF) system was determined from dynamic response analysis of an equivalent Single-Degree-of-Freedom (SDOF) system. A SDOF system is defined as that in which only one type of motion is possible, or in other words the location of the system at any instant of time can be defined in terms of single coordinate (Sen 2009). Consequently, the proposed simplified nonlinear analysis procedures and structural models are usually based on the reduction of MDOF model of structures to an equivalent SDOF system (Chopra and Goel 2002).

Takeda *et al*. (1970) proposed force-displacement relationship for calculated dynamic response of an equivalent SDOF system. An advance in the development of simplified nonlinear analysis approaches happened with introduce of many prominent nonlinear static analyses (Pushover Analysis),namely capacity spectrum Method, coefficient method, N2 method, Modal Pushover Analysis (MPA), upper-bound pushover Analysis, multi-modal pushover, pushover result combination and many more(Jan, Liu and Kao 2004; Kreslin and Fajfar 2011; Moghadam and Tso 2002; Paret *et al*. 1996).

The Coefficient Method is administered for rehabilitation and evaluation of existing structures. This method was first introduced in Federal Emergency Management Agency (FEMA) of the U.S.A (FEMA-273 1997), and was further developed and published as a pre-standard for seismic rehabilitation of buildings in FEMA-356. The method was later updated in FEMA-440 (2006). The displacement demand of the method is determined from the elastic one by using a number of modification factors based on statistical analyses. The expected maximum inelastic displacement of nonlinear MDOF system is obtained by modifying the elastic spectral displacement of an equivalent SDOF system with a series of coefficients (Lin, Chang and Wang 2004).

### *Coefficient Method of FEMA-356:*

In coefficient method the maximum inelastic displacement of an MDOF system is obtained using modifying the elastic displacement of the equivalent SDOF with an effective period ($T_e$).

**Corresponding Author:** S. Taghavipour, School of Civil Engineering, UniversitiSains Malaysia, Seri Ampangan, 14300 Nibong Tebal,Pulau Pinang, Malaysia
E-mail: taksiah@eng.usm.my

The first step of this method is constructed a bilinear representation of the capacity curve, therefore as shown in Figure 1, draw the effective elastic stiffness,$K_e$, by creating a secant line passing through the point on the capacity curve. In the Second step the effective fundamental period is calculated per equation 1.

$$T_e = T_i\sqrt{\frac{K_i}{K_e}} \tag{1}$$

where, $T_i$ is the elastic fundamental period (in seconds), $K_i$ is the elastic lateral stiffness and $K_e$ is the effective lateral stiffness.

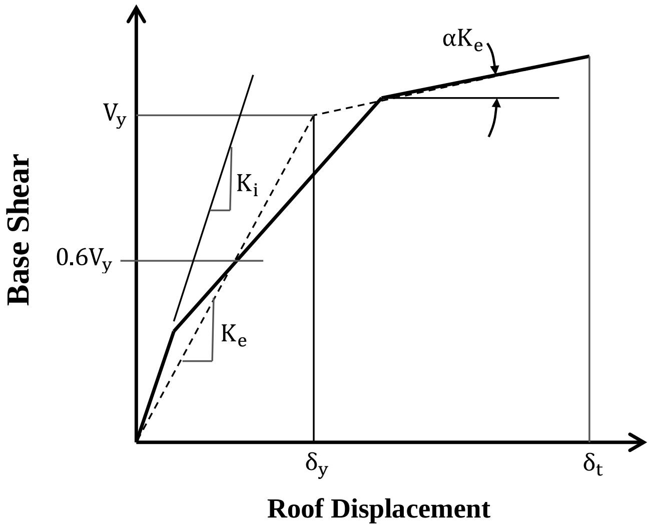


**Fig. 1:** Bilinear Representation of Capacity Curve for Displacement Coefficient Method

In addition calculated the target displacement ($\delta_t$) from equation 7.

$$\delta_t = C_0C_1C_2C_3S_a\frac{T_e^2}{4\pi^2} \tag{7}$$

where;

$\mathbf{T_e}$= Effective fundamental period as calculated in equation 6.

$\mathbf{C_0}$= Modification factor to relate spectral displacement. For estimate $C_0$ must calculated the first modal participation factor at the roof level, then calculate the appropriate value from Table 1.

$\mathbf{C_1}$=Modification factor to relate anticipated maximum inelastic displacements to displacements obtained from linear elastic responseand is given by;

$$C_1 = 1 \qquad T_e \geq T_0 \tag{8}$$

$$C_1 = [1 + (R-1)\,T_0/T_e]/R \qquad T_e \leq T_0 \tag{9}$$

where $T_0$ is the characteristic period of the response spectrum defined as the transition from constant acceleration section to the constant velocity section and R is the ratio of inelastic strength demand to calculated yield strength coefficient obtained as following equation:

$$R = \frac{W.S_a}{g.V_y}.\frac{1}{C_0} \tag{10}$$

where $V_y$ Yield strength obtained by the capacity curve and W total dead load and expected live load (usually 25% of the floor live load considered).

$\mathbf{C_2}$= Modification factor to represent the effect of hysteresis shape on the maximum displacement response. Values of C2 for diverse framing type and performance levels are listed in Table 2.

$\mathbf{C_3}$= Modification factor to represent increased displacements because of dynamic $P-\Delta$ effects. For structures with positive post yield stiffness, $C_3$ must be set equivalent to 1.0. For structures with negative post yield stiffness, $C_3$ must be calculated by Equation 11.

$$C_3 = 1 + \frac{|\alpha|(R-1)^{3/2}}{T_e} \qquad (11)$$

where R and $T_e$ are obtained from above and $\alpha$ is the post yield stiffness ratio to elastic stiffness from characterized nonlinear force-displacement relation by bilinear relation.

**Table 1:** Values for Modification Factor $C_0^1$

| Number of Stories | Shear Buildings[2] | Other Buildings |
|---|---|---|
| 1 | 1.0 | 1.0 |
| 2 | 1.15 - 1.2 | 1.2 |
| 3 | 1.2 | 1.3 |
| 4 | 1.2 – 1.3 | 1.4 |
| 10+ | 1.2 – 1.3 | 1.5 |
| 1. Linear interpolation should be used to calculate intermediate values.<br>2. Buildings in which, for all stories, inter-story drift decreases with increasing height. | | |

**Table 2:** Values of Modification Factor $C_2$

| Structural Performance | $T \leq 0.1$ sec. | | $T \geq T_s$ | |
|---|---|---|---|---|
| | Framing Type 1[1] | Framing Type 2[2] | Framing Type 1 | Framing Type 2 |
| Immediate Occupancy | 1.0 | 1.0 | 1.0 | 1.0 |
| Life Safety | 1.3 | 1.0 | 1.1 | 1.0 |
| Collapse Prevention | 1.5 | 1.0 | 1.2 | 1.0 |
| 1. Structures that more than 30 percent of story shears at any level is resisted by a combination of ordinary moment resisting frames, concentrically braced frames, frames with partially restrained connections, tension only braces, unreinforced masonry walls, shear critical, piers and spandrels of reinforced concrete or masonry.<br>2. All frames not assigned to Farming type 1. | | | | |

***Nonlinear Time-History Analysis:***

Nonlinear time-history analysis is a step by step analysis of the dynamical response of a building to an identified loading that might change with time. Nonlinear time-history analysis is used to determine the dynamic response of a structure to random loading. The dynamic stability equation is obtained via Equation 12.

$$K\,u(t) + C\,\dot{u}(t) + M\,\ddot{u}(t) = r(t) \qquad (12)$$

where k is the stiffness matrix, c is the damping matrix, m is the diagonal mass matrix, r is the applied load and $u, \dot{u}, \ddot{u}$ are the displacements, velocities, and accelerations of the structure, respectively.

The nonlinear response of buildings is very sensitive to the structural modelling and ground motion characteristics. Consequently, a set of representative ground motion records with different characteristics in distance, magnitude, frequency and duration should be used. The input motions used in this research is included the two horizontal components of 10 ground motion with different magnitude (ranging from 6.4 to 7.6) and different Pick Ground Acceleration (PGA) obtained from Pacific earthquake engineering research centre (PEER 2006). Furthermore, as show in Table 3 five of the ground motion has been selected as a far fault ground motion recorded with more than fifteen km distance and five of the other selected ground motion recorded with less than fifteen km distance.

Table 3: Parameters of Ground Motion Used in Analysis

| No | Type[1] | Earthquake | Mw | Station Name | Distance (km) | Component | PGA (g) |
|---|---|---|---|---|---|---|---|
| 1 | FF | Chi-Chi Taiwan 1999 | 7.6 | CWB 99999 | 26 | TCU045 | 0.47 |
| 2 | FF | Northridge 1994 | 6.7 | Castaic-Old Ridge Route | 21 | ORR360 | 0.48 |
| 3 | FF | Imperial Valley 1979 | 6.5 | Calipatria Fire Station | 25 | H-CAL225 | 0.10 |
| 4 | FF | San Fernando 1971 | 6.6 | Castaic-Old Ridge Route | 22 | ORR291 | 0.29 |
| 5 | FF | Victoria Mexico 1980 | 6.3 | SAHOP Casa Flores | 39 | CPE045 | 0.07 |
| 6 | NF | Cape Mendocino 1992 | 7 | Petrolia | 4 | PET090 | 0.62 |
| 7 | NF | Erzican, Turkey 1992 | 6.7 | Erzican | 9 | ERZ-EW | 0.48 |
| 8 | NF | Landers 1992 | 7.3 | Joshua Tree | 11 | JOS000 | 0.24 |
| 9 | NF | Kobe, Japan 1995 | 6.9 | KJMA | 0.96 | KJM000 | 0.7 |
| 10 | NF | Tabas, Iran 1978 | 7.3 | Tabas | 2 | TAB-TR | 0.81 |
| 1. FF: Far-Fault; NF: Near-Fault. | | | | | | | |

***Description of Case Study Modelling:***

This studyproposed three reinforced concrete frames with different heights (5, 8 and 12-stories) which used to cover a broad range of fundamental periods, as shown in Figure 2. The presented frames were designed by using ACI 318-05 (2005), then modelled as two dimensional frames using SAP2000 version 14 software (CSI 2010). The structural models were defined based on centreline dimensions that assigned columns and beams among the nodes. Rigid floor diaphragm was also assigned at each story level.Moreover, the mass centre of each story and the seismic mass of the frames were lumped. Supposed gravity loads in pushover analysis are including dead loads and 25 percent of live loads that were assigned in all frames. Furthermore, P-Δ effect for pushover analysis is not taking to account. Free vibration analyses were carried out to obtain mode shapes and elastic periods of the frames. The Table 4 lists dynamic properties of the frames.

**Table 4:** Dynamic Properties of Case Study Frames

| Frame | Period ($T_n$) | | | Modal Mass Factor ($\alpha_n$) | | |
|---|---|---|---|---|---|---|
| | $T_1$ | $T_2$ | $T_3$ | $\alpha_1$ | $\alpha_2$ | $\alpha_3$ |
| 5-Story RC | 0.956 | 0.302 | 0.143 | 0.795 | 0.116 | 0.076 |
| 8-Story RC | 1.195 | 0.424 | 0.189 | 0.721 | 0.146 | 0.090 |
| 12-Story RC | 1.690 | 0.605 | 0.282 | 0.726 | 0.132 | 0.092 |

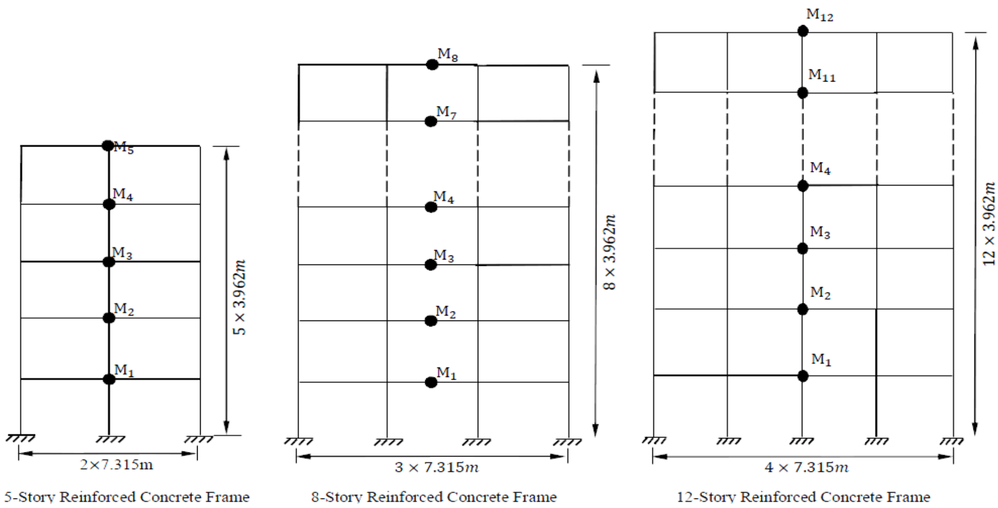


**Fig. 2:** Case Study Reinforcement Concrete Frames

***Frame Hinge Properties:***

SAP2000(CSI 2010) can be inserted plastic hinges at every location along the clear length of every frame element object. Every hinge represents concentrated post-yield conduct in single or multi degrees of freedom. Hinges only have an effect on the behaviour of the structure in pushover analysis and nonlinear time history (direct-integration) analyses. Uncoupled moment (M2 and M3), axial force (P), torsion (T) and shear hinges (V2 and V3) can be assigned in element ends as well as at any number of locations along the span of the frame element. Furthermore, can be assigned a coupled P-M2-M3 (PMM) hinge that yield based on the interaction of bending moments and axial force at the hinge location. In this study, PMM and M3are assigned to the columns and beams, respectively.

Three types of hinge properties defined in SAP2000 consisting of user-defined hinge properties, default hinge properties and generated hinge properties.It should be noted that only user-defined hinge properties and default hinge properties can be assigned to frame elements, which default hinge properties were assigned to all frame elements in this study. Default hinge properties in SAP2000 are presented based on FEMA-356 code.

***Lateral Load Distribution:***

FEMA-356 guidelines proposed the lateral loads based on the dynamic properties of each structure. Furthermore for the sake of consistency load patterns relative to mode shape, FEMA-356 suggested different lateral load distribution. Therefore first mode, Equivalent Lateral Force (ELF) and uniform lateral load distribution are considered for all structures.

- "**Uniform**" lateral load pattern is the load that assigned to each story which is proportional to the mass at that story:

$$F_i = \frac{m_i}{\sum m_i} \tag{13}$$

where $F_i$ is the lateral force at i-th storey and $m_i$ is the mass of i-th story.

- "**Elastic First Mode**" lateral load pattern is the lateral force that assigned to each story which is depended to the product of the magnitude of the elastic first mode and the mass in each story.

$$F_i = \frac{m_i \phi_i}{\sum m_i \phi_i} \qquad (14)$$

where $\phi_i$ is magnitude of the elastic first mode at i-th story.

- "**Equivalent lateral force (ELF)**" proposed by Eurocode 8 (CEN 2005)is the force at any story which is dependent to the height of story and mass in that story:

$$F_i = \frac{m_i h_i^k}{\sum m_i h_i^k} \qquad (15)$$

where$h_i$ is the height of the i-th story above the base and k is equal to 1 for main period $T_1 \leq 0.5$ sec and k is equal to 2 for main period $T_1 \geq 2.5$ sec.

***Computation Target Displacement:***

Based on the effective lateral stiffness ($K_e$) and elastic lateral stiffness ($K_i$) defined as the initial elastic gradient of the capacity curve (pushover curve) and initial elastic gradient of the bilinear idealization, the effective fundamental period ($T_e$), was calculated from Equation 1. Moreover obtained modification factor to relate spectral displacement ($C_0$) from Table 1 and calculated modification factor to relate anticipated maximum inelastic displacements to displacements ($C_1$) from Equation 3 and 4. Also obtained modification factor to represent the effect of hysteresis shape on the maximum displacement response ($C_2$). Then insert the values obtained above into Equation 2 to obtain target displacement ($\delta_t$). Table 5 to Table 7 show results of proposed values and target displacement for different lateral load distribution.

**Table 5:** Computation of Target Displacement for ELF Lateral Load Distribution

| Story | $C_0$ | $C_1$ | $C_2$ | $C_3$ | $S_a$ | $T_e$ | $T_i$ | $K_i$ | $K_e$ | $\delta_t$ |
|---|---|---|---|---|---|---|---|---|---|---|
| 5-Story | 1.296 | 1 | 1 | 1 | 0.6743 | 0.9491 | 0.9491 | 16011 | 16011 | 0.194 |
| 8-Story | 1.4187 | 1 | 1 | 1 | 0.4365 | 1.4663 | 1.1696 | 35431 | 22542 | 0.328 |
| 12-Story | 1.358 | 1 | 1 | 1 | 0.3217 | 1.9894 | 1.6628 | 38580 | 26950 | 0.426 |

**Table 6:** Computation of Target Displacement for Uniform Lateral Load Distribution

| Story | $C_0$ | $C_1$ | $C_2$ | $C_3$ | $S_a$ | $T_e$ | $T_i$ | $K_i$ | $K_e$ | $\delta_t$ |
|---|---|---|---|---|---|---|---|---|---|---|
| 5-Story | 1.2387 | 1 | 1 | 1 | 0.7456 | 0.8584 | 0.8499 | 21088 | 20672 | 0.168 |
| 8-Story | 1.3327 | 1 | 1 | 1 | 0.533 | 1.2008 | 1.0201 | 49965 | 36058 | 0.252 |
| 12-Story | 1.311 | 1 | 1 | 1 | 0.3534 | 1.8108 | 1.452 | 54383 | 34967 | 0.375 |

**Table 7:** Computation of Target Displacement for Elastic First Mode Lateral Load Distribution

| Story | $C_0$ | $C_1$ | $C_2$ | $C_3$ | $S_a$ | $T_e$ | $T_i$ | $K_i$ | $K_e$ | $\delta_t$ |
|---|---|---|---|---|---|---|---|---|---|---|
| 5-Story | 1.2943 | 1 | 1 | 1 | 0.6694 | 0.9561 | 0.9561 | 15805 | 15805 | 0.195 |
| 8-Story | 1.4305 | 1 | 1 | 1 | 0.4261 | 1.5019 | 1.1952 | 33590 | 21272 | 0.339 |
| 12-Story | 1.3595 | 1 | 1 | 1 | 0.3151 | 2.0312 | 1.69 | 37141 | 25712 | 0.435 |

***Comparison the Results of Selected Lateral Load Distribution:***

The correctness of selected lateral loads to predict the real behaviour of the structure was evaluated in this study. Therefore, nonlinear dynamic analysis method is used to evaluate the lateral loads. This evaluation can be performed via comparingbehaviour, displacement and drift of structures.

***Behaviour of Structure:***

Behaviour of structure under lateral load is shown using capacity curve in which plotted displacement variation versus lateral load changes. Capacity curves of coefficient method should be idealized using the different target displacements which are obtained from Table 5 to Table 7and evaluation using nonlinear time-history analysis (Figure 3 to 5).

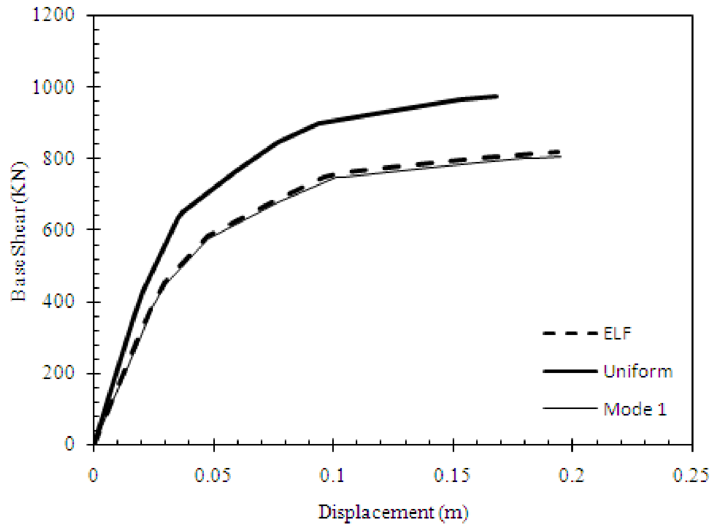


**Fig. 3:** Capacity Curve of 5-Storey Frame in coefficient method

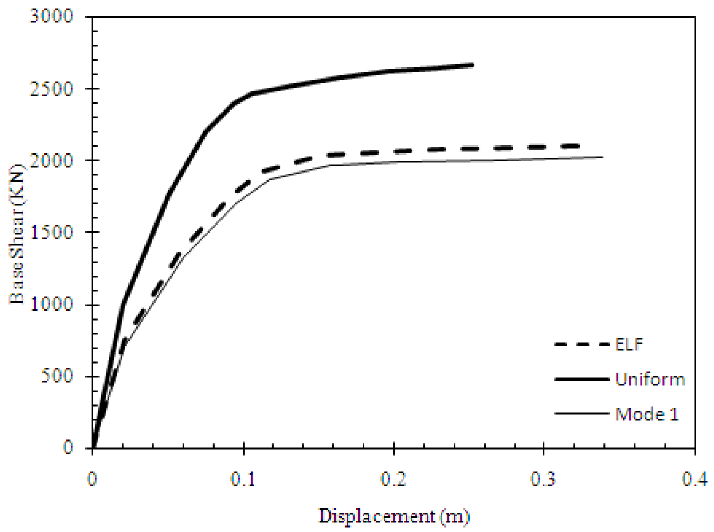


**Fig. 4:** Capacity Curve of 8-Storey Frame in coefficient method

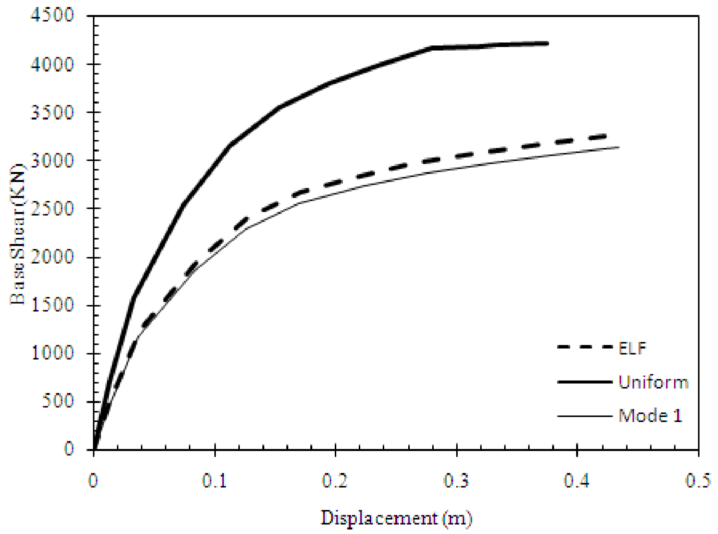


**Fig. 5:** Capacity Curve of 12-Storey Frame in coefficient method

As can be seen in above figures, capacity curve in coefficient method for ELF and first mode lateral load distribution are similar, while uniform lateral load with respect to the larger base shear, shows lower displacement.

***Storey Displacement:***

Storey displacements obtained from lateral loads are different for each frame. The selected lateral loads are evaluated by using selected ground motions. For this purpose, the floor displacements from the three lateral load distributions of coefficient methodare compared with the obtained values from the nonlinear time-history analysis.The storey displacement of 5, 8 and 12-story frames are illustrated in Figures 6 to 8.

The results show that displacements obtained from 5 and 8-story frame for ELF and First mode lateral load are overestimated at all stories especially in the middle stories. Furthermore, the displacement for uniform lateral load is slightly overestimated at lower story and underestimated at upper stories (Figure 6 and 7). The results of displacement in 12-story frame show that story displacements for uniform lateral load in comparison with ground motions are underestimated at all stories. Also, the results show that displacements for ELF and First mode lateral load are almost similar in comparison of ground motions.In general it can be concluded coefficient method ELF and firs mode lateral load, which is applicable for structures analysis based on the fundamental mode, provided rational estimate of the story displacement.

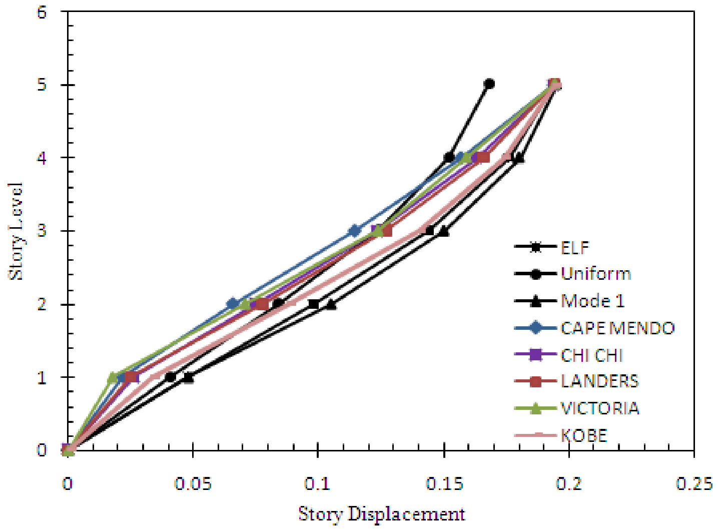


**Fig. 6:** Storey Displacement of 5-Story Frame

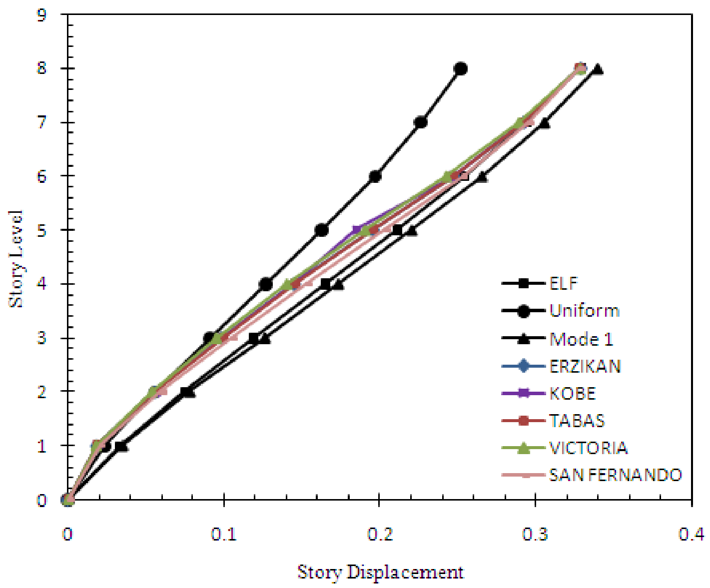


**Fig. 7:** Storey Displacement of 8-Story Frame

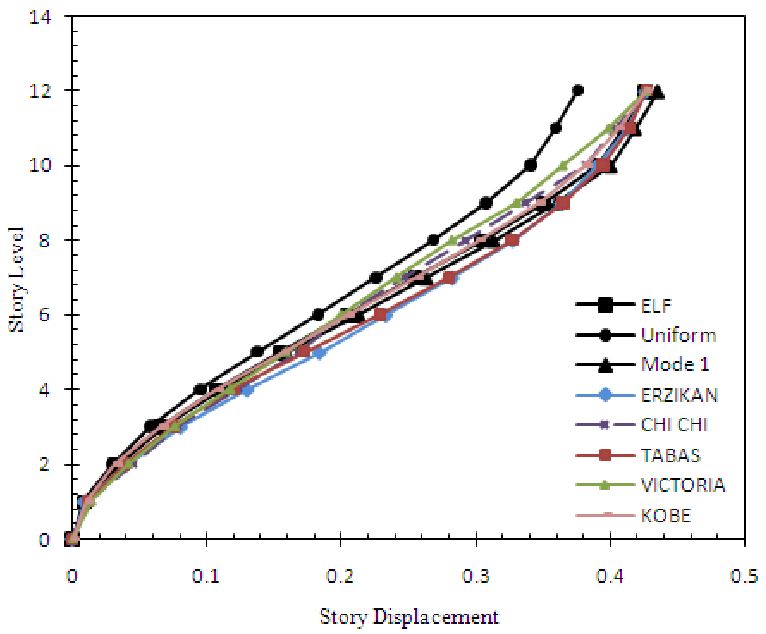


**Fig. 8:** Storey Displacement of 12-Story Frame

***Inter-Storey Drift:***

Drift is the comparative horizontal displacement of two adjoining storey in structure. Inter-storey drift is a percentage of the story height separating in the two adjoining floor. The selected lateral loads are evaluated by using selected ground motions. For this purpose, the inter-storey drift from the three lateral load distributions of coefficient method are compared with the obtained values from the nonlinear time-history. The inter-storey drift of 5, 8 and 12-storey frames are illustrated in Figures 9 to 11.

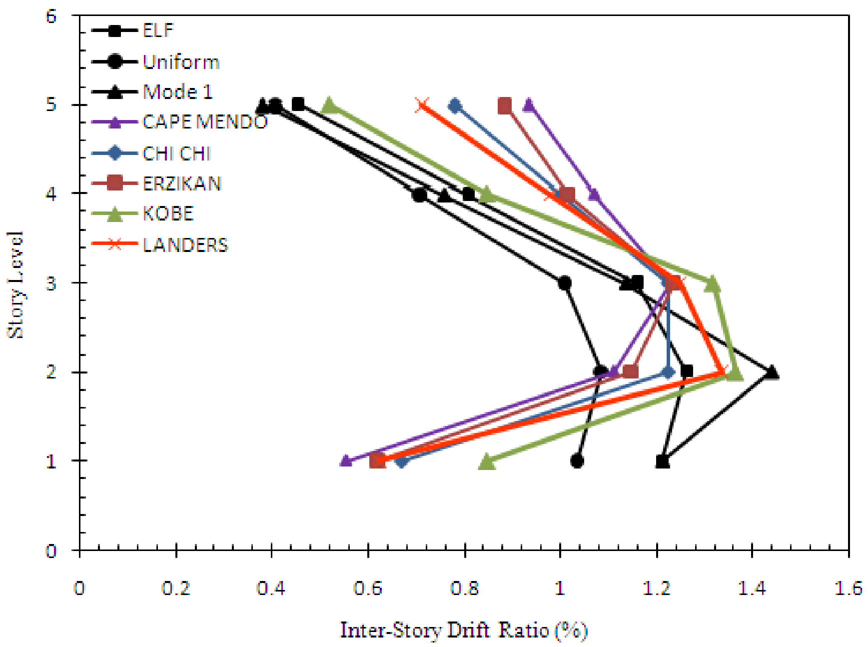


**Fig. 9:** Inter-Story Drift of 5-Story Frame

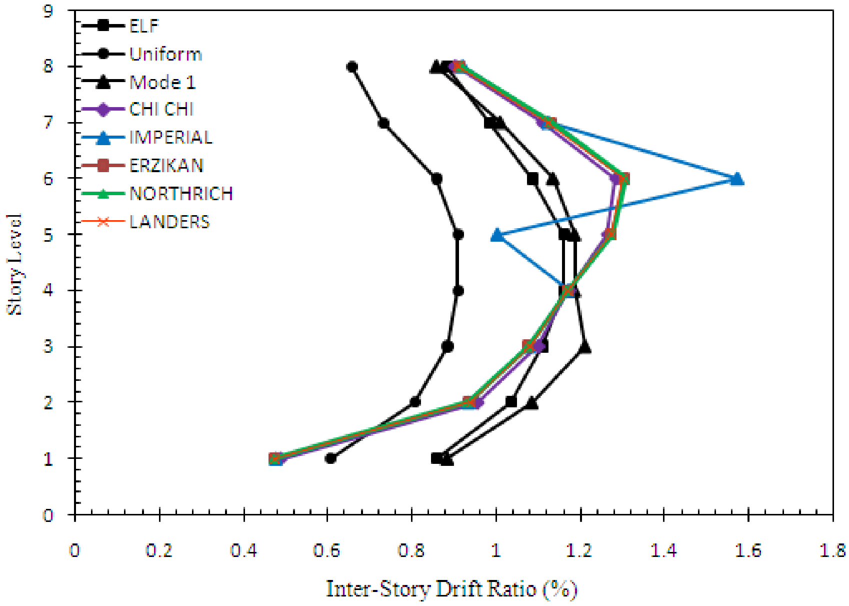


**Fig. 10:** Inter-Story Drift of 8-Story Frame

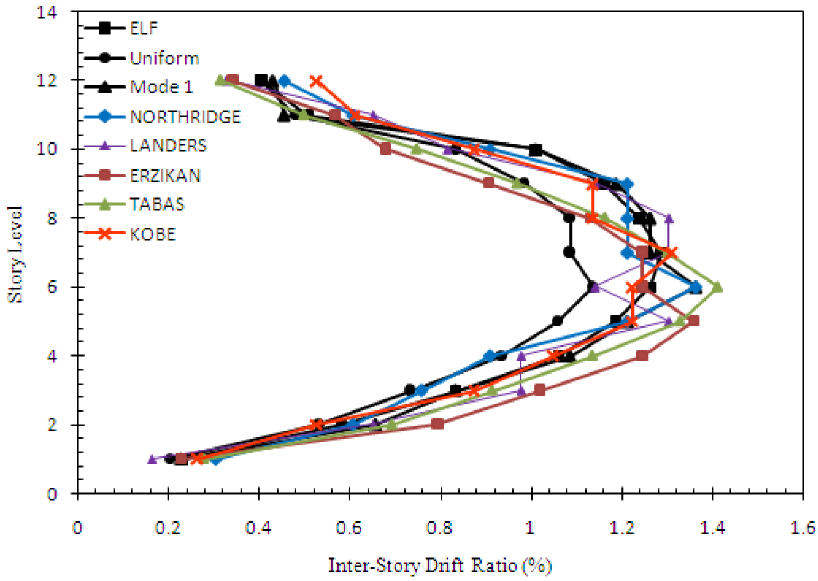


**Fig. 11:** Inter-Story Drift of 12-Story Frame

In 5-story frame, the comparison of inter-story drifts from the all lateral loads and the ground motions (Figure 9) shows that the all lateral loads, lead to gross overestimation of drifts in the lower stories. Also, results show that all lateral loads lead to slightly underestimation of drift in the upper stories.

In 8-story frame, the comparison of results shows that the all lateral loads lead to gross overestimation of drift in the lower storey (Figure 10).Results show, although uniform lateral load lead to gross underestimation of drift in the upper story, ELF and first mode lateral load lead to almost similar estimation of drift in the upper storey. For example the selected pushover methods drift results in comparison with Erzikan ground motion drift result in upper storey, shows that there are 4, 28 and 6 percent difference for ELF, uniform and first mode lateral load, respectively.

In 12 storey frame, the comparison of results shows, although uniform lateral load lead to almost similar estimation of drift in the lower and upper storey, lead to underestimation drift in the middle storey (Figure 11). Also, the results show that the ELF and first mode lateral load distribution, lead to almost similar estimation of drift in the all stories.

***Conclusion:***

Motivated by previous researches correspond to that issue, this study presents effect of different lateral load distribution on pushover analysis for the different type of structures. This research used the 5, 8 and 12-story frames to represent the real low, medium and high rise regular reinforced concrete structure. Furthermore, this research using nonlinear time-history analysis for assessment of selected lateral loads in coefficient method. Selected lateral loads are including ELF, uniform and elastic first mode.

Based on results obtained by comparing the selected lateral load distribution and nonlinear time-history the following conclusion can be made that fulfilled the objective of this research:

The comparison of capacity curves from selected lateral loads show that the behaviour of selected case study frame for ELF and First lateral load are almost similar. Also, the storey displacements of selected lateral loads in comparison with ground motions reveal that, ELF and First mode lateral load are almost similar, while that of at the roof level for uniform lateral load are different about 20 percent. In addition, the comparing of inter-storey drift from selected lateral loads reveal that ELF and First mode lateral load lead to almost similar estimation of drift in the selected frames, especially in high rise frames, while uniform lateral load lead to inaccurate estimation of selected frame. Therefore, it can be concluded that the ELF and First mode lateral load can be used for analysis of structure, while uniform lateral load cannot show the real behaviour of structure under earthquake.

## REFERENCES

ACI-318R-05. 2005. "Building code requirements for structural concrete (ACI 318-05) and commentary (ACI 318R-05)."

CEN. 2005. "Eurocode 8: Design of structures for earthquake resistance–Part 1: General rules, seismic actions and rules for buildings." CEN Brussels.

Chopra, A.K. and R.K. Goel, 2002. "A modal pushover analysis procedure for estimating seismic demands for buildings." *Earthquake Engineering & Structural Dynamic.,* 31(3): 561-582.

CSI. 2010. " SAP2000 (v14.0.0)-linear and nonlinear static and dynamic analysis and design of three-dimensional structures. ." *Berkeley: Computer & Structures,.*

Fajfar, P. and M. Fischinger, 1988. "N2-A method for non-linear seismic analysis of regular buildings." *Proceedings of 9WCEE* 5:111-116.

FEMA-273. 1997. "NEHRP Guidelines for the Seismic Rehabilitation of Buildings. FEMA 273, Federal Emergency Management Agency, Washington, D.C.".

FEMA-440. 2006. "Improvement of Nonlinear Static Seismic Analysis Procedures."

FEMA., 2000. *prestandard and commentary for the seismic rehabilitation of buildings*. Washington, D.C.: Federal Emergency Management Agency.

Freeman, S., J. Nicoletti and J. Tyrell, 1975. "Evaluations of existing buildings for seismic risk-A case study of Puget Sound Naval Shipyard, Bremerton, Washington."

Hu, Y., S.C. Liu, and W. Dong, 1996. *Earthquake Engineering*: E&FN Spon.

Jan, T.S., M.W. Liu and Y.C. Kao, 2004. "An upper-bound pushover analysis procedure for estimating the seismic demands of high-rise buildings." *Engineering Structures.*, 26(1): 117-128.

Kreslin, M. and P. Fajfar, 2011. "The extended N2 method taking into account higher mode effects in elevation." *Earthquake Engineering & Structural Dynamics.*, 40(14): 1571-1589.

Lin, Y.Y., K.C. Chang and Y.L. Wang, 2004. "Comparison of displacement coefficient method and capacity spectrum method with experimental results of RC columns." *Earthquake Engineering & Structural Dynamics.,* 33(1): 35-48.

Moghadam, A. and W. Tso, 2002. "A pushover procedure for tall buildings."

Paret, T.F., K.K. Sasaki, D.H. Eilbeck and S.A. Freeman, 1996. "Approximate inelastic procedures to identify failure mechanisms from higher mode effects."

PEER. 2006. "Pacific earthquake engineering research centre,Strong ground motion database. http://peer.berkeley.edu/." *University of California, Berkeley*.

Saiidi, M and M.A. Sozen, 1979. "Simple and complex models for nonlinear seismic response of reinforced concrete structures."

Sen, T.K., 2009. *Fundamentals of Seismic Loading on Structures*: John Wiley & Sons.

Takeda, T., M.A. Sozen and N.N. Nielsen, 1970. "Reinforced concrete response to simulated earthquakes." *Journal of the Structural Division.*, 96(12): 2557-2573.