Pushover Analysis for Estimating Resonance Factor of Tall RC Frames with Steel Eccentric Bracing

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Abstract: Since the commonplace designs are directed on linear analyses, a practical estimation on actual displacements and deformations which are taken out by linear analyses could be handled by adding a factor. On the other word, displacement resonance factor (DRF) supplemented to linear analysis responses may be held superior in order to determine actual displacements concerning time and cost concerns. According to provision of Iranian Code for seismic design, displacement resonance factor (DRF) receives same values for all structural systems. To make sure for required modifications on seismic design codes, one hundred concrete moment resisting frames with eccentric braces, designed based on the Iranian National Seismic Standard, has been considered to capture seismic parameters by performing two-dimensional nonlinear pushover analyses. Pushover Analyses have been conducted using SAP-2000 program, which can consider material nonlinearities almost near reality. In this case the applied forces have been considered as the lateral forces of the Seismic Standard. Seismic parameters including overstrength, ductility and behavior factors are excerpted by following Young Theory. Also studies based on Newmark and Hall practice has been pursued to withdraw coefficient of force reduction due to ductility. Concentrating on tall buildings, variation of DRF has been illustrated concerning bracing kind of spans, length of link beam and height of structure. Analytical results show that in the case of reminded frames the value of DRF can be much higher than that recommended by Iranian Code. On the suggestion side, this problem can be devised by multiplying a coefficient of 1.54 to the former resonance factor.
[Mohammad Reza Hoseinzadeh, Mussa Mahmoudi, Ali Edalatbehbahani, Seyed Amirodin Sadrnejad, Iraj Rasoolan. Pushover Analysis for Estimating Resonance Factor of Tall RC Frames with Steel Eccentric Bracing. Journal of American Science 2011;7(3):773-778]. (ISSN: 1545-1003). http://www.americanscience.org.

Keywords: Seismic design, Pushover analysis, Moment resisting frame, Tall buildings

1. Introduction

It's important to understand that seismic provisions in most building codes are intended to protect life and reduce property damage but not completely eliminate losses. That's recommended that structures should be able to resist minor earthquakes without damage, also resist moderate earthquakes without structural damage but with some nonstructural damage. On top of that, structures' attitude toward the major earthquake should be obliged for withstanding structural and non-structural damages. Generally, raising drift initiates structural damages though non-structural ones also leave stability in more critical levels. This put more emphasize to control lateral drift (Yaun et al., 2009). Buildings that are most vulnerable to lateral forces induced by seismic waves are unreinforced masonry, brick and mortar, and adobe constructions. Small wood frame structures are usually the safest as long as they are securely anchored to their foundations. Steel frame or reinforced-concrete construction methods are least hazardous for multi-story buildings or other tall structures (Hoseinzadeh, 2010). Focusing on tall buildings, this study has been conducted under provisions of both Seismic Resistant Design of Buildings (Iranian Code) and Rehabilitation of Existing Buildings (Code 360). Description of specific earthquake is the point which both criteria are met concerning intermediate important structures. Thus a performance level is established on life security in a way that failure is permitted for a structure providing does not lead to hazard in life safety. On the other side, specific earthquake is referred to a design base earthquake with exceeding probability of less than ten percent in 50 years in accordance with provisions of Iranian Code while Code 360 relates this notion to a risk level-1 earthquake. Presenting probability of ten percent in 50 years, the later comment keeps abreast with a return period of 475 years (Iranian code, 2005; Code

360, 2006). In summing up, considering risk level-1 earthquake can't be greater than design base earthquake, more rigorousness is anticipated on designs under Iranian Code. But lining on more detailed instructions upon checking lateral drifts, Code 360 seems to spend more severity for embracing performance level.

On the other side, under certain conditions the most conventional designs are permitted for linear analyses such as static equivalence method to draw estimation on actual displacements. Based on recommendations of Iranian Code while structures are authorized for static equivalent method, displacement resonance factor of (0.7R) together with displacements fulfilled by assumption of elastic behavior of structure shall be replaced for actual displacements. Provided regarding $P-\Delta$ effects, Equation 1 can be used to confine the displacements:

$$\begin{aligned} \Delta_m &< 0.025h \qquad for: T < 0.7s \\ \Delta_m &< 0.020h \qquad for: T > 0.7s \end{aligned} \tag{1}$$

Where h and T are height and vibration period of structure respectively. Also Δ_m is referred to actual values of base lateral drift.

This method is placed pivot point of study to make sure on the required modification of the seismic design codes for building systems. In this study one hundred concrete moment resisting frames by eccentric braces designed based on Iranian Code (issued in 1988 and revised in 2005) and Code 360 (2006) has been considered under linear equivalent static and nonlinear static (pushover) analyses.

2. Materials and Methods

Generally speaking actual displacements could be derived based either on non-linear analysis method or simplified methods. Considering time and cost concerns, simplified methods may be held superior in order to determine actual displacements. Static equivalent method supplemented with resonance factor is usually discussed as a conspicuous option. Iranian Code (2005) has broadened this method to the following occasions:

A. Regular buildings with the height less than 50 meters including base story.
B. Irregular buildings with five story limitation or the height less than 18 meters including base story.
C. Buildings with variation in lateral stiffness in which the upper levels hold less stiffness than that of the bottom side and providing first, both parts have regular configurations. Secondly, average stiffness of bottom levels is valued at least ten times greater than that of above and at the end, Fundamental period of vibration surpasses 1.1 times of the upper level supposing this part is fixed at the end also imagined separately.

These conditions bring large scope of structures into legitimacy of static equivalent method. Thus estimation on actual displacements of these structures could be written as:

$$\Delta_m = 0.7R\,\Delta_w \tag{2}$$

Where R is behavior factor of structure, and Δ_w is lateral drift from elastic analysis considering ductility reduction factors. In spite of controversy on the values of coefficients 0.02 and 0.025 (at Eq. 1), more intense is placed here on validation of Eq. 2. Now this equation can be rewritten as:

$$\frac{\Delta_m}{\Delta_w} = 0.7R \tag{3}$$

Setting right hand of Eq. 3 by displacement resonance factor yields:

$$C_d = \frac{\Delta_m}{\Delta_w} \tag{4}$$

Now parameter X can be defined as:

$$X = \frac{C_d}{R} \tag{5}$$

In accordance with Iranian Code the value of X should be taken as 0.7 generally for all buildings designed by static equivalence method without reminding any special structural system. This study is developed to include striking elaborations around the value of X for concrete moment resisting frames with eccentric braces. Accepting general behavior of a conventional structure (Fig .1) also following Young theory set the following equations:

$$C_d = \mu_s \Omega Y \tag{6}$$

$$R = R_\mu \Omega Y \tag{7}$$

Where R_μ, μ_S, Ω, Y respectively are force reduction factor due to ductility, ductility of structure, overstrength factor and allowable stress factor. Substituting equations (6), (7) into (5) yields:

$$X = \frac{\mu_s}{R_\mu} \tag{8}$$

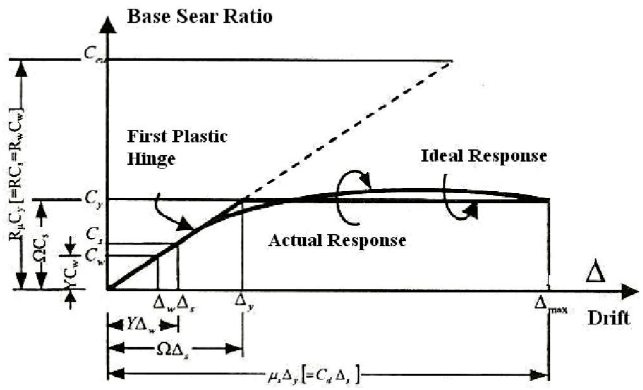

Figure 1. General behavior of a conventional structure (Tasnimi et al., 2007).

By idealizing behavior curve of structure to absolute elastic-plastic phases, ductility factor of structure could be defined as following quotient:

$$\mu_S = \frac{\Delta_{max}}{\Delta_y} \qquad (9)$$

Based on Newmark and Hall performance the relation between R_μ and μ_S is expressed as (Tasnimi et al, 2007):

$$T_e \le 0.03_{Sec} \rightarrow R_\mu = 1.0 \qquad (10)$$

$$0.03_{Sec} < T_e < 0.12_{Sec} \rightarrow R_\mu = (T_e - 0.03)\frac{\sqrt{2\mu_s - 1} - 1}{0.09} + 1$$

$$0.12_{Sec} \le T_e \le 0.50_{Sec} \rightarrow R_\mu = \sqrt{2\mu - 1}$$

$$0.50_{Sec} < T_e < 1.00_{Sec} \rightarrow R_\mu = 2(T_e - 1)[\mu - \sqrt{2\mu_s - 1}] + \mu_s$$

$$1.00_{Sec} \le T_e \rightarrow R_\mu = \mu$$

Where T_e is effective fundamental period of structure. Next step focuses on extracting the coefficients of R_μ and μ_S which are dependant on maximum of roof lateral relative displacement, fundamental period and roof lateral drift at yielding moment. Fundamental period is directly derived by solving the characteristic equation while acquiring aid of Eq. 11 sets value of initial vibration period.

$$\left|[K] - \omega^2 [M]\right| = 0 \qquad (11)$$

$$T_i = 2\pi / \omega_i \qquad (12)$$

Where $[K],[M],\omega$ respectively are stiffness matrix, mass matrix and modal frequency of structure. T_i is defined as fundamental (initial) period and ω_i is initial frequency of structure. Effective fundamental period needs referring to bilinear diagram belonging to Roof Drift/Shear Base curve (Figs. 2, 3) in case of structure under lateral displacement up to the target point.

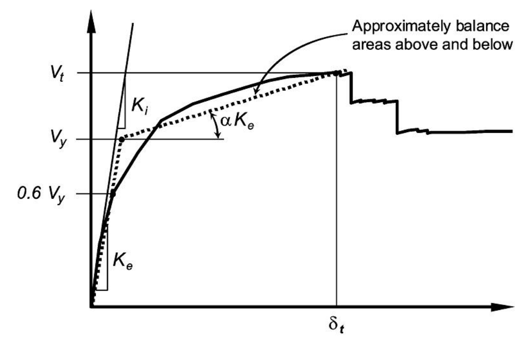

Figure 2. Idealized force displacement curves, Positive post-yield slope (FEMA-356).

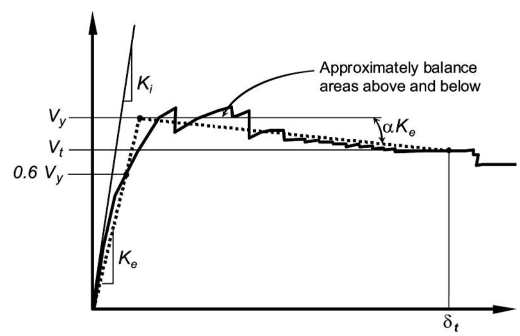

Figure 3. Idealized force displacement curves, Negative post-yield slope (FEMA-356).

Figs. 2, 3 employ K_i, K_e respectively as initial stiffness and effective stiffness. V_y is referred to shear base of total yielding, also δ_y, δ_t are displacement at yield base shear and target displacement. Now effective fundamental period can be calculated from:

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \qquad (13)$$

Roof lateral drift at yielding moment could be derived as:

$$\Delta_y = \frac{V_y}{K_e} \tag{14}$$

To satisfy performance level, maximum of lateral drift has been restrained to target displacement of structure and can be written as:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \tag{15}$$

Where g, S_a respectively are acceleration due to gravity and spectral acceleration. c represents a bunch of correction factors which could be extracted according to the Code 360.

3. Results

To draw a concrete conclusion, wide variety of concrete frame structures has been scrutinized under earthquake designs. One hundred braced moment resisting frames giving variety in number of stories (8, 10, 12, 14, 15), bracing kinds of spans, length of link beam (0.5, 1, 1.5, 2 meters) and number of spans (1,3) have been investigated under elastic-inelastic analysis procedures. Analysis methodology covering provision of Iranian Code has been followed by using SAP-2000 (version 12) computer program which consider both gravity and lateral loads. Details of the frames profile in this study are presented at Tables 1, 2.

Table 1. Characteristics of frames.

Zone Type	High Risk Level
Ground Type	Type 2
Ductility of building	Intermediate
Frame Type	Middle
Length of Loading Span	4 m
Length of Spans	4 m
Height of Stories	3.2
Dead Load	550 kg/m^2
Live Load	200 kg/m^2
Equivalent Partition Load	100 kg/m^2

4. Discussion

Because of attending wide variety of buildings and weighty structural elaborations, presenting results has been abbreviated only to averages on ultimate results. Five models applied for bracing the spans are plotted at Fig. 4. Figs. 5, 6, 7 present variation of DRF concerning bracing kind of spans, length of link beam and height of building with this in mind dashed and solid lines represent the results respectively for Iranian Code and analysis.

Table 2. Properties of materials.

Concrete	F_c	240 kg/cm^2
	V	0.15
	E	2100000 kg/cm^2
Bar	V	0.3
	E	2100000 kg/cm^2
	F_y	3000 kg/cm^2
Brace	V	0.3
	E	2100000 kg/cm^2
	F_y	2400 kg/cm^2
	F_u	3700 kg/cm^2

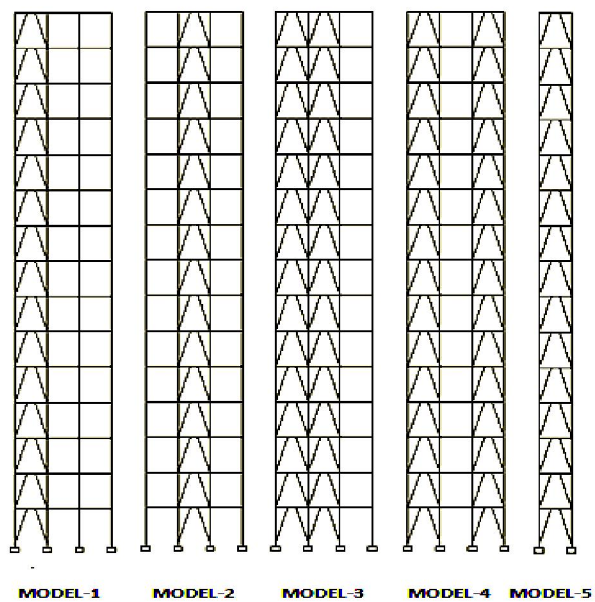

Figure 4. Number of bracing models.

Fig. 5 has represented an approximate equality in displacement resonance factors upon models 1, 2 and separately for the models 3, 4. According to the Fig. 4, these pair models can be marked out because of similarity in number of bracing spans. By this presents, resulting close stiffnesses explain similarity in displacement resonance factors. But locations of bracing spans have been left no tangible effect on DRF. Observed through Fig. 7, increasing height of building is accompanied with demoting DRF. In explaining, obligation to confining roof displacements to the target displacement acquires more stiffness together with raising height of building. This resulted in a reduction in displacement resonance factor.

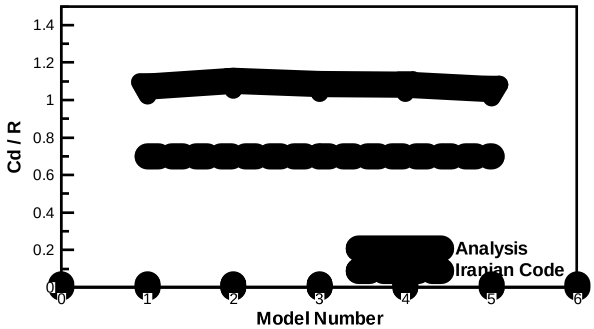

Figure 5. Variation of DRF ratio with Bracing kind of spans.

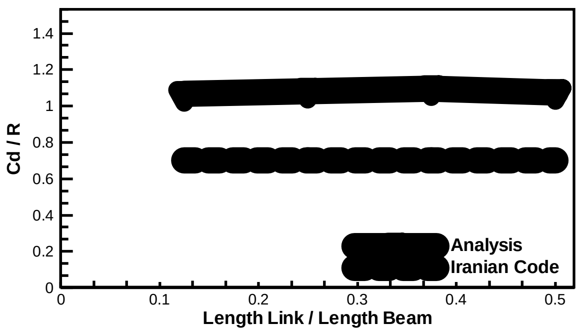

Figure 6. Variation of DRF ratio with Length of link beam.

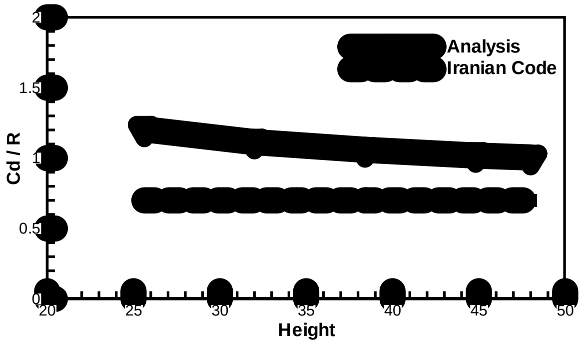

Figure 7. Variation of DRF ratio with height of building.

5. Conclusions

The main conclusions of this project can be drawn as follows:

Contemplating above discussions, seismic design based on Code 361 should be placed in priority than that of Iranian Code.

Giving a grade based on ability to present displacement resonance factor addresses equations $C_d = 1.16R$, $C_d = 1.03R$ respectively for buildings higher and lower than 10 stories.

It seems Iranian Code provisions on postulating the equation $C_d = 0.7R$ are not prepare to provide minimum strength required resistance to earthquake because of undervaluing actual displacements. To mitigate large displacement potential of existing buildings designed based on Iranian Code, retrofitting techniques should be considered.

Increasing either number of braced spans or height of structure leads declining in DRF.

Similarity in displacement resonance factors which are appraised by effect of symmetric and anti-symmetric bracing type is well observed throughout the analytical results.

Affected by increasing length of link beam over the span beam, the values of DRF grow in gradual steps (Fig. 6).

Acknowledgements

This work was supported and financed by the Islamic Azad University of Behbahan. Also the authors thank Dr. A. K. Pirasteh for scientific consulting in this project.

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3/8/2011