IJE TRANSACTIONS C: Aspects Vol. 28, No. 9, (September 2015) 1257-1267

International Journal of Engineering

Journal Homepage: www.ije.ir

Two Important Issues Relevant to Torsional Response of Asymmetric 8-Story RC Building Designed with Direct Displacement based Design Approach

E. Izadi Z.*, A. S. Moghadam

International Institute of Earthquake Engineering and Seismology (IIEES),Tehran, Iran

PAPER INFO

Paper history:
Received 08 March 2015
Received in revised form 09 July 2015
Accepted 30 July 2015

Keywords:
Asymmetric
Direct displacement based design
Diagonal displacement
Seismic
RC Frames
Torsional response

ABSTRACT

Direct displacement based design (DDBD) is a conceptual framework that directly designs a structure to achieve an expected performance level under specified seismic intensity. In this study, two important issues relevant to torsional response of mass eccentric 8-story RC building designed with DDBD approach are investigated. These issues are including the effects of unbalanced mass distribution scenario on the torsional response parameters and the study of these parameters with reference to diagonal displacement. Diagonal displacement is the SRSS combination of the displacement demands along the direction of excitation and orthogonal direction. Three different unbalanced mass distribution scenarios which produce the same mass eccentricity were applied to the plan of the generic structural model to determine the general range of the mass moment of inertia (MMI) variation due to different unbalanced mass distribution scenarios. Expressions were established to correlate MMI and mass eccentricity in each scenario. Results show that for slight eccentricities the variation of the MMI is negligible but as eccentricity is increased the range of the variation is extended. Then, sensitivity analyses based on finite element method and inelastic time history analysis have been carried out on 8-story RC building frame designed with DDBD approach with different levels of mass eccentricity and different MMI. Torsional response parameters in terms of maximum displacement demands of edge elements, diaphragm rotation, nominal relative displacement and nominal rotation is compared with diagonal and horizontal displacement demands along seismic excitation.

***doi**: 10.5829/idosi.ije.2015.28.09c.02*

1. INTRODUCTION

Most seismic design codes still focus on force rather than displacement. Over the past two decades different displacement oriented design approaches have been developed [1]. Direct displacement based design (DDBD) has been developed in conjunction with the fundamental objections on conventional force based design method [2]. This approach that utilizes the idea of 'substitute structure' [3] has been the focus of different researches. Priestley and Kowalsky presented DDBD basic formulation for RC structures in detail [2]. Sullivan et al. investigated the limitation and seismic performance of different displacement based design approaches and suggested the DDBD as one of the most reliable approaches [1]. Pettinga and Priestley investigated the seismic behavior of RC tube frames designed with DDBD approach [4]. Sullivan et al. developed the DDBD procedure for RC frame-wall structures considering interaction effects between frame and shear walls [5]. Pettinga and Priestley developed a method for explicitly accounting P-Δ effects in structures when using DDBD method [6]. Calvi and Sullivan developed a model draft code (DBD09) for displacement based seismic design [7]. Salwadeh developed DDBD procedure for vertically irregular frame-wall structures [8]. Sullivan et al. reviewed the model draft code DBD09 provisions and superseded it by DBD12 [9].

Some researchers also have been conducted for extending DDBD procedure to asymmetric structures in layout. Beyer et al. investigated the torsional inelastic response of structural wall buildings and developed a method for estimating the torsion-induced displacement

*Corresponding Author's Email: e.izadi@iiees.ac.ir (E. Izadi Z.)

Please cite this article as: E. Izadi Z. and A. S. Moghadam, Two Important Issues Relevant to Torsional Response of Asymmetric 8-Story RC Building Designed with DDBD Approach International Journal of Engineering (IJE), TRANSACTIONS C: Aspects Vol. 28, No. 9, (September 2015) 1257-1267

demands to extend DDBD method to asymmetric wall buildings [10]. Although this research, provided recommendations and provisions for DDBD of asymmetric structures, but the concept is still under development and require more contribution. Main focus of this study is to contribute with further extension of DDBD approach to design asymmetric mass eccentric structures in layout. Accurate estimate of seismic displacement demand corresponding to expected performance level of the structure is one of the most important steps of DDBD approach. For asymmetric structures subjected to seismic excitation, rotational response is expected to occur. Consequently displacement demand at a particular floor level of structure is no longer uniform and displacement demands for each of lateral force-resisting elements should be addressed directly. Torsional inelastic behavior of the asymmetric buildings has been the focus of many researches and different design procedures have been developed for considering torsion-induced displacement.

General review and description of some of these researches may be found in Rotenberg [11] and Destefano and Pintucchi [12]. Variety of absolute, relative and nominal torsional response quantitative parameters generally related to displacement demands has been introduced in the literature. These parameters describe somewhat dynamic torsional response of asymmetric structures. In this research, the effects of unbalanced mass distribution scenario and diagonal displacement on the torsional response parameters of an asymmetric 8-Story RC building designed with DDBD approach are investigated.

2. GENERIC ASYMMETRIC STRUCTURAL MODEL

The generic asymmetric model is a mass eccentric 8-story RC moment resistant building as shown in the Figure 1. Building is asymmetric in X direction and the center of geometry is taken as the origin of the X-Y co-ordinate system. Typical floor of the building have 3 spans with 5.0 m width in each direction and constant 3 m story heights. Building is located in a very high seismic hazard zone and on type III soil according to the site classification of the Iranian Seismic Code [13]. Gravity load transfer system is RC two-way slab which is considered to be rigid. The building is assumed to be residential and the non-factored gravity loads are: dead load equal to 7.5 KN/m^2 and live load equal to 2 KN/m^2. Seismic weight of each story considering dead load plus 20 percent of live load is equal to 1777.5 KN, which typically assumed to be lumped at floor levels. Characteristic material strengths are taken as concrete compression strength of 40 MPa and reinforcing steel yield strength of 400 MPa.

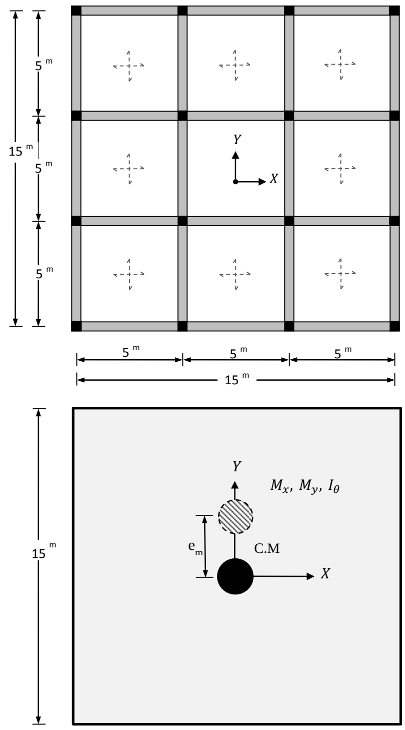


Figure 1. Generic Asymmetric Structural Model

3. BRIEF REVIEW OF THE GENERAL DDBD APPROACH

Using DDBD approach, an inelastic multi degree of freedom (MDOF) system is substituted with an elastic single degree of freedom (SDOF) system characterized by secant stiffness K_e at peak displacement Δ_d and an Equivalent Viscous Damping (EVD). This damping represents the effects of elastic damping and hysteretic damping due to inelastic responses. For a known design displacement and rational estimation of yield displacement Δ_y, it is straightforward to determine displacement ductility demand μ, and thus EVD. Afterward, using sets of elastic displacement response spectra (DRS) for different levels of damping, the effective period T_e of equivalent SDOF structure at design displacement is calculated. For a known effective period, the effective stiffness and then base shear of the equivalent SDOF structure at design displacement is determined [14]. Each of these steps is described in more detail in the following sections.

4. DESCRIPTION OF DDBD PROCEDURE FOR RC BUILDING FRAMES

In this study, DDBD procedure based on the provisions provided by Sullivan et al. [9] were applied to the 8-story RC building of Figure 1. Detailed description of the procedure is provided hereunder.

TABLE 1. Performance criteria for damage control performance level

Performance Criteria	Limit
Maximum drift	0.02
Residual drift	0.004
Concrete comp. strain	$\varepsilon_{c,dc} = 0.004 + 1.4\frac{\rho_v f_{yh} \varepsilon_{su}}{f'_{cc}} < 0.02$
Rebar tension strain	$0.6\varepsilon_{su} < 0.05$

4. 1. Expected Performance and Limitations Building is designed to achieve damage control corresponding to the life safety performance level in Iranian Seismic Code. The design displacement for the building corresponding to the expected performance level may be controlled by material strain limits; by drift limits, which are typically used to control damage to non-structural components, by residual drift limits or by any other deformation quantity that is appropriate for the structure being designed [9]. In this regard, based on the provisions provided in DBD12, for expected performance level, performance criteria in terms of maximum drift, residual drift and material strain at plastic hinge regions shall be limited to the values in Table 1 for structural elements. Where $\varepsilon_{c,dc}$ is the concrete compressive strain for damage control limit state, ρ_v the volumetric ratio of reinforcement hoops or spirals, f_{yh} the yield stress of transverse confining reinforcement, ε_{su} the ultimate strain of longitudinal reinforcement and finally f'_{cc} the confined concrete compressive strength as per Mander model [15].

4. 2. Design Displacement Profile Building is designed for drift limit value in Table 1 and material strain limits at plastic hinges are checked later in design section. The design displacement Δ_d, is defined as:

$$\Delta_d = \frac{\sum_{i=1}^{n}(m_i \Delta_i^2)}{\sum_{i=1}^{n}(m_i \Delta_i)} \tag{1}$$

where m_i is the seismic mass and Δ_i the displacement at level i which is obtained from Equation (2).

$$\Delta_i = \omega_\theta . \theta_c . h_i . \frac{(4H_n - h_i)}{(4H_n - h_1)} \tag{2}$$

where ω_Θ is the higher mode reduction factor, Θ_c the drift ratio limit, h_i the height of level i, H_n the total building height and h_1 the height of the first level.

4. 3. Yield Displacement and Ductility Demands The displacement ductility demand μ, is defined as:

$$\mu = \frac{\Delta_d}{\Delta_y} \tag{3}$$

where Δ_y is the yield displacement that for regular RC frames is calculated by Equation (4) presented by Priestley [14].

$$\Delta_y = \theta_y \times H_e \tag{4}$$

where Θ_y is the yield drift that comes from Equation (5) and H_e the effective height which is derived later in Equation (7).

$$\theta_y = 0.5\varepsilon_y \left(\frac{l_b}{h_b}\right) \tag{5}$$

where ε_y is the yield strain of longitudinal reinforcement, l_b the length of beams between column centerlines and h_b the depth of beam sections in of RC frame.

4. 4. Equevalent SDOF Characteristics Direct displacement based design approach relies on substituting an inelastic MDOF system with an elastic SDOF system characterizes by secant stiffness at design displacement and an equivalent viscous damping (EVD). SDOF representation parameters [9] are:

- The effective mass m_e

$$m_e = \frac{\sum_{i=1}^{n}(m_i \Delta_i)}{\Delta_d} \tag{6}$$

- The effective height of the equivalent SDOF structure, H_e

$$H_e = \frac{\sum_{i=1}^{n}(m_i \Delta_i h_i)}{\sum_{i=1}^{n}(m_i \Delta_i)} \tag{7}$$

- The EVD of a well detailed RC frame structure is included the sum of the elastic damping ξ_{el} and hystersis damping ξ_{hys} as Equation (8).

$$\xi_{eq} = \xi_{el} + \xi_{hys} = 0.05 + 0.565\left(\frac{\mu - 1}{\mu\pi}\right) \tag{8}$$

4. 5. Effective Stiffness, Base Shear And Its Distribution For a design displacement Δ_d and corresponding ξ_{eq}, the effective period of the equivalent SDOF structure, T_e can be found from the elastic DRS sets. Then, the effective stiffness K_e at the design displacement is obtained from Equation (9) which is the simple inversion of the equation for the period of a SDOF structure.

$$K_e = 4\pi^2 \frac{m_e}{T_e^2} \tag{9}$$

DDBD approach needs sets of DRS for higher level of damping rather than commonly used 5% one. There is no information available in Iranian seismic code to describe how to decrease DRS for higher level of damping. To develop such a sets of DRS, a suite of seven artificial records were generated to be matched with the design response spectrum of Iranian Seismic Code corresponding to the study site for 5% damping. Matching procedure based on Wavelet transformation proposed by Suarez and Montejo [16] was applied using program provided in Matlab environment. Wavelet transformation is one of the most powerful tools in time-frequency domain for record generation. In the matching procedure considered, a real original record that is compatible with the study site condition is presented as input record. The procedure manipulates the input record to generate the spectrum compatible with accelerogram. Furthermore, the non-stationary characteristic of the original record is somewhat preserved in generated record. DRS were developed for each generated records and level of damping using Seismo signal [17]. The smoothed mean values were considered as design DRS for each level of damping as shown in Figure 2. The design base shear of a SDOF structure V_b that will be used to define the strength of plastic hinge regions shall be determined in accordance with Equation (10) [9]:

$$V_b = K_e.\Delta_d + V_{P-\Delta} \le 2.5R_{\xi}.PGA.m_e + V_{P-\Delta} \tag{10}$$

$$V_{P-\Delta} = 0.5 \times \frac{\sum_{i=1}^{n} P_i \Delta_i}{H_e} \tag{11}$$

where $V_{P\text{-}\Delta}$ is the P-delta component of the base shear, R_ξ the response spectrum modification factor, PGA the peak ground acceleration and P_i the total expected gravity load on level i of the building during earthquake. Seismic characteristics and response spectrum of Iranian Seismic Code [13] corresponding to the study site was applied in this paper. Calculation of the equivalent SDOF characteristics of the building based on the procedure described before is shown in Table 2. The resulting DDBD parameters of the building are summarized in Table 3. The calculated design base shear in Table 3 is distributed to the seismic masses at different story levels according to Equation (12) [9]. For frame buildings in which the lateral resisting system forms plastic hinges over the full height of the structure, the value of k is assumed to be k=0.9.

$$F_i = \begin{cases} kV_b.(m_i\Delta_i) / \sum_{i=1}^{n}(m_i\Delta_i) & i = 1:(n-1) \\ (1-k)V_b + kV_b.(m_i\Delta_i) / \sum_{i=1}^{n}(m_i\Delta_i) & i = n \end{cases} \tag{12}$$

where F_i is the lateral force at the story level i.

TABLE 2. Calculation of the equivalent SDOF characteristics for 8-story RC building

St.	m_i (KN.S^2/m)	h_i (m)	Δ_i (m)	h_b (m)	l_b (m)	θ_y -	Δ_y (m)	μ -	ζ -
1	177.75	3	0.060	0.5	5	0.011	0.182	1.51	0.111
2	177.75	6	0.116	0.5	5	0.011	0.182	1.51	0.111
3	177.75	9	0.168	0.45	5	0.012	0.202	1.36	0.098
4	177.75	12	0.217	0.45	5	0.012	0.202	1.36	0.098
5	177.75	15	0.261	0.4	5	0.014	0.228	1.21	0.081
6	177.75	18	0.302	0.4	5	0.014	0.228	1.21	0.081
7	177.75	21	0.339	0.4	5	0.014	0.228	1.21	0.081
8	177.75	24	0.372	0.4	5	0.014	0.228	1.21	0.081

TABLE 3. DDBD parameters for 8-story RC building

Story	m_e (KN.S^2/m)	h_e (m)	Δ_d (m)	ξ_{eff} %	T_e (Sec)	K_e (KN/m)	V_b (KN)
8	1185.7	16.56	0.275	9.67	1.71	16008.5	4519.2

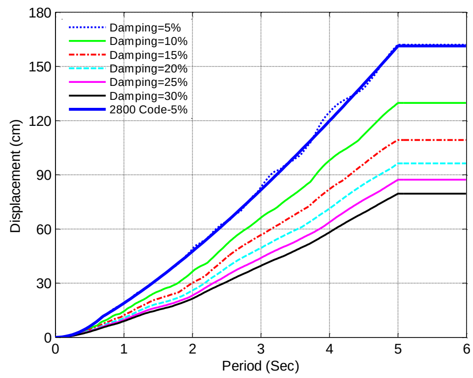


Figure 2. Design displacement response spectra (DRS)

4. 6. Structural Analysis and Section Design

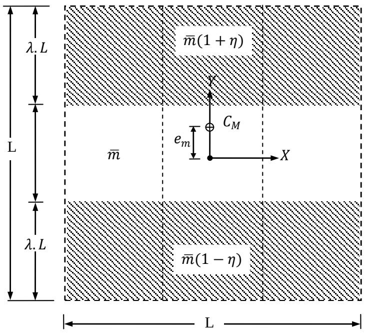
Analysis based on the equilibrium considerations [14] have been carried out on the building to determine the required moment capacity of intended plastic hinge locations. The equilibrium method lets designer to select the moment capacities of the column-based hinges, provided that the resulting moment throughout the structure are in equilibrium with external applied forces. However, to determine the seismic demands at the column-based hinges, rational engineering decision should be made. In this regard, the bottom story point of contra-flexure was assumed to be at $0.6H_1$, where H_1 is the first story height. The full procedure, details and rational decisions for equilibrium method may be found in Section 5.5.2 of Priestley et al. [14]. After determining the seismic demands, the building was designed for desired beam-sway mechanism due to inelastic response. To ensure that unwanted inelastic mechanism and brittle shear failure do not occur, capacity design based on the Section 9.3 of DBD12 was applied [9]. Moments and shears in regions required to remain elastic were amplified to account for higher mode dynamic amplification and possible increased material strength in plastic hinges location. After determining capacity design force levels, section analysis program Microsoft CUMBIA provided by Montejo and Kowalsky [18] were used to design reinforcing details of structural elements and performing moment-curvature analysis. This program implements concrete confinement model presented by Mander [15]. The ultimate concrete compression strain and rebar tension strain for each section of elements were checked with the strain limit values of Table 2 manually. Characteristic material properties without any amplification were used for designing the capacity protected actions while plastic hinge regions were designed for expected material strengths. Expected concrete compressive strength f'_{ce} and expected yield stress of steel reinforcement f_{ye} were assumed as $f'_{ce} = 1.3 \times f'_c$ and $f_{ye} = 1.1 \times f_y$ respectively, where f'_c and f_y are characteristic concrete compressive strength and characteristic yield stress of steel reinforcement. For all design actions, except those of plastic hinge regions, normal material strength reduction factor were applied in line with ACI318-99. For design actions of plastic hinge regions, strength reduction factor were not applied.

5. UNBALANCED MASS DISTRIBUTION SCENARIOS

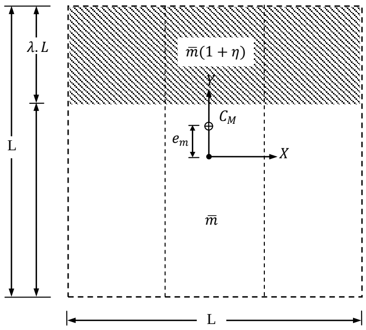
A large number of parameters affect inelastic response of asymmetric buildings. One of the most important ones which affect dynamic characteristics of the buildings is mass moment of inertia (MMI). This parameter directly depends on the unbalanced mass distribution scenario which produces the eccentricity in layout. Considering constant mass, corresponding to a given mass eccentricity, probably there are infinite unbalanced mass distribution scenarios. Therefore, for a known mass eccentricity a range of MMI is possible. In this study, to estimate the range of the MMI variation, three different unbalanced mass distribution scenarios which produce the same mass eccentricity is considered. With reference to symmetric structure, total seismic mass is kept constant in each scenario. These scenarios are: 1.unbalanced double-band mass scenario as shown in Figure 3(a), 2.unbalanced single-band mass scenario as shown in Figure 3(b), and 3.unbalanced concentrated mass scenario as shown in Figure 3(c), where λ is a geometric factor as shown in Figure (3), $\overline{m}$ the uniform mass of the diaphragm and η unbalanced uniform mass distributor. These scenarios were applied to the generic asymmetric structural model (L=15 and λ=1/3) and expressions were established to correlate MMI and mass eccentricity (e_m) for each scenario. Normalized MMI (φ) with reference to corresponding symmetric building is defined as:

$$\varphi = \frac{I_m}{I_r} \tag{13}$$

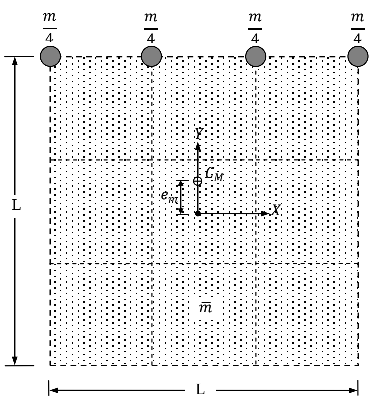
where I_m and I_r are the MMIs corresponding to unbalanced mass distribution scenario and symmetric model ($e_m = 0$), respectively. Correlation between mass eccentricity (e_m) and normalized MMI (φ) is shown in Figure 4.



(a). Unbalanced double-band mass scenario



(b). Unbalanced single-band mass scenario



(c). Unbalanced concentrated mass scenario

Figure 3. Different unbalanced mass distribution scenarios

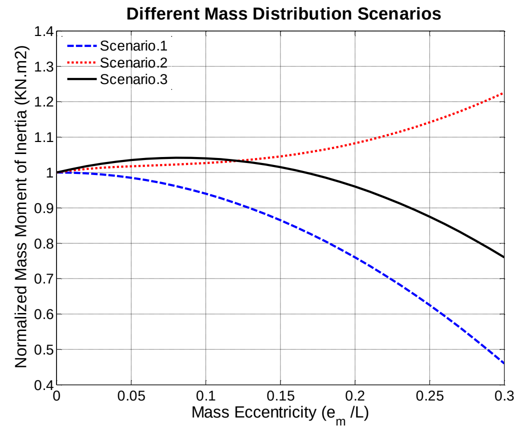


Figure 4. Correlation between mass eccentricity and normalized MMI

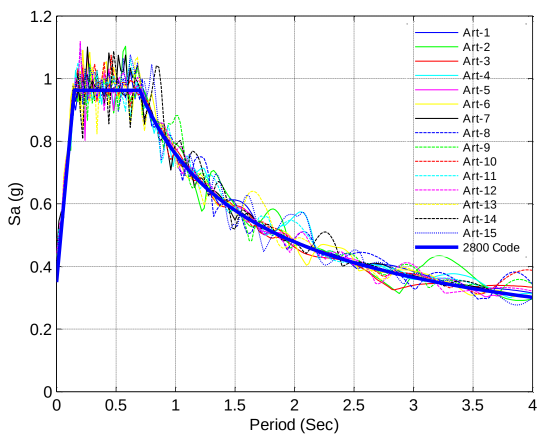


Figure 5. Comparison of artificially generated records spectra with design response spectrum

For slight eccentricities, the variation of ϕ is negligible, but as eccentricity is increased the range of the variation is extended. As shown in the Figure 4, variation range of the normalized MMI is less than ±40% even in the case of severe eccentricity. Consequently, normalized MMI is taken as ϕ = 0.6, 0.8, 1.0, 1.2 and 1.4 for sensitivity analysis.

6. GROUND MOTIONS AND MODELING

For the purpose of seismic assessment, a series of inelastic time history analysis (ITHA) have been carried out on the building. A suite of fifteen artificial accelerograms were generated to match with the design response spectrum. Matching procedure based on Wavelet transformation proposed by Suarez and Montejo [16] was applied using program provided in Matlab environment. Acceleration response spectrum of artificially generated records in comparison to the design acceleration response spectrum has been demonstrated in Figure 5. ITHA of the building is performed using OpenSees [19] which is an object oriented framework for finite element analysis. The models are subjected to one-directional records along the X direction. The structural elements are modeled as fiber elements to consider plasticity along the elements and their cross sections. Separate stress-strain characteristics were used for the unconfined cover concrete and the confined core concrete as per Mander model [15]. An artificially low damping coefficient ξ^* in the fundamental mode was specified for initial elastic damping based on the recommendation of Priestley et al. [14]. Artificially low damping coefficient is approximately equal to:

$$\xi^* = \frac{\xi(1-0.1(\mu-1)(1-r))}{\dfrac{\sqrt{\mu}}{(1+r\mu-r)}} \tag{14}$$

where r is the post yield stiffness ratio (post yield stiffness to the initial stiffness) that for a well-detailed RC frame is assumed about r=0.05.

7. SEISMIC EVALUATION OF ASYMMETRIC BUILDINGS

To study the seismic torsional behavior of the purposed asymmetric buildings, unbalanced single band mass scenario as shown in Figure 3(b) was applied to produce mass eccentricity (e_m). Four asymmetric models with e_m=5, 10, 15 and 20% of the plan dimension respectively (L=15m) were considered and corresponding MMI were calculated for each model. ITHA have been carried out on the asymmetric models with different level of eccentricity and corresponding MMI. Results are presented as mean value of the maximum responses for fifteen artificial records. Torsional response parameters in terms of maximum diaphragm rotation, maximum nominal relative displacement and maximum nominal rotation are presented hereunder.

7. 1. Maximum Diaphragm Rotation Maximum rotation is taken as one of the response parameters which represent the severity of torsional behavior. Nevertheless, this parameter is not among the common parameters used in torsional provisions of design codes. Maximum diaphragm rotation and maximum displacement demand at the center of mass do not occur at the same instant. Thus maximum diaphragm rotation is not a practical parameter for addressing maximum displacement demands of asymmetric buildings. Figure

6(a) shows maximum diaphragm rotation for different levels of mass eccentricity. As shown in the figure maximum rotation at all level of the building is increased as the eccentricity is extended. Furthermore it is seen that for severe eccentricities, maximum rotation is much less sensitive to the amount of eccentricity.

7. 2. Maximum Nominal Relative Displacement

This parameter is one of the most common practical parameters used in torsional studies which represent the dynamic torsional behavior of eccentric structures. Nominal relative displacement is defined as [10]:

$$\Delta_{rel} = \frac{\Delta_{\max}}{\Delta_{c.m}} \tag{15}$$

where Δ_{max} is maximum displacement at the story level and $\Delta_{c.m}$ maximum center of mass displacement. Figure 6(b) shows maximum nominal relative displacement for different eccentricities. It is shown in the figure that independent of the amount of eccentricity, nominal relative displacement is somehow uniform over the height of the building and for severe eccentricities ($e_m/L = 20\%$), maximum nominal relative displacement is also much less sensitive to the amount of eccentricity. Furthermore, for severe eccentricity, nominal relative displacement has diminished surprisingly.

7. 3. Maximum Nominal Rotation

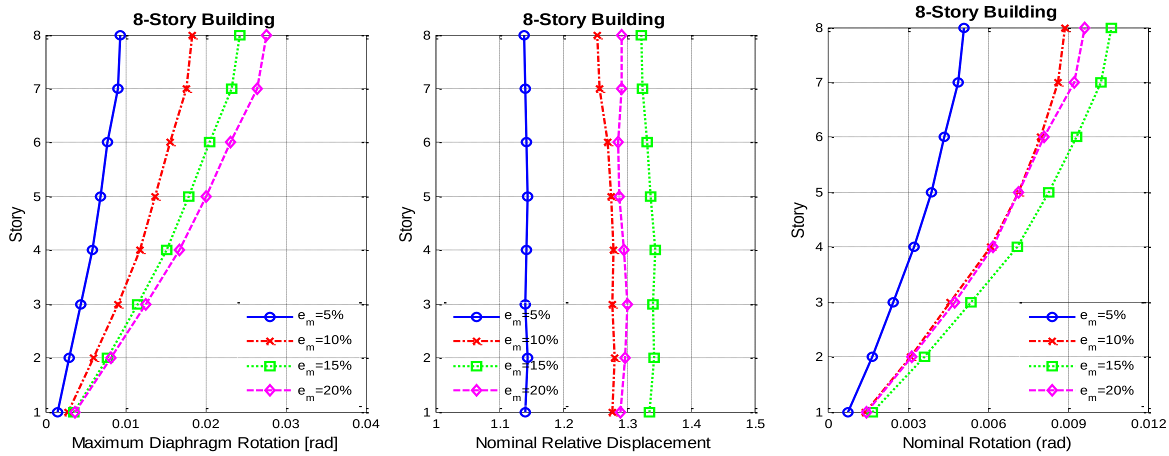
For asymmetric buildings subjected to seismic excitation, it is not possible to directly associate maximum displacement and rotation of the structure to the maximum displacement demands of the lateral load resisting elements. To overcome this problem, diaphragm nominal rotation has been suggested by Castillo [20]. Nominal rotation is widely used as one of the most common parameters in torsional provisions of some displacement based design codes. Maximum nominal rotation θ_N is defined as:

$$\theta_N = \frac{\Delta_{\max} - \Delta_{c.m}}{X_{cp-cm}} \tag{16}$$

where Δ_{max} is maximum displacement of critical lateral load resisting element, $\Delta_{c.m}$ maximum displacement at the center of mass and X_{cp-cm} the distance between critical element and center of mass. Figure 6(c) plots the variations of maximum nominal rotation for different eccentricities. Maximum nominal rotation is influenced significantly by the eccentricity level. For severe eccentricity ($e_m/L = 20\%$), maximum nominal rotation is also much less sensitive to the amount of eccentricity. Furthermore, for severe eccentricities, nominal rotation has been diminished surprisingly.

8. SENSITIVITY ANALYSIS FOR MMI

Sensitivity analysis based on ITHA was performed on the designed structure with slight (em/L=5%) and severe (em/L=15%) mass eccentricity. Normalized MMI (Φ) was changed gradually and the effects of unbalanced mass distribution scenarios on the seismic torsional response parameters was measured. Figure 7 shows maximum diaphragm rotation for slight (em/L=5%) and severe (em/L=15%) eccentricity. Significant variation of maximum rotation due to change of normalized MMI is observed. This variation, as shown in Figure 7, is significant for both models with slight and severe eccentricity. Furthermore maximum diaphragm rotation is increased as normalized MMI is increased.



(a) (b) (c)

Figure 6. Torsional response parameters due to different levels of mass eccentricity

Figure 8 shows maximum nominal relative displacement for slight and severe mass eccentricities. It is seen that nominal relative displacement at different story levels of each model is relatively uniform. Furthermore, this parameter is influenced significantly by normalized MMI (Φ) for both models with slight and severe mass eccentricities. Figure 9 plots the variations of maximum nominal rotation for slight and severe mass eccentricities. Maximum nominal rotation for both models is influenced significantly by the level of the normalized MMI (Φ). Figure 10 summarizes the sensitivity of the envelope (along height) of the maximum torsional response parameters with normallized MMI (Φ) for the models with slight and severe eccentricity. Generally, the envelope of the maximum diaphragm rotation, as well as maximum nominal rotation, occurs at the upper story levels, while the envelope of the maximum nominal relative displacement is at the lower story levels.

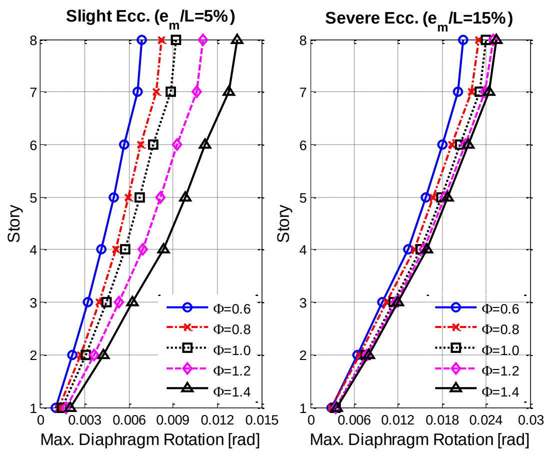


Figure 7. Sensitivity of maximum diaphragm rotation with normalized MMI

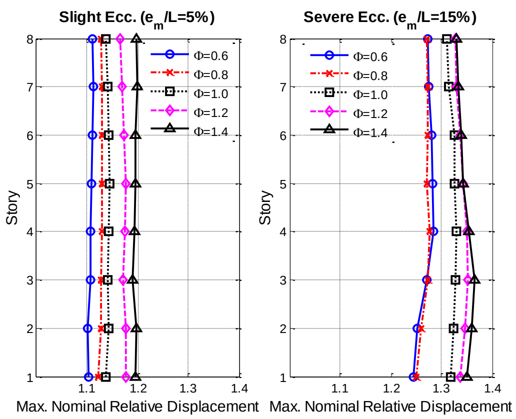


Figure 8. Sensitivity of maximum nominal relative displacement with normalized MMI

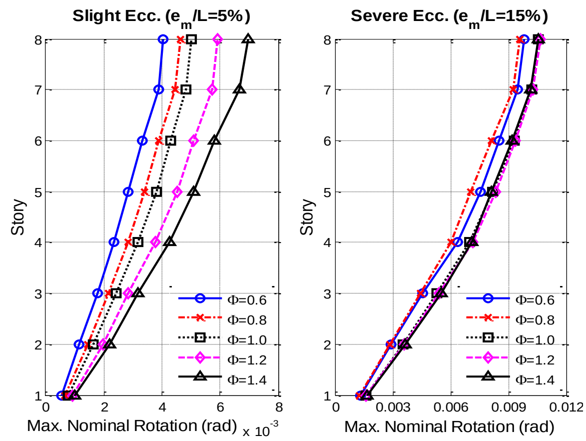


Figure 9. Sensitivity of maximum nominal rotation with normalized MMI

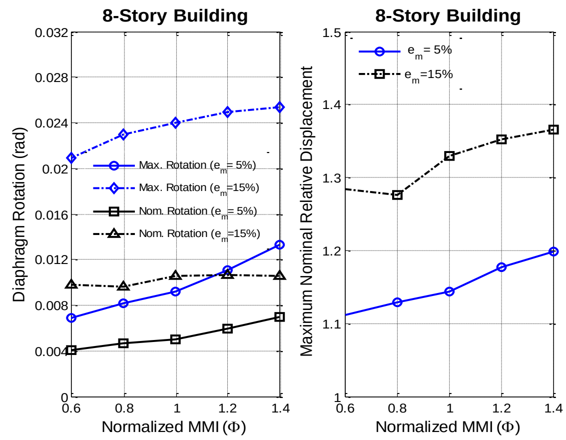


Figure 10. Summary of the sensitivity of torsional response parameters with normalized MMI

9. DIAGONAL DISPLACEMENT-BASED SEISMIC ASSESSMENT

Although foregoing torsional response parameters describe dynamic behavior of the asymmetric buildings, but variation of these parameters with reference to displacement as a vector is still open to question. Here diagonal displacement is chosen as a vector representation of displacements that is calculated as the SRSS combination of the displacement demands along the direction of excitation and orthogonal direction. Generally, diagonal displacement effects on the torsional response parameters are assumed negligible while these effects may cause severe local damages at floor corners due to stress and strain concentration. Figure 11 compares maximum demands with reference to horizontal and diagonal displacement of the 8-story RC building with slight and severe mass eccentricities.

Based on what was said in previous sections, displacement demands at a particular floor level of an eccentric structure due to torsional response is not uniform and demands should be addressed directly for each lateral load resistant element. Therefore, maximum displacement demands at the soft edge, center of mass and stiff edge are presented and described separately. It is observed in Figure 11 that maximum demand at the center of mass for both models with slight and severe eccentricities is not strongly influenced by considering diagonal displacements rather than horizontal displacements.

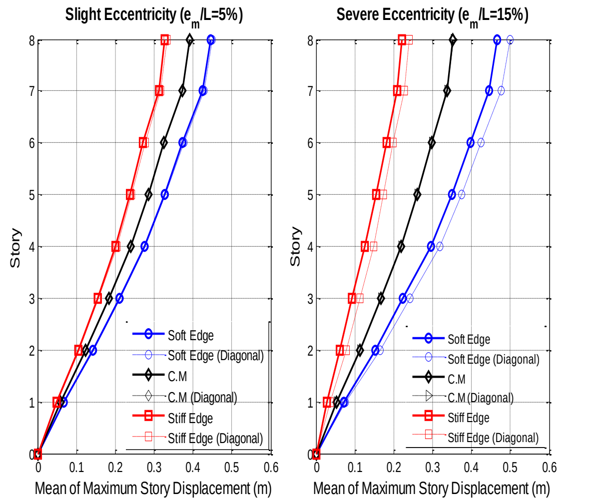


Figure 11. Comparison of maximum demands with reference to horizontal and diagonal displacement

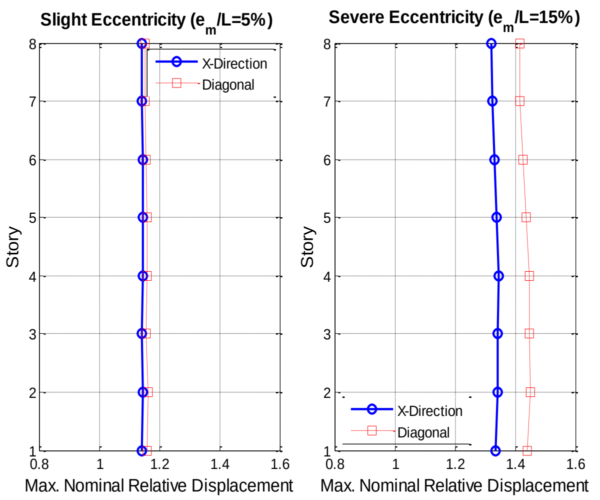


Figure 12. Comparison of maximum nominal relative displacement with reference to horizontal and diagonal displacement

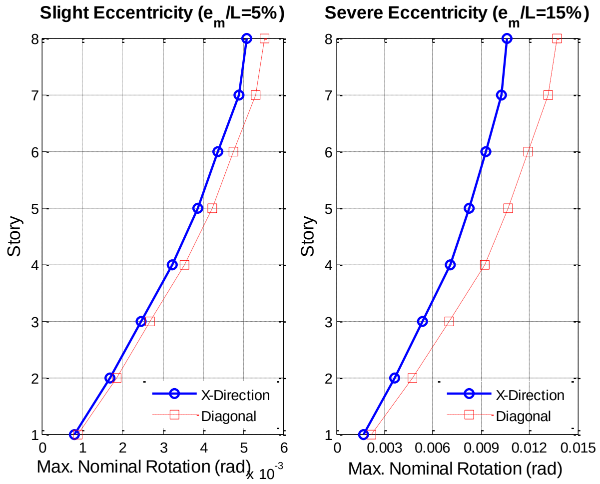


Figure 13. Comparison of maximum nominal rotation with reference to horizontal and diagonal displacement

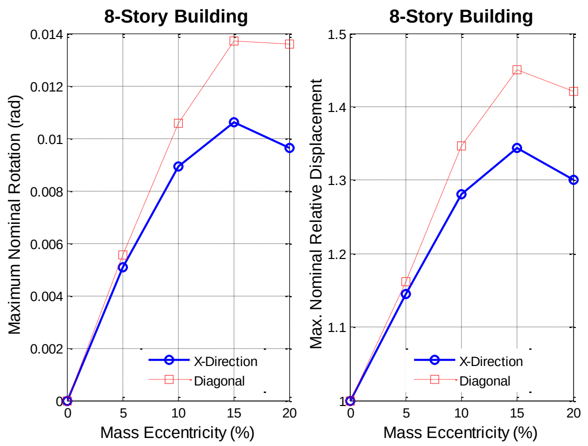


Figure 14. Comparison of maximum torsional response with reference to horizontal and diagonal displacement for different mass eccentricity levels

The effect of diagonal displacement on maximum displacement demands at the soft and stiff edge for model with slight eccentricity is not considerable, while for the models with severe eccentricity, the effect is significant. Maximum nominal relative displacement and maximum nominal rotation of the 8-story RC building with reference to horizontal and diagonal displacements are shown in the Figures 12 and 13. For the model with slight eccentricity, the variation is negligible, but for that with severe eccentricity, it is significant. Figure 14 summarizes the comparison of the envelope (along height) of maximum torsional response parameters with reference to horizontal and diagonal displacements at different levels of eccentricity. It is obvious in Figure 14 that as mass eccentricity is increased, the effect of diagonal displacement is much more sensible.

10. CONCLUSIONS

For contribution with further extension of DDBD approach to asymmetric buildings, two important issues relevant to torsional response of mass eccentric 8-story RC building designed with DDBD approach were investigated. These issues are the effects of unbalanced mass distribution scenario and diagonal displacement on the torsional response parameters. Based on the results, the following conclusions are reached:

- Three different unbalanced mass distribution scenarios which produce the same mass eccentricity are considered to apply to a generic asymmetric model. For slight eccentricities, the variation of normalized MMI is negligible, but as eccentricity is increased the range of the variation is extended. Variation range of normalized MMI is less than ±40% even in the case of severe eccentricity.
- Maximum torsional response parameters of asymmetric building in terms of diaphragm rotation, nominal relative displacement and nominal rotation for different level of mass eccentricity were investigated. Independent of the level of mass eccentricity, nominal relative displacement is relatively uniform over the height of the building. For severe eccentricities, maximum torsional response parameters are much less sensitive to the amount of eccentricity; furthermore, nominal relative displacement and also nominal rotation have diminished surprisingly.
- Sensitivity analysis was performed on the building with slight (em/L=5%) and severe (em/L=15%) mass eccentricity to study the correlation of the torsional response parameters with normalized MMI. Maximum torsional response parameters are strongly influenced by normalized MMI (Φ) for both models with slight and severe mass eccentricities.
- Torsional response parameters with reference to displacement as a vector (diagonal displacement) do not influence significantly for slight eccentricities, but as eccentricity is increased, the effect is much more sensible.

11. REFERENCES

1. Sullivan, T., Calvi, G., Priestley, M. and Kowalsky, M., "The limitations and performances of different displacement based design methods", ***Journal of Earthquake Engineering***, Vol. 7, No. spec01, (2003), 201-241.
2. Priestley, M. and Kowalsky, M., "Direct displacement-based seismic design of concrete buildings", ***Bulletin of the New Zealand National Society for Earthquake Engineering***, Vol. 33, No. 4, (2000), 421-444.
3. Shibata, A. and Sozen, M.A., "Substitute-structure method for seismic design in R/C", ***Journal of the Structural Division***, Vol. 102, No. 1, (1976), 1-18.
4. Pettinga, J.D. and Priestley, M.N., "Dynamic behaviour of reinforced concrete frames designed with direct displacement-based design", ***Journal of Earthquake Engineering***, Vol. 9, No. spec02, (2005), 309-330.
5. Sullivan, T., Priestley, M. and Calvi, G., "Development of an innovative seismic design procedure for frame-wall structures", ***Journal of Earthquake Engineering***, Vol. 9, No. sup2, (2005), 279-307.
6. Pettinga, J.D. and Priestley, M.N., "Accounting for p-delta effects in structures when using direct displacement-based design, IUSS Press, (2007).
7. Calvi, G.M. and Sullivan, T.J., "A model code for the displacement-based seismic design of structures: Dbd09, draft subject to public enquiry, Iuss Press, (2009).
8. Salawdeh, S., "Displacement based design of vertically irregular frame-wall structures", Masters dissertation, European school for advanced studies in reduction of seismic risk (ROSE School), University of Pavia, Italy, (2009),
9. Sullivan, T.J., Priestley, M. and Calvi, G.M., "A model code for the displacement-based seismic design of structures: Dbd12, IUSS Press, (2012).
10. Beyer, K., Dazio, A. and Priestley, M., Seismic design of torsionally eccentric buildings with rc u-shaped walls., IUSS Press Pavia,, Italy. (2008)
11. Rutenberg, A., "Eaee task group (tg) 8: Behaviour of irregular and complex structures, asymmetric structures–progress since 1998", in Proceeding of 12th European Conference on Earthquake Engineering, Paper. (2002).
12. De Stefano, M. and Pintucchi, B., "A review of research on seismic behaviour of irregular building structures since 2002", ***Bulletin of Earthquake Engineering***, Vol. 6, No. 2, (2008), 285-308.
13. Code, I.S., Iranian code of practice for seismic resistant design of buildings., Standard, (2005).
14. Powell, G.H., "Displacement-based seismic design of structures", ***Earthquake Spectra***, Vol. 24, No. 2, (2008), 555-557.
15. Mander, J., Priestley, M. and Park, R., "Observed stress-strain behavior of confined concrete", ***Journal of Structural Engineering***, Vol. 114, No. 8, (1988), 1827-1849.
16. Suarez, L.E. and Montejo, L.A., "Generation of artificial earthquakes via the wavelet transform", ***International Journal of Solids and Structures***, Vol. 42, No. 21, (2005), 5905-5919.
17. Antoniou, S. and Pinho, R., Software seismosignal., Version, (2004).
18. Montejo, L.A. and Kowalsky, M.J., "Cumbia—set of codes for the analysis of reinforced concrete members", ***CFL Technical Rep. No. IS-07***, Vol. 1, (2007).
19. OpenSees, "Opensees is an object-oriented framework for finite element analysis", ***Pacific Earthquake Engineering Research Center University of California [Berkeley]***, (2006).
20. Castillo, R., "Seismic design of asymmetric ductile systems", Ph.D. Thesis, University of Canterbury, Christchurch, New Zealand, (2004).

Two Important Issues Relevant to Torsional Response of Asymmetric 8-Story RC Building Designed with Direct Displacement based Design Approach

E. Izadi Z., A. S. Moghadam

International Institute of Earthquake Engineering and Seismology (IIEES),Tehran, Iran

PAPER INFO

Paper history:
Received 08 March 2015
Received in revised form 09 July 2015
Accepted 30 July 2015

Keywords:
Asymmetric
Direct displacement based design
Diagonal displacement
Seismic
RC Frames
Torsional response

چکیده

روش طراحی مستقیم بر اساس تغییر مکان، یک روش مفهومی است که سازه را به منظور رسیدن به یک سطح عملکرد مورد انتظار تحت شدت لرزه‌ای معین طراحی می‌کند. در این مطالعه به منظور مشارکت در توسعه بیشتر روش یاد شده در سازه‌های نامتقارن، دو مقوله مهم در ارتباط با پاسخ پیچشی سازه ۸ طبقه قاب خمشی بتن آرمه نامتقارن جرمی طراحی شده به روش طراحی مستقیم بر اساس تغییرمکان مورد مطالعه قرار می‌گیرد. یکی مطالعه تآثیر سناریوی توزیع نامتوازن جرم بر پارامترهای پیچشی سازه نامتقارن و دیگری مطالعه این پارامترها با در نظر گرفتن اثر تغییرمکانهای مورب می‌باشد. در اینجا تغییرمکان مورب به ترکیب جذر مجموع مربعات تغییرمکانها در امتداد اعمال تحریک زلزله و امتداد متعامد آن اطلاق می‌گردد. در این ارتباط تعداد سه سناریوی مختلف توزیع نامتوازن جرم به منظور تعیین محدوده تغییرات ممان اینرسی دورانی در نظر گرفته شده و معادلاتی برای مرتبط نمودن خروج از مرکزیت جرمی و ممان اینرسی دورانی استخراج گردیده است. نتایج نشان می‌دهد که در محدوده خروج از مرکزیتهای کم، تغییرات ممان اینرسی دورانی بسته به سناریوهای مختلف کم می‌باشد، اما هر چه مقدار خروج از مرکزیت افزایش می‌یابد محدوده تغییرات آن نیز بشدت افزایش می‌یابد. در ادامه آنالیز حساسیت بر مبنای روش المان محدود و تحلیلهای تاریخچه زمانی غیرارتجاعی برای سطوح مختلف خروج از مرکزیت و مقادیر مختلف ممان اینرسی دورانی بر روی سازه ۸ طبقه بتن آرمه طراحی شده به روش طراحی مستقیم بر اساس تغییرمکان انجام شده است. پارامترهای پاسخ پیچشی سازه شامل ماکزیمم نیازهای تغییرمکانی لبه‌ها، ماکزیمم دوران دیافراگم، ماکزیمم تغییرمکان نسبی اسمی و ماکزیمم دوران اسمی دیافراگم با در نظر گرفتن اثر تغییرمکانهای نیاز مورب و همچنین تغییرمکانهای نیاز افقی در امتداد اعمال تحریک لرزه‌ای مورد مطالعه و مقایسه قرار گرفته است.

doi*: 10.5829/idosi.ije.2015.28.09c.02*