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A study on the effects of vertical mass irregularity on seismic performance of tunnel-form structural system

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Abstract. Irregular distribution of mass in elevation is regarded as a structural irregularity by which the modes with high energy levels are excited and in addition, it can lead the structure to withstanding concentration of nonlinear deformations and consequently, suffer from unpredictable local or global damages. Accordingly, with respect to the lack of knowledge and insight towards the performance of concrete buildings making use of tunnel-form structural system in seismic events, it is of utmost significance to assess seismic vulnerability of such structures involved in vertical mass irregularity. To resolve such a crucial drawback, this papers aims to seismically assess vulnerability of RC tunnel-form buildings considering effects of irregular mass distribution. The results indicate that modal responses are not affected by building's height and patterns of mass distribution in elevation. Moreover, there was no considerable effect observed on the performance levels under DBE and MCE hazard scenarios within different patterns of irregular mass distribution. In conclusion, it appears that necessarily of vertical regularity for tunnel-form buildings, is somehow drastic and conservative at least for the buildings and irregularity patterns studied herein.

Keywords: tunnel-form structural system; performance level; irregular mass distribution; vertical irregularity; seismic reliability

1. Introduction

Irregular distribution of mass, stiffness or strength in elevation of multi-storey buildings is considered as an influential factor exciting the higher modes, which is in breach of the common code-based design approaches whose main focus is on the first vibration mode. According to Fig. 1, such irregularities inflict unpredictable and abrupt damages on the structures reducing the reliability on gaining the predefined performance levels (Khan and Javed 2015). Unforeseen measures such as change in use of some of the building's stories, altering the interior architecture, demolition or construction of additional infill walls as well as evacuated stories are usually main causes of mass irregularity.

In order to study the effect of mass irregularity, Michalis *et al.* (2006) doubled the mass of several stories and conducted an incremental dynamic analysis (IDA) on it. They found that the mass and stiffness irregularities both affect the inter-storey drifts considerably. Through a number of structured researches, Das and Nau (2003) managed to investigate the effects of diverse irregularities on a number of RC buildings. Evaluating the provisions specified by seismic codes such as UBC, they succeeded in indicating the limitations ahead of applying simplified design methods (code-based static analysis method) for vertical irregular buildings.

Fig. 1 Deterioration of Fifth Storey of a Building in the Kobe Earthquake, Japan, 1995

DeStefano *et al.* (2005) evaluated the irregular RC frames in a 2D medium, which had been designed in compliance with Euro-Code 8 (EC-8) to satisfy high ductile requirements. Based on their findings, P-Delta effects could remarkably influence on performance of these buildings.

In virtue of evaluating the demand distribution in stories, Al-Ali and Krawinkler (1998) investigated into the effect of vertical irregularities and concluded that in both linear and nonlinear states, mass irregularity insignificantly affect the shear and displacement demands and compared to the other types of irregularity, its effect is less intense. Moreover, it was specified that increase in mass of the upper stories, leaves greater effects on the displacement responses of building compared to the case in which lower and middle stories become heavier.

The studies conducted by Vinod *et al.* (2009) demonstrated that displacement responses as well as storey drifts, are sensitive to the position of mass irregularity in

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addition to amount of the mass. In addition, they specified that increase in mass of the lower and upper stories, leaves profounder effects on the structural responses compared to the middle stories.

Through another study, Choi *et al.* (2004) investigated in to the distribution of plastic hinges and frame rotation and concluded that presence of extra mass in lower and upper stories, imposes greater effects on the seismic responses.

With reference to the results obtained from the previous studies, vertical irregularities are of potential to remarkably affect the seismic responses of buildings that might result in catastrophic consequences.

1.1 An overview on the tunnel-form structural system and literature review

Multi-story reinforced concrete tunnel form buildings are one of the innovative structural systems consisting of wall and slab as structural elements counted as the lateral and vertical load bearing components which are simultaneously casted in each story. Despite the widespread application of tunnel form system in mass construction projects as well as industrial structures, very few studies have been conducted on them and almost, there is no code or standard addressing this system as an independent load-carrying system.

Moreover, this method of construction can minimize the number of cold joints in comparison with the other common construction techniques. The other notable advantages of this system include integration and 3D performance, great length of wall-slab connections and high redundancy in conjunction with elimination of stress concentration to the nodal and concentrated state (common in systems comprised of beams and columns).

Noteworthy, similar to the other industrialized systems, tunnel-form structural system is of superiorities in reducing the costs and time as well as promoting the quality and staff's safety.

Based on the observations made after Kocaeli (M_w=7.4) and Duzce (M_w=7.2) in 1999, it was concluded that the RC tunnel-form buildings take advantage of competent strength and efficiency in confrontation with the seismic events compared to the RC frames benefitting from moment-resisting system solely or together with shear walls (Balkaya and Kalkan 2004). Furthermore, it should be mentioned that despite valuable researches conducted on tunnel-form buildings, these studies are insufficient and yet, a large number of the features of such systems have not been explored.

Based on the results derived by previously performed studies, application of the empirical relation proposed by the codes and standards to compute fundamental period of the tunnel-form buildings, does not yield accurate results and subsequently, seismic loads are improperly estimated (Goel and Chopra 1998, Lee *et al.* 2000). Through a case study, Klasanovic *et al.* (2014) found that while the structure is in linear behavioral phase, the fundamental period of the structure (12-storey building constructed by tunnel-form system in Croatia) is in proximity to the period obtained from EC8 and ATC3-06.

According to the studies conducted by Tavafoghi and Eshghi (2008) on tunnel form buildings with different plans and heights, it was indicated that the fundamental period in each direction, is directly dependent on the total height and the aspect ratio as well as number of walls could not considerably affect it. Furthermore, the first three modes of vibration were reported to be irrespective of the height and number of walls in plan. In order to compute the response modification factor (*R*-Factor), they attempted to adopt the method prescribed by ATC-63 (Tavafoghi and Eshghi 2011). Based on their findings, response modification factor of 4 can be counted as a reasonable value.

Balkaya and Kalkan (2003, 2004) carried out a pushover analysis on 2 and 5-storey tunnel-form buildings with the same plan and accordingly, the 3D membrane action was found to be a dominant force mechanism for the tunnel form buildings. In conclusion, they proposed to utilize response modification factor (*R*-Factor) of 5 and 4 for shorter and taller building, respectively.

In order to study the 3D behavior of intersecting walls, Yuksel and Kalkan (2007) carried out a number of tests on specimens incorporating the minimum reinforcement. Based on the results, making use of longitudinal bars concentrated in corners of the walls, desirably affects their performance, and might be able to vary the brittle mode of failure even in low ratios of reinforcement (Kalkan and Yuksel 2007). Balkaya *et al.* (2012) investigated the effect of soil-structure interaction on the mechanical characteristics of the tunnel form structures with different with diverse geometries making use of eigen value analysis. According to the results, a new relation for calculation of the fundamental vibration period of these structures is developed taking the effect of the soil-structure into account. Beheshti-aval *et al.* (2018) evaluated the seismic performance of tunnel form system subjected to the near and far-field accelerograms including forward directivity effects. This study exhibited that however, the seismic performance of system is reported to be desirable in both cases considered the input excitation, the failure patterns were not the same.

To evaluate seismic reliability of the tunnel form structures subjected to accidental torsions, Mohsenian and Mortezaei (2018a) carried out studies. According to the results, eccentricity of mass center by 10% of the plan dimension does not affect the performance level in case of DBE and MCE scenarios. In a follow-up study, Mohsenian and Mortezaei (2018b) proposed to replace the concrete coupling beam by a replaceable steel beam so that the damages could be optimally distributed in plan and height of tunnel-form buildings. They concluded that the coupling beam can considerably affects the structural response of the system. In addition, it was argued that the challenges ahead of reinforcing the common concrete beams (raised by their dimensional limitations), disrupts their ductile nature performance and thus, it would be irrational to expect these elements act as seismic fuses.

In another study, Mohsenian *et al.* (2018) analyzed the seismic susceptibility of the tunnel form structures to accidental eccentricities of mass and stiffness as well as their configurations. They found that the structural responses are not affected by the mentioned eccentricities and their configuration. Moreover, the most critical case was explored to take place when mass and stiffness centers are moved along only one direction of the plan.

1.2 Research significance and main objectives

Despite obvious behavioral distinctions between tunnel-form and the similar systems commonly used in practice (i.e., bearing wall system), insufficient attention has been paid to this novel system and the seismic design codes does not count on it as an independent structural system.

Studies carried out on different aspects of this system, indicates the insufficiency of the research works and shortage of information to be incorporated into the codes. Currently, this system is being widely employed in mass construction projects executed in earthquake-prone areas although, its seismic behavior in some fields is engaged with serious drawbacks. Obviously, it would be of importance for seismic design codes and standards to compensate for their lack of information concerning this novel system by means of the results of numerical studies. With respect to lack of information and scarce experiences learned from the performance of tunnel-form buildings during past earthquakes, the experts responsible for preparing the seismic codes have illegalized presence of vertical or horizontal irregularities in tunnel-form buildings.

A review on the literature and previous studies demonstrate that no numerical or experimental study has been conducted yet to investigate the effect of vertical irregularity on seismic performance of such systems. According to the construction process of this structural system, it seems that mass irregularity can be counted as the most likely type of irregularity to emerge in such buildings. In this respect, the current paper deals with the nonlinear behavior of this tunnel-form buildings making use of time-history and pushover analyses for structures suffering from irregular distribution of mass in elevation determining their performance level under DBE (return period of 475 years) hazard level. Moreover, accounting for uncertainties ahead of future earthquakes, a reliability study has been carried out as well so that the fragility curves can be generated.

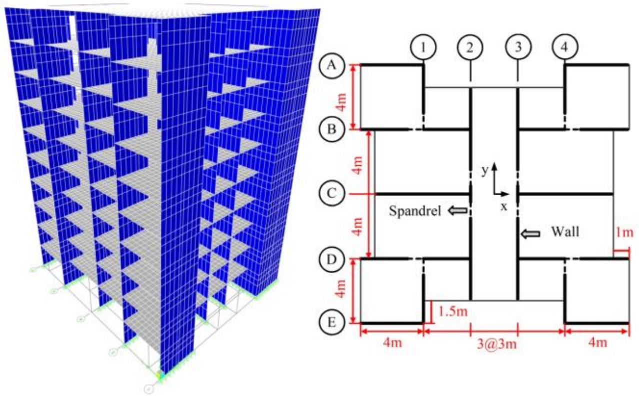

Fig. 2 Plan view of the tunnel-form buildings and a 3D view of the 10-Storey building

2. Modeling specifications

The plan view shown in Fig. 2, has been used to build the numerical models herein (Mohsenian 2013). According to this Figure, the plan is regular and symmetric in relation to both principal directions.

The dashed lines depicted on the plan, denote the coupling beams whose length and height is 1 and 0.7 m, respectively. Accounting for common heights used in mass construction projects, two buildings with 5 and 10 stories

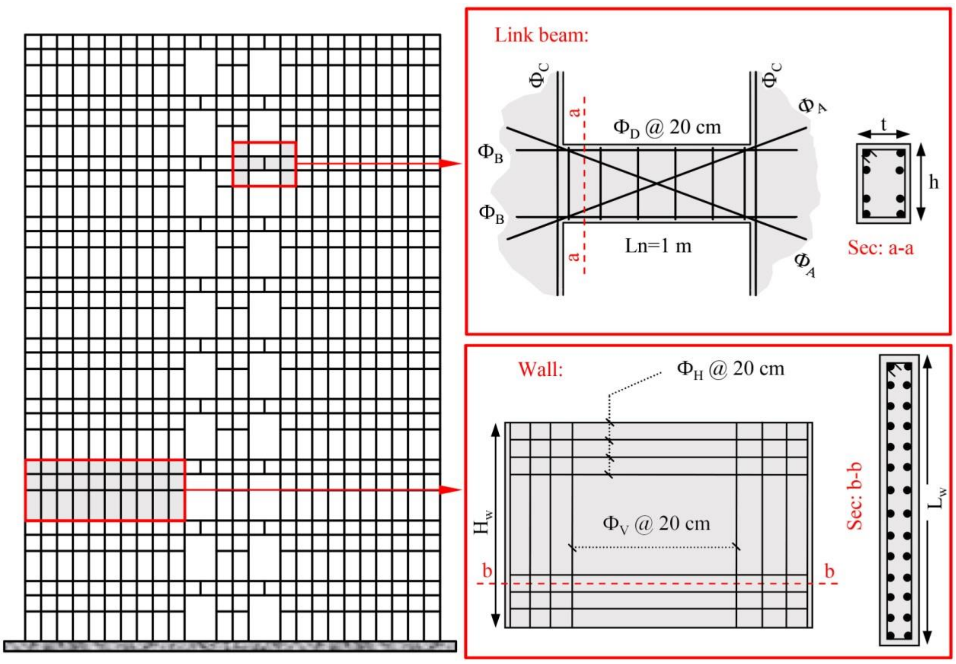

Fig. 3 Mesh sizes and reinforcement detailing for walls and coupling beams

Table 1 Values of dead and live loads

Roof	Stories	Load (kgf/m^2)
640	640	Dead load
150	200	Live load

are considered enabling to further analyze the effect of height on behavior of irregular tunnel-form buildings. The buildings are residential assumed to be located in high seismic hazard zone (i.e., PGA=0.35 g) resting on soil type “II” in compliance with classification provided by the Iranian Code of Practice for Seismic Design of Buildings (Standard No.2800, 2014) (soil type II in Standard No. 2800 is equivalent to type C as presented by NEHRP (2003) with shear wave velocity ranging from 375 to 750 m/s).

Accordingly, it is of note all models are designed by ETABS Software (CSI 2015) based on ACI-318 (2014) satisfying all design provisions specified by Iranian Building and Housing Research Center regarding this structural system (BHRCP 2007). The response modification factor for preliminary design of buildings, was selected to be 5 based on the typical values used by the practitioners for this system (Mohsenian 2013). Besides, shell behavior is considered for walls and slabs as they interactively experience in-plane and out of plane deformations and the optimal mesh size is achieved through a trial and error process as presented in Fig. 3.

Finally, thickness of 20 cm was considered for the walls reinforced at two layers by Ø 8 steel bars with spacing of 20 cm in both horizontal ($Ø_H$) and vertical ($Ø_V$) directions (only the vertical bar ($Ø_V$) of the walls in the first four floors of the taller building, are Ø 12). As the ratio of free length to height of the cross-section in the coupling beams is less than 2, it can be concluded that shear behavior certainly dominates on their general behavior (Paulay and Binney 1974, Zhao *et al.* 2004). Subsequently, diagonal reinforcement ($Ø_A$) is designed for the coupling beams to provide sufficient ductility and shear strength for them (Fig. 3). The thickness of the slabs is 15 cm and in order to design the structural elements, concrete with grade of “C25” and “AIII” reinforcing bar (yield strength of 400 MPa) are selected.

The value of dead and live loads applied to buildings are presented Table 1. According to the Iranian Code of Practice for Seismic Resistant Design of Buildings (commonly known as Standard 2800), the seismic weight includes dead loads (weight of slabs and ceiling’s belongings, lower and upper half of structural and non-structural walls) together with 20% of live loads. Consequently, the seismic weights for stories and roof were respectively computed 1200 and 940 kg/m^2.

As in almost majority of studies concerning the mass irregularity, increase in mass of the upper stories has found to be leave more critical states on the seismic responses, to trigger mass irregularity, the upper stories have become heavier herein (except for roof) as schematically shown in Fig. 4 (filled circles).

The ratio of mass in these stories in proportion to the lighter ones, has considered being 2 satisfying the mass irregularity requirements specified by Standard No. 2800 (2014).

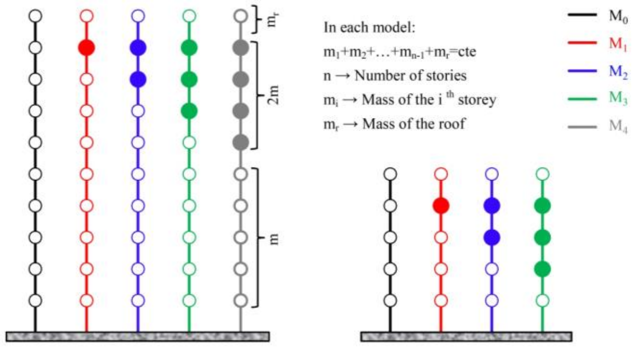

Fig. 4 Irregularity patterns considered in this study

To further elucidate, mass irregularity comes into force where the effective mass of any storey is more than 150% of the effective mass of an adjacent storey. It is of note that the seismic design codes exclude the roof and ridge stories.

To conveniently introduce the models, they were named by “M_i”. Subsequently, “M_0” represents the base model and as mentioned earlier, mass is regularly distributed along its height. Noteworthy, to ensure whether the behavioral variations are caused by just vertical irregularity, total mass of building is considered constant in all patterns.

3. Simulation of nonlinear behavior and determination of strength and deformation parameters

For nonlinear modeling and analysis of the buildings, Perform-3D Software (CSI 2016) was employed. It is worthwhile that the behavioral type of shear walls depends on the values of α and β (α=hw/lw and $\beta=M_u/(V_u.l_w)$) in a way that if “$\alpha\geq3$” or “$\beta\geq1$”, the flexural behavior will governs and shear behavior will prevail in the general response if “$\alpha\leq3$” and “$\beta\leq0.5$”. Having this said, majority of walls applied herein are long and also, attempts should be made to satisfy all provisions prescribed by the Iranian Building and Housing Research Center (BHRCP 2007) concerning application of sufficient number of walls and selection of a proper thickness for these elements, led the minimum requirements for shear design to prevailing in the analysis procedure. As a result, shear was adopted as the parameter to be controlled by displacement in most of the walls and their link beams. Hence, nonlinear shear behavior was defined for the elements (Allouzi and Alkloub 2017). It should be noted that in the above mentioned parameters (α and β), “M_u” and “V_u” are bending and shear of the wall, respectively, and “h_w” and “l_w” indicate the height and length of these elements.

Accordingly, the criteria used to ductility of the elements, differ upon their behavior. As shown in Fig. 5, in case of walls and shear-control beams in which ductility is mobilized by means of shear failure, respectively, drifts (θ) and chord rotation (γ) are chosen as the criteria (ASCE41-13 2014). In this study, nominal shear strength was considered for modeling the nonlinear shear behavior of elements as proposed by ASCE41-13 (2014). It should be

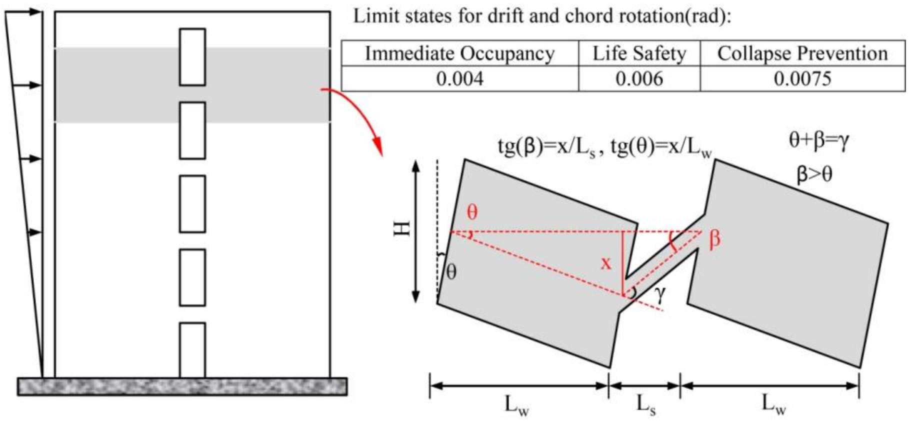

Fig. 5 Lateral relative displacement (Drift) (θ) and Chord rotation (γ) in walls and coupling beams as well as the limit states corresponding to different performance levels

Table 2 Coefficients of translational effective mass (M) and vibration period (T) of the first thee modes

Mode No. →		1		2		3	
Structures →		5-Storey	10-Storey	5-Storey	10-Storey	5-Storey	10-Storey
M_0	T (s)	0.2235	0.7485	0.1397	0.4453	0.1342	0.3187
	M (%)	0	0	79.6	75.41	74.02	67.35
M_1	T (s)	0.2411	0.8069	0.1506	0.4794	0.144	0.3383
	M (%)	0	0	82.53	76.35	78.32	69.42
M_2	T (s)	0.2394	0.8318	0.1498	0.4942	0.144	0.3486
	M (%)	0	0	85.09	80.01	80.83	72.04
M_3	T (s)	0.2316	0.8368	0.1450	0.4973	0.1396	0.3522
	M (%)	0	0	84.49	87.17	79.38	75.55
M_4	T (s)	-	0.83	-	0.4935	-	0.3512
	M (%)	-	0	-	81.45	-	74.21

mentioned that the relations used for deep beams, are applied to calculate the nominal strength of the coupling beams. The other parameters required for modeling and acceptance criteria of the nonlinear behavior, have been obtained from the load-displacement relation and Table related to the shear-control elements (ASCE41-13 2014).

Elastic out of plane behavior of the walls, rigid diaphragm for the floors, rigid connections at base of the walls and neglecting the rebar slippage in the concrete, are the other assumptions made in this study.

4. Eigen value analysis

All values considered for dead and live loads as well as the details accounted for the elements' mesh in nonlinear modeling, are exactly the same as what were adopted in preliminary design phase. Moreover, as in Eq. (1) representing the combination of gravity and lateral loads, upper limit of the gravity loads have been considered (ASCE41-13 2014).

$$QG = 1.1[\text{Dead Load+ Effective Live Load}] \quad (1)$$

Based on the number of walls in plan, the buildings are of a greater strength and stiffness in longitudinal direction (x). In this respect, main focus is placed on the transverse direction of buildings (y) and only this direction has been investigated. The data given in Table 2 concerning order of translational modes, confirms this assumption.

As it is observed, due to the considerable relative stiffness of stories, vertical irregularity has not affected the order of the first three vibration modes and only lengthens the periods as well as the coefficients of translational effective mass. Moreover, increase in building's height and pattern of irregularity, cannot affect the order of vibration modes. In all patterns, the first mode lacks the translational component in both buildings and is entirely rotational. Compared to the base model, the shorter building experiences 7% elongation in its period and also, the maximum increase in coefficients of mass contribution in longitudinal and transverse directions is respectively equal to 9 and 7%. Based on the observations, the variations mostly occur M2 model whose mass is doubled in the upper half of building's height. In addition, raise in period of the taller building in comparison with the base model, is averagely equal to 11.33% and the coefficients of mass contribution encounter an increase of 12 and 15% in longitudinal and transverse directions, respectively. Moreover, majority of variations take place in M3 model in which the mass in the upper one-third of height is twice greater than the other parts of building.

In conclusion, it is observed that the taller building is much more susceptible to irregular distribution of mass along height. Subsequently, as the fundamental period of buildings is lower than one and irregularities increase the coefficients of mass contribution in translational modes of

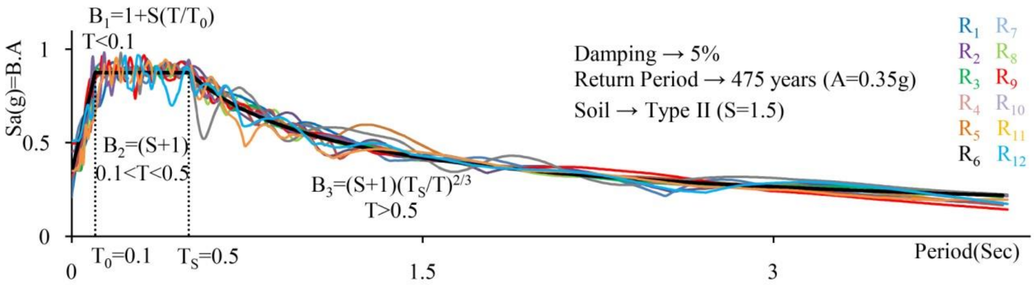

Fig. 6 Comparison between the artificial and site demand spectra

Table 3 Selected accelerograms to generate artificial records and accomplishment of dynamic analysis

Record No.	Earthquake& Year	Station	R^a (km)	Component	*Ms*	PGA(g)
R_1	Cape Mendocino, 1992	Eureka - Myrtle & West	41.97	90	7.1	0.1782
R_2	Northridge, 1994	Hollywood - Willoughby Ave	23.07	180	6.7	0.2455
R_3	Northridge, 1994	Lake Hughes #4B - Camp Mend	31.69	90	6.7	0.0629
R_4	Cape Mendocino, 1992	Fortuna - Fortuna Blvd	19.95	0	7.1	0.1161
R_5	Northridge, 1994	Big Tujunga, Angeles Nat F	19.74	352	6.7	0.2451
R_6	Landers, 1992	Barstow	34.86	90	7.4	0.1352
R_7	San Fernando, 1971	Pasadena - CIT Athenaeum	25.47	90	6.6	0.1103
R_8	Hector Mine, 1999	Hector	11.66	90	7.1	0.3368
R_9	Kobe, 1995	Nishi-Akashi	8.70	0	6.9	0.5093
R_{10}	Kocaeli (Turkey), 1999	Arcelik	53.7	0	7.5	0.2188
R_{11}	Chi Chi(Taiwan), 1999	TCU045	77.5	90	7.6	0.5120
R_{12}	Friuli(Italy), 1976	Tolmezzo	15.82	0	6.5	0.4169

[a]Closest Distance to Fault Rupture

both longitudinal and transverse directions (it approaches 75%), it makes sense to assume a triangular distribution for earthquake loads in height and utilize the static analysis method (particularly for the shorter building). Indeed, the irregularity patterns considered herein for each direction, increase the contribution of the first translational mode in the shape of lateral load distribution.

5. Time-history analysis

In order to achieve the most compatibility between the applied earthquake records and hazard-characteristics of the site, the artificial accelerograms corresponding to the design spectrum were employed. In this respect, correction of the existing records, time-domain and frequency amplitude methods are the methods at disposal to generate the artificial records, which are typically used for generation of the accelerograms that match the target design response spectrum of the project in a medium state.

In this study, 12 artificial earthquakes prepared by correction of the existing accelerograms based on wavelet transform from the site demand spectrum provided based on the Iranian Code of Practice for Seismic Design of Buildings (Standard No.2800 2014) for soil type “II” and DBE hazard level (see Fig. 6). Accordingly, the Peak Ground Acceleration of these records is approximately equal to that of the DBE hazard scenario (PGA=0.35 g).

Within wavelet transform, the selected accelerogram is taken to the wavelet domain and the details functions are corrected making use of the ratio of target to response spectrum and return to the time domain. In doing so, a motion with a spectrum closer to the target spectrum can be obtained and this process is iteratively accomplished until desired outcome is achieved (Hancock *et al.* 2006).

It should be noted that in order to produce artificial accelerograms, main component of the earthquakes presented in Table 3, has been utilized. The performance levels defined in ASCE41-13, (2014), were adopted as damage index for the building’s elements and also, the values corresponding to these limit states, were employed to evaluate the performance level (Fig. 5).

Afterwards, maximum drifts and shear forces as well as chord rotation developed respectively in walls and coupling beams were recorded and their mean values were taken as the comparison criterion (Figs. 7 to 9).

According to Fig. 7, the elements of buildings are in a level higher than that of immediate occupancy (IO) while subjected to DBE hazard level (for immediate occupancy level, the value of “γ” and “θ” is equal to 0.004). In spite of the fact that mass irregularity intensifies the seismic responses induced in the elements, it does not vary in the performance level. As it can be seen, seismic demand in coupling beams is constantly greater than that of the walls and taller building is more susceptible to the vertical irregularity.

As shown in Fig. 8, vertical mass irregularity increases the storey drifts which becomes profounder in lower stories of the short building and middle one-third stories of the tall building. The results indicate that irregularities do not change the location of maximum drift and damages, consequently. Based on Fig. 9, variation of storey shears

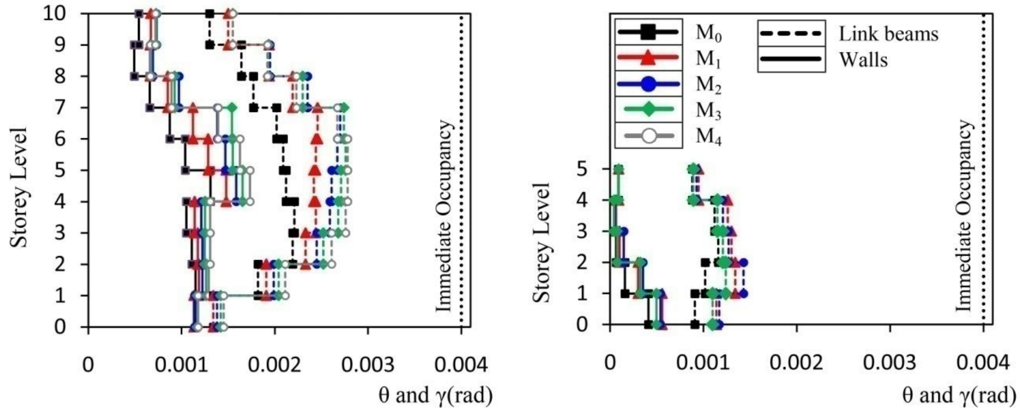

Fig. 7 Mean values of maximum drift and chord rotation developed in each storey and limit state corresponding to Immediate Occupancy (IO) performance level

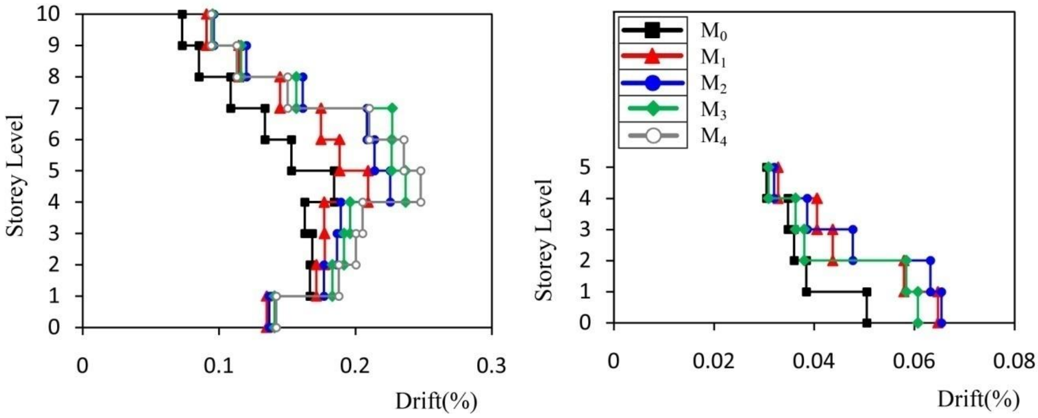

Fig. 8 Average of the maximum drift developed in stories

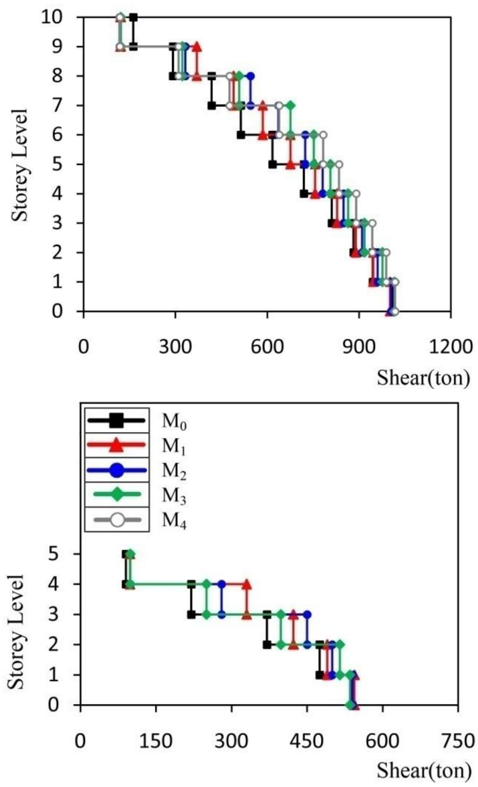

Fig. 9 Average of the Maximum Shear induced in the Stories

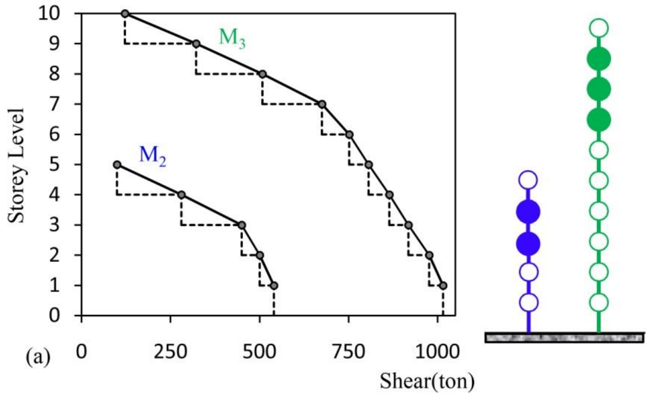

Fig. 9 Continued

form bottom to top of the building, follows a decreasing trend and in each irregularity pattern, the decreasing trend is continued with a gentler slope in the storey with double mass (Fig. 9(a)).

Based on Figs. 7 to 9, increase in number of stories with double mass, amplifies the responses in all stories until the half of building's height is not exceeded. From that point on (like M3 model in 5-storey building), the responses follow a decreasing trend and approach the base model (M_0) which is justifiable due to the fact that the irregularity pattern gradually approaches the uniform distribution pattern.

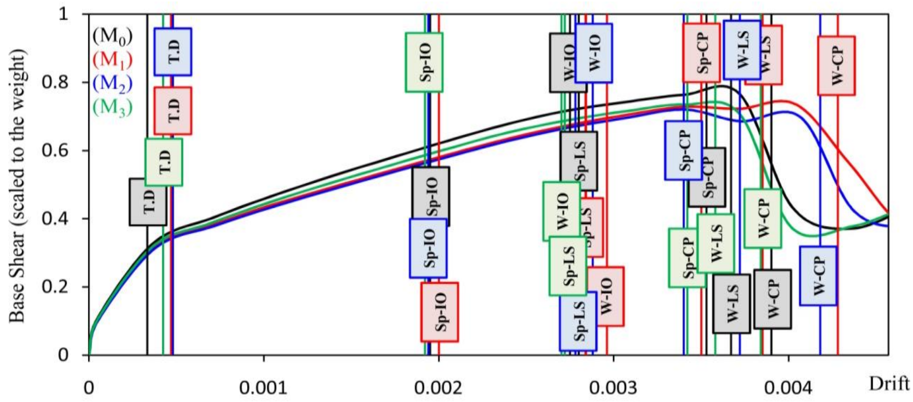

Fig. 10 Capacity curve of 5-Storey building

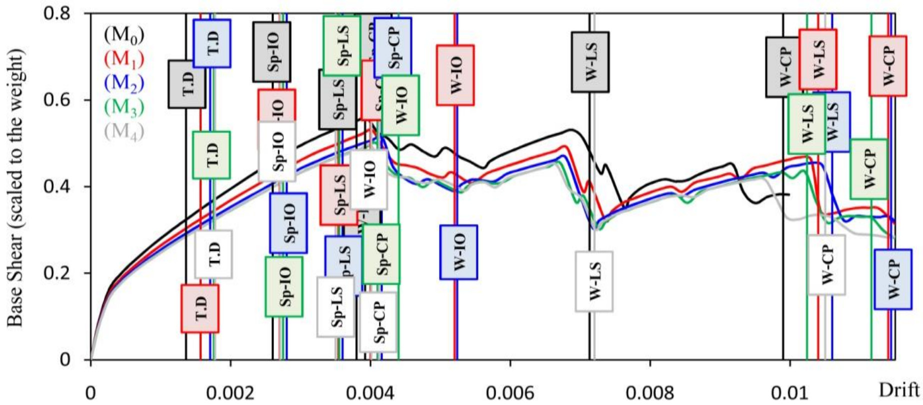

Fig. 11 Capacity curve of 10-Storey building

6. Pushover analysis

To conduct pushover analysis, modal patterns is adopted for distribution of lateral loads. This distribution is in proportion to the effective modes in desired direction (transverse direction of the plan) and number of vibration modes is selected such that at least 90% of the building's mass contributes in analyses. In this study, target displacement of the building is derived by time-history analysis in virtue of averaging the maximum displacement of roof mass center triggered by the previously generated artificial records. After that, the damages incurred in the buildings within the pushover analysis are terraced and roof drifts (ratio of displacement of roof mass center to total height of building), was determined when the first walls and coupling beams reached the performance levels of immediate occupancy (IO), life safety (LS) and collapse prevention (CP). Figs. 10 and 11 illustrate the values of these drifts together with the drift corresponding to DBE hazard level and the building's capacity curve considering various patterns for mass irregularity.

In these figures, *W* and SP denote wall and spandrels. Through the pushover analysis, it was found that mass irregularity does not vary the first elements to reach the damage states. Besides, increase in number of stories with greater masses, reduces the building's capacity and this rate is of a downward trend. In other words, increase in number of stories with double mass, shrinks the capacity curve until half of the building's height is not exceeded. However, after this limit, an increasing trend is followed approaching that of observed in the base model.

Compared to the shear walls, coupling beams always experience higher level of damage and act as the frontier vulnerable parts of the system. According to Fig. 5, as these elements seek for larger seismic demands compared to the walls, this observation can be justified.

On the other hand, in the hazard level corresponding to that of DBE, all walls and coupling beams satisfy the performance level of immediate occupancy (IO). This is because of the fact that roof drifts in this level of intensity, is smaller than the required drift to reach the mentioned performance level.

7. Seismic reliability assessment based on Engineering Demand Parameters (EDP-Based)

This section aims to assess the sensitivity of results to the intensity of input excitations and to this end, reliability and fragility analyses have been employed. Fragility curves represent the cumulative distribution of damage distribution (Cimellaro *et al.* 2006) by which distribution of structural responses in different earthquake intensities can be derived. If R represents the building's response and "LS_i" denotes the performance level or a limit state related to "R", "IM" is one of the parameters stating the earthquake intensity and "S" is the value of the desired intensity, then, fragility function is defined based on the mathematic form presented in Eq. (2).

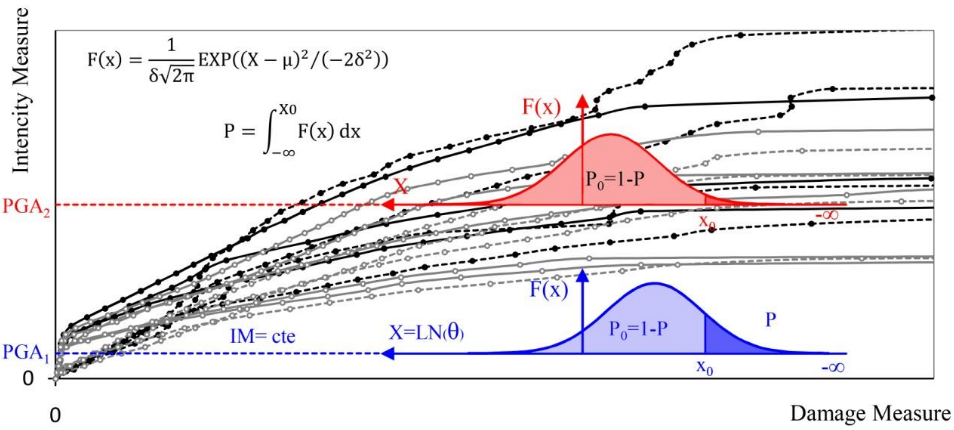

Fig. 12 Reliability on the elements to not reaching a specific damage level (X_0) at a constant hazard level

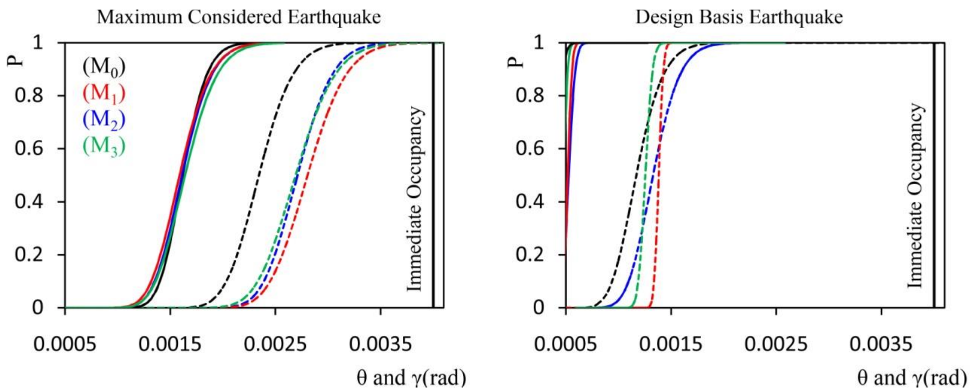

Fig. 13 Reliability on the elements to not reaching different damage levels at a DBE and MCE hazard levels (5-Storey building)

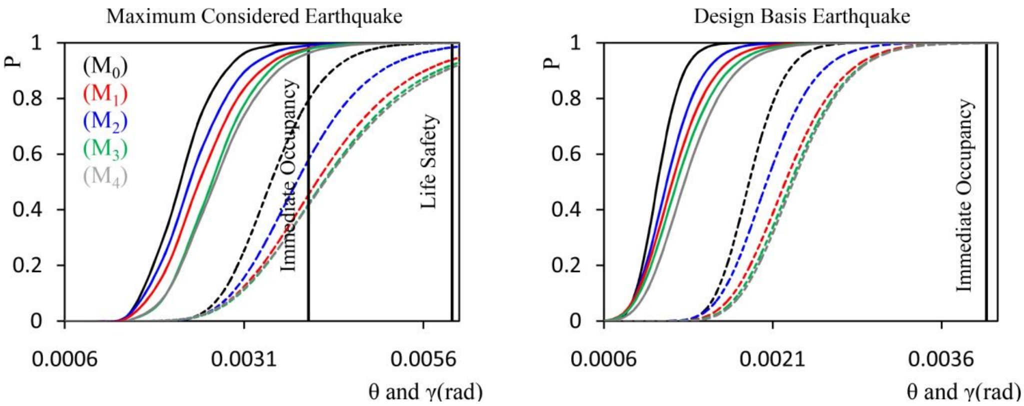

Fig. 14 Reliability on the elements to not reaching different damage levels at a DBE and MCE hazard levels (10-Storey building)

$$Fragility = \mathrm{P}[\mathrm{R} > \mathrm{LS_i} | \mathrm{IM} = \mathrm{S}] \tag{2}$$

In this respect, whenever collapse is based on the engineering demand parameter, the desired seismic response determines the limit state for damage in building (Zareian *et al*, 2010). On this basis, at a constant level of earthquake (PGA=Constant), probability of reaching to various limit states will be obtained.

Generation of the fragility curves requires a probabilistic analysis and based on the desired accuracy, different approaches can be adopted to develop these curves (Khalvati and Hosseini 2008). In this study, fragility curves are prepared masking use of an analytical method based on the time-history analysis.

In order to conduct the time-history analyses, in proportion to the site soil condition (type "II" as per the classification presented by Standard No.2800 (2014) (375 (m/s) ≤*Vs*≤750 (m/s) which is equivalent to soil type "*C*" in NEHRP (2003)), 12 pair of earthquake records were selected from the PEER database (http://peer.berkeley.edu/peer_ground_motion_database).

The selected records all are far-field. Accordingly, after drawing the spectral response for each pair of accelerograms and comparing them, the main component of the earthquake is selected based on the greater spectral values in range of vibration frequencies of the buildings as presented in Table 2.

Considering the aleatory uncertainties related to the future earthquakes and accounting for maximum drift (θ) and chord rotation (γ) respectively in walls and coupling beams as the response, the performance levels defined by ASCE41-13 (2014), were adopted as damage index for the building's elements and the values prescribed by this reference, were attributed to these limit states (Fig. 5). Next, rate of reliability on the elements under the DBE and MCE hazard levels, was estimated. The steps to be takes are as follows:

According to Fig. 12, maximum values of structural response under each scaled record to a certain PGA, are obtained. Afterwards, assuming that derived values are of a lognormal distribution, a probability density function (F(X)) is developed after computing mean (μ) and standard deviation (δ) parameters for the values obtained at this level of intensity. As shown in Fig. 12, by considering a value for "X_0" as the response corresponding to a specific damage level, the area under the curve of probability density function from "-∞" to "X_0", signifies the frame's reliability meaning that at this level of intensity, the frame response does not exceed "X_0" to probability value of P and will not experience the mentioned performance level with the same degree of probability (Mohsenian and Mortezaei 2018c). Obviously, the difference between value of "P" and "P_0", results in probability of exceedance from this performance level (fragility). Repeatedly conducting this process and deriving the probabilities for different values of the response, will result in the extraction of a curve for the desired intensity.

The mentioned curves for all irregularity patterns and DBE and MCE hazard scenarios (PGA=0.35 g and 0.55 g), have been generated in compliance with the process illustrated in Figs. 13 and 14. As it is seen, reliability on the elements for not reaching the various performance levels (P), can be easily estimated by the curves.

Analysis of Figs. 13 and 14, demonstrates that the walls are of higher reliability compared to the coupling beams. Increase in number of stories with double mass, does not downgrade the performance level of buildings. Moreover, the probability of reaching different level of performance is boosted as the building's height and intensity raises, which is profounder in case of coupling beams.

In both buildings and for all irregularity patterns, under the DBE hazard level (PGA=0.35 g), the coupling beams and walls are of zero reliability to reach the immediate occupancy performance level. This observation is also true for the MCE hazard level (PGA=0.55 g).

Under the MCE hazard level, in the most critical pattern of irregularity for the taller building ($M4$), the probability of experiencing the immediate occupancy performance level for walls and couplings beams is respectively nearly 0.5 and 59% and if performance level of life safety is of concern for coupling beams, the value of probability will be almost 1%. As the shear walls are the main lateral-load resisting elements, it can be said that the buildings suffering from the studied irregularity patterns, in the mentioned damage levels, satisfy the immediate occupancy performance level.

8. Conclusions

The results obtained herein, indicate the desirable seismic performance of tunnel-form buildings with irregular distribution of mass in building's height. Accordingly, the most notable conclusions are as follows:

- It was found that the order of vibration modes is not affected by the building's height and patterns of mass distribution in elevation.
- Irregular distribution of mass in height increases the fundamental period as well as coefficient of mass contribution of the vibration modes.
- Vertical mass irregularity amplifies the displacement responses of buildings and subsequently, deformation responses of the elements. Accordingly, the taller building was found to be more susceptible to this issue. The results indicate that until the middle of the building's height is not reached, increase in number of stories possessing extra weight leads to amplification in structural responses and reduces the building's capacity. However, exceeding this limit (half of building's height) is accompanied with a decreasing trend in the responses approaching the basic state.
- The pattern of mass irregularity does not affect the location of first damages induced by the DBE hazard level.
- Due to larger seismic demand of coupling beams compared to the walls, they are accounted as the structural fuses in tunnel-form buildings and reveal much greater susceptibility to the irregular distribution of mass in height.
- The mass irregularity does not downgrade the building's performance level for the seismic event of DBE (return period of 475 years). Under this level of intensity, all structural elements are of a level higher than that of the performance level of immediate occupancy. Considering the walls as the main lateral load-carrying elements, the same outcome in the event of MCE hazard level (return period of 2475 years) is observed.

Based on the results, it can be said that with respect to remarkable stiffness and strength, the tunnel-form structural system is capable of making ground on providing a desirable performance for the structures built using this system against the earthquake-induced loads. Furthermore, even under different scenarios for intensity of the input motions and irregular distribution of mass, the system managed to exhibit a satisfactory performance.

In conclusion, it appears that necessity of observing the mass regularity for the RC tunnel-form buildings studied herein, calls for drastic measures, which are not necessarily in demand.

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